ROCK CLASSIFICATION AND EMPIRICAL DETERMINATION OF THE IN-SITU MODULUS OF DEFORMATION OF BASALT

by

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To my dear wife, Aysun

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EMPIRICAL DETERMINATION OF THE IN-SITU
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ROCK CLASSIFICATION AND EMPIRICAL DETERMINATION OF THE IN-SITU MODULUS OF DEFORMATION OF BASALT

The objective of this thesis is to demonstrate that the rock classification systems are practical and useful tools for the determination of the in-situ mechanical characteristics of the high-strength rock formations, and to present the modern rock classification systems in a compact form.

Within the scope of this thesis, basalt samples from the Toprakkale-İskenderun Motorway Project are classified using the three most frequently implemented rock classification systems, the RQD Index, the RMR-Geomechanics, and the Q Systems. Upon the classification, the in-situ modulus of deformation is determined according to the empirical methods based on these rock classification systems. The obtained results are compared and correlated with the laboratory test and geophysical survey results.

BAZALT'IN KAYA SINIFLANDIRMASI VE ARAZİ (IN-SITU) DEFORMASYON MODÜLÜNÜN AMPİRİK OLARAK BELİRLENMESİ

Bu tezin amacı, kaya sınıflandırma sistemlerinin yüksek mukavametli kaya formasyonlarının arazideki mekanik karakteristiklerinin belirlenmesinde kullanılabilecek pratik ve yararlı araçlar olduklarını göstermek ve modern kaya sınıflandırma sistemlerini toplu olarak sunmaktır.

Bu tez kapsamında, Toprakkale - İskenderun Otoyolu'ndan alınan bazalt örnekleri günümüzde en yaygın olarak kullanılan üç kaya sınıflandırma sistemleri olan RQD İndeksi, RMR-Geomekanik ve Q Sistemlerine göre sınıflandırılmıştır. Sınıflandırmanın ardından, bazalt formasyonunun arazi deformasyon modülü bu kaya sınıflandırma sistemlerine dayalı olarak geliştirilmiş bulunan ampirik yöntemler kullanılarak belirlenmiştir. Elde edilen sonuçlar, laboratuar deney sonuçları ve jeofizik etüt sonuçlarıyla karşılaştırılmış, aralarındaki korelasyon gösterilmiştir.

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LIST OF SYMBOLS

c	cohesion (kN/m^2)
E	modulus of elasticity (GPa)
E_d	dynamic modulus of deformation (GPa)
E_m	in-situ modulus of deformation (GPa)
E_{t50}	tangent mod. of def. at 50 % of compressive strength (GPa)
G	shear modulus (GPa)
I_s	point load index (kN/m^2)
J_a	joint alteration number
J_n	joint set number
J_r	joint roughness number
J_w	joint water reduction number
M	modulus ratio
Q	rock mass quality rating
RMR	rock mass rating
SRF	stress reduction factor
V_p	primary seismic wave velocity (m/s)
V_s	shear seismic wave velocity (m/s)
τ	density of rock mass (kN/m^3)
σ_{cs}	unconfined compressive strength (kN/m^2)
σ_1	major principal stress (kN/m^2)
σ_3	minor principal stress (kN/m^2)
$\emptyset$	internal friction angle (°)
ν	poisson's ratio

I. INTRODUCTION

In almost all of the design problems in the rock mechanics, the determination of the in-situ strength or the in-situ modulus of deformation of the rock formations has a critical significance.

It is well known that the strength and the deformation modulus values determined from laboratory testing of the intact or slightly jointed or fractured rock cores are not directly applicable to the in-situ rock masses since they do not adequately represent the in-situ conditions. The in-situ properties may vary up to a large extent due to the presence of the joints and other types of geological discontinuities. The structure to be built on a certain rock formation inevitably faces with several discontinuities of various types and locations, although samples taken from this rock formation may contain no or few discontinuities. It must be noted that the laboratory conditions cannot duplicate the in-situ conditions, and the engineering design should be based upon the in-situ characteristics.

For this reason, especially over the last two decades, researchers in the field of rock mechanics have been focusing on the large-scale in-situ testing, and development of acceptable prediction methods of the in-situ rock characteristics.

With the arch dams being the most critical cases, fields of tunnelling, settlement analyses of heavy piers, high-rise buildings, nuclear reactors founded on rock, arches and walls of large-scale underground openings require that the in-situ characteristics of the underlying or surrounding rock formations to be determined with the most possible degree of accuracy possible.

The requirement for the accurate determination of the in-situ characteristics is becoming more and more significant, as the dimensions of the engineering structures are continuously increasing along with the extent of the critical nature of the design problems, and as the ideal construction sites for these structures are continuously diminishing.

1.1 Determination of the In-Situ Modulus of Deformation

Chahges in the rock types, jointing and shear zone characteristics, foliation and geological structure of the formation all contribute to the variations in the deformation modulus values. Especially, in the cases of high arch dams, the variations of the modulus value at the top and at the foundation contact of the dam throughout a stressed rock mass play important roles in the design of the engineering structure.

For the determination of the in-situ modulus of deformation, five alternative methods are being applied in the case of high-strength rock formations:

(a) Direct in-situ measurements making use of the jacking tests;

(b) Laboratory testing on the intact rock samples and the derivation of the in-situ value from test results using reduction factors depending on the case;

(c) Empirical prediction from the seismic wave velocities;

(d) Empirical prediction directly from the RQD Index measurements;

(e) Empirical prediction making use of the RQD-oriented quantitative rock classification systems.

Within the scope of this thesis, the alternatives c, d and e are considered in detail. The aim of this thesis is to examine the methods for determining the deformation modulus without the involvement of complicated and rather expensive methods such as special in-situ testing apparatus or geophysical equipment.

Borings and the RQD Index measurements are normal applications which are deemed to be necessary in almost all cases in the rock mechanics design. Therefore, if some means of predicting the in-situ modulus of deformation values with an acceptable error, and without complicated calculations or too many limiting conditions are developed, a quite practical solution for the determination of this important rock parameter would be obtained, at least for the preliminary design stage.

1.2 Rock Classification Systems in the Rock Mechanics

Rock classification systems are increasingly being used in determination of the mechanical characteristics of the rock formations (stress-strain characteristics, modulus of deformation, friction angle, cohesion, fracture degree, etc.), in establishing the support measures (rock bolting and shotcreting patterns, auxiliary or temporary support establishment) for the slope stability problems and tunnelling, and determination of the stand-up times without support and advance lengths in the tunneling applications.

Within the scope of this thesis, the two most quantitative rock classification systems, the RMR-Geomechanics System developed by Bieniawski (Section V), and the Q System developed by Barton (Section VI) will be examined, compared and correlated along with the RQD Index developed by Deere. For illustrating the rock classification systems, basalt samples from the Toprakkale-İskenderun Motorway Project will be used.

1.3 Problems Encountered in the Determination of the In-Situ Modulus of Deformation

Due to the nature of the in-situ and laboratory testing procedures, tests are generally made on the intact or near intact (slightly jointed or fractured), samples obtained from rather strong rock formations with considerable compressive strength values. Therefore, it is almost impossible to obtain data for moderately or heavily fractured rock formations or case histories of tests made on low strength rocks. This is mainly due to the fact that the in-situ or laboratory tests are generally made for the construction purposes, and not for research. It should be kept in mind that the rock formations present at a possible arch dam site must be above certain standards in order to be considered. Thus, most of the test results obtainable are for the strong and less jointed or fractured rock formations. As a conclusion, the empirical criteria developed for the in-situ strength characteristics or the modulus of deformation are generally developed for rock formations above a certain level of "quality". The lack of test results, although should be regarded as normal, result in empirical equations that are developed for specific rock types, and these equations do not yield acceptable results for all rock formations.

Therefore, the main problems encountered in the determination of the in-situ modulus of deformation may be listed as follows:

(a) Lack of reliable test results, and the absence of test results in certain ranges;

(b) Impossibility of obtaining samples due to the depth or topographic condition;

(c) Cost effects of certain in-situ tests;

(d) Specific natures of the theories and empirical criteria developed for the determination of the in-situ modulus of deformation, and their limited applicability.

Also, the proposed empirical method should satisfy the below listed conditions up to the greatest possibility;

(a) Being practical and less time consuming in application;

(b) Not requiring expensive and specific testing;

(c) Being able to predict the in-situ values without too much error.

In summary, an easy and practical tool with an acceptable reliability is sought. This thesis is intended to demonstrate that the rock classification systems are such tools for the determination of the in-situ modulus of deformation.

II. THE MODULUS OF DEFORMATION AND THE IN-SITU MODULUS OF DEFORMATION CONCEPT

2.1 Definitions of the Modulus of Deformation

For soil and rock formations, the means of expressing the deformability is referred as the stress-strain modulus or as the modulus of deformation. It is known that the soil and rock formations do not behave elastically. Some researchers working on soil mechanics use the terms "elasto-plastic" for describing the nature of soil distortion (1). This nature of the soils clearly exhibit itself in the stress-strain curves obtained from the triaxial test data. The same characteristics apply to the rock formations as well.

Two methods are commonly used in the soil and rock mechanics for the evaluation of the modulus of deformation:

(a) <u>The Tangent Modulus</u> based on the slope of the curve of a line which is just tangent to the stress-strain curve at some point. In the soil mechanics practice, the **Initial Tangent Modulus**, which is a tangent drawn from the origin, is more commonly used, since the slope at the origin is not highly subjected to the environmental factors such as the type of the testing equipment used (Fig. 2.1).

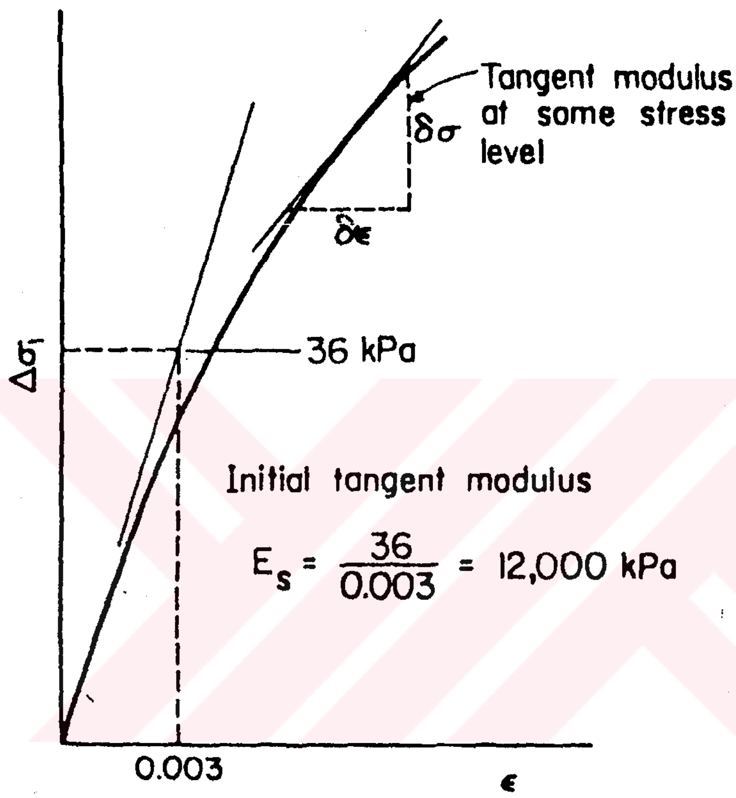


Fig. 2.1 - The Tangent Modulus of Deformation

- after Bowles (1).

In the rock mechanics practice, the recommended method for the determination of the intact modulus of deformation is the E_{t50}, which is a tangent drawn at the 50 percent of the compressive strength (2).

(b) <u>The Secant Modulus</u> which is based on the slope of a secant line, cutting the stress-strain curve at two points (Fig. 2.2).

When the data from a plate loading test is examined, it is seen that a variety of loading-unloading curves present a variety of moduli of deformation alternatives (Fig. 2.3). The modulus of deformation may be calculated from the slope of the curve from the origin to the point of total deformation and maximum load (E_{12}). Or, it may be calculated from the upper half of the unloading curve (E_{23}) or the slope of the entire unloading curve (E_{24}). These three are the most commonly used alternatives, although others may be considered also.

Among these, E_{12} is generally the lowest, and therefore, the most conservative value to use. It includes the deformation during loading and under constant load. Depending upon the general shape of the load-deformation curve, this modulus increases or decreases with increasing stress levels, but it is generally lower than E_{23} or E_{24}. This modulus is the one which is generally considered as the modulus of deformation.

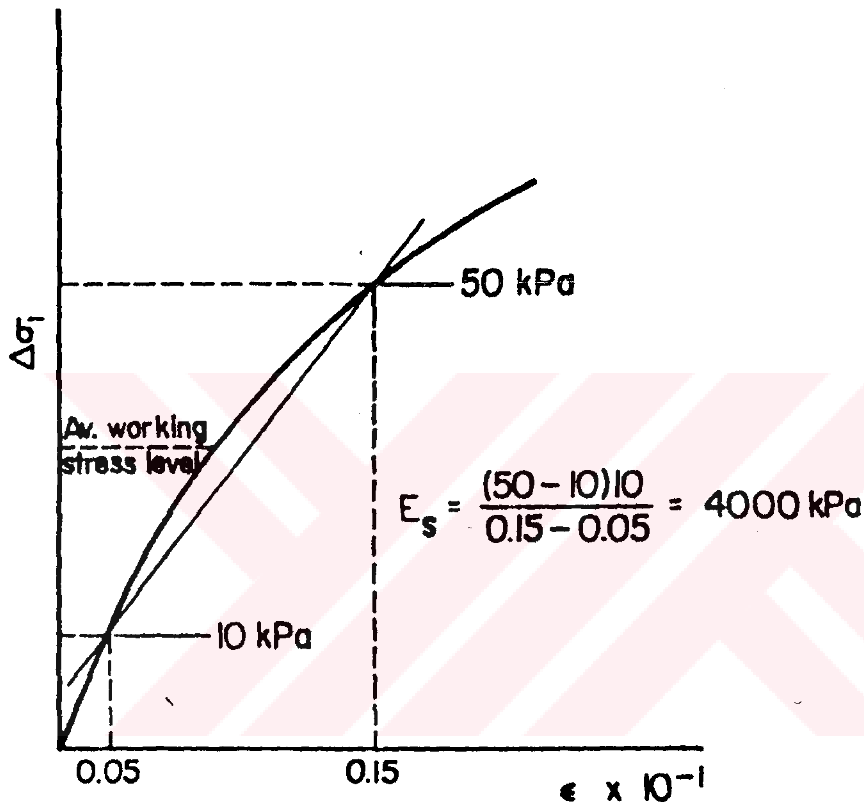


Fig. 2.2 - The Secant Modulus of Deformation - after Bowles (1).

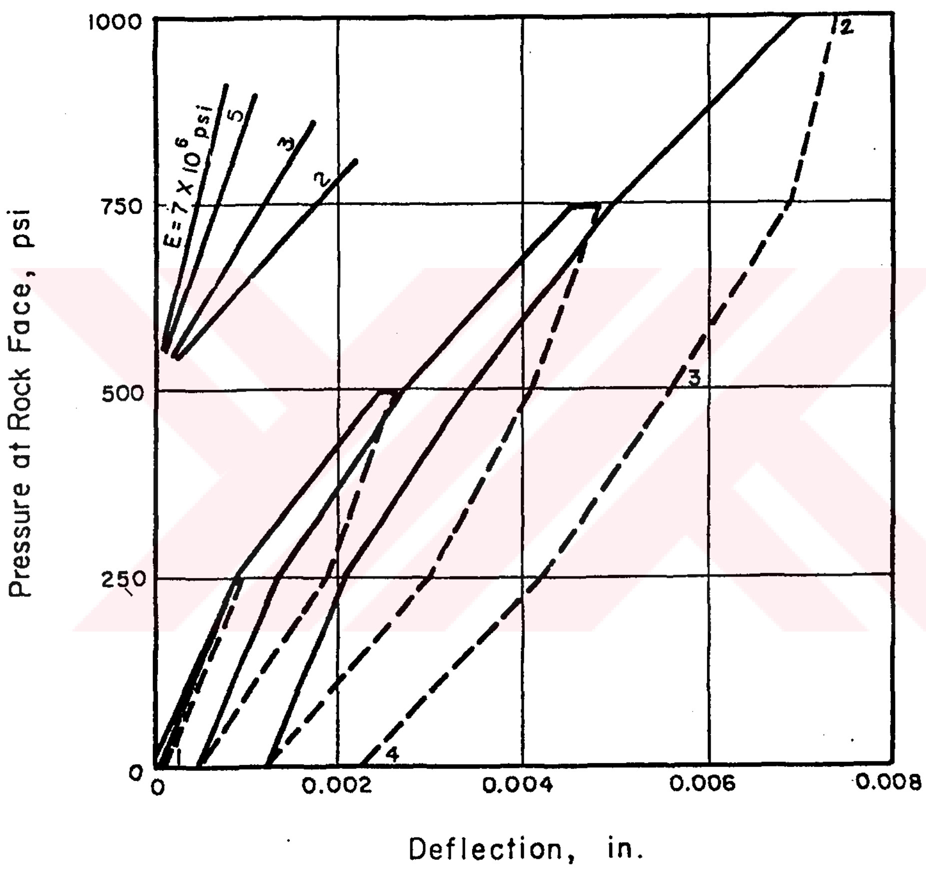


Fig. 2.3 - Typical Load-Deformation Curve From a Plate Loading Test - after Coon & Merritt (3).

The moduli values obtained from the unloading curves may be taken at any cycle of the test, but since the loading-unloading loops are very similar to each other, they are commonly measured on one of the cycles to maximum load. E_{23} is usually the highest, and furthermore, it has been observed that it often corresponds to the deformation modulus value calculated from the seismic wave velocities (3). E_{24} is lower, and its value usually ranges between the other two. In most of the cases, E_{23} and E_{24} values are not considered in the determination of the in-situ modulus of deformation.

2.2 Static Modulus vs. Dynamic Modulus

The modulus of deformation concept stated up to now is considered to be the static modulus of deformation.

Another type of the modulus of deformation is the dynamic modulus of deformation. The dynamic modulus of deformation is determined through the geophysical surveys such as the seismic refraction or sonic logging. The procedures of seismic refraction is examined in Section 7.1.

The dynamic methods for the determination of the modulus of deformation usually yield higher values than the static methods, due to the difference between the strain levels.

2.3 In-Situ Modulus of Deformation Concept

The in-situ values of the modulus of deformation are always lower than those obtained through the laboratory methods. These differences between the in-situ and the laboratory values are obviously caused by the factors such as the scale effect, samples not fully representing the actual formations, loosening on the wall faces of the exploratory adits in the course of time, etc.

Without doubt, the rock mechanics practice is much more concerned with the in-situ modulus of deformation rather than the experimental values, or the degree of difference in between. In many cases, the seismic refraction yields doubtful results, especially if the rock formations are nonhomogenous or anisotropic. In such cases, the orientations of the bedding planes and similar geomorphologic characteristics may result in quite different E_M values in different seismic refraction directions. In the case of field jacking tests, the loosening of the wall at the testing area results in unrealistic values. Every method used for the determination of the in-situ modulus of deformation has certain limitations which causes divergences from the actual, i.e. the in-situ values. Thus, the in-situ modulus of deformation concept is indeed a very complex one, and probably the best solution to the problem is to apply more than one method. However, the in-situ testing is quite expensive, and there will be always budget restrictions for testing. For these reasons, development of empirical methods for the approximate determination of the in-situ modulus of deformation, at least for the preliminary works such as route or site location, will be helpful by reducing the amount and the cost of the in-situ testing which will be required in the later design phases.

III. ROLE OF THE ROCK CLASSIFICATION SYSTEMS IN THE ROCK MECHANICS DESIGN

3.1 Introduction

A number of rock classification systems have evolved over the last 50 years, and the trend is such that every new system considered the previous systems and introduced more applicable and more developed proposals. Thus the rock classifications exhibit a striking increasing level of applicability and versatility. The systems may be grouped into engineering geology classifications, geomechanical classifications and geotechnical design classifications.

Within this section, all rock classification systems are examined in chronological order. The last two systems, the RMR-Geomechanics System and the Q System will be examined separately due to their significance in rock mechanics and their relevance in the scope of this work. Along with these, the RQD Index is also examined, since it plays a major role in these systems.

3.2 Terzaghi's Rock Load Classification System (1946)

The first notable rock classification system was developed by K. Terzaghi[(4)]. The aim of the system was to establish a weighting system for the determination of the required support for the steel-supported tunnels. The

Table 3.1 - Terzaghi's Rock Load Classification System - after Terzaghi (4).

ROCK CONDITION	ROCK LOAD Hp (feet)
1. Hard and intact	0
2. Hard stratified or schistose	0 to 0.5 B
3. Massive, moderately jointed	0 to 0.25 B
4. Moderately blocky and seamy	0.25 B to 0.35 (B + Ht)
5. Very blocky and seamy	(0.35 to 1.10) (B + Ht)
6. Completely crushed but chemically intact	1.10 (B + Ht)
7. Squezzing rock, moderate depth	(1.10 to 2.10) (B + Ht)
8. Squezzing rock, great depth	(2.10 to 4.50) (B + Ht)
9. Swelling rock	Up to 250 ft.irrespective of (B + Ht)

most popular tunnelling method in the U.S.A. then, and even today, was the usage of steel supports. The method is known as the "American Steel Support Method-ASSM". Terzaghi established means for calculating the rock loads that would be considered in the design for certain rock qualities. It should be noted that the system is not a totally quantitative one, and gives partial information regarding the mechanical characteristics of the rock formations, i.e. their in-situ strengths. The assumption of rock loads is almost completely abandoned due to its unrealistic mechanism. The new approaches consider the rock formation around an opening as not a load to be supported, but means of self-supporting. The system is not suitable for modern techniques in which shotcrete and rock bolting are used for the slope stability and tunnelling applications.

3.3 Stini's Rock Classification System (1950)

Stini's Rock Classification System is another example to a partially mechanistic rock classification system. Stini is considered to be the pioneer of "the Austrian School" in the rock mechanics [5]. In his classification system, Stini has emphasized the significance of the structural imperfections within the rock formations. However, when compared to the rock classification systems, it is rather crude and ambigious for some cases.

Stini's Rock Classification System is outlined in Table 3.2.

3.4 Lauffer's Stand-Up Time Classification System (1958)

Lauffer(6) introduced the concept of an unsupported span and its equivalent stand-up time, which are the functions of rock mass quality. Although excessively conservative when compared to the present day methods, the introduction of the "stand - up time" concept was a significant novelty in the rock mechanics and especially in tunnelling, and Lauffer's Stand-Up Time Classification System is considered to be the pioneer of Bieniawski's RMR-Geomechanics System. Lauffer's support estimates for tunnelling are not considered here, since the aim of this work is to establish the in-situ mechanical characteristics of basalt, and not to develop support systems for tunnels.

Lauffer's Stand-Up Time Classification System is outlined in Table 3.3 and Fig. 3.1.

3.5 Rabcewicz's Rock Classification System (1958)

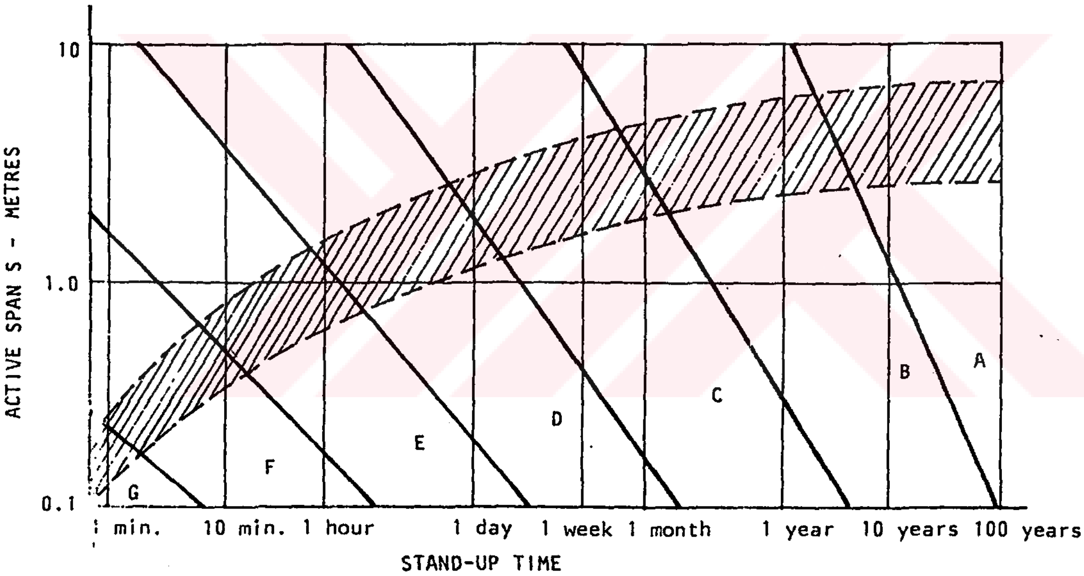
Professor Rabcewicz of Austria, is one of the pioneers of the New Austrian Tunnelling Method (NÖT-NATM),

Table 3.2 - Stini's Rock Classification System - after Ertunç (5).

CLASSIFICATION ACCORDING TO PRESSURE EXERTED	ROCK FORMATION TYPE	FRACTURE LENGTH (m)
9 Heavily pressure exerting rock formations	Schists, friable marls, crushed rock formations, heavily fractured rock zones	40 - 60
8 Moderately pressure exerting rock formations	Friable, thin layered schists, phyllites, soft marls, graphitic schists, wetted claystones	24 - 40
7 Slightly pressure exerting rock formations	Claystones, slightly deformed schists, quartz-schists with myca, hard rocks with clay fillings, medium level mylonite zones, marls with clay, wetted base morens	15 - 25
6 Highly friable rock formations	Thin layered sandstones with marl, phyllites with myca, some hard marls, calcerous clays, schists, shore morens	10 - 15
5 Friable rock formations	Marls with clay, some thin layered friable sandstones, tectonic dolomites, etc.	4 - 10
4 Moderately friable rock formations	Heavily fractured dolomites (within the fault zones)	2 - 4
3 Slightly friable rock formations	Heavily deformed and fractured quartz-phyllites, schists with chlorite, calcerous myca-schists	1 - 2
2 Sufficiently strong rock formations	Myca-schists, gneisses with thin cleavage	0.5 - 1
1 Strong and very strong rock formations		0 - 0.5

Table 3.3 - Lauffer's Stand-Up Time Classification System - after Lauffer (6).

ROCK QUALITY	CLASSIFICATION OF THE ROCK MASS	SELF SUPPORTING PERIOD
A	Strong	∞
B	Friable in time	6 months
C	Slightly friable heavily after-breaking	1 week
D	Friable	5 hours
E	Heavily friable	20 minutes
F	Pressure exerting	2 minutes
G	Heavily Pressure exerting	10 seconds



<u>Fig.</u> 3.1 - Lauffer's Stand-Up Time Chart

- after Hoek and Brown (24).

Table 3.4 - Rabcewicz's Rock Classification System

- after Rabcewicz and Golser (7).

<table>
<tr><th colspan="3">TYPE OF FORMATION</th><th>SELF SUPPORTING PERIOD T (days)</th></tr>
<tr><td>1</td><td>Strong massive rock</td><td rowspan="4">Chemically unweathered</td><td>∞</td></tr>
<tr><td>2</td><td>Strong rock with bedding and cleavage</td><td>∞ - 24</td></tr>
<tr><td>3</td><td>Highly fractured rock with discontinuities</td><td>24 - 1</td></tr>
<tr><td>4</td><td>Totally disintegrated rock</td><td>0</td></tr>
<tr><td>5</td><td>Pseudo strong rock</td><td rowspan="5">Chemically weathered</td><td rowspan="5">Depending on the geological conditions and groundwater;
from several days to several hours</td></tr>
<tr><td>6</td><td>Slightly pressure exerting surface rock</td></tr>
<tr><td>7</td><td>Heavily pressure exerting deep rock</td></tr>
<tr><td>8</td><td>Heaving-swelling rock</td></tr>
<tr><td>9</td><td>Silt, clay</td></tr>
<tr><td>10</td><td>Sand, gravel, debris, cohesionless,disintegrated formations</td><td>Chemically strong</td><td>0</td></tr>
</table>

and he has played an exceptional role in the development of the modern rock mechanics practice. Rabcewicz worked for years on the development of the application of shotcreting and rock bolting in the rock environment in a totally new manner through the consideration of the rock mass environment not as a load to be carried by the support, but as a load-carrying environment itself (7).

His rock classification system aimed to establish the self-supporting periods of the rock formations and the required support methods and quantities (temporary and permanent), depending on the chemical weathering of the rock formation.

Table 3.4 shows Rabcewicz's Rock Classification System. The required support quantities are not shown in the figure since they are developed for the establishment of the support for tunnels. Neverthless, the classification still shows the development of the rock classification systems and their increasing refinement within the elapsing years. Rabcewicz's this initial system was developed later to establish the rock classification method for the NATM (See 3.9).

3.6 Deere et al.: Rock Quality Designation (RQD) Index (1967-1970)

The RQD Index will be examined in more detail within the following section.

3.7 Wickham et al. : Rock Structure Rating (RSR) System (1972)

The RSR concept was developed by Wickham et al. in 1972[8]. It is an U.S.A. based system originally developed for the steel supported tunnels and later modified for shotcreting and rock bolting applications. This concept introduced numerical ratings and weighings to relate the rock mass quality, excavation dimensions, and the support requirements. The RSR concept was a forerunner to the RMR-Geomechanics and Q Systems. When the RSR Ratings are investigated, its similarity with the Q System is clearly observed.

The procedure of the method is quite similar to the procedures of the RMR-Geomechanics and the Q Systems, and therefore not stated in detail. However, for comparison with these two systems, the figures that are used for the establishment of the RSR ratings and the required support are given in Figs. 3.2 - 3.3 and Tables 3.5 to 3.7.

3.8 Bieniawski : Rock Mass Rating (RMR)-Geomechanics Rock Classification System (1973-1979)

The Rock Mass Rating - Geomechanics System, developed by Bieniawski, will be examined separately in the forthcoming section due to its significance in the rock mechanics.

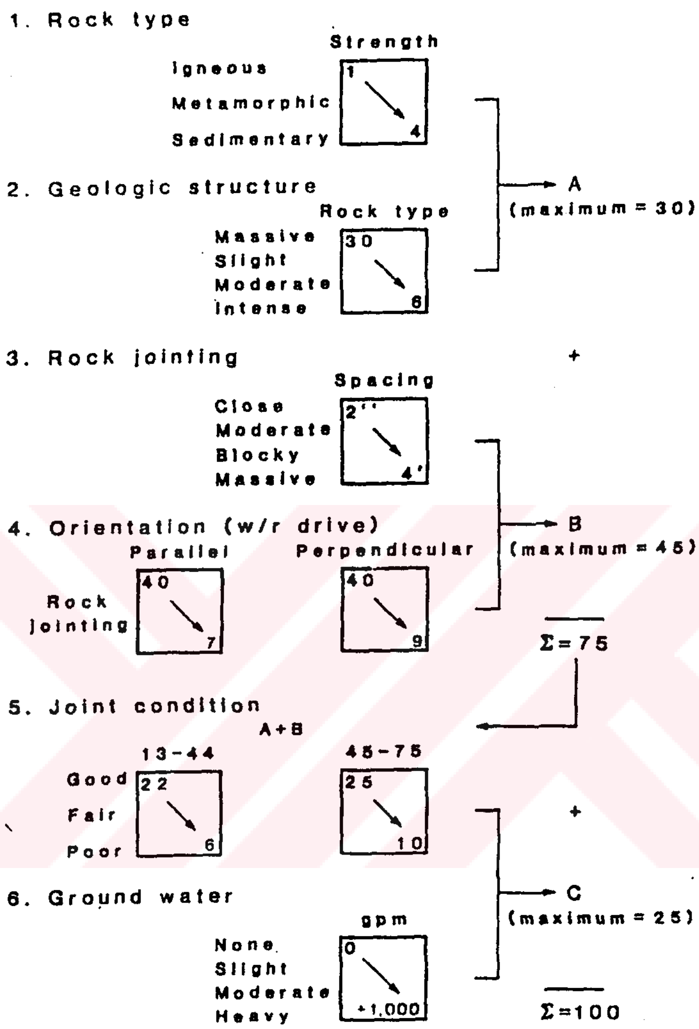


Fig. 3.2 - Establishment of the RSR Ratings

- after Wickham et al. (8).

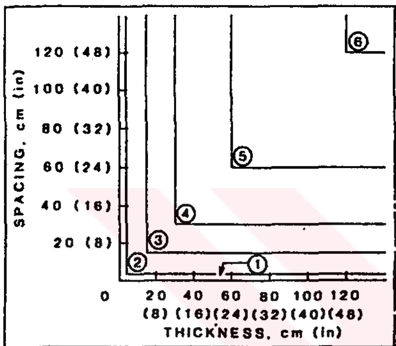
Tables 3.5-3.6-3.7 - Determination of the RSR Parameters

- After Wickham et al. (8).

Parameter A
Rock structure rating
Rock type, strength index and geologic structure
Maximum value 30

Basic rock type	Hard	Medium	Soft	Decomp
Igneous	1	2	3	4
Metamorphic	1	2	3	4
Sedimentary	2	3	4	4

	Geological structure			
	Massive	Slightly faulted or folded	Moderately faulted or folded	Intensely faulted or folded
Type 1	30	22	15	9
Type 2	27	20	13	8
Type 3	24	18	12	7
Type 4	19	15	10	6



Parameter B
Rock structure rating
Joint pattern and direction of drive
Maximum value 45

	Strike perpendicular to axis					Strike parallel to axis		
	Direction of drive					Direction of drive		
	Both	With dip		Against dip		Both		
	Dip of prominent joints					Dip of prominent joints		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
① Very closely jointed	9	11	13	10	12	9	9	7
② Closely jointed	13	16	19	15	17	14	14	11
③ Moderately jointed	23	24	28	19	22	23	23	19
④ Moderate to blocky	30	32	36	25	28	30	28	24
⑤ Blocky to massive	36	38	40	33	35	36	34	28
⑥ Massive	40	43	45	37	40	40	38	34

Flat: 0-20°; Dipping: 20-50°; Vertical: 50-90°

Parameter C
Rock structure rating
Ground water and joint condition
Maximum value 25

Anticipated water inflow m^3/min/300m (gpm/1,000 ft)	Sum of parameters A + B					
	13-44			45-75		
	Joint condition					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight <0.75 m^3/min (<200 gpm)	19	15	9	23	19	14
Moderate 0.75-3.8 m^3/min (200-1,000 gpm)	15	11	7	21	16	12
Heavy >3.8 m^3/min (>1,000 gpm)	10	8	6	18	14	10

Joint condition: Good = Tight or cemented; Fair = Slightly weathered or altered; Poor = Severely weathered, altered or open

Theoretical spacing of steel ribs, cm (ft)

Rib size \ Tunnel width or diameter, m (ft)	3.05 (10)	3.66 (12)	4.27 (14)	4.88 (16)	5.49 (18)	6.10 (20)	6.71 (22)	7.32 (24)	7.92 (26)	8.53 (28)	9.14 (30)
4I7.7	35.4 (1.16)										
4H13.0	61.3 (2.01)	46.0 (1.51)	35.4 (1.16)	28.0 (0.92)							
6H15.5	97.2 (3.19)	72.2 (2.37)	55.2 (1.81)	43.3 (1.42)	34.75 (1.14)						
6H20		92.0 (3.02)	70.7 (2.32)	55.5 (1.82)	44.5 (1.46)	36.6 (1.20)					
6H25			87.2 (2.86)	68.6 (2.25)	55.2 (1.81)	45.1 (1.48)	37.5 (1.23)	31.7 (1.04)			
8WF31				98.8 (3.24)	79.55 (2.61)	65.2 (2.14)	54.25 (1.78)	46.0 (1.51)	39.3 (1.29)	33.8 (1.11)	
8WF40					102.7 (3.37)	84.1 (2.76)	70.1 (2.30)	59.4 (1.95)	50.9 (1.67)	43.9 (1.44)	38.1 (1.25)
8WF48						101.8 (3.34)	84.7 (2.78)	71.6 (2.35)	61.3 (2.01)	53.0 (1.74)	46.0 (1.51)
10WF49								78.9 (2.59)	67.7 (2.22)	58.2 (1.91)	50.9 (1.67)
12WF53										66.75 (2.19)	58.2 (1.91)
12WF65											71.6 (2.35)

Fig. 3.3 - Theoretical Spacing of the Steel Ribs - after Wickham et al. (8).

3.9 Barton et al. : Q System of Rock Classification (1974)

Barton's Q System of Rock Classification will also be examined separately as in the case of the RMR-Geomechanics System.

3.10 Other Systems

Apart from the systems examined within this section, other rock classification systems have been developed by the rock mechanics researchers. These are not considered due to their specific natures and application areas. Some of the notable rock classification systems not examined within the scope of this work may be listed as follows;

- **United Rock Classification System (URCS)**, developed by Williamson and Kuhn in 1980 [9]. The system aims the initial assessment of the geotechnical rock conditions in the field through four fundamental properties including weathering, strength, discontinuity and density of rock materials or rock masses.

The system is not a quantitative one, and therefore not considered in more detail.

- Müller's Rock Classification System (1978), making use of the fracture frequencies. The system is developed for tunnelling, and not directly applicable to the determination of the in-situ modulus of deformation.

- Rabcewicz - Pacher Rock Classification System (1978) . This system is used in the New Austrian Tunnelling Method (NÖT-NATM). It is rather a developed form of Rabcewicz's Rock Classification system (Section 3.4), and it is beyond the scope of this work.

- French Classification, developed by Louis in 1978 for the estimation of the required support for tunnels.

3.11 General Notes on the Rock Classification Systems

It is observed that a number of rock classification systems have been developed over the last four decades. Almost all of the rock classification systems examined are at least initially, developed for tunnelling. This should be regarded as a normal trend, since the field of tunnelling is probably the most complex rock mechanics application in which healthy in-situ experimental values are quite expensive and almost impossible to obtain.

Some of the systems aim to provide certain guidelines for the classification of the rock masses, whereas some are extended to provide more quantitative support estimates.

Still, some of the rock classification systems may be extended also to appplication in other areas of the rock mechanics practice, including slope stability, foundation engineering, development of the empirical strength criteria for the jointed rock masses and the determination of the in-situ modulus of deformation. It is evident that the systems to be considered for the determination of the deformation modulus must be **quantitative** ones, and this requirement leaves only the RMR-Geomechanics and the Q systems, along with the RQD Index.

It should also be noted that a rock classification system, composed of weighing parameters for the tunnelling applications, **cannot be directly used** for the determination of other rock characteristics. So, the modifications are generally essential. The researchers focusing onto the application of the systems for the determination of these parameters have examined a number of case histories, conducted in-situ and laboratory experiments, and they have come up with certain proposals. the reliability and accuracy of these proposals are not yet totally verified. Nevertheless, current experience have shown that the rock classification systems can be used, at least at the preliminary design stages, as tools for the determination of the mechanical parameters of the rock masses.

Another striking point observed within the rock classification systems is that they are developed for the tunnelling or rock mechanics practice of their country of origin. The American tunnelling practice is built upon the steel sets and therefore, Terzaghi's Rock Loads Classification System, the Rock Structure Rating (RSR) System, The United Rock Classification System (URCS), are all developed for such a tunnelling practice. On the other hand, the Austrian school, focusing more onto the stand-up time (standzeit) concept and leading to the development of the New Austrian Tunnelling Method (NATM-NÖT), reveals itself in the Stini's Rock Classification System, Lauffer's Stand-Up Time Classification, and the works of Rabcewicz, Pacher and Müller. The South African Mining Practice reveals itself in the RMR-Geomechanics System, and the support proposals of the Q System exhibits the Norwegian trends of utilization of different and newer supporting techniques such as the usage of fiber reinforced concrete.

In summary, it may be stated that the rock classification systems are rapidly developing, becoming more and more precise, and extending into the other rock mechanics applications other than tunnelling. As stated before, they are not direct recipes, and rather general and preliminary guidelines for design. But they are getting more and more valuable tools as the role of research in the rock mechanics is increasing, and without doubt, they will be much more significant in the future.

IV. THE ROCK QUALITY DESIGNATION (RQD) INDEX CLASSIFICATION

4.1 Introduction

The Rock Quality Designation (RQD) Index, developed by D.U. Deere, has been used for over 20 years as an index of the rock mass quality. The index aims to measure the "good quality" rock amount within a borehole. The RQD is currently being used as a standard parameter in drill core logging and forms a basic element of several rock classification systems.

4.2 Development of the Concept

The RQD Index concept was originated from Deere's works between 1963 and 1967. In 1967, Deere & his team at the University of Illinois presented the first published form of the RQD concept along with some correlations with the geophysical velocity indices, fracture frequency and the in-situ modulus of deformation values [10]. The paper also included a brief discussion of some of the difficulties involved in the determination of the RQD Index.

The RQD concept found Worldwide recognition upon the publishing of the classical book "Rock Mechanics In Engineering Practice" (1968) by Deere & Hendron, in which the RQD concept and its possible applications were discussed in detail. By the 1970's, the RQD concept began to be used as a basic parameter in the rock classification systems, such as the RMR-Geomechanics System of Bieniawski, and the Q-System of Barton.

4.3 Determination of the RQD Index

The RQD Index was originally developed for NX-size cores (with 54.7 mm. diameter). According to Deere, a minimum of NX-size core obtained with double-tube core barrels should be used (10). However, the experience has shown that other core sizes and drilling techniques are also applicable. Core sizes of 36.5 mm. and 85 mm., respectively, are applicable for RQD measurements so long as proper drilling techniques are utilized such that they do not cause excess core breakage and/or poor recovery The NX and NQ size cores are considered to yield the optimal results. So, the recommended core diameters should be in between 47.5-54.7 mm. (1.87-2.16 mm.).

When the core is recovered, the core pieces with lengths (measured along their centerlines) exceeding 100 mm. (4 in.) are summed up to yield the RQD value. Although some researchers consider that 2 * diameter of core should be used instead of 100 mm. for cores with larger diameter, the standard practice is to use 100 mm.

It is very important tu utilize good drilling techniques and to measure the RQD value on the site. It is observed that some rocks, such as shales and claystones, often break up into small disks or chips with time, due to slaking, desiccation, stress relief cracking or swelling.

The core runs should not exceed 1.5 m. (5 ft.) in order to obtain realistic results. It is recommended that as zones of poor rock are encountered, the run lengths are to be shortened in order to prevent the blockage of the coring bit and to enhance the core recovery (11). The ISRM Commission On Standardization of Laboratory and Field Tests recommends that RQD logging using variable run lengths to separate individual beds, structural domains, weakness zones, etc., so as to indicate any inherent variability and provide a more accurate picture of the location and width of the zones with low RQD values.

4.4 The RQD Index Classification of Rock Masses

According to the RQD values, rock masses are divided into five classes ranging from very poor rock to excellent rock. The RQD Index of classification of rock masses is shown in Table 4.1.

This classification method is based only on the RQD Index values, i.e. the amount of more or less unfractured rock within the borehole examined. Thus, it is inevitably insufficient to classify a rock formation in a satisfactory manner. However, as it will be seen in the following sections, the RQD Index is a valuable element of modern rock classification systems.

Table 4.1 - The RQD Index Classification of Rock Masses -after Deere (11).

RQD Index	Description of Rock Quality
0 - 25 %	very poor rock
25 - 50 %	poor rock
50 - 75 %	fair rock
75 - 90 %	good rock
90 - 100 %	excellent rock

4.5 Applications of the RQD Index in the Rock Mechanics Practice

The RQD Index has been a particularly helpful tool in determining the fracture degree and the degree of weathering of the rock mass sample. Comparison of the different RQD values obtained within a large area enables a satisfactory preliminary location of structures such as bridge piers, foundation footings, etc.

4.5.1 Application to Tunnelling

It must be noted that the RQD Index was first developed for tunnelling applications, where the fracture degree and the extent of the weathering within the surrounding rock mass have considerable significance. Moreover, as stated previously, the RQD Index is an important parameter in more sophisticated quantitative rock classification systems.

The application of the RQD Index to the estimation of the tunnel support have been extensively studied between 1969 and 1972 by Peck[12], Deere[10], Cecil[13] and Merritt[14].

4.5.2 Application to Foundation Engineering

The RQD Index values are regarded as quite satisfactory guides to the fracture degree of the bedrock on which the foundation is to be constructed. It is common that the Total Core Recovery (TCR) values to be used as well as the RQD Index, since the amount of disentegrated or partially disintegrated rock mass below the foundation level is also significant for the design.

4.5.3 Determination of the In-Situ Modulus of Deformation

Another important application of the RQD Index is the determination of the in-situ modulus of deformation (E_M) of the rock masses. Researchers at the University of Illinois, at the late 1960's, investigated the relationships between the RQD Index, the seismic velocity ratios, and the in-situ modulus of deformation. They have found out that there is a strong correlation between these values; the lower the RQD value, the lower the in-situ modulus of deformation value[3].

Two evaluation methods were considered for the determination of the in-situ modulus of deformation from the RQD Index;

(a) direct determination from the RQD Index;

(b) determination using the Modulus Ratio.

Coon & Merritt worked on the data from plate loading tests for gneiss, limestone, granite and sandstone in order to develop a relationship between the RQD Index and the E_M . For their research to be more accurate, the RQD measurements to be used were weighted according to depth by using the Boussinesq stress distribution beneath a point load on the surface of a semi-elastic media. This was considered to be necessary since the near-surface fractures have more pronounced effects on the deflections measured than those at the deeper levels [3].

The two methods are given below, along with their relevance and applicability to the determination of the in-situ modulus of deformation.

4.5.3.1 Direct Determination From the RQD Index

Fig. 4.1 shows the deformation moduli values from the plate loading tests at various sites in the U.S.A., plotted against the weighted RQD Index values. The relationship obtained had a moderate correlation coefficient of 0.665 [3].

When Fig. 4.1 is examined, the following immediate conclusions are reached;

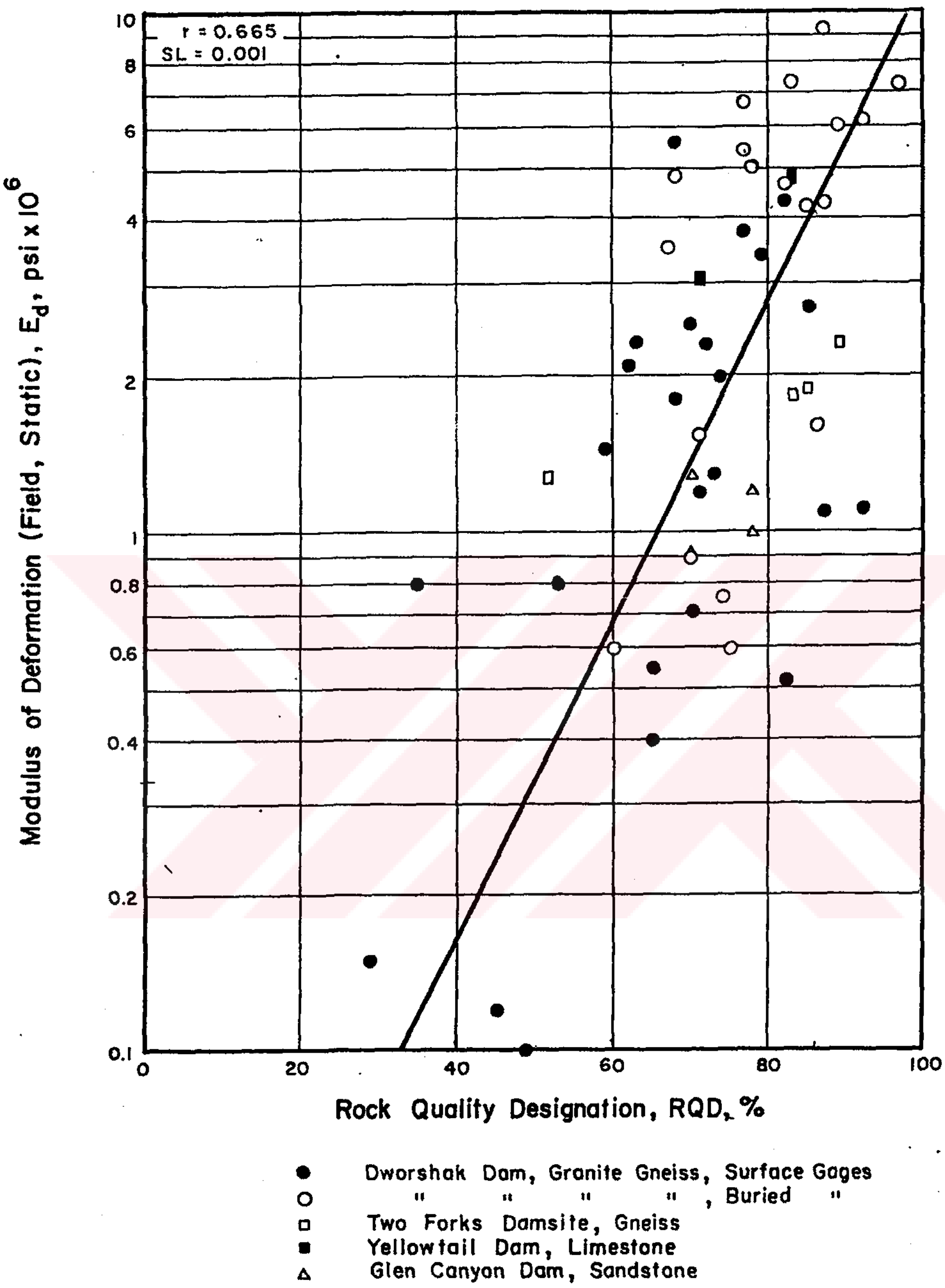


Fig. 4.1 - Relationship Between the RQD Index and the In-Situ Modulus of Deformation - after Coon & Merritt (3).

i. For RQD $\leq$ 33 per cent , $E_M = 0$.

ii. For RQD = 100 per cent (intact rock), $E_M = 98$ GPa.

Therefore, the relationship has two important limitations:

(A) For the RQD values lower than 33, the E_M value is zero. Moreover, for a rock mass with a RQD value of 60 per cent, $E_M = 7$ GPa. This value is much below the experimental values obtained from in-situ tests.

(B) The maximum E_M value foreseen for a rock mass is limited to 98 GPa. However, it is known that a number of rock types have much higher deformation moduli.

Thus, the above presented relationship is quite conservative, with limitations both for the E_M values and for the RQD values. This nature of the relationship is also noted by Coon & Merritt[(3)]; "This relationship could be used for deformation modulus estimates of similar rocks, but it cannot be used for rocks with low intact moduli". The researchers illustrate this point with a sandstone sample having a RQD Index of 80 per cent and an intact deformation modulus of 9.8 GPa. Fig. 4.1 would indicate a deformation modulus of about 26 GPa.

These limitations of that relationship caused the researchers to focus on other methods for determining the

E_M from the RQD Index values. Among these, the Modulus Ratio concept yielded more accurate results.

4.5.3.2 Determination Using the Modulus Ratio

As stated above, this method yields more acceptable results, since there are no limitations to the E_M values. In this method, the Modulus Ratio is defined as;

$$Mr = \frac{\text{in-situ modulus}}{\text{intact tangent modulus at 50 \% of unconfined strength}} = \frac{E_M}{E_{t50}} \qquad (1)$$

Data used in 4.4.3.1 has been used again by Coon & Merritt to establish a relationship between the RQD Index values and the Modulus Ratio. The relationship obtained was;

$$Mr = 0.0231 * RQD\,(\%) - 1.32 \qquad (2)$$

The graphical interpretation of this relationship is shown in Fig. 4.2. The correlation coefficient of this relationship is calculated as 0.544, which is rather low. However, when the complex nature of the rock mechanics is considered, even such a value should be regarded, at least as a normal one.

The proposed relationship indicates a E_M value of 0 for RQD < 60 per cent. Thus, it occurs that a curve,

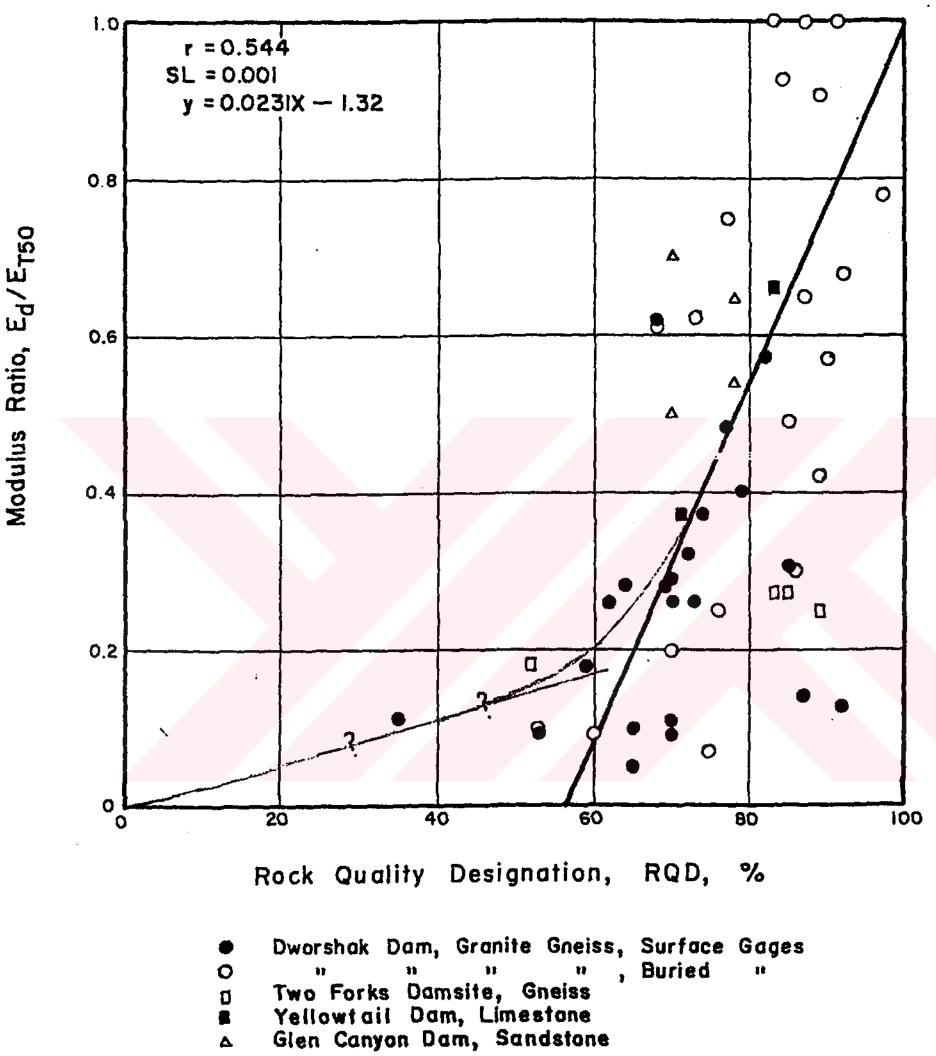


Fig. 4.2 - Relationship Between the RQD Index and the Modulus Ratio - after Coon & Merritt (3).

which is asymptotic to eq. 2, instead of a straight line, would be a much better proposal. This proposal is shown in dotted lines.

In the Appendix, the modulus of deformation values of the basalt samples from the Toprakkale-İskenderun Motorway Project are determined according to these proposals and compared with the values obtained from the calculations made by using the rock classification systems and the values from the laboratory testing.

V. THE ROCK MASS RATING (RMR) - GEOMECHANICS CLASSIFICATION

5.1 Development of the System

The Rock Mass Rating (RMR) System, also known as the Geomechanics Classification, was initiated in 1972-1973 by Z.T.Bieniawski. The system has evolved from several earlier rock classification systems, mainly the Lauffer's Rock Classification System. The system was originally based on 49 case records and was modified as more case histories became available and to conform with the international standards and procedures [15]. New versions of the system came out in 1974, 1975, 1976 and 1979. The latest number of case histories amount to 268, with applications including tunnels, underground chambers, mines, slopes and foundations, which point to the fact that the RMR system is regarded as an acceptable and versatile system.

Although the system is modified several times, the essential principles of the system still maintain their original forms. Thus, any modifications and the extensions should be regarded as the outgrowth of the same method and not as new systems [16].

5.2 Classification Procedure

The main objectives of the RMR-Geomechanics Classification System are listed below [16]:

(a) To identify the most significant parameters influencing the behavior of a rock mass.

(b) To divide a particular rock mass formation into a number of rock classes of varying quality.

(c) To provide a basis for understanding the characteristics of each rock class.

(d) To derive quantitative data for engineering design.

(e) To provide a common basis for communication between the engineer and the geologist.

In the system, the following parameters are used to classify a rock mass;

(a) Uniaxial compressive strength of the rock material.

(b) Rock Quality Designation (RQD).

(c) Spacing of the discontinuities.

(d) Condition of the discontinuities.

(e) Groundwater conditions.

(f) Orientation of the discontinuities.

To apply the RMR-Geomechanics Classification system from the field measurements and entered onto the input sheet (Figs. 5.1 - 5.2).

The classification scheme is presented in Table 5.2. In section A, five parameters are grouped into five ranges of values. Since the various parameters are not equally important for the overall classification of the rock mass, importance ratings are allocated to the different value ranges of the parameters, a higher rating indicating better rock mass conditions. In this respect, the average typical conditions are evaluated and the ratings are interpolated. Moreover, it should be noted that the importance ratings are developed for the rock masses having three sets of discontinuities, and when only two sets of discontinuities are present within the rock mass, a conservative assessment is obtained [16]. The conservative nature of the RMR-Geomechanics Classification is noted by some other researchers, including Barton [17].

Upon the establishment of the importance ratings, their summation is taken in order to obtain the unadjusted (basic) RMR value. The unadjusted RMR is adjusted through a sixth parameter in order to account for the joint orientations. With this parameter, the final RMR rating is

Name of project:
Site of survey:
Conducted by:
Date:

STRUCTURAL REGION	ROCK TYPE AND ORIGIN

DRILL CORE QUALITY R.Q.D.*

Excellent quality:	90 - 100%	....
Good quality:	75 - 90%	....
Fair quality:	50 - 75%	....
Poor quality:	25 - 50%	....
Very poor quality:	<25%	....

*R.Q.D. = Rock Quality Designation

WALL ROCK OF DISCONTINUITIES

Unweathered	....
Slightly weathered	....
Moderately weathered	....
Highly weathered	....
Completely weathered	....
Residual soil	....

GROUND WATER

INFLOW per 10 m of tunnel length litres/minute ....
or
WATER PRESSURE kPa ....
or
GENERAL CONDITIONS (completely dry, damp, wet, dripping or flowing under low/medium or high pressure:

....

STRENGTH OF INTACT ROCK MATERIAL

Designation	Uniaxial compressive strength, MPa	OR	Point-load strength index, MPa
Very high:	Over 250	....	>10 ....
High:	100 - 250	....	4-10 ....
Medium high:	50 - 100	....	2-4 ....
Moderate:	25 - 50	....	1-2 ....
Low:	5 - 25	....	< 1 ....
Very low:	1 - 5	....	

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3	Set 4
Very wide:	Over 2 m	....	....	....	....
Wide:	0,6 - 2 m	....	....	....	....
Moderate:	200 - 600 mm	....	....	....	....
Close:	60 - 200 mm	....	....	....	....
Very close:	<60 mm	....	....	....	....

NOTE: These values are obtained from a joint survey and not from borehole logs.

STRIKE AND DIP ORIENTATIONS

	Strike (average)		Dip (angle)	(direction)
Set 1	Strike:	(from .... to ....)	Dip:	....
Set 2	Strike:	(from .... to ....)	Dip:	....
Set 3	Strike:	(from .... to ....)	Dip:	....
Set 4	Strike:	(from .... to ....)	Dip:	....

NOTE: Refer all directions to magnetic north.

Fig. 5.1 - Input Data Form for the RMR-Geomechanics Classification System (Part 1).
- after Bieniawski (16).

CONDITION OF DISCONTINUITIES

PERSISTENCE (CONTINUITY)		Set 1	Set 2	Set 3	Set 4
Very low:	<1 m	....	....	....	....
Low:	1 - 3 m	....	....	....	....
Medium:	3 - 10 m	....	....	....	....
High:	10 - 20 m	....	....	....	....
Very high:	> 20 m	....	....	....	....
SEPARATION (APERTURE)					
Very tight joints:	<0,1 mm	....	....	....	....
Tight joints:	0,1 - 0,5 mm	....	....	....	....
Moderately open joints:	0,5 - 2,5 mm	....	....	....	....
Open joints:	2,5 - 10 mm	....	....	....	....
Very wide aperture	> 10 mm	....	....	....	....
ROUGHNESS (state also if surfaces are stepped, undulating or planar)					
Very rough surfaces:		....	....	....	....
Rough surfaces:		....	....	....	....
Slightly rough surfaces:		....	....	....	....
Smooth surfaces:		....	....	....	....
Slickensided surfaces:		....	....	....	....
FILLING (GOUGE)					
Type:		....	....	....	....
Thickness:		....	....	....	....
Uniaxial compressive strength, MPa		....	....	....	....
Seepage:		....	....	....	....

MAJOR FAULTS OR FOLDS

Describe major faults and folds specifying their locality, nature and orientations.

GENERAL REMARKS AND ADDITIONAL DATA

NOTE:
(1) For definitions and methods consult ISRM document: *'Quantitative description of discontinuities in rock masses.'*
(2) The data on this form constitute the minimum required for engineering design.
- The geologist should, however, supply any further information which he considers relevant.

<u>Fig.</u> 5.2 - Input Data Form for the RMR-Geomechanics Classification System (Part 2).
- after Bieniawski (16).

Table 5.1 - Classification Scheme of the RMR-Geomechanics System (Part I) - after Bieniawski (19).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

PARAMETER			RANGES OF VALUES						
1	Strength of intact roack material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5-25 MPa	1-5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0,6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces. Not continuous No seperation 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. Continous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length	None	<10 litres/min	10-25 litres/min	25 - 125 litres/min	> 125		
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	OR 0	OR 0,0-0,1	OR 0,1-0,2	OR 0,2-0,5	OR > 0,5		
		General conditions	OR Completely dry	OR Damp	OR Wet	OR Dripping	OR Flowing		
	Rating		15	10	7	4	0		

Table 5.2 - Classification Scheme of the RMR-Geomechanics System (Part II) - after Bieniawski (19).

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100← 81	80←61	60←41	40←21	< 20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2,5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°

obtained. When the importance of this parameter is considered, it is observed that this parameter is the one which really controls the final RMR rating and the rock mass classification. This means that the Geomechanics (RMR) classification requires a satisfactory level of skill of in-situ observations of the engineer or the geologist. Therefore, it is clearly seen that the classification system is field-oriented and not office-oriented. On the other hand, in many cases, a healthy in-situ joint investigation may be quite difficult or impossible. This is especially evident in regions which have undergone complex metamorphic events. So, this parameter is a source of uncertainty and according to many researchers, a weakness of the system. The joint orientation adjustment parameter is determined according to empirical rules derived from experiences gained from the case histories.

After the establishment of the final RMR rating, classification is made according to the five RMR classification ranges. The section D of the Table 5.2 shows these rock classes and the meaning of these classes, i.e. average stand-up time without support, cohesion and the friction angle of the rock mass. It must be noted that the numerical RMR rating also has a significance for the establishment of the load and support estimates [16].

A number of different opinions exist for the RMR rating and adjustment parameters. According to Kirsten [(18)], RMR is not sufficiently sensitive to the individual parameters, and as a result, the functional dependence of the RMR on any one of the parameters is not strongly represented in the system. To illustrate this point of view, Kirsten gives the following example: A certain rock mass, with a joint condition rating 26, has been rated to have a RMR rating of 79-"good rock" in which the joints are very rough, tight, discontinuous and comprise unweathered wall rock. If, instead, the joints are slickensided or contain gouge up to 5 mm. thick, the corresponding joint condition would be 9, and the RMR rating would reduce to 62. According to the classification, this result would still represent "good rock", but the behavior of the rock mass in a tunnel, foundation or a slope would be quite different from the first case. Also, Kirsten points out that the RQD and the joint spacing are treated separately, and notes that both of these two values are measures of the block size. Thus, Kirsten considers that a maximum of 40 points for the block size is excessive, and reduces the relative significance of some other important parameters, such as the joint strength.

Barton suggests that the RMR system does not adequately emphasize the significance of the joint roughness and the joint alteration. Barton considers that the framework of the RMR system is of inadequate sensitivity for application in a variety of rock mass conditions[17].

5.3 Applications of the System

The RMR-Geomechanics System has been applied in various types of engineering projects including tunnelling, slope stability, foundation engineering, mining, empirical determination of the peak strength characteristics, and the determination of the in-situ modulus of deformation. Up to date, the majority of the applications have been in the field of tunnelling.

Due to its practical nature, the system attracted the attention of many researchers including Laubscher, Serafim and Pereira, Hoek and Brown, Ünal and Nicholson.

5.3.1 Application to Tunnelling

The RMR System is specifically developed for tunnelling purposes. Bieniawski's main aim in determining such a rock classification was to establish a method for classifying rock formations around an excavation and to give empirical support characteristics for these particular rock masses. The section D of the classification table (Table 5.2) gives some empirical geotechnical parameters (cohesion, friction angle) along with the average stand-up times for certain tunnel spans. Moreover, from Fig. 5.3, the stand-up times may be estimated for all tunnel spans. The figure also locates cases where no support would be required or where the tunnel roof would collapse immediately following the excavation. Bieniawski suggests certain guidelines for the selection of the rock reinforcement for the tunnels[19] based on the RMR Ratings. These suggestions will not be completely presented here, since they are beyond the scope of this work and occupy a considerable space.

Thus, it is seen that the RMR-Geomechanics System is capable of assigning thoroughly quantitative support measures for all types of tunnels. The actual supports established at the 268 case histories up to date generally confirm these estimates[15]. In neither of the cases, the support was insufficient, but overdesign was observed in some of the case histories[17]. This is without doubt arising from the rather conservative nature of the system, as stated before in this section.

Yet, the experience confirms up to a great extent that the RMR - Geomechanics system is a helpful tool for the preliminary design of tunnels.

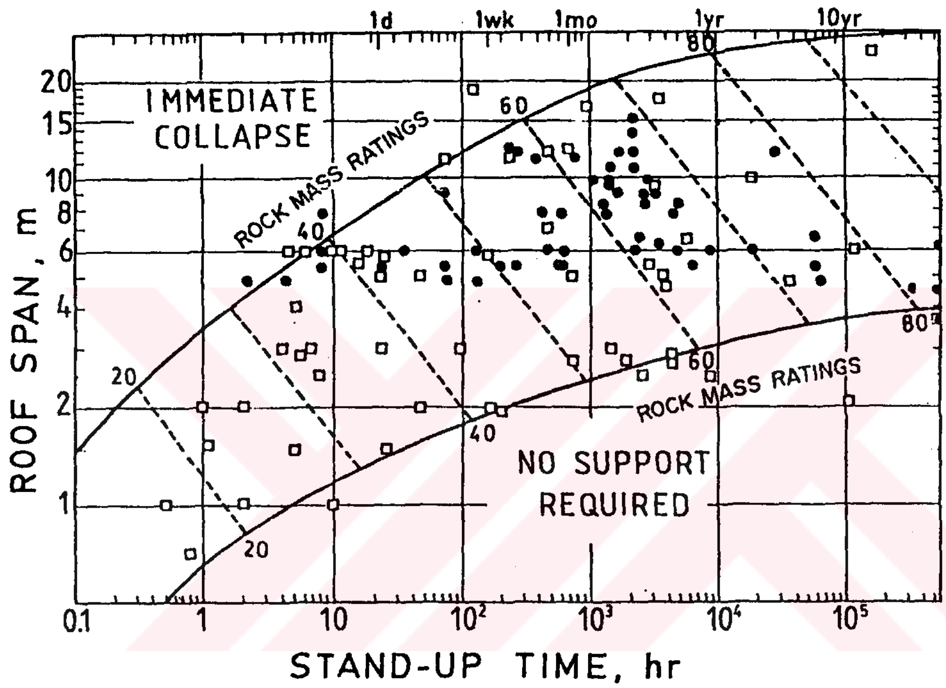


Fig. 5.3 - RMR Classification of the Rock Masses, Application To Mining and Tunnelling.

● Mining Rock Falls

□ Tunnelling Rock Falls

- after Bieniawski (19).

5.3.2 Application to Mining

The application of the system to the field of mining is first carried out by Laubscher [20] at asbestos mines in Africa. However, the main contributions belong to the noteworthy works of E. Ünal [21], who has applied the RMR-Geomechanics System to coal mines and developed an integrated approach to the roof stability assessments in mines (Fig.5.4) upon the analysis of the 49 case histories. In this approach, he incorporated the RMR ratings with roof span, support pressure, time and deformation.

Ünal proposed a quite satisfactory method for the determination of the support load;

$$P = \left[\frac{100 - RMR}{100} \right] \tau B \qquad (3)$$

Some researchers, including Laubscher [20] and Kendorski et al. [22] suggested that, especially for deep mining, the RMR ratings should be further adjusted for the stress at depth or change in stress, and reduced up to 50 percent.

5.3.3 Application to Slope Stability

As mentioned above, the Section D of the Table 5.2 gives approximate values for the cohesion and friction

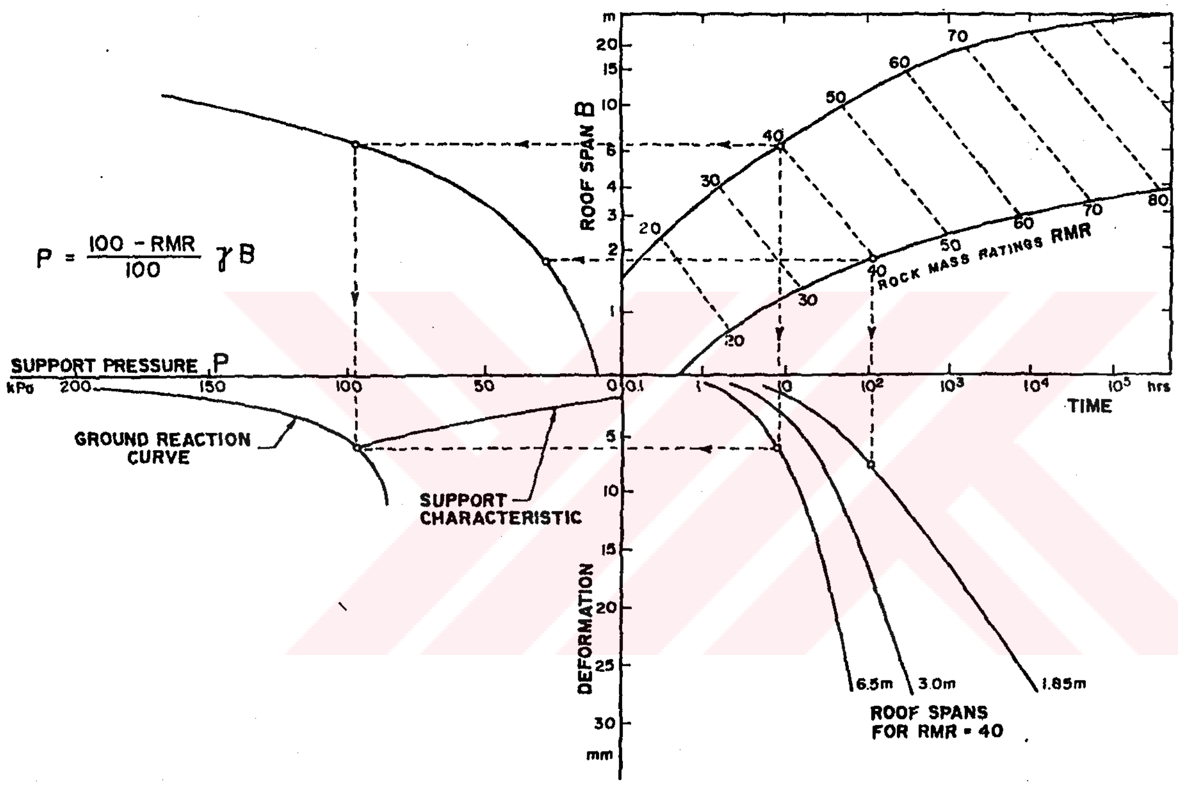


Fig. 5.4 - Roof Stability Assessment In Coal Mines Through the Usage of the RMR System - after Ünal (21).

angles of the rock masses classified according to the RMR-Geomechanics System. From these approximate values, or rather boundaries, suitable values may be chosen for the slope stability analysis.

Romana [23] has applied the system extensively for the determination of the slope stability, but to deal with this work in detail would be a diversion from the scope of the work.

5.3.4 Application To the Empirical Strength Criteria

" The design of an excavation in rock requires an assessment of the likely response of the rock mass to a set of induced stresses", state Hoek & Brown, in their famous article on the empirical strength criterion for the jointed rock masses [24] .

In order to predict this response, a knowledge of the complete stress-strain behavior and strength characteristics of the rock mass is required.It is evident that the determination of the stress-strain characteristics of a rock mass must be achieved through laboratory or in-situ testing. Neverthless, at the preliminary design stage, especially for deep rock formations where in-situ testing is not possible and the laboratory testing, which requires sampling through very deep boreholes, is quite expensive and difficult to perform, empirical determination may be both time and money saving.

Among the empirical strength criteria developed, the most striking ones are those of Bieniawski - Yudhbir et al.[25], and Hoek & Brown[26]. These two criteria are shortly described below;

(A) Bieniawski, 1974 - Yudhbir et al., 1983

$$\frac{\sigma_1}{\sigma_c} = B\left[\frac{\sigma_3}{\sigma_c}\right]^{\alpha} + A \qquad (4)$$

For the parameter B, Bieniawski suggested values for different rock formations. He also suggested that α = 0.75 be used for all cases. The parameter A, which is a dimensionless one, the value depends on the rock mass quality, with A = 1 for intact rock and A = 0 for completely disintegrated rock. However, Bieniawski gave no acceptable methods for the determination of the intermediate values of A. Thus, the criterion seems incomplete.

Yudhbir, Lemanza and Prinzi[27] modified Bieniawski's criterion and generalized it for the variable rock mass qualities. Their work may be considered as a finishing touch to the Bieniawski's criterion. For the parameter A, they have developed the following relationship;

$$A = \exp\ (\ 0.0765\ RMR - 7.65\ ) \qquad (5)$$

They suggested that $\alpha = 0.65$ gave better results, according to their experimental program. When the eq. 4 is incorporated with eq. 5, the following resulting equation is obtained;

$$\frac{\sigma_1}{\sigma_c} = \exp\ (\ 0.0765\ RMR - 7.65\ ) + B \left[\frac{\sigma_3}{\sigma_c} \right]^{0.65} \qquad (6)$$

Thus, Bieniawski's proposal has been made applicable to all types of rock masses with different rock mass qualities. The criterion gives results quite comparable to the actual test results, especially at the ductile- brittle transition range of rocks at elevated confinement[27].

(B) <u>Hoek</u> <u>&</u> <u>Brown</u>, 1980

Hoek & Brown developed a non-linear empirical strength criterion for the rock masses using the uniaxial compressive strength of the intact rock material as a scaling parameter, and introduced two dimensionless strength parameters, m and s;

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \left[m \frac{\sigma_3}{\sigma_c} + s \right]^{\frac{1}{2}} \qquad (7)$$

The parameters m and s are shown to be related with the basic (unadjusted) RMR ratings as follows[28];

$$m = m_i \exp \left[\frac{RMR - 100}{14} \right] \qquad (8)$$

$$s = \exp \left[\frac{RMR - 100}{6} \right] \qquad (9)$$

m_i is determined from a fit of the Eq. 7 to triaxial test data from the intact laboratory specimens (s = 1).

The case histories show that the strength criterion developed by Hoek & Brown yields results quite similar to the laboratory test results in the brittle range[27]. However, above the brittle-ductile transition point, which is defined by the intersection of the line $\sigma_1 = 3.4\ \sigma_3$ (suggested by Mogi[29]) and the $\sigma_1 - \sigma_3$ curve, the criterion of Bieniawski-Yudhbir et al. yields better results.

Thus, it may be seen that the RMR-Geomechanics System is applicable to the approximate determination of the stress-strain characteristics of the rock formations through the empirical strength criteria. This application of the system increases its value even more as a practical tool in the rock mechanics design.

5.3.5 Determination of the In-situ Modulus of Deformation

The most important application of the RMR-Geomechanics System as far as this work is concerned, is the determination of the in-situ modulus of deformation.

Bieniawski proposed the following relationship for the correlation of the RMR Ratings with the in-situ modulus of deformation:

$$E_M = 2\ \mathrm{RMR} - 100\ (\mathrm{GPa}) \qquad (10)$$

This relationship is developed for the rock masses with high RMR Ratings, and works only for RMR > 50. As in the case of Fig. 4.1 in the Section 4.3.2, the proposal of Bieniawski has two important limitations;

(A) For RMR < 50, $E_M = 0$. This is not the case for many rock formations, as shown by Serafim & Pereira[30].

(B) For RMR = 100 (excellent rock), $E_M = 100$ GPa. as in Fig. 4.1, the limiting E_M value is 100 GPa. It is also known that some rock masses have E_M values much higher than that value. E.g., for Gabbro and Quartzite, values much exceeding 100 GPa have been reported, with average E_M for intact Gabbro being around 90 GPa [24].

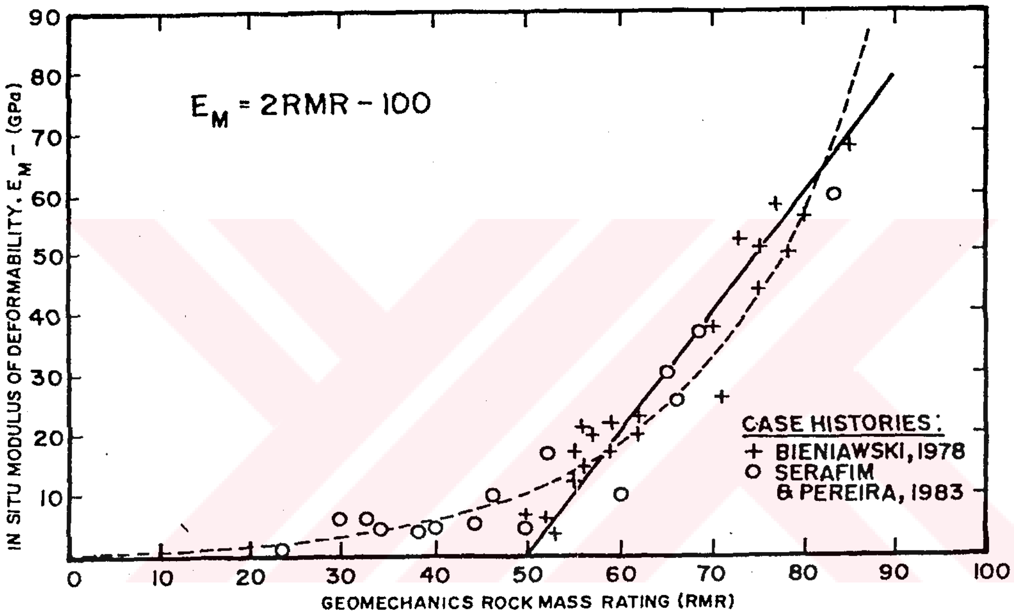
Recently, Serafim & Pereira [30] worked on a more diverse RMR horizon, and suggested the following relationship;

$$E_M = 10^{(RMR-10)/50} \quad \text{(GPa)} \qquad (11)$$

As in the Eq. 1 in the Section 4.3.2, this relationship is more general than the relationship developed by Bieniawski. It does not have a lower boundary for the RMR ratings. For RMR = 100, E_M = 178 GPa, which is a limiting value above the observed E values at the intact rock mass case histories (max. E_M about 120-130 GPa in general). So, this relationship overpredicts the E_M values at high RMR Ratings, although it is successful in the low RMR range.

So, it may be concluded that the two relationships should be used in conjunction with each other in the following manner;

$$\left.\begin{array}{l} * \text{ for } RMR > 58,\ E_M = 2\,RMR - 100 \\ \\ * \text{ for } RMR \le 58,\ E_M = 10^{(RMR-10)/50} \end{array}\right]$$



<u>Fig.</u> 5.5 - Correlation Between the RMR Ratings and the In-Situ Modulus of Deformation - after Bieniawski (15).

These two relationships are shown in Fig. 5.5. In the Appendix section, these two relationships are used for the determination of the in-situ modulus of deformation of Basalt, and the obtained values are compared with the values obtained through other techniques.

5.4 Advantages and Disadvantages

5.4.1 Advantages

The most striking characteristic of the RMR-Geomechanics System is its simplicity in usage. The classification scheme is quite simple to apply and the parameters are easily obtained from the borehole and/or underground mapping, with the uniaxial or point load test data being the only laboratory work required.

Apart from its simplicity,the system is applicable to the areas other than tunnelling, including hard rock mining, slope stability, approximate determination of the strength characteristics and the in-situ modulus of deformation.

These advantages of the system has earned it a wide and positive reputation as a trustable quantitative rock classification system.

5.4.2 Disadvantages

As stated before, in the establishment of the parameters, the system is rather conservative, which can tend to overdesign in some of the support system estimations. Bieniawski suggests the monitoring of the rock behavior during the construction and adjustments to the rock classification predictions as a vital practice to overcome the overdesign problem. From this aspect, the practice of the RMR-Geomechanics System is quite similar to the NATM practice in tunnelling.

Also, the RMR Ratings are based on the "average typical" conditions. This is a weakness especially in the tunnelling applications. In tunnelling, only the features critically affecting the stability of the excavation should be considered. For example, a number of very weak joint sets may strike nearly perpendicularly across the tunnel. Owing to their relative orientation, they do not materially affect the tunnel stability, however, including them in the classification procedure decreases the average joint condition rating [18]. The end result is overdesign of the support systems.

Another point criticized in the RMR-Geomechanics System is that it does not provide any guidelines for temporary support during excavation [17,18].

Barton [17] notes that it is impossible to separately vary the degree of joint roughness and the degree of filling, as obviously may occur in practice.

As a conclusion, it may be stated that, without an experienced user, the system, as in the case of other systems, may tend to yield unrealistic results. Although simple to use, the theory behind the RMR-Geomechanics System should be well understood and its shortcomings must not be overlooked. Bieniawski [16] suggests that at least two rock classification systems should be used together in order to obtain more realistic results.

However, it should be noted that the above stated shortcomings of the system and the critized points are mainly related to tunnelling applications, and not related to the determination of the in-situ modulus of deformation.

VI. BARTON ET AL. : Q SYSTEM OF ROCK CLASSIFICATION

6.1 Development of the System

The Q System was developed by Barton, Lien & Lunde in 1974 at the Norwegian Geotechnical Institute. For this reason, the system is also known as the NGI Rock Classification System. In the development of the system, a long analysis of 212 case histories was made. The system was developed simultaneously with, but independently from the RSR and the RMR-Geomechanics Systems. Q System is built excessively on the RQD Index, developing that concept through introducing five additional parameters to modify the RQD Index in order to account for the number of the joint sets, the joint roughness and alteration (filling), the groundwater and the various effects associated with loosening, high stress, squeezing and swelling.

The system aims to fill two major gaps in the structure of the rock classification systems developed so far;

(a) the disconnection of the rock mechanics data gathering with the decision mechanism, as suggested by Denkhaus[5].

(b) the absence of the squezzing and swelling effects of the rocks in the design of rockbolts, as suggested by Bjerrum (5).

Bjerrum also pointed out the insufficiency of the RQD Index to fully define the characteristics of the rock masses. This is a point of great significance since two separate rock masses with identical RQD values may behave in a totally different manner. A rock classification system which considered all of the characteristics of the rock mass behavior was really needed.

Barton et al.'s work helped to fulfill these requirements considerably. Especially, the detailed treatment of the joint roughness and alteration, which are features not particularly emphasized in the RMR-Geomechanics System, helped the Q System to gain international acceptance and popularity.

6.2 Classification Procedure

The Q System makes use of six parameters to describe the rock mass quality through adjusting the RQD Index values. The rock mass quality (Q) is expressed as;

$$Q = \frac{RQD}{J_n} * \frac{J_r}{J_a} * \frac{J_\omega}{SRF} \qquad (12)$$

Here, these three ratios multiplied with each other have the following meanings;

$$\frac{RQD}{J_n} = \text{Block size.}$$

$$\frac{J_r}{J_a} = \text{Minimum interblock shear strength.}$$

$$\frac{J_w}{SRF} = \text{Active stress.}$$

The determination of these three ratios and the Q value is made through the Tables 6.1, 6.2 and 6.3.

The range of the possible Q values that may be obtained is from 0.001 to 1000, covering the whole spectrum of rock mass qualities from heavily squezzing rock to sound unjointed rock.

The system is essentially a weighting process, in which the positive and negative aspects of a rock mass are assessed. While the assessment of most of the parameters is subjective, the process of support selection is organized and reasonably objective. The large number of case histories examined in the determination of the weightings for the parameters enabled the support recommendations to be quite objective (5).

The classification of the rock masses according to the Q values is shown in Table 6.4.

Table 6.1 -Classification Scheme of the Q System (Part I) - after Barton et al. (17).

1. 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

Note: (i) Where RQD is reported or measured as ≤ 10, (including 0) a nominal value of 10 is used to evaluate Q in equation (1).

(ii) RQD intervals of 5, i.e. 100,95,90, etc. are sufficiently accurate.

2. JOINT SET NUMBER (J_n)

A. Massive,no or few joints	0.5 - 1.0
B. One joint set	2
C. One joint set plus random	3
D. Two joint sets	4
E. Two joint sets plus random	6
F. Three joint sets	9
G. Three joint sets plus random	12
H. Four or more joint sets,random, heavily jointed,"sugar cube" etc.	15
J. Crushed rock,earthlike	20

Note: (i) For intersections use ($3.0 \times J_n$)

Note: (ii) For portals use (2.0 J_n)

3. JOINT ROUGHNESS NUMBER

(a) *Rock wall contact* and

(b) *Rock wall contact before 10 cms shear* (J_r)

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

Note: (i) Descriptions refer to small scale features and intermediate scale features,in that order.

(c) *No rock wall contact when sheared*

H. Zone containing clay minerals thick enough to prevent rock wall contact	1.0
J. Sandy,gravelly or crushed zone thick enough to prevent rock wall contact	1.0

Note: (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 orientated for minimum strength

Table 6.2 - Classification Scheme of the Q System (Part II) - after Barton et al. (17).

4. JOINT ALTERATION NUMBER	(J_a)	(ϕ_r) (approx.)
(a) *Rock wall contact*		
A. Tightly healed, hard, non-softening, impermiable filling i.e. quartz or epidote	0.75	(-)
B. Unaltered joint walls, surface staining only	1.0	(25-35°)
C. Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	(25-30°)
D. Silty-, or sandy-clay coatings, small clay fraction (non-soft.)	3.0	(20-25°)
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.	4.0	(8-16°)
(b) *Rock wall contact before 10 cms shear*		
F. Sandy particles, clay-free disintegrated rock etc.	4.0	(25-30°)
G. Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5 mm thickness)	6.0	(16-24°)
H. Medium or low over-consolidation, softening, clay mineral fillings. (continuous but <5mm thickness)	8.0	(12-16°)
J. Swelling -clay fillings, i.e. montmorillonite (continuous, but <5mm thickness) Value of J_a depends on percent of swelling clay-size particles, and access to water etc.	8 - 12	(6-12°)
(c) *No rock wall contact when sheared*		
K, L, M. Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6, 8, or 8-12	(6-24°)
N. Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	5.0	(-)
O, P, R. Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	10, 13, or 13-20	(6-24°)

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

Note: (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.

Table 6.3 - Classification Scheme of the Q System (Part III) - after Barton et al. (17).

6. STRESS REDUCTION FACTOR

(a) *Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.* (SRF)

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 ≤ 50m) 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 ≤ 50m) 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

Note: (i) Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence but do not intersect the excavation.

(b) *Competent rock, rock stress problems*

		σ_c/σ_1	σ_t/σ_1	(SRF)
H.	Low stress, near surface	>200	>13	2.5
J.	Medium stress	200-10	13-0.66	1.0
K.	High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-.33	0.5-2
L.	Mild rock burst (massive rock)	5-2.5	0.33-.16	5-10
M.	Heavy rock burst (massive rock)	<2.5	<0.16	10-20

Note: (ii) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to $0.8\sigma_c$ and $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where : σ_c = unconfined compression strength, and σ_t = tensile strength (point load), and σ_1 and σ_3 are the major and minor principal stresses.

(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

(c) *Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure* (SRF)

N. Mild squeezing rock pressure .... 5 - 10

O. Heavy squeezing rock pressure ....10 - 20

(d) *Swelling rock: chemical swelling activity depending on presence of water*

P. Mild swelling rock pressure .... 5 - 10

R. Heavy swelling rock pressure ....10 - 15

Table 6.4 - Classification of the Rock Masses According To the Q System - After Barton et al. (17).

Q	Rock Class	
0.001 - 0.01	1	Exceptionally Poor
0.01 - 0.1	2	Extremely Poor
0.1 - 1	3	Very Poor
1 - 4	4	Poor
4 - 10	5	Fair
10 - 40	6	Good
40 - 100	7	Very Good
100 - 400	8	Extremely Good
400 - 1000	9	Exceptionally Good

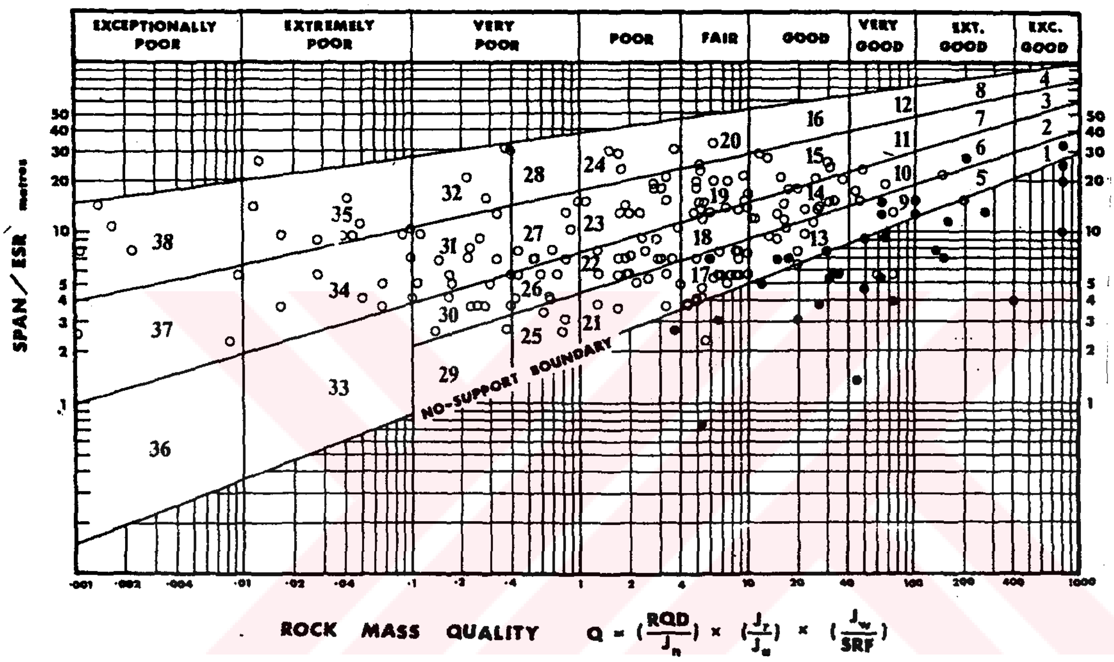
6.3 Applications of the System

6.3.1 Application to Tunnelling

The Q system provides highly quantitative guidelines for support measures for a wide horizon of rock mass qualities (Fig. 6.1). The system also emphasizes the "no-support" concept.

According to Barton et al., quite a number of tunnels to be constructed within various rock types and rock qualities, need not be supported permanently, provided that smooth blasting and thorough barring down takes place [17]. In Fig. 6.1, the first eight support categories indicate that no permanent support may be required at all. Einstein's works [31] confirms this "no-support" concept appreciably. Barton gives a thorough list of criteria to be applied for the determination of permanently unsupported tunnels, consisting of upper and lower bounds for the parameters used for the calculation of the Q values.

For the applications where permanent support is required, Barton et al. introduces a variable factor of safety, the Excavation Support Ratio (ESR), ranging from 0.8 (underground nuclear power stations) to 5.0 (temporary mine openings) [17].



<u>Fig.</u> 6.1 - Selection of the Rock Support Categories

- after Barton et al. (17).

The 212 case records in which the Q System has been applied, demonstrated that the system is an accurate quantitative rock classification system, and successfully applicable to tunnelling.

6.3.2 Application to the Empirical Strength Criteria

Bieniawski[19] proposed a correlation equation between the RMR-Geomechanics and the Q Systems;

$$RMR = 9 \ln Q + 44 \qquad (13)$$

This relationship has widely been used, and the case records reported confirm that this proposal is valid with a variation below 10 per cent. With the help of Eq. 13, it is possible to convert the empirical strength criteria developed for the RMR-Geomechanics System to the Q system.

For example, the empirical strength criteria of Bieniawski-Yudhbir et al. (Section 5.3.4) may be rewritten for the Q System in the following way;

$$\frac{\sigma_1}{\sigma_c} = 0.0176\, Q^{0.65} + B \left[\frac{\sigma_3}{\sigma_c} \right]^{0.65} \qquad (14)$$

Thus, the Eq. 13 is a useful linkage between the two important rock classification systems.

6.3.3 Determination of the In-situ Modulus of Deformation

As stated above, by making use of the Eq. 13, it is possible to convert the Eqs. 10 and 11 to the Q System.

This work proposes a simple method, which is consisting of establishing the Q ratings, converting them into the RMR ratings through Eq. 13, and obtaining the E_M values using the Eqs. 10 or 11, depending on the calculated RMR ratings.

Within the Appendix, this method is used to determine the in-situ modulus of deformation of basalt by the Q System.

6.4 Advantages and Disadvantages

6.4.1 Advantages

The main advantages of the Q System may be listed as follows;

(A) The large number of case records utilized to develop the system, which ensures that reliable support recommendations to be provided for a very wide range of tunnel sizes, types of excavation, depths and rock mass qualities.

(B) Applicability of the system to the other rock types than those described in the case records with confidence, provided that any special characteristics of the new rock type are adequately represented in the six classification parameters.

(C) The "no-support" and the ESR concepts, which are significant gap-fillings in modern tunnelling.

6.4.2 Disadvantages

The main disadvantage of the Q System is the nature of the Eq. 12, which is used to determine the Q rating. A slight change in either one of the six parameters seriously influences the numerical value of the rating. Thus, the Q system should only be used by those who have thorough knowledge in the rock mechanics, and well informed on the nature of the project that is being worked on.

Apart from that, some researchers claim that the parameters are ambigious and open to interpretation. Kirsten[32] points out that the joint alteration number (J_a) and the SRF are quite difficult to determine in a reliable way, and that the criteria for the determination of the classification parameters should be systematized and extended to overcome this problem.

It is evident that the Q System is the most quantitative and detailed rock classification system, which inevitably means that the user may easily confuse concepts and come up with unrealistic results.

VII. GEOPHYSICAL METHODS FOR THE DETERMINATION OF THE IN-SITU MODULUS OF DEFORMATION

The two most commonly used methods for the determination of the dynamic in-situ modulus of deformation are seismic refraction and sonic logging. Among these two, seismic refraction method is probably the most frequently used method for the determination of the in-situ modulus of deformation, and it will be discussed within this section.

Dynamic methods of in-situ examination of the rock masses are especially useful if, during the phase of research, they are combined with the static methods. The main advantages of the dynamic methods are that they enable fast and economic measurements, which do not require special preparatory work, and they enable the investigation of the rock mass undisturbed by the work of man in areas of the required size. By dynamic methods, the quasi-homogenous zones within a heterogenous complex can be separated relatively quickly [33].

The main difference between the static and dynamic methods for the determination of the in-situ modulus of deformation is in their durations. The static methods are those in which the loading of the rock mass, during an experiment, changes very slowly as a function of time, and the dynamic methods are those which, on the basis of caused impact, enable the gauging of the elastic waves.

In the geophysical methods, the rock mass is considered as a homogenous, elastic, isotropic medium. This consideration involves a degree of error, since the rock masses are generally non-homogenous and anisotropic, and do not behave elastically, as previously stated within this work. Thus, the geohysical methods involve this shortcoming within their theory, and the results obtained from the areas of evidently non-homogenous or anisotropic formations are generally not successful and not reliable.

7.1 Application of Seismic Refraction to the Determination of the In-Situ Modulus of Deformation

Seismic refraction surveys are used to determine the extent of surface weathering, the depth to a more dense geologic stratum, the thickness of the zone of loosened rock (de-stressed zone) around an underground opening, the dynamic in-situ modulus of deformation, etc. The refraction survey measures the arrival time of seismic energy at a number of geophones placed at known distances from the shot point. Most sismographs that are used to record the wave propagation have a number of channels so that the arrival times at the geophones can be measured. A blasting console is wired into the recording instrument, and the final record has the instant of the shot marked on the trace of each channel. As the energy arrives at each of the successive geophones, the trace shows a sudden increase in the amplitude. A plot of time versus distance can be used to calculate the seismic velocities and the depths to strata of greater density [3].

Velocity measurements can also be made between the borings by using one borehole for the shot and one or more boreholes for the geophones.

Assuming that the rock medium, in spite of its crackings, diaclases and lack of homogeneity, can be as a whole capable to present a relatively elastic mechanical behavior, it is known that when it is excited by a superficial impact, several types of propagating waves occur, each having different travelling velocities and trajectories, i.e. different modes [34].

The most important of these trajectories are the push-pull compressional or the longitudinal mode (P-Waves) and the transverse or shear mode (S-Waves). The P-Wave mode has the highest velocity and is the mode normally detected in the seismic refraction surveying. P-Wave velocity is related with the dynamic deformation modulus of elasticity of the medium material. In a similar manner, the S-Wave velocity is related with the dynamic shear or rigidity modulus of the material.

From the mathematical theory of elasticity, it can be shown that these velocities can be expressed by;

$$V_p = \left[\frac{(1-\nu)\,E}{\tau\,(1+\nu)\,(1-2\nu)} \right]^{\frac{1}{2}} \tag{15}$$

$$V_s = \left[\frac{G}{\tau} \right]^{\frac{1}{2}} \tag{16}$$

But since the shear modulus is related with the elasticity modulus, with;

$$G = \frac{E}{2\,(1+2\nu)} \tag{17}$$

$$V_s = \left[\frac{E}{2\,\tau\,(1+\nu)} \right]^{\frac{1}{2}} \tag{18}$$

From these relationships, the dynamic deformation moduli may be obtained by re-arranging;

$$E_{dp} = \frac{V_p^2\,\tau\,(1+\nu)(1-2\nu)}{(1-\nu)} \tag{19}$$

$$E_{ds} = 2\,V_s^2\,\tau\,(1+\nu) \tag{20}$$

The rock formation to be considered in the Appendix is basalt, with a rather homogenous and isotropic structure. Thus, the in-situ dynamic modulus of deformation values that will be obtained should be comparable to the results obtained from other methods.

VIII. CLASSIFICATION OF THE BASALT SAMPLES AND THE DETERMINATION OF THE IN-SITU MODULUS OF DEFORMATION

8.1 Introduction

The basalt samples used in this work are obtained from seven boreholes between the kms. 186+274-187+228 of the Toprakkale-İskenderun Motorway, from the area covered by basalt flows caused by the volcanic activity at the region. The main source of the volcanic activity is the Delihalil Hill, which is the origin of the volcanic character of the Erzin Plain, within the Hatay province. This basaltic region is locally known as the Leçelik Region. The samples are taken from the region where the basalt formation dips and covered by about 20 m. thick recent alluvial deposits. Since the region is very near the Mediterrenean Sea and with considerable amount of regular rains, the outcrops and the near-surface basalt formations are generally moderately weathered. So, they are not representative of the main basalt massive, which is estimated to be more than 160 m. thick in this area.For this reason, the sampling is made from the boreholes at which the basalt is encountered at some depth. a total of 22 samples is taken from theses boreholes. Various rock mechanics tests are conducted on the samples at the Soil Mechanics Laboratory of the General Directorate of

Highways, Ankara. The results of these tests are presented in Tables 8.2 to 8.4[35].

8.2 The Characteristics of the Basalt Samples

The basalt samples have a typical vesicular structure, with no evident anisotropy. The average unit weight is 24.97 kN/m3, ranging from 21.49 to 26.09 kN/m3. The RQD Index values are ranging from 40 to 89 per cent, with an average value of 61 per cent. TCR ranges in between 69 to 100 per cent, with an average value of 81 per cent.

The point load test results range from 4.2 to 13.0 MPa, with an average of 10.0 MPa. The compressive strength values calculated from these results range from 96.2 to 295.5 MPa, with an average of 226.9 MPa. On the other hand, the uniaxial compressive strength test results range from 65 to 146 MPa, with an average value of 114 MPa. The point load tests are conducted on the samples with lower amounts of vesicles in order to prevent premature failure. The compressive strength values calculated from the point load tests are therefore quite different from the uniaxial compressive strength tests. In order to maintain uniformity as much as possible, if two test results are available for a particular sample, one of them is considered in the calculations according to the nature of the sample and the magnitude of the point load test.

Table 81 - The RQD and TCR Values of the Basalt Samples

Borehole No.	Depth m.	RQD %	TCR %
KS 1	22.5-24.0	89	93
KS 3	20.2-21.5	56	76
KS 15	21.5-23.0	62	76
KS 16	20.2-21.7	40	69
KS 17	21.7-23.2	62	100
KS 19	21.4-22.9	52	77
KS 20	22.0-23.5	63	76
	Average	60.6	81.0

Table 8.2 - In-Situ Unit Weight Test Results[35]

Sample No.	Km.	Depth (m)	Unit Weight (kN/m3)
KS 19/1	187+147	21.4-22.9	26.09
KS 19/2	187+147	21.4-22.9	25.89
KS 19/3	187+147	21.4-22.9	25.69
KS 3/1	187+155	20.2-21.5	22.65
KS 15/1	186+274	21.5-23.0	23.83
KS 15/2	186+274	21.5-23.0	25.30
KS 16/1	186+298	20.2-21.7	21.49
KS 17/1	186+310	21.7-23.2	24.91
KS 1/1	186+330	22.5-24.0	25.79
KS 1/2	186+330	22.5-24.0	25.40
KS 1/3	186+330	22.5-24.0	25.40
KS 1/4	186+330	22.5-24.0	25.60
KS 20/1	187+228	22.0-23.5	25.50
KS 20/2	187+228	22.0-23.5	25.79
KS 20/3	187+228	22.0-23.5	25.30
		Average	24.97
		St. Dev.	9.00

Table 8.3 - Point Load Test Results (35)

Sample No.	Depth (m)	Index Is (MPa)	Calculated Comp. Strength (MPa)
KS 19/1	21.4-22.9	11.4	259.12
KS 19/2	21.4-22.9	12.0	272.76
KS 19/3	21.4-22.9	10.0	229.00
KS 3/1	20.2-21.5	7.0	157.90
KS 3/2	20.2-21.5	7.6	171.46
KS 3/3	20.2-21.5	8.6	194.02
KS 15/1	21.5-23.0	10.8	245.65
KS 15/2	21.5-23.0	12.4	281.85
KS 15/3	21.5-23.0	7.6	170.16
KS 16/1	20.2-21.7	4.2	96.18
KS 16/2	20.2-21.7	13.0	295.49
KS 16/3	20.2-21.7	12.4	279.74
KS 17/1	21.7-23.2	5.6	128.24
KS 17/2	21.7-23.2	11.0	251.90
KS 17/3	21.7-23.2	10.0	229.00
KS 1/1	22.5-24.0	11.4	265.76
KS 1/2	22.5-24.0	12.0	293.28
KS 1/3	22.5-24.0	10.0	261.70
		Average	226.85
		St. Dev.	59.76

Table 8.4 - UCS, Mod. of El., Poisson's Ratio
Test Results (35)

Sample No.	Depth m.	UCS MPa	El. Mod. GPa	Poisson's Ratio
KS 19/1	21.4-22.9	146.00	50.00	0.190
KS 13/1	20.2-21.5	65.00	75.00	-
KS 15/2	21.5-23.0	133.00	43.20	-
KS 17/1	21.7-23.2	78.00	82.00	0.250
KS 1/1	22.5-24.0	150.00	50.00	0.175
KS 20/1	22.0-23.5	112.00	66.70	0.340
	Average	114.00	61.15	0.240
	St. Dev.	35.72	15.67	0.075

The rather isotropic and homogenous structure of the basalt samples indicate that the seismic wave velocities may be successfully used for the determination of the in-situ dynamic modulus of deformation.

In 1981, The Electricity Works Survey Administration (E.İ.E.İ.) conducted seismic refraction surveys at the region for the Toprakkale-İskenderun Motorway, which was named as the Çukurova Motorway then. The wave velocities obtained for the basalt formations will be used for the cross-checking of the results obtained from the empirical methods (from the RQD Index, the RMR-Geomechanics and the Q Rock Classification Systems).

8.3 Data To Be Used In the Classification Scheme

Type of Rock Material: Basalt

Loading Axis: No evident anisotropy or bedding planes observed. Vesicular structure observed in some samples . The samples obtained from the cores are loaded on their axis.

Boreholes from which the samples are obtained:

KS15, KS16, KS17, KS1, KS19, KS3, KS20

Loading Rate: 0.5 - 1.0 MPa/s

Testing Apparatus: ELE Test Press and ELE Point Loading Device.

Fracture Mode: Separating into vertical layers and fracturing.

If the lowest and the highest values are excluded in the calculations, the average values of the geotechnical parameters will be as follows:

τ = 25.16 kN/m3

σ_{cs} = 230.72 MPa

σ_{gd} = 117.25 MPa

E = 60.43 GPa

ν = 0.220

I_s = 10.0 MPa

8.4 Classification of the Basalt Samples

Using these average values and the three rock classification systems examined within the scope of this work, the following results are obtained:

8.4.1 Classification according to the RQD Index

av. RQD = 61

According to Deere, the rock mass is classified as "fair to good rock".

8.4.1.1 Determination of the in-situ modulus of deformation directly from the RQD Index

The RQD Index varies between 40 to 89 per cent. From Fig. 4.1, the corresponding E_M values are calculated as follows;

Best Case: RQD = 89 per cent; E_M = 56.9 GPa

Worst Case: RQD = 40 per cent; E_M = 1.8 GPa

Thus, the average E_M = 29.3 GPa.

8.4.1.2 Determination of the in-situ modulus of deformation using the Modulus Ratio

From Fig. 4.2, it is observed that the best and the worst Modulus Ratios are 0.77 and 0.12, respectively, when the proposal shown on the figure is used.

Using the laboratory modulus of elasticity average for E_{t50} (since no other data exists) would not be a significant error. Thus, the following results are obtained;

Best Case: Mr = 0.77 ; E_M = 46.5 GPa

Worst Case: Mr = 0.12 ; E_M = 7.3 GPa

Thus, the average E_M = 26.9 GPa.

8.4.2 Classification according to the RMR-Geomechanics System

Relevant Information for the basalt samples:

Average $I_s \geq 10.0$ MPa.

Average RQD = 61 per cent, ranging between 40 to 89 per cent.

Average TCR = 81 per cent, ranging between 69 to 100 per cent.

One joint set to one plus random joint sets per core barrel, and general presence of groundwater (wet to dripping conditions).

8.4.2.1 Classification parameters

Using the below cases and the Tables 5.1 and 5.2;

Best Case (KS 1)	Worst Case (KS 16)
RQD = 89 %	RQD = 40 %
TCR = 100 %	TCR = 69 %
av. I_s = 11.1 MPa	av. $I_s \cong$ 10.0 MPa

i. Strength of the intact rock material

Best Case = 15/15

Worst Case = 15/15

ii. Drill core quality

Best Case = 18/20

Worst Case = 8/20

iii. Spacing of discontinuities

Best Case

At most one spacing per core (0.6-2.0 m.)

i.e. 15/20

Worst Case

One plus random (0.2-0.6 m.)

i.e. 10/20

iv. *Condition of discontinuities*

Best Case

Slightly rough surfaces, separation < 1 mm., moderately weathered walls, i.e. 22/30.

Worst Case

Slightly rough surfaces, separation 1-5 mm., highly weathered walls, i.e. 18/30.

v. **Groundwater**

Wet to dripping conditions for all cases, i.e. 6/15 for both cases.

Thus, the basic (unadjusted) RMR for the two cases are;

Best Case = 76/100

Worst Case = 57/100

8.4.2.2 *Determination of the rock classes*

Since the evaluation of the RMR Ratings and the relevant rock classes are made for the determination of the in-situ modulus of deformation of the basalt samples and not for tunnelling applications, section D of the Table 5.2 has no direct meaning. Therefore, no rating adjustments are to be made.

So, the determined rock classes are as follows:

<u>Best Case</u> :

Rating = 76/100

Class No.: II - "Good Rock"

<u>Worst Case</u> :

Rating = 57/100

Class No.: III - "Fair Rock"

Thus the basalt samples considered for this thesis are ranging from good to fair rock, with an average RMR Rating of 66.5, corresponding to "good rock".

8.4.2.3 Meanings of the rock classes

<u>Best Case</u> (Rock Class II)

Average stand-up time = 6 months for 8 m. span

Cohesion of the rock mass = 300-400 KPa

Friction angle of the rock mass = 35° - 45°

<u>Worst Case</u> (Rock Class III)

Average stand-up time = 1 week for 5 m. span

Cohesion of the rock mass = 200 -300 KPa

Friction angle of the rock mass = 25° - 35°

For the average mechanical characteristics of the basalt samples, the following values may be taken;

C = 300 KPa

ϕ = 35°

These values are within the range of the observed in-situ values for basalt (C = 200 - 400 KPa, ø = 31° -38°)[24].

8.4.2.4 In-situ modulus of deformation

<u>Best Case</u>

RMR = 76

Using Eq.(10),

E_M = 2 RMR - 100 (GPa)

E_M = 2 * 76 - 100 = 52.0 GPa

<u>Worst Case</u>

RMR = 57

Since RMR < 58, using Eq. (11),

$$E_M = 10^{(RMR - 10)/40} \quad (GPa)$$

$$E_M = 10^{47/40} = 15.0 \text{ GPa}$$

Therefore, the average E_M for the basalt samples using the RMR-Geomechanics System would be;

av. E_M = (52 + 15)/2 = 33.5 GPa

This value indicates that, in average, the in-situ value is about 59 per cent of the laboratory value.

8.4.3 Classification According to the Q System

8.4.3.1 Establishment of the parameters

(i) Rock Quality Designation (RQD)

Best Case : RQD = 89 per cent ; "good".

Worst Case : RQD = 40 per cent ; "poor".

(ii) Joint Set Number

Best Case : Massive, no or few joints ; J_n = 0.7 .

Worst Case : One joint set plus random ; J_n = 3.0.

(iii) Joint Roughness Number

For either cases, discontinuous joints ; J_r = 4.0 .

(iv) Joint Alteration Number

Best Case : Unaltered joint walls, surface staining only ; J_a = 1.0 .

Worst Case : Silty or sandy clay coatings, small clay fraction (non-softening) ; J_a = 3.0 .

(v) Joint Water Reduction Number

In all cases, large inflow or high pressure, considerable outwash of the joint fillings;

J_w = 0.5 for both cases.

(vi) Stress Reduction Factor

In all cases, single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m.);

SRF = 5.0 for both cases.

Therefore,

Best Case :

Q = (89/0.7) * (4.0/1.0) * (0.5/5.0) = 50.9,

"very good rock".

Worst Case :

Q = (40/3.0) * (4.0 /3.0) * (0.5/5.0) = 1.8,

"poor rock".

8.4.3.2 Comparison with the RMR System

Checking the correlation of the RMR and the Q ratings using the Eq. (13);

$$RMR = 9 \ln Q + 44$$

Best Case :

RMR calc. = 79.4

Actual RMR = 76

Variation = 4.5 per cent

Worst Case :

RMR calc. = 49.3

Actual RMR = 57

Variation = 13.5 per cent

Thus, the RMR-Geomechanics and the Q approaches to the determination of the rock mass quality are compatible with each other. Both systems yield quite close results when handled carefully.

8.4.3.3 In-situ modulus of deformation

Using the calculated RMR values this time, the in-situ modulus of deformation estimates of the Q System are calculated to be;

<u>Best Case</u>

RMRcalc = 79.4

Using Eq. (10),

$E_M = 2 * 79.4 - 100 = 58.8$ GPa

<u>Worst Case</u>

RMRcalc = 49.3

Since RMR < 58, using Eq. (11),

$E_M = 10^{39.3/10} = 9.6$ GPa

Thus, the average E_M for the basalt samples using the Q System would be obtained as;

av. $E_M = (58.8 + 9.6)/2 = 34.2$ GPa

8.5 Comparison with the Geophysical Survey Data

The average seismic velocities for the basalt samples, according to the 1981 survey by the E.İ.E.İ.[36] are as follows;

av. $V_p = 3900$ m/s

av. $V_s = 2400$ m/s

The surveys are made at approximately 5000 meters earlier on the motorway alignment from the location of the boreholes from which the sampling has been made. At the area of seismic refraction, the basalt formation is at the surface, and naturally, much more weathered and fractured. Thus, although the dynamic in-situ modulus of deformation is expected to be greater than the static modulus, the excessive weathering and fracturing of the basalt outcrops should result in close deformation moduli values for these two areas.

Using the equations (20) and (19) in Section 7.1 and obtaining the necessary unit weights and Poisson's Ratios from the laboratory results (Tables A.2 and A.4), since no other data is available;

$$\text{av. } \tau = 25.16 \text{ GPa}$$

$$\text{av. } \nu = 0.220$$

So,

$$E_{d_p} = 33.5 \text{ GPa}$$

$$E_{sp} = 35.4 \text{ GPa}$$

$$\text{av. } E_d = 34.5 \text{ GPa}$$

IX. CONCLUSION AND DISCUSSION OF THE RESULTS

Within the scope of this thesis, the rock classification systems are used for the empirical determination of the in-situ modulus of deformation of the basalt samples from the Leçelik Region - Hatay, Türkiye.

As stated within this thesis, determination of the in-situ modulus of deformation of the rock masses is of great importance, since the in-situ moduli differ appreciably from the intact or near-intact laboratory moduli. As it is well known, the deformation modulus is one of the key parameters of the foundation design, since it is used for the determination of the expected settlement. The in-situ testing is quite expensive, and as stated before, may not yield healthy results. Thus, for the preliminary design works such as preliminary structure location, cheaper and practical empirical methods are quite helpful.

Moreover, for the basalt, it is evident that samples are not adequately representative of the general basalt mass, as it may be observed from the poor correlation between the Point Load and UCS test results. On the other hand, when the results from the four static and two dynamic methods are considered altogether, it is seen that the results fall within a narrow range;

av. E_M from the RMR System = 33.5 GPa

av. E_M from the Q System = 34.2 GPa

av. E_M from the RQD Index = 29.3 GPa

av. E_M using RQD + Mod. Ratio = 26.9 GPa

av. E_d from the P-Waves = 33.5 GPa

av. E_d from the S-Waves = 35.4 GPa

If, instead, the Elab = 60.4 GPa value were to be considered for the design, using a safety factor of five, as in the normal practice;

$$E_M = 60.43/5 = 12.1 \text{ GPa}$$

Thus, it is observed that the combination of the empirical static methods and the seismic refraction surveys provide much refined E_M values than using a fixed reduction factor for the laboratory values.

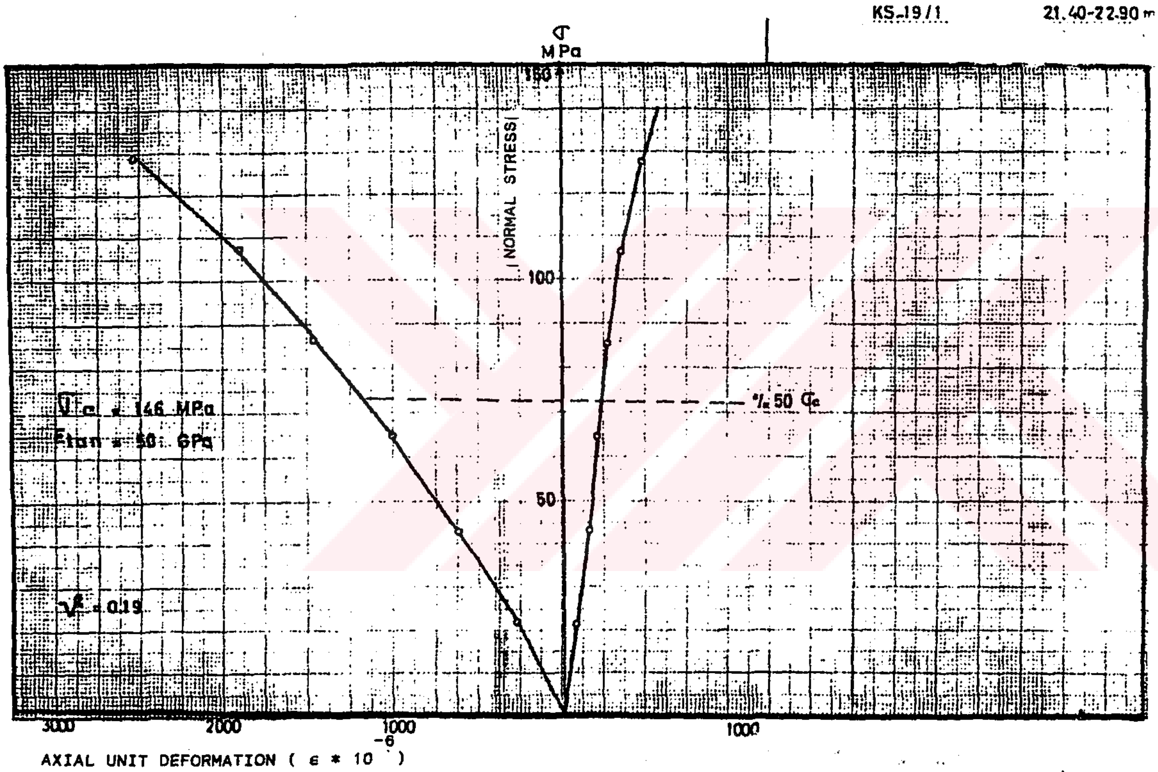
The average static in-situ modulus of deformation is, av. E_M (static) = 31.0 ± 3.5 GPa (with 11.3 per cent variation). This is about 51 per cent of the laboratory values.

Using a fixed reduction factor for all degrees of fracturing and weathering is not a good engineering judgement. On the other hand, in-situ plate loading tests or dilatometer testing is quite expensive, and still may not represent the average in-situ conditions.

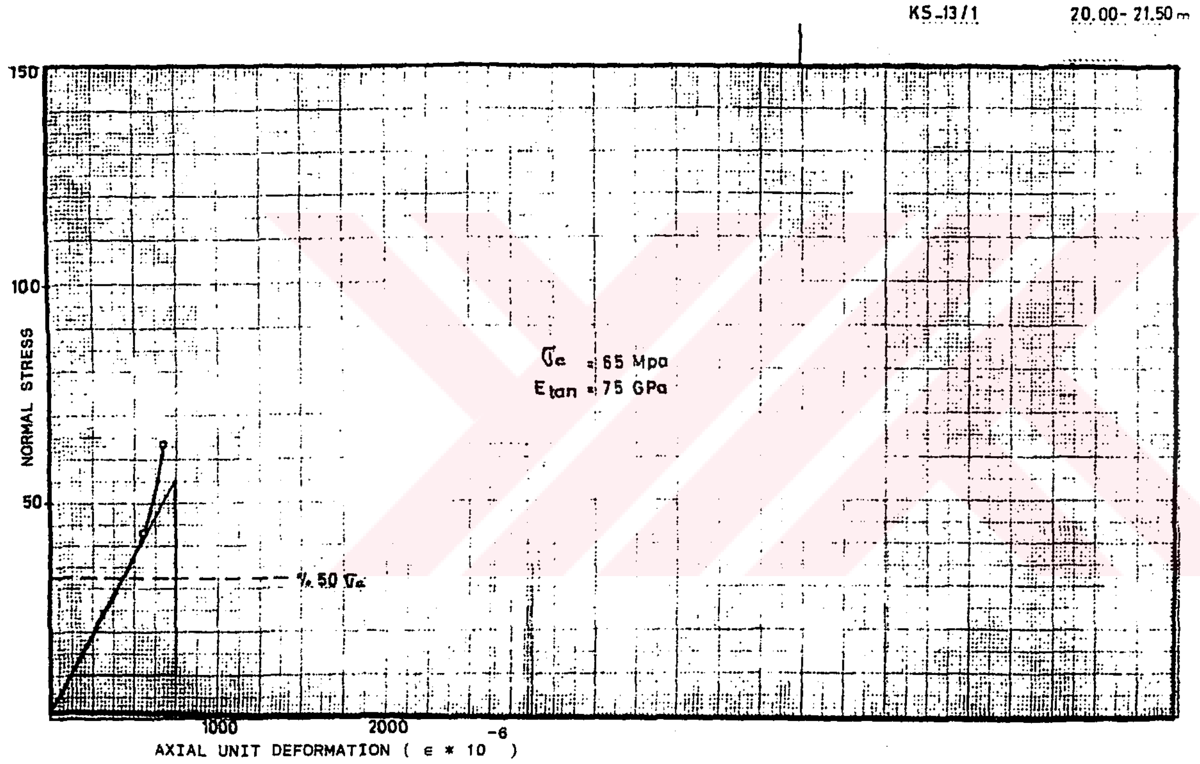
Following the recommendations of the researchers of the rock mechanics to use at least two rock classification systems at the same time, and to emply both static and dynamic methods, the average in-situ modulus of deformation of the basalt samples from the Leçelik Region have been determined in a practical and inexpensive way. Apart from that, the geophysical surveys of the year 1981 are verified, and more information is obtained for this important basalt massive of Türkiye.

APPENDIX A - ORIGINAL K.G.M. TEST RESULT SHEETS

(Results are listed in Table 8.4)

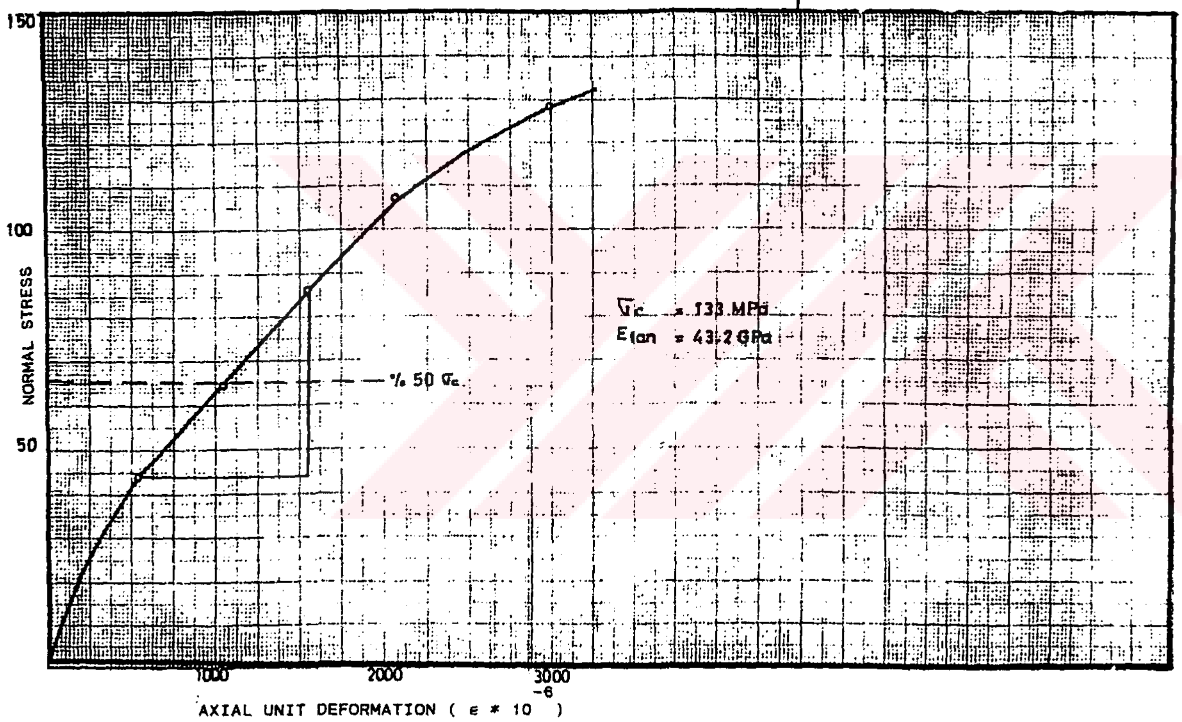


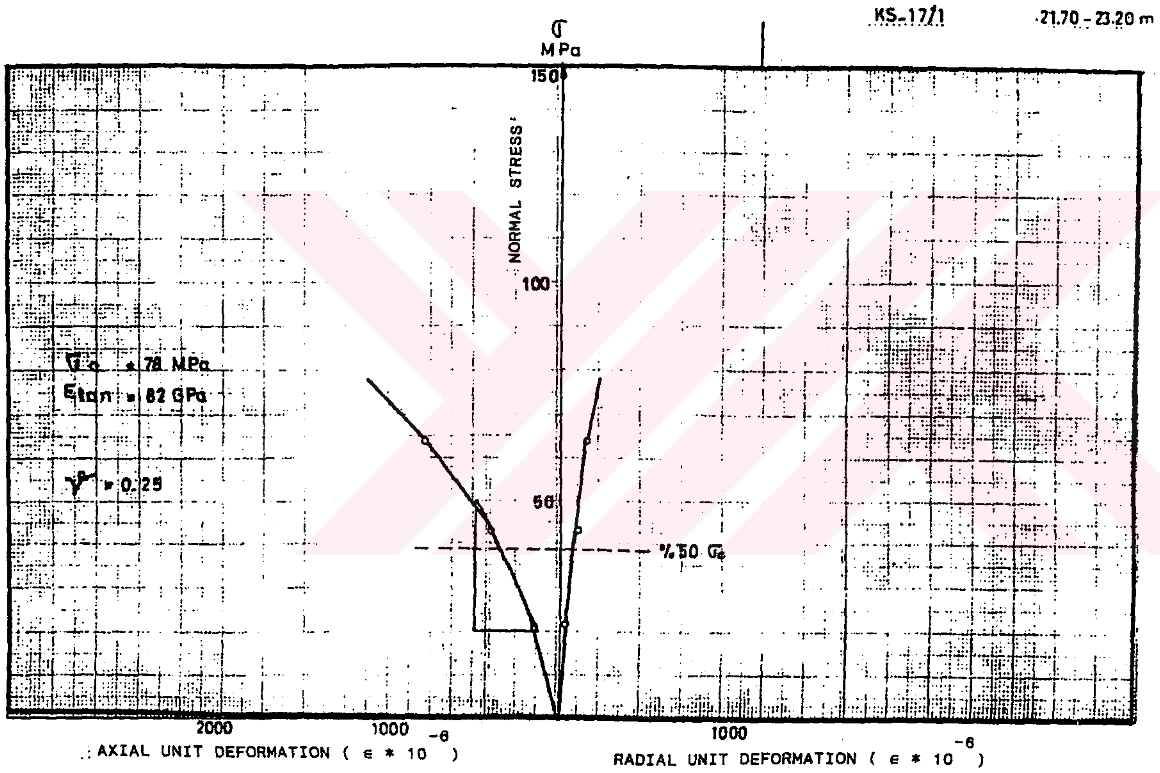
KS-19/1
21.40-22.90 m
σ
MPa
150
100
50
NORMAL STRESS
σc = 146 MPa
Etan = 50 GPa
ν = 0.19
% 50 σc
3000
2000
1000
1000
AXIAL UNIT DEFORMATION (ε * 10^-6)



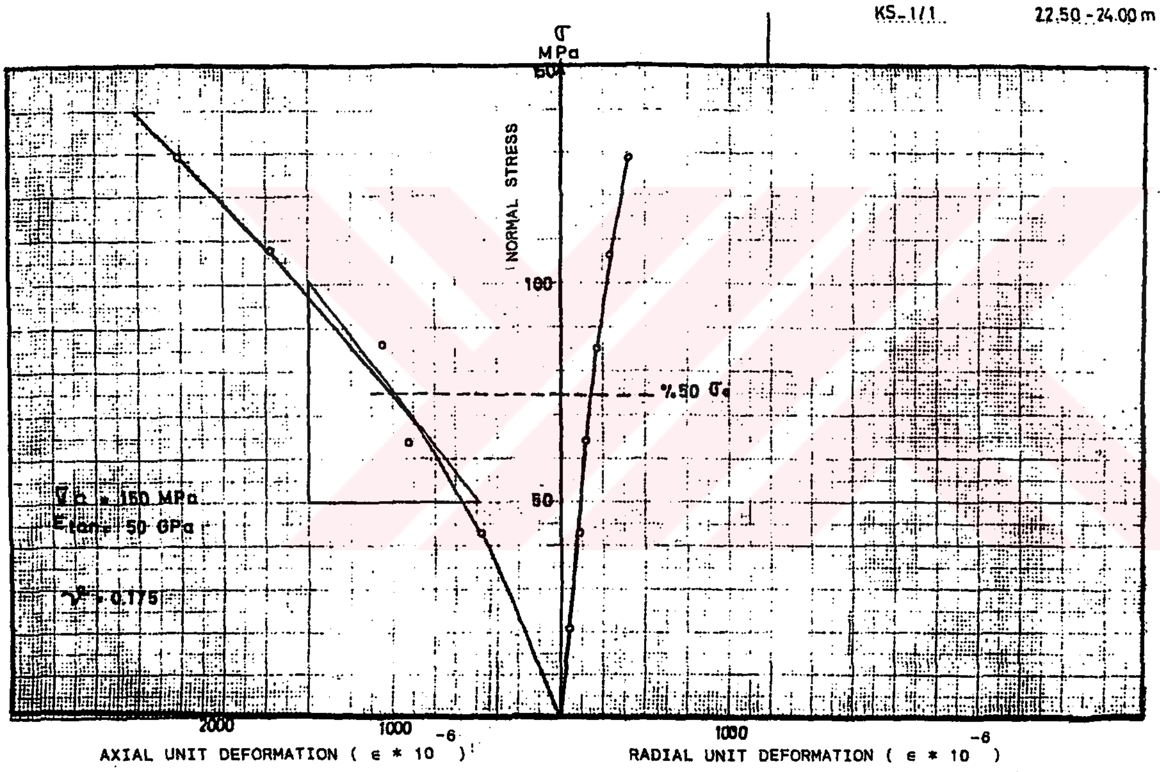
KS-13/1
20.00- 21.50 m
150
100
50
NORMAL STRESS
σc = 65 Mpa
Etan = 75 GPa
%50 σc
1000
2000
AXIAL UNIT DEFORMATION (ε * 10 -6)

KS - 15 / 2 21.50 - 23.00 m.

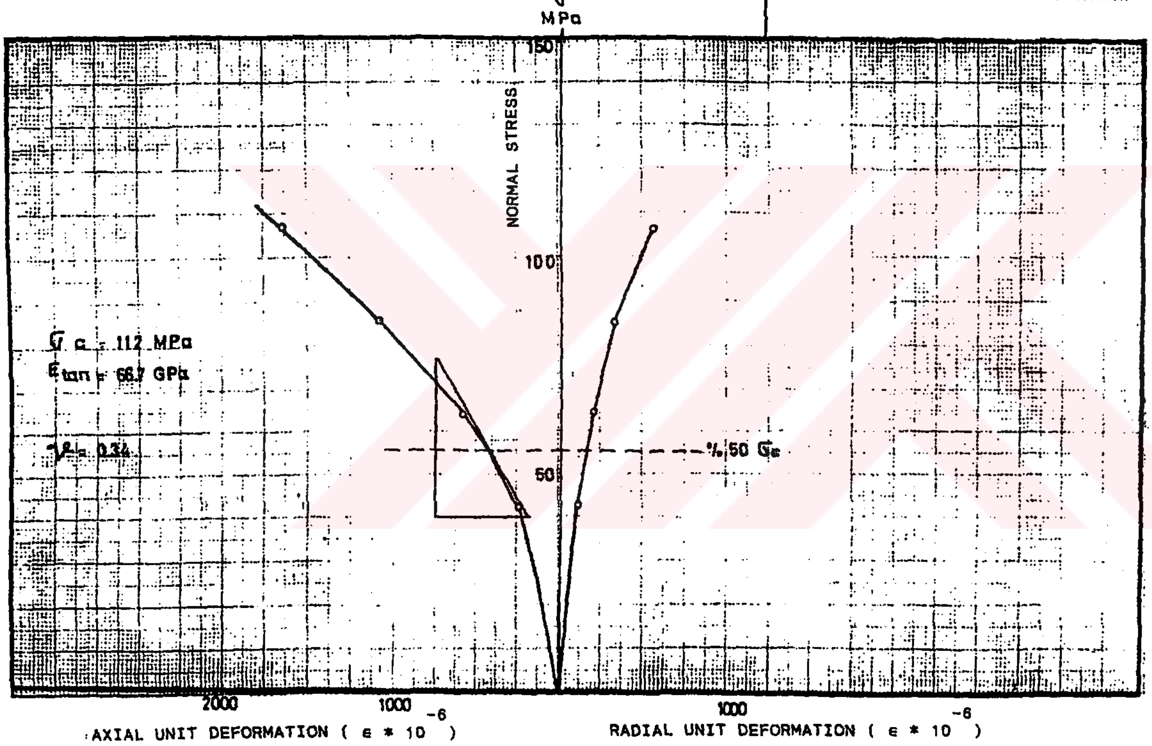




KS-17/1
21.70 - 23.20 m
σ
MPa
NORMAL STRESS
150
100
50
σc = 78 MPa
Etan = 82 GPa
ν = 0.25
%50 σc
2000
1000
1000
AXIAL UNIT DEFORMATION (ε * 10^-6)
RADIAL UNIT DEFORMATION (ε * 10^-6)


KS-1/1
22.50 - 24.00 m
σ
MPa
150
NORMAL STRESS
100
50
%50 σc
σc = 150 MPa
Etan = 50 GPa
ν = 0.175
2000
1000
1000
AXIAL UNIT DEFORMATION (ε * 10^-6)
RADIAL UNIT DEFORMATION (ε * 10^-6)

KS- 20/1 22.00-23.50 m



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