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A new approach to rock slope stability - a probability classification (SSPC)

R. Hack^{1, M}, D. Price² and N. Rengers¹

- (1) Section Engineering Geology, Centre for Technical Geosciences, International Institute for Aerospace Survey and Earth Sciences (ITC), Delft, The Netherlands
- (2) Formerly at the Technical University, Delft, The Netherlands
- ☑ E-mail: <u>hack@itc.nl</u> Phone: +31-15-2748847 Fax: +31-15-2623961

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D. Price is deceased.

Abstract. The newly developed system presented in this paper is based on a three-step approach and on the probabilistic assessment of independently different failure mechanisms in a slope. First, the scheme classifies rock mass parameters in one or more exposures and allowance is made for weathering and excavation disturbance. This gives values for the parameters of importance to the mechanical behaviour of a slope in an imaginary, unweathered and undisturbed 'reference' rock mass. The third step is the assessment of the stability of the existing slope or any new slope in the reference rock mass, taking into account both method of excavation and future weathering. From the large quantity of data obtained in the field, the Slope Stability Probability Classification (SSPC) system has been proposed, based on the probabilities of different failure mechanisms occurring. Developed during 4 years of research in Falset, Tarragona province, Spain, it has been used with good results in Austria, South Africa, New Zealand and the Dutch Antilles.

Résumé. Le nouveau système de classification présenté dans ce papier est basé sur une approche en trois étapes et sur une évaluation probabiliste de différents mécanismes de rupture de pentes.

D'abord, les paramètres du massif rocheux sont mesurés sur un ou plusieurs affleurements, tout en considérant les effets de l'altération et des perturbations du massif résultant de l'excavation. Cette démarche permet ensuite d'obtenir les valeurs des paramètres jouant un rôle important dans le comportement mécanique d'une pente pour un massif rocheux non altéré et non perturbé par le processus d'excavation. La troisième étape est l'évaluation de la stabilité des pentes dans le massif rocheux de référence, prenant en compte à la fois la méthode d'excavation et l'altération future. A partir du nombre important de données obtenues sur le terrain, le système de classification probabiliste de stabilité des pentes (SSPC) a été proposé, basé sur les probabilités d'occurrence de différents types de mécanismes de rupture. Développé pendant quatre ans à Falset, dans la province de Tarragone (Espagne), il a été utilisé avec de bons résultats en Autriche, Afrique du Sud, Nouvelle Zélande et aux Antilles néerlandaises.

Keywords. Slope stability - Probability classification - SSPC

Mots clés. Stabilité des pentes - Mécanismes de rupture de pente - Analyse probabiliste - SSPC

Introduction

In the last decades, knowledge of the behaviour of discontinuous rock masses has developed tremendously. For constructions such as slopes, foundations and shallow tunnels, it has been recognized that discontinuities have a major influence on the mechanical properties of a rock mass. This perception has had consequences for the assessment of the engineering behaviour of a rock mass. Calculations for engineering structures in or on a rock mass must include discontinuity properties. Variations in properties can be considerable along the same discontinuity plane, however. 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 may be used to obtain discontinuity properties. However, testing in large quantities is both time-consuming and troublesome.

Discontinuous 'distinct block' numerical calculations can model the discontinuities and calculate the behaviour of a rock mass in 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 values from the literature. To what extent the result is still representative for the real situation is a question that often remains unanswered.

Existing rock mass classification systems for slopes

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 (Barton 1976, 1988; Bieniawski 1989; Laubscher 1990). Some rock mass classification systems developed originally for underground excavations have been used for slopes (Barton et al. 1974; Bieniawski 1989) or have been modified for slopes (Haines and Terbrugge 1991; Romana 1985, 1991; Selby 1980, 1982). A system specially designed for slope stability has been developed by Shuk (1994).

The calculation methods and parameters in existing slope stability classification systems have been analysed and the systems used to establish the stability of existing slopes (Hack <u>1998</u>). Generally, all systems include parameters for slope geometry, intact rock strength, discontinuity spacing or block size and parameters related to the shear strength along discontinuities. Some systems include the presence of water or water pressures, deformation of the rock and rock mass, susceptibility to weathering and method of excavation. Use of the existing classification systems has shown that some parameters are difficult or impossible to measure (for example, water pressures and deformation of rock masses). Most systems present the final stability as a single point value with a description (Barton et al. <u>1974</u>; Barton <u>1976</u>; Bieniawski <u>1989</u>; Haines and Terbrugge <u>1991</u>; Selby <u>1980</u>, <u>1982</u>). This can give results that are difficult to appreciate. Parameters influence the stability rating for a slope whose instability may be caused by a physical mechanism that is independent of those parameters. For example, intact rock strength is used to calculate the stability rating, while a slope is unstable because of sliding on a discontinuity with a thick clay infill. Hence, intact rock strength is of no importance for the stability or instability of that slope.

A new approach: slope stability probability classification (SSPC)

Expressions for uncertainty in establishing rock mass properties and for variation of properties (Nilsen 2000) and the applicability of the calculation method are also absent in existing rock mass classification systems, although they are of fundamental importance in establishing the safety of a slope design. Another important problem identified in existing systems is that generally no distinct differentiation is made between the rock mass in the exposures used for the classification and the rock mass in which a slope is to be made. Local influences such as weathering and method of excavation may be the cause of major differences. The ravelling type of failure of slopes is again generally not considered in

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classification systems, although rock mass classification is the only feasible option for the predicting this type of failure (Maerz $\frac{2000}{2}$).

For these reasons and the generally unsatisfactory results obtained with existing rock mass classification systems, a new classification system for slope stability assessment has been developed (Hack <u>1998</u>). The concept of this newly developed system is based on:

- 1. The introduction of the principle of a three-step classification system to describe the 'exposure', 'reference' and 'slope' rock mass.
- 2. The assessment of stability by determining the probability of the occurrence of different failure mechanisms instead of a single-point rating value.
- 3. Unambiguous and simple procedures for collection of data in the field.

Three-step classification system

The SSPC system considers three rock masses:

- 1. The rock mass in the exposure the 'exposure rock mass' (ERM).
- 2. The rock mass in an imaginary, unweathered and undisturbed condition prior to excavation the 'reference rock mass' (RRM).
- 3. The rock mass in which the existing or new slope is to be situated the 'slope rock mass' (SRM).

Rock mass parameters of importance are described and characterized in an exposure, resulting in the 'exposure rock mass'. Local influences on the parameters measured in the exposure such as weathering and the disturbance due to the excavation method used to create the exposure are then taken into account. This converts the parameters for the ERM to those of the theoretical fresh rock mass (Fig. <u>1</u>) that exists below the zone of influence of weathering and other disturbances - the 'reference rock mass' (RRM).



1: natural exposure made by scouring of river, moderately weathered; 2: old road, made by excavator, slightly weathered; 3: new to develop road cut, made by blasting, moderately weathered to fresh.

Fig. 1. Sketch of exposures in rock masses with various degrees of weathering and different types of excavation, and indicating the concept of the 'reference rock mass'

This conversion is made with the aid of correction parameters: the exposure-specific parameters (Fig. <u>2</u>). By this technique, parameters of material in the same geotechnical unit that show different degrees of weathering and different degrees of excavation disturbance are brought back to parameters reflecting their original basic geotechnical properties.





The actual stability assessment is made in the 'slope rock mass' (SRM). This is derived from the 'reference rock mass' (RRM) by adjustment of the parameters of the RRM with the slope-specific parameters. Slope-specific parameters are correction parameters for the influence of future weathering within the engineering lifetime of the slope and the influence of the method of excavation to be used. The ERM and SRM are the same if an existing slope is examined and future weathering is not considered.

Research area

The research for the development of the classification system was done in the area around Falset in northeast Spain, in the province of Tarragona. The area is particularly suitable for this type of research as there is a large variation in the geology, lithology and tectonic environment, giving different geological environments for the development of the classification system. Rocks in the Falset area vary from Tertiary conglomerates to Carboniferous slates and include rocks containing gypsum, shales, granodiorite, limestone and sandstones. The topography is mountainous and the vegetation limited, such that large areas of rock are exposed. In addition, numerous old roads (built some 40 to 60 years ago) exist and several roads have been built in recent years creating large numbers of road cuts, excavated using different techniques. Many old and new slopes had been designed or excavated poorly, resulting in a number of slope instabilities over the years. The height of the slopes in the road cuts is typically between 5 and 25 m with a maximum of about 45 m. This has allowed a comparison of both slope stand-up times and excavation methods as well as an assessment of the influence of weathering and the method of excavation.

The climate in the Falset area is Mediterranean, characterized by dry and hot summers (temperature ranges from ≈ 15 to 35 °C) and moderate winters (10 to 15 °C). Part of the area is mountainous, ranging to about 1,200 m above sea level. Rivers and streams in the area are mostly dry from March to October, but it can rain for long periods during the winter and even into April, although this is not typical. Sometimes the rain is torrential. Occasionally, temperatures below zero occur but snowfall in the area is rare.

Description of field conditions

Visual assessment of slope stability

The research was directed towards designing a slope stability classification system incorporating all possible mechanisms and modes of slope failure. To develop a new slope stability classification system, the stability of the slopes was visually classified in the field as stable or unstable, with a further subdivision into unstable with small problems and unstable with large problems. In principle, 'large' implies that the unstable rock mass is in the order of tonnes in weight, while 'small' implies that the unstable rock mass would weigh in the order of kilograms.

Visually estimating the degree of stability of a slope is to a certain degree subjective. This is a problem for all classification systems. For the SSPC system, estimates have been made over a period of 4 years using at least 60 observers from staff and students of ITC and Delft University of Technology, working on 184 slopes. It is therefore reasonable to assume no observer bias.

Geotechnical units in a rock mass

Theoretically, a proper assessment to determine the behaviour of a rock mass should include all properties in a rock mass and all spatial variations of the properties. This would be unrealistic and is not possible without the destruction of the rock mass, hence 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 units. The smaller the allowed variability of properties in a geotechnical unit, the more accurate the assessment can be. Limiting the variability of the properties of the geotechnical units involves collecting more data, however, and is thus more costly. Higher accuracy based on more data must therefore be balanced against the economic, social and environmental value of the engineering structure to be built and the possible risks for the engineering structure, environment or human life. For a road cut along a major highway, the variations allowed within a geotechnical unit will be smaller than those for a geotechnical unit in a road cut along a local road. No standard rules are available for the division of the rock mass into geotechnical units; this depends on experience and 'engineering judgment'.

Failure mechanisms

Slope failure mechanisms (such as shear displacement) and the resulting different failure modes (plane sliding, wedge failure, partial toppling and buckling) are discontinuity-related and depend on the orientations of the slope and discontinuity. However, other mechanisms not related to the orientations of the slope and the discontinuities can also cause failure of a slope, e.g. the breaking of intact rock under the influence of the stresses in the slope and the removal of slope surface material due to surface (rain)water and seepage of water out of the rock mass (ravelling).

Traditional rock slope stability analyses are based on recognition of the failure mode in the field followed by a (back)calculation. Although the failure modes causing slope instability are theoretically well defined, it is often difficult to recognize the operating failure mode in the field. In many unstable slopes, multiple modes are at work at the same moment or successively. Not all of these may be visible or easily recognizable. Moreover, not only do the proper failure modes have to be identified, but for slopes with multiple modes at work, the contribution of each mode to the overall (in)stability should also be quantified. In cases where different modes of failure operate successively, the moment the slope is examined may determine the failure mode recognized. For these reasons, in this research both stable and unstable slopes were analysed without regard to the cause of instability, to avoid the problem of identifying the exact failure modes in the field.

Determination of rock mass parameters

The rock mass properties necessary for the SSPC system (intact rock strength and discontinuity spacing and condition) are determined by relatively simple means in the field.

Intact rock strength

Intact rock strength is established in the field by 'simple means' following the table in Fig. <u>15</u>. The method has been tested extensively and the results compared with strengths obtained by laboratory unconfined compressive strength tests. 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 laboratory UCS tests (Hack <u>1998</u>). Although the 'simple means' tests may be thought to be subjective, only a short training on rock pieces with known intact rock strengths is enough to reduce subjectivity to an acceptable level. Large numbers of 'simple means' tests can be done in a short time span and are not dependent on obtaining a sample large enough for laboratory testing. The large number of tests also gives a better indication of the variation of the intact rock strength throughout the rock mass than can be obtained from a limited number of UCS tests values (Hack <u>1998</u>).

Orientation, 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 thus has a major influence on the mechanical behaviour of a rock mass. It should first be established whether discontinuities belong to a 'set' or should be treated as a 'single' feature. Determining the parameters for a set of discontinuities requires a form of averaging of the parameters of individual discontinuities.

The average orientation of a discontinuity set can be found mathematically or by stereo-projection methods (Terzaghi <u>1965</u>; Taylor <u>1980</u>; Hoek and Bray <u>1981</u>; Davis <u>1986</u>). 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 or under-rated in importance because the discontinuity spacing is large. This and other errors that may affect the results of stereographic projection methods to determine discontinuity sets and orientations are discussed in extenso by Terzaghi (<u>1965</u>).

Alternatively, a studied assessment can be undertaken, in which the discontinuities that are representative for a set are visually selected. The properties of the selected discontinuities are then measured in detail in predetermined locations. In the opinion of the authors based on both experience during former work and this research, this method gives a result equal to or better than the results of large numbers of measurements of discontinuities for a statistical analysis. A large number of measurements are usually done on a part of the exposure that is (easily) accessible, whether representative of the rock mass or not. The same observations have been made by other researchers (Gabrielsen <u>1990</u>). Moreover, the variation of discontinuity properties in one discontinuity set is often so large that a high degree of accuracy for an individual measurement is not very important (ISRM <u>1978</u>, <u>1981</u>) and the variation of properties is covered by the probability approach of the SSPC system.

Determining properties representing shear strength of a discontinuity

The shear strength of a discontinuity is determined by the sliding criterion that converts a visual and tactile (roughness established by touch) characterization of a discontinuity into an apparent friction angle along the discontinuity plane (Hack and Price <u>1995</u>). Figure <u>15</u> lists the different descriptive terms. The method of characterization of discontinuities in the SSPC system is partly based on existing literature (Rengers <u>1971</u>; ISRM <u>1978</u>; Laubscher <u>1990</u>). The large-scale roughness is determined following the examples in Fig. <u>3</u>. The small-scale roughness factors are a combination of visible roughness on an area of about 20 $^{\times}$ 20 cm² and tactile roughness. Visible small-scale roughness (e.g. 'stepped', 'undulating' and 'planar') is established following the examples in Fig. <u>4</u>. Tactile (material) roughness is established by touch (e.g. rough, smooth and polished). The relation between the different roughness parameters is illustrated in Fig. <u>5</u>. Infill material in discontinuities and the presence of karst along discontinuities are

characterized following the table in Fig. 15. This figure also shows how the characteristics of the

discontinuity are translated into values for four factors: large-scale (Rl) and small-scale (Rs) roughness, infill material (Im) and karst (Ka)). The condition factor for a discontinuity (TC) is calculated by a simple multiplication of these four factors:

$$TC = Rl * Rs * Im * Ka \tag{1}$$



Fig. 3. Large-scale roughness profiles



Fig. 4. Small-scale roughness profiles

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Fig. 5. Interpretation of roughness as a function of wavelength and amplitude. For small 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

Non-fitting discontinuities

The contribution of the roughness to the shear strength reduces if discontinuities are non-fitting (Rengers 1971). For large-scale roughness, only an estimate can be made as to how much the contribution of the large-scale roughness to shear strength is reduced due to non-fitting. For visible small-scale roughness, a similar procedure may be followed, but tilt or shear box tests can also be undertaken and the results converted into a roughness characterization (Hack and Price 1995). If a discontinuity is completely non-fitting, the shear strength depends only on the material roughness, e.g. rough, smooth or polished. Such a discontinuity would be characterized as 'planar' for small-scale visible roughness and 'straight' for large-scale roughness.

Non-persistent discontinuities

A non-persistent discontinuity (e.g. a discontinuity ending in intact rock) is treated as a discontinuity with a small-scale roughness of 'rough stepped'. Breaking through asperities has to take place before displacement along a rough stepped discontinuity can occur. This approach is similar to that used in the Q-system (Barton <u>1976</u>).

Stability analysis

The stability is determined by two analyses: the first is related to the orientation of the discontinuities and the slope ('orientation-dependent stability') and the second to the strength of the rock mass in which the slope occurs, independent of the orientation of both the discontinuities and the slope ('orientation-independent stability').

Orientation-dependent stability

Failures in a rock slope often depend on the orientation of the slope and the discontinuities in the rock mass. The main parameter governing this type of failure is the shear strength of the discontinuity. Two criteria were developed in the SSPC system to predict the orientation-dependent stability of a slope: the sliding and toppling criteria.

Sliding criterion

A relation was found between the condition value of a discontinuity *TC* (Eq. <u>1</u>) and the apparent angle of the dip of the discontinuity plane in the direction of the slope dip (*AP*):

$$\begin{aligned} AP &= \arctan\left(\cos \delta * \tan dip_{discontinuity}\right) \\ \text{if } AP &> 0^{\circ} \rightarrow AP = \text{apparent discontinuity dip} \\ &\text{ in the direction of the slope dip} \\ \text{if } AP &< 0^{\circ} \rightarrow |AP| = \text{apparent discontinuity dip} \\ &\text{ in the direction opposite the slope dip} \\ \delta &= dipdirection_{slope} - dipdirection_{discontinuity} \end{aligned}$$
(2)

Below the dashed line in Fig. $\underline{6}$ only two combinations of values for *TC* and *AP* exist for day-lighting discontinuities in stable slopes (the two points below the line are likely due to a measuring error). The 'sliding criterion' (Eq. $\underline{3}$) is therefore considered as the boundary condition for sliding in slopes and sliding occurs if:

$$TC < 0.0113 * AP \tag{3}$$



Fig. 6. Discontinuity condition TC vs. AP for day-lighting discontinuities in stable and unstable slopes

The sliding criterion is confirmed by field and laboratory test values for discontinuity friction and by friction values for discontinuities discussed in the literature (Hack and Price <u>1995</u>).

Toppling criterion

Analogous to the sliding criterion, the 'toppling criterion' considers the interlayer slip necessary for toppling as defined by Goodman (<u>1989</u>). The SSPC toppling criterion is:

$$TC < 0.0087 * \left(-90^{\circ} - AP + dip_{discontinuity}\right) \tag{4}$$

Additional conditions

Additional conditions concern the minimum difference between the slope and discontinuity planes for the sliding criterion: $dip_{slope}>AP+5^{\circ}$. This is necessary because a day-lighting discontinuity with an apparent dip (in the direction of the slope dip) similar to that of the slope dip will form the slope face and will not cause sliding failure. A second condition is that the discontinuity plane should not be near vertical as a vertical plane cannot be a sliding plane or a cause of toppling. Therefore, for sliding to occur *AP* must be $<85^{\circ}$ and for toppling to occur it must be $>-85^{\circ}$. The value of 5° is based on field observations which indicated that measuring accuracy is normally around 5° . Figures 9 and 10 give the probabilities for sliding and toppling respectively as a function of the apparent discontinuity dip in relation to the condition of the discontinuity. Probability calculations are given below.

Methodology to optimize sliding and toppling criteria

The sliding and toppling criteria can be demonstrated on a graph but as this introduces some subjectivity, the criteria have also been established mathematically. To determine the boundary line for the sliding criterion, 300 sets of data points (*AP*, *TC*) were generated randomly out of the data set for discontinuities in stable slopes in the research area, with the standard error distribution in *AP* and *TC* for each original data point (see probability analyses below). A number of data points (*X*) with the lowest ratios of *TC/AP* were determined from each set of data points. Those with the lowest *TC/AP* were used because the boundary line should be the lower boundary of the data set. The slope and intercept of a linear regression of these *X* data points were computed for each of the 300 sets of data points, resulting in 300 regression lines for which the mean and standard error were calculated. The number of data points (*X*) used for the regression varied from 2 to 30. Figure <u>7</u> illustrates the procedure for *X*=2 and Fig. <u>8</u> the mean and standard error of the intercept and the slope versus *X*. If six points are used for regression, the values for the mean intercept and mean slope are robust (changing only slightly if more points are used) and the standard errors approximately constant. As would be expected, the value for the mean slope coincides with the visually determined boundary. The same procedure was followed for the toppling criterion.







Fig. 8. Mean and standard error of intercept and slope of boundary lines vs. χ , for 'sliding criterion'

Orientation-independent stability

A large number of the slopes were found not to be unstable using the sliding and toppling criteria, although they were assessed visually in the field as unstable. For these slopes, a mathematical model could be formulated to predict the orientation-independent stability. Most of the failures in these slopes were approximately linear, although not following one and the same existing discontinuity plane. Often, fracturing of intact rock over small distances (relative to the size of the slope) results in linear failure planes developing partly through intact rock and partly following existing discontinuity planes. This effect was more prominent in rock masses in which the block size was smaller. Intact rock strength, block size and shear strength along discontinuities thus have an influence on the development of failure planes not related to a single existing discontinuity.

The orientation-independent stability of such a slope was modelled by a linear shear plane model following the Mohr-Coulomb failure criterion (Hack <u>1998</u>). The friction and cohesion parameters in the Mohr-Coulomb failure criterion are the apparent friction and cohesion for the rock mass. The rock mass friction and cohesion are dependent on intact rock strength, block size (e.g. discontinuity spacing) and shear strength (e.g. the condition of discontinuities) along all discontinuities in the rock mass.

Block size and condition of discontinuities

Several options exist to incorporate the intact rock strength, the block size and the condition of all discontinuities in the shear plane model. Extensive analyses (Hack <u>1998</u>) gave the best results for the block size (*SPA*) if based on the factors of Taylor (<u>1980</u>); see graph in Fig. <u>16</u>:

for three discontinuity sets : $SPA = factor_{maximum} * factor_{intermediate} * factor_{n}$ for two discontinuity sets : $SPA = factor_{maximum} * factor_{minimum}$ for one discontinuity : SPA = factor (5) SPA = spacing parameter $factor_{x} =$ determined by graph included in figure 16

The best formulation for the condition of discontinuities in a rock mass (CD) was found to be based on the mean of the conditions of three discontinuity sets weighted against the spacing of the sets:

$$CD = \frac{\frac{TC_1}{DS_1} + \frac{TC_2}{DS_2} + \frac{TC_3}{DS_3}}{\frac{1}{DS_1} + \frac{1}{DS_2} + \frac{1}{DS_3}}$$
(6)

 $TC_{1,2,3}$ are the condition, and $DS_{1,2,3}$ are the spacings of discontinuity sets 1,2,3

SSPC rock mass friction and cohesion

Optimizing the Mohr-Coulomb failure criterion with the intact rock strength (*IRS*), spacing (*SPA*) and condition of discontinuities (*CD*) gives the following:

$$\begin{split} \varphi'_{mass} &= IRS * 0.2417 + SPA * 52.12 + CD * 5.779\\ coh'_{mass} &= IRS * 94.27 + SPA * 28629 + CD * 3593\\ if intact rock strength > 132 MPa \rightarrow IRS = 132 \end{split}$$
(7)
$$\varphi'_{mass} &= angle \ of \ internal \ friction \ of\\ the \ rock \ mass \ (in \ degrees)\\ coh'_{mass} &= rock \ mass \ cohesion \ (in \ Pa) \end{split}$$

The intact rock strength in Eq. (7) is maximized. Above a value of about 132 MPa, it was found that the stability of the slopes did not further increase with increasing intact rock strength. This was valid for the slopes in the research area with heights ranging up to 45 m. A higher value for the intact rock strength maximum may be necessary for significantly higher slopes with higher stresses. For both spacing (*SPA*) and condition (*CD*), the combination of discontinuity sets that results in the minimum rock mass friction value was always taken. Even if the rock mass contains three or more discontinuity sets, spacing (*SPA*) and condition (*CD*) factors calculated from only one or two discontinuity sets may give a lower result.

Linear shear plane model and maximum slope height

The model of a linear shear plane following the Mohr-Coulomb failure criterion implies that the stability of a slope is independent of the height of the slope if the slope has a dip angle less than the friction angle of the rock mass. If the dip angle is higher than the friction angle, however, the maximum slope height is determined by the stresses in the slope. For a rock mass unit weight of 25 kN/m³:

If the
$$dip_{slope} \leq \varphi'_{mass}$$
:
the maximum slope height (H_{max}) is infinite
 $else$
 $H_{max} = 1.6 * 10^{-4} * coh'_{mass} *$
 $\frac{sin(dip_{slope}) * cos(\varphi'_{mass})}{1 - cos(dip_{slope} - \varphi'_{mass})}$
(8)

Stability

In terms of probabilities, the orientation independent stability is given in Fig. <u>11</u>. The axes are horizontally normalized on the ratios of rock mass friction to slope dip and vertically on the maximum possible height (H_{max}) as a ratio of the true height (H_{slope}).

Methodology to optimise orientation independent failure criterion

It is assumed that \mathcal{P}_{mass} and coh_{mass} are dependent on the rock mass parameters measured in the field, e.g. intact rock strength (*IRS*), spacing of discontinuities (*SPA*) and condition of discontinuities (*CD*). In

addition to linear relationships between \bigcap_{mass}^{Q} and *coh_{mass}*, and *IRS*, *SPA* and *CD*, the following have also been investigated:

$$\varphi_{mass} \text{ or } coh_{mass} = IRS * e^{\frac{-w1}{SPA+CD}}$$

$$\varphi_{mass} \text{ or } coh_{mass} = IRS * SPA^{w1} * CD^{w2}$$
(9)

The research has found that both φ_{mass} and coh_{mass} can be reasonably represented by a linear combination of *IRS*, *SPA*, and *CD*. The influence of the intact rock strength on slope stability is bounded by a

maximum (cut-off) value. Linear relationships for \wp_{mass} and coh_{mass} with a cut-off value for the intact rock strength (*IRS*) result in the following:

$$\begin{aligned} \cosh_{mass} &= w0 * IRS + w1 * SPA + w2 * CD \\ \varphi_{mass} &= w3 * IRS + w4 * SPA + w5 * CD \\ \text{if } IRS &\leq \text{cut} - \text{off value} \rightarrow IRS = \text{intact rock strength} \\ & (\text{as measured in the field}) \\ \text{if } IRS &> \text{cut} - \text{off value} \rightarrow IRS = \text{cut} - \text{off value} \\ & \text{weight factors} : w0, w1, ..., w5 \geq 0 \end{aligned}$$
(10)

 φ

 φ_{mass} , the friction of the rock mass, has a value within a range from 0 to 90° (0 to $\pi/2$). In order to optimize the shear plane model, φ_{mass} has to be normalized so that the value is never outside this range. The maximum value for φ_{mass} is obtained for an intact rock strength (*IRS*) equal to the cut-off value, *SPA* equal to its maximum value of 1.00 and *CD* to its maximum value of 1.0165. Hence, the maximum for φ_{mass} is expressed by:

$$\varphi_{mass}(maximum) = w3 * cut - of f value + w4 * 1.00 + w5 * 1.0165$$
(11)

 φ_{mass} in Eq. (<u>10</u>) must thus be divided by φ_{mass} (maximum) and multiplied by $\pi/2$.

Large differences in the order of magnitude of parameter values may have an influence on the optimum values found in the non-linear optimization. The intact rock strength (*IRS*) in Eq. (10) has therefore been divided by 100 to reduce the difference with SPA and CD. Combining with Eqs. (8) and (10) with the normalization of \mathcal{P}_{mass} and the division of *IRS* by 100 leads to a set of equations describing the shear

normalization of mass and the division of *IRS* by 100 leads to a set of equations describing the shear plane model:

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$$dip_{slops} \ge \varphi_{mass} \rightarrow$$

$$H_{max} = 4 * \frac{coh_{mass}}{UW} * \frac{\sin(dip_{slops}) * \cos(\varphi_{mass})}{1 - \cos(dip_{slops} - \varphi_{mass})}$$

$$dip_{slops} < \varphi_{mass} \rightarrow H_{max} = unlimited$$

$$coh_{mass} = a0 * \frac{IRS}{100} + a1 * SPA + a2 * CD$$

$$\varphi_{mass} = \left(\frac{a3 * \frac{IRS}{100} + a4 * SPA + a5 * CD}{a3 * a6 + a4 + a5 * 1.0165}\right) * \frac{\pi}{2}$$

$$if \frac{IRS}{100} \le a6 \rightarrow IRS = intact \ rock \ strength \ (in \ MPa)$$

$$if \frac{IRS}{100} > a6 \rightarrow IRS = a6 * 100$$

$$a0 \ through \ a6 = weight \ factors \ dip_{slops} = dip \ of \ slops$$

$$H_{max} = maximum \ possible \ slops \ height$$

$$UW = Unit \ Weight \ of \ the \ rock \ mass$$

SPA is calculated following Eq. $(\underline{5})$ and CD following Eq. $(\underline{6})$.

1.3

Measured intact rock unit weights ranged between 25.5 and 27.0 kN/m³. A small proportion of open discontinuities in the rock mass indicates that the unit weight of the rock mass is approximately the same as that of the material. The value of the unit weight of the rock mass was taken to be the same for all rock masses in the research area - it was considered that the karstic rock units do not have a rock mass unit weight considerably less than the intact rock unit weight.

In Eq. (12), the values of the factors a0 through a6 are unknown. Equation (12) is therefore optimized (following the set of optimization rules in Eq. 13) over the slopes that are not unstable due to orientation-dependent stability.

For each slope
$$j$$
:
visually estimated
stability = stable
visually estimated
stability = unstable
 $ER = \sum_{j} er_{j}$
 $\begin{cases} \frac{\varphi_{mass}}{dip_{olope}} \ge 1 (stable) \to er = 1 \\ \frac{\varphi_{mass}}{H_{olope}} \ge 1 (stable) \to er = \frac{\varphi_{mass}}{dip_{olope}} \\ 1 \left\{ \frac{H_{max}}{H_{olope}} \le 1 (unstable) \to er = \frac{H_{olope}}{H_{max}} \\ \frac{\varphi_{mass}}{dip_{olope}} \ge 1 (stable) \to er = \frac{\varphi_{mass}}{dip_{olope}} \\ 1 \left\{ \frac{H_{max}}{H_{olope}} \le 1 (unstable) \to er = 1 \\ \frac{H_{max}}{H_{olope}} \ge 1 (stable) \to er = 1 \\ \frac{H_{max}}{H_{olope}} \ge 1 (stable) \to er = 1 \\ \frac{H_{max}}{H_{olope}} \ge 1 (stable) \to er = \frac{H_{max}}{H_{olope}} \\ \end{cases}$
(13)

In Eq. (13), H_{slope} and dip_{slope} are the real height and dip of the existing slope (j). H_{mass} is the rock mass friction of the rock mass in which this slope (j) is made (defined in Eq. 12) . H_{max} is the theoretical maximum height of a slope which, with the same dip as that of the existing slope (dip_{slope}), can be sustained by the rock mass in which the existing slope is made. H_{max} is defined in Eq. (12). ER in Eq. (13) is the value over which optimization is undertaken, i.e. it is the value which will be minimized during the optimization procedure. *ER* equals the summation of er_j over all slopes used in the optimization.

The procedure of the optimization is that for each slope (*j*) the \mathcal{P}_{mass} and H_{max} are calculated following Eq. (12) with (initially randomly) chosen values for the unknowns *a0* to *a6*. If for slope (*j*) the \mathcal{P}_{mass} is larger than the dip of the existing slope (*dip*_{slope}), then slope (*j*) should be stable following the shear plane model. If this is in accordance with the visually estimated stability of the existing slope (*j*) the unknowns *a0* to *a6* are correctly chosen. Hence, *er*_j is set to the value 1. If the existing slope (*j*) is not stable then the unknowns *a0* to *a6* are not correct and er_j is set to a value larger than 1, which reflects how much the calculated \mathcal{P}_{mass} differs from values that would result in stability at equilibrium (i.e., $er_j = \mathcal{P}_{mass}/dip_{slope}$).

The procedure is more complex if $P_{mass} < dip_{slope}$. The theoretical possible height (= H_{max}) should then be compared to the real height of the existing slope (*j*) (= H_{slope}). If it is more than the height of the existing slope (*j*) then slope (*j*) should be stable following the shear plane model with the chosen set of values for the unknowns *a0* to *a6*. If this is in accordance with the visually estimated stability of slope (*j*) then the calculation is correct and *er_j* is set to the value 1. If the existing slope (*j*) is visually assessed as unstable then the unknowns *a0* to *a6* are not correct and *er_j* is set to a value larger than 1 which reflects how much the calculated H_{max} differs from values that would result in stability at equilibrium following the shear plane model calculated for slope (*j*) (i.e. *er_j*= H_{max}/H_{slope}). If slope (*j*) is calculated to be unstable ($H_{max} < H_{slope}$) and slope(*j*) is also visually assessed as unstable, the unknowns are correctly estimated and *er_j*=1. If the slope is visually assessed as stable, however, the unknowns are incorrect and the value of *er_j* must be set to a value reflecting the degree of miscalculation (*er_j*= H_{slope}/H_{max}).

The above is undertaken for all slopes with one set of values for the unknowns a0 to a6. The er_j of all the slopes are then added to give an *ER*. A second set of unknowns a0 to a6 is determined following the Levenberg-Marquardt (*1963*) optimization routine and the *ER* calculated following the same procedure. The optimization routine compares the *ER* values for the different sets of unknowns and based on this determines a new set of values for the unknowns a0 to a6, such that the *ER* calculated based on these new values is likely to be lower than those previously used. The optimization routine will continue until no further reduction of *ER* is obtained. The values for the unknowns a0 to a6 resulting in the lowest value for *ER* are assumed to be the most appropriate values for which the shear plane model formulated in Eq. (12) best fits the data.

ER would equal the total number of slopes used in the optimization if the shear plane model is the completely correct model for orientation-independent stability, if the data set is ideal (no errors in any parameter of any slope), and if the factors a0 through a6 are at optimum values. The stability calculated with the shear plane model would then be the same as the visually estimated slope stability in the field for all slopes. Obviously, this is unlikely because the shear plane model is not a completely correct model and the data set is not likely to be ideal. There are thus always a certain percentage of the slopes for which the slope stability following the shear plane model is not equal to the visually estimated stability in the field. For this reason, the value of *ER* is always larger than the total number of slopes used in the optimization. The goal of the optimization is to minimize *ER*. The values for a0 through a6 in Eq. (12) belonging to the minimum value for *ER* are then taken to be the values that best fit the data set.

During the optimization process, the ratios of H_{slope}/H_{max} (for slopes visually estimated to be stable) and

 H_{max}/H_{slope} (for slopes visually estimated to be unstable) are limited to a maximum of 2. The ratio of r_{max}/dip_{slope} (for visually estimated unstable slopes) is similarly limited to 2. These limitations are necessary to avoid too strong an influence of possible outliers. In particular, H_{max} becomes (extremely)

large and influences the optimization very significantly for an outlier with φ_{mass} smaller than, but almost equal to, the slope dip.

The maximum possible height of a rock slope (H_{max}) is infinite if the slope dip angle is less than the rock

mass friction (P_{mass}). As a consequence of this and of the use of a cut-off value for the intact rock strength, the function in Eq. (12) is not continuous in the first derivative. Because of the likely errors in the data (visually estimated stability, dip, height, intact rock strength, etc.), the function also contains multiple minima. Optimization of a function that is not continuous in the first derivative and that contains multiple minima is difficult and it is often doubtful whether the absolute minimum can be found. The function was therefore examined graphically to find ranges for the factors in which the function is likely to minimize (decreasing *ER*). Then the Levenberg-Marquardt optimization routine (Marquardt 1963) was implemented with starting values for the factors within the ranges graphically determined. A lowest minimum was found for which multiple optimizations with different starting values resulted in approximately the same values for the six factors. A graphic examination of the function with these values showed that these values were probably the best possible and represented the absolute minimum of

the function. These values are used as starting values in the optimizations for the probability analyses. Figure $\underline{12}$ shows the results of the optimization with the data from the research area. Note that, as would be expected, the slopes characterized visually as 'unstable with small problems' plot near the line of equilibrium (dashed line). However, this information on the scale of the instability is not used in the optimization and therefore confirms the optimization results independently.

Influence of water

Water pressures in discontinuities are traditionally believed to be of major importance in rock slope stability. However, a more thorough examination indicates that, in general, this must be regarded as doubtful for many rock masses. Most rock masses at or near the surface contain many discontinuities and these will generally allow water to freely flow out of the rock mass in a slope cut, while the cover of topsoil frequently present above a slope will reduce the rate of water inflow as it is generally less permeable than the rock mass. This inhibits any build-up of water pressures.

Another reason for smaller or non-existent water pressures is that near the slope face, stresses will be smaller than those occurring deeper in the rock mass. Smaller stresses will cause the discontinuities to open and hence reduce water pressures in discontinuities with water flowing in the direction of the slope cut. The only situation in which water pressures will be a major influence is if a new slope intersects a groundwater table. However, it would be anticipated that good engineering practice would ensure appropriate drainage measures were taken to lower the groundwater table behind such a slope cut.

In the more established rock mass classification systems, the influence of water varies widely ranging between 3 and 15%, but lower values are more common in recently developed rock mass classification systems for slope stability (Hack 1998). For example, the maximum influence of the presence of water in the RMR system is 15% (Bieniawski 1989), in the SMR system 13% (Romana 1985), 6% in the system developed by Selby (1980) and only 3% in the system developed by Haines and Terbrugge (1991). The SSPC system does not explicitly incorporate a factor for the presence of water pressures. However, the presence of water is incorporated in the factors for the infill material in discontinuities for materials that lose strength if water is present.

Local influences: weathering and method of excavation

The three-step approach allows for correction of local influences such as weathering and the damage due to the method of excavation. The 'exposure' rock mass is first divided into geotechnical units. 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 (Hack and Price 1997) and for damage due to the method of excavation used to create the exposure. These correction factors are listed in Fig. <u>15</u>. The weathering characterization follows BS 5930 (<u>1981</u>) although for most rock masses these can easily be converted to comply with the 1999 revised standard. The parameters that characterize the 'slope' rock mass are obtained by correction of those for the 'reference' rock mass to allow for damage due to the method of excavation to be used for the new slope and to take into account present and future weathering (see Fig. <u>17</u>).

Future weathering is predicted by examining the same geotechnical unit in exposures which have been in existence for a known period of time. It should be noted that weathering may well depend on very local influences, such as orientation of the exposure, position in the landscape (wind), use of fertilizers by farmers which may influence the mineral stability (via the groundwater), etc. Although this may sometimes be difficult, experience suggests that enough information can be found to estimate the most likely degree of weathering of the geotechnical unit at the end of the engineering lifetime.

Probability analyses

A probabilistic approach using the Monte Carlo method (Hammersley and Hanscombe <u>1964</u>) was applied in the analysis to quantify the reliability of the functions found for slope stability in the SSPC system. The results of the Monte Carlo simulations were used to indicate the probability lines in Figs. 9, <u>10</u> and <u>11</u>. This methodology also allows an evaluation of the sensitivity of the result for input errors. A measured rock mass parameter has a distribution related to a combination of: (1) the variation of a parameter in a rock mass, (2) the limitations of the variation of a rock mass parameter imposed by the subdivision of the geotechnical units and (3) the error made in measuring a rock mass parameter in a geotechnical unit. The latter can be determined by repeating a measurement many times at exactly the same location such that a standard error is obtained. Clearly, only one single location should be used or the distribution of a parameter in the geotechnical unit would contribute to the standard error.



Fig. 9. Sliding criterion



Fig. 10. Toppling criterion



Fig. 11. Probability of orientation-independent stability



Fig. 12. Results of optimization for orientation-independent stability

During the research, repeated measurements of the same parameter in the same geotechnical unit were made by different students and staff members. The variation resulting from these measurements is assumed to be typical error distributions for the measurement of a characteristic value for a particular rock mass parameter within a geotechnical unit. Most of the distributions of rock mass parameters were about normal, although some were discrete or showed a non-normal behaviour near the limit values of the ranges allowed. As the differences were small, however, in the probability analyses the non-normal and discrete distributions, either direct or expressed as a percentage of the mean (characteristic) value, were taken as the standard error of the characteristic value of a rock mass parameter. The standard errors are not the same for all geotechnical units, as those with a wider range of permitted values will probably also have a wider distribution of characteristic values and thus a larger standard error. In the research area, however, they were approximately identical in different rock mass types. This implies that different observers divide different rock masses in geotechnical units for slope stability assessment such that the

variation allowed in a unit is similar. It is therefore considered realistic to assume that the error distributions are representative for measuring a characteristic parameter value in a geotechnical unit.

Results and example

Results

Figure <u>13</u> compares the results of slope stability assessments for 184 slopes following the SSPC system with results using the Haines (Haines and Terbrugge 1991) and SMR (Romana 1985) systems. The calculation of the stability of a slope with the SSPC system gives a more distinctive differentiation between stable and unstable conditions than with either the Haines or SMR systems. In addition, the correlation between the visually estimated slope stabilities and the predictions of stability of the SSPC system is better than the correlation with the other classification systems.





Example

The SSPC system, as applied to the thinly bedded units in a slope of newly blasted (1988) limestone and dolomite, originally with a face of about 75° to 80° (see Fig. 14), is shown in Figs. 15, 16, and 17. The present (2000) angle of the face is between 60° and 70° . The slope consists of interlayered thin bedded (above the seated person in Fig. 14) and medium to thickly bedded units. The same thinly-bedded units are found exposed in road cuts less than 50 m away which are known to be more than 40 years old. These

old road cuts, with slopes of 60° to 70° and heights of about 5 m, are still (1995) stable, very little or no degradation of the rock mass is observed and the material appears only slightly weathered. The method of excavation used for these old slopes was either hand shovels or small mechanical shovels.



Fig. 14. Photo showing unstable slope

ITC/TUD ENGINEERING	GEOLO	GY			expos	ure characte	rization					SS	PC - SYSTEM	
LOGGED BY: 22	0/04/96			TIME: 16:0	00r hr	exposure no: example (old slope)								
WEATHER	LOCATION			map no:		472-								
Sun:	cloudy/fa	air/bright		Map coordinates:			northing:		4,558,750					
Rain: dr	y/drizzle/	/slight/heavy	y				easting:		301 604					
METHOD	OF EXCA	AVATION (N	/ (1E)	1			ů, ř	DIMENSIO	NS/ACCES	SIBILITY				
(tick) Size total							(m) l: 200 h: 5 d:							
natural/hand-made pneumatic hammer excava	ation			✓ 1.00 0.76	mapped o	n this form	(m)	Ŀ	200	h:	5	d:	50	
pre-splitting/smooth wall	blasting			0.99	Accessibil	ity:				poor/i	air/good			
conventional biasting with	0.77													
open di disl	0.75													
fracture	ed intact ro	xck		0.67										
EODMATION NAME: 64	22 Hin La	<u></u>		0.62										
FORMATION NAME: 19	CS inin be	aaea unus			DESCRI	PTION /BS	930-1981)							
colour	8	grain size	1	stru	cture & text	ure	,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	veathering	NAME					
brownish off-white		fine		thin boddod, small tabular				slightly			imestone and dolomite			
	INI	ACT ROCK	STRENG	TH (IRS) (tick)				sample nur	nber(s):		WEATHERING (WE)			
< 1.25 MPa	1	Crumbles in	hand								61.15			
5 - 12.5 MPa	1	Thin slabs b	reak easil	y in hand	1 processo			- - -			(tick) unweathe	red	1.00	
12.5 - 50 MPa 50 - 100 MPa		Lumps brok	en by ligl	ht hammer	blows						slightly	slightly V 0.95		
100 - 200 MPa 🖌		Lumps brok	en by hea chip by h	avy hamme neavy hami	r blows ner blows (Dull ringing	highly							
> 200 MPa	1	Rocks ring c	on hamme	er blows. Sp	oarks fly		,				complete	y	0.35	
DISCONTINUITIES B=bee	iding C=C	leavage J=joi	int			B1	\$2	ß	4	5	EXISTING SLOPE? dip-direction/dip 180/ 70			
Dip direction					(degrees)	082	310	244						
Dip					(degrees	30	87	62						
Spacing (DS)					(m)	0.03	0.04	0.03						
	along str	ike			(m)	> 200	>14	0.2			height:			
Persistence	Persistence along dip				(m)	> 50	> 20	0.2				0.11		
CON	5						I	Stability (tick)					
Roughness large scale		1.00	0					stable		¥1				
(RI)	curved:				0.85 1.00	0.75	0.75			large prol	olems	3		
0.2 and 1 x 1 m2)	slightly c straight	curved			0.80									
	rough stepped				0.95				1 1					
	smooth s	stepped			0.90									
Roughness small scale (Rs)	rough un	ndulating			0.80									
(on an area of 0.2 x 0.2	smooth u polished	smooth undulating polished undulating			0.75		0.60	0.95			notes: 1) For infill 'gouge >			
m2)	rough planar smooth planar polished planar			0.65		·							>	
					0.60						irregularities' and 'flowing material' small scale roughness r			
	cemented	d/cemented i	infill		1.07						0.55.		de a branda	
	no infill -	surface stair	ung		$-\frac{1.00}{0.95}$						(e.g. rippl	 If roughness is anisotropic (e.g. ripple marks, striation, etc.) 		
	non sortening & sneared cc material, e.g. free of clay, m talc, etc. fii soft sheared material, e.g.			xoarse 0.99 nedium 0.90 ine 0.85							roughnes	s should b	e assessed	
											roughnes	s and dire	ctions noted	
Infill material (Im)				oarse	0.75 0.55		0.55	1.00			on this for 3) Non-fit	m.		
	clay, tálc,	, etc.	- in - in	nedium	0.65						 Non-fitting of discontinuities should be marked in roughness columns. 			
	gouge < i	irregularities												
	gouge > irregularities			0.17										
			0.05			- 			-					
Karst (Ka)	karst				0.92	0.92	0.92	0.92						
SUSCEPTIBILITY TO WEA	ATHERIN	G (SW)							remarks:					
degree of weathering:		date excavat	tion:		remarks:									
slightly	ars old	old road cuts, hand-made or small shovel ?									•			

Fig. 15. Example of exposure characterization

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ITC/TUD ENGINEERING GEOLOGY reference rock mass calculation SSPC - SYSTEM											
CALCULATED BY: zz DATE: 10/04/96 exposure no							exposure no: example				
REFERENCE UNIT NAME	: Jg23 thin	bedded units									
				INTACT ROC	K STRENGT	H (RIRS)					
					RIRS = IRS	(in MPa) / W	TE (correction for weathering) = 150/ 0.95=	158			
			1	DISCONTINU	ITY SPACIN	G (RSPA)					
DISCONTINUITI	ES	\mathcal{B}_1	J2	З	4	5	SPA (see figure below) =				
Dip direction	(degrees)	082	310	244			$0.43^{*} \ 0.30^{*} \ 0.34 = 0.043$				
Dip	(degrees)	30	87	62			corrected for weathering				
Spacing (DS)	(m)	0.03	and method of excavation:								
The spacing parameter (SPA) is calculated based on the three discontinuity sets with the smallest spacings following figure:											
DISCONTINUITIES		R	6	I DISC		ES (RIC & R					
Roughness large scale	(RI)	1.00	0.75	0.75							
Roughness small scale	(Rs)	0.75	0.60	0.95		1					
Infill material	(lm)	0.55	0.55	1.00							
Karst	(Ka)	0.92	0.92	0.92			RTC is the discontinuity condition of a single	e discontinuity			
Total (RI*Rs*Im*K	a = TC)	0.38	0.23	0.66			(set) in the reference rock mass corrected for	discontinuity			
RTC		0.38	0.23	0.66			RTC = TC / sqrt(1.452 - 1.220 * e^(-WE))				
Weighted by spacing:		$CD = \frac{\frac{TC}{DS}}{\frac{1}{DS}}$	$\frac{C_1}{S_1} + \frac{TC_2}{DS_2} + \frac{TC_2}{DS_2} + \frac{1}{S_1} + \frac{1}{DS_2} + \frac{1}{S_1} + \frac{1}{$	$\frac{TC_3}{DS_3} = \frac{0}{C}$ $\frac{1}{DS_3} = \frac{0}{C}$ receded for weather the second secon	$\frac{0.38}{0.03} + \frac{0.2}{0.0}$ $\frac{1}{0.03} + \frac{1}{0.0}$ athering: RCI	$\frac{23}{24} + \frac{0.66}{0.03}$ $\frac{1}{24} + \frac{1}{0.03}$ $\frac{1}{24}$ (with a max)	= 0.44 cimum of 1.0165) = CD / WE = 0.44/ 0.95=	0.46			
REFERENCE UNIT FRICTION AND COHESION (RFRI & RCOH)											
$\Phi_{\text{REM}} = \text{RIRS} * 0.2417 + \text{RSPA} * 52.12 + \text{RCD} * 5.779 \text{ (if RIRS > 132 MPa then RIRS = 132)}$ $\Phi_{\text{REM}} = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 5.779 = 132^* 0.2417 + 0.046^* 52.12 + 0.46^* 52.12 $											
coh _{RRM} = RIRS * 94.27 + RSPA * 28629 + RCD * 3593 (if RIRS > 132 MPa then RIRS = 132) coh _{RRM} = 132 * 94.27 + 0.046 * 28629 + 0.46 * 3593 =											
notes: 1) For IRS (intact rock strength) take average of lower and higher boundary of class. 2) Roughness values should be reduced or shear strength has to be tested if discontinuity roughness is non-fitting. 3) WE = 1.00 for 'soil type' units, e.g. cemented soils, etc 4) If more than three discontinuity sets are present in the rock mass then the reference rock mass friction and cohesion should be calculated based on the combination of those three discontinuities that result in the lowest values for rock mass friction and cohesion.											

Fig. 16. Example of reference rock mass calculation

ITC/TUD ENGINEERING GEO	slope stability	probability		SSPC - SYSTEM							
LOGGED BY: 22				stope no: example (new slope)							
		map no:		472.							
						northing:					
		Map coordina	easting:				4,556,6				
				DETAILSOF		521,725					
METHOD OF EXCA	VATION (SM	(E)	1	WEATHER	SLOFE SING (SWE)						
(tick)			(tick)								
natural/hand-made	ural/hand-made 1.00 un sumatic hammer excavation 0.76 sli					1.00	Slope dip dire	ection (degre	es):	180	
pre-splitting/smooth wall blastin	ıg	0.99	moderately			✓ 0.90	Slope dip (de	egrees):		70	
conventional blasting with result	ventional blasting with result: good (
open discontin	open discontinuities (Height (Hslo	pe) (m)		13.8		
dislodged t fractured intac	√0.72 0.67	note: SWE =	1 00 for 'soil tyr	ve'units e a cem	:						
crushed intac	t rock	0.62	0.62 etc.								
11/12											
SLOPE UNIT NAME: JA Car	boniferous) slat	te, v. thin cleav	age			1975 /					
			INTAC	T ROCK STRE	NGTH (SIRS)	11					
				SIRS = RIRS	(from reference n	ock mass) * S	WE (weatherin	10 slope = 1	58* 0.90=	142	
			DISCO	NTINUITY SP/	ACING (SSPA)	oen muss) - c	ine (inclution	·B (10 Pc) - 1	00 0.70	1 1 1	
SSPA = RSPA (from reference ro	ck mass) * SWI	E (weathering s	lope) * SME (m	ethod of excave	ation slope)						
							SSPA =	0.046* 0.	90* 0.72=	0.030	
			CONDITIC	ON OF DISCON	ITINUITIES (SCI	D)					
SCD = RCD (from reference rock	mass) * SWE (weathering slo	pe)								
								SCD = 0.4	64* 0.90=	0.418	
SLOPE UNIT FRICTION AND COHESION (SFRI & SCOH)											
$\phi_{SRM} = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779$ (if SIRS > 132 MPa then SIRS = 132)										2/10	
cohsem = SIRS * 94.27 + SSPA * 28	3629 + SCD * 3	593 (if SIRS > 13	32 MPa then SI	RS = 132)	ψs	RM - 132 0.	2417 + 0.050	52.12 + 0.4	470 *5.779=		
		,		,	coh	SRM = 1 32* 9	4.27 + 0.030	* 28629 + 0.	418 * 3593 =	14146Pa	
		Ifs	SFRI < slope di	p: MAXIMUM	SLOPE HEIGHT	(Hmax)					
Maximum possible height: Hma:	x = 1.6 * 10-4 * c	oh _{SRM} * sin(slop	e dip) * cos (_s	RM) / (1-cos(slo	pe dipsrm))		70		o. o.t		
				Hm	$ax = 1.6 * 10^{-4} *$	14146* sin(70°) * cos(34	°)/(1-cos(7	0° - 34°)) =	9.3m	
ratios:					_SRM / 1	0.49					
				Hmax / Hsl	0.67						
Probab	vility stable: if \$	5FRI > slope dij	p probability =	100 % else use	figure for orienta	ition indeper	dent stability:			8%	
DISCONITINUITIES			ORIENTA	TION DEPENI	DENT STABILIT	Y		-			
Discontinutilies			(4)	<i>D</i> 1	∂^2	<i>P</i> ³	4	3			
Dip direction			(aegrees)	082	310	244					
Dip			(degrees)	30	87	62	ļ				
With, Against, Vertical or Equal				a	v	w					
AP	-5	-85	40								
RTC (from reference form)	0.38	0.23	0.66		·						
STC = RTC * sqrt(1.452 - 1.220 * e	0.37	0.22	0.65								
Probability stable:	100%	100%	97 %	%	%	<u></u>					
Determination orientation stabili	ty:	die die st	- 41	ter die die ee			the second				
calculation AP: $\beta = discontinuity$	aip, o = slope stability	dip-direction,	t = discontinui	ty dip-directior	1: ο = σ - τ: AP =	arctan (cos õ	* tan β) stabiliter	cliding	. 1	toppling	
AP > 84° or AP < -84°	vertical	100 %	100 %	AP < 0° and	(-90° - AP + slop	e dip) < 0°	against	100 %	,	100 %	
(slope dip+5°) < AP < 84°	with	100 %	100 %	AP < 0° and	(-90° - AP + slop	e dip) > 0°	against	100 %	use	graph toppling	
(slope dip-5°) < AP < (slope dip+5°)	equal	100 %	100 %								
$0^\circ < AP < (slope dip-5^\circ)$	with	use graph sliding	100 %								

(graphs: sliding figure 9; toppling figure10; orientation independent stability figure 11)



The slope directions in the old and new road cuts are approximately equal and the general position of the old road cuts is comparable to that of the new road cut. Both the old and new road cuts were excavated into a hill that flattens above them. Any surface flow of heavy rainfall is therefore likely to be the same

for both the old and new cuts. In addition, with respect to geology (faults, etc.), no major differences have been noted between the old and the new road cuts.

The new road cut is clearly unstable, large parts show rill erosion and erosion of the thinly bedded units is causing undercutting of the more thickly bedded horizons, making these unstable. The general impression of the slope is extremely poor. On close examination, those parts of the slope that appear to be 'soil' are in fact the thinly bedded units, moderately to highly weathered, which are only partly covered by topsoil transported from higher parts of the slope. In some places the thinly bedded units would be classified as moderately or highly weathered for at least 0.5 to 1 m into the rock mass. The structure and coherence of the rock mass and in particular of the thinly bedded units have been disturbed by the method of excavation. Discontinuities have opened, blocks are displaced and at many locations the intact rock is fractured or occasionally crushed as a result of blasting. This has disturbed the structure of the rock mass so severely that water can flow through the near-surface rock slope and cause weathering of the thinly bedded units. The slope is not at risk due to sliding or toppling along discontinuities.

The SSPC system gives a probability of stability of >95% for the old road cuts with slope faces of 70° and a height of 5 m. The same rock mass characteristics were used for the new slope as both slopes are in the same 'reference' rock mass as far as the thinly bedded units are considered. For a new road cut with a height of 13.8 m, a 'moderate' degree of rock mass weathering and 'dislodged blocks' due to blasting, the stability assessment was about 8% for a 70° slope. This is in agreement with reality as in its present condition (2000) the rock mass is clearly not able to support a slope of 70° . According to the SSPC system, stability will be achieved if the slope angle is decreased to about 45° .

This example shows that the SSPC classification of slope stability is also applicable in situations where the stability is governed by damage due to the method of excavation and the influence of weathering. If the slope had been designed using the SSPC system, the increased weathering would not have been anticipated as the old road cuts do not show this. However, the new road cut would never have been designed with a steep face of 80° if it was anticipated that perhaps poorly executed blasting would be used.

Discussion

The SSPC system (although based on a large variation of lithologies and rock mass types) has been developed in a particular region, in a particular climate and with particular types of lithologies and rock masses, etc. As for all empirical systems, using the SSPC system on rock masses in an environment that is very different implies a risk.

The quality of a rock mass consisting of a highly inhomogeneous, intensely folded or faulted rock presents a special problem. The rock mass should be divided in geotechnical units in which the rock mass properties are broadly homogeneous and can be calculated for each specific geotechnical unit. If it is impossible to distinguish geotechnical units with a suitably small range of permitted values for properties, due to the limited size of the inhomogeneous areas, the worst case rock mass parameters can be used, although this would probably lead to an over-conservative assessment.

Rock types that are deformed very easily (gypsum, salts, etc.) were included in the development of the SSPC system. However, the stability of slopes in rock masses containing gypsum is influenced more by erosion and weathering (in particular solution of gypsum) than by mechanical deformation of the rock. The SSPC system cannot be used if the strength of the rock mass is governed by deformation of the intact rock.

It should be noted that the SSPC classification was developed for uniform plane slopes, while real slopes (and in particular, those poorly excavated) contain re-entrants, niches, overhangs, etc. which may allow slope movement in directions that would not be possible if the slope was one continuous plane. Rock falls resulting from such slope irregularities are not uncommon.

Generally, the errors made in assessing the rock mass field data by students are larger than would be expected from experienced rock mechanics engineers. Consequently, the slope stability probabilities calculated by the SSPC system may be conservative. In the opinion of the authors, this is not a problem as the SSPC system is likely to be used by experienced and inexperienced users. Experienced users will note that the results based on the SSPC system may be conservative and will interpret the results accordingly while it is highly unlikely that an inexperienced user would be able to recognize that the results are too optimistic and be able to correct for this. Conservatism in the results is therefore considered to be advantageous.

The SSPC system does not assess stability for slopes that are subject to external stresses, such as tectonic stresses or stresses induced by a (large) hill or mountain behind the investigated slope. A criterion for buckling such as proposed for the sliding and toppling criteria could not be developed. This is in agreement with field observations in the research area where buckling as a cause of slope failure is seldom found with almost none of the slopes being sufficiently high and steep for this to occur.

Conclusions

The Slope Stability Probability Classification (SSPC) system provides a better assessment of slope stability than other slope stability classification systems because its three-step approach allows for the incorporation of past and future weathering, the damage due to excavation methods and the assignment of probabilities for different failure mechanisms. The repeatability and reliability of the characterization of rock mass properties are generally good, since the more difficult to measure or ambiguous parameters such as RQD, water and elaborate testing (UCS, shear-box tests, etc.) are not required. The SSPC system was developed using data from 184 stable and unstable slopes. The amount of data and the fact that the data were collected by a large number of different persons at different times eliminate a designer bias in the system.

Susceptibility to weathering is a major factor for the stability of a slope in a rock mass prone to weathering within the engineering lifetime of the slope. The SSPC system quantifies the future strength of a discontinuity and rock mass if the future degree of rock mass weathering can be predicted. This methodology is independent of the climate. The system has recently been used with good results in Austria, South Africa, New Zealand (Lindsay et al. 2000) and in the Dutch Antilles (Rijkers and Hack 2000) and it is considered that it will also be applicable outside these areas.

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