Review: GSI and Hoek-Brown Procedure

The presence of geological structures within a rock mass (joints, shears, etc.), requires that consideration be given to the combined influence of intact rock blocks and discontinuities when calculating the rock mass' response to tunnelling. This is in contrast to most testing carried out during geotechnical investigations, which is usually restricted to laboratory testing of intact rock samples. In-situ tests are often prohibitively expensive and associated with their own issues of reliability, repeatability and scale. This has led to the development of a number of systems that link rock mass properties to observations of the rock mass characteristics. Among these, the Geological Strength Index (GSI) coupled with the Hoek-Brown failure criterion has become one of the industry standards for estimating rock mass properties on international tunnelling projects.

The Hoek-Brown failure criterion is an empirical formulation for estimating the confinement-strength relationship of a rock mass. It's non-linear form distinguishes it from the linear Mohr-Coulomb failure criterion (Fig. 1a). The criterion was originally conceived based on experiences with brittle failure in hard rock and developed to assume that rock mass failure was controlled by jointing but with no preferred failure directions (Hoek & Brown 1980); i.e. the rock mass responds as an equivalent continuum. Later revisions saw the Hoek-Brown failure criterion coupled with Bieniawski's Rock Mass Rating (RMR) system as a means to scale laboratory intact rock properties to those for the jointed rock mass (Hoek & Brown 1988), and improvements to better account for poorer quality rock masses (Hoek et al. 1992). Further experience with the latter found that it was difficult to apply RMR to very poor quality rock masses. This led to the introduction of the Geological Strength Index (GSI) as a characterization system based more heavily on fundamental geological observations and less on 'numbers' (Hoek et al. 1995). The most up-to-date version, Hoek et al. (2002), represents a major re-examination of the entire Hoek-Brown criterion and includes new derivations for the relationships between the different input parameters and GSI.

The generalized form of the non-linear Hoek-Brown failure criterion is:

$$\sigma_1' = \sigma_3' + \sigma_{ucs} \left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s \right)^a \tag{1}$$

where σ_1' and σ_3' are the major and minor effective principal stresses at failure, σ_{ci} is the uniaxial compressive strength of the intact rock, and m_b, *s* and *a* are material constants for the rock mass. These constants are determined for the rock mass using GSI as per Hoek et al. (2002):

$$m_b = m_i \, exp\left(\frac{GSI-100}{28-14D}\right) \tag{2}$$

$$s = exp\left(\frac{GSI-100}{9-3D}\right) \tag{3}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \tag{4}$$

From above, m_i is a curve fitting parameter derived from triaxial testing of intact rock. The parameter m_b is therefore a reduced value of the intact rock value m_i, which accounts for the strength reducing effects of the rock mass conditions defined by GSI. Strength reduction for the parameters *s* and *a* follow accordingly (Fig. 1b). *D* is a disturbance factor that can account for blast damage and stress relaxation, with values ranging from 0 for undisturbed conditions to 1 for very disturbed rock masses.

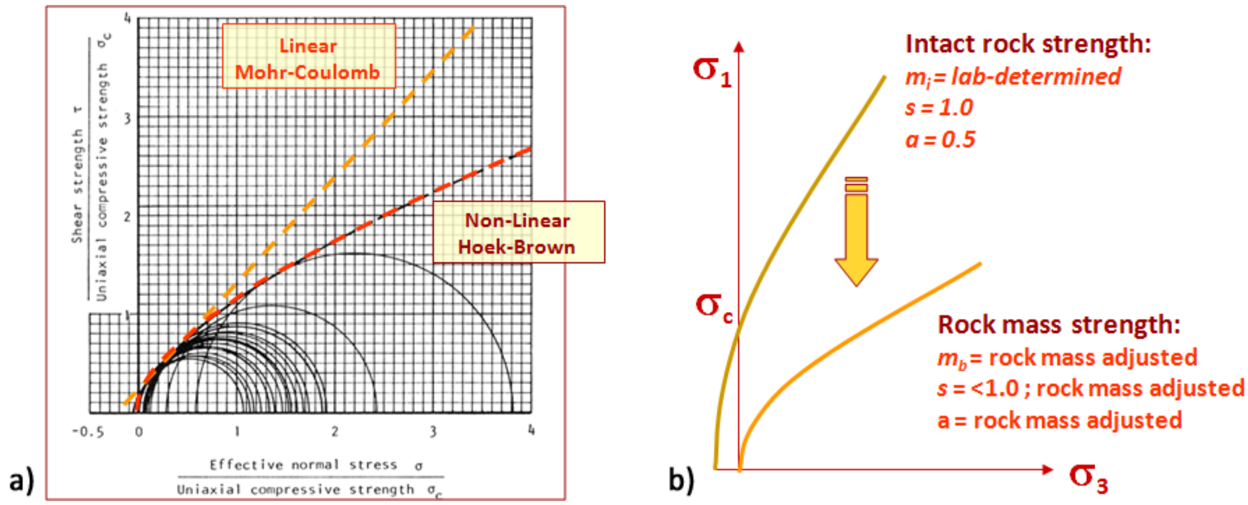

Figure 1. a) Comparison of linear Mohr-Coulomb and non-linear Hoek Brown failure envelopes plotted against triaxial test data for intact rock; b) Scaling of Hoek-Brown failure envelope for intact rock to that for rock mass strength.

GSI is estimated in the field from the chart of Marinos et al. (2005); see Figure 2. In the absence of GSI values or as a secondary check, GSI can be converted from RMR_{89} values (Bieniawski 1989), as was the practice in early applications of the GSI system using the relationship:

$$GSI = RMR'_{89} - 5 \quad \textit{(for } RMR'_{89} > 23\textit{)} \tag{5}$$

where RMR'_{89} is a modified version of RMR_{89} in which the groundwater rating is set to 15 and the adjustment for joint orientation is set to zero. These adjustments avoid double counting the effects of groundwater (an effective stress parameter in the numerical analysis) and joint orientation (treated as a specific input for structural analysis) when deriving rock mass properties to be used in numerical analyses. For very poor quality rock masses (RMR'_{89}< 23), the conversion was found to be unreliable and an alternative relationship using Barton et al.'s (1974) Q parameter was recommended instead:

$$GSI = 9Log_eQ' + 44 \quad \textit{(for } RMR'_{89} < 23\textit{)} \tag{6}$$

Similar to the adjustment for RMR'_{89}, Q' represents the modified Q value where the J_w/SRF quotient is dropped from the calculation (the influences of groundwater and in-situ stresses being explicitly accounted for in the effective stress numerical analyses being employed).

Since Eqns. (5) and (6) were first published, Hoek (2007) has found that these correlations have proven to be unreliable and he now recommends that GSI should be estimated directly by means of the charts published on its use (Fig. 2). Experience has shown that most geologists and engineering geologists are comfortable with the descriptive and largely qualitative nature of the GSI tables and have little difficulty in arriving at an estimated value.

It should also be emphasized that the GSI system is not a replacement for the RMR or Q-systems as it has no rock mass reinforcement or support design capability - its only function is the estimation of rock mass properties (Marinos et al. 2005). Furthermore, Hoek et al. (2005) emphasize that the GSI was not developed for the purpose of specifying anticipated or changing tunnelling conditions and strongly oppose its use for this purpose. Thus, as applied in the Preliminary Design report, it is recommended that RMR_{89} be used as a framework for classifying and communicating changing ground conditions and changing support requirements during tunnel construction.

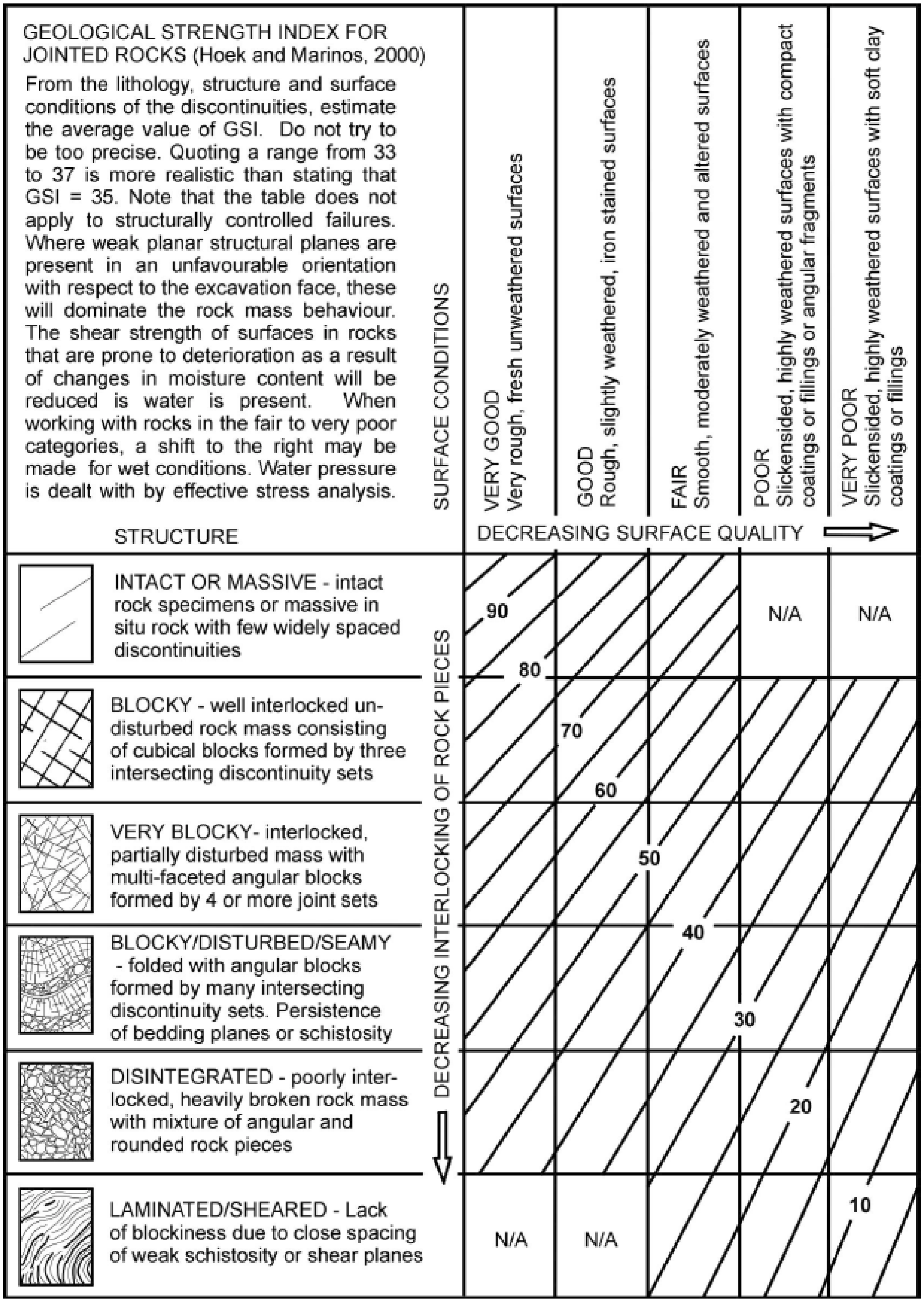

Figure 2. Geological Strength Index (GSI) lookup chart for jointed rock masses (after Marinos & Hoek 2000).

A further consideration is that most geotechnical design calculations are written for the Mohr-Coulomb failure criterion, it is often necessary to calculate equivalent rock mass friction angles and cohesive strengths from the Hoek-Brown parameters. Moreover, most practitioners have more experience, and therefore an intuitive feeling for the physical meanings of cohesion and friction, which is not the case for m_b, s and a. In terms of equivalencies, the parameter m_b is related to the frictional strength of the rock mass, and s, which is a measure of how fractured the rock mass is, is related to the rock mass cohesion. These are only descriptive relationships, however. Where Mohr-Coulomb parameters are required, the fitting of the linear Mohr-Coulomb envelope to the non-linear Hoek-Brown envelope results in the following equations for friction ϕ' and cohesive strength c':

$$\phi' = sin^{-1}\left[\frac{6am_b\,(s+m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b\,(s+m_b\sigma'_{3n})^{a-1}}\right] \tag{7}$$

$$c' = \frac{\sigma_{ci}\,[(1+2a)s+(1-a)m_b\sigma'_{3n}](s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1+\left(\frac{6am_b(s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)}\right)}} \tag{8}$$

where $\sigma'_{3n} = \sigma'_{3max}/\sigma_{ci}$. Note that the value of σ'_{3max} represents the upper limit of confining stress over which the relationship between the Hoek-Brown and Mohr-Coulomb failure envelopes is considered. For deep tunnels, σ'_{3max} can be calculated from the empirical relationship (Hoek et al. 2002):

$$\frac{\sigma'_{3max}}{\sigma'_{cm}} = 0.47\left(\frac{\sigma'_{cm}}{\gamma H}\right)^{-0.94} \tag{9}$$

where γ is the unit weight of the rock mass, H is the depth of the tunnel below surface, and σ'_{cm} is the "global" rock mass strength for the stress range $\sigma_t < \sigma'_3 < \sigma_{ci}$ given by:

$$\sigma'_{cm} = \sigma_{ci}\left[\frac{[m_b+4s-a(m_b-8s)]\left(\frac{m_b}{4+s}\right)^{a-1}}{2(1+a)(2+a)}\right] \tag{10}$$

In cases where the horizontal stress is higher than the vertical stress, the horizontal stress value should be used in place of γH in Eqn. (9).

Software

The procedures for determining the Hoek-Brown and equivalent Mohr-Coulomb parameters are incorporated in the Rocscience software package ROCDATA, which is available in the EOAS computing labs. These relationships can also be easily implemented through an Excel spreadsheet.

In carrying out these calculations, it must be emphasized that the quantitative conversion of Hoek-Brown to Mohr-Coulomb parameters is done by fitting an average linear relationship to the non-linear Hoek-Brown envelope for a range of minor principal stress values defined by $\sigma_t < \sigma_3 < \sigma'_{3max}$ (Hoek et al. 2002). Note that the value of σ'_{3max}, the upper limit of confining stress over which the relationship between the Hoek-Brown and Mohr-Coulomb criteria is considered, has to be determined for each individual case (Fig. 3).

Where geotechnical design software accepts Hoek-Brown input directly, it is preferable to use this input rather than estimates of the Mohr Coulomb parameters c and ϕ extrapolated from the non-linear Hoek-Brown failure envelope. This eliminates the uncertainty associated with the conversion that is demonstrated in Figure 3.

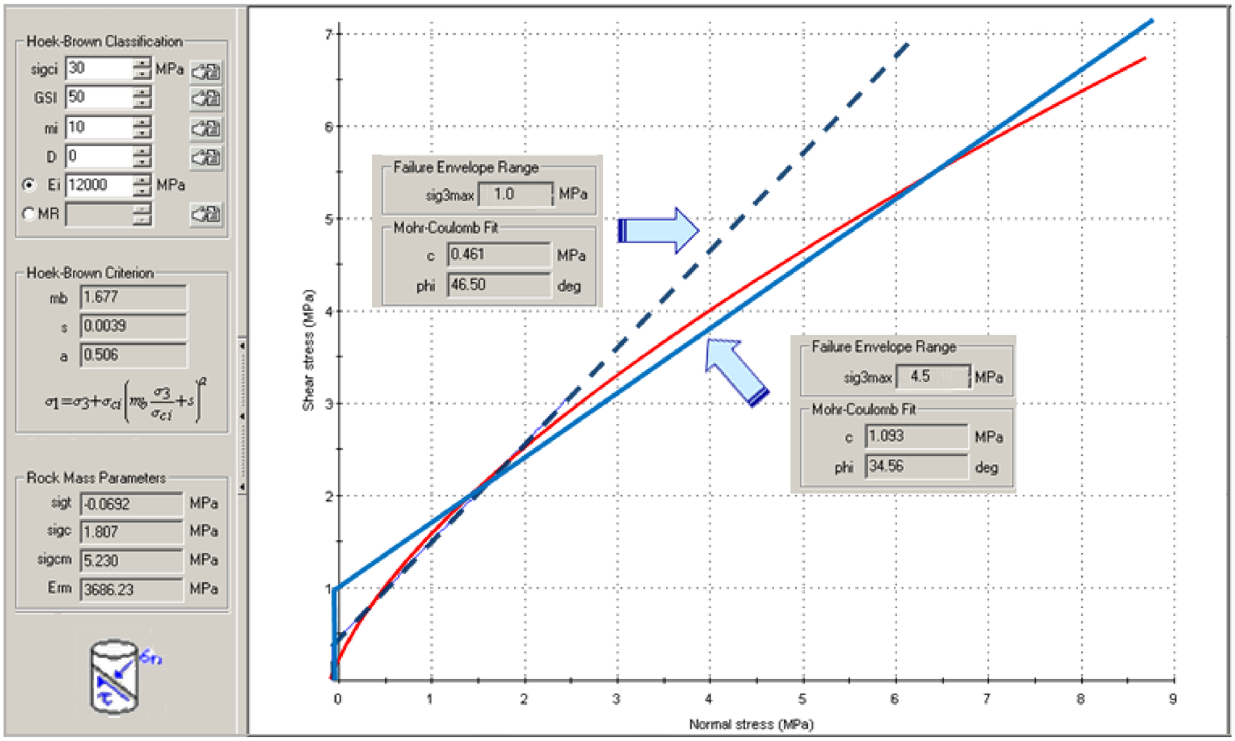

Figure 3. Fitting of linear Mohr-Coulomb failure envelopes along two different stress ranges of a non-linear Hoek-Brown failure envelope. Note the change in cohesion and friction angle values for the two different stress ranges specified.

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