Journal of Rock Mechanics and Geotechnical Engineering 6 (2014) 227–239

Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full length article

Tunnel behaviour and support associated with the weak rock masses of flysch

V. Marinos*

Laboratory of Engineering Geology and Hydrogeology, School of Geology, Faculty of Sciences, Aristotle University of Thessaloniki, Thessaloniki 54124, Greece

ARTICLE INFO

Article history:
Received 2 April 2014
Received in revised form
21 April 2014
Accepted 4 May 2014
Available online 14 May 2014

Keywords:
Flysch
Geological strength index (GSI)
Weak rock
Tunnel behaviour
Temporary support

ABSTRACT

Flysch formations are generally characterised by evident heterogeneity in the presence of low strength and tectonically disturbed structures. The complexity of these geological materials demands a more specialized geoengineering characterisation. In this regard, the paper tries to discuss the standardization of the engineering geological characteristics, the assessment of the behaviour in underground excavations, and the instructions–guidelines for the primary support measures for flysch layer qualitatively. In order to investigate the properties of flysch rock mass, 12 tunnels of Egnatia Highway, constructed in Northern Greece, were examined considering the data obtained from the design and construction records. Flysch formations are classified thereafter in 11 rock mass types (I–XI), according to the siltstone–sandstone proportion and their tectonic disturbance. A special geological strength index (GSI) chart for heterogeneous rock masses is used and a range of geotechnical parameters for every flysch type is presented. Standardization tunnel behaviour for every rock mass type of flysch is also presented, based on its site-specific geotechnical characteristics such as structure, intact rock strength, persistence and complexity of discontinuities. Flysch, depending on its types, can be stable even under noticeable overburden depth, and exhibit wedge sliding and wider chimney type failures or cause serious deformation even under thin cover. Squeezing can be observed under high overburden depth. The magnitude of squeezing and tunnel support requirements are also discussed for various flysch rock mass types under different overburdens. Detailed principles and guidelines for selecting immediate support measures are proposed based on the principal tunnel behaviour mode and the experiences obtained from these 12 tunnels. Finally, the cost for tunnel support from these experiences is also presented.

1. Introduction

Since the last decades of the 20th century, there has been a rapid development in various stages of geotechnical design, analysis and computational methods. Yet, regardless of the capabilities offered by the numerical tools, the results can still involve uncertainties when parameters are used directly without considering the actual failure mechanism of the rock mass in tunnelling. Understanding the rock mass behaviours in tunnelling can ensure selecting appropriate design parameters (for rock mass and/or discontinuities) and failure criteria to be used in numerical analysis and consideration of the principles in association with tunnel support.

Engineers can design reinforced concrete or steel structures using certain checks for specifically predefined failure mechanism. Specifically, design should consider bending moment, axial force, shear, penetration and deflection (serviceability limit state). In tunnelling, however, there is no specific procedure to check against a predefined failure mechanism. This paper points out that the first step is not to start performing numerous calculations (probably misleading or useless), but to define what the potential failure mechanisms are and to qualitatively consider the support theories to account for them. This process is thus applied for the heterogeneous rock masses of flysch (Fortsakis, 2014).

Rock mass behaviour evaluation in tunnelling and its relation with the design process have been significantly reported. Goricki et al. (2004), Schubert (2004), Potsch et al. (2004) and Poschl and Kleberger (2004) have studied rock mass behaviours with respect to design and construction experiences of Alpine tunnels and Palmstrom and Stille (2007) from other tunnels. Flysch rock is

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* Tel.: +30 2310998518.
E-mail addresses: marinosv@geo.auth.gr, vmarinos14@gmail.com.

Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

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http://dx.doi.org/10.1016/j.jrmge.2014.04.003

composed of varying alternations of clastic sediments associated with orogenesis, since it ends the cycle of sedimentation before the paroxysm folding process. Intense folding and heavy shearing with numerous overthrusts thus characterise the environment in areas of flysch formations. It is characterised mainly by rhythmic alternations of sandstone and pelitic layers (siltstones, silty or clayey shales), where the thickness of sandstone or siltstone beds ranges from centimetres to metres. Consequently, conglomerate beds may also be included. The main thrust movement is associated with smaller reverse faults within the thrust body. The overall rock mass is highly heterogeneous and anisotropic, and thus may be affected by extensional faulting producing mylonites. The tectonic deformation drastically degrades the quality of the rock mass, a reason that flysch is characterised by diverse heterogeneity (Fig. 1) and the presence of low strength and tectonically disturbed structures (Fig. 2). Such formations are classified into 11 rock mass types (I–XI) according to the siltstone–sandstone proportion and their tectonic disturbance.

The design of tunnels in weak rock masses such as disturbed and sheared flysch presents a major challenge to geologists and engineers. The complex structure of these materials, resultant from their depositional and tectonic history, means that they cannot easily be classified in terms of the commonly used characterisation schemes.

The variety of geological conditions under different in situ stresses, in both mild and heavy tectonism examined here, provided significant amount of information regarding the engineering geological conditions and geotechnical behaviour of several flysch rock mass types. These behaviours were analysed and evaluated so as to define the geotechnical characteristics for each flysch type.

This study is based on experiences obtained from the design and construction of 62 mountainous twin tunnels of the Egnatia Highway in Northern Greece. The cross-section of these tunnels is 100–120 m^2, constructed conventionally using the top heading and bench method. In this context, a database named "Tunnel Information and Analysis System" (TIAS) was created (Marinos, 2007; Marinos et al., 2013). Using this database, the evaluation of huge geological and geotechnical data from the design and the construction of 12 tunnels is presented. These cases comprise tunnelling up to 500 m of overburden depth.

The data processed by TIAS are obtained from geological mapping (design and face mapping records), boreholes, laboratory tests, site testing, geotechnical classifications (design and construction records) and designation of design parameters. Data were also collected and processed in view of the geotechnical behaviour, such as deformations, overbreak, structural failures and groundwater inflow. Data from detailed information on temporary support measures and tunnel construction cost were also included. The processing and evaluation of this information contributed to assessing the correlations between behaviours of the ground and the formulation and the temporary support requirements. The use of TIAS database enabled then the determination of the possible rock mass types of flysch and the engineering geological characterisation in terms of properties and their behaviour in underground construction (Marinos et al., 2013).

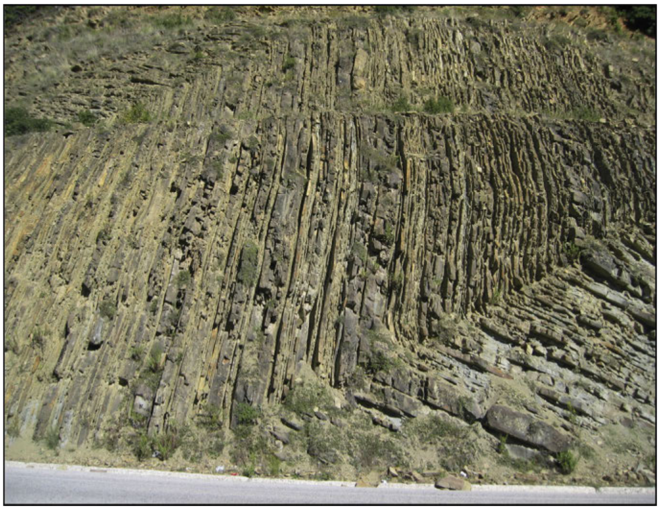


Fig. 1. Moderately disturbed rock mass with sandstone and siltstone alternations in similar amounts.

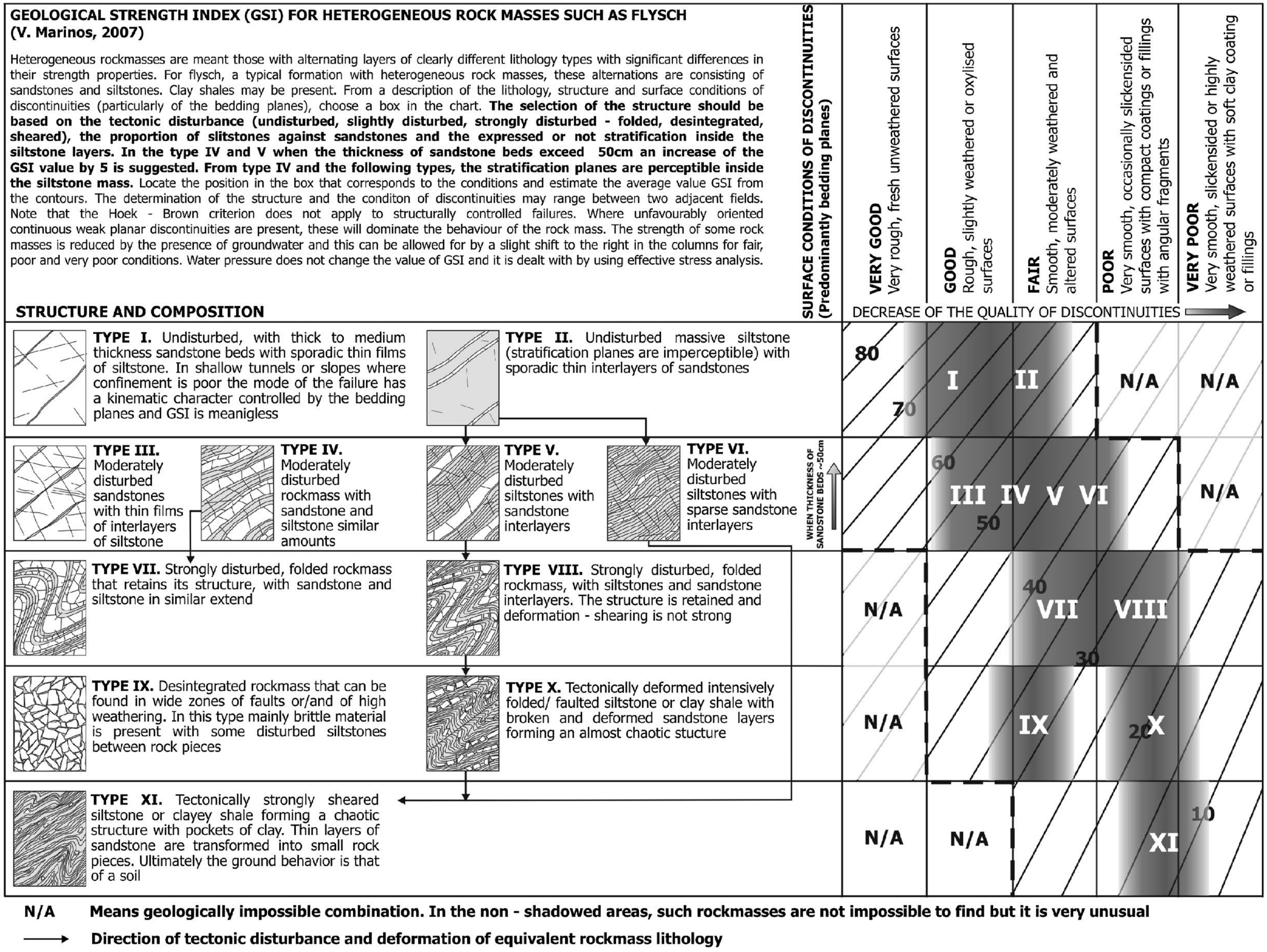
Fig. 2. Tectonically disturbed sheared siltstone with broken deformed sandstone layers. These layers have almost lost their initial structure, almost a chaotic structure.

2. Geotechnical properties

The development of powerful microcomputers and of user-friendly software prompted a demand on data related to rock mass properties required as inputs for numerical analysis or close-form solutions for designing tunnels. This necessity preceded the development of a different set of rock mass classifications, where the geological strength index (GSI) is such a classification. The Hoek–Brown failure criterion (Hoek et al., 2002) is closely connected to the GSI, covering a wide range of geological conditions affecting the quality of the rock masses, including heavily sheared weak rock masses (Hoek et al., 1998). The GSI considered as such a tool for assessment was initially introduced by Hoek (1994) and developed by Marinos and Hoek (2000). Marinos et al. (2005) further discussed its applications and limitations.

The GSI system was extended to heterogeneous rock masses, such as flysch, by Marinos and Hoek (2001), and then modified by Marinos (2007), and Marinos et al. (2007, 2011a) with adjustments in values and additions of new rock mass types. Flysch formations are thus classified into 11 rock mass types (I–XI) according to the siltstone–sandstone proportion and their tectonic disturbance. Hence, a new GSI diagram for heterogeneous rock masses such as flysch has been presented, where a certain range of GSI values for every rock mass type is proposed (Fig. 3). It is highlighted again that the Hoek–Brown failure criterion and consequently the GSI value should be used when the rock mass behaves isotropically.

The case in the presence of better quality blocks along with the sheared mass may improve the "overall" rock mass strength,



GEOLOGICAL STRENGTH INDEX (GSI) FOR HETEROGENEOUS ROCK MASSES SUCH AS FLYSCH (V. Marinos, 2007)

Heterogeneous rockmasses are meant those with alternating layers of clearly different lithology types with significant differences in their strength properties. For flysch, a typical formation with heterogeneous rock masses, these alternations are consisting of sandstones and siltstones. Clay shales may be present. From a description of the lithology, structure and surface conditions of discontinuities (particularly of the bedding planes), choose a box in the chart. **The selection of the structure should be based on the tectonic disturbance (undisturbed, slightly disturbed, strongly disturbed - folded, desintegrated, sheared), the proportion of siltstones against sandstones and the expressed or not stratification inside the siltstone layers. In the type IV and V when the thickness of sandstone beds exceed 50cm an increase of the GSI value by 5 is suggested. From type IV and the following types, the stratification planes are perceptible inside the siltstone mass.** Locate the position in the box that corresponds to the conditions and estimate the average value GSI from the contours. The determination of the structure and the conditon of discontinuities may range between two adjacent fields. 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.

TYPE I. Undisturbed, with thick to medium thickness sandstone beds with sporadic thin films of siltstone. In shallow tunnels or slopes where confinement is poor the mode of the failure has a kinematic character controlled by the bedding planes and GSI is meanigless

TYPE II. Undisturbed massive siltstone (stratification planes are imperceptible) with sporadic thin interlayers of sandstones

TYPE III. Moderately disturbed sandstones with thin films of interlayers of siltstone

TYPE IV. Moderately disturbed rockmass with sandstone and siltstone similar amounts

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. Desintegrated rockmass that can be found in wide zones of faults or/and of high weathering. In this type mainly brittle material is present with some disturbed siltstones between rock pieces

TYPE X. Tectonically deformed intensively folded/ faulted siltstone or clay shale with broken and deformed sandstone layers forming an almost chaotic stucture

TYPE XI. 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

N/A Means geologically impossible combination. In the non - shadowed areas, such rockmasses are not impossible to find but it is very unusual

⟶ Direction of tectonic disturbance and deformation of equivalent rockmass lithology

Fig. 3. The new GSI classification chart for heterogeneous rock masses such as flysch (Marinos, 2007; Marinos et al., 2007).

depending on their location and size. In the case where strong sandstone blocks are numerous and continuous and are with defined geometry, the rock mass properties can be evaluated by different approaches. Such an approach, the block in matrix approach (beamrocks), has effectively described by Wakabayashi and Medley (2004).

Basic inputs of the Hoek–Brown failure criterion, apart from the GSI value, are the uniaxial compressive strength (σ_{ci}) and the material constant (m_i) that is related to the frictional properties of the intact rock. Furthermore, in order to calculate the rock mass deformation modulus E_{rm}, Hoek and Diederichs (2006) proposed a new equation, which includes the intact rock deformation modulus

E_i, the GSI value and a disturbance factor due to the excavation method or a distressed character of rock mass D. Values of characteristic geotechnical parameters likely to prevail, for every flysch rock mass type (I–XI), are presented in Table 1. These values are resultant from the Roclab application (Rocscience Inc.). They are only indicative, since they cannot replace the detailed examination and the application of engineering judgement needed for each site-specific project separately.

The higher σ_{ci} values are presented in sandstone flysch with a mean value of 45–50 MPa. In siltstone flysch, a mean σ_{ci} value of approximately 15–20 MPa is promised. When the E_i is considered, a mean value of around 13 GPa is measured for sandstone flysch and 45 GPa for siltstone flysch (Marinos and Tsiampaos, 2010). Estimation of the mechanical parameters of a sheared siltstone or shale is a difficult task since the strength of the intact parts can hardly be measured in the laboratory (Figs. 4 and 5). Representative strength values can, however, be assessed by back analysis (Tsatsanifos et al., 2000; Marinos et al., 2006b).

In addition, it is necessary to take into account the parameters of the "intact" rock properties σ_{ci}, m_i and E_i, and considerer the heterogeneous rock mass as a unit. Some quantitative estimates of heterogeneous intact rock properties via laboratory tests (Mihalis et al., 2010) have already been reported. In cases when laboratory tests are not feasible, a "specific weighted average" of the intact strength properties of the strong and weak layers was proposed by Marinos et al. (2011a).

The influence of groundwater upon the mechanical properties of the intact rock components, more particular on shales and siltstones that are susceptible to changes in moisture content in tunnelling is very important and has to be considered in the estimation of potential tunnelling problems.

Flysch, a typical impermeable formation, has the character of presenting alternations of strong brittleness with weak rocks. The latter strongly influences the development tendency of permeability due to the fracturing in the strong beds. Data collected in Northern Greece from 213 packer tests from 108 boreholes during site investigation for 8 tunnels in flysch environment showed the permeability values of about 4.5×10^{-7} m/s (Marinos et al., 2011b). The difference of different flysch types is very small, which can be explained with respect to the tectonic history of the flysch formation where a "homogenization" has achieved from the compression and folding process. The low values in the sandstone type are imposed by the barriers of the thin interlayers of siltstones, which may also intrude in major fractures of the sandstone beds. The decrease in relation to depth is progressive but with significant scatter (Marinos et al., 2011b). As a result of the low permeability, the water is not easily drained and it reduces the effective stresses and thus the shear strength of the rock mass. Many of these materials will disintegrate very quickly if they are allowed to dry out and not supported immediately.

Table 1
Characteristic geotechnical parameters for each flysch rock mass type (I–XI). These values are indicative and have resulted from the Roclab application (Rocscience Inc.). Yet, they cannot replace the detailed examination and the application of engineering judgement adjusted for each particular project distinctly. The deformation modulus E_m is calculated here based on the empirical relation of Hoek and Diederichs (2006).

Flysch type	GSI	σ_{ci} (MPa)	m_i	E_i (GPa)	σ_{cm} (MPa)	E_m (GPa)
I	65	40	17	10	12	7
II	60	15	7	3	3	1.5
III	55	40	17	9	10	3.5
IV	50	23	10	5.5	4	1.5
V	45	18	8	4	2.5	0.9
VI	40	15	7	3	1.7	0.5
VII	35	23	10	5.5	2.5	0.6
VIII	25	18	8	4	1.5	0.25
IX	30	22	9.5	5.2	2	0.4
X	20	15	7	3.3	1	0.15
XI	15	<10	6	2	0.5	0.08

Fig. 4. Tectonically strongly sheared red siltstone forming a chaotic structure with pockets of clay (rock mass type X).

3. Engineering geological behaviour during tunnelling

A further classification of flysch rock masses based on their geotechnical behaviour (deformation due to overstressing, overbreaks or wedge failure, "chimney" type failure, ravelling and their corresponding scale) is presented hereafter. Flysch, depending on its type, can present a variety of behaviours: being stable even under a noticeable overburden depth, exhibiting wedge sliding and wider chimney type failures, or showing serious deformation even under low to medium overburden. Its behaviour is basically controlled by its main geotechnical characteristics, considering of course the in situ stress and groundwater conditions. The study of the varying behaviours of various flysch types was based on the large set of data from the TIAS database.

After the identification of the failure mechanism, the suitable design parameters can be selected according to the principles of the

Fig. 5. Tectonically strongly sheared siltstone: a chaotic structure with pockets of clay from a great thrust of different geotectonic units (Anthochori tunnel–Egnatia highway, Northern Greece).

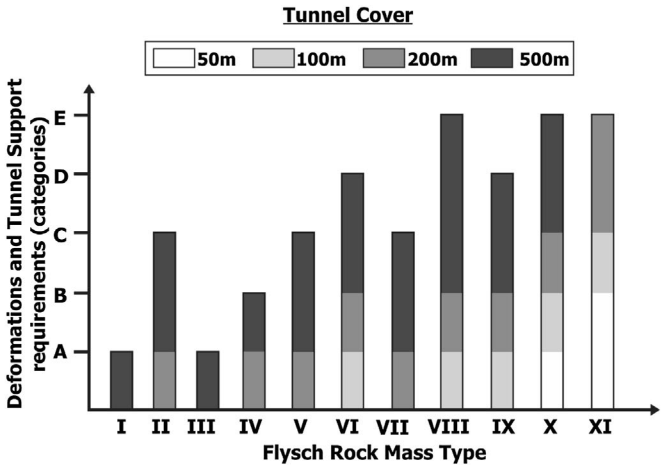


Fig. 6. Deformations and tunnel support requirements for each flysch rock mass type (I–XI) under different overburdens. Strain categories A–E are determined according to Hoek and Marinos (2000) (see Fig. 7.).

failure mechanism. If the behaviour of the rock mass can be considered as isotropic and is governed by stress-induced failures, the user must focus on rock mass parameters. On the other hand, if the principal behaviour type is gravity-controlled failures (e.g. wedge sliding, chimney failures, ravelling ground), the user must focus on parameters related to discontinuities. If the rock mass is weak but also anisotropic (e.g. due to schistosity or well defined bedding planes), both the rock mass parameters and the persisting joint properties must be considered.

A reliable first estimate of potential problems of tunnel strain can be given by the ratio of the uniaxial compressive strength σ_{cm} of the rock mass to the in situ stress p_o (Hoek and Marinos, 2000). This is usually followed by a detailed numerical analysis of the tunnel's response to sequential excavation and support stages. The strain estimation for the weak flysch rock mass type X of 4 different tunnel covers is shown in Fig. 6. It is evident that minor squeezing (category B) can be developed in the very poor flysch rock mass types X and XI from 50 m to 100 m tunnel cover, while severe to very severe squeezing (categories C and D) from 100 m to 200 m cover. Undisturbed rock mass types of sandstone or conglomerate (types I and III) do not exhibit significant deformations under 500 m.

Fig. 8. Overstressed steel sets due to squeezing. Long cables have been implemented to secure stability (Driskos tunnel in Northern Greece).

More analytically, the strain estimation for one of the weakest flysch type for 4 different tunnel covers is shown in Fig. 7 (strain categories A–E according to Marinos and Hoek (2001)). An overstressed support shell due to squeezing is presented in Figs. 8 and 9.

The presence of better quality blocks along the sheared mass may improve the stability of the surrounding rocks, depending on their location and size. A tunnel driven through this geomaterial requires continuous geological and geotechnical characterisation, as well as state of the art monitoring, to comprehend the complex interaction of internal block/matrix structure and their impact on the excavation and can only be conducted during tunnel construction. Such an effort was described in Button et al. (2004).

As far as the rheological characteristics of flysch formations are concerned, the creep potential of the sandstone formations is considered to be negligible. On the other hand, in the case of tunnel excavation in siltstone or shale formations, especially under high overburden, a time-dependent displacement or loads should be developed.

A detailed presentation of the range of geotechnical behaviour in tunnelling for each flysch rock mass type (I–IX) based on engineering geological characteristics is presented in Fig. 10. Generally,

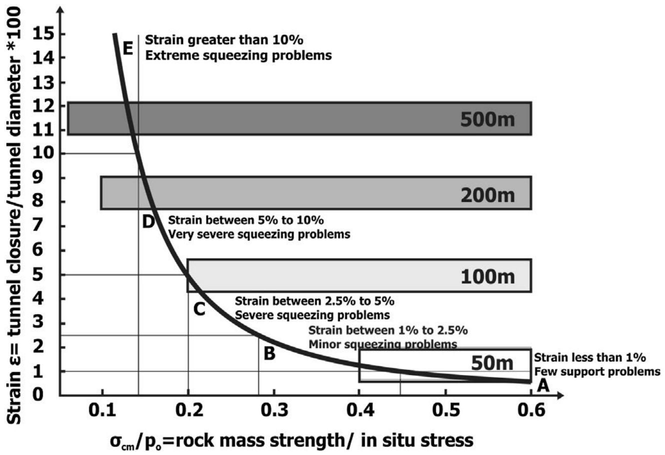
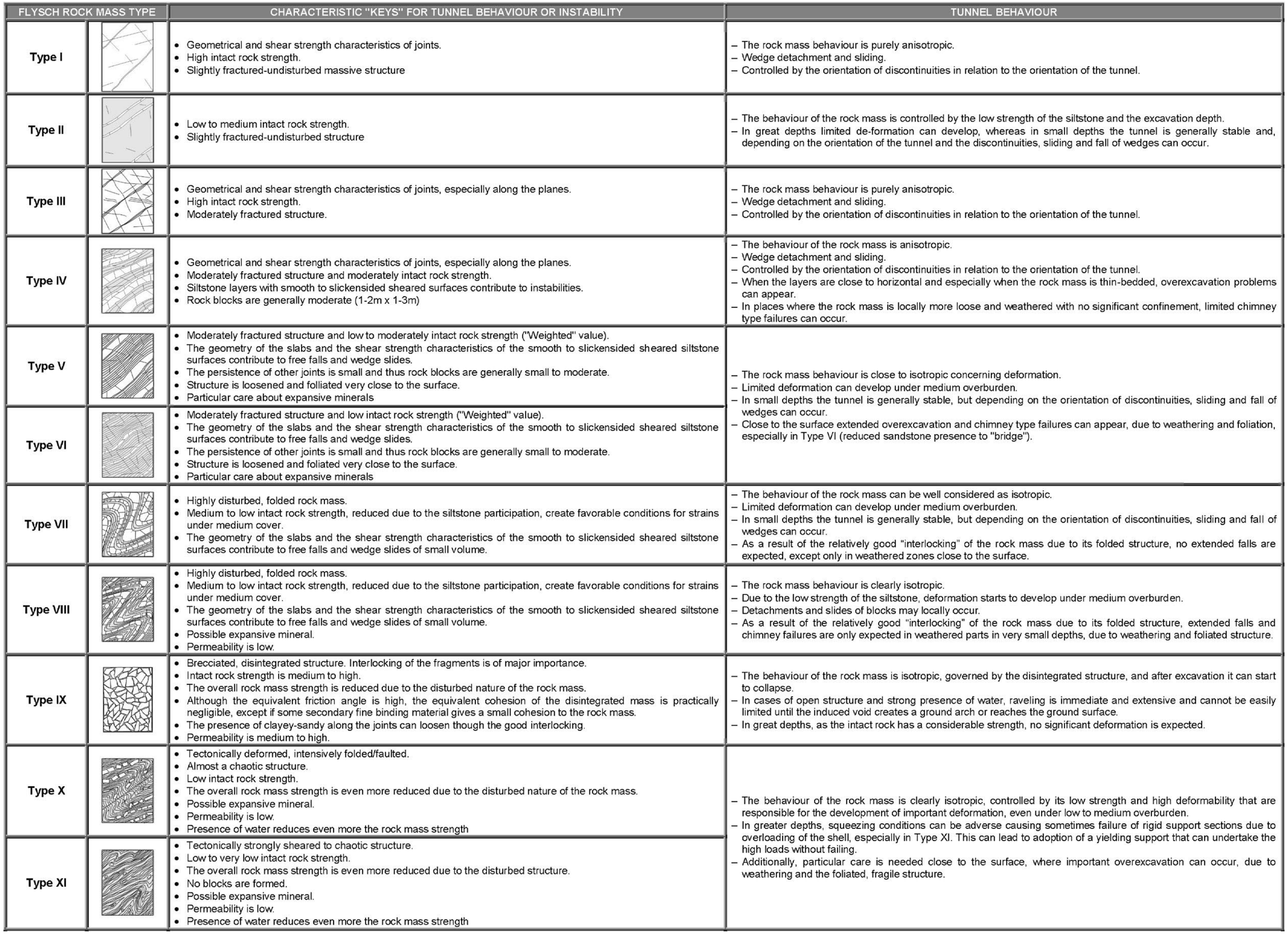


Fig. 7. Strain estimation of the flysch rock mass type X for 4 different tunnel covers categories A–E according to Hoek and Marinos (2000).

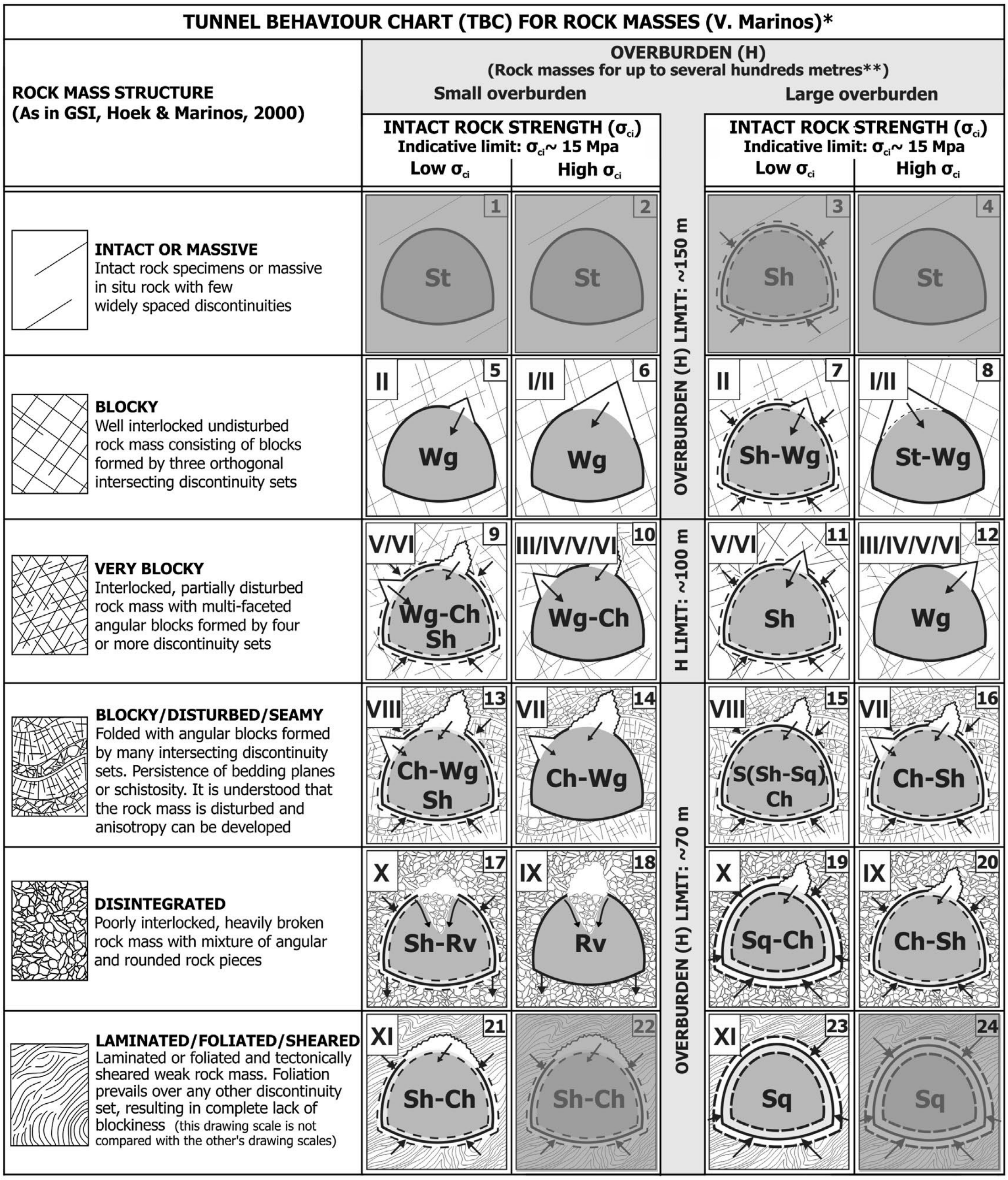
Fig. 9. Overstressed support shell due to squeezing (Anthochori tunnel in Northern Greece).



FLYSCH ROCK MASS TYPE		CHARACTERISTIC "KEYS" FOR TUNNEL BEHAVIOUR OR INSTABILITY	TUNNEL BEHAVIOUR
Type I		• Geometrical and shear strength characteristics of joints. • High intact rock strength. • Slightly fractured-undisturbed massive structure	– The rock mass behaviour is purely anisotropic. – Wedge detachment and sliding. – Controlled by the orientation of discontinuities in relation to the orientation of the tunnel.
Type II		• Low to medium intact rock strength. • Slightly fractured-undisturbed structure	– The behaviour of the rock mass is controlled by the low strength of the siltstone and the excavation depth. – In great depths limited de-formation can develop, whereas in small depths the tunnel is generally stable and, depending on the orientation of the tunnel and the discontinuities, sliding and fall of wedges can occur.
Type III		• Geometrical and shear strength characteristics of joints, especially along the planes. • High intact rock strength. • Moderately fractured structure.	– The rock mass behaviour is purely anisotropic. – Wedge detachment and sliding. – Controlled by the orientation of discontinuities in relation to the orientation of the tunnel.
Type IV		• Geometrical and shear strength characteristics of joints, especially along the planes. • Moderately fractured structure and moderately intact rock strength. • Siltstone layers with smooth to slickensided sheared surfaces contribute to instabilities. • Rock blocks are generally moderate (1-2m x 1-3m)	– The behaviour of the rock mass is anisotropic. – Wedge detachment and sliding. – Controlled by the orientation of discontinuities in relation to the orientation of the tunnel. – When the layers are close to horizontal and especially when the rock mass is thin-bedded, overexcavation problems can appear. – In places where the rock mass is locally more loose and weathered with no significant confinement, limited chimney type failures can occur.
Type V		• Moderately fractured structure and low to moderately intact rock strength ("Weighted" value). • The geometry of the slabs and the shear strength characteristics of the smooth to slickensided sheared siltstone surfaces contribute to free falls and wedge slides. • The persistence of other joints is small and thus rock blocks are generally small to moderate. • Structure is loosened and folliated very close to the surface. • Particular care about expansive minerals	– The rock mass behaviour is close to isotropic concerning deformation. – Limited deformation can develop under medium overburden. – In small depths the tunnel is generally stable, but depending on the orientation of discontinuities, sliding and fall of wedges can occur. – Close to the surface extended overexcavation and chimney type failures can appear, due to weathering and foliation, especially in Type VI (reduced sandstone presence to "bridge").
Type VI		• Moderately fractured structure and low intact rock strength ("Weighted" value). • The geometry of the slabs and the shear strength characteristics of the smooth to slickensided sheared siltstone surfaces contribute to free falls and wedge slides. • The persistence of other joints is small and thus rock blocks are generally small to moderate. • Structure is loosened and foliated very close to the surface. • Particular care about expansive minerals	
Type VII		• Highly disturbed, folded rock mass. • Medium to low intact rock strength, reduced due to the siltstone participation, create favorable conditions for strains under medium cover. • The geometry of the slabs and the shear strength characteristics of the smooth to slickensided sheared siltstone surfaces contribute to free falls and wedge slides of small volume.	– The behaviour of the rock mass can be well considered as isotropic. – Limited deformation can develop under medium overburden. – In small depths the tunnel is generally stable, but depending on the orientation of discontinuities, sliding and fall of wedges can occur. – As a result of the relatively good "interlocking" of the rock mass due to its folded structure, no extended falls are expected, except only in weathered zones close to the surface.
Type VIII		• Highly disturbed, folded rock mass. • Medium to low intact rock strength, reduced due to the siltstone participation, create favorable conditions for strains under medium cover. • The geometry of the slabs and the shear strength characteristics of the smooth to slickensided sheared siltstone surfaces contribute to free falls and wedge slides of small volume. • Possible expansive mineral. • Permeability is low.	– The rock mass behaviour is clearly isotropic. – Due to the low strength of the siltstone, deformation starts to develop under medium overburden. – Detachments and slides of blocks may locally occur. – As a result of the relatively good "interlocking" of the rock mass due to its folded structure, extended falls and chimney failures are only expected in weathered parts in very small depths, due to weathering and foliated structure.
Type IX		• Brecciated, disintegrated structure. Interlocking of the fragments is of major importance. • Intact rock strength is medium to high. • The overall rock mass strength is reduced due to the disturbed nature of the rock mass. • Although the equivalent friction angle is high, the equivalent cohesion of the disintegrated mass is practically negligible, except if some secondary fine binding material gives a small cohesion to the rock mass. • The presence of clayey-sandy along the joints can loosen though the good interlocking. • Permeability is medium to high.	– The behaviour of the rock mass is isotropic, governed by the disintegrated structure, and after excavation it can start to collapse. – In cases of open structure and strong presence of water, raveling is immediate and extensive and cannot be easily limited until the induced void creates a ground arch or reaches the ground surface. – In great depths, as the intact rock has a considerable strength, no significant deformation is expected.
Type X		• Tectonically deformed, intensively folded/faulted. • Almost a chaotic structure. • Low intact rock strength. • The overall rock mass strength is even more reduced due to the disturbed nature of the rock mass. • Possible expansive mineral. • Permeability is low. • Presence of water reduces even more the rock mass strength	– The behaviour of the rock mass is clearly isotropic, controlled by its low strength and high deformability that are responsible for the development of important deformation, even under low to medium overburden. – In greater depths, squeezing conditions can be adverse causing sometimes failure of rigid support sections due to overloading of the shell, especially in Type XI. This can lead to adoption of a yielding support that can undertake the high loads without failing. – Additionally, particular care is needed close to the surface, where important overexcavation can occur, due to weathering and the foliated, fragile structure.
Type XI		• Tectonically strongly sheared to chaotic structure. • Low to very low intact rock strength. • The overall rock mass strength is even more reduced due to the disturbed structure. • No blocks are formed. • Possible expansive mineral. • Permeability is low. • Presence of water reduces even more the rock mass strength	

Fig. 10. Engineering geological characteristics keys for assessing tunnel instability for each flysch type (I–XI).

the behaviours of the flysch formations during tunnelling depend on 3 major parameters: (i) the structure, (ii) the intact strength of dominant rock type and (iii) the depth of the tunnel. The expected behaviour types (stable, wedge failure, chimney type failure, ravelling ground, shear failures, squeezing ground) can be illustrated in a tunnel behaviour chart (TBC) (Marinos, 2012). The main failure mechanism for every flysch rock mass type (I–XI) is projected in a TBC chart in Fig. 11.



St: Stable ground
Gravity induced failures: Wg:Wedge failure Ch:Chimney type failure Rv:Ravelling ground
Stress induced failures: Sh:Shear failure Sq:Squeezing ground

Notes:

*** The data used in the TBC were obtained from tunnels excavated by the conventional method with top heading and bench in a non-urban environment with the overburden cover up to several hundred metres (generally not exceeding 500m) with a tunnel diameter=12m**

****The chart does not refer to very high overburden (e.g. many hundreds of m or >1000m), where the scale and the mechanism of failure may differ**

- **The limit-ranges of the uniaxial compressive strength (σ_{ci}) of the intact rock and the overburden thickness (H) are indicative. This is done to avoid standardisation by an inexperienced user. The purpose of this diagram is to predict the failure mechanism of several common rock mass types.**
- **Groundwater presence mainly affects the factor of safety and not the behaviour type. Though, in some cases, such as "Blocky-Disturbed" & "Disintegrated" rock mass, the groundwater presence may "shift" a Chimney (Ch) or Ravelling (Rv) behaviour type to Flowing ground (Fl)**
- **Cases number 4, 8 and 12 may develop brittle failures (Br) when overburden increases considerably (e.g. >800 m) depending on the intact rock strength**
- **The illustrations of the tunnel are sketches; this shape corresponds to the usual top heading**

Fig. 11. Modified tunnel behaviour chart (TBC) from Marinos (2012) with projections of the principal failure mechanisms for the rock mass types of flysch (I–XI).

GROUND CHARACTERIZATION, BEHAVIOUR AND SUPPORT FOR TUNNELS (1/2) (V. Marinos, 2012)

Location:	Metsovo area (Northern Greece)
Classification phase (primary, evaluation, construction)**:**	Final design phase
Date:	

I. GEOLOGICAL CONDITIONS

a) Lithology	
Geotectonic unit:	“Pindos” geotectonic unit-zone
General formation to which it belongs (e.g. Flysch):	Flysch
Rock mass name:	Type X. Siltstone - claystone chaotic structure with sandstone fragments. Claystone chaotic structure with sandstone intrusions Note: information concerning significant alteration of the rock mass, intact rock or surface weathering, presence of hosted - clayey geomaterial and bedding thickness if it is stratified
b) Tectonism	
Tectonic zones:	
Major thrust zones which affect the project in great scale:	The area is disturbed by several thrusts. The rock mass is sheared in a wider area (tens of m).
Localized fault or disturbed zones:	The disturbance is not localized in a specific fault zone (several m).
Fracturing or Shearing:	
Fracturing degree:	**Slightly fractured** ☐ **Fractured** ☐ **Very fractured** ☒ **Brecciated** ☐
Continuation- persistence of fracturing with depth:	The tectonic disturbance continues in depth for tens of m. The sandstone beds are broken, almost transformed into small pieces.
Shearing or foliation across the rock mass:	Siltstone bedding is highly sheared and foliation planes of very low strength are present. / Intact rock of siltstone has been intensively sheared
Folding:	
Type:	Several folds (recumbent type)
Geometry:	
c) Weathering	
Discontinuities:	Weathering is strongly favored along the siltstone planes. Clay minerals are formed
Intact rock:	The rock is disintegrated in small pieces due to slaking
Persistence with depth:	Weathering is limited only close to the surface
d) Permeability	
Qualitative appraisal:	**High** ($k>10^{-3}$m/sec) ☐ **Low** (k:10^{-7}-10^{-9}m/sec) ☐ **Practically impermeable** ($k<10^{-9}$m/sec) ☐ **Medium**(k:10^{-5}-10^{-3}m/sec) ☐ **Very low** (k:10^{-9}-10^{-7}m/sec) ☐
Quantitative appraisal:	**k:** ☐ **m/sec**

II. IN SITU CONDITIONS AND TUNNEL CHARACTERISTICS

a) Tunnel Geometry	
Tunnel Size:	12m
Shape:	Horseshoe
Tunnel Direction:	Horizontal in E-W direction
b) Overburden	
Overburden range with similar behaviour:	**H:** 150-200 **m**
Insitu stresses ($P_o=\gamma H_{min}$ to γH_{max}):	**p_o:** 5 **MPa**
c) Stress field particularities	
Particular presence of lateral pressures (k_o):	
d) Adjacent zone close to tunnel perimeter	
Weak zone close to tunnel perimeter:	**Dip:** / **Dip Direction** **Thickness:** **m** **Geological characteristics:**
Competent zone close to tunnel perimeter:	**Dip:** / **Dip Direction** **Thickness:** 3 **m** **Geological characteristics:** Sandstone beds overlying the weak sheared rock mass. If these beds are close to the tunnel perimeter, strains could be reduced
e) Hydrogeological conditions (location of aquifer according to the tunnel axis)	Aquifer is located above the tunnel axis
f) Other boundaries	

III. CHARACTERISTIC “KEYS” FOR TUNNEL BEHAVIOUR OR INSTABILITY

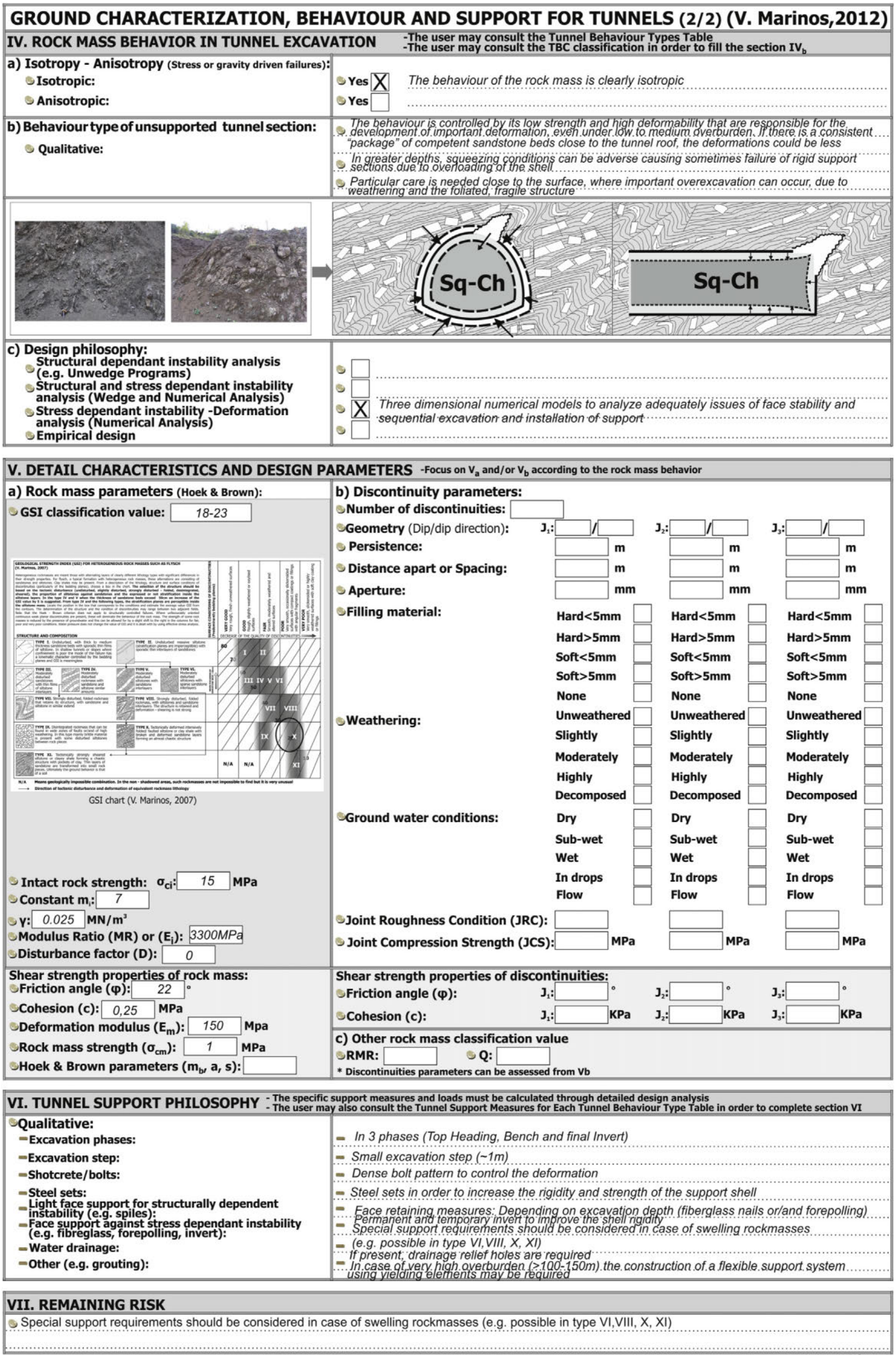
Intact rock strength:	Siltstone strength has been considerably reduced due to shearing. Sandstone strength does not significally contribute to the to weighted “intact rock”
Rock mass strength to insitu stress ratio(σ_{cm}/p_o):	**$\sigma_{cm}/p_o>0.6$** ☐ **$0.3<\sigma_{cm}/p_o<0.6$** ☐ **$\sigma_{cm}/p_o<0.3$** ☒
Structure “interlocking”:	Sandstone beds marginally “follows” the folding of the siltstone beds
Presence of low strength minerals:	Clayey minerals maybe present. Possible swelling minerals
Intact rock weathering, clay zones:	Clay zones due to intensive shearing
Groundwater presence:	Groundwater is present but cannot be drained easily
Block geometry - bed thickness:	Only small sandstone blocks may be present
Rock mass structure (based to GSI classification)	**Blocky** ☐ **Very blocky** ☐ **Blocky/Disturbed/Seamy** ☐ **Disintegrated** ☐ **Laminated/Sheared** ☒
Discontinuity geometry:	The geometry “shifts” within 1-2m
Discontinuity persistence:	The discontinuities, if possible to be measures in some points, are not persistent
Discontinuity quality (based to GSI classification)	**Very good** ☐ **Good** ☐ **Fair** ☐ **Poor** ☒ **Very poor** ☒
Rock Quality Index (RQD):	**RQD:** N/A
Other characteristic:	
Fundamental engineering geological characteristics - ”Keys”:	Low intact rock strength (5-15MPa), Very disturbed-sheared structure (GSI=15-20) Very low σcm/po. Water pressure

☒ **The behaviour is controlled by the overall rock mass:** ↓ **It is essential to enter the relevant rock mass parameters in field “Va”**	☐ **The behaviour is controlled by the discontinuities:** ↓ **It is essential to enter the relevant parameters of the discontinuities in field “Vb”**

Page 1 from 2

(a)

Fig. 12. Modified example of a Ground Characterisation, Behaviour and Support for Tunnels (modified from Marinos (2012)). Illustrated, in light characters, by an example of tunnelling in a tectonically deformed intensively folded siltstone (flysch rock mass type X).



GROUND CHARACTERIZATION, BEHAVIOUR AND SUPPORT FOR TUNNELS (2/2) (V. Marinos,2012)

IV. ROCK MASS BEHAVIOR IN TUNNEL EXCAVATION
-The user may consult the Tunnel Behaviour Types Table
-The user may consult the TBC classification in order to fill the section IV_b

a) Isotropy - Anisotropy (Stress or gravity driven failures):
- **Isotropic:** Yes ☒ *The behaviour of the rock mass is clearly isotropic*
- **Anisotropic:** Yes ☐

b) Behaviour type of unsupported tunnel section:
- **Qualitative:**
  - *The behaviour is controlled by its low strength and high deformability that are responsible for the development of important deformation, even under low to medium overburden. If there is a consistent "package" of competent sandstone beds close to the tunnel roof, the deformations could be less*
  - *In greater depths, squeezing conditions can be adverse causing sometimes failure of rigid support sections due to overloading of the shell*
  - *Particular care is needed close to the surface, where important overexcavation can occur, due to weathering and the foliated, fragile structure*

c) Design philosophy:
- **Structural dependant instability analysis (e.g. Unwedge Programs)** ☐
- **Structural and stress dependant instability analysis (Wedge and Numerical Analysis)** ☐
- **Stress dependant instability -Deformation analysis (Numerical Analysis)** ☒ *Three dimensional numerical models to analyze adequately issues of face stability and sequential excavation and installation of support*
- **Empirical design** ☐

V. DETAIL CHARACTERISTICS AND DESIGN PARAMETERS -Focus on V_a and/or V_b according to the rock mass behavior

a) Rock mass parameters (Hoek & Brown):
- **GSI classification value:** 18-23

GSI chart (V. Marinos, 2007)

- **Intact rock strength:** σ_{ci}: 15 MPa
- **Constant m_i:** 7
- **γ:** 0.025 MN/m³
- **Modulus Ratio (MR) or (E_i):** 3300MPa
- **Disturbance factor (D):** 0

Shear strength properties of rock mass:
- **Friction angle (φ):** 22 °
- **Cohesion (c):** 0,25 MPa
- **Deformation modulus (E_m):** 150 Mpa
- **Rock mass strength (σ_{cm}):** 1 MPa
- **Hoek & Brown parameters (m_b, a, s):**

b) Discontinuity parameters:
- **Number of discontinuities:**

	J_1	J_2	J_3
Geometry (Dip/dip direction):	/	/	/
Persistence:	m	m	m
Distance apart or Spacing:	m	m	m
Aperture:	mm	mm	mm
Filling material:	Hard<5mm ☐ Hard>5mm ☐ Soft<5mm ☐ Soft>5mm ☐ None ☐	Hard<5mm ☐ Hard>5mm ☐ Soft<5mm ☐ Soft>5mm ☐ None ☐	Hard<5mm ☐ Hard>5mm ☐ Soft<5mm ☐ Soft>5mm ☐ None ☐
Weathering:	Unweathered ☐ Slightly ☐ Moderately ☐ Highly ☐ Decomposed ☐	Unweathered ☐ Slightly ☐ Moderately ☐ Highly ☐ Decomposed ☐	Unweathered ☐ Slightly ☐ Moderately ☐ Highly ☐ Decomposed ☐
Ground water conditions:	Dry ☐ Sub-wet ☐ Wet ☐ In drops ☐ Flow ☐	Dry ☐ Sub-wet ☐ Wet ☐ In drops ☐ Flow ☐	Dry ☐ Sub-wet ☐ Wet ☐ In drops ☐ Flow ☐
Joint Roughness Condition (JRC):			
Joint Compression Strength (JCS):	MPa	MPa	MPa

Shear strength properties of discontinuities:

	J_1	J_2	J_3
Friction angle (φ):	°	°	°
Cohesion (c):	KPa	KPa	KPa

c) Other rock mass classification value
- **RMR:**
- **Q:**

*** Discontinuities parameters can be assessed from Vb**

VI. TUNNEL SUPPORT PHILOSOPHY
- The specific support measures and loads must be calculated through detailed design analysis
- The user may also consult the Tunnel Support Measures for Each Tunnel Behaviour Type Table in order to complete section VI

Qualitative:
- **Excavation phases:** *In 3 phases (Top Heading, Bench and final Invert)*
- **Excavation step:** *Small excavation step (~1m)*
- **Shotcrete/bolts:** *Dense bolt pattern to control the deformation*
- **Steel sets:** *Steel sets in order to increase the rigidity and strength of the support shell*
- **Light face support for structurally dependent instability (e.g. spiles):** *Face retaining measures: Depending on excavation depth (fiberglass nails or/and forepolling)*
- **Face support against stress dependant instability (e.g. fibreglass, forepolling, invert):** *Permanent and temporary invert to improve the shell rigidity*
  Special support requirements should be considered in case of swelling rockmasses (e.g. possible in type VI,VIII, X, XI)
- **Water drainage:** *If present, drainage relief holes are required*
- **Other (e.g. grouting):** *In case of very high overburden (>100-150m) the construction of a flexible support system using yielding elements may be required*

VII. REMAINING RISK
- Special support requirements should be considered in case of swelling rockmasses (e.g. possible in type VI,VIII, X, XI)

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(b)

Fig. 12. (*continued*).

Apart from the characterisation in Figs. 10 and 11, the estimation of the tunnel behaviour and the philosophy of the support measures should be also performed on the basis of a detailed ground characterisation. This detailed characterisation cannot ignore the geological and/or in situ characteristics dictating or influencing the tunnel behaviour compared with a standardised classification (Marinos, 2012). This characterisation, named "Ground Characterization, Behaviour and Support for Tunnels" (Marinos, 2012) prompts user to evaluate the data in detail in order to assess the tunnel behaviour and adopt the appropriate support measures. An example of this characterisation in a tectonically disturbed flysch types is presented in Fig. 12.

The rock mass is often considered as an equivalent "mean isotropic geomaterial", where rock mass properties are quantified through classification systems. This assumption is usually acceptable in cases of uniformly jointed, highly tectonised or disintegrated rock mass without persisting discontinuities of stable orientation controlling the rock mass behaviour. This is the case of the types VII–IX. In the case of bedded rock masses, at a scale of the tunnel section, the engineering geological behaviour during tunnel construction is significantly controlled by the characteristics of the stratification planes. This case may apply to flysch rock mass types IV–VI. A simulation of this anisotropic behaviour was analysed in Fortsakis et al. (2012).

4. Temporary support measures

The implementation of empirical tunnel design methods based on rock mass classification or simplified methods such as the convergence–confinement method should be of limited use in the design of tunnels in most of the flysch rock mass types. Such design cannot deal adequately with issues of face stability and the sequential excavation and installation of support. Therefore, the design of tunnels in weak flysch rock masses must involve the use of numerical methods. In some critical cases, like the simulation of the effectiveness of forepoling, tunnel advance and sequential support installation, three-dimensional numerical models should be used. However, in weak rock masses, the uses of sound engineering judgement and experiences from similar cases are valuable for the design and the construction of tunnel. The geotechnical properties of the material used for these analyses were calculated based on Hoek–Brown failure criterion. It should be highlighted here that in most of all cases the results of the model studies have been validated by the interpretation of convergence measurements and by the observation of the tunnel and installed support performance. Detailed principles and guidelines for selecting the immediate support measures are proposed based on the principal tunnel behaviour mode and the experiences from these 12 tunnels. In terms of permanent support concerned, different systems were presented in Fortsakis et al. (2004).

The tunnels under consideration are large in size with span of about 12 m. Apart from some cases of straightforward tunnelling in areas of good rock masses of flysch (types I–V), most of the studied tunnels were excavated under difficult geological conditions (types VII–XI). These tunnels have been excavated using top heading and bench method. Special measures were taken to stabilise the face like forepoling or/and installation of long grouted fibreglass dowels in the face. In addition, immediate shotcreting and leaving a core for buttressing have been used in different combinations for face stabilisation. After the stabilisation of the face, the application of the primary support system, consisting of shotcrete layers, rockbolts, steel sets or lattice girders embedded in the shotcrete in various combinations was necessary to ensure the stability of the tunnel. Elephant's foot and micropiles in rare cases were used to assist the foundation of the top heading shell and to secure stability when benching. Temporary and permanent invert closure was implemented in order to face squeezing conditions. A typical support design for weak flysch rock masses, using top heading and bench method, is presented in Fig. 13 (Marinos et al., 2006a).

Under severe squeezing, the application of yielding systems was an alternative solution. The applied system was described in Schubert (1996) and Hoek et al. (2008). In the case of tectonically sheared siltstone rock masses under high cover (e.g. up to 250 m), where tunnel squeezing is a significant problem, the pillar stability in these twin tunnels requires careful evaluation.

The wide range of engineering geological behaviour leads to a corresponding range of temporary support measures. The temporary support in the specific tunnels discussed here varies from very light to very rigid or yielding. Temporary support measures concept and principles for every rock mass type are presented, based on the available tunnelling experiences, as shown in Fig. 14. It is not in the scope of this paper to provide analytical support measures. This work requires detailed design analysis of the tunnel support, adapted to the in situ conditions and particularities of each project. Here, the support proposals are reasonable considerations of both the rock mass behaviour and the critical failure mechanism, which are different for every flysch rock mass type. The necessity, the amount and the combination of the various elements of this typical section are results of numerical analysis and the optimization is a matter of reliable monitoring. The time of constructing temporary support is related with the support principle. A quick construction of a stiff support is usually implemented in case that there is a very small tolerance for displacements, whereas a yielding support that decreases the loads corresponds to a larger time interval.

The average excavation step for the top heading excavation of flysch rocks is presented in Fig. 15. The excavation step must be decided upon: (i) the anticipated size of wedges in the case of not tectonically stressed rock masses, (ii) the size of the wedges and the loosening prevention of the structure, in the case of disturbed rock masses without deformation problems, (iii) the prevention of structure loosening and (iv) decrease of deformation in association with the other appropriate measures in the case of weak rock masses where significant deformation is anticipated. For the cases (i)–(iii), the installation of spiles allows the increase of the excavation step. Excavation step is very difficult to exceed 1–1.5 m in very weak rock masses, while a mean value for the undisturbed rock masses could be 3 m.

The cost (Euros/linear metre of tunnel) of the temporary support system for the flysch formations from the experience of the Egnatia

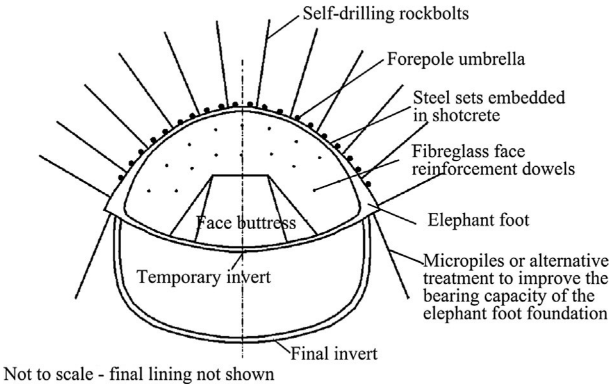
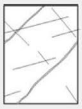
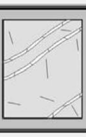
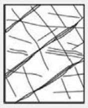
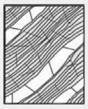
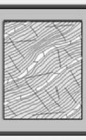
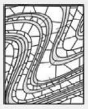


Fig. 13. A typical support design for weak flysch rock masses using top heading and bench method. The necessity, the amount and the combination of various elements of this typical section are results of numerical analysis. The optimisation is a matter of reliable monitoring. For highly squeezing ground, the philosophy of a yielding support is recommended (sketch from Hoek (Marinos et al., 2006a)).

ROCKMASS TYPE	STRUCTURE	TEMPORARY SUPPORT RECOMMENDATIONS
Type I. Undisturbed, with thick to medium thickness sandstone beds with sporadic thin films of siltstone.		• Excavation step: ≥3.0m • Installation of split-set bolts (e.g. Swellex) to support the unstable wedges (Sparse installation is not recommended due to the large dimensions of typical transportation tunnels)
Type II. Undisturbed massive siltstone with sporadic thin interlayers of sandstones.		• Excavation step: 2-3m • Bolts installation to support the unstable wedges and control the deformation in case of high overburden • Light steel sets in case of weathered rockmass, depending on excavation depth
Type III. Moderately disturbed sandstones with thin of siltstone interlayers.		• Excavation step: 1.5-2m • Installation of split-set bolts (e.g. Swellex type) for the support of unstable wedges • Light steel sets in case of loose structure
Type IV. Moderetaly disturbed rock mass with sandstone and siltstone similar amounts.		• Excavation step: 1.5-2m • Systematic bolt installation to support the unstable wedges, prevent the rockmass loosening and control the deformation in case of high overburden • Spiles and light steel sets in case of loose structure and weathered rockmass to avoid local chimney type failures
Type V. Moderately disturbed siltstones with thin sandstone interlayers.		• Excavation step: 1.5-2m • Systematic bolt installation to support the unstable wedges, prevent rockmass loosening and control the deformation under high overburden • Light steel sets to increase the rigidity and strength of the support shell • Spiles in case of loose and weathered structures to avoid chimney type failures • Face retaining measures: Depending on excavation depth (fibreglass nails)
Type VI. Moderately disturbed siltstones with sparse sandstone interlayers.		• Excavation step: 1.5-2m • Dense bolt pattern to control the deformation and prevent rockmass loosening • Steel sets to increase the rigidity and strength of the support shell • Spiles to stabilise loose and weathered structures and avoid chimney type failures • Face retaining measures: Depending on excavation depth (fibreglass nails) • Depending on bedding orientation, anisotropic stress induced deformations may be observed
Type VII. Strongly disturbed, folded rock mass that retains its structure, with sandstone and siltstone in similar extent.		• Excavation step: 1.5-2m • Dense bolt pattern to control of deformation and rockmass loosening prevention • Steel sets to increase the rigidity and strength of the support shell • Face retaining measures: Depending on excavation depth (fibreglass nails or/and forepolling)
Type VIII. Strongly disturbed, folded rock mass with siltstones and sandstone interlayers. The structure is retained and deformation – shearing is not strong.		• Excavation step usually small: 1-1.5m • Dense bolt pattern to control the deformation • Steel sets to increase the rigidity and strength of the support shell • Face retaining measures: Depending on excavation depth (fibreglass nails or/and forepolling) • Permanent and probably temporary invert to improve the shell rigidity.
Type IX. Disintegrated rockmass that can be found in wide zones of faults or/and of high weathering.		• Excavation step usually small (~1m) • Face buttress • Dense pattern of self-drilling anchors. Grouting to locally increase the rockmass cohesion • Steel sets to increase the rigidity and strength of the support shell • Spiles to presupport tunnel roof and prevent the development of chimney type failure • Alternatively in case of completely cohesionless rockmass grouting around tunnel section is proposed (e.g. through perforated forepolles)
Type X. Tectonically deformed intensively folded/faulted siltstone or clay shale with broken and deformed sandstone layers forming an almost chaotic structure.		• Small excavation step (~1m) • Dense bolt pattern to control the deformation • Steel sets in order to increase the rigidity and strength of the support shell • Face retaining measures: Depending on excavation depth (fibreglass nails or/and forepolling) • Permanent and temporary invert to improve the shell rigidity
Type XI. Tectonically strongly sheared siltstone or clayey shale forming a chaotic structure with pockets of clay.		• Small excavation step (~1m) • Dense bolt pattern and steel sets to increase the rigidity and strength of the support shell • Face retaining measures: Depending on excavation depth (fibreglass nails or/and forepolling) • Permanent and temporary invert to improve the shell rigidity • In case of very high overburden (>100-150m) the construction of a flexible support system using yielding elements may be required.

Remarks:

- The excavation is referred to Top heading and Bench method. Full face excavation in weak rockmasses imposes strong face retaining measures and small distance between temporary support and final lining.
- Shotcrete is not referred in the recommendations due to its wide application. More specifically, when shotcrete is used to avoid rockmass loosening and to ensure the personnel safety, its thickness is generally small and it is determined according to experience and evaluation of the magnitude of possible wedge failure. In stress induced phenomena due to the combination of weak rockmass and high excavation depth or/and swelling phenomena, shotcrete should be analysed as a structural element and the requisite thickness and reinforcement is determined through numerical analyses.
- The excavation step will be determined according to: (a) the anticipated size of wedges in the case of competent undisturbed rockmasses (b) the size of the wedges and the structure loosening prevention, in the case of disturbed rockmasses with no deformation problems (c) the prevention of structure loosening and decrease of deformation, in the case of weak rock masses where significant deformation is anticipated. However, the installation of spiles allows the increase of the excavation step.
- Drainage holes are proposed in case of permeable sandstone beds and relief holes in case of trapped, low permeable, groundwater zones under the water table.
- Special support requirements should be considered in case of swelling rockmasses (e.g. possible in type VI, VIII, X, XI).

Fig. 14. General directions for the immediate support measures for every flysch type (Marinos et al., 2011a).

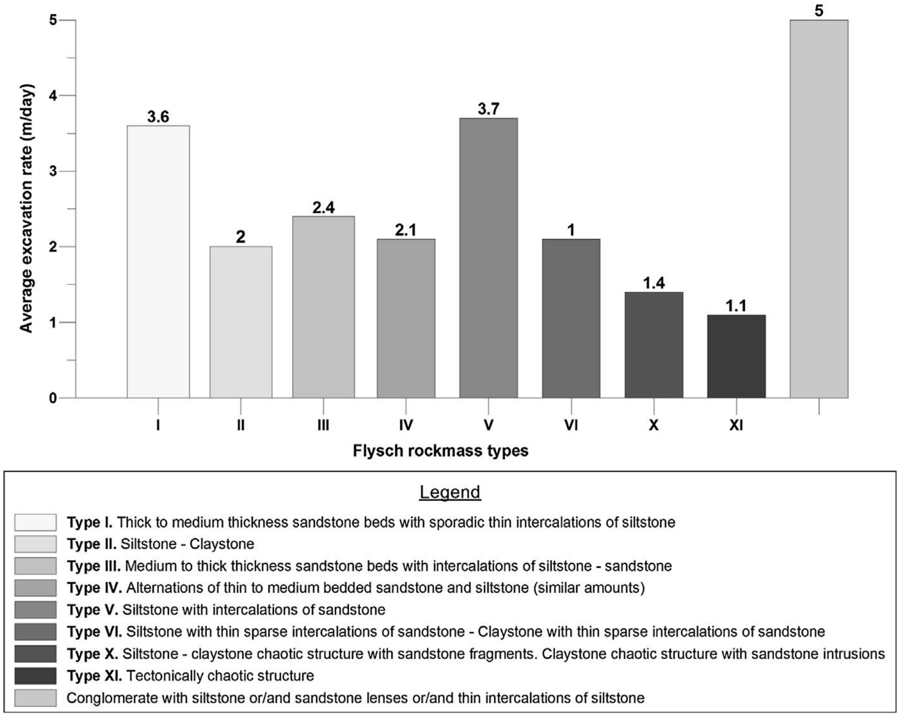


Fig. 15. Average top heading excavation step for flysch rock masses (types I, II, III, IV, V, VI, X and XI). A conglomerate mass is also projected in the last column of the diagram.

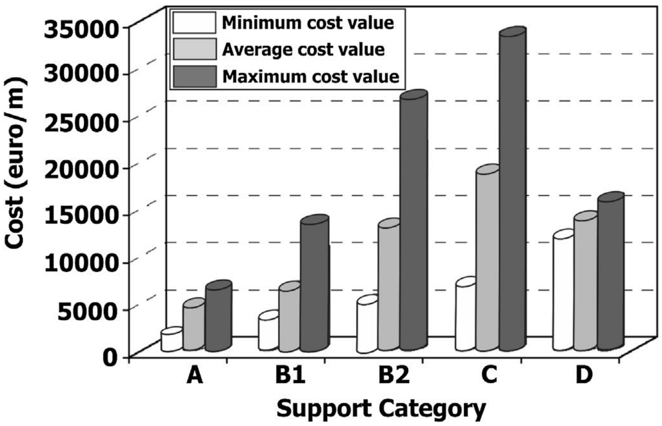


Fig. 16. Cost (Euros/linear metre of tunnel) of the temporary support system for the flysch formations. A–D is the "weight" of the support measures (A: shotcrete and bolts; B1: shotcrete, bolts and steel sets; B2: shotcrete, bolts, steel sets and light face support measures like spilling; C: shotcrete, bolts, steel sets and forepoling and D: yielding support system). Category D was only used in one case study.

highway tunnels is projected in Fig. 16. This cost is presented in accordance with the "weight" of the support category.

5. Conclusions

The processing and evaluation of a great amount of geological and geotechnical information, obtained from the design and construction of 12 tunnels driven in flysch in Northern Greece, contributed to assessing the behaviours of the ground and the formulation in association with the correlations between ground and the formulation behaviours and the temporary support required.

Flysch formations are generally characterised by strong heterogeneity in the presence of low strength and tectonically disturbed structures, which may produce heavily sheared and chaotic masses. Flysch rock masses can be composed of sandstone and siltstone beds (undisturbed to folded) and inherently weak materials subjected to strong shearing where the original structure of the rock mass is no longer recognizable. The rock mass strength parameters needed for design can be sufficiently estimated by the Hoek–Brown failure criterion as long as the rock mass reacts isotropically to the underground excavation. Thus, a specialised GSI chart for the heterogeneous rock masses such as flysch can be used.

Flysch of various types can either be stable even under noticeable overburden and exhibit wedge sliding and chimney type failures, or cause serious deformation even under low to medium overburden. The rock mass behaviour in undisturbed to moderately undisturbed structures is highly anisotropic and controlled by the orientation and properties of discontinuities, mainly the bedding, in relation to the orientation of the tunnel. As a result, there is a possibility of wedge detachment and sliding along thin siltstone layers with low shear strength. The behaviour of the disturbed structures and even more of the heavily sheared rock mass types is generally isotropic, controlled by their low strength and low modulus of deformability. These masses may develop a significant deformation, even under low to medium overburden, while at greater depths squeezing prevails.

A wide range of temporary support can be applied in flysch rock masses, varying from very light to very rigid or yielding under severe squeezing conditions. Specific suggestions for the theory of temporary support in tunnel excavation through each flysch type are presented. These proposals take into account both the rock mass behaviour and the critical failure mechanism, which yet cannot replace the detailed analysis. They should be always back-analysed by engineering judgement and adjusted for each site-specific project.

Conflict of interest

We wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

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Dr. V. Marinos is Assistant Professor of Engineering Geology in the Department of Geology at the Aristotle University of Thessaloniki. He got Bachelor degree in Geology from the National University of Athens, a Master of Science degree of Engineering Geology with Distinction as well as a diploma DIC from Imperial College and a PhD in geotechnical engineering from the National Technical University of Athens. His research interests cover engineering geological assessment and geotechnical characterisation of weak and complex rock masses, assessment of their behaviour and support in tunnelling, creation of geotechnical database for tunnels, rock slope stability assessment, evaluation and analysis of ground movement in mines and evaluation of landslide hazard analysis. Dr. V. Marinos has published 61 papers in the field of Engineering Geology, Geotechnical Engineering and Hydrogeology–Engineering Geology in peer-reviewed scientific journals and conferences. He has participated in 13 granted research programs. He has been consulting in design and construction on engineering geological topics for major engineering projects (metro works and mountain tunnels, dams, road works and landslides) under a variety of geological conditions. Dr. V. Marinos has over than 150 citations of his research work in international journals and international conferences, inclusions in the design specifications of metro works with sections of his research work and use of parts of his work (geotechnical characterisation of weak and complex rock masses with the GSI system) in the design of large engineering works. Dr. V. Marinos is an active member of several professional bodies and learned international societies like the IAEG, ISSMGE, ISRM (member of the Soft Rock Commission), National Committee of Tunnels and Underground Structures and the International Association of Hydrogeology and other national scientific Committees.