Design Considerations for Artificial Support of Open Pit Walls

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ABSTRACT

Design approaches for the artificial support of open pit walls have been reviewed and the main factors affecting support performance have been identified. An example of passive support in a potentially unstable pit wall has been used to demonstrate the importance of slope displacement as part of the support design process.

INTRODUCTION

Artificial support of open pit walls to improve wall stability has now become well accepted, particularly in Australia and Canada. In general, the technique is used to reduce the risk of surface unravelling from exposed batter faces and to provide increased resistance to movement on deeper seated, continuous structures. Current methods of artificial support have evolved from the application of large capacity, pre-tensioned anchors in open pit mines (Seegmiller, 1974, 1975) and the use of untensioned, fully grouted strand cables used for rock reinforcement in underground metalliferous mines (Fuller, 1980, 1983).

When steel strand cables were first introduced to open pits in Australia, the majority were untensioned during installation and were intended to perform as "passive" supports (Rosengren, 1986). However, within the last two years, there has been a move to both pre-tensioned and post-tensioned cable installations. Pre-tensioning is where the support is tensioned as part of the installation. The load is applied by jacking the support against a face plate and the end anchorage is provided by a short grouted length of cable. The pre-tensioned support is intended to develop an active load on the rock mass with an associated increase in pit wall stability. Post-tensioning is normally carried out by applying a small load to a face plate on the slope surface after the support has been fully grouted. Post tensioned supports are usually specified to ensure the firm contact of the face plate with the surface and hence control unravelling around the support.

APPROACHES TO ARTIFICIAL SUPPORT DESIGN

Until recently, most artificial support design was based on engineering judgement (Rosengren et al, 1988). Textbooks on the subject of slope engineering have treated the influence of artificial support as a resistance force which combines with resistance forces developed within the rock mass (e.g. shear resistance on a basal slide plane) to develop a total resistance force. The aim in these designs was to ensure that the total resistance force was greater than the total driving force and therefore to obtain a "factor of safety" greater than 1.0. This approach was demonstrated by Das and Stimpson (1986) in stability analyses of two sliding failure modes. These authors concluded that it may be possible for passive artificial support to improve the stability of small to medium height slopes. In all analyses however, it was assumed that the design load in all support elements would be developed immediately after installation and no account of slope displacement appears to have been considered. Also, this approach assumed that these loads could be totally transferred to the rock mass.

A different approach has been proposed by Dight (1983) and Rosengren (1988). Both methods emphasize the increase in shear resistance developed by supports as displacement occurs within the rock mass. The approach by Dight (1983) involved a design approach based on limiting the movements on critical surfaces so that peak resistance could be mobilized within the rock mass. This meant that support forces could be a function of the amount of displacement permitted in the design instead of some arbitrary design value such as the force required to yield the steel. Design curves for single and twin strand cable were determined from a theoretical model and showed how the support forces changed with displacement. These gave good agreement with experimental results. The curves were used to determine the support forces that could be developed after reaching the movement limit on a critical surface.

Rosengren et al (1988) developed an alternative model to describe the increase in shear force with displacement for various cable support combinations. These authors quote significant benefits in shear resistance by pre-tensioning 50 tonne capacity cable bolts to 25 tonnes. It is important that this result is not taken out of context because it assumes that the pre-tension force is totally transferred to the sliding surface and, that this surface is highly dilatant at small displacements (dilation angle approx. 50°).

Regardless of the model used to describe the shear force-displacement relationship for each support element, it is essential to recognise that

1. passive supports (untensioned during installation) only develop resistance after some movement has occurred on structures intersected by the supports,
2. the amount of movement allowed in a design will influence the resistance developed by the supports, and
3. the load developed by a support element in response to any shear movement will depend on the support type, the angle of the support to the movement direction and whether the shear movement results in any dilatant behaviour.

In the example that follows, it is intended to demonstrate the use of a shear force-displacement model to design passive support. For simplicity, it has been assumed that only one type of passive support is used and that all supports are installed at one angle.

DESIGN EXAMPLE—PASSIVE ARTIFICIAL SUPPORT

Pit wall geometry

In order to illustrate the influence of passive artificial support on stability, a cross-section through a pit wall in Figure 1 has been considered. Siltstone forms the immediate wall, with sandstone occurring deeper into the slope. Bedding is steeply dipping near the surface and is over-turned at depth due to the presence of a fold.

A major fault dipping at 20° into the pit was known to intersect the bedding and be undercut when the pit floor reached a level 75m below surface as illustrated in Figure 1. Clearly, a failure involving sliding on this basal fault could potentially be a problem

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when the fault was undercut. A potential failure extending back to the siltstone-sandstone contact was taken as a worst case for an analysis of wall stability in this area.

The slope consists of 8m wide benches spaced at 15m vertical intervals with batter faces cut at 60'. For the purpose of this example, this geometry is assumed to be uniform for 50m along the wall.

Material properties

The material properties used in stability calculations are given in Table 1.

Stability analysis

Unsupported Wall

A limiting equilibrium stability analysis of the pit slope was conducted using the program GENSAM which is available from the Mining Research Laboratories of CANMET in Ottawa, Canada. GENSAM is a vertical slice analysis method covering circular and non-circular shear surfaces. Using the material properties in Table 1, a linear shear failure criterion and dry slope conditions, the factor of safety was calculated to be *0.95*. This indicates that there would be a high potential for failure involving slip on the fault.

Provided this was known before the pit reached the −75m level and undercut the fault, Figure 1 shows that it is possible to install artificial support from the −45m and −60m levels to intersect the basal fault and potentially increase its shear resistance.

Passive support design

The simplest method of including the effect of passive support in the GENSAM analysis is to treat the additional shear resistance as an increase in cohesion over an appropriate area of influence on the basal plane (Dight, 1983). To calculate the additional resistance, the relationship between shear resistance

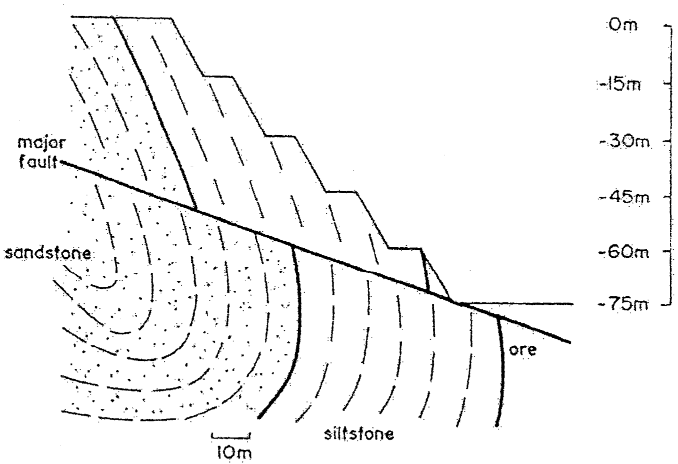


Fig. 1 – Pit Slope Cross Section

TABLE 1 – Material Properties for Stability Calculations

Material	Cohesion (kPa)	Friction Angle (deg)	Density (t/m^3)
Siltstone	100	30	2.0
Siltstone/ Sandstone contact	27	25	–
Fault	29	25	–

versus shear displacement must be known for the support member operating in these ground conditions at the installed angle to the direction of sliding. For this example, it was assumed that all supports would be installed through siltstone in 45° dipping downholes to intersect the fault at 65°. Under these conditions, each support was assumed to develop shear resistance according to the curve shown in Figure 2. Note that this support type develops peak shear resistance at a shear displacement (slip) on the fault of 25mm. The curve in Figure 2 is typical of the shear force-displacement response for a multiple strand cable bolt.

One row of supports

The least expensive artificial support option would be to install one row of support from the −60m bench at a close spacing (say 1.5m) along the bench. Each support would need to be long enough to develop at least a 3m long anchorage in the siltstone below the fault as shown in Figure 3. Once the pit floor was mined to −75m, sliding on the fault would commence and each support would develop an increasing shear resistance according to the curve in Figure 2.

At each displacement value, the total support resistance was calculated and divided by the area of influence on the fault. This represents the increase in cohesion due to the support. The GENSAM analysis was re-run with the increased cohesion value along part of the fault. The effect of the increased shear resistance on the factor of safety can be seen in Figure 4. This shows that there is only a small increase in the factor of safety with shear movement due to the single row of supports. It reaches a maximum value after 25mm shear movement which is where the support develops its maximum shear resistance (Figure 2). However, the factor of safety is still less than 1.0, so the slope would continue to fail, regardless of the influence of the supports. Therefore, if the support design is to be restricted to a single row for any reason (such as restricted access or a lack of available time for support installation), higher capacity supports would be required to achieve a factor of safety greater than 1.0 and give confidence that the design was adequate.

Multiple rows of supports

If there were no operational constraints on the number of rows that could be installed then the next step is to look at suitable access locations and determine their relative priority in terms of cost. The two obvious locations are the benches −40m and −60m but if the decision to install support was made early enough, the pit floor at these levels could also be used. Because the −60m level is closer to the fault, this should have priority over the −45m level. By working off the pit floor at this level, Row 2 could be installed along the batter line and if necessary Row 3 could be "countersunk" as shown in Figure. 5.

The effect of these additional rows of support on the factor of safety is shown in Figure 6.The designer is then faced with the decision as to whether it would be worthwhile installing Row 3. This clearly depends on the level of confidence in the original factor of safety (0.95) and hence the cohesion and friction angle on the fault. If there was a likelihood of the factor of safety being as low as 0.85-0.90, then all three rows would be required.

Another consideration may be the reliability of the support performance data. If this is in doubt, the design factor of safety should be at least 1.1. The installation of a fourth row of supports from the −45m bench would be required to achieve this (see Figure 7).

In theory, shear movement on the fault should cease once the factor of safety reaches 1.0. However, due to the variability in material properties and support performance, this should not be relied upon. It is suggested that a margin above a factor of safety of 1.0 be allowed in design and that this should reflect both the confidence level of the data used in the analysis as well as the

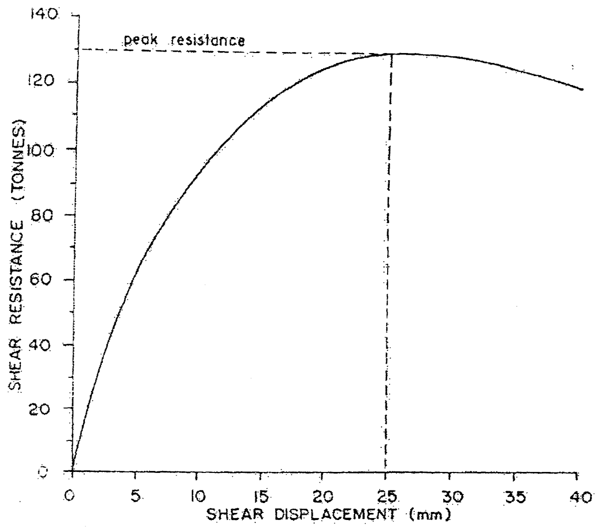


Fig. 2—Shear resistance response for support at 65° to the shear direction.

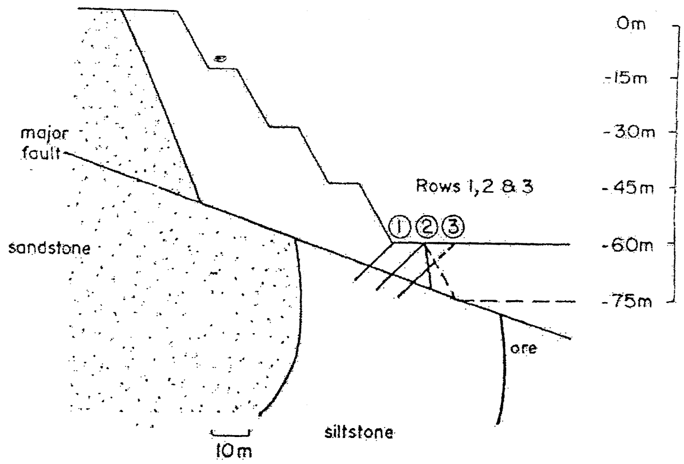


Fig. 5—Three rows of support from −60m level

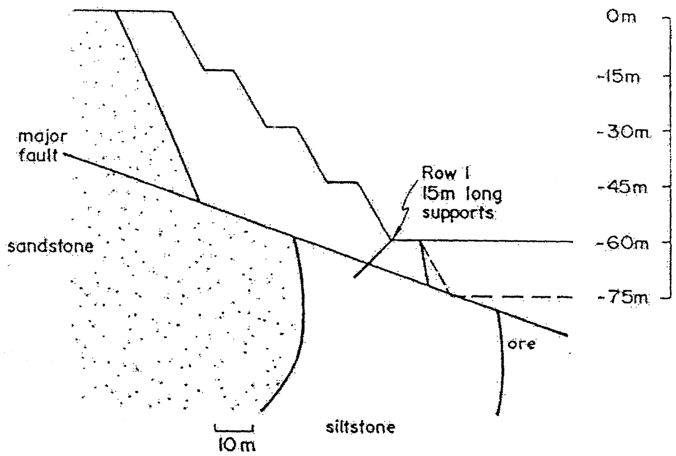


Fig. 3—Row 1 support installed from −60m bench.

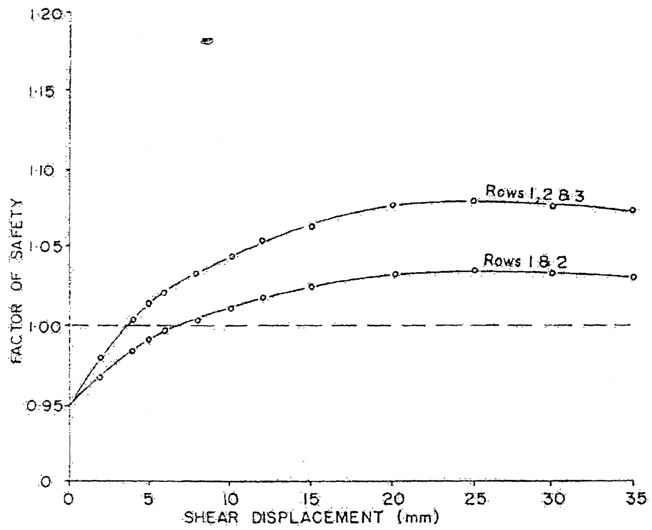


Fig. 6—Change in Factor of Safety with shear displacement

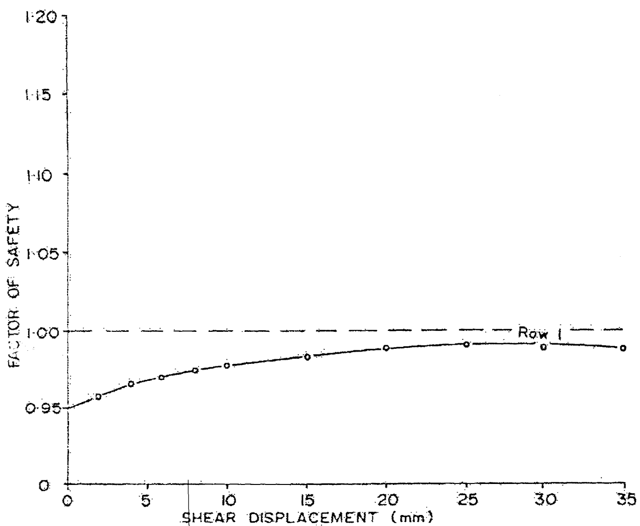
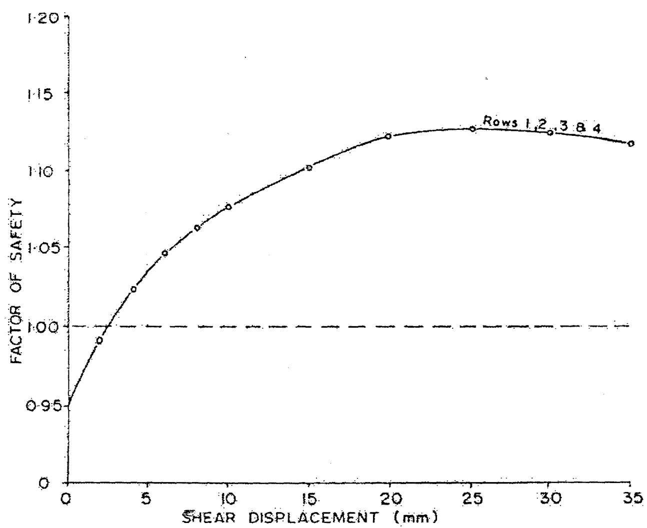


Fig. 4—Change in Factor of Safety with shear displacement



practical and safety implications of a failure. However, the greater the safety factor margin, the greater the support cost and it is essential for the designer to recognize that conservatism is costly.

All analyses conducted on the pit slope in Figure 1 have assumed that the shear strength parameters on the fault remain constant with shear displacement. This is reasonable provided residual strength parameters are used. Should the actual shear strength be greater than the residual value at small displacement, this will add a degree of conservatism to the design.

CONCLUSIONS

The example of passive support design considered in this paper demonstrates that the critical factors in design are:

1. A sound understanding of the mechanism of failure and the strength properties on the failure surfaces.
2. Detailed knowledge of the shear response for the support elements to be used and its variation with angle to the shear direction on the failure surface.
3. Practical constraints such as suitable access from benches or the pit floor to allow support to be installed before stability problems develop.

The example also shows that passive supports affect pit slope stability only when some movement occurs.

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