DESIGN OF EXCAVATION AND SUPPORT SYSTEMS FOR THE ÇUBUKBELİ TUNNEL IN ANTALYA

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

ERCÜMENT KARAHAN

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
MINING ENGINEERING

JANUARY 2010

Approval of the thesis:

DESIGN OF EXCAVATION AND SUPPORT SYSTEMS FOR THE ÇUBUKBELİ TUNNEL IN ANTALYA

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ABSTRACT

DESIGN OF EXCAVATION AND SUPPORT SYSTEMS
FOR THE
ÇUBUKBELİ TUNNEL IN ANTALYA

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January 2010, 116 pages

In this thesis, suggestion of appropriate excavation and support systems and selection of rock mass strength parameters for the determination of these systems were carried out for the Çubukbeli Tunnel in Antalya.

Çubukbeli Tunnel is a twin tube flute shaped tunnel with 1985 m length, 12 m width, 10 m height and maximum overburden thickness of 130 m. The tunnel area consists of limestone, clayey limestone, claystone, marl and siltsone. Rock mass classification systems are used for evaluation of rock mass characteristics and estimation of strength parameters. Selection of appropriate numerical method and software tool, namely Phase^2, is accomplished after an extensive literature survey.

The rock mass was divided into sections according to the RMR, Q, NATM and GSI classification systems along the tunnel and excavation and support systems were determined empirically along these sections. Thereafter,

geomechanical parameters (i.e. modulus of deformation E_m, Hoek-Brown material constants m and s etc.) were selected based on these classification systems.

Finite element analysis was carried out as the final step of the design in order to investigate deformations and stress concentrations around the tunnel, analyze interaction of support systems with excavated rock masses and verify and check the validity of empirically determined excavation and support systems.

As the result of design studies accomplished along tunnel route, B1, B2, B3 and C2 type rock classes are assumed to be faced during construction of Çubukbeli Tunnel and appropriate excavation and support systems are proposed for these rock classes.

Keywords: Çubukbeli Tunnel, Classification Systems, Rock Mass Strength Parameters, Excavation and Support Systems, Finite Element Analysis.

ÖZ

ANTALYA ÇUBUKBELİ TÜNELİ İÇİN
KAZI VE DESTEK SİSTEMLERİ
TASARIMI

Karahan, Ercüment
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Tez Yöneticisi: Doç. Dr. H. Aydın Bilgin

Ocak 2010, 116 sayfa

Bu tezde, Antalya Çubukbeli Tüneli boyunca uygulanacak kazı ve destek sistemleri ve bu sistemlerin belirlenmesine yönelik olarak da kaya kütlesi dayanım parametrelerinin seçilmesi ile ilgili çalışmalar ortaya konmuştur.

Çubukbeli Tüneli 1985 m uzunluğunda, 12 m genişliğinde, 10 m yüksekliğinde ve maximum 130 m örtü kalınlığına sahip, çift tüplü flüt şeklinde bir tüneldir. Tünel alanı, kireçtaşı, killi kireçtaşı, kil taşı, marn ve kumtaşından oluşmaktadır. Kaya kütle özelliklerinin değerlendirilmesi ve dayanım parametrelerinin belirlenmesi amacıyla kaya kütlesi sınıflandırma sistemleri kullanılmıştır. Uygun sayısal metod ve Phase^2 isimli bilgisayar programının seçimine yönelik olarak geniş çaplı bir kaynak taraması yapılmıştır.

Kaya kütlesi, tünel boyunca RMR, Q, NATM ve GSI sınıflandırma sistemlerine göre bölümlere ayrılmış ve her bir bölüm için kazı ve destek

sistemleri gözlemsel yöntemle belirlenmiştir. Daha sonra, bu sınıflandırma sistemlerine dayalı olarak jeomekanik parametreler (deformasyon modülü E_m, Hoek-Brown malzeme sabitleri m ve s vb.) seçilmiştir.

Tasarının son aşaması olarak ise, hem tünel etrafındaki deformasyon ve gerilme konsantrasyonlarının araştırılması, hem kazılmış kaya kütlesi ile destek sistemleri etkileşiminin analiz edilmesi ve hem de gözlemsel yöntemle önerilmiş kazı ve destek sistemlerinin uygunluğunun kanıtlanması amacıyla sonlu eleman analizleri yapılmıştır.

Tünel boyunca gerçekleştirilen tasarım çalışmalarının sonucuna göre, Çubukbeli Tüneli inşaatı sırasında B1, B2, B3 ve C2 sınıfı kaya ile karşılaşılacağı öngörülmüş ve bu kaya sınıflarına uygun kazı ve destek sistemleri önerilmiştir.

Anahtar Kelimeler: Çubukbeli Tüneli, Sınıflandırma Sistemleri, Kaya Kütle Dayanım Parametreleri, Kazı ve Destek Sistemleri, Sonlu Eleman Analizleri.

ACKNOWLEDGEMENTS

I would like to express my indebted appreciation to my supervisor Assoc. Prof. Dr. H. Aydın Bilgin for his very kind supervision, valuable suggestions and friendship throughout this study.

I express my gratitude to the members of the examining committee namely: Prof. Dr. Bahtiyar Ünver, Prof. Dr. Tamer Topal, Prof. Dr. Naci Bölükbaşı and Prof. Dr. Celal Karpuz for their valuable comments.

I would like to express my special thanks to Songül Coşar, Nail Eren and Boğaçhan Yalvaç for their contribution to this thesis.

Finally, but most deeply, I would like to thank my valuable wife Gülay Karahan for her support and patience during preparation of this thesis.

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B	Span or width of the tunnel (m)
c	cohesion (kPa)
D	Disturbance factor
E_m	Deformation modulus (GPa)
ESR	Excavation support ratio
h_t	Rock-load height (m)
H	Overburden or tunnel depth (m)
GSI	Geological strength index
J_a	Joint alteration number
J_n	Joint set number
J_r	Joint roughness number
J_v	Volumetric joint count (joint/m^3)
J_w	Joint water reduction factor
J_S	Joint spacing
k	Horizontal to vertical stress ratio
KGM	General Directorate of Highways
M-RMR	Modified rock mass rating
NATM	New Austrian tunneling method
NGI	Norvegian Geotechnical Institute
Q	Rock mass quality
P	Support pressure (kN/m^2 or MPa)
r_f	Mass Reduction Factor
RMi	Rock mass index
RMR	Rock mass rating
RMS	Rock mass strength
RQD	Rock quality designation
RSR	Rock structure rating

S	Strength factor
SCR	Surface condition rating
SR	Structure rating
SRF	Stress reduction factor
TBM	Tunnel boring machine
URCS	Unified rock mass classification system
WCS	Weakening coefficient system
Φ	Internal friction angle (^{o})
γ	Density of the rock (N/m^3)
σ_c	Uniaxial compressive strength (MPa)
σ_h	Horizontal stress (MPa)
σ_v	Vertical stress (MPa)

CHAPTER I

INTRODUCTION

1.1 General Remarks

The main purpose of a tunnel design is to use the rock itself as the principal structural material with little disturbance during the excavation and to provide as little support system as possible. For this purpose, determinations of geological and geotechnical conditions existing in a tunnel area and optimum simulation of these circumstances for estimation of appropriate excavation and support systems are absolutely necessary. Rock mass classification systems and numerical analysis methods are (and should be) used together for this purpose to provide safety, economy, performance and conformity during construction and operation of an underground opening.

1.2 Statement of the Problem

To provide input data for empirical and numerical design of tunnels, it is necessary to determine the geological and geotechnical conditions in the study area and carry out rock mass classification systems and determine geomechanical parameters in the tunnel ground. The Rock Mass Rating (RMR), Rock Mass Quality (Q), Geological Strength Index (GSI), and New Austrian Tunnelling Method (NATM) are commonly used in rock mass classification systems. The above mentioned classification systems are used in Çubukbeli Tunnel project area and correlations among these classification systems are performed. For the determination of rock mass strength parameters, worldwide commonly used Hoek & Brown Failure Criterion and the approaches of Hoek&Diederichs (2006), Sönmez et.al. (2006), Hoek (2002) and Barton (2002) are utilized. And for finite element analysis, ***Phase²*** program (version 6.03), developed by Rocscience Group, was used.

1.3 Objectives of the Thesis

This study has three main objectives. The first objective consists of two stages, namely i) classification of the rock mass in the tunnel area according to the Rock Mass Rating (RMR), Rock Mass Quality (Q), Geological Strength Index (GSI), and New Austrian Tunnelling Method (NATM) and correlation among these classification systems, and ii) suggestion of empirical excavation and support systems according to New Austrian Tunnelling Method (NATM)

The second objective is to determine the rock mass strength parameters based on Geological Strength Index (GSI) Classification and laboratory test results.

The third objective is to perform finite element analysis to verify and check the validity of empirically determined excavation and support systems.

1.4 Thesis Outline

Following the introduction, Chapter 1, the rock mass classification systems and their excavation and support recommandations as empirical approach and estimation of rock mass strength parameters and application of finite element analysis as numerical approach to tunnel design are reviewed in Chapter 2.

Information about Çubukbeli Tunnel, geological and geotechnical investigations accomplished around tunnel area and enginnering geological characteristics of formations along tunnel route are presented in Chapter 3.

Chapter 4 includes rock mass classifications and empirical determination of excavation and support systems prior to estimation of rock mass strength parameters and modulus of deformation E_m.

Finite element analysis for verification of empirical methods was carried out in Chapter 5.

Finally, conclusions and recommendations related to this study are presented in Chapter 6.

CHAPTER II

LITERATURE SURVEY

2.1 Introduction

Provision of reliable input parameters for tunnel design in rock is one of the most difficult tasks for engineering geologists and tunnel design engineers. It is extremely important that the quality and accuracy of input parameter match the sophisticated design methods. Obviously, it must be realized that the reliability of computer results is mainly controlled by the accuracy of input parameter, that means, incorrect input parameters lead incorrect results. Therefore each computer or analysis results should be carefully checked by an experienced tunnelling engineer for its reliability. In addition, the rock parameters may vary in a wide range over the length of a tunnel.

It is the task of the designer to select and determine the most significant parameters of the ground and the support materials, interpret investigation results, and make the best use of available design methods and models.

Basically, there are three different methods used in engineering design. These are empirical, analytical and numerical methods. Empirical design method relates practical experience gained on previous projects to the conditions anticipated at a proposed site and requires experience as well as engineering judgement. Rock mass classification systems are an integral part of empirical tunnel design and have been successfully applied throughout the world as a uniqe method for design.

During the feasibility and preliminary design stages of a project, when very little information on the rock mass and its stress and hydrogeological characteristics is available, the use of a rock mass classification can be of considerable benefit. At its simplest, this may involve using the classification scheme as a check list to ensure that all relevant information has been considered. At the other end of the spectrum, one or more rock mass classification schemes can be used to build up a picture of the composition and characteristics of a rock mass to provide initial estimates of support requirements, and to provide estimates of the strength and deformation properties of the rock mass (Hoek et al., 1995) , those will also be input parameters for numerical analysis.

A rock mass classification system has the following purposes in application (Bieniawski, 1976):

a. To divide a particular rock mass into groups of similar behavior,
b. To provide a basis for understanding the characteristics of each group,
c. To facilitate the planning and design of excavations in rock by yielding quantitative data required for the solution of real engineering problems,
d. To provide a common basis for effective communication among all persons concerned with a geotechnical project.

The classification systems are not recommended for use in detailed and final design, especially for complex underground structures. For these purposes and to provide an optimum engineering design, especially in tunnel engineering, empirical methods should be used in combination with numerical methods to correlate and verify each other.

2.2 Rock Mass Classification Systems

There are many different rock mass classification systems and the most common ones are shown below in Table-2.1.

Rock mass classification systems have been developing for more than 60 years since Terzaghi (1946) firstly attempted to classify the rock masses for engineering purposes. Terzaghi (1946) classified rock conditions into nine categories ranging from hard and intact rock, class 1, to swelling rock, class 9.

Lauffer (1958) proposed that the stand up time for an unsupported span is related to the quality of the rock mass in which the span is excavated.

The Rock Quality Designation index (RQD) was developed by Deere et al. (1967) to provide a quantitative estimate of rock mass quality from drill core logs. RQD is defined as the percentage of intact pieces longer than 100 mm (4inches) in total length.

Palmström (1982) suggested that, when no core is available, but discontinuity traces are visible in surface exposures or exploration adits, the RQD might be estimated from the number of discontinuities per unit volume. The most important use of RQD is as a component of the RMR and Q rock mass classifications.

Wickham et al. (1972) proposed a quantitative method for describing the quality of a rock mass and for selecting appropriate support on the bases of their Rock Structure Rating (RSR) classification. Although the RSR classification system is not widely used, Wickham et al.'s work played a significant role in the development of the classification systems, which will be mentioned, in the following paragraphs.

Table 2.1 Major rock mass classification systems (Bieniawski, 1989; Özkan and Ünal, 1996; Ulusay and Sönmez., 2002).

Rock Mass Classification System	Originator	Country of Origin	Application Areas
Rock Load	Terzaghi, 1946	USA	Tunnels with steel Support
Stand-up time	Lauffer, 1958	Australia	Tunnelling
New Austrian Tunneling Method (NATM)	Pacher et al., 1964	Austria	Tunnelling
Rock Quality Designation (RQD)	Deere et al, 1967	USA	Core logging, tunnelling
Rock Structure Rating (RSR)	Wickham et al, 1972	USA	Tunnelling
Rock Mass Rating (RMR)	Bieniawski, 1973 (last modification 1989-USA)	South Africa	Tunnels, mines, (slopes, foundations)
Modified Rock Mass Rating (M-RMR)	Ünal and Özkan, 1990	Turkey	Mining
Rock Mass Quality (Q)	Barton et al, 1974 (last modification 2002)	Norway	Tunnels, mines, foundations
Strength-Block size	Franklin, 1975	Canada	Tunnelling
Basic Geotechnical Classification	ISRM, 1981	International	General
Rock Mass Strength (RMS)	Stille et al, 1982	Sweden	Metal mining
Unified Rock Mass Classification System (URCS)	Williamson, 1984	USA	General Communication
Weakening Coefficient System (WCS)	Singh, 1986	India	Coal mining
Rock Mass Index (RMi)	Palmström, 1996	Sweden	Tunnelling
Geological Strength Index (GSI)	Hoek and Brown, 1997	Canada	All underground excavations

For a preliminary tunnel design, at least two classification systems should be applied (Bieniawski, 1989). In this study the most commonly used and applicable classification systems; Rock Mass Rating (RMR), Rock Mass Quality (Q), Geological Strength Index (GSI) and New Austrian Tunneling Method (NATM) are used. More detailed information about these classification systems will be given in the following paragraphs.

2.2.1 Rock Mass Rating (RMR) System

The Geomechanics Classification or the Rock Mass Rating (RMR) system was developed by Bieniawski in 1973. Significant changes have been made over the years with revisions in 1974, 1976, 1979 and 1989; in this study the discussion is based upon the latest version (Bieniawski, 1989) of the classification system.

The RMR classification has found wide applications in various types of engineering projects, such as tunnels, foundations, and mines but, not in slopes. Most of the applications have been in the field of tunneling.

Originally 49 case histories used in the development and validation of the RMR Classification in 1973, followed by 62 coal mining case histories that were added by 1984 and a further 78 tunneling and mining case histories collected by 1987. To the 1989 version, the RMR system has been used in 351 case histories (Bieniawski, 1989).

This classification of rock masses utilizes the following six parameters, all of which are measurable in the field and some of them may also be obtained from borehole data (Bieniawski, 1989):

a. Uniaxial compressive strength of intact rock material,
b. Rock quality designation (RQD),
c. Spacing of discontinuities,
d. Condition of discontinuities,
e. Groundwater conditions,
f. Orientation of discontinuities.

To apply this classification system, the rock mass along the tunnel route is divided into a number of structural regions, e.g., zones in which certain geological features are more or less uniform within each region. The above six parameters are determined for each structural region from measurements in the field and entered into the standard input data sheets.

The RMR system is presented in Table 2.2. In Section A of Table 2.2, the first five parameters are grouped into five ranges of values. Since the various parameters are not equally important for the overall classification of a rock mass, importance ratings are allocated to the different value ranges of the parameters, a higher rating indicating better rock mass conditions (Bieniawski, 1989).

It is suggested by Bieniawski (1989), however, that the charts A-D in Appendix A in Figures A.1, A.2, A.3, A.4 should be used instead of A1 (uniaxial compressive strength), A2 (RQD) and A3 (spacing of discontinuities) in Table 2.2. These charts are helpful for borderline cases and also remove an impression that abrupt changes in ratings occur between categories. Chart D is used if either RQD or discontinuity data are lacking.

Table 2.2 Rock mass rating system (After Bieniawski, 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock metarial	Point-load strength index	> 10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive		
		Uniaxial comp. strength	> 250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality RQD		90 % - 100 %	75 % - 90 %	50 % - 75 %	25 % - 50 %	< 25 %		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0,6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surface Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1 - 5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length(1/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0,1	0,1 - 0,2	0,2 - 0,5	> 0,5		
		General Conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Ratings	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2,5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS**

Discontinuity length (persistence)	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m
Ratings	6	4	2	1	0
Separation (aperture)	None	< 0,1 mm	0,1 - 1,0 mm	1 - 5 mm	> 5 mm
Ratings	6	5	4	1	0
Roughness	Very rough	Rough	Slightyl rough	Smooth	Slickensided
Ratings	6	5	3	1	0
Infilling (gouge)	None	Hard filling <5 mm	Hard filling >5 mm	Soft filling <5 mm	Soft filling >5 mm
Ratings	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately	Highly Weathered	Decomposed
Ratings	6	5	3	1	0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip-Dip 45 - 90°	Drive with dip-Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very favourable	Fair
Drive against dip-Dip 45 - 90°	Drive against dip-Dip 20 - 45°	Dip 0 - 20° - Irrespective of strike	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly. ** Modified after Wickham et al. (1972) .

*** Instead of A.1, A.2, and A.3 use the charts A-D given in Figure A.1. included in App.A **** Section E is used to calculate basic RMR.

After the importance ratings of the classification parameters are established, the ratings for the five parameters listed in Section A of Table 2.2 are summed up to yield the basic rock mass rating for the structural region under consideration.

At this stage, the influence of strike and dip of discontinuities is included by adjusting the basic rock mass rating according to Section B of Table 2.2. This step is treated separately because the influence of discontinuity orientation depends upon engineering application e.g., tunnel (mine), slope or foundation. It will be noted that the value of the parameters discontinuity orientation is not given in quantitative terms but by qualitative descriptions such as favorable. To facilitate a decision whether strike and dip orientations are favorable or not, reference should be made to Section F in Table 2.2, which is based on studies by Wickham et al. (1972).

After the adjustment for discontinuity orientations, the rock mass is classified according to Section C of Table 2.2, which groups the final (adjusted) rock mass ratings (RMR) into five rock mass classes, the full range of the possible RMR values varying from zero to 100. Note that the rock mass classes are in groups of twenty ratings each.

Next, Section D of Table 2.2 gives the practical meaning of each rock mass class by relating it to specific engineering problems. In the case of tunnels and chambers, the output from the RMR System may be used to estimate the stand-up time and the maximum stable rock span for a given RMR.

Lauffer (1988) presented a revised stand-up time diagram specifically for tunnel boring machine (TBM) excavation. This diagram is most useful because it demonstrates how the boundaries of RMR classes are shifted for TBM applications. Thus, an RMR adjustment can be made for machine-excavated rock masses.

Support pressures can be determined from the RMR System as (Ünal, 1992) :

$$P = \left(\frac{100 - RMR}{100}\right) . \gamma . B . S = \gamma . h_t \qquad (2.1)$$

$$h_t = \left(\frac{100 - RMR}{100}\right) . B . S \qquad (2.2)$$

where

P : is the support pressure in kN/m^2,

h_t : is the rock-load height in meters,

B : is the tunnel width in meters,

S : strength factor ,

γ : is the density of the rock in kN/m^3.

Using the measured support pressure values from 30-instrumented Indian tunnels, Goel and Jethwa (1991) proposed Equation 2.3 for estimating the short-term support pressure for underground openings in the case of tunneling by conventional blasting method using steel rib supports:

$$P = \left(\frac{0.75xB^{0.1}xH^{0.5} - RMR}{2RMR}\right) \qquad (2.3)$$

where

P : is the support pressure in MPa,

H : is the overburden or tunnel depth in meters (>50 m),

B : is the span of opening in meters.

RMR System provides a set of guidelines for the selection of rock support for tunnels in accordance with Table 2.3. These guidelines depend on such factors as the depth below surface (in-situ stress), tunnel size and shape, and the method of excavation. Note that the support measures given in Table 2.3 are for 10 m span horseshoe shaped tunnel, vertical stress less than 25 MPa and excavated using conventional drilling and blasting procedures.

Table 2.3 Guidelines for Excavation and Support of 10 m span rock tunnels in accordance with the RMR System (Bieniawski, 1989)

Rock Mass Class	**Excavation**	**Rock bolts (20 mm diameter, fully grouted)**	**Shotcrete**	**Steel sets**
I - Very good rock RMR: 81-100	Full face, 3 m advance	Generally no support required except spot bolting		
II - Good rock RMR: 61-80	Full face, 1 – 1.5 m advance. Complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
III - Fair rock RMR: 41-60	Top heading and bench 1.5 - 3 m advance in top heading. Commence support after each blast. Complete support 10 m from face	Systematic bolts 4 m long , spaced 1.5 - 2 m in crown and walls with wire mesh in crown	50 - 100 mm in crown and 30 mm in sides	None
IV - Poor rock RMR: 21-40	Top heading and bench 1.0 – 1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face	Systematic bolts 4-5 m long, spaced 1 – 1.5 m in crown and walls with wire mesh	100 - 150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
V - Very poor rock RMR : < 20	Multiple drifts 0,5 – 1.5 m advance in top heading. Install support concurrently with possible after blasting	Systematic bolts 5 - 6 m long, spaced 1 – 1.5 m in crown and walls with wire mesh. Bolt invert	150 - 200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and fore poling if required. Close invert

2.2.2 Rock Mass Quality (Q) System

Barton et al. (1974) at the Norvegian Geotechnical Institute (NGI) proposed the Rock Mass Quality (Q) System of rock mass classification on the basis of about 200 case histories of tunnels and caverns. It is a quantitative classification system, and it is an engineering system enabling the design of tunnel supports.

The concept upon which the Q system is based upon three fundamental requirements:

a. Classification of the relevant rock mass quality,
b. Choice of the optimum dimensions of the excavation with consideration given to its intended purpose and the required factor of safety,
c. Estimation of the appropriate support requirements for that excavation.

The Q-System is based on a numerical assessment of the rock mass quality using six different parameters:

$$Q = \left(\frac{RQD}{J_n}\right) . \left(\frac{J_r}{J_a}\right) . \left(\frac{J_w}{SRF}\right) \qquad (2.4)$$

where

RQD	is the Rock Quality Designation
J_n	is the joint set number
J_r	is the joint roughness number
J_a	is the joint alteration number
J_w	is the joint water reduction factor
SRF	is the stress reduction factor

The numerical value of the index Q varies in logarithmic scale from 0.001 to a maximum of 1000.

The numerical values of each of the above parameters are interpreted as follows (Barton et al., 1974). The first quotient (RQD/J_n), representing the structure of the rock mass, is a crude measure of the block or particle size. The second quotient (J_r/J_a) represents the roughness and frictional characteristics of the joint walls or filling materials. The third quotient (J_w/SRF) consists of two stress parameters. SRF is a measure of:

i. loosening load in the case of an excavation through shear zones and clay bearing rock,
ii. rock stress in competent rock, and
iii. squeezing loads in plastic incompetent rocks. It can be regarded as a total stress parameter.

The parameter J_w is a measure of water pressure. The quotient (J_w/SRF) is a complicated empirical factor describing the active stress.

Barton et al. (1974) consider the parameters, J_n, J_r, and J_a, as playing a more important role than joint orientation, and if joint orientation had been included, the classification would have been less general. However, orientation is implicit in parameters J_r, and J_a, because they apply to the most unfavorable joints.

The traditional use of the Q-system for rock mass classification and empirical design of rock reinforcement and tunnel support has been extended in several ways in the paper published by Barton (2002a). The classification of individual parameters used to obtain the tunneling Quality Index Q for a rock mass is given in Table 2.4.

Table 2.4 Classification of individual parameters used in the Q System (Barton, 2002a).

A1

Rock quality designation		**RQD (%)**
A	Very poor	0–25
B	Poor	25–50
C	Fair	50–75
D	Good	75–90
E	Excellent	90–100

Notes: (i) Where RQD is reported or measured as ≤10 (including 0), a nominal value of 10 is used to evaluate Q. (ii) RQD intervals of 5, i.e.,100, 95, 90, etc., are sufficiently accurate.

A2

Joint set number		**J_n**
A	Massive, no or few joints	0.5–1
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random, heavily jointed, 'sugar-cube', etc.	15
J	Crushed rock, earthlike	20

Notes: (i) For tunnel intersections, use (3.0 x J_n). (ii) For portals use (2.0 x J_n).

A3

Joint roughness number		**J_r**
(a) Rock-wall contact, and (b) rock-wall contact before 10 cm shear		
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	1.0
G	Slickensided, planar	0.5
(c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact.	1.0
J	Sandy, gravely or crushed zone thick enough to prevent rock-wall contact	1.0

Notes: (i) Descriptions refer to small-scale features and intermediate scale features, in that order. (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. (iii) J_r = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are oriented for minimum strength. (iv) Jr and J_a classification is applied to the joint set or discontinuity that is least favourable for stability both from the point of view of orientation and shear resistance, τ (where $\tau \approx \sigma_n \tan^{-1} (J_r/J_a)$.

Table 2.4 (Continued)

A4

Joint alteration number		ϕ_r approx. (deg)	J_a
(a) Rock-wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote	—	0.75
B	Unaltered joint walls, surface staining only	25–35	1.0
C	Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25–30	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20–25	3.0
E	Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays	8–16	4.0
(b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc.	25–30	4.0
G	Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5mm thickness)	16–24	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5mm thickness)	12–16	8.0
J	Swelling-clay fillings, i.e., montmorillonite (continuous, but <5mm thickness). Value of J_a depends on per cent of swelling clay-size particles, and access to water, etc.	6–12	8–12
(c) No rock-wall contact when sheared (thick mineral fillings)			
KLM	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6–24	6, 8, or 8–12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	—	5.0
	OPR Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6–24	10, 13, or 13–20

A5

Joint water reduction factor		Approx. water pres. (kg/cm^2)	J_w
A	Dry excavations or minor inflow, i.e., <5 l/min locally	<1	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1–2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5–10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5–10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	>10	0.2–0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	>10	0.1–0.05

Notes: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed. (ii) Special problems caused by ice formation are not considered. (iii) For general characterization of rock masses distant from excavation influences, the use of J_w = 1.0, 0.66, 0.5, 0.33, etc. as depth increases from say 0–5, 5–25, 25–250 to >250 m is recommended, assuming that RQD=J_n is low enough (e.g. 0.5–25) for good hydraulic conductivity. This will help to adjust Q for some of the effective stress and water softening effects, in combination with appropriate characterization values of SRF. Correlations with depth dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed.

Table 2.4 Continued.

A6

Stress reduction factor				**SRF**
(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10
B	Single weakness zones containing clay or chemically disintegrated rock) (depth of excavation ≤50 m			5
C	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation >50m)			2.5
D	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)			7.5
E	Single shear zones in competent rock (clay-free), (depth of excavation ≤50 m)			5.0
F	Single shear zones in competent rock (clay-free), (depth of excavation >50m)			2.5
G	Loose, open joints, heavily jointed or 'sugar cube', etc. (any depth)			5.0
		σ_c/σ_1	**σ_θ/σ_c**	**SRF**
(b) Competent rock, rock stress problems				
H	Low stress, near surface, open joints	200	<0.01	2.5
J	Medium stress, favorable stress condition	200–10	0.01–0.3	1
K	High stress, very tight structure. Usually favorable to stability, may be unfavorable for wall stability	10–5	0.3–0.4	0.5–2
L	Moderate slabbing after >1h in massive rock	5–3	0.5–0.65	5–50
M	Slabbing and rock burst after a few minutes in massive rock	3–2	0.65–1	50–200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock	<2	>1	200–400
			σ_θ/σ_c	**SRF**
(c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure				
O	Mild squeezing rock pressure		1–5	5–10
P	Heavy squeezing rock pressure		>5	10–20
				SRF
(d) Swelling rock: chemical swelling activity depending on presence of water				
R	Mild swelling rock pressure			5–10
S	Heavy swelling rock pressure			10–15

Notes: (i) Reduce these values of SRF by 25–50% if the relevant shear zones only influence but do not intersect the excavation. This will also be relevant for characterization. (ii) For strongly anisotropic virgin stress field (if measured): When $5 \leq \sigma_1/\sigma_3 \leq 10$; reduce σ_c to $0.75\sigma_c$: When $\sigma_1=\sigma_3 > 10$; reduce σ_c to $0.5\sigma_c$; where σ_c is the unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ the maximum tangential stress (estimated from elastic theory). (iii) Few case records available where depth of crown below surface is less than span width, suggest an SRF increase from 2.5 to 5 for such cases (see H). (iv) Cases L, M, and N are usually most relevant for support design of deep tunnel excavations in hard massive rock masses, with RQD=Jn ratios from about 50–200. (v) For general characterization of rock masses distant from excavation influences, the use of SRF=5, 2.5, 1.0, and 0.5 is recommended as depth increases from say 0–5, 5–25, 25–250 to >250 m. This will help to adjust Q for some of the effective stress effects, in combination with appropriate characterization values of J_w: Correlations with depth- dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed. (vi) Cases of squeezing rock may occur for depth $H > 350Q^{1/3}$ according to Singh [34]. Rock mass compression strength can be estimated from $\sigma_{cm} \approx 5\gamma Q^{1/3}c$ (MPa) where γ is the rock density in t/m^3, and Qc = Q x σ_c / 100; Barton (2000).

Most recently, some suggestions, related to Q-System, were made by Ünal (2002). These suggestions are based on the experience gained in applying rock mass classification systems. As experienced before, it was quite difficult to apply the Q-System as suggested by Barton et al. (1974). The difficulty arises, especially in determining the joint alteration number (J_a) and stress reduction factor (SRF) parameters during geotechnical logging, which is not defined by Barton et al. (1974). In order to bring a modest solution to this problem Ünal (2002) made some suggestions for J_a and SRF parameters.

In relating the value of the index Q to the stability and support requirements of underground excavations, Barton et al. (1974) defined a parameter that they called Equivalent Dimension, D_e, of the excavation. This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio, ESR.

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio, ESR}} \qquad (2.5)$$

The value of ESR is related to the intended use of the excavation and to the degree of security which is demanded of the support system installed to maintain the stability of the excavation as shown below in Table 2.5.

The equivalent dimension, D_e, plotted against the value of Q, is used to provide 38 support categories in a chart published in the original paper by Barton et al. (1974). This chart has been updated by Grimstad and Barton (1993) to reflect the increasing use of steel fibre reinforced shotcrete in underground excavation support. The reproduced updated Q-support chart (Barton, 2002a) is shown in Figure 2.1.

Table 2.5 Excavation support categories and their ESR values (After Barton et al., 1974).

Excavation Category		ESR Values
A	Temporary mine openings	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for excavations	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, civil defense chambers, portal intersections.	1.3
D	Power stations, major road and railway tunnels, civil defense chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories	0.8

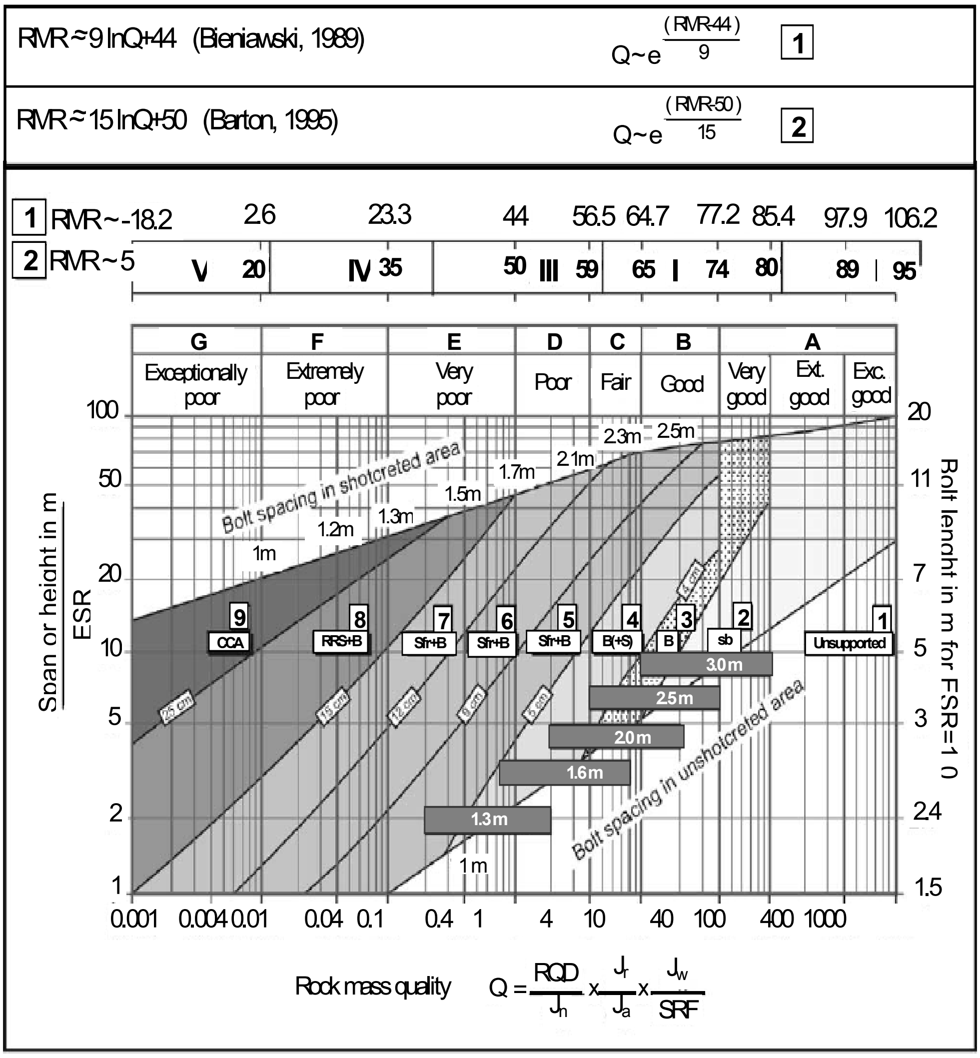
Barton et al. (1980) provide additional information on rock bolt length, maximum unsupported spans and roof support pressures to supplement the support recommendations published in the original 1974 paper.

The length (L) of rockbolts can be estimated from the excavation width (B) and the Excavation Support Ratio (ESR):

$$\mathrm{L} = \frac{2 + 0.15B}{ESR} \tag{2.6}$$

The maximum unsupported span can be estimated from the following expression:

$$\text{Maximum unsupported span} = 2 \,.\, \mathrm{ESR} \,.\, \mathrm{Q}^{0.4} \tag{2.7}$$



REINFORCEMENT CATEGORIES

1. Unsupported.
2. Spot bolting (Sb).
3. Systematic bolting (B).
4. Systematic bolting with 40-100 mm unreinforced shotcrete.
5. Fibre reinforced shotcrete (S(fr)), 50-90 mm, and bolting.
6. Fibre reinforced shotcrete, 90-120 mm, and bolting.
7. Fibre reinforced shotcrete, 120-150 mm, and bolting.
8. Fibre reinforced shotcrete, >150 mm, with reinforced ribs of shotcrete and bolting.
9. Cast concrete lining (CCA).

Figure 2.1 The 1993 updated Q-support chart for selecting permanent B+S(fr) reinforcement and support for tunnels and caverns in rock. The black, highlighted areas show where estimated Q-values and stability are superior in TBM tunnels compared to drill-and-blast tunnels. This means ‘nosupport’ penetrates further (After Barton, 2002a).

Based upon analyses of case records, Grimstad and Barton (1993) suggest that the relationship between the value of Q and the permanent roof support pressure P is estimated from:

$$P = \frac{2\sqrt{J_n}Q^{-1/3}}{3J_r} \tag{2.8}$$

The original Q-based empirical equation for underground excavation support pressure (Barton et al., 1974), when converted from the original units of kg/cm^2 to MPa, is expressed as follows (Barton, 2002a):

$$P = \frac{J_r}{20xQ^{1/3}} \tag{2.9}$$

2.2.3 Geological Strength Index (GSI)

One of the major problems in designing underground openings is estimating the strength parameters of in situ rock mass. The strength and deformation modulus of closely jointed rock masses cannot be directly determined, since the dimensions of representative specimens are too large for laboratory testing. This limitation results in an important difficulty when studying in jointed rock masses. Hoek and Brown (1980) suggested an empirical failure criterion to overcome this difficulty. The rock mass rating (RMR) classification was introduced into the Hoek–Brown criterion by its originators (Hoek and Brown, 1988) to describe the quality of rock masses. This empirical criterion has been re-evaluated and expanded over the years due to the limitations both in Bieniawki's RMR classification and the equations used by the criterion for very poor-quality rock masses (Hoek, 1983, 1990, 1994; Hoek and Brown, 1988, 1997; Hoek et al., 1992, 2002).

Hoek (1994), Hoek et al (1995), and Hoek and Brown (1997) proposed a new rock mass classification system called "Geological Strength Index, GSI" as a replacement for Bieniawski's RMR to eliminate the limitations rising from the use of RMR classification scheme. The GSI System seems to be more practical than the other classification systems such as Q and RMR when used in the Hoek–Brown failure criterion. Therefore, the GSI value has been more popular input parameter for the Hoek–Brown criterion to estimate the strength and deformation modulus of the jointed rock masses.

In the original form of the GSI System (Hoek and Brown, 1997), the rock mass is classified into 20 different categories with a letter code based upon the visual impression on the rock mass and the surface characteristics of discontinuities and the GSI values ranging between 10 and 85 are estimated. Two additional rock mass categories, is called foliated / laminated rock mass structure and massive or intact rock, were introduced into the GSI system by

Hoek et al. (1998) and Hoek (1999), respectively. Due to the anisotropic and heterogeneous nature of the foliated/laminated rock mass structure category, Marinos and Hoek (2001) also proposed a special GSI chart only for the classification of the heterogeneous rock masses such as flysch.

However, the GSI classification scheme, in its existing form, leads to rough estimates of the GSI values (Sönmez and Ulusay, 1999). Therefore, Sönmez and Ulusay (1999) made an attempt for the first time to provide a more quantitative numerical basis for evaluating GSI as a contributory use of the GSI system by introducing new parameters and ratings, such as surface condition rating (SCR) and structure rating (SR). In this modification, the original skeleton of the GSI System has been preserved, and SR and SCR are based on volumetric joint count (J_v) and estimated from the input parameters of RMR scheme (e.g. roughness, weathering and infilling). Then this chart was slightly modified by Sönmez and Ulusay (2002) and defined by fuzzy sets by Sönmez et al. (2003). In this version of the quantitative GSI chart, intact or massive rock mass included into the system as previously suggested by Hoek (1999) are given in Figure 2.2.

In recent years, the GSI system has been used extensively in many countries and lots of studies have been done to quantify GSI system parameters to better classify jointed rock masses for engineering purposes. The quantified GSI chart, building on the concept of block size and condition, developed by Cai. et al. (2003), and fuzzy-based quantitative GSI chart of Sönmez et al. (2004a) are results of some of these studies. A computer program "***RocLab***" was developed (Hoek et al., 2002) to determine the rock mass strength parameters (m,s, c, Ø, E_m etc.) by using GSI.

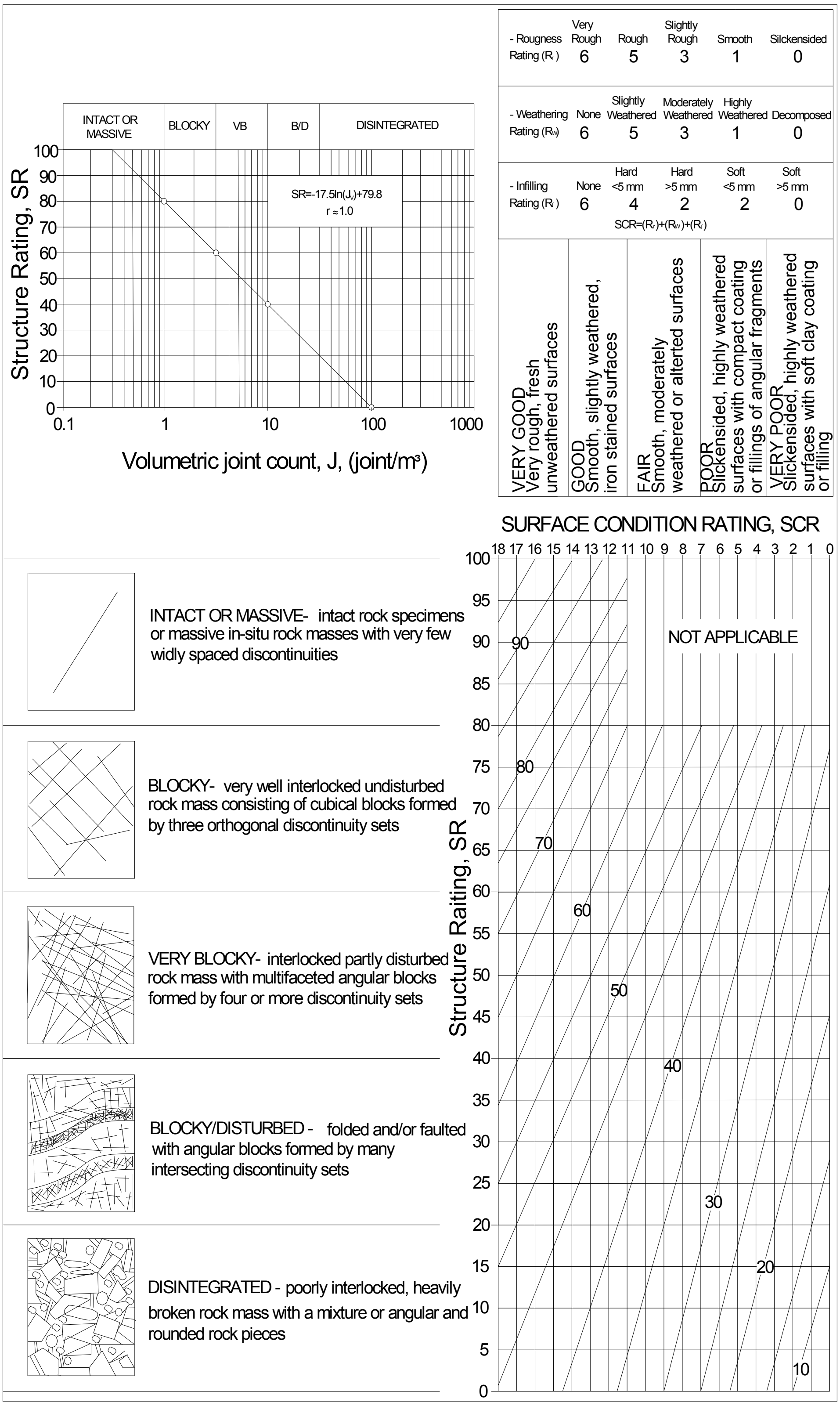


Figure 2.2 The modified GSI classification suggested by Sönmez and Ulusay (2002).

2.2.4 The New Austrian Tunneling Method (NATM)

The New Austrian Tunneling Method (NATM) was developed by Rabcevicz, Müller and Pacher between 1957 and 1965 in Austria. NATM features a qualitative ground classification system that must be considered within the overall context of the NATM (Bieniawski, 1989).

The NATM is based on the philosophy of "*Build (*or *Design) as you go*" approach with the following caution.

"*Not too stiff, Nor too flexible*
Not too early, Nor too late"

In essence, NATM is an approach or philosophy integrating the principles of the behaviour of rock masses under load and monitoring the performance of underground excavations during construction. The NATM is not a set of specific excavation and support techniques. It involves a combination of many established ways of excavation and tunneling, but the difference is the continual monitoring of the rock movement and the revision of support to obtain the most stable and economical lining. However, a number of other aspects are also pertinent in making the NATM more of a concept or philosophy than a method (Bieniawski, 1989).

Müller (1978) considers the NATM as a concept that observes certain principles. Although he has listed no less than 22 principles, there are seven most important features on which the NATM based (Bieniawski, 1989):

1. *Mobilization of the Strength of the Rock Mass.* The method relies on the inherent strength of the surrounding rock mass being conserved as the main component of the tunnel support. Primary support is directed to enable the rock to support itself. It follows that the support must have suitable load deformation characteristics and be placed at the correct time.

2. *Shotcrete Protection.* In order to preserve the load-carrying capacity of the rock mass, loosening and excessive rock deformations must be minimized. This is achieved by applying a thin layer of shotcrete, sometimes together with a suitable system of rock bolting, immediately after face advance. It is essential that the support system used remains in full contact with the rock and deforms with it. While the NATM involves shotcrete, it does not mean that the use of shotcrete alone constitutes the NATM.

3. *Measurements.* The NATM requires the installation of sophisticated instrumentation at the time the initial shotcrete lining is placed, to monitor the deformations of the excavation and the buildup of load in the support. This provides information on tunnel stability and permits optimization of the formation of a load-bearing ring of rock strata. The timing of the placement of the support is of vital importance.

4. *Flexible Support.* The NATM is characterized by versatility and adaptability leading to flexible rather than rigid tunnel support. Thus, active rather than passive support is advocated, and strengthening is not by a thicker concrete lining but by a flexible combination of rock bolts, wire mesh, and steel ribs. The primary support will partly or fully represent the total support required and the dimensioning of the secondary support will depend on the results of the measurements.

5. *Closing of Invert.* Since a tunnel is a thick walled tube, the c1osing of the invert to form a load-bearing ring of the rock mass is essential. This is crucial in soft-ground tunneling, where the invert should be closed quickly and no section of the excavated tunnel surface should be left unsupported even temporarily. However, for tunnels in rock, support should not be installed too early since the load-bearing capability of the rock mass would not be fully mobilized. For rock tunnels, the rock mass must be permitted to deform sufficiently before the support takes full effect.

6. *Contractual Arrangements.* The preceeding main principles of the NATM will only be successful if special contractual arrangements are made. Since the NATM is based on monitoring measurements, changes in support and construction methods should be possible. This, however, is only possible if the contractual system is such that changes during construction are permissible (Spaun, 1977).

7. *Rock Mass Classification Determines Support Measures.* Payment for support is based on a rock mass classification after each drill and blast round. In some countries this is not acceptable contractually, and this is why the method has received limited attention in the United States.

According to NATM, the rock mass is classified without a numerical quality rating; ground conditions are described qualitatively. The Austrian ONORM B2203 of October 1994 is based on the suggestions by Rabcewicz et al. (1964). The main rock mass classes and behaviour of rock masses for each rock mass group according to the ONORM B2203 are given in Table 2.6.

A critical analysis of the principles of the complete New Austrian Tunneling Method (NATM) "edifice of thoughts" has been published by Kovari (1994). The author claimed that: "The NATM is based on two basic erreneous concept". The most recently published paper by Kovari (2004) traces the fascinating history of rock bolts and the NATM or the sprayed concrete lining method from its beginnings and shows how it developed on a broad international front in its theoretical and technological aspects. This paper describes numerous examples of civil engineering work worldwide with early application of rock bolting. In concluding, it is demonstrated that NATM is in many respects borrowed and has created much confusion amongst professional engineers by dint of its pseudo-scientific basis (Kovari, 2004).

Table 2.6 NATM Rock Mass Classes (Geoconsult, 1993 and ONORM B 2203, 1994).

Rock Mass Class	Behaviour of Rock Mass		Explanations
	ONORM B 2203 After Oct. 1994	ONORM B 2203 Before Oct. 1994	
A	A1 Stable	A1 Stable	The rock mass behaves elastically. Deformations are small and decrease rapidly. There is no tendency of overbreaking after scaling of the rock portions disturbed by blasting. The rock mass is permanently stable without support.
	A2 Sligthly Overbreaking	A2 Sligthly Overbreaking	The rock mass behaves elastically. Deformations are small and decrease rapidly. A slight tendency of shallow overbreaks in the tunnel roof and in the upper portions of the sidewalls caused by discontin- uities and the dead weight of the rock mass exists.
B	B1 Friable	B1 Friable	Major parts of the rock mass behave elastically. Deformations are small and decrease rapidly. Low rock mass strength and limited stand-up times related to the prevailing discontinuity pattern yield overbreaks and loosening of the rock strata in tunnel roof and upper sidewalls if no support is installed in time.
	B2 Very Friable	B2 Very Friable	This type of rock mass is characterised by large areas of nonelastic zones extending far into the surrounding rock mass. Immediate installation of the tunnel support, will ensure deformations can be kept small and cease rapidly. In case of a delayed installation or an insufficient quantity of support elements, the low strength of the rock mass yields deep loosening and loading of the initial support. Stand-up time and unsupported span are short. The potential of deep and sudden failure from roof, sidewalls and face is high.
	B3 Rolling		
C	C1 Rock Bursting	C1 Squeezing	C1 is characterized by plastic zones extending far into the surrounding rock mass and failure mechanisms such as spalling, buckling, shearing and rupture of the rock structure, by squeezing behaviour or by tendency rock burst. Subject rock mass shows a moderate, but distinct time depending squeezing behaviour; deformations calm down slowly except in case of rock bursts. Magnitude and velocity of deformations at the cavity boundary are moderate.
	C2 Squeezing		
	C3 Heavily Squeezing	C2 Heavily Squeezing	C2 is characterized by the development of deep failure zones and a rapid and significant movement of the rock mass into the cavity and deformations which decrease very slowly. Support elements may frequently be overstressed.
	C4 Flowing	L1 Short-term-stable with high cohesion	By limitation of the unsupported spans at arch and face, the rock mass remain stable for a limited time.
	C5 Swelling	L2 Short-term-stable with low cohesion	No stand up time without support by prior installation of forepolling or forepiling and shotcrete sealing of faces simultaneously with excavation. The low cohesion requires a number of subdivisions.

2.2.5 Correlations between the RMR, Q, GSI and NATM

The RMR, Q and GSI classification systems are based on the quantitative properties of rock mass, but NATM is qualitative classification system. However, the basic idea of the support systems is close to each other. For the tunnel design, these classification systems are used together as empirical aproach.

Various empirical correlations have been made between RMR and Q classification in previous studies. The most popular and applicable one is proposed by Bieniawski (1976) is given in Table 2.5. Also different correlations proposed between GSI and RMR (Hoek, et al., 1995) and GSI and Q (Hoek, et al., 1995) as given in Table 2.7.

Table 2.7 Correlations between classification systems (RMR,Q and GSI)

Originator of empirical equation	Equation
Bieniawski (1976)	RMR = 9 lnQ + 44
Hoek et al. (1995)	$GSI = RMR_{76}$ (use of 1976 version of RMR) $GSI = RMR_{89} - 5$ (use of 1989 version of RMR)
Hoek et al. (1995)	GSI = 9 lnQ' + 44 (Q': $\frac{RQD}{Jn} \frac{Jr}{Ja}$)

Most widely used empirical correlation between RMR, Q and NATM is presented in Figure 2.3 providing relation between quantitative properties of rock mass , based on RMR and Q and suggested empirical excavation and support system acoording to the NATM.

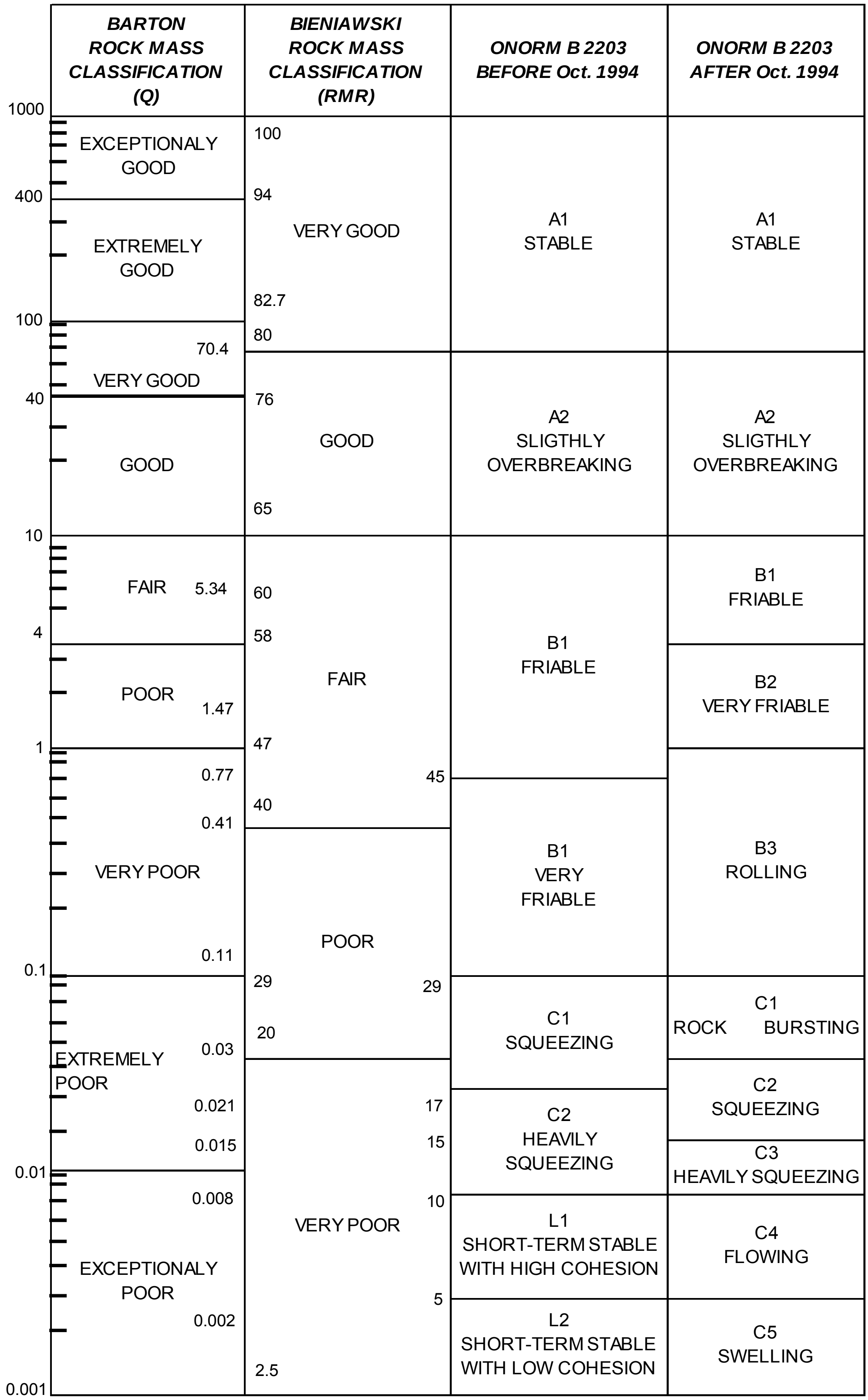


Figure 2.3 Correlations between classification systems (RMR,Q and NATM)

2.3 Estimation of Rock Mass Strength and Deformation Modulus

One of the major problems in designing underground openings is estimating the strength parameters of in-situ rock mass. Estimation of the strength of closely jointed rock masses is difficult since the size of representative specimens sometimes is too large for laboratory testing.

This difficulty can be overcome by using the Hoek-Brown failure criterion. Since its introduction in 1980, the criterion has been refined and expended over the years (1983, 1988. 1992, 1995, 2002, 2006). A brief history of the development of the Hoek-Brown failure criterion and summary of equations, which are used for estimation of rock mass strength parameters are published by Hoek (2006).

The results of the back analysis of the slope instabilities in closely jointed rock masses by Sönmez and Ulusay (1999 and 2002) indicated that the disturbance effect due to the influence of the method of excavation could not be ignored. For this reason, a disturbance factor, which should be used in the determination of rock mass constants considered by the Hoek-Brown failure criterion, was suggested by these investigators.

The latest version of Hoek-Brown failure criterion was proposed by Hoek et al. (2002, 2006). It represents a major re-examination of the entire Hoek-Brown failure criterion and new derivations of the relationships between rock mass strength parameters (m, s) and GSI. A disturbance factor (D), which is also considered by the empirical equation for estimating the deformation modulus of rock masses in conjuction with the GSI, was also included to deal with blast damage. The guidelines for estimating disturbance factor D are given in Appendix D , Figure D.1 with Hoek-Brown Failure Criterion 2002. Also a computer program ***RocLab***, which includes all of these new derivations, was developed to determine the rock mass strength parameters (m,s, c, Ø, E_m etc.) by using GSI.

The deformation modulus (E_m) of a rock mass is another important parameter in any form of numerical analysis and in the interpretation of monitored deformation around underground openings. Since this parameter is very difficult and expensive to determine in the field, several attempts have been made to develop methods for estimating its value, based upon rock mass classifications (Hoek et al., 1995).

The first empirical model for prediction of the deformation modulus of rock masses was developed by Bieniawski (1978). After Bieniawski's empirical equation, some other empirical approaches such as Barton et al. (1980), Serafim and Pereira (1983), Nicholson and Bieniawski (1990), Mitri et al. (1994), Hoek and Brown (1997), Palmström and Singh (2001), Barton (2002), Hoek, et al. (2002) and Kayabaşı et al. (2003) have been proposed to estimate the deformation modulus of rock masses. Such empirical approaches are open to improvement because they are based on limited collected data.

The equations proposed by Bieniawski (1978), Serafim and Pereira (1983), Nicholson and Bieniawski (1990) and Mitriet al. (1994) consider Bieniawski's RMR (1989) while Barton's equation (1980, 2002b) estimates the deformation modulus by considering the Q-values. The equation proposed by Hoek and Brown (1997, 2002) is a modified form of Serafim and Pereira's equation (1983) and it is based on the GSI and a new constant D (disturbance factor). Palmström and Singh (2001) also suggested an empirical equation depending on RMi (Palmström, 1996) values for the prediction of deformation modulus. Kayabaşı et al. (2003) proposed the most recent empirical equation by considering the RQD, elasticity modulus of intact rock and weathering degree for estimating the deformation modulus of rock masses. Recently, with the study conducted by Gökçeoğlu et al. (2003), the prediction performance of the existing empirical equations was checked and some contributions to the work of Kayabaşı et al. (2003) was provided. Therafter, a prediction model, based on an approach which considers that modulus ratios of the rock mass and intact rock should be theoretically equal to each other when GSI is equal to 100, was developed by Sönmez et. al. (2004a).

Most recently, in close periods, Hoek and Diederichs (2006) and Sönmez et. al. (2006b) have improved the empirical relation about modulus of deformability based on elasticity of intact rock material E_i , GSI , RMR , disturbance factor D and mass reduction factor r_f. Appendices E and F present these two approaches with its original paper of Hoek and Diederichs (2006) in Appendix E and with two important graphs evaluating the mass reduction factor r_f depending on disturbance factor D and Elasticity Modulus E_i of intact rock of Sönmez et. al. (2006b) in Appendix F.

Mostly known and widely used empirical equations for the estimation of deformation modulus in the history are given in Table 2.8.

Table 2.8 List of empirical equations suggested for estimating the deformation modulus with required parameters and limitations (Sönmez and Ulusay, 2007)

Originator of empirical equation	Required parameters	Limitations	Equation
Bieniawski (1978)	RMR	RMR > 50	E_m= 2RMR-100 GPa
Serafim and Pereira (1983)	RMR	RMR ≤ 50	$E_m=10^{[(RMR-10)/40]}$ GPa
Barton (2002)	Q, σ_c	σ_c ≤100MPa	$E_m= 10[(\sigma_c /100)Q]^{1/3}$ GPa
Hoek et al. (2002)	GSI, σ_c, D	σ_c≤100 MPa	$E_m= [1-(D/2)]\sqrt{(\sigma_c/100)}\, 10^{(GSI-10)/40}$ GPa
		σ_c>100 MPa	$E_m= [1-(D/2)]10^{(GSI-10)/40}$ GPa
Kayabaşı et al. (2003)	E_i, RQD , WD	-	$E_m= 0.135 [(E_i (1 + RQD / 100)) / WD]^{1.1811}$ GPa
Gökçeoğlu et al. (2003)	E_i, RQD, WD, σ_c	-	$E_m= 0.001 [((E_i /\sigma_c)(1 + RQD / 100))/ WD]^{1.5528}$ GPa
Sönmez et al. (2004a)	E_i,s,a	-	$E_m= E_i (s^a)^{0.4}$ GPa
Hoek and Diederichs (2006)	GSI,D	-	$E_m=100\,000[(1-(D/2))/(1+e^{((75+25D-GSI)/11)})]$ MPa
Hoek and Diederichs (2006)	E_i,GSI,D	-	$E_m = E_i [(1-(D/2))/(1+e^{((60+15D-GSI)/11)})]$ GPa
Sönmez et al. (2006b)	Ei,RMR,r_f	-	$E_m = E_i\, 10^{[((RMR_d-100)(100-RMR_d)/4000\, exp(-RMR_d/100))]}$ GPa $RMR_d= GSI_d =r_f x (RMR_{89} -5)$

2.4 Analytical Design Methods

The construction material “rock” is a natural, non-homogeneous material. In most cases, rock deformations are of plastic nature, or at least partly elastic or partly plastic. Mathematical modeling of main support elements like shotcrete, rock bolts, etc. is also very complex and still unsatisfactory. It is well recognized that rock and shotcrete show in general a very distinct rheological behaviour.

Therefore, approximations and simplifications must be made in mathematical modeling of rock tunneling, in particular for application of closed form solutions. The result of that kind of computations cannot be exactly conform with the reality. That is why analytical method in rock engineering is mainly used for comparative design and parametric studies and must be supplemented by other design approaches for solving practical engineering problems.

The earliest analysis of the elasto-plastic stress distribution around a cylindrical opening was published by Terzaghi (1925) but this solution did not include a consideration of support interaction. Fenner (1938) published the first attempt to determine support pressures for a tunnel in rock mass in which elasto-plastic failure occurs. Brown et al. (1983) and Duncan (1993) have reviewed several of the analytical solutions which have been published since 1938. The major difference between these solutions lies in the assumed post failure characteristics of the rock mass surrounding the tunnel. All these solutions are restricted to the case of cylindrical opening in a rock mass subjected to hydrostatic stress field.

The stress field in the rock surrounding most mining and civil engineering excavations is not hydrostatic and very few of these excavations are circular in shape. Consequently, practical applications of the analytical solutions are severly limited. The main value of these solutions is the understanding of basic principals of rock support interaction which can be gained from parametric studies involving different material properties, in stress levels and support systems.

In order to overcome the limitations of analytical solution and to provide practical design calculations, numerical design methods have been taken into consideration in the last few decades.

2.5 Numerical Design Methods

2.5.1 Introduction

As many complex structures could not be defined analytical, methods of discretisation for such problems were searched to solve these structures. Figure 2.4 shows the “Family tree” of Finite Element Methods (Zienkiewicz,2005).

From the beginning it was known that the occuracy of the solution increases with the degree of discretisation, i.e. the results come closer to the analytical solution, if the number of elements increases.

Up to early 1960’s the main problem for the solution of ‘discretised structures’ was the missing of suitable ‘machineries’ to solve the problems accurate enough.

Since that time much progress has been made, as the available ‘machines’, the computers increased their capabilities and hand in hand became cheaper. Parallel to this development the possibilities to apply more complex material models to the discretisation procedure became possible.

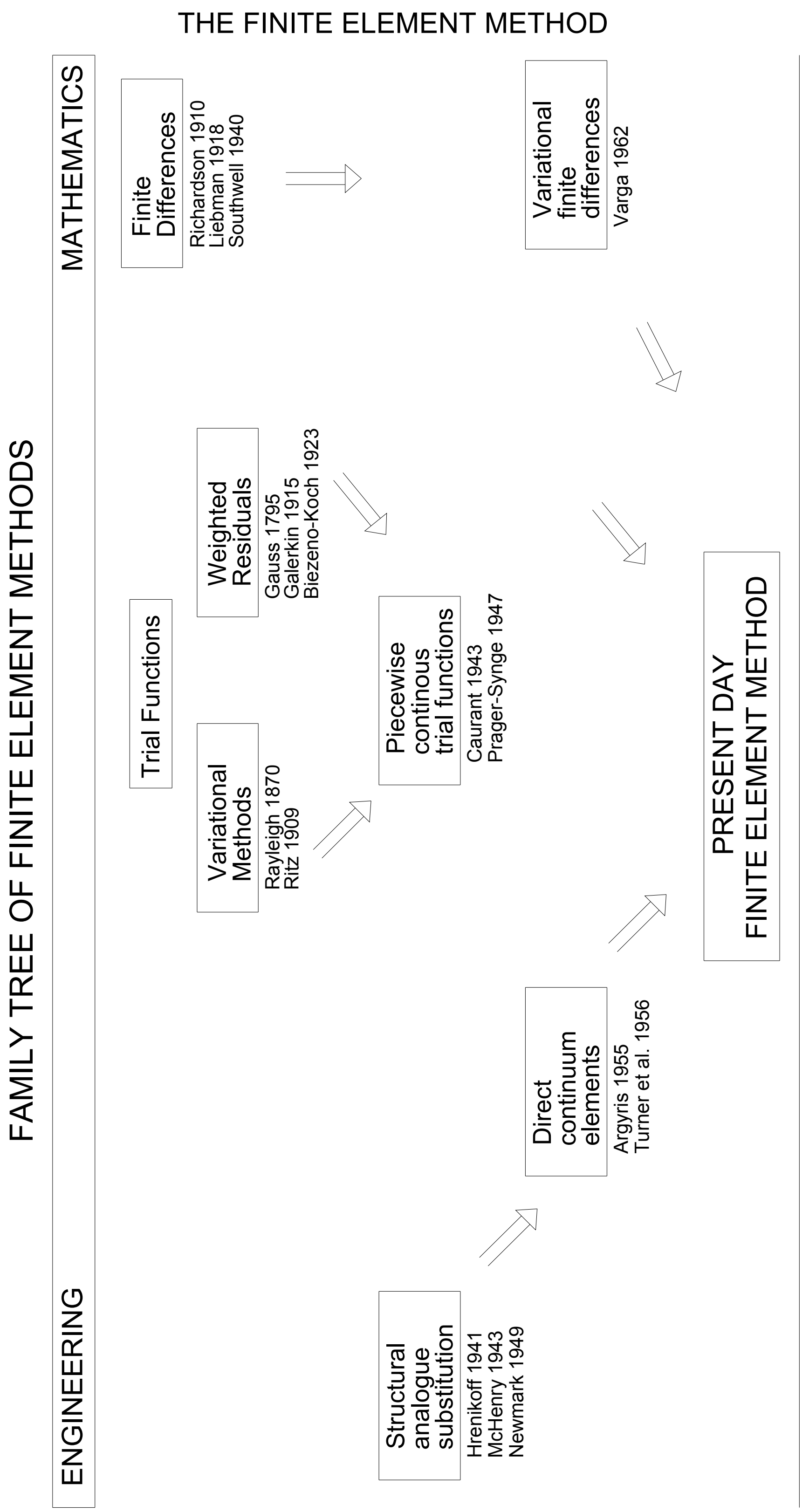


Figure 2.4 Development of finite element method (Zienkiewicz, 2005)

2.5.2 Aplication of Numerical Models for Geomechanics

The rapid development of computers made it possible to apply more complex material behavior to numerical methods, especially the Finite Element Method. Especially geomechanic problems provide a wealth of problems for the numerical mathematician. So it was logical sequence and only a question of time for the application of this method for geomechanics.

Two approaches to numerical modeling of rock masses can be identified, both recognizing geological structures as being discontinuous due to joints, faults and bedding planes. A continuum approach treats the rock mass as a continuum intersected by a number of discontinuities, while a discontinuum approach views the rock mass as an assemblage of independent blocks or particles (Goodman and Shi, 1985)

2.5.2.1 Discontinuum Models

These models feature numerical procedures involving the equations of motion of particles or blocks rather than the continuum (Cundall and Hart, 1993). Discontinuum models should be used whenever independent rock block movements must be specially recognized.

In the beginning of the application of these models for engineering work only pure elastic problems were solved. Today's models already can cover a wide range of material problems in Geomechanics, but still some work has to be done to make them more accurate.

In essence, for geomechanical purposes, a wide range of two and three dimensional modeling capabilities is available with computer codes developed for many material behaviours. This includes the constitutive behaviours such as linear elastic, non-linear elastic, linear visco-elastic, elastoplastic, anisotropic, dilatant, thermal-dependent and stochastic.

However much more model application and verification are required for the future. The reason for this situation is that numerical techniques have outstripped the ability of engineering geology to provide necessary input parameters. It is, therefore, essential that full consideration be given to the availability of realistic input data before applying sophisticated numerical methods.

2.5.2.2 Continuum Models

There are two types of continuum models,

- Integral
- Differential

Integral or Boundary Element Models (BEM) feature discretization only along interior or exterior boundaries (Brebbia and Walker, 1978). The interfaces between different material types and discontinuities are treated as internal boundaries which must be similarly discretized. Boundary element procedures are the most appropriate for modeling linear elastic system, although certain forms of nonlinearity may be treated. The boundary element procedures provide an economical means of two- and three- dimensional analysis of rock masses. They are particularly suitable for use when conditions at the boundary are of most concern (St. John et. al.,1979)

Differential models characterize the entire region of interest and include Finite Difference Method (FDM) and Finite Element Method (FEM). The Finite Element Method is uniquely capable of handling complex geometries and inhomogenities. Complex systems involving fluid flow and heat transfer can also be handled (Zienkiewicz, 2005)

2.5.3 Application of Finite Element Method for Tunnel Design

While, finite element methods have been used in many fields of engineering practice for over many years, it is only relatively recently that it has begun to be widely used for analysing geotechnical problems in tunnel engineering. This is probably because there are many complex issues which are specific to tunnel engineering and which have only been resolved relatively recently.

Many attempts for analytical solutions were made, but could never fit the requirements to simulate the possible factors influencing the excavation of underground structures. A list of such items without any demand on completeness may demonstrate the above state.

- Primary stress conditions
- Stress distribution after excavation/carrying rock arch
- Influence of the ‘allowed preformation’
- Influence of time dependent installation of support measures
- Description of rock and soil characteristics as:
    - -Failure mechanism/behavior after failure/fractioning etc.
    - -Creep phenomena for special rocks
    - -Material properties for shotcrete

This list could be extended for a large number of items, which cannot be covered by analytical methods but only with the help of numerical models using a discretised area represented by a finite number of elements, where application of above listed items can be done more easily than for the whole structure.

The finite element method is currently by far the most popular and useful technique for application as an analytical design tool. This method is particularly usefull in assessing the merits of various design schemes on a comparative basis. In this respect it represents a powerful technique where the effects of various parameters on the overall design may be studied and design variations may be compared with one another.

Using a FE program with a high analytical capacity for tunnel design it is necessary to choose an appropriate model for simulation of the specific rock behaviour. Normally, a two-dimensional analysis based on the assumption of plain strain condition is an adequate tool to simulate excavation procedures and the interaction between ground and rock.

Usually a continuum model is suitable to simulate the surrounding rock and interaction with the installed support. The rock continuum and the support are divided into small segments, connected by a number of nodes. Within each element the displacements are approximated via interpolation functions (shape functions) using element nodal point displacements. The status at the boundary of elements is approximated in the same manner. For this purpose a local coordinate system is introduced for each element. Transformation from the local to global system is performed by using the shape function.

In this study, Finite Element Program ***Phase***^{2} (version 6.028, Rocscience Inc., Toronto - Canada, 2008) is used to verify and check the validity of empirically determined excavation and support systems. ***Phase***^{2} is a worldwide used 2-dimensional elasto-plastic finite element program for calculating stresses and displacements around underground openings. Complex, multi-stage models can be created and analyzed with this program such as tunnels in poor or jointed rock, underground powerhouse caverns, open pit mines etc. ***Phase***^{2} offers a wide range of support modeling options. Liner elements can be applied in the modeling of shotcrete, concrete, steel set systems and multi-layer composite liners. Material models for rock and soil include Mohr-Coulomb, Generalized Hoek-Brown and Cam-Clay.

CHAPTER III

GENERAL DESCRIPTION OF ÇUBUKBELİ TUNNEL ROUTE AND ENGINEERING GEOLOGY

3.1 Introduction

This chapter comprises general information about Çubukbeli Tunnel and geological and geotechnical studies carried out around the project area which provides base information for rock mass classifications.

3.2 General Description of the Tunnel Route and Geometry

Antalya-Burdur-Keçiborlu-Sandıklı State Highway, on which Çubukbeli Tunnel is situated, is one of the main transportation arter, providing the link between middle and north Anatolia - Antalya. The tunnel area is located approximately 50 km. north of Antalya City Center where uncomfortable and unsafe traffic flow is present because of the inconvenient topographical conditions. (Figure 3.1)

The twin tube unidirectional Çubukbeli tunnel is located on Antalya-Burdur-Keçiborlu-Sandıklı State Highway between Km: 42+607- Km:44+592 and has a length of 1985 m. The horizontal geometry consists of a curve with R=3000 m radius in the middle part. The rest part is composed of alignment.

Figure 3.1 Location map of the tunnel area

The vertical geometry has 2.38 % longitudinal slope. Both horizontal and vertical geometry are suitable for a twin tube highway tunnel providing optimum traffic flow, comfort and safety. Figures 3.2 and 3.3 show the typical crossection of the tunnel.

The selection of optimum distance between tubes is a very important paratemer both for the design of route geometry and support. Since, greater distance causes greater open-cut excavation volume at portals, and smaller distance may cause interaction of tubes so that excavation and support of each tube may be problematic especially in poor rock conditions. In Çubukbeli Tunnel, the distance between tubes is selected as 18 m (approx. 1.5 times of width), which is an optimum value for the minimization of open-cut excavation and elimination of adverse effects of excavation and support of each tube on one another. This is verified by finite element analysis in Chapter 5.

According to KGM (General Directorate of Highways) specifications (Karayolu Teknik Şartnamesi, 2006) since the length of tunnel is more than 1000 m, an emergency vehicle passage at middle part and two emergency pedestrian passages at both ends of tunnel route are located. The distance between emergency passages do not exceed 500 m according to Tunnel Safety Specifications of KGM.

The plan and profile sheets of the tunnel area showing the geology, topography, entrance and exit portals, and their excavation geometries, horizontal and vertical geometry of the tunnel route, overburden thickness, distance between tubes, location of emergency pedestrian passages etc. are presented in Figure 3.4 and Figure 3.5.

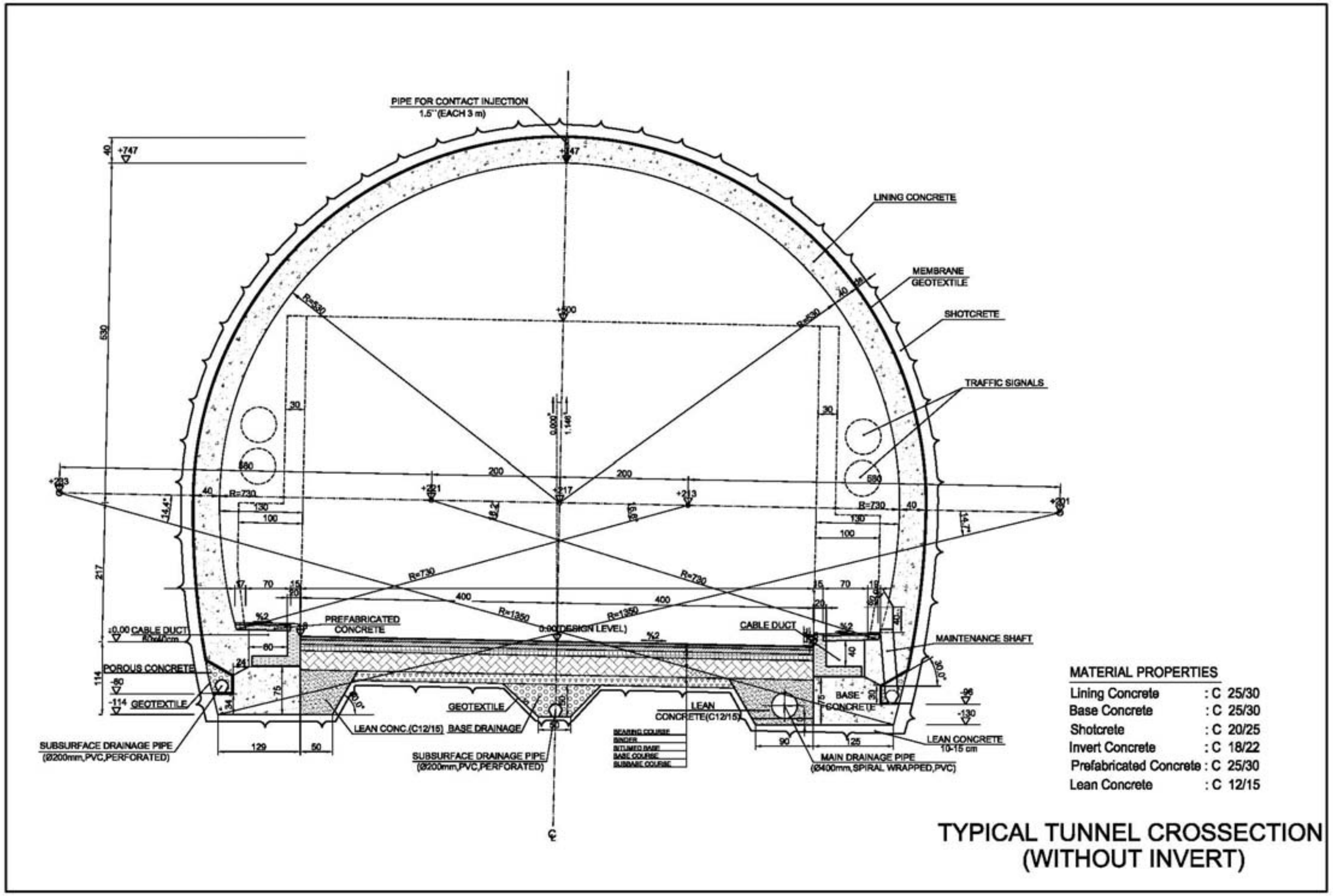


Figure 3.2 Typical tunnel crossection (without invert)

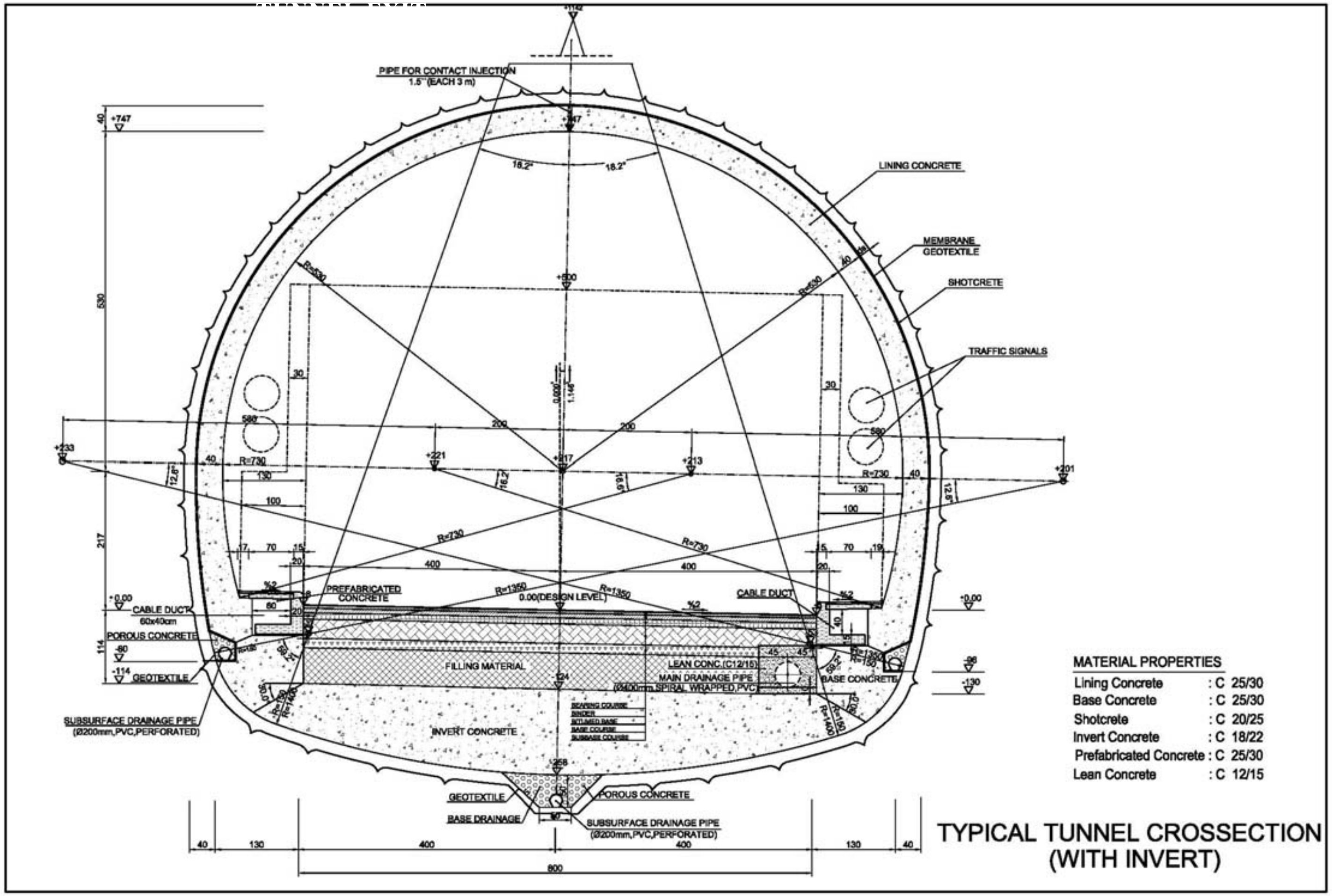
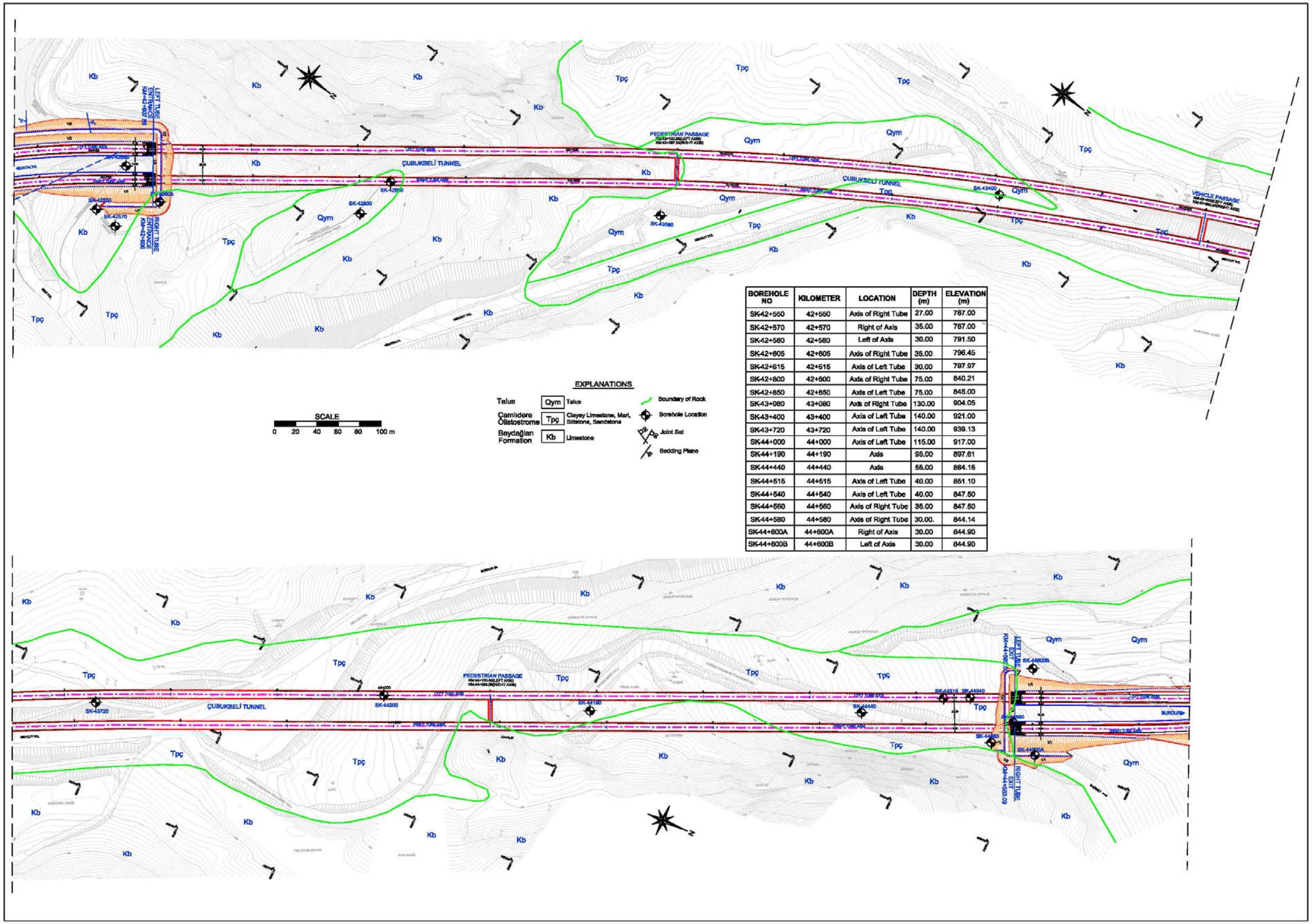
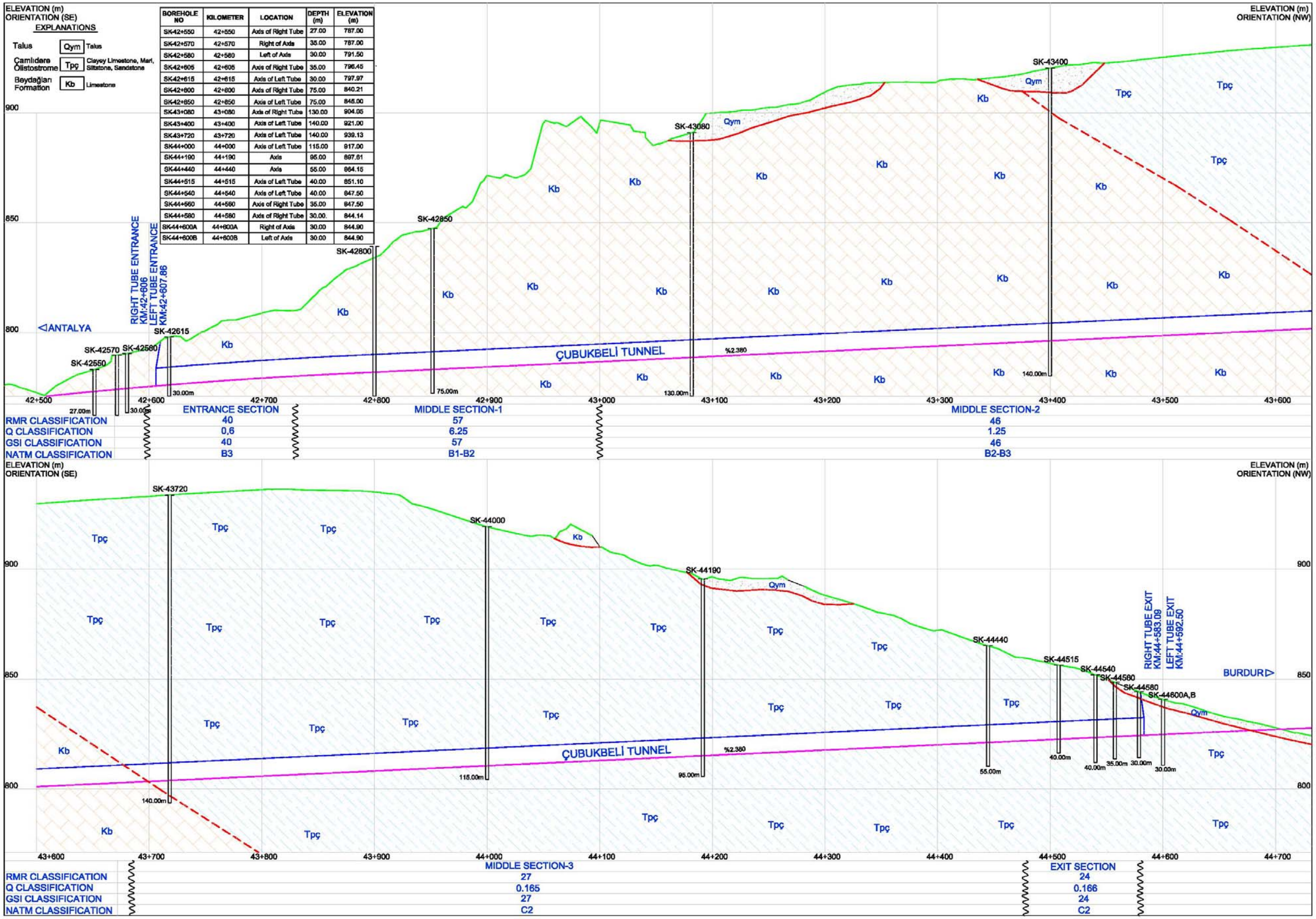


Figure 3.3 Typical tunnel crossection (with invert)



BOREHOLE NO	KILOMETER	LOCATION	DEPTH (m)	ELEVATION (m)
SK42+550	42+550	Axis of Right Tube	27.00	767.00
SK42+570	42+570	Right of Axis	35.00	787.00
SK42+580	42+580	Left of Axis	30.00	791.50
SK42+605	42+605	Axis of Right Tube	35.00	796.45
SK42+615	42+615	Axis of Left Tube	30.00	797.97
SK42+800	42+800	Axis of Right Tube	75.00	840.21
SK42+850	42+850	Axis of Left Tube	75.00	845.00
SK43+080	43+080	Axis of Right Tube	130.00	904.05
SK43+400	43+400	Axis of Left Tube	140.00	921.00
SK43+720	43+720	Axis of Left Tube	140.00	939.13
SK44+000	44+000	Axis of Left Tube	115.00	917.00
SK44+190	44+190	Axis	95.00	897.61
SK44+440	44+440	Axis	55.00	864.15
SK44+515	44+515	Axis of Left Tube	40.00	851.10
SK44+540	44+540	Axis of Left Tube	40.00	847.50
SK44+560	44+560	Axis of Right Tube	35.00	847.50
SK44+580	44+580	Axis of Right Tube	30.00.	844.14
SK-44+600A	44+600A	Right of Axis	30.00	844.90
SK-44+600B	44+600B	Left of Axis	30.00	844.90

Figure 3.4 Geological strip map of Çubukbeli tunnel



BOREHOLE NO	KILOMETER	LOCATION	DEPTH (m)	ELEVATION (m)
SK-42+550	42+550	Axis of Right Tube	27.00	787.00
SK-42+570	42+570	Right of Axis	35.00	787.00
SK-42+580	42+580	Left of Axis	30.00	791.50
SK-42+605	42+605	Axis of Right Tube	35.00	796.45
SK-42+615	42+615	Axis of Left Tube	30.00	797.97
SK-42+800	42+800	Axis of Right Tube	75.00	840.21
SK-42+850	42+850	Axis of Left Tube	75.00	845.00
SK-43+080	43+080	Axis of Right Tube	130.00	904.05
SK-43+400	43+400	Axis of Left Tube	140.00	921.00
SK-43+720	43+720	Axis of Left Tube	140.00	939.13
SK-44+000	44+000	Axis of Left Tube	115.00	917.00
SK-44+190	44+190	Axis	95.00	897.61
SK-44+440	44+440	Axis	55.00	864.15
SK-44+515	44+515	Axis of Left Tube	40.00	851.10
SK-44+540	44+540	Axis of Left Tube	40.00	847.50
SK-44+560	44+560	Axis of Right Tube	35.00	847.50
SK-44+580	44+580	Axis of Right Tube	30.00.	844.14
SK-44+600A	44+600A	Right of Axis	30.00	844.90
SK-44+600B	44+600B	Left of Axis	30.00	844.90

Figure 3.5 Geological longitudinal section of Çubukbeli tunnel

3.3 Engineering Geology

This part comprises the evaluation of engineering geological properties of rocks exposed and cut along the tunnel route on the basis of field measurements, core-box survey and laboratory tests. The rock descriptions include both rock mass and rock material characteristics based on ISRM method (1981).

The geological and geotechnical evaluations, descriptions and rock mass classifications made in this study are based on borehole logs, laboratory test results, figures, tables and photographs given in Geological and Geotechnical Final Report (Altınok, 2007a)

The rock types in Çubukbeli Tunnel area are; Beydağları Formation comprising neritic limestones belonging to Jurassic-Cretaceous aged Beydağları Autoctone, Danien aged Çamlıdere Olistostrome and neo-autoctone situated and Plio-Quaternary aged talus above these units as described in plan and profile sheets in Figures 3.4 and 3.5.

In order to determine the engineering properties of these rock masses, detailed field investigations and measurements and total 19 boreholes drilled along the entrance, exit and middle sections of tunnel route are accomplished. Table 3.1 shows the location, depth, elevation and kilometers of these boreholes.

For the rock classifications and estimation of geomechanical parameters for finite element analysis , rock mechanics testing (uniaxial compressive strength, modulus of elasticity, unit weight, poisson ratio etc.) is performed on samples taken from the core borings drilled in the study area. Laboratory tests were conducted by Rock Mechanics Laboratories of General Directorate of Highway Research Department and summarized in Table 3.2.

Table 3.1 The location, depth, elevation and kilometer of boreholes

BOREHOLE NO	KILOMETER	LOCATION	DEPTH (m)	ELEVATION (m)
SK-42+550	42+550	Axis of Right Tube	27.00	787.00
SK-42+570	42+570	Right of Axis	35.00	787.00
SK-42+580	42+580	Left of Axis	30.00	791.50
SK-42+605	42+605	Axis of Right Tube	35.00	796.45
SK-42+615	42+615	Axis of Left Tube	30.00	797.97
SK-42+800	42+800	Axis of Right Tube	75.00	840.21
SK-42+850	42+850	Axis of Left Tube	75.00	845.00
SK-43+080	43+080	Axis of Right Tube	130.00	904.05
SK-43+400	43+400	Axis of Left Tube	140.00	921.00
SK-43+720	43+720	Axis of Left Tube	140.00	939.13
SK-44+000	44+000	Axis of Left Tube	115.00	917.00
SK-44+190	44+190	Axis	95.00	897.61
SK-44+440	44+440	Axis	55.00	864.15
SK-44+515	44+515	Axis of Left Tube	40.00	851.10
SK-44+540	44+540	Axis of Left Tube	40.00	847.50
SK-44+560	44+560	Axis of Right Tube	35.00	847.50
SK-44+580	44+580	Axis of Right Tube	30.00.	844.14
SK-44+600A	44+600A	Right of Axis	30.00	844.90
SK-44+600B	44+600B	Left of Axis	30.00	844.90

Table 3.2 Summary of laboratory test results

Borehole No	Depth (m)	Lithology	Density (g/cm^3)	Uniaxial Comp-ressive Strength (MPa)	Pois-sons Ratio	Cohesion (c, MPa)	Internal Friction Angle (Ø, ^{o})
42+550	17.0-17.25	Limestone	2.59	26.0	0.23	-	-
42+580	13.5-13.65	Limestone	2.59	31.6	0.22	-	-
42+580	22.7-22.85	Limestone	2.51	7.8	0.27	-	-
42+850	55.1-55.25	Limestone	2.44	27.4	-	-	-
42+800	56.7-58.0	Limestone	-	-	-	72	65.5
43+080	115.8-117.8	Limestone	-	-	-	75	63.9

3.3.1 Beydağları formation (Kb)

Beydağları formation is present along the Çubukbeli Tunnel area from the entrance portal (Km:42+607) to Km:43+700.

Formation, composed of Jurassic-Cretaceous aged neritic limestones, is medium-thick layered, gray-dark gray coloured, occasionally dolomitic and macro fossil tracked. Top cretaceous aged limestones are medium-thick layered, beige, gray and light brown coloured. Unit sometimes comprises macro fossils such as coral, gastropod, lamelli. A layered view of Beydağları formation is shown in Figure 3.6 in the following page, where Figure 3.7 present limestones in the entrance portal.

There are two uniform and one random discontinuity set observed in the Beydağları formation. The discontinuity length (persistence) is more than 20 m and the aperture is between 0.1-1 mm. The joint walls are slightly to moderately wheathered, rough and planar. They are also dry and occasionally hard filling with below 3 mm filling thickness. Fillings are slightly weathered.

According to these observations, the Beydağları formation is estimated as fresh-sligthly weathered, middle-rare jointed, highly strong, poor-fair quality rock. The uniaxial compressive strength of intact rock ranges between 8-31 MPa.

Karstic spaces are widely present in Beydağları formation including water and clay fillings. Many openings are developed inside the unit. The formation is accepted as moderately permeable-permeable. Water inflow may be faced during excavation especially at formation boundaries and fault zones.

The strikes of layers and joints are perpendicular or nearly perpendicular to tunnel axis. Therefore, the locations of these discontinuities are evaluated as “fair” in accordance with tunnel excavation.

Figure 3.6 The layered view of Beydağları formation

Figure 3.7 The leftside limestones in the entrance portal

Table 3.3 gives the Rock Quality Designation (RQD) values of boreholes drilled in Beydağları limestones and evaluated according to last 50 m depth. It ranges from 0% to 100%. According to the average RQD percentages, the Beydağları limestone is very poor-poor-fair quality rock.

Table 3.3 RQD values of boreholes – Beydağları Formation

Borehole No	Lithology	Range of RQD (%)	Average of RQD (%)
SK-42+550	Limestone	10-100	45
SK-42+570	Limestone	0-80	22
SK-42+580	Limestone	10-27	10
SK-42+615	Limestone	0-33	9
SK-42+800	Limestone	5-95	66
SK-42+850	Limestone	0-83	29
SK-43+080	Limestone	0-46	12
SK-43+400	Limestone	0-22	1

It should be emphasized that, one of the main stability risk for limestone is structurally controlled instability problem during construction. But because of the lack of structural information about discontinuities at excavation levels, it is impossible to determine these risky regions in design stage.

3.3.2 Çamlıdere Olistostrome (Tpç)

The other main lithological group present along the Çubukbeli tunnel area is Çamlıdere olistostrome. Formation, situated from Km:43+700 to the exit portal (Km:44+592) of the tunnel, is composed of clayey limestone, siltstone, marl and sandstone at the bottom and fragmented rocks including various blocks at the top. A general view Çamlıdere olistostrome is shown in Figure 3.8 in the following page.

Figure 3.8 General view of Çamlıdere olistostrome.

The unit comprises thin-middle layered, beige, gray, greenish gray, pink, dirty yellow etc. coloured micrit, clayey micrit, claystone, marl, sandstone etc. rock types at the bottom. The top part is ended with the olistostrome, including parts of Antalya Naps (ofiolit, radyolite, limestone, sandstone etc.) and Beydağları Otoctone. The unit comprises matrixed sandstone, claystone and conglomerates. Sometimes coarse and sometimes fine fragments are dominant in the formation.

Çamlıdere olistostrome is highly jointed with 10-20 m persistence. The apertures are between 0.1-1 mm with highly weathered, rough, wet and soft clay filling joints. According to these observations, the rocks of the Çamlıdere olistostrome are estimated as highly weathered, highly jointed, very weak-weak, very poor-poor quality rock. The uniaxial compressive strength is supposed to be between 0 and 10 MPa.

Because of its lithological character, Çamlıdere olistostrome is accepted as low – middle permeable. Its sanstone, conglomerate and limestone levels may contain groundwater. Therefore, there is a possibility of facing groundwater flow during excavation.

The strikes of joints are perpendicular or nearly perpendicular to tunnel axis. Therefore, the locations of these discontinuities are evaluated as "fair" in accordance with tunnel excavation. Table 3.4 gives the Rock Quality Designation (RQD) values of borelholes drilled in Çamlıdere olistostrome and evaluated according to last 50 m depth. It ranges from 0% to 16%.

Table 3.4 RQD values of Boreholes – Çamlıdere olistostrome

Borehole No	Lithology	Range of RQD (%)	Average of RQD (%)
SK-42+605	Clayey Limestone	0	0
SK-43+720	Claystone, Siltstone, Marl	0	0
SK-44+000	Siltstone, Marl	0-10	1
SK-44+190	Siltstone, Marl	0-16	2
SK-44+440	Claystone, Siltstone	0	0
SK-44+515	Claystone, Siltstone	0	0
SK-44+540	Claystone, Siltstone	0-11	1
SK-44+560	Claystone, Siltstone	0	0
SK-44+580	Claystone, Siltstone	0	0
SK-44+600A	Claystone, Siltstone	0	0
SK-44+660B	Claystone, Siltstone	0	0

3.3.3 Talus (Qym)

Talus is another lithologic unit present along the Çubukbeli tunnel area between Km: 44+560- tunnel exit – open cut slope. It is composed of limestone originated gravels, blocks and rubbles both in loose and tight form.

CHAPTER IV

EMPIRICAL DETERMINATION OF EXCAVATION AND SUPPORT SYSTEMS FOR ÇUBUKBELİ TUNNEL IN ANTALYA

4.1 Introduction

Aim of this chapter is to empirically determine the excavation and support systems for the Çubukbeli tunnel in Antalya. For this purpose, rock mass classifications along the tunnel route are carried out. Thereafter, excavation and supports systems are suggested based on these rock classifications according to NATM. Finally, geomechanical parameter estimations along the tunnel alignment is accomplished prior to numerical analysis.

4.2 Rock Mass Classification for Çubukbeli Tunnel

Main geological formations and their geotechnical specifications along Çubukbeli tunnel route have been explained in the previous chapter. According to these informations and evaluations, common geological formation (GMU) present along tunnel route will be determined in this part.

According to site investigations and measurements, laboratory tests and drilling results, Rock Mass Rating (RMR) and Rock Mass Quality (Q) classifications have been accomplished for the rocks encountered along the tunnel route. Based on these two widely used classifications, those also are correlated to each other, New Austrian Tunnelling Method (NATM) classification is carried out to empirically determine excavation and support systems and Geological Strength Index (GSI) values have been found in order to estimate the rock mass strength parameters used in finite element analysis.

As explained in Literature Survey in detail, GSI is a qualitative classification system arisen from the need to describe discontinuities in highly jointed weak rock masses. It may be used individually and independently as a separate classification system. But in this study, it is preferred to achieve GSI values indirectly by using RMR classifications especially to estimate rock mass strength parameters, since the tunnel is not under construction yet.

4.2.1 Entrance Section (Beydağları formation)

Boreholes SK-42550, SK-42580 and SK-42615 are evaluated for the classification.

4.2.1.1 RMR Classification

			Rating
Rock Quality Designation (RQD)		= 15	4
Uniaxial Comressive Strength (σ_c)		= 22 MPa	3
Spacing of Discontinuities		= 0.05-1.00m	8
Discontinuity Conditions			
	Persistance	> 20 m	0
	Aperture	= 0.1-1 mm	4
	Roughness	= slightly rough	3
	Infilling	= hard filling < 5 mm	4
	Weathering	= mod.-slightly weathered	4
Groundwater Condition		= dry	15
	Basic RMR Point (RMR_o)	**=**	**45**
Discontinuity Orientation		= fair	-5
	Total RMR Point (RMR) (poor to fair rock)	**=**	**40**

4.2.1.2 Q Classification **Rating**

Rock Quality Designation (RQD) = 15	15
Joint Set Number (Jn) 2 set+random	6
Joint Roughness Number (Jr) rough undulating joints	1.5
Joint Alteration Number (Ja) moderate-slightly weathered joints	2.5
Joint Water Reduction Number (Jw) dry	1
Stress Reduction Factor (SRF) portal section	2.5

Q = (RQD/Jn)*(Jr/Ja)*(Jw/SRF)

Q = (15/6)*(1.5/2.5)*(1/2.5) = 0.60
(very poor rock)

4.2.1.3 Correlation of RMR and Q

RMR=9lnQ+44

Q=0.60 ═> RMR ≈ 40 which is equal to calculated RMR=40.

4.2.1.4 NATM Classification

RMR=40 and Q=0.60 is thought to be **B3 Rock Class** according to NATM Classification.

4.2.1.5 GSI Classification

GSI = RMR_o – 5 = 45 - 5 = 40

4.2.2 Middle Section-1 (Km:42+650-43+000 - Beydağları formation)

Boreholes SK-42800 and SK-42850 are evaluated for the classification.

4.2.2.1 RMR Classification

		Rating
Rock Quality Designation (RQD)	= 50	10
Uniaxial Comressive Strength (σ_c)	= 27 MPa	4
Spacing of Discontinuities	= 0.10-2.00m	15
Discontinuity Conditions		
Persistance	= 3-10 m	2
Aperture	= 0.1-1 mm	4
Roughness	= slightly rough	3
Infilling	= hard filling < 5 mm	4
Weathering	= slightly weathered	5
Groundwater Condition	= dry	15
Basic RMR Point (RMR_o)	**=**	**62**
Discontinuity Orientation	= fair	-5
Total RMR Point (RMR)	**=**	**57**

(fair to good rock)

4.2.2.2 Q Classification

	Rating
Rock Quality Designation (RQD) = 50	50
Joint Set Number (Jn) 2 set+random	6
Joint Roughness Number (Jr) rough undulating joints	1.5
Joint Alteration Number (Ja) unweathered joints	2.0
Joint Water Reduction Number (Jw) dry	1
Stress Reduction Factor (SRF)	1

Q = (RQD/Jn)*(Jr/Ja)*(Jw/SRF)

Q = (50/6)*(1.5/2.0)*(1/1) = 6.25

(fair rock)

4.2.2.3 Correlation of RMR and Q

RMR=9lnQ+44

Q=6.25 ═> RMR ≈ 60 which is close to calculated RMR=57.

4.2.2.4 NATM Classification

RMR=57 and Q=6.25 is thought to be at the boundary of **B1-B2 Rock Class** according to NATM Classification.

4.2.2.5 GSI Classification

$GSI = RMR_o - 5 = 62 - 5 = 57$

4.2.3 Middle Section-2 (Km:43+000-43+650 - Beydağları Formation)

Boreholes SK-43080 and SK-43400 are evaluated for the classification.

4.2.3.1 RMR Classification

		Rating
Rock Quality Designation (RQD)	= 10	4
Uniaxial Comressive Strength (σ_c)	= 27 MPa	4
Spacing of Discontinuities	= 0.05-1.00m	10
Discontinuity Conditions		
Persistance	= 3-10 m	2
Aperture	= 0.1-1 mm	4
Roughness	= slightly rough	3
Infilling	= hard filling < 5 mm	4
Weathering	= slightly weathered	5
Groundwater Condition	= dry	15
Basic RMR Point (RMR_o)	**=**	**51**
Discontinuity Orientation	= fair	-5
Total RMR Point (RMR) (fair rock)	**=**	**46**

4.2.3.2 Q Classification **Rating**

Rock Quality Designation (RQD) = 10	10
Joint Set Number (Jn) 2 set+random	6
Joint Roughness Number (Jr) rough undulating joints	1.5
Joint Alteration Number (Ja) slightly weathered joints	2.0
Joint Water Reduction Number (Jw) dry	1
Stress Reduction Factor (SRF)	1

Q = (RQD/Jn)*(Jr/Ja)*(Jw/SRF)

$$\underline{\mathbf{Q = (10/6)*(1.5/2.0)*(1/1) \quad = \quad 1.25}}$$

(poor rock)

4.2.3.3 Correlation of RMR and Q

RMR=9lnQ+44

Q=1.25 ═> RMR ≈ 46 which is equal to calculated RMR=46.

4.2.3.4 NATM Classification

RMR=46 and Q=1.25 is thought to be at the boundary of **B2-B3 Rock Class** according to NATM Classification.

4.2.3.5 GSI Classification

$$\underline{\mathbf{GSI = RMR_o - 5 = 51 - 5 = 46}}$$

4.2.4 Middle Section-3 (Km:43+650-44+530 - Çamlıdere olistostrome)

Boreholes SK-43720, SK-44000, SK-44190 and SK-44440 are evaluated for the classification.

4.2.4.1 RMR Classification

		Rating
Rock Quality Designation (RQD)	< 10	3
Uniaxial Comressive Strength (σ_c)	< 10 MPa	1
Spacing of Discontinuities	= 0.02-0.50m	7
Discontinuity Conditions		
Persistance	= 3-10 m	2
Aperture	= 0.1-1 mm	4
Roughness	= rough	5
Infilling	= soft filling < 5 mm	2
Weathering	= highly weathered	1
Groundwater Condition	= wet	7
Basic RMR Point (RMR_o)	**=**	**32**
Discontinuity Orientation	= fair	-5
Total RMR Point (RMR)	**=**	**27**

(poor rock)

4.2.4.2 Q Classification

	Rating
Rock Quality Designation (RQD) < 10	10
Joint Set Number (Jn) 3 set+random	12
Joint Roughness Number (Jr) rough joints	3.0
Joint Alteration Number (Ja) highly weathered joints	4.0
Joint Water Reduction Number (Jw) wet	0.66
Stress Reduction Factor (SRF) sqeezing condition H>50 m	2.50

Q = (RQD/Jn)*(Jr/Ja)*(Jw/SRF)

Q = (10/12)*(3.0/4.0)*(0.66/2.50) = 0.165

(very poor rock)

4.2.4.3 Correlation of RMR and Q

RMR=9lnQ+44

Q=0.165 => RMR ≈ 28 which is close to calculated RMR=27.

4.2.4.4 NATM Classification

RMR=27 and Q=0.165 is thought to be **C2 Rock Class** according to NATM Classification. Actually RMR=27 and Q=0.165 seems to be C1 Rock Class according to Figure 2.4 of Chapter II. But C1 Clas Rock indicates rock bursting which occurs in deep excavations below 500. In our case maximum overburden is 130 m. Therefore both for this section and for exit section C2 Rock Class is selected according to NATM.

4.2.4.5 GSI Classification

$GSI = RMR_o - 5 = 32 - 5 = 27$

4.2.5 Exit Section (Çamlıdere Olistostrome)

Boreholes SK-44515, SK-44540, SK-44560, SK-44580, SK-44600A and SK-44600B are evaluated for the classification.

4.2.5.1 RMR Classification

		Rating
Rock Quality Designation (RQD)	< 10	3
Uniaxial Comressive Strength (σ_c)	< 10 MPa	1
Spacing of Discontinuities	= 0.02-0.30m	5
Discontinuity Conditions		
Persistance	= 10-20 m	1
Aperture	= 0.1-1 mm	4
Roughness	= rough	5
Infilling	= soft filling < 5 mm	2
Weathering	= highly weathered	1
Groundwater Condition	= wet	7
Basic RMR Point (RMR_o)	**=**	**29**
Discontinuity Orientation	= fair	-5
Total RMR Point (RMR) (poor rock)	**=**	**24**

4.2.5.2 Q Classification **Rating**

Rock Quality Designation (RQD) < 10	10
Joint Set Number (Jn) heavily jointed	15
Joint Roughness Number (Jr) rough joints	3.0
Joint Alteration Number (Ja) highly weathered joints	4.0
Joint Water Reduction Number (Jw) wet	0.66
Stress Reduction Factor (SRF) sqeezing condition H<50 m	5.00

Q = (RQD/Jn)*(Jr/Ja)*(Jw/SRF)

<u>Q = (10/15)*(3.0/4.0)*(0.66/5.00) = 0.066</u>
(extremely poor rock)

4.2.5.3 Correlation of RMR and Q

RMR=9lnQ+44

Q=0.066 => RMR ≈ 20 which is close to calculated RMR=24.

4.2.5.4 NATM Classification

RMR=24 and Q=0.066 is thought to be **<u>C2 Rock Class</u>** according to NATM Classification.

4.2.5.5 GSI Classification

<u>GSI = RMR_o – 5 = 29 - 5 = 24</u>

According to above classifications, the entrance section of tunnel is evaluated as poor to fair rock in limestone (Beydağları formation) which indicates B3 Rock Class and exit section of tunnel is evaluated as extremely poor to poor rock in clayey limestone, marl, sandstone and claystone (Çamlıdere olistostrome) which indicates C2 Rock Class according to NATM. The middle sections of tunnel is estimated as fair to good rock in Beydağları limestones and very poor to poor rock in Çamlıdere olistostrome indicating B1, B2 and C2 Rock Classes according to NATM. Table 4.1 summarizes overall rock mass classifications along Çubukbeli Tunnel.

Table 4.1 Rock Mass Classifications along Çubukbeli Tunnel

	LITHOLOGY	BASIC RMR	Q	NATM	GSI
ENTRANCE SECTION	Limestone	45	0.60	B3	40
MIDDLE SECTION-1 (KM:42+650-43+000)	Limestone	62	6.25	B1-B2	57
MIDDLE SECTION-2 (KM:43+000-43+650)	Limestone	51	1.25	B2-B3	46
MIDDLE SECTION-3 (KM:43+650-44+530)	Clayey limestone, marl, sandstone, claystone	32	0.165	C2	27
EXIT SECTION	Clayey limestone, marl, sandstone, claystone	29	0.066	C2	24

4.3 Suggested Excavation and Support Systems based on NATM

In the preceding section, Rock Mass Classifications along Çubukbeli Tunnel are accomplished and NATM Classification is achieved for the determination of excavation and support suggestions for a highway tunnel with two lanes. Table 4.2 shows the summarized excavation and support systems for Çubukbeli tunnel according to NATM. Figure 4.1 presents the excavation and support system details on geological longitudinal section of tunnel. Figures C.1, C.2, C.3 and C.4 in Appendix C show final drawings and details of application of these excavation and support systems based on KGM (General Directorate of Highways) tradition.

Table 4.2 Summary of excavation and support systems of Çubukbeli tunnel according to NATM

SUPPORT SYSTEMS	**B1**	**B2**	**B3**	**C2**
DIAMETER OF ROCK BOLT	∅ 28	∅ 28	∅ 28	∅ 28
INTERVAL OF ROCK BOLT	2.50x2.50 (TOPHEADING)	2.00x2.00	1.50x1.00-1.50	1.00x1.00-1.25
LENGTH OF ROCK BOLT	4m	4m	4-6m	4-6m
THICKNESS AND CLASS OF SHOTCRETE	C20-25(10cm)	C20-25(15cm)	C20-25(20cm)	C20-25(25cm)
TYPE OF WIRE MESH	Q221/221 SINGLE LAYER	Q221/221 SINGLE LAYER	Q221/221 DOUBLE LAYER	Q221/221 DOUBLE LAYER
TYPE AND INTERVAL OF STEEL RIB	-	-	I160,1.00-1.50m	I160,1.00-1.25m
INTERVAL AND LENGTH OF FOREPOLE	-	-	0.3-0.4m,4-6 m	0.3-0.4m,4-6 m
ADVANCE LENGTH				
TOPHEADING	2.00-3.00 m	≤1.50-2.00 m	≤1.25-1.50 m	≤0.75-1.25 m
BENCH	4.00 m	≤3.00-3.50 m	≤3.00 m	≤2.00 m
INVERT	-	-	≤3.00 m (IF REQUIRED)	≤2.00 m

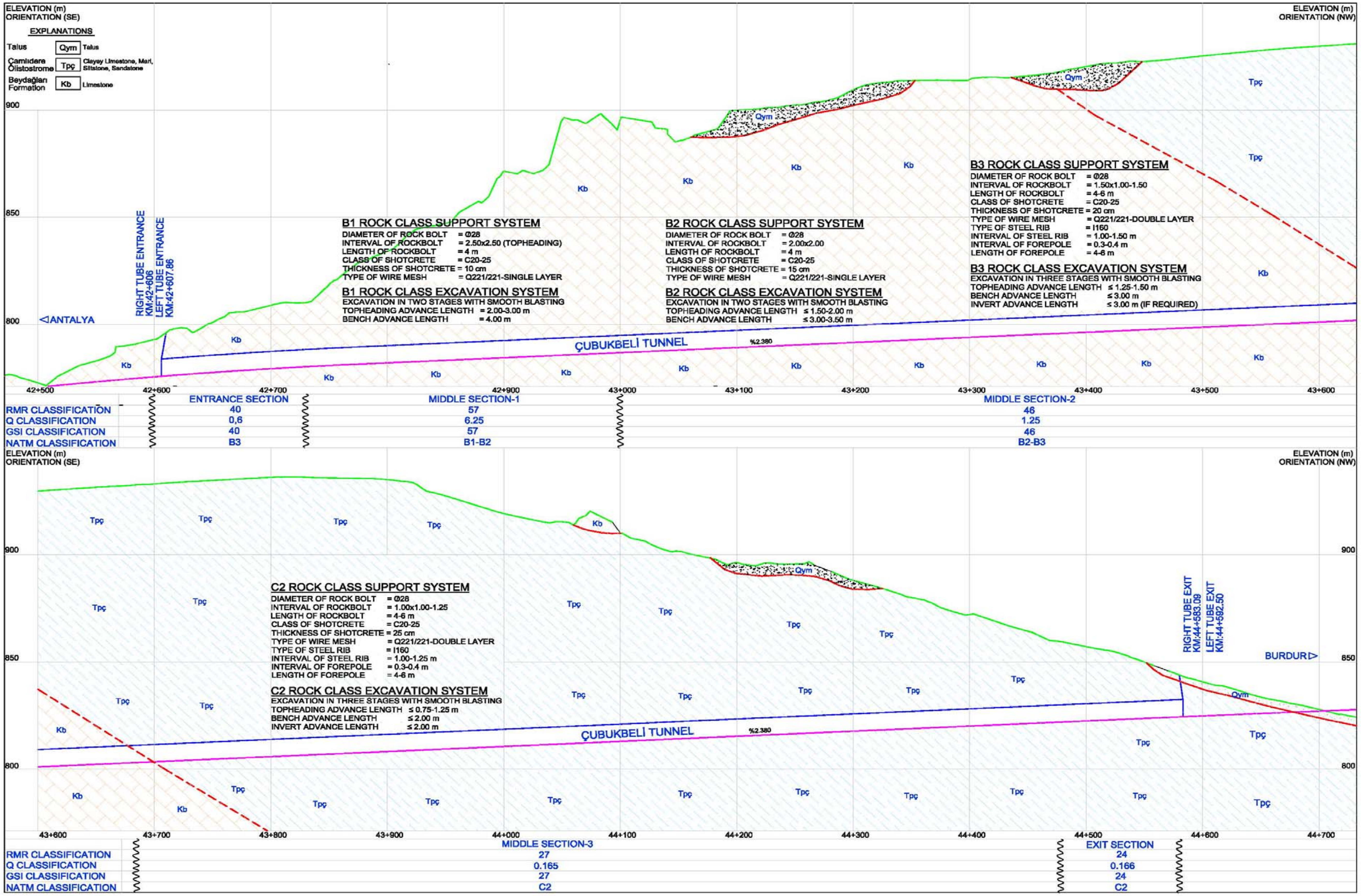


Figure 4.1 Geological longitudinal section of Çubukbeli tunnel showing excavation and support systems according to NATM

4.4 Estimation of Rock Mass Strength and Modulus of Deformation

One of the major obstacles which is encountered in the field of numerical modeling for rock mechanics, is the problem of data input for rock mass properties. The usefulness of elaborate constitutive models, and powerful numerical analysis programs, is greatly limited, if the analyst does not have reliable input data for rock mass properties.

The Hoek - Brown failure criterion, in conjunction with its implementation in ***RocLab***, which provides a widely used approach for remedying this situation, allowing users to easily obtain reliable estimates of rock mass properties, and to visualize the effects of changing rock mass parameters on the failure envelopes.

The task of determining rock mass properties is not usually an end in itself. It is carried out in order to provide input for numerical analysis programs, which require material properties in order to perform a stability or stress analysis. In this study, ***Roclab*** (ver. 1.031) is used to determine the rock mass properties and to provide input data for numerical analysis program ***Phase***^{2} (finite element stress analysis and support design for excavations). Table 4.3 shows the summary of geomechanical parameters of rock mass classes for each section at the project area. Appendix B shows the ***Roclab*** program outputs presenting these parameters.

As seen in the Table 4.3 and ***Roclab*** program outputs in the Appendix B, Disturbance Factor (D) is taken into consideration for rock mass classes. D is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses. This factor is taken as a new approach for the effect of excavation to rock mass in 2002 Edition of Hoek-Brown Failure Criterion by Hoek, Torres and Corkum.

In our case, D is estimated as 0,6 for Beydağları limestones due to fair to good quality smooth blasting in poor to fair rock mass and as 0,4 for Çamlıdere olistostrome due to fair to good quality smooth blasting in very poor to poor quality rock mass. These values lead to mass reduction factor (Sönmez et. al., 2006) r_f=0.67 for D=0.6 and r_f=0.83 for D=0.4 according to Figure F.1 in Appendix F.

A very important parameter in any form of numerical analysis and in the interpretation of monitored deformation around underground opening is Modulus of Elasticity (E_m). The short history of approaches for estimation of E_m is mentioned in Chapter II. In our case, Tables 4.4 and 4.5 show the Modulus of Elasticity (E_m) results of disturbed and undisturbed rock masses according to Barton (2002), Hoek (2002), Hoek & Diederichs (2006) and Sönmez et. al. (2006) .

As seen in Tables 4.4 and 4.5, highest results are obtained from Barton (2002) and lowest results are obtained from Sönmez et.al. (2006) and Hoek and Diederichs (2006) in undisturbed case. Hoek and Diederichs (2006) gives very low results where uniaxial compressive strength of intact rock is low (in middle section-3 and exit sections). Hoek (2002) gives similar results to Hoek and Diederichs (2006) with average uniaxial compressive strength values. In disturbed case, highest results are obtained from Hoek (2002), and lowest results are obtained from Sönmez et. al. (2006) and Hoek and Diederichs (2006) at low uniaxial compressive strength values.

If all the results are evaluated, it is seen that, these empirical derivations give considerably different E_m values for the same quality rock. So, it is assumed that, Modulus of Elasticity (E_m) values to be used for finite element analysis of Çubukbeli Tunnel excavation and support systems are determined according to Hoek and Diederichs (2006) empirical approach since it gives average values and also it is a recent approach and used worldwide.

Table 4.3 Summary of geotechnical parameters of rock mass sections along Çubukbeli tunnel

	ROCK MASS CLASSIFICATIONS				LABORATORY TEST RESULTS				HOEK-BROWN PARAMETERS		ROCK MASS PARAMETERS (UNDISTURBED ROCK)			ROCK MASS PARAMETERS (DISTURBED ROCK)		
	GSI	BASIC RMR	Q	NATM	σ_c (MPa)	Ei (MPa)	γ (kN/m^3)	Over-burden (m)	mi	D	E_m (MPa)	m	s	E_m (MPa)	m	s
ENTRANCE SECTION	40	45	0,600	B3	22	15400	26	10	10	0,6	2459	1.173	0.0013	1028	0.468	0.0002
MIDDLE SECTION-1	57	62	6,250	B1-B2	27	18900	26	110	10	0,6	8547	2.153	0.0084	3705	1.115	0.0025
MIDDLE SECTION-2	46	51	1,250	B2-B3	27	18900	26	130	10	0,6	4513	1.454	0.0025	1833	0.636	0.0006
MIDDLE SECTION-3	27	32	0,165	C2	10	3750	23	130	7	0,4	253	0.516	0.0003	159	0.269	0.0001
EXIT SECTION	24	29	0,066	C2	5	1875	23	10	7	0,4	106	0.464	0.0002	70	0.235	0.0001

Table 4.4 Modulus of deformation (E_m) values of undisturbed rock mass

	MODULUS OF ELASTICITY (UNDISTURBED)				
	$E_m=Ei\,(0.02+(1-D/2)/(1+e^{((60+15D-GSI)/11)}))$ (MPa) (Hoek&Diederichs, 2006)	$E_m=(1-D/2)*((\sigma_c/100)^{0.5})*(10^{(GSI-10)/40})$ (MPa) (Hoek, 2002)	$E_m=Ei10^{((RMRd-100)*(100-RMRd))/(4000*exp(-RMRd/100))}$ (MPa) (Sönmez et. al., 2006)	$E_m=10Q_c^{1/3}$ $Q_c=Q*(\sigma_c/100)$ (MPa) (Barton, 2002)	$E_m=10^{(15logQ+40)/40}$ (MPa) (Barton, 2002)
ENTRANCE SECTION	2459	2638	909	5092	8257
MIDDLE SECTION-1	8547	7775	3045	11906	19882
MIDDLE SECTION-2	4513	4127	1400	6962	10873
MIDDLE SECTION-3	253	841	144	2546	5088
EXIT SECTION	106	501	117	1489	3609

	GSI	BASIC RMR	Q	NATM	σ_c (MPa)	Ei (MPa) (Hoek, 2002)	mi	MR	Ei (MPa) (Sönmez, 2006)	γ (kN/m^3)
ENTRANCE SECTION	40	45	0,600	B3	22	15400	10	700	20000	26
MIDDLE SECTION-1	57	62	6,250	B1-B2	27	18900	10	700	20000	26
MIDDLE SECTION-2	46	51	1,250	B2-B3	27	18900	10	700	20000	26
MIDDLE SECTION-3	27	32	0,165	C2	10	3750	7	375	8000	23
EXIT SECTION	24	29	0,066	C2	5	1875	7	375	8000	23

Table 4.5 Modulus of deformation (E_m) values of disturbed rock mass

	MODULUS OF ELASTICITY (DISTURBED)		
	$E_m=Ei\ (0.02+(1-D/2)/(1+e^{((60+15D-GSI)/11)}))$ (MPa) (Hoek&Diederichs, 2006)	$E_{m=}(1-D/2)*((\sigma_c/100)^{0.5})*(10^{(GSI-10)/40})$ (MPa) (Hoek, 2002)	$E_m=Ei10^{((RMRd-100)*(100-RMRd))/(4000*exp(-RMRd/100))}$ (MPa) (Sönmez et. al., 2006)
ENTRANCE SECTION	1028	1846	355
MIDDLE SECTION-1	3705	5442	797
MIDDLE SECTION-2	1833	2889	471
MIDDLE SECTION-3	159	673	105
EXIT SECTION	70	400	88

	D	rf	GSI	BASIC RMR	Q	NATM	σ_c (MPa)	Ei (MPa) (Hoek, 2002)	mi	MR	Ei (MPa) (Sönmez, 2006)	γ (kN/m^3)
ENTRANCE SECTION	0,6	0,67	40	45	0,600	B3	22	15400	10	700	20000	26
MIDDLE SECTION-1	0,6	0,67	57	62	6,250	B1-B2	27	18900	10	700	20000	26
MIDDLE SECTION-2	0,6	0,67	46	51	1,250	B2-B3	27	18900	10	700	20000	26
MIDDLE SECTION-3	0,4	0,83	27	32	0,165	C2	10	3750	7	375	8000	23
EXIT SECTION	0,4	0,83	24	29	0,066	C2	5	1875	7	375	8000	23

CHAPTER V

FINITE ELEMENT ANALYSIS

5.1 Introduction

The objective of numerical analysis is to check and verify the validity of empirically determined excavation and support systems for an underground opening. For this aim, the finite element software package ***Phase²*** is used to determine induced stresses and deformations developed around tunnel excavation and to investigate the interaction of proposed support systems with the tunnel ground.

The excavation section of Çubukbeli tunnel is twin tube flute shape with 12 m width and 10 m height. As explained in detail in the preceding chapter, the tunnel ground is divided into sections with respect to rock mass geomechanical properties that have been evaluated at the borehole locations along the tunnel route. Dominating rock class of the Çubukbeli tunnel according to ONORM B2203 is B1, B2 and B3 between entrance - middle part along Beydağları formation and C2 between middle part - exit of tunnel along Çamlıdere olistostrome.

So, four different ***Phase²*** excavation and support model (B1, B2, B3 and C2) is constituted to evaluate the interaction between proposed support systems with surrounding ground for the Çubukbeli tunnel. In this study, the model prepared for B3 Excavation and Support System, one of the most severe conditions that would be faced during tunneling, is explained and validity and verification of empirically determined B3 Excavation and Support Elements presented in Figure 4.3 is provided. Table 5.1 shows the material parameters of support elements used in the finite element analysis.

Table 5.1 Material parameters of support elements

	SHOTCRETE (C20-25)	**BOLT**	**STEEL RIB**	**WIRE MESH**
		Φ28	**(I160)**	**(Q221/221)**
Modulus of Elasticity (MPa)	15.000	210.000	210.000	210.000
Compressive Strength (MPa)	20	420	420	420
Crossectional Area (cm^2)	-	6,157	22,80	1,31
Tensile Strength	1,6 (MPa)	0,16 (MN)	-	-

5.2 Finite Element Analysis of B3 Excavation and Support System

B3 excavation and support model is formed based on the middle part crossection Km:43+600 of Çubukbeli tunnel route, where the overburden height is approximately 130 m, which is the biggest value along the tunnel. The Hoek-Brown Failure Criterion is used for the finite element analysis.

According to the geotechnical evaluations presented in Chapter III, the Beydağları limestones cut between Km:43+000-43+650 is in poor to fair rock conditions with GSI=46 which indicates B2-B3 Rock Class acoording to NATM. But the model shown in Figures 5.1 and 5.2 is constituted by entrance section rock mass parameters (GSI=40), where it is assumed that this is the poorest quality rock faced in Beydağları Limestones along Çubukbeli Tunnel. The model aims to verify the validity of empirical B3 excavation and support sytem in its poorest conditions.

As seen in Figures 5.1 and 5.2 in the following page, the boundary of the Beydağları limestone and Çamlıdere olistostrome (clayey limestone, siltstone, claystone and marl) is passing approximately 25 m above of right tube right upper section. By this way, the probable effect of very poor to poor quality (C2 Rock Class) Çamlıdere olistostrome on Beydağları limestones is included to the model. The model is constituted according to geological maps and sections based on borehole results and site investigation as explained in Chapter III.

The distance between excavation perimeters of left and right tubes are 18 m. A disturbed zone of 3 m thickness around tunnel excavation is supposed to be formed after blasting and bolt installation (Figure 5.2).

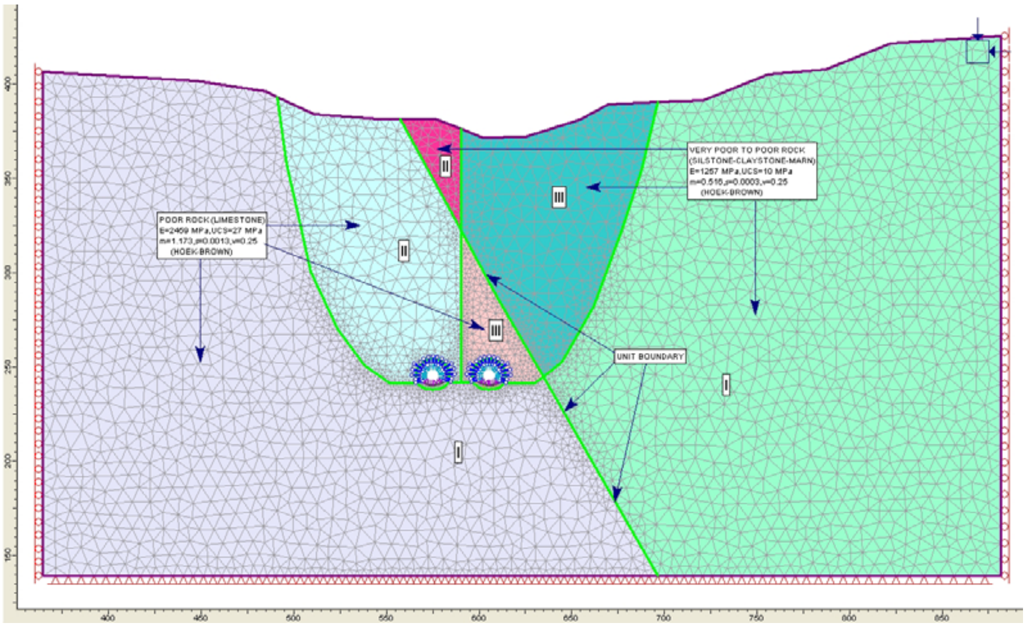


Figure 5.1 General view of B3 excavation and support model including finite element meshes.

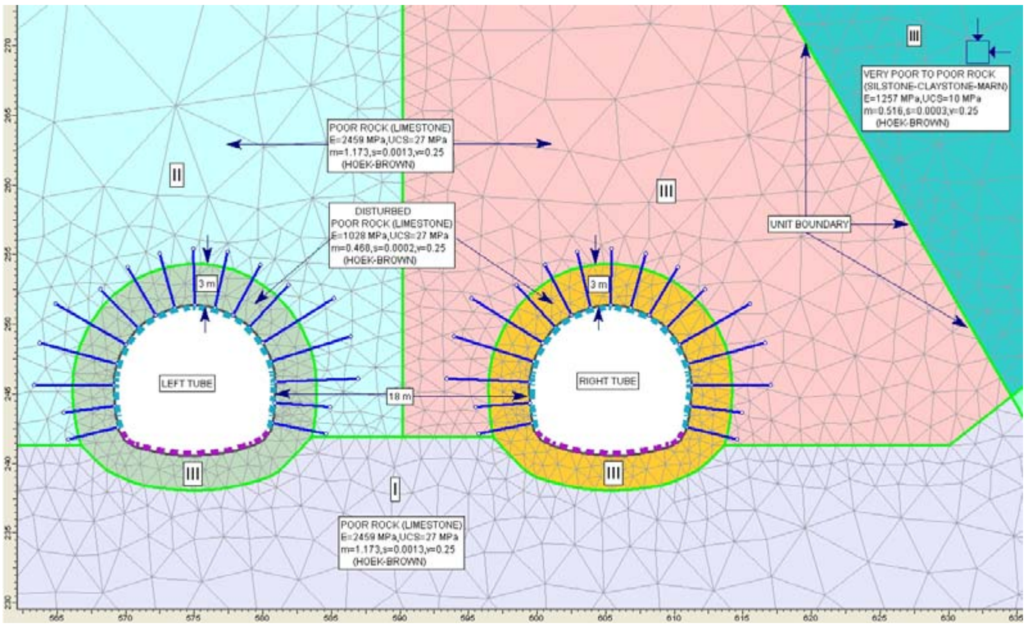


Figure 5.2 Detailed view of B3 Excavation and Support Model at its last stage when all excavation and supporting activities has been finished

B3 excavation and support model is constituted by nine stages to simulate the real excavation and support phases. In the first stage, insitu stress distribution (gravity loading due to thickness of overburden) is examined. The overburden height is the biggest value along tunnel route (approx. 130 m). In the following two stages, left tube top heading excavation and supporting activities are completed. In the forth and fifth stages, right tube top heading excavation and supporting activities are finished in the same manner. In the last four stages, bench excavation and supporting of left and right tubes are completed.

For the optimum simulation of excavation and support stages, **Phase²** provides Load Split possibility to split the load between stages. This means that, loads carried by an unsupported excavation is not the total load, but only part of it. The rest of load is carried by the unexcavated section of tunnel. It is assumed that (and should be), total load is carried by fully supported section of the tunnel which is also at a sufficient distance of tunnel face.

According to this, as shown in Figures 5.1 and 5.2, Load Split has been utilized for undisturbed rock masses II and III and disturbed Rock Masses IV and V. The elasto-plastic parameters of rock masses II and III are the same with each other and with undisturbed rock mass I. Rock masses I, II and III of Çamlıdere olistostrome and Beydağları formation have also same elasto-plastic parameters in themselves. At the second stage, where the top heading excavation of left tube is finished, II and IV masses are loaded with a percentage of r1. In the third stage where the support of top heading (20 cm thick shell of shotcrete, fully bonded 4-6 m long and 28 mm diameter untensioned grouted rock bolt, wire mesh and steel rib) is completed, the rest of load (1-r1) is added and these masses will be fully activated. In the same manner, III and V rock masses are loaded with a percentage of r2 at the fourth stage of right tube, and the rest (1-r2) is loaded in the fifth stage for full activation.

The determination of r1 and r2 percentages is quite difficult, because it depends on various parameters such as rock mass quality, time deviations during excavation and supporting phases, application quality of supports etc. But it is clear that, r1 and r2 percentages of rock masses will be decreased with increasing quality of rock mass because of increasing time and dimension of unsupported span. For B3 excavation and support system, the advance length of top heading is 1.00-1.50 m and full supporting should be completed before a new advance. So an unsupported section will be at max. 1.50 m distance from tunnel face and r1 percentage is estimated as 60% with the consideration of safety conditions. The rest 40% is added in the third stage where the supporting is completed. The construction period of right tube will be started after left tube top heading is finished. Since the left tube top heading excavation creates extra load on right tube, a new ınduced stresss condition occurs around this rock mass which leads to a lower value of r2 compared to r1. So r2 percentage is estimated as 40 % and the rest 60% is added in the fifth stage supporting of right tube is finished.

Total displacements are shown in Figure 5.3 where top heading (second stage) and top heading support systems (third stage) are completed in the left tube. As seen , there is no change in the 10 mm total displacements in the roof, between second and third stages where 60% r1 loading and total loading is applied. But base displacements increased from 5 mm to 30 mm.

Total displacements are shown in Figure 5.3 where top heading (second stage) and top heading support systems (third stage) are completed in the left tube. As seen , there is 6 mm increase (from 4 mm to 10 mm) total displacements in the roof, between second and third stages where 60% r1 loading and total loading is applied. Also, base displacements increased from 14 mm to 31 mm.

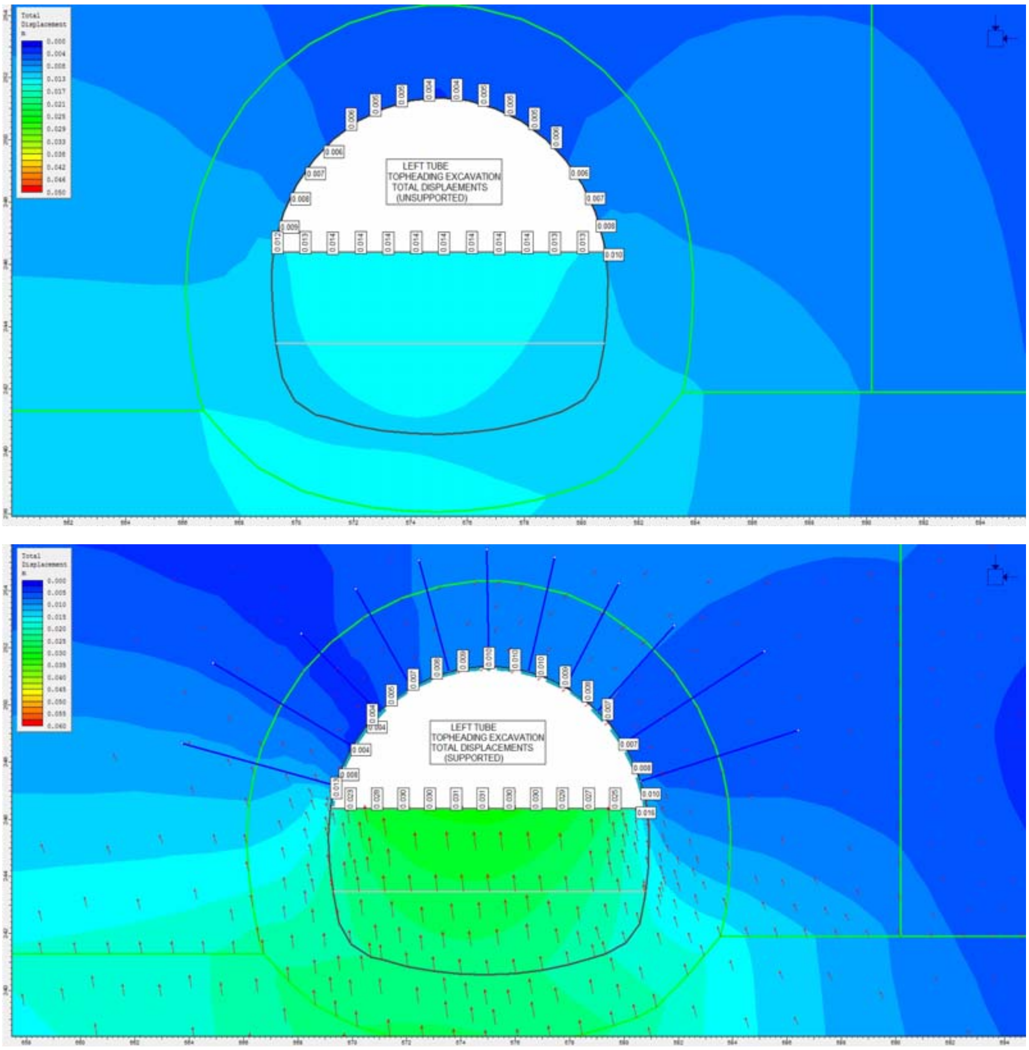


Figure 5.3 Total displacements at the second and third stages on left tube

Figure 5.4 shows total displacement in fourth and fifth stages where top heading excavation and support systems are finished in the right tube. As seen, max. displacement, originated as 8 mm after 40% r2 loading at the roof, is increased to 13 mm after full loading. Base displacements also increased to 37 mm from 17 mm between partial and full loading stages.

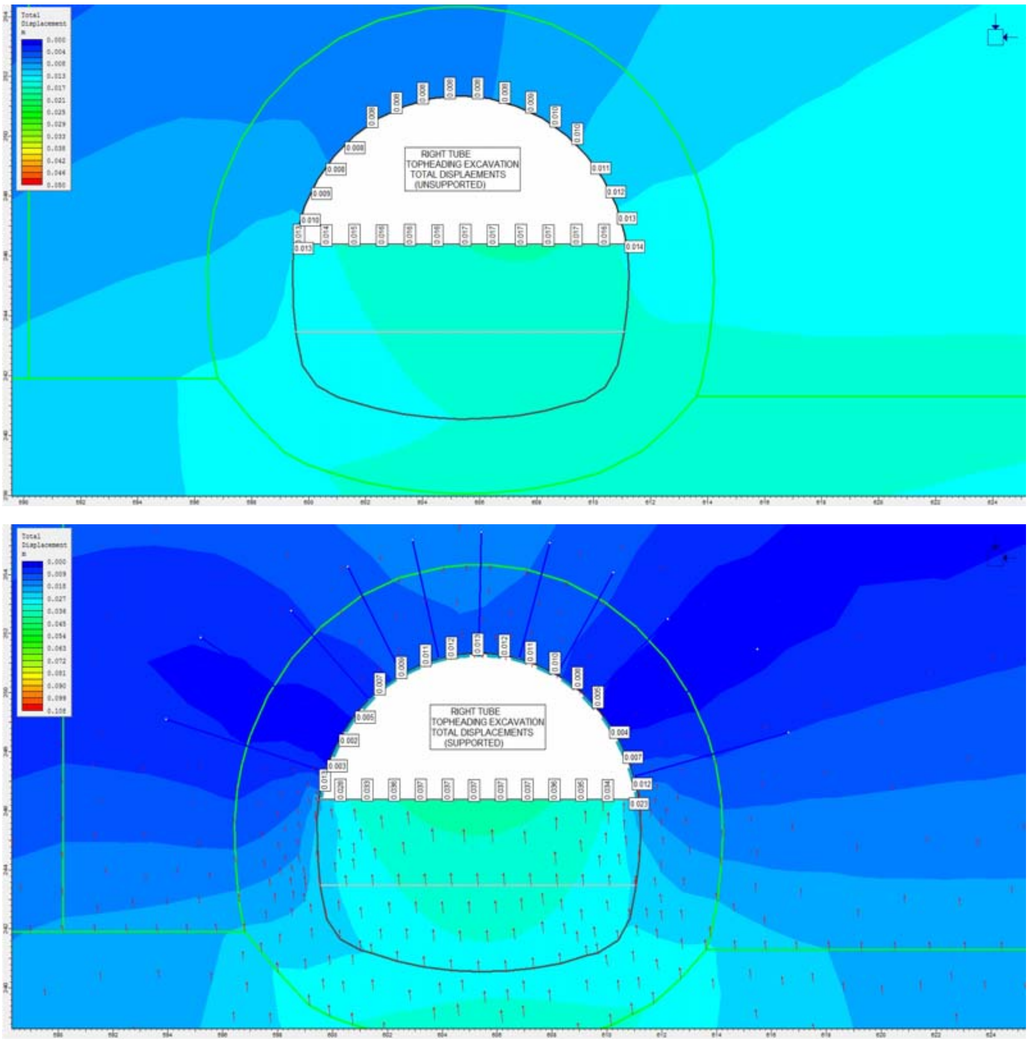


Figure 5.4 Total displacements at the forth and fifth stages on right tube

Total displacements originated on the left tube top heading is shown in Figure 5.5 after right tube top heading excavation and supporting is completed. Max. value is increased to 23 mm from 10 mm in the roof which shows the impact of right tube excavation on left tube excavation in poor rock conditions. This also verifies that r1 (60%) and r2 (40%) are correctly estimated and some displacement of rock mass is provided before installation of support and before the excavation of second tube in poor rock conditions as expected and as suitable for NATM philosophy.

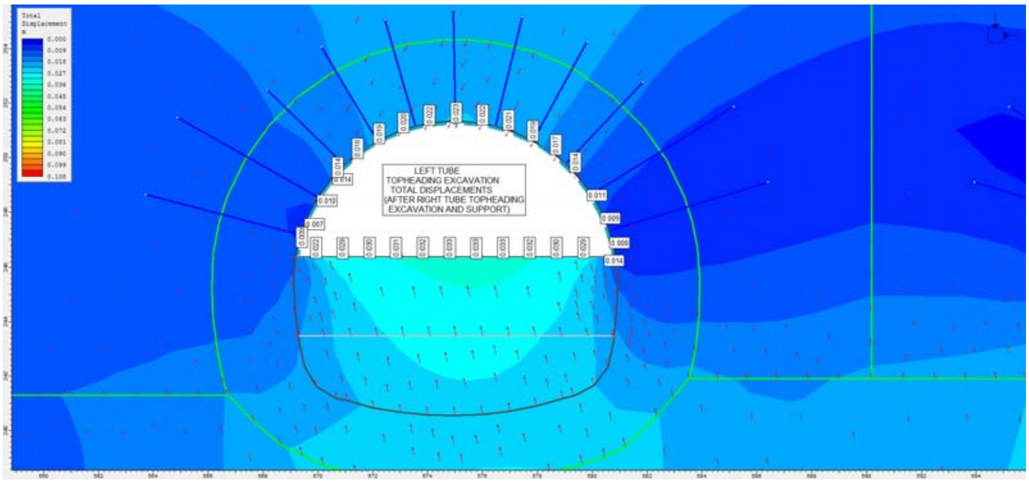


Figure 5.5 Total displacements at the fifth stage on left tube

Figure 5.6 shows the element numbers of rockbolt and shotcrete at nineth stage where full section excavation and support for both left and right tubes are finished. This figure will be reference for the following graphs presenting loads and moments on these support elements.

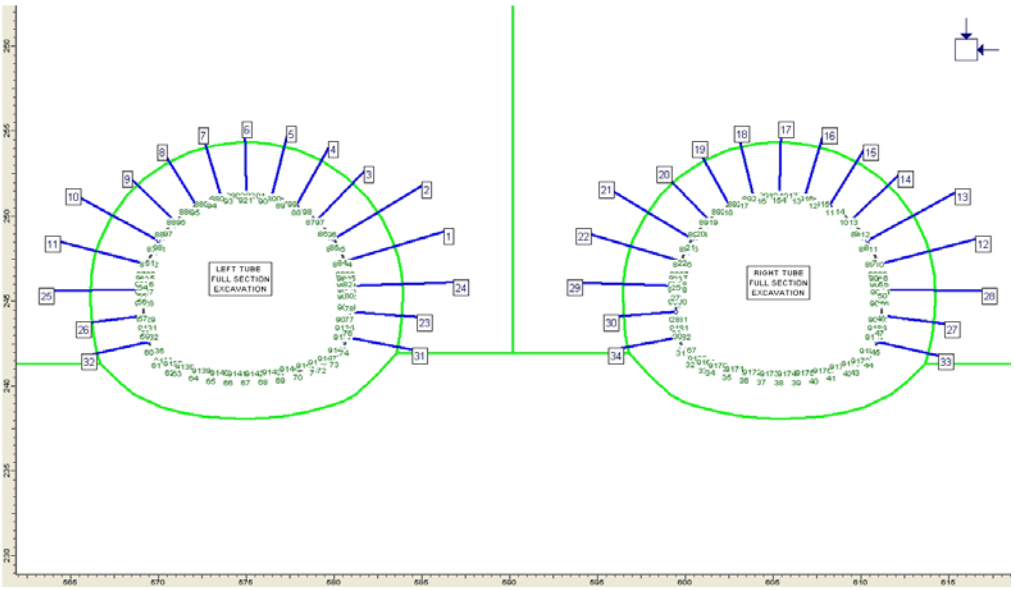
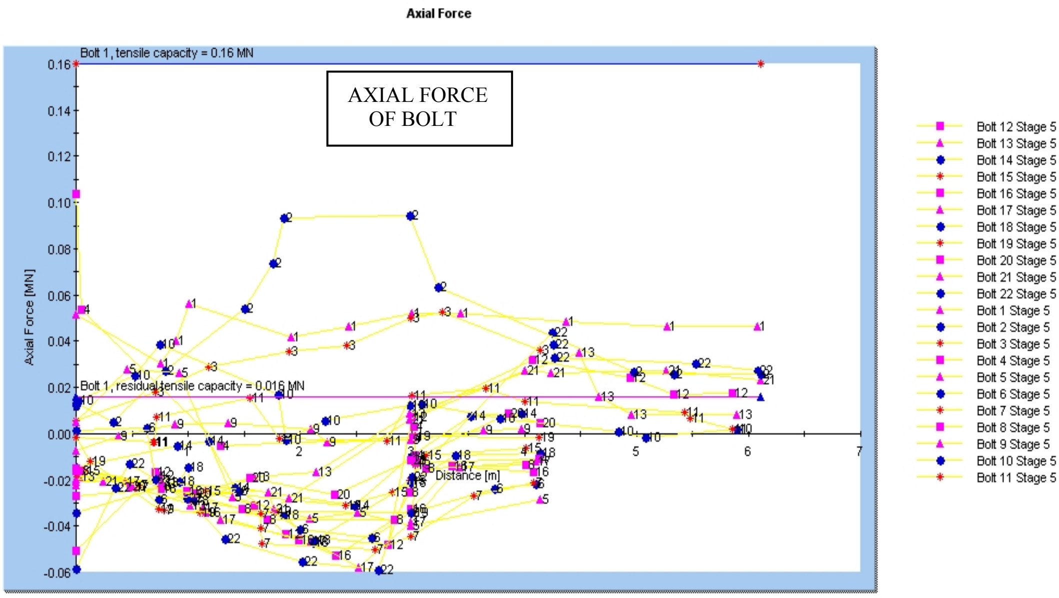


Figure 5.6 Numbers of bolt and shotcrete elements at nineth stage

Axial forces originated on rockbolts and shotcrete elements at the top heading of left and right tubes are shown in Figure 5.7. Maximum axial force on bolts is 9 tonnes which is originated at second bolt element at side wall of left tube. Maximum axial force developed on shotcrete elements are 340 tonnes on 23-24 element.



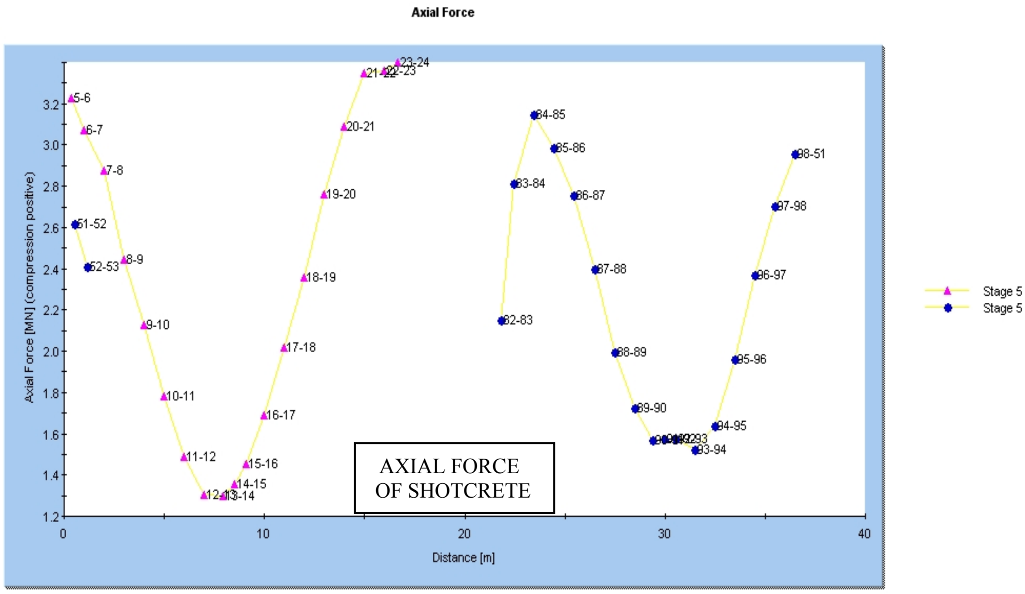
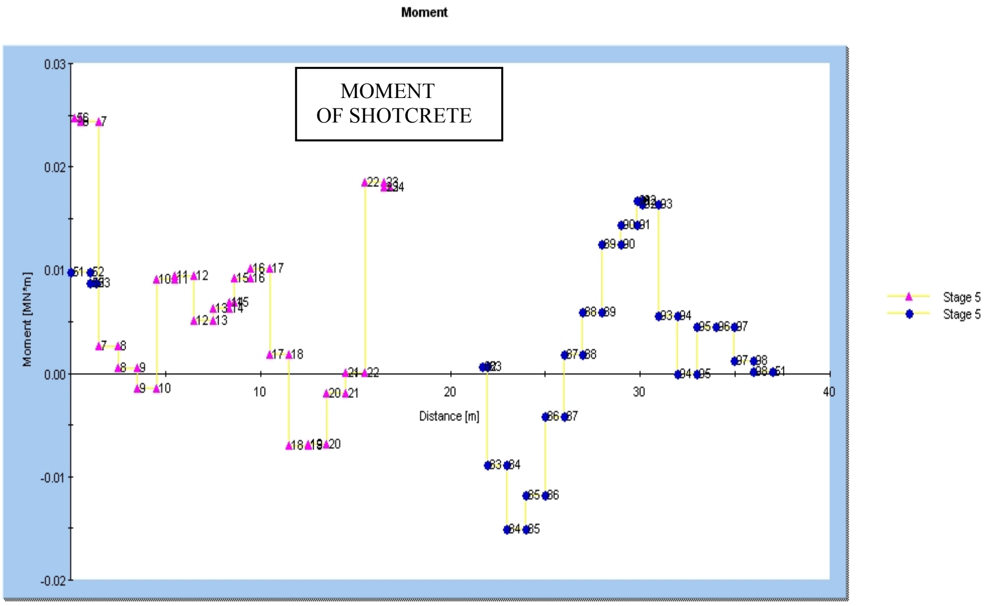


Figure 5.7 Axial forces on rockbolts and shotcrete elements at fifth stage

Maximum bending moment and shear forces developed on shotcrete elements are 2.5 ton*m and 16 tonnes respectively at roofs and side walls of right tube. It is clear that, the loads carried by support elements are quite below their capacities at fifth stage.



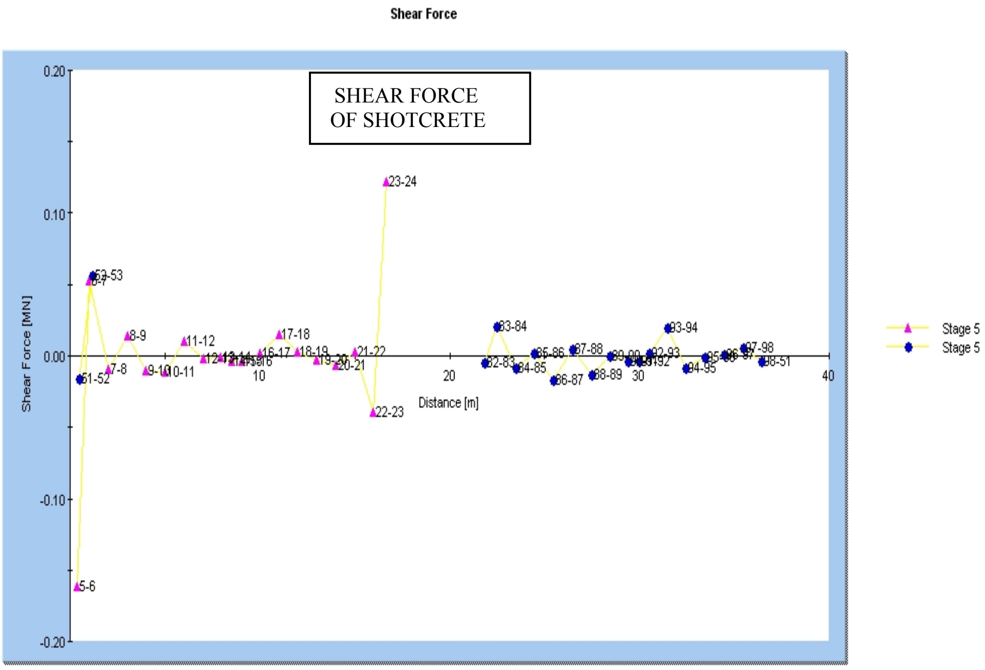


Figure 5.8 Moment and shear forces on shotcrete elements at fifth stage

Total displacements originated after bench excavation and support at left tube and right tubes are present in Figure 5.9 with their displacements vectors. There is no change in total displacements both at sixth stage where left tube bench excavation and supporting finished and seventh stage where right tube bench excavation and supporting is completed as expected in this homogenous rock conditions without big scale discontinuities. Total displacements are 23 mm at the roof and 33 mm at the base of left tube excavation and 14 mm at the roof and 37 mm at the base of right tube excavation respectively.

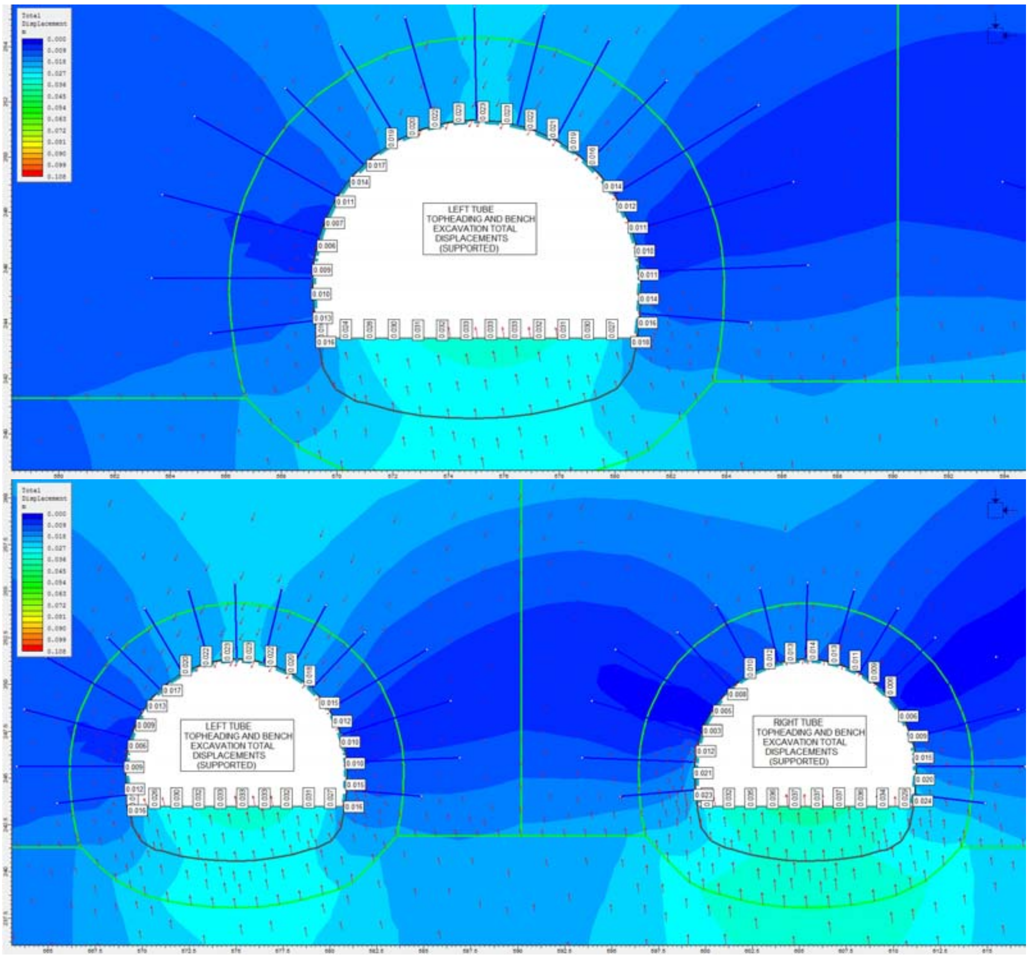
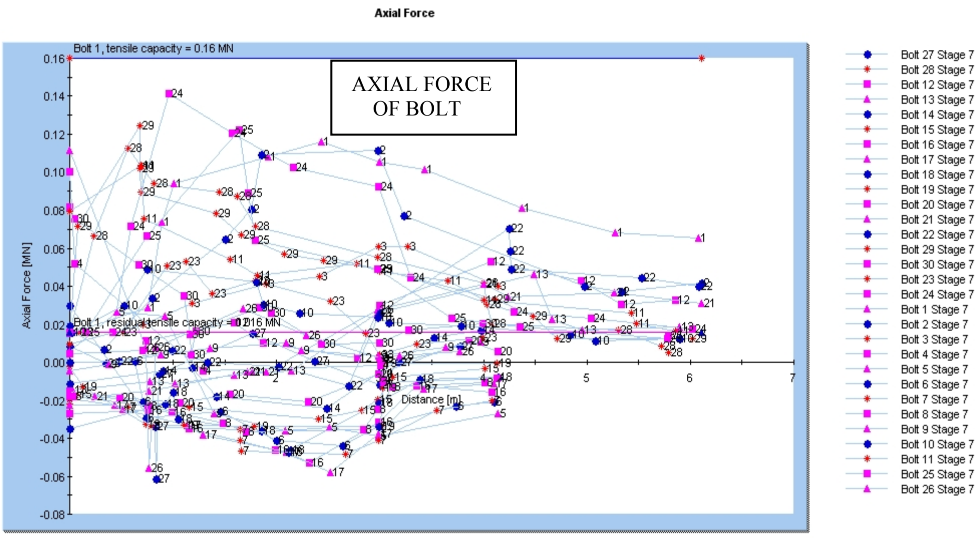


Figure 5.9 Total displacements of left and right tubes at sixth and seventh stage

Figure 5.10 show axial forces developed on rockbolts and shotcrete elements at the bench of left and right tubes. Maximum axial force on bolts is increased to 14 tonnes from 9 tonnes at twentyforth bolt element at side wall of left tube indicating that the capacity (16 tonnes) came close. Maximum axial force on shotcrete is slightly decreased to 310 tonnes from 340 tonnes.



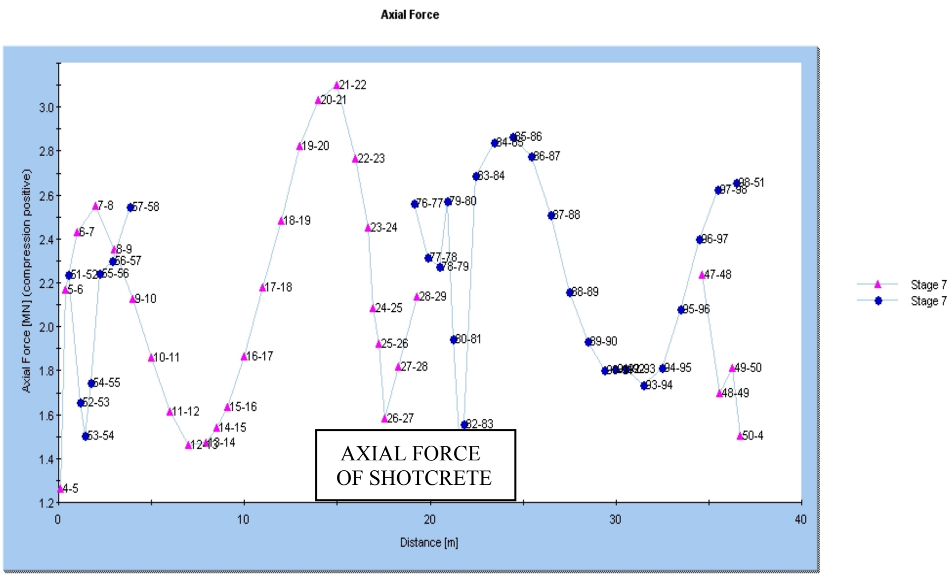


Figure 5.10 Axial forces on rockbolts and shotcrete elements at seventh stage

Opposing to slight decrease in axial force on shotcrete elements, bending moment and shear forces are considerably increased to 6.5 ton*m and

20 tonnes from 2.5 ton*m and 16 tonnes respectively at roofs and side walls of right tube as shown in Figure 11. This load increase on bolt and shotcrete elements clearly indicate the effect of growth of excavation geometry on total stability.

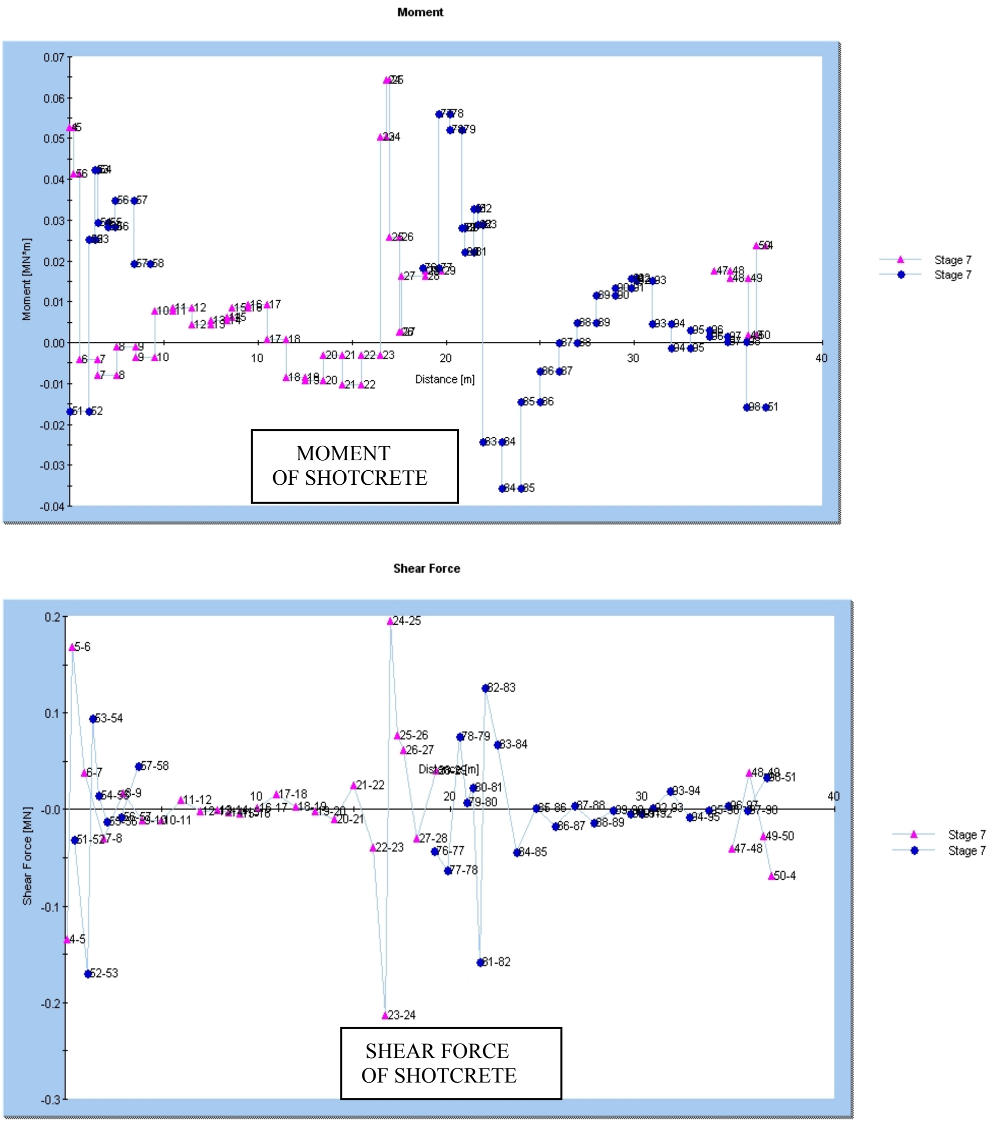


Figure 5.11 Moment and shear forces on shotcrete elements at seventh stage

After completion of full section excavation and support of left and right tubes (stages eight and nine), total displacements are present in Figure 5.12 with their displacements vectors. There is again no change in total displacements compared to previous stages and total displacements are **24 mm for left tube** and **14 mm for right tube**. Total displacements at base are about 35 mm even with invert geometry and shotcrete with 15 cm thickness. This indicates the necessity of invert geometry.

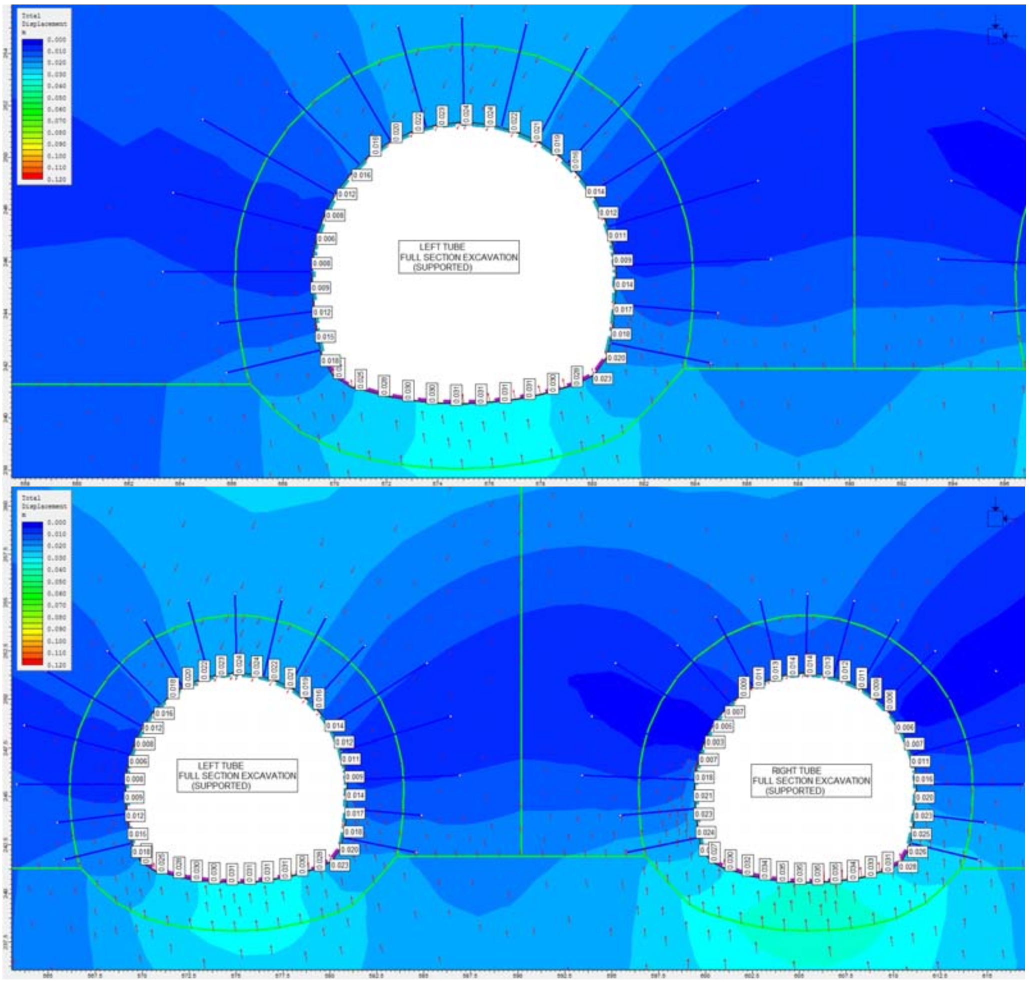


Figure 5.12 Total displacements of left and right tubes at full section excavation

Axial forces developed on rockbolts and shotcrete elements at the full section excavation (nineth stage) of left and right tubes are shown in Figure 5.13. Maximum axial force originated on bolt elements are **14 tonnes** for element number 23 of left tube and 29 of right tube side walls. There is no change in max. load compared to previous stage. Only element number is changed. Maximum axial loads carried by shotcrete elements are **310 tonnes** on element number 21-22 as axial force at right tube.

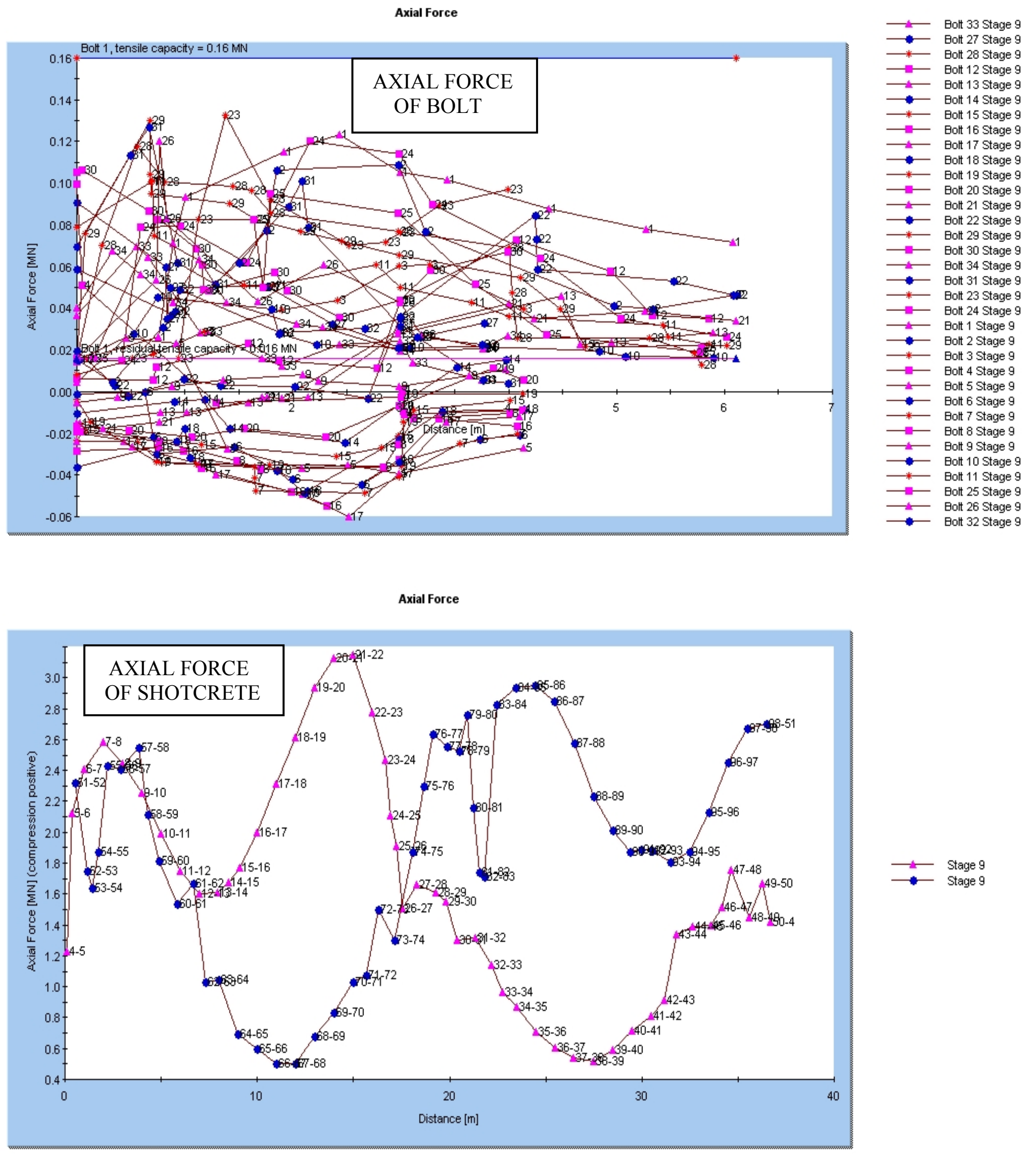
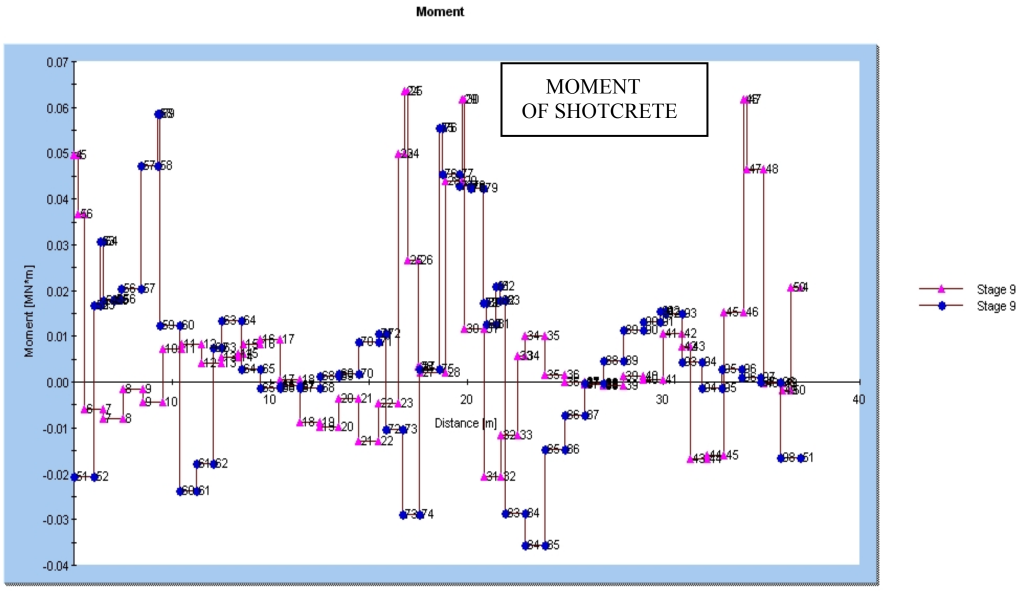


Figure 5.13 Axial forces on rockbolts and shotcrete elements at full section excavation

Figure 5.14 shows moment and shear forces originated on shotcrete elements at nineth stage (full section excavation) of left and right tubes. Maximum loads carried by shotcrete elements are **6.5 ton*m** on element number 24-25 as bending moment at right tube and **33 ton** on element number

46-47 as shear force at right tube. Compared to previous stage, there is an increase on shear forces and no change on axial forces and moments. This indicates the positive effect of ring geometry with invert excavation on the total stability, altough the excavation geometry is increased.



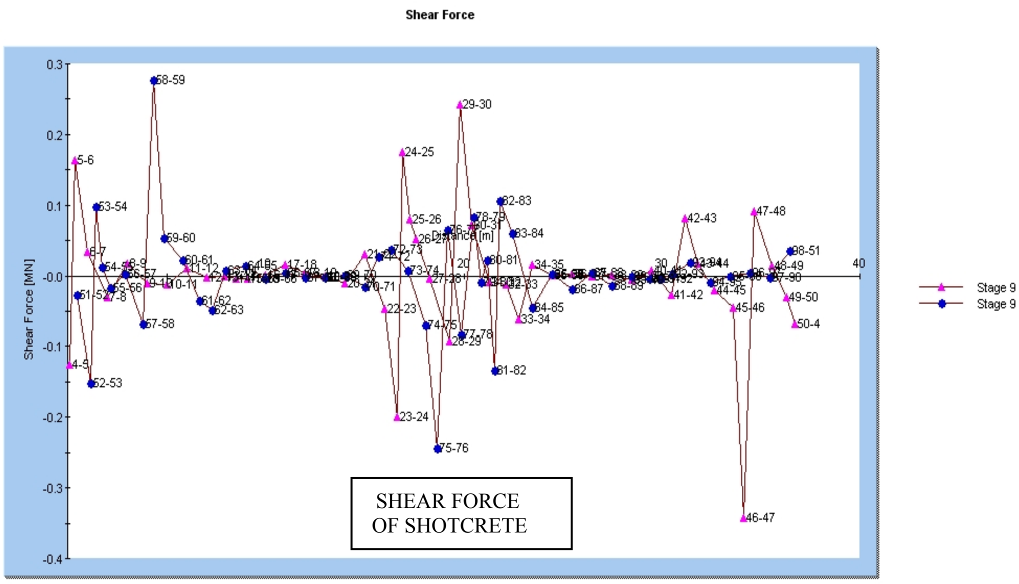


Figure 5.14 Moment and shear forces on shotcrete elements at nineth stage

As seen from the previous figures, the loads developed on support elements, which considerably increased between fifth stage, where topheading excavation and support are completed, and seventh stage, where bench excavation and support are finished, is almost not changed because of the ring geometry of excavation, between seventh stage and nineth stage, where full section excavation of left and right tubes is completed. The same condition is valid for total displacements. There is also no yield in support elements and finite element solutions are converged according to principals and explanations emphasized in this study. This also means that, distance between two tubes is properly selected and creates no instability even in weak rock conditions.

Maximum total displacements originated in the last stage of the model is about 2.4 cm in the left tube and 1.4 cm in the right tube accordingly. Smaller displacements are expected results especially in nonsqueezing conditions. But for safety and economical conditions to obtain final theoretical crossection during excavation, appropriate displacement tolerance is supposed to be **3-5 cm** for B3 excavation and support system.

In Figure 5.13, it is seen that maximum axial force originated on bolt elements are 14 tonnes for element number 23 of left tube and 29 of right tube side walls. This maximum load is below the bearing capacity of rock bolt which verifies that the interval, pattern, length and diameter of bolts are appropriately selected for B3 excavation and support system.

In the last stage of the model, maximum axial and shear forces and bending moments that bolt and shotcrete elements have been exposed are;

21-22 Element;
M=0.013 MN*m=1.3 ton*m=130 000 kg*cm
N=3.10 MN=310 tonnes=310 000 kg (max. value)

24-25 Element;
M=0.065 MN*m=6.5 ton*m=650 000 kg*cm (max. value)
N=2.0 MN=200 tonnes=200 000 kg

46-47 Element;
T=0.33 MN=33 tonnes= 33 000 kg (max. value)

Minimum and maximum stress concentrations constituted by the axial force-bending moment on the shotcrete shell of 20 cm in thickness will be;

21-22 Element
Sxx,max =310000/(100*20)+6*130000/(100*20^2)
=155+19.5=174.5 kg/cm^2
Sxx,min =155-19.5 =135.5 kg/cm^2

24-25 Element
Sxx,max=200000/(100*20)+6*650000/(100*20^2)
=100+97.5=197.5 kg/cm^2
Sxx,min =100-97.5=2.5 kg/cm^2

Under these maximum stress concentrations, there will not be any instability with C20-25 class shotcrete of 200 kg/cm^2 load carrying capacity. Convergence of the finite element solutions and no yield in support elements verify this situation.

Also average shear stress constituted by shear force on shotcrete will be;
t= 33000/(20*100)=13.50 kg/cm^2 which can be easily carried by shotcrete, steel rib and wire mesh combination.

As a result, it is concluded that excavation and support systems suggested B3 Rock Class of Çubukbeli Tunnel in Table 4.2 in Chapter 4 and Figure C.3 in Appendix C are found satisfactory.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

The main objective of this study is to provide the assessment of engineering geological characteristics of rock masses and to achieve the appropriate excavation and support systems and stabilization techniques along Çubukbeli tunnel. For this aim, a detailed engineering geological study is carried out in project area including field investigations, drillings and laboratory testing.

5.1 Conclusions

Based on the explanations and investigations carried out in this study, the following conclusions are drawn:

1. The rock masses are classified based on the RMR, Q, NATM and GSI classification systems and divided into five categories. i) Entrance Section in Beydağları limestones with poor to fair quality rock classified as B3 according to NATM , ii) Middle Section between Km:42+650-43+000 in Beydağları formation with fair to good rock classified as B1-B2 according to NATM, iii) Middle Section between Km:43+000-43+650 in Beydağları formation with fair rock classified as B2-B3 according NATM, iv) Middle Section between Km:43+650-44+530 in Çamlıdere olistostrome with poor rock classified as C2 according to NATM, v) Exit Section in Çamlıdere olistostrome with very poor to poor rock classified as C2 according to NATM.

2. Empirical Excavation and Support Systems are suggested based on NATM Classification for these sections assessing common rock characteristics.

3. The Hoek-Brown material constants m and s and elastic modulus of rock masses are obtained by using RMR (Rock Mass Rating) and GSI (Geological Strength Index).

4. The finite element software ***Phase²*** is used to determine the loads, deformations and induced stresses developed around tunnel excavation, to investigate the interaction of proposed support system with the rock mass and to verify and check the validity of suggested support systems.

5. It is concluded that the suggested empirical excavation and support systems are found satisfactory.

6. Excavation and support systems may change at any time during construction of the Çubukbeli tunnel according to rock mass classifications accomplished after each advance and monitoring measurements. This is also one of the most important feature of NATM.

5.2. Recommendations

1. During this study, a detailed engineering geological study is carried out in project area including field investigations, drillings and laboratory testing. The geological and geotechnical investigations for the future studies should be more extensive even including excavation of test galleries especially in complicated very poor and poor rock conditions to obtain insitu characteristics of rock masses. This may bring economical responsibilities during design stages, but on the other hand, will supply appropriate input both for empirical design and numerical analysis leading reduction of cost during construction.

2. Based on rock mass classifications carried out in this study, the tunnel ground is charaterized according to the RMR, Q, NATM and GSI systems. During future studies, excavation and support recommendations and strength parameters based on these classification studies should be determined with other empirical methods and numerical studies to provide crosscheck.

REFERENCES

1- Altınok Mühendislik ve Müşavirlik Ltd. Şti., 2007a. Antalya – Burdur – Keçiborlu – Sandıklı Devlet Yolu, Çubukbeli Tüneli Kesin Proje Raporu, TC Ulaştırma Bakanlığı Karayolları Genel Müdürlüğü.

2- Altınok Mühendislik ve Müşavirlik Ltd. Şti., 2007b. Antalya – Burdur – Keçiborlu – Sandıklı Devlet Yolu, Çubukbeli Tüneli Jeolojik-Jeoteknik Kesin Proje Raporu, TC Ulaştırma Bakanlığı Karayolları Genel Müdürlüğü.

3- Barton, N., Lien, R. And Lunde, J., 1974. Engineering Classification of Masses for the Design of Tunnel Support, Rock Mechanics, Vol.6, No.4, pp.189-236.

4- Barton, N., Loset, F., Lien R., Lunde, J., 1980. Application of the Q-System in Design Decisions Concerning Dimensions and Appropriate Support for Underground Installations. Int. Conf. Subsurface Space, Rockstore, Stockholm, Subsurface Space, Vol.2, pp. 553-561.

5- Barton, N., 2002a. Some New Q-value Correlations to Assist in Site Characterisation and Tunnel Design, International Journal of Rock Mechanics and Mining Sciences, Vol. 39, pp. 185-216.

6- Barton, N., 2002b. Deformation Moduli and Rock Mass Characterization, Tunneling & Underground Space Technology., Vol. 17, pp. 221-222.

7- Bieniawski, Z.T., 1973. Engineering Classification on Jointed Rock Masses. Trans. South African Inst. Civil Engineering, Vol.15, pp. 335-344.

8- Bieniawski, Z.T., 1974. Geomechanics Classification of Rock Masses and its Application in Tunneling Proc. 3rd Congress of International Society for Rock Mechanics, Denver, Vol. 2, pp. 27-32.

9- Bieniawski, Z.T., 1976. Rock Mass Classifications in Rock engineering, Proceedings Symposium on Exploration for rock Engineering, (ed. Z.T. Bieniawski), A.A. Balkema, Rotterdam, pp. 97-106.

10- Bieniawski, Z.T., 1978. Determining Rock Mass Deformability Experience from Case Histories, International Journal of Rock Mechanics and Mining Sciences, Vol. 15, pp. 237-247.

11- Bieniawski, Z.T., 1979. The Geomechanics Classification in Rock Engineering Applications, Proc. 4th International Congress Rock Mechanics, ISRM, Montreaux, A.A.Balkema, Rotterdam, Vol.2, pp. 51-58.

12- Bieniawski, Z.T., 1989. Engineering Rock Mass Classifications, Wiley, Newyork, 251 pages.

13- Brebbia C.A., Walker S., 1978. Recent advances in boundary element methods. Conference, Boundary Element Methods, Southampton University.

14-Brown, E.T., Bray, J.W., Ladanyi, B. and Hoek, E. , 1983. Characteristic line calculations for rock tunnels. J. Geotech. Engng. Div., ASCE 109, 15-39.

15-Cai, M., Kaiser, P.K., Uno, H., Tasaka, Y., Minami, M., 2004. Estimation of Rock Mass Deformation Modulus and Strength of Jointed Hard Rock Masses Using the GSI System, International Journal of Rock Mechanics and Mining Sciences, Vol. 41, pp. 3-19.

16-Coşar, S., 2004. Application of Rock Mass Classification Systems for Future Support Design of the Dim Tunnel near Alanya-M.Sc. Thesis, 217 p.

17-Cundall, P. A. 1976. "Explicit Finite Difference Methods in Geomechanics , Numerical Methods in Engineering," Proceedings of the EF Conference on Numerical Methods in Geomechanics, Blacksburg, Virginia. Vol. 1: pp. 132-150.

18-Cundall, P. A., and Hart, R. D. 1993. “Numerical Modeling of Discontinue,” Comprehensive Rock Engineering, Vol.2 , J. A. Hudson , ML , Pergammon Press: pp. 231-243.

19-Duncan Fama, M.E. 1993. Numerical modeling of yield zones in weak rocks. In comprehensive rock engineering, (ed. J.A. Hudson) 2, 49-75. Oxford: Pergamon.

20- Deere, D.U., Hendron, A.J., Patton, F.D. and Cording, E.J., 1967. Design of Surface and Near Surface Construction in Rock. In Failure and breakage of Rock, Proc. 8th U.S. Symposium Rock Mechanics, (ed. C.Fairhurst), New York. Soc. Min. Engr. Am. Inst. Metall. Petrolm. Engrs., pp. 237-302.

21-Fenner, R. 1938. Untersuchungen zur Erkentniss des Gebirgsdrucks. Glukauf 74, pp. 681-695,705-715.

22-Feder, G. (1978). Versuchsergebnisse und analytische Ansaetze zum Scherbruchmechanismus im Bereich tiefliegender Tunnel. Rock Mechanics, Vol.6, pp. 71-102

23-Geoconsult ZT GmbH, 1993. Technical Specification for Civil Underground Tunnel Works, Prepared for the General Directorate of Highways Republic of Turkey , Austria.

24-Geoconsult/Tekfen Impresit J.V., 1994. Training Lectures on Engineering Geology and NATM Tunnelling, Prepared for the General Directorate of Highways Republic of Turkey , Ankara.

25-Goodman, R.E., Shi, G., 1985. Block Theory and its application in Rock Engineering. by Prentice Hall, Englewoods Cliffs NJ, 338 p.

26-Goodman, R.E., 1993. Engineering Geology : Rock in Engineering Construction, by John Wiley & Sons, Inc., 412 p..

27-Goel, R.K. and Jethwa, J.L., 1991. Prediction on Support Pressure Using RMR Classification. Proc. Indian Conf., surat, India, pp. 203-205.

28-Gökçeoğlu, C., Aksoy, H., 2000. New Approaches to the Characterization of Clay-bearing, Densely Jointed and Weak Rock Masses, Engineering Geology, Vol. 58, pp. 1-23.

29-Gökçeoğlu, C., Sönmez, H., Kayabaşı, A., 2003. Predicting the Deformation Moduli of Rock Masses, International Journal of Rock Mechanics and Mining Sciences, Vol. 40, pp. 701-710.

30-Grimstad, E. and Barton, N., 1993. Updating the Q-System for NMT. Proc. International Symposium on Sprayed Concrete – Modern Use of Wet Mix Sprayed Concrete for underground Support, Oslo, Norwegian Concrete Association.

31-Günay, Y. ve diğ.,1982. Beydağlarının Stratigrafisi ve Yapısı: Türkiye Altıncı Petrol Kongresi, Ankara, pp. 91-100.

32-Hoek, E. 1983. Strength of jointed rock masses, 23rd. Rankine Lecture. Géotechnique 33(3), pp.187-223.

33-Hoek, E. 1990. Estimating Mohr-Coulomb friction and cohesion values from the Hoek-Brown failure criterion. Intnl. J. Rock Mech. and Mining Sci. and Geomechanics Abstracts. 12(3), 227-229.

34-Hoek, E. 1994. Strength of rock and rock masses, ISRM News Journal, 2(2), pp.4-16.

35-Hoek, E., 1999. Putting Numbers to Geology – An Engineer's Viewport, Quarterly Journal of Engineering Geology, Vol. 32, pp. 1-19.

36-Hoek, E., 2000. Rock Engineering Course Notes, http://www.rocscience/ Hoek's Corner, last date accessed April 2009.

37-Hoek, E., 2004a. A Brief History the Development of the Hoek - Brown Failure Criterion, Discussion Paper # 7, http:// www.rocscience.com/Hoek's Corner, 9 p, last date accessed June 2004

38-Hoek, E., 2004b. Estimates of Rock Mass Strength and Deformation Modulus, Discussion Paper # 4, http://www.rocscience.com/Hoek's Corner, 6 p.

39-Hoek, E. and Brown E.T., 1980. Underground Excavations in Rock, The Institution of Mining and Metallurgy, London, pp.31-34.,

40-Hoek E and Brown E.T. 1988. The Hoek-Brown failure criterion - a 1988 update. Proc. 15th Canadian Rock Mech. Symp. (ed. J.H. Curran), pp. 31-38. Toronto:Civil Engineering Dept., University of Toronto

41-Hoek, E. and Brown, E.T. 1997. Practical estimates or rock mass strength. Intnl. J. Rock Mech. and Mining Sci. And Geomechanics Abstracts. 34(8), pp.1165-1186.

42-Hoek, E., Wood, D. and Shah, S. 1992. A modified Hoek-Brown criterion for jointed rock masses. Proc. rock characterization, symp. Int. Soc. Rock Mech.:Eurock '92, (J. Hudson ed.), pp. 209-213.

43-Hoek, E., Kaiser, P.K., Bawden, W.F., 1995. Support of Underground Excavations in Hard Rock, A.A. Balkema, Roterdam, Brookfield, 217 p.

44-Hoek, E., Marinos, P. and Benissi, M., 1998. Applicability of the GeologicalStrength Index (GSI) classification for very weak and sheared rock masses. The case of the Athens Schist Formation. Bull. Engg. Geol. Env. 57(2), pp. 151-160.

45-Hoek, E., Carranza-Torres, C., Corkum, B., 2002. Hoek-Brown Failure Criterion – 2002 Edition, Proc. NARMS-TAC Conference, Toronto, 2002, **1**, pp. 267-273.

46-Hoek, E., Marinos, P., Marinos, V. 2005. The Geological Strength index:applications and limitations,Bull. Eng. Geol. Environ.,Vol. 64, pp.55-65

47-Hoek, E., Marinos, P., Marinos, V. 2005. Characterization and engineering properties of tectonically undisturbed but lithologically varied sedimentary rock masses, International Journal of Rock Mechanics and Mining Sciences, Vol. 42,pp. 277-285

48-Hoek, E., Marinos, P., Marinos, V. 2006. Variability of the engineering properties of rock masses quantified by the geological strength index: the case of ophiolites with special emphasis on tunnelling. Bull. Eng. Geol. Env., Vol. 65, pp.129-142.

49-Hoek, E., Diederichs, M.S., 2006. Empirical estimation of rock mass modulus. International Journal of Rock Mechanics and Mining sciences, Vol. 43, pp.203–215.

50-ISRM (International Society for Rock Mechanics), 1981. Rock Characterization, Testing and Monitoring – ISRM Suggested methods, Pergamon Press, Oxford, E.T. Brown (ed), 211 p.

51-Karahan E., Coşar S., Sözer Ş., Kaya K., 2005, Rock Mass Classification and Support Design of Kuskunkıran Tunnel in Van, Proceedings of the 31st ITA-AITES World Tunnel Congress,7-12 May 2005, İstanbul, Turkey, Vol.2, pp. 701-706.

52-Karayolları Genel Müdürlüğü, 1997, NATM Uygulamalı Yeraltı Tünel İşleri Teknik Şartnamesi, 105 p.

53-Karayolları Genel Müdürlüğü, 2006, Karayolu Teknik Şartnamesi, pp.306-454

54-Kayabaşı, A., Gökçeoğlu, C., and Ercanoğlu, M., 2003. Estimating the Deformation Modulus of Rock Masses: A Comparative Study", International Journal of Rock Mechanics and Mining Sciences, Vol. 40, pp. 55-63.

55-Kovari, K., 1994. Erroneous Concepts behind the New Austrian Tunneling Method, Tunnel, Vol. 1.

56-Kovari, K. 2004. History of Rock Bolts and the Spread Concrete Lining Method; Aachen International Mining Symposium, Aachen, pp. 18-66.

57-Lauffer, H., 1958. Gebirgsklassifizierung für den Stollenbau. Geol. Bauwesen, Vol. 24, pp. 46-51.

58-Lauffer, H., 1988. Zur Gebirgsklassifizierung bei Frasvortrieben, Felsbau. Vol. 6, pp. 137-149.

59-Mitri, H.S., Edrissi, R. and Henning, J., 1994. Finite element Modeling of Cable-bolted Stopes in Hard Rock Ground Mines. SME Annual Meeting. Albuquerque, Mexico, pp. 94-116.

60-Müller, L., 1978. Removing Misconceptions on the New Austrian Tunneling Method, Tunnels Tunneling, Vol. 10, pp. 667-671.

61-Nicholson, G.A. and Bieniawski, Z.T., 1990. A nonlinear deformation Modulus Based on Rock Mass Classification, International Journal of Min. Geol. Eng., Vol.8, pp. 181-202.

62-Özan, İ. And Ünal, E., 1996. Kaya kütlesi sınıflama sistemleri üzerine kritik bir değerlendirme, 3. Ulusal Kaya Mekaniği Sempozyumu Bildirileri Kitabı, Ankara, 181-193.

63-Palmström, A., 1982. The Volumetric Joint Count – A Useful and Simple Measure of the Degree of Rock Jointing. Proc. 4th Congress IAEG, New Delhi, Vol. 5, pp. 221-228.

64-Palmström, A., 1996. Characterizing Rock Masses by Rmi for Using in Practical Rock engineering (Part 1: The Development og the Rock Mass Index Rmi, tunneling Underground Space Technology, Vol. 11 (2), pp. 175-188.

65-Palmström, A. and Singh, R., 2001. The Deformation Modulus of Rock Masses – Comparisons between In-situ Tests and Indirect Estimates, Tunneling Underground Space Technology, Vol. 16, pp. 115-131.

66-Phase^2, ver. 6.028, 2008. A two-dimensional elasto-plastic finite element program and its user's manual, by Rocscience Inc, Toronto-Canada.

67-Rabcewicz, L. 1964. The New Austrian Tunneling Method, Part I. Water Power, pp. 453-457.

68-Roclab, ver. 1.031, 2007. A software program for determining rock mass strength parameters based on the generalized Hoek-Brown failure ctriterion, by Rocscience Inc, Toronto-Canada.

69-Serafim, J.L. and Pereira, J.P., 1983. Considerations of the Geomechanics Classification of Bieniawski. Proc. International Symposium on Geol. And Underground Const., LNEC, Lisbon, Portugal, Vol. 1, pp. 33-42.

70-Singh B., Goel R.K.., 1999. Rock Mass Classification , A Practical Approach in Civil Engineering, by Elsevier Ltd., 267 p.

71-Sönmez, H. and Ulusay, R., 1999. Modifications to the Geological Strength Index (GSI) and Their Applicability to Stability of Slopes, International Journal of Rock Mechanics and Mining Sciences, Vol. 36, pp.743-760.

72-Sönmez, H. and Ulusay, R., 2002. A Discussion on the Hoek-Brown Failure Criterion and Suggested Modifications to the Criterion Verified by Slope Stability Case Studies. Yerbilimleri (Earth Sciences), Vol 26, pp. 77-99.

73-Sönmez, H., Gökçeoğlu, C. and Ulusay, R., 2003. An Application of Fuzzy Sets to the Geological Strength Index (GSI) System Used in Rock Engineering, Engineering Application of Artificial Intelligence, Vol. 16, pp 251-269.

74-Sönmez, H., Gökçeoğlu, C., Ulusay, R., 2004a. A Mamdani Fuzzy Inference System for the Geological Strength Index (GSI) and its Use in Slope Stability Assessments-Sinorock2004 Symposium, International Journal of Rock Mechanics and Mining Sciences, Vol. 41, pp.413-514.

75-Sönmez, H., Gökçeoğlu, C., Ulusay, R., 2004b. Indirect Determination of the Modulus of Deformation of Rock Masses Based on the GSI System, International Journal of Rock Mechanics and Mining Sciences, Vol. 41, pp. 849-857.

76-Sönmez, H., Gökçeoğlu, C., Medley, E.W., Tuncay, E., and Nefislioğlu, H.A., 2006a. Estimating the uniaxial compressive strength of a volcanic bimrock, International Journal of Rock Mechanics and Mining Sciences, Vol. 43, pp. 554-561.

77-Sönmez, H., Nefislioğlu, H.A., Gökçeoğlu, C., and Kayabaşı, A., 2006b. Estimation of rock modulus: for intact rocks with an artificial neural network and for rock masses with a new empirical equation. International Journal of Rock Mechanics and Mining Sciences, Vol. 43, pp. 224-235.

78-Spaun, G., 1977. Contractual Evaluation of Rock Exploration in Tunnelling, Exploration of Rock Engineering, Z.T. Bieniawski (ed.), A.A.Balkema, Johannesburg, Vol. 2, pp. 49-52.

79-Terzaghi, K. 1925. Erdbaumechanik auf Bodenphysikalischer Grundlage. Vienna: Franz Deuticke.

80-Terzaghi K., 1946. Rock Tunneling with Steel Supports, R.V. Practor and T. White (ed), Commercial Sheving Co., Youngstown, Ohio. 296 p.

81-Ulusay, R., 1991. Geotechnical evaluations and deterministic design considerations for pitwall slopes at Eskihisar (Yatağan-Muğla) strip coal mine. PhD. Thesis, Middle East Technical University, Geological Engineerng Department, Ankara, Turkey, 340 p.

82-Ulusay, R., Özkan, İ. and Ünal, E., 1995. Characterization of Weak, Stratified, and Clay Bearing Rock Masses for Engineering Applications. Proc. Of the Fractured and Jointed Rock Masses Conference, Lake Tahoe, California, L.R. Myer, N.G.W. Cook, R.E. Goodman, and C.F. Tsans (eds.), pp. 229-235.

83-Ulusay, R. and Sönmez, H., 2007. Kaya Kütlelerinin Mühendislik Özellikleri, TMMOB Jeoloji Müh. Odası Yayınları, No: 60, Ankara, 292 p.

84-Ünal, E., 1992. Rock Reinforcement Design and its Application in Mining, Proc. International Symposium on Rock Support, Sudbury, Ontario, Canada, pp.541-546.

85-Ünal, E., 1996. Modified Rock Mass Classification: M-RMR System – Milestones in Rock Engineering: A Jubilee Collection; Z.T. Bieniawski, Balkema, pp.203-223.

86-Ünal, E. and Özkan, İ., 1990. Determination of Classification Parameters for Clay-bearing and Stratified Rock Mass, 9th Conference on Ground Control in Mining, Morgantown, USA, 250-259.

87-Ünal, E., 2002, Rock Mass Classification Studies at Kışladağ Gold-Project Site. Project No: METU AGUDOS 02-03-05-1-00-07, 39 p., (unpublished).

88-Wickham, G.E., Tiedemann, H.R. and Skinner, E.H., 1972. Support Determination Based on Geological Predictions, International Proc. North American Rapid Excavation Tunneling Conference, Chicago, K.S. Lane and L.A. Garfield (Eds), New York, p. 43-64.

89-Yalçınkaya, S. ve diğ., 1986. Batı Torosların Jeolojisi: MTA Rap:7898, Ankara.

90-Zienkiewicz, O.C., Taylor, R.L., 2005. The Finite Element Method for Solid and Structural Mechanics, by Elsevier Ltd., 619 p.

APPENDIX A

RATINGS FOR INPUT PARAMETERS FOR ROCK MASS RATING (RMR) SYSTEM USED IN THIS STUDY

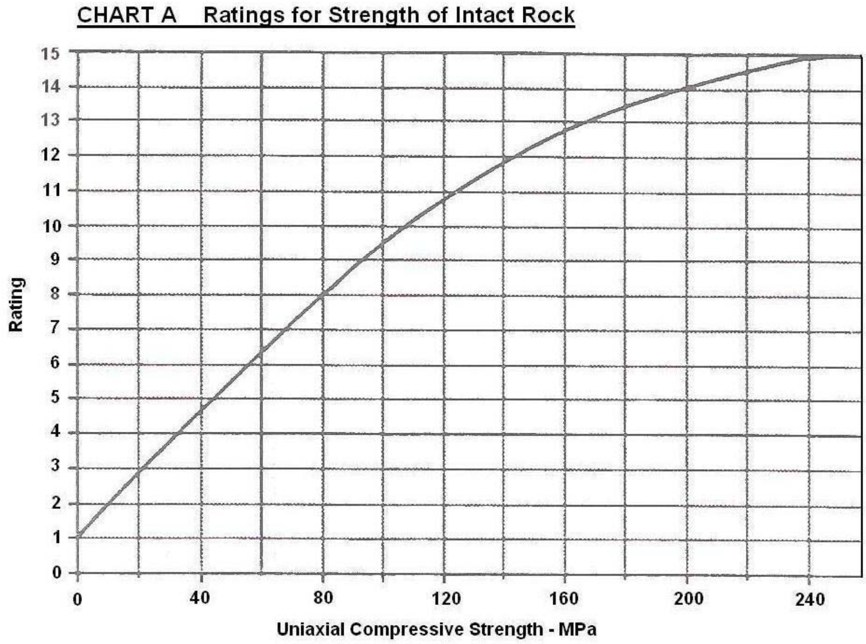


Figure A.1 Ratings for strength of intact rock (After Bieniawski, 1989)

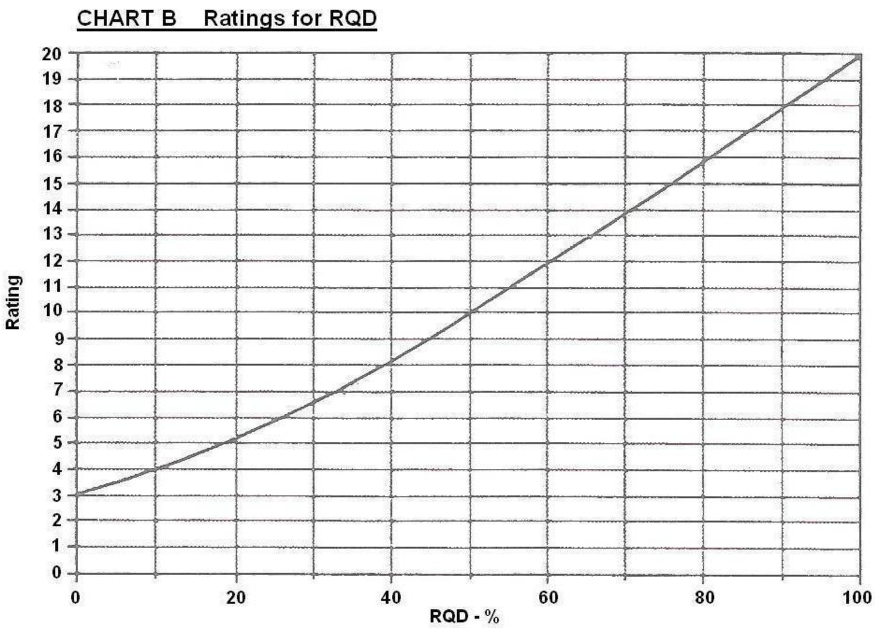


Figure A.2 Ratings for RQD (After Bieniawski, 1989)

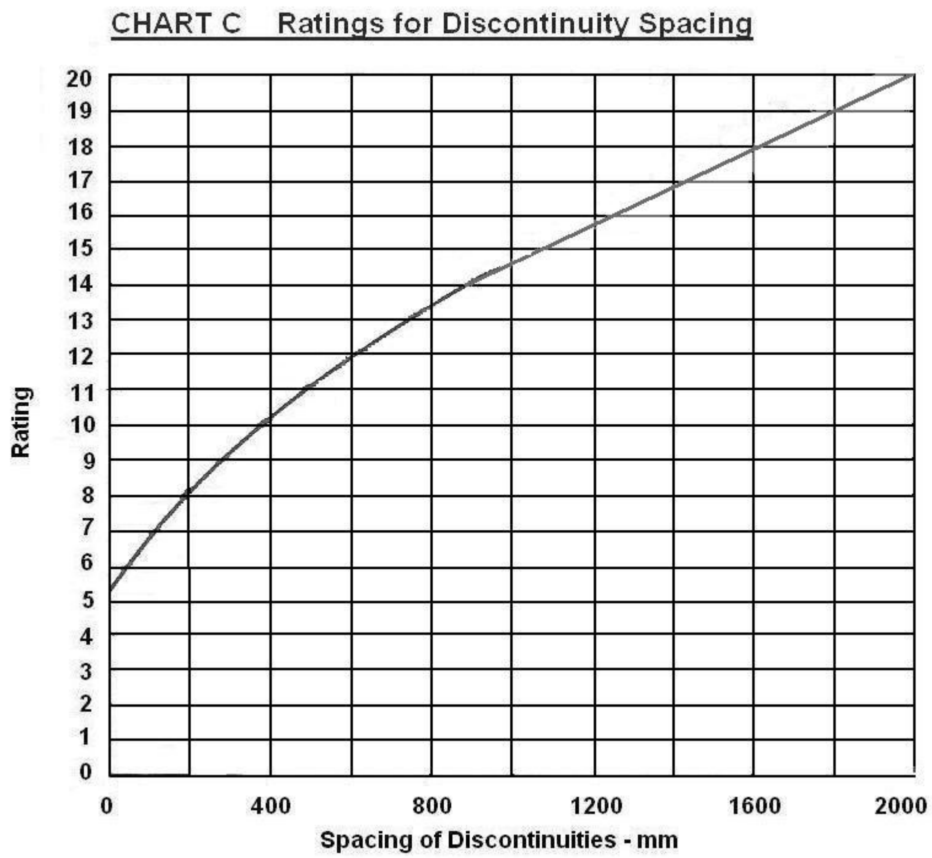


Figure A.3 Ratings for discontinuity spacing (After Bieniawski, 1989)

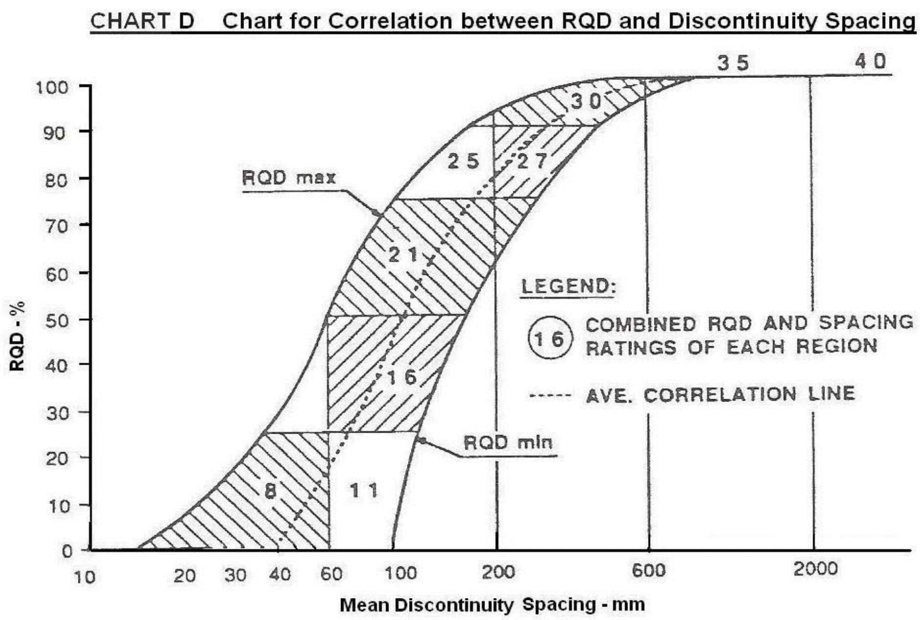


Figure A.4 Chart for correlation between RQD and discontinuity spacing (After Bieniawski, 1989)

APPENDIX B

ROCLAB OUTPUTS FOR ROCK MASS STRENGTH PARAMETERS

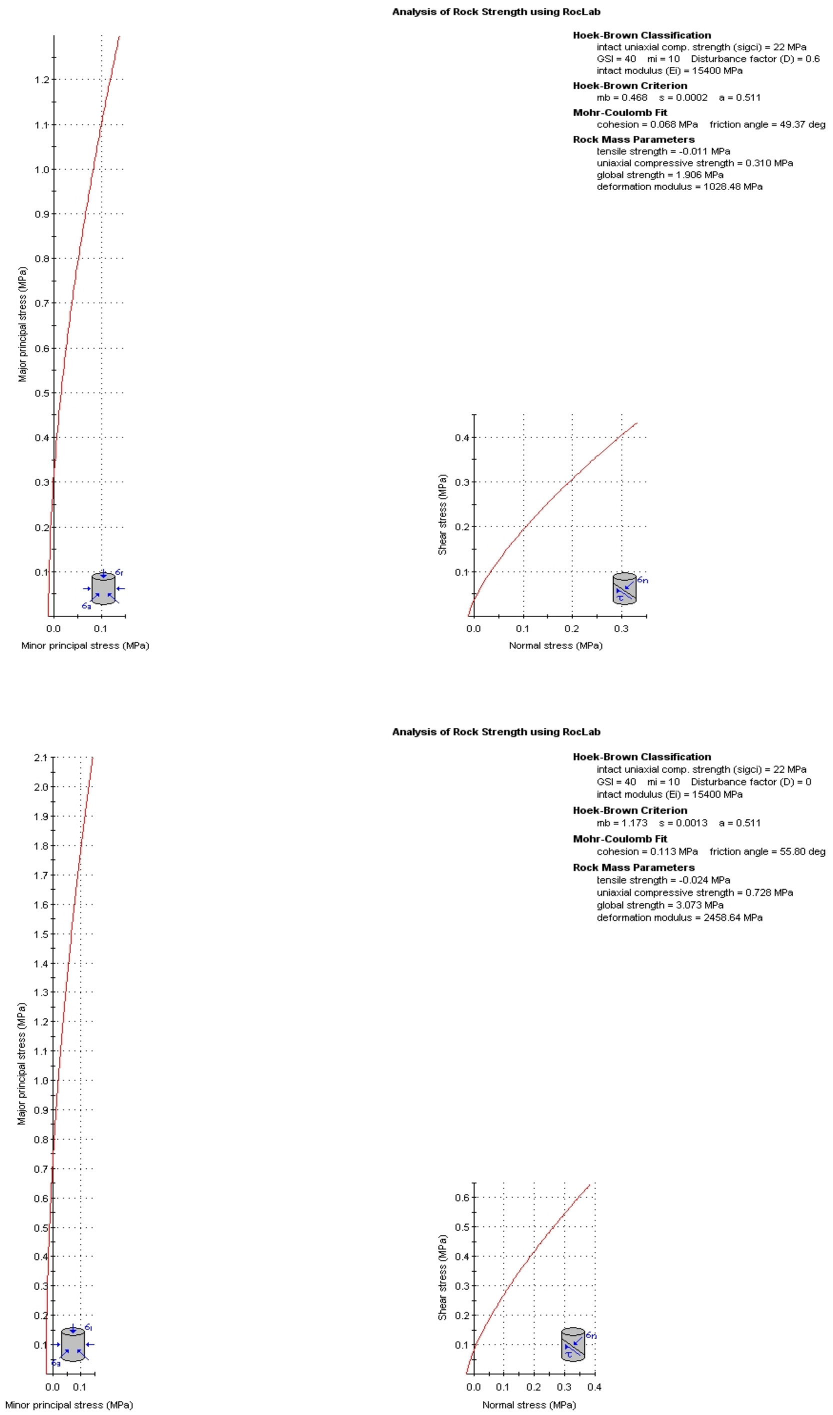
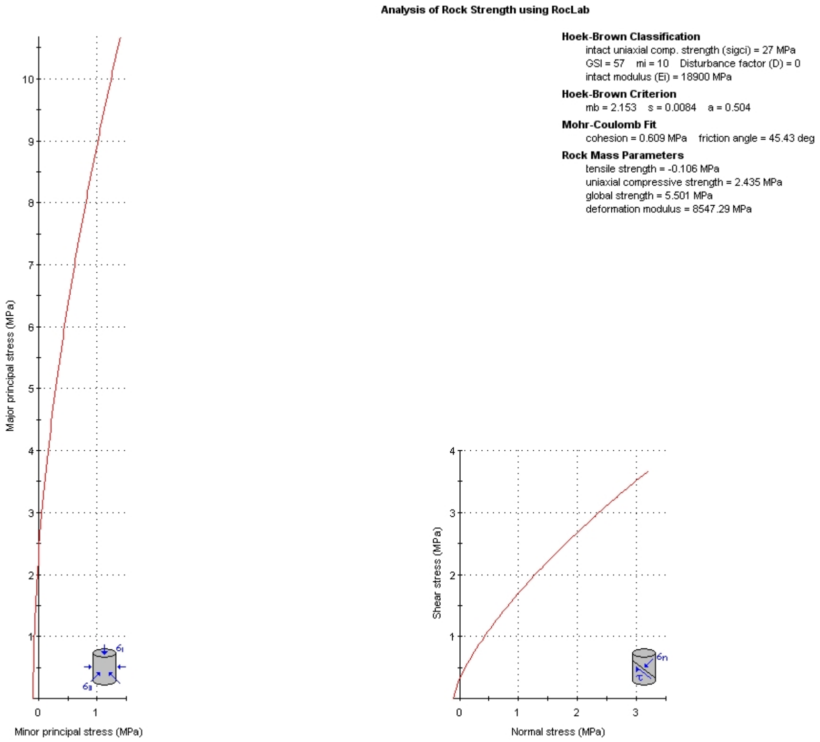


Figure B.1 ***Roclab*** Output of entrance section of Çubukbeli tunnel



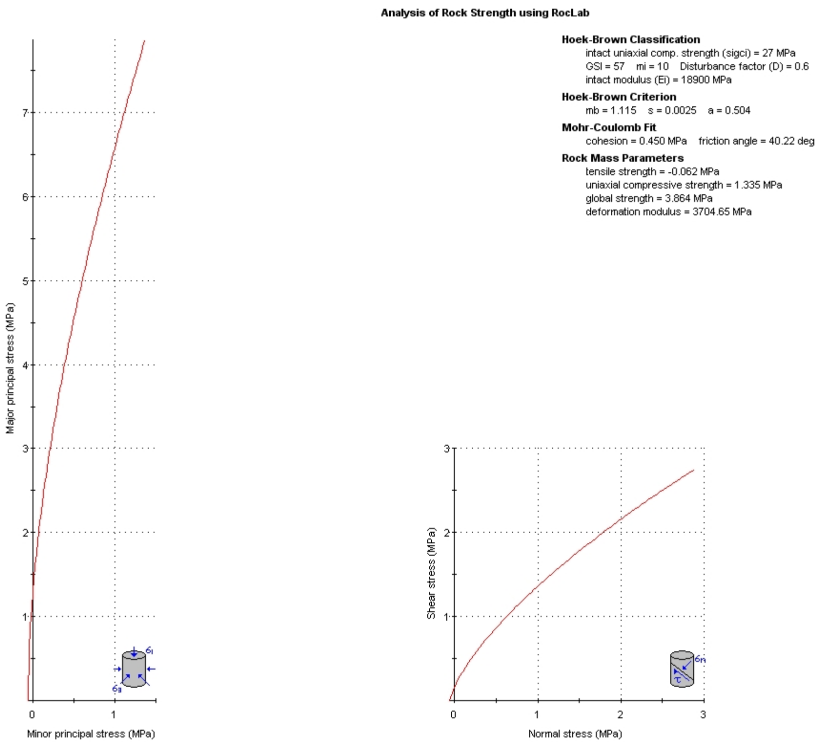
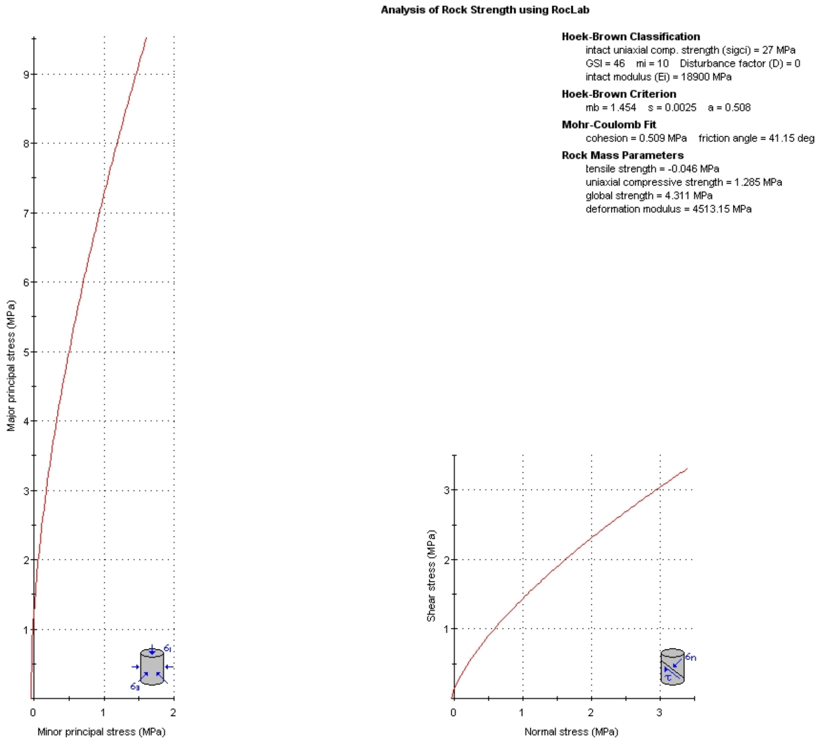


Figure B.2 ***Roclab*** Output of middle section-1 of Çubukbeli tunnel



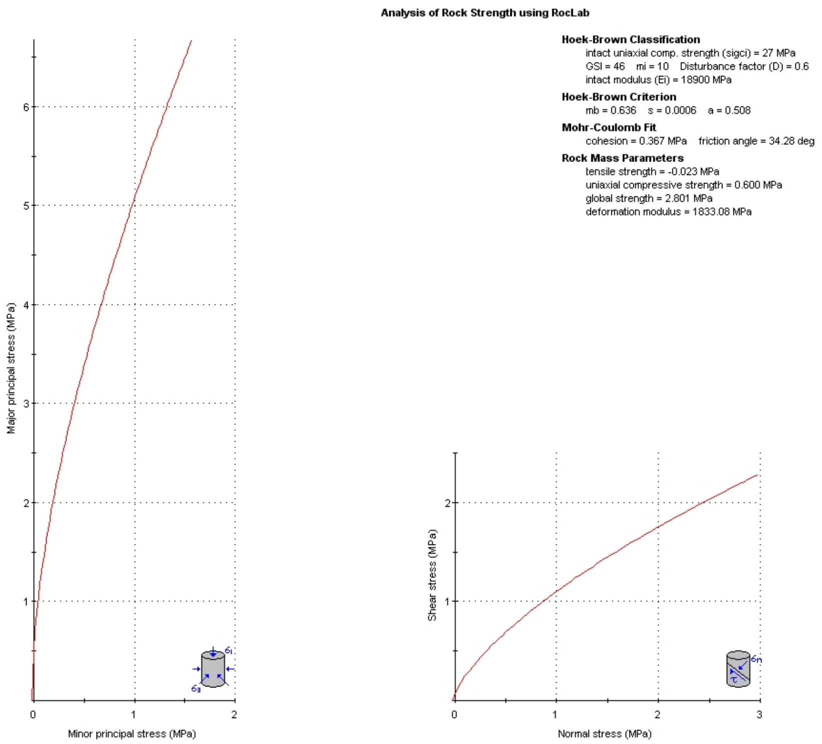
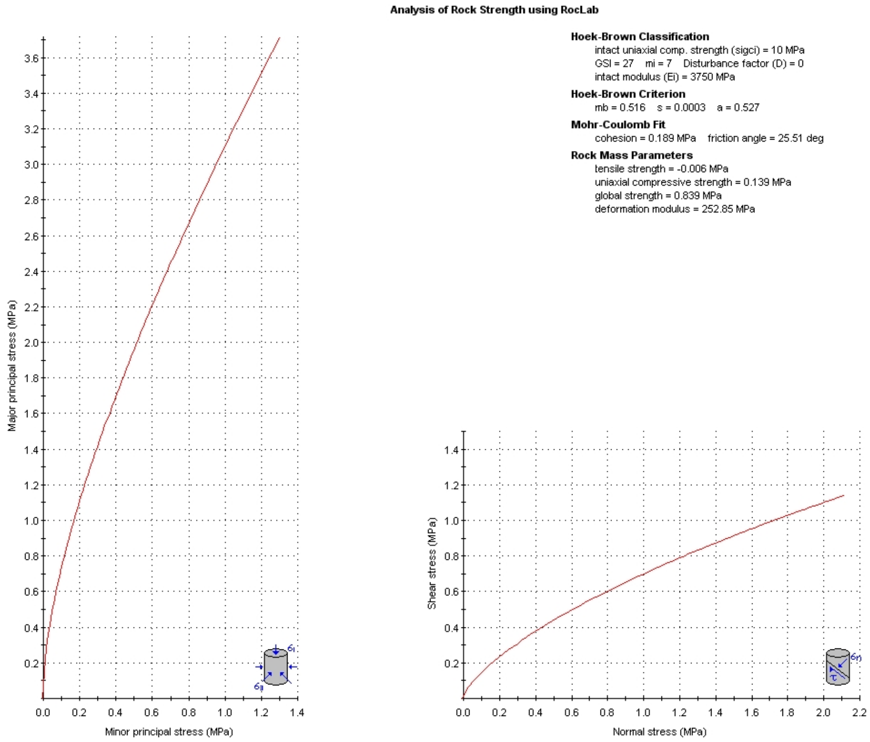


Figure B.3 ***Roclab*** Output of middle section-2 of Çubukbeli tunnel



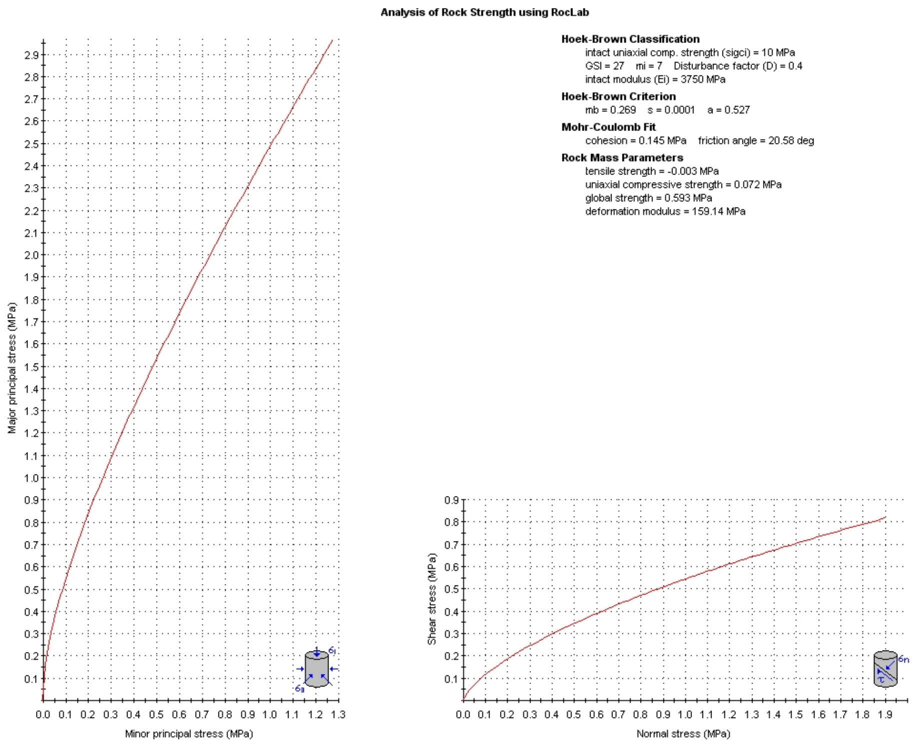
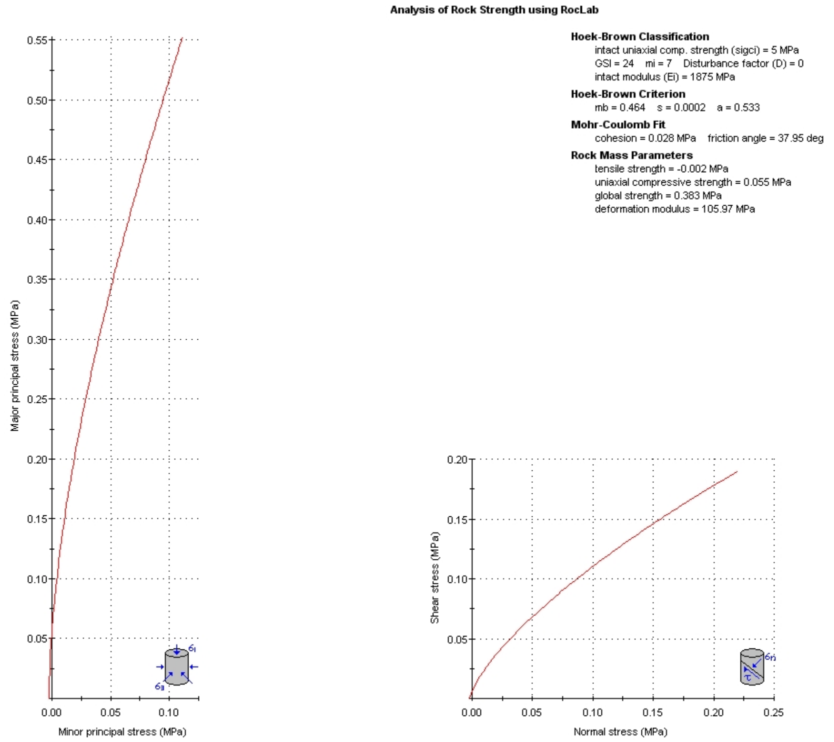


Figure B.4 ***Roclab*** Output of middle section-3 of Çubukbeli tunnel



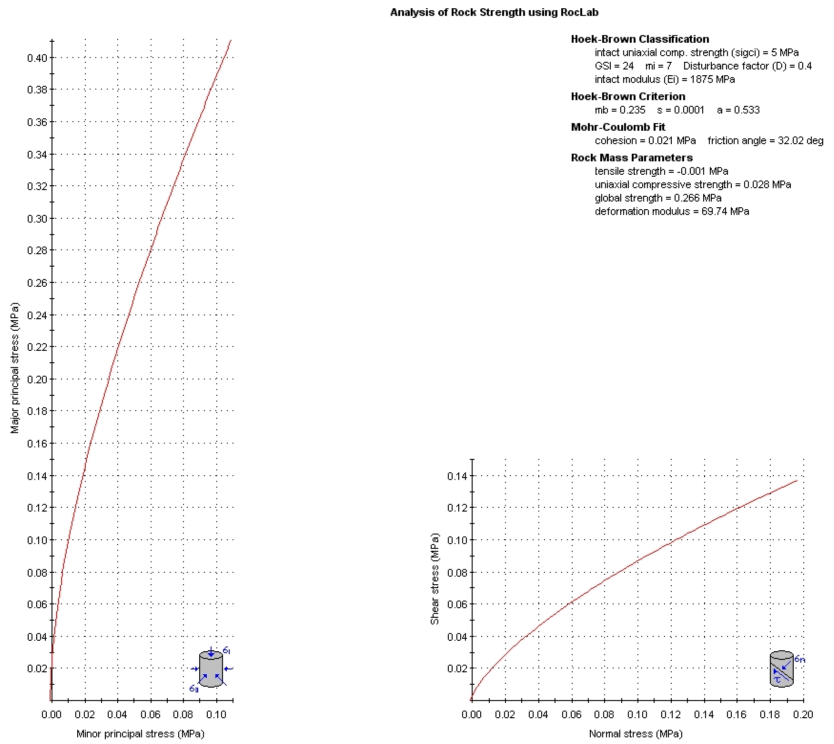


Figure B.5 ***Roclab*** Output of exit section of Çubukbeli tunnel

APPENDIX C

APPLICATION DRAWINGS OF EXCAVATION AND SUPPORT SYSTEMS OF ÇUBUKBELİ TUNNEL

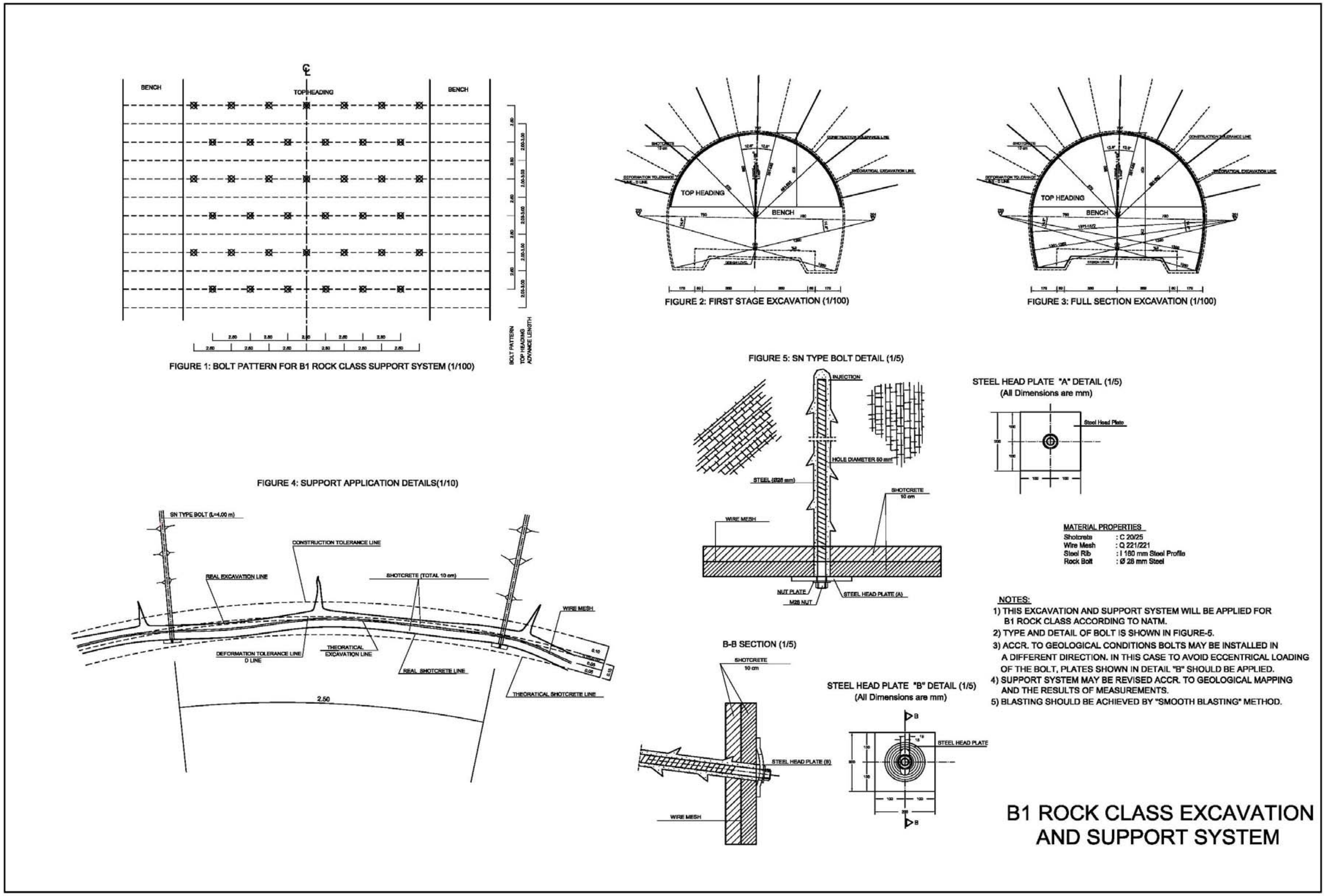


Figure C.1 B1 Rock class excavation and support system of Çubukbeli tunnel

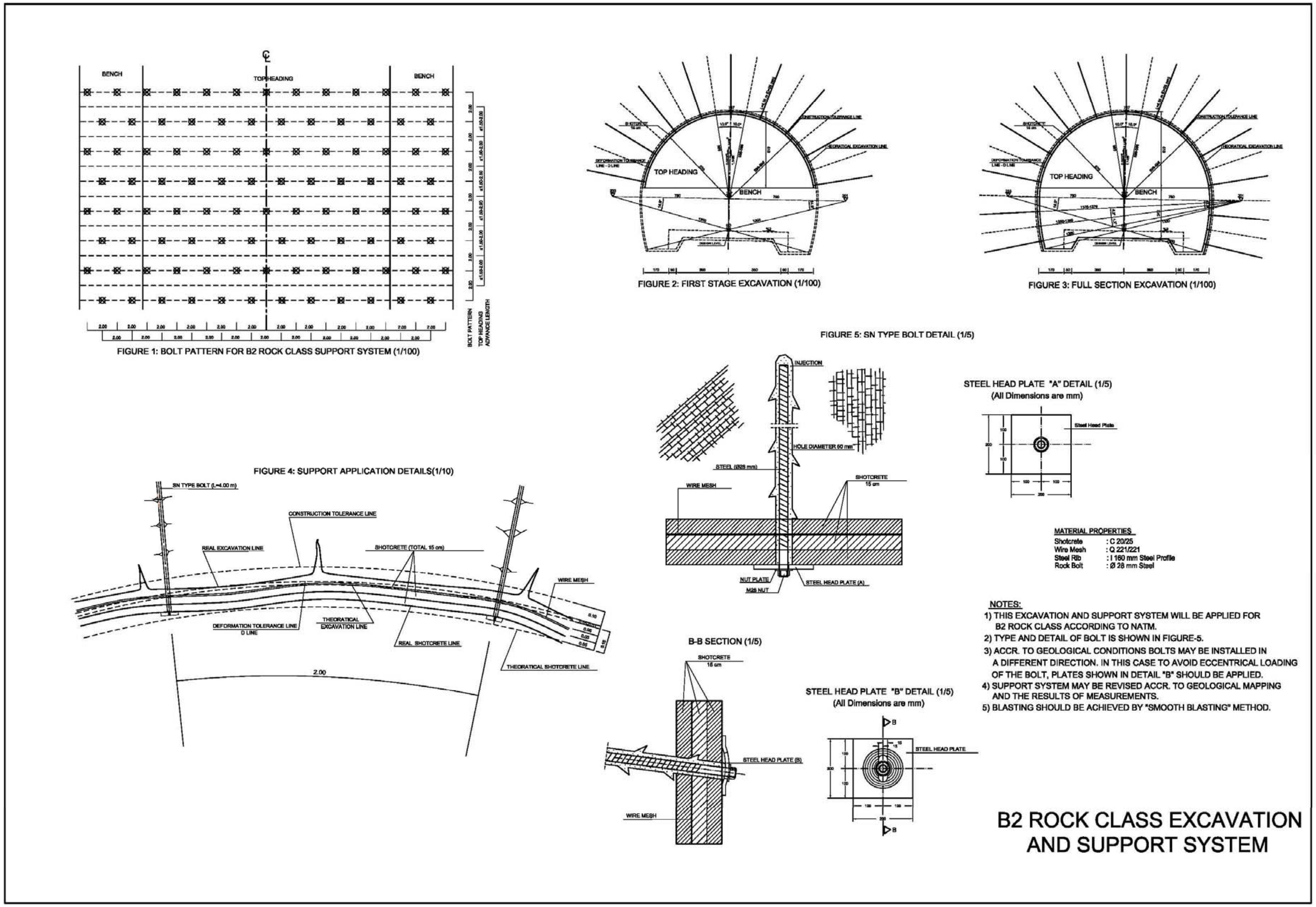


Figure C.2 B2 Rock class excavation and support system of Çubukbeli tunnel

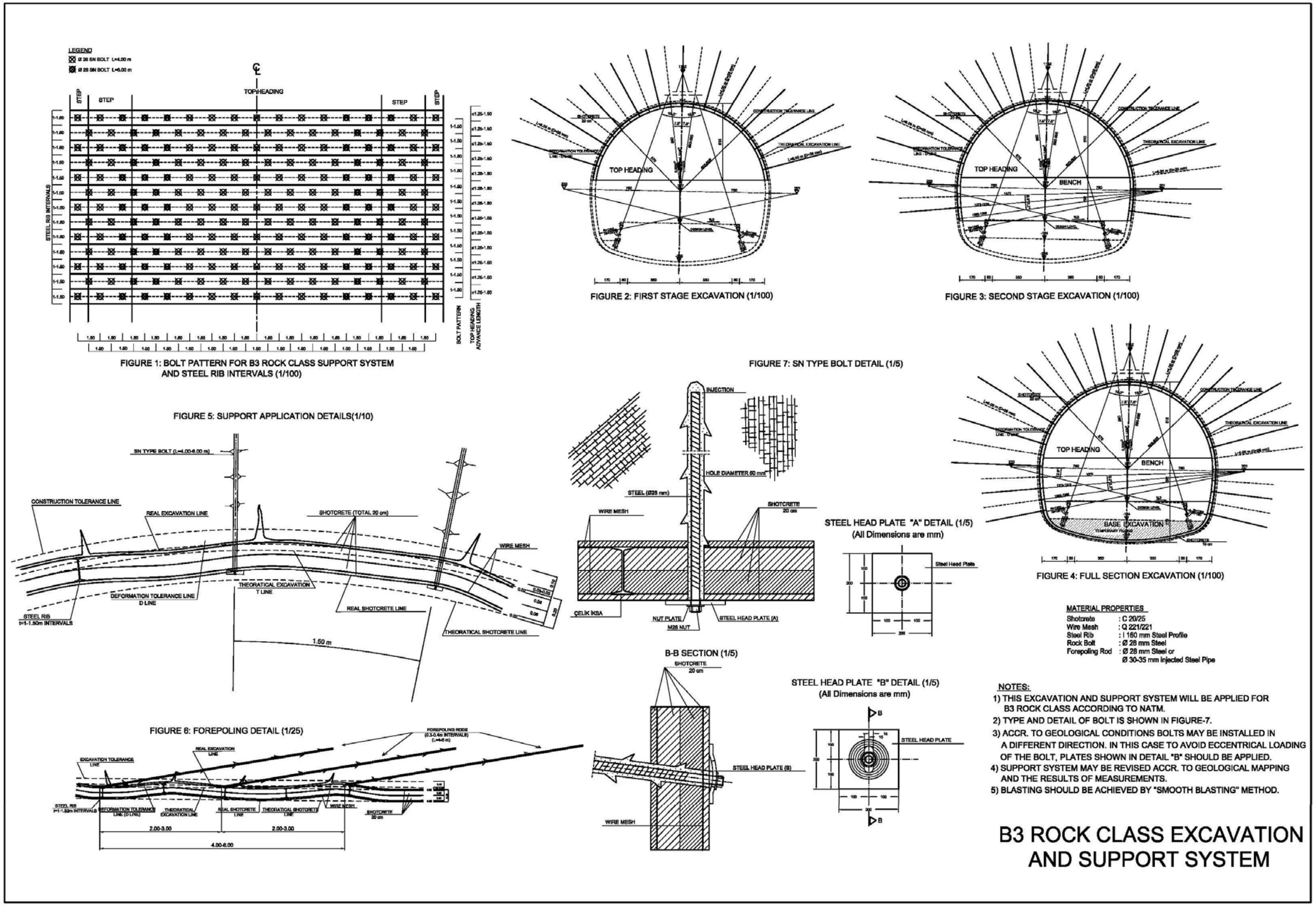


Figure C.3 B3 Rock class excavation and support system of Çubukbeli tunnel

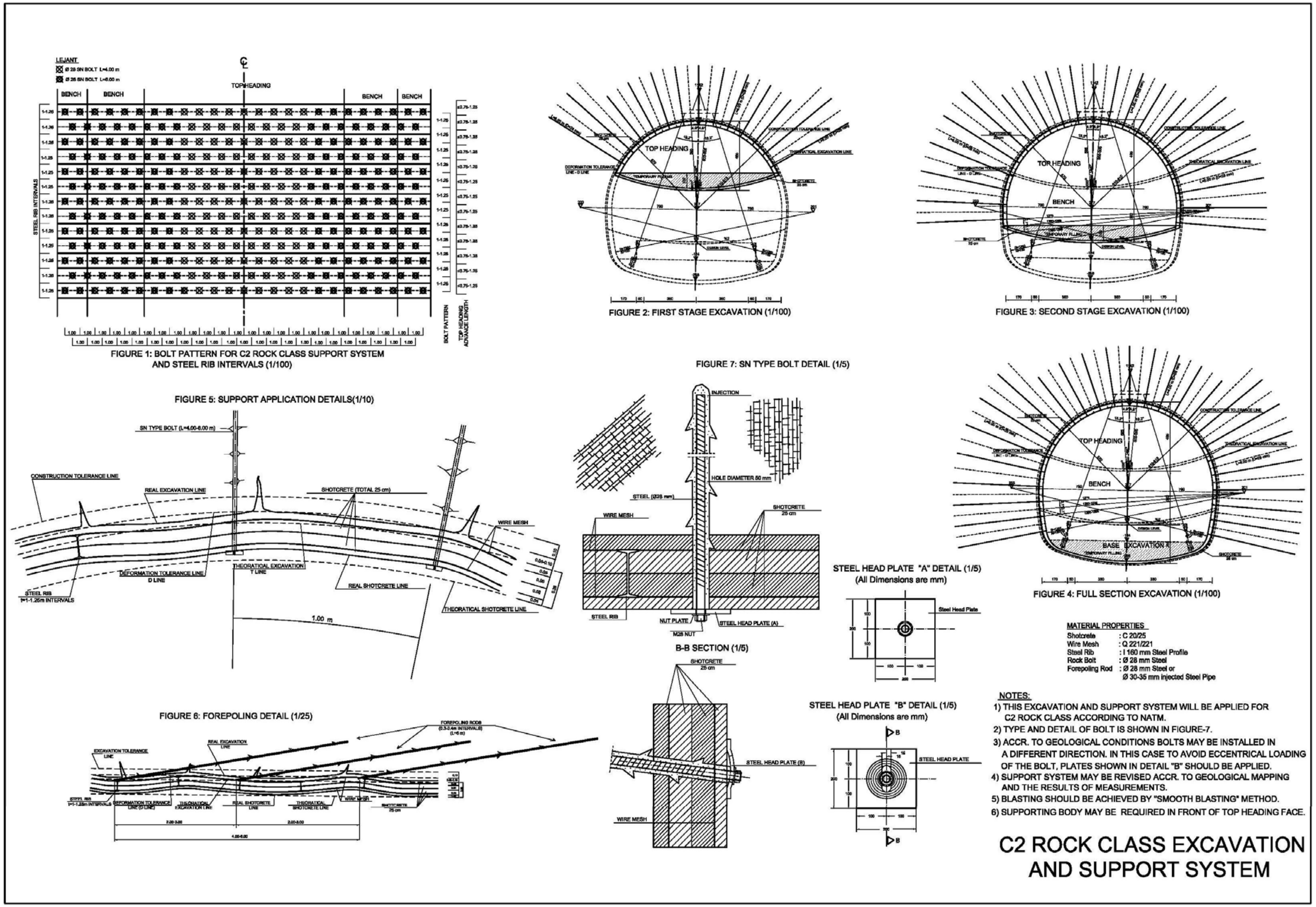


Figure 4.4 C2 Rock class excavation and support system of Çubukbeli tunnel