Numerical Modeling of Paste Sills in Underhand Cut & Fill Stopes

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ABSTRACT

This paper describes a focus of work presently being conducted at the Rock Mechanics Research Group at the University of British Columbia. The underhand method under consolidated fill ensures a high recovery under an engineered back that is comprised of cemented rock fill and/or cemented paste fill. This method of mining has been employed successfully in mines throughout North America as a method of mitigating exposure to the operator having to work under a seismically active "back". This paper reviews design methodology in the placement and analysis of sill mats with reference to site observation coupled with on going numerical and analytically derived solutions.

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1 INTRODUCTION

With increasing demand on minerals and escalating market prices, the need to mine deeper depths in high stress mining environments is becoming more common. Within these high stress environments the frequency of rock bursts have increased. In order to provide a safe working environment underhand cut and fill with paste has become a more widely used mining method. The placement of consolidated backfill requires one to understand the overall factors affecting design. Figure 1 graphically summarizes some of the parameters that are being investigated in terms of their implication on developing a design span enabling man entry access. A sill for this study is defined as a consolidated layer of previously placed fill immediately above the mine opening that is being excavated.

Through numerical modelling, the UBC Geomechanics Centre has done a sensitivity analysis of paste unconfined compressive strength (UCS) versus sill width. A further comparison has been with modelled UCS strengths compared to existing literature for determining span width based on UCS.

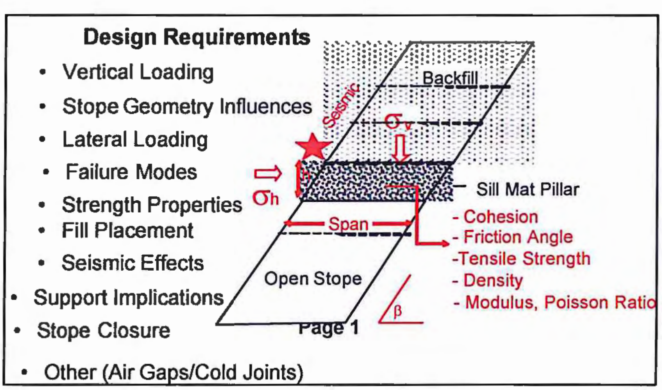

Figure 1: Mining Under Consolidated Backfill

2. DESIGN CONSTRAINTS

Figure 1 shows the factors that have to be accounted for in terms of mining under an engineered back. These will be outlined in this paper from a general perspective with focus on the analytical and numerical assessment of span and applied loading conditions.

2.1 Design Load

A critical factor is estimating the design loads onto the sill mat. Caceres (2005) employs an existing database of a Canadian mine as a case study that looks at the loading conditions that exist on a cemented rockfill sill mat. Design loads are critical to determine (or "in determining") the strength required of the sill mat for the given stope geometry as under-estimating can cause a premature failure of the sill mat once mining exposes the mat, whereas overestimating can result in unnecessary expenses due to the cost of the cement in place. Estimating the vertical loading is not a trivial solution as many factors affect the overall loading conditions as evident from the many theoretical derivations that are available as per Janssen (1895), Terzaghi et al.(1996), Reimbert (1976) and Blight (1984) all of which have significant assumptions in terms of coefficient of lateral earth pressure "K" as detailed by Marcinyshyn (1996). The typical geometry was modelled employing FLAC^{2D} (Itasca, 2005) which did not have the constraints the analytical methods had in terms of 'K" and stope inclination. An analytical approximation, as shown by Eqn. 1, was derived by Caceres (2005) relating the numerical simulation to an equivalent relationship.

Eqn. 1:

$$\sigma_y(z) = \left(\frac{\gamma \cdot L}{2 \cdot K \cdot \tan(\phi)}\right) \cdot \sin^2(\beta) \cdot \left[1 - \exp\left(-\frac{2 \cdot K \cdot \tan(\phi) \cdot z}{L \cdot \sin^2(\beta)}\right)\right]$$

Where:

-L = Span of stope

-z = Height of fill

-? = Coefficient of lateral earth pressure

-? = Unit weight of fill

-F = Friction Angle of Fill

-? = Stope dip angle

As mentioned, the above was derived for unconsolidated or uncemented rock fill, however, the analytical solution would be similar to that of unconsolidated paste as the input parameters would define the loading conditions.

2.3 Failure Mechanism

The methodology of span design under consolidated fill is complex as many factors control the overall stability as shown in Figure 1. The failure modes and combination thereof must be analysed with respect to the placed fill, stope geometry, loading conditions, seismic effects, stope closure, and support placement as well as other factors that are due to filling practises such as cold joints and gaps between successive lifts among others. This paper employs analytical, numerical and empirical tools to attempt to provide an initial tool for design by the operator. The database of underhand stopes observed by the author is shown in Table 1, which is comprised of twelve(12) operations which include seven cemented rock fill and five having paste within the immediate back.

The unconfined compressive strength is typically the parameter employed to benchmark the overall stability of the immediate back. The compiled database (Table 1, found at the end paper) of backfill unconfined compressive strengths was adapted from Souza et al.(2003).

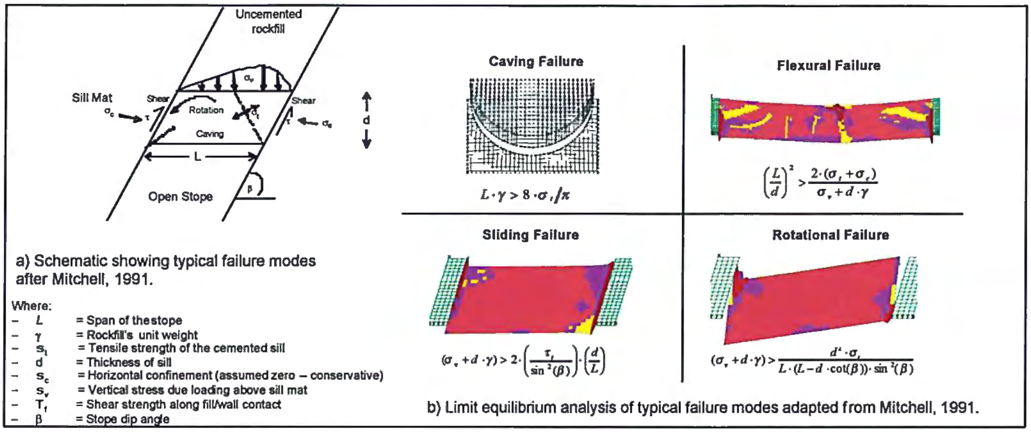

Figure 2: Limit equilibrium criteria adapted from Mitchell, 1991.

The design methods (Table 1) all employed a form of limit equilibrium analysis coupled with modelling. The failure modes are summarized by Mitchell (1991) and shown in Figure 2.

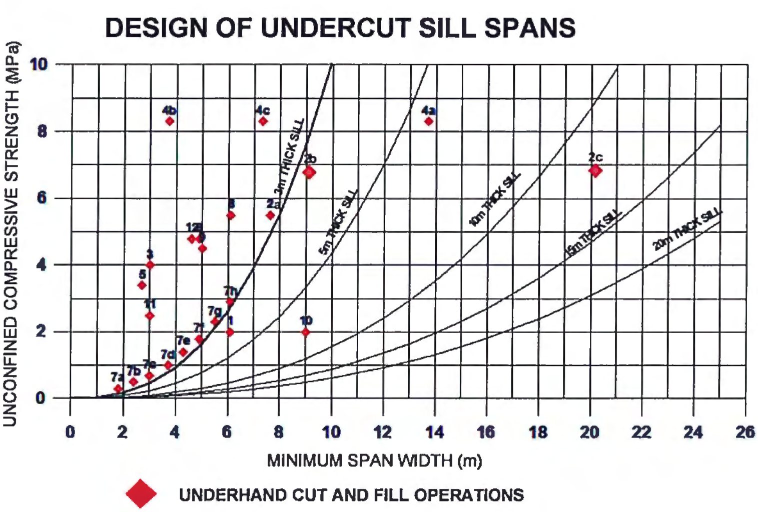

Figure 3: Stability chart for the design of undercut sills with vertical sidewalls with a FS of 2. Chart is based upon fixed beam bending failure with surcharge, adapted from Stone, 1993. (Pakalnis, 2005)

Flexural instability was found to be most critical in the absence of rotational instability and closure stresses (s_c) which have to be evaluated separately. Stone (1993) had concluded that, for cemented rock fills, crushing, caving and sliding are generally negated when the sill thickness exceeds 0.5 x span. In the absence of closure stresses, when the unconfined compressive strength of the cemented rock fill is greater than 1.5MPa, the kinematically possible rotational instability has to be analysed separately. Figure 3 shows the database that has been compiled in Table 1 and plotted onto a stability chart adapted from Stone(1993) and developed for the design of sills with vertical sidewalls with a Factor of Safety of two. The chart is based upon flexural instability employing fixed beam analysis with surcharge loading after Eqn. 1. It shows the unconfined compressive strength required (FS=2) for a given sill thickness and span exposed and related to actual field observations. Generally the mine data was found to be more conservative than the required for a Factor of Safety of 2.0. This may reflect the quality control requirements at individual operations, along with other factors such as seismicity and stope geometry among others as shown in Figure 1.

Caceres (2005) simulated the limit equilibrium approach shown in Figure 2 employing FLAC^{2D} models (finite difference code) for a given value of cohesion, friction angle, vertical surcharge, span and stope dip. The backfill properties assigned are for a Mohr-Coulomb type of material with strain-softening behaviour where integrity is lost after 1.5% strain (Swan and Brummer, 2001). The resultant mode of failure was analysed for various stope dips with cohesion on the hanging wall contact varying from zero to maximum value (equal to the cohesion value of sill). From the simulations, it was found that the analytical approach after Mitchell,(1991) which assumes no hanging wall cohesion for the rotational instability and this resulted in a high degree of conservatism.

2.3 Other Factors

The above attempts to outline a methodology for span design. It is critical that the method be calibrated for individual sites, incorporating critical factors such as seismic conditions, installed support, wall closure and methods of fill placement as these all play a significant role in ensuring a safe exposed operating span. A major benefit of mining under paste is the mitigation of the hazards posed by bursting (Blake et al, 2004).

3 SEISMIC CASE HISTORY – MANAGING ROCKBURSTS BY EMPLOYING UNDERHAND PASTE FILL AT HECLA'S LUCKY FRIDAY MINE, MULLAN IDAHO

The following have been compiled by Blake and Hedley, 2003. Its importance is that the underhand mining as practised at Lucky Friday (Mine #12 in Table 1), is the first to incorporate paste to mitigate burst damage and the method has been adopted at mines throughout North America such as the Red Lake Mine in Ontario (Mah et al.,2003) and the Stillwater Mine in Montana (Jordan et al., 2003).

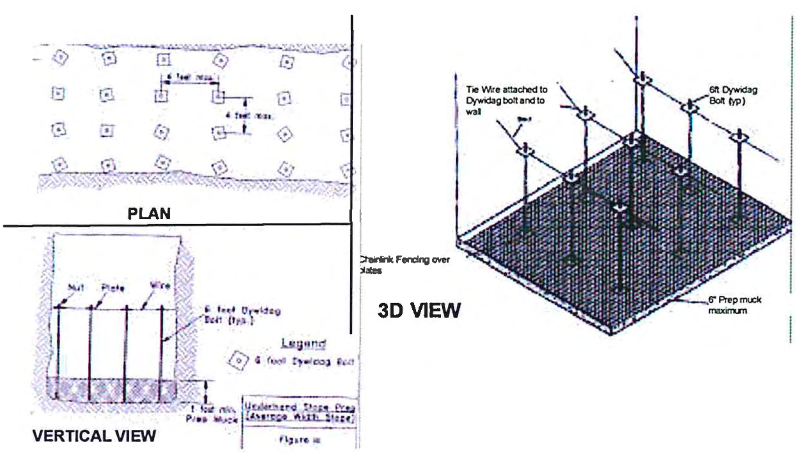

Figure 4: Lucky Friday Mat

Hecla initiated overhand cut-and-fill mining on the Silver Vein at the Lucky Friday Mine in the late 1950's. By the mid 1960's mining had progressed down to the 3050 level (~930m below surface), and the mining geometry consisted of long, flat-backed stopes, all at the same elevation, being carried up from two or more levels simultaneously. A burst prone sill pillar was formed when mining from below would approach the overlying mined out level. As a result of a double rockburst fatality in 1969, the mining front was changed to"centre lead stope" geometry. In 1973 the first computer controlled seismic monitoring system was installed, and pillar distressing was routinely carried out when a sill pillar was mined to approximately 12m (thickness).

This rockburst strategy allowed mining to proceed safely down to below the 4660 level (~1420m below surface). In 1982 the mining front entered a highly burst prone formation, and serious rock burst problems were encountered. As a result of rockburst fatalities in 1984 and 1985, Hecla initiated an experimental underhand cut-and-fill stope along the east abutment of the mine. After another rockburst fatality in March 1986 Hecla realized that it was not possible to manage their rockburst problem with overhand cut-and-fill mining. Production mining at the Lucky Friday was stopped in April 1986, and plans were made to convert the entire mine to mechanized underhand cut-and-fill mining geometry, which they named LFUL – Lucky Friday underhand longwall. The key features of this mining method were that pillars would never be formed, and the mining would be carried out under a stable, engineered, paste type fill back as shown in Figure 4.

Production mining at Lucky Friday resumed in October 1987 incorporating the above changes. Despite increased rates of rockbursting, as well as larger magnitude bursts (Ml 4.1), underhand cut-and-fill mining at the Lucky Friday has been carried out without any serious rockburst injuries or fatalities, and with greatly increased productivity at significantly reduced costs. Underhand mining has allowed Hecla to very effectively manage their rockburst problem. The miners have a higher sense of security working below an engineered back. *Management has said that the mine would likely have never reopened after 1986 had it not been for the all the benefits of LFUL mining.*

Finally, the paste backfill is only very rarely damaged by the effects of nearby rockbursts. The only burst induced fill failure at the mine occurred in 1991 during mining of a remnant pillar where a 3.5Ml burst caused the wall to fail and in turn undercutting the past back which collapsed. The peak particle velocity at the hanging wall/fill mat was approximately 1m/s. Despite closure from ongoing mining, as well as closure and shock loading from the burst, the fill was not rubbilized as might have been expected.

4 ONGOING RESEARCH

The following is a summary of current work being completed at the University of British Columbia's Norman B. Keevil Institute of Mining on paste backfill.

4.1 Numerical Model Description

A FLAC^{2D} was constructed to represent a high angle ore body with competent host rock in which underhand cut and fill mining is being implemented. The model was constructed in such a way that span widths, span heights, rock properties, paste properties and wall closure can be specified. The model was set up with far field stress equal to a plane strain, $s_z = s_v$ =0.027MPa/m depth, $s_x = s_y$ = 2/5(s_v) based on gravity loading (Figure 5) with a 42^o friction angle. The footwall and hanging wall have an angle of 70°.

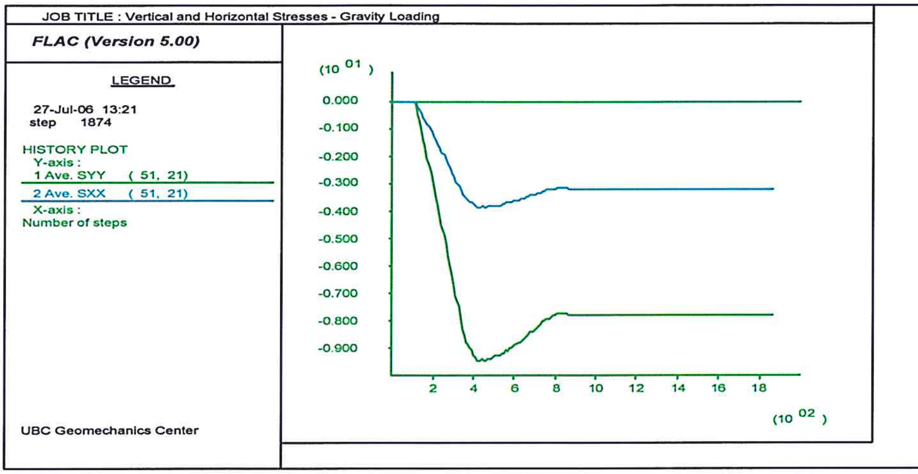

Figure 5: Plot of Horizontal and Vertical Stress used within FLAC^{2D} model

The rock properties were equivalent to a competent limestone with properties shown in Table 2. The cohesion and tensile strength was equivalent to ¼ and 1/10 of the UCS (Jaeger et al., 1976) respectively and Elastic properties values were obtained from various limestone tests performed at the UBC Geomechanics Centre. The rock behaved according to a Mohr-Coulomb constitutive model. The amount of wall closure is user specified and has initially been tested for 0 and 10mm of wall closure.

Table 2: Rock: Properties

Properties	Value
UCS Friction Angle	22Mpa 42°
Elastic Modulus	25GPa
Poisson's Ratio	0.22

The paste fill properties were user defined and were typically based on values obtained in literature from Pakalnis (2005) and Caceres (2005). The elastic modulus was based on the UCS of the paste and was determined according to Eqn 2.. This equation serves only the purpose of allowing the elastic modulus to reflect a change in UCS during coding. It should be noted that Eqn 2. is a best fit line of limited test samples.

$$E = \begin{cases} 0.15\,GPa \quad UCS < 0.4\,MPa \\ -371.6 \cdot UCS^2 + 1317.9 \cdot UCS - 274.4 \qquad 0.4 < UCS < 1.75\,MPa \end{cases} \qquad (2)$$

The paste behaved according to a Mohr-Coulomb strain softening constitutive model with cohesion friction angle and tension decreasing in value as the strain increases as seen in Figure 5 a,b,c.

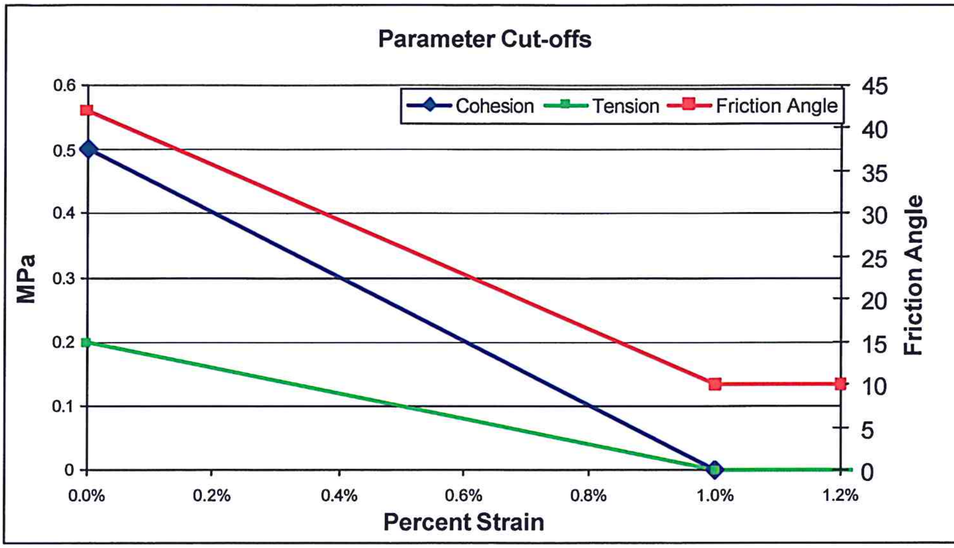

FIGURE 6: (a)Cohesion vs. Strain cut-off; (b) Friction Angle vs. Strain cut-off (c) Tensile Strength vs. Strain cut-off

The following geometries were modelled:

- 3m Span with sill thickness of 3, 3.5, 4 and 23.5 m thickness
- 6 m Span with sill thickness of 3, 3.5, 4 and 23.5 m thickness
- 8 m span with sill thickness of 3, 3.5, 4 and 23.5 m thickness

The air gaps of 0.5m were assigned between subsequent lifts with exception of the 23.5m thickness which is supposed to represent a near infinite span. Models were run with only the UCS of the paste being modified. This had a trickle down effect of altering the cohesion, tension and elastic modulus of the material. Span width and UCS paste combinations were marked as stable or unstable. An unstable scenario was considered a model with no numerical convergence or sill mat failure.

4.2 Initial Findings

Figure 7 shows the initial findings of the model for a span of 23.5 m height. Note that the wall closure consist of 5mm of closure on the hanging wall and footwall respectively. It can be seen that as the span width decreases the effect of wall closure affects the necessary strength of paste required for a stable span. This is due to the increase in strain within the paste and is considered a crushing failure. The zero convergence line follows a similar trend as the 10mm of wall closure line for large spans yet a deviation occurs as the spans become smaller than 4 meters. As the wall closure is mine sensitive and is independent of span width, it is more beneficial to real world applications to plot wall closure rather than strain within the paste.

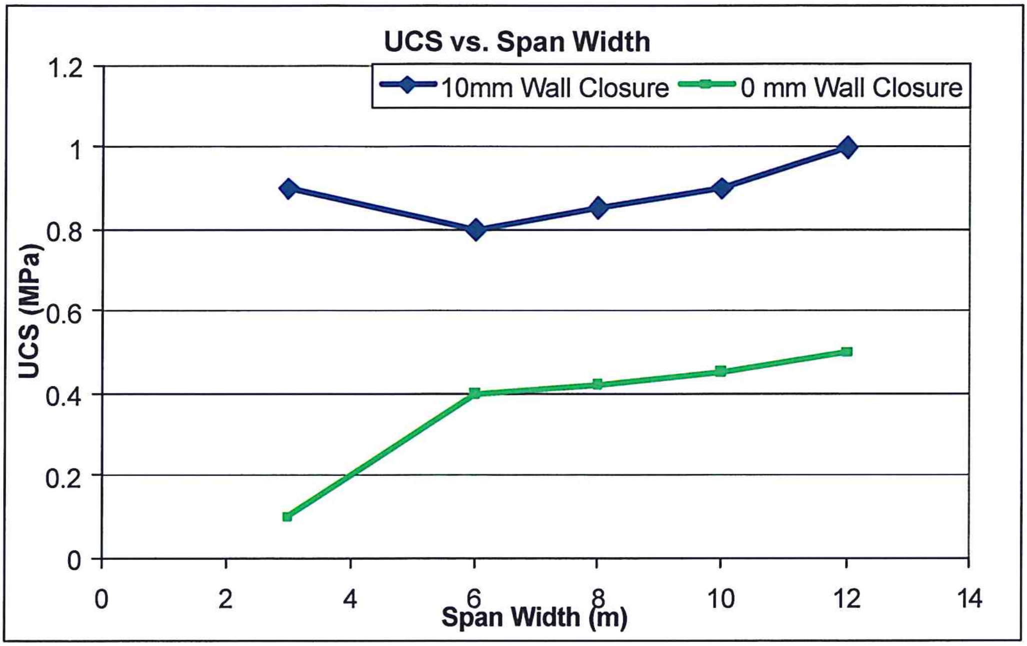

Figure 7: Span vs. Paste UCS

4.3 Future Work

Further research will be done with respect to Figure 6 to determine the effect of wall closure on the UCS of paste required for a safe working environment. This will consists of refining the mesh along the contacts of the paste with the host rock and by measuring total strain across the paste span.

Currently the work is in the theoretical stage and is not calibrated to a specific mine. Future work would be to calibrate the model to a mine currently employing underhand paste fill. From this verification, an optimization of span width vs. UCS of paste can be derived.

4 OTHER OBSERVATIONS

A recent report from NIOSH (Tesarik et al., 2005) discussed the earth pressure cells are significantly affected by the paste cure temperature which can reach 40° C with stresses measured in excess of 69kPa due to the temperature difference. Figure 8 shows the uncorrected values and corrected values based on the temperature of the paste curing. The applied corrections are on top of the corrections supplied by the pressure cell manufacturers, the manufacturer's correction factor applies only to the transducer and not to the entire earth pressure instrument body. By inspecting Figure 7, it can be seen that not only are the uncorrected stress greatly lower than the true stress but the uncorrected stress show a negative trend while the corrected stress show a positive increase of stress. The great difference in uncorrected stress vs. corrected stress is very important in the case of paste as the UCS of paste is very low and that without the proper correction an increase in loading causing failure could go undetected. Furthermore, without the correction over time, the pressure within the fill has a negative trend when in actual fact the fill is being loaded.

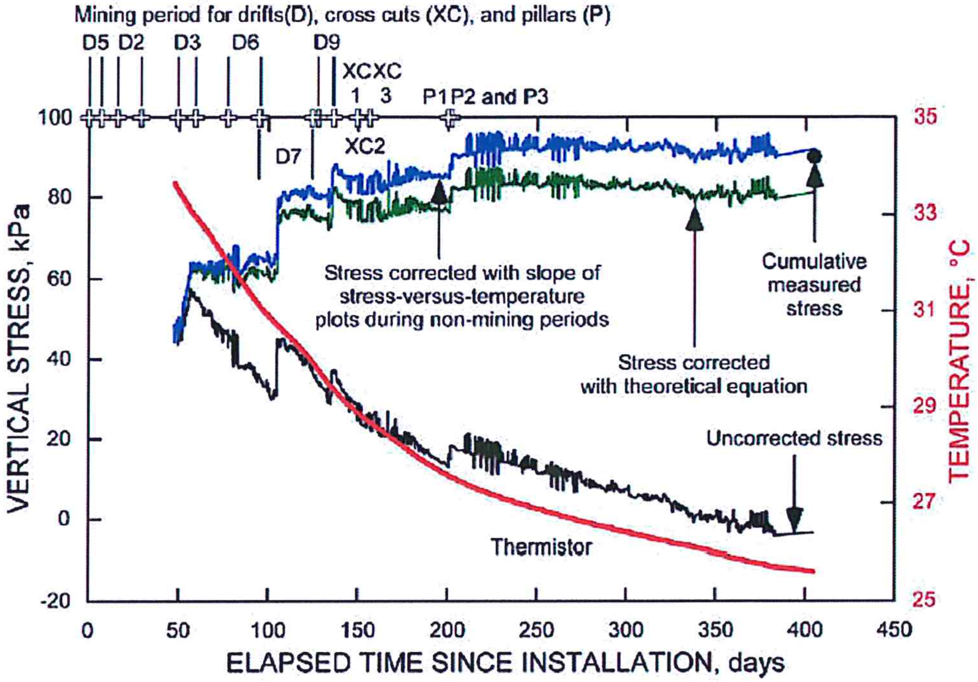

Figure 8: Corrected vs. Uncorrected Earth Pressure Cell readings. Tsearik, (2005).

Current work is underway on lateral loading mechanisms of fill fences. It is the goal of the research to understand the loading mechanisms, determine fill fence construction guidelines and understand paste/ fill fence interaction. The research will consist of field work inspecting current fences, placing instruments on fences to determine loading mechanisms and, hopefully, a destructive test of an instrumented fill fence. Once the field has been completed a model will be constructed based on recorded values and further analysis will be carried out.

6 CONCLUSION

Mining under consolidated fills is becoming competitive to conventional cut and fill mining as increased spans and productivities are realized through reduced placement of ground support and more control on the mine cycle due to working under an engineered back. This requires a thorough understanding of the mechanism of support that one is relying upon which is the consolidated fill immediately above. The fill may be supported in terms of conventional bolts and screen in order to counter "cold joints" that may develop in the fill, account for variability in fill quality control and/or increase the overall factor of safety required due to seismic events in the close proximity. This requires an understanding of the stabilization affect of the consolidated fill and the mine environment that it is placed within. Through the gathering of site data, modelling of behaviour either analytically and/or numerically coupled with observation and measurement one will be able to advance the overall design criteria to provide a safe and cost effective workplace.

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Table 1:

Table 4.—Updated Support Capacity

Rock properties, tonnes		
Bolt strength	Yield strength	Breaking strength
5/8-in mechanical	6.1	10.2
Split-Set (SS 33)	8.5	10.6
Split Set (SS 39)	12.7	14.0
Standard Swellex	NA	11.0
Yielding Swellex	NA	9.5
Super Swellex	NA	22.o
*20-mm rebar, No. 6	12.4	18.5
*22-mm rebar, No. 7	16.o	23
*25-mm rebar, No. 8	20.5	30.8
No. 6 Dywidag	11.9	18.0
No. 7 Dywidag	16.3	24.5
No. 8 Dywidag	21.5	32.3
No. 9 Dywidag	27.2	40.9
No. 10 Dywidag	34.6	52.0
1/2-in cable bolt	15.9	18.8
5/8-in cable bolt	21.6	25.5
1/4 by 4-in strap	25.o	39.0

Note: No. 6 gauge = 6/8-in diameter; No. 7 gauge = 7/8-in diameter; No. 8 gauge = 1-in diameter.

NA = Not applicable.

Screen	Bag strength, tonnes
4- by 4-in welded mesh, 4 gauge	3.6
4- by 4-in welded mesh, 6 gauge	3.3
4- by 4-in welded mesh, 9 gauge	1.9
4- by 2-in welded mesh, 12 gauge	1.4
2-in chain link, 11 gauge, bare metal	2.9
2-in chain link, 11 gauge, galvanized	1.7
2-in chain link, 9 gauge, bare metal	3.7
2-in chain link, 9 gauge, galvanized	3.2

Note: 4 gauge = 0.23-in diameter; 6 gauge = 0.20-in diameter;
9 gauge = 0.16-in diameter; 11 gauge = 0.125-in diameter; 12
gauge = 0.11-in diameter
Shotcrete shear strength = 2 MPa (200 t/m^2)

Bond strength	
Split-Set, hard rock	0.75-1.5 mt per 0.3 m
Split-Set, weak ground	0.25-1.2 mt per 0.3 m
Swellex, hard rock	2.70-4.6 mt per 0.3 m
Swellex, weak rock	3-3.5 mt per 0.3 m
Super Swellex, weak rock	>4 mt per 0.3 m
5/8-in cable bolt, hard rock	26 mt per 1 m
No. 6 rebar, hard rock	18 mt per 0.3 m, ~12-in granite