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Engineering Geology and Tunnels

Vassilis Marinos

Abstract

Currently, knowledge and understanding of the role of geological material and its implication in tunnel design is reinforced with advances in site investigation methods, the development of geotechnical classification systems and the consequent quantification of rock masses. However, the contribution of engineering geological information in tunnelling cannot be simply presented solely by a rock mass classification value. What is presented in this chapter is that the first step is not to start performing numerous calculations but to define the potential failure mechanisms. After defining the failure mechanism that is most critical, selection of the suitable design parameters is undertaken. This is then followed by the analysis and performance of the temporary support system based on a more realistic model. The specific failure mechanism is controlled and contained by the support system. A tunnel engineer must early assess all the critical engineering geological characteristics of the rock mass and the relevant mode of failure, for the specific factors of influence, and then decide either he or she will rely on a rock mass classification value to characterise all the site-specific conditions. Experiences from the tunnel behaviour of rock masses in different geological environments in Alpine mountain ridges are presented in this chapter.

Keywords: weak rocks, ground types, tunnel behaviour, tunnel design, tunnel behaviour chart, geotechnical classifications

1. Introduction

Currently, knowledge and understanding of the role of geological material and its implication in design is reinforced with advances in site investigation methods, the development of geotechnical classification systems and the consequent quantification of rock masses. Rock mass rating (RMR) [1] and Q [2] were developed to provide tunnel support estimates through a rating of rock masses. In addition, the advancement of the geotechnical software that is easier to use led to an increased requirement of data related to mass properties. This kind of data is needed as input for analysis in the numerical solutions for designing tunnels. The onset of numerical tools to handle rock-support interaction and the advancement of concepts related to ground reaction curve permitted issues to be managed well beyond the ultimate extent for application of different tunnel support classification systems such as the RMR and Q systems. Practice picked up from the early application of more modern numerical modelling recommended that there was great correspondence between the rules from these classifications and the displaying outcomes about reality when rock mass behaviour was generally simple; for example, the RMR system does not give good outcomes beyond the range of values between 30 and 70 under moderate stresses. Good results and realistic outcomes may well be produced where the

sliding and rotation of intact rock pieces essentially controlled the overall failure process, comparing to an encounter database on which the early classifications were built. Truly hundreds of kilometres of tunnels were effectively excavated on the sole premise of this application.

Solid appraisals of the strength and deformation characteristics of rock masses are required for nearly any procedure of investigation concerning an underground work. Subsequently, an approach for the estimation of rock mass properties from intact rock properties and joint characteristics is fundamental. The Hoek-Brown failure criterion [3–5] would be of no benefit in the event that it might not be promptly connected with engineering geological perceptions for the nature and fabric of the rock mass. Hoek [6] proposed a methodology for getting estimates of the strength of fractured rock masses based on an appraisal of the interlocking of rock pieces and the condition of the surfaces between these pieces. For such an evaluation, the Geological Strength Index (GSI) was presented. The GSI has been established over a long period [3, 5, 7–13] to meet the desires of practitioners and cases that were not at first realised. The application of the mass properties from the GSI values basically accepts that the rock mass behaves isotopically. It is not factional where there is a clear anisotropic behaviour, e.g. clearly characterised favoured failure surface or discontinuities. The appropriate use of rock mass characterisation systems, notably the GSI (for details, see [14, 24]), allowed the quantification of difficult ground for the evaluation of the geotechnical properties and the selection of the design parameters. An extension of the original GSI application charts for heterogeneous and structurally complex rock masses, such as flysch, was initially introduced by Marinos and Hoek [10] and recently updated and extended by Marinos [11]. Specific GSI charts for molassic formations [12], ophiolites [13], gneiss in its disturbed form [14, 15] and particular cases of limestones [15] and under particularly difficult geological conditions have been developed from experience gained during excavation of 62 tunnels as part of the Egnatia project in Northern Greece, along Alpine mountain belts.

In expansion to the GSI values, it is additionally fundamental to consider the choice of the 'intact rock' properties σ_{ci} and m_i for these rock masses with different mineral composition. The fundamental inputs of the Hoek-Brown failure criterion are assessments or measurements of the uniaxial compressive strength (σ_{ci}) and the material constant (m_i) related to the frictional properties of the rock and of the GSI. Furthermore, to assess the deformation modulus of the rock mass (E_m), Hoek and Diedrichs [16] proposed a formula based on the values of the intact rock deformation modulus (Ei) or the modulus ratio (MR).

The role of the ground characteristics and its effect in tunnel design, strengthened with progresses in site investigation techniques, cannot be exclusively based on the advancement of geotechnical classification frameworks and the following quantification of rock masses. Temporary support measures for rock masses with equivalent classification values can be diverse. The engineering geology appraisal displayed in this chapter cannot bypass the geological and/or in situ characteristics managing or affecting the tunnel behaviour compared with a regular classification that might miss the specifics and particularities of and around a tunnel segment. The likely ground types must be assessed, and after that, combined with the components of the tunnel geometry, the primary in situ stresses and the groundwater regime, the possible failure modes must be considered. These classified behaviour modes, followed by the appropriate mechanical properties that are required for sound tunnel design, are the premise for the numerical design of the appropriate primary support measures to achieve stable tunnel conditions.

There has been a serious effort to develop guidelines and procedures for tunnel design in which the observation of rock mass behaviour is incorporated in

the determination of excavation and support classes [17–21]. The first step of this methodology involves the definition of the possible rock mass type, the second step involves the evaluation of the rock mass behaviour in tunnelling, the third step suggests the type of tunnel excavation-support system, and the final step is the definition of tunnel length with equal support requirements and the appraisal of time and cost for incorporation in the tender documents.

The design methodology discussed here incorporates the assessment of the tunnel behaviour type in the selection of design parameters and the definition of temporary support measures. A flowchart in **Figure 1**, based on Schubert [20] with modifications, presents this design methodology. As shown in this flowchart, the fundamental link between the rock mass model and the excavation and support classes is the definition of the tunnel behaviour type.

Hence, the contribution of engineering geological information for safe and economical tunnelling cannot be simply presented solely by a rock mass classification value (e.g. RMR, Q, GSI, or others). A classification rating, if used, must be accompanied by an understanding of the actual rock mass behaviour in tunnelling [22]. The tunnel behaviour may vary from one rock mass to another, indeed on the off chance that they have the same classification rating within the same stress field and the same groundwater conditions. An illustration of two different ground types with the same classification value but distinctive tunnel failure mode is displayed in **Figure 2** [22]. The two frameworks in **Figure 2** outline that the choice of the immediate support measures cannot be based solely on a classification rating (either GSI or RMR or Q) but that it moreover requires an understanding of the tunnel failure type.

Attention, therefore, should be given to the evaluation of the failure mechanism that 'fits' the ground type after its excavation. For instance, it is clear that in the process of design, the structure of the rock mass must be considered together with the classification index. Taking after the assessment of the failure mechanism, one can be more certain either in utilising the rating of the associated classification value or in deciding the particular geological or in situ characteristics—'keys' that oversee the tunnel behaviour of the ground type. This procedure assists the designer in the analysis of tunnel behaviour and the selection of support measures and in the establishment of the contractual documents and guidelines for the construction.

After the appraisal of the tunnel failure mode, the appropriate numerical modelling can be performed, the conditions can be more soundly analysed, and the principles of tunnel support can be more precisely considered. The appropriate and critical design parameters can also be chosen in connection with the standards of the failure mode. If the behaviour of the rock mass can be considered isotropic and governed by induced stress, the tunnel engineer must be focused on the rock mass parameters (e.g. GSI in the Hoek-Brown transfer equations relating intact and rock mass properties with respect to the GSI) [3, 4]. On the other hand, in the event that the main failure mode is gravity-induced instability, the practitioner must focus on parameters related to the joints. In the event that the rock mass is weak but moreover anisotropic (e.g. due to schistosity or well-defined bedding planes), both the rock mass parameters and the persistent discontinuity properties must be considered [23]. Being that most tunnel designs presently incorporate numerical analysis, the issue is whether to utilise rock mass parameters (such as shear strength of the rock mass, c_{mass}, φ_{mass} and E_{mass}) when the rock mass behaves isotopically or to incorporate the joint parameters (orientation, distribution, persistence, shear strength c_{joint} and φ_{joint}) when the behaviour is controlled by the discontinuities or impacted by the resulting anisotropy.

Recent research regarding weak rock masses and their engineering geological behaviour, as well as the experience gained by the recent tunnelling projects in the Greek mountains, offered sound and adequate information for the investigation of
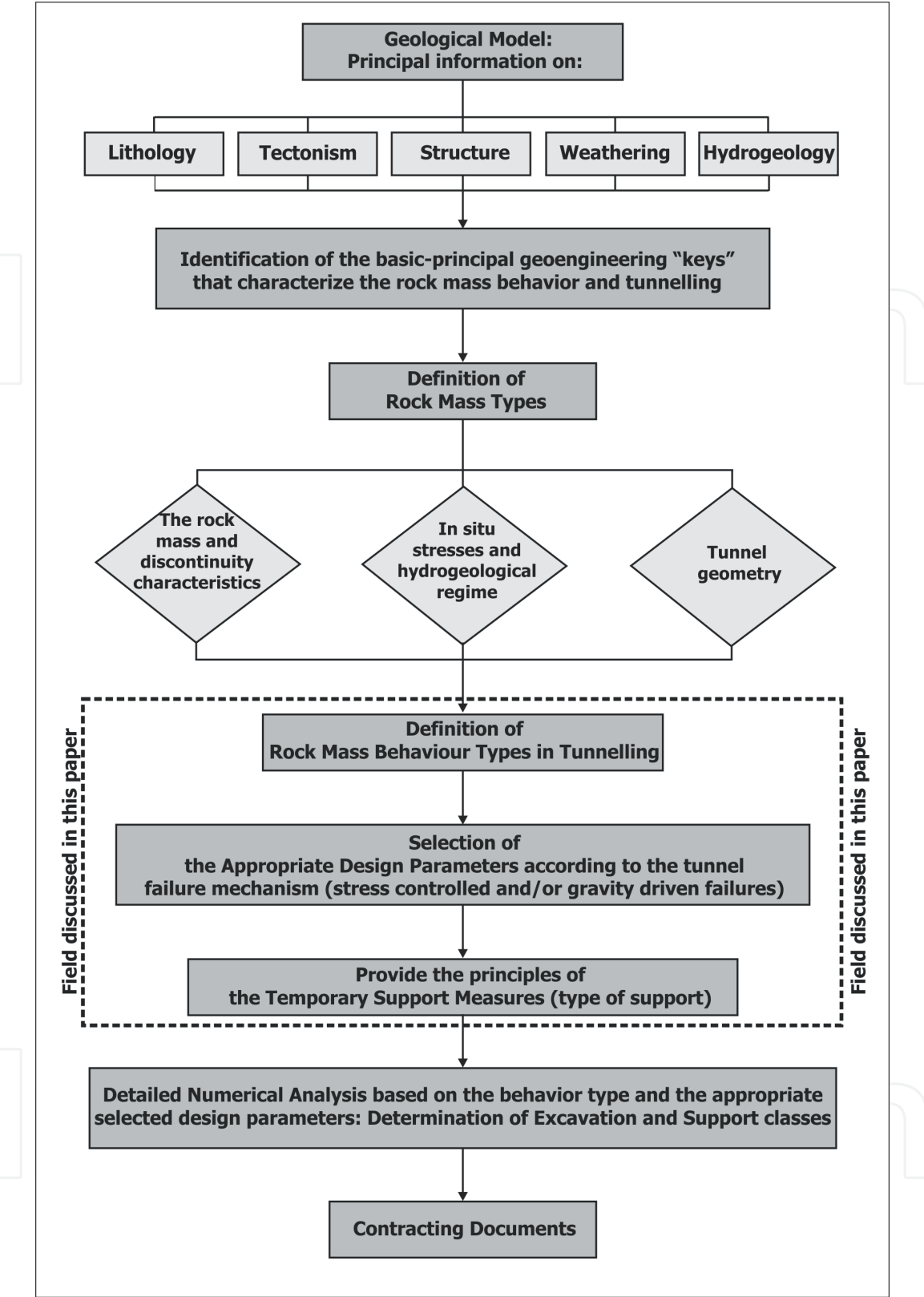
Figure 1.
Flowchart of the design procedure for tunnelling using conventional drill and blast excavation from the geological model to the definition of rock ground types and from the appreciation of the tunnel behaviour modes to the numerical design. Based on Schubert [21] and modified by the author [22].

the impact of these conditions on the behaviour of the geological material, as well 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 correlate this material, a database was built, i.e. 'Tunnel Data Examination System' (TIAS), which was outlined and created for 62 bored tunnels within the Greek region along
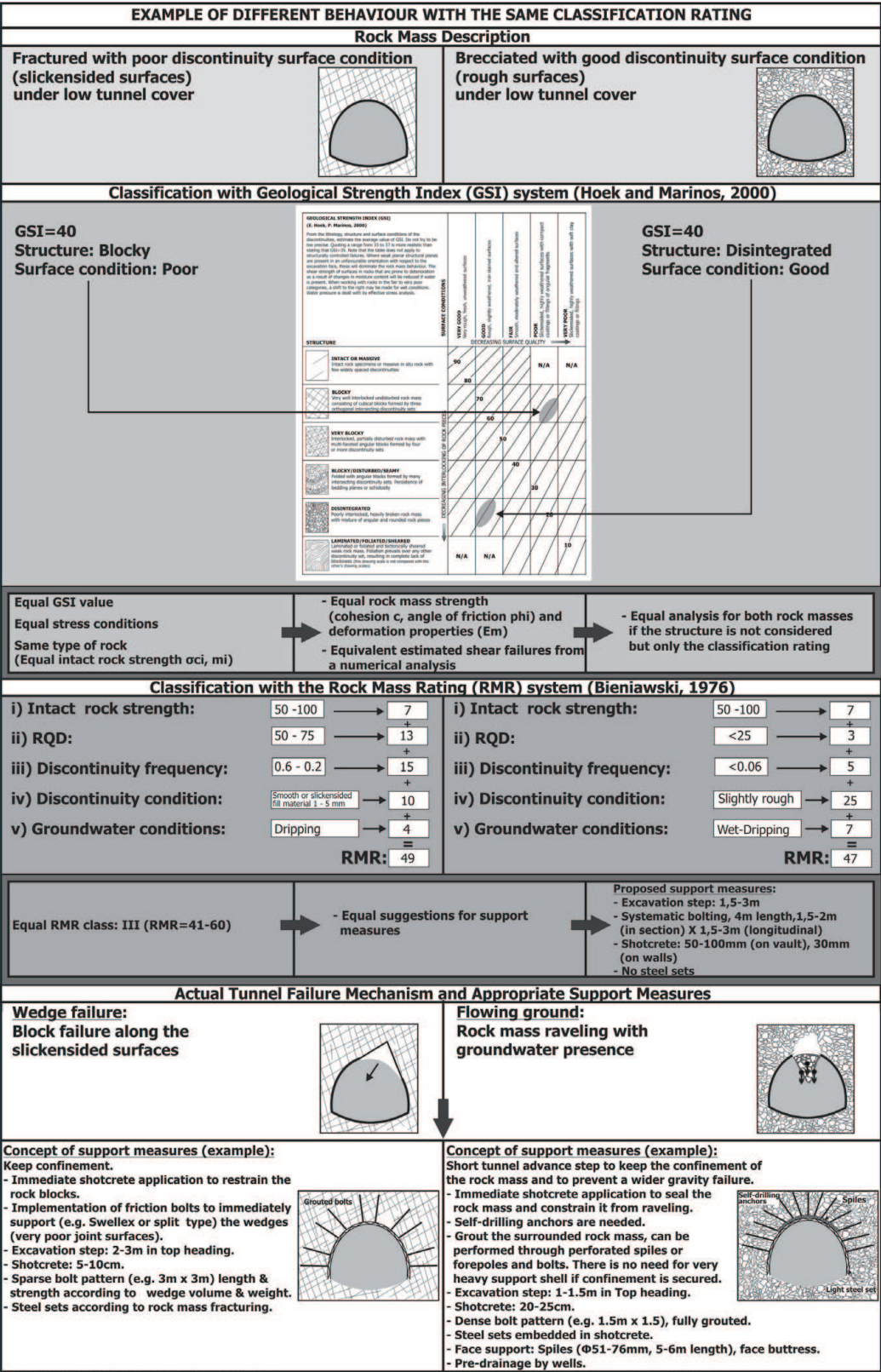
Figure 2.
Example of two equally rated rock masses with the GSI or RMR system but with completely different behaviours in tunnelling [22]. The selection of the temporary support measures should not be based only on the classification ratings but also on the understanding of the tunnel failure mechanism, which is greatly dependent on the rock mass structure.

the Egnatia Highway [25] mainly spanned in Alpine mountain ridges under difficult geological conditions in weak rock masses, excavated with conventional methods, in the concept of top heading and bench excavation. This database is built to 'relate' all available data through all the phases of a tunnel project and premises deep knowledge from the geological and geotechnical investigation to the final design

and construction. The data processed by TIAS came from a variety of sources such as geological mapping, boreholes, laboratory and in situ testing, geotechnical classifications, engineering geological behaviour, groundwater, design parameters, information concerning immediate support measures, construction records and cost. The scope of the system is to provide a tool for the evaluation of anticipated and encountered geological and geotechnical conditions, the evaluation of geotechnical classification and design methods and the relations regarding rock mass conditions and behaviour and immediate support methods and types.

The variety of geological formations under different in situ stress conditions, not only in both mildly and heavily tectonised rock masses but also in altered and/or weathered rock masses, provided a significant amount of information regarding the engineering geological behaviour of several rock mass types. The general geological and engineering geological characteristics and the behaviour in tunnelling of specific rock masses, such as heterogeneous rock masses of flysch and molassic formations (tectonically undisturbed heterogeneous sediments) as well as sound, disturbed and altered ophiolites, are briefly presented in the next paragraphs as examples.

2. Tunnel behaviour appraisal

Engineers can analyse reinforced concrete or steel structures utilising certain checks for a particularly predefined failure mechanism. Particularly, analysis is performed against the bending moment, axial force, shear, penetration and deflection (serviceability limit state). In the case of tunnelling, there is no particular methodology to check against a predefined failure mode.

It is pointed out that the primary step is not to begin performing various calculations but to characterise the potential tunnel behaviour modes. After the evaluation of the ground behaviour in tunnelling, the analysis of the temporary support system can be utilised, in two stages: the choice of the appropriate support elements and their detailed analysis. The selection of support measures should be established equally on experience and geotechnical data and on the analytical solutions but must be confirmed or re-evaluated during construction, supported by the monitoring of the tunnel.

Rock mass behaviour evaluation in tunnelling and its relationship with the design process have been significantly researched. Goricki et al. [18], Schubert [20], Potsch et al. [26] and Poschl and Kleberger [19] studied rock mass behaviour from the design and construction experiences of Alpine tunnels and Palmstrom and Stille [27] from other tunnels.

2.1 Tunnel behaviour types

The term 'failure mechanism-behaviour type', as alluded here, includes all the components that endanger the tunnel segment when the ground has not yet been supported after excavation.

This paragraph presents the tunnel failure modes as they have been designated by Terzaghi [28] and Schubert [21] and also suggested by the author from the tunnel experience of 62 designed and constructed tunnels along Egnatia Motorway and from other cases in Greece. The tunnel failure modes, apart from stable (St) conditions, are separated into gravity-driven failures (wedge and chimney-type failures and ravelling and ravelling ground) and stress-driven failures (failures, squeezing and swelling, anisotropic deformations and brittle failures). The limits and ranges where each behaviour type is connected are briefly depicted and
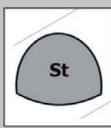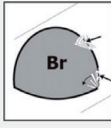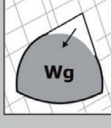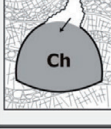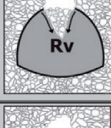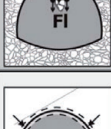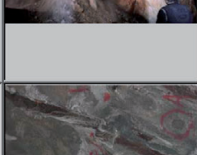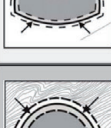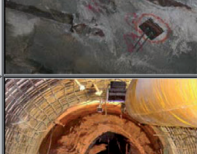
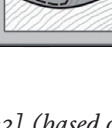|		TUNNEL BEHAVIOUR TYPES		
St	**Stable ground:** Stable tunnel section with local gravity failures. Rock mass is compact with limited and isolated discontinuities			
Br	**Brittle failure:** Brittle failure or rock bursting at great depths			
Wg	**Wedge failure:** Wedge sliding or gravity driven failures. Insignificant strains. The rock mass is blocky to very blocky, blocks can fall or slide. The stability is controlled by the geometrical and mechanical characteristics of the discontinuities. The ratio of rock mass strength to the in situ stress (σ_{cm}/p_o) is high (>0.6-0.7) and there are very small strains ($\varepsilon<1\%$)			
Ch	**Chimney type failure:** Rock mass is highly fractured, maintaining most of the time its structure (or at least that of the surrounded rock mass). Rock mass does not have good interlocking (open structure) and in combination with low confinement (lateral stress) can tend to block falls which develop to larger overbreaks of chimney type. The overbreaks may be stopped and "bridged" by better quality rock masses, depending on the in situ conditions. This type may be applied also in cases of brecciated and disintegrated rock mass in ground with high confinement (high lateral stress)			
Rv	**Ravelling ground:** The rock mass is brecciated and disintegrated or foliated with practically zero cohesion and depending on the intact rock interlocking (Rv1 case: without infilling) and possible secondary hosted geomaterial, (Rv2 case: with infilling, e.g. clay), rock mass can generate immediate rock mass ravelling in face and tunnel perimeter. The difference with Ch type lies in the block size, which is very small here, the self support timing, which is very limited here and the failure extension, where it is unrestricted due to the lack of better rock mass quality in the surrounding zone			
Fl	**Flowing ground:** The rock mass is disintegrated with practically zero cohesion and intense groundwater presence along the discontinuities. Rock fragments flow with water inside the tunnel			
Sh	**Shear failure:** Minor to medium strains, with the development of shear failures close to the perimeter around the tunnel. Rock mass is characterized by low strength intact rocks ($\sigma_{ci}<15MPa$) while the rock mass structure reduces the overall the rock mass strength. Strains develop either at a small to medium tunnel cover (around 50-70m) in case of poor sheared rock masses, or in larger cover in case of better quality rock masses. The ratio of rock mass strength to the in situ stress (σ_{cm}/p_o) is low ($0.3<\sigma_{cm}/p_o<0.45$) and strains are measured or expected to be medium (1-2.5 %)			
Sq	**Squeezing ground:** Large strains, due to overstressing with the development of shear failures in an extended zone around the tunnel. Rock mass consists of low strength intact rocks while the rock mass structure reduces the overall rock mass strength. The ratio of rock mass strength to the in situ stress (σ_{cm}/p_o) is very low ($\sigma_{cm}/p_o<0.3$) and strains are measured or expected to be >2.5%, and they can be also take place at the face			
Sw	**Swelling ground:** Rock mass contains a significant amount of swelling minerals (montmorillonite, smectite, anhydrite) which swell and deform in the presence of groundwater. Swelling often occurs in the tunnel floor when the support ring is not fully closed			
San	**Anisotropic strains:** The rock mass is stratified or schistose or consists of specific weak zones and develops increased strain characteristics along a direction defined by the schistosity.			

Figure 3.
Brief descriptions and schematic presentations of tunnel behaviour types [22] (based on data from Schubert [21], Terzaghi [28] and from the author). Photos from the author except for 'Sq' from E. Hoek (personal communication) and for 'San' from Seingre [29].

appeared in **Figure 3**. The failure modes are assembled based on the examination of tens of rock mass types, their rock mass and joint quality properties and their actual behaviour below different stress conditions (from 30 m to 500 m overburden).

Stress-driven failures: The advancement of critical strains around a tunnel is characterised by the ratio of σ_{cm}/p_o [30]. Specifically, when σ_{cm}/p_o is between 0.3 and 0.6, shear failures can develop in a shallow zone around the tunnel perimeter (Sh failure mode). Such cases include rock masses with poor to very poor fabric and low intact rock strength (< 10–15 MPa) under medium overburden or with

more competent structure and low intact rock strength below high tunnel cover. Squeezing conditions (Sq failure mode) with severe tunnel strains can be induced when $\sigma_{cm}/p_o < 0.3$.

Gravity-driven failures: They are generally differentiated with relation to the rock mass fabric (original conditions and tectonic deformation) and to the conditions of being kept in confinement or not. These gravity-controlled failures occur in rock masses that are clearly characterised by the joints. When the rock masses are just excavated, wedges may fall or slide, depending on the tunnel geometry, the orientation and the shear strength characteristics of the discontinuity planes. Wedge (Wg), chimney (Ch) or ravelling (Rv) failure types can take place in rock masses with poor interlocking of rock blocks due to fracturing degree and/or low confinement. The rock mass cannot arch after the falling, and the crown failure may be significant and irregular. The volume and recurrence of these sorts of tunnel behaviour depend on the structure of the rock mass ('blocky-disturbed' and 'disintegrated'), its relaxation ('open structure') and the tunnel cover/lateral confinement conditions. With an increase in the depth of the tunnel, the rock mass quality is generally improved, and the confinement pressure 'tightens' the structure of the mass.

Of course, there are cases where both stress and gravity-driven failures can be met in a rock mass. In such cases, particular consideration ought to be given to the principal failure mode for the choice of suitable support measures.

A tunnel behaviour chart (TBC) [22], illustrated in **Figure 4**, has been proposed for assessing the rock mass behaviour in tunnelling and covers a wide range of rock mass conditions. This assessment is based on the structure of the rock mass, the strength of the intact rock and the overburden depth.

This classification frame, the TBC, joins the rock mass characteristics straightforwardly with the design and the tunnel support standards and covers a wide extent of conditions. The TBC could be a classification for the estimation of tunnel behaviour and requires three parameters: the rock mass structure, the overburden (H) and the intact strength of the rock (σ_{ci}). This is an integrated classification based on the TIAS database and the data from the design and construction of 62 tunnels in Greece [25]. The purpose of this chart is to foresee the basic failure modes of several rock mass qualities and conditions. The cases that were investigated elaborated intact rock strengths up to 100 MPa and depths not more than 500 m, while many tunnels were less than 300 m deep. It is noted that the values of the uniaxial compressive strength of the intact rock (σ_{ci}) and the overburden thickness (H) utilised within the chart are reasonable trends but should only be considered as indicative.

This chart can be applied in a wide range of geological and geotechnical conditions, since numerous geological formations with various tectonic, weathering and alteration intensity, commonly found worldwide, have been excavated and effectively supported, under a large range of tunnel covers (up to 500 m). The chart does not refer to very high overburden (e.g. many hundreds of m or > 1000 m) and very large intact rock strengths, where brittle failures (spalling or rock burst) can be developed. Hence, TBC can be really useful in any mountain formations in a tunnel excavated with the conventional principles within this wide application range.

The rock mass structure is an essential parameter to appraise its prompt reaction in underground opening in the TBC chart. From the structure of the rock mass, one can 'read' the tectonic disturbance, the blockiness of the mass, the probable size of blocks, the shape of rock elements (massive, blocky, foliated or sheared) or the ability of the rock blocks to rotate. Rock mass fabrics were categorised after the GSI system [8].

For gravity-controlled failures, the tunnel depth impacts the degree of a failure since the degree of interlocking between the rock blocks changes and the confinement
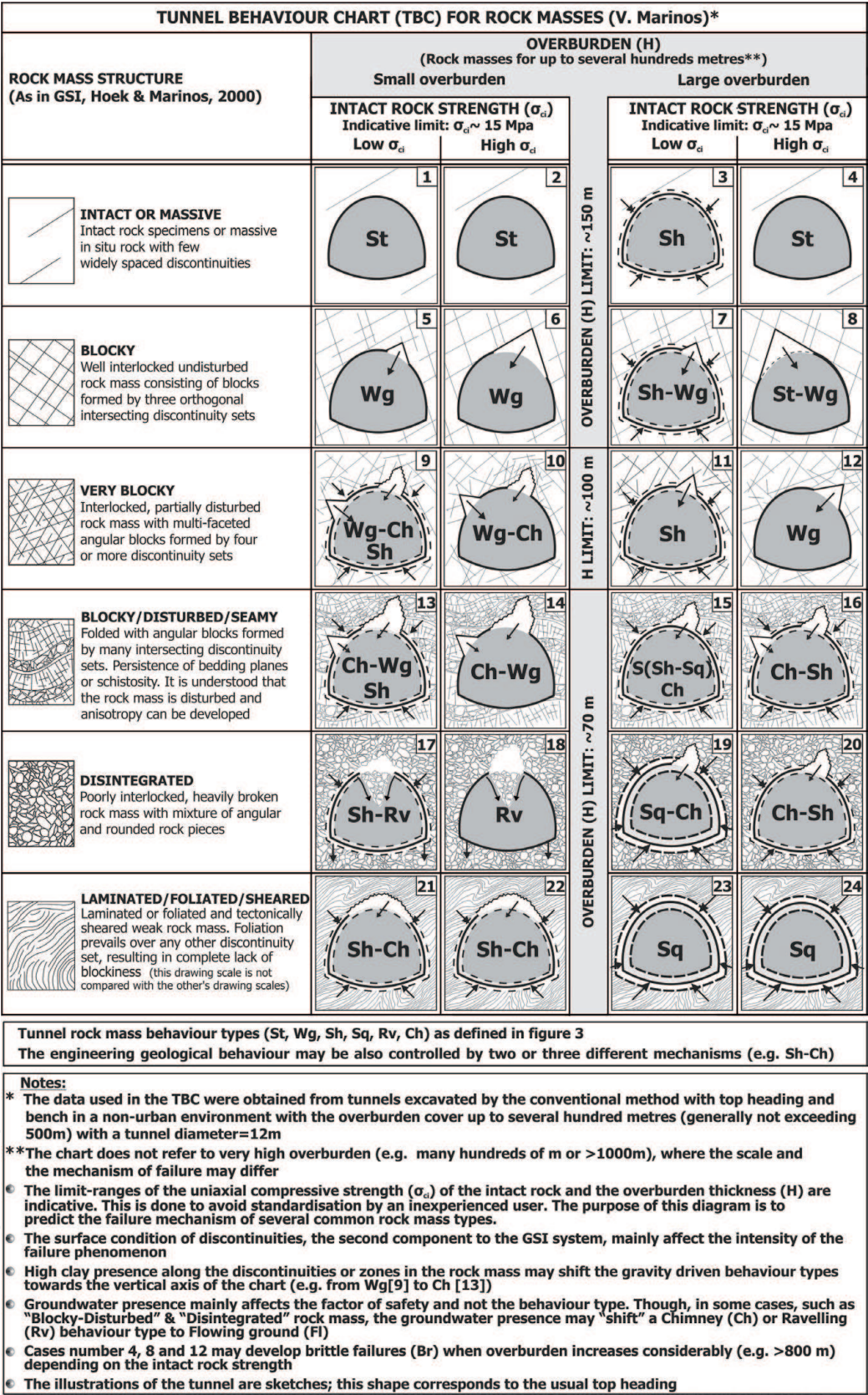
Figure 4.
Tunnel behaviour chart: an assessment of rock mass behaviour in tunnelling [22].

stress varies with depth. For instance, the rock mass may ravel (Rv) near the ground surface, but under higher overburden, a chimney-type (Ch) failure may be developed.

As far as the stress-driven modes are concerned, tunnel cover H characterises when shear failures and deformations are formed. These limits are appraised in the following manner: 150 m for competent structure (intact and blocky-seamy undisturbed),

100 m for very blocky structure and 70 m for the very poor fabric (seamy-disturbed, disintegrated or laminated-sheared). These values are basically evaluated by back analysis and by the calculated values of the ratio σ_{cm}/p_o, with p_o, the in situ stress, considered isotropic. For example, in the event that $\sigma_{cm}/p_o < 0.3$, squeezing conditions are likely; in case $0.3 <$ squeezing <0.6, minor to medium strains may happen; and on the off chance that $\sigma_{cm}/p_o > 0.6$, minor or no deformations are expected.

The limit of intact rock strength (σ_{ci}), i.e. 'low' vs. 'high', considered to characterise the rock mass behaviour in tunnelling, in the TBC chart, is based on the value when shear failures and deformations initiate. This limit is assessed at 15 MPa. The σ_{ci} values that were analysed in the design of the investigated tunnels are extended between 5 and 100 MPa.

3. Examples of engineering geological appreciation and behaviour in tunnelling of particular rock masses

The general geological and engineering geological characteristics and the behaviour in tunnelling of specific rock masses, such as heterogeneous rock masses of flysch (tectonically disturbed heterogeneous formation) and molassic formations (non-tectonically disturbed heterogeneous sediments) as well as sound, disturbed and altered ophiolites, are briefly presented in the next paragraphs as examples. For more details on each of the specific rock types, their engineering geological characteristics, their specific GSI characterisation and their tunnel behaviour, the interested reader is referred to the original publications presenting the individual charts [9–13, 15, 22, 23, 30, 31].

3.1 Flysch formations: tectonically disturbed heterogeneous rock masses

Due to the generally poor characteristics and uncertainties with respect to its geotechnical characterisation, flysch frequently causes problems or challenges towards the design and construction of engineering projects. Flysch is composed of variable alternations of clastic sediments that are related with orogenesis, since it closes the cycle of sedimentation prior to the paroxysm folding process. It is characterised basically by rhythmic alternations of sandstone and pelitic layers (siltstones and silty or clayey shales). The thickness of the sandstone or siltstone beds ranges from centimetres to metres. Conglomerate beds may also be included. Heavy folding and highly shearing with various overthrusts characterize the environment in areas of flysch formations. The main thrust movement is associated with smaller satellite reverse faults within the thrusted body. The overall rock mass is profoundly heterogeneous and anisotropic and moreover may be influenced by extensional faulting creating mylonites. The structural deformation due to tectonism radically debases the quality of the rock mass. Hence, flysch rock mass types are associated with undisturbed, fractured, heavily sheared or even chaotic structures. Such flysch qualities are classified into 11 rock mass types (I to XI) [11] according to the siltstone-sandstone proportion and their tectonic disturbance (**Figure 5**).

The design of tunnels in poor rock masses such as folded and sheared flysch presents a major challenge to geologists and engineers. The complex structure of these materials, coming about from their depositional and structural history, implies that they cannot effectively be classified in terms of the broadly used characterisation systems. The range of geological conditions under varied in situ stresses, in both mild and heavy tectonism investigated here, offered valuable data with respect to the engineering geological conditions and geotechnical behaviour of several flysch rock mass types.

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.			

Figure 5.
Rock mass types in tectonically disturbed heterogeneous formations such as flysch [11].

A classification of flysch rock masses depending on their geotechnical behaviour (strain due to overstressing, overbreaks or wedge failure, 'chimney' type failure, ravelling and their corresponding scale) is displayed from now on. Depending on its type, flysch can show a range of behaviours: be stable even under significant over-burden, display wedge sliding and more extensive chimney type-crown failures, or show large deformations even under low to medium overburden.

In a general sense, the behaviour of flysch arrangements amid tunnelling depends on three major parameters: (i) the structure, (ii) the intact strength of the governing rock type and (iii) the depth of the tunnel. The anticipated behaviour types (stable, wedge failure, chimney type failure, ravelling ground, shear failures and squeezing ground) can be outlined within the tunnel behaviour chart [22]. A detailed introduction of the range of geotechnical behaviours in tunnelling for each flysch rock mass type (I–IX), which is based on engineering geological characteristics, is displayed in **Figure 6**.

The rock mass is frequently taken as a 'mean isotropic geomaterial', in the case that rock mass properties are quantified through a classification system. This presumption is normally accepted in conditions of a uniformly jointed, highly tectonised rock mass without persistent joints of certain unfavourable orientation. This condition can be quite true for types VII to IX. Where bedded rock masses are involved, at a scale of the tunnel segment, the engineering geological behaviour during tunnel construction is controlled by the properties of the bedding planes. This case may be applied to the flysch rock mass type III to VI.

A reliable first estimate of potential problems of tunnel strain can be given by the ratio of the rock mass strength to the in situ stress σ_{cm}/p_o [30]. This is usually followed by a detailed numerical analysis of the tunnel's response to sequential excavation and support stages. Minor squeezing (1–2.5%) can be developed in the very poor flysch rock mass types X and XI from 50 to 100 m tunnel cover, while severe (2.5–5.0%) to very severe squeezing (5–10%) can be developed 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.

Regarding the rheological characteristics of flysch formations, the creep potential of sandstone formations is considered to be negligible. However, in the case of tunnel excavation in siltstone or shale formations, especially under high overburden, time-dependent displacements or loads may be developed.
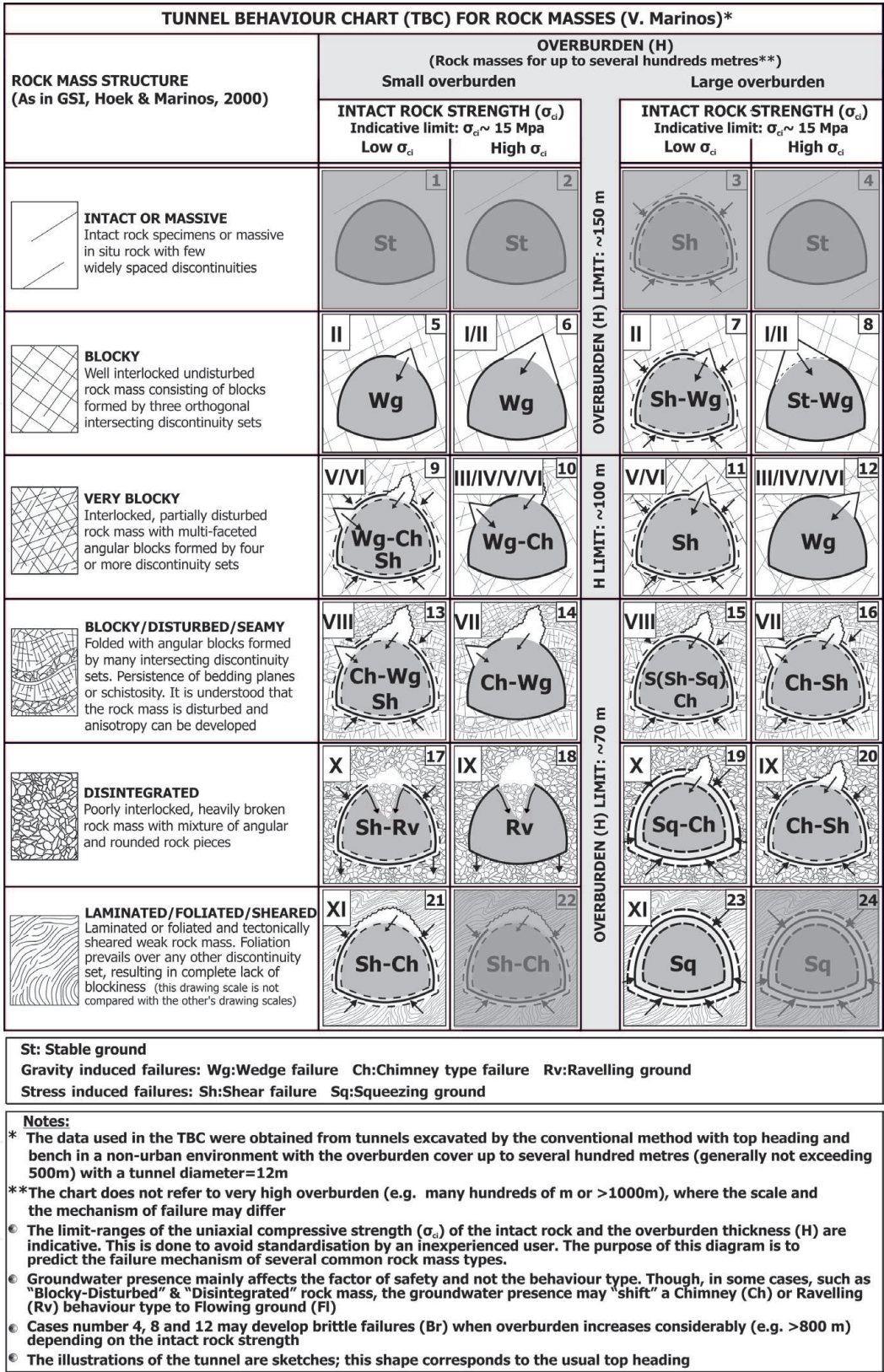
TUNNEL BEHAVIOUR CHART (TBC) FOR ROCK MASSES (V. Marinos)*								
ROCK MASS STRUCTURE (As in GSI, Hoek & Marinos, 2000)	**OVERBURDEN (H)** (Rock masses for up to several hundreds metres**)							
	Small overburden				Large overburden			
	INTACT ROCK STRENGTH (σ_ci) Indicative limit: σ_ci ~ 15 Mpa				INTACT ROCK STRENGTH (σ_ci) Indicative limit: σ_ci ~ 15 Mpa			
	Low σ_ci		High σ_ci		Low σ_ci		High σ_ci	
INTACT OR MASSIVE Intact rock specimens or massive in situ rock with few widely spaced discontinuities	1 St		2 St		3 Sh		4 St	
BLOCKY Well interlocked undisturbed rock mass consisting of blocks formed by three orthogonal intersecting discontinuity sets	II 5 Wg		I/II 6 Wg		II 7 Sh-Wg		I/II 8 St-Wg	
VERY BLOCKY Interlocked, partially disturbed rock mass with multi-faceted angular blocks formed by four or more discontinuity sets	V/VI 9 Wg-Ch Sh		III/IV/V/VI 10 Wg-Ch		V/VI 11 Sh		III/IV/V/VI 12 Wg	
BLOCKY/DISTURBED/SEAMY Folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity. It is understood that the rock mass is disturbed and anisotropy can be developed	VIII 13 Ch-Wg Sh		VII 14 Ch-Wg		VIII 15 S(Sh-Sq) Ch		VII 16 Ch-Sh	
DISINTEGRATED Poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces	X 17 Sh-Rv		IX 18 Rv		X 19 Sq-Ch		IX 20 Ch-Sh	
LAMINATED/FOLIATED/SHEARED Laminated or foliated and tectonically sheared weak rock mass. Foliation prevails over any other discontinuity set, resulting in complete lack of blockiness (this drawing scale is not compared with the other's drawing scales)	XI 21 Sh-Ch		22 Sh-Ch		XI 23 Sq		24 Sq	

OVERBURDEN (H) LIMIT: ~150 m
H LIMIT: ~100 m
H LIMIT: ~70 m
OVERBURDEN (H) LIMIT: ~70 m

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

Figure 6.
Tunnel behaviour chart with projections of the principal failure mechanisms for the rock mass types of flysch (I–XI) [31].

The influence of groundwater on the rock mass behaviour in tunnelling is very important and has to be taken into great consideration in the estimation of potential tunnelling problems. The most basic impact of groundwater is on the mechanical properties of the intact rock components, particularly on shales and siltstones that are susceptible to changes in moisture content.

The evaluation of tunnel behaviour and the conceptual assessment of the support measures must be also based on detailed ground characterisation. This detailed characterisation cannot bypass the geological and/or in situ characteristics managing or affecting the tunnel behaviour compared with a standardised classification. This classification, named 'Ground Characterization, Behaviour and Support for Tunnels' [22], urge the practitioner to assess the information in detail, to appraise the tunnel behaviour and to select the appropriate design parameters and the suitable support measures. An illustration of this characterisation in tunnelling through tectonically disturbed flysch type is displayed in **Figure 7** [31].

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 [X] 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>10⁻³m/sec) [ ] Low (k:10⁻⁷-10⁻⁵m/sec) [ ] Practically impermeable (k<10⁻⁹m/sec) [ ]
  Medium(k:10⁻⁵-10⁻³m/sec) [ ] Very low (k:10⁻⁹-10⁻⁷m/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 (P₀=γH_min to γH_max): p₀: *5* MPa

c) Stress field particularities
- Particular presence of lateral pressures (k₀):

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 significantly contribute to the weighted "intact rock"*
- Rock mass strength to insitu stress ratio(σcm/p₀): σcm/p₀>0.6 [ ] 0.3<σcm/p₀<0.6 [ ] σcm/p₀<0.3 [X]
- 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 [X]
- 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 [X] Very poor [X]
- 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*

[X] 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)

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 [X] — *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*

Sq-Ch   Sq-Ch

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) [X] — *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:
- Geometry (Dip/dip direction): J$_1$: / J$_2$: / J$_3$: /
- Persistence: m m m
- Distance apart or Spacing: m m m
- Aperture: mm mm mm
- Filling material:
  - Hard<5mm / Hard<5mm / Hard<5mm
  - Hard>5mm / Hard>5mm / Hard>5mm
  - Soft<5mm / Soft<5mm / Soft<5mm
  - Soft>5mm / Soft>5mm / Soft>5mm
  - None / None / None
- Weathering:
  - Unweathered / Unweathered / Unweathered
  - Slightly / Slightly / Slightly
  - Moderately / Moderately / Moderately
  - Highly / Highly / Highly
  - Decomposed / Decomposed / Decomposed
- Ground water conditions:
  - Dry / Dry / Dry
  - Sub-wet / Sub-wet / Sub-wet
  - Wet / Wet / Wet
  - In drops / In drops / In drops
  - Flow / Flow / Flow
- Joint Roughness Condition (JRC):
- Joint Compression Strength (JCS): MPa MPa MPa

Shear strength properties of discontinuities:
- Friction angle (φ): J$_1$: ° J$_2$: ° J$_3$: °
- Cohesion (c): J$_1$: KPa J$_2$: KPa J$_3$: 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)

(b)

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Figure 7.
(a) Modified example of the ground characterisation, behaviour and support for tunnels illustrated [31], in light characters by an example of tunnelling in a tectonically deformed intensively folded siltstone (flysch rock mass type X) [for page 2 from 2 see (b)].

Apart from a few cases of simple tunnelling conditions in areas of good rock mass types of flysch (sorts I–V), most of the investigated tunnels were excavated under challenging geological conditions (sorts VII–XI). These tunnels have been excavated utilising top heading and bench methods. Particular measures were applied to stabilise the face, such as forepoling or/and establishment of long

grouted fibreglass dowels in the face. Furthermore, immediate shotcreting and face buttressing have been utilised in several combinations for face stabilisation. After the stabilisation of the tunnel face, the application of the immediate support shell, consisting of shotcrete layers, rockbolts and steel sets embedded within the shotcrete in different combinations, was essential to ensure the stability of the tunnel. Elephant's foot and, in uncommon cases, micropiles were utilised to help the establishment of the top heading foundation zone and to secure stability when benching. Temporary and final invert closure was applied to meet the squeezing conditions.

Under severe squeezing, the application of yielding systems is an alternative solution [e.g. in Schubert, 1996, 20]. 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.

3.2 Molassic formations (non-tectonically disturbed heterogeneous rock masses)

The term molasse comes from a Swiss local title at first allotted to soft sandstones related to marls and conglomerates belonging to the tertiary that had an extraordinary advancement within the lowland parts of Switzerland. They are as a result of debris of weathering and erosion of the Alpine mountains. The term is currently used to describe the deposits from the erosion of a mountain belt after the ultimate stage of orogenesis behind the mountain building zone. Molasse comprises of a sequence of tectonically undisturbed sediments of sandstones, conglomerates, siltstones and marls. Molassic rock masses may have exceptionally distinctive structures near to the surface compared to those restricted at depth, where bedding strata do not show up as clearly characterised joint surfaces that separate the rock mass into blocks [12].

Tunnelling through molassic rocks is based on the experience picked up from the design and construction of 12 tunnels along the Egnatia Highway in northern Greece. A context is displayed here concerning the distinctive rock masses of molassic rocks, the geotechnical behaviour of each type in tunnelling and the temporary support philosophy, both for underground construction and portal zones. The major characteristics of the investigated geomaterial that cause its specific tunnel behaviour are (a) the lithological heterogeneity, as the series comprises of a nearly continuous units of sandstones, siltstones, marls or claystones and conglomerates, with alternations of layers from some centimetres to some metres thick; (b) the low to moderate strength of the intact rock of these units; (c) the compact, nearly intact structure at depth, indeed when sandstone strata alternate with siltstones; and (d) the problematic behaviour of the siltstone-marly units near to the surface due to slaking and weathering.

Molassic rocks display noteworthy differences between the surface and at depth. These contrasts lie within the rock mass fabric, weathering and permeability and thus are exceptionally critical for the rock mass quality and behaviour in tunnelling. Molassic rocks, especially sandstone and well-cemented conglomerates, tend to be profoundly frictional. Due to the narrow deformation to which they have been subjected in deposition, the discontinuity in these rocks is by and large free from the impacts of shear development (slickensides).

Siltstone or claystone beds, being restricted shortly beneath the surface, are compact enough to create a nearly unbroken medium. Their presence may, be that as it may, diminish the quality of the whole rock mass, due to its nature. In any case, there are occasions where siltstones are fairly competent and below low stress; their behaviour does not essentially contrast from that of sandstones. The bedding is the basic joint set in a molassic rock mass but is only communicated on and close to the surface.

At depth, the bedding is mostly concealed. For the cases examined in this chapter, rock quality designation (RQD) values close the surface run from 0 to 50%. At low depths (~5 m), the rock masses ended up medium broken and weathered, whereas bedding planes are still apparent. At depths greater than 10–15 m, the rock masses are as a rule homogeneous in structure and continuous, with RQD values >60%.

Weathering usually transforms the rock mass strength. Siltstone (or marly) members are susceptible to weathering, and fissility may be built parallel to the bedding when these rocks are uncovered to the surface or are close to it. Siltstone (or marly) members in outcrops show up thin layered or even schistosed, and when they alternate with sandstones, the appearance of the rock mass takes after that of flysch. This appearance in outcrops can be deceiving when considering the behaviour of molassic rocks in a limited underground environment, in which the slaking is confined and the rock mass is massive. There are conditions where sandstones are loose and may be treated as dense sands. In such poor molasses, clays and silts also present, and the fabric can be treated like a soil. It is not the goal of this chapter to address these soil-like molasses that have constrained spatial dissemination. In any case, it ought to be underlined that molasses close to the surface may make a cover with such soil characteristics.

Based on the outcomes of numerous in situ permeability tests (Marinos et al. [9]), the overall permeability of the molassic series is rapidly reduced with depth. Though, the permeability of the sandstone members within the molasse is altogether higher than that of the siltstone ones. Within the case of variations of the two types of rocks, the permeability approaches the value of the siltstone since the siltstone layers do not permit the water flow through the rock mass and decrease the overall permeability. Besides, the frequent horizontal transitions do not allow the development of a uniform aquifer. Fault zones, in spite of the fact that they are more permeable, are neither frequent nor extensive. Thus, in spite of the fact that the water table will ordinarily be over the tunnel, only minor water inflows are expected, in spite of the fact that in a few circumstances it may be essential to relieve water pressures by drilling.

The high strength of the molassic rock mass in relation to the in situ stresses at shallow to medium depths does not qualify stress-driven failures. The prevailing failure mode in tunnels is the gravity-induced falls and slides of rock blocks and wedges characterised by intersecting joints and bedding planes. It ought to be stressed, however, that this behaviour has been confirmed with tunnel construction in depths up to 110 m and should not be reflected for much larger depths.

These types of behaviours are differentiated in two regions (see **Figure 4**):

- Stable (St) within the case of massive structure and shallow to medium tunnel covers. As the tunnel depths increases, stable behaviour with no deformation can be assumed for sandstone- or conglomerate-dominated series (zones #1, #2 and #4 in **Figure 4**).

- Stable (St) with limited strains (Sh), particularly in cases of siltstone-dominated rock mass types, under notable tunnel cover. The size of the resulting deformation depends on the strength of the siltstone and the overburden. Serious strains have not been experienced along the Egnatia tunnels, as the greatest depth was restricted to 110 m (range #3 in **Figure 4**).

- Wedge failure (Wg) in cases of blocky rock masses and shallow to medium depths (ranges #5 and #6 in **Figure 4**). Similar characteristics with minor deformations are witnessed for sandstone or conglomerate formations with the increase in tunnel depth. The developing confinement with depth may result in

less wedge sliding events (St-Wg) (range #8 in **Figure 4**). A marginally differ-ent failure mode can be presented within the case of thin-bedded series with nearly horizontal bedding planes. Failure of rock blocks due to self-weight from the crown section may be occasional and extensive once their base is exposed due to deconfinement which might cause subvertical tension joints. Such unfavourable conditions must be controlled to face systematic crown failures.

- Wedge failure with limited deformation (Sh-Wg), within the case of siltstone governed formations, beneath critical tunnel cover (region #7 in **Figure 4**). The size of the deformation depends on the intact rock strength of the siltstone and the depth of the tunnel.

- Broad wedge failures that can advance into chimney-type failure (Ch-Wg) within the case of weathered and disturbed fabric near to the surface (portal zones or under shallow depths beneath streams or gullies) due to slaking and loosening of the siltstone parts (ranges #13 and #14 in **Figure 4**).

- Repeated wedge failure that can slowly transform into chimney-type failure (Wg-Ch) along the case of exceptionally blocky rock masses due to fault-ing. In the case of siltstone-dominated rock mass types, at medium to large depths, limited deformations (Sh) can be developed (region #13 in **Figure 4**). However, considerable strains have not been experienced within the tunnels of the Egnatia Highway since the greatest depth of the tunnels, within these rock masses, was 110 m.

With regard to water inflows, minor occurrence of water has been met along the 12 tunnel projects, which develop basically within the form of increased moisture to drips. In a few uncommon cases, periodic or continuous low flow at different areas, primarily in sandstone-siltstone contact layers and along major discontinui-ties, has been experienced. However, this presence degrades the characteristics of the discontinuities and ought to be taken under consideration when evaluating the geotechnical characteristics of the rock mass types. The low geotechnical properties of the molassic formations, near to the surface, have driven to numerous failures in the portal areas. These instabilities were not directed by pre-existing discontinui-ties, such as the bedding planes, but they were related to the advancement of a new circular-shaped failure surface across the weathered poor rock mass.

The tunnel support concept in molassic rock mass types must take into consid-eration the rock mass fabric and the expected failure modes in connection to the depth as depicted above. These approaches for the philosophy of temporary support measures have been formed based on the geotechnical behaviour of molassic series as well as on construction data. Along the tunnels of the Egnatia Highway through molassic formations, 54% of the whole length was excavated employing a support category with shotcrete shells, anchors, steel sets and light spilling. A really light support category containing a thin shotcrete shell and a sparse pattern of bolts was received for 38% of the entire length. At long last, a heavy support category with thick shotcrete shells, steel sets, forepoling and fibreglass nails was executed in only 6.5% of the full length tunnel basically within the region of the portals. Hence, absent from the ground surface, where the rock mass is subjected to surface weathering conditions and any credible fault zones, there are two basic types of immediate support systems that could be implemented.

The first type concerns stable conditions with solely gravity-controlled failures and minor to zero deformation. This is often the foremost common case for all molasses at depth and ought to be connected for low to medium overburden or

indeed under higher overburden in cases of sandstone or conglomerate mastery. The immediate support comprises of a thin shotcrete layer and a pattern of rock bolts, while the advance step can be 3–4 m. The primary 3–5-cm-thick layer of shotcrete, implemented, as soon as possible, on the uncovered rock mass surfaces, seals and secures the siltstone layers from slaking. The rock bolt design reinforces the rock mass, keeps it restricted and prevents likely gravity-controlled falls of loose, fundamentally defined blocks or wedges due to decompression of the otherwise sealed bedding planes. The introduction of another layer of shotcrete, strengthened either by wire mesh or by fibres, makes a complementary shell, engaging the heads of the rock bolts and guaranteeing the stability of the tunnel. The next type of support system for competent molasses absent from portals or faults alludes (a) to conditions with frequent wedge failures due to the geometry of major joint systems and the conditions already displayed (horizontal bedding planes) and/or (b) to cases of weak rock (e.g. siltstone) governed by molassic formations below considerable to large overburden. In expansion to the shotcrete and the rock bolts, light steel sets may be necessary, while the advance step must be restricted (around 2 m) to prevent any wedge formation or critical strains in the case of large depths.

For weathered molassic series near to the tunnel portals or intensely jointed and poor molassic rock masses along fault zones, stiffer support is required by the use of heavier steel sets and a thicker shotcrete shell. Consideration ought to be given to limiting disturbance to the encompassing geomaterial by reducing the excavation step (~1 m). Furthermore, it may be essential to stabilise the tunnel face utilising face support measures (e.g. fibreglass nails) or face protection schemes (e.g. spiles or forepole umbrella) to avoid progressive detachment, deconfinement and creation of chimney-type failures.

3.3 Ophiolitic complex

The term ophiolite was at first given to a series of basic and ultrabasic rocks, more or less serpentinised and transformed, appearing within the Alpine chains. Ophiolites are presently considered as pieces of the oceanic crust produced at an oceanic ridge and the upper mantle of an ancient ocean, thrust up on the continental crust during mountain building [32].

The ophiolitic complex is in a general sense characterised by underlying peridotitic rocks that are overlied by gabbroic/peridotitic rocks, which, in turn, are covered by basalts or spilites. The basal peridotites are laminated ('tectonites'). The subsequent alternations of peridotites and gabbros frequently have a layered structure of cumulates and are taken after by enormous gabbros, norites or other basic rocks richer in SiO_2. The overlying basalts are either continuous or within the frame of pillow lavas. In between these rocks, sedimentary rocks of deep sea may be stored. This geometry is exceedingly exasperated since the ophiolitic complexes happen primarily in tectonic zones with superposition of various overthrusts. Metamorphism, which is additionally displayed, changes the initial nature of the materials. The high degree of serpentinisation and the intensity of shearing can make it difficult to distinguish any lattice mineral of either fibrous or laminar shape. This unordinary alteration is a phenomenon of autohydration that occurs amid the final phases of the crystallisation of magma where there's an abundance of water. In other scenarios, serpentinisation compares to a low initial cumulate texture [33].

Serpentinisation is the change of ferromagnesian minerals, specifically olivine, to serpentine—a grade metamorphism of peridotites. In all these cases, the peridotites can be changed into serpentinite. This new rock is initially compact, moderately soft and more naturally sheared by tectonic processes. Serpentinisation can moreover be created due to exogenic conditions with meteoric water under regular

weathering conditions. In this case, the alteration deteriorates the parent peridotite to a schistosed mass and later to clayey soil-like mass. The development at depth of weathered peridotites is less generalised up and clearly restricted compared with the endogenic serpentinisation portrayed already [13].

Rock masses in an ophiolitic complex display a wide variety of engineering behaviour in tunnelling. Typically, this is true due to their petrographic variety and structural complexity. An advanced degree of serpentinisation together with the increased shearing may result in a mass in which it is hard to recognise any initial surface or texture. Thus, behaviour can change from stable to severe squeezing conditions in cases where ophiolites are related with overthrusts. The main rock mass types are peridotites, gabbros, pillow lavas, peridotites that are more or less serpentinised, serpentinites, schisto-serpentinites, sheared serpentinites and chaotic masses in melanges. Peridotites are sound and behave as typical brittle materials. Serpentinisation can be found along the discontinuity surfaces, and the conditions of the joints are significantly reduced to very poor with coatings of 'slippery' minerals such as serpentine or talc. In a disturbed ophiolitic mass, the serpentinisation procedure regularly loosens and disintegrates parts of the rock matrix itself, contributing to lower GSI values and reducing the intact strength [13].

The extraordinary assortment of numerous rock mass types, the unpredictable changes and the alteration mark the ophiolites a formation where great care is required in the tunnel design. This is often true for tunnel projects due to their linearity and their depth that increase the possibility of experiencing the unfavourable zones related with the ophiolites, whereas the uncertainty as to their exact location and extent impairs the difficulty.

In sound and competent rock masses of peridotite, simple and straightforward tunnelling conditions can be anticipated, where consideration has got to be concentrated on maintaining wedge failures. Within the case of a more broken peridotite, schistose or great serpentinite, the behaviour is controlled by sliding and rotation on joint surfaces with generally little failure of the intact rock pieces. In this case, the control of stability can be amended during tunnel excavation by keeping the rock mass confined. In poor quality serpentinite, due to alteration or shearing, blockiness may be totally missing, and clayey areas with swelling materials may be present. Tunnel instability will at that point be due to stress-dependent rock mass failure with severe squeezing at depths [13].

Peridotites: In great quality masses of peridotite, straightforward tunnelling conditions can be anticipated. Consideration must be concentrated on controlling gravity-driven instabilities from wedges. For these failures comprising some joints, the issue is basically one of three-dimensional geometry and stereographic tools or numerical analyses such as UnWedge (see http://www.rocscience.com) ought to be utilised for an investigation of design of support measures.

However, compared with other rock masses of comparative structures, the peridotites in a general sense have smoother joints with poor frictional properties. As clarified previously, it's due to the existence of serpentinised material, which is regularly present even if the serpentinisation has not affected the rock itself. This makes the gravity-driven failures more challenging and for the most part requests heavier rock bolting patterns and/or thicker shotcrete (zones #2, #4, #6 and #8 in **Figure 4** depending on the depth and intact rock strength). In exceptionally hard rock masses at large depths, spalling, slabbing and rockbursting are the mechanisms of failure which will be developed and controlled by brittle fracture propagation in the intact rock with the joints having as it were a minor influence. In these cases, the utilisation of the brittle rock failure models must be considered, such as that proposed by Kaiser et al. [34].

Disturbed peridotites or schistose serpentinites: Within the case of a more disturbed peridotite, schistose or weaker serpentinite, the behaviour is controlled

by sliding and rotation on joint surfaces with generally minor failure of the intact rock fragments (ranges #10, #12, #14 and #16 in **Figure 4** depending on the depth and intact rock strength). In this extent of GSI values, the RQD values can be exceptionally low. This is typical, given the structure of the rock masses, but some of the frictional behaviour of the unaltered fragments of the mass is reserved. In such cases, the control of the stability can be effectively achieved during tunnel excavation by maintaining the rock mass confined.

Sheared serpentinite, squeezing behaviour: In low-quality serpentinite, as a result of alteration or shearing, blockiness may be nearly totally lost, and clayey areas with swelling materials may be available. Tunnel stability will at that point be controlled by stress-dependent rock mass failure with significant squeezing at depths (regions #21 to #24 in **Figure 4** depending on the depth and intact rock strength). In these cases, a detailed numerical analysis must be performed that permits progressive failure and support interaction analysis to be demonstrated. In any case, it is exceptionally instructive to carry out a closed form analysis of the tunnel behaviour to get an indication of the significance and value of deformation [13]. The 'strain' can be evaluated from the proportion of the rock mass strength to the in situ stress [30]. This plot is valid to single circular-shaped tunnels.

4. Conclusions

In general, the application of well-known classification systems has the drawback of not necessarily displaying information concerning rock mass behaviour in tunnels. Consequently, there are many cases in which the geological 'identity' of the geomaterial is lost since it is not involved in the analysis, and in that way, it is possible that its special characteristics are mislaid. Despite the capabilities offered by the rapid advance of the numerical tools in the geotechnical design, the outcomes can still include uncertainties when parameters are utilised straightforwardly without considering the real failure mode of the rock mass in tunnelling. This chapter points out that the assessment of the principle tunnel failure mode is an essential information for the temporary support measure definition. The work presented in this chapter was based on a large set of data, incorporated in a tunnel information and analysis system (TIAS), from the design and construction of 62 tunnels through a wide variety of geological conditions.

Two classifications and characterisation schemes have been presented to assess tunnel behaviour based on the engineering geological identity of the rock masses. The primary, called the tunnel behaviour chart, is a classification framework for predicting the rock mass behaviour in tunnelling and covers a wide extent of rock mass conditions. This evaluation is based on the fabric of the rock mass, the strength of the intact rock and the tunnel cover. The moment, called Ground Characterisation, Behaviour and Support for Tunnels, is a step-by step appreciation of a rock mass quality, with detailed engineering geological and geotechnical characteristics, towards the evaluation of the foremost tunnel behaviour and its support requirements.

After defining the most possible failure types for every kind of the predicted rock mass, the most appropriate design parameters are identified, either of the rock mass, if it displays an isotropical behaviour, or characteristics of discontinuities if it behaves in an anisotropic manner. These proposals allow an early assessment of the principles for the choice of appropriate support measures and their basic dimensioning, as dictated by the ground behaviour and the associated mode of failure. The accuracy of the classifications and the support system can be managed directly from direct tunnel observation and monitoring.

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References

[1] Bieniawski ZT. Rock mass classification in rock engineering. In: Bieniawski ZT, editor. Exploration for Rock Engineering, Proc. of the Symp. Vol. 1. Cape Town, Balkema; 1976. pp. 97-106

[2] Barton NR, Lien R, Lunde J. Engineering classification of rock masses for the design of tunnel support. Rock Mechanics. 1974;**6**(4):189-239

[3] Hoek E, Brown ET. Practical estimates of rock mass strength. International Journal of Rock Mechanics and Mining Sciences. 1997;**34**:1165-1186

[4] Hoek E, Brown ET. The Hoek-Brown failure criterion and GSI – 2018 edition. Journal of Rock Mechanics and Geotechnical Engineering. 2018;**11**(3):445-463. DOI: 10.1016/j.jrmge.2018.08.001

[5] Hoek E, Carranza-Torres C, Cokcum B. Hoek–Brown failure criterion-2002 edition. In: Bawden HRW, Curran J, Telsenicki M, editors. Proceedings of the North American Rock Mechanics Society (NARMS-TAC 2002)—Mining Innovation and Technology. Toronto; 2002. pp. 267-273

[6] Hoek E. Strength of rock and rock masses. ISRM News Journal. 1994;**2**(2):4-16

[7] Hoek E, Marinos P, Benissi M. Applicability of the geological strength index (GSI) classification for weak and sheared rock masses—The case of the Athens schist formation. Bulletin of Engineering Geology and the Environment. 1998;**57**(2):151-160

[8] Marinos P, Hoek E. GSI A geologically friendly tool for rock mass strength estimation. In: Proceedings of the International Conference on Geotechnical and Geological Engineering (GeoEng 2000). Lancaster: Technomic Publishers; 2000. pp. 1422-1446

[9] Marinos V, Fortsakis P, Prountzopoulos G, Marinos P. Permeability in flysch— Distribution decrease with depth and grout curtains under dams. Journal of Mountain Science. 2011b;**8**(2):234-238

[10] Marinos P, Hoek E. Estimating the geotechnical properties of a heterogeneous rock mass such as flysch. Bulletin of Engineering Geology and the Environment. 2001;**60**(2):85-92

[11] Marinos V. A revised, geotechnical classification GSI system for tectonically disturbed heterogeneous rock masses, such as flysch. Bulletin of Engineering Geology and the Environment. 2017;**78**:899-912. DOI: 10.1007/s10064-017-1151-z

[12] Hoek E, Marinos P, Marinos V. Characterisation and engineering properties of tectonically undisturbed but lithologically varied sedimentary rock masses. International Journal of Rock Mechanics and Mining Sciences. 2005;**42**(2):277-285

[13] Marinos P, Hoek E, Marinos V. Variability of the engineering properties of rock masses quantified by the geological strength index. The case of ophiolites with special on tunnelling. Bulletin of Engineering Geology and the Environment. 2005;**65**(2):129-142

[14] Marinos V, Carter TG. Maintaining geological reality in application of GSI for design of engineering structures in rock. Journal of Engineering Geology. 2018;**239**:282-297

[15] Marinos V. Geotechnical Classification and Engineering Geological Behaviour of Weak and Complex Rock Masses in Tunnelling

(in Greek), Doctoral Thesis. Athens, Greece: School of Civil Engineering, Geotechnical Engineering Department, National Technical University of Athens (NTUA); 2007

[16] Hoek E, Diederichs MS. Empirical estimation of rock mass modulus. International Journal of Rock Mechanics and Mining Sciences. 2006;**43**(2):203-215

[17] Austrian Society for Geomechanics. Guideline for the Geotechnical Design of Underground Structures with Conventional Excavation. Translated from version 2.1, 29 p, 7-page Appendix; 2010

[18] Goricki W, Schubert G, Riedmueller G. New developments for the design and construction of tunnels in complex rock masses. International Journal of Rock Mechanics and Mining Sciences. 2004;**41**(3):497-498

[19] Poschl I, Kleberger J. Geotechnical risks in rock mass characterization. Tunnels and Tunnelling International. 2004. Part 1—September 2004, 37-9. Part 2—October 2004, 36-8

[20] Schubert W. Dealing with squeezing conditions in alpine tunnels. Rock Mechanics and Rock Engineering. 1996;**29**(3):145-153

[21] Schubert W. Basics and application of the austrian guideline for the geomechanical design of underground structures. In: EUROCK2004 and 53th Geomechanics Colloquium. VGE; 2004

[22] Marinos V. Assessing rock mass behaviour for tunneling. Environmental and Engineering Geoscience. 2012;**18**(4):327-341

[23] Fortsakis P, Nikas K, Marinos V, Marinos P. Anisotropic behaviour of stratified rock masses in tunnelling. Engineering Geology. 2012;**141-142**:74-83

[24] Marinos V, Marinos P, Hoek E. The geological strength index—Applications and limitations. Bulletin of Engineering Geology and the Environment. 2005;**64**(1):55-65

[25] Marinos V, Prountzopoulos G, Fortsakis P, Koumoutsakos D, Papouli D. Tunnel information and analysis system: A geotechnical database for tunnels. Geotechnical and Geological Engineering. 2012;**31**:891-910. DOI: 10.1007/s10706-012-9570

[26] Potsch M, Schubert W, Goricki A, Steidl A. Determination of rock mass behaviour types, a case study. In: EUROCK2004 and 53th Geomechanics Colloquium. VGE; 2004

[27] Palmstrom A, Stille H. Ground behaviour and rock engineering wtools for underground excavations. Tunnelling and Underground Space Technology. 2007;**22**(4):363-376

[28] Terzaghi K. Rock defects and load on tunnel supports. In: Proctor RV, White TL, editors. Introduction to Rock Tunnelling with Steel Supports. Youngtown, Ohio, USA: Commercial Shearing & Stamping Co; 1946

[29] Seingre G. Tunnel de base du Lötschberg - Bilan de l'excavation aux tunneliers. In: Arnould M, Ledru P, editors. GEOLINE 2005, 23-25 May. Lyon, France; 2005. BRGM editions

[30] Hoek E, Marinos P. Predicting tunnel squeezing problems in weak heterogeneous rock masses, part 1: Estimating rock mass strength & part 2: Estimating tunnel squeezing problems. Tunnels and Tunnelling International. 2000;**32**:45-51(Part 1—November 2000); Part 2—December 2000. 34-36

[31] Marinos V. Tunnel behaviour and support associated with the weak rock masses of flysch. Journal of

Rock Mechanics and Geotechnical
Engineering. 2014;**6**:227-239

[32] Debelmas A, Mascle G. Les grandes
structures geologiques. Paris: Dunod;
1997. 312 pp

[33] Skemp A, McCaig M. Origins and
significance of rocks in an imbricate
thrust zone beneath the Pindos
ophiolite, northwestern Greece. In:
Dixon JE, Robertson HF, editors.
Geological Evolution of Eastern
Mediterranean, Special Publication
17. London: Geological Society; 1984.
pp. 569-580

[34] Kaiser PK, Diederichs MS, Martin D,
Sharp J, Steiner W. Underground works
in hard rock tunnelling and mining.
In: Proceedings of the Geo Eng
2000, International Conference
on Geotechnical and Geological
Engineering, Melbourne. Lancaster:
Technomic Publishers; 2000.
pp. 841-926