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THE APPROPRIATE USE OF GEOLOGICAL INFORMATION IN THE DESIGN AND CONSTRUCTION OF THE EGNATIA MOTORWAY TUNNELS

Εισηγητής : Paul G. Marinos and Evert Hoek

Ιωάννινα, 7 & 8/12/2001
"ΕΓΝΑΤΙΑ ΟΔΟΣ Α.Ε."
& Ε.Ε.Σ.Υ.Ε.

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ABSTRACT

The main geological conditions are presented for the Egnatia Motorway, an immense 680 km long project currently under construction across northern Greece. The need for an understanding of the regional geological "rationale" at any location is stressed in order that an accurate evaluation can be made of problems that might arise along the route. The Egnatia Highway runs across the entire width of Greece traversing almost perpendicularly the main geotectonic units. Thus, there are a great variety of geological situations that impose the need of a different approach in designing the engineering structures of the Highway. From a geological point of view, each geotectonic unit displays different particularities in terms of weak rock masses and of the possibility of unstable arrangements of rocks. Such knowledge is used in the choice of alignments to avoid areas of instability or areas of old large landslides.

In turn, for the chosen alignment, the emphasis is placed in defining the exact geotechnical rock mass model. Only when this model has been constructed can the necessary geotechnical parameters for the design of cuts, tunnels and embankments be chosen.

1. INTRODUCTION

In Roman times, during the 2nd century B.C, Via Egnatia was the first highway built by the Romans outside Italy and the first to cross the Balkan peninsula from the Adriatic Sea to the west to Marmara and Black Sea to the east.

The four-lane highway, which is currently under construction in northern Greece, is named after that old Roman road. The Egnatia Highway, with a length of 680km and containing 77 twin tunnels, runs across the entire width of Greece traversing the Pindos Mountain and almost perpendicularly the main geotectonic units of the country (Fig. 1). Thus, there are a great variety of geological situations and this fact imposes the need different approaches in designing the Highway. Each geotectonic unit displays different particularities in terms of weak rock masses and of the potential for instability.

Such knowledge led to a selection of alignments so as to avoid, often through tunnels, areas of slope instability or areas affected by old large landslides. The same knowledge allows design of all tunnels on a sound basis.

The authors are responsible for reviewing route selection, tunnel design and construction issues as well as safety and cost implications of the designs.

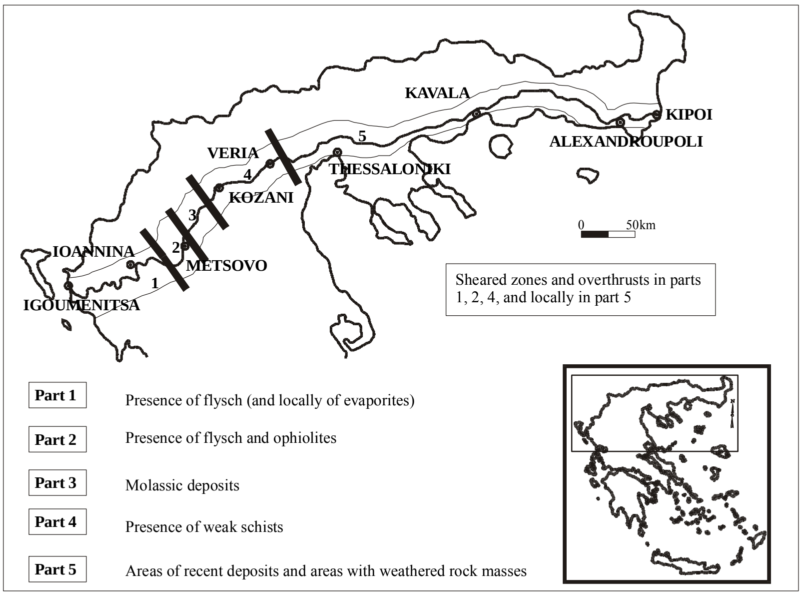


Figure 1: Alignment of Egnatia Highway and spatial setting of the major geological sections along with the geotechnically unfavorable characteristics.

2. THE GEOLOGICAL ENVIRONMENT

2.1 BASIC PRINCIPLES

From west to east, the Egnatia Highway can be subdivided into the following parts:

- **From Igoumenitsa to Metsovitikos River. Ionian geotectonic unit** (part 1 in Fig. 1)
  Flysch and alternations of various carbonate formations, mainly limestone, with very limited occurrence of schist are the typical rock masses. Local occurrences of gypsum and anhydrite in diapiric intrusions can be also encountered. The rocks are folded while large scale overthrusts, big faults and mylonitized zones are present in this region (Fig. 2).
- **From the Metsovitikos River to Metsovo tunnel. Pindos geotectonic unit** (part 2 in Fig. 1, 2)
  The area to be crossed consists mainly of flysch in various forms, characterized by intense folding, heavily sheared with numerous overthrusts. The degree of the tectonic deformation at some places drastically degrades the quality of the rock mass (Fig. 2).

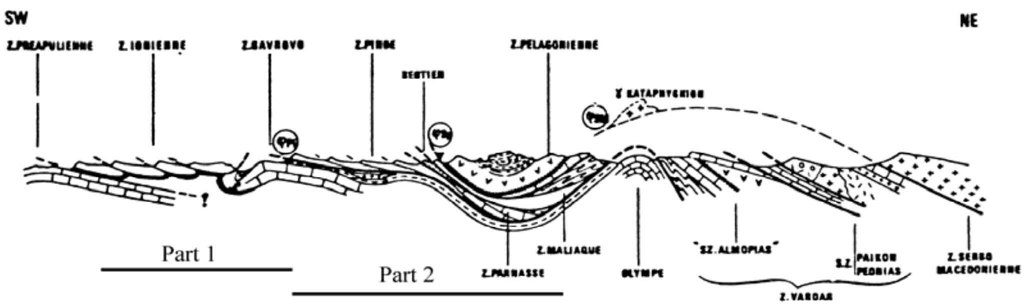


Figure 2: Schematic cross-section of the Hellenic Alps (AUBOUIN, 1979, from PAPANIKOLAOU, 1986).

- **From Metsovo tunnel to Panagia region. Nappe of Pindos ophiolites** (part 2 in Fig. 1, 2)
  Ophiolites comprise the predominant rock mass in this area, however they exhibit great heterogeneity regarding their degree of weathering and the occurrence of shear zones. Weak flysch is also present.
- **From Panagia to Siatista** (part 3 in Fig. 1)
  This region consists of molassic formations in the form of alternating thick-bedded conglomerates, sandstones and marls. From a tectonic point of view, the area is of relatively low disturbance and although weak rocks are present there is not any dramatic decrease of geotechnical qualities due to the absence of tectonic shearing.
- **From Siatista to Lefkopetra. Pelagonian geotectonic unit** (part 4 in Fig. 1)
  The area is characterized by the predominance of hard rocks such as marbles, gneisses and granites. The presence of tectonically weakened zones through faulting is very localized.
- **From Lefkopetra to Veria. Axios to Almopia geotectonic units** (part 4 in Fig.1)
  Phyllites, limestones and ophiolites are the usual rock masses in this area, while overthrusts and sheared zones are the main tectonic structures.
- **From Aliakmon River to Axios River flood plane to Thessaloniki region.**
  The entire area consists of recent alluvial fill which often exhibits insufficient natural compaction.
- **Section east of Thessaloniki to borders**
  The Serb-Macedonian massif and the Rhodope massif comprise the region. The basement consists of hard crystalline marbles, gneisses and granites. At some localities, the latter two appear weathered and are locally crosscut by faults with sheared zones within the rock mass. The Egnatia Highway passes also through areas of younger sediments such as marls and sandstones and areas of recent geological deposits with soft soils of loose or open structure.

2.2 TECTONICALLY ACTIVE AREAS

Although areas of tectonic activity do not exhibit intense dynamic characteristics in the northern part of the Hellenic region, as is the case for the central and southern Greece, the Egnatia Highway crosscuts active tectonic depressions such as the Volvi – Lagkadas trench and the Grevena – Kozani as well as possibly active structures at the western part of the alignment (the area of Paramithia has potential for strike slip activity).

2.3 SOME GEOMORPHOLOGIC PECULIARITIES ASSOCIATED WITH THE TECTONIC MODEL

The spatial distribution of the various geologic formations and geotectonic units has yielded some geomorphologic features whose final arrangement creates conditions of generalized slope instability on mountain slope scale. This fact is particularly significant in western Greece and in the Pindos mountain range. For instance, masses of competent formations (limestones) have overthrusted the soft and “ductile” flysch. The former comprise the highest parts of the eastern valley sides forming high cliffs which produce screes covering the flysch of the lower parts which are deformed and sheared due to the overthrusting. The screes allow water to percolate and dampen the geotechnically poor mass of flysch. Earthflows as well as old and new landslides are typical phenomena, often on impressive scales on such slopes. Additionally, the development of this instability of the flysch undermines the upper parts of the slopes provoking falls of limestone blocks which further weaken the flysch slopes downhill. Such unstable areas cannot be avoided along the entire Egnatia alignment and difficult tunnels and slopes have to be faced in several sections.

2.4 THE WEAK ROCK MASSES ALONG THE EGNATIA MOTORWAY

In the area of the civil engineering works of the highway, the weak rock masses, often in heterogeneous and chaotic forms, are the following:

1. different types of flysch (depending on the participation of weak siltstone-clayey members and the tectonic deformation). This tectonically deformed family of sedimentary rocks has properties that range from fair to extremely poor and designing tunnels, foundations and slopes in these materials presents serious challenges to geotechnical engineers
2. ophiolites (depending on tectonic deformation and on degree and non uniformity of weathering)
3. phyllitic schists (depending on weathering and tectonic pre-shearing)
4. sheared zones in hard rocks
5. weathered zones in gneisses, granites and schists.

3. ENGINEERING THE ALIGNMENT

Many times the situations described above result in very difficult engineering conditions that necessitate specific investigations and the use of sophisticated design methods. The ignorance of situations yielding such conditions may lead, at least, to delays but also to failures. These failures may develop not only during the course of construction, when engineering solutions can generally be found, but also in the operational stage of the Egnatia Highway. In many cases, the early detection of potential problems can justify even a drastic change of the alignment, when the cost is not prohibitive and the operational safety does not inherit the uncertainties of the initial alignment.

Along the route of the Egnatia Highway, the difficult geotechnical cases are classified as follows:

1. Areas with entire slope scale instability (fig. 3 and 4):
   a. marginally unstable areas or areas with deep relict earthflows (left slopes of Metsovitikos river valley)
   b. areas which have suffered old large scale landslides, even if at equilibrium today, as the mechanisms of instability continue to exist in a continuously weakened material (operational strengths equal to residual) (left slopes of Metsvitikos valley and Lefkopetra)
   c. areas of heavy tectonic disturbance (Kristallopigi – Psilorachi at Paramythia area)

In such cases, drastic modifications of the Highway alignment are considered (Georganopoulos and Kazilis, 1999).

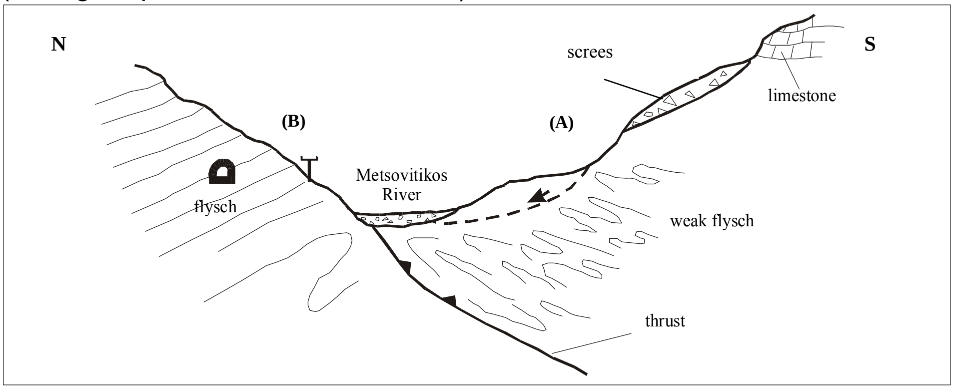


Figure 3: Drive through the Anthohori - Anilio section. Geotechnical problem: initial alignment (A)selected through a mild topography. However, this morphology was due to a series of old landslides in flysch not stabilised. Proposal for extensive draining including drainage gallery 5km of length. Solution: relocation of alignment at the northern side of Metsovitikos River (B) with tunnels alternating with bridges in rock. The assessed risk is much smaller than the initial proposal for draining of the old alignment (drawing not scaled).

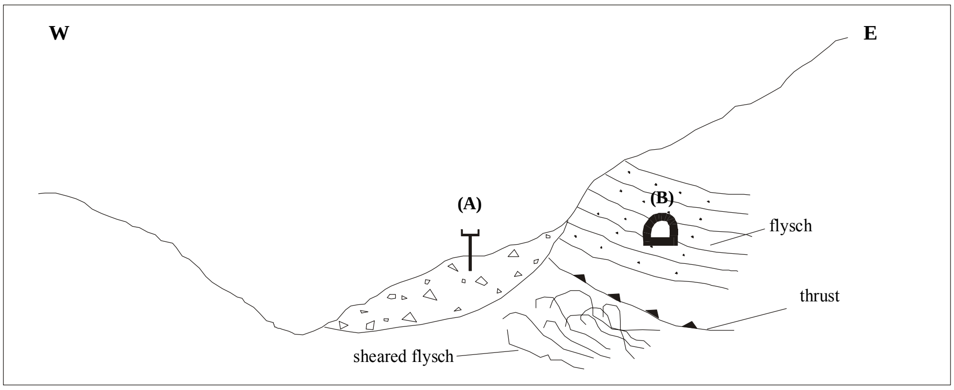


Figure 4: Section Anilio - west portal of Metsovo tunnel. Geotechnical problem: foundation of initially proposed long viaduc bridge in deep (40m) active landslide of flysch (A). Solution: avoiding of the whole unstable area with the construction of tunnel with a length of 2.120m (B) (not scaled).

2. Areas with instability emerging only from the construction of the cuts or the foundation of the embankments or bridges in case of improper design due to erroneous selection of geotechnical parameters for a predominantly weak rock mass (instability on the scale of slopes surrounding the road).

3. Areas of tunnel constructions consisting of tectonised and generally weak rock masses or rock masses crosscut by sheared zones. Design, after selection of the driving parameters, requires the use of advanced methods. It has been found that a reliable first estimate of potential problems can be given by the ratio of rock mass strength to in situ stress (HOEK and MARINOS, 2000). This is usually followed by a detailed numerical analysis of the tunnel’s response to sequential excavation and support stages. However, in some cases of extremely weak rock masses, the capability for theoretical analysis is limited and here sound technical judgment and experience from similar cases are valuable aids in design. In order to cope with face stability and high squeezing deformation, the use of heavy support with application of forepole umbrellas may be applied.

4. THE PROPER USE OF THE GEOLOGICAL INFORMATION IN THE DESIGN OF TUNNELS

Cases from tunnels with particular interest for the design are succinctly presented in this paragraph.

4.1 TUNNELS IN SECTION 1.1.6, PARAMYTHIA AREA

Tunnels S1 and S2 of 430 and 645 m replace the bridges originally planned for this route. Site investigations revealed potential instability problems in the bridge foundations and this led to the decision to re-route the highway into tunnels. The area is at the edge of the cliff of a “balcony” overlooking the plain of Paramythia and dominated by the mountain of Khionistra (a continuation of the Paramythia mountains).

The broader area was studied by geologists during the early 60s and an excellent publications by the Greek Geological Survey and the Institut Francais du Petrol (1966) is available. In Figure 5 we have reproduced a conceptual profile and a block geological model from the latter publication.

The predominant rockmass of the area is heavily fractured and brecciated limestone resulting from thrusting or from tectonic displacements and falls (tectonic rugs). The brecciated limestone in this area is suitable for tunnelling under conditions where an adequate vertical and lateral cover is available. This opinion was in response to a question regarding the use of the RQD index for evaluating rock mass conditions and, in this case, the RQD is often zero. We consider that RQD is not an appropriate tool for evaluating closely jointed rock masses and that the Geological Strength Index (GSI) is much more suitable. For the same reasons the RMR classification is inappropriate as it is heavily dependent upon the value of RQD. Hence, even when RQD and RMR values are very low, this type of rockmass, composed of tightly interlocking angular pieces of strong limestone, exhibits good stability under confined conditions (fig. 6). We believe that a GSI value of about 35-40 is appropriate for the most frequently occurring limestone breccia.

Excavation of the tunnel allows the rock mass ahead and above the tunnel crown to dilate and to lose its interlocking. This results in gravity driven ravelling which, if not controlled, propagates upwards to form a chimney. This ravelling continues until the chimney enters a stress environment in which arching of the material can occur. The correct way to prevent this type of failure is to confine the exposed surface by means of shotcrete or to increase the cohesive strength of the rock mass by grout injection.

In addition to the brecciated limestones described above, the tunnels may also cross adverse conditions associated with the occurrence of flysch. The breccias mentioned earlier are the result of the fragmentation of the limestones due to the compression they suffered as a result of the thrust of the Khionistra mountains from the E-NE and of another thrust over the flysch of the “Paramythia” synclinal. Thus flysch is present under the breccia and this material has been found in a number of boreholes. Furthermore due to the high tectonic compression during the alpine orogenesis, diapiric intrusion of evaporitic masses with gypsum, occurred in weak zones. This gypsum is present either as homogeneous compact masses, associated with the thrust zones next to the flysch, or as penetrations in the latter, often in a diffused manner.

No particular difficulties are normally associated in driving through gypsum. However, the flow of water through the gypsum may give rise to solution and hence it may be prudent to seal the tunnel, without the provision of drainage, in order that it does not act as a drain. Grouting of voids due to overbreak or solution may also be necessary to ensure that flow paths are minimised. Due to the potential presence of anhydrites, swelling may be present although, with the shallow cover, the concrete lining should accommodate this without problems. Checks should also be carried out on the chemical reactions of the gypsum and the concrete and, if necessary, specially formulated concrete may have to be used in tunnelling through gypsum rich areas. In the contact between the diapiric mass and the flysch poor quality a melange material will probably be present and this may require heavy support measures.

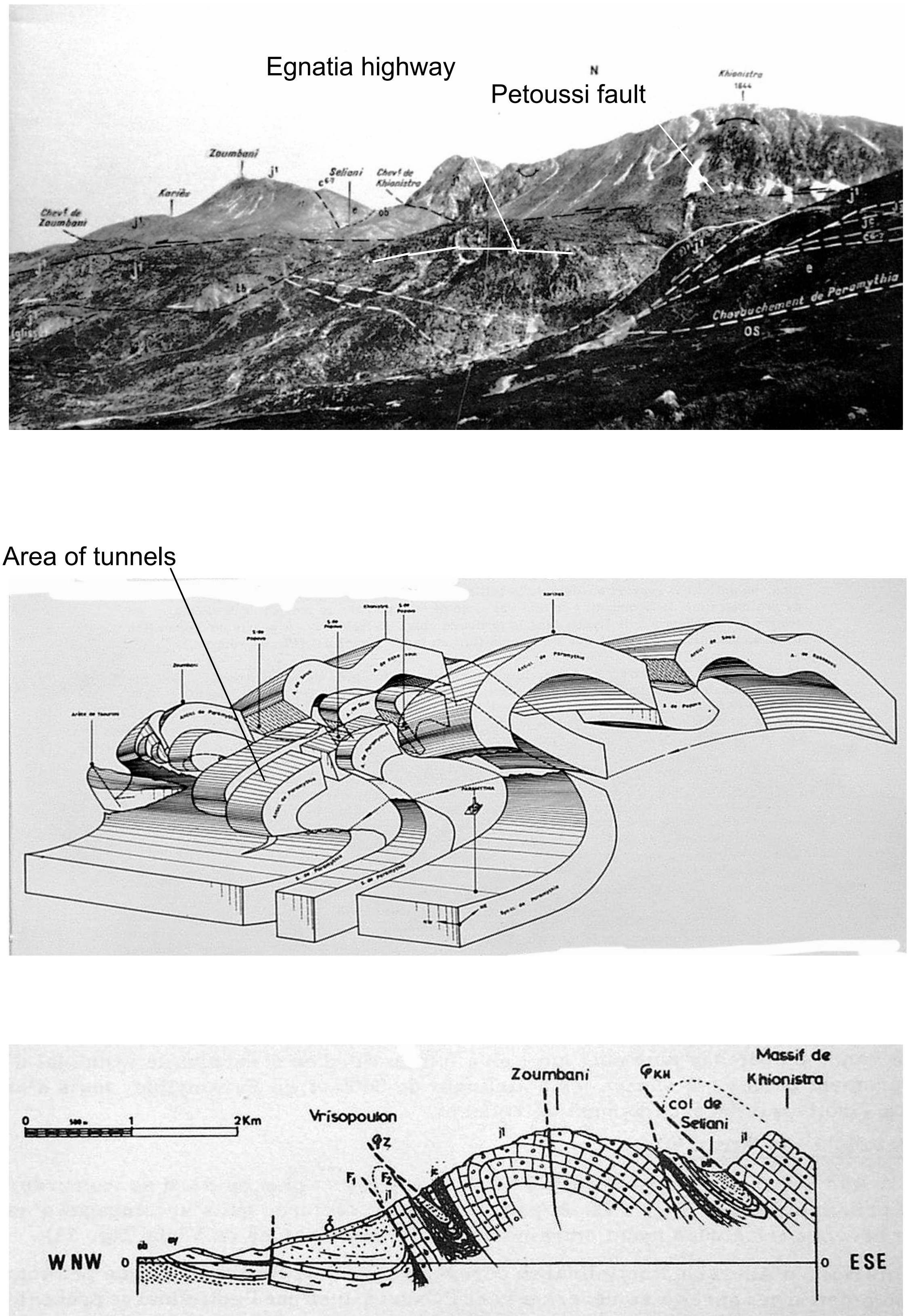


Figure 5: The geological model for the Khionstra-Paramythia trust and the Petoussi fault (from IGME – IFP, 1966)

Figure 6: Unraveling of the rock surrounding a borehole drilled into brecciated limestone.

4.1 DODONI TUNNEL

The Dodoni tunnel in northern Greece, with a length of 3.3 km is being driven in a limestone sequence with well developed bedding and possible local intercalations of siltstones or cherts a few cm or dm of thickness. The limestone encountered so far has behaved well and this behavior is expected to continue. However, significant overbreaks have occurred at some locations and these overbreaks were mainly due to instability of the fill in karstic cavities even in depths under a cover of 100 m (Fig. 7).

Figure 7: Typical appearance of a small karstic void partially filled with clay and silt; Dodoni Tunnel, northwest Greece, 2000 (Photo taken from site engineer)

Two major collapses occurred related to the presence of sinkholes at the surface with outcropping chimneys almost 100 m of height. The voids were filled with clayey material and pieces of broken rock and were prominently wet. The main collapse had a diameter of approximately 1.5 m in the tunnel and 3 m on the surface (Fig. 8), leading to 1200 m^3 of material falling into the tunnel.

In order to detect karstic cavities, pockets filled with soft and broken material, shear zones and gouge-filled faults, it was recommended that routine probe drilling ahead of the tunnel face should be carried out. Ideally, the probe hole should always be kept one tunnel diameter ahead of the advancing face and the most convenient way to achieve this is by drilling long holes (30 to 50 m) during maintainance shifts. As in all karstic voids, because of the irregular and unpredictable shape and location of weak zones, it is recommended that at least three probe holes should be drilled from the face at 10, 12 and 2 o'clock positions.

When a significant weak zone is detected additional probe holes were drilled to define the extent and shape of the zone as accurately as possible. In exceptional cases, one or two cored holes may be required to determine the nature of the filling material.

As a general rule grouting of the filling material within the cavity is a primary consideration in order to improve its cohesive strength. However, it has to be realized that the effects of such grouting are highly unpredictable, depending on the nature of the filling materials. Thus the most secure solution involves the creation of a protective umbrella over the tunnel.

4.2 DRISKOS TUNNEL

The main part of the tunnel of 4.570 m long will be excavated through flysch comprising zones of siltstone or claystone with thin beds of sandstone alternating with sandstones with thin beds of siltstones. Conglomerates may also occur. Folds and faulted zones are present (fig. 9).

Figure 8: Collapse of the filling of a karstic chimney crossed by Dodoni Tunnel. The collapse outcropped on the surface about 100 m over the tunnel

The most serious issue in the Driskos tunnel is the possibility of significant squeezing conditions in the central part and the provision of very heavy support in this stretch in the central part where weather flysch may occur (fig. 10).

Figure 9: Medium to thick-bedded sandstones and thin-bedded siltstones. It is noted the angular folds and the fracturing (faults) along the axial plains (Photo taken by the designer).

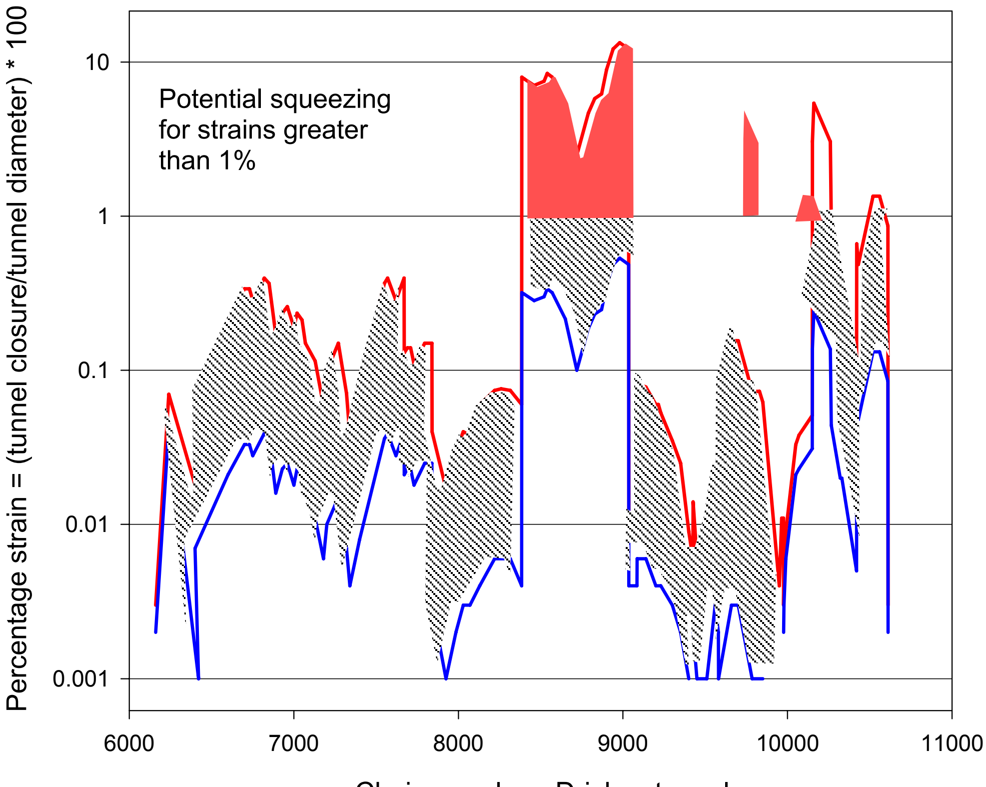


Figure 10: Analysis of potential squeezing problems along the Driskos tunnel, based upon strength of the rock mass and the cover depth. Plot of calculated percentage strain along the tunnel for upper and lower bound rock mass strength values assumed by the Designer.

Indeed sections from the Western portal of the Driskos tunnel have exhibited larger than anticipated deformations (fig. 11). In some stretches of the tunnel this deformations are obviously the result of overloading of the primary support system. The reasons for the overstressing and subsidence are established on the basis of back-analysis of the behaviour of the already constructed tunnels. It was recommended that the rock mass classification should be revised, if necessary, to take into account the experience to date and that the support design be critically examined to see whether changes are required.

Figure 11a: Spalling of the shotcrete surrounding a highly stressed lattice girder.

Figure 11b: Deformed rockbolt face plate due to overstressing of the primary support system.

4.4 ANTHOCHORI TUNNEL

This tunnel is located at the left side of the Metsovitikos valley. The area to be crossed by this 690 long tunnel is located close to the frontal area of the big thrust where the Pindos Unit has moved tectonically over the Ionian Unit. Obviously the main thrust movement is associated with satellite shears within the thrusted body. These shears are generally marked by the reddish siltstone sequence of the Pindos flysch. The tunnel is located in such a tectonic body of the Pindos Unit. This body is between two shear zones (satellite thrusts) close to the big Pindos/Ionian thrust (see fig. 12). It has thus suffered from large compression and very weak rock masses have been produced and are likely to be found close the shear zones (fig. 13).

In the central part of the body it is reasonable to expect less deformed sandstone flysch with more rock-like behaviour. However, this material is likely to be cut by small shears. As the planes of the thrust are undulating on a large scale, the distance between the tunnel and the presheared weak zones will vary. Consequently, frequent shifts from good sandstones or sandstones/siltstones to weak flysch with a significant presence of sheared and very weak siltstone (claystone) has to be anticipated.

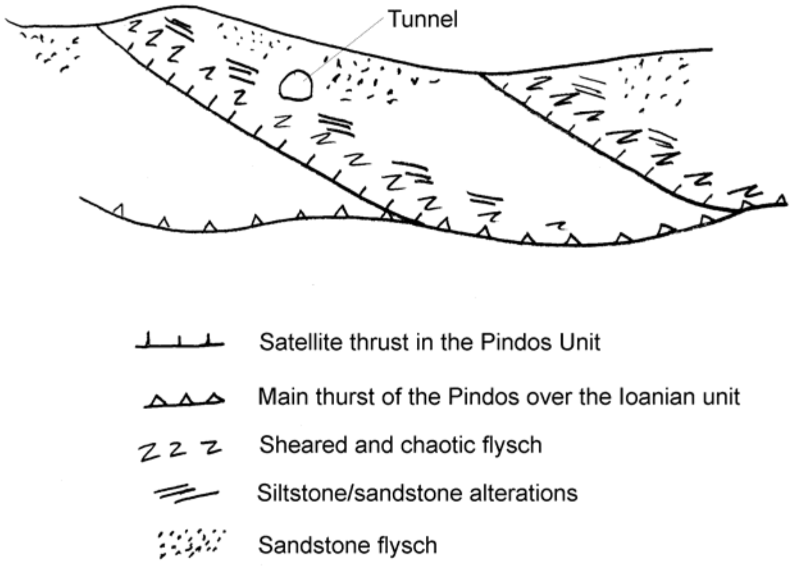


Figure 12: Indicative cross-section at the eastern part of the Anthohori tunnel. Shear-thrust planes have highly variable direction and dip and may intersect the central part of the tunnel alignment at frequent intervals. Not to scale

Figure 13: Core from Borehole MB3, Anthochori tunnel

The geological conditions in the Anthochori tunnel are as severe as any encountered thus far on the Egnatia project. The sheared flysch, very weak and deformable produced large deformations ahead of and around the tunnel and great care is taken to control these deformations at present.

Figure 14: Panoramic view of the east portal of the Anthochori tunnel.

4.5 ANILIO TUNNEL

The solution of a tunnel of 2050 m long has been selected in order to bypass an old landslide and areas of potential instability near the Anilio village.

The tunnel not yet under construction will be driven in a relatively disturbed area and it will cross a series of thrusts and faults associated with weak flysch material. In the western part of the tunnel, zones of sheared clay, shales have been recognised in outcrops and in depths. However, it is very probable that zones of more competent rock will also be encountered. At the eastern part of the tunnel the situation should be significantly better due to the presence of thick massive sandstones. However, a change in the strike and dip of the strata in this region may reduce the length of this competent unit of sandstones at the tunnel level.. The groundwater level is anticipated to be above the tunnel elevation but the low permeability of the weaker rock units will probably limit the water flow into the tunnel.

The eastern portal area is covered by landslide debris. The depth of the unstable material is not very great and protection measures are implemental.

Thus the geological conditions will be difficult over a significant proportion of the tunnel but can be controlled by the effective implementation of conventional construction and support techniques for weak or, locally, very weak rock masses.

4.6 TUNNEL S4 AT LEFKOPETRA

The presence of an active landslide in the initially proposed eastern portal area of the S4 tunnel has been clearly recognized.

From the comprehensive geological mapping that was performed by the designer, the chaotic structure of the material that forms the area of the S4 tunnel was largely proved. Additionally, the geological data allow the conclusion that the area is situated at the thrust zone of two distinct geotectonic units: the phyllites and slates of the Almopia Unit over the marbles and gneisses of the Pelagonian Unit. These chaotic materials are interpreted by the designers either as fluvioglacial deposits (due to their extreme heterogeneous granulometry) or as an old debris flow. In both cases the loose brecciated initial nature of the bed rocks in the front of this big thrust contributed to the production of this chaotic melange of more recent geological age. A recommendation for realignment of the tunnel was accepted.

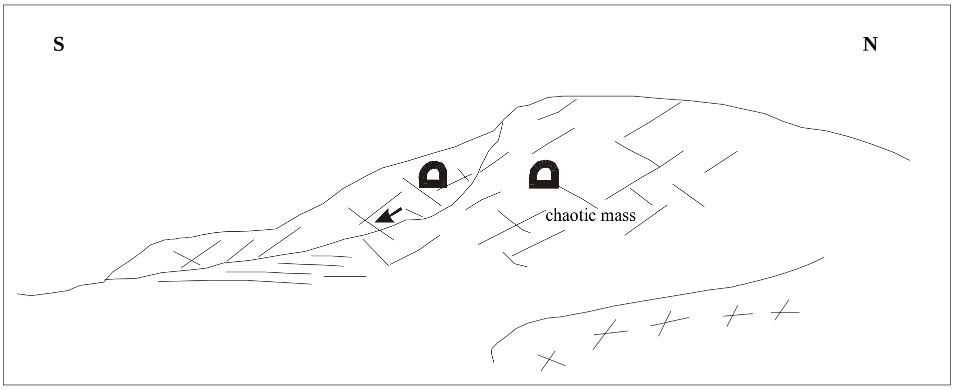


Figure 15: Section Lefkopetra-Veria. Geotechnical problem: landslide in area of portal of tunnel Σ4 and in the area of associated B4 bridge with chaotic rock mass. Solution: modification of alignment and avoidance of the area of the great landslide. Embankment at the site of the bridge (not scaled).

Special and heavy design was performed for the portals in these chaotic and unstable materials. For the stability of the west portal extensive drainage has been installed and is operating well; piles for reinforcement of the lower portal slopes were cast too (fig. 16).

Figure 16: West portal excavation for S4 tunnel. Drainage pipes are shown projecting from the slope and a partially completed cast in place pile is shown in the foreground.

This east portal is located in the loose and clayey chaotic rock mass at the boundary of the major slide. In addition to trimming the slopes and the installation of drainage, the principal reinforcing elements proposed for stabilising the slopes are 1.2 m diameter cast in place piles spaced at 2.4 m centre to center (fig. 17). The stability of the slopes below the portal is also considered. The highway embankment will provide buttressing for part of this lower slope and it will be necessary to extend a fill buttress further along the toe of the slope. Consideration is given to driving the tunnel from the western portal only until stabilisation measures have been implemented at the eastern portal.

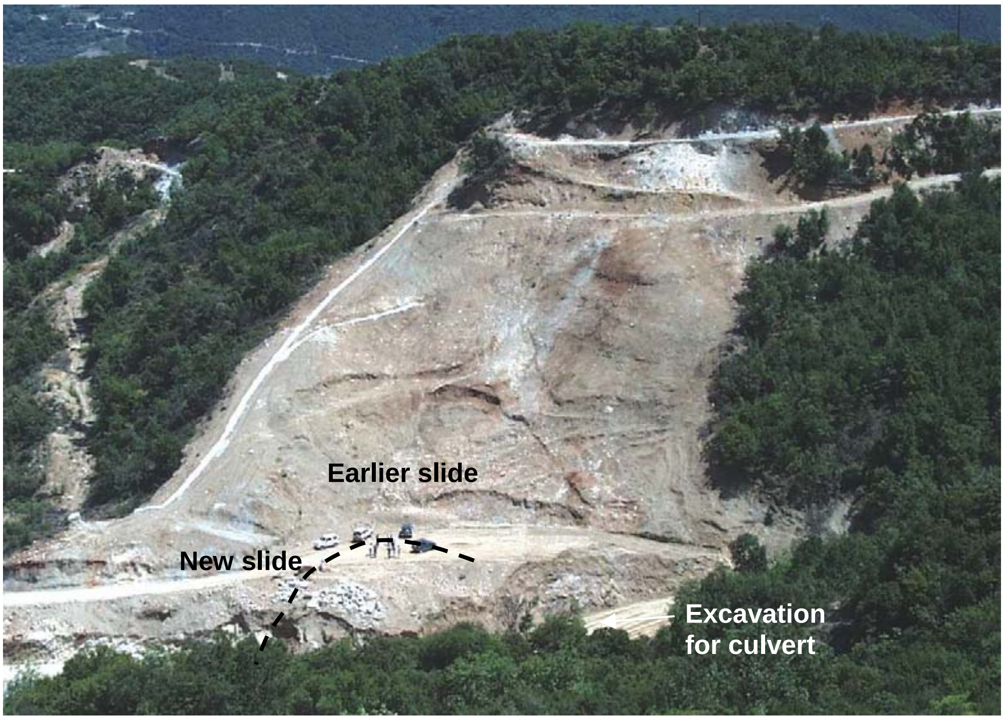


Figure 17: Small slide in the east portal excavation of S4 tunnel in Section 5.2.3.

In tunneling the rock mass was, as anticipated, heterogeneous and generally weak. According to the Designer, the face experienced in cases malanges as in the following table imposing heavy support (fig. 18)

Property	
GSI	5-15
σ_{ci} MPa	2
m_i	15
σ_{cm} MPa	0.05-0.11
E MPa	106-188
Friction angle	16-24
Cohesion (kPa)	20-40
Strength/stress ratio	0.046 - 0.1

Figure 18: Appearance of face of tunnel S4 under construction. Depth below surface 40 m and maximum recorded displacement is 45 mm. Support: 12 m long 114 mm diameter forepoles at 0.6 m centres, steel sets with elephant foot support, fibre-reinforced shotcrete and fibreglass face reinforcement

4.7 TUNNEL S3

During construction a number of cracks were observed in the slope above the portal excavations (fig. 19 and 120). In spite of the implementation of changes or support, cracks have continued to develop as the tunnels have been advanced.

The rock consists of mixed gneiss and phyllite, with an almost chaotic structure. We believe that the disturbed nature of this material is due to its tectonic history and that it is in situ material rather than landslide debris.

Figure 19: Current appearance of the portals for Tunnel S3 in Section 5.2.3. Note the very shallow cover over the tunnels.

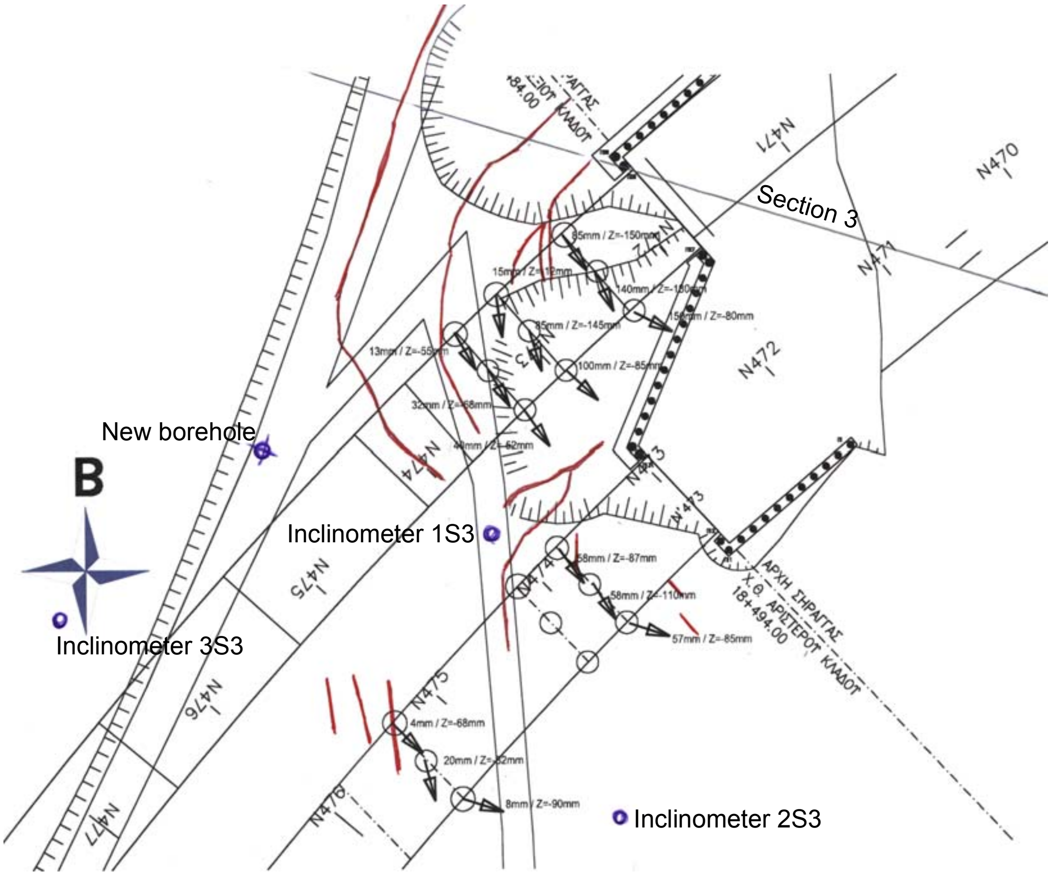


Figure 20: Plan of the S3 eastern portal area showing approximate crack locations, inclinometer and borehole locations and vertical and horizontal displacements measured in the tunnels.

Surface cracking is consistent with the subsidence that could be anticipated for large tunnels excavated in poor quality rock masses with a very small cover, both vertically and laterally. By design modification the tunnel proceed in the top heading excavation under controlled deformations.

5. THE GEOLOGICAL INPUT FOR THE ESTIMATE OF ROCK MASS PROPERTIES

Having define the basic geological model, design can start on a sound background and selection of rock mass properties is the next step. However the outcropping geological materials do not other permit high quality sampling and consequently performance of laboratory tests to derive the design geotechnical parameters is difficult. Additionally sampling is not representative of the rock mass due to heterogeneity of most formations. Nor is it realistic or even feasible to carry out in situ tests. Therefore, the only solution in order to acquire reasonable geotechnical parameters, is indeed to rely upon the use of the rock mass classification schemes, that have become customary in geotechnical engineering (RMR, Q, GSI) and that are correlated with the basic geotechnical parameters needed for the design.

The basic input consists of estimates or measurements of the uniaxial compressive strength (σci) and a material constant (mi) 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 (1994) 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).

But the most important component of the Hoek – Brown failure criterium is the process of reducing the material constants σci and mi from their "laboratory" values to appropriate in situ values. This is accomplished through the Geological Strength Index GSI first introduced by E. Hoek, 1997 and extended by E. Hoek, P. Marinos and M. Benissi, 1997 and E. Hoek and P. Marinos, 2000.

Careful consideration has been given to the precise wording in each box of the GSI chart and to the relative weights assigned to each combination of structural and surface conditions, in order to respect the geological conditions existing in nature.

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

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.

[1] 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[2]. 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 * 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.

[2] 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.

Having defined the parameters σci, mi and GSI as described above, the follow 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.

6. THE 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 of discontinuities 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 failure 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.

In order to give the most probable range of GSI values for rock masses of various rock types that most usually occur the Egnatia road route, a series of indicative charts are presented in tables 3 to 9. Deviations may certainly occur but these are the exceptions. From the charts it can be seen:

6.1 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.

6.2 SILSTONES, CLAYSTONES, MARLS, CLAYSHALES

Siltstones, claystones or marls 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 if effected by tectonic processes). In shales, either silty or clayey, the role of weak schistosity planes is more pronounced, which cannot however induce a real 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.

6.3 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 great depths the confinement of the mass reduce the significance of the bedding planes and GSI may be higher than primary considered; thus in this cases GSI may be of limited value. In the chart of Table 5 the limestone series with thin interlayers or films of clayey, marly or silty nature is of course not considered.

6.4 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).

6.5 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 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.

6.6 GNEISS

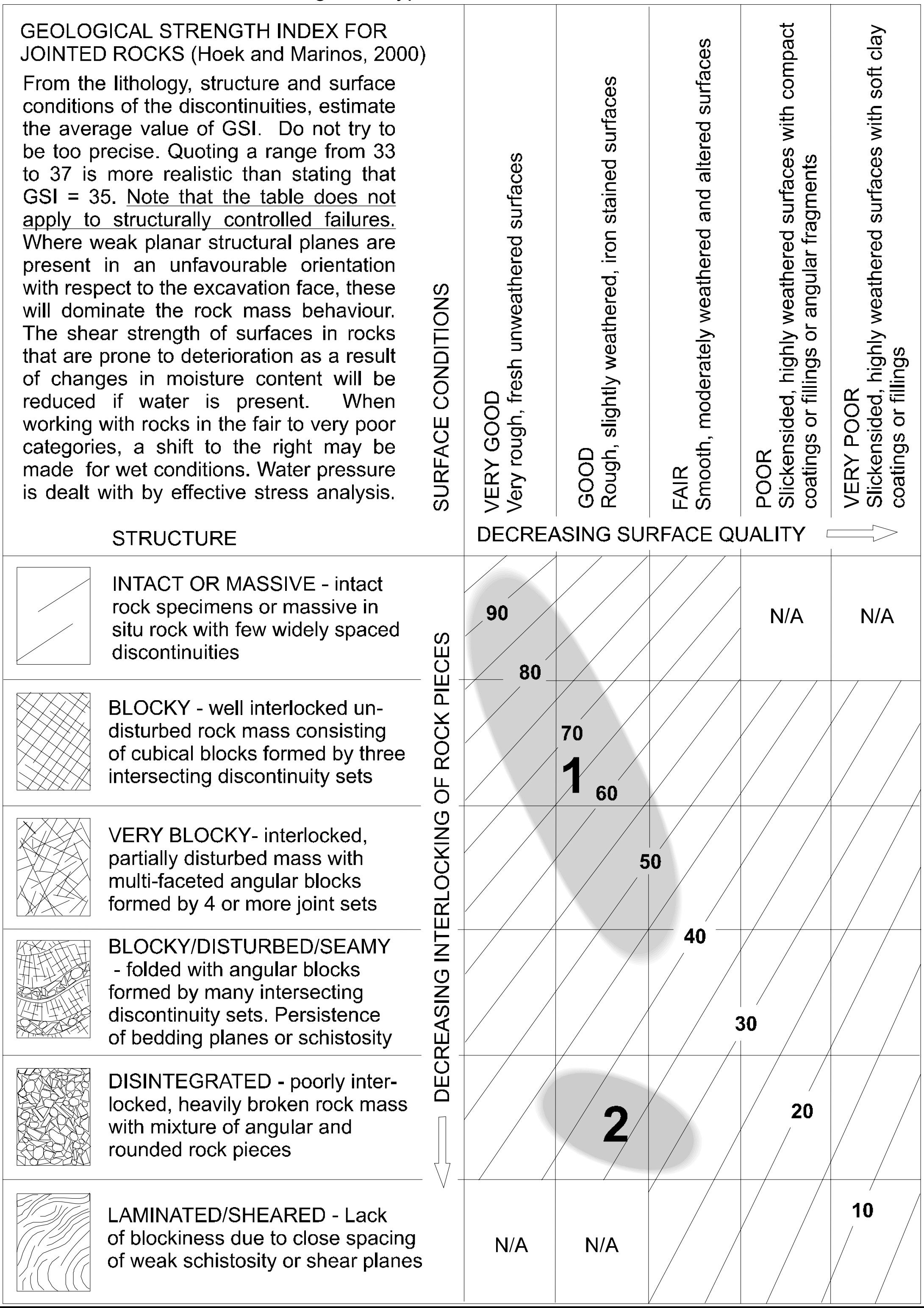
Compared to sound granitic masses a slight not significant displacement of the assigned range downward and to the right of the GSI chart may be seen. Same comments as for the granite apply when gneiss is weathered.

6.7 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. However in greater depths the defective role of schististity plane is considerably reduced and much higher GSI values have to be considered.

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.

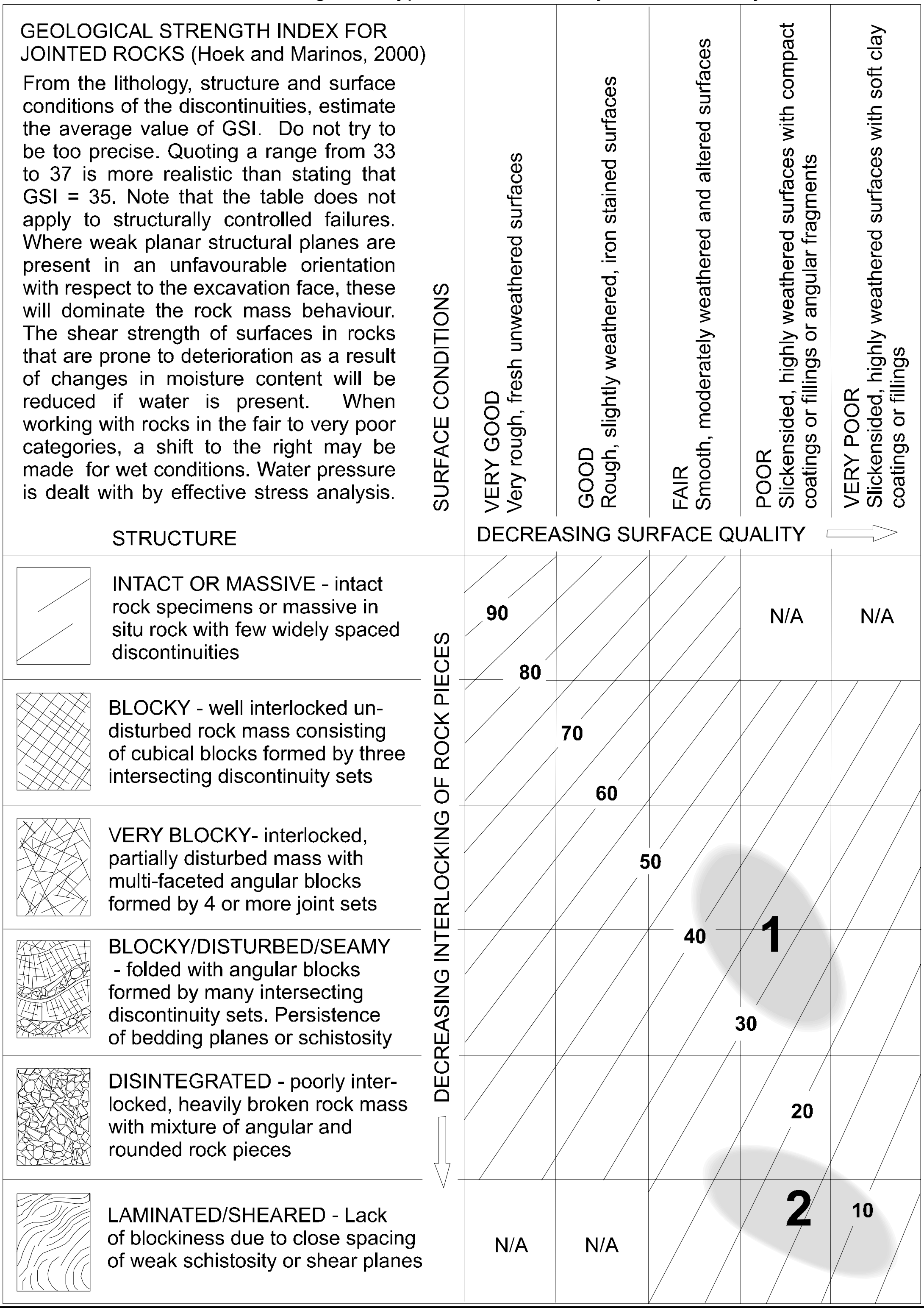
Table 3: Most common GSI ranges for typical sandstones.*



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

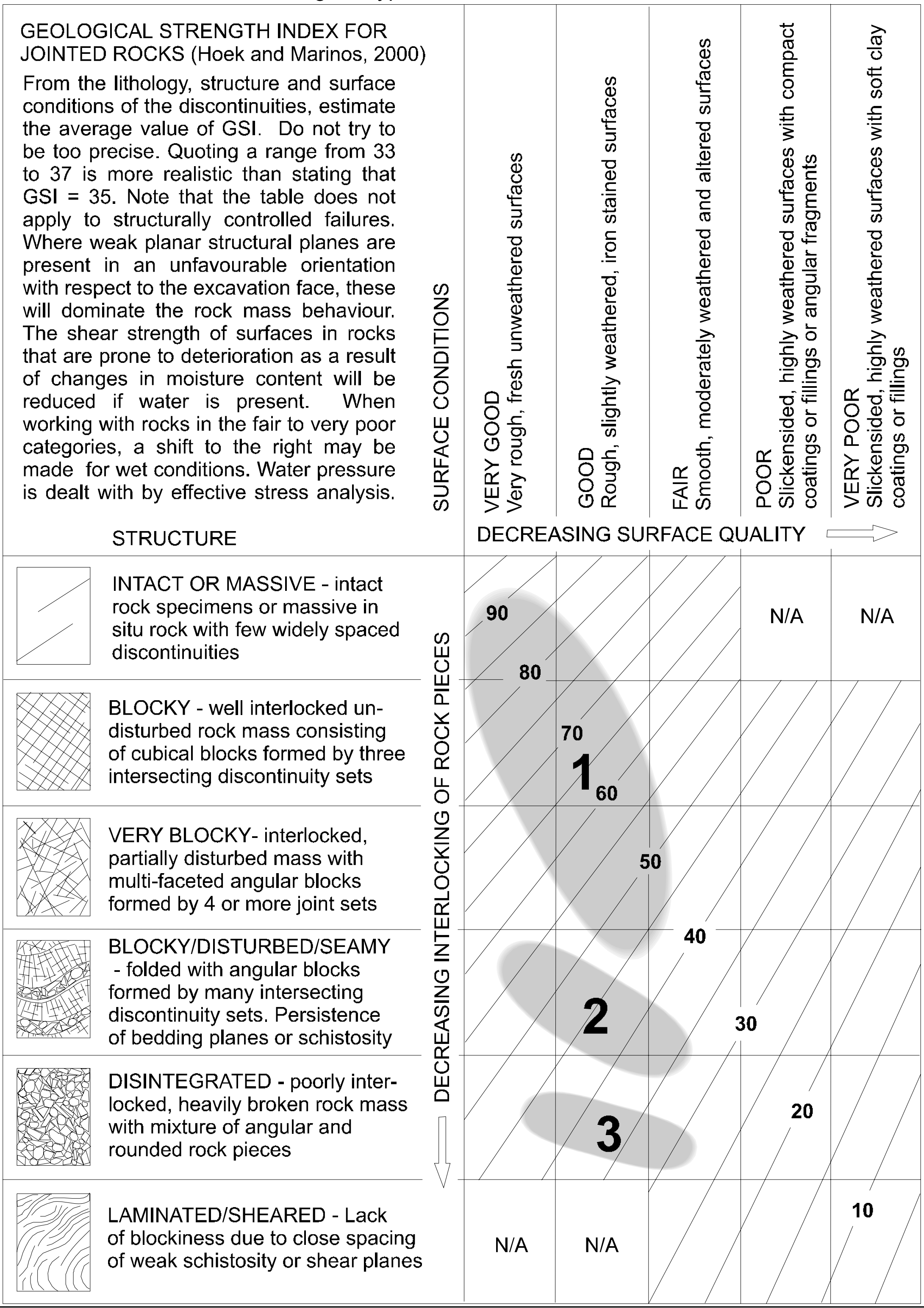


*_WARNING_:
The shaded areas are indicative and may not be appropriate for site specific design purposes.
Mean values are not suggested for indicative characterisation; the use of ranges is recommended

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

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

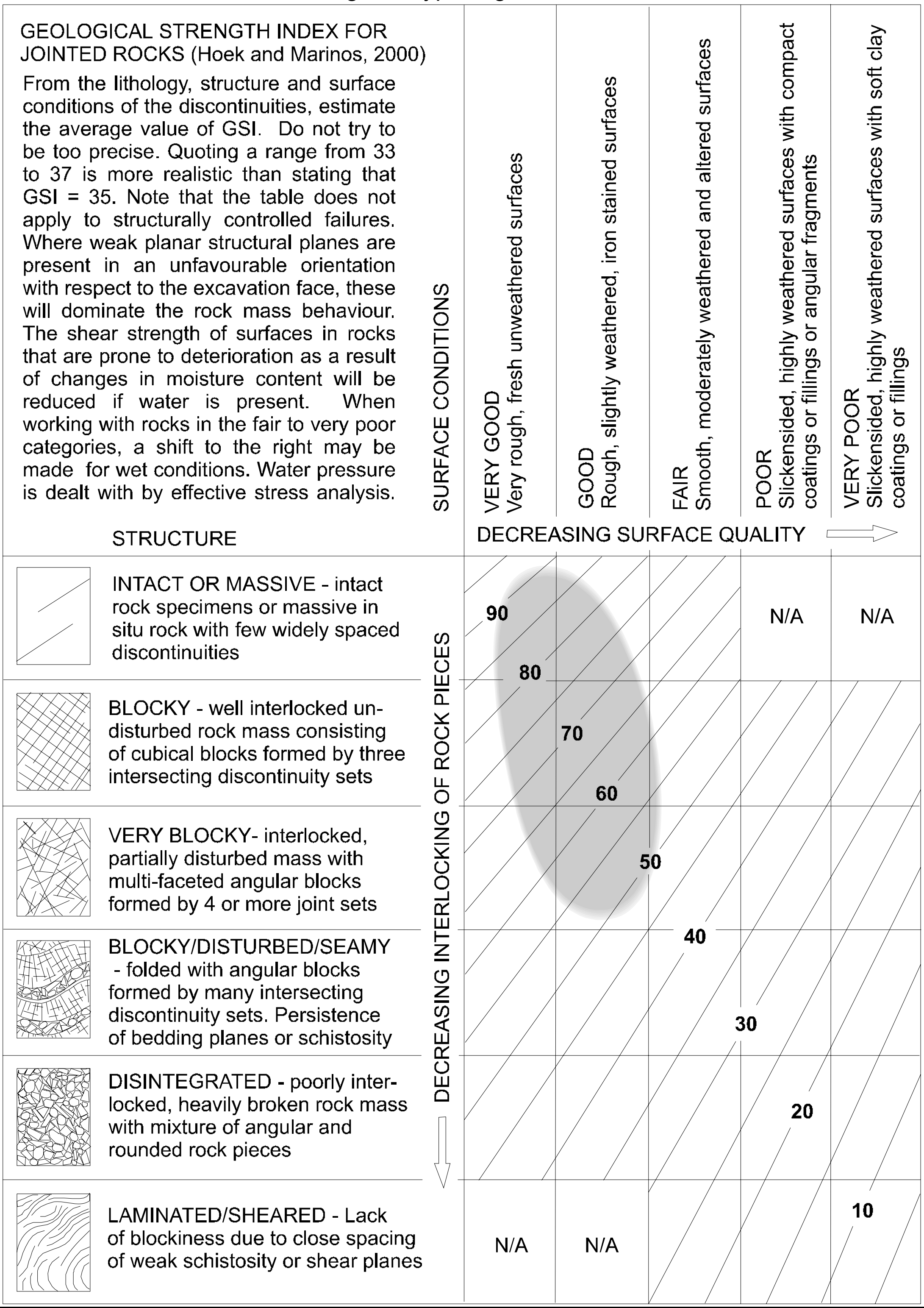
Table 5: Most common GSI range of typical limestone.*



*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
2. Thin bedded
3. Brecciated

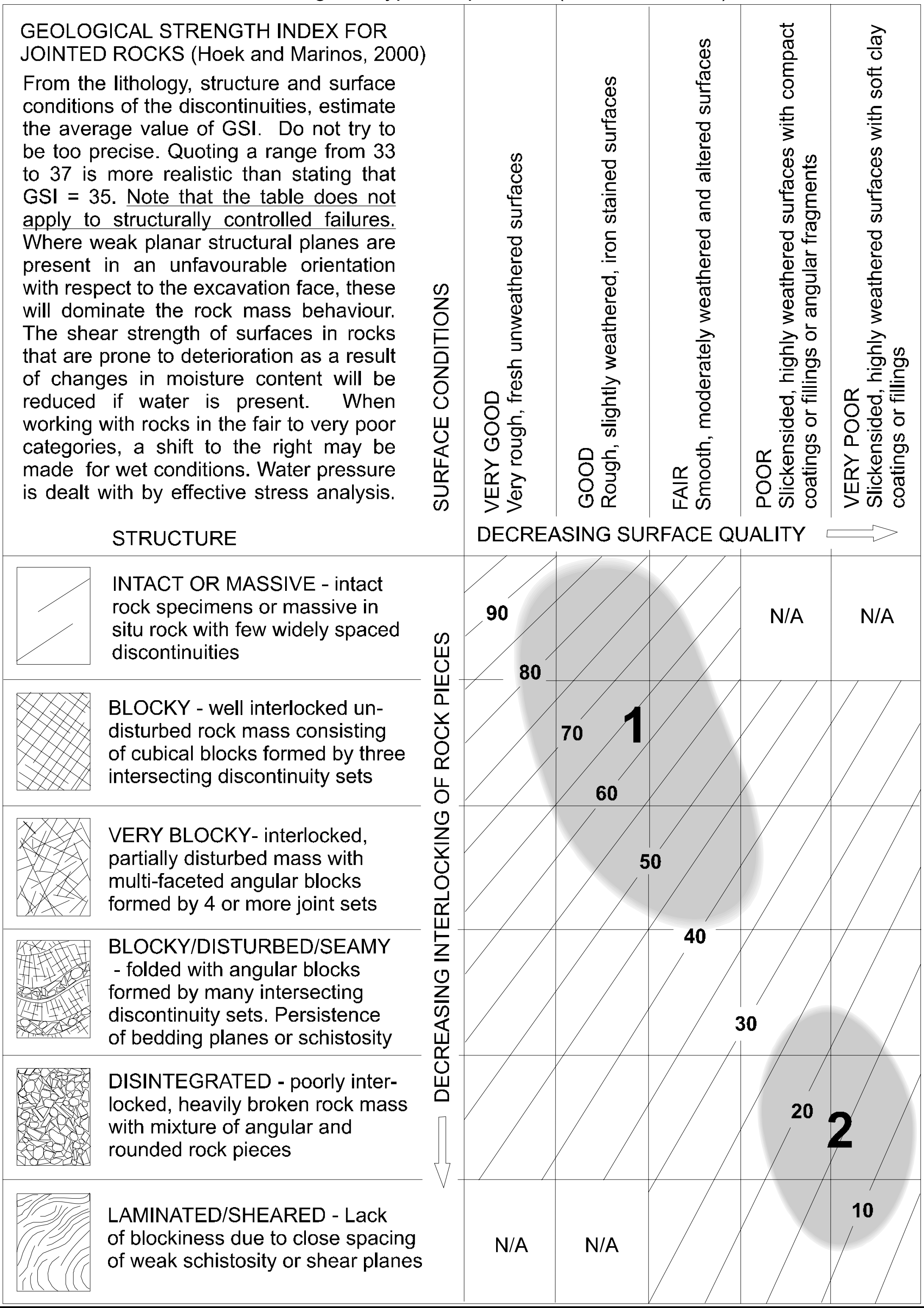
Table 6: Most common GSI range for typical granite.*



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.

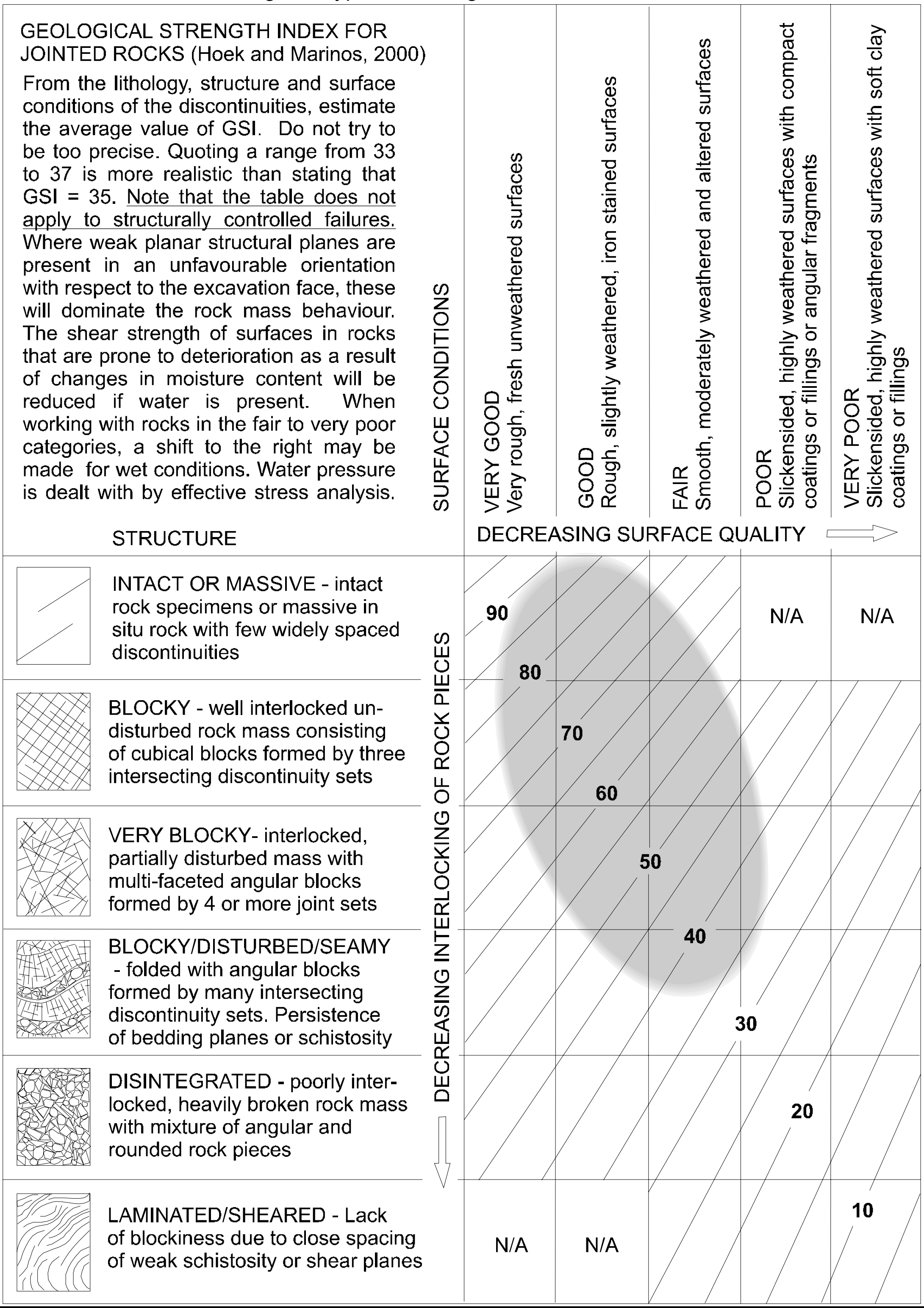
Table 7: Most common GSI range for typical ophiolites (ultrabasic rocks).*



*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. Fresh
2. Serpentinised with brecciation and shears

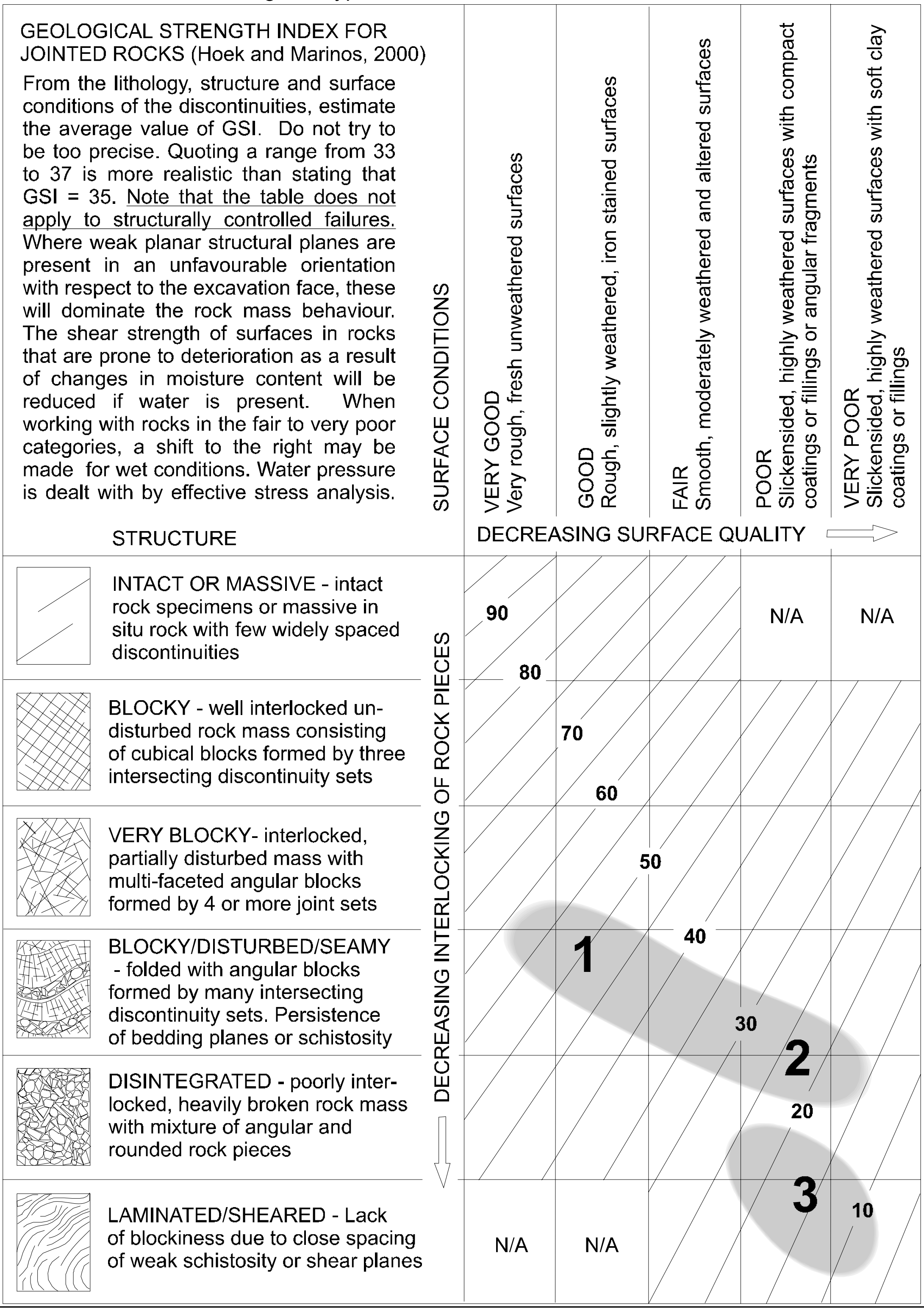
Table 8: Common GSI range for typical sound gneiss.*



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 9: 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

6.8 THE PARTICULAR CASE OF HETEROGENEOUS ROCK MASSES SUCH AS FLYSCH

Flysch consists of alternations of clastic sediments that are associated with orogenesis. Typical 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. 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 10.

Selection of σci and mi 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 σci and mi 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 (Marinos and Hoek, 2000) 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 11.

Table 10: GSI estimates for heterogeneous rock masses such as flysch.

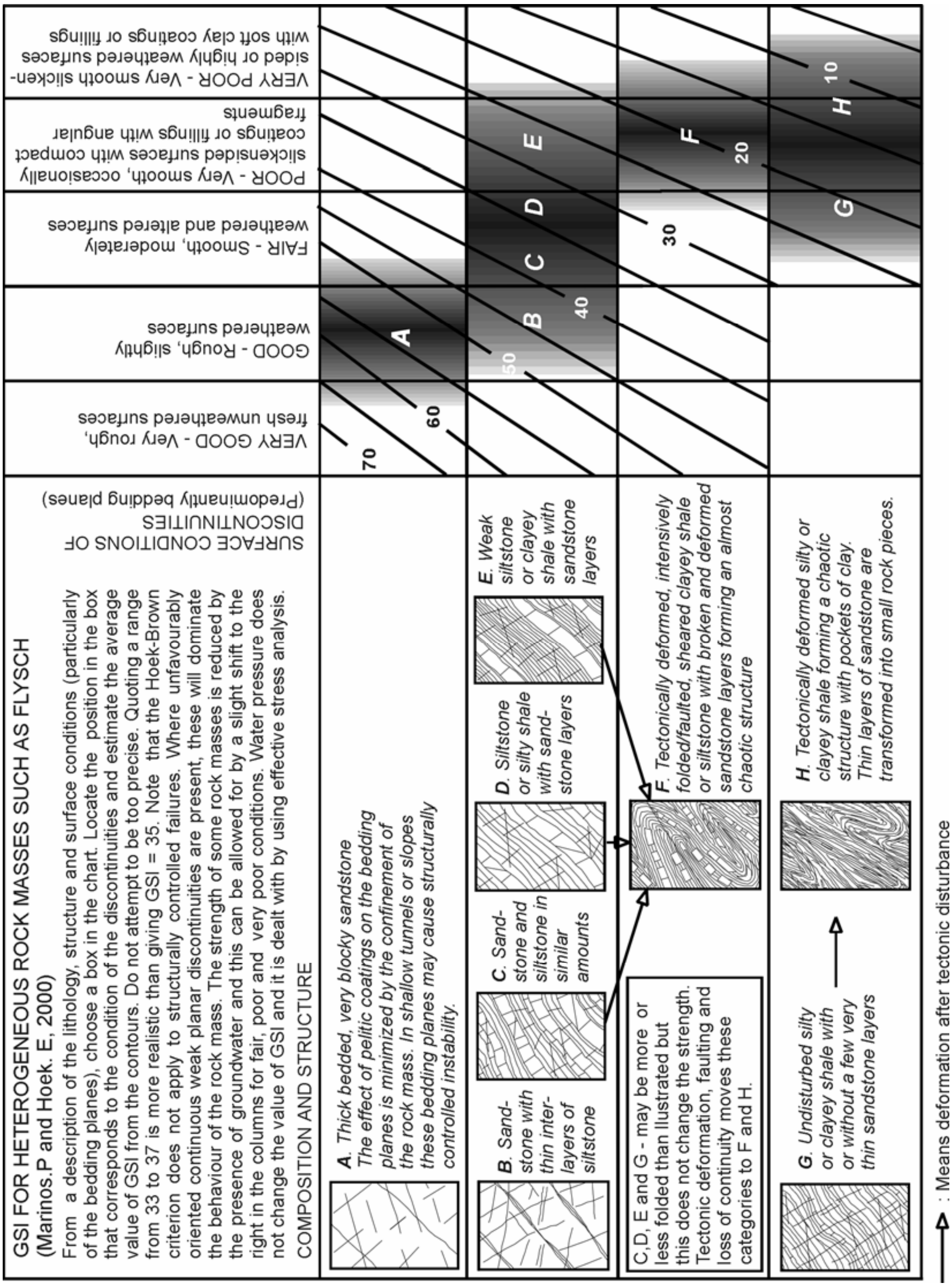


Table 11: Suggested proportions of parameters σ_{ci} and m_i for estimating rock mass properties for flysch (Marinos, P., Hoek, E., 2000).

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

CONCLUSIONS

The Egnatia Highway runs across areas with many problematic geological behavior regarding the structure and quality of the rock mass of the various formations, particularly in the western sections of the project. In the eastern sections the problems are more localized. The detection of the geological conditions on a mega-scale was necessary and of critical importance for the choice of the alignments.

This project has emphasised the need for an integrated approach that involves engineering geology, soil mechanics and rock mechanics in solving problems that cover the entire spectrum of geotechnical engineering.

In particular, a sound understanding of the geology of the region and of the origin and tectonic history of each of the rock units is essential in tackling the engineering problems of tunnel deformation and crossing effectively all geological deficiencies

ACKNOWLEDGEMENTS

Particular acknowledgments are due to “Egnatia Odos S.A.” and to Mr. Nikos Kazilis, Head of the Tunnelling Department and all his colleagues in the Department.

REFERENCES

- Georganopoulos, Chr. And Kazilis, N. (1999). The Northern alignment of Metsovo: Conception, design, construction. Egnatia Scientific Meeting, Ioannina.
- Hoek, E. (1994). Strength of rock and rock masses, ISRM News Journal, 2(2), 4-16.
- Hoek, E. and Brown, E.T. (1980), b). Empirical strength criterion for rock masses. J. Geotech Engng. Div., ASCE, 106 (GT 9), 1013-1035.
- Hoek, E. and Brown, E.T. (1997). Practical estimates or rock mass strength. Int. J. Rock Mech. & Mining Sci. & Geomechanics Abstracts, 34(8), 1165-1186.
- Hoek, E., Marinos, P. and 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. Engg. Geol. Env. 57(2), 151-160.
- Hoek, E. and Marinos, P., (1998-2000). Report of Panel of Experts on Egnatia Highway project. Reports prepared for Egnadia Odos S.A.

- Marinos, P. and Hoek, E., (2000). Estimating the geotechnical properties of heterogeneous rock masses such as flysch. (submitted for publication). Bulletin of Engineering Geology 60, 85-92.
- Marinos, P. and Hoek, E. (2001). From the Geological to the rock mass model. Driving the Egnatia Highway through difficult geological conditions. 4th Congress of the Hellenic Society for Soil Mechanics and Foundation Engineering.
- Rawlings, C.G., De Silva, R., Kazilis, N. (2001) et al. The Tunnels of the Egnatia Motorway project, Northern Creece, Underground Construction 2001, IMM, BTS, London.