WEAK ROCK MASS DESIGN FOR UNDERGROUND MINING OPERATIONS

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ABSTRACT

A major focus of ground control research presently being conducted by the Geomechanics Group at the University of British Columbia, Canada, in conjunction with the National Institute for Occupational Safety and Health's (NIOSH) Spokane Research Laboratory, is the development of design guidelines for underground mining within weak rock masses. The study expands upon the span design curve for man-entry operations and the stability graph for nonentry operations developed at UBC by extending the application to weak rock masses. The original database has been augmented by weak rock mass information from mines throughout the United States, Canada, Australia, Indonesia, and Europe. The common factor in all of these mines is the presence of a weak back and/or walls. This paper expands on the North American database and how the design curves have been employed at mining operations throughout the world. The definition of a weak rock mass for this study has been defined as having an RMR_{76} under 45% and/or a Q-value under 1.0.

INTRODUCTION

A comparative analysis by the Mine Safety and Health Administration for Nevada gold mine operations for 1990–2004 (Figure 1) shows that the number of injuries from roof falls in 13 Nevada underground gold mines ranged from a low of 8 in 1990 to a high of 28 in both 1995 and 1997 [Hoch 2001]. This high injury rate was the prime motive for the initial study by NIOSH. The goal was to address the extremely difficult ground conditions associated with mining in a weak rock mass and provide mine operators with a database that could lead to a better understanding of the failure mechanism associated with mining within a weak rock mass. The database summarized in this paper is composed of seven [Potvin 1988] mines in Nevada and several others, which are summarized in Table 1.

A weak rock mass upon review of site conditions observed was identified as having an RMR_{76} less than 45% and/or a rock mass quality (Q) under 1.0. These values are largely described by Barton [2002] and Bieniawski [1976] as being "very poor" and "fair," respectively, and are shown schematically in Figure 2.

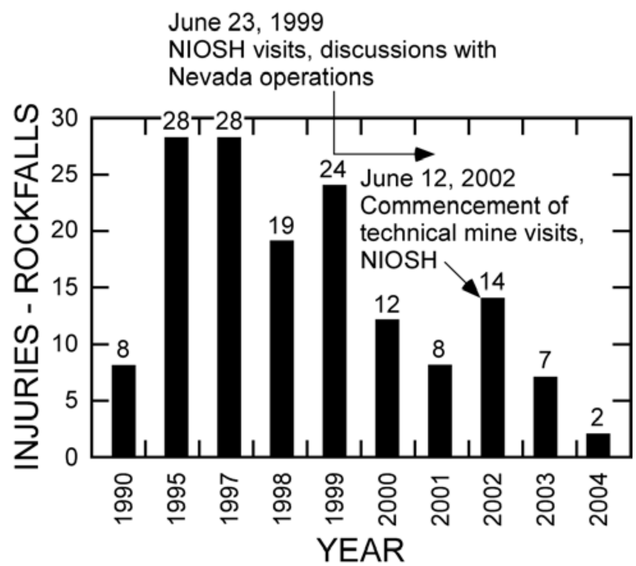

Figure 1.—Injuries from rock falls in Nevada underground mining operations [Hoch 2001].

Table 1.—Weak rock mass database (RMR_{76} < 45%, Q < 1.0)

East Carlin Mine (Newmont)	Nevada.
Deep Post Mine (Newmont)	Nevada.
Midas Mine (Newmont)	Nevada.
Rodeo Mine (Barrick)	Nevada.
Turquoise Ridge Mine (Barrick) ..	Nevada.
Getchell Mine (Barrick)................	Nevada.
Murray Mine (Queenstake)	Nevada.
SSX Mine (Queenstake)	Nevada.
Nye Operation (Stillwater)	Montana.
Eskay Creek Mine (Barrick)	British Columbia, Canada.
Eagle Point Mine (Cameco)	Saskatchewan, Canada.
Quinsam Mine (Hillsborough)......	British Columbia, Canada.
Kencana Mine (Newcrest)...........	Halmahera, Indonesia.

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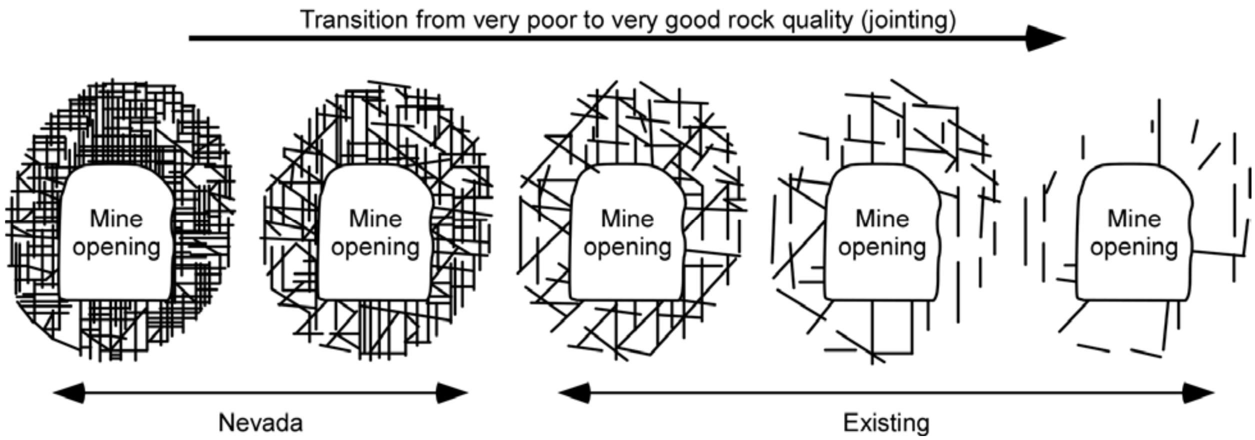

Figure 2.—Schematic showing transition of weak rock mass to stronger and existing databases.

The mining methods practiced in Nevada are largely longhole mining, underhand cut-and-fill, and cut-and-fill. The underhand mining method relies upon an engineered back composed of rock/paste fill and is not addressed within this study, as mining under/adjacent to weak rock masses is largely negated. Underground mining methods as practiced in Nevada dictated which specific databases and stope design curves NIOSH would focus upon. Rock mass values were calculated during mine visits and varied from an RMR_{76} high of 70% to a low of 16% in gold-bearing fault gouge. Several rock mass design curves developed by the Geomechanics Group at the University of British Columbia (UBC) [Pakalnis 2002] are available, but they were not thought to be relevant to the mining methods employed within the weak ground of Nevada gold mines and therefore not augmented.

Research began with visits to Nevada operators in June 1999 to address concerns and determine where NIOSH would be able to assist. The first technical site visit was on June 12, 2002, and initial data were collected (Figure 1). The major objectives were to obtain information on weak rock masses and incorporate this information into existing design curves [Lang 1994] for back spans of manned entries and a stability graph [Clark and Pakalnis 1997] for longhole wall design for nonentry operations. The distribution of the original databases was based on Canadian mining data, as summarized in Figure 3, and shows the lack of data for weak rock masses.

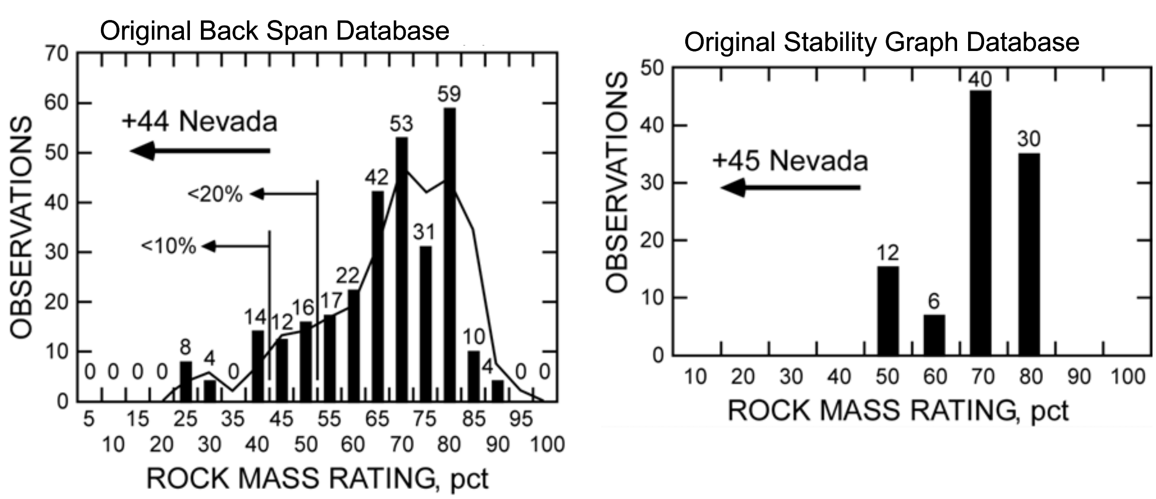

Figure 3.—Distribution of the original database for back span *(left)* and stability graph *(right)*.

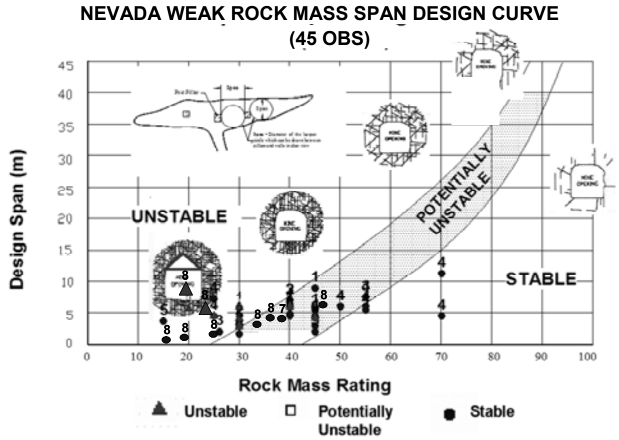

Figure 4.—Critical span curve augmented with Nevada operations (Table 1) (45 observations).

SPAN DESIGN, MAN ENTRY

The initial span curve was developed by the UBC Geomechanics Group to evaluate back stability in cut-and-fill mines. It consists of two straight lines that divide a graph into three zones: stable, potentially unstable, and unstable. The database for this graph initially consisted of 172 data points from the Detour Lake Mine of Placer Dome, Inc., Ontario, Canada, with most of the points having RMR values in excess of 60% [Lang 1994]. The database was expanded to 292 observations in the year 2000 with case histories from an additional six mines [Wang et al. 2000]. The successful use of empirical design techniques is based upon interpolation rather than extrapolation. Thus, a decision was made to develop a database for a critical span curve in weak rock masses. The term "critical span" refers to the largest circle that can be drawn within the boundaries of the excavation when seen in plan view (Figure 4).

The term "design span" refers to spans that have no support and/or spans incorporating a limited amount of local support (e.g., pattern bolting in which 1.8-m-long mechanical bolts are installed on a 1.2- by 1.2-m pattern). Local support is deemed as support used to confine blocks that may be loose or that might open or fall because of subsequent mining in surrounding areas. The Nevada study added an additional 44 observations to the span design curve as shown in Figure 4 and summarized by Brady et al. [2005], of which 35 had an RMR_{76} less than 45%.

The span design curve is used throughout North America. Three operations and their database are summarized in this paper: Stillwater Mining Co.'s Nye Operation in Montana, Barrick Gold, Inc.'s Eskay Creek in British Columbia, Canada, and Cameco, Ltd.'s Eagle Point Mine operation in Saskatchewan, Canada, with the design span curves shown in Figures 5–7, respectively. Of note in Figure 6, where Eskay Creek has established guidelines for support based on RMR as well as if conventional overhand mining versus underhand will be practiced as a function of rock mass and span. Figure 7 shows two data points within the unstable zone. In this area of the operations, these zones were observed to previously cave and required either increased support (dead weight) and/or mining employing "short rounds" in order to ensure stable conditions for subsequent mining.

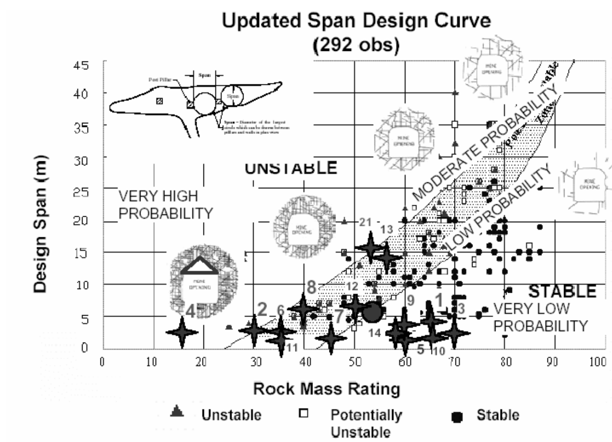

Figure 5.—Critical span curve at Stillwater Mining's Nye Operation in Montana (292 observations).

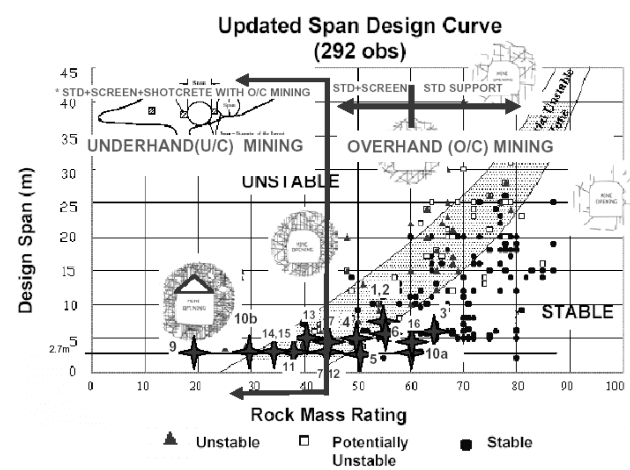

Figure 6.—Critical span curve at Barrick Gold, Inc.'s Eskay Creek Mine in British Columbia, Canada (292 observations).

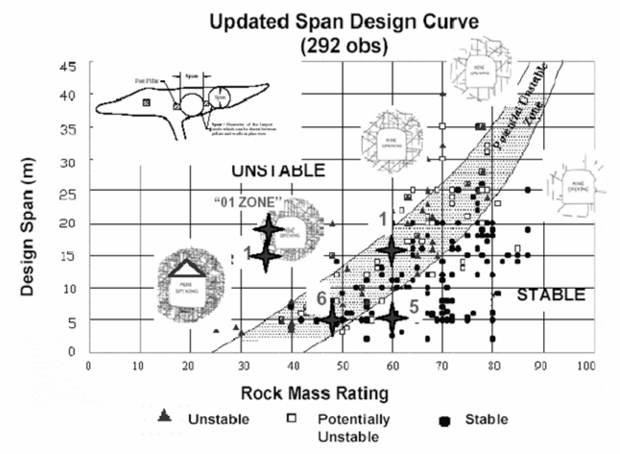

Figure 7.—Critical span curve at Cameco, Ltd.'s Eagle Point Mine, Saskatchewan, Canada (292 observations).

A brief description of the use of the critical span curve is presented; more detail is outlined by Pakalnis [2002].

Excavation stability is classified into three categories; each category is further divided into three subcategories.

1. Stable excavation (S)
   a) No uncontrolled falls of ground have occurred.
   b) No movement of the back has been observed.
   c) No extraordinary support measures have been employed.
2. Potentially unstable excavation
   a) Extra ground support has been installed to prevent falls of ground.
   b) Movement has occurred in the back.
   c) Increased frequency of ground movement has been observed.
3. Unstable excavation (U)
   a) Area has collapsed.
   b) Depth of failure of the back is 0.5 times the span (in the absence of major structures). Within a weak rock mass, the depth of failure has been noted as 1 times the span and sometimes even greater.
   c) Limited local support was not effective in maintaining stability.

A minus-10 correction factor is applied to the final RMR when evaluating rock with shallow dipping or flat joints. However, the applicability of this factor in weak ground is being reassessed because of its amorphous nature. Where discrete ground wedges have been identified, they must be supported before employing the critical span curve. Stability is generally defined in terms of short-term stability because the database is based largely on stoping methods that, by their nature, are of short duration. Movement of the back greater than 1 mm within a 24-hr period has also been defined as a critical amount of movement for safe access [Pakalnis 2002]. This value is also being addressed for weak rock masses as it applies to the initial database identified in Figure 4. This critical value may be much greater than 1 mm.

STABILITY GRAPH METHOD: NONENTRY

The original stability method for open-stope design was based largely on Canadian operations and was proposed by Matthews et al. [1981], modified by Potvin [1988], and updated by Nickson [1992]. In all instances, stability was qualitatively assessed as being either stable, potentially unstable, or caved. Recent research at UBC has augmented the stability graph by using stope surveys in which cavity monitoring systems were employed [Clark and Pakalnis 1997]. This research has enabled the amount of dilution to be quantified. A parameter termed the "equivalent linear overbreak/slough" (ELOS) was introduced by Clark and Pakalnis [1997] and was used to express volumetric measurements of overbreak as an average depth over an entire stope surface. This has resulted in a design curve as shown in Figure 8.

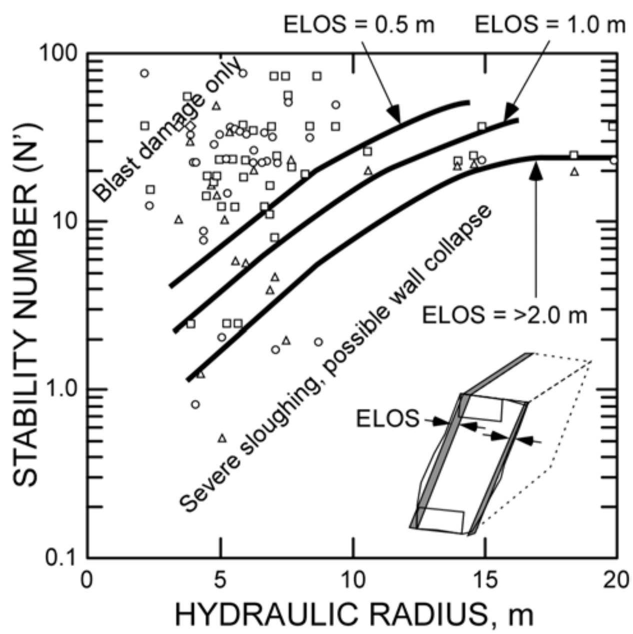

Figure 8.—Stability graph, after Clark and Pakalnis [1997].

A limited number of observations existed for RMR values under 45% (Figure 3). An additional 45 data points were added on the stability graph—nonentry from Nevada operations having an RMR under 45%. In addition, Mine 4 (Table 1) reflects more than 338 observations that have been averaged to reflect the design points as discussed by Brady et al. [2005]. The stability graph relates hydraulic radius of the stope wall to empirical estimates of overbreak slough. Hydraulic radius is defined as the surface area of an opening divided by perimeter of the exposed wall being analyzed.

Equation 1 was used for calculating parameters for the database shown in Figure 8:

$$N' = Q' * A * B * C \tag{1}$$

where N' = modified stability number;

Q' = modified Norwegian Geotechnical Institute (NGI) rock quality index [Unal 1983] where the stress reduction factor and joint water reduction factor are equal to 1, as they are accounted for separately within the analysis;

A = stress factor equal to 1.0 due to relaxed hanging wall;

B = rock defect factor. This value results from parallel jointing and the amorphous state of the weak rock mass being set to 0.2 and 0.3, respectively [Brady et al. 2005];

and C = stope orientation factor as defined in Figures 5–7, i.e., $C = 8 - 6 \times \cos \varphi$ (dip of hanging wall).

An initial observation from Figure 9 is that the classical design curves (ELOS) as shown in Figure 6 are inaccurate at low N′ and hydraulic radius values. If hydraulic radius is kept below 3.5 m in a weak rock mass, the ELOS value should remain under 1 m. It seems that a hydraulic radius under 3 m would not result in ELOS values much greater than 1 m. This result is being further evaluated.

SUPPORT CAPACITY GUIDELINES

The development of support capacity guidelines is critical to the overall success of the mining method selected in terms of ensuring a safe workplace. Ground support in weak rock presents special challenges. Underdesign can lead to costly failures, whereas overdesign can lead to high costs for unneeded ground support. Figure 10 depicts a classic wedge failure controlled by structure. It is critical to design for the dead weight of the wedge in terms of the breaking load of the support, as well as the bond strength associated with embedment length [Brady et al. 2005].

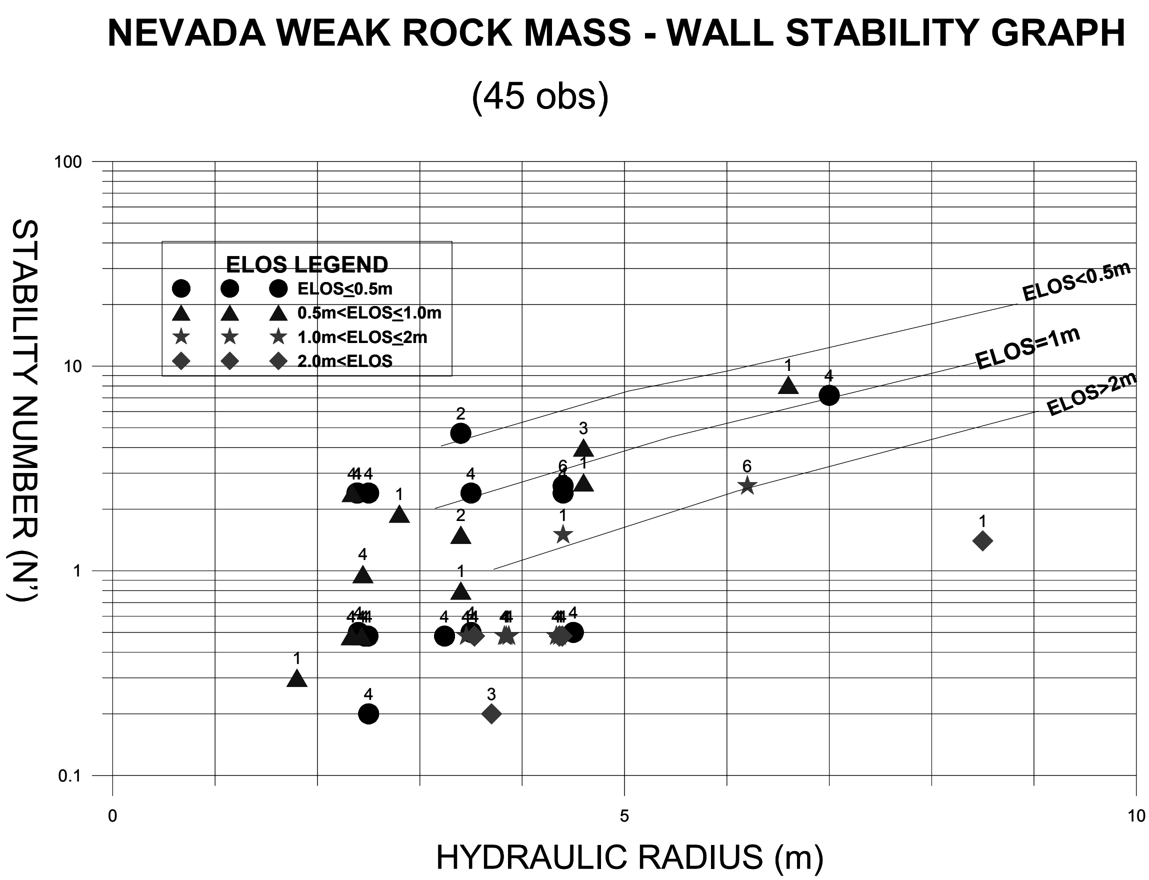

Figure 9.—Wall stability graph as developed for Nevada operations (45 observations).

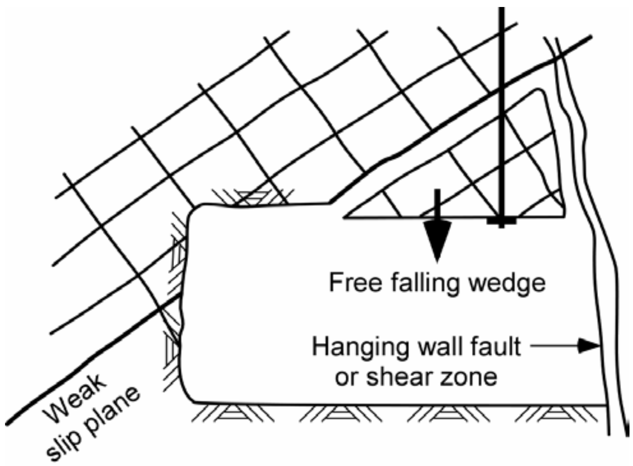

Figure 10.—Structurally controlled wedge.

More than 400,000 Split Set [Brady et al. 2005] friction bolts are used in Nevada mines as primary support. Friction bolts are particularly useful in fissile, buckling, or sheared ground where it is difficult to secure a point anchor. Caution must be used with this method of primary support because of the low bond strength between broken rock and the bolt and because of the susceptibility of the bolt to corrosion. In Mine 4, Split Set bolts had a life of 6 months because of corrosion resulting from acidic ground conditions. An analysis of the performance of friction bolts in mines with weak rock (as determined by RMR) needed to be conducted. With one exception, Nevada mines use 39-mm Split Set bolts (the exception uses 46-mm Split Set bolts). Mines in Canada, however, use 33-mm Split Set bolts. Canadian mines generally use these bolts only in the walls and not in the back. The 46-mm bolts are common in Indonesian operations (Newcrest's Kencana Mine) and Australian underground mines.

Data points gathered from several pull tests in weak rock were plotted as shown in Figure 11, with bond strengths (SS39) shown for Mine 4 in Figure 12. The graph shows a strong trend between RMR and bond strength.

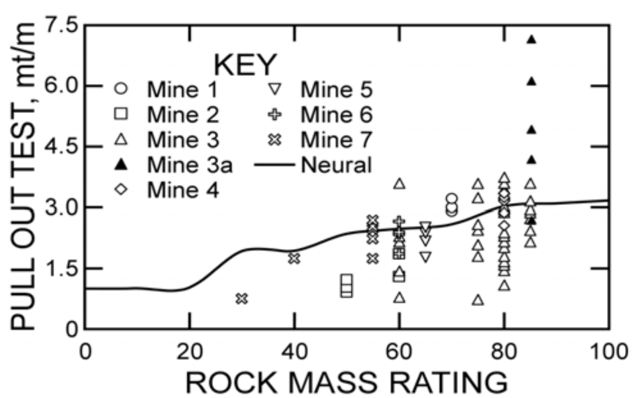

Figure 11.—Pullout load versus RMR for SS39.

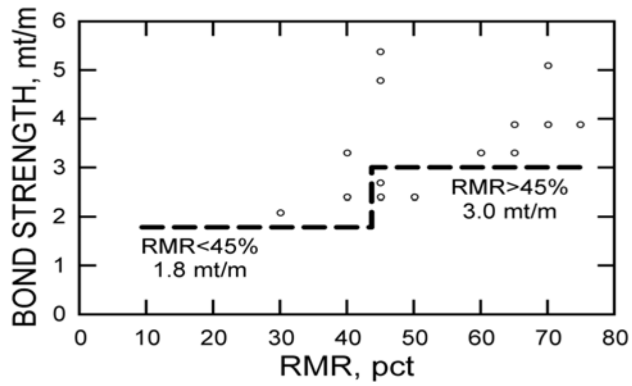

Figure 12.—Pullout load versus RMR for SS39 (Mine 4).

Variability in test results shows the difficulty in assessing overall support for a given heading. Thus, it is important that mines develop a database with respect to the support used so they can design for variable ground conditions. Factors critical to design, such as bond strength, hole size, support type, bond length, and RMR, should be recorded in order to determine where they lie on the design curve. Table 2 shows the design bond strengths determined through field testing at Newcrest's Kencana Mine in Indonesia. Design values differed for grouted versus ungrouted split sets [Villaescusa and Wright 1997] and enabled one to assess the benefit with respect to alternative support, such as Swellex bolts. The Pm12 Swellex bolt provided 8.6 t/m of bond for RMR_{76} values ranging from 25% to 55%. These values compare similarly to design field strengths observed at Barrick's Eskay Creek Mine in British Columbia, Canada.

Table 2.—Bond strength SS46/Pm12: Kencana Mine, Indonesia

Type	RMR_{76}	Bond strength (SS46 mm) Ungrouted (tonnes/m)	Grouted (tonnes/m)
I…………	RMR > 55%	4.1	7.2
II………..	35% < RMR ≤ 55%	2.6	5.8
III……….	25% ≤ RMR ≤ 35%	1.5	4.4

Swellex Pm12 bond strength = 8.6 t/m (RMR 25%–55%).

Table 3 summarizes the updated support table incorporating the weak mass pull-test results conducted at operations throughout this database (Table 1).

Table 3.—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

MINING OPERATIONS GUIDELINES

With weak rock masses, blast control is critical to ensure that the weak rock mass is not further disturbed from overblasting. Guidelines for blasting based on RMR values for Queenstake's SSX Mine and Barrick's Goldstrike operation are summarized in Figures 13–14, respectively. In addition, the length of round pulled is related to the rock mass rating for a 5-m supported back span (Figure 13). For example, when the RMR is 15%–20%, only a 1.2-m (4-ft) advance is possible; otherwise, failure of the unsupported back will result. Spiling is recommended at these RMR thresholds. Figure 14 shows the degree of loading of a development round at Barrick's Goldstrike Mine for a 5-m by 5-m heading with respect to the RMR.

DEPTH OF FAILURE

Recent parametric analysis employing discrete-element methods of numerical modeling of discontinuous materials were employed by MacLaughlin et al. [2005], whereby

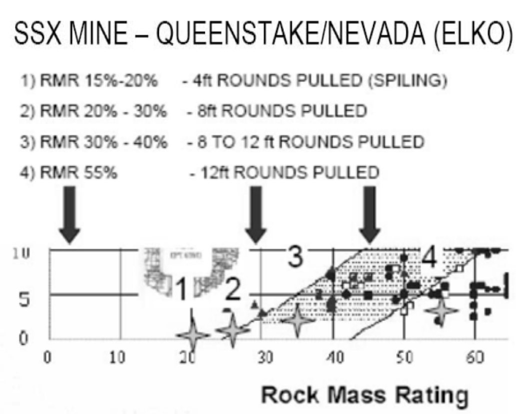

Figure 13.—RMR versus round advance at Queenstake's SSX Mine.

BARRICK GOLDSTRIKE
RMR>45% LOAD FULL FACE (5m X 5m) ~115kg-160kg EXPLOSIVE
RMR 25%-45% LOAD HALF FACE LOADED ~40kg EXPLOSIVE AND UNDER
RMR<25% LIFTERS ONLY LOADED - OR FREE DIG/MUCK
TYPICAL ADVANCE IS 1.5 to 2.8m DEPENDING UPON RMR

Figure 14.—Loading of 5-m by 5-m face at Barrick's Goldstrike Mine.

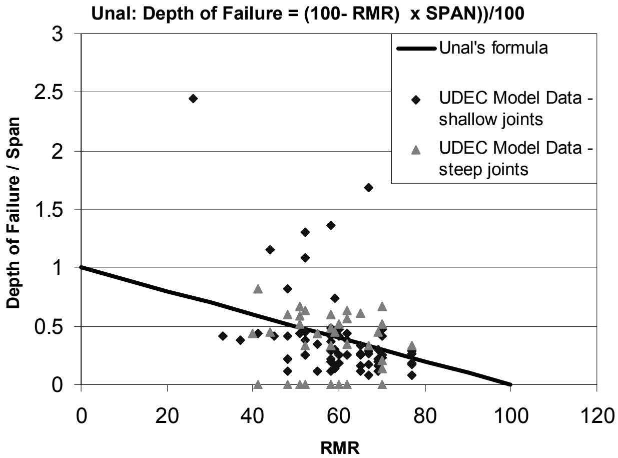

Figure 15.—Modeled depth of failure as a function of RMR [Villaescusa and Wright 1997].

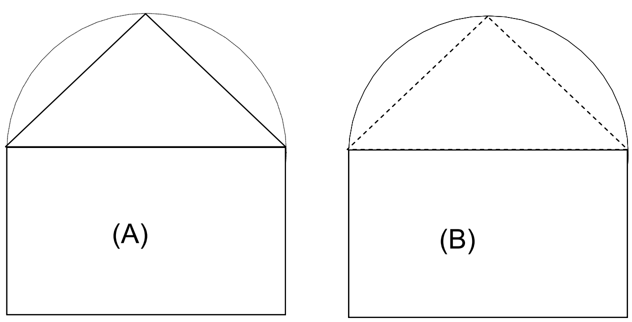

Figure 16.—Illustration of reduction of wedge volume due to arching of back. Depth of failure has been approximated by 0.5 times the span. Note arch has removed the potential for dead-weight failure.

UDEC[5] was employed to determine the depth of failure for a characterized rock mass ranging in RMR_{76} from 26% to 77% with variable joint orientations. The results of the modeling showed that the failure mode was highly correlated with both the RMR value and joint condition. The depth of failure was largely found to be a function of drift geometry, with depth largely explained by the Unal [1983] relationship, as shown in Figure 15. This study also showed that the depth of failure was largely confined to 0.5 times the span for RMR values modeled.

In practice [Unal 1983] it was found that for weak rock masses, arching the back dramatically increased the overall stability for a given span. This is schematically shown in Figure 16, whereby the potential wedge volume is significantly decreased by employing an arched back and the effectiveness of the applied support increased as a greater length of bolt passes beyond the failure plane. This has been shown to be a major contributing factor to the overall stability of mines operating within weak rock masses.

[5]Universal Distinct Element Code.

CONCLUSIONS

The NIOSH Spokane Research Laboratory and the UBC Geomechanics Group are focusing on developing safe and cost-effective underground design guidelines for weak rock masses having an RMR in the range of 15%–45%. Weak ground conditions, ground support, and mining methods used in several North American underground mines were observed. The RMR_{76} values were calculated to update the span design graph and the stability graph to weak rock mass conditions. The greatest benefit is the implementation of these design relationships and methodologies at the participating mines, as their relevance and ability to predict design requirements have been shown to be workable, safe, and cost-effective.

Variability in field conditions showed the difficulty in assessing overall support for a given heading. It is imperative that mines develop their own databases based on the type of support used in their mines so that unexpected ground conditions can be analyzed and mine stability predicted. The results from augmented design curves and pullout tests are presented in the hope that they will aid mine professionals in their task of designing a safe workplace. A systematic approach allows an operator to understand overall failure mechanisms and resultant loads that could affect the system. This approach would allow an engineer to develop an optimal support strategy for the mining method employed.

This work would not have been possible without the partnership between NIOSH, the UBC Geomechanics Group, and North American mining company personnel. This continued partnership is critical to the development of safe and cost-effective mine design strategies. Figure 1 shows that since the inception of the team approach and resultant collaboration, injury statistics have declined dramatically in Nevada. This decline may be a result of many factors; however, it is clear that this approach is important and relevant to mine operations.

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