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Rock mass strength by rock mass classification

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ABSTRACT: The strength of a rock mass for foundation purposes is for a large part determined by the discontinuities in the rock mass. Numerical calculations of discontinuous rock masses prove often to be cumbersome and unreliable. Rock mass classification may be an equal or more reliable methodology. The Slope Stability Probability Classification (SSPC) system designed for slope stability may be used for this purpose. The system has been developed during four years of research in Falset, province Tarragona, Spain. The rock slope classification scheme assesses orientation dependent and orientation independent stability. The orientation independent stability assessment leads to a rock mass strength criterion based on classification data, e.g. intact rock strength, discontinuity spacing and discontinuity condition. The criterion is developed in the context of a slope stability classification system, however, there is no reason that the criterion is not also valid for the determination of rock mass strength for other purposes, such as foundations on a discontinuous rock mass. The results of the strength criterion are compared to the results of the 'modified Hoek-Brown strength criterion' and to the rock mass strength as determined by Bieniawski's classification system.

1 INTRODUCTION

In the last decades the study of discontinuous rock mechanics has developed tremendously. For constructions, such as slopes, foundations and shallow tunnels it has been recognised that discontinuities have a major influence on the mechanical properties of a rock mass. This perception has major consequences for the assessment of the engineering behaviour of a rock mass. Calculations for engineering structures in or on a rock mass have to include discontinuity properties. Variations in properties, however, can be considerable along the same discontinuity plane. As there may be hundreds of discontinuities in a rock mass, each with its own variable properties, these, taken together with inhomogeneities in the rock material, require that in order to describe or calculate the mechanical behaviour of the rock mass accurately, a large amount of data is required. Laboratory and field tests are available to obtain discontinuity properties. Testing in large quantities is, however, time consuming and troublesome.

Continuum calculations for engineering structures in or on a rock mass, whether analytical or numerical, cannot be appropriate, as the simplifications needed to present the rock mass as a continuum are so substantial that it is nearly always highly questionable to what extent the final calculation model still represents reality. Discontinuous 'distinct block' numerical calculations can model the discontinuities and calculate the behaviour of a rock mass in all detail, provided that property data are available. Apart from the need to have powerful computers to do the large number of calculations required by the vast quantity of discontinuities, the test data needed for a detailed numerical discontinuous calculation are never available. An often applied practice to avoid these problems is to simplify the discontinuity model, and estimate or guess the properties or to use literature values. To what extent the result is still representative for the real situation is a question that often remains unanswered.

2 ROCK MASS CLASSIFICATION SYSTEMS

An altogether different approach to assess the engineering behaviour of a rock mass is rock mass classification. In a classification system empirical relations between rock mass properties and the behaviour of the rock mass in relation to a particular engineering application, are combined to give a method of designing engineering structures in or on a rock mass. Rock mass classification has been applied successfully for some years in tunnelling and underground mining in, for example, Southern Africa, Scandinavia and Canada (Barton, 1976, 1988, Bieniawski, 1989, Laubscher, 1990). Some rock mass classification systems (Bieniawski, 1989, Hack et al., 1993, 1996, Hoek et al., 1992, Serafim et al., 1983) result in empirical strength criteria for a rock mass.

3 SLOPE STABILITY PROBABILITY CLASSIFICATION (SSPC)

Recently the Slope Stability Probability Classification (SSPC) system has been developed (Hack et al., 1993, 1996). The research for the development of this system has been carried out in northeast Spain. The system has successfully been applied to slope stability problems in the same area as well as in Austria, Costa Rica and the Dutch Antilles (Hack, 1997b). The SSPC rock slope classification scheme assesses orientation dependent and orientation independent stability of a slope. The orientation independent stability of a slope assessment leads to a rock mass strength criterion. The rock mass strength criterion is based on a Mohr-Coulomb model for shear strength in which the cohesion and friction are those of the rock mass and determined by classification of the rock mass properties, e.g. by determining intact rock strength, discontinuity spacing and discontinuity condition.

The classification system is based on a three step classification system and allows for correction for weathering (Hack et al., 1996, 1997a) and excavation disturbance (Hack, 1996). The system classifies rock mass parameters in one or more exposures (step 1). These parameters are corrected for the influence of weathering and excavation disturbance in the exposures and parameters important for the mechanical behaviour of a the rock mass in an imaginary unweathered and undisturbed 'reference' rock mass are calculated. The slope stability assessment thence allows assessment of the stability of the existing or any new slope in the 'reference rock mass', with allowance for any influence of excavation method to be used for the new slope, and future weathering. This procedure allows a rock mass assessment based on rock mass parameters that are independent from local weathering and excavation disturbance as found in the exposures, but allows for the influence of future weathering on the rock mass at the location of the slope and the disturbance caused by the method of excavation used for the slope (Hack, 1996).

4 SSPC SYSTEM APPLIED TO FOUNDATIONS

The Slope Stability Probability Classification (SSPC) system uses a rock mass strength criterion (eq. [5]). The criterion is not necessarily restricted to slope stability and can likely also be used to determine the strength of a rock mass for a foundation. The expressions for rock mass friction and cohesion (eq. [4]) have been established for slopes in which no instability due to the orientation of discontinuities was present. The instability of the slope can then only be caused by overall rock mass instability due to the stress configuration in the slope. The stress configuration causes, for example, new fractures forming in intact rock, opening of existing discontinuities, or movements along discontinuities, generally resulting in a loss of structure of the rock mass (Hack, 1996). These mechanisms are similar as those occurring in a rock mass for foundations.

5 GEOTECHNICAL UNITS IN A ROCK MASS

Theoretically a proper description or geotechnical calculation to determine the behaviour of a rock mass should include all properties in a rock mass including all spatial variations of the properties. This would be unrealistic and is also not possible without disassembling the rock mass. Therefore a standard procedure is to divide a rock mass into homogeneous geotechnical units. In practice, such homogeneity is seldom found and material and discontinuity properties vary within a selected range of values within the unit. The allowable variation of the properties within one geotechnical unit depends on: 1) the degree of variability of the properties within a rock mass, and 2) the context in which the geotechnical unit is used. A rock mass containing a large variation of properties over a small distance necessarily results in geotechnical units containing larger variations in properties because it is impossible to establish all boundaries between the various areas with different properties within the rock mass with sufficient accuracy. The smaller the allowed variability of the properties in a geotechnical unit the more accurate the geotechnical calculations can be. Smaller variability of the properties of the geotechnical units involves, however, collecting more data and is thus more costly. The higher accuracy obtained for a calculation based on more data has, therefore, to be balanced against the economic and environmental value of the engineering structure to be built and the possible risks for the engineering structure, environment or human life. For the foundation of a highly sensitive engineering structure (e.g. nuclear power station) the variations allowed within a geotechnical unit will be smaller than for a geotechnical unit in a calculation for the foundation of an ordinary house. No standard rules are available for the division of the rock mass into geotechnical units and this transformation depends on experience and 'engineering judgement'. Features such as changes in lithology, faults, shear zones, etc. will, however, be often the boundaries of a geotechnical unit.

6 DETERMINATION OF ROCK MASS PARAMETERS

The determination of discontinuous rock mass properties within a geotechnical unit is mostly not highly accurate. The determination of the properties can therefore be done with relatively simple means in the field (Hack, 1996). In the SSPC system the rock mass properties intact rock strength, and discontinuity spacing and condition are determined. These are determined separately for each geotechnical unit.

IRS intact rock strength range (MPa)	'simple means' test
< 1.25	crumbles in hand
1.25 - 5	thin slabs break easily in hand
5 - 12.5	thin slabs break by heavy hand pressure
12.5 - 50	lumps broken by light hammer blows
50 - 100	lumps broken by heavy hammer blows
100 - 200	lumps only chip by heavy hammer blows
> 200	rocks ring on hammer blows - sparks fly

note: the hammer should be a normal geological hammer of about 1 kg.

Table 1. Determination of intact rock strength (IRS) (modified after Burnett (1975).

6.1 Intact rock strength

Intact rock strength is established in the field by 'simple means' following Table 1. The method has extensively been tested and compared to strength determination by laboratory unconfined compressive strength and point load strength tests. The assessment in the field by 'simple means' is obviously partly subjective. However, the strengths determined by 'simple means' by about 50 different people showed that the results of the 'simple means' field tests are at least comparable to the quality of results obtained by the laboratory tests (Hack, 1996).

6.2 Spacing and condition of discontinuities

The orientation of discontinuities in combination with the shear strength along discontinuities determines the possibility of movement along discontinuities and, hence, has a major influence on the deformation of a rock mass. A rock mass classification system should thus include one or more parameters describing the influence of discontinuities. Considerable differences exist in the methodologies used to incorporate shear strength of discontinuities in the existing classification systems. A basic problem is that shear strength along discontinuities is not fully understood. Some deterministic and empirical models do exist to calculate shear strength from discontinuity characteristics (form of discontinuity, type of infill material, etc.), however, most of these methods are not without criticism and do not always work in all circumstances. The literature describing shear strength of discontinuities is extensive and often contradictory. For a classification system the emphasis is therefore on parameters that can be determined in the field without extensive testing.

The shear strength of a discontinuity is influenced by a number of discontinuity parameters. To be able to determine parameters for a classification system describing discontinuity properties it is necessary to define whether discontinuities can be incorporated in a classification system as belonging to a 'set' or should be treated as a single phenomenon. Determining the parameters for a 'set' of discontinuities requires a form of averaging of the parameters of individual discontinuities. This can be done by various methodologies. The rock mass strength criterion described in this article is only applicable for the strength of a rock mass; it is not applicable to the strength of a single discontinuity. The strength of a single discontinuity can be calculated by other means, for example, the 'sliding criterion' (Hack et al., 1995). Hence, if a single discontinuity has an overriding influence on the rock mass behaviour the criterion described in this article is not applicable.

The average orientation of a discontinuity set can be found mathematically or by stereo projection methods and subsequent contouring (Davis, 1986, Hoek et al., 1981, Mardia, 1972, R.D. Terzaghi, 1965). The characteristic properties of each discontinuity set are the average of the properties of each measured discontinuity belonging to that set. A disadvantage of these methods is that it may be difficult to distinguish between the different discontinuity sets. Furthermore, an important discontinuity set may be missed out or underrated in importance because the discontinuity spacing is large. Such a discontinuity set may be masked by a less important but far more often measured discontinuity set. This and other errors which may affect the results of stereographic projection methods to determine discontinuity sets and orientations are, in extenso, discussed by R.D. Terzaghi (1965).

Making visually an inventory of the different discontinuity sets (based on orientation, spacing and the character of the discontinuity, e.g. infill, roughness, etc.) is a more proper method. A mean orientation value for a discontinuity set can then be calculated by using only those discontinuities that belong to the discontinuity set in a stereo plot or by a vector analysis. For this method separate scanlines for each discontinuity set can be oriented in order to cross a maximum number of the discontinuities of the set being measured. Alternatively, all discontinuities belonging to a set in the whole exposure or in part of the exposure can be measured and analyzed. Experience shows, however, that scanlines or measuring large quantities of discontinuities in a part of the exposure are still likely to be done only on easily accessible parts of the exposure.

The distinction of different discontinuities or discontinuity sets and the determination of the characteristic orientation, spacing and parameters describing the shear strength can be best done by a studied assessment. Discontinuities within an exposure and within a geotechnical unit should first be grouped visually into sets. The discontinuity properties and parameters of each set can then be measured at an easy accessible location.
In a studied assessment to determine discontinuity properties the discontinuities that are representative for the set are visually selected. In this selection is incorporated the whole exposed area (as this selec-

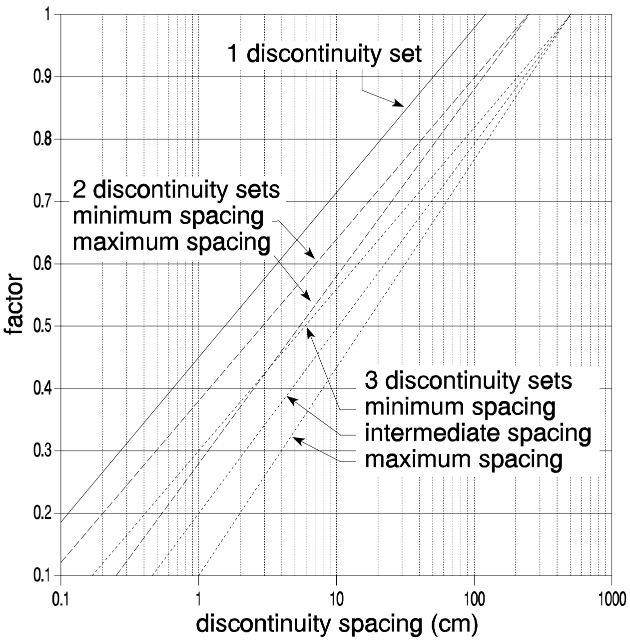
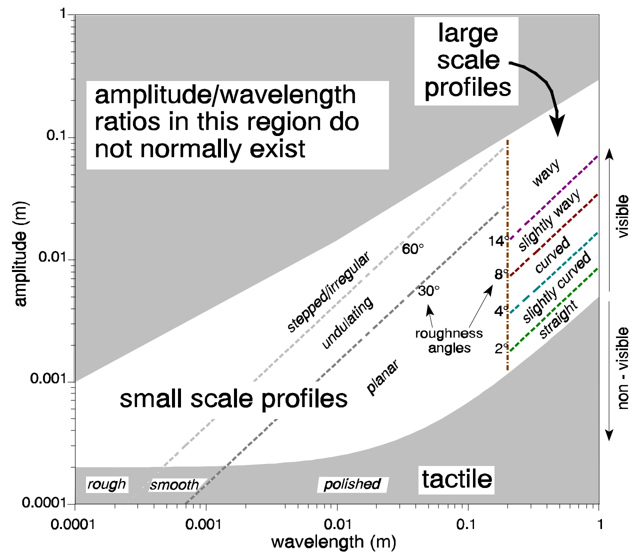


Fig. 1. Discontinuity spacing factors (after Taylor, 1980).



For small amplitudes and wavelengths the roughness is of a triangular/sawtooth form whereas with larger amplitudes and wavelengths the roughness changes to a more sinusoidal form. Lustre is not included in the boundary non-visible to visible roughness. The boundaries in the graph are dashed as these are not exact.

Fig. 2. Interpretation of regular forms of roughness as function of scale and angle.

tion is done visually it does not matter whether the discontinuity is accessible or not) and the character of the discontinuity (infill, roughness, etc.). After the selection the properties of the selected discontinuities are measured in detail in pre-selected locations. In the opinion of the author based on experience during former work and during this research this method gives an equal or better result than the results of extensive measurements of discontinuities for a statistical analysis. If extensive amounts of measurements of discontinuity properties and parameters have to be done, they are always done on a part of the exposure that is (easily) accessible whether representative for the rock mass or not. The same observations have been made by other researchers (Gabrielsen, 1990). It may be thought that a studied assessment for the determination of discontinuity properties would not be accurate enough, but it should be kept in mind that the variation of discontinuity properties in one discontinuity set is often so large that a high accuracy is not very important (ISRM, 1978b, 1981a).

Geological and structural geological approaches can be used to determine the properties and parameters at locations where the rock mass is not exposed.
It should be noted that borehole cores show only a very small part of a discontinuity surface and that consequently the determination of properties based on borehole cores may be less accurate.

6.3 Spacing of discontinuities in a rock mass

In the research for the SSPC system three options have been tested how discontinuity spacing of multiple discontinuity sets in a rock mass could be incorporated in the strength of a rock mass. These are: the spacing of the discontinuity set with the minimum spacing, as, for example, used in the RMR classification system (Bieniawski, 1989), the average of all spacings of all discontinuity sets, and a spacing factor calculated following Taylor (1980) based on the three discontinuity sets with minimum spacings. The results of the comparison of the three methods showed that the methodology following Taylor gave the best results. The graphical representation is shown in Fig. 1. The parameter is calculated for a maximum of three discontinuity sets with the lowest spacings. The spacing factor is:

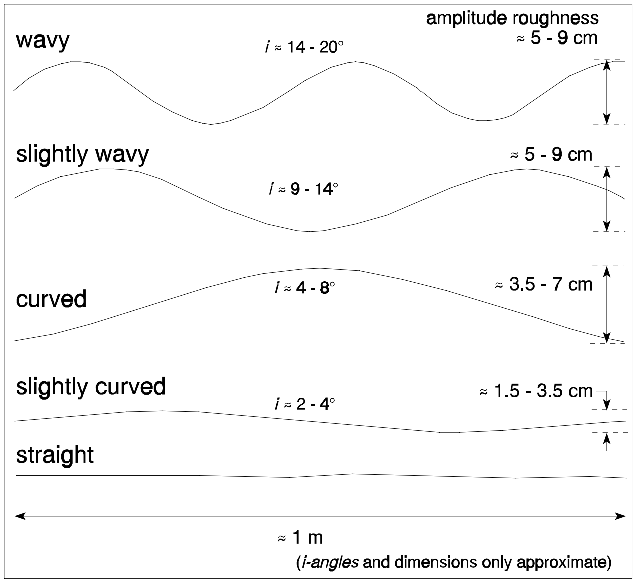


Fig. 3. Large scale roughness profiles.

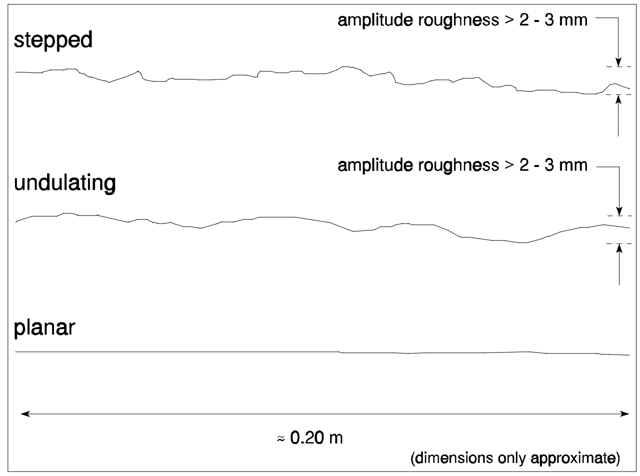


Fig. 4. Small scale roughness profiles.

$$
\begin{aligned}
SPA &= factor_{maximum} * factor_{intermediate} \\
&\quad * factor_{minimum} \\
&\quad (for\ three\ dscontinuity\ sets) \\
SPA &= factor_{maximum} * factor_{minimum} \\
&\quad (for\ two\ discontinuity\ sets) \\
SPA &= factor \\
&\quad (for\ one\ discontinuity\ set)
\end{aligned}
\qquad \textbf{[1]}
$$

6.4 Determining properties representing shear strength of a discontinuity

The condition of discontinuities is determined for each discontinuity set by visually observing the discontinuity. The properties large scale roughness (following the examples in Fig. 3), small scale roughness, infill, and the presence of karst along a discontinuity set are established for each discontinuity set following Table 2. The small scale roughness factors in Table 2 are a combination of visible roughness on an area of about 20 x 20 cm^2, e.g. ’stepped’, ’undulating’ and ’planar’ (following the examples in Fig. 4), with the material roughness that can be established by touch, e.g. ’rough’, ’smooth’ and ’polished’ (tactile roughness). The different roughness parameters are visualized in Fig. 2.

If discontinuities are non-fitting the contribution of the roughness to the shear strength is reduced and depends on the amount of non-fitting. For large scale roughness only an estimate can be made how far the contribution of the roughness to the shear strength is reduced due to non-fitting and the roughness characterization for large scale roughness in Table 2 should be reduced accordingly (normally no large scale shear tests can be done). For small scale roughness a similar procedure may be followed, however, also tilt or shearbox tests can be done and converted in a roughness characterization (Hack et al., 1995). If a discontinuity is completely non-fitting the shear strength depends only on the material roughness, e.g. ’rough’, ’smooth’ and ’polished’. Such a discontinuity would be characterized by ’planar’ for small scale visible roughness and ’straight’ for large scale roughness.

The condition factor for a discontinuity (*TC*) equals:

The discontinuity condition (TC) can be related to shear friction along the discontinuity (Hack et al., 1995).

6.5 Overall condition of discontinuity sets in a rock mass

Several options have been tested to incorporate the overall discontinuity properties describing the influence of the condition of multiple discontinuity sets in a rock mass. Options tested included the average

Roughness large scale (**Rl**)	wavy		:1.00
	slightly wavy		:0.95
	curved		:0.85
	slightly curved		:0.80
	straight		:0.75
Roughness small scale (**Rs**) (on an area of 20 x 20 cm^2)	rough stepped/irregular		:0.95
	smooth stepped		:0.90
	polished stepped		:0.85
	rough undulating		:0.80
	smooth undulating		:0.75
	polished undulating		:0.70
	rough planar		:0.65
	smooth planar		:0.60
	polished planar		:0.55
Infill material (**Im**)	cemented/cemented infill		:1.07
	no infill - surface staining		:1.00
	non softening & sheared material, e.g. free of clay, talc, etc.	coarse	:0.95
		medium	:0.90
		fine	:0.85
	soft sheared material, e.g. clay, talc, etc.	coarse	:0.75
		medium	:0.65
		fine	:0.55
	gouge < irregularities		:0.42
	gouge > irregularities		:0.17
	flowing material		:0.05
Karst (**Ka**)	none		:1.00
	karst		:0.92

notes:
1) For infill 'gouge > irregularities' and 'flowing material' small scale roughness = 0.55
2) If roughness is anisotropic (e.g. ripple marks, striation, etc.) the roughness is the average of the minimum and maximum roughness along the discontinuity plane.

Table 2. Factors for characterization of discontinuity condition.

of the condition (*TC*) of all discontinuity sets in the rock mass, the condition of the discontinuity set with the minimum condition (*TC*), and a weighted mean of the condition (*TC*) of the three discontinuity sets with minimum condition. A weighted mean of the condition of the three discontinuity sets with minimum condition weighted against the spacing of the discontinuity set worked out to be the most satisfying (eq. [3]).

$$CD = \frac{\frac{TC1}{DS1} + \frac{TC2}{DS2} + \frac{TC3}{DC3}}{\frac{1}{DS1} + \frac{1}{DS2} + \frac{1}{DS3}} \qquad [3]$$

TC1, TC2 and TC3 are the discontinuity condition (TC) of three discontinuity sets (1, 2 and 3)
DC1, DC2 and DC3 are the discontinuity spacing for each set (1, 2 and 3)

6.6 SSPC rock mass friction and cohesion, and failure criterion

The rock mass friction and cohesion of a rock mass have been obtained by optimizing a Mohr-Coulomb failure criterion with the intact rock strength (*IRS*), spacing (*SPA*) and condition of discontinuities (*CD*). The rock mass friction and cohesion are defined as:

$$\begin{gathered}\phi'_{rock\ mass} = \\ IRS * 0.2417 + SPA * 52.12 + CD * 5.779 \\ coh'_{rock\ mass} = \\ IRS * 94.27 + SPA * 28629 + CD * 3593\end{gathered} \qquad [4]$$

IRS = intact rock strength
SPA = spacing factor following Taylor
CD = weighted mean of condition of discontinuities

Water pressures are not expected to have been of importance in the slopes that have been used for the development of the SSPC system (Hack, 1996). The rock mass cohesion and friction are therefore defined in terms of effective stresses.

The SSPC failure criterion equals:

$$\begin{gathered}\sigma'_1 = 2 * coh'_{rock\ mass} * \tan\left(45° + \frac{\phi'_{rock\ mass}}{2}\right) \\ + \sigma'_3 * \tan^2\left(45° + \frac{\phi'_{rock\ mass}}{2}\right)\end{gathered} \qquad [5]$$

σ'_1, σ'_3 = *major respectivily minor effective principal stress at failure*

7 COMPARISON OF RESULTS OF SSPC SYSTEM, BIENIAWSKI, AND THE MODIFIED HOEK-BROWN FAILURE CRITERION

The strength of a confined rock mass under a compressive stress is calculated according to the Mohr-Coulomb failure criterion with rock mass cohesion and friction determined with the SSPC system. This strength is compared to the strength of a rock mass calculated according to the Mohr -Coulomb failure criterion with the rock mass cohesion and friction determined with the RMR system (Bieniawski, 1989, Serafim et al., 1983), and is

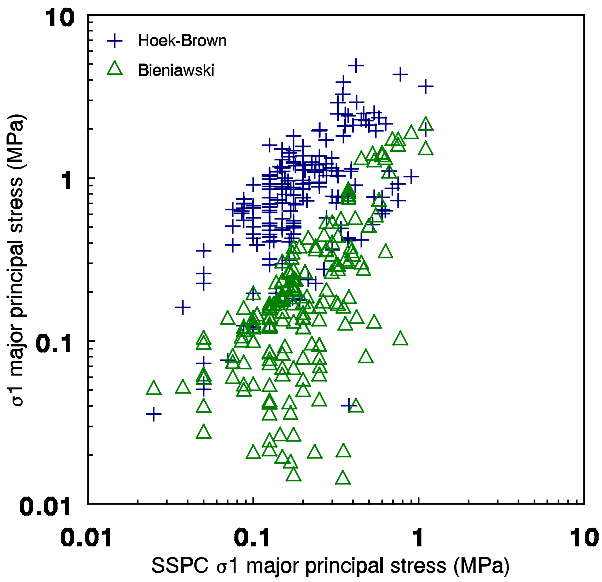


Fig. 5. Comparison of major effective principal stress values.

Weathering	
Degree of rock mass weathering following British Standard for rock mass weathering (BS5930; 1981)	*WE*
fresh	1.00
slightly	0.95
moderately	0.90
highly	0.62
completely	0.35

note: the factors for highly and completely weathered are based on few exposures mainly in granite and granodiorite, and may be less reliable than those for other degrees of weathering.

Table 3. Correction values for the degree of rock mass weathering (*WE*).

compared to the strength of a rock mass as calculated with the 'modified Hoek- Brown failure criterion' (Hoek et al., 1992).

The 'modified Hoek-Brown failure criterion' for rock masses incorporates a simplified rock mass classification system (Hoek et al., 1992). The failure criterion is formulated as follows:

$$\sigma_1' = \sigma_3' + \sigma_c * \left(m_b * \frac{\sigma_3'}{\sigma_c} \right)^a \qquad [6]$$

σ_1', σ_3' = *major respectivily minor effective stress at failure*
σ_c = *intact rock strength*
m_b *and a are parameters describing the rock mass material and structure, and surface condition of discontinuities*

For the comparison the data of the slopes are used that also have been used for the development of the SSPC system. The absolute values for major and minor principal stresses resulting from the three criteria is not very important but should be in the same order as the that exists in reality in the slopes in the research area and should be the same for the calculations of the different criteria. The stress configuration compared, is representative for the stress configurations at a point in the rock mass in the slope, located at about the same level as the toe of the slope but at some distance behind the slope face. The major and minor principal stress values at failure are calculated only for the purpose of the comparisons.

The comparison is for effective stresses. Therefore RMR ratings for a dry rock mass have been used.

The major principal stress values at failure from the SSPC system correlate with the major principal total stress values at failure from the RMR system, and from the 'modified Hoek-Brown failure criterion' (Fig. 5). The major principal stress values following the 'modified Hoek-Brown criterion' are mostly higher than the corresponding values following the SSPC system. The values found following the Mohr-Coulomb failure criterion with rock mass cohesion and friction following Bieniawski correlate for most rock masses, however, show large differences for other.

8 LOCAL INFLUENCES, WEATHERING AND METHOD OF EXCAVATION

The influence of local influences such as weathering and the damage due to the method of excavation may be incorporated analogous to the methodology used in the SSPC system for slope

METHOD OF EXCAVATION (*ME*)			
natural/hand-made			1.00
pneumatic hammer excavation			0.76
blasting	pre-splitting/smooth wall		0.99
	conventional blasting with the following result:	good	0.77
		open discontinuities	0.75
		dislodged blocks	0.72
		fractured intact rock	0.67
		crushed intact rock	0.62

Table 4. Correction factors for the method of excavation (*ME*).

stability assessments. A three- step approach should be used also for foundations. The three steps consist of the characterization of the 'exposure' rock mass, establishment of a fresh and undisturbed 'reference' rock mass and finally the conversion of the parameters that characterize the 'reference' rock mass into parameters that characterize the 'foundation' rock mass. The 'exposure' rock mass is first divided into geotechnical units. Then for each geotechnical unit the rock mass parameters are determined and converted into parameters for the 'reference' rock mass by correction for local weathering in the exposure characterized (Table 3) (Hack et al., 1997a) and for the damage due to the method of excavation used to make the exposure (Table 4) (Hack, 1996). The rock mass cohesion and friction of the reference rock mass are:

$$\begin{aligned} \phi_{reference} &= \phi_{exposure} \;/\; \left(WE_e * ME_e\right) \\ coh_{reference} &= coh_{exposure} \;/\; \left(WE_e * ME_e\right) \end{aligned} \qquad [7]$$

WE_e = *degree of weathering of rock mass in exposure*

ME_e = *method of excavation used for the exposure*

The 'reference' rock mass thus describes the geotechnical units in an unweathered state prior to excavation. The parameters characterizing the 'reference' rock mass can be compared from different exposures and can be combined or averaged. The parameters that characterize the 'foundation' rock mass are obtained by correction of the parameters that characterize the 'reference' rock mass for the damage due to the method of excavation to be used for excavation of the rock mass for the new foundation and taking into account present and future weathering:

$$\begin{aligned} \phi_{foundation} &= \phi_{reference} * \left(WE_f * ME_f\right) \\ coh_{foundation} &= coh_{reference} * \left(WE_f * ME_f\right) \end{aligned} \qquad [8]$$

WE_f = *correction for present and future weathering of the rock mass of the foundation*

ME_f = *method of excavation to be used for the foundation*

9 DISCUSSION

The SSPC system has been designed in a particular region in a particular climate with particular types of rocks and rock masses, etc.. As for all empirical systems, using the SSPC system on rock masses under conditions and in areas that are very different implies a risk. The SSPC system is, however, based on a large number of different rock masses with a wide variety of rock materials.

Susceptibility to weathering is a major factor in determining the strength of a rock mass at the end of the engineering lifetime of a foundation on a rock mass prone to weathering within the engineering lifetime of the foundation. The SSPC system quantifies the future strength of a rock mass if the future degree of rock mass weathering can be determined. This methodology is independent of the climate.

The strength of a rock mass consisting of a highly inhomogeneous, intensively folded or faulted rock masses present a special problem. The rock mass should be divided in geotechnical units in which the rock mass properties are broadly homogeneous. The rock mass strength properties should be calculated per geotechnical unit. If the definition of geotechnical units with a suitable small range of allowed values for properties, becomes impossible due to small scale inhomogeneity, folding or faulting, the worst case rock mass parameters can be used, although this likely leads to a too conservative assessment.

Rock types that are deformed very easily (gypsum, salts, etc.) have been used for the design of the SSPC system. The stability of the slopes in rock masses containing gypsum is, however, more governed by erosion and weathering (in particular solution of gypsum) than by deformation of the rock material. The SSPC system cannot be used if the strength of the rock mass is governed by deformation of the intact rock.

10 CONCLUSIONS

The SSPC system results in a better assessment of slope stability than other slope stability classification systems because of the three-step approach that allows for the incorporation of past and future weathering and the damage due to excavation methods, and the absence of ambiguous or difficult to measure parameters like RQD, water and elaborate testing (UCS, shearbox tests, etc.). The repeatability and reliability of the characterization is generally good because difficult to measure or ambiguous parameters are not required. It is likely that the same is applicable if the SSPC system is used for the determination of the strength of a rock mass in foundation engineering.

The major principal total stress values at failure from the SSPC system correlate with the major principal total stress values at failure from the RMR system and with the values derived from the 'modified Hoek-Brown failure criterion'.

The Slope Stability Probability Classification (SSPC) system is developed based on data from 184 stable and unstable slopes. The amount of data and the fact that the data were collected by different persons at different times eliminates a designer bias in the system.

An advantage of a rock mass strength classification system derived from slope stability is that, generally, in slope stability problems the rock mass can extensively be studied. The rock mass is normally assessable and can be investigated in detail. Virtually always the slope and the rock mass the slope has been made in stay assessable after the slope has been made. This in contrary to rock masses around tunnels or rock masses for foundations. The rock mass may be assessable at the time of building a tunnel or foundation, however, often after construction a tunnel is covered with a support lining and a foundation is covered (nearly always) with backfill. This prohibits the study of the behaviour of the rock mass after the tunnel or foundation has been made. A system for strength classification of a rock mass based on slopes may therefore be more reliable.

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