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Limitations in Tunnel Portal Design: An Evaluation Using Numerical Models and Line Surveys

Tünel Portal Tasarımında Kısıtlar: Sayısal Model ve Hat Etütleri Kullanarak Bir Değerlendirme

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ABSTRACT

In this study, the engineering geology and the geotechnical design studies of the Aslankayası Tunnel Project are explained. Owing to the low overburden thickness, the tunnel in question, which is located near a slope face, will be exposed to asymmetrical loading after commencement of excavation. The asymmetrical loadings will especially affect the right tube, in the direction of increasing kilometer markings. Furthermore, the thickness between the right tube's right wall and the slope face in this section has decreased down to 6 m. Moreover, as the tunnel is passing under a 1st degree protected archeological area. Some of the site investigation studies, such as geotechnical drilling and site laboratory works, could not be performed. The excavation support system of the tunnel was determined using empirical studies and numerical models with the help of line surveys, local sampling, and internationally accepted rock mass classification studies (RMR, Q, GSI). These studies were performed on rock mass outcrops. Rock mass engineering properties were determined through the utilization of empirical equations that incorporate data derived from site investigation studies and laboratory test results as input. By using geotechnical properties obtained from line surveys and engineering geology studies, a numerical model was generated. The numerical model results corroborated the asymmetrical loading predictions obtained from line surveys and engineering geology studies. The main aim of this study is to emphasize the importance of interpretation of the geological units and their post excavation behaviors on the excavation stability.

Keywords: Line surveys, Numerical modeling, Rock mass classifications, Slope tunnel design

ÖZ

Bu çalışmada Aslankayası tünel projesine ait mühendislik jeolojisi ve jeoteknik tasarım çalışmalarını anlatmaktadır. Düşük örtü kalınlığı nedeni ile şev yüzeyine yakın olan yamaç tüneli, özellikle kazı işlemine başladıktan sonra asimetrik yüklemeye maruz kalacaktır. Asimetrik yükleme, proje artış kilometresi yönünde özellikle sağ tüpte etkili olacaktır. Ayrıca, bu bölgedeki sağ tüp sağ duvarı ile yamaç yüzeyi arasındaki et kalınlığı 6 m seviyesine kadar düşmektedir. Ayrıca, tünelin 1. derece arkeolojik koruma alanı altından geçiyor olması nedeniyle, tünel ekseni üzerinde yapılması gerekli olan bazı saha araştırma çalışmaları, örneğin jeoteknik sondaj çalışmaları ve ilgili saha deneyleri yapılamamıştır. Tünelin kazı destek sistemi; hat etütleri, yerinden örnek alma ve uluslararası kabul görmüş

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kaya sınıflama sistemleri kullanılarak (RMR, Q, GSI) ampirik çalışmalar ve nümerik modellemeler aracılığı ile belirlenmiştir. Bu çalışmalar için sahada yüzlek veren kaya kütlesi kesimleri kullanılmıştır. Saha çalışmalarından ve laboratuvar testlerinden elde edilen sonuçlar girdi olarak kullanılarak ampirik eşitlikler yardımı ile kaya kütlesi mühendislik parametreleri hesaplanmıştır. Hat etütleri ve mühendislik jeolojisi çalışmalarından elde edilen veriler ile nümerik modeller oluşturulmuştur. Nümerik modellerden elde edilen sonuçlar, mühendislik jeolojisi aşamasında asimetrik yükleme koşullar için yapılan tahminleri doğrulamıştır. Bu çalışmanın esas amacı da jeolojik birimlerin ve onların kazı sonrası davranışlarının doğru yorumlanmasının tünel kazı stabilitesi üzerindeki önemini göstermektir.

Anahtar Kelimeler: *Hat etütleri, Sayısal modelleme, Kaya kütle sınıflamaları, Yamaç tünel dizaynı*

Introduction

Due to high traffic load and the inadequacy of the existing roads, it was decided to construct a double-tube highway tunnel at the Zonguldak-Kilimli road city crossing. Not only would this make travel more comfortable, but total travel time would be reduced and gasoline consumption of the vehicles would decreased. The tunnel is located on the west Black Sea coastline in the Northern part of Türkiye (Figure 1). The tunnel dimensions are 340 m in length, 10 m in width and 8 m in height.

The tunnel is located on an incline and, as the tunnel progresses, the wall thickness on the right side of the right tube is not thick enough. In other words, there is not enough overburden thickness for construction stability. In other words, there will be asymmetrical loads when the construction starts that will threaten the tunnel's stability. During the mapping stage, it was noticed that the wall thickness decreased to 6 m between the right wall of the tube and the outer face of the slope. In this case, the arch effect will not occur and this will threaten both short-term and long-term tunnel stability. Since the tunnel road is located in a 1st degree protected archaeological site, the necessary site investigation studies could not be carried out. Site investigation studies have been done using line surveys and in-situ sampling on the right side of the route where there are outcrops. Internationally accepted rock mass classification methodologies have been used. Using the results of all these studies and laboratory test results, an attempt has been made to estimate the rock mass strength parameters. Numerical models were created for the right tube entrance portal which may be affected by asymmetrical loads. There are several studies in literature about solutions for similar problems. (Kun and Onargan 2013, Xiao et. al. 2014, Das et al. 2017, Zhang et. al. 2017, Hu et al. 2021, Zhou et. al. 2022, Guo et al. 2023). Kun and Onargan (2013) studied the Metro tunnel in İzmir, which is located in a faulty area with low overburden thickness. Geological and geotechnical conditions were modeled with finite element software. Tunnel stability was ensured by using rock bolts, steel wire mesh and shotcrete, as well as an umbrella arch with lattice girders. Xiao et. al. (2014) studied the cracking mechanism of shallow and asymmetrically-loaded tunnels in loose deposits. Similar to this study, the left tube of the tunnel portal is under asymmetrical loading conditions and low overburden thickness. After the application of the final lining, cracks were observed on the concrete lining surface which is the result of surface settlement. The problem was modeled using a numerical method. After the completion of the excavation and lining, asymmetrical loads which threaten tunnel stability were prevented using a retaining wall.

Figure 1: Location of the tunnel
Şekil 1: Tünelin Türkiye haritasındaki lokasyonu

Surface subsidence in asymmetrically parallel highway tunnels located in the Himalayan terrain was studied by Das et. al. (2017). In this study the tunnel tubes are asymmetric both in terms of diameter (12 m and 8.5 m) and overburden depths (26 m and 36 m). This study shows the difference in surface settlement and deformations for the larger in diameter and deeper tunnel tube even though the same supporting pattern is applied for both tubes. A cracking mechanism of an asymmetrically-loaded entrance portal to a highway tunnel was studied by Hu et al. (2021). In this case study, cracks developed on the lining at the entrance section after the excavation was completed. A three-dimensional finite element model was used to understand the failure mechanism. Ground reinforcements and reverse loading were suggested to prevent such failures in future studies.

Geology of the Tunnel Axis

Early Cretaceous age Kilimli Formations are outcropped on the route of Aslankayası Tunnel and its close surroundings. The Kilimli Formation which is located with conformity on top of the Inalti Formation, is formed from sandstone, siltstone, claystone, clayed limestone and marns. Grey, dark grey, and yellowish beige are the distinctive colors of the formation. The bedding thickness ranges from thin to thick. The Kilimli formation was surveyed by dividing it into three sub-members; yellow-colored quartz sandstones were named Velibey, glauconitic sandstones and clayed limestones were named Sapca, and the marned levels were named Tasmaca. Among these groups, Sapca is outcropped on the Aslankayası tunnel route. Sandstone, claystone, and siltstone intercalation is observed in the Sapca. There are partly sandy and clayey limestone levels observed in the Sapca. The sandstone grains are composed of quartz, glauconite, metamorphic rock segments and magmatic rock grains. A photo taken from the entrance portal of the tunnel is provided below in Figure 2.

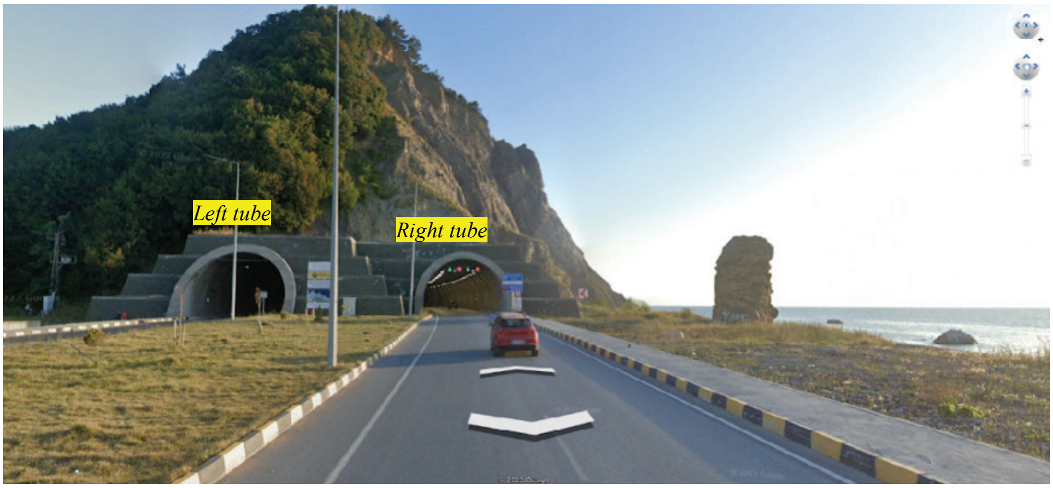

Figure 2: Photo of the tunnel entrance portal

Şekil 2: Tünel giriş portalinin fotoğrafı

Engineering geology and geotechnical parameters

Sandstones of Kilimli formation are mostly grey colored, fair to thick and linearly bedded, the discontinuity spacings are 15 – 50 cm, the apertures are less than 5 mm, cracks are mostly clear, partly calcite plastered, rough to slightly rough, moderately to highly weathered, observationally moderately strong medium strength and moderately hard (Bieniawski 1989). Beddings are the main discontinuity sets. Other joint sets are also observed that cut this bedding vertically and diagonally. Free sandstone blocks can be seen in both tunnel portal sections. Due to the tunnel route being located in a 1st degree archaeological protection site, geotechnical drillings were not carried out. However, as the rock outcrops are clearly observed on the right slope side of the tunnel route (Figure 3), some of the necessary geotechnical measurements and line surveys were taken from these sections. The geological plan and engineering geology map are provided in Figure 4.

In order to obtain the strength parameters of the geological units of the tunnel route, block samples were taken from both tunnel portal sections and the necessary laboratory tests were carried out. The laboratory test results are provided in Table 1.

Structural geology

The rocks located on the tunnel route are highly jointed and fractured. Joints, folds and faults are formed due to north-south directional compressional forces and laned through the northeast-southwest direction. Dominated joint sets are determined in the northeast-southwest direction. The less distinct joint sets are located in a roughly perpendicular direction to the main joint sets. Discontinuity fillings are hard and intact. In the massive rocks joint set spacings are fairly large. However, discontinuities in the claystone and shale formations are in the form of irregular fractures and discontinuity spacings are frequent. Fillings are mostly closed, smooth to rough, sometimes containing calcite infillings. The thickness of the beddings in the tunnel can vary from laminate up to very thick. The bedding spacings are closed. Although the bedding directions and slopes are variable due to local folding and faults, they are mostly distinct in the NE-SW directions, with dip angles 30° or steeper (KGM, 2015).

Figure 3: A descriptive geological section of the Kilimli Formation's Sapca Member from the entrance portal

Şekil 3: Giriş portal Kilimli formasyonu Sapça Üyesinin jeolojik yapısını tanımlayıcı fotoğrafı

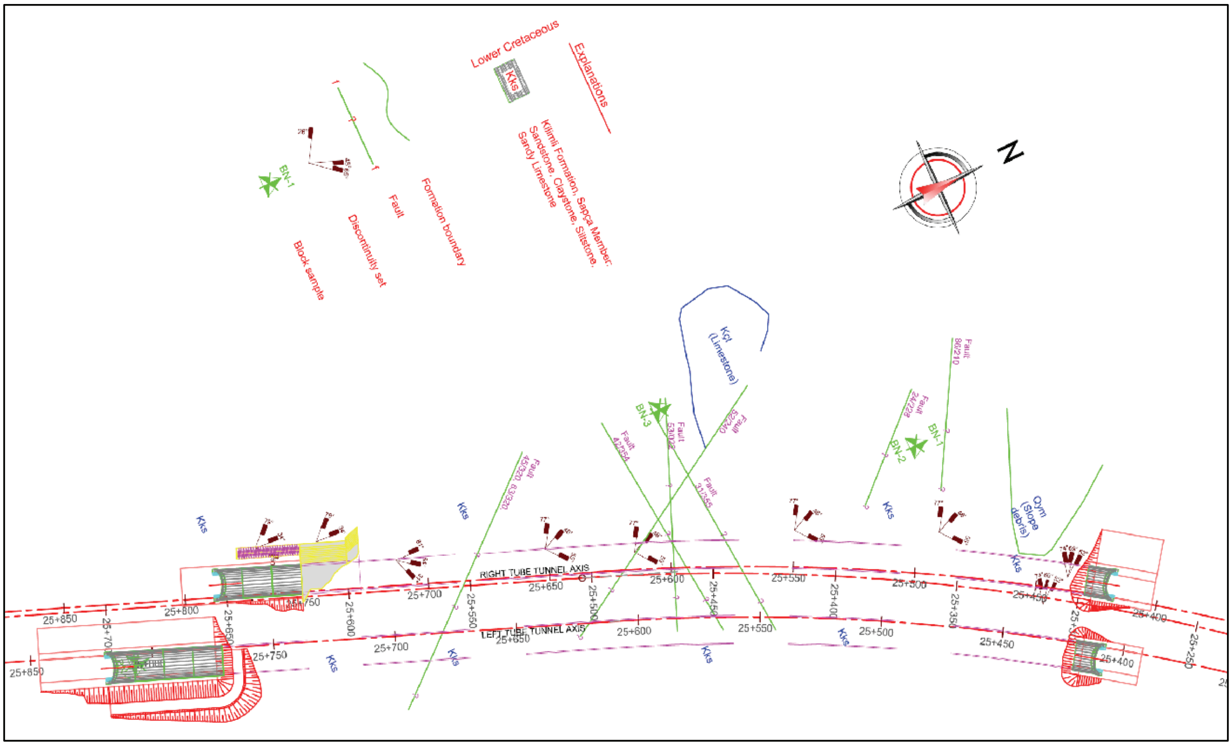

Figure 4: Geological map and plan of the tunnel axis

Şekil 4: Tünel eksenine ait plan ve jeolojik harita

Table 1. Laboratory test results of block samples taken from tunnel route

Tablo 1. Tünel güzergahından alınan blok numunelere ilişkin laboratuvar sonuçları

Sample No	W_n (%)	γ_n (kN/m^3)	E_i (MPa)	ν	UCS (MPa)	$I_{s(50)}$ (MPa)
BS-1	0,22	24,82	9826.00	0,257	38,19	1,22
BS-2	0,14	24,01	7423.00	0,250	30,15	1,27
BS-3	0,17	25,75	10585,67	0,263	41,67	1,47
BS-4	0,20	24,26	5833.00	0,253	23,51	1,19

W_n: Natural water content, γ_n: Natural unit volume weight, v: Poison ratio, I_s: Point load indice, UCS: Uniaxial compressive strength, E_i: Intact rock modulus of elasticity

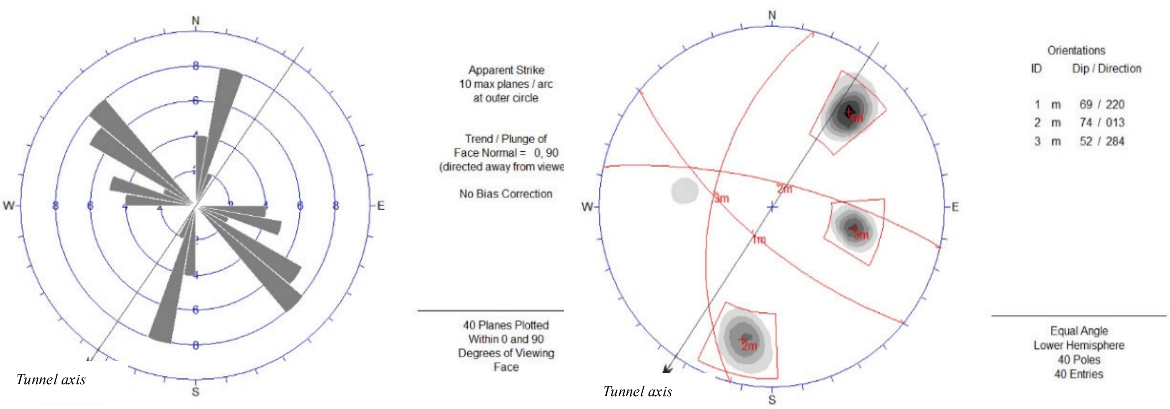

Figure 5. Contour planes and rose diagrams of the main discontinuity sets in the tunnel entrance portal

Şekil 5. Tünel giriş portalı ana süreksizlik setlerine ilişkin kontur ve gül diyagramları

Kinematical evaluations

The Dip and dip directions of the joint set were measured on the field for a kinematical analysis of the tunnel entrance portal section. According to measurements, dominating joint sets were specified by using contour and rose diagrams. A commercial software, Dips, was used for this aim. The density of the measured joints indicated that there are 3 main joint sets and some other random joints. Based on the measurements taken, dip/dip direction of the main bedding planes are 69°/220° and 74°/013° and dip/dip direction of the main joint sets are 52°/284°. The rose diagram and contour planes were given below in Figure 5.

According to the kinematical evaluations (Figure 6), a planar type failure stability problem is not expected entrance portal area. Wedge and toppling type failures were detected in the kinematical analysis (Figure 6). However, all wedges detected by the kinematical analysis have greater safety values and are stable (Figure 6 and 7). Toppling type failure was detected for only one discontinuity plane and it was excavated and removed from its housing once the excavation started.

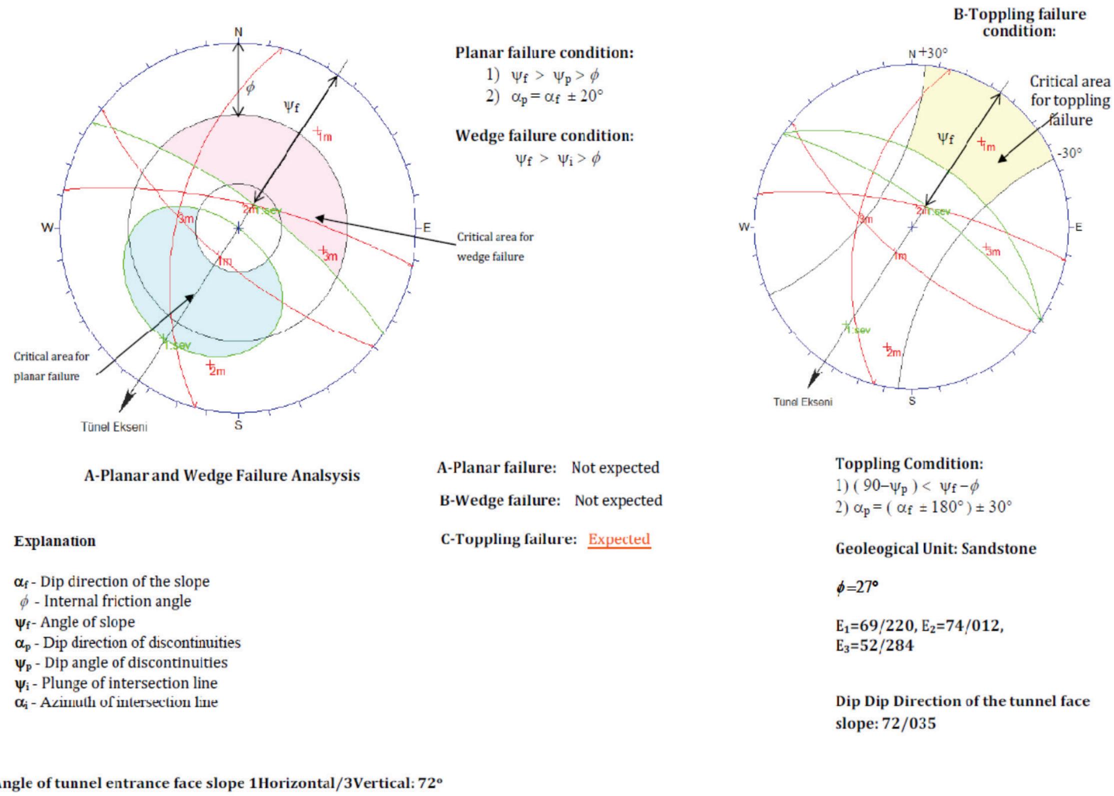

Figure 6. Tunnel entrance right tube (Km 25+427 – 25+434) kinematical analysis

Şekil 6. Tünel giriş sağ tüp (Km 25+427 – 25+434) kinamatik analizler

Geotechnical Evaluation of Tunnel Portal

The classifications and geotechnical parameters of the rock mass at the tunnel entrance were determined according to line surveys, rock surface analysis, laboratory tests applied to block samples, discontinuity measurements and engineering geology studies. RMR, Q, GSI scores and the geotechnical parameters of the entrance portal are given below. The estimation parameters for the RMR evaluation are provided in detail (Table 2).

Besides the rock mass parameters, a rock mass deformation modulus is also necessary for the numerical analysis. Accordingly, the estimation methodology is outlined briefly below. To obtain more accurate results, the deformation modulus was determined by using various approaches.

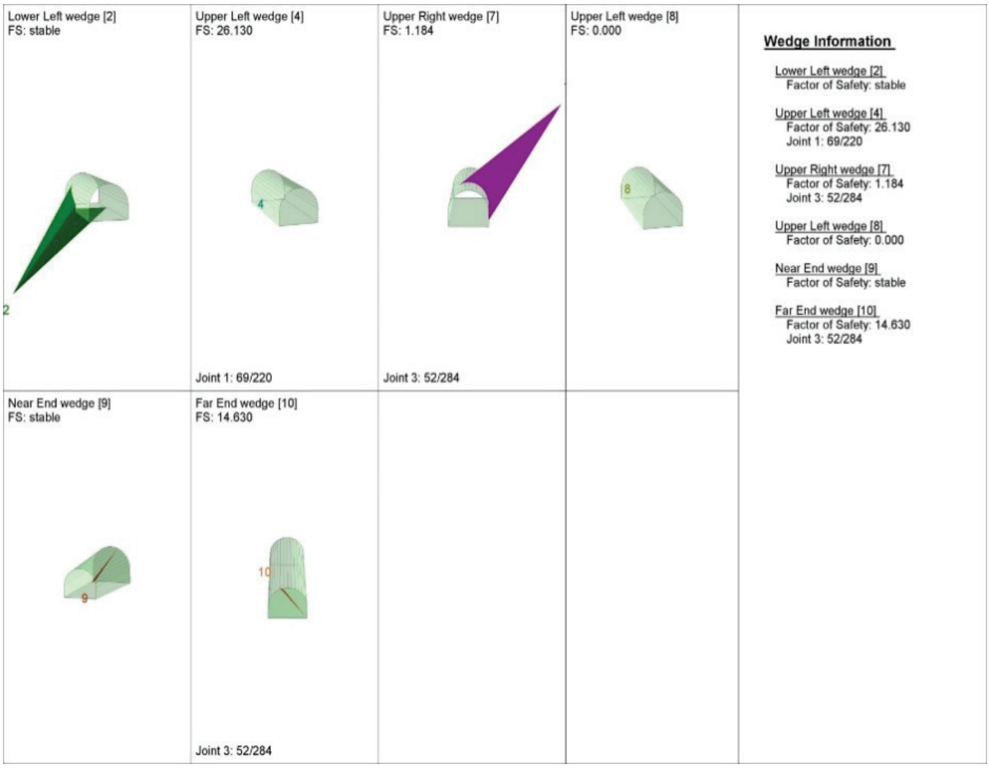

Figure 7. Tunnel entrance right tube (Km 25+427 – 25+434) probable wedge failures and their factor of safety values
Şekil 7. Tünel giriş sağ tüp (Km 25+427 – 25+434) olası kama tipi kaymalar ve bunlara ait güvenlik sayısı değerleri

- Nicholson&Bieniawski (1990)

$$\frac{E_m}{E_i} = \frac{1}{100}\left(0{,}0028RMR^2 + 0{,}9e^{\left(\frac{RMR}{22{,}82}\right)}\right) \quad [1]$$

For RMR=49 and E_i=9826 MPa;

$$\frac{E_{rm}}{9826} = \frac{1}{100}\left(0{,}0028x49^2 + 0{,}9e^{\left(\frac{49}{22{,}82}\right)}\right)$$

E_{rm}=1418 MPa

- Hoek&Diederich (2005)

$$E_m = E_i\left(0{,}02 + \frac{1-\frac{D}{2}}{1+e^{[(60+15D-GSI)/11]}}\right) \quad [2]$$

For GSI=47, Ei=9826 MPa and disturbance factor D=0,5;

$$E_m = 9826\left(0{,}02 + \frac{1-\frac{0{,}5}{2}}{1+e^{[(60+15x0{,}5-47)/11]}}\right)$$

E_{rm}=1186 MPa

As the Q estimation uses the parameters based on the RMR chart, details of the Q value estimation (Barton 2002) are not provided here for the sake of brevity (*$Q = RQD/J_n * J_r/J_a * J_w/SRF$; J_n=12, J_r=3, J_a=1, J_w=1, SRF=5*).

Table 2: Tunnel entrance portal right tube (Km 25+427 – 25+434) RMR analysis
Tablo 2: Tünel girişi sağ tüp (Km 25+427 – 25+434) RMR analizi

1	Strength of intact rock material	Point load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range- uniaxial compressive test is preferred		
	Uniaxial compressive strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa	
	Rating		15	12	7	4	2	1	0
2	Drill core Quality RQD		%90-%100	%75-%90	%50-%75	%25-%50	<%25		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2m	0,6-2m	200-600 mm	60-200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Discontinuity length		<1m	1-3m	3-10m	10-20m	>20m		
	Rating		6	4	2	1	0		
	Separation (Aperture)		Yok	<0,1mm	0,1-1,0mm	1-5mm	>5mm		
	Rating		6	5	4	1	0		
	Roughness		Very rough	Rough	Slightly rough	Smooth	Slickensided		
	Rating		6	5	3	1	0		
	Infilling <5mm		None >5mm	Hard filling		Soft filling			
				<5mm	>5mm				
	Rating		6	4	2	2	0		
	Weathering		Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed		
	Rating		6	5	3	1	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	Yok	<10 l/m	10-25 l/m	25-125 l/m	>125 l/m		
		(Joint water press)/ (Major principal σ)	0	0.0-0.1	0.1-0.2	0.2-0.5	>0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

GSI value and Hoek & Brown failure criterion were also provided in Figure 8.

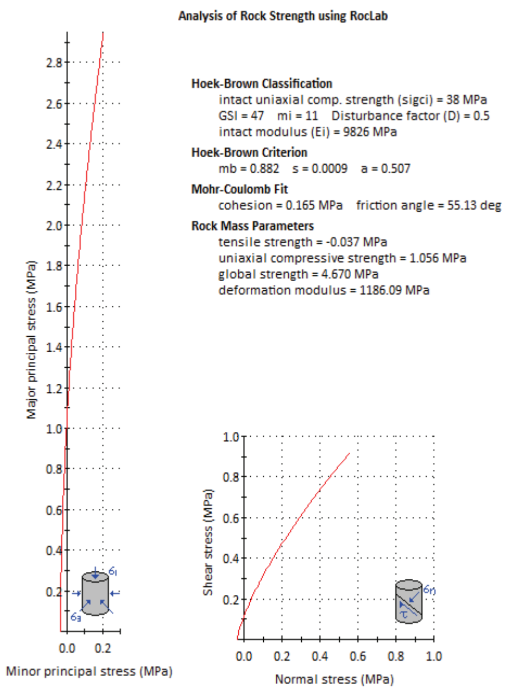

Figure 8. Tunnel entrance right tube (Km 25+427 – 25+434) GSI value ve Hoek&Brown rock mass failure criterion

Şekil 8. Tünel giriş sağ tüp (Km 25+427 – 25+434) GSI değeri ve kaya kütlesi için Hoek&Brown yenilme kriteri

All of the obtained rock mass geotechnical parameters and results are shown in Table 3.

Support Design According to Rock Mass Classification

The tunnel excavation was designed in accordance with the sequential excavation method. Accordingly, the NATM philosophy was chosen as an excavation methodology. The tunnel entrance portal support system was designed in accordance with the universal rock mass classification systems by using Table 3 and site investigation studies. Support design suggestions of rock mass classification systems and NATM were listed in Table 4.

Table 3. Geotechnical parameters of entrance portal section

Tablo 3. Tünel giriş portal jeoteknik parametreleri

Parameters	Score
UCS, Uniaxial Compressive Strength (MPa)	38
Basic RMR score (Bieniewski, 1989)	49
Adjusted RMR score	37
Q, Tunnel Quality Indice (Barton, 2002)	1,25
GSI, Geological Strength Indice score (Sönmez et.al., 2002)	47
GSI_r Residual Geological Strength Indice score (Cai vd., 2007)	25,03
m_i (Hoek-Brown rock constant)	11
m_b (Hoek-Brown rock constant)	0,882
s (Hoek-Brown rock constant)	0,0009
a (Hoek-Brown rock constant)	0,507
D, Disturbance factor	0,5
E_i, Modulus of Elasticity (MPa)	9826
ɣ, Unit Volume Weight (kN/m^3)	25
H, Overburden thickness (m)	15
C_{rm}, Cohesion (kPa) RMR (Bieniewski, 1989)	100-200 165*
C_{rm}, Cohesion (kPa) (Hoek vd.,1997)	
$Ø_{rm}$, Internal Friction Angle (°) (Bieniewski, 1989)	15-25 55,13*
$Ø_{rm}$, Internal Friction Angle (°) (Hoek vd., 1997)	
E_{rm}, Deformation Modulus (MPa) (Nicholson vd., 1990)	1418 1186*
E_{rm}, Deformation Modulus (MPa) (Hoek vd., 2005)	

Table 4. Support suggestions of rock classification system and NATM for tunnel entrance portal excavation
Tablo 4. Tünel giriş portal kazısına ilişkin kaya sınıflama sistemelerinin ve NATM nin destek sistemi önerileri

Excavation support classes according to rock mass classifications
– **RMR Support and Excavation Suggestion, Class IV (App.I):** Excavation should be top heading and bench, max advance length should be between 1.0 – 1.5 m in the top heading, all supports should be completed 10 m before the excavation face, 4.0 – 5.0 m in length systematic rock bolts should be applied at 1.0 – 1.5 m intervals, together with welded wire mesh, on the top heading at a thickness of 100 – 150 mm; and on the side walls at a thickness of 100 mm shotcrete should be applied; steel ribs, when necessary, should be applied in 1.5 m intervals (Bieniawski, 1989).
– **Q Support and Excavation Suggestion, Class 23A for the roof (App.II);** tensioned systematic rock bolts at 1.0 – 1.5 m intervals; with cement-injection and welded wire mesh with 10 – 15 cm thick shotcrete
– **Q Support and Excavation Suggestion, Class 23B for the walls;** systematical rock bolts at 1.0 – 1.5 m intervals; with cement-injection, 10 – 15 cm thick shotcrete (Barton et. al., 1974 and Barton, 2002).
– **NATM Support and Excavation Suggestion, Class B2;** Very brittle rock mass, water inflow has minor effect on the strength of weathered and disintegrated rock mass, excavation divided into sections depending on the excavation cross-section, excavation advance length depending on the unsupported stand-up time and distance. Advance length should not be more than 1.5 – 2.0 m in the top heading excavation and 3.0 – 3.5 m in the bench. Excavation can be done by using soft blasting. Systematic support is necessary on the roof and the walls. Forepoling should be used, when necessary, on the roof (KTŞ, 2013).

As seen in Table 4, NATM provides subjective information about the support system and does not contain detailed specific data about the supporting system (Cording and Deere, 1972). Therefore, a support design based solely on NATM would be incomplete, if not inaccurate. Therefore, numerical models supported with rock mass classifications can overcome this shortcoming if appropriate geotechnical inputs are obtained and accurate geological models are created to use in the models (Carter, 1992).

Numerical Excavation Model of The Tunnel Entrance Portal

As explained in the previous section, in addition to the support and excavation suggestions of rock mass classification systems, stress distributions that vary depending on the excavation medium, the overburden thickness and tectonic factors should also be considered in support design. To achieve this goal, finite element numerical analyses were employed to assess support design and deformations that follow the excavation. For this aim, a 15-stage model was generated. As each of the tunnel tubes are going to be excavated using conventional methods, i.e. mechanically, top heading excavation was completed in three steps in the model. When it comes to bench excavation, it was conducted in two steps to simulate real excavation and supporting conditions in the field. 5 cm shotcrete and wire mesh were applied in the numerical models to observe controlled deformations of the top heading excavation. Although discontinuities such as joints and bedding planes were observed and measured during line surveys and field investigations, most of the discontinuity sets were closed and unfilled, and their continuity was not observed in the host rock; therefore, they were not integrated into the numerical models. Instead, material conditions were incorporated into the numerical model by selecting the "Generalized Hoek-Brown Criterion" as the failure criterion in the model. In this approach, instead of specifying a specific rock mass strength value, rock mass strength values were adjusted based on depth, excavation conditions, and stress distributions. In this way, the Kilimli Formation's Sapca member, which is the main geological unit of the tunnel route consisting of sandstone, claystone, and siltstone intercalation, could be modeled as a whole (Hoek and Brown 1997, Sönmez and Ulusay 2002). The cross-section of the entrance portal and the created model structure with the input rock mass parameters are depicted in Figure 9.

There are various approaches to determine in-situ stress in literature e.g., Jamison and Cook (1979), Hoek and Brown (1980), Sheorey (1994), Amadei and Stephansson (1997), Hudson and Harrison (1997), Reinecker et al. (2004). However, it has been suggested to use these relationships with caution (Zhang 2017). Most of these approaches determine "k" value, the average horizontal stress to vertical stress ratio, in general, to be greater than 1. This approach is accurate when the surface topography is horizontal. However, if the lateral boundaries are limited, such as homogeneous rock mass with a complex topography consisting of hills and no surface loads, the rock mass is under gravity alone with no lateral displacements (Zhang 2017). The case explained in this study is very close to the surface and lateral loads are limited. Therefore tunnel excavation will mainly be under gravity loading. For this reason, a numerical model is employed to determine the "k" value by using actual topography. The "actual ground surface condition - gravity" option was used in numerical modelling.

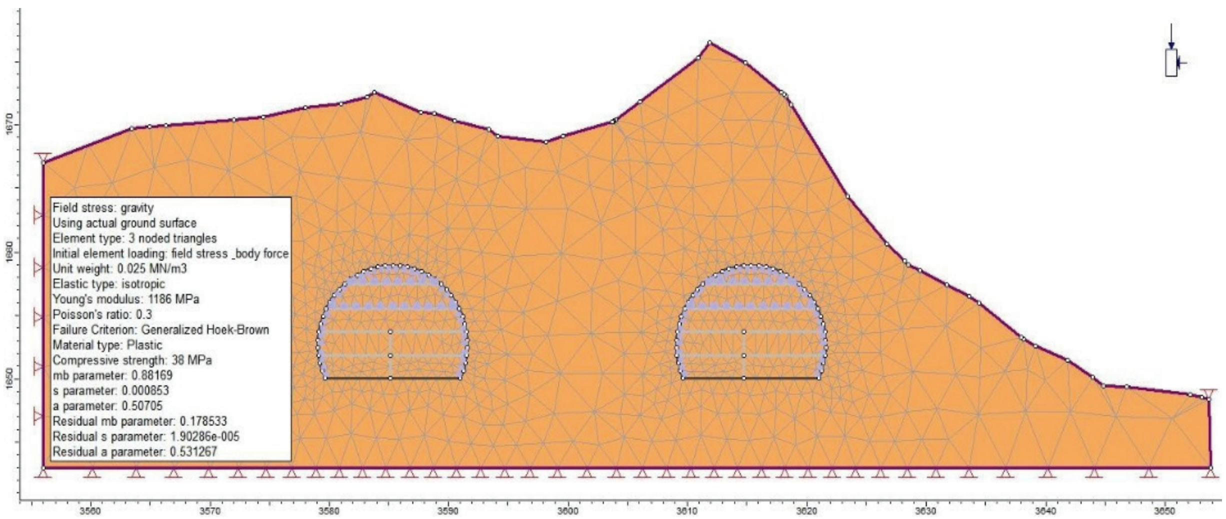

Figure 9. Model cross-section, mesh structure and input parameters

Şekil 9. Model kesiti, ağ yapısı ve girdi parametreleri

Pre-excavation stress trajectories, total stresses, and stress distributions found through numerical analysis were presented in Figure 10. As depicted in Figure 10, the topographical conditions (in the form of asymmetrical loading conditions) and geological structure are expected to result in asymmetrical stress distributions around the excavation area upon commencement of excavation, potentially exerting adverse effects on tunnel stability.

In the 15-stage excavation model, the excavation of the left tube's top-heading was completed in three steps. Following this, a 5 cm shotcrete layer was applied for preliminary support to prevent rock falls and facilitate controlled deformations and stress relief. Subsequently, top-heading of the right tube was excavated in a similar manner. Bench excavations were carried out in two steps and a 5 cm shotcrete layer was applied in the numerical model in accordance with the field application.

The evolution of horizontal and vertical stress distributions after excavation and the corresponding shear and tensile-strain state are shown in Figures 11 and 12. As is evident in the results obtained, the vertical stresses after excavation are approximately 3.50 times higher than the horizontal stresses. After this stage, to ensure tunnel stability, the selection of support type, pattern, and structure should be carried out to mitigate horizontal and vertical stresses based on critical cross-sections. However, details and application of rock bolt and other supporting elements were not given here. This is because the aim of this study as to emphasize the improtance of geological structure and the behavior of geological units after the excavation.

As observed in the following figures, in this example, the highest stresses have predominantly developed in a nearly vertical direction, sloping towards the slope. During the design of the support system, not only the pressures that the selected supports can withstand but also the directions in which these pressures would act were taken into account.

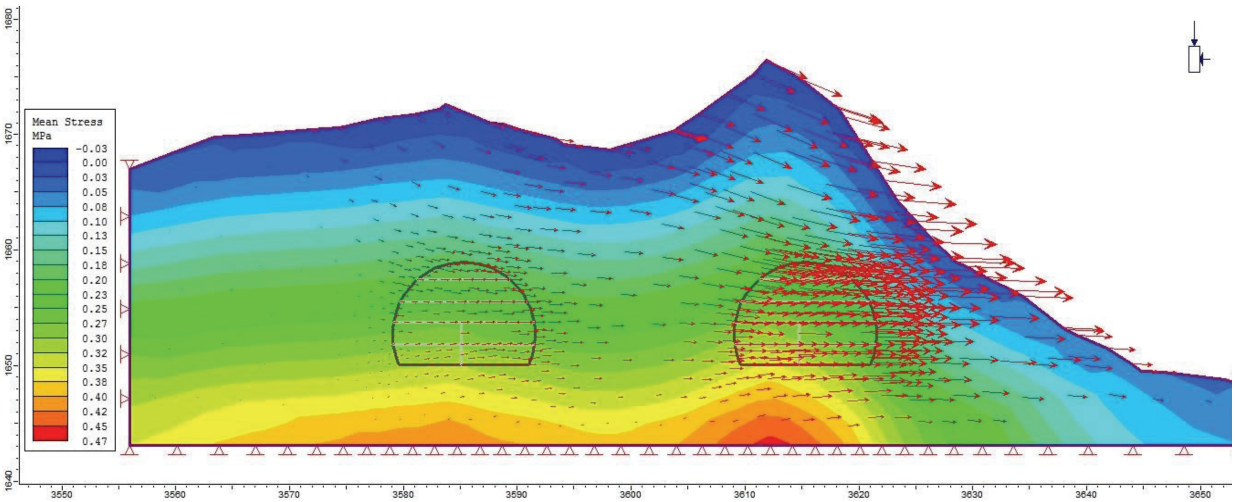

Figure 10. Total stress distributions during the pre-excavation stage.
Şekil 10. Kazı öncesi aşamada toplam gerilme dağılımları

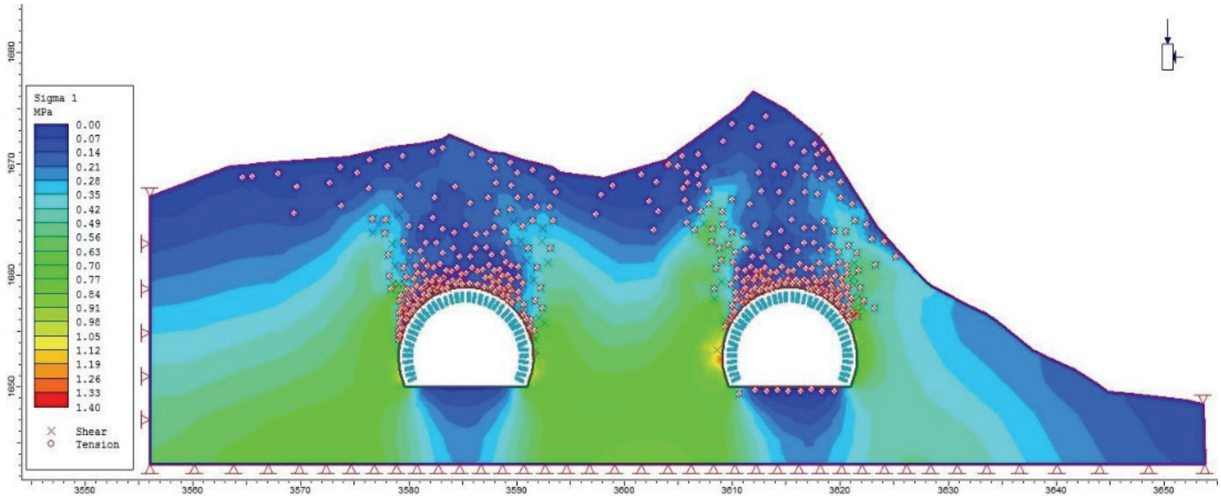

Figure 11. Vertical stress distributions after the excavation
Şekil 11. Kazı sonrası aşamada düşey gerilme dağılımları

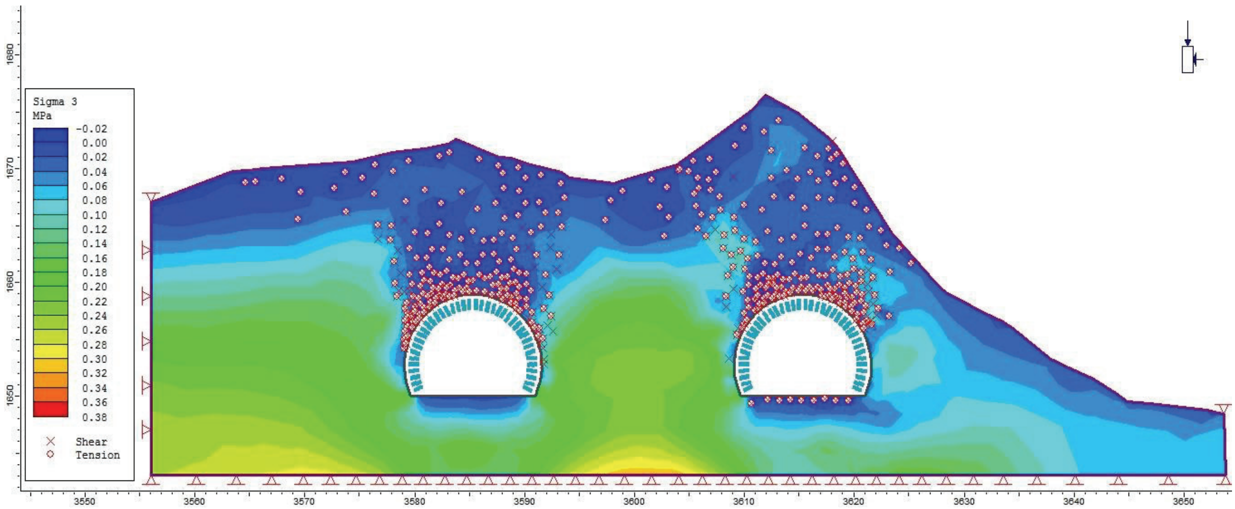

Figure 12. Horizontal stress distributions after the excavation
Şekil 12. Kazı sonrası aşamada yatay gerilme dağılımları

5. Discussion

Obtained modeling results show that there will be asymmetrical loads on the tunnel's right side walls after the excavation. So, the support suggestions or post failure behavior predictions may give misleading results if they are only based on empirical rock mass classifications. These suggestions mostly give a good starting point for the designer but should be verified once excavation commences. Numerical models are also useful and practical tools for this aim. However, designs that depend only on numerical models will also give misleading results. It should be noted that, despite the fact that most rock mass evaluation systems have not undergone significant revisions, excavation and supporting technologies have evolved. Accordingly, while high-capacity excavation machinery and blasting technology enable larger-scale excavation at once, they also may induce greater deformation in the host rock. Hence, deformations occurring in the host rock after blasting or machinery excavation should be incorporated into numerical models. As a result, supporting types and strategy need to be revised both in the rock mass evaluation systems and numerical modelling strategy in accordance with the developed excavation technology. Tunnel excavation and support design should have to consider field studies, geological models, stratigraphical relations of the rock masses, historical tectonism of the area and engineering geology model and laboratory studies. Numerical models considering both this data and engineering input will give more accurate results. The results of the numerical models will directly depend on what the designer puts into the models. Additionally it must be noted that because of the limitations that obstruct geotechnical field studies, there will always be some assumptions made. In conclusion, tunnel designs and field studies should be carried out with an experienced geological – geotechnical engineer. In this way, accurate numerical models that reflect real conditions in the field can be produced.

6. Conclusion

The aim of this study is to show how empirical predictions regarding tunnel excavation support may give misleading results if they are not supported with geological field evaluations and engineering geology studies. This is because empirical supporting strategies such as RMR, Q or NATM do not take into account post excavation stress distributions, depth and topographical conditions. As a result, once the geological and engineering geology model has been generated, the selection of support type and choices should be undertaken in collaboration with experienced civil, mining or geological engineers or competent engineers with postgraduate education in these fields. Finally, excavation and support designs, as well as the application project, derived from numerical models, must be monitored in the field. This monitoring is necessary not only to ensure that the construction team is working in accordance with the outputs of the numerical model design, but also to verify the accuracy of the geological model estimation. In this manner, excavation safety can be established, and a more accurate and economical project can be executed.

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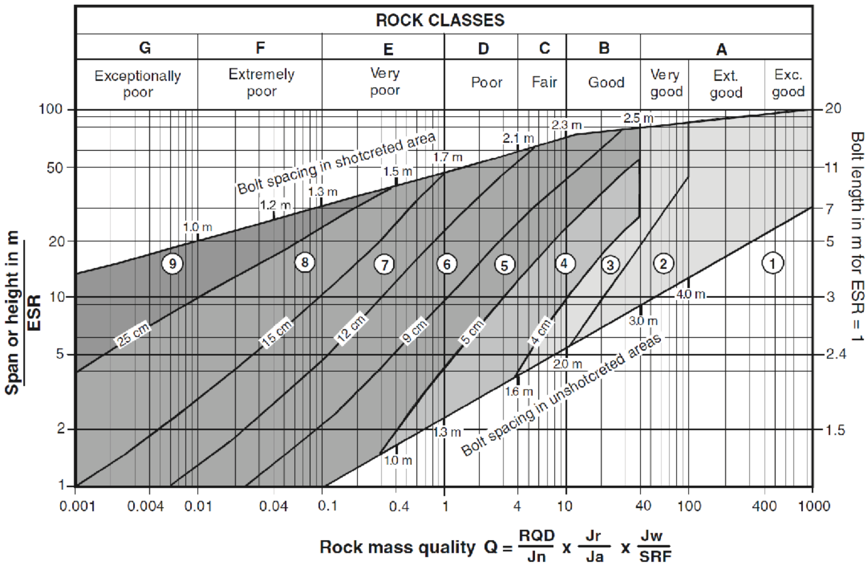
Appendices

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock ***RMR*****: 81-100**	Full face, 3 m advance.	Generally no support required except spot bolting.		
II - Good rock ***RMR*****: 61-80**	Full face , 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock ***RMR*****: 41-60**	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None.
IV - Poor rock ***RMR*****: 21-40**	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor rock ***RMR*****: < 20**	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Appendix I: Guidelines for excavation and support of 10 m span rock tunnels in accordance with the *RMR* system (After Bieniawski 1989).



REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting, (and unreinforced shotcrete, 4 - 10 cm)
5) Fibre reinforced shotcrete and bolting, 5 - 9 cm
6) Fibre reinforced shotcrete and bolting, 9 - 12 cm
7) Fibre reinforced shotcrete and bolting, 12 - 15 cm
8) Fibre reinforced shotcrete, > 15 cm, reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Appendix II: Estimated support categories based on the tunnelling quality index Q (After Grimstad and Barton, 1993, reproduced from Palmstrom and Broch, 2006).

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