Applicability of the geological strength index (GSI) classification for very weak and sheared rock masses. The case of the Athens Schist Formation

E. Hoek · P. Marinos · M. Benissi

Abstract The Athens Schist Formation includes a wide variety of metasedimentary rocks, varying from strong or medium strong rocks such as sericite metasandstone, limestone, greywacke, sericite schist through to weak rocks such as metasiltstone, clayey and silty shale and phyllite. The overall rock mass is highly heterogeneous and anisotropic owing to the combined effect of advanced weathering and severe tectonic stressing that gave rise to intense folding and shearing followed by extensional faulting, which resulted in highly weathered rock masses and numerous shear and/or mylonite zones with distinct downgraded engineering properties. This paper is focused on the applicability of the GSI classification system to these highly heterogeneous rock masses and proposes an extension of the GSI system to account for the foliated or laminated weak rocks in the lower range of its applicability.

Résumé La formation des Schistes d' Athènes correspond à une large variété de roches légèrement métamorphiques, comprenant de roches à résistance élevée, comme les grès à séricite, des calcaires crystallins, des schistes à séricite, mais aussi des roches tendres comme les schistes argilleux et les phyllades. La masse rocheuse constitue un ensemble très hétérogène et anisotrope, surtout si l'on y ajoute une altération souvent avancée et une tectonique intense. Une phase compressive sévère a en effet provoqué des cisaillements importants, le massif ayant été par la suite affectée de failles normales; les zones mylonitiques sont donc très fréquentes. Cet article engage la discussion sur l'application de la classification GSI proposée par Hoek à ces masses rocheuses tres hétérogènes et propose une extension de son champ d'application aux roches feuilletées et cisaillées à faible résistance.

Key words Rock mass classification · Weak rocks · Strength parameters · Deformability modulus · Rock mass structure · Sheared shales

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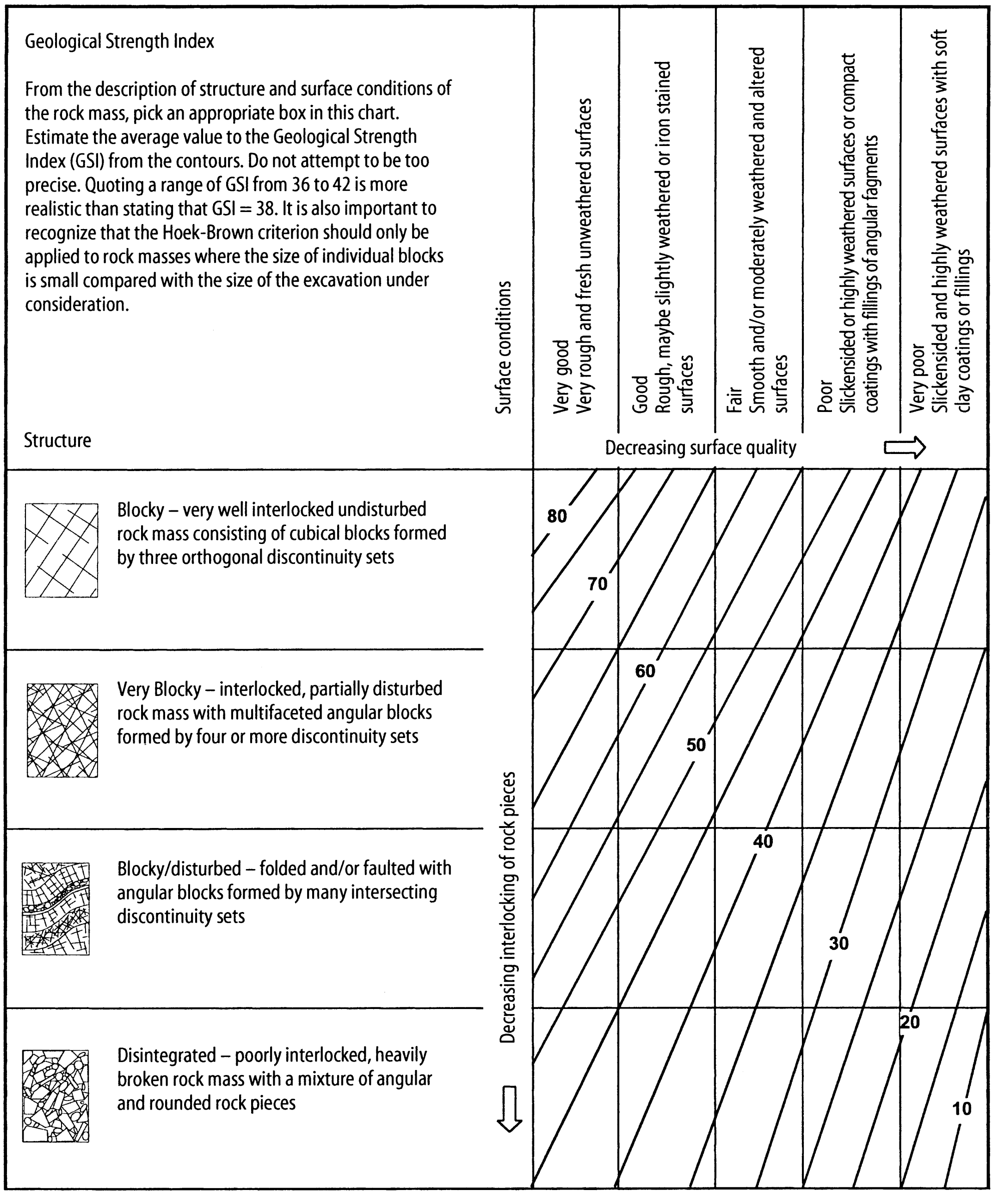
Introduction

The Geological Strength Index (GSI), introduced by Hoek (1994), Hoek et al. (1995) and Hoek and Brown (1998) provides a system for estimating the reduction in rock mass strength for different geological conditions as identified by field observations. The rock mass characterisation is straightforward and it is based upon the visual impression of the the rock structure, in terms of blockiness, and the surface condition of the discontinuities indicated by joint

Table 1
Characterisation of rock masses on the basis of interlocking and joint alteration (Hoek and Brown 1998 adjusted from Hoek 1994)



roughness and alteration (Table 1; from Hoek and Brown 1998). The combination of these two parameters provides a practical basis for describing a wide range of rock mass types, with diversified rock structure ranging from very tightly interlocked strong rock fragments to heavily crushed rock masses. Based on the rock mass description the value of GSI is estimated from the contours given in Table 1.

The uniaxial compressive strength σ_{ci} and the material constant m_i are determined by laboratory testing or estimated from published tables, reproduced here as Tables 2 and 3 respectively. Wherever possible the values of these constants should be determined by statistical analysis of the results of a set of triaxial tests on carefully prepared core samples. The shear strength of the rock mass, defined by the angle of internal friction ϕ and cohesion c, are estimated from the curves plotted in Figs. 1 and 2.

Using the GSI system, provided the UCS value is known, the rock mass deformation modulus E_m for $\sigma_{ci} < 100$ MPa is estimated in GPa from the following equation (Hoek and Brown 1998):

$$E_m = \sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{\left(\frac{GSI-10}{40}\right)} \qquad (1)$$

The relationships between the parameters incorporated in this equation are illustrated in Fig. 3.

The Athens Schist rock masses

The Athens Schist Formation is a term used to describe a highly heterogeneous, flysch-like formation of Cretaceous

Table 2
Field estimates of the uniaxial compressive strength of intact rock pieces

Grade[a]	Term	Uniaxial compressive strength (MPa)	Point load index (MPa)	Field estimate of strength	Examples
R6	Extremely strong	>250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100–250	4–10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff
R4	Strong	50–100	2–4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, phyllite, sandstone, schist, shale
R3	Medium strong	25–50	1–2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Claystone, coal, concrete, schist, shale, siltstone
R2	Weak	5–25	[b]	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, rocksalt, potash
R1	Very weak	1–5	[b]	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock
R0	Extremely weak	0.25–1	[b]	Indented by thumbnail	Stiff fault gouge

[a] Grade according to Brown (1981)
[b] Point load tests on rocks with a unaxial compressive strength below 25 MPa are likely to yield ambiguous results

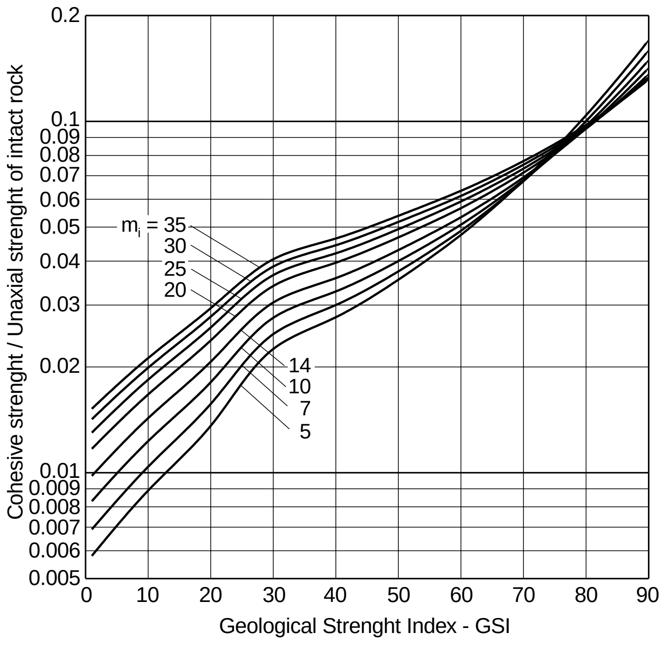


Fig. 1
Relationship between cohesive strength and Geological Strength Index (*GSI*); example: GSI = 20, $m_i = 10$, $c/\sigma_{ci} = 0.018$

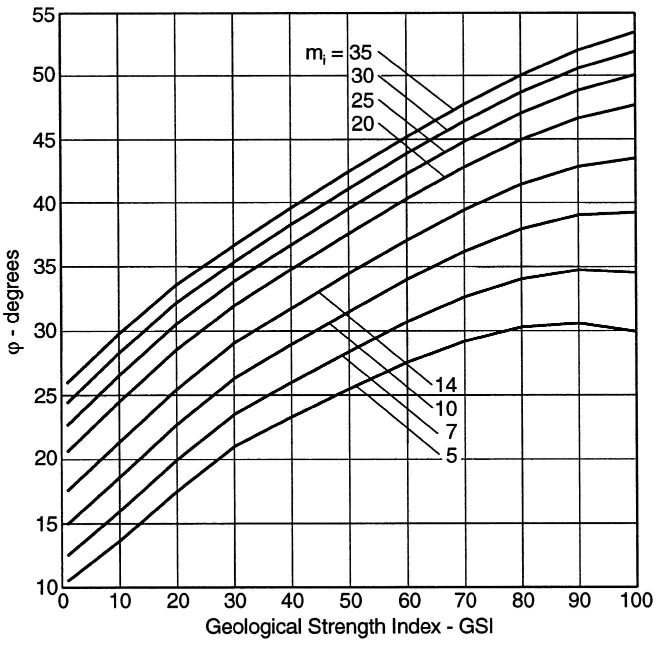


Fig. 2
Relationship between friction angle and GSI; example: GSI = 20, $m_i = 10$, $\phi = 23°$

Table 3
Values of the constant m_i for intact rock, by rock group. Note that values in parenthesis are estimates

Rock type	Class	Group	Texture			
			Coarse	**Medium**	**Fine**	**Very fine**
Sedimentary	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
				Greywacke (18)		
	Non-clastic	Organic		Chalk 7		
				Coal (8–21)		
		Carbonate	Breccia (20)	Sparitic limestone (10)	Micritic limestone 8	
		Chemical		Gypstone 16	Anhydrite 13	
Metamorphic	Non-foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 25–31	Mylonites (6)	
	Foliated[a]		Gneiss 33	Schists 4–8	Phyllites (10)	Slate 9
Igneous	Light		Granite 33		Rhyolite (16)	Obsidian (19)
			Granodiorite (30)		Dacite (17)	
			Diorite (28)		Andesite 19	
	Dark		Gabbro 27	Dolerite (19)	Basalt (17)	
			Norite 22			
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

[a] These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane

age (Marinos G. et al. 1971). It comprises schists, phyllites and metasedimentary shales, siltstones and sandstones. Limestones and marls may also occur while igneous activity has introduced peridotitic and diabasic intrusions at certain localities. The Athens Schist bedrock is primarily marked by an advanced degree of weathering and intense folding, shearing and extensional faulting, which completed the structural "downgrading" of the rock mass.

The Athens Schist rock mass is characterised by:

1. Frequent changes of lithological facies at short distances accentuated by an irregular alteration and weathering pattern.
2. Variability of materials ranging from hard rocks to soils in terms of strength (frequently mixed at the scale of the engineering structures).
3. A highly complex structural pattern of numerous structural shears and faults.

The tectonic activity whenever associated with weak rocks often produces engineering soil materials. The tectonic fabric of the Athens Schist Formation includes mylonite materials that are not only limited to the major fault zones but also occur as thick gouge infilling of systematic or non-systematic shears. The response of rock mass volumes composed of hard rock and weak rock intercalations to the severe tectonic activity gave rise to disharmonic folding and faulting that often entailed a clearly visible chaotic structure of isolated lensed blocks of hard rock 'floating' within a soft clayey matrix (Marinos P. et al. 1997a and b).

Most of the Athens Schist Formation members are aptly described by the term *blocky/disturbed* as the rock masses are often folded and faulted. The Athens Schist Formation rock mass exhibits well-defined shears, frequently oriented parallel to the foliation planes that constitute the prevailing structural feature of the rock mass. These shear surfaces are commonly polished and slickensided with clayey coatings or in some cases thick (>10 cm) mylonitic clay gouge. The discontinuity condition falls between the *fair*

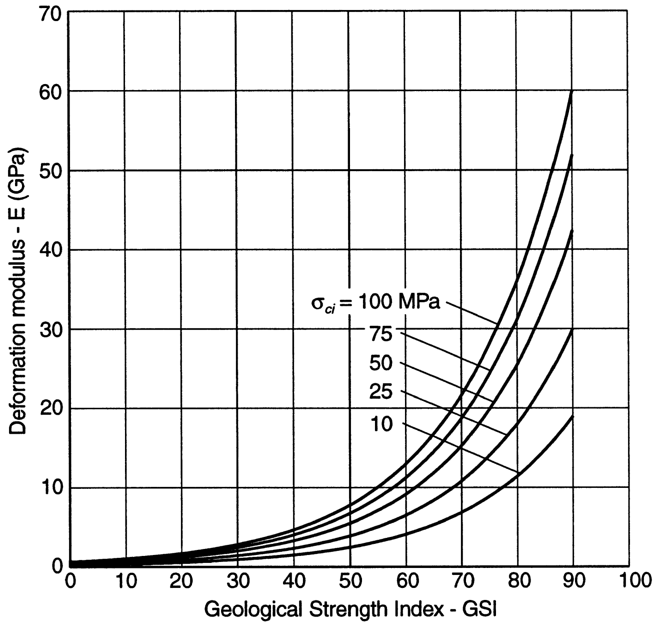


Fig. 3
Relationship between GSI, intact rock strength (σ_{ci}) and in situ modulus of deformation E_m for $\sigma_{ci} < 100$

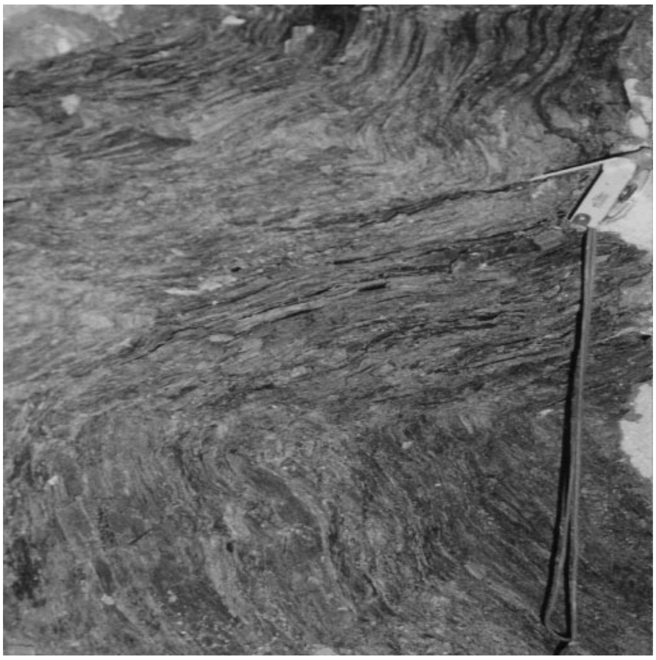


Fig. 4
The foliated sequences of sericite sandstone and schist of slight difference in competence are classified as blocky/disturbed, with a fair to very poor discontinuity condition

Fig. 5
Well interlocked very blocky strong sandstone rock mass

and the *very poor* range of categories. The categories *blocky/disturbed – fair to blocky/disturbed – very poor* (Table 1) are typically assigned to rock masses composed of sericite metasandstone, greywacke, metasiltstone, marly limestone, schist (Fig. 4) or to alternations of these rocks where however they exhibit a slight difference in competence.

Competent rocks and well interlocked rock masses with three or more joint set systems that better fit to the model of a *blocky* rock structure are less common in the Athens Schist Formation and have been classified as *very blocky* (see Fig. 5). They include slightly weathered, medium strong to strong, rock types such as arkosic metasandstone, limestone and fresh diabase-peridotite, which are encountered as isolated occurrences within the Athens Schist Formation.

However, several rock mass types, which are quite abundant in the Athens Schist Formation, cannot be adequately described by the above classification. This category involves primarily originally weak laminated non-competent rocks of low strength and high deformability, such as the dark grey clayey and silty shales or phyllites of the Athens Schist Formation. Also the same category involves rock masses of downgraded strength and enhanced deformability, as a combined result of intense *shearing* and mylonitization along the lamination or foliation planes assisted by significant weathering of the intact rock pieces. As for regards the rock structure a well-defined persistent and closely spaced lamination or foliation system is dominant and is clearly recognisable by the slickensided surfaces and the gouge-infilled shears (Fig. 7). These types of rock masses, an example of which is shown in Fig. 6, are of a non-blocky/non-anglular structure, and cannot be adequately described by any of the available GSI rock structure categories. The closest fit, considering a *very poor* surface condition, is that of the *disintegrated* rock structure.

The ranges of GSI values corresponding to the rock mass descriptions given above indicated by the ellipses plotted in Table 4.

Fig. 6
The intensely sheared and mylonitized argillaceous shales could only fit to the disintegrated category of Table 1 with a very poor surface condition

Fig. 7
a Foliated/laminated/sheared rock structure. **b** The seamy rock mass type consisting of intercalated rock members of strikingly different competence which are differentially deformed (sheared, folded and faulted). **c** A chaotic rock mass comprising lensified hard rock bodies and boudinaged quartz or calcite lenses floating in a sheared soil-like environment. The rock mass structure is scale-independent and its influence depends upon the scale of the engineering structure

Extension of GSI to accommodate the weakest Athens Schist rock masses

The uniaxial strength of some of the rock types comprising the Athens Schist Formation was established by testing approximately 60 samples.
In the case of weak dark grey shale, weak laminated metasiltstone or highly weathered sericite sandstone for which it was generally not possible to form testable samples, a strength range was empirically estimated in the field on the basis of the descriptions given in Table 2.
Based on the measured/estimated UCS strengths and ranges of material constants (m_i) values and the GSI values attributed to the different rock mass types, the cohesive strength and friction angle for each rock mass type were estimated from Figs. 1 and 2 (see upper section of Table 5).
The range of the rock mass deformability modulus E_m was calculated for each rock mass by means of equation (1); the values are shown in Table 6.
In the case of the 'dark grey clayey shales', the weakest among the Athens Schist Formation, *Menard* pressuremeters show a typical range of E_m values between 50–150 MPa. Back analysis of settlements from underground excavations in the city of Athens yield values of between 150–250 MPa. These low E_m values derived from both *Menard* pressuremeters and from back analysis of settlements are not always consistent with the calculated E_m values by use of equation (1) when the input GSI value is the minimum that falls in the lower right portion of the *disintegrated* category of Table 4 (see rock mass types C^+ and partly B^- of table 6). This fact alone necessitated the addition of a new rock mass category where the calculated E_m values are in better accordance with the measured ones.
Moreover the mechanism of deformation in the above described foliated and sheared rocks is not governed by rock-to-rock contacts of *angular or subrounded rock fragments* as in the *disintegrated* category, but it is rather controlled by the displacements along the numerous very thinly spaced presheared foliation planes of the rock mass.
A new *foliated/laminated/sheared* rock mass category has thus been considered to better represent thinly laminated or foliated and structurally sheared weak rocks. In these rock masses the lamination or foliation is the predominant structural feature which prevails over any other discontinuity set, resulting in complete lack of blockiness. The new *foliated/laminated/sheared* rock mass structure, shown in Table 7, is not associated with *good* or *very good* discontinuity surface quality, since it entails a degree of preshearing along the lamination/foliation surfaces. For the remaining *fair* to *very poor* surface qualities the equivalent GSI contours range from the new value of 5 up to 30 and the derived E_m values are shown in the graph of Table 6 as type C^- rock mass.
More specifically, in terms of shear strength and deformability, by moving the Athenian black shales down to the

Table 4
Field of GSI classification of distinct rock mass types encountered in the Athenian substratum: the prevailing foliated sandstone-schist rock mass (*diagonal hatch*), the occasional strong blocky metasandstone or limestone (*vertical hatch*) and the sheared mylonitic black shales (*horizontal hatch*)

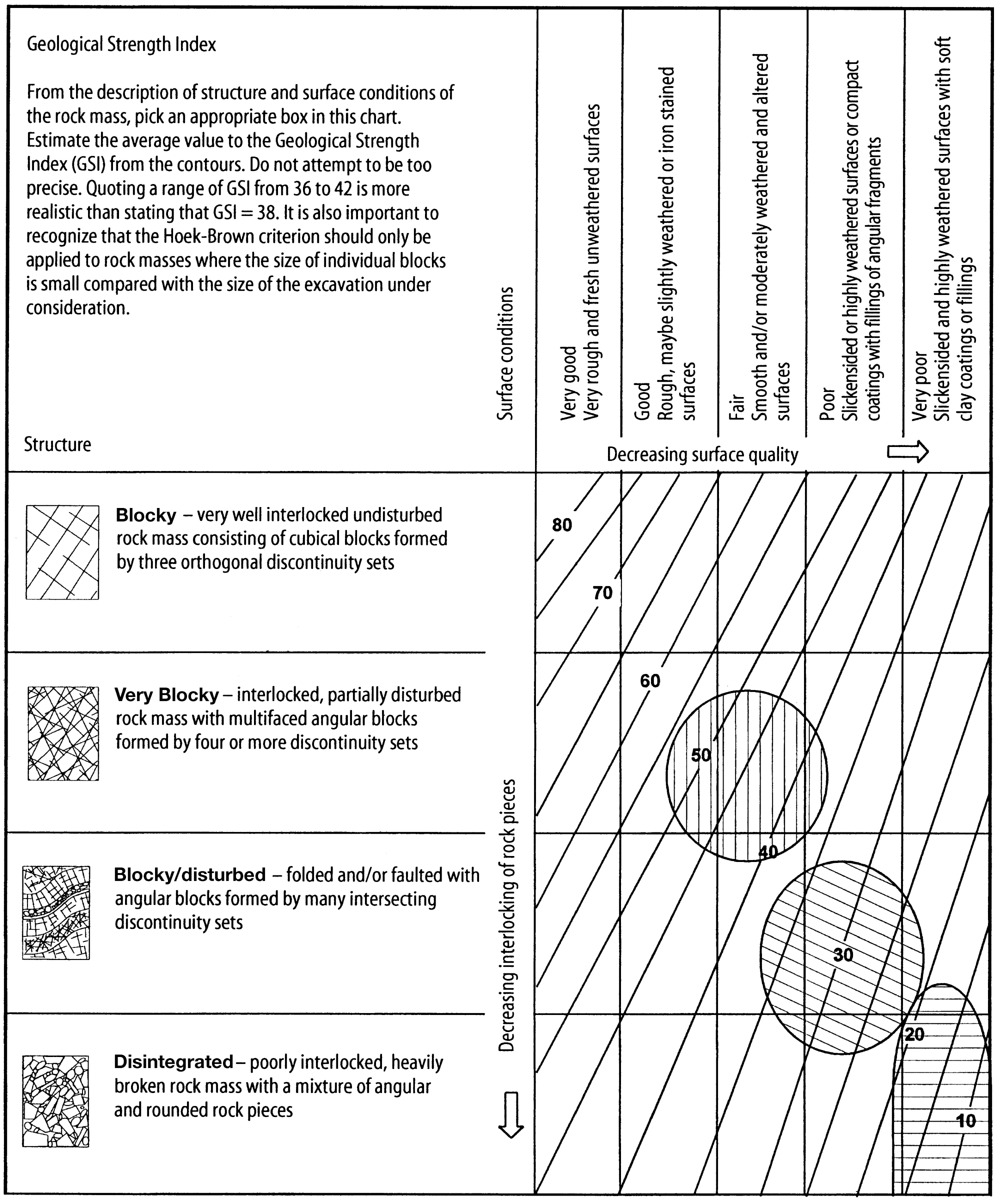
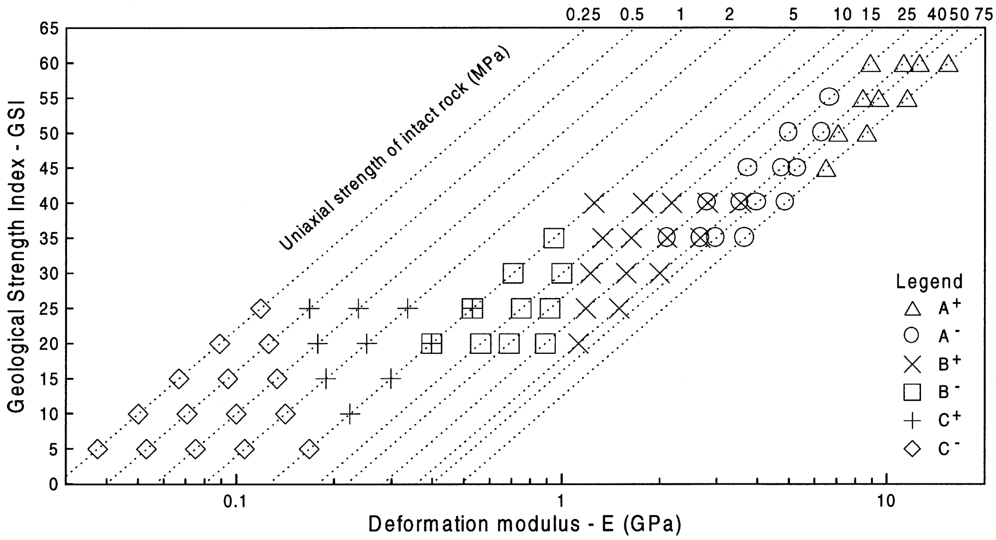


Table 5
Rock mass characteristics and estimated shear strengths for different rock mass units

Rock mass type	UCS σ_{ci} MPa	Constant m_i	Estimated GSI	Cohesion c-MPa	Friction angle $\phi°$
Limestone	54	10	50 ± 10	2.3 ± 0.4	35 ± 2
Sericite sandstone	37	19	50 ± 10	1.7 ± 0.2	37 ± 2
Greywacke	25	18	30 ± 8	0.7 ± 0.1	31 ± 2
Dark grey siltstone	18	9	30 ± 8	0.55 ± 0.2	25 ± 2
Black shales (classified as Disintegrated)	1–5	8	15 ± 8	0.05 ± 0.04	19 ± 3
Black shales (classified in the new Folliated laminated/ sheared, rock structure)	1–5	8	10 ± 6	0.04 ± 0.03	17 ± 2

Table 6
Estimated deformation modulus values (in GPa) for various rock mass types of the Athens Schist' Formation (A^+ to C^-)



A^+	Medium strong to strong metasandstone or limestone rock mass, interlocked, with angular blocks formed by four or more discontinuity sets with rough and MW to SW surface condition.
A^-	The lower E_m values of this field derive either from lower σ_{ci} values due to weathering-tectonic weakening of the intact rock material, or from lower GSI values reflecting the decreased surface quality due to shearing / mylonitization and weathering.
B^+	Medium strong to weak, thinly foliated sericite sandstone / schist or greywacke, with occasional boudins of quartz.
B^-	The lower E_m values may derive either from lower σ_{ci} values due to weathering-tectonic weakening of the intact rock material, or from lower GSI values reflecting the decreased surface quality due to shearing / mylonitization and weathering.
C^+	Weak to very weak, laminated argillaceous shale or phyllite, with boudins of quartz or lenticular blocks of intact rock in a soft rock environment.
C^-	The lower E_m values bottom left derive either from lower σ_{ci} values due to weathering-weakening of the intact rock material, or to lower GSI values reflecting the absence of blockiness and the decreased surface quality due to shearing / mylonitization and weathering.

foliated/laminated/sheared category, the estimated GSI is reduced to 10 ± 6 and this gives a cohesive strength of 0.04 ± 0.03 MPa, a friction angle of 17 ± 2 deg and an E_m value of 70–300 MPa (see Table 5 and Table 6 where 1 MPa $< \sigma_{ci} <$ 5 MPa lower row).

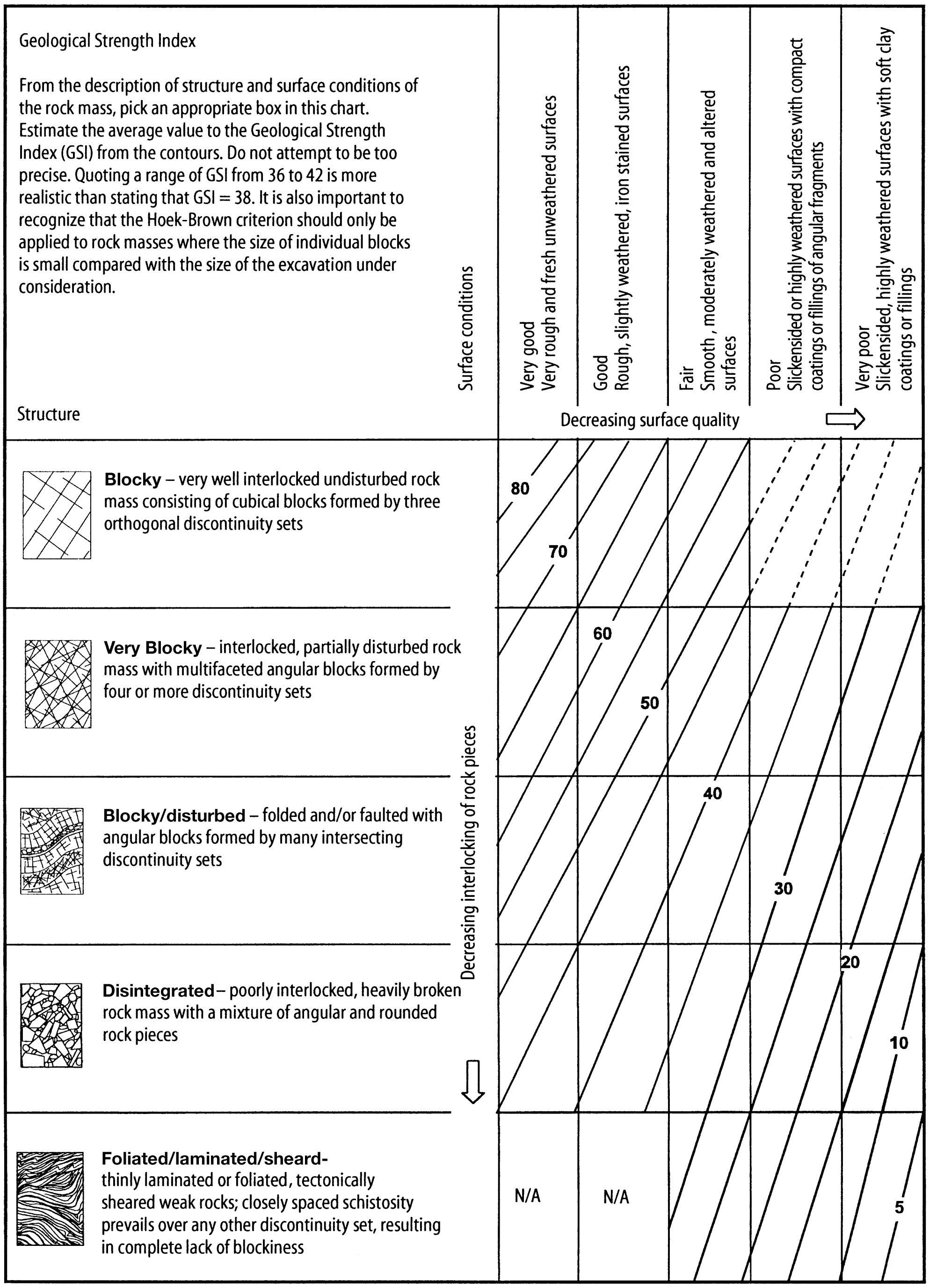
Rock mass types not described by the GSI classification

In addition to the weak sheared rock masses which have been integrated in the GSI classification system as described above, there are two more distinct rock mass types frequently encountered in the Athens Schist complex that cannot be accomodated within the existing GSI classification.

The first category involves seamy structures consisting of intercalated rock members of strikingly different competence, which are differentially deformed (sheared, folded and faulted), e.g. sandstones vs. mudstones as a common feature for flysch (Fig. 7b). In this case the geotechnical behaviour of the rock mass is beyond the philosophy of the GSI concept of rock mass structure, since it is always controlled by the persistent interfaces between the two media of strikingly different strength and deformability.

The second rock mass type is where extended tectonic fatigue has produced chaotic structures comprising lensified intact rock bodies and boudinaged quartz/calcite lenses that healed former structural discontinuities, and which now 'float' in a sheared soil-like environment (Fig. 7c). The geotechnical behaviour of this type of rock mass can be identified in between the *disintegrated* and *foliated/laminated* categories of Table 7.

Table 7
A new laminated/foliated/sheared rock structure accounts for presheared thinly foliated very weak rocks, in which the prevailing rock mass feature controlling strength and deformability are not the rock-to-rock contacts of the broken rock pieces (as in breccias) but rather the shear strength of the fines along the numerous clayey coated foliation or shear surfaces



Conclusions

The Geological Strength Index (GSI) classification scheme, through which the rock mass strength and deformability parameters are estimated based on the rock mass structure and discontinuity surface condition does not adequately describe some of the rock mass types commonly encountered in the Athens' bedrock. The materials not included are the thinly foliated or laminated, folded and predominantly sheared weak rocks of non-blocky structure. In these rock masses the strength and deformability characteristics are not governed by rock-to-rock contacts of angular or rounded rock pieces but rather by the displacements along the numerous very thinly spaced presheared and slickensided foliation planes of the rock mass.
A new *foliated/laminated* rock mass structure category is proposed to accommodate these rock types in the lowest range of applicability of the GSI system. Given the presheared nature of the rock's discontinuities their surface

condition could not be classified either as *very good* or as *good* and therefore the classification is non-applicable. For the remaining *fair* to *very poor* surface qualities the equivalent GSI contours now range from the new value of 5 up to 30.

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GSI: A GEOLOGICALLY FRIENDLY TOOL FOR ROCK MASS STRENGTH ESTIMATION

Paul Marinos[1] and Evert Hoek[2]

ABSTRACT

This paper presents a review of the estimation of rock mass strength properties through the use of GSI. The GSI classification system greatly respects the geological constraints that occur in nature and are reflected in the geological information. A discussion is given regarding the ranges of the Geological Strength Index for typical rock masses with specific emphasis to heterogeneous rock masses.

1.0 INTRODUCTION

Reliable estimates of the strength and deformation characteristics of rock masses are required for almost any form of analysis used for the design of surface excavations. Hoek and Brown (1980a, 1980b) proposed a method for obtaining estimates of the strength of jointed rock masses, based upon an assessment of the interlocking of rock blocks and the condition of the surfaces between these blocks. This method was modified over the years in order to meet the needs of users who applied it to problems that were not considered when the original criterion was developed (Hoek 1983, Hoek and Brown 1988). The application of the method to poor quality rock masses required further changes (Hoek, Wood and Shah, 1992) and, eventually, the development of a new classification called the Geological Strength Index (Hoek 1994, Hoek, Kaiser and Bawden 1995, Hoek and Brown 1997, Hoek, Marinos and Benissi, 1998), extended recently for heterogeneous rock masses (Marinos and Hoek, 2000). A review of the development of the criterion and the equations proposed at various stages in this development is given in Hoek and Brown (1997).

2.0 ESTIMATE OF ROCK MASS PROPERTIES

The basic input consists of estimates or measurements of the uniaxial compressive strength (σ_{ci}) and a material constant (m_i) that is related to the frictional properties of the rock. Ideally, these basic properties should determined by laboratory testing as described by Hoek and Brown (1997) but, in many cases, the information is required before laboratory tests have been completed. To meet this need, tables that can be used to estimate values for these parameters are reproduced in Tables 1 and 2. Note that both tables are updated from earlier versions (Marinos and Hoek, 2000).

The most important component of the Hoek – Brown system for rock masses is the process of reducing the material constants σ_{ci} and m_i from their "laboratory" values to appropriate in situ values. This is accomplished through the Geological Strength Index GSI that is defined in Table 3.

GSI has been developed over many years of discussions with engineering geologists with whom E. Hoek has worked around the world. Careful consideration has been given to the precise wording in each box and to the relative weights assigned to each combination of structural and surface conditions, in order to respect the geological conditions existing in nature.

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Table 1: Field estimates of uniaxial compressive strength of intact rock.[3]

Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field estimate of strength	Examples
R6	Extremely Strong	> 250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100 - 250	4 - 10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite , rhyolite, tuff
R4	Strong	50 - 100	2 - 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium strong	25 - 50	1 - 2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concrete, phyllite, schist, siltstone
R2	Weak	5 - 25	**	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rocksalt,
R1	Very weak	1 - 5	**	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock, shale
R0	Extremely Weak	0.25 - 1	**	Indented by thumbnail	Stiff fault gouge

* Grade according to Brown (1981).

** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.

[3] Note that this table contains a few changes in the column of examples from previously published version.

Table 2: Values of the constant m_i for intact rock, by rock group[4]. Note that values in parenthesis are estimates. The range of values quoted for each material depends upon the granularity and interlocking of the crystal structure – the higher values being associated with tightly interlocked and more frictional characteristics.

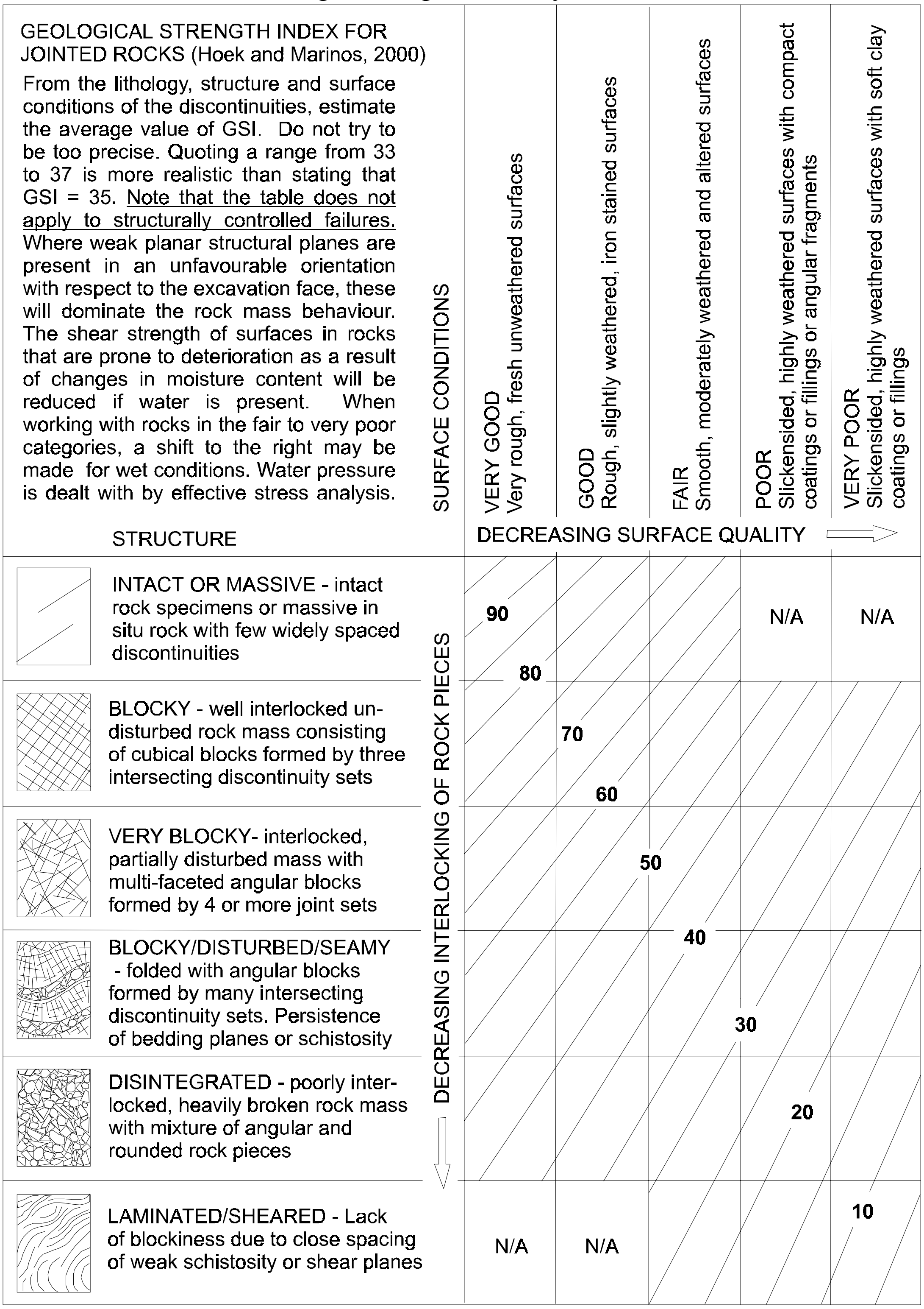
Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerates * Breccias *	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6	Gneiss 28 ± 5	
	Foliated**			Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

* Conglomerates and breccias may present a wide range of m_i values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone, to values used for fine grained sediments (even under 10).

** These values are for intact rock specimens tested normal to bedding or foliation. The value of mi will be significantly different if failure occurs along a weakness plane.

[4] Note that this table contains several changes from previously published versions, These changes have been made to reflect data that has been accumulated from laboratory tests and the experience gained from discussions with geologists and engineering geologists.

Table 3: Geological strength index for jointed rock masses.



Having defined the parameters σ_{ci}, m_i and GSI as described above, the next step is to estimate the mechanical properties of the rock mass. The procedure for making these estimates has been described in detail by Hoek and Brown (1997) it will not be repeated here. A spreadsheet for carrying out these calculations is given in Table 4 [5].

Table 4: Spreadsheet for the calculation of rock mass properties

Input:	sigci =	10	MPa	mi =	10		GSI =	30	
	Depth of failure surface or tunnel below slope =				25	m	Unit wt. =	0.027	MN/n3
Output:	stress =	0.68	MPa	mb =	0.82		s =	0.0004	
	a =	0.5		sigtm =	-0.0051	MPa	A =	0.4516	
	B =	0.7104		k =	3.95		phi =	36.58	degrees
	coh =	0.136	MPa	sigcm =	0.54	MPa	E =	1000.0	MPa
Calculation:									
									Sums
sig3	1E-10	0.10	0.19	0.29	0.39	0.48	0.58	0.68	2.70
sig1	0.20	1.01	1.47	1.84	2.18	2.48	2.77	3.04	14.99
ds1ds3	21.05	5.50	4.22	3.64	3.29	3.05	2.88	2.74	46.36
sign	0.01	0.24	0.44	0.62	0.80	0.98	1.14	1.31	5.54
tau	0.04	0.33	0.50	0.64	0.76	0.86	0.96	1.05	5.14
x	-2.84	-1.62	-1.35	-1.20	-1.09	-1.01	-0.94	-0.88	-10.94
y	-2.37	-1.48	-1.30	-1.19	-1.12	-1.06	-1.02	-0.98	-10.53
xy	6.74	2.40	1.76	1.43	1.22	1.07	0.96	0.86	16.45
xsq	8.08	2.61	1.83	1.44	1.19	1.02	0.88	0.78	17.84
sig3sig1	0.00	0.10	0.28	0.53	0.84	1.20	1.60	2.05	7
sig3sq	0.00	0.01	0.04	0.08	0.15	0.23	0.33	0.46	1
taucalc	0.04	0.32	0.49	0.63	0.76	0.87	0.97	1.07	
sig1sig3fit	0.54	0.92	1.30	1.68	2.06	2.45	2.83	3.21	
signtaufit	0.14	0.31	0.46	0.60	0.73	0.86	0.98	1.11	
Cell formulae:									
stress =	if(depth>30, sigci*0.25,depth*unitwt*0.25)								
mb =	mi*EXP((GSI-100)/28)								
s =	IF(GSI>25,EXP((GSI-100)/9),0)								
a =	IF(GSI>25,0.5,0.65-GSI/200)								
sigtm =	0.5*sigci*(mb-SQRT(mb^2+4*s))								
sig3 =	Start at 1E-10 (to avoid zero errors) and increment in 7 steps of stress/28 to stress/4								
sig1 =	sig3+sigci*(((mb*sig3)/sigci)+s)^a								
ds1ds3 =	IF(GSI>25,(1+(mb*sigci)/(2*(sig1-sig3))),1+(a*mb^a)*(sig3/sigci)^(a-1))								
sign =	sig3+(sig1-sig3)/(1+ds1ds3)								
tau =	(sign-sig3)*SQRT(ds1ds3)								
x =	LOG((sign-sigtm)/sigci)								
y =	LOG(tau/sigci)								
xy =	x*y	x sq =	x^2						
A =	acalc =	10^(sumy/8 - bcalc*sumx/8)							
B =	bcalc =	(sumxy - (sumx*sumy)/8)/(sumxsq - (sumx^2)/8)							
k =	(sumsig3sig1 - (sumsig3*sumsig1)/8)/(sumsig3sq-(sumsig3^2)/8)								
phi =	ASIN((k-1)/(k+1))*180/PI()								
coh =	sigcm/(2*SQRT(k))								
sigcm =	sumsig1/8 - k*sumsig3/8								
E =	IF(sigci>100,1000*10^((GSI-10)/40),SQRT(sigci/100)*1000*10^((GSI-10)/40))								
phit =	(ATAN(acalc*bcalc*((signt-sigtm)/sigci)^(bcalc-1)))*180/PI()								
coht =	acalc*sigci*((signt-sigtm)/sigci)^bcalc-signt*TAN(phit*PI()/180)								
sig3sig1=	sig3*sig1	sig3sq =	sig3^2						
taucalc =	acalc*sigci*((sign-sigtm)/sigci)^bcalc								
s3sifit =	sigcm+k*sig3								
sntaufit =	coh+sign*TAN(phi*PI()/180)								

[5] For an electronic version of this Excel spreadsheet, contact Evert Hoek <ehoek@attglobal.net>

2.1 Deep tunnels

For tunnels at depths of more than 30 m, the rock mass surrounding the tunnel is confined and its properties are calculated on the basis of a minor principal stress or confining pressure of $0 < \sigma_3 < 0.25\sigma_{ci}$, in accordance with the procedure defined by Hoek and Brown (1997).

For the case of "deep" tunnels, equivalent Mohr Coulomb cohesive strengths and friction angles can be calculated by means of the spreadsheet given in Table 4. Note that any depth greater than 30m can be used for this calculation. In addition, the deformation modulus E and the uniaxial compressive strength σ_{cm} of the rock mass can be estimated. Plots of these estimated values are given in Figures 1 to 4.

The uniaxial compressive strength of the rock mass σ_{cm} is a particularly useful parameter for evaluating potential tunnel squeezing problems. The following equation, obtained by a curve fitting process on the plots presented in Figure 4, gives a very close approximation of σ_{cm} for selected values of the intact rock strength σ_{ci}, constant m_i and the Geological Strength Index GSI :

$$\sigma_{cm} = (0.0034 m_i^{0.8})\sigma_{ci}\{1.029 + 0.025 e^{(-0.1 m_i)}\}^{GSI} \tag{1}$$

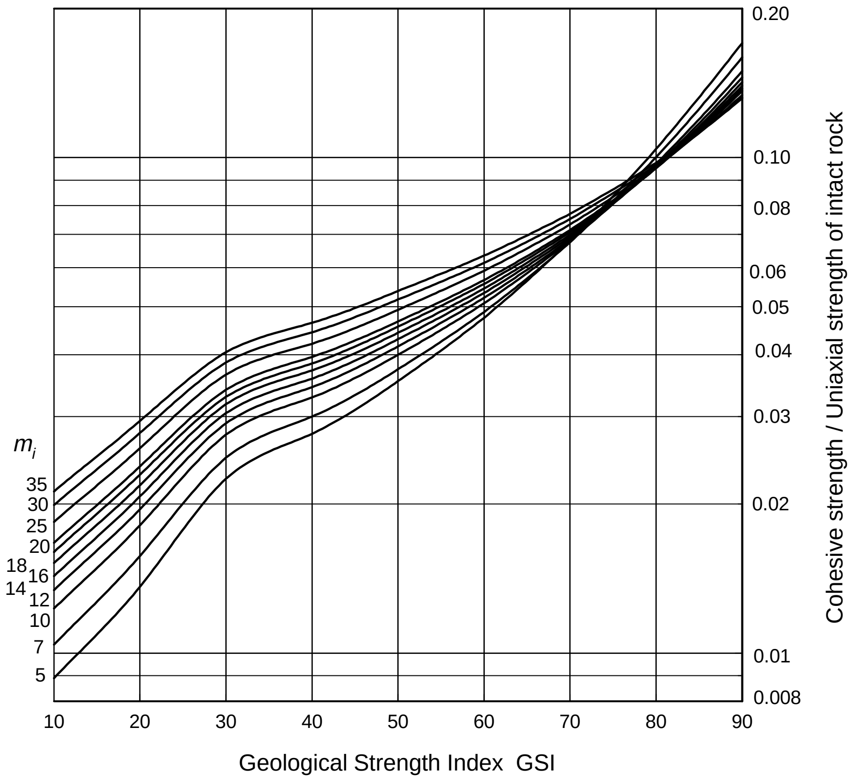


Figure 1. Relationship between ratio of cohesive strength to uniaxial compressive strength of intact rock c/σ_{ci} and GSI for different m_i values, for depths of more than 30m.

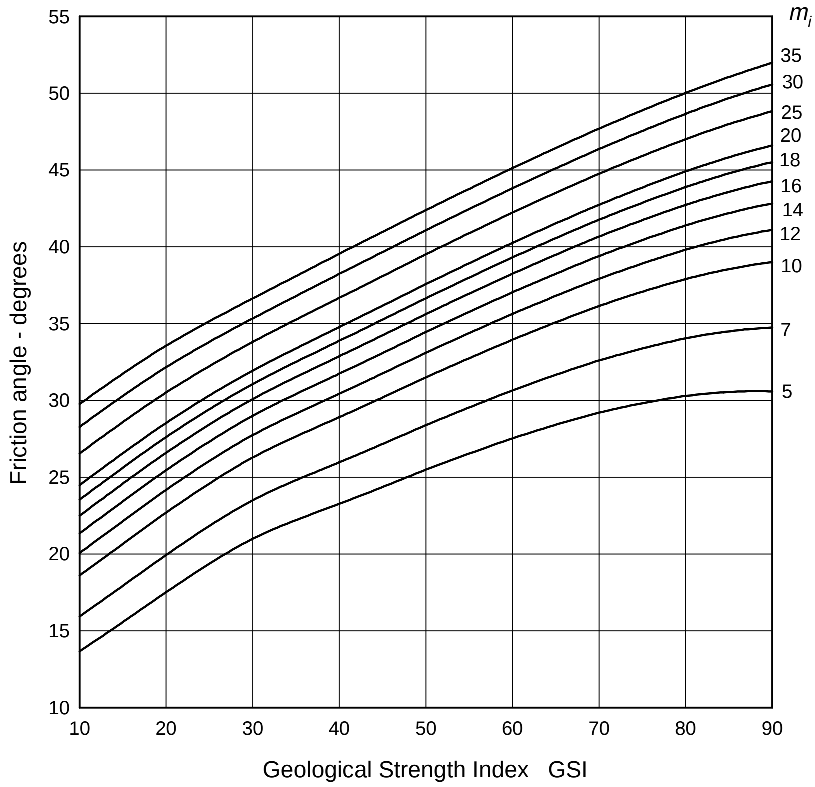


Figure 2. Friction angle ϕ for different GSI and m_i values, for depths more than 30m.

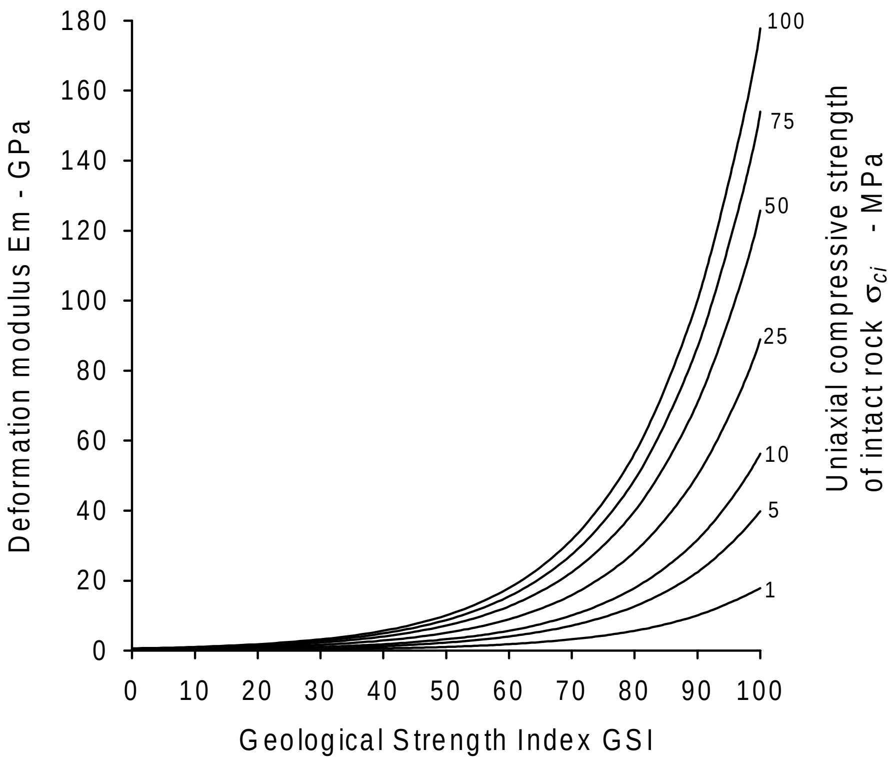


Figure 3. Rock mass Deformation modulus *E* versus Geological Strenth Index *GSI*.

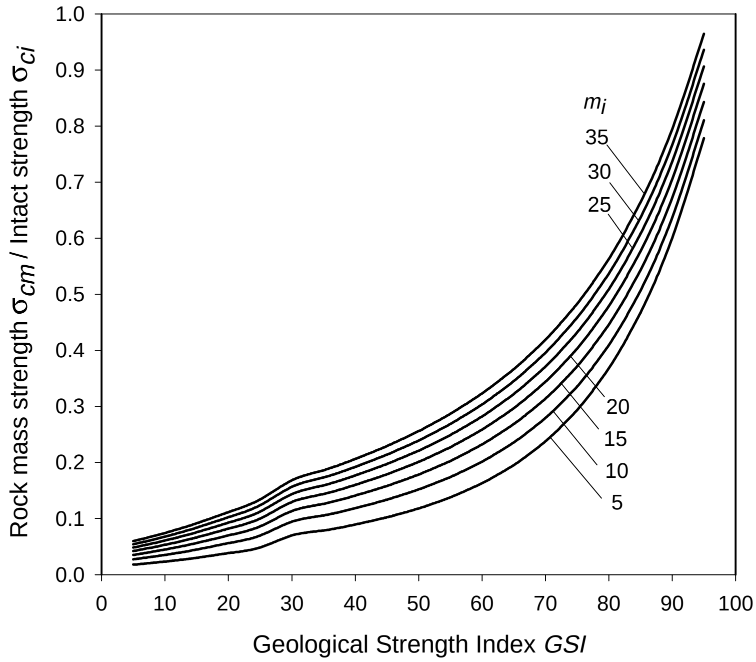


Figure 4. Relationship between rock mass strength σ_{cm}, intact rock strength σ_{ci}, constant m_i and the Geological Strength Index *GSI*, for depths of more than 30m.

2.2 Shallow tunnels and slopes

For shallow tunnel and slopes in which the degree of confinement is reduced, a minor principal stress range of $0 < \sigma_3 < \sigma_v$ is used, where σ_v = depth x unit weight of the rock mass. In this case, depth is defined as the depth below surface of the tunnel crown or the average depth of a failure surface in a slope in which a circular type can be assumed, i.e. where the failure is not structurally controlled.

In the case of shallow tunnels or slopes, the spreadsheet presented in Table 4 allows the user to enter the depth below surface and the unit weight of the rock mass. The vertical stress σ_v calculated from the product of these two quantities is then used to calculate the rock mass properties.

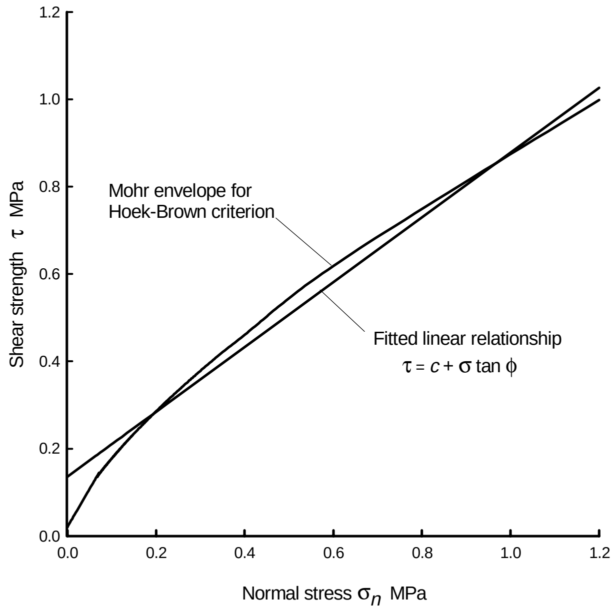


Figure 5. Mohr envelope for Hoek Brown criterion and fitted linear relationship for the normal stress range $0 < \sigma_n < \sigma_v$ where σ_v = depth x unit weight. As shown in the spreadsheet in Table 4, the friction angle ϕ = 36.6° and the cohesive strength c = 136 kPa for σ_{ci} = 10 MPa, m_i = 10, *GSI* = 30 and a depth below surface of 25 m.

3.0 TYPICAL RANGES OF GSI FOR VARIOUS ROCK MASSES

The strength of a jointed rock mass depends on the properties of the intact rock pieces and also upon the freedom of these pieces to slide and rotate under different stress conditions. This freedom is controlled by the geometrical shape of the intact rock pieces as well as the condition of the surfaces separating the pieces. Angular rock pieces with clean, rough discontinuity surfaces will result in a much stronger rock mass than one which contains rounded particles surrounded by weathered and altered material, or sheared flakes of the initial rock.

Note that the Hoek and Brown criterion and indeed any of the other published criteria that can be used for this purpose, assume that the rock mass behaves isotropically. In other words, while the behaviour of the rock mass is controlled by movement and rotation of rock elements separated by intersecting structural features such as bedding planes and joints, there are no preferred failure directions.

This failure criteria should not be used when the rock mass consists of a strong blocky rock such as sandstone, separated by clay coated and slickensided persisting bedding surfaces. The behaviour of such rock masses will be strongly anisotropic and will be controlled by the fact that the bedding planes are an order of magnitude weaker that any other features. In such rock masses the predominant failure mode will be planar or wedge slides in slopes, or gravitational falls of wedges or blocks of rock defined by the intersection of the weak bedding planes with other features which act as release surfaces in tunnels. However, if the rock mass is heavily fractured, the continuity of the bedding surfaces will be disrupted and the rock may behave as an isotropic mass.

This GSI 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 surface excavations such as roadcuts, in tunnel faces and in borehole core.

The Geological Strength Index, by the combination of the two fundamental parameters of geological process, the blockiness of the mass and the conditions of discontinuities, respects the main geological constraints that govern a formation and is thus both a geologically friendly index and practical to assess.

The petrographic characteristics of each and every rock do not however allow all the combinations that can be derived from the GSI charts to exist. A limestone mass, for instance, can not present “poor” conditions in discontinuities or a thin bedded sequence of rock cannot be better than “seamy” in a folded geological environment; a siltstone or clayshale cannot present better conditions in the discontinuities than “fair”.

In order to give the most probable range of GSI values for rock masses of various rock types that most usually occur in nature, a series of indicative charts are presented in tables 5 to 13. Deviations may certainly occur but these are the exceptions. From the charts it can be seen:

- *Sandstones*: A typical rock mass varies in the majority of cases between 45 and 90, but if tectonically brecciated from 30 to 45. It is understood that in all cases weak interlayers do not interfere and that in a typical sandstone no clayey or gypsiferous cement is involved; if yes the GSI values may move to the right of the chart.
- *Silstones, clayshales*: Siltstones and claystones may be homogeneous with no discontinuities other than bedding planes, if they are of recent geological age and have not suffered from major tectonic effects. In these cases the GSI classification is not applicable and its use, even approximately, is not recommended. In these cases laboratory testing is to be applied. However GSI may be applied when siltstones exhibit joints and shears (common deformational features in orogenetic belts, etc). In shales, either silty or clayey, the role of weak schistosity planes is in that case more pronounced, which cannot however induce an anisotropic character to the mass, as they are developed in thin discontinuous flake-like sheets. By their nature the condition of discontinuities will usually be poor, and it cannot be classified beyond the fair type, even in extreme cases. In many cases siltstones and clayshales are present as thin interlayers (e.g. of few millimetres of thickness) between stronger rocks; in that case a downgrading of the rock mass towards the right part of the chart is brought about, unless other unfavourable situations arise from instability on preferred failure orientations.
- *Limestones*: Limestones in term of bedding may be massive, bedded, thin bedded (few to 10-20cm thickness of beds). Jointing from the tectonic history is added. In all cases the surface of discontinuities is mainly “good” and can hardly be “fair”. The thin bedded type is more keen to differential movement of beds during folding thus lower GSI values are expected. In this type the many intersecting discontinuity sets diminish the role of the persisting orientations of the bedding planes, making GSI applicable. In the chart of Table 7 the limestone series with thin interlayers or films of clayey, marly or silty nature is of course not considered.
- *Granite*: The range shaded in the chart is considered for sound or non significantly weathered granite. Thus there is no remarkable decrease of the surface condition or the interlocking of the rock pieces with fracturing. In case of weathered granite, care has to be taken in the assignment of GSI values, owing to the enhanced heterogeneity that usually arises at the scale of the excavation, especially where

poorly interlocked rock masses with smooth planes (e.g. GSI of 30-35) may transpass irregularly to engineering soils (arrenites).

- *Ultrabasic rocks (ophiolites)*: In ophiolithic rocks (mainly peridotites, diabases) the characteristic is that, even where they are sound, their discontinuities may be coated by weak minerals that originate from alteration or dynamic metamorphosis. So they decline a bit to the right in the GSI chart comparing to a sound granitic mass. Ophiolites are often transformed to serpentinites which along with the tectonic fatigue may produce very weak masses.
- *Gneiss*: Compared to sound granitic masses a slight displacement of the assigned range downward and to the right of the GSI chart may be seen. Same comments as for the granite apply when gneiss is weathered.
- *Schists*: They vary from strong micaschists and calcitic schist types to weak chloritic, talcic schists and phyllites. The persisting schistosity planes and their usually "poor" surface conditions restrain the range of GSI values.

It is strongly underlined that the shaded areas illustrated in the charts are indicative and should not be used for design purposes as deviations may occur. But even for indicative cases or for rough approaches the use of mean values is not, again recommended. For design purposes it is obviously necessary to base the assessment on detailed site inspection and evaluation of all geological data derived from site investigation.

4.0 HETEROGENEOUS ROCK MASSES

The design of tunnels and slopes in heterogeneous rock masses such as *flysch* presents a major challenge to geologists and engineers. The complex structure of these materials, resulting from their depositional and tectonic history, means that they cannot easily be classified in terms of widely used rock mass classification systems.

Flysch consists of alternations of clastic sediments that are associated with orogenesis. It closes the cycle of sedimentation of a basin before the "arrival" of the poroxysmic folding process. The clastic material derived from erosion of the previously formed neighbouring mountain ridge. Flysch is characterised by rhythmic alternations of sandstone and fine grained (pelitic) layers. The fine grained layers contain siltstones, silty shales and clayey shales. The thickness of the sandstone beds range from centimetres to metres. The siltstones and schists form layers of the same order but bedding discontinuities may be more frequent, depending upon the fissility of the sediments.

Different types of alternations occur in the flysch series: e.g. predominance of sandstone, or typical sandstone/siltstone alternations, or predominance of siltstone. The overall thickness of the formation has often been reduced considerably by erosion or by thrusting. In fact, the formation is often affected by reverse faults and thrusts. This, together with consequent normal faulting, results in a significant degradation of the geotechnical quality of the flysch rock mass. Thus, sheared or even chaotic rock masses can be found at the scale of a typical engineering design.

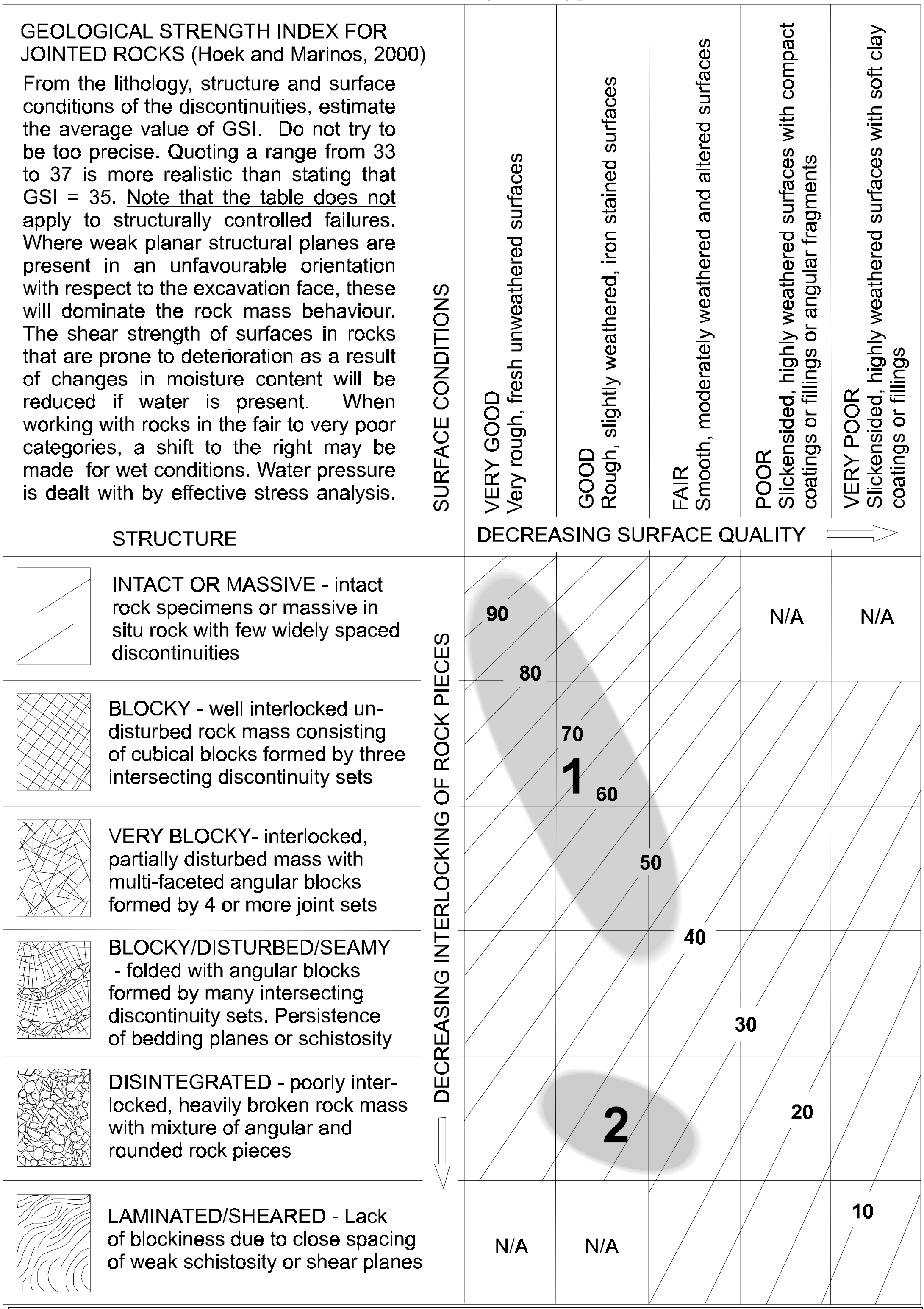
The determination of the Geological Strength Index for these rock masses, composed of frequently tectonically disturbed alternations of strong and weak rocks, presents some special challenges. However, because of the large number of engineering projects under construction in these rock masses, some attempt has to be made to provide better engineering geology tools than those currently available. Hence, in order to accommodate this group of materials in the GSI system, a chart for estimating this parameter has been developed recently (Marinos and Hoek, 2000) and is presented in Table 12.

4.1 Selection of σ_{ci} and m_i for flysch

In addition to the GSI values presented in Table 12, it is necessary to consider the selection of the other "intact" rock properties σ_{ci} and m_i for heterogeneous rock masses such as flysch. Because the sandstone layers or usually separated from each other by weaker layers of siltstone or shales, rock-to-rock contact between blocks of sandstone may be limited. Consequently, it is not appropriate to use the properties of the sandstone to determine the overall strength of the rock mass. On the other hand, using the "intact" properties of the siltstone or shale only is too conservative since the sandstone skeleton certainly contributes to the rock mass strength.

Therefore, it is proposed that a 'weighted average' of the intact strength properties of the strong and weak layers should be used. Suggested values for the components of this weighted average are given in Table 13.

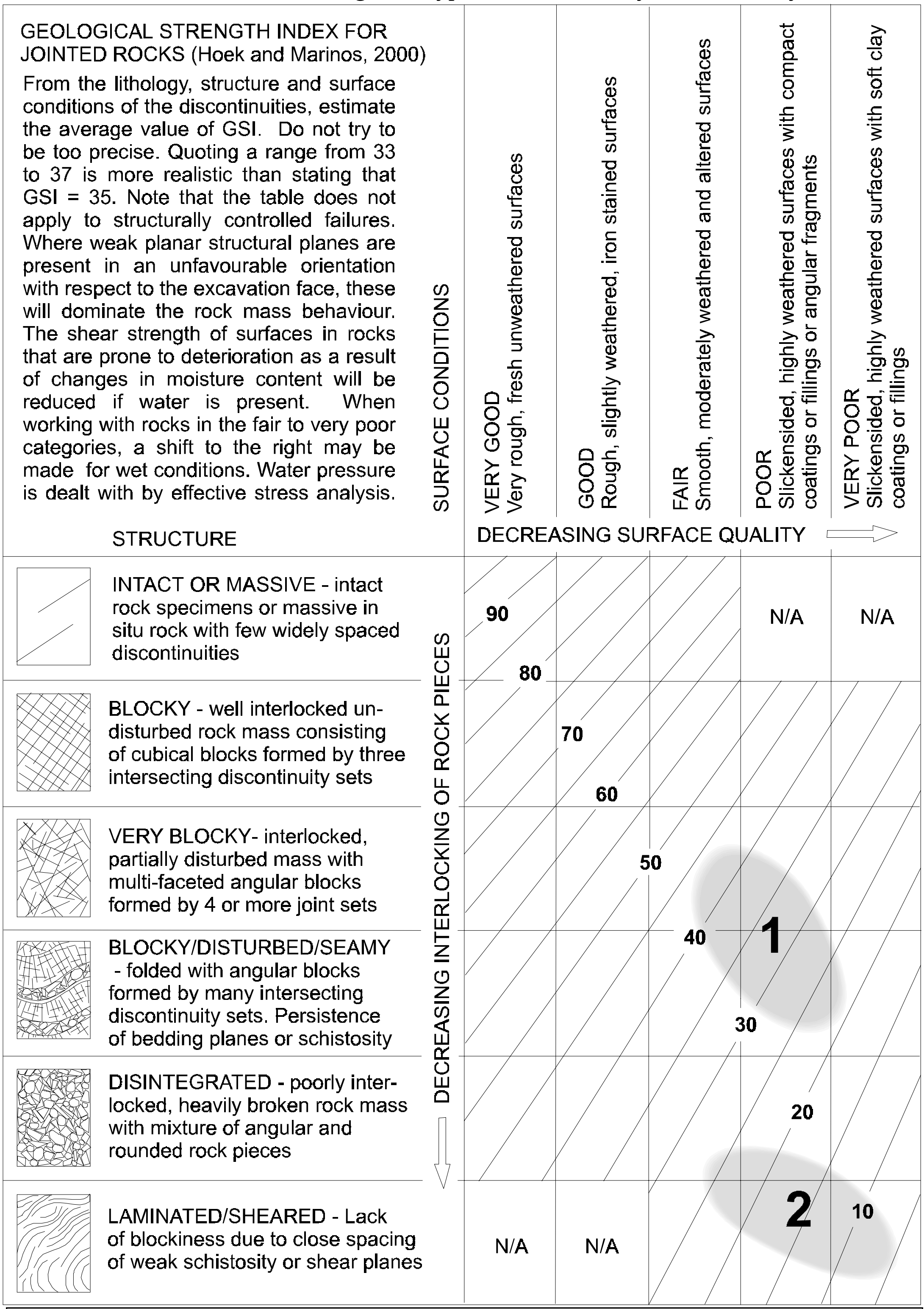
Table 5: Most common GSI ranges for typical sandstones.*



*_WARNING_:
The shaded areas are indicative and may not be appropriate for site specific design purposes.
Mean values are not suggested for indicative characterisation; the use of ranges is recommended

1. Massive or bedded (no clayey cement present)
2. Brecciated (no clayey cement present)

Table 6: Most common GSI ranges for typical siltstones, claystones and clay shales.*

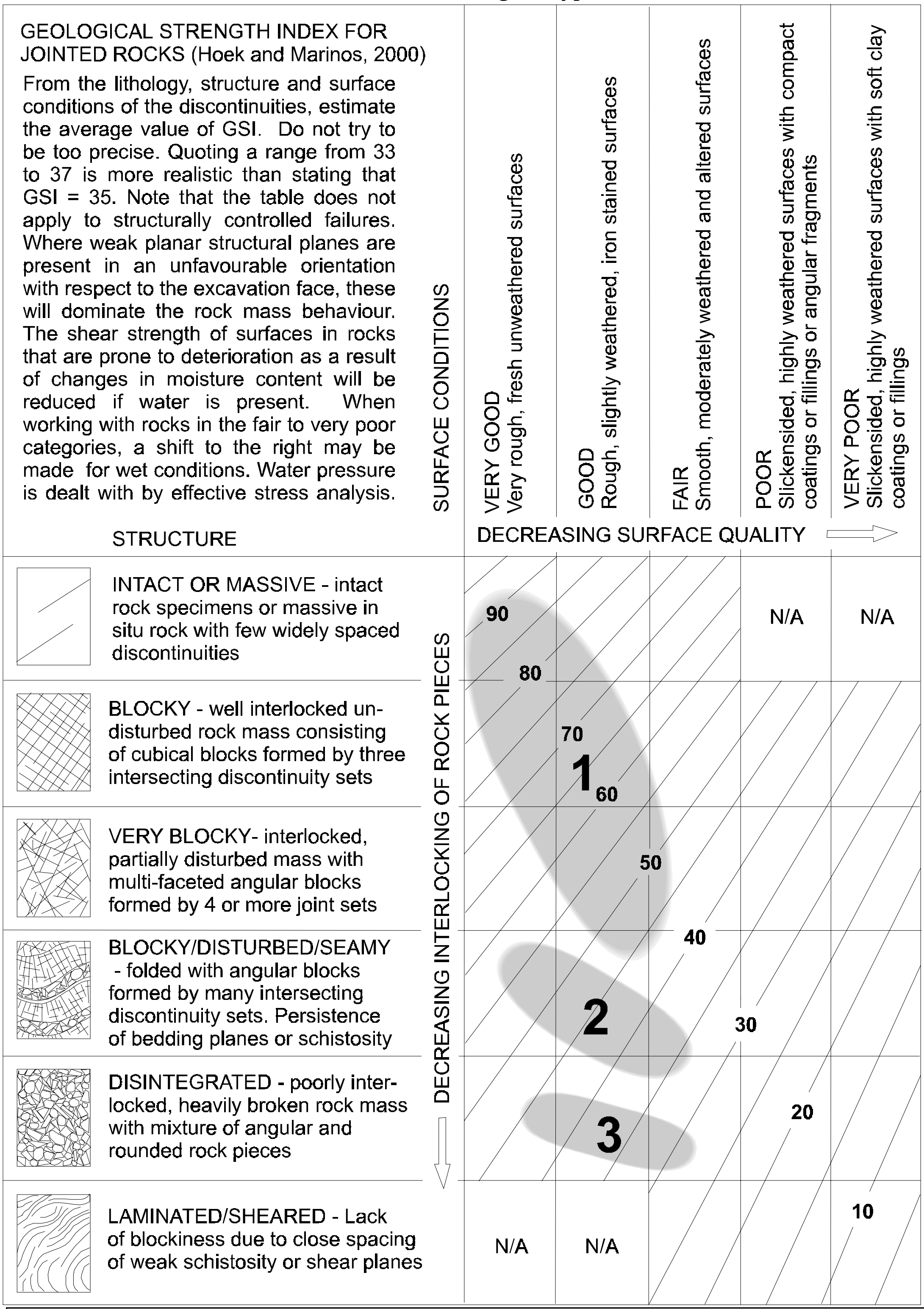


*<u>*WARNING*</u>:
The shaded areas are indicative and may not be appropriate for site specific design purposes. Mean values are not suggested for indicative characterisation; the use of ranges is recommended

1. Bedded, foliated, fractured
2. Sheared, brecciated

These soft rocks are classified by GSI as associated with tectonic processes. Otherwise, GSI is not recommended. The same is true for typical marls.

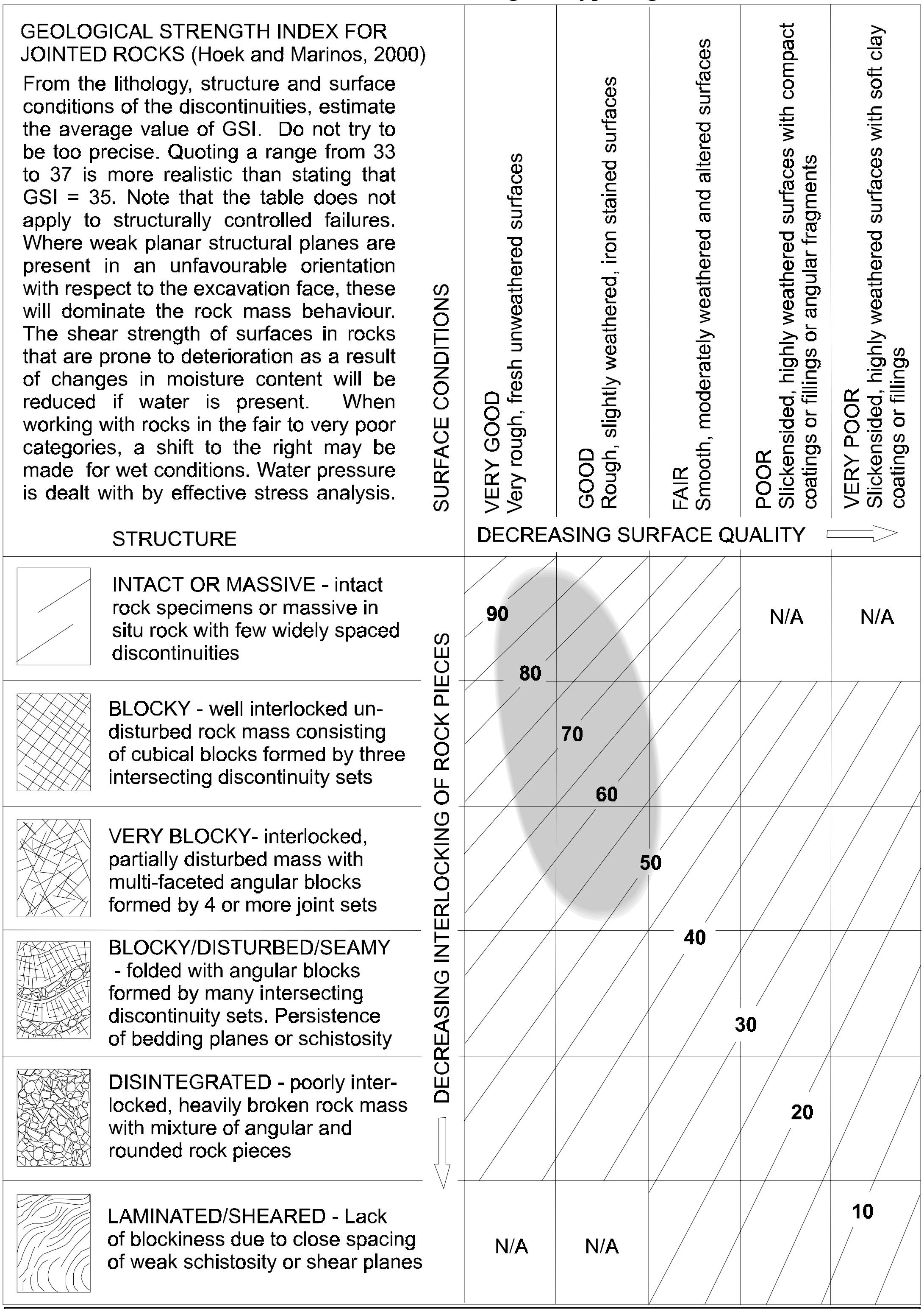
Table 7: Most common GSI range of typical limestone.*



*<u>*WARNING*</u>:
The shaded areas are indicative and may not be appropriate for site specific design purposes. Mean values are not suggested for indicative characterisation; the use of ranges is recommended

1. Massive
2. Thin bedded
3. Brecciated

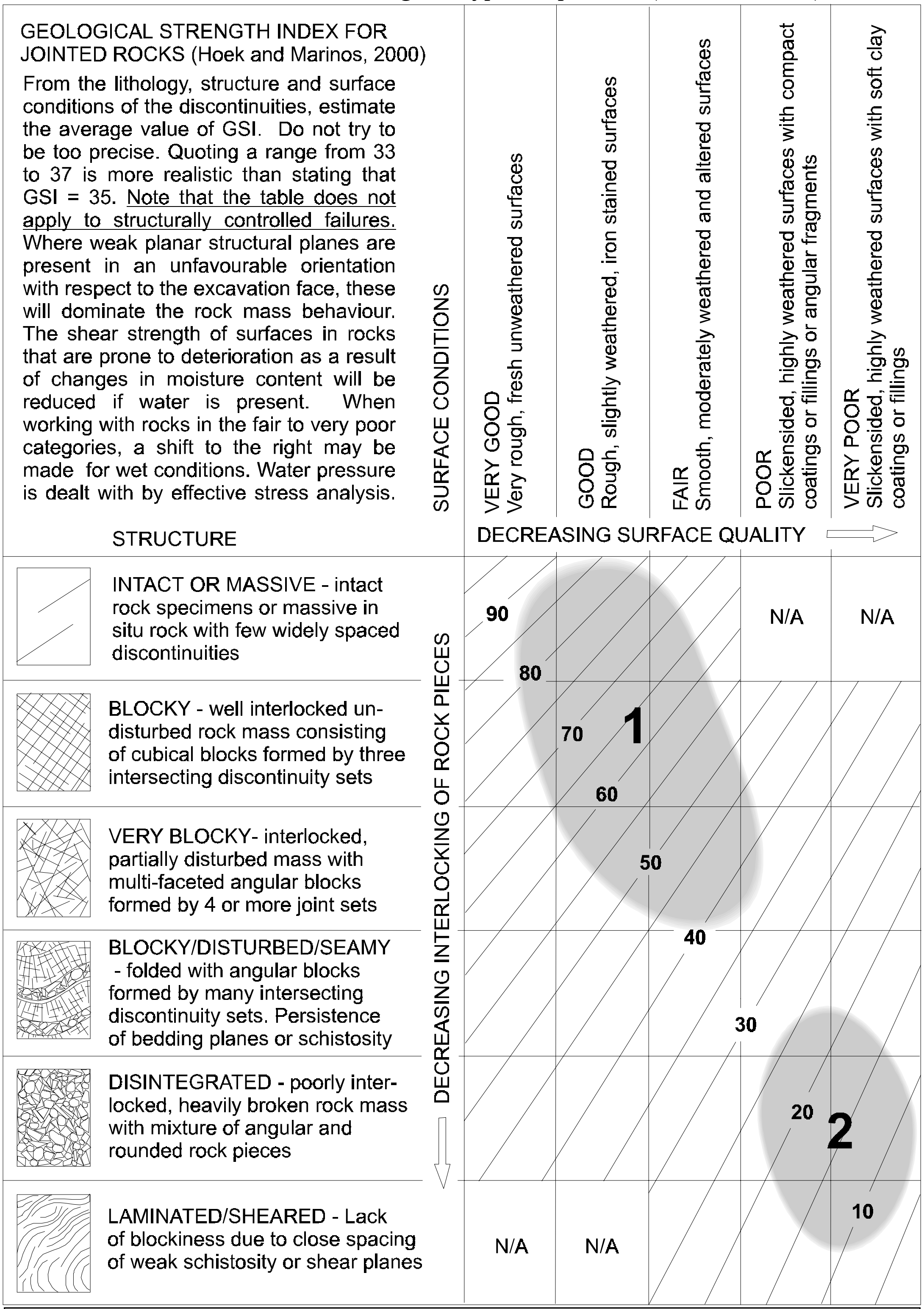
Table 8: Most common GSI range for typical granite.*



*_WARNING_:
The shaded areas are indicative and may not be appropriate for site specific design purposes.
Mean values are not suggested for indicative characterisation; the use of ranges is recommended

Only fresh rock masses are shown. Weathered granite may be irregularly illustrated on the GSI chart, since it can be assigned greatly varying GSI values or even behave as an engineering soil.

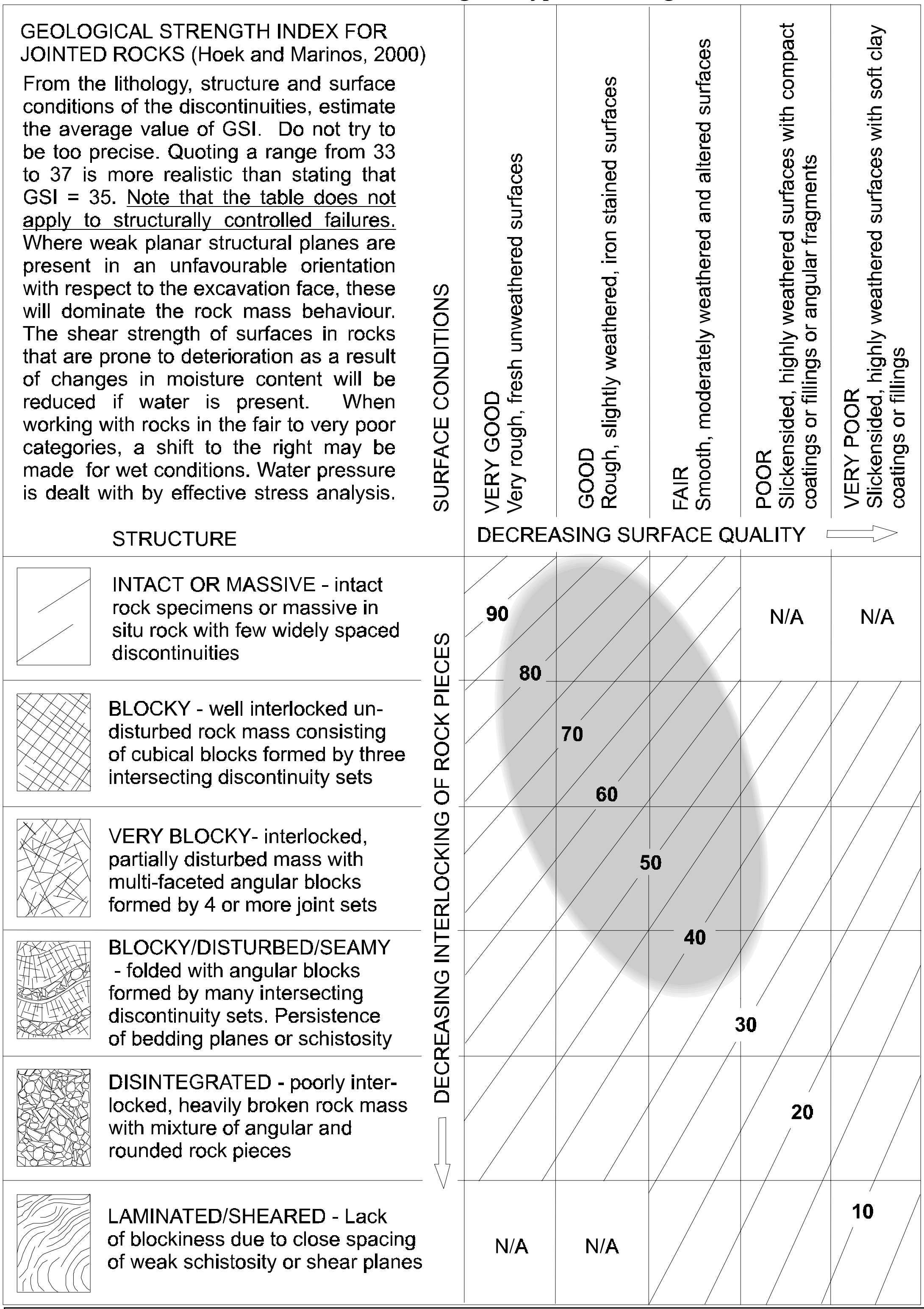
Table 9: Most common GSI range for typical ophiolites (ultrabasic rocks).*



*<u>*WARNING*</u>:
The shaded areas are indicative and may not be appropriate for site specific design purposes. Mean values are not suggested for indicative characterisation; the use of ranges is recommended

1. Fresh
2. Serpentinised with brecciation and shears

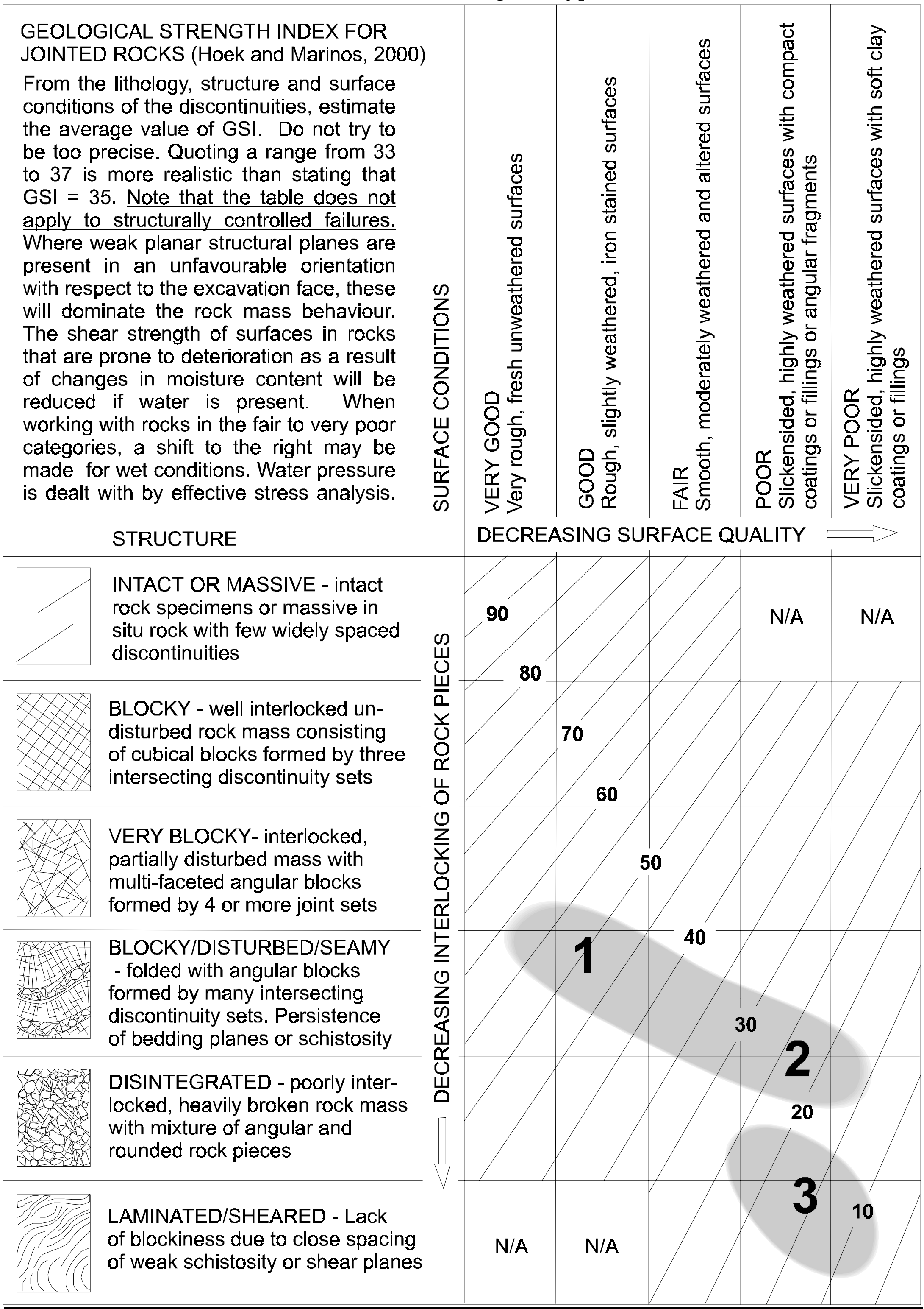
Table 10: Common GSI range for typical sound gneiss.*



*_WARNING_:
The shaded areas are indicative and may not be appropriate for site specific design purposes.
Mean values are not suggested for indicative characterisation; the use of ranges is recommended

Sound gneiss. Shaded area does not cover weathered rockmasses.

Table 11: Common GSI range for typical schist.*



*_WARNING_:
The shaded areas are indicative and may not be appropriate for site specific design purposes.
Mean values are not suggested for indicative characterisation; the use of ranges is recommended

1. Strong (e.g. micaschists, calcitic schists)
2. Weak (e.g. chloritic schists, phyllites)
3. Sheared schist

Table 12. GSI estimates for heterogeneous rock masses such as flysch.

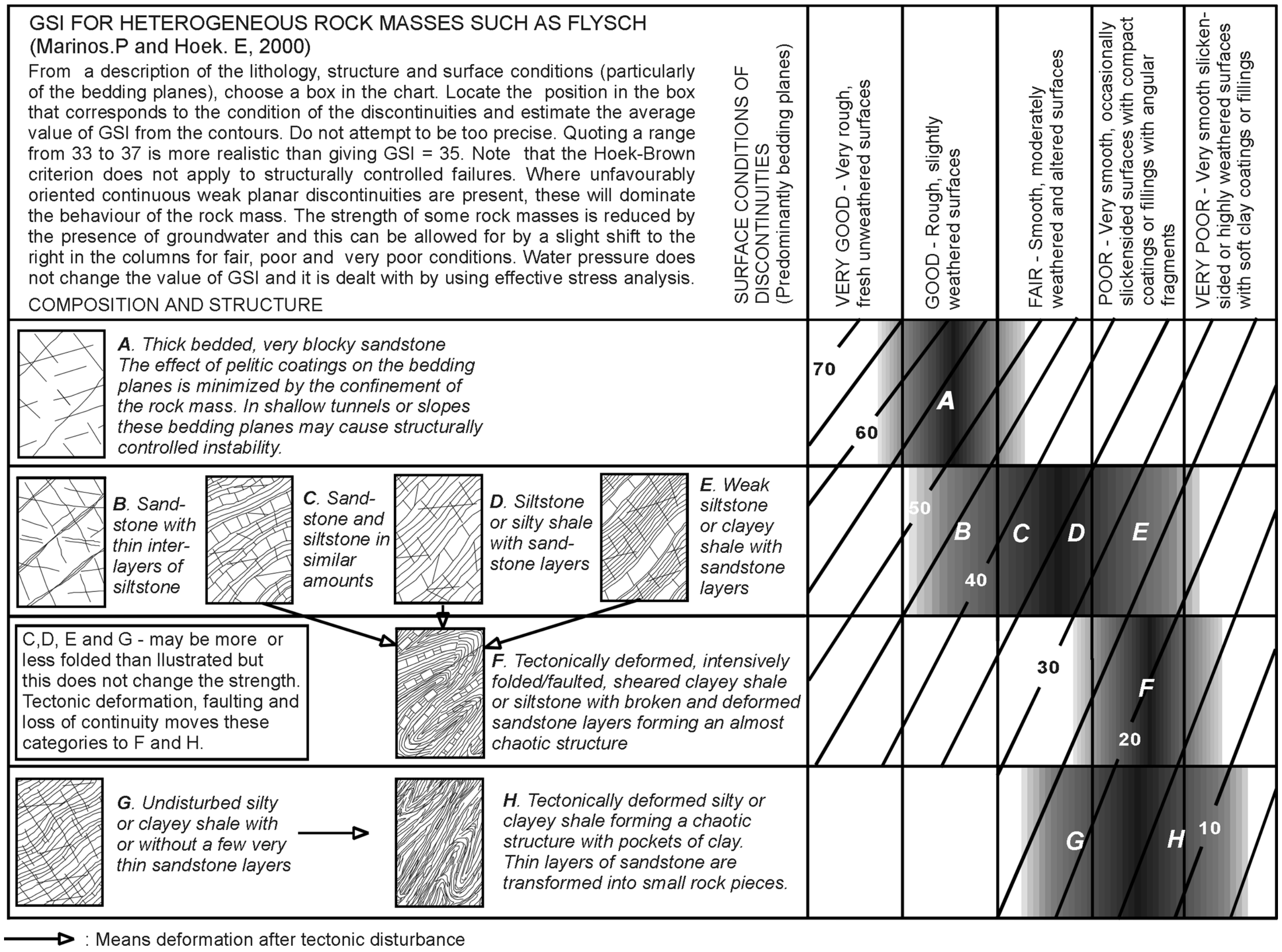


Table 13: Suggested proportions of parameters σ_{ci} and m_i for estimating rock mass properties for flysch (Marinos, P., Hoek, E., 2000).

Flysch type see Table 12	Proportions of values for each rock type to be included in rock mass property determination
A and B	Use values for sandstone beds
C	Reduce sandstone values by 20% and use full values for siltstone
D	Reduce sandstone values by 40% and use full values for siltstone
E	Reduce sandstone values by 40% and use full values for siltstone
F	Reduce sandstone values by 60% and use full values for siltstone
G	Use values for siltstone or shale
H	Use values for siltstone or shale

5.0 ACKNOWLEDGEMENTS

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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).

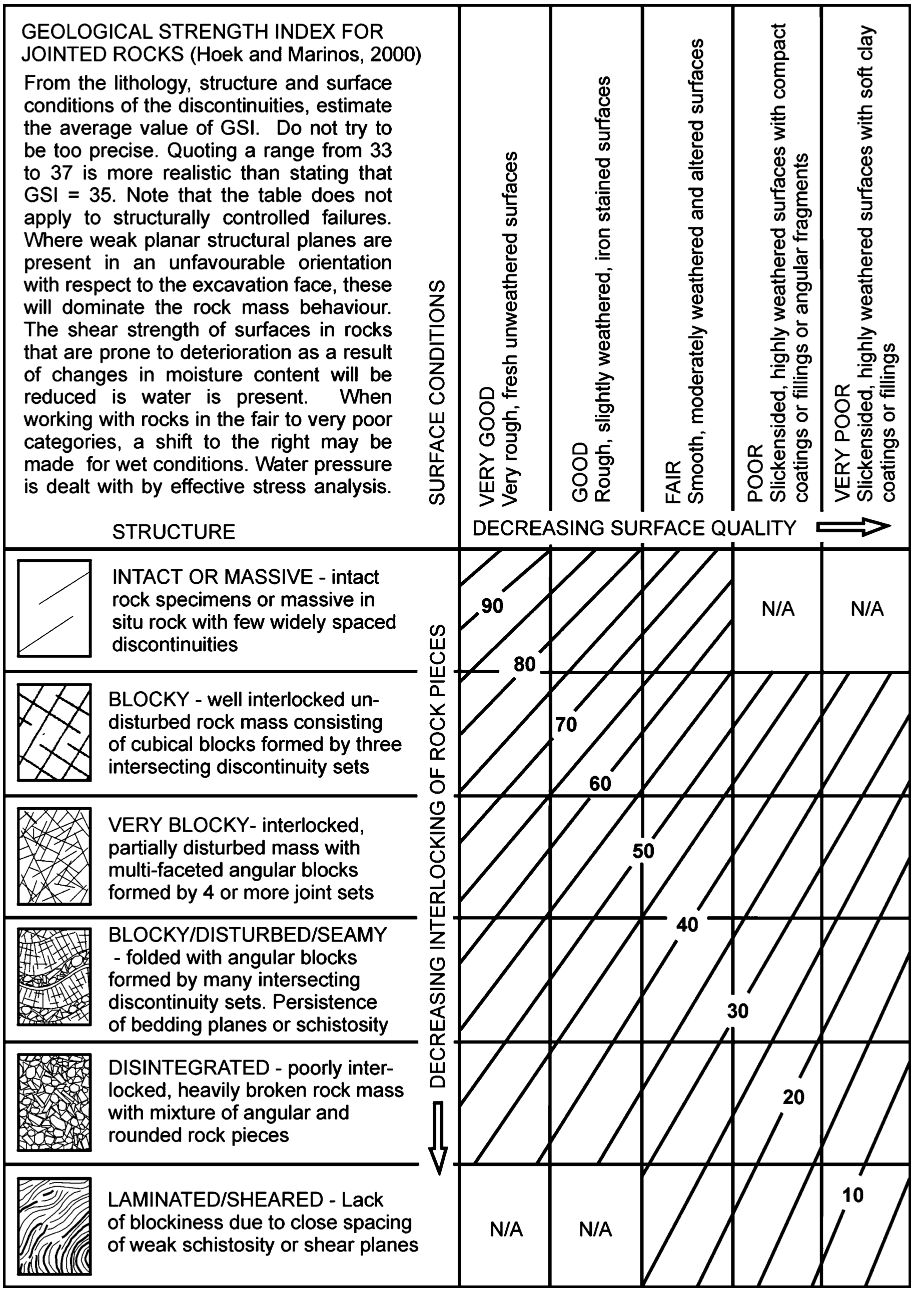

Fig. 1 General chart for GSI estimates from the geological observations

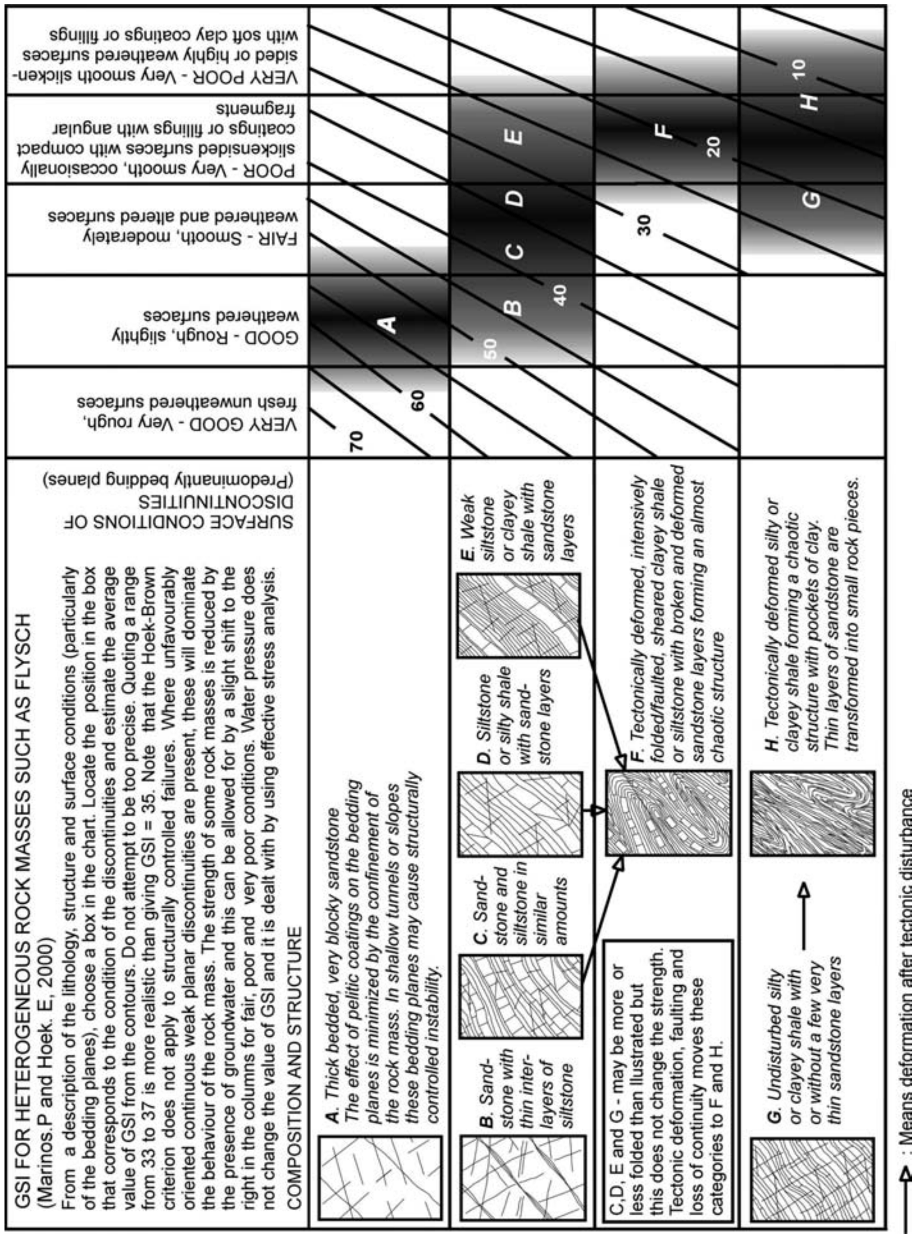

Fig. 2 Geological strength index estimates for heterogeneous rock masses such as Flysch

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

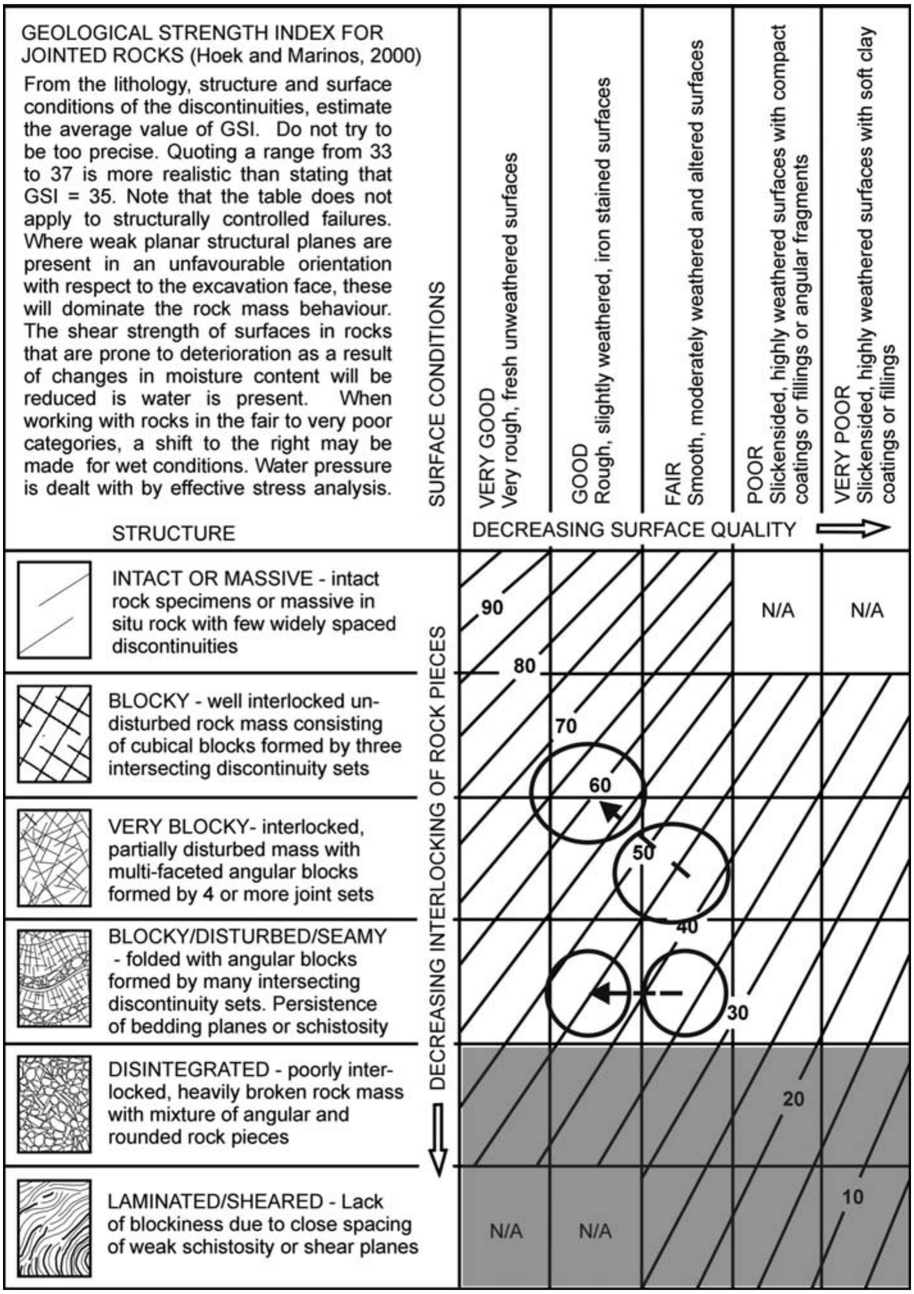

Fig. 3 Suggested projection of information from observations in outcrops to depth. *White area*: a shifting to the left or to the left and upwards is recommended; the extent of the shift shown in the chart is indicative and should be based on geological judgement. *Shadowed area*: shifting is less or 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

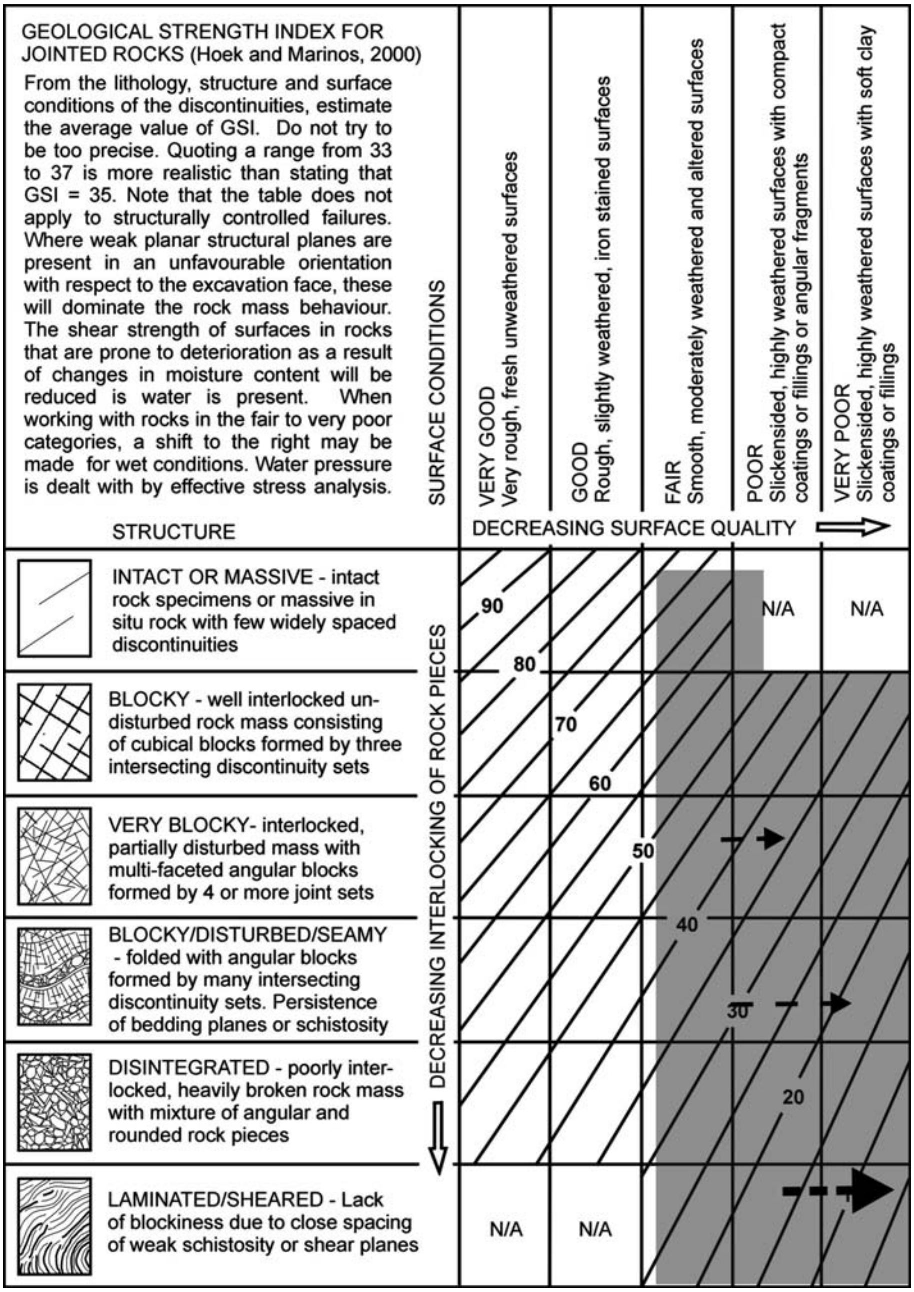

Fig. 4 In fair to very poor categories of discontinuities, a shift to the right is necessary for wet conditions as the surfaces of the discontinuities or the filling materials are usually prone to deterioration as a result of change in the moisture content. The shift to the right is more substantial 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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ORIGINAL PAPER

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Variability of the engineering properties of rock masses quantified by the geological strength index: the case of ophiolites with special emphasis on tunnelling

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Abstract The paper presents a quantitative description, using the Geological Strength Index (GSI), of the rock masses within an ophiolitic complex including types with large variability due to their range of petrography, tectonic deformation and alteration. This description allows the estimation of the range of rock mass properties and the understanding of the dramatic changes in behaviour which can occur during tunnelling, from stable conditions to severe squeezing within the same formation at the same depth. The paper presents the geological model in which the ophiolitic complexes develop, their various petrographic types and their tectonic deformation, mainly due to overthrusts. The structure of the various rock masses includes all types from massive strong to sheared weak, while the conditions of discontinuities are in most cases fair to poor or very poor due to the fact that they are affected by serpentinisation and shearing. Serpentinisation also affects the initial intact rock itself, reducing its strength. Associated pillow lavas and tectonic mélanges are also characterised. Based on the GSI, a classification of the behaviour in terms of tunnelling is presented, including stable conditions, structural instability, mild overstressing, stress dependant instability, squeezing and ravelling.

Keywords Ophiolites · Rock mass classification · Geological strength index · Tunnels

Résumé Une description quantitative des massifs rocheux des complexes ophiolitiques est présentée par le moyen de l'index GSI. Les ophiolites forment un cas particulier à cause de leur variété pétrographique, leur déformation tectonique et leur altération. Cette description permet l'estimation des propriétés géotechniques et la compréhension des différents types de comportements souvent très variables rencontrés lors du creusement de tunnels. L'article discute brièvement le modèle géologique de ces formations, leurs variétés pétrographiques et leur déformation à cause surtout des charriages. La structure des massifs rocheux ophiolitiques inclut tous les types, (du milieu continu au cisaillé), tandis que l'état des joints est toujours faible à cause de la serpentinisation de leurs épontes. La serpentinisation peut aussi affecter la masse entière de la roche saine. Une classification du comportement en tunnel est présentée basée sur l'index GSI: conditions stables, instabilité structurale, instabilité due à des convergences.

Mots clés ophiolites · massif rocheux · index GSI · tunnels

Introduction

Over the past decades, rock mass classification methods have been developed in order to meet the needs for designing specific projects. The rock mass classification systems of Bieniawski (1973) and Barton et al. (1974) were originally developed to provide guidance on the selection of support for tunnels in blocky rock masses and they played an important role in the expansion of the tunnelling industry.

During the last decade, the development of ''user-friendly'' software has provided alternative design tools that are more appropriate in many cases. 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 classification schemes for the characterisation of rock masses has been developed. A system that is now widely used is the Geological Strength Index (GSI) developed by Hoek (1994) and extended by Hoek et al. (1998), Marinos and Hoek (2001) and Hoek et al. (2005) to incorporate weak, heterogeneous rock masses and rock masses with lithological variability. This classification system is used in conjunction with the Hoek and Brown failure criterion to estimate the geotechnical parameters of rock masses which fall within the range specified by the authors. A presentation and a discussion on the use of GSI can be found in Marinos and Hoek (2000) and more recently in Marinos et al. (2005) hence there is no need to repeat the details here.

The GSI has considerable potential for use in rock engineering because it permits the manifold aspects of rock to be quantified, enhancing geological logic and reducing engineering uncertainty. Its use allows the influence of the variables which make up a rock mass to be assessed and hence the behaviour of rock masses has to be explained more clearly. One of the advantages of the Index is that the geological reasoning that it embodies allows adjustments of its ratings to cover not only a wide range of rock masses and conditions but also variations that may develop within the same rock type. It also allows the limits of its application to be understood.

This paper presents a quantitative description, through the GSI, of the rock masses of a particular type of geological formation with both petrographic variety and structural complexity due to tectonic deformation and alteration. This description allows an estimation of the variation in rock mass strength and how much this rock mass strength can be reduced by shearing or alteration as well as an understanding of the dramatic changes in behaviour where, in tunnelling, stable conditions and severe squeezing can occur within the same formation at the same depth.

Ophiolites: geological model

Setting

The term ophiolite was initially given to a sequence of mafic (basic) and ultramafic (ultrabasic) rocks, more or less serpentinised and metamorphosed, occurring in the Alpine chains. These complexes were considered, not long ago, as enormous submarine volcanic effusions inside which magmatic differentiations took place sheltered by a cuirass of pillow lavas. Although this can be the case in some regions, ophiolites are at present considered as pieces of the oceanic crust generated at an oceanic ridge and the upper mantle of an ancient ocean, thrust up on the continental crust during mountain building (e.g. collision between two continents or between a continent and an insular arc; see Fig. 1). They can exhibit sections of more than 10 km in thickness which leads to the conclusion that not only the oceanic crust (6–7 km) but also part of the mantle are included in the process (Debelmas and Mascle 1997).

The ophiolitic complex

The ophiolitic sequence (or complex in order to emphasise the diversity of materials) is fundamentally characterised by underlying peridotitic rocks which are covered by gabbroic/peridotitic rocks which, in turn, are covered by basalts or spilites. The basal peridotites are foliated (''tectonites''). The subsequent alternations of peridotites and gabbros often have a layered structure of cumulates and are followed by massive gabbros, norites or other basic rocks richer in SiO_2. The overlying basalts are either massive or in the form of pillow lavas. In between these lavas sedimentary rocks may be deposited. In Fig. 2 a synthetic and theoretical column of an ophiolitic complex is presented. The succession is idealised and in many cases some members may be absent, as for instance the volcanic lava at the top.

This geometry is highly disturbed as the ophiolitic complexes occur mainly in tectonic zones with superposition of numerous overthrusts. Metamorphism, which is also present, changes the original nature of the materials. The high degree of serpentinisation and the intensity of shearing can make it difficult to identify any initial cumulate texture or fabric (Skemp and McCraig 1984).

Serpentinisation

Serpentinisation is the transformation of ferromagnesian minerals, olivine in particular, to serpentine—a

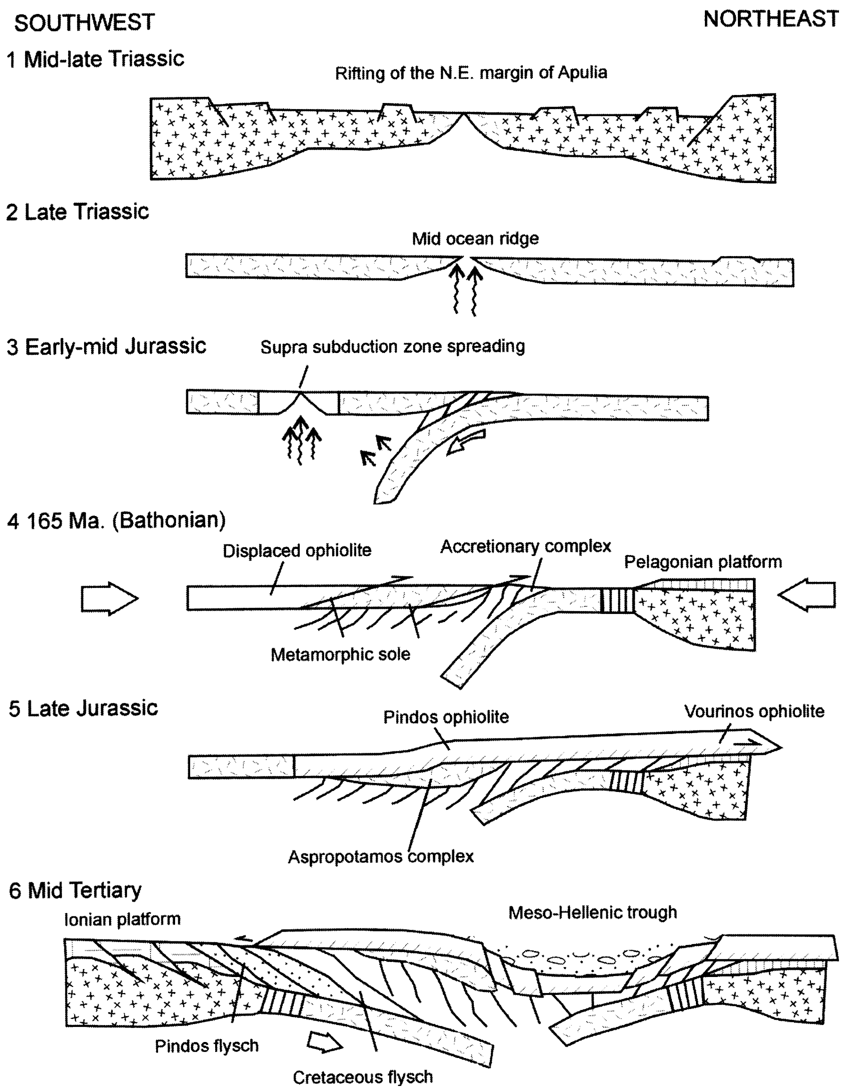

Fig. 1 Tectonic model for the evolution of the Pindos and Vourinos mountain ophiolites in Northern Greece (modified from Jones et al. 1991, in Pe-Piper and Piper 2002)

lattice mineral of either fibrous or laminar form. This unusual alteration is a phenomenon of autohydratation which takes place during the last phases of the crystallisation of magma where there is an excess of water. Thus it can be considered as a type of autometamorphism. In other cases the serpentinisation corresponds to a low grade metamorphism of peridotites (Foucault and Rault 1995). In all these cases the peridotites can be transformed into serpentinite. This new rock is originally compact, relatively soft and more easily sheared by tectonic processes.

Serpentinisation can also be developed under exogenic conditions with meteoric water under usual weathering processes. In this case the alteration disintegrates the parent peridotite to a clayey soil-like mass. The development at depth of weathered peridotites is less generalised and obviously limited compared with the endogenic serpentinisation described previously.

Pillow lavas

Pillow lavas, usually of basaltic or andesitic composition, have been extruded under water and consist of a mass of more or less ellipsoidal bodies each with a billowy surface. The pillows range from 10 cm to a few metres in diameter and lie merged with one another, not unlike an irregular collection of “sofa pillows” (in Visser 1980). Radial joints are conspicuous in cross sections, forming radial columnar fragments (Pantazis 1973).

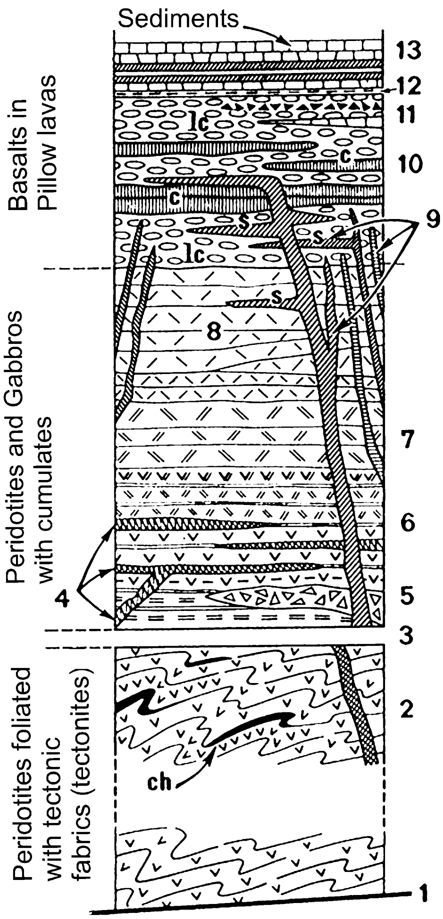

Fig. 2 Ophiolites: synthetic and theoretical column (from Foucault and Raoult 1995 with simplified descriptions). *1* basal contact, overthrust; *2* basal body of peridotites with foliation (tectonites), generally harzburgites with chromites (*ch*) layered with dunites; *3* tectonic cut or confused zone, *4* dykes, sills and layers of basic and ultrabasic rocks; *5* layered peridotites (cumulates of dunites, lherzolites) and magmatic breccia; *6* alternations of peridotites with gabbros, *7* non layered gabbros over layered gabbros, *8* variety of basic rocks, dolerites, diorites and granophyres (increase of SiO_2); *9* dykes, veins, sills (*s*) of basic rocks; *10* and *11* basalts (spilites) with compact flows (*c*) or pillow-lavas (*lc*), a tectonic contact is often present at the base of *10*, *12* Argilites rich in Fe and Mn, *13* sedimentary rocks with siliceous beds (radiolarites) over or interlayered with the lavas (volcano–sedimentary complex). Total thickness: usually 4–5 km, can achieve 10–15 km (as an example: 11 and 10 = 0, 5–1 km, 8 = 0, 5 km, 7 and 6 = 0, 5 km, 5 = 0, 2 km, 2 = 2–3 km). These numbers can change dramatically due to overthrusts

Mélanges

As ophiolites are associated with large-scale overthrusts, tectonic mélanges can be formed in the base and at the front of such megastructures. These ophiolitic mélanges contain ophiolitic rocks and other rocks of various paleogeographic origins; the whole entity being in considerable tectonic disorder with chaotic masses where blocks and packages of various sizes of any kind of rock (sedimentary or volcanic) ‘‘float’’ inside a sheared soil-like mass.

Sketches of models around the world

Most of the ophiolites belong to the Alpine cycle (their age ranges from 180 to 60 MA) but older deposits are also known (e.g. in the Apalaches—Paleozoic, or in Maroc—Precambrian) (Debelmas and Mascle 1997).

Figures 3, 4 and 5 show schematically the general geological model of the ophiolitic formations in a number of mountain chains of the world, where petrographic and tectonic complexities exist (Fig. 6).

Geological engineering characterisation of various rock masses in the ophiolitic bodies

The geotechnical parameters

From the discussion on the geological model of ophiolites it is clear that this formation contains a variety of rock types with geotechnical qualities varying from excellent to fair, becoming poor to very poor when serpentinisation is extensive and/or shearing present. These last processes are both very frequent in the ophiolitic complexes.

The main fundamental types are peridotites, gabbros, peridotites more or less serpentinised, serpentinites, schisto-serpentinites, sheared serpentinites, pillow lavas and chaotic masses in ophiolitic mélanges.

In this section the engineering geological characterisation of these various rock type areas are discussed. The discussion concludes with the assignment of the range of values of the (GSI) which are most likely to occur for the fundamental types of rock masses occurring in the ophiolites. The field data are from outcrops, cuts in slopes, borehole cores and tunnel excavations from various significant ophiolitic complexes and mélanges in northern and central Greece. It is hoped that this assignment is not simply site specific but of general value and can be applied in a more universal way, given the similarities between the geological setting of numerous ophiolitic complexes of the Alpine cycle around the world.

The GSI values characterising the various masses, together with the strength of the intact rock, σ_{ci}, and the petrographic parameter, m_i, allow the geotechnical parameters of strength and deformability of the various rock masses to be estimated with a level of accuracy which is generally adequate for engineering design. For this estimation the program RocLab can be used. The program can be downloaded free from http://www.rocscience.com.

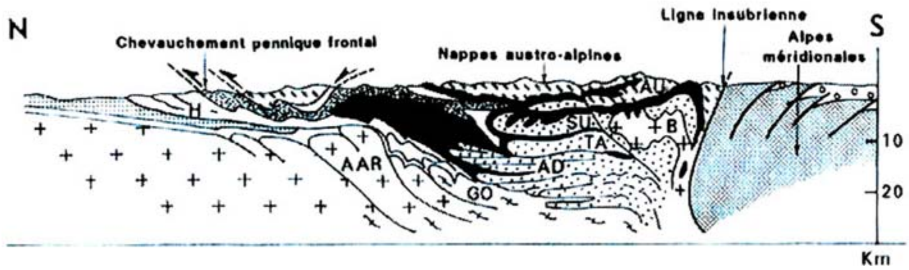

Fig. 3 Schematic section of nappes (overthrusts) in central Alps. Nappes of Mesozoic ophiolites in *black* (details in Pollino et al. 1990 in Mercier and Vergely 1999)

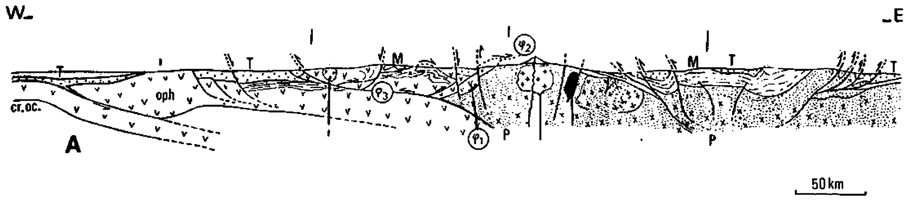

Fig. 4 The ophiolites (*oph*) in the context of Andes. Model in the Colombian Andes. (details in Megard 1987)

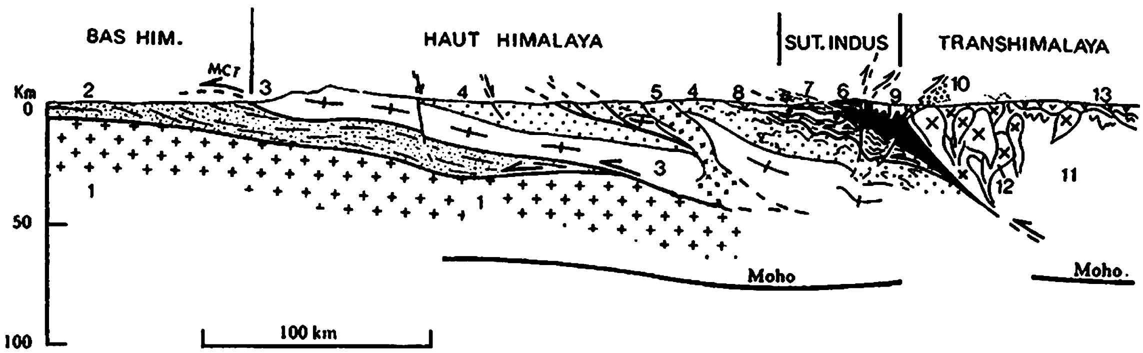

Fig. 5 Simplified section in the Himalayas with ophiolites in *black* (details in Bassoulet et al. 1984)

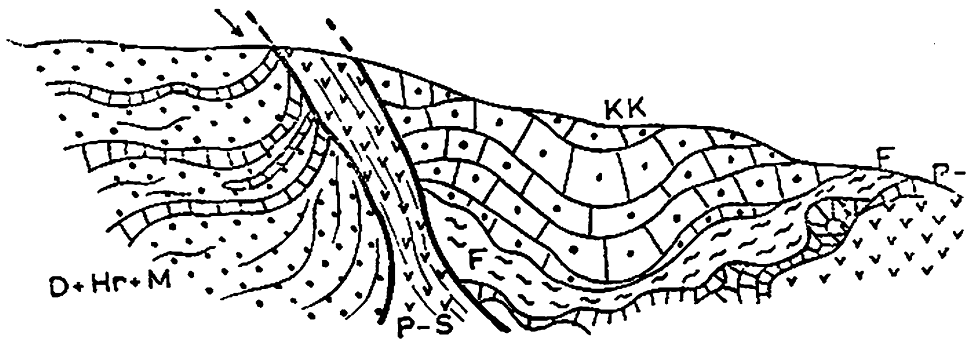

Fig. 6 Geological sketch in Orthrys mountain, central Greece with ophiolites (P–S) in tectonic relation with sedimentary series of rocks (limestones and siliciferous schists) (from Marinos 1974). Scale of tens of meters

Peridotites

Peridotites (hartsbourgites, dunites etc.) are strong with a range of unconfined strength for the intact mass from many tens of MPa to more than 100 MPa at which stage they behave as typical brittle materials. Koumantakis (1982) gives mean values of about 90 MPa from tests on 130 samples of peridotites, more or less serpentinised, from various locations in Greece. The values of σ_{ci} used in tunnelling design approaches in Greece are at least 50 MPa. Their tectonic disturbance is expressed in terms of intersecting joint sets distributed in accordance with the state of stress under which they were developed.

Serpentinisation, as a result of a typical weathering process or a process of alteration from endogenic causes, can be present on the surface of discontinuities. In such

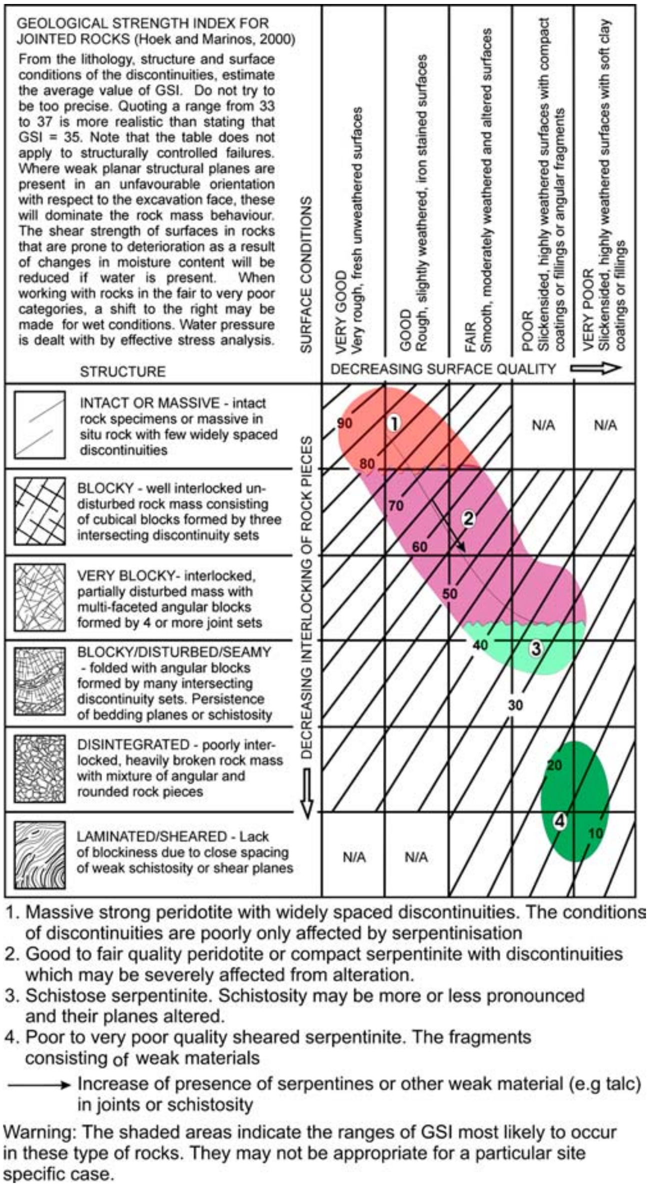

Fig. 7 Ranges of GSI for various qualities of peridotite–serpentinite rock masses in ophiolitic complexes

Fig. 8 Good quality peridotite from the mountain of Orthrys in central Greece. The conditions of the widely spaced discontinuities are only mildly affected by serpentinisation. Tunnel stability is controlled by occasional structural failures. Depth of the cores in the photograph about 380 m. GSI 80 ± 5

Fig. 9 Good quality surface outcrop of blocky structure in peroditite with fair (smooth, moderately weathered and altered) surface conditions of discontinuities. Width of photograph about 1.5 m. GSI$\sim$55

cases the initial rough conditions of the joints are dramatically reduced to poor or very poor with coatings of smooth and slippery minerals such as serpentine or even talc.

The range of GSI for peridotitic types of rock masses of the ophiolitic complex is shown in Fig. 7 (areas 1 and 2). The rock mass can be almost massive, with only a few widely spaced discontinuities, even close to the surface in tectonically quiet areas or in zones of "tectonic shadow". High values of GSI are to be attributed to this type of peridotitic mass (GSI greater than 65—area 1 in Fig. 7). Figure 8 shows representative cores of this good quality peroditite. Figures 9 and 10 show outcrops of sound and weathered peridotite.

When the rock mass is jointed or fractured the GSI values drop as low as 35, not only due to a disturbed structure but also because of the conditions of the discontinuities which become smooth and slippery due to serpentinisation. In a disturbed peridotitic mass, the serpentinisation process often affects and disintegrates parts of the rock, not only contributing to lower GSI values but also reducing the intact strength values. Such

Fig. 10 Weak weathered ophiolitic outcrop of serpentinised peridotite

Fig. 11 Fair quality peridotite, from the mountain of Orthrys in central Greece, with discontinuities of low frictional properties due to the presence of films of seprentinised material. Blocky-jointed with short lengths of disintegrated or serpentinised sections. Tunnel stability will be controlled by structural stability of small blocks or by mild overstressing. Depth of the cores in photograph about 200 m. GSI 35 ± 5

disturbed peridotites fall in the lower bound of area 2 of the GSI diagram of Fig. 7 and are shown in Fig. 11.

Gabbros

The gabbros follow the same principles in their engineering geological characterisation as all strong rocks. If they are sound their behaviour depends on their degree of fracturing. Their discontinuities can have better conditions than those of the peridotites as they suffer less from alteration. When weathered, the disintegrated materials contain clay which may be highly expansive.

Serpentinites

When the serpentinisation is due to weathering which has affected all of the mass, in addition to the reduction of the intact strength there is a dramatic disintegration of the structure of the rock mass. If this process of serpentinisation is due to autometamorphism and/or associated with tectonic thrust, the rock mass is poor, with a schistose disturbed structure which may reduce the GSI to values to 30 or less (area 3 in the GSI diagram of Fig. 7).

Measuring the strength of the intact rock, σ_{ci}, from such rock masses is always a problem. When testing schisto-serpentinites, the influence of "schistosity" results in a significant reduction in the strength of a large proportion of the specimens. Consequently, it is very difficult to obtain reliable values for σ_{ci} from laboratory tests and it is suggested that the uniaxial compressive strength of the schisto-serpentinite should be estimated from that of the normal serpentinite and reduced by about 30% to account for the schistosity. From cases in northern Greece it is considered that 40 MPa may be a realistic value for the uniaxial compressive strength (σ_{ci}) of the serpentinite (Fig. 12) and 30 MPa for the schisto-serpentinite. Koumantakis (1982) gives values of 45 MPa from 12 samples of serpentine.

It is essential to differentiate between intact rock strength and rock mass strength as this has a significant impact on the assessment of potential tunnelling conditions. Assigning the rock mass a low value of GSI and a low value of intact strength penalises the rock mass twice and results in too low a value of the estimated rock mass strength.

Fig. 12 Compact serpentinite

Fig. 13 Poor quality sheared serpentinite from the mountain of Orthrys in central Greece. Completely disintegrated peridotite with loss of blockiness and presence of clayey sections. Tunnel stability will be controlled by stress dependent rock mass failure with significant squeezing at depth. Depth of samples in photograph about 175 m, GSI 15–20

Fig. 14 Piece of sheared ophiolite, which has been disintegrated into flakes of weak serpentinite

Sheared serpentinites

In the sheared zones of serpentinites there is a lack of blockiness, which allows the rock to disintegrate into slippery laminar pieces and small flakes of centimetres or millimetres in size. GSI values can drop to less than 20 (Fig. 7, area 4). Such sheared serpentinite is shown in Figs. 13 and 14. The intact strength may vary from 20 to 5 MPa or less.

In a recent paper, Glawe and Upreti (2004) illustrate and discuss the differences that occur in two serpentinites, one in Turkey and one in Indonesia. Varying strength values may result from differences in local lithological factors, micro- and macro- structures, mineralogical compositions, variations in interlocking of smaller grains, sheared and angular rock fragments and re-cementation of matrix in serpentinite with bimrock fabric (Glawe and Upreti 2004).

Fig. 15 Basic volcanics in pillow lava structure. Blocky disturbed with poor condition of discontinuities (highly weathered with coatings or fillings). GSI approximately 30 in this particular site. A thin shear plane is also present (indicated by the notebook)

Pillow lavas

Pillow lavas of basaltic nature exhibit exfoliated spherical zones. Friable or sheared material surrounds a stronger main mass thus reducing the overall quality (Fig. 15). The condition is particularly likely to occur in areas of low overburden and in the tectonic zones of the ophiolitic complex. Thus, the quality can be poor only when weathering and tectonic shearing is generalised. The range of the geotechnical quality of the pillow lava given in terms of GSI may vary from 50 to 25. The GSI chart for these pillow lava structures is given in Fig. 16. Occasional shear planes occur (Fig. 15) and in tunnelling these could result in local structural failures unless adequately supported. Under such conditions tunnelling in this mass may be fair to good and only poor when the rock mass is at its lower bound (sheared and weathered). However, it will be essential to maintain confinement during the excavation procedures in order to optimise the temporary support.

Mélanges

Low to very low GSI values can be attributed to masses in ophiolitic mélanges where, as discussed earlier, rocks of the ophiolitic sequences are mixed in complete

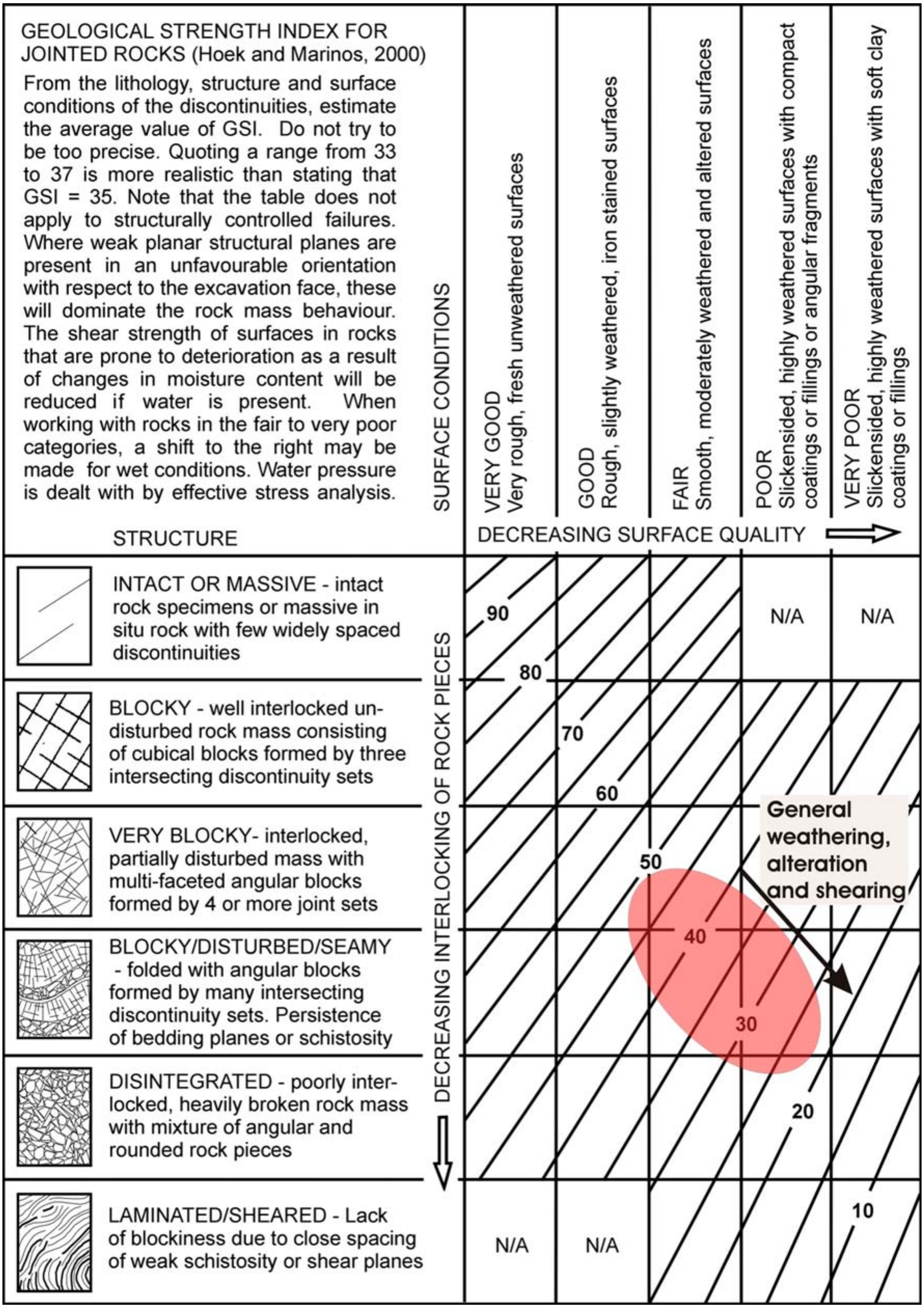

Fig. 16 Range of GSI ratings for basaltic pillow lavas. (Warning: the *shaded area* indicates the ranges of GSI values most likely to occur in this type of rock. It may not be appropriate for a particular site specific case)

disorder with other rocks of various origins and are situated at the base or in the front of great ophiolitic overthrust nappes. In soil-like material a GSI assignment is meaningless. However, it is possible that during tunnelling inside these masses, extensive blocks of sedimentary rocks of good engineering quality (e.g. limestones or sandstones) can be encountered. Nevertheless, the transition to the surrounding sheared rock mass of either ophiolitic or other clayey sedimentary rocks (flysch, siltstones, argillites) is sharp and unpredictable. Probe drilling ahead of the face is always prudent when tunnelling in such conditions.

The m_i values

In the Hoek and Brown failure criterion the m_i value reflects the frictional characteristics of the component minerals and grains of the intact rock. In the ophiolitic rocks in the Greek Alpine context the m_i values can be: peridotites more than 20; schistose serpentine: 12 ± 2; altered material due to shearing: 8 ± 2.

Behaviour in tunnelling

The great variety of the rock mass types, the irregular changes and the alteration make the ophiolites a formation where extreme care is needed in the design of any engineering structure founded on or crossing them. This is particularly true for tunnels as their linearity and their depth increase the possibility of encountering the adverse conditions and weak zones associated with the ophiolites while the uncertainty as to their occurrence and extent exacerbate the difficulty. In Fig. 17 the tunnelling behaviour of peridotites—serpentinites is classified following the characterization of their rock masses discussed in the previous section and shown in Fig. 7.

Peridotites

In good quality masses of peridotite, simple straight forward tunnelling conditions can be expected. Attention has to be concentrated on avoiding structural instabilities from wedges. For these structurally controlled failures involving only a few discontinuities, the problem is essentially one of three-dimensional geometry and stereographic techniques or numerical analyses such as Unwedge (see http://www.rocscience.com) should be used for an analysis of failure processes and the design of reinforcement. However, compared with other rock masses of similar structure, the peridotites generally have smoother discontinuities with low frictional properties. As explained earlier this is due to the presence of serpentinised material which is very often present even if the serpentinisation has not affected the fundamental rock material. This makes the structurally dependant instability more critical and generally demands heavier rock bolting patterns and/or thicker shotcrete (zone II in Fig. 17). In very hard massive rock masses at great depths, spalling, slabbing and rockbursting are the modes of failure that may develop, controlled by brittle fracture propagation in the intact rock with the discontinuities having only a minor influence (zone I of Fig. 17). In these cases the use of brittle rock failure criterion should be considered, such as that proposed by Kaiser et al. (2000).

Fractured peridotites or schistose serpentinites

In the case of a more fractured peridotite, schistose or weaker serpentinite (GSI values of 25–40), the behaviour is controlled by sliding and rotation on discontinuity surfaces with relatively little failure of the intact rock pieces (zone II/III of Fig. 17). In this range of GSI values the RQD values can be very low. This is normal, given the structure of the rock masses, but some of the frictional behaviour of the unaltered pieces of the mass is retained. Thus, the control of stability can be effectively improved during excavation of the tunnel by keeping the rock mass confined.

Sheared serpentinite—squeezing behaviour

In the poor quality serpentinite, due either to weathering or shearing, blockiness may be almost completely lost and clayey sections with swelling materials may be present. Tunnel stability will then be controlled by stress dependant rock mass failure with significant squeezing at depths (Fig. 17, zone III). In these cases detailed design has to be carried out using a numerical analysis which permits progressive failure and support interaction analysis to be modelled. However, it is very instructive to carry out a closed form analysis of the behaviour of the tunnel in order to get some idea of the significance and magnitude of convergence and squeezing. The plot presented in Fig. 18 is taken from a paper by Hoek and Marinos (2000) in which it was shown that, for tunnels in weak rocks, the ‘‘strain’’ can be estimated from the ratio of rock mass strength to in situ stress by means of the equation shown in the figure. This plot is for single circular shaped tunnels. The strain for twin tunnels which are reasonably close together is expected to be higher than that indicated by the plot, which is for unsupported tunnels in a hydrostatic stress field.

Two cases from a tunnel in Greece in ophiolites in the form of more or less sheared serpentinites are plotted in points 7 and 8 in Fig. 18. Point 7 corresponds to a rock mass of a strength, σ_{cm}, of about 1.4 MPa and a deformation modulus, $E = 800$ MPa, under a cover of

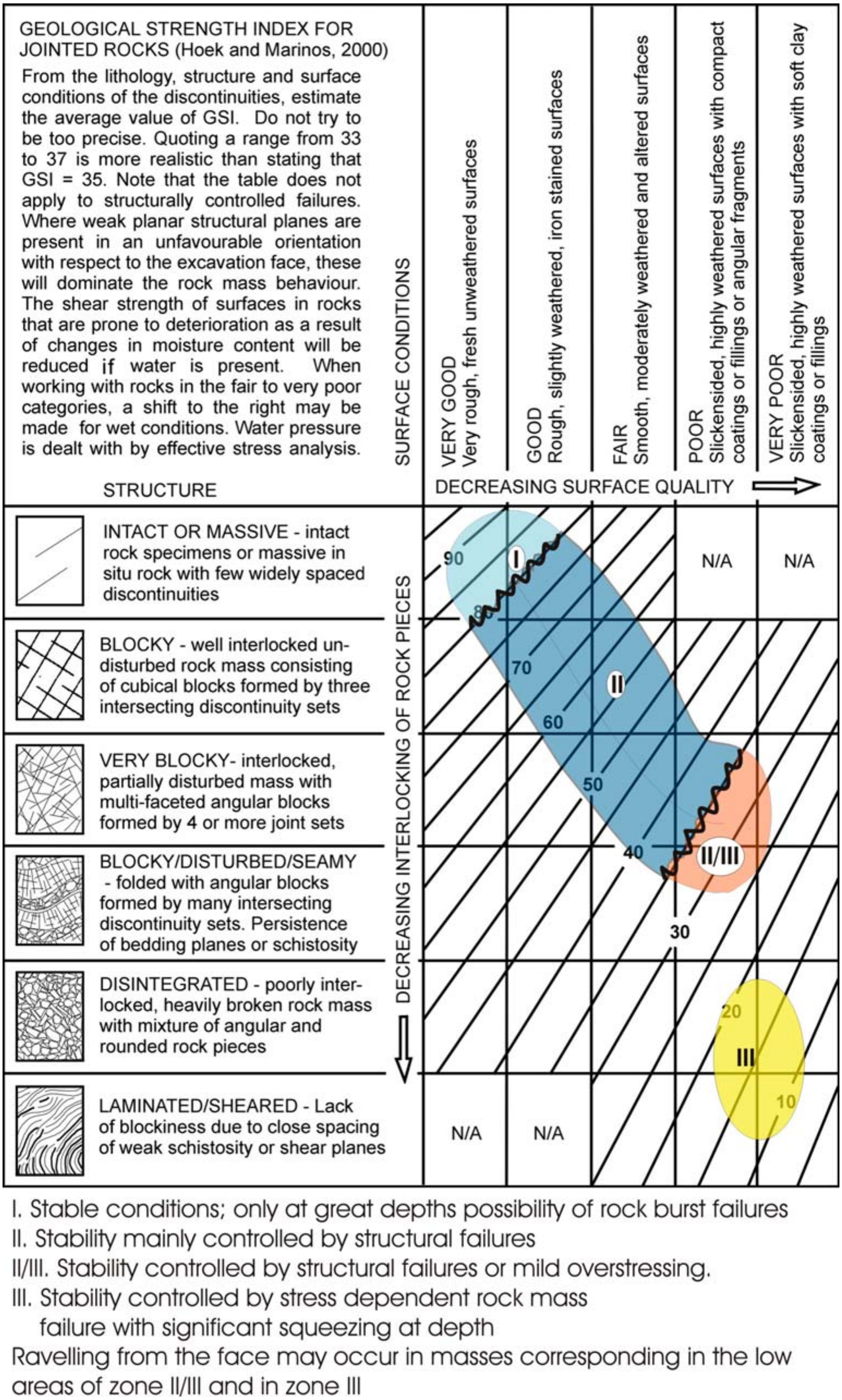

Fig. 17 Classification of the behaviour in tunnelling for peridotite–serpentinite rock masses in ophiolitic complexes (to be read in conjunction with Fig. 7)

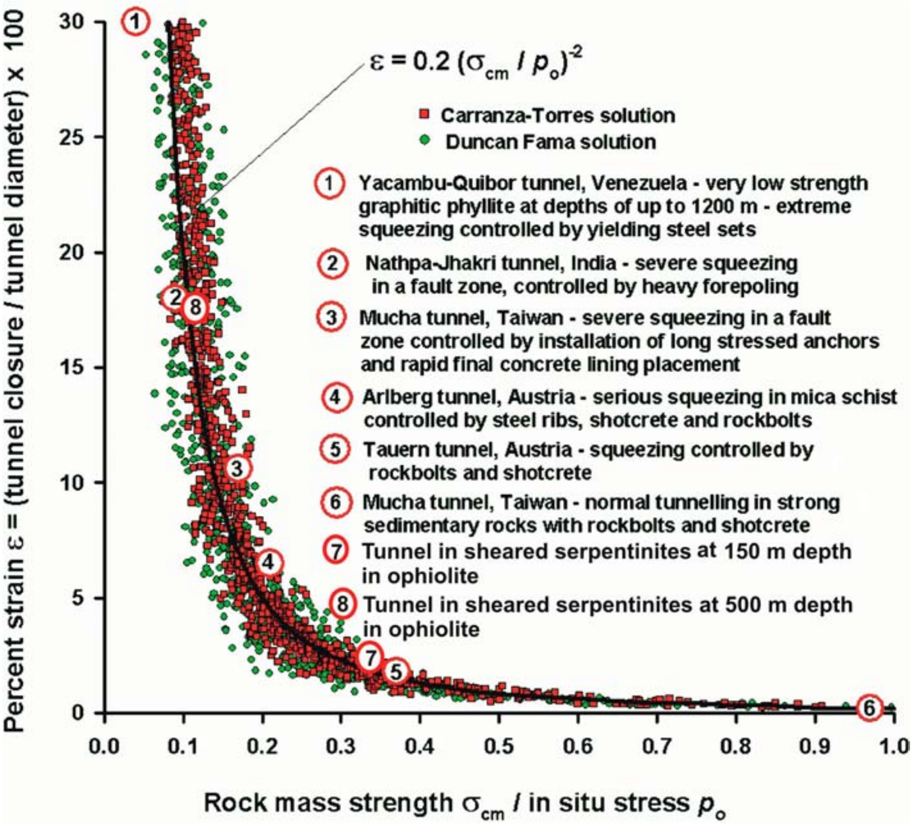

Fig. 18 Plot of percentage strain versus the ratio of rock mass strength to in situ stress (after Hoek and Marinos 2000). Calculated and predicted strains for a tunnel in ophiolite are plotted as *points 7* and *8* at ratios of rock mass strength to in situ stress of 0.35 and 0.11, respectively

about 150 m. These values were derived from a back analysis of the behaviour of a tunnel section where the measured and computed displacements of about 1.5% were in close agreement. These properties correspond to a weak ophiolitic rock mass (sheared serpentinite) with a $GSI = 20$, $\sigma_{ci} = 16$ MPa and $m_i = 10$.

For a depth of 500 m, the ratio of rock mass strength to in situ stress is 0.11 and this gives a strain of 17% for a single tunnel. This means that there may be a closure of as much as 2 m in a 12 m span tunnel unless appropriate steps are taken to control this deformation.

Squeezing and face stability can be controlled by forepoling, commonly used in many weak rock tunnels in southern Europe. However, when severe squeezing is anticipated in very weak serpentinite in areas of thick cover, the use of yielding primary support (sliding joints in steel sets or gaps in shotcrete) in conventional tunnelling may be required. In these cases the ideal tunnel section is circular. Where such difficult tunnelling conditions are encountered it is recommended that a robust design be provided with the possibility of varying the amount of yielding depending on the overburden and the rock mass quality. In general, it is recommended that the chosen support types should be able to accommodate changes without the need to change the principal elements of the support system.

It is unlikely that rockbolts will be effective in severe squeezing conditions as they are too stiff in relation to the surrounding rock mass and the resulting strain differential causes shearing of the grout bond. Ravelling of completely disintegrated serpentinite may also be a problem and keeping confinement of the face is the key action to be undertaken.

Special provisions may be required to eliminate the possibility of trapping the machine in the case of excavation by TBM. It is critically important that a TBM should never be stopped in zones of severe squeezing. Experience has shown that this squeezing occurs relatively slowly and that a moving machine is seldom trapped. Overboring devices are necessary and in some cases TBMs have been specially designed to permit reduction of the shield diameter. However, several machines have been lost when they have been parked for one reason or another. An appropriately designed precast segmental lining is generally installed directly behind the machine.

As the occurrence of weak or sheared rock masses is randomly distributed along the tunnel alignment due to the particular geological model of the ophiolites, it is prudent to probe ahead of the tunnel face continuously, keeping a length of probe hole of at least 20 m ahead of the face at all times. In the case of TBM excavation,

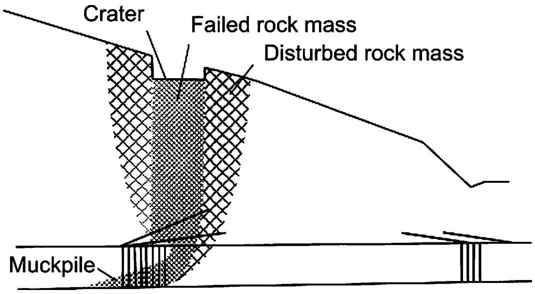

Fig. 19 Failure and interpretation of possible extent of rock mass disturbance resulting from the collapse due to ravelling of weathered peridotite during tunnelling in a heavily disturbed ophiolitic complex in Greece. The overburden is about 35 m. Forepoling and a grout umbrella for remediation are shown

Fig. 20 Appearance of stabilised muckpile at the face. No material has been removed since the collapse and stabilisation has been carried out by the installation of a double forepole umbrella and by grouting

provision should be made for probe drilling through the cutter head or by means of inclined holes drilled over the shield.

Weathered peridotite. A case of ravelling

Tunnelling through weathered peridotite (serpentinite) close to the surface will require great care in order to avoid subsidence and slope movement and a light forepole umbrella (75 or 100 mm diameter pipes) can be used. Pre-grouting an umbrella in the rock mass over the forepoles may be advisable in order to increase the cohesive strength of the rock mass. Figures 19 and 20 illustrate a failure in weathered peridotite.

The tunnel face has been stabilised by the installation of a double forepole umbrella and by extensive grouting through the forepoles and also through horizontal holes drilled through the muckpile. Figure 20 shows a situation in which no material has been removed from the muckpile which has been covered with shotcrete.

Groundwater conditions

Groundwater can be present in fractured but otherwise sound peridotites. However, weak masses are usually of low permeability; thus, water pressure has to be considered in the design and systematic relief holes may be required during construction.

Conclusions

Rock masses in an ophiolitic complex exhibit a wide range of engineering behaviour, particularly in tunnelling. This is due to their petrographic variety and structural complexity. The GSI, enhanced by geological logic permits the characterisation of this wide range. Its use allows adjustments of the rating to cover not only the wide range of the ophiolitic rock masses and conditions but also the variations that may occur within the same rock type. As a consequence a classification of tunnelling behaviour can be formulated for this wide range of ophiolitic rock masses from stable to severe squeezing conditions.

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Quantification of the Geological Strength Index chart

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ABSTRACT:

The original Geological Strength Index chart was developed on the assumption that observations of the rock mass would be made by qualified and experienced geologists or engineering geologists. With the ever increasing use of the GSI chart as the basis for the selection of input parameters for numerical analysis, often by individuals without the strong geologic understanding of rock mass variability necessary to interpret the graphical GSI chart properly, some uniformity and quantification of the chart seems necessary. This paper presents a proposed quantification of the GSI chart on the basis of two well-established parameters - Joint Condition and RQD. Recommendations for future development of more robust scales are presented.

1. INTRODUCTION

The original Geological Strength Index (GSI) chart was developed on the assumption that observations of the rock mass would be made by qualified and experienced geologists or engineering geologists. When such individuals are available, the use of the GSI charts based on the descriptive categories of rock mass structure and discontinuity surface conditions have been found to work well. However, there are many situations where engineering staff rather than geological staff are assigned to collect data, which means that the mapping of rock masses or core is carried out by persons who are less comfortable with these qualitative descriptions.

As part of an ongoing evaluation of the uses and abuses of the Hoek-Brown and Geological Strength Index systems for estimating the mechanical properties of rock masses, the issue of quantifying GSI has been given priority. GSI is the first point of entry into the system and, unless this Index is well understood and applied correctly, the reliability of the estimated properties is open to question.

Figure 1 illustrates the data flow when using the GSI/Hoek-Brown method for estimating the parameters required for a numerical analysis of underground or surface excavations in rock. Depending on whether the users have a geological or an engineering background, there tend to be strongly held opinions on whether the observed geological conditions should be entered either descriptively or quantitatively into the characterization table for GSI. Both of these approaches are catered for in the discussion that follows.

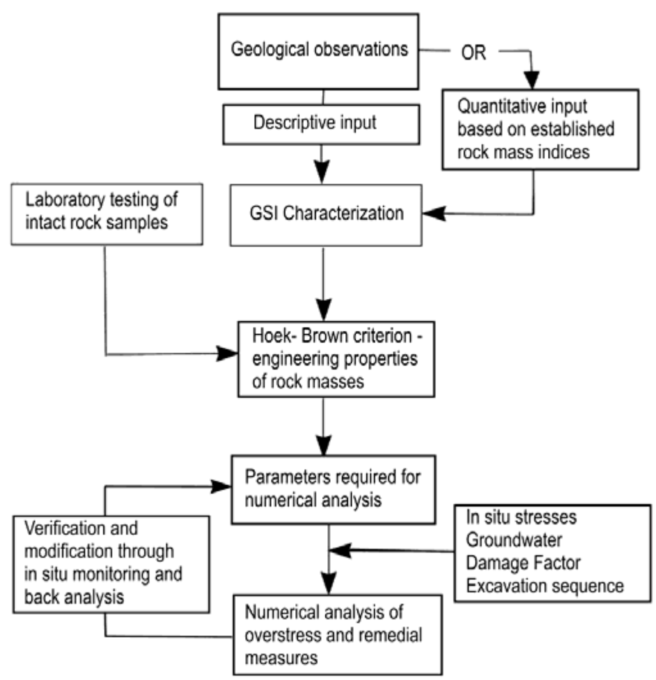

Figure 1: Data entry stream for using the Hoek-Brown system for estimating rock mass parameters for numerical analysis.

2. CONSTRUCTION OF THE BASIC GSI CHART

The GSI chart published by Hoek and Marinos (2000) [1] is reproduced in Figure 2. Scale A has been added to represent the 5 divisions of surface quality with a range of 45 points, defined by the approximate intersection of the GSI = 45 line on the axis. Scale B represents the 5 divisions of the block interlocking scale with a range of 40 points in the zone in which quantification is applied.

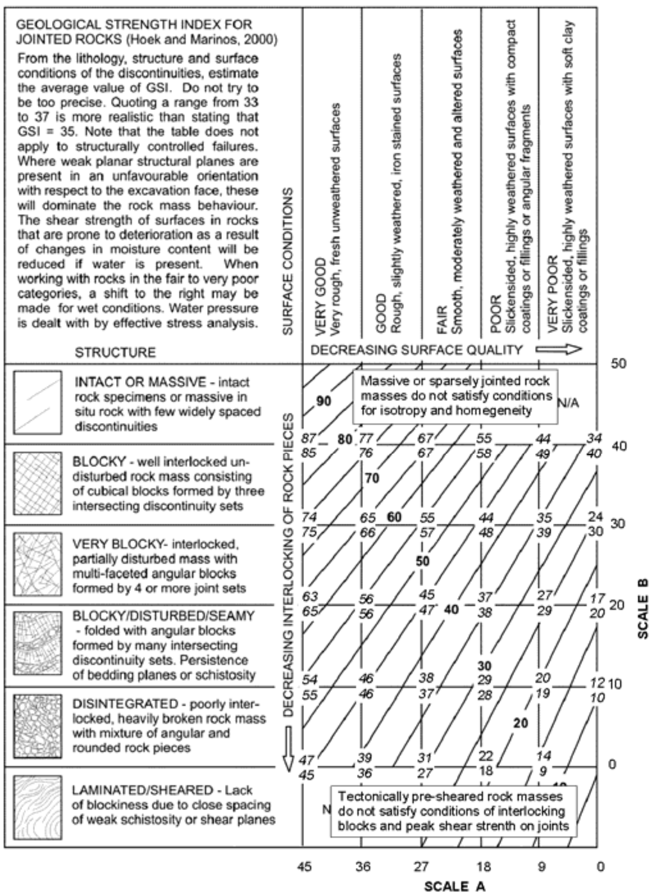

Figure 2: The basic structure of the Hoek and Marinos (2000) GSI chart and possibilities for quantification.

At each intersection of the A and B scales the value of GSI has been estimated from the GSI lines on the chart. These values are shown as the upper italicized number at the intersection point. At the same intersection points the lower italicized number equals the sum of the A and B values. The two numbers at each intersection point are then plotted against each other in Figure 3.

This plot demonstrates that there is a high potential for quantifying GSI by means of two linear scales representing the discontinuity surface conditions (scale A) and the interlocking of the rock blocks defined by these intersecting discontinuities (scale B).

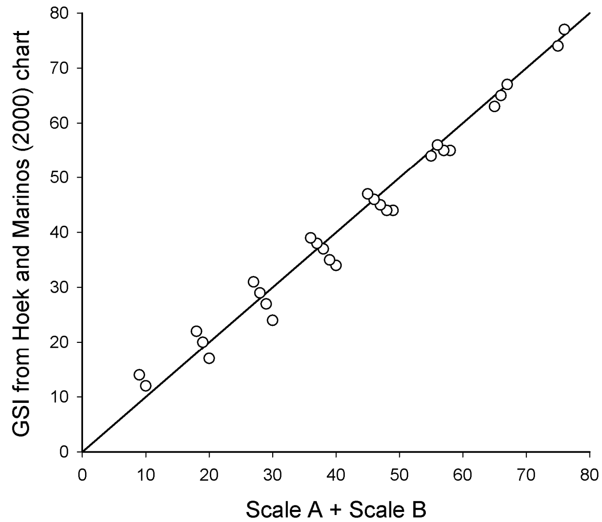

Figure 3: Plot of GSI estimated from the basic GSI chart against the sum of the A and B values.

Figure 3 also shows that there is a systematic trend in each group of plotted points and, from an examination of the chart in Figure 2, it is obvious that this trend is due to the fact that the original GSI lines, which were hand drawn, are neither parallel nor equally spaced.

With a modest correction to the original GSI lines to make them parallel and equally spaced, the error trends in Figure 3 can be eliminated completely. This correction has been applied to Figure 5.

Note that the correction of the GSI lines and the addition of the A and B scales do not change the chart's original function of estimating GSI from field observations of blockiness and joint condition, characterized in terms of the descriptive axis title blocks. Hence the chart shown in Figure 5 has the potential for satisfying both the descriptive and quantitative user camps.

Before proceeding any further with this discussion it is necessary to define a number of conditions and limitations of the proposed quantitative GSI chart.

1. The addition of quantitative scales to the GSI chart should not limit the use for which it was originally designed – the estimation of GSI values from direct visual observations of the rock conditions in the field.

2. A fundamental assumption of the Hoek-Brown criterion for the estimation of the mechanical properties of rock masses is that the deformation and the peak strength are controlled by sliding and rotation of intact blocks of rock defined by intersecting discontinuity systems. It is assumed that there are several discontinuity sets and that they are sufficiently closely spaced, relative to the size of the structure under consideration, that the rock mass can be considered homogeneous and isotropic. These concepts are illustrated diagrammatically in Figure 4.

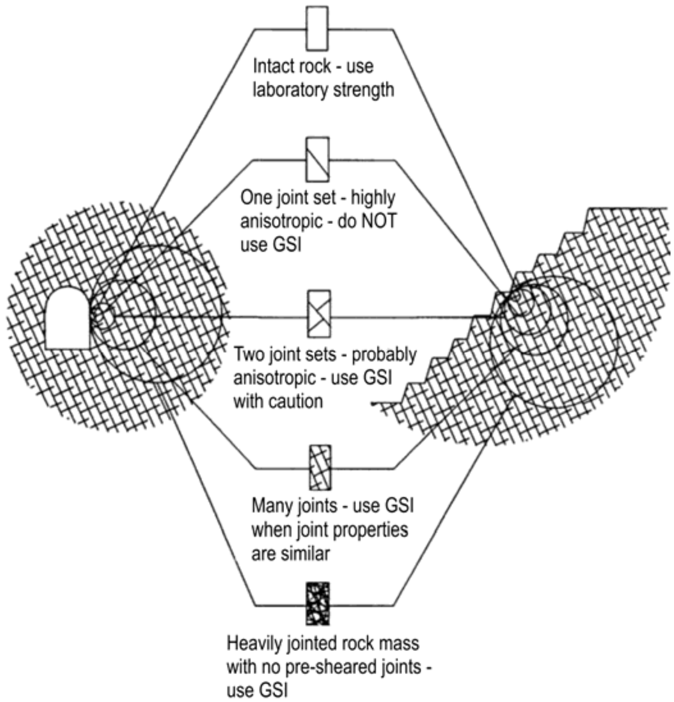

Figure 4: Limitations on the use of GSI depending on scale.

3. For intact massive or very sparsely joined rock, the GSI chart should not be used for input into the Hoek-Brown criterion as shown in Figure 1. This is because there are insufficient pre-existing joints to satisfy the conditions of homogeneity and isotropy described above. Hence, in order to avoid confusion, the upper row of the chart shown in Figure 2 has been removed in the development of the quantified GSI chart. Brittle failure processes such as rockbursts and spalling are specifically excluded from the section of the quantified GSI chart since these processes do not involve the rotation and translation of interlocking blocks of rock as defined in 2 above. Similarly, structurally controlled failure in sparsely jointed rock does not fall within the definition of homogeneity inherent in the definition of GSI.

4. The lower row of the original 2000 GSI chart has also been removed since this represents previously sheared or transported or heavily altered materials to which the conditions defined in item 2 above also do not apply. A second GSI chart for heterogeneous, pre-sheared materials such as flysch has been published by Marinos and Hoek (2002) [2] and Marinos et al (2007) [3]. Where applicable this flysch chart could be used or a similar site specific chart could be developed for rock masses that fall below the last row of the chart given in Figure 5.

Some approaches for tackling both ends of the rockmass competency scale addressed in paragraphs (3) and (4) are suggested by Carter et al, 2008, [4].

5. In order to quantify GSI using the chart, the quantities used to construct the A and B scales have to be practical ratings that are familiar to engineering geologists and geotechnical engineers operating in the field. They should also be well established in the literature as reliable indices for characterizing rock masses intersecting discontinuity systems. It is assumed that there are a sufficient number of discontinuities and that they are sufficiently closely spaced, relative to the size of the structure under consideration, that the rock mass can be considered homogeneous and isotropic.

3. ESTIMATION OF GSI IN TERMS OF RQD AND JOINT CONDITION

Scale A in Figure 2 represents discontinuity surface conditions while Scale B represents the blockiness of the rock mass. Prime candidates for these scales are the Joint Condition ($JCond_{89}$) rating defined by Bieniawski (1989) [5] and the Rock Quality Designation (RQD) defined by Deere (1963) [6]. These ratings are given in Appendix 1.

The $JCond_{89}$ rating corresponds well with the surface conditions defined in the text boxes of the x axis of the GSI chart in Figure 5. This rating parameter has been in use for many years and users have found it to be both simple and reliable to apply in the field.

The RQD rating has been in use for 50 years and some users have defined it as boringly reliable. Hence these two ratings appear to be ideal for use as the A and B scales for the quantification of GSI.

Figure 5 shows a chart in which the A scale is defined by 1.5 $JCond_{89}$ while the B scale is defined as RQD/2. The value of GSI is given by the sum of these scales which results in the relationship:

$$GSI = 1.5\ JCond_{89} + RQD/2 \qquad (1)$$

4. CHECK OF QUANTIFIED GSI AGAINST MAPPED GSI

In order to check whether or not the proposed quantification of GSI works it is necessary to check the values of GSI predicted from equation 1 against field mapped GSI values. At the time of writing only one set of reliable field data, from a drill and blast tunnel, is available to the authors. The GSI values calculated from $JCond_{89}$ and RQD are plotted against mapped GSI values in Figure 6. This plot shows that the correlation between the calculated and mapped GSI values is reasonably close to the ideal 1:1 relationship for a perfect fit. This suggests that, once additional field data are obtained, the application of this quantification of GSI may justify the transition from proposed to recommended.

It is possible that some adjustments in the positions of the $JCond_{89}$ and RQD scales in Figure 5 may be required as more mapped GSI data becomes available and as experience is gained in using this quantification.

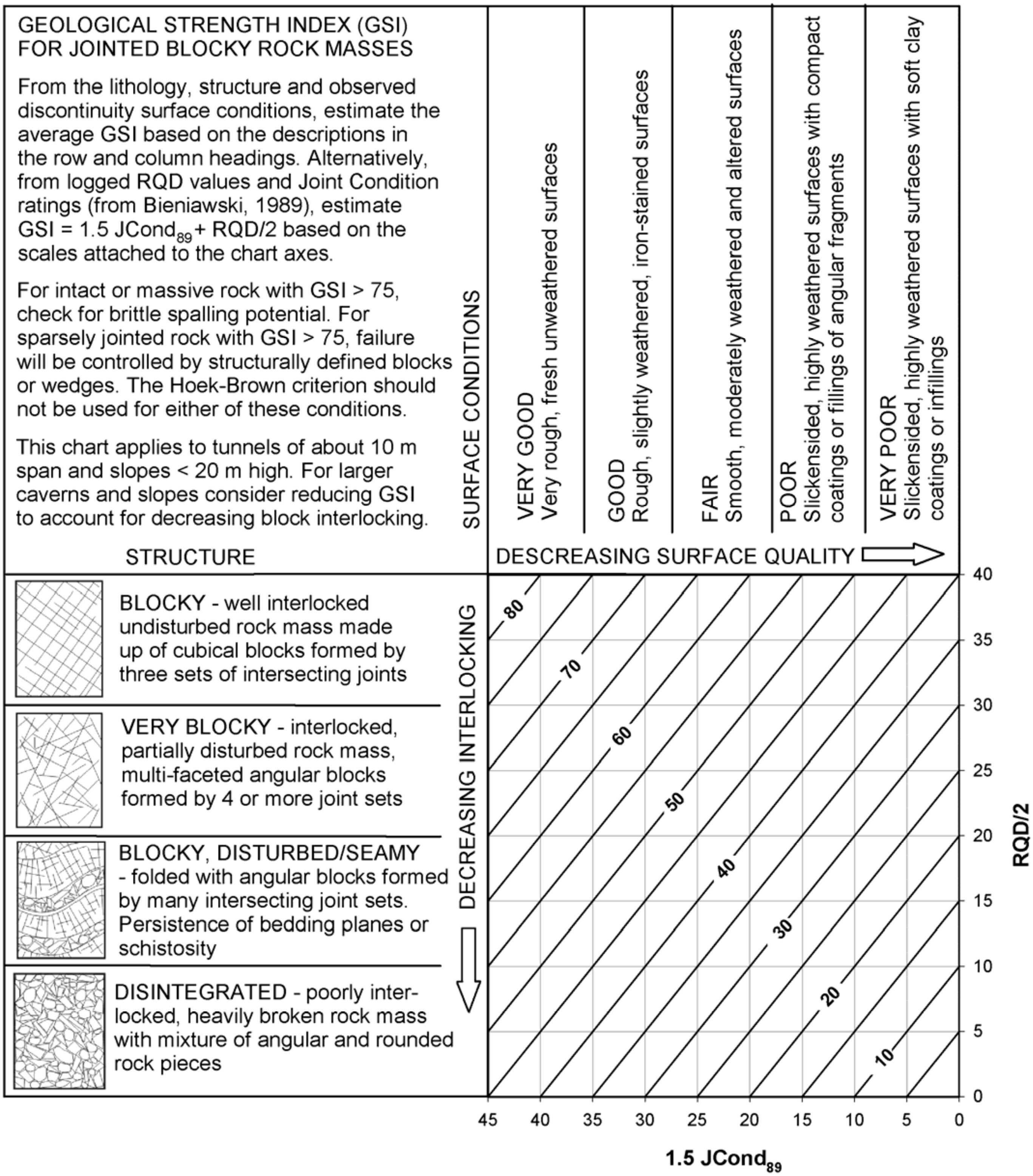

Figure 5: Quantification of GSI by Joint Condition and RQD.

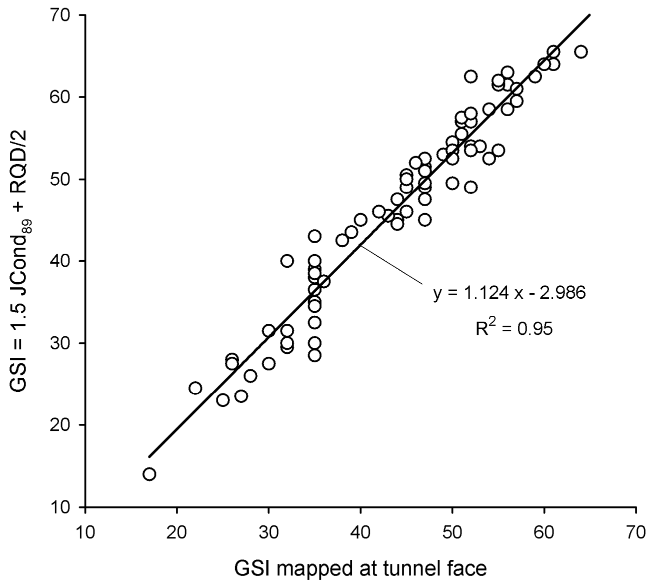

Figure 6: Comparison between mapped GSI and GSI predicted from $JCond_{89}$ and RQD.

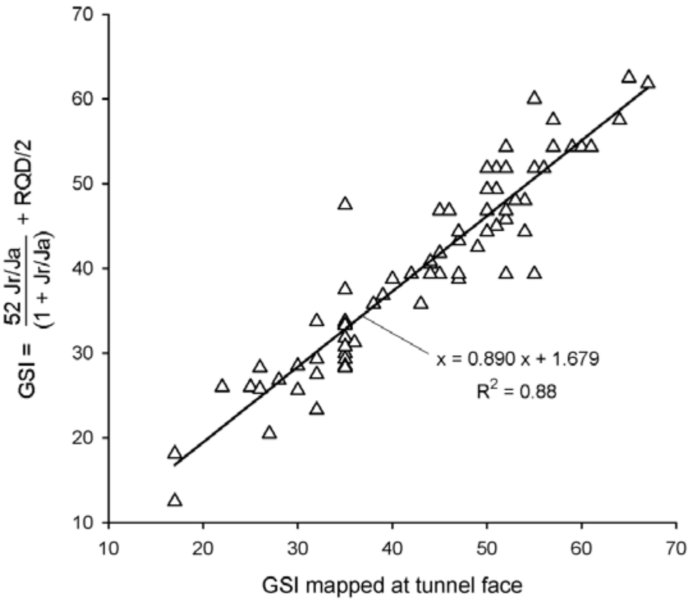

Figure 7: Comparison between mapped GSI and GSI predicted from Jr/Ja and RQD.

5. ALTERNATIVE JOINT CONDITION SCALE

In recognition of the fact that values of $JCond_{89}$ are not always available in data from field mapping, the authors have examined two options for alternative scales for the surface quality axis in Figure 5.

The first candidate is the version of Joint Condition rating ($JCond_{76}$) included in the paper by Bieniawski (1976) [7] (see Appendix 1). Regression analysis of a plot of individual values assigned to $JCond_{76}$ and $JCond_{89}$ gives $JCond_{89}$ = 1.3 $JCond_{76}$ which, when substituted into equation 1, gives

$$GSI = 2\ JCond_{76} + RQD/2 \qquad (2)$$

A second candidate is the quotient Jr/Ja, included in the Tunnelling Quality Index (Q) of Barton et al (1974) [8]. This quotient (Jr/Ja) represents the roughness and frictional characteristics of the joint walls or fillings.

Comparing the ratings for $JCond_{89}$ with those allocated to Jr and Ja by Barton et al (1974) [7] (see Appendix 1) gives the relationship $JCond_{89}$ = 35 Jr/Ja/(1 + Jr/Ja). Substitution of this relationship into equation 1 yields:

$$GSI = \frac{52\ Jr/Ja}{(1 + Jr/Ja)} + RQD/2 \qquad (3)$$

For the same data set used in the preparation of Figure 6, the predicted values of GSI are plotted against field mapped values of GSI in Figure 7. While the results for a linear regression analysis are not as good as those obtained for equation 1, the fit is an acceptable approximation for engineering applications.

6. RQD DETERMINED FROM FACE MAPS

When no core is available and RQD has to be determined from the mapping of tunnel faces, tunnel walls or slope faces, three methods are available.

The first involves a simple physical measuring rod or tape held against or in front of the face. The length of intact rock segments greater than 10cm falling between natural fractures intersecting the rod or tape are summed in a fashion similar to core-based RQD. This procedure is described in Hutchinson and Diederichs (1996) [9]. A virtual version of this approach can be carried out on high quality face photos or Lidar scans.

Priest and Hudson (1976) [10] found that a reasonable estimate of RQD could be obtained from discontinuity spacing measurements made on core or from an exposure by use of the equation:

$$RQD = 100\ e^{0.1\lambda}\ (0.1\lambda + 1) \qquad (4)$$

where λ is the average number of discontinuities per meter.

Palmström (1982) [11], also studied RQD but in relation to the Volumetric Joint Count, Jv, a measure of the number of joints crossing a cubic meter of rock. Based on mapping of exposures or on orthogonal scanline mapping underground, the following expression was derived:

$$RQD = 115 - 3.3\ Jv \qquad (5)$$

More recently, Palmström (2005) [12] extended his analysis by including computer generated blocks of different sizes and shapes. A new correlation between RQD and Jv was found to give somewhat better results that the commonly used RQD = 115 – 3.3Jv. He suggested that this relationship (equation 5) given in his 1982 paper should be modified to:

$$RQD = 110 - 2.5\ Jv \tag{6}$$

7. CONCLUSION AND RECOMMENDATIONS

With some minor modifications to the GSI chart published by Hoek and Marinos (2000) [1] it has been found that two simple linear scales, $JCond_{89}$ and RQD, can be used to represent the discontinuity surface conditions and the blockiness of the rock mass. These ratings are well established in engineering geology practice, are simple to measure or estimate in the field and are possibly the ratings that give the highest degree of consistency between different geologists working on a single project. Most importantly, in a direct check between GSI estimated from the sum of these ratings and GSI obtained by direct tunnel face mapping, the agreement is acceptable for the characterization of jointed rock masses in order to obtain properties for input for numerical models.

In recognition of the fact that values of $JCond_{89}$ are not always available in data from field mapping, two alternative scales for the surface quality axis have been investigated. One of these is a relationship between $JCond_{89}$ and the $JCond_{76}$ version of this parameter, used in older data sets, which can be used as a direct replacement of $JCond_{89}$. The second alternative is the quotient Jr/Ja that gives a relationship to $JCond_{89}$ which provides an acceptable approximation for engineering applications.

The goal of this paper was to construct a practical set of scales for the GSI chart, based on existing and well established scales used in either the RMR or Q classifications. Cai et al (2004) [13], Somnez and Ulusay (1999) [14] and Russo (2007, 2009) [15, 16] have published quantified GSI charts which incorporate joint surface and rock structure scales based on parameters related to those used by the authors in constructing Figure 5. All of these quantified GSI charts, including that proposed in Figure 5 of this paper, have advantages and disadvantages. However, they all suffer from two significant shortcomings.

Firstly, the parameters used to specify the joint surface conditions (the equivalent of Scale A in Figure 5) are all based on ratings of joint roughness, joint alteration and joint waviness. These ratings, with the exception of joint waviness, are based upon assessment of the degree of surface roughness and alteration rather than on any physical measurements of the shear strength of the surfaces themselves. It is this shear strength that is a controlling parameter in the behavior of the jointed rock mass and it is questionable whether the somewhat arbitrary nature of the roughness and alteration ratings can provide a reliable assessment of this shear strength.

Secondly, the use of RQD by the authors or some variation of the volumetric joint count Jv or the block volume Vb, by the other authors, limits the definition of rock structure to the dimension of the blocks. This takes no account of the ratio of block size to the size of the tunnel or slope which, as shown in Figure 4, has a significant influence on the application of the GSI chart for characterizing the rock mass.

Direct measurement of physical properties and numerical modeling of the progressive failure and deformation of the rock mass, while not devoid of challenges and abuses by over-enthusiastic users, offer the potential for resolving some of these deficiencies.

Measurement of the frictional strength of sawn or ground surfaces of small specimens is simple enough in a field laboratory with basic equipment. Similarly, measurement of small and large scale surface undulations, at a scale relevant to the problem under consideration, and combining these measurements with the basic friction angle of the rock surface is a well-established procedure described by Barton and Choubey (1977) [17].

Numerical techniques such as the Synthetic Rock Mass model (Mas Ivars et al. (2011) [18]) provide the means of incorporating the joint fabric of a rock mass at different scales. In the long run these methods have the potential to allow direct three-dimensional modeling of all of the physical components of a rock mass and provide a much more rigorous alternative to the empirical characterization and rockmass parameter estimation approach using the GSI chart. In the short term, numerical modeling techniques can be used to develop rock structure scales which incorporate both the scale of the rock blocks and the scale of the engineering structure in which they exist.

Rating-based rock mass characterization scales, such as those used in this paper, have played a critical role in the development of practical design tools for rock engineering. However, while practitioners may continue to apply these methods for some time, researchers should turn their attention to the actual physical properties of rock joints and numerical modeling of rock fracture networks to develop and apply a better understanding of jointed rock mass behavior.

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10. APPENDIX 1 –PARAMETER DEFINITION

The Rock Quality Designation (RQD) was developed by Deere (1963) [6]. The index was developed to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact core pieces longer than 100 mm (4 inches) in the total length of core. The core should be at least NW size (54.7 mm or 2.15 inches in diameter) and should be drilled with at least a double-tube core barrel. The correct procedures for measurement of the length of core pieces and the calculation of RQD are summarized in Figure 8.

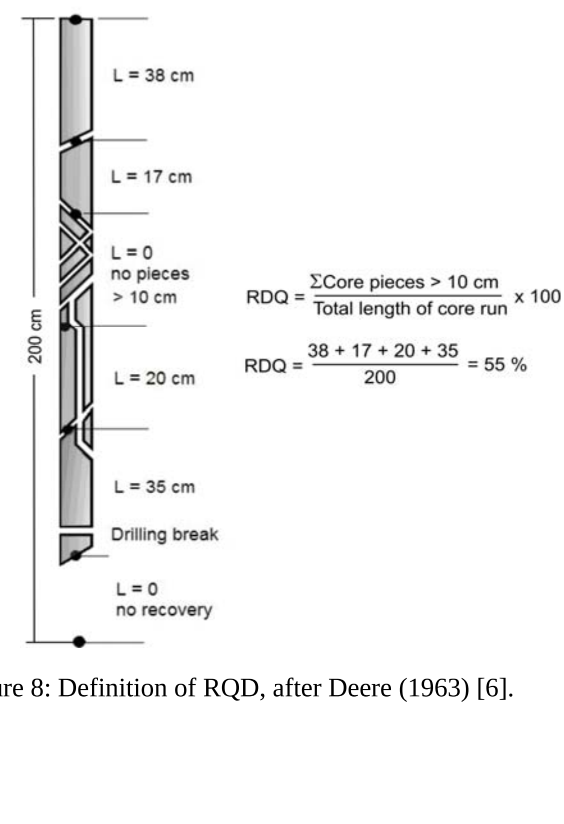

Figure 8: Definition of RQD, after Deere (1963) [6].

The definition of $JCond_{89}$ in Table 1 is reproduced directly from Bieniawski (1989) [5] while $JCond_{76}$, from Bieniawski (1976) [7], is defined in Table 2.

The parameters Jr and Ja, for rock wall contact, from Barton et al (1974) [8], are defined in Table 3

Table 1: Definition of $JCond_{89}$, after Bieniawski (1989) [5].

Condition of discontinuities	Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1 – 5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous
Rating	30	25	20	10	0

Guidelines for classification of discontinuity conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 to 3 m 4	3 to10 m 2	10 to 20 m 1	More than 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 – 1.0 mm 4	1 – 5 mm 1	More than 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard infilling < 5 mm 4	Hard filling > 5 mm 2	Soft infilling < 5 mm 2	Soft infilling > 5 mm 0
Weathering Rating	Unweathered 6	Slightly weathered 5	Moderate weathering 3	Highly weathered 1	Decomposed 0

Table 2: Definition of $JCond_{76}$, after Bieniawski (1976) [7]

Condition of discontinuities	Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Soft joint wall rock	Slickensided surfaces or Gouge < 5 mm thick or Joints open 1 – 5 mm Continuous joints	Soft gouge > 5 mm thick or Joints open > 5 mm Continuous joints
Rating	25	20	12	6	0

Table 3: Definition of Jr and Ja for rock wall contact (no pre-shearing), after Barton et al (1974) [8].

JOINT ROUGHNESS NUMBER Jr	Rating	JOINT ALTERATION NUMBER Ja	Rating
Discontinuous joints	4	Tightly healed, hard, non-softening, impermeable filling	0.75
Rough and irregular, undulating	3	Unaltered joint walls, surface staining only	1.0
Smooth, undulating	2	Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0
Slickensided, undulating	1.5	Silty-, or sandy-clay coatings, small clay fraction (non-softening)	3.0
Rough or irregular planar	1.5	Softening or low friction clay, mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 – 2 mm or less in thickness)	4.0
Smooth, planar	1.0		
Slickensided, planar	0.5		