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ROCK MASS CHARACTERISATION IN ENGINEERING PRACTICE

1. INTRODUCTION

The rock mass classification systems of Bieniawski (1973) and Barton, Lien and Lunde (1974) were originally developed to provide guidance on the selection of support in blocky rock masses. When applied correctly, within the bounds of the original assumptions upon which they were based, they worked well and there are thousands of kilometres of stable tunnels around the world, which attest to their effectiveness.

The type of rock masses where the behaviour is controlled by sliding and rotation on discontinuity surfaces with relatively little failure of the intact rock pieces is a domain in which the RMR and Q systems work very well. Less successful were attempts to apply these classification systems to very hard massive rock masses at great depths and also to very weak rock masses.

The development of extremely powerful microcomputers and of “user-friendly” software has provided alternative tools for the design of tunnels. However, this has brought with it the need for reliable input data particularly that related to rock mass properties. In order to meet this requirement a different set of rock mass classification schemes have been developed. Geological Strength Index (GSI) is such a system developed by Hoek and extended by Hoek and Marinos (2000). This classification system is used in conjunction with the Hoek and Brown failure criterion and its sole function in the estimation of rock mass properties such as the strength and deformability characteristics of rock masses (rf program RocLab from www.rocscience.com).

2. THE GEOLOGICAL STRENGTH INDEX: AN AID TO ROCK CHARACTERISATION

The structural complexity of rock masses has always presented a challenge to an analyses of their in-situ strength and deformability. In certain circumstances the problem can be simplified with justification if the rock mass contains well-defined, regularly orientated /and persistent discontinuities that are weaker than the rock they bound, and upon which the rock mass will move given the freedom to do so. Here, rigid body movements can be expected and an appropriate analysis generated when the behaviour of the discontinuities is adequately known. The many slope failures by translation, either on a single plane or as a wedge, that have been prevented from occurring by the application of such analyses bear witness to the value of this approach. In other circumstances, a rock may be uniform and reasonably isotropic by reason of the geometry of the discontinuities within it and their mutual intersections or by the fact that the difference between the properties of the rock and the discontinuities are small, and when failure does not depend on anisotropy. When the deformability and strength of such rocks and the masses they form, is unable to carry the loads upon them without serious deformation and displacement, it may be reasonable to consider whether the mass can be treated as “homogenous” in terms of its strength. There may, of course, be some reservations as to the correctness of such an approximation, increasing the need to exercise judgment when assessing the meaning of any analyses based on this precept (Chandler, de Freitas and Marinos 2004). Such a process has been of great assistance to many tunnel designers. Both these approaches require approximations but have a clear basis upon which they are founded. This becomes increasingly difficult to achieve with rock masses where there is a mix of weak and strong materials, cut by surfaces of different strength and orientations. Here the designers have resorted to qualitative descriptions of the rock mass to which numbers may be allocated, reflecting an intuitive understanding of the ability of the feature in question to resist load. This was the basis of rock mass classifications. (Chandler, de Freitas and Marinos 2004). The rock mass ratings have further more continued to provide valuable design elements for engineering work in rock masses and many engineers attempted to use them as a starting place for a more basic analysis.

Hoek and Brown (1980) approached the problem assuming that a rock mass would have the same strength as the rock from which it was composed provided it contained no other sources of weakness, most notably joints, bedding plains, cleavage and similar discontinuities. A basic expression could therefore be expected where the strength of the rock mass equals the strength of the intact rock reduced by factors to account for the character of the rock material and the mass it forms. From this came the “Hoek – Brown failure criteria” for rock masses, subsequently modified (Hoek, et al., 1992, 2002) to

$$\sigma_1' = \sigma_3' + \sigma_c \{ [m_b \, \sigma_3' / \sigma_c] + s \}^a$$

where

σ_1' and σ_3'	are the axial and confining effective stresses at failure;
σ_c	is the unconfined compressive strength of intact fragments of rock material;
m_b	is a constant derived from the triaxial testing of intact rock material in the laboratory, producing the parameter (m_i), which is modified to reflect the rock mass, for example the extent to which the blocks making up the mass are interlocked;
s and a	are constants that reflect other aspects of the rock mass, (s) for example the degree of fracturing present, so that s = 1,0 for intact rock.

One major problem with the Hoek – Brown failure criterion was the difficulty field geologists had in knowing what input was required in quantitative terms. For them, rock mass ratings provided that quantification of visible features (number of joints, roughness of surfaces, etc.). Thus, already existing classifications such as RMR, appropriately adjusted were initially used for imput data. However, incorporating such ratings without further thought can cause aspects of the ground to be considered in an inappropriate way.

It was necessary to introduce a system that placed greater emphasis on basic geological observations of rock mass characters and that reflected the material, its structure and its geological history. This was introduced by Hoek, Wood and Shah (1992), and formed the basis of what is now called the Geological Strength Index (Hoek 1994; Hoek, Kaiser and Bawden, 1995; Hoek and Brown, 1997; Hoek, Marinos and Benissi, 1998; Marinos and Hoek, 2000)

GSI determination is based on qualitative visual descriptions of the rock mass which are easy to make and use; Fig. 1 (Hoek and Marinos, 2000; Marinos, Marinos and Hoek, 2004). Once a value for the Index is decided, it can be used with appropriate values for σ_c and m_i to calculate other mechanical properties of a rock mass, in particular the compressive strength of the rock mass and its deformation modulus (E); the procedures are explained in Hoek and Brown, (1997). The relationship between the Hoek – Brown criterion and the Mohr – Coulomb criterion was addressed by and Hoek, Caranza-Torres and Corkum (2002), where a method for calculating the cohesive strength and angle of friction for a rock mass is presented, for appropriate ranges of stress encountered in tunnels and slopes. These values can be further adjusted by a “damage criterion”, a number that accounts for the reduction in strength arising from stress relaxation and blasting[1].

[1] Solution is provided by the RocLab programme which can be downloaded free from the site: www.rocscience.com

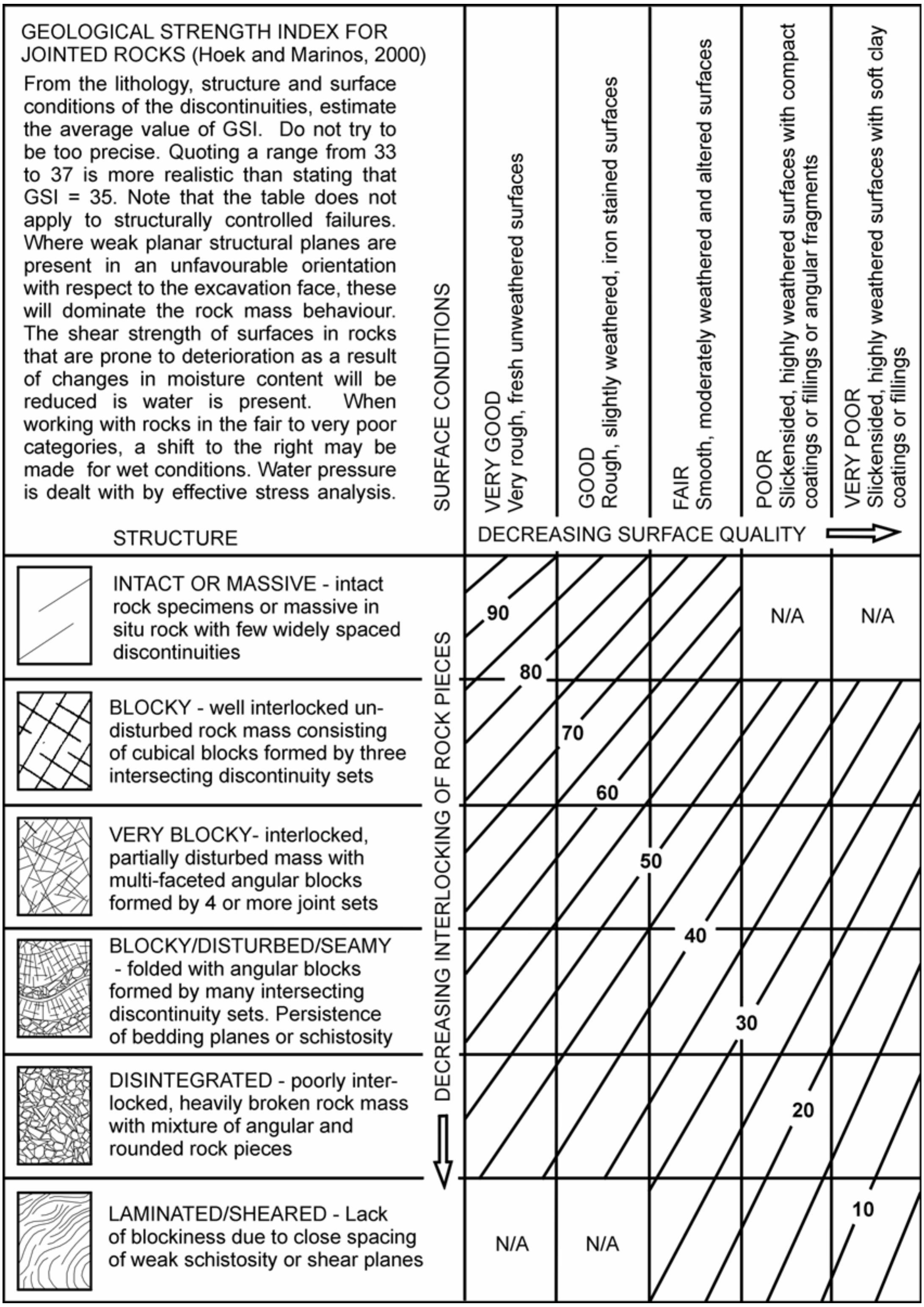

Fig. 1 Geological Strength Index estimates from geological observations (Hoek and Marinos, 2000)

Such a form of rock mass characterization as the Geological Strength Index has considerable potential for use in engineering geology because it permits the manifold aspects of rock to be quantified (Chandler, de Freitas and Marinos 2004). This allows the influence of variables, which make up a rock mass to be assessed, and hence the behaviour of rock masses to be better explained. Examples of this potential are given in the way the index has been extended to accommodate some of the most variable of rock masses, such as flysch and molasse (Hoek and Marinos, 2000; Marinos and Hoek, 2001; Marinos, Marinos, and Hoek, 2004). In such rock masses there is some justification for disregarding the anisotropy arising from “schistosity”, because in many cases the difference in the strength of the rock and that of the discontinuities within it is small. Here the Index has been developed to include extremely poor quality sheared siltstones, clay – shales and chaotic mélanges; Fig. 2 (Marinos and Hoek, 2000). One of the advantages of the Index is that the geological reasoning it ambodies allows quantitative adjustments of its ratings to be made for local ground conditions.

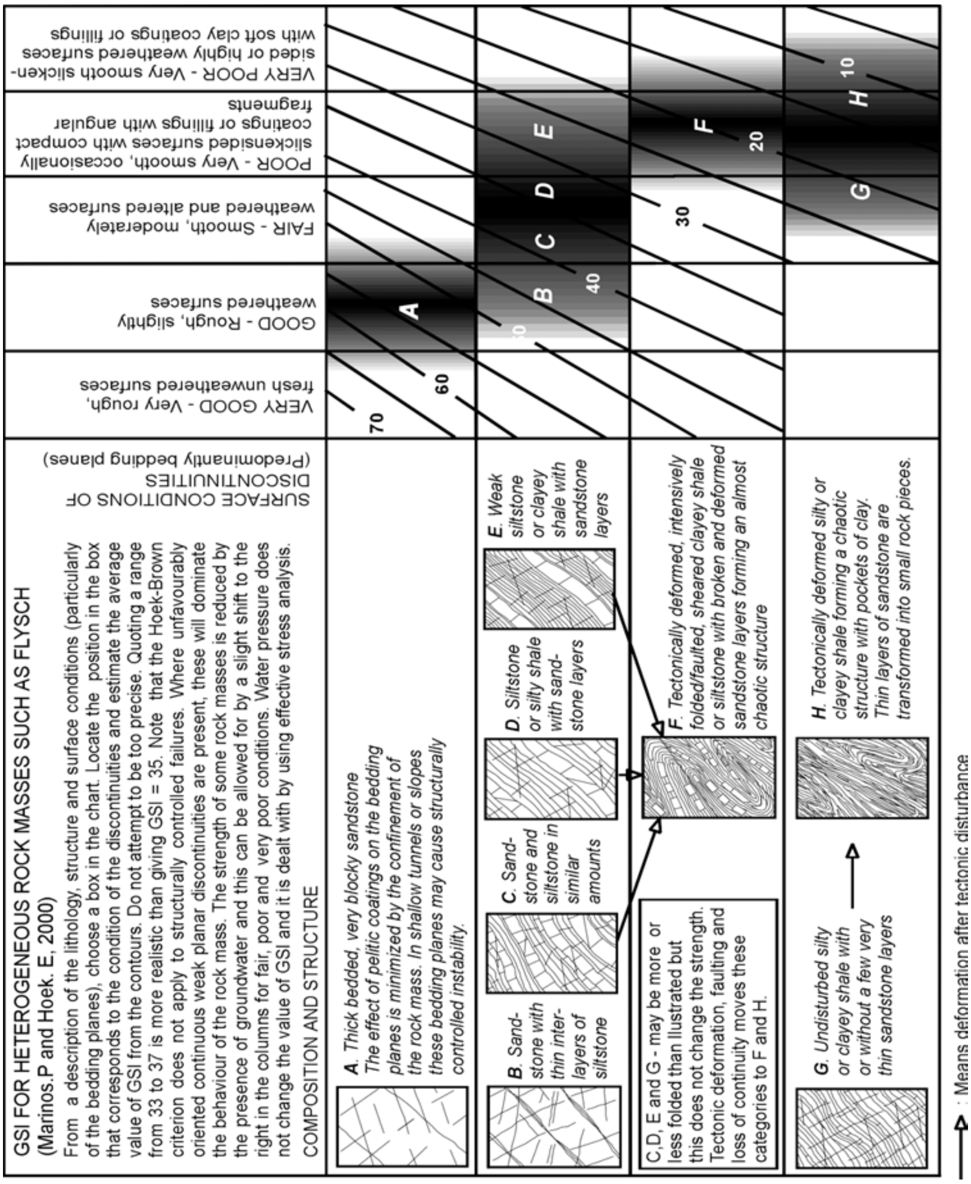

Fig. 2 Geological Strength Index estimates for heterogeneous rock masses such as flysch (Marinos and Hoek, 2001)

3. CONCLUSIONS

The final conclusion of a key note presentation in the "Skempton Conference" in 2004, in London, given by R. Chandler, M. de Freitas and P. Marinos can be reproduced here: Rock mass characterization appears to hold the way forward for engineering geology to extend its usefulness, not only so as to define a conceptual model of the site geology, from which much of great value to design can be gained, but also to the quantification needed for analyses "to ensure that the idealisation (for modelling) does not misinterpret actuality" (Knill, 2003). If such work is completed in conjunction with numerical modeling, it presents the prospect of understanding far better the reasons for rock mass behaviour (Chandler, de Freitas and Marinos 2004).

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