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ΑΡΘΡΑ

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

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 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.

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.

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).

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 HoekeBrown failure criterion and consequently the GSI value should be used when the rock mass behaves isotropically.

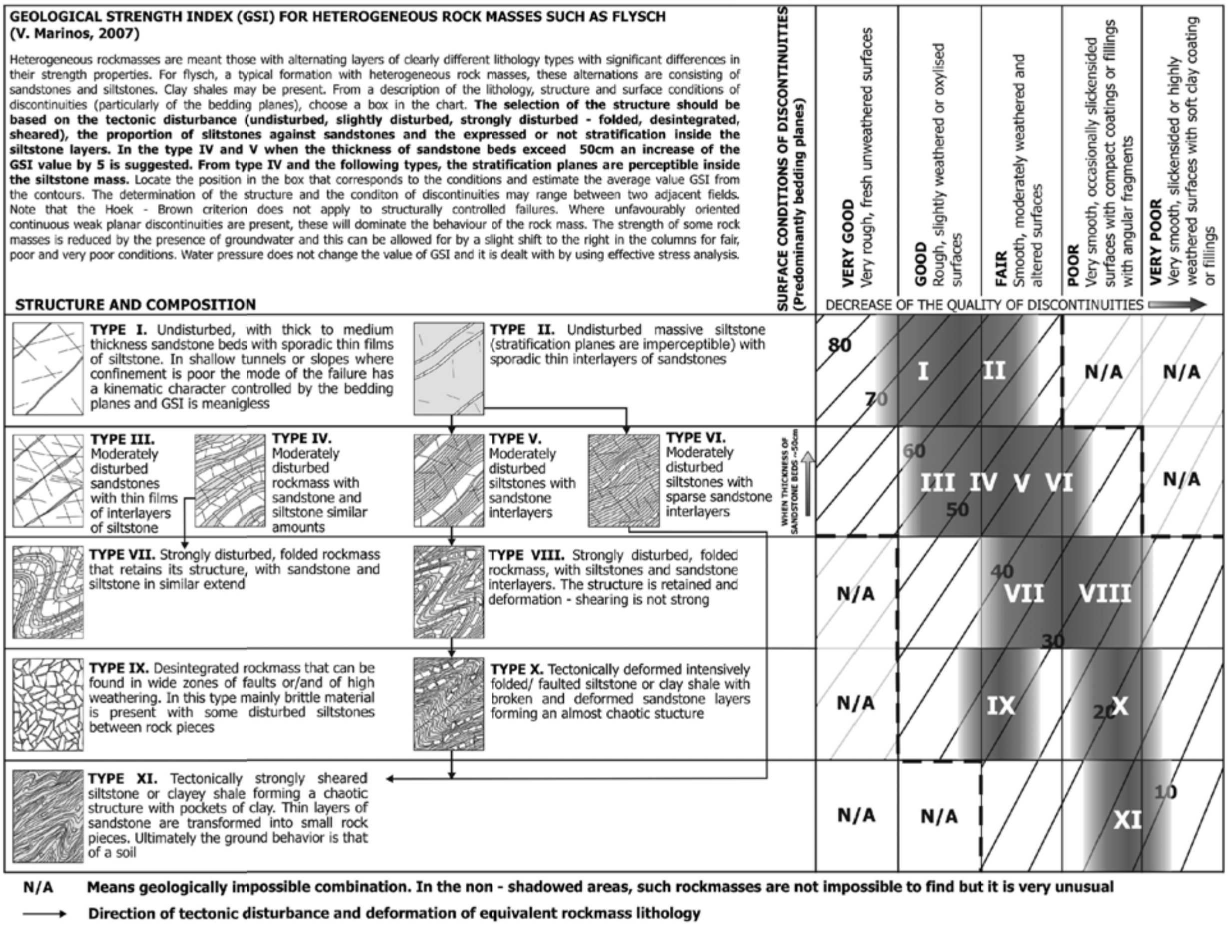

Fig. 3. The new GSI classification chart for heterogeneous rock masses such as flysch (Marinos, 2007; Marinos et al., 2007).

The case in the presence of better quality blocks along with the sheared mass may improve the "overall" rock mass strength, 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 (s_{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 Erm, 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 sitespecific project separately.

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

The higher s_{ci} values are presented in sandstone flysch with a mean value of 45 - 50 MPa. In siltstone flysch, a mean s_{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 s_{ci}, mi 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.

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

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).

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 $4.5x10^{-7}$ 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.

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 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.

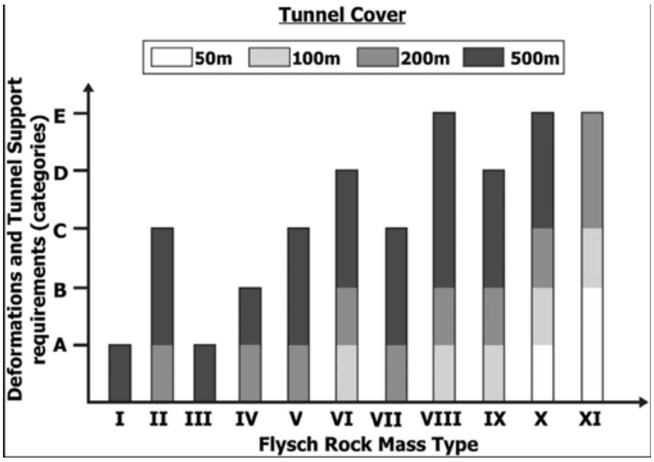

Fig. 6. Deformations and tunnel support requirements for each flysch rock mass type (I - XI) under different overburdens. Strain categories AeE are determined according to Hoek and Marinos (2000) (see Fig. 7.).

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).

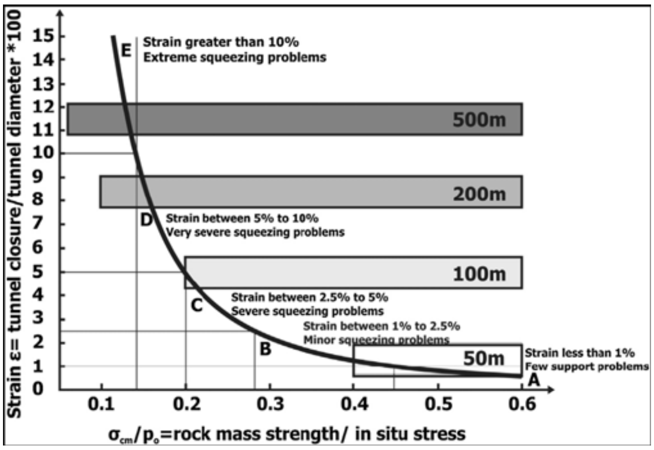

Fig. 7. Strain estimation of the flysch rock mass type X for 4 different tunnel covers categories A - E according to Hoek and Marinos (2000).

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

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

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, 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.

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).

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 select-

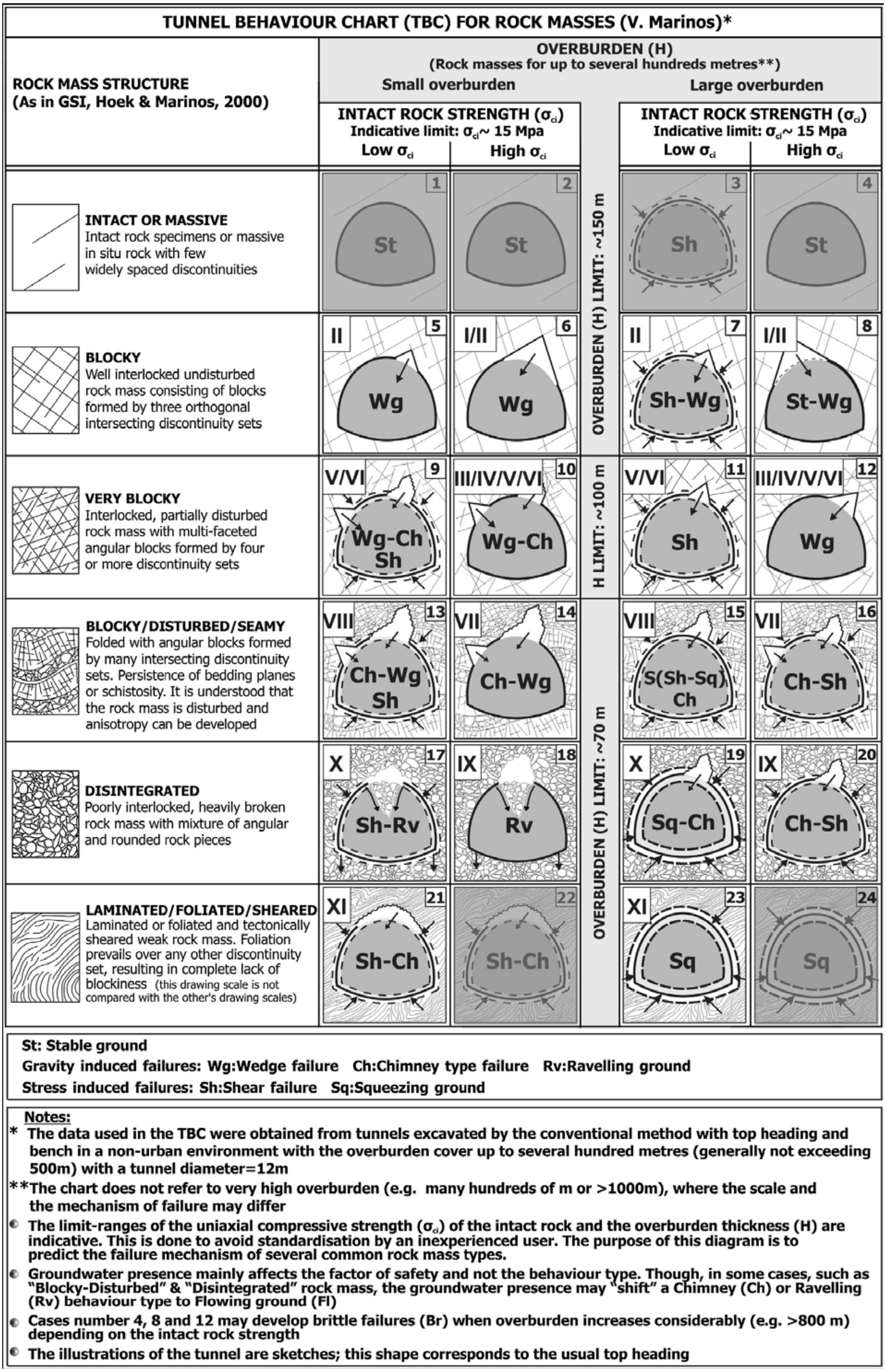

Fig. 11. Modified tunnel behaviour chart (TBC) from Marinos (2012) with projections of the principal failure mechanisms for the rock mass types of flysch (I - XI).

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^{-5}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"**

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(a)

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

IV. ROCK MASS BEHAVIOR IN TUNNEL EXCAVATION

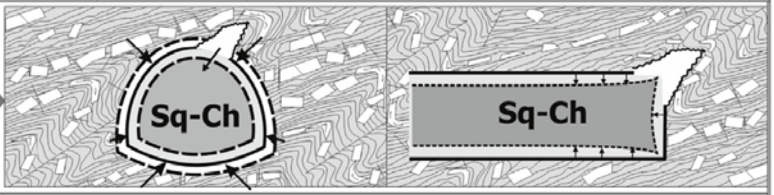
-The user may consult the Tunnel Behaviour Types Table
-The user may consult the TBC classification in order to fill the section IV_b

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*



c) Design philosophy:

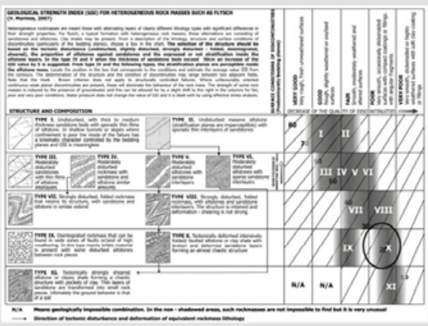
- Structural dependant instability analysis (e.g. Unwedge Programs) [ ]
- Structural and stress dependant instability analysis (Wedge and Numerical Analysis) [ ]
- Stress dependant instability -Deformation analysis (Numerical Analysis) [X] *Three dimensional numerical models to analyze adequately issues 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:

	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. 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).

ing 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).

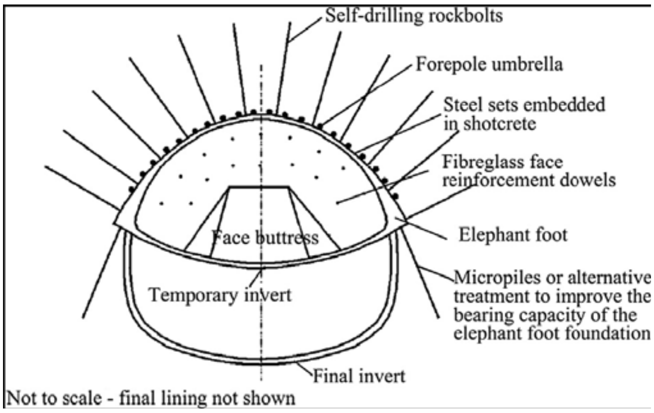


Fig. 13. A typical support design for weak flysch rock masses using top heading and bench method. The necessity, the amount and the combination of various elements of this typical section are results of numerical analysis. The optimisation is a matter of reliable monitoring. For highly squeezing ground,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

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

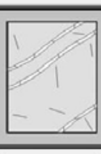
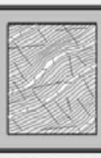
5. Conclusions

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

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).

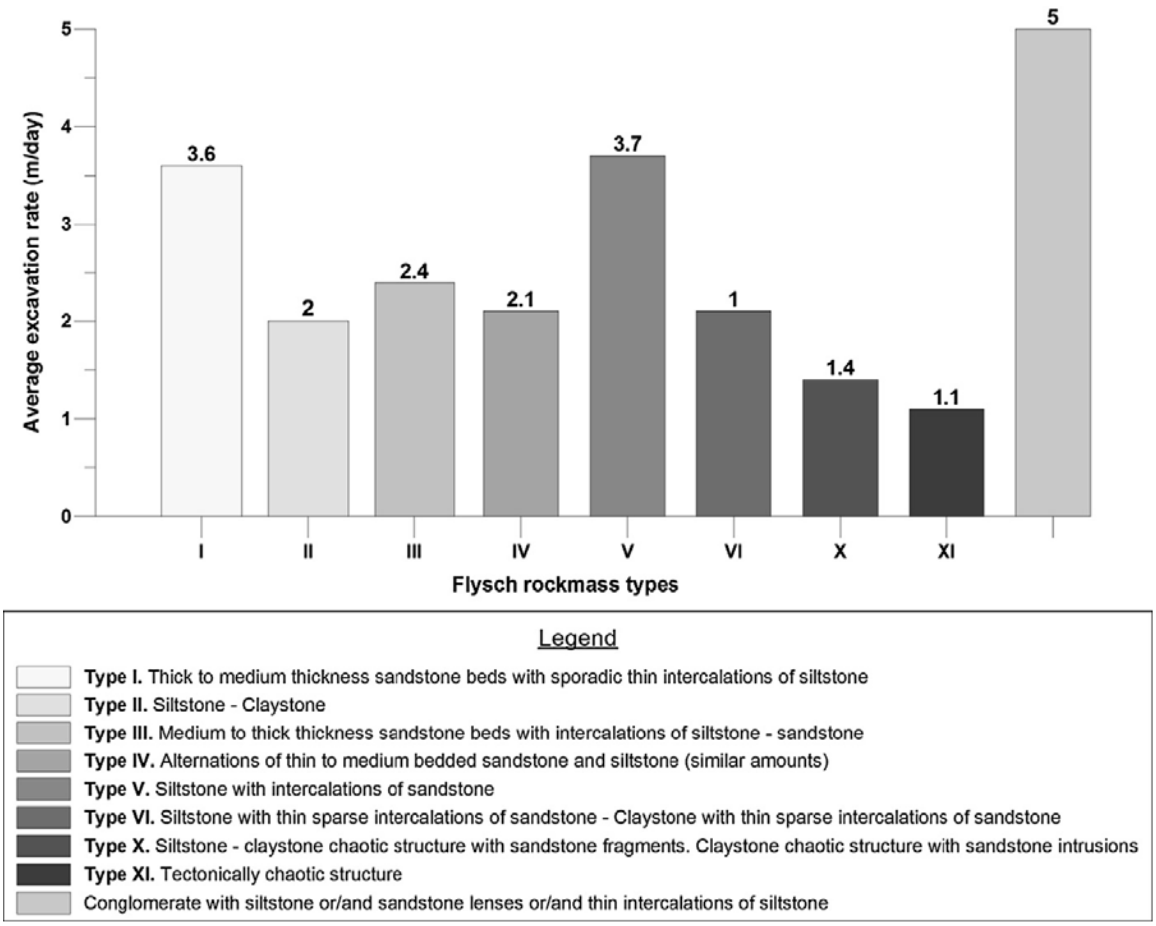


Fig. 15. Average top heading excavation step for flysch rock masses (types I, II, III, IV, V, VI, X and XI). A conglomerate mass is also projected in the last column of the diagram.

detailed analysis. They should be always backanalysed by engineering judgement and adjusted for each sitespecific project.

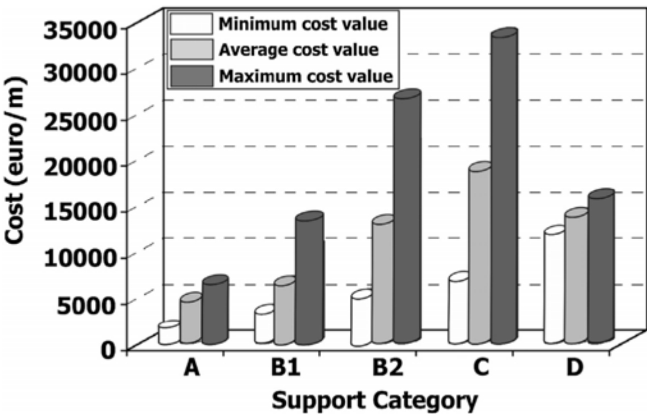


Fig. 16. Cost (Euros/linear metre of tunnel) of the temporary support system for the flysch formations. A-D is the "weight" of the support measures (A: shotcrete and bolts; B1: shotcrete, bolts and steel sets; B2: shotcrete, bolts, steel sets and light face support measures like spilling; C: shotcrete,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

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Use of GSI for Rock Engineering Design

T.G. Carter
Golder Associates, Toronto, Ontario, Canada
V. Marinos
Aristotle University of Thessaloniki, Thessaloniki, Greece

ABSTRACT: In rock engineering design significant advances have occurred in recent years in numerical modelling capability. Better and more advanced insight is now possible of rock-support interaction and rockmass progressive failure processes. However one major drawback to more realistically evaluating these processes has been and still remains the lack of reliable estimates of strength and deformation characteristics. Use of the GSI rockmass classification system and the associated *m*, *s* and *a* parameter relationships linking GSI with the Hoek-Brown failure criterion provides a proven, effective and reliable approach for prediction of rockmass strength for surface and underground excavation design and for rock support selection. Backanalyses of tunnels, slopes and foundation behaviour using the approach attest to its reliability. One of the key advantages of the index is that the geological reasoning it embodies allows rating adjustments to be made to cover a wide range of rockmasses and conditions, whilst also allowing some understanding to be gained of applicability limits. This paper attempts to outline approaches for the application of GSI for the quantitative characterization of rockmasses for inclusion in the Hoek-Brown failure criteria for rock engineering design, not just for the original range of applicability, but also for both ends of the rock competence scale.

1 INTRODUCTION

In recent years, significant advances have occurred within almost every area of geotechnical design. In rock engineering arguably the greatest developments have been in numerical modelling capability. Codes are now available that can not only afford better and more advanced insight into rock-support interaction and rockmass progressive failure processes, but they are now capable enough to allow synthetic rockmasses to be efficiently built so that design layouts can be more realistically evaluated. The drawback to maximizing the advantages that this progress allows is that to date similar levels of improvement have not occurred in the observational characterization of geological variability existing in natural rockmasses.

For effective rock engineering design it is necessary that reliable estimates be available of strength and deformation characteristics of the rockmasses on which or within which engineering structures are to be created, be it a tunnel, a foundation or a slope.

Definition of rockmass properties for a particular design problem usually involves one or more of the following data acquisition methods: a) laboratory testing; b) in situ testing, c) use of rockmass classifications and/or d) back analysis. However, there can be significant differences in scale between results that each of these approaches may yield. Additional complications in representativeness may also arise because, for instance, laboratory scale samples may not be truly representative of the rockmass due to natural heterogeneity present for most formations. Cost and time issues additionally, complicate data acquisition, particularly when it comes to carrying out in situ tests. To estimate reasonable geotechnical parameters for the design of many engineering projects, and in particular for design of tunnel support for long deep tunnels beneath mountain ranges where drilling is difficult before construction starts, and where a back analysis approach would not be possible, there is no option but to rely upon the use of some form of rockmass classification scheme that is correlated with the basic parameters needed for design. Back-analysis is indisputably the best way to estimate appropriate geotechnical parameters, when construction has started, provided the analysis approach being used for the parameter definition is valid. For almost all rock engineering problems, back-analysis evaluation of deformation measurements generally provides the most insight, and oftentimes yields the most credible parameter understanding, such that it can be used to validate or modify any parameters previously used for design.

The need for design input parameters is satisfied nowadays mainly through the use of geotechnical classification systems. When the earliest of the more well-known and more commonly applied rockmass classification systems were initially developed (Terzaghi 1946, Lauffer 1958, Barton et al. 1974, Bieniawski 1973), they were intended principally for tunnelling and mainly as an empirical design method for the purpose of estimating underground support. The knowledge base for these approaches were the hundreds of kilometres of tunnels that had to that date been successfully constructed under moderate stresses and in good quality rock conditions. With the rapid growth of improved numerical design tools, which now allow progressive failure processes and also sequentially installed support to be analyzed, and synthetic rockmasses to be built, the need for acquisition of more reliable rockmass parameters has also grown.

Introduction of the Hoek-Brown failure criterion in 1980 has over the last three decades provided a sound basis for the evaluation of rock mass strength parameters when predicated on reliable engineering geological input, but only when parameters are carefully selected with good geological judgment. While the Hoek-Brown criterion has aided the revolution in rock engineering, it has over the years also suffered a lack of precision in definition of input constants. It was recognized that this was only partially initially addressed through use of the then available rockmass classification systems. As these were considered at the time somewhat restrictive and not sufficiently linked with real geological observation, and thus were perceived to be over-rigid, almost mechanical in their application, the "Geological Strength Index", GSI, was developed, initially by Hoek (1994) but then amplified and improved in subsequent papers published by Hoek, Marinos and Benissi (1998) and Marinos & Hoek (2000, 2001) and Marinos, Marinos and Hoek (2007), resulting in the now familiar basic chart, as per Figure 1.

According to the original Hoek-Brown failure criterion publication of 1980, the criterion is based upon an assessment of the interlocking of rock blocks and the condition of the surfaces between these blocks. The geological strength index (GSI) more than the other available classification systems was formulated to attempt to characterize rockmasses from a more geological rather than a typical engineering approach so as to better meet the need for delivering reliable input data, particularly related to those rockmass properties required as inputs into numerical analysis or into closed form solutions for designing tunnels, slopes or foundations in rocks.

One of its great advantages over other approaches is that it allows characterization of difficult to describe rockmasses.

This paper attempts to outline the approaches for application of GSI for quantitative characterization of rockmasses for inclusion in the Hoek-Brown failure criteria for rock engineering design.

2 THE GEOLOGICAL STRENGTH INDEX

2.1 *Background to development of GSI*

Right from the inception of the Hoek-Brown failure criterion, these two authors recognized that for it to have practical value it must be related to geological observations that could be made quickly and easily by an engineering geologist or geologist in the field. Initially, they considered developing a completely new classification system during the evolution of the criterion in the late 1970s, but soon gave up the idea and settled with using the already published RMR system. In the early days this worked well because most of the problems being evaluated in the 1970s were in reasonable quality rockmasses (RMR>30) under moderate stress conditions. In the context of the Hoek-Brown criterion the estimation of a rock quality value was needed so that it could be used as a primary scaling correction on intact strength for use in the criterion. Since the RMR classification value was being solely utilized in the Hoek-Brown criterion, for estimating rockmass properties that would then be used in numerical analyses, it was recognized that many such analysis methods already accounted for a number of facets of the basic RMR description parameters.

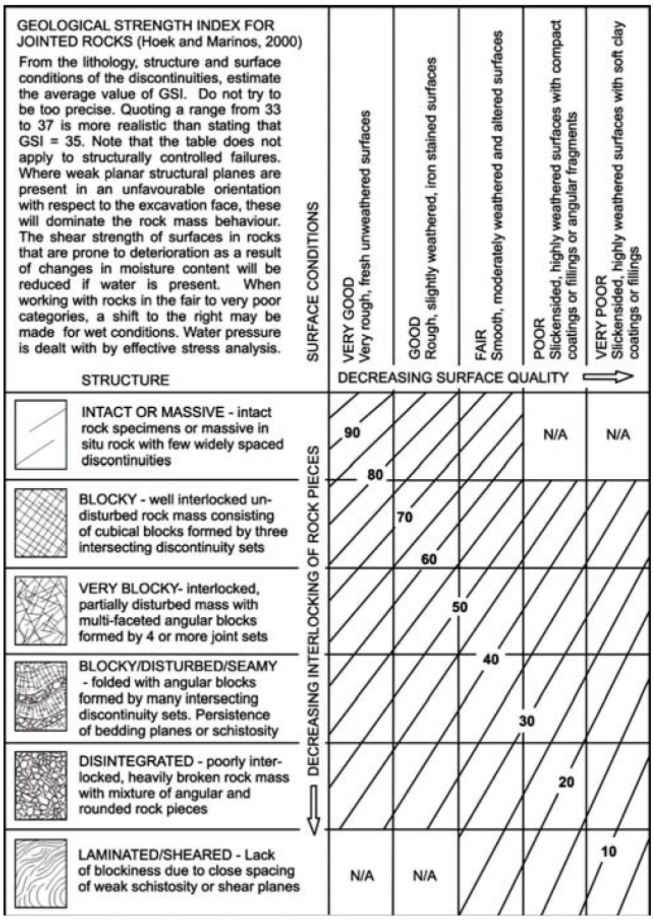


Figure 1. Basic GSI Chart for Visual Geologic Characterization of Rockmasses

In particular the strength term, the structural orientation term and the ground-water term were thought to likely be being double counted. Similar issues were evident for the Q system, due to the parameters for groundwater and stress. While it was appreciated that both RMR and Q were basically developed for estimating underground excavation support, and therefore included some parameters not required for the estimation of rockmass properties, it was considered that these specific parameters were better dealt with by means of full structural and effective stress analyses approaches. It was therefore felt incorporation of these parameters into the methodology for rockmass property estimation was inappropriate.

It soon also became obvious that both the RMR and Q systems were difficult to apply to rockmasses of very poor quality. It was found in particular that the relationship between RMR and the constants *m* and *s* of the Hoek-Brown failure criterion began to break down for severely fractured and/or weak rockmasses. Other classification systems, such as that by Palmstrøm, 2000, 2005 were reviewed and also found to suffer many of the same problems at both the low and high ends of the rock competence scale. While Palmstrøm had made an attempt to look at classifying a broader range of block size than feasible using the RQD classification introduced by Deere, 1964, both of the RMR and the Q systems include and are heavily dependent upon RQD. Since RQD for most weak rockmasses is essentially zero or meaningless, and as none of the existing systems seemed to cope well with this problem, it became necessary to consider an alternative classification approach. It was decided that the required system should not include RQD, but would place greater emphasis on basic geological observations of rockmass characteristics to better reflect the material, its structure and its geological history. Further, it would be developed specifically for the estimation of rockmass properties rather than for estimating tunnel reinforcement and support. This new classification, which was considered more an index of rock competence than a mechanical quality rating was thus termed GSI (geological strength index) in 1992 when work was started on definition of mi relationships for various rock types (Hoek et al. 1992). The index and its use in the transfer equations from rockmass description through to *mi* and *s* definitions for application in the Hoek-Brown failure criterion were further developed by Hoek (1994), Hoek et al. (1995) and Hoek & Brown (1997) but remained still a "hard rock" system, roughly equivalent to RMR at this stage.

The initial GSI classification was set up to match the earliest tables for *m* and *s* (as per Table 1) and to address the two principal factors considered important influences on the mechanical properties of a rockmass – the structure (or blockiness), and the condition of the joints.

It specifically was set up to avoid including other factors, such as intact strength, in situ stresses and/or groundwater pressures, because these factors normally would be allowed for in any sophisticated analyses. As GSI was initially introduced into the transfer equations as a direct replacement for RMR at the time it was proposed that a new variation of the previously published Hoek-Brown transfer equations should be used for disturbed and undisturbed rock conditions with the limit GSI for the equation set equal to 25.

The most recent major revision of the Hoek-Brown criterion was published by Hoek, Carranza-Torres and Corkum in 2002, which resulted in the following suite of equations:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b\frac{\sigma_3}{\sigma_{ci}} + s\right)^a$$

where

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right)$$

and

$$s = \exp\left(\frac{GSI-100}{9-3D}\right)$$

with

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

One of the most important goals of this revision had been to remove the "switch" at GSI = 25 which had been required in the version published by Hoek, Kaiser and Bawden (1995). In place of "disturbed" rock, defined by GSI < 25 and "undisturbed" rock for GSI > 25, a more general Blast Damage factor D was introduced to handle the transition. While also not perfect this new D factor has proved workable, although some clarification and refinements of definition have over the years also been necessary, as discussed

Empirical failure criterion $\sigma_1' = \sigma_3' + (m\sigma_c \sigma_3' + s\sigma_c^2)^{1/2}$ σ_1'=major principal stress σ_3'=minor principal stress σ_c=uniaxial compressive strength of intact rock m, s=empirical constants	Carbonate rocks with well developed crystal cleavage, e.g. dolomite, limestone and marble	Lithified argillaceous rocks, e.g. mudstone, siltstone, shale and slate (tested normal to cleavage)	Arenaceous rocks with strong crystals and poorly developed crystal cleavage, e.g. sandstone and quartzite	Fine grained polymineralic igneous crystalline rocks, e.g. andesite, dolerite, diabase and rhyolite	Coarse grained polymineralic igneous and metamorphic crystalline rocks, e.g. amphibolite, gabbro, gneiss, granite, norite and quartzdiorite
Intact rock samples Laboratory size samples free from pre-existing fractures Bieniawski, 1974b (CSIR)* rating 100 Barton et al, 1974 (NGI)** rating 500	m=7 s=1	m=10 s=1	m=15 s=1	m=17 s=1	m=25 s=1
Very good quality rock mass Tightly interlocking undisturbed rock with rough unweathered joints spaced at 1 to 3 m Bieniawski, 1974b (CSIR)* rating 85 Barton et al, 1974 (NGI)** rating 100	m=3.5 s=0.1	m=5 s=0.1	m=7.5 s=0.1	m=8.5 s=0.1	m=12.5 s=0.1
Good quality rock mass Fresh to slightly weathered rock, slightly disturbed with joints spaced at 1 to 3 m Bieniawski, 1974b (CSIR)* rating 65 Barton et al, 1974 (NGI)** rating 10	m=0.7 s=0.004	m=1 s=0.004	m=1.5 s=0.004	m=1.7 s=0.004	m=2.5 s=0.004
Fair quality rock mass Several sets of moderately weathered joints spaced at 0.3 to 1 m, disturbed Bieniawski, 1974b (CSIR)* rating 44 Barton et al, 1974 (NGI)** rating 1	m=0.14 s=0.0001	m=0.20 s=0.0001	m=0.30 s=0.0001	m=0.34 s=0.0001	m=0.50 s=0.0001
Poor quality rock mass Numerous weathered joints spaced at 30 to 500 mm with some gouge. Clean, compacted rockfill Bieniawski, 1974b (CSIR)* rating 23 Barton et al, 1974 (NGI)** rating 0.1	m=0.04 s=0.00001	m=0.05 s=0.00001	m=0.08 s=0.00001	m=0.09 s=0.00001	m=0.13 s=0.00001
Very poor quality rock mass Numerous heavily weathered joints spaced at 50 mm with gouge. Waste rock Bieniawski, 1974b (CSIR)* rating 3 Barton et al, 1974 (NGI)** rating 0.01	m=0.007 s=0	m=0.010 s=0	m=0.015 s=0	m=0.017 s=0	m=0.025 s=0
*CSIR: Commonwealth Scientific and Industrial Research Organization **NGI: Norway Geotechnical Institute					

Table 1. Original GSI tabulations for *m* and *s* for various geological conditions

subsequently in this paper.

With continuing use worldwide, the GSI system has continued to evolve. Additional publications, from 1998 onwards mainly have been aimed towards providing clarification of application methodology, primarily aimed at addressing difficulties with usage, particularly for weak inhomogeneous rockmasses.

Many of the most notable improvements have come from work that Hoek and Marinos had undertaken during tunnelling in difficult ground in Northern Greece (Hoek et al. 1998, Marinos and Hoek 2000), (Figure. 2).

An extension of the original GSI application charts for heterogeneous and structurally complex rockmasses, such as flysch, was initially introduced by Marinos and Hoek (2001) and recently updated and extended by Marinos et al. (2007), Marinos et al. (2012) and Marinos (2014). Specific GSI charts for molassic formations (Hoek et al. 2005), ophiolites (Marinos et al. 2005), gneiss (in its disturbed form), and particular cases of limestones (Marinos 2007, 2010) 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.

Figure 2. Difficult Ground Conditions for undertaking Rockmass Classification (Photo from the Egnatia Highway).

In its current form the index is based first upon an assessment of lithology and secondly on structure and condition of discontinuity surfaces within the rockmass. As outlined above, historically it was considered that GSI should be divorced from any quantitative, numerical coding approach, like RMR or Q or most other classification systems. Indeed the focus had been to only "allow" visual description of the rockmass as the basis for ascribing a GSI value. Needless to say this unfortunately led to confusion and mis-definition of GSI values, especially when other classifications had been in use on the same project or scheme. Accordingly, last year, rather reluctantly a set of quantitative codified rules were introduced on the side of the GSI chart to attempt to benchmark the ranges so some global consistency could be achieved between different observers, as discussed in the next few paragraphs. A two pronged approach to characterization use was also proposed (Hoek et al. 2013) – purely observational, as per the original intent using the various already published charts, and – quantitative, defined per specifically selected scales.

Figure 3 shows a flow chart to aid selection of approach for arriving at appropriate Hoek-Brown m, s and a parameters based on either entry point.

2.2 *Observational characterization*

When a rockmass is exposed in outcrops, in surface excavations, such as road cuts and/or in tunnel faces and if sufficient core is available even in borehole cores, visual assessment of its competence is feasible and assigning a zone of appropriate character is straightforwardly done directly on the standard GSI chart shown in Figure 1. In this chart the two basic parameters of GSI, the blockiness of the mass and the conditions of discontinuities, are plotted on the axes respecting the main geological constraints that govern a formation, thus geologically defining a sound index that almost all geologists find simple to assess in the field.

2.3 *Quantitative characterization*

Experience has shown over the years that quite a number of mining engineers and equally many civil engineers without strong geological backgrounds are less comfortable assigning a quality range based solely on visual assessment, and as a consequence some quantified scales were included on the standard GSI chart to aid application in a more quantitative manner. (Hoek. et al, 2013). This modified chart (which is shown in Figure 4), has been drawn up to include one of the parameters most wanted to be avoided – RQD, but this was with good reason. It may have its faults, but these are now well known, and can be worked around. Moreover, despite these issues, RQD concept is generally well understood and accepted to give reasonable estimates of brokenness, provided that one remains within blocky rockmass conditions, such as are defined in Figure 4.

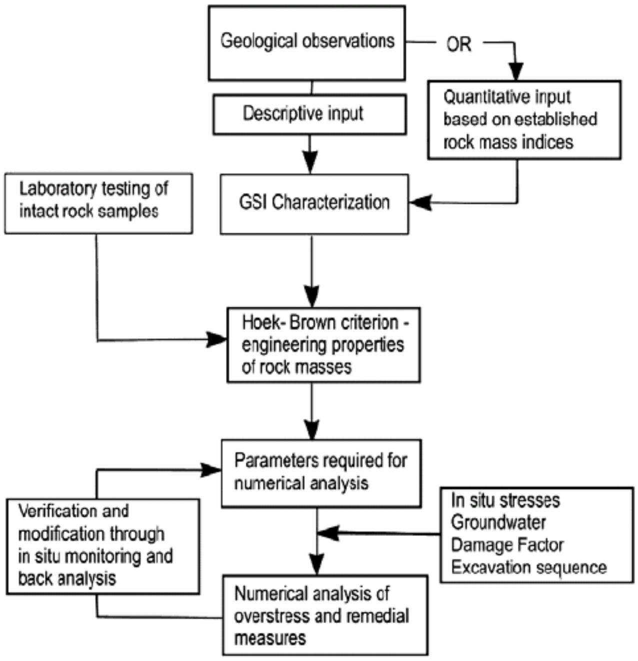


Figure 3. Data entry stream for using the Hoek-Brown system for estimating rock mass parameters for numerical analysis (Hoek et al., 2013).

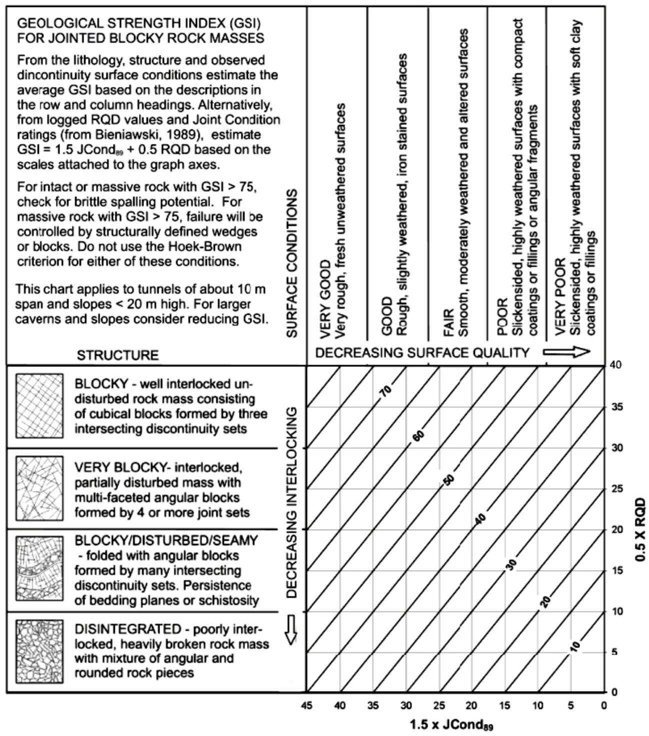


Figure 4. Quantification of GSI by Joint Condition, as graded by RMR, and RQD only for the central section of the basic chart (Hoek et al., 2013).

The chart in Figure 4 was also introduced because the lack of quantified scales on the original GSI chart had already prompted several other authors to attempt to quantify the original chart by adding scales derived from RMR, Q, RMi or other rockmass classifications. Various attempts by Cai et al. (2004), Russo (2007) and Sonmez & Ulusay (1999), each individually have merit, but also some limitations. Similarly the quantified chart proposed by Hoek at al. (2013), shown in Figure 4, has some limitations, but as explained in more detail in the 2013 paper it is thought reliable enough in most circumstances to provide a good starting point for characterization.

The quantification processes suggested for use in these cases are related to frequency and orientation of discontinuities and are limited to rockmasses for which these numbers can quite easily be measured. In consequence the quantifications do not work well in tectonically disturbed rockmasses in which the structural fabric has been destroyed. In such types of rockmasses the authors recommend the use of the original qualitative approach based on careful visual observations.

It must be noted here though, that is meaningless to attempt to assign a precise unique GSI number for a rockmass. In almost all situations, rockmasses have variability, thus GSI is best assigned as a range. For analytical purposes this range may best be defined by a normal distribution with the mean and standard deviation values assigned on the basis of common sense.

2.4 *Transfer equations*

As is evident from the suite of standard equations for using the Hoek-Brown criteria, the GSI term is used in conjunction with appropriate values for the unconfined compressive strength of the intact rock σ_{ci} and the petrographic constant m_i, and a measure of the brokenness as defined by the *s* and *a* parameters to allow estimation of the mechanical properties of a rockmass, in particular, compressive strength (σ_{cm}). For defining the deformation modulus (E_{rm}) of the rockmass, the following relationship, proposed by Hoek and Diederichs (2006) is recommended:

$$E_{rm} = Ei\left(0.02 + \frac{1 - D/2}{1 + e^{((60+20D-GSI)/12)}}\right)$$

In the above expression and in the previous equations for *m* and *s* the Disturbance Factor, D can be assessed from Table 2 or from the graphic table in Hoek et al, 2002. It is noted that D should only be applied to the blast damage zone (typically up to a few metres into the wall of a tunnel and for some proportion of depth into each bench face based on the slope height for a slope) and definitely should not be applied to the entire rockmass.

Location	D-Factor	Disturbance Characteristics
Underground Excavations (confined conditions)	0	High Quality Perimeter Blasting (100% half barrel traces) or Mechanical Excavation with TBM or Roadheader
		NATM excavation in weak rock with mechanical excavation
	0.5	As above (but with invert heave issues)
	0.8	Poor Quality Blasting (<50% half barrel traces)
Open Cuts and Open Pits (de-stressed conditions)	0.7	Controlled Blasting (>80% half barrel traces)
	1	Poor Quality Blasting (<50% half barrel traces)
	0.7	Mechanical Excavation in Weak Rock with Face Shovel etc
	1	Typical Open Pit Production Blasting/Quarry Blasting

Table 2. Guidelines for the selection of the Disturbance (Blast Damage) Factor D (after Hoek et al. 2002)

In examining these transfer equations between GSI and the Hoek-Brown criteria constants *m*, *s* and *a* and also the rockmass modulus expression E_{rm} it should be noted these equations have been set up to account for any basic differences in characteristics between different parent rock fabrics and rockmass competence. As such they allow the prediction of an appropriate m_b value properly normalized by the intact m_i for that rock type or rock fabric while *s* and the exponent *a* are scale invariant, reflecting simply the degree of brokenness of the rockmass. Defining an appropriate rockmass modulus value for use in any modelling however, like m/m_i also requires full normalization by the intact modulus so that it also appropriately takes into account the inherent stiffness of the parent intact material.

2.5 *GSI and specific rockmasses*

Figure 5 shows a compilation of all of the charts that have been published in papers in terms of probable m_i and σ_{ci} for the parent rock material. Most of the common GSI ranges for typical gneisses, granites, ophiolites, limestones, schists, siltstones / mudstones / shales, molassic and flysch formations have been illustrated here, highlighting how minor geological differentiations affect the geotechnical properties of the different formations. For more details the reader is referred to the original publications presenting these charts for each specific formation as published by Marinos and Hoek (2000), Hoek et al. (2005), Marinos et al. (2005) and Marinos et al. (2011).

In this overall chart it should also be appreciated that differences in assigned GSI commonly occur due to different tectonism, weathering and alteration effects on either brittle or soft rock materials. These effects due to influence of different geological processes are discussed in subsequent paragraphs.

These charts cover a wide range of rockmass competence from extremely competent hard rockmasses with non-degradable fabrics, typical of the deep mines in high-strength/high stress conditions where spalling and bursts characterize behaviour in the top left corner of the chart through to the lower right corner of the matrix diagram where rockmass competence is low and where squeezing and often significant closure problems are the characteristic hallmarks (e.g. flysch, shales etc.).

For any rock engineering design the first issue is geological characterization. This basically defines where you are within Figure 5 and this then allows one to establish the potential range of characteristics for the rockmass, based on the competence of the parent rock material. For example if one were working in the flysch conditions of central and southern Europe then one would be looking at a point plotting in the lower right third of the diagram, while for rock conditions within a hard rock mine in Canada or South Africa at significant depth, one might then be looking more towards the top left corner.

Himalayan, Andean or Alpine mountain belt conditions would cover the complete suite top left to bottom right.

The size and spread of one's GSI chart on a sitespecific basis could therefore be expected to vary depending on parent rock type and mineralogy and thence on macrofabric and overall competence.

An overprint to this whole matrix would be created by the changes in parent rock competence created by natural processes such as weathering or mineralogical alteration or solution effects. Indeed, for tropically weathered rockmasses, such as exist in many parts of the world, competence ratings could be expected to change several orders of magnitude in the vertical plane within the matrix in Figure 5.

For Hong Kong granites, for example, one might move from intact, high-strength, fine-grained grey granites with high m_i and high σ_{ci} which plot towards the top left of the diagram, progressively downwards and to the right through the weathering grades to completely degraded granite – essentially a soil. However *mi* for this range of rock characteristics would vary less severely (as is shown by many triaxial tests undertaken on the granites from Hong Kong for all different weathering grades).

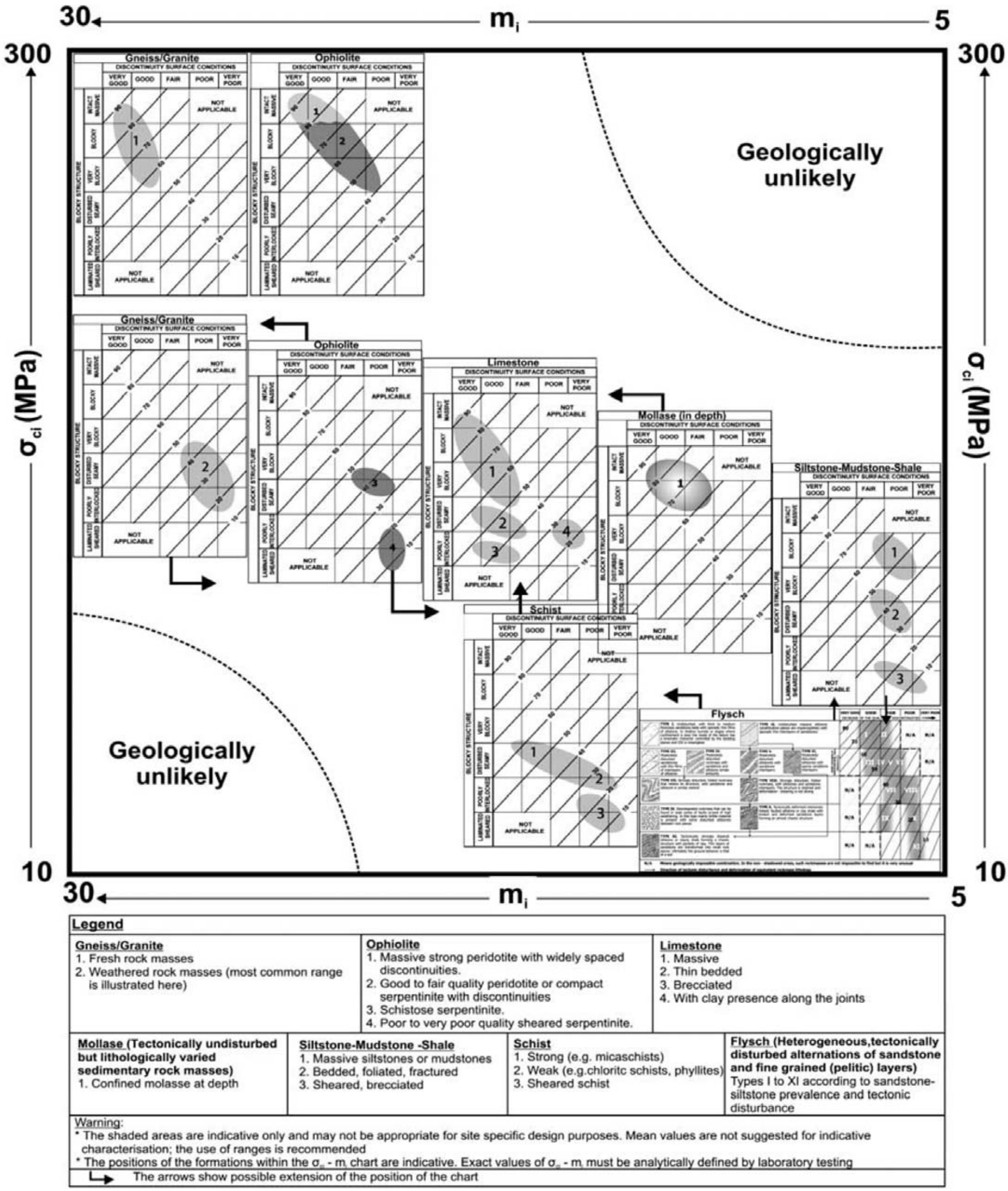


Figure 5. Most common GSI ranges for typical gneisses, granites, ophiolites, limestones, schists, siltstones/mudstones/shales, molassic and flysch formations in conjunction with a range of m_i and σ_{ci}. (Refer to text and reference list for original papers for more details on charts)

2.6 *Intact rock properties*

Right from its original formulation in 1980, through to the current generalized expression:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

The concept behind the development of the Hoek-Brown criterion has remained consistent with the aim being to allow definition of the strength of any given rockmass based on its composition, considered as a matrix of blocks of intact material set within a discontinuity framework, thus replicating a typical blocky rockmass in hard competent rock conditions. In the expression the value of σ_{ci} defines the uniaxial compressive strength of the intact rock fragments making up the blocks within the rockmass, with the dimensionless parameters σ_3/σ_{ci}, m and s modifying the contribution of σ_{ci} to reflect the brokenness of the rockmass due to the extent of fracturing present within the rockmass. Depending on the confining stress (σ_3/σ_{ci}), the degree of block interlocking (m_b) and the condition of the inter-block surfaces (s), these dimensionless components act to degrade the intact strength of the matrix blocks to a strength considered representative for the overall rockmass, when considered as an equivalent "new" material.

Figure 6 shows Hoek-Brown strength envelopes for a range of commonly encountered rock types. Approximate values of the ratio between σ_{ci} and σ_{ti} are listed for each rock type in the Figure 6.

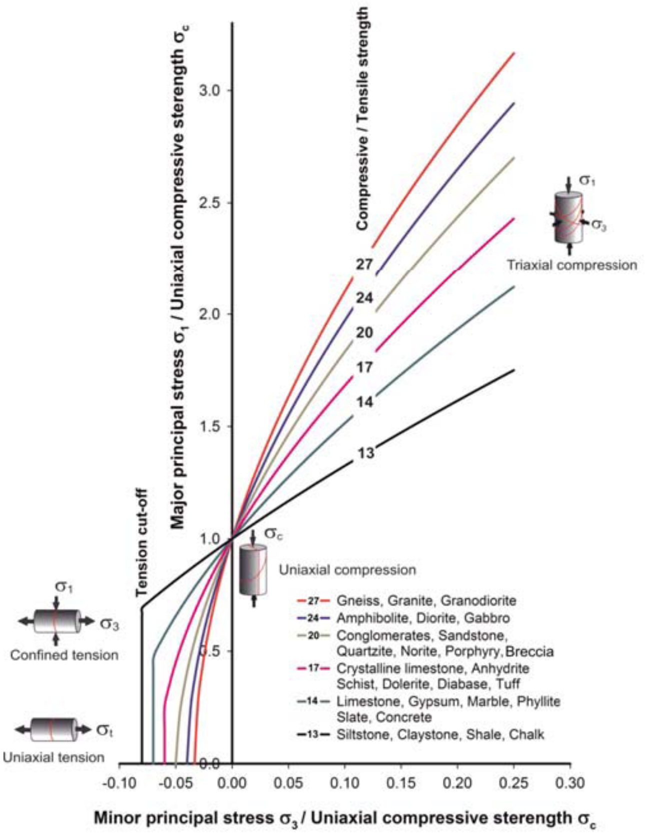


Figure 6. Hoek-Brown typical Intact Strength envelopes for a range of igneous, metamorphic and sedimentary rocks (Hoek and Martin, 2014)

Ideally, a representative *σci* should be determined by direct laboratory tests under carefully controlled conditions thereby generating curves similar to those shown in Figure 6 and in many earlier publications on the Hoek-Brown criterion. However, in many cases, this is not possible in practice because of time or budget constraints or because it is not feasible to recover samples for laboratory testing (particularly the case in weak, schistose or tectonically disturbed rockmasses where the influence of discontinuities cannot be avoided within laboratory samples).

Under such circumstances estimates of the value of σ_{ci} are quite often made on the basis of published information, simple index type tests (such as point load tests, where appropriate) or by use of various descriptive assessments such as have been published by the International Society of Rock Mechanics (Brown 1981). The values given in many case are often general and usually it is better to avoid them for design purposes. This same problem of strength estimation is discussed again later, as it is one of the most thorny issues complicating reliable design.

Experience has shown that there is a common tendency to underestimate the value of the intact strength in many cases where actual laboratory data is not available. This is particularly true for weak and/or tectonically disturbed rockmasses where the characteristics of the intact rock components tend to be masked either by structural tectonic elements or by surrounding sheared or weathered material. These underestimates can have serious implications for any engineering design and care has to be taken to ensure that realistic estimates of intact strength are made as early as possible in the project. In some situations, early estimates can be refined through detailed back-analysis, for example of tunnel deformation, and, while this may require considerable effort and even the involvement of numerical analysis, the attempt will generally be repaid many times over in time and cost savings achieved by more realistic designs.

Care must also be paid to ensuring that tensile tests are carried out so that the strength envelope can be properly defined. This is of major importance at the intact sample scale. Because the original Hoek-Brown criterion had problems with predicting too high an axial strength and also a finite tensile strength in areas of the rockmass with limited to no confinement (i.e. with the minor principal effective stress (σ_3) close to zero), the modified generalized equation that was formulated in 1992, was structured to allow greater curvature of the envelope through the tensile range, while still covering the full range from $0 < s < 1$. This however has given rise to other problems for rocks of particularly low strength and competence, requiring inclusion of a tensile cut-off, such as shown in Figure 6 for the siltstone/mudstone group. This approach is one method to tackle this problem. Other approaches include using the use of transition function equations as explained later in this paper. The issue can also be tackled during the analysis stage as changing envelope curvature may not be necessary if the need for a tension cut off is understood and is used correctly (Hoek and Martin, 2014).

As basis for decisions, where data is missing, some typical values for m_i and σ_{ci} for various rock material types commonly encountered worldwide in engineering projects are presented in Table 3.

Typical σ_{ci} (MPa)	Metamorphic	Igneous			Sedimentary	m_i
		Intrusive		Extrusive (Volcanic)		
		Felsic	Mafic			
125-250		Coarse (Granite)				31-33
100-300	Granular Texture (Granulites, Quartz Gneiss)	Medium (Grano-diorite-Diorite)				28-30
85-350	Medium, amorphous (Amphibolite)		Coarse (Gabbro-member of ophiolites)	Mafic (Basalt) Intermediate (Andesite) Felsic (Rhyolite)	Coarse (Conglomerate-not clayey)	25-27
75-350	Fine, amorphous (Homfels, Qua-rtzite)		Medium (Dolerite / Diabase – member of ophiolites)		Medium quartz cemented (Sandstone/ Sandstone members of flysch or molasse/greywacke)	17-20
50-200	Bended/ Gneissose (Biotitic Gneiss)		Fine (Serpentinite-member of ophiolites)		Medium carbonates (Limestone)	13-16
30-100	Folliated (Phyllite, Slate)				Fine, (clastics) (Siltstone/ Siltstone members of flysch or molasse/tuff)	10-12
20-60	Schistose (Schist)				Fine, Calc-rock (Chalk/marl)	7-9
10-50	Mylonitized (Sericitic Schist, Mylonite)				Ultrafine (Claystone, Mudstone / sheared Siltstone, Shale members of flysch)	4-6

Table 3. Typical values for *σci* and m*i* for range of igneous, metamorphic and sedimentary rocks (to be read in conjunction with Figure 6 regarding parent rock type characteristics)

This table can be used for preliminary estimating, but it is far better that the value of the constant m_i, as for the intact strength σ_{ci}, be determined by triaxial tests, yielding envelopes similar to those in Figure 6. When this is not possible, estimates can be made based on published values (e.g. from the program RocLab or from published tables from Brown 1981 and/or from Stacey & Page 1986 or other sources).

Note though that great care must be taken when applying such strengths depending on the required engineering application, as the data in Table 3 and much of the data in Rocklab is for dry rock. Values of m_i can also differ somewhat depending on the state of saturation. In this regard it is always better to carry out site specific laboratory triaxial testing in an attempt to obtain realistic test data, not just for intact strength σ_{ci}, but also for determining intact ten sile strength σ_{ti}, so that the full Hoek-Brown envelope can be established and a representative m_i value properly established. This is particularly important for both ends of the rock competence scale – i.e. for both the weak and for the very competent rocks, and also for weathered or altered intact rock materials.

For brittle rocks towards the high end of the rock competence scale, where designs might be needed for high stress situations, it is critical that the onset of brittle failure behaviour is established by proper laboratory testing. For such rock, and for the weak rocks also there is a need for more attention being directed to undertaking representative laboratory tests. In the opinion of Professor Brown (Hoek, pers. comm.), the fact that "plug-in" strength tables are available in codes such as Rocklab has resulted in a significant decrease in the amount and the quality of triaxial testing carried out on projects because many users tend to take the easy way out by simply relying on tabulated data. In this light, the relative expense of acquiring a little testing data should be seen as trivial compared with the potential cost implications of a lack of good reliable data that might otherwise compromise subsequent design decisions.

2.7 *GSI and intact rock strength*

Moving on from the discussion above, it must be remembered that the strength of the intact rock does not affect the GSI value. GSI is only dependent on the fracturing degree and on discontinuity surface condition. GSI values can be high (e.g. >70) in a rock of low *σci*. For example marl or siltstone with low σ_{ci} can form a very good structure that may be described as blocky or even intact.

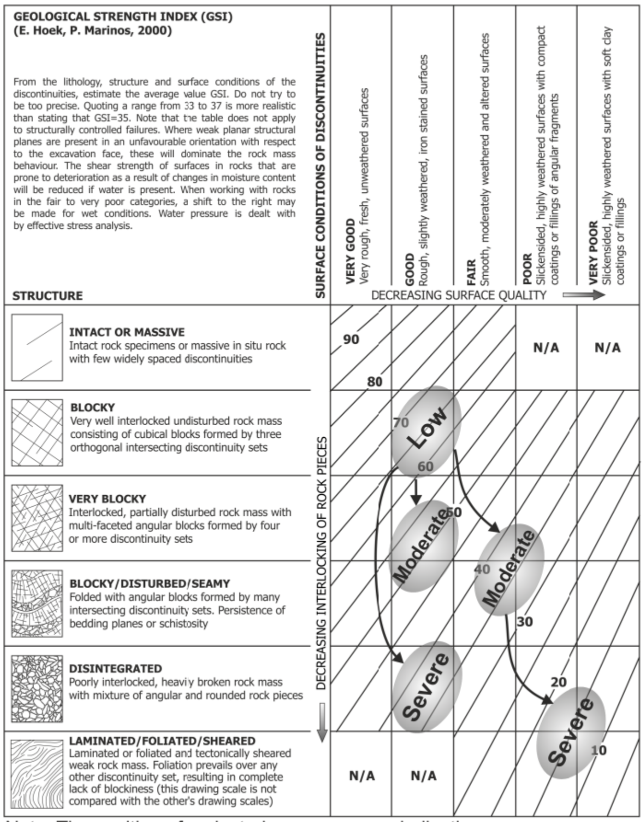
On the contrary, a rock with high strength can be initially characterised as Good due to its high σ_{ci} but may become Poor when it is heavily fractured (e.g. along a fault zone). In this case, GSI can be low (e.g. 25-30), if the discontinuity surface condition is also Poor. An indicative example of how tectonism (low to severe) affects the GSI is shown in Figure 7.

However, if the low strength of the intact rock has derived from weathering, alteration or shearing through tectonism then GSI is also reduced with the disturbance of the structure.

2.8 *GSI and tectonism*

When tectonism is low, GSI values are high to very high (Intact to Blocky structure). Lower GSI values can however be acquired when rockmasses are crossed by discontinuities formed by genesis such as frequent bedding or schistosity planes, often with Fair or Poor conditions on these discontinuities.

In tectonic areas, particularly if compressional, GSI values may be considerably reduced since the structure is more fractured (more joints) or even sheared; such rock behaves in a more ductile manner (e.g. mudstones, shales, siltstones). In the last case joint condition is probably Poor to Very Poor (with slickensided or soft clay coatings due to shearing). The intact rock strength σ_{ci} and the m_i value may also be reduced in the case of shearing.



Note: The position of projected grey areas are indicative

Figure 7. Indicative example of how tectonism (from low to severe) affects the GSI.

2.9 *GSI, weathering and alteration*

Before leaving the topic of correct input parameter definition it is important to realize that rarely will a project be executed entirely in fresh rock. Only in deep tunnels and mining situations is it likely that one would be entirely in dry, fresh and unweathered rock. On the other hand, alteration can be from none to severe in different depths. Accordingly, some consideration needs to be given to the impact that these natural processes of weathering and alteration can have in degrading intact rock material quality, strength and deformability from initial intact state.

There is much confusion in the engineering rock mechanics (non-geological) literature regarding these two processes, as the terms are so often used interchangeably, largely because there is overlap in characteristics, but the origin is different.

Alteration is an effect brought about by generally deep geological processes – hydrothermal alteration, metamorphic alteration etc. Many rocks that are heavily altered are common in mining situations as they tend to be located in immediate proximity to the main ore body, so oftentimes form the hanging wall or footwall of the stopes for which rock engineering design is needed.

Weathering by contrast, is almost universally a shallow depth decomposition process brought about by two different, but linked processes – mechanical and chemical degradation. Again in many engineering situations one has to design engineering projects in weathered rocks. The various stages of weathering of intact rock, rockmass and discontinuities have been described by ISRM (1981) in certain grades (from fresh rock W-I to clayey-sandy soil W-VI). Other descriptions for weathering have been made by the engineering group of the British Geological Society (Anon, 1995).

In the context of GSI, the influence that either process exerts on the ascribed value of GSI is that both degrade not just the parent intact rock material but also they change the character and competence of the rockmass fabric. According to the weathering degree the discontinuity surface condition becomes poorer and the interlocking of rock blocks becomes loosened. The structure on the other hand may not be in principle affected, at least if weathering is not very advanced. An indicative example of how this increase in weathering degree affects the GSI value is illustrated in Figure 8. In weathering degrees W-II and W-III, discontinuity condition is shifted to the middle or right columns of the GSI chart (Figure 8). In W-IV to W-V, joint condition is Poor to Very Poor due to the weathering products along the joints.

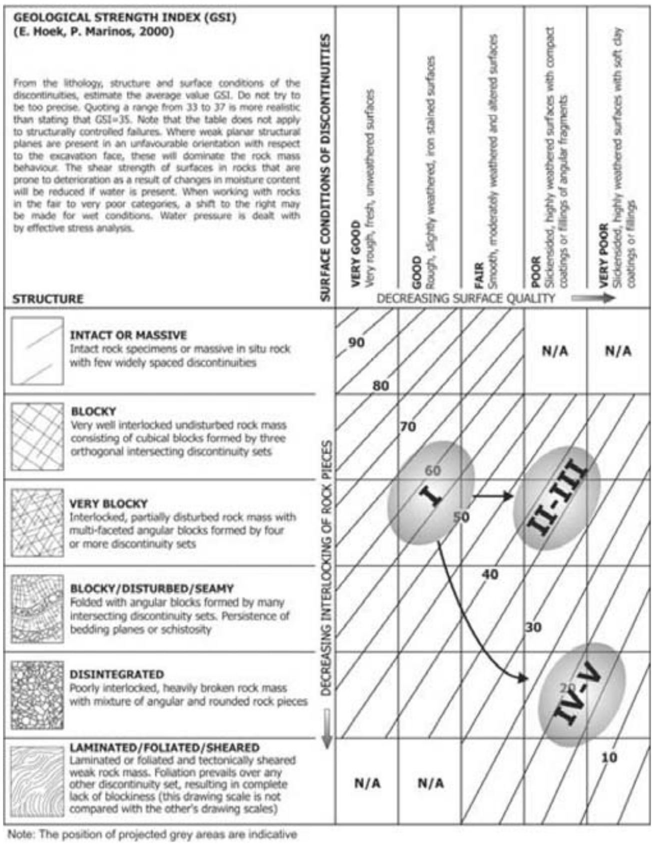


Figure 8. Indicative example of how the weathering degree (W-I – W-V) affects the GSI.

By the same token, almost all of the processes of mechanical weathering - which change the rockmass fabric and break down the blocks into smaller pieces are describable through routine application of the GSI definition scales on the standard chart.

The degradation of the parent material within the intact pieces within the fabric though needs special consideration. Figure 9 provides some guidance for assessing the influence of chemical/mineralogical changes that occur due to pervasive weathering.

The main effect of weathering though is mainly concentrated in the intact rock strength properties like σ_{ci} and m_i, since interlocking and bonding of grains is loosened. Reduction factors from intact rock strength (Stacey and Page, 1986) according to the weathering grade are presented in Figure 9. Note that at W-III state the rock material is not yet friable like in W-IV. The change from W-III to W-IV is however a critical boundary since there is generally considerable reduction to intact properties at this change, often way in excess of 50% of fresh intact material, while GSI may also by affected to some degree by reduction of the structure also.

However, the actual changes that will occur in a given situation are very much rock-type dependent. Some rocks are resistant to intact material fabric change. These are the rocktypes that the aggregate industry chooses for high durability, some diabase, some hornsfels, some diorites are typical of very high durability rock materials. Other rocks tend to decompose completely with penetrative weathering or alteration. Even rocks, which are quite competent, can be quite prone to degradation and decomposition. Many feldspathic granites, for instance can weather appreciably, with the feldspars turning to clay minerals. Some rock materials absorb water and disintegrate as a consequence, some by swell processes, others by drying processes, depending on the internal mineralogy. Control of decomposition is often cementitious material controlled, commonly quartz cements are stronger and more durable than calcite/carbonate cements and even better than iron oxide/haematitic cements. Processes of laterization - which are typical in deep, tropically weathered areas, also reduce rock material strength, particularly at the saprolite boundary. In such rocks the intact structure may become pervasively degraded right within the rock material itself.

Most weathering and alteration processes create major changes in rock material character, which in turn alters the parent material m_i, σ_{ci} and E_i, let alone the changes that occur to the rockmass fabric - all of which affect GSI. This is why it is not recommended just taking published m_i and strength values for dry unweathered intact material based on tables, such as Table 3 or from RockLab listings as these could be seriously in error for actual site conditions.

Alteration in principle also affects both the intact rock properties of the material and the joint surface condition. In fresh conditions, unaltered rockmasses can be generally massive, strong rocks with sparsely spaced discontinuities. Slightly to moderately altered rockmasses often exhibit smoother or slickensided joint surfaces (e.g. though serpentinization). The structure, σ_{ci} and m_i in this case are not or only slightly affected.

With severe alteration, GSI values can be reduced considerably. The structure becomes disturbed (e.g. from Blocky to Very Blocky or perhaps to Sheared) according to the alteration degree (e.g. formation of schistose or laminated planes). Joint condition in this case is Poor to Very poor. The intact properties σ_{ci} and m_i are considerably reduced. An indication of how alteration affects GSI is illustrated in Figure 10. However there are cases, e.g., contact metamorphism, where alteration may result in stronger rocks (such as Quartzites, Keratites etc).

Measuring intact rock strength, σ_{ci}, from altered rockmasses is always problematic. When testing for example schisto-serpentinites, the influence of "schistosity" results in a significant reduction in the strength of a large proportion of the specimens. As a consequence, it is very difficult to obtain reliable σ_{ci} values from laboratory tests. For schisto-serpentinite rocks it is suggested that the uniaxial compressive strength should be estimated from that of the normal serpentinite and reduced by about 30% to account for the schistosity (Marinos et al. 2005).

2.10 *GSI and other classification systems*

Global correlation to other classifications has been an ongoing discussion for many years, with the pros and cons of different approaches debated intensely.

Several decades ago, with arrival of the personal computer, the tools for rock engineering design started to change. Although still crude, a number of useful numerical methods

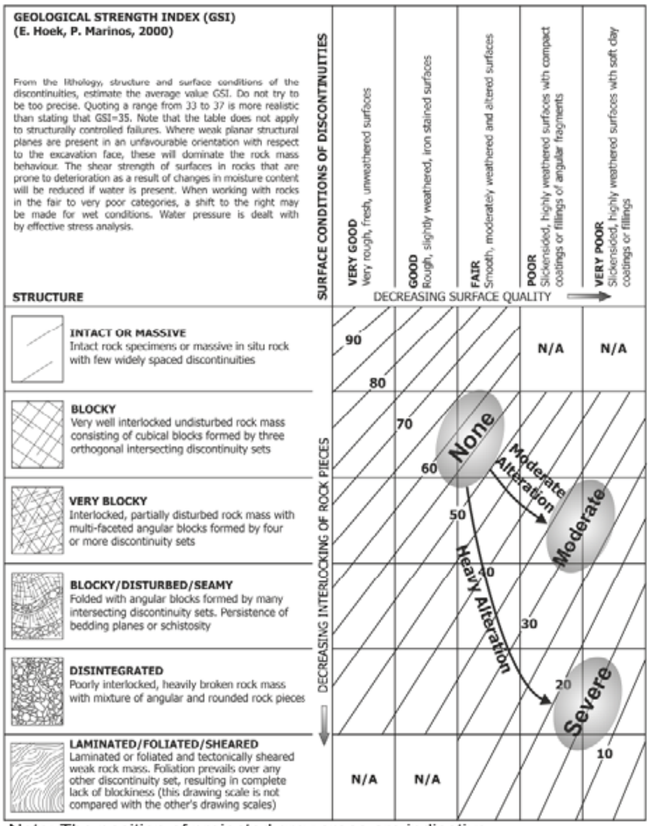
GRADE SCALE (ISRM)	TERM	Description	σ_{ci} reduction factor (After Stacey and Page,1986)	GSI notes
VI	Residual soil	Soil derived from in situ weathering (100% soil) (from grades IV,V)	N/A (advise soil mechanic testing)	N/A (advise soil mechanic testing)
V	Completely weathered	All rock material is decomposed and/or disintegrated to soil (less than 30% rock of grades I,II,III). The original mass structure is still visable Shearing can be affected through matrix.	0.001-0.004	Area where GSI is marginally applicable. The structure has been severely disturbed and the interlocking between the fragments has been lost. Clayey-sandy zones follow the original structure and rock fragments are not interlocked. Joint condition is Very Poor.
IV	Highly weathered	More than a half of the rock material is decomposed and/or disintegrated to a soil (30% to 50% rock of grades I,II,III). Severe weathering along the surfaces. Fresh or discoloured rock is present either as a discontinuous framework or as corestones. The rock material is friable. Corestones still affect shear behaviour of the rock mass.	0.04	The structure has been highly disturbed and the interlocking between the fragments has been highly loosened. Clayey and sandy products are filling all the discontinuities. Joint condition is Very Poor. The GSI shifts down and right in the chart.
III	Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil (50% to 90% rock of grades I,II,III). High to severe weathering along the surfaces. Fresh or discoloured rock is present either as a discontinuous framework or as corestones. The rock material is not friable. The structure is locked.	0.1	The interlocking between the fragments has been considerably loosened. Weathering coatings and fragments are filling principle discontinuities (e.g. gneissic bands) and other joints.Joint condition is Poor. The GSI shifts to the poorer structure(e.g. from Very Blocky to Blocky/Disturbed and to the right in the chart.
II	Slightly weathered	Discolouration indicates weathering of rock material and discontinuity surfaces (>90% rock of grades I,II,II). All the rock material may be discoloured by weathering and may be somewhat weaker than its fresh condition.	0.4	The structure is not changed but the quality of the discontinuity surfaces is (shift to the right). The GSI is reduced to Fair conditions.
I	Fresh	No visible sign of rock material weathering (100% rock); perhaps slight discolouration on major discontinuity surfaces	1.0	Fresh rocks are generally massive (Intact to Very blocky). Joint condition is Very Good(very rouch) to Good(rouch). Blocks and surfaces are strongly interlocking. Rock mass may be even more fractured but only in depth (along a fault zone) where wethering has not been favored. In surface, a fractured rock mass is rarely fresh.

Figure 9. GSI and intact strength change for weathering grade W-I to grade W-VI (grades according to ISRM 1981)

were being developed at that time that offered the promise for being able to undertake much more detailed analysis of difficult surface and underground excavation problems. In tunnelling the advent of numerical techniques to tackle rock-support interaction and the development of ideas associated with the ground reaction curve concept allowed problems to be tackled sometimes well outside the ideal range for application of the various then available tunnel support classification systems such as RMR, Bieniawski (1973, 1976) and the Q system Barton et al. (1974), Barton (1976). Experience in the early application of more sophisticated modelling suggested that there was good correspondence between guidelines from these classifications and modelling results and reality when rockmass behaviour was relatively simple, for example for RMR values between about 30–70 and with moderate stress levels; in other words where sliding and rotation of intact rock pieces essentially controlled the overall failure process. The database of experience on which the classifications had been built and the fact that there were literally hundreds and hundreds of kilometres of tunnels that had been successfully constructed on the sole basis of their application attest to this. However, consistently it was found that the classifications or for that matter simplistic modelling was less reliable for predicting excavation behaviour in squeezing or swelling ground or for cases with clearly defined structural failures or where spalling, slabbing and/or rockbursting was feasible under very high insitu stress conditions. It was also found difficult to apply the then available classification systems for the design of sequentially installed temporary reinforcement or for establishing the support required to control progressive failure in difficult tunnelling conditions, where for example, the excavation and installation of support is needed to be defined for a tunnel being advanced by a drill and blast operation involving top heading and benching. In such a system an intricate sequence of excavation, face support, installation of rockbolts, steel sets and/or shotcrete, support of the top heading while the bench is excavated, then the completion of the lower sidewalls and the invert and then installation of the final concrete lining (if one is required) all require design. In such a case, defining the proper sequence of excavation and establishing the timing of support installation is very critical elements of achieving a successful tunnel design. Describing such as case, Hoek et al. (2007) state …

…" *it is unacceptable to assume that an adequate design can be based upon a simple estimate of the final bolt pattern and lining thickness …"*

Needless to say though, there is in fact merit in undertaking more than one classification of a project rockmass, if not for any other reason than simply from the point of view of providing a redundancy check to ensure that one's classification estimates are not out of line. It has long been recognized (Carter 1992), that for example RMR is more readily applied when only core is available than Q, as there are elements of the joint system that are difficult to describe from core alone, estimating JN in particular is particularly problematic. Similarly it has long been established that RMR76 and GSI are interchangeable in the mid-range blocky rockmass-zone, defined in the quantitative paper, (Hoek et al. 2013) but that RMR is difficult to apply for discrimination between rockmasses of low general competence. This is an area where the GSI system excels in that it provides the descriptive controls allowing ready observational characterization that are perhaps missing from the other systems.



Note: The position of projected grey areas are indicative

Figure 10. Indicative example of how alteration (none to severe) affects the GSI.

At the upper end of the scale of rock competence it can be problematic discriminating characteristics of importance, for example, differentiating between different rock types from a brittle strain bursting perspective. Again this is an area within which the observational approach of GSI characterization can be useful. This is where the Q system and the RMR system sometimes have been found by practitioners to be difficult to apply to get the correct numbers that excavation performance, for example, might show. Doing more than one classification has value in that gaps in what might be termed "classification accuracy", can be filled and a proper spectrum of characteristics assigned for the rockmass into or upon which engineering structures are required.

3 METHODOLOGY FOR GSI APPLICATION

3.1 *Understanding geological reality*

Irrespective of whether one wishes to use the now available quantitative parameter scales as an aid to characterization or not, the very heart of the GSI classification and what makes it different from the other systems is that is requires a careful engineering geology description of the rockmass. In dealing with specific rockmasses it is suggested that the selection of an appropriate zone location on a GSI chart not be limited merely to checking the visual similarity with the sketches of the structure of the rockmass as they appear in the charts, but rather should be based on gaining real understanding by examining actual face exposures or outcrops. Just using the charts and even reading the associated descriptions carefully, is no substitute for examining conditions in situ so that the most suitable rockmass structure is chosen. The most appropriate case may well lie at some intermediate point between the limited number of rockmass sketches or descriptions included around the charts.

Outcrops, excavated slopes, tunnel faces and drill cores are the most common sources of information for the estimation of a GSI value for a rockmass, but these are not of equal calibre. Outcrops are an extremely valuable source of good observational data, particularly in the initial stages of a project, but like any surface exposure, they suffer from the disadvantage that relaxation and weathering may have significantly influenced the appearance of the exposed rockmass as compared with what might be met deep underground; but in the context of a surface problem, such insight may be very valuable for comparison to core to assess degree of weathering decomposition. Improved data on weathering depth and influence can sometimes be acquired (where permissible) by trial trenches or specific drilling. Judgment is however required in order to allow for weathering effects in assessing the most probable GSI value applicable at the depth of the proposed excavation.

Excavated slopes and exposed tunnel faces are probably the most reliable source of information for estimating GSI provided that these faces are related with the project. For design of tunnels for example, in hard strong rockmasses it is important that some appropriate allowance be made for damage due to mechanical excavation or blasting. As the purpose of estimating GSI values for such situations is to assign properties to the insitu undisturbed rockmass, failure to allow for effects of blast damage when assessing GSI will result in assigning values that may be too conservative. Therefore, attempts should be made in such circumstances to properly estimate the degree of disturbance, and thereby define D. This problem becomes less significant in weak and tectonically disturbed rockmasses as excavation then is generally carried out by more gentle mechanical means but also because the rocks are more ductile and are less influenced by blast damage. Hence, surface damage effects are negligible compared to the damage which already exists in the rockmass.

Drillhole cores of reasonable diameter provide one of the best sources for acquiring data from depth, but one has to be cognizant that it is necessary to extrapolate the very one-dimensional information provided by the core to the threedimensional insitu rockmass. However, this is a problem common to all borehole investigations, and almost all experienced engineering geologists are comfortable with this extrapolation process. Multiple boreholes and also inclined boreholes are of great help to interpretation of rockmass characteristics at depth.

For stability analysis of a slope, the evaluation should be based on the rockmass through which it is anticipated that a potential failure plane could pass. The estimation of GSI values in these cases requires considerable judgment, particularly when the failure plane can pass through several zones of different quality. Mean values may not be appropriate in such situations.

For tunnels, the index should be assessed for the volume of rock involved in carrying loads, e.g. for about one diameter around the tunnel in the case of overall tunnel behaviour or more locally in the case of an engineered structure such as an "elephant foot" at the end of lattice girders or steel rib elements.

For particularly sensitive or critical structures, such as underground powerhouse caverns, the data and information obtained from the sources discussed above may not be considered adequate, particularly as the design advances beyond preliminary stages. In these cases, the use of small exploratory tunnels can be considered, allowing better collection of reliable data. Remarkably, often when compared to multiple programs of deep drillholes this type of approach to data gathering has often been found cost effective.

4 APPLICATION WITHIN THE HOEK-BROWN CRITERIA

Use of GSI as a basic input parameter within the GSI system is straightforward throughout the range of possible description, but is more problematic at the two ends of the rock competence scale – weak soft rocks and high strength, high competence rocks. Figure 11 shows the range of applicability of GSI when described qualitatively and quantitatively.

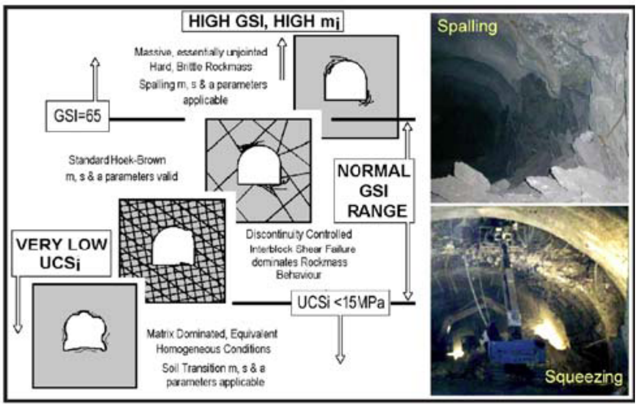


Figure 11. Zones of applicability of GSI and conventional Hoek-Brown equations with relation to transitions to brittle spalling criterion (top right) and (lower left) to Mohr-Coulomb criterion (with potential in tunnelling for squeezing, (photo, courtesy E. Hoek, from Carter et al. 2008).

In the upper quartile of the standard GSI chart rockmass conditions can be considered near intact, and descriptive techniques work best, so the chart with quantitative scales should not be used. For the weak, low competence rocks, which typically are trending ever closer to soils, the same basic precepts apply. In the middle, both charts work equally well.

4.1 *Conventional applicability range*

For the conventional range where rockmasses can be thought of as blocky, the qualitative and quantitative charts (Figures 1 and 4 respectively) both can be used for assessing GSI. Input of GSI values into the Hoek-Brown criterion is then straightforward, and depending on the scale of the engineering problem with respect to the rockmass scale (Figure 12) the appropriate *m*, *s* and *a* constants can be derived.

Getting the relative scale of the problem to the scale of the rockmass correct is the key factor in any application of GSI into the Hoek-Brown criterion. While there has been some criticism that GSI is only applicable to heavily jointed rockmasses, which at the scale of an engineering problem can be treated as homogeneous and isotropic, this limitation can be readily overcome with a little judgment.

For example, if there are weak discontinuities or a pervasive weak fabric throughout a rockmass, such as bedding or foliation, that would render the overall rockmass anisotropic, it would be recommended that that weak fabric be ignored in developing the global GSI and then it should be modelled for or analyzed for explicitly. In this regard even at a crude modelling scale, any such fabrics can be readily examined as a ubiquitous joint set within the overall Hoek-Brown material considered appropriate for characterizing the problem.

It is reasonable to extend this argument further and to suggest that, when dealing with large scale rockmasses, the strength will reach a constant value when the size of individual rock pieces is sufficiently small in relation to the overall size of the structure being considered. This suggestion is embodied in Figure 12, which schematically shows the transition from an isotropic intact rock specimen, through a highly anisotropic rockmass in which behaviour is controlled by one or two discontinuities, to again a fully isotropic rockmass, in this case heavily jointed.

The Hoek-Brown failure criterion, which assumes isotropic rock and rockmass behaviour, should only be applied to those rockmasses in which there are a sufficient number of closely spaced discontinuities that isotropic behaviour involving failure on these discontinuities can be assumed. Where the block size of the rockmass is of the same order as that of the engineering structure being analyzed, GSI and the Hoek-Brown criterion should not be used.

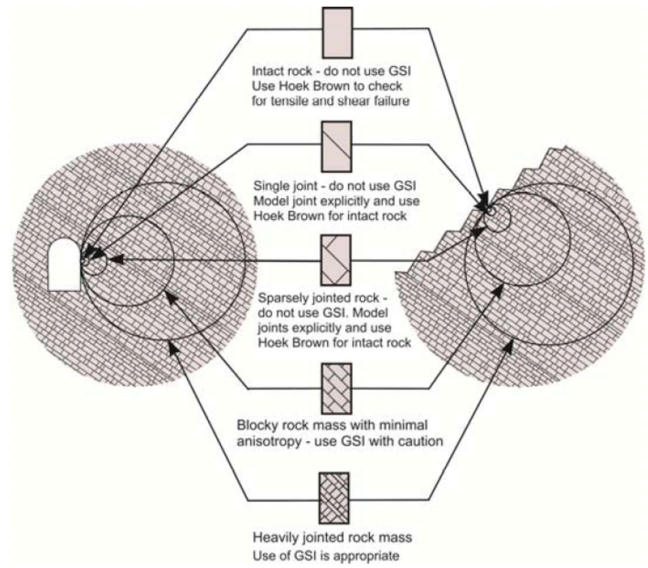


Figure 12. Limitations on the use of GSI depending on scale (Hoek et al, 2013)

Rather, the overall stability of the engineering structure itself should be analyzed by considering the behaviour of blocks and/or wedges defined by the intersection of geological structural features. When the slope or the underground excavation is large and the block size small in comparison, the rockmass can be treated as a Hoek-Brown material.

An example of this transition is frequently seen in large open pit mines in which individual structure or intersecting structural features control bench scale stability, while the overall slope can sometimes be considered as a homogeneous rockmass to which the Hoek-Brown criterion can be applied.

These basic differences in the various scales of structures of importance are apparent in the photograph in Figure 13 of the overall slopes of the more than 1000m deep Chuquicamata open pit mine in Chile. In this case many scales of structure are involved and for much of the mechanics, the rockmass can be considered as a Hoek-Brown material.

4.2 *Application at ends of Rock Competence Scale*

As it will be apparent from Figure 11 and previous discussions about the charts within Figures 1 and 4, because of their development, both GSI and the Hoek-Brown criterion admirably characterize most "normal" rockmasses from the viewpoint of their behaviour for rock excavations. While descriptive GSI spans the full range of rockmass characteristics, the governing equations describing a conventional Hoek-Brown material however run into difficulties when applied at the two ends of the rock competence scale. This is largely because incipient strength and block size is such that rockmass behaviour in these domains tends not to be controlled by inter-block shear strength, but rather by material strength. Three rockmass competence regimes can thus be postulated as shown in Figure 11.

Figure 13. Examples of different scales of fabric - stability of individual benches clearly controlled by intersecting structural features while the overall slope can be characterized as a Hoek-Brown materials on which major faults and shear zones must be superimposed (Hoek, pers comm)

As also illustrated in Figure 11, the behaviour of the rockmass is different in these three zones, and thus it is suggested that use of the conventional transfer equations from GSI values to the *m*, *s* and *a* parameters for the Hoek-Brown criterion equations be restricted to the central range of applicability shown in Figure 11, i.e. above GSI=30 and below GSI = 65. At the low end of the rock competence scale (where σ_{ci} <15MPa and GSI <30), it can be taken that discontinuities play less of a role and rockmass strength tends towards matrix strength. Similarly, at the high end of the scale (where GSI>65, m_i>15), because discontinuities are now widely spaced, block size becomes so significant that once again, intact material behaviour rather than the fracturing becomes dominant in controlling rock mass strength.

At the high end of the scale, in competent hard rock, particularly at great depth (e.g. 1,000m or more), rockmass structure is often so tight that the mass behaviour again approaches that of the intact rock. These conditions can be modelled using the original Hoek-Brown equation directly applied for the intact rock (Hoek 1983) as rockmass behaviour can now be considered that of intact rock (i.e. equivalent GSI = 100). For example, in the case of the Drakensberg and Ingula projects (Hoek, pers comm), where rockmass behaviour was controlled by intact rock strength, (since GSI for the sandstones and mudstones was equal to 100) the through going major horizontal bedding planes could then each be modelled explicitly.

Where the rockmass is more brittle and spalling is an important issue, consideration needs to be given to brittle fracture initiation which occurs at about 40% of σ_{ci}. In this case the non-linear relationship between brittle spalling initiation and confining stress can still be represented (at least as well as any other criterion) by the Hoek-Brown equation with adjusted parameters. In this situation, one is towards the top of the GSI charts and absolute GSI values likely meaningless (Hoek, pers. comm.). Here the failure process that controls stability of underground excavations under these conditions is dominated by brittle fracture initiation and propagation, which leads often to spalling, slabbing and, rock-bursts in extreme cases. Considerable research effort has been devoted in recent years to the study of these brittle fracture processes, and a specific spalling criterion developed to characterize rockmass behaviour under such conditions. The paper by Diederichs, et al. (2004) provides a useful summary of much of this work, while that by Diederichs et al. (2010) outlines practical application methods for spalling criterion application for tunnelling problems.

At the other (low) end of the rock competence scale, again the rockmass structure has diverged from a blocky fabric to something much closer to an intact material – in this case a soil. In this regime cohesive strength becomes more important and thus the traditional Mohr-Coulomb criterion has more applicability. It is possible however to extend the applicability of the Hoek-Brown criterion to cover both these ends of the competence scale if use is made of the two transition equations suggested by Carter et al., 2008. These two transition relationships allow continued use of the basic Hoek-Brown strength criterion concept out into these both end domains. Figure 14 illustrates the behaviour of these transition functions with respect to the conventional Hoek-Brown and Mohr-Coulomb relationships, as a function of GSI, while Figure 15 summarizes the transition relationships. It should be noted that since the initial publication of these low and high end transition relationships by Carvalho et al. (2007) and by Diederichs et al. (2007) respectively, several minor modifications and improvements have been made by other authors, e.g. Castro et al. (2013).

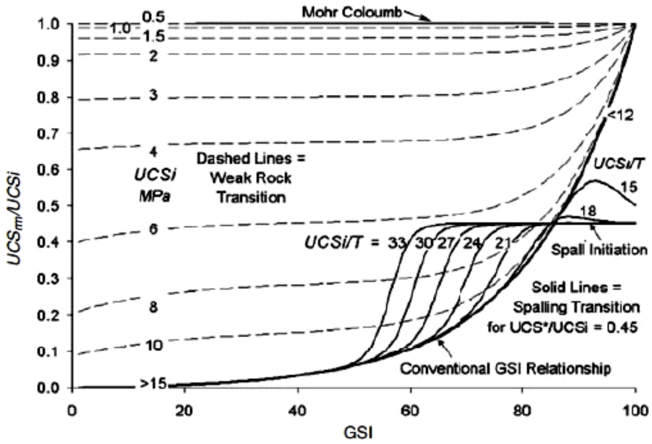


Figure 14. Normalized rockmass strength (σ_{cm}/σ_{ci}) as a function of rock quality (GSI), illustrating marked differences between conventional Hoek-Brown behaviour and transition functions. Spall transition threshold set at typical value, UCS*=0.45σ_{ci}) (Carter et al. 2008)

5 GSI LIMITATIONS

GSI classification is based upon the assumption that the rockmass contains a sufficiently large number of intersecting discontinuities that it can be considered to behave as an isotropic mass (Figures 11 and 12). The GSI system therefore should not be applied for rockmasses with clearly defined dominant structural fabrics. Undisturbed slate is an example of a highly anisotropic rockmass, where applying a GSI value should only be done if the mode of potential failure won't be governed by shear strength of the incipient discontinuities within the slate. Of importance though, in the confined conditions of a typical tunnel situation it is possible that stress dependent effects may provoke very different degrees of deformations dependent on location around the periphery of the tunnel as a result of such anisotropy. An approach to relate such anisotropy in deformations with GSI is described in Fortsakis et al. 2012. For rockmasses with completely sheared structure, such as that shown in the sixth (very last) row of the GSI chart in Figure 1, anisotropy is not an issue as the difference in strength between the rock and the discontinuities within it will likely be small.

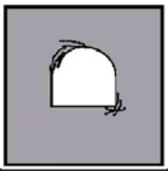
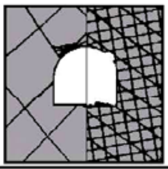
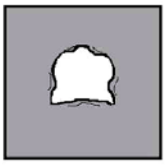
SPALLING CONDITIONS Massive, essentially unjointed Hard, brittle Rock Masses m_i>15 GSI>65		$m_{SP} = s_{SP}(UCSi/\lvert T\rvert)$ $s_{SP} = (UCS*/UCSi)^{1/a_{SP}}$ $a_{SP} = 0.25$ (peak) & $= 0.75$ (residual) and $X_{TRANS} = X_{GSI} + (X_{GSI} - X_{SP}) f_{SP}$ where $f_{SP} = \frac{1}{1+e^{100\left(2+\frac{D}{5}-\left(\frac{GSI}{60}\right)^{1/3}-\left(\frac{UCSi}{34T}\right)^{1/5}\right)}}$
NORMAL FRACTURED ROCKMASS CONDITIONS Classic Hoek-Brown & Classifications Applicable m_i<15 full GSI range m_i>15 GSI<65		$\sigma_1 = \sigma_3 + UCSi\left(m_b \frac{\sigma_3}{UCSi} + s\right)^a$ $\frac{m_b}{m_i} = e^{\frac{GSI-100}{28-14D}}$; $s = e^{\frac{GSI-100}{9-3D}}$ $a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$
SQUEEZING CONDITIONS Matrix Dominated Equivalent Homogeneous Soil-like Rock Mass UCSi < 10–15MPa		$m_b^* = [m_b + (m_i - m_b) f_T(\sigma_{ci})]/(4a^* - 1)$ $s^* = s + (1-s) f_T(\sigma_{ci})$ $a^* = a + (1-a) f_T(\sigma_{ci})$ where $f_T(\sigma_{ci}) = \begin{cases} 1, & UCSi \le 0.5MPa \\ e^{\frac{-(UCSi-0.5)^2}{25}}, & UCSi > 0.5MPa \end{cases}$

Figure 15. Suggested Applicability Ranges and suggested Transition and Conventional Hoek-Brown Relationships (from Carter et al. 2008); (where *UCSi* = σ_{ci} = Uniaxial Compressive Strength; *T* = σ_i = Tensile Strength; *UCS** = crack initiation threshold strength; and where f_{SP} and $f_T(\sigma_{ci})$ are the originally suggested Spalling and Weak rock transition functions)

GSI values should thus not be indiscriminately estimated and used in the conventional Hoek-Brown equations for rockmasses that tend towards either the upper or lower end of the rock competence scale.

When rocks such as marls, claystones, siltstones and weak sandstones are deposited and remain through diagenesis in stable environments, with little tectonism, they can often be found as quite massive looking rocks with few discontinuities. In such rocks, even when bedding planes exist they often do not appear as clearly defined discontinuity surfaces. In such cases, the GSI chart can still be used, but care needs to be taken in defining the discontinuity spacing range, based on observed conditions. The character of the discontinuities needs to be carefully, defined, as if they exert control on blockiness and fabric, then it is likely that rockmass quality cannot be much better than fair (usually fair or poor) and hence GSI values should be in the range of 40–60. If the discontinuities do not appear to exert any block size or fabric control, such rocks form continuous masses with very few to no discontinuities evident. For such cases the GSI of the rockmass is essentially 100 and as such, the rockmass can be treated as an intact rock material with engineering parameters given directly by laboratory testing. This situation can be handled by the low end transition function as included in Figure 15, but with any specific discrete discontinuities treated explicitly.

At the upper end of the rock competence range, again care must be taken in assigning GSI values and ensuring that these are not indiscriminately used within the conventional Hoek-Brown equations. The upper end transition equations allow one to deal with the initiation of fracture in hard brittle rocks. The conventional equations do not deal with initiation of spalling failure, without modification, but they can be used for modelling failure of already damaged rock and hence will give an acceptable prediction of damaged strength for hard brittle rocks.

For numerical analysis of a rockmass containing a single well-defined shear zone or fault or other type of similar major geological structure, it is sometimes appropriate to model the overall rockmass as a Hoek–Brown material and superimpose the specific discontinuity as a significantly weaker element. In this case, the GSI value assigned to the rockmass should ignore the single major discontinuity. The properties of this particular discontinuity may fit the lower portion of the GSI chart or it may require a different approach such as laboratory shear testing of soft clay fillings in order to model its influence.

Last, but not least, care must be taken to ensure that adequate attention is given to defining a tensile strength cut-off for rocks with low m_i, values, or else the Hoek-Brown envelopes will be unrealistic. This is the range where the low-end transition equations might be better applied. Another possibility is to use the basic Hoek-Brown relationship but incorporating a tension cut-off, which is easy to implement in most modelling codes.

6 CONCLUSIONS

Use of the GSI rockmass classification system and the associated *m*, *s* and *a* parameter relationships linking GSI with the Hoek-Brown failure criterion provides a proven, effective and reliable approach for strength prediction for surface and underground excavation design and for rock support selection for most "normal" rockmasses:

- with intact rock material strength, σ_{ci} > 5–10MPa,

and

- with matrix yield behaviour dominated by shear mechanisms
  for (m_i < 15) for the full GSI range and
  for (m_i > 15) for GSI < 65.

The observational GSI approach of making visual assessments of rockmass conditions and assigning appropriate quality on charts such as Figure 1 seems to work very effectively, not just for the original range of applicability of the Hoek-Brown criterion for the "normal range of fractured rockmasses" but also for describing rockmasses at either end of the rock competence scale. The quantitative GSI chart however should not be used outside the original blocky rockmass range.

Rockmass characterization has an important role in the future of engineering geology in extending its usefulness, not only to define a conceptual model of the site geology, but also for the quantification needed for analyses to ensure that the idealization (for modelling) does not misinterpret actuality (Knill 2003). If GSI rockmass characterization is carried out in conjunction with careful numerical modelling, together they can provide more insight on ground behaviour than either approach would on its own (Chandler et al. 2004). Use of GSI has considerable potential in rock engineering because it permits the multiplicity of fabrics within a given rockmass to be quantified, thereby enhancing geological logic and reducing engineering uncertainty. The use of the GSI system allows the influence of variables, which make up a rockmass, to be assessed and hence the behaviour of the rockmass to be explained more clearly.

One of the key advantages of the index is that the geological reasoning it embodies allows adjustments to be made of its ratings to cover a wide range of rockmasses and conditions, but also allowing some understanding to be gained on applicability limits. Having become familiar with the GSI system most people have no difficulty in using it for describing the complete range of rock characteristics including better and worse rockmasses than the blocky midrange for which the original Hoek-Brown criterion was initially formulated.

The qualitative GSI chart provides an intelligent tool for the user to rapidly assess the numerous key combinations of rockmass characteristics that truly influence the strength and deformation properties that might control design of any structure on or within that rockmass.

As a system, visual GSI characterization linked with Hoek-Brown strength determination as a basis for modelling has been widely adopted by engineers and geologists involved in design and construction of structures in or on rock. One reason is that there are very few practical alternatives. There are quite a number of rockmass classification systems and rules of thumb that can be applied for estimating tunnel support, with several of them developed specifically to provide first order estimates of final tunnel rock support, foundation bearing capacity or rockslide risks. Some of these classifications have been pressed into service to provide estimates of rockmass strength and deformation modulus but that was never the original aim of their developers. The GSI approach has been solely developed for parameter estimation, and this is the key other reason that its use for input into the Hoek-Brown failure criteria, both empirical in concept, have been proved to work so well. Analytic or practical back-analyses of rockmasses where it has been applied are continuously certifying this.

In spite of criticisms of the qualitative nature of the original GSI classification, it appears to work well within the hands of experienced observers. With the advent of a parallel, more quantified chart, it is hoped that those who felt uncomfortable with the solely qualitative visual approach will be able to more readily ascribe appropriate GSI's for their projects. There is no question than GSI should only be expressed as a range, as rockmasses by their very nature are variable. As such, the expectation must be that GSI characterization will produce approximate answers only, but when these answers are refined as a project typically progresses through its various stages the final solutions are generally credible and robust. We are still a long way perhaps from being able to completely define a rockmass, but we have come a long way towards this goal, as expressed several years ago by Hoek … … *«My long term hope is that numerical tools such as the Synthetic Rock Mass and its off-shoots will eventually enable us to replace classification type approaches or at least to calibrate these classifications. It may be a while before these hopes can be realized».*

ACKNOWLEDGEMENTS

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6.1 *References*

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Keynote Lecture

ΝΕΑ ΑΠΟ ΤΙΣ ΕΛΛΗΝΙΚΕΣ ΚΑΙ ΔΙΕΘΝΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΝΩΣΕΙΣ

6th ISRM Online Lecture

The sixth ISRM Online Lecture was broadcasted on 23 July, at 9 a.m. GMT. It was delivered by Prof. Herbert Einstein, MIT, USA and the title will be "Puzzles in rock – Have they been solved?". The lecture can be downloaded from http://www.isrm.net/gca/index.php?id=1148

The previous ISRM Online Lectures were given by Prof. Wulf Schubert, Prof. John Hudson Dr. Pierre Dufaut, Prof. Eduardo Alonso and Dr. John Reed. All the ISRM Online Lectures will remain available on the ISRM website in this dedicated webpage http://www.isrm.net/gca/?id=1104

☙ ❧

MUIR WOOD LECTURE 2014
WTC 2014, Iguassu, Brasil, 16 juin 2014

Some Critical Aspects of Subaqueous Tunnelling

Georgios Anagnostou - ETH Zurich, Switzerland

High potential for damage, relatively high pore pressures and limited pre-construction accessibility are all features of subaqueous tunnels. Potential hazards include high water inflows or even a complete flooding of the tunnel in the case of a connection opening up to the seabed. In subaqueous tunnels, very high pore pressures may occur at small depths of cover, i.e. often in combination with a low shear strength ground, resulting in particularly adverse effects in terms of stability and deformations of the opening. This lecture illustrates some of the geomechanical issues relating to subaqueous tunnels (face stability in fault zones, the limits of open mode TBM operation in weak sedimentary rocks and the effect of advance drainage in squeezing ground) with reference to five case studies – the Storebælt tunnel, the "Melen 7" Bosphorus tunnel, the Lake Mead Intake No 3 tunnel, the Zurich Cross Rail and the future Gibraltar Strait tunnel project.

http://www.ita-aites.org/fr/news/1052-muir-wood-lecture-2014

MUIR WOOD LECTURE 2015

Στη συνάντηση του Iguassu, η ΕΕΣΥΕ είχε προτείνει ως υποψήφιο για την Muir Wood Lecture 2015 στο Dubrovnik, στα πλαίσια του WTC2015, τον καθηγητή Παύλο Μαρίνο. Ο κος Μαρίνος είχε προτείνει ως θέμα διάλεξης "Engineering Geology for tunneling in difficult ground". Κατά τη σχετική ψηφοφορία στη συνεδρίαση που έλαβε χώρα στη Βραζιλία, τα αποτελέσματα ήταν Lunardi:10, Marinos:8, Dix:4, Lykke:2. Σημειώνουμε την ιδιαίτερα θετική αποδοχή του κ. Μαρίνου, δεδομένου ότι ο πλειοψηφών κ. Lunardi είχε προταθεί επί τρία συνεχή χρόνια από την Ιταλία καθώς και το γεγονός ότι η αποδοχή δύο Ελλήνων επί δύο συνεχόμενα έτη για τη διάλεξη αυτή δημιουργούσε άλλα προβλήματα και συνειρμούς σε μια κοινότητα που φροντίζει και τις ισορροπίες.

Το Bergen της Νορβηγίας υπερίσχυσε στις ψηφοφορίες των άλλων δύο υποψηφίων (Παρίσι και Άμστερνταμ) και ανέλαβε τη διοργάνωση του WTC 2017. Το θέμα του συνεδρίου: "Surface problems – Underground solutions".

(από «Το Δελτίο των Σηράγγων», Τεύχος 01 – 2014, Ιούνιος 2014, ΕΕΣΥΕ)

THE ITA PHOTO CONTEST

At the occasion of its 40th anniversary,
The International Tunnelling and Underground Space Association
organized a photo contest. See the winning pics below :

During the WTC in Iguaçu, the winner for each category of photos has been voted by participants. Please find in the following link the photos that won (http://www.ita-aites.org/en/?option=com_content&view=article&id=676&catid=233&Itemid=990).

IN TUNNELS AND UNDERGROUND SPACES IN OPERATION

Turin Metro in Operation by s210173

IN TUNNELS AND UNDERGROUND SPACES IN CONSTRUCTION

TBM breakthrough by batuhan nazar salihoglu

IN SITE PERSONNEL

Coming through by Lillejord

IN BLACK & WHITE PHOTOS

Cutterhead of TBM Tonda by Jan Tatar

Ελληνική Επιτροπή Σηράγγων και Υπογείων Έργων (Ε.Ε.Σ.Υ.Ε.)
Μέλος της International Tunnelling Association (I.T.A.)
www.eesye.gr1

ΑΠΟΤΕΛΕΣΜΑΤΑ ΕΚΛΟΓΩΝ ΚΑΙ ΣΥΓΚΡΟΤΗΣΗ ΤΟΥ ΝΕΟΥ Δ.Σ.

Στη Γενική Συνέλευση της ΕΕΣΥΕ την 28η Απριλίου 2014 για την ανάδειξη νέου Διοικητικού Συμβουλίου και νέας Εξελεγκτικής Επιτροπής ψήφισαν συνολικά 47 ταμειακώς εν τάξει μέλη. Σύμφωνα με την καταμέτρηση των ψήφων, που διενήργησε η Εφορευτική Επιτροπή (Χρήστος Τσατσανίφος και Βασίλης Βαντόλας), έλαβαν:

Για το Διοικητικό Συμβούλιο:

1. Αλιφραγκής Δημήτρης	13
2. Γιούτα – Μήτρα Παρασκευή	24
3. Θανόπουλος Ιωάννης	14
4. Μπακογιάννης Ιωάννης	41
5. Περγαντής Ευάγγελος	14
6. Προυντζόπουλος Γεώργιος	21
7. Ραπτόπουλος Σταύρος	27
8. Ραχανιώτης Νικόλαος	14
9. Τσιφουτίδης Γεώργιος	16
10. Φίκιρης Ιωάννης	16

Για την ΕΕΕ της ΕΕΣΥΕ:

1. Γεωργίου Δημήτριος	23
2. Νικολάου Δημήτριος	28
3. Ντουνιάς Γεώργιος	34
4. Φορτσάκης Πέτρος	18

Λόγω ισοψηφίας των μελών Θανόπουλου, Περγαντή και Ραχανιώτη, διενηργήθη κλήρωση από την οποία προέκυψε η σειρά κατάταξης μεταξύ των τριών: 1. Περγαντής, 2. Ραχανιώτης, 3. Θανόπουλος.

Το Διοικητικό Συμβούλιο συγκροτήθηκε σε σώμα στη συνεδρίαση της 13.05.2014 ως εξής:

Πρόεδρος:	Στ. Ραπτόπουλος
Αντιπρόεδρος:	Π. Γιούτα-Μήτρα
Γενικός Γραμματέας:	Γ. Τσιφουτίδης
Ταμίας:	Ι. Φίκιρης
Μέλη:	Ι. Μπακογιάννης
	Γ. Προυντζόπουλος
	Ευ. Περγαντής

Από τα Μέλη ο Γ. Προυντζόπουλος ανέλαβε την εποπτεία της νεο-συσταθείσας Ομάδας Νεαρών Μελών - Young Members' Group, και ο Ι. Μπακογιάννης την εποπτεία έκδοσης του Δελτίου.

ΠΡΟΣΕΧΕΙΣ ΕΚΔΗΛΩΣΕΙΣ ΓΕΩΤΕΧΝΙΚΟΥ ΕΝΔΙΑΦΕΡΟΝΤΟΣ ΣΤΗΝ ΕΛΛΑΔΑ

GREEK TUNNELLING SOCIETY YOUNG MEMBERS GROUP

2nd EASTERN EUROPEAN TUNNELLING CONFERENCE
Tunnelling in a challenging environment

Ημερίδα Σταδιοδρομίας για Επαγγελματίες Υπογείων και Γεωτεχνικών Έργων

Στο πλαίσιο του συνεδρίου 2nd Eastern European Tunnelling Conference (EETC2014) που διοργανώνεται από την ΕΕΣΥΕ στην Αθήνα (28 Σεπτεμβρίου - 1 Οκτωβρίου 2014), συμπεριλήφθηκε, με πρωτοβουλία της «Ομάδας Εργασίας Νέων Μελών» της Ελληνικής Επιτροπής Σηράγγων και Υπογείων Έργων* η εκδήλωση «**Ημερίδα Σταδιοδρομίας για Επαγγελματίες Υπογείων Έργων**».

Τι είναι η ημερίδα σταδιοδρομίας;

Στόχος της εκδήλωσης είναι η διασύνδεση υποψηφίων επαγγελματιών με γνώση ή/και εμπειρία στο χώρο των υπογείων έργων με εταιρείες από την Ελλάδα και την Ευρώπη με σημαντική δραστηριότητα στο συγκεκριμένο χώρο. Πιο συγκεκριμένα, η Ημερίδα θα περιλαμβάνει (α) την παρουσίαση του προφίλ των εταιριών που συμμετέχουν, των έργων στα οποία δραστηριοποιούνται και των προοπτικών σταδιοδρομίας που προσφέρουν και (β) συζήτηση μεταξύ των υποψηφίων και των εκπροσώπων των εταιρειών.

Πότε και που διοργανώνεται;

Η Ημερίδα θα πραγματοποιηθεί στην Αθήνα, στο ξενοδοχείο Athens Royal Olympic Hotel όπου διοργανώνεται και το συνέδριο EETC2014, την **Κυριακή 28 Σεπτεμβρίου 2014 (14:00 έως 19:00)**.

Σε ποιόν απευθύνεται;

Η Ημερίδα απευθύνεται σε μηχανικούς και επαγγελματίες κάθε ηλικίας με αποδεδειγμένη ειδική εκπαίδευση ή/και εργασιακή εμπειρία στον τομέα των υπογείων και γεωτεχνικών έργων (ενδεικτικά και όχι περιοριστικά αναφέρονται πολιτικοί μηχανικοί, μεταλλειολόγοι, γεωλόγοι, τοπογράφοι).

Πως μπορώ να συμμετάσχω;

Για τη δήλωση συμμετοχής απαιτείται η αποστολή email στην ηλεκτρονική διεύθυνση **ymg.gts@gmail.com** με συνημμένα (α) βιογραφικό σημείωμα στα αγγλικά και (β) συνοδευτική επιστολή / cover letter στα αγγλικά. Στην εκδήλωση θα υπάρχει συγκεκριμένος αριθμός συμμετεχόντων και θα τηρηθεί σειρά προτεραιότητας. Για το λόγο αυτό παρακαλούνται οι ενδιαφερόμενοι να υποβάλουν άμεσα δήλωση συμμετοχής.

Που μπορώ να βρω επιπλέον πληροφορίες;

Website του συνεδρίου EETC2014:
http://www.eetc2014athens.org

Facebook group «Ομάδας Εργασίας Νέων Μελών»:
www.facebook.com/groups/YMGGTS

Twitter «Ομάδα Εργασίας Νέων Μελών»:
@YMG_GTS

LinkedIn Group «Ομάδας Εργασίας Νέων Μελών»:
YMG - GTS

Για περισσότερες πληροφορίες και εγγραφή όσο αφορά στην Ομάδα Εργασίας Νέων Μελών της ΕΕΣΥΕ μπορείτε να απευθυνθείτε στην ηλεκτρονική διεύθυνση **ymg.gts@gmail.com.*

*** ***

2nd EASTERN EUROPEAN TUNNELLING CONFERENCE
Tunnelling in a challenging environment
ATHENS 30 September - 3 October 2014 Royal Olympic Hotel Athens, Greece

30 September - 3 October 2014, Athens, Greece
www.eetc2014athens.org

It is our pleasure to inform you that the Greek Tunnelling Society is organizing the 2ndEastern European Tunnelling Conference in Athens on September 28 – October 1 2014 (EETC2014, Athens).

The Eastern European Tunnelling Conference is a biennial regional traveling conference. It aims to promote the sharing of knowledge, experience, skills, ideas and achievements in the design, financing and contracting, construction, operation and maintenance of tunnels and other underground facilities among the countries of Eastern Europe, on an organized basis and with agreed aims. EETC2014 aims mainly to bring together colleagues from Eastern Europe but people from the rest of the world are also welcome.

The theme of EETC2014 Athens is:

"Tunnelling in a Challenging Environment"
Making tunnelling business in difficult times

The construction of underground projects is becoming increasingly demanding as new challenges are emerging in every aspect and sector of this multidisciplinary and multivarious business. Further to the usual geological, geotechnical, structural and operational challenges, we are now facing a difficult business and financial environment, which requires the deployment of even more intelligent and effective tools and solutions.

I really do hope that the EETC2014 Athens will contribute and further facilitate the growth of the tunnelling business and will be a forum for scientific and professional collaboration.

TOPICS:

- Innovative methods for Analysis and Design
- Tunnelling in difficult ground conditions
- Conventional urban or shallow tunnelling
- Mechanized tunnelling
- Hydraulic tunnels

- Underground complexes
- Caverns for Hydropower or Storage
- Pipe jacking and microtunnelling
- Innovations in tunnelling construction technology
- Tunnels and shafts for mining
- Rehabilitation and repair
- Safety and security in tunnels and tunnelling
- Contractual and financial issues
- Education and training
- Case histories
- Underground space use
- Tunnels and monuments

7ο ΠΑΝΕΛΛΗΝΙΟ ΣΥΝΕΔΡΙΟ ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

5 – 7 Νοεμβρίου 2014, ΑΙΓΛΗ Ζαππείου, Αθήνα
http://www.7hcge2014.gr

Η Ελληνική Επιστημονική Εταιρεία Εδαφομηχανικής και Γεωτεχνικής Μηχανικής, στο πλαίσιο των δραστηριοτήτων της, διοργανώνει το 7ο Πανελλήνιο Συνέδριο Γεωτεχνικής Μηχανικής υπό την αιγίδα του Δήμου Αθηναίων και του Τεχνικού Επιμελητηρίου Ελλάδας. Στόχος του Συνεδρίου είναι να καταγράψει τις προόδους της γεωτεχνικής μηχανικής στην Ελλάδα του 21ου αιώνα όπως αντικατοπτρίζονται στα σημαντικά γεωτεχνικά αλλά και άλλα έργα (σιδηροδρομικά, οδοποιίας, λιμενικά, υδραυλικά, κτιριακά, περιβαλλοντικά) με σημαντικό γεωτεχνικό αντικείμενο, που έχουν μελετηθεί και κατασκευαστεί ή κατασκευάζονται, καθώς και στα αποτελέσματα της ερευνητικής δραστηριότητας των ελληνικών πολυτεχνείων και πολυτεχνικών σχολών. Επιδίωξη είναι οι εργασίες του Συνεδρίου να αναδείξουν πρωτότυπα στοιχεία συμβολής της γεωτεχνικής μηχανικής αλλά και να προβάλουν θεωρητικές και πειραματικές έρευνες σε εδαφικά, βραχώδη και ημιβραχώδη υλικά που βρήκαν ή μπορούν να βρουν εφαρμογή στην πράξη."

Θεματικές Ενότητες

1. Συμπεριφορά Εδαφών: Έρευνες Υπαίθρου και Εργαστηρίου
2. Συμπεριφορά Εδαφών: Προσομοιώματα
3. Επιφανειακές και Βαθειές Θεμελιώσεις
4. Αλληλεπίδραση Εδάφους - Κατασκευής
5. Πρανή - Κατολισθήσεις
6. Βαθειές Εκσκαφές - Αντιστηρίξεις
7. Σήραγγες
8. Βελτιώσεις Εδαφών
9. Φράγματα, Άοπλα Επιχώματα
10. Οπλισμένα Επιχώματα
11. Εφαρμογή Ευρωκωδίκων
12. Εφαρμογές Γεωσυνθετικών Υλικών
13. Εδαφοδυναμική / Τεχνική Σεισμολογία
14. Βραχομηχανική
15. Περιβαλλοντική Γεωτεχνική
16. Ενεργειακή Γεωτεχνική (energy geotechnics)
17. Πολιτιστική Κληρονομιά και Γεωτεχνική Μηχανική
18. Διδασκαλία και Μάθηση Γεωτεχνικής Μηχανικής

ΠΡΟΣΕΧΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΚΔΗΛΩΣΕΙΣ

Για τις παλαιότερες καταχωρήσεις περισσότερες πληροφορίες μπορούν να αναζητηθούν στα προηγούμενα τεύχη του «περιοδικού» και στις παρατιθέμενες ιστοσελίδες.

Second European Conference on Earthquake Engineering and Seismology, 24-29 August 2014, Istanbul, Turkey www.2eceesistanbul.org

TC204 ISSMGE International Symposium on "Geotechnical Aspects of Underground Construction in Soft Ground" - IS-Seoul 2014, 25-27 August 2014, Seoul, Korea, csyoo@skku.edu

ACESD 2014 International Conference on Advances in Civil Engineering for Sustainable Development, 27-29 August 2014, Nakhon Ratchasima, Thailand, http://acesd.sut.ac.th/index.php?acesd=9c847ec878ac085f8c0c829a241d5a35

International Symposium on Geomechanics from Micro to Macro (TC105), 01 - 03 September 2014, Cambridge, United Kingdom, http://is-cambridge.eng.cam.ac.uk

International Conference on Industrial and Hazardous Waste Management "CRETE 2014", September 2nd – 5th, 2014, Chania, Crete, Greece, http://www.hwm-conferences.tuc.gr

Geosynthetics mining solutions 2014, September 8 – 11, 2014, Vancouver, Canada, http://www.geosyntheticssolutions.com

JUBILEE CONFERENCE 50th Anniversary of Danube-European Conferences on Geotechnical Engineering Geotechnics of Roads and Railways, 9 - 11 September 2014, Vienna, Austria, www.decge2014.at

IAEG XII CONGRESS Torino 2014 Engineering Geology for Society and Territory, IAEG 50th Anniversary, September 15-19, 2014, Torino, Italy, www.iaeg2014.com

Reclaim - 1ο Πανελλήνιο Συνέδριο για την Εξόρυξη Αποβλήτων, 16 Σεπτεμβρίου 2014, http://us3.campaign-archive1.com/?u=234903e30e14c301e8f4d6547&id=7108cb488a&e=e6374bee25

10th International Conference on Geosynthetics – 10ICG, Berlin, Germany, 21 – 25 September 2014 www.10icg-berlin.com

14th International Conference of the International Association for Computer Methods and Advances in Geomechanics (14IACMAG), September 22 – 25, 2014, Kyoto, Japan, www.14iacmag.org

14th World Conference of the Associated Research Centers for the Urban Underground Space (ACUUS 2014), September 24-26, 2014, Seoul, Korea http://acuus2014.com

EETC 2014 ATHENS 2nd Eastern European Tunnelling Conference, 28 September - 1 October 2014, Athens, Greece, www.eetc2014athens.org

7th Basements and Underground Structures Conference, 30 September – 1 October 2014, London, geevents@emap.com

5th International Forum on Opto-electronic Sensor-based Monitoring in Geo-engineering (5th OSMG-2014), Oct 12-14, 2014, Nanjing, China, http://www.osmg2014.com

International Congress Tunnels and Underground Space risks & opportunities, 13-15 October 2014, Lyon, France, www.congres.aftes.asso.fr/en/content/invitation

Building on Recent Development Progress
13 to 15 October 2014, Villa Erba, Cernobbio, Italy
www.hydropower-dams.com/pdfs/hydro2014.pdf

The annual events organized by Hydropower & Dams have become the world's most important gatherings held in Europe for the hydropower profession. HYDRO 2013, which took place in Innsbruck, Austria, last year, was the largest and most international event for the hydro profession in 2013, with more than 1450 participants representing 85 countries.

The main emphasis each year is on encouraging the advancement of carefully planned hydropower plants in the less developed countries of Africa and Asia, where there is vast unexploited potential, and the greatest need for new capacity.

HYDRO 2014 Mission

Policy makers are today taking a much more positive and balanced view of the role and benefits of hydropower. The IEA predicts a doubling of hydropower capacity and production by 2030, and current trends indicate that this is achievable. Vast regional projects (such as Inga) are taking off in Africa, and large-scale schemes continue in Asia. HYDRO 2014 will build on the tremendous progress which has been made in the planning and implementation of large and small hydro schemes, pumped-storage projects, and marine energy systems over the past few years.

Session 1: Policies, developments and priorities - Africa
Session 2: The structuring of hydropower projects in developing countries
Session 3: Civil works - Materials and design
Session 4: Hydraulic machinery – Part I
Session 5: Policies, developments and priorities - Europe
Session 6: The perception and management of risk
Session 7: Civil works: monitoring and rehabilitation
Session 8: Hydraulic machinery – Part II
Session 9: Policies, developments and priorities - Asia
Session 10: Flood discharge works
Session 11: Social aspects

Session 12: Maximizing the benefits of hydropower
Session 13a: Pumped storage - Case studies
Session 13b: Pumped storage - Integration of storage and renewable energy technologies
Session 13c Pumped storage: Panel Discussion
Session 13d Pumped storage: Latest developments in equipment performance and O&M
Session 14: Small hydro - Hydrology, machinery and rehabilitation
Session 15: Environmental aspects
Session 16: Civil works - challenging sites and tunnels
Session 17: Small hydro - civil works and retrofitting
Session 18: Hydropower and fish
Session 19: Dam Safety
Session 20: Hydro plant rehabilitation and refurbishment
Session 21: Hydro expertise: ensuring a legacy for the next generation
Session 22: Project implementation and contractual aspects
Session 23: Hydro plant rehabilitation and refurbishment
Session 24: Sedimentation management
Session 25: Electrical engineering
Session 26: Operation and maintenance
Session 27: Hydrology and flood management
Session 28: Hydro plant management
Session 29: Ensuring the safety of penstocks
Session 30: Hydropower and cultural heritage
Session 31: The design and safety of hydraulic gates
Session 32: Valuing hydropower services
Session 33: Turbine flow measurement

For further details of the programme, please contact: Mrs Margaret Bourke at: Hydropower & Dams, PO Box 285, Wallington, Surrey SM6 6AN, UK.
Tel: + 44 (0)20 8773 7244 • Fax: + 44 (0)20 8773 7255
Email: hydro2014@hydropower-dams.com

ARMS 8 - 8th ISRM Rock Mechanics Symposium, 14-16 October 2014, Sapporo, Japan www.rocknet-japan.org/ARMS8/index.htm

The nature of construction has changed over the past 30 to 40 years, primarily in Singapore but also Worldwide. Moving towards new frontiers, great deal of construction now involves deep basement and other forms of underground construction such as caverns, tunnels, subway constructions, power plants, waste repositories, underground cities and "underground space creation" activities.

One of the biggest challenges faced by geotechnical experts is related to the highly non-uniform nature of the ground and the complex behavior of the soil. Ground improvement plays a major role in geoengineering and earthwork construction projects of many types in preceding years. The use of ground improvement has increased significantly in recent decades owing to new construction sites increasingly being located in areas of complex ground conditions.

The next big challenge revolves around balancing risk and cost. The impact of geotechnical risk is well understood by most ground engineering practitioners but the problem and methods for mitigation are frequently misjudged or undervalued by other construction professionals. Geotechnical risk is better managed, if it is integrated with overall project risk.

GeoStructures by **Trueventus** is a platform showcasing dynamic discussions on the current technical challenges in ground improvement - how to adopt cost effective solutions, how geotechnical risk is managed and the impacts this has in each construction market and construction practices of geotechnical structures.

This unique conference will bring delegates the benefits of:

Improving the accuracy of geotechnical investigations to enhance project planning and the design process

Promoting best practice in design, construction and maintenance

Mastering the state-of-art techniques for soil improvement

Identifying geological risk and how best to address geological risk in civil engineering contracts

Exploring the effectiveness of GBR to better manage risk for subsurface engineering works

Moving towards accurate geotechnical investigations

Leveraging risk to attain well define project feasibility

For more information:
John Karras
T: +603 2781 1501
F: +603 2781 1505
Email: **johnk@trueventus.com**

9th International Conference on Structural Analysis of Historic Constructions, 14 – 17 October 2014, Mexico City, Mexico, www.linkedin.com/groups/SAHC-2014-Mexico-City-3930057.S.213150607

6th International Conference on Protection of Structures Against Hazards, 16-17 October 2014, Tianjin, China, http://cipremier.com/page.php?764

2nd International Conference Innovations on Bridges and Soil - Bridge Interaction IBSBI 2014, Athens, 16 - 18 October, 2014, http://ibsbi2014.ntua.gr

1st International Conference on Volcanic Landscapes (VOLAND 2014), 16 - 18 October 2014, Santorini Island, Greece, voland@heliotopos.net

1st International Conference on Discrete Fracture Network Engineering, October 19 - 22, 2014, Vancouver, British Columbia, Canada, www.dfne2014.ca

12th International Conference Underground Infrastructure of Urban Areas, 22-23th October 2014, Wroclaw, Poland, http://www.uiua2011.pwr.wroc.pl

Η χρήση νέων τεχνολογιών στην πρόληψη και τη διαχείριση φυσικών καταστροφών - Ο ρόλος της πολιτικής προστασίας, 24 ÷ 26 Οκτωβρίου 2014, Ρόδος, http://saferhodes.blogspot.gr

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AusRock 2014
3rd Australasian Ground Control in Mining Conference - an ISRM Specialized Conference
5 – 6 November 2014, Sydney, Australia
www.groundcontrol2014.ausimm.com.au

AusRock2014: Third Australasian Ground Control in Mining Conference is aimed at practical mine site operators, technical support staff, geotechnical engineers, mining engineers, consultants and researchers in the field of mining geomechanics and ground control. The conference will provide an update to all mining industry geotechnical personnel involved in best practice in both Australasia and overseas, and an information exchange vehicle between the coal and metalliferous sectors of the industry, with a focus on new technologies and developments; industry needs and mine site problem solving; and practical case studies.

Conference Themes

- Ground support – tendon systems, surface liners, injection systems in opencut and underground mining
- Geotechnical instrumentation and monitoring
- Mine design – geotechnical considerations
- Geotechnical design methodologies
- Geomechanics of multiseam, multireef and complex orebodies
- Geotechnical challenges in extreme mining environments
- Backfill technologies
- Pillar design and performance
- Rock mass characterisation techniques and practice
- Regional stability
- Geotechnical risk management
- Best practice case studies
- New challenges in ground control
- Numerical modelling in design
- Mine subsidence – prediction and control
- Caving mechanics and control
- Dynamic mining and managing large deformations
- Geotechnical education and training
- Civil engineering application in mining

For further information, please contact:
Matthew Hadley, Coordinator, Publishing, The AusIMM
Telephone: +61 3 9658 6104 |
Email: mhadley@ausimm.com.au

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3rd ISRM International Young Scholars' Symposium on Rock mechanics - an ISRM Specialized Conference
8 – 10 November 2014, Xi'an, China
http://www.isrm.net/fotos/editor2/NI26/sysrock2014_copy.pdf

The ISRM Commission on Education has the pleasure to present the 3rd ISRM International Young Scholars' Symposium on Rock Mechanics two years after successful holding of the 2nd Symposium. This inaugural meeting will be held in Xi'an, the famous Chinese ancient capital, on Nov. 8-10, 2014.

This symposium aims to provide an ideal platform for idea and information exchange, knowledge and experience sharing. To develop and improve any subject requires continuity – through young researchers advancing the knowledge based on past information, and incorporating new techniques and new experiences. The meeting will provide stimulus and encouragement to the Young Scholars so that they will tackle our outstanding rock mechanics and rock engineering problems with renewed vigor.

Welcome all young scholars in rock mechanics, including young rock mechanics scientists, engineers and students to attend the symposium. All professors, research scientists and engineers who concern about education of rock mechanics and training of young rock mechanics scholars are also welcome to the symposium.

TOPICS

- Field investigation and measurements
- Experimental study of physical and mechanical properties of rock
- Analysis and design methods for rock engineering
- Numerical and physical modeling
- Multi-fields coupling analysis methods
- Rock slope, tunnel and foundation engineering
- Monitoring and control of ground pressure in underground rock engineering
- Dynamic rock mechanics and blasting
- Support and reinforcement techniques for geotechnical engineering
- Prediction and control of artificial hazards with excavation in roc

Contact

Telephone: +86 10 62332 464
Fax: +86 10 62334 098
E-mail: caimeifeng@ustb.edu.cn

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JTC-1's First International Landslide Workshop
November 2014, Seoul, Korea
sglee@uos.ac.kr

JTC-1's first International Landslide Workshop will be held in Seoul in November 2014 and Prof. Su-Gon Lee is the chairman of the organizing committee of the workshop. If colleague is interested in the workshop, please contact Prof. Su-Gon Lee: sglee@uos.ac.kr.

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7th International Congress on Environmental Geotechnics, 10-14 November 2014, Melbourne, Australia, www.7iceg2014.com

GEOMATE 2014 Fourth International Conference on Geotechnique, Construction Materials + Environment, 19 - 21 Nov. 2014, Brisbane, Australia, www.geomate.org

International Symposium "Geohazards" Science, Engineering & Management, 20-21 November 2014, Kathmandu, Nepal, www.ngeotechs.org/ngs/index.php/geohazards-2014

7th International Conference on Scour and Erosion (ICSE-7), 2nd – 4th December 2014, Perth, Western Australia, http://www.2014icse.com

Third Australasian Ground Control in Mining Conference 2014, Sydney, Australia, www.mining.unsw.edu.au/node/608

Proceedings of the Institution of Civil Engineers, Geotechnical Engineering, THEMED ISSUE 2015, Construction processes and installation effects, Editors: Benoît Jones, University of Warwick, UK and Stuart Haigh, University of Cambridge, UK, sarah.walker@ice.org.uk

IGS Chennai 2015 6th International Geotechnical Symposium on Disaster Mitigation in Special Geoenvironmental Conditions, January 21-23, 2015, IIT Mandras, Chennai, India, http://igschennai.in/6igschennai2015

Geosynthetics 2015, February 15 – 18, 2015, Portland, Oregon, USA, http://geosyntheticsconference.com

12th Australia New Zealand Conference on Geomechanics (ANZ 2015), 22-25 February 2015, Wellington, New Zealand, http://www.anz2015.com

GeoProc2015: International Conference on Coupled THMC Processes in Geosystems
25-27 February 2015, Salt Lake City, USA
robert.podgorney@inl.gov

Water Storage and Hydropower Development for Africa
10 to 12 March 2015, Marrakesh, Morocco
http://www.hydropower-dams.com/AFRICA-2015.php?c_id=89

Aqua~Media International, in partnership with the International Commission on Large Dams, and with the strong support of the Government of the Kingdom of Morocco, is pleased to invite world water and hydropower specialists to AFRICA 2015 in Marrakesh. This follows the successful first conference in this series, AFRICA 2013 in Addis Ababa, when practical aspects of advancing hydropower and water resources development in Africa were discussed by more than 600 participants from 67 countries. H.E. Dr Elham M.A. Ibrahim, Commissioner for Energy and Infrastructure of the African Union, H.E. Alemayehu Tegenu, Ethiopia's Minister of Water and Energy, and H.E. Mamounata Belem Ouedraogo, Minister of Water Resources and Hydraulic Infrastructure of Burkina Faso, all presided at the event, which was co-hosted by Aqua-Media, ICOLD and the Ethiopian Electric Power Corporation.

As well as ministerial delegations, the conference brought together utility CEOs, officers of UNECA, leading experts from the IFIs, Presidents, Vice-Presidents and Secretaries of the water- and energy-related professional associations, eminent engineering consultants, researchers, leading contractors and equipment suppliers. The conference made a practical contribution to the Programme for Infrastructure Development in Africa. Outcomes, including a Final Declaration, were widely disseminated globally. Building on the success of AFRICA 2013, and working closely with ICOLD as well as an International Steering Committee, Aqua~Media is putting together a programme to focus on issues of special relevance to the whole African region. Experts from all parts of the world will bring their knowledge and experience to the sessions, panel discussions and inter-active workshops.

Major multipurpose water resources schemes, including many large regional hydro projects, are moving ahead in Africa at an unprecedented rate. A status update, potential and development opportunities, as well as technical, environmental and financial challenges, covering all regions of the African continent, will be the focus of the presentations and discussions.

Themes

- Potential and planned developments in Africa
- African multipurpose water storage including hydropower, irrigation, water supply, navigation and fisheries
- Quantifying and qualifying the benefits of water infrastructure
- African small dams for irrigation
- Role of storage in river basin management for sustainable development
- The role of risk mitigation in making hydro more competitive
- Concession agreements and construction contracts
- Finance options including resource mobilization and the Africa 50 fund
- Implementation and review of dam safety legislation in Africa
- Monitoring the safety of dams, gates and powerplants
- Public safety around dams
- Dam engineering: design and construction
- Institutions and institutional arrangements
- Case studies from the 'Water Towers': the Congo Basin, the Ethiopian Highlands, the Fouta Djallon and the Lesotho Highlands
- Effects of climate change in Africa: adaptation and mitigaion
- Flood control
- The role of hydro in African regional development
- Update on the PIDA Energy Priority Action Plan
- Pumped storage; hydro in synergy with other renewables
- Hydro's role in electrical system stability in Africa
- Small hydro in Africa
- Rural electrification in Africa
- Hydro machinery: research and operational issues and, practical examples of innovative low cost technologies
- Environmental and social aspects of African schemes, including water conservation and transfer, and social challenges of transboundary projects
- Reservoir sedimentation mitigation

- Operation, maintenance and rehabilitation challenges; obstacles and solutions

In addition to plenary and parallel sessions exploring the themes above, there will be focused workshops and panel discussions on topics such as project finance, regional co-operation and progress with the Programme for Infrastructure Development in Africa, including Grand Inga and other large regional hydro projects.

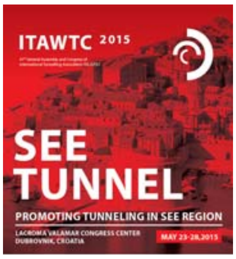
For more information please contact:
Mrs Margaret Bourke, Conference Project Manager,
Aqua~Media International, PO Box 285, Wallington, Surrey SM6 6AN, UK.
Tel: +44 20 8773 7244 Fax: + 44 20 8773 7255.
Email: mb@hydropower-dams.com

16th African Regional Conference on Soil Mechanics and Geotechnical Engineering, April 27 to 30, 2015 in Hammamet, Tunisia, http://www.cramsg2015.org

ISP7-PRESSIO2015 1 to 2 May 2015, Hammamet, Tunisia, http://www.cramsg2015.org/isp7-pressio2015

13th ISRM International Congress on Rock Mechanics Innovations in Applied and Theoretical Rock Mechanics 10–13 May 2015, Montreal, Canada, www.isrm2015.com

Shale and Rock Mechanics as Applied to Slopes, Tunnels, Mines and Hydrocarbon Extraction, Special One day Symposium, May 12, 2015, Montreal, Quebec, Canada, www.isrm2015.com/Page/PageContent/ShaleSymposium



World Tunnel Congress 2015
and 41st ITA General Assembly
Promoting Tunnelling in South East European (SEE) Region
22 - 28 May 2015, Dubrovnik, Croatia
http://wtc15.com

The Croatian Association for Tunnels and Underground Structures – ITA Croatia and the ITA-AITES invite you to join us at the WORLD TUNNEL CONGRESS AND GENERAL ASSEMBLY IN DUBROVNIK May 22 - 28, 2015.

The topic of the 2015 world congress is dedicated to the development of using underground space in south-east Europe. The goal is to introduce participants to new and contemporary methods and technologies which through their developing trends and achievements can contribute to the development in the field of underground usage and tunneling in Croatia as well as the wider region of south-east Europe.

It will certainly be a great opportunity for engineers, designers, contractors and investors to learn about the possibilities and advantages of new methods as well as products in their technical and financial shape.

Main topics

1. Planning and Designing Tunnels and Underground Structures
2. Mechanized Tunnelling in Development and Use
3. Conventional Tunnelling Methods in Development and Use
4. Immersed and Floating Tunnelling
5. Operation and Maintenance of Tunnels and Underground Structures
6. Equipment of Tunnels and Underground Structures
7. Fire Safety of Tunnels and Underground Structures
8. Intelligent Systems, Mechatronics and Robotics in Tunnelling
9. Developments in Use of Underground Space: Case Studies (Traffic, Waste, Energy, Water, Sewer, Flood Protection, Commercial and other Uses)
10. Urban planning and Using of Underground Space
11. SEE Session: Soft Ground Urban Tunnelling / Rock Tunnelling in Karst
12. Risk Analyses and Techniques for Underground Structures
13. Cost Optimization and Financing of Underground Structures

Contact
ITA Croatia - Croatian Association for Tunnels and Underground Structures
Davorin KOLIC, Society President
Trnjanska 140
HR-10 000 Zagreb
Croatia
info@itacroatia.eu

83rd ICOLD Annual Meeting & Congress
Hydropower' 15
June 2015, Stavanger, Norway
www.icoldnorway2015.org

With its 4 000 river courses and 250 000 lakes, Norway is a blessed country when it comes to electricity production from hydropower. An installed capacity of 27 000 MW ensures that virtually all electricity consumption is covered from hydropower. Hence, hydropower is one of Norway´s most important natural resources. This has been an important factor in the favourable economic development of the country for the last century.

More than 1 000 hydropower stations, including nearly half of the world´s underground power plants, 330 large dams, an active dam and hydropower sector, with power companies, manufacturers, major consulting and engineering companies, contractors and R&D institutions, provide an excellent framework for international gatherings in Norway. In addition the scenic surroundings of Western Norway, with its fjords, mountains, glaciers and rivers constitute a breath-taking framework of the Stavanger events.

We are looking forward to seeing you in Norway in June 2015!

Conference topics

- Challenges in dam constructions under challenging conditions
- Reservoir optimizations and design
- Operation and maintenance of electro-mechanical equipment
- Innovative hydropower and dam projects in emerging economies
- Project financing and contractual aspects
- Industry- Academia cooperation
- Managing Risks in hydropower– Emergency preparedness and public safety
- Climate change challenges (Environment, Social Aspects, Floods, Generation and Dam Safety)

If you would like to receive information about ICOLD 2015, please send an e-mail to: icold2015@gyro.no

ISFOG 2015 3rd International Symposium on Frontiers in Offshore Geotechnics, Oslo, Norway, 10-12 June 2015, www.isfog2015.no

DMT 15 The 3rd International Conference on the Flat Dilatometer, Rome 15-17 June 2015, www.dmt15.com

ICGE 2015 International Conference in Geotechnical Engineering – Colombo-2015, 10 - 11 August 2015, Colombo, Colombo, Sri Lanka, http://www.slgs.lk/?p=564

China Shale Gas 2015
an ISRM Specialized Conference
6-8 September 2015, Wuhan, China
http://english.whrsm.cas.cn/ic/ic/201405/t20140509_120692.html

China Shale Gas 2015 (CSG2015) will take place in Wuhan, China from 6 to 8 September 2015. This international conference will be organized by the Institute of Rock and Soil Mechanics, Chinese Academy of Sciences in conjunction with a dozen of other institutions.

The objective of the conference is to provide a platform for international researchers and practitioners across the whole range of disciplines to examine pressing issues, exchange ideas, develop innovative solutions and explore emerging technologies in key technical areas of shale and coal seam gas extraction. This is the key to replicating the success of the US shale gas revolution both in China and in other parts of the world.

The objective of this conference will be achieved through a combination of 10 keynote lectures, 150 presentations, 200 posters, and post-conference technical tours. Keynote lectures will be given by a combination of world leading scientists and world-renowned specialists in different disciplines of shale and coal seam gas engineering.

Institute of Rock and Soil Mechanics, Chinese Academy of Sciences, tel. +86 1368 3361 028, jsliu@whrsm.ac.cn

16th European Conference on Soil Mechanics and Geotechnical Engineering "Geotechnical Engineering for Infrastructure and Development", 13 - 17 September 2015, Edinburgh, UK, www.xvi-ecsmge-2015.org.uk

Workshop on Volcanic Rocks & Soils, 24 - 25 September 2015, Isle of Ischia, Italy, www.associazionegeotecnica.it

ISRM European Regional Symposium
64th Geomechanics Colloquy
7 – 9 October 2015, Salzburg, Austria
www.oegg.at/eurock-2015

The Austrian Society for Geomechanics has the pleasure to invite you to the ISRM Regional Symposium EUROCK 2015 to be held in conjunction with the 64th Geomechanics Colloquy in Salzburg, the city where the International Society for Rock Mechanics (ISRM) was founded in 1962. The Geomechanics Colloquy in Salzburg since its initiation in 1951 has always been a perfect and distinguished meeting place for researchers and practitioners. The success of this concept not only shows in the continuous meetings over more than 60 years, but also in the attendance of regularly around 1000 participants.

PROPOSED SESSION TOPICS

The Symposium will cover following themes

- Long-term behaviour of engineering structures in rock
- Tunnelling under challenging conditions
- Mass movements
- Rock mechanical aspects of excavation
- Monitoring and safety management
- Determination of rock properties
- Rock mechanical aspects of nuclear waste repositories
- Micromechanics of rocks
- Modelling in rock and rock masses
- Mining rock mechanics
- Young researchers session

WORKSHOPS

On Wednesday the 7th of October Workshops on special technical topics are scheduled

- Rock mass characterization
- Design practices
- Contractual models

Prof. Wulf Schubert
AUSTRIAN SOCIETY FOR GEOMECHANICS
Österreichische Gesellschaft für Geomechanik (ÖGG)
Innsbrucker Bundesstrasse 67
5020, Salzburg, Austria
P.: +43 662 875519
F.: +43 662 886748
E.: Salzburg@oegg.at, info@EUROCK2015.com

European Conference in Geo-Environment and Construction, October/November 2015, Tirana, Albania, Prof. Dr. Luljeta Bozo, lulibozo@gmail.com; luljeta_bozo@universitetipolis.edu.al

International Conference on
Engineering Geology in New Millennium
26-31 October 2015, New Delhi, India
http://isegindia.org/pdfs/1st%20circular-international-IAEG.pdf

Having been established in October 1965, Indian Society of Engineering Geology (ISEG) will be completing glorious fifty years of its engagement in the allied domain of geology and engineering geology in October 2015. In order to commemorate the golden jubilee, I am pleased to announce that an International conference titled “Engineering Geology in New Millennium” will be hosted by ISEG in October 2015 in New Delhi. While an active participation and support from International Association of Engineering Geology and the Environment (IAEG) is being solicited, I take this opportunity to invite all the geologists, engineering geologists, geotechnical engineers and other geo-scientific practitioners to join us in the celebration and participate in the conference. Regular updates and subsequent circulars will be available in ISEG websites isegindia.org and joegindia.com

Themes

1. Ambit of engineering geology, international viewpoints, Indian references, historical developments, roles and functions of engineering geologists.

2. Engineering geological education, undergraduate and graduate level study, pure and applied geology, geology in civil engineering, geology in geotechnical engineering, cross disciplinary and collaborative research, opportunities, new approach in engineering geological education.

3. Recent developments in explorations, new techniques in investigations, exploratory drilling tools, geophysical surveys, geological & geophysical logging, accelerated investigation programs.

4. Application of rock mechanics tests in investigation and construction stages, importance of lab and insitu testing, International standards and methods, Indian Standards, ASTM and British standards: similarities and differences.

5. How to use test results? Recent techniques of derivation of engineering properties from rock mass classifications.

6. Engineering geological monitoring in construction stage projects, tackling geological problems, role of best construction practices, recent advances in open air and underground excavations and support techniques, slope stability.

7. Soil Mechanics: Slope stability, rock fill & earthen dams and embankments.

8. Investigation and construction methodology for long tunnels, use of TBM’s, success stories and pitfalls.

9. Geological input in contract documents, understanding obligations of owners and contractors, risk assessment and sharing, use of specialized agencies, do’s and don’ts for engineering geologists while handling contracts.

10. Exploration and testing for natural construction materials, quarrying, environmental aspects, disposal of tunnel waste, reclamation.

11. Blasting techniques, importance in deep open excavations, blast design for safe and speedy tunneling, methodology for excavation and support of large caverns. Effects of blasting on engineering properties.
12. Rock support elements: Theory & practice, recent advancements both in soil & rock stabilization.

13. Geological case histories for CFRD and RCC dams.

14. Role of rock mechanics and engineering geology in mining, case studies for mining industry, mining methodology, environmental issues of open cast and underground mining.

15. Geo-environmental studies for large civil engineering projects, sustainable development, environmental aspects of river valley schemes.

16. Seismotectonic studies and earthquake engineering.

17. Geo-hydrological studies in hilly terrain, ground water modelling for underground projects, use of new techniques, assessment of water ingress.

18. Ground water contamination, cities and country side expansion and water balance studies.

19. Landslide studies and mitigation, hazard zonation, exploration technique, monitoring, engineering solutions, long term measures.

20. Urban engineering geology: Geotechnical evaluation of cities, metro tunnels, underground space technology, waste water treatment plants.

Correspondence

Imran Sayeed
Editor,

Indian Society of Engineering Geology (ISEG)
Email: iseg2015@gmail.com;
editor@joegindia.com

M. Raju
Secretary, Indian Society of Engineering Geology (ISEG)
C/o Director, Monitoring Division (Engineering Geology),
2nd Floor, GSI, 27, Jawahar Lal Nehru Road, Kolkata 700 016, India.
Mobile : 09432672087; Email: geolraju@gmail.com

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6th International Conference on Earthquake Geotechnical Engineering, 2-4 November 2015, Christchurch, New Zealand, www.6icege.com

The 15th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, 9-13 November 2015, Fukuoka, Japan, http://www.15arc.org

15th Pan-American Conference on Soil Mechanics and Geotechnical Engineering, 15 - 18 November 2015, Buenos Aires, Argentina, http://conferencesba2015.com.ar

VIII South American Congress on Rocks Mechanics, 15 - 18 November 2015, Buenos Aires, Argentina, http://conferencesba2015.com.ar

Sixth International Conference on Deformation Characteristics of Geomaterials IS Buenos Aires 2015, November 15th to 18th 2015, www.saig.org.ar/ISDCG2015

2015 6th International Conference Recent Advances in Geotechnical Engineering and Soiul Dynamics, December 7-11, 2015, New Delhi (NCR), India, wason2009@gmail.com; wasonfeq@iitr.ernet.in, sharmamukat@gmail.com; mukutfeq@iitr.ernet.in, gvramanaiitdelhi@gmail.com, ajaycbri@gmail.com

☙ ❧

Southern African Rock Engineering Symposium
an ISRM Regional Symposium
5 January 2016, Cape Town, South Africa
http://10times.com/southern-african-rock

Contact Person: William Joughin
SRK Consulting SA. PTY LDA
Tel. +27-11-441-1214
wjoughin@srk.co.za

☙ ❧

GeoAmericas 2016
3RD PAN-AMERICAN CONFERENCE ON GEOSYNTHETICS
11-14 APRIL 2016 • MIAMI BEACH • USA

NAGSDirector05@gmail.com

☙ ❧

CIGB ICOLD
SANCOLD

84th ICOLD Annual Meeting
May 2016, Johannesburg, South Africa
www.sancold.org.za/index.php/activities/icold-annual-meeting-2016

☙ ❧

7th In-Situ Rock Stress Symposium 2016
An ISRM Specialised Conference
10-12 May 2016, Tampere, Finland
www.ril.fi/en/international-conferences/rs2016-symposium.html

Contact Person: Erik Johansson
erik.johansson@rs2016.org
Finnish ISRM Group and Finnish Association of Civil Engineers - RIL

☙ ❧

GEOSAFE: 1st International Symposium on Reducing Risks in Site Investigation, Modelling and Construction for Rock Engineering - an ISRM Specialized Conference
25 – 27 May 2016, Xi'an, China

Contact

Telephone: 0086 27 87198913
Fax: 0086 27 87198413
E-mail: xtfeng@whrsm.ac.cn

☙ ❧

NGM 2016 - The Nordic Geotechnical Meeting, 25 - 28 May 2016, Reykjavik, Iceland, www.ngm2016.com

EUROCK 2016
ISRM European Regional Symposium
Rock Mechanics & Rock Engineering: From Past to the Future
29-31 August 2016, Ürgüp-Nevşehir, Cappadocia, Turkey
resat@hacettepe.edu.tr

Contact Person: Prof. Resat Ulusay
Turkish National Society for Rock Mechanics
Telephone: +90 312 2977767
Fax: +90 312 2992034

3rd ICTG
International Conference on Transportation Geotechnics

4 - 7 September 2016, Guimaraes, Portugal
www.spgeotecnia.pt/cpgt

The Transportation Geotechnics International Conference series began under the auspices of ISSMGE-TC 3 and was initiated in 2008 at the University of Nottingham, UK, as an International event designed to address the growing requirements of infrastructure for societies. The 2nd International Conference on Transportation Geotechnics took place in 2012, at Sapporo, Japan, under the ISSMGE-TC202 that follows the TC-3 activities for the period 2009-2013. To continue the successful of these conferences and the output of ISSMGE-TC-202, the 3rd was scheduled for 2016, at Guimarães, Portugal. Following the previous one, the challenges addressed by this conference will include a better understanding of the interactions of geotechnics on roads, rails, airports, harbours and other ground transportation infrastructure with the goal of providing safe, economic, environmental, reliable and sustainable infrastructures. The 3rd ICTG will be composed of workshops and several types of sessions, as well as a technical exhibition, to better disseminations of findings and best practices. A special attention will be paid to the publication of all the peer review papers, some of them in specialised international journals. On behalf of the organizing committee I am honoured to invite you to the 3rd ICTG in the City of Guimarães, UNESCO World Heritage (September 4-7, 2016).

Contact person: Prof. A. Gomes Correia (Chair)
Address: University of Minho, School of Engineering
Campus de Azurém
4800-058, Guimarães, Portugal
Phone: +351253510200, +351253510218
Fax: +351253510217
E-mail: 3ictrgeo2016@civil.uminho.pt, agc@civil.uminho.pt

EuroGeo 6 – European Regional Conference on Geosynthetics
25 – 29 Sep 2016, Istanbul, Turkey
eguler@boun.edu.tr

ARMS 9
9th Asian Rock Mechanics Symposium
ISRM Regional Symposium
October 2016, Bali, Indonesia
rkw@mining.itb.ac.id

Contact Person: Dr Ridho Wattimena
Indonesian Rock Mechanics Society (IRMS)
Telephone: +22 250 2239

6th Asian Regional Conference on Geosynthetics
November 2016, New Delhi, India
uday@cbip.org

11th International Conference on Geosynthetics (11ICG)
16 - 20 Sep 2018, Seoul South Korea
csyoo@skku.edu

ΕΝΔΙΑΦΕΡΟΝΤΑ ΓΕΩΤΕΧΝΙΚΑ ΝΕΑ

Field Investigation of Geosynthetics used for Subgrade Stabilization

The main objective of this project is to determine material properties of geosynthetics that affect in-field performance of geosynthetics used for subgrade stabilization, so that DOT personnel can objectively and confidently specify appropriate geosynthetics based on material properties and cost for a specific situation, while also allowing competition from different manufacturers.

Filling the large-scale test trench ahead of installing materials, screeding surfaces, etc.

From December 2011 through the end of May 2014, the Montana Department of Transportation led a pooled fund study on "Performance of Geosynthetics in Subgrade Stabilization." The research, which was carried out by the Western Transportation Institute (WTI, based at Montana State University) follows a smaller Phase I, which was carried out in 2008 and 2009 by WTI.

"In Phase I, the testing conditions were more severe," says Eli Cuelho, P.E., who led the research and co-authored the report with Steve Perkins and Zachary Morris. "The research in Phase I helped understand what happens under more severe conditions, which do occur in the real world, but in Phase II we wanted to look at performance in more typical, less severe conditions."

The research picked up where Phase I had left off. This included utilizing WTI's large outdoor laboratory—the TRANSCEND research facility in Lewistown, Montana. Different subgrade stabilization designs were evaluated.

Phase I Final Report http://www.geosynthetica.net/wp-content/uploads/PhaseI_4W2012_Final_Report.pdf

"With this second phase we expanded the number of products in the study," Cuelho says. "We wanted to cover the gamut of the types of geosynthetics that could be used in subgrade stabilization applications—whether it's a welded, woven or integrally-formed geogrid, or a geotextile."
Just as in Phase I, a number of manufacturers were represented in the study, including Colbond (Bonar), Huesker, NAUE, Propex, Synteen, SynTec, TenCate and Tensar.

Screeding gravel cover.

Phase I of the research was co-sponsored by NAUE GmbH & Co. KG. The results drew the attention of DOTs and brought about Phase II's funding mix, which was entirely supported by a geographically broad group of state DOTs: Idaho, Montana, New York, Ohio, Oklahoma, Oregon, South Dakota, Texas and Wyoming.

Phase II Final report
http://www.mdt.mt.gov/other/research/external/docs/research_proj/subgrade/final_report-2.pdf

Geotextiles Move Forward with Geogrids

Geotextiles have been part of the geosynthetics field so long that there are some people who still refer to all geosynthetics generically as geotextiles. At times, this has caused confusion or overshadowed the technical characteristics and performance of geotextiles, including in reinforcement applications. Their common presentation as a "commodity" material has hampered their acceptance in higher strength work.

But the Federal Highway Administration's (FHWA) use of geotextiles in abutments for its Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS) has helped raised the profile of geotextiles for designers.

Example of tension in geosynthetic in the wheel path.

Also, a number of companies have stepped up R&D and promotion of geotextiles in reinforcement applications, highlighting not only geotextiles on their own but the performance of geotextile-geogrid composite materials.

WTI's Phase II Report will certainly continue to keep geotextiles elevated in stabilization discussions.

"Textiles performed very well," says Cuelho. "The woven geotextile performed very well and even the 8 oz. nonwoven geotextile did very well. I think people will be interested, and maybe a little surprised, at how well these materials performed."

As expected, most of the geogrids also performed well, and that performance was directly linked to the strength and stiffness of the junctions as well as tensile strength in the cross-machine direction.

The combined results of geotextiles and geogrids highlights the range of choice and performance that geosynthetics provide to soil stabilization applications. They have different price points. They can substantially reduce aggregate thickness. They present traffic benefit ratio (TBR) improvements. Ultimately, they can provide strong options for maximizing economics and performance.

Phase II has verified what most already perceive in geogrids. Geotextiles, however, may have earned a boost. Phase I showed resiliency in geogrids and geotextile-geogrid composites in more severe conditions. In Phase II, under less severe conditions, geotextiles revealed stabilization characteristics that may outperform the perception many engineers have of them.

"Nonwoven geotextiles are primarily used in separation applications," Cuelho says, "but based on their performance in this study, perhaps secondary structural benefits are also relevant – and that might change some perceptions."

Currently, geotextiles are not considered for their reinforcement benefit in subgrade stabilization applications; but that may change if additional quantification of the mechanical characteristics of this performance can be better understood.

Phase III on Subgrade Stabilization?

The ink is just drying on the Phase II report, but Mr. Cuelho is already in discussion with some DOTs and private industry on another expansion of the research. More materials, loading conditions, and designs would be welcomed to help further quantify the real mechanisms at work on and within these materials.

Companies and agencies interested in the work and potentially contributing materials or financial resources to support future studies on geosynthetics in a variety of transportation applications should contact Eli Cuelho at elic@coe.montana.edu.

Visit the study website
http://www.mdt.mt.gov/research/projects/geotech/subgrade.shtml

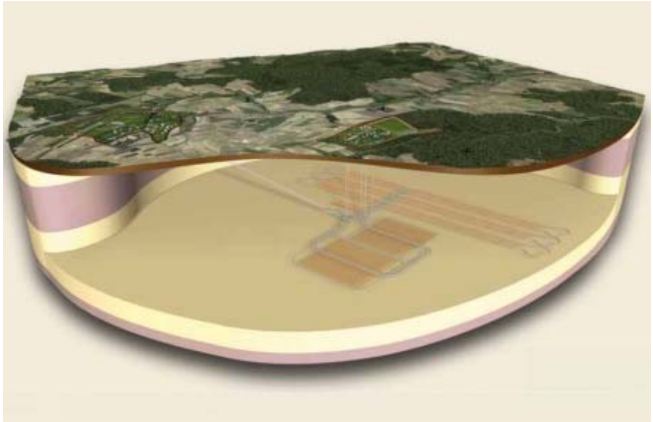
(geosynbthetica.net, June 11, 2014, http://www.geosynthetica.net/final-report-phase2-geosynthetics-subgrade-stabilization)

Designing the Cigéo disposal site

Work is to start on the design of waste handling procedures for Cigéo, France's future underground disposal site for radioactive waste.

The four-year handling project, worth €20 million ($27 million), is to be carried out by engineering firms Assystem, Cegelec and Spretec. They will undertake what they called "high-level studies" followed by "detailed design of the technical procedures for the transfer and storage of waste packages" at Cigéo, said Assystem.

The waste in question totals around 2700 cubic metres of high-level radioactive waste and about 40,000 cubic metres of long-lived intermediate-level radioactive waste. Between them these contain 99% of the radioactivity from nuclear power generation that has provided the majority of France's electricity over the last few decades.



Cigéo (Image: Andra)

Cigéo will be an underground system of disposal tunnels, known as galleries, in a natural layer of clay near Bure, to the east of Paris in the Meuse/Haute Marne area.

The purpose of the studies is to specify Cigéo's systems and procedures in detail ahead of developing a licence application, said Assystem's Laurent Doher. For five months Assystem will work to "confirm the industrial feasibility of the project," which he said was "paramount". After that will be the development of a "roadmap for [Assystem's] industrial partners so that we can put forward the best technical and economic solution."

Managed by national radioactive waste disposal organisation Andra, plans for Cigéo were last year put out for public comment. Based on public input, Andra revised its plans to provide for a pilot plant to test, under real conditions, all of the disposal functions: the technical measures to control operating risks, the capacity to remove packages being disposed of, the disposal monitoring sensors, the techniques for sealing cavities and galleries, among other things.

The public also wanted Andra to allow for reversibility, so future generations could more easily remove the waste packages, should they decide to do so. A master plan for the entirety of Cigéo's development and operation will be constantly updated with input from stakeholders and approval by government, also as a result of the consultation, Andra said.

Next year Andra plans to submit its master plan for operation and disposal to government, as well as a set of options for security and retrievability. The application to regulators to construct Cigéo should come in 2017, with construction itself following in 2020. The pilot phase of disposal could start in 2025.

(World Nuclear News, 02 July 2014, http://world-nuclear-news.org/WR-Designing-Cigeo-disposal-site-0206141.html)

Αψίδες υπό τάση
Γεωλογικά γλυπτά αποδεικνύονται έργα της βαρύτητας

Στην εικόνα, το εμβληματικό «Delicate Arch» στο Εθνικό Πάρκο Αψίδων στις ΗΠΑ. Στο βίντεο του Nature, τα πειράματα της μελέτης σε γρήγορη κίνηση https://www.youtube.com/watch?v=OBDjyd1aByQ

Εκατοντάδες χιλιάδες τουρίστες επισκέπτονται κάθε χρόνο το «Εθνικό Πάρκο Αψίδων», μια περιοχή της πολιτείας Γιούτα με περισσότερες από 2.000 μνημειώδεις αψίδες από ψαμμίτη. Οι σχηματισμοί αυτοί πιστεύεται ότι δημιουργήθηκαν λόγω της διάβρωση των εύθρυπτων πετρωμάτων από το νερό και τον αέρα. Όμως τα τελευταία πειράματα δείχνουν ότι πρόκειται στην πραγματικότητα για γλυπτά της βαρύτητας.

Ακόμα ένα γεωλογικό γλυπτό στο Εθνικό Πάρκο Αψίδων στις ΗΠΑ (Πηγή: Cacophony / Wikimedia Commons)

Ερευνητές από τις ΗΠΑ και την Τσεχία ήταν περίεργοι να κατανοήσουν πώς ο ψαμμίτης, ένα μάλλον σαθρό πέτρωμα από κόκκους άμμου που συγκρατούνται χαλαρά μεταξύ τους, μπορεί να σχηματίζει πανύψηλες κατασκευές που δεν καταρρέουν από το ίδιο τους το βάρος.

Μετέφεραν λοιπόν στο εργαστήριο δείγματα ψαμμίτη από ένα ορυχείο στην Τσεχία στο οποίο οι εργασίες εξόρυξης οδήγησαν στον αυθόρμητο σχηματισμό αψίδων.

Όταν τα δείγματα βυθίστηκαν σε νερό, ο ψαμμίτης διαλύθηκε αμέσως. Όταν όμως βυθίστηκαν με βάρη τοποθετημένα πάνω τους, η τάση που ασκούσε το βάρος στους κόκκους άμμου διατήρησε τη δομή σταθερή, αναφέρει η ερευνητική ομάδα στο Nature ("Sandstone landforms shaped by negative feedback between stress and erosion", Jiri Bruthans, Jan Sou-kup, Jana Vaculikova, Michal Filippi, Jana Schweigstillova, Alan L. Mayo, David Masin, Gunther Kletetschka & Jaroslav Rihosek, *Nature Geoscience* 7, 597–601 (2014), doi:10. 1038/ngeo2209, http://www.nature.com/ngeo/journal/v7/n8/full/ngeo2209.html#affil-auth).

Το εντυπωσιακό είναι ότι οι κατακόρυφες τάσεις που ασκεί η βαρύτητα μέχρι τη βάση του σχηματισμού δεν κατανέμεται ομοιόμορφα στον ψαμμίτη. Οι περιοχές που δεν δέχονται τάση παρατηρήθηκαν να καταρρέουν και να αφήνουν πίσω τους σχηματισμούς που θυμίζουν τα φυσικά μνημεία του Εθνικού Πάρκου Αψίδων.

Ο «Πύργος της Βαβέλ» στο Εθνικό Πάρκο Αψίδων (Πηγή: Shannon Martin / Wikimedia Commons)

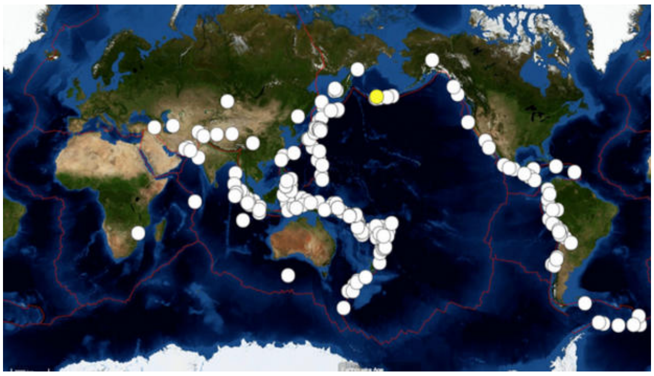
Το μεγαλύτερο αξιοθέατο του πάρκου, μέσα στην κοιλάδα του ποταμού Κολοράντο στις ΗΠΑ, είναι το λεγόμενο «Delicate Arch», ή «Λεπτεπίλεπτη Αψίδα», ένα γεωλογικό γλυπτό ύψους 20 μέτρων.

Εδώ και χιλιάδες χρόνια η βαρύτητα επιβραδύνει τη διάβρωση αυτών των επιβλητικών σχηματισμών, δεν αρκεί όμως για να την σταματήσει εντελώς: πάνω από 40 αψίδες έχουν καταρρεύσει στο πάρκο από το 1970.

(Βαγγέλης Πρατικάκης / Newsroom ΔΟΛ, 21 Ιουλ. 2014, http://news.in.gr/science-technology/article/?aid=1231336107)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΣΕΙΣΜΟΙ

Big earthquakes double in 2014, but scientists say they're not linked



Earthquakes larger than magnitude-7 since 2000 (USGS)

If you think there have been more earthquakes than usual this year, you're right. A new study finds there were more than twice as many big earthquakes in the first quarter of 2014 as compared with the average since 1979.

"We have recently experienced a period that has had one of the highest rates of great earthquakes ever recorded," said lead study author Tom Parsons, a research geophysicist with the U.S. Geological Survey (USGS) in Menlo Park, California.

But even though the global earthquake rate is on the rise, the number of quakes can still be explained by random chance, said Parsons and co-author Eric Geist, also a USGS researcher. Their findings were published online June 21 in the journal Geophysical Research Letters. [Image Gallery: This Millennium's Destructive Earthquakes]

With so many earthquakes rattling the planet in 2014, Parsons actually hoped he might find the opposite -- that the increase in big earthquakes comes from one large quake setting off another huge shaker. Earlier research has shown that seismic waves from one earthquake can travel around the world and trigger tiny temblors elsewhere.

"As our group has been interested in the ability of an earthquake to affect others at a global scale, we wondered if we were seeing it happening. I really expected we would see evidence of something we couldn't explain by randomness," Parsons told Live Science's Our Amazing Planet in an email interview.

The new study isn't the first time researchers have tried and failed to link one earthquake to another in time and across distance. Earlier studies found that the biggest earthquakes on the planet -- the magnitude-8 and magnitude-9 quakes -- typically trigger much smaller jolts, tiny magnitude-2 and magnitude-3 rumblers. Yet, no one has ever proven that large quakes unleash other large quakes. Finding a statistical connection between big earthquakes is a step toward proving such connections takes place.
But despite the recent earthquake storm, the world's great earthquakes still seem to strike at random, the new study found.

The average rate of big earthquakes -- those larger than magnitude 7 -- has been 10 per year since 1979, the study reports. That rate rose to 12.5 per year starting in 1992, and then jumped to 16.7 per year starting in 2010 -- a 65 percent increase compared to the rate since 1979. This increase accelerated in the first three months of 2014 to more than double the average since 1979, the researchers report.

The rise in earthquakes is statistically similar to the results of flipping a coin, Parsons said: Sometimes heads or tails will repeat several times in a row, even though the process is random.

"Basically, we can't prove that what we saw during the first part of 2014, as well as since 2010, isn't simply a similar thing to getting six tails in a row," he said.

But Parsons said the statistical findings don't rule out the possibility that the largest earthquakes may trigger one another across great distances. Researchers may simply lack the data to understand such global "communication," he said.

"It's possible that global-level communications happen so infrequently that we haven't seen enough to find it among the larger, rarer events," Parsons said.

However, earthquakes smaller than magnitude-5.6 do cluster on a global scale, the researchers found. This suggests these less-powerful quakes are more likely to be influenced by others -- a finding borne out by previous research.
For example, the number of magnitude-5 earthquakes surged after the catastrophic magnitude-9 earthquakes in Japan and Sumatra, even at distances greater than 620 miles (1,000 kilometers), earlier studies found.

(Becky Oskin / LiveScience.com, July 1, 2014, http://www.cbsnews.com/news/big-earthquakes-double-in-2014-but-scientists-say-theyre-not-linked/#postComments)

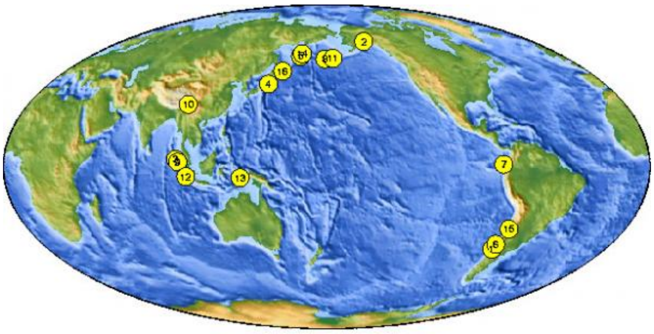
The 2010–2014.3 global earthquake rate increase

Tom Parsons and Eric L. Geist

In light of a heightened global earthquake rate during the first quarter of 2014 and recent studies concluding that large earthquakes affect global seismicity for extended periods, we revisit the question whether the temporal distribution of global earthquakes shows clustering beyond that expected from a time-independent Poisson process. We examine a broad window from 1979 to 2014.3 for $M \geq 7.0$ shocks, and a narrow window for $M \geq 5.0$ seismicity since 2010 that has higher than average rates. We test whether a Poisson process can be falsified at 95% confidence to assess the degree of dependent clustering in the catalogs. If aftershocks within at least one rupture length from main shocks/foreshocks are filtered, then we find no evidence of global scale $M \geq 5.2$–5.6 (depending on parameters) clustering since 2010 that demands a physical explanation. There is evidence for interdependence below this threshold that could be a consequence of catalog completeness or a physical process.

Geophysical Research Letters, Volume 41, Issue 13, pages 4479–4485, 16 July 2014

The 10 Biggest Earthquakes in History



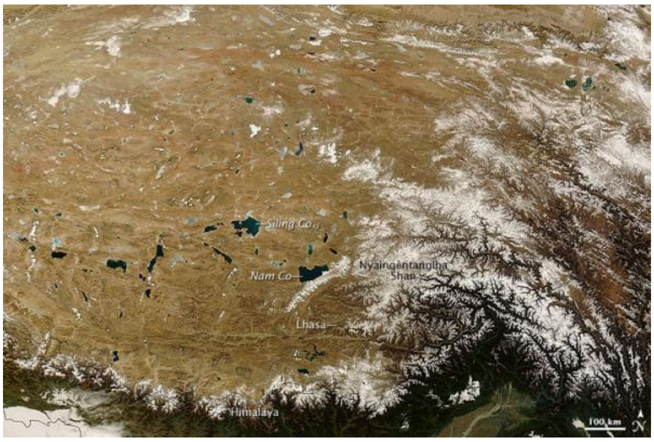
USGS National Earthquake Information Center

Intro

As massive and deadly as Japan's recent magnitude 9.0 earthquake was, it's not the world's biggest recorded quake.

It is Japan's largest quake, but dating back to 1900, four other earthquakes of magnitude 9.0 or greater have ruptured across the globe, according to data from the U.S. Geological Survey (USGS). We countdown the top 10 biggest recorded earthquakes in the world.

10 - Assam-Tibet, 1950 - Magnitude 8.6



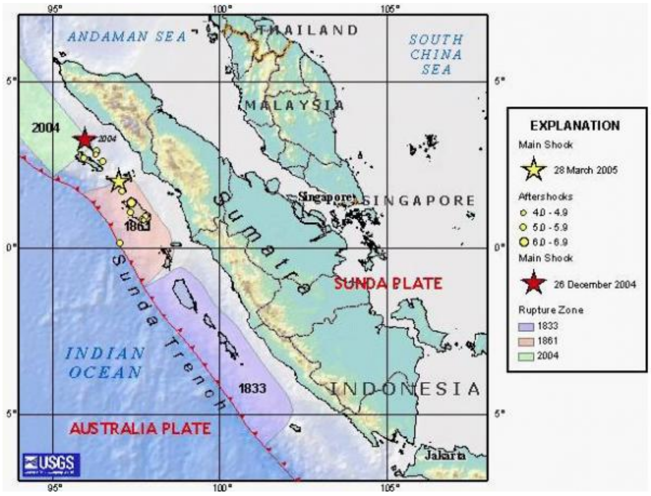
At least 1,500 people were killed across eastern Tibet and Assam, India, when this temblor shook the region. Ground cracks, large landslides and sand volcanoes hit in the area. The quake was felt in the Sichuan and Yunnan Provinces of China, and as far away as Calcutta, India.

The quake caused large landslides that blocked rivers. When the rivers finally burst through the walls of debris, waves inundated several villages and killed hundreds of people.

This quake is commonly called the Assam-Tibet earthquake or the Assam earthquake, even though the epicenter was in Tibet. The quake struck at the intersection of the most vigorous collision of continental plates on the planet, where the Indian continental plate smashes into the Eurasian plate and dives beneath it. The slow-motion crash helped create the massive Himalayas.

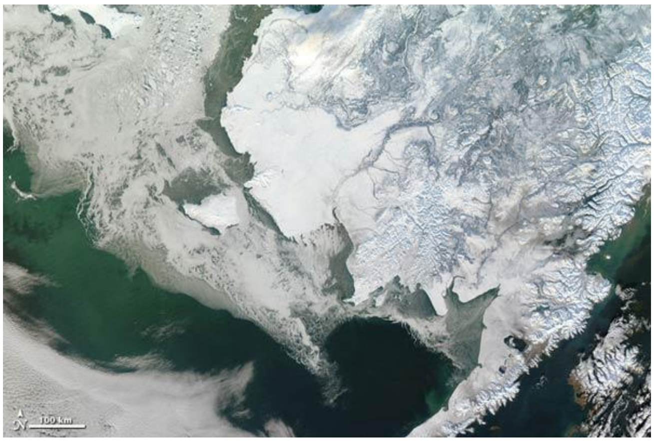
9 - Northern Sumatra, Indonesia, 2005 - Magnitude 8.6

More than 1,000 people were killed, with hundreds more injured, mostly in Nias, in northern Sumatra, Indonesia. The quake hit just months after an even bigger earthquake destroyed the region (see entry #3).



The quake ruptured below the surface of the Indian Ocean, where the Indo-Australian Plate is pushing under the Eurasian plate at the Sunda trench, similar to the 2004 quake.

8 - Rat Islands, Alaska, 1965 - Magnitude 8.7



Alaska had been a state for only 7 years when this huge earthquake triggered a tsunami of over 30 feet (10 meters). Despite its size, the quake caused little damage due to its remote location at the tip of the Aleutian Islands.

The tsunami was reported in Hawaii and spread as far away as Japan.

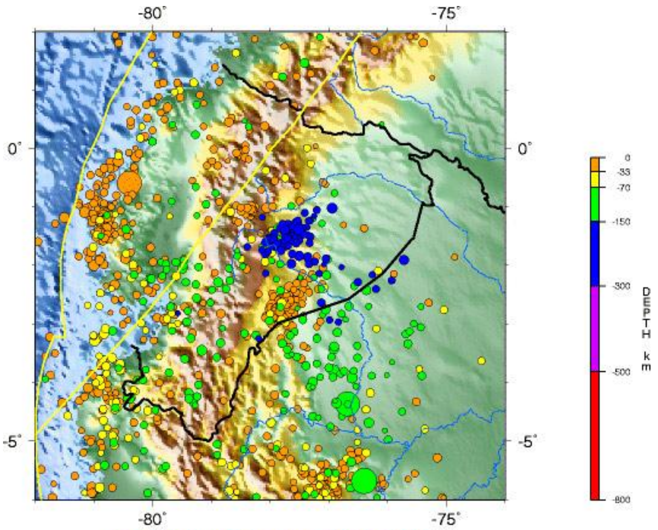
The temblor was the result of the Pacific Plate diving beneath the North American Plate at the Alaska-Aleutian megathrust, which has been the location of many megathrust earthquakes.

The quake cracked wood buildings and split an asphalt runway. Hairline cracks also formed in the runways at the U.S. Coast Guard Loran Station.

7 - Off the Coast of Ecuador, 1906 - Magnitude 8.8

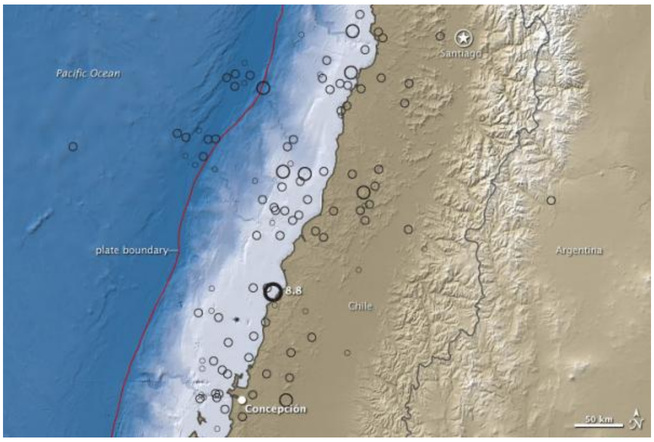
A catastrophic magnitude 8.8 earthquake ruptured off the coast of Ecuador and Colombia and generated a strong tsunami that killed 500 to 1,500 people. The tsunami spread along the coast of Central America, and even stretched to San Francisco and Japan.

The earthquake occurred along the boundary between the Nazca Plate and the South American Plate. It hit more than 100 years ago, so reports are spotty, but according the USGS, witnesses reported a huge rush of water in Honolulu Bay. All the steam and sailboats in the bay were turned around, and then a sudden flood tide roared inland.



Seismicity of Ecuador, 1990 - 2006

6 - Offshore Maule, Chile, 2010 - Magnitude 8.8

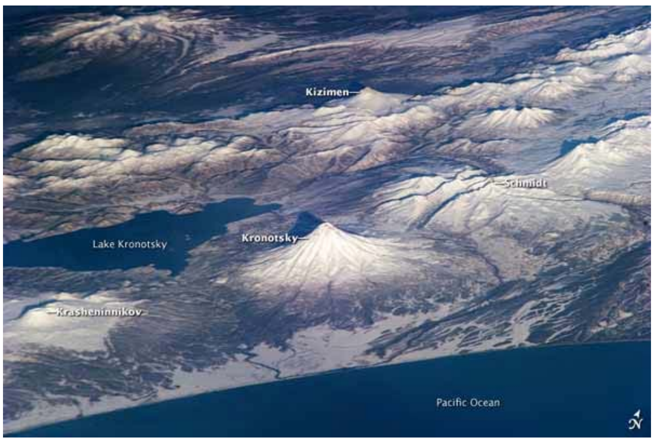


Just last year, at least 500 people were killed and 800,000 were displaced by the earthquake and tsunami that hit central Chile. More than 1.8 million people were affected and the total economic loss was estimated at $30 billion USD. Central Chile is still feeling aftershocks to this day.

The earthquake took place along the boundary between the Nazca and South American tectonic plates.

The quake hit just over a month after the disastrous magnitude 7.0 quake in Port-Au-Prince, Haiti, which killed more than 200,000 people.

5 - Kamchatka Peninsula, Russia, 1952 - Magnitude 9.0

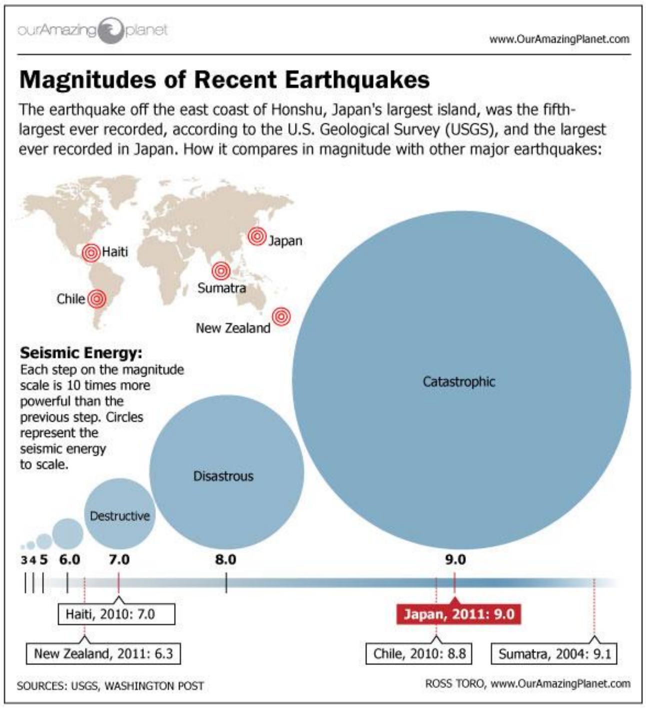


The world's first recorded magnitude 9.0 earthquake struck off the east coast of Kamchatka in 1952. The quake generated a 43-foot tsunami (13 m) locally. The tsunami rocked Crescent City, Calif., which was also hit hard by the recent Japan earthquake.

No lives were lost, but in Hawaii, property damage was estimated at up to $1 million USD. The waves tossed boats onto the beach, caused houses to collide, destroyed piers, scoured beaches and moved road pavement.

Kamchatka has a rumbling past and many active volcanoes. It was also hit by an 8.5 magnitude quake in 1923.

4 - Near the East Coast of Honshu, Japan, 2011 - Magnitude 9.0



On March 11, a magnitude 9.0 quake triggered a tsunami that killed an estimated 29,000 people and damaged some nuclear reactors. This earthquake is the largest ever recorded in Japan.

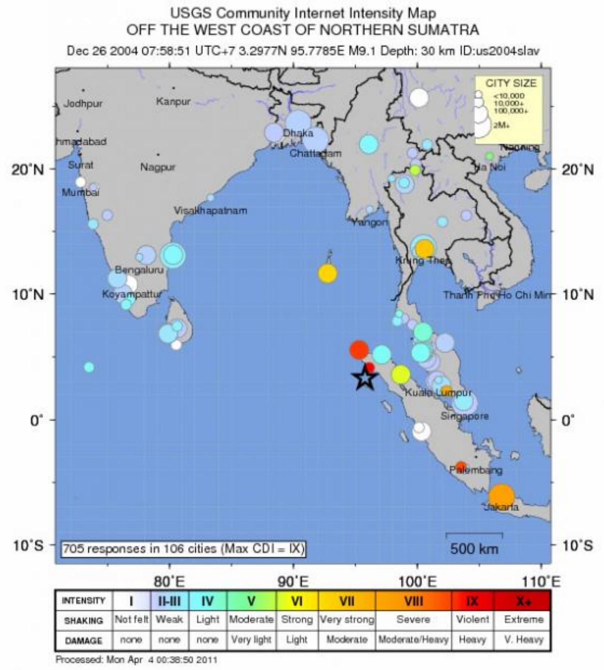
Aftershocks continue to rock the island of Honshu. The aftershocks include more than 50 of magnitude 6.0 or greater, and three above magnitude 7.0.

The quake was caused by thrust faulting near the Japan Trench, the boundary between the Pacific and North America tectonic plates. Thrust faulting happens when one tectonic plate dives under another. In this case, the Pacific plate is diving under the North America plate.

3 - Off the West Coast of Northern Sumatra, 2004 - Magnitude 9.1

This quake was the third largest earthquake in the world, and the largest since the 1964 earthquake in Prince William Sound, Alaska (see entry #2). In total, 227,898 people were killed or missing and presumed dead and about 1.7 million people were displaced by the earthquake and subsequent tsunami in 14 countries in Southeast Asia and East Africa.

The tsunami caused more casualties than any other in recorded history, although some estimates say the death toll from the 2010 Haiti earthquake was larger. The tsunami was recorded nearly world-wide on tide gauges in the Indian, Pacific and Atlantic Oceans.



This quake struck one day after Christmas along the interface of the India and Burma tectonic plates (huge, moving slabs of the Earth's crust) and was caused by the release of stresses that develop as the India plate dives beneath the Burma plate.

2 - Prince William Sound, Alaska, 1964 - Magnitude 9.2

This great earthquake and ensuing tsunami took 128 lives and caused about $311 million USD in property loss. The earthquake damage was heavy in many towns, including Anchorage, which was about 75 miles (120 kilometers) northwest of the epicenter. The quake ruptured along a seismically active fault between the North American and Pacific plates. The shaking lasted about 3 minutes.

Landslides in Anchorage caused heavy damage. Huge slides occurred in the downtown business section and water mains and gas, sewer, telephone and electrical systems were disrupted throughout the area.

1 - Chile, 1960 - Magnitude 9.5

Approximately 1,655 people were killed during the largest earthquake ever recorded. Thousands more were injured, and millions were left homeless. Southern Chile suffered $550 million USD in damage.

The quake triggered a tsunami that killed 61 people in Hawaii, 138 in Japan and 32 in the Philippines.

The earthquake ruptured where the Nazca Plate dives underneath the South American Plate, on the Peru-Chile Trench.

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΛΟΙΠΑ

Elastic Connections May Aid Bridge Design

Shape shifting materials sound like the stuff of science fiction, but they've been known for decades and are used in a wide range of applications, from bio and mechanical engineering to dentistry. Seismic engineering researchers now have latched on to the material and promise to expand its use even more by designing "elastic" bridge connections that would deform during an earthquake but then spring back to its original shape. If successful, the design would prevent serious structural damage or collapse and allow a bridge to remain open after a quake, a time when roads and bridges are critical for emergency response.

Nickel titanium is the primary Shape Memory Alloy civil engineering professor M. Saiid Saiidi is working with at the University of Nevada, Reno, because of its "superelasticity," he says. Other SMAs are generally only temperature sensitive, requiring a heat source to return to its original shape. Nickel titaniumis a bit different and has 10 to 30 times the elasticity of steel or other standard metals. In many ways it performs like steel, but it differs in its ability "to undergo a large deformation and come back to its original shape," Saiidi adds.

Building robust bridges and other significant infrastructure in highly seismic areas such as California and the western U.S. could significantly lessen damages after a large quake. The 1994 Northridge earthquake in southern California caused an estimated $20 billion in damage to roads and buildings.

Shake Tests

With funding from the National Science Foundation using the foundation's George E. Brown, Jr. Network for Earthquake Engineering Simulation, Saiidi and colleagues tested the material at Nevada-Reno's shake table. They used bridge models of 120 feet to 130 feet long, with three types of columns: standard reinforced steel and concrete, nickel titanium and concrete, and nickel titanium and engineered cementitious composites, with cement, fiber, water, and chemicals. The columns were first tested using OpenSEES, and earthquake simulation program developed at the University of California, Berkeley, and then built and tested on the shake table. The columns with nickel titanium outperformed the standard design under forces equaling or exceeding those of a magnitude 6 quake on the Richter scale, Saiidi says.

"In bridges, typically the column is most susceptible [to seismic shaking], specifically at the end zone at the top and at the bottom, by the footing," he says. Because nickel titanium is much more costly than standard steel reinforcing bar, the researchers specified its use only in those zones, a little more than one-tenth the length of the overall column. Nickel titanium bars were assembled into cages just as a standard rebar cage, and connected to the column's main cage in the plastic hinge zones, he says.

Saiidi's test caught the attention of the Federal Highway Administration and Washington State's Department of Transportation, which funded similar research. The agencies are using the design for a highway ramp that is expected to begin construction in 2015, he says.

Completed plastic hinge zone with SMA bars. Image: WSDOT

History

Nickel titanium—the generic name for the family of alloys is nitino—has been around since the early 1960s from work at the Naval Ordnance Laboratory. It has many applications in a number of disciplines. Among other applications, the U.S. military has used nitinol couplers in F-14 aircraft to join hydraulic lines. In medicine, the material is used as a guide for catheters in blood vessels and anchors to attach tendons to bone in orthopaedic surgery. It also is used in robotic actuators and micromanipulators to simulate muscular movement.

"The first time I learned about it was from a mechanical engineer building an actuator," says Saiidi. "Later, the question I asked myself was, 'What if we used it inside a concrete column in a bridge in a high seismic zone?'"

That was about twelve years ago, and Saiidi's research has progressed, also using other copper-based shape memory alloys. He says copper holds promise but civil seismic work is hampered by its scarcity. Only one manufacturer—in Japan—currently fabricates the material.

Shake table testing of a quarter-scale bridge. Image: UNR.edu

Cost-Benefit

Nitinol also costs more than standard steel and suffers from limited sourcing. Saiidi says a bridge built with columns incorporating nickel titanium will cost about 3% more than a structure with standard materials. The savings will come in reduced maintenance costs—nickel titanium is highly resistant to corrosion—and its ability to withstand seismic forces, he says.

"Several manufacturers make SMA wires," he says, noting nitinol is commonly used for orthodontic braces and eyeglass frames that "bounce" back to their original shape if bent. "Only a few make bars." In Washington, the bridge design using the material calls for sizes up to 30 millimeters, he says.

"But if you look at lifecycle costs, it will become economical after an earthquake. You won't have to shut the bridge down," he says. "After an earthquake is when you need bridges the most."

(John Kosowatz, Senior Editor, ASME.org, June 2014, https://www.asme.org/engineering-topics/articles/construction-and-building/elastic-connections-may-aid-bridge-design)

ΝΕΕΣ ΕΚΔΟΣΕΙΣ ΣΤΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΠΙΣΤΗΜΕΣ

Response of Piled Buildings to the Construction of Deep Excavations

Mandy Korff

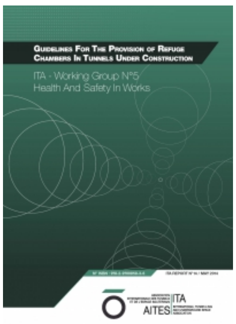
Deltares Select Series, Vol. 13

Deep excavations in densely populated urban areas around the world pose specific challenges due to the increasingly complex conditions in which they are undertaken. The construction of underground car parks, cellar storage areas and major infrastructure in deep excavations helps to preserve the quality of space above ground. Despite the considerable effort that goes into their design and construction, such projects often encounter problems, such as damage to existing structures, delays and cost overruns.

This book presents the results of an extensive research project conducted at the University of Cambridge, in cooperation with the Netherlands Centre of Underground Construction (COB) and Deltares, the Dutch Institute for water, subsurface and infrastructure issues. The study gained insight into mechanisms of soil-structure interaction for piled buildings adjacent to deep excavations and resulted in suggestions for designing and monitoring deep excavations in urban areas with soft soil conditions. Monitoring data of the construction of three deep excavations for the North–South metro line in Amsterdam, the Netherlands, have been used to validate the methods described.

This book aims to contribute to the reduction of failure costs in the building industry and in underground construction in particular.

(IOS Press, 2013)



Guidelines for the Provision of Refuge Chambers in Tunnels Under Construction

ITA Working Group 5

At its meeting in Helsinki in 2011, International Tunnelling Association Working Group 5, identified the need for guidance on the provision of refuge chambers in tunnels under construction. A refuge chamber is a place of relative safety in a shaft or tunnel where tunnellers can be accommodated with access to basic life support services until rescued or it is safe for them to exit the tunnel.

Refuge chambers should be easily identifiable and readily accessible by tunnellers at risk and by the emergency services

This document will be reviewed and revised as necessary in the light of practical experience with the provision of refuge chambers in tunnel under construction.

(ITA, 2014, http://www.ita-aites.org/en/component/k2/1051-guidelines-for-the-provision-of-refuge-chambers-in-tunnels-under-construction)

Guidelines on Best Practices for Segment Backfilling

ITATech

This document is a guideline to the best practices on backfill grouting. It is intended to address the various considerations when applying backfill grout. It does not address the theory or details of applying, or mixtures of, the backfill grout itself, but does give reference for such theory and application.The document gives general guidelines: site and machine-specific guidelines should be developed based on this document as well as the project specifications.

(ITA, 2014, http://www.ita-aites.org/en/component/k2/1045-guidelines-on-best-practices-for-segment-backfilling)

Guidelines on Monitoring Frequencies in Urban Tunnelling

ITATech

Responding to an increasing demand of underground infrastructures monitoring of hydro-geotechnical and structural parameters during the construction of urban tunnels in soils and rock (except hard rock) is a field of activity which has seen impressive technological changes and progress in the past years.

Monitoring has thus become an essential part of the overall risk management which normally is implemented for such type of construction works.

(ITA, 2014, http://www.ita-aites.org/en/component/k2/1046-guidelines-on-monitoring-frequencies-in-urban-tunnelling)

Core Principles of Soil Mechanics - An ICE Textbook

Sanjay Kumar Shukla

Core Principles of Soil Mechanics presents an overview of the fundamentals of soil mechanics, drawing on optimum texts and including numerous illustrations and worked examples. Covers the essential topics of geological, basic, thermal and electrical characteristics; classification; stresses; fluid flow; consolidation; compressibility and shear strength.

(ICE Press, May 2014)

ΗΛΕΚΤΡΟΝΙΚΑ ΠΕΡΙΟΔΙΚΑ

International Society for Soil Mechanics and Geotechnical Engineering

http://www.issmge.org/en/resources/issmge-bulletin/658-vol-8-issue-3-june-2014

Κυκλοφόρησε το Τεύχος 3 του 8ου Τόμου του ISSMGE Bulletin (Ιουνίου 2014) με τα παρακάτω περιεχόμενα:

- Message from CAPG Chairman
- Message from thw President
- From thw Indonesian Society – Activities of the Indonesian Society for Geotechnical Engineering (HATTI)
- From the German Society - 80th Birthday of Prof. Dr.-Ing. habil. Dr.-Ing. E.h. Walter Wittke, Honorary Chairman of the German Geotechnical Society
- Report on ISSMGE Foundation on Conference Attendance
- News on Upcoming Conference – 6th International Conference on Earthquake Geotechnical Engineering (6ICEGE)
- Report from Prof. Askar Zhussupbekov
- News on Recent Conference - 2014 DFI Middle East Conference
- New Books from ICE
- Event Diary
- Corporate Associates
- Foundation Donors
- ISSMGE'S International Journal of Geoengineering Case Histories

International Journal of Geoengineering Case Histories

Vol. 2, Issue 4
http://casehistories.geoengineer.org/volume/volume2/issue4/issue4.html

Christopher T. Senseney "Expedient Mitigation of Collapsible Loess in Northern Afghanistan", pp. 252-257

Abdeltawab Samir "Karst Limestone Geohazards in Egypt and Saudi Arabia", pp. 258-269

Anastasios Batilas, Panagiotis Pelekis, Vasileios Vlachakis, and George Athanasopoulos "Soil Liquefaction/Nonliquefaction in the Achaia-Ilia (Greece) 2008 Earthquake: Field Evidence, Site Characterization and Ground Motion Assessment", pp. 270-287

Phan, T.L., Matsumoto, T., and Nguyen, H.H. "Errata for Comparison of Static and Dynamic Pile Load Tests at Thi Vai International Port in Viet Nam", pp. 288-290

Tara, D., Middendorp, P., and Verbeek, G. "Discussion of Comparison of Static and Dynamic Pile Load Tests at Thi Vai International Port in Viet Nam", pp. 291-297

Phan, T.L., Matsumoto, T., and Nguyen, H.H. "Closure to Discussion of Comparison of Static and Dynamic Pile Load Tests at Thi Vai International Port in Viet Nam", pp. 298-299

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Deepankar Choudhury, Rolf Katzenbach "Editorial", pp. i-ii

Alvin K.M. Lam, Daman D.M. Lee "Combined Pile Foundation System for a Residential Complex", pp. 1-9

Biswas, S., Choudhary, S.S., Manna, B., and Baidya, D.K. "Field Test on Group Piles under Machine Induced Coupled Vibration", pp. 10-23

Bhattacharya, S., and Tokimatsu, K. "Collapse of Showa Bridge Revisited", pp. 24-35

Phan, T.L., Matsumoto, T., and Nguyen, H.H. "Comparison of Static and Dynamic Pile Load Tests at Thi Vai International Port in Viet Nam", pp. 36-66

No. 26 - June 2014
http://www.isrm.net/adm/newsletter/ver_html.php?id_newsletter=97&ver=1

Κυκλοφόρησε το Τεύχος 26 / Ιούνιος 2014 του Newsletter της International Society for Rock Mechanics. Περιεχόμενα:

- ARMS8 - 2014 ISRM International Symposium, Sapporo, Japan, 14-16 October 2014
- VI Brazilian Rock Mechanics Symposium, Goiβnia, Brazil, September 9-13, 2014 an ISRM Specialized Conference
- Puzzles in rock – Have they been solved?" - the 6th ISRM Online Lecture by Prof. Herbert Einstein
- Rock Mechanics and Empirical Methods in Rock Engineering - by Dr. Nick Barton. A short course organised by the ISRM NG Singapore
- ISRM Commission on Petroleum Geomechanics
- ISRM is on Twitter and Linkedin

- Dedication Ceremony for ISRM Past-President John Franklin
- 80th Birthday of Prof. Dr.-Ing. habil. Dr.-Ing. E.h. Walter Wittke, Honorary Chairman of the German Geotechnical Society
- 13th International ISRM Congress, May 2015, Montreal, Canada
- AusRock 2014: 3rd Australasian Ground Control in Mining Conference, Sydney, 5-6 November 2014
- 3rd ISRM International Young Scholars' Symposium on Rock Mechanics, Xi'an, 8-11 November 2014 An ISRM Specialized Conference
- ISRM Rocha Medal 2016 - nominations to be received by 31 December 2014
- ISRM sponsored meetings
- EUROCK 2014 was held in Vigo, Spain
- Tunisia, the youngest ISRM National Group, presents its 1st activity report

www.geoengineer.org

Κυκλοφόρησαν τα Τεύχη #112 και #113 του **Newsletter του Geoengineer.org** (Ιούνιος και Ιούλιος 2014 αντίστοιχα) με πολλές χρήσιμες πληροφορίες για όλα τα θέματα της γεωμηχανικής. Υπενθυμίζεται ότι το Newsletter εκδίδεται από τον συνάδελ-φο και μέλος της ΕΕΕΕΓΜ Δημήτρη Ζέκκο (secretariat@geoengineer.org).

Συγκεκριμένα, στο Τεύχος #112, Ιουνίου 2014, υπάρχουν αναφορές στα παρακάτω ενδιαφέροντα γεωτεχνικά θέματα:

The 2014 Terzaghi Lecture, delivered by Prof. J. C. Santa-marina, during GeoCongress 2014 "Energy Geotechnology: Enabling New Insights Into Soil Behavior"

The 2nd Annual de Alba Lecture, delivered by Prof. Stokoe "The Increasing Role of Seismic Measurements in Geotechnical Engineering"

The two GEER Reconnaissance Reports, on the Cephalonia island, Greece earthquakes 2014, and the March 2014 Flood Event Post 2010-2011 Canterbury Earthquake Sequence, New Zealand

New Youtube video from Keynetix: Modelling Geology fault in AutoCAD Civil 3D

Video: Mudslide in Minneapolis undermines hospital's stability

Στο Τεύχος #113, Ιουλίου 2014, υπάρχουν αναφορές στα παρακάτω ενδιαφέροντα γεωτεχνικά θέματα:

Massive landslide in western India, killing 25, attributed to extensive deforestation

Geosynthetic Reinforced Soil - Integrated Bridge System (GRS-IBS)

Magnitude 6.3 quake hits eastern Mexico - no damage reported

CPT Testing, the Piezocone and Measuring Soil Moisture

Στο Τεύχος #108, Ιανουαρίου 2014, υπάρχουν αναφορές στα παρακάτω ενδιαφέροντα γεωτεχνικά θέματα:

Buildings prone to cliff collapse are being demolished in Port Hills, Christchurch

Video shows derailed train in Italy ready to fall in the ocean due to landslide

Ground collapse near riverbank in Brazil [video]

New study shows 16 earthquakes were triggered by landslide

India TBMs blamed for building cracks

Landslide in Mexico results in coastal highway collapse [video]

"Bubble curtains" around Tappan Zee Bridge's pilings protect fish from noise [video]

Huge underwater landslide contributed to Japanese devastating tsunami

Στο Τεύχος #109, Φεβρουαρίου – Μαρτίου 2014, υπάρχουν αναφορές στα παρακάτω ενδιαφέροντα γεωτεχνικά θέματα:

Massive mudslide east of Arlington causes casualties and buries homes under the mud

Unsaturated Soils Theory in the Undergraduate Civil Engineering Curriculum

Tom Lantos twin tunnels provide a sustainable solution to the Devil's slide problem

The 4.4 magnitude earthquake in L.A. causes scientific surprise

Climate change may increase landslide risk by up to 25% in the Gota river valley

Rockfall at Birling Gap, U.K. is caught live!

Powerful storms trigger mudslides in hills scarred by Colby fire last January

Landslide in Sardoa, Brazil, caught live, as crews were searching for survivors

Sinkhole damages eight vintage valuable Corvettes at National Corvette Museum, in Bowling Green, Kentucky

Cephalonia earthquakes: EERI and GEER to investigate the seismic activity impact

Cephalonia earthquake, preliminary damage observations from Geoengineer.org's geotechnical engineer

Second Major Earthquake hits Cephalonia island in Greece; causes significant damage; information hub established on GeoWorld; updates to follow

Pile-driving to blame for building cracks in Seattle [video]

Στο Τεύχος #110, Απριλίου 2014, υπάρχουν αναφορές στα παρακάτω ενδιαφέροντα γεωτεχνικά θέματα:

Sinkhole in Florida re-opens under two homes a few days after its filling with cement

[Slow moving landslide in Jackson, Wyoming, urges for evacuation of the area]

[Mudslide in Tajikistan kills six children and buries village under the mud]

[A West Coast earthquake early warning system at stake due to lack of funding, as Japan's is active since 2007]

[Opening the classroom to the geo-profession 2: Web-based class projects on Ground Improvement prepared by University of Michigan students are available now for everybody to review!]

[A landslide in Jalisco Mexico is caught live by local driver]

[Anti-liquefaction pilot program in Canterbury, New Zealand]

[8.2 quake hits northern Chile, triggering tsunami warning]

[La Habla earthquake and the role of social media in the information transmission]

[The mystery behind the stone spheres of Costa Rica]

[50th anniversary of the 9.2 magnitude Great Alaska Earthquake and Tsunami]

Στο Τεύχος #111, Μαΐου 2014, υπάρχουν αναφορές στα παρακάτω ενδιαφέροντα γεωτεχνικά θέματα:

[Major earthquake hits Mexico, most likely an aftershock of April 18th event]

[Massive landslide in Japan, recorded on video!]

[Video: Building under construction in South Korea tilts 5 times more than the tower of Pisa]

[Railroad retaining wall collapses and is caught live on video!]

[Illegal gold mine collapses in Colombia, trapping 30 people]

[A 6.0 magnitude earthquake hits northern Thailand, causing significant damage]

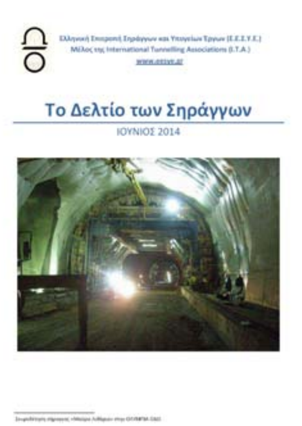
[Jet Grouting Video by Hayward Baker]

INTERNATIONAL TUNNELLING AND UNDERGROUND SPACE ASSOCIATION
ita@news n°54, June 2014
www.ita-aites.org

Κυκλοφόρησε το Τεύχος No. 54 – Ιούνιος 2014 των ita@news της International Tunnelling Association με τα παρακάτω περιεχόμενα:

- Message from Soren Degn Eskesen, ITA President
- 2014 Annual Meetinh - Iguassu
- WTC 2015 is On Track
- International Tunnelling Awards
- Photo Contest Winners

Ελληνική Επιτροπή Σηράγγων και Υπογείων Έργων (Ε.Ε.Σ.Υ.Ε.)
Μέλος της International Tunnelling Associations (I.T.A.)
www.eesye.gr1



Κυκλοφόρησε το Τεύχος 1 του 6ου Τόμου του ηλεκτρονικού περιοδικού της ΕΕΣΥΕ (Ιουνίου 2014) με ανακοινώσεις για τις δραστηριότητες της ΕΕΣΥΕ και ενδιαφέροντα νέα για της σήραγγες και τα υπόγεια έργα ανά τον κόσμο.

www.itacet.org/Newsletter/19_2014/index.php

Κυκλοφόρησε το Τεύχος No. 19 (Ιούλιος 2014) του ITACET Foundation με τα παρακάτω περιεχόμενα:

- President's address
- ITACET Foundation Award
- Training Session in Mexico City
- Training Session at the WTC 14
- A Blog of Interest

  Mr. Senthil Nath G T, current beneficiary of the ITACET Foundation Grant to follow the Specializing Masters in Tunnelling and Tunnel Boring Machines in Politecnico di Torino, opened a blog to share his experience and reference materials with other prospective students or interested engineers.

  His blog "Geotech and Tunnel Engineering , Experiences & Excerpts; Ideas & Impressions; Reviews & Ramblings of a Civil Engineer " is very lively, full of essential references and original content. It is sure to be of interest to any professional of the tunneling and underground space domain. Visit it here : http://geo-technical.blogspot.it

Geosynthetics International

www.thomastelford.com/journals

Κυκλοφόρησε το τεύχος αρ. 3 του 21ου τόμου (Ιουνίου 2014) του περιοδικού **Geosynthetics International** με τα ακόλουθα περιεχόμενα:

"Hyperbolic models for a 2-D backfill and reinforcement pullout", C.-C. Huang; H.-Y. Hsieh; Y.-L. Hsieh

"Performance of reinforced soil walls during the 2011 Tohoku earthquake**"**, J. Kuwano; Y. Miyata; J. Koseki

"Performance of three GCLs used for covering gold mine tailings for 4 years under field and laboratory exposure conditions", M.S. Hosney; R.K. Rowe

"A parametric study of geosynthetic-reinforced column-supported embankments", N.N.S. Yapage; D.S. Liyanapathirana

"Treatment of an expansive soil by mechanical and chemical techniques", A.R. Estabragh; H. Rafatjo; A.A. Javadi
Please find the download of the articles at:
http://www.icevirtuallibrary.com/content/issue/gein/21/3
For the IGS members to have FREE access to the papers they MUST log in through the IGS website.

Geotextiles & Geomembranes

www.geosyntheticssociety.org/journals.htm

Κυκλοφόρησαν τα τεύχη αρ. 3 και 4 του 42ου τόμου (Ιουνίου και Αυγούστου 2014) του περιοδικού **Geotextiles & Geomembranes** με τα ακόλουθα περιεχόμενα:

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Sébastien Bourgès-Gastaud, Guillaume Stoltz, Fabienne Sidjui, Nathalie Touze-Foltz "Nonwoven geotextiles to filter clayey sludge: An experimental study", Pages 214-223

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S.G. Chung, H.J. Kweon, W.Y. Jang "Observational method for field performance of prefabricated vertical drains", Pages 405-416

Technical Note

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Please find the download of the articles at:
http://www.sciencedirect.com/science/journal/02661144/42/3
http://www.sciencedirect.com/science/journal/02661144/42/4
For IGS members to have FREE access to the G&G journal articles they MUST log in through the IGS website.

http://www.icold-cigb.org/article/GB/News/newsletter/newsletter-14

Κυκλοφόρησε το Τεύχος 14 (Μάιος 2014) του **The Dams Newsletter** της **International Commission on Large Dams** με τα παρακάτω περιεχόμενα:

- Editorial
- News about Dams
  - Grand Inga: a dream comes true, p.3
  - China: accelerating the building of hydroelectric dams, p.6
  - ICOLD European Club news: Summary of activities during the period 2011-2013, p.7
  - ICOLD meets in Seattle with great success, p.8
- ICOLD News
  - ICOLD President answers Oxford misleading study, p.9
  - ICOLD activities, p.12

International Association for Engineering Geology and the Environment Newsletter
Issue No.1, 2014
http://www.iaeg.info/index.php/newsletter/cat_view/110-iaeg-newsletter/193-iaeg-newsletter-2014

Κυκλοφόρησε το νέο τεύχος του Newsletter της **International Association for Engineering Geology and the Environment** με τα παρακάτω περιεχόμενα:

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ΕΕΕΕΓΜ
Τομέας Γεωτεχνικής
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Πολυτεχνειούπολη Ζωγράφου
15780 ΖΩΓΡΑΦΟΥ

Τηλ. 210.7723434
Τοτ. 210.7723428
Ηλ-Δι. secretariat@hssmge.gr ,
geotech@central.ntua.gr
Ιστοσελίδα www.hssmge.org (υπό κατασκευή)

«ΤΑ ΝΕΑ ΤΗΣ ΕΕΕΕΓΜ» Εκδότης: Χρήστος Τσατσανίφος, τηλ. 210.6929484, τοτ. 210.6928137, ηλ-δι. pangaea@otenet.gr, ctsatsanifos@pangaea.gr, editor@hssmge.gr

«ΤΑ ΝΕΑ ΤΗΣ ΕΕΕΕΓΜ» «αναρτώνται» και στην ιστοσελίδα www.hssmge.gr