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Direct shear tests on cemented paste backfill–rock wall and cemented paste backfill–backfill interfaces

Nabassé J.F. Koupouli [a], Tikou Belem [a,*], Patrice Rivard [b], Hervé Effenguet [c]

[a] Research Institute in Mining and Environment (RIME), Université du Québec en Abitibi-Témiscamingue, Rouyn-Noranda, Québec, Canada
[b] Department of Civil Engineering, Université de Sherbrooke, Sherbrooke, Québec, Canada
[c] Department of Geology, Université Marien NGouabi, Brazzaville, People's Republic of Congo

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ABSTRACT

This paper presents the results of the shear strength (frictional strength) of cemented paste backfill-cemented paste backfill (CPB-CPB) and cemented paste backfill–rock wall (CPB-rock) interfaces. The frictional behaviors of these interfaces were assessed for the short-term curing times (3 d and 7 d) using a direct shear apparatus RDS-200 from GCTS (Geotechnical Consulting & Testing Systems). The shear (friction) tests were performed at three different constant normal stress levels on flat and smooth interfaces. These tests aimed at understanding the mobilized shear strength at the CPB-rock and CPB-CPB interfaces during and/or after open stope filling (no exposed face). The applied normal stress levels were varied in a range corresponding to the usually measured in-situ horizontal pressures (longitudinal or transverse) developed within paste-filled stopes (uniaxial compressive strength, $\sigma_c \leq 150$ kPa). Results show that the mobilized shear strength is higher at the CPB-CPB interface than that at the CPB-rock interface. Also, the perfect elastoplastic behaviors observed for the CPB-rock interfaces were not observed for the CPB-CPB interfaces with low cement content which exhibits a strain-hardening behavior. These results are useful to estimate or validate numerical model for pressures determination in cemented backfill stope at short term. The tests were performed on real backfill and granite. The results may help understanding the mechanical behavior of the cemented paste backfill in general and, in particular, analyzing the shear strength at backfill–backfill and backfill-rock interfaces.

1. Introduction

Cemented paste backfill (CPB) technology is increasingly and widely used in many underground mines throughout the world and has become very popular over the last decade (Potvin et al., 2005; Belem and Benzaazoua, 2008). CPB is obtained from mixing tailings with water and a binding agent called hydraulic binder. This technology was initially implemented in Canadian mines in the early 1990s (e.g. Landriault and Tenbergen, 1995; Nantel, 1998; Landriault et al., 2007). This popularity is primarily observed due to the numerous environmental directives implemented in many developed and developing countries. This implies the reuse of at least 50% of the tailings as CPB for secondary ground support in underground mine stopes (Mitchell, 1989a; Belem et al., 2000). Thus, CPB provides stable working platform for miners and reduces the amount of open space that could potentially be filled with a collapse of the surrounding pillars (Barret et al., 1978). In order to retain the CPB during the open stope filling, the constructed barricades are designed to prevent any failure induced by high pressures generated by the saturated fill mass (excess pore water pressure). In most cases, the sequence of filling an open stope is to first pour a plug fill of a few meters high (up to 7 m), followed by pouring the residual fill (Fig. 1). The binder content in the plug fill is larger than 5 wt% (on average 7 wt% of Portland cement or a blended binder; wt% is the weight percentage), while the binder content in the residual fill is not more than 5 wt% (on average in the range 2–5 wt% of a blended binder). The plug fill is usually left between 2 d and 5 d of curing prior to the residual filling in order to avoid excess pressure on the barricade. However, the proper design of a barricade requires a good estimate of barricade loads which in turn depend on the pressure/stress distribution within the backfilled stope (Belem et al., 2013).

* Corresponding author. Tel.: +1 8197620971x2359.
E-mail address: Tikou.Belem@uqat.ca (T. Belem).
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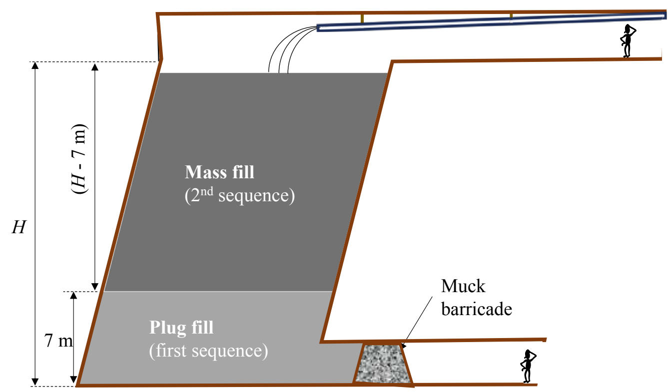

Fig. 1. Schematic of a typical underground stope filling sequences.

In many cases, the adjacent rock sidewalls actually help supporting the fill through boundary shearing and arching. Therefore, CPB and rock sidewalls may be mutually supported (Mitchell, 1989b). When arching occurs in a filled stope, the vertical pressure at the bottom of the fill is less than the overburden weight due to the horizontal transfer of pressure, somewhat like a trap door (Marston, 1930; Terzaghi, 1943). This pressure transfer is primarily associated with the frictional and/or cohesive interaction between CPB and rock sidewall (Belem and Benzaazoua, 2008). It should also be noticed that some "chemical" consolidation can occur within the fill mass due to the chemical shrinkage also designated as self-desiccation (Helinski et al., 2007). In fact, the binder hydration leads to the dissipation of pore water pressure which will increase the vertical effective stress causing consolidation. In-situ measurements conducted by Bridges (2003) show that pore water pressure on barricades is negligible after a few days. If the CPB permeability is very low and water does not drain out under gravity, no settlement of the CPB occurs. Without settlement, no shear stress is mobilized at the CPB-rock sidewall interface and no arching occurs. However, if the CPB is draining freely, the fill begins to settle virtually as soon as it is placed and the distortion associated with this settlement generates the mobilization of shear stresses at the fill-rock sidewall interface. The shear strength that can be mobilized at the interface will depend on the level of friction. This friction in turn is a function of the horizontal effective stress acting on the interface (Fourie et al., 2007). The determination of shear stress development allows understanding how arching effect can occur (and thus stress relief on barricades). Then this effect can be taken into account during preliminary backfill design process (de Souza et al., 2009). It is therefore necessary to quantify experimentally the shear strength parameters (interface cohesion or adhesion, interface friction angle) and the shear stiffness of CPB.

To the authors' knowledge, very few experimental studies have been conducted on CPB-rock sidewall interface behavior. A study on the shear behavior of artificial paste backfill-limestone smooth interface was carried out by Nasir and Fall (2008), followed by another one on artificial paste backfill-concrete and brick interfaces (Fall and Nasir, 2010). The normal stress ranging from 100 kPa to 200 kPa and four different curing times (i.e. 1 d, 3 d, 7 d, and 28 d) were tested with a single cement content of 4.5% (by dry mass of ground silica). The main observation was that, for the same stress conditions, the shear strength of the artificial CPB materials is greater than that of the artificial CPB-rock/concrete/brick interfaces. Their results also showed that the angle of friction of artificial CPB-rock/concrete/brick interfaces was greater than 2/3 of the angle of internal friction of artificial CPB. However, the results presented by these authors were obtained from tests conducted on purely artificial cemented backfill prepared with ground silica, namely SIL-CO-SIL 106, which is different from true tailings. A third study on the investigation of backfill–rock mass (simulated by concrete) interface failure mechanisms was conducted by Manaras (2009) who highlighted the importance of the binder content, the curing time and the rock (concrete) sidewall roughness quantified by the JRC (joint roughness coefficient) values ranging from 3 to 19. The normal stress ranging from 35 kPa to 1500 kPa and three different curing times (14 d, 28 d, and 56 d) were tested. The CPB samples were prepared at 80% of solid content with three different binder contents (2.5%, 5% and 8% by dry mass of tailings).

Although the results of these previous studies contribute to the understanding of interfaces phenomena, the fact remains that it would be more interesting to have results on the behaviors of interfaces between real CPBs and real rocks. But to the authors' knowledge, such a study has not been conducted to date on the interfaces between a real CPB and a real rock. Hence, the main objective of this paper is to conduct a laboratory investigation of the shear stress-shear displacement behavior and the determination of shear strength parameters of early age CPB-granite sidewall and early-age CPB-CPB interfaces using a direct shear machine. The curing times tested are 3 d and 7 d. The results of these tests will allow estimating the shear strength that develops in the short term (between 1 d and 7 d of curing times), during which the authors

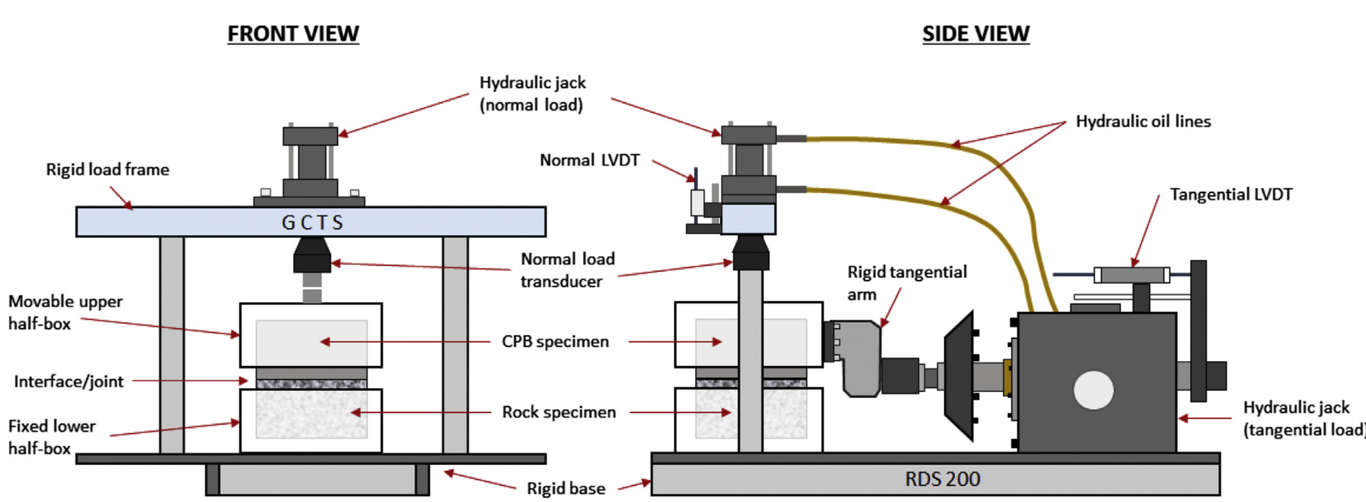

Fig. 2. Direct shear test machine RDS-200 from GCTS (Geotechnical Consulting & Testing Systems).

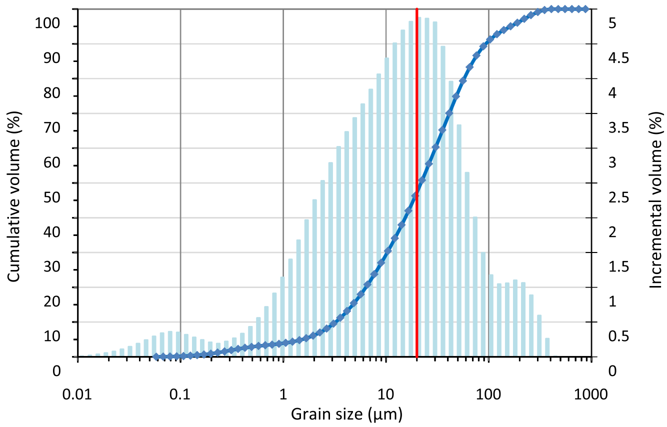

Fig. 3. Histogram and cumulative grain size distribution curve of Perseverance mine tailings.

believe that the arching effects may develop in backfilled stopes. The results will also serve to better estimate the pressures in the backfilled stopes using analytical models, taking into account the arching effect (e.g. Aubertin et al., 2003; Li et al., 2005). The results may help understanding the mechanical behavior of early-age backfill in general and, in particular, analyzing the shear strength at the interfaces of real backfill–backfill and real backfill-rock. Finally, these results could be used as input parameters in numerical modeling of the pressure distribution in backfilled stope at short term using numerical codes (e.g. FLAC^{2D}, FLAC^{3D}, UDEC, and PLAXIS).

2. Materials

2.1. Direct shear apparatus

The direct shear tests were performed on cemented paste backfill-granite (CPB-rock) and cemented paste backfill-cemented paste backfill (CPB-CPB) interfaces in order to assess the frictional shear strength parameters. The direct shear apparatus (see Fig. 2) can generate pressure using a hydraulic pump. This pressure allows sample shearing through the relative displacement of two halves shear boxes (one fixed and one movable). The fixed lower half-box contains the granite specimen and the upper movable half-box contains the CPB specimen (Fig. 3). The shear box is connected to a digital control system composed of a computer and software and a data acquisition system.

2.2. Tailings sample, binder, mixing water and rock

(1) Tailings sample

The CPB specimens will be prepared with filtered tailings cake (87% solid concentration) sampled from PVE mine (now closed) in Matagami, Quebec, Canada (Xstrata Zinc). The semi-quantitative mineralogical analysis by X-ray diffraction (XRD) of the tailings sample shows predominance of pyrite (31.96 wt%), magnetite (20.64 wt%), pyrrhotite (15.51 wt%), talc (11.73 wt%), quartz (9.43 wt%) and chlorite (9.28 wt%). The average specific gravity G_s of the Perseverance mine tailings sample was 3.75. Fig. 3 presents the histogram and grain size distribution curve of the tailings sample and Table 1 provides their physical parameters. PVE mine tailings can be considered as medium size tailings: the fines (<20 μm grain size) content is between 35% and 60%, according to Landriault (1992) classification system. Also, these tailings can be considered as a fine-grained soil.

Table 1
Physical parameters of Perseverance mine tailings.

Parameter	Unit	Value
Specific gravity G_s (or *SG*)		3.75
D_{10} (grain diameter at 10% passing)	μm	3.23
D_{30} (grain diameter at 30% passing)	μm	10.3
D_{50} (grain diameter at 50% passing)	μm	21.9
D_{60} (grain diameter at 60% passing)	μm	30.2
D_{90} (grain diameter at 90% passing)	μm	94.2
$C_u = D_{60}/D_{10}$ (coefficient of uniformity)		9.36
$C_c = D^2_{30}/(D_{10} \times D_{60})$ (coefficient of curvature)		1.08
$\Delta = (D_{90} - D_{10})/D_{50}$ (relative span factor)		4.15
Fines content (<20 μm)	%	46.3

(2) Binding agent

The binder used is a generally used Portland cement (GU type) provided by Lafarge North America. Three different cement contents $B_{w\%}$ were tested: 2.3%, 5.3% and 8.2% (by dry mass of tailings). The cement content $B_{w\%}$ is written as

$$B_{w\%} = 100M_{\text{binder}}\Big/M_{\text{dry-tailings}} \tag{1}$$

where M_{binder} is the mass of binder and $M_{\text{dry-tailings}}$ is the mass of dry tailings.

(3) Mixing water

The water used for the mixture is potable municipal water (tap water).

(4) Rock sample

The rock sample is a medium-grained granite taken from the Abitibi area in Quebec (Canada) and precut using a circular saw (120 mm × 120 mm × 60 mm). The granite surface to be in contact with the CPB is considered flat and smooth.

2.3. Cemented paste backfill specimens preparation

The required mass of tailings, water and binder (GU type Portland cement) were mixed and kneaded using a Hobart mixer for about 10 min. Table 2 summarizes the CPB mixture characteristics such as the solid mass concentration $C_{w\%}$ and corresponding slump height, gravimetric water content w%, and also the average unconfined compressive strength (UCS) for each mix. The solid mass concentration $C_{w\%}$ and the gravimetric water content w(%) can be expressed as

$$\left.\begin{aligned} C_{w\%} &= 100M_{\text{solid}}/M_{\text{total}} \\ w(\%) &= M_{\text{w}}/M_{\text{solid}} \end{aligned}\right\} \tag{2}$$

where M_{w} is the mass of water, M_{solid} is the mass of dry solids (tailings + binder), and M_{total} is the total mass of backfill (water + tailings + binder).

Table 2
CPB mixtures' characteristics.

Backfill recipe	Binder content, $B_{w\%}$ (%)	Solid mass concentration, $C_{w\%}$ (%)	Water content, w (%)	Slump height, S (mm)	UCS (kPa)	
					3 d	7 d
Mix 1	2.3	70	42.9	18	98	137
Mix 2	8.2	70	42.9	18	350	490

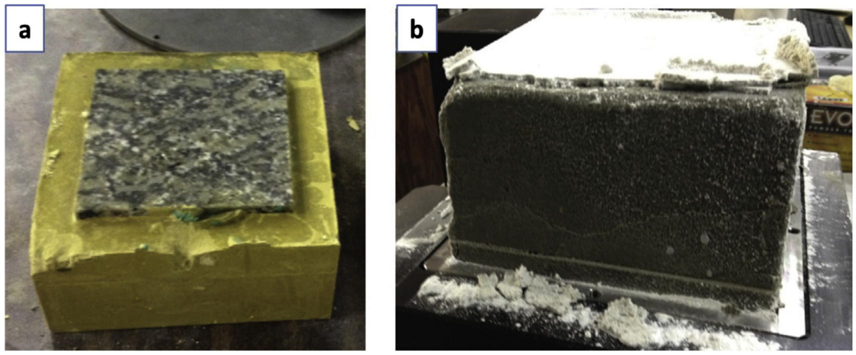

Fig. 4. Direct shear test samples: (a) Granite wall, and (b) CPB sample.

Once cast in 120 mm × 120 mm square and 65 mm height PVC molds, the CPB samples were cured in a humid chamber at room temperature (23 °C) and relative humidity >90%. The granite rock specimens were precut to the same size as the CPB specimens. The surface of the samples was characterized by the surface roughness coefficient R_s (EL-Soudani, 1978). It appears that R_s (= actual surface area/projected surface area) is close to 1 for all surfaces. This allows us considering that all surfaces are as smooth.

Because the objective of the study is to understand the early age behavior of the CPB, two curing times were considered: 3 d and 7 d. These curing times correspond to the filling time of a typical stope of 30 m high at an average constant filling rate of about 0.18 m/h. Fig. 4 shows two granite rock footwalls with smooth surface (Fig. 4a) and two CPB specimens with smooth surface (Fig. 4b) for the direct shear test of interfaces.

3. Methods

3.1. Experimental approach

Fig. 5 illustrates schematically the experimental approach of the direct shear tests which should mimic the in-situ conditions of a backfilled stope. This figure shows that the direct shear tests represent a small portion of about 120 mm × 120 mm section of a backfilled stope interface. However, for a typical underground mine stope of 30 m in height and 25 m in length (cross-sectional area of 750 m^2), the shear box section represents about 0.002% of the interface. Moreover, it should be noted that the normal stress (σ_n) to be applied during shearing should correspond to the horizontal pressure σ_h (longitudinal or transverse) developed within the stope during its filling.

3.2. Experimental program

As mentioned previously, two cement contents were used (2.3 wt% and 8.2 wt%) for CPB preparation, and the specimens were cured for 3 d and 7 d. For each curing time, both types of interfaces were tested (CPB-CPB and CPB-rock interfaces). CPB-CPB was considered here as a particular case of interface. For each interface, three normal stress levels were applied (50 kPa, 100 kPa, and 150 kPa) and a single CPB specimen was used for each stress level (3 different specimens in total). These normal stress levels were chosen based on in-situ measurement of pressures in backfilled instrumented stopes at DYN mine. Indeed, these measurements showed that the maximum horizontal (longitudinal) pressure in the backfill mass did not exceed 200 kPa (Belem et al., 2004). For each testing start, a normal stress level is applied on the shear box and the interface is sheared at a constant shear rate of 0.5 mm/min for a maximum tangential displacement of 4 mm.

4. Results and discussion

The shear stress–shear displacement curves obtained from the test specimens are given for Mix 1 (Fig. 6) and Mix 2 (Fig. 7). These curves are characterized by two different behaviors: the nearly perfect plateau elastoplastic curves (Figs. 6a, b and 7a, b) and the elastoplastic curves with strain-hardening (Fig. 6c, d). At the CPB-rock interface, when the CPB is blended at 2.3% of binder content, the shear stress continues to increase slightly (around 2%) even after the elastic stage (Fig. 6a, b). These curves have an elastoplastic nearly perfect plateau behavior. When the binder content is 8.2%, the shear stress does not vary after elastic stage (Fig. 7a, b) and these curves have an elastoplastic with nearly perfect plateau behavior. The effect of binder content is even more significant with CPB-CPB interface. Indeed, Fig. 6c, d shows that when the binder proportion is 2.3%, the stress–displacement curves exhibit an elastoplastic behavior with strain-hardening. When the curing times went from 3 d to 7 d, the shear strength was increased by 8%, 9%, and 10%, which corresponded to the applied normal stresses of 50 kPa, 100 kPa, and 150 kPa, respectively. The UCS was increased by 39% for the same 2.3% binder content when the curing times went from 3 d to 7 d. However, when the binder proportion was increased up to 8.2%, the stress–displacement curves exhibited a nearly perfect plateau elastoplastic behavior, regardless of the curing time (Fig. 7c, d). When the applied normal stresses were 50 kPa, 100 kPa, and 150 kPa and the curing time increased from 3 d to 7 d, the shear strength was increased by 12%, 21%, and 13%, respectively. The UCS was increased by 40% in the same conditions.

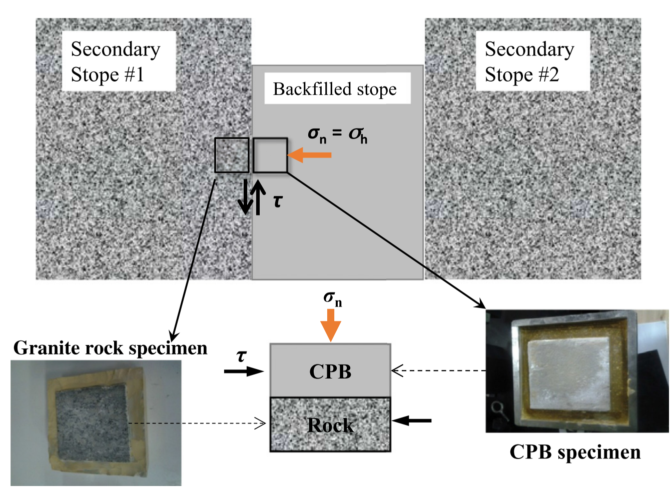

Fig. 5. Schematic of a backfilled mine stope illustrating the experimental approach.

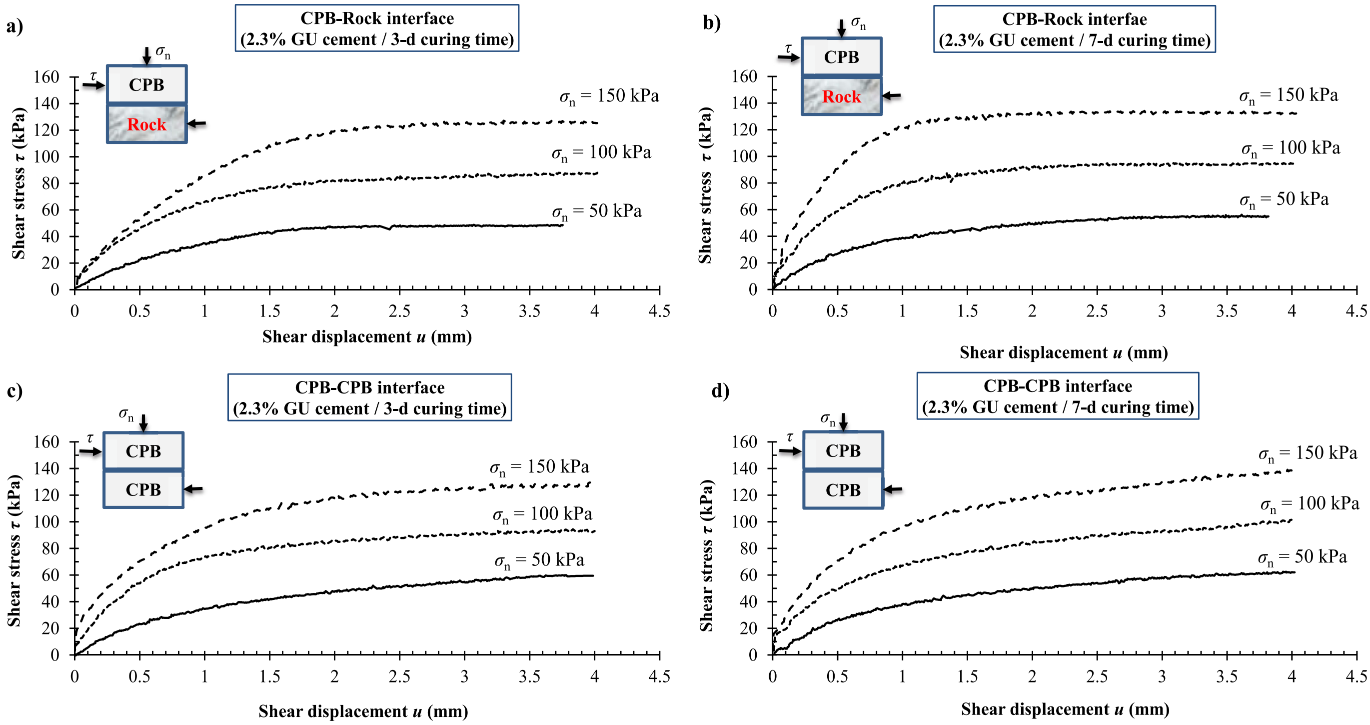

Fig. 6. Shear stress-shear displacement curves for CPB specimens containing 2.3% GU cement: (a) CPB-rock interface with CPB cured at 3 d, (b) CPB-rock interface with CPB cured at 7 d, (c) CPB-CPB interface with CPB cured at 3 d, and (d) CPB-CPB interface with CPB cured at 7 d.

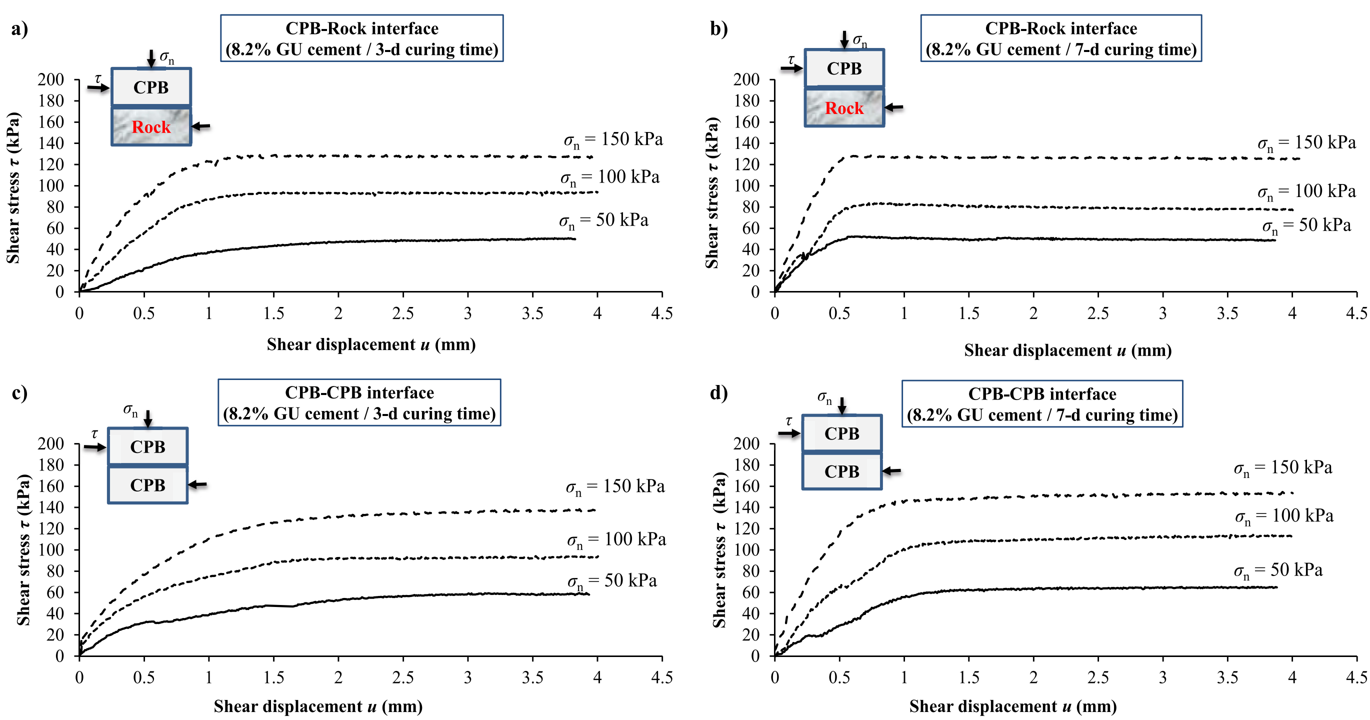

Fig. 7. Shear stress–shear displacement curves with CPB specimens containing 8.2% GU cement: (a) CPB-rock interface with CPB cured at 3 d, (b) CPB-rock interface with CPB cured at 7 d, (c) CPB-CPB interface with CPB cured at 3 d, and (d) CPB-CPB interface with CPB cured at 7 d.

These behaviors suggest that when the CPB surfaces (the same binder content and the same curing time) are in contact, an apparent cohesion or adhesion (amplified by capillary effect) is generated at the interface. This apparent cohesion is promoted by the low hardness (or stiffness) of CPB compared with the granite. Tables 3 and 4 summarize the shear parameters of CPB-rock and CPB-CPB interfaces obtained from these tests. The shear parameters are the peak shear stress (or shear strength) τ_p and the tangential (or shear) stiffness K_s. Table 3 also shows that the interface stiffness K_s slightly increases with curing time, except for CPB-rock interface. This can be explained by the increase of UCS as described above and the plastic stage with strain-hardening at the CPB-CPB interface curves. In fact, due to the presence of water and binder at each CPB surface, some adhesion is created at the CPB-CPB interface, unlike

Table 3
Interface shear parameters with CPB containing 2.3% GU cement.

σ_n (kPa)	CPB-CPB interface				CPB-rock interface			
	3 d curing		7 d curing		3 d curing		7 d curing	
	τ_p (kPa)	K_s (kPa/mm)	τ_p (kPa)	K_s (kPa/mm)	τ_p (kPa)	K_s (kPa/mm)	τ_p (kPa)	K_s (kPa/mm)
50	59	40	64	52	48	40	55	45
100	95	85	103	68	88	51	95	90
150	127	100	140	91	125	113	132	137

Table 4
Interface shear parameters with CPB containing 8.2% GU cement.

σ_n (kPa)	CPB-CPB interface				CPB-rock interface			
	3 d curing		7 d curing		3 d curing		7 d curing	
	τ_p (kPa)	K_s (kPa/mm)	τ_p (kPa)	K_s (kPa/mm)	τ_p (kPa)	K_s (kPa/mm)	τ_p (kPa)	K_s (kPa/mm)
50	58	60	65	56	50	43	48	93
100	94	87	114	113	93	103	86	143
150	137	120	154	187	127	176	125	260

the CPB-rock interface where the interaction is limited by the rock surface. The results show that the variation of K_s is strongly dependent on the normal stress level applied during direct shear testing. At high normal stress ($\sigma_n \geq 100$ kPa), the stiffness of the CPB-rock interface is higher than that of CPB-CPB interface. This can partly explain the fact that the shear stresses are higher for CPB-CPB interfaces compared with CPB-rock interfaces.

Fig. 8 shows the failure envelopes of CPB-rock and CPB-CPB interfaces with backfill incorporating 2.3% and 8.2% GU cements after curing times of 3 d and 7 d, respectively. As can be observed, the failure envelopes of all smooth interfaces are well described by the linear Mohr-Coulomb criterion, which is given by

$$\tau_f = c_a + \sigma_n \tan\varphi_j \tag{3}$$

where τ_f is the shear strength (kPa).

Fig. 8 suggests the existence of a non-zero apparent cohesion (adhesion) for all tested interfaces, lying between 8 kPa and 15 kPa. Table 5 summarizes the apparent cohesion (c_a) and interface friction angle (φ_j) values. It can be noted that both the apparent cohesion and the interface friction angle are close for all tested interfaces, regardless of the cement contents (2.3% and 8.2%) and the curing times (3 d and 7 d). The general trend is that the apparent cohesion varies in the range of 10–15 kPa for the CPB-CPB interfaces, while 8–9 kPa for the CPB-rock interfaces. Indeed, the interface friction angle varies in the range of 37°–44° for the CPB-CPB interfaces, and 38°–40° for the CPB-rock interfaces for both cement contents and both curing times. The slight increase of the interface friction angle at the CPB-CPB interface may be explained by the high peak shear stress observed at CPB-CPB interface as shown in Tables 3 and 4. This means that the friction is higher at the CPB-CPB interface than that at the CPB-rock interface, leading to an increase in the interface friction angle.

Several mechanisms may be responsible for the frictional resistance at the interfaces during shearing. The presence of binder and water within the paste backfill leads perhaps to the development of induced capillary forces (only assumed but not measured) at the interface which may partly explain the strain-hardening behavior at the CPB-CPB interface. This can also be due to the interpenetration of the CPB surfaces under the effect of the normal stress applied. Also, the presence of presumable micro-asperities on the CPB surface may also explain this strain-hardening frictional behavior. Indeed, the CPB surface is not perfectly smooth and there may have some tailings grain that can create small grooves on the opposite surface, which in turn increases the frictional shear strength during shearing.

5. Concluding remarks

From the results of direct shear tests on CPB-rock and CPB-CPB smooth interfaces, the following concluding remarks can be drawn:

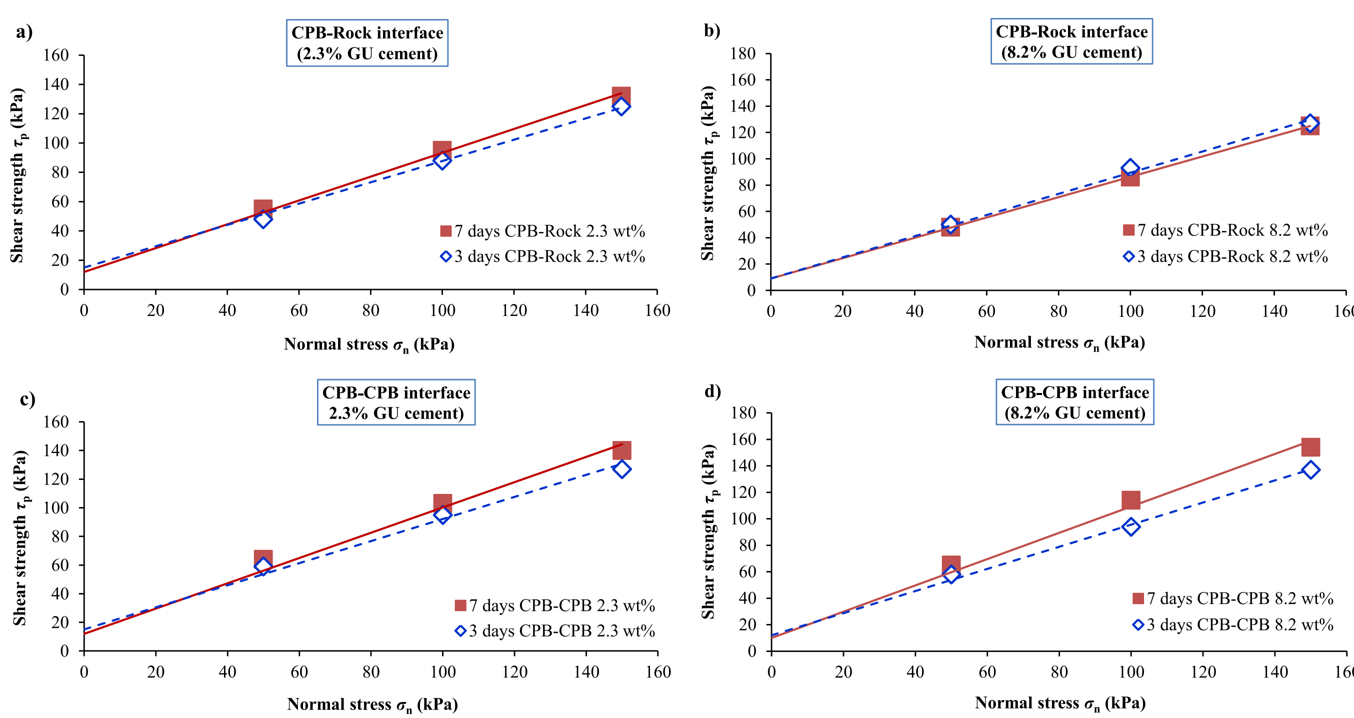

Fig. 8. Mohr-Coulomb failure envelopes for the interface with CPB cured at 3 d and 7 d: (a) CPB-rock interface with 2.3% GU cement, (b) CPB-rock interface with 8.2% GU cement, (c) CPB-CPB interface with 2.3% GU cement, and (d) CPB-CPB interface with 8.2% GU cement.

Table 5
CPB-rock and CPB-CPB interfaces apparent cohesion and interface friction angle.

$B_{w\%}$ (%)	CPB-CPB interface				CPB-rock interface			
	3 d curing		7 d curing		3 d curing		7 d curing	
	c_a (kPa)	φ_j (°)	c_a (kPa)	φ_j (°)	c_a (kPa)	φ_j (°)	c_a (kPa)	φ_j (°)
2.3	15	37	12	41	8	38	9	40
8.2	12	40	10	44	9	38	9	39

Note: c_a is the apparent cohesion or adhesion (kPa), and φ_j is the interface angle friction (°).

(1) The results show that the stress–displacement behavior of the CPB-rock smooth interfaces is much more dependent on the CPB binder content (2.3% and 8.2%) than the curing time (3 d and 7 d).
(2) The stress–displacement curves show a nearly perfect elastoplastic behavior of the CPB-rock interfaces, and elastoplastic with strain-hardening behavior at the CPB-CPB interface.
(3) The results show that more the cement content is high and more the stress–displacement behavior is nearly perfect elastoplastic for CPB-rock smooth interfaces.
(4) For the CPB-CPB interfaces, results show that with the lowest binder content (2.3% GU cement) the stress–displacement curves exhibit an elastoplastic behavior with strain-hardening, regardless of the curing times (3 d and 7 d). For the highest binder content (8.2% GU cement), the shear stress-shear displacement curves exhibit a perfect elastoplastic behavior, regardless of the curing time.
(5) The results show that the interface stiffness K_s slightly increases with the curing time, except for CPB-rock interfaces. In particular, the variation of K_s is strongly dependent on the normal stress level applied during direct shear testing.
(6) The results show the existence of an apparent cohesion (adhesion) for all tested interfaces. The apparent cohesion varies in the range of 10–15 kPa for the CPB-CPB interfaces, while 8–9 kPa for the CPB-rock interfaces.
(7) It is observed that the interface friction angles are very close for all tested interfaces, regardless of curing time and binder content. The interface friction angle varies in the range of 37°–44° for the CPB-CPB interfaces, while 38°–40° for the CPB-rock interfaces for the range of cement content (2.3% and 8.2%) and the range of curing times (3 d and 7 d) tested. The angle of internal friction of the CPB materials varies in the range of 22°–36°.
(8) The interface shear strength is slightly or not affected by the binder content that has been used, but is clearly dependent on the applied normal stress level, regardless of the type of interface and the curing time that have been tested.

This study gives some values of shear strength, cohesion, interface roughness and interface friction angle at the CPB-CPB and CPB-rock interfaces for short curing time. These values are useful for numerical modeling and to know paste backfill behavior at short time. However, the parameters at long term were not determined in this study. Therefore, an extrapolation is not recommended because the mechanical behavior of the CPB changes as a function of time due to the heterogeneity of mine tailings.

Conflict of interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

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Mr. Nabassé Jean-Frédéric Koupouli is a master student at the Research Institute on Mining and Environment (RIME) of the Université du Québec en Abitibi-Témiscamingue (UQAT) in Canada. His research topic is entitled "Compression and shear behavior of cemented paste backfill and study of the frictional behavior of cemented paste backfill/rock wall (CPB-rock) and backfill/backfill (CPB-CPB) interfaces". This research topic is part of a research project that aims at experimentally characterizing all the mechanical properties that play a role during the stability analysis process of backfilled stopes having one exposed face after a certain curing time (usually 28 d). **Contact information:** Research Institute on Mining and Environment (RIME), Université du Québec en Abitibi-Témiscamingue, 445, boul. de l'Université, Rouyn-Noranda, Quebec, J9X 5E4, Canada. Tel.: +1 819 762 0971x4307. E-mail: NabasseJeanFrederic.Koupouli@UQAT.ca

Dr. Tikou Belem is a full professor at the Université du Québec en Abitibi-Témiscamingue (UQAT) in Canada. He has over 17 years of experience in industry-related research in mine fill and environmental geomechanics. His research interests cover mine backfilling systems design, ground control in underground mines, environmental geomechanics, rheology of mine paste backfill, mine backfill mix formulation, hydromechanics of mine backfill, physical modeling of mine backfill, bulk backfill/rock mass interface behavior, in situ instrumentation of backfilled stopes and geomechanics of cemented paste aggregate fill. Dr. Belem also participated in many international conferences, congresses and seminars (more than seventy) covering all his fields of research, in addition to having authored and co-authored several journal papers and conference papers on Mine Backfill and Rock Mechanics. **Contact information:** Research Institute on Mining and Environment (RIME), Université du Québec en Abitibi-Témiscamingue, 445, boul. de l'Université, Rouyn-Noranda, Quebec, J9X 5E4, Canada. Tel.: +1 819 762 0971x2359. E-mail: Tikou.Belem@UQAT.ca

Dr. Patrice Rivard is working as a full professor at Université de Sherbrooke, Canada. His research interests cover applied rock mechanics in civil engineering, non-destructive testing methods and aggregate in concrete. His recent contributions to rock mechanics include the characterization of shear mechanism using acoustic emission (AE), the evaluation of concrete-rock interface for dam safety analysis and the characterization of rock joint roughness. **Contact information:** Department of Civil Engineering, Université de Sherbrooke, 2500 Université, Sherbrooke, Québec, J1K2R1, Canada. Tel.: +1 819 821 8000x63378. E-mail: Patrice.Rivard@USherbrooke.ca

Dr. Hervé Effenguet is working as a professor at the Department of Geology of the Marien NGOUABI University (Republic of Congo). His research interests cover unsaturated soil mechanics, laboratory testing, constitutive modeling, and environmental geotechnics. **Contact information:** Université Marien NGOUABI, Faculté des Sciences, Département de Géologie, Boîte Postale 69, B/ville, République du Congo. Tel.: +242 05 594 5764. E-mail: herve.effenguet@oges-congo.org