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ORIGINAL PAPER

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The geological strength index: applications and limitations

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Abstract The geological strength index (GSI) is a system of rock-mass characterization that has been developed in engineering rock mechanics to meet the need for reliable input data, particularly those related to rock-mass properties required as inputs into numerical analysis or closed form solutions for designing tunnels, slopes or foundations in rocks. The geological character of rock material, together with the visual assessment of the mass it forms, is used as a direct input to the selection of parameters relevant for the prediction of rock-mass strength and deformability. This approach enables a rock mass to be considered as a mechanical continuum without losing the influence geology has on its mechanical properties. It also provides a field method for characterizing difficult-to-describe rock masses. After a decade of application of the GSI and its variations in quantitative characterization of rock mass, this paper attempts to answer questions that have been raised by the users about the appropriate selection of the index for a range of rock masses under various conditions. Recommendations on the use of GSI are given and, in addition, cases where the GSI is not applicable are discussed. More particularly, a discussion and suggestions are presented on issues such as the size of the rock mass to be considered, its anisotropy, the influence of great depth, the presence of ground water, the aperture and the infilling of discontinuities and the properties of weathered rock masses and soft rocks.

Résumé Le Geological Strength Index (GSI) est un système de classification des massifs rocheux développé en mécanique des roches. Il permet d'obtenir les données relatives aux propriétés de masses rocheuses, données nécessaires pour des simulations numériques ou permettant le dimensionnement d'ouvrages:tunnels, pentes ou fondations rocheuses. Les caractéristiques géologiques de la matrice rocheuse ainsi que celles relatives à la structure du massif correspondant sont directement utilisées pour obtenir les paramètres appropriés relatifs à la déformabilité et la résistance de la masse rocheuse. Cette approche permet de considérer une masse rocheuse comme un milieu continu, le rôle des caractéristiques géologiques sur les propriétés mécaniques n'étant pas oblitéré. Elle apporte aussi une méthode de terrain pour caractériser des masses rocheuses difficiles à décrire. Après une décennie d'application du Geological Strength Index et de ses variantes pour caractériser des masses rocheuses, cet article tente de répondre aux questions formulées par les utilisateurs concernant le choix le plus approprié de cet index pour une large gamme de massifs rocheux.

Des recommandations quant à l'usage du GSI sont données et, de plus, des cas où le GSI n'est pas applicable sont discutés. Plus particulièrement, des suggestions sont apportées sur des questions relatives à la taille de masse rocheuse à considérer, son anisotropie, l»influence des grandes profondeurs, la présence d'eau, l'ouverture et le remplissage des discontinuités ainsi que les propriétés des masses rocheuses altérées et des roches tendres.

Keywords Geological Strength Index · Rock mass · Geological structure · Mechanical properties · Selection of the GSI

Mots clés Geological Strength Index · Massif rocheux · Structure géologique · Propriétés mécaniques · Conditions d»utilisation du GSI

Introduction

Design in rock masses

A few decades ago, the tools for designing tunnels started to change. Although still crude, numerical methods were being developed that offered the promise for much more detailed analysis of difficult underground excavation problems which, in a number of cases, fall outside the ideal range of application of the tunnel reinforcement classifications such as the RMR system introduced by Bieniawski (1973) and the Q system published by Barton et al. (1974) both furthermore expanded in the following years. There is absolutely no problem with the concept of these classifications and there are hundreds of kilometres of tunnels that have been successfully constructed on the basis of their application. However, this approach is ideally suited to situations in which the rock mass behaviour is relatively simple, for example for RMR values between about 30–70 and moderate stress levels. In other words, sliding and rotation of intact rock pieces essentially control the failure process. These approaches are less reliable for squeezing, swelling, clearly defined structural failures or spalling, slabbing and rock-bursting under very high stress conditions. More importantly, these classification systems are of little help in providing information for the design of sequentially installed temporary reinforcement and the support required to control progressive failure in difficult tunnelling conditions.

Numerical tools available today allow the tunnel designer to analyse these progressive failure processes and the sequentially installed reinforcement and support necessary to maintain the stability of the advancing tunnel until the final reinforcing or supporting structure can be installed. However, these numerical tools require reliable input information on the strength and deformation characteristics of the rock mass surrounding the tunnel. As it is practically impossible to determine this information by direct in situ testing (except for back-analysis of already constructed tunnels) there was a need for some method for estimating the rock-mass properties from the intact rock properties and the characteristics of the discontinuities in the rock mass. This resulted in the development of the rock-mass failure criterion by Hoek and Brown (1980).

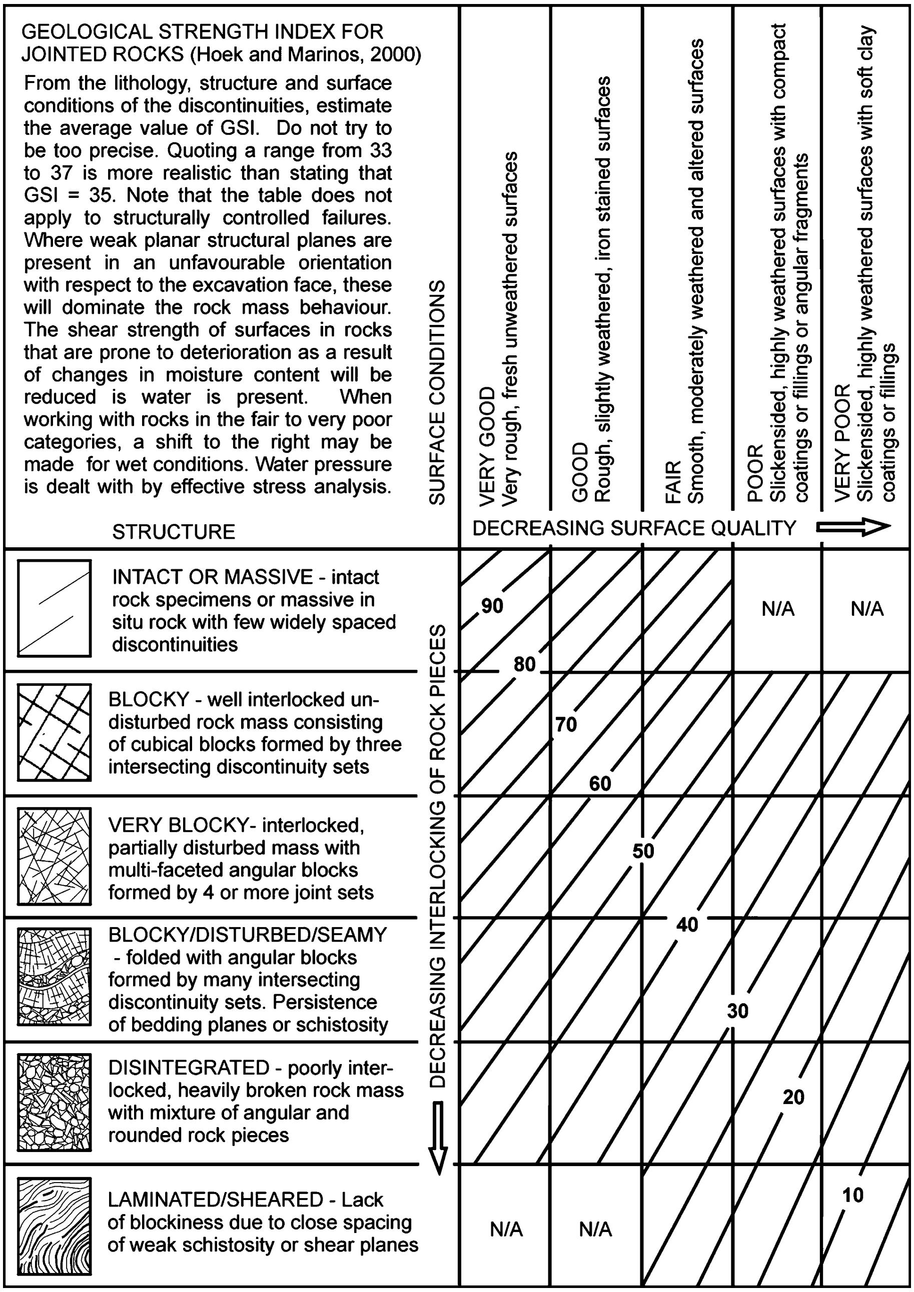
The Geological Strength Index (GSI): development history

Hoek and Brown recognized that a rock-mass failure criterion would have no practical value unless it could be related to geological observations that could be made quickly and easily by an engineering geologist or geologist in the field. They considered developing a new classification system during the evolution of the criterion in the late 1970s but they soon gave up the idea and settled for the already published RMR system. It was appreciated that the RMR system (and the Q system) were developed for the estimation of underground excavation and support, and that they included parameters that are not required for the estimation of rock-mass properties. The groundwater and structural orientation parameters in RMR and the groundwater and stress parameters in Q are dealt with explicitly in effective stress numerical analyses and the incorporation of these parameters into the rock-mass property estimate results is inappropriate. Hence, it was recommended that only the first four parameters of the RMR system (intact rock strength, RQD rating, joint spacing and joint conditions) should be used for the estimation of rock-mass properties, if this system had to be used.

In the early days the use of the RMR classification (modified as described above) worked well because most of the problems were in reasonable quality rock masses ($30 < \mathrm{RMR} < 70$) under moderate stress conditions. However, it soon became obvious that the RMR system was difficult to apply to rock masses that are of very poor quality. The relationship between RMR and the constants m and s of the Hoek–Brown failure criterion begins to break down for severely fractured and weak rock masses.

Both the RMR and the Q classifications include and are heavily dependent upon the RQD classification introduced by Deere (1964). Since RQD in most of the weak rock masses is essentially zero or meaningless, it became necessary to consider an alternative classification system. The required system would not include RQD, would place greater emphasis on basic geological observations of rock-mass characteristics, reflect the material, its structure and its geological history and would be developed specifically for the estimation of rock mass properties rather than for tunnel reinforcement and support. This new classification, now called GSI, started life in Toronto with engineering geology input from David Wood (Hoek et al. 1992). The index and its use for the Hoek and Brown failure criterion was further developed by Hoek (1994), Hoek et al. (1995) and Hoek and Brown (1997) but it was still a hard rock system roughly equivalent to RMR. Since 1998, Evert Hoek and Paul Marinos, dealing with incredibly difficult materials encountered in tunnelling in Greece, developed the GSI system to the present form to include poor quality rock masses (Fig. 1) (Hoek et al. 1998; Marinos and Hoek 2000, 2001). They also extended its application for heterogeneous rock masses as shown in Fig. 2 (Marinos and Hoek 2001).

Fig. 1 General chart for GSI estimates from the geological observations



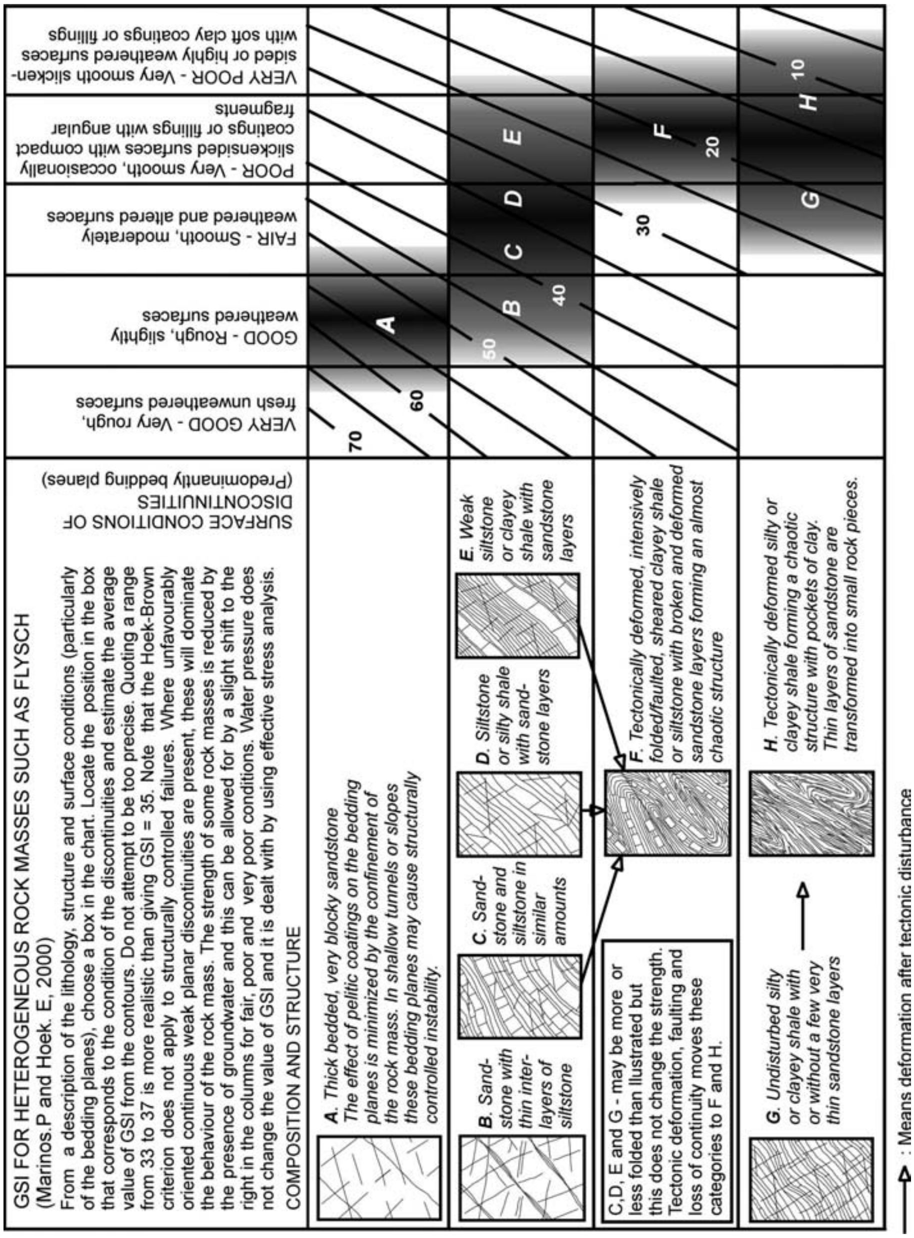


Fig. 2 Geological strength index estimates for heterogeneous rock masses such as Flysch

Functions of the Geological Strength Index

The heart of the GSI classification is a careful engineering geology description of the rock mass which is essentially qualitative, because it was felt that the numbers associated with RMR and Q-systems were largely meaningless for the weak and heterogeneous rock masses. Note that the GSI system was never intended as a replacement for RMR or Q as it has no rock-mass reinforcement or support design capability—its only function is the estimation of rock-mass properties.

This index is based upon an assessment of the lithology, structure and condition of discontinuity surfaces in the rock mass and it is estimated from visual examination of the rock mass exposed in outcrops, in surface excavations such as road cuts and in tunnel faces and borehole cores. The GSI, by combining the two fundamental parameters of the geological process, the blockiness of the mass and the conditions of discontinuities, respects the main geological constraints that govern a formation and is thus a geologically sound index that is simple to apply in the field.

Once a GSI ''number'' has been decided upon, this number is entered into a set of empirically developed equations to estimate the rock-mass properties which can then be used as input into some form of numerical analysis or closed-form solution. The index is used in conjunction with appropriate values for the unconfined compressive strength of the intact rock σ_{ci} and the petrographic constant m_i, to calculate the mechanical properties of a rock mass, in particular the compressive strength of the rock mass (σ_{cm}) and its deformation modulus (E). Updated values of m_i, can be found in Marinos and Hoek (2000) or in the RocLab program. Basic procedures are explained in Hoek and Brown (1997) but a more recent refinement of the empirical equations and the relation between the Hoek–Brown and the Mohr–Coulomb criteria have been addressed by Hoek et al. (2002) for appropriate ranges of stress encountered in tunnels and slopes. This paper and the associated program RocLab can be downloaded from http://www.rocscience.com.

Note that attempts to "quantify" the GSI classification to satisfy the perception that "engineers are happier with numbers" (Cai et al. 2004; Sonmez and Ulusay 1999) are interesting but have to be applied with caution. The quantification processes used are related to the frequency and orientation of discontinuities and are limited to rock masses in which these numbers can easily be measured. The quantifications do not work well in tectonically disturbed rock masses in which the structural fabric has been destroyed. In such rock masses the authors recommend the use of the original qualitative approach based on careful visual observations.

Suggestions for using GSI

After a decade of application of the GSI and its variations for the characterization of the rock mass, this paper attempts to answer questions that have been raised by users about the appropriate selection of the index for various rock masses under various conditions.

When not to use GSI

The GSI classification system is based upon the assumption that the rock mass contains a sufficient number of "randomly" oriented discontinuities such that it behaves as an isotropic mass. In other words, the behaviour of the rock mass is independent of the direction of the applied loads. Therefore, it is clear that the GSI system should not be applied to those rock masses in which there is a clearly defined dominant structural orientation. Undisturbed slate is an example of a rock mass in which the mechanical behaviour is highly anisotropic and which should not be assigned a GSI value based upon the charts presented in Figs. 1, 2. However, the Hoek–Brown criterion and the GSI chart can be applied with caution if the failure of such rock masses is not controlled by their anisotropy (e.g. in the case of a slope when the dominant structural discontinuity set dips into the slope and failure may occur through the rock mass). For rock masses with a structure such as that shown in the sixth (last) row of the GSI chart (Fig. 1), anisotropy is not a major issue as the difference in the strength of the rock and that of the discontinuities within it is small.

It is also inappropriate to assign GSI values to excavated faces in strong hard rock with a few discontinuities spaced at distances of similar magnitude to the dimensions of the tunnel or slope under consideration. In such cases the stability of the tunnel or slope will be controlled by the three-dimensional geometry of the intersecting discontinuities and the free faces created by the excavation. Obviously, the GSI classification does not apply to such cases.

Geological description in the GSI chart

In dealing with specific rock masses it is suggested that the selection of the appropriate case in the GSI chart should not be limited to the visual similarity with the sketches of the structure of the rock mass as they appear in the charts. The associated descriptions must also be read carefully, so that the most suitable structure is chosen. The most appropriate case may well lie at some intermediate point between the limited number of sketches or descriptions included in the charts.

Projection of GSI values into the ground

Outcrops, excavated slopes tunnel faces and borehole cores are the most common sources of information for the estimation of the GSI value of a rock mass. How should the numbers estimated from these sources be projected or extrapolated into the rock mass behind a slope or ahead of a tunnel?

Outcrops are an extremely valuable source of data in the initial stages of a project but they suffer from the disadvantage that surface relaxation, weathering and/or alteration may have significantly influenced the appearance of the rock-mass components. This disadvantage can be overcome (where permissible) by trial trenches but, unless these are machine excavated to considerable depth, there is no guarantee that the effects of deep weathering will have been eliminated. Judgement is therefore required in order to allow for these weathering and alteration effects in assessing the most probable GSI value at the depth of the proposed excavation.

Excavated slope and tunnel faces are probably the most reliable source of information for GSI estimates provided that these faces are reasonably close to and in the same rock mass as the structure under investigation. In hard strong rock masses it is important that an appropriate allowance be made for damage due to mechanical excavation or blasting. As the purpose of estimating GSI is to assign properties to the undisturbed rock mass in which a tunnel or slope is to be excavated, failure to allow for the effects of blast damage when assessing GSI will result in the assignment of values that are too conservative. Therefore, if borehole data are absent, it is important that the engineering geologist or geologist attempts to "look behind" the surface damage and try to assign the GSI value on the basis of the inherent structures in the rock mass. This problem becomes less significant in weak and tectonically disturbed rock masses as excavation is generally carried out by "gentle" mechanical means and the amount of surface damage is negligible compared to that which already exists in the rock mass.

Borehole cores are the best source of data at depth, but it has to be recognized that it is necessary to extrapolate the one-dimensional information provided by the core to the three-dimensional in situ rock mass. However, this is a problem common to all borehole investigations, and most experienced engineering geologists are comfortable with this extrapolation process. Multiple boreholes and inclined boreholes can be of great help in the interpretation of rock-mass characteristics at depth.

For stability analysis of a slope, the evaluation is based on the rock mass through which it is anticipated that a potential failure plane could pass. The estimation of GSI values in these cases requires considerable judgment, particularly when the failure plane can pass through several zones of different quality. Mean values may not be appropriate in this case.

For tunnels, the index should be assessed for the volume of rock involved in carrying loads, e.g. for about one diameter around the tunnel in the case of tunnel behaviour or more locally in the case of a structure such as an elephant foot.

For particularly sensitive or critical structures, such as underground powerhouse caverns, the information obtained from the sources discussed above may not be considered adequate, particularly as the design advances beyond the preliminary stages. In these cases, the use of small exploration tunnels can be considered and this method of data gathering will often be found to be highly cost effective.

Figure 3 provides a visual summary of some of the adjustments discussed in the previous paragraphs. When direct assessment of depth conditions is not available, upward adjustment of the GSI value to allow for the effects of surface disturbance, weathering and alteration are indicated in the upper (white) part of the GSI chart. Obviously, the magnitude of the shift will vary from case to case and will depend upon the judgement and experience of the observer. In the lower (shaded) part of the chart, adjustments are not normally required as the rock mass is already disintegrated or sheared and this damage persists with depth.

Anisotropy

As discussed above, the Hoek–Brown criterion (and other similar criteria) requires that the rock mass behave isotropically and that failure does not follow a preferential direction imposed by the orientation of a specific discontinuity or a combination of two or three discontinuities. In these cases, the use of GSI is meaningless as the failure is governed by the shear strength of these discontinuities and not of the rock mass. Cases, however, where the criterion and the GSI chart can reasonably be used were discussed above.

However, in a numerical analysis involving a single well-defined discontinuity such as a shear zone or fault, it is sometimes appropriate to apply the Hoek–Brown criterion to the overall rock mass and to superimpose the discontinuity as a significantly weaker element. In this case, the GSI value assigned to the rock mass should ignore the single major discontinuity. The properties of this discontinuity may fit the lower portion of the GSI chart or they may require a different approach such as laboratory shear testing of soft clay fillings.

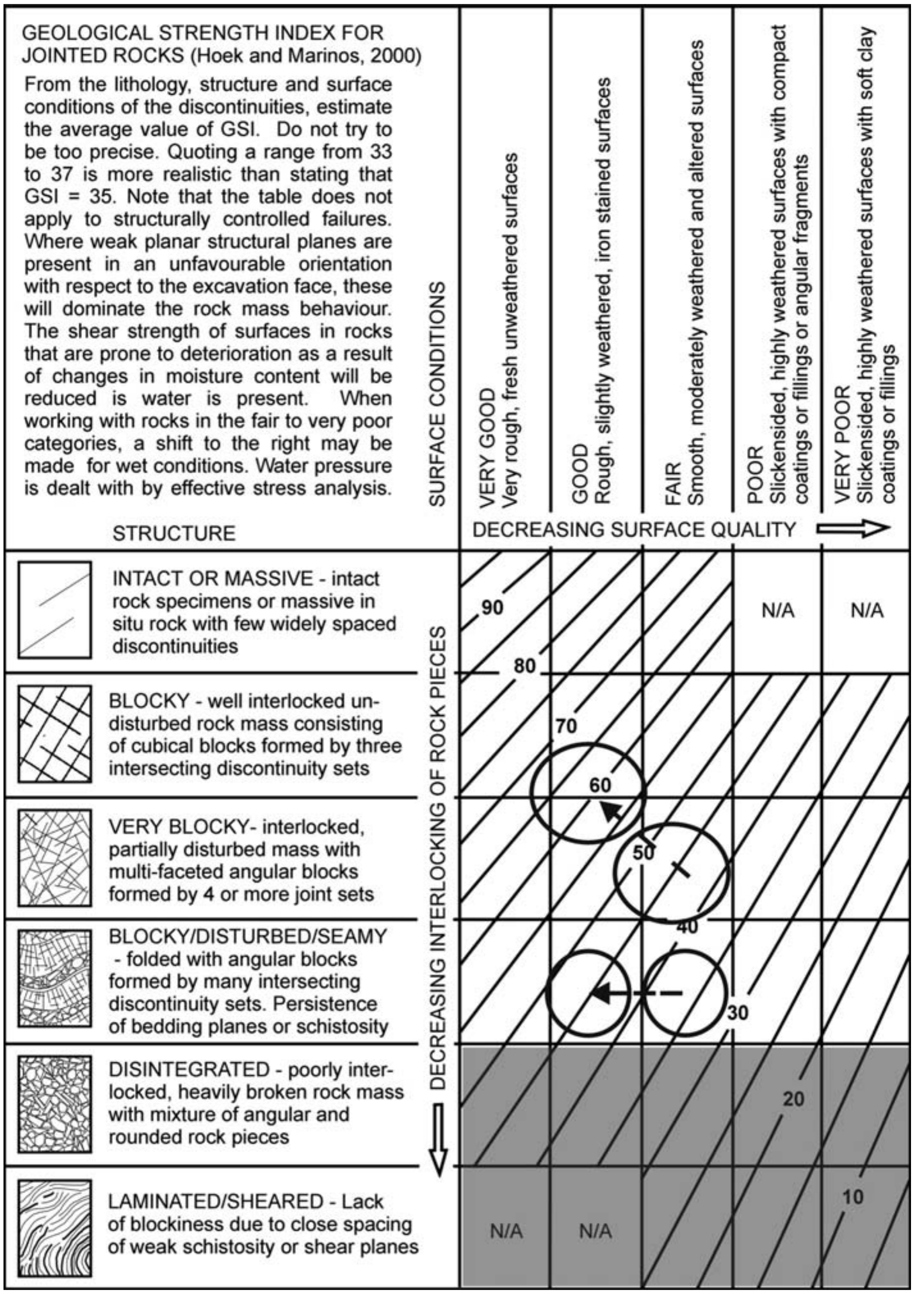
Aperture of discontinuities

The strength and deformation characteristics of a rock mass are dependent upon the interlocking of the individual pieces of intact rock that make up the mass. Obviously, the aperture of the discontinuities that separate these individual pieces has an important influence upon the rock-mass properties.

There is no specific reference to the aperture of the discontinuities in the GSI charts but a "disturbance factor" D has been provided in the most recent version of the Hoek–Brown failure criterion (Hoek et al. 2002). This factor ranges from $D=0$ for undisturbed rock masses, such as those excavated by a tunnel boring machine, to $D=1$ for extremely disturbed rock masses such as open pit mine slopes that have been subjected to very heavy production blasting. The factor allows for the disruption of the interlocking of the individual rock pieces as a result of opening of the discontinuities.

The incorporation of the disturbance factor D into the empirical equations used to estimate the rock-mass strength and deformation characteristics is based upon back-analysis of excavated tunnels and slopes. At this stage (2004) there is relatively little experience in the use of this factor, and it may be necessary to adjust its participation in the equations as more field evidence is accumulated. However, the limited experience that is available suggests that this factor does provide a

Fig. 3 Suggested projection of information from observations in outcrops to depth. *White area*: a shifting to the left or to the left and upwards is recommended; the extent of the shift shown in the chart is indicative and should be based on geological judgement. *Shadowed area*: shifting is less or not applicable as poor quality is retained in depth in brecciated, mylonitized or shear zones



reasonable estimate of the influence of damage due to stress relaxation or blasting of excavated rock faces.

Note that this damage decreases with depth into the rock mass and, in numerical modelling, it is generally appropriate to simulate this decrease by dividing the rock mass into a number of zones with decreasing values of D being applied to successive zones as the distance from the face increases. In one example, which involved the construction of a large underground powerhouse cavern in interbedded sandstones and siltstones, it was found that the blast damaged zone was surrounding each excavation perimeter to a depth of about 2 m (Cheng and Liu 1990). Carefully controlled blasting was used in this cavern excavation and the limited extent of the blast damage can be considered typical of that for civil engineering tunnels excavated by drill and blast methods. On the other hand, in very large open pit mine slopes in which blasts can involve many tons of explosives, blast damage has been observed up to 100 m or more behind the excavated slope face. Hoek and Karzulovic (2000) have given some guidance on the extent of this damage and its impact on rock mass properties.

Geological Strength Index at great depth

In hard rock, great depth (e.g. 1,000 m or more) the rock-mass structure is so tight that the mass behaviour approaches that of the intact rock. In this case, the GSI value approaches 100 and the application of the GSI system is no longer meaningful.

The failure process that controls the stability of underground excavations under these conditions is dominated by brittle fracture initiation and propagation, which leads to spalling, slabbing and, in extreme cases, rock-bursts. Considerable research effort has been devoted to the study of these brittle fracture processes and a recent paper by Diederichs et al. (2004) provides a useful summary of this work. Cundall et al. (2003) have introduced a set of post-failure flow rules for numerical modelling which cover the transition from tensile to shear fracture that occurs during the process of brittle fracture propagation around highly stressed excavations in hard rock masses.

When tectonic disturbance is important and persists with depth, these comments do not apply and the GSI charts may be applicable, but should be used with caution.

Discontinuities with filling materials

The GSI charts can be used to estimate the characteristics of rock-masses with discontinuities with filling materials using the descriptions in the columns of poor or very poor condition of discontinuities. If the filling material is systematic and thick (e.g. more than few cm) or shear zones are present with clayey material then the use of the GSI chart for heterogeneous rock masses (Fig. 2) is recommended.

The influence of water

The shear strength of the rock mass is reduced by the presence of water in the discontinuities or the filling materials when these are prone to deterioration as a result of changes in moisture content. This is particularly valid in the fair to very poor categories of discontinuities where a shift to the right may be made for wet conditions (Fig. 4).

Water pressure is dealt with by effective stress analysis in design and it is independent of the GSI characterization of the rock mass.

Weathered rock masses

The GSI values for weathered rock masses are shifted to the right of those of the same rock masses when these are unweathered. If the weathering has penetrated into the intact rock pieces that make up the mass (e.g. in weathered granites) then the constant m_i and the unconfined strength of the σ_{ci} of the Hoek and Brown criterion must also be reduced. If the weathering has penetrated the rock to the extent that the discontinuities and the structure have been lost, then the rock mass must be assessed as a soil and the GSI system no longer applies.

Heterogeneous and lithologically varied sedimentary rock masses

The GSI has recently been extended to accommodate some of the most variable of rock masses, including extremely poor quality sheared rock masses of weak schistose materials (such as siltstones, clay shales or phyllites) sometime inter-bedded with strong rock (such as sandstones, limestones or quartzites). A GSI chart for flysch has been published in Marinos and Hoek (2001) and is reproduced in Fig. 2. For lithologically varied but tectonically undisturbed rock masses, such as the molasses, a new GSI chart is (Hoek et al. 2005).

Rocks of low strength

When rocks such as marls, claystones, siltstones and weak sandstones are developed in stable conditions or a post tectonic environment, they present a simple structure with few discontinuities. Even when bedding planes exist they do not always appear as clearly defined discontinuity surfaces.

In such cases, the use of the GSI chart for the ''blocky'' or ''massive'' rock masses (Fig. 1) is applicable. The discontinuities, although they are limited in number, cannot be better than fair (usually fair or poor) and hence the GSI values tend to be in the range of 40–60. In these cases, the low strength of the rock mass results from low values of the intact strength σ_{ci} and the constant m_i.

When these rocks form continuous masses with no discontinuities, the rock mass can be treated as intact with engineering parameters given directly by laboratory testing. In such cases the GSI classification is not applicable.

Precision of the GSI classification system

The ''qualitative'' GSI system works well for engineering geologists since it is consistent with their experience in describing rocks and rock masses during logging and mapping. In some cases, engineers tend to be uncomfortable with the system because it does not contain parameters that can be measured in order to improve the precision of the estimated GSI value.

The authors, two of whom graduated as engineers, do not share this concern as they feel that it is not meaningful to attempt to assign a precise number to the GSI

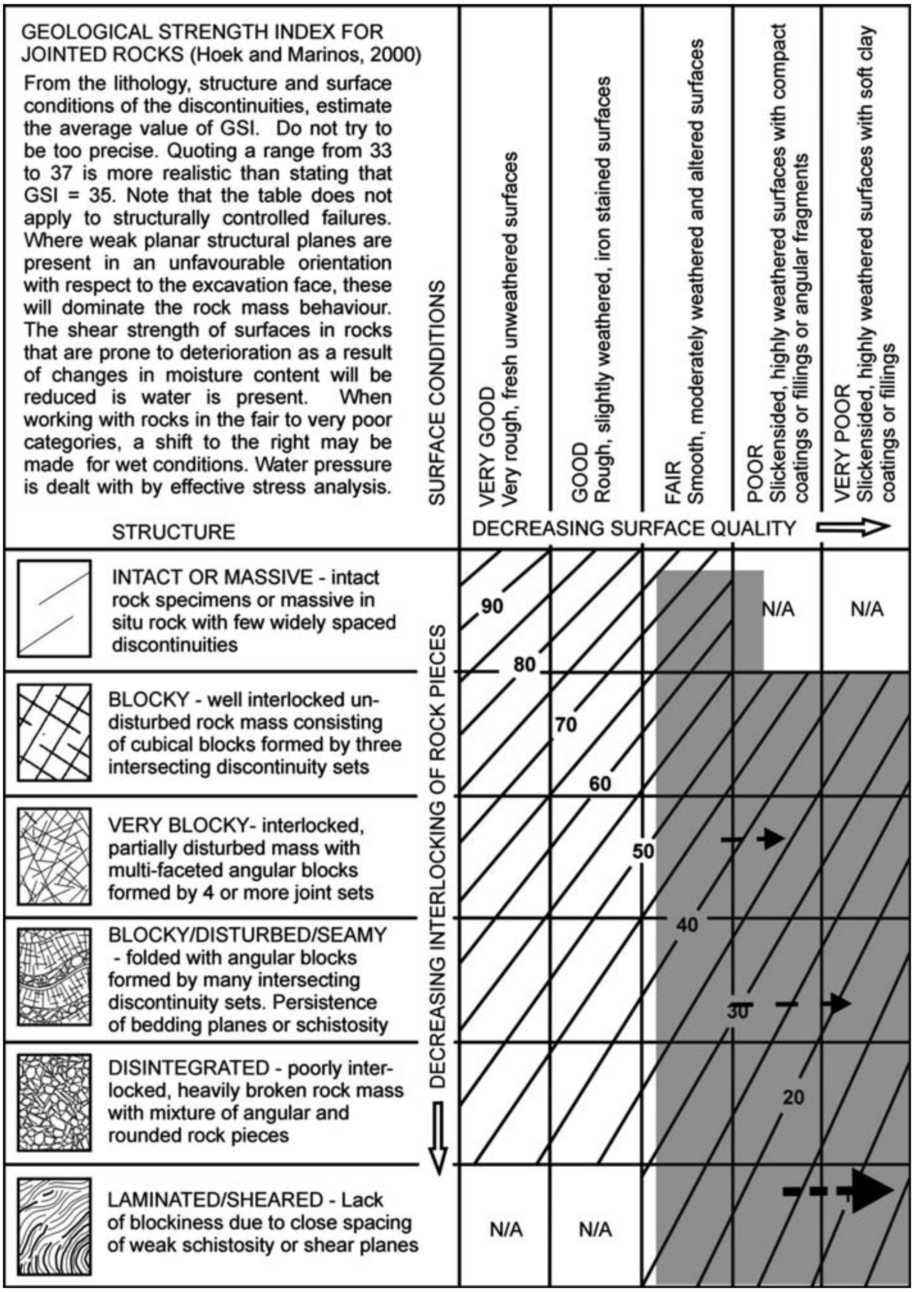


Fig. 4 In fair to very poor categories of discontinuities, a shift to the right is necessary for wet conditions as the surfaces of the discontinuities or the filling materials are usually prone to deterioration as a result of change in the moisture content. The shift to the right is more substantial in the low quality range of rock mass (*last lines and columns*)

value for a typical rock mass. In all but the very simplest of cases, GSI is best described by assigning it a range of values. For analytical purposes this range may be defined by a normal distribution with the mean and standard deviation values assigned on the basis of common sense.

In the earlier period of the GSI application it was proposed that correlation of "adjusted" RMR and Q values with GSI be used for providing the necessary input for the solution of the Hoek and Brown criterion. Although this procedure may work with the better quality rock masses, it is meaningless in the range of weak (e.g. GSI < 35), very weak and heterogeneous rock masses where these correlations are not recommended.

Estimation of intact strength σ_{ci} and the constant m_i

While this paper is concerned primarily with the GSI classification, it would not be appropriate to leave the related topic of the Hoek–Brown failure criterion without briefly mentioning the estimation of intact strength σ_{ci} and the constant m_i.

The influence of the intact rock strength σ_{ci} is at least as important as the value of GSI in the overall estimate of rock mass properties by means of the Hoek–Brown criterion. Ideally, σ_{ci} should be determined by direct laboratory testing under carefully controlled conditions. However, in many cases, this is not possible because of time or budget constraints, or because it is not possible to recover samples for laboratory testing (particularly in the case of weak, thinly schistose or tectonically disturbed rock masses where discontinuities are included in the laboratory samples). Under such circumstances, estimates of the value of σ_{ci} have to be made on the basis of published information, simple index tests or by descriptive grades such as those published by the International Society for Rock Mechanics (Brown 1981).

Experience has shown that there is a tendency to underestimate the value of the intact rock strength in many cases. This is particularly so in weak and tectonically disturbed rock masses where the characteristics of the intact rock components tend to be masked by the surrounding sheared or weathered material. These underestimations can have serious implications for engineering design and care has to be taken to ensure that realistic estimates of intact strength are made as early as possible in the project. In tunnelling, such estimates can be refined on the basis of a detailed back-analysis of the tunnel deformation and, while this may require considerable effort and even the involvement of specialists in numerical analysis, the attempt will generally be repaid many times over in the cost savings achieved by more realistic designs.

The value of the constant m_i, as for the case of the intact strength σ_{ci}, is best determined by direct laboratory testing. However, when this is not possible, an estimate based upon published values (e.g. in the program RocLab) is generally acceptable as the overall influence of the value of m_i on the rock-mass strength is significantly less than that of either GSI or σ_{ci}.

GSI and contract documents

One of the most important contractual problems in rock construction and particularly in tunnelling is the issue of "changed ground conditions". There are invariably arguments between the owner and the contractor on the nature of the ground specified in the contract and that actually encountered during construction. In order to overcome this problem there has been a tendency to specify the anticipated conditions in terms of the RMR or Q tunnelling classifications. More recently some contracts have used the GSI classification for this purpose, and the authors are strongly opposed to this trend. As discussed earlier in this paper, RMR and Q were developed for the purposes of estimating tunnel reinforcement or support whereas GSI was developed solely for the purpose of estimating rock-mass strength. Therefore, GSI is only one element in a tunnel design process and cannot be used, on its own, to specify tunnelling conditions.

The use of any classification system to specify anticipated tunnelling conditions is always a problem as these systems are open to a variety of interpretations, depending upon the experience and level of conservatism of the observer. This can result in significant differences in RMR or Q values for a particular rock mass and, if these differences fall on either side of a major "change" point in excavation or support type, this can have important financial consequences.

The geotechnical baseline report (Essex 1997) was introduced in an attempt to overcome some of these difficulties and has attracted an increasing amount of international attention in tunnelling[1]. This report, produced by the Owner and included in the contract documents, attempts to describe the rock mass and the anticipated tunnelling conditions as accurately as possible and to provide a rational basis for contractual discussions and payment. The authors of this paper recommend that this concept should be used in place of the traditional tunnel classifications for the purpose of specifying anticipated tunnel conditions.

Conclusions

Rock-mass characterization has an important role in the future of engineering geology in extending its usefulness, not only to define a conceptual model of the site geology, but also for the quantification needed for analyses "to ensure that the idealization (for modelling) does not misinterpret actuality" (Knill 2003). If it is carried out in conjunction with numerical modelling, rock-mass characterization presents the prospect of a far better understanding of the reasons for rock-mass behaviour (Chandler et al. 2004). The GSI has considerable potential for use in rock engineering because it permits the manifold aspects of rock to be quantified thereby enhancing geological logic and reducing engineering uncertainty. Its use allows the influence of variables, which make up a rock mass, to be assessed and hence the behaviour of rock masses to be explained more clearly. One of the advantages of the index is that the geological reasoning it embodies allows adjustments of its ratings to cover a wide range of rock masses and conditions but it also allows us to understand the limits of its application.

[1]A simple search for "geotechnical baseline report" on the Internet will reveal the extent of this interest.

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