Estimating the geotechnical properties of heterogeneous rock masses such as flysch

Paul Marinos · Evert Hoek

Abstract 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. A methodology for estimating the Geological Strength Index and the rock mass properties for these geological formations is presented in this paper.

Résumé L'étude des tunnels et des talus dans des masses rocheuses hétérogènes, telles que le flysch, représente un défi majeur pour les géologues et les ingénieurs. La complexité de ces formations, résultat de leur histoire de sédimentation et de leur mise en place tectonique, pose des problèmes pour leur classification par des systèmes reconnus de classifications géotechniques. Dans ce travail une méthodologie pour l'estimation du GSI (Geological Strength Index) et l'évaluation des propriétés des masses rocheuses de flysch est présentée.

Keywords Rock mass classification · GSI · Geotechnical properties · Flysch

Mots clés Classifications géotechniques · Masses rocheuses · GSI · Propriétés géotechniques · Flysch

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P. Marinos (✉)
Civil Engineering Department,
National Technical University of Athens,
Athens, Greece
E-mail: marinos@central.ntua.gr

E. Hoek
Consulting Engineer,
Vancouver, Canada

Introduction

Many large civil engineering projects are currently under construction in countries where flysch is a very common geological formation. The design of surface and underground excavations in these materials requires knowledge of the mechanical properties of the rock masses in which these excavations are carried out. The paper presents a methodology for estimating these properties.

Estimation of rock mass properties

One of the most widely used criteria for estimating rock mass properties is that proposed by Hoek and Brown (1997) and this criterion, with specific adaptations to heterogeneous rock masses such as flysch, is briefly summarised. This failure criterion should not be used when the rock mass consists of a strong blocky rock such as sandstone, separated by clay-coated and slicken-sided 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 than any other features. In such rock masses the predominant failure mode will be 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. However, if the rock mass is heavily fractured, the continuity of the bedding surfaces will have been disrupted and the rock may behave as an isotropic mass. In applying the Hoek and Brown criterion to "isotropic" rock masses, three parameters are required for estimating the strength and deformation properties. These are:

1. The uniaxial compressive strength σ_{ci} of the "intact" rock elements that make up the rock mass (as described below, this value may not be the same as that obtained from a laboratory uniaxial compressive strength or UCS test).
2. A constant m_i that defines the frictional characteristics of the component minerals in these rock elements.
3. The Geological Strength Index (GSI) which relates the properties of the intact rock elements to those of the overall rock mass.

These parameters are discussed below.

Uniaxial compressive strength σ_{ci} of intact rock

In dealing with heterogeneous rock masses such as flysch, it is extremely difficult to obtain a sample of "intact" core for uniaxial compressive testing in the laboratory. The typical appearance of such material in an outcrop is illustrated in Fig. 1. Practically every sample obtained from a rock mass such as that illustrated in Fig. 1 will contain discontinuities in the form of bedding and schistosity planes or joints. Consequently, any laboratory tests carried out on core samples will result in a strength value that is lower than the uniaxial compressive strength σ_{ci} required for input into the Hoek–Brown criterion. Using the results of such tests in the Hoek–Brown criterion will impose a double penalty on the strength (in addition to that imposed by GSI) and will give unrealistically low values for the rock mass strength.

In some special cases, where the rock mass is very closely jointed and where it has been possible to obtain undisturbed core samples, uniaxial compressive strength tests have been carried out directly on the "rock mass" (Jaeger 1971). These tests require an extremely high level of skill on the part of the driller and the laboratory technician. The large-scale triaxial test facilities required for such testing are only available in a few laboratories in the world and it is generally not economical or practical to consider such tests for routine engineering projects.

One of the few courses of action that can be taken to resolve this dilemma is to use the point load test on samples in which the load can be applied normal to the bedding or schistosity of block samples. The specimens used for such testing can be either irregular pieces or pieces broken from the core, as illustrated in Fig. 2. The direction of loading should be as perpendicular to any weakness planes as possible and the fracture created by the test should not show any signs of having followed an existing discontinuity. It is strongly recommended that photographs of the specimens, both before and after testing, should accompany the laboratory report as these enable the user to judge the validity of the test results. The uniaxial compressive strength of the intact rock samples can be estimated with a reasonable level of accuracy by multiplying the point load index I_s by 24[1], where $I_s = P/D^2$. P is the load on the points and D is the distance between the points.

In the case of very weak and/or fissile rocks such as clayey shales or sheared siltstones, the indentation of the loading points may cause plastic deformation rather than fracture of the specimen. In such cases the point load test does not give reliable results. Where it is not possible to obtain samples for point load testing, the only remaining alternative is to turn to a qualitative description of the rock material in order to estimate the uniaxial compressive strength of the intact rock. A table listing such descriptions is given in Table 1, based on Hoek and Brown (1997).

Fig. 1
Appearance of sheared siltstone flysch in an outcrop

Constant m_i

The Hoek–Brown constant m_i can only be determined by triaxial testing on core samples or estimated from a qualitative description of the rock material as described by Hoek and Brown (1997). This parameter depends upon the frictional characteristics of the component minerals in the intact rock sample and has a significant influence on the strength characteristics of rock.

When it is not possible to carry out triaxial tests (for the reasons discussed above), an estimate of m_i can be obtained from Table 2. Most of the values quoted have been derived from triaxial tests on intact core samples and the range of values shown is dependent upon the accuracy of the geological description of each rock type. For example, the term "granite" describes a clearly defined rock type and all granites exhibit very similar mechanical characteristics. Hence the value of m_i is defined as 32±3. On the other hand, the term "volcanic breccia" is not very precise in terms of mineral composition and hence the value of m_i is shown as 19±5, denoting a higher level of uncertainty.

Fortunately, in terms of the estimation of rock mass strength, the value of the constant m_i is the least sensitive of the three parameters required. Consequently, the average values given in Table 2 are sufficiently accurate for most practical applications.

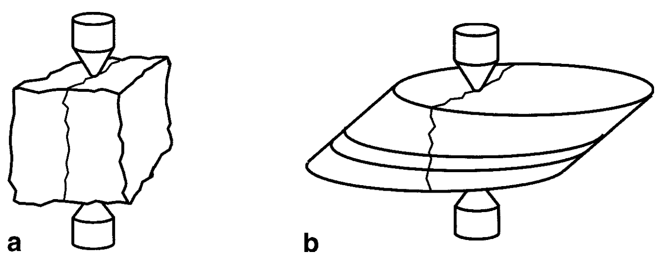


Fig. 2
Point load test options for intact rock samples from heterogeneous rock masses. **a** Test on sample chosen from surface exposure. **b** Test on sample broken from diamond drill core

[1]The constant of 24 is for a 54 mm core sample (Bieniawski 1974)

Table 1
Field estimates of uniaxial compressive strength of intact rock. (After Hoek and Brown 1997 with some changes in the examples)

Grade[a]	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	[b]	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, rock salt
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, shale
R0	Extremely weak	0.25–1	[b]	Indented by thumbnail	Stiff fault gouge

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

Table 2
Values of constant m_i for intact rock, by rock group. Values in *parentheses* are estimates. Range of values quoted for each material depends upon granularity and interlocking of crystal structure, higher values being associated with tightly interlocked and more frictional characteristics. (This table contains several changes from previously published versions. These changes have been made to reflect data that have been accumulated from laboratory tests and experience gained from discussions with geologists and engineering geologists.)

Sedimentary	Clastic		Conglomerates[a] Breccias[a]	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[b]			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)	

[a]Conglomerates and breccias may have a wide range of values, depending on the nature of cementing material and degree of cementation. Values may range between those of sandstones to those of fine-grained sediments
[b]These values are for intact rock specimens tested normal to bedding or foliation. Values of m_i will be significantly different if failure occurs along a weakness plane

Geological Strength Index (GSI)

The Geological Strength Index (GSI) was introduced by Hoek (1994), Hoek, Kaiser and Bawden (1995), Hoek and Brown (1997) and extended by Hoek, Marinos and Benissi (1998). A chart for estimating the GSI for flysch is presented in Table 3.

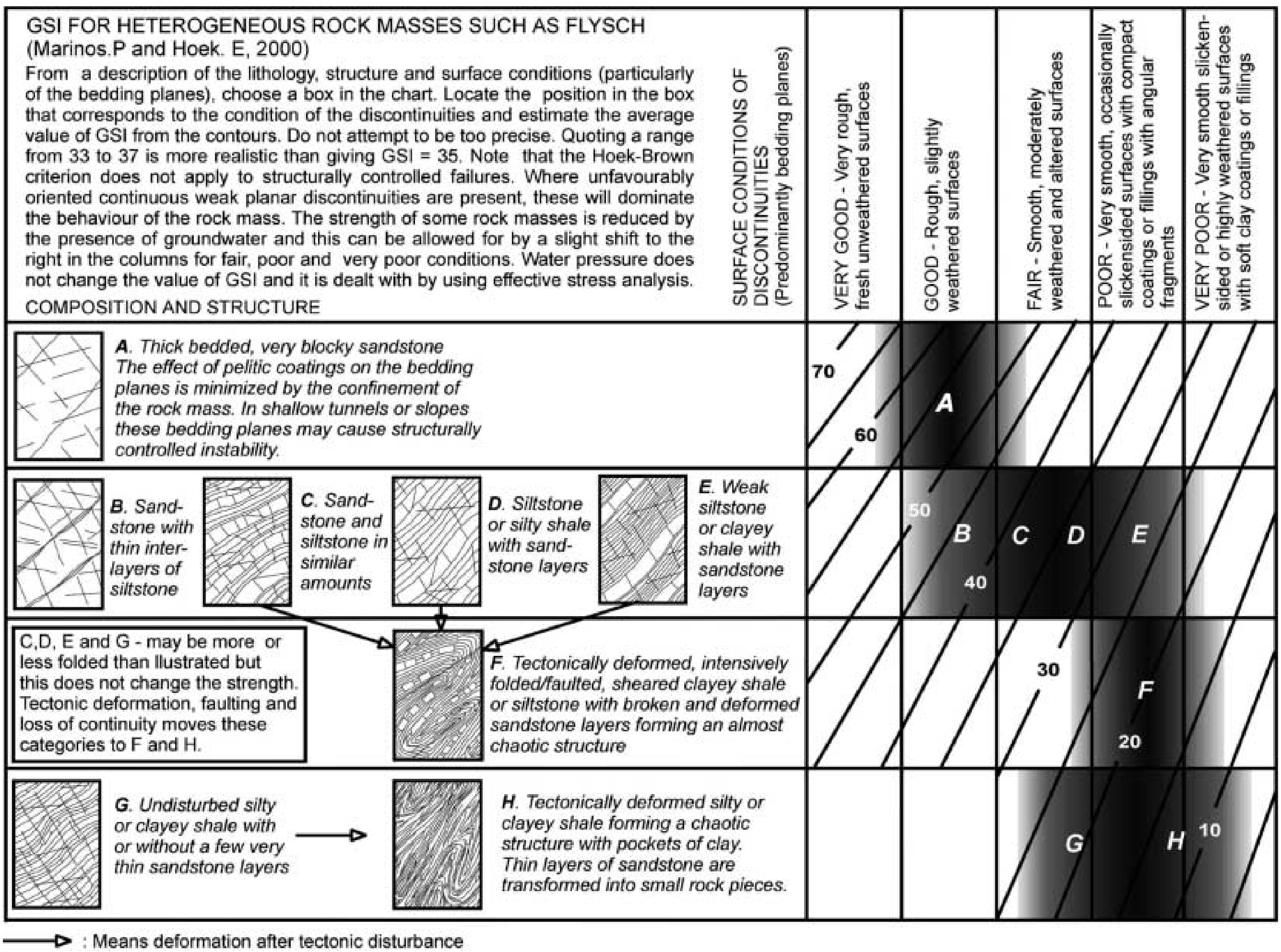
Mechanical properties of flysch

The term flysch is attributed to the geologist B. Studer and it comes from the German word "*fliessen*" meaning flow, probably denoting the frequent landslides in areas consisting of these formations. Flysch consists of varying alternations of clastic sediments that are associated with orogenesis. It closes the cycle of sedimentation of a basin before the "arrival" of the paroxysm folding process. The clastic material derives from erosion of the previously formed neighbouring mountain ridge.

Flysch is characterised by rhythmic alternations of sandstone and fine-grained (pelitic) layers. The sandstone may also include conglomerate beds. The fine-grained layers contain siltstones, silty shales and clayey shales. Rarely, limestone beds or ophiolitic masses may be found close to its margins. The thickness of the sandstone beds range

Fig. 3
"Portable" point load test device for use in the field

Table 3
GSI estimates for heterogeneous rock masses such as flysch



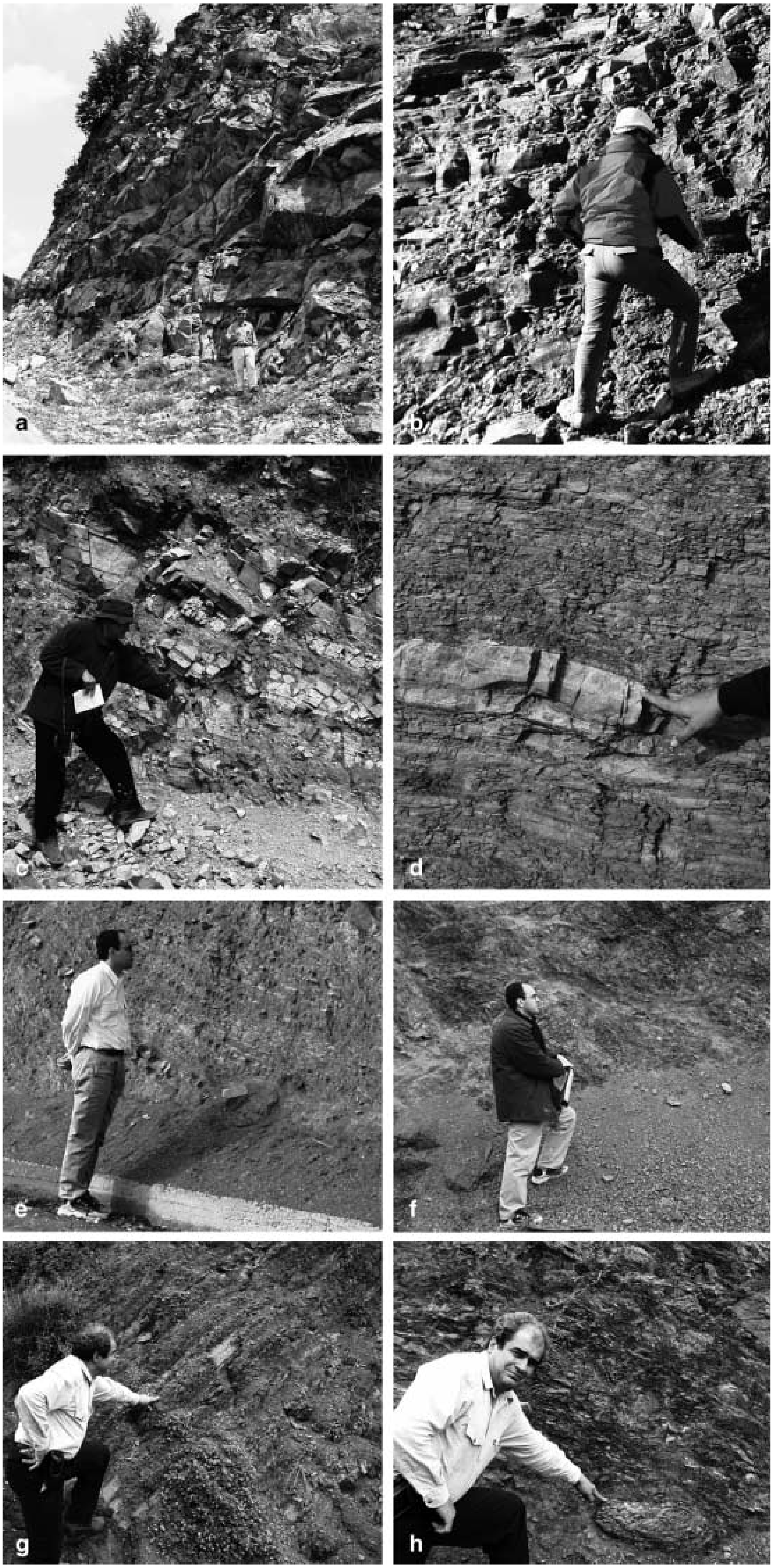


Fig. 4
Examples of flysch corresponding to descriptions in Table 3. **A** Thick-bedded blocky sandstone. Note that structural failure can occur when the dip of bedding planes is unfavourable. **B** Sandstone with thin siltstone layers. Small-scale structural failures can occur when bedding dip is unfavourable. **C** Sandstone and siltstone in equal proportions. **D** Siltstone or silty shale with sandstone. **E** Weak siltstone or clayey shale with sandstone layers. **F** Tectonically deformed clayey shale or siltstone with broken sandstone. **G** Undisturbed silty or clayey shale with a few thin sandstone layers. **H** Tectonically deformed clayey shale

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.

The overall thickness of the flysch is often very large (hundreds to a few thousand metres), albeit it may have been reduced considerably by erosion or by thrusting. The formation may contain different types of alternations and is often affected by reverse faults and thrusts. This, together with consequent normal faulting, results in a 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.

Geotechnically, a flysch rock mass has the following characteristics:

1. Heterogeneity: alternation of competent and incompetent members.
2. Presence of clay minerals.
3. Tectonic fatigue and sheared discontinuities, often resulting in a soil-like material.
4. The permeability of flysch rock masses is generally low and, because of the presence of clay minerals, the rock mass may be weakened to a significant degree where free drainage is not present.

Molasse is a term used to define a rock mass of similar composition but of post-orogenic origin associated with newly formed mountain ridges. It has the same alternations of strong (sandstones and conglomerates) and weak (marls, siltstones and claystones) materials, but there is not the same compressional disturbance.

Determination of the GSI 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 geological 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 and is presented in Table 3.

Influence of groundwater

The most basic impact of groundwater is upon the mechanical properties of the intact rock components of the rock mass. This is particularly important when dealing with shales, siltstones and similar rocks that are susceptible to changes in moisture content. Many of these maerials will disintegrate very quickly if they are allowed to dry out after removal from the core barrel. For this reason, testing of the "intact" rock to determine the uniaxial compressive strength σ_{ci} (see above) and the constant m_i must be carried out under conditions that are as close to the in-situ moisture conditions as possible. Ideally, a field laboratory should be set up very close to the drill rig and the core prepared and tested immediately after recovery.

In one example in which a siltstone was being investigated for the construction of a power tunnel for a hydro-electric project, cores were carefully sealed in aluminium foil and wax and then transported to a laboratory in which very high-quality testing could be carried out. In spite of these precautions, the deterioration of the specimens was such that the test results were meaningless. Consequently, a second investigation programme was carried out in which the specimens were transported to a small laboratory about 5 km from the exploration site and the samples were tested within an hour of having been removed from the core barrel. The results of this second series of tests were very consistent and the values of uniaxial compressive strength σ_{ci} and constant m_i were considered reliable.

When laboratory testing is not possible, point load tests, using equipment similar to that illustrated in Fig. 3, should be carried out as soon after core recovery as possible in order to ensure that the moisture content of the sample is close to the in-situ conditions.

Examples of typical flysch

In order to assist the reader in using Table 3, examples of typical flysch outcrops are given in the photographs reproduced in Fig. 4.

Selection of σ_{ci} and m_i for flysch

In addition to the GSI values presented in Table 3, it is necessary to consider the selection of the "intact" rock properties σ_{ci} and m_i for heterogeneous rock masses such as flysch. As the sandstone layers are 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

Table 4
Suggested proportions of parameters σ_{ci} and m_i for estimating rock mass properties for flysch

Flysch type (see Table 3)	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

Table 5
Spreadsheet for calculation of rock mass properties. (After Hoek and Brown 1997)

Input:	sigci=10 MPa			mi=10			GSI=30		
	Depth of failure surface or tunnel below slope[a]=25 m						Unit wt.=0.027 MN/m^3		
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=1,000 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:
(σ_n) stress=if(depth>30, sigci×0.25,depth×unit wt×0.25)
(m_b) 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)
(σ_{tm}) sigtm=0.5×sigci×(mb–SQRT(mb^2+4×s))
(σ_3) sig3=start at 1E-10 (to avoid zero errors) and increment in seven steps of stress/28 to stress/4
(σ_1) sig1=sig3+sigci×(((mb×sig3)/sigci)+s)^a
($\delta\sigma_1/\delta\sigma_3$) ds1ds3=IF(GSI>25,(1+(mb×sigci)/(2×(sig1–sig3))),1+(a×mb^a)×(sig3/sigci)^(a–1))
(σ_n) sign=sig3+(sig1–sig3)/(1+ds1ds3)
(τ)tau=(sign–sig3)×SQRT(ds1ds3)
x=LOG((sign–sigtm)/sigci)
y=LOG(tau/sigci)
xy=x×y
xsq=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)
(ϕ) phi=ASIN((k–1)/(k+1))×180/PI()
(c) coh=sigcm/(2×SQRT(k))
σ_{cm} 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)
tangent = coht+signt×TAN(phit×PI()/180)
[a]For depths below surface of less than 30 m, average stress on the failure surface is calculated by the spreadsheet. For depths greater than 30 m average stress level is kept constant at the value for 30 m depth
The example included in Table 5 is for evaluation of the mechanical properties of a rock mass of flysch consisting of weak siltstone and sandstone layers. Tables 1, 2, 3 and 4 are used to obtain the following input parameters: (1) Equivalent intact rock strength σ_{ci}=10 MPa (weighted from values of σ_{ci} of sandstone and siltstone from Table 1 and Table 4); (2) equivalent constant m_i = 10 (weighted from values of m_i of sandstone and siltstone from Table 2 and Table 4); (3) geological Strength Index GSI = 30 (flysch type E in Table 3)

conservative as the sandstone skeleton certainly contributes to the rock mass strength. 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 4.

Estimating rock mass properties

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. As 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 5.

Deep tunnels

For tunnels at depths of greater 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 σ_3 up to 0.25 σ_{ci}, in accordance with the procedure defined by Hoek and Brown (1997). In the case of "deep" tunnels, equivalent Mohr Coulomb cohesive strengths and friction angles together with the uniaxial compressive strength σ_{cm} and the deformation modulus E of the rock mass can be estimated by means of the spreadsheet given in Table 5 by entering any depth greater than 30 m.

Shallow tunnels and slopes

For shallow tunnels and slopes in which the degree of confinement is reduced, a minor principal stress range of $0<\sigma_3<\sigma v$ is used, where σv=depth × unit weight of the rock mass. In this case, depth is defined as the depth below the surface of the tunnel crown or the average depth of a failure surface in a slope in which a circular failure 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 5 allows the user to enter the depth below surface and the unit weight of the rock mass. The vertical stress σv calculated from the product of these two quantities is then used to calculate the rock mass properties. An example is given for a tunnel or a failure surface at a depth of 25 m below the surface. The estimated properties of this heterogeneous rock mass, from the Hoek–Brown criterion, are:

1. Cohesive strength c=0.136 MPa
2. Friction angle ϕ=36.6°
3. Rock mass compressive strength σ_{cm}=0.54 MPa
4. Deformation modulus E=1,000 MPa

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References

Bieniawski ZT (1974) Estimating the strength of rock materials. J SAfr Inst Min Metall 74:312–320

Brown ET (ed) (1981) Rock characterization, testing and monitoring – ISRM suggested methods. Pergamon, Oxford, pp 171–183

Hoek E (1994) Strength of rock and rock masses. ISRM News Journal 2(2):4–16

Hoek E, Brown ET (1997) Practical estimates of rock mass strength. Int J Rock Mech Min Sci Geomech Abstr 34(8):1165–1186

Hoek E, Kaiser PK, Bawden WF (1995) Support of underground excavations in hard rock. AA Balkema, Rotterdam

Hoek E, Marinos P, Benissi M (1998) Applicability of the Geological Strength Index (GSI) classification for very weak and sheared rock masses. The case of the Athens Schist Formation. Bull Eng Geol Environ 57(2):151–160

Jaeger JC (1971) Friction of rocks and stability of rock slopes. The 11th Rankine Lecture. Géotechnique 21(2):97–134