Assessment of Parameter Uncertainty Associated with Dip Slope Stability Analyses as a Means to Improve Site Investigations

Brendan R. Fisher, M.ASCE[1] and Erik Eberhardt[2]

Abstract: Uncertainty is inherent in geotechnical design. In regard to estimating the stability state of dip slopes, most of the uncertainty lies in the geologic model assumed and the geotechnical parameters used in the evaluation. Biplanar (or active–passive) sliding in dip slopes occurs along a slope-parallel sliding surface with toe breakout occurring at the base of the failure. Internal shearing is required to facilitate kinematic release. All three of these release surfaces work together for the slope to fail, but with different degrees of importance depending on the dip-slope inclination. Increased efficiency and value with respect to the site investigation resources can be gained by working toward minimizing the uncertainty of those parameters that have the greatest bearing on the outcome of the slope stability analysis. This can be done quickly and inexpensively by performing scoping calculations facilitated by the use of Spearman rank correlation coefficients. This paper demonstrates that for shallow-dipping dip slopes, stability is primarily dictated by the shear strength of the slope-parallel sliding surface, and therefore, efforts should be focused on constraining the shear strength of this surface. For steep dip slopes, the shear strength related to the toe breakout and internal shear release surfaces becomes dominant, and therefore, the rock-mass shear strength and that for any adversely dipping persistent discontinuities should be the focus of the geotechnical investigation. **DOI:** 10.1061/(ASCE)GT.1943-5606.0000515.

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Introduction

Uncertainty in rock-slope engineering is inherent. Often, field data (e.g., geological structure, rock-mass properties, and groundwater) are restricted to surface observations or limited by inaccessibility, and can never be known completely. This leads to two forms of uncertainty: (1) model uncertainty, which arises from gaps in understanding required to make predictions on the basis of causal inference; and (2) parameter uncertainty, in which geological heterogeneity contributes to spatial variations in rock-mass properties (Morgenstern 1995). For most rock-slope stability problems, these two forms of uncertainty are interconnected. Without a proper understanding of the acting failure mechanism, it is not possible to select the correct set of equations to carry out a stability analysis. Once the failure mechanism is understood, the issue then becomes selecting a specific set or range of values to be used in the analysis.

This is certainly true in the case of biplanar (or active–passive) dip-slope problems, in which in the absence of daylighting bedding, the geologic conditions dictate the mechanism by which failure may occur. Fisher and Eberhardt (2007) show that the dip-slope failure mechanism consists of (Fig. 1):

- Sliding along a slope-parallel sliding surface, most commonly bedding in sedimentary sequences but also a fault, shear, or lithologic contact that is subparallel to the slope face.
- Toe breakout that may occur along a persistent discontinuity, but more often occurs by means of a step path developing along discontinuities of limited persistence and/or shearing through weak rock at the toe of the slope.
- Kinematic release by shearing through the rock mass near the toe of the slope.

On the basis of these findings, Fisher (2009) addresses the issue of model uncertainty by providing a set of procedures for carrying out a dip-slope stability assessment. These include empirical constraints that relate the slope height to failure depth (H/D) ratio (as observed through a number of case histories), together with the application of different analytical tools (e.g., limit equilibrium, finite element, finite difference, and discrete element), the combination of which depends on the geological conditions present. Fisher (2009) used the Sarma (1973) limit-equilibrium method to analyze a range of scenarios and to provide a parametric study on the basis of rational and somewhat conservative factors of safety.

Problem Statement: Parameter Uncertainty

Managing parameter uncertainty is one of the key aspects to understanding the reliability of a slope design (Duncan et al. 2003). Authors that have focused on evaluating circular-slope failures suggest that the stability state of a slope is contingent on the averaged shear strength of the slope lithologies as opposed to the presence of local heterogeneities. Therefore, the spatial distribution of the soil shear strength can be represented by the "averaged" strength along the slip surface (El-Ramly et al. 2002). This is not the case with the stability state of dip slopes because the failure mechanism consists

[1]Principal, Fisher & Strickler Rock Engineering, Radford, VA 24143 (corresponding author). E-mail: bfisher@fsrengineering.com

[2]Professor, Geological Engineering, Univ. of British Columbia, Vancouver, BC, Canada V6T 1Z4.

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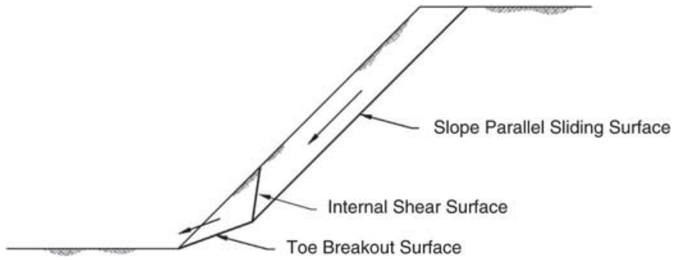

Fig. 1. Simplified dip slope showing biplanar failure mechanism

of sliding along a slope-parallel persistent discontinuity and toe breakout surface (Fig. 1), with the latter often developing through the rock mass (Fisher 2009). Therefore, unlike other slope types documented in the geotechnical literature, a dip slope must be treated as both a continuum and discontinuum.

Another consideration is that a thorough understanding of the influence of the geotechnical input on the outcome of the geotechnical evaluation (e.g., parameter sensitivity) can greatly assist in planning the geotechnical investigation. This is especially important when access and budget may be limited. A typical geotechnical budget for a given project may range between 0.5 and 3% of the project, with up to 8% being reported for large tunneling projects (Parker 2004). Therefore, it is paramount that the geotechnical investigation (e.g., background research, field investigation, and laboratory testing) be planned as efficiently as possible. Of course, this applies to all geotechnical projects, not only dip slopes.

Planning of the geotechnical investigation is usually on the basis of the judgment of the engineer who is in charge of the project. Although experience and judgment cannot be replaced, there are methods available for project planning that aid in the decision-making process with simple up front scoping calculations that provide a means for justifying the site investigation tasks undertaken.

Uncertainties specific to dip slopes can be quantified through a sensitivity analysis that effectively brackets the influence of the geotechnical input parameters on the outcome of the slope stability calculations. These can be carried out relatively quickly when performing a limit-equilibrium analysis, although in the case of numerical slope stability analyses, much time and effort can be expended to test the sensitivity of the model results to parameter uncertainty.

Recently, other authors have proposed using Monte Carlo simulations and first-order second moment (FOSM) calculations to predict the reliability of geotechnical engineering designs (El-Ramly et al. 2002; Harr 1987; Hoek 1987, 2007; Duncan 2000). El-Ramly et al. (2002) discusses Spearman rank correlation coefficients (Spearman 1904) that relate the uncertainty and statistical distribution of geotechnical input parameters to the outcome of geotechnical calculations. Given an understanding of the simple methodology associated with calculating the Spearman rank correlation coefficients, and the efficiency with which these calculations can be accomplished, it becomes clear that this procedure is suitable for prioritization of limited budget funds during planning of the geotechnical investigations.

Spearman Rank Correlations

Spearman coefficients are calculated using rankings of the input values and not the actual values themselves (as is done with a typical linear correlation). The correlation is a value between −1 and 1, and provides an indication of the influence of one input parameter on the result of another. The closer the correlation is to −1 or 1, the better the fit. A positive correlation suggests that a high value of the input results in a high value of the output value. A negative correlation suggests that a high input value results in a low output. Spearman rank correlation coefficients can be calculated using Eq. (1),

$$R^2 = 1 - \frac{6\sum d^2}{n^3 - n} \tag{1}$$

where R^2 = Spearman rank correlation coefficient; d = difference in the ranks between the input and output variables; and n = number of samples.

The Spearman rank is calculated by ranking the input parameters on the basis of their numerical values from highest to lowest. For instance, if the sample population consists of five input values and the numerical values ranged from 1 to 5, 5 would be ranked first (as "one") and 1 would be ranked last (as "five"). Similarly, if the input value of 5 corresponded with an output value of −10 while an input of 1 corresponded with an output of −1, the −1 would be ranked as "one" and the −10 would be ranked as "five." The difference in the rankings is the ranking of the input minus the ranking of the associated output.

A simple spreadsheet can be written to perform a Spearman rank correlation calculation, although there are also spreadsheet add-ons such as @RISK (Palisade Corporation) that perform the calculations for numerous inputs very efficiently. Monte Carlo simulations facilitated by the @RISK add-on were used for the following example, which shows the influence of the different Rock-Mass Rating (RMR) system input parameters on the estimated RMR value.

The RMR system by Bieniawski (1989) was initially developed to empirically aid in tunnel support design and has since become a standard rock-mass mapping index. (The same could be said for Barton's Q-system, for which the following analysis could have been similarly performed.) There are five geotechnical parameters required to estimate the basic RMR (RMR_{89}):

1. Intact rock strength (σ_{ci}),
2. Drill-core quality [e.g., rock quality designation (RQD)],
3. Discontinuity spacing,
4. Discontinuity condition (e.g., roughness), and
5. Groundwater condition.

Although the groundwater conditions are an important consideration when using the RMR_{89} system directly for empirical design or classification purposes, for the purpose of characterization and establishing rock-mass properties, it is often not included as being a characteristic of the rock mass (e.g., Hoek and Brown 1997). Instead, the maximum rating value is assigned (e.g., 15), and groundwater and pore pressures are treated explicitly in an effective stress analysis. This modified system is referred to as RMR^*_{89}.

A rating is assigned to each of the input parameters, and those ratings are summed to arrive at the RMR^*_{89} value. Each one of the input parameters has an influence on the outcome of the RMR^*_{89} although depending on the rock-mass quality, the influence of the input is not equal. For example, consider the input parameters and distributions presented in Table 1. These distributions are on the basis of a data set collected for a dip slope located in southern California (Fisher 2009).

The mean RMR^*_{89} generated suggests that the rock mass may be described as "fair rock" and follows a log normal statistical distribution with a standard deviation (σ) of approximately 6. A similar mean value and standard deviation were obtained using a normal distribution.

A Spearman rank correlation coefficient chart is presented in Fig. 2. It is clear from Fig. 2 that for the rock mass considered,

Table 1. Uncertainty Associated with RMR^*_{89}, Based on Detailed Dip-Slope Data Set from Southern California

Geotechnical property	Distribution	Mean	σ	Minimum	Maximum
		Input parameters			
σ_{ci} (MPa)	Normal	15	6	7	25
RQD (%)	Triangular	63	26	0	100
Discontinuity spacing (mm)	Log normal	450	1,260	10	3,000
Discontinuity-condition rating	Log normal	17	3.5	0	30
Groundwater condition	None	15	0	15	15
		Output values			
RMR^*_{89}	Log normal	54	6	34	76

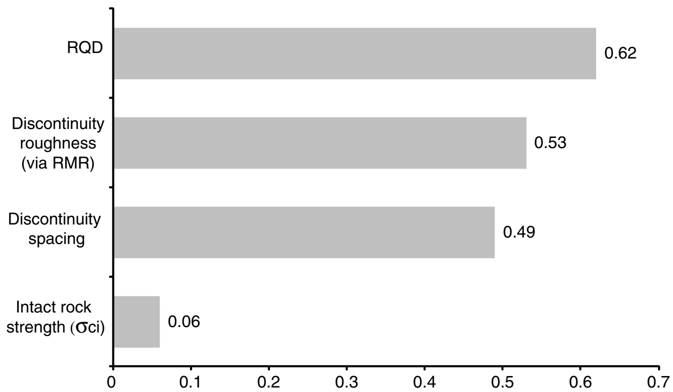

Fig. 2. Spearman rank correlation coefficient for an RMR^*_{89} rated "fair" rock mass

the influences of the input parameters are not equal. The most influential rating parameters are the RQD, discontinuity spacing (directly related to RQD), and the condition of the discontinuities. The intact rock strength (σ_{ci}) has little influence on the RMR^*_{89} estimated as shown by the low ranking in Fig. 2.

An alternative presentation of the correlation coefficient is provided in Fig. 3, which shows the correlation between the calculated RMR^*_{89} and the inputs RQD and σ_{ci}. Fig. 3 suggests that there is a much better correlation between RQD and the calculated RMR^*_{89} than that with σ_{ci}. Clearly, if the goal of a geotechnical investigation is to calculate RMR^*_{89}, and there is a preliminary indication that the rock mass consists of "fair" rock, more resources should be focused on constraining the distribution of RQD than that of the uniaxial compressive strength. An approximation of the latter would be sufficient.

Fig. 4 expands on the previous example by now considering each of the different rating classes as defined by Bieniawski (1989), from very poor to very good quality rock masses. The relative influence of each geotechnical input parameter on the calculated RMR^*_{89} is shown across the different classes. The input distributions used to generate this chart are on the basis of the authors' experience and judgment. In the case of the "good" quality rock mass, the parameters are equally weighted, corresponding to the majority of the case histories that Bieniawski (1989) used to develop the RMR system. This chart was generated by "normalizing" the correlation coefficients for each of the rock grades.

The influence of the geotechnical input parameters on the outcome of the RMR^*_{89} is directly related to the recommended numerical values assigned by Bieniawski (1989) to describe the individual rock-mass properties. For example, poor and very poor rock masses are sensitive to the roughness of the discontinuities. Bieniawski (1989) suggests that the range of numerical ratings for discontinuities typically found in poor to very poor rock masses (gouge infilled to being slightly rough) is 20. Better quality rock (in which the discontinuities are typically slightly rough to very rough) has a numerical rating range of 10. Therefore, RMR^*_{89} for the poorer quality rock masses is more sensitive to the discontinuity characteristics than better quality rock masses.

Rock-quality designation and discontinuity spacing are directly related until the spacing of the joints reaches approximately 600 mm, according to Bieniawski (1989). At this point, the rating for spacing increases but RQD is approximately 100%, and there is

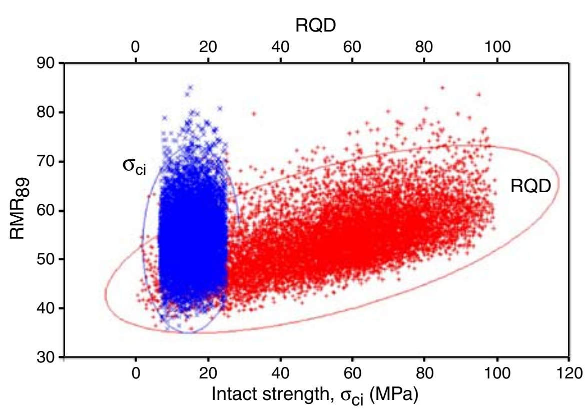

Fig. 3. Influence of uniaxial compressive strength, σ_{ci}, and RQD on RMR^*_{89} (10,000 data points)

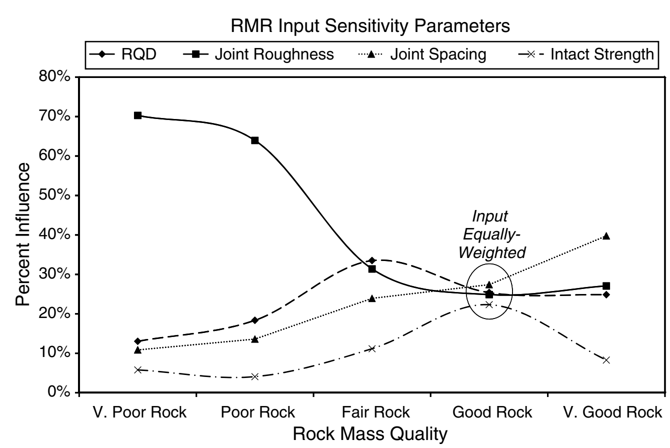

Fig. 4. Influence of different RMR inputs for different RMR^*_{89} classes

no increase in the RQD numerical value assigned. This trend is shown in Fig. 4 by inspection of the RQD and spacing "lines." For the poorer quality rock, there is a general increase in the influence of RQD and spacing. Where the rock quality is fair or better, the influence of RQD begins to diminish, whereas an increase in discontinuity spacing continues to affect the RMR^*_{89} estimated.

The uniaxial compressive strength of the intact rock is seen to have only a small influence on the RMR^*_{89} rating because of the narrow range of ratings assigned to the relatively large ranges of σ_{ci}. Consequently, ranges of σ_{ci} can be easily constrained using simple field tests [International Society for Rock Mechanics commission on Standardization of Laboratory and Field Tests (ISRM) 1978] as opposed to requiring more expensive laboratory testing programs. As illustrated in the next example, this is not the case when establishing Mohr-Coulomb rock-mass strength parameters.

Mohr-Coulomb Rock-Mass Shear Strength Parameters

In many situations, the goal of a geotechnical investigation is to establish the rock-mass shear-strength properties required to carry out a stability analysis. However, issues arise with respect to appropriately scaling laboratory-based values to those that are more representative at the rock-mass scale. For this, Hoek et al. (2002) provide a methodology for establishing rock-mass shear-strength parameters based on the geological and geotechnical site conditions. This procedure uses a nonlinear Hoek-Brown failure envelope to define the laboratory-based intact rock strength, and the geological strength index (GSI) to account for the strength-reducing effects of the rock-mass conditions. The GSI is based on the blockiness of the rock mass and the surface conditions of the discontinuities (Hoek et al. 1995); it can be evaluated directly in the field (Marinos et al. 2005) or estimated from the RMR (Hoek and Brown 1997).

Where possible, the Hoek-Brown criterion should be used directly. However, because many geotechnical design calculations are written for Mohr-Coulomb, it is often necessary to calculate equivalent rock-mass cohesion and friction angle from the Hoek-Brown parameters. Moreover, most practitioners have more experience and therefore an intuitive feeling for the physical meanings of cohesion and friction. The quantitative conversion of Hoek-Brown to Mohr-Coulomb parameters is done by fitting an average linear relationship to the nonlinear Hoek-Brown envelope for a range of minor principal stress values with an upper bound of $\sigma'_{3\,\max}$ (Hoek et al. 2002). The value of $\sigma'_{3\,\max}$ has to be determined for each individual case.

Using this procedure, a Spearman rank correlation simulation was carried out for the purpose of testing the sensitivity of the Mohr-Coulomb shear-strength parameters calculated for the "fair" rated rock mass characterized previously in Table 1. Table 2 lists the Hoek-Brown geotechnical input parameters used and estimates of their statistical distributions. These are assumed from the same data set used for Table 1.

Fig. 5 shows the Spearman rank correlation coefficients for the "fair" rock-mass friction angle (ϕ_{rm}) and cohesion (c_{rm}), generated for a 30-m slope. The height of the slope in this case is used to calculate $\sigma'_{3\,\max}$ using the relationship provided in Hoek et al. (2002). As shown, the intact compressive strength (σ_{ci}) and GSI have the greatest influence on ϕ_{rm} and c_{rm}. Slope height, unit weight (γ), and the disturbance factor (D) are ranked next, with m_i having the least influence on the calculated Mohr-Coulomb shear-strength parameters. More specifically, as either slope height or γ increases, $\sigma'_{3\,\max}$ increases resulting in the linear Mohr-Coulomb envelope being fitted to the flattening gradient of the nonlinear Hoek-Brown envelope. This results in an increasing c_{rm}, but a decreasing ϕ_{rm}. Consequently, the Spearman rank correlation highlights the rock unit weight as a sensitive parameter given its use in the calculation of $\sigma'_{3\,\max}$. Fortunately, unit weight is easily constrained and routinely measured during laboratory testing.

On the basis of Fig. 5, it is clear that if completing a geotechnical investigation directed toward estimating Mohr-Coulomb rock-mass shear-strength parameters for the "fair" rock mass in question, with properties as shown in Tables 1 and 2, the limited resources available for site investigation would be better spent on quantifying the distribution and uncertainty of σ_{ci} and GSI as opposed to m_i. From this, it can be noted that σ_{ci} is a sensitive parameter with respect to estimating rock-mass shear strength [when applying Hoek et al. (2002) procedure], whereas it is of limited influence when establishing RMR^*_{89}. Hoek et al. (1995) and other authors have commented on this difference; in response, Hoek et al. (2002), Hoek (2007), and Marinos et al. (2005) have promoted their development of the GSI system as a means to overcome this and other perceived shortcomings of the RMR system.

Geotechnical Input Distributions

For the previous calculations, it may appear that a detailed data set is required to determine which input parameters the site investigation resources should be directed toward. However, Spearman rank correlations are independent of the statistical distribution of the input parameters insomuch as more reliance on the mean values of the input is not required to provide a meaningful correlation.

Table 2. Uncertainty of Mohr-Coulomb Rock-Mass Properties Derived Using Hoek et al.'s (2002) Estimation Procedure

Geotechnical property	Distribution	Mean	σ	Minimum	Maximum
Input parameters					
GSI	Log normal	49	6	31	79
σ_{ci} (MPa)	Normal	15	6	7	25
m_i	Normal	17	0.67	15	19
γ (kN/m^3)	Normal	22.5	1.67	17.5	27.5
Disturbance, D	Normal	0.4	0.07	0.2	0.6
Slope height (m)	Normal	30	2	24	36
Output values					
c_{rm} (kPa)	Log normal	217	62	90	728
ϕ_{rm} (deg)	Normal	47	3.9	32	58

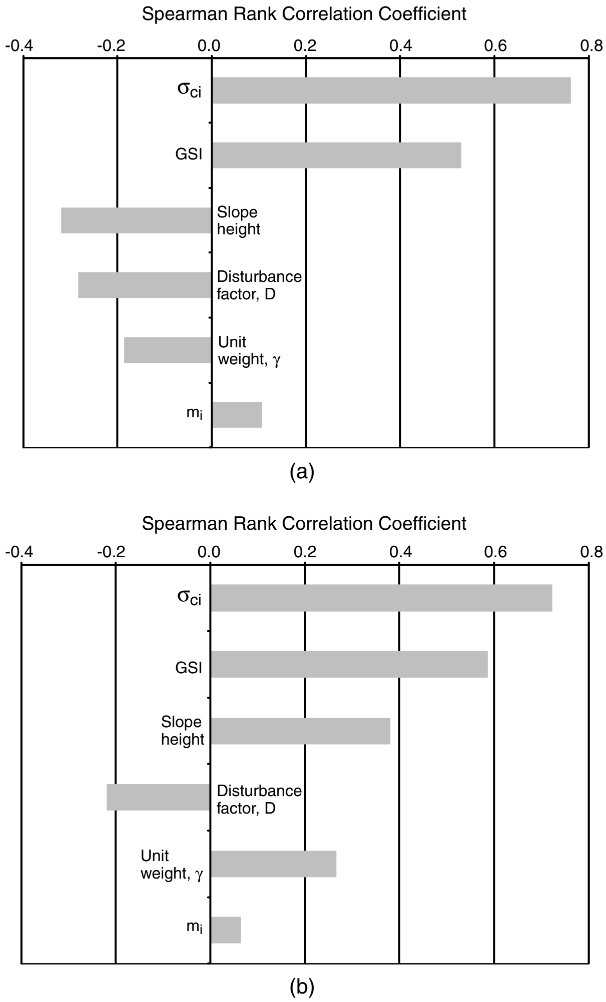

Fig. 5. Spearman rank correlation coefficients for (a) rock-mass friction, ϕ_{rm}; (b) rock-mass cohesion, c_{rm}

The expected minimum and maximum values simply provide a means for constraining the correlation. In other words, similar (but not identical) Spearman rank correlation coefficients would be calculated for the previous example if assuming uniform distributions for the input parameters in Table 2.

For reliability-based calculations, input distributions are paramount and the goal of establishing the correlation coefficients is to give an indication regarding which input parameter distributions have the most influence on the outcome of a deterministic or reliability-based engineering calculation. Therefore, there is merit in discussing typical distributions for the geotechnical input parameters required to establish output variables such as RMR_{89}^* or the Mohr-Coulomb rock-mass shear-strength parameters generated from the Hoek-Brown/GSI procedure.

Hoek (2007) suggests that a normal distribution is the most common distribution used in geotechnical engineering and that when the actual distribution is unknown, a normal distribution should be chosen and that the distribution should in many cases be truncated so that numerical stability using statistical sampling techniques such as Monte Carlo (Harr 1987) or Latin hypercube (Iman et al. 1980; Startzmann and Wattenbarger 1985) can be maintained. Besides normal distributions, other distributions such as beta, exponential, log normal, or triangular have been used in geotechnical engineering (Wyllie and Mah 2004).

Hudson and Harrison (2000) suggest that discontinuity spacing data are best represented by a negative exponential distribution. According to Wyllie and Mah (2004), triangular distributions are common for data sets in which the minimum, maximum, and most likely value can be estimated (such as RQD). Normal and log normal distributions are common for describing joint roughness. Hoek (1989, 2007) suggests that variables, such as GSI, mi, and σ_{ci}, and Mohr-Coulomb values, such as ϕ_{rm} and c_{rm}, can be adequately described using normal distributions.

Sensitivity analyses completed as part of this study suggest that the distribution of RMR_{89}^* varies according to the average or mean rock quality with all but poor and very poor rock masses having a normal distribution. Poor and very poor rock masses showed a log normal distribution, but could be described using a normal distribution with minimal error. If a linear correlation between RMR_{89}^* and GSI is used (e.g., Hoek and Brown 1997), then it follows that similar distributions would be expected for GSI. Hoek (2007) states that normal distributions are appropriate for GSI if it is determined using qualitative descriptions of the rock mass.

The authors have also found that the distribution of the safety factor calculated using Sarma's (1973) method for biplanar failures in dip slopes is log normal. This alleviates concerns regarding inaccuracies associated with estimating probability of failure (P_f) using simplified methods such as Rosenblueth's (1981) point estimate (also referred to as FOSM calculations). The FOSM method has been suggested by Harr (1987), Hoek (1989), and Duncan (2000) for estimating probability of failure for geotechnical applications.

Parameter Uncertainty and Dip-Slope Stability

The influence of geotechnical input parameters on the calculated biplanar dip-slope factor of safety was evaluated by modifying a limit-equilibrium solution published by Sarma (1973), Hoek (1987), and Watson (2000). The solution allows nonvertical slices, and explicitly accounts for internal shear forces and shear strength. The solution is iterative and solves for force equilibrium only. The evaluation was carried out using the following inputs:

- Slope geometry including height, slope angle, and dip-slope bedding/slab thickness;
- Rock-mass properties including unit weight and Mohr-Coulomb shear-strength parameters; the latter were derived on the basis of RMR_{89}^* data using correlations provided by Hoek and Brown (1997) to establish the GSI input; unit weight is used to provide an estimate of $\sigma_{3\,max}$; and
- Discontinuity properties including the Mohr-Coulomb shear strength of the slope-parallel sliding surface (see Fig. 1).

Thus, for dip slopes, a distinction is made between the rock-mass shear strength, which pertains to the internal shearing and toe breakout failure mechanisms (i.e., release surfaces that must develop through the rock mass), and the discontinuity shear strength of the slope-parallel sliding surface (in which it is not appropriate to use an equivalent continuum rock-mass shear strength). Statistical input parameters were used for each of the input variables. Slope geometries are the same as that shown in Fig. 1 with the depth of the slope-parallel sliding surface on the basis of empirical relationships for a large number of case histories reported by Fisher (2009).

The shear strength of the slope-parallel sliding surface was estimated assuming the discontinuity in question would be a bedding plane within a siltstone layer. On the basis of the relatively low plasticity index of the fine-grained siltstone, a fully softened friction angle was assumed having a normal distribution with a mean value of 25° and standard deviation of 1°. The toe breakout and release surface was assumed to develop and dip out of the slope at the most critical angle predicted by plasticity theory. Two different shear-strength scenarios for this feature were considered: toe breakout through shearing of the rock mass, thus requiring use of the rock-mass shear-strength parameters provided in Table 2, and toe breakout along a persistent daylighting discontinuity dipping out of the slope. The second scenario provides a means for comparing the effect of using a "friction only" shear-strength model for the toe breakout surface.

Fig. 6 provides a summary of the sensitivity of the dip-slope stability analysis to the slope-parallel sliding surface and toe breakout and internal shear strengths. Fig. 7 provides a further breakdown of this analysis to include the different input parameters used in the Hoek-Brown procedure to derive the Mohr-Coulomb rock-mass shear-strength parameters. For this, a negative correlation coefficient was assigned to c_{rm} and ϕ_{rm} so that during the Monte Carlo simulation, if a higher value for c_{rm} was chosen, a lower value of ϕ_{rm} was chosen. The correlation coefficient reflects the curvature of the Hoek-Brown failure envelope.

The input parameters shown in Figs. 6 and 7 that have not yet been described include ϕ'_{bedding} and $\phi'_{\text{toe joint}}$, which is the friction angle used for the slope-parallel sliding surface and persistent toe breakout discontinuity (when assumed), respectively.

The first key finding that can be drawn from these results is that the influence of the geotechnical input on the calculated factor of safety is directly related to the inclination of the slope and the resulting distribution of shear stresses that develop in the toe of the slope. Figs. 6 and 7 illustrate this change in parameter influence.

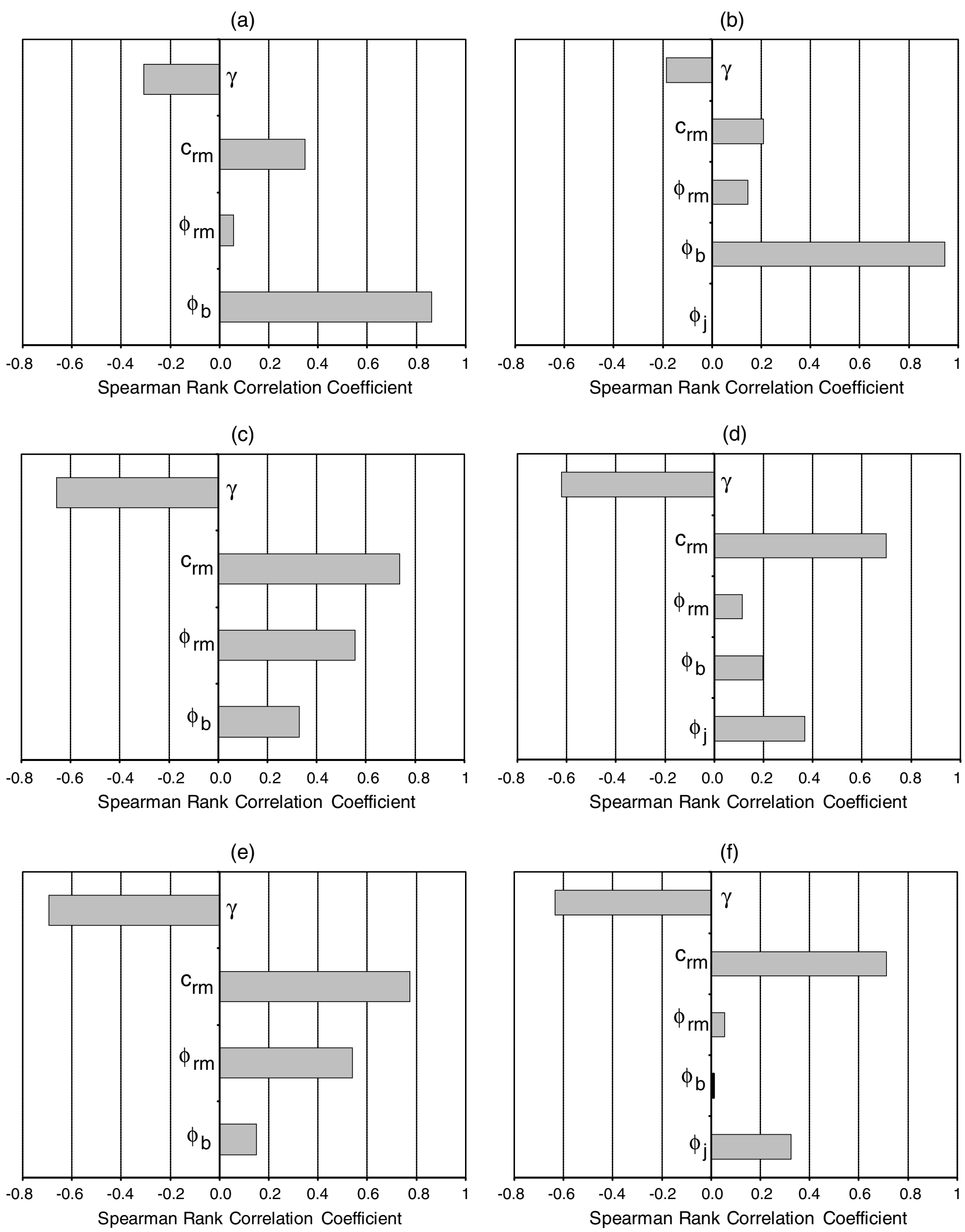

Fig. 6. Influence of Mohr-Coulomb rock-mass shear-strength parameters on dip-slope stability: (a), (c), (e) toe breakout through rock-mass shear failure for dip slopes of 30, 45, and 60°, respectively; (b), (d), (f) toe breakout along a persistent daylighting discontinuity for dip slopes of 30, 45, and 60°, respectively

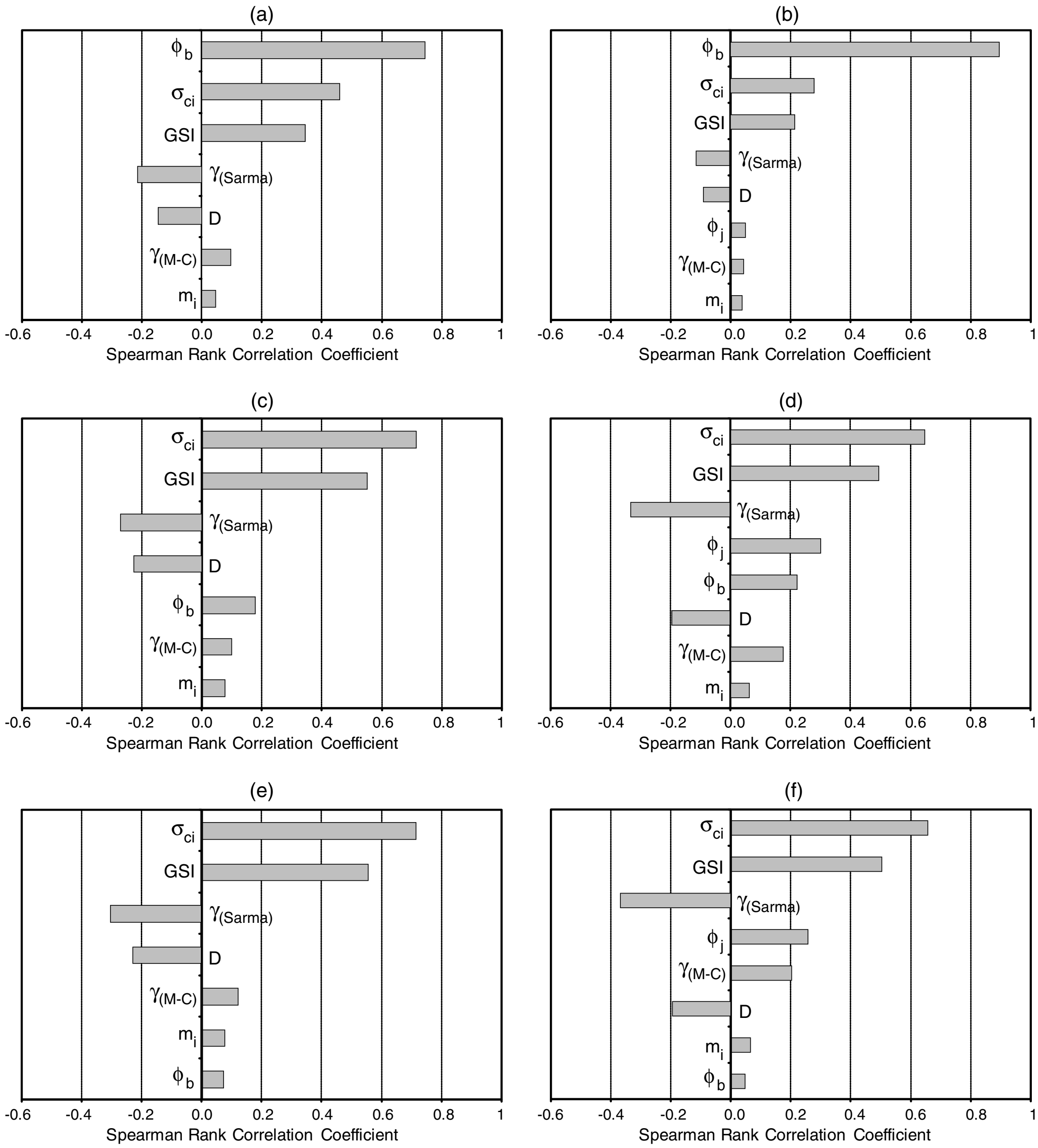

Fig. 7. Influence of Hoek-Brown input parameters on dip-slope stability: (a), (c), (e) toe breakout through rock-mass shear failure for dip slopes of 30, 45, and 60°, respectively; (b), (d), (f) toe breakout along a persistent daylighting discontinuity for dip slopes of 30, 45, and 60°, respectively

In effect, as the slope inclination increases, the normal stresses acting across the sliding surface decrease, and thus the frictional component of shear strength decreases (frictional strength is a function of the acting normal stress). This reduction in shear strength indicates that more active load is transferred to the passive toe of the slope so that stability of the slope becomes more contingent on the shear strength on the internal shear and toe breakout surfaces. Thus, the influence of shear strength along the slope-parallel sliding surface diminishes with an increase in the slope angle, whereas the influence of the toe breakout and internal shear surfaces increases with an increase in slope inclination.

In the case of toe breakout along a continuous, adversely dipping, daylighting discontinuity [Figs. 6 and 7(d)–7(f)], the importance of frictional strength increases with an increase in the slope angle. Once again, this is because of the increase in stress at the toe of the slope as the slope inclination increases. At low slope angles, the influence of the toe joint is lessened, and the shear strength of the bedding dictates the slope's stability state. Furthermore, where the slope is shallow, internal shear plays a more important role in the dip-slope stability state, thus increasing the importance of ϕ_{rm} and c_{rm}. This suggests that applying an infinite slope analysis (Duncan 1996) would produce a very conservative result because the infinite slope solution does not account for the increase in the slope's stability state because of the internal shearing required for kinematic release.

The influence of the internal shear strength can be observed by comparing Figs. 6, 7(b), 7(d), and 7(f). At shallow angles, the toe-joint strength [and thus the rock-mass shear strength at the toe in Figs. 6, 7(a), 7(c), and 7(e)] does not have much influence on the slope's stability state. As the slope inclination increases, more influence is placed on the shear strength at the toe breakout surface and the internal shear.

Another finding is that c_{rm} influences the factor of safety calculation much more than ϕ_{rm}. This is demonstrated by comparing both the rankings of the c_{rm} and the influence of the unit weight (γ) relative to ϕ_{rm}. Where the slope is steeper than 30°, c_{rm} receives the highest ranking and γ has a large negative influence on the factor of safety.

Carrying out the same analysis, but for weak, poor quality rock masses (RMR < 20) with low σ_{ci} values, the results suggest that although the shear strength of the slope-parallel sliding surface remains influential for shallow-dipping angles, ϕ_{rm} becomes more influential than the c_{rm} as the slope angle increases.

Whether represented as ϕ_{rm} with or without c_{rm}, it is clear that the rock-mass shear strength at the toe is the most important input parameter to constrain for a dip-slope stability analysis when dealing with steep dip slopes, whereas the bedding shear strength becomes the more important parameter to constrain for shallow-dipping dip slopes.

Conclusions and Practical Recommendations

The preceding sections illustrate the use of Spearman rank correlation coefficients as a means to quantify the influence of geotechnical input parameters to the calculated outcome. The RMR^*_{89} is sensitive to different inputs depending on the rock-mass quality class, but in general, it would seem unnecessary to expend considerable effort and budget to determine the intact rock strength, σ_{ci}, specifically for estimating RMR. When calculating the Mohr-Coulomb rock-mass shear-strength parameters for "fair" rock using the procedures by Hoek and Brown (1997), GSI and σ_{ci} hold the greatest influence in the output of the results. Therefore, establishing these values with greater certainty during the geotechnical investigation should be a primary focus, whereas expending resources to establish m_i values is not well merited.

In the case of biplanar dip-slope failures, the influence of the input parameters can be directly related to the slope inclination and the transference of the active driving forces (e.g., distribution of stresses) in the slope. For shallow-dipping dip slopes, the shear strength of the slope-parallel sliding surface is most critical, and therefore considerable effort should be spent quantifying its shear strength. For steeper dip slopes, more load is transferred from the upper slope to the slope toe, resulting in the shear strength of the internal shear and toe breakout surfaces having more influence. If shearing is expected through the rock mass, then the focus should be on quantifying the strength of the rock mass. If a persistent joint or fault daylights the slope toe, then establishing its shear strength holds the greatest value.

Thus, there appears to be great value in performing quick scoping calculations to establish the influence of parameter uncertainty on the outcome of design calculations (e.g., factor of safety) before planning the geotechnical data collection campaign. This, of course, moves toward the objective of increasing the efficiency and value return of the field and laboratory investigations. Value of the return can be measured by evaluating the decrease of excess cost that may be incurred if the uncertainty associated with the dip slope's stability state is decreased to a minimal or "residual" value.

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