CrossMark

ORIGINAL PAPER

A revised, geotechnical classification GSI system for tectonically disturbed heterogeneous rock masses, such as flysch

Vassilis Marinos[1]

Abstract Use of the geological strength index (GSI) rock mass classification system and the associated m, s and a parameter relationships linking GSI with the Hoek–Brown failure criterion provides a demonstrated, effective and reliable approach for prediction of rock mass strength for surface and underground excavation design and for rock support selection for most "normal" rock masses. One of the key advantages of the index is that it allows characterization of rock masses difficult to describe, such as flysch, and the geological reasoning it embodies, allowing adjustments to be made to its ratings to cover a wide range of rock masses and conditions compared to a typical engineering approach. Flysch, having high heterogeneity in its petrographic nature and a tectonically disturbed structure, forms very weak rock masses in many cases and needs a particular geotechnical classification according to the engineering geological characteristics it presents. After a decade of application of the GSI for the classification of heterogeneous rock masses (Marinos and Hoek 2001), this paper attempts to re-evaluate or verify the original values and to contribute to the appropriate selection of the index for various conditions. A revised GSI diagram for heterogeneous rock masses, such as flysch, is presented, where a certain range of GSI values for every rock mass type is proposed according to the siltstone-sandstone participation and their tectonic disturbance. Data from the design and construction of a large number of tunnels in a variety of geological conditions were assessed for this purpose. In addition to the GSI values, the selection of the appropriate "intact" rock properties for this type of heterogeneous rock mass is also discussed, where characteristic σ_{ci}, E_i and modulus ratio (MR) values are proposed.

Keywords GSI classification system · Flysch · Heterogeneous rock masses · Rock engineering design · Tunnel · Hoek and Brown failure criteria · Intact rock properties

Introduction

In the last several decades, there has been rapid development in almost all stages of geotechnical design. Analysis and computational methods are fields where great progress has been made. However, regardless of the great capabilities offered by the state-of-the-art computational tools, the results still involve uncertainties due to the difficulties in defining design parameters. Hence, attention should be given to the definition of the geotechnical parameters and behavior of the rock mass in engineering works.

Geotechnical design requires numerical estimation of a series of geotechnical properties and parameters of the geomaterial for a safe, sound and realistic analysis of an engineering project. Estimation of rock mass properties can be achieved by one or a combination of more than one of the following methods: a) laboratory testing, b) in situ testing, c) use of rock mass classifications (GSI, RMR, Q, etc.) and d) back-analysis. However, samples may not be representative of the rock mass due to the structural disturbance, jointing and heterogeneity of most formations. Additionally, in situ tests are often not realistic or feasible to carry out, and in any case, uncertainties regarding them being representative of the rock mass always remain. To estimate, for instance, reasonable geotechnical parameters for the design of tunnel support categories before

✉ Vassilis Marinos
marinosv@geo.auth.gr

1 School of Geology, Faculty of Sciences, Aristotle University of Thessaloniki, GR-541 24 Thessaloniki, Greece

construction, where back-analysis is not possible, there is no option but to rely upon the use of a rock mass classification scheme that is correlated with the basic parameters needed for the design. Back-analysis is the best way to estimate the geotechnical parameters when construction has started by evaluation of the deformation measurements, provided such measurements exist and are reliable. This process can be used to validate or modify the parameters for subsequent stages of construction.

This need is currently satisfied primarily with the development of engineering geological investigation tools, powered here by geotechnical classification systems. Initially, classification systems were generated to estimate the immediate support measures for underground excavations as an empirical design. On the basis of this approach, hundreds of kilometers of tunnels have been successfully excavated and supported. More recently, though, geotechnical design tools have changed, and numerical methods enable detailed analysis of rock mass progressive failure and its interactions with support measures.

Reliable estimates of the strength and deformation characteristics of rock masses are required for almost any form of analysis concerning a tunnel, a slope or a foundation. Hence, a method for the estimation of rock mass properties from the intact rock properties and the joint characteristics was necessary (Carter and Marinos 2014). The Hoek and Brown failure criterion (Hoek and Brown 1997; Hoek et al. 2002, Hoek and Marinos 2007) would be of no use if it could not be immediately linked with engineering geological observations for the nature and structure of the rock mass. Hoek (1994) proposed a method for obtaining estimates of the strength of jointed rock masses based upon an assessment of the interlocking of rock blocks and the condition of the surfaces between these blocks. For such an assessment, the "Geological Strength Index' (GSI) was introduced. GSI has been developed over the years (Hoek et al. 1998; Marinos and Hoek 2000; Hoek et al. 2002; Marinos, Marinos, and Hoek 2005; Hoek, Marinos and Marinos 2007) to meet the needs of users and cases that were not initially considered.

Use of the GSI rock mass classification system and the associated m, s and a parameter relationships linking GSI with the Hoek–Brown failure criterion provides a proven, effective and reliable approach for prediction of rock mass strength and deformability for most "normal" rock masses. The geological strength index (GSI), more than the other available classification systems, was formulated to attempt to characterize rock masses from a more geological perspective rather than a typical engineering approach to better meet the need for delivering reliable input data. Back-analyses of tunnels, slopes and foundation behavior using the approach attest to its reliability.

One of the most significant problems of numerically based classification systems is that they suffer markedly when stretched to cove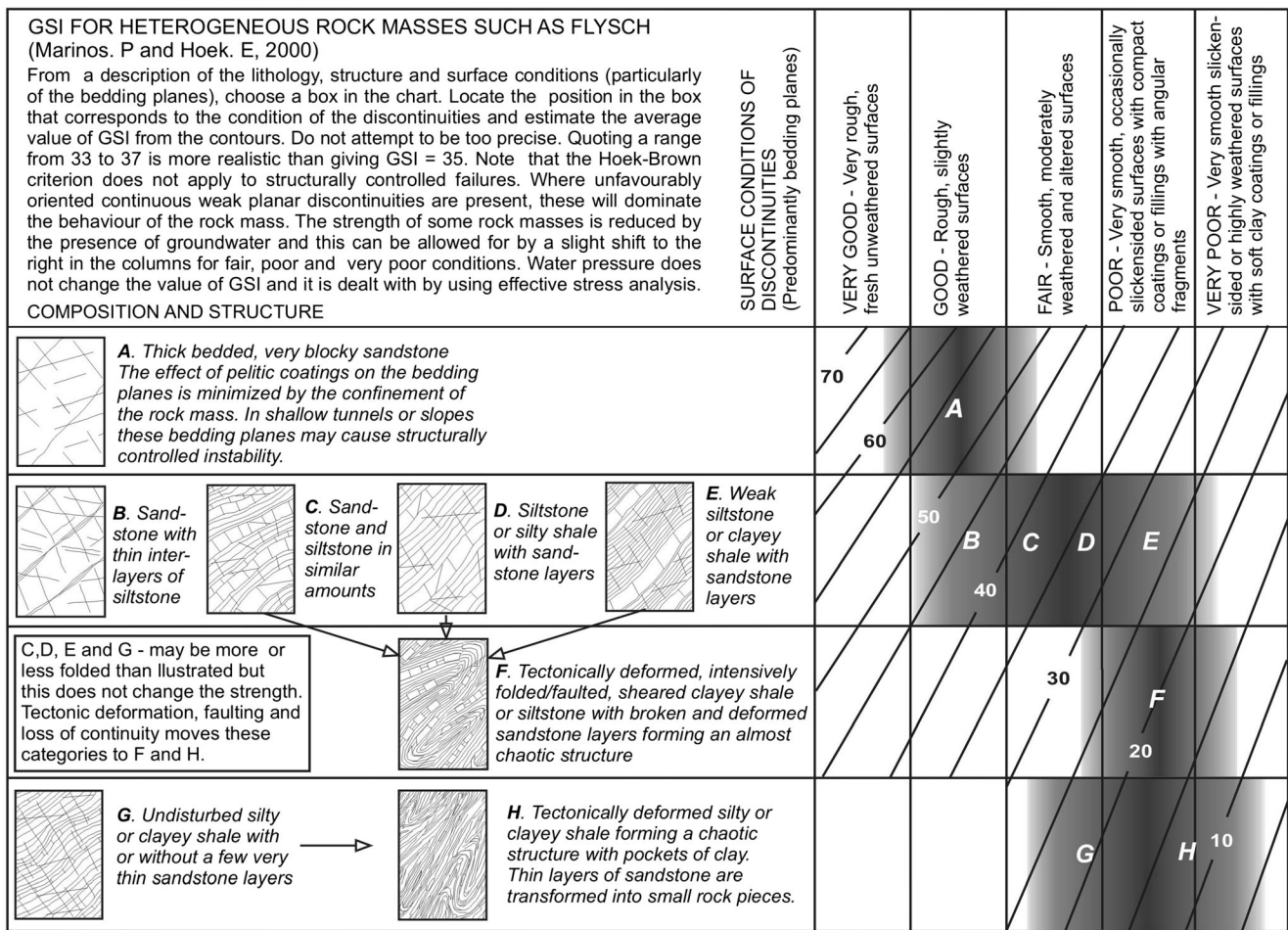r the wide range of geological conditions met in actual situations, including weak and complex ones. The experience gained from the design and construction of the 62 tunnels along the Egnatia Highway in Northern Greece in difficult and diverse geological conditions (Marinos et al. 2012) using the Hoek–Brown failure criterion and the GSI system led to one of the first geological extensions of the GSI geotechnical classification approach to weaker, geologically more complex and heterogeneous rock masses.

In the case of heterogeneous and often tectonically disturbed rock masses, such as flysch, a specific classification was introduced in 2001 by Marinos and Hoek (Figure 1). Due to the generally poor properties and ambiguities regarding its geotechnical characterization, flysch often imposes problems or difficulties on the design and construction of engineering projects. Flysch formations are generally characterized by diverse heterogeneity, the presence of members with low-strength geomaterial, and tectonically disturbed structures. Hence, the rock mass quality of flysch formations can be associated with undisturbed, fractured, heavily sheared or even chaotic structures. The principle of this classification is based upon the sandstone-siltstone (or shales) participation and the intensity of the tectonic disturbance to the overall rock mass.

After a decade of application of GSI for the classification of these heterogeneous rock masses, this paper attempts to re-evaluate or verify the original value and assist in the appropriate selection of the index for various conditions with the addition of new rock mass types. Hence, a revised GSI diagram for heterogeneous and structurally disturbed rock masses is presented, where a certain range of GSI values for every rock mass type is proposed.

To investigate and evaluate the rock mass properties of such rock masses, 12 tunnels constructed in a flysch environment, under various tectonic conditions, were examined. The variety of geological settings under different stresses regimes, in mild and heavy tectonism, provides a great amount of information regarding the engineering geological conditions and geotechnical behavior of several flysch rock mass types. More specifically, a fundamental tool for the re-evaluation of the GSI values for every rock mass type has been the assessment of the encountered deformations (Marinos et al. 2011; Marinos 2014). This behavior was analyzed and evaluated to define the engineering geological and geotechnical characteristics for every flysch rock mass type. As a result, flysch formations are classified here into 11 rock mass types (I to XI) according to the siltstone-sandstone participation and their tectonic disturbance.

Recent research regarding weak rock masses and their engineering geological behavior, as well as the experience gained by the recent tunneling projects in the Greek mountains under particularly difficult geological conditions, provided reliable and sufficient data for the study of the influence of these conditions on the behavior of the geological material, as well

GSI FOR HETEROGENEOUS ROCK MASSES SUCH AS FLYSCH
(Marinos. P and Hoek. E, 2000)

From a description of the lithology, structure and surface conditions (particularly of the bedding planes), choose a box in the chart. Locate the position in the box that corresponds to the condition of the discontinuities and estimate the average value of GSI from the contours. Do not attempt to be too precise. Quoting a range from 33 to 37 is more realistic than giving GSI = 35. Note that the Hoek-Brown criterion does not apply to structurally controlled failures. Where unfavourably oriented continuous weak planar discontinuities are present, these will dominate the behaviour of the rock mass. The strength of some rock masses is reduced by the presence of groundwater and this can be allowed for by a slight shift to the right in the columns for fair, poor and very poor conditions. Water pressure does not change the value of GSI and it is dealt with by using effective stress analysis.

COMPOSITION AND STRUCTURE

SURFACE CONDITIONS OF DISCONTINUITIES (Predominantly bedding planes)

VERY GOOD - Very rough, fresh unweathered surfaces

GOOD - Rough, slightly weathered surfaces

FAIR - Smooth, moderately weathered and altered surfaces

POOR - Very smooth, occasionally slickensided surfaces with compact coatings or fillings with angular fragments

VERY POOR - Very smooth slickensided or highly weathered surfaces with soft clay coatings or fillings

A. Thick bedded, very blocky sandstone
The effect of pelitic coatings on the bedding planes is minimized by the confinement of the rock mass. In shallow tunnels or slopes these bedding planes may cause structurally controlled instability.

B. Sandstone with thin interlayers of siltstone

C. Sandstone and siltstone in similar amounts

D. Siltstone or silty shale with sandstone layers

E. Weak siltstone or clayey shale with sandstone layers

C, D, E and G - may be more or less folded than Illustrated but this does not change the strength. Tectonic deformation, faulting and loss of continuity moves these categories to F and H.

F. Tectonically deformed, intensively folded/faulted, sheared clayey shale or siltstone with broken and deformed sandstone layers forming an almost chaotic structure

G. Undisturbed silty or clayey shale with or without a few very thin sandstone layers

H. Tectonically deformed silty or clayey shale forming a chaotic structure with pockets of clay. Thin layers of sandstone are transformed into small rock pieces.

⟶▷ : Means deformation after tectonic disturbance

Fig. 1 The original GSI chart with estimates for heterogeneous rock masses such as flysch (Marinos and Hoek 2001)

as on the design and construction methods. To make substantial use of the experience accumulated from the design and construction of these tunnels and to process this material, a database was constructed, i.e., "Tunnel Information Analysis System" (TIAS), which was designed and created for a great number of tunnels in the Greek territory along the Egnatia Highway (Marinos et al. 2012). The already constructed highway, 680 km long, composes a very important and modern infrastructure for the communication of Greece with Europe, the Balkans and East Asia and has a total of 76 road tunnels with an overall combined length of 99 km. It runs across the entire width of Greece, traversing a great variety of geological situations and geotectonic units, displaying different particularities in terms of occurrence of weak rock masses, tectonic structure and in situ stresses. The data input and processed by TIAS has to do with results from boreholes, laboratory or on-site testing, classifications, engineering geological behavior, groundwater inflows, design parameters and geotechnical analysis in addition to information concerning primary measures of immediate support. Important analyses of the database include the comparison of the anticipated and the actual

geological and geotechnical conditions, the evaluation of the applied classification methods of the rock masses, and the correlations regarding their behavior and the immediate support measures. All 62 tunnels were designed using the Hoek–Brown failure criterion and, subsequently, the GSI system, and were successfully constructed.

In addition to the GSI values, it is also necessary to consider the selection of the "intact" rock properties σ_{ci} and m_i for these heterogeneous rock masses. The basic inputs of the Hoek and Brown failure criterion are estimates or measurements of the UCS (σ_{ci}) and the material constant (m_i) related to the frictional properties of the rock and of the GSI. In addition, to estimate the deformation modulus of the rock mass (E_m), Hoek and Diederichs (2006) proposed an equation based on the values of the intact rock deformation modulus (E_i) or the modulus ratio (MR). Based on numerous lab test results from the investigation of 12 of these tunnels constructed in flysch, characteristic σ_{ci}, E_i and MR values are proposed in this paper for the flysch members. Finally, a 'weighted average' of the intact strength properties of the strong and weak layers that should be used is also suggested.

Engineering geological environment of a flysch formation

Flysch is composed of varying alternations of clastic sediments and is associated with orogenesis since it marks the end of a cycle of sedimentation of a basin before the paroxysm folding process. It is characterized mainly by rhythmic alternations of sandstone and pelitic (siltstones, silty or clayey shales). The thickness of the individual sandstone or siltstone beds ranges from centimeters to metres. Sandstones and pelitic beds are present in various analogies. As a result, different lithotypes of flysch are created. Typical lithotypes are the thick-bedded sandstones series, the sandstone-siltstone alternations and the siltstone series with thin layers of sandstones (Figure 2). Some conglomerate masses can also be present but not with the same frequency. Apart from these lithotypes due to sedimentation processes, flysch series that are often deformed by tectonism create additional rock mass types. Tectonism is expressed from light fracturing and/or folding to severe thrusting and shearing. There are cases when the main thrust movement is associated with satellite shears within the thrusted body. These shears are generally marked by weak siltstone beds, which act as a 'soap layer.' The flysch has thus suffered from large compression, and very weak rock masses have been produced. In these cases, the original structure is no longer recognizable, and blockiness is lost. Inside the sheared body mass, it is reasonable to expect less deformed sandstone blocks ($1.5–2$ m^3) with more rock-like behavior. However, this material is likely to be cut by small shears, and even at the scale of an engineering project, these boulders do not contribute significantly to the overall strength of the rock mass. In the cases where a flysch type, composed mainly by siltstones with even thin layers of sandstone, is further folded, the product is a very poor rock mass. Folding and faulting often entail a clearly visible chaotic structure of isolated lensed blocks of hard rock «floating» within a soft clayey-silty sheared matrix. A conceptual model, where flysch series have suffered from large compression, severe thrusting and shearing, has been drawn in Fig. 3.

Weathering alters the rock mass strength significantly. Weathering is generally confined close to the surface with the generation of laminated and disintegrated forms due to slaking. Siltstone (or marly) members are very vulnerable to weathering, and fissility may be developed parallel to the bedding when these rocks are exposed to the surface or are very close to it from wetting/drying processes.
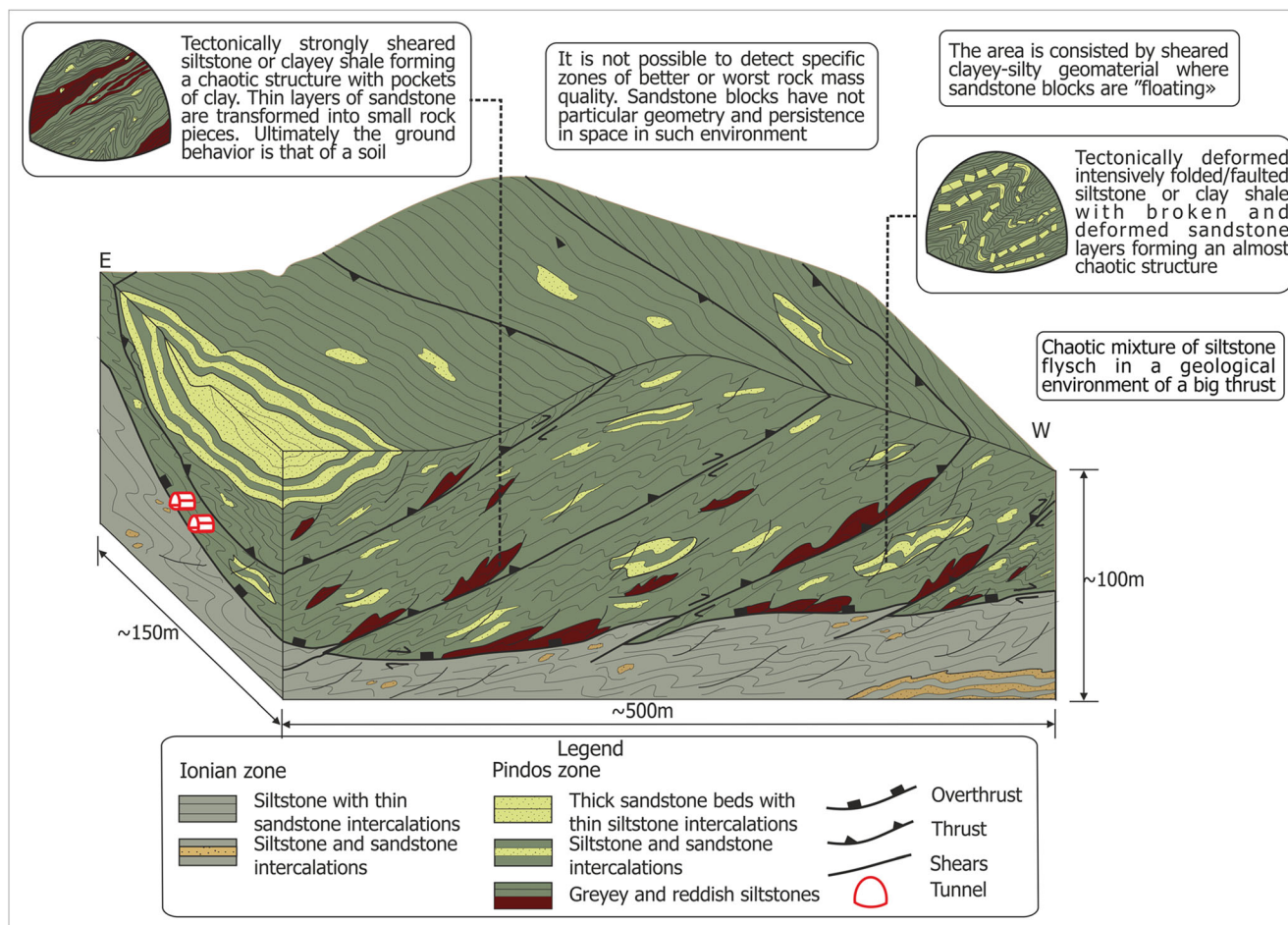
Hence, weak and complex rock masses in flysch have the following engineering geological characteristics:

I. *Complex rock masses*

- Alternations of competent-high-strength sandstone beds with incompetent-low-strength siltstone beds or clayey shales.
- Intense stratification, distinctly developed, with high bedding persistence.
- Diversity of bedding thickness (thin-bedded structure of a few cm to thick-bedded strata of a few m).
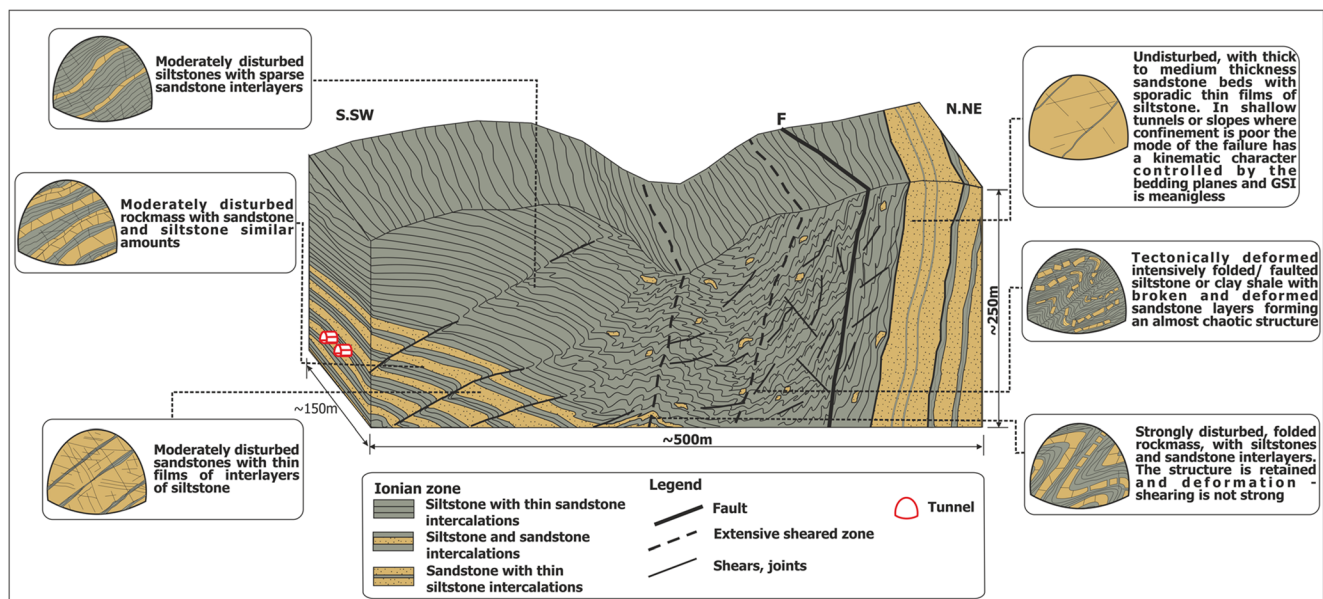
Fig. 2 Conceptual engineering geological model and related rock mass types in a typical flysch environment

The box labels in the figure read:

Tectonically strongly sheared siltstone or clayey shale forming a chaotic structure with pockets of clay. Thin layers of sandstone are transformed into small rock pieces. Ultimately the ground behavior is that of a soil

It is not possible to detect specific zones of better or worst rock mass quality. Sandstone blocks have not particular geometry and persistence in space in such environment

The area is consisted by sheared clayey-silty geomaterial where sandstone blocks are "floating»

Tectonically deformed intensively folded/faulted siltstone or clay shale with broken and deformed sandstone layers forming an almost chaotic structure

Chaotic mixture of siltstone flysch in a geological environment of a big thrust

Legend

Ionian zone
- Siltstone with thin sandstone intercalations
- Siltstone and sandstone intercalations

Pindos zone
- Thick sandstone beds with thin siltstone intercalations
- Siltstone and sandstone intercalations
- Greyey and reddish siltstones

- Overthrust
- Thrust
- Shears
- Tunnel

Fig. 3 Conceptual engineering geological model and related rock mass types, where flysch series have suffered from large compression, severe thrusting and shearing

- Tectonic disturbance that alters the initial structure and can create tectonic mixtures and chaotic structures.

II. *Weak rock masses*

- Presence of low-strength-clayey rocks.
- High tectonic fracturing and shearing degrades the original rock mass quality
- Weathering of silty-clayey members due to slaking that disintegrates and forms a fissile rock mass.
- Possible water presence reduces the rock strength of the silty-clayey members.

To evaluate the engineering geological characteristics and categorize the geotechnical characterization, the flysch rock masses were divided and classified into distinct rock mass types. These rock mass types are distinguished based on the sandstone or siltstone predominance and participation and the intensity of the tectonic disturbance and are illustrated in Fig. 4. Indicative photographs for rock mass types I through VI and VII through XI are presented in Figs. 5 and 6, respectively.

Revised GSI system for tectonically disturbed heterogeneous rock masses, such as flysch

Revision of the original 2001 chart: modifications and validation

The revised classification system, presented in this publication, is based on a GSI chart that was introduced in 2001 by Marinos and Hoek during the design phase and before the construction of a number of tunnels along the Egnatia Highway (Table 1). The design of these tunnels was based on the 2001 chart, and thus the monitoring experience from the construction of these tunnels provided valuable information. Hence, after a decade of application of the GSI for the classification of these heterogeneous rock masses, this paper attempts to re-evaluate or verify the original value and assist in the appropriate selection of the index for various conditions with the addition of new rock mass types. Hence, a revised GSI diagram for heterogeneous and structurally disturbed rock masses is presented, where a certain range of GSI values for every rock mass type is proposed. The revised GSI chart

ROCKMASS TYPE	STRUCTURE	ROCKMASS TYPE	STRUCTURE	ROCKMASS TYPE	STRUCTURE
TYPE I. Undisturbed, with thick to medium thickness sandstone beds with sporadic thin films of siltstone.		**TYPE II.** Undisturbed massive siltstone with sporadic thin interlayers of sandstones		**TYPE III.** Moderately disturbed sandstones with thin films of interlayers siltstone	
TYPE IV. Moderately disturbed rockmass with sandstone and siltstone similar amount		**TYPE V.** Moderately disturbed siltstones with sandstone interlayers		**TYPE VI.** Moderately disturbed siltstones with sparse sandstone interlayers	
TYPE VII. Strongly disturbed, folded rockmass that retains its structure, with sandstone and siltstone in similar extend		**TYPE VIII.** Strongly disturbed, folded rockmass, with siltstones and sandstone interlayers. The structure is retained and deformation - shearing is not strong		**TYPE IX.** Disintegrated rockmass	
TYPE X. Tectonically deformed intensively folded/ faulted siltstone or clay shale with broken and deformed sandstone layers forming an almost chaotic structure		**TYPE XI.** Tectonically strongly sheared siltstone or clayey shale forming a chaotic structure with pockets of clay.			

Fig. 4 Rock mass types of flysch, based on the sandstone-siltstone participation and predominance and the intensity of the tectonic disturbance
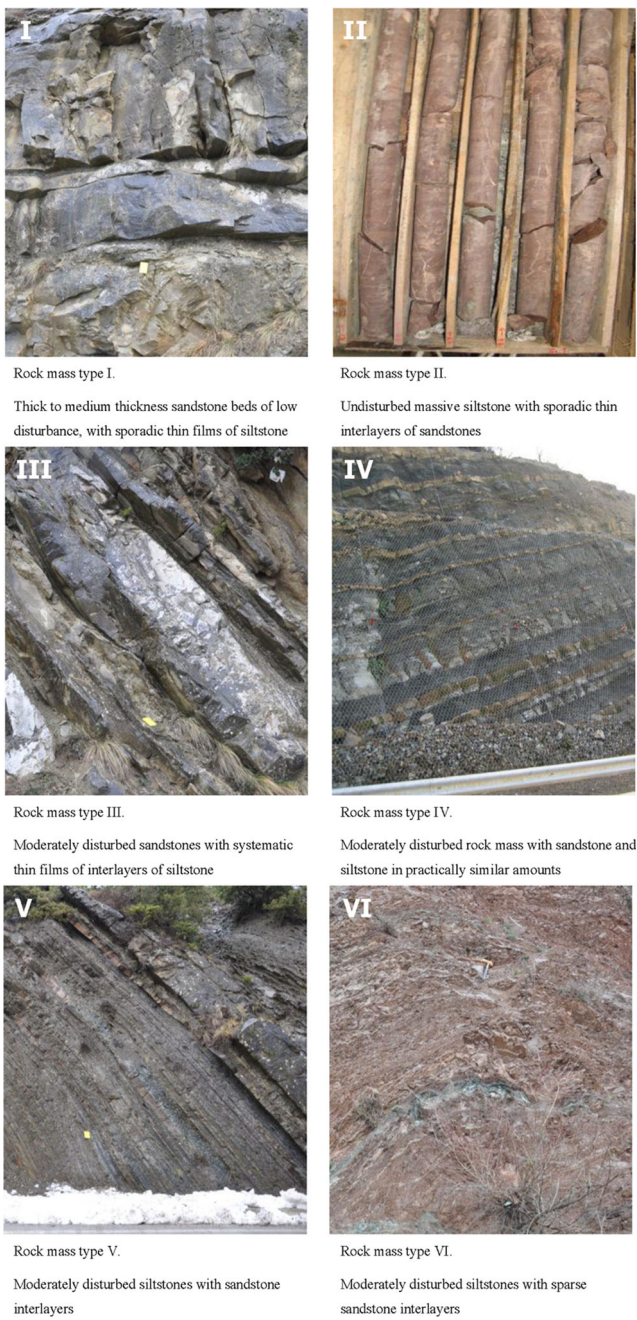
for heterogeneous rock masses such as flysch is presented in Fig. 7. This chart has several differences from the 2001 one without changing its conceptual principles (Figure 1).

In the new diagram, GSI values are increased from 10 to 35 units for the "Blocky" to "Undisturbed" structures, respectively, particularly for the siltstone type, and this an important modification. The high presence of siltstone rocks does not decrease the GSI value, but only in the highly disturbed forms. Only the intact strength is decreased. When a rock mass is undisturbed or slightly disturbed, independently of siltstone or sandstone predominance, much higher GSI ratings have to be considered. This was confirmed in tunnel construction, where lighter temporary support categories (correlated with high GSI values) were implemented and experienced marginally measurable deformations. For example, in the Kalamion tunnel, one excavation and support category consisting of 20-cm-thick shotcrete, lattice guiders, a 2X1.5 4-m bolt pattern and a 2-m excavation step was designed under 150 m of overburden in a rock mass with characteristics similar to rock mass type I and a designed GSI value of 45. However, for the same sections, a lighter support category, designed for a GSI value of 60, was implemented, comprising 5-cm-thick shotcrete, a 2X2 4-m bolt pattern and a 4-m excavation step. This value is in agreement with the proposed range within the revised GSI chart for rock mass type I. This section of the tunnel was constructed without deformation problems. Another example involves the experiences from tunneling in undisturbed siltstones with thin sandstone intercalations in the Anilio tunnel under a maximum overburden of 245 m. The quality and the

behavior of the rock mass, with characteristics similar to rock mass type II, were very satisfactory. During the design phase, a heavy excavation and support category was proposed for these sections. A support category, designed for a GSI value of approximately 20–25, included light forepoling, fiberglass nails in the tunnel face, 30-cm-thick shotcrete, lattice guiders with an elephant foot, a dense bolt pattern of 9 m in length and a 1-m excavation step. However, for the same sections, a much lighter support category, designed for a GSI value of 45, was applied, comprising 20-cm-thick shotcrete, 11 bolts of 6 m in length and a 2-m excavation step. This value is also in agreement with the proposed range within the revised GSI chart for rock mass type II. Again, this section of the tunnel was constructed without deformation problems. Other similar cases in other rock mass types were evaluated for this study, not only showing changes in the GSI values but also verifying the 2001 chart (Marinos and Hoek 2001).

Hence, the selection of the structure should be initially based on the tectonic disturbance (from undisturbed to sheared rock masses), then on the proportion of siltstones against sandstones, and finally on the expressed or non-bedding stratification inside the siltstone layers. From type IV and the following types, the stratification planes are perceptible inside the siltstone mass and thus separate in smaller sheetblocks of the rock mass.

According to the Hoek–Brown failure criterion, the GSI value is used to reduce the intact rock strength to the rock mass strength based on the structure and the condition of the discontinuity surfaces. The possible low strength of the

Rock mass type I.

Thick to medium thickness sandstone beds of low disturbance, with sporadic thin films of siltstone

Rock mass type II.

Undisturbed massive siltstone with sporadic thin interlayers of sandstones

Rock mass type III.

Moderately disturbed sandstones with systematic thin films of interlayers of siltstone

Rock mass type IV.

Moderately disturbed rock mass with sandstone and siltstone in practically similar amounts

Rock mass type V.

Moderately disturbed siltstones with sandstone interlayers

Rock mass type VI.

Moderately disturbed siltstones with sparse sandstone interlayers

Fig. 5 Examples of rock mass types (types I to VI) of flysch corresponding to descriptions in Fig. 4

Rock mass type VII.

Strongly disturbed, folded rock mass that retains its structure, with sandstone and siltstone in similar extend

Rock mass type VIII.

Strongly disturbed, folded rock mass, mainly from siltstones with sandstone interlayers. The structure is retained and shearing is still not intense

Rock mass type IX.

Disintegrated rock mass (good quality sampling with split barrel)

Rock mass type X.

Tectonically deformed intensively folded/ faulted siltstone or clay shale with broken and deformed sandstone layers forming an almost chaotic structure

Rock mass type XI.

Tectonically strongly sheared siltstone or clayey shale forming a chaotic structure with clayey zones formed from the shear process.

Fig. 6 Examples of rock mass types (types VII to XI) of flysch corresponding to descriptions in Fig. 4

siltstone or the clayey shale is considered in the failure criterion through their intact rock strength. Thus, the overall rock mass strength will be "penalized" only by this σ_{ci} value. As opposed to that, siltstone presence must be negatively considered in the GSI values in the case of highly tectonically disturbed rock masses, where shearing is intense. Otherwise, when the rock mass is medium disturbed to undisturbed, the presence of siltstone should not impose a double penalization on the rock mass strength, through the GSI value as well. Therefore, GSI values in these rock masses must be much

higher. However, the presence of siltstones may also be stated through the fair to poor quality of the discontinuities that reduce the GSI values. These values range from 25 for the highly disturbed-folded rock masses that retain though their structure to 65 for the undisturbed rock masses.

One more addition in the GSI chart that affects the rock mass strength is the consideration of the competent sandstone
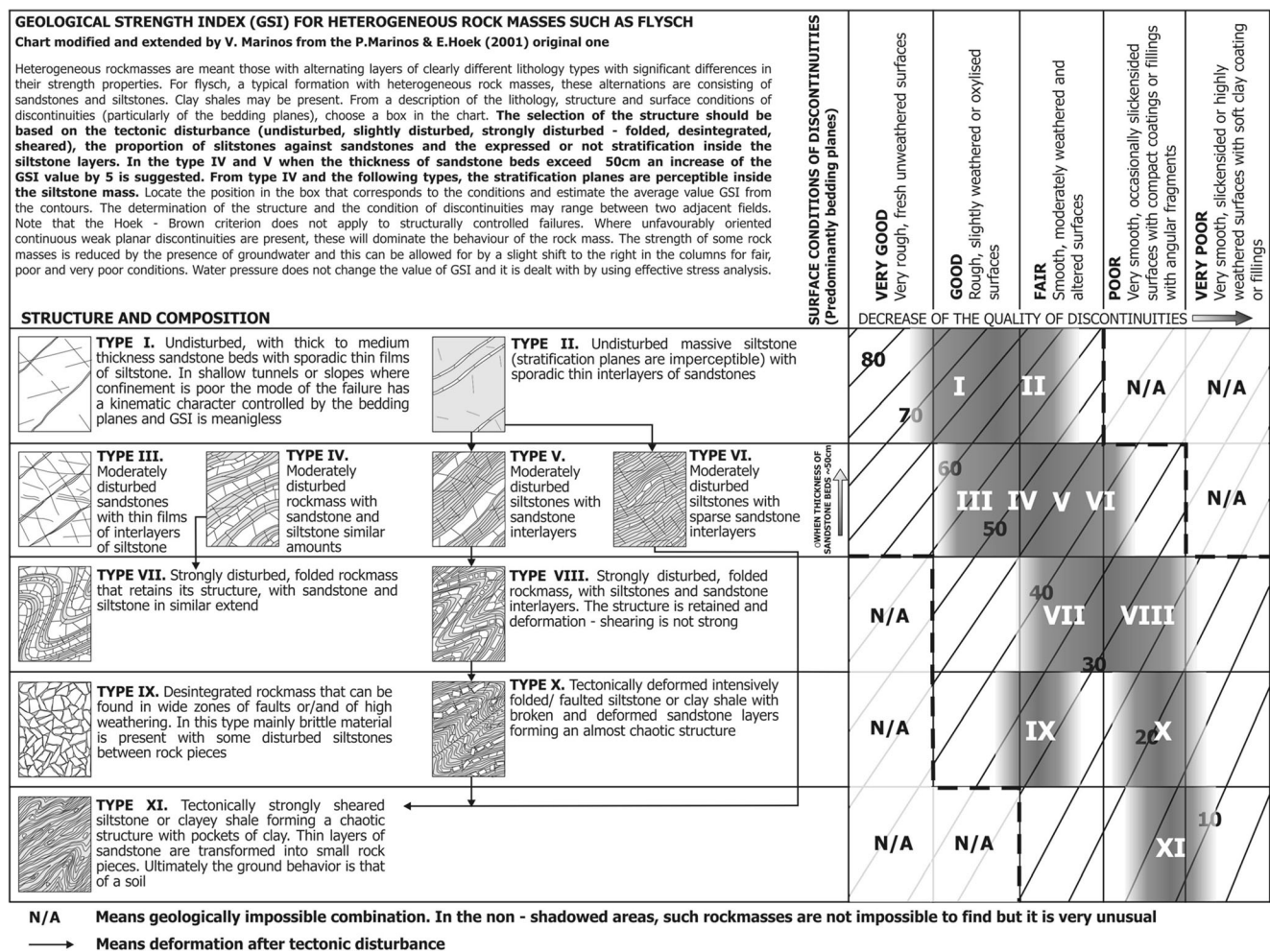
Fig. 7 A new, revised, geotechnical classification GSI system for tectonically disturbed heterogeneous rock masses, such as flysch, with the fields of most probable occurrence. The chart is modified and extended from the original one (Marinos and Hoek 2001)

bedding thickness. When the presence of sandstones is regular and is consistently alternating with siltstones, the thickness of the competent member plays a significant role in the overall

Table 1 List of investigated road tunnels in an environment of heterogeneous rock masses, such as flysch

Tunnel name	Location area	Tunnel length (m)	Maximum overburden (m)
Σ2	Ioannina	850	65
Driskos	Ioannina	4600	220
T6	Ioannina–Metsovo	300	80
T8	Ioannina–Metsovo	2600	200
Agios Nikolaos	Metsovo	1100	80
Anthochori	Metsovo (Peristeri)	770	90
Votonossi	Metsovo (Peristeri)	530	65
Dyo Koryfon	Metsovo (Peristeri)	750	150
Kalamion	Metsovo	850	160
Krimnou	Metsovo	1100	150
Anilio	Metsovo (Anilion)	2200	245
Metsovo	Metsovo	3600	600

stability of the rock mass regarding its strength. When the thickness of the sandstones beds is small (10–20 cm), both members contribute according to their presence (if 50%–50%, then equally) to the structure and the rock mass strength, especially when they are not highly disturbed but just jointed. Alternately, when the sandstone beds have higher thickness (~50 cm) and preserve their persistence, then the structure is more massive, and the siltstone beds are "missing" inside the rock mass. This difference, though, concerns the overall rock mass strength that is going to receive the loads and not a structurally controlled failure, where the presence of siltstone or clayey shale, regardless of the sandstone thickness, dictates its stability, creating plane or wedge sliding. As a result, in types IV and V (slightly disturbed structures) when the thickness of sandstone beds exceeds 50 cm, an increase of the GSI value by 5 is suggested.

It is noticed that, for the non-disturbed types, anisotropy is clearly present due to the bedding planes, and in analysis, this fact should be taken into consideration. This is the case specifically for types I through IV, while in the cases of V and VI,

the high presence of siltstone slightly reduces this impact. There has been some criticism that GSI is only applicable to heavily jointed rock masses, which at the scale of most engineering problems can be treated as isotropic. However, this limitation can be readily overcome with some judgment.

For example, if there are weak discontinuities throughout a rock mass, such as bedding, that would render the overall rock mass anisotropic, it would be recommended that the weak fabric be ignored in developing a global GSI value, and then the rock mass should be modeled or analyzed explicitly. In this regard, any such bedding can be readily examined as a global joint set within the overall Hoek–Brown material considered appropriate for characterization of the case. Such an example is presented in a paper by Fortsakis et al. (2012), where a reduced GSI was selected for the general rock mass ignoring the persistent discontinuity for which its own strength parameters were taken into account in the design process.

Classification per rock mass type

The main engineering geological characteristics and numerical classification for those rock mass types that are differentiated, more or less, from the original chart for flysch (Marinos and Hoek 2001) are presented in the following paragraphs.

Rock mass type I consists of undisturbed thick- to medium-bedded sandstones with thin sporadic films of siltstone. This rock mass cannot be very fractured in a wide scale but only in small zones. GSI values range from 60 to 75. Light support measures, such as thin shotcrete and bolts, were implemented in tunneling in this rock mass type without deformation phenomena. Such temporary support measures were designed, even under noticeable high tunnel cover (~150 m), with the Hoek–Brown failure criteria with GSI values of 60–75 as input. Under low-medium overburden in tunneling or in a slope cut, the rock mass may present structurally dependent instability along the persistent bedding, facilitated by the presence of siltstone films between the sandstone beds due to their low-friction properties. In these cases, the Hoek–Brown failure criterion and, hence, GSI must not be used in the design analysis, critical to select the appropriate support measures. Under higher stresses, failure of the brittle sandstone beds may be observed, and stress-dependent analysis has to be performed. In this case, GSI is a useful tool to evaluate the properties of the rock mass needed for the analysis.

The projection of rock mass type II consists of one of the fundamental modifications related to the original diagram. A difference of 20–35 units in the diagram results, according to how tectonically undisturbed and thin-layered the rock mass is. When tunnels were excavated in such an environment, much lighter support categories were applied under large tunnel cover (100–250 m). These applied support categories were analyzed and designed, before construction, for higher GSI values. These values correspond to this proposed range. This

is reasonable since the rock mass is generally undisturbed and massive, with no shear zones and infrequent discontinuities with low persistence. Furthermore, the rock mass quality should not be penalized twice by the presence of the siltstone members. If the incompetent member, here the siltstone, has low intact rock strength, the overall rock mass strength will be penalized by this σ_{ci} value through the Hoek–Brown transfer equations relating intact and rock mass properties with respect to the GSI (Hoek et al. 2002). Moreover, the presence of siltstones may also be stated through the fair to poor surface quality of the discontinuities. Thus, it should not be penalized through both the GSI and σ_{ci}. The proposed GSI values in the new diagram range from 45 to 60.

The rock mass type IV projection in the proposed GSI chart creates a modification of 10 units higher. This rock mass consists of sandstone and siltstone alternations in similar amounts with medium tectonic disturbance (fracturing and/or folding). Bedding planes are distinct with high persistence, while joints have generally small development due to the significant difference in deformability of the two geomaterials and the diffraction of the fractures from the sandstone inside the siltstones. According to the quality of the discontinuities, GSI values range between 45 and 55 due to the medium fracturing and folding of the rock mass. This value can become higher (by approximately 5 units) if the sandstone beds have greater thickness (> 40 cm). The rock mass may behave anisotropically according to the trend of the bedding planes in relation to the geometry of the project (tunnel or slope). A comment was already made on this issue.

Rock mass type VI presents significant changes in GSI values in relation to the 2001 diagram that rated this rock mass lower by approximately 20 units (type G in the original diagram). The experience from tunneling along this type showed that lighter support measures were implemented (designed for GSI values of approximately 35) with regard to the predicted ones. The rock mass is continuous and slightly folded, but highly stratified. Sandstone beds assist the overall good structure. GSI values range from 35 to 45 according to the discontinuity quality.

A weaker rock mass, type VIII, is of the same lithology as type V or even VI with increased tectonic disturbance and a folded structure. This rock mass is rated in the new GSI system from 20 to 35, depending on the joint surface condition. The structure is highly disturbed with the siltstone beds being sheared, but the beds are still parallel with the sandstone ones and retain their stratification planes. The rock mass here approaches the isotropic behavior due to its disturbance and folding. Bedding planes, while persistent, frequently change their geometry, and as a result, no considerable blocks can be formed. Siltstone beds preserve their original strength without significant shearing inside their mass. Alternately, joints do not persist due to the ductile behavior of the siltstones. Discontinuities are, anyway, very smooth to slickensided, while locally, they can be coated with clay.

Rock mass type IX is heavily fractured and brecciated due to tectonic disturbance. This situation in flysch formations is only able to occur in the sandstone persisting type, i.e., in brittle-type material. Such a type can sometimes be found along siltstones when they have a high intact strength (e.g., meta-siltstone), though this is not so typical. An example of this type is frequently found in a flysch-type series, the Athenian schist and was encountered along the Metropolitan Railway of Athens. The rock mass behaves isotropically, resulting from the intense fracturing. GSI values range from 25 to 40.

Rock mass type X is characterized by an almost chaotic structure, a result of intensive folding/faulting, and consists of siltstone or clay shale with broken and deformed sandstone layers. This type may also be formed without the presence of thin beds of sandstones. This rock mass is similar to type V or VI and then VIII, as far as the lithology is concerned, but even more tectonically disturbed. Siltstones are heavily folded and sheared, but the thin layers of sandstone can follow this disturbance, though severely fractured. Hence, these layers help to maintain a lesser poor quality and hold their parallel geometry with the siltstone ones, but they cannot form continuous beds. The rock mass is always heterogeneous because of the presence of both geomaterials but now behaves isotropically. The intact strength of siltstones has been significantly reduced, but this should not penalize the GSI value further. This is because the intact rock strength σ_{ci} is already considered in the Hoek–Brown failure criteria, and if it was also reflected in the GSI value, it would twice penalize the rock mass strength and the deformation modulus. The rock mass is rated from 18 to 25 according to the joint surface conditions. This projection in the new diagram is identical to the one in the original diagram (Marinos and Hoek 2001). These values were verified during the tunnel excavation, where respective support measures designed for such values were implemented without deformation problems.

The weakest rock mass type that can be met along flysch series is type XI. This rock mass is included in a classic geological environment derived from overthrusts of one geotectonic unit formation over another or from other satellite thrusts and along zones formed from great inverse faults. The produced rock mass is a heavily sheared clayey flysch (siltstones-clayey shales with spare sandstone layers). Rock discontinuities are randomly dispersed, very slickensided and often coated with clay. The corresponding rock masses can be classified as a tectonic mélange since they consist of chaotic, heterogeneous geological mixtures of blocks of different types and sizes. Inside the weaker, sheared-foliated finer-grained body mass, it is reasonable to expect less deformed sandstone blocks (e.g., 1–2 m^3). However, this material is likely to be cut by small shears, and at the scale of the project, these boulders do not contribute significantly to the overall strength of the rock mass. This particular geological formation, given its

chaotic structure, is considered as a «pseudo-isotropic mixture» (Marinos and Hoek 2000), and thus the rock mass behaves isotropically. GSI values for this rock mass type range from 15 to 20. In the case where strong sandstone blocks are numerous, continuous and with defined geometry, the rock mass properties are evaluated by a different approach. One such approach, the block in matrix approach (beamrocks), is effectively described by Wakabayashi and Medley (2004).

Intact rock properties of lithological members of flysch

While this paper is concerned primarily with the GSI classification, it would not be appropriate to leave the related topic of the Hoek–Brown failure criterion without briefly mentioning the estimation of intact strength σ_{ci} and the constant m_i. These parameters are used as an input in the transfer equations relating intact and rock mass properties with respect to GSI (Hoek et al. 2002).

The influence of the intact rock strength σ_{ci} is at least as important as the GSI value for the Hoek–Brown criterion. Ideally, σ_{ci} should be determined by direct laboratory testing under carefully controlled conditions. However, in many cases, this is not possible in practice because of time or budget constraints or because it is not possible to recover samples for laboratory testing (particularly in the case of weak, thinly schistose or tectonically disturbed rock masses where discontinuities are included in the laboratory samples). Under such circumstances, estimates of the value of σ_{ci} are often made on the basis of published information or simple index tests (such as point load tests if appropriate, published by the International Society of Rock Mechanics 1985).

Experience has shown that there is a tendency to underestimate the value of the intact rock strength in many cases. This is particularly true in weak and tectonically disturbed rock masses where the characteristics of the intact rock components tend to be masked by the structural tectonic elements or the surrounding sheared or weathered material. 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 the GSI) and will give unrealistically low values for the rock mass strength (Marinos and Hoek 2001). These underestimates can have serious implications for engineering design, and care has to be taken to ensure that realistic estimates of intact strength are made as early as possible in the project. In tunneling, such estimates can be refined on the basis of a detailed back-analysis of the tunnel deformation, and while this may require considerable effort, the attempt will generally be repaid many times over in the cost savings achieved by more realistic designs.

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 (Marinos and Hoek 2001). The UCS of the intact rock samples can be estimated, with a reasonable level of accuracy, by multiplying the point load index $Is_{(50)}$ by 13 (Tsiambaos and Sabatakakis 2004) for $Is_{(50)}$ values up to 2 MPa and by 24 for $Is_{(50)}$ values over 2 MPa, where $Is_{(50)}$ is determined according to ISRM (1985) suggestions. Point load tests should be carried out as soon after core recovery as possible to ensure that the moisture content of the sample is close to the in situ conditions. It should be highlighted here that siltstones and silty sandstones are susceptible to slaking, and thus the rock can be rapidly altered, schistosed or even disintegrated after its atmospheric exposure. In addition, point load tests may not be adequate to measure the strength since the pressure point may "invade" the weak rock (Marinos and Hoek 2001). Nevertheless, testing must be applied vertically in the direction of the rock fissility.

Where it is not possible to obtain samples for unconfined compression testing and point load testing or because of time or budget constraints, the only remaining alternative is to turn to a qualitative description of the rock material to estimate the UCS of the intact rock according to Hoek and Brown (1997) or to other published values of intact rock parameters, particularly in the case of weak, schistose or tectonically disturbed rock masses, such as flysch.

Based on numerous lab test results from the design of 12 tunnels of the Egnatia Highway in flysch environments, characteristic σ_{ci}, E_i and MR values are proposed here for the flysch members. The great number of lab tests of these rocks enabled this process, reducing the possibility of error or of confusing values (Marinos and Tsiampaos 2010).

The mean, minimum and maximum values of the UCS (UCS-σ_{ci}) and E_i of the intact rock materials for flysch formations are presented in Table 2. The typical range of a mean σ_{ci} value for sandstone is approximately 40–45 MPa, and for siltstone, it is 15–20 MPa. Nevertheless, when siltstones have a high presence of clayey minerals, the UCS ranges between 5 MPa and 10 MPa. In conglomerates, the UCS of the flysch formations is approximately 15–17 MPa due to the poorer cement. As far as E_i is concerned, a mean value of approximately 10–15 GPa is observed for the sandstones of flysch and 4–5 GPa for the siltstones. Do notice though the wide range of values, especially for E_i, of the flysch members in Table 2. These values should not be used uncritically since they present noticeable variability due to the member's deposition environment and tectonic evolution. It is always better to avoid their use for design purposes. Rather, site-specific testing should be utilized.

In the case of tectonically disturbed weak geomaterial, however, it is very difficult to perform tests. Regarding the values of the intact strength σ_{ci}, the chaotic structure and the sensitivity to the environmental factors that characterize the heavily sheared flysch demand high-quality laboratory testing

Table 2 Characteristic σ_{ci} and E_i values and their typical range and standard deviation (STD) for the individual lithological members of flysch. Data from Northern Greece from the Egnatia Highway

Rock material	Rock material properties	Number of tests	Min	Max	Mean	STD
Sandstone	UCS (MPa) (σ_{ci})	238	10.00	119.60	46.07	28.85
	E_i (MPa)	130	2019	56,000	13,354	12,892
Siltstone	UCS (MPa)	107	2.43	62.00	17.36	12.68
	E_i (MPa)	76	698	17,000	4628	4116
Conglomerate	UCS (MPa)	61	5.18	54.00	16.16	2.37
	E_i (MPa)	48	669	14,400	4588	3321

standards to acquire representative values, as a wide scatter of values can easily be present. Back-analysis has been performed by Marinos et al. (2006) to determine the parameters of heavily sheared flysch, utilizing the convergence measurements along two tunnels of the Egnatia Highway that have suffered major deformation by ground squeezing. The ranges of the geotechnical parameters that come up and best satisfy the field observations were 13–17 for the GSI, 4–5 MPa for the intact UCS (σ_{ci}) and 5–6 for the material constant m_i. These results are in agreement with another case study in similarly sheared flysch described by Tsatsanifos et al. (2000). Such parameters, such as the unconfined strength value (σ_{ci}), are very crucial for geotechnical design. For example, in tunneling in heavily sheared flysch (e.g., type X or XI), the σ_{ci} parameter has been found to be the most critical in the evaluation of the deformations of the tunnel (Marinos et al. 2006).

As far as the estimation of the rock mass deformation modulus E_{rm}, Hoek and Diederichs (2006) proposed Eq. (1).

$$E_{rm} = E_i \left[0.02 + \frac{1 - D/2}{1 + e^{\left((60 + 15D - GSI)/11 \right)}} \right] \quad (1)$$

where E_i is the intact rock deformation modulus, GSI is the geological strength index, and D is the disturbance factor depending on the excavation method (Hoek et al. 2002). In the cases where E_i is not known or cannot be measured, the MR after Deere (1968) is used for several rock types. The MR for the intact rocks is calculated from Eq. (2):

$$E_i = MR\sigma_{ci} \quad (2)$$

where σ_{ci} is the UCS of the intact rock.

Hoek and Diederichs (2006) note that the MR values are difficult to precisely identify since a slight disturbance in rock state and structure (especially for weak rocks) can be crucial to the σ_{ci} measurement. A certain range of MR values are suggested by Deere (1968) and Palmstrom and Singh (2001). The

Table 3 Characteristic modulus ratio (MR) values and their typical range for the individual lithological members of flysch from data of Northern Greece from the Egnatia Highway

Rock material	Number of tests	MR value	
		Range	Mean*
Sandstone	72	80–300	140
Siltstone	65	100–400	245
Conglomerate	45	110–400	230

suggested MR values from Deere (1968) and Palmstrom and Singh (2001) are 275 ± 75 for sandstone, 375 ± 25 for siltstone and 350 ± 50 for conglomerate. These values present notable differences from those found in this present work (Table 3) since the mean values are generally lower. Previous research on sedimentary rocks in Greece provided MR value ranges for sandstone specimens from 120 to 727, with a mean value of 303 (Sabatakakis et al. 2008). Moreover, Tziallas et al. (2009) proposed a typical range of MR values for sandstones between 200 and 400. From all these, it is nevertheless clear that the MR is an "unstable" factor since it is case-specific; it should then be used only when direct measurements of E are missing and even then with care.

However, the large range of MR (as given in Hoek and Diederichs 2006) when considering only rock type, such as sandstone (80 to 300), siltstone (100 to 400) and conglomerate (110 to 400), decreases the practical value of the use of MR for the prediction of E_i. Of course, some rough estimation may be obtained by using MR. Detailed discussion is available in Sonmez et al. 2006, including a proposed ANN-based prediction chart for E_i from UCSi and the unit weight of rock with considerable prediction correlation. This chart can be used together with MR for selection of the value for E_i.

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

In addition, it is necessary to consider values of the "intact" rock properties σ_{ci}, m_i and E_i for the heterogeneous rock mass considered as a unit. Some quantitative estimates of heterogeneous intact rock properties, as a whole, via laboratory tests (Mihalis et al. 2010) have already been presented. For cases in which laboratory tests are not feasible, which is the most common case, a "weighted average" of the intact strength properties of the strong and weak layers is proposed in Table 4. These suggested proportions for the components of this weighted average are based on Marinos and Hoek's (2001) assessment and on geological judgment based on the experience gained from the tunnel construction in flysch

Table 4 Suggested proportions of values of the properties of participating lithological members for each flysch rock type to be considered for the "intact rock" property determination (σ_{ci}, m_i and E_i; adjusted from Marinos and Hoek 2001 and reasonable judgment)

Flysch rock mass type	Proportions of values for each member of rock type to be considered for the weighted "intact rock" property determination
I, III	Use values for sandstone beds
II, VI, XI	Use values for siltstone or shale
IV	Thin beds: Reduce sandstone values by 10% and use full values for siltstone
	Thick beds: Use equivalent values for siltstone and sandstone beds
V, VII, VIII	Reduce sandstone values by 20% and use full values for siltstone
IX	Use equivalent values for siltstone and sandstone beds according to their participation
X	Reduce sandstone values by 40% and use full values for siltstone

Note: if adjusted values are found lower than the value of the weak layer, use that latter value

environments. It is strongly noted that this weighted value must be calculated in accordance with the percentage every composite contributes.

Impact of groundwater on the mechanical properties

When the water is not drained, it reduces the effective stresses and thus the shear strength along discontinuities and finally, in all cases, the strength of the rock mass. In addition, particularly important when dealing with shales, siltstones and similar rocks is that these rocks are susceptible to changes in moisture content, which directly affect their strength. The analysis of a good number of in situ permeability tests in the various lithotypes of flysch reveal the low permeability of its rock mass with very small differences among the types. The role of siltstone interlayers is predominant in all types, even in those types where their participation is very low. Additionally, the history of compression tectonics, from which the formation suffered, led to a homogenization of all types in terms of mean permeability values. This value is approximately 5×10^{-7} m/s for the first tens of metres below the surface (Marinos et al. 2011).

Conclusions

In concept, the original Hoek–Brown failure criterion was predicated on the assumption that rock mass strength was dominated by the interaction and interlocking of rock blocks controlled by the condition of the surfaces between these blocks. The GSI was formulated to attempt to characterize

such rock masses from a more descriptive geological perspective rather than a typical numerical parameter engineering approach to better meet the need for delivery of reliable geologically based input data in the associated m, s and a parameter relationships linking GSI with the Hoek–Brown failure criterion (Carter and Marinos 2014).

The use of the GSI system allows the influence of variables that define a rock mass to be assessed and, hence, the behavior of the rock mass to be explained more clearly. One of the key advantages of the index is that it allows characterization of difficult-to-describe rock masses such as flysch, and the geological reasoning it embodies allows adjustments to be made to its ratings to cover a wide range of rock masses and conditions, as well as allowing some understanding to be gained on applicability limits.

Many locations of tectonically disturbed heterogeneous rock masses such as flysch exist in the margins of major mountain chains worldwide. The opportunity created by the construction of the Egnatia Highway across Northern Greece involving a significant number of geotechnical investigations plus extensive experience gained from the excavation of 62 tunnels (Marinos 2007) provided remarkable insight and plenty of data concerning the engineering geological conditions of several rocks and formations and, especially in a flysch environment, enabled study of their distinction variability of rock mass types and their quantification. From the characterization of flysch rock mass types, it is clearly established how GSI respects the various geological particularities and how its numerical value is governed by the engineering geological-specific characteristic "keys" the geomaterial embodies.

Flysch formations create particular rock masses due to the varying petrographic heterogeneity, the participation of low-strength members and the tectonic disturbance until the complete distraction of the initial structure. The complexity of this geological formation needs a particular geological characterization due to the special features of its rock masses. Recent experience from the design and construction of a large number of tunnels in flysch environments, under various tectonic and particularly difficult geological conditions, provided sound and sufficient data for the study of the behavior of the geological material, as well as its properties appropriate for the design.

To investigate and evaluate the rock mass properties of flysch, 12 tunnels were considered. Flysch formations are classified into 11 rock mass types (I to XI) according to the siltstone-sandstone participation and their tectonic disturbance.

In the new, revised, GSI chart for heterogeneous and tectonically disturbed rock masses, new rock mass types are included, compared to the initial proposals for flysch rock masses by Marinos and Hoek in 2001, whereas increased GSI values are generally assigned to undisturbed rock masses, even if they are dominated by siltstone. GSI values are increased from 10 to 35 units for the blocky-medium disturbed to undisturbed structures, respectively, as it was confirmed from tunnel construction along those types.

In addition to the GSI values, the additional basic inputs of the Hoek–Brown failure criterion, the "intact" rock strength and deformability properties, were discussed. Although intact rock properties ideally should be determined by direct laboratory tests under carefully controlled conditions, in practice, this is not always possible because of time or budget constraints or because it is not feasible to recover samples for laboratory testing, particularly in the case of weak, schistose or tectonically disturbed rock masses, such as flysch. Approximate values of σ_{ci}, E_i and MR are proposed, while a "weighted average" of the intact strength properties for each rock type is suggested based on a large number of laboratory tests. However, care should be exercised when using these given values for rock design purposes since they present noticeable variability due to the member's deposition environment and tectonic evolution.

References

Carter T, Marinos V (2014) Use of GSI for Rock Engineering Design. Proceedings of the 1st International Conference on Applied Empirical Design Methods in Mining, Lima

Deere DU (1968) Chapter 1: geological considerations. In: Stagg KG, Zienkiewicz OC, editors. Rock mechanics in engineering practice. London: Wiley; 1968. p. 1–20

Fortsakis P, Nikas K, Marinos V, Marinos P (2012) Anisotropic behaviour of stratified rock masses in tunnelling. Engineering Geology, Volumes 141–142(19):74–83

Hoek E (1994) Strength of rock and rock masses. In: News J Int Soc Rock Mech, 2, 2, pp 4–16

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

Hoek E, Diederichs MS (2006) Empirical estimation of rock mass modulus. In: Int J Rock Mech Min Sci, 43, pp 203–215

Hoek E, Marinos P (2007) A brief History of the Development of the Hoek–Brown Failure Criterion. Soils and Rocks, São Paulo, 30(2): pp 85–92, May–August, 2007

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

Hoek E, Caranza-Torres CT, Corcum B (2002) Hoek–Brown failure criterion - 2002 edition. In: Bawden HRW, Curran J, Telsenicki M (eds) Proc. north American rock mechanics society (NARMS-TAC 2002). Mining Innovation and Technology, Toronto, pp 267–273

ISRM (1985) Suggested method for determining point load strength. In: Int J Rock Mech Min Sci and Geomech Abstr 22, pp 51–62

Marinos V (2007) Geotechnical classification and engineering geological behaviour of weak and complex rock masses in tunneling, Doctoral thesis, School of Civil Engineering, Geotechnical Engineering

Department, National Technical University of Athens (NTUA), Athens. (In greek)

Marinos V (2014) Tunnel behaviour and support associated with the weak rock masses of flysch. J Rock Mech Geotech Eng 6:227–239

Marinos P, Hoek E (2000) GSI: A geologically friendly tool for rock mass strength estimation. In: Proc. GeoEng2000 at the Int. Conf. on Geotechnical and Geological Engineering, Melbourne, Technomic publishers, Lancaster, Pennsylvania, pp 1422–1446

Marinos P, Hoek E (2001) Estimating the geotechnical properties of heterogeneous rock masses such as flysch. In: Bull Eng Geol Env, 60, pp 82–92

Marinos P, Tsiampaos G (2010) «Strength and deformability of specific sedimentary and ophiolithic rocks». Proccedings of the 12th International Congress of Geological Society of Greece, Patra, Greece, Vol 43, No3, pp. 1259–1266

Marinos V, Fortsakis P, Prountzopoulos G (2006) Estimation of rock mass properties of heavily sheared flysch using data from tunnelling construction. In: Proceedings of the 10th IAEG International Congress, Nottingham, United Kingdom

Marinos V, Fortsakis P, Prountzopoulos G (2011) Estimation of geotechnical properties and classification of geotechnical behaviour in tunnelling for flysch rock masses. Proceedings of the 15th European Conference on Soil Mechanics and Geotechnical Engineering, A. Anagnostopoulos et al. (Eds.), Part 1, pp. 435–440, Athens

Marinos V, Marinos P, Hoek E (2005) The geological strength index – applications and limitations». Bull Eng Geol Environ, 64(1):55–65

Marinos V, Prountzopoulos G, Fortsakis P, Korkaris K, Koumoutsakos D, Papouli D (2012) Tunnel information and analysis system: A geotechnical database for tunnels. In: J Geotech Geol Eng, special issue "Information Technology for Geotechnical Engineering" (Accepted, DOI: https://doi.org/10.1007/s10706-012-9570-x)

Mihalis A Parathyras G Prountzopoulos A Pa-pazachariou and Schwarz L (2010) "The Tunnels of Panagia-Anilio Section of the New Kalambaka-Ioannina-Igoumenitsa Railway. Evaluation of Geotech-nical Investigation". Proceedings of the 6th Hellenic Conference on Geotechnical and Geoenvironmental Engineering, Volos 29 September – 1 October 2010, 2, 343–350. (In Greek)

Palmstrom A, Singh R (2001) The deformation modulus of rock masses: comparisons between in situ tests and indirect estimates. In: Tunnelling and Underground Space Technology, 16, pp 115–131

Sabatakakis N, Koukis G, Tsiambaos G, Papanakli S (2008) Index properties and strength variation controlled by microstructure for sedimentary rocks. In: Engineering Geology, Elsevier, 97, pp 80–90

Sonmez H, Gokceoglu C, Nefeslioglu HA, Kayabasi A (2006) Estimation of rock modulus: for intact rocks with an artificial neural network and for rock masses with a new empirical equation. IJRMMS 43(2):224–235

Tsatsanifos CP, Mantziaras PM, Georgiou D (2000) Squeezing rock response to NATM tunneling: A case study. Geotechnical Aspects of Underground Construction in Soft Ground. Kusakabe, Fujita & Miyazaki (eds), Balkema, Rotterdam, pp 167–172

Tsiambaos G, Sabatakakis N (2004) Considerations on strength of intact sedimentary rocks. In: Engineering Geology. 72, pp 261–273

Tziallas GP, Tsiambaos G, Saroglou H (2009) Determination of rock strength and deformability of intact rocks. In: Electron J Geotech Eng, 14 G, pp 1–12

Wakabayashi J, Medley EW (2004) Geological characterization of mélanges for practitioners. Felsbau 22(5):10–43