Rock Mass Characterization and Assessment of Ground Behavior for the Trikokkia Railway Tunnel in Central Greece

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ABSTRACT: *The Trikokkia Railway tunnel is a 5000 m long single tube tunnel, foreseen to be constructed along the future Kalambaka–Kozani railway line in central Greece. The tunnel alignment is crossing the Molasse formations of the meso-Hellenic trench and a design approach for the tunneling works that account for the specific properties of this formation has been adopted. Mixed modes of failure governed by rock overstressing and discontinuity controlled slides were tackled by considering both rock bulk strength and deformability properties and rock mass architecture. The ground characterization process, was performed using well established empirical rock mass characterization indices. Rock mass characterization was assisted by hand sketching of rock mass structure as well as by generating discrete fracture networks, compatible with the joint statistics of each rock mass class. The identification of different rock mass behavior types is based on metrics that compare rock mass bulk strength with in situ stress state, and in parallel on the visualization of the rock mass structure to describe and quantify structurally controlled failure modes. Both the intuitive procedure to draw rock mass structure sketches, as well as the random joint generation procedure, are found to be effective tools to identify and assess gravity driven modes of failure. The adopted process to assess rock mass behavior contributes significantly to design with confidence the tunneling and support process for tendering purposes.*

KEYWORDS: Rock Tunneling, Rock mass Characterization, Rock mass Behavior, Discrete Fracture Networks, Molasse.

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

Prediction of ground response to tunneling is the most essential task of the design of tunneling works. Assessment of ground behavior is inevitably associated with ground characterization, which is the process of quantified description of the ground conditions anticipated along a proposed tunnel alignment for engineering design purposes. The outcome of ground characterization is used in subsequent design stages, as direct input for empirical design methods or for numerical analyses used to verify and adjust design decisions. Subsequently, ground characterization during construction forms the basis to quantitatively describe encountered ground conditions, interpret observed rock mass response and update the design accordingly.

Review of recent literature on rock mass characterization reveals two diverging schools of thought: The first may be called "intuitive", insists to employ simple rock mass characterization tools based on geological reasoning, which, however require engineering geology expertise for their use. The second may be called "quantitative" and exploits rock mass structure measurements and quantitative indicators of rock mass properties, to characterize the rock mass in a less subjective and more dependable manner. In the same lines, facets of rock mass behavior may be assessed in a more intuitive or in a quantitative manner. In the first case, the use of well-established rock mass rating systems (RMR, GSI etc) form the basis to derive design parameters and consequently simple equivalent continuum analysis are used as a guide to assess development of stress-induced modes of failure. Hand drawn sketches of the rock mass structure may also provide the means to demonstrate how certain geologic features will affect the development of gravity driven modes of failure and to guide the selection of tunnel support measures. In the latter case, a more quantitative approach would employ a rating system which is based on measurable parameters of rock mass structure to assess the design parameters more objectively.

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Discrete fracture networks (DFN), produced on the basis of rock mass discontinuity statistics, can be used to visualize rock mass fabric (in lieu of hand drawn sketches) and to provide a sound basis to identify prevailing failure mechanisms. During the final design stages, these discrete fracture networks (DFN) can also be used as a basis for discontinuoum numerical analysis.

Undeniably, both approaches have their merits and they should not be perceived as alternative, but merely as complementary methods. Deep understanding of geology should not be overlooked in any underground project and engineering geology assessments and intuition, should find their way in the design process. On the other hand in long tunneling projects when large amount of data is available a quantitative approach becomes more feasible and can easier be linked with probabilistic analysis and assessment of tunneling risks. This paper presents the design of the Trikokkia railway tunnel, a 5000 m long tunnel, foreseen to be bored in molassic formations in central Greece, where elements from both approaches were used in a complementary manner, leading to a confident and dependable design of the tunneling works for tendering purposes.

THE TRIKOKKIA RAILWAY TUNNEL

For the extension of the existing north-south railway axis of the Greek network towards western Greece, two new lines are planned by the Greek Railway Organization (OSE). These lines will eventually link the current OSE terminal in the city of Kozani with the cities of Kalambaka, Ioannina and the Port of Igoumenitsa, located on the Ionian Sea coast (Figure 1). The Trikokkia railway tunnel is one of the longest tunnels foreseen to be bored along the new Kalambaka–Kozani section, with a length of 5000 m. It is located approximately 50 km north to the city of Kalambaka, between the villages of Theotokos and Trikokkia. The tunnel is planned as a single track tunnel, of uni-directional operation to accommodate passenger and freight rail traffic. A number of escape galleries, at regular intervals of 1000 m or less are foreseen to provide emergency exit pathways from the running tunnel. The location and layout of the proposed railway tunnel are outlined in Figure 1.

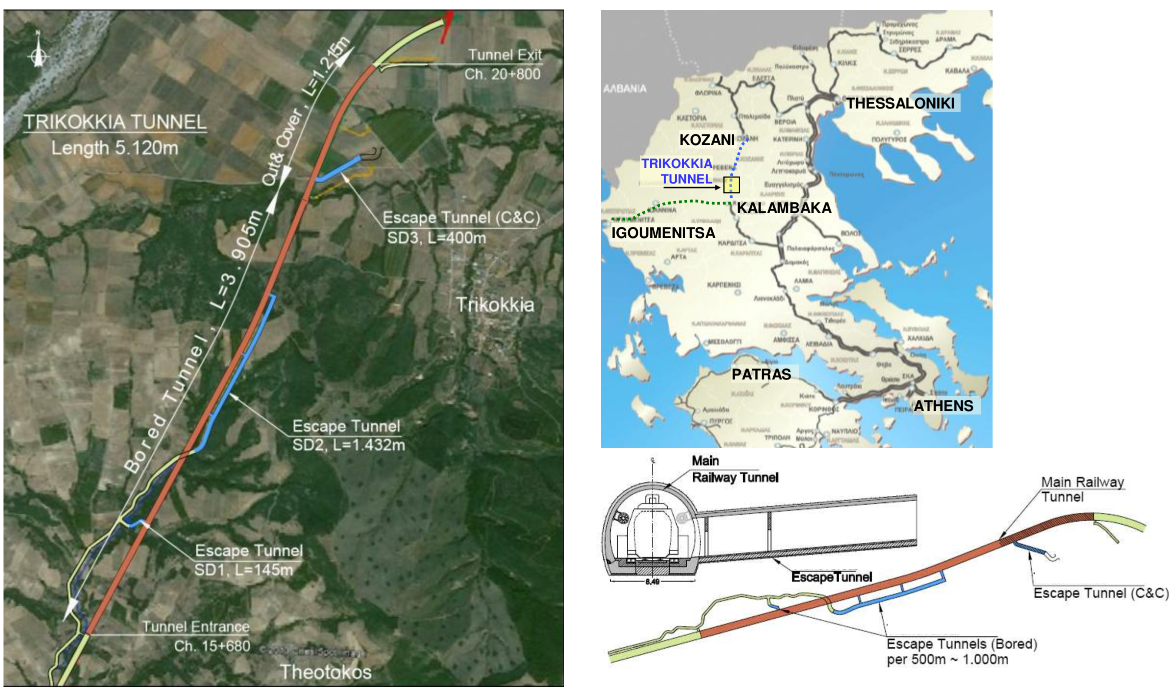


Figure 1. Location, layout and typical section of the Trikokia railway tunnel. The inset presents an outline of the Greek Railway Network and the proposed new lines of the future Western axis (dashed lines), linking the port of Igoumenitsa to the cities of Kalambaka and Kozani.

The proposed tunnel is long enough (5000 m) to justify the deployment of a Tunnel Boring Machine, to mechanize the construction. However, since the last kilometer of the tunnel runs through soil formations of low overburden, it is feasible and more efficient to construct this particular tunnel stretch by the cut and cover method. Moreover, considering that the evacuation galleries can be constructed in advance to provide access to a number of intermediate excavation fronts, the drill and blast method of construction becomes equally attractive with the mechanized option. A dual design approach for the procurement of the tunnel project was proposed to stimulate contractors to focus on performance and promote competition.

The tunnel cross section for the TBM and the NATM options is presented in figure 2, but this presentation will focus on the NATM option. A full review and more detailed account of the planning issues of this tunneling project is presented by Alexandris et al. (2014).

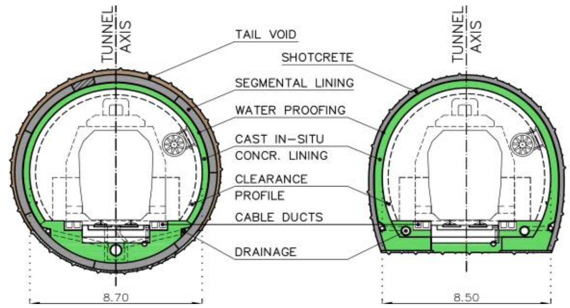


Figure 2. Alternative cross sections of the proposed Trikokkia Tunnel (TMB and NATM sections)

REGIONAL GEOLOGY AND GEOTECHNICAL CONDITIONS

The Trikokkia tunnel is crossing mainly the molassic sediments of the Mesohellenic trough. The Mesohellenic trough is a 130 km long and about 30 km wide subsidence area, which runs in a NE-SW direction, parallel to the isopic zones of the Hellenides. The molassic formation of the trough is a sequence of fairly undisturbed clastic sediments, mainly medium to fine sandstones, siltstones, mudstones, and conglomerates. These sediments have been deposited after the paroxysmic phases of the Alpine orogeny and they have not been significantly compressed or thrusted over. However, close to the western border of the trench, in the vicinity of the contact of the trough with the Alpine flysh, thrust faults have been active after the deposition of the molassic clastic sediments, disturbing fairly locally the molasse. In the area of interest, the Mesohellenic trough is divided in two small sub – basins, separated by an Oligocene age thrust fault: the Theotokos thrust (Doutsos et al., 1994). The western sub-basin is filled with sediments of the “Eptachori” formation, consisting of fine to medium sandstones with siltstone interbeds. The Eastern basin is filled with the sediments of the “Tsotylio” formation, which consists of thick layers of polygenetic conglomerates of variable degree of cementation and diagenesis, interbedded with coarse to medium sandstones. The Theotokos thrust is trending SW-NE and carries also ophiolithic rocks above the “Eptachori” formation. The Northern section of the tunnel reaches the plain of “Karpero”, which is an area covered by alluvial deposits, mainly dense sands and dense silts. The geological setting of the project area is depicted in the synthetic map of Figure 3, which is based on the combination of the regional geological map of the project area (scale 1:50.000-IGME 1979) and the ground relief.

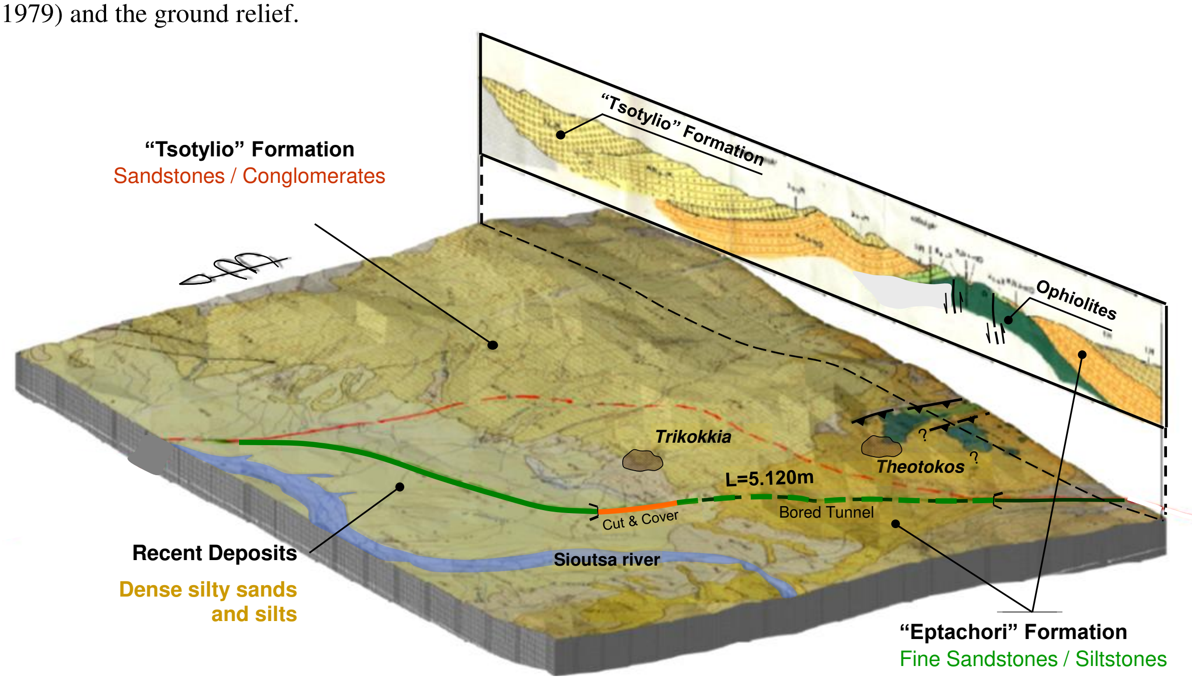


Figure 3. General geological conditions along the tunnel alignment. Geology after the Geological Map of Greece, sheet Theotokos, 1:50.000 IGME 1979, slightly modified.

Field observations in the area of interest, did not reveal any wealth of information concerning geology and tectonics, due to the absence of natural or man-made rock exposures in the area. The weak sandstones and siltstones of the molasse disintegrate rapidly to soil when exposed to the surface and hence, the presence of a thick soil cover, in most sites, prohibited direct rock mass observation. Even when rock outcrops were spotted, the quality of rock is known to have been significantly affected by weathering and slaking (Hoek et al., 2005) and these observations were treated with caution. However a comprehensive geotechnical site investigation campaign, which was undertaken along the proposed tunnel alignment, in combination with the piecemeal field observations on rock outcrops and cuttings, provided sufficient data to establish a reliable design basis. The site investigation works comprised 25 exploratory boreholes (some of them reaching 150 m in depth) with 1350 m of total core run. During borehole sinking, numerous in situ borehole deformability tests (dilatometer) and permeability tests were executed and the field works were followed by a comprehensive laboratory testing program on retrieved rock and soil samples.

The tunnel longitudinal profile of the tunnel presented in Figure 4, has been produced by combining both the subsurface data retrieved by the exploratory boreholes and the field geological mapping. For the first 4000 m of its length, the proposed tunnel is crossing the Molassic strata of the “Eptachori” formation, consisting mainly of fine sandstones with siltstone interlayers, while the last kilometer is crossing the recent alluvial deposits of the Karpero plain, comprised of dense fine sands and silts. A more detailed examination of the geotechnical conditions along the tunnel route, reveals that in the first 1500 m of tunnel run, the overburden height is small and the molasse is more variable, characterized by alternations of thin bedded sandstones and siltstones. Beds of well cemented conglomerates between siltstones and claystones are also present and a tunnel stretch which is crossing a layer of stiff clayey marl that was identified as a potential source of tunneling problems. In the remaining 2500 m long section of the tunnel route through the molasse, where the overburden is higher, medium strength massive sandstones with occasional siltstone beds dominate and the geotechnical conditions are generally more favorable. The hydrogeologic regime along the tunnel route is characterized by a groundwater table which follows the ground relief. Given the low permeability of the molasse, the water inflows in the tunnel are expected to be limited and controllable.

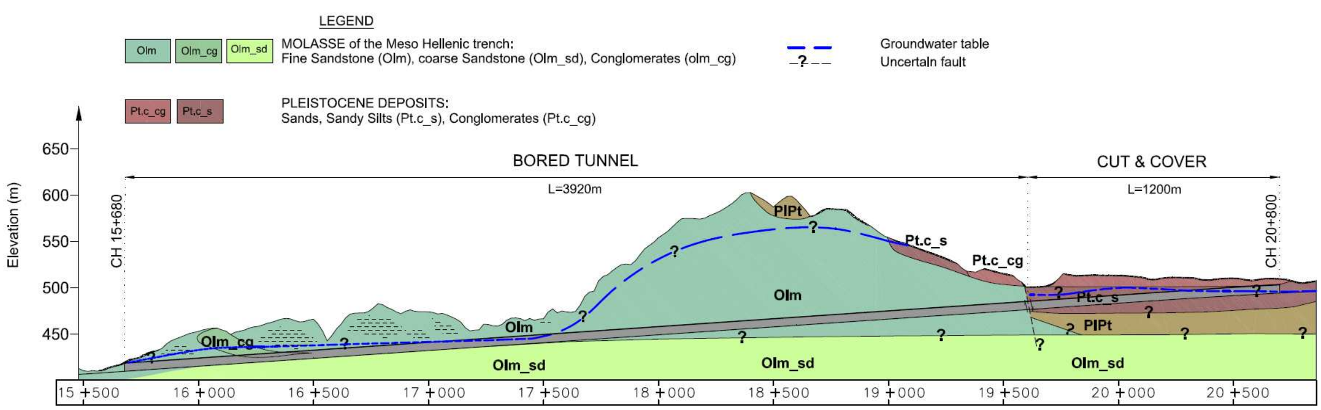


Figure 4. Tunnel Geological profile. Vertical exageration 4:1.

ROCK MASS CHARACHTERISATION

Tunnel design is based currently on empirical methods to a priori evaluate the response of the rock mass to tunneling and to assess the required support. From the mid ’70s, rock tunnel (and mine) design was based on empirical rock mass rating systems (RMR, Q). These were developed for direct tunnel or mine design, through design charts and tables of support requirements, produced largely on past experience. Despite the popularity of the direct design methods, it soon became apparent that this approach leaves little space for analysis, tailoring and optimization. The development of numerical analysis tools provided gradually the means to evaluate the adequacy of the proposed support systems and the tools to optimize them. This development was followed by a growing need to determine representative rock mass design parameters, compatible with the available numerical tools. From the mid 80’s rock mass characterization systems have evolved from the pre-existing rating systems-benefitting from past experience-and were primarily focused on the assessment of rock mass strength and deformability parameters. Subsequent trends are broadening the scope of ground characterization process, incorporating also behavioral parameters, targeting to the establishment of a general geotechnical baseline for tunnel design.

For the particular project, two systems were employed for rock mass characterization, namely GSI (Hoek et al., 1995), and RMi (Palmström, 1995). The GSI index has been evolved from RMR rating system by gradually expanding its initial field of application from more competent rock masses to weaker ones (Hoek et al., 1998) and by the development of various charts to assist the engineering geologist to cope with complex geological formations such as flysh, (Marinos and Hoek, 2001), molasse, (Hoek et al. 2005), ophiolites (Marinos et al. 2004) as reviewed by Hoek and Marinos (2000) and Marinos et al. (2005). In the design and construction practice, GSI soon became very popular due to both its simplicity and its geological logic, which provide dependable assessments when engineering geology expertise is used. However some engineering geologists are skeptical about the subjective manner on which the GSI is assessed in practice, while many engineers are critical to the simplistic character of diagram recognition method (Barton 2010), which gives ground for raising contractual claims during construction (Carter 2010). Attempts to quantify the GSI chart by various practitioners, including E.Hoek himself, demonstrates the need for a classification procedure that would not rely strongly on user's expertise.

The RMi index has been evolved along the lines of the Q system and uses as characterization parameters the average block volume of the rock mass and a parameter expressing the discontinuities characteristics. The system is believed to be less subjective, but it is considered by many too complex to use and that it lacks the intuitive input of GSI. To cope with these issues Cai et al. (2004) proposed a more quantitative version of the GSI chart, which considers the average block volume (V_b) as rock mass structure metric and the joint condition factor (Jc) as a descriptor of joint condition. The affinity of this system with the RMi, led Russo (2008) to propose a combined method to assess the index. In the same direction Hoek et al., (2013) proposed their own quantification of the GSI chart, which uses the RQD as a rock structure descriptor and the $Jcond_{89}$ rating of the RMR_{89} rating system as joint surface descriptor. This chart is indeed easier to use, but it fails to use fundamental rock mechanics parameters, such as the average block size and a quantifiable joint parameter. For this reason, the GSI chart proposed by Cai et al. (2004) was implemented on this particular project and both quantitative and qualitative input was used for GSI assessment.

The rock mass types anticipated along the tunnel alignment were categorized by considering prevailing rock mass lithology and by assessing the feasible range of joint density and quality. As described in the previous section, a major portion of tunnel length intersects massive sandstones with some siltstone interbeds, represented by rock types SS1-SS4 to cover various degrees of jointing. Along the first, low overburden section of tunnel alignment, alternations of conglomerates with claystones and weak sandstones are anticipated and represented by rock types ASS1 (prevailing weak sandstones) and ASS2 (prevailing claystones). Some tunnel stretches through weak or disintegrated conglomerates are also anticipated and are represented by rock type DCG. The presentation that follows focuses on sandstone/siltstone units (SS) being the most significant for this project, without implying that they are the only rock mass types studied.

Figure 5 depicts the distinguished sandstone rock mass types (SS1-SS4) on the GSI chart, aside photographs of rock outcrops of the same formation. Molassic units, are encountered either sound/undisturbed (registering very high GSI values), or weathered/disturbed by slaking, usually encountered at shallow depths and slopes (registering low GSI values), as reported also by Marinos et al. (2004). However, in this particular geological area, at the western margin of the meso-Hellenic trench and at close proximity of the Theotokos thrust, the molasse is found at places in a more distressed condition than in most sites to the North. The retrieved cores from the boreholes sunk along the tunnel alignment, showed that the entire spectrum of rock mass structure, from massive to blocky disturbed, may be encountered (classes SS1 to SS4) during tunneling. Joint conditions are generally good in such rock masses, but poor joint conditions, associated with the presence of siltstone beds or with weathering and slaking of the sandstone-siltstones units, are also anticipated and this is reflected by a gradual shift to the right on the GSI chart for the poorer rock mass classes. Figure 6 shows the distribution of RQD with depth in five of the exploratory boreholes. The presence of narrow zones of disturbed molasse, registering very low RQD values, is emphatically noted in this figure. The succession of these zones of fractured molasse with massive rock units is an imprint of the true meso-structure of the actual geological formation, which cannot be described solely by a single rock mass rating/indexing number.

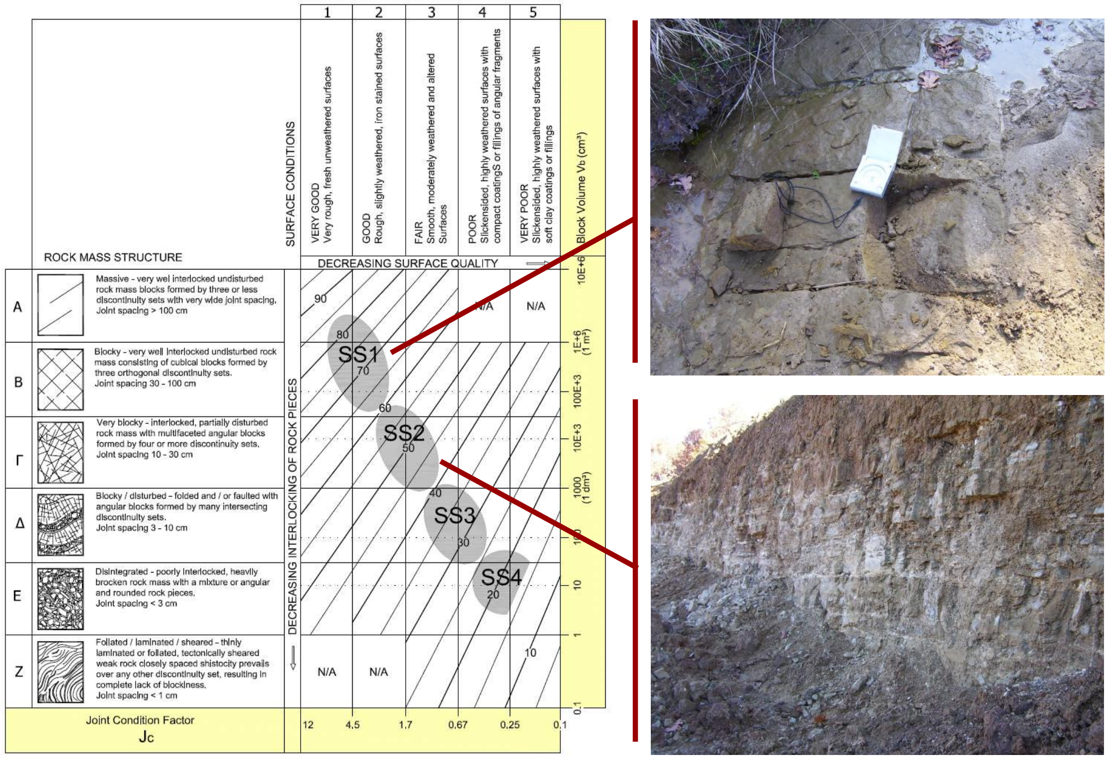


Figure 5. GSI chart and Rock Mass Types at the sandstone layers of the molasse

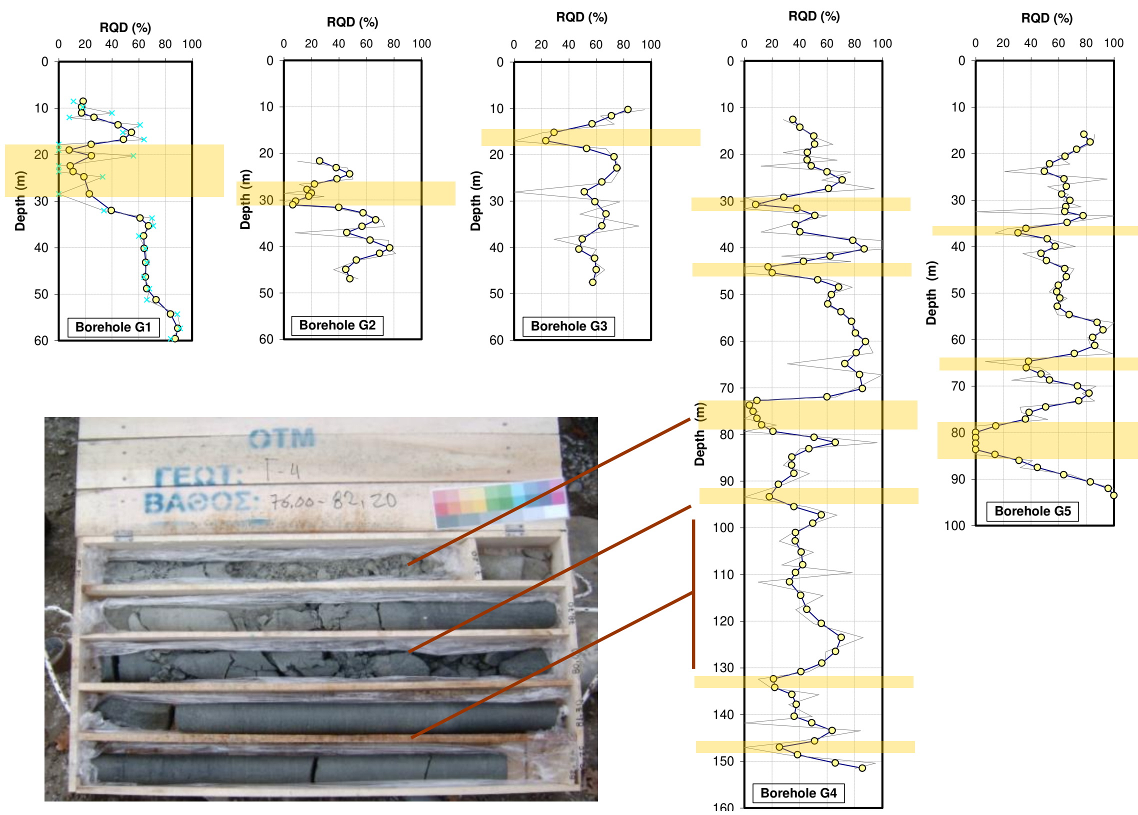


Figure 6. Identification of intense fracturing zones in the exploratory boreholes.

Lithological alternations, mainly between fine sandstones and siltstones, although not systematic to necessitate the use of the charts introduced by Hoek et al. (2005) for inhomogeneous molasse, in some zones are an essential rock fabric characteristic. The presence of siltstone beds even when they are widely spaced introduce anisotropy which will evidently affect rock mass behavior and it was felt that it is not fairly represented by a singular assigned value of GSI (or RMi). To this end, generic sketches of rock mass structure at tunnel scale were produced for all the distinguished rock mass classes, with an effort to reflect surface and subsurface observations and measurements, consistent with the broader geologic and tectonic regime. The aim of these sketches is to provide a basis to intuitively assess rock mass response to tunneling, considering both gravity driven modes of failure as well as shear failures from overstressing. Sample sketches for rock mass classes, along with typical core runs are depicted in Figure 7. Systematic inspection of the borehole cores (Figure 6) showed that intensively fractured zones are of limited width, and hence a low GSI value of the rock mass around the entire tunnel would not reflect in a fair way rock mass properties and tunneling conditions, unless assigned to distinct zones. This is reflected to the sketches of classes SS3 and SS4 depicted in Figure 7.

Rock mass types were in parallel visualized by means of random joint generation procedure, based on joint statistics performed on discontinuity surveys on borehole cores and rock outcrops. This is a less subjective procedure to describe rock mass structure, which in this case depends largely on joint spacing, persistence and orientation. By evaluating discontinuity stereonets from joint surveys on rock outcrops, it became apparent that, apart from bedding, the joints can be grouped in one or two joint sets (sub-vertical/sub-horizontal). In that respect all the joints identified in the borehole cores were characterized as i) bedding ii) sub-horizontal joint set iii) sub-vertical joint set. Consequently joint data collected by surveying more than 200 m of core runs, retrieved in the fine sandstones of the molasse, which was the prevailing lithological unit along the tunnel alignment, were statistically analyzed. For the joint statistics, the rock core runs were allocated in distinct classes (bins), on the basis of lithological similarities and specific GSI and RMi ranges. As stated previously, spatial variation of joint frequency showed that the most intensely fractured zones were only a few meters wide and could not represent the rock mass in tunnel scale. Hence, while collecting data for joint statistics three bins were considered namely; class SS1 (GSI 60-80), class SS2 (GSI 40-60) and fracture zones with GSI<40. Classes SS3 and SS4 were represented by rock mass type SS2 with a few fracture zones (SS3) or with many fracture zones (SS4). Typical core runs from the three classes described, as well as the relevant joint statistics, are presented in Figure 8.

In the next step, the mean values and the variance of the geometrical characteristics of the joints, was used to produce the random joint patterns representing the architecture of each rock mass class. The commercial software UDEC (Itasca 2004) was used to generate these joint patterns in two dimensions, following a procedure similar to the one reported by Kim et al. (2007). The produced joint patterns were used to cross check the intuitive interpretation of rock mass structure previously performed. Moreover, after producing the discrete fracture network, the block size distribution as well as average block size was calculated on the basis of the equivalent block area and this value was checked for consistency with the assumed values for GSI and RMi.

The procedure of data collection, generation of discrete fracture network and re-evaluation of GSI through the average block size, is outlined graphically in Figure 9. Rock mass types SS1 and SS2 were represented by homogeneous joint patterns produced using the joint parameters derived from the joint statistics of the respective classes. Rock mass types SS3 and SS4 were developed considering the presence of one or more zones of intense fracturing (sheared zones), as described in the previous paragraph, consistently with the borehole findings. For both classes the background joint pattern was the same as for class SS2 and on certain zones with increased fracturing considered by assigning the jointing parameters of the sheared zones.

In order to check the sensitivity of the procedure, a number of random realizations for all the rock mass types was performed by varying the loosely controlled parameters (e.g. joint persistence) or other uncertainties. Figure 10 present six random realizations for rock mass type SS2. The average block size in all the cases was found to be comparable and the joint patterns were equivalent.

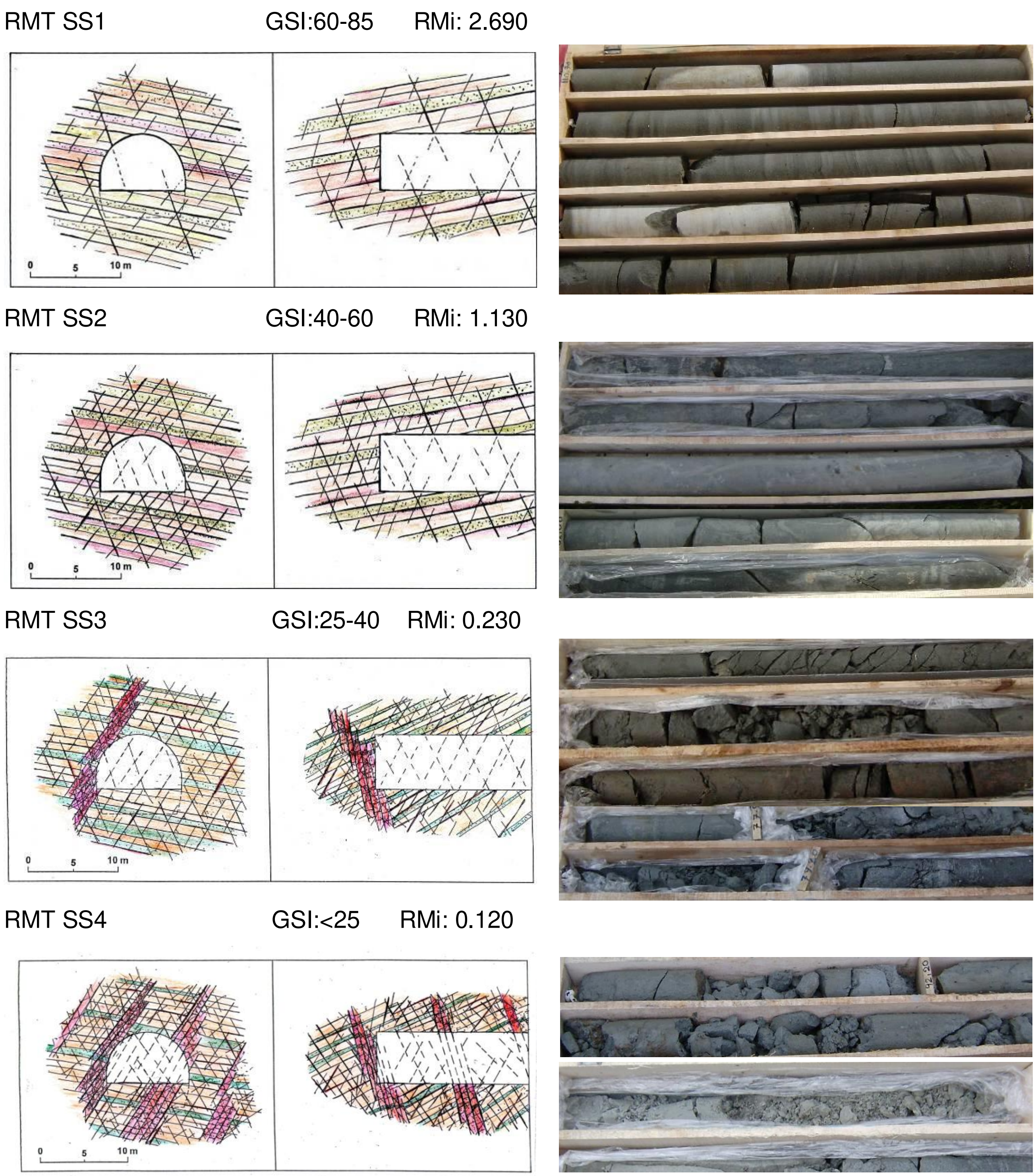


Figure 7. Typical core samples and engineering geology interpretation for the sandstone- siltstone units SS1 to SS4 (Note: RMT: Rock Mass Type).

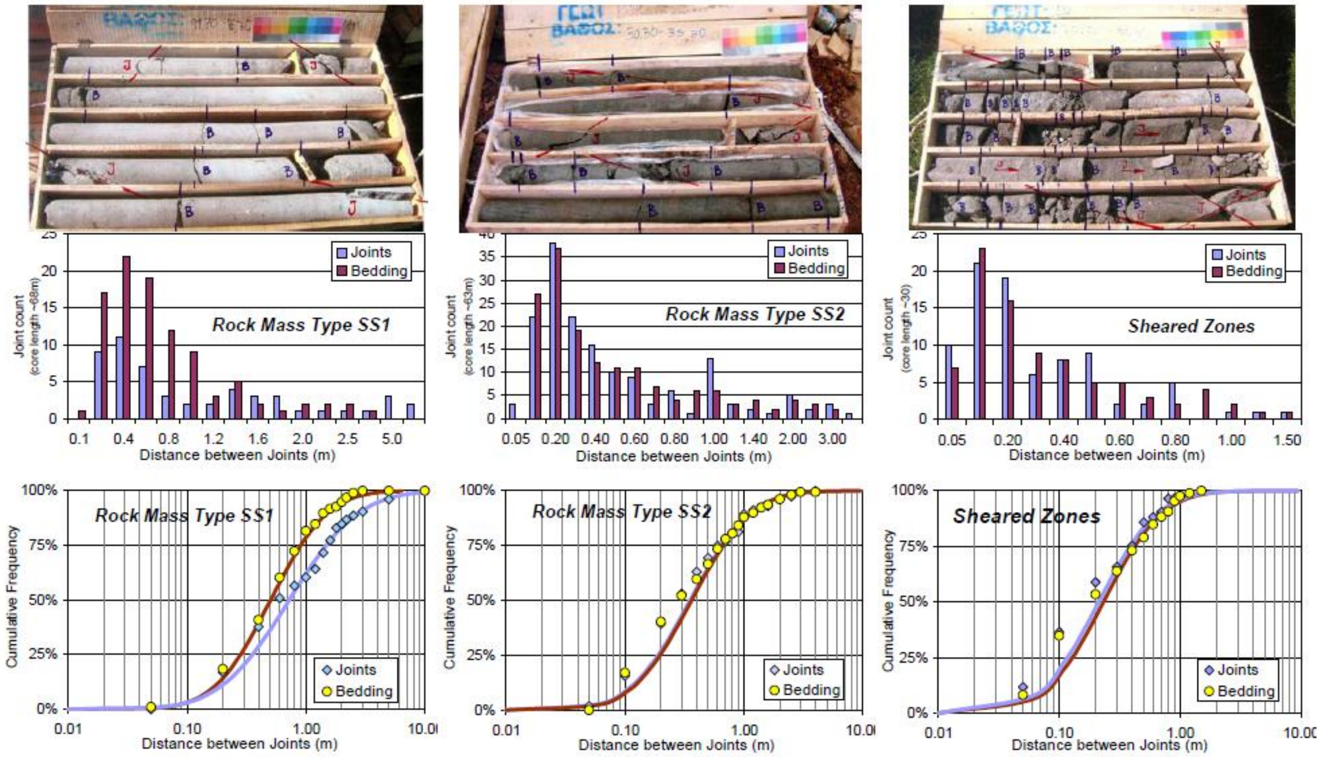


Figure 8. Joint statistics for Rock Mass Type SS1, SS2 and for sheared zones

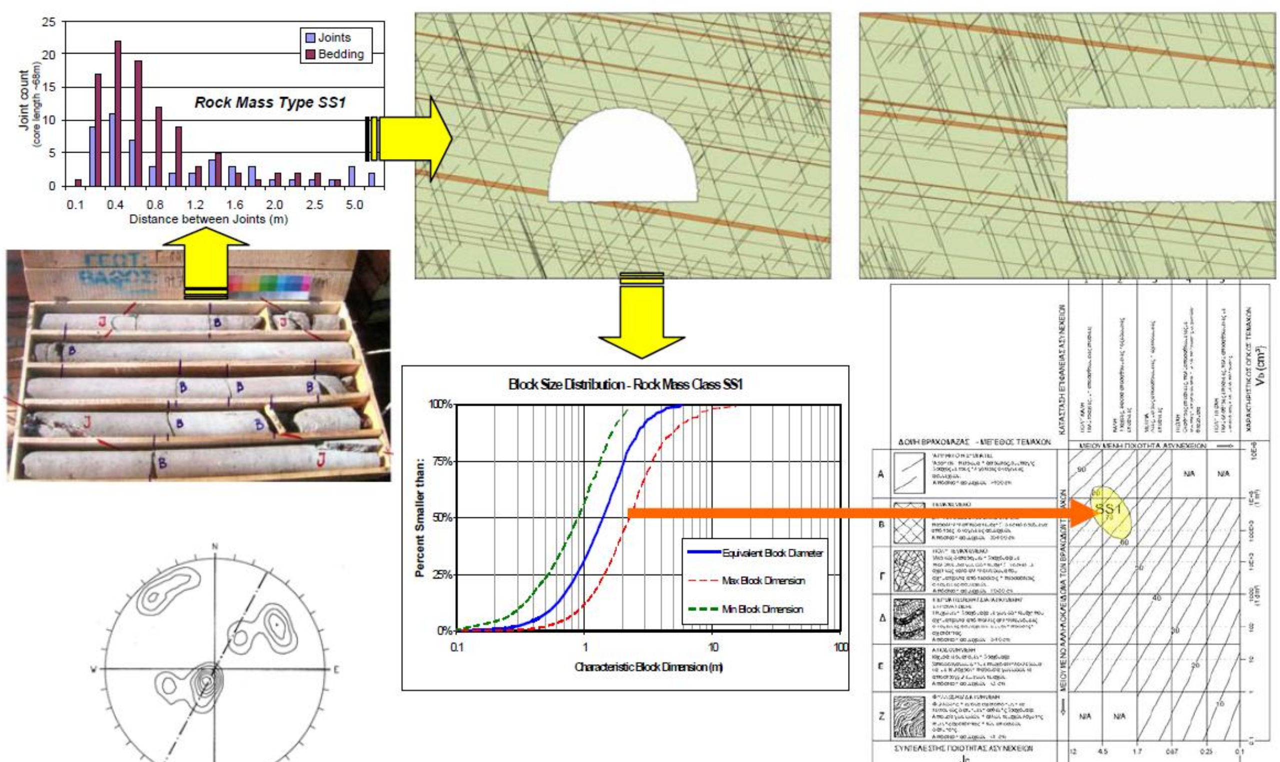


Figure 9 Joint count, discrete fracture network generation and assesment of average block size

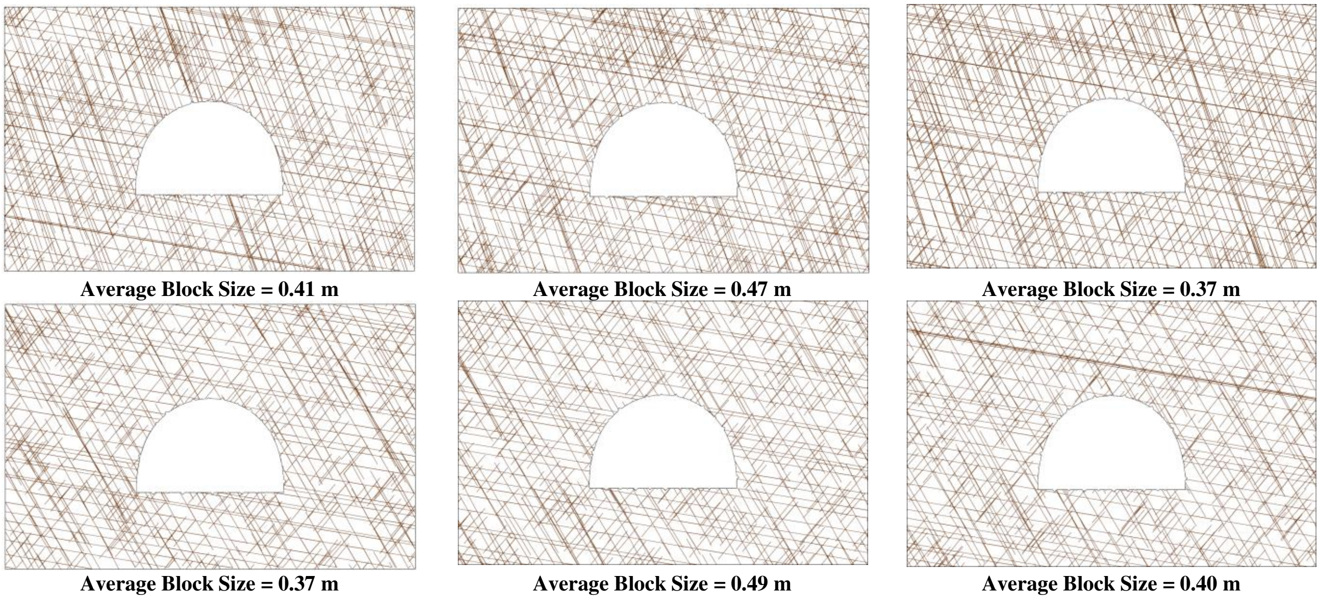


Figure 10. A series of realzations of random joint patterns for rock mass type SS2 (GSI = 40-60)

Rock Mass Strength and Deformability Parameters

Laboratory tests on sandstone cores, retrieved by the exploratory boreholes, revealed that the intact rock strength is fairly low, averaging σ_c =15 MPa, while the average tensile strength is σ_t =1.0 MPa. The frequency distribution of the test results in compression and indirect tension performed in sandstone samples is presented in Figure 11. A few slake durability tests performed in the same material yielded values in the range I_s=65-85, which classifies the rock as low durability on Gamble's classification scheme.

The modulus of elasticity of the intact rock samples was derived from the load deformation curves routinely measured during the unconfined compression tests, which displayed values that varied widely between E_i=1.5 GPa to E_i=5.5 GPa with an average value of E_i=3.3 GPa. The same dataset demonstrated that the MR value (=E_i/σ_{ci}) varied within the range MR=130÷330, with an average value of 225, which is in general agreement with published values for Greek molassic units presented by Marinos and Tsiambaos (2010).

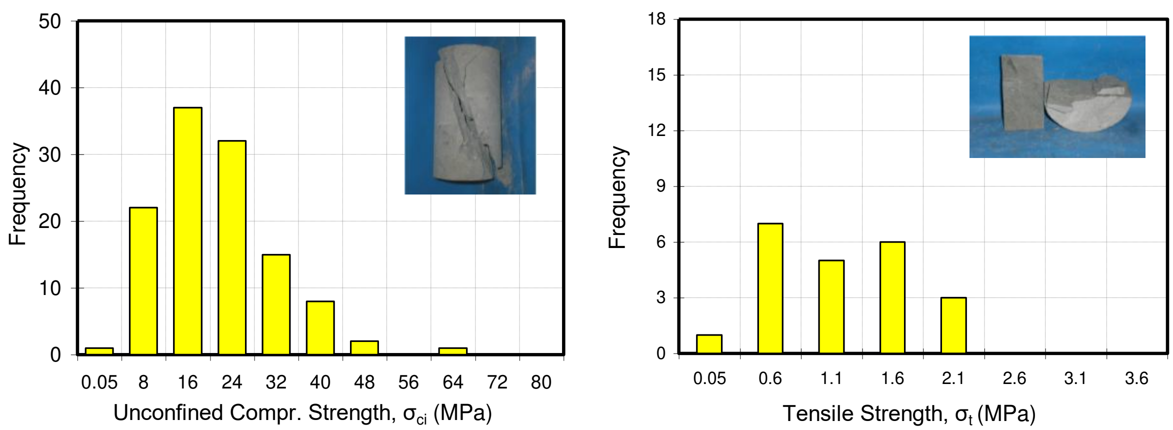


Figure 11.Statistical distribution of UCS and tensile strength values of sandstone cores

Rock mass strength parameters were estimated in terms of the Hoek-Brown failure criterion, by scaling down the intact rock strength parameters, accounting for rock mass structure (jointing) effects expressed by means of the GSI or the RMi indices, which reflect the interlocking of rock blocks and the condition of the surfaces between these blocks.

In that respect, GSI and RMi values for each rock mass class were used to derive corresponding rock mass strength parameters, following the Hoek et al. (2002) and the Palmström (1996) methodologies respectively. Table 1 and Figure 12, compare m and s parameters of the Hoek - Brown failure criterion, estimated for the sandstone unit rock mass classes on the basis of GSI and RMi, which demonstrate the consistency of the two methodologies as well as of the index assignments of each respective rock mass class.

Table 1. Shear strength parameters for the dominant rock mass classes along the tunnel alignment.

Rock Mass Class			Hoek - Brown				RMi			
	σ_{ci} (MPa)	m_i	GSI	σ_{cm} (MPa)	m_b	s	RMI	σ_{cm} (MPa)	m_b	s
SS1	15.0	17	60-85	2.820	5.82	3.6E-02	2.69	2.840	5.85	3.6E-02
SS2	15.0	17	40-60	0.903	2.85	3.9E-03	1.13	1.109	3.21	5.5E-03
SS3	15.0	17	25-40	0.258	1.39	4.0E-04	0.23	0.239	1.20	2.5E-04
SS4	15.0	17	<25	0.119	0.98	1.0E-04	0.12	0.141	0.86	8.9E-05

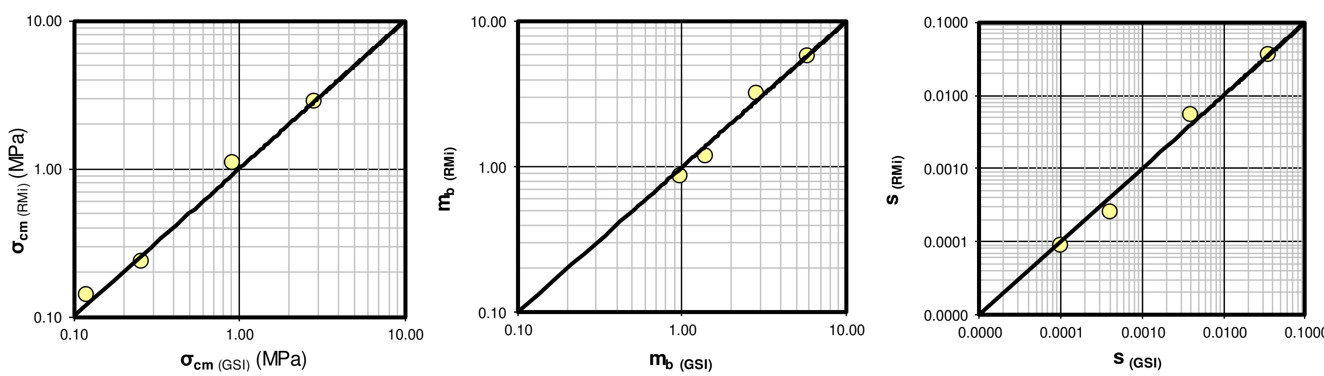


Figure 12. Comparison of rock mass H&B strength parameters (σ_{cm}, m_b, s respectively) determined on the basis of the GSI and RMi indexes for four Rock Mass Classes.

The deformation modulus was estimated on the basis of the GSI and the Hoek and Diederichs (2006) empirical relation. During the geotechnical investigation campaign, the deformation modulus of the rock mass was also directly measured by performing a number of in situ dilatometer tests. The Goodman jack tests were performed at the most competent sections of the exploratory boreholes, where borehole jack testing was feasible. Figure 9a depict a typical test with three loading – unloading cycles. The test results demonstrated that E_d values vary between 1.0 and 2.0 GPa. A comparison of the measured E_d values at each test location, with the respective empirical rock mass modulus E_m estimates, was performed to justify the validity of the empirical approach. At each Goodman jack test location, the GSI assessment was based on the retrieved rock cores and the modulus of elasticity E_i of the intact rock on the basis of the most relevant unconfined compression tests available for each respective test location. The outcome of the comparison outlined in Figure 13b was satisfactory and provided confidence on the use of the Hoek and Diederichs (2006) relation at this particular formation, at least for the GSI range tested, i.e., GSI >40.

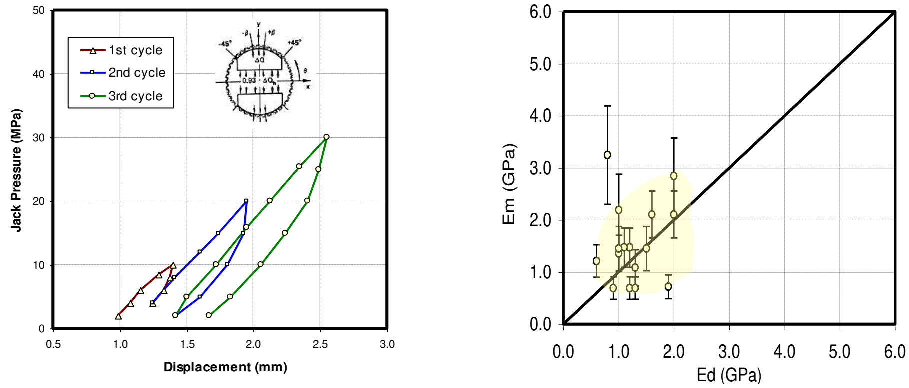


Figure 13. Typical Example of Dilatometer test results on the sandstone unit (left) and a comparison of the rock mass moduli (right) measured directly by dilatometer testing (E_d) and estimated indirectly through the Hoek and Diederichs (2006) empirical relation (E_m).

ROCK MASS BEHAVIOR AND DESIGN OF TEMPORARY SUPPORT SYSTEM

Successful tunnel design does not fail to identify all types of critical ground behavior and adjust the excavation and support process to account for them. Experience shows that deficiencies at this stage are the main causes for contractual disputes, leading to cost overruns and project delays. A robust design process involves the identification of rock mass behavior types likely to develop at certain tunneling sections by combining the previously defined rock mass types with other tunnel specific parameters, such as size of the tunnel opening, in situ stresses, groundwater regime, joint orientation etc.

A comparison of bulk rock mass strength to the developed stresses around the underground opening, provide input on modes of behavior governed by overstressing, while analysis of wedge stability around the opening provide some input on likely development of discontinuity controlled modes of failure. However, in practice, the design engineer faces frequently cases (especially in low strength rocks like the molasses sediments) where the dominant behavior type is not apparent, and where combined modes of response are more likely to develop. Rock mass architecture has in such cases a profound impact on the mode of response of the underground opening, which has to be evaluated in parallel with intact rock strength and joint properties. While the latter (rock and joint properties) are more straightforward to define through laboratory or in situ testing, the former (rock mass architecture) requires more data and both geological expertise and engineering judgment.

The Austrian guideline for the design of underground structures with conventional excavation (ÖGG 2001), which was largely used as a guide in this study, encourage the use of sketches of rock mass architecture (like the ones presented in Figure 7) to describe and communicate the characteristics of anticipated rock mass types and interpret the likely tunneling conditions. The tunnel engineer in this context, is provided with a clear picture of ground conditions and with a fairly solid basis to address risks. A more quantitative processes of depicting rock mass architecture based on joint statistics, like the one presented in the previous section, can assist to more confidently address discontinuity driven modes of failure and quantify their likely extend in a quantitative manner. Such exercises require more data, but allow the use of modern analysis tools and potentially will put a strain to gather more data and develop the methods of analysis further. The holistic description of specific rock mass types, in terms of rock and joint properties, rock mass architecture and bulk rock mass strength and deformability parameters, presented in the previous section, form the basis for a rational, justifiable and eventually successful identification of the dominant ground response.

The assessment of prevailing behavior types in this case accounted for specific characteristics of the molasse; the most notable being: a) the lithological variations (mainly sandstones-siltstones but in some tunnel stretches also conglomerates), b) the presence of joints and intense jointing zones and c) the medium to low intact strength of the constituent sediments (sandstone or siltstone) and the low cohesion of disintegrated conglomerates where present, d) the presence and inflow conditions of groundwater. As described before, assessment of rock mass behavior accounted in a well-balanced manner, ground response associated with gravity driven-structurally controlled modes of failure, and response associated with transgression of rock mass strength, likely to dominate the response in deeper sections (Stille & Palmström 2008). A wealth of observations of behavioral response and lessons learned during tunneling the Egnatia tunnels (N.Greece) in molassic formations (Marinos et al., 2013) and in Flysh, (Marinos 2014) was used as a solid and proven base to identify and predict the development of the dominant rock mass behavior types.

An initial systematic assessment of the dominant modes of failure was performed by means of the "Multiple Graph" approach, proposed by Russo (2008) and Russo (2014). This method considers in a very general manner the effect of rock mass fabric, strength, competence and self-supporting capacity on the basis of the GSI, RMR indexes, as well as the rock intact strength and tunnel overburden. As an example, Figure 14 plots the four prevailing sandstone rock mass types: SS1-SS4 on the multiple graph, which suggests that tunneling in the weaker rock mass class SS4 under high in situ stress in the higher overburden (>100 m) area may encounter squeezing conditions, while for the remaining classes, only gravity type failures may be anticipated, ranging from unstable wedges to retrogressive block failures and caving.

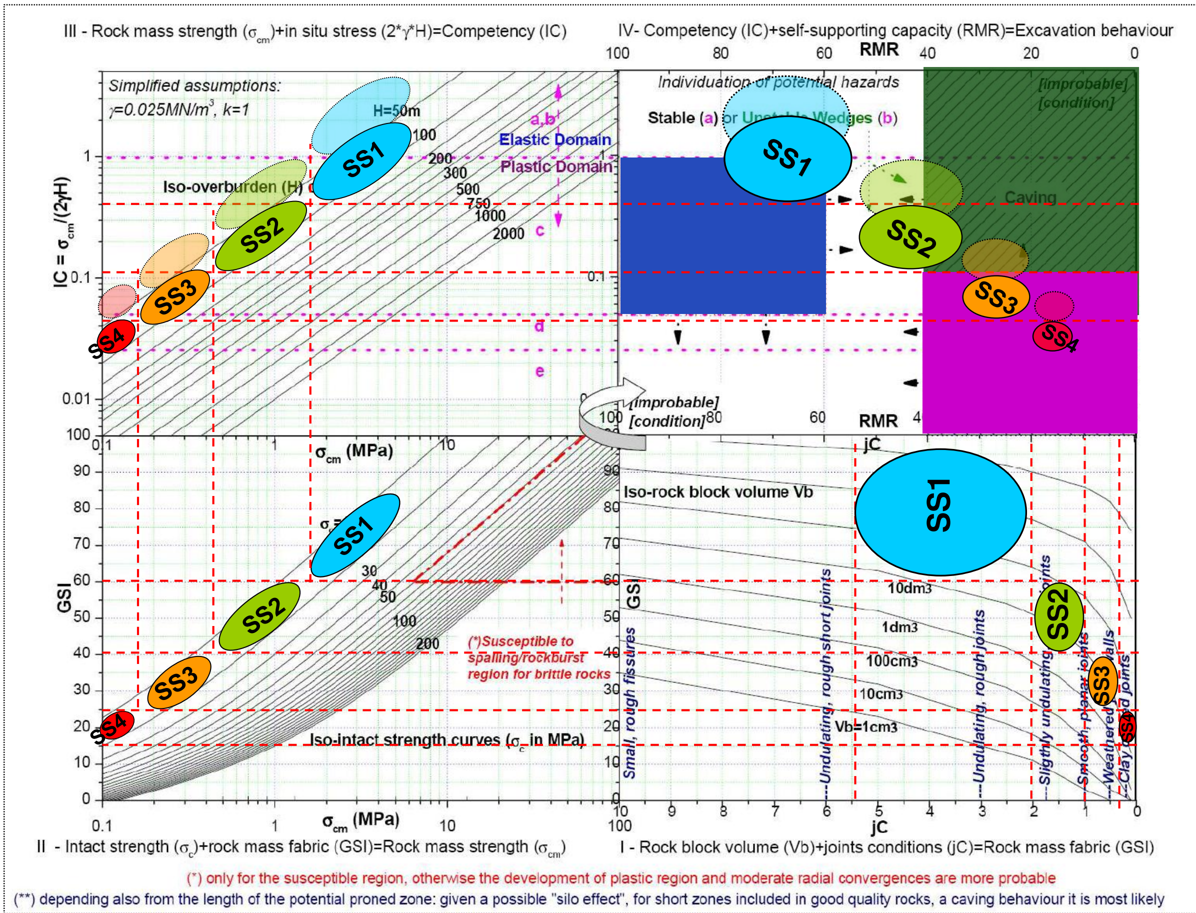


Figure 14. Preliminary assesment of Rockmass Behaviour Types for the sandstone units according to the "Multiple Graph" method proposed by Russo (2014). Dashed line elipses indicate Rock Mass Types in low overburden

A more detailed analysis of squeezing potential was performed by calculating the closure of the unsupported tunnel by employing a close form solution. In this case the analytical solution for a circular tunnel in hydrostatic stress field, in an elastoplastic medium, obeying the Hoek-Brown failure criterion, presented by Carranza-Torres and Fairhurst (2000) was selected. The strength and deformability parameters of the equivalent continuous medium (presented in table 1) were used to derive expected convergence values of the unsupported opening. Table 2 presents the calculated maximum normalized tunnel closures for all the sandstone rock mass classes, for various levels of overburden. Only the weaker rock mass classes, encountered in the deeper tunneling sections, is expected to encounter some minor overstressing and squeezing problems (ε>1%). At a later stage, more elaborate equivalent continuum analyses were undertaken, using the commercial finite difference code FLAC (Itasca 2008), to explicitly model the geometry of the tunnel section, and the gravitational in situ stress field, and verified the findings of the closed form solutions.

More effort is required to assess structurally controlled modes of failure. The presence of specific structural features like stratification and lithological variation, the presence of persistent joints, faults or folds will definitely control the detachment of single blocks, or the fall of a series of blocks in a retrogressive manner. The rock mass structure sketches as well as the discrete fracture networks, which were developed in the previous rock mass characterization step, were proven a valuable tool for this purpose. The envisaged modes of gravity driven failures, for the four rock mass sandstone classes using both approaches are presented as an example in Figure 15. The extent of the unstable rock blocks and the size of the potentially unstable wedges, is used as a guide to dimension rock anchor length and spacing. In subsequent design stages, rock wedge stability analysis, or analysis of the discontinuum (e.g. with UDEC) is also required to justify the design decisions.

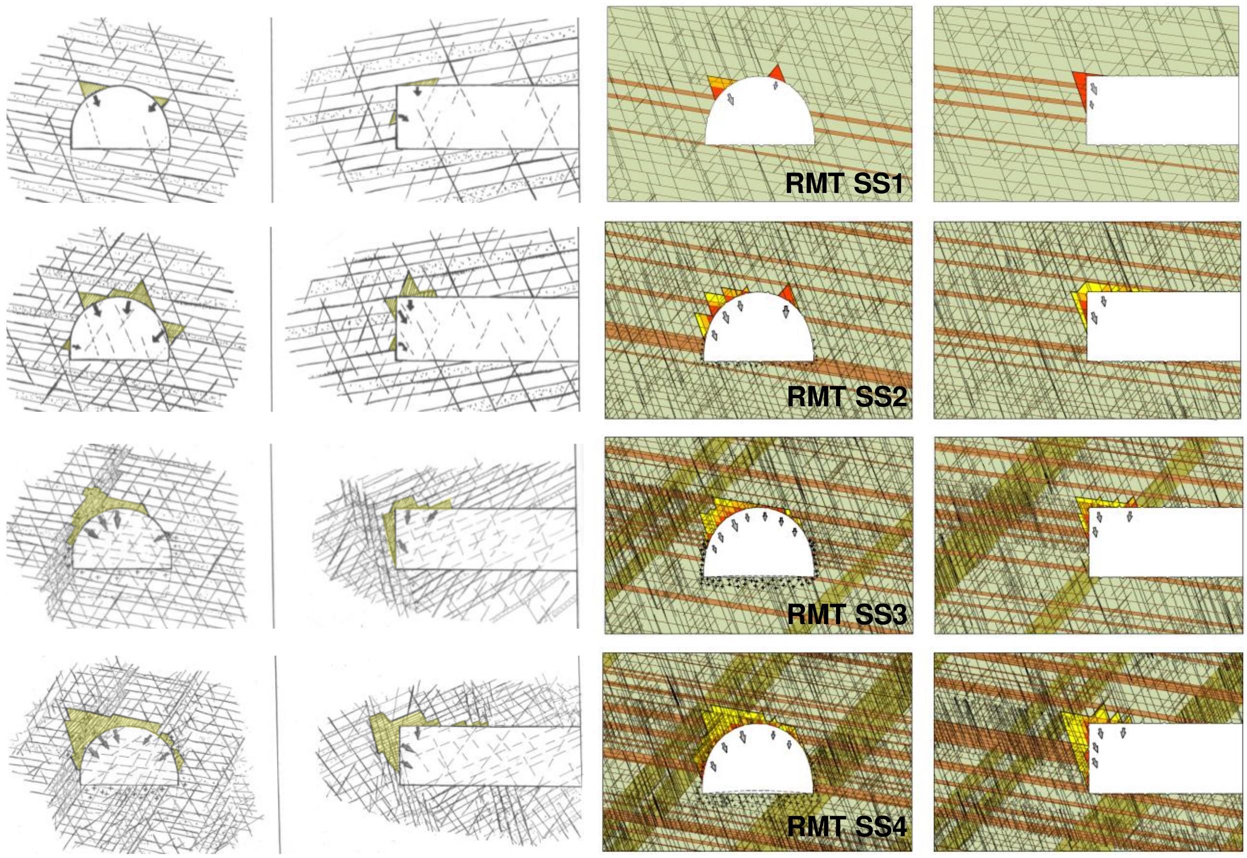


Figure 15. Rock Mass Behavior Types controlled by structural features

In general, past experience from tunneling in similar formations is indispensable to support the design decisions and to verify the results of any individual analysis. In this particular case significant experience has been available from the construction of 12 road tunnels of the "Egnatia Odos" motorway, to the North of the studied area, that were excavated in similar formations (molasse), as reported by Dounias et al. (2010) and Marinos et al. (2013). Experience from the construction of these tunnels shows that the most notable problems are associated with gravity driven types of failure, wedge falls and block detachment falls, taking place either at tunnel face (Figure 16) or at tunnel crown. In the latter case, block detachment takes place mainly along the bedding during excavation and it only appears as overexcavation of the tunnel section, which should be filled with shotcrete or concrete during casting of the final lining (Figure 16).

Figure 16. Response of molassic rocks during tunnelling of the "Egnatia odos" tunnels. Left: Structurally controlled overexcavation - Ag.Paraskevi tunnel (after Dounias et al 2010). Right: structurally controlled instability at tunnel face in thick sandstone layers separated by sparse thin siltstone intercalations (after Marinos et al. 2013)

Despite the fact that most of the tunnel length is driven deep below the water table, the low permeability of the molasse (measured by numerous in situ borehole permeability tests) ensure limited groundwater inflows. Flush inflows have been estimated to be of the order of 2÷10 m^3/h/m, for a rock mass permeability within the range 5×10^{-7} m/sec to 5×10^{-8} m/sec. The presence of fracture zones (rock mass class types SS3 and SS4) is expected to convey larger quantities of groundwater within the tunnel excavation, but since their lateral extent is not expected to be substantial, their inflow potential is considered limited. Experience from the "Egnatia" tunnels, mentioned before, verify these assumptions, although the overburden and the hydraulic head in those cases was significantly smaller. High pore-water pressures on the other hand is necessary to be quickly dissipated after tunnel face arrival, by means of drilling relief holes to control rock overstressing. Since the molasses are dominated lithologically by sandstones, joint quality is not expected to deteriorate substantially due to groundwater flows, as would be the case with poorly lithified siltstones and claystones.

Table 2 presents a synthesis of anticipated rock mass behavior types, for each rock mass type at various overburden depths, as derived by the examination of the rock mass architecture as well as by simplified convergence calculations.

Table 2. Rock mass behavior for the dominant rock mass classes

Rock Mass Types/Ground Types (GT)	Response Factors (Circular Tunnel D=9m)			Response of Unsupported Tunnel							BT*
				Dominant Mode (**DM**) Probable/Contributing mode (**CM**) Less Likely/Less Contributing mode (**TM**) Unlikely mode (**UM**)				Quantitative Response Indices.			
	Overburden (m)	Groundwater	Lithology/Structure	Generally stable localized block falls in unsupported span Limited shear distress around the excavation (BT 2)	Raveling (BT 8)	Shallow Shear Failures (BT 3)	Deep sated shear failures (BT 4)	Thickness Plastic zone (m)	Tunnel radial strain ε%	Max Convergence (mm)	Dominant Rock Mass Behavior type (RMBT)
SS1	50	√	Three joint sets with an orientation not detrimental to face and roof stability.	DM	UM	UM	UM	0.1	0.1	2.9	2/1
	80	√		DM	UM	UM	UM	0.3	0.1	4.8	2/1
	120	√		DM	UM	TM	UM	0.50	0.2	7.8	2/1
SS2	50	√		DM	UM	UM 4	UM	0.50	0.2	10.6	2/2
	80	√		DM	UM	CM	UM	0.9	0.4	19.1	2/2
	120	√		CM	UM	DM	UM	1.3	0.7	33.3	3/1
SS3	50	√	Three joint sets as well as well as presence fracture zones with a sub-vertical orientation.	DM	TM	CM	UM	1.0	0.8	33.5	2/2
	80	√		CM	TM	DM	UM	1.6	1.4	64.4	3/1
	120	√		CM	TM	DM	TM	2.4	2.7	120.7	3/2
SS4	50	√		DM	CM	DM	UM	1.3	1.1	50.5	2/2-3/1
	80	√		CM	DM	DM	TM	2.0	2.2	99.4	8/1-3/2
	120	√		TM	DM	CM	DM	3.0	4.2	190.8	8/1-4/1

BT 2: *Potential of discontinuity controlled block/wedge falls, local **(2/1)** or retrogressive **(2/2)**. Occasional local shear failure on discontinuities.*
BT 3: *Shallow, local **(3/1)**, or more extensive **(3/2)**, stress induced failures, in combination with discontinuity and gravity controlled failures.*
BT 4: *Deep-seated, stress induced failures, involving large ground volumes and medium **(4/1)**, to large **(4/2)**, tunnel strain and deformation.*
***BT 8**: Raveling of dry or moist, intensely fractured, poorly interlocked rocks or soil with low cohesion **(8/1)**.*

TUNNELING METHOD AND TUNNEL SUPPORT TYPES

The design of the sequencing of the tunneling works and the required support measures is designed to control the response of the rock-support system as a whole and meet the following requirements:

- General safety of tunneling works.
- Limiting tunnel strain / convergence (<5 cm).
- Safety against general stability at tunnel face area.
- Safety against rock block/wedge falls at tunnel face area.
- Structural safety of shotcrete lining.
- Control of rock bolt strain within permissible limits.
- Control groundwater inflows during tunneling.
- Meet the requirements of construction (equipment, materials, constructability, productivity, schedule et.c.).

Based on these requirements, in combination with the prescribed patterns of tunnel behavior (BT) established in the previous step, four (4) main support schemes (ST) were developed. (ST II, III, IV and V) to cover all anticipated tunneling conditions. These support types were further divided into three subcategories (a, b and c) to cater for low, medium and high overburden, that largely control stress-induced modes of failure and affect shotcrete shell compressive stresses. The tunnel support measures consist of shotcrete applied in layers and reinforced with fibers, in lighter support classes and with lattice girders and wire mesh in heavier ones, to meet the structural requirements. Experience with tunneling in weak molassic sandstones or siltstones, has shown that it is essential to apply immediately after excavation of each round a thin sealing layer of shotcrete, to prevent deterioration of the rock and to provide a safer environment for further support works. For rock bolting, swellex type bolts were selected for the top heading, where speed of construction is critical, while grouted rebar anchors were used for the bench support. Drainage holes are drilled subsequently with the rock bolting operations to provide a quick relief of groundwater pressures. Heavier support classes include also pre-support by means of a spilling umbrella to stabilize retrogressive wedge failures, or cope with raveling conditions at the face area. Where more severe face and unsupported span instabilities are anticipated, a forepolling umbrella is required and since this occurs in mild to moderate squeezing conditions a temporary invert is also foreseen for those cases.

The initial assessment of the thickness of the shotcrete shell and the spacing of the rock anchors exploited the empirical design diagrams of RMR, Q or the RMi system. As an example, Figure 17 presents the support requirements for the sandstone rock mass types (SS1-SS4), according to the empirical design chart of Palmström (2000). Similar input is provided by charts based on Q or RMR values summarized by Bieniawski (1989). Round length is also selected, considering face and unsupported span stability, in conjunction with constructability requirements. The 12 support classes were adjusted to these initial support requirements and the support types (ST) established in Figure 18 were developed.

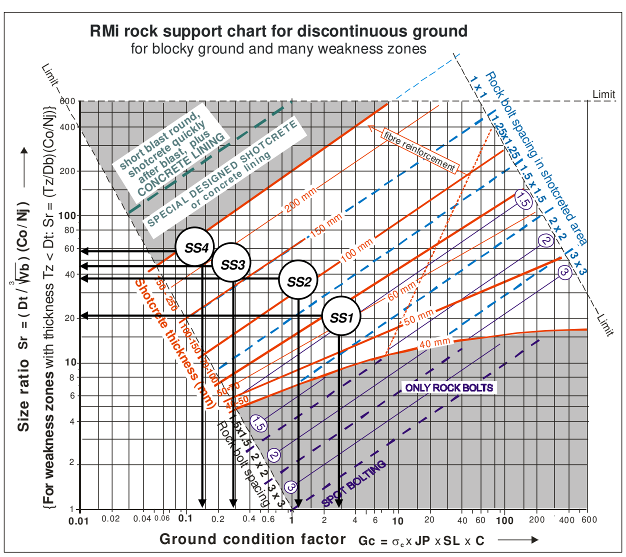
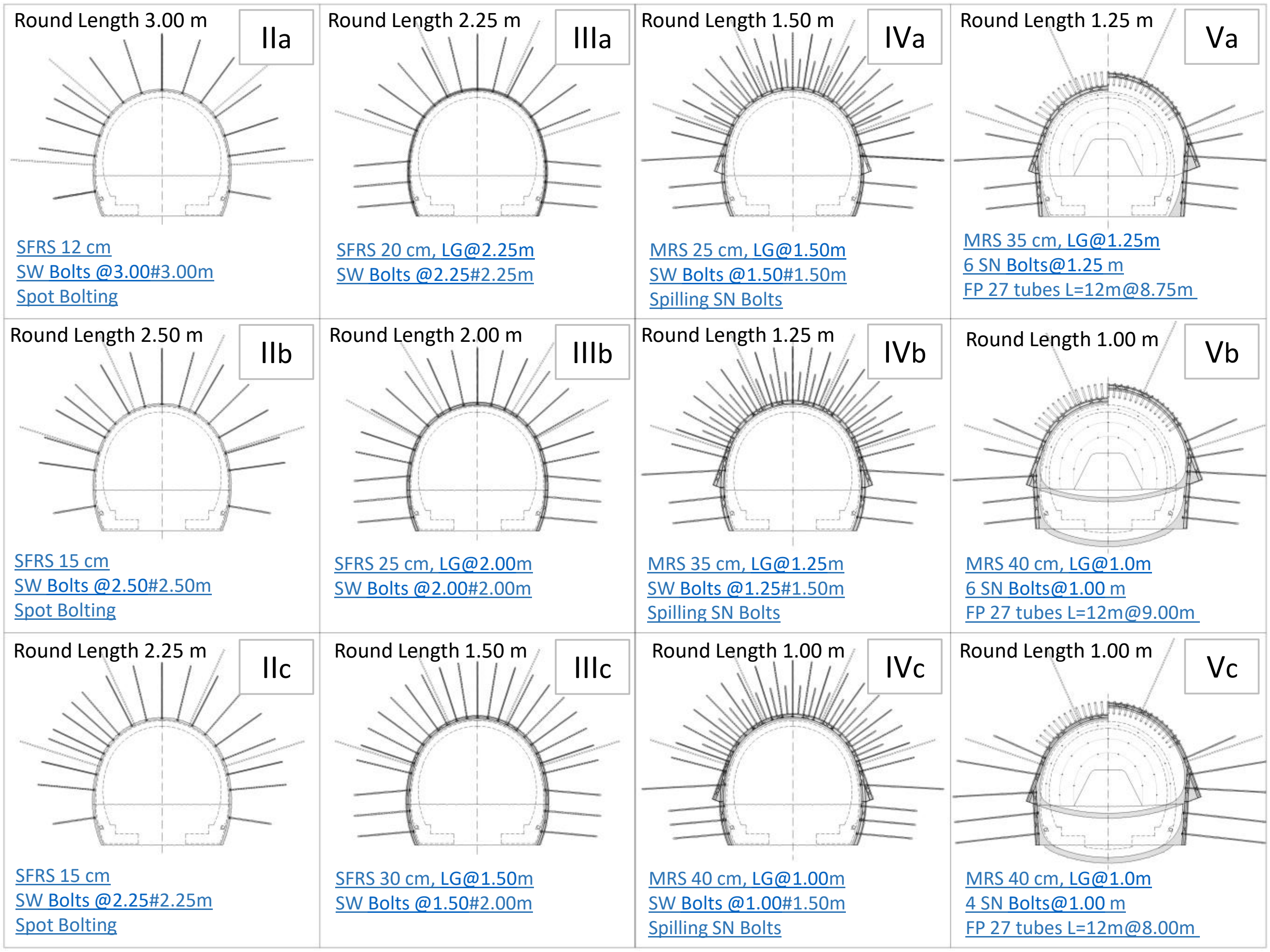


Figure17. Rock support requirements for rock mass types SS1 to SS4 (design chart after Palmström 2000)

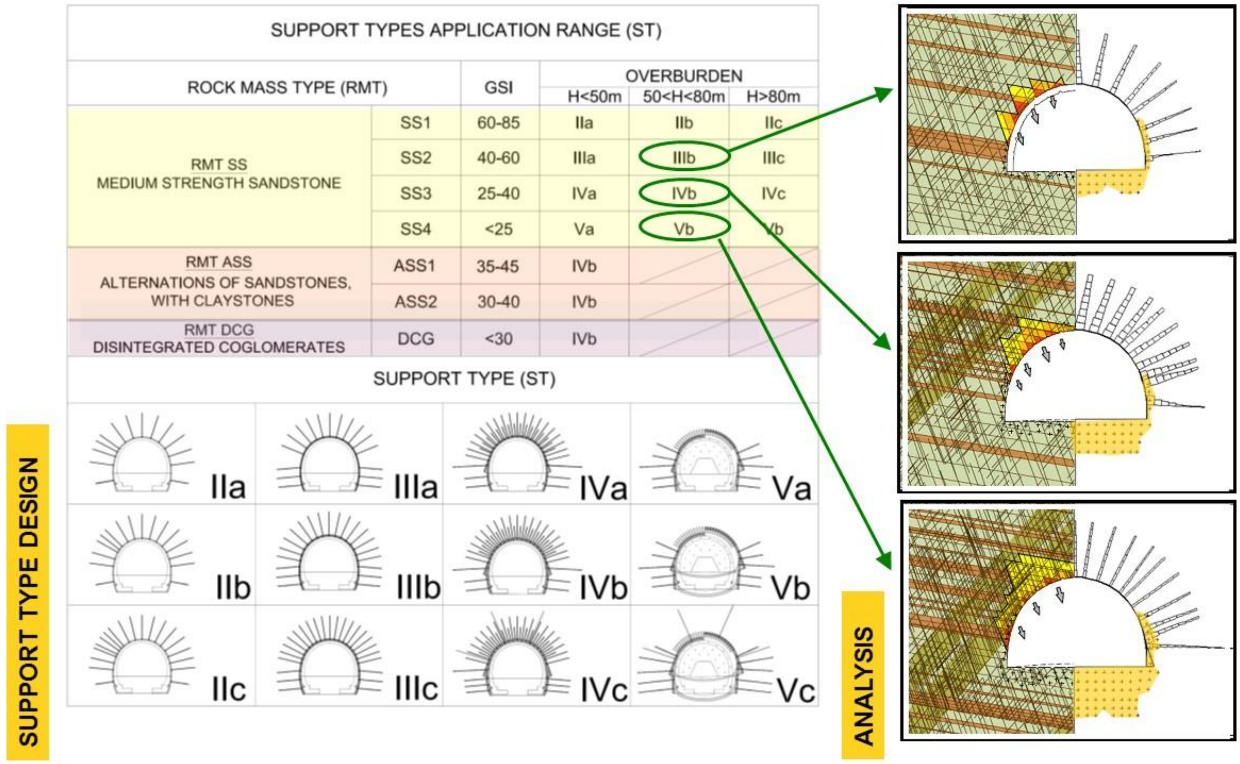


Notation:

SFRS: Steel Fibre Reinforced Shotcrete MRS: Mesh Reinforced Shotcrete
SN Bolts: Fully Grouted Rebar Bolts SW Bolts : Swellex type Bolts
LG: Lattice Girders FP: Forepolling

Figure 18. Support types for rock and stiff soil. Round lengths and support requirement noted on the figure correspond to top heading drive.

The final correspondence of rock mass types and rock support types was performed on the basis of actual distress of support measures (stresses in shotcrete and bolts) by modelling explicitly ground-tunnel-support interaction, as well as by considering the likely development of structurally controlled modes of failure. Figure 19 presents in a concise form, the assigned application field of each support class. The performance of each support schedule was tested for the respective rock mass type and applicable overburden, against stress-induced modes of failure, by means of finite difference analysis undertaken by FLAC, and against wedge stability and gravitational type failures and a comparison of these modes of response is also outlined in Figure 19. It is pointed out that although analyses based on an equivalent continuum may indicate that a safe tunneling and support sequence is achieved, consideration of wedge detachment and block sliding may require additional support (anchors) and in some cases pre-support (spiles, forepoles). Sample results of the finite difference analyses are presented in Figure 20, demonstrating the response of the ground-support system and the associated displacement field. The developed support classes and their respective field of application, form a baseline tunneling and support schedule, appropriate for tendering purposes. Observed response during actual tunneling will provide the basis to optimize this process to the benefit of safety and economy of the project.



SUPPORT TYPES APPLICATION RANGE (ST)					
ROCK MASS TYPE (RMT)		GSI	OVERBURDEN		
			H<50m	50<H<80m	H>80m
RMT SS MEDIUM STRENGTH SANDSTONE	SS1	60-85	IIa	IIb	IIc
	SS2	40-60	IIIa	IIIb	IIIc
	SS3	25-40	IVa	IVb	IVc
	SS4	<25	Va	Vb	Vb
RMT ASS ALTERNATIONS OF SANDSTONES, WITH CLAYSTONES	ASS1	35-45	IVb		
	ASS2	30-40	IVb		
RMT DCG DISINTEGRATED COGLOMERATES	DCG	<30	IVb		

Figure 19. Left: Table of Support classes and field of application. Right: Performance of primary support measures on controlling rockmass deformation and stabilizing rock blocks.

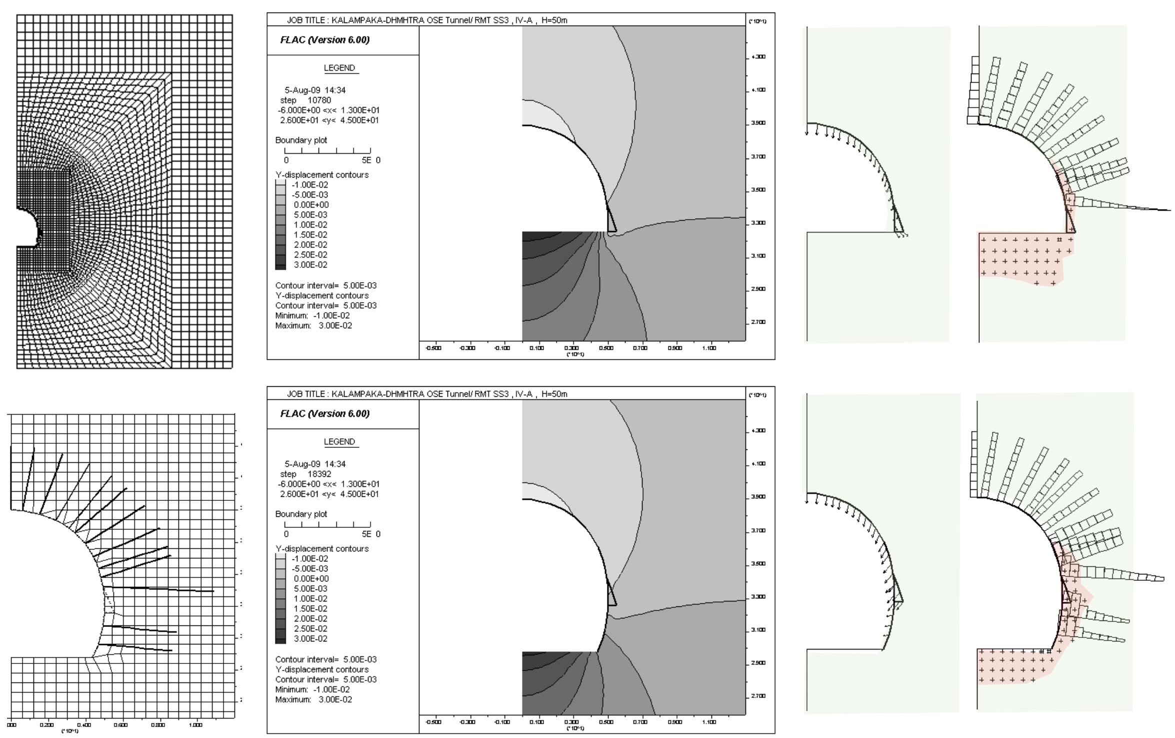


Figure 20. Sample numerical analysis results; Left:mesh, Center: Vetical displacements, Right: Vectors of lining convergence, rock bolt axial forces and plastic zone.

The definition of the field of application of each support class allowed the prediction of the likely application length of each rock support class which is essential for project scheduling and budgeting. This is undertaken by the compilation of the synthetic tunnel longitudinal profile, presented in simplified form on Figure 21. In this figure, it is demonstrated how ground conditions, overburden and groundwater, affect the development of possible rock mass behavior types which in turn lead to the definition of the support requirements.

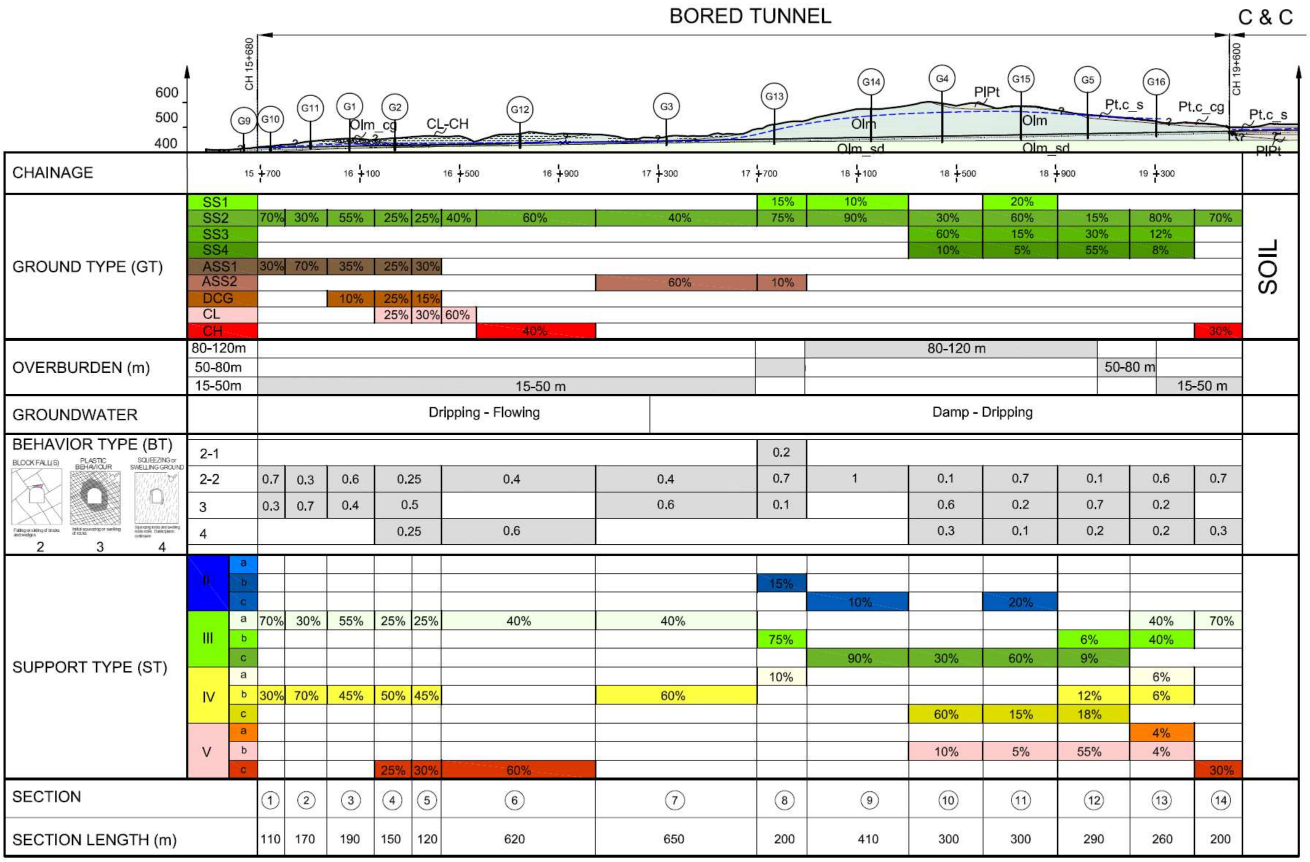


Figure 21. Distribution of Rock Mass Types, Rock Mass Behavior types and Support Types along the tunnel alignment

SUMMARY AND CONCLUSIONS

This paper outlines the procedures for rock mass characterization, ground behavior assessment and support design, followed in the design of the 5000 m long Trikokkia tunnel, planned to be constructed in central Greece through the molassic formations of the southern edge of the meso-Hellenic trench.

The dominant geologic units are medium strength massive sandstones with occasional siltstones interbeds, moderately and locally intensively jointed. The design approach that was followed focused on the consideration of rock overstressing modes of failure, on gravitational modes of failure driven by discontinuities and other structural features as well as on mixed modes of failure. In that respect, ground characterization was performed by implementing empirical rock mass characterization indices (GSI and RMi) that complemented the explicit description of rock mass structure by hand sketching of each rock mass class and by the generation of discrete fracture networks, compatible with joint statistics. The joint patterns produced by the latter method were compared with the intuitional drafting of the rock mass and consequently they were both used to assess more confidently the potential for structurally controlled modes of failure. The identification of different rock mass behavior types and the subsequent design of appropriate support, was based on metrics that compare rock mass strength and deformability with in situ stress state, as well as on the rock mass structure characteristics, associated with structurally controlled modes of failure and assisted by wedge stability calculations. The adopted process to assess rock mass behavior contributes significantly to design with confidence the tunneling and support process for tendering purposes.

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