Estimating the geotechnical properties of heterogeneous rock masses such as Flysch

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Paper published in *Bull. Engg. Geol. Env.* **60**, 85-92, 2001

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Estimation des propriétés géotechniques des masses rocheuses hétérogènes, comme le flysch

Paul Marinos[1] and Evert Hoek[2]

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, comme 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 problémes à leur classification par les systèmes reconnus des classifications géotechniques. Dans ce travail une méthodologie pour l' estimation du GSI et l' évaluation des propriétés des masses rocheuses de flysch, est présentée.

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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 following 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 in the following text.

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 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:

- 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 of the obtained from a laboratory uniaxial compressive strength or UCS test),
- a constant m_i that defined the frictional characteristics of the component minerals in these rock elements, and
- the Geological Strength Index (GSI) that relates the properties of the intact rock elements to those of the overall rock mass.

These parameters are dealt with in the following sub-sections.

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 Figure 1.

Practically every sample obtained from rock masses such as that illustrated in Figure 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 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.

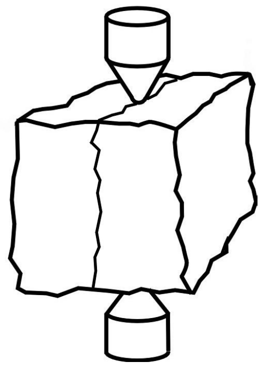
Figure 1: Appearance of sheared siltstone flysch in an outcrop

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 considering such tests for routine engineering projects.

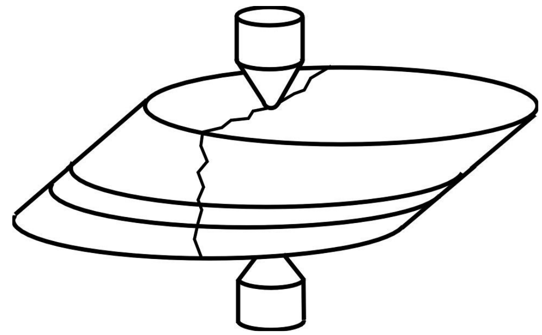
One of the few courses of action that can be taken to resolve this dilemma is to use the Point Load Test on samples in which the load can be applied normal to bedding or schistosity block samples. The specimens used for such testing can be either irregular pieces or pieces broken from the core as illustrated in Figure 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 since 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, where $I_s = P/D^2$. P is the load on the points and D is the distance between the points (Brown, 1981).

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 a qualitative description is given in Table 1, based on Hoek and Brown (1997).



a. Test on sample chosen from surface exposure.



b. Test on sample broken from diamond drill core.

Figure 2: Point Load test options for intact rock samples from heterogeneous rock masses.

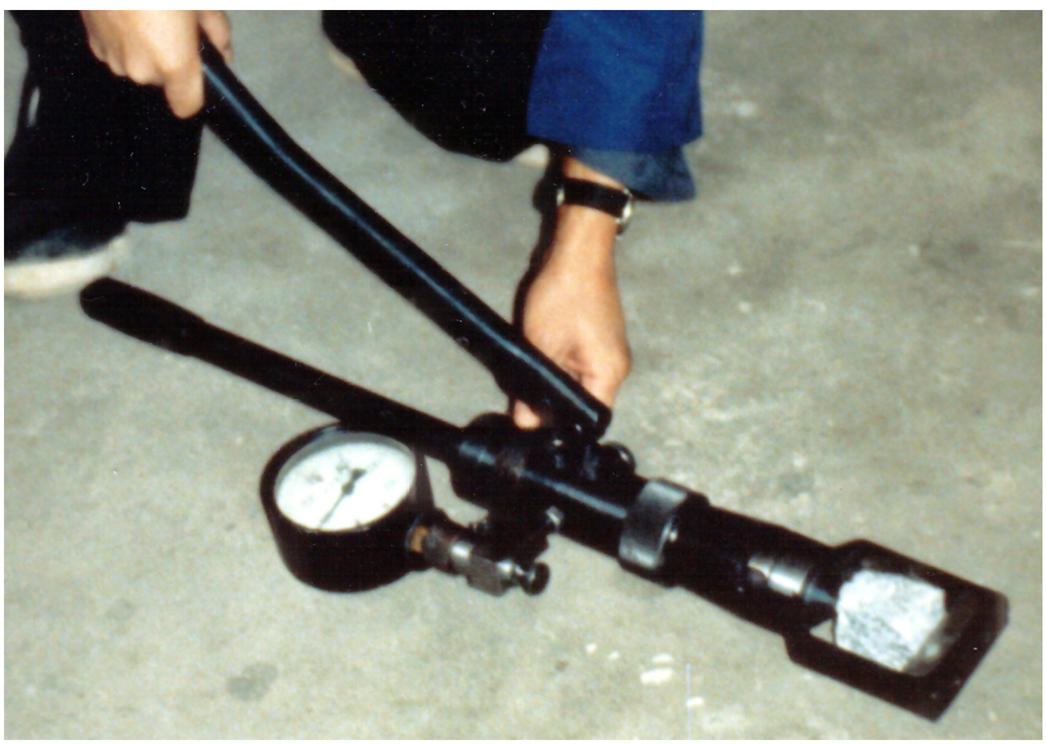


Figure 3: “Portable” point load test device for use in the field.

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

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	Concete, 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.

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 it has a significant influence on the strength characteristics of rock.

When it is not possible to carry out triaxial tests, for the reasons discussed in the previous section, 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” described 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.

Geological Strength Index GSI

The Geological Strength Index (GSI) was introduced by 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 poroxysme 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 sandstone may also include conglomerate beds. The fine grained layers contain siltstones, silty shales and clayey shales. Rarely and close to its margins, limestone beds or ophiolitic masses may be found. 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.

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 alterations 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.

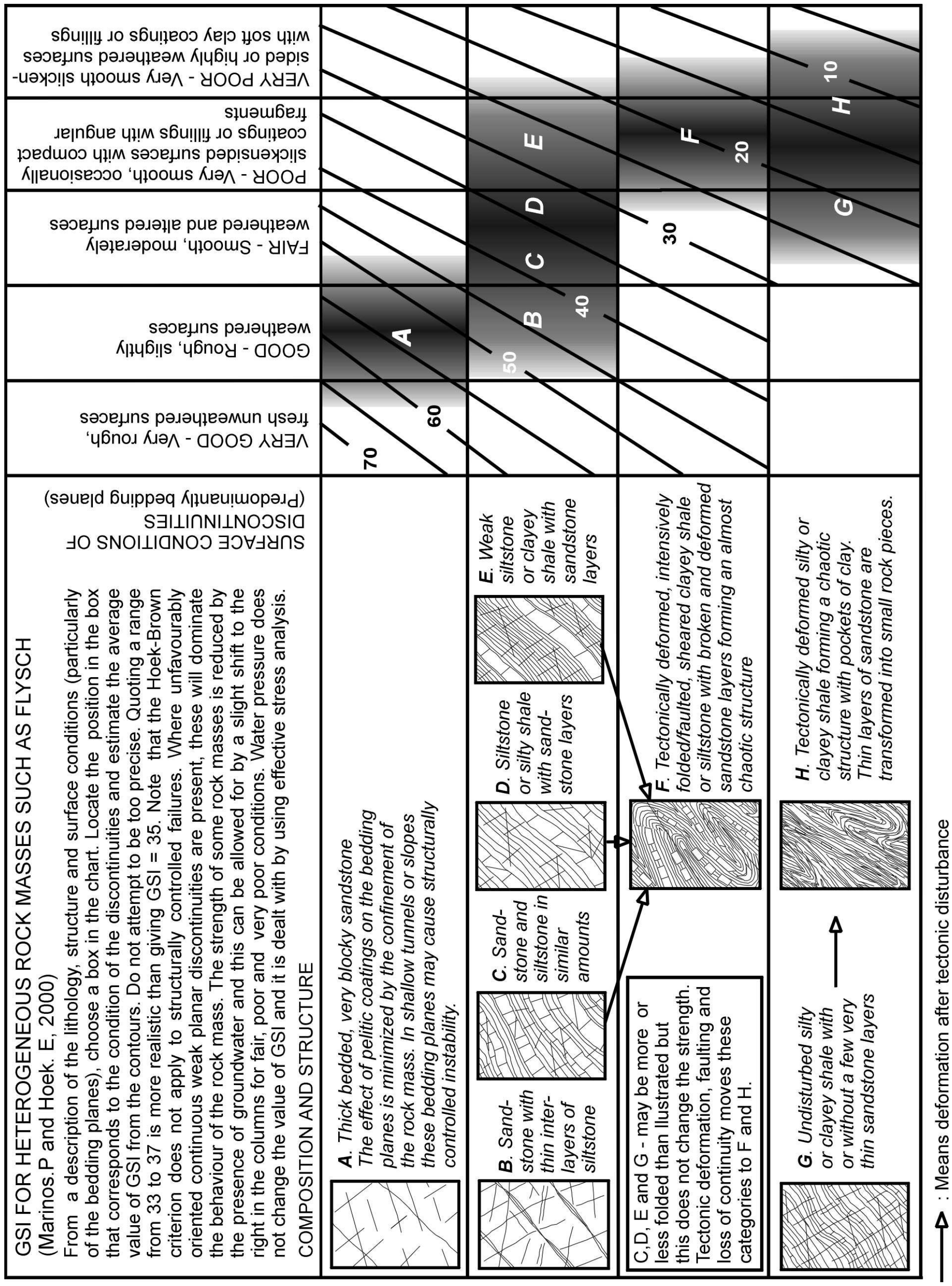
Table 2: Values of the constant m_i for intact rock, by rock group[3]. Note that values in parenthesis are estimates. The range of values quoted for each material depends upon the granularity and interlocking of the crystal structure – the higher values being associated with tightly interlocked and more frictional characteristics.

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
SEDIMENTARY	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
SEDIMENTARY	Non-Clastic	Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
SEDIMENTARY	Non-Clastic	Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
METAMORPHIC	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6	Gneiss 28 ± 5	
METAMORPHIC	Foliated*			Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Diorite 25 ± 5 Granodiorite (29 ± 3)			
IGNEOUS	Plutonic	Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
IGNEOUS	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
IGNEOUS	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	
IGNEOUS	Volcanic	Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

* 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.

[3] 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. GSI estimates for heterogeneous rock masses such as Flysch.



Geotechincally, a flysch rock mass has the following characteristics:

- Heterogeneity: alterations of competent and incompetent members,
- Presence of clay minerals,
- Tectonic fatigue and sheared discontinuities, often resulting in a soil-like material,
- 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 that is 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) but there is no compressional disturbance.

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 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 materials 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 hydroelectric 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 program was carried out in which the specimens were transported to a small laboratory about 5 kilometres 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 gave very consistent results and values of uniaxial compressive strength σ_{ci} and constant m_i that were considered reliable.

When laboratory testing is not possible, point load tests, using equipment similar to that illustrated in Figure 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 Figure 4.

Figure 4 A. Thick bedded blocky sandstone. Note that structural failure can occur when dip of bedding planes is unfavourable.

Figure 4 B. Sandstone with thin siltstone layers. Small scale structural failures can occur when bedding dip is unfavourable.

Figure 4 C. Sandstone and siltstone in equal proportions

Figure 4 D. Siltstone or silty shale with sandstone

Figure 4 E. Weak siltstone or clayey shale with sandstone layers

Figure F. Tectonically deformed clayey shale or siltstone with broken sandstone

Figure 4 G. Undisturbed silty or clayey shale with a few thin sandstone layers

Figure 4 H. Tectonically deformed clayey shale

Figure 4: Examples of Flysch corresponding to descriptions in Table 3.

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 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 4.

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

Flysch type see Table 4.	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

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. The procedure 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).

For 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 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 5 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.

Table 5: 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:

σ_n	stress =	if(depth>30, sigci*0.25,depth*unitwt*0.25)
m_b	mb =	mi*EXP((GSI-100)/28)
s	s =	IF(GSI>25,EXP((GSI-100)/9),0)
a	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 7 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	x =	LOG((sign-sigtm)/sigci)
y	y =	LOG(tau/sigci)
	xy =	x*y x sq = x^2
A	A =	acalc = 10^(sumy/8 - bcalc*sumx/8)
B	B =	bcalc = (sumxy - (sumx*sumy)/8)/(sumxsq - (sumx^2)/8)
k	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	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)

* For depths below surface of less than 30 m, the average stress on the failure surface is calculated by the spreadsheet. For depths greater than 30 m the average stress level is kept constant at the value for 30 m depth.

The example included in Table 5 is for a rock mass with an intact rock strength σ_{ci} = 10 MPa, a constant m_i = 10 and a Geological Strength value of GSI = 30. The depth below surface is 25 m. The estimated properties for this rock mass are a cohesive strength c = 0.136 MPa, a friction angle ϕ = 36.6°, a rock mass compressive strength σ_{cm} = 0.54 MPa and a deformation modulus E = 1000 MPa.

Acknowledgements

The development of the methodology presented in this paper was carried out by the authors as part of their consulting services to Egnatia Odos S.A., an organisation responsible for the design and construction of a 680 km highway across northern Greece. Particular acknowledgements are due to Mr Nikos Kazilis, head of the tunnelling department of Egnatia Odos S.A, Mr George Agistalis, Mr Nikos Syrtariotis and Miss Maria Benissi for their assistance of in the preparation of this paper.

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