SLOPE STABILITY PROBABILITY CLASSIFICATION

SSPC

2nd edition

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To my mother

for Hanneke

ABSTRACT

The need to include discontinuity properties in slope stability analyses and the poor results of existing rock mass classification systems applied to slope stability has led to the development of a rock Slope Stability Probability Classification (SSPC) system. The system has been developed during four years of research in Falset, province Tarragona, Spain.

The rock slope classification scheme, which has been developed, classifies rock mass parameters in one or more exposures. These are compensated for weathering and excavation disturbance in the exposures and parameters important for the mechanical behaviour of a slope for 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 the influence of excavation method and future weathering. The large quantity of data allowed for the development of a classification system based on probabilities. This resulted in a classification system based on a probability approach: the 'Slope Stability Probability Classification' (SSPC).

Alles Vergängliche
Ist nur ein Gleichnis;

Goethe, Faust II

CONTENTS

Figures and tables with numbers starting with 'A' are included in the appendices and starting with 'G' in the glossary.

Preface

The ideas forming the basis for the research started to develop while the author was employed as underground rock mechanics engineer in the copper mines in Zambia. The experience gained in underground rock mechanics applied in one of the largest underground metal mines in the world, gave the opportunity to develop an insight in rock mechanics and rock mass classification not easily obtained elsewhere. The ideas have been further developed while being employed by ITC[1] and the Technical University Delft[2]. Data could be collected for the development of a rock slope stability classification system during the fieldwork organized for the graduate students in engineering geology of ITC and the TU Delft in the area around Falset in Spain. This resulted in a classification scheme for probability assessment of slope stability: Slope Stability Probability Classification (SSPC).

Part of the research described has already been published in various articles (Hack et al., 1993a, 1993b, 1993c, 1995). For fieldwork also handouts and fieldwork manuals have been published in the years 1990 through 1996. The publications describe the findings of the research at that time. These may not be the same as those described here. During the research the classification system developed via various intermediate systems to the final Slope Stability Probability Classification (SSPC) system. This was the best possible with the data available. However, it is likely that in the future the system will be changed or adjusted when more data will be available. As for all empirical classification systems further development and improvement are possible if more data are available.

Apart from the research done for the development of the Slope Stability Probability Classification (SSPC) system also research for engineering geological mapping has been done during the same period in the area of Falset, Spain. The results of this engineering geological mapping research will be reported on in an engineering geological map and accompanying report and legend (Price et al., in preparation). Some data gathered for the engineering geological map have been used in the development of the SSPC system.

October 1996

Preface 2nd edition

The popularity of the SSPC system necessitated a second edition. This edition is basically unchanged compared to the first edition. The opportunity has been used to correct two inaccuracies which were pointed out to me by John Hutchinson (lubrication by water) and Milkar Vijlbrief (modified Hoek-Brown failure criterion) for which they are acknowledged.

Robert Hack
22 May 1998

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Acknowledgement

I am sincerely in debt to David Price and Niek Rengers for guiding and helping during the research. The regular discussions during coffee or while having a Spanish lunch in 'El Hostal' with a clear view on a spectacular slope (Fig. 5) will not be easily forgotten. To both I am clearly also in debt for the tedious job of critically reviewing and editing of the manuscript.

Johan Kaashoek of the Erasmus University, Rotterdam, Mathisca de Gunst of the Department of Mathematics and Computer Science of the Free University of Amsterdam and Dieter Genske of the Technical University Delft are acknowledged for their help with and review of the statistical methods used.
I also thank my colleagues in ITC and in the Technical University Delft for their help during the fieldwork. A special word of thanks should be given to Willem Verwaal and Arno Mulder of the Technical University Delft for getting the samples and test results used in the research.
I thank ITC, being my employer, for giving the opportunity to do the research and providing the financial support.

The largest contribution to this research is probably made by the graduate students from ITC and the Technical University Delft who collected the data. They provided the data that allowed me to establish the relations and to develop the classification system. Without knowing it, their ideas and sometimes blunt comments on preliminary versions of the classification system helped me to eliminate ambiguous elements and to improve the system.

I like to express my sincere gratitude to the inhabitants of the Falset area in Spain, the City Council of Falset, the Quardia Civil and, in particular, to the staff of the Hostal Sport in Falset. For years they had to put up with students, staff members and me doing 'strange' things to their rocks, hampering traffic, using the swimming pool as site laboratory, being late or too early for dinner and generally being very prominently present. No complaints have ever reached us, on the contrary, they helped us when and wherever possible and provided all facilities necessary for doing the research.

I thank Hanneke for her assistance and moral support while having suffered my often irritating moods, and for her loving care provided.

Robert Hack
1 October 1996

A INTRODUCTION

A.1 THE RESEARCH

A.1.1 Problem definition

In the practice of constructing engineering structures, such as buildings, tunnels and slopes, an interaction takes place between the 'ground' and the engineering structure. The knowledge of the consequences of the influence of the 'ground' on the engineering structure and vice versa are often critical for the economic and safe design of an engineering structure. In particular the mechanical response of the 'ground' under influence of the engineering structure should be known before an engineering structure is built. 'Ground' is a very broad term. The 'ground' is any natural material present at the site where the engineering structure is to be built on or in. 'Ground' is normally divided in 'soil' and 'rock'. 'Soil' consists of loose particles not cemented together whereas the particles in rock are cemented together, resulting in a tensile strength. This difference in characteristics between 'soil' and 'rock' has also resulted in the development of different methodologies for the calculation of the mechanical behaviour of the 'soil' or 'rock'. Most 'rocks' are not continuous, but contain fractures, faults, bedding planes or more general: 'discontinuity[(1)] planes' that divide the 'rock' into blocks of rock bounded by discontinuities. The whole array of blocks of rocks and discontinuity planes is then designated the 'rock mass' or 'discontinuous rock mass'. The research described has been done to develop an improved methodology for the assessment of 'rock' slope stability for 'discontinuous rock masses'.

Discontinuous rock masses

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 recognized 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. Descriptions and characterizations, engineering geological maps and 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. Analytical or numerical calculations should be performed in three dimensions because discontinuities usually make a rock mass

(1) The terms *discontinuous* rock mass and *discontinuity* are used in a rock mechanical sense. A *discontinuity* is a plane that marks an interruption in the continuity and normally has low or zero tensile strength. A *discontinuous* rock mass is a rock mass containing discontinuities. (see further chapter A.2 and glossary, page 241)

three-dimensionally anisotropic. Calculations are, however, usually in two dimensions because of the amount of data needed and the number of calculations required for three-dimensional analyses.
Alternatively numerical methods can be used not as a deterministic method but to produce sensitivity analyses that will give the most likely and worst case scenarios for a rock mass calculation. This, however, may result in a colossal quantity of calculations. The same applies to the various methods of stochastic calculations incorporated in analytical or numerical calculations. The near infinite number of parameters for which values and distributions of values are not or only partly known, prohibits acceptable and fast calculations.

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. A limiting factor is that any classification system is empirical and thus only applicable to engineering applications within the range of experience used to develop that particular system.
The application of rock mass classification systems in civil engineering is, however, still limited because the existing systems were not developed for civil engineering practice, and most civil engineers are unfamiliar with classification systems. An obvious application for a classification system is the assessment of slope stability. Slopes made for road alignments are normally extensive and an appropriate sampling and testing program for an analytical or numerical calculation of slope stability is expensive and often unreliable. It would therefore be very attractive to have a classification system for slopes available that produces stability assessments of equal or better quality.

A.1.2 Scope

The research described is directed towards the development of a new classification system for rock slope stability. The data for the research were collected during four years of research in the Falset area in the province of Tarragona in the northeast of Spain. Within the context of four years of fieldwork with groups of graduate students from ITC and TU Delft in Falset, it was possible to make an extensive study for the development of a classification system for slope stability assessment. In the area new roads have recently been built through a mountainous terrain, necessitating a large number of new road cuts. The height of the slopes in the road cuts is typical between 5 and 25 m with a maximum of about 45 m. Rocks in the Falset area vary from Tertiary conglomerates to Carboniferous slates and include rocks containing gypsum, shales, granodiorite (fresh to completely weathered), limestone and sandstone, thus giving the opportunity to assess rock masses in different lithologies. Different methods of excavation were used for the old and the new road cuts, allowing comparison of the effects of different excavation methods. The road cuts made for old roads some forty to sixty years ago could be compared to road cuts not more than four years old. Also local variations in the degree of weathering, the influence of weathering on rock and rock mass parameters, and the susceptibility of the rocks and rock masses to weathering as a parameter in slope stability could be studied in detail in the area.

Slope Stability Probability Classification (SSPC)
The rock slope classification scheme, which has been developed, classifies rock mass parameters in one or more exposures. These parameters are corrected for the influence of weathering and excavation disturbance in the exposures and parameters important for the mechanical behaviour of a slope 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 and future weathering. This procedure allows a slope design 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 of the rock mass at the location of the slope and the disturbance caused by the method of excavation used for the slope. The large quantity of data collected allowed the development of a classification system based on probabilities, the 'Slope Stability Probability Classification' (SSPC).

Outline
The outline is as follows:
Section A - Introduction
The introduction gives the setting for the research, necessary terminology and definitions for rock and rock masses and a brief description of the fieldwork area used for the research.

Section B - Existing rock mass characterization & classification
Section B comprises a review of existing rock mass characterization and classification systems and parameters used in these systems. Subsequently the classification systems and the parameters are evaluated on their merits for slope stability classification. The conclusions form the basis for the definition of parameters in section C and the development of the new classification system in section D.

Section C - Parameter definition and 'initial point rating' system
This section treats the definition of parameters to be used for slope stability classification, and the results of the 'initial point rating' classification system. The results of the 'initial point rating' system were such that the concept of a point rating system was abandoned.

Section D - Slope stability probability classification - SSPC
The newly developed SSPC system that classifies slope stability based on probabilities, is described and evaluated in section D.

A.2 INTACT ROCK VERSUS ROCK MASS

A rock mass may consist of intact rock only, but is more commonly formed from an array of intact rock blocks with boundaries formed by discontinuities (Fig. 1). Within the rock mass the mechanical properties of both the intact rock blocks and the discontinuities may be inhomogeneous and anisotropic. A common relation between rock, rock mass and engineering is (Price, 1984):

$$\textit{material properties} + \textit{mass fabric} = \textit{mass properties}$$
$$\textit{mass properties} + \textit{environment} = \textit{the engineering geological matrix}$$

$$\frac{\textit{the engineering geological matrix}}{\textit{changes produced by the engineering work}} = \textit{the engineering behaviour of the ground}$$

Exact descriptions of rock material and rock mass are required for understanding the analyses in this research and follow below.

A.2.1 Rock mass components

Intact rock material
Intact rock blocks are blocks of rock that do not contain mechanical discontinuities and do have tensile strength.

Discontinuities
A discontinuity is a plane or surface that marks a change in physical or chemical characteristics in rock material. A division is made between integral discontinuities and mechanical discontinuities. The latter are planes of physical weakness. Bedding planes, joints, fractures, faults, etc. are mechanical discontinuities if the tensile strength perpendicular to the discontinuity or the shear strength along the discontinuity are lower than those of the surrounding rock material (ISRM, 1978b, 1981a). Integral discontinuities are discontinuities which are as strong as the surrounding rock material. Integral discontinuities can change into mechanical discontinuities due to weathering or chemical reactions that change the mechanical characteristics. Throughout this book 'discontinuities' denote mechanical discontinuities except where stated otherwise.

Discontinuity set
Discontinuities exist as single features (fault, isolated joint or fracture, etc.) and as discontinuity sets or families (bedding planes, schistosity, cleavage, joints, etc.)[(2)]. A set denotes a series of discontinuities for which the geological origin (history, etc.), the orientation, spacing and the mechanical characteristics (friction angle, roughness, infill material, etc.) are broadly the same. In some circumstances a discontinuity is treated as a single discontinuity although it belongs to a discontinuity set, in particular if the spacing is very wide compared to the size of the engineering application or to the size of the geotechnical unit (ch. C.3.4.1).

[(2)] Various geological processes create discontinuities at a broadly regular spacing. For example, bedding planes are the result of a repeated sedimentation cycle with a change of sedimentation material at regular intervals, folding creates joints at regular separations to allow for shrinkage or expansion of the rock material, etc.. Normally discontinuities with the same origin have broadly the same characteristics in terms of roughness, infill, etc.. The orientations of discontinuities with the same origin are related to the process that has created them and to the geological history of the rock mass.

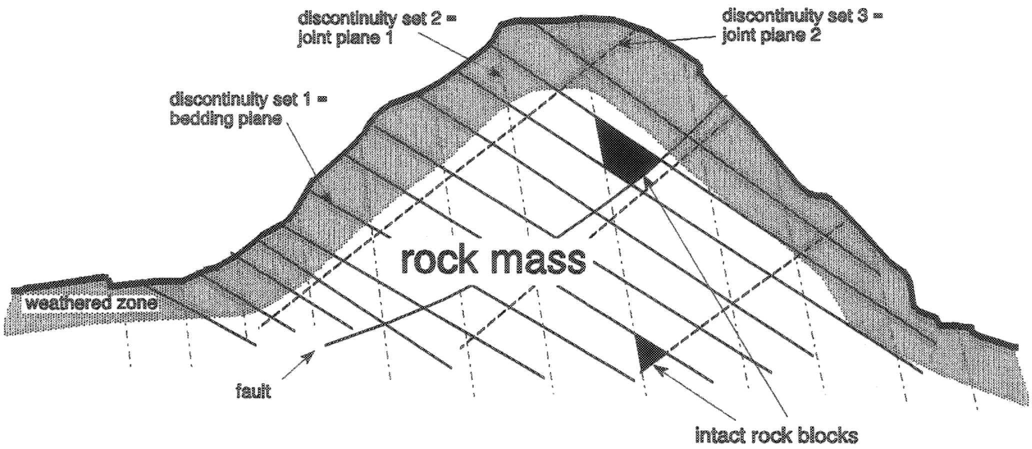


Fig. 1. Intact rock vs rock mass.

Inhomogeneity

Inhomogeneity is the spatial variation of rock material or rock mass properties. For example, an intact rock strength variation within a block of intact rock material causes the intact rock material to be inhomogeneous; a variation in the orientation of discontinuities causes a rock mass to be inhomogeneous. In this research it is taken that inhomogeneity results in new boundaries in the rock mass. This is not a discontinuity boundary but a boundary defined by a change in intact rock material or rock mass properties. Normally this boundary will coincide with a geotechnical unit boundary (ch. A.2.2). Similarly a gradual change in the orientation of a discontinuity set causes a rock mass to be inhomogeneous, also leading to an arbitrarily established geotechnical unit boundary.

Anisotropy

An isotropic body has equal properties in all directions. Discontinuities in a rock mass induce anisotropy. A simple case of anisotropy is illustrated in Fig. 2. The rock is regularly intersected by series of discontinuities ('d') filled with a material different from the rock material ('i') between the discontinuities. The properties in any direction in the xy plane are of rock material or of the infill material. Properties in the z direction depend on the combination of the properties of the rock and the infill material. The total block including discontinuities will, as a result, have different properties in different directions.

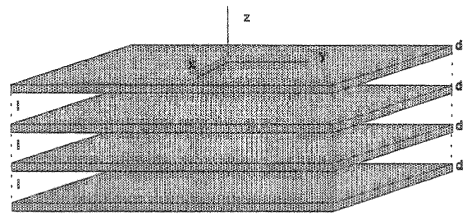


Fig. 2. Anisotropic rock mass.

Rock mass

A rock mass is an assemblage of rock blocks with discontinuities, with or without inhomogeneity and with anisotropy (Fig. 1). The overall effect of discontinuities is that a rock mass that contains discontinuities, is weaker than the intact rock because shear and tensile strengths of the discontinuities are lower than those of the intact rock material. A rock mass containing discontinuities will be more deformable than intact rock. Such deformation will normally take place by relative movement along discontinuities and be plastic rather than elastic (ch. A.2.4). The tensile strength of a rock mass containing discontinuities is low and for many rock masses zero. The porosity of a discontinuous rock mass is higher due to the storage capacity of the discontinuities and the permeability is often considerably higher due to the conductivity via the discontinuities. Discontinuities always lead to an anisotropic behaviour of the rock mass and all rock mass properties, such as deformability, permeability, etc.. Therefore a discontinuous rock mass is a three-dimensional feature that is anisotropic in three dimensions.
A classical example of the influence of discontinuities in a rock mass on the stability of a tunnel is illustrated in Fig. 3. During excavation of the diversion tunnel for the Castaic dam (40 miles north of Los Angeles, USA) an overbreak occurred. The overbreak was improperly backfilled, which allowed de-stressing of the rock mass around the tunnel. By de-stressing the rock mass the clay lined bedding plane on the left of the tunnel was de-stressed in the direction normal to the plane resulting in a lower shear strength along the bedding plane. This allowed movement of part of the rock mass in the direction of the tunnel, destroying the support and resulting in a complete collapse of the tunnel.

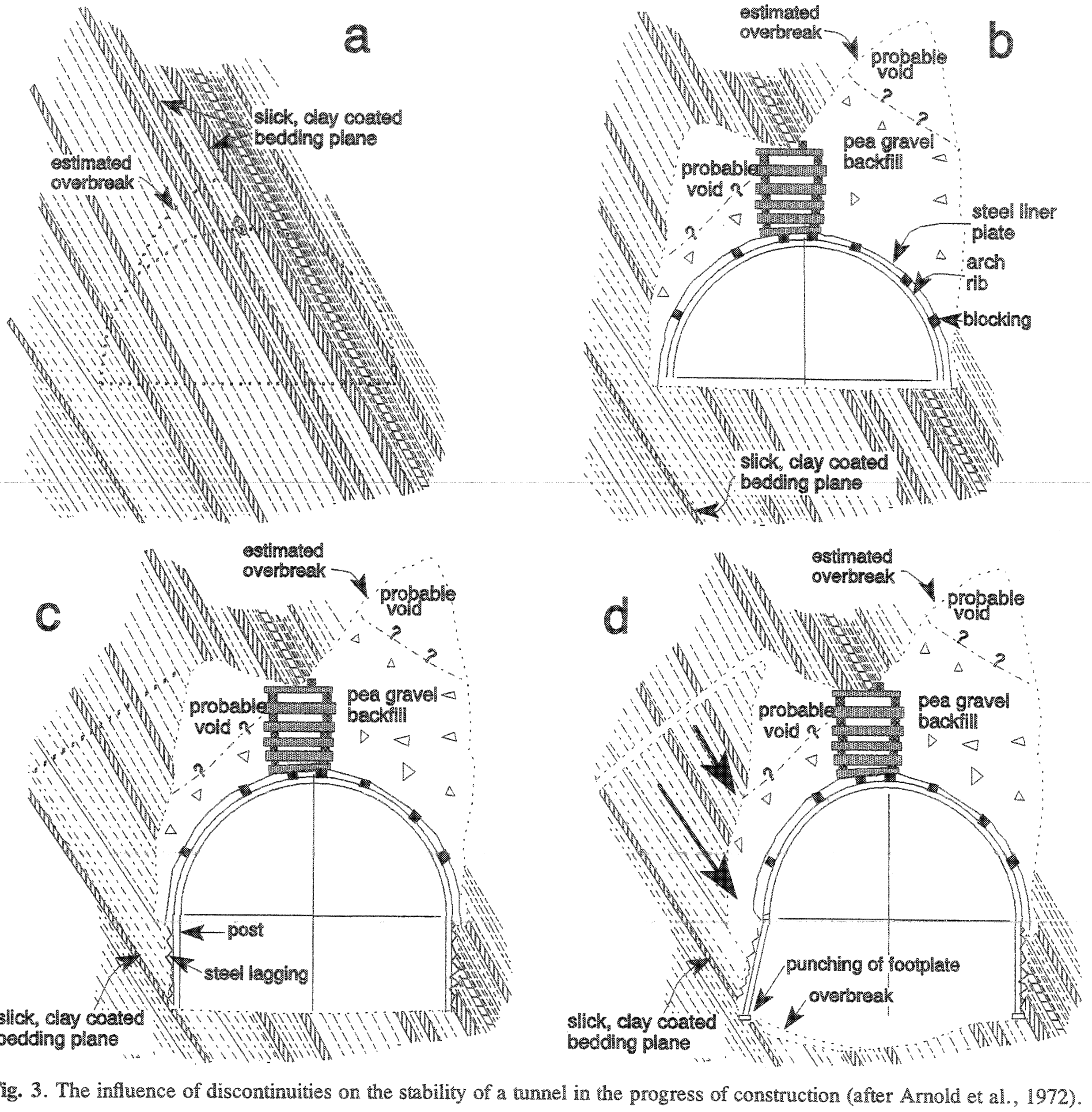


Fig. 3. The influence of discontinuities on the stability of a tunnel in the progress of construction (after Arnold et al., 1972).

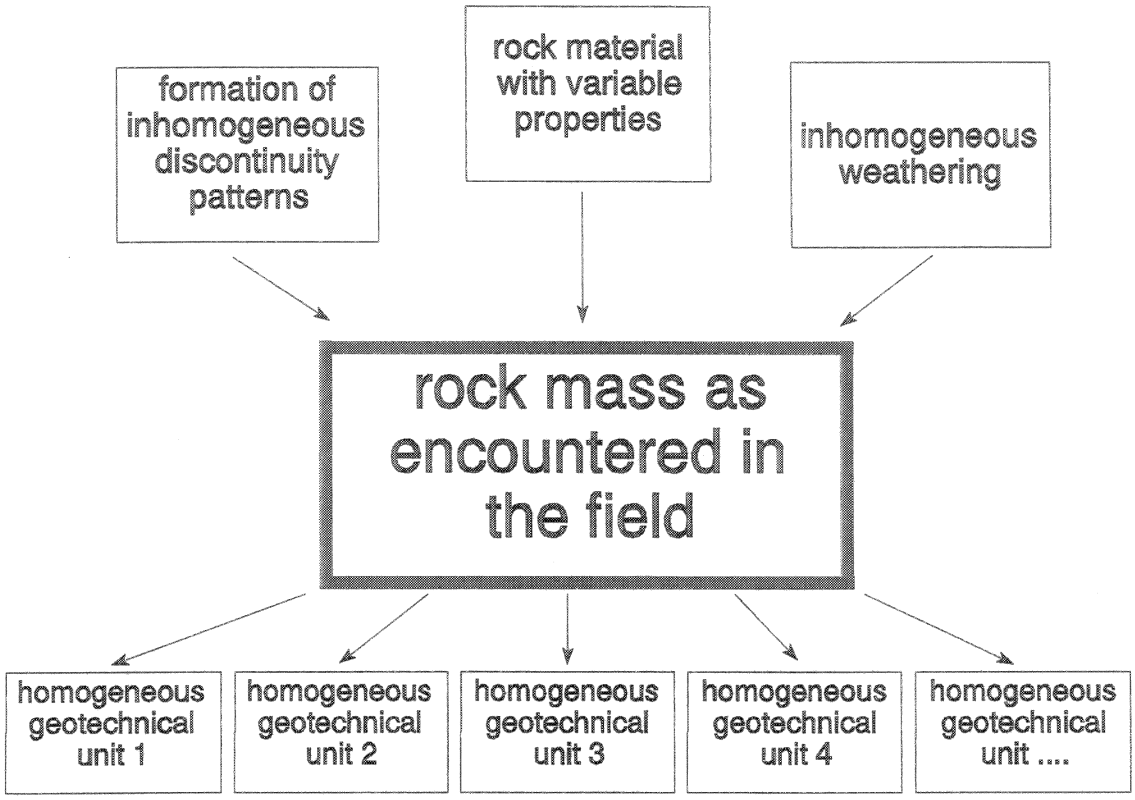


Fig. 4. Rock mass components.

A.2.2 Geotechnical units

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[3]. Fig. 4 shows a schematic visualization of a rock mass and its division in 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.

(3) A geotechnical unit is, in theory, a part of the rock mass in which the mechanical properties of the intact rock material are uniform and the mechanical properties of the discontinuities (including anisotropy of properties) within each set of discontinuities are the same. In this research the anisotropy of properties in a geotechnical unit is also uniform. This additional condition is not always specified in the literature, however, in engineering it is an obvious requirement because of the large influence of anisotropic features (e.g. discontinuities, etc.) on engineering as explained in the previous pages.

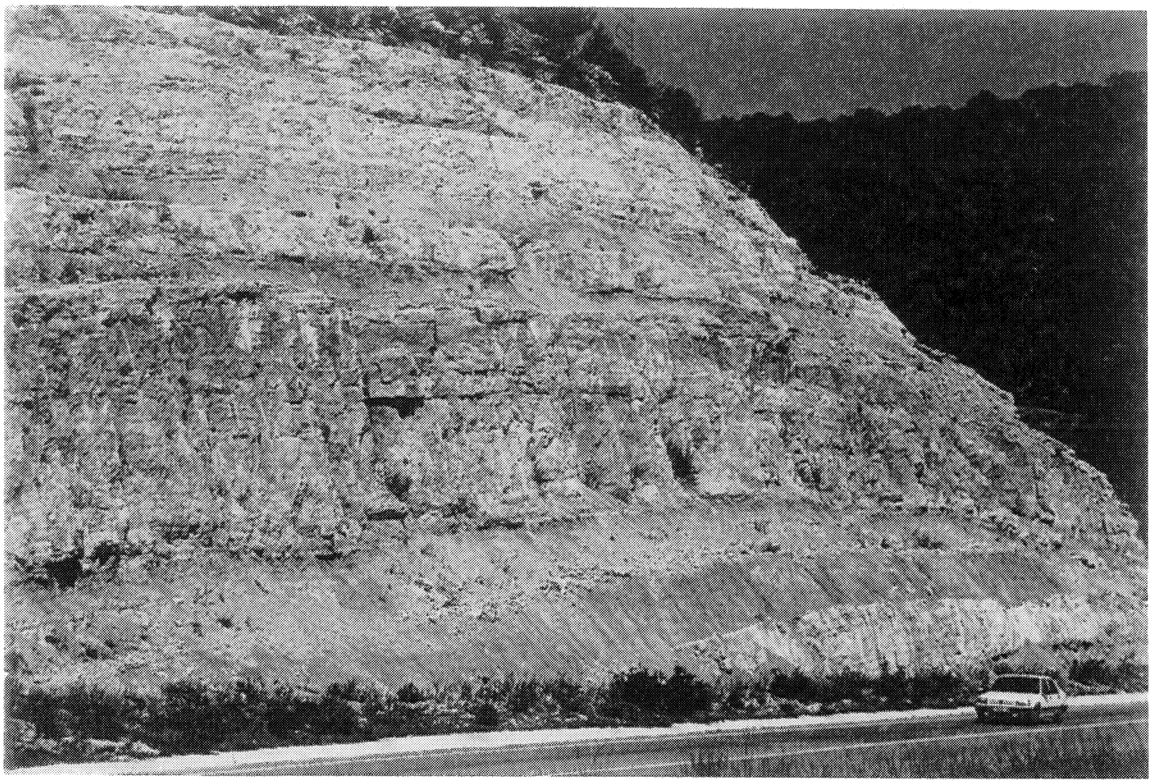


Fig. 5. Different geotechnical units in a single slope. Greenish and blueish grey layers consist of calcareous shale and brownish, pinkish off-white layers consist of dolomite and limestone.

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. In Fig. 5 and Fig. 6 a slope is shown in which different geotechnical units are present. The influence of the different geotechnical units on the form of the slope is clearly visible through the changes in slope surface steepness.

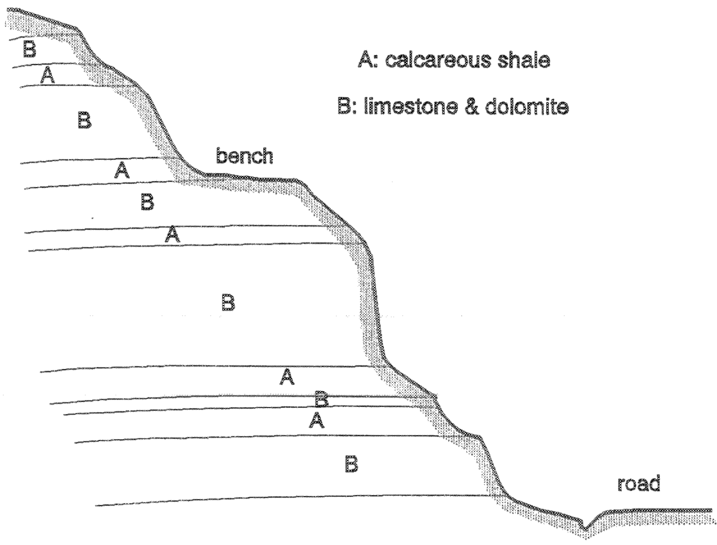


Fig. 6. Section through the slope of Fig. 5.

A.2.3 Water

Water influences the mechanical characteristics of a rock mass. Water adds to the weight of the rock mass, acts as a lubricant in discontinuities, causes softening of some infill materials (e.g. clay), and water pressure in discontinuities reduces the shear strength along discontinuities and thus also the (yield-) strength of a rock mass. Therefore it is necessary to consider whether water should be treated as part of the rock mass and the geotechnical units. In this respect it must be noted that water is often not a continuous feature in time. Water can be present during and just after rainfall and absent during long dry periods. Also the engineering structure to be built might influence the presence of water (e.g. drainage around tunnels, saturation of the rock mass due to an impounded reservoir, etc.).

With time, most rock masses weather, a process strongly influenced by the presence of water, which causes the intact rock strength and the discontinuity strength parameters to decrease. To what extent weathering influences the mechanical behaviour of a rock mass depends on the type of engineering application, type of intact rock material and discontinuity infill material, amount and chemistry of percolating water, etc..

Reduction of shear strength of discontinuities due to water
Water pressures in a discontinuity reduce the normal stress on the discontinuity and therefore reduce the shear strength along the discontinuity. Sliding over a discontinuity plane is then possible at a lower dip angle than over a discontinuity without water pressure (Fig. 7a, b and c). In traditional limiting-equilibrium calculations for slope stability, water pressures in discontinuities are therefore a main reason for slope instability to occur (Hoek et al., 1981, Giani, 1992). In Fig. 7a, b and c it can easily be seen that the discontinuity dip angle for which equilibrium exists decreases ($\alpha > \beta > \gamma$)[4].

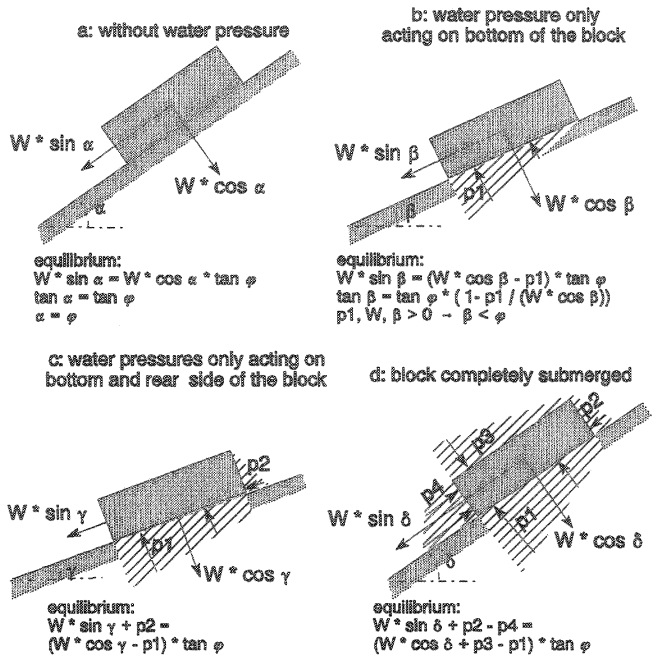


Fig. 7. Block on slope with and without water pressure (W is the weight of the block, cohesion along discontinuities is zero).

Accordingly, because both effects (pressure and weathering) of the presence of water might or might not be present, water is not included in the rock mass or in the geotechnical unit. The influence of water should, however, be included in any calculation of the behaviour of the geotechnical units.

A.2.4 Characteristics of intact rock and rock mass

A description of some geotechnical properties and characteristics of rock and rock mass is given hereafter. The properties and characteristics are described as far as important for the development of a slope classification system and not in all detail. The underlying mechanisms are only briefly addressed as a full description of all mechanisms in discontinuous rock mechanics would be beyond the scope of this study. The reader is referred to the standard literature for further details (Giani, 1992, Goodman, 1989, Hoek et al., 1980, 1981, etc.).

Stress distribution in a rock mass
The stress distribution in a rock mass is strongly influenced by the presence of discontinuities. Fig. 8 shows examples of a stress distribution in intact rock and in discontinuous rock masses. The figures clearly show the variation in the stress contours due to the presence and orientation of discontinuities.

Deformation
Deformation of intact rock is the change in volume or shape of intact rock under the influence of deforming loads. In general, the deformation of intact rock is partly elastic and partly plastic and some rocks also show a time dependent deformation (see also creep, below). Deformation of a rock mass is the change in volume or shape of the rock mass. The deformation is mainly caused by displacements of intact rock blocks along or perpendicular to discontinuities.

[4] If rock blocks are completely submerged in water (Fig. 7d) the normal stress on the discontinuity is reduced (p1 > p3) causing a reduction in shear strength, but also the driving forces are reduced (p4 > p2). In a completely submerged slope the equilibrium between driving forces and shear strength is, therefore, less disturbed than in a situation with water pressures acting only on bottom and rear sides of the block (Fig. 7b and c). In slopes the rock blocks near the surface of the slope are normally not completely submerged in water and therefore water pressures cause a reduction in normal stress along the discontinuity plane (Fig. 7b) and driving forces may increase if a discontinuity at the rear of the block is filled by water (Fig. 7c).

The discontinuities cause a dramatic change in deformation behaviour of a rock mass in comparison to that of intact rock. The deformation in a rock mass is for a large part due to shear displacements along discontinuities or opening or closure of discontinuities. The shear deformations are non-elastic for larger displacements. Whether the opening or closure of discontinuities is elastic or non-elastic depends on the infill material in the discontinuities and the discontinuity wall material but usually the displacements are non-elastic (e.g. for a common infill material such as clay). Therefore, a rock mass shows mostly non-elastic deformation behaviour. Fig. 9 and Fig. 10 illustrate the non-elastic deformation behaviour of rock masses.

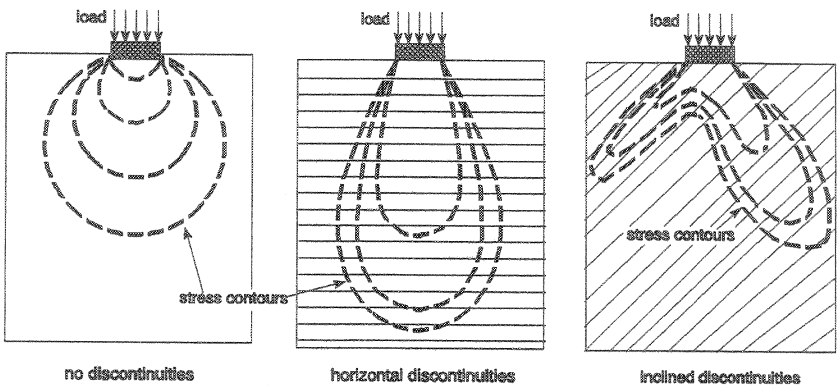


Fig. 8. Stress distribution (bulbs of pressure - lines of equal major principal stress) in a rock mass due to a vertically oriented plane load (after Gaziev et al., 1971).

Rock mass failure

In general a rock mass does not fail and therefore failure of a rock mass is usually defined as the deformation of the rock mass larger than allowed for a particular engineering construction.

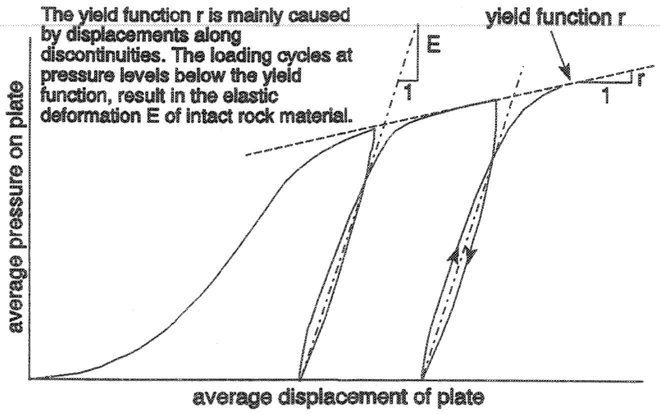


Fig. 9. Example of a cyclic plate-bearing test on fractured rock (after Schneider, 1967).

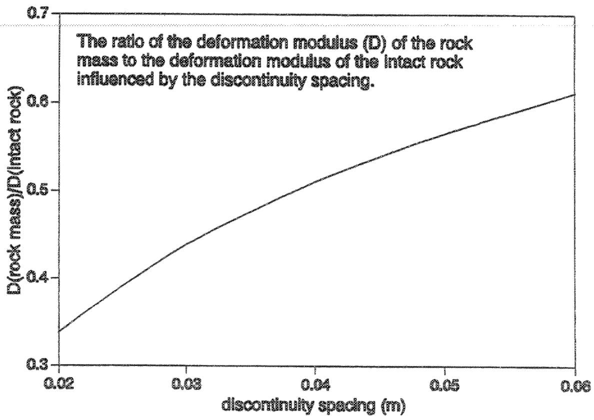


Fig. 10. $D_{intact\ rock}/D_{mass}$ vs discontinuity spacing for plate diameter 8 cm on a model rock mass (after Berkhout, 1985).

Compressive, tensile and shear strength of intact rock

Intact rock material has compressive[(5)], tensile and shear strength. Rock material consists of mineral grains completely or partially bonded together by cement or another bonding agency. If loaded to failure under a compressive, tensile or shear stress, intact rock material will break into smaller pieces of rock when the compressive, tensile or shear strength is reached ('the rock fails'). Intact rock strength behaviour may be approximated with a 'Mohr-Coulomb failure criterion'[(6)]. This allows definition of the intact rock strength in terms of intact rock cohesion and intact rock friction.

Strength of a rock mass

The 'strength' of a rock mass, as often used in the literature or in day-to-day practice, is a confusing and false expression. A rock mass may be considered to have strength, but, due to the discontinuities in a rock mass, this strength is dependent on a variety of factors: the shape and size of the rock mass considered, the environment (e.g.

(5) The compressive strength is dependent on the test method, see glossary, page 241.

(6) See glossary, page 241. Note: the Mohr-Coulomb failure criterion does not suit all rocks in all situations and different theoretical or empirical models for which the strength of intact rock have been defined. These will not be repeated here as these can be found in any standard text book on rock mechanics (e.g. Goodman, 1989, Hoek et al., 1992).

the engineering application, the confining stresses, etc.), the amount and orientation of discontinuities and, although in many situations of minor importance, the intact rock strength. Consider the sketch in Fig. 11. The rock mass (including the orientation of the discontinuity) and the stresses on the rock mass are in both cases the same. Only the volume of the rock mass is changed. It is easily seen that the rock mass in Fig. 11a has a higher 'strength' than in Fig. 11b. In Fig. 11a intact rock has to be broken, and in Fig. 11b sliding along the discontinuity is sufficient for 'failure'.

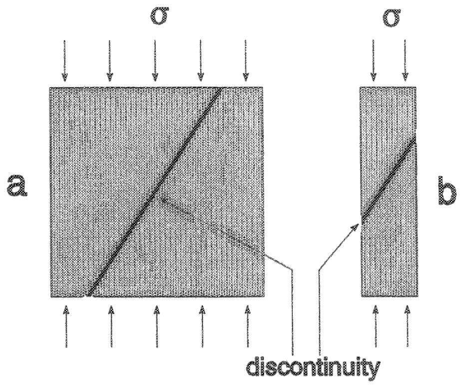


Fig. 11. Rock mass under stress.

Tensile strength of a rock mass
The bonding strength between the particles causes the tensile strength of intact rock. A rock mass with discontinuities has only a tensile strength if the discontinuities have a tensile strength or are filled, coated or cemented with a material that has a gluing or bonding effect between both sides of the discontinuity. For most rock masses at (near-) surface this is not true and most rock masses have a tensile strength equal to zero.

Compressive and shear strength of a rock mass
A rock mass consists of rock blocks bounded by discontinuities which have shear strength and may have some tensile strength. The rock mass could thus be considered as a large scale rock material, rock blocks replacing mineral grains. In a rock mass with discontinuities which have a tensile strength, the bonding agent causing the tensile strength may be broken due to compressive or shear loading. This is comparable to the failure of intact rock material and compressive and shear 'strength' may be defined, although these 'strengths' are likely anisotropic and may still depend on the environment. If the discontinuities do not have tensile strength the rock mass may be compared to not cemented dense sand, where grains, being the intact rock blocks, fit closely together. The environment (confinement, etc.), the shear strength along the discontinuities, and the intact rock strength determine the maximum compressive and shear load that can be sustained[(7)]. Thus 'failure' depends on the configuration of the rock mass and the orientation and variation of the stress fields. Generally valid compressive and shear strength values can therefore not be defined[(8)]. In some situations where anisotropy is absent or not very important, it is, however, possible to approximate the strength behaviour of a rock mass in models analogous to the methods used for intact rock, but with strongly reduced values for compressive and shear strength.

Weathering
Weathering is the chemical and physical change in time of intact rock and rock mass material under the influence of the atmosphere and hydrosphere. Two main processes are distinguished: physical and chemical weathering. Physical weathering results in the breakdown of rock material into progressively smaller fragments. The rock and rock mass break up due to temperature variations resulting in differential expansion and shrinkage of minerals, freezing and thawing of water, pressures of water in pores and discontinuities, (re-) crystallization pressures, hydration, and frequent swelling and shrinkage of clays due to water absorption, etc.. Chemical weathering results in decomposition of minerals. Water and groundwater with dissolved chemical agents are of major importance as these react with rock and rock mass material. Normally biotic influences, induced by living organisms, plants, bacteria, worms, etc., are included and cause physical as well as chemical weathering. On or near to the surface the influence of these processes (due to larger temperature variations, influence of vegetation and rain, etc.) is more distinct than deeper below the surface. In this research also the effects of stress relief, intact rock creep and rock mass creep are included in the definition of weathering as proposed by Price (1995). Intact rock, and rock

(7) Comparing a rock mass to intact rock or to an uncemented sand is only partly valid. The elements in a rock mass (rock blocks) fit together like dry masonry, whereas the grains in intact sedimentary rock or in a sand do usually not fit together. The cement in a rock mass is in the discontinuities whereas in intact rock or in a sediment the elements (grains) are bound together by a cement filling the pores between the grains.

(8) An alternative way to understand rock mass 'strength' is as follows: If loaded to failure under a compressive or shear stress a piece of intact rock will break into smaller pieces of rock when the compressive or shear strength is reached ('the rock fails'). Effectively it then becomes a rock mass (intact pieces of rock with boundaries by fractures = discontinuities). Reversed this leads to the conclusion that a rock mass does not have a compressive or shear strength; it already consists of blocks with boundaries by discontinuities.

mass creep and stress relief can lead to new cracks in intact rock, develop integral discontinuities into mechanical discontinuities and open existing discontinuities.
A distinction is made between 1) the degree (state) of weathering and 2) the susceptibility to weathering. The degree of weathering denotes the state of weathering of a particular rock mass or geotechnical unit at a certain moment. Susceptibility to weathering is the susceptibility of the rock mass to further weathering in the future.
The influence of weathering on intact rock and on discontinuities is as follows:
Weathering of intact rock - Weathering of intact rock discolours the material and decreases intact rock strength. Further progressive weathering of minerals and cement may lead to a decomposition of the intact rock ultimately resulting in a residual soil. New cracks may develop in blocks of intact rock.
Weathering of discontinuities -The discontinuity wall material and the infill material are, in general, weakened, resulting in lower shear strength along the discontinuities. The material resulting from weathering of the discontinuity walls will often form an infill in the discontinuities. The discontinuity wall loses its asperities and becomes smoother. Integral discontinuities can develop into mechanical discontinuities. The discontinuities become visible and can therefore be measured (Price, 1993) resulting in lower values for discontinuity spacings.

Creep

Creep in rock mechanics is a confusing term. Various forms of plastic or time dependent deformation processes which are governed by totally different physical or chemical processes are all described as creep. In this study the term creep is avoided as much as possible, but if used, the process responsible for the creep will be named. The following are examples of 'creep'.
Creep in intact rock - Creep in intact rock usually means that the intact rock deforms with time under a constant load. The velocity of the deformation depends on the level of the load. Creep deformation takes place by solution and recrystallisation of minerals, or by the growth of microcracks into larger cracks, sometimes leading to failure. Both require time and are dependent on stress levels.
Creep in a rock mass - In a rock mass all processes of creep in intact rock may occur, together with time and stress level dependent deformation along and perpendicular to discontinuities.
Slope creep - Slopes are said to creep if the surface layer of the slope moves downhill in a slow process under influence of gravity. Underlying mechanisms are: deformation of intact rock and displacements along existing discontinuities. Processes such as weathering of the intact rock (growth of new mechanical discontinuities) and discontinuity infill material, and creep in intact rock and rock mass are normally also included. The process is facilitated by fluctuations in pore and discontinuity water pressures from water flowing over and through the surface of the slope.

Porosity

Porosity is defined as the pore space (space not occupied by rock material and filled by vapours or fluids) in intact rock or in a rock mass. Porosity is divided in primary and secondary porosity. Primary porosity is the porosity of intact rock and secondary porosity is the porosity of the rock mass due to discontinuities.

Permeability

Permeability is a property of the rock material or mass and describes the ease with which a fluid may move through it. Primary permeability is the permeability of intact rock whereas the secondary permeability is the permeability of the discontinuity system in a rock mass. The permeability of intact rock is usually lower than that of the rock mass.

A.3 THE RESEARCH AREA

The research for the development of a slope classification system has been carried out in the area around Falset in northeast Spain, in the province of Tarragona (Fig. 12). The area around Falset is particularly suitable for the type of research described because:

1. The variation in geology, lithology and tectonic environment is large, giving different geological environments for the development of the classification system.
2. The topography is mountainous and vegetation is limited, exposing large areas of rock.
3. Access to the area and to rock exposures along existing roads and paths is not difficult.
4. Numerous old roads exist and several new roads have been built in recent years creating large numbers of road cuts, excavated with different excavation techniques. This has allowed for the comparison of stand-up times of slopes, excavation methods and for an assessment of weathering influences.
5. Aerial and satellite images, topographical and geological maps at various scales are available.

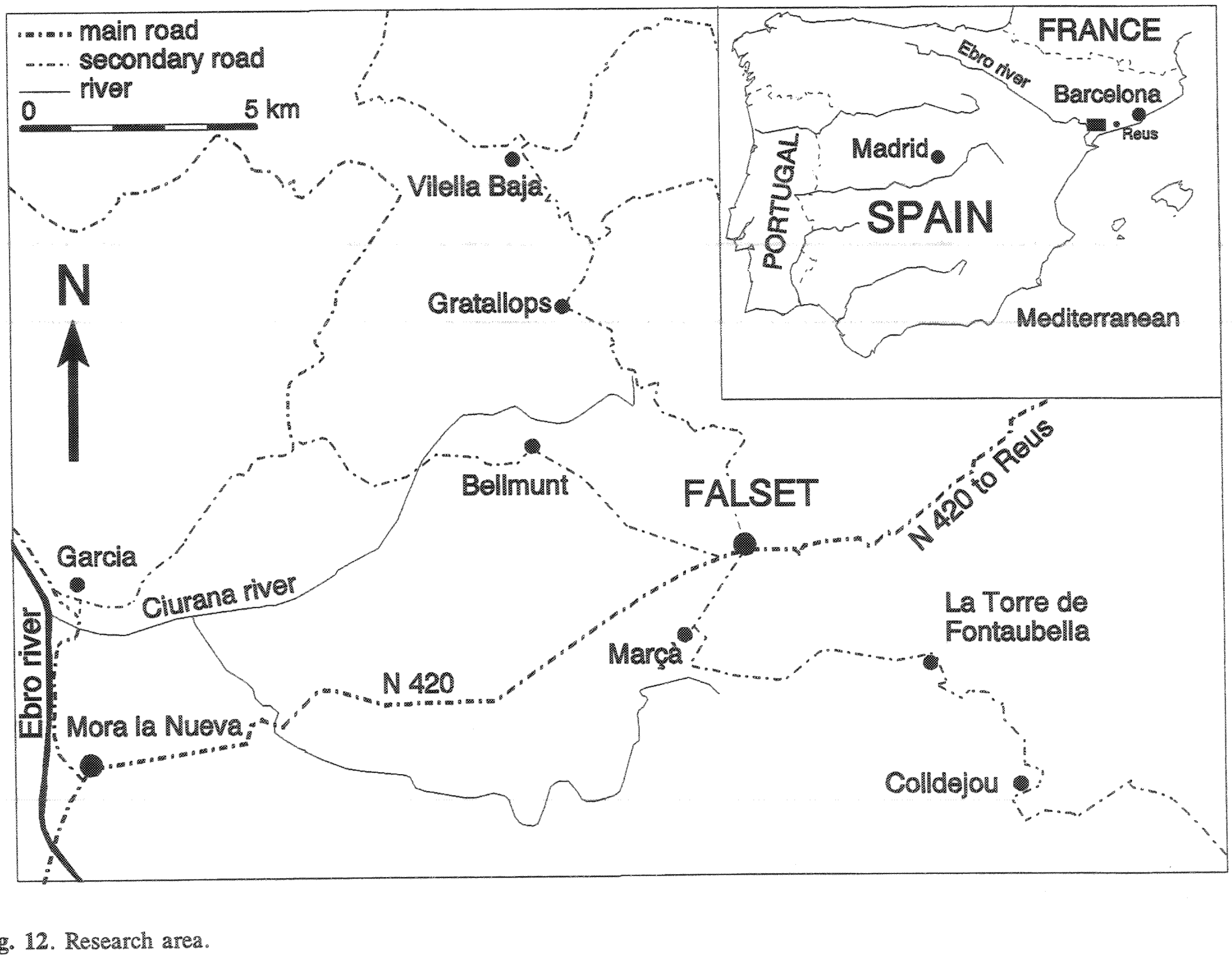


Fig. 12. Research area.

Apart from the research for the development of a slope stability classification system also engineering geological mapping has been carried out in the area. The results of this mapping will be reported on in the form of an engineering geological map and accompanying report and legend (Price et al., in preparation). The engineering geological map and report are, however, not part of this study. Detailed descriptions of topography, geology, and engineering geological mapping units are thus omitted. Details of the area and the geology are summarized below as far as is necessary for understanding the analyses that result in the slope stability classification system.

A.3.1 Climate and vegetation of the Falset research area

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°). Part of the area is mountainous, ranging up to about 1000 m above sea level. Rivers and streams in the area are mostly dry from March through October/November. It can rain for long periods during the winter and even up to March/April although this is not typical. Sometimes the rain is torrential. Occasionally temperatures below zero do occur. Snowfall is seldom in the area, but can fall in March which is the fieldwork season.
Extensive agricultural use is made of the soft soils and weathered rocks in the valleys. The more mountainous areas are covered with forests or are barren rock.

A.3.2 Geological and engineering geological characteristics of the Falset research area

In the Falset area the stratigraphy is composed of sediments of Devonian through Quaternary age and intrusive rocks from Carboniferous through Permian age. A generalized geological table with the lithology and the main engineering characteristics is given in Table 1. The table only presents a broad impression of the engineering geological mapping units found in the area and is in no way complete in all details.

Sedimentary rocks

- The Palaeozoic consists predominantly of slates interbedded with micro-conglomerates, sand- and siltstones. A low degree of regional metamorphism developed cleavage in the slates. Contact metamorphism has affected the Carboniferous rocks near granodiorite intrusions.
- The Triassic corresponds with the Germanic facies type for Triassic sediments. It is characterized by massive or very thick bedded sandstones with some conglomerate beds at the base (Buntsandstone), followed by thick bedded limestones and dolomites (Lower Muschelkalk), intensely folded and deformed sandy clayey siltstone with gypsum (Middle Muschelkalk) and limestones and dolomites of the Upper Muschelkalk. The youngest formation in the Triassic (Keuper) is a sequence of shales, in the lower part interbedded with limestones and dolomites.
- The Jurassic consists of a series of formations of limestones and dolomites, with broadly similar engineering characteristics.
- The Cretaceous is represented from the Albian upwards. The Albian consists of (cemented) sands and clays. The remaining Upper Cretaceous consists of limestones and dolomites, with broadly similar engineering characteristics.
- The Tertiary is mainly marly-arenitic, with an alternation of cemented conglomerates and (not or very weakly cemented) sand and clay layers. The upper part contains limestones and marls.
- The Quaternary is widespread, mainly as superficial gravelly and sandy slope deposits, fine grained sand and silt deposits on flat areas which are likely of aeolian origin (loess), and gravel in river beds and as terraces.

Intrusive rocks

Extensive bodies of igneous rocks occur intruded into the Carboniferous formations as granodiorite bodies and aplitic dykes. The intrusions are from Carboniferous through to Permian age and are probably associated with the Hercynian orogeny.

GENERALIZED GEOLOGICAL TABLE & DESCRIPTION AND MAIN ENGINEERING CHARACTERISTICS OF THE LITHOLOGY IN THE FALSET AREA(1)					
CENOZOIC	Quaternary			Q	GRAVEL terraces along and in rivers; SAND/SILT/CLAY often on flat agricultural areas (also eolian origin).
	Tertiary(3)	Miocene, Oligocene, Eocene		T	Brown/yellowish, cemented, CONGLOMERATE layers (massive up to metres thickness) interbedded with brown/yellow, clayey SILT AND SAND layers, in top: LIMESTONE and calcareous silty SAND layers.
MESOZOIC (3)	Cretaceous	Upper Cretaceous(4)		C	Off-white/l.grey, argillaceous to arenaceous, medium bedded to massive, medium to v.large blocky, jointed, slightly weathered, LIMESTONE AND DOLOMITE, strong.
		Albian		C 16	Red/ochre, SANDS AND CLAYS, at some locations weakly cemented.
	Jurassic(4)			J	Off-white/l.grey, argillaceous to arenaceous, medium bedded to massive, medium to v.large blocky, jointed, slightly weathered, LIMESTONE AND DOLOMITE, strong.
	Triassic	Keuper		Tg3	Red/green/greenish blue/brown/yellow/off-white, argillaceous to fine arenaceous, thinly laminated to v.thin bedded, v.small blocky, jointed, often folded and deformed, slightly to completely weathered, calcareous sandy silty SHALES, v. to mod. weak, with (small) quantities of gypsum. Bottom: Interbedded with layers (20 - 100 cm) off-white/l.grey, argillaceous to fine arenaceous, v.thin bedded, v.small blocky, jointed, LIMESTONE AND DOLOMITE, mod.weak to mod. strong.
		Muschel-kalk	upper	Tg23	Off-white/l.grey/yellowish grey, argillaceous to fine arenaceous, thick laminated to massive, v.small to v.large blocky, jointed, slightly weathered, LIMESTONE AND DOLOMITE, mod.strong to strong.
			middle	Tg22	Red (occasionally greenish grey), argillaceous to fine arenaceous, thinly laminated to v.thin bedded, jointed, often folded and deformed, slightly to completely weathered, gypsiferous clayey sandy SILTSTONE, v. to mod. weak; large quantities of gypsum up to occasionally more than 80 %.
			lower	Tg21	Off-white/l.grey, arenaceous, medium to thick bedded, medium to large blocky, jointed, slightly weathered, LIMESTONE AND DOLOMITE (CALCARENITE), strong.
		Buntsandstone.		Tg1	Red/brown, rudaceous (bottom) to fine arenaceous (top), v.thick bedded to massive, slightly weathered, SANDSTONE, mod.strong.
PALAEO-ZOIC	Carbonife-rous(2)			Hs, H	Thick sequences (> 100 m) of d.grey, argillaceous, thinly spaced cleavage, thinly bedded, small to medium blocky or tabular, jointed, folded, slightly to mod. weathered, SLATE, mod. strong to strong, interbedded with sequences (5 - 100 m thick) of grey/brown, thin to thick bedded, medium to large blocky, jointed, folded, slightly weathered, MICRO CONGLOMERATES, SANDSTONES AND SILTSTONES, mod. strong to extr. strong; folding 1 to > 10 m in slate and > 10 m in other. At two locations 10 to 50 m thick layer of black (with white 5 - 10 mm bands), medium grained, massive, fresh, GNEISS, v. strong to extr. strong.
	Devonian(2)			D	Layers (6 cm) of black, argillaceous, thinly laminated, schistose, folded, slightly to mod.weathered, ORGANIC SHALE, v.weak, interbedded with layers (10 cm) of off-white/brown, amorphous, v.small to small blocky, jointed, folded, RADIOLARIAN CHERT, v.strong; intensive multiple folding from 10 cm to > 10 m.
	Intrusives in Carboniferous	late Carboniferous through Perm		γη/ FO2	L. to d. grey, fine to coarse grained, small to medium blocky, jointed, slightly to highly weathered (also residual soil), GRANODIORITE (sometimes porphyritic), v.weak to extr.strong.
					D. grey, v.fine to fine grained, v.small to small blocky, jointed, slightly to mod. weathered, APLITIC DYKES, mod. strong to v.strong (intrusive in Carboniferous sediments and granodiorite).

Codes (Q, T, C16, etc.) refer to codes used on the geological map sheets, no. 444, 445, and 471, of the area prepared at a 1 : 50 000 scale by the Instituto Geologico y Minero de España. Only main codes are included. Notes:

1 Description for rock units according to BS 5930 (1981). (l. = light; d. = dark; v. = very; mod. = moderately; extr. = extremely)
2 Palaeozoic sedimentary rocks intensively folded under Hercynian orogeny (Carboniferous through Perm); folded on a scale of metres to 10's of metres.
3 Mesozoic folded under influence of the Alpine orogeny (late Cretaceous through Miocene); folding on a scale of 100 m to km's, Tertiary faulted and tilted.
4 Jurassic and Upper Cretaceous consist out of various formations with similar engineering characteristics.
5 Weathering indication characterizes the degree of weathering typically found in surface exposures.

Table 1. Geological table and description and main engineering characteristics of the lithology of the Falset area.

Tectonics and structural geology

Hercynian orogeny
Devonian and Carboniferous sediments have been intensively folded during the Hercynian orogeny (Upper Carboniferous through Permian). The scale of the tight to isoclinal folds starts at metre scale up to 10's of metres; larger scale folding might be present but has not been observed due to the absence of clear marker beds. The axial planes are mostly shallowly dipping in north-northeast directions. The folding is asymmetric with north flanks considerably larger than south flanks, and is probably associated with a regional metamorphism of chlorite to lower greenschist facies. This has altered the Devonian and Carboniferous rock minerals and resulted in the development of a cleavage. The dip-direction of the cleavage is throughout the area approximately north-northeast. In many Carboniferous rocks, in particular the slates, the bedding is fully overprinted by the cleavage and is barely or not at all recognizable.

Alpine orogeny
The Alpine orogeny has influenced all rocks up to Miocene age. The Alpine orogeny has broken the Palaeozoic basement into several blocks. These block faults also affect the Mesozoic-Cenozoic cover. Regarding the Mesozoic-Cenozoic cover, two tectonic areas can be distinguished. A large, roughly south-dipping thrust separates a northern from a southern area. In the northern area the structure of the cover is mainly the result of tectonic movements in the underlying basement, although also minor folds and thrusts, originating above a Triassic detachment-plane, were formed independently from the basement. In the southern area the Mesozoic cover is deformed independently from the basement. It is characterized by folding and thrusting above a Triassic detachment-plane. This resulted in large scale (> 100 m to kilometre scale) open to gentle folds. Thrusting took place in a north-northwest direction.
Towards the northwest the elevation of the series of tectonic blocks decreases, creating depressions that were rapidly occupied by the sea in the Tertiary. Towards the southeast the marine episodes are progressively shorter and of decreasing age, although not younger than Eocene. Further extensive deltaic zones exist, of decreasing age towards the south. The pre-orogenic Mesozoic and Tertiary emerged when they were folded, resulting in syn-orogenic sedimentation of conglomerates on a progressively developing unconformity. These Tertiary deposits are characterized by a fluvial regime (delta deposits) with marine influx (beach deposits).
During a phase of decompression northwest-southeast striking normal faults were produced. Tertiary post-orogenic sediments consisting of limestones interbedded with calcareous silty sand layers and conglomerates interbedded with calcareous silty sand layers were deposited in a lagoonal environment with some marine influx.

Quaternary
During the Quaternary all previous deposits were eroded, resulting in Quaternary slope deposits of variable thickness and extent. Along the Ebro river two terraces are well developed. Indications for a widespread glaciation during the Pleistocene have not been found. The periglacial climate will, however, have had an influence on the geomorphology and the forming of the present landscape of the area. Also associated with the periglacial climate are likely the silt deposits of eolian origin (loesses) that cover part of the area as a blanket with a thickness of up to 7 m. Recent are the colluvial deposits with a thickness of up to 6 m that are found on most of the natural slopes in the area.

A.3.3 Lithostratigraphic units and sub-units

Many geomechanical relations discussed are related to a particular formation, a lithostratigraphic unit or a lithostratigraphic sub-unit. Broad descriptions of the geology of the research area can be found in Table 1 (page 17). A sub-division into lithostratigraphic sub-units is based on bedding or cleavage spacing (Table A 17, appendix I, page 181).

B EXISTING ROCK MASS CHARACTERIZATION & CLASSIFICATION

B.1 INTRODUCTION

Rock masses have been described from the earliest geological maps onwards. The descriptions of the rocks were initially in lithological and in other geological terms. With increasing knowledge of geology, geological features and the influence of geology on engineering the amount of information to be included in a description for geotechnical purposes increased, leading to sets of rules for the description or characterization of a rock mass geotechnically. These are briefly reviewed in ch. B.2.1.

Parallel with this development, a movement took place in mining and engineering geology to combine the characterization of a rock mass with direct recommendations for tunnel support. This resulted in rock mass classification systems. Rock mass classification procedures were developed for underground excavations as an alternative to analytical analyses of a discontinuous rock mass. The systems were developed primarily empirically by establishing the parameters of importance, giving each parameter a numerical value and a weighting. This led, via empirical formulae, to a final rating for a rock mass. The final rating was related to the stability of the underground excavation used for the development of the classification system. In more elaborate systems the rating was also related to the support installed in the excavation. Any other underground excavation made in a rock mass with a similar final classification rating is assumed to have the same stability appraisal or to require the same support as the excavations used for the development of the classification system. The reason for the development of classification systems is that analytical stability calculations for tunnels in discontinuous rock masses are nearly impossible. In the time before computers became generally available, a deterministic, and even remotely realistic, analytical calculation was not really feasible. This brought some engineers onto the idea that empirical relations might be an alternative.

Various classification systems have been developed since 1946. A division is often made between so-called 'early' systems and 'recent' systems (Bieniawski, 1989). This division is also maintained in this description of existing characterization and classification systems. The main difference between the two groups of classification systems is the number of parameters used in the systems. The 'early' (ch. B.2.2) systems often depend on only one or two parameters and were developed for underground excavations. 'Recent' (ch. B.2.3) classification systems use more parameters. The 'recent' systems also have been primarily developed for underground use but in the last decades some extensions to surface applications (e.g. slope stability, foundations and rippability) have been published (ch. B.2.4). The New Austrian Tunnelling Method (NATM) (Pacher et al., 1974) is also discussed in ch. B.2.3.6. This system includes legal and contractual parameters not found in any of the other systems, and is strictly related to tunnelling. Therefore it cannot readily be compared to the other systems and is for this study not very significant but is included to present a comprehensive overview of the main classification systems presently in use. In ch. B.2.3.7 the Rock Engineering Systems (RES) methodology is briefly discussed. Although the RES methodology itself is not a classification system as the other systems discussed, applications of the RES to slope stability, such as presented in ch. B.2.4.9., resemble the application of a classification system.

In ch. B.3 correlations between the different existing systems are discussed as well as calculation methods, the parameters used in the existing classification systems, and the influence of these parameters on the final classification result. A summary of the findings in the literature concludes the literature review and serves as the basis for the development of a new classification system (ch. B.4).

B.2 EXISTING SYSTEMS

The review of existing systems covers the characterization and classification systems, which are the main and (in the opinion of the author) most interesting systems with good published documentation. Most of these systems have been used in different geological and geotechnical environments for different projects. In many civil engineering or mining projects systems have been developed or existing systems have been modified. Often these have been modified to the particular needs of a project and might not be applicable to other projects or other geological or geotechnical environments. Sometimes parameters or factors of different systems are combined (Japan, 1992).
This review only describes the main parameters and characteristics of the systems. All characterization and classification systems are accompanied by (extensive) tables for descriptions of parameters and, if appropriate, by tables with recommendations for civil or mining engineering applications. These tables have not been copied and the reader is referred for the details to the cited literature.

B.2.1 Descriptive and characterization systems

Two standard systems that characterize a rock mass and express rock mass characteristics in standard terms are those in BS 5930 (1981) and the ISRM Basic Geotechnical Description (ISRM, 1981b). A third, mainly used in the USA, is the Unified Rock Classification System (URCS) (Williamson, 1980, 1984). The systems do not result in a numerical value or direct design recommendation. The systems facilitate communication on rock mass characteristics and are widely used for various purposes.

Borehole core and exposure logging
The work by Deere et al. (1964, 1967) and Moye (1967), who published detailed instructions and recommendations for the description of rock masses and the presentation of rock mass data in the form of borehole core logs, has been adapted by the working party of the Geological Society Engineering Group in the report 'The logging of rock cores for engineering purposes' (Anon., 1970).

British Standard BS 5930
The present version BS 5930 (1981) gives recommendations for a standard description of a rock mass. The characteristics are described according to a series of standard terms and phrases and lead to an extensive rock mass name. The geological units of the research area for this study are described according to BS 5930 (Table 1, page 17). An interesting feature of the British Standard is the recognition of the importance of intact rock block size and form (Fig. 13). Rock blocks are described as very large blocky, very small columnar, etc.. Although not quantified, the descriptive terms relating to block form are very useful in engineering geology.

ISRM Basic Geotechnical Description
ISRM (1981b) recommends the following geotechnical rock mass parameters to be described or measured:
1) Rock lithology, with geological description
2) Discontinuity spacing (bedding or layer thickness and joint/fracture spacing)
3) Unconfined compressive strength (UCS)
4) The friction angle of the discontinuities

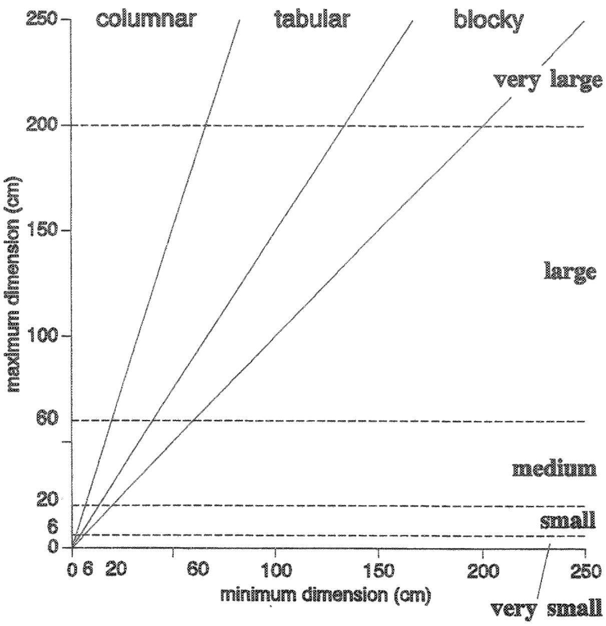
The far more extensive ISRM 'Suggested methods for rock and discontinuity characterization, testing and monitoring' (1978b, 1981a) recommends the quantitative description of a very extensive and complete set of rock mass parameters for the characterization of a rock mass.

Unified Rock mass Classification System (URCS)

The Unified Rock mass Classification System (Williamson, 1980, 1984) has been specially designed to facilitate the communication on rock mass parameters. The parameters described are: 1) degree of weathering, 2) strength, 3) discontinuities and 4) density. Hsein (1990) extended the system to give an overall 'performance index' for a rock mass.

Discussion

It is regrettable that the descriptive systems do not use the same descriptions for the same parameters. For example, Table 2 shows the description of strength of intact rock for three systems published within a period of two years. The systems use different intervals and terms to describe the strength of intact rock. Similar differences are found for discontinuity spacing, degree of weathering, etc.. The differences are often based on futile reasons that do not justify the differences. For example, in the ISRM system (1981b) interval boundaries are used which resemble the particle size[9] boundaries, for which the philosophy is that it is easily remembered. Other differences are caused by cultural background (for example: the use of psi interval boundaries in the URCS system, 1980).



note: the ratios separating columnar, tabular and blocky (continuous lines) are an interpretation from Price (1992); these are not quantified in BS 5930 (1981).

Fig. 13. Block size and form description according to British Standard (BS 5930, 1981) with ratios for block form (Price, 1992).

The British Standard, ISRM and URCS systems are presented as basic description systems and allow for additional information to be provided with the basic description. Standard guidelines for the additional information are, however, not given. The only system, whether classification or characterization, that includes the parameter of rock material density is the URCS. Probably this parameter is included because a main user of the system is the Soil Conservation Service of the U.S. Department of Agriculture. Apart from applications for construction materials the author is not aware of any application for which density is of major importance.

strength of intact rock						
BS 5930 (1981)		ISRM (1981b)		URCS (1980)		
interval MPa	class	interval MPa	class	interval psi	interval MPa	class
> 200	extremely strong	> 200	very high	> 15,000	> 103	rebounds (elastic)
100 - 200	very strong	60 - 200	high			
50 - 100	strong			8,000 - 15,000	55 - 103	pits (tensional)
12.5 - 50	moderately strong	20 - 60	moderate	3,000 - 8,000	21 - 55	dents (compression)
5 - 12.5	moderately weak	6 - 20	low	1,000 - 3,000	7 - 21	craters (shears)
1.25 - 5	weak	< 6	very low	< 1,000	< 7	mouldable (friable)
< 1.25	very weak					

Table 2. Characterization of intact rock strength according to BS 5930 (1981), ISRM (1981b) and URCS (1980).

An interesting method of describing intact rock strength is included in the URCS system (Table 2). The determination of intact rock strength in the field is related to the deformation properties of intact rock, rather than to the unconfined compressive strength of intact rock as used in BS 5930 and ISRM. A similar intact rock strength description with testing method has been designed by Burnett (1975)[10] and was later used for the British Standard (BS 5930, 1981).

(9) Soil particle size intervals: 0.002, 0.06, 2, 60 mm, etc. (BS 5930, 1981).

(10) This method of establishing intact rock strength is included in the slope stability probability classification (SSPC) system (ch. C.3.2.1.2).

B.2.2 Early classification systems

Terzaghi - rock load classification system

K. Terzaghi (1946) classified rock masses with the objective of predicting the load on steel arch support sets in tunnelling. The parameters taken into account are the 'rock condition', the dimensions of the tunnel, the depth below the terrain surface and below the water table (Fig. 14). The rock volume supposed to be supported by the steel arch set is hatched in Fig. 14.
The assumption that the steel arch set has to support a certain volume of rock above the tunnel, implies that the rock is allowed to deform until it can exert a force on the support. Terzaghi modelled deformation zones (a crack or shear zone) starting at the toes of the steel arch set in upward direction to allow the volume of rock above the tunnel to rest on the set. The load on the set is assumed to be the weight of the rock volume in-between the deformation zones up to a certain height above the tunnel (H_p) and the water load (W) (Fig. 14).
The 'rock condition' parameter describes the rock mass in various classes such as 'hard and intact', 'hard stratified or schistose', etc.. Also classes for crushed and swelling rock are distinguished. A table is provided which, based on the 'rock condition', gives the 'rock load (H_p)' parameter as a factor of the width and height of the tunnel. The table also includes estimates of the variation in pressure on the support (e.g. the presence or absence of side-pressure on the steel arch sets)[11].

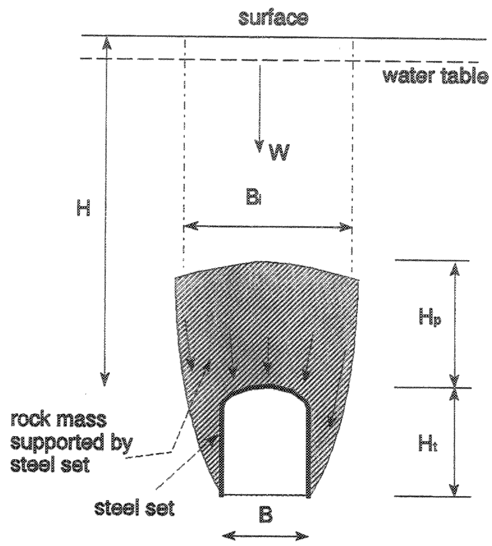


Fig. 14. Terzaghi - rock load classification (after K. Terzaghi, 1946).

Lauffer - stand-up time classification

Lauffer (1958) related the stand-up time of an un-supported span to standard rock mass types. Compared to the Terzaghi approach this was a major improvement as discontinuities (structural defects) were considered. The characterization of the rock mass was, however, not done by describing different rock mass parameters but had to be selected from a number of characterizations of standard type rock masses prescribed by Lauffer. Later the Lauffer system became the basis for the New Austrian Tunnelling Method (ch. B.2.3.6).

Deere - RQD index classification

Deere et al. (1967, 1988, 1989) introduced the Rock Quality Designation (RQD). The RQD index is measured on borehole cores, following eq. [1].

$$RQD = \frac{\sum \textit{length pieces of intact core with length} > 10\ cm}{\textit{total length drilled}} * 100\ \% \qquad [1]$$

The intact pieces of core (highly weathered pieces of rock or infill material should not be included) should be measured along the centre line of the core and the RQD values should be calculated separately for each lithostratigraphic unit. Core runs should preferably be not longer than 1 or 1.5 m. The RQD values provide a measure of the brokenness of the rock mass. Deere et al. (1967) related the RQD index to support types for tunnels. It is therefore the first classification system incorporating an index for the amount and quality of discontinuities in a rock mass. Recently 'rock quality charts (RQC)' have been based on RQD measurements by Şen et al. (1991, 1992).

(11) Severe doubt has been expressed about the concept of a deformation zone starting at the toe of the steel support and developing in upward direction. The development of deformation zones as indicated is only likely in a massive, not jointed (thus continuous), rock mass. In a discontinuous rock mass the deformations will follow existing discontinuities and may well lead to a totally different volume of rock to be supported. Secondly, the deformation zones will develop in upward direction only under low horizontal stress. With a higher horizontal stress the normal stress on the proposed deformation zones will be too high to allow shearing or tension cracking, thus preventing the development of deformation zones, whereas if the horizontal stresses are considerably larger than the vertical stresses the deformation zones may well develop horizontally rather than vertically. The assumption that the water load has to be supported by the steel set over the full height up to the water table is also unlikely as this would only be the case for a tunnel with impermeable lining capped by a fully permeable waterlogged rock mass.

Wickham - Rock Structure Rating (RSR)
Wickham et al. (1972, 1974) developed the Rock Structure Rating. The system is based on quantitative parameters for:

- parameter A - rock structure (origin, hardness, geological structure),
- parameter B - discontinuity pattern with respect to the direction of the tunnel (joint-spacing and -orientation relative to direction of tunnel drive),
- parameter C - groundwater inflow (based on overall rock mass quality described by parameters A and B, joint condition, amount of water inflow in tunnel),
- factor AF for type of excavation (drilling - blasting)

The final rating is:

$$\begin{gathered} RSR\ (rock\ structure\ rating) = A + B + C \\ RSA\ (adjusted\ RSR) = RSR * AF \end{gathered} \qquad [2]$$

The outcome of eq. [2] is used to design rib, bolt and shotcrete support for tunnels via the support recommendations of the Terzaghi system. The RSR (or RSA) system is the first system that resembles the recent systems, which are based on a number of rock mass parameters.

B.2.3 Recent classification systems

B.2.3.1 Bieniawski's RMR

The Rock Mass Rating (RMR) system from Bieniawski (1973, 1976, 1989) is one of the oldest of the often so-called 'recent' systems. The system has been developed in South Africa for underground mining. The system is based on a combination of six parameters (eq. [3]). Each parameter is expressed in a point rating and the final RMR ranges between 0 (very poor rock for tunnelling) to 100 (very good rock for tunnelling).

$$\begin{gathered} RMR = (IRS + RQD + spacing + condition + groundwater) + reduction\ factor \\ \\ RMR = Rock\ Mass\ Rating \\ IRS = Intact\ Rock\ Strength \quad RQD = Rock\ Quality\ Designation \\ spacing = discontinuity\ spacing\ of\ one\ set\ (see\ text) \\ condition = expression\ for\ condition\ (shear\ strength)\ of\ one\ set\ (see\ text) \\ groundwater = expression\ for\ groundwater\ inflow\ (pressure) \\ reduction\ factor = depending\ on\ orientation\ of\ engineering\ structure\ relative\ to \\ the\ main\ discontinuity\ set \end{gathered} \qquad [3]$$

In the latest modification published by Bieniawski (1989) the 'condition of the discontinuity' parameter has been extended and has been more specified (Table 4, page 35). Also, the RMR has been related to the span and stand-up time of the excavation.
The spacing and condition parameters are determined by the weakest discontinuity set or by the discontinuity set with the most adverse influence on stability (ch. B.3.4.5). Support of an underground excavation is determined by the RMR parameter and results in five different support classes.

B.2.3.2 Barton's Q-system

The Q-system of Barton et al. (1974, 1976a, 1988) expresses the quality of the rock mass in the so-called Q-value. The Q-value is determined with eq. [4]. The first term RQD (rock quality designation) (ch. B.3.4.2) divided by J_n (joint set number) is related to the size of the intact rock blocks in the rock mass. The second term J_r (joint roughness number) divided by J_a (joint alteration number) is related to the shear strength along the discontinuity planes and the third term J_w (joint water parameter) divided by SRF (stress reduction factor) is related to the stress environment for the discontinuities around the tunnel opening.

$$Q = \frac{RQD}{J_n} * \frac{J_r}{J_a} * \frac{J_w}{SRF}$$

Q = Rock Mass Quality
RQD = rock quality designation J_n = joint set number [4]
J_r = joint roughness number J_a = joint alteration number
J_w = joint water reduction factor
SRF = stress reduction factor (depending on intact rock strength and stress environment)

A multiplication of the three terms results in the 'Q' parameter, which can range between 0.00006 for an exceptionally poor rock mass to 2666 for an exceptionally good rock mass. The numerical values of the class boundaries for the different rock mass types are subdivisions of the Q range on a logarithmic scale (Fig. 16, page 33).

Intact rock strength influences the result only when the intact rock strength is relatively low compared to the stress environment. J_r and J_a are the parameters for the discontinuity roughness and alteration of the weakest discontinuities (Barton et al., 1974) or the discontinuity most likely to allow failure to initiate (Barton, 1976a) (ch. B.3.4.5). The Q-value determines the quality of the rock mass, but the support of an underground excavation is based not only on the Q-value but is also determined by the different terms in eq. [4]. This leads to a very extensive list of classes for support recommendations.

B.2.3.3 Laubscher's MRMR

Laubscher (1977, 1981, 1984, 1990) modified the RMR classification of Bieniawski. In his system the stability and support are determined with eq. [5]. The main parameters are the same as for the Bieniawski system but the parameter for groundwater is included in the condition parameter. The number of classes for the parameters and the detail of the description of the parameters is more extensive than in the RMR system.

$$RMR = IRS + RQD + spacing + condition$$

RMR = Rock Mass Rating   IRS = Intact Rock Strength
RQD = Rock Quality Designation
spacing = expression for the spacing of discontinuities
condition = condition of discontinuities (parameter also dependent on
groundwater presence or quantity of groundwater inflow in tunnel) [5]
(parameters for RQD and spacing can be replaced by the fracture frequency)

$$MRMR = RMR * adjustment\ factors$$

MRMR = Mining Rock Mass Rating
adjustment factors are compensation factors for: the method of excavation,
orientation of discontinuities and excavation, induced stresses and future weathering.

The resulting RMR parameter is multiplied by adjustment factors depending on future (susceptibility to) weathering, stress, orientation, method of excavation and the amount of free block faces that facilitate gravity fall, and then becomes the MRMR (Mining Rock Mass Rating). The values of RMR and MRMR determine the so-called 'reinforcement potential'. A rock mass with a high rock mass rating before the adjustment factors are applied has a particular reinforcement potential. A high RMR rated rock mass can be reinforced by for example rock bolts whatever the MRMR value might be after excavation. Contrariwise, rock bolts are not a suitable reinforcement for a rock mass with a low RMR (has a low potential for reinforcement) even if after excavation the MRMR is not much lower than the RMR.

Laubscher uses a graph for the spacing parameter. The parameter is dependent on a maximum of three discontinuity sets that determine the size and the form of the rock blocks. The condition parameter is determined by the discontinuity set with the most adverse influence on the stability (further discussed in ch. B.3.4.5). The Laubscher system specifies values for the discontinuity condition parameter depending on different situations with respect to water or water pressure and does not have a separate parameter for water in the RMR equation (eq. [5]).

The concept of adjustment factors for the rock mass before and after excavation is very attractive (Laubscher, 1990). This allows for compensation of local variations, which may be present at the location of the rock mass observed, but might not be present at the location of the proposed excavation or vice versa. Also this allows for quantification of the influence of excavation and excavation induced stresses, excavation methods and the influence of past and future weathering of the rock mass.

B.2.3.4 Franklin's Size Strength Classification

Franklin et al. (1970, 1974, 1975a, 1986) and Louis (1974) developed a classification system based on intact rock strength and the block size of intact rock blocks. The intact rock strength can be established by hammer and scratch tests or Point Load Strength (PLS) tests. Block size is defined as the diameter of a typical rock block and is determined either by observing an exposure or rock core from bore holes. The intact rock strength, the influence of rock block diameter and tunnel size have been related to tunnel stability and potential failure mechanisms.
In particular the determination of intact rock strength by hammer tests and the determination of the block size by observing an exposure are interesting in the context of the development of a slope stability classification system and are further discussed in ch. B.3.4.

B.2.3.5 Modified Hoek-Brown failure criterion for jointed rock masses

The Hoek-Brown failure criterion for rock masses has recently been adjusted and incorporates now a simplified rock mass classification system (Hoek et al., 1992). The failure criterion is formulated as follows:

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

σ_1' = *major principal effective stress at failure* **[6]**
σ_3' = *minor principal effective stress at failure*
σ_c = *intact rock strength*
m_b *and* a *are parameters describing the rock mass structure and surface condition*

The rock mass parameter σ_c (intact rock strength) is derived from a field estimate which resembles the system for estimation of field intact rock strength by Burnett (1975, ch. B.2.1), however, the classes, descriptions and class boundaries are different. The parameters m_b and a are derived from a matrix describing the 'structure' and the 'surface condition' of the rock mass. The 'structure' is related to the block size and the interlocking of rock blocks while the 'surface condition' is related to weathering, persistence and condition of discontinuities. The parameter for rock mass 'structure' is divided in four classes, ranging from 'blocky' (well interlocked, undisturbed rock mass, large to very large block size) to 'crushed' (poorly interlocked, highly broken rock mass, very small blocks). The parameter for 'surface condition' is divided in five classes, ranging from 'very good' (unweathered, discontinuous, very tight aperture, very rough surface, no filling) to 'very poor' (highly weathered, continuous, narrowly spaced discontinuities, polished/slickensided surfaces, soft infilling).

B.2.3.6 NATM - New Austrian Tunnelling Method

The New Austrian Tunnelling Method (NATM) (Müller, 1978, Kovári, 1993, Pacher et al., 1974, Rabcewicz et al., 1964, 1972) comprises characterization and classification but also includes rock mass modelling, deformation monitoring, legal contract aspects and the construction of a tunnel. Various modifications, adjusted to local circumstances, have been developed worldwide, noticeably in Japan (Japan, 1992). The system is solely designed for tunnelling and a total description of the system is beyond the scope of this study.

B.2.3.7 Hudson's RES - Rock Engineering Systems

The Rock Engineering Systems (RES) methodology developed by Hudson (1992), relates the interaction of parameters that have an influence on engineering in discontinuous rock masses. As well the influence of a parameter on the engineering structure as the influence of a parameter on other parameters is quantified and result in a rating for a parameter of the engineering structure. This last-named parameter can be, for instance, the stability or instability of a tunnel or slope. Parameters can be parameters describing properties of a rock mass, such as intact rock strength, discontinuity orientation, etc., but also parameters describing external influences on rock mass parameters or engineering structures, such as climate, geomorphological processes, etc.. The quantification of all the interactions results in a matrix with which the required parameter, for example, the

stability of a tunnel, is determined. Quantification of the interactions or influences between parameters and between parameters and engineering structure can have any form. These can be, for example, differential equations, binary operations (0 or 1, for example, for features that are either present or not present), classifications or numerical calculations. How these relations are established (e.g. by engineering judgement or actually proved by testing) is of no importance. The reliability and accuracy of the final result depend, however, on the reliability and accuracy of the relations (and obviously of the input data). The methodology resembles the working of a neural network as also pointed out by Hudson, however, the relations between in- and output parameters in a neural network are normally of a simpler form.
The methodology is not a classification system, but rather a methodology of thinking for engineering in or on discontinuous rock masses. Hudson gives no detailed applications nor relations between parameters, however, suggestions are given for implementation of the methodology in various forms of engineering in or on discontinuous rock masses.

B.2.4 Rock mass classification systems for surface engineering applications

Some rock mass classification systems developed for underground excavations have been used for surface engineering structures such as slopes directly (Bieniawski, 1976, 1989, Barton et al., 1974) or in a modified form (Haines et al., 1991, Robertson, 1988, Romana, 1985, 1991, Selby, 1980, 1982). The system developed by Shuk (1994) is specially designed for slope stability. Also systems have been designed specially for excavation, rippability, etc..

B.2.4.1 Barton's Q-system applied to slope stability

Barton et al. (1974) included in his system an estimate of the friction angle for the shear strength of discontinuities. This friction angle can be used in, for example, slope stability calculations.

B.2.4.2 Bieniawski's RMR applied to slope stability

Bieniawski (1976, 1989) included not only recommendations for underground excavations but also for foundations and slope stability. The author is not aware whether the system has actually been used for slope stability analyses in the form as presented by Bieniawski.

B.2.4.3 Vecchia - Terrain index for stability of hillsides and scarps

Vecchia (1978) designed a classification system to quantify the stability of a hillside or scarp, e.g. natural slopes, based on parameters for 'lithology' and 'attitude', and a 'friction' parameter which is depending on the 'lithology' and 'attitude' parameters. The 'lithology' parameter is determined by the presence of clay and shale in the rock mass and by characteristics of the rock mass such as loose, coherent or massive rock masses. This, combined with interbedded lithologies, results in a series of different standard classes for the lithology, e.g. from shale with a few coherent beds (rating 10 points) to massive rocks with few or no discontinuities (rating 90 points). The rock mass in the field is visually compared to the standard classes provided by Vecchia (1978), classified and rated. The 'attitude' parameter assigns a rating ranging from 0 (unfavourable) to 12 (favourable) to the orientation of discontinuities with respect to the orientation of slope or scarp. The 'friction' parameter is a rating for the friction along the main discontinuity (set) allowing sliding. The 'friction' parameter with a rating between 2 and 10, is assigned on the bases of the classes determined for the 'lithology' and 'attitude' parameters. The 'friction' parameter is thus not a separate parameter established in the field. A terrain index (I_T) is calculated as follows:

$$I_T = \textit{terrain index} = \textit{lithology} + \textit{attitude} - \textit{friction} \qquad [7]$$

The simplicity of the system and the limited number of parameters, effectively only two, which have to be assessed in the field, are very attractive. This simplicity, however, may also be its largest drawback. The quantity of standard lithologies given is limited, will not always fit a rock mass in the field and the visual comparison may be ambiguous. The definition of standard lithologies resembles the approach of standard rock mass classes as used by Lauffer (1958, ch. B.2.2) for underground excavations.

Other drawbacks are that there are no provisions for more than one discontinuity set and the limited options for the friction along the discontinuities. An interesting observation (Vecchia, 1978) is made that water in surface hillsides or scarps is generally limited to surface water. Water pressures in the rock mass are therefore not considered.

B.2.4.4 Selby - Geomorphic rock mass strength classification

Selby (1980, 1982) designed the Geomorphic Rock Mass Strength classification. The classification is designed with emphasis on geomorphology rather than engineering. The system resembles the Bieniawski system (ch. B.2.3.1) and includes for a large part the same parameters. Parameters assessed and rated are: intact rock strength (which can also be assessed by Schmidt hammer, ch. C.3.2.1.1), degree of weathering, spacing of joints, joint orientations, widths (aperture) of joints, continuity (persistence) of joints combined with joint infill, and outflow of water (ratings are given in Table 4, page 35). The ratings obtained for each parameter are added and the total rating is an expression for the rock mass strength. The rock mass strength is divided in five classes ranging from very strong to very weak. The total rating is not directly related to slope stability but is used in the qualification and quantification of geomorphologic processes.

B.2.4.5 Robertson' RMR (modified Bieniawski)

Robertson (1988) modified the Bieniawski (RMR) system for use in slope stability analyses. The main distinction with the original system is that for RMR $>$ 40 the stability of the slope is fully governed by the discontinuities whereas for an RMR $<$ 40 the slope stability can be assessed by a modified Bieniawski system. In Table 4 (page 35) the parameters are listed that are used for determining the slope stability for an RMR $<$ 40.

B.2.4.6 Romana's SMR (modified Bieniawski)

Romana (1985, 1991) extended the RMR classification system to slope stability problems expressed in the slope mass rating (SMR).

$$SMR = RMR - (F_1 * F_2 * F_3) + F_4$$

SMR = *Slope Mass Rating*
RMR = *Rock Mass Rating (same as Bieniawski's RMR)*
F_1 = *factor for parallelism of the strikes of discontinuities and slope face* **[8]**
F_2 = *factor for discontinuity dip angle*
F_3 = *factor for relation between slope face and discontinuity dip*
F_4 = *factor for method of excavation*

The parameters F_1, F_2 and F_3 are for one discontinuity only and therefore the SMR should be calculated for each discontinuity set and the lowest resulting SMR value gives an indication for the stability of the slope. The SMR value predicts the possibility of a 'soil-type' failure (normally for low values) and the amount of plane and wedge failures (normally for higher SMR values). The SMR value is also used to indicate the support measures to be taken for (partially) unstable slopes.

B.2.4.7 Haines (modified Laubscher)

The Laubscher (ch. B.2.3.3) system is used to forecast rock slope stability in open pits in South Africa (Haines et al., 1991). The adjustment ratings incorporated in the Laubscher system are reported to be of great benefit for slope stability estimation. The design chart to determine the slope dip related to slope height and factor of safety using the MRMR of the Laubscher classification is shown in Fig. 15. Haines et al. point out that the system is designed in a mining environment where safety requirements are generally lower than in civil engineering. However, they also incorporated slope dips for slopes with a factor of safety equal to 1.5. These might be suitable for civil engineering. The system has been designed empirically based on existing slopes in open pit mines and analytical calculations.

The intact rock strength value necessary in the Laubscher system can be replaced by an estimate with Schmidt hammer values for soil and 'softer' rocks and by the density of the material for 'harder' rocks (Haines et al., 1991). The orientation of the slope with respect to the discontinuity orientations is incorporated in an adjustment percentage.

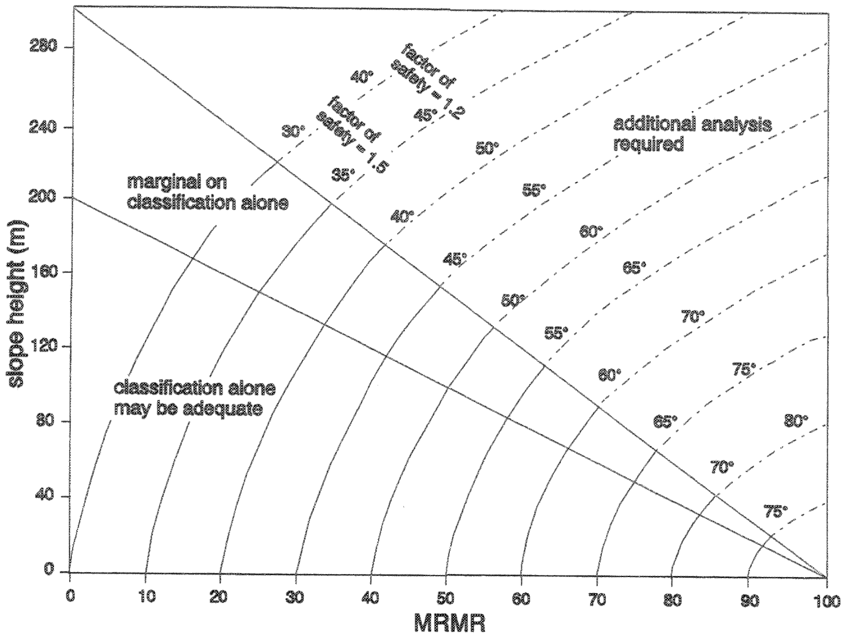


Fig. 15. Design chart to determine slope dip and height using MRMR classification data (after Haines et al., 1991).

B.2.4.8 Shuk - Natural slope methodology (NSM)

Designing the inclination of a new slope based on slope dips measured on existing natural and artificial slopes is often used in the design of new slopes to be excavated. Normally no formal characterization or classification of the rock mass is applied.

The Natural Slope Methodology (NSM) (Shuk, 1994a, 1994b, 1994c, 1994d) is based on this principle. This method uses a statistical analysis of existing natural slopes to predict rock mass and soil parameters, and the probability of slope stability. The method is based on a presumed relation (eq. [9]) between the height and length of a natural slope.

$$Height_{slope} = a * \left(Length_{slope}\right)^{b}$$

$$a = f\left[\left(\frac{c}{\gamma}\right)^{(1-b)}\right] \qquad b = s_a + p_a \qquad s_a = \tan\varphi_r + \frac{c}{\gamma * Height_{slope}} \qquad [9]$$

p_a = *non-dimensional pressurization parameter (related to tectonics, water pressures, etc.)*
φ_r, c = *residual friction angle, residual cohesion of rock mass or soil*
γ = *unit weight of rock mass or soil*
a *and* b = *weighting factors*

Equation [9] is only one of the possible relations. Other more complicated relations have not been investigated in depth by Shuk at present. Back analyses of a large number of natural slopes and optimization of eq. [9] result in estimates for different rock (mass) or soil parameters. The method can also be combined with anisotropic behaviour of rock masses and soils. The methodology is very attractive as it does not require extensive field investigations.

A problem with the methodology as reported, is that not all relations, parameters and especially the methods used to optimize the non-linear relations on the data are clear from the articles published. It is thus impossible to perceive the methodology, or comment on it in detail at present[12]. It is understood that the methodology has been still further developed and future versions and publications may show the full potential.

[12] Therefore this system has not been included in Table 4.

B.2.4.9 Hudson's RES - rock mass characterization applied to assess natural slope instability

Mazzoccola et al. (1996) presented an example for determining natural slope instability following the Rock Engineering Systems (RES) methodology (ch. B.2.3.7, Hudson, 1992). The rock mass characterization evaluates the interactions between and the influence of all parameters that may be of influence on slope stability. Twenty parameters are evaluated ranging from parameters as the geology, folding, etc. to parameters describing the rock mass such as weathering, the number of discontinuity sets, slope orientation, etc.. Also external influences are included such as climatological influences, as rainfall, freeze and thaw, etc.. The instability of the slopes is determined following the Rock Engineering Systems (RES) methodology.
The publication shows that a good correlation is obtained with a predictability rating for slope instability based on indicators of potential instability of the natural slopes (Nathanail et al., 1992).

B.2.4.10 Excavatability, rippability and blasting assessment

Various classifications have been developed to assess the excavatability and rippability of rock masses at terrain surface (Franklin et al. 1971, Weaver, 1975, Kirsten, 1982). Franklin et al. based the excavatability on strength (unconfined compressive or point load strength) and discontinuity spacing in accordance with the Franklin size - strength classification (B.2.3.4). Weaver based his rippability assessment on the Bieniawski classification for underground excavations (B.2.3.1) while the approach of Kirsten is based on the Barton classification (B.2.3.2). Most excavatability or rippability assessment systems are equipment specific, e.g. give recommendations for a particular type of excavation or ripping equipment. Some systems also include seismic velocities to assess rippability (Weaver, 1975).

B.3 CALCULATION METHODS AND PARAMETERS IN EXISTING CLASSIFICATION SYSTEMS

The following evaluation of the methods of calculation, correlation between existing classification systems and the evaluation of the various parameters in the existing classification systems is made to identify the parameters that should be included in a newly to develop classification system. Consideration is also given to establishing the relative importance of each of the parameters to be included and possibilities to establish a value for the parameters either in the field or by laboratory testing. This chapter (and the summary following in ch. B.4) provides the basis for the development of the new classification system for slope stability (SSPC) in section D.

B.3.1 Method of calculation

Addition, subtraction, multiplication and division of logarithmic, linear or non-linear parameters are used in the different existing classification systems. These are used either solely or in combination and no clear benefit from using a particular type of numeric representation or calculation method seems to exist. Some slope classification systems that use a method of calculation based on combining different parameters to give one single rating number, can give results difficult to perceive (for example: Robertson's RMR, ch. B.2.4.5, Romana's SMR, ch. B.2.4.6). In these classification systems parameters have an influence on the stability rating for a slope which instability may be caused by a physical mechanism that is independent from 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 and hence intact rock strength is of no importance for the stability or instability of the slope. In a newly to design classification system such illogical calculation methods should be avoided.

B.3.2 Correlations between different classification systems

Various relationships have been established between the different existing classification systems (Cording et al., 1972, Rudledge et al., 1978, Yufu, 1995). An important correlation is that between the systems of Bieniawski and Barton. The existence of a correlation of the numerical rating values was already established in 1976 (Bieniawski, 1976, 1989) and is shown in Fig. 16[13]. The two systems (Bieniawski and Barton) have been developed in different parts of the world, in different types of mines, in different rock types and, above all, these use partly different parameters and have defined differently the parameters included in both systems. That two so very different systems do correlate is strange but tentative reasons for this correlation might be:

1. Correlation between parameters; e.g. a rock mass with a low intact rock strength has often also a small discontinuity spacing or a low shear strength along discontinuities or both. A correlation between different classification systems is always obtained for the majority of possible rock masses.
2. Biased users: The parameter difference is compensated by adjusting parameter(s) to values which the experienced user considers to be appropriate for the rock mass. Thus, if the user knows from experience or by other means that the rock mass is poor, he unconsciously creates also a poor rock mass rating by

[13] It should be noted that the quality classes do not perfectly correlate (continuous lines in Fig. 16) and the scatter allows for one to two classes difference between the two systems (dashed lines). This may be due to the definition of the classes. A more correct comparison between the two systems should be based on the recommended support for underground excavations. The recommended types of support are, however, different for the two systems and a comparison cannot be easily made.

taking lower values for the individual parameters of the system he uses (see also ch. B.4). Because of this, systems should be designed to be operator-independent.

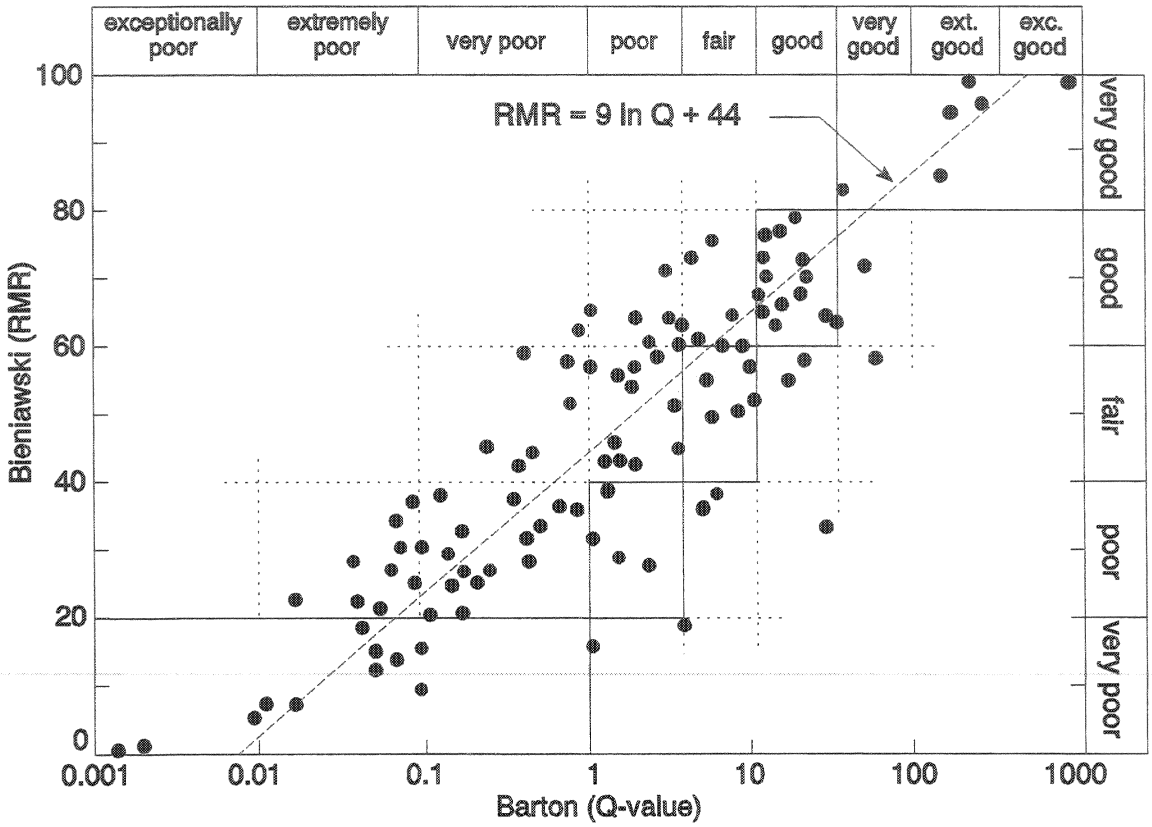


Fig. 16. Correlation between Bieniawski (RMR) and Barton (Q). Data from case histories with RMR and Q-system (after Bieniawski, 1989). (Continuous lines indicate correlating classes of rock mass quality.)

B.3.3 Influence of parameters in existing classification systems

An inventory of the most important rock mass parameters of interest for engineering structures in or on a rock mass is presented in Table 3. This table is based on the experience and intuition of the author and on the literature. The parameters listed are, in part, those occurring in some of the existing characterization and classification systems previously discussed (ch. B.2). Many systems do, however, not contain one or more of the parameters from Table 3 and also the influence of parameters in the existing classification systems is not for all classification systems the same. Table 4 presents the various parameters used in the existing rock mass classification systems and gives a crude indication of the maximum influence of each parameter on the final rating or recommendations for tunnel support or slope geometry. It is impossible for all systems to indicate the influence per parameter exactly because in some systems parameters are not independent or parameters are not linear. The percentages indicate the reduction of the final rating when that parameter is given its minimum value and all other parameters have their maximum value, compared to the rating based on the maximum value of all parameters. If a parameter is linked to another parameter then the other parameter is also changed as required(14).

Noteworthy differences in the influence of parameters (Table 4) are:

- The absence of the intact rock strength (except for a low intact rock strength/environment stress ratio), in the Barton system.
- The absence of discontinuity spacing in the Barton system.

(14) Take for example, the link between J_r and J_a in the Barton system; the lowest value for J_a is 20 but this cannot be combined with the maximum value (5) for J_r but only with J_r = 1.

- The strong reduction in influence of the water parameter in the Laubscher and Haines systems as compared to the systems of Bieniawski and Barton.
- The absence of a water/water pressure parameter in the Robertson modification for slopes of the Bieniawski system and in the slope stability system of Vecchia.
- The strong influence of the susceptibility to weathering in the Laubscher system.
- The strong increase in influence of orientation of discontinuities in relation to the orientation of the walls and roof of underground excavations in the Laubscher system compared to the Bieniawski system.
- The systems (except for Haines) for surface applications do not include the height of the slope whereas the height of the slope likely has an influence on the stability.

Since the systems are based on back calculation (regression analysis) of case histories that are mostly unpublished, an exact determination of the origin of the differences cannot be given. In this respect it should also be mentioned that empirical systems are never 'final'. In the last two decades the systems have continuously developed. Experience with the systems and subsequent changes in or fine-tuning of weighting factors and parameters cause some of the differences between the systems. It is also likely that the added experience with classification systems makes the latest systems the most reliable. In this respect the decrease of the influence of water in some of the newer systems and, in particular, in systems focused on slope stability should be noted[15].

<table>
<tr><td rowspan="12">Rock mass</td><td colspan="4">Intact rock strength</td></tr>
<tr><td rowspan="9">Discontinuities</td><td></td><td colspan="2">orientation (with respect to engineering structure)</td></tr>
<tr><td rowspan="3">rock block size and form</td><td colspan="2">amount of sets</td></tr>
<tr><td colspan="2">spacing per set</td></tr>
<tr><td colspan="2">persistence per set</td></tr>
<tr><td rowspan="5">shear strength along discontinuity (condition of discontinuity)</td><td rowspan="4">surface characteristics of discontinuity wall</td><td>material friction</td></tr>
<tr><td>roughness (dilatancy)</td></tr>
<tr><td>strength</td></tr>
<tr><td>deformation</td></tr>
<tr><td colspan="2">infill material</td></tr>
<tr><td colspan="4">Susceptibility to weathering</td></tr>
<tr><td colspan="4">Deformation parameters of intact rock/rock mass</td></tr>
<tr><td>Engineering structure</td><td colspan="4">Geometry of engineering structure (size and orientation of a tunnel, height and orientation of a slope, etc.)</td></tr>
<tr><td rowspan="2">External influences</td><td colspan="4">Water pressure/flow, snow and ice, stress relief, external stress, etc.</td></tr>
<tr><td colspan="4">Type of excavation</td></tr>
</table>

Table 3. Rock mass parameters of interest for engineering structures in or on rock.

(15) A reduced importance of water pressures in slope stability assessments is also found in this research (ch. D.1.7).

Table 4. Parameters and their influence in existing classification systems.

MAXIMUM NEGATIVE INFLUENCE OF PARAMETERS (in percentage from final maximum rating)(1)(2)																	
				Discontinuities									pressure or load		excavation		
classification system(2)	rating range	Intact Rock Strength	RQD	amount of sets	persist-ence	spacing	aperture	roughness (scale) large	roughness (scale) small	infill	altera-tion walls	Future weather-ing	rock	water	dimension	orienta-tion	method of excavation
EARLY SYSTEMS (for underground excavations)																	
Deere (RQD)	0 - 100		100														
Wickham (RSR)	19 - 120	24 (general area geology parameter)				35	7							7		11	17
RECENT SYSTEMS (for underground excavations)																	
Bieniawski (RMR)	0 - 100	15	20		6	20	6	6		6	6		15		100(4)	12	
		(reductions are not enough for a change of class)															
Barton(3) (Q)	0.00006 - 2666	with rock load parame-ter(3)	90 extr. good	97 very good				90 extr. good		99 good			97 very good	95 extr. good	100(4)		
Laubscher	0 - 120	17	13(5)	21(5)(6) good				5	9	15	5	70 poor	40 good	3(7) (no change)	100(4)	37 good	20 (no change)
		(no change of class)						(no change in class)									
SLOPE SYSTEMS																	
Selby	0 - 100	20			7	30	7			*(8)		10(9)		6		20	
Bieniawski (RMR)	0 - 100	15	20		6	20	6	6		6	6		15			60	
Vecchia	0 - 100	88														12	
Robertson (RMR)(10)	0 - 100	30	20		6	20	6	6		6	6					(100) (10)	
Romana (SMR)	0 - 115	13	17		5	17	5	5		6	6		13			52	13
Haines	0 - 100	17	13(5)	21(5)(6)				5	9	15	5	70	40	3(7)	(note 11)		20

Notes:

1 Influence percentages are only an approximate indication. Some systems are combinations of adding/subtracting, multiplier/divider, and/or logarithmic parameters, not independent and/or non-linear parameters (see text). Influence percentage = (maximum final rating - rating with the parameter minimum and all other parameters maximum) / maximum final rating x 100 %. For the recent classification systems also the class is indicated that results if the particular parameter has its minimum value. This allows comparison of classes between the logarithmic scale of the Q-system and the linear scales of the Bieniawski and Laubscher systems.

2 Terzaghi, Lauffer and NATM systems are not included as they do not use a rating for different parameters.

3 Intersections and portals are not considered. Intact rock strength is only of influence if low compared to stress environment.

4 Graphical (approximately logarithmic) relations between roof span or hydraulic radius, final rating and stand-up time.

5 Laubscher's system. Parameters for RQD and discontinuity spacing can be replaced by discontinuity frequency.

6 Amount of discontinuity sets, spacing and persistence combined in logarithmic relation (Fig. 33 & eq. [13]).

7 Water influence combined with discontinuity ratings.

8 Infill combined with persistence.

9 Selby rates present degree of weathering (thus not future weathering) for the whole rock mass following BS 5930 (1981).

10 Robertson: If RMR < 40 points slope stability governed by the RMR rating; if RMR > 40 points the stability is fully governed by the orientation and strength of the discontinuities.

11 Haines: Final result from graph relating slope height, dip, safety factor and (MRMR) rating. Adjustment parameter for slope orientation in relation with orientation of discontinuities with maximum of 100%.

B.3.4 Problems with parameters in existing rock mass classification systems

In the previous chapter it is shown that not all systems use the same parameters, that not all systems include all parameters thought to be important for geotechnical purposes and that the influence of a parameter on the final classification result is not the same for all systems. Apart from these differences the implementation of some parameters can also be questioned. A further discussion of the parameters thought to be important for a classification system for geotechnical engineering is therefore necessary.

B.3.4.1 Intact rock strength

Intact rock strength is, in most classification systems, defined as the strength of the rock material between the discontinuities. Strength values used are often from laboratory unconfined compressive strength (UCS) tests. Problems caused by the definition of intact rock strength and using strength values based on UCS laboratory tests are:

1 The UCS includes discontinuity strength for rock masses with a small discontinuity spacing. The UCS test sample is most often about 10 cm long and if the discontinuity spacing is less than 10 cm the core may include discontinuities[16].

2 Samples tested in the laboratory tend to be of better quality than the average rock because poor rock is often disregarded when drill cores or samples break (Laubscher, 1990), and cannot be tested.

3 The intact rock strength measured depends on the sample orientation if the intact rock exhibits anisotropy.

4 UCS is not a valid parameter because, in reality, most rock will be stressed under circumstances resembling conditions of triaxial tests rather than UCS test conditions.

Some classification systems (Franklin et al., ch. B.2.3.4) use the Point Load Test solely or as alternative for UCS or hammer tests as the intact rock strength index test. The same problems applying to using the UCS test also apply to the PLS test. The inclusion of discontinuities in the rock will cause a PLS value tested parallel to this discontinuity to be considerably lower than if tested perpendicular. This effect is stronger for the PLS test than for a UCS test, as the PLS test is basically a splitting test.

The size-strength system of Franklin et al. (ch. B.2.3.4), the Unified Rock mass Classification System (URCS, ch. B.2.1), the slope stability system of Haines et al. (ch. B.2.4.7), the geomorphic rock mass strength classification of Selby (ch. B.2.4.4), and the modified Hoek-Brown failure criterion (Hoek et al., 1992, ch. B.2.3.5) allow for an estimate or 'engineering guess' of intact rock strength using 'simple means' (geological hammer, Schmidt hammer, scratching, breaking by hand, etc.). Although Laubscher (ch. B.2.3.3) also recognises the problems inherent to testing of intact rock strength he actually does not explicitly allow for an 'engineering guess' with 'simple means'.
The disadvantage of using a Schmidt hammer for estimation of intact rock strength is the influence of discontinuities behind the tested surface. Schmidt hammer values may be influenced by a large and un-quantifiable loss of rebound if a discontinuity is present inside the rock behind the tested surface (ch. C.3.3.3).

B.3.4.2 Rock Quality Designation (RQD)

Rock quality designation (RQD)[17] is defined as eq. [10] (Deere et al., 1967).

$$RQD = \frac{\sum \textit{length pieces of intact core with length} > 10\ cm}{\textit{total length drilled}} * 100\ \% \qquad [10]$$

The RQD is measured on the borehole core. Normally the RQD is determined for every metre length of borehole core per lithostratigraphic unit. The length of unbroken pieces of sound core that are of more than 10 cm (4

(16) With discontinuities are denoted mechanical discontinuities, see glossary, page 241.

(17) RQD is used as an indicator for rock mass quality directly (ch. B.2.2), but also it is a parameter that is included in many classification systems together with other rock mass parameters. The discussion in this chapter considers the RQD only as a parameter in a rock mass classification system and not as an indicator for rock mass quality itself.

inches) length along the centre line of the core (ISRM, 1978b, 1981a), are added and the ratio, as percentage, to the length drilled is the RQD. Recommended is a drilled length of 1 or 1.5 m. In principle the RQD is a very simple test and used worldwide. However, the definition of the RQD and the day-to-day practice of determining the RQD introduces several severe disadvantages that cause the RQD often to be inaccurate or to result in totally misleading values. Many authors have commented on the disadvantages of RQD measurements (R.D. Terzaghi, 1965). Some major problems with RQD measurements are:

1 The value of 10 cm (4 inches) unbroken rock is arbitrary.

2 The value of 10 cm for unbroken pieces of rock core is an abrupt boundary. A rock mass with a discontinuity spacing of 9 cm perpendicular to the borehole axis will result in an RQD value of 0 % while a discontinuity spacing of 11 cm will result in an RQD of 100 %. Although a (small) quality difference might result from the difference in spacings, this is certainly not such a large difference that it should result in a difference between minimum and maximum of the quality assignment. Obviously in a real rock mass the spacings between discontinuities are not all the same and therefore the 10 cm boundary effect is more or less abrupt depending on the distribution of the spacings.

3 The RQD is biased through orientation with respect to discontinuity orientation (Fig. 17 - compare vertical borehole to horizontal borehole A). If a discontinuity is in the borehole core parallel to the borehole (borehole B) then ISRM (1978b, 1981a) recommends measuring the length of the core offset from the centre line if sound pieces of > 10 cm length are present in that stretch of the core. Depending on the infill thickness of the discontinuity, this might solve the problem of borehole B (RQD = 0 %) in Fig. 17.

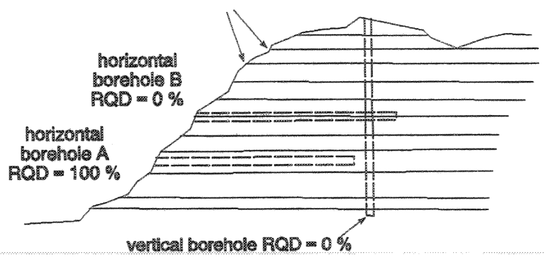


Fig. 17. Bias of RQD due to orientation of borehole.

4 Weak rock pieces (weathered pieces of rock or infill material) that are not sound should not be considered for determining the RQD (Deere et al., 1967, 1988). To exclude infill material will usually not be too difficult; however, excluding pieces of weathered, not sound rock is fairly arbitrary.

5 The RQD value is influenced by drilling equipment, drilling operators and core handling. Especially RQD values of weak rocks can be considerably reduced due to inexperienced operators or poor drilling equipment.

6 The equipment and especially the core barrels used for geotechnical rock drilling are not standard. It is obvious that the number of breaks caused by the drilling process will be strongly dependent on whether single-, double- or triple-tube core barrels are used. ISRM recommends measuring RQD on cores drilled with a double-tube core barrel only. The borehole is, however, normally not only made to determine the RQD. Often triple-tube core barrels are used for weaker rock or fractured rock masses to obtain a decent core for test samples. The RQD measured on this core is overrated but the amount of overrating is not known. Alternatively two boreholes should be drilled; one for the RQD with a double-tube core barrel and one for the samples with a triple-tube core barrel. The author does not know of any site where this has been the case. On the contrary the author has noticed many sites were the RQD was determined and compared from borehole to borehole irrespective of the core barrels used.

7 The diameter of the borehole core is not standard in geotechnical drilling. A core diameter of not less than 70 mm (H size) is recommended for geotechnical drilling. In massive rocks, however, a reduction is allowed to 55 mm (N size) and in very weak or fractured rock the diameter should be increased between 100 and 150 mm (BS 5930, 1981). The author has noticed that in practice very often N or NQ sized boreholes (approximately 47 to 55 mm core diameter) are used independent of the quality of the rock. Bieniawski (1989) allows borehole diameters from BQ to PQ (36.5 to 85 mm) for RQD determination. A larger diameter will result in: 1) fewer breaks during drilling and core handling after drilling, 2) a larger chance that a parallel discontinuity is intersected and 3) a larger chance that pieces of sound rock will be present in the core if a (near-) parallel discontinuity is intersected. In general, smaller core diameters lead to lower values for the RQD and larger diameters to higher values for the RQD.

8 Pieces of rock that are clearly broken through drilling or transport are supposed to be fitted together and the length should be measured as unbroken (ISRM, 1978b, 1981a). If this is done properly it partly solves the problems mentioned in points 5, 6 and 7, however it is not always easy to distinguish between natural discontinuities and breaks from drilling or core handling. In particular in a fresh rock mass this distinction

is often almost impossible and a less experienced engineer or drilling master might make considerable errors.

9 Although the RQD should be established per lithology, many establish the RQD irrespective of the lithology. Partly because of inexperience, partly because lithological boundaries are often uncertain. This problem is emphasized if core loss occurs in interbedded lithologies where the weaker lithology is not present in the borehole core.

The above leads to the conclusion that the RQD is not very strictly defined, that the definition is not very logical, that the result may not express the rock mass quality and that comparison of RQD values might be deceptive. Thus the incorporation of the RQD in rock mass classification systems can be questioned.

In many classification systems the RQD is incorporated as a parameter while the classification system also contains a parameter for discontinuity spacing. This seems not very logical. It effectively doubles the influence of the spacing of discontinuities on the final rating.

RQD values determined without a borehole

Various methods have been proposed to determine the RQD value for situations where no borehole core is available. Palmstrøm (Barton, 1976a, Bieniawski, 1989, Palmstrøm, 1975) recommends measuring all discontinuities along a scanline on an exposure and to calculate the RQD following eq. [11].

$$\begin{array}{lll} IF\ J_v \geq 4.5 & ===> & RQD = (115 - 3.3 * J_v)\ \% \\ IF\ J_v < 4.5 & ===> & RQD = 100\ \% \end{array}$$
$$J_v = \textit{total number of discontinuities per } m^3 \quad [11]$$
$$(= \textit{sum of number of discontinuities per metre length of all discontinuity sets})$$

A more sophisticated approach is a three-dimensional model to calculate the RQD from discontinuity spacing and orientation (Eissa et al., 1991, Şen et al., 1991). The methods are vulnerable to criticism because 1) the relations are only approximate, 2) an exposure might show more discontinuities than a borehole in the same rock mass (certainly when the exposure has been created by blasting), 3) weak rock pieces (highly weathered pieces of rock or infill material) that should be excluded in the determination of RQD cannot be excluded in these theoretical models and 4) influences of drilling and core handling are completely excluded, whereas the RQD measured in a borehole is always influenced by the drilling and core handling. A more fundamental error might be caused by the orientation of the measurement. A borehole is nearly always vertical and a scanline nearly always horizontal. As classification systems are empirical the orientation of the measurement might well have an influence although this is not quantified (or known) in the existing classification systems that use RQD.

B.3.4.3 Spacing of discontinuity sets

In many classification systems the spacing of discontinuities is used as a parameter. However, often the spacing of only one discontinuity set can be incorporated (except for Laubscher and Franklin and modifications, and the 'modified Hoek-Brown failure criterion'). This is no problem if only one discontinuity set is present in the rock mass or if one discontinuity set has a considerably smaller spacing than the other discontinuity sets. The mechanical behaviour of the rock mass with respect to discontinuity spacing is, in such rock masses, mainly governed by one discontinuity set. However, these classification systems do not describe what should be done if the mechanical behaviour of the rock mass is governed by more than one discontinuity set, for example, if more sets with a similar discontinuity spacing are present (see also B.3.4.5).

B.3.4.4 Persistence of discontinuities

Non-persistent discontinuity sets do not have the same influence on the stability of a rock mass as persistent discontinuities have (glossary, page 241, and ch. C.3.3.1). How to deal with persistence is described in detail in the Q-system (Barton et al., 1974, 1976a, 1988) and the geomorphic rock mass strength classification of Selby (1980, 1982). These systems combine persistence with the description of the shear friction parameters of the discontinuity. In the RMR and Laubscher systems and modifications discontinuities are only considered if: 1) the discontinuity is larger than visible; thus the discontinuity can be followed for a distance equal to or larger than, for example, the dimensions of a tunnel or exposure, or 2) the discontinuity abuts against another discontinuity. Discontinuities that do not comply with 1 or 2 are not considered as discontinuities in these classification systems.

B.3.4.5 Condition of discontinuities

The condition of the discontinuities (material friction, roughness, discontinuity wall strength and infill material) determines the shear and tensile strength characteristics of the discontinuities. It is a problematic parameter in all existing systems that use the condition of discontinuities. Most systems separate the condition of discontinuities in different parameters (for example: Barton, Bieniawski, Laubscher and modifications) that are independently rated in the classification system. The Laubscher system uses four parameters (large and small scale roughness, alteration of discontinuity walls and infill), to establish the quality of the discontinuity. The Barton system uses only two parameters (discontinuity roughness number and discontinuity alteration number), but the number of options for these parameters is so large that most discontinuity conditions can be described.

A major problem with the existing systems is that these use an expression for the condition of the discontinuities for one discontinuity set only. Obviously there is no problem if all discontinuity sets have the same characteristic condition but for a rock mass with discontinuity sets with different characteristics it is often difficult to decide which discontinuity set should be considered in the determination of the rock mass quality. Some authors (Bieniawski, 1989, Barton, 1976a, Laubscher, 1990 and modifications) indicate that: 1) the condition of the discontinuity set with the poorest condition should be included or 2) the condition of the discontinuity set that has the most adverse influence on the rock mass quality or engineering application should be included. Romana (1985, 1991) recommends that the rating should be calculated for each discontinuity set and the lowest resulting rating be used to determine the slope stability. In Bieniawski (RMR) and modifications and the slope classification by Romana the problem is more pronounced because also the spacing parameter is defined for one discontinuity set only. According to Bieniawski the discontinuity set with the most adverse influence on the stability should be taken into account. A discontinuity set with a large spacing but with a bad condition could, however, have a worse influence on stability than a discontinuity set with a small spacing but with a good condition. It is not clear how the worst discontinuity set should be selected in such a situation. The problem is illustrated in Fig. 18.

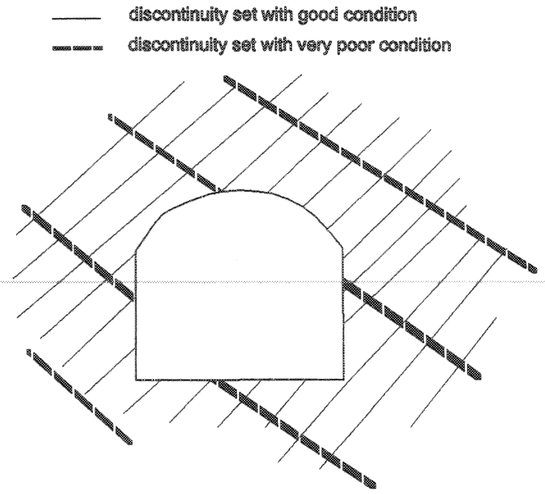


Fig. 18. Influence of discontinuity condition. It is not clear which discontinuity set has the worst influence on the stability of the tunnel.

B.3.4.6 Anisotropic discontinuity roughness

The roughness of a discontinuity can be anisotropic, e.g. ripple marks, striations, etc.. The shear strength resulting from anisotropic discontinuity roughness will also be anisotropic. Thus roughness should be assessed in relation with the orientation of the discontinuity and the roughness used in a classification system should be the roughness in the direction that is most important for the stability of a slope.
None of the existing classification systems incorporate anisotropic roughness. Robertson (1988) recommends assessing the roughness in the direction where possible sliding can occur. Systems that do not include the influence of discontinuity and slope orientation (ch. B.3.4.10) can obviously also not include anisotropic roughness.

B.3.4.7 Discontinuity karst features

Karst features have been found to be of importance in slope stability. The open holes considerably weaken the rock mass. Karst features are nearly always found to originate from solution along discontinuities. Solution leaves cavities supported by points of contact across opened discontinuities. The shear strength is reduced by a diminished contact area if (apparent) cohesion is present, and points of contact may break due to overstressing. The presence of karst holes during excavation has also an adverse effect on the slope stability. During blasting the blasting gasses will force their way out of the rock mass via the karstic discontinuities rather than by breaking intact rock or by following discontinuities in the direction of the next borehole. None of the existing systems incorporates a parameter that allows for an influence of karst features.

B.3.4.8 Susceptibility to weathering

Susceptibility to weathering is only considered, to a certain extent, in the classification system by Laubscher (1990) and in the modifications of this classification system. Susceptibility to weathering is an important factor in slope stability. Within the life span of a civil engineering structure future weathering of discontinuities and rock material may well lead to instability.

B.3.4.9 Deformation of intact rock and rock mass, stress relief

Deformation of intact rock is not considered in any of the existing systems, however, it is used for an indirect estimation of the intact rock strength by impact methods (ch. B.2.1). Deformation of intact rock is likely not important for engineering structures which cause low stresses on the rock, e.g. slopes of relatively small heights. Deformation of a rock mass is considered in the Q-system (e.g. Barton et al., 1974, 1976a, 1988, ch. B.2.3.2) in relation to stress relief due to weak or sheared zones in the rock mass. Deformation of a rock mass in relation to stress relief, not particularly related to weak or sheared zones, may, however, be of importance for slopes. Stress relief and related deformation may cause movements along discontinuities, increase of slope dips, etc., which influence the stability of a slope. A problem with deformation of a rock mass and with stress relief is that these cannot be tested, otherwise than with costly tests.

B.3.4.10 Relative orientation of slope and discontinuities

The orientation of discontinuities in relation with the orientation of the slope has a marked and often decisive effect on the stability of a slope (sliding, toppling failure, etc.) but not all classification systems used for slope stability assessment incorporate a parameter that allows for this influence (for example, Robertson, 1988 for an RMR of less than 40). In the other systems the parameter is fairly crude or not fully decisive or both. For example Bieniawski allows for a reduction of the final RMR rating by 60 % if the slope is unfavourably oriented, and Romana allows a reduction of 52 % (Table 4). In some systems (for example, Bieniawski and Romana) only the major discontinuity set or the discontinuity set with the most adverse influence on the slope stability has an influence on the final ratings, with respect to orientation of discontinuities and slope. This results in the same problem as outlined above for the condition of the discontinuity (ch. B.3.4.5).

B.3.4.11 Slope height

The height of the slope has a direct influence on the stress levels in the rock mass of the slope. High stress levels, comparatively to the intact rock strength, may cause failure of the slope due to intact rock failure (Gama, 1989). A high slope may also present more opportunities for discontinuity related failure as the quantity of discontinuities intersected by the slope is larger. Hence, although slope height is likely to be of importance in a slope stability system, none of the existing rock mass surface classification systems for slopes incorporates the slope height, except Haines (ch. B.2.4.7) and Shuk (ch. B.2.4.8).

B.3.4.12 Water

The presence, or the pressure of water in discontinuities, is a parameter incorporated in most systems. Water pressures and water flow in discontinuities may exercise pressures on rock blocks. The shear strength along discontinuities is unfavourably influenced because water pressure reduces the normal pressure on the discontinuity and therefore reduces the shear strength, while the presence of water gives a lubricating effect and may lower the

shear strength of the infill material and of the discontinuity wall (ch. A.2.3). Weathering of discontinuities through the passage of water can also strongly reduce the shear strength (ch. A.2.4).
The incorporation of a 'water' parameter in classification systems to allow for an influence of water pressure on the stability of an engineering structure is questionable for the following reasons:

1 Establishing the value for a parameter for the influence of water determined by the amount of water flowing out of the rock mass can cause some problems. Mostly they are defined by a certain quantity of water flowing out of the rock mass per time unit over a certain length of tunnel. Discontinuities will, in virtually all rock masses, be the major conduits for water discharge. In the classification systems the size or the form of the tunnel is, however, not considered in relation with the parameter for water, whereas it can easily be seen that the number of water discharging discontinuities and thus the quantity of water discharged is dependent on the form and size of the tunnel.

2 An important shortcoming in the existing water class determination is that the quantity of water is not necessarily related to the pressure of the water in the discontinuities. A small quantity of water discharged by a low permeability rock mass might be related to a higher water pressure in the discontinuities than a large quantity of water discharged by a (free draining) rock mass with high permeability.

3 The discharge of water is often not constant over the slope height. In the rock mass of the lower part of the slope the water pressure and consequently water discharge will be higher than in the higher part of a slope. Whether an average of the water discharged should be used in a single classification or whether this should lead to two or more different classifications applicable to different levels of the slope is not described in the existing slope stability classification systems.

4 In underground excavations the stress configuration around the opening will generally result in a higher compressive stress on discontinuities perpendicular to the wall of the opening and near to the underground opening than the compressive stress on discontinuities further away from the opening. Higher compressive stress causes a closing of the discontinuities in the direction of the underground opening. Water pressures are therefore present in the discontinuities adjacent to the opening. In slopes stress relief causes the discontinuities nearest to the slope face to open and the storage capacity increases in the direction of the slope face, resulting in a decrease of water pressures. The pressure decrease in the direction of a slope face can be large; in most slopes the discontinuities at the slope surface are free draining. This difference in water pressures between underground openings and slopes is likely to cause that water should be treated in a different way in slope than in underground excavation classification systems[(18)].

5 It has been shown that the water flow through discontinuities is often restricted to channels in the discontinuity (Abelin et al., 1990, Bear et al., 1993, Genske et al., 1995, Hakami, 1995, Neretnieks et al., 1982, 1985, Rasmussen et al., 1987). Probably this can be extended to water pressures. Water pressure acting on a plane only at the location of a channel would result in a total water pressure on the plane considerably smaller than if the water pressure would act over the full discontinuity plane[(19)].

6 Water run-off over the slope can lead to instability, but such run-off is not related to water seepage.

7 Water presence in slopes is not a continuous feature in time. During and shortly after rain high water pressures may build up in a slope or, alternatively, there may be no water at all after a dry period.

8 During rain it will be virtually impossible to distinguish between water discharged by discontinuities in the rock mass of the slope and surface run-off water over the slope.

9 Drains will normally be present in a wet tunnel, in which the quantity of water flowing in and out a section can be simply measured with, for example, a weir. The difference between the quantity of water flowing in and out of the section is the amount of water discharged by the rock mass surrounding the tunnel. Slopes, however, will usually not have a drain at the toe and measuring the quantity of water will be a practical problem.

10 In the existing classification systems for underground excavations the water parameter is normally expressed in classes such as: 'dry', 'moist', 'dripping', 'wet' or in classes that are directly related to an

(18) This applies to flowing - dynamic - water; the water pressures of static water are independent of the storage capacity. The slope face is, however, always free draining, except if a slope face is covered by an impermeable material, such as shotcrete, without draining facilities, and an underground opening mostly, and thus there is a flow of water in the direction of the slope face or underground opening.

(19) Water flow may be restricted to channels while the whole discontinuity is filled by static, not flowing, water, then the water pressure still acts over the whole surface of the discontinuity. In underground excavations has, however, been found that in some rock masses the majority of the discontinuities is not water bearing while the rock mass is water bearing (Neretnieks et al., 1985).

amount of water flowing out of the rock mass into the excavation. Classes such as 'dry' and 'moist' are not very difficult to establish but classes such as 'dripping' or 'wet' are subjective.

The above leads to the conclusion that the methodology used in the existing classification systems that incorporate the influence of water pressures on the mechanical behaviour of a rock mass, should be reconsidered.

B.3.4.13 Ice and snow influence

Ice and snow can have a severe influence on the stability of a slope. Freezing of water leads to an expansion in volume. Water frozen in a discontinuity will exert a very high pressure on the discontinuity walls. In underground applications this virtually will never be a problem as temperatures underground are normally not below zero. In surface applications and certainly in slope stability applications freezing of water in discontinuities can, however, be a major factor for the stability of a slope. Freezing of water may lead to opening and widening of discontinuities, displacements of rock blocks out of the slope face, but also to closure of discontinuities, blocking the discharge of seepage water that may lead to water pressure build-up in the slope. Snow may cause a problem for slope stability because of the additional weight of snow on the slope face. The influence of ice and snow is also dependent on the orientation of the slope with respect to the direction of the sun as daily temperature changes, especially a regular variation between freezing and thawing, has a negative influence on the quality of the rock mass. The problem of ice and snow influence is not addressed in any of the existing systems for slope stability.

B.3.4.14 Method of excavation

The way the exposure has been established has a considerable influence on the parameters measured or observed in the exposure. For example, an exposure in a river bed created by slow scouring of the river over probably hundreds to thousands of years creates an exposure with a relatively small amount of visible discontinuities. Stress concentrations have not occurred or were minimal during the creation of the exposure due to the slow process. The tendency for discontinuities to open is minimal and therefore a larger part of the discontinuities is not clearly visible. Contrariwise a blasted excavation shows considerably more discontinuities because partly intact rock has been cracked due to the blasting but also, and often more important, existing internal planes of incipient weakness, which before blasting were not visible, have opened or widened due to the pressure of the blasting gasses and the shock wave, and therefore become visible and thus will be measured as mechanical discontinuities.

Some existing classification systems take this effect into account (Haines, ch. B.2.4.7, Laubscher, ch. B.2.3.3, Romana, ch. B.2.4.6, Wickham, ch. B.2.2). These systems reduce the rock mass rating with a parameter to compensate for the damage that will be caused by the method of excavation.

B.3.4.15 Seismic velocity in a discontinuous rock mass

Some systems include seismic parameters, usually the velocity or apparent velocity of the wave, to assess the quality of the rock or rock mass (Japan, 1992, Weaver, 1975). For rippability, excavation and blasting assessment this is a fairly standard procedure, but assessments are often specific for types and brands of (excavation) equipment, for blasting procedures or for types and brands of explosives. In excavation or blasting assessment the interpretation is in general simpler than for other applications. The influence of intact rock strength and spacing and orientation of discontinuities (the main rock mass parameters defining excavatability) on seismic waves is comparatively straightforward. To relate seismic velocities to other rock mass or discontinuity parameters (for example, shear strength) is far more complicated. The behaviour of a seismic wave in a rock mass and the relationships between the rock mass parameters and the seismic parameters are not known in all details and consequently the interpretation is often ambiguous (Cervantes, 1995, Hack et al., 1982, 1990)[20].

[20] A research project has recently been started at ITC and TU Delft to further investigate relations between seismic waves and detailed rock mass classification in near surface rocks.

B.3.4.16 Operator experience and familiarity with a classification

Assigning values to some of the parameters in the systems discussed is often subjective and depends upon the operator's experience and the familiarity of the operator with the system. Examples for which this is of major importance are: 'the discontinuity set with the most adverse influence on the rock mass or for the engineering application' (B.3.4.5) and classes such as 'wet', 'dripping' for water influence (B.3.4.12). The merits of a system are clearly reduced if a system depends on the operator's experience or familiarity with the system.

B.4 SUMMARY

The review of existing characterization and classification systems leads to a series of conclusions and provides some directions for further improvement of parameters and calculation methods for slope stability assessment. These conclusions will be used to develop a new classification system for slopes (SSPC) which is the main topic of this research (section D). The conclusions derived from the review of existing classifications systems are:

Method of calculation and parameter type

1 Different systems with different parameters lead sometimes to approximately the same outcome for the description of the same rock mass, e.g. Bieniawski compared with Barton. These two systems have been used extensively by different users, so it is unlikely that the outcome of the systems is totally wrong, however, operator bias may be present.

2 In the literature only the final rock mass classification systems are described and not the underlying data analyses that resulted in the choice of weighting factors in the systems. In general, back analysis by linear regression has been used to fit the weighting factors for most systems.

3 Addition, subtraction, multiplication and division of logarithmic, linear and non-linear parameters are used. No clear advantage from one type of calculation or numeric representation of parameters above another seems to exist.

4 Methods of calculation which combine different parameters in one rating number may not express properly the slope stability because parameters will have an influence on the rating that may not be important for the stability of the slope.

5 The concept of a rock mass quality assessment before and after excavation should be considered as this concept seems logical and has been reported to be beneficial for slope stability assessment (Haines' slope stability assessment, ch. B.2.4.7).

6 Parameters with fixed class boundaries but also with gradational boundaries are used. No specific preference can be found in the literature. Intuitively a scale with gradational boundaries seems to be more appropriate for a real rock mass.

7 Most classification systems have changed during the years of application. This is logical for all systems are empirical. The number of case histories used determines the quality of the system. The use of any empirical relation is restricted to the geological and engineering conditions of the case histories on which the system was developed. Extensive new data may stimulate an update of the system. No system is 'final' for there will always be new case histories to either expand its range of use or to improve its quality.

Parameters

8 Parameters that need revision or should not be used at all in a new system are:
- Intact rock strength,
- Rock Quality Designation,
- Spacing of discontinuities,
- Persistence of discontinuities,
- Condition of discontinuities,
- Presence of water,
- Deformation of the rock mass in relation to stress relief.

9 Parameters that should be included are:
- Susceptibility to weathering,
- Method of excavation.

10 Parameters not used in existing systems but may be considered necessary are:
- Surface run-off of water over slopes,

- Ice and snow influence - freezing of water in discontinuities and weight of snow on a slope face,
- Karstic features.

11 No new terms or definitions should be introduced unless absolutely necessary because this might result in confusion.

Water

12 Water pressures in discontinuities will generally decrease in the direction of the slope face, due to stress relief and consequent opening of discontinuities. This is different from the situation around tunnels where, generally, water pressures in discontinuities are present directly behind the tunnel wall. Consequently the influence of water pressures in discontinuities on the final rating of a classification system for slope stability assessment should be smaller than on the final rating of a classification system for the stability assessment of underground excavations.

13 Water flow and water pressures may be restricted to channels in discontinuities only.

14 The tendency to reduce the influence of water, water flow or water pressure in some of the more recent classifications systems for slope stability may suggest that water has a less strong influence on slope stability than often assumed in the past.

15 The influence of water on infill material in discontinuities, the effect of lubrication of discontinuities and the influence of water on weathering of the rock mass is likely to be important.

Expressions for spacing and condition of a number of discontinuity sets in a rock mass

16 Parameters for spacing and condition of discontinuity sets should be revised so that multiple sets with different discontinuity spacings and conditions can be accounted for.

Parameter determination

17 Determination of parameters should be possible using the simplest means. Any form of (complex) testing should be avoided where possible. If any test is incorporated then the benefits of this test should be clear. Certainly it should be recognized that the need to do a field or laboratory test will reduce, for economic reasons, the amount of data available. Less data of probably better quality might not be preferable to more data of lower quality.

18 Characterization and classification should be operator independent. Different users of the system should come to the same result.

19 Classification systems should be accompanied by exact and detailed descriptions of how to obtain the parameters.

C PARAMETER DEFINITION AND INITIAL POINT RATING SYSTEM

C.1 INTRODUCTION

The review of existing classification systems (section B) shows that classification of a rock mass is generally accepted as a useful tool to estimate the influence of the mechanical behaviour of a rock mass on an engineering structure. However, the methodologies and parameters applied in the existing systems may not be appropriate or have to be adjusted to be fully effective in a classification system for slope stability assessment. In this section C parameters are defined such that these are suitable for slope stability assessment. These parameters, and more, were measured in the early stages of this research, which began in 1990. Slope stability was analysed by a point rating system which was modified and developed as the research progressed to give the 'initial point rating' system. It was eventually concluded that a point rating system is not a suitable approach to slope stability classification. Therefore in section D the approach is changed and the final result - a slope stability classification system based on probabilities; the SSPC system - is developed.

The outline of section C is as follows:
chapter C.2 - Slope geometry and standards for visual assessment of slope stability
The slope stability classification system developed is designed by describing and analysing existing slopes. The standards for measuring the geometry of the slopes and standards for the visual assessment of the stability of these slopes are defined and described in this chapter.

chapter C.3 - Parameters in rock slope stability
Parameters of importance in slope stability and possibilities to measure these in the field, are defined.

chapter C.4 - 'initial point rating' system
Based on the results of the parameter analyses an 'initial point rating' system was developed. This 'initial point rating' system and the results obtained with the initial system are briefly discussed.

C.1.1 Data quality and storage

Students and staff of ITC and the Technical University Delft characterized slopes according to standard procedures outlined in the following chapters and produced reports with photographs and descriptions of the slopes. The four years of data collection resulted in 286 characterizations of slopes in the Falset area. Obviously not all data were of high quality as students were in a learning process. This was, however, anticipated, for the involvement of a large number of different persons, not all experienced specialists in rock mechanics, was a preset requirement to avoid operator bias in the development of the system. Nonetheless some of the data received were incomplete, obviously erroneous or inconsequential and could not be used for the research. Because of this all described slopes have also been visited by the author and one or more staff members of ITC or the Technical University Delft. Incomplete data have been completed during these visits. Changing inconsequent or erroneous data incorporated, however, the risk of introducing operator bias from the author or from other staff members. Therefore it was decided that rather than changing the erroneous or inconsequent data these characterizations were altogether disregarded. This resulted in abandoning 36 characterizations[21], so that 250 acceptable characterizations resulted. Appendix I, Table A 17 shows the number of slope assessments per lithostratigraphic (sub-) unit.

Each characterization consists of a maximum of 35 parameters. For 250 characterizations this results in a maximum of 8750 data items. This quantity of data can obviously not be handled manually to develop a classification system. Therefore all data have been introduced into a database (DbaseIII Plus and IV). A programme in the programming language Clipper has been made for the necessary calculations (SSPCCLAS).

[21] From which 20 had been made by one group of students. The work of this group was abandoned altogether because the sites where they reported to have made the characterizations could not be precisely located. These were thus not abandoned because of the characterizations or the slope assessments itself.

C.2 SLOPE GEOMETRY AND STANDARDS FOR VISUAL ASSESSMENT AND CLASSIFICATION OF SLOPE STABILITY

The development of the classification system was based on existing slopes. The geometry and the stability of the existing slopes had therefore to be properly described and assessed.

C.2.1 Geometry of slopes

The orientation of a slope (dip and dip-direction) and the height of a slope assessed should be uniform and the stability assessments, whether visual or established by classification, should be made per individual geotechnical unit. However, not all slopes comply to these requirements and rules have been set up how to describe the geometry of a slope.

Laterally curved slopes
If a slope is curved laterally, the slope has to be subdivided in different sections where in each section the dip-direction is broadly uniform. The same applies if a slope dip or slope height changes along a slope laterally. The visually estimated stability (ch. C.2.2) and the stability assessment by classification are also established per section.

Slope height and dip
Slope height and dip can be difficult to establish, for the slope is almost never a straight plane. Most slopes tend to become less steep towards the top and often flatten out. In this research the height and dip of the slope have been measured from the toe to the point where curvature indicates a flattening of the slope (Fig. 19).
If, in vertical direction, a slope consists of different sections with different slope dips, the dip of each section is measured and the visually estimated slope stability (ch. C.2.2) is assessed in each section separately. A classification of the stability of the slope is done for each section individually. In each section the height is taken as the height from the bottom of the section to the top of the slope because the weight of the material above the section will have an influence on the stability of the section.

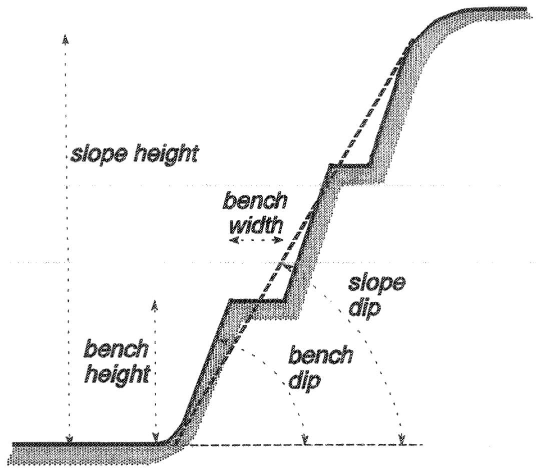


Fig. 19. Standards for the geometry of a slope.

Stepped or benched slopes
Steps and benches on slopes have been measured because the stability of a stepped or benched slope is determined either by the dip and height of the bench or by the dip and height of the total slope (Fig. 19). If the width of the step or bench is large compared to the height of the slope and the rock mass is not prone to large deformations, the influence of the rock mass weight above the bench will, in general, not have a large influence on the outer layers of the rock mass forming the slope below the bench and its stability is governed by the bench dip and height. However, if the width of the bench is small or if the rock mass is prone to large deformations, the stability is governed by the dip and height of the whole slope. Classification of slope stability is done for the sections in-between benches and for the whole slope and the lowest result is assumed to be valid for the whole slope.

Multiple geotechnical units in one slope

If a rock mass in a slope consists of a number of geotechnical units with approximately horizontal boundaries, the visually estimated stability is established per geotechnical unit. Slope stability classification is also done for each geotechnical unit independently. Slope dips can be different for each geotechnical unit and in each classification the slope dip is used that is characteristic for that geotechnical unit. The slope height used in the classification is the height from the bottom of the geotechnical unit assessed to the top of the slope. If the rock mass in a slope consists of multiple geotechnical units with vertical or inclined boundaries the visually estimated slope stability is established per geotechnical unit and also the classification is done per geotechnical unit. The height used in the calculations is again the height to the top of the slope. In some slopes a slope stability classification per geotechnical unit is not possible, for example, because the geotechnical units are folded. In such a slope, the slope stability classification is done as if the whole slope consists of the geotechnical unit that has the most adverse influence on slope stability. The visually estimated stability is established for the whole slope.

C.2.2 Visual estimation of slope stability

The research described was directed towards designing a slope stability classification system incorporating all possible mechanisms and modes of failure. To be able to reference such a newly designed slope stability classification system the stability of the slopes classified in the field has been assessed visually. The stability has been classified in five classes depending upon the absence, presence or impending presence of stability problems. These problems may be 'small' or 'large' depending on the size of the potential or actual rock falls. Table 5 gives the five stability classes and the number of slopes assessed in each stability class.

This visual estimation of slope stability is a subjective judgement. The division between 'large' and 'small' failures is particularly sensitive to the experience of the observer. In principle 'large' implies that the unstable rock mass is in the order of tonnes weight while 'small' implies that the unstable rock mass is in the order of kilograms weight.

Class		Description	Number of slopes
1	Stable	No signs of present or future slope failures	109
2	Small problems in near future	The slope shows all the signs of impending small failures but no failure has taken place	48
3	Large problems in near future	The slope shows all the signs of impending large failures but no failure has taken place	18
4	Small problems	The slope presently shows signs of active small failures and has the potential for future small failures	20
5	Large problems	The slope presently shows signs of active large failures and has the potential for future large failures	55
		Total:	250

Note: - The description large or small is independent of slope size.
- 'Near future' implies within the engineering lifetime of the slope.

Table 5. Standards for the visual estimation of slope stability and the number of slopes per stability class.

The problem of estimating the degree of stability for referencing a classification system is, however, a problem for all classification systems, whether for slopes or for tunnels. For most systems this estimation has been made by a group of observers. For the slope stability classification system described, estimates have been made over a period of four years using at least sixty observers from staff and students of ITC and Delft University of Technology working on 250 slopes. The large number of observers and observations must have significantly reduced the effects of individual observer bias.

The purpose of visually assessing slope stability was to compare this with the stability of the slope as assessed by one or another form of classification. However, it should be noted the classification measurement is for a uniform plane slope, while the real slopes (and in particular, those excavated by poor quality blasting) contain re-entrants, niches, overhangs, etc. which may allow slope movement in directions that could not be possible if the slope was one continuous plane. Rock falls resulting from such slope irregularity are not uncommon in the research area, where slopes whose geological structure and geotechnical characteristics give promise of stability, are unstable because of their irregularity of shape. In consequence, some visually assessed stabilities reflect poor construction rather than adverse geotechnical conditions.

C.3 PARAMETERS IN ROCK SLOPE STABILITY

C.3.1 Introduction

The results of the review of the existing classification systems in section B showed that parameters to be used for rock slope classification should be carefully reconsidered and defined to be most effective in slope stability classification. The following parameters are discussed in this chapter:
- parameters determining the mechanical behaviour of the rock mass material: intact rock strength and susceptibility to weathering (material properties, ch. C.3.2),
- shear strength along a discontinuity (ch. C.3.3),
- sets of discontinuities versus single discontinuities, concept of discontinuity spacing (ch. C.3.4),
- parameters that are specific to the rock mass at the location of an exposure or slope (exposure and slope specific parameters, ch. C.3.5) and
- parameters that have an influence on slope stability, but are not directly related to the rock mass or the slope (external influences, ch. C.3.6).

The results of the evaluation are summarized in ch. C.3.7.

C.3.2 Material properties

Material properties include the intact rock strength and the susceptibility to weathering of the rock mass.

C.3.2.1 Intact rock strength (*irs*)

In most existing classification systems for slope stability assessment intact rock strength is a parameter and is it necessary to obtain the characteristic or mean value of the intact rock strength of the geotechnical unit in which the slope is made or to be made. To assess whether and how intact rock strength should be a parameter in a rock slope stability classification system, the following should be considered:

1. Intact rock strength is not always included in existing underground or surface classification systems as a (main) parameter.
2. In existing underground excavation and slope stability classification systems (those which include intact rock strength) the contribution of intact rock strength to the final rating is considerably less than other parameters such as discontinuity spacing or condition of discontinuities.
3. Stresses in slopes will be nearly always considerably less than in underground excavation work so that it is unlikely that the influence of intact rock strength is as important in slope stability.
4. Failure in slopes is often associated with the shear strength of discontinuities[22][23].

[22] Some of the existing classification systems for slopes attribute slope failure fully to discontinuity failure if the rock mass rating is higher than a certain preset value, e.g. if the rock mass is of a certain quality. For example, the RMR modification by Robertson (1988, ch. B.2.4.5) assumes that slope failure is influenced by a number of parameters, including intact rock strength, for rock masses with a low rating (RMR < 40), but for a high rating (RMR > 40) the stability is dependent on discontinuity shear strength only.

5 An analysis of the influence of steps on discontinuity planes prohibiting sliding along a discontinuity plane (appendix II) shows that the intact rock strength will not be very critical for most slopes with dimensions as in the research area.

Summarized, this leads to the conclusions that the importance of intact rock strength in governing the stability of a slope diminishes with increasing intact rock strength and that a high accuracy in establishing intact rock strength is not necessary. A cut-off value for intact rock strength is used to incorporate the decrease of importance of intact rock strength. Above the cut-off value the contribution of the intact rock strength to the stability assessment of a slope remains constant. The limited importance of intact rock strength[24] does not require that sophisticated tests are done to establish the intact rock strength. Relatively easy to execute field tests with an impact method (ch. C.3.2.1.1) or with a 'simple means' field test (hammer, scratching, moulding, breaking by hand, etc., ch. C.3.2.1.2) lead to intact rock strength values adequate for slope stability assessment.

C.3.2.1.1 Impact methods

The Schmidt hammer determines the rebound of a piston activated by a spring. The rebound values measured on rock surfaces have been correlated to intact rock strength. Schmidt hammer values are, however, influenced by the material to a fairly large depth behind the surface. If a discontinuity lies within the influence sphere the Schmidt hammer values will be affected. The Schmidt hammer is thus not considered suitable to measure rock material strength in the field. The same applies to any other impact/rebound devices whose released energy per surface unit area is of the same order of magnitude as the Schmidt hammer of L or N design (ch. C.3.3.3). Equotip or other rebound impact devices (ch. C.3.3.3) might be suitable, but as these devices are only recently applied to rock mechanics it is not yet certain whether the relationships between rebound values and intact rock strength are correct.

C.3.2.1.2 'Simple means' intact rock strength field estimates

'Simple means' field tests that make use of hand pressure, geological hammer, etc. (Burnett, 1975), are used to determine intact rock strength classes in the British Standard (BS 5930, 1981) (the test classes are listed in Table 6). The 'simple means' field tests to estimate intact rock strength following Table 6 have been extensively used throughout the research. For all classifications multiple estimates of the intact rock strength, often more than ten, have been made per geotechnical unit and per exposure. The values obtained were averaged. Additional to these estimates also large amounts of unconfined compressive strength (UCS) tests[25] have been done in the same geotechnical units and in the same exposures to establish the reliability of the strength estimates. If possible, estimates and UCS tests were done both perpendicular and parallel to the bedding or cleavage[26].

(23) Sometimes a rock mass with a low intact rock strength (based on unconfined compressive strength - UCS tests) appears to have failed through intact rock failure, but, on closer examination, the low intact (UCS) strength is a consequence of a large number of (mechanical) discontinuities in the rock test specimen. Thus a shale may have a very low intact rock strength as determined by conventional UCS testing (ch. B.3.4.1), but this is not caused by the low strength of the intact material but by the numerous closely spaced bedding planes.

(24) For very high slopes, as in deep open pit mines, stresses can become so high that intact rock failure and shearing through asperities can occur also for high intact rock strengths. The intact rock strength may then be more important. The slope stability classification system developed in this research is, however, not designed for very high slopes.

(25) 14 UCS tests (one test from slope 92/5/3004 and all tests of student group 93/4) out of a total of 955 UCS tests were clearly outliers with values from 2 to 10 times higher than those measured by other groups in the same area and unit. These UCS tests have been excluded from the analysis.

(26) 'Simple means' field tests and UCS tests have also been used for the engineering geological mapping research (see preface), which data is included in the analyses of 'simple means' testing in this and following chapters.

The extensive quantity of tests allowed a thorough analysis of the accuracy and reliability of the 'simple means' field tests for estimating the intact rock strength. This analysis is presented in the following chapters. The estimated strength values in the graphs in this chapter are plotted as the mid values of the ranges of Table 6. If the strength was estimated to be on the boundary between two classes the boundary value is used.

intact rock strength	'simple means' test (standard geological hammer of about 1 kg)
< 1.25 MPa	Crumbles in hand
1.25 - 5 MPa	Thin slabs break easily in hand
5 - 12.5 MPa	Thin slabs break by heavy hand pressure
12.5 - 50 MPa	Lumps broken by light hammer blows
50 - 100 MPa	Lumps broken by heavy hammer blows
100 - 200 MPa	Lumps only chip by heavy hammer blows
> 200 MPa	Rocks ring on hammer blows. Sparks fly.

Table 6. Estimation of intact rock strength.

C.3.2.1.3 Intact rock strength field estimates versus UCS tests

In Fig. 20a the estimated values of intact rock strength by 'simple means' field tests are plotted versus UCS test values for all locations for which both were available, in Fig. 20b[27] the differences between the UCS test values and the estimated values as percentage of the estimated values are plotted, and in Fig. 20c the averages of estimated and UCS values per unit. In Fig. 20 no differentiation is made for the direction of the measurements. Fig. 20a shows that the scatter is wide and consequently only low or no correlation can be seen. In Fig. 20b is clearly visible that the differences between UCS and estimated values do not show a normal distribution for lower strength values. The distribution is skewed to higher values, e.g. the UCS values are higher than the estimated values. For high strength values the distribution of the differences is more normal but the average values of the UCS tests per estimated strength class are lower than the averages of the estimated values. A quite good correlation is found for the averages per unit (Fig. 20c). The standard deviation of the UCS values per unit is for most units considerably higher than the standard deviation for the estimated strength value per unit (Fig. 20d). If is assumed that a unit has a characteristic strength distribution with a characteristic mean strength value, which is very likely for the units assessed in the research area, then the estimated value will be nearer the mean value of the distribution because it is an average of more tests. The UCS test value is, however, only a single value or the average of few test values (normally less than three or four) and is likely to differ more from the mean value. This leads to the conclusion, as expected, that the characteristic mean strength value of a unit is better determined by a large quantity of estimated values than by few UCS tests. The skew of the distribution of the differences between UCS and estimated values for low strength (Fig. 20b) is probably caused by the fact that samples are not taken randomly. Samples are very seldom taken from the worst parts of a rock exposure. This is also confirmed by an analysis of the results of intact rock strength estimation and UCS tests for granodiorite with various degrees of rock mass weathering in the same exposure (description rock mass weathering: appendix V, Table A 20).

In Fig. 21 UCS values are considerably higher than the estimates of intact rock strength for the higher degrees of weathering of the rock mass. The granodiorite has weathered starting from the discontinuities and often a complete sequence of weathering is found. The weathered material and certainly the highly weathered parts, will break from the sample during transport and sawing of the sample. The UCS test is thus done on pieces of rock material less weathered than the average degree of weathering in the unit and therefore leads to a too high strength value.

The difference between UCS test values and estimated values for high intact rock strength might be due to a similar, but reversed effect. For high intact rock strength (> 100 MPa) it is often difficult to get sample blocks out of an exposure without equipment (saw, blasting, etc.) and a tendency exists to do tests on loose blocks that are more easily obtained. These may, however, have a lower strength. This effect is also observed in the granodiorite for which the estimated strength of the fresh exposures is higher than the UCS strength values

[27] The averages of UCS values are the averages of all UCS values belonging to the range of estimated strength. A grouping of the UCS values in the same classes as used for the estimate, before averaging leads to about the same values.

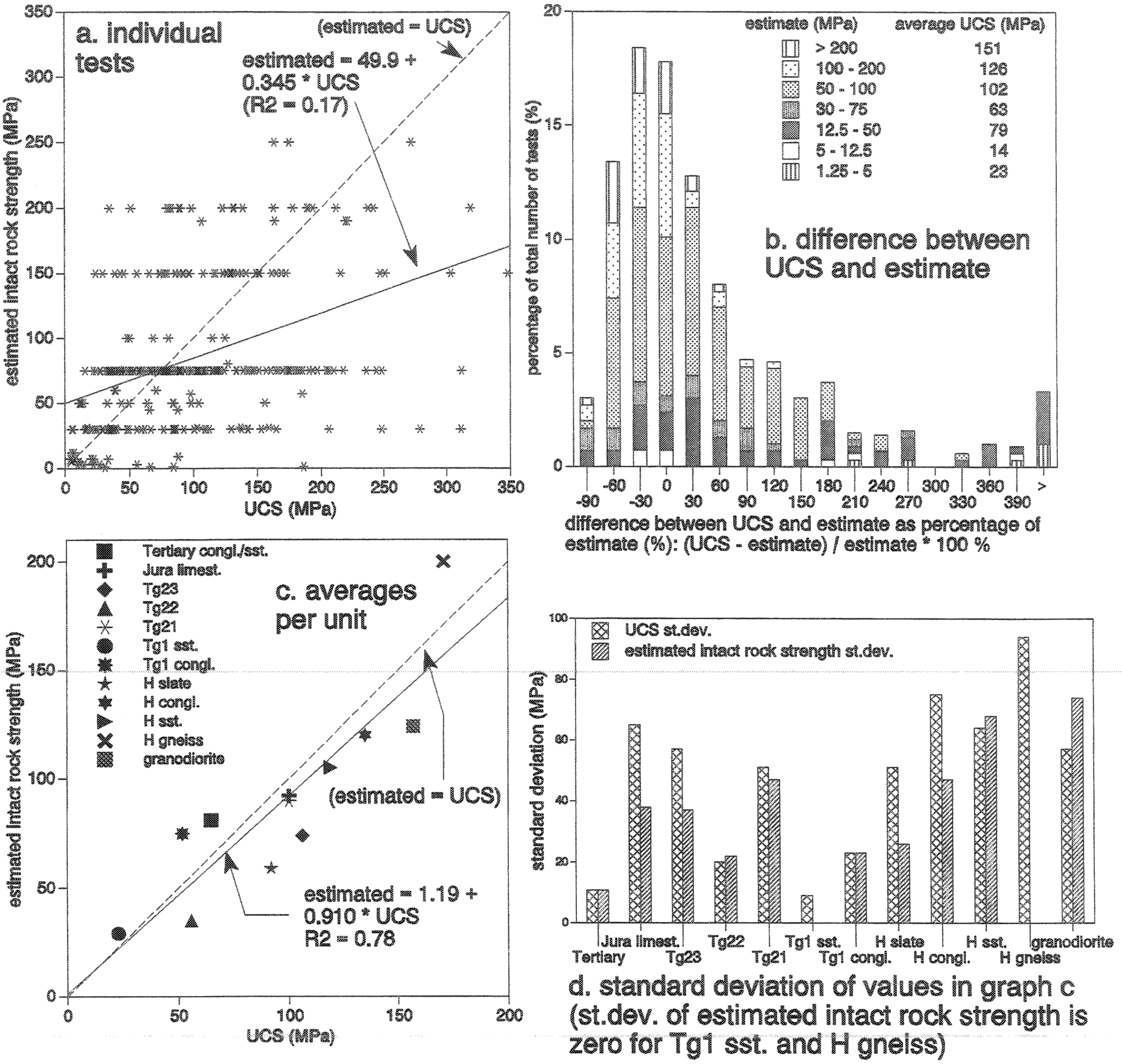


Fig. 20. Estimated intact rock strength vs strength values determined by UCS tests. (The dashed lines in A and C indicate the relation if estimated strength equals UCS strength.) (Number of UCS tests: 941)

(Fig. 21). The same effects, but for all rock units, are obvious in Fig. 22, which shows the percentages of UCS tests falling in the ranges for the estimate of intact rock strength different from the estimated range value. For lower intact rock strength values the UCS values are higher than the estimated values while for the higher intact rock strength values the UCS value is lower than the estimated value.

C.3.2.1.4 Repeatability of intact rock strength estimates

The repeatability of estimating the intact rock strength is fairly good. In the field intact rock strength has been estimated by different students and staff members in the same exposure and in the same geotechnical unit. The results show that the majority estimate the strength to be in the same class and a minority estimate the strength to be in a one class lower or higher. Strength estimates more than one class different from the class estimated by the majority were rare and could often be attributed to real variability in intact rock strength within a unit. An argument against estimating intact rock strength by classifying following Table 6, is that it would be dependent on the person who does the estimation, e.g. a large or physically strong person estimates the strength lower than a small or fragile person. This has not or only rarely been observed. The class ranges are obviously large enough

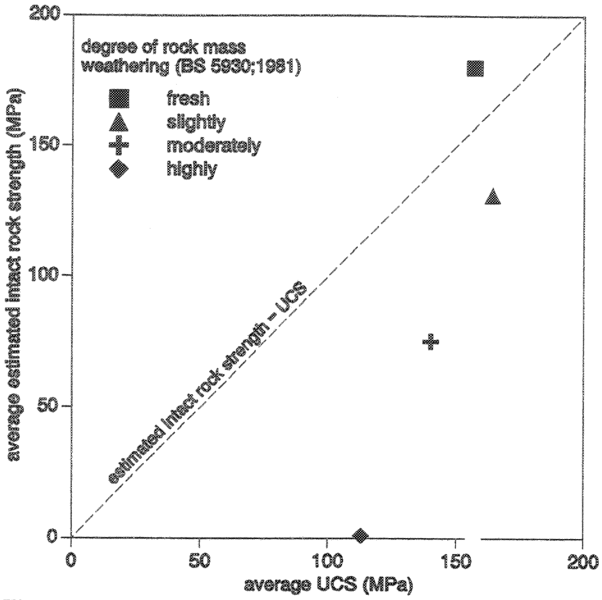


Fig. 21. Average estimated intact rock strength vs average UCS for granodiorite units with various degrees of rock mass weathering.

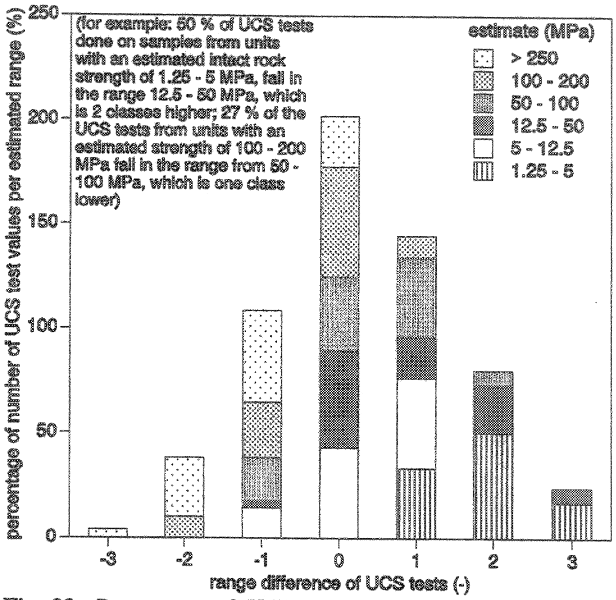


Fig. 22. Percentage of UCS test values falling in a range different from the estimated range value.

to accommodate for most physical strength differences. The possible error made by using estimation by 'simple means' of intact rock strength is discussed in more detail in ch. D.2.1 (Table 15, note 2, page 130).

C.3.2.1.5 Influence of degree of water saturation on intact rock strength

Some porous rocks exhibit a difference in intact rock strength depending on the degree of water saturation when tested by UCS tests (Bekendam et al., 1993). The permeability and porosity of the intact rocks in the research area is generally low (the porosity is generally less than a few percent) and the differences in UCS strength due to the degree of water saturation are therefore likely also very small and less than the scatter of the test results for most units. Only the Tg1 sandstone unit (Tg1 sst.) exhibits a larger porosity, is permeable, and could have shown a strength difference similar to that found in the literature. However, the quantity of tests done on this single unit does not allow for conclusive statements. Therefore it is not known whether a strength estimate is influenced in the same way by the degree of water saturation as the strength value obtained by a UCS test.

C.3.2.1.6 Strength anisotropy

The correlation of the estimated value of intact rock strength with the UCS tested in a particular direction could not be proven. Only in strongly anisotropic rocks (e.g. slate) the estimate is in agreement with the results from UCS tests. The highest strength is expected perpendicular to the cleavage direction. For the other rocks the estimation of intact rock strength results in higher values parallel to the bedding direction. In Fig. 23 are shown, per unit, the ratios of the strength perpendicular over the strength parallel for average UCS test values and for average field estimated values.

Although this effect has not been studied in detail a possible (and tentative) explanation could be as follows. All rocks included in Fig. 23 have intact rock strengths that are in 'intact rock strength estimate' classes established by hammer blows (≥ 12.5 MPa). The field estimate by hammer blows is a form of impact (dynamic) testing by which the rock breaks due to the impact energy (e.g. hammer blow). The impact energy is a limited quantity of energy induced into the rock in a small amount of time. Energy induced per time unit is thus high. The UCS test is a static test by which an unlimited amount of energy is induced into the rock until failure in a relatively large time span. The energy induced per time unit is low.
Deformation of rock is a time dependent phenomenon. It requires a certain amount of time before a stress is converted into a deformation and vice versa. Stress and deformation are linked and it requires time to transfer stress and deformation throughout a test specimen. In an impact test part of the energy dissipates due to crack

forming directly at the impact point. The remaining energy travels through the rock as a stress/deformation wave (e.g. shock or seismic wave). This wave is reflected at layer boundaries and at the end of the sample. When the incident and reflected waves are at the same location and have the same phase, the stresses (and deformations) are added and may cause the rock layer to break. In a layered sample the distance between layers is smaller than the length of the sample. The wave will loose energy (due to spherical dispersion, non-elastic deformation, absorption, etc.) during travelling through the rock. A wave reflected against the end of the sample with a longer travel distance, has thus less energy than a wave reflected against a layer boundary. The concentration of energy at a certain point due to the coincidence of direct and reflected waves will also be less.

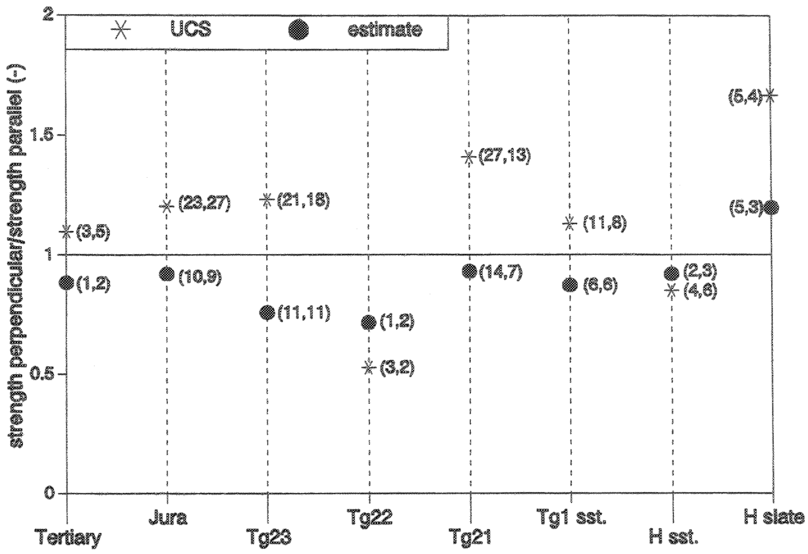


Fig. 23. Ratio of average intact rock strength perpendicular over average intact rock strength parallel for UCS and field intact rock strength estimate per unit (values in brackets are the numbers of UCS tests respectively estimates).

This may be the explanation why a rock sample when tested (by hammer blows) breaks more easily perpendicular than parallel to the layering and thus that the strength estimate for a sample tested perpendicular is lower than tested parallel. It is likely that this mechanism is less (or does not occur) in very thin spaced layered material (e.g. slate) because the rock at the impact point is easily fractured and broken whatever the orientation.

In a UCS test the induction of energy in the sample is so slow that a stress/deformation wave will not occur. The whole sample will be stressed and deformed. The tensile strength perpendicular to the layer boundary planes in a layered material is normally less than the tensile strength of the material. In a UCS test of layered material tested parallel to the layering, failure will occur due to bending and separation of the individual layers, resulting in breaking of layers (starting with the layers at the rim of the sample). Perpendicular to the layering failure occurs due to stress concentrations in the intact rock of individual layers. Bending of the layers and consequent cracking/failure requires mostly less stress/deformation than breaking the rock due to stress concentrations and thus is the measured strength perpendicular larger than parallel to the layering.

C.3.2.1.7 Conclusions

The estimate of the characteristic strength of intact rock in a geotechnical unit with a 'simple means' test, following Table 6, is equally good as executing a limited number of UCS tests. Therefore, intact rock strength (*irs*) in the classification system for slope stability ('initial point rating' system, ch. C.4, and SSPC, section D) has been taken as the intact rock strength established with a 'simple means' test, following Table 6. The higher accuracy that might be obtained by using UCS tests exists often only in theory. In practice the number of strength tests is so limited in comparison to the variations in strength in the rock mass that very many simple field tests will give a better estimate of the intact rock strength at various locations in the rock mass than a limited number of more complex tests.

A cut-off value is used above which the influence of intact rock strength on the estimated slope stability is constant. For the initial slope stability point rating classification system (ch. C.4) the cut-off value was set at 100 MPa. This was an engineering guess. In the SSPC system (section D) the cut-off value is optimized based on data from existing slopes and results in a value of 132 MPa.

C.3.2.2 Susceptibility to weathering

Weathering takes place in all rock masses whether underground or on surface. On surface, however, the influence of weathering is considerably more distinct. As it is in general not possible to determine which weathering process caused the weathering, all processes are included in weathering and are not individually treated in the classification systems. The different processes causing weathering are discussed in ch. A.2.4. The degree of rock mass weathering is classified following the British Standard (BS 5930, 1981, Table A 20, appendix V).

expected future degree of weathering	percentage adjustment for a rock mass weathered from fresh				
	after 1/2 year	after 1 year	after 2 years	after 3 years	after ≥ 4 years
fresh	100	100	100	100	100
slightly	88	90	92	94	96
moderately	82	84	86	88	90
highly	70	72	74	76	78
completely	54	56	58	60	62
residual soil	30	32	34	36	38

Note: The adjustment is applied to the rating for the stability of the underground excavation of Laubscher's rock mass classification to predict the future stability. The degrees of rock mass weathering follow BS 5930 (1981).

Table 7. Adjustment values for susceptibility to weathering for classification of stability of underground excavations in mining (after Laubscher, 1990).

For engineering applications in a rock mass it is very important to determine whether any significant weathering of the rock mass can be expected within the design life span of the engineering works[(28)]. There is no readily available test for determining the susceptibility to weathering for rock masses over long (up to 50 years) periods. This is obvious if one considers the amount of different materials, the amount of different processes, the variation in local circumstances, the necessary size of samples and the time span over which tests have to be executed[(29)]. Available tests describe the resistance of rock materials to erosive or climatological influence or to a particular use as construction material (Fookes et al., 1988, Selby, 1982). Mostly these tests are done on relatively small samples not representative of a rock mass. Susceptibility to weathering has been correlated with rock used as building material in existing engineering structures, e.g. buildings, gravestones, etc.. Tests for establishing susceptibility to weathering of discontinuities in a rock mass are not available for the same reasons. Generally, it is assumed that an increase in the degree of weathering causes a decrease of the shear strength along the discontinuities.
Attempts have been made to quantify the influence of susceptibility to weathering on the stability of underground excavations (Laubscher, 1990, Table 7). The percentages given in Table 7 are multiplied with the rock mass rating calculated following Laubscher, e.g. the rock mass rating is reduced by about 50 % if a rock mass is expected to weather from fresh to completely weathered within a half year. It should be noted that conditions in underground excavations are considerably different and, in general, with less variation than the conditions which influence weathering at surface.

Conclusions
In the 'initial point rating' stability classification system for slopes (ch. C.4) susceptibility is incorporated in a way similar to that by Laubscher (1990). Susceptibility to weathering is defined as the time necessary to weather a rock mass one degree down in the British Standard definition for rock mass weathering (BS 5930, 1981) (appendix V, Table A 20). A maximum time span of 50 years has been taken as this is about the maximum design lifetime for engineering works. The class denoted with '> 50 year' means that within the life span of the engineering structure

(28) The weighting factors used in the slope stability classification system, both the weighting factors in the 'initial point rating' system (ch. C.4) as in the SSPC system (section D), are optimized by referencing against existing slopes that have been subject to one or more of the mechanisms causing weathering. Therefore the weighting factors include the influence of existing weathering of the rock mass and a separate parameter for the degree of weathering is not necessary in the classification systems. The degree of weathering as a parameter for correction for the influence of past and future weathering is discussed under exposure and slope-specific parameters (ch. C.3.5.1).

(29) A possible means to establish susceptibility to weathering is to determine the slake durability (ISRM, 1981a). This test is, however, only a crude simulation of some of the processes involved in weathering.

no significant weathering is expected. In the SSPC system (section D) susceptibility to weathering is incorporated by establishing the expected degree of weathering at the end of the engineering lifetime of the slope (ch. D.1.6). The amount of time is established by comparing exposures with a known time of existence within the same geotechnical unit.

C.3.3 Shear strength along a discontinuity

The orientation of discontinuities in combination with the shear strength along discontinuities determines the possibility of movement along discontinuities. The influence of discontinuities on various engineering and mining structures and on slope stability is extensively described in the literature (Barton et al., 1990a, Goodman, 1989, Hoek et al., 1980, 1981, etc.). In the literature review (section B) is shown that virtually all rock mass classification systems do include parameters that describe the shear strength along discontinuities in a rock mass. A new-to-develop slope classification system should thus also include one or more parameters describing the shear strength 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. The discussion in this chapter covers only those aspects necessary to illustrate the problems involved in defining a relation for shear strength along discontinuities in a slope stability 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. The discussion of the different parameters leads to a preliminary description of discontinuity parameters determining the shear strength of a discontinuity for implementation in a classification system. This was used in the 'initial point rating' system (ch. C.4) and further developed and adjusted for the SSPC system (section D).

C.3.3.1 Persistence

Persistence[30] determines the possibilities of relative movement along a discontinuity. Discontinuities are usually differentiated in:[31] 1) persistent discontinuities; the discontinuity is a continuous plane in the geotechnical unit, 2) abutting discontinuities; the discontinuities abut against other discontinuities, or 3) non-persistent discontinuities; the discontinuities end in intact rock (ISRM, 1978b, 1981a). This definition does not consider differences in persistence in different directions. It is assumed that the discontinuity is persistent in any direction for the same length. This is not necessarily true. A discontinuity might be persistent in dip direction but not persistent perpendicular to the dip direction or vice versa (ISRM, 1978b, 1981a). The literature review showed that different classification systems treat persistence in different ways. Some systems (Barton et al., 1974, 1976a, 1988) treat persistence combined with roughness of the discontinuity walls while Selby (1980, 1982) combines persistence with the classification of infill material. In his classification Laubscher (1990) includes only those discontinuities which are larger than visible, thus those extending for a length larger than the exposure or tunnel, or those abutting against another discontinuity. Further quantitative descriptions of persistence are few and probably not fully satisfactory (Bandis, 1990).

The differences in the methodology to incorporate persistence in a classification system were the reason to try to define a new implementation of persistence in the new slope classification system. In the 'initial point rating' system (ch. C.4) the persistence is related to the height of the slope. A non-persistent discontinuity can only move along the discontinuity if the intact rock pieces are broken through. This is dependent on the level of the shear stresses along the discontinuity and hence related to the height of the slope[32].

(30) Persistence is treated as a discontinuity property in many of the existing classification systems and often also in the literature (e.g. Barton, 1974, 1976a, 1988, Selby, 1980, 1982, ch. B.3.4.4).

(31) See also glossary, page 241.

(32) The number of non-persistent discontinuities in the rock masses that were used for the design of the new classification system were, however, few and this methodology to incorporate persistence could not be tested. Therefore in the SSPC system (section D) this approach is abandoned and the persistence is incorporated in the characterization of the condition of a discontinuity.

C.3.3.2 Discontinuity roughness

The contribution of discontinuity roughness to the shear strength of a discontinuity can directly be measured with, for example, a shearbox test (ch. C.3.3.8), but only for relatively small surfaces. In theory the contribution of roughness to the shear strength of a large surface can be determined from other easily determined discontinuity parameters, such as the friction of the material (φ) and the measurement of roughness profiles (Patton, 1966)[33]. This is, however, too simple for natural irregular discontinuity surfaces. More complicated theories about roughness profiles, methods to characterize roughness profiles and relations between roughness profiles and shear strength can be found in the literature (Bandis et al., 1981, Barton et al., 1977, Fecker et al., 1971, Grima, 1994, Hsein et al., 1993, ISRM, 1981a, Rengers, 1970, 1971, etc.). However, many of these relations between roughness and shear strength are hampered by scale effects (Cunha, 1990, 1993) or do not consider all discontinuity properties that are important. In fact the determination of the contribution of roughness to the shear strength is so complicated that exact methods for large planes can probably not exist other than by full scale shear tests. Variation of roughness properties throughout a rock mass and the impossibility to establish the roughness properties for discontinuity surfaces that are not exposed, complicate the matter even further. Obtaining the properties in the required detail to make it worthwhile to apply a sophisticated methodology, is therefore mostly impossible or impractical. The conclusion is that a relatively simple method to describe the roughness that has a relation with the shear strength, based on as many as possible simple assessments of outcropping discontinuities, is the only feasible method in a classification system.

C.3.3.2.1 Roughness parameters important in slope stability

The importance of the roughness of a discontinuity partly depends upon the stress configuration on the discontinuity plane in relation with the strength and deformation characteristics of the discontinuity wall material and asperities. To clearly understand the mechanisms involved, the three following theoretical situations are distinguished. These situations apply to a discontinuity without infill (discontinuities with infill are discussed in ch. C.3.3.4).

1. Overriding of asperities - the rock blocks on both sides of the discontinuity are not confined[34] and no shearing through asperities occurs.
2. Deformation of asperities - the rock blocks on both sides of the discontinuity are confined[34] and no shearing through asperities occurs.
3. Shearing through asperities - the rock blocks on both sides of the discontinuity can be confined[34] or not be confined, but shearing through asperities occurs.

1) *Overriding of asperities*

For a plane sliding situation the normal stress (= the weight of the block under gravity) on the shear plane is constant in time (influences that can change the stress, such as snow, water, etc. are not considered for this theoretical situation). If is assumed that no asperities can be sheared off, because, for example, the strength is too high, the asperities have to be overridden for movement along the discontinuity to be possible. Then the most important roughness parameters are the friction of the discontinuity wall material (φ_{basic}) and the maximum roughness angle (i_{max}) from the datum reference plane (Fig. 24 left). The deformation characteristics of the rock material adjacent to the discontinuity and the geometry of the asperities at other locations along the shear plane are of no or minor importance. If $\varphi_{basic} + i_{max}$ is equal to or larger than 90° movement becomes impossible.

2) *Deformation of asperities*

If a discontinuity is confined and no shearing through asperities can occur, then the angle of the roughness is less important but the geometry (in particular the maximum height) of the asperities, the amount of asperities and the deformation characteristics will mainly determine the shear strength (Fig. 24 right, deformation is hatched).

[33] Formulated in the 'bi-linear shear criterion', see glossary, page 241.

[34] Confined denotes here that the rock blocks on both sides of the discontinuity are not free to move in the direction perpendicular to the discontinuity.

3) *Shearing through asperities*

If shearing through asperities can take place then all parameters are of importance, e.g. the strength of the asperity material, the geometry and the deformation characteristics (Fig. 25). Not only all parameters are of importance but also all variations of these parameters everywhere along the plane where contact between the walls will occur during displacement.

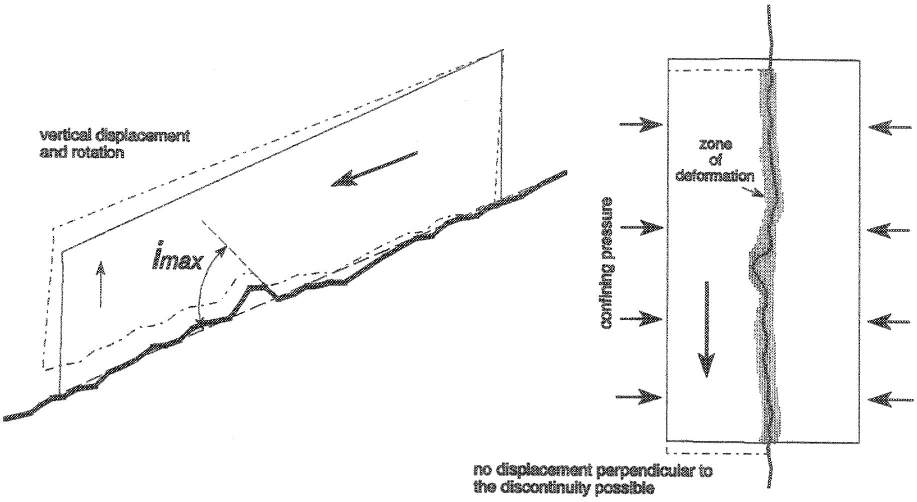


Fig. 24. Influence of roughness on displacement without shearing through asperities (left figure: unconfined; right figure: confined).

A complicating factor is that a piece of intact rock will often break under stress. Where and when a block of rock breaks is virtually impossible to establish by analytical calculations and highly complicated in a numerical analysis (Baardman, 1993).
Situation 3) is the common situation and nearly all shear displacement along discontinuities is governed by a combination of overriding of asperities, deformation of asperities and shearing through asperities. In slopes, however, the stresses perpendicular to the discontinuities are normally low which reduce the importance of shearing through asperities and the deformation of asperities.

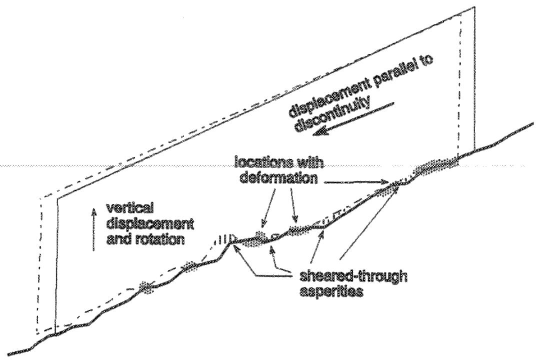


Fig. 25. Displacement of block (shearing through asperities and deformation).

C.3.3.2.2 Measuring roughness

Measuring a roughness profile on an exposed plane is theoretically simple. All that is necessary is to measure the height of the surface above and below a certain datum plane at regular intervals. There are, however, practical problems with regard to the datum plane, the measuring interval and the three-dimensionality of roughness.

Datum plane

Fig. 26 (left) shows a single block on a slope with the datum plane for this particular block. The datum plane is established by a least squares regression analysis of the profile. The roughness profile can be determined by sampling at a regular interval, measuring the distance below and above the datum plane. Fig. 26 (right) shows the same block but the block contains a (vertical) not-cemented mechanical discontinuity across which no tensile stresses can be transmitted. Thus block B can move while block C remains stable, then datum planes have to be established for both blocks and do not have the same orientation. In a discontinuous rock mass each independent block of rock material has therefore its own datum plane.

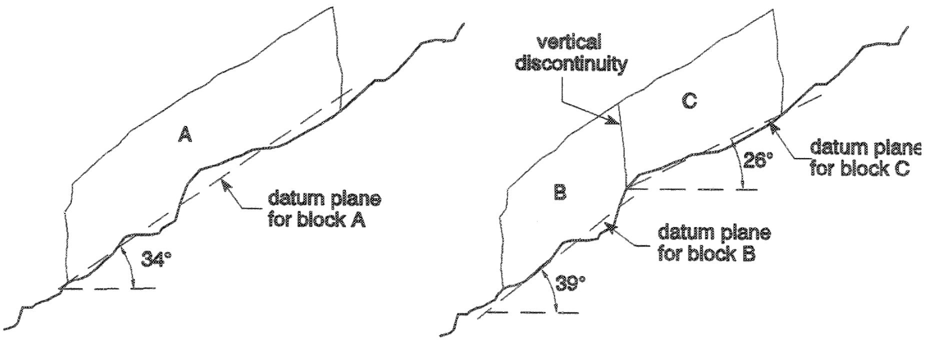


Fig. 26. Roughness datum plane for single block (left) and same block intersected by vertical discontinuity (right).

Measuring interval
The friction along the discontinuity plane is determined by the roughness of the discontinuity surfaces together with the friction properties of the material. 'Roughness' may range from the scale of atoms (e.g. irregularities in crystal structures) up to large scale roughness of the order of metres. A uniform measuring interval is therefore not practical and roughness measurements have to be confined to certain ranges. The measured roughness (*i-angles*) depends, however, on the measuring interval and consequently also the shear strength calculated from this roughness(35).

Three or two dimensions
Most empirical shear strength relations or roughness profiles (e.g. Barton et al., 1977, ISRM, 1978b, 1981a, Laubscher, 1990, etc.) that include discontinuity roughness are based on two-dimensionality whereas the reality is three-dimensional. Discontinuity surfaces can be highly irregular in three dimensions. Fig. 27 shows a series of parallel roughness profiles measured with a laser roughness meter on one discontinuity plane (Baardman, 1993). It is clear that the profiles are considerably different and that a shear strength calculation based on one profile will be different from those calculated on the other profiles. A complicating factor is that during displacement the contact points between two irregular surfaces can be anywhere. For these reasons measurement of roughness should be done in three-dimensions (Fecker et al., 1971, Rengers, 1971).

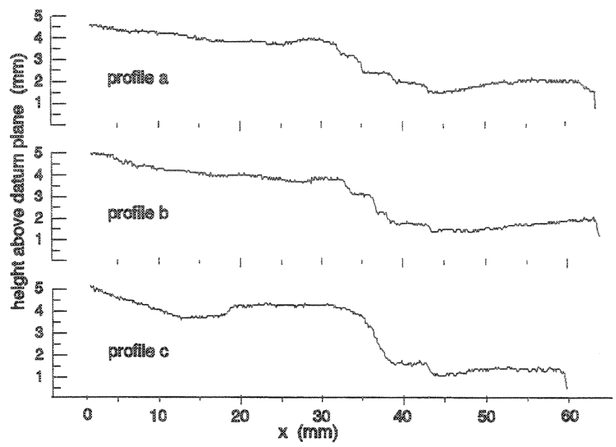


Fig. 27. Parallel roughness profiles of one discontinuity plane. Spacing between profiles ≈ 1.5 cm (after Baardman, 1993).

The problems with defining datum planes, measuring interval and the three-dimensionality of irregular surfaces make that measuring of roughness is highly complicated. Therefore it is not practical to include the measuring of roughness in a classification system.

C.3.3.2.3 Estimating roughness and roughness profiles

The foregoing chapters show that a simple theory of shear strength based on material friction and measured roughness angles is not satisfactory for natural discontinuities. The theory does not consider all parameters (e.g. deformation, etc.) and measuring of roughness of a natural discontinuity plane is not realistic on a large scale. Estimating the contributions of roughness to the shear strength of a discontinuity is an alternative approach. This is easiest done if it is divided in ranges. A simple practical division can be established by the naked eye to give (Fig. 28): 1) roughness which cannot be seen and 2) visible roughness which can be estimated by visual comparison with standard roughness profiles(36). A large advantage of this method is that it does not need an extensively exposed discontinuity plane. It is often enough to see traces of the discontinuity in different directions. An example of this approach are the standard roughness profiles and the empirical relation that relates the profiles to shear strength values that have been developed by Barton et al. (1977). Barton introduced the JRC standard roughness profiles as a means to be able to describe roughness profiles, which are also related to shear strength:

(35) Fractal representation of roughness is proposed as a solution for this problem (Carr, 1989, Lee et al., 1990, etc.). Research showed, however, that the results published may be accidental. Fractal representation is therefore not suitable without further research and a proper definition of the used methodology (Den Outer et al., 1995).

(36) Visible roughness is that which can actually be seen. Light reflection characteristics (lustre) partly depend on roughness but on a far smaller scale, and are not included in visible roughness. Measuring of roughness can be done by means of a laser-profile meter, by photogrammetry or, for larger scale roughness with rulers, theodolites, etc.. The range for visible roughness is normally limited to a maximum. For the SSPC classification system the maximum is 1 m (section D, Fig. 69). Roughness on a larger scale than the maximum, for example, large waviness in bedded rocks, implies a change in dip and hence a new geotechnical unit.

$$\frac{\tau_{peak}}{\sigma'_n} = \tan\left[JRC * \log_{10}\left(\frac{JCS}{\sigma'_n}\right) + \varphi_r\right]$$

τ_{peak} = *peak shear strength* [12]
σ'_n = *effective normal stress on discontinuity plane*
JCS = *discontinuity wall compression strength*
JRC = *discontinuity roughness coefficient*
φ_r = *residual friction angle*

A problem with the JRC roughness profiles is that they do not include stepped surfaces and require measurement of the residual friction angle. Also, in the author's experience it is often very difficult to establish the proper JRC number visually.
Laubscher (1990) developed a thorough set of descriptive terms for roughness of discontinuities with factors rating the influence on the stability of underground excavations. The descriptions used by Laubscher are partly based on the profiles published by ISRM (1978b, 1981a). The roughness is divided in roughness that cannot be seen, but can be felt by using fingers (tactile roughness), and roughness that can be seen, which is described visually at different scales. This set of descriptions is used in Laubscher's classification system for underground excavations (ch. B.2.3.3). Drawbacks are that dimensions for the roughness profiles are not given, the profiles are partly ambiguous, representative profiles for large scale roughness have not been published, and in particular the combination of tactile roughness and small scale roughness is not clearly defined.

C.3.3.2.4 Stepped roughness planes

Stepped roughness planes are planes on which asperities with sides occur for which applies that φ + *i-angle* ≥ *90°*. These asperities are normally denoted as steps on the discontinuity plane, although the *i-angle* does not have to be 90°. If a step is present perpendicular to the direction of sliding then either the step has to be sheared off before the block can slide or so much dilatancy deformation has to be possible that the block can slide over the asperity. Steps on surfaces often prohibit sliding (appendix II). None of the empirical relations take this into account. The standard profiles by ISRM (1978b, 1981a) and Laubscher (1990) do, however, include stepped planes (Fig. 69, page 142, and Fig. 70, page 143).

C.3.3.2.5 Anisotropic roughness

Roughness of a surface can be anisotropic (e.g. ripple marks, striation, etc.), and thus the shear strength will be direction dependent. Theoretically the roughness should therefore be measured in different directions. The number of different directions that should be measured depends on the type of the roughness. For example, it is sufficient to measure the roughness in one direction only for a regular striation; perpendicular to the striation the contribution to the shear strength of the roughness due to the striation is maximum while parallel to the striation no influence of the striation is present. For less regular surfaces the number of directions in which the roughness has to be measured increases, but roughness in all directions will be again about equal for a fully irregular surface and one measurement will be sufficient. Alternatively the roughness can be measured only in the direction in which shear displacement over the discontinuity is expected (this direction will often be known in slope instabilities).
In practice it will mostly be sufficient to determine the roughness in one direction or in two perpendicular directions only; parallel and perpendicular to the maximum roughness. The accuracy of roughness determination and subsequent translation into friction angles is, in general, not high enough to justify the determination of roughness in more than two directions.

C.3.3.2.6 Discontinuity history

The history and origin of a discontinuity have an influence on the shearing characteristics of the discontinuity. If movement along a discontinuity has taken place in the past then two situations are possible:

1 Due to the movement asperities have sheared off completely and the roughness of the discontinuity is nil. The roughness of the discontinuity is determined as found and thus the history is included in the assessment of the roughness.

2 The movement happened without shearing off the asperities or the asperities are only partially sheared off. The resulting discontinuity has then a non-fitting roughness profile and the dilatancy necessary to allow further displacement is lower (Rengers, 1971). In this situation testing might help to guess an accurate value of the shear strength or an estimate can be made by which amount the necessary dilatancy (or *i-angle*) is reduced due to the displacement. For example, the shear strength of a discontinuity that is not fitting at all, is governed by the material friction only.

C.3.3.2.7 Conclusions

A summary of the different ranges for roughness with wavelengths and amplitudes for regular forms of roughness is shown in Fig. 28. The boundary lines are dashed as these are not exact. The wavelengths and amplitudes for the roughness profiles are an indication only. The figure is an attempt to combine normally occurring different types, scales, and measuring methods of roughness and is not expected to cover all forms of roughness[37].

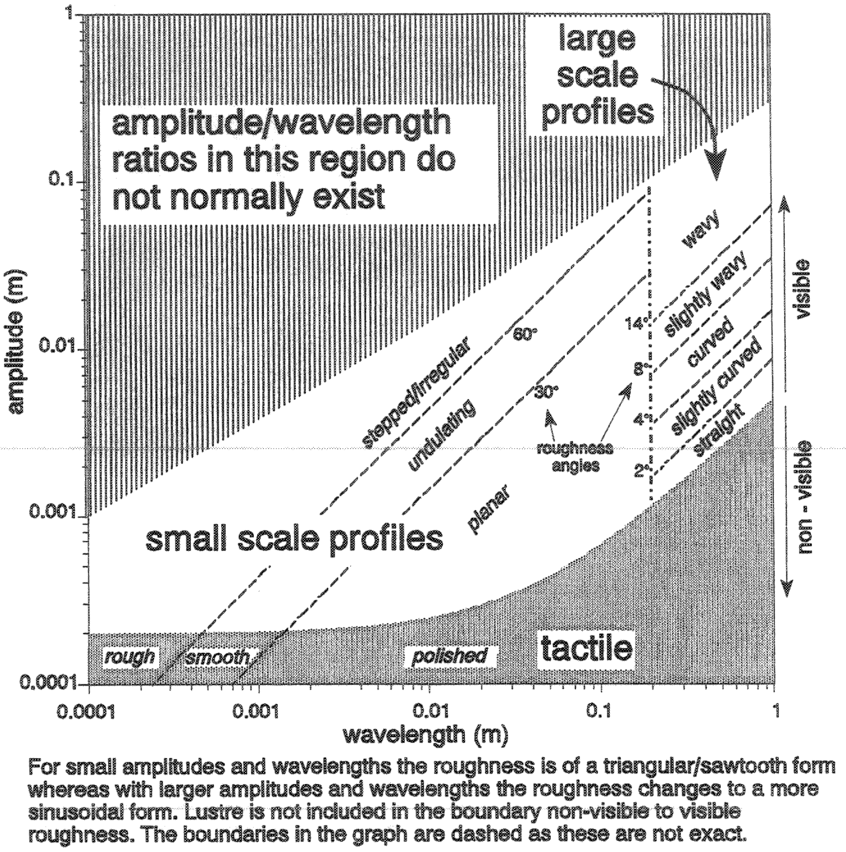


Fig. 28. Interpretation of regular forms of roughness as function of scale and angle.

In this research, a new empirical relation between tactile and visible roughness based on the ISRM (1978b, 1981a) profiles, and the friction along a discontinuity plane resulting from roughness, is developed because of the problems with existing shear strength theory and roughness as described above. For this purpose the roughness profiles of ISRM (and Laubscher) have been modified. Tactile roughness is to be distinguished by feeling with fingers and described in three classes: rough, smooth and polished. The small scale roughness determined on an area of 20 x 20 cm^2 of the discontinuity surface, should be visible and is described in three classes: stepped, undulating and planar. Representative example profiles including scales are provided in Fig. 70[38] (page 143). The vertical scale of these profiles is based on the minimum step height required to prohibit crushing effects in steps (ch. C.3.2.1). The large scale roughness determined on an area larger than 20 x 20 cm^2 but smaller than 1 x 1 m^2, is described in five classes: wavy, slightly wavy, curved, slightly curved and straight. For large scale roughness examples of profiles with scales and *i-angles*[39] are presented in Fig. 69[38] (page 142). The roughness profiles are included in Fig. 28. Values for each roughness description that rate the influence on slope stability, have been copied from Laubscher for the 'initial point rating' system. In the SSPC system (section D) the values have been adjusted based on the data obtained in this research.

(37) For example, stylolites in limestones or very coarse grained rocks (e.g. porphyritic granites) could plot in the region which is indicated as 'do not normally exist'.

(38) Changes between roughness profiles for the 'initial point rating' and SSPC system are only minor. Therefore the profiles are not repeated in this chapter.

(39) The *i-angles* were not included in the 'initial point rating' system but have been derived from data gathered during the fieldwork for this research (Fig. A 98, ch. D.1.2.1).

If the roughness is direction-dependent the roughness should be assessed in two perpendicular directions. If movement along a discontinuity has taken place in the past then the influence of this movement on the shear strength along the discontinuity should be quantified by estimating the remaining *i-angle* or the discontinuity has to be tested.

C.3.3.3 Alteration of a discontinuity wall

The discontinuity wall is the rock material directly adjacent to the discontinuity. It is the material which, if in contact, will determine the shear strength along the discontinuity. Determining the shear strength characteristics of discontinuities requires that the joint wall condition or joint wall strength should be established. Various authors have commented on the influence of the strength of the discontinuity wall on shear strength (Bandis, 1990, Barton et al., 1973a, 1973b, 1976b, 1977, 1985, Laubscher, 1990, Fishman, 1990, Rengers, 1970, 1971, Rode et al., 1990). Often the 'quality' (strength) of the discontinuity wall is lower than the intact rock strength (also ch. C.3.2.1). The decrease in strength may have been caused by weathering features, brought about by chemically charged water percolating through discontinuities that reacted with the wall, etc.. The thickness of the layer having a lower strength may range from microscopic thickness up to many centimetres or more. In shearbox tests the discontinuity wall strength is incorporated in the results, however, shearbox tests can only be done on samples of limited size. Strength and thickness of the joint wall must be known to understand the shear strength test results.

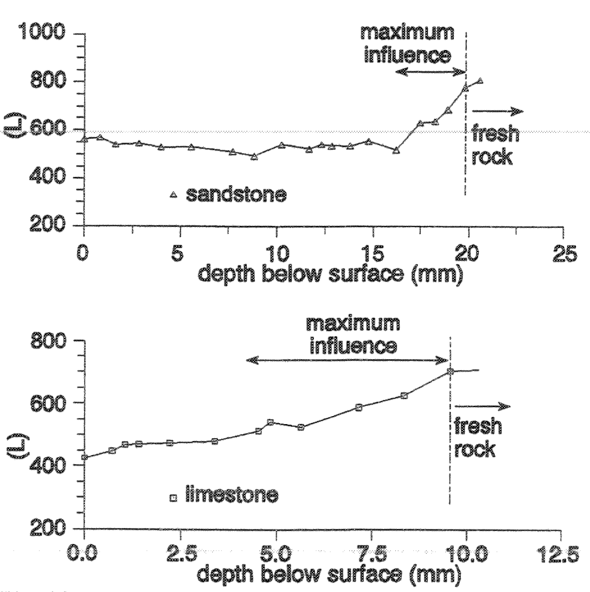


Fig. 29. Equotip rebound values on weathered discontinuity walls progressively ground down to fresh rock (after Hack et al., 1993a).

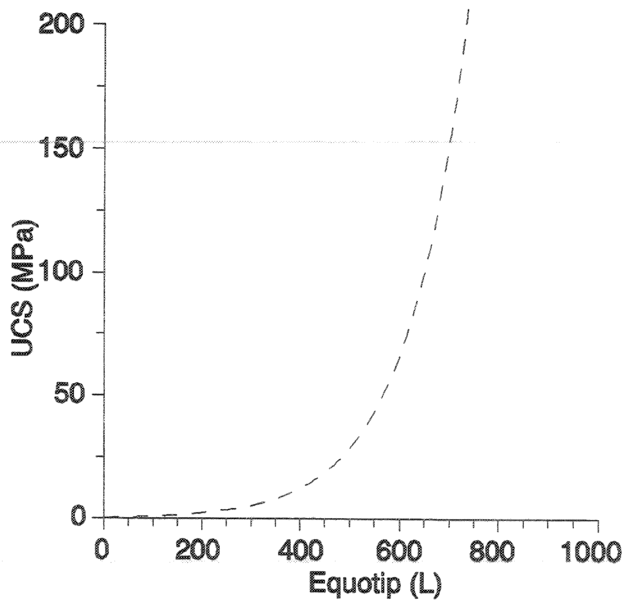


Fig. 30. UCS vs Equotip (after Verwaal et al., 1993).

Rebound tests are a method which may be suitable to assess discontinuity wall strength. The best-known rebound test is the Schmidt hammer[40] (ISRM, 1978a, 1981a, Rode et al., 1990, Stimpson, 1965). Other rebound measurements are based on a hammer, ball or piston which drops from a certain height on to the surface to be measured (Equotip, 1977, Hack et al., 1993a, ISRM, 1978a, 1981a, Pool, 1981, Price et al., 1978, Stimpson, 1965, Verwaal et al., 1993). The rebound of the piston, hammer or ball after hitting the surface is dependent on the elastic parameters of the tested material and on the strength of the material at the surface of the discontinuity. This latter effect is caused by the crushing of surface asperities and surface material, which dissipates energy. Most of the rebound tests reported in the literature are not developed to measure the discontinuity wall strength but to measure the intact rock strength.

The standard form of the Schmidt hammer releases so much energy over such a large area that in most rocks a layer of up to centimetres depth influences the measurement. The ball rebound device (Pool, 1981, Price et al., 1978) and the Equotip device (Equotip, 1977, Hack et al., 1993a, Verwaal et al., 1993) release considerably less

[40] Different designs of Schmidt hammers for different impact energies exist. 'L' and 'N' design are most commonly used in the field.

energy and may therefore be better methods of establishing discontinuity wall parameters. Fig. 29 shows that the depth of the material influencing the Equotip measurement is maximum about 5 mm and Fig. 30 shows the correlation between Equotip rebound and unconfined compressive strength (UCS). This correlation suffers, however, from scale effects, the Equotip testing a much smaller volume of rock than the UCS. Thus exact correlation is unlikely to be achieved. In the development of the new classification system the quality of the discontinuity wall has been established by using the Equotip measuring device. The Equotip was used to determine whether the discontinuity wall had a lower, equal or higher strength than the intact rock strength. The values rating the influence of the alteration of the discontinuity wall on slope stability have been copied from Laubscher (1990) for the 'initial point rating' system (ch. C.4). For the SSPC system (section D) the parameter was found not to be necessary and the parameter is not used in the SSPC system.

C.3.3.4 Discontinuity infill material

The importance of discontinuity infill was recognized in nearly all rock geotechnical disciplines and many different description systems for discontinuity infill have been proposed, often as part of a rock mass classification system (ch. B) (Barton et al., 1974, 1980, Bieniawski, 1973, 1976, 1989, Holtz et al., 1961, Lama, 1978, Laubscher, 1990, Tulinov et al., 1971, etc.). The type of infill material and whether the walls of the discontinuity will be in contact or not during shearing, have a very strong influence on the shear strength characteristics. Types of infill materials may considered to be either cemented, non-softening or softening under influence of water, deformation or shear displacement. The material itself is reported to be generally of minor influence (Barton et al., 1974, 1980, Tulinov et al., 1971). Testing the shear strength of discontinuities that include infill material is very difficult. Proposals for testing discontinuities with disturbed infill material have been made by Goodman (1989), but only in-situ tests are a reliable means to test shear strength of undisturbed filled discontinuities. In most situations estimating the shear strength is therefore the only option. Because the infill material itself is of minor importance, the number of classes necessary to describe a filled discontinuity and to estimate the shear strength parameters can be done with a relatively simple and limited set of classes.

C.3.3.4.1 Aperture or width of discontinuity

A parameter often used in the description of discontinuities in relation to infill, is aperture or width of the discontinuity (Bieniawski, 1973, 1976, 1989, Brekke et al., 1972, ISRM, 1981a, etc.). In these descriptions an aperture or width of a discontinuity larger than zero, implies that the discontinuity is filled with material over a certain distance with a continuous layer (band) of infill material with more or less the same thickness, or is completely open without any contact between the walls of the discontinuity. The latter, completely open discontinuities, can occasionally be observed at surface and are usually vertical. Completely open discontinuities are obviously not common in other situations as the stresses working on the discontinuities will cause closure. Although aperture is included in some descriptions, other research has shown that aperture itself is often of minor importance for the shear strength characteristics. The shear strength characteristics of a smooth, planar discontinuity can be roughly divided in three ranges (Phien-wej et al., 1990):

1) If there are points of contact between the discontinuity walls, the shear strength is mainly determined by the properties of the discontinuity walls.
2) If the infill thickness in the discontinuity is less than about the grain size of the intact rock grains or minerals in the discontinuity walls or of the grain size of the infill material, the shear strength of the discontinuity is that of the infill but influenced by the discontinuity wall material.
3) If the infill thickness is larger than the grain size of the discontinuity wall and the grain size of the infill, the shear strength is that of the infill material.

Aperture for irregular discontinuity walls is therefore meaningful only if the aperture of a discontinuity is related to the amplitude of the roughness of the discontinuity walls or is related to the grain or mineral size of the infill or rock material. The above does not apply to non-filled discontinuities (such as karstic discontinuities) for which aperture can be important (ch. B.3.4.7).

C.3.3.4.2 Origin of a discontinuity or origin of infill material

Some classification systems describe discontinuity infill material based on the origin of the discontinuity (Brekke et al., 1972) because the origin of a discontinuity can have a relation with the shear strength characteristics of the discontinuity. For example: bedding planes will often be a potential discontinuity because the plane is formed by more softer or easier weathered materials (e.g. clay) than the rest of the rock mass, whereas tectonic joints will normally have an infill material consisting of weathered intact rock material. This method of description implies the risk of totally wrong assessments. The author has often observed bedding planes that did not contain any clay infill material and observed tectonic joints filled with clay material that was not weathered intact rock. It could even have been that the clay material of the bedding plane had been washed out of the bedding plane and accumulated in the joints. In this situation it would be obviously totally wrong to determine the shear strength parameters based on the origin of the discontinuity. Origin of infill material is obviously a better means of describing the discontinuity characteristics (Welsh, 1994). However, often the origin of the infill material can only be established by a detailed description of the infill material. Therefore it seems more logical to relate the infill material to shear characteristics directly than by first establishing the origin of the infill.

C.3.3.4.3 Conclusions

The classes used in this study (and described in this chapter), roughly follow those established by Laubscher (1990). The system is relatively simple and no expert knowledge of geology is necessary. An additional class for 'cemented/cemented infill' discontinuities has been included.

No infill, cemented or not cemented

The first distinction to be made is between: no infill, cemented, cemented infill or non-cemented infill. 'No infill' describes a discontinuity that may have coated walls but no other infill. For most discontinuity surfaces friction is virtually independent of the minerals of the intact rock. This has been established by many researchers doing tests on smooth, planar surfaces to obtain φ_{basic} and is also confirmed by tests done for this research (Hack et al., 1995, appendix III). Apparent cohesion of the discontinuity walls does depend on the type of mineral but at low levels of low normal stress apparent cohesion is less important (ch. C.3.3.2.1). For mineral coatings on discontinuity walls the same applies (Welsh, 1994), also confirmed by tests done for this research (Hack et al., 1995, appendix III). Therefore one class describing the shear strength of a non-cemented, non-filled discontinuity is sufficient.

A cemented discontinuity or a discontinuity with cemented infill has a higher shear strength than a non-cemented discontinuity if the cement or cemented infill is bonded to both discontinuity walls. If there is no cement bond between the discontinuity walls or between the cemented infill and one or both discontinuity walls the discontinuity behaves as a non-cemented, non-filled discontinuity. Two classes should be distinguished for discontinuities with a cement bond or with cemented infill bonded to both discontinuity walls: 1) the cement or cemented infill and bonding to both discontinuity walls are stronger than the surrounding intact rock (failure will be in intact rock), and 2) the cement or cemented infill and bonding are weaker than the surrounding rock but still stronger than a non-filled discontinuity. Those that are stronger than the surrounding rock do not need to be considered as a discontinuity, those weaker are described with the class 'cemented/cemented infill'.

Non-softening and softening infill

A major distinction should be made between non-softening and softening material for discontinuities without cement but with infill material (Barton, 1974, 1980, Laubscher, 1990, Tulinov et al., 1971). Non-softening infill material is material that does not change in shear characteristics under the influence of water nor under the influence of shear displacement. The material may break but no greasing effect will occur. The material particles can roll but this is considered to be of minor influence because, after small displacements, the material particles will generally still be very angular. Softening infill material will under the influence of water or displacements, attain in a lower shear strength and will act as a lubricating agent. Both classes of softening and non-softening infill material can be further sub-divided in classes according to the size of the grains in the infill material or the size of the grains or minerals in the discontinuity wall. The larger of the two should be used for the description (Tulinov et al., 1971, Laubscher, 1990).

Gouge

The so-called 'gouge[41] filled' discontinuities are a special case. Gouge filled discontinuities are often the larger discontinuities in a rock mass such as faults. Gouge layers are relatively thick and continuous layers of infill material, mainly consisting of clay but often also containing rock fragments. The common feature of gouge is the presence of clay material that surrounds the rock fragments in the clay completely or partly, so that these are not in contact with both discontinuity walls. The initial shear strength of such a discontinuity will be that of the clay. If the gouge is thicker than the amplitude of the roughness of the discontinuity walls, shear movement is governed by the clay material. If the thickness is less than the amplitude of the roughness the shear strength will be influenced by the wall material and the discontinuity walls will be in contact after a certain displacement; for further displacement the shear strength is governed by the friction along the discontinuity walls in combination with the clay infill and the friction of the rock fragments in the gouge.

Flowing material

Very weak and not compacted infill in discontinuities flows out of the discontinuities under its own weight or as a consequence of a very small trigger force (such as water pressure, vibrations due to traffic or the excavation process, etc.).

For the 'initial point rating' system (ch. C.4) values that rate the influence of the different infill materials on slope stability have been copied from Laubscher (1990) or are studied guesses from the author. The values have been adjusted for the SSPC system (section D) based on the data obtained during this research.

C.3.3.5 Weathered discontinuities

Weathering of discontinuities results, in most rock material, in weakening of the discontinuity wall and in the formation of infill material in the discontinuity. The shear strength of such a weathered discontinuity is determined more by the presence of infill material than by the reduction of the shear strength due to the weakening of the discontinuity walls[42]. Reduction of the shear strength of the discontinuity walls become important only if the weathered material is flushed out of the discontinuity completely. However, usually a thin layer or coating of weathered material stays behind in the discontinuity. For example, in carbonate rock masses containing some clay, it is often found that the discontinuity walls are slightly weathered and that a thin clay infill is found in the discontinuities, this being all that remains of the weathered rock material. The remaining infill significantly determines the shear strength of the discontinuity. A separate parameter for weathered discontinuities is therefore not necessary.

C.3.3.6 Discontinuity karst features

Karst features have been found to be of importance in slope stability. The open holes considerably weaken the rock mass (ch. B.3.4.7). In the 'initial point rating' system (ch. C.4) karst was described per discontinuity set in terms of occurrence and size of the karst holes. The values used for the karst parameter (Fig. 36, page 84) in the 'initial point rating' system are studied guesses of the author as no literature references were found. In the SSPC system the values are calculated from the influence of karst on the stability of existing slopes (ch. D.1.2.1.2).

C.3.3.7 Effect of water pressure in discontinuities

Water pressures in discontinuities reduce the shear strength of the discontinuities (ch. A.2.3), which is the reason that many classification systems for underground excavations include a separate parameter quantifying this influence. In ch. B.3.4.12 is already discussed that the influence of water pressures on slope stability may be less important than often assumed. The methodology used in this research to develop the classification system for slope

(41) 'Gouge' is an ancient mining term which implies soft, easily extracted material (see glossary, page 241).

(42) This has been confirmed during this research for slope stability assessment where was found that the reduction of discontinuity wall strength is not important if even small quantities of infill material are present (ch. D.1.2.1.2).

stability, may even further reduce the need for a separate parameter for water pressures (whether surface run-off water should be included as a separate parameter is discussed in ch. C.3.6.1). Consider the two following situations: 1) a new slope is made in similar conditions, with respect to water, as the exposures used for the classification (Fig. 31, cut A), and 2) a new slope is made which is totally different with respect to water conditions (Fig. 31, cut B).

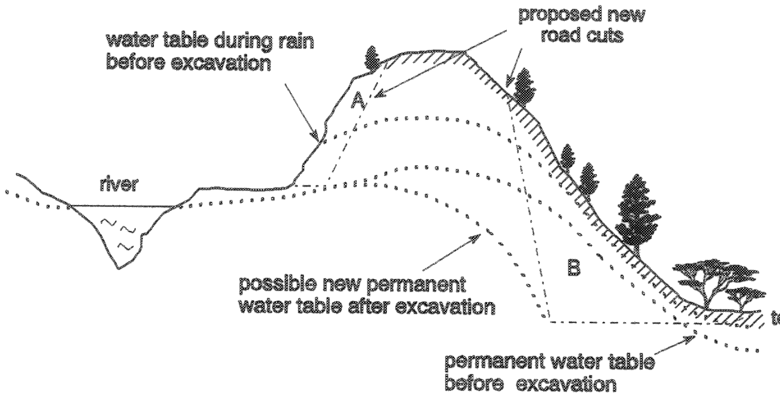


Fig. 31. New slopes in different conditions with water table.

1 *New slope in similar conditions with respect to water*

All slopes used for the development of the slope stability classification system in this research are situated in a temperate (Mediterranean) climate (ch. A.3.1) and with a recurrence period of a few years heavy rainfall takes place. Therefore, all slopes being used for referencing the slope stability classification system have been subject to rainfall appropriate to the climate, leading to the presence of water and probably to water pressures in the discontinuities (Fig. 31, cut A). This influence of water in discontinuities is thus likely already incorporated in the weighting of the parameters in the slope stability system.

2 *New slope in different conditions with respect to water*

If a new slope is made which intersects the permanent water table (Fig. 31, cut B) and the classification for the reference rock mass has been made on exposures not intersecting a permanent water table a correction on the classification system may have to be applied to allow for the unfavourable water condition(43).

A correction to the classification for slope stability is thus likely only necessary in those very few occasions where a new slope intersects a permanent water table with water pressures in the rock mass directly behind the slope face(44). In the 'initial point rating' system (ch. C.4) a parameter was incorporated that corrects the stability assessment in case the slope shows permanent water seepage, thus for a slope intersecting a permanent water table. The correction values used for this parameter were the same as those used by Laubscher (ch. B.2.3.3). The quantities of water flowing out of the rock mass as used by Laubscher, have, however, been reduced by the author to be feasible for slopes. In the SSPC system (section D) a parameter for the influence of water has been omitted on the basis of the results of the analysis of the data of the existing slopes (ch. D.1.7).

C.3.3.8 Practical aspects of shear tests on discontinuities

Testing the shear strength of a discontinuity can be done by field and laboratory tests. In practice the various tests contain serious shortcomings and will only give crude estimates of reality. All non-in-situ field and laboratory tests on discontinuities are hampered by difficulties in sampling and executing of the tests. Therefore, no testing of discontinuities is required for the slope stability classification system developed in this research. The problems involved in testing of shear strength have been commented on by many authors (Goodman, 1989, Cunha, 1990, 1993). Example II (ch. D.5.2) illustrates the problems encountered with testing shear strength carried out for this research.

(43) It should, however, be considered that: i) a new slope cut will be unlikely to allow free drainage of a permanent water table and artificial measures would be taken to lower the water table behind the slope (drains above the slope, drainage holes in the slope, etc.) and, ii) often exposures used for the classification of the reference rock mass, not intersecting a permanent water table, intersect the increased water table during rain (Fig. 31, cut A). In both situations a correction is not necessary.

(44) In the research area this situation does not occur and, in the author's experience, also in other areas this rarely happens.

C.3.3.9 Conclusions

The evaluation of discontinuity shear strength properties and the possibilities to measure parameters to describe these properties lead to the conclusion that a simple classification of parameters based on tactile and visual observations of outcropping discontinuities is the only feasible possibility to include discontinuity properties in a classification system. More sophisticated measuring methodologies are not necessarily better, mostly highly unpractical and not suitable for field use for a classification system. The various simple methodologies for the different parameters as described in this chapter, are implemented in the 'initial point rating' system and are shown on the field exposure characterization form in Fig. 36 (page 84). Some methodologies are modified for the SSPC system in section D based on the data obtained in this research.

C.3.4 Sets of discontinuities versus single discontinuities, concept of discontinuity spacing

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. These are briefly described, with their advantages and disadvantages. This evaluation leads to conclusions of how parameters describing discontinuity orientation, spacing and discontinuity shear strength are best implemented and measured for a rock mass classification system for slope stability. All parameters are determined separately for each geotechnical unit.

C.3.4.1 Discontinuity sets

A description of each single discontinuity in a rock mass would lead to an unreasonable quantity of work; calculations with hundreds or thousands of discontinuities are very time-consuming. However, discontinuities occur often as a regular feature, e.g. bedding planes, cleavage, regular sets of joints or fractures, etc. (ch. A.2). Therefore a normal procedure in discontinuous rock mechanics is to group discontinuities in sets (or families). All discontinuities in a set are then considered to have broadly the same characteristics such as orientation, spacing, roughness, etc.. Shear zones or faults may also occur in a set but occur usually as a single phenomena on the scale of engineering works.

Sets of very widely spaced discontinuities or single discontinuities
If the discontinuities in a set are very widely spaced (for example, if the spacing is considerably more than the dimensions of the tunnel, slope, borehole, etc., or if the discontinuities are very widely spaced compared to the dimensions of the geotechnical unit) then each discontinuity of the set may be treated as a single discontinuity in subsequent analyses. For each single occurring discontinuity all characteristics should be described and measured so that in further calculations these can be dealt with individually.

C.3.4.2 Grouping discontinuities and determining characteristic discontinuity properties and parameters

Grouping discontinuities in sets based on their properties and finding the characteristic properties and orientation of a discontinuity set can be done with various methods.

C.3.4.2.1 Geological and structural analyses.

A geological and structural geological analysis consists of the determination of the discontinuity properties in various exposures, the relations between the different discontinuities and discontinuity sets, and the origin and history of the discontinuity sets. These are plotted on plans and sections representing the volume of the rock mass or directly into a three-dimensional GIS. Stereographic representations can be used (Maurenbrecher et al., 1990, 1995, Phillips, 1972). A geological and structural geological analysis will in many situations allow for prediction of the properties of discontinuity sets in-between the exposures. Fig. 32 shows a simple example where the bedding is estimated to be about horizontal at the location of the new slope. A complete discussion on how to determine discontinuity properties from structural and geological analyses is outside the scope of this research and can be found in books on geology and structural geology.

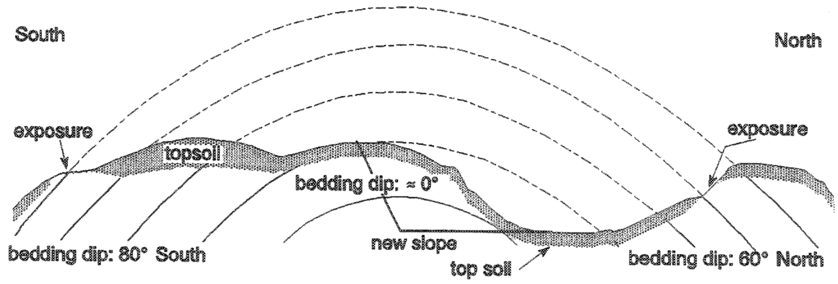


Fig. 32. Geological and structural geological analyses to obtain discontinuity properties.

C.3.4.2.2 Scanline method

An often applied practice to determine characteristic discontinuity properties in an exposure is by a form of averaging of the properties of all individual discontinuities crossing a scanline. Scanlines are normally positioned at an easily accessible location on the exposure, mostly about 1.5 m above the ground and horizontally oriented. This implies that: 1) depending on the height of the exposure only a part (and often very small part) is represented in the measurements, 2) discontinuity sets with a (very) large spacing may be missed, and 3) all discontinuities with a trace (near-) parallel to the scanline are under-sampled. The last-mentioned is, in particular for slope stability, a large shortcoming as these discontinuities may be the planes of instability for plane sliding.

C.3.4.2.3 Exposure - measuring and averaging discontinuity properties and parameters

If discontinuities occur as a set the average orientation 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 contouring is that it may be difficult to distinguish between the different discontinuity sets in a stereo plot. Furthermore, an important discontinuity set may be missed out or underrated in importance because the discontinuity spacing is large. In a stereo plot such a discontinuity set may be masked by a less important but far more often measured discontinuity set. A single discontinuity with an unfavourable dip and dip-direction for the slope can well determine slope stability while the mean or average direction and dip of the whole set do not indicate an unstable situation. If all discontinuities in a rock mass have been measured and plotted this is no problem because the stereo scatter plot will show this critical discontinuity. Often, however, only the discontinuities along a scanline or in a particular (accessible) area of the exposure are measured. Then the most unfavourable discontinuity may well have been missed if it happened not to cross the selected scanline, not to be present in the area of the exposure where the measurements are done, or unfavourably oriented with respect to the exposure, e.g. discontinuities near parallel to the face of the exposure. The 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 analysed. 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.

C.3.4.2.4 Exposure - studied assessment and interpreted properties and parameters

In a studied assessment to determine discontinuity properties in an exposure, those discontinuities that are most unfavourable for the engineering structure or if that is not a priori known, the discontinuities that are representative for the set are visually selected. In this selection is incorporated the whole exposed area (as this selection 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(45). In the opinion of the author based on experience during former work(46) and during

(45) If selected discontinuities happen not to be accessible the orientation can often be measured from a distance by simple means, such as clinometer and compass or photogrammetry. However, properties of a not-accessible discontinuity (set) have to be estimated.

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).

C.3.4.2.5 Borehole cores

Grouping the discontinuities in sets and determining the mean or characteristic discontinuity properties and parameters of the sets can be done by the methods discussed for exposures[(47)]. It should be noted that borehole cores show only a very small part of a discontinuity surface and that consequently the determination of properties may be less accurate.

C.3.4.3 Overall spacing of discontinuity sets in a rock mass

Various expressions have been defined to quantify in a single qualitative or quantitative expression the spacings of a number of discontinuity sets in a rock mass. One of the simplest expressions is the RQD (Deere, 1967, 1988, 1989, ch. B.2.2), more detailed expressions, which describe block size and block form in a rock mass, can be found in BS 5930 (1981, ch. B.2.1), Price (1992, ch. B.2.1). Taylor (1980) developed eq. [13] for the description of the spacing for a maximum of three discontinuity sets.

For a rock mass with one discontinuity set:

$$factor_1 = 0.45 + 0.264 * \log_{10} x \qquad factor_2 = factor_3 = 1$$

(x = *discontinuity spacing in cm*)

with two discontinuity sets:

$$factor_1 = 0.38 + 0.259 * \log_{10} x_{minimum} \qquad factor_2 = 0.28 + 0.300 * \log_{10} x_{maximum}$$

$$factor_3 = 1 \qquad [13]$$

with three discontinuity sets:

$$factor_1 = 0.30 + 0.259 * \log_{10} x_{minimum} \qquad factor_2 = 0.20 + 0.296 * \log_{10} x_{intermediate}$$

$$factor_3 = 0.10 + 0.333 * \log_{10} x_{maximum}$$

$$spacing\ factor\ for\ rock\ mass = factor_1 * factor_2 * factor_3$$

(minimum, intermediate and maximum refer to the spacing of the discontinuity sets)

The graphical representation is shown in Fig. 33. The parameter is calculated for a maximum of three discontinuity sets with the lowest spacings. The method according to Taylor is used in Laubscher's classification system

(46) Experiments (unpublished) done by the author while employed in an underground mine showed that scanline analyses compared to studied assessments of the orientation and spacing of various discontinuity sets resulted in nearly the same values if the discontinuity sets were clearly distinct and if done in small (maximum 2 x 2 m) tunnels with crosscuts allowing for scanlines in all directions (also along the roof). The studied assessments and statistical analyses were done by different qualified engineers who also incorporated discontinuity type and properties in the analyses. The statistical analyses often, however, missed discontinuity sets if the same comparison was done in large tunnels or in tunnels without crosscuts (thus not allowing for scanlines in all directions), or if the sets were not clearly distinct or had a (very) large spacing.

(47) In borehole cores spacing is often measured irrespective of the discontinuity sets, as, for example, in measuring the RQD. This is often inevitable because the borehole cores are drilled without marking the orientation. The orientation of discontinuities is, however, a main factor in determining the stability of a slope and boreholes drilled for slope stability assessments should thus always be drilled to produce orientated cores.
The discontinuity spacing measured in borehole cores may be effected by new discontinuities formed due to the stress relief as a consequence of drilling. The measured discontinuity spacing is then lower than in-situ. This effect is more severe for borehole cores from a large depth than for cores from a relatively shallow depth as would be drilled for the type of slopes for which the classification system is developed and is therefore not further discussed.

for underground excavations (1990, ch. B.2.3.3). Many engineers, including the author, have extensively used, with success, the Laubscher system for classification of the stability of underground excavations in a mining environment. The good results obtained with Laubscher's classification system for underground excavations is the reason to investigate the possibility to include a parameter describing the spacings of a number of discontinuity sets in a rock mass, calculated analogous to Taylor (1980), in the classification system for slope stability developed in this research[48].

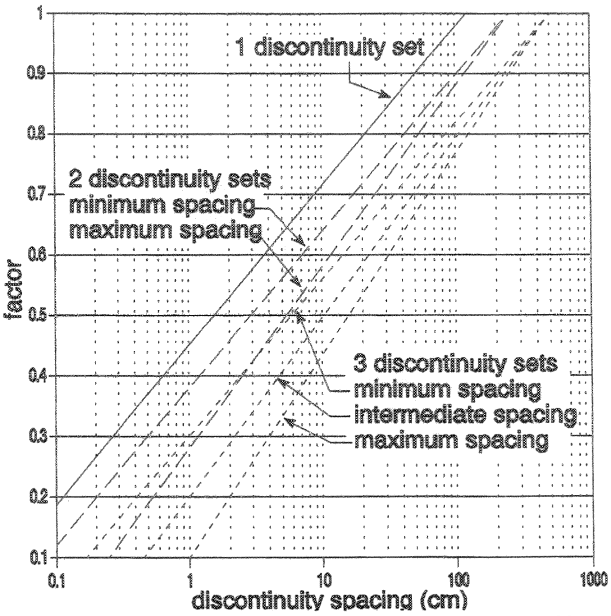


Fig. 33. Discontinuity spacing factors (after Taylor, 1980).

C.3.4.4 Overall condition of discontinuity sets in a rock mass

Several options exist to describe the overall properties describing the shear strength of discontinuity sets in a rock mass. In most existing classification systems only that discontinuity set is considered that has the most adverse condition. This can lead to problems as discussed in ch. B.3.4.5. A solution to these problems is to use an average or a weighted mean of the condition of the different discontinuity sets. In the 'initial point rating' system (ch. C.4) the parameter describing the overall condition of the discontinuities is the mean value of the three discontinuity sets with the lowest condition ratings, weighted inversely against the spacing. For the SSPC system (section D) different methods to quantify an overall condition have been investigated.

C.3.4.5 Conclusions

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. Geological and structural geological approaches can be used to determine these properties and parameters at locations where the rock mass is not exposed. It should be realized that these methods do not result in highly accurate values because the variation of properties and parameters in most rock masses is large. This implies that a very high accuracy in determining parameters in an exposure is mostly not necessary.
In the 'initial point rating' system (ch. C.4) the methodology according to Taylor is used for the overall spacing of a number of discontinuity sets in a rock mass and a weighted mean is used for the overall condition. For the development of the SSPC system (section D) various options are investigated to quantify the overall spacing and condition of a number of discontinuity sets in a rock mass.

[48] In the 'initial point rating' system (ch. C.4) the parameter calculated following Taylor is multiplied with 25 to achieve a point rating for the spacings of a number of discontinuity sets in a rock mass. For the SSPC system a comparison is made between different approaches to calculate a quantitative parameter for the spacings of a number of discontinuity sets in a rock mass of which one is calculated analogous to Taylor (ch. D.1.3.3).

C.3.5 Exposure and slope specific parameters

C.3.5.1 Degree of rock mass weathering

The degree of rock mass weathering[49] is classified following the British Standard[50] (BS 5930, 1981, Fig. 36 and appendix V, Table A 20) and expressed as a value. This value is used to compensate intact rock strength and spacing and condition of discontinuities for the influence of local weathering on the rock mass in exposures and on the rock mass in which a new slope is excavated. The British Standard is used for the weathering classification because it is worldwide known and used. The degrees are described by characteristics that can be determined visually. The degree of weathering does not contain information on the susceptibility to future weathering or a time parameter indicating the time it has taken to arrive at the observed weathering state. The values associated to the degrees of weathering used in the 'initial point rating' system (ch. C.4) are studied guesses. The values for the SSPC system are determined in ch. D.1.5.

C.3.5.2 Method of excavation

The origin of an exposure has a considerable influence on quality of the rock mass and thus also on the stability of the slope (ch. B.3.4.14). The slope will show more mechanical discontinuities if high stress levels occurred in the rock mass during the natural development or excavation of a slope. Table 8 shows a division of excavation methods in use for underground excavations with quantitative values for the damaging influence of the method of excavation on the rock mass. Natural slopes have in general not been subjected to high stress levels and therefore show fewer mechanical discontinuities than an excavated slope in the same rock mass. The excavation of a rock mass by hand and by mechanical excavators causes mostly also a relatively small amount of damage[51] and thus fewer induced mechanical discontinuities.

technique	adjustment (%)
Boring	100
Smooth wall blasting	97
Good conventional blasting	94
Poor conventional blasting	80

Table 8. Adjustments for method of excavation (after Laubscher, 1990).

Blasting can cause severe damage of the rock mass (Hoek et al., 1980, 1988, Franklin, 1975b, Laubscher, 1990, Rosenbaum et al., 1994, Swindells, 1985, etc.). The shock wave from the detonation causes high stress levels leading to cracking of intact rock, the opening of integral discontinuities into mechanical discontinuities, and widening of existing mechanical discontinuities. Gasses discharged by the explosion cause similar effects, but giving more opening of existing discontinuities than cracking intact rock or opening of integral discontinuities. Additionally the roughness of discontinuity planes may be affected by blasting. Steps on discontinuity planes can be, partly or completely, sheared off or crushed due to the shock wave. Also rock blocks may be displaced, giving a non-fitting discontinuity plane resulting in a lower shear strength[52]. The effects from blasting are reduced if special techniques are used like pre-splitting or smooth wall blasting. Therefore a separate class for pre-splitting and smooth wall blasting is incorporated. The differentiation of blasted rock masses into the classes 'good' and 'poor' as proposed by Laubscher (1990) (Table 8) and Romana (1985, 1991) was thought to be too simple. Some more classes, recognisable in the field, have been included (Fig. 36, page 84).

(49) Processes included in weathering and the effects weathering has on a rock mass are described in chs. A.2.4 and C.3.2.2.

(50) BS 5930 (1981) is chosen as standard because it is widely known and properly described. However, a considerable amount of criticism is lately expressed about this characterization system. The possible replacement of the British Standard by a newer standard is therefore discussed in appendix V.

(51) In the research area no difference has been observed between natural exposures (e.g. made by scouring of a river) and exposures created by hand. The damage due to excavation with pneumatic hammers is, however, considerable (ch. D.1.4).

(52) Both effects were expected in the research area, but no evidence of these effects could be found. The condition of discontinuities (including roughness) was generally found to be independent of the type of excavation (ch. D.1.4).

Blasting techniques have changed over the years; the number of boreholes blasted in one round and the size (diameter and length) of the boreholes have increased. Also, where in the past blasting was directed towards creating gasses (slow detonation explosives), nowadays blasting is more directed towards creating a shock wave (fast detonation explosives). A rock mass will have most open discontinuities in the direction of the free face. A blast creating gasses will therefore work more in the direction of the free face than inwards into the rock mass. Shock waves work in all directions and therefore in more recent excavations, the rock mass is more damaged in the direction away from the free face than in older excavations. In this research old fashioned blasting (creating gasses rather than shock waves) has been incorporated in the class for pre-splitting and smooth wall blasting as the results are comparable.
In the 'initial point rating' system (ch. C.4) the different classes and the values are partly based on the work of Laubscher but modified as described above. The values for the additional classes are studied guesses. In the SSPC system (section D) the values for all classes for the method of excavation have been determined by analysis of the data obtained in the research area.

C.3.6 External influences

C.3.6.1 Surface run-off water

Water run-off[53] over a slope and through the near surface of a slope can lead to instability, but it is not proposed that surface run-off should be treated as a separate parameter in a classification system. All slopes used for referencing the classification system have been subject to rainfall and surface run-off water and thus the calculation method, parameters and weighting factors in the classification system include the influence of surface run-off water. For example, surface run-off water will have a larger influence on a slope in a rock mass with a small block size than with a larger block size because smaller blocks are more easily flushed away by the water. Block size (discontinuity spacing) is a parameter in the classification system and because the classification system is referenced against existing slopes, and existing stability, the weighting factors for discontinuity spacing incorporate the influence of surface run-off water.

C.3.6.2 Snow and ice

The influence of snow and ice in the weathering of a rock mass is discussed in ch. A.2.4. Snow and ice may also block seepage from the discontinuities where discontinuities are outcropping at the slope face which may lead to water pressures in the discontinuities. Additionally snow and ice add weight to a slope. Snow and ice do not commonly occur in the research area, however, during the fieldwork in 1992 it snowed, followed by the failure of some small slopes. It can therefore be assumed that the slopes have been occasionally subject to limited amounts of snow and ice characteristic for the Mediterranean climate. Hence, the classification system and weathering parameters incorporate the influence on slope stability caused by these limited quantities of snow and ice, because existing slopes and existing stability are used for calculating the weighting factors in the system. A separate parameter is thus not necessary for snow and ice.

C.3.6.3 Rock mass creep and stress relief

Rock mass creep and stress relief can lead to new cracks in intact rock, develop integral discontinuities into mechanical discontinuities and open existing discontinuities. These effects are included in weathering (ch. A.2.4). Creep movements and stress relief can also cause displacements along discontinuities, resulting in non-fitting discontinuity planes (ch. C.3.3.2.6). This is included in the characterization of the shear strength along discontinuities. Large movements of the rock mass in a slope may cause an increase in the slope dip angle leading to slope instability. In a classification of slope stability this can be incorporated by taking the slope dip angle that will exist due to rock mass creep and stress relief at the end of the engineering lifetime of the slope. For these reasons a separate parameter for rock mass creep and stress relief is not necessary in the classification system.

[53] The presence and pressure of water in discontinuities in the slope and the influence this has on slope stability and how it can be implemented in a slope stability classification is already discussed in ch. C.3.3.7.

C.3.6.4 External stresses

External stresses working on the rock mass in which a slope is or will be excavated can make a slope unstable. External stresses do not originate in the rock mass of the slope, but are, for example, stresses due to a high hill or mountain behind the slope or tectonic stresses. Generally, it is impossible to determine external stresses without stress measurements and their influence on the stability of a slope can mostly be only quantified with detailed numerical or analytical calculations. Therefore external stress influence cannot be included in a classification system[54] and consequently the classification system developed in this research cannot be used for slopes in rock masses that are under influence of external stresses.

C.3.6.5 Vegetation

The engineering lifetime, for example 50 years, of a slope is more than sufficient to allow some types of trees to develop to full growth. Root wedging will dislodge blocks, allow water infiltration, etc.. The prevention of such growth falls within the province of slope maintenance, which is not dealt with in this research.

C.3.7 Summary - parameters in rock slope stability

The review of parameters important in rock slope stability and to be included in a classification system for rock slope stability results in the following conclusions:

Intact rock strength:
Intact rock strength in the classification system for slope stability can be established with a 'simple means' test in the field. A cut-off value should be used above which the influence of intact rock strength on the calculation of the stability of a slope is constant.
Susceptibility to weathering:
In the 'initial point rating' system (ch. C.4) susceptibility to weathering is incorporated by estimating the time it takes for a rock mass to go one degree down in weathering according to BS 5930 (1981). In the SSPC system (section D) the expected degree of weathering at the end of the engineering lifetime is estimated.
Discontinuity shear strength:
Roughness of discontinuity walls, alteration of discontinuity walls, type of infill material, and the occurrence of karst are described in classes that can be established by visual observation of outcropping discontinuities.
Determining discontinuity properties and parameters:
Discontinuities within an outcrop and within a geotechnical unit should first visually be grouped into sets. Discontinuities with characteristic or mean properties (e.g. orientation, spacing, and properties describing the shear strength of each set) should be selected whereafter these can be measured at an easy accessible location. Single parameters describing the overall discontinuity spacing and condition of a number of discontinuity sets in a rock mass are described in respectively chs. C.3.4.3 and C.3.4.4.
Exposure and slope specific parameters:
The degree of weathering and the method of excavation of an exposure and a slope are established and are used respectively to correct for local and future weathering, and to correct for the damage due to the method of excavation with which an exposure or slope has been made or is to be made.
External influences:
No parameters are used for external influences such as surface run-off water on a slope face, snow and ice influences, rock mass creep and stress relief, external stresses, and vegetation.

[54] Most slopes in the research area are in a rock mass that is unlikely to be under influence of external stresses and those few slopes in a rock mass that might be under influence of external stresses have not been used for the development of the classification system.

C.4 INITIAL POINT RATING SYSTEM

The parameters and factors as outlined in chs. C.2 and C.3, have been used to design a point rating classification system for the classification of slope stability (ch. C.4.1). This 'initial point rating' system was used during the four years of collecting data. The weight factors in the first version of this system were mainly based on studied guesses from the author and values for weight factors found in the literature. Later the system has been improved and weight factors were modified to give ratings that better correlated with the visually estimated stability of the slopes in the fieldwork area. The system has, however, never given a really satisfying classification of slope stability. The reasons are discussed in ch. C.4.3. The poor results of the point rating system led to the abandoning of the point rating concept and to the development of an altogether new concept for the design of a classification system which is described in section D. A brief outline of the 'initial point rating' system is given in this chapter to be able to understand the reasons why the concept of a point rating has been abandoned.

C.4.1 Concept of initial point rating system

The concept of the point rating classification system is based on the principle of three rock masses, e.g. an 'exposure', 'reference' and 'slope' rock mass. This is the main core of the concept. The values for the parameters determined in an exposure are converted into parameters for a 'reference' rock mass by correction of the parameters for the influence of weathering of the rock mass in the exposure and for the damage due to the method of excavation used to make the exposure. The parameters of this 'reference rock mass' are then the parameters for the rock mass before it has undergone the effects of weathering and of the excavation process. A reference rock mass rating is calculated to facilitate the determination of geotechnical units. Correction of the parameters of the 'reference' rock mass for future weathering and the method of excavation to be used for excavation of the new slope results in parameters of the 'slope' rock mass. These latter parameters with parameters for the geometry of the new slope and the presence of water result in the 'slope' rating. This 'slope' rating determines the stability of the slope. The flow diagram of the 'initial point rating' system is shown in Fig. 34 and the form used to describe and classify exposures is given in Fig. 36.

C.4.2 Results

The 'initial point rating' system with optimum weight factors has been used to calculate the point rating of all 250 slopes. The results are presented in Fig. 35. In Fig. 35 the range of possible point ratings is divided in ten intervals of 10 points and the number of slopes obtaining a point rating in each interval is shown for each class of visually estimated stability as percentage from the total number of slopes in each stability class[55]. Evidently the 'initial point rating' system gave a poor distinction between stable (visually estimated stability class 1) and unstable slopes (class 4 and 5). Slopes judged to be stable are often rated as unstable (< 50 points). The ratings from slopes assessed to be unstable in the future (visually estimated stability classes 2 and 3) do not show any distinction from unstable slopes at present (class 4 and 5).

[55] The visually estimated stability classes are described in Table 5 (page 52).

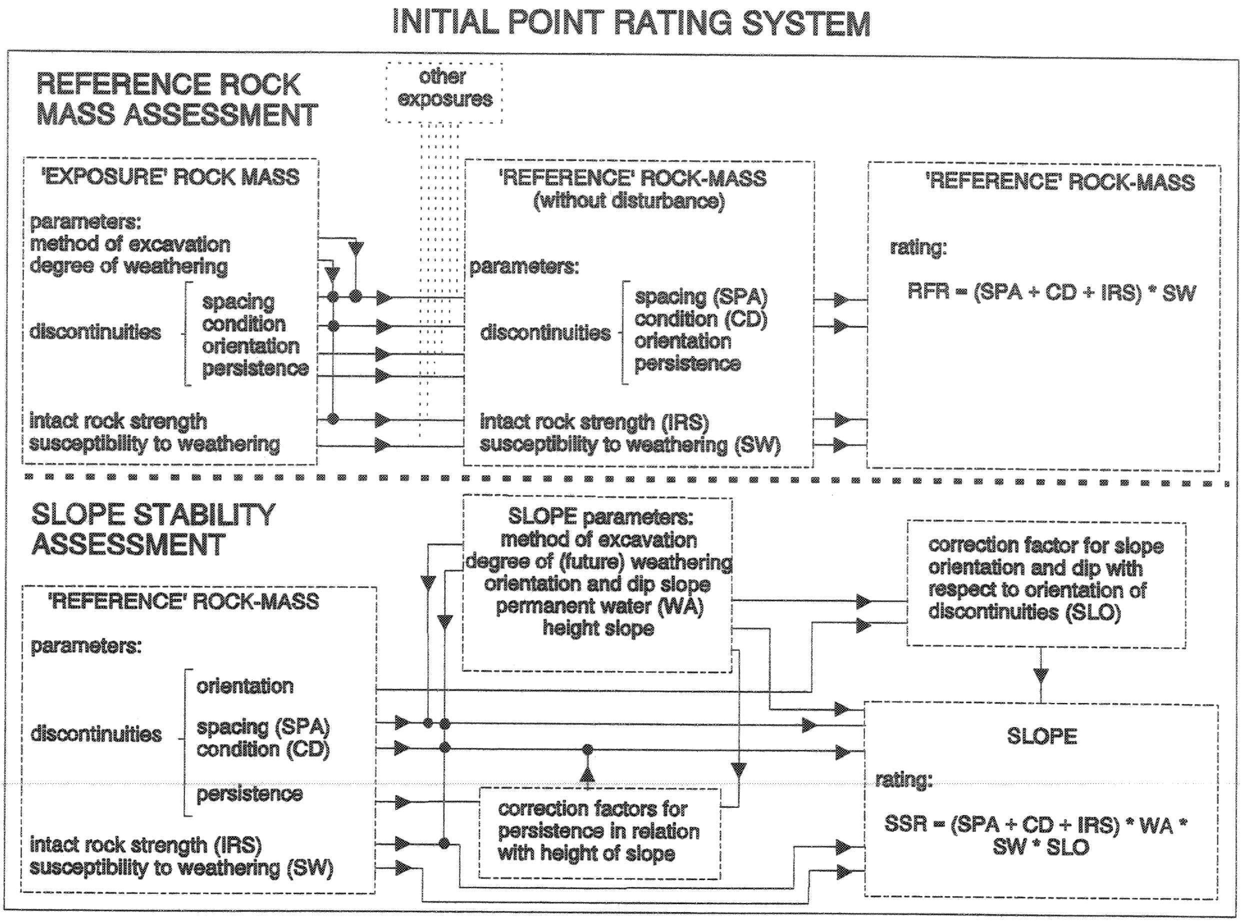


Fig. 34. Flow diagram of the concept of the 'initial point rating' system.

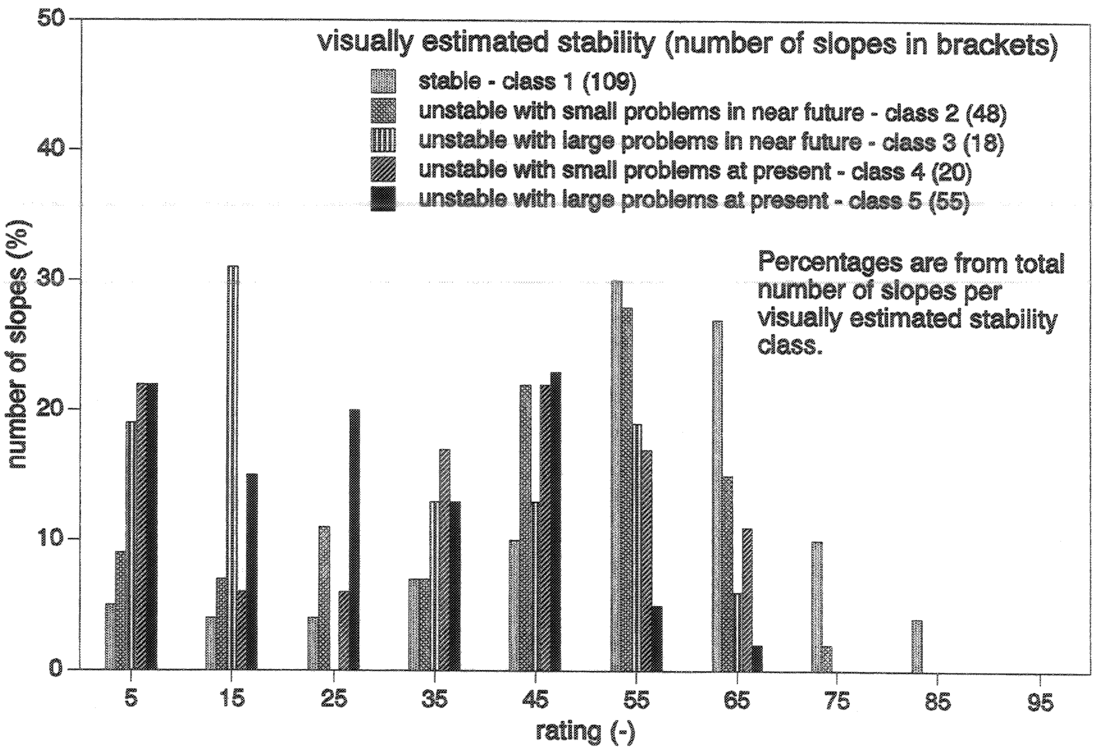


Fig. 35. Results of 'initial point rating' system with optimum weight factors based on 250 slopes (Definition of visually estimated stability classes - Table 5, page 52).

C.4.3 Discussion

The poor results of the 'initial point rating' system can be contributed to:

Adding or multiplying parameters without considering the type of failure related to the parameter
In a point rating system the rock mass parameters and parameters describing the slope (geometry, etc.) are expressed in a number and are added or multiplied. The resulting single numerical value, e.g. the slope rating, should reflect the slope stability. This inherently results in some illogical effects. Consider a slope that is unstable due to sliding over a discontinuity filled with a thick clay layer. In such a slope the spacing of discontinuities is of none or minor importance for the stability of the slope. The spacing of discontinuities is, however, a parameter included in the calculation of the slope rating and hence will influence the slope rating(56). This leads to the conclusion that different failure mechanisms should be rated independently and that only those parameters should be used in the rating for a particular failure mechanism that have a direct influence on the failure mechanism.

Severity of the failure
Expressing the stability of a slope in a single value does for many slopes not express the severity of the failure. This is also shown in Fig. 35 in which the ratings for visually estimated stability class 4 (small problems) and class 5 (large problems) slopes are similar. This occurs in particular if a single rock mass feature causes the instability. The influence on the final point rating value of rock mass and slope parameters and properties that are not related to the failure mechanism, causes that the final point rating does not reflect the severity of the failure. The severity of a failure can be properly included if different failure mechanisms are rated independently (see also above).

Measuring accuracy and variation in parameter and property values
No allowance is made in the 'initial point rating' system for the accuracy of measurements and the variation of parameters and properties of the rock mass and slope.

Susceptibility to weathering & future instability
The estimation of the time span for susceptibility to weathering, as required in the 'initial point rating' system, and the assessment of future instability of a slope (visually estimated stability class 2 and 3 slopes) are very difficult. Already during the fieldwork it was doubted whether these were reliable as both were found to depend for a large part on the experience of the person making the estimation and assessment. In all attempts to improve the 'initial point rating' system it was found that the slopes assessed to be unstable in the future, were assigned ratings that were not distinctively different from the ratings obtained for other stability classes. This also suggests that estimating the time span for susceptibility to weathering or the assessment of future instability of a slope or both are not reliable. For these reasons the susceptibility to weathering is incorporated differently in the SSPC system (section D) than in the 'initial point rating' system and the slopes assessed to be unstable in the future (class 2 and 3 slopes) are not used for the development of the SSPC system.

C.4.4 Conclusion

The 'initial point rating' system did not lead to a satisfying assessment of the stability of the slopes in the research area. Therefore the concept of a point rating classification system for slope stability assessment has been abandoned. In section D a different approach for a slope stability assessment system is designed.

(56) The same effect is also present in some of the existing classification systems for slopes as discussed in ch. B.3.1.

INITIAL POINT RATING SYSTEM - EXPOSURE CLASSIFICATION FORM			
LOGGED BY:	DATE: / /19	TIME: : hr	exposure no:

WEATHER CONDITIONS		LOCATION	map no:	
Sun:	cloudy/fair/bright	Map coordinates:	northing:	
Rain:	dry/drizzle/slight/heavy		easting:	

METHOD OF EXCAVATION (ME)	DIMENSIONS/ACCESSIBILITY			
(tick) hand-made : 0.99 unknown : 1.00 excavator : 0.97 natural : 1.00 boring : 1.00 ----------- BLASTING ----------- smooth wall blasting : 0.97 conventional blasting with result: good: 0.94 open joints: 0.80 dislodged blocks: 0.60 fractured intact rock: 0.40 crushed intact rock: 0.20	Size total exposure: (m)	l:	h:	d:
	mapped on this form: (m)	l:	h:	d:
	Accessibility:	poor/fair/good		

FORMATION NAME:

DESCRIPTION (BS 5930: 1981)					
colour	grain size	structure & texture (bedding thickness)	weathering	NAME	strength

INTACT ROCK STRENGTH (IRS)	sample number(s):
< 1.25 MPa : Crumbles in hand 1.25 - 5 MPa : Thin slabs break easily in hand 5 - 12.5 MPa : Thin slabs broken by heavy hand pressure 12.5 - 50 MPa : Lumps broken by light hammer blows 50 - 100 MPa : Lumps broken by heavy hammer blows 100 - 200 MPa : Lumps only chip by heavy hammer blows (Dull ringing sound) > 200 MPa : Rocks ring on hammer blows. Sparks fly	

DISCONTINUITIES B=bedding J=joint		.. 1	.. 2	.. 3	.. 4	.. 5
Dip direction (degrees)						
Dip (degrees)						
Discontinuity spacing (DS) (metres)						
persistence	along strike (meters)					
	along dip (meters)					
CONDITION OF DISCONTINUITIES						
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 about 0.2 x 0.2 m)	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					
Alteration of discontinuity walls (Al)	stronger :1.10 no change :1.00 weaker (only if<infill):0.80					
Infill material (Im)	no infill-surface staining :1.00 cemented/cemented infill :0.97 non softening & sheared material only, 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 opening: < 0.1[m] / 0.1-1 / >1 [m] occasional: 0.75 / 0.55 / 0.35 common: 0.25 / 0.15 / 0.05 frequently: 0.10 / 0.05 / 0.00					

WEATHERING (WE) (at observation point)
(tick) unweathered :1.00 slightly :0.93 moderately :0.87 highly :0.80 completely :0.74
SUSCEPTIBILITY (SW)TO WEATHERING
(tick) 1 year :0.05 5 year :0.25 10 year :0.50 50 year :0.90 > 50 year :1.00
sample no:
EXISTING SLOPE ?
dip-direction/dip /
height: (m)
permanent water (seepage) yes / no
Stability ?(tick) 1 stable 2 small problems in near future 3 large problems in near future 4 small problems 5 large problems

Fig. 36. Initial point rating - exposure classification form.

D SLOPE STABILITY PROBABILITY CLASSIFICATION - SSPC

D.1 THE DEVELOPMENT OF THE SSPC SYSTEM

The results of slope stability classification with the 'initial point rating' system were not very satisfactory. The reasons are discussed in the foregoing chapter. Therefore a new classification system for slope stability assessment has been developed taking into account the deficiencies of the 'initial point rating' system. The result is the Slope Stability Probability Classification (SSPC) system. The main concept of the classification system is outlined in ch. D.1.1. The rock mass parameters measured (intact rock strength, discontinuity properties, etc.) as described in ch. C.3, are analysed and some modifications were necessary. The newly designed classification system divides the failures possible in a slope into slope orientation dependent failures and slope orientation independent failures. The slope dependent failures are sliding, toppling and buckling and depend on the orientation of the discontinuities and the orientation of the slope. Failures of a slope that are not directly related to the presence of an unfavourably oriented discontinuity (set) are orientation independent failures. These failures are thus not related to the orientation of the slope. The new classification system is based on a probability assessment of the stability of a slope for the different possible failure mechanisms.

The analyses for the development of the SSPC system are presented as follows:

- Concept of the SSPC system (ch. D.1.1)
- Orientation dependent stability (ch. D.1.2):
  - 'Sliding criterion' (ch. D.1.2.1)
  - 'Toppling criterion' (ch. D.1.2.2)
  - Buckling (ch. D.1.2.3)
- Orientation independent stability (ch. D.1.3):
  - Linear model (ch. D.1.3.4)
  - Shear plane model (ch. D.1.3.5)
- The exposure and slope specific parameters (method of excavation and degree of rock mass weathering) are quantified and presented in chs. D.1.4 and D.1.5.
- The implementation of susceptibility to weathering is discussed in ch. D.1.6 based on the results obtained for the parameter for weathering.
- Statistical probability analyses of orientation dependent and independent stability are presented in ch. D.2.

The analyses in chs. D.1 and D.2 result in the SSPC system which is presented in ch. D.3. In ch. D.4 the results of the SSPC system are presented and the results of the SSPC system are compared to other existing rock mass classification systems. In the same chapter also the merits of the rock mass strength parameters calculated with the SSPC system are evaluated and compared to other methods to calculate rock mass strength parameters. Examples of the application of the SSPC system to four slopes in the research area are given in ch. D.5. In two examples the results of the SSPC system are also compared to analytical and numerical calculations of the stability of the slope. The limitations of the SSPC system and the conclusions follow in ch. D.6.

D.1.1 Concept

The concept for the development of the rock mass classification system for slope stability probability classification (SSPC) is based on:

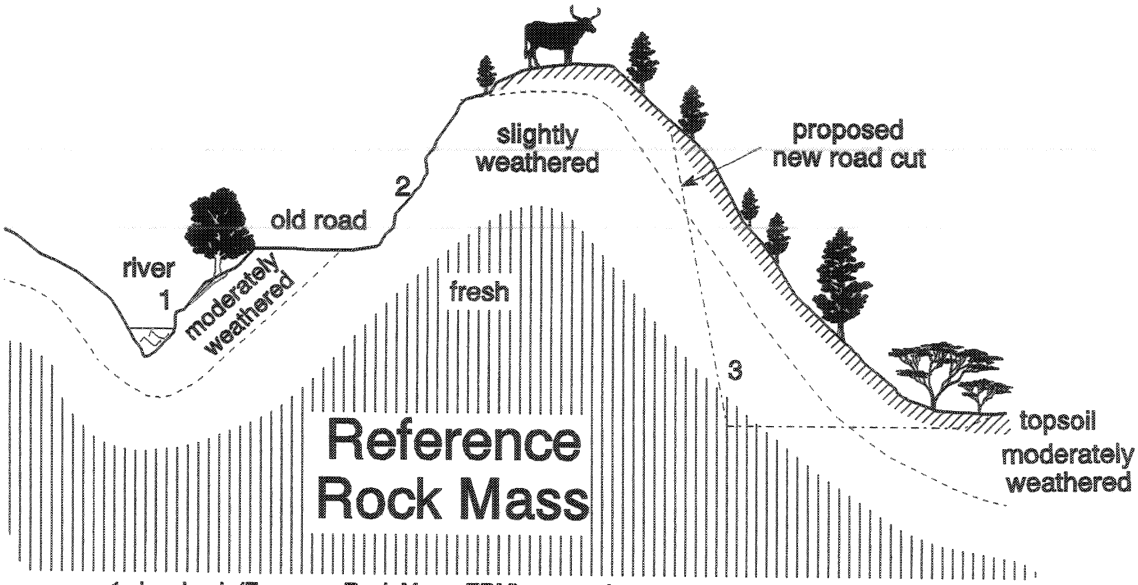
1. The introduction of the principle of a 'Reference Rock Mass' (RRM).
2. The development is not dependent on failure mechanisms recognized in the field.
3. The system leads to the assessment of a probability of slope stability (expressed as a percentage) instead of a rating.

D.1.1.1 'Reference Rock Mass'

The rock slope classification system developed considers three rock masses. These are:

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 new slope is to be made; the 'slope rock mass' (SRM).

The 'Reference Rock Mass'.
Rock mass parameters of importance are described and characterized in an exposure resulting in the 'exposure rock mass' (ERM). Local influences on the parameters measured in the exposure such as weathering and the disturbance due to the excavation method used to make the exposure, are then compensated for in order to convert the parameters for the 'exposure rock mass' to that of the theoretical rock mass that exists below the influence zones of weathering (thus fresh) and other disturbances: the 'reference rock mass' (RRM). This compensation is done with the aid of correction parameters: the exposure specific parameters. The resulting rock mass parameters are those of the 'Reference Rock Mass'. By this technique parameters that, in the same geotechnical unit, show different degrees of weathering and different degrees of excavation disturbance are brought back to parameters reflecting their original basic geotechnical properties. Fig. 37 shows exposures with various degrees of weathering and excavated by different means. Fig. 38 shows a flow diagram and the parameters that are of importance in slope stability with their relation to exposure, reference and slope rock mass.



1 river bank (Exposure Rock Mass, ERM) - natural exposure, moderately weathered
2 old road (Exposure Rock Mass, ERM) - made by excavator, slightly weathered
3 new road (Slope Rock Mass SRM) - to be made by blasting, moderately, slightly weathered and fresh rock mass (not exposed)

Fig. 37. Sketch of exposures with various degrees of weathering, different types of excavation and showing the concept of the 'reference rock mass'.

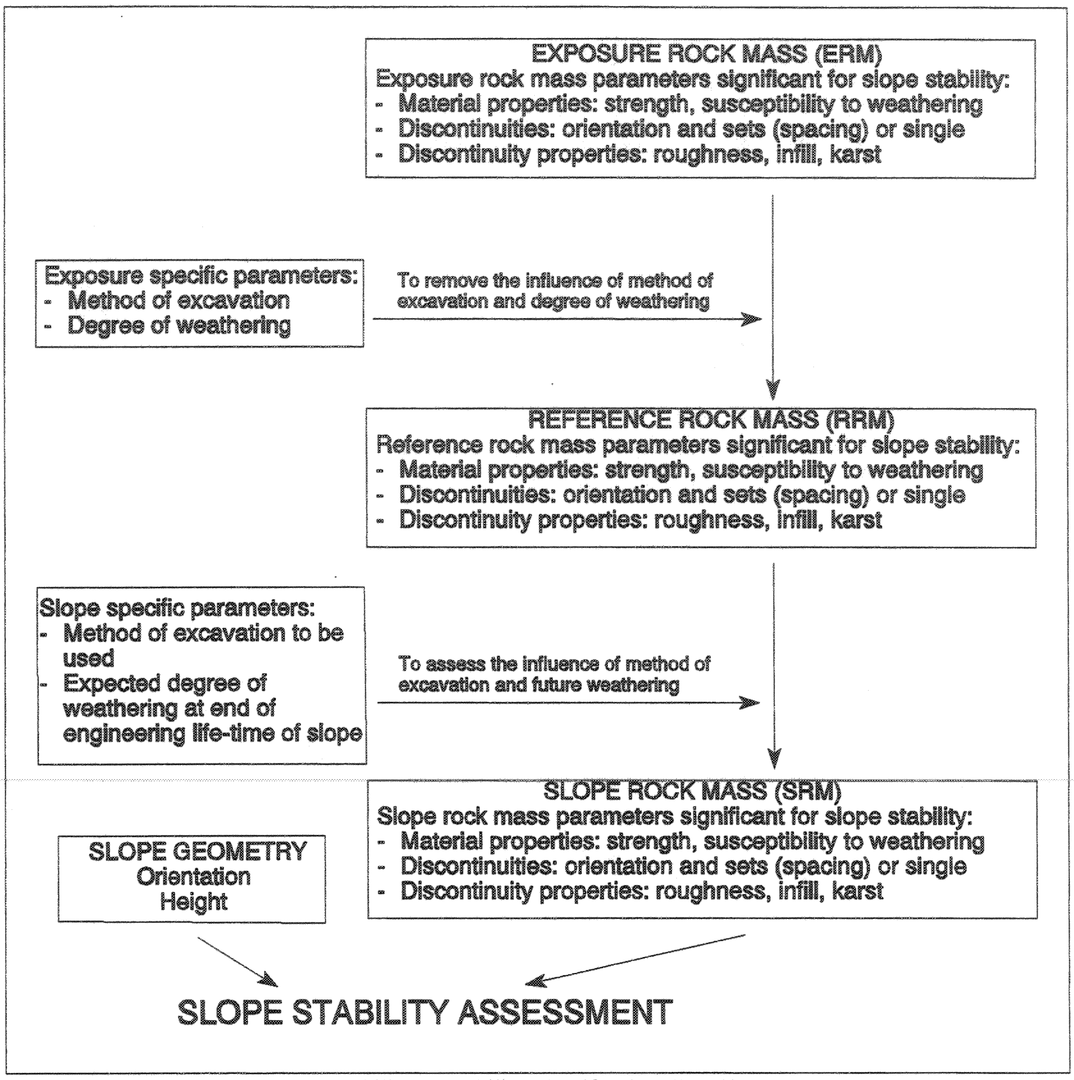


Fig. 38. Parameters in the slope stability probability classification (SSPC).

The stability assessment of a slope in the 'slope rock mass'.
The actual stability assessment is made in the 'slope rock mass' (SRM). This is derived from the 'reference rock mass' (RRM) by correction of the parameters of the 'reference rock mass' with the slope specific parameters for the influence of future weathering within the engineering lifetime of the slope and for the influence of the method of excavation to be used[57].

D.1.1.2 Determination of parameters & weighting factors

Slope failure mechanisms such as shear displacement and the different resulting failure modes (plane sliding, wedge failure, partially toppling and buckling) are discontinuity related and are dependent on the orientations of slope and discontinuity. Slope failure mechanisms that are usually not related to the orientations of the slope and the discontinuities can also cause failure of a slope. Examples of these latter causes are: breaking of intact rock under influence of the stresses in the slope, and removal of slope surface material due to (surface) water and internal water (flow and pressure).

[57] The 'Exposure Rock Mass' and 'Slope Rock Mass' are the same if an existing slope is examined and future weathering is not considered. Then it also not necessary to use the exposure and slope specific parameters (method of excavation and weathering) as these are the same.
Engineering lifetime denotes the time for which an engineering structure is designed. Slopes are often designed for a lifetime of about 50 years.

Traditionally rock slope stability analyses are based on recognition of the failure mechanism and mode in the field followed by a particular (back) calculation. Although the mechanisms and modes causing slope instability are theoretically well defined, it is often difficult to recognize the operating mechanism(s) and mode(s) in the field. In many unstable slopes multiple mechanisms and modes are at work at the same moment or successively (Fig. 39 and Fig. 40). Not all of these may be easily recognisable or visible, and in case of successive mechanisms and modes, the moment the slope is examined may determine the failure mechanism and mode recognized. In this research stable and unstable slopes have, therefore, been analysed without regard to the cause of instability to avoid the problem of exactly identifying failure mechanisms and modes in the field. Only if the result of an analysis indicates a certain mechanism or mode then conclusions on the failure mechanism and mode are assigned to the result; for example, slope orientation dependent and independent failures. If this procedure had not been followed then not only the proper failure mechanisms and modes had to be identified but, for slopes with multiple mechanisms and modes at work, also the contribution of each mechanism or mode to the overall (in-)stability should have been quantified. Quantifying the failure mechanisms and modes would have required detailed quantitative knowledge of rock mass parameters, which, as described before, is difficult or impossible to obtain.

D.1.1.3 Mathematical modelling

Three different approaches are possible to establish the relative importance of (and weighting factors for) the different parameters in the slope stability assessment system.

First a neural network can be used that optimizes[58] the influence of each of the different parameters on the engineering structure, in this case slope stability. This method has been used for the development of a system for the assessment of the stability of underground excavations (Lee et al., 1992). In this research this method has not been followed because: 1) the relationships between the parameters and the slope stability are not perceived in detail and 2) bias, if present in the data set, will most likely not be detected with a neural network (overfitting)[59].

A second possibility is to optimize a function relating all possible rock mass and engineering parameters to the stability of the slopes. The inherent problem of this method is that the type of function is likely highly complicated and that the type of function must, a priori, be known. If not, then each possible type of function should be considered. This leads to a virtually infinite number of functions and consequently an infinite number of optimizations. A more fundamental problem is that optimization of a complicated, and most likely discrete and non-linear, function on data with a presumed large scatter is often impossible. The function will not converge to a minimum or the global minimum is not found due to the presence of a large number of local minima. The result of the optimization can be a fit on an arbitrarily clustered subset of the data. Which subset will be fitted will depend on the start values used in the optimization process. In a complicated function such a misfit may not be recognized. Computing time would also be extremely long for this type of mathematical modelling because the number of variables and the amount of data in this research is large.

A third alternative is to use a set of relatively simple relationships that can easily be perceived and understood, for example, graphically, to determine the influence of parameters on other parameters or on slope stability. Each relation is optimized individually. The advantage is that at all times a fairly good control is possible over the relationships and that 'strange' results caused by data bias, scatter of the data or errors are likely to be recognised immediately. This method has been used in this research. In some ways the method resembles the Rock Engineering Systems methodology (Hudson, 1992, ch. B.2.3.7) as also in this methodology interactions in-between parameters, and interactions between parameters and engineering structure are individually analysed.

A probabilistic approach using the Monte Carlo method (Hammersley et al., 1964), is applied in this analysis to quantify the reliability of the functions found and the sensitivity of the result for input errors (Gama, 1994, Muralha, 1991, Scavia et al., 1990).

(58) Optimization is the art of obtaining the best result under given circumstances (Rao, 1979).

(59) If an amount of the data inhibits a consequent error (bias), the neural network will adjust the factors until they fit the data, including the data with the bias. More generally: the individual characteristics and not the structure of the data set are fitted, leading to weighting factors that are data set specific. This is also denoted overfitting (see glossary, page 241).

Fig. 39. Different failure mechanisms making a single slope unstable. Over the whole slope relatively small sliding, toppling and transport of rock blocks during rain and a 'wedge' sliding failure in the middle.

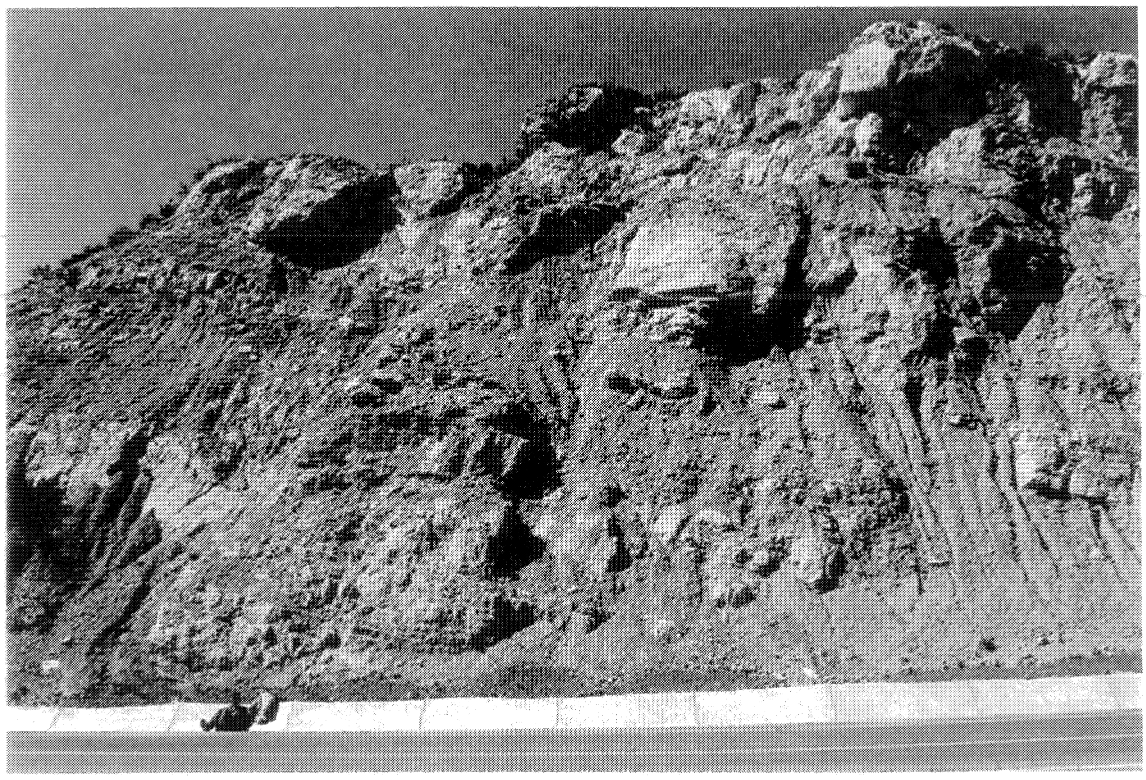


Fig. 40. Poor blasting, weathering and surface (rill) erosion making a single slope unstable. This slope is discussed in more detail as example IV in ch. D.5.4.

D.1.2 'Orientation dependent stability' - sliding, toppling and buckling criteria

Failures in a rock slope may depend on the orientation of the slope and the discontinuities. These are mostly related to shear displacement along a discontinuity. The main parameters governing this type of failure are the orientation of the discontinuity in relation to the orientation of the slope and the shear strength of the discontinuity. The parameters described in the field that are likely to have a relation with the shear strength of a discontinuity are the parameters describing the roughness of the discontinuity (*Rl* and *Rs*), the alteration of the discontinuity wall (*Al*), the infill material in the discontinuity (*Im*) and the presence of karst (*Ka*)[60]. This chapter investigates whether these parameters together with the orientation of the discontinuity can be related to failure modes of slopes due to shear displacement and whether this results in criteria that can be incorporated in the SSPC system. Three different modes of slope failure related to shear displacement along discontinuities are investigated: sliding, toppling and buckling.
The relationship found for failures related to 'sliding' are such that a 'sliding criterion' can be defined that relates the maximum dip of a discontinuity and parameters describing the condition of the discontinuity in the field (ch. D.1.2.1). This 'sliding criterion' has been verified with field and laboratory test values for discontinuity friction and with friction values for discontinuities found in the literature, which confirms that the 'sliding criterion' is properly defined. Analogous to the 'sliding criterion' a 'toppling criterion' is defined (ch. D.1.2.2). A similar criterion could not be developed for buckling. This is in agreement with field observations as buckling as a cause of slope failure is seldom found in the fieldwork area. Almost none of the slopes are high enough to cause buckling in the rock masses of the slopes (ch. D.1.2.3). The sliding and toppling criteria are incorporated in the SSPC system to predict the 'orientation dependent stability' of a slope (ch. D.1.2.4).

D.1.2.1 'Sliding criterion'

The 'sliding criterion'[61] relates the orientation of a discontinuity that allows kinematically sliding, to the parameters describing the condition of a discontinuity. The relation found in ch. D.1.2.1.1 is refined in ch. D.1.2.1.2 by examining different parameters and values used in the description of the condition of a discontinuity. The 'sliding criterion' with refinements of parameters is presented in ch. D.1.2.1.5.

D.1.2.1.1 Initial 'sliding criterion'

Failure in slopes related to sliding along a discontinuity means that the driving force along the discontinuity is larger than the restraining shear strength of the discontinuity. In the 'initial point rating' system the shear strength is described in the field with the parameter *TC*. *TC* is a multiplication of the parameters for the roughness of the discontinuity (*Rl* and *Rs*), alteration of discontinuity wall (*Al*), infill material in the discontinuity (*Im*), and the presence of karst along the discontinuity (*Ka*). The values used for the parameters are those included in the exposure characterization form of the 'initial point rating' system (Fig. 36, page 84). The driving forces in the direction of the slope dip are related to the (apparent) dip of the discontinuity in the direction of the slope dip. The larger the driving force is, the more likely it is that a block of rock laying on the discontinuity will slide out of the slope. The discontinuity dip in the direction of the slope dip (β) is defined as follows:

$$\begin{gathered} \textit{if:}\ \left|\delta\right| < 90° \\ \textit{then:}\ \beta = \arctan\left(\cos\delta * \tan dip_{discontinuity}\right) \end{gathered} \qquad [14]$$

$$\begin{gathered} \beta = \textit{apparent discontinuity dip in direction slope dip} \\ \delta = \textit{dip direction}_{slope} - \textit{dip direction}_{discontinuity} \end{gathered}$$

[60] Spacing of discontinuities was not expected to have an influence on the shear strength of a discontinuity, which was confirmed in this research as no influence of spacing on discontinuity shear strength was found. The influence of intact rock strength on the shear strength along a discontinuity (ch. C.3.2.1) is discussed in chs. D.1.2.1.1 and D.1.2.1.2, *alteration of discontinuity wall.*

[61] The 'sliding criterion' is published in Hack et al., 1995.

Fig. 41 shows the relation of the initial discontinuity condition parameter (*TC*) against β for 'day-lighting'[62] discontinuities in slopes that show no signs of present or future slope failures (stability class 1, Table 5, page 52)[63]. In Fig. 41 a vague relation is visible; fewer discontinuities plot in the lower right corner of the graph if the karst parameter is not included in the calculation of *TC*. For β between 30° and 80° it is possible, by visual examination, to draw (by hand) a boundary condition line below which only five discontinuity condition values for stable discontinuities in non-karstic rock masses are present. This boundary line is considered to be the 'sliding criterion'. In Fig. 41 many other boundary lines would have been possible, but the linear relationship between *TC* and β as indicated in Fig. 41, is the most simple possible boundary.

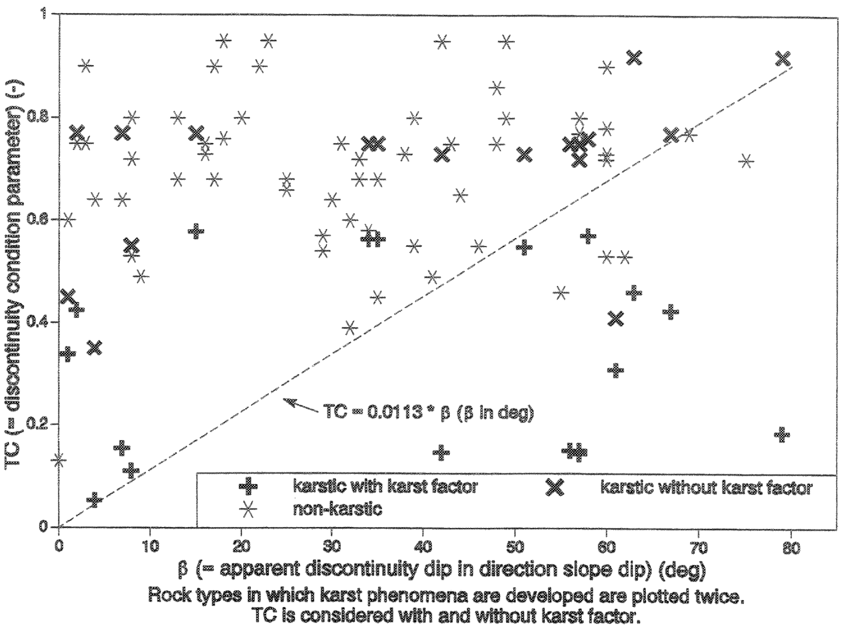


Fig. 41. Discontinuity condition parameter (*TC*) vs β, for 'day-lighting' discontinuities in stable slopes (stability class 1, Table 5, page 52).

Discussion

A significant number of discontinuities in karstic rock masses have a value for the *TC* parameter plotting below the boundary line in Fig. 41. It seems therefore that either including karst in the discontinuity condition parameter is not a proper approach or that the reduction of *TC* by the karst parameter is too strong. The discontinuities in karstic rock masses for which the *TC* parameter is calculated without the karst parameter, plot above the dashed line, except for one value[64]. Four of the five[65] values for non-karstic discontinuities plotting below the dashed line are cemented discontinuities in limestone (Tg21) (see below - *cemented/cemented infill*). Fig. 42 shows the initial discontinuity condition parameter (without considering the karst parameter in the calculation of the discontinuity condition parameter, *TC*) for different rock lithologies. The relation between *TC* and the apparent discontinuity dip in the direction of the slope dip does not show a dependency on the type of lithology.

(62) 'Day-lighting' of a discontinuity means that the discontinuity has a dip less than, but in the same direction as, the slope dip, and is outcropping in the slope (see also glossary, page 241).

(63) The accuracy of measuring dip and dip directions is such that the accuracy of dip and apparent dip values is not less than 5° (the accuracy of field measurements and derived data is discussed in more detail in ch. D.2.1), therefore only discontinuities are included for which applies that $dip_{slope} > \beta + 5°$. If the difference is less than 5° the dip_{slope} and β (apparent discontinuity dip) are assumed to be equal and the discontinuity plane forms the slope. The latter are obviously not a cause for slope instability due to sliding and cannot be used to determine a relation for sliding. Also are not included discontinuities whose apparent dip is almost vertical, e.g. discontinuities for which the apparent dip (β) > 84°.

(64) The karstic discontinuity at $\beta = 61°$ is a near vertical discontinuity with a dip of 85° (in slope 91/7/9.1/2; discontinuity orientation 078°/85°). For (near) vertical discontinuities the accuracy in measuring the orientation of discontinuity and slope becomes very important. Small inaccuracies will lead to large differences in the apparent dip. Therefore it is not unlikely that a small error in the measurement of the orientation causes this discontinuity to plot below the dashed line.

(65) At β = 55, 60, 62 and 69° for slopes respectively: 91/6/1/s3a, 91/6/1/s2 (2 x for two discontinuity sets) and 93/13/1. The fifth non-karstic discontinuity at $\beta = 75°$ has an apparent dip that is just over 5° less than the slope dip and is likely to be slope forming (slope 93/15/1; discontinuity orientation 128°/75° with slope orientation 142°/80° results in a difference between dip_{slope} and β of 5.4°).

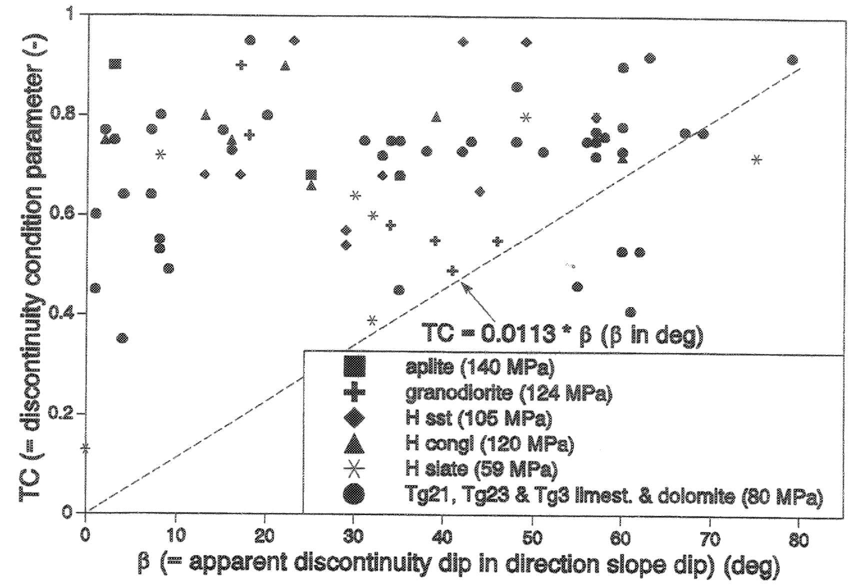


Fig. 42. *TC* without the karst parameter in the calculation of *TC* vs β for 'day-lighting' discontinuities in stable slopes for different rock materials (values in-between brackets are average estimated intact rock strength).

D.1.2.1.2 Refinement of initial sliding criterion

The 'sliding criterion' presented in ch. D.1.2.1.1 with parameters and their values of the 'initial point rating' system is refined as follows:

Infill: 'gouge > irregularities'
The value for the initial discontinuity condition parameter (*TC*) is calculated by multiplying, among others, the parameter values for small scale roughness (*Rs*) and infill material (*Im*). For discontinuities with an infill description of 'gouge > irregularities' it is, however, unlikely that the small scale roughness will be important for the shear strength along the discontinuity plane. Logically the calculation for this class of infill material should be changed to a small scale roughness parameter of 0.55 (minimum possible value).

Alteration of discontinuity wall (Al)
The parameter for the alteration of the discontinuity wall had been incorporated in the initial *TC* parameter to accommodate for possible shearing through discontinuity walls. It is, however, likely that this in reality is virtually never important. The weathering process causing weakening of the discontinuity walls is usually accompanied by the development of infill material in the discontinuity (ch. C.3.3.5). The shear strength of the infill material is normally considerably lower than the shear strength through weakened intact rock and consequently shearing will take place through the infill. The boundary condition line does not change if the *TC* parameter is calculated without considering the parameter for alteration of the discontinuity wall. Thus a parameter for the alteration of the discontinuity wall is not necessary for the 'sliding criterion' for the lithologies represented in the graph (for lithologies see Fig. 42).

Cemented/cemented infill
The contribution to the shear strength along the discontinuity of cement in the discontinuity or of cemented infill material will depend on the type of cement. Cemented infill in the research area has nearly been always of a calcitic type. In Fig. 41 the *TC* of the four cemented discontinuities[65] that plot below the 'sliding criterion', have been calculated with a value of 0.97 for the infill material parameter. If this value is changed to 1.07[66] then

[66] For the four slopes values of 1.07, 1.06, 1.02 and 1.01 are necessary for equilibrium.

these discontinuities will plot above the 'sliding criterion'(67). This leads to the conclusion that the value for infill material for the class 'cemented/cemented infill' has to be 1.07. Whether this value determined for a calcitic type of cement, is valid for other types of cement or cemented infill (e.g. quartz, salts, etc.) could not be determined. Many types of cement (e.g. quartz, etc.) are, however, so strong that shearing through the cement or cemented infill will not occur under the level of stresses occurring in slopes up to 40 or 50 m high. Cement types that consist of an easily dissolvable and deformable material, like salts or gypsum, are unlikely to be permanent during the engineering lifetime of a slope in most climates. Also their easily deformable character means that the discontinuity might move, not actually by shear but by deformation of the cement. Therefore it is a safer approach to characterize these latter types of cement as non-cemented infill.

Karst (Ka)
In retrospect it is clear that the karst parameter (*Ka*) should not be used to calculate *TC* in the form as applied in the 'initial point rating' system. The parameter for karst is, in the 'initial point rating' system, dependent on the frequency of the occurrence of karst along the discontinuity planes. For a 'sliding criterion' this is obviously not relevant as only a single discontinuity is enough to make the slope unstable. Moreover it can be questioned whether the parameter for karst should be dependent on the size of the solution holes along the discontinuities. Although karstic solution along discontinuities reduces the contact area between the two sides of a discontinuity, the normal stress on the contact area increases linearly with the reduction of the contact area and the shear strength resulting from friction remains the same(68). The contribution to the shear strength from the discontinuity cohesion reduces linearly with the reduction in contact area. Most discontinuities do not contain cement or cemented infill (causing real cohesion). This leaves discontinuities with an apparent cohesion that could have been influenced by karst. The discontinuities likely to show apparent cohesion are those with a small scale roughness of 'irregular/stepped'. Discontinuities in limestones do, however, seldom have a stepped surface but rather a plane or undulating surface for which the apparent cohesion is low or nonexistent. For these reasons it is likely that the influence of karst is considerably less than initially expected and the values have to be increased accordingly. Discontinuities with karst features in stable slopes will not plot below the 'sliding criterion' if the value for the karst parameter is fixed at 0.92(69) (independent of frequency of occurrence and independent from the size of the solution holes). The *TC* (condition of discontinuity) parameter should therefore be calculated including a parameter for karst along discontinuities that should have a fixed value of 0.92.

Persistence
All discontinuities in unstable slopes that are prone to sliding according to the 'sliding criterion' are persistent, but also other discontinuities in slopes in the research area are virtually always persistent. Non-persistent discontinuities or discontinuities that abut against other discontinuities are very seldom. Because of this the influence of non-persistence of discontinuities could not be investigated. It is suggested that non-persistent

(67) The influence of cement or cemented infill on the friction along a discontinuity as calculated with the 'sliding criterion' can be compared to the Q-system (Barton et al., 1990b). In the Q-system the difference in friction angle between a discontinuity with tightly healed, hard, non-softening, impermeable filling (i.e. quartz and epidote) and a discontinuity with unaltered joint walls, with surface staining only, is between 4° and 7°. The first value is for a rough undulating surface and the second is for a polished planar surface (roughness descriptions refer to small and intermediate scale roughness in the Q-system, Fig. A 97, footnote 147). In the 'sliding criterion' the difference between a discontinuity with cement or cemented infill and a discontinuity with no infill is between 4° and 2.5°, if a value of 1.07 is used for the class 'cemented/cemented infill'. The first value is for a straight (large scale roughness) rough undulating (small scale roughness) surface and the second for a straight polished planar surface. Thus, the value of 1.07 in the 'sliding criterion' results in a good correlation with the Q-system for the rough discontinuity surfaces but less for more smooth surfaces.

(68) The shear strength could increase if, due to the larger stresses, the friction parameters change. This effect can occur, by example, for a weathered discontinuity wall where, due to the larger stresses, the penetration of asperities into the weathered zone reaches less weathered material resulting in higher friction angles. For pure limestones, however, no weathering of the discontinuity wall material has been observed. This is different for limestones that also contain clay minerals because, as the limestone dissolves, the clay minerals may stay behind as a coating on the discontinuity wall.

(69) Values for the karst parameter to obtain equilibrium for the karstic discontinuities which plot below the 'sliding criterion' in Fig. 41, are: 0.60, 0.74, 0.87, 0.88, 0.92, 0.89, 0.72, 0.90 and 0.90.

discontinuities should be treated as stepped discontinuity planes. The step has to shear before movement can occur, a mechanism comparable to the breaking of intact rock in non-persistent discontinuities(70).

Conclusions

The foregoing results in the following conclusions on the refinement of parameters and values for the condition of discontinuity parameter (*TC*):

1. If the infill material of a discontinuity is in class 'gouge > irregularities', the small scale roughness parameter should be 0.55 (minimum possible value).
2. A parameter for the alteration of the discontinuity wall is not necessary in a failure criterion for slope stability.
3. The value used for cemented discontinuities with bonding between the discontinuity walls, or for discontinuities containing cemented infill with bonding to both discontinuity walls should be 1.07 for discontinuities containing a calcitic type of cement. Values for other types of cement could not be established, however, the following approach is likely logical. If the type of cement is very strong the discontinuity should not be considered as a discontinuity in the classification system. If the discontinuity contains cemented infill from which the cement easily dissolves, the loose material that may remain after dissolving of the cement, should be accounted for as a non-cemented infill. Also if the cement or cemented infill easily deforms, the discontinuity should be regarded as a discontinuity containing a non-cemented infill material.
4. The value for the karst parameter should be 0.92, independent from the size and frequency of the karst phenomena.
5. It is suggested that non-persistent discontinuities should be treated as stepped discontinuity planes.

D.1.2.1.3 Correlation of the threshold friction values of the 'sliding criterion' to test and literature friction values

The 'sliding criterion' is based on the assumption that the friction angle along the discontinuity plane, is equal or larger than β (= apparent discontinuity dip in the direction of the slope dip). This allows for comparison of threshold friction values found with the 'sliding criterion' with test and literature values (Hack et al., 1995, appendix III). The correlation found between the threshold friction angles determined with the 'sliding criterion' and the friction angles obtained from testing or found in the literature confirm the correctness of the 'sliding criterion' and the discontinuity condition parameter (*TC*) describing the discontinuity shear strength.

D.1.2.1.4 Reliability of friction angle values based on 'sliding' criterion

The reliability of the 'sliding criterion' for estimating friction values along discontinuities from field descriptions can be perceived from a visual examination of the data and graphs(71). There are a total number of 155 characterizations of discontinuities that kinematically allow sliding from about 100 slope stability assessments. These have been carried out by different persons in different years and it can be assumed that a consistent operator bias is absent in the data set. The 'sliding criterion' as defined above, is based on 98 % of the data plotting above the line. In Fig. 43 two other criteria for sliding are indicated (at 95 %: upper dashed line and at 99 %: lower dashed line). The influence these changes have on the friction angle is marginal and gives a change of a few degrees only. The differences can safely be neglected for an empirical field classification system and they lie also

(70) This approach is comparable to the treatment of non-persistent discontinuities in the Q-system (Barton et al., 1990b). In the Q-system non-persistent discontinuities are treated as continuous discontinuities, but the parameter for joint roughness is taken higher. The friction values found by Barton for non-persistent discontinuities are approximately 4° (for a discontinuity with unaltered joint walls, surface staining only) to 8° (for a discontinuity with softening or low friction clay coating) higher than the friction found for rough undulating but persistent discontinuities. In the 'sliding criterion' a straight (large scale roughness) non-persistent (which is thus classified as having a small scale roughness of 'rough stepped/irregular') discontinuity without infill has a friction angle about 10° higher than a straight rough undulating persistent discontinuity, while a straight non-persistent discontinuity filled with softening fine material has a friction angle about 6° higher than a straight rough undulating persistent discontinuity filled with the same material.

(71) It is also possible to perceive the reliability from the probability analysis in ch. D.2.2.

within the measuring accuracy of, for example, a shear test.

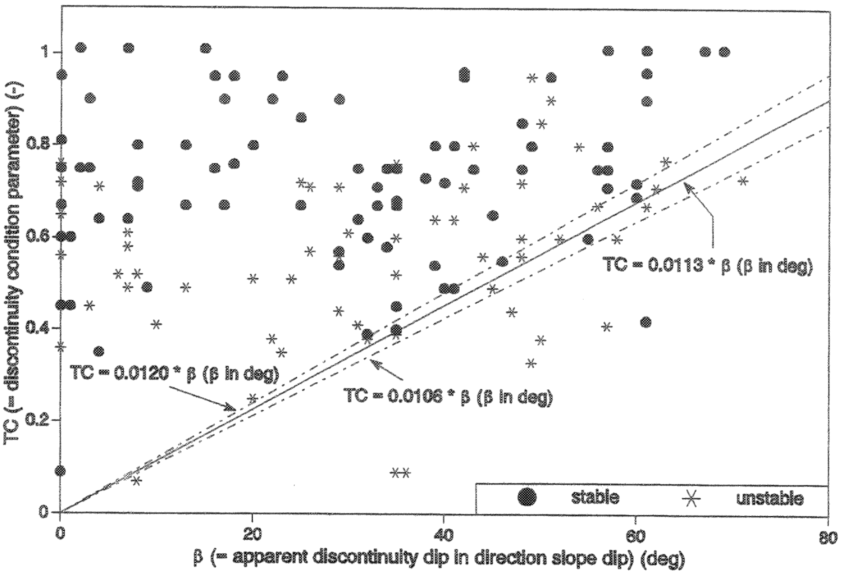


Fig. 43. Discontinuity condition parameter (*TC*) vs β for 'day-lighting' discontinuities in stable and unstable slopes (visually estimated stability class 1, 4 & 5).

The reliability of the 'sliding criterion' as an estimate for shear friction parameters is, however, dependent on the accuracy of the description of the discontinuity. During the research it was found that although different persons made the descriptions, these rarely differed more than one class. For example: instead of describing a surface as rough undulating it was described as smooth undulating. The difference in the friction angle is then 3° (rough undulating: 53°, smooth undulating: 50°; large scale roughness straight and no infill and karst). Obviously if for all parameters the class is consequently taken one lower, then the difference in friction value for the discontinuity becomes larger. This has, however, not been observed to happen, rather the differences were randomly a class lower or higher for the different parameters, which resulted in approximately the same results for *TC* values.

D.1.2.1.5 Discussion and conclusion

The correlation found between the friction angles determined with the 'sliding criterion' and the friction angles obtained from testing or found in the literature confirm the correctness of the 'sliding criterion' and the discontinuity condition parameter (*TC*) describing the discontinuity shear strength.

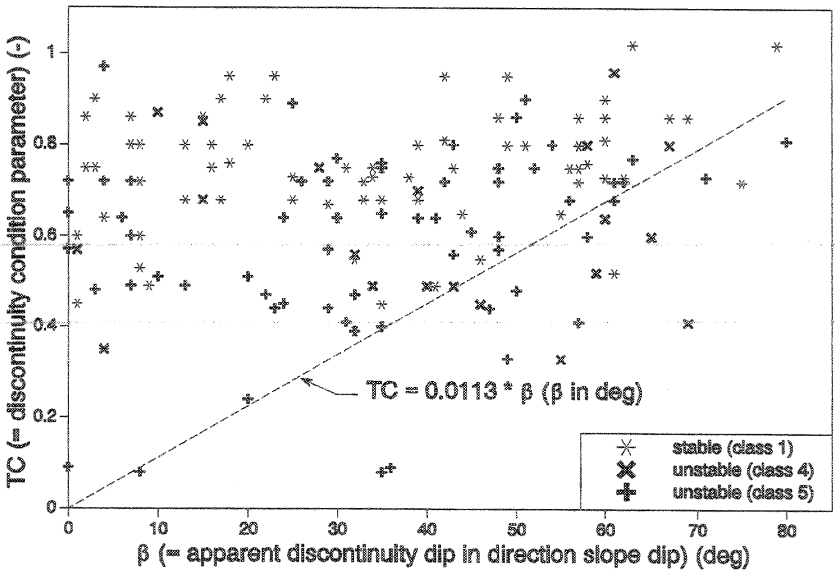


Fig. 44. Discontinuity condition parameter (*TC*) with refinements vs β for 'day-lighting' discontinuities in stable and unstable slopes (visually estimated stability classes 1, 4 & 5).

Fig. 44[72] shows *TC* versus β for 'day-lighting' discontinuities in both stable (class 1) and unstable (class 4 and 5) slopes. Some discontinuities from slopes with visually estimated stability 4 and 5, plot below the dashed line and it is likely that sliding is the cause of the instability of the slopes containing these discontinuities. The discontinuities in unstable slopes resulting in points that plot above the dashed line can, however, not be the cause of sliding instability in the slope and other causes (like toppling, buckling, etc.) have to be investigated for these slopes.

[72] The two discontinuities in stable slopes which plot below the 'sliding criterion' are discussed in footnotes 64 and 65.

The 'sliding criterion' is formulated as follows:

$$\begin{array}{l} \textit{sliding if:} \\ \qquad \left| \textit{dip direction}_{\textit{discontinuity}} - \textit{dip direction}_{\textit{slope}} \right| < 90° \\ \textit{and} \\ \qquad \textit{dip}_{\textit{slope}} > \beta + 5° \\ \textit{and} \\ \qquad \beta < 85° \\ \textit{and} \\ \qquad TC < 0.0113 * \beta \end{array} \qquad [15]$$

*TC = discontinuity condition parameter (= Rl * Rs * Im * Ka)*
β = apparent discontinuity dip in the direction of the slope dip

The parameter for the condition of the discontinuity (*TC*) is calculated with the refinements (ch. D.1.2.1.2). The 'sliding criterion' has been incorporated in the slope stability probability classification (SSPC) system to predict the 'orientation dependent stability' with respect to sliding along a discontinuity (ch. D.3). The exposure characterization form of the SSPC system (Fig. 71, page 145) includes the parameter and values for the discontinuity condition parameter (*TC*) with the refinements.

D.1.2.2 'Toppling criterion'

The second slope failure mode related to shear displacement and depending on the orientation of the slope and the discontinuities is toppling. Toppling of rock blocks on a slope can occur if the height of the blocks is larger than the width of the blocks or if columns of blocks overturn. The block or column of blocks rotates while the base of the block or column remains fixed. Different types of toppling have been identified (Goodman et al., 1976, Hoek et al., 1981). Shear displacement along discontinuities together with deformation, breaking and crushing of the corners of a rock block, or flexure of the rock blocks or columns of blocks is necessary before toppling is possible. Shear displacement in the form of so-called 'interlayer slip', is important in all forms of toppling (ch. D.1.2.2.1). Investigated is whether the interlayer slip can be related to the parameters describing the condition of the discontinuity (ch. D.1.2.2.2). The result is a 'toppling criterion' that relates the orientation of the discontinuity and the slope with the parameter describing the condition of a discontinuity (*TC*) (ch. D.1.2.2.3).

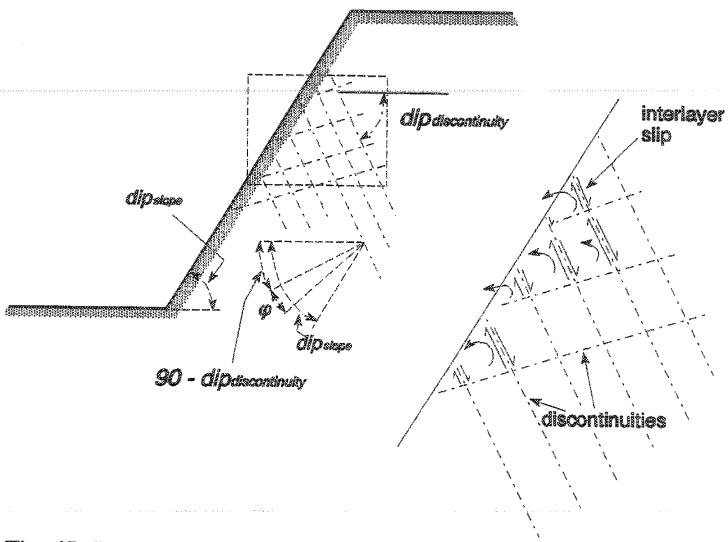


Fig. 45. Toppling. Blocks on the surface of the slope are pushed out due to the forces of the rotating blocks behind. Interlayer slip and deformation or crushing of block corners govern the rotation of the blocks (free after Goodman, 1989).

D.1.2.2.1 Interlayer slip and toppling

Fig. 45 shows toppling of rock blocks on a slope. The toppling is governed by a combination of interlayer slip (determined by the shear strength) along the steeply right dipping discontinuities and deformation or crushing of the corners of intact rock blocks due to rotation of the blocks. The toppling blocks push the smaller blocks directly underneath the slope surface out of the slope surface. Analytical methods to analyse toppling calculate for each block or series of blocks the force and rotational momentum equilibrium. In some methods also a dynamic aspect is taken into account to calculate displacements and consequent toppling possibilities (Hoek, 1981, Giani, 1992). The determination of the stress and deformation related to rotation and crushing of corners is usually not considered in analytical methods because the calculations are almost impossible. Numerical distinct computer programmes can incorporate the rotation and crushing in the toppling mechanism (UDEC, 1993, 3DEC, 1993, etc.), however, require detailed slope and rock mass parameters.

Toppling can be formulated following eq. [16] (Fig. 45) if only the friction component of the shear strength along a discontinuity is considered (Goodman, 1989). This holds only for discontinuities dipping into the slope, and not for vertical discontinuities nor for discontinuities dipping in the same direction as the slope[73].

$$\begin{aligned} \textit{toppling if:} \quad & dip_{slope} > (90° - dip_{discontinuity}) + \varphi \quad \rightarrow \\ & -90° + dip_{discontinuity} + dip_{slope} > \varphi \end{aligned} \qquad [16]$$

$$\varphi = \textit{friction along discontinuity plane}$$

Only if eq. [16] is satisfied toppling can occur. Equation [16] forecasts toppling before it usually happens in reality because rotational deformation and crushing are not considered. If the dip direction of the toppling plane is not approximately opposite to the dip direction of the slope then the blocks at the side of the block prone to toppling will prevent toppling. Different empirically established boundary conditions are defined in the literature. The boundary condition formulated by Goodman (1989) is formulated as follows:[74]

$$150° < \left|(dip\ direction_{slope} - dip\ direction_{discontinuity})\right| < 210° \qquad [17]$$

D.1.2.2.2 Discontinuity condition and toppling

An apparent dip of the discontinuity plane in the direction opposite to the dip direction of the slope can be formulated:

$$\begin{aligned} \gamma &= \textit{apparent discontinuity dip in direction opposite to the slope dip} = \\ & \arctan\left[\left|\cos(dip\ direction_{slope} - dip\ direction_{discontinuity})\right| * \tan(dip_{discontinuity})\right] \end{aligned} \qquad [18]$$

Fig. 46 shows the discontinuity condition parameter (*TC*) versus φ determined with eq. [16] for discontinuities in stable and unstable slopes[75]. The $dip_{discontinuity}$ in eq. [16] is replaced by γ following eq. [18]. The discontinuity condition parameter (*TC*) has been calculated with the refinements as for the 'sliding criterion' (ch. D.1.2.1.5). In Fig. 46 is indicated for all stable and unstable slopes whether the difference in dip direction between the slope and the discontinuity fulfil the boundary condition formulated in eq. [17]. Analogous to sliding, a boundary line, the 'toppling criterion', can be drawn below which no values plot[76]. For comparison also the 'sliding criterion' is shown. For a particular discontinuity surface type with a discontinuity condition parameter (*TC*), the φ found via the 'toppling criterion' is higher than the value found for the same type of surface via the 'sliding criterion'. Rotational and crushing effects likely cause this difference. Apart from one discontinuity[77], all values plotting below the 'toppling criterion' in Fig. 46 are within the boundaries set by eq. [17]. Therefore it is likely that a boundary on the dip directions of slope and discontinuity is not necessary if for the discontinuity dip γ (= the

(73) The only form of toppling discussed is that caused by stresses originating in the rock mass in which the slope is excavated or will be excavated. Other forms of toppling, for example, toppling of vertical blocks, may occur if additional external stresses work on the rock mass, however, these are not considered in this research (ch. C.3.6.4).

(74) In the literature also other lower and higher limits are reported, for example, 165° and 195°, or a differentiation in likelihood is used: for example, if the difference in directions is between 165° and 195° toppling is very likely whereas in the ranges between 150° - 165° and 195° - 210° toppling may happen (both under the condition that eq. [16] is satisfied). In general, it is likely that the boundary is not absolute but that a gradual boundary should be applied.

(75) Only included are discontinuities with $\gamma < 85°$ (discontinuities with $\gamma \geq 85°$ are assumed vertical and cannot enable toppling according to the criterion formulated in eq. [16], see footnote 73).

(76) Note that γ is the apparent dip of the discontinuity in the direction opposite to the direction of the slope dip; the value is always positive. The 'toppling criterion' in this chapter is formulated as $\varphi < -90° + \gamma + dip_{slope}$. This in contrary to the 'toppling criterion' formulated in ch. D.3.3 which is, more generally, defined in terms of apparent dip of the discontinuity plane (AP): $AP > 0°$ for planes dipping in the same direction as the direction of the slope dip and $AP < 0°$ for planes dipping in the direction opposite to the direction of the slope dip. The 'toppling criterion' is then: $\varphi < -90° - \gamma + dip_{slope}$.

(77) Slope: 91/10/1002; $(dip\ direction_{slope} - dip\ direction_{discontinuity}) = 213°$. This is just above the boundary condition of 210°.

apparent discontinuity dip opposite to the direction of the slope dip) is used.

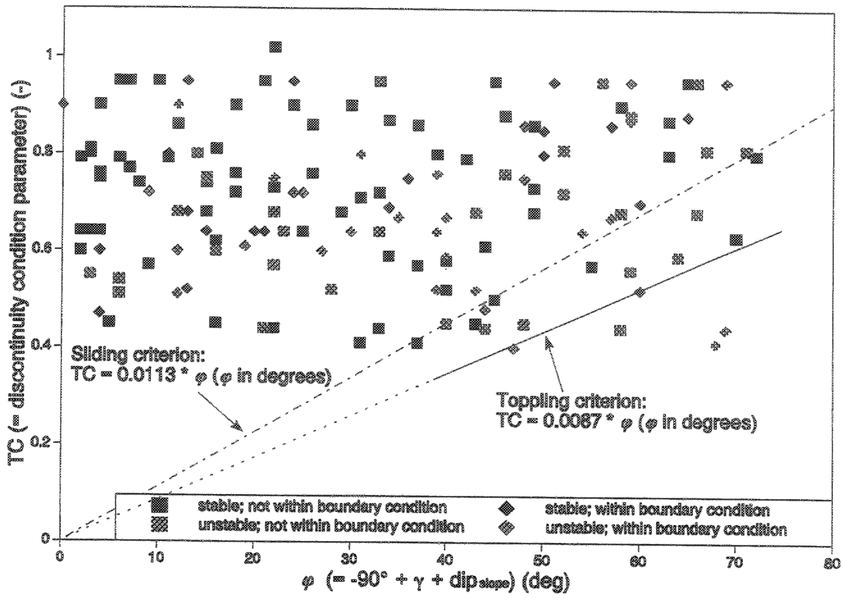


Fig. 46. *TC* vs φ for discontinuities dipping opposite slope dip direction in visually estimated stability class 1 (stable) and 4 & 5 slopes (unstable); boundary condition refers to eq. [17].

D.1.2.2.3 Conclusions

Following the 'toppling criterion' only a few slopes in the research area are failing at present due to toppling failure and the discontinuities in these slopes allowing toppling plot near to the 'toppling criterion' (Fig. 46). This is in agreement with field observations which showed only few slopes to be possibly unstable due to toppling.

The 'toppling criterion' relates the discontinuity condition parameter (*TC*) with the orientation of the discontinuity and the orientation of the slope and describes the possibility of slope failure due to toppling:

toppling occurs if:

$$\left| dip\ direction_{discontinuity} - dip\ direction_{slope} \right| > 90°$$

and

$$\gamma < 85° \qquad [19]$$

and

$$TC < 0.0087 * (-90° + \gamma + dip_{slope}) \quad (\gamma > 0°)$$

TC = discontinuity condition parameter
γ = apparent discontinuity dip in direction opposite the slope dip

The discontinuity condition parameter (*TC*) is calculated as for the 'sliding criterion' (ch. D.1.2.1.5). The 'toppling criterion' has been incorporated in the slope stability probability classification (SSPC) system to predict the 'orientation dependent stability' with respect to toppling along a discontinuity (ch. D.3).

D.1.2.3 'Buckling criterion'

The third failure type depending on orientation of slope and discontinuity investigated, is due to buckling of layers of rock that are parallel or near parallel to the slope face (Fig. 47). Buckling can only occur after shear displacement because layers move downwards to exhibit the force necessary for buckling of the layers at a lower position along the slope. This shear displacement is counteracted by the friction between the layers. Equation [20] (Giani, 1992) describes the equilibrium for flexural buckling under the following conditions: 1) the slab prone to buckling is taken to be a rectangular block of intact rock material that behaves elastically, and 2) the column axis is straight.

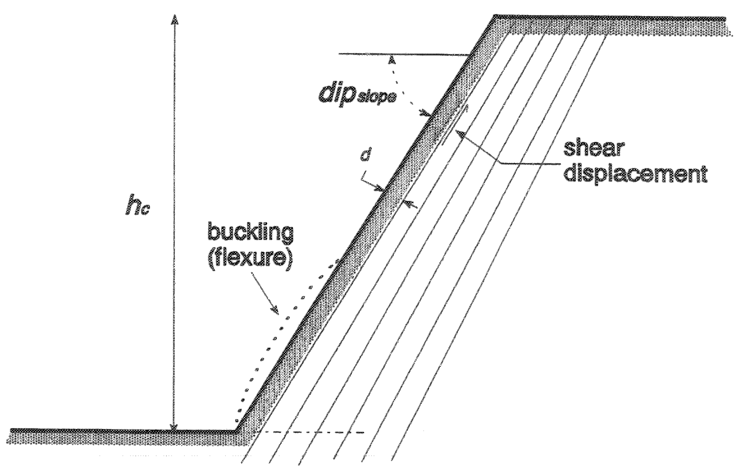


Fig. 47. Flexural buckling failure (layers flex under the load of the rock above) (free after Giani, 1992).

$$h_c^3 = \frac{\sin^3(dip_{slope}) * \pi^2 * E * d^2}{2.25 * \left(UW * \sin(dip_{slope}) - UW * \cos(dip_{slope}) * \tan\varphi_j - \frac{coh}{d}\right)} \qquad [20]$$

h_c = critical slope height d = thickness of layer prone to buckling
E = intact rock elastic modulus UW = unit weight rock material
coh = cohesion along discontinuity plane φ_j = friction along discontinuity plane

Equation [20] can be expanded to more complicated forms of buckling (three hinge or more beam models(78) for straight or curved slopes) but the assumptions necessary for the more complicated models are manifold and it becomes questionable whether the critical slope height resulting of more complicated models represents reality. Following eq. [20] the slope height would have to be about 100 m to create flexural buckling failure if the discontinuity spacing (d in eq. [20]) is about 0.1 m(79). Heights in the order of 100 m are more than the heights of the slopes in the research area and thus flexural buckling is unlikely. This agrees with field observations as slopes in the research area have not been noted to fail due to buckling. Buckling, however, has been observed to occur in very localized zones in slopes (generally zones of less than 1 m^2). In these zones cleavage planes in slates have become detached due to weathering, reducing the discontinuity spacing to about 1 mm, allowing localized buckling.
A 'buckling criterion' has not been defined or incorporated in the SSPC system because the slopes in the research area are not failing due to buckling. Also in other areas it is likely that buckling causes only failure if the slopes are higher than those for which the SSPC system has been developed.

D.1.2.4 Discussion and conclusions

The 'sliding criterion' and 'toppling criterion' are valid for all discontinuities that fulfil the kinematic requirements (e.g. 'day-lighting' for sliding and dipping opposite to the slope dip for toppling). Both criteria have been incorporated in the SSPC system for predicting 'orientation dependent stability' of a slope. The values of the angle of friction determined from the 'sliding criterion' are comparable to the result of laboratory and field tests and confirmed by friction angle values reported in the literature. Therefore the values determined from the 'sliding criterion' can be used to estimate friction angles for discontinuity planes.

(78) The boundaries of the hinges or beams are formed by discontinuities with strike parallel to the slope strike but dip opposite to the slope dip.

(79) This is in rock masses with $E_{intact\ rock}$ = 45 GPa, $UCS_{intact\ rock}$ = 100 MPa and $\varphi_j = 45°$, which are typical values for the rock types in the research area.

D.1.3 Orientation independent stability

The slope failures that could be attributed to discontinuity shear displacement and are dependent on the orientation of the slope and the discontinuities have been analysed in the previous chapters. It has been shown that a number of the investigated slopes are unstable following the criteria set in the foregoing chapters for orientation dependent stability, but a large number of the slopes are not unstable following these criteria. This chapter examines whether the rock mass parameters of the slopes in the research area that are not unstable due to the criteria for orientation dependent stability, show a correlation with the visually estimated stability of the slopes (ch. D.1.3.1). Parameters that are analysed do not depend on the orientation of a discontinuity nor depend on the orientation of the slope and hence slope failures due to a combination of these parameters have been named 'orientation independent stability'. Moreover the rock mass parameter data from the slopes in the research area are examined to see whether a mathematical model can be formulated to predict the 'orientation independent stability'. Two mathematical models are analysed: a linear model and a shear plane model (ch. D.1.3.2). The rock mass parameters in these models that depend on the overall spacing and condition of discontinuities of multiple discontinuity sets in the rock mass, can be calculated in different ways. Three different options have been selected for the spacing as well as for the condition of the discontinuities (ch. D.1.3.3). The linear model is optimized with all different options for the spacing and condition of the discontinuities (ch. D.1.3.4) and the results are used in optimizing the shear plane model (ch. D.1.3.5). The good capability of the shear plane model to predict the 'orientation independent stability' of a slope and, however less significant, the possibility to interpret the shear plane model as a physical model that describes the mechanical behaviour of the rock mass of the slope at failure, are the justification to use the shear plane model for the SSPC system for determining the 'orientation independent stability' of a slope (ch. D.1.3.6).

D.1.3.1 Correlation of rock mass parameters with visually estimated slope stability

An analysis of the rock mass parameters of the slopes that are not unstable following the orientation dependent stability criteria for sliding and toppling as discussed in ch. D.1.2[80], shows that there is a marked difference between stable and unstable slopes for the main parameters describing rock mass quality. Fig. 48 shows the frequency distributions of these parameters (e.g. intact rock strength - *irs*, spacing parameter[81] - spa_{mass}[82], and condition of discontinuities parameter[81] - con_{mass}[82]) for stable and unstable slopes. All three distributions show a shift from higher to lower values from stable slopes via unstable slopes class 4 to unstable slopes class 5. It is therefore likely that unstable slopes that are not unstable following the toppling or sliding criteria, are unstable due to a combination of the parameters for intact rock strength, spacing of the discontinuities and the condition of the discontinuities.

D.1.3.2 Models

In the previous chapter is shown that the rock mass parameters describing intact rock strength, spacing of discontinuities and the condition of the discontinuities, correlate with the visually estimated slope stability for 'orientation independent stability'. A mathematical relation between these rock mass parameters and the visually estimated stability is likely to be also dependent on slope dip and slope height:

(80) Only slopes have been used for the development of criteria for orientation independent stability with a probability to sliding or toppling instability following the sliding or toppling criteria of less than 5 % (for probability analyses see ch. D.2). Slopes assessed to be unstable in the future (class 2 and 3) are not used because the results of the 'initial point rating' system (ch. C.4.3) showed that the assessment of future instability may be not reliable. This results in a total of 141 slopes that are used for the development of orientation independent stability criteria, from which are 94 visually estimated to be stable (class 1), 10 to be unstable with small problems (class 4) and 37 unstable with large problems (class 5).

(81) In Fig. 48 the spacing parameter (spa_{mass}) is calculated following eq. [13], and the condition of discontinuities parameter (con_{mass}) following eq. [22].

(82) To avoid confusion with spacing and condition of a single discontinuity set, the characteristic value for the spacing and condition of a rock mass with one or more discontinuity sets are denoted by the subscript 'mass', e.g. spa_{mass} and con_{mass}.

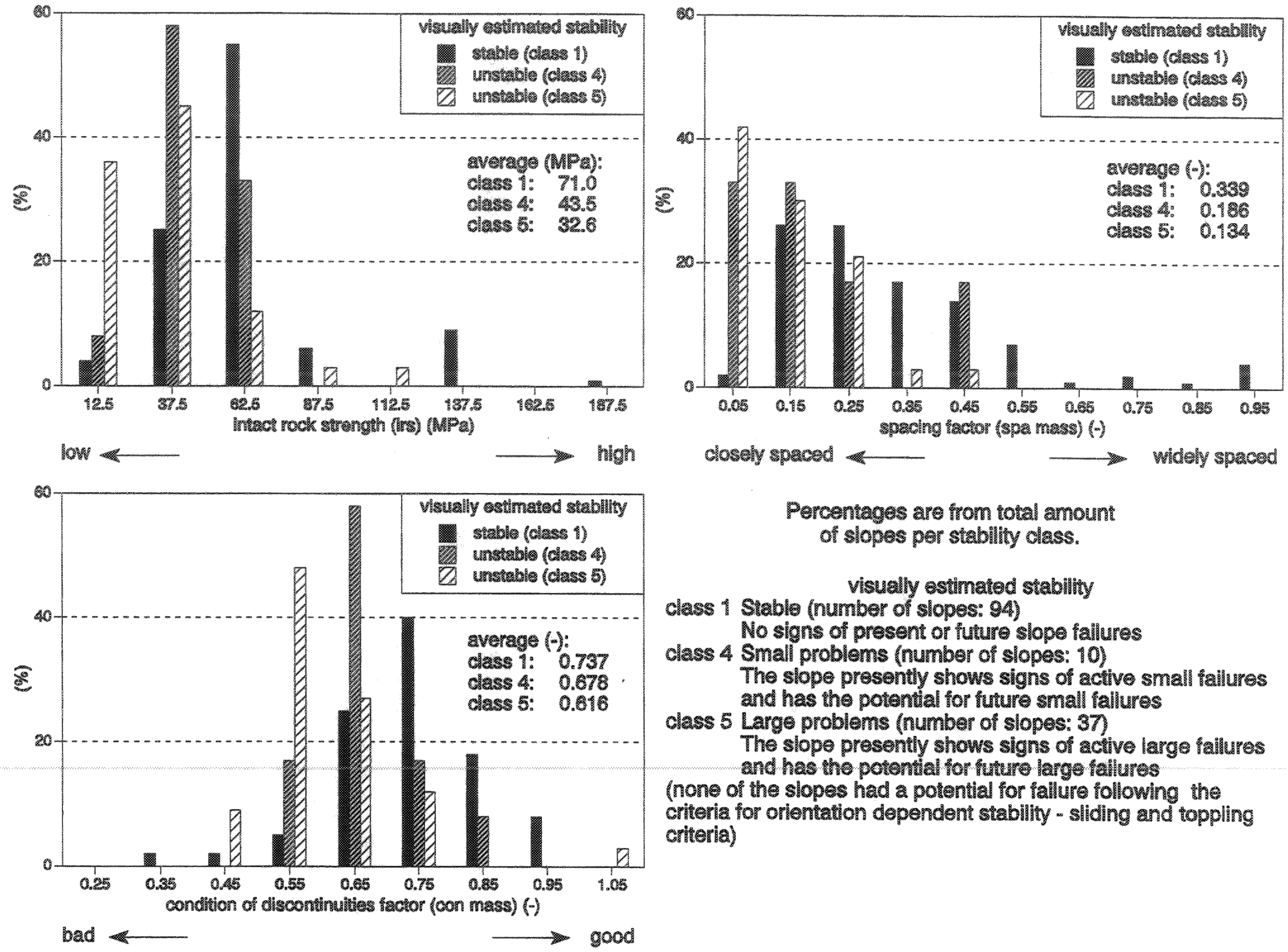


Fig. 48. Frequency distribution of irs, spa_{mass} and con_{mass}.

$$f\left(irs,\ spa_{mass},\ con_{mass},\ dip_{slope},\ height_{slope}\right) = stability$$

$$irs = intact\ rock\ strength \qquad [21]$$

$$spa_{mass},\ con_{mass} = the\ spacing\ respectively\ the\ condition\ of\ the\ discontinuities\ in\ the\ rock\ mass$$

$$height_{slope} = vertical\ height\ of\ the\ slope \qquad dip_{slope} = dip\ of\ the\ slope$$

Obviously the number of possible relations that could fit is large. Two relations have been tested: 1) a linear model and 2) a shear plane model (Fig. 50) such as used for slope stability calculations in soils.

D.1.3.3 Options for spacing of discontinuities (spa_{mass}) and condition of discontinuities (con_{mass})

Most rock mass classification systems consider only the spacing and condition parameter of the most prominent discontinuity set or the discontinuity set with the most adverse influence on the stability of an underground excavation or slope (ch. B.3.4). This is too simple for slopes, for failure is often not determined by one main discontinuity set but by more than one set. Multiple options exist to implement the spacing and the condition of discontinuities. Averaging or a form of weighting of the parameters for spacing and condition of discontinuities give a large number of possibilities so that a choice had to be made. Three options for the spacing parameter and three options for the condition of discontinuities parameter, leading to a total of nine different combinations, are analysed in the linear model to establish which fitted the data best. The different options are analysed according to the following rules.

Spacing of discontinuities value (spa_{mass}):

1 Minimum

spa_{mass} equals the spacing value of the discontinuity set with the smallest spacing in metre. The value is taken as 10 m if no discontinuity set is present in the rock mass of the slope.

2 Average

spa_{mass} equals the average of the spacing values (in metre) of all discontinuity sets present in the slope. The value is taken as 10 m if no discontinuity set is present in the rock mass of the slope.

3 Taylor

spa_{mass} equals the spacing parameter calculated following eq. [13] (page 76) and Fig. 33 (Taylor, 1980). The value is taken as 1.00 if no discontinuity set is present in the rock mass of the slope.

Condition of discontinuities value (con_{mass}):

1 Minimum

con_{mass} equals the condition parameter (*TC*) of the discontinuity set with the lowest condition value. The value is taken as 1.0165[83] if no discontinuity set is present in the rock mass of the slope.

2 Average

con_{mass} equals the average of the condition parameters (*TC*) of all discontinuity sets present in the slope. The value is taken as 1.0165[83] if no discontinuity set is present in the rock mass of the slope.

3 Weighted

If no discontinuity set is present in the rock mass of the slope the con_{mass} is taken as 1.0165[83]. If only one discontinuity set is present in the slope con_{mass} is taken as the condition parameter (*TC*) of that set. If more than one discontinuity set exists in the slope, the condition parameter (con_{mass}) is taken as the lower value of:

- the condition parameter (*TC*) of the discontinuity set with the lowest condition value, or
- the lowest value of the weighted mean values of the condition parameter (*TC*) of any two or three discontinuity sets present in the rock mass, weighted inversely against the spacing.

Thus con_{mass} may equal a value based on only one or two discontinuity set(s) even if the rock mass contains more than one or two discontinuity set(s). For three discontinuity sets the weighted mean value equals:

$$con_{mass}\ (condition\ of\ discontinuities) = \frac{\dfrac{condition_1}{spacing_1} + \dfrac{condition_2}{spacing_2} + \dfrac{condition_3}{spacing_3}}{\dfrac{1}{spacing_1} + \dfrac{1}{spacing_2} + \dfrac{1}{spacing_3}} \qquad [22]$$

The nine different combinations have only been analysed in the linear model because optimization times in the (non-linear) shear plane model would have resulted in an infeasible calculation time[84].

(83) 1.0165 is the maximum possible value of *TC*.

(84) The author does not think that this is a weakness in the analysis as the outcome of the analysis show that the most logical choices for spacing, e.g. Taylor, and condition, e.g. weighted, parameters are the best. Also the results of the whole SSPC system are so good that it is unlikely that these choices are erroneous.

D.1.3.4 Linear model

The linear model is calculated using:

$$stability = a0 + a1 * dip_{slope} + a2 * height_{slope} + a3 * irs + a4 * spa_{mass} + a5 * con_{mass}$$

$$\text{if } stability \leq 0.5 \rightarrow \text{slope stable (visually estimated class 1)}$$
$$\text{if } stability > 0.5 \rightarrow \text{slope unstable (visually estimated class 4 and 5)}$$

[23]

$a0$ through $a5$ = factors
dip_{slope}, $height_{slope}$ = slope dip, height
irs = intact rock strength spa_{mass} = spacing parameter
con_{mass} = condition of discontinuities parameter

The visually estimated stability (classes 1, 4 and 5) is an integer. To keep the model linear a distinction is impossible between classes 4 and 5 and the stability is only expressed as a value ≤ 0.5 (stable) or > 0.5 (unstable). Thus the stability of a slope is correctly calculated with the linear model if a slope that is visually estimated to be stable (class 1), obtains a stability value less than or equal 0.5 and a slope that is visually estimated to be unstable (class 4 or 5), obtains a stability value larger than 0.5. The $height_{slope}$ and dip_{slope} are the height and the dip of the slope as defined in ch. C.2.1. The linear model is calculated for each option for calculating the spacing and condition of discontinuity parameters (ch. D.1.3.3) and the percentages are determined of slope stabilities that are incorrectly calculated by the model (that is to say, those that conflict with the visually estimated stability)(80). The calculation is done by a Monte Carlo simulation with distributions on the parameters. This procedure is described in ch. D.2.3.1. Fig. 49 shows the percentages of incorrectly calculated stabilities of slopes for the three different options for the parameters for spacing (spa_{mass}) and for the three different options for the condition of discontinuities (con_{mass}). The factors and the standard errors obtained for the linear model for a spa_{mass} calculated following the Taylor method and a weighted con_{mass} are listed in Table 9(85).

factor	mean value	standard error
a0	0.784	0.076
a1	0.184	0.027
a2	0.021	0.0009
a3	-0.256	0.032
a4	-0.867	0.037
a5	-0.662	0.118

note: factors *a3* through *a5* are negative because the model (eq. [23]) becomes stable for smaller values and unstable for larger values of *a3* through *a5*.

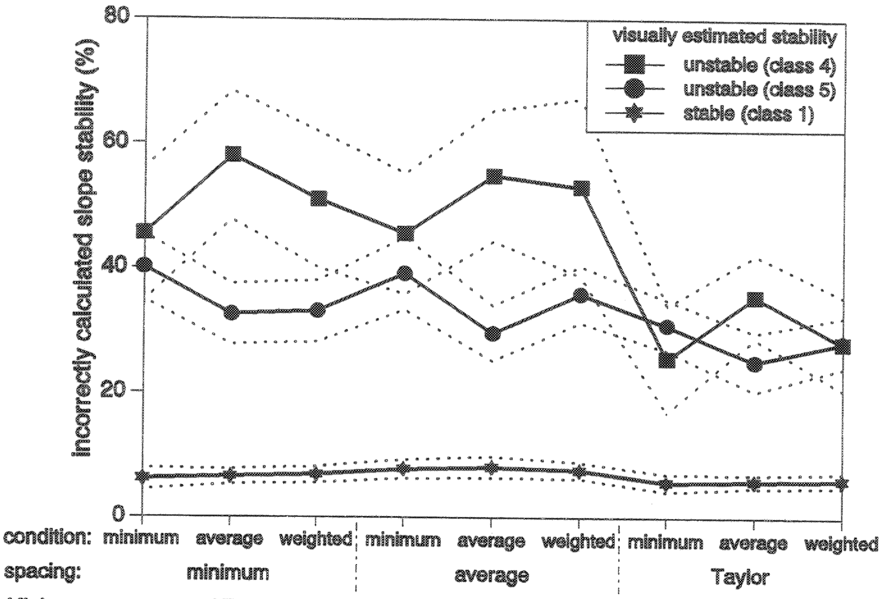
Table 9. Factors for linear model with spa_{mass} following Taylor and weighted con_{mass} (for calculation see ch. D.2.3.1).

D.1.3.4.1 Discussion and conclusions linear model

The lowest percentages for incorrectly calculated slope stability are clearly obtained if the Taylor calculation method is used for the spacing (spa_{mass}) parameter. The differences in percentages for incorrectly calculated slope stability obtained for the different options for the condition parameter (con_{mass}) are very small if the spa_{mass} is calculated following the Taylor method, and probably statistically not very significant. However, the lowest percentages for unstable slopes, class 5, are found if an average or a weighted condition parameter is used, whereas the lowest percentages for unstable slopes, class 4, are found if a minimum or weighted condition parameter is used. A weighted mean for the condition of a number of discontinuity sets in a rock mass is thus the best approach to calculate the condition of discontinuities in the linear model to predict 'orientation independent stability'. The methods of calculating spa_{mass} following Taylor and con_{mass} with a weighted mean are used in the optimization of the shear plane model in the following chapter(86).

(85) The results presented are for optimizations without weight factors to compensate for the difference in the numbers of stable and unstable slopes. This is because: 1) optimizations with weight factors showed only small differences with those without weight factors, 2) the difference in numbers of stable versus unstable slopes is only a factor 2 (94 stable slopes versus 47 unstable slopes; no differentiation is made between class 4 and class 5 slopes), and 3) introducing a weight factor also increases the influence of outliers on the optimization result.

(86) This approach of calculating spa_{mass} and con_{mass} also avoids the problems with the spacing and condition of discontinuities as included in some of the existing rock mass classification systems as discussed in chs. B.3.4.3 and B.3.4.5.



Minimum, average and Taylor or weighted refer to the method of calculation for spacing and condition of discontinuities parameters (as defined on the forgoing pages). Continuous lines are mean values and dashed lines are the standard error (the lines are plotted for easy understanding and have no meaning in between the data points).

Fig. 49. Percentages incorrectly calculated slope stabilities with linear model (for calculation of mean values and standard error see ch. D.2.3.1).

D.1.3.5 Shear plane model

The shear plane model can be applied as just a 'mathematical model' to predict the 'orientation independent stability' of a slope. The shear plane model may, however, also represent the physical mechanical mechanism in a slope at failure. A model that has a physical meaning would clearly be a more attractive model than just 'a mathematical model' as the linear model is in the previous chapter.

D.1.3.5.1 The shear plane model and its physical meaning

Failure in a rock slope that is not directly related to the orientation of a discontinuity, results in a new slope surface that is curved or linear or a combination of both. In the research area the slope surfaces resulting from this type of failure are often approximately linear, but not necessarily parallel to an existing discontinuity. Breaking of intact rock under the influence of gravity and of the stresses in the slope due to the weight of the rock mass, is a possible mechanism. The size and strength of intact rock blocks in relation to the stresses in the rock mass, govern this mechanism. Another mechanism, more common in the research area, which creates slopes with a linear failure plane, not parallel to a discontinuity, is observed in rock masses with a relatively small block size. In these rock masses small blocks tumble down the slope under the influence of a water flow over the surface of the slope and the water pressures in discontinuities near the surface of the slope[87]. Underlying mechanisms are small (compared with the slope height) scale sliding, toppling, buckling or wedge failure or combinations, under influence of water pressures caused by the flowing water over the surface. The process may be facilitated by wash-out of infill material from discontinuities making blocks unstable. The mechanism is most common in rock masses with many discontinuity sets with different orientations. This creates more possibilities for movement in different directions and resembles the possibilities of relative movement between individual grains in a soil.

The strength of a soil can be modelled following the 'Mohr-Coulomb failure criterion' by expressing the strength of the soil in terms of cohesion and friction between the individual grains in the soil. This model is similar to the

[87] This is not contrary to the conclusions about water pressures and the 'sliding criterion' in ch. D.1.7. The water in the discontinuities is likely only locally present and likely stems more from influx water from the slope surface than from the rock mass behind the slope.

model for intact rock strength approximated with the 'Mohr-Coulomb failure criterion' (ch. A.2.4). The failure mechanisms that cause orientation independent slope failure resemble for some slopes intact rock failure and for other slopes failures in a soil. This may allow the strength of a rock mass that fails through these orientation independent failure mechanisms to be approximated by a 'Mohr-Coulomb failure criterion'. Fig. 50 shows a slope in a rock mass following this criterion. Mathematically this is formulated as follows (Das, 1985):

$$\begin{aligned}
&\textit{for:}\quad dip_{slope} > \varphi_{mass} \;\rightarrow\; H_{max} = 4 * \frac{coh_{mass}}{UW} * \frac{\sin(dip_{slope}) * \cos(\varphi_{mass})}{1 - \cos(dip_{slope} - \varphi_{mass})}\\
&\textit{for:}\quad dip_{slope} \leq \varphi_{mass} \;\rightarrow\; H_{max} = \textit{unlimited}
\end{aligned} \qquad [24]$$

$$H_{max} = \textit{maximum possible height} \qquad UW = \textit{Unit Weight of rock mass}$$
$$coh_{mass},\ \varphi_{mass} = \textit{cohesion and friction angle of the rock mass}$$

The maximum possible height (H_{max}) of the slope in relation to the dip of the slope (dip_{slope}) is governed by the rock mass cohesion (coh_{mass})(88) and friction (φ_{mass})(88) if the slope dip is larger than the rock mass friction. The material above the slope plane following the 'Mohr-Coulomb failure criterion' (Fig. 50) will fail if the excavated slope height or dip is larger than permitted by this criterion. There is no maximum to the slope height if the rock mass friction is larger than the slope dip.

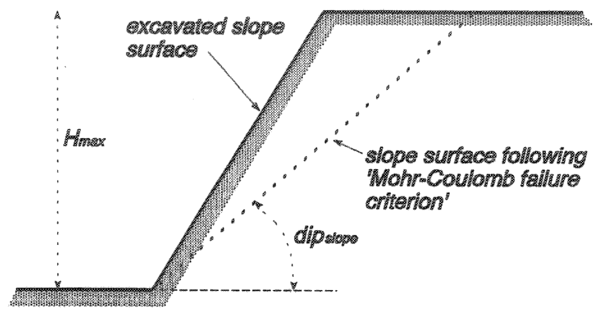


Fig. 50. Shear plane model for $dip_{slope} > \varphi_{mass}$.

D.1.3.5.2 Parameters in the shear plane model

φ_{mass} and coh_{mass} are assumed to be dependent on the rock mass parameters measured in the field, e.g. intact rock strength (irs), spacing of discontinuities (spa_{mass}) and condition of discontinuities (con_{mass}). In this research has been found that both φ_{mass} and coh_{mass} can be reasonably represented by a linear(89) combination of irs, spa_{mass} and con_{mass}. Chapter C.3.2.1 discusses the likelihood that the influence of the intact rock strength on slope stability is bounded by a maximum, i.e. a cut-off value. Linear relationships for φ_{mass} and coh_{mass} with a cut-off value for the intact rock strength (irs) result in the following:

$$\begin{aligned}
coh_{mass} &= w0 * irs + w1 * spa_{mass} + w2 * con_{mass}\\
\varphi_{mass} &= w3 * irs + w4 * spa_{mass} + w5 * con_{mass}
\end{aligned}$$

$$\begin{aligned}
&\textit{with cut-off value for irs:}\\
&\textit{if } irs \leq \textit{cut-off value} \rightarrow irs = \textit{intact rock strength (as measured in the field)}\\
&\textit{if } irs > \textit{cut-off value} \rightarrow irs = \textit{cut-off value}
\end{aligned} \qquad [25]$$

$$\textit{weight factors: } w0,\ w1,\ \ldots,\ w5 \geq 0$$

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

$$\varphi_{mass}\ (\textit{maximum}) = w3 * \textit{cut-off value} + w4 * 1.00 + w5 * 1.0165 \qquad [26]$$

φ_{mass} in eq. [25] 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

(88) To avoid confusion between friction and cohesion along discontinuities the friction and cohesion for the rock mass are denoted with respectively φ_{mass} and coh_{mass}.

(89) Not only linear relations between φ_{mass} and coh_{mass}, and irs, spa_{mass} and con_{mass} have been investigated. Also relations have been investigated of the following forms:

$$\varphi_{mass} = irs * e^{\frac{-w1}{spa_{mass} * con_{mass}}} \qquad \varphi_{mass} = irs * (spa_{mass})^{w1} * (con_{mass})^{w2}$$

The results are, however, not leading to better results than a linear combination.

optimization which one is necessary for the shear plane model. In the relations for φ_{mass} and coh_{mass} an order of magnitude difference exists of 100 between the values for *irs* and the values for spa_{mass} and con_{mass}. Therefore the intact rock strength (*irs*) in eq. [25] has been divided by 100 to reduce the difference.
The normalization of φ_{mass} and the division of *irs* by 100 combined with eqs [24] and [25] lead to a set of equations describing the shear plane model:

$$
\begin{gathered}
dip_{slope} \geq \varphi_{mass} \rightarrow H_{max} = 4 * \frac{coh_{mass}}{UW} * \frac{\sin(dip_{slope}) * \cos(\varphi_{mass})}{1 - \cos(dip_{slope} - \varphi_{mass})} \\
dip_{slope} < \varphi_{mass} \rightarrow H_{max} = unlimited \\
coh_{mass} = a0 * \frac{irs}{100} + a1 * spa_{mass} + a2 * con_{mass} \\
\varphi_{mass} = \left(\frac{a3 * \frac{irs}{100} + a4 * spa_{mass} + a5 * con_{mass}}{a3 * a6 + a4 + a5 * 1.0165} \right) * \frac{\pi}{2} \\
if \ \frac{irs}{100} \leq a6 \rightarrow irs = intact\ rock\ strength\ (in\ MPa) \\
if \ \frac{irs}{100} > a6 \rightarrow irs = a6 * 100
\end{gathered}
\qquad [27]
$$

a0 through a6 = factors dip_{slope} *= dip of slope* *irs = intact rock strength*
spa_{mass} *= spacing parameter* con_{mass} *= condition of discontinuities parameter*
H_{max} *= maximum possible slope height* *UW = Unit Weight of the rock mass*

In eq. [27] spa_{mass} is calculated following Taylor and con_{mass} is calculated with a weighted value as these are the calculation methods which gave the best results in the analysis of the linear model (ch. D.1.3.4). The value of the unit weight of the rock mass in eq. [24] is taken the same for all rock masses in the research area[90].

D.1.3.5.3 Optimization procedure for the shear plane model

In eq. [27] the values of the factors *a0* through *a6* are unknown. Equation [27] is optimized following the set of optimization rules in eq. [28] over the slopes with stability classes 1, 4 and 5[80] to find the values for the factors. In eq. [28] H_{slope} and dip_{slope} are the height and dip of the existing slope as defined in ch. C.2.1, and φ_{mass} and H_{max} are defined in eq. [27]. *ER* in eq. [28] is the value over which is optimized. *ER* equals the summation of er_j over all slopes used in the optimization. er_j is set to the value 1 if for a slope (*j*) the φ_{mass} and H_{max} are calculated following the shear plane model (eq. [27]), such that these imply a stability of the slope equal to the visually estimated stability in the field. If the result of the calculation is not in accordance with the visually estimated stability in the field er_j is set to a value, larger than 1, which reflects how much the calculated φ_{mass} or H_{max} is different from values that would result in a stability at equilibrium following the shear plane model calculated for slope *j*.

[90] Measured intact rock unit weights averaged per lithostratigraphic sub-unit, of the rocks in the research area are between 25.5 and 27.0 kN/m^3. The range is generally less than the scatter of repeated unit weight determinations within one sub-unit. Rock mass unit weight determinations have, for obvious reasons, not been done. However, the quantity of discontinuities, the generally small opening of the discontinuities and the fact that open and not filled discontinuities hardly exist, do not give reason to assume that the unit weight of the rock mass is considerably lower than the intact rock unit weight. Also the karstic rock units are not thought to have a rock mass unit weight considerably less than the intact rock unit weight. The actual value of the unit weight used in eq. [24] is not important as it will be optimized together with coh_{mass}.

For each slope j:

$$
\text{visually estimated stability} = \text{class } 1 \begin{cases} \dfrac{\varphi_{mass}}{dip_{slope}} \geq 1 \quad (stable) \quad \rightarrow er = 1 \\ \dfrac{\varphi_{mass}}{dip_{slope}} < 1 \begin{cases} \dfrac{H_{\max}}{H_{slope}} \geq 1 \ (stable) \quad \rightarrow er = 1 \\ \dfrac{H_{\max}}{H_{slope}} < 1 \ (unstable) \rightarrow er = \dfrac{H_{slope}}{H_{\max}} \end{cases} \end{cases}
$$

$$
\text{visually estimated stability} = \text{class } 4 \text{ or } 5 \begin{cases} \dfrac{\varphi_{mass}}{dip_{slope}} \geq 1 \quad (stable) \quad \rightarrow er = \dfrac{\varphi_{mass}}{dip_{slope}} \\ \dfrac{\varphi_{mass}}{dip_{slope}} < 1 \begin{cases} \dfrac{H_{\max}}{H_{slope}} \leq 1 \ (unstable) \rightarrow er = 1 \\ \dfrac{H_{\max}}{H_{slope}} > 1 \ (stable) \quad \rightarrow er = \dfrac{H_{\max}}{H_{slope}} \end{cases} \end{cases} \qquad [28]
$$

$$ER = \sum_j er_j$$

(visually estimated stability: clas 1 is stable, class 4 is unstable with small problems and class 5 is unstable with large problems; between brackets is indicated the stability calculated with the shear plane model)

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 is thus always a certain percentage of the slopes for which the calculated slope stability following the shear plane model is not equal to the visually estimated stability in the field. Hence, the value of *ER* is always larger than the total number of slopes used in the optimization. The goal of the optimization is therefore to minimize *ER*. The values for *a0* through *a6* in eq. [27] 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 maximal 2. The ratio of $\varphi_{mass}/dip_{slope}$ (for visually estimated unstable slopes) is also maximal 2. These limitations are necessary to avoid a too strong influence of possible outliers. In particular H_{max} becomes (extremely) large and influences the optimization very strongly for an outlier with φ_{mass} smaller than, but almost equal to, the slope dip.

The maximum possible height of the slope (H_{max}) is infinite if the slope dips less than the rock mass friction (φ_{mass}). As a consequence of this and of the use of a cut-off value for the intact rock strength, the function in eq. [27] is not continuous in the first derivative. Because of errors in the data (visually estimated stability, dip, height, intact rock strength, etc.) the function contains multiple minima. Optimization of a function that is not continuous in the first derivative and that also contains multiple minima, is difficult and it is often doubtful whether the absolute minimum can be found. The function is therefore examined graphically to find ranges for the factors in which the function is likely to minimize (decreasing *ER*). Then an optimization routine (Levenberg-Marquardt, Marquardt, 1963) is started with starting values for the factors within the ranges graphically determined. The procedure has been repeated multiple times(91). Multiple optimizations without the outliers(92) result in minima which are

(91) The order of magnitude of the factors is considerably different. *a0*, *a1* and *a2* are about 10 000 times larger than *a3* through *a6*. This difference could have influenced the optimization results and therefore an optimization with scaled factors *a0*, *a1* and *a2* has been done (e.g. coh_{mass} in eq. [28] is multiplied by 10 000 which results in *a0*, *a1* and *a2* to be divided by 10 000). The results are the same as with none scaled factors apart for the divider of 10 000. This implies that the optimization is not sensitive for this order of magnitude differences in the factors. The Levenberg-Marquardt routine used for the non-linear optimization which is part of the computer programme MathCad does not use scaling of the factors.

obtained for approximately the same values for the six factors. A graphical examination of the function with these values showed that these values were likely the best possible and represent the absolute minimum of the function. The values are used as starting values in the optimizations for the probability analyses, resulting in mean values and standard errors(93) for the factors and for the percentages of slopes with an incorrectly calculated stability (Table 10)(85).

D.1.3.5.4 Discussion of the shear plane model

An unexpected result of the optimization of the shear plane model is the high standard error for $a5$ (Table 10) which may imply that $a5$ is not significantly different from 0 and hence that the condition of discontinuities (con_{mass}) is not significant for the determination of the rock mass friction (φ_{mass}). This, however, conflicts with common sense because friction along discontinuities (expressed in the condition of discontinuities) should have an influence on the rock mass friction. An explanation could be that the description of the discontinuity condition in the field is not correct; it correlates, however, via the 'sliding criterion' with literature values as is shown in appendix III. For these reasons it was decided to maintain the condition of discontinuities in the equation to calculate the rock mass friction.

Fig. 51 shows H_{max}/H_{slope} versus $\varphi_{mass}/dip_{slope}$ for the slopes in the research area. H_{max} and φ_{mass} are calculated following eq. [27] with the mean values for the six factors as listed in Table 10. It should be noted that H_{max}/H_{slope} has no meaning for $\varphi_{mass}/dip_{slope} \geq 1$, because H_{max} is then infinite. Slopes with a $\varphi_{mass}/dip_{slope} \geq 1$ have been plotted in the graph at an arbitrary value for H_{max}/H_{slope} ($H_{max}/H_{slope} = 1$) to show that these plot in the area of the graph where slopes are stable following the shear plane model.

The slopes with visually estimated stability class 4, slopes being unstable with small problems thus with a stability likely almost at equilibrium, are also calculated to have a stability almost at equilibrium with the shear plane model (these plot near the boundary line between stable and unstable in Fig. 51). This supports the correctness of the model because in the optimization no differentiation is made between slopes with visually estimated stability classes 4 and 5, and thus also no 'a priori' knowledge is used to steer the optimization process in this direction.

D.1.3.6 Discussion and conclusions on 'orientation independent stability'

Percentages of slopes for which the visually estimated stability and the stability calculated with the linear or shear plane model did not lead to the same result, are slightly better for the shear plane model than for the linear model. The linear model resulted in incorrectly calculated slope stabilities of 6, 28 and 28 % while the shear plane model resulted in 8, 28 and 20 % (including outliers) for respectively stability class 1, 4 and 5. The shear plane model also correctly differentiates between slopes with small problems (visually estimated stability class 4) and slopes with large problems (visually estimated stability class 5). A further advantage of the shear plane model compared to the linear model is that the model may have a physical meaning(94), whereas a physical meaning is difficult

(92) Four slopes give in all optimizations a non-realistic result for the maximum height (H_{max}) or the friction (φ_{mass}) and are therefore considered to be outliers. The descriptions given of these slopes explain why these slopes should not be used. The four slopes are:

90/10/2.2	The slope is parallel to and very near to a major fault. The slope is situated in an associated shear zone area; the discontinuity orientations are irregular.
92/13/1401	The rock mass consists of (meta-) sandstones interbedded with slates. The instability is caused by the presence of slates but the rock mass characterization is done for sandstone.
92/18/lc	Doubt about the stability; by some observers classified as unstable (visually estimated class 5) by others as small problems in the near future (visually estimated class 2).
93/11s/11s	The slope is visually estimated to be stable but all calculations result in an absolutely unstable slope. The same slope was then characterized by additionally two other persons who measured considerably larger discontinuity spacings, that resulted in a stable slope following the shear plane model.

(93) Procedures for calculation of mean value and standard errors are discussed in ch. D.2.3.2.

(94) This allows for comparison of the rock mass 'strength' calculated with Bieniawski's RMR system and the 'modified Hoek-Brown strength criterion' with the rock mass 'strength' of the SSPC system (ch. D.4.2).

to perceive for the linear model. The shear plane offers therefore an appropriate method for the calculation of the 'orientation independent stability' and is used in the SSPC system. Using in eq. [27] the mean values for the factors, listed in Table 10, and simplifying eq. [27] results in:

$$
\begin{gathered}
coh_{mass}\ (in\ Pa) = irs * 94.27 + spa_{mass} * 28629 + con_{mass} * 3593 \\
\varphi_{mass}\ (in\ degrees) = irs * 0.2417 + spa_{mass} * 52.12 + con_{mass} * 5.779 \\
\\
if\ intact\ rock\ strength < 132\ MPa \rightarrow irs = intact\ rock\ strength\ (in\ MPa) \\
else\ irs = 132 \\
\\
If\ dip_{slope} \leq \varphi_{mass} \rightarrow maximum\ slope\ height\ (H_{max})\ is\ infinite \\
else\ the\ maximum\ slope\ height\ is\ determined\ by: \\
\\
H_{max} = 1.6 * 10^{-4} * coh_{mass} * \frac{\sin(dip_{slope}) * \cos(\varphi_{mass})}{1 - \cos(dip_{slope} - \varphi_{mass})}
\end{gathered}
\qquad [29]
$$

spa_{mass} is calculated following Taylor (eq. [13], page 76, Taylor, 1980) and con_{mass} is calculated with the option for a weighted con_{mass} following eq. [22] (page 104).

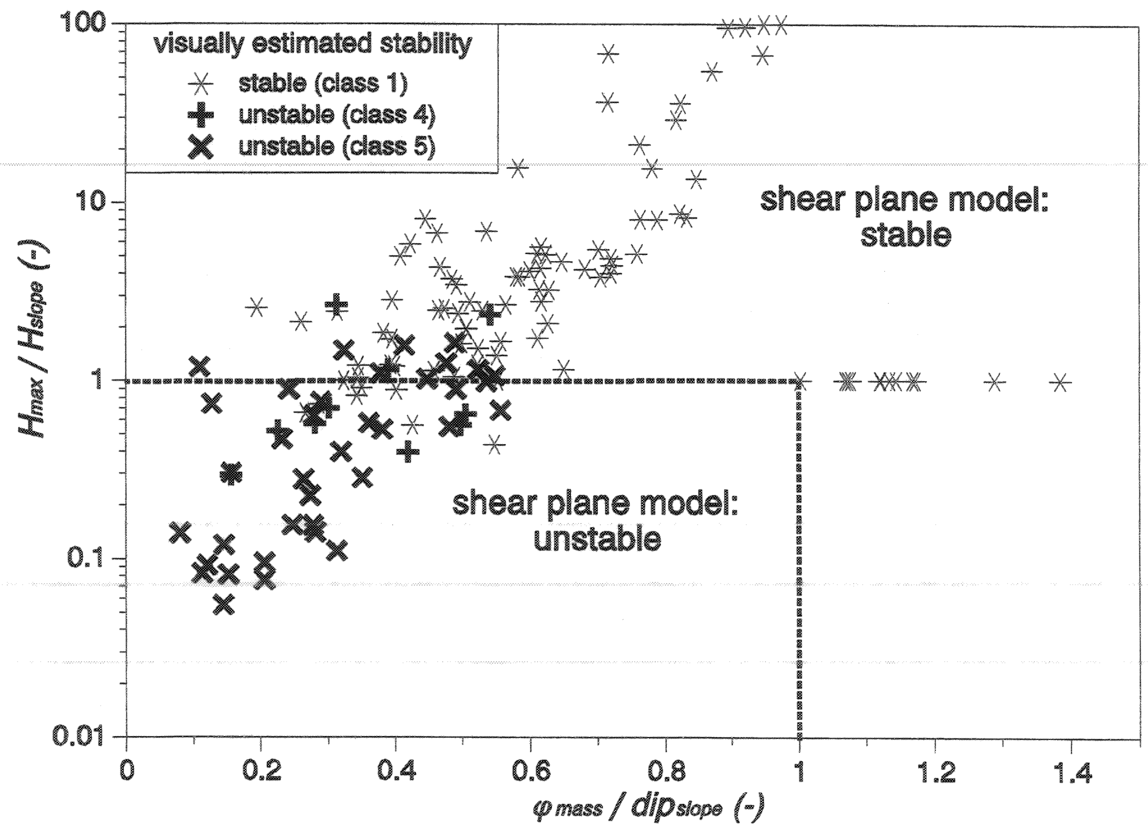


Fig. 51. H_{max}/H_{slope} vs $\varphi_{mass}/dip_{slope}$ (for the graph H_{max}/H_{slope} has a maximum value of 100, and H_{max}/H_{slope} = 1 for $\varphi_{mass}/dip_{slope} \geq 1$).

factors in the shear plane model		
factor	mean value [-]	standard error [-]
a0	9427	2458
a1	28629	5667
a2	3593	1083
a3	0.3150	0.09135
a4	0.6792	0.1006
a5	0.07531	0.04551
a6	1.325	0.3149
percentage incorrect calculated slope stability		
visually estimated stability	mean value [%]	standard error [%]
stable (class 1)	8	2
unstable (class 4)	28	8
unstable (class 5)	20	3

note: factors and percentages for optimization without outliers.

Table 10. Factors for the shear plane model and percentages of slopes with a calculated stability that conflict with the visually estimated stability (for calculation see ch. D.2.3.2).

D.1.4 Parameter for the method of excavation

A quantification of the influence of the method of excavation is necessary in the SSPC system to be able to exclude any influence of the method of excavation on the parameters measured in the exposure, and subsequently to accommodate for the influence of the proposed method of excavation on the rock mass in which the new slope is made (ch. D.1.1.1). The methods of excavation used in the research area and the geotechnical parameters influenced are evaluated in ch. D.1.4.1. The methodology to quantify this influence is presented in ch. D.1.4.2. 'A priori' existing interdependencies between the method of excavation and the geotechnical characteristics of the rock mass do not exist or can be neglected. This allows for a fairly straightforward calculation of the values for the parameter for the method of excavation by calculating ratios of the discontinuity spacing from different exposures excavated with different methods of excavation (ch. D.1.4.2.2). The reliability of the values and a comparison of the values with values found in the literature are evaluated (in respectively chs. D.1.4.3 and D.1.4.4).

D.1.4.1 Methods of excavation used for slopes in the research area and geotechnical parameters influenced by these methods

The different classes for the method of excavation used for field description of the slopes are listed on the exposure field characterization form for the 'initial point rating' system (Fig. 36 and Table 11). Exposures made by 'boring' techniques have not been found in the research area. Exposures in the research area made by excavator were done by a pneumatic hammer[95] mounted on a shovel frame. 'Hand-made' exposures have also not been found.
No influence of the method of excavation on the intact rock strength nor on the condition of discontinuities has been found. Only the spacing of discontinuities was found to be influenced by the method of excavation. Fig. 52 shows examples of the average of the discontinuity spacing per lithostratigraphic sub-unit and per type of discontinuity versus the method of excavation[96].

<table>
<tr><th colspan="3">METHOD OF EXCAVATION (ME)</th></tr>
<tr><td colspan="2">unknown</td><td>excavator</td></tr>
<tr><td colspan="2">natural</td><td>boring</td></tr>
<tr><td colspan="2">hand-made</td><td></td></tr>
<tr><td rowspan="6">blasting</td><td colspan="2">pre-splitting/smooth wall</td></tr>
<tr><td rowspan="5">conven-tional with the following result:</td><td>good</td></tr>
<tr><td>open discontinuities</td></tr>
<tr><td>dislodged blocks</td></tr>
<tr><td>fractured intact rock</td></tr>
<tr><td>crushed intact rock</td></tr>
</table>

Table 11. Initial classes for the method of excavation.

D.1.4.2 Influence of the method of excavation on the discontinuity spacing

In principle it is very simple to determine the relation between discontinuity spacing and the method of excavation. A comparison of the discontinuity spacing in exposures excavated with different methods of excavation to the discontinuity spacing in 'natural' exposures in the same lithostratigraphic sub-unit and with the same degree of rock mass weathering should give the required parameter (ME_j) (eq. [30]). The 'natural' exposures are assumed to be representative for the rock mass prior to excavation; thus without influence of the method of excavation.

$$ME_j = \frac{\textit{discontinuity spacing}_j}{\textit{discontinuity spacing}_{natural}} \qquad [30]$$

$$j = \textit{method of excavation}$$

[95] The steel rod of the hammer which had been mostly used, is approximately 2 m long with a diameter of about 0.15 m.

[96] In ch. D.1.3.6 is established that a shear plane model with a parameter for the overall spacing of a number of discontinuity sets in a rock mass (spa_{mass}) calculated according to Taylor (eq. [13], page 76, Taylor, 1980) is to be used in the SSPC system. Therefore the parameter for the method of excavation is determined for spa_{mass} calculated according to Taylor. Fig. 52 shows, however, the average spacing per discontinuity type to show that the dependency between spacing and method of excavation applies to all types of discontinuities without major differences. spa_{mass} calculated according to Taylor shows the same trend, but necessarily irrespective of the type of discontinuity.

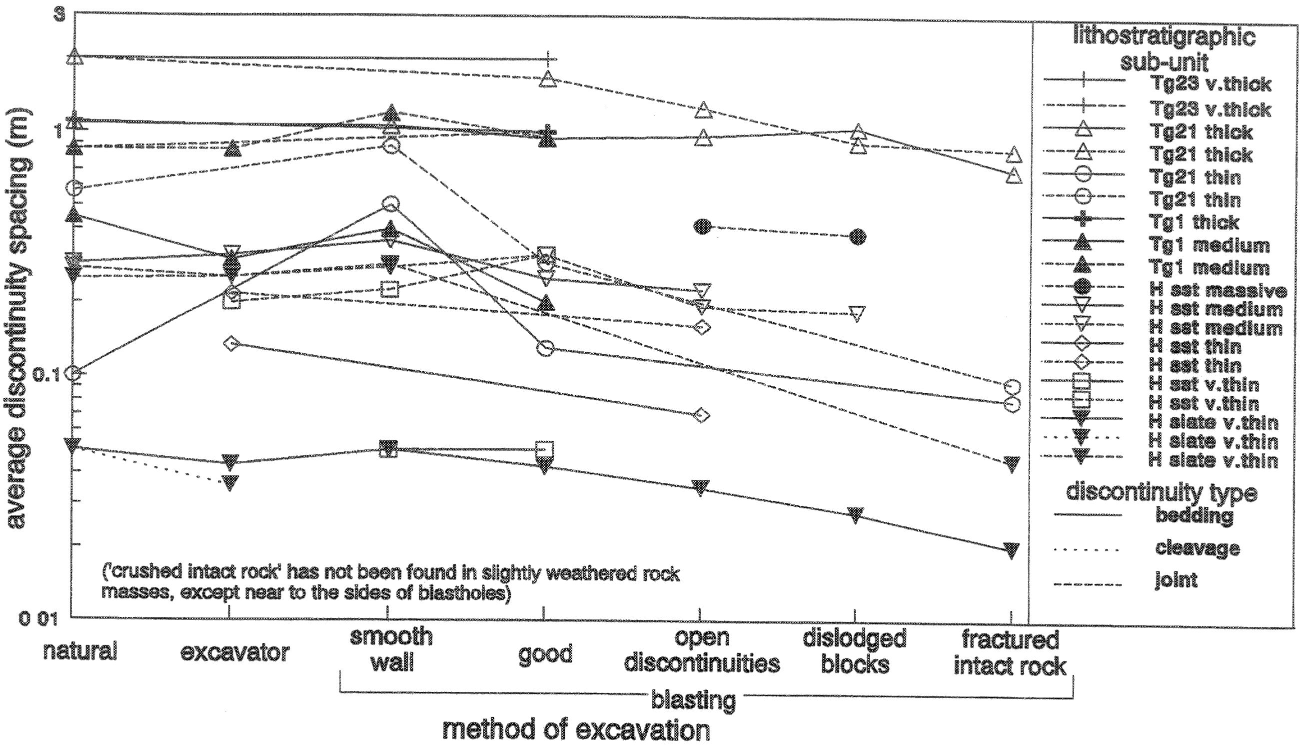


Fig. 52. Examples of average discontinuity spacing per lithostratigraphic sub-unit and type of discontinuity vs the method of excavation in slightly weathered exposures.

This procedure to determine the values is possible if the same lithostratigraphic sub-unit with the same degree of weathering can be followed in connected exposures excavated with different excavation methods(97). In the research area this was, however, only possible at very few locations; not enough to establish the values with sufficient certainty. The alternative is to determine the values in different not connected exposures in the same lithostratigraphic sub-unit with the same degree of weathering, however, this is possible only if no interdependencies exist between discontinuity spacing and method of excavation.

D.1.4.2.1 Interdependency between discontinuity spacing and method of excavation

The goal is to calculate the change in discontinuity spacings before and after excavation as a factor dependent on the method of excavation. Because the spacings before excavation could not be measured and the calculation has to be done from spatially independent exposures (see above), it is necessary to calculate the influence of the excavation methods by comparing different exposures made with different excavation methods. The rock mass in the exposures used for a comparison should belong to the same lithostratigraphic (sub-) unit with the same degree of weathering as well before as after excavation. The results of such a methodology can, however, be influenced by the definition of lithostratigraphic sub-units or by the choice of the excavation method if this choice was based on discontinuity spacing. Another problem may be the dependency of the relation between spacing and damage due to the excavation method, on the absolute spacing or on other properties of the rock mass such as material type, strength, etc..

The definition of lithostratigraphic sub-units
The lithostratigraphic sub-units are defined based on the spacing of bedding or cleavage. So, if this spacing changes due to the excavation method it is possible that the sub-unit observed in an exposure after excavation, is

(97) For example, in Tg21 limestones the decrease in discontinuity spacing from natural exposures and exposures created by pre-splitting/smooth wall blasting compared to exposures created by normal (bulk hole) blasting is striking.

not the same sub-unit as before excavation. Comparison of the influence of the excavation method would then lead to erroneous results[98].

The choice of the excavation method based on discontinuity spacing
The method of excavation may be chosen because of the block size of the (sub-) unit. The influence of the method of excavation can then not be determined from average spa_{mass} values (Fig. 53)[99].

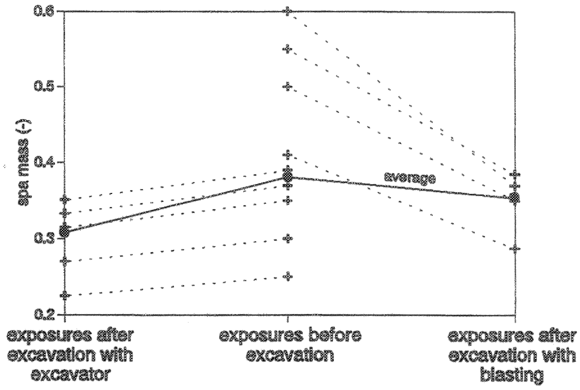


Fig. 53. The average spa_{mass} of a series of exposures in a sub-unit or unit excavated with a particular method may depend more on the absolute spa_{mass} rather than on the influence of the excavation method.

These problems cannot be solved, however, it is likely that their influence is not too serious and can be neglected. This is based on the following observations:

1 A change in sub-unit cannot occur in a lithostratigraphic unit that contains only one typical bedding or cleavage spacing over the whole research area (thus the 'unit' equals the 'sub-unit'). Calculation of the parameter for the method of excavation with only these units results in values which are in the same order as those calculated with all lithostratigraphic sub-units and units.

2 Calculation of the values for the parameter for the method of excavation with only the sub-units found in the research area with the most widely spaced bedding or cleavage, results in values which are in the same order as the values calculated for all lithostratigraphic sub-units and units[100].

3 The values for the influence of the excavation methods (calculated with all sub-units or with the sub-units as calculated in points 1 and 2 above) are generally (far) smaller than the ratios between minimum and maximum bedding or cleavage spacing allowed within one lithostratigraphic sub-unit. This reduces the chance that a sub-unit is different before and after excavation.

4 An a priori block size related choice of the method of excavation is not always made. Interbedded lithostratigraphic sub-units of large and small block size will normally be excavated with one type of excavation method. The choice of a particular type of excavation may also have been based on other reasons such as availability of excavation equipment, etc.. For a unit with large block size and high intact rock strength blasting is the normal choice; assuming that the engineers concerned knew the nature of the rock mass they were about to excavate. Hence, the choice of the method of excavation was not based on the bedding or cleavage spacing if such a unit has been excavated with other means than blasting. Calculation of the values for the method of excavation based on only the lithostratigraphic sub-units with the most widely spaced bedding or cleavage and with a high intact rock strength, avoids therefore a possible dependency of the choice of the method of excavation based on discontinuity spacing. The values calculated in this way are in the same order as for all sub-units.

[98] The problem is illustrated with the following example. Assume two exposures in the same limestone unit; one is a natural exposure and the other is excavated with good conventional blasting. The bedding spacing is 0.5 m in the first exposure and is 0.45 m in the second exposure determined after excavation. The lithostratigraphic sub-unit is in both exposures medium spaced limestone. A comparison of the damage due to the excavation method would lead to the conclusion that conventional blasting reduces spacing only by a factor of 0.9 (= 0.45/0.5). However, it may well be that the original spacing in the rock mass in which the second exposure has been made was 0.7 m, thus thick spaced limestone, and that the reduction of spacing due to conventional blasting is 0.65 (= 0.45/0.7).

[99] This is not applicable to the classes for conventional blasting resulting in a particular type of damage, e.g. open discontinuities, dislodged blocks, etc.. These are never dependent on an a priori choice of excavation method.

[100] Sub-units of a unit with the most widely spaced bedding or cleavage found after excavation in exposures excavated with a particular type of excavation method have a discontinuity spacing which is the original discontinuity spacing (which was likely also the most widely spaced sub-unit) influenced by the damage due to the method of excavation (there are no 'wider' spaced units).

Excavation damage depending on rock mass structure or material

The damage inflicted on a rock mass by a method of excavation may depend on the structure of the rock mass (spacing and orientation of discontinuities, etc.) and on the rock material (strength). The dependency between the excavation damage and rock mass structure and material could, however, not be proved in this research. For some units (very) vague relations with other rock mass properties may be present but statistically these relations are not significant[101] (Fig. 52).

Conclusion

Interdependencies between the spa_{mass} averaged from different exposures and the method of excavation may be present. The effects on calculated factors for the influence of the excavation method on the discontinuity spacing are, however, small and can be neglected for the slopes in the research area.

D.1.4.2.2 The values of the parameter for the method of excavation

The parameter for a particular method of excavation is the ratio of the spa_{mass} (calculated according to Taylor) measured in an exposure made with that method over the spa_{mass} value determined in a 'natural' exposure in the same lithostratigraphic sub-unit and with the same degree of weathering (eq. [30]). This can be calculated directly per lithostratigraphic sub-unit and per degree of weathering, but requires that in a lithostratigraphic sub-unit with a particular degree of weathering at least two observations have been made from which one is made in a 'natural' exposure. Some methods of excavation have, however, been only used in a lithostratigraphic (sub-) unit in which no 'natural' exposures have been found. For these methods of excavation a parameter cannot be calculated if this direct approach is used. However, by using ratios between all methods of excavation it is still possible to calculate values even if no 'natural' exposures are available. For example, assume a lithostratigraphic sub-unit $u1$ with degree of weathering $w1$. In this lithostratigraphic sub-unit two observations are available: one in a 'natural' exposure with $(spa_{mass})_{u1,\,w1,\,n}$ and one excavated with 'pre-splitting/smooth wall blasting' with $(spa_{mass})_{u1,\,w1,\,s}$, the ratio between the two is $ratio_{u1,\,w1,\,s,\,n}$. If is accepted that the damage due to the method of excavation is independent from the lithostratigraphic (sub-) unit and from the degree of weathering as argued in ch. D.1.4.2.1, the ratio found for 'pre-splitting/smooth wall blasting' over 'natural' is independent from the sub-unit and from the degree of weathering. Hence, the value for the method of excavation parameter for 'pre-splitting/smooth wall blasting' ME_s is:

$$ratio_{u1,\,w1,\,s,\,n} = ratio_{s,\,n} = ME_s \qquad [31]$$

No further observations are available in this lithostratigraphic (sub-) unit with this degree of weathering. Therefore it is not possible to establish ratios for other methods of excavation in this lithostratigraphic (sub-) unit with this degree of weathering. Assume another two observations in lithostratigraphic sub-unit $u2$ with degree of weathering $w2$. In this sub-unit no 'natural' exposures have been found, but say, only one exposure made with 'pre-splitting/ smooth wall blasting' with $(spa_{mass})_{u2,\,w2,\,s}$, and one exposure made with conventional blasting with as result 'fractured rock' with $(spa_{mass})_{u2,\,w2,\,f}$. The ratio between these two is $ratio_{u2,\,w2,\,f,\,s}$. With the direct approach (eq. [30]) it would not be possible to establish any values for the method of excavation parameter because no 'natural' exposures are available. If, however, is assumed that the ratio is independent from the lithostratigraphic sub-unit and from the degree of weathering then:

$$ratio_{f,\,s} = \frac{(spa_{mass})_{u2,\,w2,\,f}}{(spa_{mass})_{u2,\,w2,\,s}} = \frac{\dfrac{(spa_{mass})_{u2,\,w2,\,f}}{(spa_{mass})_{u2,\,w2,\,n}}}{\dfrac{(spa_{mass})_{u2,\,w2,\,s}}{(spa_{mass})_{u2,\,w2,\,n}}} = \frac{ratio_{u2,\,w2,\,f,\,n}}{ratio_{u2,\,w2,\,s,\,n}} = \frac{ratio_{f,\,n}}{ratio_{s,\,n}} = \frac{ME_f}{ME_s} \qquad [32]$$

$$or: \quad ME_f = ratio_{f,\,s} * ME_s$$

Hence, a combination of eqs [31] and [32] gives the option to calculate a value for a method of excavation parameter (in this example: ME_f) even if no 'natural' exposures are available in a particular lithostratigraphic (sub-) unit with a particular degree of weathering. Another benefit of calculating ratios rather than calculating the values for the method of excavation parameter directly, is that all ratios can be used between any two methods of

(101) The adjustment factors for the method of excavation from Laubscher (1990) are also not dependent on rock mass structure or material (ch. B.2.3.3).

excavation in any lithostratigraphic sub-unit and with any degree of weathering. This increases the accuracy of the values for the method of excavation parameter considerably because more observations can be used.

The above is implemented as follows. The spa_{mass} values of the exposures are averaged per lithostratigraphic sub-unit (u), per degree of weathering (w) and per method of excavation (i), resulting in average $(spa_{mass})_{u,\,w,\,i}$. Then the ratios are determined between any two types of excavation per lithostratigraphic sub-unit and per degree of weathering:

$$ratio_{u,\,w,\,i,\,j} = \frac{average\ (spa_{mass})_{u,\,w,\,i}}{average\ (spa_{mass})_{u,\,w,\,j}} \qquad [33]$$

u = *lithostratigraphical (sub-) unit*
w = *degree of rock mass weathering*
i, j = *method of excavation*

The ratios are independent of the lithostratigraphic sub-unit and of the degree of weathering (ch. D.1.4.2.1), hence $ratio_{u,\,w,\,i,\,j}$ can be averaged:

$$average\ ratio_{i,\,j} = \frac{1}{W} * \sum^{W}\left(\frac{1}{U_w} * \sum^{U_w} ratio_{u,\,w,\,i,\,j}\right) \qquad [34]$$

U_w = *number of lithostratigraphical (sub-) units per degree of weathering*
W = *number of different degrees of weathering*

With actual data the calculated average ratios are formulated in the following set of equations:

$$\begin{aligned}
x_i &= average\ ratio_{i+1,\,i} & i &= 0, 1 .. 6\\
x_{i+1} * x_i &= average\ ratio_{i+2,\,i} & i &= 0, 1 .. 5\\
x_{i+2} * x_{i+1} * x_i &= average\ ratio_{i+3,\,i} & i &= 0, 1 .. 4\\
x_{i+3} * x_{i+2} * x_{i+1} * x_i &= average\ ratio_{i+4,\,i} & i &= 0, 1 .. 3\\
x_{i+4} * x_{i+3} * x_{i+2} * x_{i+1} * x_i &= average\ ratio_{i+5,\,i} & i &= 0, 1 .. 2\\
x_{i+5} * x_{i+4} * x_{i+3} * x_{i+2} * x_{i+1} * x_i &= average\ ratio_{i+6,\,i} & i &= 0, 1\\
x_{i+6} * x_{i+5} * x_{i+4} * x_{i+3} * x_{i+2} * x_{i+1} * x_i &= average\ ratio_{i+7,i} & i &= 0
\end{aligned} \qquad [35]$$

and the values for $x_{0,1\,..\,6}$ are found by optimization. The values (ME_j) for the method of excavation are then:

$$\begin{aligned}
ME_{natural} &= 1.00\\
ME_{j+1} &= \frac{1}{\prod_{i=0}^{j} x_i} \qquad j = 0, 1,.., 6
\end{aligned} \qquad [36]$$

j = 0: *pneumatic hammer excavator*, j = 1: *pre-splitting/smooth wall blasting*,
conventonal blasting with result: j = 2: *good*, j = 3: *open discontinuities*,
j = 4: *dislodged blocks*, j = 5: *fractured intact rock*, j = 6: *crushed intact rock*

A weighting factor is used in the optimization of $x_{0,1\,..\,6}$ because the numbers of exposures excavated with each particular method of excavation are not all the same:

$$weighting\ factor\ for\ average\ ratio_{i,\,j} = number\ of\ exposures_i * number\ of\ exposures_j \qquad [37]$$

The resulting values for the parameter for the method of excavation are shown in Fig. 54 and Table 12. The values and standard errors are calculated by using a Monte Carlo simulation on the above methodology (ch. D.2.4.1).

D.1.4.3 Reliability of the parameter for the method of excavation

The values for the parameter for the method of excavation are as reliable as the number of exposures and number of different units they are based on (Table 12). The more exposures in different lithostratigraphical (sub-) units the more reliable the values. The values for 'pneumatic hammer excavation', 'pre-splitting/smooth wall blasting', 'good conventional blasting' and 'conventional blasting with as result 'open discontinuities' are based on a considerable quantity of exposures in different lithostratigraphical (sub-) units. Values for conventional blasting resulting in dislodged blocks and fractured and crushed intact rock are, however, based on fewer exposures in less different (sub-) units and consequently the reliability is lower. The standard errors reflect the scatter and uncertainty in the data. These do not reflect whether the values are also applicable to other lithologies than those used for the calculation of the values. However, considering the number of observations and the number of different lithologies observed, the values are likely to be applicable also outside the research area.

D.1.4.4 Discussion, comparison to literature values and conclusion

Fig. 54 shows that the value reflecting the damage inflicted by 'pneumatic hammer excavation' is approximately the same as the value for 'good conventional blasting'. The type of excavator used in the research area is a pneumatic hammer mounted on a shovel frame. In particular in rock units with small discontinuity spacing (e.g. slate) the pneumatic hammer causes a severe damage to the rock mass by fracturing of intact rock.
The values for the method of excavation are based on data with a high scatter. The data is established in different units consisting of different rock masses and rock material. The parameter values do not depend on the type of rock mass, e.g. lithology, intact rock strength, discontinuity spacing, etc., however, this may be a consequence of the high scatter in the data. In particular, slopes excavated with conventional blasting in karstic rock masses were found mostly to be more unstable than slopes excavated with conventional blasting in other rock masses (ch. C.3.5.2). This is likely caused by the method of excavation, however, the scatter in the data is too large to allow a different class for blasting in karstic rock masses to be established.

The adjustment factors for the method of excavation[102] from Romana's SMR slope classification system (Romana, 1991) and from Laubscher's MRMR classification for underground excavations (Laubscher, 1990) are plotted in Fig. 54 as the ratio of a rock mass obtaining maximum points (100 points) in an exposure excavated with the method of excavation, to the same rock mass but in a natural exposure. A comparison of the absolute values is not possible, but the relative differences between the excavation classes are comparable. Note that Romana also found that the damage to a rock mass caused by mechanical excavation is about equal to the damage caused by 'good' conventional blasting.

As expected the method of excavation influences the discontinuity spacing. The values established for the method of excavation parameter are used in the SSPC system to correct for damage of the rock mass due to the method of excavation.

[102] The SMR adjustment factor for 'mechanical excavation' is compared to 'pneumatic hammer excavation' in the SSPC system. Romana's SMR and Laubscher's MRMR have only one class for deficient or poor blasting. This class is arbitrarily plotted at the position of 'blasting with result dislodged blocks' in the SSPC system.

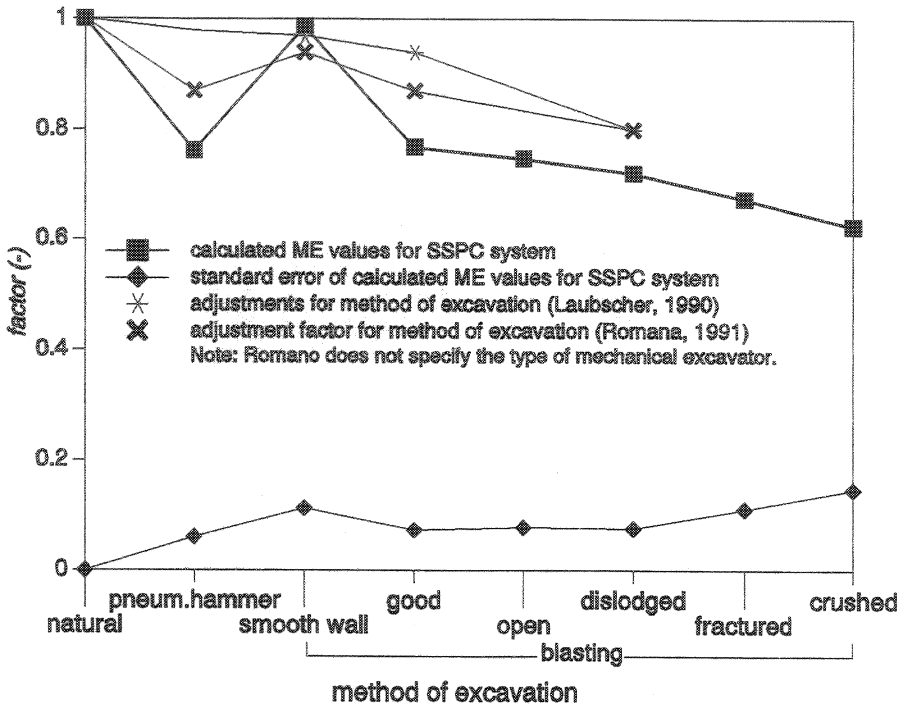


Fig. 54. Values for the parameter for the method of excavation compared to values from Laubscher (1990) and Romana (1991) (lines in-between data points have no meaning, and serve only for identification).

METHOD OF EXCAVATION (*ME*)(1)							
method			*ME*		number of observa-tions(3)	lithostratigraphic	
			mean value	standard error		sub-units (2)(3)	units(2)
natural/hand-made			1.00	-	92	23	6
pneumatic hammer excavation			0.76	0.06	173	21	6
blasting	pre-splitting/smooth wall		0.99	0.11	57	19	6
	conventional with the following result:	good	0.77	0.07	131	28	7
		open discontinuities	0.75	0.08	54	20	7
		dislodged blocks	0.72	0.08	14	8	4
		fractured intact rock	0.67	0.11	18	7	4
		crushed intact rock	0.62	0.15	5	4	3
				Total:	544	30	7

notes:
1 Data used for calculation are the combined data gathered for the SSPC system and for the engineering geological mapping (see preface).
2 Columns 'sub-units' and 'units' are respectively the number of lithostratigraphic sub-units and lithostratigraphic units used for the calculation of the *ME* values.
3 Used for the calculation of *ME* values and included in the column 'lithostratigraphical sub-units' are only those in which at least two different methods of excavations have been used in exposures with the same degree of weathering, so that excavation damage could be compared in the same lithostratigraphic sub-unit with the same degree of weathering.

Table 12. Parameter for the method of excavation (*ME*) (for calculation see ch. D.2.4.1).

D.1.5 Parameter for the degree of weathering

The values for the parameter for the degree of weathering are the factors with which the value of a geotechnical parameter is reduced due to weathering. The determination of the values for the parameter for the degree of weathering is for a large part analogous to the methodology used for the parameter of the degree of weathering in the foregoing chapter. Influence of weathering and the values for the parameter for weathering can be determined from geotechnical parameters of the rock mass in spatially independent exposures. The calculation method is described in ch. D.1.5.2. Values for the parameter for weathering are determined for different geotechnical parameters (ch. D.1.5.3). Implementation of the parameter for weathering in the SSPC system as a single parameter is described in ch. D.1.5.4. The reliability of the values and a comparison of the values with values found in the literature are evaluated (in respectively chs. D.1.5.5 and D.1.5.6). The incorporation into the SSPC system is described in ch. D.1.5.7.

D.1.5.1 Interdependencies between weathering and lithostratigraphical (sub-) unit

As with the parameter for the method of excavation it is, in principle, very simple to determine the values for the parameter for the degree of rock mass weathering. A simple comparison of a rock mass parameter important in slope stability as defined before (e.g. *irs*, *TC*, spa_{mass} and con_{mass}) per lithostratigraphic (sub-) unit in exposures with different degrees of weathering gives the required quantitative values (WE_j):

$$WE_j = \frac{rock\ mass\ parameter_j}{rock\ mass\ parameter_{fresh}} \qquad [38]$$

$$j = degree\ of\ weathering$$

In some locations it is possible to follow a unit through different degrees of weathering in one exposure and to establish with certainty the influence of rock mass weathering(103). This was, however, not enough to establish the values for WE_j accurately because the number of exposures in which a lithostratigraphical (sub-)unit could be followed through different degrees of weathering was very small. Alternatively the values for the parameter for the degree of weathering can be determined from spatially independent exposures. The definition of the lithostratigraphic sub-units may, however, not be independent from the influence of rock mass weathering and the decrease of geotechnical parameters due to weathering may be dependent on the rock mass structure or material (these problems are analogous to those discussed in ch. D.1.4.2.1). The influence of both is, however, supposed to be small and can be neglected because:

1. The trends of weathering influence on the different rock mass parameters are roughly the same for all the lithostratigraphic (sub-) units, except for the 'soil type'(104) units (see graphs in appendix IV). This may seem strange as weathering is dependent on rock mass material and structure. The description of rock mass weathering following BS 5930 (1981) is, however, based on decomposition and disintegration of rock material which result in features such as a decrease of intact rock strength, decrease of discontinuity spacing and increase in quantity of infill material in discontinuities. These are the same features that determine the geotechnical parameters.
2. Calculations with only the thickest spaced members of a unit and the units that have only one typical bedding or cleavage spacing throughout the research area, resulted in values approximately equal to the values calculated based on all lithostratigraphic (sub-) units (this is the same argument as used for the method of excavation (ch. D.1.4.2.1).

(103) The influence of weathering is clearly visible in the Tg21 formation limestones and dolomites. These can be followed in one exposure with a clear decrease of the bedding and joint spacings for an increasing degree of weathering.

(104) 'Soil type' units consist of loosely cemented grains or small particles, generally either without clearly defined mechanical discontinuities or with highly irregular and thinly laminated mechanical discontinuities, and have a low intact rock strength. 'Soil type' units resemble more cemented soils than a rock mass.

D.1.5.2 Calculation method

The values for the parameter for weathering are calculated as were the values for the parameter for the method of excavation (ch. D.1.4.2.2). The parameter investigated is averaged per lithostratigraphic (sub-) unit (u) and degree of weathering (i), resulting in: average $(rock\ mass\ parameter)_{u,\ i}$. Then the ratios are determined between any two degrees of weathering per lithostratigraphic (sub-) unit:

$$ratio_{u,\ i,\ j} = \frac{average\ (rock\ mass\ parameter)_{u,\ i}}{average\ (rock\ mass\ parameter)_{u,\ j}} \qquad [39]$$

u = *lithostratigraphical (sub-) unit*
i, j = *degree of weathering*

The ratios are independent of the lithostratigraphic sub-unit; hence, the $ratio_{u,\ i,\ j}$ can be averaged:

$$average\ ratio_{i,\ j} = \frac{1}{U} * \sum^{U} ratio_{u,\ i,\ j} \qquad [40]$$

U = *number of lithostratigraphical units*

Hence, the calculated average ratios are formulated in the following set of equations:

$$\begin{aligned} x_i &= average\ ratio_{i+1,\ i} && i = 0,\ 1\ ..\ 3 \\ x_{i+1} * x_i &= average\ ratio_{i+2,\ i} && i = 0,\ 1\ ..\ 2 \\ x_{i+2} * x_{i+1} * x_i &= average\ ratio_{i+3,\ i} && i = 0,\ 1 \\ x_{i+3} * x_{i+2} * x_{i+1} * x_i &= average\ ratio_{i+4,\ i} && i = 0 \end{aligned} \qquad [41]$$

and the values for $x_{0,1\ ..\ 3}$ are found by optimization. The values for weathering are then:

$$\begin{aligned} WE_{fresh} &= 1.00 \\ WE_{j+1} &= \frac{1}{\prod_{i=0}^{j} x_i} && j = 0,\ 1,..,\ 3 \end{aligned} \qquad [42]$$

j = 0: *slightly*, j = 1: *moderately*, j = 2: *highly*, j = 3: *completely*

A weighting factor is used in the optimization of $x_{0,1\ ..\ 3}$ because the numbers of exposures for each particular degree of weathering are not all the same:

$$weighting\ factor\ for\ average\ ratio_{i,\ j} = number\ of\ exposures_i * number\ of\ exposures_j \qquad [43]$$

The mean values for the parameter for the degree of weathering and the standard errors are calculated by using a Monte Carlo simulation (ch. D.2.4.2).

D.1.5.3 Influence of weathering on rock mass parameters used in the SSPC system

The influence of weathering has been investigated for the following parameters: intact rock strength (*irs*), overall discontinuity spacing in a rock mass (spa_{mass} calculated following Taylor, 1980, eq. [13], page 76), condition of a single discontinuity (set) (*TC*), the overall weighted condition of discontinuities in a rock mass (con_{mass}), and the friction (φ_{mass}) and cohesion (coh_{mass}) of a rock mass (ch. D.1.3.6). Appendix IV shows examples of the influence of weathering on these different geotechnical parameters used in the SSPC system for different lithology groups. These groups, e.g. coarse and fine grained and calcareous[(105)] lithologies, show a similar decrease in the values of geotechnical parameters and can be grouped. However, the differences between the different groups are not so large that no parameter for weathering could be determined applicable to all lithologies. Fig. 55 and Table 13 present a summary of the influence of weathering on all investigated geotechnical parameters. For 'soil type' units

[(105)] Calcareous lithologies show a similar decrease of the values of geotechnical parameters with increasing weathering independent of the contents of other than calcitic minerals, e.g. clay minerals. It should, however, be noted that pure limestones or dolomites do not occur in a 'highly' and 'completely' weathered form. The values for calcareous units for 'highly' and 'completely' weathered are therefore based on units that contain a certain quantity of other than calcitic minerals.

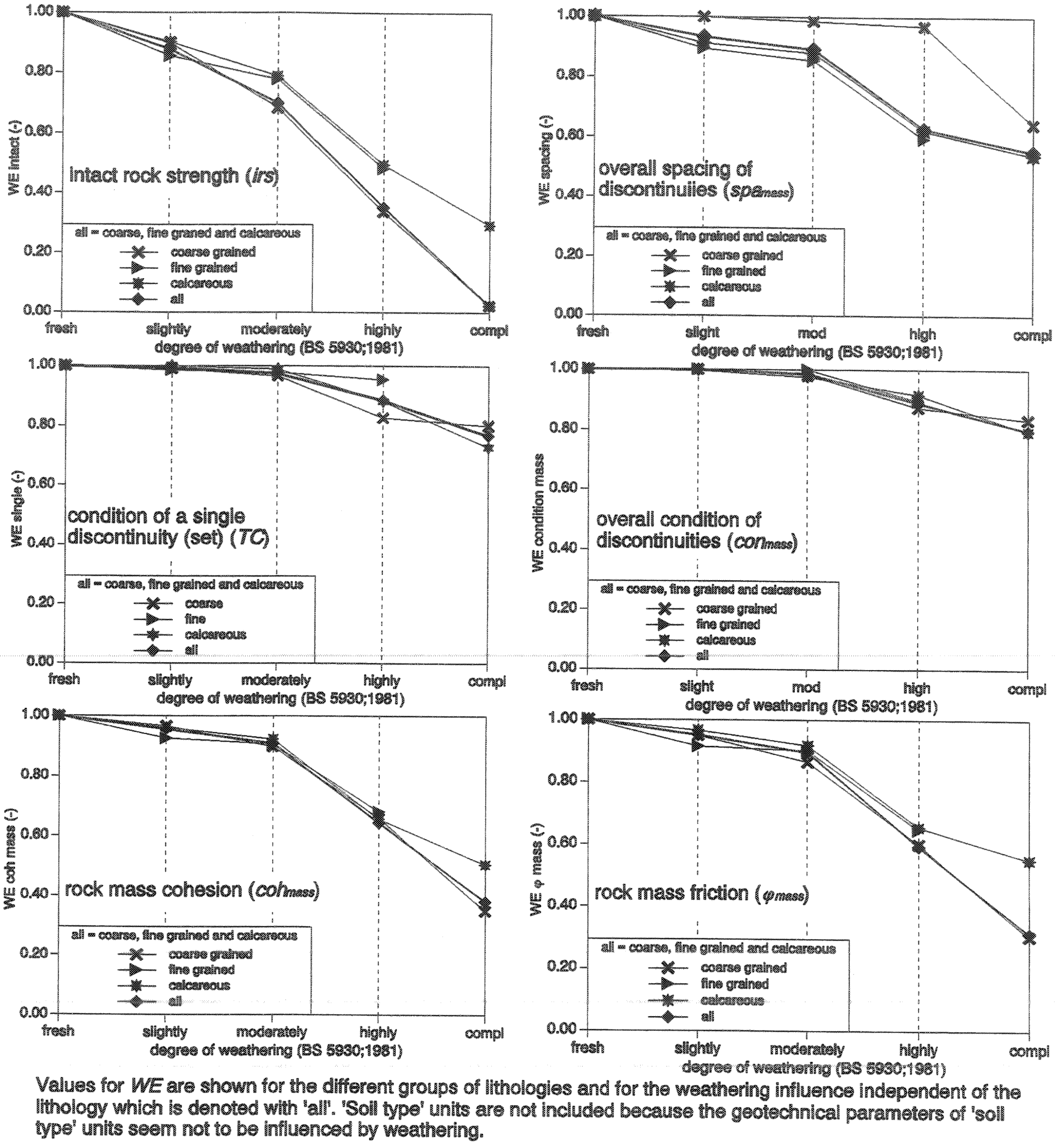


Fig. 55. Overview of the influence of weathering on different geotechnical parameters.

the influence of weathering is either absent or is smaller than the scatter in the data and could be neglected. The following remarks can be made on the interpretation of the figures in appendix IV, Fig. 55 and Table 13 (the remarks do not apply to 'soil type' units).

- *Influence of weathering on the intact rock strength*

The degrees of weathering described as moderately, highly and completely weathered imply that a proportion of the rock mass has decayed to a geotechnical soil. The intact rock strength is that of rock blocks remaining in the particular degree of weathering. The intact rock strength decreases with increasing degree of weathering.

- *Influence of weathering on the overall discontinuity spacing in the rock mass*

The decrease of the spacing due to weathering is independent of the type of discontinuity. A general decrease of the overall spacing parameter with increasing degree of weathering is evident for all lithologies.

WEATHERING									
degree of rock mass weathering (BS 5930; 1981)	intact rock strength	overall spacing of discontinuities (1) (*spa mass*)	condition of a single discontinuity (set)	overall condition of discontinuities (1) (*con mass*)	rock mass (1)		number of observations (3)	lithostratigraphic	
	WE intact	*WE spacing*	*WE single*	*WE con mass*	*WE coh mass*	*WE φ mass*		sub-units (2)(3)	units (2)
fresh	1.00	1.00	1.00	1.00	1.00	1.00	12	7	5
slightly	0.88	0.93	0.99	1.00	0.96	0.95	168	20	6
moderately	0.70	0.89	0.98	0.99	0.91	0.90	27	12	6
highly	0.35	0.63	0.89	0.89	0.64	0.59	6	3	3
completely(4)	0.02	0.55	0.77	0.80	0.38	0.31	2	1	1
						Total:	215	24	7

notes:
1 Values have been calculated after correction for damage due to the method of excavation.
2 Columns 'sub-units' and 'units' are respectively the number of lithostratigraphic sub-units and the number of lithostratigraphic units used for the calculation of *WE* values.
3 Used for the calculation of *WE* values and included in the column 'lithostratigraphic sub-units' are only those in which at least two different degrees of weathering have been observed so that weathering effects could be compared in the same 'lithostratigraphic sub-unit'.
4 'completely weathered' is assessed in granodiorite only.

Table 13. Values for the parameter for weathering.

- Influence of the weathering on condition of a single discontinuity (set) and on the overall condition of discontinuities parameter in a rock mass
No major differences are evident between the influence of weathering on bedding or on cleavage and joint planes (Fig. A 102, appendix IV). The general decrease of the condition of a discontinuity (as well for a single discontinuity as for the condition of discontinuities parameter, con_{mass}) with increasing degree of weathering is evident beginning with a slightly weathered rock mass, but is considerably less than the decrease of intact rock strength and spa_{mass}.
- Influence of weathering on rock mass strength parameters
The influence of weathering on the rock mass cohesion coh_{mass} and friction φ_{mass} is evident and is similar for both parameters.

D.1.5.4 *WE* parameter in SSPC system

In the SSPC system three rock mass parameters are of importance. For 'orientation dependent stability' the rock mass parameter influenced by weathering is the condition of a single discontinuity (set): WE_{single}. For 'orientation independent stability' the rock mass parameters influenced by weathering are coh_{mass} and φ_{mass}, expressed in respectively WE_{coh} and $WE_{\varphi\ mass}$. Using several parameters for weathering in the SSPC system may be confusing and therefore in the SSPC classification system only one parameter for weathering is used: WE_{mass}. WE_{mass} is the average of WE_{coh} and $WE_{\varphi\ mass}$ because the values for both are very similar. To be able to determine the influence of weathering for a single discontinuity (set) with a $WE_{\varphi\ mass}$ relation has been established between WE_{mass} and WE_{single}:

$$WE_{single} = \sqrt{1.452 - 1.220 * e^{-WE_{mass}}} \qquad [44]$$

$$(correlation\ coefficient = 0.999)$$

Table 14 and Fig. 56[106] show the mean values and standard errors for the parameter of weathering used in the SSPC system.

[106] In the forms for the calculation of the SSPC system the parameter is denoted with *WE* without subscript as only one weathering parameter is used.

WEATHERING(1)								
degree of rock mass weathering (BS 5930; 1981)	condition of a single discontinuity (set)		rock mass					
	WE single		*WE coh mass*		*WE φ mass*		*WE mass*(2)	
	mean value	standard error	mean value	standard error	mean value	standard error	mean value	standard error
fresh	1.00	-	1.00	-	1.00	-	1.00	-
slightly	0.99	0.04	0.96	0.06	0.95	0.06	0.95	0.06
moderately	0.98	0.03	0.91	0.06	0.90	0.07	0.90	0.07
highly	0.89	0.05	0.64	0.11	0.59	0.12	0.62	0.12
completely(3)	0.77	0.09	0.38	0.11	0.31	0.10	0.35	0.11

notes: 1 Values have been calculated after correction for damage due to the method of excavation.
2 *WE mass* is the average of *WE coh mass* and *WE φ mass*.
3 'completely weathered' is assessed in granodiorite only.

Table 14. Values for the degree of weathering for a single discontinuity (set) and for a rock mass as used in the SSPC system (for calculation see ch. D.2.4.2).

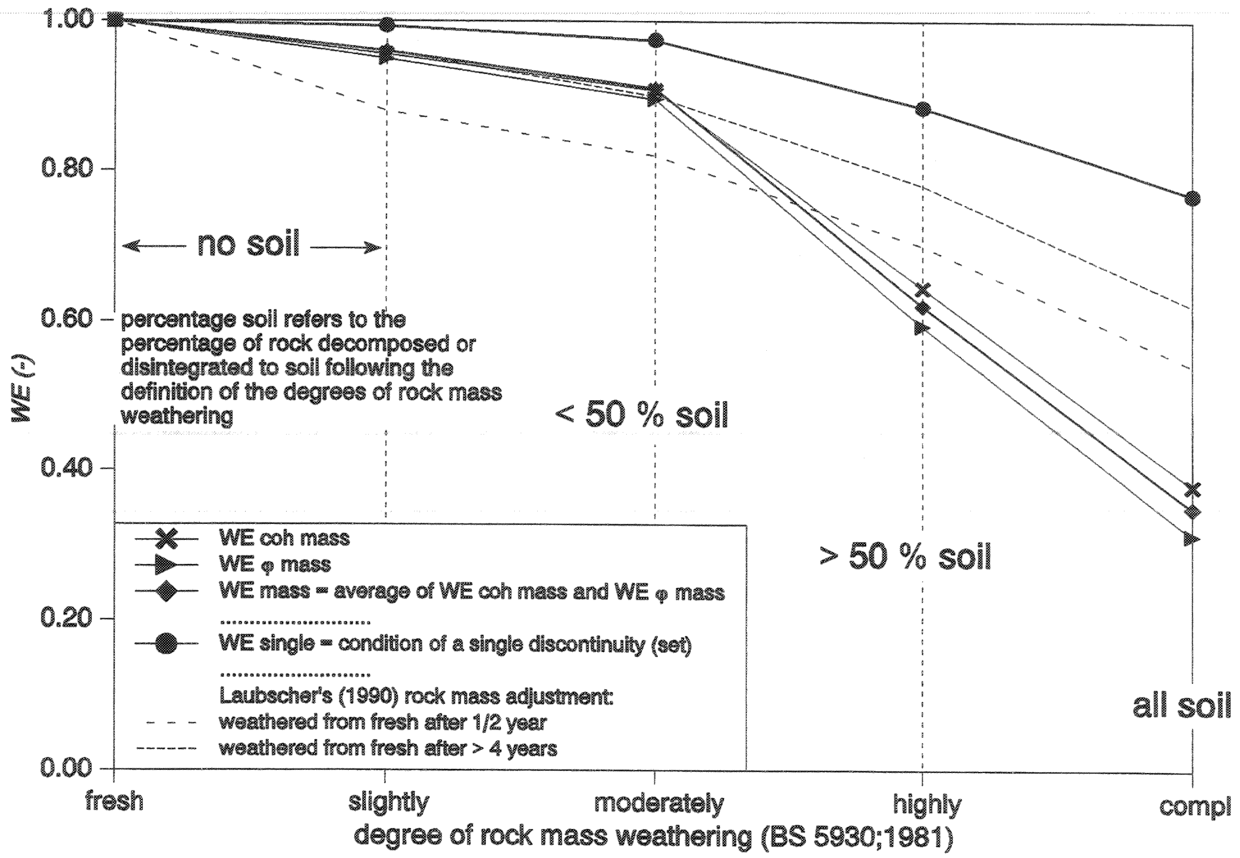


Fig. 56. Weathering parameters vs degree of rock mass weathering (refer for the rock mass adjustments following Laubscher to Table 7, page 60).

D.1.5.5 Reliability

The values established for weathering are as reliable as the number of exposures (observations) and the number of different lithostratigraphic (sub-) units on which they are based. The values for 'slightly' and 'moderately' weathered are based on a large number of exposures and different units, but the values for 'highly' and 'completely' weathered are based on fewer exposures and units[107]. Consequently, these values are expected to be less reliable. The standard error for the weathering values (Table 14) reflects the scatter and uncertainty in the data. These do not reflect whether the values are also applicable to other lithologies than those used for the calculation of the values. However, considering the number of observations and the number of different lithologies observed, the values are likely to be applicable also outside the research area. Some of the uncertainties in the data may stem from the practical difficulties of applying the weathering classification given in BS 5930; 1981 (see also appendix V).

D.1.5.6 Comparison to literature values

The rock mass adjustment factors for susceptibility to weathering according to Laubscher's rock mass classification system (Table 7, page 60) are adjustment factors describing the future influence of weathering in a mining environment. At the time of excavation the rock mass is supposed to be fresh and to weather within a certain time span to another degree of weathering. The degradation of the rock mass due to weathering and thus the reduction of its mechanical characteristics is expressed, by Laubscher, as a factor. The rock mass rating, calculated following Laubscher, obtained for the fresh rock mass after excavation is multiplied by this factor to obtain Laubscher's rock mass rating in a weathered state.
A time-span for the weathering process could not be defined for the SSPC system (ch. D.1.6), however, the influence of an increase in weathering on Laubscher's rock mass rating, e.g. Laubscher's factors for susceptibility to weathering, can be compared to the influence of the degree of weathering on the mechanical parameters used in the SSPC system. Although the SSPC system uses different parameters than Laubscher's rock mass rating a comparison is likely valid because both describe the mechanical characteristics of the rock mass. The adjustment factors of Laubscher are included in Fig. 56 and show that these factors for a rock mass weathered from 'fresh' to 'slightly' or 'moderately' for a time-span of more than 4 years, are the same as the parameter for rock mass weathering (WE_{mass}) obtained in this research. For an increase in weathering to 'highly' and 'completely' weathered the factors according to Laubscher are larger, e.g. the influence of weathering on the rock mass parameters is less than according to the SSPC system. This is likely due to the difference in the influence of the condition of discontinuities on the final rock mass parameter. The condition of discontinuities, which is the parameter least influenced by weathering for 'highly' and 'completely' weathered rock masses, has an influence of about 34 % on the MRMR rating while in the SSPC system the influence of the condition of discontinuities on the rock mass friction and cohesion is in the order of 7 to 9 %.
The correlation of the weathering parameter (WE) of the SSPC system with the adjustment factors of Laubscher supports the concept of the weathering parameter (WE) as defined in the SSPC system. Laubscher's values are based on research for a different application (underground excavations) and on different rock types than the SSPC system. The validity of the SSPC weathering factor is thus likely not restricted to use for the mechanical behaviour of surface slopes in the rock types studied in the research area.

D.1.5.7 Conclusions

The preceding chapters have demonstrated how weathering influences the intact rock strength, the overall spacing of discontinuities and the overall condition of discontinuities. Weathering is of obvious importance in the estimation of the stability of existing slopes and in the forecasting of the stability of new slopes for which the degree of weathering may increase in the future. For this reason the values for WE_{mass} (Table 14) and the relation between WE_{mass} and WE_{single} in eq. [44], are incorporated in the SSPC system to correct geotechnical parameters for past and future weathering.

[107] Highly and completely weathered exposures have not been found for all formations, because of erosion, vegetation and agricultural use. Highly or completely weathered exposures of pure limestone or dolomite are nonexistent.

The condition of discontinuities is considerably less influenced by weathering than the intact rock strength and the spacing of discontinuities. The influence of weathering for all rock mass parameters is low for an increase in the degree of weathering from 'fresh' to 'slightly' and 'moderately', but strongly increases for 'highly' and 'completely' weathered. This corresponds to the percentage (as indicated in Fig. 56) of the rock material which is decomposed or disintegrated into a soil following the definition of the degrees of rock mass weathering (BS 5930; 1981). 'Soil type' units seem not to be influenced by weathering. The scatter in the data is larger than a possible decrease of intact rock strength, spacing or condition of discontinuities. The correlation between the adjustment factors of Laubscher and the weathering parameters of the SSPC system supports the correctness of the approach to determine the weathering parameters and it extends the validity of the weathering parameters also to rock types not occurring in the research area.

D.1.6 Susceptibility to weathering

The susceptibility to weathering of a rock mass as a function of time is one of the parameters most difficult to determine. Not only is the parameter dependent on the lithology, texture and structure of the rock and rock mass material but also on the climate, quantities of water percolating through the rock mass, chemicals and salts dissolved in the water, the orientation of the exposure, etc.. The type and quantity of chemicals and salts dissolved may change in time due to change of landuse, change in fertilizer use, etc.. These influences cannot be incorporated in enough detail to give a parameter for susceptibility to weathering leading to a universally valid function of time. Slope failure may, however, occur due to the rock mass weathering within the engineering lifetime of the slope. For this reason the degree of weathering of the rock mass at the location of the slope that will be reached at the end of the engineering lifetime is estimated in the SSPC system. The rock mass parameters (ch. D.1.5) at the location of the slope are corrected for the estimated degree of rock mass weathering expected at the end of the engineering lifetime of the new slope, and the slope stability is calculated as if the slope is made in this more weathered rock mass. The determination of the degree of rock mass weathering for an existing exposure is, to a certain extent, subjective. The accuracy with which the degree of rock mass weathering can be determined at the end of the engineering lifetime is, however, not only partly subjective, but will depend heavily on the experience of the observer. The accuracy of the estimation depends also on rock mass specific factors and local circumstances such as the regularity of weathering over the years of the rock mass considered, the quantity of exposures in the area, the differences in time of existence of the exposures, the number of different degrees of rock mass weathering present and the homogeneity of the rock mass.

Susceptibility to weathering is a major factor in determining the slope stability at the end of the engineering lifetime of a slope excavated in a rock mass prone to weathering within the engineering lifetime of the slope (see also page 152). The SSPC system is not designed to quantify susceptibility to weathering as a function of time, however, with the SSPC system the future stability of a slope can be determined if the future degree of rock mass weathering can be predicted. In most other classification systems for slope stability (except Haines et al., 1991, ch. B.2.4.7), the influence of future rock mass weathering is neither discussed nor quantified.

D.1.7 Water pressures in discontinuities

Water pressures in a discontinuity counteract the normal stress across the discontinuity and therefore reduce the shear resistance along the discontinuity. Water pressures in discontinuities are therefore an important reason for slope instability in traditional limiting-equilibrium stability calculations (Hoek et al., 1981, Giani, 1992, Fig. 7a and b, page 11). However, in ch. B.3.4.12 is shown that this influence may be considerably less than often assumed because of the stress distribution in a slope and the possible restriction of water flow and pressures to discontinuity channels. The reduction of the influence of water in more recent classification systems supports this (ch. B.3.3). Moreover in ch. C.3.3.7 is shown that a classification system for slopes should contain a parameter for water pressures only if the system is used for the design of a new slope that will intersect a permanent water table. This led to the introduction of a parameter for permanent water pressure in the 'initial point rating' system (ch. C.4).

Whether this parameter should be maintained in the SSPC system can be questioned. The friction angles determined with the 'sliding criterion' should have been considerably lower than laboratory and literature friction values if water pressures in the order of magnitude as normally assumed in traditional limiting-equilibrium calculations had been present in the stable slopes that determine the 'sliding criterion'. The friction angles from the 'sliding criterion' are, however, very well comparable with laboratory and literature values (appendix III) and there is no reason to assume any water pressure influence. Obviously water pressures may have only been present in unstable slopes. This is, however, highly unlikely because the rock masses of stable and unstable slopes are not fundamentally different with respect to the possibilities for water pressure build-up.

In the research area it is thus unlikely that water pressures are important. This is also supported by the fact that virtually no evidence of water under pressure, such as water spurting out of discontinuities, has been observed in stable or unstable slopes, not even during or after heavy and prolonged rainfall. The evidence of water in discontinuities has been some limited and localized seepage out of some discontinuities. It is likely that more evidence of water under pressure in discontinuities had been observed if the instability of many slopes in the research area had been caused by water pressures in discontinuities (see also chs. D.5.2, D.5.3, examples II and III). Moreover, for the majority of the failed slopes it is difficult to imagine how the discontinuities could ever have been filled, completely or for a large part, with water because the water can flow out of the discontinuities sideways or via other connecting discontinuities. The pressure build-up in such rock masses is equivalently smaller and considerably less than those normally assumed in a traditional limiting-equilibrium calculation (see also chs. D.5.2, D.5.3, examples II and III).

Notwithstanding the above it should be noted that most slope failures occur during or directly after rainfall, this also happened in the research area. This does not conflict with the observation that water pressures may be of less importance in slope failures. Discontinuities will become saturated during rainfall. Lubrication and the reduction of the friction angle of infill material that softens under the influence of water, e.g. clay, cause the slope to fail. The observation that slopes often fail directly after rainfall and not always during rainfall may be further evidence that the softening of infill material is the reason for failure. If the water pressures had been the cause for the slope failure, failures would occur during the rainfall because water levels drop after rainfall ceases. The saturation process of infill material is, however, time dependent because most softening infill material has a low permeability, and it is thus very well conceivable that the maximum saturation is reached after rainfall.

A separate parameter for water pressures in discontinuities for the slopes in the research area is not incorporated in the SSPC system. The presence of water causing lubrication and softening of infill material is already incorporated in the parameters describing the infill material in a discontinuity. Whether in other areas with more rainfall or different rock types a parameter for water pressures is needed cannot be conclusively answered. However, considering that the area has been subject to heavy and prolonged rainfall and the amount of different lithologies and rock mass types, it is not likely that a parameter for water would be needed elsewhere.

D.2 PROBABILITY ANALYSES

The quantity of data collected during the research allows for a statistical analysis of the relations found in the foregoing chapters. A probabilistic quantification of the stability results in the slope stability probability classification (SSPC). Such probabilistic analyses require an analysis of the distributions of the input (field) data and parameters (ch. D.2.1). This is followed by probability analyses of the 'sliding' and 'toppling' criteria for orientation dependent stability (ch. D.2.2), of the linear and shear plane models for orientation independent stability (ch. D.2.3), and of the parameters for the method of excavation and the degree of rock mass weathering (ch. D.2.4).

D.2.1 Distributions of field data and derived parameters

A discussion of the distributions and errors of field data used for the development of the SSPC system should consider what different types of distributions and possible errors are present for each rock mass parameter measured in the field, for each parameter describing the geometry of the slope, and for the visually estimated stability.

Rock mass parameters

A rock mass parameter measured has a distribution that is the combined result of:

1 the distribution of a parameter in a rock mass, and
2 the limitations of the distribution of a rock mass parameter imposed by the subdivision in geotechnical units, and
3 the error made in measuring a rock mass parameter in a geotechnical unit.

Parameters describing the geometry of the slope

The distribution of a parameter describing the geometry of a slope is the combined result of:

1 the distribution of the geometrical parameter, and
2 the error made in measuring a geometrical parameter.

Visually estimated slope stability

The error made in visually estimating the stability of a slope.

Rock mass parameters

The distribution of a parameter within the rock mass is not relevant for the SSPC system, which is applied per geotechnical unit (ch. C.2), and is not further discussed. The distribution of a rock mass parameter within a geotechnical unit depends on how the rock mass is subdivided into geotechnical units. A parameter within a geotechnical unit is never a single value but a certain range for a parameter is allowed. The allowed width of the range depends on the context in which the geotechnical unit is used (e.g. the risk of a slope failure), on the variation of a parameter in the rock mass, and on the experience of the observer (as discussed in ch. A.2.2). The error made in measuring a rock mass parameter within a geotechnical unit can be determined. Repeating a measurement multiple times at exactly the same location will result in a standard error of a parameter measurement. Clearly only one single location should be used as otherwise the distribution of a parameter in the geotechnical unit would be contributed to the standard error. A combination of the distribution of the parameter in the geotechnical unit and the error made in measuring the parameter is obtained if several measurements of a rock mass parameter are made all over the geotechnical unit. This is the distribution needed for a probabilistic assessment of slope stability. However, to obtain this distribution is often difficult, time consuming or impossible in many situations (as already discussed in relation with the discontinuity orientation in ch. C.3.4). Therefore, in

the SSPC system the input field data should be the characteristic value for a rock mass parameter in a geotechnical unit. Ideally, the characteristic value will be the mean value of the combined distribution of the error and the distribution of a parameter in a geotechnical unit.
For the development of a probabilistic classification system the distributions of measured rock mass parameters are, however, necessary. Therefore, during the research, multiple measurements of the same parameter in the same geotechnical unit have been done by different students and staff members. The distributions resulting from these measurements are assumed to be typical for the error distributions[108] for the measurement of a characteristic value for a particular rock mass parameter within a geotechnical unit.
Most of the distributions are normal. Some, however, are discrete or show a non-normal behaviour near limit values of the ranges allowed for a parameter. In the probability analyses the non-normal distributions and the discrete distributions are replaced by a continuous normal distribution because the differences between the obtained distributions and a normal distribution are generally small. The standard deviations of these normal distributions, either direct or expressed as a percentage of the mean (characteristic) value, are taken as the standard error[108] of the characteristic value of a rock mass parameter. Table 15 gives these standard errors. The standard errors are not exact for all geotechnical units because a geotechnical unit with a wider range of allowed values will likely also have a wider distribution of the characteristic value and thus a larger standard error. The error distributions of the characteristic values were, however, approximately identical in different rock mass types in the research area and are assumed to be representative for the error made in measuring a characteristic parameter value in all geotechnical units.

Parameters describing the geometry of the slope
The slope height and orientation have been measured as described in ch. C.2.1. These have only rarely the same values everywhere along a slope. Also an error may be made while measuring the slope geometry. The combination of the two results in a distribution, called the 'error distribution' and the standard deviation of this distribution is the standard error[108] (Table 15).

Derived parameters
The distributions of parameters derived from a parameter or combination of parameters measured in the field are established by Monte Carlo simulations. The simulations are done by randomly selecting sample data points out of the distributions of the parameter or parameters that form the basis for the derived parameter. Enough samples are used to obtain a stable 'robust' distribution for the derived parameter. Most of the resulting error distributions[108] are normal[109], but some distributions show a non-normal behaviour near limit values of the ranges allowed for a parameter, or are discrete. Such distributions are replaced by a continuous normal distribution because the differences between the obtained distributions and a normal distribution are generally small.

Visually estimated stability
The visually estimated stability is a discrete parameter (ch. C.2.2). It classifies the stability in stable or unstable with a further subdivision in future instability and present instability. The unstable classes are further subdivided in unstable with small problems and unstable with large problems. It has been established in ch. C.4.3 that the visual estimation of future instability is unreliable and therefore slopes estimated to be unstable in the future have not been used in the development of the SSPC system and are also not used for the probability analyses in this chapter. In the calculation of the relations and in the probability analyses only the difference between stable (visually estimated stability class 1) or unstable (visually estimated stability classes 4 and 5) slopes has been used. Thus, the visually estimated stability of a slope in the probability analyses can be only stable or unstable and this is assumed to be a certainty.

(108) For reason of simplicity the distribution which is a combined distribution of an error and the variation of a parameter in a geotechnical unit, is denoted with 'error distribution'. Consequently the standard deviation of the 'error distribution' is denoted by 'standard error'.

(109) A normal distribution is expected because of the Central Limits Theorem from basic statistics (Davis, 1972).

parameter		standard error[108]	note
DISTRIBUTIONS OF FIELD PARAMETERS			
dip (of slope and discontinuity planes)		2.5°	(1)
dip-direction (of slope and discontinuity planes)		5°	(1)
slope height		5 % of measured height (mean value)	(1)
characteristic discontinuity spacing		5 % of measured spacing (mean value)	(1)
intact rock strength (*irs*)		32 % of midpoint of estimated strength class in MPa	(2)
discontinuity characteristics (roughness, infill and karst)		-	(3)
DISTRIBUTIONS OF PARAMETERS DERIVED FROM FIELD PARAMETERS			
apparent friction (φ)	sliding	5°	(4)
	toppling	6°	
average spacing of discontinuities		0.28 m	(5)
spa_{mass} parameter (following Taylor)		0.003	(5)
condition of discontinuity (TC)		0.068	(3)
average condition of discontinuities parameter		0.050	(6)
weighted con_{mass} parameter		0.065	(6)

Table 15. Distributions of field and derived parameters (numbers in brackets refer to the notes in the text).

Notes on Table 15:

1 *Dip, dip-direction (of slope and discontinuities), height of slope, and characteristic discontinuity spacing*
Analyses of field data have shown that the error distributions for dip and dip-direction are normal. Standard errors for dip and dip-direction are independent from its mean value[110]. The distributions for spacing and height were also found to be normal[111]. The standard errors for spacing and height are expressed as a percentage of the mean value.

2 *Intact rock strength (irs)*
In the field intact rock strength has been estimated by using a classification scale. In ch. C.3.2.1.2 was already concluded that the average of a series of estimated strengths is usually nearer to the characteristic (mean) value than the average of a limited amount of UCS tests. Students and staff have estimated the intact rock strength in the same exposure in the same geotechnical unit (Fig. 57). The estimates of strength classes are converted to MPa by taking the midpoint of the range of the strength class (e.g. for strength class 4 with a range from 12.5 to 50 MPa the midpoint of the range is 31.25 MPa). The strength is taken equal to the boundary value if the strength was estimated to be on the boundary of two classes. The resulting distribution of the estimated strength in MPa is discrete, however, with a form which resembles a normal distribution. The standard deviations (the standard errors) are in the range from 27 to 40 % of the mean value. The distributions are not normal at the extreme classes 1 and 7. In the probability calculations an error distribution for all classes of intact rock strength estimation is assumed with a standard error of 32 % of the midpoint value or of the boundary value of two classes.

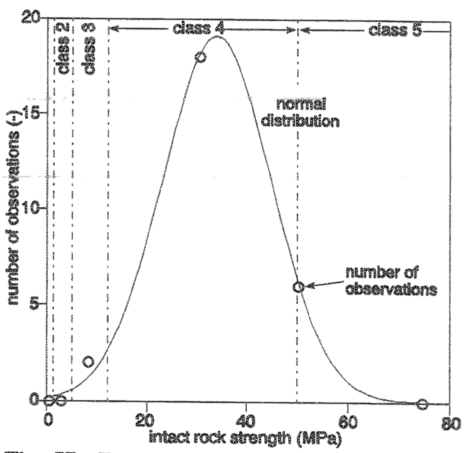


Fig. 57. Example of the distribution of *irs* estimates (made in exposure: 95/stu/2).

[110] The error in dip-direction measurement increases with decreasing dip angle. This influences the results only if the dip angle of a plane is very low. Such planes are, however, almost never a cause for slope instability.

[111] The error distribution of the 'characteristic' discontinuity spacing of the rock masses in the research area was found to be approximately normal. The distribution of the discontinuity spacings of a discontinuity set have not been investigated separately. Such a distribution is not necessarily normal (Genske, 1988, Giani, 1992).

3 *Condition of discontinuity (TC)*

A Monte Carlo simulation to determine the error in the condition of discontinuity parameter (*TC*) resulted in a standard error of 0.068. 1400 samples are generated randomly out of uniform distributions of all parameters determining the parameter *TC* (e.g. parameters for large and small scale roughness, infill and karst). On each parameter for each sample 200 disturbances (errors) are introduced randomly out of a uniform and discrete distribution of one class below until one class above the sample class, except for the classes at the extremes, for which a uniform and discrete distribution is used from one class above the minimum respectively one class below the maximum through the estimated class. The disturbances are the errors made in the description of discontinuities (ch. D.1.2.1.4). The result is for most samples a discrete distribution, which can be approximated by a normal distribution (Fig. 58). For some samples a distribution with clear preference for certain values is obtained but the distribution of all samples can still be approximated by a normal distribution. Near the extremes for *TC* (minimum: 0 and maximum: 1.0165) the distribution is not normal and is not independent of the mean *TC* value. The differences are, however, very small, certainly if is considered that the input distributions are not known in detail and are assumed to be uniform. For simplicity is assumed that the error distribution is normal around the mean *TC* value and is independent from the mean *TC* value. The average of the standard errors of all samples results in a standard error of 0.068. This value is robust and repeated simulations with newly randomized samples and disturbances resulted in maximum differences of 0.003.

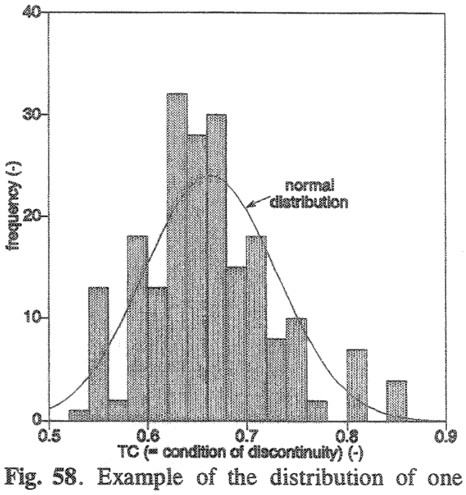


Fig. 58. Example of the distribution of one sample of *TC*.

4 *Apparent friction* φ

In the 'sliding' and 'toppling' criteria use is made of the apparent friction along discontinuity planes, φ (ch. D.1.2). The apparent friction (φ) is derived from the apparent dip of the discontinuity plane in the direction of the slope for the 'sliding' criterion (Fig. 44, ch. D.1.2.1.5) and is derived from the apparent dip of the discontinuity in the direction opposite the slope for the 'toppling' criterion (ch. D.1.2.2). To determine the possible error in φ the empirical sliding and toppling criteria are intrinsically included in the calculations. This is for sliding: $\varphi = \beta$, in which β is the apparent dip in the direction of the slope dip; and this is for toppling: $\varphi = -90° + \gamma + dip_{slope}$, in which γ is the apparent discontinuity dip in the direction opposite the slope dip. 1400 samples are generated randomly out of uniform distributions of all possible values for dips and dip-directions (for discontinuity and slope). For each sample 200 disturbances are introduced randomly out of normal distributions with standard errors of 2.5° for dips and 5° for dip-directions (Table 15, note 1). The result is a maximum standard error of about 5° for φ for sliding and 6° for toppling. The results are approximately normally distributed around the mean value and independent of the mean value. Repeated simulations with newly randomized samples and disturbances differed less than 0.5° and thus the standard errors are fairly robust. The standard errors are weakly dependent on φ and are not normally distributed near the end values of the range for φ (0° and 90°), however, the differences are less than 2° and are neglected.

5 *Average spacing of discontinuities and spa_{mass} parameter following Taylor*

The error distribution of the average spacing of discontinuities as used in the orientation independent slope stability analyses (ch. D.1.3), is normally distributed with a maximum standard error of 0.28 m. The error distribution of the spa_{mass} parameter calculated according to Taylor (eq. [13], page 76) as used in the orientation independent slope stability analyses, is normally distributed with a maximum standard error of 0.003. The results are obtained by a Monte Carlo simulation. 50 samples are randomly generated for three discontinuity sets out of uniform distributions for all possible spacings between 0 and 10 m. On each sample 50 disturbances are generated randomly out of normal distributions with standard errors of 5 % of the value (Table 15, note 1). The results for average spacing of discontinuities and for the spa_{mass} parameter calculated according to Taylor, are normally distributed and virtually independent from the average spacing or from the value of the weighted spa_{mass} parameter. Repeated simulations with newly randomized samples and disturbances result in approximately the same values for the standard errors.

6 *Average condition of discontinuities parameter and weighted con_{mass} parameter*

The error distribution of the average condition of discontinuities parameter and the weighted con_{mass} parameter as used in the orientation independent slope stability analyses (ch. D.1.3), are normally distributed with a maximum standard error of 0.050 respectively 0.065. The results are obtained by a Monte Carlo simulation. 50 samples are randomly generated for three discontinuity sets out of uniform distributions for all possible *TC* values between 0 and 1.0165. On each sample 50 disturbances are generated randomly out of the error distributions. The error distribution for the condition of discontinuity (*TC*) is normally distributed with a standard error of 0.065 (Table 15, note 3) and for the spacing of discontinuities is normally distributed with a standard error of 5 % of the value for the spacing (Table 15, note 1). Repeated simulations with newly randomized samples and disturbances give values for the standard error with differences less than 0.005.

D.2.2 Probability of orientation dependent stability

The sliding and toppling criteria are based on a boundary line below which no discontinuities in stable slopes plot. Visual determination of these boundary lines as done in ch. D.1.2 is possible but does not quantify the reliability of the lines determined. Therefore an alternative procedure has been applied to the 'sliding' and 'toppling' criteria that is discussed in the following chapters.

D.2.2.1 Probability of 'sliding criterion'

Determining boundary line

To determine the boundary line for the 'sliding criterion' (ch. D.1.2.1.5, Fig. 44) 300 sets of data points (φ, *TC*) have been generated randomly out of the original data set for discontinuities in stable slopes, with on each original data point the standard error distribution in φ and in *TC* (Table 15). A number of data points (*X*) with lowest ratio of TC/φ are determined from each set of data points. Data points with lowest TC/φ are used because the boundary line should be the lower boundary of the data set (ch. D.1.2.1). Slope and intercept of a linear regression of these *X* data points are computed for each of the 300 sets of data points, resulting in 300 regression lines. The mean and standard error of the slopes and intercepts of these 300 lines are calculated. The number of data points (*X*) used for the regression, is varied from 2 through 30. Fig. 59 illustrates the procedure for $X = 2$.

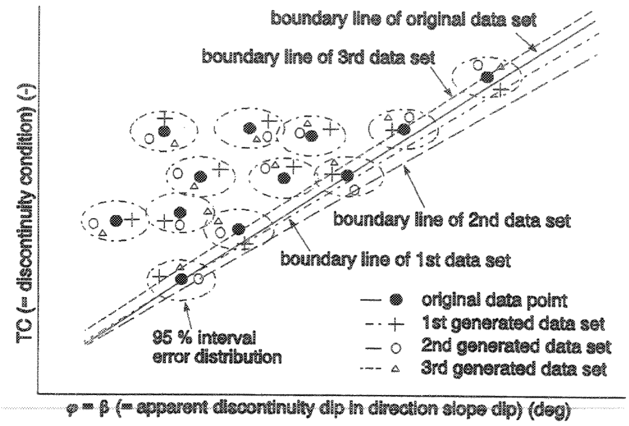


Fig. 59. Sketch showing the procedure to calculate the boundary line for the 'sliding criterion' for $X = 2$ (e.g. boundary line based on 2 data points).

The mean and standard error of the intercept and the slope of the 300 lines are shown versus *X* in Fig. 60. If 6 points are used for regression, the values for the mean intercept and mean slope become robust, e.g. change only slightly if more points are used, and the standard errors become approximately constant. The value for the mean slope coincides, as expected, with the visually determined boundary with a slope of 0.0113 in Fig. 41 (ch. D.1.2.1). Having determined the number of points (6) necessary to compute a lower boundary, the next step is to compute the reliability of this boundary.

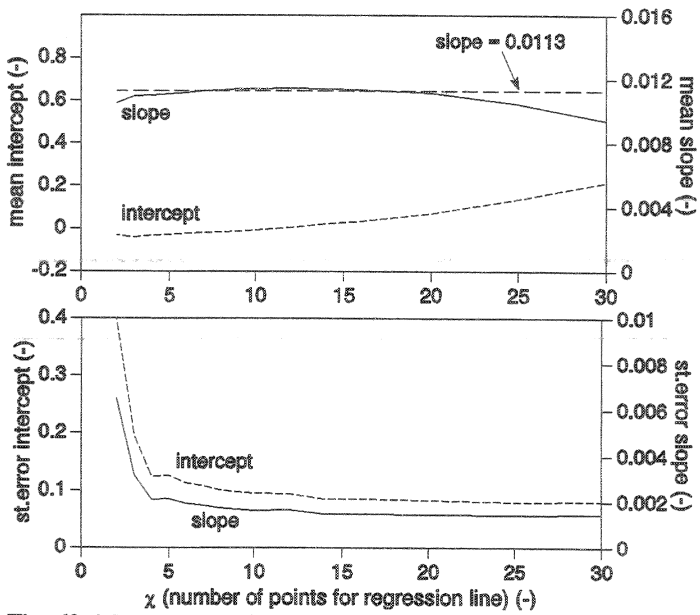


Fig. 60. Mean and standard error of intercept and slope of boundary lines vs *X*, for 'sliding criterion'.

Determining lines of equal probability

For each of the 300 regression lines (which are based on 6 data points with the lowest TC/φ ratio) the *TC* value is computed for $\varphi = 5°, 10°, 20°$, .., .., 80° and 85°. The distributions of the *TC* values for $\varphi = 5°, 10°$, etc. are determined and the cumulative probability is calculated for 5%, 30%, 50%, 70% and 95% (Fig. 61). The percentages indicate the probability that a discontinuity with a measured φ and *TC*, will not cause a slope to be unstable due to sliding over this discontinuity. The probability lines, except 50%, are clearly curved due to the lower data densities for low and high values of φ. The probability lines are fitted to second degree polynomials with correlation coefficients over 0.999. The coefficients of the polynomials are listed in Table A 18 (Appendix I).

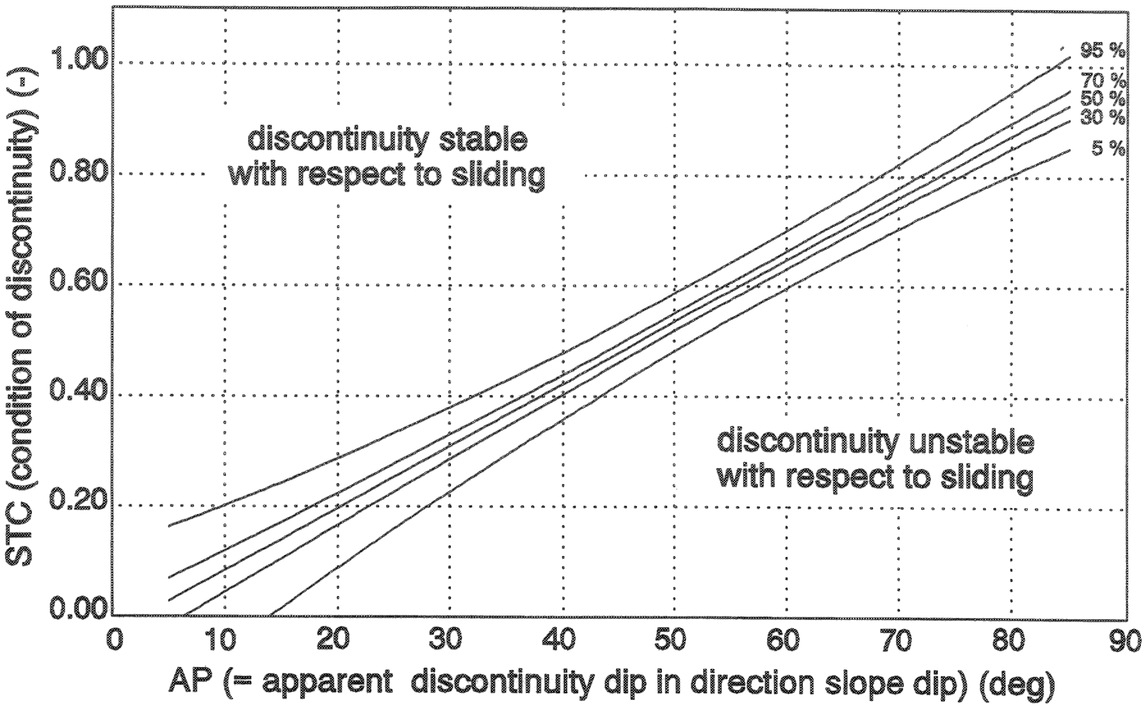


Fig. 61. Sliding probability for orientation dependent slope stability.

D.2.2.2 Probability of 'toppling' criterion

Determining boundary line

Determining the boundary line for toppling (ch. D.1.2.2.2, Fig. 46) is done in the same way as for the 'sliding criterion'. All discontinuities which kinematically allow for toppling, irrespective of the orientation according to eq. [17] (page 99) are included. In Fig. 62 the mean and standard error of intercept and slope are plotted (analogous to Fig. 60 for sliding). For 6 data points, with the lowest TC/φ ratios, the mean values of intercepts and slopes become robust and the standard errors approximately constant. The value for the mean slope coincides with the visually determined boundary with a slope of 0.0087 in Fig. 46. The mean intercept value starts to rise rapidly with more than 14 data points. Between 6 and 14 data points the value is, however, approximately constant. As the minimum number of data points is required, the increase of the mean intercept value above 14 data points is not important[112].

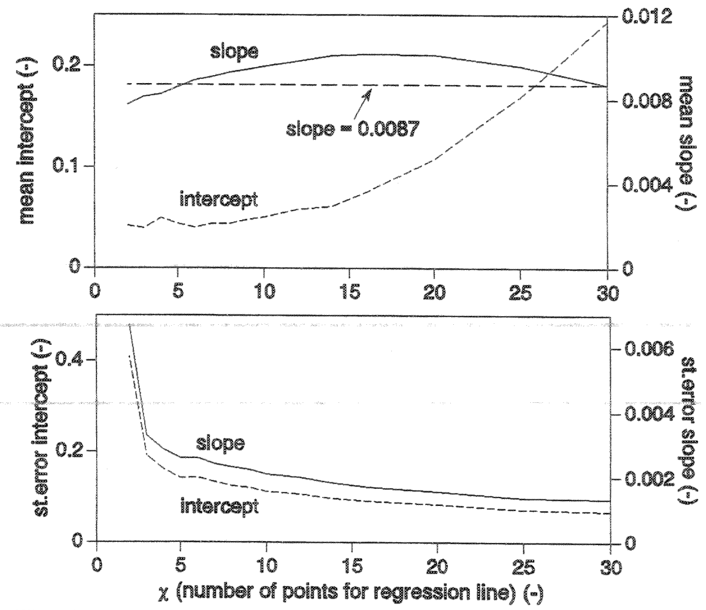


Fig. 62. Mean and standard error of intercept and slope of boundary lines vs *X*, for 'toppling criterion'.

Determining lines of equal probability

The cumulative probabilities for toppling are computed analogous to the 'sliding criterion' and polynomials are fitted. The coefficients for the polynomials are listed in Table A 18, Appendix I. The cumulative probabilities are the probability that a discontinuity in a slope is not the cause for toppling failure. The lines are plotted in Fig. 63.

(112) The more data points are used in the regression the more the line moves into the data set. If all points of the data set are used, the line is the linear regression line of the whole data set.

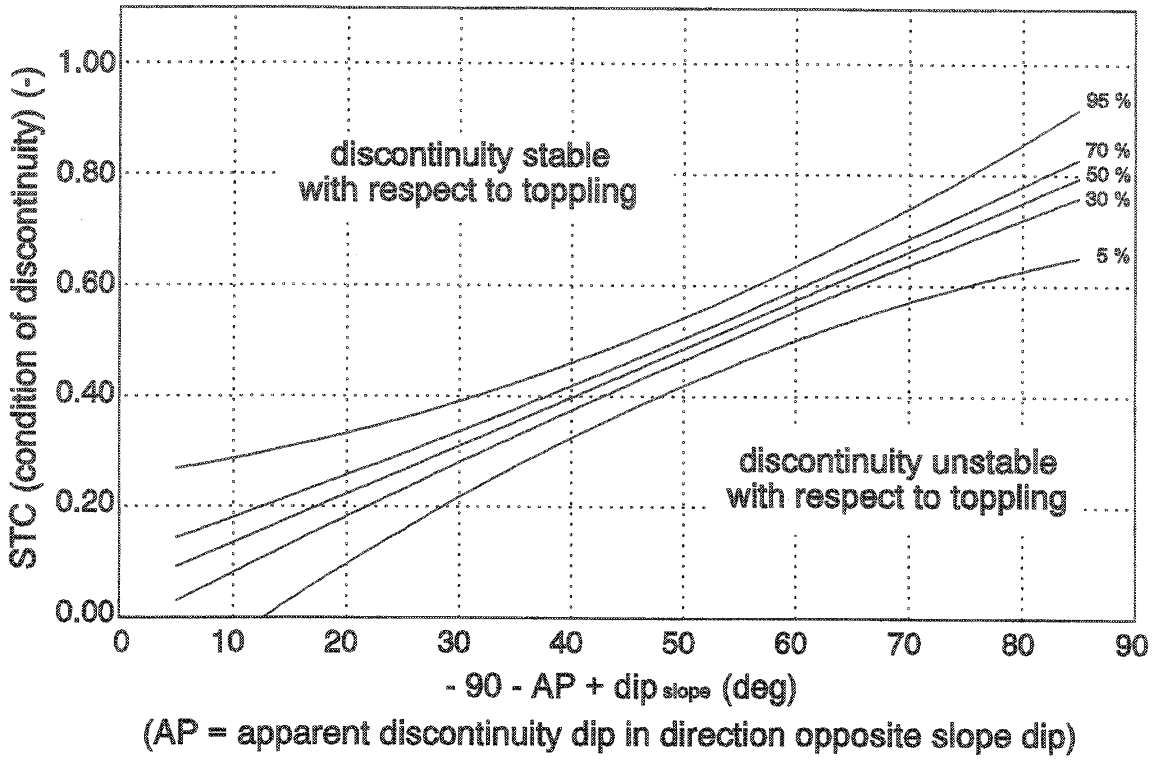


Fig. 63. Toppling probability for orientation dependent slope stability.

D.2.3 Probability of the orientation independent slope stability

The probabilities of orientation independent slope stability are calculated for the shear plane model (ch. D.1.3.5). For the linear model (ch. D.1.3.4) only the mean values and standard errors of the factors (*a0* through *a5*) are calculated.

D.2.3.1 Probability of the linear model for orientation independent slope stability

The linear model relates linearly the visually estimated stability class with the slope geometry parameters (dip_{slope}, $height_{slope}$) and the rock mass parameters (*irs*, spa_{mass} and con_{mass}). A set of these data points is generated randomly out of the original data set with on each parameter of the original data points, an error distribution. The error distributions are normal distributions with mean values 0 and standard deviations as discussed in ch. D.2.1. The visually estimated stabilities of the slopes belonging to the generated data points are the same as the visually estimated stabilities of the slopes belonging to the original data points. The factors (*a0* through *a5*) in the linear model (eq. [23], page 105) are calculated with this generated set of data points. The procedure is repeated with newly generated sets of data points, leading to new values for the factors. The mean values and standard errors of the factors belonging to all generated data sets are then calculated. New sets of data points are generated and the newly calculated factors are included in the calculation of the mean values and standard errors of the factors until the mean values and standard errors become constant. Fig. 49 and Table 9 (ch. D.1.3.4) show the resulting mean values and standard errors.

D.2.3.2 Probability of the shear plane model for orientation independent slope stability

Determining mean values and errors for weight factors of the shear plane model
A probability analysis analogous to the linear model is done for the shear plane model. Sets of data points are generated randomly out of the original data set with on each parameter (dip_{slope}, $height_{slope}$, *irs*, etc.) of the original data point an error distribution. The number of points in each of the newly generated sets is thus the same as the number of points in the original data set. The error distributions on the parameters are normal distributions with mean values 0 and standard deviations according to ch. D.2.1.

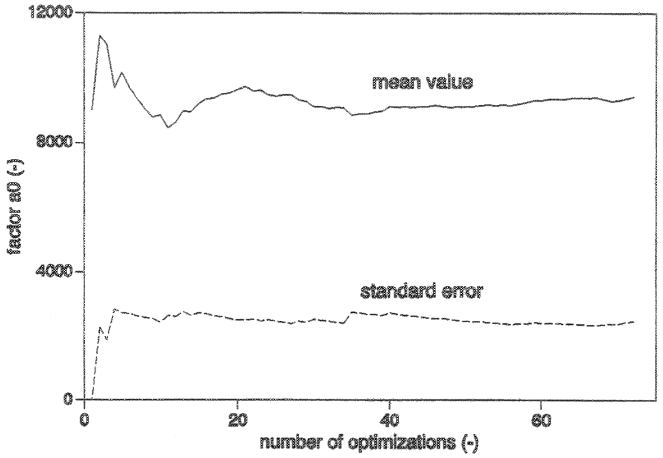


Fig. 64. Mean value and standard error for factor $a0$ in shear plane model vs number of optimizations.

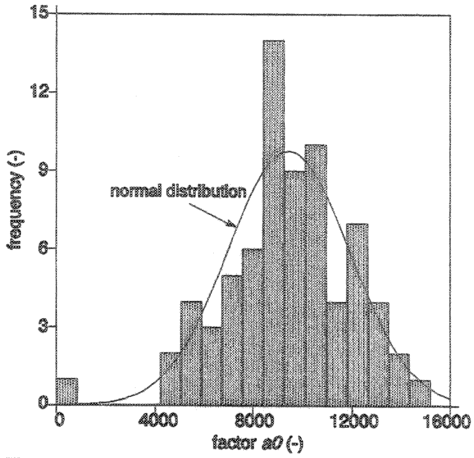


Fig. 65. Distribution of $a0$ after 72 optimizations.

The factors $a0$ through $a6$ in eqs [24] and [27] are (non-linear) optimized following eq. [28] (page 109). The procedure is repeated with newly generated sets of data points, leading to new values for the factors. The mean values and standard errors of the factors belonging to all generated data sets are calculated. New sets of data points are generated and the newly calculated factors are included in the calculation of the mean values and standard errors of the factors until the mean values and standard errors become constant. For factor $a0$ this is shown in Fig. 64 and the distribution of $a0$ is shown in Fig. 65. The mean values and standard errors of factors $a0$ through $a6$ are listed in Table 10 (ch. D.1.3.5).

Lines of equal probability

The lines of equal probability for the orientation independent stability of a slope according to the shear plane model (Fig. 67) are obtained as follows. 640,001 (j = 0 to 640,000) sample data points are randomly generated out of uniform distributions from all possible intact rock strength values (0 through 150 MPa), spa_{mass} values (0 through 1) and con_{mass} values (0 through 1.0165). dip_{slope} values are randomly generated out of the range from 10° to 90° and values for H_{slope} are randomly generated out of the range from 2 m through 25 m or 50 m[113]. For each of these sample data points (j) are calculated the $(\varphi_{mass})_j$ and $(H_{max})_j$ (following eq. [27], page 108) with the factors ($a0$ through $a6$) equal to the mean values (Table 10, ch. D.1.3.5). The ratios of $(\varphi_{mass})_j$ over $(dip_{slope})_j$ and $(H_{max})_j$ over $(H_{slope})_j$ are calculated and result in the points: $(\varphi_{mass}/dip_{slope}, H_{max}/H_{slope})_j$. φ_{mass} and H_{max} are also calculated with all the pairs $a0$ through $a6$ found in the optimization of the shear plane model (see above). For the factors $a0$ through $a6$ pairs of $a0$ through $a6$ are used, e.g. $(a0_0, a1_0, a2_0, a3_0, a4_0, a5_0, a6_0)$, $(a0_1, a1_1, a2_1, a3_1, a4_1, a5_1, a6_1)$, etc., because the factors are likely not independent. There have been calculated 72 pairs of factors $a0$ through $a6$ (i = 0 through 71) and thus for every point j are calculated 72 points i, resulting in: $(\varphi_{mass}/dip_{slope}, H_{max}/H_{slope})_{j,\,i}$. If $(\varphi_{mass}/dip_{slope})_{j,\,i} < 1$ and $(H_{max}/H_{slope})_{j,\,i} < 1$ the point

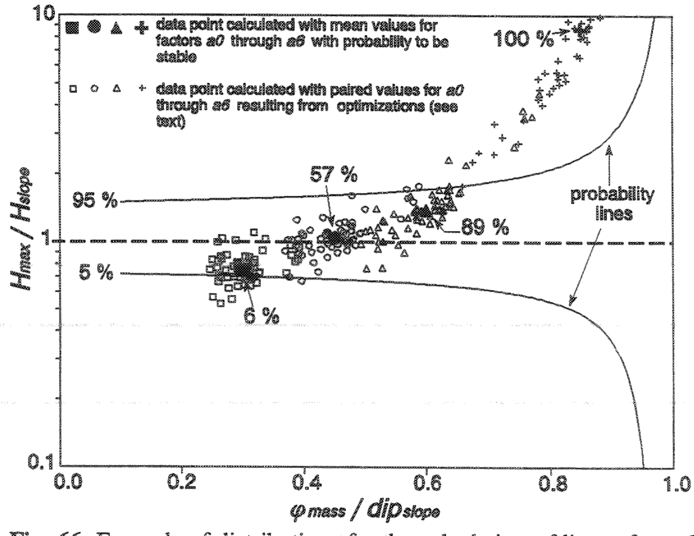


Fig. 66. Example of distributions for the calculation of lines of equal probability for orientation independent stability for the shear plane model.

[113] The calculations have been done for two ranges for the slope height: up to 25 m and up to 50 m. Most of the slopes in the research area have a height less than 25 m so that the probability lines for these slopes may be regarded as more certain than the probability lines for slopes with a height up to 50 m. Therefore the probability lines in Fig. 67 are continuous for slopes with a height up to 25 m and dashed for slopes with a height up to 50 m. For higher slopes no probabilities have been calculated as no field data are available in the research area.

represents an unstable slope whereas if $(\varphi_{mass}/dip_{slope})_{j,\,i} \geq 1$ or $(H_{max}/H_{slope})_{j,\,i} \geq 1$ the point represents a stable slope. The points representing a stable slope are counted for every point j. The total is divided by 72 and multiplied by 100 %. Hence, for every point $(\varphi_{mass}/dip_{slope},\ H_{max}/H_{slope})_j$ is thus established the percentage of the points i representing stable slopes and thus what the probability is that a point $(\varphi_{mass}/dip_{slope},\ H_{max}/H_{slope})_j$ represents a stable slope. The procedure is illustrated in Fig. 66. Curves[114] are fitted with a least squares method through the points with an equal probability of 5, 10, 30, 50, 70, 90, 95 % and are shown in Fig. 67.

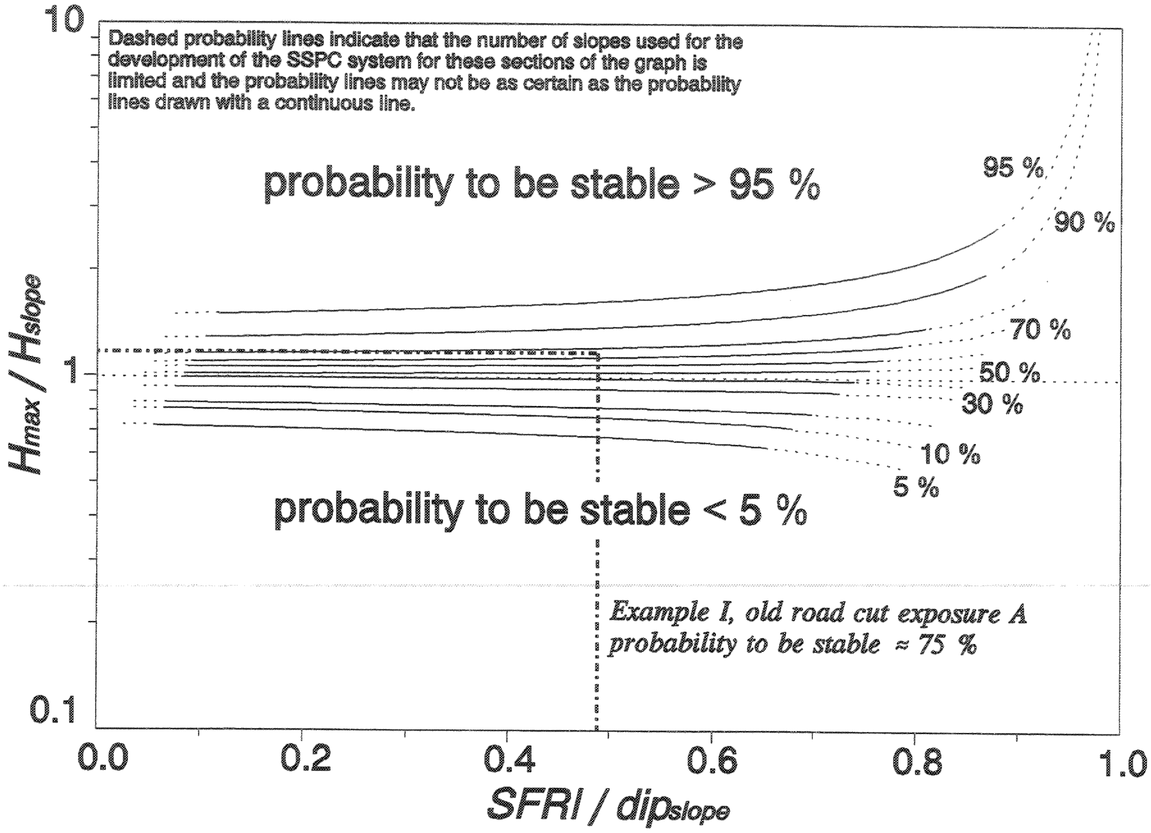


Fig. 67. Probability of orientation independent slope stability. Values indicate the probability of a slope to be stable.

D.2.3.3 Probability of the coh_{mass} and φ_{mass}

The distributions of rock mass parameters coh_{mass} and φ_{mass} of the shear plane slope stability model are used for the quantification of the error in the parameter for weathering only (ch. D.2.4.2). The error distributions of coh_{mass} and φ_{mass} are determined as follows. 401 sample data points are generated randomly out of uniform distributions from all possible intact rock strength values (*irs*, 0 through 150 MPa), from all possible spacing of discontinuities values according to Taylor (spa_{mass}, 0 through 1), and from all possible weighted condition of discontinuities values (con_{mass}, 0 through 1.0165). On each sample data point 401 disturbances are introduced out of the error distributions, which are normal distributions with mean values 0 and standard deviations conform Table 15 (page 130), giving 400 data sets. For each data point of each set coh_{mass} and φ_{mass} are calculated following eq. [29] (page 111). The mean value and error distribution are determined for each sample data point. Fig. 68 gives an example of the data set of one sample data point. The error distributions are normal distributions except for the distributions for sample data points which are calculated from values at the extremes of the range, e.g. irs = 0 MPa or spa_{mass} = 1.00, etc. For simplicity these are also

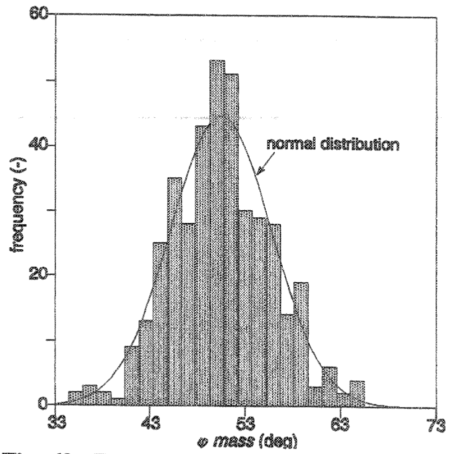


Fig. 68. Example of the distribution of one data set of φ_{mass}.

[114] The formulae and factors used for the curves have no meaning other than giving a best line representation through the points (the formulae and factors are given in appendix I, Table A 19).

assumed to be normal. The average of the standard errors of all data sets equals 7.5 % respectively 9.5 % of the mean value of coh_{mass} and φ_{mass}.

D.2.4 Probability of the values for the method of excavation and degree of weathering parameters

The probability of the values for the parameters for the method of excavation and the degree of weathering (chs. D.1.4 and D.1.5) are found by Monte Carlo simulations.

D.2.4.1 Probability of the values for the parameter of the method of excavation

The method of excavation influences only the spacing of the discontinuities (ch. D.1.4). The standard error distribution on the spa_{mass} parameter calculated according to Taylor, equals 0.003 (Table 15). The error distribution on the method of excavation is as follows. The estimation of the method of excavation is assumed to be certain for the classes 'natural', 'pneumatic hammer excavation', and 'pre-splitting/smooth wall blasting' (these can normally be easily recognized in the field). The classes for the quality of conventional blasting are subjective and the assumption is made that there is a uniform and discrete distribution from one class above until one class below the estimated method of excavation, e.g. each class has a probability of 1/3. For the classes at the extremes, 'good conventional blasting' and 'conventional blasting with result crushed rock' the distribution is uniform and discrete from one class below respectively from one class above the estimated class through the estimated class, e.g. each class has a probability of 1/2.

A Monte Carlo simulation is run with randomly generated data sets out of the original data from the field with the error distributions as described above (Table 15). Equations [33] through [37] (page 117) are calculated with those data, resulting in values for the parameter for the method of excavation. The procedure is repeated with newly generated sets of data points, leading to new values for the parameter for the method of excavation. The mean values and standard errors of the values for the parameter for the method of excavation belonging to all generated data sets are calculated. New sets of data points are generated and the values for the parameter for the method of excavation are included in the calculation of the mean values and standard errors of the values for the parameter for the method of excavation until the mean values and standard errors become constant (approximately 150 times). The results are listed in Table 12 (ch. D.1.4.2.2).

D.2.4.2 Probability of the values for the parameter of the degree of weathering

The degree of weathering influences all rock mass parameters (as discussed in ch. D.1.5). The error distributions of the intact rock strength, spacing of discontinuities and discontinuity condition (*TC*) are calculated in ch. D.2.1 and the rock mass parameters coh_{mass} and φ_{mass} in ch. D.2.3.3. For all calculations of the error in the values quantifying the influence of weathering on the different rock mass parameters, the same procedure is used. This procedure is analogous to the procedure used for the parameter for the method of excavation (ch. D.2.4.1).
The error distribution on the degree of rock mass weathering of the exposures is assumed to be uniform and discrete from one degree above until one degree below the estimated degree of weathering, e.g. each class has a probability of 1/3. For the degrees at the extremes, 'unweathered' and 'completely weathered', the distribution is uniform from one degree below respectively from one degree above the estimated degree through the estimated degree, e.g. each class has a probability of 1/2.

A Monte Carlo simulation is run with randomly generated data sets out of the original data from the field with the error distributions as described above. Equations [39] through [43] (page 121) are calculated with those data, resulting in values for the parameter for the degree of weathering. The procedure is repeated with newly generated sets of data points, leading to new values for the parameter for the degree of weathering. The mean values and standard errors of the values for the parameter for the degree of weathering belonging to all generated data sets are calculated. New sets of data points are generated and the values for the parameter for the degree of weathering are included in the calculation of the mean values and standard errors of the values for the parameter for the degree of weathering until the mean values and standard errors become constant (after approximately 100 re-calculations). The results are listed in Table 14 (ch. D.1.5.5).

D.2.5 Conclusions

The large number of field observations allowed for a probability approach of the SSPC system. The different probabilities analyses calculated in this chapter have been incorporated into the Slope Stability Probability Classification (SSPC) system as described in ch. D.3.

Generally the error distributions of the rock mass field data are conservative. It should be noted that the same observations done by experienced users of rock mass classification systems would likely result in lower errors. In the opinion of the author this is no problem as the SSPC system is likely to be used by experienced and unexperienced users. Experienced users will note that the results based on the SSPC system may be conservative and will interpret the results accordingly. It is, however, highly unlikely that an unexperienced user would be able to recognize that the results are too optimistic and be able to correct for too optimistic results. A conservatism in the results is therefore rather advantageous.

D.3 THE COMPLETE SSPC SYSTEM

The Slope Stability Probability Classification (SSPC) system is presented in this chapter[115] and its parameters, factors and calculation methods as result from the analyses in the previous chapters are described. A three-step classification system is the core of the SSPC system. The three steps consist of the characterization of the rock mass in the exposure, e.g. 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 'slope' rock mass. The stability of the slope is assessed for a slope made in the 'slope' rock mass. The 'exposure', 'reference' and 'slope' rock mass may consist of more than one geotechnical unit[116], however, the characterization and calculation of the rock masses in the three steps in the SSPC system are in principle done for each geotechnical unit separately.
The 'exposure' rock mass is first divided into geotechnical units. Then for each geotechnical unit the intact rock strength and the susceptibility to weathering, and the orientation, spacing and condition of each discontinuity (set) are determined (ch. D.3.1). The intact rock strength, and the spacing and condition of the discontinuity (sets) are converted into parameters for the 'reference' rock mass by correction for local weathering in the exposure characterized and for the damage due to the method of excavation used to make the exposure. The 'reference' rock mass describes thus 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 (ch. D.3.2).
The parameters that characterize the 'slope' 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 a new slope and taking into account future weathering. The probability of the slope to be stable is then determined with the additional parameters for the geometry of the slope. Separate probabilities are determined for orientation dependent stability and for orientation independent stability (ch. D.3.3).

The SSPC system is illustrated with a fully worked out characterization of an exposure and calculation of the reference rock mass and slope stability. The 'exposure', 'reference' and 'slope' rock mass in this example consist of one geotechnical unit only. This example is described ch. D.5.1 (example I). Blank forms and graphs for the SSPC classification system are provided in appendix VII.

(115) Fully calculated examples are presented in ch. D.5.

(116) In a geotechnical unit the geotechnical characteristics including the orientation of anisotropic features such as discontinuities, should be broadly uniform.

D.3.1 Exposure characterization

The complete exposure characterization form is presented in Fig. 71 (page 145). The entries in the form are discussed step by step in this chapter.

General information

LOGGED BY: ??		DATE: 11/04/92	TIME: 10:30 hr	exposure no: old road cut exposure A		
WEATHER CONDITIONS		LOCATION	map no:	445-iii		
Sun:	cloudy/fair/<u>bright</u>	Map coordinates:	northing:	739.940		
Rain:	<u>dry</u>/drizzle/slight/heavy		easting:	983.640		
		DIMENSIONS/ACCESSIBILITY				
		Size total exposure: (m)	l: 100	h: 9	d: 4	
		mapped on this form: (m)	l: 24	h: 9	d: 2	
		Accessibility:	poor/fair/<u>good</u>			
FORMATION NAME: tg21 limestone and dolomite						
DESCRIPTION (BS 5930: 1981)						
colour	grain size	structure & texture	weathering	NAME		
off-white	fine	medium bedded, medium blocky	slightly	calcisiltite		

The size of the exposure and the part of the exposure mapped on the form may be of help if at a later stage the significance of the description has to be determined. Accessibility and weather are recorded because experience teaches that if accessibility is poor or when the weather is poor the descriptions and measurements are less accurate.

Exposure specific parameter: Method of excavation (ME)

The classes and values for the method of excavation have been determined in ch. D.1.4.

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

Material property: Intact rock strength (IRS)

INTACT ROCK STRENGTH (IRS) (tick)		sample number(s):
< 1.25 MPa	Crumbles in hand	
1.25 - 5 MPa	Thin slabs break easily in hand	
5 - 12.5 MPa	Thin slabs broken by heavy hand pressure	
12.5 - 50 MPa	Lumps broken by light hammer blows	
50 - 100 MPa ✓	Lumps broken by heavy hammer blows	
100 - 200 MPa	Lumps only chip by heavy hammer blows (Dull ringing sound)	
> 200 MPa	Rocks ring on hammer blows. Sparks fly	

Intact rock strength is estimated with 'simple' field tests that are related to the strength classes of the British Standard (BS 5930, 1981) (ch. C.3.2.1). A standard geological hammer should be used (weight about 1 kg). A space is provided for sample numbers for intact rock strength laboratory testing. The values resulting from such testing should, however, be used with care, as discussed in ch. C.3.2.1.2.

Exposure specific parameter: Weathering (WE)

The degree of rock mass weathering is classified following British Standard (BS 5930, 1981, Table A 20, appendix V) (ch. D.1.5.7).

WEATHERING (WE) (tick)	
unweathered	1.00
slightly	✓ 0.95
moderately	0.90
highly	0.62
completely	0.35

Discontinuities: Type, orientation, spacing (DS)

DISCONTINUITIES B=bedding C=Cleavage J=joint		B1	J2	J3	4	5
Dip direction	(degrees)	110	044	002		
Dip	(degrees)	02	86	86		
Spacing (DS)	(m)	0.40	0.50	0.50		

Discontinuity sets and the type of discontinuity, e.g. B(edding), C(leavage), J(ointing), etc. are established visually and indicated in the appropriate boxes. Characteristic orientations and spacings (DS) are measured and recorded for each discontinuity set. If necessary, scanline and statistical methods are used to establish mean values, although the comments made in ch. C.3.4 concerning the accuracy of measuring methods should be considered. More forms should be used if more than five discontinuity sets are present. Single discontinuities (e.g. a single fault, etc.) are also recorded because the SSPC system can also be used for a single discontinuity to determine the probability for sliding or toppling failure. Spacing is obviously not applicable for a single discontinuity and an *S* (indicating single) is written in the appropriate space before the discontinuity set number[117].

Discontinuity property: Persistence

DISCONTINUITIES B=bedding C=Cleavage J=joint			B1	J2	J3	4	5
persistence	along strike	(m)	>24	>2	>2		
	along dip	(m)	>20	>2	>2		

Discontinuity persistence (along strike and along dip) is recorded for each discontinuity (set). A prefix indicating 'larger than' ('>') means that the discontinuity is continuous as far as visible in the exposure.

Discontinuity property: Large (Rl) and small (Rs) scale roughness

DISCONTINUITIES B=bedding C=Cleavage J=joint			B1	J2	J3	4	5
CONDITION OF DISCONTINUITIES							
Roughness large scale (Rl)	wavy slightly wavy curved slightly curved straight	:1.00 :0.95 :0.85 :0.80 :0.75	0.75	0.80	0.80		
Roughness small scale (Rs) (on an area of 20 x 20 cm^2)	rough stepped/irregular smooth stepped polished stepped rough undulating smooth undulating polished undulating rough planar smooth planar polished planar	:0.95 :0.90 :0.85 :0.80 :0.75 :0.70 :0.65 :0.60 :0.55	0.80	0.80	0.80		

The roughness of each discontinuity (set) is visually estimated according to Fig. 69 for large scale roughness (on an area > 20 x 20 cm^2) and Fig. 70 for small scale roughness (on an area $\leq$ 20 x 20 cm^2). The tactile roughness classes, e.g. rough, smooth and polished, are established by touch. If the discontinuity roughness is anisotropic (e.g. ripple marks, striation, etc.) the roughness is estimated both perpendicular and parallel to the direction with the maximum roughness. The directions are noted on the form.
If roughness profiles of both discontinuity sides are non-fitting (ch. C.3.3.2.6), this is noted on the form. The reduction of the friction along the discontinuity plane that is expected due to non-fitting may be estimated and samples for tilt or shearbox tests can be taken. Considering the difficulties and uncertainties related to shearbox tests, the estimation of the reduction of the friction angle, for example, with the Rengers envelope (Rengers, 1970, 1971) and tilt tests are almost always more appropriate than shearbox tests. The estimated or determined friction angle is converted into a value for the roughness parameter by multiplying this friction angle with 0.0113[118]

[117] Sometimes a single discontinuity may be better characterized and described as a separate geotechnical unit. This may be necessary if the infill in the discontinuity is very thick. Often major faults and fault zones can be better classified as a separate geotechnical unit. The comments in ch. C.3.4.1 can be used as guidelines to decide whether to include a discontinuity in a discontinuity set or to classify a discontinuity as a separate geotechnical unit.

[118] Use is made of the 'sliding criterion' (ch. D.1.2.1).

if the estimated or determined friction angle is determined for the large scale roughness (*Rl*) only, or if the angle is determined for both the large and small roughness combined (*Rl* and *Rs* combined). The friction angle is multiplied with 0.0151 if the friction angle is only applicable to the small scale roughness (*Rs*). A separate value for the parameter for infill material is not required if the estimated or determined friction angle is applicable to the discontinuity including the influence of infill material, for example, if a tilt test has been done with infill material present. The same applies if karst is present.

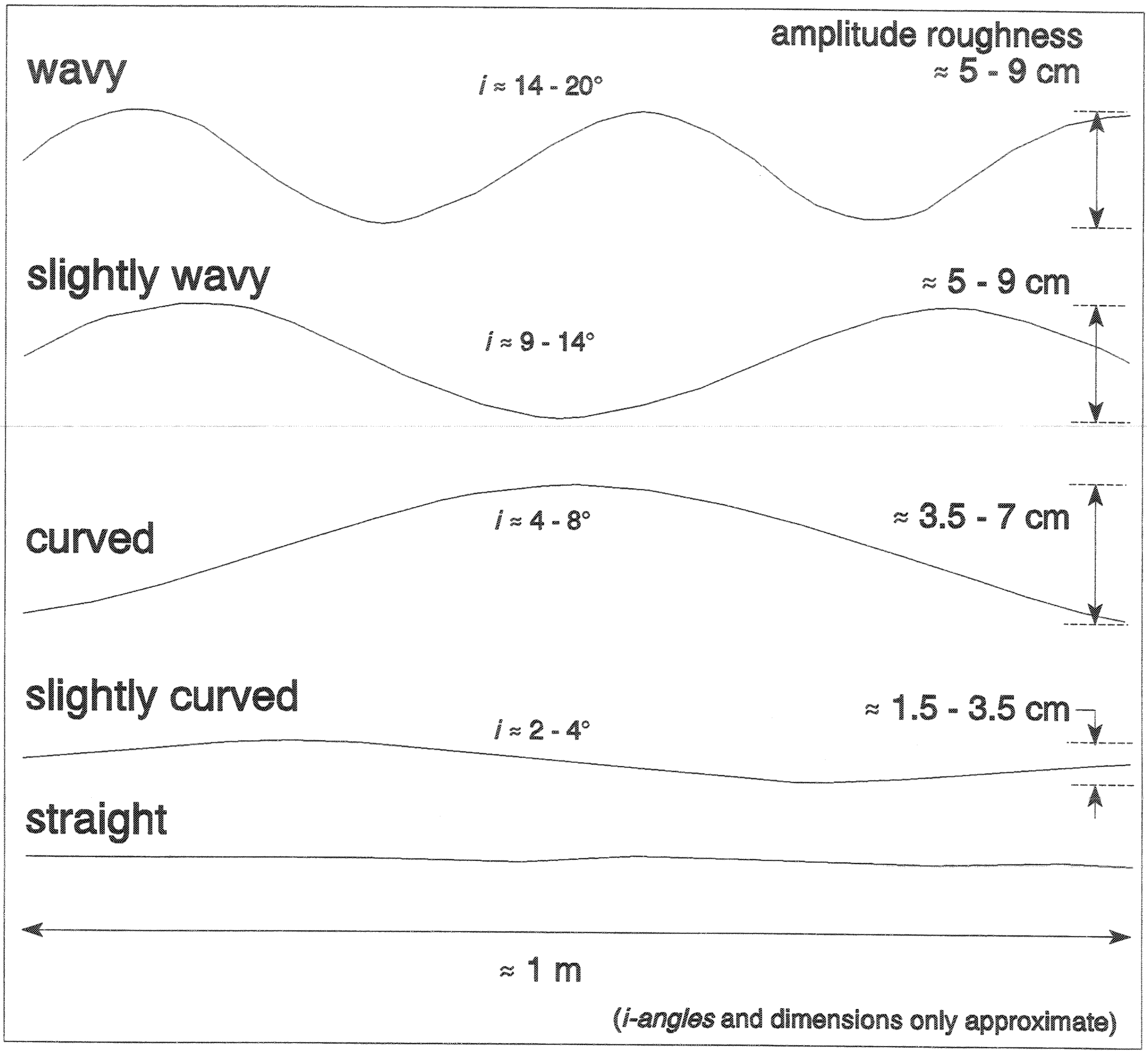


Fig. 69. Large scale roughness profiles used for the slope stability probability classification (SSPC).

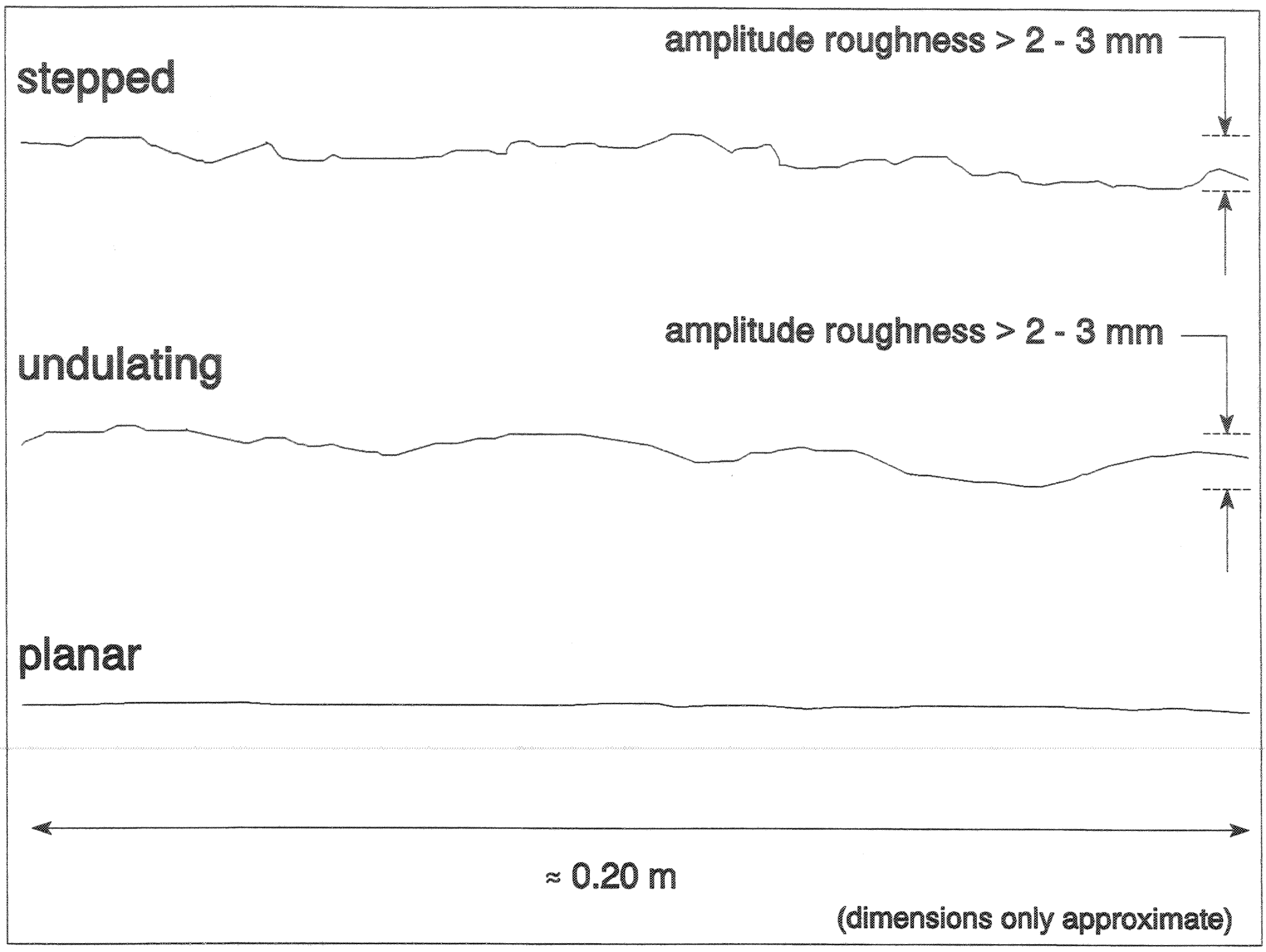


Fig. 70. Small scale roughness used for the slope stability probability classification (SSPC).

Discontinuity property: Infill material (Im)

DISCONTINUITIES B=bedding C=Cleavage J=joint		B1	J2	J3	4	5
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	1.00	1.00	1.00		

Descriptions of the infill material classes are given in ch. C.3.3.4.3. If the infill material is characterized as 'gouge > irregularities' the small scale roughness equals 0.55 (polished planar) (ch. D.1.2.1.2).

Discontinuity property: Karst (Ka)

DISCONTINUITIES B=bedding C=Cleavage J=joint		B1	J2	J3	4	5
Karst (Ka)	none :1.00 karst :0.92	1.00	0.92	0.92		

The presence of karst features should be noted for each discontinuity (ch. D.1.2.1.2).

Material property: Susceptibility to weathering (SW)

SUSCEPTIBILITY TO WEATHERING (SW)			remarks: Road cut along old road. Excavated with small (gun powder) holes.
degree of weathering:	date excavation:	remarks:	
slightly	> 40 years ago	all outcrops only slightly weathered	All existing outcrops in surroundings are slightly weathered whatever orientation.

The assessment of the susceptibility to weathering (SW) in the SSPC system may be done by noting the degree of weathering in surrounding exposures that are in the same lithologic unit, together with the length of time these exposures have existed. If special circumstances have influenced the rate of weathering in the other exposures (for example: different orientation, permanent water flow over the exposure, etc.) this should be noted in the space provided.

Existing slope ?

This information can be a reference for the reliability of the slope stability probability classification (SSPC) system. The stability classes are visually established. The description of the classes for the visual estimation of stability as used for this research can be used as guidelines (Table 5, page 52). The classes that indicate a possible likelihood for failure in the future, e.g. 'small problems in near future', class 2, and 'large problems in near future', class 3, may be difficult to distinguish.

EXISTING SLOPE?	
dip-direction/dip	
040/80	
height:	8.0 m
Stability (tick)	
stable	✓ 1
small problems in near future	2
large problems in near future	3
small problems	4
large problems	5

ITC/TU ENGINEERING GEOLOGY	exposure characterization	SSPC - SYSTEM	
LOGGED BY: zz	DATE: 11/04/92	TIME: 10:30 hr	exposure no: old road cut exposure A

WEATHER CONDITIONS		LOCATION		
Sun:	cloudy/fair/bright		map no:	445-iii
Rain:	dry/drizzle/slight/heavy	Map coordinates:	northing:	739.940
			easting:	983.640

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

DIMENSIONS/ACCESSIBILITY				
Size total exposure:	(m)	l: 100	h: 9	d: 4
mapped on this form:	(m)	l: 24	h: 9	d: 2
Accessibility:		poor/fair/good		

FORMATION NAME: tg21 limestone and dolomite

DESCRIPTION (BS 5930: 1981)				
colour	grain size	structure & texture	weathering	NAME
off-white	fine	medium bedded, medium blocky	slightly	calcisiltite

INTACT ROCK STRENGTH (IRS) (tick)		sample number(s):
< 1.25 MPa	Crumbles in hand	
1.25 - 5 MPa	Thin slabs break easily in hand	
5 - 12.5 MPa	Thin slabs broken by heavy hand pressure	
12.5 - 50 MPa	Lumps broken by light hammer blows	
50 - 100 MPa ✓	Lumps broken by heavy hammer blows	
100 - 200 MPa	Lumps only chip by heavy hammer blows (Dull ringing sound)	
> 200 MPa	Rocks ring on hammer blows. Sparks fly	

WEATHERING (WE)	
(tick)	
unweathered	1.00
slightly	✓ 0.95
moderately	0.90
highly	0.62
completely	0.35

DISCONTINUITIES B=bedding C=Cleavage J=joint			B1	J2	J3	4	5
Dip direction		(degrees)	110	044	002		
Dip		(degrees)	02	86	86		
Spacing (DS)		(m)	0.40	0.50	0.50		
persistence	along strike	(m)	> 24	> 2	> 2		
	along dip	(m)	> 20	> 2	> 2		
CONDITION OF DISCONTINUITIES							
Roughness large scale (Rl)	wavy :1.00 slightly wavy :0.95 curved :0.85 slightly curved :0.80 straight :0.75		0.75	0.80	0.80		
Roughness small scale (Rs) (on an area of 20 x 20 cm²)	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		0.80	0.80	0.80		
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		1.00	1.00	1.00		
Karst (Ka)	none :1.00 karst :0.92		1.00	0.92	0.92		

EXISTING SLOPE?	
dip-direction/dip	040/80
height:	8.0 m
Stability (tick)	
stable	✓ 1
small problems in near future	2
large problems in near future	3
small problems	4
large problems	5

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.) roughness should be assessed perpendicular and parallel to the roughness and directions noted on this form.
3) Non-fitting of discontinuities should be marked in roughness columns.

SUSCEPTIBILITY TO WEATHERING (SW)		
degree of weathering:	date excavation:	remarks:
slightly	> 40 years ago	all outcrops only slightly weathered

remarks: Road cut along old road. Excavated with small (gun powder) holes.

All existing exposures in surroundings are slightly weathered whatever orientation.

Fig. 71. Exposure characterization (example I, old road cut exposure A, see ch. D.5.1).

D.3.2 Reference rock mass

The reference rock mass is the imaginary unweathered and undamaged rock mass prior to excavation. A form for the determination of the parameters characterizing a geotechnical unit in the reference rock mass is presented in Fig. 72 (page 148) and filled in for example I also used for the exposure characterization (see also ch. D.5.1).

General information

REFERENCE UNIT NAME: *tg21, limestone and dolomite, medium bedded*

The unit name should be such that it clearly identifies the lithology and any other characteristic of the unit that may be important for the susceptibility to weathering.

Material property: Reference intact rock strength (RIRS)

INTACT ROCK STRENGTH (RIRS)	
If IRS > 132 MPa then RIRS = 132 else RIRS = IRS (in MPa) / WE (correction for weathering) = *75 / 0.95* =	*79*

The reference intact rock strength (RIRS) equals the intact rock strength (IRS) up to a maximum of 132 MPa and below that value it is divided by the parameter for the degree of rock mass weathering at the location of the exposure (WE) [119].

Discontinuities: Reference overall discontinuity spacing (RSPA)

DISCONTINUITY SPACING (RSPA)								
DISCONTINUITIES		1	2	3	4	5	SPA (see figure) = factor1 * factor2 * factor3 = *0.71* * *0.70* * *0.67* = *0.33* corrected for weathering and method of excavation: RSPA = SPA / (WE * ME) RSPA = *0.33* / (*0.95* * *0.99*) =	*0.35*
Dip direction	(degrees)	*110*	*044*	*002*				
Dip	(degrees)	*02*	*86*	*86*				
Spacing (DS)	(m)	*0.40*	*0.50*	*0.50*				
The spacing parameter (SPA) is calculated based on the three discontinuity sets with the smallest spacings following figure:								

The three discontinuity sets with the smallest spacings are used to calculate the spacing factor using eq. [45] or the figure included on the reference rock mass calculation form (Fig. 72). The resulting $factor_1$, $factor_2$ and $factor_3$ are multiplied and give SPA.

For a rock mass with one discontinuity set:

$$factor_1 = 0.45 + 0.264 * \log_{10} x \qquad factor_2 = factor_3 = 1$$

with two discontinuity sets:

$$factor_1 = 0.38 + 0.259 * \log_{10} x_{minimum} \qquad factor_2 = 0.28 + 0.300 * \log_{10} x_{maximum}$$

$$factor_3 = 1 \tag{45}$$

with three discontinuity sets:

$$factor_1 = 0.30 + 0.259 * \log_{10} x_{minimum} \qquad factor_2 = 0.20 + 0.296 * \log_{10} x_{intermediate}$$

$$factor_3 = 0.10 + 0.333 * \log_{10} x_{maximum}$$

$SPA = factor_1 * factor_2 * factor_3$

(x = *discontinuity spacing in cm of the three discontinuity sets with the smallest spacings*)

A correction for the method of excavation (ME) and for the degree of rock mass weathering (WE) [119] is applied to determine the overall discontinuity spacing of the geotechnical unit in the reference rock mass: RSPA = SPA / (ME * WE).

[119] Correction for weathering is not necessary for 'soil type' units, e.g. cemented soils, etc. (see glossary, page 241) as the mechanical parameters of 'soil type' units seem not to be influenced by weathering (ch. D.1.5). Therefore, for these type of units the parameters for the degree of weathering in the exposure (WE) and at the location of the slope (SWE) equal 1.00 in the calculations of the reference rock mass and the slope stability.

Discontinuity property: Reference condition of each discontinuity (set) (RTC) and Reference overall condition of discontinuities (RCD)

CONDITION OF DISCONTINUITIES (RTC & RCD)						
Discontinuity:	B1	J2	J3	4	5	
Roughness large scale (Rl)	0.75	0.80	0.80			RTC is the discontinuity condition of a single discontinuity (set) in the reference rock mass corrected for discontinuity weathering. RTC = TC / sqrt(1.452 -1.220 * e^(-WE))
Roughness small scale (Rs)	0.80	0.80	0.80			
Infill material (Im)	1.00	1.00	1.00			
Karst (Ka)	1.00	0.92	0.92			
Total (Rl*Rs*Im*Ka = TC)	0.60	0.59	0.59			
RTC	0.61	0.60	0.60			

Weighted by spacing:

$$CD = \frac{\frac{TC1}{DS1} + \frac{TC2}{DS2} + \frac{TC3}{DS3}}{\frac{1}{DS1} + \frac{1}{DS2} + \frac{1}{DS3}} = \frac{\frac{0.60}{0.40} + \frac{0.59}{0.50} + \frac{0.59}{0.50}}{\frac{1}{0.40} + \frac{1}{0.50} + \frac{1}{0.50}} = 0.59$$

corrected for weathering RCD = CD / WE = 0.59 / 0.95 = 0.62

The condition of discontinuity (*TC*) for each discontinuity (set) is a multiplication of the parameters for large (*Rl*) and small (*Rs*) scale roughness, infill material (*Im*) and karst (*Ka*): $TC = Rl * Rs * Im * Ka$[120]. The condition of discontinuity in the reference rock mass for each discontinuity (set) (RTC), is the condition of discontinuity (*TC*) corrected for the degree of weathering in the exposure. The correction parameter for the degree of weathering should be the correction parameter for the condition of a single discontinuity (set):
RTC = TC / sqrt(1.452 - 1.220 e^{-WE}) [119] (ch. D.1.5.7).
No distinction is made between continuous and abutting discontinuities. Non-persistent discontinuities (thus discontinuities ending in intact rock) are characterized by changing the parameter for the discontinuity small scale roughness to 'rough stepped/irregular' (= 0.95) (ch. D.1.2.1).
CD is the weighted overall condition of a number of discontinuity sets in the exposure rock mass unit. RCD equals CD divided by the parameter for the degree of rock mass weathering (WE) [119].

Anisotropic roughness
The calculation of TC and RTC should be done for the minimum roughness and for the maximum roughness if the roughness is anisotropic. The condition of the discontinuities in the reference rock mass (RCD) and the reference rock mass friction (RFRI) and cohesion (RCOH), should be calculated with the average of minimum and maximum roughness.

Reference rock mass friction and cohesion (RFRI & RCOH)

REFERENCE UNIT FRICTION AND COHESION (RFRI & RCOH)	
Rock mass friction: RFRI = RIRS * 0.2417 + RSPA * 52.12 + RCD * 5.779	
RFRI = 79 * 0.2417 + 0.35 * 52.12 + 0.62 * 5.779 =	41°
Rock mass cohesion: RCOH = RIRS * 94.27 + RSPA * 28629 + RCD * 3593	
RCOH = 79 * 94.27 + 0.35 * 28629 + 0.62 * 3593 =	19875 Pa

Rock mass friction and rock mass cohesion are calculated according to the formulae on the form.

D.3.2.1 Determination of number of geotechnical units in a reference rock mass

In the exposures the rock mass is divided in geotechnical units and the parameters of each geotechnical unit are described. After correction for the degree of weathering and for the method of excavation, parameters are determined that characterize each geotechnical unit in the reference rock mass. However, if the differences between the geotechnical units in the exposure(s) are caused only by a different degree of weathering or a different method of excavation then after correction for the degree of weathering and the method of excavation, these can be combined in one geotechnical unit in the reference rock mass. A form of averaging is necessary because the

(120) The parameter determined with the testing of discontinuities including infill material or karst or both (ch. D.3.1) already includes the influence of these parameters.

values for parameters obtained from a number of different exposures or zones in one exposure and will usually not be exactly the same. As a result the number of geotechnical units in the reference rock mass is not necessarily the same as the number of geotechnical units in the exposures but may be smaller.

ITC/TU ENGINEERING GEOLOGY	reference rock mass calculation	SSPC - SYSTEM
CALCULATED BY: zz	DATE: 11/04/92	exposure no: old road cut exposure A

REFERENCE UNIT NAME: tg21, limestone and dolomite, medium bedded

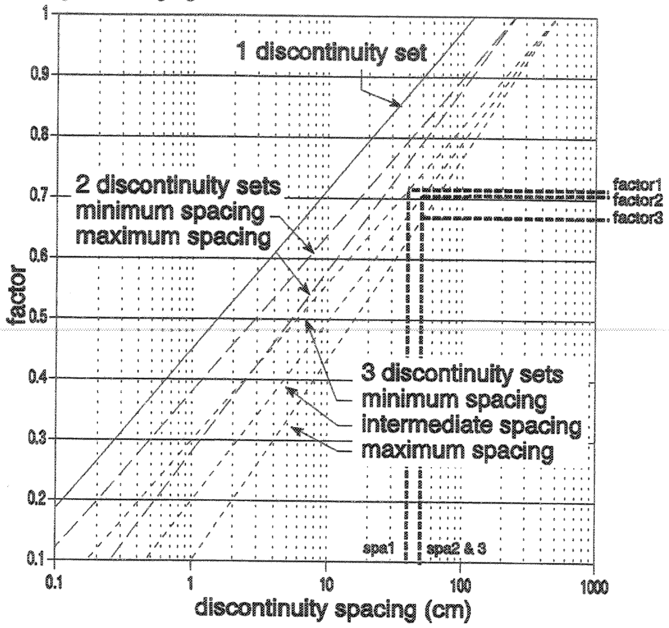
INTACT ROCK STRENGTH (RIRS)

If IRS > 132 MPa then RIRS = 132 else RIRS = IRS (in MPa) / WE (correction for weathering) = 75 / 0.95 = 79

DISCONTINUITY SPACING (RSPA)

DISCONTINUITIES		B1	J2	J3	4	5
Dip direction	(degrees)	110	044	002		
Dip	(degrees)	02	86	86		
Spacing (DS)	(m)	0.40	0.50	0.50		

The spacing parameter (SPA) is calculated based on the three discontinuity sets with the smallest spacings following figure:



SPA (see figure) = factor1 * factor2 * factor3 =
0.71 * 0.70 * 0.67 = 0.33
corrected for weathering and method of excavation:
RSPA = SPA / (WE * ME)
(with a maximum of 1.00)
RSPA = 0.33 / (0.95 * 0.99) = 0.35

CONDITION OF DISCONTINUITIES (RTC & RCD)

DISCONTINUITIES		B1	J2	J3	4	5
Roughness large scale	(Rl)	0.75	0.80	0.80		
Roughness small scale	(Rs)	0.80	0.80	0.80		
Infill material	(Im)	1.00	1.00	1.00		
Karst	(Ka)	1.00	0.92	0.92		
Total	(Rl*Rs*Im*Ka = TC)	0.60	0.59	0.59		
RTC		0.61	0.60	0.60		

RTC is the discontinuity condition of a single discontinuity (set) in the reference rock mass corrected for discontinuity weathering.
RTC = TC / sqrt(1.452 -1.220 * e^(-WE))

Weighted by spacing:

$$CD = \frac{\frac{TC1}{DS1} + \frac{TC2}{DS2} + \frac{TC3}{DS3}}{\frac{1}{DS1} + \frac{1}{DS2} + \frac{1}{DS3}} = \frac{\frac{0.60}{0.40} + \frac{0.59}{0.50} + \frac{0.59}{0.50}}{\frac{1}{0.40} + \frac{1}{0.50} + \frac{1}{0.50}} = 0.59$$

corrected for weathering: RCD (with a maximum of 1.0165) = CD / WE = 0.59 / 0.95 = 0.62

REFERENCE UNIT FRICTION AND COHESION (RFRI & RCOH)

Rock mass friction: RFRI = RIRS * 0.2417 + RSPA * 52.12 + RCD * 5.779

RFRI = 79 * 0.2417 + 0.35 * 52.12 + 0.62 * 5.779 = 41 °

Rock mass cohesion: RCOH = RIRS * 94.27 + RSPA * 28629 + RCD * 3593

RCOH = 79 * 94.27 + 0.35 * 28629 + 0.62 * 3593 = 19875 Pa

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 soil, 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. 72. Reference rock mass calculation (example I, old road cut exposure A, see ch. D.5.1).

D.3.3 Slope stability probability

The stability of a new slope is assessed with the new slope made in the 'slope' rock mass. The parameters characterizing the geotechnical units in the 'slope' rock mass are obtained by correcting the parameters characterizing the geotechnical units in the 'reference' rock mass for the damage due to the method of excavation to be used for a new slope and are corrected for the decay of the rock mass due to future weathering[121]. This latter is achieved by estimating or guessing the degree of weathering of the geotechnical unit in the slope rock mass at the end of the engineering lifetime of the new slope. The probability of the slope to be stable is then calculated for a slope made in this 'slope' rock mass. For each geotechnical unit in the slope rock mass the orientation dependent and orientation independent stability are determined. The orientation dependent stability assesses for each of the discontinuity (sets) the probability for sliding and toppling along that discontinuity or discontinuity set, and the orientation independent stability assesses the probability of a slope to be stable with respect to failure mechanisms that are not directly related to a discontinuity. The form to calculate the slope stability probability is presented in Fig. 74 (page 153), with the data for the slope of example I that has also been used for the explanation of the exposure characterization and reference rock mass calculation.

General information

ITC/TU ENGINEERING GEOLOGY		slope stability probability		SSPC - SYSTEM
LOGGED BY: *zz*	DATE: *11/04/92*			slope no: *old road cut exposure A*
		LOCATION	map no:	*445-iii*
		Map coordinates:	northing:	*739.940*
			easting:	*983.640*

Slope geometry

DETAILS OF SLOPE	
	Slope dip direction (degrees): *040*
	Slope dip (degrees): *80*
	Height (Hslope) (m): *8.0*

The SSPC system can only be used for a slope of which dip, dip-direction and height are broadly uniform. This means that if a slope is curved laterally, e.g. the dip direction of the slope is varying, the stability of the slope has to be assessed in different vertical sections where in each section the dip-direction is broadly uniform. The same applies if a slope dip or height changes laterally along a slope. If the slope dip changes vertically the slope should be assessed in different horizontal sections for which the slope dip is broadly uniform. The height of the slope is the height from the bottom of the section assessed to the top of the slope. It may also be necessary to divide the slope in horizontal sections and to determine the slope stability per section if the slope is benched (ch. C.2.1). If more than one geotechnical unit is present at the location of the slope (thus in the 'slope' rock mass) then the stability of the slope should be calculated per geotechnical unit. The height of the slope is taken as the height from the bottom of the geotechnical unit assessed to the top of the slope.

Slope specific parameters: Method of excavation (SME) and degree of weathering (SWE)

DETAILS OF SLOPE				
METHOD OF EXCAVATION (SME)		WEATHERING (SWE)		
(tick)		(tick)		
natural/hand-made	1.00	unweathered	1.00	
pneumatic hammer excavation	0.76	slightly	✓ 0.95	
pre-splitting/smooth wall blasting	✓ 0.99	moderately	0.90	
conventional blasting with result:		highly	0.62	
good	0.77	completely	0.35	
open discontinuities	0.75			
dislodged blocks	0.72	note: SWE = 1.00 for 'soil type' units, e.g.		
fractured intact rock	0.67	cemented soil, etc.		
crushed intact rock	0.62			

The method of excavation which is going to be used for the new slope (SME) and the degree of weathering of the rock mass at the location of the slope (SWE) that is expected at the end of the engineering lifetime of the slope, according to the British Standard (BS 5930, 1981) (appendix V, Table A 20) for rock mass weathering, are noted.

[121] If an existing slope is assessed the value for the parameter for the method of excavation of the slope (SME) is equal to the value for the parameter for the method of excavation of the exposure (ME). The same applies for the value for the parameter for the degree of weathering (SWE = WE).

Slope unit name

SLOPE UNIT NAME: *tg21, limestone and dolomite, medium bedded*

Orientation independent stability

Material property: Slope intact rock strength (SIRS)

INTACT ROCK STRENGTH (SIRS)	
SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = *79* * *0.95* =	*75*

The slope intact rock strength (SIRS) equals the reference intact rock strength (RIRS) multiplied by the parameter for rock mass weathering at the location of the slope[122] (SWE) [119][123].

Discontinuity: Slope overall discontinuity spacing (SSPA)

DISCONTINUITY SPACING (SSPA)	
SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope)	
SSPA = *0.35* * *0.95* * *0.99* =	*0.33*

The overall discontinuity spacing parameter for the slope is determined by multiplying the reference overall discontinuity spacing (RSPA) by the parameter for the method of excavation for the new slope (SME) and by the parameter for rock mass weathering at the location of the slope[122] (SWE) [119].

Discontinuity property: Slope overall condition of discontinuities (SCD)

CONDITION OF DISCONTINUITIES (SCD)	
SCD = RCD (from reference rock mass) * SWE (weathering slope)	
SCD = *0.62* * *0.95* =	*0.59*

The slope overall condition of discontinuities (SCD) equals the reference overall condition of discontinuities (RCD) multiplied by the parameter for rock mass weathering at the location of the slope[122] (SWE) [119].

Rock mass friction and cohesion (SFRI & SCOH)

SLOPE UNIT FRICTION AND COHESION (SFRI & SCOH)	
Rock mass friction: SFRI = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779	
SFRI = *75* * 0.2417 + *0.33* * 52.12 + *0.59* * 5.779 =	*39* °
Rock mass cohesion: SCOH = SIRS * 94.27 + SSPA * 28629 + SCD * 3593	
SCOH = *75* * 94.27 + *0.33* * 28629 + *0.59* * 3593 =	*18638* Pa

The rock mass friction and rock mass cohesion for the slope are calculated according to the formulae on the form.

(122) The existing degree of rock mass weathering of the rock mass at the location of the slope should be used if the stability of an existing slope is assessed. The degree of rock mass weathering that is expected to exist at the end of the engineering lifetime of a new slope is to be used if the stability of a new slope is assessed.

(123) A problem can arise if the stability of an existing slope is determined. The maximum of the intact rock strength for the *reference rock mass* of a value at 132 MPa causes that for an existing slope the intact rock strength could become lower than the intact rock strength measured in the rock mass of an existing slope. To avoid this problem SIRS should be taken equal to the intact rock strength as measured and described on the 'exposure characterization form' with a maximum of 132 MPa.

Probability to be stable for orientation independent stability

If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (Hmax)	
Maximum possible height: Hmax = 1.6 * 10^{-4} * SCOH * sin(slope dip) * cos(SFRI) / (1-cos(slope dip - SFRI))	
Hmax = 1.6 * 10^{-4} * 18638 * sin(80 °) * cos(39 °) / (1-cos(80 ° - 39 °)) =	9.3 m
ratios: SFRI / slope dip = 39 ° / 80 ° =	0.49
Hmax / Hslope = 9.3 m / 8.0 m =	1.2
Probability stable: if SFRI > slope dip probability = 100 % else use figure for orientation independent stability:	75 %

If the slope rock mass friction (SFRI) is smaller than the dip of the slope the maximum possible height (H_{max}) for the slope is calculated and ratios are determined of SFRI / slope dip and Hmax / Hslope. The probability to be stable for orientation independent stability is determined with Fig. 67 (page 136). The probability to be stable is 100 % if the rock mass friction (SFRI) is larger than the dip of the slope. For the slope in example I the resulting probability to be stable for orientation independent stability is about 75 %, which agrees with reality as no major problems with the stability of the slope have been noted and for any practical application the slope would be considered stable (ch. D.5.1).

Orientation dependent stability

Apparent discontinuity dip

ORIENTATION DEPENDENT STABILITY						
DISCONTINUITIES		B1	J2	J3	4	5
Dip direction	(degrees)	110	044	002		
Dip	(degrees)	02	86	86		
With, Against, Vertical or Equal		w	v	v		
AP	(degrees)	01	86	85		

The apparent discontinuity dip in the direction of the slope and the relation between orientation of the slope and the orientation of the discontinuity, e.g. W(ith), A(gainst), V(ertical), or E(qual), are determined according to the formulae included on the form.

Discontinuity property: Slope condition of discontinuity (STC)

ORIENTATION DEPENDENT STABILITY					
DISCONTINUITIES	B1	J2	J3	4	5
RTC (from reference form)	0.61	0.60	0.60		
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))	0.60	0.59	0.59		

The condition of a discontinuity is determined by multiplying the reference condition of a discontinuity (RTC) by the value for the parameter for the degree of weathering that is expected at the end of the engineering lifetime of the slope. The weathering parameter for a discontinuity (set) should be used,
e.g. STC = RTC * (1.452 - 1.220 e^{-SWE}) (119) and not the parameter for weathering of the entire rock mass.

Probability to be stable for sliding and toppling

ORIENTATION DEPENDENT STABILITY					
DISCONTINUITIES	ß1	J2	J3	4	5
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))	0.60	0.59	0.59		
Probability stable:	> 95 %	100 %	100 %		

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: δ = σ - τ: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling
AP > 84° or AP < -84°	vertical	100 %	100 %
(slope dip + 5°) < AP < 84°	with	100 %	100 %
(slope dip-5°) < AP < (slope dip + 5°)	equal	100 %	100 %
0° < AP < (slope dip-5°)	with	use graph sliding	100 %

	stability:	sliding	toppling
AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling

Depending on the orientation of the discontinuity in relation with the orientation of the slope and the value of STC the probability to be stable is determined for each discontinuity (set) following the rules at the bottom of the form. The graphs refer to Fig. 61 page (133) for sliding and to Fig. 63 (page 134) for toppling. For the slope of example I there are no discontinuities orientated in such a way that they allow a toppling failure and the dip of the discontinuity set that is dipping in the same direction as the slope is nearly horizontal, so sliding along this discontinuity set will not occur. The resulting probabilities for the slope to be stable are over 95 % and no problems are to be expected for orientation dependent stability. This slope had been assessed to be stable during the slope description in the field (ch. D.5.1).

Notes:

Estimation of future weathering
A probability for the susceptibility to future weathering could not be determined because quantification of time in a parameter for susceptibility was impossible as described before. Alternatively the SSPC system incorporates the estimation of the future degree of weathering. The importance of a correct estimation of the degree of rock mass weathering at the end of the engineering lifetime is illustrated in Fig. 73. The slope in the example is excavated in a rock mass that at the time of excavation is slightly weathered. During the lifetime of the slope the degree of rock mass weathering increases from slightly via moderately to highly weathered. The orientation independent slope stability following the SSPC system, reduces from stable (probability to be stable: 92 %) to unstable (probability to be stable: 3 %). A similar example can be given for orientation dependent slope stability.

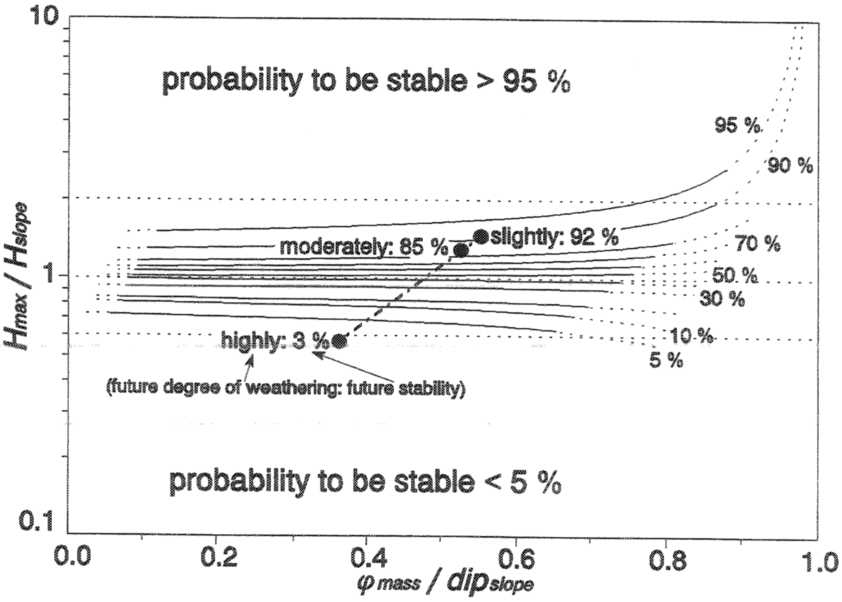


Fig. 73. Example of future orientation independent slope stability as function of the future degree of weathering.

Anisotropic roughness
If the roughness along a discontinuity (set) is anisotropic the condition of discontinuity (RTC) of the reference rock mass calculation cannot be used. For a slope stability assessment the roughness parameters for large and small scale roughness should be determined in the direction in which the displacement is expected with eq. [46].

$$roughness\ factor = mir + (mar - mir) * |\sin(\alpha)|$$

$$mir = minimum\ roughness\ factor \quad mar = maximum\ roughness\ factor$$
$$\alpha = (direction_{slope} - direction\ of\ minimum\ roughness) \qquad [46]$$

The roughness parameters (for large and small scale) calculated with eq. [46] should then be multiplied with the value for the parameter for infill material and with the value for the parameter for karst. The resulting value replaces RTC in the calculation of orientation dependent slope stability.

Unit weight$_{mass}$

The factors in the SSPC system have been optimized based on rock masses with an intact rock unit weight of 25.5 to 27 kN/m^3 and rock mass unit weights of around 25 kN/m^3. A correction of the calculated maximum possible height (H_{max}) should be applied if a rock mass unit weight is different. This correction equals 25/Unit Weight$_{mass}$ (in kN/m^3). Rock masses with a high porosity and permeability (e.g. with a large storage capacity) may exhibit a large difference between wet and dry mass unit weight.

ITC/TU ENGINEERING GEOLOGY	slope stability probability	SSPC - SYSTEM
LOGGED BY: zz	DATE: 11/04/92	slope no: old road cut exposure A

LOCATION	map no:	445-iii
Map coordinates:	northing:	739.940
	easting:	983.640

DETAILS OF SLOPE

METHOD OF EXCAVATION (SME)		WEATHERING (SWE)			
(tick)		(tick)		Slope dip direction (degrees):	040
natural/hand-made	1.00	unweathered	1.00		
pneumatic hammer excavation	0.76	slightly	✓ 0.95		
pre-splitting/smooth wall blasting	✓ 0.99	moderately	0.90	**Slope dip** (degrees):	80
conventional blasting with result:		highly	0.62		
good	0.77	completely	0.35	Height (**Hslope**) (m):	8.0
open discontinuities	0.75				
dislodged blocks	0.72	note: SWE = 1.00 for 'soil type' units,			
fractured intact rock	0.67	e.g. cemented soil, etc.			
crushed intact rock	0.62				

SLOPE UNIT NAME: tg21, limestone and dolomite, medium bedded

ORIENTATION INDEPENDENT STABILITY

INTACT ROCK STRENGTH (SIRS)

SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = 79 * 0.95 = 75

DISCONTINUITY SPACING (SSPA)

SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope)

SSPA = 0.35 * 0.95 * 0.99 = 0.33

CONDITION OF DISCONTINUITIES (SCD)

SCD = RCD (from reference rock mass) * SWE (weathering slope)

SCD = 0.62 * 0.95 = 0.59

SLOPE UNIT FRICTION AND COHESION (SFRI & SCOH)

Rock mass friction: SFRI = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779

SFRI = 75 * 0.2417 + 0.33 * 52.12 + 0.59 * 5.779 = 39 °

Rock mass cohesion: SCOH = SIRS * 94.27 + SSPA * 28629 + SCD * 3593

SCOH = 75 * 94.27 + 0.33 * 28629 + 0.59 * 3593 = 18638 Pa

If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (Hmax)

Maximum possible height: Hmax = 1.6 * 10^{-4} * SCOH * sin(slope dip) * cos(SFRI) / (1-cos(slope dip - SFRI))

Hmax = 1.6 * 10^{-4} * 18638 * sin(80 °) * cos(39 °) / (1-cos(80 ° - 39 °)) = 9.3 m

ratios: SFRI / slope dip = 39 ° / 80 ° = 0.49

Hmax / Hslope = 9.3 m / 8.0 m = 1.2

Probability stable: if SFRI > slope dip probability = 100 % else use figure for orientation independent stability: 75 %

ORIENTATION DEPENDENT STABILITY

DISCONTINUITIES		B1	J2	J3	4	5
Dip direction	(degrees)	110	044	002		
Dip	(degrees)	02	86	86		
With, Against, Vertical or Equal		w	v	v		
AP	(degrees)	01	86	85		
RTC (from reference form)		0.61	0.60	0.60		
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))		0.60	0.59	0.59		
Probability stable:		> 95 %	100 %	100 %		

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: δ = σ - τ: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling		stability:	sliding	toppling
AP > 84° or AP < -84°	vertical	100 %	100 %	AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
(slope dip+5°) < AP < 84°	with	100 %	100 %	AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling
(slope dip-5°) < AP < (slope dip+5°)	equal	100 %	100 %				
0° < AP < (slope dip-5°)	with	use graph sliding	100 %				

Fig. 74. Slope stability probability calculation (example I, old road cut exposure A, see ch. D.5.1).

D.4 RESULTS AND COMPARISON

In this chapter the stability probability of the slopes[124] to be stable in the research area is determined with the SSPC system. The stability of the slopes is also determined with two other existing classification systems for slope stability, e.g. the Haines and Romana classification systems. The results of the three classification assessments in relation with the visually estimated stability in the field are evaluated. In the second part of this chapter the merits of the rock mass cohesion and friction as calculated with the SSPC system are investigated. The 'strength' of a confined rock mass under a compressive stress is calculated with the rock mass cohesion and friction from the SSPC system, and is compared with the 'strength' of a rock mass calculated according to Bieniawski's RMR and with the 'strength' of a rock mass calculated according to the 'modified Hoek-Brown failure criterion'.

D.4.1 Slope stability

D.4.1.1 Application of SSPC system

The stability probability of the slopes in the research area is determined with the Slope Stability Probability Classification (SSPC) and the frequency distribution per visually estimated stability class is shown in Fig. 75a. The percentages of slopes that obtain a stability probability that does not agree with the visually estimated stability are around 15 % for the stable slopes (class 1) as well as for the unstable slopes (class 4 and 5 combined). Fig. 75a also shows that a large quantity of visually estimated stable slopes (class 1) obtains a stability probability of larger than 95 % with relatively few slopes being assessed as having a stability between 50 and 95 %. The same but reversed, is obtained for unstable slopes class 5 (slopes with large problems); a large quantity of these slopes has a stability probability of less than 5 % with relatively few slopes being assessed as having a stability between 5 and 50 %. The frequency distribution for unstable slopes class 4 (slopes with small problems) is more spread over the possible stability probabilities with the majority of the slopes assessed as having a stability probability of less than 50 %. The larger spread for class 4 slopes agrees with common sense, as slopes with small problems are expected to have a stability nearer to unity (which equals a stability probability of 50 %).

D.4.1.2 Application of Haines' slope classification

The slope stability classification system of Haines (Haines et al., 1991, ch. B.2.4.7) is developed to determine the design slope dip depending on the slope height for safety factors of 1.2 and 1.5. The system is based on Laubscher's MRMR rating (Laubscher, 1990, ch. B.2.3.3). Heights of the slopes in the research area range characteristically from 2 to 25 m. The average is about 8 m; the maximum is about 45 m. This is small compared to the heights of the slopes used for the development of the Haines system (Fig. 15, page 30). Therefore the influence of the height in the stability calculation according to Haines can be neglected for the slopes in the research area. The MRMR ratings are calculated with the adjustment factor for the method of excavation. This is in accordance with the Haines classification system. No adjustment for the orientation of discontinuities and slope has been used. This adjustment would cause the design slope dips following Haines' classification system to be smaller, increasing the percentage of slopes being unstable and decreasing the percentage estimated to be

[124] The slope data used in this chapter are only the slopes that have been visually assessed to be stable or unstable at present (stability classes 1, 4 and 5; Table 5, page 52). This gives a total of 184 slopes.

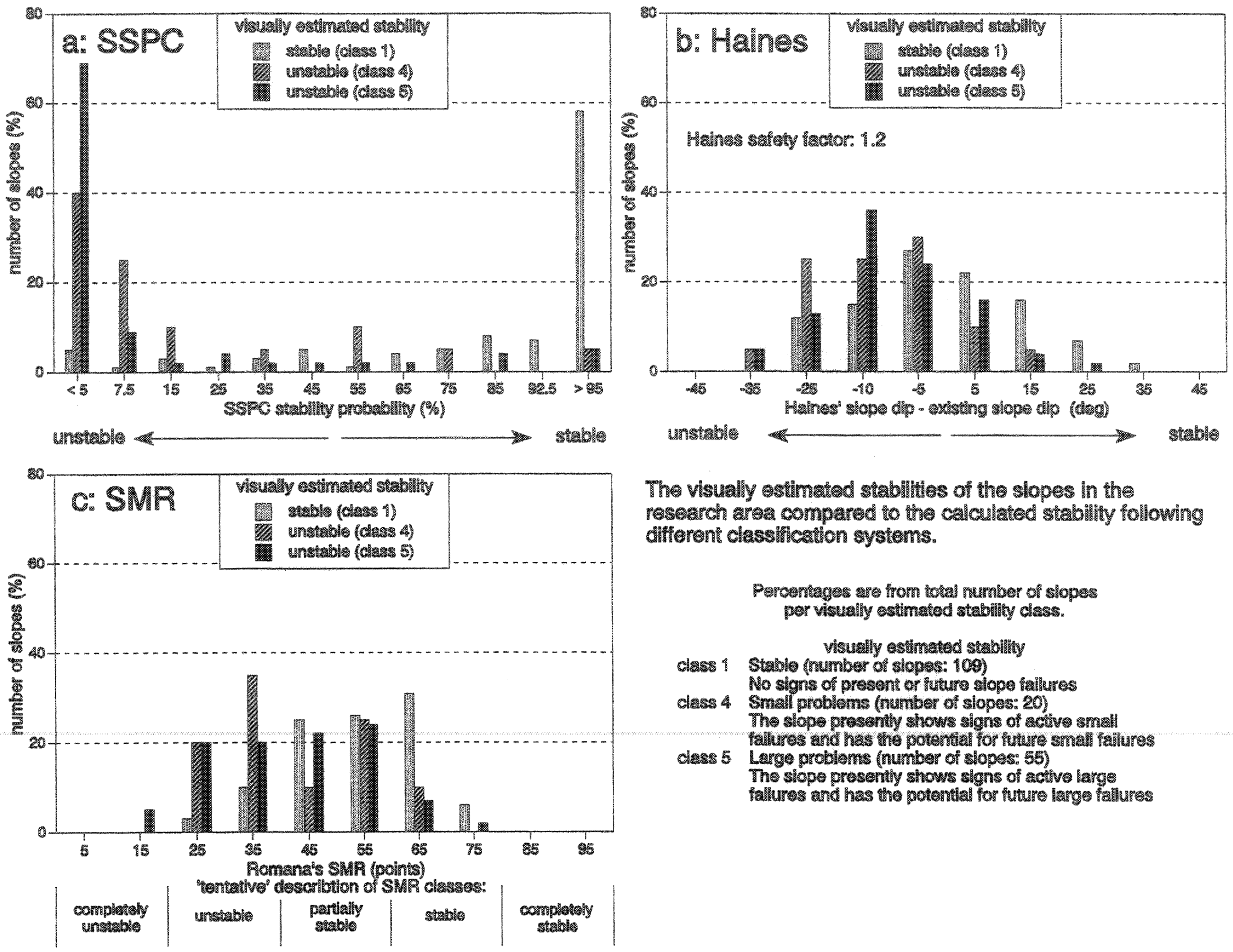


Fig. 75. Comparison of slope stability assessments by different classification systems. a: SSPC system, b: Haines' slope stability classification and c: Romana's SMR rating.

stable, and thus increasing the conservatism in the system(125). Fig. 75b shows the frequency distribution of the differences between Haines' design slope dip for a safety factor of 1.2 and the existing slope dip. Slopes with a negative difference are unstable and slopes with a positive difference are considered stable according to the slope classification system of Haines et al.. The system has been published for design slope dips with safety factors of 1.2 and 1.5 only. A comparison with the other classifications systems is thus limited as the other systems do not include a safety factor.

(125) The Laubscher MRMR rating is calculated from the data collected for the SSPC system as follows:

1) The intact rock strength estimation is used for the MRMR.
2) The MRMR rating for the RQD is calculated from the number of discontinuities per cubic metre (Jv = sum of number of discontinuities per metre from all discontinuity sets): RQD = 115 - 3.3 * Jv (%), if Jv > 4.5 then RQD = 100 % (Palmstrøm, 1975).
3) The MRMR rating for the spacing of discontinuities is calculated following eq. [45] (page 146) for a maximum of three discontinuity sets. These are the sets with a maximum negative influence on the rating value.
4) The MRMR rating for the condition of discontinuities is calculated with the discontinuity condition parameters from the discontinuity set with the minimum condition value (thus with the minimum *TC* value in the SSPC system). The values used for the different parameters (large and small scale roughness, alteration of discontinuity wall and infill) are those published by Laubscher (1990) which incorporate a maximum water influence, e.g. the values for a high pressure; > 125 litre/minute. The discontinuity set with the minimum condition value is not necessarily a discontinuity set which is used for the determination of the rating of the spacing of the discontinuities.
5) No adjustment parameters for weathering nor orientation are used but the adjustment parameter and values for the method of excavation are used as defined by Laubscher (Table 8, page 78).

D.4.1.3 Application of Romana's SMR system

The stability of the slopes in the research area is classified according to Romana's SMR system (Romana, 1985, 1991, ch. B.2.4.6). Fig. 75c shows the frequency distribution of the number of slopes versus the SMR points per visually estimated stability class. Bieniawski's RMR rating, which is used to determine the SMR rating, is calculated from the field data from the SSPC system(126). The parameter for the excavation method (F_4) has been applied according to the classification system.

D.4.1.4 Discussion

Table 16 shows, in the second column, for each of the three classification systems which percentage of the slopes that are visually estimated to be stable, are falling in this same category by using the classification systems. The third column gives the same comparison for slopes visually estimated to be unstable. Fig. 75 and Table 16 clearly show that the SSPC system is more distinctive in predicting between stable and unstable and the predictions have a better correlation with the visually estimated slope stabilities than the predictions of the other two systems for the slopes in the research area.

classification system	correctly classified stable slopes (visually estimated stability class 1) (%)	correctly classified unstable slopes (visually estimated stability class 4 & 5) (%)
SSPC	86	84
Haines	47	80
SMR	63	62

Table 16. Comparison of slope stability classification systems.

The better correlation of the visual estimated slope stabilities with the predictions from the SSPC system than with the predictions from other systems could have been due to observer bias in the visual estimations of the slope stabilities. The SSPC system has been developed using the same slopes with the same visual stability assessments and an observer bias would have resulted in a good correlation. However, as discussed before (ch. C.2.2), the visually estimated slope stabilities are very likely free of observer bias and therefore the better correlation very likely proves that the SSPC system is more reliable for predicting the stability of the slopes in the research area.

The determination of the slope dip following the Haines classification system leads to a percentage of correctly classified unstable slopes (80 %) in the same order as the percentage obtained with the SSPC system (84 %). However, the percentage of correctly classified stable slopes (47 %) is considerably less than the percentage obtained with the SSPC system (86 %). The system is thus more conservative than the SSPC system which is likely due to the incorporation of a safety factor of 1.2. Adjustments for the influence of orientation of slope and discontinuities, would even further increase the percentage correctly classified unstable slopes and further decrease the percentage correctly classified stable slopes and thus further increase the conservatism in the system.

The Haines and the SMR classification systems are calculated with parameters that represent a slope in a maximum 'wet' state for all slopes. For the SRM system this choice is likely justified as the percentage of correctly classified slopes for stable and unstable slopes is rather well balanced, 63 versus 62 %. For both systems the percentage of

(126) RMR ratings are calculated from the data collected for the SSPC system as follows:

1) The intact rock strength estimation is used for the RMR.
2) The RMR rating for the RQD is calculated from the number of discontinuities per cubic metre (Jv = sum of number of discontinuities per metre from all discontinuity sets): RQD = 115 - 3.3 * Jv (%), if Jv > 4.5 then RQD = 100 % (Palmstrøm, 1975).
3) The RMR rating for the spacing of discontinuities is calculated for the discontinuity set with the minimum spacing.
4) The RMR rating for the condition of discontinuities is taken linearly proportional to the discontinuity condition parameter *TC* (ch. D.1.2.1) from the SSPC system. The range of the values of the RMR rating for the condition of discontinuities is between 0 and 30 and the range for the *TC* parameter is from 0 to 1.0165. Hence, to obtain the rating for the condition of discontinuities for the RMR system from the *TC* value of the SSPC system the *TC* value has been multiplied with 30 / 1.0165. The discontinuity set with the minimum value for the condition of discontinuities has been used to calculate the RMR rating. This is not necessarily the same discontinuity set as the discontinuity set with the minimum spacing.
5) Parameters related to orientation of the slope and discontinuities (F_1, F_2 and F_3) are calculated for the discontinuity (set) which has a maximum adverse influence the stability.
6) The parameter for the excavation method (F_4) has been applied.
7) The RMR rating for water is for a maximum water influence ('flowing conditions'), which leads to a rating of 0 points.

correctly classified stable slopes had been increased but the percentage of correctly classified unstable slopes had been decreased if parameters had been used for a 'dry' state. It is, however, unlikely that for slopes at the terrain surface the system would ever be used with parameters for a 'dry' state as it is clear that the slopes will be regularly in a 'wet' state.

In the Haines and SMR systems the parameter for the method of excavation has been applied following the classification systems. This causes Haines' design slope dip and Romana's SMR rating to be lower than necessary. The damage due to the method of excavation inflicted on the rock mass of the slope, is already included in the rock mass parameters measured (as shown in this research, ch. D.1.4). Both systems do, however, not allow for a positive correction for existing damage due to the method of excavation as does the SSPC system.

D.4.2 SSPC system's rock mass 'strength' parameters - rock mass cohesion and friction

The 'strength' of a rock mass calculated according to the Mohr-Coulomb failure criterion[127] with the rock mass cohesion and friction determined with the SSPC system (ch. D.3.3) 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, ch. B.2.3.1), and is compared to the 'strength' of a rock mass as calculated with the 'modified Hoek-Brown failure criterion' (Hoek et al., 1992, ch. B.2.3.5).

The 'strength' of a confined rock mass under a compressive stress is expressed in the major and minor principal stress at failure. According to the Mohr-Coulomb failure criterion this is formulated as follows:

$$\sigma_1 = 2 * cohesion * \tan\left(45° + \frac{friction}{2}\right) + \sigma_3 * \tan^2\left(45° + \frac{friction}{2}\right) \qquad [47]$$

σ_1 = major principal stress at failure $\quad$ σ_3 = minor principal stress at failure
cohesion, friction = cohesion and friction of the rock mass

and according to the 'modified Hoek-Brown failure criterion' this is formulated as:

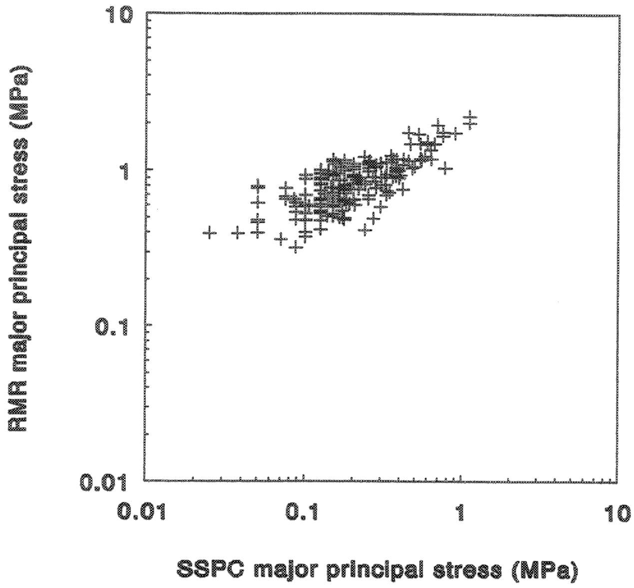
$$\sigma_1' = \sigma_3' + \sigma_c * \left(m_b * \frac{\sigma_3'}{\sigma_c}\right)^a \qquad [48]$$

σ_1' = major principal effective stress at failure $\quad$ σ_3' = minor principal 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 (see text)

Hence, the 'strength' of a rock mass according to different criteria can be compared by comparing the major principal stress values at failure. The absolute value of the σ_3 is not very important but should be in the same order as the σ_3 that exists in reality in the slopes in the research area[128], 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. Whether the calculated stress values at failure are true in reality is not evaluated and probably not very likely as the actual values will generally depend on more factors that are not considered in the calculations, such as the orientation of discontinuities, etc. (ch. A.2.4).

[127] See also glossary, page 241.

[128] The minor principal stress (σ_3) for each slope is calculated with the Mohr-Coulomb failure criterion with the rock mass cohesion and friction calculated with the SSPC system and with the major principal stress equal to the overburden pressure, e.g. equal to the height of the slope multiplied with the unit weight$_{mass}$. This σ_3 is then used in the calculation of the major principal stress at failure according to the Mohr-Coulomb failure criterion with the rock mass cohesion and friction calculated with the RMR system, and is used in the calculation of the major principal stress at failure according to the 'modified Hoek-Brown failure criterion'.



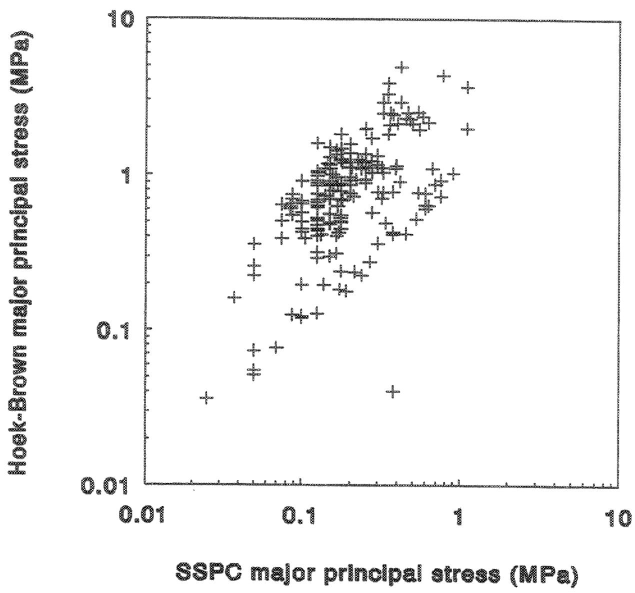


Fig. 76. Comparison of total major principal stress values at failure; left: RMR vs SSPC; right: 'modified Hoek-Brown failure criterion' vs SSPC.

D.4.2.1 SSPC system versus Bieniawski's RMR system

The values of the major principal total stress at failure (σ_1) calculated according to the Mohr-Coulomb failure criterion (eq. [47]) with the rock mass cohesion and friction determined with the SSPC system, are compared to the major principal total stress values calculated with the Mohr-Coulomb failure criterion with the rock mass cohesion and friction determined with the RMR classification system (Fig. 76 left). The total minor principal stress σ_3 in each slope is calculated following footnote 128. The RMR rating is derived from the SSPC field data following footnote 126 and converted to rock mass cohesion and friction values.

D.4.2.2 SSPC system versus the 'modified Hoek-Brown failure criterion'

The rock mass parameters in the 'modified Hoek-Brown failure criterion' (σ_c, m_b and a, eq. [48]) are derived from parameters measured for the SSPC system[(129)]. Fig. 76 right shows the major principal total stress at failure calculated according to the 'modified Hoek-Brown failure criterion' versus the major principal total stress at failure according to the Mohr-Coulomb failure criterion calculated with the rock mass cohesion and friction derived from the SSPC system for each slope. The 'modified Hoek-Brown failure criterion' is defined in terms of effective stresses while the rock mass cohesion and friction from the SSPC system, used in the Mohr-Coulomb failure criterion, are defined in total stresses. A calculation done with effective stresses for as well the SSPC system as for the 'modified Hoek-Brown failure criterion' showed virtually the same relation as shown in Fig. 76 right.

[(129)] The parameter σ_c (intact rock strength) is taken as the intact rock strength field estimate from the SSPC system. Hoek et al. (1992, ch. B.2.3.5) also suggest a determination of the intact rock strength by field estimation (although their classes and boundaries are slightly different from those used in the SSPC system). Hoek et al. derive the parameters m_b and a from a matrix describing the rock mass 'structure' in four classes, and the 'surface condition' in five classes. For the analysis described in this chapter the 'structure' and 'surface condition' parameters have to be derived from the SSPC system parameters. The 'structure' parameter in the 'modified Hoek-Brown failure criterion' is related to the size of the blocks in the rock mass, and is therefore taken to be linear with the spa_{mass} in the SSPC system (e.g. $spa_{mass} > 0.75$: 'structure' = class 1, $0.50 < spa_{mass} < 0.75$: 'structure' = class 2, etc.). The 'surface condition' parameter in the 'modified Hoek-Brown failure criterion' is related to the condition of the discontinuities and is therefore taken to be linear with con_{mass} in the SSPC system (e.g. $con_{mass} > 0.81$: 'surface condition' = class 1, $0.61 < con_{mass} < 0.81$: 'surface condition' = class 2, etc.). The parameter m_b is adjusted for the type of material of the intact rock. Not all positions in the matrix are defined. In the comparison only those rock masses are compared for which the matrix gives values for m_b and a.

D.4.2.3 Discussion

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 (Fig. 76 left) and the 'modified Hoek-Brown failure criterion'[130] (Fig. 76 right). The absence of a difference between calculations done with total stresses or calculations done with effective stresses in the comparison of the 'modified Hoek-Brown failure criterion' with the SSPC system, may indicate that the SSPC system is defined in terms of effective stresses and that thus water pressures in the slopes in the research area have been small or absent. The overall reasonable correlation proves that the SSPC system methodology for non-oriented slope stability is justified.

D.4.3 Conclusions

The calculation of the stability of a slope with the SSPC system gives a more distinctive differentiation between stable and unstable than with the Haines and SMR systems and is a clear advantage of the SSPC system over these classification systems. 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. This very likely proves that the SSPC system is more reliable in predicting the slope stabilities of the slopes in the research area. The 'strength' of a rock mass as determined with the SSPC system is good comparable to other methods. This proves that the calculation methodology used for the orientation independent slope stability incorporated in the SSPC system is justified.

[130] The rock mass parameters in the Hoek-Brown failure criterion, 'structure' and 'surface condition', are calculated following procedures from the SSPC system. The good correlation suggests that the rock mass parameters in the Hoek-Brown failure criterion could standard be calculated following SSPC procedures.

D.5 EXAMPLES AND VALIDATION

Four slopes in the research area are presented as examples of the application of the SSPC system. For two of the examples also an extensive analytical and numerical modelling, and a sampling program have been carried out which are included in this chapter to validate the results obtained with the SSPC method. The worked out classification forms for the SSPC system of each example are included in appendix VI.

The computer software programme used for numerical calculations is UDEC (Cundall et al., 1971, 1985, Hart et al., 1988, UDEC, 1993). This programme models the rock mass as individual blocks separated by discontinuities. The intact rock blocks are allowed to deform, rotate and translate. The movements of blocks along each other are governed by the shear and discontinuity stiffness criteria defined for the discontinuities. The programme calculates, therefore, a fairly realistic model of a rock mass. However in complicated situations that require a model containing many individual blocks, calculations become extremely time-consuming. The programme is two-dimensional which requires a transformation from a three-dimensional reality to a two-dimensional computer model. In many situations this is virtually impossible therefore the programme has only been used for slopes where it was 'a priori' recognized that a simplification from three- to two-dimensions would not have a too large influence on the calculated slope stability. Slopes have been used that can be modelled in a vertical cross section perpendicular to the slope and in which the discontinuities determining the stability are approximately perpendicular and parallel to this cross section[131].

D.5.1 Example I. Predicting the stability of a slope in Lower Muschelkalk (Tg21)

This example demonstrates how the SSPC system is applied to design a slope in a new road cut from old exposures. The road cut is situated in Lower Muschelkalk (Tg21) at km 494 along the road N-420 from Falset to Reus. Fig. 78 and Fig. 79 show two exposures of Tg21 limestone and dolomite along the old road. The first photo shows an excavated road cut made by small hole blasting (blast hole diameter ≈ 2.5 cm, length ≈ 0.75 m) probably blasted by gunpowder about 40 years ago while the second photo shows a similarly blasted exposure with a natural exposure above in the same unit along the same road. In the same unit a new road cut has been made in 1989 (Fig. 80). The new road cut was excavated by blasting (blast hole diameter ≈ 7.5 cm, length ≈ 8 m equal to the full slope height). The blasting was done with care. Fig. 77 shows a sketch of the locations of the exposures and of the new road cut.

The rock mass characterization, reference rock mass calculation[132] and the calculation of the slope stability probability of exposure A are used for the description of the SSPC system (respectively Fig. 71, page 145, Fig. 72, page 148 and Fig. 74, page 153). The forms for exposure B are Fig. A 108, Fig. A 109 and Fig. A 110 in appendix VI. The calculations of the stability of both slopes result in stable slopes which (as the photos show)

[131] Numerical distinct three-dimensional procedures and software programmes have been developed (3DEC, 1993, Cundall et al., 1985, 1988, Hart et al., 1988) but the cost of these programmes and of the required hardware (to obtain results in a reasonable amount of time) is so high that it was not possible to use these programmes in this research. It is also unlikely that such programmes will be used in day-to-day slope cutting practice in the near future.

[132] The values on the forms have been calculated with the computer programme SSPCCLAS in a higher precision than that shown on the forms. The rounding of the values may cause slight discrepancies in the calculations shown on the forms.

are also stable after 40 years or more. The exposures A and B are in the same lithological sub-unit; the orientations of discontinuities, the reference condition of each discontinuity set (*RTC*), and the reference friction (*RFRI*) and cohesion (*RCOH*) of the exposures are approximately the same so that they can be assumed to belong to the same geotechnical unit in the reference rock mass. The only difference between both exposures is discontinuity set 2 (J2, see exposure characterization forms). The location of the new road cut exposure C is nearby and also before the road cut had been made, it must have been obvious from the simple geology, that the road cut was going to be in the same lithological sub-unit as the old road cut and natural exposure. Characterization of the new road cut C confirmed this, and also that the rock mass belongs to the same reference rock mass unit as exposures A and B. However, in contrast to the good condition of the slopes in exposures A and B, the condition of the slope in exposure C is generally very poor with large blocks falling from the slope (Fig. 80).

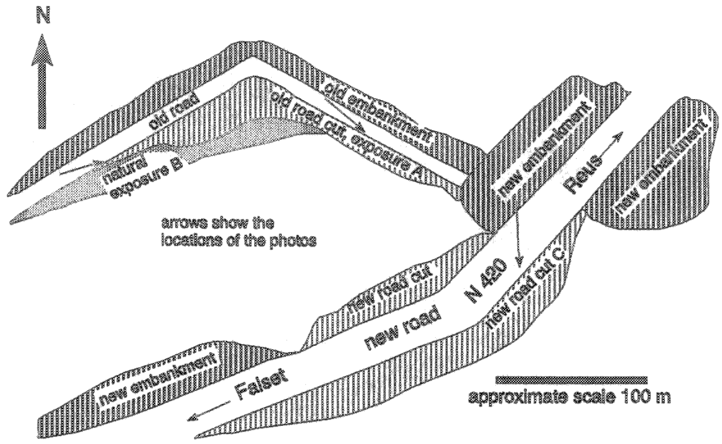


Fig. 77. Example I. Location sketch.

D.5.1.1 Slope stability by classification

The original design of the new road cut C has been made with a slope dip of approximately 85° to 90° (based on the inclination of the blast holes). Fig. A 111 shows the calculation of the slope stability probability for a slope dip of 85°. The calculation shows that sliding will occur along discontinuity 2 (044/86) and that the calculated Hmax of the slope is less than the design height of the slope of 8 m. In the old road cut A sliding does not occur along discontinuity 2 (044/86) because discontinuity 2 forms the slope while in the new road cut C the dip direction of this discontinuity set is about 73° different from the dip direction of the slope, resulting in an apparent dip of 76° in the direction of the slope dip.
Fig. A 112 shows the slope calculation for a design slope dip of 70°. No sliding nor toppling along discontinuities will occur and the calculated Hmax is larger than the slope height. This calculation is confirmed by the present (1995) overall slope dip measured about six years after excavation, which is at some locations along the slope already approximately 70° to 75° and at other locations expected to become 70° to 75° in due time. This example shows that with the SSPC system a good new road cut low in maintenance costs could have been designed and that the present poor result could have been predicted.

Fig. 78. Example I. More than 40 year old road cut A (blasted height about 8 metres from road level).

Fig. 79. Example I. Natural exposure B along old road. The natural exposure starts at about 2 m from road level and is partly overgrown. The lower part of the exposure is blasted. Note the small gunpowder blastholes in the lower part.

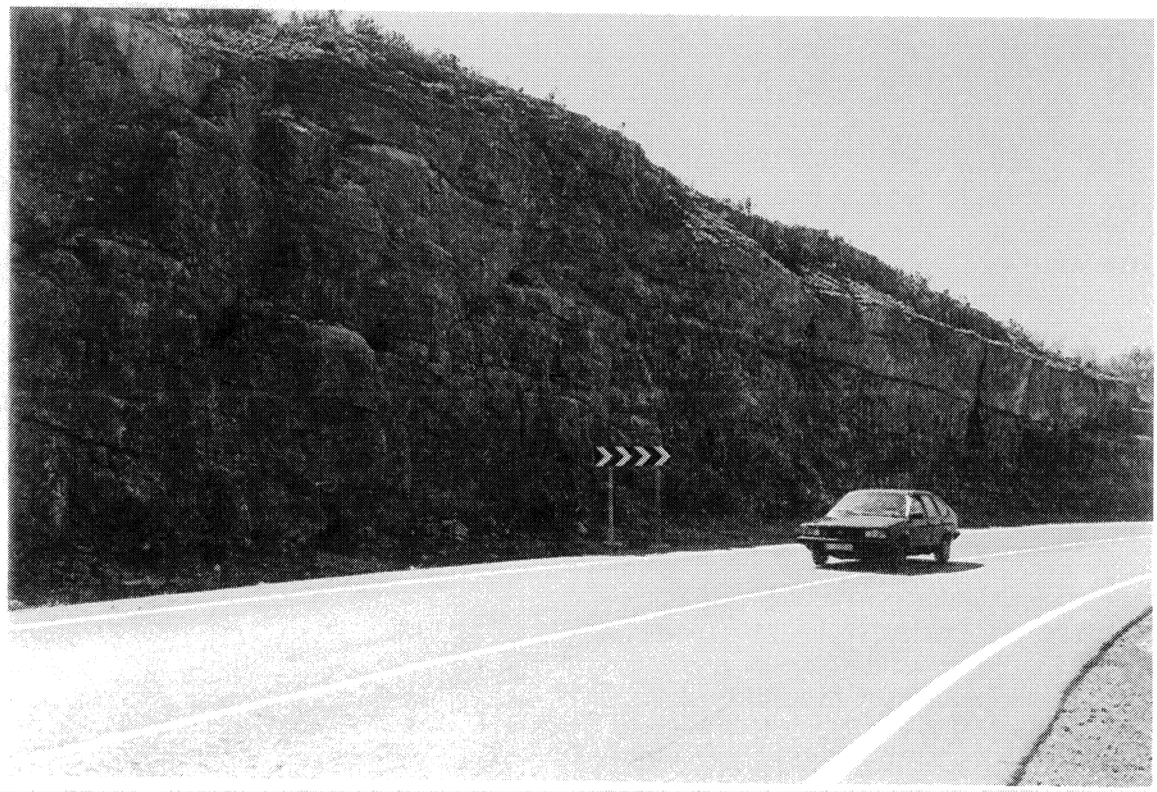


Fig. 80. Example I. New road cut C (bedding dips about 6°; gradient of road to the left causes a seemingly larger dip of the bedding to the right). Blast holes are clearly visible at the left.

Fig. 81. Example II. The sliding plane is clearly visible on the right; left side still standing part of the road cut (scale: road lining about 0.1 m wide).

D.5.2 Example II. Plane sliding failure in a 40 year old slope in Upper Muschelkalk (Tg23)

This example shows the application of the SSPC system to predict the stability of an existing slope, and the usefulness of analytical and numerical computer calculations compared to using the SSPC system. The 40 year old road cut is situated in Upper Muschelkalk (Tg23) limestone and dolomite at about 2.5 km from Marçá along the road from Marçá to La Torre de Fontaubella.

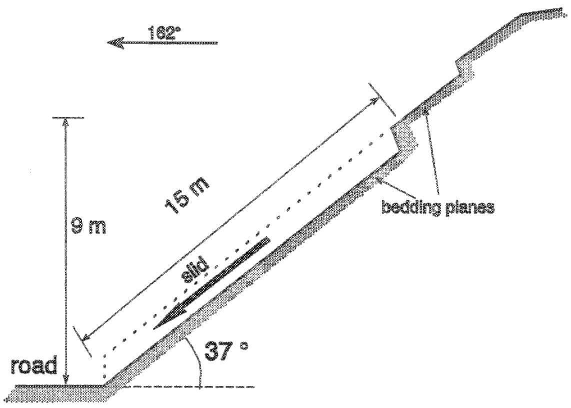


Fig. 82: Example II. Geometrical cross section of the slope (in the direction of the dip of the bedding and slope, 162°).

The slope (photo: Fig. 81; cross-section: Fig. 82) is cut in Upper Muschelkalk light grey calcisiltite (Tg23), medium bedded, very widely jointed, slightly weathered, strong, and impermeable except along the joints and bedding planes. The bedding (dip-direction/dip = 162/37) strikes parallel to the slope of the terrain (dip slope) and dips 36 - 37° towards the road excavation. Two joint sets are present. One set is vertical (265/85) and strikes approximately perpendicular to the road cut. A second joint set (337/48) is striking parallel to the slope face and bedding, but dips ≈ 50° against the slope face direction and thus approximately perpendicular to the bedding plane. The spacing of this discontinuity set is approximately 15 m. However, this discontinuity set showed a far smaller spacing of about 5 m in parts of the slope directly below the part that had slid and in parts directly adjacent to the sliding plane. This is likely due to the slope geometry that caused existing joints to open and new cracks to form because of tensile stresses. These additional joints with orientation 337/48 (at a spacing of about 5 m) are further called 'internal joints'. The bedding plane and the vertical jointing contain some clay infill. The clay in the bedding planes is likely to result from weathering as the bedding planes often contain some minor contents of clay. Clay infill in the vertical discontinuity set (265/85) is likely topsoil flushed in from the terrain surface above. Some karstic solution was observed along the vertical discontinuity set. For further details see the SSPC exposure characterization form (Fig. A 113).

The slope has originally been cut at an angle of 80 - 90° at least some 40 years ago. The slope was stable until April 1990 but already showed the 'internal joints', likely caused by tensile stresses, and it was found to have failed in March 1991. Because the slope is such a good example of plane sliding, the slope has been investigated in more detail. Samples for UCS and shearbox testing have been sawn out of the rock and a detailed survey of the topography of the slope has been executed. The slope has also been subject to analytical and numerical modelling with the distinct element method (UDEC, 1993). Numerical modelling was possible because the slope could be modelled for the two-dimensional UDEC programme without too many simplifications (Cindarto et al., in preparation).

D.5.2.1 Slope stability by classification

Fig. A 113, Fig. A 114 and Fig. A 115 (appendix VI) show respectively the exposure characterization, reference rock mass calculation and the slope stability probability calculation for this slope. The exposure characterization has been done in 1990 before the slope failed. The slope calculation shows that for a slope dip (road cut) of 90° the stability probability for sliding along the bedding plane was 55 % before the sliding happened. This indicates that the slope stability against sliding was almost unity, and for example, a very slight decrease of the condition of the bedding plane due to weathering was sufficient to cause failure. The fact that tension cracks developed during the lifetime of the slope also indicates that the slid block above the bedding plane was not fully supported by shear strength along the lower parts of the bedding plane. The clay infill in the vertical joint set (265/85) has not been included in the calculations of the reference rock mass and in the slope stability probability, as the infill was expected to be flushed into the discontinuities from the terrain surface and not to be present deeper in the rock mass. Whether the spacing of the second joint set (337/48) is taken as 15 or 5 m does not make a difference for the calculation of the reference rock mass nor for the probability of the slope stability.

D.5.2.2 Laboratory tests

Shear tests samples of the discontinuity planes have been done with the Golder shearbox (Hencher et al., 1989). Samples have been obtained from the debris of the failed slope and have been sawn out of still standing parts of the road cut. The samples from the debris were used for shear tests on non-fitting surfaces whereas the samples sawn out of the rock-mass were used to test fitting discontinuity surfaces. Only samples could be tested which did not contain steps. No significant differences were found between tests on the bedding planes and on the other discontinuities. The shearbox friction angle from these tests is 45° (this is the average of six tests which are not corrected for dilatancy, standard deviation 1°). The clay infill on the bedding surface as observed in the field has not been present on the surfaces of the samples for testing. For the debris samples this is obvious but also for the sawn samples the clay infill (which is very thin; 1 - 2 mm) was lost during the sawing and preparation of the sample.

The laboratory shearbox friction values for the bedding plane are representative for a rough planar surface (the sample with steps could not be tested) without infill and a large scale roughness equal to straight. This results in a friction angle of about 43° according to the 'sliding criterion'(133). The description of the bedding plane in the field is, however, straight, rough stepped with fine soft sheared infill and equivalent to about 35° friction angle along the plane ('sliding criterion')(133). The value from the laboratory shearbox test of 45° is thus in agreement with the sliding criterium for the sample tested, however, is not representative for the bedding plane in reality. That the difference between the test result and reality is not larger is pure coincidence. The absence of steps on the surface of the samples is compensated by the absence of the infill material in the laboratory tests. This illustrates the limited usefulness of shearbox testing, even for discontinuities which have no large scale roughness.

D.5.2.3 Slope stability by limiting-equilibrium back calculation

A traditional limiting-equilibrium back analysis was made of the slope of example II (Fig. 83). The cohesion along the sliding plane is taken as zero. The length of the sliding block is defined by the second joint set (337/48) approximately perpendicular to the failure plane, the so-called 'internal joint'. In the calculations the spacing of this joint set and thus the length of the sliding block is varied between 3 and 15 m. Whether the failure occurred under the influence of water pressures in the discontinuities was also investigated. Three different levels of water in the 'internal joint' were used in the calculation: hw = 100 %, 50 % and 25 % (hw = the height of the water as percentage of hj, the height of the joint above the bedding plane). The friction angle along the sliding plane is calculated with:

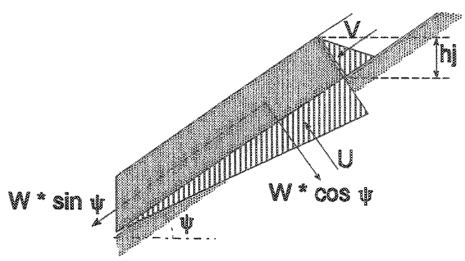


Fig. 83. Example II. Limiting-equilibrium analysis.

$$\varphi = \arctan\left(\frac{W \sin\psi + V}{W \cos\psi - U}\right) \qquad [49]$$

φ = friction along sliding plane W = weight of block ψ = dip of sliding plane
U = water force at bottom of block V = water force at rear of block

Fig. 84 shows the relation between the length of the sliding block along the sliding plane and the friction angle for different water heights (hw) in the 'internal joint'. The friction angle decreases if the length of the sliding block increases. This relation is less pronounced if the water level in the joints decreases. For a friction angle of 45° (shear test result) along the sliding plane, sliding should not have occurred for a full 15 m length block, even not if the 'internal joint' would have been completely filled with water (hw = 100 %). However, for a block length of 5 m sliding would just have been possible if the 'internal joint' was completely filled with water. For a friction

(133) Condition of discontinuity for the laboratory samples: *TC* = 0.75 (straight) * 0.65 (rough planar) * 1.00 (no infill) * 1.00 (no karst) = 0.49. 'Sliding criterion': φ = *TC* / 0.0113; φ = 43°.
Condition of discontinuity for the bedding plane in the field: *TC* = 0.75 (straight) * 0.95 (rough stepped) * 0.55 (fine soft sheared infill) * 1.00 (no karst) = 0.39. 'Sliding criterion': φ = *TC* / 0.0113; φ = 35°.

angle of 35° ('sliding criterion', ch. D.5.2.2) sliding would have been possible without the influence of water and independent from the block size.

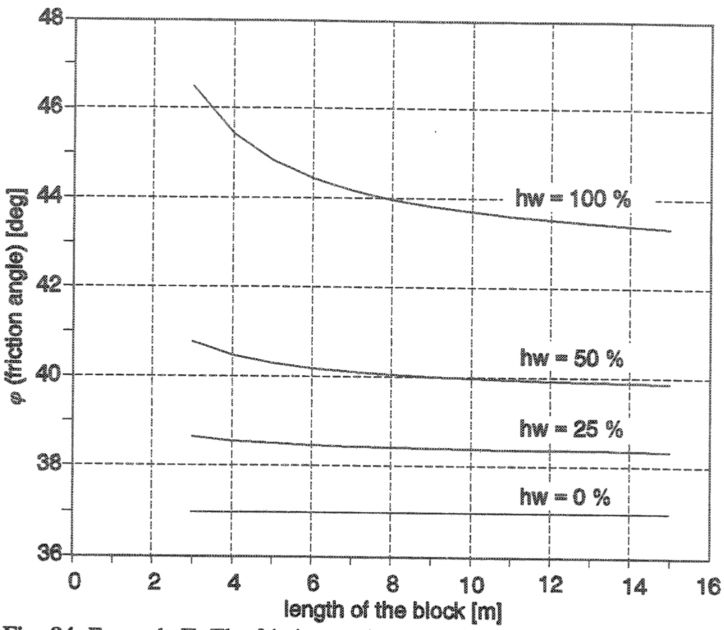


Fig. 84: Example II. The friction angle as function of block length and the height of the water in the second joint set (337/48).

D.5.2.4 Slope stability by numerical analysis - UDEC simulation

The UDEC simulations were made with fully deformable intact rock blocks[134]. The intact blocks behave elasto-plastic with a Mohr-Coulomb failure criterion and the discontinuities behave as an elasto-plastic area contact with Mohr-Coulomb failure criterion. The programme allows for fluid flow and thus water pressure modelling to investigate the influence of water pressures. Permeability parameters of discontinuities are, however, extremely difficult to obtain, and the parameters used in the modelling are studied guesses. The parameters have been chosen such that the 'internal joint' fills up with water.
The internal discontinuities striking parallel to the sliding plane, are modelled as tension joints. The angle between sliding plane and internal discontinuity is perpendicular. The joints are modelled with a tension strength of 7.5 MPa perpendicular to the plane (Brazilian indirect tensile strength for intact rock). The UDEC programme allows for setting a so-called 'flag' which causes the tensile strength to become zero after opening of the joint. The friction angle along the sliding plane is varied. The displacement of the block at the toe of the slope versus the number of calculation cycles (representing time) is shown in Fig. 85a and b; Fig. 85a for a slope without tension joints and Fig. 85b with tension joints. The result of the modelling with internal joints shows that the internal joints open and that the lowest block at the toe of the slope is moving down the slope and causing the slope to be unstable for friction angles smaller than 38° without any water being present. Fig. 85b shows the displacement and the stress distribution in the slope.

D.5.2.5 Conclusions example II

The classification, and the limiting-equilibrium and numerical analyses show that this slope was prone to failure. The limiting-equilibrium and numerical analyses become unstable if a friction angle along the bedding plane is less than about 37° to 38° without water pressures. This is the friction angle resulting from the SSPC system. The slope would have been stable if the friction angle from the shearbox testing was used in these analyses, except if high water pressures in the discontinuities had been assumed. Completely water filled discontinuities as an explanation for instability are unlikely. Even if the bedding plane was completely blocked at the toe of the slope, then in this type of rock mass water must have been able to flow out the bedding plane sideways via the vertical joint set.

[134] Only the main parameters for the modelling are included here. Parameters not mentioned are at a default value as suggested by the manual of the programme and have no or minor influence on the modelling results. For more detailed descriptions the reader is referred to the literature (UDEC, 1993).

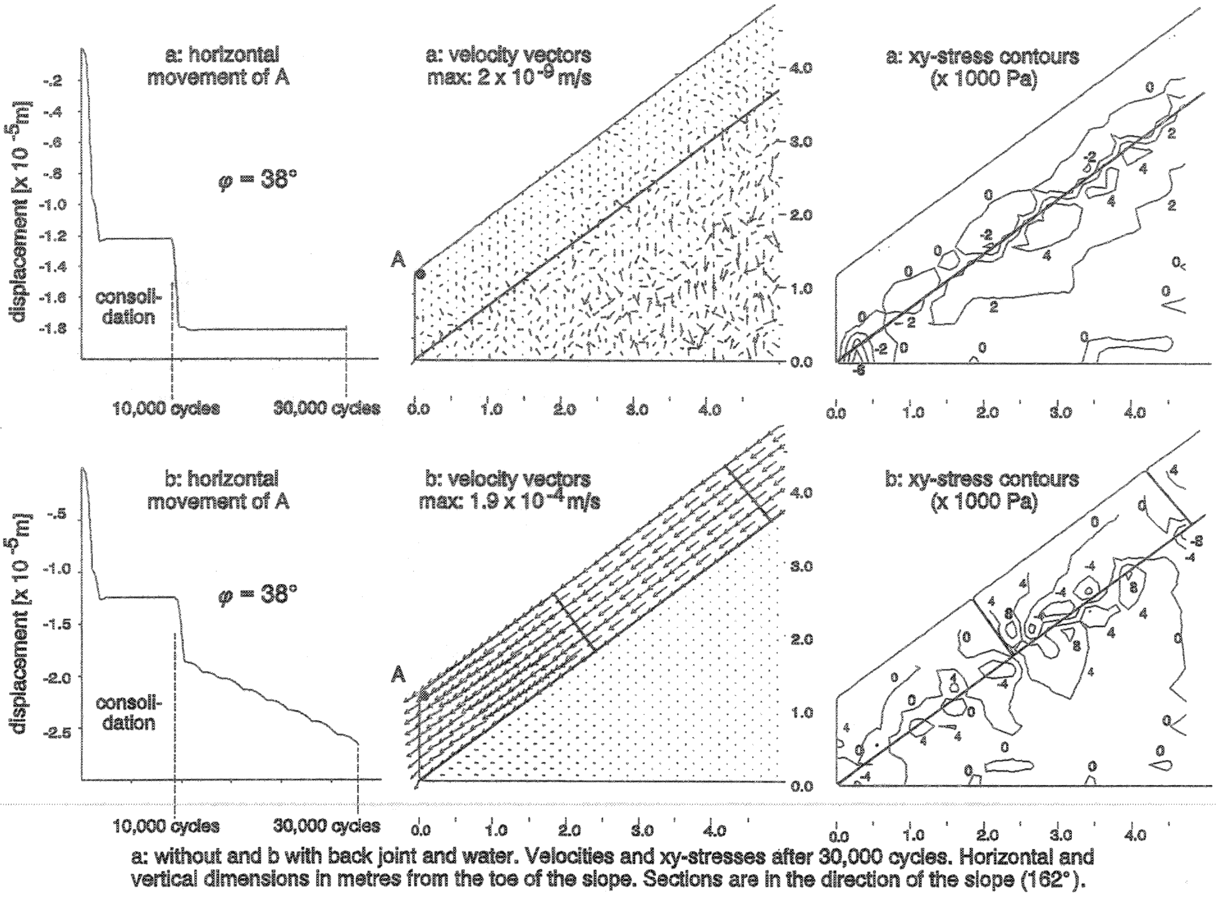


Fig. 85: Example II. UDEC simulation. Enlarged part of the toe of the slope showing displacement, velocity and xy-stresses along sliding plane.

The most likely explanation for this failure after 40 years, is therefore that the slope always had a stability almost unity. However, the weathering over the years caused a very slight decrease in the shear strength of the bedding plane, reducing its shear strength slightly. The final trigger for failure may have been water, but with water pressures considerably smaller than associated with a fully filled up discontinuity. Also it is likely that water caused a softening of the infill material in the bedding plane and acted as lubricating agent in the discontinuity.

This example clearly shows the relative uselessness of taking samples and testing these samples for shear strength. Apart of the size of the samples, which only represent a very small part of the rock mass, all problems with taking samples and testing these are encountered. If results of such tests are not very carefully scrutinized, the conclusions based on subsequent analyses might be incorrect. Classification does not involve difficult sampling and testing and, more importantly, it does describe a large part of the rock mass. The analytical and numerical calculations are only possible because the slope is relatively simple, so that the degree of simplification is minimal. Even then these calculation methods including the testing take many hours or even days to execute, while a classification probability study is carried out in less than a quarter of an hour.

D.5.3 Example III. Non discontinuity related failure in a 4 year old slope in Carboniferous slate

Example III illustrates the use of the SSPC system for the stability of a slope which stability is not directly related to the orientation of discontinuities and slope. The slope is situated in a 4 year old road cut in Carboniferous slate at about km 9.5 along the road from Falset to Gratallops.

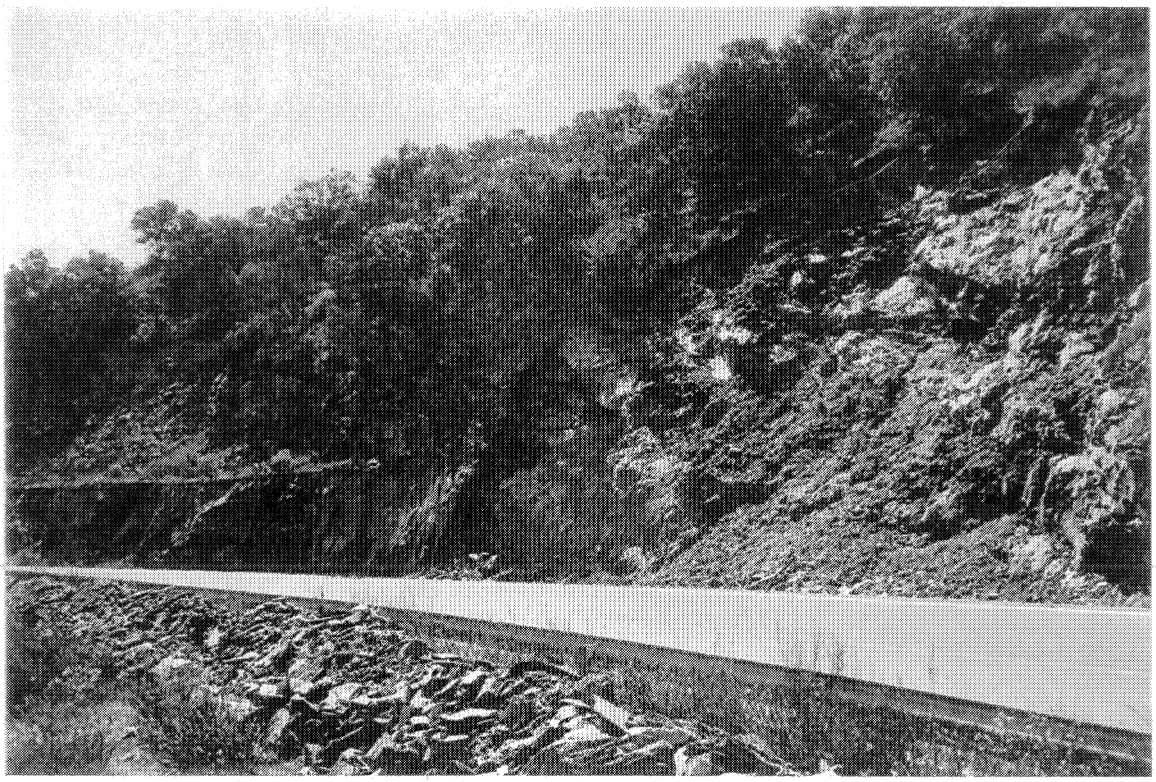


Fig. 86. Example III. The slope in April 1995 after the main failure of April 1992 and the partial failure of the top part of the slope (the terrace on the left is the old road).

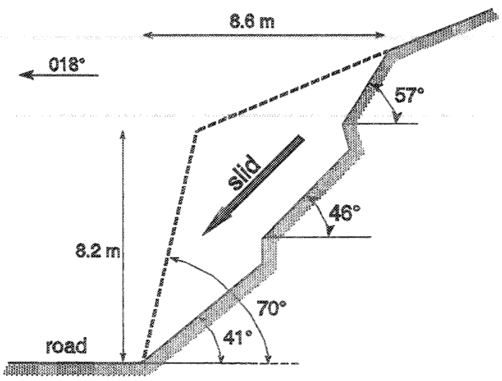


Fig. 87: Example III. Geometrical cross section of the slope. Situation in april 1992 after the main failure occurred (section in direction 018°).

The slope (photo: Fig. 86, cross section: Fig. 87) is cut in Carboniferous light to dark grey, argillaceous, narrow to very thinly spaced cleavage, very closely jointed, small tabular, slightly weathered slate, moderately strong, impermeable except along joints and open cleavage planes. The well developed cleavage is folded resulting in variations in dip but its strike is about constant. The cleavage plane is highly irregular and dips about 41° in the bottom part of the slope and 60° at the top part; both in a direction of about 340°. The average dip of the cleavage plane is about 46°. Orientations of the joint planes in this slope are also influenced by folding. There is no significant infill material to be found in the cleavage discontinuities (C1) nor in the joint discontinuities but a 5 cm thick gouge type of infill is found in the approximately horizontal joint discontinuity set (J4). The bedding is about parallel to the cleavage at locations where visible, however, the bedding is almost completely overprinted by the cleavage and of no importance for the stability of the slope.

Some cleavage and joint surfaces are very slightly weathered and locally stained. Fig. A 116a (appendix VI) shows the poles of the slope face and of the cleavage and joint discontinuities. The quantity of joints in different directions allows for kinematic sliding or toppling in virtually any direction. The discontinuities included are those which could be measured and are thus at easily accessible locations mainly at the bottom of the slope. Also not all discontinuities are continuous or have the same orientation throughout the rock mass of the slope. Fig. A 116b and c (appendix VI) show stereo projections and contour plots of the poles of cleavage and main joint discontinuity planes.

The slope was excavated in 1989 during the construction of the new road alignment and the slope has been cut at about 70° in the direction 018° (comparable to the situation to the left in Fig. 86). In April 1992 the slope failed. The slope face after failure has highly irregular surface. The overall dip of the slope became about 53° with a slope dip of 41° in the lower part of the slope, 46° in the middle part and 57° towards the top of the slope. The upper part of the slope became undercut (Fig. 87) and in 1995 also the undercut top part of the slope had partially failed (Fig. 86) reducing the overall slope dip to about 45°. Visually assessed the slope is now expected to be stable, although some minor blocks which are undercut and not fully supported, or which have already moved during the forgoing slides are expected to fall in the near future. The night before the main failure occurred (April 1992) it had been raining and a very small amount of snow had fallen. The actual temperature had probably not been below zero at ground level.

This slope can be analysed by a limiting-equilibrium method and numerically in two dimensions with some simplifications. Samples for UCS and shearbox testing have been sawn out of the rock and a detailed survey of the topography of the slope has been carried out.

D.5.3.1 Slope stability probability by SSPC classification

Because of the variation in dip of the cleavage plane (from 41° in the lower part of the slop towards 60° in the upper part of the slope) the number of geotechnical units in the rock mass of the slope is infinite. The most unfavourable dip for the stability of the slope is, however, the 60° dip in the top part of the slope and, therefore it is sufficient to calculate the slope stability probability as if the dip of the cleavage is 60° throughout the slope. Fig. A 117 and Fig. A 118 (appendix VI) show the exposure characterization and reference rock mass calculation. Fig. A 119 shows the slope stability calculation before failure and Fig. A 120 after failure. These are based on average slope dip angles. The exposure characterization had been done before the slope failed in 1991. Already at that time it was obvious that failure was imminent and accordingly the slope was visually assessed as 'unstable with large problems'.
The slope calculation shows that for an overall slope dip (road cut) of 70° (Fig. A 119) it could be expected that the slope would fail because the calculated H_{max} (3.2 m) is far below the real height (8.2 m) resulting in a probability to be stable of < 5 %. This instability is not caused by sliding along the cleavage discontinuity plane but results from the orientation independent slope stability probability. The friction along the cleavage planes is about 57° according to the 'sliding criterion'[(135)]. This is more than the apparent dip of the cleavage plane in the direction of the slope dip and sliding along this plane is not expected according to the SSPC system, even not along the steepest parts of the cleavage plane. The slope calculation (Fig. A 120) with an overall slope dip of 45° which is the overall slope dip in 1995, results in an SSPC slope stability calculation which is about unity (55 % stable). This corresponds with common sense as the stability of a slope is expected to be unity after failure. A stability almost unity also corresponds with the visually assessed stability in 1995.

D.5.3.2 Slope stability by kinematic analysis

A kinematic analysis of the mean orientations of the discontinuities (Fig. A 116, appendix VI) shows that for a slope with a slope dip of 70° sliding along cleavage planes is possible if the friction angle along the cleavage planes is less than 54° for cleavage planes in the top of the slope (dip of cleavage plane 60°) and is less than 40° for the lower part of the slope where the dip of the cleavage planes is about 46°. In the top of the slope, where the cleavage plane dips 60°, kinematically wedge failure is possible for the wedges formed by the cleavage plane C1 with joint system J2, J4 and J5 and for the wedges formed by J4 with J2, J3 and J5. The kinematically possible wedge failures formed by J4 are not relevant as J4 is nearly horizontal and the friction along the planes intersecting

(135) Cleavage discontinuity plane: *TC* (condition of discontinuity) = 0.85 (large scale roughness 'curved') * 0.75 (small scale roughness 'smooth undulating') * 1.00 (no infill) * 1.00 (no karst) = 0.64. Friction along cleavage discontinuity plane: 0.64 / 0.0113 ≈ 57° ('sliding criterion').

J4 are high enough to prevent movement. For the wedges formed by C1 with J2, and C1 with J5 the friction angle has to be less than about 54°. To allow wedge failure in the lower part of the slope where the cleavage plane dips 46° the friction along the cleavage and joint planes has to be in the order of 40° or less to allow for wedge failure. Concluding, a kinematic analysis resulting in an unstable slope is only possible if for the upper part of the slope the friction angle along the cleavage plane is assumed to be less than 54° and for the lower part of the slope less than about 40°.

D.5.3.3 Laboratory tests

Field tilt tests (45°) and shearbox tests have been done (cohesion = 0 MPa, friction = 47° average of five tests of the cleavage plane and not corrected for dilatancy, standard deviation 2°). Samples have been taken from the slope after failure and have been sawn out of still standing parts of the slope. The test results of the cleavage planes did not show significant differences in friction angle between fitting and non-fitting surfaces(136). Shearbox tests on joints were not possible as no suitable samples could be obtained. The cleavage plane is described in the field as large scale roughness 'curved' and small scale roughness 'smooth undulating' with no infill and no karst. Tilt angles and shearbox friction angles are obtained from samples whose size is too small for large scale roughness to be included. The description of the surfaces of the samples is thus large scale 'straight, and small scale roughness 'smooth undulating' with no infill and no karst. According to the 'sliding criterion' this results in a friction angle of 49°(137) which is in agreement with the 45° and 47° resulting from the tests (compare to example II, ch. D.5.2.2).

D.5.3.4 Slope stability by limiting-equilibrium back calculation

Janbu's method of vertical slices (Janbu, 1973) and Sarma's method of non-vertical slices (Sarma, 1979) have been used for the limiting-equilibrium calculation. Back calculation according to Janbu resulted in friction angles along the sliding plane of 55.4° (under wet conditions) and 42.6° (under dry conditions). Sarma's method allows for the incorporation of the true orientation of the jointing. The internal discontinuities crossing the slid rock mass form the sides for the calculation according to Sarma. Two discontinuities are modelled under an angle (δ) of +6° and +11° with the vertical (see cross sections in Fig. 90). In addition Sarma's method incorporates the friction resistance along the side of the slices in its calculation. The calculations resulted in a back calculated friction angle of 57.4° under wet conditions and 43.9° under dry conditions.

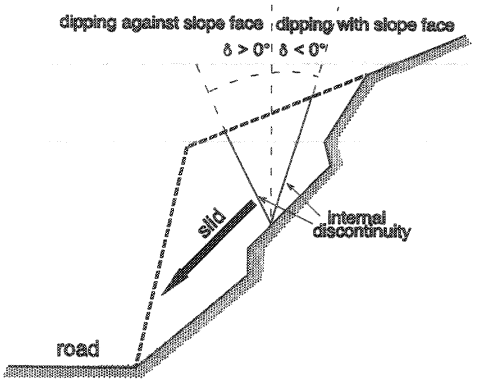


Fig. 88. Example III. Definition of inclination angles for the internal discontinuities.

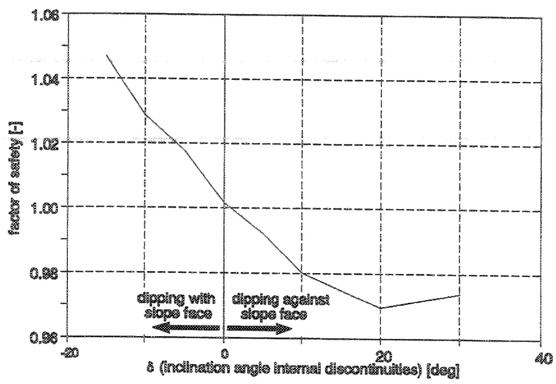


Fig. 89: Example III. The factor of safety as function of the inclination angle of the internal discontinuities. Friction angle ϕ = 43.5° for base and side friction; calculated by Sarma's method.

(136) The difference in friction angle between fitting and non-fitting surfaces diminishes with less rough surfaces.

(137) Condition of discontinuity for the test samples: large scale roughness 'straight' (it is too small for a large scale roughness) and small scale roughness 'smooth undulating' and no infill or karst: $TC = 0.75 * 0.75 * 1.00 * 1.00 = 0.56$. 'Sliding criterion': $\varphi = 0.56 / 0.0113 = 49°$.

The orientation of the internal discontinuities crossing the slid rock mass shows a variation due to folding. In order to study the influences of this variation of the orientation a sensitivity analysis has been carried out. A series of geometrical cross sections with different inclination angles (δ) for the internal discontinuities has been modelled. Fig. 88 illustrates the way in which the orientation of δ is defined. The factor of safety of each geometrical cross section is calculated by Sarma's method for non-vertical slices. The results of the calculations are illustrated in Fig. 89. As the inclination angle of internal discontinuities increases from a negative to a positive value (from dipping "with" to dipping "against" the slope face) the factor of safety decreases. The extent of the influence of the orientation and the friction angle of the internal discontinuities on the stability depends upon the inclination angle of the internal discontinuities. As the inclination angle of the internal discontinuities increases from negative through zero to positive (from dipping "with" to dipping "against" the slope face) the influence of the shear strength becomes more pronounced. This is caused by the increase of the normal stresses on the internal discontinuities. The variation of the friction angle of the internal discontinuities has only a minor influence on the total safety if the inclination of the internal discontinuities is in a range between +10° and -10°. The analysis results in a safety factor of unity for a friction angle of about 42°, virtually independent from the orientation of the internal discontinuities.

D.5.3.5 Slope stability by numerical analysis - UDEC simulation

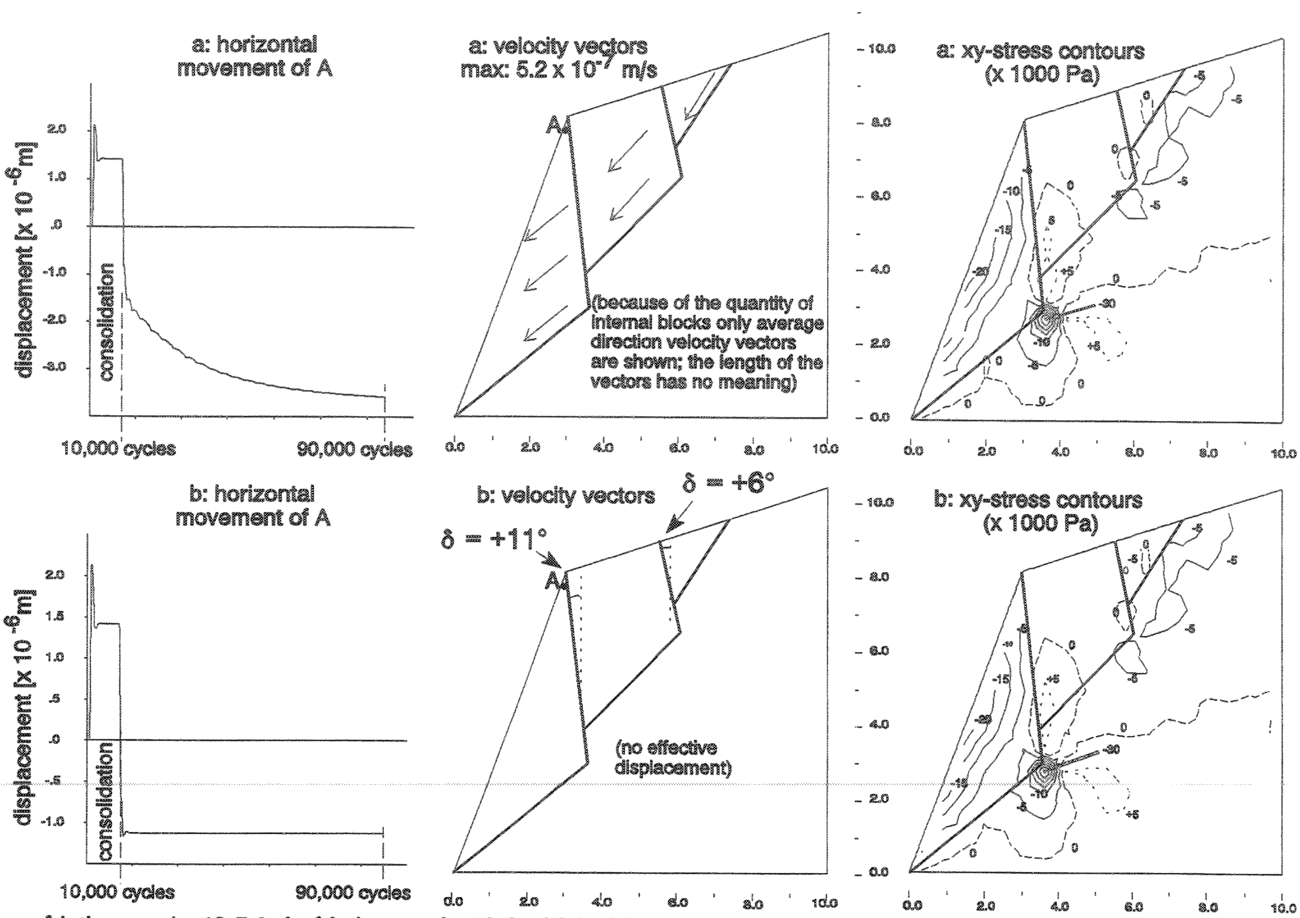
The UDEC programme (example II, ch. D.5.2.4) has been used for a numerical back calculation[134] of a model with different oriented internal discontinuities. The model in which the orientation of the internal discontinuities varies between +6.04° and +11.07° resulted in a φ of 43.7° (Fig. 90 a: unstable, b: stable). A sensitivity analysis comparable to the limiting-equilibrium back analysis was not possible due to the large calculation time necessary. As water may have had an influence on the failure of the slope a numerical back analysis including water has been executed. The results were, however, totally unrealistic as during the calculations water pressures in discontinuities became larger than defined by the level of the water table. This is obviously not possible and the calculations have been abandoned[138].

D.5.3.6 Conclusions example III

The classification, limiting-equilibrium and numerical calculations come to the same result: the original slope dip of approximately 70° was unstable. According to the SSPC system the slope was too high for a slope dip of 70° while none of the discontinuities was the cause for sliding or toppling instability. This is in contradiction to the limiting-equilibrium and numerical analyses which both show that sliding instability can occur. However, the sliding is only possible if the friction angle along the discontinuity planes is lower than the friction angles determined from testing and considerably lower than friction angles determined with the 'sliding criterion', or if is assumed that high water pressures existed in the slope at the time of failure.
The 'sliding criterion' gives a reasonably accurate estimate of the friction along the discontinuities (appendix III). In this example this is confirmed by the tilt tests (45°) and laboratory shearbox tests (47°) for the friction angle without large scale roughness, for which the 'sliding criterion' results in 49°. In the limiting-equilibrium and numerical analyses sliding is only possible if the friction angle along the discontinuities is around 43° which is not only less than the test values, but also implies that large scale roughness would have been of no importance. This is unlikely. The existence of high water pressures in the slope at the time of failure is also unlikely for the same reasons as for example II (ch. D.5.2.5), e.g. water can flow out via connecting discontinuities and no evidence of water under pressure has been observed.

Although water pressures are not the sole reason for the failure, the presence of water will have had a negative influence on the stability. Water will have caused a softening of the infill material in joint discontinuity J4, will have lubricated all discontinuities, and will have created some, however, very limited, pressures in the discontinuities. Additionally the little bit of snow in April 1992 will have caused a (very) little additional weight on the slope.

[138] These erroneous results have been discussed with the manufacturers of UDEC however a reason could not be pinpointed and it is likely that these were caused by an error in the UDEC programme code (version 1.8). In later versions of UDEC this problem is reported to be solved, however these were not available for this research.



a: friction angle 43.5 °. b: friction angle 43.9°. Velocities and xy-stress contours after 90,000 cycles.
Horizontal and vertical distances from the toe of the slope. Sections are in the direction of the slope (018°).

Fig. 90. Example III. UDEC simulation. Displacement, velocity vectors and xy-stresses in the slope.

The reason for failure of the slope is more likely that the rock mass as a whole has not been able to sustain the stresses in the rock mass caused by excavation the slope with a dip of 70°. After the excavation of the slope the new stress situation in the rock mass caused a progressive weakening by breaking pieces of intact rock, small movements along existing discontinuities, opening of cleavage planes and existing joints and possibly forming of tension joints. This progressive weakening of the rock mass continued until in April 1992 the water and snow triggered the already weakened rock mass to fail. The orientation independent stability probability of the SSPC classification system is empirically developed on the basis of existing slopes from which the rock mass of several must have been subject to progressive weakening of the rock mass. Therefore, the SSPC system can successfully assess the stability probability of the slope in this example.

D.5.4 Example IV. Influence of weathering and method of excavation on the stability of a slope in Upper Muschelkalk (Tg23)

Example IV demonstrates how the SSPC system considers poor blasting and future weathering in a slope stability probability assessment. The slope is situated in Upper Muschelkalk (Tg23) limestone and dolomite in a road cut at km 492 along the road N-420 from Falset to Reus. A photo of the example slope is shown in Fig. 40 (page 91)[139]. The slope is newly cut in Tg23 (Upper Muschelkalk) in 1988. The slope has been excavated by blasting originally with a dip of about 80°. The present dip of the slope is between 60° and 70°. The length of the blasting holes cannot any more be determined, but it is likely that blasting has been done in one pass over the full height of the slope (≈ 14 m) with blasting holes with a diameter of about 7.5 cm. This procedure for blasting has been standard for the road cuts in rock along this road (N-420) when the road was renewed in 1988, and it is likely that the same procedure has been followed for this slope.

The Tg23 consists of interlayered thin bedded (visible in Fig. 40 just above the sitting person) and medium to thick bedded units. The same thin bedded units are found exposed in nearby (less than 50 m away) old road cuts of more than 40 years old. Old road cuts made in the thin bedded units with dips of 60 to 70° and heights of about 5 m, are still (in 1995) stable and no or very little degradation of the rock mass is observed in these old road cuts. The rock mass in these old road cuts is still only slightly weathered. The method of excavation used for these old slopes is not known, but no remnants of blasting boreholes are visible at all, so that it is likely that these road cuts are excavated by hand or by a small shovel. An exposure characterization of the thin bedded units is given in Fig. A 121 (appendix VI).
The dip direction of the slopes in the old and new road cuts are approximately equal and the general position of the old road cuts in the topography is fully comparable to the new road cut. Both the old and new road cuts are cut into a hill that flattens above the road cuts. Quantities of water flowing from above over the road cuts are, therefore, likely comparable, although this has not been tested. Also with respect to geology (faults, etc.) no major differences have been noted between the old and the new road cuts.

The new road cut (Fig. 40, page 91) is clearly unstable, large parts show rill erosion and erosion of the thin bedded units causes undercutting of the thicker bedded parts, making these unstable. The general impression of the slope is extremely poor. On close examination those parts of the slope which appear to be 'soil' are in fact the thin bedded units which are partly covered by top soil transported from higher parts of the slope. For another part the soil is derived from weathering of the thin bedded units. In some places these have been weathered to a moderate or high degree of rock mass weathering for at least 0.5 to 1 m into the rock mass. The structure and coherence of the rock mass, and in particular the structure and coherence of the thin bedded units, are disturbed by the method of excavation. Discontinuities have opened, blocks are displaced, and at many locations the intact rock is fractured and occasionally also crushed due to the blasting for the excavation. The slope is not unstable due to sliding or toppling along discontinuities.

Although a back analysis of such a slope can never be very exact, the following reasonable assumptions can be made to explain the instability. The damage due to blasting has disturbed the structure of the rock mass so severely that water could flow through the near-surface parts of the rock slope. This has caused the weathering of the thin bedded units[140]. The disturbed and moderately to highly weathered thin beds cannot sustain a slope with a dip of 60 to 70° at a height of about 14 m.

[139] The slope discussed here has not been used for the development of the SSPC system. Although located in the research area, the slope was considered to be too unstable to safely be analysed by students during the years used for collecting data (1990 - 1993). By 1995 most larger blocks had fallen of the slope (reducing the slope dip) and the slope was deemed safe enough to carefully assess the slope stability and rock mass.

[140] A rapid, within a few years, weathering of intact rock as well as the rock mass has been noted to occur in some of the units with a thin or smaller than thin bedding spacing of the Tg23 and Tg3 formations.

D.5.4.1 Slope stability by kinematic analysis or calculation

A kinematic analysis results in an assessment for the slope to be stable because the slope is not unstable due to discontinuity related sliding or toppling. A limiting-equilibrium or numerical analysis is extremely difficult for such a rock mass as it is almost impossible to obtain suitable samples for testing. It is also impossible to quantify the reduction in strength of the rock mass due to the loss of structure and coherence without large scale testing.

D.5.4.2 Slope stability by classification

The SSPC system results in a probability to be stable of > 95 % for the old road cuts with a slope dip of 70° and a height of 5 m. The new road cut with a height of 13.8 m, with a 'slight' degree of rock mass weathering and 'dislodged blocks' due to blasting, results in a probability to be stable of less than 5 % for a slope dip of 80°. For a slope dip of 60° the probability to be stable increases to 85 %. If also the increased degree of rock mass weathering (highly) is taken into account, the probability to be stable decreases again to < 5 % for a slope dip of 60°. In the present condition the rock mass is clearly not able to support a slope with a dip of 60° (Fig. 40, page 91), and according to the SSPC system, stability will be achieved if the slope dip is decreased to 45° (probability to be stable 55 %).

D.5.4.3 Conclusions example IV

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 weathering influence. 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 the steep slope dip of 80° if sloppy executed blasting was going to be used.

D.5.5 General conclusions from the examples

The kinematic analyses, and the limiting-equilibrium and numerical calculations executed for the examples give results for the stability which are, in general, comparable to the stability probability obtained by the SSPC classification system. However, assumptions have to be made in the kinematic, limiting-equilibrium and numerical analyses, e.g. water pressures, low friction angles along discontinuities, etc., which are not supported or which are even contradicted by field observations or testing. The SSPC classification system gives feasible results without contradicting field observations or test results. Kinematic, limiting-equilibrium or numerical analyses would not have predicted the instability of the slope in example IV. The examples presented in this chapter are typical for the slopes in the research area.

D.6 CONCLUSIONS

The Slope Stability Probability Classification (SSPC) system is developed based on data from 184 stable and unstable slopes with heights ranging between 2 and 45 m. 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. 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, the assessment of stability in probabilities for separate failure mechanisms, and the absence of ambiguous or difficult to measure parameters like RQD, water and elaborate testing (UCS, shearbox tests, etc.). The SSPC system, unlike many other methods to design or classify slopes, also gives a stability assessment if the discontinuities are not the main source of failure. The SSPC system assesses the stability of the slopes three-dimensionally because the system is verified against existing slopes. This is obviously a major advantage compared to methods that are either implicitly two-dimensional or require massive calculations. The repeatability and reliability of the characterization is generally good because difficult to measure or ambiguous parameters are not required.

A three-step classification system is the core of the SSPC system. 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 'slope' 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 and for the damage due to the method of excavation used to make 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 'slope' 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 a new slope and taking into account future weathering. The probability of the slope to be stable is then determined with the additional parameters for the geometry of the slope.

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 slopes under conditions and in areas that are very different implies a risk. The SSPC system is, however, based on a large number of different slopes in a wide variety of rock materials and rock masses, and thus may be reliable for slopes in more rock mass types than those used for the design of the system. The influence of using the system in a climate different from the climate where the system has been developed may be limited. The intensity and duration of the rainfall determine the water levels (and related water pressures) in and on the slopes, but whether it rains occasionally or daily, does not likely change the maximum possible water levels and pressures substantially. Also, should be considered that water pressures may be less important in slope stability than it is often assumed. Snow and ice forming in and on the slope may, however, influence its stability in climates with lower temperatures. Generally: it is always prudent to check the system against existing slopes before applying it to the design of new slopes.

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

Slope instability can best be described in probabilities separate for different types of failure and not in an overall 'point rating' that includes all failure mechanisms and modes. The degree of stability or instability is easier to perceive with a probability than with a 'point rating' because a 'point rating' normally requires previous experience from the user. The probability of failure of a slope indicates the hazard that a slope is going to fail. Depending on the importance of the slope, for example, along a major highway or along a secondary road, the risk of a slope failure, in terms of loss of human lives, economic losses or environmental damage, etc., can be calculated.

The methodology followed for the development of the SSPC system did not require identification of failure mechanisms in the field. This is in contrast with most other slope stability research which analyses the failure mechanism(s) in unstable slopes. Such an approach requires the proper identification of the failure mechanism(s) in the field which is often not or not in all detail possible. Therefore the methodology used for the development of the SSPC system increases its reliability.

Shear strength along a discontinuity plane
The friction determined by the 'sliding criterion' is a useful and powerful tool in rock mechanics. The shear friction is determined along discontinuity planes in-situ without the necessity of taking samples, without the ambiguous interpretation of shearbox results, and without the translation of small scale shearbox values into large scale friction values. The 'sliding criterion', together with the weathering parameter, allows for an assessment of the friction angle along a discontinuity plane in the future.

Water pressures in discontinuities
The stability of slopes in this research is not or only very little influenced by water pressures in discontinuities. Surely water has had an influence on slope stability due to softening of infill, lubricating discontinuities, etc., but water pressures as normally assumed in traditional limiting-equilibrium calculations for slope stability cannot have been present at failure in many of the slopes assessed. The assumptions normally made in traditional slope stability calculations with respect to water pressures in discontinuous rock masses are likely to be too conservative, because the discontinuities are not completely filled by water or because the water pressure in a discontinuity does not act over the full plane of the discontinuity.

Intact rock strength by estimation
The determination of intact rock strength estimation by a large number of simple tests (e.g. hammer blows, etc.), as used in this research, is more suitable for the determination of intact rock strength and its variation in large inhomogeneous rock masses than a limited number of unconfined compressive strength tests.

Natural slopes
The system has been designed on man-made excavations and slopes. However, there is no reason the system could not also be used for natural slopes.

Minimum and maximum height and slope dip - overhanging or undercut slopes
The slopes used for the design of the system have heights characteristically between 2 and 25 m and the average height of all slopes is 7.7 m; maximum height is 45 m. It is likely that for slopes much higher than 45 m the system does not work because mechanisms related to size, such as breaking of intact rock, shearing through steps, buckling, etc., will become more important. The minimum slope dip used for the design of the slope stability probability classification (SSPC) system is 25°. Whether the system is still valid for slope dips lower than this value has not been investigated. The maximum slope dip in the research has been 90°. Overhanging or undercut slopes have not been considered. The slope stability probability classification (SSPC) system should not be used for such slopes. Friction angles along discontinuity planes may, however, be determined for such slopes and may be used in an assessment of their stability.

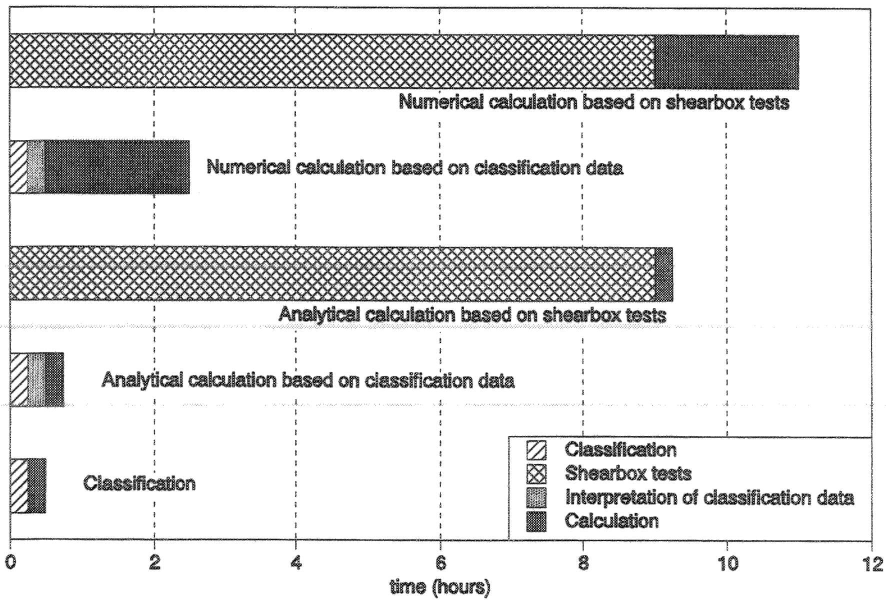
Highly inhomogeneous, folded and faulted rock masses
Slopes in highly inhomogeneous, intensively folded or faulted rock masses present a special problem. The rock mass should be divided in geotechnical units in which the rock mass properties are broadly homogeneous, and the slope stability probability 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 and orientations can be used, although this likely leads to a too conservative assessment.

Strongly deforming intact rock
Rock types that are deformed very easily (gypsum, salts, etc.) are present in the research area and 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 stability of the slope is governed by deformation of the intact rock.

External stresses
The system has not been designed for slopes that are or will be excavated in a rock mass that is under influence of external stresses. External stresses do not originate in the rock mass in which the slope is or will be excavated, but are, for example, tectonic stresses or stresses due to a high hill or mountain behind the slope. It may sometimes be possible to simulate the presence of a hill or mountain behind the slope by taking the slope height in the SSPC system as if the slope extends to the top of the hill or mountain. This has, however, not been tested and often this height will exceed the maximum height for which the system has been designed (see above).

System structure
The system, in comparison with other rock mass classification systems, is more elaborate in structure and calculation. This is, however, not likely to be a drawback of the system in a time where computers are widely available both for office and field use. The system is suitable to be incorporated into a GIS environment. The parameters can be interpolated independently and rock mass parameters and slope stability probabilities can be calculated at required locations.

Time saving
Fig. 91 shows time estimates for various methods to arrive at a stability assessment of a slope. Classification is an attractive option, in particular because the calculation may be done while standing in front of the slope. If curious results are obtained it is still possible to check the observations and data.



Shearbox tests for a slope are estimated to take a minimum of 9 hours (obtaining the sample and sawing: 2 hours; testing: 1 hour, and 3 samples per slope), excluding the time taken to transport the samples to the laboratory. An analytical computer calculation takes at least 15 minutes; a numerical computer calculation will take a minimum of about 2 hours.

Fig. 91. Time estimates for the stability calculation of a 15 m high slope.

APPENDIX I TABLES - SLOPE STABILITY PROBABILITY CLASSIFICATION (SSPC)

time unit or formation	lithostratigraphic unit	lithostratigraphic sub-unit(1)	number of classifications: SSPC	number of classifications: engineering geological mapping (see preface)	description(2)
Tertiary	congl/sst			52	Brown/yellowish, CONGLOMERATE and SANDSTONE.
Jura	limest.			32	Off-white/l.grey, argillaceous to arenaceous, LIMESTONE AND DOLOMITE.
Keuper (Tg3)	Tg3 dolo(mite)		18	13	20 - 100 cm thick off-white/l.grey, argillaceous to fine arenaceous, LIMESTONE AND DOLOMITE.
	Tg3 shale		16	16	Red/green/greenish blue/brown/yellow/off-white, argillaceous to fine arenaceous, calcareous sandy silty SHALES, with (small) quantities of gypsum.
Upper Muschelkalk (Tg23)	Lime-stone and dolomite	Tg23 v.thick		5	Off-white/l.grey/yellowish grey, argillaceous to fine arenaceous, LIMESTONE AND DOLOMITE.
		Tg23 thick		8	
		Tg23 medium	6	38	
		Tg23 thin		22	
		Tg23 v.thin	3	26	
		Tg23 thick lam.	2	14	
		Tg23 thin lam.			
Middle Muschelkalk (Tg22)	Tg22		6	28	Red (occasionally greenish grey), argillaceous to fine arenaceous, gypsiferous clayey sandy SILTSTONE; large quantities of gypsum up to occasionally 80 %.
Lower Muschelkalk (Tg21)	Lime-stone and dolomite	Tg21 v.thick	3	8	Off-white/l.grey, arenaceous, LIMESTONE AND DOLOMITE (CALCARENITE).
		Tg21 thick	20	35	
		Tg21 medium	27	48	
		Tg21 thin	9	17	
		Tg21 v.thin	6	6	
Buntsand-stone (Tg1)	sand-stone	Tg1 sst mass.		4	Red/brown, coarse arenaceous (bottom) to fine arenaceous (top), SANDSTONE.
		Tg1 sst v.thick		6	
		Tg1 sst thick	1	19	
		Tg1 sst medium	1	17	
		Tg1 sst thin		3	
		Tg1 sst v.thin		2	
	conglom-erate	Tg1 congl.		7	Red/brown, rudaceous, CONGLOMERATE.
Carbonifer-ous (H)	slate (Hslate)	Hslate thick	1	2	Thick sequences (> 100 m) of d.grey, argillaceous, SLATE.
		Hslate medium	4	10	
		Hslate thin	21	54	
		Hslate v.thin	21	50	
		Hslate thick lam.	14	24	
		Hslate thin lam.	1	8	
	Hcongl		9	15	Grey/brown, MICRO CONGLOMERATES.
	sand-stone (Hsst)	Hsst mass.		4	Grey/brown, SANDSTONES AND SILTSTONES.
		Hsst v.thick	1	1	
		Hsst thick	6	16	
		Hsst medium	21	42	
		Hsst thin	5	20	
		Hsst v.thin	5	8	
		Hsst thick lam.	1	3	
	Hgneiss			2	Black (white foliated), GNEISS.
intrusive	granodiorite		18	64	L. to d. grey, fine to coarse grained, GRANODIORITE (sometimes porphyritic).
	aplite		4	21	D. grey, v. fine grained, APLITIC DYKES
		Total:	250	770	

Codes refer to the codes used on the geological map sheets of the area (Table 1, page 17) (sst = sandstone, congl = conglomerate).

Notes:

1 Lithostratigraphic sub-units are defined on bedding spacing for limestone, dolomite and sandstone, and on cleavage spacing for slate. Cleavage or bedding spacing according to BS 5930 (1981): mass. = no bedding visible, v.thick = > 2 m, thick = 0.6 - 2 m, medium = 0.2 - 0.6 m, thin = 0.06 - 0.2 m, v.thin = 0.02 - 0.06 m, thick lam. = 0.006 - 0.02, thin lam. = < 0.006 m.

2 Descriptions according to BS 5930 (1981) (l. = light; d. = dark; v. = very; mod. = moderately; extr. = extremely).

Table A 17. Formations, lithostratigraphic units and sub-units.

probability %	$TC = a + b * \varphi + c * \varphi^2$					
	sliding			toppling		
	a	*b*	*c*	*a*	*b*	*c*
5	-2.11e-01	1.58e-02	-3.85e-05	-1.62e-01	1.46e-02	-5.71e-05
10	-1.75e-01	1.49e-02	-3.15e-05	-1.13e-01	1.30e-02	-4.14e-05
20	-1.12e-01	1.33e-02	-1.75e-05	-6.99e-02	1.21e-02	-3.27e-05
30	-7.86e-02	1.25e-02	-1.06e-05	-2.11e-02	1.07e-02	-1.90e-05
40	-5.37e-02	1.18e-02	-4.70e-06	1.22e-02	9.73e-03	-9.87e-06
50	-2.76e-02	1.12e-02	5.06e-07	4.51e-02	8.77e-03	-4.47e-07
60	-1.08e-02	1.09e-02	3.65e-06	8.80e-02	7.43e-03	1.29e-05
70	1.89e-02	1.00e-02	1.20e-05	1.13e-01	6.73e-03	2.02e-05
80	4.89e-02	9.29e-03	1.86e-05	1.53e-01	5.60e-03	3.17e-05
90	8.58e-02	8.26e-03	2.87e-05	1.98e-01	4.51e-03	4.32e-05
95	1.25e-01	7.35e-03	3.74e-05	2.45e-01	3.19e-03	5.65e-05

Table A 18. Coefficients for polynomials of equal probability for sliding and toppling criteria (correlation coefficients > 0.999) (SSPC).

probability %	$$\frac{H_{\max}}{H_{slope}} = \left(p0 + \frac{p1}{\log_e(x)}\right)^2 \qquad x = \frac{\varphi_{mass}}{dip_{slope}}$$ *ranges*: $$0.1 \le \frac{\varphi_{mass}}{dip_{slope}} < 1$$ $$10° \le dip_{slope} \le 90° \quad and \quad 2\ m \le H_{slope} \le 50\ m$$ φ_{mass}, dip_{slope} *in degrees*; $H_{\max}$, H_{slope} *in metres*		
	p0	p1	R2
without influence of uncertainty in weathering and method of excavation			
5	0.8592	0.02732	0.25
10	0.9074	0.02341	0.46
20	0.9211	0.01205	0.11
30	0.9655	0.00444	0.25
40	0.9955	0.00219	0.17
50	1.0047	-0.00607	0.36
60	1.0260	-0.00941	0.78
70	1.0416	-0.01676	0.69
80	1.0665	-0.02341	0.83
90	1.1160	-0.04117	0.73
95	1.1978	-0.05644	0.49

The formula and factors have no meaning other than representing a best fit for the points of equal probability within the indicated ranges. The scatter strongly increases for probabilities less than 50 % which causes the low correlation coefficients (R2).

Table A 19. Lines of equal probability for orientation independent slope stability (SSPC).

APPENDIX II STEPS ON DISCONTINUITY PLANES

Introduction
The importance of intact rock strength in slope stability is limited. Slope failures are virtually never due to intact rock failure (ch. C.3.2.1), but slope failures are often related to shear failure along a discontinuity. If both sides of the discontinuity are interlocked by, for example, steps on the discontinuity plane, the steps have to be broken before displacement along the discontinuity can take place (Fig. A 92). This mechanism is related to the intact strength of the steps and may have an impact on how intact rock strength should be included in a slope classification system. The influence of interlocking by steps of discontinuity planes on slope stability can be analysed only very generally as the location and existence of 'steps' inside the rock mass is unknown. The following analysis is done to establish how intact rock strength should be included in a slope stability classification system with respect to the stabilising effect of steps on discontinuities. Two questions are raised:

1 How accurately must intact rock be measured to be certain that the stabilising effect of steps on discontinuity planes on slopes can be estimated with enough accuracy ?
2 Is there a certain value (cut-off value) where above the intact rock strength is of no importance for the stabilising effect of steps, and can dimensions of steps, necessary to stabilise a slope, be established ?

The intention of the analysis is not to produce an exact analysis or weighting factors for a slope stability classification system.

1 How accurately must intact rock be measured to be certain that the stabilising effect of steps on discontinuity planes on slopes can be estimated with enough accuracy ?
The first question can be answered very simply. Steps on discontinuity planes cannot be observed and their location is unknown. As all rock material and rock masses inhibit inhomogeneity in their intact rock strength, it is obvious that the intact rock strength at the location of a step will never be established accurately. A highly accurate method to establish the intact rock strength is thus not necessary.

2 Is there a certain value (cut-off value) where above the intact rock strength is of no importance for the stabilising effect of steps, and can dimensions of steps, necessary to stabilise a slope, be established ?
Consider a slope with a discontinuity plane dipping in the same direction as the slope dip and 'day-lighting' (Fig. A 92). The surface of the discontinuity plane is smooth planar for small scale roughness and straight for large scale roughness (for roughness descriptions see ch. C.4) except for a single step somewhere near the bottom end of the discontinuity plane. Most stepped discontinuity planes have numerous steps spread over the plane but for simplicity a single step is used in this example. In this hypothetical situation what are the minimum dimensions and what is the minimum intact rock strength of the step for which the rock of the step will not be sheared off or crushed?

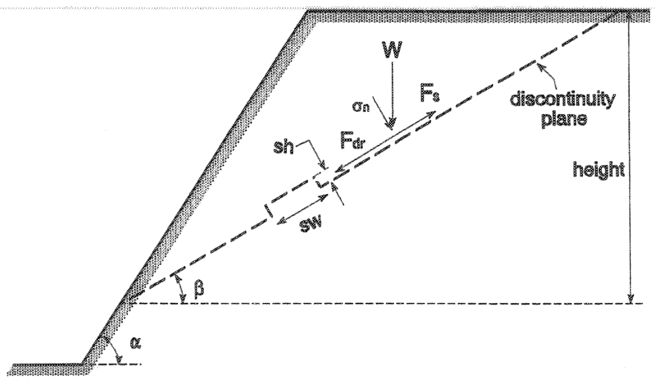


Fig. A 92. Cross-section of a slope with one step on a discontinuity plane.

All formulae and calculations are for a cross section with a length of 1 m along the slope. *UW* is the dry unit weight of the rock. The weight (*W*) of the block above the discontinuity plane is:

$$W = \frac{1}{2} * height^2 * UW * \left(\frac{1}{\tan\beta} - \frac{1}{\tan\alpha}\right) \quad [50]$$

This results in a normal stress (σ_n) on and a driving stress along the discontinuity plane (σ_{dr}):

$$\begin{aligned} \sigma_n &= \frac{1}{2} * height * UW * \left(\frac{1}{\tan\beta} - \frac{1}{\tan\alpha}\right) * \sin(\beta) * \cos(\beta) \\ \sigma_{dr} &= \frac{1}{2} * height * UW * \left(\frac{1}{\tan\beta} - \frac{1}{\tan\alpha}\right) * \sin^2(\beta) \end{aligned} \quad [51]$$

The driving force (F_{dr}) along the plane is:

$$F_{dr} = \frac{1}{2} * height^2 * UW * \left(\frac{1}{\tan\beta} - \frac{1}{\tan\alpha}\right) * \sin(\beta) \quad [52]$$

Assuming that there is no cohesion along the discontinuity, the restraining force from the shear friction (φ_d) along the discontinuity plane (F_s)[141] is:

$$F_s = \frac{1}{2} * height^2 * UW * \left(\frac{1}{\tan\beta} - \frac{1}{\tan\alpha}\right) * \cos(\beta) * \tan(\varphi_d) \qquad [53]$$

The remaining force along the discontinuity plane (F_r) is:

$$F_r = F_{dr} - F_s \qquad [54]$$

The remaining force F_r, if > 0, has to be counteracted by the step in order to ensure stability. The relation following the Mohr-Coulomb failure criterion between intact rock cohesion ($cohesion_i$), angle of internal friction (φ_i) and the unconfined compressive strength (UCS)[141] is:

$$cohesion_i = \frac{1}{2} * \frac{UCS}{\tan\left(45 + \frac{\varphi_i}{2}\right)} \qquad [55]$$

Assume that a shear plane through the step will be parallel to the discontinuity surface, then the force (F_b) necessary for shearing through a step with width sw is:

$$F_b = \left(cohesion_i + \sigma_n * \tan\varphi_i\right) * sw \qquad [56]$$
$$sw = width\ of\ the\ step$$

For equilibrium $F_r = F_b$. The width of the step necessary to prevent shearing of the step is then:

$$sw = \frac{F_r}{cohesion_i + \sigma_n * \tan\varphi_i} \qquad [57]$$

The rock material can also be crushed by the stresses working on it. This is also a form of shear failure. However, the shear plane in the step will be inclined with respect to the discontinuity plane. The height of the step controls this mechanism. The area of the upper side surface of the step is sh. σ_{sh} is the stress on this surface caused by the remaining force ($\sigma_{sh} = F_r / sh$). σ_n is the confining pressure on the step. This leads to a triaxial stress configuration. Using the Mohr-Coulomb failure criterion the equilibrium value for the UCS of the intact rock for which crushing will not take place is[142]:

$$UCS_{equilibrium} = \sigma_{sh} - \sigma_n * \tan^2\left(45 + \frac{\varphi_i}{2}\right) \qquad [58]$$

Interlocking by steps of discontinuity planes in slopes in the research area

The height of the slopes in the research area ranges characteristically between 2 and 25 m with a maximum of about 45 m. For this example assume a block of rock on the discontinuity with a height of 15 m, unit weight (UW)[143] = 25 kN/m^3 and the overall friction angle for the discontinuity plane (φ_d) without the step is 25° (this is the lowest value measured in the research area). The intact rock cohesion ($cohesion_i$) is 23 MPa and the angle of internal friction for intact rock is φ_i = 40° (UCS = 100 MPa). This φ_i is not very critical because the normal stress on the step is small. Fig. A 93 shows the width of the step (sw) necessary for equilibrium, versus the discontinuity dip (β) for various slope dips (α). The maximum step width of approximately 5 cm occurs for a slope dip (α) of 90° and a discontinuity dip (β) of approximately 52°.

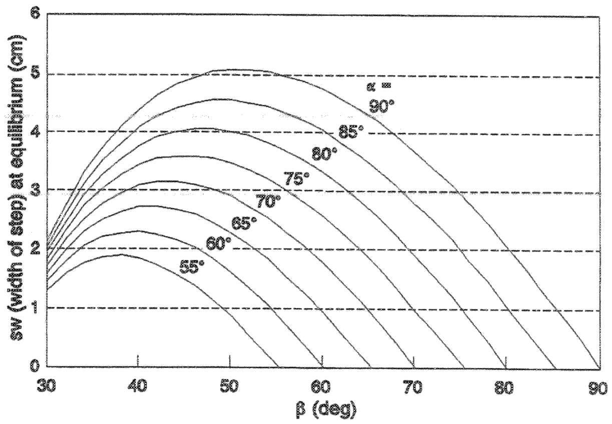


Fig. A 93. Width of step (sw) necessary for equilibrium, vs dip of discontinuity (β) for various slope dips (α).

(141) The shear strength along discontinuities and the strength of intact rock in this chapter are calculated according to the Mohr-Coulomb criterion. This may be too simple and not accurate for most rock material, but it is accurate enough to illustrate the influence of steps on a discontinuity plane.

(142) In this example the stress on the side of the step (σ_{sh}) is due to the whole block (there is only a single step) and is therefore very large compared to the normal stress on the step (σ_n).

(143) Rock mass unit weights for the units in the research area are around 25 kN/m^3.

Fig. A 94 shows the *UCS* value necessary for equilibrium (for which no crushing occurs), versus the height (*sh*) of the step. The curves are for each slope dip (α) with a discontinuity dip for which the maximum F_r is obtained ($\delta F_r / \delta \beta = 0$, the maxima in Fig. A 93).
Provided that the step is wide enough to prevent shearing-off the step completely, then Fig. A 94 shows that for a *UCS* value of 100 MPa with a slope dip (α) of 90° and a discontinuity dip (β) of approximately 52° (the maximum in Fig. A 93), a step height of approximately 13 mm is enough to prevent crushing of the intact rock material. The *UCS* has to be > 150 MPa for equilibrium if the step is less than ≈ 3 mm high. Most rocks have an intact rock strength of less than 150 MPa so that the height of steps should be in the order of ≈ 3 mm or more to prevent crushing of the step[144]. The conclusion is that a relatively small step (in width and height) is enough to stabilize a slope.

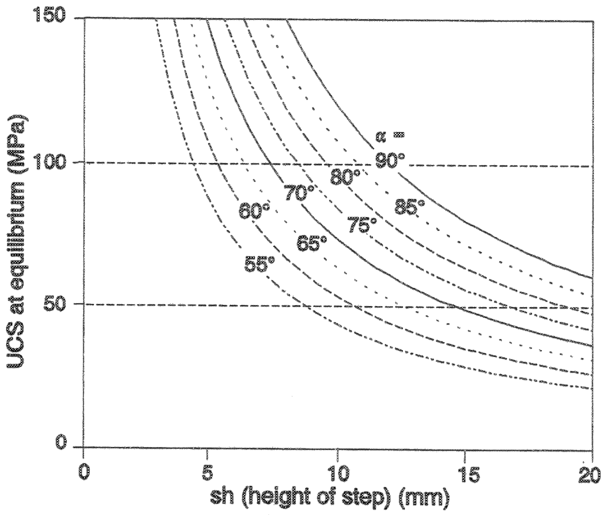


Fig. A 94. $UCS_{equilibrium}$ vs height of step (*sh*).

Dynamic effects
At many locations in the research area steps on discontinuity surfaces in failed slopes have been sheared off. As this cannot always be explained by static force equilibrium (see above), alternatives as weakening of the intact rock material due to weathering, intact rock creep, progressive failure or dynamic effects should be considered as possible causes for the shearing off of steps. On many of the surfaces with sheared off steps no indication of weakening was observed visually or determined by Equotip measurements (ch. C.3.3.3). If the steps are not weakened then the shearing of the steps may have been caused by dynamic effects.
Dynamic effects can be triggered, for example, by earthquakes[145], blasting and vibrations caused by heavy road traffic, thunder storms, etc.. Blasting is likely the reason for steps to be sheared off during the excavation of the slope. Blasting, however, together with stress relief or rock mass creep can also have caused displacements in the rock mass so that opposing step faces are not any more interlocked. The discontinuity shear strength is then determined by the friction along the discontinuity plane only.
If opposing steps on a discontinuity plane are not in contact, it can be calculated that dynamic impact of steps creates stresses in the steps that cause shearing or crushing of the step. This is illustrated with the following example. Assume that equilibrium exists between the restraining force and the driving force of the weight of the block in Fig. A 92. Assume that the rock mass on top of the discontinuity in Fig. A 92 can move by 1 mm before the opposing step faces are in contact. For a slope dip of 90° and a discontinuity dip of 52° the energy of the rock mass acquired by moving over 1 mm is:

$$\begin{aligned} Energy_{mov} &= mass * acceleration * displacement \Rightarrow \\ Energy_{mov} &= F_r \ (remaining\ force\ along\ discontinuity\ plane) * 1\,mm \Rightarrow \\ Energy_{mov} &= 1.1\,MN * 0.001\,m = 1100\,N.m \end{aligned} \qquad [59]$$

This energy has to dissipate in the rock material of the step at the moment of contact between the opposing step faces. The energy will then be changed into elasto-plastic deformation of the rock of the step. The elasto-plastic energy of the deformed step is:

$$\begin{aligned} Energy_{elas} &= \sigma_{def} * area * dsw \quad and \quad \frac{dsw}{sw} = \frac{\sigma_{def}}{E_{elas}} \\ Energy_{mov} &= Energy_{elas} = \frac{\sigma^2_{def} * area * sw}{E_{elas}} \end{aligned} \qquad [60]$$

dsw = *the deformation of the step in down dip direction* sw = *width of the step* = 0.05 m
area = $height_{step}$ * *length of step* (= 0.013 * 1 m^2)

[144] This value is used as a guide for the indication of the scales for the roughness profiles (ch. D.3.1).

[145] The research area is not known to have undergone any earthquakes in recent times.

For example with an elastic modulus (E_{elas}) of the limestone of the Lower Muschelkalk which equals 45 GPa[146]:

$$\sigma_{def} = \sqrt{\frac{Energy_{mov} * E_{elas}}{area * sw}} = \sqrt{\frac{1100\ N.m * 45\ GPa}{0.013\ m^2 * 0.05\ m}} = 276\ MPa \qquad [61]$$

The maximum stress during impact is then 276 MPa. This is three to four times the intact rock strength of Lower Muschelkalk and will lead to crushing or shearing of the step. Similar results are obtained for other units in the research area.

Conclusions

A highly accurate method to establish the intact rock strength is not necessary as far as the stabilising effect of steps on discontinuities is concerned because the location of steps is unknown and as every rock mass inhibits inhomogeneity in the intact rock strength it will never be possible to establish the strength of steps with a high accuracy. The above analyses are done for the situation that only one step on the discontinuity plane is present. This is hypothetical because (nearly) always multiple steps will be present along a discontinuity plane. The widths, heights and required intact rock strength of the steps necessary to stabilize a slope are then equivalently lower. In the field has also been observed that steps are normally considerably wider than the minimum dimensions calculated above. This leads to the conclusion that the intact rock strength will usually be too high to allow shearing or crushing of steps.

(146) Laboratory test value for Lower Muschelkalk limestone which is determined on a UCS sample: diameter ≈ 4.5 cm and length ≈ 10 cm.

APPENDIX III CORRELATION OF THRESHOLD VALUES OF SLIDING CRITERION TO TEST AND LITERATURE VALUES

Correlation of the threshold values of 'sliding criterion' to test and literature friction values

The 'sliding criterion' is based on the assumption that the friction angle along the discontinuity plane, is equal or larger than β (= apparent discontinuity dip in the direction of the slope dip). In this appendix the threshold friction angles obtained from the 'sliding criterion' are compared to the friction angles resulting from laboratory and field tests done in the context of this research and to friction values found in the literature. The discontinuity condition parameter (*TC*) and the 'sliding criterion' in this chapter are calculated as defined in ch. D.1.2.1.5 and include thus the refinements for the calculation of the parameter *TC*. In the following analyses the 'sliding criterion' is re-calculated for the different parameters in the 'sliding criterion'. For example, in the analysis of the small scale roughness (*Rs*) the 'sliding criterion' is calculated for a situation that only small scale roughness is present, thus large scale roughness is straight, and that no infill or karst are present in or along the discontinuity.

Small scale roughness (Rs)

The threshold friction angles from the 'sliding criterion' are plotted versus the small scale roughness description in Fig. A 95 a and b for planes with large scale roughness straight, no infill and no karst. The threshold friction angles are then only dependent on the small scale roughness. Observed planes, measured and characterized in the field, with these specifications and that plot within a 20 % band of the 'sliding criterion' are plotted to verify that these planes actually exist in reality.

'Sliding criterion' compared to tilt and shearbox tests

Fig. A 95a shows the results of field tilt tests (tilt angle) and Fig. A 95b shows the results of laboratory shearbox tests. The shearbox values are not corrected for dilatancy. Also plotted are the results of shearbox tests performed on (artificial) plaster samples (Grima, 1994). The linear regression lines between roughness description and friction angle found for tilt tests and shearbox tests are approximately the same. The tilt tests and shearbox tests show neither a dependency on rock material type nor on non-softening mineral coatings on the discontinuity surface (e.g. hematite coatings that were present on the discontinuity surfaces of some of the slate samples). This is in accordance with the literature (ch. C.3.3.4.3). The graphs show a fairly large scatter which does not allow for a statistical evaluation; the linear regression lines in the graphs are an indication of a trend rather than a correlation. A good fit between β, tilt angle and shearbox friction values cannot be expected. The tilt and shearbox tests are done on sample blocks extracted from the slope. The extraction process can easily break the cohesion and damage the discontinuity planes. In particular sharp asperities, that cause the highest *i-angle*, are easily broken. Secondly during extraction and preparation of the sample, the sample halves are nearly always taken apart and re-fitted for the tilt or shearbox tests. The cohesion that might have been present is broken and the re-fitting will often not be as good as the original in-situ fit of the sample halves. A not so good fit will result in a lower *i-angle* (Rengers, 1970, 1971, ch. C.3.3.2.6) and thus also in a lower tilt angle or shearbox friction value and as it is likely that the higher values are resulting from a high *i-angle* rather than a high φ value, the influence of the sample preparation is obvious.

This is confirmed by the tests on the artificial plaster samples (Grima, 1994). The samples were made exactly according to the ISRM standard graphs (ISRM, 1978b, 1981a) and testing started with perfectly fitting sample halves. Each value is the average of 11 or 12 tests. The average values are considerably higher than the shearbox results on real rock samples but confirm the 'sliding criterion'.

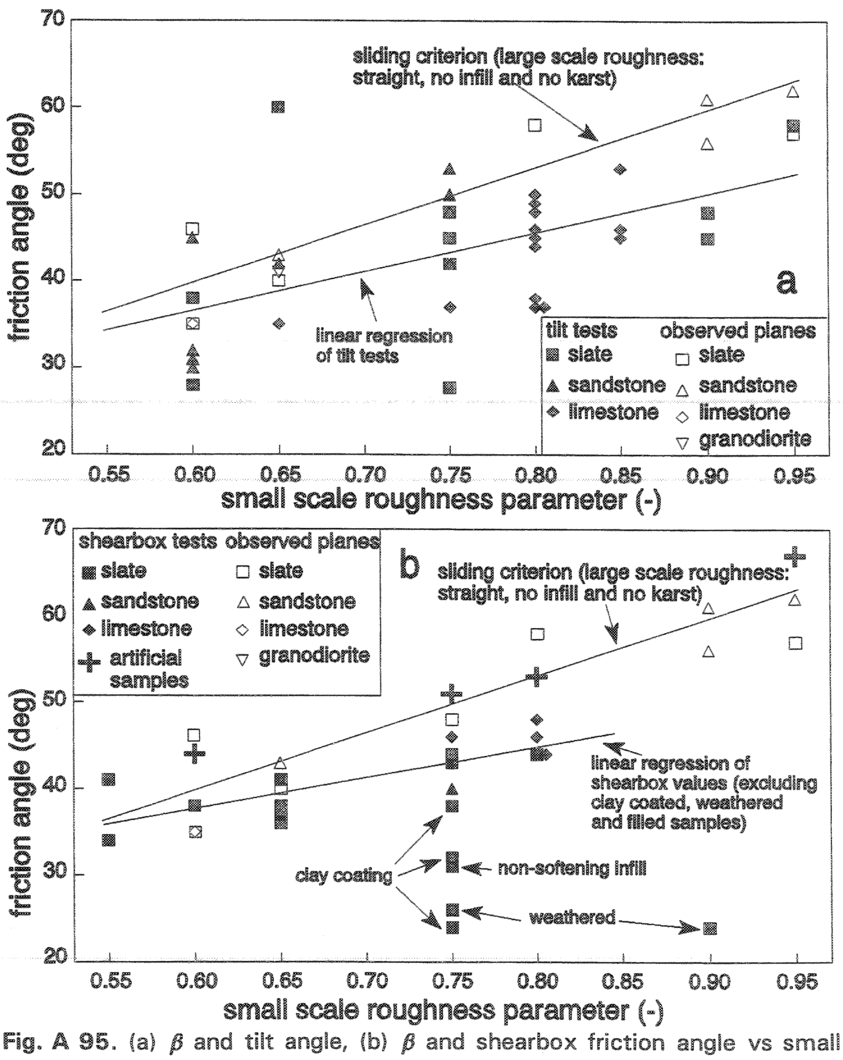


Fig. A 95. (a) β and tilt angle, (b) β and shearbox friction angle vs small scale roughness parameter (roughness parameter values see Fig. 71, artificial samples: Grima, 1994).

'Sliding criterion' compared to literature φ_{basic} values

Values reported in the literature for φ_{basic} range from 23° to 40° (Giani, 1992, Barton, 1973a) (Fig. A 96). The friction for a straight polished planar surface (without infill and no karst) equals 36.5° according to the 'sliding criterion', falling within the range of published values. The differences between φ_{basic} for the different rock types reported in the literature are small and for many less than the range measured for one type of rock. This was also found for the 'sliding criterion' which does not show any significant difference in friction values for different rock types (Fig. 42).

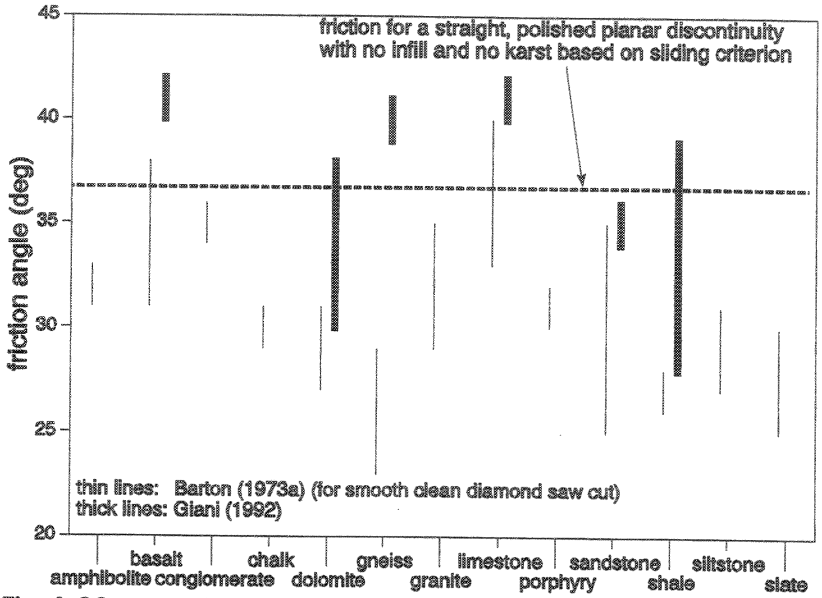


Fig. A 96. φ_{basic} values following 'sliding criterion', Barton (1973a) and Giani (1992).

'Sliding criterion' compared to small scale roughness literature values

It is difficult to compare literature values for small scale roughness with the 'sliding criterion' because the descriptions of the roughness in the literature are not uniform, standards are often not reported or a reference is made to the JRC number. The conversion of JRC numbers into the ISRM roughness descriptions is subjective and possible without ambiguity only for some roughness profiles (Barton, 1987, 1990b). However, an attempt to compare literature friction values with the threshold friction values obtained from the 'sliding criterion' has been undertaken in Fig. A 97. The friction values for small and intermediate scale roughness description (J_r) from the Q-system classification as reported by Barton et al. (1990b) are dependent on the joint alteration number (J_a parameter. J_a = 1.0[147], surface staining only should be compared with the 'sliding criterion' in Fig. A 97. The values are reasonably in agreement with the 'sliding criterion'. The values reported by Barton et al. are established by tilt tests that are reported to be unreliable for stepped surfaces (Barton et al., 1990b). Rough stepped roughness in the 'sliding criterion' is therefore compared with discontinuous joints in the Q-system.

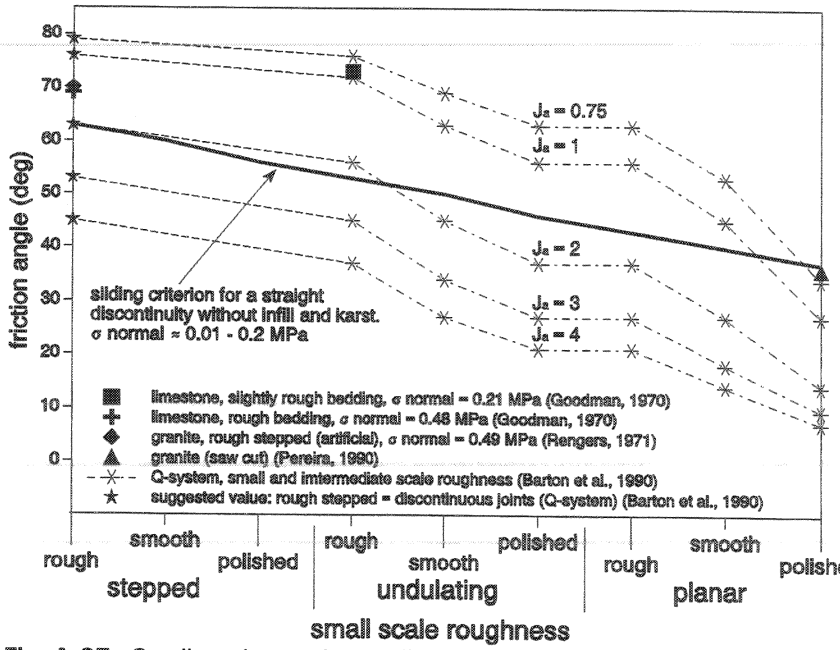


Fig. A 97. Small scale roughness literature values (Barton et al., 1990, Goodman, 1970, Rengers, 1971, Pereira, 1990) and 'sliding criterion' vs surface description.

(147) J_a descriptions (Barton et al., 1990b):

J_a	= 0.75	Tightly healed, hard, non-softening, impermeable filling, i.e. quartz or epidote.
J_a	= 1.0	Unaltered joint walls, surface staining only.
J_a	= 2.0	Slightly altered joint walls. Non-softening mineral coatings, sand particles, clay-free disintegrated rock, etc..
J_a	= 3.0	Silty- or sandy-clay coatings, small clay fraction (non-softening).
J_a	= 4.0	Softening or low friction clay mineral coatings, i.e. kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays.

Discussion influence of small scale roughness

McMahon (1985) reported that small scale roughness is not important for shear friction along large (e.g. 30 to 250 m) discontinuity planes. This is based on comparison between peak and residual friction values from laboratory tests, intermediate and large scale field roughness measurements and back-calculated friction values from failed slopes. Bandis (1983) found that the peak friction angle value (in laboratory tests) decreased with a larger test surface as did the difference between peak and residual friction angles. During this research the friction values derived from the 'sliding criterion' are considerably higher than the values obtained by shear and tilt testing and they show an increase in friction angles with increasing small scale roughness. The difference between the tendencies obtained during this research and those reported in these literature may be the following:

1) The literature values from real failed slopes are based on shearbox tests and roughness descriptions and measurements on discontinuities. After a sliding failure the discontinuity plane that failed will have a different roughness profile and is unsuitable for back analysis (ch. C.3.3.2.6). For this reason it is good practice that both the roughness profiles are measured and the test samples are taken from other discontinuity planes in the same slope that are representative for the failure plane. However, then the question arises: why did these planes not fail? Obviously a number of reasons are possible (differences in orientation, water pressures, etc.). It is, however, also possible that the friction along these planes is (slightly) higher than the plane that failed and that thus also a larger value for roughness friction is obtained. A friction value back calculated from the failed plane compared to the friction (roughness measurement and shearbox tests) from the non-failed discontinuities results in seemingly less important roughness of the discontinuity plane.
2) The laboratory test results by Bandis (1983) are presently questioned because the results are based on averages while the scatter of results from individual tests is large. It is not unlikely that due to the equipment used (non-computerized shearbox) inaccuracies in the individual results masked the influence of small scale roughness. It is doubted whether the conclusions would be the same if the tests are repeated in a modern computerized shearbox (discussion: Second international workshop on scale effects in rock masses, Lisbon, Portugal, 1993).
3) The scale effect between smaller and larger surfaces was also reported to be absent by Ohnishi et al. for artificial samples, and the relation was vague or absent for a repetition of the tests of Bandis on replicas of natural discontinuity surfaces (Ohnishi et al., 1993).
4) Another reason for the seemingly reduced influence of small scale roughness may be the handling of samples in laboratory and field tests. The larger the sample, the more difficult it is to perfectly fit two discontinuity halves together without damaging the asperities. The steepest asperities which are normally the smaller asperities, contribute most to the friction but especially the highest and sharpest asperities are most easily damaged and broken. Secondly the broken parts of these asperities may stay in the discontinuity and cause a (lower) rolling friction. Hence, the influence of small scale roughness seemingly reduces with larger sample size.

Large scale roughness (Rl)

Threshold friction values obtained from the 'sliding criterion' for discontinuities without infill and karst are shown in Fig. A 98 versus the descriptions for large and small scale roughness.

'Sliding criterion' compared to large scale field roughness measurements

During the research a limited number of large scale roughness profiles have been measured. Large scale *i-angles* (20 cm < base < 100 cm) measured on discontinuity planes in slate and limestone resulted in large scale roughness *i-angles* of between 6° and 10° for respectively slightly wavy and wavy surfaces and 5° for slightly curved surfaces. These are lower than the threshold friction values for large scale roughness obtained from the 'sliding criterion' (Fig. A 98). The large scale *i-angles* measurements done for this research have been done on exposed planes. The exposed planes are exposed because the material originally above has slid. This sliding may have reduced the large scale roughness *i-angles*.

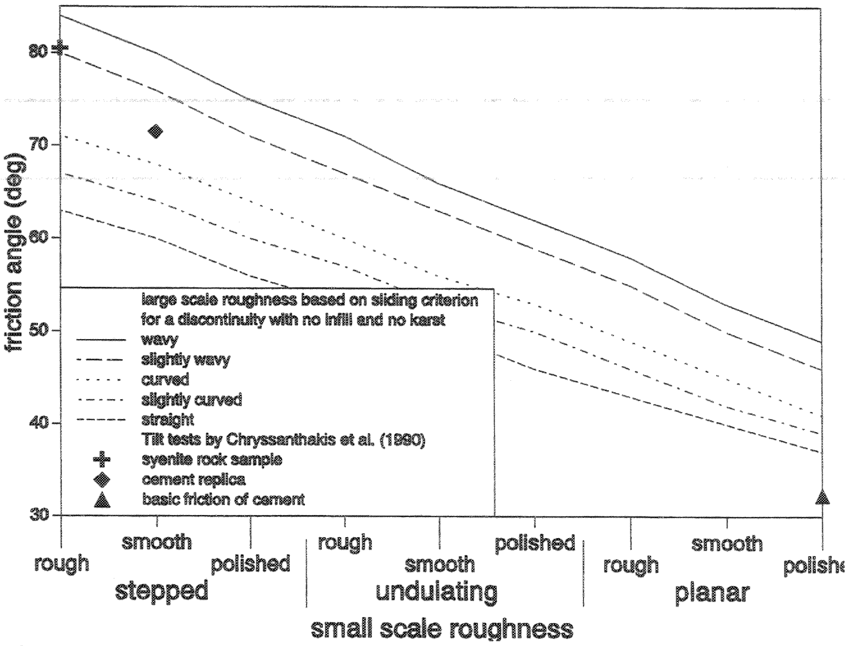


Fig. A 98. Friction vs large & small scale roughness and literature tilt test values of Chryssanthakis et al. (1990).

'Sliding criterion' compared to literature large scale friction values
Literature reporting large scale friction values from tests and that include a complete description of the roughness following ISRM roughness profiles (ISRM, 1978b, 1981a), has not been found. However, tilt test angles from a syenite rock sample containing a discontinuity of 1 metre length and from cement replicas of this sample have been reported by Chryssanthakis et al. (1990) and are plotted in Fig. A 98. The large and small scale roughness is estimated from scale drawings in the publication. The original syenite sample has a large scale roughness of slightly wavy or curved while the visible small scale roughness is stepped. From the description of the sample can be derived that the rock surface is rough. The tilt test angles obtained from the replicas is plotted at a small scale roughness of smooth stepped as it is likely that a cement and fine sand matrix will be less rough than the original rock surface. The large scale tests are reasonably in agreement with the 'sliding criterion'. The roughness description of these profiles was, however, not reported and the description has been estimated from scale drawings in the publication. This might well underestimate the large and small scale roughness.

Infill material (Im)
Fig. A 99 shows the threshold friction values for different infill materials obtained from the 'sliding criterion' plotted as a continuous line. The values are calculated for discontinuities with no karst, large scale roughness straight and small scale roughness polished planar.

'Sliding criterion' compared to literature values for infill material in natural discontinuities
In Fig. A 99 shows literature values for different infill materials (Hoek et al., 1981). The literature values are peak shear strength φ for filled natural discontinuities. As far as the infill thicknesses were reported these are included in the graph. Also indicated are the residual friction ranges listed in the Q-system (Barton, 1988). The shear strength friction values based on the 'sliding criterion' are correlate with the literature values.

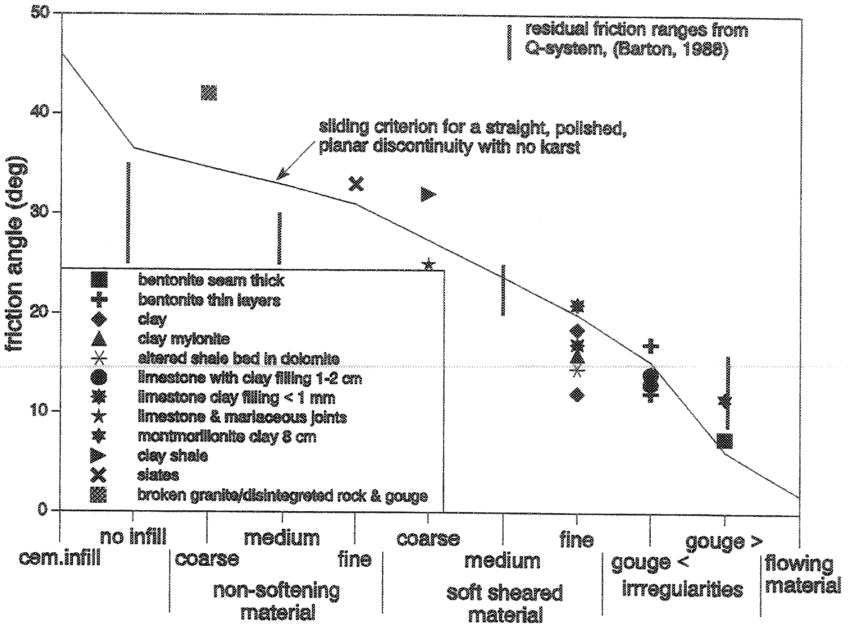


Fig. A 99. Friction angle vs infill material (values from Hoek et al., 1981, vertical lines from Barton, 1988).

'Sliding criterion' compared to literature values for infill material from artificial samples
It is difficult to compare threshold friction values obtained from the 'sliding criterion' with laboratory tests on artificial discontinuities or on discontinuities with infill materials as reported in the literature (Papaliangas et al., 1990, Pereira, 1990). The materials and the circumstances under which these discontinuities were tested are, in general, very different from natural materials and circumstances. Additionally the normal stress on the samples during testing is often far higher than the stresses in the slopes in this research. In Fig. A 100 values from Papaliangas et al. (1990) are compared to the infill friction values resulting from the 'sliding criterion'. The roughness of the sample discontinuity was not described following ISRM recommendations, but the friction angle (33°) for a saw cut (planar surface) and the friction angle (52°) for the surface of the test sample without infill, were reported and from these values the roughness parameter according to the 'sliding criterion' could be back calculated and resulted in 'rough undulating' (small scale roughness). The samples used were not large enough to give a large scale roughness, which therefore is taken as 'straight'. The values (Fig. A 100) are fairly well in agreement with the 'sliding criterion' except for the thick infill (infill thickness/roughness amplitude > 1). The high value (24°) for the thick infill compared to the 'sliding criterion' (7°) might well be attributed to the following differences between the circumstances during slope failure and tests:

1 Most failures of slopes occur during or directly after rainy periods, it is therefore not unlikely that in a natural state thick layers of cohesive infill material (clay gouge) causing slope failure will be nearly always saturated. During failure it is likely that this leads to pore water pressures in the infill and thus low friction angles. The kaolin in the tests of Papaliangas et al. was tested with a moisture content of 50 %, however, the degree of saturation is not reported so that these samples might have been tested in a not saturated state with no or less pore water pressure.

2 The discontinuity surface in a slope is far larger than in the sample. This reduces the possibility for discharge of water and thus reduction of pore water pressures in a slope compared to the laboratory tests.

3 The normal stress on the discontinuity plane in the laboratory tests is 50 MPa whereas the normal stresses in the slopes in the research area are in the order of 0.01 MPa. The far higher normal stresses for the tests

can lead to a collapse of the infill material structure and allowing easier discharge of (pore) water and reduction of pore water pressures.

4 The shear velocity in real slopes is often far higher than in the laboratory tests (laboratory: 0.4 - 1 mm/min), also reducing the possibilities for water discharge in slopes.

It is suggested that in slopes water pressures in the clay gouge cause an undrained shear behaviour whereas in the laboratory tests, with none or smaller water pressures, the shear behaviour is drained. The values found by Pereira (1990) (Fig. A 100) and Phien-wej (1990) (not in graph) for an open air dried, silty clay infill and oven dried bentonite infill (38° for 20 mm infill, roughness amplitude 10 mm) respectively, seem to support this suggestion.

The values for non-cohesive soils of Pereira (1990) show that for the two larger grain sizes the friction angle is reduced rather than increased. This effect is attributed by Pereira to rolling friction rather than shear friction (the silicious river sand was rounded).

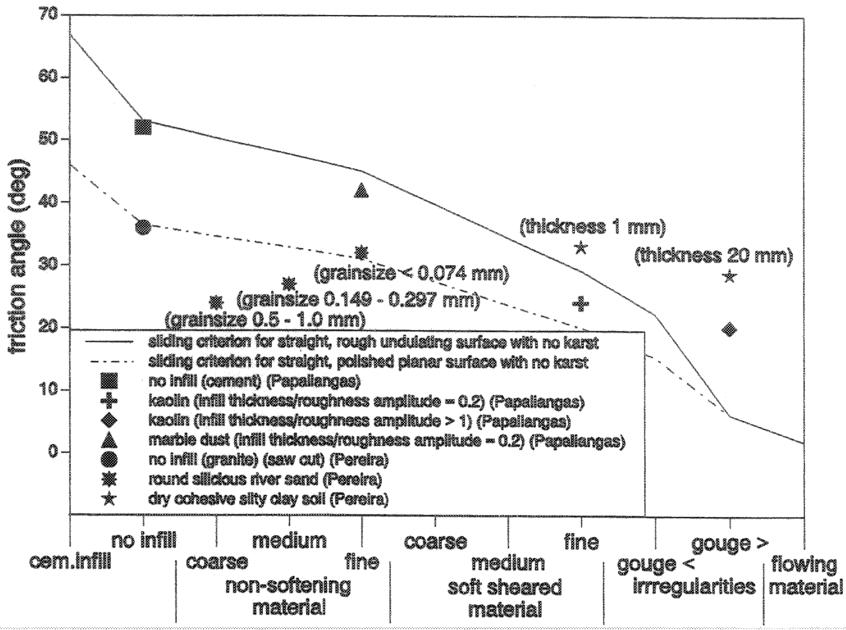


Fig. A 100. Friction angle vs infill material compared to infill thickness laboratory tests. Papaliangas et al. (1990) tests with straight, rough undulating surfaces and Pereira (1990) with straight, polished planar sample surfaces.

Discussion and conclusion

The friction angle values for discontinuities (Bieniawski, 1989, Serafim et al., 1983) related to the descriptions in Bieniawski's (RMR) rock mass classification system (ch. B.2.3.1) are difficult to compare with the threshold friction values found for the 'sliding criterion'. However, according to Serafim et al. the maximum friction along a discontinuity is 45° for a dry discontinuity and 37° for a wet discontinuity. The 'sliding criterion', laboratory and field tests, and the literature references cited in the foregoing chapters, allow for considerably higher maximum values and the merits of the values reported by Serafim et al. should be questioned.

Apparent cohesion is not found for the 'sliding criterion'. This is expected for the more smooth discontinuity planes as the normal stresses in slopes are low compared to the intact rock strength, so that the asperities will mostly not be sheared through, but are overridden. For the rough or stepped surfaces an apparent cohesion was expected but, however, not found. For larger test sample sizes the shear behaviour of a discontinuity is more ductile than brittle (Bandis et al., 1981, 1983, Muralha et al, 1990) and the apparent cohesion decreases. This may explain that apparent cohesion is not present because the rock slopes studied have surfaces ranging between 3 m^2 and 300 m^2 which is, even for the smallest slope, considerably larger than the maximum discontinuity sample size ever tested.
Cohesion resulting from infill material has also not been found. This may be due to the relative small number of discontinuities with a thick infill (gouge) which would have showed cohesion. The other parameters (roughness and karst) may mask the presence of cohesion.

The generally good correlation found between the threshold friction angles determined with the 'sliding criterion' and the friction angles obtained from testing and found in the literature confirm the correctness of the 'sliding criterion' and the definition of the discontinuity condition parameter (*TC*). The 'sliding criterion' is therefore an appropriate method to determine the friction angle along a discontinuity, which is formulated as follows:

$$TC = 0.0113 * \varphi \qquad [62]$$

TC = discontinuity condition parameter
φ = friction angle along discontinuity (in degrees)

APPENDIX IV INFLUENCE OF WEATHERING ON GEOTECHNICAL PARAMETERS

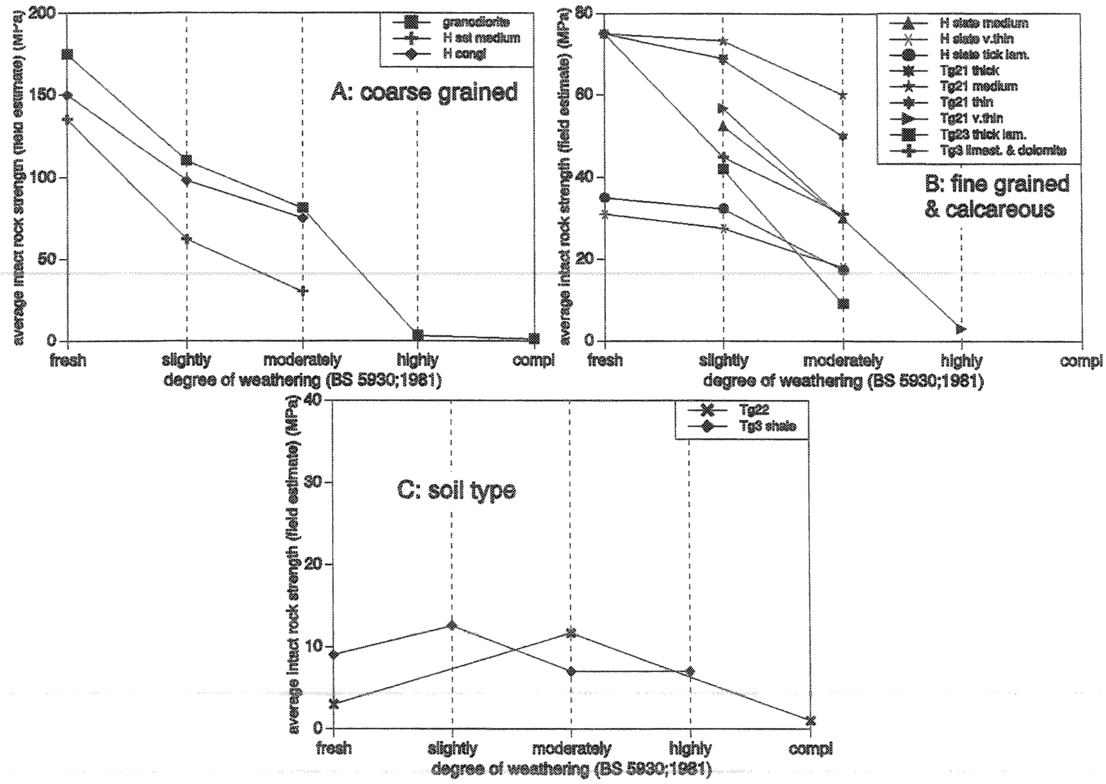


Fig. A 101. Examples of average intact rock strength (field estimate) vs degree of rock mass weathering per lithostratigraphic (sub-) unit.

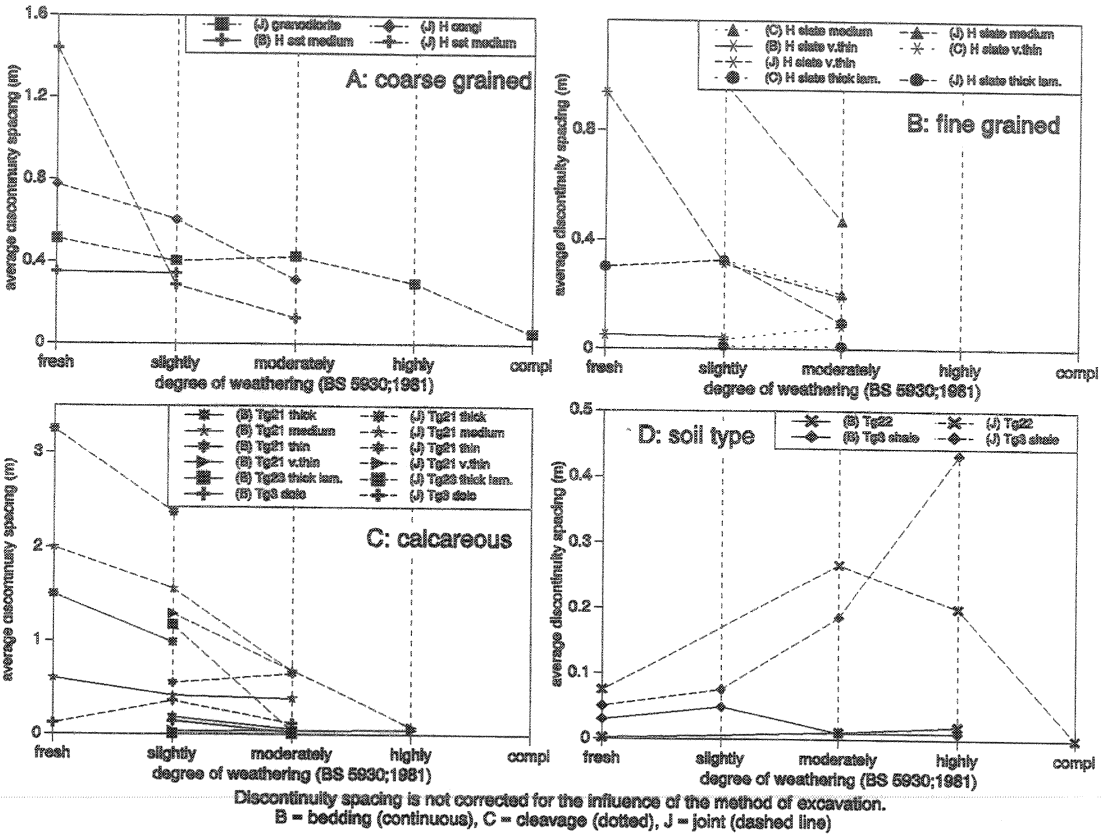


Fig. A 102. Examples of average discontinuity spacing vs degree of rock mass weathering per lithostratigraphic (sub-) unit and per type of discontinuity.

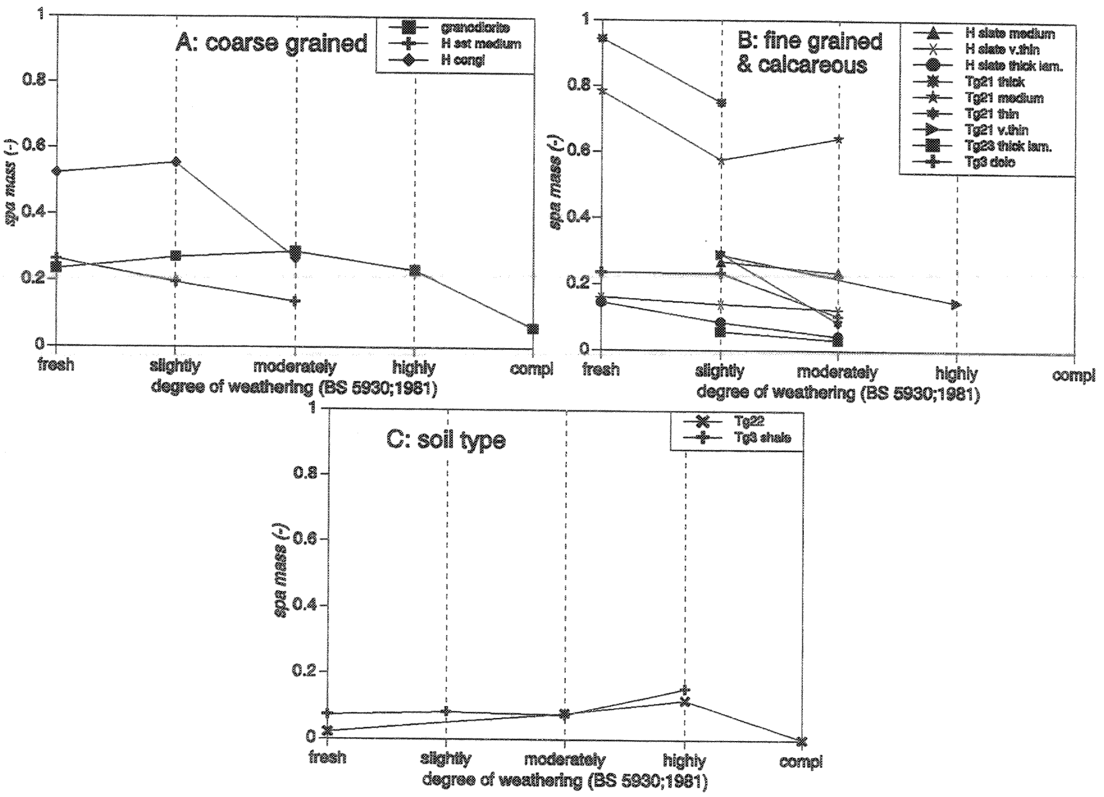
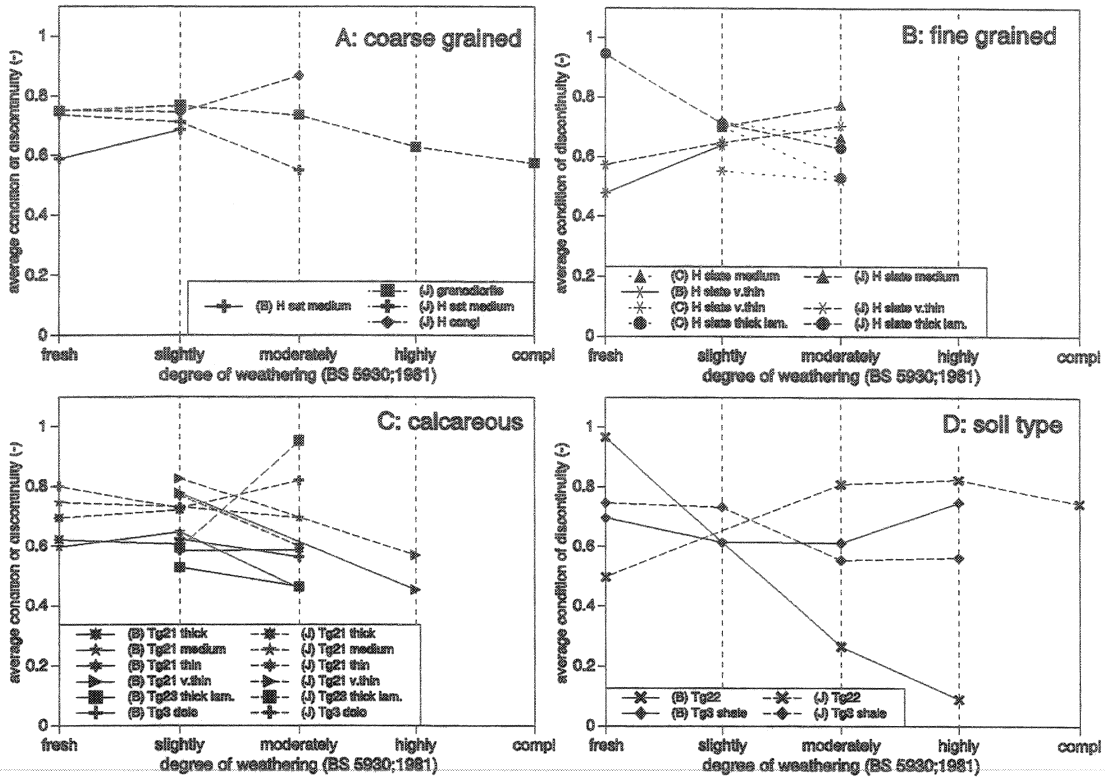
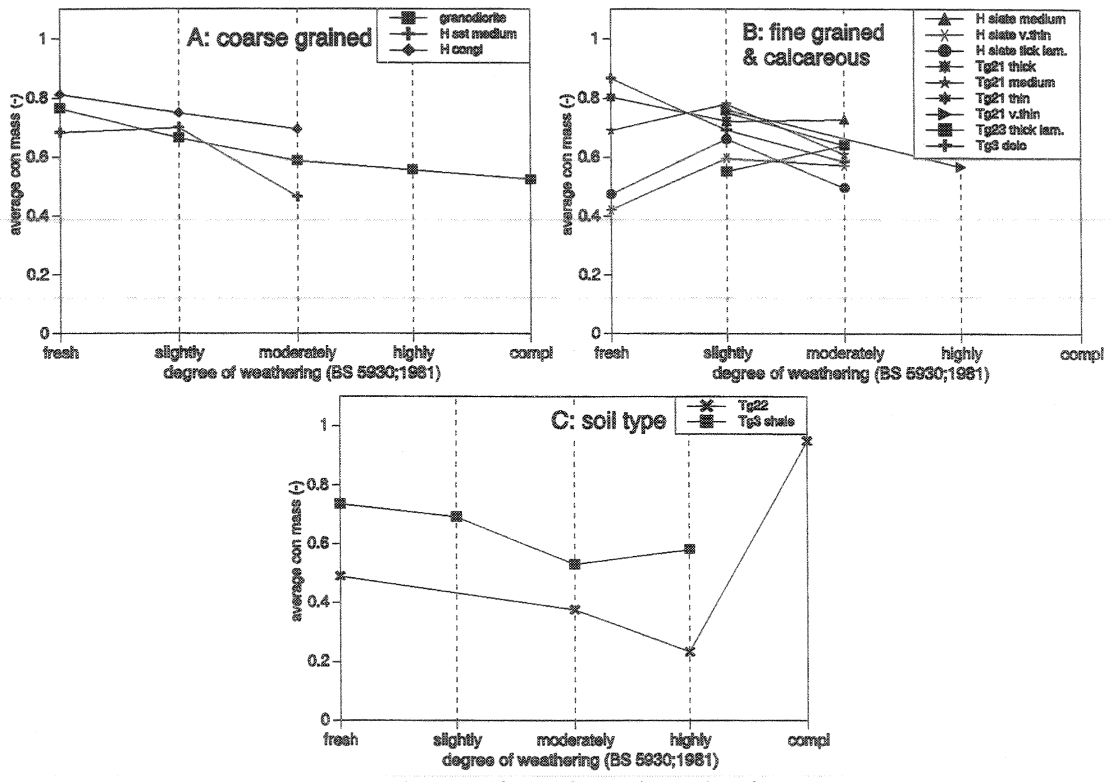


Fig. A 103. Examples of average spa_{mass} vs degree of rock mass weathering per lithostratigraphic (sub-) unit (spa_{mass} corrected for method of excavation).



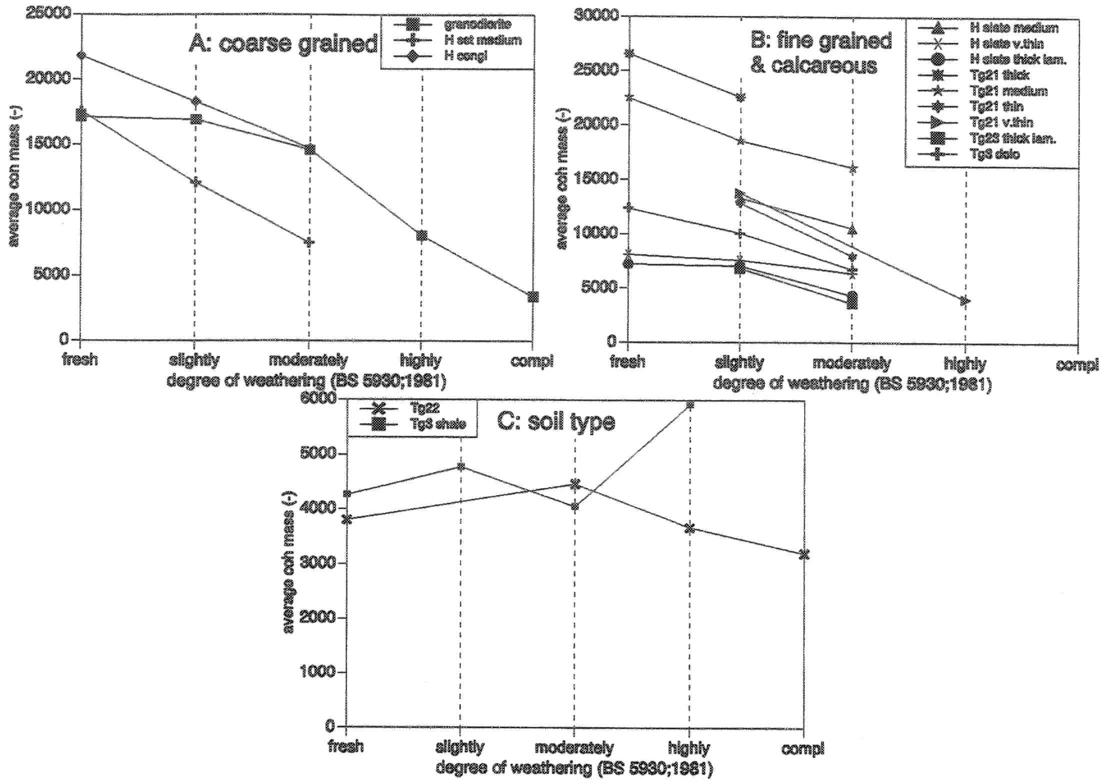
B = bedding (continuous), C = cleavage (dotted), J = joint (dashed line)

Fig. A 104. Examples of the average condition of discontinuity parameter (*TC*) of a single discontinuity (set) vs degree of rock mass weathering per lithostratigraphic (sub-) unit and per type of discontinuity.



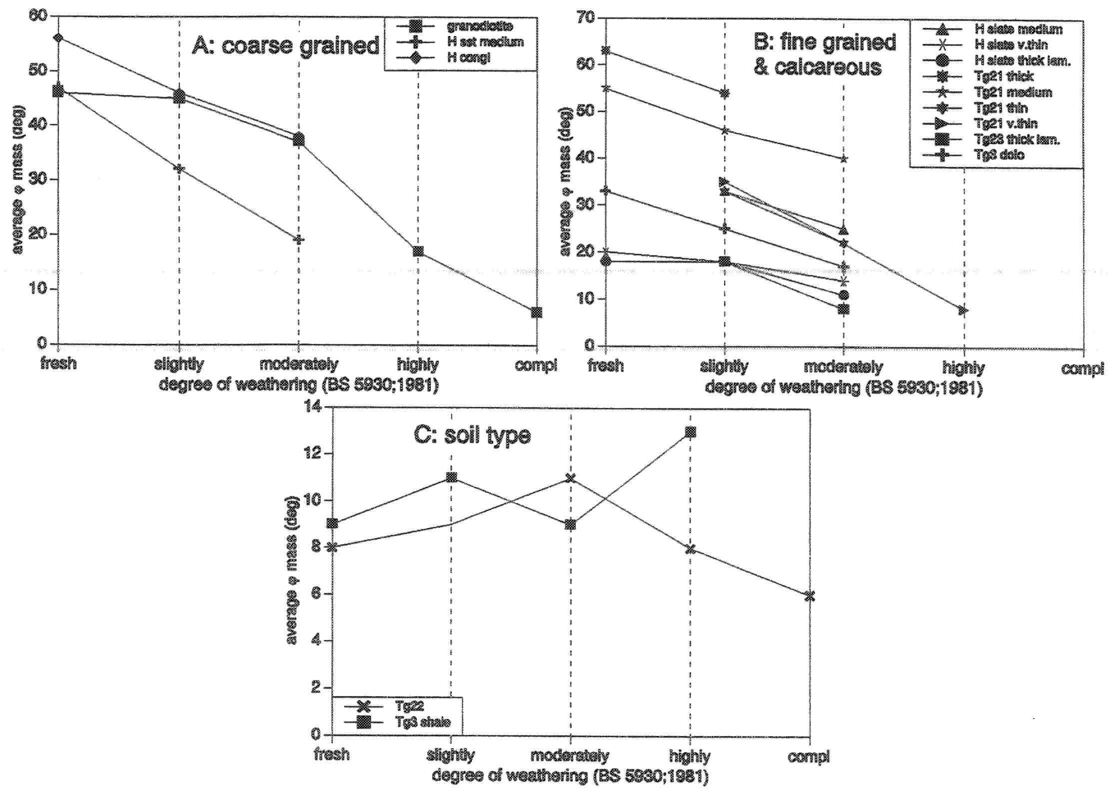
con mass is corrected for the influence of the method of excavation

Fig. A 105. Examples of the average overall condition of discontinuities (con_{mass}) vs degree of rock mass weathering per lithostratigraphic (sub-) unit.



coh mass is corrected for the influence of the method of excavation

Fig. A 106. Examples of average coh_{mass} vs degree of rock mass weathering per lithostratigraphic (sub-) unit.



φ mass is corrected for the influence of the method of excavation

Fig. A 107. Examples of the average φ_{mass} vs degree of rock mass weathering per lithostratigraphic (sub-) unit.

APPENDIX V WEATHERING CLASSIFICATION

Term	Description	degree
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.	I
Slightly weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All rock material may be discoloured by weathering.	II
Moderately weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as core stones.	III
Highly weathered	More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as core stones.	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric is destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

Table A 20. Degrees of rock mass weathering - BS 5930 (1981).

Introduction

In the design of a slope the future degradation of the rock mass due to weathering is of major importance. In the SSPC classification system quantitative reduction values have been defined to accommodate for existing or future weathering. These values are related to the degrees of rock mass weathering as described by BS 5930 (1981, Table A 20). This classification for rock mass weathering has been under criticism and different alternative classifications for weathering have been proposed since its publication in 1981. The author has not noticed that any of these alternative classifications have been widely applied. Recently a new classification scheme for rock and rock mass weathering has been proposed by the Engineering Group of the (British) Geological Society (Anon, 1995). Whether the recommendations given by the Engineering Group will be widely accepted cannot be predicted, however, a comment on this scheme and possibilities to apply this scheme in the SSPC classification system is presented.
The approach proposed by the Engineering Group (Anon, 1995) is composed of a general description of the weathering of the rock and rock mass (named: approach 1) and followed by different classification schemes (approaches 2 through 5) for different types of rock and rock masses (Table A 21).

Quantification of BS 5930 (1981)

Although the scatter in the data is large it is shown in ch. D.1.5 that it is possible to quantify the influence of weathering classified according to BS 5930 (1981). Some differences between the influence of weathering on the geotechnical parameters of different types of rocks and rock masses have been noticed, however, these differences are generally not large. Averaging over different lithologies and types of rock masses was possible and overall parameters for weathering influence could be calculated. Exceptions are the very weak 'soil type' units for which were found, in this research, that weathering does not influence the geotechnical parameters or has only a minor influence (the scatter in the data is larger than the influence of the weathering, ch. D.1.5).

approach 1	General description
approach 2	Prescriptive classification for uniform materials
approach 3	Prescriptive classification for heterogeneous masses
approach 4	Prescriptive classification incorporating both material and mass features.
approach 5	Classification of rocks that cannot be classified with approach 2 through 4, such as limestones developing karst. This classification may be based on associated characteristics (landform, etc.) of the rock mass but not on rock mass parameters directly.

Table A 21. Classification approaches (Anon., 1995).

Absence of weathering degrees
Criticism on the BS 5930 classification of rock mass weathering focuses on the fact that it is not always possible to 'fit' the rock mass into one of the degrees of weathering or that degrees are not applicable to particular rock masses. This criticism particularly focuses on the percentages material decomposed or disintegrated into 'soil' which is one of the main criteria for the BS 5930. Some rocks do not produce 'soil'. As noted before (footnote 107), highly and further weathered rock masses following BS 5930, do not result from weathering of pure limestones or dolomites. The carbonates dissolve in surface and subsurface water. This may result in a karstic rock mass. Whether this should be classified in a different weathering classification system is disputable.

Replacing BS 5930 (1981) by a new classification scheme following the Engineering Group of the Geological Society
The newly designed scheme for weathering classification following the recommendations of the Engineering Group (1995) can be used for the SSPC classification system if the 'approaches' in the newly designed classification system are correlated to the old BS 5930 system in the following way (see also Table A 22):

- *Approach 2 - uniform material*
  Grades I through V from approach 2 describe mainly the weathering of the intact rock in the rock mass. It is proposed to correlate grades III through V of approach 2 of the new system to the degree 'moderately' in the old BS 5930. In these rock types 'highly' and 'completely' weathered according to BS 5930 do not exist.
- *Approach 3 - heterogeneous masses*
  This approach can be correlated directly to BS 5930 if the term 'soil' in the description of BS 5930 is not taken too strict, but is taken equal to the material descriptions of grades IV - VI of approach 2 of the new system.
- *Approach 4 - material and mass*
  This approach can be correlated to the degrees of weathering in the old BS 5930, if class B of 'approach 4' includes both the degrees of 'slightly' and 'moderately' weathered in the BS 5930 classification. The value for $WE_{spacing}$ (ch. D.1.5) decreases considerably from 'moderately' ($WE_{spacing}$ = 0.89) to 'highly' ($WE_{spacing}$ = 0.63) weathered. Therefore, the reduction of discontinuity spacing in the description for class C ('much closer fracture spacing') is more comparable to the degree of 'highly' weathered in the BS 5930 classification than to the degree of 'moderately' weathered.
- *Approach 5 - rock masses not fitting into approach 2 through 4*
  A classification based on, for example, landforms cannot readily be correlated to the rock mass weathering classification following BS 5930.

PROPOSAL FOR THE COMPARISON OF THE WEATHERING SYSTEM BS 5930 (1981) WITH THE NEW PROPOSAL FROM THE ENGINEERING GROUP OF THE GEOLOGICAL SOCIETY (1995)(1)								
degree of rock mass weathering - BS 5930 (1981)		new proposal working group geological society (1995)						
degree	quantitative reduction values for weathering (ch. D.1.5) rock mass strength	approach 2 uniform materials (moderately strong or strong rock in fresh state)		approach 3 heterogeneous masses (mixture of relatively strong and weak material)		approach 4 material and mass (moderately weak or weaker in fresh state)		
	WEmass	grade	description	zone	description(2)	class	description	
fresh	1.00	I fresh	Unchanged from original state	1	100 % grades I - III	A unweather-ed	Original strength, colour, frac-ture spacing	
slightly	0.95	II slightly weathered	Slight discolouration, slight weakening	2	> 90 % grades III < 10 % grades IV - VI	B partially weathered	Slightly reduced strength, slightly closer fracture spacing, weathering penetrating in from fractures, brown oxidation	
moder-ately	0.90	III moderately weathered	- Considerable weakened, penetrative discolouration - Large pieces cannot be broken by hand	3	50 to 90% grades I -III 10 to 50% grades IV - VI			
		IV highly weathered	-Large pieces can be broken by hand - Does not readily disintegrate (slake) when dry sample immersed in water					
		V completely weathered	- Considerably weakened - Slakes in water - Original texture apparent					
highly	0.62			4	30 to 50% grades I - III 50 to 70% grades IV - VI	C distinctly weathered	Further weakened, much closer fracture spacing, grey reduction	
complete-ly	0.35(3)			5	< 30% grades I - III 70 - 100% grades IV - VI	D de-struc-tured	Greatly weakened, mottled, lithorelicts in matrix becoming weakened and disordered, bed-ding disturbed	
residual soil	(4)	VI residual soil	Soil derived by in-situ weathering but having lost retaining original texture and fabric	6	100% grades IV - VI	E residual or reworked	Matrix with occasional altered random or apparent lithorelicts, bedding destroyed. Classed as reworked when foreign inclusions are present as a result of transportation	

notes:
(1) The correlation showed in this table is made by the author and is not a proposal by the Engineering Group of The Geological Society.
(2) Grades refer to approach 2 (uniform materials).
(3) Quantitative reduction values based on granodiorite only.
(4) Not applied in this research.

Table A 22. Proposal for correlation of the degree of rock mass weathering following BS 5930 (1981) and quantitative weathering with the proposal of the Engineering Group of the Geological Society (Anon., 1995).

APPENDIX VI EXAMPLES - SSPC FORMS

ITC/TU ENGINEERING GEOLOGY	exposure characterization	SSPC - SYSTEM

LOGGED BY: zz	DATE: 11/04/92	TIME: 12:00 hr	exposure no: natural exposure B

WEATHER CONDITIONS		LOCATION	map no:	445-iii
Sun:	cloudy/fair/bright	Map coordinates:	northing:	739.698
Rain:	dry/drizzle/slight/heavy		easting:	983.390

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

DIMENSIONS/ACCESSIBILITY				
Size total exposure:	(m)	l: 175	h: 6	d: 8
mapped on this form:	(m)	l: 23	h: 6	d: 8
Accessibility:		poor/fair/good		

FORMATION NAME: tg21 limestone and dolomite

DESCRIPTION (BS 5930: 1981)				
colour	grain size	structure & texture	weathering	NAME
off-white	fine	medium bedded, medium blocky	slightly	calcisiltite

INTACT ROCK STRENGTH (IRS) (tick)		sample number(s):
< 1.25 MPa	Crumbles in hand	
1.25 - 5 MPa	Thin slabs break easily in hand	
5 - 12.5 MPa	Thin slabs broken by heavy hand pressure	
12.5 - 50 MPa	Lumps broken by light hammer blows	
50 - 100 MPa ✓	Lumps broken by heavy hammer blows	
100 - 200 MPa	Lumps only chip by heavy hammer blows (Dull ringing sound)	
> 200 MPa	Rocks ring on hammer blows. Sparks fly	

WEATHERING (WE)	
(tick)	
unweathered	1.00
slightly	✓ 0.95
moderately	0.90
highly	0.62
completely	0.35

DISCONTINUITIES B=bedding C=Cleavage J=joint			B1	J2	J3	4	5
Dip direction		(degrees)	310	287	002		
Dip		(degrees)	06	85	87		
Spacing (DS)		(m)	0.40	0.60	0.40		
persistence	along strike	(m)	> 23	> 8	> 23		
	along dip	(m)	> 8	> 6	> 6		
CONDITION OF DISCONTINUITIES							
Roughness large scale (Rl)	wavy slightly wavy curved slightly curved straight	:1.00 :0.95 :0.85 :0.80 :0.75	0.80	1.00	1.00		
Roughness small scale (Rs) (on an area of 20 x 20 cm²)	rough stepped/irregular smooth stepped polished stepped rough undulating smooth undulating polished undulating rough planar smooth planar polished planar	:0.95 :0.90 :0.85 :0.80 :0.75 :0.70 :0.65 :0.60 :0.55	0.80	0.95	0.80		
Infill material (Im)	cemented/cemented infill no infill - surface staining non softening & sheared material, e.g. free of clay, talc, etc.: coarse / medium / fine soft sheared material, e.g. clay, talc, etc.: coarse / medium / fine gouge < irregularities gouge > irregularities flowing material	:1.07 :1.00 :0.95 / :0.90 / :0.85 :0.75 / :0.65 / :0.55 :0.42 :0.17 :0.05	1.00	0.55	1.00		
Karst (Ka)	none karst	:1.00 :0.92	1.00	0.92	0.92		

EXISTING SLOPE?	
dip-direction/dip	358 / 45
height:	6.0 m
Stability (tick)	
stable	✓ 1
small problems in near future	2
large problems in near future	3
small problems	4
large problems	5

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.) roughness should be assessed perpendicular and parallel to the roughness and directions noted on this form.
3) Non-fitting of discontinuities should be marked in roughness columns.

SUSCEPTIBILITY TO WEATHERING (SW)		
degree of weathering:	date excavation:	remarks:
slightly	> 50 years ago	

remarks: All existing exposures in surroundings are slightly weathered whatever orientation, method of excavation, etc.

Fig. A 108. Example I. Natural exposure B. Exposure characterization.

ITC/TU ENGINEERING GEOLOGY	reference rock mass calculation	SSPC - SYSTEM
CALCULATED BY: zz	DATE: 11/04/92	exposure no: natural exposure B

REFERENCE UNIT NAME: tg21, limestone and dolomite, medium bedded

INTACT ROCK STRENGTH (RIRS)

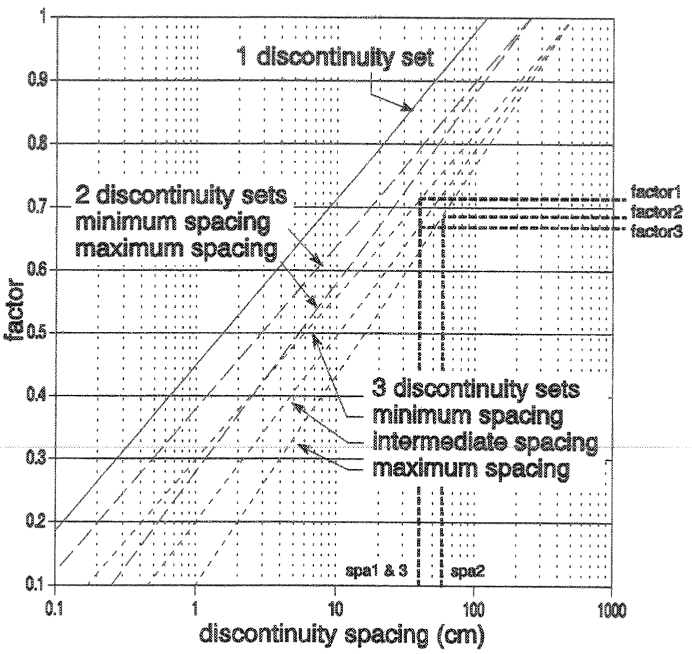
If IRS > 132 MPa then RIRS = 132 else RIRS = IRS (in MPa) / WE (correction for weathering) = 75 / 0.95 = **79**

DISCONTINUITY SPACING (RSPA)

DISCONTINUITIES		B1	J2	J3	4	5
Dip direction	(degrees)	310	287	002		
Dip	(degrees)	06	85	87		
Spacing (DS)	(m)	0.40	0.60	0.40		

The spacing parameter (SPA) is calculated based on the three discontinuity sets with the smallest spacings following figure:

SPA (see figure below) =
factor1 * factor2 * factor3 =
0.71 * 0.69 * 0.68 = 0.33
corrected for weathering and method of excavation:
RSPA = SPA / (WE * ME)
(with a maximum of 1.00)
RSPA = 0.33 / (0.95 * 1.00) = **0.35**



CONDITION OF DISCONTINUITIES (RTC & RCD)

DISCONTINUITIES		B1	J2	J3	4	5
Roughness large scale	(Rl)	0.80	1.00	1.00		
Roughness small scale	(Rs)	0.80	0.95	0.80		
Infill material	(Im)	1.00	0.55	1.00		
Karst	(Ka)	1.00	0.92	0.92		
Total	(Rl*Rs*Im*Ka = TC)	0.64	0.48	0.74		
RTC		0.65	0.49	0.74		

RTC is the discontinuity condition of a single discontinuity (set) in the reference rock mass corrected for discontinuity weathering.
RTC = TC / sqrt(1.452 -1.220 * e^(-WE))

Weighted by spacing:

$$CD = \frac{\frac{TC1}{DS1} + \frac{TC2}{DS2} + \frac{TC3}{DS3}}{\frac{1}{DS1} + \frac{1}{DS2} + \frac{1}{DS3}} = \frac{\frac{0.64}{0.40} + \frac{0.48}{0.60} + \frac{0.74}{0.40}}{\frac{1}{0.40} + \frac{1}{0.60} + \frac{1}{0.40}} = 0.64$$

corrected for weathering: RCD (with a maximum of 1.0165) = CD / WE = 0.64 / 0.95 = **0.67**

REFERENCE UNIT FRICTION AND COHESION (RFRI & RCOH)

Rock mass friction: RFRI = RIRS * 0.2417 + RSPA * 52.12 + RCD * 5.779

RFRI = 79 * 0.2417 + 0.35 * 52.12 + 0.67 * 5.779 = **41 °**

Rock mass cohesion: RCOH = RIRS * 94.27 + RSPA * 28629 + RCD * 3593

RCOH = 79 * 94.27 + 0.35 * 28629 + 0.67 * 3593 = **19875 Pa**

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 soil, 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. A 109. Example I. Natural exposure B. Reference rock mass calculation.

ITC/TU ENGINEERING GEOLOGY	slope stability probability	SSPC - SYSTEM
LOGGED BY: zz	DATE: 11/04/92	slope no: natural exposure B

	LOCATION	map no:	445-iii
	Map coordinates:	northing:	739.698
		easting:	983.390

DETAILS OF SLOPE

METHOD OF EXCAVATION (SME)		WEATHERING (SWE)			
(tick)		(tick)			
natural/hand-made	✓ 1.00	unweathered	1.00	Slope dip direction (degrees):	358
pneumatic hammer excavation	0.76	slightly	✓ 0.95	**Slope dip** (degrees):	45
pre-splitting/smooth wall blasting	0.99	moderately	0.90	Height (**Hslope**) (m):	6.0
conventional blasting with result:		highly	0.62		
good	0.77	completely	0.35		
open discontinuities	0.75				
dislodged blocks	0.72	note: SWE = 1.00 for 'soil type' units, e.g. cemented soil, etc.			
fractured intact rock	0.67				
crushed intact rock	0.62				

SLOPE UNIT NAME: tg21, limestone and dolomite, medium bedded

ORIENTATION INDEPENDENT STABILITY

INTACT ROCK STRENGTH (SIRS)	
SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = 79 * 0.95 =	75
DISCONTINUITY SPACING (SSPA)	
SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope)	
SSPA = 0.35 * 0.95 * 0.99 =	0.33
CONDITION OF DISCONTINUITIES (SCD)	
SCD = RCD (from reference rock mass) * SWE (weathering slope)	
SCD = 0.67 * 0.95 =	0.64
SLOPE UNIT FRICTION AND COHESION (SFRI & SCOH)	
Rock mass friction: SFRI = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779	
SFRI = 75 * 0.2417 + 0.33 * 52.12 + 0.64 * 5.779 =	39 °
Rock mass cohesion: SCOH = SIRS * 94.27 + SSPA * 28629 + SCD * 3593	
SCOH = 75 * 94.27 + 0.33 * 28629 + 0.64 * 3593 =	18817 Pa
If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (Hmax)	
Maximum possible height: Hmax = 1.6 * 10^{-4} * SCOH * sin(slope dip) * cos(SFRI) / (1-cos(slope dip - SFRI))	
Hmax = 1.6 * 10^{-4} * 18817 * sin(45 °) * cos(39 °) / (1-cos(45 ° - 39 °)) =	302 m
ratios: SFRI / dip $_{slope}$ = 39 ° / 45 ° =	0.87
Hmax / Hslope = 302 / 6.0 =	50
Probability stable: if SFRI > dip $_{slope}$ probability = 100 % else use figure for orientation independent stability:	> 95 %

ORIENTATION DEPENDENT STABILITY

DISCONTINUITIES		B 1	J 2	J 3	4	5
Dip direction	(degrees)	310	287	002		
Dip	(degrees)	06	86	87		
With, Against, Vertical or Equal		w	w	v		
AP	(degrees)	04	75	87		
RTC (from reference form)		0.65	0.48	0.75		
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))		0.64	0.48	0.74		
Probability stable:		> 95 %	100 %	100 %		

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: δ = σ - τ: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling		stability:	sliding	toppling
AP > 84° or AP < -84°	verti-cal	100 %	100 %	AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
(slope dip+5°) < AP < 84°	with	100 %	100 %	AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling
(slope dip-5°) < AP < (slope dip+5°)	equal	100 %	100 %				
0° < AP < (slope dip-5°)	with	use graph sliding	100 %				

Fig. A 110. Example I. Natural exposure B. Slope stability probability calculation.

ITC/TU ENGINEERING GEOLOGY	slope stability probability	SSPC - SYSTEM
LOGGED BY: zz	DATE: 11/04/92	slope no: design new road cut C, dip 85°

LOCATION	map no:	445-iii
Map coordinates:	northing:	739.568
	easting:	983.485

DETAILS OF SLOPE

METHOD OF EXCAVATION (SME)		WEATHERING (SWE)			
(tick)		(tick)			
natural/hand-made	1.00	unweathered	1.00	Slope dip direction (degrees):	330
pneumatic hammer excavation	0.76	slightly	✓ 0.95	Slope dip (degrees):	85
pre-splitting/smooth wall blasting	0.99	moderately	0.90	Height (Hslope) (m):	8.0
conventional blasting with result:		highly	0.62		
good	0.77	completely	0.35		
open discontinuities	0.75	note: SWE = 1.00 for 'soil type' units, e.g. cemented soil, etc.			
dislodged blocks	0.72				
fractured intact rock	✓ 0.67				
crushed intact rock	0.62				

SLOPE UNIT NAME: tg21, limestone and dolomite, medium bedded

ORIENTATION INDEPENDENT STABILITY

INTACT ROCK STRENGTH (SIRS)

SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = 79 * 0.95 = **75**

DISCONTINUITY SPACING (SSPA)

SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope)

SSPA = 0.35 * 0.95 * 0.67 = **0.22**

CONDITION OF DISCONTINUITIES (SCD)

SCD = RCD (from reference rock mass) * SWE (weathering slope)

SCD = 0.65 * 0.95 = **0.62**

SLOPE UNIT FRICTION AND COHESION (SFRI & SCOH)

Rock mass friction: SFRI = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779

SFRI = 75 * 0.2417 + 0.22 * 52.12 + 0.62 * 5.779 = **33 °**

Rock mass cohesion: SCOH = SIRS * 94.27 + SSPA * 28629 + SCD * 3593

SCOH = 75 * 94.27 + 0.22 * 28629 + 0.62 * 3593 = **15596 Pa**

If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (Hmax)

Maximum possible height: Hmax = 1.6 * 10^{-4} * SCOH * sin(slope dip) * cos(SFRI) / (1-cos(slope dip - SFRI))

Hmax = 1.6 * 10^{-4} * 15596 * sin(85 °) * cos(33 °) / (1-cos(85 ° - 33 °)) = **5.4 m**

ratios:
SFRI / slope dip = 33 ° / 85 ° = **0.39**
Hmax / Hslope = 5.4 m / 8.0 m = **0.68**

Probability stable: if SFRI > slope dip probability = 100 % else use figure for orientation independent stability: **8 %**

ORIENTATION DEPENDENT STABILITY

DISCONTINUITIES		B1	J2	J3	J4	5
Dip direction	(degrees)	310	044	287	002	
Dip	(degrees)	06	86	86	87	
With, Against, Vertical or Equal		w	w	e	e	
AP	(degrees)	06	76	85	85	
RTC (from reference form)		0.61	0.60	0.48	0.60	
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))		0.60	0.59	0.48	0.59	
Probability stable:		> 95 %	< 5 %	100 %	100 %	

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: δ = σ - τ: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling		stability:	sliding	toppling
AP > 84° or AP < -84°	vertical	100 %	100 %	AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
(slope dip+5°) < AP < 84°	with	100 %	100 %	AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling
(slope dip-5°) < AP < (slope dip+5°)	equal	100 %	100 %				
0° < AP < (slope dip-5°)	with	use graph sliding	100 %				

Fig. A 111. Example I. New road cut C, design slope dip 85°. Slope stability probability calculation.

ITC/TU ENGINEERING GEOLOGY	slope stability probability	SSPC - SYSTEM
LOGGED BY: zz	DATE: 11/04/92	slope no: design new road cut C, dip 75°

	LOCATION	map no:	445-iii
	Map coordinates:	northing:	739.568
		easting:	983.485

DETAILS OF SLOPE

METHOD OF EXCAVATION (SME)		WEATHERING (SWE)			
(tick)		(tick)			
natural/hand-made	1.00	unweathered	1.00	Slope dip direction (degrees):	330
pneumatic hammer excavation	0.76	slightly	✓ 0.95		
pre-splitting/smooth wall blasting	0.99	moderately	0.90	Slope dip (degrees):	75
conventional blasting with result:		highly	0.62		
good	0.77	completely	0.35	Height (Hslope) (m):	8.0
open discontinuities	0.75				
dislodged blocks	0.72	note: SWE = 1.00 for 'soil type' units, e.g. cemented soil, etc.			
fractured intact rock	✓ 0.67				
crushed intact rock	0.62				

SLOPE UNIT NAME: tg21, limestone and dolomite, medium bedded

ORIENTATION INDEPENDENT STABILITY

INTACT ROCK STRENGTH (SIRS)

SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = 79 * 0.95 = **75**

DISCONTINUITY SPACING (SSPA)

SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope)

SSPA = 0.35 * 0.95 * 0.67 = **0.22**

CONDITION OF DISCONTINUITIES (SCD)

SCD = RCD (from reference rock mass) * SWE (weathering slope)

SCD = 0.65 * 0.95 = **0.62**

SLOPE UNIT FRICTION AND COHESION (SFRI & SCOH)

Rock mass friction: SFRI = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779

SFRI = 75 * 0.2417 + 0.22 * 52.12 + 0.62 * 5.779 = **33 °**

Rock mass cohesion: SCOH = SIRS * 94.27 + SSPA * 28629 + SCD * 3593

SCOH = 75 * 94.27 + 0.22 * 28629 + 0.62 * 3593 = **15596 Pa**

If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (Hmax)

Maximum possible height: Hmax = 1.6 * 10^{-4} * SCOH * sin(slope dip) * cos(SFRI) / (1-cos(slope dip - SFRI))

Hmax = 1.6 * 10^{-4} * 15596 * sin(75 °) * cos(33 °) / (1-cos(75 ° - 33 °)) = **8.6 m**

ratios:	SFRI / slope dip = 33 ° / 75 ° =	0.44
	Hmax / Hslope = 8.6 m / 8.0 m =	1.075

Probability stable: If SFRI > slope dip probability = 100 % else use figure for orientation independent stability: **55 %**

ORIENTATION DEPENDENT STABILITY

DISCONTINUITIES		B 1	J 2	J 3	J 4	5
Dip direction	(degrees)	310	044	287	002	
Dip	(degrees)	06	86	86	87	
With, Against, Vertical or Equal		w	e	v	v	
AP	(degrees)	06	76	85	85	
RTC (from reference form)		0.61	0.60	0.48	0.60	
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))		0.60	0.59	0.48	0.59	
Probability stable:		> 95 %	100 %	100 %	100 %	

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: δ = σ - τ: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling		stability:	sliding	toppling
AP > 84° or AP < -84°	vertical	100 %	100 %	AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
(slope dip+5°) < AP < 84°	with	100 %	100 %	AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling
(slope dip-5°) < AP < (slope dip+5°)	equal	100 %	100 %				
0° < AP < (slope dip-5°)	with	use graph sliding	100 %				

Fig. A 112. Example I. New road cut C, design slope dip 70°. Slope stability probability calculation.

ITC/TU ENGINEERING GEOLOGY	exposure characterization	SSPC - SYSTEM
LOGGED BY: *zz*	DATE: *27/03/90*	TIME: *10:00* hr | exposure no: *example ii*

WEATHER CONDITIONS		LOCATION		
Sun:	cloudy/fair/bright	Map coordinates:	map no:	*472-i*
Rain:	dry/drizzle/slight/heavy		northing:	*4.555.100*
			easting:	*318.950*

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

DIMENSIONS/ACCESSIBILITY			
Size total exposure: (m)	l: *25*	h: *15*	d: *10*
mapped on this form: (m)	l: *15*	h: *15*	d: *10*
Accessibility:	poor/fair/good		

FORMATION NAME: *tg23 limestone and dolomite*

DESCRIPTION (BS 5930: 1981)				
colour	grain size	structure & texture	weathering	NAME
off-white/yellowish	*fine*	*medium bedded, large tabular*	*slightly*	*calcisiltite*

INTACT ROCK STRENGTH (IRS) (tick)	
< 1.25 MPa	Crumbles in hand
1.25 - 5 MPa	Thin slabs break easily in hand
5 - 12.5 MPa	Thin slabs broken by heavy hand pressure
12.5 - 50 MPa ✓	Lumps broken by light hammer blows
50 - 100 MPa	Lumps broken by heavy hammer blows
100 - 200 MPa	Lumps only chip by heavy hammer blows (Dull ringing sound)
> 200 MPa	Rocks ring on hammer blows. Sparks fly

sample number(s):

WEATHERING (WE)	
(tick)	
unweathered	1.00
slightly	✓ 0.95
moderately	0.90
highly	0.62
completely	0.35

DISCONTINUITIES B=bedding C=Cleavage J=joint			*B* 1	*J* 2	*J* 3	4	5
Dip direction		(degrees)	*162*	*265*	*337*		
Dip		(degrees)	*37*	*85*	*48*		
Spacing (DS)		(m)	*0.50*	*5.00*	*5.00*		
persistence	along strike	(m)	*>25*	*>10*	*5*		
	along dip	(m)	*>15*	*>15*	*5*		
CONDITION OF DISCONTINUITIES							
Roughness large scale (Rl)	wavy :1.00 slightly wavy :0.95 curved :0.85 slightly curved :0.80 straight :0.75		*0.75*	*0.85*	*0.95*		
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		*0.95*	*0.95*	*0.90*		
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		*0.55*	*0.55*	*1.00*		
Karst (Ka)	none :1.00 karst :0.92		*1.00*	*0.92*	*1.00*		

EXISTING SLOPE?

dip-direction/dip *162 / 90*

height: *5.0* m

Stability (tick)	
stable	1
small problems in near future	2
large problems in near future	3
small problems	4
large problems	✓ 5

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.) roughness should be assessed perpendicular and parallel to the roughness and directions noted on this form.
3) Non-fitting of discontinuities should be marked in roughness columns.

SUSCEPTIBILITY TO WEATHERING (SW)		
degree of weathering:	date excavation:	remarks:
slightly	*> 40 years ago*	

remarks: *J3 likely tension joint. Clay infill in J2 is likely flushed in topsoil from above.*

All exposures in surroundings slightly weathered, except for 0.5 m of rock near surface which is moderately weathered.

Fig. A 113. Example II. Exposure characterization.

ITC/TU ENGINEERING GEOLOGY	reference rock mass calculation	SSPC - SYSTEM
CALCULATED BY: zz	DATE: 27/03/90	exposure no: example ii

REFERENCE UNIT NAME: tg23, limestone and dolomite, medium bedded

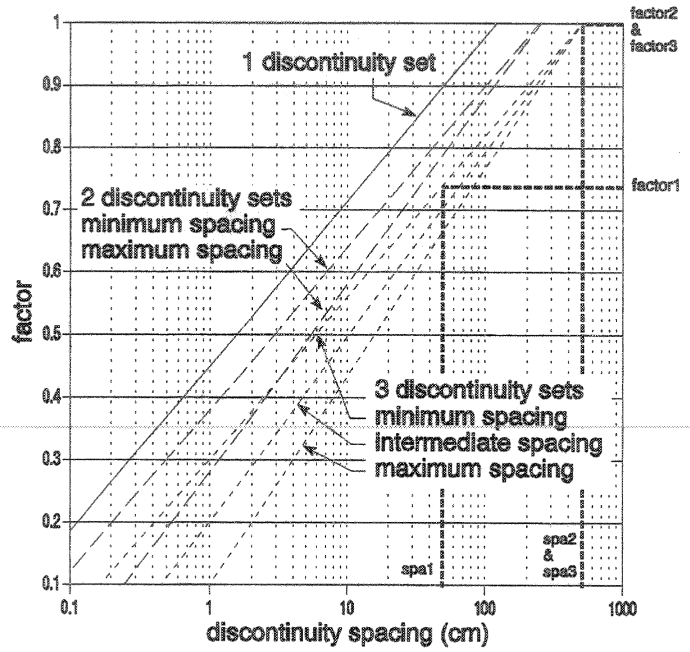
INTACT ROCK STRENGTH (RIRS)

If IRS > 132 MPa then RIRS = 132 else RIRS = IRS (in MPa) / WE (correction for weathering) = 31 / 0.95 = **33**

DISCONTINUITY SPACING (RSPA)

DISCONTINUITIES		B1	J2	J3	4	5
Dip direction	(degrees)	162	265	337		
Dip	(degrees)	37	85	48		
Spacing (DS)	(m)	0.50	5.00	5.00		

The spacing parameter (SPA) is calculated based on the three discontinuity sets with the smallest spacings following figure:



SPA (see figure below) = factor1 * factor2 * factor3 =
0.74 * 1.00 * 1.00 = 0.74
corrected for weathering and method of excavation:
RSPA = SPA / (WE * ME)
(with a maximum of 1.00)
RSPA = 0.74 / (0.95 * 0.99) = **0.79**

CONDITION OF DISCONTINUITIES (RTC & RCD)

DISCONTINUITIES		B1	J2	J3	4	5
Roughness large scale	(Rl)	0.75	0.85	0.95		
Roughness small scale	(Rs)	0.95	0.95	0.90		
Infill material	(Im)	0.55	1.00	1.00		
Karst	(Ka)	1.00	0.92	1.00		
Total	(Rl*Rs*Im*Ka = TC)	0.39	0.74	0.86		
RTC		0.40	0.75	0.86		

RTC is the discontinuity condition of a single discontinuity (set) in the reference rock mass corrected for discontinuity weathering.
RTC = TC / sqrt(1.452 - 1.220 * e^(-WE))

Weighted by spacing:

$$CD = \frac{\frac{TC1}{DS1} + \frac{TC2}{DS2} + \frac{TC3}{DS3}}{\frac{1}{DS1} + \frac{1}{DS2} + \frac{1}{DS3}} = \frac{\frac{0.39}{0.50} + \frac{0.74}{5.00} + \frac{0.86}{5.00}}{\frac{1}{0.50} + \frac{1}{5.00} + \frac{1}{5.00}} = 0.46$$

corrected for weathering: RCD (with a maximum of 1.0165) = CD / WE = 0.46 / 0.95 = **0.48**

REFERENCE UNIT FRICTION AND COHESION (RFRI & RCOH)

Rock mass friction: RFRI = RIRS * 0.2417 + RSPA * 52.12 + RCD * 5.779

RFRI = 33 * 0.2417 + 0.79 * 52.12 + 0.48 * 5.779 = **52 °**

Rock mass cohesion: RCOH = RIRS * 94.27 + RSPA * 28629 + RCD * 3593

RCOH = 33 * 94.27 + 0.79 * 28629 + 0.48 * 3593 = **27280 Pa**

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. A 114. Example II. Reference rock mass calculation.

ITC/TU ENGINEERING GEOLOGY	slope stability probability	SSPC - SYSTEM
LOGGED BY: *zz*	DATE: *11/04/92*	slope no: *example ii*

	LOCATION	map no:	*472-i*
	Map coordinates:	northing:	*4.555.100*
		easting:	*318.950*

DETAILS OF SLOPE

METHOD OF EXCAVATION (SME)		WEATHERING (SWE)			
(tick)		(tick)			
natural/hand-made	1.00	unweathered	1.00	Slope dip direction (degrees):	*162*
pneumatic hammer excavation	0.76	slightly	✓ 0.95		
pre-splitting/smooth wall blasting	✓ 0.99	moderately	0.90	**Slope dip** (degrees):	*90*
conventional blasting with result:		highly	0.62		
good	0.77	completely	0.35	Height (**Hslope**) (m):	*5.0*
open discontinuities	0.75				
dislodged blocks	0.72	note: SWE = 1.00 for 'soil type' units, e.g. cemented soil, etc.			
fractured intact rock	0.67				
crushed intact rock	0.62				

SLOPE UNIT NAME: *tg23, limestone and dolomite, medium bedded*

ORIENTATION INDEPENDENT STABILITY

INTACT ROCK STRENGTH (**SIRS**)

SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = *33* * *0.95* = *31*

DISCONTINUITY SPACING (**SSPA**)

SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope)

SSPA = *0.79* * *0.95* * *0.99* = *0.74*

CONDITION OF DISCONTINUITIES (**SCD**)

SCD = RCD (from reference rock mass) * SWE (weathering slope)

SCD = *0.48* * *0.95* = *0.46*

SLOPE UNIT FRICTION AND COHESION (**SFRI & SCOH**)

Rock mass friction: SFRI = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779

SFRI = *31* * 0.2417 + *0.74* * 52.12 + *0.46* * 5.779 = *49* °

Rock mass cohesion: SCOH = SIRS * 94.27 + SSPA * 28629 + SCD * 3593

SCOH = *31* * 94.27 + *0.74* * 28629 + *0.46* * 3593 = *25702* Pa

If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (**Hmax**)

Maximum possible height: Hmax = 1.6 * 10^{-4} * SCOH * sin(slope dip) * cos(SFRI) / (1-cos(slope dip - SFRI))

Hmax = 1.6 * 10^{-4} * *25702* * sin(*90* °) * cos(*49* °) / (1-cos(*90* ° - *49* °)) = *13.3* m

Probability stable: if SFRI > slope dip probability = 100 % else use figure for orientation independent stability: **> *95* %**

ORIENTATION DEPENDENT STABILITY

DISCONTINUITIES		*B*1	*J*2	*J*3	4	5
Dip direction	(degrees)	*162*	*265*	*337*		
Dip	(degrees)	*37*	*85*	*48*		
With, Against, Vertical or Equal		*w*	*a*	*a*		
AP	(degrees)	*37*	*-69*	*-48*		
RTC (from reference form)		*0.40*	*0.75*	*0.86*		
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))		*0.39*	*0.74*	*0.86*		
Probability stable:		*55* %	> *95* %	> *95* %		

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: δ = σ - τ: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling
AP > 84° or AP < -84°	vertical	100 %	100 %
(slope dip + 5°) < AP < 84°	with	100 %	100 %
(slope dip-5°) < AP < (slope dip + 5°)	equal	100 %	100 %
0° < AP < (slope dip-5°)	with	use graph sliding	100 %

	stability:	sliding	toppling
AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling

Fig. A 115. Example II. Slope stability probability calculation.

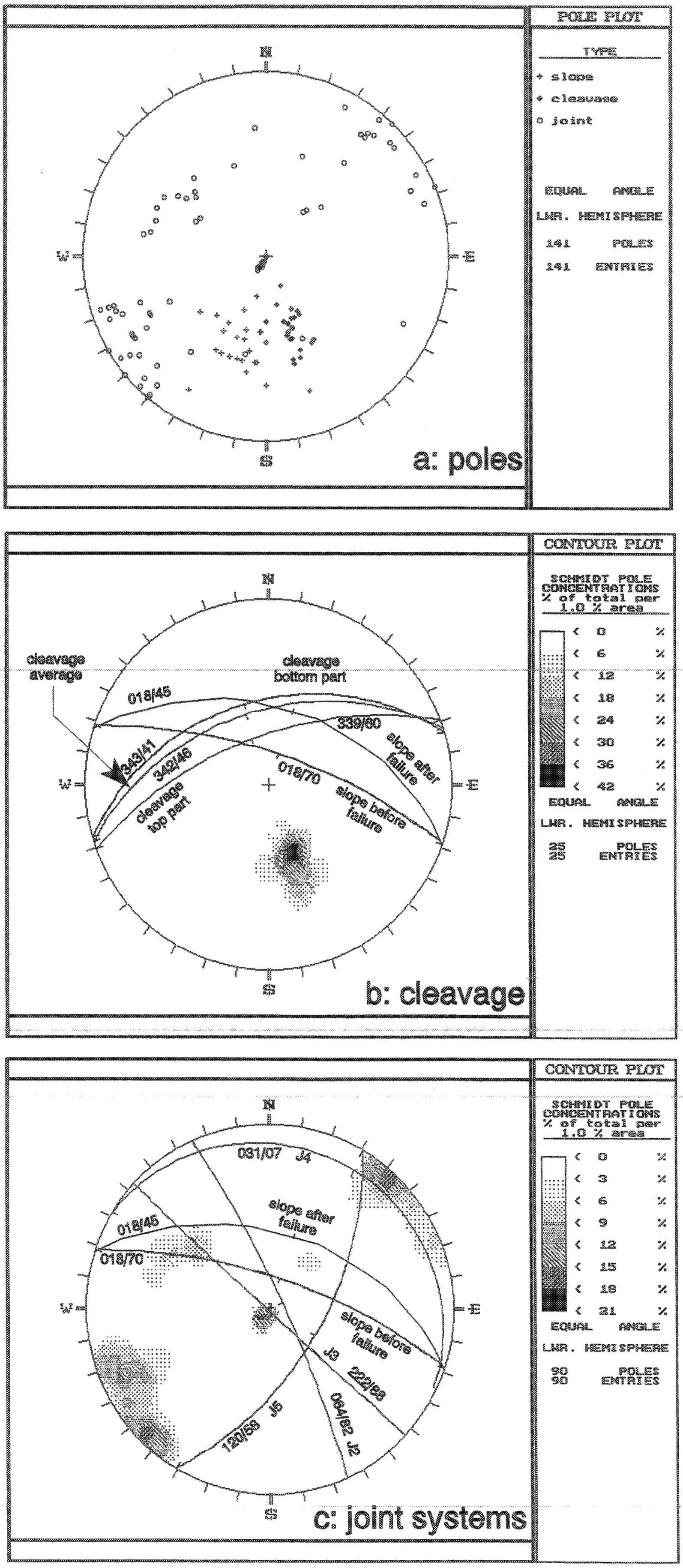


Fig. A 116. Example III. Stereo projection. a: poles; b and c: contours of poles and great circles of planes. Indicated orientations are dip-vectors.

ITC/TU ENGINEERING GEOLOGY	exposure characterization		SSPC - SYSTEM
LOGGED BY: *zz*	DATE: *10/04/91*	TIME: *12:00* hr	exposure no: *example iii*

WEATHER CONDITIONS		LOCATION	map no:	*444-ii*
Sun:	cloudy/fair/bright	Map coordinates:	northing:	*740,840*
Rain:	dry/drizzle/slight/heavy		easting:	*974,790*

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

DIMENSIONS/ACCESSIBILITY			
Size total exposure: (m)	l: *200*	h: *15*	d: *20*
mapped on this form: (m)	l: *20*	h: *15*	d: *20*
Accessibility:	poor/fair/good		

FORMATION NAME: *H (Carboniferous) slate*

DESCRIPTION (BS 5930: 1981)				
colour	grain size	structure & texture	weathering	NAME
light to dark grey	*fine*	*v.thin bed./cleavage,small tabular*	*slightly*	*slate*

INTACT ROCK STRENGTH (IRS) (tick)		sample number(s):
< 1.25 MPa	Crumbles in hand	
1.25 - 5 MPa	Thin slabs break easily in hand	
5 - 12.5 MPa	Thin slabs broken by heavy hand pressure	
12.5 - 50 MPa ✓	Lumps broken by light hammer blows	
50 - 100 MPa	Lumps broken by heavy hammer blows	
100 - 200 MPa	Lumps only chip by heavy hammer blows (Dull ringing sound)	
> 200 MPa	Rocks ring on hammer blows. Sparks fly	

WEATHERING (WE)	
(tick)	
unweathered	1.00
slightly	✓ 0.95
moderately	0.90
highly	0.62
completely	0.35

DISCONTINUITIES B=bedding C=Cleavage J=joint			*C*1	*J*2	*J*3	*J*4	*J*5
Dip direction		(degrees)	*342*	*064*	*222*	*031*	*120*
Dip		(degrees)	*60*	*82*	*88*	*07*	*58*
Spacing (DS)		(m)	*0.02*	*0.20*	*0.20*	*0.50*	*1.00*
persistence	along strike	(m)	*> 200*	*> 20*	*> 20*	*> 200*	*> 20*
	along dip	(m)	*> 20*	*> 15*	*> 15*	*> 20*	*> 15*
CONDITION OF DISCONTINUITIES							
Roughness large scale (Rl)	wavy slightly wavy curved slightly curved straight	:1.00 :0.95 :0.85 :0.80 :0.75	*0.85*	*1.00*	*0.75*	*1.00*	*1.00*
Roughness small scale (Rs) (on an area of 20 x 20 cm²)	rough stepped/irregular smooth stepped polished stepped rough undulating smooth undulating polished undulating rough planar smooth planar polished planar	:0.95 :0.90 :0.85 :0.80 :0.75 :0.70 :0.65 :0.60 :0.55	*0.75*	*0.90*	*0.65*	*0.55*	*0.95*
Infill material (Im)	cemented/cemented infill no infill - surface staining non softening & sheared material, e.g. free of clay, talc, etc.: coarse / medium / fine soft sheared material, e.g. clay, talc, etc.: coarse / medium / fine gouge < irregularities gouge > irregularities flowing material	:1.07 :1.00 :0.95 / :0.90 / :0.85 :0.75 / :0.65 / :0.55 :0.42 :0.17 :0.05	*1.00*	*1.00*	*1.00*	*0.17*	*1.00*
Karst (Ka)	none karst	:1.00 :0.92	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*

EXISTING SLOPE?	
dip-direction/dip	*018 / 70*
height:	*8.2* m
Stability (tick)	
stable	1
small problems in near future	2
large problems in near future	3
small problems	4
large problems	✓ 5

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.) roughness should be assessed perpendicular and parallel to the roughness and directions noted on this form.
3) Non-fitting of discontinuities should be marked in roughness columns.

SUSCEPTIBILITY TO WEATHERING (SW)			remarks:
degree of weathering:	date excavation:	remarks:	
moderately	*> 40 years ago*		
slightly	*4 years ago*		

Fig. A 117. Example III. Exposure characterization.

ITC/TU ENGINEERING GEOLOGY	reference rock mass calculation	SSPC - SYSTEM
CALCULATED BY: *??*	DATE: *10/04/91*	exposure no: *example iii*

REFERENCE UNIT NAME: *H (Carboniferous) slate, within cleavage*

INTACT ROCK STRENGTH (RIRS)

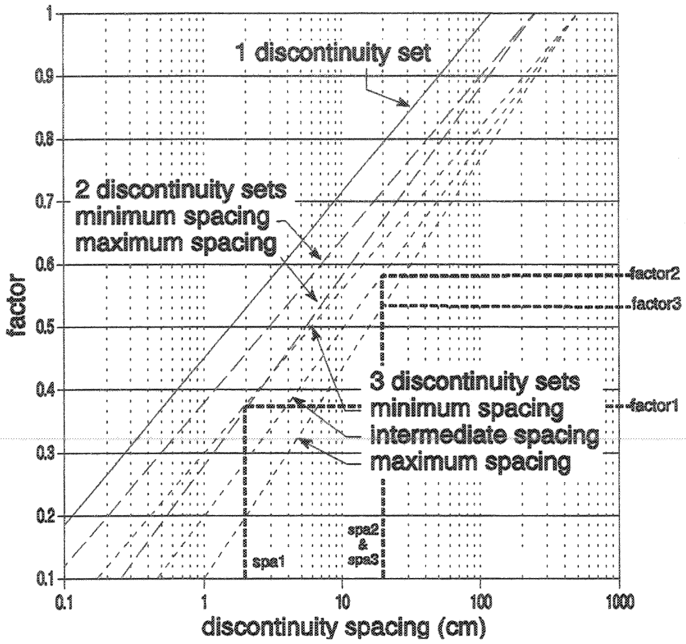
If IRS > 132 MPa then RIRS = 132 else RIRS = IRS (in MPa) / WE (correction for weathering) = *32* / *0.95* = *34*

DISCONTINUITY SPACING (RSPA)

DISCONTINUITIES		*C*1	*J*2	*J*3	*J*4	*J*5
Dip direction	(degrees)	*342*	*064*	*222*	*031*	*120*
Dip	(degrees)	*60*	*82*	*88*	*07*	*58*
Spacing (DS)	(m)	*0.02*	*0.20*	*0.20*	*0.50*	*1.00*

SPA (see figure below) = factor1 * factor2 * factor3 = *0.38* * *0.58* * *0.53* = *0.12* (1)
corrected for weathering and method of excavation:
RSPA = SPA / (WE * ME)
(with a maximum of 1.00)
RSPA = *0.12* / (*0.95* * *0.76*) = *0.16*

The spacing parameter (SPA) is calculated based on the three discontinuity sets with the smallest spacings following figure:



CONDITION OF DISCONTINUITIES (RTC & RCD)

DISCONTINUITIES		*C*1	*J*2	*J*3	*J*4	*J*5
Roughness large scale	(Rl)	*0.85*	*1.00*	*0.75*	*1.00*	*1.00*
Roughness small scale	(Rs)	*0.75*	*0.90*	*0.65*	*0.55*	*0.95*
Infill material	(Im)	*1.00*	*1.00*	*1.00*	*0.17*	*1.00*
Karst	(Ka)	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
Total	(Rl*Rs*Im*Ka = TC)	*0.64*	*0.90*	*0.49*	*0.09*	*0.95*
RTC		*0.64*	*0.91*	*0.49*	*0.09*	*0.96*

RTC is the discontinuity condition of a single discontinuity (set) in the reference rock mass corrected for discontinuity weathering.
RTC = TC / sqrt(1.452 - 1.220 * e^(-WE))

Weighted by spacing:

$$CD = \frac{\frac{TC1}{DS1} + \frac{TC2}{DS2} + \frac{TC3}{DS3}}{\frac{1}{DS1} + \frac{1}{DS2} + \frac{1}{DS3}} = \frac{\frac{0.64}{0.02} + \frac{0.91}{0.20} + \frac{0.49}{0.20}}{\frac{1}{0.02} + \frac{1}{0.20} + \frac{1}{0.20}} = 0.65^{(1)}$$

corrected for weathering: RCD (with a maximum of 1.0165) = CD / WE = *0.65* / *0.95* = *0.68*

REFERENCE UNIT FRICTION AND COHESION (RFRI & RCOH)

Rock mass friction: RFRI = RIRS * 0.2417 + RSPA * 52.12 + RCD * 5.779

RFRI = *34* * 0.2417 + *0.16* * 52.12 + *0.68* * 5.779 = *21* °

Rock mass cohesion: RCOH = RIRS * 94.27 + RSPA * 28629 + RCD * 3593

RCOH = *34* * 94.27 + *0.16* * 28629 + *0.68* * 3593 = *10306* Pa

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.

note (1): SPA and CD based on discontinuities 1, 2 and 3.

Fig. A 118. Example III. Reference rock mass calculation.

ITC/TU ENGINEERING GEOLOGY	slope stability probability	SSPC - SYSTEM
LOGGED BY: *zz*	DATE: *11/04/92*	slope no: *example iii before failure*

LOCATION	map no:	*444-ii*
Map coordinates:	northing:	*740,840*
	easting:	*974,790*

DETAILS OF SLOPE

METHOD OF EXCAVATION (SME)		WEATHERING (SWE)			
(tick)		(tick)			
natural/hand-made	1.00	unweathered	1.00	Slope dip direction (degrees):	*018*
pneumatic hammer excavation	✓ 0.76	slightly	✓ 0.95		
pre-splitting/smooth wall blasting	0.99	moderately	0.90	**Slope dip** (degrees):	*70*
conventional blasting with result:		highly	0.62		
good	0.77	completely	0.35	Height (**Hslope**) (m):	*8.2*
open discontinuities	0.75				
dislodged blocks	0.72	note: SWE = 1.00 for 'soil type' units, e.g. cemented soil, etc.			
fractured intact rock	0.67				
crushed intact rock	0.62				

SLOPE UNIT NAME: *H (Carboniferous) slate, v. thin cleavage*

ORIENTATION INDEPENDENT STABILITY

INTACT ROCK STRENGTH (SIRS)

SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = *34* * *0.95* = *32*

DISCONTINUITY SPACING (SSPA)

SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope)

SSPA = *0.16* * *0.95* * *0.76* = *0.12*

CONDITION OF DISCONTINUITIES (SCD)

SCD = RCD (from reference rock mass) * SWE (weathering slope)

SCD = *0.68* * *0.95* = *0.65*

SLOPE UNIT FRICTION AND COHESION (SFRI & SCOH)

Rock mass friction: SFRI = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779

SFRI = *32* * 0.2417 + *0.12* * 52.12 + *0.65* * 5.779 = *18* °

Rock mass cohesion: SCOH = SIRS * 94.27 + SSPA * 28629 + SCD * 3593

SCOH = *32* * 94.27 + *0.12* * 28629 + *0.65* * 3593 = *8725* Pa

If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (Hmax)

Maximum possible height: Hmax = 1.6 * 10^{-4} * SCOH * sin(slope dip) * cos(SFRI) / (1-cos(slope dip - SFRI))

Hmax = 1.6 * 10^{-4} * *8725* * sin(*70* °) * cos(*18* °) / (1-cos(*70* ° - *18* °)) = *3.2* m

ratios: SFRI / slope dip = *18* ° / *70* ° = *0.26*

Hmax / Hslope = *3.2* m / *8.2* m = *0.39*

Probability stable: if SFRI > slope dip probability = 100 % else use figure for orientation independent stability: *< 5* %

ORIENTATION DEPENDENT STABILITY

DISCONTINUITIES		*C*1	*J*2	*J*3	*J*4	*J*5
Dip direction	(degrees)	*342*	*064*	*222*	*031*	*120*
Dip	(degrees)	*60*	*82*	*88*	*07*	*58*
With, Against, Vertical or Equal		*w*	*w*	*v*	*w*	*a*
AP	(degrees)	*54*	*79*	*88*	*07*	*-18*
RTC (from reference form)		*0.64*	*0.91*	*0.49*	*0.09*	*0.95*
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))		*0.63*	*0.90*	*0.49*	*0.09*	*0.96*
Probability stable:		*92* %	*100* %	*100* %	*75* %	*100* %

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: δ = σ - τ: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling		stability:	sliding	toppling
AP > 84° or AP < -84°	vertical	100 %	100 %	AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
(slope dip + 5°) < AP < 84°	with	100 %	100 %	AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling
(slope dip-5°) < AP < (slope dip + 5°)	equal	100 %	100 %				
0° < AP < (slope dip-5°)	with	use graph sliding	100 %				

Fig. A 119. Example III. Slope stability probability calculation before failure.

ITC/TU ENGINEERING GEOLOGY	slope stability probability	SSPC - SYSTEM
LOGGED BY: *zz*	DATE: *11/04/92*	slope no: *example iii after failure*

	LOCATION	map no:	*444-ii*
	Map coordinates:	northing:	*740.840*
		easting:	*974.790*

DETAILS OF SLOPE

METHOD OF EXCAVATION (SME)		WEATHERING (SWE)		
(tick)		(tick)		
natural/hand-made	1.00	unweathered	1.00	Slope dip direction (degrees): *018*
pneumatic hammer excavation	✓ 0.76	slightly	✓ 0.95	**Slope dip** (degrees): *45*
pre-splitting/smooth wall blasting	0.99	moderately	0.90	Height (**Hslope**) (m): *8.2*
conventional blasting with result:		highly	0.62	
good	0.77	completely	0.35	
open discontinuities	0.75			
dislodged blocks	0.72	note: SWE = 1.00 for 'soil type' units,		
fractured intact rock	0.67	e.g. cemented soil, etc.		
crushed intact rock	0.62			

SLOPE UNIT NAME: *H (Carboniferous) slate, v.thin cleavage*

ORIENTATION INDEPENDENT STABILITY

INTACT ROCK STRENGTH (SIRS)

SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = *34* * *0.95* = *32*

DISCONTINUITY SPACING (SSPA)

SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope)

SSPA = *0.16* * *0.95* * *0.76* = *0.12*

CONDITION OF DISCONTINUITIES (SCD)

SCD = RCD (from reference rock mass) * SWE (weathering slope)

SCD = *0.68* * *0.95* = *0.65*

SLOPE UNIT FRICTION AND COHESION (SFRI & SCOH)

Rock mass friction: SFRI = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779

SFRI = *32* * 0.2417 + *0.12* * 52.12 + *0.65* * 5.779 = *18*°

Rock mass cohesion: SCOH = SIRS * 94.27 + SSPA * 28629 + SCD * 3593

SCOH = *32* * 94.27 + *0.12* * 28629 + *0.65* * 3593 = *8725* Pa

If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (Hmax)

Maximum possible height: Hmax = 1.6 * 10^{-4} * SCOH * sin(slope dip) * cos(SFRI) / (1-cos(slope dip - SFRI))

Hmax = 1.6 * 10^{-4} * *8725* * sin(*45*°) * cos(*18*°) / (1-cos(*45*° - *18*°)) = *8.4* m

ratios:	
SFRI / slope dip = *18*° / *45*° =	*0.40*
Hmax / Hslope = *8.4* m / *8.2* m =	*1.02*

Probability stable: if SFRI > slope dip probability = 100 % else use figure for orientation independent stability: *55* %

ORIENTATION DEPENDENT STABILITY

DISCONTINUITIES		*C*1	*J*2	*J*3	*J*4	*J*5
Dip direction	(degrees)	*342*	*064*	*222*	*031*	*120*
Dip	(degrees)	*60*	*82*	*88*	*07*	*58*
With, Against, Vertical or Equal		*w*	*w*	*e*	*w*	*a*
AP	(degrees)	*54*	*79*	*88*	*07*	*-18*
RTC (from reference form)		*0.64*	*0.91*	*0.49*	*0.09*	*0.95*
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))		*0.63*	*0.90*	*0.49*	*0.09*	*0.96*
Probability stable:		*100* %	*100* %	*100* %	*75* %	*100* %

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: δ = σ - τ: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling		stability:	sliding	toppling
AP > 84° or AP < -84°	vertical	100 %	100 %	AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
(slope dip+5°) < AP < 84°	with	100 %	100 %	AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling
(slope dip-5°) < AP < (slope dip+5°)	equal	100 %	100 %				
0° < AP < (slope dip-5°)	with	use graph sliding	100 %				

Fig. A 120. Example III. Slope stability probability calculation after failure.

ITC/TU ENGINEERING GEOLOGY	exposure characterization		SSPC - SYSTEM
LOGGED BY: *zz*	DATE: *10/04/95*	TIME: *16.00* hr	exposure no: *example iv*

WEATHER CONDITIONS		LOCATION		
Sun:	cloudy/fair/bright	Map coordinates:	map no:	*472-i*
Rain:	dry/drizzle/slight/heavy		northing:	*4.558.750*
			easting:	*321.625*

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

DIMENSIONS/ACCESSIBILITY			
Size total exposure: (m)	l: *200*	h: *5*	d: *50*
mapped on this form: (m)	l: *200*	h: *5*	d: *50*
Accessibility:	poor/fair/good		

FORMATION NAME: *to23 thin bedded units*

DESCRIPTION (BS 5930: 1981)				
colour	grain size	structure & texture	weathering	NAME
brownish off-white	*fine*	*thin bedded, small tabular*	*slightly*	*limestone and dolomite*

INTACT ROCK STRENGTH (IRS) (tick)		sample number(s):
< 1.25 MPa	Crumbles in hand	
1.25 - 5 MPa	Thin slabs break easily in hand	
5 - 12.5 MPa	Thin slabs broken by heavy hand pressure	
12.5 - 50 MPa	Lumps broken by light hammer blows	
50 - 100 MPa	Lumps broken by heavy hammer blows	
100 - 200 MPa ✓	Lumps only chip by heavy hammer blows (Dull ringing sound)	
> 200 MPa	Rocks ring on hammer blows. Sparks fly	

WEATHERING (WE)	
(tick)	
unweathered	1.00
slightly	✓ 0.95
moderately	0.90
highly	0.62
completely	0.35

DISCONTINUITIES B=bedding C=Cleavage J=joint			*B* 1	*J* 2	*J* 3	4	5
Dip direction		(degrees)	*082*	*310*	*244*		
Dip		(degrees)	*30*	*87*	*62*		
Spacing (DS)		(m)	*0.03*	*0.04*	*0.03*		
persistence	along strike	(m)	*> 200*	*> 14*	*0.2*		
	along dip	(m)	*> 50*	*> 20*	*0.2*		
CONDITION OF DISCONTINUITIES							
Roughness large scale (Rl)	wavy slightly wavy curved slightly curved straight	:1.00 :0.95 :0.85 :0.80 :0.75	*1.00*	*0.75*	*0.75*		
Roughness small scale (Rs) (on an area of 20 x 20 cm²)	rough stepped/irregular smooth stepped polished stepped rough undulating smooth undulating polished undulating rough planar smooth planar polished planar	:0.95 :0.90 :0.85 :0.80 :0.75 :0.70 :0.65 :0.60 :0.55	*0.75*	*0.60*	*0.95*		
Infill material (Im)	cemented/cemented infill no infill - surface staining non softening & sheared material, e.g. free of clay, talc, etc.: coarse / medium / fine soft sheared material, e.g. clay, talc, etc.: coarse / medium / fine gouge < irregularities gouge > irregularities flowing material	:1.07 :1.00 :0.95 / :0.90 / :0.85 :0.75 / :0.65 / :0.55 :0.42 :0.17 :0.05	*0.55*	*0.55*	*1.00*		
Karst (Ka)	none karst	:1.00 :0.92	*0.92*	*0.92*	*0.92*		

EXISTING SLOPE?	
dip-direction/dip	*180 / 70*
height:	*5* m
Stability (tick)	
stable	✓ 1
small problems in near future	2
large problems in near future	3
small problems	4
large problems	5

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.) roughness should be assessed perpendicular and parallel to the roughness and directions noted on this form.
3) Non-fitting of discontinuities should be marked in roughness columns.

SUSCEPTIBILITY TO WEATHERING (SW)			remarks:
degree of weathering:	date excavation:	remarks:	
slightly	*> 40 years old*	*old road cuts, hand-made or small shovel ?*	

Fig. A 121. Example IV. Exposure characterization.

APPENDIX VII BLANK SSPC CLASSIFICATION FORMS

Blank forms that can be used for the SSPC system are provided on the following pages. The values for the reference rock mass and the probability of slope stability include expressions for spacing and discontinuity condition. These are calculated based on the discontinuity or combination of discontinuities that result in the lowest possible values for reference rock mass friction and in the lowest probability for the slope stability. This requires that calculations are done for each discontinuity set and for all possible combinations of discontinuity sets. This calculation is tedious and it is normally done by computer. However, a rock mass does not always contain more than one discontinuity set, or it is obvious which discontinuity set(s) will result in the lowest possible values, or a computer is not available. Therefore forms are provided which can be used for the calculations. One form should be used for each geotechnical unit.

ITC/TU ENGINEERING GEOLOGY	exposure characterization		SSPC - SYSTEM
LOGGED BY:	DATE:	TIME: hr	exposure no:

WEATHER CONDITIONS		LOCATION		
Sun:	cloudy/fair/bright	Map coordinates:	map no:	
Rain:	dry/drizzle/slight/heavy		northing:	
			easting:	

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

DIMENSIONS/ACCESSIBILITY			
Size total exposure: (m)	l:	h:	d:
mapped on this form: (m)	l:	h:	d:
Accessibility:	poor/fair/good		

FORMATION NAME:

DESCRIPTION (BS 5930: 1981)				
colour	grain size	structure & texture	weathering	NAME

INTACT ROCK STRENGTH (IRS) (tick)		sample number(s):
< 1.25 MPa	Crumbles in hand	
1.25 - 5 MPa	Thin slabs break easily in hand	
5 - 12.5 MPa	Thin slabs broken by heavy hand pressure	
12.5 - 50 MPa	Lumps broken by light hammer blows	
50 - 100 MPa	Lumps broken by heavy hammer blows	
100 - 200 MPa	Lumps only chip by heavy hammer blows (Dull ringing sound)	
> 200 MPa	Rocks ring on hammer blows. Sparks fly.	

WEATHERING (WE)	
(tick)	
unweathered	1.00
slightly	0.95
moderately	0.90
highly	0.62
completely	0.35

DISCONTINUITIES B=bedding C=Cleavage J=joint			..1	..2	..3	..4	..5
Dip direction		(degrees)					
Dip		(degrees)					
Spacing (DS)		(m)					
persistence	along strike	(m)					
	along dip	(m)					
CONDITION OF DISCONTINUITIES							
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 cm²)	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					

EXISTING SLOPE?	
dip-direction/dip	
/	
height:	m
Stability (tick)	
stable	1
small problems in near future	2
large problems in near future	3
small problems	4
large problems	5

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.) roughness should be assessed perpendicular and parallel to the roughness and directions noted on this form.
3) Non-fitting of discontinuities should be marked in roughness columns.

SUSCEPTIBILITY TO WEATHERING (SW)			remarks:
degree of weathering:	date excavation:	remarks:	

Fig. A 122. Exposure characterization.

ITC/TU ENGINEERING GEOLOGY	reference rock mass calculation	SSPC - SYSTEM
CALCULATED BY:	DATE:	exposure no:
REFERENCE UNIT NAME:		

INTACT ROCK STRENGTH (RIRS)

If IRS > 132 MPa then RIRS = 132 else RIRS = IRS (in MPa) / WE (correction for weathering) = ... / ..... = ...

DISCONTINUITY SPACING (RSPA)

DISCONTINUITIES	..1	..2	..3	..4	..5
Dip direction (degrees)					
Dip (degrees)					
Spacing (DS) (m)					

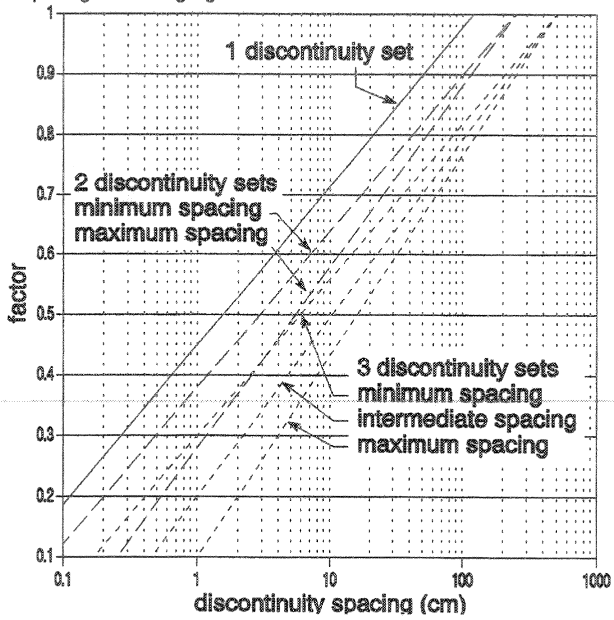
SPA (see figure below) = factor1 * factor2 * factor3 =

.... * .... * .... = ...

corrected for method of excavation and weathering:
RSPA = SPA / (ME * WE)
(with a maximum of 1.00)

RSPA = ... / (.....*.....) = ...

The spacing parameter (SPA) is calculated based on the three discontinuity sets with the smallest spacings following figure:



CONDITION OF DISCONTINUITIES (RTC & RCD)

DISCONTINUITIES	..1	..2	..3	..4	..5
Roughness large scale (Rl)					
Roughness small scale (Rs)					
Infill material (Im)					
Karst (Ka)					
Total (Rl*Rs*Im*Ka = TC)					
RTC					

RTC is the discontinuity condition of a single discontinuity (set) in the reference rock mass corrected for discontinuity weathering.
RTC = TC / sqrt(1.452 - 1.220 * e^(-WE))

Weighted by spacing:

$$CD = \frac{\frac{TC1}{DS1} + \frac{TC2}{DS2} + \frac{TC3}{DS3}}{\frac{1}{DS1} + \frac{1}{DS2} + \frac{1}{DS3}} = \frac{\frac{.....}{.....} + \frac{.....}{.....} + \frac{.....}{.....}}{\frac{1}{.....} + \frac{1}{.....} + \frac{1}{.....}} = ...$$

corrected for weathering: RCD (with a maximum of 1.0165) = CD / WE = ... / .... = ...

REFERENCE UNIT FRICTION AND COHESION (RFRI & RCOH)

Rock mass friction: RFRI = RIRS * 0.2417 + RSPA * 52.12 + RCD * 5.779

RFRI = ... * 0.2417 + ... * 52.12 + ... * 5.779 = ... °

Rock mass cohesion: RCOH = RIRS * 94.27 + RSPA * 28629 + RCD * 3593

RCOH = ... * 94.27 + ... * 28629 + ... * 3593 = ... Pa

notes: 1) For IRS (intact rock strength) take average of lower and higher boundary of class.
2) Roughness parameters 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 soil, 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. A 123. Reference rock mass calculation.

ITC/TU ENGINEERING GEOLOGY	slope stability probability	SSPC - SYSTEM
LOGGED BY:	DATE:	slope no:

LOCATION	map no:	
Map coordinates:	northing:	
	easting:	

DETAILS OF SLOPE

METHOD OF EXCAVATION (SME)		WEATHERING (SWE)		
(tick)		(tick)		Slope dip direction (degrees):
natural/hand-made	1.00	unweathered	1.00	
pneumatic hammer excavation	0.76	slightly	0.95	
pre-splitting/smooth wall blasting	0.99	moderately	0.90	**Slope dip** (degrees):
conventional blasting with result:		highly	0.62	
good	0.77	completely	0.35	Height (**Hslope**) (m):
open discontinuities	0.75			
dislodged blocks	0.72	note: SWE = 1.00 for 'soil type' units,		
fractured intact rock	0.67	e.g. cemented soil, etc.		
crushed intact rock	0.62			

SLOPE UNIT NAME:

ORIENTATION INDEPENDENT STABILITY

INTACT ROCK STRENGTH (SIRS)

SIRS = RIRS (from reference rock mass) * SWE (weathering slope) = ... * ..... = ...

DISCONTINUITY SPACING (SSPA)

SSPA = RSPA (from reference rock mass) * SWE (weathering slope) * SME (method of excavation slope)

SSPA = ... * .... * .... = ...

CONDITION OF DISCONTINUITIES (SCD)

SCD = RCD (from reference rock mass) * SWE (weathering slope)

SCD = ... * ..... = ...

SLOPE UNIT FRICTION AND COHESION (SFRI & SCOH)

Rock mass friction: SFRI = SIRS * 0.2417 + SSPA * 52.12 + SCD * 5.779

SFRI = ... * 0.2417 + ... * 52.12 + ... * 5.779 = ...°

Rock mass cohesion: SCOH = SIRS * 94.27 + SSPA * 28629 + SCD * 3593

SCOH = ... * 94.27 + ... * 28629 + ... * 3593 = ... Pa

If SFRI < slope dip: MAXIMUM SLOPE HEIGHT (Hmax)

Maximum possible height: Hmax = $1.6 * 10^{-4}$ * SCOH * sin(slope dip) * cos(SFRI) / (1-cos(slope dip - SFRI))

Hmax = $1.6 * 10^{-4}$ * .... * sin(....°) * cos(....°) / (1-cos(....° - ....°)) = ... m

ratios:

SFRI / slope dip = ... ° / ..... ° = ...

Hmax / Hslope = ... m / ..... m = ...

Probability stable: if SFRI > slope dip probability = 100 % else use figure orientation independent stability: ..... %

ORIENTATION DEPENDENT STABILITY

DISCONTINUITIES		..1	..2	..3	..4	..5
Dip direction	(degrees)					
Dip	(degrees)					
With, Against, Vertical or Equal						
AP	(degrees)					
RTC (from reference form)						
STC = RTC * sqrt(1.452 - 1.220 * e^(-SWE))						
Probability stable:		.... %	.... %	.... %	.... %	.... %

Determination orientation stability:
calculation AP: β = discontinuity dip, σ = slope dip-direction, τ = discontinuity dip-direction: δ = σ - τ: AP = arctan (cos δ * tan β)

	stability:	sliding	toppling
AP > 84° or AP < -84°	vertical	100 %	100 %
(slope dip+5°) < AP < 84°	with	100 %	100 %
(slope dip-5°) < AP < (slope dip+5°)	equal	100 %	100 %
0° < AP < (slope dip-5°)	with	use graph sliding	100 %

	stability:	sliding	toppling
AP < 0° and (-90° - AP + slope dip) < 0°	against	100 %	100 %
AP < 0° and (-90° - AP + slope dip) > 0°	against	100 %	use graph toppling

Fig. A 124. Slope stability probability calculation.

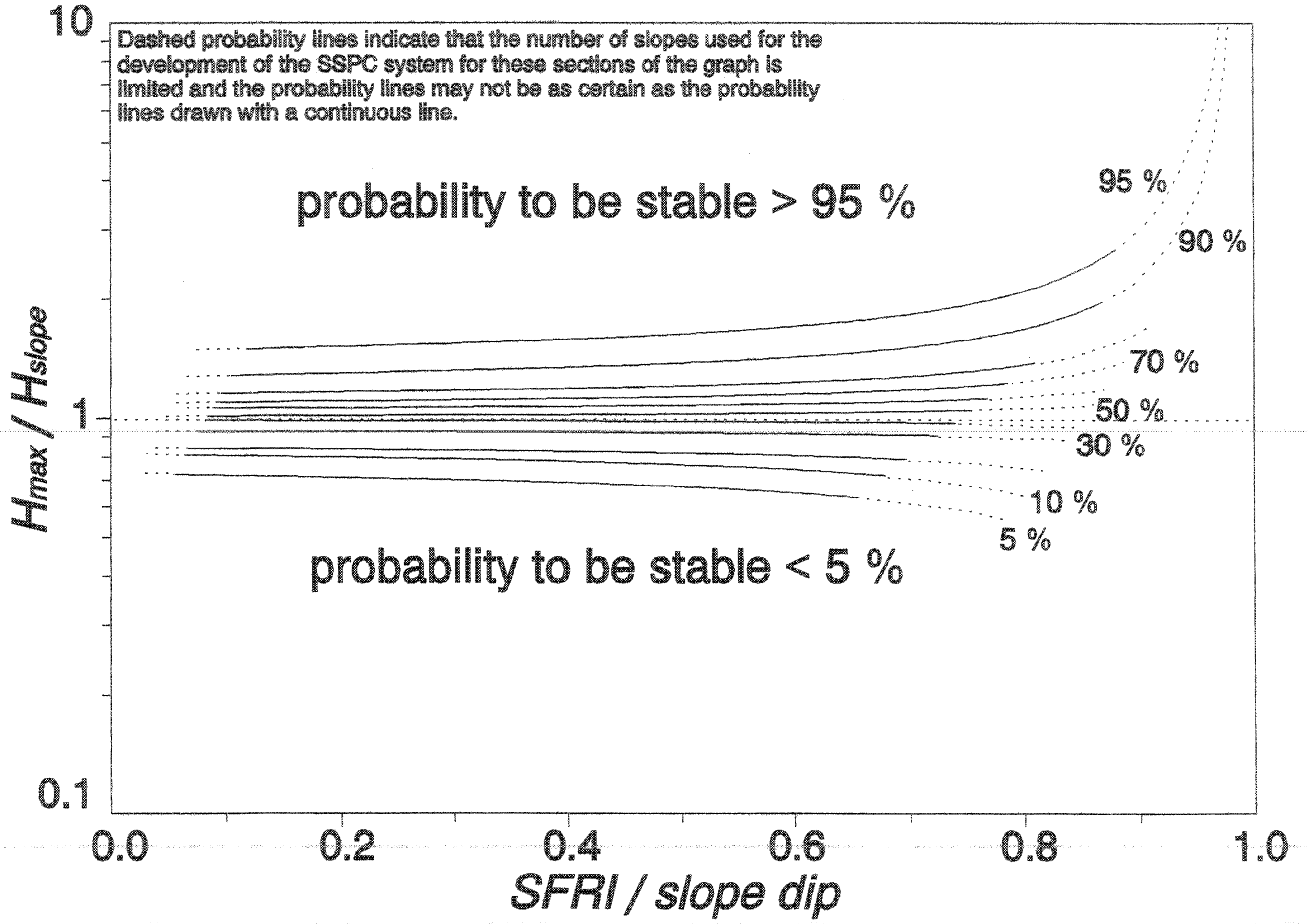


Fig. A 125. Probability of orientation independent slope stability. Values indicate the probability of a slope to be stable.

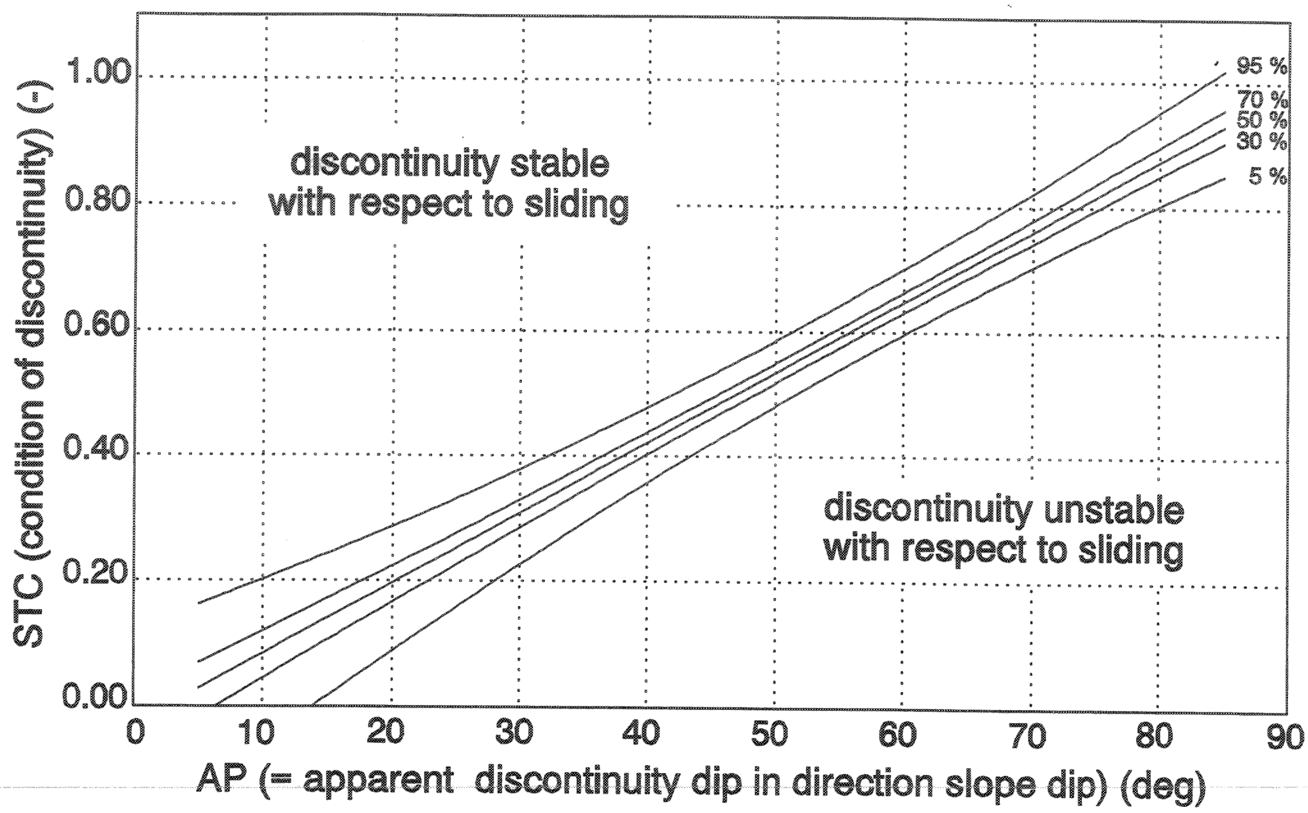


Fig. A 126. Sliding probability for orientation dependent slope stability.

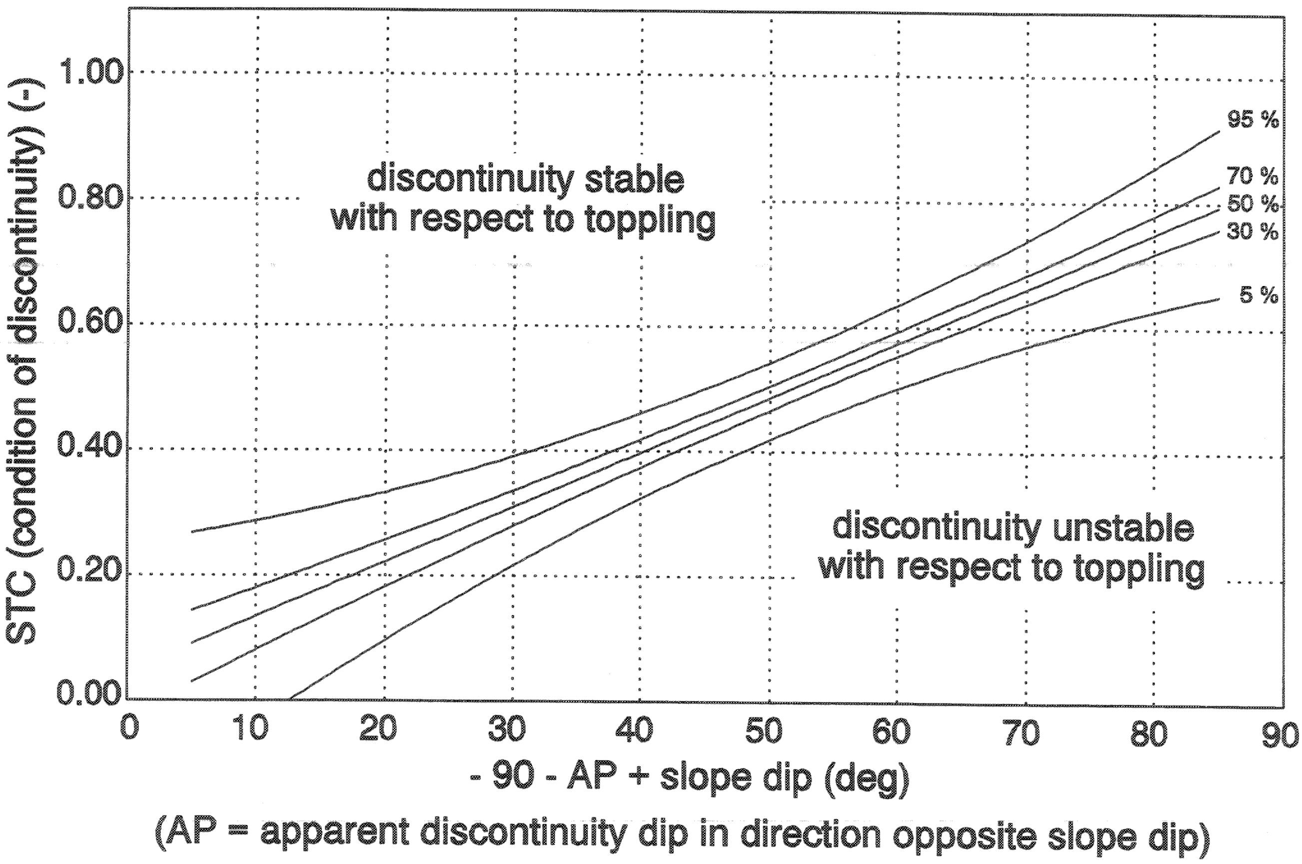


Fig. A 127. Toppling probability for orientation dependent slope stability.

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GLOSSARY

Definitions for rock, rock mass and their properties are not used uniformly in the literature. Therefore definitions of terminology which is frequently used are listed below to avoid confusion. In section A the main terms and relations are described in more detail. Geological terminology is explained as far as necessary for the understanding of the research and analyses for the SSPC system and is based on the Dictionary of Geological Terms of the American Geological Institute (AGI, 1976) and the Geological Nomenclature of the Royal Geological and Mining Society of the Netherlands (KNGMG, 1980).

i-angle
See 'bi-linear shear criterion'.

φ_{basic}
In this research φ_{basic} denotes the friction of a non-displaced (fitting) discontinuity which friction does not cause opening of the discontinuity (dilatancy). Confusion has arisen in the literature about φ_{basic}. Some authors use φ_{basic} also for the φ_m of rock material, for $\varphi_{residual}$ (which is the φ obtained after large displacements), or use the term for artificial surfaces (saw cuts). The φ of these surfaces might be the same as the φ_{basic} of a discontinuity but this is not necessarily so. See further 'bi-linear shear criterion'.

φ_m
See 'bi-linear shear criterion'.

Abutting discontinuities
See persistence.

Anisotropy
The dependency on direction of properties of rock or rock mass.

Bi-linear shear criterion
Terminology of shear strength along a discontinuity is easiest explained with the 'bi-linear shear criterion' (Patton, 1966). For more sophisticated relations for shear strength along discontinuities is referred to the appropriate literature. The shear strength along a discontinuity is for a discontinuity with a regular set of triangular shaped asperities formulated by Patton in the 'bi-linear shear criterion' (Fig. G 128). The angle of friction (φ_{basic}) is a material constant depending on the structure, texture, type of material, roughness and degree of interlocking of the discontinuity surfaces. The roughness included in φ_{basic} should not cause dilatancy of the discontinuity (opening in the direction perpendicular to the shearplane, δv in Fig. G 128). The roughness that causes dilatancy of the discontinuity is described by the angle of roughness (*i-angle* = *arctan* $\delta v/\delta h$). In Fig. G 128 the roughness are the triangular asperities. Depending on the steepness of the asperities and the normal stress across the discontinuity, the asperities break rather than are overridden. The shear strength is then described by the rock material parameters cohesion S_m and friction φ_m. If there is no gluing or bonding agent (for example, cement) between the discontinuity walls the cohesion is described as apparent cohesion. The cohesion, or a part of it, may be real cohesion if a gluing or bonding agent is present. The parameters cohesion S_m and friction φ_m are normally not the same as the cohesion

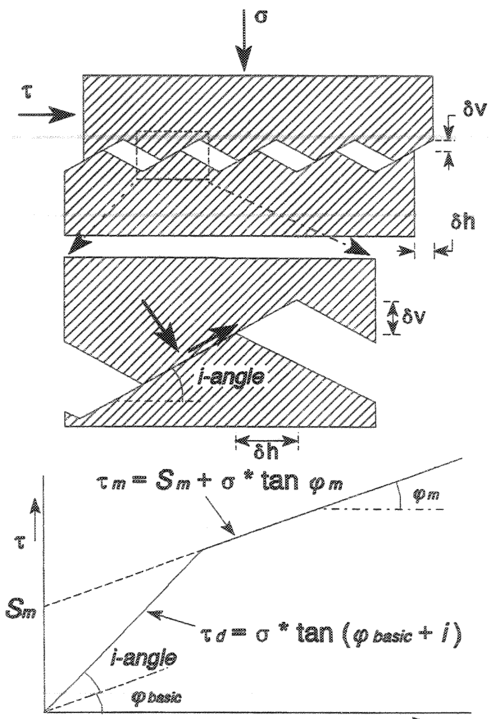

Fig. G 128. 'Bi-linear shear criterion' for a discontinuity with a regular set of triangular shaped asperities (modified after Patton, 1966).

and friction (φ) of the rock material defined in the 'Mohr-Coulomb failure criterion'.

Characteristic discontinuity orientation
The characteristic discontinuity orientation is the mean of the orientations of the discontinuities in a discontinuity set.

Characteristic discontinuity spacing
The spacing of discontinuities within one set of discontinuities is defined as the perpendicular distance between two discontinuity planes (Fig. G 129). The characteristic discontinuity spacing is the mean of the spacings between discontinuities in a discontinuity set.

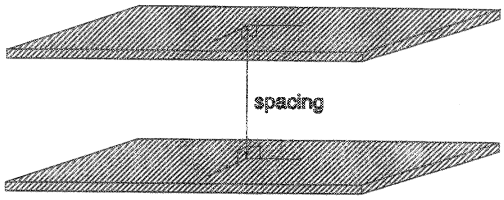

Fig. G 129. Discontinuity spacing.

Characterization
Characterization is the description of a unit. A characterization is not automatically a classification.

Cleavage (slaty cleavage)
A tendency to cleave or split along definite, parallel, closely spaced planes, which may be highly inclined to the bedding planes. It is a secondary structure, commonly confined to bedded rocks, is developed by pressure, and ordinarily is accompanied by at least some recrystallization of the rocks. (In this study used for Carboniferous rocks which contain a 'slaty cleavage').

Classification
Classification is the characterization (description) of a unit by standard parameters which are empirically related to an engineering application. A weighting of the parameters according to standard rules will lead to a recommendation for an engineering application.

Cohesion (apparent)
For the strength description of rock, rock mass and soil see 'Mohr-Coulomb failure criterion'; for discontinuities see 'bi-linear shear criterion'.

Compressive strength
The compressive strength is the compressive stress at failure of a sample under a compressive stress (σ_1). Compressive strength of rock or rock mass material can be tested under different stress configurations. Depending on the type of test done the compressive strength is denoted as Unconfined Compressive Strength (UCS), triaxial compressive strength or true triaxial compressive strength. Unconfined compressive and triaxial tests are normally done on cylindrical samples and a true triaxial test is done on a rectangular (mostly cubic) sample.

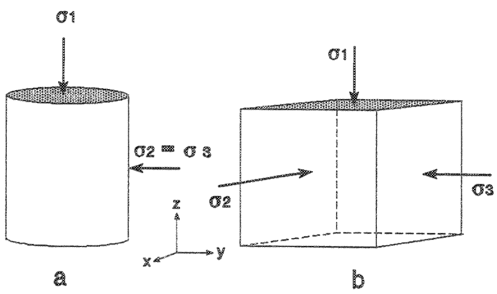

Fig. G 130. Compressive strength.

- *Unconfined Compressive Strength (UCS)*
The compressive stress (σ_1) is measured at failure of the sample under the condition that the confining pressure is zero ($\sigma_2 = \sigma_3 = 0$) (Fig. G 130a).
- *Triaxial compressive strength*
The compressive stress (σ_1) is measured at failure of a sample that is under a confining pressure. The confining pressure is equal in x and y direction ($\sigma_2 = \sigma_3$) (Fig. G 130a).
- *True triaxial compressive strength*
The compressive stress (σ_1) is measured at failure of a sample that is under confining pressure. The confining pressure is not equal in x and y direction ($\sigma_2 \neq \sigma_3$) (Fig. G 130b).

Creep
Creep in rock mechanics is a confusing term. Various forms of plastic or time dependent deformation processes which are governed by totally different physical or chemical processes are all described as creep (ch. A.2.4).

Day-lighting
'Day-lighting' denotes that a discontinuity has a dip less than, but in the same general direction as, the slope dip, and is outcropping in the slope: the difference between slope dip-direction and discontinuity dip-direction should be less than 90° and the slope dip should be steeper than the discontinuity dip.

Deformation
Deformation of intact rock or of a rock mass is the change in volume or shape.

Dilatancy
The tendency of a fitting discontinuity to open perpendicular to the discontinuity plane if sheared along the discontinuity because the asperities on the discontinuity surface are overridden.

Discontinuity
A discontinuity is a plane which marks a change in the physical or chemical characteristics of rock material.

- Single discontinuity	A single discontinuity denotes a single isolated discontinuity (single fault, isolated crack or joint, etc.) that is not part of a discontinuity set or, if part of a discontinuity set, then the spacing between the different discontinuities is so large that for practical engineering purposes the discontinuity may be considered as a single feature..
- Discontinuity set	A discontinuity set or discontinuity family denotes a series of discontinuities of which the geological and mechanical characteristics as well as their orientation are broadly the same (examples are: sets of bedding planes, schistosity planes, cleavage planes, joint sets, etc.).
- Integral discontinuities	Integral discontinuities are discontinuities for which there is no change in strength compared to the surrounding rock material. Intact rock may contain integral discontinuities.
- Mechanical discontinuities	Mechanical discontinuities are planes of physical weakness. Bedding, joints, fractures, faults, etc. are mechanical discontinuities if the tensile strength perpendicular to the discontinuity or the shear strength along the discontinuity are lower than in the surrounding rock material. Mechanical discontinuities will in general be the boundaries for 'banks' of intact rock. The term bank is, however, not used as the definition of a bank is based on sedimentological characteristics.

In this study 'discontinuities' is used for mechanical discontinuities except where otherwise stated.

Discontinuous rock mass
A rock mass containing discontinuities (see also *rock mass)*.

Engineering lifetime
Engineering lifetime denotes the expected existence of an engineering structure. Slopes are often designed for a lifetime of about 50 years.

Failure mechanisms and modes
Processes leading to slope failure are divided into different mechanisms that are sub-divided into different modes. For example, slope failure mechanisms are shear displacement, deterioration of rock material, intact rock creep, etc.; the resulting failure modes of the shear displacement mechanism are plane sliding, wedge failure, partially toppling and, to some extent, buckling.

Fitting discontinuity
A discontinuity in which the asperities of both discontinuity walls are complementary and the discontinuity walls are not displaced. Displacement along a fitting discontinuity can only take place if the asperities are sheared off, deformed or if the asperities are overridden (causing dilatancy; see before).
Non-fitting discontinuity: the asperities are not complementary or the opposing discontinuity walls have been displaced causing that the asperities are not fitting.

Formation
The primary unit of formal geological mapping or description. Most formations possess certain distinctive or combinations of distinctive (lithological) features. Boundaries are not based on time criteria.

Friction (φ)
For the strength description of rock, rock mass and soil see 'Mohr-Coulomb failure criterion'; for discontinuities see 'bi-linear shear criterion'.

Geotechnical unit
See unit - geotechnical.

Gouge
Claylike material containing rock fragments of the surrounding rock, occurring between the walls of a continuous discontinuity (mostly: faults or major discontinuities) as a result of wear during displacement.
In a discontinuity described as a 'gouge filled' discontinuity the rock fragments in the discontinuity do not make contact with both discontinuity walls; thus the initial shear strength is governed by the clay material.

Identification
Identification describes the effect that a relation is defined that includes more parameters then necessary to relate the data. The parameters are not determined by the relation. For example:

$$y = (a + b) * x$$
$$x, y = data \quad a, b = parameters$$

Both a and b can never be determined from this relation whatever the number of (x, y) data pairs. (Obviously for determination of (a + b) only one data pair (x, y) is sufficient.)
In optimization of complex relation(s) identification problems might not be recognized leading to ambiguous results.

Inhomogeneity
Inhomogeneity is the spatial variation of properties of intact rock or of a rock mass.

Intact rock
Intact rock blocks are blocks of rock for which: 1) The physical and mechanical properties are roughly uniform. 2) The particles (mineral grains, rock grains, etc.) are bounded by a cementing agent which causes a block of intact rock to have a tensile strength. 3) An intact rock block does not contain mechanical discontinuities.

Isotropy
Isotropy designates that properties of intact rock or of a rock mass are not direction dependent.

Lithology - lithological
The science of the rocks; in this study lithology denotes the type of minerals, their origin or sedimentation environment.

Lithostratigraphic (sub-) unit
See unit - lithostratigraphic

Lubrication
Lubrication by water may reduce the shear strength of discontinuities. The effect may be caused by the water itself which changes the mechanical characteristics of some materials. Another more general effect is that the presence of water will cause a reduction in shear strength because the surface stresses of water will cause a reduction of the normal stresses on the discontinuity walls. The quantity of water is not necessarily so large that an overall water pressure is established.

Lustre
The appearance of a stone's surface (or of a mineral in general) in reflected light. Refraction index and perfection of polish possessed by the stone are the main factors affecting lustre, while hardness is also of some importance.

Mapping unit
See unit - mapping.

Mohr-Coulomb failure criterion
The 'Mohr-Coulomb failure criterion' consists of a linear envelope (eq. [63]) touching all Mohr's circles representing critical combinations of principal stresses in the rock or rock mass, or soil (Fig. G 131).

$$\tau_{failure} = cohesion + \sigma_{failure} * \tan(\varphi) \qquad [63]$$
cohesion and φ are the cohesion and angle of internal friction of the material

Expressed in the 'Mohr-Coulomb failure criterion' the unconfined compressive strength (UCS) equals:

$$UCS = 2 * cohesion * \tan\left(45° + \frac{\varphi}{2}\right) \qquad [64]$$

The relation between minor (σ_3) and major (σ_1) principal stress at failure is:

$$\sigma_1 = UCS + \sigma_3 * \tan^2\left(45° + \frac{\varphi}{2}\right) \qquad [65]$$

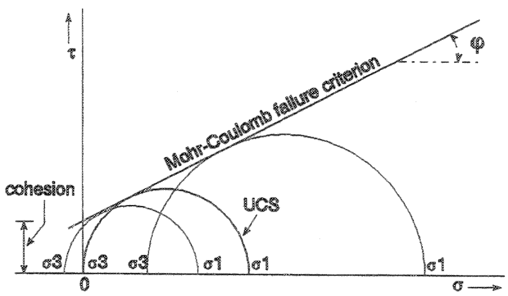

Fig. G 131. Mohr-Coulomb failure criterion.

Non-fitting discontinuity
See fitting discontinuity.

Non-persistent discontinuities
See persistence.

Orientation
See characteristic discontinuity orientation.

Overfit
Overfit describes the effect that a relation is defined that includes more parameters then necessary to relate the data. In optimization scatter on the data will cause that multiple, equally good, solutions are found. Each solution is a solution on different (clustered) subsets of the data set. None of these solutions need to be the solution for the full data set.

Outlier
An outlier is a data point which is clearly detached, or out from the main set of data points.

Persistence (Fig. G 132)
- *Persistent discontinuities*
Persistent discontinuities are formed by a continuous discontinuity plane. Shear displacement takes place if the shear stress along the discontinuity plane exceeds the shear strength of the discontinuity plane. If unfavourable orientated it is often a sliding plane in slopes.
- *Abutting discontinuities*
Abutting discontinuities are discontinuities which stop at the intersection with another discontinuity plane. Abutting discontinuities might continue at the other side of the intersecting discontinuity, however, with a displacement to give so-called 'stepped planes'[(148)]. Shear displacement along the discontinuity can take place if 1) the shear strength along the discontinuity plane is exceeded and 2) the blocks of rock against which the discontinuity abuts can move.
- *Non-persistent discontinuities*
Non-persistent discontinuities are discontinuities ending in intact rock. Before movement of the blocks on both sides of a non-persistent discontinuity is possible, the discontinuity has to extend and break through intact rock material. As intact rock material has virtually always a far higher shear strength than the discontinuity, a non-persistent discontinuity will have a larger shear strength than a persistent discontinuity.

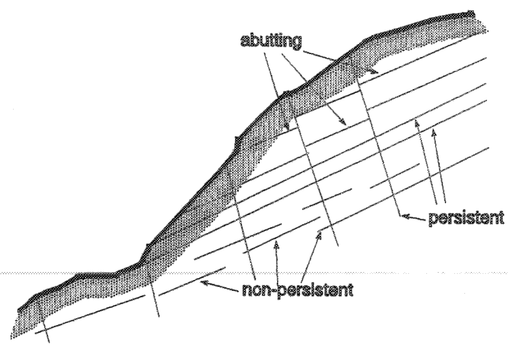

Fig. G 132. Persistent, non-persistent and abutting discontinuities.

Porphyritic, porphyrite
A textural term for those igneous rocks in which larger crystals are set in a finer groundmass.

Rock mass
A rock mass is a mass of rock blocks with or without discontinuities. A rock mass may be homogeneous or inhomogeneous. Based on rock mass parameters the rock mass is divided in homogeneous geotechnical units.

Rock (mass) failure
A rock mass is supposed to have failed if the rock mass deforms more than allowed for a safe engineering application.

Shear strength
The shear strength is the shear stress at failure of a sample under a shear stress. See for shear strength along a discontinuity 'bi-linear shear criterion'.

Slaty cleavage
See cleavage.

Slickensided
Usually striated surface of rock produced by friction.

Soil type units
'Soil type' units describe units which consist of loosely cemented grains or small particles, generally either without clearly defined mechanical discontinuities or having highly irregular and thinly laminated mechanical discontinuities, and having a low intact rock strength. 'Soil type' units resemble cemented soils rather than a rock mass.

(148) Stepped discontinuity planes should not be confused with discontinuity planes with steps. A discontinuity plane with a step is described in appendix II.

Spacing
See characteristic discontinuity spacing.

Striated
Surface of rock characterized by fine, narrow, curved or straight parallel grooves.

Stylolite
A term applied to parts of certain limestones which have a columnlike development; the columns being generally at right angles or highly inclined to the bedding planes, having grooved, sutured or striated sides, and irregular cross sections. Stylolites result from solution under pressure of limestone. The clay particles which were originally in the limestone, remained on the solution surface.

Susceptibility to weathering
See weathering.

Tactile roughness
Roughness that can be felt by using fingers.

Tensile strength
The tensile strength is the tensile stress at failure of a sample under a tensile stress.

Triaxial compressive strength
See compressive strength.

Unconfined Compressive Strength (UCS)
See compressive strength.

Unit
The following definitions are used in this study:
- *Lithostratigraphic unit*
A layer or a body of layers characterized by consisting dominantly of a certain lithologic type (sand, clay, sandstone, shale, granodiorite, etc.).
- *Lithostratigraphic sub-unit*
A lithostratigraphic unit which characteristic bedding or cleavage spacing is within the ranges for discontinuity spacing as given by BS 5930 (1981) (Table A 17, page 181).
- *Geotechnical unit*
A geotechnical unit is a part of the rock mass in which the mechanical characteristics of the intact rock material are uniform in each block of intact rock and the mechanical properties (including orientation) of the discontinuities within each set of discontinuities are uniform. Anisotropy of properties, if present, is uniform (ch. A.2.2).
- *Mapping unit*
The divisions made on an engineering geological map.
(note: in this study 'lithostratigraphic sub-units' are a subdivision defined on bedding or cleavage spacing, of the 'lithostratigraphic units' found in the research area.)

Weathering
Weathering is the chemical and physical change in time of intact rock and rock mass material under influence of atmosphere and hydrosphere (temperature, rain, circulating ground water, etc.) (ch. A.2.4). A distinction is made between 1) the degree (state) of weathering (at a certain moment) and 2) the susceptibility to weathering (in a certain time-span).

SYMBOLS & ABBREVIATIONS

SSPC indicates that the expression is used in the slope stability probability classification system. Symbols and codes used in the forms for the 'initial point rating' system (ch. C.4) are not included.

(-)	used in graphs to indicate that the parameter has no dimension (e.g. H_{max}/H_{slope} in Fig. 67)
φ	friction angle
φ_{mass}	angle of internal friction of a rock mass
AP	apparent angle of dip of a discontinuity in the direction of the slope dip (AP > 0°) or opposite the direction of the slope dip (AP < 0°) (SSPC)
CD	parameter for the weighted overall condition of a number of discontinuity sets in an exposure rock mass unit (SSPC)
coh_{mass}	cohesion of a rock mass
con_{mass}	parameter for the overall condition of a number of discontinuity sets in a rock mass
DS	characteristic spacing (in metres) between the discontinuities in one discontinuity set in an exposure rock mass unit (SSPC)
e	natural base of logarithms (e = 2.7182818...)
Hmax	maximum possible height of a slope if SFRI is lower than the slope dip (SSPC)
Hslope	height of a slope (SSPC)
i-angle	angle of roughness for discontinuities
Im	parameter for discontinuity infill material in an exposure rock mass unit (SSPC)
IRS or *irs*	intact rock strength; in the SSPC system used for the intact rock strength of an exposure rock mass unit
Ka	parameter for karst along a discontinuity in an exposure rock mass unit (SSPC)
$\log_e x$	natural logarithm of x (base e)
$\log_{10} x$	common logarithm of x (base 10)
ME	parameter for the method of excavation used for an exposure (SSPC)
RCD	parameter for the weighted overall condition of a number of discontinuity sets in a reference rock mass unit (SSPC)
RCOH	cohesion of a reference rock mass unit (SSPC)
RFRI	angle of internal friction of a reference rock mass unit (SSPC)
RIRS	intact rock strength of a reference rock mass unit (SSPC)
Rl	parameter for the large scale roughness of a discontinuity in an exposure rock mass unit (SSPC)
Rs	parameter for the small scale roughness of a discontinuity in an exposure rock mass unit (SSPC)
RSPA	parameter for the overall spacing of a number of discontinuity sets in a reference rock mass unit (SSPC)
RTC	parameter for the condition of a discontinuity (set) in a reference rock mass unit (SSPC)
SCD	parameter for the weighted overall condition of a number of discontinuity sets in a slope rock mass unit (SSPC)
SCOH	cohesion of a slope rock mass unit (SSPC)
SFRI	angle of internal friction of a slope rock mass unit (SSPC)
SIRS	intact rock strength of a slope rock mass unit (SSPC)
SME	parameter for the method of excavation used for a new slope (SSPC)
SPA	parameter for the overall spacing of a number of discontinuity sets in an exposure rock mass unit (SSPC)
spa_{mass}	parameter for the overall spacing of a number of discontinuity sets in a rock mass
SSPA	parameter for the overall spacing of a number of discontinuity sets in a slope rock mass unit (SSPC)
STC	parameter for the condition of a discontinuity (set) in a slope rock mass unit (SSPC)
st.dev.	standard deviation
SW	susceptibility to weathering (SSPC)

SWE (= WE_{mass})	parameter for the degree of weathering of a slope rock mass unit at the end of the engineering lifetime of a slope (SSPC)
TC	parameter for the condition of a discontinuity (set) in an exposure rock mass unit (SSPC)
WE (= WE_{mass})	parameter for the degree of weathering of an exposure rock mass unit (SSPC)
WE_{single}	weathering parameter for the condition of a discontinuity (set) in a rock mass unit
$WE_{coh\ mass}$	weathering parameter for the cohesion of a rock mass unit
$WE_{\varphi\ mass}$	weathering parameter for the angle of internal friction of a rock mass unit
WE_{mass}	weathering parameter for a rock mass unit

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FIGURES

TABLES

COMPUTER PROGRAMMES

Name	Manufacturer	Application
Clipper	Computer Associates International, Inc., One Computer Associates Plaza, Islandia, New York 11788-7000, USA.	Compiler (and programming language) for calculation and database programmes for slope stability probability classification.
Dbase III Plus & IV	Borland, 100 Borland Way, Scotts Valley, California 95066-3249, USA.	Relational databases for field and classification data.
DIPS 2.0	E. Hoek & M. Diederichs, Rock Engineering Group, Dept. of Civil Engineering, University of Toronto, Canada.	Stereo projection and contour plots for example III.
ILWIS, version 1.4	ITC (Int. Inst. for Aerospace Survey and Earth Sciences), P.O. Box 6, 7500 AA Enschede, The Netherlands.	Map of 'research area' in chapter A.
Mathcad for Windows, version 5.0	MathSoft Inc. 101 Main Street, Cambridge, Massachusetts, 02142 USA.	Optimization of linear and non-linear functions in orientation independent slope stability and probability analyses.
SlideWrite Plus, version 2.0	Advanced Graphics Software Inc. 5825 Avenida Encinas, Suite 105, Carlsbad, CA 92008-4404, USA.	Curve fitting for orientation dependent and independent slope stability.
SSPCCLAS	H.R.G.K. Hack, Section Engineering Geology, ITC (Int.Inst. for Aerospace Survey and Earth Sciences), Kanaalweg 3, 2628 EB Delft, The Netherlands.	Calculation programme for SSPC system (written in Clipper).
Statgraphics, version 5	Manugistics Inc., 2115 East Jefferson St., Rockville, Maryland, 20852, USA.	Histograms and normal distributions for probability analyses.
UDEC, version 1.8	UDEC, Universal Distinct Element Code. ITASCA Consulting group. Minneapolis, Minnesota, USA.	Distinct element analyses for examples II and III.
WordPerfect, version 5.1+ for DOS	Corel Corporation. Corel Building, 1600 Carling Avenue, Ottawa, Ontario, K1Z 8R7, Canada.	Text and layout.
WordPerfect Presentations, version 2.0 for DOS	idem	Figures.

CURRICULUM VITAE

The author is employed by ITC in Delft in the section Engineering Geology since 1990. The author is a staff member of the research school 'Centre for Technical Geoscience' and is president of the Dutch Association of Engineering Geologists (Ingeokring). The author is a member of the Institution of Mining and Metallurgy and a Chartered Engineer.

Besides the research done for the SSPC the author is also involved in research projects for engineering geological mapping, detailed seismic wave behaviour around discontinuities in a rock mass, and in research towards the optimizing of the use of three-dimensional geographical information systems (GIS) and knowledge base systems for engineering geology.

The author worked as engineering geologist and geotechnical engineer in the Middle East, Far East and Africa. Several years he worked as senior rock mechanics engineer in the underground copper mines of Zambia.

The author studied geology at the University of Leiden, The Netherlands, (BSc. 1979) and engineering geology with exploration geophysics at the Technical University Delft and the University of Utrecht, The Netherlands (MSc. 1983). He obtained the Doctor degree at the Technical University Delft in 1996.