Mechanical Properties of Cemented Paste Backfill under Low Confining Stress

by

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A thesis submitted in conformity with the requirements for the degree of Master of Applied Science
Department of Civil & Mineral Engineering
University of Toronto

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2019

Abstract

Cemented paste backfill (CPB) plays an increasingly important role in the mining industry due to rapid delivery rate, tight-filling characteristics, relatively insignificant water management issues during filling, and generally uniform as-placed mechanical properties. The most common basis for assessing backfill strength is Unconfined Compressive Strength (UCS), and some of the most commonly used design methods have UCS as a fundamental design parameter. However, actual CPB properties and behavior under the range of confining stresses relevant to these design methods has been studied very little. This work uses UCS tests and a novel direct tensile test method and compares test results with strengths obtained from direct shear tests. A form of punching shear was also investigated, but the results were found unsatisfactory for design purposes. For materials with UCS up to about 1 MPa, the UCS and tensile strengths are consistent with the Mohr-Coulomb failure envelope obtained from direct shear tests.
Acknowledgments

First of all, I would like to thank my supervisor, Dr. Murray Grabinsky for his guidance and advice. Throughout my master studies, I made each step by virtue of his guidance and encouragement. His suggestions and advice were invaluable for my academic development.

I would also like to thank Dr. Kamran Esmaeili for reviewing my thesis, and his helpful comments and suggestions.

I am truly thankful for my lab mates Mohammadamin Jafari, Mohammad Shahsavari, and Wendal Yue for their always prompt help whenever I need. I also want to thank them for their companionship and for providing a pleasurable and friendly working atmosphere. A special thank you to Dr. Lijie Guo from Beijing General Research Institute of Mining and Metallurgy for his assistance and participation.

On a personal note, my most heartful thank you must be saved for my parents, Yuxing & Xiaoxia for their unconditional support, encouragement, and love, without which I would not have come this far.

This research was supported by the Natural Sciences and Engineering Research Council Canada and Barrick Gold Corporation; as well as Lassonde Entrepreneurship Scholarship, and University of Toronto Graduate fellowship.
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1 Introduction

Cemented Paste Backfill (CPB) is a composite mine backfill technique used as regional ground support. Composite backfilling is classified into 3 categories: rock fill, hydraulic fill, and pastefill, based in part on their particle sizes, but also on water content and water retention properties. Rock fill is predominantly composed of crushed rock and is typically placed dry or possibly coated with a slurried binder. Hydraulic fill generally contains a significant fraction of sands and places a limit on fines content (e.g., no more than 10% passing 0.010 mm). It is transported in pipelines as a slurry and the fines restriction is intended to promote drainage of the significant water volumes during placement. Even so, the filling must be conducted in stages with times between successive pours allowing the drainage to occur (Patterson & Cooke, 2019).

Paste fills are characterized by significantly more finer particles (e.g., at least 15% passing 0.020 mm). The consistency resembles toothpaste, and the material is so thick that it must be transported in pipelines using positive displacement pumps. The fines generate significant capillary suction and the water content is low enough that very little water is lost from the backfill upon placement in the stope (Patterson & Cooke, 2019).

The use of CPB is currently practiced in many modern mines throughout the world, particularly in Canada and Australia. CPB plays an increasingly important role due to safety, cost-efficiency and environmental benefits. CPB is a homogeneous mixture obtained by mixing tailings with water and hydraulic binder. Its designs are based on regional ground conditions, tailing behaviors, and operational requirements. CPB is consisted 70 to 85% fines by weight and incorporates 2% to 9% binder (generally Portland cement and possibly fly ash or ground blast furnace slag) (Simons and Grabinsky, 2013). In this work, binder content is determined in proportion to solid weight. CPB is placed in the mined-out stope to form a self-supporting structure which is used as regional ground support. The stability of the CPB structure is governed by its mechanical properties under low confining stress conditions.
CPB technology was implemented in Canadian mines since the 1980s (Mitchell et al. 1982; Grabinsky and Simms 2006, Grabinsky 2010). It has developed into the optimal backfilling technique and become common practice in mining operations. Since late the 1990’s, research groups at the University of Toronto have conducted extensive research on CPB both in the laboratory and at operating mine sites (Grabinsky and Simms 2006, Grabinsky 2010, Grabinsky et al., 2013, Grabinsky et al., 2014, Grabinsky 2019), and these have led to significant improvement in the design and application of paste backfill in underground hard rock mines.

One of the most common design methods for exposed backfill sidewalls (for any form of backfill) is based on Mitchell, Olsen, and Smith (1982) and is commonly referred to as “Mitchell’s Method”. Laboratory scale models of sand backfills were conducted with sidewall exposure, and interpretation of the observed failure mechanism led the authors to propose a sliding block model based on Figure 1, where L is the exposed side length, H is the exposed face height, and w is the backfill depth.

![Figure 1: Mitchel’s wedge mechanism schematic](image)

It is assumed the back face becomes detached through tensile failure and so does not contribute to fill mass stability. It is further assumed that the two sidewalls remaining in contact develop a constant “bond strength” equal to half of the Unconfined Compressive Strength (UCS). The
authors first developed a solution based on sliding along the inclined base plane, but then go on to make some rationalizations and finally recommend a simplified expression for the limiting backfill strength, namely:

\[
UCS = \frac{\gamma H}{1 + \frac{H}{L}}
\]

In this expression \( \gamma \) is the backfill’s unit weight. Importantly, the assumption of a constant bond strength equal to \( \frac{1}{2} \) UCS is based on direct shear and uniaxial compressive strength tests conducted by the authors on their backfill materials, and presented in their 1982 paper as follows:

Figure 2: Assumed strength envelope (Mitchell, Olsen, & Smith, 1982)
Note that test results for two materials are presented on this figure, one for material with UCS about 200 kPa, and the other for material with UCS a little more than 100 kPa. The authors never presented stress-displacement results to justify their claim that, at low strain levels, the bond strength is equal to ½ UCS. Furthermore, the sloped portion of the failure envelope for the 100 kPa material is inconsistent with what should otherwise be a tangent to the UCS’s Mohr Circle.

It is surprising that, despite the wide acceptance of Mitchell’s method for design, these material test results were never called into question nor compared to similar types of tests conducted on other backfill materials. Therefore, the primary objective of this work is to determine the nature of the failure envelope for a specific backfill material and compare the results with those presented in Mitchell et al. (1982). Although only one material is tested in this work, it is broadly representative of paste backfills used at many hard rock mines. In support of this main objective, the planned approach was to complement the “standard” UCS test with a new kind of direct tension test and to compare the resulting Mohr circles with direct shear test results. A form of punching shear test was to be investigated based on promising work demonstrated for conventional concrete. The supposed advantage of the punching shear test, in addition to its simplicity, is that it is supposed to combine shear and tensile normal stresses on the failure surface, which cannot be achieved with conventional direct shear.

In general, the backfill design engineer needs to know not only the final backfill strength, but how this strength evolves during the curing process. As a result, it was decided to test at a range of binder contents and curing times: 4.2%, 6.9%, and 9.7% binder contents; and 3, 7, 14, and 28 day curing times.

The basis for each of the indicated material test methods is given next.

1.1 Direct Shear Tests

Direct shear is a rapid method to determine strength properties through a predefined shear plane. It is characterized by relatively short drainage path which allows pore pressure to dissipate rapidly (ASTM Standard D3080, 2011). The shear response of CPB has been the focus of several research projects including the shear response of cement mine tailings, and backfill-rock
interfaces (Nasir and Fall, 2018, Belem et al., 2016), simulated tailings (Kesimal et al., 2005; Klein and Simon, 2006; Fall et al., 2007; Veenstra, 2013; Niroshan et al., 2017) and cemented soils, shotcrete, and concrete materials (Cresswell and Barton, 2003; Nasir and Fall, 2008; Kesimal et al., 2005). Direct shear provides an effective approach to determine shear strength properties (Mitchell et al., 1982).

The direct shear tests were performed on samples in order to assess the shear strength of CPB. The apparatus generates pressure with a fixed load. This pressure allows sample shearing through the relative displacement of 2 halves of a shear box while the horizontal and vertical displacements, as well as the forces, are completely controlled. The shear box is connected to a data acquisition system.

The shear behavior particularly the cohesion and the angle of friction is an important parameter in CPB numerical analysis. The geomechanical design of CPB is determined by the normal stress, $\sigma_n$, and the shear stress, $\tau$.

$$\sigma_n = \frac{F_n}{A}$$

$$\tau = \frac{F_s}{A}$$

Where, $F_n$, is the applied normal force, $F_s$, is the applied shear force, and $A$ is the original area.

There is a need to increase our understanding of the shear properties of CPB. The results have important implications including allowing mine engineers to design and optimize the filling strategy and the cement content and curing time of CPB.

1.2 Unconfined compressive Strength Test

The Unconfined Compressive Strength test (UCS) is widely used in industry and research due to its simple and effective way to characterize a material’s response to compressive loading. By subjecting samples to compressive displacement along a single axis, the change in dimensions and measured load are used to determine the stress-strain behavior.
Although the test technique appears simple, there are many practical details to consider in order to get a reliable test result. These are considered in detail later in the thesis.

1.3 Direct Tensile Test

The tensile strength of CPB is an important parameter particularly in the design of undercut structures. In this thesis, the direct tensile strength is determined from dog-bone shaped specimens by using a compression-to-tension load converter (Perras and Diederichs, 2014). The dog-bone shape was initially suggested as the optimal condition for creating a uniform tensile stress field in the specimen’s mid-section while mitigating extraneous effects from end conditions (Hoek, 1964). Subsequent studies focused on extending the technique to different materials, for example, soft organic rock (Tolooiyan et al., 2014). However, this is the first work to adapt the proposed test technique and demonstrate its effectiveness for relatively soft and weak materials.

1.4 Punching Shear Test

Strength testing under tensile normal stress is particularly challenging, and not readily carried out using conventional direct shear devices. Boulifa, et al. (2013) used a punching shear test method to develop coupled tensile and shear stress states. In their studies, the sample is prepared with a reinforcement cage and 2 Styrofoam molds as shown in Figure 3 which creates two stress-free zones along the top and base.

![Figure 3: Punching shear test setup (Boulifa et al., 2013)](image)
The sample is loaded on the top which forces a shear band to form through to the base, which results in coupled shear and tensile stresses. The advantage of punching shear is that it can be carried out easily. However, the tensile behavior of the CPB is not fully understood due to its soft and brittle behavior. This research aims to increase our understanding of the tensile and shear properties of the CPB by integrating interpretations of punching shear results with the results from the other previously described test methods.

The remainder of this thesis provides details of the experimental setups, sample preparation techniques, specifics of executing the tests, experimental design and test results, and interpretation and significance of the test results.
Chapter 2
Experimental Setup

2 Experimental Setup

2.1 Materials

2.1.1 Mill tailing samples

The samples were collected from Barrick’s Williams Operation in the Helmo mining camp. The Williams mine is in North West Ontario, approximately 35km west of the township Marathon. The ores are primarily amphibolite-facies Archean load gold deposits (Roscoe Postle Associates, 2017). The deposits were extracted with a mixture of long hole stope and Alimak stope which were subsequently backfilled with CPB. The tailings in Williams operation consisted of silica, quartz, feldspar, and plagioclase. Material tests conducted in the fall of 2017 showed that the sulfides in the tailings can oxidize which alters the material’s initial gray color to orange color when exposed to oxygen. The tailings consisted of predominately silt size particles.

Starting in 1997, research at the University of Toronto was conducted on laboratory properties and in situ monitoring at William Mine, Golden Giant Mine, Kidd Mine, and Cayeli Mine (Roux et al., 2005; Grabinsky, 2010; Thompson et al., 2012; Grabinsky et al., 2013). These studies showed that mechanical properties of CPB depend upon their physical and chemical characteristics such as the tailings mineralogy, particle size, water content, and binder type, etc. These mechanical properties varied significantly during the 0 – 28 day curing time. The findings have important implications for the research as well as mine design.

The paste backfill must be designed so that it would reach its target compressive strength values at 28 days of curing and beyond. This can be done by choosing optimal mixtures for each type of tailings. The short- and long-term mechanical properties depend on the binder type and concentration. However, the “optimal” binder design must also consider the total cost of the backfilling operation.
2.1.2 Hydraulic binders and Paste backfill mixtures

In this study, Normal Portland Cement was used as the basic binding agent in the experiments, reflecting current practice at Williams Mine. 4.2% and 6.9% cement contents were used to simulate typical mining practice; 9.7% cement content was an upper bound and reflects the maximum concentrations used for critical applications such as undercut structures. The cement content was calculated by total solids, i.e.:

$$c_s = \frac{M_{cs}}{M_D} \times 100\%$$

Where $c_s$ is cement content, $M_{cs}$ is the mass of cement, and $M_D$ is the total dry mass (tailings and cement).

The samples were prepared with 28% mine water content based on recent field study which showed mine water content ranged from 26.5% to 29.6 % in Williams Mine (Grabinsky et al. 2013). The water content was calculated based on the mass of water over the total mass, which shown as:

$$w = \frac{M_w}{M_D + M_w} \times 100$$

Where $w$ is water content, $M_w$ is the mass of water, and $M_D$ is the total dry mass.

2.2 Equipment

The tests were conducted with constant strain uniaxial, and constant displacement direct shear loading apparatus. The constant displacement controlled direct shear apparatus was used for the direct shear experiments. A load frame was used for UCS, punching shear, and direct tensile test.

2.2.1 Direct Shear Apparatus

The direct shear apparatus has been accepted as the most efficient approach to characterize the strength envelope. The normal stress was applied by a fixed load in the increment of 6kg and 10kg plates. The samples were fitted into a 60x60mm shear box and sheared at a constant strain
rate with the electric motor as shown in Figure 4. The top and bottom were spaced 0.68mm based on ASTM D3080.

![Direct shear apparatus](image)

**Figure 4: Direct shear apparatus**

The loading rate was controlled by mechanical gearbox. The tests were performed at constant shear displacement. The linear displacement was recorded with LVDT placed against the sample apparatus. The vertical dilation was tracked with LVDT placed perpendicular against the sample. The shear resistance was determined with a load cell in the direction of travel. The LVDTs and load cell were connected to a data acquisition system. The values were tracked at a frequency of 1 Hz.

The data acquisition system consisted of 2 ICP CON i-7016 data acquisition modules linked to ICP CON i-7520 RS323 to RS485 converter as shown in Figure 5. The system was powered by a programmable power source. The ICP CON i-7016 converted the analog signal to digital signal. The ICP CON -7520 combined the results and transmitted the data into a single serial port. The data was processed through the Data Acquisition System Laboratory (DiasyLab) System.
DaisyLab software is a combination of real-time acquisition, analysis, and control systems. The system was used for calibration and data acquisition. The horizontal and vertical LVDTs were calibrated using second-order equation differential based on displacement and the change in conductivity. The load cell was calibrated with the same mechanism. The process schematic is shown in Figure 6.
The data acquisition system consisted of a load cell input and 2 LVDT inputs as shown in Figure 7. The input values were converted into digital readings based on the calibration. The values were displayed in real-time and saved into CSV files.

![Data acquisition schematic](image)

**Figure 7: Data acquisition schematic**

### 2.2.2 Loading Frame

The UCS, punching shear, and direct tensile tests were carried out with a Wille Geotechnik Tabletop Electromechanical Apparatus as shown in Figure 8. The loading apparatus was a servo-controlled machine with variable loading rate. The system was equipped with dual linear variable displacement transistor (LVDT) and 10kN load cell. The duel LVDT consisted of internal and external LVDT units. The internal sensor was used to track the limit of the servo travel and served as redundancy measurement. The external sensor was used to control the rate of loading and displacement.
The system can be controlled manually or linked to the computer through Geosys software provided by Wille Geotechnik. This allowed the system to function both as a standard load frame and programable oedometer. In this study, the system was used as a load frame based on the loading rate of 1% per minute for the unconfined compressive strength, and 0.5 mm per minute for the punching shear test and the direct tensile test.

2.3 Experimental Protocol

From October 2017 to July 2019, 4 experimental methods and a total of 180 tests were conducted to examine the low confining stress properties of CPB. These included direct shear, unconfined compressive strength (UCS), punching shear, and direct tensile strength. The direct shear and UCS tests were performed in accordance with ASTM D2166 and ASTM D3080. The
punching shear and direct tensile test were performed in conjunction with numerical simulations described subsequently.

The 4-curing time of 3, 7, 14, and 28 days were selected for the testing program. The 3-day cured material represented early material strength. The 7 and 14-day tests were used to quantify the strength development, and 28-day tests were for the full strength. The tests were classified into 4 main groups of: direct shear, UCS, punching shear, and direct tensile tests. The experimental protocols are summarized below.

2.3.1 Direct shear

The direct shear tests followed standards of:

- AASHTO T236: Standard Method of Test for Direct Shear Test of Soil Under Consolidated Drained Conditions

In the study, a total of 72 trials were conducted with the combination of 3 cement contents of 4.2%, 6.9%, and 9.7%; and 4 curing time of 3, 7, 14, and 28-day at the 6 stress intervals, the configurations are shown in Table 1.

<table>
<thead>
<tr>
<th>Mix reference</th>
<th>Binder content (%)</th>
<th>Curing time (days)</th>
<th>Trial per mix</th>
<th>Number of samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>DS-CS04</td>
<td>100% GPC</td>
<td>4.2</td>
<td>3, 7, 14, &amp; 28</td>
<td>6</td>
</tr>
<tr>
<td>DS-CS07</td>
<td>100% GPC</td>
<td>6.9</td>
<td>3, 7, 14, &amp; 28</td>
<td>6</td>
</tr>
<tr>
<td>DS-CS10</td>
<td>100% GPC</td>
<td>9.7</td>
<td>3, 7, 14, &amp; 28</td>
<td>6</td>
</tr>
</tbody>
</table>

The stress ranges were determined from field research, an example of which is shown in Figure 9. The study showed the maximum in situ stress range from 150 to 200kPa.
Figure 9: In-situ stress regime (Grabinsky, 2010)

The 6 load increments of 1kPa, 20kPa, 60kPa, 80kPa, 130kPa, and 210kPa confining stress were selected for this study which corresponded to observed in-situ stress condition. The load increments were applied using load plates in increments of 6kg or 10kg. The loads were placed at intervals of 45s to allow excess porewater pressure to dissipate.

The rate of the loading was determined by the time to achieve 50 % consolidation. The consolidation was plotted on a linear-log scale for each load increment. The best fit lines were placed to determine primary consolidation. The time to achieve 50 % consolidation was determined by linear regression. The loading rate $R_d$ was determined by the distance to failure, $d_f$, and the time to failure, $t_f$.

$$R_d = \frac{d_f}{t_f}$$

The time to failure, $t_f$, was determined by 50 % consolidation:

$$t_f = 50 \ t_{50}$$
The nominal confining stress was determined by the applied force and the original area.

\[ \sigma_n = \frac{F_n}{A} \]

The nominal shear stress was calculated by the shear force and the original area.

\[ \tau = \frac{F_s}{A} \]

The samples were prepared in the 4-part spilt mold based on Veenstra’s (2013) experiments as shown in Figure 10. The molds were modified to use sulfate resistant plastic to limit the effect of oxidation potential. The preliminary test showed that the steel mold has a significant amount of rust built up which led to the concern of sample degradation. The modified mold was made of PGN plastic which resulted in significantly less rust.

The 4-part spilt mold consisted of 2 side enclosures with sill inserts and 2 base enclosures with O-ring inserts. The O-ring and the sill were coated with a layer of vacuum grease to facilitate a better seal. The sample enclosure was sprayed with silicone lubricant to facilitate the demolding process. The sample case was held together with 12 #10-32 screws.

Figure 10: Mold setup
Direct shear tests were performed in drained and consolidated conditions. The samples were loaded in increments of 45s to alleviate the excess pore pressure and thus consolidate prior to the onset of the test. The sample enclosure was filled with water to reduce the effect of suction during the test.

The displacement-strain curve was used to calculate the shear modulus. The shear modulus was determined indirectly by the definition of engineering strain. The shear modulus, $G$, was taken based on the shear stress, $\tau_{xy}$; and approximate gauge length for the shear, $l$; and the lateral displacement, $\Delta x$. It must be noted that this definition of shear strain is generally not accepted, however for CPB it has been considered to be reasonable in the elastic range given the very discrete form of the ultimate failure surface, as shown later in photographs. The shear modulus was calculated based on the shear stress, $\tau_{xy} = F/A$, and engineering shear strain, $\gamma_{xy} = \Delta x/l$, giving:

$$G = \frac{\tau_{xy}}{\gamma_{xy}} = \frac{Fl}{A\Delta x} = \frac{\tau_{xy}l}{\Delta x}$$

2.3.2 Unconfined compressive strength

Unconfined compressive strength (UCS) tests were conducted in accordance with standards:


- ISRM Commission on Standardization of Laboratory and Field Tests: Suggested Methods for Determining the Unconfined Compressive Strength and Deformability of Rock Materials

In this study, a total of 36 trials were conducted with the combination of 3 cement contents of 4.2%, 6.9%, and 9.7%; and 4 curing time of 3, 7, 14, and 28-day with 3 trials each, the configurations are shown in Table 2.
Table 2: Unconfined compressive strength mix reference

<table>
<thead>
<tr>
<th>Mix reference</th>
<th>Binder</th>
<th>Binder content (%)</th>
<th>Curing time (days)</th>
<th>Trial per mix</th>
<th>Number of samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS-CS04</td>
<td>100% GPC</td>
<td>4.2</td>
<td>3, 7, 14, &amp; 28</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>UCS-CS07</td>
<td>100% GPC</td>
<td>6.9</td>
<td>3, 7, 14, &amp; 28</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>UCS-CS10</td>
<td>100% GPC</td>
<td>9.7</td>
<td>3, 7, 14, &amp; 28</td>
<td>3</td>
<td>12</td>
</tr>
</tbody>
</table>

The samples were prepared with 4-part 35mm split molds as shown in Figure 11. The molds consisted of the base cap, top cap, 2-side enclosures, O-ring inserts and the seals at the contact area. The O-ring insert and the seals were coated with vacuum grease to seal the sample. The sample mold was coated with silicone lubricant to facilitate the demolding process. The samples were filled in 3 lifts and rodded 20 times to limit air voids.

![Figure 11: Schematic of Unconfined compression strength mold](image-url)
The samples were loaded at 1% per minute which ranged from 0.65 to 0.75mm/min. The tests were carried out underwater in a sample enclosure to reduce suction effects, as shown in Figure 12. The vertical deformation was measured with an LVDT placed on top of the sample enclosure.

![Diagram of Unconfined Compression Strength Test Setup](image)

**Figure 12: Unconfined compression strength test setup**

The UCS was calculated using the ASTM standard. The UCS was corrected by the cross-sectional area by longitudinal strain. This is shown as:

\[ A = A_0 \times \frac{1}{1 - \frac{\varepsilon}{100}} \]

\[ \varepsilon = \frac{\Delta L}{L_0} \times 100 \]
Where, $A$ is the cross-sectional area, $A_0$ is the original area, and $\varepsilon$ is the longitudinal strain. The UCS strength is calculated by the ultimate load, $P$, and the calculated cross-sectional area, $A$. This is shown as:

$$\sigma_c = \frac{P}{A}$$

2.3.3 Punching shear

The punching shear tests were carried out with a strain control loading frame, which was modified from the apparatus used in concrete tests by Boulifa et al. (2013). The sample was placed between a base plate and a loading plate and constricted with an external ring as shown in Figure 13. The base plate and the loading ring formed a preferential failure surface along the frustum. The external ring restricted the lateral expansion of the sample.

![Figure 13: Punching shear conceptual setup](image)

The punching shear tests were used to characterize the coupling of tensile and shear stresses. The punching shear tests were performed at the same curing intervals as the direct shear and the UCS tests. The tests were performed in 20 mm, 30 mm, and 50 mm puncher diameters with base
angles of 42°, 59°, and 71° respectively as shown in Figure 14. The stress distributions for each setup were analyzed using finite element models.

Figure 14: Punching shear test configurations

The punching shear samples were analyzed with RS2.0 v9.0. The analyses were modelled axisymmetrically around the center to stimulate three-dimensional stress distributions, and a strain softening material model was used. The model used parameters from UCS and direct shear tests. The model was laterally restrained with pin connections as shown in Figure 15. The model was analyzed by displacement control at the top of the sample. The load step was taken based on the rate of deformation.

Figure 15: Punching shear numerical simulation

The samples were prepared with 4-part 76.2 mm spilt molds as shown in Figure 16. The mold consisted of base cap, top cap, 2 side enclosures, O-ring and seals. The samples were prepared using the same method as the UCS specimens. The O-ring and seals were coated with vacuum
grease. The base and the side enclosure were coated with silicone lubricant to facilitate the demolding process. The sample was poured in 3 lifts and rodded 20 times per lift and then placed in the water to eliminate the effect of oxidation and suction. The samples were loaded at 0.5mm per minute. The test was performed underwater to reduce the effect of suction and oxidation.

36 trials were conducted with combinations of 3 cement contents of 4.2%, 6.9%, and 9.7%; and 4 curing time of 3, 7, 14, and 28 day with 3 configurations. The configurations are shown in Table 3.

Table 3: Punching shear mix reference

<table>
<thead>
<tr>
<th>Mix reference</th>
<th>Binder</th>
<th>Binder content (%)</th>
<th>Curing time (days)</th>
<th>Trial per mix</th>
<th>Number of samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>PS-CS04</td>
<td>100% GPC</td>
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<td>3, 7, 14, &amp; 28</td>
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<td>12</td>
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<td>3, 7, 14, &amp; 28</td>
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<tr>
<td>PS-CS10</td>
<td>100% GPC</td>
<td>9.7</td>
<td>3, 7, 14, &amp; 28</td>
<td>3</td>
<td>12</td>
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</table>
2.3.4 Direct tensile

The direct tensile tests were used to determine the uniaxial tensile strength of CPB. Direct tensile tests were carried out with compression to tension load converter. The compression to tension load converter was designed to accommodate a castable specimen. The loading apparatus consisted of upper and lower apparatus as shown in Figure 17. The upper apparatus was composed of the base plate, 2 bottom plates, and 4 rods, and the lower apparatus consisted of a base plate, 4 rods, and 2 top plates.

![Compression to tension load converter](image)

**Figure 17: Compression to tension load converter**
The samples were prepared with 4-part rectangular dog bone mold. The mold consisted of the top cap, bottom cap, and 2 side caps as shown in Figure 18. The base and top caps contained seal inserts. The mold was held together with 16 #10-32 screws. The seal was coated with vacuum grease. The base and side enclosure were coated with silicone lubricant to facilitate the demolding process. The sample was filled 3 lifts with 20 taps per lift to remove air voids. The samples were placed in water to reduce the effects of suction and oxidation.

![Figure 18: Schematics of Direct tensile mold conceptual setup](image)

The geometry of the specimen was designed with RS2 v9.0. The model replicated the loading conditions with displacement boundary at the edge of the sample. The analysis showed curvature around the neck of the sample reduced stress with the optimal curvature to the diameter of 1.5, and the height of the sample around 1 as shown in Figure 19. The result showed the tensile stress at the middle was virtually uniform with the variation was less than 2.5%.
The tests were conducted in the compression to tension loading apparatus. The samples were secured in the upper and lower load apparatus as shown in Figure 18. The lower apparatus functioned as the support component which suspended the tensile specimen. The upper apparatus functioned as the loading component which transmitted compression load to tensile force. The samples were loaded at a constant rate of 0.5mm per minute. The displacement was measured with the internal LVDT in the load frame.

Figure 19: Direct tensile numerical simulation
The tensile strength was determined by the area and the weight of loading apparatus and sample. This is calculated as:

$$\sigma_t = \frac{P + w_l + \frac{1}{2}w_s}{A}$$

Where, A is the cross-sectional area measured at the center of the sample, P is the ultimate load, $w_l$ is the weight of the top-loading apparatus, and $w_s$ is the weight of the sample.

In the study, tensile strength was analyzed with 2 cement contents of 6.9% and 9.7% CPB (the higher content is typical for sill mat), at 3 curing intervals of 7, 14, and 28-day with 3 trials for each configuration. The combination totaled to 18 trials are shown in Table 4.

**Table 4: Direct tensile mix reference**

<table>
<thead>
<tr>
<th>Mix reference</th>
<th>Binder</th>
<th>Binder content (%)</th>
<th>Curing time (days)</th>
<th>Trial per mix</th>
<th>Number of samples</th>
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<td>PS-CS6.9</td>
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<td>6.9</td>
<td>7, 14, &amp; 28</td>
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<td>9</td>
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<tr>
<td>PS-CS9.7</td>
<td>100%GPC</td>
<td>9.7</td>
<td>7, 14, &amp; 28</td>
<td>3</td>
<td>9</td>
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</table>
3 Results and Discussion

In developing the experimental protocols, a series of preliminary experiments were conducted in October 2017 to determine the range of factors, including cement content and curing time, as well as the sample preparation techniques for the main experiments. Sample results are plotted in Figure 21 which show that increased cement content results in decreased compressibility. The trends are observed in all samples of 4.2%, 6.9%, and 9.7% CPB, along with the rationale for the conditions selected for subsequent experiments.

![Figure 21: Vertical settlement with curing time and cement content of CPB](image)

Preliminary experiments show that at lower cement content and curing time, the samples are easy to manipulate in the direct shear box. The misalignment between the samples and the testing...
apparatus are minute. As cement content increases, the samples become stiffer which results in stress rotation with misalignment that results in a nonlinear failure surface as shown in Figure 22.

**Figure 22: Non-linear failure surface**

The misalignment between the sample and the testing apparatus were addressed by 3-D printed molds. The mold is prepared to be within +/- 0.010 mm of the sample apparatus. The samples are made within the tolerance of 0.5% of the enclosure dimensions. The sides of the samples are lubricated with the silicon-based lubricant to reduce friction. The failure surface is characterized by linear fracture as shown in Figure 23.

**Figure 23: Typical test direct shear view**
3.1 Direct Shear

In the main experiments, direct shear results are presented by 3 types of analysis: strength properties, stress-displacement, and dilation-displacement. The combination of 3 cement contents of 4.2%, 6.9%, and 9.7%; and 4 curing time of 3, 7, 14, and 28 days were presented and compared in each analysis.

3.1.1 Strength properties

Figures 24-26 show the impact of curing time and cement content on the shear envelopes of CPB. It can be observed that regardless of the curing time and cement contents, the strength data follows a Mohr–Coulomb envelope.

\[ \tau = \sigma_n \tan \phi + c \]

Where, \( \tau \) is the shear stress, \( \sigma_n \) is the normal stress, \( \phi \) is the angle of frictional resistance, and \( c \) is the cohesion. The results indicate that shear resistance increases linearly with applied normal confining stresses with the coefficient of determination \( R^2 \) ranging from 0.98 to 0.99.

The peak shear strength is determined by the cohesion and the angle of frictional resistance, while the residual shear strength is governed by the frictional resistance alone. Results show cohesion increasing from early to advanced curing time, and from 4.2% to 9.7% cement contents.
Figure 24: Peak strength and shear strength of 4.2% CPB
Figure 25: Peak strength and residual strength of 6.9% CPB
Neither the peak nor the residual friction angles show a consistent trend with increasing cement content or increasing curing time. The peak friction angle ranges from 32 to 44 degrees with average of 36 degrees and standard deviation of 3 degrees. The residual frictional angle ranges from 39 to 43 degrees with average of 41 degrees and standard deviation of 1 degree.
The CPB strength property with time can be expressed by its cohesion, in which increase rate is characterized by a power law:

\[ c = a \cdot t^b \]

Where \( c \) is cohesion, \( a \) is a constant coefficient, \( t \) is the time in days, and \( b \) is a power coefficient. The coefficient of determination, \( R^2 \) value, is over 0.95 for 4.2%, 6.9%, and the 9.7% CPB. Strength increases with CPB cohesion, which is characterized by a gradual increase during early cure time from 3 to 7 days followed by a steady increase from 14 day to 28 days as shown in Figure 27. The results are more obvious in 9.7% than for 4.2% and 6.9%. This can be attributed to more developed cement bonds.

Figure 27: Cohesion and curing time
The rate of strength increase with cement content can be characterized by the linear relationship.

\[ c = m \cdot c_s + b \]

Where, \( c \) is cohesion, \( m \) is a constant coefficient, \( c_s \) is cement content, and \( b \) is a constant. The result shows cohesion increases with cement content. The rate of increase is consistent in all the sample from 4.2%, 6.9%, and 9.7%. The trend is more obvious in 28-day curing time than for 3, 7, and 14-day curing time as shown in Figure 28. This can be attributed to a longer curing time which results in higher degree of binder hydration.

![Figure 28: Cohesion and cement content](image)

The results of the direct shear test show the strength behavior is governed by cement content and curing time. CPB follows the Mohr-Coulomb envelope with the peak strength controlled by cohesion and the angle of frictional resistance, and the residual strength controlled by the angle
of frictional resistance alone. Cohesion increases at a constant rate with cement content and curing time. This can be explained in that the cohesion is controlled by the cement bond and the increase in cohesion is proportional to the cement content.

3.1.2 Stress-displacement

The stress displacement results of 4.2%, 6.9% and 9.7% cement contents at 3, 7, 14, and 28 days curing time are presented in Figure 29-31 for comparison. Typical stress-displacement curves follow 3 stages: the initial linear-elastic behavior, post-yield behavior from 0.5% onward strain, and post-fracture behavior from 1% strain onward.

The effects of curing time and cement content on stress-displacement behavior of CPB can be observed from Figure 29-31. Figure 29 shows that under low cement contents and early curing time, the shear stress at level 130 kPa and 210 kPa gradually increased with shear displacement until it reached the maximum, after which it became relatively constant with respect to shear displacement. In the case, the high stress could not reach the residual shear condition and led to the remolding of tailing particles. By comparison, the shear stress at a lower level (≤ 60 kPa) exhibited post-failure strain-softening, while stress =80 kPa, the stress-displacement curve showed slight to no post-failure strain softening due to the fact that a residual shear condition could not be achieved.

By contrast, high cement content in Figure 31 shows the shear stress of 9.7% CPB increases with shear displacement until it reaches peak stress. This behavior can be attributed to the breakage of cement bonds. After the peak, the shear stress decreases as the shear displacement increases, attributed possibly to the mobilization of the full frictional resistance or remolding of tailing particles. As the shear displacement continues to increase, the shear stress remains relatively constant (for stress≤80 kPa) or slightly decreases (for stress=130 kPa & 210 kPa). This behavior can be explained by dilation. It can be observed that the curve for 14 days curing time starts to reach peak stress in 28 days. The reason for this is the increasing degree of hydration with time. Similar behaviors were observed by Nasir and Fall (2008) on paste-rock interfaces.
Figure 29: Stress displacement curve of 4.2% CPB
Figure 30: Stress displacement curve of 6.9% CPB
Figure 31: Stress displacement curve of 9.7% CPB

The post-peak behavior is controlled by the frictional resistance which can be generalized into strain-softening or strain-hardening based on the confining stress. The softening behavior is characterized by an elastic perfectly plastic curve with an instantaneous drop in shear strength
following the fracture. The strain hardening behavior is characterized by ductile behavior with continuous strength increase following the fracture. The stress-displacement curve shows shear modulus increase with cement content and curing time. The trend is observed in all the trials of 4.2%, 6.9%, and 9.7% CPB.

Shear modulus with curing time is characterized by a gradual increase from 3 to 7 days followed by steady increase from 14 day to 28 days as shown in Figure 32. The results are consistent in 4.2%, 6.9% and 9.7% CPB. The rate of increase can be characterized by the power relationship:

\[ S = a \cdot t^b \]

Where S is shear modulus, a is a constant coefficient, t is the time in days, and b is the power coefficient. The rate of increase is consistent with cohesion behavior. The coefficient of determination, the \( R^2 \) value, is above 0.9 for the fit with the average shear modulus. The results are more obvious in 9.7% than that of 4.2% and 6.9%. This can be attributed to higher cement contents developing more cement bonds.

Figure 32: Shear modulus and curing time
The shear modulus with cement content is characterized by a linear relationship. The results are consistent in all 4-curing time of 3, 7, 14, and 28-day as shown in Figure 33. The rate of increase with the cement content is characterized by:

\[ S = m \cdot c_s + b \]

Where \( S \) is shear modulus, \( m \) is a coefficient, \( c_s \) is cement content, and \( b \) is a constant. The coefficient of determination, \( R^2 \) value, is greater than 0.8 when fit with the average shear modulus.

Figure 33: Shear modulus and cement content
3.1.3 Dilation-displacement

Figures 34-36 summarize the results of dilation analysis of 4.2%, 6.9%, and 9.7% cement contents, at 3, 7, 14, and 28 days of curing time. The dilative curve is characterized by an initial stationary phase prior to fracture followed by a dilation stage which is generalized into contraction or dilation. Dilative behavior varies with normal stress, curing time, and cement content.

The data suggest that dilative behavior depends on the stress level. Under low stress (≤80 kPa), samples exhibited expansion due to dilatancy as loading progressed. When stresses reach 130 kPa and 210 kPa, no dilative behavior was observed after the contraction phase. The reason for this is that the relatively high applied normal stresses degraded the test samples and thus, altered their dilative behavior. The results are consistent with the transition from strain-softening to strain hardening.

The comparison of dilation results for 4.2%, 6.9% and 9.7% CPB suggest that 9.7% CPB exhibits greater dilative response. The peak strength behavior is governed by the cement bond which is consistent with the stress displacement curve.
Figure 34: Dilation displacement of 4.2% CPB
Figure 35: Dilation displacement of 6.9% CPB
Overall, the strength behavior of the CPB is controlled by cure time and cement content. The strength of CPB increases with cement content and curing time. The trends are illustrated in peak strength characteristics, stiffness, and dilation behavior. The strength characteristics evolve from
frictional governed behavior at low cement content to cohesion governed behavior at higher cement content. The trends are observed in all samples of 4.2%, 6.9%, and 9.7% at 3, 7, 14, and 28 day of curing time.

The observed trends are consistent with the studies by Fall et al. (2007), Veenstra (2013), and Niroshan (2017). The shear strength increases with cement content and curing time. The results show peak strength behavior follows the Mohr-Coulomb envelope which is controlled by the angle of frictional resistance and cohesion, while residual strength is governed by friction alone.

### 3.2 Unconfined compressive strength (UCS)

The unconfined compressive strength (UCS) results are presented by 2 types of analysis: strength and stress-strain curves. The combination of 3 cement contents of 4.2%, 6.9%, and 9.7%; and 4 curing time of 3, 7, 14, and 28 days were presented and compared in each analysis.

Figure 37 shows the progressive failure mechanism of the UCS sample. The failure mechanism is characterized by 3 phases: the initiation of the hairline fractures in the sample near the contact and along the samples (Figure 37A), the propagation of the hairline fractures (Figure 37B), the coalesce of the hairline fractures and the formation of the extension of the crack leading to the failure (Figure 37C).
3.2.1 Strength properties

The UCS strength properties of CPB with curing time can be characterized by the power relationship as shown in Figure 38:

\[ \sigma_c = a \cdot t^b \]

Where, \( \sigma_c \) is the unconfined compressive strength, \( a \) is a constant coefficient, \( t \) is curing time, and \( b \) is the power coefficient. The coefficient of determination, the \( R^2 \) value, is greater than 0.95 in all of 4.2\%, 6.9\%, and the 9.7\% trials. The increase is more evident in 9.7\% than that of 4.2\% and 6.9\% CPB. This could be attributed to higher cement content resulting in more developed cement bond.
Figure 38: UCS with curing time

Figure 39 shows strength variation with the cement content. The UCS with cement content can be characterized by a linear relationship:

$$\sigma_c = m \cdot c_s + b$$

Where, $\sigma_c$ is the unconfined compressive strength, $m$ is a constant coefficient, $c_s$ is cement content, and $b$ is a constant. The coefficient of determination, $R^2$ value, is greater than 0.9 in all the 4.2%, 6.9%, and 9.7% CPB tests. The result shows UCS increase is more evident in the 28-day curing time than that of 3, 7, and 14 day. This could be attributed to more developed cement bonds.
The UCS strength results are analyzed with the direct shear envelope as shown in Figures 40-42. The UCS results are plotted in the Mohr-Coulomb space with 2 principal stresses, the minimum principal stress, $\sigma_3$, taken as 0 and the maximum principal stress, $\sigma_1$, taken as the unconfined compressive strength. The UCS result is consistent with the direct shear results. The rate of UCS increase corresponds to the rate of cohesion increase.
Figure 40: UCS Strength envelope of 4.2% CPB
Figure 41: UCS Strength envelope of 6.9% CPB
Figure 42: UCS Strength envelope of 9.7% CPB
The 4.2%, 6.9%, and 9.7% UCS show good fits with the direct shear results. UCS plots are tangent to the strength envelope with the confidence interval of 95%. The back-analysis of UCS strengths are determined through the Mohr-Coulomb parameter.

\[
\sigma_c = \frac{2c}{\tan \left(45 \frac{\varphi}{\pi} \right)}
\]

Where, \( \varphi \) is the angle of the frictional resistance, and \( c \) is cohesion. The back analyzed UCS strength is consistent with the experimental result. The trend is consistent in all of 4.2%, 6.9%, and 9.7% trials as shown in Table 5. The result shows a good fit with the direct shear results.
Table 5: UCS strength back-analysis

<table>
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<th>Sample ID</th>
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</thead>
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3.2.2 Stress-strain properties

The stress strain properties of 4.2%, 6.9% and 9.7% cement contents at 3, 7, 14, and 28 days curing time are presented in Figure 43-45. It can be seen regardless of cement content and curing time, stress-strain properties of UCS show 3 phases: an initial elastic behavior, a plateau region
with peak strength around 1% strain, and post-peak behavior from 1% onward. This phenomenon can be attributed to the propagation of cracks generated in the pre-peak and peak regions.

The result shows stress and strain graphs vary with time and cement content. The stiffness and strength of 4.2%, 6.9%, and 9.7% increase successively with curing time. The increase could be attributed to the development of cement bonds. The higher cement content specimens showed greater strength gain as observed in the Figures 43-45.

Figure 43: Stress-strain curve of 4.2% CPB
Figure 44: UCS stress-strain curve of 6.9% CPB
Figure 45: Stress-strain curve of 9.7% CPB

The shape of the UCS curve is controlled by cement content and curing time. At low cement content and early cure time, the stress and strain curves show more plastic behavior, the elastic behavior extends up to 30 to 40% of peak strength. The high cement content and curing time have more defined response, the elastic behavior extends to 60 to 70% of peak strength. This could be attributed to more developed cement bonds. Previous studies have also shown similar trends (Fall et al., 2007).
Figure 46 and Figure 47 show the modulus of elasticity increases with curing time and cement content. The modulus of elasticity with curing time of CPB is characterized by gradual increase from 3 to 7 days followed by steady increase from 14 day to 28 days as shown in Figure 46. The results are consistent in 4.2%, 6.9% and 9.7% CPB. The rate of increase can be characterized by the power relationship.

\[ E = a t^b \]

Where, E is the modulus of elasticity, a is a constant coefficient, t is curing time, and b is a power coefficient. The results are consistent in all of 4.2%, 6.9%, and 9.7% CPB. The coefficient of determination, the \( R^2 \) value, is greater than 0.95 in all of 4.2, 6.9 and 9.7% trials. The trend is more obvious in 9.7% than that of 4.2% and 6.9%. This could be attributed to the more developed cement bonds.

**Figure 46: Modulus of elasticity with curing time**
The modulus of elasticity with cement content is characterized by a linear relationship as shown in Figure 47. The trend is consistent in all 4.2%, 6.9%, and 9.7% at 3, 7, 14, and 28-day. The rate with cement content is characterized by:

\[ E = m c_s + c \]

Where, E is the modulus of elasticity, m is the rate of increase with cement content, \( c_s \) is the cement content, c is a constant. The coefficient of determination, \( R^2 \) value, ranged from 0.85 to 0.9.

**Figure 47: Modulus of elasticity with cement content**

The result shows the modulus of elasticity increased at the same rate in UCS strength, cohesion and shear modulus. The linear relationship with the cement content is attributed to the amount of cement hydration products.
3.3 Punching shear

The strength properties and stiffness of punching shear tests are analyzed with numerical simulation and stress-displacement graphs. Three configurations of the tests with 20mm, 40mm, and 50mm punching plates are conducted with 3 cement contents of 4.2%, 6.9% and 9.7% at 4 curing times of 3, 7, 14, and 28 days.

Figure 48 shows the failure surface of 20 mm punching shear. The failure surface develops from the loading plate to the base plate. The separation is characterized by a constant increase in the resistance with the displacement. The upper frustrum forms a steep angle which transfers to a milder angle at the base.

Figure 48: Punching shear with 20 mm load plate

Figure 49 shows the failure surface of 40mm punching shear. The failure surface develops from the base of the 40mm puncher and follows a steeper angle than the 20mm tests. The fracture is more defined than for the 20mm tests. The failure is characterized by a constant increase in resistance with displacement. The rate of resistance increase is consistent with the 20mm puncher. The conic shape is characterized by a linear and rough surface which is consistent with brittle failure.
Figure 49: Punching shear with 40 mm puncher

Figure 50 shows the failure surface of the 50 mm punching shear. The result is more definite and consistent than those of 20mm and 40mm tests. The rate of resistance increase with the punching is consistent with the 20mm and 40mm tests. It should be noted at lower cement content, the 20mm tests are overly fragile. The failure surface is characterized by a continuous failure surface which is consistent with brittle failure.

Figure 50: Punching shear with 50 mm puncher
3.3.1 Numerical simulation

A finite element model is used to analyze the resultant stress condition. The stress condition is modeled axisymmetrically with displacement-controlled boundary condition. Figures 51-53 show the simulation of 20 mm, 40 mm, and 50 mm punching test. The result shows stress localization forms along the preferential failure surface. The results of stress distribution follow the shape of the failure envelope. The analysis shows stress progression follows 3 phases: an initial stress concentration along the base of the plate and above the support; the propagation of the stress concentration along the failure surface, and; the coalesce of the stress concentration leading to nonconvergence.

Figure 51 shows the typical differential stress of 20mm simulation prior to nonconvergence. The analysis shows the differential stress initiates at the base of the loading plate. The increased displacement leads to a coalesce of the differential stress along the failure surface and the formation of tensile stress zone under the loading plate. The formation of a tensile zone and differential stress at upper edge of the sample show the development of bending moments which results in a nontrivial progressive failure mechanism.

Figure 51: Numerical simulation for 20 mm loading plate
Figure 52 shows the typical differential stress of 40mm simulation. The differential stress initiates at the base of the loading plate. The increased displacement results in stress localization along the failure plane and stress concentration above the failure plane. The development of the differential stress corresponds to the increase in the normal stress along the failure plane. The stress localizations are more definite than those of the 20mm plate tests.

Figure 52: Numerical simulation for 40 mm loading plate

Figure 53 shows the typical differential stress of 50mm simulation. The differential stress initiates at base of the loading plate. The increased displacement results in stress concentration along the failure plane and secondary stress zone above the failure surface. The secondary stress zone shows higher differential stress which could be attributed to the normal stress along the failure plane. The stress concentrations are more definite than those of the 20mm and 40mm plate tests.
Figure 53: Numerical simulation for 50mm loading plate

The resultant stress state is resolved into normal and shear components. The normal stress, $\sigma_n$, is taken as:

$$\sigma_n = \frac{P \cos \theta}{\pi (r_1 + r_2) \left[ (r_2 - r_1)^2 + h^2 \right]^{\frac{1}{2}}}$$

The shear component is taken as:

$$\tau = \frac{P \sin \theta}{\pi (r_1 + r_2) \left[ (r_2 - r_1)^2 + h^2 \right]^{\frac{1}{2}}}$$

Where, $P$ is the point load, $\theta$ is angle of the frustum, $r_1$ is the radius of loading plate, $r_2$ is the radius of the support ring, and $h$ is the height of the disk as shown in Figure 54.
The punching shear results are compared with the direct shear, UCS, and direct tensile properties. The normal and shear stress components are plotted in Mohr-Coulomb stress space. Figure 55 shows the typical punching shear stress state. All the stress states with 20mm tests are below the strength envelope. All the stress states with 40mm tests are consistent with the direct shear and tensile properties. All the stress states with 50mm tests are above the strength envelope. The trends are consistent with the numerical simulations. The simulation of 20mm shows the development of bending moment resulting in lower reading. The simulation of 40mm is consistent with observed failure. The simulation of 50 mm shows the development of secondary stress concentration which would attribute to higher normal stress resulting in higher load readings.
3.3.2 Stress displacement

The stress displacement properties of 4.2%, 6.9% and 9.7% cement contents at 3, 7, 14, and 28 days with 20mm, 40mm, and 50mm puncher are presented in Figures 54-56. The stress displacement curve can be classified into 3 phases, an initial elastic region, a plateau region, and post-peak behavior. In the elastic region, the 20mm, 40mm, and 50mm follow a similar rate of elastic resistance increase with displacement; the elastic region of 20mm ranges from 0-0.25mm with peak behavior to around 0.5mm; the elastic region of 40mm range from 0-.5mm with the peak behavior to around 0.75mm, and; the elastic region of 50mm range from 0-0.75mm with the peak behavior to around 1mm. Figures 56-58 show the stress-displacement curve of 4.2%, 6.9%, and 9.7% CPB. The stress displacement curve is dependent on curing time and cement content.
Figure 56: Stress and displacement of 4.2% CPB
Figure 57: Punching shear stress and strain curve of 6.9% CPB
Figure 58: Punching shear stress and strain curve of 9.7% CPB

The rate of increase is consistent with the trends observed in direct shear. The resistance is shown to increase at a steady rate between 3-7 days followed by gradual increase from 7 to 28 days. These results are consistent with the direct shear and the UCS tests. The peak strain, particularly with 50mm tests, is significantly higher than both other tests. The direct shear and UCS tests have peak strain around 1%, the punching shear test showed 10% strain prior to failure.

The analysis illustrated the challenges in working with CPB. In previous studies, Tolooiyan et al. (2014) showed that the frustum angle could range from 20 to 75 degrees with concrete materials.
The current study showed that the indirect test method is not suited for CPB, because the soft and brittle properties of CPB affect the variability of the tests. The tests with 20mm loading plate showed the development of bending moment. The tests with 50mm loading plate showed the development of secondary stress localization resulting in plastic flow both along and above the failure plane. The development of secondary stress concentration results in both higher normal stress and shear stress. The analysis shows the challenges in obtaining a nontrivial result.

### 3.4 Direct tensile

The direct tensile results are analyzed with the direct shear and UCS results and the stress and displacement curve. The tests were conducted with 2 cement contents of 6.9% and 9.7% and 3 curing time of 7, 14, and 28-day to analyze the effect of curing time and cement content. Lower cement contents and curing times produced specimens that were too weak to successfully remove from the molds and therefore no results are available for comparison.

#### 3.4.1 Strength properties

The results show the tensile strength is controlled by cement content and the curing time as shown in Figure 59 and Figure 60. The tensile strength increases successively with time. The trend is consistent with that of direct shear and UCS. The rate of increase can be characterized by the power law.

\[
\sigma_t = a \cdot t^b
\]

Where \( \sigma_t \) is the tensile strength, \( a \) is a constant coefficient, \( t \) is the time in days, and \( b \) is power coefficient. The trend is observed in all the 6.9% and 9.7% CPB with coefficient of determination, the \( R^2 \) value, greater than 0.9 in both cases. The increase is more evident in 9.7% than the 6.9% which can be attributed to the more developed cement bond. It should be noted that 6.9% was the lowest binder content for which tests could be completed. Lower binder content samples failed under their self-weight conditions during sample handling after debonding from the sample molds.
The strength increases with cement content is shown in Figure 60. The trend is consistent with the direct shear and UCS results.
The tensile properties are analyzed with direct shear and UCS results. Figure 61 and Figure 62 show the strength envelope of 6.9% and 9.7% CPB at 7, 14, and 28 days. The direct tensile results are plotted in Mohr-Coulomb space with 2 principal stresses, the maximum principle pressure, $\sigma_1$, is taken as 0 and the minimum principal stress, $\sigma_3$, is taken as the tensile strength. The tensile behavior exhibited similar trends as the direct shear result. The result shows the tensile strength increase with the cement content and curing time. The trend is consistent in 6.9% and 9.7% CPB, except for 9.7% at 28 day cure time which will be discussed later.
Figure 61: Tensile strength envelop of 6.9% CPB
Figure 62: Tensile strength envelop of 9.7% CPB
The results show Mohr-Coulomb behavior extends to uniaxial tension. The tensile stress states are tangent to the strength envelope at 95% confidence interval. The tensile strength can be estimated with the Mohr-Coulomb parameters.

\[
\sigma_t = \frac{2c}{\tan\left(45 + \frac{\phi}{2}\right)}
\]

Where, \(\sigma_t\) is the tensile strength, \(\phi\) is the angle of the frictional resistance, and \(c\) is cohesion.

The back-analysis from Mohr-Coulomb parameter shows a good fit with the experimental results as shown in Table 6. The average of 6.9% tests show difference of ±10%, and the average of 9.7 test show difference of ±5% with the experimental result as shown in Table 6. The 9.7% CPB has a better fit which could be attributed to more developed cement bonds.

**Table 6: Tensile strength back-analysis**

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<th>Sample ID</th>
<th>Tensile Strength [kPa]</th>
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3.4.2 Stress and displacement

The stress and displacement characteristic of 6.9% and 9.7% CPB are characterized by linear elastic behavior followed by abrupt failure which is consistent with tensile failure. Figure 63 and Figure 64 shows this trend in all 6.9% and 9.7% CPB tests.

Figure 63: Stress displacement graph of 6.9 CPB
The stress-displacement behavior depends on the cement content and the curing time. Figure 65 and Figure 66 show the stiffness increase with cement content and curing time. The observed stiffness is around 20 to 30% lower than the modulus of elasticity from UCS tests.

The results show tensile stiffness increases with the cement content. The trend is observed in all the 6.9% and 9.7% CPB trials as shown in Figure 66. The tensile stiffness increased successively with curing time with more gradual increase from 14 days onward. The trend is characterized by the power relationship.
The 7, 14, and 28-day tests of 4.2% and 6.9% CPB show the stiffness increases with cement content. The result is consistent with the shear modulus found in direct shear and the modulus of elasticity in UCS tests. The result showed the tensile stiffness is 20-30% less than compressive modulus in all the 6.9% and 9.7% tests.

Figure 65: Stiffness with curing time
Figure 66: Stiffness with cement content

Direct shear, UCS, and direct tensile results illustrated consistent behavior with the Mohr-Coulomb envelope. The UCS results are tangent to the Mohr-Coulomb envelope within a 95% confidence interval. The direct tensile results are consistent with the extended envelope. The observations are supported by the back-calculated strength which shows a good fit with the experimental result. The UCS is back-calculated as:

$$\sigma_c = \frac{2c}{\tan \left(45 - \frac{\varphi}{2}\right)}$$

Where, $\varphi$ is the angle of frictional resistance, and $c$ is cohesion. The back analyzed UCS strength is consistent with all the tests with 4.2%, 6.9%, and 9.7% CPB at 3, 7, 14, and 28 days. The tests show the estimated values are within +/- 5% for the majority of results.
The tensile strength is back analyzed as:

\[ \sigma_t = \frac{2c}{\tan\left(45 + \frac{\varphi}{2}\right)} \]

Where, \( \sigma_t \) is the tensile strength, \( \varphi \) is the angle of the frictional resistance, and \( c \) is cohesion. The estimates are consistent with the experimental results for 6.9% and 9.7% tests conducted at 7, 14, and 28 days. The results show a larger spread than the UCS tests with the percent difference ranging from 5 to 10% of the experimental results.

The direct shear and UCS results showed that frictional resistance and cohesion are mobilized together rather than independent as previously assumed (Mitchel et al. 1982). Historically, the strength behavior has been assumed to follow a constant strength envelope. The cohesion is characterized by UCS under the assumption of:

\[ c = \frac{\sigma_c}{2} \]

Where, \( c \) is cohesion and \( \sigma_c \) is the unconfined compressive strength. The result of this study suggests the assumption is invalid with a non-zero frictional angle. The cohesion is analyzed with the angle of frictional analysis and unconfined compressive strength. The analysis normalized cohesion with the unconfined compressive strength. This is shown as:

\[ \frac{\sigma_c}{2c} = \frac{1}{\tan(45 - \varphi/2)} \]

Where, \( \sigma_c \) is the unconfined compressive strength, \( c \) is cohesion, and \( \varphi \) is the angle of the frictional resistance. The sensitivity analysis shows while the assumption is valid when the angle of frictional resistance equals zero, the analyses with typical frictional resistance ranging from 35 to 38 degrees gives the cohesion significantly less than the half of the UCS. The assumption would overestimate the cohesion by a factor 1.4 to 1.5 which would potentially result in underestimating the required strength for the backfill material.
The tensile test illustrated the tensile strengths are higher than previously anticipated. Historically, the tensile strength is taken as 1/10 to 1/12 of the UCS strength of the sample. The result shows the tensile strength follows the Mohr-Coulomb envelope which could be calculated with Mohr-Coulomb parameters. The tensile strength is analyzed with the unconfined compressive strength and angle of friction resistance. The analyses normalized the tensile strength with compressive strength. This is shown as:

\[
\frac{\sigma_c}{\sigma_t} = \frac{1 + \sin \varphi}{1 - \sin \varphi}
\]

Where, \(\sigma_c\) is the UCS strength, \(\sigma_t\) is the tensile strength, and \(\varphi\) is the angle of frictional resistance. The ratio of UCS strength to tensile strength is shown in Figure 68. For frictional angles ranging from 30 to 37 degrees the compressive strength to tensile strength ratio is 3 to 4.5. This is significantly lower than the ratio of about 10 typically assumed. Put another way, the
usual assumption would under-estimate the actual tensile strength by a factor of approximately 2-3 (i.e., about 10/4.5 to 10/3.0).

![Graph showing the relationship between UCS and tensile strength variation with angle of frictional resistance](image)

**Figure 68: UCS and tensile strength variation with angle of frictional resistance**

The direct shear, UCS, and direct tensile tests are effective means to quantify the strength behavior of the cement pastefill. The direct shear and UCS results are consistent which suggest the historical assumption that cohesion equals ½ the unconfined compressive strength is invalid. The direct tensile strength shows the Mohr-Coulomb envelope extends to unconfined tension which illustrates that tensile strength is higher than 1/10 to 1/12 the unconfined compressive strength as previously assumed. The study shows the testing regime based on UCS alone is inadequate in characterizing the strength behavior of CPB. The results show the strength envelope should be analyzed with at least a few direct shear tests, combined with UCS and direct tensile tests which account for friction as well as cohesion.
The result showed the strength behavior of the CPB is controlled by cement content and curing time. The trend is consistent in all the direct shear, unconfined compression strength (UCS), and the direct tensile test of 4.2%, 6.9%, and 9.7% cement content. The stiffness and the strength increase at a higher rate between 3 to 7 days followed by a more gradual rate from 14 to 28 days which are modeled well using power relationships. The trend is observed in experimental results with strength parameters such as cohesion, compressive strength, and tensile strength, as well as the stiffness characteristics such as shear modulus, modulus of elasticity, and tensile stiffness.

The 3, 7, 14, and 28-day tests of 4.2%, 6.9%, and 9.7% CPB show strengths vary linearly with cement content which could be generalized using a linear relationship. The trend is consistent in all the direct shear, UCS, and punching shear results. The linear trend is observed in strength characteristics such as cohesion, unconfined compressive strength, tensile strength and stiffness characteristics such as shear modulus of elasticity, and tensile stiffness.
Chapter 4
Conclusion

4 Conclusion

This study has analyzed the mechanical properties of CPB under low confining stress and quantified the strength variation with cement content and curing time of 4.2%, 6.9%, and 9.7% CPB at 3, 7, 14, and 28 days, and illustrated the challenges working with CPB.

The three-dimensional plastic printer was highly effective in rapid prototyping plastic molds for sample preparation. All samples were prepared by pouring fresh CPB into plastic molds, ensuring no air voids were entrained, and sealing and curing the samples underwater. This effectively prevented oxidation and any subsequent strength degradation. All samples were tested at rates established by previous research to be slow enough to prevent excess pore water pressure generation, but also all samples were tested underwater so that any dilation potential would not result in suction development which could give artificially high apparent strength.

Although the punching shear test has the advantages of easy sample preparation and easy test preparation and execution, interpreting the punching shear results for the CPB materials requires nonlinear numerical analysis and the resulting stress fields are non-uniform, and the failure mechanisms progressive, which makes a unique interpretation of the stresses at failure (and hence determination of material strength) virtually impossible. Operations may find some variation of the punching shear test useful as a quality control device, but it is not appropriate from a strict strength determination perspective.

For CPB material with UCS less than about 300 kPa, the dog bone tensile specimens were too weak to handle and instead broke under their own self-weight during test preparation. Future work needs to consider alternate test methods and/or test preparation methods using the existing device in order to determine tensile strengths for relatively weak CPBs.

The CPBs tested in this work had UCSs in the range about 100 kPa (4.2% binder, 3 day cure) to almost 1 MPa (9.7% binder, 28 day cure) and in all cases excellent agreement was obtained between the UCS and direct shear test results. The direct shear tests were conducted in the
normal stress range from essentially zero up to almost the UCS value, and excellent linearity was obtained. The UCS Mohr Circles were tangent to the corresponding linear failure envelopes. The results are in stark contrast to the material test results shown in Mitchell et al. (1982), which suggested the shear strength was constant and equal to \( \frac{1}{2} \) UCS for normal stresses less than \( \frac{1}{2} \) UCS. Future work needs to critically assess the significance of these strength differences, because the actual strength of these CPBs will be less than that assumed in the design of exposed backfill sidewalls.

The extent of the linear Mohr-Coulomb failure envelope into the tensile range was evaluated using tensile test data for samples with UCS generally 300 kPa or greater, and specifically for 6.9 \% and 9.7 \% binder contents and 7, 14, and 28 day cure times. As for the UCS Mohr Circles, the tensile Mohr Circles were tangent to the extrapolated Mohr Coulomb envelope in the tensile range. It should be noted, however, that the failure surface in the tensile test is normal to the tensile loading direction, suggesting that the actual failure envelope should be Mohr-Coulomb with a tensile cutoff. Fortunately, such constitutive models are widely available in commercial software for geotechnical engineering.

Finally, it is important to note that all the testing was done for a specific mine tailings and binder type. The generality of the results obtained in this work should not automatically be assumed for other tailings and binder combinations. However, this work provides an excellent starting point for others to test their materials and establish the appropriate failure envelope for their CPB designs under low confining stress conditions.
References


Appendices
Appendix A: Loading Apparatus
## Loading Apparatus Drawing List

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Appendix B: Sample Apparatus
### Sample Apparatus Drawing List

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SHEET: 02/02
SCALE: NTS

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CHECK: 2/28/2019 M. GRABINSKY
APPR.: 2/28/2019 M. GRABINSKY
ISSUED: 2/28/2019 A. PAN
REV: REV 0

SA-DSSM-TD-02
ORTHOGONAL VIEW

ASSEMBLY

X2

COMPONENTS

FILE NAME: SAMPLE APPARATUS

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CHECK: 2/28/2019 M. GRABINSKY
APPR: 2/28/2019 M. GRABINSKY
ISSUED: 2/28/2019 A. PAN

REV: 0

REVISION NO.

SIGNATURE

CHECKED

SA-DTSM-0D-01

DIRECT TENSILE SAMPLE MOLD
ORTHOGRAPHIC DRAWINGS
ORTHOGRAPHIC VIEW
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