Integrated engineering design of economic mine backfill systems

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Abstract

In order to maximize the recovery of ore in modern and highly productive mining methods, cemented backfill is normally placed in underground excavations to provide structural support. In order to save on costs, backfill of low cement content is used to fill the mined out excavations. This backfill mass is supported by a sill pillar structure cast from cemented backfill of very high strength. Mining excavations progress under the sill pillar, which must remain stable when exposed and subjected to mine induced stresses. The stability behavior of the sill pillar elements must be carefully studied to provide very effective, safe and economic mining operations. Improper design of these support structures may result in catastrophic fill mass failure, with substantial economic losses. This paper presents a methodology, procedures and engineering standards for backfill design based on three engineering modeling approaches: analytical, centrifuge and numerical modeling. This novel integrated modeling approach represents a powerful design tool in engineering practice.

Keywords: underground mining, backfill, sill pillar, centrifuge modeling.

1 Introduction

Sill pillars are strong structural elements used to support uncemented or low cement content backfill. Conventional sill pillars are cast from cemented sand backfill materials, often underlain by a timber mat; these structures have to be self-supporting when exposed by undercut mining. Improper design would result in failure of the fill mass, and extensive economic losses associated with loss of production and ore dilution, as well as in safety problems.

The traditional stability design of cemented sill pillars is based on experience and on property data from standard static physical model tests such as uniaxial
and triaxial compression. Laboratory test data is normally used to describe or predict the behaviour of sill pillars in empirical, analytical or numerical models. This approach offers inherent limitations and generally results in conservative designs.

To overcome the limitations of traditional designs, a unique and powerful design approach has been developed. In an integrated engineering design methodology, centrifuge physical modeling is combined with analytical and numerical modeling analysis to describe and predict the behavior and potential failure modes of sill pillars. Centrifuge modeling is the primary tool used not only to dynamically test sill pillar performance on a time-dependent basis but also to accommodate the three-dimensional aspects of the problem. Analytical modeling was carried out using limiting equilibrium analysis. Numerical modeling was carried out using FLAC (Fast Lagrangian Analysis of Continua), a powerful two-dimensional elastic plastic-finite difference code. This novel integrated modeling approach represents a powerful design tool in engineering practice.

Application of the design approach is demonstrated from an engineering design study conducted for an underground gold mine, aimed at minimizing backfill binder content and at producing cost efficient backfill recipes for sill pillar construction. The study aimed at establishing the effect of excavation geometry, excavation wall roughness, wall closure, and varying backfill heights on fill stability behaviour, when exposed or undercut during mining. Models were developed to conform with two excavation conditions applied in the mine: excavations 3 m wide, 15 m long, and 30 m high, and excavations 75 m wide, 15 m long, and 40 m high. In all cases the excavation walls were inclined at 75° and smooth, medium-rough and rough rock wall conditions were established for simulating typical excavation boundary modes encountered in the mine. Wall closure strains of 0.9% and 2% were applied to the model. Backfill was prepared at 80% pulp density using unclassified tailings mixed with Type 10 Normal Portland cement (NPC) and Type C fly ash (FA). All sill pillar recipes were prepared at 7% binder content; backfill recipes were prepared at 2.5% binder content. The backfill was cured for 28 days before testing. The results of all three modeling techniques suggested that the 2.5% binder content backfill supported by a 7% binder sill pillar seems to be appropriate for the simulated mining conditions. Application of the engineering design recommendations will result in annual cost savings to the mine in the order of hundreds of thousands of dollars.

2 Engineering modeling design

2.1 Centrifuge modeling

Centrifuge modeling is based on the development of scaled models to study the complex behavior of materials. When designing scaled models, a geometrically equivalent model to the field situation (prototype) is first formed, scaled down at 1/N, using the same materials of identical properties as that being used in the field. The scaling of the prototype must ensure that the model should observe
conditions of similitude such as the same relative inclination, position, proportions, boundary conditions, values and structure. The model is, in essence, a small prototype. The scaled model is then placed in the centrifuge strongbox and accelerated to N gravities. During flight, a centripetal force acceleration is experienced by the model, inducing self-weight forces in the test material. The inertial mass induced stresses at any point in the model will be identical to the gravitational mass induced stresses at any similar point in the prototype, provided that the boundary conditions are identical at all equivalent points [1]. Thus, the model accelerated in the centrifuge experiences analogous behavior as the prototype.

The centrifuge used is a 5.3 m diameter machine driven by a 45 kW hydraulic motor and speed control assembly. The centrifuge modeling facility is located in the Department of Mining Engineering, Queen’s University, Canada. It is designed to rotate up to a maximum speed of 350 rpm and it is capable of exerting up to 300 gravities on scaled models. It is rated at 30g-tonne and can carry payloads of up to 300 kg. An aluminum swing strong box, 0.75 m long, 0.56 m wide and 0.42 m high, is used to house scaled models. The centrifuge has a total of 14 slip rings and a data acquisition system is utilized for model data transmission and storage.

2.2 Analytical modeling

An analytical solution, developed by Mitchell and Roettger [2] on the basis that sill pillars are normally of such length that the problem can be idealized as a two-dimensional plane strain stability problem, has been used. The critical operational stage of a sill pillar after the underlying ore is removed is shown is Figure 1. The sill supports a non-uniform vertical stress, $\sigma_v$, of unknown magnitude, and a lateral closure stress, $\sigma_c$, resulting from rock deformations.

![Figure 1: Sill pillar loadings (after Mitchell and Roettger).](image)
Possible sill pillar failure modes include slippage, caving, shear, rotational shear or flexural failure. For the case application later presented in this paper, slippage and shear failure are of particular importance. Sill pillar slippage failure is likely to occur when,

$$\gamma H = \delta E \frac{\mu}{L^2} \left(\sin^2 \beta - 0.15 \sin^2 \beta\right)$$  \hspace{1cm} (1)$$

where, $\gamma$ is the unit weight of the uncremented fill (kN/m$^3$); $H$ is the height of the uncremented fill (m); $\delta$ is the estimated wall closure; $E$ is the stiffness modulus of fill (kPa); $L$ is the sill pillar width (m); $\mu$ is the coefficient of friction at the fill rock walls interface; $\ell$ is the total contact length between sill and wall rocks (m); and $\beta$ is the excavation wall dip.

Sill pillar caving or crushing occurs if the closure stress, $\sigma_c$, exceeds the compressive strength (UCS) of the cemented sill material. If the expected wall closure, $\delta$, is estimated, then the cemented sill can be engineered, where the lateral prestress is considered the design condition, giving $\sigma_c = E \delta L$. The order of magnitude of the sill pillar stiffness modulus, $E$, can be controlled by the cement content used. Mitchell and Roettger [2] suggested that $\sigma_c$ should be less than 50% of the UCS of the sill pillar.

Wall rock interface conditions play an important role on fill stability. Irregular excavation walls provide sill pillar and wall rock interlocking, thus preventing sill slippage. The rougher the wall conditions the lower the vertical stress due to arching of the fill mass, specially in narrow orebodies. Theoretical considerations, model studies and field measurements indicate that arching between the HW/FW rocks reduces the vertical stress, $\sigma_v$ (kN/m$^3$) to,

$$\sigma_v = \frac{\gamma L}{2K \tan \phi}$$  \hspace{1cm} (2)$$

where, $\gamma$ is the fill unit weight (kN/m$^3$) and $K$ is a constant often taken to be unity. Although it is more likely that the footwall supports a considerable portion of the adjacent stress, it is prudent to assume that this stress acts uniformly on the sill.

For equilibrium, block sliding of the sill due to side shear failure occurs when,

$$w > 2 \left(\frac{\tau_f}{\sin^2 \beta}\right) \left(\frac{d}{L}\right)$$  \hspace{1cm} (3)$$

where, $w$ is the uniform loading (kPa) which include the self-weight of the sill (i.e., $w = \sigma_c + d\gamma$), $d$ is the sill depth (m), $\tau_f$ is the shear strength in the fill-wall rock contact zone (kPa), and $\beta$ is the excavation wall dip.
2.3 Numerical modeling

Numerical modeling was carried out using FLAC, an explicit finite difference program for engineering mechanics computation. The program simulates the behaviour of structures built of soil, rock or other materials that may undergo plastic flow when their yield limits are reached. The materials are represented by elements, or zones, which form a grid that can be adjusted to fit the shape of the object to be modeled. Each element behaves according to a prescribed linear or non-linear stress/strain law in response to the applied forces or boundary restraints. The material can yield and flow, and the grid can deform (in large-strain mode) and move with the material that is represented [3].

FLAC program offers a wide range of capabilities to solve complex problems in mechanics; for this particular application, the program was used to validate the centrifuge modeling work. Two-dimensional elastic-plastic models were constructed and the excavation walls were modeled as fixed boundaries. Because sill pillar stability is dependent on the strength of the fill and fill-wall rock interface, the interface strength was simulated in three different scenarios. The smooth wall rock boundary condition was simulated with very low or no interface strength. The medium-rough wall rock condition was simulated by reducing the interface strength until sill failure would occur. For the rough wall rock condition, the interface strength was considered to be the same as the fill strength, defined by cohesion and friction angle.

3 Validation study

3.1 Centrifuge modeling

Eighteen centrifuge models were designed to represent prototype conditions consisting of backfilled excavations, 3 m and 7.5 m wide and 30 m to 40 m high, detailed in Table 1. All models were prepared using unclassified tailings supplied by an underground gold mine. The sill pillar was constructed using unclassified tailings, mixed with 7% binder (4% of Type 10 NPC and 3 Type C FA) and the overlying fill consisted of fill prepared with 2.5% binder content (1.5% of Type 10 NPC and 1% of Type C FA). Prototype excavations 3 m wide by 15 m long by 30 m high were represented by models 6 cm by 30 cm by 60 cm high, scaled at 50 gravities. Prototype excavations 7.5 m wide by 15 m long by 40 m high were represented by models 11.5 cm by 23 cm by 61.5 cm high, scaled at 65 gravities. All model walls were sloped at 75 degrees from horizontal. Backfill was placed at 80% pulp density and models were cured for 28 days, under controlled conditions. The simulated boundary conditions were smooth (surface of form ply), medium-rough (0.15 m breakage for every 2 m vertical spacing at 50 g) and rough (0.19 m breakage for every 1 m spacing at 65g). The effect of excavation wall closure on sill pillar stability was investigated by applying closure strains of 0.9% and 2% to the model. Minimum closure strains required to prevent sill pillar failure, at different mining conditions, were determined. Figure 2 shows a typical model ready for testing.
Table 1: Centrifuge model data.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Model Dimensions (cm)</th>
<th>Model Sill Pillar Thickness (cm)</th>
<th>Design Gravity (g)</th>
<th>Boundary Conditions</th>
<th>Centrifuge Gravity at Failure</th>
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<tr>
<td>1</td>
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<td>60</td>
<td>50</td>
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<td>50</td>
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<td>65</td>
<td>MRW</td>
<td>-</td>
</tr>
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<td>60</td>
<td>50</td>
<td>RW</td>
<td>-</td>
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<td>61.5</td>
<td>65</td>
<td>RW</td>
<td>-</td>
</tr>
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<td>50</td>
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<tr>
<td>8</td>
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<td>65</td>
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</tr>
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<td>65</td>
<td>SW</td>
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<tr>
<td>17</td>
<td>6 x 30 x 60</td>
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<td>50</td>
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<td>2</td>
</tr>
<tr>
<td>18</td>
<td>11.5 x 23 x 61.5</td>
<td>23</td>
<td>65</td>
<td>SW</td>
<td>2</td>
</tr>
</tbody>
</table>

SW = Smooth Wall, MRW = Medium Rough Wall, RW = Rough Wall.

Figure 2: Model prior to testing and three-dimensional schematic.

After curing, the backfill column was spray painted with white stripes to facilitate visual observation of deformation behavior during flight. Each cured specimen was placed in a model box, and secured in the centrifuge swing strongbox. During test, the model was conditioned for 10-15 minutes at a centrifuge speed of 60 rpm (producing accelerations of 10 g). The centrifuge speed was then increased at a rate of approximately 2-3 rpm per minute until either sill pillar failure or the design scale was achieved.
Models 1-6 were designed with the entire fill column consisting of high cement content backfill. Models 7-18 were designed with sill pillars of different thicknesses, overlain by backfill of low cement content. Excavation wall closure was applied to models 11-18. Design details are shown in Table 1. Figure 3 presents failure mechanisms for models 6, 11 and 16.

![Figure 3: Typical model failures. a. model 6; b. model 11; c. model 16.](image)

Models 1 (representing prototypes 3 m wide, 15 m long, and 30 m high) and 2 (representing prototypes 7.5 m wide, 15 m long, and 40 m high), with smooth walls, exhibited total collapse of the complete fill mass (slip failure) prior to achieving their design scale. Model 1 failed at 39 g, and model 2 at 19 g. The failure process for models 1 and 2 developed suddenly without any warning. The fill self-weight plays a significant effect on the failure mode of models with smooth excavation wall conditions.

Models 3 and 4 represented excavations with medium-rough wall conditions. The models exhibited collapse of the fill mass (slip failure) below the design scale, with fill masses of different heights plunging into the undercut. Model 3 failed at 48 g, slightly below the design scale of 50 g, with a 6 m high fill mass sliding into the undercut. The first indication of failure propagation was the dislodging of the entire fill mass along the hanging wall contact occurring at approximately 45 g. The mechanism was immediately followed by failure of the fill mass caused by a sub-horizontal rupture plane (60–65° from the horizontal), which extended from the hanging wall to the footwall. Failure at the footwall was observed to be a combination of internal failure of the fill mass and failure at the fill-rock wall interface. Model 4 experienced a similar failure mechanism to model 3. It failed at 51 g, with a 15 m high fill mass sliding into the undercut. The failure height of the fill masses, delimited by sub-horizontal rupture planes, can be attributed to strain softening or strain hardening of the fill column.

Models 5 and 6 were constructed with rough excavations walls. Model 5 did not fail at the design gravity. Model 6 could not be brought to failure at the design gravity of 65 g. The acceleration was increased and failure of the model occurred at 69 g (6% above the design gravity). The first indication of failure propagation, with the entire fill mass dislodging from the hanging wall, occurred...
at 52 g. The mechanism was followed by the development of a sub-horizontal rupture plane (60–65° from the horizontal) at the hanging wall, then extending to the footwall, at a height of approximately 31.2 m and leading to the eventual failure of the fill mass. For rough excavation wall conditions, the failure process appears to occur within the fill mass and not at the fill-rock interface.

Models 7 and 8 represented conventional sill pillars, constructed with a thickness equivalent to one excavation width, and overlain by a low binder content fill. The models were constructed with smooth boundary conditions. The two models exhibited collapse of the fill mass at different heights (slip failure) and at design scales well below the required targets. Although the failure of models 7 and 8 occurred at approximately the same gravity of 9 g, different modes were observed. For the narrow excavation (model 7), the failure process happened fast. It was initiated by hanging wall separation at 8.31 g and immediately followed by the occurrence of a sub-horizontal rupture plane and by plunging of 5 m of the fill mass height into the undercut. For the wider excavation (model 8), no initial evidence of failure was observed. Sliding failure of around 4 m of the fill mass height, delimited by a sub-horizontal rupture plane, into the undercut happened slowly; it responded to the increases in acceleration and stress loading.

Models 9 and 10 had equivalent dimensions and boundary conditions to models 7 and 8, respectively but with sill pillars of thickness twice the excavation width. The two models exhibited total collapse of the entire fill mass into the undercut at stresses well below the design scales. The mechanism of failure of model 9 is similar to that of model 8; it responded with increases in acceleration and stress loading. No warning or indication of failure was observed until the initial sliding of the fill mass occurring at around 10 g. Total collapse of the entire fill mass into the undercut occurred at 11 g. The mechanism of failure of model 10 is, to some extent, similar to the failure mode of model 7. The failure happened suddenly. Initial failure was represented by the dislodgement of the overlying backfill along the hanging wall contact at 7.5 g, followed by consolidation of the fill, at approximately 12 g. Sudden failure (slip) of the entire fill mass, then occurred at 19 g.

It is apparent that, although models 7-10 were constructed to simulate smooth boundary conditions, frictional resistance at the hanging wall-footwall contacts seemed to affect the stability of the system. The increase in sill height, and therefore the surface area of the sill pillar fill in contact with the wall, increased the stability of the system. This is evidenced by the gravity at failure for models with increasing sill pillar thickness (models 7-8 versus models 9-10 versus models 1-2).

Models 11 and 12 represented sill pillars, in 3 m and 7.5 m wide excavations and with 0.54 mm (0.9% strain) wall closure. Model 11 had equivalent dimensions and sill pillar thickness to model 7 and model 12 to model 8. Both models failed below their design scales of 50 g and 65 g, respectively. Failure of model 11 occurred at approximately 28 g, with about 7 m height of the fill column collapsing into the undercut. During modeling, consolidation of the overlying backfill was first observed at 12 g, although further increases in
acceleration and stress loading did not result in increased consolidation. At 22 g, a sub-horizontal rupture plane developed in the hanging wall and propagated towards the footwall as the acceleration was increased to 25 g. At this stress level, sliding of the lower 7 m fill occurred, with this fill mass collapsing at 28 g. The failure of model 12 happened without warning with the entire fill column plunging into the undercut at 27 g. No initial evidence of failure was observed, except for some consolidation of the overlying fill occurring at 14 g.

Models 13 and 14 had equivalent dimensions and sill pillar thickness to models 9 and 10, respectively, but underwent excavation wall closure of 0.54 mm. The failure mechanism of model 13 was similar to that observed in model 11. Model failure occurred at 34 g with a 7.5 m high fill mass plummeting into the undercut. The failure was acceleration and stress dependent. At approximately 9 g a sub-horizontal rupture plane occurred 12 m above the undercut. Consolidation of the overlying backfill initiated at 16 g but did not continue with increases in acceleration and stress loading. A second sub-horizontal rupture plane, 7.5 m above the undercut, developed at 32 g and the fill mass started down-sliding until total collapse at 34 g. Model 14 failed at almost the same g level as model 13 with entire fill column plunging into the undercut. The failure mechanism of the model was similar to that of model 12. Consolidation of the overlying backfill mass was first observed to occur at 18 g, followed by the dislodgement of the overlying backfill mass at 22 g and by separation of the sill pillar along the hanging wall contact at 27 g. Sliding of the fill mass then initiated and continued with increases in acceleration and stress loading until total failure at 34 g.

Models 15, 16, 17 and 18 represented sill pillars placed in 3 m wide excavations and with 1.2 mm (2% strain) wall closure. Model 15 had equivalent dimensions and sill pillar thickness to models 7 and 11, model 16 to models 8 and 12, model 17 to models 9 and 13, and model 18 to models 10 and 14. In all cases, except for model 17, sill pillar failure occurred below the design g scales. Failure of model 15 occurred at approximately 22 g, with 2.5 m of the sill dropping into the undercut. The failure mechanism was similar to that observed for model 13. Consolidation of the overlying backfill was first observed at 14 g, followed by the formation of a sub-horizontal rupture plane, approximately 7 m above the undercut, at 17 g. At increased acceleration, a second rupture plane, 2.5 m above the undercut, was formed, followed by sliding failure of the 2.5 m high fill mass into the undercut, at 22 g. Failure of model 16 was of a sudden nature, with the entire fill column plunging into the undercut, at 44 g. No initial evidence of failure was observed prior to failure, except for some consolidation of the overlying backfill, which occurred at 17 g. The failure mechanism was similar to that observed in model 20.

Model 17 failed at 51 g, slightly above the design gravity of 50 g. The failure mechanism was localized caving, extending 5 m within the sill. No forewarning of failure was observed except for some consolidation of the overlying backfill, at 19 g. The failure process formed a stable arc within the fill which extended along the strike length at a slope of 60 to 65° and reached a model height of 10.16 cm (5.2 m prototype height). The arc height was found to be >L/2 (where
L is the strike length of the excavation). The caving failure process can be attributed to the 2% applied excavation wall closure, which was twice that of the strain at peak stress in uniaxial compression strength tests. The magnitude of the wall closure seems to be too high for the particular sill pillar recipe and may have resulted in crushing of the material. The failure mechanism of model 18 was similar to that observed for model 14. The model failed at 34 g, which is below the design scale of 65 g. The failure process was acceleration and stress dependent. Aside from the consolidation of the overlying backfill, which occurred at around 16 g, the first indication of failure was the dislodgement of the backfill and sill pillar, along the hanging wall contact at 32 g. This mechanism was immediately followed by continuous sliding of the fill mass along the hanging wall/footwall-fill contact, until total failure at 34 g.

3.2 Analytical modeling

The limit equilibrium analysis assumed that sill pillars are stable when the factor of safety against failure is more than 1.

Application of the model described by equation (3) was based on a series of parametric calculations as follows. The fill shear strength parameters were obtained from triaxial tests and calculated from, $\tau = c + \sigma_n \tan \phi$, where $c$ is the cohesion, $\sigma_n$ is the normal stress and $\phi$ is the friction angle. The calculated shear strength, $\tau$, of the fill was 551 kPa. The surcharge stress was calculated from equation (2), with $\gamma = 19.37$ kN/m$^3$, $L = 3$ m, 5 m and 7.5 m, $K = 1$, and $\phi = 33^\circ$, to be 44.74 kPa, 74.57 kPa and 111.80 kPa. The values of $d\gamma$ were 774.8 kPa for the 40 m high fill, and 581.1 kPa for the 30 m high fill. The values of the uniform loading, $w$, were 625.84 kPa, 849.37 kPa and 886.60 kPa for 3, 5 and 7.5 m wide excavations, respectively.

Trial calculations showed that sill pillars of varying excavation widths and heights would remain stable with the shear resistance at the rock wall-fill interface being as low as 29% of the fill shear strength. Rock wall interfaces with shear strengths above this threshold value might thus be considered rough. Smooth rock wall conditions are assumed when the factor of safety against failure is less than 1, in which slippage failure occurs due to the self-weight of the fill material. Smooth rock wall interface conditions would thus be characterized by having a shear resistance at the rock wall-fill interface below approximately 27% of the fill shear strength. Validation of the analytical models was highly dependent on results from the centrifuge models; yet this modeling approach may be used to predict the behavior of a sill pillar.

3.3 Numerical modeling

A series of two-dimensional elastic-plastic models were constructed using FLAC in an attempt to validate the physical models. The excavation geometry and equivalent planar representation are illustrated in Figure 4. The fill was discretized into square elements with an area of 0.25 m$^2$. The backfill was modeled as a Mohr-Coulomb material using effective shear strength parameters.
obtained from consolidated-undrained triaxial tests. The model input parameters are shown in Table 2.

Two-dimensional interface elements were placed between the fill and the fixed excavation wall boundaries to represent the fill-rock interface. Both the y-displacement and y-velocity histories were used to evaluate whether the system was reaching equilibrium at each step. Increased steps were continued until active collapse of fill occurred or the model reached equilibrium.

![Figure 4: 3-D mining under fill geometry and equivalent 2-D representation.](image)

**Table 2:** Material properties of the backfill and rock strata.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Backfill</th>
<th>Rock</th>
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<td>Density ($\rho$)</td>
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</tr>
<tr>
<td>Bulk Modulus ($K$)</td>
<td>80 MPa</td>
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<tr>
<td>Shear Modulus ($G$)</td>
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<tr>
<td>Cohesion ($c$)</td>
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<td>Tensile Strength</td>
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</table>

A series of models were developed for 5 m and 7.5 m wide excavations, and considering the three different wall rock conditions. Numerical model results corresponded well with centrifuge models and were very important for design validation purposes.

Figure 5a shows typical displacement vectors in a 5 m wide excavation with rough rock wall conditions. In this case, the model came to equilibrium indicating stable mining conditions. Displacement vectors indicated a rotational response of the sill about the footwall contact. Figure 5.b shows histories of y-displacement at the bottom of the 5 m wide fill, a maximum displacement of approximately 0.0325 m was determined when equilibrium was attained.
Figure 5: Displacement vectors (a) and history of y-displacement (b) in a 5 m wide excavation of rough wall conditions.

4 Conclusions

A sill pillar design methodology based on centrifuge, analytical and numerical and centrifuge modeling was successfully developed. For the materials tested and mining conditions modeled, although the three methods exhibited comparable results, centrifuge modeling seemed to be more effective in treating complex sill pillar and excavation interactions. Model studies have indicated that excavation width, wall roughness and wall closure play an important role in excavation and sill pillar stability. Model studies have also indicated that sill pillar stability develops as a function of frictional effects and of cohesion between particles and fill-rock wall contacts. Stronger arching seems to develop in fills placed in narrow excavations with rough wall conditions than in wider excavations. Undoubtedly, wall closure represents a significant factor in increasing stability; higher closure stresses are required in wider excavations in order to enable stable arching to develop across the fill.

References