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A Geomechanical Classification for Slopes: Slope Mass Rating

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INTODUCTION

This chapter presents a new geomechanical classification for slopes in rock, the 'Slope Mass Rating' (SMR). SMR can be very useful as a tool for the preliminary assessment of slope stability. It gives some simple rules about instability modes and the required support measures. It cannot be a substitute for detailed analysis of each slope, which must combine both good commonsense engineering and sound analytical methods.

SMR classification is a development of the Bieniawski 'Rock Mass Rating' (RMR) system which has become known worldwide, and applied by many technicians as a systematic tool to describe rock mass conditions. The **RMR** concept has been proven to be particularly useful in assessing the need for support in tunnel studies.

Application of the RMR system to slopes has not been possible to date. The SMR system provides adjustment factors, field guidelines and recommendations on support methods which allow a systematic use of geomechanical classification for slopes. Bieniawski [1] has included an abridged version of SMR in his latest book on rock classification.

1. RMR CONCEPT

1.1 DEVELOPMENT OF RMR

ln 1973 Bieniawski [2] introduced 'Rock Mass Rating' (RMR), a new system of rock mass classification, also known as CSIR classification. It included eight rock 'parameters', one of which was 'strike and dip orientations of joints'. Emphasis was given to the use of RMR classification in tunnels.

In the second version of RMR classification [3] some major changes were introduced. Five rock mass parameters were added to obtain the numerical RMR value. From this RMR value, a 'rating adjustment for discontinuity orientations' (always a negative number) was subtracted. Some minor modifications were made in 1979 [4], and the actual form of RMR rating was established (see Table 1).

Bieniawski and Orr [5] applied RMR to dam foundations, correlating the RMR value to the *in situ* modulus of deformation. Serafim and Pereira [6] completed this correlation. Kendorski *et al.* [7] developed a new classification 'Modified Basic RMR' (BMR) for mining with caving methods. Several new parameters were included—blasting damage, induced stress, major geological structures, distance to cave-line and block panel size.

1.2 PREVIOUS APPLICATION OF RMR TO SLOPES

In the 1976 version, the ‘rating adjustments for discontinuity orientation< for slopes were

very favorable	O
favorable	— 5
fair	— 25
unfavorable	— 50
very unfavorable	— 60

No guidelines have been published for the definition of each class. ln the same Symposium Steffen [8] stated that 635 slopes, of which 20 have failed, were classified, and the average values of cohesion and friction were used (to obtain) factors of safety with Hoek design charts for circular failure. Figure 1 shows Steffen’s results, with ‘a definite statistical trend’. It was concluded that ‘the scope for using classification alone as a design method is still very limited’. No reference is given by Bieniawski [9] in 1984 for the use of the RMR classification in slopes. The reason for this lack of use is probably the extremely high values of the ‘adjustment rating value’, which can reach 60 points out of 100. A mistake in this value can supersede by far any careful evaluation of the rock rnass, and classification work would be both difficult and arbitrary.

Table 1. Bieniawski [4] Ratings for RMR

Parameter		Ranges of values						
Strength of intact rock material	Point load index	> 10 MPa	4-10 MPa	2-4 MPa	1-2 MPa		For this low range uniaxial compressive test in preferred	
	Uniaxial compressive	> 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
Drill core quality RQD		90-100%	75-90%	50-75%	25-50%		< 25%	
	Rating	20	17	13	8		3	
Spacing of discontinuities		> 2 m	0.6-2 m	200-600 mm	60-200 mm	<	< 60 mm	
	Rating	20	15	10	8		5	
Condition of discontinuities		Very rough surfaces. Not discontinuous. No separation. Unweathered wall rock	Slightly rough surfaces. Separation < 1 mm. Slightly weathered walls	Slightly rough surfaces. Separation < 1 mm. Highly weathered walls	Slickensided surfaces or gouge < 5 mm thick or separation 1-5 mm. Continuous		Soft gouge > 5 mm or separation > 5 mm. Continuous	
	Rating	30	25	20	10		0	
Groundwater in joints		Completely dry	Damp	Wet	Dripping		Flowing	
	Rating	15	10	7	4		0	

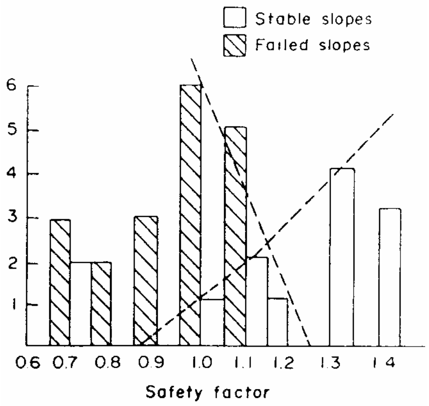

Figure 1. Frecuency distribution of slope stability as pedicted by Hoek´s design charts for the Geomechanics Classification (RMR) (Steffen [8])

2. FAILURE MODES IN SLOPES

Any classification method has to cope with the different possible modes of failure. In a majority of cases, the slope failures in rock mass are governed by joints, and develop across surfaces formed by one or several joints. Basic modes are well known, and are summarized below.

(i) Plane failures along prevalent and/or continuous joints dipping towards the slope, with strike near parallel to the slope face. There are two instability conditions: when critical joints dip less than the slope, and when the mobilized shear strength in the joint is not enough to assure stability. The plane failures depend on joint continuity. The difference between the dip direction values of the slope and the failed joint is less than 90^o .

(ii) Wedge failures along two joints from different families whose intersect dips towards the slope. A ‘wedge factor’ depending on the geometry, multiplies the joint mobilized shear strength. This mode of failure depends on the joint attitude and conditions, and is more frequent than plane failure, but many apparent wedge failures resolve to plane failures when studied in detail. The size of the failure depends on the joint frequency, and is usually minor compared to plane failures.

(iii) Toppling failures along a prevalent and/or continuous family of joints which dip against the slope, and with strike near-parallel to the slope face. Joints sup between them, and are frequently weathered. In practice, two kinds of instability can exist: minor toppling occurring near the surface of slope, and deep toppling which can produce big deformations. In both cases the failures develop slowly. Surface toppling can cause rock falls, but deep toppling seldom fails suddenly. The difference between the dip direction values of slope and joint is more than 90^o.

(iv) Soil-type failure along a surface which only partially develops along joints, but mainly crosses them. These failures can only happen in heavily jointed rock masses with a very small block-type size and/or very weak or heavily weathered rock. In both cases, the RMR value is very low, and the material is borderline with a soil.

Any classification system has to take account of the following ‘parameters’.

(i) Rock mass global characterization (including joints frequency, state and water inflow).

(ii) Differences in strike between slope face and prevalent joints.

(iii) Differences between joint dip angle and slope dip angle, as they control the ‘daylighting’ of a joint in the slope face, a necessary condition for plane and/or wedge failure.

(iv) Relationship of joint dip angle with normal values of joint friction (for plane and/or wedge failure).

(v) Relationships of tangential stresses, developed along a joint, with friction (for toppling failure).

3. SLOPE MASS RATING (SMR)

The proposed 'Slope Mass Rating' (SMR) is obtained from RMR by subtracting a factorial adjustment factor depending on the joint—slope relationship and adding a factor depending on the method of excavation

$$SMR = RMR + (F_1 \cdot F_2 \cdot F_3) + F_4$$

The RMR (see Table 1) is computed according to Bieniawski's 1979 proposal, adding rating values for five parameters: (i) strength of intact rock; (ii) RQD (measured or estimated); (iii) spacing of discontinuities; (iv) condition of discontinuities; and (v) water inflow through discontinuities (estimated in the worst possible conditions). RMR has a total range of 0 - 100.

The adjustment rating for joints (see Table 2) is the product of three factors as follows:

(i) F_1 depends on parallelism between joints and slope face strikes. Its range is from 1.00 (when both are near parallel) to 0.15 (when the angle between them is more than 300 and the failure probability is very 10^o). These values were established empirically, but afterwards were found to approximately match the relationship

$$F_1 = (1 - \sin A)^2$$

where A denotes the angle between the strikes of the slope face and the joint.

(ii) F_2 refers to joint dip angle in the planar mode of failure. In a sense it is a measure of the probability of joint shear strength. Its value vanes from 1.00 (for joints dipping more than 45^o) to 0.15 (for joints dipping less than 200). Also established empirically, it was found afterwards to match approximately the relationship

$$F_2 \; - \mathrm{tg}^2 \, \beta_j$$

where β_j denotes the joint dip angle. For the toppling mode of failure F_2 remains 1.00.

(iii) F_3 reflects the relationship between the slope face and joint dip. Bieniawski's 1976 figures have been kept. In the planar mode of failure E_3 refers to the probability that joints 'daylight' in the slope face. Conditions are fair when slope face and joints are parallel. When the slope dips 100 more than joints, very unfavourable conditions occur.

For the toppling mode of failure, unfavourable or very unfavourable conditions cannot happen in view of the nature of toppling, as there are very few sudden failures and many toppled slopes remain standing. The Goodman-Bray [10] condition has been used to evaluate toppling probability, with the hypothesis that this failure is more frequent in weathered slopes and there is a small reduction (around 50) of shear strength due to rotational friction, as proposed by Goodman [11].

The adjustment factor for the method of excavation (see Table 3) has been fixed empirically as follows:

(i) Natural slopes are more stable, because of long time erosion and built-in protection mechanisms (vegetation, crust desiccation, etc.): $F_4 = +15$.

(ii) Presplitting increases slope stability for half a class: $F_4 = \pm 10$.

(iii) Smooth blasting, when well done, also increases slope stability: $F_4 = \pm 8$.

(iv) Normal blasting, applied with sound methods, does not change slope stability: $F_4 = 0$.

Table 2. Adjustment Rating for Joints

Case		**Very favorable**	**Favorable**	**Fair**	**Unfavorable**	**Very unfavorable**
P	$\lvert \alpha_j - \alpha_s \rvert$	> 30°	30-20°	20-10°	10-5°	5°
T	$\lvert (\alpha_j - \alpha_s) - 180° \rvert$					
P/T	F_1	0.15	0.40	0.70	0.85	1.00
P	$\lvert \beta_j \rvert$	< 20°	20-30°	30-35°	35-45°	45°
P	F_2	0.15	0.40	0.70	0.85	1.00
T	F_2	1	1	1	1	1
P	$\beta_j - \beta_s$	> 10°	10-0°	0°	0° to 10°	< -10°
T	$\beta_j - \beta_s$	< 110°	110-120°	> 120°	-	-
P/T	F_3	0	-6	- 25	-50	- 60

P, plane failure; T, toppling failure; α_j, joint dip direction; α_s slope dip direction; β_j, joint dip; β_s, slope dip

Table 3. Adjustment Rating for Methods of Excavation of Slopes

Method	**Natural Slope**	**Presplitting**	**Smooth blasting**	**Blasting or mechanical**	**Deficient blasting**
F_4	+ 15	+ 10	+ 8	0	- 8

(v) Deficient blasting, often with too much explosive, no detonation timing and/or nonparallel boles, damages stability: $\boldsymbol{F_4} = -8$.

(vi) Mechanical excavation of slopes, usually by ripping, can be done only in soft and/or very fractured rock, and is often combined with some preliminary blasting. The plane of slope is difficult to finish. The method neither increases nor decreases slope stability: $F_4 = 0$.

A tentative description of the SMR classes is given in Table 4.

Swindells [12] presented the results of an investigation on 16 different cuts, in five locations in Scotland (natural slopes, railways, highways, quarries), to assess the influence of blasting methods on slope stability. All the cuts were in igneous or metamorphic rocks. The cuts were investigated with different techniques (visual inspection, field seismic refraction profiling, borehole TV camera, and laboratory testing). Swindells concluded that 'the degree of measurable disturbance is related to excavation technique. Faces excavated by uncontrolled or bulk blasting, or quarry blasting, exhibit greater thicknesses of measurable disturbances'. The numerical data presented by Swindells have been reworked and compared with the SMR corrective factor F_4 in Table 5.

Figure 2 reproduces Swindells data and Figure 3 compares the depth of the disturbed zone with the value of F_4. There is a general similarity between the thickness of disturbed zone and SMR F_4 correction factor. The most important difference occurs with smooth blasting techniques. The two cases investigated were registered by Swindells as 'unsuccessful presplit' and 'bulk-smooth blasting'. Therefore it is possible that these data do not correspond to real, successful excavations by smooth blasting methods.

Kendorski *et al.* [7] evaluated blasting damage in underground mines, by caving, with a factor AB which multiplies RMR. Table 6 compares AB with the SMR corrective factor F_4.

The final Slope Mass Rating is

$$\text{SMR} = \text{RMR} + (F_1 \cdot F_2 \cdot F_3) + F_4$$

No special factors taken for the wedge mode of failure are different from those applied for the plane mode of failure. The practice of classification seems to prove that wedge failures are no more dependent on RMR value than plane failures. Therefore, the classification must be applied for each joints system. The minor value of SMR is retained for the slope.

Weathering cannot be assessed with rock mass classification as it is a temporal process which depends mostly on the mineralogical conditions of rock and the climate. In certain evolutive rocks (like some rnarls and clay-shales) the slopes are stable when open, and fail some time afterwards (usually one to two years later). The classification must be applied twice: for actual fresh and future weathered conditions.

Table 4. Tentative Description of SMR Classes

Class	SMR	Description	Stability	Failures	Support
I	81-100	Very good	Completely stable	None	None
II	61-80	Good	Stable	Some blocks	Occasional
III	41-60	Normal	Partially stable	Some joints or many wedges	Systematic
IV	21-40	Bad	Unstable	Planar or big wedges	Important/corrective
V	0-20	Very bad	Completely unstable	Big planar or soil-like	Reexcavation

Table 5. Comparison between Disturbance effects of blasting methods and F_4 [12]

Excavation method	N	Thickness of disturbance		SMR F_4
		Range (m)	Mean (m)	
Natural slope	4	0	0	+15
Presplitting	3	0-0.6	0.5	+10
Smooth blasting	2	2-4	3	+8
Bulk blasting	3	3-6	4	0

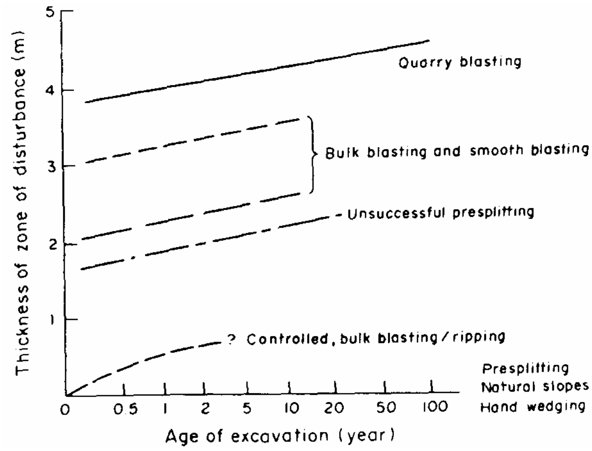

Figure 2. Generalized relationship between excavation technique, age of excavation and seismically detectable disturbance (Swindells [12])

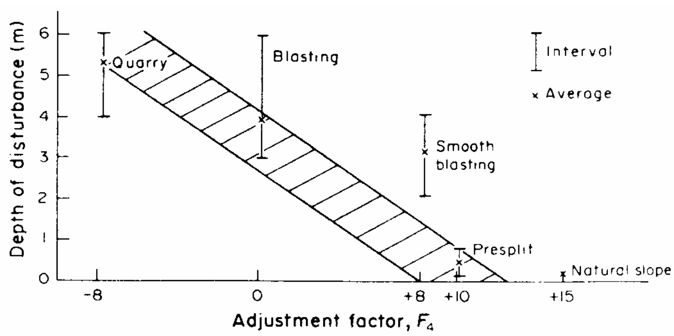

Figure 3. Relationship between depth of disturbed zone, according to Swindells [12], and SMR correction factor F_4

Table 6. Comparison between Kendorski [7] Blasting Coefficient and F_4

Excavation method	**A_B**	**(%)**	**SMR F_4**
Controlled blasting	0.97-0.94	108-104	+8
Good blasting	0.94-0.90	104-100	0
Poor blasting	0.90-0.80 (worst)	100-89	- 8

Water conditions govern the stability of many slopes which are stable in summer and fail in winter because of heavy ram or freezing. The worst possible water conditions must be assumed. The stability of a dry slope is evaluated by an increase of 15 in the RMR.

4. GUIDELINES FOR SMR EVALUATION

4.1 CHOOSING THE OUTCROPS

Classification can be done on the following.

(i) ***Drillhole cores***

Good for rock mass conditions.

Difficult for joints orientation.

Groundwater conditions can be assessed only from general groundwater levels.

(ii) ***Natural rock outcrops***

Only the more sound rock outcrops so general conditions can be masked.

It is easy to measure joints orientation.

When comparing the failure modes the adjusting factor for 'natural slope' must be used.

(iii) ***Other slopes***

The conditions of rock mass depend on slope age, excavation method and weathering conditions. Joints can be more frequent and more open than in rock mass if deficient blasting has been used. It is easy to forecast and compare the failure modes. It is easy to assess the water conditions.

Each rock exposure has advantages and disadvantages. The best classification will be done with a combination of natural outcrops and man-made slopes.

In order to cope with data variability it is necessary to individualize different structural regions in the field for classification purposes. In each region the mean values will represent only the mean conditions. But many slope failures happen in slopes which are stable in normal conditions but which fail because some factor reached an extreme value. So classification systems must take account of extreme conditions.

There are three common problems.

(i) Water conditions are much better during dry seasons, when most field work is likely to be done. In cold climates ice can increase groundwater pressure.
(ii) Some materials show a different behavior at 'long term' because of weathering, pore pressure redistribution, erosion or other causes. Time to failure can be one or several years.

(iii) Frequently, failure is governed by a ‘special’ joint or set, which exhibits different features and has a lower shear strength than ‘normal’ ones.

Classification cannot be a routine task done by people inexperienced in field work. A form for field use is included at the end of this chapter. This form is better suited for classification of existing natural or man-made slopes than for other outcrops.

4.2 STRENGTH OF INTACT ROCK

Adequate input data for the strength of intact rock is the uniaxial compressive strength ***(C_0)*** determined according to ¡SRM Suggested Methods or any other reliable testing standard. However, often it is necessary lo assess strength in the field without the aid of laboratory tests.

Table 7 has been adapted from ¡SRM ‘Suggested Method for the Quantitative Description of Discontinuities in Rock Masses’ [13] and can be helpful lo assess the uniaxial compressive strength from manual index tests performed on rock specimens with a pocket knife and/or geological hammer. Extremely strong rocks are very rare, and very strong rocks are not common, so in most cases it is only necessary lo assess the strength of rock in the lower categories, where the parameter values are low and the possible error not too big. Intact rock strength can be tested in the field with the help of a ‘Schmidt Impact Hammer’ (also known as a ‘Sclerometer’).

Haramy and DeMarco [14] have summarized the procedures and results of several authors, concluding that the test is ‘inexpensive, fast and reliable’ to obtain estimates of compressive strength from core samples, most of them from NX gauge ***(55*** mm diameter). They mention correlations by Deere (1966) and Beverly (1979) between the Schmidt rebound index (obtained when holding the hammer vertically downwards) and uniaxial compressive strength. Testing procedures are described in ISRM ‘Suggested Method for Determining Hardness and Abrasiveness of Rocks’ [15] and ‘Suggested Method for the Quantitative Description of Discontinuities in Rock Masses’ [13]. They can be summarized as follows.

Table 7. Manual Index Text for Assessing Rock Strength (ISRM)

Rock description	Range of C_o (MPa)	Pocket knife	Field identification Geological hammer
Ext. strong	250	No peeling	Only chips after impact
Very strong	100-250	No peeling	Many blows to fracture
Strong	50-100	No peeling	Several blows to fracture
Med. strong	25-50	No peeling	A firm blow to fracture
Weak	5-25	Difficult peeling	Can indent
Very weak	1-5	Easy peeling	Can crumble

(i) Use Schmidt hammers: L type for hard rock; R-710 type for soft materials.

(ii) Apply the hammer in a direction perpendicular to the wall of specimen being tested.

(iii) The test surface must be smooth, flat and free from cracks and discontinuities to a depth of 6 cm.

(iv) Clamp individual specimens to a rigid base.

(v) Discard 'anomalous' tests, easily detected through lack of rebound and 'hollow' sound, or those causing cracks or visible failure.

(vi) Conduct 10 to 20 tests on each series. Test locations should be separated by at least one diameter of the hammer.

(vii) Record the angle of orientation of the hammer. Use the correction curves supplied by the manufacturer for test results.

(viii) Discard the half on the tests giving lower results.

(ix) The rebound index is obtained as the mean of the higher half of the results.

In practice most tests on rock outcrops must be done in a horizontal (or near horizontal) direction. In these conditions the maximum estimated strength will be 60 MPa (for the L type hammer). Strength is lower when the rock surface is saturated. The average dispersion is 40% of estimated strength (and minimum error 10%).

4.3 RQD

'Rock Quality Designation' (RQD) was defined by Deere [16] as the total length of all the pieces of sound core over 10 cm long, expressed as a percentage of the length drilled. If the core is broken by handling or by the drilling process (giving a fresh fracture) the broken pieces must be fitted together and counted as one piece. The length of individual core pieces must be measured along the axis of the core, trying lo avoid a joint parallel lo the drill hole that penalizes the RQD values loo much. RQD must be estimated for variable length, logging separately structural domains, weakness zones, individual beds and any other significant features in the rock mass. RQD was first established for igneous rocks, where it is much easier lo apply than in metamorphic foliated rocks. It has become a widespread method of assessing rock mass quality.

Reliable RQD values are obtained only when: (i) the drill core is NX diameter; (mi) drilling has been done with a double battery; and (iii) logging takes place as soon as possible after drilling.

Palmstrom [17] proposed an approximate correlation between RQD and the 'volumetric joint count' (number of joints per cubic meter), which can be used lo estimate RQD when drill cores are not available

$$RQD = 115 — 3.3J_v \quad (RQD > 100)$$

$$J_v = \sum 1/\overline{S_i}$$

where $\overline{S_i}$ is the mean spacing for the discontinuities of family i (m).

Priest and Hudson [18] proposed a correlation between the mean spacing of joints and RQD value in the direction perpendicular lo joints

$$RQD = 100(0.1/S + 1)\exp(0.1/S)$$

where ***S*** is the mean spacing in meters.

The Priest—Hudson formulation is based on a Poisson probabilistic distribution of frequency for joints. It has been validated for

$$RQD > 50 \quad (\overline{S} > 0.06 \text{ m})$$

Both correlations give the same values for a rock mass with typical block dimensions of 1 m x 1 m

x S_1 for which

$$J_v = 1/\overline{S_i}\ , + 2$$

4.4 JOINT SPACING (S_i)

Spacing of discontinuities is the distance between them, measured along a line perpendicular lo discontinuity planes.

The ISRM [13] suggest the use of minimum, modal and maximum values of spacing to characterize a set of joints. This procedure has been superseded in practice by the use of mean spacing. Bieniawski defines the spacing as the 'mean distance' so the mean spading is the appropriate input in RMR and SMR classification. Spacing is measured with a tape along the rock outcrop, counting the number of joints in a fixed distance and multiplying by the corresponding cosines of angles between the normal to joints and the plane of rock outcrop.

In practice this is an easy task for set of joints with vertical dip and strike not parallel to the slope. But many times the dangerous set of discontinuities for slope stability happens to be composed of joints with strike parallel lo slope. In these cases systematic tape measurements are seldom possible. It is suggested to assess visually the model value of spacing of dangerous joints and measure it carefully afterwards.

RMR uses the classification of discontinuity spacings proposed by the ISRM [13] and presented in Table 8. Bieniawski [9] has added a description of rock mass conditions.

4.5 JOINT CONDITION

This is a very complex parameter which includes several subparameters: (i) roughness; (Ii) separation; (iii) filling material; (iv) persistence; and (v) weathering of walls.

4.5.1. <u>Roughness / filling</u>

Bieniawski [9] has proposed a roughness scale which is very easy to check in the field.

(i) *Very rough.* Near vertical steps and ridges occur on the joint surface.
(ii) *Rough.* Some ridges are visible. Asperities happen. Joint surface feels very abrasive.
(iii) *Slightly rough.* Some asperities happen. Joint surface feels asperous.
(iv) *Smooth.* No asperities. Smooth feeling of joint surface.
(v) *Slickensided.* Visual evidence of polashing exists.

The most important consequence of joint roughness is the display of dilatant behaviour when close, coupled joints are subject to shearing stresses. The nature of fillings govern the shearing stress of open, uncoupled joints and is a related parameter to roughness.

A classification of fillings is out of the scope of Ibis chapter. Anyway, for practical purposes it is necessary lo distinguish between gouge and soft gouge: (i) ‘gouge’ is no filling or filling with a material of high friction (calcite, sand, crushed rock, *etc.);* and (ii) ‘soft gouge’ is filling with a material of low friction (clay, mica, platy minerals, *etc.).*

4.5.2. <u>Separation</u>

Separation is the perpendicular distance between the rock walls of an open joint. If the joint is air or water-filled the separation becomes the aperture of the joint. It the joint has filling the appropriate term is width (ISRM [13]).

Measurement of model apertures is very difficult. RMR classification is very simple.

(i) *Close.* Opening <0.1 mm, which cannot be resolved by naked eye.
(ii) *Moderately open.* Opening < 1 mm. Walls come into contact with a small shearing movement.
(iii) *Open.* Opening 1-5 mm. Walls come into contact after a shearing movement.
(iv) *Very open.* Opening > 5 mm. Walls can remain separated until a big shearing displacement has happened.

The separation of joints governs the displacement necessary to mobilize the joint shear stress. Moreover, open or very open joints can show nondilatant behavior.

Table 8. Classification for Joints Spacing (ISRM, Bieniawski)

Description	Spacing (m)	Rock mass condition
Very wide	> 2	Solid
Wide	0.6-2	Massive
Moderate	0.2-0.6	Blocky/seamy
Close	0.06-0.2	Fractured
Very close	> 0.06	Crushed/shattered

4.5.3. **Persistence**

ISRM [13] classifies the joints as follows.

(i) *Persistent.* Continuous.

(ii) *Subpersistent.* Not continuous but several joints can coalesce lo form a continuous separation surface.

(iii) *Not persistent.* Not continuous.

RMR classification uses only the first and third classes. Subpersistent joints can be classified as not continuous before shearing, and continuous after shearing.

4.5.4. **Weathering of walls**

Table 9 summarizes the recommendations of ISRM [13] for the classification of wall weathering.

RMR classification mentions only grades I, II and IV. Grade V (completely weathered) is equivalent lo grade IV (highly weathered) because in both cases the frictional strength of the joint becomes very low. Grade III (moderately weathered) is an intermediate case.

4.5.5. **Parametric rating**

RMR descriptions of joint condition classes are clear enough. In many cases the field conditions fit clearly into one of the classes. But in some intermediate cases the field evidence does not appear grouped as in the table and some doubts are raised about the correct rating.

The RMR classes represent the frictional component of shear strength of joints and it is possible to establish the appropriate rating through an estimation of the apparent friction angle. Some people prefer lo rate separately each one of the subparameters and add the partial rating in order to obtain an overall rating for condition of discontinuities. Such a method is not encouraged as a general one but it can be used by less-experienced operators and has the advantage of being a checking list. Bieniawski [1] has produced a parametric rating of joint conditions.

Table 10 presents another list of partial parametric ratings for joint conditions which has been us eful lo the author when classifying rock slopes. When using this method each of these four subparameters is assessed and the partial ratings are added to obtain the final rating for the condition of the joints.

4.6 GROUNDWATER

Groundwater conditions can be estimated in RMR geomechanical classification in three different ways: (i) inflow of water in tunnels; (ii) pore pressure ratio; and (iii) general conditions.

For slopes the general conditions are usually sufficiently adequate. The ISRM [13] have proposed a seepage classification which has been ad

apted lo surfacing joints in order lo estimate groundwater conditions. See Table 11.

Table 9. Classification for Wall Weathering (ISRM)

Grade	Term	Decomposed rock (%)	Description
Ia	Fresh	-	No visible weathering
Ib	Fresh	-	Slight discoloration of walls
II	Slightly weathered	< 10	General discoloration
III	Moderately weathered	10-50	Part of rock is decomposed. Fresh rock is a continuum
IV	Highly weathered	50-90	General decomposition of rock. Some fresh rock appears
V	Completely weathered	> 90	All rock is decomposed. Original structure remains
VI	Residual soil	100	All rock is converted to soil. Original structure is destroyed

Table 10. Partial parametric Ratings for Joint Conditions (Romana)

Roughness/filling		**Rating**
Very rough		10
Rough		9
Slightly rough		8
Smooth		6
Slickensided or gouge		5
Soft gouge		0
Separation	***Opening***	***Rating***
Closed	< 0.1 mm	9
Moderately open	0.1-1 mm	7
Open	1-5 mm	5
Very open	> 5 mm	0
Persistence		***Rating***
Not persistent, not continuous		5
Subpersistent		3
Persistent, continuous		0
Weathering	***Grade***	***Rating***
Fresh	I	6
Slightly weathered	II	5
Moderately weathered	III	3
Highly weathered	IV	0
Completely weathered	V	0

Table 11. Groundwater Conditions (ISRM, Romana)

Description	**Unfilled joints**		**Filled joints**	
	Joint	**Flow**	**Filling**	**Flow**
Comp. Dry	Dry	No	Dry	No
Damp	Stained	NO	Damp	No
Wet	Damp	No	Wet	Some drips
Dripping	Wet	Occasional	Outwash	Dripping
Flowing	Wet	Continuous	Washed	Continuous

4.7 ORIENTATIONS

4.7.1. Joints

For each family of joints the orientation data are:

(i) *Dip* (0 to 900)

Measured with clinometer

Measuring error ± 2^o

Normal data scatter ± **5^o**

(ii) *Dip direction* (0 to 360^o)

Measured with geological compass

Measuring error ± 2^o

Normal data scatter ± **5^o**

The most convenient compass is the CLAR type, which can give directly the values of dip and dip direction. It data scatter is the normal one the modal values of dip and dip direction can be used. It data scatter is higher than ± 5^o classification can be done with the modal values and checked with the extreme values. Adjusting factors can be different.

4.7.2. Slope

The orientation data for the slope are difficult to measure. The normal error is ± 5^o (or even more). Classification must be done with the estimated values for slope face dip and dip direction and checked with the extreme values. Adjusting factors can be different.

4.8 BLASTING METHODS

The general conditions for every blasting method are defined, in SMR classification, as follows.

Presplitting

A row of holes is drilled along the final face.

Each hole is carefully marked in the field.

Holes must be parallel (to ± 2%).

Distance between boles is in the order of *50—80* cm.

Charges are decoupled from blasthole walls, leaving air space.

Charges are very light.

Row is fired before the main blast.

Smooth blasting

A row of boles is drilled along the final face.

Each hole is carefully marked in the field.

Holes must be parallel (to ± 2%).

Distance between holes is in the order of 60—100 cm.

Charges are light.

Row is fired after the main blast (sometimes using microdelays). *Normal blasting*

Each blast is done according to a previously fixed scheme.

Each hole is marked in the field.

Charges are kept to the minimum possible.

Blast is fired sequentially, using delays or rnicrodelays.

Deficient blasting

The blasting scheme is only a general one.

Charges are not the minimum possible.

Blast is not fired sequentially.

If blasting is done nominally in one of these categories but some condition is not fulfilled use the adjusting factor of the next lower one. Most production blasts in open pits and quarries are designed to gel maximum fragmentation of rock debris. Usually they must be rated as ‘deficient’ blasting.

5. CASE RECORDS

To establish the SMR classification, 28 slopes with several degrees of instabilities have been registered and classified. Six of these failed completely and have been reexcavated. In several cases the failure was due to weathering and happened a long time after construction (at least one year). Results are shown in Tables 12, 13 and 14, and seem to offer a good concordance with stability classes as predicted by the proposed classification. That gave us enough encouragement to publish the classification in ***1985*** [19 - 21].

Collado and Gili [22] applied the SMR classification to 44 slopes during the geotechnical studies for a new highway in highway 420 (Coll de la Teixeta—Coll Negre) in Tarragona (Spain). The slopes had been recently excavated (one to two years old). Many of them were small in height. All the cases are listed in Table 15.

Figure 4 shows the correlation between the observed behaviour and that estimated from SMR classification. Collado and Gili concluded that ‘there is a good correlation ... with more stable behaviour in slopes with bigger SMR values. However, actual behaviour is slightly better than predicted’. They explain the difference from the fact that ‘SMR tries to evaluate long term behaviour and (we have observed) slopes one to two years old’.

Romana and Izquierdo [23] applied SMR classification to the study of final slopes of a quarry for dolomitic materials at Cartagena harbor. Slopes had different orientations, a maximum height of 35 m and a total length of 260 m. In this area ENAGAS installed a big tank for liquified natural gas (LNG). Previously, a total correction of slopes was done in order lo avoid any instability risk, after a very detailed study. In Table 16 a comparison is shown between actual and SMR predicted stability chasses and correction methods.

Table 12. Case Records / Standing slopes / Plane failure

Case number	Rock	Excavation method	SMR	Class	Failures	Support	Ref. (only 1 st author)
1	Limestone	P	85	I	None	None	Romana (1985)
5	Sandy marl	N	82	I	None	None	Romana (1985)
2	Limestone	P	77	II	Three small blocks W	Toe ditch	Romana (1985)
18	Gneiss	P	74	II	Small wedges d.c.W	None, instrumented	González (1982)
10	Limestone	B	72	II	None	None	Romana (1985)
22	Dolostone	B	64-76	II	Small planes d.c.P	None	Romana (1985)
19	Limestone	SB	61-73	II	None	None, instrumented	Roman Buj (1982)
11 a	Marl	SB	71	II	None. Failure a.w. P	None, see case 11 b	Cedrun (1976)
25 a	Limestone	B	70	II	Small blocks W	Spot bolting	Intecsa (1984)
13	Sandstone/siltstone	N	64	II	Some blocks W	None. Scaling	Uriel (1976)
4	Limestone	DB	63	III	Many blocks W	Insufficient toe ditch	Romana (1985)
14	Marls/limestone	M	59	III	Local problems	None	R. Miranda (1972)
21	Gypsum rock	N	53	III	Some blocks (1 m^3) W	Toe ditch. Fence	Intecsa (1983)
26 a	Claystone/sandstone	B	52	III	Big Wedge (15 m^3) W	Systematic bolts. Net.	Intecsa (1984)
3	Claystone	M	47	III	Surface erosion	Toe ditch	Romana (1985)
8	Sandstone/marl	B	43	III	Many blocks W	Systematic bolting	López (1981)
28	Limestone	DB	40	IV	Many failures P	Concrete wall	Correcher (1985)
20	Gypsum rock	N	31-43	IV	Big failure (100 m^3) W	None	Intecsa (1983)
6	Sandy marl	M	32	IV	Blocks. Mud flows S	None	Romana (1985)
7	Sandstone/marl	B	30	IV	Big plane failures d.c.P.	Systematic bolting. Reexcavation	López (1981)
9	Limestone	B	29	IV	Several blocks (50 m^3) W	None	Romana (1985)

Table 13. Case Records / Toppling

Case number	Rock	Excavation method	SMR	Class	Failures	Support	Ref. (only 1 st author)
16 b	Volcanic tuff/diabase	P	74	II	None	Systematic bolting	R. Oyanguren
25 b	Limestone	B	56	III	Some blocks	Syst. bolting/net	Intecsa (1984)
26 b	Sandstone/claystone	B	56	III	No data	Syst. bolting/2 net	Intecsa (1984)
15 a	Claystone/marls/limestone	M	21-37	IV	Total failure (T)	Reexcavated (to 15b)	R. Miranda (1972)
15 b	Claystone/marls/limestone	M	60	III	Some cracks	None	R. Miranda (1972)
27 b	Slates/grauwacks	M	23	IV	Some failures	Shotcrete/nets/bolting	Intecsa (1984)

Table 14 Case Records / Failed and Rebuilt Slopes /Plane Failure

Case number	Rock	Excavation method	SMR	Class	Failures	Support	Ref. (only 1 st author)
11 b	Marl	SB	36	IV	Almost total a.w. P	Reexcavated to joints	Cedrun (1976)
16 a	Volcanic tuff / diabase	B	30	IV	Big plane failure P	Reexcavated (to 16 b)	R. Oyanguren (1972)
23 a	Marls	B	16	V	Total failure a.w.P	Reexcavated (to 23 b)	Intecsa (1984)
23 b	Marls	B	42	III	Small blocks W	Toe wall/anchors/fence	Intecsa (1984)
24 a	Marls	B	17	V	Total failure a.w.P	Reexcavated (to 24 b)	Intecsa (1984)
24 b	Marls	B	43	III	Small blocks W	Toe wall/anchors/fence	Intecsa (1984)
27 a	Slates/grauwacks	M	17	V	Soil-like failure S	Reexcavated (to 27b)	Intecsa (1984)

Excavation methods: P, presplitting; DB , deficient blasting; SB, smooth blasting; M, mechanical excavation; B, blasting; N, natural slope. Failure: d.c., during construction; W, wedge; a.w., after weathering; T, toppling; P, plane; S, soil-like

Table 15. Slopes studied by Collado and Gili [22]

Heigh (m)	Number	Lithology
42	1	Sandstone and porphyry
33.5	1	Sandstone and quartzite
25	1	Slate and porphyry
16-20	3	Slate and sandstone
12-16	5	Sandstone and slate
8-12	18	Varied
< 8	15	Varied
	TOTAL 44	

The behaviour of the slopes was slightly better than predicted by SMR, with an average difference of 5 to 10 points. In one case the slope was in a worse state than predicted. The correction work was very careful with a daily definition of operations. Corrective measures actually used were those predicted by SMR classification but with less intensity. Overall the predictions were slightly conservative.

6. STABILITY CLASSES

Table 4 shows the different stability classes, and these are summarized below.

Class I	Completely stable. No failures
Class II	Stable. Some block failures
Class III	Partially stable. Planar failures in some joints and many wedge failures
Class IV	Unstable. Planar failures in many joints or big wedge failures
Class V	Completely unstable. Big planar failures or soil-hike failures

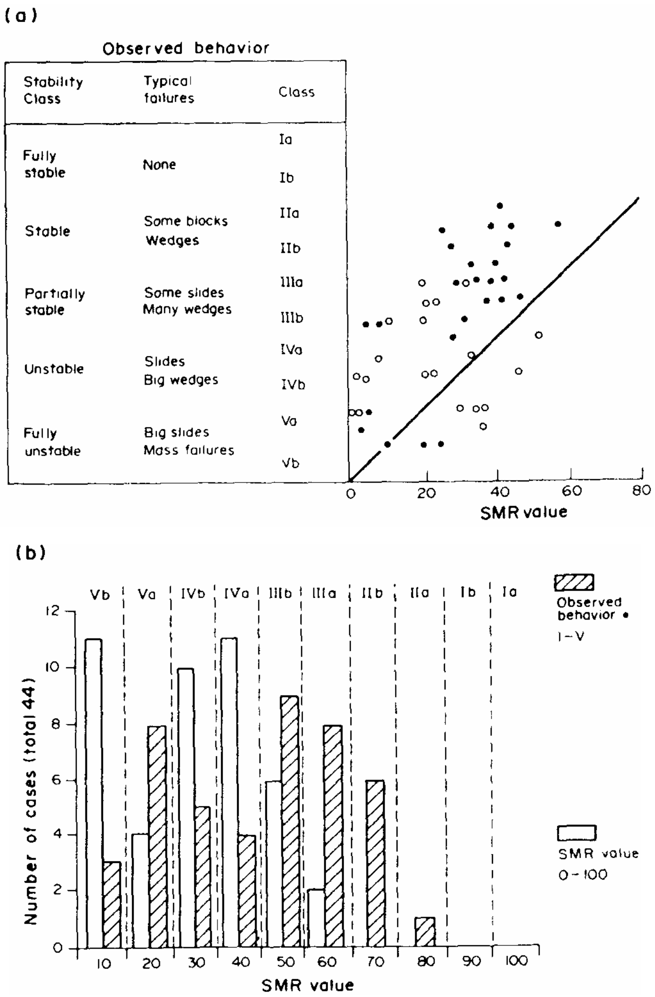

Figure 4. SMR in 44 slopes (1 to 2 years old) in Tarragona (Collado and Gili [22]; (a) observed behaviour against SMR value; (b) histogram of cases for each class

Table 16. Predicted and Actual Behavior of Cap Negre Slopes (Romana and Izquierdo [23])

RMR	Stability class		Correction[(a)]	
	SMR	Actual	SMR	Actual
54-61	III a	II b	sB sS	pS
20-40	IV	III	W	W
55-65	III a – II b	III a	sB pS	pB pS
42-54	III b	III b	sB sG (M)	pB R

The empirically found limit values of SMR for the different failure modes are listed in Table 17. All slopes with SMR values below 20 fail very quickly. No slope has been registered with SMR value below 10. These slopes would not be physically feasible.

7. SUPPORT MEASURES

7.1 GENERAL

Many different remedial measures can be taken to support an unstable slope, or to prevent a believed potential instability. There is not enough technical literature about the real effects of support measures in rock slopes, especially when different measures are adopted simultaneously. On the other hand many case histories document analytically the correction of landslides in soil using deep drainage and/or resistant inclusions in the slopes.

SMR	**Plane/failures**	**Wedge**	
> 75 60-75 40-55 15-40	None None Big Major	None Some Many No	
SMR	**Toppling failures**	**SMR**	**Soil-like failures**
> 65 50-65 30-35	None Minor Major	30 10-30	None Possible

The study of potentially unstable rock slopes is a difficult task requiring careful field work, detailed analysis and good engineering sense in order to understand the relative importance of the several instability factors acting on the slope. No classification system can replace all that work. However, they may be of son utility in indicating the normal limits of use for each class of support measures. The choice between them is out of the scope of the classification system.

The support measures can be grouped in six different classes.

(i) No support	None
	Scaling
(ii) Protection	Toe ditches
	Fences (at toe or in the slope)
	Nets (over the slope face)
(iii) Reinforcement	Bolts
	Anchors

(iv) Concreting	Shotcrete
	Dental concrete
	Ribs and/or beams
	Toe walls
(v) Drainage	Surface
	Deep
(vi) Reexcavation	

From the collected case histories Table 18 presents the more common support measures for each class interval (see also Figure 5).

Normally no support measures are needed for slopes with SMR values of 75-100. There are some stable slopes with SMR values of 65.

Total reexcavation of a slope is a drastic measure, normal in soil slopes, but less practical in rock ones, except in the instability mode is planar through a big continuous joint. It may be adopted in order to reduce its grade, to take away weight in its upper part and/or to add a stabilizing weight at the toe. No totally reexcavated slope has been found with SMR value over 30. No slope has been found with a SMR value below 10. Probably such a low value would imply total and instant instability, the excavation of the slope (even during a very short time) would not be physically feasible.

In a broad sense, the ranges of SMR for each class of support measures are listed in Table 19. Selection of the adequate measures must be made taking into account the prevalent failure mechanism and also the frequency of joints. Two parameters can be useful to quantify frecuency of joints.

(i) Joint spacing, S. The modal value of joint spacing distribution in a family. Frequently the governing joint spacing value corresponds to the joints family which originates the instability.

(ii) Joint volumetric count, J_v. The number of joints per cubic meter, J_v, can evaluated with the formula.

$$J_v = \sum 1/\overline{S_i}$$

where $\overline{S_i}$ is the mean (not the modal) spacings for each joints family. The ISRM 'Suggested Method for the Quantitative Description of Discontinuities in Rock Masses' [13] gives the following:

Table 18. Recommended Support Measures for Each Stability Class

Class	SMR	Support
Ia	91-100	None
Ib	81-90	None. Scaling
II a	71-80	(None. Toe ditch or fence) Spot bolting
II b	61-70	Toe ditch or fence. Nets Spot or systematic bolting
III a	51-60	Toe ditch and/or nets Spot or systematic bolting Spot shotcrete
III b	41-50	(Toe ditch and/or nets) Systematic bolting. Anchors Systematic shotcrete Toe wall and/or dental concrete
IV a	31-40	Anchors Systematic shotcrete Toe wall and/or concrete (Reexcavation) Drainage
IV b	21-30	Systematic reinforced shotcrete Toe wall and/or concrete Reexcavation. Deep drainage
V a	11-20	Gravity or anchored wall Reexcavation

(i) Very often several different support methods are used in the same slope.

(ii) Less usual support measures are in brackets

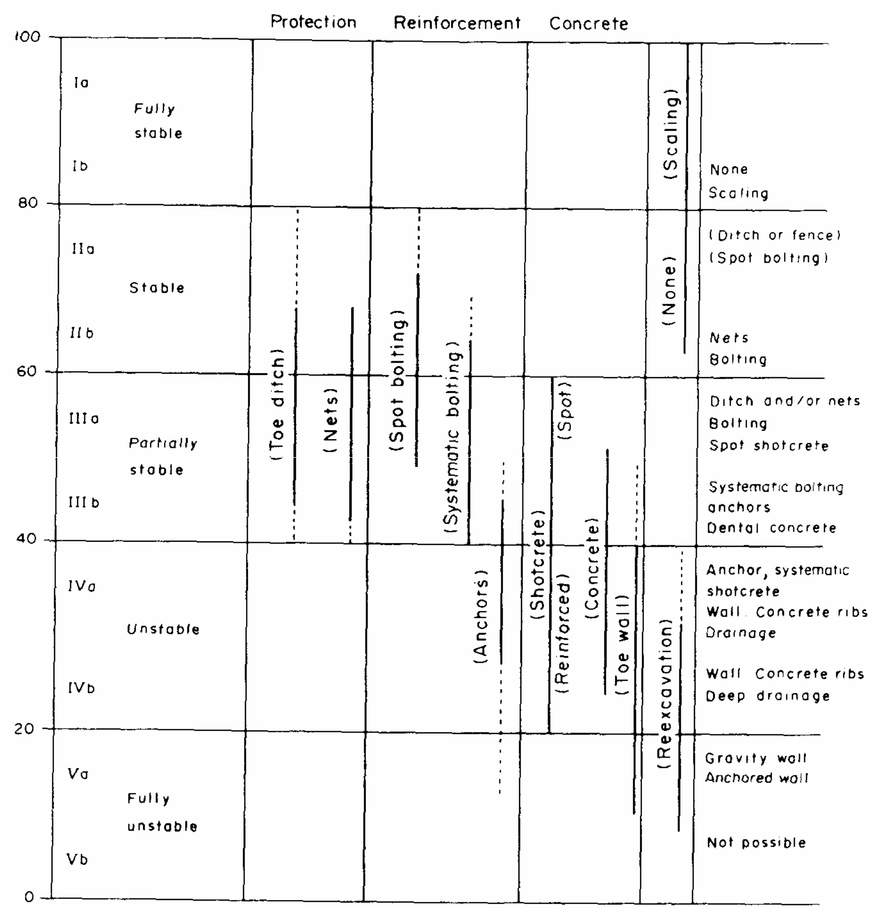

Figure 5. Correction methods according to SMR range

Table 19. Range of SMR for Support Measure Classes

SMR	Support measures
65-100	None. Scaling
45-70	Protection
30-75	Reinforcing
20-60	Concreting
10-40	Drainage
10-30	Toe walls. Reexcavation

Description of block sizes according to J_v.

Description of blocks	J_v (joints m^{-3})
Very large	< 1
Large	1-3
Medium	3-10
Small	10-30
Very small	30-60
Crushed rock	> 60

J_v and RQD can be approximately correlated through the Palmstrom [17] formula

$$RQD = 115 – 3.3\ J_v \quad RQD \leq 100$$

7.2 PROTECTION MEASURES

7.2.1. Toe ditch

Toe ditches are useful to keep fallen rocks out of the road when failures are wedges, planes and/or minor topples.

Ritchie [24] filmed rockfalls in several slopes, identifying three fall modes: (i) direct fall, for slopes 1H:4V and steeper; (ii) rebound, for slopes around 1H:2V; and (iii) roll, for slopes 1H:1V and flatter.

Ritchie also proposed empirical criteria for dimensioning ditches and fences (Table 20). These values have been widely reproduced and quoted. Nevertheless many engineers believe that Ritchie´s values are too big and lead to designs that are too expensive. Whiteside [25] (based on Fookes and Sweeney [26]) has published an abacus reducing Ritchie´s proposed dimensions.

On the other hand Ritchie seems to have worked with slopes in hard rocks, therefore having high rebound coefficients. In softer rocks part of the rebound energy is lost in breakout, and the distances to the slope toe are smaller.

Castañeda [27] has proposed, and used ssuccessfully, a reduction of Ritchie´s criteria for highway slopes in the north of Spain, excavated in marls, lutites, soft sandstones, etc (Table 21). His results are very similar to these given by Whiteside. Ritchie´s rules seem more adequate for slopes when C_o > 25 MPa and $F_4 \leq 0$ (normal blasting). Castañeda´s reductions can be used for slopes when C_o > 25 MPa (soft rock) and $F_4 \leq 0$ (careful blasting).

Table 20. Ditch Dimensions According to Ritchie [24]

Height (m)	1H:4V/1H:3V	1H:2V	Slopes 3H:4V	1H:1V	5H:4V
4.5-9	3.0 x 0.9	3.0 x 0.9	3.0 x 1.2	3.0 x 0.9	3.0 x 0.9
9-18	4.5 x 1.2	4.5 x 1.2	4.5 x 1.8	4.5 x 1.2	3.0 x 1.5 F
18-30	6.0 x 1.2	6.0 x 1.8 F	6.0 x 1.8 F	4.5 x 1.8 F	4.5 x 1.8 F
> 30	6.0 x 1.2	705 x 1.8 F	7.5 x 2.4 F	4.5 x 1.8 F	4.5 x 1.8 F

W width (m); D, depth (m) (W x D), F means that ditch depth can be 1.20 m with a fence to total depth.

Table 21. Ditch Dimensions According to Castañeda [27]

Height (m)	Slope 1H:4V / 2H:3V	Height (m)	Slope 2H:3V/1H:1V
10-25	2.2 x 1.2	6 – 20	2.2 x 1.2
25-40	3.2 x 1.6	> 20	3.5 x 1.8
> 40	3.7 x 2.0		

W width (m); D, depth (m) (W x D)

Table 22. Indicative Conditions for Use of Nets (Romana)

J_v	Type of net	Block weight (kN)
5-10	Reinforced	1.5-5
> 10	Normal	> 1.5

7.2.2. Nets

Nets over the slope are used to avoid free fall of rock pieces. Therefore they are useful for wedge failures and also with minor topples (although in this case securing the net at the top of the slope can be difficult). To avoid breaks of the net caused by the excessive weitht of rock fragments, nets must be used only when slopes have big values of J_v. Table 22 gives some indications about the use of nets in slopes.

7.3 REINFORCEMENT

7.3.1. <u>Bolting</u>

Bolting in slopes is a worldwide used technique, but no specific rules for design and layout of bolts are offered in the technical literature. The following were derived from the author´s experience and are partially inspired by the excellent and concise manual by Schach, Garshol and Heltzen [28], dedicated mostly to underground bolting.

Bolts in slopes are used as a combined immediate and permanent support. The bolt types are detailed below.

(i) *Fuly grouted. Not tensioned*

Normally rebar type (20-25 mm diameter)

Grouted with resin or mortal

Not (or very lightly) tensioned at head.

Sometimes ‘perfo’ type, with a thin, perforated metal tube split longitudinally, filled with mortar.

(ii) *Tensioned*

Normally expansion type (split and wedge).

With a bearing plate

Tensioned at head.

Grouted alterwards with mortar to prevent corrosion.

For the sake of simplicity only the untensioned, fully grouted bolts are referred to in this section. The undergrouted and/or tensioned bolts are best included with the anchors. Bolts are defined then as a ‘passive’ reinforcement, anchors being an ‘active’ one.

Simple passive, grouted bolts have the following characteristics:

Length	Normally 3 to 4 m
	Should reach 1-2 m in solid rock, across the unstabilizing joint
	As a rule of thumb, bigger than height of slope divided by ten
Diameter	Normally 22 mm
Strength	120 to 150 kN

They are very appropiate to support slopes with wedges, planes and/or minor topples.

From the point of view of bolting, rock masses can be classified according to the joint frequency in the following types of rock.

(i) *Blocky, hard rock*

Typical joint spacing over 1 m

Joint volumetric count, J_v = 1-3

Systematic bolting at 3-3.5 m distance

(ii) *Fractured, hard rock*

Typical joint spacing between 0.3 and 1 m

Joint volumetric count, J_v = 3-10

Systematic bolting at 1-3 m distances (three times the prevalent joint spacing).

(iii) *Very fractured, hard rock*

Typical joint spacing smaller than 0.3 m

Joint volumetric count $J_v > 10$.

Systematic bolting at 1 m distance combined with a continuous, thick layer of shotcrete (15-25 cm) if J_v = 10-18.

(iv) *Weathered rock, with open or clay-filled joints*

Bolting to secure hanging blocks, through the joints to sound rock.

Bolting distance and length in a selected pattern to suit rock and joint disposition.

(v) *Soft rocks*

Normally bolts are of limited use in very soft rocks, because they cannot develop full tension.

In soft rocks bolts can be used combined with continuous reinforced thick layers of shotcrete.

The rock conditions for an adequate use of bolting are summarized in Table 23.

Some indications of possible bolt pattern in relation to the SMR stability classes are noted in Table 24.

7.3.2. Anchoring

Anchors are long steel bars which apply an active force in the surface of the slope, transferring it to the ground behind the unstable zone. They introduce a stabilizing force and simultaneously increase shear strength in joints.

Many types of anchors can be used. Their characteristics can be summarized as follows.

(i) Length	Most usual, 12 to 20 m
	Possible, 8 to 30 m
(ii) Strength	Most usual, 700 to 1000 kN
	Possible, 300 to 2000 kN
(iii) Layout	Most usual, one anchors every 10 to 35 m^2
	Disposed in rows and files

Table 23. Conditions for Adequate Use of Bolting (Romana)

Rock strength C_o (MPa)	**Joint volumetric count J_v (joints m^{-3})**	**Bolting**
< 5	> 18	Not adequate
5-25	40-18	Systematic with shotcrete
> 25	3-10	Systematic bolting
> 25	< 3	Spot bolting

Table 24. Indicative Patterns for Rock Bolting in Slopes (Romana)

Class	**SMR**	**Bolt ($S\ m^{-2}$)**	**D (m)**	**Force density ($kN\ m^{-2}$)**	**Shotcrete**
II	65	0.08	3.5	10-12.5	No
II b	65-60	0.11	3.0	13.3-16.6	No
III	60-45	0.40	1.6	48-60	Spot
		0.70	1.2	84-105	Spot or systematic
		1.00	1.0	120-150	Systematic
III b	45-40	1.00	1.0	120-150	Systematic reinforced

D, distance between bolts

(iv) Heads	Isolated concrete monuments (1.00 x 1.00 x 0.50 m)
	Concrete ribs or beams (width 1.00-1.50 m)
	Anchored walls
	Toe walls (anchors can be a complement for gravity toe wall stability)

Anchoring systems are a major support method and therefore their pattern and quantity must be analytically studied in each case, and their behavior monitored after installation. They are very useful for coping with big planar slides, major toppling and general slope instabilities.

A rough guide for the preliminary quantification of the anchoring needs in a slope in presented in Table 25. The author has produced the table, deriving the data from some cases where major nachoring was successful as the principal means of stabilizing the slope.

When anchors are used to help as an additional measure to increase the stability of gravity walls or rib systems, the force density should be in the order of 25-50 kN m^{-2} (minimum 15 kN m^{-2}).

7.4 CONCRETING

7.4.1. Shotcrete

Shotcreting a slope is easy, it can be done quickly, and very often it is a profitable work. Therefore many slopes are shotcreted when the first signs of instability appear. It is difficult to assess the real effect of shotcrete in slopes. Often the shotcrete layer decays with time, cracking and falling. Sometimes a surface net has been installed to prevent shotcrete pieces from falling into the road after cracking.

Spot shotcrete can be useful when local corrections and/or protections are needed (e.g. against overhanging) and when differential erosion can damage a slope. Systematic shotcrete is necessary in slopes supported with systematic bolting when the rock mass is fragmented (joint volumetric mass, J_v = 10-18), and can be used to distribute the forces of isolated anchor heads.

If shotcrete is used as a general protection for erodible or soft rock in a slope care should be taken to ensure that the following rules are observed.

(i) Clear previously the slope (with compressed air and water).

(ii) Use several layers. A convenient layout includes a preliminary surface layer (e = 3 cm), and two protection layers with reinforcement (e = 2 x 10 cm).

(iii) Use short bolts to secure shotcrete to the rock mass.

(iv) Absolutely avoid shotcreting the areas with natural drainage to the slope face in order to avoid developing bigger internal water pressures in joints and/or pores, which can be dangerous.

(v) Try to install drains to alleviate internal water pressures. Experience shows that most of these drains do not work properly, and remain dry even when they are very close to water-bearing cracks in shotcrete.

The beneficial effects of systematic shotcrete are doubtful, and it can be harmful for the natural drainage of the rock mass. Futhermore, the aesthetic effect of shotcrete is very bad, although it can be bettered by using clear pigments in the final layer.

7.4.2. Dental concrete

Dental concrete is adequate for local corrections in generally stable slopes. It can be substituted by mansory. This has advantages when the mansory is formed from the same rock as the slope (similar resistance to weathering and a better, unobtrusive view). In any case, dental concrete must avoid the disturbance of the natural drainage system in rock masses.

7.4.3. Ribs, beams and walls

Concrete ribs and beams can serve as a resistant grid for the slope. Often the crossings include anchors, the support system being a combination of both factors. Toe walls have similar functions and can also be combined with anchors. Fully unstable slopes can be stabilized with gravity walls, with or without anchors. In these cases the force densities of the anchors can be smaller. Continuous walls must include effective provisions for deep drainage of the rock mass.

7.5 DRAINAGE

7.5.1. Surface drainage

The surface drainage can be a great help for the stability of a slope. At the top of a slope water can be ponded in open tension cracks. Water pressure develops, proportionally to the square of crack depth, and is a very dangerous destabilizing force. In the face of the slope the running water may cause erosion in soft zones. This can lead to local instabilities.

Surface drains can be ditches at the top of the slope, more or less parallel to it. Across the face vertical ditches, at regular spacing, can collect the water falling from the upper part, protecting the slope.

Surface drainage must be very well done to be effective. Concrete ditches can crack and inject water into the joints instead of draining the slope. Drainage conduits must be lined preferably with soft and/or extensible materials which can accommodate to the slope deformations. They must be provided with ample and safe evacuation devices.

7.5.2. Deep drainage

Water percolates in rock masses through the joints system. The conductivity of the joints is proportional to the cube of their width. The presence of fill in the joints makes them nonpermeable. Near the surface, joints tend to be open and very permeable. For these reason internal water pressure is a less important cause of instability in rock masses than in soil slopes.

Many soil landslides can be fully corrected (or at least slowed) by internal drainage only. In rock slopes internal drainage must be used in conjunction with other support measures (anchoring and/or walls).

The possible deep drainage systems are as follows.

(i) Horizontal toe drains (‘French’ or ‘Californian’)

Bored horizontally (with a very small inclination) from the slope toe.
Must include filters to prevent suffusion.
Short lived if the slope undergoes deformation.
Very effective to eliminate water pressure from the slope surface.

(ii) Vertical drains

Bored vertically from the slope.

Very effective if there is a perched water level in th slope.

(iii) Horizontal drainage adits

Parallel to the slope.

The most effective measure.

Not usual in civil engineering (except dams).

Deep drainage is only useful when a continuous groundwater level surfaces the slope, a situation which requires a very humid climate and/or joints with big horizontal conductivity. Deep drainage is a good support measure for big planar slides or mass instabilities.

The design details of drainage by subhorizontal (‘Californian’ or ‘French’) drains can be derived from Louis.

(i) Optimal length of drains

0,20 to 0,30 H_w (H_w is the height of the groundwater level over the toe of the slope at a distance of Usually from 6 to 12 m.

(ii) Optimal distance between drains

0,33 to 0,50 the length of the drains

Usually from 2 to 6 m.

(iii) Optimal direction

Theoretically 10^o to 15^o downslope

Usually horizontal.

5^o to 10^o upslope is water flow has to clear eventual debris in the borings.

(iv) Optimal material

Plastic ranured PVC tube.

Geotextile filter around the tube to protect drains against suffusion.

8. CONCLUSIONS

The new method presented, called Slope Mass Rating (SMR), allows the use of the Bieniawski (CSIR) classification for slopes. It requires the same data and gives a forecast of stability problems and support techniques for slopes in each stability class. More research is needed, and will be welcome, to check the proposed classification system.

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