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A STUDY ON TUNNELING SUPPORT SYSTEMS WITH RMR AND Q METHODS FOR ERDEMLI-SILIFKE-TAŞUCU TUNNEL PROJECT

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M.Sc. THESIS

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Ulaş Eren KAYA
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İNŞAAT MÜHENDİSLİĞİ ANABİLİM DALI

ERDEMLİ-SİLİFKE-TAŞUCU TÜNEL PROJESİNİN RMR VE Q TÜNEL DESTEK SİSTEMLERİYLE İNCELENMESİ ÜZERİNE BİR ÇALIŞMA

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ÖZET

Doğal kaya kütlesi, diğer mühendislik malzemelerinin aksine belirli bir jeolojik ortamda oluşmakta ve değişen özelliklere sahip farklı kaya türlerinin ve yapısal düzlemlerin kombinasyonlarından oluşmaktadır. Kaya kütlesinin karmaşıklığı nedeniyle, mühendislik bilgi ve deneyime dayalı olarak kaya kütlesi, kalite ve dayanıma göre farklı sınıflara ayrılmaktadır. Tünel projeleri için gerekli geomekanik dayanım parametreleri, tünel güzergahı boyunca yapılan jeolojik değerlendirmeler ve laboratuvar deneyleri aracılığıyla belirlenmiştir. Kaya kütlesinin davranışını matematiksel olarak modelleyebilmek için kaya kütlesi derecelendirme sistemlerinden türetilen ilişkiler, Jeolojik Dayanım İndeksi (GSI) kavramı ve literatürde bulunan değerler kullanılarak belirlenmiştir. Saha çalışmalarından toplanan tüm bilgilerin analiz edilmesi, deneysel ve analitik yöntemler kullanarak tünel kazıları ve destek sistem tasarımları için değişkenler belirlenmiştir. Bu çalışma, T-8 tünel inşaatı sırasında karşılaşılan dayanım sorunlarını ele almak üzere geliştirilen bir sayısal model ile gerçek durum arasındaki durumu karşılaştırmaktadır. 13. bölgede Erdemli-Silifke-Taşucu T-8 güney tünel tüpü (km: 175+640-175+950) ile T-8 kuzey tünel tüpü (km: 175+700-176+084) arasındaki tünel destek sistemlerini sonlu elemanlar (SE) analiz yöntemi kullanılarak incelemiştir. Tünel ayna kazıları sırasında jeolojik ve geoteknik çalışmalar yürütülmüş olup, Kaya Tünel Kalite İndisi (Q) ve Kaya Kütle İndeksi (RMR) sınıflandırmaları ile deformasyon ölçümleri hazırlanmıştır. Sayısal modelde tahmin edilen destek sistemlerinin gerçek uygulama ile uyumu analiz edilmiştir. Tünel destek sistemleri mevcut jeolojik ve geoteknik verilere dayalı olarak belirlenmiştir. Kazı ve destek imalatı sırasında karşılaşılan jeolojik yapı ile tahmin edilen modelin uyumlu olduğu görülmüştür. Tünel kazısı sırasında öngörülen destek sistemlerinde revizyon gerekmediğinden, sayısal model doğrulandığı ve başarılı olduğu görülmüştür.

Anahtar Kelimeler: Tünel, kaya sınıflamaları, tünel destekleme sistemleri, sayısal modelleme.

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MASTER THESIS

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ABSTRACT

The natural rock mass, unlike other engineering materials, is formed in a specific geological setting and consists of different combinations of rocks and structural planes with varying properties. Due to the complexity of the rock mass, it is categorized into different classes based on rock mass quality and stability in accordance with engineering knowledge and experience. The geomechanically strength parameters required for tunnel projects are determined through geological assessments along the tunnel route and laboratory experiments. In order to mathematically model the behavior of rock masses, strength and deformation parameters are determined using relationships derived from rock mass rating systems, the concept of Geological Strength Index (GSI), and values found in literature. Variables for tunnel excavations and support system designs using empirical and analytical methods are identified by analyzing all the information gathered from the field. This study focuses on the agreement between a numerical model, developed to address stability issues encountered during the construction of the T-8 Tunnel, and the actual case. The finite element (FE) analysis method is employed to examine the model and support systems used in the rock masses between the T-8 Tunnel South Tube (Km: 175+640-175+950) and the T-8 Tunnel North Tube (Km: 175+700-176+084) sections of the Erdemli - Silifke - Taşucu 13th Zone. Geological and geotechnical studies were conducted during tunnel face excavations, resulting in the preparation of Tunneling Quality Index (Q) and Rock Mass Rating (RMR) rock classifications, as well as deformation measurements. The compatibility between predicted support systems in the numerical model and the actual implementation is analyzed. Tunnel support systems are determined based on existing geological and geotechnical data. It has been observed that the estimated model is compatible with the geological structure encountered during excavation and is supporting the construction. Since no revision was required in the support systems envisaged during the tunnel excavation, the numerical model was validated and found to be successful.

Keywords: Tunneling, rock classifications, tunnel support systems, numerical modeling.

PREFACE

I am grateful to have had the opportunity to embark on this journey of academic exploration and discovery. This thesis is the result of my hard work, dedication, and perseverance, and I am proud to share it with the academic community. I would like to extend my sincere thanks to my advisor, Assist. Prof. Dr. Nurullah Akbulut, who has been an incredible mentor, providing invaluable guidance, support, and encouragement throughout this process. I would also like to thank my family for their encouragement and support, as well as for being a source of inspiration for me. Their unwavering faith in me has made it possible for me to pursue my academic goals. Also, I dedicate this thesis to our citizens who lost their lives on 6 February 2023 Kahramanmaras earthquake in Türkiye. I hope this thesis will make a meaningful contribution to the field of study and serve as a source of inspiration for future generations of researchers.

Ulaş Eren KAYA
Gaziantep - 2023

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ABBREVIATIONS OR SYMBOLS LIST

FE	Finite Element Method
Q	Rock Mass Quality Index
RMR	Rock Mass Rating
GSI	Geological Strength Index
RQD	Rock Quality Designation
NATM	New Austrian Tunneling Method
KTŞ	Technical Specification for Highways (Türkiye)
IBO	Self-Drilling Anchor Bolt System
Jr	Joint Roughness Status
Jw	Joint Water Reduction Factor
Jn	Number of Joint Sets
Ja	Joint Alternation Number
SRF	Stress Reduction Factor
ɣ	Unit Volumetric Weight of the Rock
B	Tunnel Width
S	Stress Factor
C2	C2 Rock Classification
ÖNORM	Underground Works Austrian Standard
C30/37	C30/37 Concrete Class
E	Young’s Modulus

1. INTRODUCTION

1.1. General

The process of designing tunnels that are bored underground for transportation is highly difficult although complex geological and geotechnical features are defined in detail and interpreted by experienced engineers. There are different engineering design approaches, which are yet related to each other, that are used for underground tunnel faces.

There are two methods that involve empirical methods which are prevalently used by implementers due to their practicality of implementation and are mentioned in detail in the technical specifications of construction works. These are the Rock Mass Quality (Q) and Rock Mass Rating (RMR) systems.

The input parameters of the system are the strength of the rock material, rock quality designation (RQD), discontinuity spacing, discontinuity surface conditions, and underground water conditions. After adding the scores given to all parameters, the basic score is obtained. Following necessary adjustments such as discontinuity orientation adjustment, the adjusted score value is obtained. Based on this value, a support system is recommended for the tunnel or face to be created. These operations are carried out based on detailed data obtained at tunnel construction excavation stages, and they allow the rapid determination of support systems on the site.

Rock behaviors are modeled by using the Phase-2D computer program that is based on the finite element method which can provide quantitative information about face stability and support performance in tunnel construction and is prevalently used (Rocscience, 2009). In light of the data that is collected from the site, the tunnel that is to be constructed is modeled in the computer environment. With this method, the opportunity to examine the pre-boring, post-boring, and temporary and permanent support system construction stages step by step before the construction process is provided, and analyses of cases that can be encountered at the stage of construction are conducted. The performance of support elements that are recommended by the system based on actual on-site data is tested based on numerical modeling for different conditions, and this allows the identification of existing shortcomings. The proper use of the computer program is as important as the appropriate collection of data from the site. With errors that can occur while transferring the accurately collected data to the digital environment, it is possible to encounter modeling outputs that are far from reality.

It is not advisable to classify rocks by using a single method, and this often results in deviations from the actual case. Using on-site data that are compiled under the default stress conditions, multiple calculations should be made. Every method has unique advantages and disadvantages. Therefore, all methods that are used in tunnel designs should be correlated with each other, and their compatibility with the actual case should be investigated.

1.2. Objective of the Study

A sinkhole occurred between the Km:176+143.55 and 176+104.73 of the North Tube section located between the Erdemli – Silifke – Taşucu which resulted in subsidence on the existing road surface along the Antalya – Mersin D400 Highway that had already resided above the tunnel, and solution works were carried out by additional tunnel support in this study.

This study employs the finite element analysis method to analyze face mapping, Q and RMR rock classifications, and support design works in the tunnel. The study's aim is to enhance tunnel stability and safety with broader implications for tunnel engineering practices.

1.3. Organization of the Study

Chapter 1: This chapter discusses the importance of using some methods that are designed for the identification of rock classes, the significance of correlations among these methods, and the place of data transfer in classification.

Chapter 2: In this chapter, studies conducted on rock classification systems, the general properties of the study site, its geology, and seismicity are mentioned. Additionally, information is provided about NATM.

Chapter 3: This chapter includes descriptions of the methods that were used in the study. Informations about geotechnical conditions, geotechnical analyses, numerical analyses and quantities of the support elements for the T-8 North and South Tube are given in this chapter.

Chapter 4: This chapter includes the results and discussions of support elements that are used in the tunnel, the boring and support methodology, and the deformation measurements.

Chapter 5: The conclusions and recommendations of the thesis are presented in this chapter.

2. LITERATURE REVIEW

2.1. Rock Mass Classification Systems

Rock masses do not have homogeneous structures, and their structures include several fractures, joints, gaps, and other features. The presence of these conditions is highly significant in the construction of underground engineering structures.

Rock mass classification systems are utilized while designing many underground access structures that are opened for mining and civil engineering purposes. Some of these classification systems not only grade the rock mass but also provide recommendations of support systems that are required for its stability. Among classification systems that include support recommendations, the Rock Mass Quality (Q), Rock Mass Rating (RMR), New Austrian Tunneling Method (NATM), and Finite Element (FE) Modeling methods were included in this study. The fact that these systems are open to improvement with new studies increases the interest of researchers in these systems.

2.1.1. Advantages of rock mass classification

Classification of rock mass improves the quality of site investigations by calling for a systematic identification and quantification of input data. A rational, quantified assessment is more valuable than a personal (non-agreed) assessment. Classification provides a checklist of key parameters for each rock mass type (domain) i.e. it guides the rock mass characterization process. Classification results in quantitative information for design purposes and enables better engineering judgment and more effective communication on a project (Bieniawski, 1993). A quantified classification assists proper and effective communication as a foundation for sound engineering judgment on a given project (Hoek, 2007).

Correlations between rock mass quality and mechanical properties of the rock mass have been established and are used to determine and estimate its mechanical properties and its squeezing or swelling behavior.

2.1.2. Disadvantages of rock mass classification

According to Bieniawski (1993), the major pitfalls of rock mass classification systems arise when:

- using rock mass classifications as the ultimate empirical book, i.e. ignoring analytical and observational design methods;
- using one rock mass classification system only, i.e. without cross-checking the results with at least one other system;

- using rock mass classifications without enough input data;
- using rock mass classifications without full realization of their conservative nature and their limits arising from the database on which they were developed.

Some people are of the opinion that:

i) natural materials cannot be described by a single number,

ii) other important (often dominating) factors are not considered,

iii) results of rock mass classification are prone to misuse (e.g., claims for changed conditions) (Bieniawski, 1989).

Most of the multi-parameter classification schemes (Wickham et al (1972) Bieniawski (1973, 1989) and Barton et al (1974)) were developed from civil engineering case histories in which all of the components of the engineering geological character of the rock mass were included. In underground hard rock mining, however, especially at deep levels, rock mass weathering and the influence of water usually are not significant and may be ignored. Different classification systems place different emphases on the various parameters, and it is recommended that at least two methods be used at any site during the early stages of a project.

2.1.3. Terzaghi's rock mass classification

The earliest reference to the use of rock mass classification for the design of tunnel support is in a paper by Terzaghi (1946) in which the rock loads, carried by steel sets, are estimated on the basis of a descriptive classification. While no useful purpose would be served by including details of Terzaghi's classification in this discussion on the design of support, it is interesting to examine the rock mass descriptions included in his original paper, because he draws attention to those characteristics that dominate rock mass behaviour, particularly in situations where gravity constitutes the dominant driving force. The clear and concise definitions and the practical comments included in these descriptions are good examples of the type of engineering geology information, which is most useful for engineering design.

Terzaghi's descriptions (quoted directly from his paper) are:

- *Intact rock* contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as a spalling condition. Hard, intact rock may also be encountered in the popping condition involving the spontaneous and violent detachment of rock slabs from the sides or roof.

- *Stratified rock* consists of individual strata with little or no resistance against separation along the boundaries between the strata. The strata may or may not be weakened by transverse joints. In such rock the spalling condition is quite common.
- *Moderately jointed rock* contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked those vertical walls do not require lateral support. In rocks of this type, both spalling and popping conditions may be encountered.
- *Blocky and seamy rock* consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support.
- *Crushed but chemically intact rock* has the character of crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.
- *Squeezing rock* slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or clay minerals with a low swelling capacity.

  Swelling rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks that contain clay minerals such as montmorillonite, with a high swelling capacity.

2.2. Rock Mass Quality (Q) System

The Q system is a classification method based on six parameters which was introduced to the literature in 1974 by the Norwegian Geotechnical Institute and modernized by Barton et al. (1980).

$$Q = \frac{RQD}{Jn} x \frac{Jr}{Ja} x \frac{Jw}{SRF}$$

Definitions of the parameters:

RQD : Rock quality designation,

Jn : Number of joint sets,

Jr : Joint roughness coefficient,

Ja : Joint alternation number,

Jw : Joint water reduction factor,

SRF : Stress reduction factor.

In the formula, RQD/Jn is an indicator of the general structure of the rock mass and its block size, Jr/Ja is an indicator of interblock sliding strength, and Jw/SRF is an indicator of the active stress factor. According to this classification system, 6 parameters are used to calculate the Q value, and these parameters are presented with their explanations in Table 2.1

Table 2.1. RQD input parameter of the Q system and its values (Barton et al., 1974).

	Definition	RQD Values	Explanation
A	Very Poor	0-25	*Where RQD≤10, a nominal value of 10 is used to calculate the Q-slope value. *RQD is taken as 5 or its multiples.
B	Poor	26-50	
C	Fair	51-75	
D	Good	76-90	
E	Excellent	91-100	

Table 2.2. Jn input parameter of the Q system and its values (Barton et al., 1974).

	Definition	Jn Values	Explanation
A	Solid, little or no joints	0.5-1	*3xJn for intersecting tunnels *2xJn for tunnel portals.
B	One joint set	2	
C	One joint set and random joints	3	
D	Two joint sets	4	
E	Two joint sets and random joints	6	
F	Three joint sets	9	
G	Three joint sets and random joints	12	
H	Four or more joint sets, multiple random joints in a form resembling sugar cubes	15	
J	Crushed rock, earthlike	20	

Table 2.3. Jr input parameter of the Q system and its values (Barton et al., 1974).

	Definition	Jr Value	Explanation
	a) Rock-wall contact, **b) Rock-wall contact before 10 cm of shear deformation**		*1.0 added of mean spacing of the relevant joint set is greater than 3 meters. *Jr=0.5 can be used for planar, slickensided joints with lineations if the lineations are suitable. *Classes from B to G are small-scale features, others are large-scale features.
A	Discontinuous joints	4	
B	Rough or irregular, undulating	3	
C	Smooth, undulating	2	
D	Slickensided, undulating	1.5	
E	Rough or irregular, planar	1.5	
F	Smooth, planar	1	
G	Slickensided, planar	0.5	
	c) No rock-wall contact when sheared		
H	Zones containing clay minerals thick enough to prevent rock-wall contact	1	
J	Sandy, gravely or crushed zone thick enough to prevent rock-wall contact	1	

Table 2.4. Ja input parameter of the Q system and its values (Barton et al., 1974).

	Definition	Values		Explanation
	a) Rock-Wall Contact	**Ja**	**~ϕr**	
A	Tightly healed, hard non-softening, impermeable filling(e.g., quartz or epidote)	0.75	-	*ϕr values are given to approximately present the mineralogical properties of alteration products if any.
B	Unaltered joint walls, surface staining only	1	25 – 35	
C	Slightly altered joint walls, non-softening mineral coatings, clay-free disintegrated rock	2	25 – 35	
D	Silty- or sandy-clay coatings, small clay disintegrated rock	3	20 – 25	
E	Softening or low-friction clay mineral coatings (e.g., kaolinite, mica), also chlorite, talc, gypsum, graphite, or small quantities of swelling clays (discontinuous coatings at thicknesses of 1-2 mm or less)	4	8 – 16	
F	Sandy particles, clay-free disintegrated rock	4	25 – 30	
G	Strongly over-consolidated non-softening clay mineral fillings (continuous, less than 5 mm in thickness)	6	16 – 24	
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, less than 5 mm in thickness)	8	12 – 16	
J	Swelling-clay fillings (continuous, less than 5 mm in thickness). Ja varies based on the fraction of particles at swelling-clay size and the presence of water effects	8-12	6 – 12	
	b) No Rock-Wall Contact			
K	Zones or bands of crushed rock and clay	6, 8, or 8-12	6 – 24	
L	Zones or bands of silty- or sandy-clay, small non-softening clay fraction	5	-	
M	Thick, continuous zones or bands of clay	10, 13, or 13-20	6 - 24	

Table 2.5. Jw input parameter of the Q system and its values (Barton et al., 1974).

	Definition	Values: Jw	Values: Water pressure (kg/cm²)	Explanation
A	Dry excavations or inflow of <5 l/min	1	<1.0	*Factors C to F are crude estimates. Jw should be increased if the rock is drained. *Different problems can be seen in the case of ice formation. These problems are not considered here.
B	Medium inflow or pressure, occasional outwash of joint fillings	0.66	1 - 2.5	
C	Jet inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	
D	Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5 - 10.0	
E	Exceptionally high inflow or water pressure decaying with time, causes outwash of material and perhaps cave-in	0.2 - 0.1	>10.0	
F	Exceptionally high inflow or water pressure continuing without noticeable decay, causes outwash of material and perhaps cave-in	0.1 - 0.05	>10.0	

Table 2.6. SRF input parameter of the Q system and its values (Barton et al., 1974).

	Definition			SRF	Explanation
a)	**Weak zones intersecting the underground opening, which may cause loosening of rock mass**				*Reduce SRF by 25-50% if the weak zones influence but do not intersect the opening.
A	Multiple occurrences of weak zones containing clay or disintegrated rock (any depth)			10.0	
B	Single weak zones containing clay or chemically disintegrated rock (any depth)			5.0	
C	Single weak zones containing clay or chemically disintegrated rock (depth ≤50 m)			2.5	
D	Multiple shear zones in competent clay-free rock with loose surrounding rock (any depth)			7.5	
E	Single shear zones in competent clay-free rock (depth ≤50 m)			5.0	
F	Single shear zones in competent clay-free rock (depth >50 m)			2.5	
G	Loose, open joints, heavily jointed, sugar cube-shaped(any depth)			5.0	
b)	**Competent rock, rock stress problems**	**σc/σ1**	**σt/σ1**		*Increase SRF from 2.5 to 5 when the depth of the crown below the surface is less than the span.
H	Low stress, near surface	>200	>13	2.5	
J	Medium stress	200-10	13-0.66	1.0	
K	High stress (very tight structure)	10-5	0.66-0.33	0.5-2.0	
L	Moderate (solid) rock burst	5-2.5	0.33-0.16	5-10	
M	Heavy (solid) rock burst	<2.5	<0.16	10-20	
c)	**Squeezing rock: pressure dependent on the presence of water**				
N	Mild squeezing rock pressure			5-10	
O	Heavy squeezing rock pressure			10-20	
d)	**Swelling rock: chemical swelling dependent on the presence of water**				
P	Mild swelling rock pressure			5-10	
R	Heavy swelling rock pressure			10-15	

Table 2.7. Excavation support ratio (ESR) and its value (Barton et al., 1974).

	Type of Excavation	ESR Value
A	Temporary mine openings	3.0-5.0
B	Vertical shafts (circular/rectangular shafts)	2.5-2.0
C	Permanent mine openings, water tunnels for hydropower (except high-pressure penstock pipes), pilot tunnels, drifts, and headings for large openings	1.6
D	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
E	Power stations, major road and railway tunnels, civil defense chambers	1
F	Underground nuclear power stations, factories, sports and public facilities	0.8

Regarding the stability values and support requirements of underground excavations related to the Q value, Barton et al. (1974) defined the parameter they named the "Equivalent Dimension 'De' of the Excavation". This parameter is calculated using the following equation.

$$De = \frac{Diameter(height)}{ESR}$$

In this equation, ESR is a safety factor that affects the support system that is put in place for the underground excavation to stay stable. After this ESR value was proposed by Barton in 1974, it was updated by Barton and Grimstad (1994), and it is used in its updated form today. The Q system has 9 different support categories from the weakest to the strongest (Table 2.1). The support categories determined according to the rock quality designation (RQD) and the excavation support ratio (ESR) factor are as follows:

1. Unsupported
2. Spot bolting
3. Systematic bolting
4. Systematic bolting (and reinforced concrete, shotcrete, 4-10 m)
5. Fiber-reinforced shotcrete, 5-9 m
6. Fiber-reinforced shotcrete and bolting, 9-12 cm
7. Fiber-reinforced shotcrete and bolting, 12-15 cm
8. Fiber-reinforced shotcrete, 15 cm.
9. Shotcrete with cast concrete lining and bolting Concrete lining.

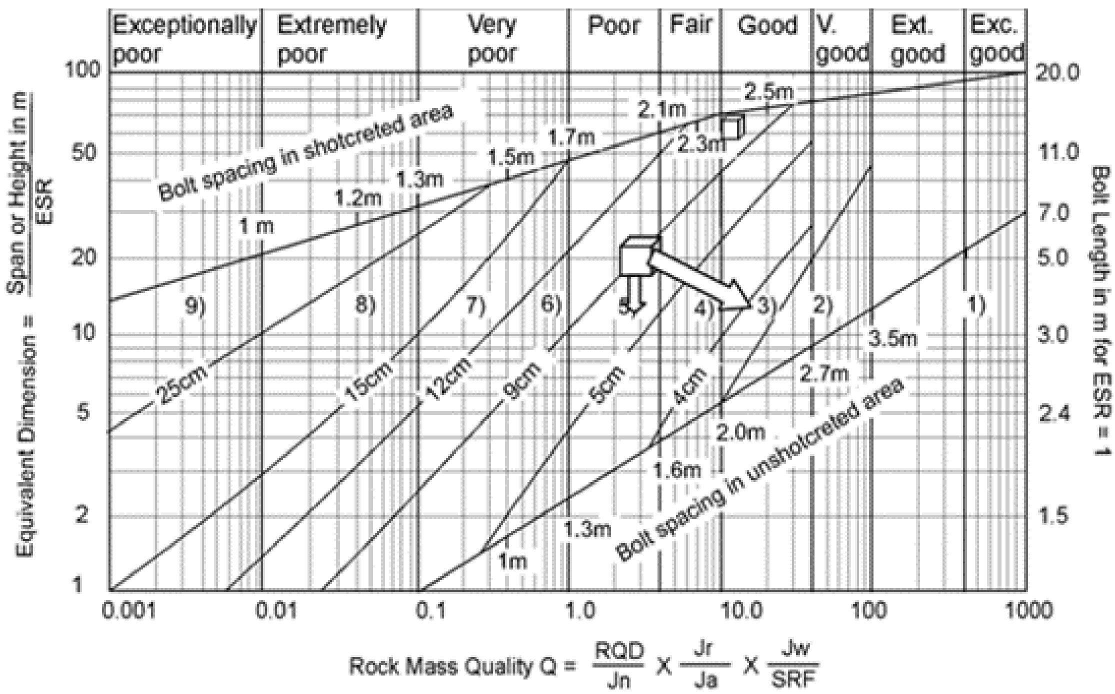


Figure 2.1. Relationship between Q, equivalent dimension (De) and rock mass classes in the Q system (Barton et al., 1974).

2.3. Rock Mass Rating (RMR) System

The RMR system is a rock mass classification system that was developed in 1973 and finalized in 1989 by Bieniawski. The system is based on investigations made in more than 350 mining tunnels and other tunnels located in South Africa and the US. The system was finalized by Bieniawski (1989) based on data compiled from a total of 351 applications for tunnels, large underground excavations, and mining locations from 1973 to 1989 and relevant experiences (R. Ulusay & H. Sönmez, 2007). Bieniawski (1988) listed the main purposes of the rock mass rating system, which is an empirical method, as follows (R. Ulusay & H. Sönmez, 2007).

- Determining the significant parameters affecting the behavior of the rock mass,
- Determining rock mass classes of different qualities by dividing the rock mass into zones that show similar characteristics amongst each other,
- Establishing baselines for understanding the properties of each rock mass class,
- Comparing experiences gained regarding the conditions of a rock mass on any site to the conditions on other sites and establishing correlations,
- Creating a numerical database and guidelines for engineering designs,
- Achieving technical communication among engineers on a common basis.

Excavation-support calculations are made easier with the plot including the unsupported stand-up time and dimensions of the span recommended by the system. The RMR system has been subjected to various changes made by many researchers from the day it was proposed until now. These changes include discontinuity adjustment, blast damage adjustment, stress adjustment, and the adjustment of planes of weakness (e.g., faults, dikes, folds). The guide for the selection of support systems based on the RMR value is given in table 3. The support system recommended in the guide is applicable to vertical stress values of <25 MPa and spans of 10 m.

Although the original form of the system is prevalently used, some researchers have suggested different adjustment requirements. While these adjustments are not frequently used in applications, they mainly include the adjustments of stress, blast damage, and planes of weakness (Ulusay & Sönmez, 2007).

Excavation geometry, rock features, and stresses are among the most significant parameters that affect stability. However, the RMR system recommends the same support system for excavations to be created at different depths, and therefore under different stress conditions, in the same rock mass. The purpose of this study is to assess the performance of the support system that is recommended by RMR in the default conditions for increased depths or varying stress conditions.

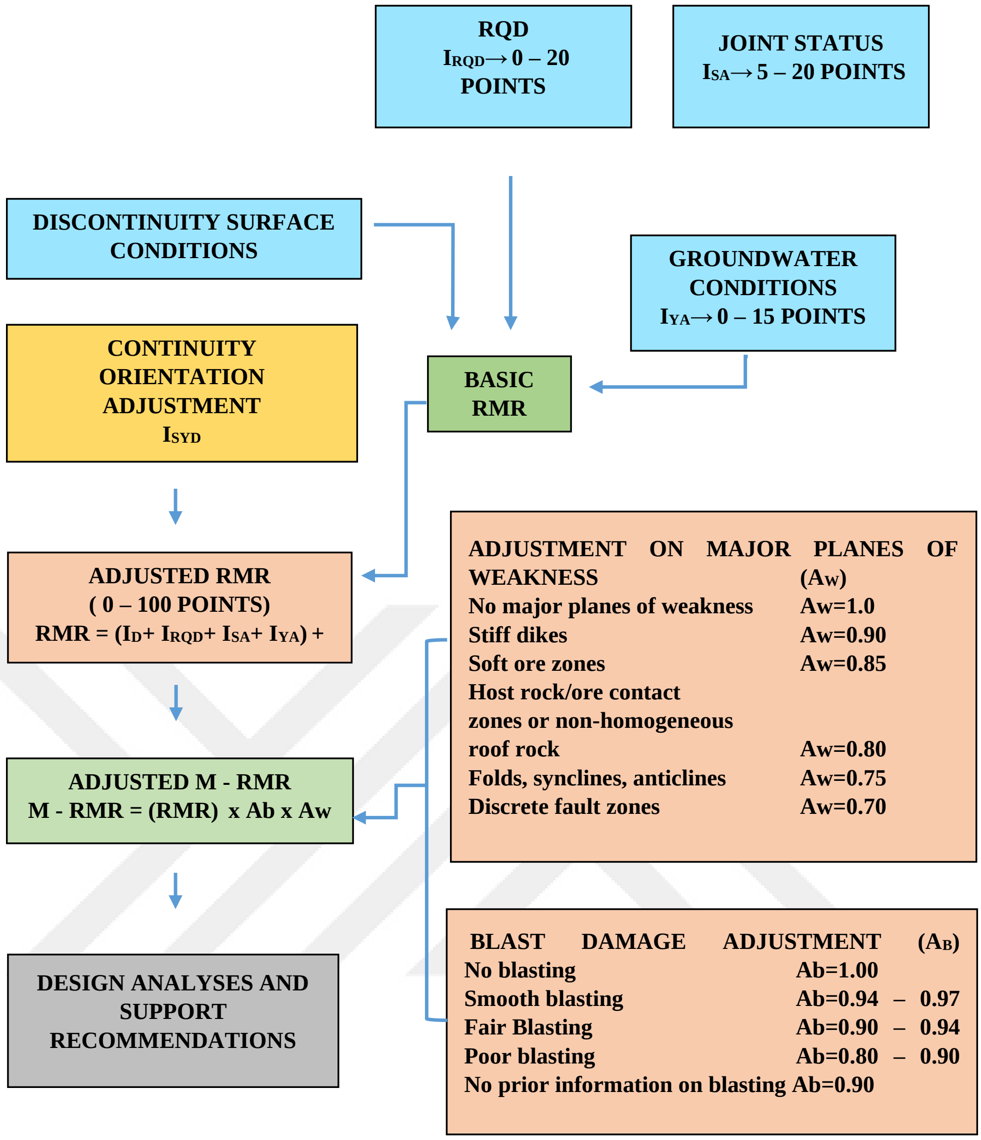


Figure 2.2. Adjusted RMR classification operating chart (Sönmez & Ulusay, 2002).

2.3.1. Initial version of the system in 1973

In the system that was proposed for the first time in 1973, the 8 parameters given below are taken into consideration, and the classification is made according to the scores given to these parameters (Ulusay & Sönmez, 2007). These parameters are:

- Uniaxial compressive strength of the rock material
- RQD (Rock quality designation)
- Conditions of weathering
- Separation of discontinuities
- Continuity of discontinuities
- Conditions of discontinuities
- Groundwater conditions

- Orientation of discontinuities.

Because the definition criteria and variation intervals that were proposed by the ISRM (1981) for the definition of the parameters mentioned above and have found a broad area of use were not present in that period, the parameters except RQD had been assessed based on the definition criteria proposed by different researchers before 1973. The criteria that used to be utilized by the system in that period to define parameters such as strength (a), discontinuity separation (b), and weathering (c) are shown in Table 2.8. Definitions and variation intervals for the other parameters were proposed by Bieniawski (1973).

Table 2.8. Criterias that used to be utilized by the system such as (a) strength, (b) spacing of discontinuities and (c) weathering (Ulusay & Sönmez, 2007).

(a) Uniaxial Compressive Strength of Intact Rock		
Definition	**Uniaxial Compressive Strength (MPa)**	
Very weak	1 – 25	
Weak	25 – 50	
Strong	50 – 100	
Very strong	100 – 200	
Extremely strong	> 200	
(b) Spacing of Discontinuities		
Definition	**Spacing**	**Rock mass class**
Very widely spaced	>3m	Solid
Widely spaced	1-3m	Massive
Medium spaced	0.3-1.0m	Blocky / seamy
Closely spaced	50-300mm	Shattered
Very closely spaced	<50mm	Disintegrated
(c) Weathering	**Definition Criteria**	
Unweathered rock	No sign of weathering in the rock. The rock is fresh, and the minerals are bright. A few discontinuities. Minimal oxidation can be observed on the surface.	
Slightly weathered rock	While there is weathering penetrating the rock through open discontinuity surfaces, the rock material is minimally weathered. There are color changes in the discontinuities, and the change progresses from the discontinuity surface into the rock by approximately 10 mm.	
Moderately weathered rock	Mild color changes are observed in a significant proportion of the rock mass. The rock material is not brittle, except for weakly cemented sedimentary rocks. The discontinuity surfaces are covered in oxidized and/or decomposed infilling material.	
Highly weathered rock	Weathering is observed in the entire rock mass, and the rock mass is partially brittle. No brightness in the rock. Except for quartz, there are color changes in the entire material. The rock is weak to an extent that it can disintegrate with the impact of a geologist's hammer.	
Decomposed	The rock has lost its original color completely, decomposed, and brittle. Particles preserving the structure and texture of the rock are observed. From the outside, it has a soil surface appearance.	

Table 2.9. Definition criteria and variation intervals of the parameters in the RMR system that were used in 1973 (Ulusay & Sönmez, 2007).

No	Parameter	1 Excellent	2 Good	3 Moderate	4 Weak	5 Very weak
1	RQD (%)	90-100	75-90	50-75	25-50	<25
2	Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
3	Rock material strength (MPa)	>2000	100-200	50-100	22-50	<25
4	Discontinuity spacing	>3 m	1-3 m	0.3-1 mm	50-300 mm	<50 mm
5	Discontinuity separation	<0.1 mm	<0.1 mm	0.1-1 mm	1-5 mm	>5 mm
6	Continuation of discontinuities	Not continuous	None	Continuous, no filling	Continuous with filling	Continuous with filling
7	Groundwater flow (at every 10 m of the excavation)	None	None	Minimal, <25 l/min	Moderate, 25-125 l/min	Excessive > 125 l/min
8	Slope and orientation	Very suitable	Suitable	Moderate	Unsuitable	Very unsuitable

Changes have been made in the system several times. Table 2.10 shows the determinant features for which different score ranges were given in these changes by years.

Table 2.10. Changes in the RMR system from 1973 to 1989

	1973	1974	1975	1976	1989
Rock strength	10	10	15	15	15
RQD	16	20	20	20	20
Discontinuity separation	30	30	30	30	20
Spacing	5	-	-	-	-
Continuation of discontinuities	5	-	-	-	-
Groundwater	10	10	10	10	15
Decomposition	9	-	-	-	-
Discontinuity conditions	-	15	30	25	30
Dip orientation	-	15	-	-	-
Dip orientation in tunnel	3-(-15)	-	0-(-12)	0-(-12)	0-(-12)

2.3.2 Final version of the system in 1989

The RMR system, which has been modified several times since 1973 when it was first proposed, took its final form with the changes made by Bieniawski (1989), and it has been used in this form since then. The main changes made in the system and new aspects brought later may be listed as follows (Ulusay & Sönmez, 2007):

- Direct determination of scores for the rock mass strength, RQD, and continuation of discontinuities parameters with the newly developed "parameter-score" plots in addition to the intervals given in the classification parameters table 5 for more sensitive classification,

- More sensitive classification by the grouping of parameters for the conditions of discontinuities including the continuity, spacing, and separation of discontinuities, as well as weathering, based on the definitions proposed by the ISRM (1981) and the assignment of separate scores for each group,

- Proposal of a set of adjustment factors for the additional consideration of the effects of blast damage, fault proximity, and stress changes on RMR, especially in tunnels excavated for underground mining operations,

- Rearrangement of the unsupported stand-up time plot (Figure1) based on long-term observations and measurements.

2.4. Strength of the Rock Materials

The strength of the rock material is used as input for many classification methods. Uniaxial compressive strength is obtained by conducting experiments on samples that are prepared using borehole samples with a height/diameter ratio between 2.5 and 3 whose top and bottom surfaces are smoothened [ISRM, 1981]. In situations where experiments are not possible, the point load strength index can also be utilized. Figure-2.4. presents the classification and relevant definitions (D. U. Deere, R. P. Miller, 1966). For the easier calculation of the rock material strength score, Bieniawski (1989) proposed the plot given in Figure-2.3.

Table 2.11. Classification based on uniaxial compressive strength and point load strength (Ulusay & Sönmez, 2007).

Class	Uniaxial compressive strength (MPa)	Definition	Point load strength (MPa)
A	>250	Extremely strong	> 10
B	100-250	Very strong	4 – 10
C	50-100	Moderately strong	2 – 4
D	25-50	Low strength	1 – 2
E	< 25	Very low strength	< 1

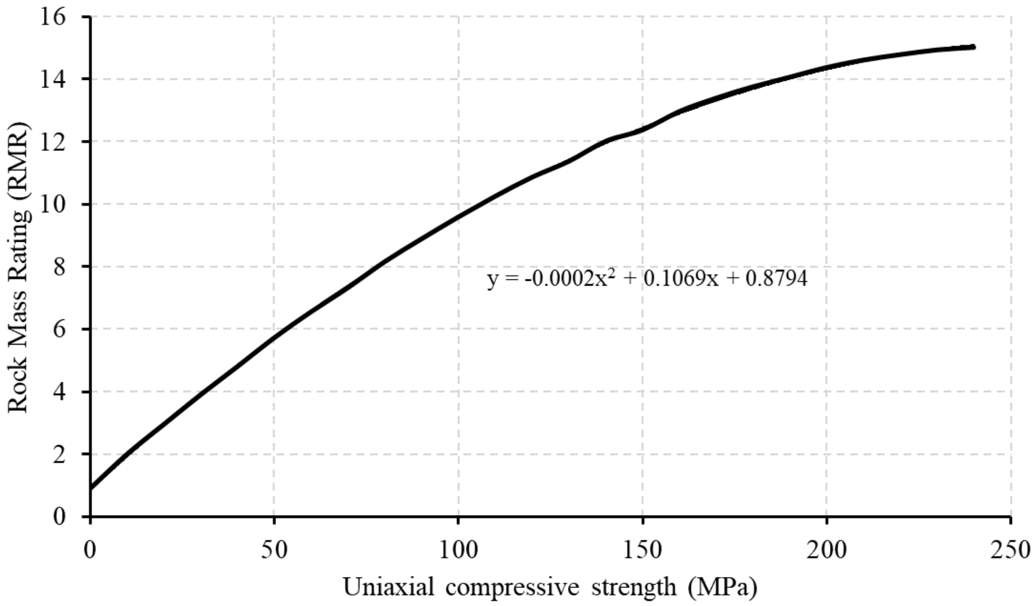


Figure 2.3. Plot of uniaxial compressive strength (MPa) versus discontinuity separation (mm) plot (Z. T. Bieniawski, 1989).

2.5. Rock Quality Designation (RQD)

RQD is an input that is frequently used in rock mass definitions, and it was developed first by Deere (1964). RQD (%) is defined as the ratio of the total length of the sound cores extracted from a borehole with a length of more than 10 cm each to the total core run (drilling) length (Figure 2.4).

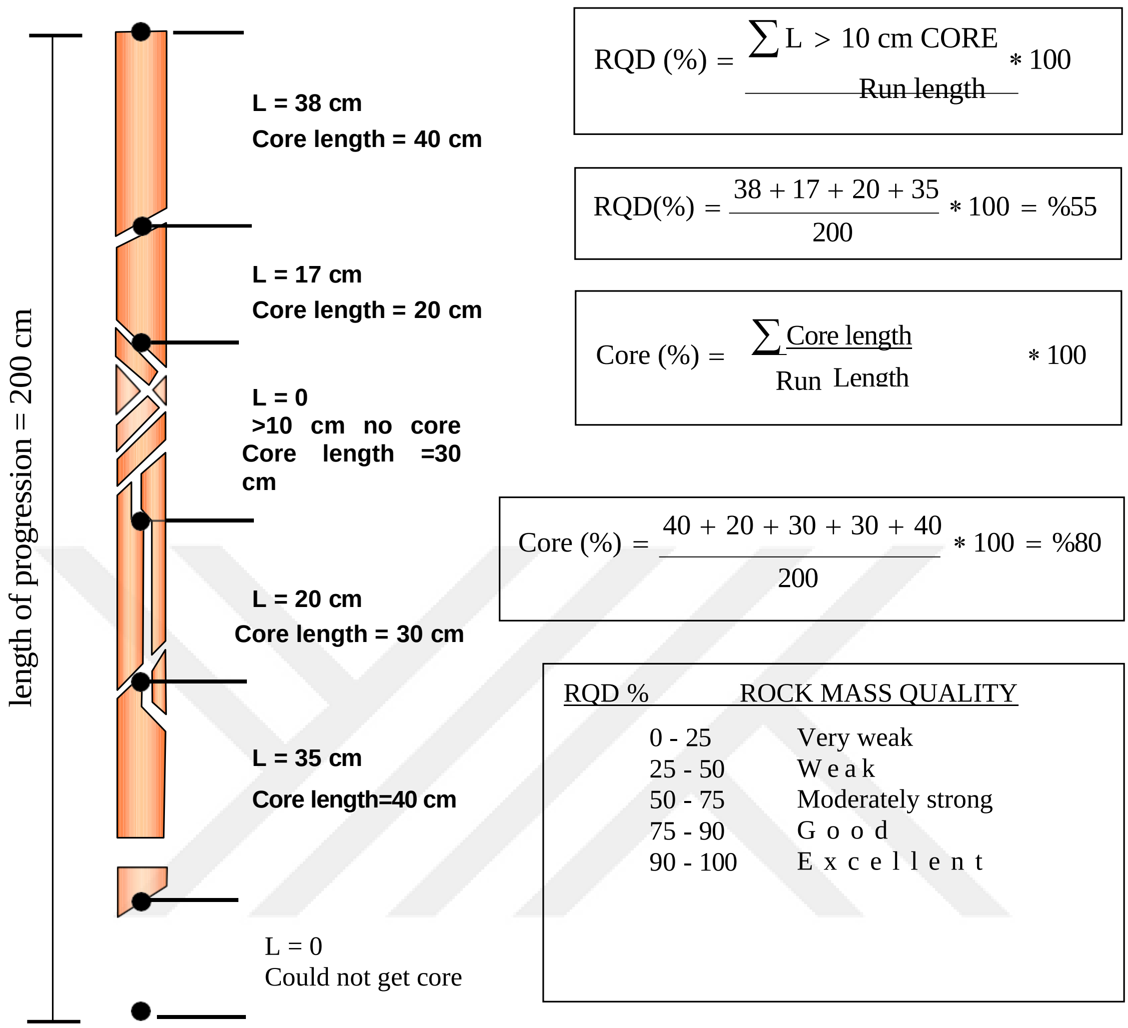


Figure 2.4. Calculation of the rock quality designation value (H. Başarır & M. Karakuş, 2006).

The RQD value that is needed for calculating the RMR value is easily obtained from the “rock quality designation – score” plot created by Bieniawski (1989) (see Figure 2.5).

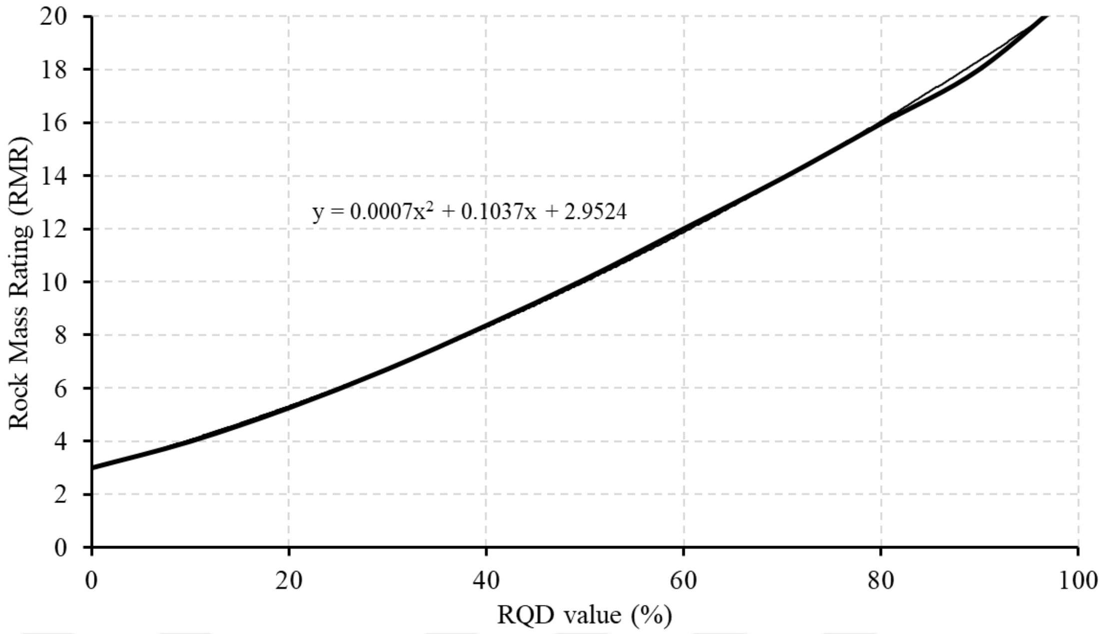


Figure 2.5. Plot of rock quality designation versus RMR (Z. T. Bieniawski, 1989).

2.6. Spacing of Discontinuities

It is the mean spacing of discontinuities such as faults, joints, and dikes. Bieniawski created the plot shown in Figure 2.6 for easier calculation. The higher this separation, as shown in the plot, the higher the rock mass.

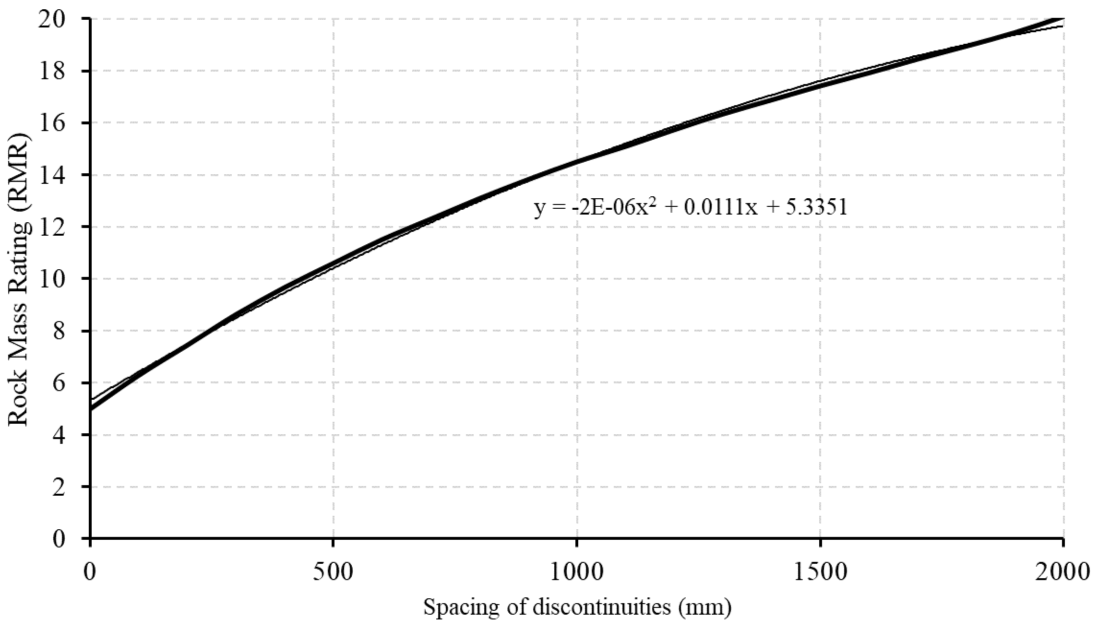


Figure 2.6. Plot of separation of discontinuities versus RMR (Z. T. Bieniawski, 1989).

2.6.1. Factor of surface discontinuity

Although this factor is considered a uniform structure for the same rock type, rock masses cannot preserve their uniformity even within the same rock type due to structural causes such as changes in the continuity of discontinuities and weathering, as well as the presence of discontinuities like faults, shear zones, and dikes in the structure. Therefore, it is important to identify these discontinuities and their shapes inside the rock mass (Figure 2.7).

Parameters such as the continuation, separation, and roughness of discontinuities, as well as the degree of filling and weathering, that are determined are grouped based on the definitions recommended by the ISRM (1981), each group is given a separate score, and these scores are included in the RMR system.

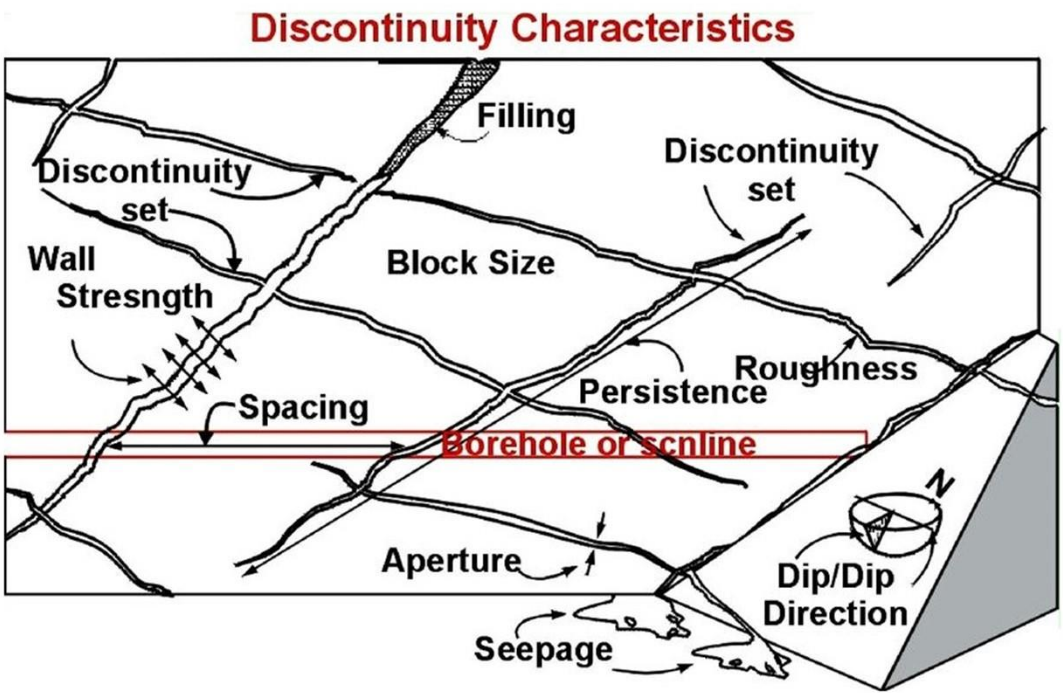


Figure 2.7. Discontinuity conditions (J. A. Hudson, 1989).

2.6.2. Effect of water on rock mass

The flow of water in rock masses occurs by its passage between interconnected discontinuities (Ulusay & Sönmez, 2007). The amount of groundwater increases depending on the faulting and joints in the rock mass. Figure 2.8 shows the flow status of water in these discontinuities and its potential effects (e.g., pressure). In the RMR system, groundwater conditions are scored based on the amount of flow of water. How these 5 parameters that are found in the latest version of the RMR system are scored is shown in Table 2.12.

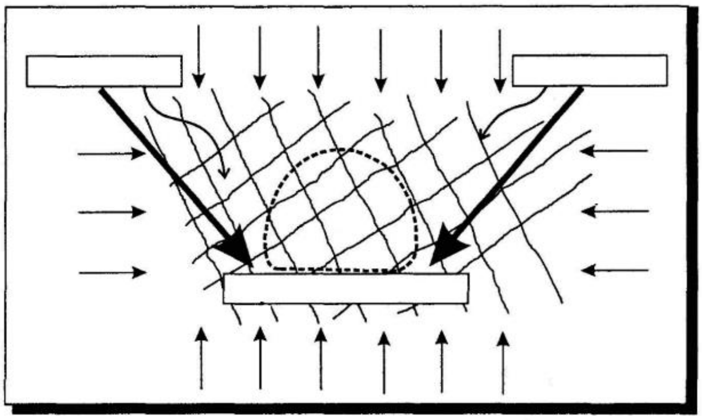


Figure 2.8. Water flow along discontinuities in rock masses and its potential effects (J. A. Hudson, 1989).

Table 2.12. Latest version of the RMR system (Z. T. Bieniawski, 1989).

Parameter			Value range						
1 (*)	Strength of Intact Rock	Point Load Strength	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	Uniaxial compressive strength is preferred in this range.		
		Uniaxial Compressive Strength	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Assessment		15	12	7	4	2	1	0

2 (*)	Borehole Core Quality (RQD)	90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%
	Assessment	20	17	13	8	3

3 (*)	Discontinuity Separation	> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm
	Assessment	20	15	10	8	5

4	Discontinuity Conditions (Found using Table 4)	Very rough, discontinuous, no separation, unweathered	Rough walls, separation < 1 mm, slightly weathered	Slightly rough, separation < 1 mm, highly weathered	Slickenslides or gouge, < 5-mm-thick or continuous separation 1-5 mm	Soft gouge, > 5-mm-thick or separation > 5 mm, continuous, decomposed wall rock
	Assessment	30	25	20	10	0

5	Groundwater	Water flow for every 10 m of tunnel (l/min)	None	< 10	10 - 25	25 – 125	> 125
		(Joint water pressure) / (Principal stress)	0	< 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5
		General Conditions	Completely Dry	Damp	Wet	Drip-leakage	Flow
	Assessment		15	10	7	4	0

The “Basic RMR” score is obtained with these 5 parameters, whereas the “Adjusted RMR” score is obtained with discontinuity orientation adjustment.

2.6.3. Discontinuity orientation

Discontinuity orientation is a parameter that is as important as the other parameters that determine the movement status of the rock mass in the excavation created underground. Figure 2.9 shows a tunnel cross-section to be opened along the horizontal axis in a rock mass with layers at a 45° angle. The score for this parameter is calculated based on whether the tunnel will progress against this slope or along this slop using Table 2.13 and Table 2.14.

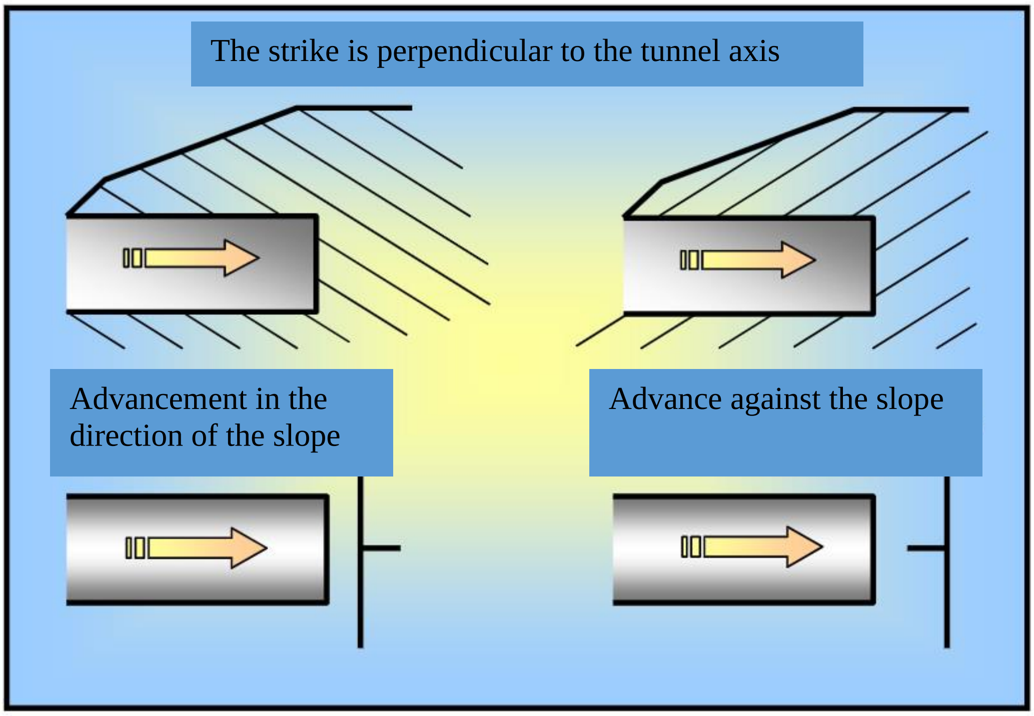


Figure 2.9. Selection of tunnel direction based on discontinuity orientation.

Table 2.13. Effects of discontinuity, slope, and orientation in tunnels (Z. T. Bieniawski, 1974 ve E. Hoek, 1995).

Orientation Perpendicular to Tunnel Axis		Orientation in Parallel with Tunnel Axis	
Progression in slope direction, dip 45 – 90°	Progression in slope direction, dip 20 – 45°	Slope 45 – 90°	Slope 20 – 45°
Very Suitable	Suitable	Very Suitable	Moderate
Progression against slope direction, slope 45 – 90°	Progression against slope direction, slope 20 – 45°	Direction-independent, slope 0 – 20°	
Moderate	Poor	Moderate	

Table 2.14. Adjustment scores based on discontinuity orientation (Z. T. Bieniawski, 1974).

Discontinuity Direction and Slope		Very suitable	Suitable	Moderate	Poor	Very poor
Assessment	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Bevels	0	-5	-25	-50	-60

For the first version of the RMR system, Bieniawski (1973) recommended caution while making classifications in the presence of shale and swelling rocks. These types of rock mass that show variations in a broad range in terms of their engineering properties, have a low resistance to weathering and disintegration, especially as a result of wetting and drying processes. This issue was stated as a limitation by Bieniawski, who developed the system. In the same period, it was recommended to include the slate durability experiments conducted by Franklin and Chandra (1972) and Gamble (1971) on samples collected from these types of rocks and the degradation classification made by Olivier (1973) as an additional criterion of classification (R. Ulusay & H. Sönmez, 2007).

2.7. Score Adjustment of RMR System

Several researchers who have used the RMR system have had a substantial role in the improvement of the system. After the system started to be used, three types of adjustment were proposed by Kendorski et al. (F. S. Kendorski, R. A. Cummings, Z. T.

Bieniawski, & E. H. Skinner, 1983). Today, these adjustments are used for the calculation of the RMR score (R. Ulusay & H. Sönmez, 2007).

2.7.1. Stress adjustment (As)

Kendorski et al. (1983) proposed stress adjustment based on horizontal and vertical stresses underground. The stress adjustment coefficient is found with the help of the curves shown in Figure 2.10 by looking at the relationships between the horizontal and vertical stresses that are calculated. Because the calculation of this adjustment has a high cost, implementers usually do not include it in the calculation. Therefore, a more practical adjustment factor to be added to the calculation is needed.

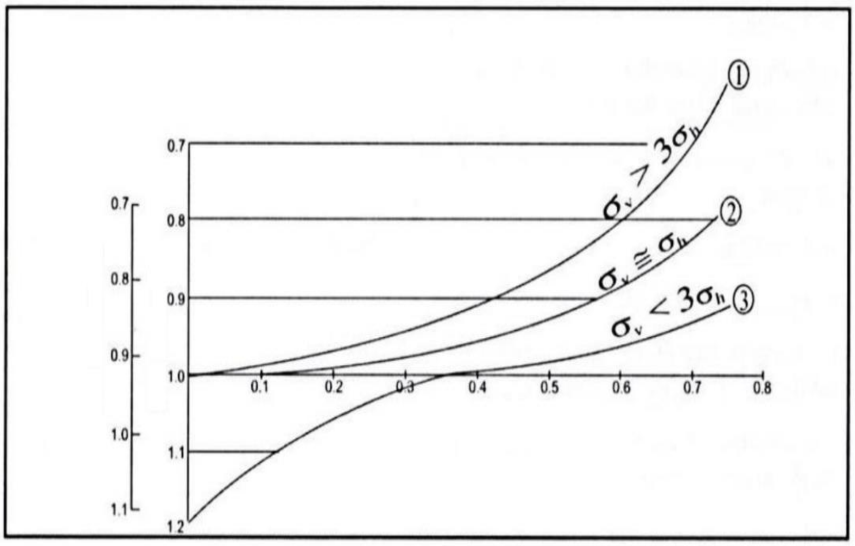


Figure 2.10. Stress adjustment (F. S. Kendorski, R. A. Cummings, Z. T. Bieniawski, & E. H. Skinner, 1983).

2.7.2. Blast damage adjustment (AB)

If blasting is going to be performed in the underground excavation, adding the blast damage adjustment (AB) factor proposed by Kendorski et al. (1983) to the adjusted RMR value will provide a more accurate RMR score. Table 2.15 shows the method for obtaining this coefficient (R. Ulusay & H. Sönmez, 2007).

Table 2.15. Blast damage adjustment (F. S. Kendorski, R. A. Cummings, Z. T. Bieniawski, & E. H. Skinner, 1983).

Blast Damage Adjustment (A_B)				
Conditions/Method			**Applicable term**	**Coefficient of adjustment, A_B**
1.	Mechanical excavation		No damage	1.0
2.	Controlled blasting		Slight damage	0.94 – 0.97
	a.	The traces of all holes can be observed in practice.		
	b.	No loosened blocks or separated discontinuities.		
	c.	Overbreak: Usually less than 15 cm, rarely between 15 and 30 cm.		
	d.	Little or no new fractures between joints.		
3.	Fair blasting		Moderate damage	0.90 – 0.94
	a.	The traces of some blastholes can be observed.		
	b.	There can be a few loose blocks, some joints can be separated.		
	c.	Overbreak: Mostly up to 30 cm, can locally exceed 30 cm.		
	d.	Hair cracks can develop in intact rock blocks and between joints.		
4.	Poor blasting		Severe damage	0.90 (best) 0.80 (worst)
	a.	Only a few blastholes can be observed.		
	b.	Several loose blocks are observed at the roof. Several joints are separated, blocks can fall.		
	c.	Overbreak: Usually larger than 30 cm, locally 1 m or larger.		
	d.	No information on blasting.		
			Moderate damage	0.90 (relative)

2.7.3. Adjustment of major planes of weakness (Aw)

Table 2.16 shows how the score of adjustment for the major planes of weakness (Aw) is found as another RMR adjustment parameter.

Table 2.16. Adjustment of planes of weakness (F. S. Kendorski, R. A. Cummings, Z. T. Bieniawski, & E. H. Skinner, 1983).

Adjustment of Major Planes of Weakness (A_W)	
Condition	**Coefficient of Adjustment, A_W**
1. No major planes of weakness	1.0
2. Stiff dikes	0.90
3. Soft ore zones	0.85
4. Host rock / ore contact zones or non-homogeneous roof rock	0.80
5. Folds (synclines and anticlines)	0.75
6. Discrete fault zones	0.70

After obtaining these 3 adjustment values, the following formula is used to calculate the “Adjusted RMR” score.

$$Adjust\ RMR\ Score = RMR\ x\ As\ x\ AB\ x\ Aw$$

As : Stress adjustment

AB : Blast damage adjustment

Aw : Adjustment of major planes of weakness

2.8. Use of the RMR System in Tunnel Support Design

The pressure that will be applied to the support systems that will be used in underground excavations can be predicted using the RMR score. As a result of their studies in coal mines, Ünal (1983) argued that the support pressure for underground excavations can be predicted using RMR. With the latest form of the expression proposed by Ünal, the following formula is obtained:

$$P = \left[\left(\frac{100 - RMR}{100}\right) x\ \gamma\ x\ B\ x\ S\right]$$

Here,

P : Support pressure (MPa)

ɣ : Unit weight of the rock (MN/m^3)

B : Tunnel diameter (m)

S : Stress factor

The concept of unsupported excavation was proposed first by Lauffer (1958). The original classification by Lauffer was rarely used in that period, but its usage has become prevalent by its inclusion in the RMR system by modifications. Figure 10 shows the relationship between the dimensions of an underground excavation and the unsupported stand-up time of the excavation based on its RMR score. The method shown in Table 2.17 which was developed by Bieniawski is used to make the RMR section demonstrated in Figure 10 more comprehensible. Using the plot in question, the application boundaries of known or selected excavation dimensions and the unsupported stand-up time of the excavation are predicted (R. Ulusay & H. Sönmez, 2007).

Table 2.17. Rock mass classes and RMR score variation intervals for each class in the 1973 version of the RMR system (Z. T. Bieniawski, 1973).

Class No.	**1**	**2**	**3**	**4**	**5**
Class definition	Very strong rock	Strong rock	Moderately strong rock	Weak rock	Very weak rock
Total RMR score	100-90	90-70	70-50	50-25	< 25

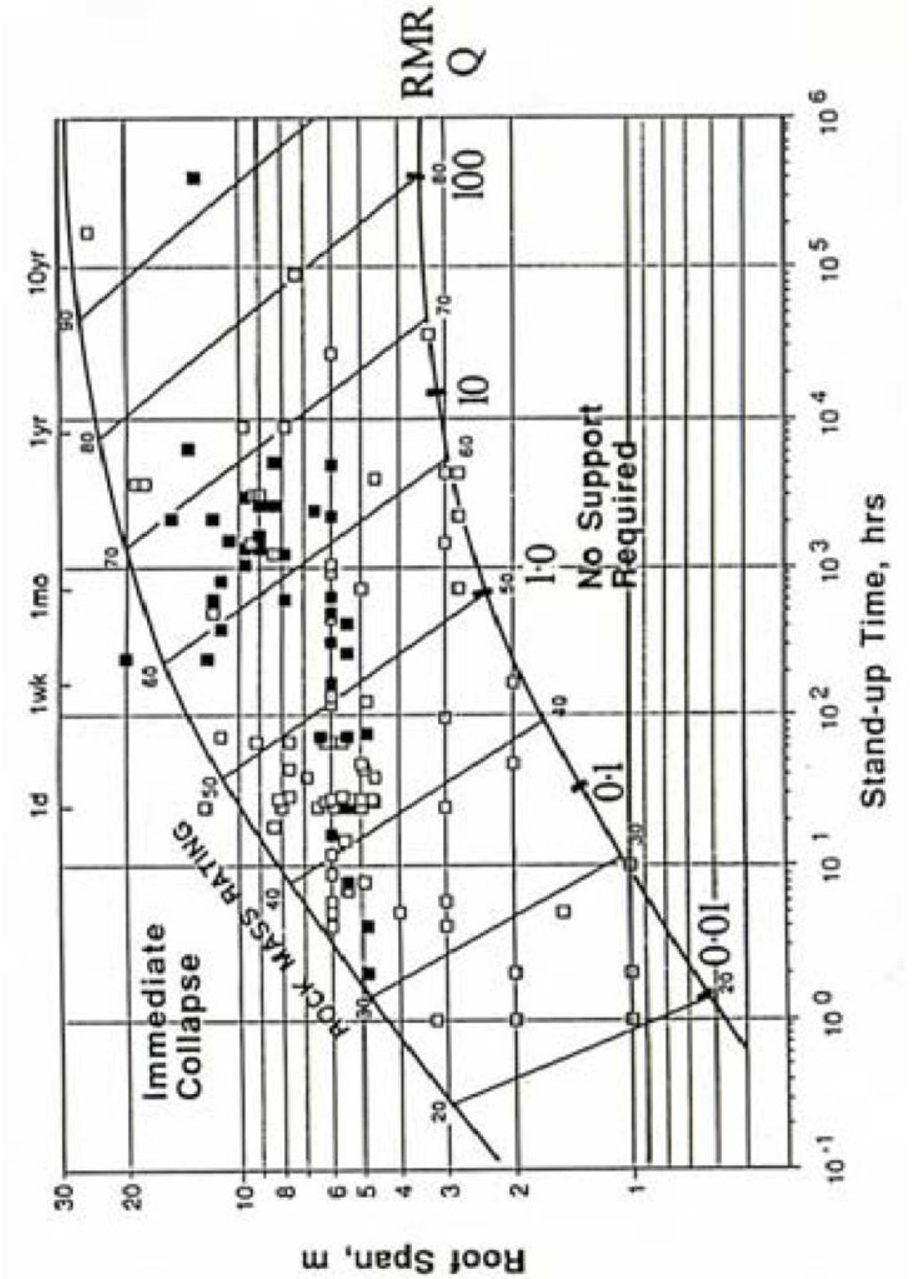


Figure 2.11. Plot showing the “unsupported excavation dimensions versus unsupported stand-up time” relationship proposed by Bieniawski (1973) (C. Karpuz & M. A. Hindistan, 2008)

Table 2.18. Guide for selecting front support in the RMR system (Z. T. Bieniawski, 1973).

		PERMANENT SUPPORT (LINING)		
ROCK MASS CLASS	**Excavation**	**Rock bolts (*) (length for tunnels with 10 m span)**	**Shotcrete**	**Steel sets**
I	Full face, 3 m advance			
II	Full face, 1.0 - 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	Top heading and bench, 1 .5 - 3 m advance from top heading, complete support, 10 m from face.	Systematic bolts 3 - 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	Top heading and bench, 1.0 - 1.5 m advance from 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	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, 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Closed invert.

- (*) 20 mm diameter, fully grouted
- Tunnel shape: Horseshoe, Construction: Drilling and blasting, Span: 10 m, Vertical stress: <25 MPa

2.9. Limitations of the RMR System

Although the RMR system is prevalently used, it has some shortcomings. Some important shortcomings of the system may be listed as follows:

- Because the RMR system is an experimental method that is mainly used in underground excavations and based on multiple observations, it can lead to erroneous assessments in terms of engineering applications and design.
- Although attempts have been made to standardize the orientations of discontinuities and their scoring schemes, these are currently limiting factors for this system.

- While the system accounts for groundwater conditions as a parameter, there is no parameter to define the negative effects of water on weak rock masses that have thin layers and contain clay.
- In the system, the strength of the rock material is also taken into account as a classification parameter, and for this purpose, it is recommended to score rock materials using uniaxial compressive strength values or the point load strength index. On the other hand, for rock material samples taken from rock masses that include planes of weakness such as laminae and schistosity surfaces that are repeated at frequent intervals, it is usually impossible to prepare samples at the dimensions required for these experiments.
- The system also falls short in evaluating rock masses that contain rock types with different properties.
- Every excavation on earth has different properties, and thus, different behaviors can be observed in different underground excavations. As the RMR system is an experimental method, some limitations can be encountered in the design to be made due to these different behaviors.
- Errors regarding the definition of parameters caused by inexperience or failure to monitor some changes in the classification system lead to negative effects on the calculation of the rock mass classification score.
- As seen in Figure 2.12, in drilling processes carried out along the route of a tunnel, the rock masses right above the tunnel's top heading constitute the most critical part in terms of the bearing of loads to be transferred from the cover during the boring of the tunnel. Therefore, the scoring process for rock masses should be followed meticulously (R. Ulusay & H. Sönmez, 2007).

2.10. New Austrian Tunneling Method (NATM)

The NATM system is a tunnel excavation method that was developed in 1964 by Prof. Rabcewicz. The method also includes a rock mass classification system. This system simply categorizes rock masses as A, B, and C, and it provides excavation and support recommendations according to these classes. The rock mass classification system in NATM has been updated several times, and today, it is used in compliance with the Austrian standard ÖNORM B2203. Table 2.19 shows the currently used rock mass classes. The NATM method divides rock masses into ten classes and makes the support recommendations that are shown in Table 2.19 according to these classes.

The NATM method first started to be used following the articles of Rabcewicz (1964) that were published in three pieces in the magazine Waterpower in November 1964, December 1964, and January 1965 (Sauer, 1990). The dual lining supports idea in NATM found by Rabcewicz is considered to be based on the theoretical works of Engesser, where allowing the rock to deform before the implementation of the final lining results in reduced loads on the lining. It was also stated that shotcrete was used for the first time by Carl E. Akeley in Chicago in 1920 to preserve dinosaur skeletons, the main contributions of Rabcewicz, Leopold Müller, and other Austrians to this method are systematic bolting and on-site measurement (Kahyaoğlu, 2008).

The patent application document for NATM prepared by Rabcewicz in 1948 includes the definition of "reducing rock pressure by placing a thin temporary support and allowing deformations, ultimately distributing this pressure to the surrounding rock" (Kahyaoğlu, 2008). This way, the final support system will be subjected to lower loads, and this will allow an even thinner support lining to be placed. In practice, the formation of deformations should be observed through deformation measurements, and the results should be assessed along with structural analyses and structural design.

In time, with the application of NATM in different countries and projects, different definitions of it have emerged. Nevertheless, the principles that are commonly accepted in all definitions can be summarized as follows:

1. The robustness of the rock mass for the tunnel should be preserved as much as feasible and maximized.

2. For the rock to reach stability completely and without a problem, deformations that will lead to loss of bearing resistance and unacceptable settling should be avoided, and deformations should be kept under control to increase the safety factor.

3. To prevent deformations, the ground or rock mass surrounding the tunnel can be allowed to deform alongside thin or thick shotcrete lining and systematic bolting elements that will contact the surrounding mass.

4. The timely application of support elements and shotcrete use is important for deformation control.

5. The initial support is created by observing the stresses in the support elements and tunnel deformations in real-time. The values obtained in the initial measurements and those obtained in later measurements are compared to identify the amounts of motion.

6. The part that is left unsupported during tunnel excavation should be minimized to a feasible extent.

7. All stakeholders involved in the design and construction of the tunnel (e.g., designers, inspection staff, contractors) should comprehend the approach and principles of NATM well and act in collaboration at the stages of decision-making and problem-solving.

As NATM is more subjective than other classification methods, the project designs of tunnels and other underground structures are made by using it in combination with systems such as RMR and Q that score rock masses.

In this method, the main purpose is to make the surrounding rock carry itself before producing the final lining in the ground and rock around tunnels by taking a set of reinforcement precautions and keep the stresses and deformations that will occur with the excavation of the tunnel at acceptable levels.

NATM, which has led to the emergence of a new approach in tunnel construction, consists entirely of practical experience. It is based on making the excavation that is created in the rock mass where the tunnel is constructed carry its own weight. In this excavation, temporary support is placed, a certain degree of deformation is allowed, and the pressure of the rock mass is distributed inside the rock. Support systems allow the balancing of the ground. To carry the ground, the other parts of the support are created using rock bolts, shotcrete, or concrete lining and invert concrete. Certain systematic measurements of the deformations are made, and the results are analyzed along with structural analyses and design.

A set of changes can be needed as a result of on-site measurements. With these changes, it is aimed to find solutions in a short time and a safe and economical manner. It is the optimum decision to combine practices that are directed by empirical methods with analytical methods.

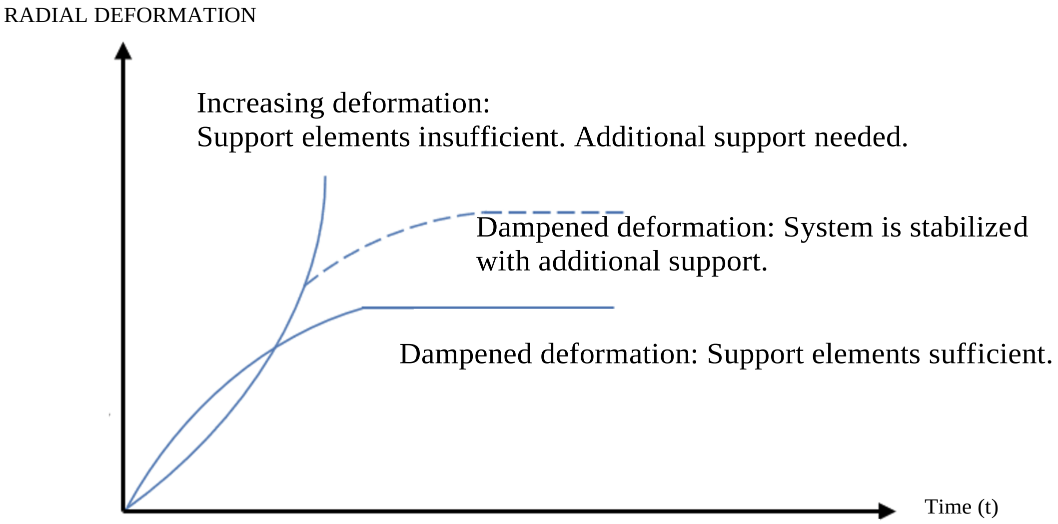


Figure 2.12. Interpretation of deformation measurements.

Table 2.19. Rock classification according to ÖNORM B2203 (1994)

Rock Class	ÖNORM B2203 October 1994 and later	ÖNORM B2203 Before October 1994
A	A1 stable rock A2 slightly overbreaking	A1 stable rock A2 slightly overbreaking
B	B1 friable B2 very friable B3 rolling (loose)	B1 friable B2 very friable
C	C1 rock bursting C2 squeezing C3 heavily squeezing C4 flowing C5 swelling	C1 squeezing C2 heavily squeezing L1 short-term stable with high cohesion L2 short-term stable with low cohesion

The A1 Rock Class is known as stable rock. Its degrees of deformation are very low. The tendency of overbreaking after the scaling of rock pieces is little to none. There is no water in this rock class, and controlled blasting application will be required for excavation.

The A2 Rock Class is also known as strong rock, while it is solid and slightly overbreaking. Its deformation amounts are low, and they decrease rapidly. Overbreaking, albeit little, can be seen in the tunnel axis parts (top heading) and side faces due to the weight of the rock mass. Groundwater conditions are negligible. Drilling-blasting is necessary for excavation. Lining may be required partially for the roof and side walls of the tunnel. Except for sites with a tendency to collapse that need to be immediately supported, supports with rock bolts will be created at most by one round after face construction. The direction of the rock bolts is determined based on the orientation of discontinuities.

In the B1 Rock Class, deformations are small and decrease rapidly. These are known as “friable” rock masses. As a result of loosening that is observed in the rock after blasting, there can be overbreaks on the roof and side walls of the tunnel. Dripping water is negligible. The round length is dependent on the unsupported stand-up time, support separation, and the placement of support elements. Blasting is required for excavation. Front support elements can usually be needed. Systematic support can be needed in local parts. Supports will be created at most by one round after face construction. Parts susceptible to failure must be immediately supported with a lining support system.

In the B2 Rock Class, the support system should be installed in a timely manner to rapidly reduce tunnel deformations. These are “very friable” rock masses, in which deep loosening and sudden failure can be encountered in the case that support systems are not installed on time or sufficiently. The inflow of water inside the separated or disintegrated rock does not have an effect on the rock’s strength. Round lengths vary based on the unsupported stand-up time and unsupported span. Blasting is preferred for excavation. Systematic lining is required for the roof and side walls of the tunnel. If needed, a forepoling system is used along the axis, and the necessary precautions are taken against the negative effects of forepoling on the rock mass.

The B3 Rock Class is defined as “rolling rock”. Overbreaks are observed even during partial excavation. It causes instability in the excavation due to insufficient binder material (cementing) and cohesion. The inflow of water has an effect on the strength of the rock mass for disintegrated and separated rock. While excavation is usually carried out by blasting, mechanical excavation methods are used in areas sensitive to vibrations. A support system is required beforehand for face progression. Local forepoling may be needed.

The C1 Rock Class refers to “slightly squeezing” rock masses. It involves the accumulation of elastic energy in the brittle rock mass with joints where initial stresses are high. With the transfer of energy, deep fractures and buckling are seen in the rock structure. Free material is likely to rupture in unsupported parts. Water inflow does not have an effect on the strength inside the rock mass. In the selection of the excavation method, smooth blasting or mechanical excavation is chosen, and the advance is made by two-stage excavation. For the support elements, in addition to wire mesh, short but narrowly spaced rock bolts are needed. Holes to reduce the pressure in the rock mass can be opened as an additional precaution. With this method, rock bursts can also be prevented.

The C2 Rock Class is described as “pressure exerting”, and it is known for behaviors that create pressure inside the rock and behaviors with deep-lying plastic zones. The deformations in the rock mass are long-lasting, moderate, and slowly calming. The speeds and effects of the deformations seen during excavation are moderate. Water leakage inside the decomposed and separated rock mass does not have a significant effect on the rock. In tunnels where the excavation is wide, multiple excavation stages are required. Blasting or mechanical excavation can be used as the excavation method. After the excavation, it is needed to clear the free material and apply shotcrete.

The C3 Rock Class is described as “highly pressure exerting”. Because there are weak zones in the C3 rock class, rapid and high-level deformations are seen, and the time these deformations take to calm down is long. As fractures are formed in deep parts, plastic displacements are seen in these parts. Water leakages or flows have effects on the rock mass. Blasting or mechanical excavation operations are needed, and these are followed by the creation of a safe zone with shotcrete and the addition of lining support systems. Staged excavations should be carried out, and after creating an outer shell with shotcrete by using forepoling and lining systems in the roof parts, temporary invert applications with bolts and injections can be required.

The C4 Rock Class is known as “flowing”. In this rock class, friction, little cohesion, and minimally plastic behaviors lead to the flow of the material in places that are left unsupported. The low cohesion brings about several stages needed during excavation. Mechanical excavation (with an excavator) is suitable as the excavation method. For the material to carry itself after the excavation, it is needed to apply shotcrete lining and forepoling. The round length must not exceed 1.5 meters.

The C5 Rock Class is known as “short-term stable with low cohesion”. With the increase in volume after the combination of salts, anhydrides, and clay minerals with swelling potential with water, the volume of the rock mass increases, and this results in shedding, breaks, and flows. The excavation method for this class involves using a tunnel excavator. After the excavation, shotcrete is used as a covering material, forepoling or sheathing with steel plates is applied in proximity to the axis, the permanent support is set, shotcrete is applied, and bolting is performed.

The final tunnel lining usually consists of shotcrete and a surface protected with a waterproof geomembrane. The felt placed between the shotcrete and the geomembrane, as well as the drainage texture cover, protect this geomembrane and allow a point of drainage. The drainage canals at the bottom end of the geomembrane collect water. In the second part of the support, before applying the final interior lining with crown concrete, it is ensure that the outer crown reaches a balance (arching). This method can also be used with other tunnel excavation methods.

NATM is actually not a rock classification system but an approach that observes the deformations of underground structures during tunnel construction and includes principles to assess the behaviors of rock masses under load. Therefore, NATM does not include precisely determined excavation and support systems. While the main principle is the constant inspection of the support and observation of rock movements for obtaining the most stable and economical support system, all tunneling and excavation methods can be used. This feature makes NATM a philosophy more than a method. The method is a novel tunneling method that is adapted especially for weak grounds where the surface is reinforced with a thin layer of shotcrete, strengthened with rock bolts, and covered with invert concrete as soon as possible.

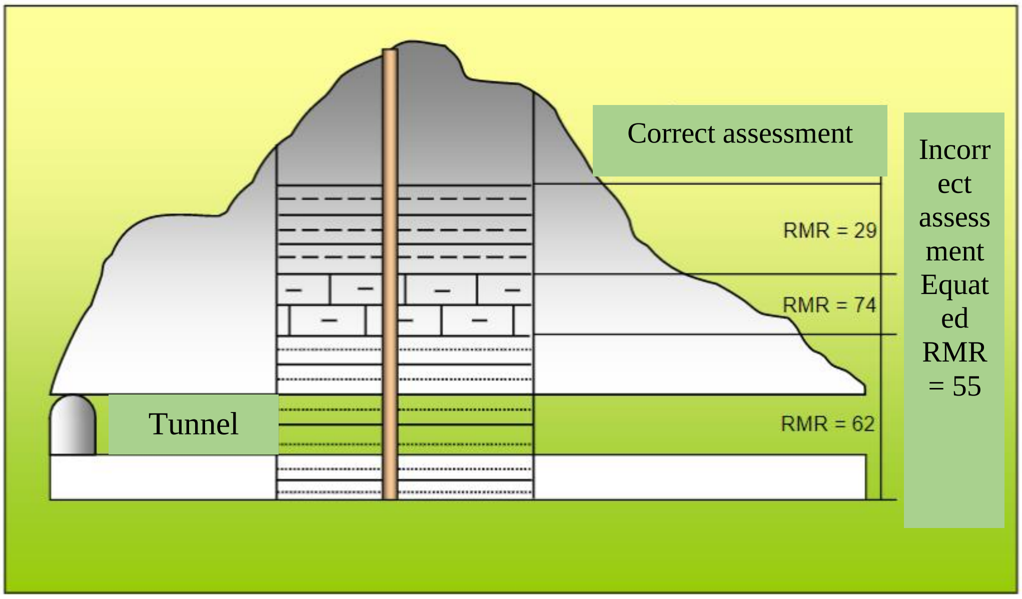


Figure 2.13. Errors made during RMR score assessment and support design.

2.11. Prediction of Rock Mass Properties

The Q, RMR, and NATM systems provide researchers with a preliminary idea by rating rock masses. This preliminary idea is not sufficient for project planning. To obtain more detailed information on the rock mass, researchers conduct laboratory and field experiments on the rock material and need to calculate values such as the modulus of elasticity (deformation), uniaxial compressive strength, angle of refraction, Poisson's ratio, and Hoek-Brown constants. As it is not always possible to conduct these experiments, for these values that are required by many researchers, approximate equations have been proposed. Thanks to these equations, using Q, RMR, and other scores, rock material properties such as the modulus of elasticity and uniaxial compressive strength can be predicted. Predicted rock mass properties constitute the input parameters that are important for numerical modeling that is utilized in project planning. Rock mass properties and numerical modeling outputs (e.g., deformation, stress, plastic zones) are used along with different equations to calculate the support pressure that is required. The prediction of these values not only prevents the occurrence of unwanted situations during project construction but also allows the implementers to increase structural safety and reduce costs by identifying the optimum quantities of support materials.

3. MATERIALS AND METHODS

This section mentions the study area's general features, geographical location, geology, and the numerical application of Phase 2D.

3.1. Location and Properties of the Tunnel Route

The route is an important part of the axis of the Mediterranean Coastal Road that is located along the East-West corridor of Turkey, and it connects the province of Mersin to the province of Antalya. The route is generally in the east-west direction parallel to the coasts of the Mediterranean.

The project includes 6 double-tube tunnels at a length of 6060 m each and 5 double-viaducts at a length of 2050 m each. The Aydıncık - Gözce Road is 17.5-km-long, and its standard is BY-BSK according to the Turkish General Directorate of Highways. With the standard improvements in the horizontal line, a 2.5-km reduction is provided in road length. With the completion of this project, the contribution of the region, which has high tourism potential, to the country’s economy will increase, and substantial savings will be achieved in terms of the shipping times and costs of agricultural products that are grown in the region and distributed to the entire country (e.g., bananas, strawberries). Moreover, it is known that due to the abundance of sharp curves on the current road, the traffic is disrupted for long durations by tractor-trailers getting stuck at times of rain. This problem will be resolved after the completion of the project. The figures below display the site location map of the T-8 Tunnel.

Figure 3.1. Site location map of the study area and its vicinity.

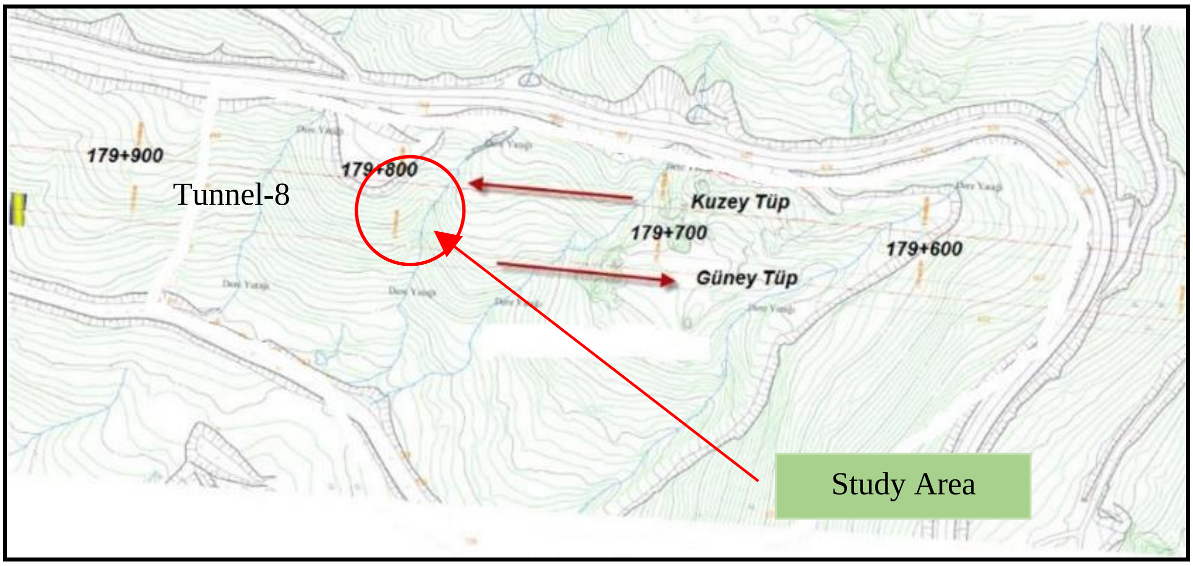


Figure 3.2. Site location of the T-8 north and south tube.

The T-8 Tunnel that is being constructed under the Aydıncık-Gözce Road project is within the borders of the 5th Regional Directorate of Highways. Since August 2012, several field works have been carried out, engineering geology studies have been conducted on the route and the tunnels, measurements of contacts between geological units, faults, weak zones, and discontinuities have been made using maps on a 1/1000 scale, and the outputs have been recorded on map sheets. Furthermore, by conducting drilling and laboratory studies in the portals and tunnels, the mechanical properties of geographical units have been determined.

In the T-8 Tunnel North and South Tubes, where modeling processes and analyses were conducted with the finite element (FE) method, projects that have been prepared for solving the stability issues encountered in these parts were examined. Examinations were made on the tunnel face during the excavation of the T-8 Tunnel that was opened based on the NATM principles, and the relevant parameters of the rock mass were identified. Using the obtained data, engineering parameter calculations were made for geological units, and the face-mapping processes conducted for the tunnel, Q and RMR rock classification results, and support elements applied to the tunnel are presented.

Table 3.1. T-8 Tunnel project features

Tunnel	Tube	Portal entrance Km.	Tunnel entrance Km.	Tunnel exit Km.	Portal exit Km.	Tube length (m)	Tunnel length (m)
T-8	South	174+043.04	174+058.1	176+330.45	176+340.4	2272.35	2297.36
	North	174+033.04	174+048.1	176+336.05	176+346.0	2287.95	2312.96

Table 3.2. T-8 Tunnel production and installation quantities

Application	Amount	Unit
Excavation	47,187.83	m^3
Shotcrete	71,733.60	m^3
Support	3,966.98	ton
Mesh	3,026.39	ton
Bolt	650,981.00	M
Forepoling	453,381.00	M
Waterproofing	121,909.83	m^2
Crown concrete	58,978.83	m^3
Grouting	14,862.74	m^3
200 mm drainage	9,271.08	M
400 mm drainage	4,569.10	M

3.2. Typical Cross-Section of the Tunnel

The T-8 Tunnel to be constructed in the scope of the Erdemli – Silifke – Taşucu 13th Zone Boundary Road Project will include two tubes with two lanes each, and the clearance dimensions of the tubes are 8.00 x 5.00 meters. The maximum excavation height of the tubes is approximately 9.5 m, while their excavation width is 12.0-12.5 m. The typical cross-section used in the scope of the project includes designs both with and without an invert.

Precast reinforced concrete elements to be placed under the pavement were used as cable and fire equipment passages. Surface water drainage was set up with grilles placed on the sides of the pavement at 50-m intervals converging at the main drainpipe with a diameter of 400 mm. Superelevation was applied by a slope of 2% for the T-8 Tunnel. C20/25 concrete was used as shotcrete, and C25/30 concrete was used for the inside lining.

For the T-8 Tunnel to be constructed within the scope of the Erdemli – Silifke – Taşucu 13th Zone Boundary Road Project, the tunnel’s typical cross-section without an invert is given in Figure 3.3, whereas the one with an invert is given in Figure 3.4.

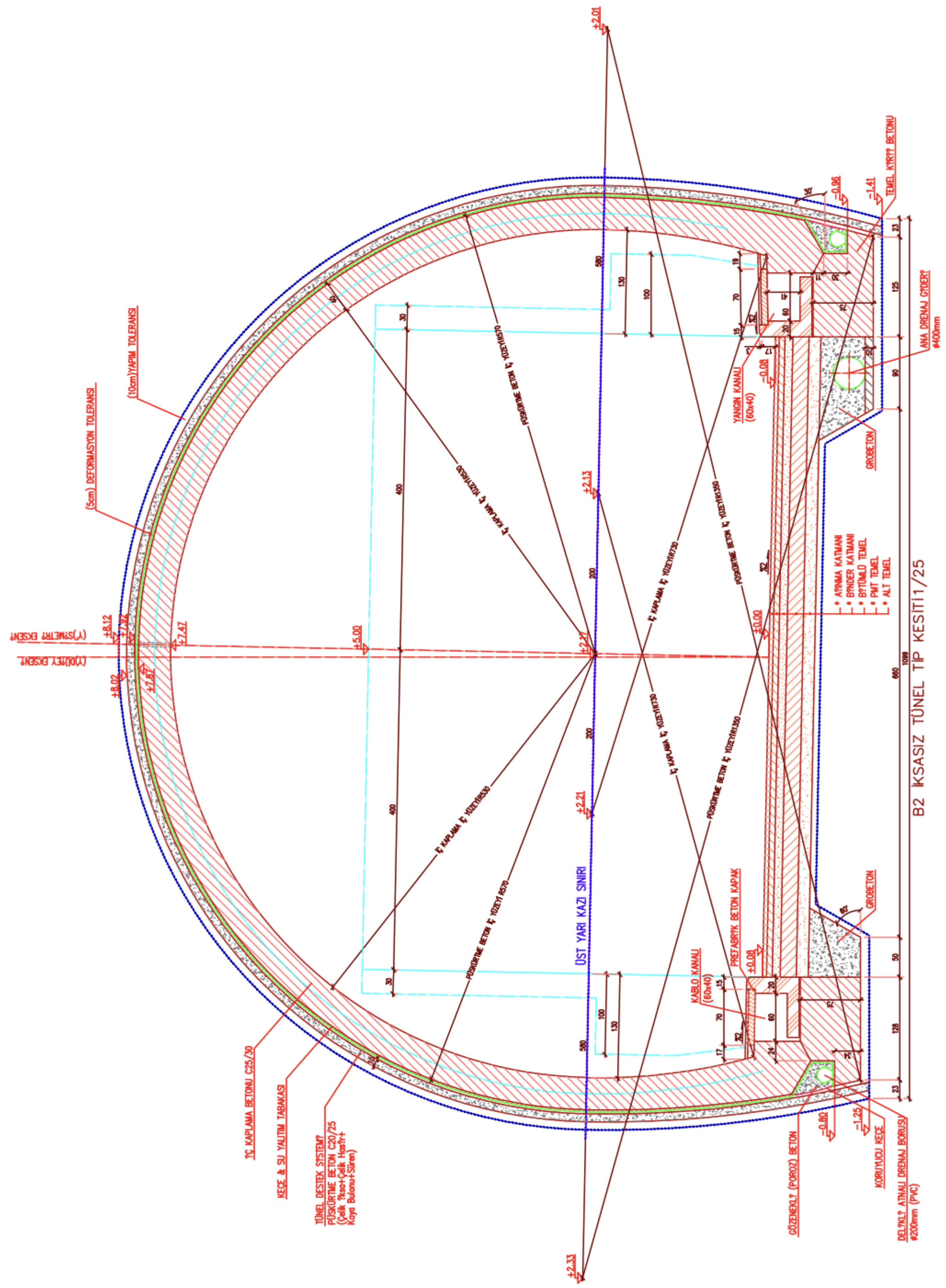


Figure 3.3. Typical tunnel cross-section without invert.

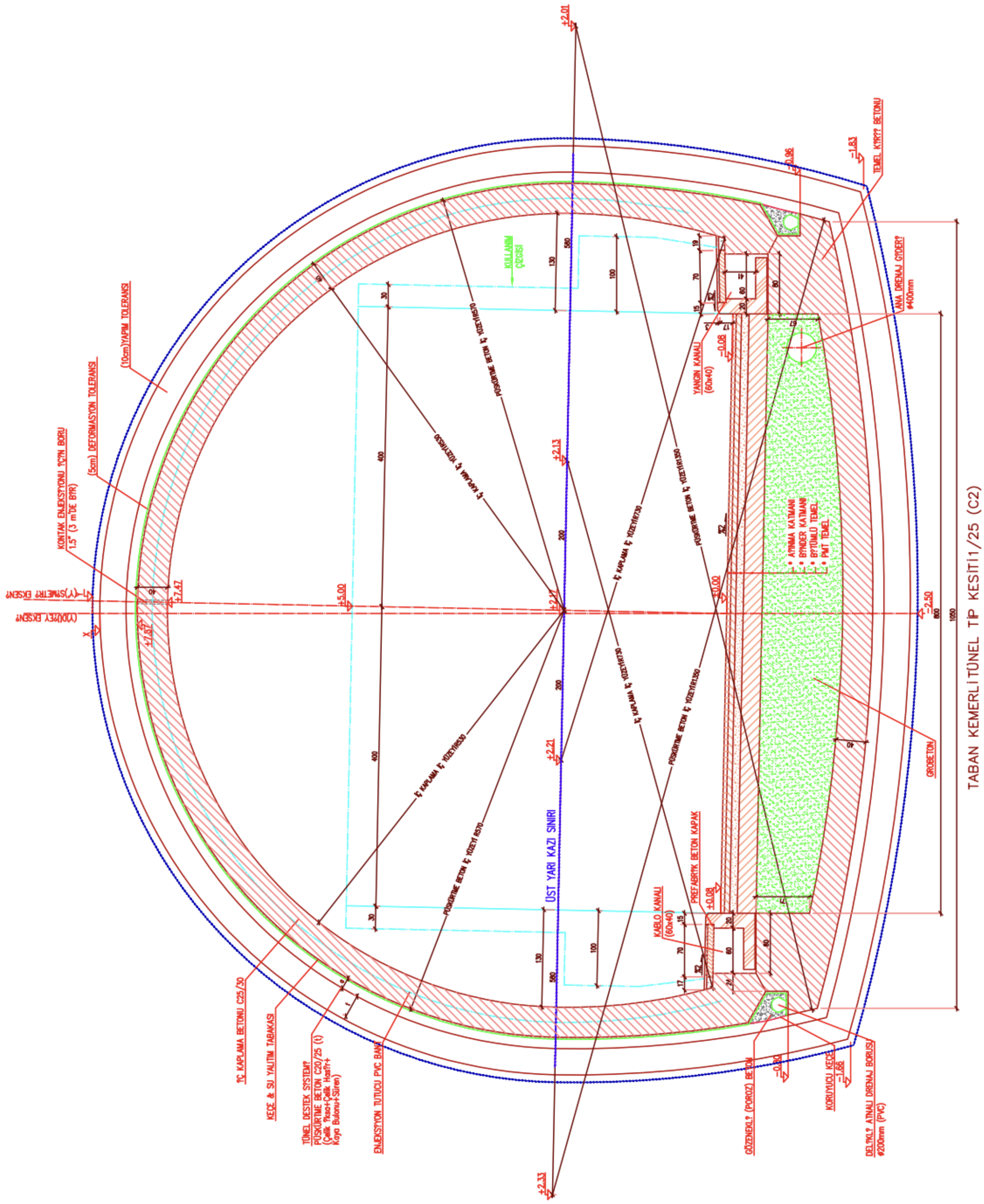


Figure 3.4. Typical tunnel cross-section with invert.

3.3. Geographical and Geological Conditions

The route of the tunnel that is still being constructed is a highly significant route because of its touristic importance, proximity to agricultural land, its connection between the Southeastern Anatolia Region and the Aegean and Mediterranean Regions, and the substantial additional length of alternative routes between these regions. Around the T-8 Tunnel, engineering geological surveys (rock quality classifications, discontinuity and layer orientation-slope measurements, roughness, weathering status, joint infilling determination) were conducted, and these data were analyzed for the tunnel faces with disrupted stability.

During the excavation of the T-8 Tunnel South Tube, a collapse occurred from the intersection of the face and the roof, also containing the face. This also led to a deformation in the part where the support had been completed. As seen in the face photographs and rock classifications given below, the units that were encountered during the tunnel excavation were identified as decomposed, brown-light brown colored, very firm, dry-minimally damp, low-plasticity, and gritty-silty-clayey units.

Figure 3.5. Photo of deformation and collapse in the T-8 north tube.

Based on the sinking and openings occurring on the topography above the tunnel, it was inferred that the weak unit spanned from the ground elevation to the base of the tunnel. It was observed that the unit, which became residual with the collapse, lost its rock properties when it contacted water. Additionally, it was determined that the deformations in the right shoulder part of the left tube reached up to a thickness of 60 cm. It was understood that these deformations occurred as a result of the stratification observed in the face from the right shoulder to the left of the tunnel at a slope of approximately 45 degrees.

Figure 3.6. Cracks on the D400 highway above the T-8 tunnel.

According to the deformation measurements that were made at Km: 175+970, deformation dimensions were 60 cm in the tunnel's right shoulder, 25 cm in the tunnel roof, and 26 cm in the left shoulder. The deformations, which progressed stably throughout the dry season, showed a sudden change with precipitation and an increase of approximately 20 cm. In the field observations of the collapsed part of the tunnel, cracks in the tunnel support elements and fatigue and failure in the rock bolts were identified. It was inferred that the collapse in the South Tube part occurred due to the low burden thickness (Figure 3.5), and the deformations at Km: 176+000-175+970 reached the elevation of the Antalya-Mersin D400 Highway and caused cracks and caved areas along the highway (see Figure 3.6).

Figure 3.7. Surface of the sinkhole that is occurred in the T-8 north tunnel.

Furthermore, it was thought that these parts, which contained the North and South Tubes of the tunnel at Km:175+550-175+880 of the project line, had low wall thicknesses which could lead to instabilities in the higher elevations in time.

The T-8 Tunnel covers the section between Km: 174+020 and 176+320 in the North and South Tubes and contains siltstone-shale units. As the burden thickness from the tunnel roof to the topography in this section was 30 meters on average, it was considered a shallow overburden tunnel.

As a result of the discontinuity measurements, 3 planes of discontinuities that could be included in a certain system were randomly determined. One of these planes was a foliation plane, and the others were joints. The layer thicknesses varied in the range of 5-25 cm, and the discontinuity separations were usually narrow (1-5 mm). They consisted of smooth, planar, and occasionally glossy surfaces. The joints had soft clay-silt infilling. The unit was identified as a shale unit with moderate-weak strength and multiple faults, in addition to occasional decomposition.

3.4. Phase-2D Software

The numerical analyses were carried out using the Phase-2D (Version 8.0) program. Phase-2D is a two-dimensional finite element (FE) analysis program that was developed by researchers at the University of Toronto and models rock masses and the supported behaviors of these rock masses. In the program, an underground excavation can be modeled in stages, and support can be applied using bolts, steel sets, wire mesh, and shotcrete. Moreover, during the excavation modeling process, load split and material softening effects can be applied to the model.

Decisions that are based on practice and experiences constitute the foundation of support systems, and numerical analyses are considered guidelines for practical decisions. Support systems may need to be revised based on actual situations to be encountered in the field and the results of geological mapping and measurement operations.

It is known that in underground excavations, the appropriate modeling of the ground material is very difficult due to the abundance and complexity of uncertainties. Thus, a detailed modeling process that takes into account all possible conditions is neither possible nor useful.

A model can be simplified using the following conditions:

- Reduction of three-dimensional conditions to two dimensions,
- Assumption of the symmetry of the cross-section on the axis,
- Simplification of the ground material with basic descriptions,
- Concise and comprehensible definition of tunnel progression and excavation conditions,
- Assumption that the ground is homogeneous and isotropic.

3.5. Numerical Analyses in the T-8 South and North Tubes

The numerical analyses were carried out based on elastic and plastic solutions, and the results are presented in this section. By using load split and material softening effects in the Phase-2D program, the NATM principles are completely reflected in the calculations.

The main philosophy of NATM is the consideration of the ground around the tunnel as a load-bearing element. Therefore, while following the modeling steps, it was assumed that the load coming to the tunnel at the moment of excavation was not equal to the load that will come from the entire burden thickness above the tunnel.

Table 3.3. Phase-2D model and material parameters prepared for the analyses.

Material Name	Color	Initial Element Loading	Unit Weight (MN/m3)	Elastic Type	Young's Modulus (MPa)	Poisson's Raito	Failure Criterion	Material Type	Tensile Strenght (MPa)	Dilation Angle (deg)	Friction Angle (peak) (deg)	Cohesion (peak) (MPa)	Incact Compressive Strenght (MPa)	mb (peak)	s (peak)	a (peak)
Rezidüel Zemin		Field Stress and Body Forece	0,016	Isotropic	200	0,3	Mohr Coulomb	Plastic	0	0	25	0,25				
Ayrışmış Şeyl (Yamaç Molozu)		Field Stress and Body Forece	0,018	Isotropic	250	0,3	Mohr Coulomb	Plastic	0	0	25	0,075				
Rezidüel Zemin Ei %10 azaltma		Field Stress and Body Forece	0,016	Isotropic	180	0,3	Mohr Coulomb	Plastic	0	0	25	0,05				
Süren		Field Stress and Body Forece	0,027	Isotropic	8241	0,31	Generalized Hoek-Brown	Plastic					57	1,03874	0,0753	0,503773
Beton		Field Stress and Body Forece	0,027	Isotropic	20000	0,3	Mohr Coulomb	Elastic	0		35	10,5				
Şeyl-Silttaşı		Field Stress and Body Forece	0,027	Isotropic	347,2	0,23	Generalized Hoek-Brown	Plastic					23	1,05587	0,00127	0,511368
Ayrışmış Şeyl (Yamaç Molozu) Ei %10 azaltma		Field Stress and Body Forece	0,018	Isotropic	225	0,3	Mohr Coulomb	Plastic	0	0	25	0,075				

After this first assumption, considering that the support would be reinforced with a self-piercing two-row forepoling system at a length of 9 meters with 6 meters of overlay at 135 degrees in the first upper half around the T-8 South Tube, the parameters of this section were augmented. After this, shotcrete and bolts were added to the model. Shotcrete floor invert was applied to the upper half. At other grades, the model included respectively lower half excavation and support systems, followed by invert excavation and support systems.

In the model, the shotcrete lining was added as a composite liner in two stages. In the first stage, the shotcrete was set at a thickness of 35 cm with I200 steel ribs. The second stage shotcrete was included in the model at a thickness of 5 cm with wire mesh. These two stages of shotcrete application were defined as composite liner in the model.

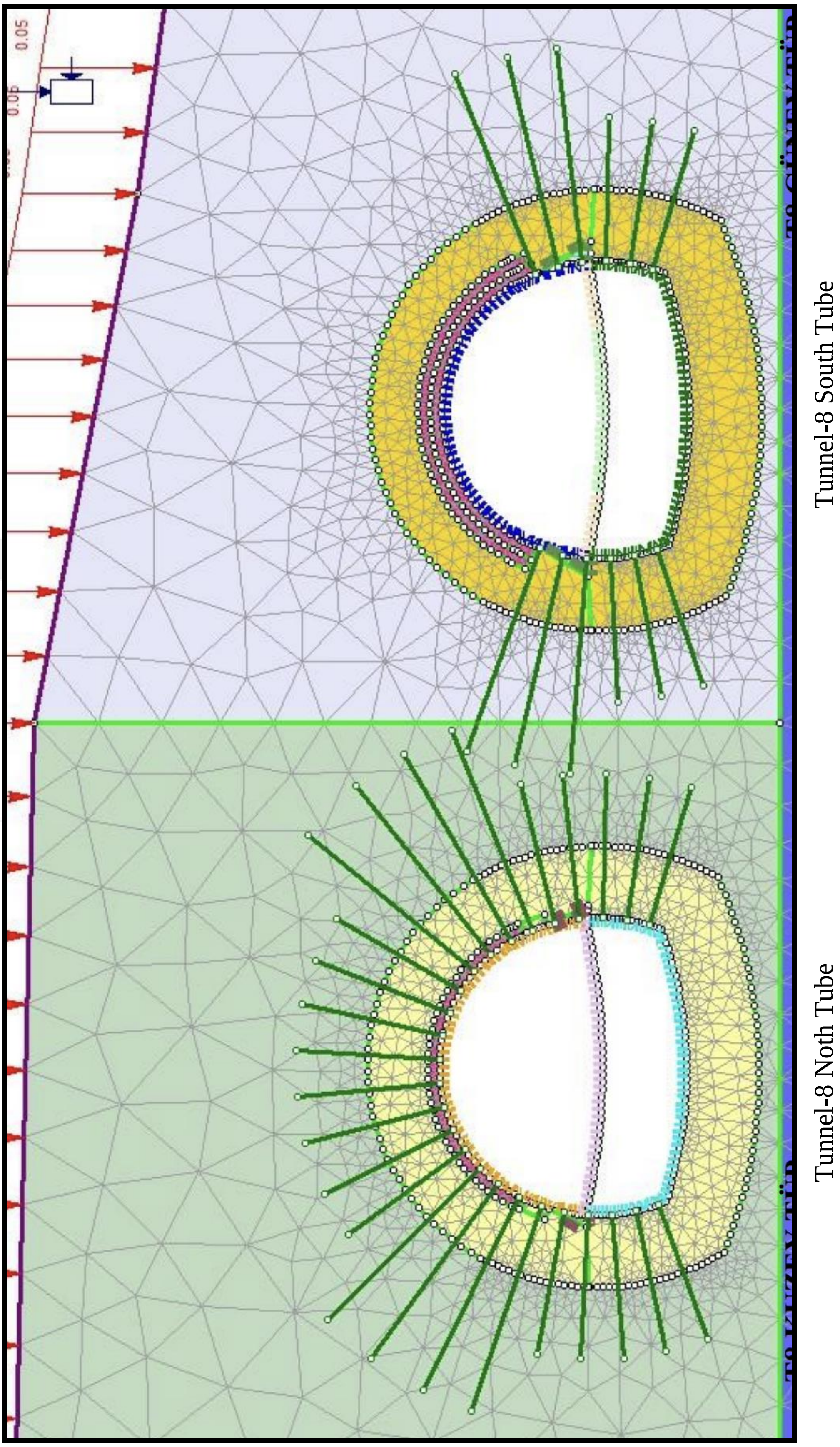


Figure 3.8. North and south tube modeling (Phase-2D ver.8.0).

Table 3.4. T-8 South Tube support elements and quantities

Support Elements and Quantities		KTŞ C2-Specific Type-2
Shotcrete	Concrete Class	C25/30
	Thickness (cm)	40
Wire Mesh	Quantity	2
	Type	Q589/378
Forepoling	Length	9 m
	Spacing	20 cm
	Overlay	6 m
	Diameter	3.5”
	Type	Two-Row Self-Piercing
Lining	Type	Cage Shoring (Φ32 reinforcement) or IPN 200
Rock Bolts	Type	IBO
	Diameter	32
	Length (m)	6-9 (6 m for upper half, 9 m for lower half)
	Spacing (radial)-Longitudinal	1.10-0.50
Round Length	Upper half (m)	0.50
	Lower half (m)	1.50-2.00
Tolerances	Deformation (cm)	25
	Construction (cm)	10

Table 3.5. T-8 North Tube support elements and quantities

Support Elements and Quantities		KTŞ C2-Specific Permanent	KTŞ C2-Specific Temporary
Shotcrete	Concrete Class	C25/30	C25/30
	Thickness (cm)	40	40
Wire Mesh	Quantity	2	2
	Type	Q589/378	Q589/378
Forepoling	Length	9 m	
	Spacing	20 cm	
	Overlay	4-5 m	
	Diameter	3.5”	
	Type	Self-Piercing	
Lining	Type	Cage Shoring (Φ32 reinforcement) or IPN 200	
Rock Bolts	Type	IBO	
	Diameter	32	
	Length (m)	6-9	
	Spacing (radial)-Longitudinal	1.10	
Round Length	Upper half (m)	0.75-1.00	
	Lower half (m)	1.50-2.00	
Tolerances	Deformation (cm)	25	
	Construction (cm)	10	

After the face excavation, the displacements around the tunnel were found approximately as 1 cm around the tunnel’s upper half, 2 cm around its lower half, and 4 cm around the base. The strength factor around the tunnel was above 1, and it would increase farther from around the tunnel. Moreover, it was seen that the fault zones around the tunnel completely covered its surroundings.

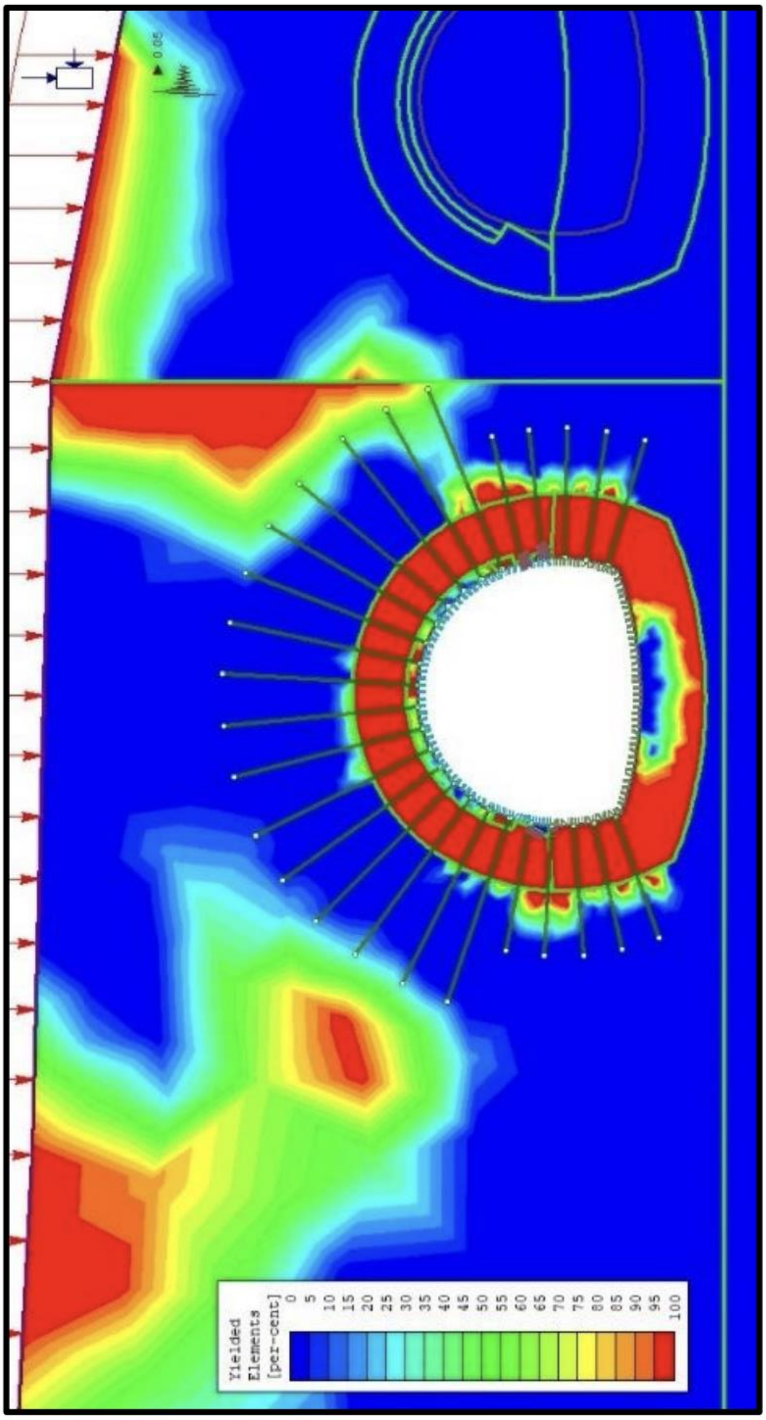


Figure 3.9. North and south tube plasticization zones (Phase-2D ver. 8.0).

The loads on the bolts were approximately 240 kN, and the bolts were loaded within their limits.

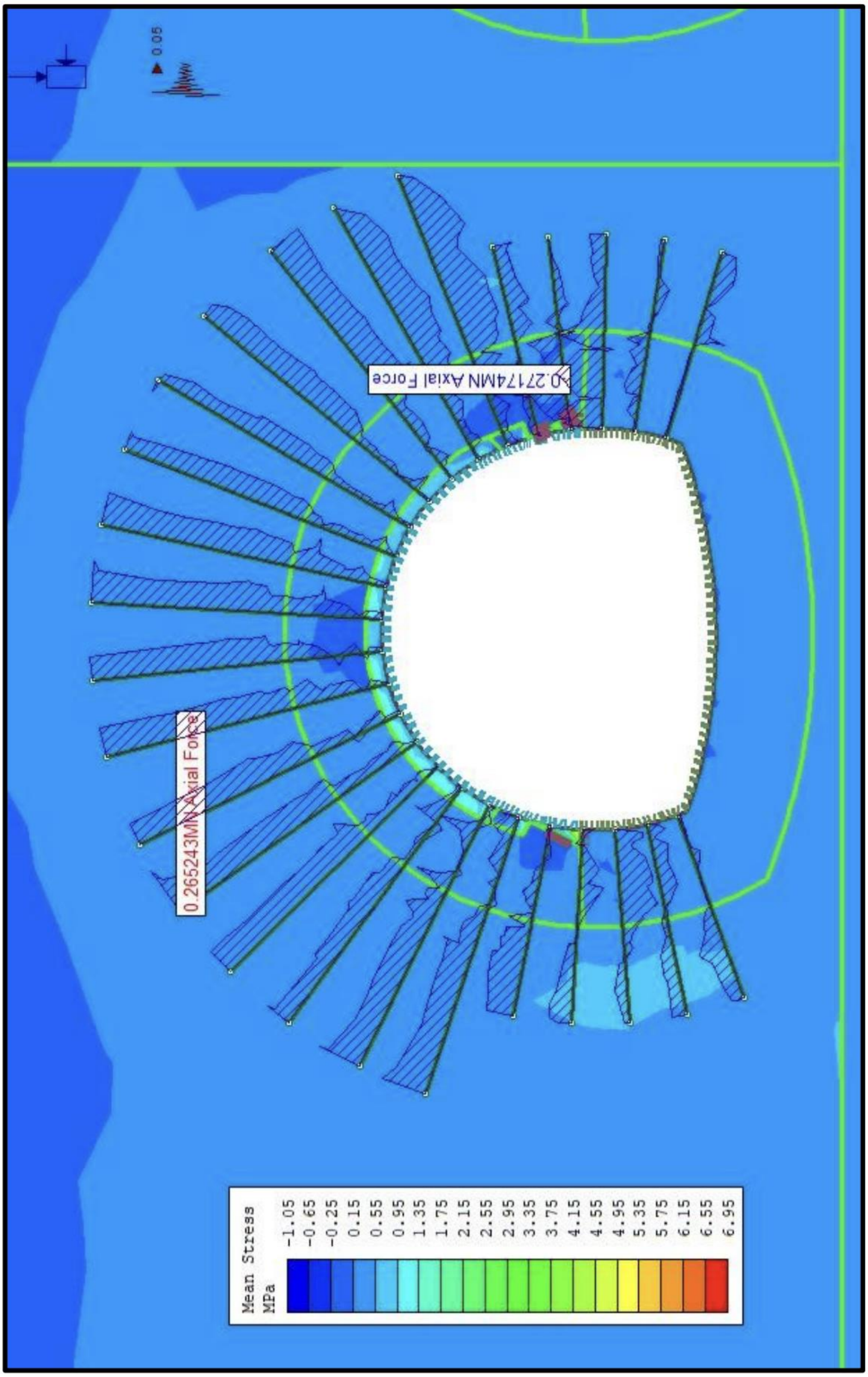


Figure 3.10. Forces acting on the bolts.

To support the tunnel face wedge during excavation (IBO Q32 L=12 meters (1.00x1.00 grid), support was provided with two-row wire mesh with 9 meters of overlay (Q221/221, 10 cm shotcrete) and temporary invert (40 cm shotcrete, 2 layers of wire mesh (Q589/378)).

With the change made in the application angle of forepoling based on the results of the analysis, improvements were seen in the deformations. Furthermore, occasional failures and strains were determined in the shotcrete implementations, and it was considered appropriate and safe for this section to have reinforcement in the lining concrete recommended in the South Tube.

3.6. RMR and Q Classification of T-8 South Tube

The engineering parameters to be used in the tunnel calculations were determined based on (Bieniawski), and the rock mass classifications for the T-8 South and T-8 North Tubes were calculated separately.

From the tunnel roof to the topography in this section, the mean burden thickness was 9 meters. Layer measurements were made on the units along the existing road, and these measurements were analyzed for the tunnel. The layers included 3 discontinuity planes, and the layer thicknesses varied in the range of 5-10 cm. The unit was very weak-weak, shattered-disintegrated, and moderately-highly weathered.

a) Rock Quality Designation (RQD)

The RQD values to be used in the calculations were obtained from the measurements that were made during tunnel face excavations. The mean RQD value was found 10, and the <25% (3 points) range was selected for the RMR calculations.

b) Rock Strength

After the uniaxial compressive strength and point load strength tests that were made on the borehole samples obtained in the drilling works for the tunnel, the mean strength value for this section was found as 9 MPa, and the 5-25 (2 points) range was selected for the RMR calculations.

c) Discontinuity orientation

Because the orientation of the sandstone-shale unit that was measured at the tunnel face was in parallel with the tunnel axis and at an angle of 65°, the condition of "unsuitable" was accepted.

AYDINCIK-GÖZCE SECTION OF ERDEMLİ-SİLİFKE-TAŞUCU 13TH ZN-BOUND KM:158+779.790 - 177+664.477

T8 TUNNEL GEOLOGICAL FACE MAP AND ROCK CLASSIFICATION
SOUTH (LEFT) TUBE EXIT
(Prj. Km. 174+048 - 176+320.60)

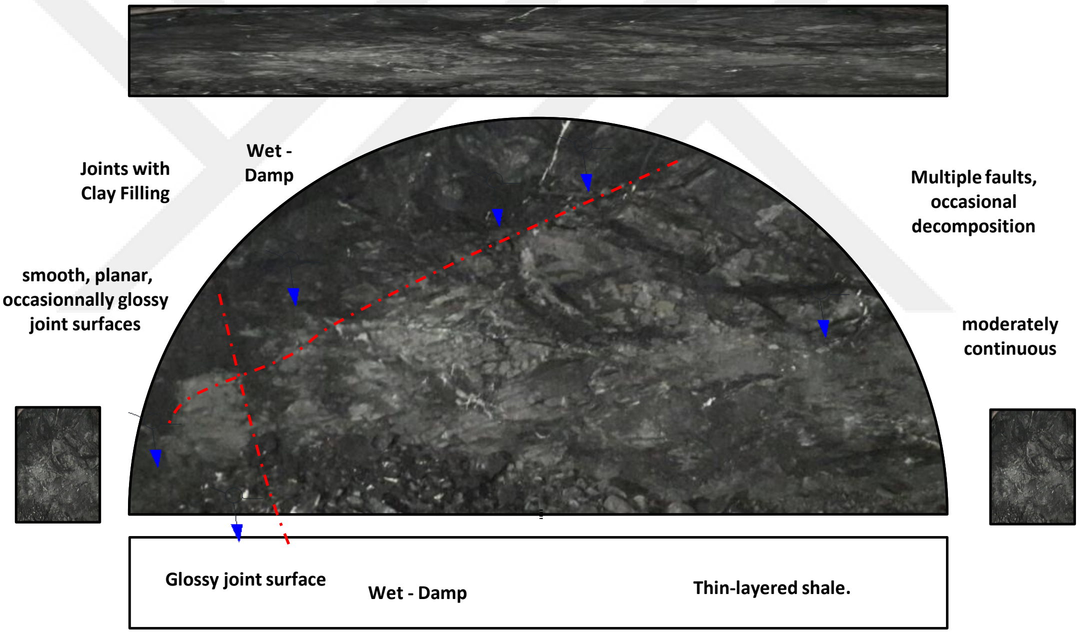
Mapping Km.	: 175+936,50	DATE	: 5.07.2019
		MAP NO	: T8ÇGA-481

Q Value	0,020	Rock Class. for Q and RMR Values (ÖNORM B 2203 AFTER 1994)	C2 BASKILI	Recommended Support Type	C2
RMR Value	16,0				

Shale | Sandstone | Eklem Doğrultu ve Eğimi

Layer	-
Discontinuities:	-

Siltstone | Layer Orientation and Slope | Driping Water | Moderate Water Inflow | Excessive Water Inflow



ROCK DEFINITION	
Geological Definition	*Multiple faults, occasional decomposition, moderately strong-weak SHALE*

PHYSICAL PROPERTIES

Grey colored, fine-medium grained, multiple joints, narrow separation (60 mm.), moderate continuity (3-10 m.), low separation (1 - 5 mm.), clay-silt filling, softening, smooth planar, occasionally glossy surfaces, 3 joint sets and random joints, wet-damp (<5 l/min), multiple faults, occasional decomposition, moderately strong-weak SHALE.

Figure 3.11. T-8 south Km. 175+936.50 tube Q 0.020 and RMR 16, C2 tunnel face

With the fieldwork that was carried out for the face of the T-8 South Tube Km. 175+936.50 section, engineering parameters, including lithological identifications and discontinuity measurements, were obtained. The units were classified according to their Q rock mass quality parameters. The calculations were made based on the methods reported by Barton et al. (1974) and Barton (2000).

a) Joint Roughness (Jr)

The sandstone-shale unit observed in the T-8 South section had open and closed joint systems. The closed systems were mostly filled with clay-silt, and the open systems were cemented with iron oxide. The open joint systems were observed to be flat, and their joint roughness coefficient in the Q classification system was found as Jr = 1.5.

b) Water Reduction Factor (Jw)

The sandstone-shale unit was determined as a semipermeable system, and low-moderate amounts of water flow were observed during the tunnel excavation process. Accordingly, based on the Q classification system, its joint water reduction factor was found as Jw = 0.2.

c) Joint Set Number (Jn)

In the exit portal section, where 3 different discontinuity planes were measured, the "three joint sets and random joints" value was selected, and the joint set number was found as Jn = 12.

d) Joint Alteration Number (Ja)

In the general assessment of the joint systems that were observed in the sandstone-shale units, the "silty or sandy zones, bands" option where joints are infilled with silty-clayey material was selected. According to this, in the Q classification system, the joint alteration number was found as Ja = 5.

e) Stress Reduction Factor (SRF)

The burden thickness in the calculations made for the T-8 South Tube Km. 175+936.50 section was 9 meters. Considering the weathered rock conditions, in the Q classification system, the stress reduction factor for "low stress, near surface" was determined as SRF = 2.5.

AYDINCIK-GÖZCE SECTION OF ERDEMLİ-SİLİFKE-TAŞUCU 13TH ZN-BOUND KM:158+779.790 - 177+664.477

T8 TUNNEL GEOLOGICAL FACE MAP AND ROCK CLASSIFICATION
SOUTH (LEFT) TUBE EXIT
(Prj. Km. 174+048 - 176+320.60)

Mapping Km.	: 175+936,50		
		DATE	: 5.07.2019
		MAP NO	: T8ÇGA-481

With the data obtained from the observations, drilling, and laboratory analyses using the geomechanical and engineering properties of the excavated tunnel rock, RMR and Q classification system results, KGM - NATM Practical Technical Specifications for Underground Tunnel Works, and post-October 1994 ÖNORM B results are given below.

1. RMR (Geomechanical classification)- Bieniawski Rock Quality Rating

Parameters			Explanations	Score
1	Rock Material Strength, MPa		5 - 25 Mpa	2,0
2	Rock Qualiy Designation (RQD %)		< 25%	3,0
3	Continuity Separation, mm.		60 mm.	6,0
4	Groundwater q, l/min.		<10	10
5	Discontinuity Conditions			
	a	Length (Continuity), m.	3-10	2
	b	Separation, mm.	1-5	1
	c	Roughness	Smooth	1
	d	Filling, mm.	Soft filling >5 mm	0
	e	Weathering	Highly Weathered	1
6	Discontinuity Orientation Adjustment		Unsuitable	-10
7	TOTAL SCORE		16,0	
8	ROCK CLASS		V Çok Zayıf Kaya	
9	Some Rock Class Properties			
	a	Mean Unsupported Stand-Up Time	1 metre açıklık için 30 dakika	
	b	Rock Mass Cohesion, kPa	< 100	
	c	Rock Mass Angle of Internal Friction, degrees	< 15	
	d	Support Pressure, kPa, P=((100-RMR)/100))ɣBS	294,86 kPa	
10	RMR SUPPORT TYPE (Önorm B 2203 after October 1994)		C2 BASKILI	

ɣ: Rock density; B: Tunnel span; S: Stress factor

Figure 3.12. T-8 south Km. 175+936.50 tube RMR89 classification system.

AYDINCIK-GÖZCE SECTION OF ERDEMLİ-SİLİFKE-TAŞUCU 13TH ZN-BOUND KM:158+779.790 - 177+664.477

T8 TUNNEL GEOLOGICAL FACE MAP AND ROCK CLASSIFICATION
SOUTH (LEFT) TUBE EXIT
(Prj. Km. 174+048 - 176+320.60)

DATE : **5.07.2019**
Mapping Km. : 175+936,50 MAP NO : **T8ÇGA-481**

2. Q (Rock Mass Classification)- Barton Rock Quality Classification

	Parameters		Explanations	Score
1	RQD %	Rock Quality Designation		10
2	Jn	Joint Set Number*	3 joint sets, random joints	12
3	Jr	Joint Roughness	Smooth, planar	1,5
4	Ja	Joint Alteration Number	Silty or sandy zones, bands	5
5	Jw	Water Reduction Factor	Moderate	0,2
6	SRF	Stress Reduction Factor	Low stress, near surface	2,5
7	ESR	Excavation Type-For Tunnels		1
* Jn x 2 is used in the portal parts of the tunnel.				

FORMULA $Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF} = 0{,}020$

ion diameter 12 m.

A	Block Size	RQD / Jn	0,83	E	Unsupported excavation, B	2(ESR) x Q ^0.4	0,41826
B	Block Internal Strength	Jr / Ja	8,00	F	Roof Support Pressue, Proof, MPa	2/Jr x Q^-1/3	22,2222
C	Effective Stress Factor	Jw / SRF	0,08	F*	Roof Support Pressure, Proof, MPa	(2Jn^0.5 x Q^-1/3) / 3 x Jr	57,735
D*	RMR value based on Q index: RMR=13.5 logQ + 43		20,1	G	Equivalent Dimension, De	(Excavation diameter / ESR)	12
H	Q SUPPORT TYPE (ÖNORM B 2203 after October 1994)		C2 SQUEEZING		**This equation is used if joint set number is lower than three.*		

D RMR and Q relationships (Chaqaet & Hadjigeorgiu, 1993, New Zealand Tunnels)*

3. Excavation class based on the Q index and RMR values (ÖNORM B 2203 after October 1994)

Excavation Class **C2 BASKILI** **(according to correlation between RMR and Q)**

4. Tunnel Support Type

Support Type **C2** **support recommended**

***NOTE:** Engineer will be notified of changes observed in the ground, rock classification will be made again if required.*

Figure 3.13. T-8 south Km. 175+936.50 tube Q classification system.

In the kinematic analyses, it was seen that there was shedding due to the narrow separation of the discontinuities. This required staged excavation. Additionally, as the excavation span of a tunnel gets larger, plasticization zones increase. Because the excavation span in the T-8 South Tube was approximately 12 m, precautions against plasticization zones were needed. Considering the conditions mentioned above, the rock support class for the Turkish Technical Specification for Highways (KTŞ) was determined as KTŞ C2.

Table 3.6. T-8 south tube rock classification according to the Q and RMR systems

Zone for RMR and Q Correlation;	BARTON ROCK MASS QUALITY (Q)	Q	RMR	BIENIAWSKI ROCK MASS RATING (RMR)	ÖNORM B2203 (Before October 1994)	ÖNORM B2203 (After October 1994)
1000			101			
400	Exceptional		94		A1 Stable	A1 Stable
100	Excellent		82.7	Excellent		
		70.4	80			
40	Very Good		76		A2 Slightly Overbreaking	A2 Slightly Overbreaking
10	Good		65	Good		
	Fair	5.34	60			B1 Friable
4			58		B1 Friable	
1	Poor	1.47	47	Fair		B2 Very Friable
		0.77		45		
	Very Poor	0.41	40		B2 Very Friable	B3 Rolling (Loose)
0.1		0.11	29	Poor 29		
		0.03	20		C1 Squeezing	C1 Rock Bursting
	Excessively Poor	0.021		17		C2 Squeezing
		0.015		15	C2 Heavily Squeezing	C3 Heavily Squeezing
0.01				10 Very Poor	L1 Short-Term Stable with High Cohesion	C4 Flowing
	Extremely Poor	0.002		5		
0.001			2.5		L2 Short-Term Stable with Low Cohesion	C5 Swelling

3.7. RMR and Q Classification of T-8 North Tube

From the tunnel roof to the topography in this section, the mean burden thickness was 8 meters. Layer measurements were made on the units along the existing road, and these measurements were analyzed for the tunnel. The layers included 3 discontinuity planes, and the layer thicknesses varied in the range of 2-12 cm. The unit was very weak-weak, shattered-disintegrated, and moderately-highly weathered.

a) Rock Quality Designation (RQD)

The RQD values to be used in the calculations were obtained from the measurements that were made during tunnel face excavations. The mean RQD value was found 10, and the <25% (3 points) range was selected for the RMR calculations.

b) Rock strength

After the uniaxial compressive strength and point load strength tests that were made on the borehole samples obtained in the drilling works for the tunnel, the mean strength value for this section was found as 9 MPa, and the 5-25 (2 points) range was selected for the RMR calculations.

c) Discontinuity orientation

Because the orientation of the sandstone-shale unit that was measured at the tunnel excavation face was in parallel with the tunnel axis and at an angle over 65°, the condition of "unsuitable" was accepted.

AYDINCIK-GÖZCE SECTION, ERDEMLİ-SİLİFKE-TAŞUCU 13TH ZN. BOUND KM:158+779.790 - 177+664.477

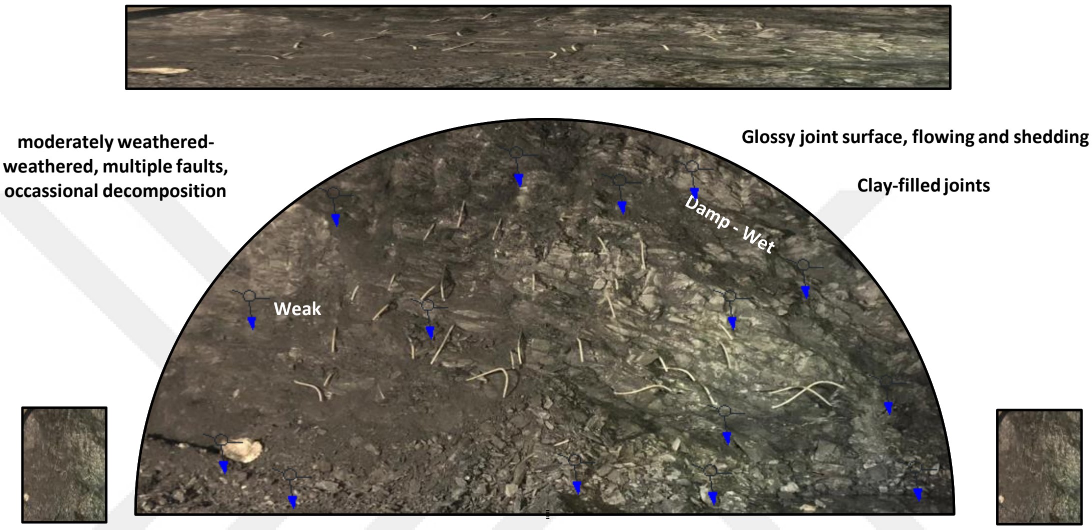
T8 TUNNEL GEOLOGICAL FACE MAP AND ROCK CLASSIFICATION
NORTH (RIGHT) TUBE ENTRANCE
(Prj. Km. 174+048 - 176+320.60)

Haritalama Km.si	: 175+837,80	DATE	: 26.02.2020
		MAP NO	: T8-GKA-1367

Q Value	0,017	Rock Classification Based on Q and RMR Valuesı (ÖNORM B 2203 AFTER OCTOBER 1994)	C2 SQUEEZING	Recommended Support Type	C2
RMR Value	15,4				

Shale | Sandstone | Joint Orientation and Slope | Silttaşı | Layer Orientation and Slope | Dripping Water | Moderate Water Inflow | Excessive Water Inflow

Layer	-
Discontinuities:	-



3 random discontinuity sets, shedding in the axis and left shoulder parts

ROCK DEFINITON	
Geological Definition	*Multiple faults, occasional decomposition, moderately strong-weak SHALE.*

PHYSICAL PROPERTIES
Grey colored, fine-medium grained, multiple joints, very narrow spacing (60-200 mm.), low separation (1 - 5 mm.), clay-silt filling, softening-loose, local calcite zones, slippery glossy surfaces, moderate continuity, 3 joint sets and random joints, dripping groundwater, multiple faults, occasional decomposition, moderately strong-weak SHALE.

Figure 3.14. T-8 north Km. Q 0.017 and RMR 15.4 C2 rock tunnel face.

With the fieldwork that was carried out for the face of the T-8 North Tube Km. 175+845.80 section, engineering parameters, including lithological identifications and discontinuity measurements, were obtained. The units were classified according to their Q rock mass quality parameters. The calculations were made based on the methods reported by Barton et al. (1974) and Barton (2000).

a) Joint Roughness (Jr)

The sandstone-shale unit observed in the T-8 North section had open and closed joint systems. The closed systems were mostly filled with clay-silt, and the open systems were cemented with iron oxide. The open joint systems were observed to be flat, and their joint roughness coefficient in the Q classification system was found as Jr = 1.5.

b) Water Reduction Factor (Jw)

The sandstone-shale unit was determined as a semipermeable system, and moderate amounts of water flow were observed during the tunnel excavation process. Accordingly, based on the Q classification system, its joint water reduction factor was found as Jw = 0.66.

c) Joint Set Number (Jn)

In the exit portal section, where 3 different discontinuity planes were measured, the "three joint sets and random joints" value was selected, and the joint set number was found as Jn = 12.

d) Joint Alteration Number (Ja)

In the general assessment of the joint systems that were observed in the sandstone-shale units, the "silty or sandy zones, bands" option where joints are infilled with silty-clayey material was selected. According to this, in the Q classification system, the joint alteration number was found as Ja = 5.

e) Stress Reduction Factor (SRF)

The burden thickness in the calculations made for the T-8 North Tube Km. 175+845.80 section was 9 meters. Considering the weathered rock conditions, in the Q classification system, the stress reduction factor for "weak zones, containing disintegrated rock" was determined as SRF = 10.

AYDINCIK-GÖZCE SECTION, ERDEMLİ-SİLİFKE-TAŞUCU 13TH ZN. BOUND KM:158+779.790 - 177+664.477

T8 TUNNEL GEOLOGICAL FACE MAP AND ROCK CLASSIFICATION
NORTH (RIGHT) TUBE ENTRANCE
(Prj. Km. 174+048 - 176+320.60)

		DATE	: **26.02.2020**
Mapping Km.	: **175+837,80**	MAP NO	: **T8-GKA-1367**

With the data obtained from the observations, drilling, and laboratory analyses using the geomechanical and engineering properties of the excavated tunnel rock, RMR and Q classification system results, KGM - NATM Practical Technical Specifications for Underground Tunnel Works, and post-October 1994 ÖNORM B results are given below.

1. RMR (Geomechanical classification)- Bieniawski Rock Quality Rating

Parameters			Explanations	Score
1	Rock Material Strength, MPa		5 - 25 Mpa	2,0
2	Rock Qualiy Designation (RQD %)		< 25%	3,0
3	Continuity Separation, mm.		10 mm.	5,4
4	Groundwater q, l/min.		10-25	7
5	Discontinuity Conditions			
	a	Length (Continuity), m.	3-10	2
	b	Separation, mm.	0.1-1	4
	c	Roughness	Smooth	1
	d	Filling, mm.	Soft Filling >5 mm	0
	e	Weathering	Highly Weathered	1
6	Discontinuity Orientation Adjustment		Unsuitable	-10
7	TOTAL SCORE		15,4	
8	ROCK CLASS		V Very Weak Rock	
9	Some Rock Class Properties			
	a	Mean Unsupported Stand-Up Time	30 minutes for 1-meter span	
	b	Rock Mass Cohesion, kPa	< 100	
	c	Rock Mass Angle of Internal Friction, degrees	< 15	
	d	Support Pressure, kPa, P=((100-RMR)/100))ɣBS	296,78 kPa	
10	RMR SUPPORT TYPE (Önorm B 2203 after October 1994)		C2 SQUEEZING	

ɣ: Rock density; B: Tunnel span; S: Stress factor

Figure 3.15. T-8 north tube Km. 175+845.80 RMR89 classification system.

AYDINCIK-GÖZCE SECTION, ERDEMLİ-SİLİFKE-TAŞUCU 13TH ZN. BOUND KM:158+779.790 - 177+664.477

T8 TUNNEL GEOLOGICAL FACE MAP AND ROCK CLASSIFICATION
NORTH (RIGHT) TUBE ENTRANCE
(Prj. Km. 174+048 - 176+320.60)

Mapping Km.	: **175+837,80**	DATE	: **26.02.2020**
		MAP NO	: **T8-GKA-1367**

2. Q (Rock Mass Classification)- Barton Rock Quality Classification

Parameters			Explanations	Score
1	RQD %	Rock Quality Designation		10
2	Jn	Joint Set Number*	3 joint sets, random joints	12
3	Jr	Joint Roughness	Smooth, undulating	1,5
4	Ja	Joint Alteration Number	Silty or sandy zones	5
5	Jw	Water Reduction Factor	Moderate water inflow	0,66
6	SRF	Stress Reduction Factor	Weakness zones, containing disintegrated rock	10
7	ESR	Excavation Type-For Tunnels		1
* Jn x 2 is used in the portal parts of the tunnel.				

FORMULA $Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF} = 0{,}017$

ation Diameter 12 m.

A	Block Size	RQD / Jn	0,83	E	Unsupported excavation, B	2(ESR) x Q ^0.4	0,38728
B	Block Internal Strength	Jr / Ja	8,00	F	Roof Support Pressue, Proof, MPa	2/Jr x Q^-1/3	26,936
C	Effective Stress Factor	Jw / SRF	0,07	F*	Roof Support Pressure, Proof, MPa	(2Jn^0.5 x Q^-1/3) / 3 x Jr	69,9819
D*	RMR value based on Q index: RMR=13.5 logQ + 43		18,9	G	Equivalent Dimension, De	(Excavation diameter / ESR)	12
H	Q SUPPORT TYPE (ÖNORM B 2203 after October 1994)		C2 SQUEEZING	*This equation is used if joint set number is lower than three.			

D RMR and Q relationships (Chaqaet & Hadjigeorgiu, 1993, New Zealand Tunnels)*

3. Excavation class based on the Q index and RMR values (ÖNORM B 2203 after October 1994)

Kazı Sınıfı **C2 SQUEEZING** **(Based on the correlation between RMR and Q)**

4. Tunnel Support Type

Support Type **C2** **support recommended**

NOTE:* *Engineer will be notified of changes observed in the ground, rock classification will be made again if required.

Figure 3.16. T-8 north tube Q classification system.

In the kinematic analyses, it was seen that there was shedding due to the narrow separation of the discontinuities and the presence of weakness zones containing disintegrated rock. This required staged excavation. Additionally, as the excavation span of a tunnel gets larger, plasticization zones increase. Because the excavation span in the T-8 North Tube was approximately 12 m, precautions against plasticization zones were needed. Considering the conditions mentioned above, the rock support class was determined as KTŞ C2.

Table 3.7. T-8 north tube rock Classification according to the Q and RMR systems

Zone for RMR and Q Correlation;		BARTON ROCK MASS QUALITY (Q)	BIENIAWSKI ROCK MASS RATING (RMR)	ÖNORM B2203 (Before October 1994)	ÖNORM B2203 (After October 1994)
	1000				
	400	Exceptional	101 94	A1 Stable	A1 Stable
	100	Excellent	Excellent 82.7		
	40	70.4 Very Good	80 76	A2 Slightly Overbreaking	A2 Slightly Overbreaking
	10	Good	Good 65		
	4	Fair 5.34	60 58	B1 Friable	B1 Friable
	1	Poor 1.47	Fair 47		B2 Very Friable
	0.1	0.77 0.41 Very Poor 0.11	45 40 Poor 29 29	B2 Very Friable	B3 Rolling (Loose)
	0.01	0.03 Excessively Poor 0.021 0.015	20 17 15 10	C1 Squeezing C2 Heavily Squeezing	C1 Rock Bursting C2 Squeezing C3 Heavily Squeezing
	0.001	Extremely Poor 0.002	Very Poor 5 2.5	L1 Short-Term Stable with High Cohesion L2 Short-Term Stable with Low Cohesion	C4 Flowing C5 Swelling

3.8. Rock Support Classification according to Turkish Highway Regulations (KTŞ)

a) Behavior: This rock class is known as "squeezing" rock. C2 is characterized by plastic zones within the surrounding rock mass and pressure-exerting behaviors. In this rock mass, moderate but noticeably long-lasting deformations with slow calm-down behaviors are observed. The degree and speeds of the deformations around the excavation are moderate. Stress is observed in rock masses with plastic behavior and high cohesion.

b) Water Effects: Water leakages and concentrated water flow have significant effects on the rock mass.

c) Excavation: The tunnel excavation must be staged as the upper half, the lower half, and the floor excavations. Except for special cases in portal sections, the upper half excavation will be separated into segments. A support element will be needed for the face of the upper half excavation. Round length will not exceed 0.75-1.25 in the upper half and 2 m in the lower half. The excavation can be made with the smooth blasting and mechanical excavation methods. Shotcrete cover must be applied immediately after clear-cutting.

d) Support and Timing: The tunnel face is generally stable. Systematic support is needed around the entire cross-section. For each round, the next round will start after the completion of the support system. Forepoling will be needed in the roof of the tunnel. During and after drilling for the implementation of forepoling, unwanted effects on the face of the construction and rock mass in the roof must be prevented. The purpose of the support elements is to limit deep plastic fractures. In compliance with the geological conditions, an invert is needed, and this invert must not be behind the upper half face by more than 50-100 meters.

The support that is shown for a certain rock class in the approved project sheets is accepted as a standard for the relevant classification. Rock bolt types and numbers, positions and inclinations, shotcrete thickness, wire mesh layers, steel ribs, and forepoling numbers and spacings can be changed. To adapt to the changing ground conditions, additional support element implementations and/or project support revisions can be made in the standard support system with the approval of engineers. Changing ground conditions require alterations in the support systems of the tunnel throughout the tunnel excavation process with experience.

4. RESULTS AND DISCUSSIONS

4.1. Tunnel Excavation and Supporting Works

The tunnel excavation was carried out in four parts with the staged excavation method as shown on Figure 4.1.

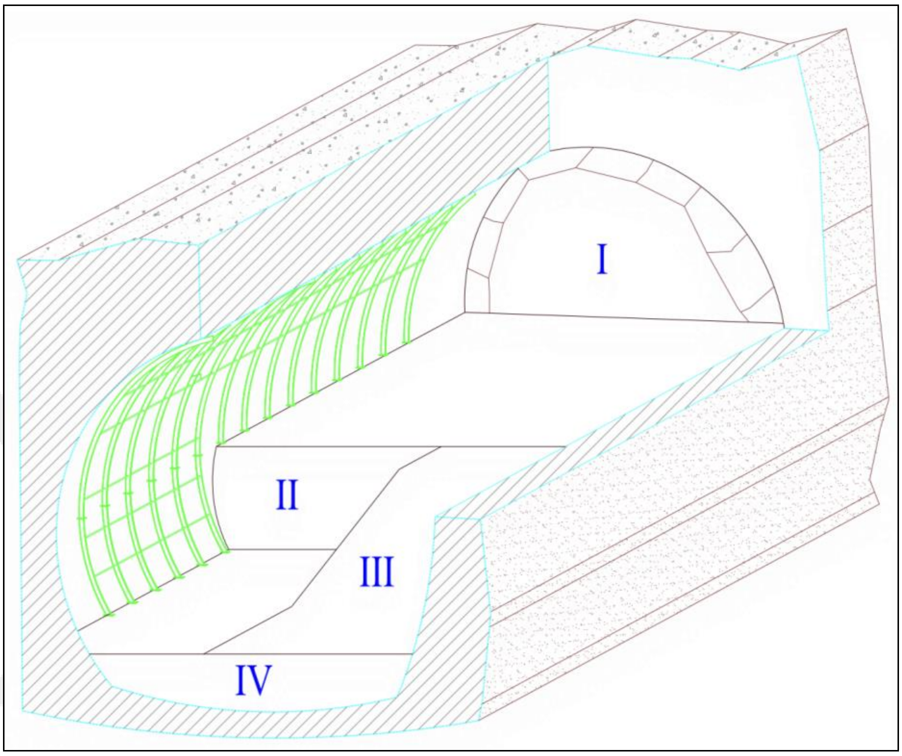


Figure 4.1. Advancement in the tunnel excavation.

The tunnel construction began with excavating and supporting the first part known as the "Top Heading". Progress was made in the upper half of the tunnel, with a maximum round length of 30 meters between the upper and lower halves.

Next, the excavation and support for the second part, called the "Bench", started from a point 40 meters behind the excavation face. Once sufficient progress was made in the second part, the ramp in the third part was moved to the second part, and the excavation and support for the third part were carried out.

Following that, the excavation and support steps for the "invert", which is the fourth part, were conducted from a point 40 meters behind the face of the upper half. A ring was formed for the tunnel at this stage.

In the tunnel, where the excavation and support stages of the first and fourth parts were completed, deformation monitoring stations were set up at 6-meter intervals. Each station contained 7 deformation monitoring instruments. The monitoring process was conducted systematically according to the following procedure.

- Daily for the 1st week,
- Twice a week for the 2nd week,
- Once a week for the 3rd and 4th weeks, and
- Monthly afterwards.

The interior lining and concrete crown works commenced after the anticipated project deformations of up to 25 cm were dampened, and the production phases were finalized.

In tunnel construction, one of the most crucial processes is taking deformation readings, using calibrated and sensitive measuring devices. Deformation readings are then plotted on graphs, as illustrated in Figure 4.7, to identify potential curve formations. The deformation graphs help to observe any upward or downward trends, leading to the determination of whether the support system in place is adequate or not.

Increasing deformation:
Support elements insufficient. Additional support needed.

Dampened deformation:

System is stabilized with additional support.

Dampened deformation:

Support elements are sufficient.

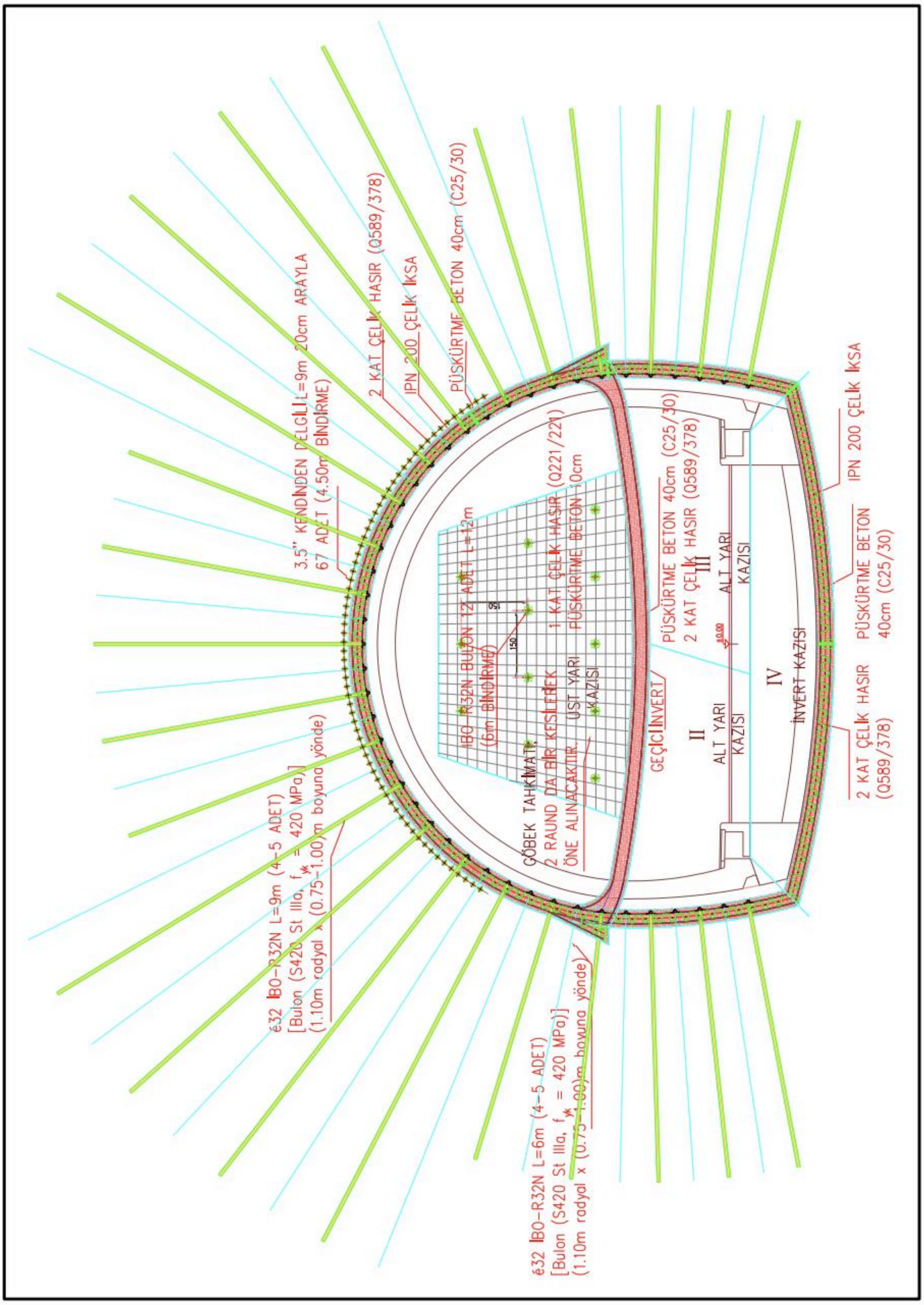


Figure 4.2. Tunnel excavation and support in a typical cross-section.

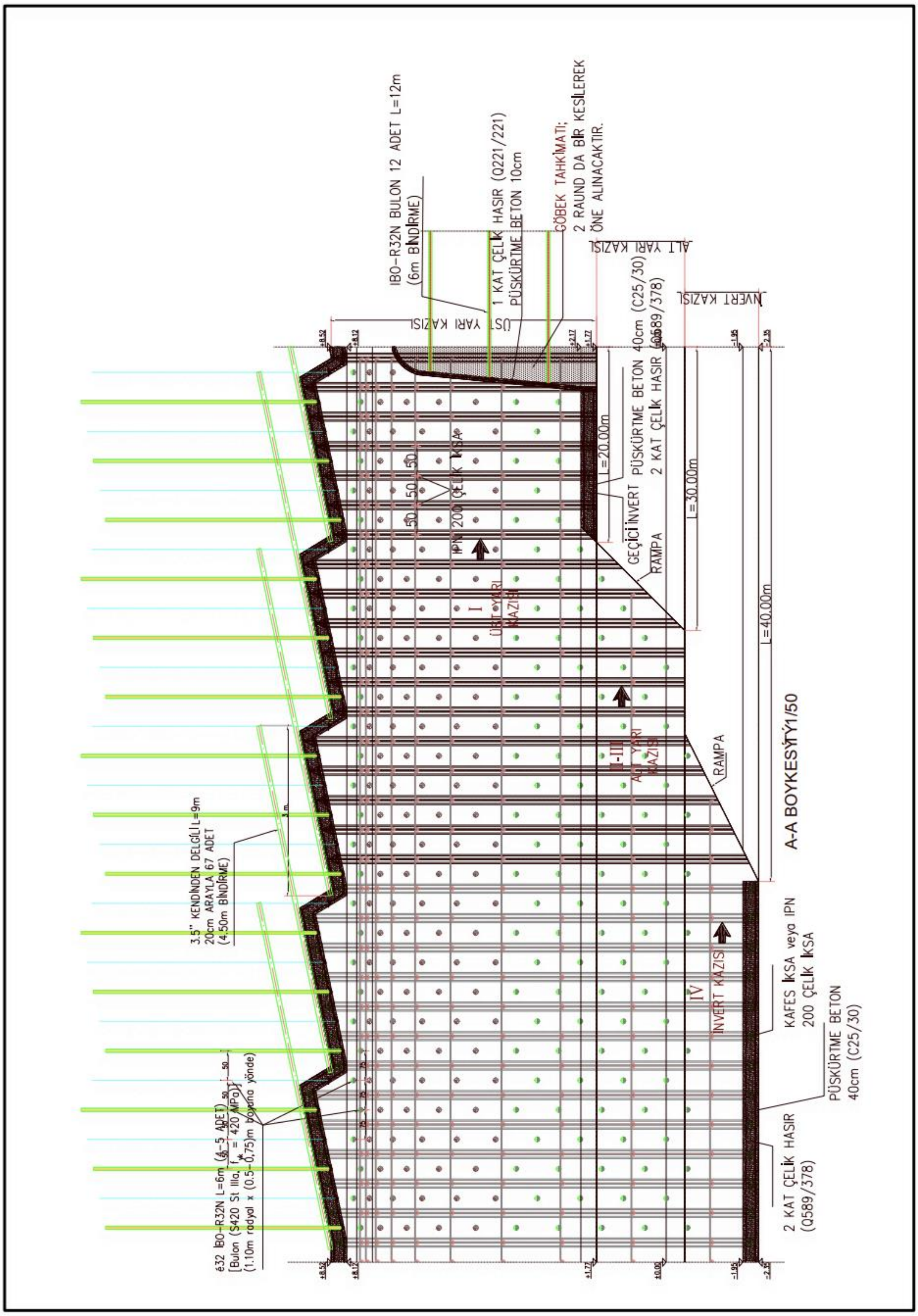


Figure 4.3. Tunnel excavation and support progress.

Figure 4.4. Upper half support installation of the tunnel.

The initial phase of tunnel excavation in the upper section involved carrying out excavation and support construction stages in a sequential manner, while also recording deformation measurements.1- L=9.00 m (6 m. overlab), self-piercing forepoling

2- Top heading excavation

3- Covering of the excavation surface with t=8 cm protection shotcrete

4- Support of the 1st layer of steel wire mesh (Ø589/378)

5- Support of the upper half NPI 200 steel rib part (see Figure 4.3.)

6- Covering with t=12 cm 1st-layer shotcrete

7- Placement of the 2nd layer of steel wire mesh (Ø589/378)

8- Covering with t=12 cm 2nd-layer shotcrete

9- Support of R32N IBO bolts and implementation of injections

10- Installation of bolt anchor plates and torquing

11- Completion of shotcrete up to 40 cm.

Figure 4.5. IBO bolt drilling and injections of the tunnel.

The excavation and support construction phases for the second and third parts of the tunnel excavations in the lower half were carried out in a similar order, with deformation measurements taken and monitored more frequently when compared to the upper half.

1- Bench excavation

2- Covering of the excavation surface with t=8 cm protection shotcrete

3- Support of the 1st layer of steel wire mesh (Ø589/378)

4- Support of the upper half NPI 200 steel rib part

5- Covering with t=12 cm 1st-layer shotcrete

6- Support of the 2nd layer of steel wire mesh (Ø589/378)

7- Covering with t=12 cm 2nd-layer shotcrete

8- Support of R32N IBO bolts and implementation of injections (see Figure 4.4.)

9- Installation of bolt anchor plates and torquing

10- Completion of shotcrete up to 40 cm.

Figure 4.6. Bench excavation and waste transport.

For the fourth segment of the tunnel excavation, the invert excavation and support construction went ahead in a planned order, with deformation measurement carried out at frequent intervals higher than those of the upper and lower sections.1- Invert excavation

2- Covering of the excavation surface with t=8 cm protection shotcrete

3- Support of the 1st layer of steel wire mesh (Ø589/378)

4- Support of the upper half NPI 200 steel rib part

5- Covering with t=12 cm 1st-layer shotcrete

6- Support of the 2nd layer of steel wire mesh (Ø589/378)

7- Completion of shotcrete up to 40 cm.

The excavation process was divided into smaller sections, allowing for observation of the behavior of the rock surrounding the excavation to determine appropriate measures for deformation. The excavation and support stages were conducted in line with the project methodology, and the entire tunnel construction process was supervised by engineers.

4.2. Deformation Measurements and Analyses

All the instruments were installed during the most recent round of excavation, near the excavation face. Baseline readings were taken immediately after each instrument was installed or as soon as it was ready to be used. Readings in sections with faster deformation were taken frequently, at least once a day, until the deformation rate eventually slowed down. The deformation measurements for the T-8 South Tube, starting from round 481 support, can be found in Table 4.1. The readings were recorded for the X, Y, and Z axes, and a deformation plot was generated.

Table 4.1. T-8 south tube Km. 175+936.50 deformation readings.

T8 Güney R 481		DÜŞEY (Z)		Yatay (X)		Boyuna (Y)		T8 Güney R 481		DÜŞEY (Z)		Yatay (X)		Boyuna (Y)	
Ok.no	TARİH	Z	Depl. mm.	X	Depl. mm.	Z	Depl. mm.	Ok.no	TARİH	Z	Depl. mm.	X	Depl. mm.	Z	Depl. mm.
1	05.07.2019	139,810	0	4000343,406	0	518878,442	0	26	29.08.2019	139,790	20	4000343,400	6	518878,456	-14
2	06.07.2019	139,812	-2	4000343,398	8	518878,437	5	27	30.08.2019	139,786	24	4000343,401	5	518878,454	-12
3	07.07.2019	139,808	2	4000343,400	6	518878,437	5	28	31.08.2019	139,795	15	4000343,399	7	518878,454	-12
4	08.07.2019	139,808	2	4000343,400	6	518878,437	5	29	01.09.2019	139,770	40	4000343,384	22	518878,446	-4
5	09.07.2019	139,807	3	4000343,399	7	518878,438	4	30	02.09.2019	139,796	14	4000343,384	22	518878,446	-4
6	10.07.2019	139,804	6	4000343,409	-3	518878,428	14	31	03.09.2019	139,766	44	4000343,386	20	518878,449	-7
7	11.07.2019	139,808	2	4000343,408	-2	518878,422	20	32	06.09.2019	139,786	24	4000343,401	5	518878,454	-12
8	12.07.2019	139,803	7	4000343,409	-3	518878,424	18	33	09.09.2019	139,795	15	4000343,399	7	518878,454	-12
9	15.07.2019	139,808	2	4000343,411	-5	518878,422	20	34	12.09.2019	139,797	13	4000343,398	8	518878,451	-9
10	18.07.2019	139,808	2	4000343,410	-4	518878,422	20	35	15.09.2019	139,778	32	4000343,372	34	518878,453	-11
11	21.07.2019	139,809	1	4000343,411	-5	518878,425	17	36	30.09.2019	139,763	47	4000343,371	35	518878,454	-12
12	22.07.2019	139,802	8	4000343,411	-5	518878,426	16	37	15.10.2019	139,772	38	4000343,373	33	518878,444	-2
13	23.07.2019	139,805	5	4000343,409	-3	518878,424	18	38	30.10.2019	139,774	36	4000343,372	34	518878,452	-10
14	24.07.2019	139,802	8	4000343,410	-4	518878,424	18	39	14.11.2019	139,775	35	4000343,371	35	518878,456	-14
15	25.07.2019	139,803	7	4000343,409	-3	518878,426	16	40	29.11.2019	139,778	32	4000343,372	34	518878,453	-11
16	26.07.2019	139,791	19	4000343,415	-9	518878,450	-8	41	14.12.2019	139,778	32	4000343,372	34	518878,454	-12
17	27.07.2019	139,789	21	4000343,411	-5	518878,454	-12	42	29.12.2019	139,779	31	4000343,372	34	518878,444	-2
18	28.07.2019	139,785	25	4000343,417	-11	518878,420	22	43	13.01.2020	139,775	35	4000343,371	35	518878,446	-4
19	31.07.2019	139,786	24	4000343,397	9	518878,453	-11	44	28.01.2020	139,774	36	4000343,371	35	518878,445	-3
20	03.08.2019	139,789	21	4000343,400	6	518878,454	-12	45	12.02.2020	139,778	32	4000343,372	34	518878,446	-4
21	06.08.2019	139,786	24	4000343,397	9	518878,453	-11	46	27.02.2020	139,779	31	4000343,372	34	518878,441	1
22	09.08.2019	139,786	24	4000343,397	9	518878,453	-11	47	13.03.2020	139,779	31	4000343,373	33	518878,440	2
23	12.08.2019	139,790	20	4000343,399	7	518878,454	-12	48	28.03.2020	139,778	32	4000343,372	34	518878,441	1
24	27.08.2019	139,790	20	4000343,391	15	518878,444	-2	49	12.04.2020	139,778	32	4000343,373	33	518878,441	1
25	28.08.2019	139,798	12	4000343,416	-10	518878,452	-10	50	27.04.2020	139,778	32	4000343,373	33	518878,440	2

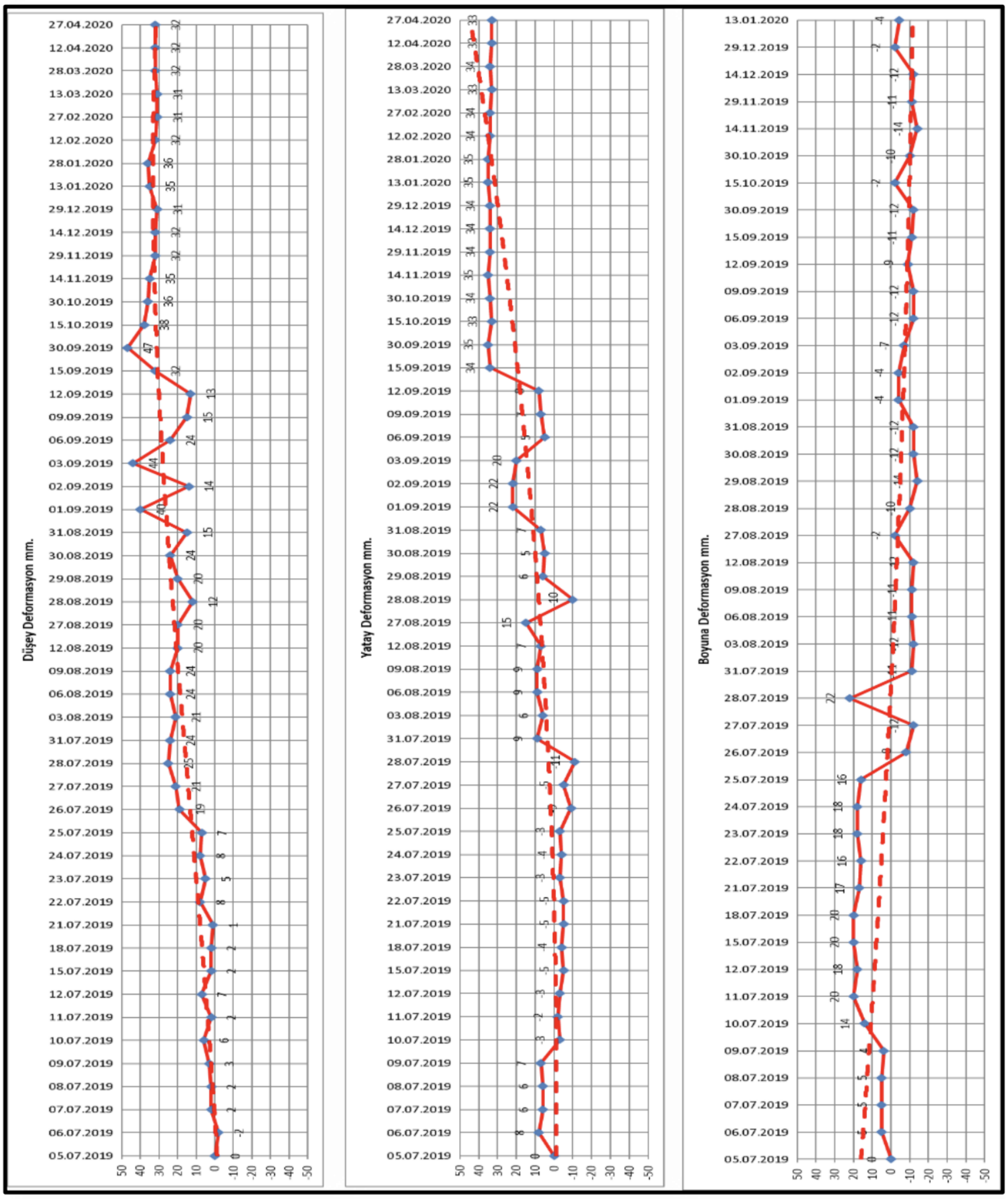


Figure 4.7. T-8 south tube Km. 175+936.50 deformations.

The spikes observed on the plot are usual reading deviations that may occur when taking measurements. In order to avoid these errors from impacting the analysis of the plot, they were not corrected, and a second curve was plotted. As evident from the deformation plot, the deformations showed a noticeable decrease in rate, remained within the expected range, and gradually diminished.

Upon achieving a reduction in the movements, the production of the concrete lining was initiated. This involved the removal of the deformation measurement instruments, application of the final shotcrete patches, and smoothing of rough surfaces. Geotextile felt and geomembrane were then applied to ensure waterproofing, as it was undesirable for water to seep into the tunnel. Prior to the final concrete application, both the waterproofing and drainage systems were inspected.

4.3. Final Concrete Lining

Prior to pouring the inner lining concrete in the tunnel, a check was performed in the tube where the concrete was intended to be placed to ensure adequate clearance. It was determined that the section width was satisfactory. Following this, the final shotcrete application was conducted, and the arch was prepared for further application.

The main component of the waterproofing material is the membranes. These membranes were applied to the surface in a manner that would ensure they remain intact by using adhesive material, specifically in areas where there was water or moisture presence. This method effectively prevented the entry of water and moisture through the surface.

Pre-fabricated concrete elements with reinforcement were used underneath the pavement in order to accommodate cable passages, drainage pipes for surface water, and fire-protection water pipes. If the tunnel was in a straight line, a 2% roof slope was used, while the superelevation slope determined by the tunnel project was used in the case of a curve. Surface water was drained at 50-meter intervals using grilles placed underneath the pavement. C30/37 concrete was utilized for the interior lining.

Figure 4.8. Application of geomembrane isolation in the tunnel.

Figure 4.9. Concrete application inside the tunnel.

5. CONCLUSIONS

A sinkhole occurred between the Km:176+143.55 and 176+104.73 of the North Tube section located between the Erdemli – Silifke – Taşucu which resulted in subsidence on the existing road surface along the Antalya – Mersin D400 Highway that had already resided above the tunnel. To resolve instabilities, the model that was formed by conducting analyses using the finite element (FE) method and revised tunnel support projects were examined.

- In this study, revised support system models were summarized, geotechnical studies were carried out for the face excavation of the tunnel, and face mapping and Q and RMR rock classification processes were carried out. The Q and RMR rock classifications were separately assessed, and correlations were found between them by using transformation formulae. Consequently, the rock mass was classified as C2 according to the ÖNORM B 2203 system after October 2014 as seen in Tables 3.6. and 3.7..
- With the numerical modeling process that was carried out, the orientations of the displacement vectors during tunnel excavation, their cluster points, and failure zones were identified. These analyses were useful in the identification of zones that need to be particularly kept in mind during excavation.
- The excavation works were advanced so that the maximum unsupported excavation specified in the classification would be achieved. After the excavation, earth-moving, and face clearance steps, measurements were made, and support works were completed without delay.
- The NATM-based excavation and support works were carried out in 4 parts as the upper half, lower half-right, lower half-left, and invert parts. As the advance of the excavation in the tunnel approached the measurement stations, the reading frequencies in the previous stations were increased, and thus, it was ensured that the momentary deformations caused by the excavation were measured.
- After the implementation of support elements, deformation measurements were also made to check the deformations in the rock based on the degrees of deformation that were anticipated in the project and to ensure that arching occurred, and the tunnel was stabilized. The deformation measurements in the rock mass were monitored, and deformation plots were prepared.
- The success of the numerical model that was established before excavation and its compatibility with the actual case were examined based on the results

encountered in the field, and it was determined that the support systems that were predicted in the model were sufficient, and the numerical model was compatible with the actual environment.

- The deformation measurements and records collected from the tunnel were analyzed and compared to the expected deformation values outlined in the project plan. Through the implementation of designated support systems, it was determined that the total displacement did not exceed 32 mm, which ensured adequate safety measures were in place.
- In tunnel construction works, the systematic excavation and support stages that are determined in approved projects should be reevaluated based on the changing underground conditions. Approved projects serve as forecasting, and tunnel support systems should be determined after making measurements on the face of the tunnel.
- Decisions based on practice and experiences are the main factors in the determination of support systems, and numerical analyses should be considered guidelines for practical decisions. Support systems may need to be revised based on the actual situations encountered in the field, geological and geotechnical measurements to be made, and realistic deformation results.

The higher the compatibility of the numerical model that is created with actual conditions and its success, the fewer the project revision recommendations needed during excavation will be.

APPENDIX

Phase-2D Analysis Information

General Settings
Number of Stages: 10
Analysis Type: Plane Strain
Solver Type: Gaussian Elimination
Units: Metric, stress as MPa

Analysis Options
Maximum Number of Iterations: 500
Tolerance: 0.001
Number of Load Steps: Automatic
Convergence Type: Absolute Energy
Tensile Failure: Reduces Shear Strength
Joint tension reduces joint stiffness by a factor of 0.01

Groundwater Analysis
Method: Piezometric Lines
Pore Fluid Unit Weight: 0.00981 MN/m3
Probability: None

Field Stress
Field stress: gravity
Using actual ground surface
Total stress ratio (horizontal/vertical in-plane): 0.577
Total stress ratio (horizontal/vertical out-of-plane): 0.577
Locked-in horizontal stress (in-plane): 0
Locked-in horizontal stress (out-of-plane): 0

Seismic Loading
Horizontal seismic load coefficient: 0.05 (positive to the right)
Vertical seismic load coefficient: 0 (positive up)
Seismic load applied in: Stage 9
Seismic Stage Factors:
Stage 1: 0, Stage 2: 0, Stage 3: 0, Stage 4: 0, Stage 5: 0, Stage 6: 0, Stage 7: 0, Stage 8: 0, Stage 9: 1, Stage 10: 0

Material Properties
Material: Residual Ground

Color	
Initial element loading	field stress & body force
Unit weight	0.016 MN/m3
Elastic type	isotropic
Young's modulus	200 MPa
Poisson's ratio	0.3
Failure criterion	Mohr-Coulomb
Peak tensile strength	0 MPa
Residual tensile strength	0 MPa
Peak friction angle	25 degrees
Peak cohesion	0.05 MPa
Material type	Plastic
Dilation Angle	0 degrees
Residual Friction Angle	25 degrees
Residual Cohesion	0.05 MPa
Piezo to use	None
Ru value	0

Material: Clay Shale (Slope Wash)

Color	
Initial element loading	field stress & body force
Unit weight	0.018 MN/m3
Elastic type	isotropic
Young's modulus	250 MPa
Poisson's ratio	0.3
Failure criterion	Mohr-Coulomb
Peak tensile strength	0 MPa
Residual tensile strength	0 MPa
Peak friction angle	25 degrees
Peak cohesion	0.075 MPa
Material type	Plastic
Dilation Angle	0 degrees
Residual Friction Angle	25 degrees
Residual Cohesion	0.075 MPa
Piezo to use	None
Ru value	0
Field Stress	Gravity
Ground surface elevation	0 m
Unit weight of overburden	0.027 MN/m3

Total stress ratio (horizontal/vertical in-plane)	a=1 b=0 c=0
Total stress ratio (horizontal/vertical out-of-plane)	a=1 b=0 c=0
Locked-in horizontal stress (in-plane)	0
Locked-in horizontal stress (out-of-plane)	0

Material: Residual Ground Ei 10% reduction

Color	
Initial element loading	field stress & body force
Unit weight	0.016 MN/m3
Elastic type	isotropic
Young's modulus	180 MPa
Poisson's ratio	0.3
Failure criterion	Mohr-Coulomb
Peak tensile strength	0 MPa
Residual tensile strength	0 MPa
Peak friction angle	25 degrees
Peak cohesion	0.05 MPa
Material type	Plastic
Dilation Angle	0 degrees
Residual Friction Angle	25 degrees
Residual Cohesion	0.05 MPa
Piezo to use	None
Ru value	0
Field Stress	Gravity
Ground surface elevation	0 m
Unit weight of overburden	0.027 MN/m3
Total stress ratio (horizontal/vertical in-plane)	a=1 b=0 c=0
Total stress ratio (horizontal/vertical out-of-plane)	a=1 b=0 c=0
Locked-in horizontal stress (in-plane)	0
Locked-in horizontal stress (out-of-plane)	0

Material: Forepoling

Color	
Initial element loading	field stress & body force
Unit weight	0.027 MN/m3
Elastic type	isotropic
Young's modulus	8241 MPa
Poisson's ratio	0.31
Failure Criterion	Generalized Hoek-Brown
Material type	Plastic
Dilation Parameter	0

Compressive strength	57 MPa
mb parameter	1.03874
s parameter	0.00753
a parameter	0.503773
Residual mb parameter	1.03874
Residual s parameter	0.00753
Residual a parameter	0.503773
Piezo to use	None
Ru value	0

Material: Concrete

Color	
Initial element loading	field stress & body force
Unit weight	0.027 MN/m3
Elastic type	isotropic
Young's modulus	20000 MPa
Poisson's ratio	0.3
Failure criterion	Mohr-Coulomb
Peak tensile strength	0 MPa
Residual tensile strength	0 MPa
Peak friction angle	35 degrees
Peak cohesion	10.5 MPa
Material type	Elastic
Piezo to use	None
Ru value	0

Material: Shale-Siltstone

Color	
Initial element loading	field stress & body force
Unit weight	0.027 MN/m3
Elastic type	isotropic
Young's modulus	347.2 MPa
Poisson's ratio	0.23
Failure Criterion	Generalized Hoek-Brown
Material type	Plastic
Dilation Parameter	0
Compressive strength	23 MPa
mb parameter	1.05587
s parameter	0.001273
a parameter	0.511368
Residual mb parameter	0.687837

Residual s parameter	0.000335
Residual a parameter	0.525561
Piezo to use	None
Ru value	0

Material: Clay Shale (Slope Wash) Ei 10% reduction

Color	
Initial element loading	field stress & body force
Unit weight	0.018 MN/m3
Elastic type	isotropic
Young's modulus	225 MPa
Poisson's ratio	0.3
Failure criterion	Mohr-Coulomb
Peak tensile strength	0 MPa
Residual tensile strength	0 MPa
Peak friction angle	25 degrees
Peak cohesion	0.075 MPa
Material type	Plastic
Dilation Angle	0 degrees
Residual Friction Angle	25 degrees
Residual Cohesion	0.075 MPa
Piezo to use	None
Ru value	0

Liner Properties

Liner: Temporary Invert (South Tube)

Color	
Liner Type	Reinforced Concrete
Equivalent Young's modulus	26900.2 MPa
Equivalent thickness	0.399581 m
Poisson ratio	0

Reinforcement Properties

Type	Wire Mesh (Canada): #8 (diameter=8mm)
Spacing	0.5 m
Section Depth	0.008 m
Area	5.03e-005 m2
Moment of inertia	2.01e-010 m4
Young's modulus	210000 MPa
Poisson ratio	0.25
Compressive strength	365 MPa
Tensile strength	365 MPa
Unit weight	0.79 kg/m

Concrete Properties

Thickness	0.4 m
Young's modulus	25743 MPa
Poisson ratio	0.2
Compressive strength	30 MPa
Tensile strength	1.9 MPa
Unit weight	0.024 MN/m3

Liner: South Tube Lower Half

Color	
Liner Type	Reinforced Concrete
Equivalent Young's modulus	25607.4 MPa
Equivalent thickness	0.383859 m
Poisson ratio	0

Reinforcement Properties

Type	I-beam (IPN): IPN 200
Spacing	0.5 m
Section Depth	0.2 m
Area	0.00334 m2
Moment of inertia	2.14e-005 m4
Young's modulus	210000 MPa
Poisson ratio	0.25
Compressive strength	365 MPa
Tensile strength	365 MPa
Unit weight	26.2 kg/m

Concrete Properties

Thickness	0.4 m
Young's modulus	20000 MPa
Poisson ratio	0.2
Compressive strength	30 MPa
Tensile strength	1.8 MPa
Unit weight	0.024 MN/m3

Properties changed in Stage 4

Young's modulus: 5000 MPa (factor = 0.25)

Properties changed in Stage 5

Young's modulus: 20000 MPa (factor = 1)

Liner: Elephant’s Feet (South Tube)

Color	
Liner Type	Reinforced Concrete
Equivalent Young's modulus	31577 MPa
Equivalent thickness	0.387072 m
Poisson ratio	0

Reinforcement Properties

Type	I-beam (IPN): IPN 200
Spacing	0.5 m
Section Depth	0.2 m
Area	0.00334 m2
Moment of inertia	2.14e-005 m4
Young's modulus	210000 MPa
Poisson ratio	0.25
Compressive strength	365 MPa
Tensile strength	365 MPa
Unit weight	26.2 kg/m

Concrete Properties

Thickness	0.4 m
Young's modulus	25743 MPa
Poisson ratio	0.2
Compressive strength	30 MPa
Tensile strength	1.9 MPa
Unit weight	0.024 MN/m3

Liner: North Tube Upper Half

Color	
Liner Type	Reinforced Concrete
Equivalent Young's modulus	25607.4 MPa
Equivalent thickness	0.383859 m
Poisson ratio	0

Reinforcement Properties

Type	I-beam (IPN): IPN 200
Spacing	0.5 m
Section Depth	0.2 m
Area	0.00334 m2
Moment of inertia	2.14e-005 m4
Young's modulus	210000 MPa
Poisson ratio	0.25
Compressive strength	365 MPa
Tensile strength	365 MPa
Unit weight	26.2 kg/m

Concrete Properties

Thickness	0.4 m
Young's modulus	20000 MPa
Poisson ratio	0.2
Compressive strength	30 MPa
Tensile strength	1.8 MPa
Unit weight	0.024 MN/m3

Properties changed in Stage 7

Young's modulus: 5000 MPa (factor = 0.25)

Properties changed in Stage 8

Young's modulus: 20000 MPa (factor = 1)

Liner: Temporary Invert (North Tube)

Color	
Liner Type	Reinforced Concrete
Equivalent Young's modulus	26900.2 MPa
Equivalent thickness	0.399581 m
Poisson ratio	0

Reinforcement Properties

Type	Wire Mesh (Canada): #8 (diameter=8mm)
Spacing	0.5 m
Section Depth	0.008 m
Area	5.03e-005 m2
Moment of inertia	2.01e-010 m4
Young's modulus	210000 MPa
Poisson ratio	0.25
Compressive strength	365 MPa
Tensile strength	365 MPa
Unit weight	0.79 kg/m

Concrete Properties

Thickness	0.4 m
Young's modulus	25743 MPa
Poisson ratio	0.2
Compressive strength	30 MPa
Tensile strength	1.9 MPa
Unit weight	0.024 MN/m3

Liner: Elephant’s Feet (North Tube)

Color	
Liner Type	Reinforced Concrete
Equivalent Young's modulus	31577 MPa
Equivalent thickness	0.387072 m
Poisson ratio	0

Reinforcement Properties

Type	I-beam (IPN): IPN 200
Spacing	0.5 m
Section Depth	0.2 m
Area	0.00334 m2
Moment of inertia	2.14e-005 m4
Young's modulus	210000 MPa
Poisson ratio	0.25
Compressive strength	365 MPa
Tensile strength	365 MPa
Unit weight	26.2 kg/m

Concrete Properties

Thickness	0.4 m
Young's modulus	25743 MPa
Poisson ratio	0.2
Compressive strength	30 MPa
Tensile strength	1.9 MPa
Unit weight	0.024 MN/m3

Liner: Lining Concrete, North Tube

Color	
Liner Type	Reinforced Concrete
Equivalent Young's modulus	34093.8 MPa
Equivalent thickness	0.404661 m
Poisson ratio	0

Reinforcement Properties

Type	Rebar (US): #25 (diameter=25.4mm)
Spacing	0.15 m
Section Depth	0.254 m
Area	0.001018 m2
Moment of inertia	1.6461e-005 m4
Young's modulus	210000 MPa
Poisson ratio	0.25
Compressive strength	400 MPa
Tensile strength	400 MPa
Unit weight	7.964 kg/m

Concrete Properties

Thickness	0.4 m
Young's modulus	30000 MPa
Poisson ratio	0.15
Compressive strength	25 MPa
Tensile strength	2 MPa
Unit weight	0.024 MN/m3

Liner: Lower Half, Pb KT

Color	
Liner Type	Reinforced Concrete
Equivalent Young's modulus	25607.4 MPa
Equivalent thickness	0.383859 m
Poisson ratio	0

Reinforcement Properties

Type	I-beam (IPN): IPN 200
Spacing	0.5 m
Section Depth	0.2 m
Area	0.00334 m2
Moment of inertia	2.14e-005 m4
Young's modulus	210000 MPa
Poisson ratio	0.25
Compressive strength	365 MPa
Tensile strength	365 MPa

Unit weight	26.2 kg/m

Concrete Properties

Thickness	0.4 m
Young's modulus	20000 MPa
Poisson ratio	0.2
Compressive strength	30 MPa
Tensile strength	1.8 MPa
Unit weight	0.024 MN/m3

Properties changed in Stage 8

Young's modulus: 5000 MPa (factor = 0.25)

Properties changed in Stage 9

Young's modulus: 20000 MPa (factor = 1)

Liner: Lining Concrete (South Tube)

Color	
Liner Type	Reinforced Concrete
Equivalent Young's modulus	34093.8 MPa
Equivalent thickness	0.404661 m
Poisson ratio	0

Reinforcement Properties

Type	Rebar (US): #25 (diameter=25.4mm)
Spacing	0.15 m
Section Depth	0.254 m
Area	0.001018 m2
Moment of inertia	1.6461e-005 m4
Young's modulus	210000 MPa
Poisson ratio	0.25
Compressive strength	400 MPa
Tensile strength	400 MPa
Unit weight	7.964 kg/m

Concrete Properties

Thickness	0.4 m
Young's modulus	30000 MPa
Poisson ratio	0.15
Compressive strength	25 MPa
Tensile strength	2 MPa
Unit weight	0.024 MN/m3

Bolt Properties

Bolt name	32 mm
Color	
Bolt Type	Fully bonded bolt
Diameter	32 mm
Young's modulus	210000 MPa
Tensile capacity	0.29 MN
Residual Tensile capacity	0.01 MN
Pre-tensioning	0 MN
Pre-tensioning force	Constant in install stage
Out-of-plane spacing	0.5 m
Allow Joints to Shear Bolt	Yes

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