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Discussion on how to classify and estimate strength of weak rock masses

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Abstract

Weak rocks can be found in many mines around the world, such as the weathered (saprolite/saprock) rocks in tropical areas, the (argillic) altered rocks in the Andes and several gold mines in Nevada, and the soft iron ore deposits in Brazil and Africa. However, it is difficult to classify these materials from drilling core and obtain representative strength for these weak rock masses. This paper discusses the rock mass classification and proposes a transition function for estimating their strengths.

Current application of the rock mass rating (RMR) – Bieniawski classification system and its subsequent input into the Hoek–Brown strength criterion yields low strength parameters that do not represent high stable slopes excavated within weak rock masses, as observed in many mine operations and road cuts.

This paper presents some modifications to the RMR_{76} system, which somewhat takes into account the Robertson (1988) proposed classification system for weak rock masses, by allowing the collection of ratings for RQD and joint condition to obtain higher RMR values for the upper portion of the R1 (i.e. R1+ or R1/R2) and R2 category rock masses that would be greater than the current minimum value of 18 for dry slopes. The RMR classification should not be applied to R0 type materials (UCS<1 MPa), as they should be treated as soil.

It is recognised that at the low end of the rock quality scale, in the transition from inter-block shear failure towards a more matrix controlled rock mass behaviour, a gradual change in the strength curve can be created by considering the reduction in the cohesion component. For the estimation of the weak rock mass strength, a low-end transition Hoek–Brown relationship originally proposed by Carvalho et al. (2007) has been calibrated with additional data and considering the strength range from R1 to R2 materials.

Sá (2010) carried out laboratory strength tests and back-analysis of failed slopes for the N4E open pit iron mine in the Vale's Carajás Mineral Complex, located in the north of Brazil. The calibrated strength parameters were used to assist in defining the lower strength limit, where this transition function should not be applied. Examples for other mines are also included, where weathered/altered rocks exist and were compared with strength parameters estimated from this low-end transition relation.

1 Introduction

Weak rocks have been defined as constituting an intermediate stage between (cohesive) soils and hard rocks, with the potential to degrade within a period of days to several years when exposed to water and climatic changes. The loss of strength is not reversible under normal conditions, whereas with cohesive soils the strength loss can be reversed by changes in water content or loading (Nickmann et al., 2006).

Through field assessment of rock strength (ISRM, 1981), weak rocks are classified as R0 (UCS<1 MPa), R1 (UCS – 1 to 5 MPa) and R2 (UCS – 5 to 25 MPa). There is a common agreement in the technical community that R0 materials should be treated as (cohesive) soils; and that for the upper strength values of the R2 range, these materials can be assumed as behaving like a rock mass. The problem is how to handle the so called R1 to R2 materials, i.e. for materials with UCS in the range of 1 to 15 MPa, which are in the transition from soil to rock behaviour.

Weak rocks can be found in many mines around the world, such as the weathered rocks (or saprolites/saprock) in tropical areas, the (argillic) altered rocks in the Andes and many Nevada gold mines, and the soft iron ore deposits in Brazil and Africa, which feature the leaching of silica from thinly bedded/banded rock. This variety in geological background presents a challenge to practitioners in classifying and estimating the strengths for these weak rock masses for pit slope designs.

The strength of rock tends to decrease with increasing degree of weathering. Weathered rocks have been termed saprock (R3/W3, moderately strong and altered, using ISRM (1981) classifications), saprolite (R1−R2/W4 to W5, very weak to weak, and highly to completely weathered) or residual soil (R0). In general, core recovery of transitional soil-rock materials, such as saprolite, is usually good with high RQD values, with low fracture frequency (wide spacing) and strengths varying from R1 to R2. It is known that slope performance in these materials is affected by:

1. Surface water run-off and groundwater.
2. The saprolite composition, which varies according to its host rock and resultant mineralogy and proportions of sand, silt and/or clay.
3. The relict structural fabric remaining from original rock mass discontinuities.
4. The time of exposure as it deforms with the deepening of the pit and passes through periods of heavy rains and dry seasons.

In some cases, these saprolites can be collapsible, due to their internal arrangement and porosity.

Hydrothermal alteration can reduce the rock strength. The amount of strength reduction depends on the degree and type of alteration and the type of clay minerals, for example: propylitic (e.g. montmorillonite and illite), potassic (feldspar and biotite), and moderate argillic (illite, smectite, montmorillonite, kaolinite) to highly argillic (pyrophilitic, dickite, kaolinite). Other types of alteration, such as silicification, may increase rock strength. The occurrence of swelling clays (e.g. smectite and montmorillonite), will further accelerate the degradation of rock strength when the surface is exposed to the atmosphere and/or to water. Drilling fluids tend to affect core recovery and the parameters for rock mass classification such as RQD, fracture frequency. The surface conditions along discontinuities also tend to be very difficult if not impossible to collect. Suggestions have been given to drill using foam as the circulation fluid and use of specialised barrels such as the Mazier (or triple barrel) with plastic (mylar) inner tubes. This is a common practice in Hong Kong (Carter et al., 1998). Even with these improved drilling methods, the quality of core obtained from these weak rocks can be variable. Sometimes it is similar to that obtained from competent weathered rocks, i.e. good recovery, high RQD, low fracture frequency, but very weak to weak strength. In many other situations, core recovery is poor with low RQD (hard to measure), high fracture frequency and strengths varying from R0 to R2 for the moderate to highly altered rocks.

Attempts have been made to create a Clay Intensity Model for areas with swelling clays through investigations including: spectrometer (identify type of clay), XRD (clay mineralogy), Atterberg limits, slake durability tests, and ethylene glycol tests. However, there is no rock mass classification system that can properly take into account strength degradation as measured by clay intensity and estimate the rock mass strength for use in slope design.

In some soft iron ore deposits, supergene alteration of the host rocks (e.g. phyllites and itabirites) causes leaching of silica and changes primary minerals into clays (Fietze et al., 2013). In the banded (or laminated) iron formations the situation is more complex, with most or all of the silica having been leached and oxidation of the iron minerals forming martite and hematite, resulting in a friable rock mass easily damaged by drilling activity. There can also be re-deposition of iron oxides as a replacement of leached silica. Again, collection of appropriate data from core can be very difficult. Attempts have been made to adapt the GSI classification system for flysch proposed by Marinos and Hoek (2000) to these laminated/banded hematite rocks of similar style. However, more work is still required to classify these soft iron ore materials, for instance, with some form of modified GSI approach.

Table 1 presents a general summary of the conditions observed in the core for several types of weak rocks.

Table 1 Overview of general conditions of weak rocks

Weak Rock Type	Strength of Intact Material	Alteration or Weathering	RQD Generally Observed in the Core	Spacing of Discontinuities	Conditions of Joints
Saprolite	R0 to R2	W4 to W6	>50%	>0.5 to 1 m	Weathered to slickensided
Altered rocks	R0 to R2	A4 to A6	>50%	<50 mm to 300 mm from the core, but tends to be higher when observed from road cuts	Altered surfaces
Banded / leached	R0 to R2	W4 to W5	<40%	<50 mm from the core but not from road cuts	Intercalation of hard and soft material
Rock with swelling clays	R3, degrading to R1 / R0	A3 to A5	>60%	0.3 to 1 m	Can be hard with no infilling

Despite poor observed conditions in drill core, mining slopes greater than 100 m in height have been excavated within these materials, with good performance behaviour. The current practice for pit slope design in weak rock relies more on previous experience, laboratory testing and back-analyses of failures that occurred, rather than strictly on rock mass classification approaches. The limitation of this practice is that previous experience may not be available or truly representative of the conditions in question, and may require unnecessary factoring of design values to account for this uncertainty.

For soil and very weak rocks, their intact strength, derived from laboratory testing and often represented by the Mohr–Coulomb strength criterion, can represent the mass strength behaviour. For stronger materials, rock mass strength can be approximated by downgrading the intact rock strength, obtained from laboratory testing, using a rock mass classification system and applying, for example, the Hoek–Brown strength criterion (Hoek et al., 2002). Rock mass classifications typically attempt to describe the strength of a rock mass as a combination of intact rock strength, block size, and inter-block shear strength. Weak rock in the R1 to R2 strength range is the subject of differing interpretations of rock mass classification schemes such as rock mass rating (Bieniawski, 1976, 1989) and lies between two material strength criteria – the Mohr–Coulomb criterion used for soils, and the Hoek–Brown criterion used for rock. Which strength criterion is used in slope design will require different types of material testing and data collection such as core logging. Therefore clarifying the use of rock mass classification scheme and the approximation of the boundary between use of soil and rock strength criterions will be a valuable tool when designing slopes in weak rock.

Until an acceptable classification system is developed for weak rock masses, the next sections present an approach to carry out this classification by providing guidelines for the application of the RMR_{76} and propose a transition function for the estimation of the weak rock mass strength that bridges the gap between the Mohr–Coulomb strength criterion for soil and the Hoek–Brown strength criterion for rock.

2 Rock mass classifications

There are several rock mass classifications systems used by rock mechanics practitioners including: rock mass rating (RMR, Bieniawski, 1976 and 1989), Q (Barton et al., 1974), MRMR, Laubcher (1990), and GSI (Hoek and Marinos, 2000). It is assumed that readers are familiar with these systems. In an effort to provide some insight for discussion, this paper uses the RMR_{76} system to derive weak rock mass strengths, using its correlation with the approach applied for moderately strong to strong rock masses by Hoek−Brown (Hoek et al., 2002) failure criterion.

The RMR_{76} system assigns ratings to five parameters: Uniaxial Compressive Strength of the intact rock material; rock quality designation (RQD); Spacing of Discontinuities; Conditions of Discontinuities; and Groundwater Conditions. These ratings for each parameter are added to define the RMR values representing the rock mass condition of the zone under consideration and estimate block size and inter−block shear strength for the rock mass. Five rock mass classes, based on the RMR values, classify the rock mass conditions as follows: Class I – very good rock (RMR>80); Class II – good rock (60<RMR<80); Class III – fair rock (40<RMR<60); Class IV – poor rock (20<RMR<40); and Class V – very poor rock (RMR<20).

The ISRM (1981) divides the rock strengths from R0 to R6 according to the UCS of the intact rock, as shown in Table 2. R0 has UCS<1 MPa, R1: 1≤UCS≤5 MPa, R2: 5≤UCS≤25 MPa and R3: 25 ≤UCS≤50 MPa.

It is recognised that for weak rock masses (R1 and R2), block size and inter-block shear strength are not always the main controls on rock mass behaviour. Instead there may be a significant or even greater contribution by matrix strength (and alteration). As a result, some practitioners prefer to abandon the RMR system completely. In terms of RQD, some practitioners accept it to be measured for materials with strengths ≥R3, others would consider it for strengths in the upper range of R2 (i.e. R2/R3), and some like Robertson (1988) would recommend to measure it for the upper range of R1 (i.e. also referred to as R1+ or R1/R2) rocks. It is recognised that these approaches might work for a local region or a mine operation; however, the experience becomes difficult to extrapolate to other regions. A definitive globally applicable classification procedure could assist engineers.

Robertson (1988) proposed modifications to the RMR_{89} system in order to estimate weak rock mass strength from logging of drillhole core. He defined a lower bound for weak rock masses by effective shear strength parameters of less than c′ = 200 kPa and ϕ′ = 30° (i.e. less than UCS ≅ 700 kPa). His approach consisted of dropping the groundwater condition rating from the RMR_{89} and instead adding the 15 points from the groundwater condition to the intact rock strength rating, resulting in the rating assignment of the highest intact rock strength class increasing from 15 to 30. This allowed the strength rating assigned to R1 rocks to increase to 15. In addition, for R1 and stronger rocks defined by not breaking when firmly twisting the core and bending it without substantial force or use of any tool or instruments, Robertson proposed that the RQD be measured and rated in the RMR classification for R1+ (or R1/R2) materials.

Table 2 presents some considerations for applying the RMR_{76} classification system for weak rocks in order to differentiate what material can be treated as 'soil' (R0), transition (R1–R2) and rock (R>R2+, i.e. UCS>10 to 15 MPa). The table was adapted to the changes in defining the rock mass strength and to somewhat take into account the Robertson (1988) proposal to estimate weak rock strength, in that all RMR parameters are measured for all rocks, though penalties are applied to weaker rock to limit RMR values. This is in contrast to other approaches where parameters such as RQD might not be measured for weak rock and a default lower RMR is assigned.

For R0 strength materials (very weak saprolites, very altered rock, and very friable hematite) the rock mass classification should not be applied. Instead they should be treated as soils, using the Mohr–Coulomb criterion to describe strength. Laboratory soil testing alone would be applicable without logging rock mass classifications (e.g. RMR) and their results used for the design of the pit slopes in these R0 materials.

Table 2 Modified guidelines for RMR76 when considering weak materials

Parameter		Range of Values and RMR Parameter Ratings						
Strength of intact rock	PLT, Is_{50}	>10 MPa	4–10 MPa	2–4 MPa	1–2 MPa	UCS, Triaxial Tests Recommended		R0 = Soil Range
	ISRM (1981)	R6	R5	R4	R3	R2	R1	R0
	UCS	>250 MPa	100–250 MPa	50–100 MPa	25–50 MPa	5–25 MPa	1–5 MPa	<1 MPa
	Rating	15	12	7	4	2	1	0
RQD	Strength limits	Rock ≥R3	Rock ≥R2	Rock ≥R1/R2	Rock ≥R1/R2	Rock ≥R1		R0
	RQD	90%–100%	75–90 %	50–75%	25–50%	<25%		
	Rating	20	17	13	8	3		N.A.
Spacing of joints	Strength limits	Rock ≥R3	Rock ≥R3	Rock ≥R3	Rock ≥R1/R2	Rock ≥R1		R0
	Spacing	>3 m	1–3 m	0.3–1 m	50–300 mm	<50 mm		
	Rating	30	25	20	10	5		N.A.
Condition of joints	Strength limits	Rock ≥R3	Rock ≥R3	Rock ≥R2	Rock ≥R1/R2	Rock <R1		R0
	Joint condition	Very rough surfaces Not continuous No separation Hard joint wall rock Unweathered wall rock	Slightly rough surfaces Separation <1 mm Hard joint wall rock Slightly Weathered	Slightly rough surfaces Separation <1 mm Soft joint wall rock Highly Weathered	Slickenside surfaces or gouge <5 mm thick or joint open 1–5 mm Continuous joints	Soft gouge >5 mm thick or separation >5 mm Continuous joints		N.A.
	Rating	25	20	12	6	0		N.A.
Groundwater	Water conditions	Completely dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			R0
	Rating	10	7	4	0			N.A.

For the lower range of R1 materials (UCS<2 MPa), the minimum ratings would be added, to yield an RMR = 18 for dry slopes (Class V), and again laboratory test results could be used to represent the strength of the weak rock mass.

For the upper range of R1 materials (or R1/R2, UCS>2 to 5 MPa) the RMR could be as high as 40 (Class IV), and for R2 materials (UCS>5 MPa) the RMR could reach 51 (Class III) for dry slopes. In both cases, the RQD ratings have been included in the estimation of the RMR values.

3 Estimation of weak rock mass strength

At the low end of the rock mass quality scale (UCS<10 to 15 MPa and RMR generally <40), rock mass strength tends toward matrix (intact) strength. In the transition from inter-block shear failure (which is well modelled by the Hoek–Brown failure criterion and the RMR system) towards a more matrix controlled rock mass behaviour, a gradual change in the strength curve can be created by taking into account a reduction in the cohesion component.

A transition function has been developed to modify the Hoek–Brown criterion to replicate strength behaviour of these rock-soil materials (Carvalho et al., 2007). A practical upper 'soil strength' limit was initially considered for a material with a minimum intact strength of $UCS_i = \sigma_{ci}$ = 0.5 MPa. Below this lower strength, it is difficult for physically meaningful structural discontinuities to influence the soil/rock mass strength and intact strength is more appropriate in representing behaviour. Above this limit, remnant structure results in transitional behaviour from soil to rock, up to approximately 10 to 15 MPa, beyond which structural features are discrete and dominant.

In this paper, both the shape of the transition function and the limits of application have been re-visited to use the ISRM (1981) common strength ranges of R0 through R2 materials, as proposed in the previous section. The lower limit of the transition function has been redefined as $UCS_i = \sigma_{ci}$ = 1.0 MPa and its shape has been modified to extend further into the R2 domain.

3.1 Classical treatment of soil and rock

Although there are many specialised constitutive models for soils, the Mohr–Coulomb strength criterion has always been considered the classical relationship for such materials. For this historical reason and for simplicity in developing this transition function for defining rock mass behaviour at the lower-bound of the rock-soil transition zone, the Mohr–Coulomb criterion linear strength model has been maintained. It is represented in σ_1–σ_3 space by Equations (1) and (2).

$$\sigma_1 = q_u + \sigma_3 \tan^2\left(45^\circ + \phi/2\right) \tag{1}$$

$$q_u = 2c \cdot \tan\left(45^\circ + \phi/2\right) \tag{2}$$

Where:

q_u = uniaxial compressive strength.

ϕ = internal friction angle; and c = cohesion.

To replicate the upper end of the soil-rock (R1 to R2) transition zone, the intact strength of the rock mass has been implicitly modified and rock mass strength appropriately reduced within the governing Hoek−Brown strength criterion. This was achieved through the introduction of the *m* and *s* parameters which attempt to capture inter-block shear strength and the degree of rock mass blockiness. Expressed in σ_1–σ_3 space in the same way as the Mohr–Coulomb criterion, the generalised Hoek–Brown strength model can be represented by Equation (3).

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \tag{3}$$

Where:

m_b and s are respectively the rock mass frictional parameter and the intrinsic strength parameter given by Equations (4) and (5).

$$\frac{m_b}{m_i} = e^{\left(\frac{GSI-100}{28-14D}\right)} \tag{4}$$

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \tag{5}$$

In these Hoek–Brown relationships, two additional parameters have been introduced from the original 1980's formulations: i) blasting damage factor, D, reflecting degree of disturbance due to blasting, varying from 0 (no damage) to 1 (heavy production blasting) (Hoek et al., 2002), and ii) the parameter 'a', to control the degree of non-linearity of the strength function and reflect weathering or other change in competence of lower strength rock masses. The latter is defined by Equation (6).

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \tag{6}$$

For the soil-rock transition strength zone, the following was considered: a) D = 0, considering that the material is likely to be excavated by mechanical equipment and b) by linearising the strength envelope in the generalised Hoek–Brown criterion by varying the parameter 'a' from 0.5 (rock end) to 1 (soil end), Equation (7).

$$\sigma_1 = s\sigma_{ci} + (m+1)\sigma_3 \tag{7}$$

which is equivalent to the Mohr–Coulomb strength criterion shown by Equations (8) and (9).

$$q_u = s\sigma_{ci} \tag{8}$$

$$\left(m^* + 1\right) = \tan^2\left(45^\circ + \phi/2\right) \tag{9}$$

The flexibility of the generalised non-linear Hoek–Brown criterion makes it suitable by means of a transition function for use as the governing model for definition of weak rock mass strength throughout the full range of the soil-rock (R1 to R2) transition.

3.2 Transition function development

The initially proposed transition function (Carvalho et al., 2007 and Carter et al., 2008) to incorporate the approach described in Section 2 for the treatment of R0 through R2 materials has been calibrated using some laboratory and back analyses results. Soil-like material tends to exhibit linear strength behaviour (as 'a' approaches 1, the strength envelope becomes a linear Mohr–Coulomb type of envelope) compared to the non-linear strength envelope for rock (e.g. Hoek–Brown envelope). Parameter m_b* in the linearised form of the envelope was therefore adjusted from the conventional Hoek–Brown specifications which are based on 'a' ≈ 0.5 in order to develop a modified m_b* which is also a function of a*. Thus, when full linearisation is used to fit available data, a value of m* is obtained that is of the order of 1/3 m_i, as would be obtained by classical Hoek–Brown fitting with a = 0.5. The updated transition function can be expressed by:

$$f_T = \left(UCS_i\right) = \begin{cases} 1, & UCS_i \leq 1.0\ \mathrm{MPa} \\ \dfrac{1}{UCS_i^2} & \end{cases}, \ UCS_i \text{ in MPa} \tag{10}$$

This transition function has been adjusted since the last proposal to fit proposed ranges of strength by ISRM (1981) as well as new field data. A graphical representation of the transition function is shown in Figure 1.

The original concept still applies, namely, there is a transfer of control from the joint fabric near the rock end of the transition to the 'intact' material (as tested in the lab) at the soil end of the transition. The transition function, f_T, represents the proportion of the contribution from the 'intact' material to the strength of the rock mass.

As the transition function approaches 1, the matrix strength (UCS for the intact rock) dominates the behaviour and the Mohr–Coulomb envelope provides adequate representation of the rock mass strength. On the other hand, as it approaches 0, the contribution of the matrix strength to the behaviour of the rock mass reduces and the block size and inter-block shear strength becomes prevalent, with the current Hoek–Brown strength envelope representing the rock mass strength. Incorporating this into the Hoek–Brown criterion is accomplished by modifying the controlling input parameters by the transition function relationships, as follows:

$$s^* = s + [(1-s) f_T]/(4a^* - 1) \tag{11}$$

$$a^* = a + (1-a) f_T \tag{12}$$

$$m_b^* = \left[m_b + (m_i - m_b) f_T\right] / \left(4a^* - 1\right) \tag{13}$$

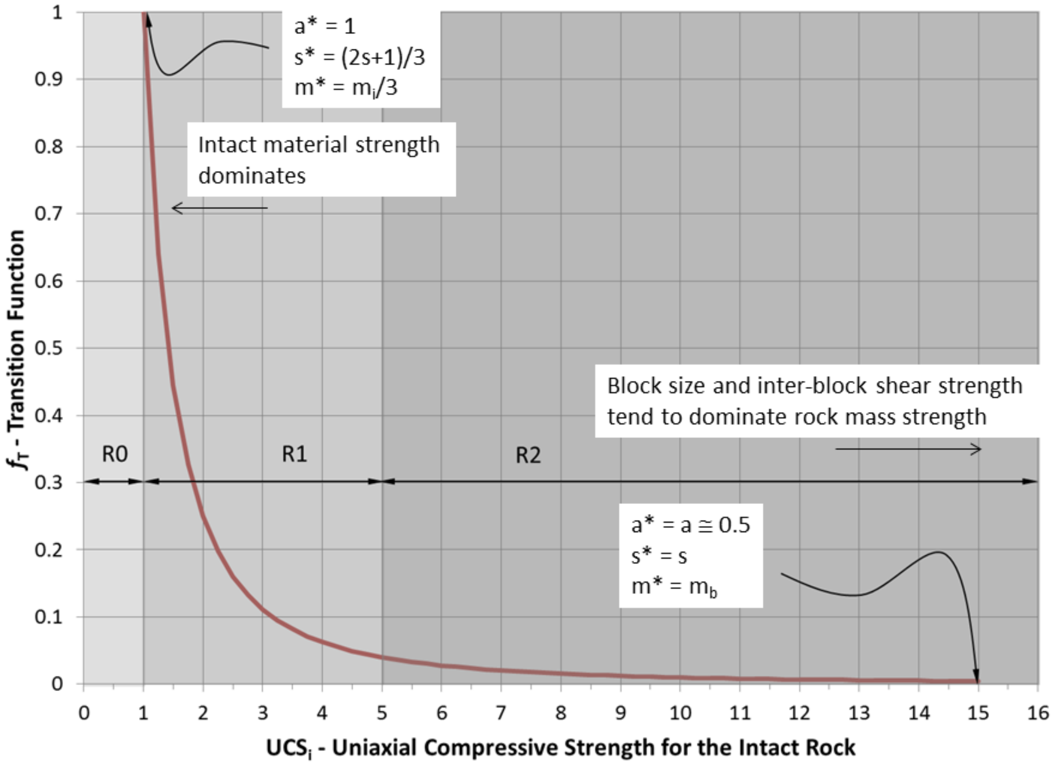


Figure 1 Updated transition function

Three examples are presented to illustrate the approach for using the transition function for weak rocks in the range of R0 (e.g. iron ore), R1 (e.g. Betze–Post mine) and R2 (e.g. mine in the Andes). They emphasise that for the R0 material (example 1) the laboratory intact strength can be used to analyse the slope behaviour. For R1 and R2 materials (examples 2 and 3), in addition to the laboratory strength testing, rock mass classification and the transition function can be used to estimate the weak rock mass strength, as described next.

4 Example: iron ore – north of Brazil

The iron formation in the N4E open pit mine in the Vale's Carajás Mineral Complex located in the north of Brazil contains weak friable hematite and altered mafic (host) rocks with different strengths and degrees of weathering and alteration; depicted in Figure 2 and properties summarised in Table 3.

A) Altered mafic rock B) Hematite friable, slightly to moderately laminated

Figure 2 Examples of altered mafic and friable hematite rocks

Table 3 Summary of the estimated parameters (after Sá, 2010)

Lithology	Description	Degree of Alteration	Field Strength Assessment	Comments	Overall RMR_{76} Values
Altered mafic rock	Lower strength	W5	R0/R1-	Material altered to stratified (or foliated) soil Develops were the fractures occur in the original rock	0, should be treated as soil
	Average	W4/W5	R1	Oxided matrix. Fractures opened (2< thick. <5 mm) and oxided/altered surfaces	18 (min. RMR_{76})
	Upper	W4/W5	R1/R1+	Slightly less oxidation and higher strength Intervals less fractured	18–30
Friable hematite	Friable/poorly laminated (foliated)	W5	R0	Grain size distribution predominantly sandy	0 – should be treated as soil
	Poor to moderately laminated		R1	Millimetric to centimetric, moderately hard to hard, some continuity layers	18 (min. RMR_{76})
	Strongly laminated		R1/R1+	Centimetric, hard and continuous layers	18–30

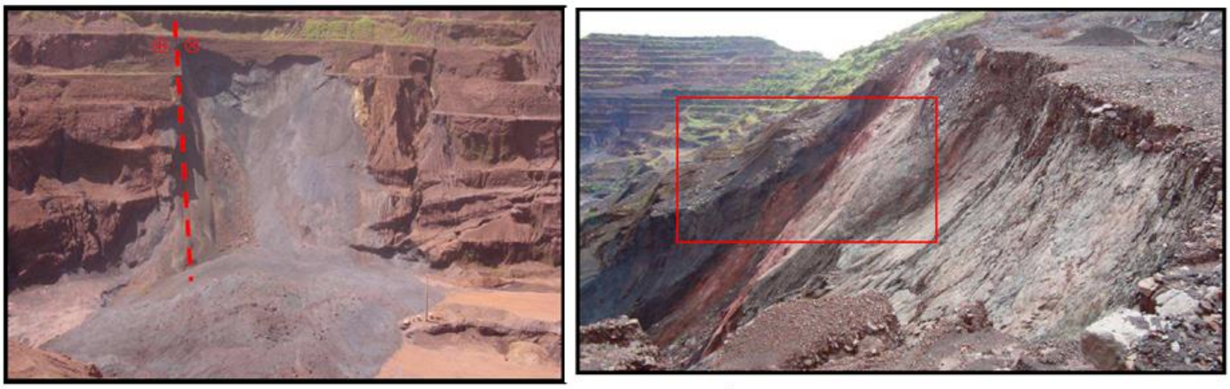
For the characterisation of friable hematite and altered mafic rock, laboratory rock strength tests were carried out including both undrained (CU) and drained (CD) triaxial compressive strength tests, where samples were saturated (CUsat or CDsat) or kept at their natural moisture content (CUnat) prior to consolidation. In addition, direct shear tests were performed on 25 × 25 cm size samples and ring shear tests were executed to estimate peak and residual strengths under large deformations of the samples (Sá, 2010).

Table 4 presents the estimated Mohr–Coulomb strength envelopes. Since the envelopes for the CUsat and CUnat for the altered mafic were about the same, it was decided to also evaluate all test results combined. Table 4 also indicates that with the saturation of the friable hematite samples there was a loss of cohesion.

Table 4 Laboratory strength envelopes (after Sá, 2010)

Strength Envelope	Altered Mafic (AM)		Estimated UCS (kPa)	Friable Hematite (FH)		Estimated UCS (kPa)
	c′ (kPa)	ϕ (°)		c′ (kPa)	ϕ (°)	
Mean CUsat	107	28	356	63	42	283
Mean CUnat	109	28	363	99	38.5	410
Average (CU) – all triaxial data	94	26	300	123	38.5	500
CD sat	–	–	–	143	38	586
Average direct shear test	122	24.6	–	181	42	–
Ring shear test (peak)	32	37	–	–	–	–
Ring shear (residual)	9	18	–	–	–	–

Figure 3 shows two slope failures that occurred during the operation of the N4E pit. The failure in Figure 3(a) occurred through the friable hematite and involved a slope height of about 87 m. Figure 3(b) illustrates a significant 10 m high slope instability, which caused floor heaving adjacent to the toe of the slope and created tension cracks behind the pit crest. The area marked on Figure 3(b) shows the contact between the iron formation and host rock (fresh mafic), located in the south to southeast portion of the N4E open pit. This contact has been interpreted as containing a 1 to 2 m wide shear zone (reddish colour) with very altered mafic rock (Sá, 2010).



A) Failure on the Central Pit B) Section of the failure on the SE pit

Figure 3 Slope instabilities used for back-analyses

The back-analysis results yielded cohesion values in the range of 100 to 125 kPa and friction angles in the range of 38 to 42° for the friable hematite, which is comparable with the triaxial test results. These cohesion and friction angles would suggest UCS values varying from 450 to 560 kPa.

This example is useful for highlighting the consideration of an upper 'soil strength' limit, which for friable hematite could be set at about 0.5 MPa, emphasising that weak rock mass strengths can best be estimated through laboratory rock testing without need for rock mass classification. This is of course assuming homogeneous conditions in the rock, as major geological discontinuities such as faults could initiate failure at otherwise acceptable slope angles.

5 Example: other cases

Data from other mines are presented and their strength parameters used in the pit slope design are compared with those estimated from the transition function, as summarised below.

5.1 Goldstrike betze-post mine, Elko, Nevada, US

The Carlin Formation exposed at Goldstrike (Sharon et al., 2005) contains subunits including the Carlin Silt, Carlin Sand (ash) and Carlin Gravel, with material and strength parameters summarised in Table 5. Two cases of RMR_{76} are applied for each rock type to illustrate the difference between assigning a default RMR_{76} to weak rock (RMR_{76} = 18) and instead measuring the RMR as proposed in Table 2 (in this case giving RMR_{76} = 40).

Table 5 Carlin formation parameters

Lithology	γ (kN/m^3)	σ_{ci} (MPa)	Used in the Design		RMR_{76} (assumed)	Estimated Using the Transition Function (Slope Height = 50 m)[1]	
			c (kPa)	Phi (°)		c (kPa)	Phi (°)
Carlin silt	17.3	2.8	105	30	18	183	28
					40	200	32
Carlin sand (ash)	14.1	1.4	105	35	18	118	35
					40	122	36
Carlin gravel	19.6	2.1	170	35	18	167	30
					40	181	32

[1] Assumed m_i = 12

The estimated values for cohesion and friction angle for the Carlin Sand and Carlin Gravel, applying the transition function, produced reasonable results compared to those used in the design. For the Carlin Silt, the estimated cohesion value was about twice that used in the project, however the intact rock strength of 2.8 MPa is high compared to the c and ϕ used in the design, if applying the relationship $UCS = 2c \cos(\phi)/[1 + \sin(\phi)]$.

Table 5 also illustrates that varying RMR_{76} from 18 to 40 does not significantly change the shear strength parameters determined using the transition function, reducing the concern as to whether or not the RQD should be measured and counted for the RMR classification of R1 type materials.

5.2 Mine in the Andes

Based on laboratory triaxial rock testing carried out for weak (R2) to very weak (R1-R2) volcaniclastic rocks and on local experience, values of cohesion and friction angle, summarised in Table 6, were used in the pit slope design to provide inter-ramp and overall slope angles for dry slopes.

The recommended slope angles were achieved by the mine, suggesting that strength parameters used in the design were adequate to represent field conditions.

Table 6 Laboratory test results for weak volcaniclastic rocks

Lithology	γ (kN/m^3)	σ_{ci} (MPa)	m_i	Used in the Design		RMR_{76} (Assumed)	Estimated Using the Transition Function (Slope Height = 50 m)	
				c (kPa)	Phi (°)		c (kPa)	Phi (°)
Volcaniclastic (R2) Sandstone, gritstones and breccias	20.3	8.8	11.8	250	40	53 (measured)	293	41
Volcaniclastic (R1–R2) Fine, clast poor sandstones, siltstones and tuffs	20.1	3.2	14.2	300	30	18 40	191 215	29 34

Table 6 shows that the transition function would have estimated strength parameters that were close to those used in the design. This example further illustrates that if laboratory strength testing and rock mass classification (including measurement of RQD) are carried out for R1/R2 and R2 materials, the transition function can provide a reasonable approach for deriving the weak rock mass strength parameters that can be used in the pit slope design.

6 Final remarks

In applying the transition function, it was found that the parameter m_i from the Hoek–Brown strength criterion continues to strongly influence the friction angle (the higher m_i, the higher the friction angle). Recognising that the value of m_i varies with the type of rock (as it is related to approximately UCS/σ_T), which is generally lower for sedimentary rocks and higher for metamorphic and igneous rocks, it is expected that the alteration of these rocks would also provide m_i values that take into account the origin of the host rock. Generally m_i values tend to be further reduced for weathered rocks of metamorphic or igneous origin, and slightly reduced for altered sedimentary and volcaniclastics rocks, when compared to their unweathered and unaltered states.

For lower values of m_i and/or values of ‘a’ close to the value of 1.0, varying slope height (which controls confinement), for example, from 10 to 100 m high, does not substantially alter c and ϕ values estimated from the transition function.

The ultimate goal achieved by developing this transition function is to provide more realistic estimates of c and ϕ values for slope design for transitional soil-rock (R1 to R2) materials. The methodology is simple to apply and requires laboratory strength testing and geotechnical core logging. If done correctly with application of good quality data collection procedures, it is cost-effective in allowing extrapolation of the results for other intermediate degrees of alteration (or weathering) thereby allowing reasonable estimation of weak rock mass strengths for large scale open pit design.

As this model is in formative development, the authors would appreciate receiving data from other case histories in weak rock masses to further improve the concepts.

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