An overview on the use of paste backfill technology as a ground support method in cut-and-fill mines

T. Belem & M. Benzaazoua
Université du Québec en Abitibi-Témiscamingue, Dépt. des Sciences appliquées, Rouyn-Noranda, Canada

ABSTRACT: This is a general overview on the use of paste backfill for ground support in underground mining operations and as such, concerns backfill design parameters (internal pressure development, required strength and mix optimisation), its pumping qualities (consistency and rheological) and delivery to an underground operation through pipelines. Emphasis was placed on the optimization of the paste backfill mix for backfill design, work safety and expense to the mining operation. This is because of the 20% representative costs related to backfilling, 15% represents the binder costs.

1 INTRODUCTION

The use of cemented paste backfill (CPB) is an increasingly important component of underground mining operations and is becoming a standard practice for use in many cut-and-fill mines around the world (Landriault et al. 1997, Naylor et al. 1997). Backfill material is placed into previously mined stopes to provide a stable platform for the miners to work on and ground support for the walls of the adjacent adits as mining progresses by reducing the amount of open space which could potentially be fill by a collapse of the surrounding pillars (Barret et al. 1978). The use of underground paste backfill provides ground support to the pillars and walls, but also helps prevent caving and roof falls, and enhances pillar recovery, which enhances productivity (Coates 1981).

Thus, the CPB placement provides an extremely flexible system for coping with changes in geometry of the orebody, that result in changing stope width, dip, and length (Wayment 1978). The method of the fill delivery depends upon the amount of energy required to deliver the backfill material underground which depends on its distribution cone (Arioglu 1983). The CPB is usually transported underground through reticulated pipelines.

Paste backfill is composed of mill tailings generated during mineral processing which are mixed with additives such as Portland cement, lime, pulverized fly ash, and smelter slag. The purpose of the binding agents is to develop cohesion within CPB so that exposed fill faces will be self-supporting when adjacent stopes are extracted (Mitchell 1989). With the current low metal prices, the survival of many mines depends on their ability to maximize productivity while minimizing costs. At underground cut-and-fill operation, the costs associated with backfilling must be looked at critically so that potential cost savings can be identified (Stone 1993). Backfilling is expensive in some ways, but indispensable for most underground mines to provide ground support for mine safety and mining operations. Therefore, the fill should be cost effective and capable of achieving the desired ground support and stability.

Analysis of the fill stability must consider the geometric boundaries of the fill for the best economic use of CPB. Mine openings and exposed fill faces in large underground mines vary in shape from high and narrow to low and wide. Additionally, wall rock next to the backfill may be either steeply dipping or relatively flat-lying (Mitchell 1989). The stoping sequence can be modified to reduce the number of CPB-filled stopes, or the stope geometries could be revised to reduce the strength required of CPB exposures (Stone 1993).

This paper is an overview of the use of CPB for underground ground support in mining operations, from preparation to placement underground. The paper will first briefly introduce the notion of arching effects and their importance in stability analysis of filled stopes. This will be followed by presenting the design of the required fill strength from reviews of various current design methods. The paper will discuss the optimization of CPB-mix designs (as a means to reduce costs and improve fill strength) followed by a discussion on the rheological properties of CPB.
Finally, the paper will discuss CPB delivery systems and underground placement of CPB.

2 DESIGN OF THE HORIZONTAL PRESSURE ON THE FILLED STOPE SIDEWALLS

In general, self-support stresses govern backfill design and the traditional design has been that of a free standing wall, requiring a uniaxial compressive strength (UCS) equal to the overburden stress at the bottom of the filled stope. However, in many cases, the adjacent rock walls can actually help support the fill through boundary shear and arching effects. Therefore, the backfill and rock walls can be mutually supporting (Mitchell 1989). In backfilled stopes, when arching occurs (which is the case in many mines) the vertical pressure at the bottom of filled stope, an analogy similar to a trap door, is less than the weight of overlying fill (overburden weight) due to horizontal pressure transfer (Martson 1930, Terzaghi 1943). This pressure transfer is due to frictional and/or cohesive interaction between the fill and wall rock. When the pillars or stope walls begin to deform into the filled opening the fill mass will provide lateral passive resistance. Passive resistance is defined as the state of maximum resistance mobilized when force pushes against a fill mass and the mass exerts resistance to the force (Hunt 1986).

The magnitude of pressure transferred horizontally to the sidewalls can be included into the design for the required fill strength. Horizontal pressures affected by the fill arching can be evaluated by four analytical solutions which account for the existence of cohesion at the fill-sidewall interface and/or the frictional sliding along the sidewalls. These solutions are the Martson’s model and its modified version, the Terzaghi’s model and a proposed 3D model.

2.1 Martson’s cohesionless model

Martson (1930) developed a two-dimensional arch solution to predict the horizontal pressure $\sigma_h$ along the sidewalls of the pillars as follows:

$$\sigma_h = \frac{\gamma B}{2 \mu^*} \left[1 - \exp \left(-\frac{2K_m H}{B}\right)\right]$$

(1)

$$\sigma_v = \sigma_h / K_s$$

(2)

$$K_s = \tan^2 \left(45^\circ - \phi / 2\right)$$

(3)

where $\gamma = $ fill bulk unit weight (kN/m$^3$); $B =$ width of stope (m); $H =$ total height of filled stope (m); $\mu^* =$ coefficient of sliding friction between fill and sidewalls; $\phi =$ angle of wall friction (may be assumed between 1/3$\phi$ to 2/3$\phi$); $\phi =$ angle of internal friction of fill (degree); $\sigma_v =$ vertical pressure at the floor of the stope (kPa); $K_s =$ coefficient of active earth pressure (see Eq. 3).

2.2 Modified Martson’s cohesionless model

Aubertin et al. (2003) proposed a modified version of the Martson’s two-dimensional arch solution which was originally defined using active earth pressure ($K_a$) and wall sliding friction. The modified version for predicting the horizontal pressure ($\sigma_{h1}$), at a depth $H$, along the sidewalls of the pillars is given as follows:

$$\sigma_{h1} = \frac{\gamma B}{2 \tan \phi'} \left[1 - \exp \left(-\frac{2KH \tan \phi'}{B}\right)\right]$$

(4)

$$\sigma_{v1} = \sigma_{h1} / K$$

(5)

where $\gamma =$ fill bulk unit weight (kN/m$^3$); $B =$ width of stope (m); $H =$ total height of filled stope (m); $\phi' =$ fill effective angle of internal friction (degree); $\sigma_{v1} =$ vertical pressure at the floor of the stope (kPa); $K =$ coefficient of fill pressure. $K$ will correspond to three different states ($K_a, K_0, K_p$) given by the following relationships:

$$\begin{cases}
K = K_0 = 1 - \sin \phi' \\
K = K_a = \tan^2 \left(45^\circ - \phi' / 2\right) \\
K = K_p = \tan^2 \left(45^\circ + \phi' / 2\right)
\end{cases}$$

(6)

where $K_0 =$ coefficient of fill pressure at rest or in place (0.4 to 0.6); $K_a =$ coefficient of active fill pressure (0.17 to 1); $K_p =$ coefficient of passive fill pressure (1 to 10).

However, in a filled stope the active fill pressure condition ($K_a$) seems improbable. In Equation 6, the coefficient of fill at rest pressure can alternatively be evaluated using this well known relationship as follows:

$$K_p = \frac{v}{1 - v}$$

(7)

where $v =$ Poisson’s ratio of the fill ($0.3 \leq v \leq 0.4$).

2.3 Terzaghi’s cohesive model

Terzaghi (1943) also developed a two-dimensional arch theory for predicting the horizontal pressure ($\sigma_h$) along the pillar walls and this is given by:

$$\sigma_h = \frac{(\gamma B - 2c)}{2 \tan \phi} \left[1 - \exp \left(-\frac{2KH \tan \phi}{B}\right)\right]$$

(8)
\[ \sigma_y = \sigma_s / K \]  
\[ K = 1 / [1 + 2 \tan^{-1}(\phi)] \]  
where \( \gamma \) = fill bulk unit weight (kN/m\(^3\)); \( c \) = cohesive strength of fill (kPa); \( B \) = width of stope (m); \( H' \) = depth below fill toe (m); \( \tan \phi \) = coefficient of internal friction of fill; \( \phi \) = angle of internal friction of fill (degree); \( K \) = coefficient of fill pressure (see Eq. 10).

2.4 Proposed 3D model

Belem et al. (2004) proposed a three-dimensional model (companion paper) which implicitly takes into account the arching effects to predict the horizontal pressures, both the longitudinal pressure (\( \sigma_x \)) and the transverse pressures (\( \sigma_z \)). The model is given as follows:

\[ \sigma_x = \frac{\gamma H_m (H_m - z)}{3(B + L)} \left[ 1 - \exp \left( - \frac{2(h - z)}{B} \right) \right] \]  
\[ \sigma_z = \frac{0.185 \cdot \gamma H_m (H_m - z)}{B + L} \left[ 1 - \exp \left( - \frac{2(h - z)}{B} \right) \right] \]  
\[ \sigma_y = \sigma_y \]  
where \( \gamma \) = bulk unit weight of the fill (kN/m\(^3\)); \( H_m \) = total height of filled stope (m); \( z \) = elevation point of measurement (m); \( z = 0 \) at the floor of the stope; \( z = H_m \) at the fill toe (\( z \leq h \leq H_m \)); \( B \) = width of stope; \( L \) = strike length of stope (m).

3 DESIGN FOR CPB REQUIRED STRENGTH

The required strength for paste backfill depends upon its intended function. For a ground support role, the required uniaxial or unconfined compressive strength (UCS) of the fill should be at least 5 MPa whereas, for free-standing fill applications, the UCS can be commonly lower than 1 MPa (Stone 1993). Previous work indicates that the UCS of the fill mass can range to between 0.2 MPa and 5 MPa, while the UCS of the surrounding rock mass is between 5 MPa and 240 MPa.

3.1 Vertical support of backfill

The mechanical effects of fill are different from those of primary ore pillars. Research and in situ testing have shown that fill is incapable of supporting the total weight of overburden (\( \sigma_y \)) and acts only as a secondary support system (Cai 1983). The fill rigidity can range from 0.1 GPa to 1.2 GPa while the surrounding rock mass rigidity varies from 20 GPa to 100 GPa. As discussed by Donavan (1999), it is possible to assume that any vertical loading will be a result of roof deformation (Fig. 1) and that the design UCS can be estimated by the following relationships:

\[ \text{UCS}_{\text{design}} = \left( \frac{E_p \Delta H_p}{H_p} \right) FS \]  
where \( E_p \) = rock mass or pillar elastic modulus (kPa); \( \Delta H_p \) = strata length variation (m); \( FS \) = factor of safety.

When the stope walls deform before backfilling, the maximum load will probably never approach the total weight of the deformed overlying strata (Donavan 1999) and the design UCS can be estimated by following relationships:

\[ \text{UCS}_{\text{design}} = k(\gamma_p H_p) FS \]  
where \( k \) = scaling constant which must vary from 0.25 to 0.5; \( \gamma_p \) = strata unit weight (kN/m\(^3\)); \( H_p \) = strata height below surface (m); \( FS \) = factor of safety.

Numeric modeling can also be used to determine the required stiffness or strength of a CPB to prevent subsidence due to the roof deformation. The results can be very useful in indicating the amount of the paste backfill desired. Modeling can be done with either of the FLAC (2D and 3D) codes. Physical modeling, such
as with a centrifuge, also can offer an alternative to numeric modeling, but its application is usually limited to simple gravitational models without high tectonic or \textit{in situ} horizontal stresses (Stone 1993).

3.2 Development through backfill mass

When one wants to cut an access gallery to a new orebody through the paste backfill (Fig. 2), it is necessary to consider the original design criteria. This design considers a fill mass to be more than two contiguously exposed faces after blasting adjacent pillars or stopes. As a result, the walls confining the fill are removed and the fill block is subjected to gravity loading similar to a uniaxial compression sample (Yu 1992). The design UCS can be estimated by the following relationships:

\[
\text{UCS}_{\text{design}} = \left( \frac{\gamma_f}{H_f} \right) \text{FS}
\]  

(16)

where \( \gamma_f \) = fill bulk unit weight (kN/m\(^3\)); \( H_f \) = total fill height (m); \( \text{FS} \) = factor of safety.

3.3 Pillar recovery

In order to maximize ore recovery, it is very common to return for mine pillars after primary ore recovery. While this is being done, large vertical heights of massive paste backfill may be exposed. For delayed paste backfill, as used in open stoping operations, the fill must be stable when free-standing wall faces are exposed during pillar recovery (Fig. 3). It is necessary that the fill has sufficient strength to remain free-standing during and after the process of pillar extraction by resisting the blast effects. Figure 3 illustrates a possible failure mechanism which can occur after a stope blast. Depending upon the mining schedule, high strength for such engineering materials may not be required for the short term (Hassani & Archibald 1998).

In the absence of numeric modeling, many mine engineers still rely on two-dimensional limit equilibrium analyses along with a calculated safety factor (FS) to determine fill exposure stability. These analyses typically result in an over-conservative estimate of the limiting strength (Stone 1993) which increase the costs of backfilling operations.

In recent years, however, 2D- and pseudo-3D empirical models have been developed to account for arching effects, cohesion and friction along sidewalls (Mitchell et al. 1982, Smith et al. 1983, Arioglu 1984, Mitchell 1989a & b, Mitchell & Roettger 1989, Chen and Jiao 1991, Yu 1992). All these design methods use the concept of a confined fill block surrounded by the wall rock.

3.3.1 More than two exposed faces

Equation (16) should be used if there are more than two contiguously exposed faces after blasting adjacent pillars or stopes (Fig. 4).

3.3.2 Narrowly exposed fill face

This design method accounts for arching effects on confined fill by adjacent stope walls (Fig. 5) using

Figure 2. Digging an access gallery through the fill mass.

Figure 3. Fill block failure mechanism during secondary stope mining.

Figure 4. Schematic of a fill block with three exposed faces.
Terzaghi’s vertical pressure model (Eq. 9). Based on 2D finite element modeling, Askew et al. (1978) proposed the following formula to determine the design fill compressive strength:

\[ UCS_{\text{design}} = \frac{1.25B}{2K \tan \phi} \left( \gamma - \frac{2c}{B} \left[ 1 - \exp \left( \frac{-2HK \tan \phi}{B} \right) \right] \right) \times FS \]  

(17)

where \( B \) = width of stope; \( K \) = coefficient of fill pressure (see Eq. 10); \( c \) = cohesive strength of fill (kPa); \( \phi \) = angle of internal friction of fill (degree); \( \gamma \) = bulk unit weight of the fill (kN/m\(^3\)); \( H \) = total height of filled stope (m); FS = factor of safety.

The fill cohesion \( c \) and its angle of internal friction \( \phi \) can be obtained from triaxial tests performed on laboratory or in situ backfill samples.

3.3.3 Exposed frictional fill face

This design refers to an exposed fill where both opposite sides of the fill are against stope walls (Fig. 6). By assuming that there is shear resistance between the fill and stope walls due to the fill cohesion, the design UCS can be evaluated by the following relationship (Mitchell 1982):

\[ UCS_{\text{design}} = \frac{(\gamma L - 2c) \left( H - \frac{B}{2} \tan \left( 45^\circ + \frac{\phi}{2} \right) \right) \sin \left( 45^\circ + \frac{\phi}{2} \right)}{L} \times FS \]  

(18)

where \( \gamma \) = fill bulk unit weight (kN/m\(^3\)); \( c \) = cohesive strength of fill (kPa); \( L \) = strike length of stope (m); \( B \) = width of stope (m); \( H \) = total height of fill (m); \( \phi \) = angle of internal friction of fill (degree); FS = factor of safety.

Again, the fill cohesion \( c \) and its angle of internal friction \( \phi \) can be obtained from triaxial tests performed on laboratory or in situ backfill samples.

3.3.4 Exposed frictionless fill face

The compressive strength of paste backfill is mainly due to the binding agents and any strength contributed from friction can be considered negligible for the long term (i.e. \( \phi = 0 \)). For a frictionless material (Fig. 7), cohesion is assumed to be half of the UCS (\( c = UCS/2 \)). Thus, the design UCS can be evaluated by the following relationship proposed by Mitchell et al. (1982):

\[ UCS_{\text{design}} = \frac{(\gamma L - 2c) \left( H - \frac{B}{2} \right) \sin \left( 45^\circ \right)}{L} \times FS \]  

(19)

where \( \gamma \) = fill bulk unit weight (kN/m\(^3\)); \( c \) = cohesive strength of fill (kPa); \( B \) = width of stope (m); \( L \) = strike length of stope (m); \( H \) = total height of fill (m); FS = factor of safety (ca. 1.5).

In Equation 19, the fill cohesion \( c \) can be obtained from laboratory confined compression tests on backfill samples.

![Figure 5. Stability analysis of a narrowly exposed fill face.](image)

![Figure 6. Confined block with shear resistance mechanism (after Mitchell et al. 1982).](image)

![Figure 7. Confined block without shear resistance mechanism of frictionless fill (adapted from Mitchell et al. 1982).](image)

641
The stability of a free standing backfill (Fig. 7) can also be determined from physical model tests. Based on centrifugal modeling tests, Mitchell (1983) proposed a formula for evaluating the design UCS which is given by:

\[ UCS_{\text{design}} = \frac{\gamma LH}{L + H}FS \]  

(20)

where \( \gamma \) = fill bulk unit weight (kN/m\(^3\)); \( L \) = strike length of stope (m); \( H \) = total height of fill (m); \( FS \) = factor of safety.

3.4 Ground support

After passive resistance has been mobilized by the fill, the strength increase in the surrounding pillars will be equal to the magnitude of the passive fill pressure. So, the main stabilizing effect of the fill is to give increased lateral confinement pressure to the pillars (Fig. 8). The compressive strength of the pillar increases according to the following formula (Guang-Xu & Mao-Yuan, 1983):

\[ UCS'_{p} = UCS_{p} + [(\gamma H + q)K_{p-f}]K_{p-p} \]  

(21)

\[ K_{p-f} = \tan^{2}\left(45° + \frac{\phi_{f}}{2}\right) \]  

(22)

\[ K_{p-p} = \tan^{2}\left(45° + \frac{\phi_{p}}{2}\right) \]  

(23)

where \( UCS'_{p} \) = pillar compressive strength with fill (kPa); \( UCS_{p} \) = pillar strength before the stope filling (kPa); \( \gamma \) = fill bulk unit weight (kN/m\(^3\)); \( q \) = surcharge loading (kPa); \( H \) = total height of fill (m); \( \phi_{f} \) = angle of internal friction of fill (degree); \( \phi_{p} \) = angle of internal friction of pillar (degree); \( K_{p-f} \) = coefficient of passive pressure of the fill; \( K_{p-p} \) = coefficient of passive pressure of the pillar.

3.5 Working platform

For cyclic backfilling operations, as in cut-and-fill stoping, the fill in each operation must serve as a platform for both mining equipment and personnel and typically requires high strength development for the short term. A standard bearing capacity relationship that has been developed from civil engineering techniques for design of shallow foundations has been found to be applicable to paste backfill. The fill top surface bearing capacity, \( Q_{f} \) (kPa), can be determined using Terzaghi's expression (Craig 1995):

\[ Q_{f} = 0.4\gamma BN_{\gamma} + 1.2cN_{c} \]  

(24)

\[ N_{\gamma} = 1.8(N_{q} - 1)\tan \phi \]  

(25)

\[ N_{c} = \left(\frac{N_{q} - 1}{\tan \phi}\right) \]  

(26)

\[ N_{q} = \tan^{2}\left(45° + \frac{\phi}{2}\right) \exp(\pi \tan \phi) \]  

(27)

where \( \gamma \) = bulk unit weight of the fill (kN/m\(^3\)); \( c \) = cohesive strength of fill (kPa); \( B \) = width of square footing at surface contact position (m); \( N_{\gamma} \) = unit weight bearing capacity factor; \( N_{c} \) = cohesion bearing capacity factor; \( N_{q} \) = surcharge bearing capacity factor; \( \phi \) = angle of internal friction of fill (degree).

Equation 24 assumes that backfill bearing is by a square footing, which is a reasonable representation of the footprint of a mine vehicle tire (Hassani & Archibald 1998). Equation 25 was developed by Hansen (1968). For the mine vehicles (Fig. 9), the...
contact width, $B$, corresponds to the tire contact width and can be determined by the following relationship (Hassani & Bois 1992):

$$B = \sqrt{\frac{F_t}{p}}$$

where $F_t =$ tire loading force (kN); $p =$ tire air pressure (kN/m$^2$).

4 OPTIMIZATION OF PASTE BACKFILL MIX DESIGNS

Once the required strength has been determined, the mix variables can be optimized to provide the desired mix, which achieves the target strength with the lowest cement usage. The mix variables under consideration include the binder content and type, mill tailings grain size distribution and mineralogy, solids concentration, and the mixing-water chemistry. For the design of a certain uniaxial compressive strength ($\text{UCS}_{\text{design}}$), these variables can be adjusted to produce an optimal mix design (Stone 1993).

The other essential requirement is that backfill must be inexpensive. Typical costs of backfill range from $2$ to $20$ per cubic meter, depending on the service required. These costs can be a significant contribution to the operating costs of the mine. Where cemented backfill is used, these costs tend to be between 10 and 20% of the total operating cost of the mine and cement represents up to 75% of that cost (Grice 1998). Optimization of CPB-mix designs can reduce binder usage and can offer significant cost savings (Fall & Benzaazoua 2003).

4.1 Laboratory optimization of CPB mix designs

Figure 10 shows the main components which can affect the quality of cemented paste backfill such as the binding agents, mill tailings mineralogy, mill tailings grain size, the density and solids percentage of tailings and finally, the mixing-water geochemistry (Benzaazoua et al. 2002).

Each component plays an important role for the backfill transportation, its delivery and its strength development in the course of curing time. Typical binder percentages are 3 to 7% by weight of the paste fill. Numerous laboratory test results have reported that the backfill strength is a function of binder content for a given curing age (Fig. 11), but this relationship is specific to each mine (e.g. Benzaazoua et al. 1999, 2002, 2004).

4.1.1 Cement and others binders

Hardening of the fill occurs as bonds are formed between fill particles at grain contact points. Many different types of binding agents are used, but the most common is ordinary Portland cement (OPC). Admixtures with pozzolanic materials are also used to curb costs by reducing the amount of Portland cement needed for hardening. Fly ash (FA) and smelter ground blast furnace slags (BFS) are the most popular pozzolans used as admixtures. The results of cement dissolution tests performed by Benzaazoua et al. (2004) showed that in either concrete or mortar, the hardening processes within the pastefill are not only due to the cement hydration but also to the precipitation of hydrated phases from the pore water of the paste. Figure 12 illustrates that paste backfill hardening occurs in two main stages: the first stage (dissolution-hydration) which is dominated by the dissolution reactions and the second stage (precipitation and hydration) which is characterized by the precipitation reactions and direct hydration of the binder. More details on this subject can be found in Benzaazoua et al. (1999, 2002, 2004).

Water is necessary to ensure that proper hydration of the cement occurs. If proper hydration of the cement does not occur, the fill will not meet its required strength and stiffness. Since tailings backfill is fairly
saturated to begin with and additional water is usually required to pump it underground, the water content of tailings backfill is always in far excess of what is required for hydration of the Portland cement. The main concern then is the pH of the water and the amount of sulfate salts present in the water. Acidic water and sulfate salts can attack the cement bonds within the fill, leading to a loss of strength, durability, and stability (Benzaazoua et al. 2002, 2004).

Figure 13 shows that when using cement–slag binder with the same tailings sample mixed with three different waters, the strength development is slow for all three waters for a curing time of 14 days (Benzaazoua et al. 2002). Beyond this curing date and at a curing time of 28 days UCS reached a value of about 1600 kPa with the sulfate-free waters (municipal and lake waters) and only 1000 kPa with the sulfate-rich water (mine A process water).

4.1.2 Mixing process

Some amount of water is added to set the resultant paste backfill to attain the desired slump value. The slump must vary from 12.4 to 25.4 cm (5 to 10 in) which correspond to solids concentrations of 78% to 82% by weight. Slump is a measure of the drop in height a material undergoes when it is released from a cone-shaped slip mold. Determination of slump provides a way to characterize a material's consistency that can be related to its transportability. The resultant paste backfill mixtures were poured into plastic cylinders 10.25 cm in diameter and 20.5 cm height, sealed and cured in a humidity-controlled chamber at approximately 90–100% relative humidity (similar to underground mine working conditions). The pastebackfill samples were then subjected to uniaxial compression tests for periods of 7, 14, 28, 56 and 91 days.

4.2 CPB preparation at the backfill plant

Figure 14 shows a typical flow chart for a backfill plant. The final mill tailings are first fed to a high-capacity thickener to increase their solids percentage to approximately 55% to 60% by weight. To aid filtration some flocculent is added. The thickened tailings are then pumped from the thickener to a high-capacity holding tank (after cyanide destruction). From the surge tank, the thickened tailings are fed by a gravity circuit to disc filters operating alone or in parallel to produce a filter cake with a solids percentage of approximately 70% to 82%. The filter cake is then discharged onto a belt (or reversible) conveyor and is then fed to a screw feeder for weighing. Filter cake batches are mixed in a spiral (or screw) mixer with cement and water and is added to produce a paste with a specified slump (127 to 254 cm). The mixed paste is dropped into a surge hopper and discharged underground under vacuum (by gravity or using concrete pump).

5 RHEOLOGICAL PROPERTIES OF CPB

Paste backfill consists of the full size fraction of the tailings stream prepared as a high slurry density. The slurry behaves as a non-Newtonian fluid, which means
that it requires an applied force to commence flowing (Fig. 15).

Toothpaste is an example of a non-Newtonian fluid that is commonly used and the yield stress (applied force) explains why you have to squeeze the toothpaste out of the tube. The paste has a higher viscosity and exhibits plug flow when transported in a pipe. The outer portions of the slurry shear against the sidewall of the pipe and the central core travels as a plug (Grice 1998). The flow of paste backfill in pipeline is entirely governed by their rheological properties. Rheology is the science about flow and deformation of matter.

5.1 Rheological models of CPB

The main mode for paste backfill flow in pipelines is the full-fill. Full-pipe flow refers to the situation where the flowing paste forms a continuum and there is no air-filled gap or discontinuities (vacuum "holes") anywhere in the pipeline segment under consideration (Li et Moerman. 2002).

The most fundamental relationship in the rheology of a non-Newtonian fluid is that between the shear rate, $\gamma$ (s$^{-1}$) and pipe wall shear stress, $\tau_w$ (Pa). Once this relationship is known, the behaviour of the fluid in all flow situations can be deduced. All non-Newtonian fluid rheology can be derived from the most general Herschel-Bulkley model given by:

$$\tau_w = \tau_0 + k\left(\frac{dV}{dr}\right)^n = \tau_0 + k\dot{\gamma}^n$$ (29)

where $\tau_0 =$ yield stress (Pa), $k =$ consistency parameter or viscosity (Pa.s), $(dV/dr) =$ paste angular velocity or shear rate (s$^{-1}$); $r =$ point of velocity profile (m), $R =$ radius of the pipe (m), $V =$ paste linear velocity (m/s), $n =$ flow parameter.

For the Newtonian fluids, $\tau_0 = n = 0$; for the pseudoplastic fluids, $\tau_0 = 0$ and $n < 0$; for dilatant fluids, $\tau_0 = 0$ and $n > 0$; for Bingham plastic fluids, $\tau_0 > 0$, $n = 0$ and $k = \eta =$ plastic viscosity in Pa.s (Fig. 16); for yield pseudoplastic fluids, $\tau_0 > 0$ and $n > 0$ (Fig. 16); for yield dilatant fluids, $\tau_0 > 0$ and $n < 0$.

Paste backfills are non-Newtonian fluids and their rheology is approximately time-independent during its transport in pipeline. Most paste backfill show an appreciable yield stress and are Herschel-Bulkley fluids (Eq. 29). Some paste backfills are Bingham plastic in limited shear rate ranges. Others are yield pseudoplastic or yield dilatant, with the former more common than the latter.

The relationship between the pseudo shear rate, $8V/D$, and the shear stress at the pipe wall, $\tau_w$, is given by:

$$\tau_w \approx \frac{D\Delta P}{4L} = \eta \frac{8V}{D^2} \left[ 1 - \frac{4}{3} \left( \frac{\tau_0}{D \Delta P} \right) + \frac{1}{3} \left( \frac{4L}{D \Delta P} \right)^{1/3} \right]$$

(30)

where $\tau_0 =$ yield stress (Pa), $\eta =$ paste plastic viscosity (Pa.s), $\Delta P =$ differential pressure in the pipe (Pa); $D =$ internal pipe diameter (m), $L =$ pipe length (m); $V =$ paste laminar velocity (m/s).

The effective pipes diameter ($D$) for paste backfill transport is ranged between 10 cm and 20 cm (4 and 8 in). Paste flow velocity varies from 0.1 m/s to 1 m/s. The practical pumping distance of paste can reach 1000 m longitudinally ($L_b$) and unlimited vertically ($L_v$).

5.2 Standard measurements of the CPB consistency

In practice, it is not easy to obtain the true rheological properties of pastes due to the complexity of the experimental devices. This makes difficult, even impossible,
the determination or the prediction of a pastes viscosity which depends on several factors. That is why the standard slump test (used in concrete experiments) is widely used, due to its simplicity, to determine paste backfill consistency. Slump is a measure of the drop in height of a material when it is released from a truncated metal cone, open at both ends and sitting on horizontal surface (Fig. 17). Determination of the slump provides a way to characterize a material's consistency that can be related to its transportability (Clark et al. 1995). According to Landriault et al. (1997), the ideal slump of the paste must be in a range between 150 mm (6 in) and 250 mm (10 in) to facilitate the flow of cemented paste backfill by its pumping underground.

Solids concentration is often used to compare the composition of mixes, particularly in batch. Although solids percentage does not provide a direct indication of a material's consistency, in some cases it can be correlated to the slump, which does.

In order to achieve the same mix consistency from batch to batch, consistency can be measured by monitoring the electrical power used by a motor turning the paddles of a mixer. The mixer is started and water is added until the power required by the motor corresponds to the target power for the mix consistency desired (Brackebusch 1994, Landriault & Lidkea 1993). Using this arrangement requires only that slump be correlated to consistency and consistency be correlated to power. It is also possible to predict what pressure gradient a mix will produce based on power once a correlation has been established between slump and pressure loss.

5.3 Alternative methods for rheological factor measurements

To correctly define the rheology of paste backfill both the yield stress ($\tau_0$) and the viscosity ($\eta$) need to be measured. Most current tests measure only one rheological factor. The relationship between the factor measured and either of the two fundamental rheological parameters is not obvious. In most cases, $\tau_0$ and $\eta$ cannot be calculated from the factor measured, but can only be assumed to be related. According to Ferraris (1999), slump, penetrating rod and K-slump tests are related to the yield stress ($\tau_0$) because they measure the ability of paste to start flowing. The remolding test, LCL apparatus, vibrating testing apparatus, flow cone, turning tube viscometer, filling ability and Orimet apparatus are related to the viscosity because they measure the ability of paste to flow after the applied stress (vibration or gravity) exceeds the yield stress.

Recently, a modification of the slump cone was developed to allow the measurement of viscosity (Ferraris & de Larrard 1998). As mentioned earlier, the standard slump test can only be correlated with the yield stress ($\tau_0$). The modification consists of measuring not only the final slump height but also the speed at which the concrete (or paste backfill) slumped. The method consists of measuring the time ($T$) for a plate resting on the top of the concrete to slide down with the concrete (or paste backfill) a distance of 100 mm (Fig. 18).

The yield stress, $\tau_0$, can be calculated from the final slump ($S$), using the following empirical equation proposed by Ferraris & de Larrard (1998):

$$\tau_0 = \frac{\rho(H - S)}{a} + b$$

where $\rho$ = paste density (kg/m$^3$); $S$ = final slump (mm); $a$, $b$ = material constants, $H = 300$ mm is the cone height. For the concrete paste, $a = 347$ and $b = 212$.

From a range of paste backfill slump values (130–250 mm), the viscosity can be determined from the 100 mm slump time ($T$) using an empirical equation that was developed by Ferraris & de Larrard (1998):

$$\eta = k\rho T$$

where $\eta$ = viscosity (Pa.s); $k$ = material constant ($k = 0.025$ for concrete); $\rho$ = paste density (kg/m$^3$); $T$ = slumping time (s).

Other authors (Nguyen & Boger 1985) have suggested adapting the laboratory vane shear test for the

![Figure 17. Paste backfill consistency measurement by slump tests: a) slump cone mold; b) schematic view of the slump test.](image)

![Figure 18. Schematics of the modified slump cone test based on slumping time $T$ measurement (after Ferraris & de Larrard 1998).](image)
measure of paste yield stress ($\tau_0$). This test allows obtaining a torque-angular deformation curve of the paste whose peak corresponds to the maximum torque ($\Gamma_m$). If these these parameters are known, the yield stress can then be calculated by the following relationship:

$$\tau_0 = \frac{\pi D^2}{2} \left( \frac{H}{D} + \frac{1}{4} \right) \left( \frac{1}{3} \right)$$

where $\tau_0 =$ paste yield stress (Pa), $\Gamma_m =$ maximum peak torque value (N.m), $D =$ vane diameter (cm); $H =$ vane height (cm).

6 CPB TRANSPORT BY PIPELINES

6.1 Type of underground distribution systems

There are three possible configurations for moving fill material from a point on the surface to the underground stopes as shown on Figure 19 (Thomas 1979).

As discussed by Thomas (1979), the “gravity/pump” system (Fig. 19) has the advantage of being totally contained underground, thus causing no disruption to surface activities. Furthermore, the ratio of the vertical to horizontal distance is usually so favourable that little or no pumping energy is required.

The “gravity” system (Fig. 19) has the advantage of by converting vertical head to horizontal pressure progressively which allows shorter and lighter pipes to be used. The pressure at the take-off points are moderate and line failures, if any, do not disrupt the main shaft or main level of operation. The circuit can be developed progressively as the mine expands.

The “pump/gravity” system (Fig. 19) has the advantage of easy installation, inspection and maintenance, with no special underground level requirements and no disruption of the main shaft. However, such a system makes the filling operation dependent upon a pumping operation and requires a long borehole to place fill underground which results in a high pressure take-off point.

6.2 CPB transport underground

The paste backfill is delivered by pipeline to the disposal point in the stope and the friction factors generated require that high pressure pipelines be used to transport the pastefill. Pressures typically exceed 5 MPa for this type of laminar flow system. Early systems used high pressure reciprocating pumps but experience has shown that pastefill can be readily transported by gravity alone, provided that the reticulation geometry is favourable (Grice 1998).

6.2.1 Flow-loop tests of the CPB

For a given mine, a fully instrumented pipes for paste backfill flow-loop tests must be performed to determine the paste transport characteristics. Usually this is an instrumented, closed-circuit pipeline system powered by a diesel engine positive-displacement pump. The instrumentation on the paste flow-loop tests provides essential engineering data such as flow rate ($Q$), friction head loss per unit length of pipe ($f = \Delta P/L$), shutdown and restart capabilities, and power consumption needed to design full-scale pipelines. Figure 20 is an example of paste flow-loop tests performed at the USBM’s Spokane Research Center (Clark et al. 1995).

The calculation of the friction head loss ($\Delta P/L$) will allows determination of the running pressures of the paste distribution system: type of volumetric displacement pump, choice of pipe diameters ($D$), flow rate ($Q$), and paste flow velocity ($V$). For a Bingham plastic fluid

![Figure 19. Basic configurations for paste backfill distribution systems (adapted from Thomas et al. 1979).](image)

![Figure 20. Pastefill flow-loop tests configuration and pressure monitoring locations (after Clark et al. 1995).](image)
flowing in laminar regime (pastefill), the friction head loss \( f \) is given by the following relationship:

\[
f = \frac{\Delta P}{L} = \frac{32\eta_B V}{D^2 \left( 1 - \frac{4\tau_w + \frac{1}{3} \tau_0}{3\tau_0} \right)^{\frac{1}{4}}}
\]

(34)

where \( f \) = friction head loss (Pa/m); \( \eta_B \) = Bingham plastic viscosity (Pa.s); \( \tau_0 \) = yield stress (Pa); \( \tau_w \) = wall shear stress in Pa \( \tau_w = \frac{\Delta P}{4L} \); \( D \) = pipe diameter (m); \( \Delta P \) = differential pressure in the pipe (Pa).

The use of rheological models such as Equation 33 requires the \textit{a priori} knowledge of the paste Bingham plastic viscosity \( \eta \) which is very difficult to predict because it depends on several factors. That is why it is important to relate the slump value to the plastic viscosity as the relationships (Eqs. 30 & 31) proposed by Ferraris & de Larrard (1998). The pipe diameters often used vary between 100 mm (4 in) and 200 mm (8 in). For example, a paste backfill with a slump value of 180 mm (7 in) can be transported by gravity at a flow rate of 100 ton/hour in boreholes/pipes system with a 150 mm (6 in) diameter.

### 6.2.2 Horizontal transport distance

The horizontal transport distance \( L_h \) generated by a standing column of material is obtained by dividing the pressure at the bottom of the standing column \( P_{\text{bottom}} \) by the frictional pressure gradient or pressure loss (Clark et al. 1995). The pressure at the bottom of a standing column is obtained by taking the difference between the pressure imparted by gravity and pressure lost through frictional pressure gradient, so that horizontal transport distance \( L_h \) is given par the following relationship (Fig. 21):

\[
L_h (m) = \frac{P_{\text{bottom}}}{f} = \frac{\left( \frac{\gamma H - \Delta P}{L} \right) L}{\Delta P}
\]

(35)

![Figure 21. Schematic illustrating the calculation of the horizontal distance of paste flow.](image)

where \( \gamma \) = fill bulk unit weight (kN/m\(^3\)); \( H \) = maximum free-fall height of the paste in the paste (m); \( \Delta P/L \) = friction head loss (Pa/m).

### 7 BACKFILL DELIVERY IN THE STOPES

Once all the transport parameters are correct, the paste backfill can be delivered to underground openings through pipelines. Figure 22 shows a general outline of a backfilled stope with its various components (fill mass, barricade, rock mass, adjacent filled stope) as well as the stress field distribution.

After the stope is backfilled with CPB its mechanical integrity can be threatened by several macroscopic factors (in opposition to the hydration process) which are going to influence the mechanical strength of the CPB and the structural stability of the filled stope. These factors which result from interactions between CPB and rock walls are, fill settlement and the drainage of its excess water, fill consolidation, stope volume, stress field distribution within the backfill mass (pressures at the floor of the stope and on the barricade), wall convergence against the fill mass, shrinkage and the arching effect.

Drainage and settlement will favour the development of a high mechanical strength of the CPB (Belem et al. 2001, 2002). On the other hand, the fill mass will be stable due to the development of arching effects depending upon the stope dimensions.

The pressures at the floor of the stope and on the barricade will have a harmful effect on the stability of the filled stope when these pressures are too high (see more details in Belem et al. in the companion paper). Consequently, it is necessary to understand these various factors which influence stope stability to ensure better ground control.

The knowledge of the magnitude of the pressures on the barricade will allow better planning of the mining sequences. The knowledge of the stress field within

![Figure 22. Schematic showing the components of a backfilled stope and the stress field distribution.](image)
the fill mass will facilitate its stability analysis when it is considered that one of its faces may be exposed or when one wants to cut an access gallery to a new orebody through the CPB.

8 CONCLUSION

This paper is a general overview on the use of cemented paste backfill, from its design to its underground delivery. When a mining method uses paste backfill, initially one must determine the limiting strength and the pressures which will be developed in the fill according to the geometry of the opened stopes.

To meet these criteria, laboratory optimization of paste backfill mix design will be essential to determine the ideal mixture to achieve the desired limiting strength. But before beginning the stoping filling, it would be necessary to know the rheological properties of the fill material. For that purpose, one will select a rheological model of paste backfill behaviour (Bingham or Pseudo-plastic) to determine the two essential parameters, yield stress and viscosity.

The pumpability of the paste backfill can be also estimated using the standard or modified slump tests. This last would allow relating the slump and the "slumping time" to the yield stress and the plastic viscosity. According to existing distribution system at the mine concerned (e.g. gravity, pumping, etc.), paste flow-loop tests are necessary to estimate the friction head loss of the pipelines for better control of the operating pressures.

With this last parameter, it would be also possible to calculate the maximum horizontal distance for the paste flow without any additional pressure. Once the paste backfill is transported underground through the pipelines to the open stopes, it will interact with the stopes and pillar walls and its initial physical and mechanical properties will evolve in the course of its curing time.

ACKNOWLEDGMENTS

This research was supported by the IRSST and parts of NSERC and NATEQ. The authors gratefully acknowledge their support. The authors would also like to thank our mining partner, Cambior Inc. (Mine Doyon) for their collaboration in the completion of this work.

REFERENCES


