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Ground Support Design Curves: Squeezing Ground in Nevada

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ABSTRACT: Researchers at Spokane Mining Research Division of the National Institute for Occupational Safety and Health (NIOSH) are working with mine personnel to aid in providing a safe working environment through the development of empirical ground support design tools. Weak rockmass conditions at underground gold mines in Nevada, which can result in hazardous mining conditions, pose challenges to installing an appropriate amount of ground support. The design of ground support based on dead-weight roof failure is commonplace and appropriate in preventing catastrophic collapse; however, these methods are not particularly effective at predicting the required support capacity to withstand overstressed, squeezing ground. This paper presents empirically derived ground support guidelines for squeezing ground conditions in familiar terms of a factor of safety (FOS) against dead-weight failure versus Rock Mass Rating (RMR). These guidelines are intended to aid the engineer in the design of ground support capacity appropriate to the rockmass conditions and intended use of the excavation, thereby improving safety in underground mines.

1. INTRODUCTION

Weak rockmass conditions at underground gold mines in Nevada can pose challenges to installing an appropriate amount of ground support. This can result in: (1) excessive excavation deformation (squeeze) damaging excavation support and reducing its capacity, and (2) hazardous groundfalls or fall of ground (FOG). This paper presents empirically derived ground support guidelines for squeezing ground conditions in Nevada. Support guidelines are presented in terms of a factor of safety (FOS) against dead-weight failure versus W-RMR (Warren et al., 2016), a modified Rock Mass Rating (RMR) (Bieniawski 1976, 1989) for weak rockmass.

The design of ground support based on dead-weight roof failure is commonplace and appropriate to prevent catastrophic collapse; however, these methods are not particularly effective at predicting the required support capacity needed to withstand overstressed, squeezing ground. Experience has shown that estimating an appropriate amount of ground support in overstressed squeezing ground is not trivial, and few design tools have proven effective at estimating the appropriate amount of ground support required for underground gold mines in Nevada.

Researchers at NIOSH's Spokane Mining Research Division are attempting to increase the reliability and confidence in installed ground support by presenting a support design tool capable of estimating the appropriate amount of ground support based on the operation's accepted risk of ground squeeze and required rehabilitation.

Previous empirical work on weak rock support design in Nevada includes:

- Drifting in Very Poor Rock–Experience and Analysis (Mathis and Page, 1995)
- Design in Weak Rock Masses: Nevada Underground Mining Operations (Brady et al., 2005)
- Update of Span Design Curve for Weak Rock Masses (Ouchi et al., 2008), and Empirical Design of Span Openings in Weak Rock (Ouchi 2008)
- Empirical Ground Support Recommendations for Underground Gold Mines in Nevada (Warren and Kallu, 2016)

The ground support design curves presented in this paper build on the Ouchi et al., (2008) and Warren and Kallu (2016), and incorporate case-history data from both studies.

2. MINING CONDITIONS

Long-hole stoping and underhand cut-and-fill are the primary extraction methods used in underground gold mines located in Nevada. Mining depths range from surface to approximately 3,300 ft with plans to go deeper. Gold host rocks are generally deep ocean sedimentary sequences of limestone and shale (Cline et al., 2005). Late-stage extensional tectonics (Basin and Range Provence) has influenced stress conditions resulting in an estimated horizontal-to-vertical stress ratio (k) from 0.6 to 0.75. (Personal communication, 2014).

2.1. Geotechnical Conditions

Production zones within ore bodies are typically composed of intensely fractured and highly altered rock with RMR less than 45 (Sandbak and Rai, 2013; Sun and Chen, 2013). Access drifts and infrastructure often intersect faults and altered material of varying thickness and geotechnical quality ranging from blocky competent rock to saturated soil-like material.

The host rock at underground mines in Nevada is generally of fair quality (RMR 41-60) unless faulted or highly altered, however, ore bodies are generally comprised of poor rock (RMR 21-40). Very poor ground conditions (RMR<20) are usually encountered as fault gouge or highly argillic altered dikes of limited thickness. Figure 1 shows the distribution of geotechnical conditions documented in this study in terms of the Weak Rock Mass Rating (W-RMR), a modified RMR system discussed below.

2.2. Development of W-RMR for weak rock masses

Very poor geotechnical conditions in Nevada (RMR <20) represent the transition between rock and soil, and the RMR system is known to be difficult to apply and unreliable in this range (Hoek et al., 1995; Hoek and Marinos, 2007; Mathis and Page, 1995). Two major issues with RMR are that, as described in Bieniawski (1976, and 1989), the lowest RMR rating is eight, and RMR is insensitive to changes in RQD <25%, and fracture spacing < 2.5 in. To address issues with RMR in the lower range, the Weak Rock Mass Rating (W-RMR) was developed by incorporating engineering soil classification and W-RMR calculation formulas that are roughly equivalent to traditional RMR calculation methods in moderate to good quality rock, but increase the sensitivity of RMR in the lower range (0-25) (Warren et al. 2016). W-RMR is described in more detail in Section 5.

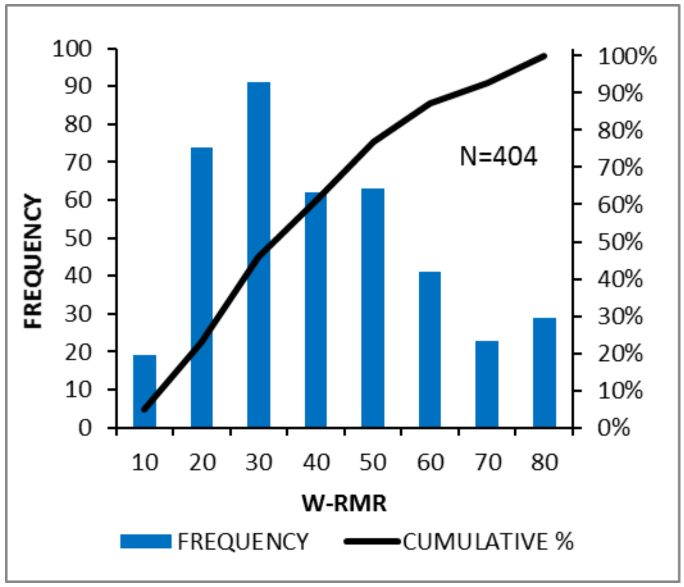

Figure 1. Distribution of W-RMR ground conditions documented in this study.

3. CASE HISTORY DATA COLLECTION

Roughly 400 ground control case studies were collected from eight operating underground gold mines in Nevada from approximately 2006 through 2014. These datasets were originally developed by Ouchi (2008) and Warren (2016) and combined for this study. The ground control case-history database was compiled from a combination of the authors' direct documentation of case-histories, and operators' geotechnical records including:

- Operator-documented case histories
- Groundfall investigations
- Rehabilitation documentation
- Ground support design plans
- Discussion with operators

A majority of the case-study database was developed by the authors' firsthand by visiting locations underground and documenting geotechnical conditions, installed ground support, and stability evaluation. The documented case-history parameters are presented in Table 1.

Table 1: Case-history Data Documentation Parameters

Data Collected	**Description**
Installed ground support	Rock bolt type, spacing, capacity, surface support
Geotechnical conditions	RMR, Q, GSI
Excavation parameters	Dimensions, depth, use, age
Stability of excavation	Stable, squeeze, groundfall.

3.1. *Excavation instability – squeezing ground*

Squeezing ground and groundfalls result in hazardous conditions to miners and are disruptive to operations. Hazards associated with groundfalls are obvious; however, hazards associated with squeezing ground can be more subtle and develop over time, as described below.

Squeezing ground is ground that is considered prone to time-dependent closure of an underground excavation. Squeezing ground results in broken-up shotcrete, bolt heads pulling through mesh and shotcrete (Figure 2), bolt heads popping off (Figure 3), shoulder shear (Figure 4), and buckling shotcrete ribs (Figure 5). As the support system is deformed and damaged, its capacity to support ground is reduced, increasing the likelihood of a groundfall. In addition, the ground support system itself can become a groundfall hazard, particularly when shotcrete is used. The result, if not dealt with in a timely manner, can become a hazard to miners.

Figure 2. Blot head pulled through welded wire mesh.

Figure 3. Bolt head beginning stages of popping off.

Figure 4. Shoulder shear.

Figure 5. Buckled shotcrete rib.

Excavation squeeze closure rate can range from essentially zero to inches per month, and total convergence has been measured over 2 ft (>13%) in a 16-ft-wide heading. Excavations that experience squeeze are rehabilitated to the extent possible until the excavation is deemed “lost” because it cannot be used for its intended purpose. Under these circumstances, the excavation is typically backfilled with cemented rockfill (CRF), and a decision may be made to attempt to drive an excavation next to the CRF plug.

The purpose and expected life span of an excavation determines the tolerance to squeeze rates and total convergence. Mine-related infrastructure, such as shops, crusher chambers, and backfill plants are particularly sensitive to convergence as these facilities are crucial to the operation of the mine and typically challenging to rehabilitate. In some cases, large fixed equipment is structurally attached to the ribs and/or back. Convergence that causes deformation of the fixed equipment superstructure can cause severe operational challenges (Figure 6). Relocation of infrastructure is costly and sometimes not possible within the financial constraints of an operation.

Figure 6. Bent structural beam in critical infrastructure caused by convergence (Internal mine memo).

Production levels, however, are typically open for a short duration (months) and therefore can withstand higher squeeze rates and total convergence. These areas are much easier to rehabilitate as mobile equipment can be moved out of the area with the possibility of temporarily relocating mining activities while rehabilitation takes place.

The design capacity of a ground support system, and its ability to resist or accommodate squeezing, is influenced by the tolerance of an excavation to squeezing conditions and the length of time it will be open. It is the authors' intent in this paper to estimate the amount of ground support appropriate to the requirements of the excavation.

4. GROUND SUPPORT CAPACITY

The capacity of a ground support system needs to be quantifiable and comparable to other ground support systems for meaningful ground support design. The term "support pressure" is common language in the tunneling industry and is defined as the equivalent resisting normal stress the support system is capable of applying to the excavation boundary as described in Barton et al. (1974) and in Hoek and Brown (1980). Estimated support pressure requirements for underground mines in Nevada are presented in Warren and Kallu (2016) and Warren (2016).

However, many underground mine operators in Nevada use a ground support Factor of Safety (FOS) against deadweight groundfall calculation, developed by Pakalnis (2008) for ground support design and for comparative purposes. This paper takes the FOS deadweight design concept and applies it to the prevention of squeezing ground excavations.

4.1. Factor of Safety (FOS) Deadweight

The deadweight FOS design method was proposed by Pakalnis (2008), where the depth of failure is approximated as a wedge whose height is equal to one half the span, with support capacity defined by the breaking and bond strength of a plated bolt. The FOS calculation is presented in equation 1 below.

$$FOS = \frac{Capacity\ of\ Support\ System}{Weight\ of\ Wedge} \qquad (1)$$

Factors contributing to FOS calculation are presented in Table 2.

Table 2: Factors Affecting FOS Calculation

Parameter	**Comment**
Bolt capacity	Manufacturer defined
Bolt bond strength	User estimated (Table 3) or measured
In-plane bolt spacing	User defined
Out-of-plane bolt spacing	User defined
Excavation span	Operational requirement
Ground unit weight	Measured or assumed

The calculation is essentially a kinematic wedge analysis (Figure 7) with support capacity defined as the value of the sum of the individual bolt capacities, which is either the breaking strength or the bond strength of the bolts passing through the failure surface, whichever is less. The wedge weight is determined by calculating the area of the wedge (height = ½ span), assuming an in-plane thickness equal to the in-plane bolt spacing and unit weight.

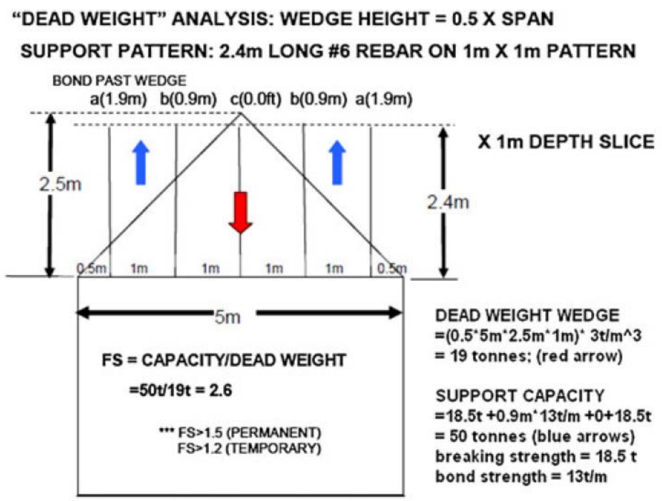

Figure 7. Deadweight factor of safety (FOS) calculation. From Pakalnis (2008)

The deadweight FOS method is easily understood, effective, and has a robust history for designing against groundfalls. It is also useful for this study in that it considers the most important variables in support capacity and reduces them to a single number, simplifying the empirical analysis.

4.2. Bond Strength

Bolt bond strength is an important variable in the FOS, and a reasonable value must be estimated for meaningful calculations. Pakalnis (2014) published some suggested values for the bond strength of various bolts, and the assumed bond strengths used in this study are provided in Table 3. Note that mine-specific bolt pull testing should be conducted to verify actual bond strengths in the ground. Some strength properties for common bolts are provided in Table 4.

Table 3: Suggested Bond Strength for Various Bolts. After Pakalnis (2014)

Bolt Type	**Bond Strength (ton/ft)**	**Assumed Bond Strength in the Study (ton/ft)**
39–mm split set weak ground RMR <45	0.3–1.2	1
39–mm split set hard ground RMR>55	0.8–1.7	
Standard Swellex weak ground RMR<45	2.7–4.6	4
Standard Swellex hard ground RMR>55	3–5	
Cable bolt weak ground RMR<45	8.1	9
Cable bolt hard ground RMR>55	9.4	
#6 Rebar weak ground RMR<45	4–4.7	12
#6 Rebar hard ground RMR>55	19.8	

Table 4: Suggested Bolt Strength Properties. After Pakalnis (2014)

Bolt Type	**Yield Strength (ton)**	***Breaking Strength (ton)**
5/8" mechanical	6.7	11.2
Split set (SS-33)	9.4	11.7
Split set (SS-39)	14	15.4
Standard Swellex	NA	12.1
Yielding Swellex	NA	10.5
Super Swellex	NA	24.2
#6 rebar	13.7	20.4
#7 rebar	17.6	25.3
#8 rebar	22.6	33.9
#6 Dywidag	13.1	19.8
#7 Dywidag	17.9	27
#8 Dywidag	23.7	35.6
#9 Dywidag	30	45.1
#10 Dywidag	38.1	57.3
½-inch cable	17.5	20.7
5/8-inch cable	23.8	28.1
¼-inch x 4-ft strap	27.5	28.1

*Breaking strength was assumed for analysis in this study.

5. CALCULATION OF RMR (W-RMR)

The Rock Mass Rating (RMR) system is widely used at underground gold mines in Nevada. It is also used as input into empirical design tools because of its ease of use and proven reliability in moderate to good quality rockmass. However, it is widely recognized that the RMR system lacks sensitivity and is difficult to apply to very weak rock (Hoek et al., 1995; Hoek and Marinos, 2007; Mathis and Page, 1995). This reduces the reliability of empirical design methods in weak rock, resulting in uncertainty in excavation stability or in expensive and oversupported ground control measures.

To increase the applicability of the RMR system and associated empirical systems, Warren et al. (2016) developed RMR calculation formulas tailored to weak rock (W-RMR) that are roughly equivalent to traditional RMR calculation methods in moderate to good quality rock.

Below RMR = 30, the W-RMR calculation procedure is recommended because it increases RMR sensitivity in weak ground, and allows RMR to be zero. Above RMR = 30, RMR and W-RMR are essentially equal. W-RMR is the classification procedure used in developing support guidelines in this paper and, for the reader's convenience, the equations are presented below.

Fracture Frequency Rating =

$$-5.383 \ln\left(\frac{Fractures}{Foot}\right) + 17.149 \qquad (2)$$

Where fractures/foot (FF) is 0.1<FF<24)

$$\textbf{\textit{RQD Rating}} = 0.2\ (\text{RQD\%}) \tag{3}$$

Where RQD = Rock Quality Designation (Deere and Deere, 1988)

$$\textbf{\textit{Intact Rock Strength Rating}} =$$

$$0.3772(R)^2 + 0.3147(R) \tag{4}$$

Where R = ISRM R strength index according to Brown (1981) and described in Appendix 1.

For convenience, **groundwater** and **joint condition rating** (JCR) are also described in Appendix 2.

The above formulas were those used to calculate W-RMR in the empirical design chart presented in this paper. The formulas are visually depicted in Appendix 2 for the reader's convenience.

5.1. Soil Classification for Very Weak Rock Mass

The use of the Unified Soil Classification System (USCS) (ASTM D2488, 2009) for classifications of very weak rock masses is discussed by Warren et al. (2016). In that paper, a USCS/RMR_{76} correlation is presented to expedite and standardize W-RMR calculation in soil-like conditions.

The USCS-RMR_{76} correlation (W-RMR_{76}) was developed to assign a numeric RMR value to USCS soil classified material for the purpose of developing RMR-based support design guidelines for the wide range of geotechnical conditions in Nevada. It also puts a number on a USCS classified material, which aids in statistical calculation of RMR data. For reference, descriptions of USCS group symbols are presented in Appendix 2.

6. DATA ANALYSES

Roughly 400 case studies were collected for this analysis. In much of the database, different ground support is used in the back versus the rib of the excavation because of differences in stress conditions or differences in geotechnical conditions. In addition, back span and rib height are often different dimensions for operational reasons. For these reasons, the case-study database was split into rib and back case histories, with roughly 400 case histories in each category.

6.1. FOS Calculation

Rock Mass Rating and Deadweight FOS were calculated in spreadsheets using W-RMR formulas presented in Section 5, the deadweight FOS equation in Section 4.1, and the bond strength assumptions presented in Table 3. For simplification and consistency, rib FOS calculations are calculated the same as in the excavation roof/back.

In the author's experience, hand calculations are typically used to calculate deadweight FOS. To expedite the calculation of some 800 FOS calculations, the authors developed a Visual Basic Code function in Microsoft Excel capable of drag-and-drop FOS calculations of the database entries.

6.2. Stability Definitions

Empirical analysis of the database requires a stability classification for each case history. For the purpose of this paper, excavation stability and degree of instability is defined in Table 5.

Table 5: Excavation Squeeze Stability Definitions

Excavation Stability	**Description**
Stable	No sign of instability
Minor squeezing	Cracks in shotcrete to 1/8 inch or bolt plates clearly taking load.
Moderate squeezing	Prolific shotcrete cracking to 1.5 inches, bolt heads starting to pull through screen, or shotcrete being pushed off bolt heads.
Severe squeezing	Shotcrete shelling creating a fall-of-ground hazard (FOG), bolt heads popping off. Shotcrete ribs or back buckling.
Groundfall	Failure above installed rock bolts

6.3. Data Visualization

For user simplification, the one-half span Factor of Safety (FOS) was plotted against W-RMR, and the data series were sorted to reflect the stability of each excavation represented by different data point shapes and colors (Figures 8 and 9). Excavation stability is defined as stable, squeezing, or groundfall and is described in Table 5, and stability zone descriptions are presented in Table 6.

Table 6: Stability Zone Descriptions

Zone Definition	**Squeeze Potential**	**Example**
Stable Zone	Low potential for squeeze	Long-term and critical infrastructure. Areas difficult to rehabilitate
Transition Zone	Likely to Squeeze over time	Short-term and production headings. Areas that can be rehabilitated easily
Unstable Zone	Unreliable	High probability of instability, including bolt heads popping off or groundfall occurring

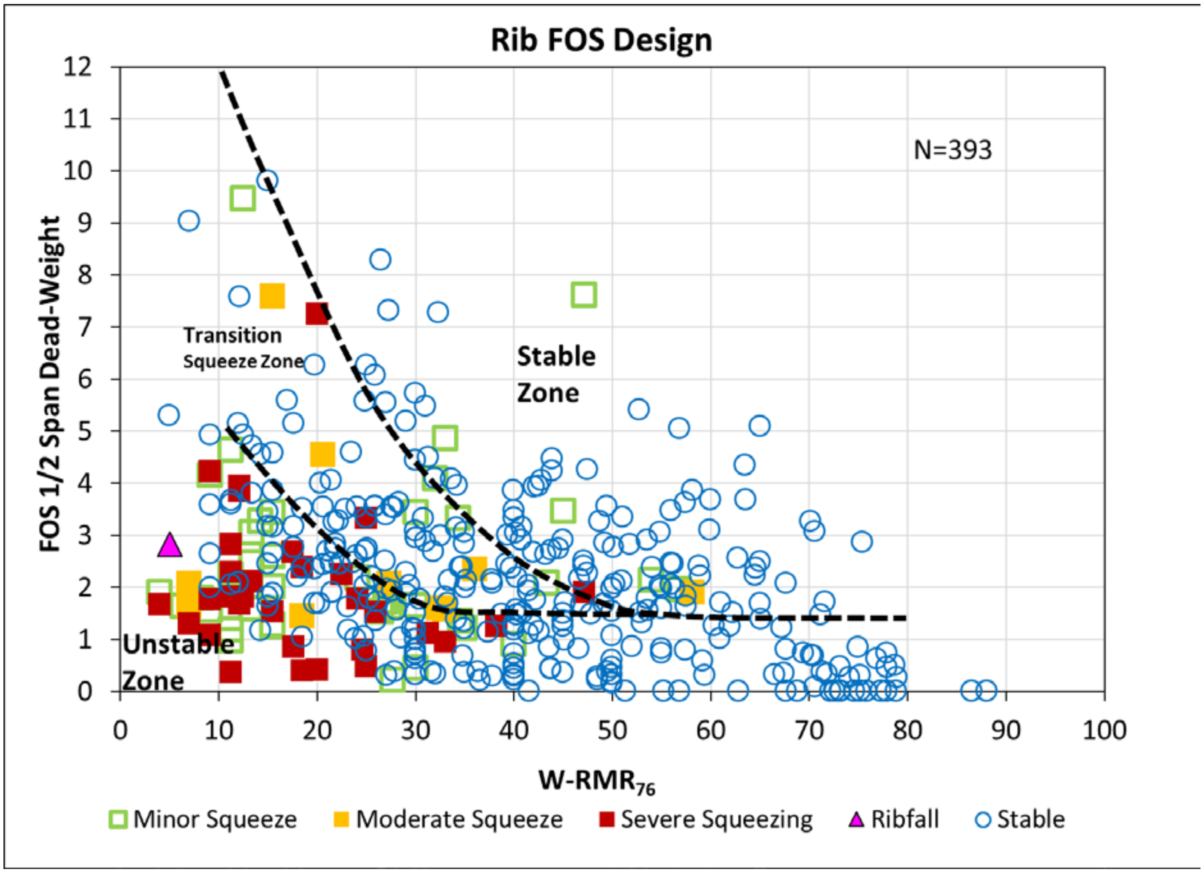

Figure 8. Rib Squeeze Case-Study Database and Design Curves

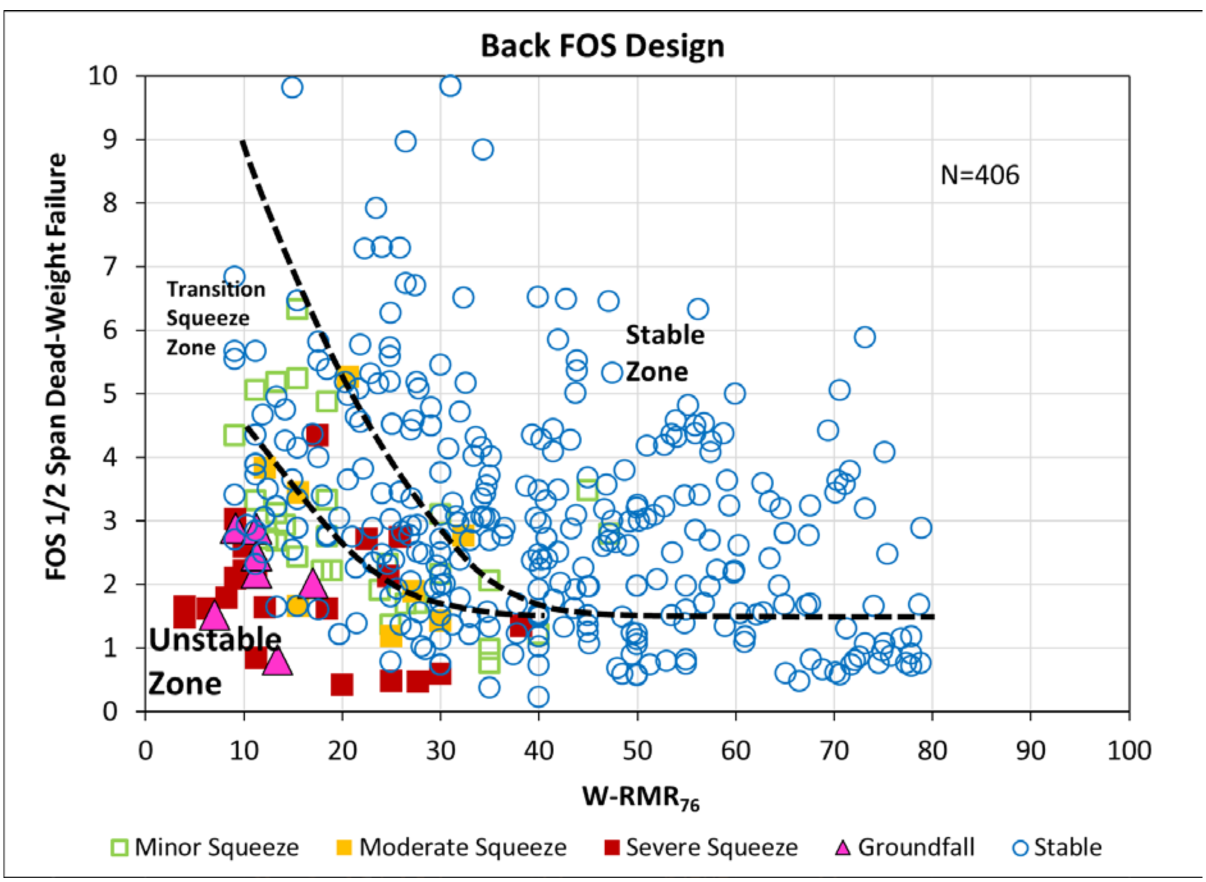

Figure 9. Back Squeeze Case-Study Database and Design Curves

6.4. *Stability Zone Discussion*

Figures 8 and 9 clearly indicate that low W-RMR combined with low FOS result in unstable ground, including squeeze and ground/rib fall situations. Higher FOS and W-RMR result in more stable excavations.

The dashed lines drawn on Figures 8 and 9 represent the authors' opinion of the boundaries between stable and unstable design zones. The Stable and Unstable Zone design lines are above 90% and 80% of the instabilities for the rib design graph (Figure 8). For back design (Figure 9) the stable and unstable design lines are above 90% and 70% of the instabilities.

The **Stable Zone** represents a high degree of certainty that the excavation will not squeeze. Design in this zone is applicable to critical infrastructure and development, such as shops, crusher chambers, and backfill plants and main haulage routes.

The **Transition Zone** represents a higher risk of squeeze compared to the Stable Zone, particularly over time. Design in this zone requires judgement and tolerance to the risk of excavation squeeze, and is appropriate for temporary mine and production openings that can withstand some amount of squeeze and/or can be rehabilitated easily. Design in the upper and lower end of the transition results in higher and lower degrees of certainty that the excavation will remain serviceable for the duration of its use.

Secondary and tertiary development deserves special consideration. These excavations tend to be open for a shorter duration than main ramps and haulages, but longer than production zones. They are typically located near or within the ore body and are, hence, more subject to mining-induced stress that can increase the likelihood of squeezing. Depending on the mine plan, these excavations may be open for several years as panels are mined. Therefore, it may make sense to design these drifts with a higher FOS than production zones, but less than permanent openings.

The **Unstable Zone** represents a high probability of squeeze or groundfall. Ground support design within this zone, particularly in the back, is not advised.

6.5. *Use of the Design Curves*

The main purpose of the design curves (Figure 8 and 9) is to aid the engineer in selecting a ground support FOS that is appropriate to the ground conditions and intended use of the excavation. The steps recommended to arrive at an appropriate ground support design FOS are:

1. Determine or estimate the design geotechnical conditions of the excavation (W-RMR) from Appendix 1 (attached) if in the field, or from equations 2 through 4 if using spreadsheet calculations.
2. Locate the design W-RMR on the X-axis of Figure 8 or 9, depending on whether the design application is in the rib or back.
3. Determine the appropriate design zone (stable or transition) for the ground support design on Figures 8 or 9. This is based tolerance risk of squeeze of the excavation, and user discretion discussed in Section 6.4.
4. Move up from the W-RMR to the user identified stability design zone.
5. From the stability zone, move directly across to the ½ span FOS. This number will be the target FOS stability design.
6. Determine the necessary values for input support parameters of the FOS calculation (Equation 1) from Tables 2-4) or bolt manufacture specifications.
7. Adjust input parameters in FOS calculation until a suitable FOS is achieved. These parameters will obviously have to fall within operational constraints, such as capabilities of available crews, available bolts, excavation dimension requirements, and practical, achievable bolt spacing.

The design curves presented as Figures 8 and 9 are not intended as precision design tools. Rather, they are data-supported guidelines to aid the engineer in making support design decisions.

The use of spreadsheet or GUI programs in the calculation of the FOS will obviously expedite the calculation and provide opportunity for sensitivity analyses and what-if scenarios.

8. SUPPORT DESIGN IN SQUEEZING GROUND

The selection and design of ground support to withstand squeezing is not straightforward and is often a trial-and-error process at underground mines. Guidelines presented in this paper are a starting point, however, local experience should evolve and refine more site specific design guidelines.

6.6. *Bolt Pattern Design*

Bolt spacing requirements can be determined through calculations described in Section 4, and if primary and secondary support are used, FOS calculations are additive. Figures 10 and 11 are intended to aid the user in determining a practical bolting pattern based on common bolt types and lengths used in Nevada. Note that site specific design charts can be made for any combination of available rock bolts or span designs.

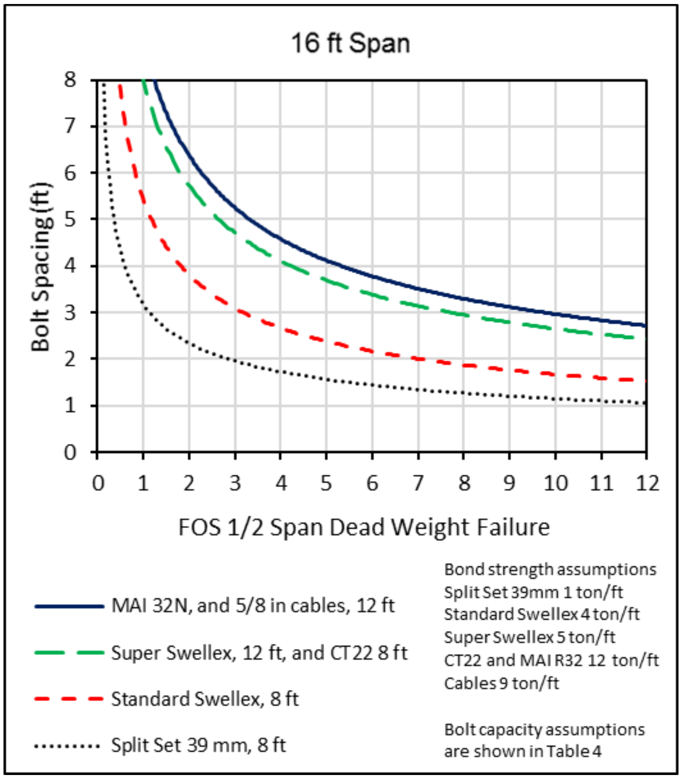

Figure 10. Bolt spacing design curves for a 16 ft span

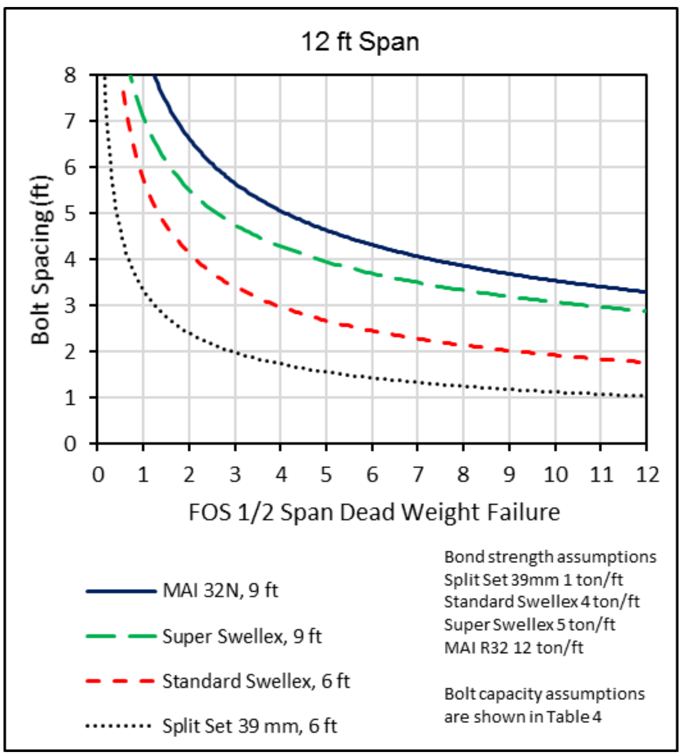

Figure 11. Bolt spacing design curve for a 12 ft span

6.7. Soft versus Stiff Support

Choosing a support strategy will depend on the intended excavation use. Slow squeezing is generally tolerable in temporary mine openings, as long as the support system remains intact and hazards are not created. If the goal is to maximize the life of an opening, higher strain capacity ground support such as inflatable (Swellex type) rock bolts is appropriate. These bolts can withstand relatively high strain (10%) before breaking. The idea is to mine and backfill before the squeezing becomes a serious problem.

However, if the opening is for long-term or critical infrastructure, the squeezing must be stopped or drastically reduced. Stiffer, high-capacity bolts, such as Minova MAI, DSI CT, and Rebar bolts, may provide the appropriate stiffness to resist squeezing. However, these bolts can only withstand a few percent strain before breaking and may provide less warning compared to softer support systems.

Shotcrete arches or ribs reinforced with 00-gauge-wire mesh straps or formed mesh channels have been successful at controlling squeezing at some locations. Variations on this theme are observed, but the basic idea is to mimic the support mechanism of a shotcrete-encapsulated lattice girder by constructing wire-mesh-reinforced shotcrete-covered arches every 2–4 ft. Detailed drawings of the system are beyond the scope of this paper, but an example is available in Marlow and Mikula (2013). The tunnel industry is employing a similar design of “reinforced ribs of sprayed concrete” or RRS using rebar. Details of this design are presented in NGI (2013) and are available online. Also note that shotcrete-reinforced lattice girders on a 4-ft spacing have been successful for spans <20 ft in Nevada.

6.8. Surface Support and Sequencing

From an operational point of view, single pass ground support is ideal; i.e. install mesh and bolts only for several rounds, then apply shotcrete to several advances at once reducing cycle times. However, recall that for the support system to interact, bolts must go over the shotcrete giving the bolt heads a solid platform to resist squeezing. Shotcrete over mesh and bolts in weak ground often results in the shotcrete being pushed in off the bolt/ mesh as the ground deforms around the bolt heads. This also causes cracked shotcrete in the roof, resulting in a fall-of-ground risk.

Although challenging from an operational point of view, immediate flash shotcrete has the advantages of reducing unraveling (chimney risk) and providing the primary bolts a bearing surface. One of the more successful strategies involved developing a 16x16-ft ramp through wet soil-like ground conditions. Similar conditions have required lattice girders. The support sequence was: (1) two inches flash shotcrete, (2) welded wire mesh and 8-ft bolts, (3) two inches shotcrete, and (4) 12-ft bolts. The first and second shotcrete cycles were combined by applying the second coat of shotcrete to the previously bolted round at the same time as the initial shotcrete for the most recent blast round. Twelve-foot bolts were added every couple of rounds to reduce cycle times. The strategy appears to have been a success with no sign of squeeze over the last couple of years.

Surface support guidelines and support installation sequencing are also discussed in Raffaldi et al. (2018) and presented on Table 7 below

Table 7. Surface support and sequencing guidelines based on RMR. After Raffaldi (2018)

RMR	Energy[1] (Joules)	Surface Support and Sequencing
60-80	0	Mesh and bolts
55-65	200	Shotcrete over mesh and bolts or Bolts over fibercrete (3.5-5 lb/yd^3)
45-55	200-280	Bolts over fibercrete (5-7.5 lb/yd^3)
25-45	280-440	Flash shotcrete, mesh and bolts, shotcrete or Bolts over fibercrete (7.5-12 lb/yd^3)
0-25	440-460	Flash shotcrete, mesh and bolts, shotcrete, longer high capacity bolts or Bolts over fibercrete (12-18 lb/yd^3)

[1] Suggested round panel (RDP) test toughness ASTM C1550

6.9. Excavation Strategy

The term “excavation engineering” (Windsor et al., 1995) is used to include all aspects of developing an underground opening, including blasting and excavation strategy. Underground blast round length, blast intensity and timing, spiling, support timing, and support sequencing all play a role in preventing heading loss (chimney) and achieving final excavation stability.

Excavation surface support, including metal straps, wire mesh, and shotcrete, aid in tying small discontinuous rock blocks to the deep tendon (rock bolt) anchorages. Surface support also acts to prevent unraveling of the back and ribs that can result in unstable situations. In weak and squeezing ground situations, surface support acts as a foundation for rock bolts to pull against, effectively tying the entire support system together.

Table 8 presents guidelines for excavation surface support and excavation strategies based on a review of: (1) ground control management plans, (2) consulting documents and internal mine memos, (3) discussions with engineers and miners, (4) the author’s experience, and (5) analysis of case-history data. The reader is referred to Warren (2016) where data analysis for surface support design guidelines and detailed blast design guidelines for weak rock masses are presented.

Table 8: Excavation surface support and excavation strategy. After Warren (2016)

W-RMR$_{76}$	Description
80–100	10-ft rounds. Mesh and bolts in back to spring line.
70–80	10ft rounds. Mesh and bolts in back to spring line, bolts to 4 ft above floor.
45–70	10-ft rounds. Mesh and bolts to 4 ft above floor. 2” delayed shotcrete in the roof/back optional.
35–45	8–10 ft rounds. Mesh and bolts to 2 ft above floor, 2” shotcrete. Possible reduced standup time.
25–35	6–8 ft caution blast rounds. 2” flash shotcrete or spiling if needed. Mesh and bolts to 2 ft above floor. 2” shotcrete roof and ribs. Long bolts over shotcrete. Shotcrete to face every round. Line drill back. Reduced standup time. Mix rock-soil like material. Long-term development discouraged.
15–25	4–6 ft light blast rounds. Line drill back. 2” flash shotcrete or spiling likely needed. Mesh and bolts to floor. 2” shotcrete after bolting. Long bolts over 2nd coat shotcrete. Shotcrete to face every round. Reduced standup time. High risk of chimney. Air-slack prone. Expect long-term squeezing conditions. Long-term development highly discouraged.
10–15	4 ft very light blast rounds or muck-advance. 2” immediate flash shotcrete or spiling likely needed. Bolts and mesh to floor. 2” shotcrete after bolting. Long bolts over 2nd coat of shotcrete. Soil-like material with very low unsupported standup time (hours). High risk of chimney. Air-slack prone. Line drill back or possible grouted hollow-tube spiling. **Temporary openings**: Expect squeezing. Mine and backfill as soon as possible. **Permanent openings:** Long-term development highly discouraged. Lattice girders likely for long-term ramps and access development in clay conditions.

Guidelines presented in Table 7 and 8 are intended to assist the engineer based on empirical data and experience as described in this study. These guidelines should be adjusted based on site experience.

7. RIB SUPPORT VERSUS BACK SUPPORT FOS

Close examination of the design curves (Figures 8 and 9) indicate that higher design factors of safety are required to prevent squeeze in the ribs compared to the back, particularly in weak rock mass (W-RMR<30). This can be counterintuitive for those used to operating in mines that heavily bolt the back compared to the ribs. Recognize that this bolting practice is typically designed to prevent gravity-driven groundfalls to which the back is more prone compared to the ribs.

In the case of squeezing ground, the ground support is attempting to reinforce or confine ground that is overstressed in that the strength of the rockmass has been exceeded by the induced stress. Note that in northern Nevada, the horizontal stress is considered to be less than the vertical stress for a given depth/location as discussed in Section 2. This results in higher loads being shed onto the abutments (ribs) compared to the back and sill, which take a relatively lower horizontal load. The result is that the ribs typically become overstressed before the back in a given mining situation.

A simple RS2 model (Rocscience, 2017), shown in Figure 12, visually describes the overstressed conditions of the ribs compared to the back. This model represents a 16x16-ft excavation roughly 2,500 ft below ground surface in moderate quality rock, with Swellex PM12 bolts on a 4x4-ft spacing. The horizontal stress ratio (K value) is 0.7, indicating higher vertical stress compared to horizontal stress.

The warmer colors about 8 ft back into the ribs represent higher differential stress conditions, which are a result of load shedding of the vertical stress onto the abutments (ribs). Note that these increased stress conditions are not present in the back or sill. Also note that in this model, bolts installed in the ribs have yielded, while the bolts in the back have not.

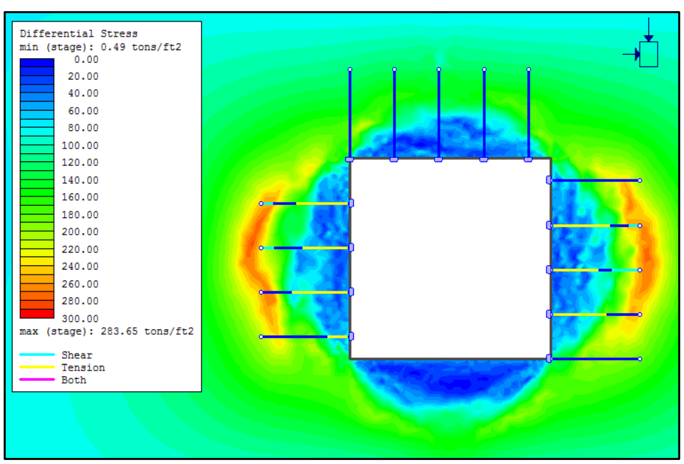

Figure 12. Differential elasto-plastic model of an excavation in a typical Nevada underground scenario. Note higher differential stress shedding onto the ribs.

This theory provides a reasonable explanation for the need for more ground support in the ribs compared to the back in northern Nevada squeezing ground conditions. Experience by the authors and colleagues in the industry supports the observation that the ribs tend to have much more stability problems than the back in terms of squeezing ground issues.

8. CASE-HISTORY RECORD ATTRIBUTES

Empirically derived guidelines should be applied within the context of the case-study database used to develop them. The case histories for this study were collected from eight underground gold mines in northern Nevada, and ground support guidelines developed from this data are considered most applicable in this region. The application of these guidelines outside of underground gold mines in Nevada is not addressed in this paper; however, basic case-study statistics are presented in Tables 9 and 10 so that practitioners can evaluate to what extent a particular site fits within the database of case histories collected for this study.

Table 9: Excavation Attributes from the Case-History Database

Excavation Attribute	Mode	Practical Range
Depth (ft)	1,750	900–3,000
Span (ft)	18	10–38
Rib Height (ft)	18	10–30
W-RMR	30	10–80
RQD	0	0–100
Intact Rock Strength (psi)	7,500	100–15,000
Groundwater Condition	Dry	Wet–Dry

Table 10: Installed Bolt Types from the Case-History Database

Bolt Type	% of Back Support Database	% of Rib Support Database
Inflatable (Swellex type)	60	72
Inflatable (Swellex type) + Split Sets	15	4
Rebar + Split Sets	9	4
Cables + other bolts	7	1
Split Set	6	14
Combination Split Set, Rebar, Inflatable (Swellex)	3	NA
No Support	0	5

9. GEOTECHNICAL CORRELATIONS, W-RMR, Q, AND GSI

While RMR is probably the most commonly applied geotechnical classification system in Nevada, there are operations that use other systems, including the Geologic Strength Index (GSI) (Marinos et al., 2005) and the Tunnel Quality Index (Q) (Barton et al., 1974). Correlation between these systems and the W-RMR system allows these operations to benefit from the design guidelines presented in this paper.

9.1. *W-RMR and Q Correlation*

Whenever possible, a site-specific or regional correlation between W-RMR and Q should be determined. Figure 13 represents a Q versus a W-RMR correlation based on geotechnical data collected at Nevada underground gold mines of:

$$W\text{-}RMR=6.5LnQ+40 \quad (5)$$

Note that a common Q-RMR correlation RMR=9LnQ+44 (Bieniawski, 1976) is also plotted for comparison.

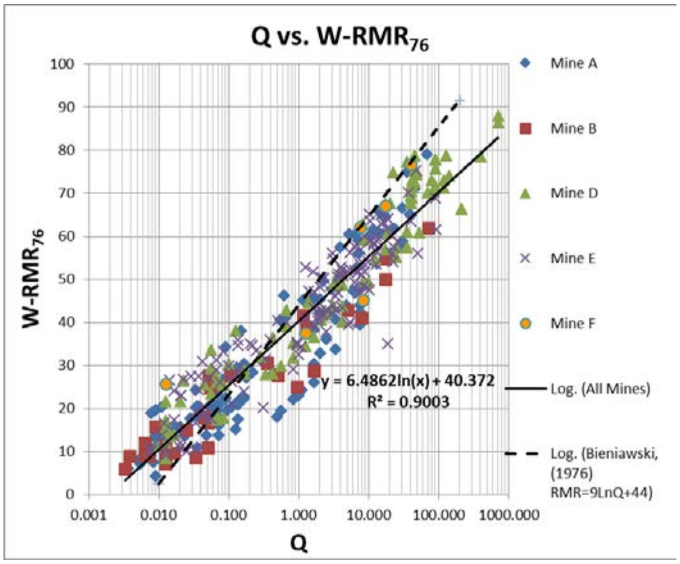

Figure 13. A Q versus a W-RMR correlation for underground gold mines in Nevada. (From Warren and Kallu, 2016)

9.2. *W-RMR and GSI Correlation*

GSI data is simpler to collect than RMR data, and good quality GSI data is probably more useful and reliable than poor quality W-RMR data. This is particularly relevant when geotechnical documentation may be considered an additional burden during exploration core logging or grade control activities.

Recognizing this, some of the underground gold mines in Nevada are switching from an RMR-based geotechnical mapping/modeling system to a GSI system. With this in mind, a data-supported GSI/W-RMR_{76} correlation is useful and presented in Figure 14 below. Note that W-RMR_{76} is generally about 10% lower than GSI with a more precise correlation of:

$$W\text{-}RMR = 0.903GSI - 0.80 \quad (6)$$

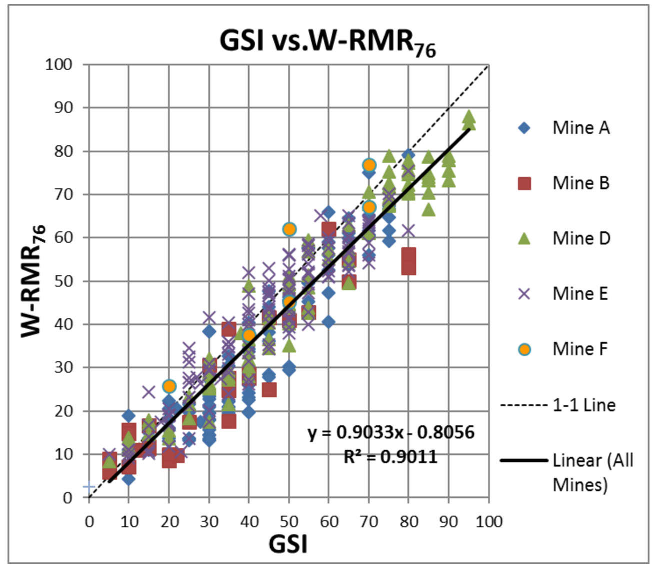

Figure 14. A GSI versus W-RMR correlation for Nevada underground mines. Note that most of the data set is in dry groundwater conditions. (From Warren, 2016)

The most significant differences between the W-RMR_{76} and GSI are that GSI purposely does not include a ground water or an intact rock strength component, as these are handled separately in the Hoek-Brown Failure Criterion and stability modeling. Both of these parameters can have a significant effect on the rock mass support requirements.

When documenting GSI, Marinos et al. (2005) suggested that "when working with rocks in the fair to very poor categories, a shift to the right [on the GSI chart] may be made for wet conditions." When collecting GSI data, it is good practice to document intact rock strength (ISRM R) and groundwater conditions for later reference.

Practitioners are reminded that the GSI system was never intended as a replacement for RMR or Q, and its primary function is the estimation of rock mass properties for numerical modeling or closed form solutions. (Marinos et al., 2005). In general, the use of GSI as a standalone classification system requires more engineering judgment compared to the RMR or Q systems. Users are cautioned to use good engineering judgment when substituting the GSI as input into the ground support guidelines presented in this paper.

10. SUMMARY

Ground support guidelines presented in this paper are based on experience at eight underground gold mines in Nevada, including: a review of ground control management plans and consulting documents, discussions with engineers and miners, and analyses of roughly 400 ground control case studies. The design charts presented in this paper do not represent precision design tools, but rather are a set of data-supported guidelines to aid the engineer in making support design and excavation strategy decisions. It is the hope of the authors that these ground support guidelines result in higher confidence and performance of installed ground support at underground gold mines in Nevada.

11. DISCLAIMER

Ground support guidelines presented in this report are based on a USCS/W-RMR_{76} correlation, W-RMR_{76} equations discussed in Section 5. Use of traditional RMR_{76} classifications procedures as presented in Bieniawski (1976) will result in higher RMR values for RMR<30 and could result in inadequately supported ground, particularly in weak ground.

The findings and conclusions in this paper are those of the author(s) and do not necessarily represent the views of the National Institute for Occupational Safety and Health (NIOSH). Mention of any company or product does not constitute endorsement by NIOSH.

12. ACKNOWLEDGEMENTS

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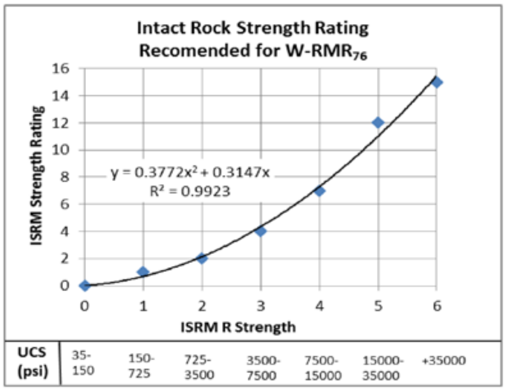
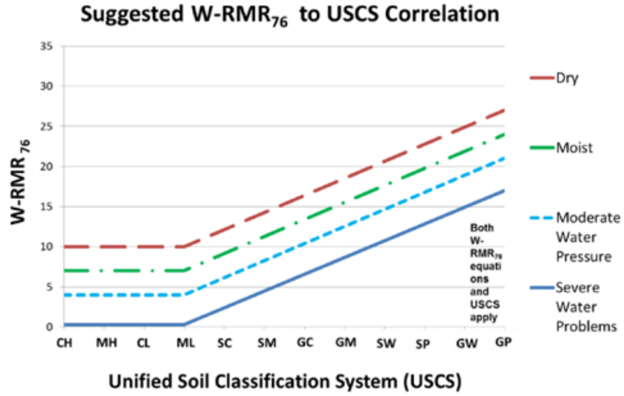
ISRM R strength	Description	Field identification	Uniaxial compressive strength (MPa)	Uniaxial compressive strength (psi)	Examples
R0	Extremely weak rock	Indented by thumbnail	0.25–1	36–145	Stiff fault gouge
R1	Very weak rock	Crumbles under firm blows with a geological hammer	1–5	145–725	Highly weathered or altered rock
R2	Weak rock	Can be peeled with a pocket knife, shallow indentations made by firm blow of a geological hammer	5–25	725–3625	Chalk, rocksalt, potash
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of a geological hammer	25–50	3625–7250	Concrete, limestone shale, siltstone, coal, schist
R4	Strong rock	Specimen requires more than one blow of a geological hammer to fracture	50–100	7250–14,500	Limestone, marble, phyllite, sandstone, shale, dolomite
R5	Very strong rock	Specimen requires several blows of a geological hammer to fracture	100–250	14,500–36,250	Basalt, gneiss, granite, dolomite
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250	>36,250	Quartzite, chert, granite, gneiss

Appendix 1. ISRM strength modified from Brown (1981)

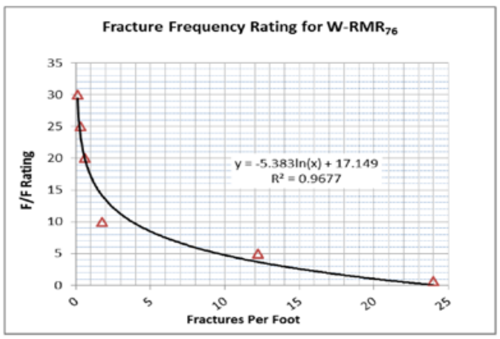
W-RMR$_{76}$ Ratings



After Bieniawski (1989) and Brown (1981)



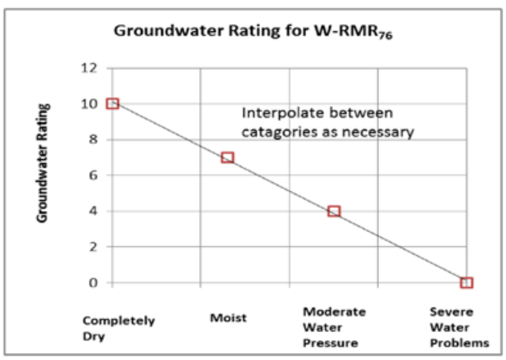
From Warren et al. 2015



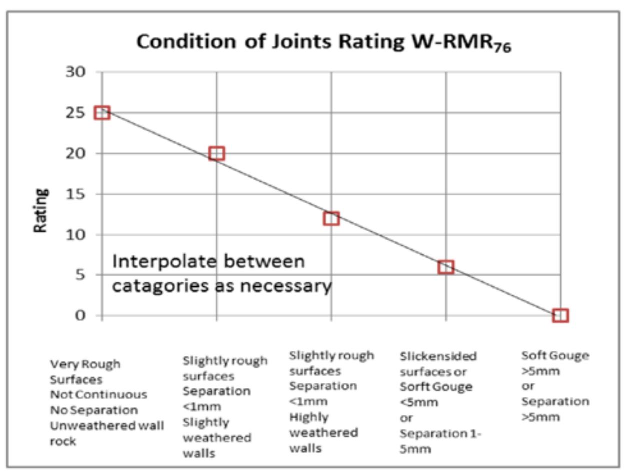
Modified from Bieniawski (1976)

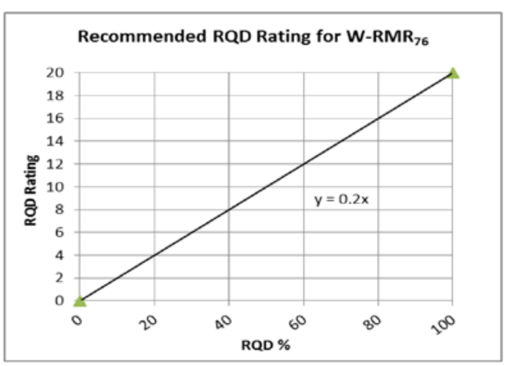
Major Division	USCS Group Symbol	Typical Description
Fine grained soils. >50% by weight clay or silt	CH	High plasticity clay
	CL	Low plasticity clay
	MH	High plasticity silt
	ML	Low plasticity silt
Coarse grained soils. >50% by weight sand and gravel	SC	Clayey sand
	SM	Silty sand
	SW	Clean sand, well graded
	SP	Clean sand, poorly graded
	GC	Clayey gravel, gravel-sand-clay mixtures
	GM	Silty gravel, gravel, sand, silt mixtures
	GW	Well graded gravel, gravel-sand mixtures, <5% fines
	GP	Poorly graded gravel gravel-sand mixtures, <5% fines

After ASTM D2488-09



After Bieniawski (1976)





After Lowson and Bieniawski (2013)

After Bieniawski (1976)

Appendix 2. Calculation of W-RMR. From Warren et al. (2016)