# Monotonic triaxial response of early age cemented paste backfill

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ABSTRACT
Modern mines require systems that quickly deliver backfill to support the rock mass surrounding underground openings. Cemented Paste Backfill (CPB), which contains fine grained mine tailings, water and binder agents, is one such backfilling method. However, concerns have been raised about CPB's liquefaction susceptibility especially when the material has just been placed in an underground opening. In this paper, test results will be shown for uncemented mine tailings in compression and 4-hour cure CPB in compression and extension over a range of confining pressures from 20 kPa to 400 kPa. In addition, the effect of axial strain rate on the monotonic response of mine tailings will be investigated. A higher axial strain rate might be required for CPB specimens to minimize the time of triaxial experiment since the properties of the material might change during the experiment due to the hydration of binder agents. Monotonic triaxial tests conducted on normally consolidated uncemented mine tailings demonstrated that the material is initially contractive up to a phase transition point, beyond which dilation occurs. Most importantly the material never exhibits unstable strain softening behaviour in compression, and only temporary or limited liquefaction in extension. The addition of 3% Portland cement results in initial sample void ratios that are even higher than their uncemented counterparts, and yet the material friction is slightly enhanced when tested at 4 hours cure. These results suggest that the flow liquefaction phenomenon commonly associated with undrained loose sands (e.g., Toyoura sand) might not occur with uncemented mine tailings and cemented paste backfill.

RÉSUMÉ
Les mines modernes exigent des systèmes qui livrent vite les remblais cimentés en pâte (RCP) pour soutenir la masse de roche l'encerclement des ouvertures souterraines. Les remblais cimentés en pâte, qui contient parfait grained le mien tailings, l'eau et les agents de classeur, est une telle méthode backfilling. Pourtant, les inquiétudes ont été levées la susceptibilité de liquéfaction de RCP surtout quand la matière a juste été placée dans une ouverture souterraine. Dans ce papier, les résultats d'essai seront montrés pour non cimenté des remblais en pâte dans la compression et la 4 heure guérissent RCP dans la compression et l'extension sur une gamme de confiner des pressions de 20 kPa à 400 kPa. En plus, l'effet de taux d'effort axial sur la réponse monotonique non cimenté des remblais en pâte sera enquêté. Un plus haut taux d'effort axial pourrait être exigé pour les exemplaires des RCP de minimiser le temps d'expérience de triaxial depuis que les propriétés de la matière pourraient changer pendant l'expérience en raison de l'hydratation d'agents de classeur. La monotonique triaxial les épreuves accomplies sur normalement uni a non cimenté des remblais en pâte a démontré que la matière est au départ serrée jusqu'à un point de transition de phase, au-delà lequel la dilatation se produit. De la manière la plus importante la matière n'expose jamais l'effort instable adoucissant le comportement dans la compression et la liquéfaction seulement temporaire ou limitée dans l'extension. L'adjonction de Portland de 3 % cimente des résultats dans les indices des vides initiaux de promotion qui sont encore plus hauts que leurs contreparties non cimentées et encore la friction matérielle est légèrement améliorée quand évalué à cure de 4 heures. Ces résultats suggèrent que le phénomène de liquéfaction d'écoulement fréquentait communément de sables desserrés non égouttés (par ex., le sable de Toyoura) ne pourrait pas se produire avec non cimenté des remblais en pâte et a les remblais cimentés en pâte.

## 1 INTRODUCTION

Modern mines require systems that quickly deliver backfill to support the rock mass surrounding underground openings. Cemented Paste Backfill (CPB) is one such backfilling method, but concerns have been raised about CPB's liquefaction susceptibility especially when the freshly placed material is subjected to static loading.

The state of practice in paste technology is to add a small quantity of cementitious materials (i.e., binder agents) to mine tailings as backfill material in order to improve short term and long term strengths. 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). It has been shown that the 100 kPa UCS can be achieved by adding a small quantity of binder to CPB in a short period of time (Aref 1988, Pierce 1998, le Roux 2004). However, there might still be a risk of liquefaction at early ages when the cement in CPB has not hydrated significantly.

A few studies exist on the liquefaction potential of CPB. Aref (1989) showed that the response of CPB to monotonic loading in triaxial tests is dilative and CPB is not susceptible to liquefaction. Been et al. (2002) also investigated the response of cured CPB using undrained compression triaxial tests. The CPB specimens showed dilative behaviour with no significant pore pressure development during monotonic loading. More recently, le Roux (2004) showed that the monotonic response of Golden Giant CPB with 5% binder content and cured for 3 hours is similar to that of non-plastic silts. In addition, she showed the evolution of cohesion and the angle of the phase transformation line due to the hydration of cement in CPB.

In the most recent studies on silt-sized mine tailings, Crowder (2004) showed that the mine tailings specimens (void ratios ~ 0.70-0.72) tested show strain hardening behaviour under monotonic loading and are not susceptible to flow liquefaction. In contrast, Al-Tarhuni (2008) has recently reported the flow liquefaction of silt-sized gold mine tailings using the direct simple shear test. He showed that the tailings specimens even with a low void ratio of 0.585±0.01 might be susceptible to flow liquefaction at the high effective confining stress of 400 kPa. In another instance, he also showed that the specimens with a void ratio of 0.660±0.01 experience flow liquefaction at the effective confining stress of 100 kPa. It seems the difference in monotonic behaviour of the tailings might be attributed to the differences in laboratory sample preparation techniques and the stress conditions due to the methods of shearing. Al-Tarhuni (2008) was concerned with tailings on surface that were desiccated and re-wetted. This may have influenced sample fabric and subsequent response to direct shear.

In addition to silt-sized mine tailings, the monotonic behaviour of silt made of limestone was investigated under compression and extension triaxial tests. Hyde et al. (2006), in these two types of triaxial test, showed that silt specimens experience both contractive and dilative behaviours, respectively, while the dilative behaviour continued until an ultimate effective stress line was reached. This indicates that the silts are not susceptible to liquefaction under monotonic loading. It is noteworthy that the angle of the ultimate line in a compression test is higher than that of the ultimate line in an extension test. This may be comparable to the monotonic behaviour of sands in compression and extension.

Hyodo et al. (1994) showed that the undrained behaviour of Toyoura sand in extension is different from compression. In their studies, although the sand specimen at 100 kPa exhibited stain softening behaviour with limited liquefaction in the compression test, the specimen showed flow liquefaction behaviour in the extension test. The difference between the behaviour of the specimens in compression and extension has been attributed to specimen anisotropy induced during sample preparation. Imam et al. (2005) also compared the undrained triaxial extension behaviour of medium loose Toyoura sand consolidated to a void ratio of 0.802 to 0.817 and tested at confining stresses between 50 kPa and 500 kPa with their critical-state constitutive model. They showed that the sand specimens exhibit both contractive and dilative behaviours although there is a drop in stress ratio predicted by the model at high axial strains. Imam et al. (2005) also showed that complete liquefaction in undrained triaxial extension might occur for Toyoura sand consolidated to void ratios higher than 0.860 and tested at confining stresses between 50 kPa and 500 kPa. They noted that zero strength is reached at or before PT in triaxial extension, whereas zero strength is reached at critical state in triaxial compression. In addition, larger contraction takes place before the PT is reached in triaxial extension, compared with triaxial compression. Vaid and Thomas (1995) also showed that the deviator stress beyond the peak value in extension drops more significantly than in compression for Fraser sand specimens, which were formed by the water pluviation technique. They also attributed the difference in the undrained responses between extension and compression to anisotropy of the specimens due to the sample preparation technique. Boukpeti et al. (2002) developed an elastoplastic model for predicting the undrained triaxial response of soils in extension and compression. This model also suggests a slope of 3M/(3+M) for the steady state line in the p'-q stress space in triaxial extension, which is lower than that in triaxial compression (i.e., M = slope of steady state line in compression).

Vick (1990) represented the data from Wahler (1974), and showed that the slime tailings exhibit strain hardening behaviour without reaching a steady state point within the range of axial strain (i.e., 12%) and effective confining stresses tested in that study. Unlike the slime tailings, Ishihara (1993) showed that the medium-loose Toyoura sands (i.e., e = 0.833) reach a steady state or critical state point at high axial strains (between 25% and 30%). The sand specimens showed fully dilative behaviour at a low effective confining stress of 100 kPa or fully contractive behaviour (flow liquefaction) at a high effective confining stress of 2000 kPa. However, the Toyoura sand specimens consolidated to a void ratio of 0.735 only exhibited dilative behaviour at different effective confining stresses (100- 2000 kPa), while reaching a steady state point at a high axial strain of 20%. Ishihara (1993) stated that Toyoura sand consolidated to a void ratio of 0.930 or higher completely liquefies in an undrained triaxial test. In another instance, Ishihara (1996) showed that the undrained behaviour of Tia Juana silty sand specimens prepared by the method of dry deposition and consolidated to void ratios between 0.840 and 0.890 can be categorized as strain softening with "limited liquefaction" followed by strain hardening behaviour. For these specimens tested at different effective confining stresses (50-200 kPa), no steady state was reached, similar to the slime tailings represented by Vick (1990). Chu et al. (2003) showed the typical undrained response of isotropically consolidated medium dense sand specimens with void ratios ranging from 0.643 to 0.695. The stress paths of the sand specimens in undrained conditions approached a constant stress ratio line (CSRL) rather than a steady state line while showing dilative behaviour.

This paper addresses the monotonic triaxial response of cemented silt-sized tailings in the form of CPB at early age. The liquefaction potential of CPB will then be compared with that of uncemented tailings and sands.

## 2 MATERIAL TESTED

Cemented pate backfill used in this study contains mine tailings from a hard rock gold mine, process water and

Portland cement. Therefore, the characteristics of mine tailings that make the solid portion of CPB on one hand and the hydration of Portland cement that changes the mechanical properties of CPB on the other hand must be determined.

### 2.1 Properties of Mine Tailings and Cement

The particle size distribution of the tailings was determined using the standard sieve analysis method (ASTM C136-06), followed by the standard hydrometer test (ASTM D422-63) without addition of the deflocculating agent. The sieve analysis test was used to categorize the particles larger than 75 microns while the hydrometer test was used to determine the distribution of the particles smaller than 75 microns. As shown in Figure 1, the tailings mainly contain silt with 14% of fine sands and 4% of clay-sized particles. The percentage of fine particles appropriate for CPB, which are less than 20 microns, is about 40%. The specific gravity of the tailings, determined using (ASTM D854-06), is 2.72.

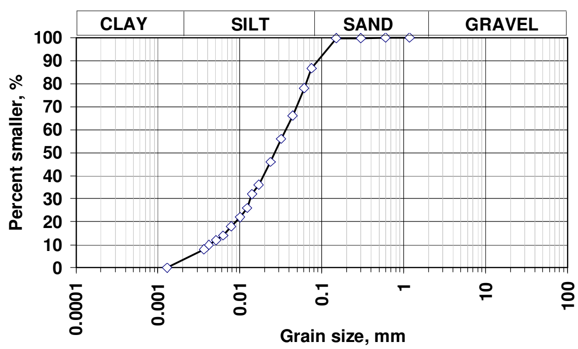


Figure 1. Particle size distribution of the tailings tested.

Using the standard test method (ASTM D4318 – 05) the liquid limit of the tailings is 30% and the plastic limit is 26% resulting in a plasticity index (PI) of 4%. As recommended by Boulanger and Idriss (2004), fine grained soils are classified as “sand like” if PI < 7, “clay like” if PI ≥ 7. Therefore, the silt-sized tailings in this study would classify as “sand like” with low plasticity.

The chemical composition of the tailings, determined using the X-Ray Fluorescence Spectroscopy (XRF) method, is shown in Table 1. The major compounds are $SiO_2$ and $Al_2O_3$, and the minor compounds are $CaCO_3$, iron and alkalis. Note that Ca was not detected in the form of oxide. In addition to these major and minor compounds, the tailings have a low sulphur content (S = 1.081%) relative to other hard rock mine tailings, such as those reported in Benzaazoua et al. (2002). This table also shows the chemical composition of the Portland cement used in this study. The X-ray diffraction (XRD) analysis, used to identify the mineral components of the tailings, showed that quartz and albite are the major minerals. The minor minerals can be identified as microcline and clinochlore. No sulphide mineral, such as Pyrite, was identified in the tailings, which is consistent with the low percentage of sulphur determined in the XRF test. The XRD analysis on Portland cement revealed that the major minerals are $C_3S$ and $C_2S$ while other PC minerals were not detected.

Table 1. Chemical composition of the tailings and Portland cement.

| Oxide | $SiO_2$ % | $Al_2O_3$ % | $CaCO_3$ % | $Fe_2O_3$ % | MgO % |
|---|---|---|---|---|---|
| Tailings | 58.30 | 11.85 | 5.13 | 2.89 | 3.70 |
| Cement | 16.27 | 3.37 | 71.18 | 2.26 | 3.01 |
| Oxide | $Na_2O$ % | $K_2O$ % | MnO % | $TiO_2$ % | $P_2O_5$ % |
| Tailings | 3.25 | 3.42 | 0.03 | 0.36 | 0.18 |
| Cement | 0.20 | 0.45 | 0.10 | 0.20 | 0.11 |

### 2.2 Time of Setting for CPB

The percentage of Portland cement used to make CPB was 3% of solids mass. To investigate the setting of the CPB due to the hydration of 3% cement, a series of standard tests for the time of setting (ASTM C191-08) was conducted by using a Vicat needle apparatus. According to this test method, the initial set is the time required for the needle to reach a penetration depth of 25 mm. The final set occurs when the needle does not penetrate into the CPB specimen.

Figure 2 shows the results of the Vicat needle tests for 3% CPB specimens along with the electrical conductivity (EC) tested by Simon and Grabinsky (2007). The initial and final set for these specimens are approximately 630 min (10.5 hours) and 1680 min (28 hours), respectively. Note that the Vicat needle penetration readings started giving non-maximum values at 300 min (5 hours), which coincides with the appearance of the EC peak. Similar correlation between the onset of the penetrative resistance (i.e., non-maximum values) and the maximum EC has been reported by Levita et al. (2000) for Portland cement mixtures in the early age of hydration. Furthermore, Hwang and Shen (1991) showed that the onset of the penetrative resistance (the equivalent of non-maximum in the Vicat needle test) corresponds well to the onset of the acceleration phase determined from the heat evolution curve, thus implying that the onset of penetrative resistance, as well as maximum EC, takes place during the acceleration phase of hydration process.

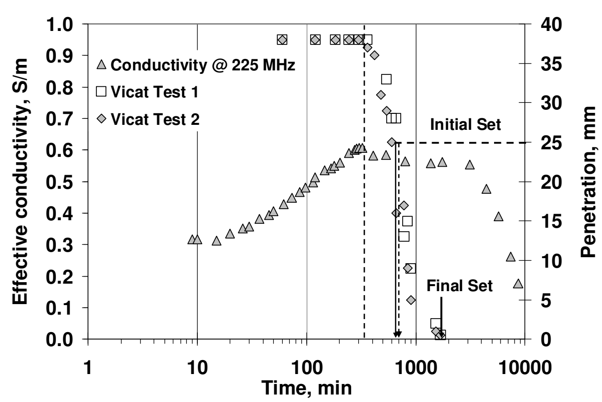


Figure 2: The results of Vicat needle tests and electrical conductivity for CPB.

## 3 LABORATORY EXPERIMENTS

The purposes of investigating the monotonic response of early age CPB (i.e., 4 hours old) were (i) to determine whether or not the material was liquefaction resistant, and (ii) to determine the contractive or dilative behaviour of the material at different effective stresses. Therefore, the consolidated undrained (CU) triaxial compression test (ASTM D4767-04) was recommended. In addition, the consolidated undrained triaxial extension test was also recommended for CPB specimens.

### 3.1 Sample Preparation

There is no standard triaxial specimen preparation method for the paste material, which has a high water content and slump. However, Crowder (2004) and le Roux (2004) suggested the pre-consolidation method of preparation for creating a triaxial specimen of paste material. To create a CPB specimen, a 600 gram sample of well blended tailings/water mixture with specific initial water content was used. Having the initial water content of the tailings mixture, it was possible to calculate the mass of cement required based on the mass of the solids ($Mass_{cement}/Mass_{solids}$ = 3%). The 600 gram sample was then mixed using a hand mixer, with additional water to have a mixture with a total water content of 39% considering the amount of cement. The 3% Portland cement was then mixed with the tailings mixture for an additional 10 minutes. The water content of the CPB sample was then measured prior to casting the triaxial specimen.

The basic setup for a triaxial CPB sample involves placing the material between two porous stones in a latex membrane while the specimen stands on its own after removing the split triaxial mould. Once the CPB sample is ready, it is poured into the mould by using about 4-5 scoops of a spoon after placing the bottom porous stone and paper filter. The mould is filled in three stages while each stage should be accompanied by removing any large voids using a 5 mm diameter glass rod to rod the sample about 20 times. When the mould is filled to the desired height, the top filter paper and porous stone are placed on top of the sample, followed by the top platen.

To calculate the void ratio and degree of saturation of the specimen, the mass and volume (i.e., area × height) of the placed specimen are required. The mass of CPB specimen used for the triaxial testing can be calculated by subtracting the masses of bowl containing the CPB sample before and after pouring. The initial height of specimen is measured after placing the top platen. At this point, the specimen is ready for dead-weight consolidation while the latex membrane is not yet attached to the top platen.

The procedure of dead weight consolidation is to apply 2.5 kg (equivalent to about 12.5 kPa) on the top of the sample while the water is collected from the bottom drainage line and at the top between the membrane and the top platen (the top drainage valve is closed). Since the mass of the top platen and triaxial ram is 500 g, two additional 1 kg weights should be applied within a few minutes. The dead-weight consolidation phase takes less than an hour. Considering the duration of mixing and the dead-weight consolidation phase, the CPB specimen has cured for an hour at the end of this phase.

After dead-weight consolidation, the bottom drainage valve is closed, the final mass of water collected is recorded, and the membrane is then fixed onto the top platen with the o-rings. The weights are then removed, followed by removing the split mould. The final height of the specimen is measured to calculate the final void ratio of the triaxial specimen. Once the triaxial sample stands on its own, the triaxial cell is assembled.

To achieve a fully saturated specimen, the back saturation process is followed in accordance with ASTM D4767-04. Generally, a back pressure of about 350 kPa was required to obtain a specimen with a B-value of more than 0.96. The specimen is then consolidated at a desired effective confining stress, $\sigma'_c$ (i.e., 20 – 400 kPa).

The minimum time required to prepare a fully saturated, consolidated CPB specimen was about 4 hours since adding the cement to the tailings mixture in this study. Therefore, the 4 hour cured CPB specimens were subjected to loading prior to reach their onset of acceleration phase of hydration (i.e., 5 hours) as well as their initial set (i.e., 10.5 hours). Note that the sample preparation procedure was the same for uncemented mine tailings specimens except the stage of addition of cement.

### 3.2 State Parameters

Depending on the effective confining stress, the final void ratios of the uncemented tailings and CPB after the consolidation stage or prior to loading range from 0.675 to 0.750 and from 0.770 to 0.840, respectively.

In addition, the isotropic loading during consolidation stage suggests that the compression index, $C_c$, of CPB is 0.11±0.01, which is similar to that of uncemented tailings tested by Crowder (2004) using the small-strain one-dimensional consolidation test. Note that these result suggest that the uncemented tailings and CPB are highly compressible compared to Ottawa sand tested by Sasitharan (1994) and Toyoura sand tested by Ishihara (1993).

### 3.3 Loading Rate

In all triaxial compression and extension tests, fully saturated CPB or uncemented tailings specimens were subjected to a monotonic load under a constant strain rate in an undrained condition. A relatively “high” axial strain rate of 2% /min has been considered for CPB because its mechanical properties depend on the time of hydration. The maximum axial strain during triaxial shearing was 20% and this took 10 minutes at an axial strain rate of 2%/min. Therefore, for the specimens cured for four hours at the start of shearing, the actual shearing phase took place between 240 minutes and 250 minutes. This indicates that in this timeframe, the hydration process has not yet achieved a stage where any significant resistance to penetration of the Vicat needle can be detected, and thus the mechanical behaviour of the sample can be taken as essentially constant during the triaxial shearing phase. Although the high axial strain rate reduces the duration of the experiment, that might affect the monotonic response of the material. Therefore, the effect of axial strain rate on the monotonic behaviour of the materials was also investigated by conducting a series of CU triaxial compression tests on uncemented mine tailings at different axial strain rates.

## 4 EFFECT OF AXIAL STRAIN RATE

In this paper, stress path results will be plotted in $(\sigma'_1+\sigma'_3)/2$ and $(\sigma'_1-\sigma'_3)/2$ space. The stress points based on these invariants can be interpreted as the top of a corresponding stress circle in Mohr stress space. State lines of inclination α´ are the locus of these stress invariants. In contrast, state lines of inclination φ´ are tangent to the Mohr's circle. The two angles can be related by tan α' = sin φ'. Some results are reported in the literature in terms of invariants of the stress tensor, $p' = (\sigma'1+2\sigma'_3)/3$ and $q = (\sigma'1-\sigma'_3)$, and the slopes of the corresponding state lines in this stress space conventionally denoted using the symbol M. The stress invariant ratio M is obtained from $M_c = 6 \sin \varphi'_c/(3-\sin \varphi'_c)$ in compression and $M_e = 6 \sin \varphi'_e/(3+\sin \varphi'_e)$ in extension in which $\varphi'_c$ and $\varphi'_e$ are friction angles at the steady state in compression and extension, respectively. Hereafter, the conversions from α´ to φ´and from α´ or φ´ to M will be given where appropriate.

As described previously, the triaxial monotonic tests on CPB are intended to be performed at a relatively high axial strain rate (2%/min) in this study to minimize the effect of cement hydration on the monotonic response of CPB during the time frame of the experiment. To investigate the effect of axial strain rate, a series of triaxial compression tests were performed on the uncemented tailings at an effective confining stress of 400 kPa. This stress level was chosen simply because it was possible to obtain specimens at relatively similar starting void ratios (~0.680±0.005).

Figure 3a shows the stress path of four tailings specimens (MTCU4, MTCU5, MTCU6, and MTCU7) tested at different axial strain rates (2, 1, 0.5, and 0.1 %/min, respectively). All the specimens exhibit both contractive and dilative behaviours. The contractive behaviour of the specimens is accompanied by increasing pore pressure with axial straining, resulting in deviation of the stress path from the drained path. The contractive behaviour changes to a dilative behaviour at the maximum excess pore pressure (Figure 3b), corresponding to the phase transformation point (PT point) shown on the stress path. The stress state corresponding to maximum excess pore water pressure, $\Delta u_{max}$, is also plotted in Figure 3a. Beyond the PT point, the pore pressure ratio decreases with continued axial strain, indicating a dilative behaviour. Determination of $\Delta u_{max}$, and therefore the PT point, was unique for each of the datasets (i.e., Δu increased monotonically to $\Delta u_{max}$, and then decreased monotonically). A good statistical fit of the phase transformation line (PTL) to the PT data was obtained with a corresponding angle of $\alpha'_{PT}$ = 32.2° ($\varphi'_{PT}$ = 39.0°) in the stress space (Figure 3a).

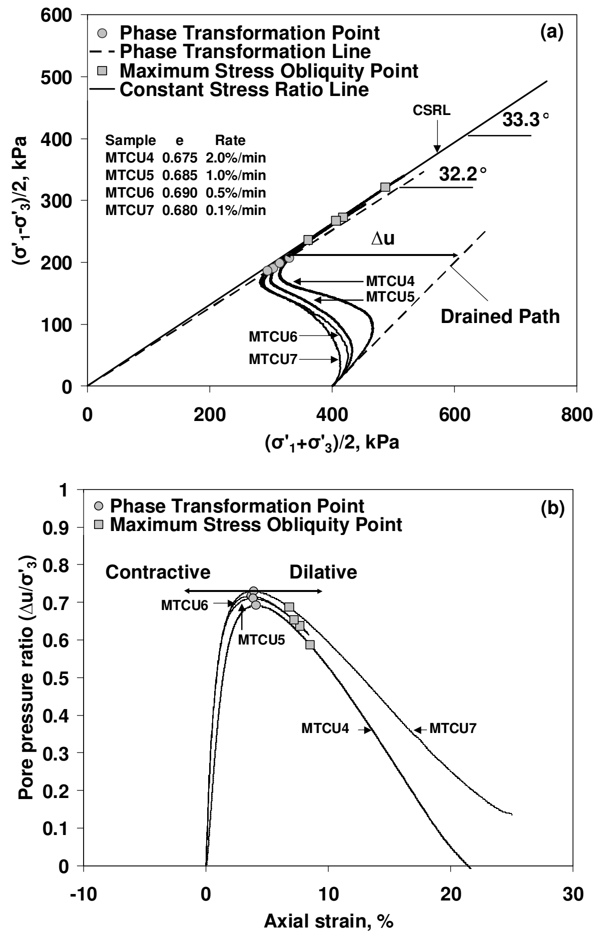


Figure 3: The effect of axial strain rate on the monotonic behaviour of the mine tailings in an undrained condition.

Determining the "failure line" for this material is more problematic. None of the deviator stresses of these specimens reaches either a peak or a steady state within the maximum axial strain range used in testing; nor does the excess pore water pressure, Δu, asymptotically approach zero. However, the stress paths coincide in a unique line at their dilative states, as shown in Figure 3a. This unique line corresponds to a temporary constant stress ratio around the maximum stress obliquity (i.e., maximum of $(\sigma'_1-\sigma'_3)/(\sigma'_1+\sigma'_3)$). To obtain this unique line, named as the constant stress ratio line (CSRL), a good statistical fit is applied to the data points at which the maximum stress obliquity is reached. Determination of the maximum stress obliquity point was unique for each of the data sets, as demonstrated in Figure 3d. The CSRL fit to these points has an angle of $\alpha'_{CSRL}$ = 33.3° in the invariant stress space with an equivalent Mohr angle of $\varphi'_{CSRL}$ = 41.1°.

Two of those four specimens (i.e., MTCU4 and MTCU7) shown in Figure 3 were tested to higher axial strains. For these specimens, the stress-strain curves almost linearly increase after PT points, particularly between 5% and 15% axial strain. None of these have plateaus; they have not reached either a peak or a steady state. The implications of not achieving the steady state within 15% axial strain could be important and are considered in more detail in Moghaddam and Grabinsky (2010).

All the specimens tested at different axial strain rates approach the same constant stress ratio line after passing through the PT point. In other words, the different axial strain rates tested (0.1-2%/min) have no effect on the Mohr friction angle ($\varphi'_{CSRL}$) and the angle at the phase transformation state ($\varphi'_{PT}$). Although each stress path follows a similar trend, the lower the axial strain rate is the lower the stress path is located in the stress space. On the other hand, the stress paths for the specimens with low axial strain rates start deviating from the drained path earlier than the specimen tested at high axial strain rates. Also, the highest pore pressure ratio belongs to the specimen tested at the lowest strain rate. This suggests that the generation of pore water pressure is more developed in the specimens tested at lowest axial strain rates. The higher the axial strain rate, the lower the pore pressure ratio. In addition, although the uncemented tailings specimens exhibit different stress paths, the strain rate has no effect on the monotonic behaviour of the uncemented tailings. In other words, all the specimens show strain hardening type of behaviour within limits of void ratio and strain rates used for these tests.

Since the axial strain rate has no significant effect on the Mohr friction angle and the slope of PTL, the axial strain of 2%/min is used in this study to minimize the effect of cement hydration on the response of the material during testing.

## 5 UNDRAINED TRIAXIAL RESPONSE OF TAILINGS

The undrained monotonic response of three uncemented tailings specimens (i.e., MTCU1, MTCU3, and MTCU4) at different effective confining stresses (50-400 kPa) is shown in Figure 4.

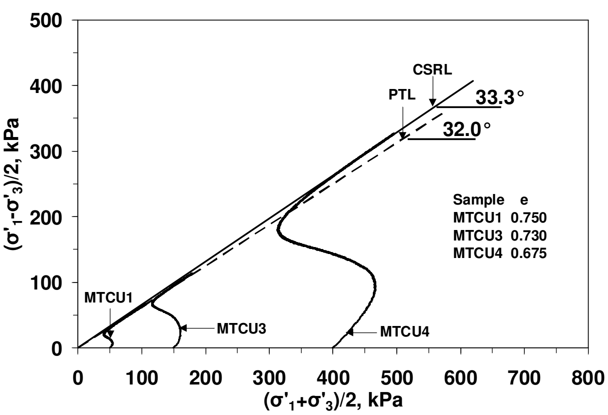


Figure 4: Undrained behaviour of normally consolidated mine tailings at different effective confining stresses.

All the undrained specimens show a contractive behaviour followed by a dilative behaviour after passing their PT points within the range of void ratio and effective confining stresses tested. A unique PTL with $\alpha'_{PT} = 32.0°$ and a unique CSRL with $\alpha'_{CSRL} = 33.3°$ ($\varphi'_{CSRL} = 41.1°$) were obtained for these undrained specimens, as explained previously.

No steady state was reached for these silt-sized, uncemented mine tailings at different effective confining stresses indicating a strain hardening behaviour type of response with no sign of flow liquefaction or limited liquefaction. This behaviour is consistent with the behaviour medium dense sand tested by Chu et al. (2003), silt-sized tailings tested by Crowder (2004) and slime tailings presented by Vick (1990).

## 6 UNDRAINED TRIAXIAL RESPONSE OF CPB IN COMPRESSION AND EXTENTION

The monotonic compression test results for three 3%CPB specimens cured for 4 hours before shearing (i.e., CPBCU3, CPBCU5, and CPBCU6) at different effective confining stresses (i.e., 50, 100, and 200 kPa, respectively) are shown in Figure 5. Generally, the CPB specimens showed both contractive and dilative behaviours similar to the response of the uncemented mine tailings in this study. A unique PTL with a corresponding angle of $\alpha'_{PT} = 32.7°$ ($\varphi'_{PT} = 39.9°$) was identified for these specimens in the stress space (Figure 5a). All the stress paths approached a unique CSRL with a corresponding angle of $\alpha'_{CSRL} = 33.8°$. This angle is corresponding to a Mohr friction angle of $\varphi'_{CSRL} = 42.0°$.

Similar to the response of uncemented tailings, no steady state is reached for these CPB specimens in the triaxial compression. Although the void ratio of the specimens in compression varies from 0.775 to 0.840, a unique angle of the CSRL was determined for these specimens suggesting that the effect of void ratio is not significant. Note that this line was obtained with a good statistical fit ($R^2 = 0.9999$) to the data points at which the maximum stress obliquity is reached, as explained in Section 4.

The monotonic extension results of three 3%CPB specimens cured for four hours (i.e., CPBCU7, CPBCU8, CPBCU9) at different effective confining stresses (i.e., 50, 100 and 150 kPa, respectively) are also shown in Figure 5. Generally, the stress paths show both contractive and dilative behaviours, while the contractive behaviour is accompanied by a temporary instability (TI) state. Both the PT and TI points are shown in Figure 5a. The PT points were strictly determined as the maximum pore pressure ratio, as demonstrated in Figure 5b. The temporary instability points can be defined as the temporary peak values for deviator stress. Similar to the response of CPB in compression, no steady state is reached for the material in extension. A unique PTL was identified for CPB in extension with an angle of $\alpha'_{PT} = 27.6°$ ($\varphi'_{PT} = 31.5°$). A good statistical fit of the temporary instability line (TIL) to the TI data was obtained with a corresponding angle of $\alpha'_{TI} = 22.1°$, as shown in Figure 5a. This figure also shows that the stress paths in extension approach a unique CSRL after passing the PT points by showing a dilative behaviour. The CSRL obtained for the CPB in extension tests has an angle of $\alpha'_{CSRL} = 32.9°$ ($\varphi'_{CSRL} = 40.3°$).

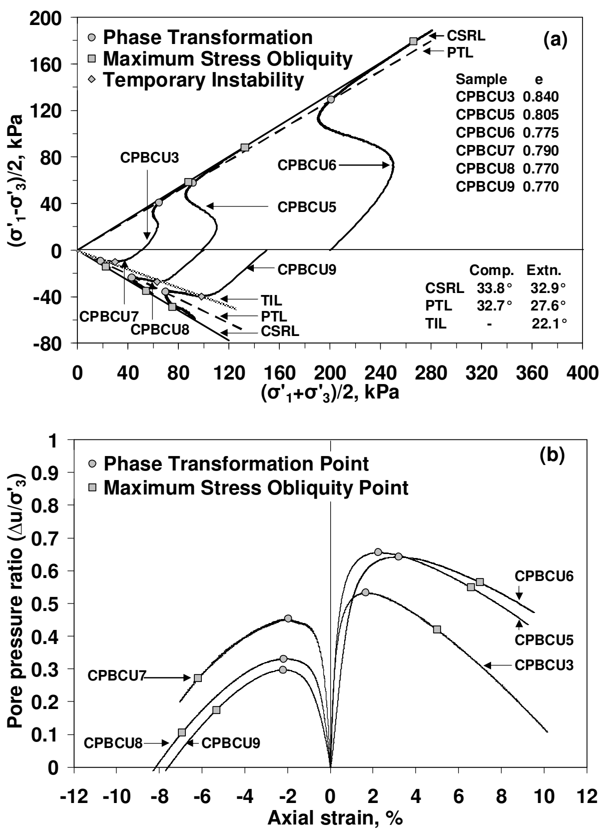


Figure 5: The undrained behaviour of CPB specimens cured for four hours in compression and extension.

## 7 CONCLUSIONS

Flow liquefaction was not achieved for the normally consolidated uncemented mine tailings in the triaxial consolidated undrained compression monotonic tests within the range of effective stresses (50 to 400 kPa) tested in this study. This suggests that the stress path for the tailings is below any potential flow liquefaction surface (FLS) or steady state point if, in fact, these concepts are even applicable to the material under consideration in the context of liquefaction behaviour of sands (e.g., Kramer 1996 and Ishihara 1996).

The compression monotonic response of the tailings showed that the PTL and the CSRL pass through the origin in the stress space, which is consistent with the purely frictional behaviour expected of the low plasticity index tailings. Note that the angle of the CSRL for the uncemented tailings is high relative to that of sands. For example, $\varphi'_{CSRL}$ for the uncemented tailings (i.e., 41.1º) is higher than that of sand (i.e., $\varphi'_{CSRL}$ = 36.9º) tested by Chu et al. (2003) even though the void ratios of the uncemented tailings (e = 0.675-0.750) is higher than those of the sands.

Flow liquefaction was also not achieved for the 3%CPB specimens cured for four hours in the undrained compression monotonic tests in this study. These normally consolidated CPB specimens showed both contractive and dilative behaviour at different void ratios (i.e., 0.805-0.840) and initial effective confining stresses (50-200 kPa) tested in this study. This result is consistent with the result of the same mine tailings without cement tested in this study.

The CSRL and the PTL determined for the 3% CPB cured for four hours passed through the origin of the stress space. This indicates that the monotonic behaviour of CPB after four hours of curing has no cohesion, the same as for uncemented mine tailings or purely frictional soils. However, the angle of the CSRL and the PTL for the CPB is higher than that for the uncemented mine tailings. In other words, the addition of 3% cement and 4 hours cure would increase $\alpha'_{CSRL}$ and $\alpha'_{PT}$, but not cohesion. This result is consistent with the results of Vicat needle test and EC measurements presented in Figure 2. As the electrical conductivity and Vicat needle results showed, the critical time is 5 hours after the addition of cement where the acceleration phase of hydration begins. Therefore, no significant amount of hydration products is formed at four hours where the CPB specimens tested, resulting in no development of cohesion. Nevertheless, the addition of 3% cement results in an increase in the percentage of fines in the CPB specimens in comparison with the uncemented mine tailings, and consequently the packing density of CPB increases. Therefore, the higher packing density might be responsible for having higher friction angle for CPB.

Flow liquefaction was also not achieved for the CPB specimens in extension. However, the undrained extension response of CPB is slightly different from the compression response. The specimens in extension show a strain softening type of behaviour with temporary instability. This temporary instability type of behaviour is similar to the behaviour of loose clean sand suggested by Yamamuro and Covert (2001). Similar to the behaviour of CPB in compression, the TIL, the PTL and the CSRL passed through the origin of the stress path space in the extension tests. This suggests that the 3%CPB after 4 hours of curing in extension exhibits a purely frictional behaviour. Note that the angle or slope of the CSRL in extension is lower than that in compression, which is consistent with the behaviour of sand in compression and extension (e.g., Boukpeti et al. 2002 and Imam et al. 2005). To interpret the difference between the behaviour for soils in extension and compression, the specimen anisotropy due to laboratory sample preparation technique has been noted (e.g., Vaid and Thomas 1995). In other words, the response of soil in extension might not be an actual response since the natural deposition of soil is different from the laboratory sample preparation techniques. In the case of CPB, however, the sample preparation method has been developed based on the actual placement of CPB in the field; therefore, the response of CPB in an extension test is most likely to be the same as actual response in the field.

## 8 ACKNOWLEDGEMNETS

Some part of the financial support received through the National Sciences and Engineering Research Council of Canada (NSERC) is gratefully appreciated.

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