DETERMINATION OF CLASSIFICATION PARAMETERS FOR WEAK AND STRATIFIED ROCKS BASED ON RMR AND Q-SYSTEMS

A MASTER'S THESIS

in

Mining Engineering

Middle East Technical University

By

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February,1989

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156 pages, February 1989

ABSTRACT

The investigations carried out in Bigadiç-Mine Region indicated that RMR and Q-systems although widely used in mining and tunnelling practice could not fully describe the characteristics of clayey and stratified weak-rocks. Consequently, the applications that would be carried out based on originally suggested RMR and Q index values could be misleading.

In this study, the ore and rock units encountered in Bigadiç Region were classified in terms of original RMR and Q-systems. During determination of original RMR and Q index values a number of difficulties were encountered. These difficulties were associated with the

following: Obtaining suitable cores for preparation of standard specimens that will be used in uniaxial compressive strength tests; determination of RQD, effect of clay and water, joint spacing, and joint conditions in RMR-systam; and determination of RQD, joint alteration number, and joint set number in Q-system. Based on original and modified RMR and Q values; regression equations were determined between RMR and Q. Considering the original and modified index values the following output parameters were compared with each other: rock mass description, rock class, maximum unsupported span, span, roof span versus stand-up time relationship, support pressures, Modulus of Deformation, m and s material properties, and internal friction angles.

Key Words: Rock-mass classification, rock mass description, unsupported span, stand-up time, support pressure, m and s constants,internal friction angle.

ZAYIF KAYALARDA RMR VE Q SİSTEMLERİNE

BAGLI OLARAK SINIFLAMA PARAMETRELERİNİN

BELİRLENMESİ

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Tez Yöneticisi: ~~Doç.Dr~~ Prof. Dr.Erdal Ünal

156 sayfa, Şubat 1989

ÖZET

Bigadiç-Simav Ocağı bölgesinin kaya kütlesi sınıflandırmaları üzerindeki araştırmalar ve çalışmalar göstermiştir ki; RMR ve Q sınıflama sistemleri özellikle killi ve tabakalaşma gösteren zayıf kaya kütlesi özelliklerini yeterli derecede tanımlayamamakta dolayısı ile bulunan indeks değerlerine göre yapılan uygulamalar yanıltıcı olabilmektedir.

Bu çalışmada, Bigadiç Bölgesi'ndeki kaya ve cevher birimleri orijinal RMR ve Q sistemlerine göre sınıflandırılmıştır. Bu çalışma esnasında, orijinal girdi parametrelerinin anlatımında birtakım güçlükler ile karşılaşıl-

mıştır. Bu güçlükler, RMR-sistemi için; RQD, suyun ve kilin etkisi, çatlaklar arası mesafe ve çatlakların konumunun belirlenmesi; Q sistemi için ise RQD, süreksizlik değişme sayısı ve süreksizlik sayısının belirlenmesi ile ilgilidir. RMR ve Q indeks değerleri bulunmuş; RMR ve Q değerleri arasındaki regrasyon eşitlikleri hesaplanmıştır. Orijinal ve değiştirilmiş indeks değerleri aşağıdaki parametreler için karşılaştırılmıştır: RMR ve Q sistemlerinin önerdiği tahkimatsız durabilen en büyük galeri genişliği, RMR değerlerine göre en büyük açıklığın tahkimatsız durabilme zamanları, bunun yanısıra tahkimatsız bir galerinin üzerinde oluşabilecek tavan yükleri, kaya kütlesinin deformasyon modülü ve ayrıca, kaya kütlesinin malzeme özelliklerine bağlı olan m ve s sabitleri RMR sistemine göre hesaplanmıştır. Son olarak değişik kayaların içsel sürtünme açıları her iki sınıflandırma sistemine bağlı olarak analiz edilmiştir.

Anahtar kelimeler: Kaya kütlesi, sınıflama sistemleri, kaya kütlesi tanımlaması, en büyük tahkimatsız durma açıklığı, en fazla durma zamanı, kaya yükü, m ve s sabitleri, içsel sürtünme açısı.

ACKNOWLEDGEMENTS

I wish to express my appreciation to Assoc.Prof.Dr. Erdal Ünal for his kind supervision, valuable comments and suggestions throughout this research.

I also express my thanks to Assoc.Prof.Dr.A.Günhan Paşamehmetoğlu and Assoc.Prof.Dr.Levent Tutluoğlu for their valuable suggestions and advices. Particular thanks are extended to Prof.Vedat Doyuran and Assoc.Prof.Dr. Abdurrahim Özgenoğlu for serving on the Master's Thesis Committee.

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CHAPTER I

INTRODUCTION

Roof falls in underground mines continue to be a major problem in terms of safety and economy. Today, vast majority of the fatallities occuring in underground mines are due to roof falls. Although loss of production due to strata control problems is very difficult to estimate, it is undoubtadly significant. Estimating roof control costs for new areas is another major problem. Therefore, economics and safety demands the realization of mining conditions prior to and during any mining activity. This obviously requires more precise assessment, particulary determination of the rock mass conditions.

Mining in larger working places, at increasing depths and in steeply dipping seams necessitates better ground control methods and in particular better prediction of unstable roof conditions. Unexpected changes in roof conditions are often encountered in underground mines. The result of this is roof falls before preventive measures can be taken. Intensive studies of the roof falls indicate that they can often be attributed to one,or a combination of the following parameters:

(i) lithology of the roof strate;

(ii) characteristics of discontinuities (bedding planes, joints, faults, etc.)

(iii) humidity;

(iv) groundwater inflow;

(v) in-situ stress;

(vi) stress relief due to surface erosion and stream channel;

(vii) geological irregularities (clay veins, fossils, concretions, etc.).

Therefore, stability can be assessed best by a thorough study of these parameters, and by classification of the rock mass.

1.1. Statement of the Problem

In underground mining today, the safety and economy aspects demand a better understanding of the mining conditions, particularly from the roof stability stand points. The question is whether or not the rock-mass classification systems which have been used successfully in rock tunnelling and hard-rock mining can be directly utilized for the purpose of roof-stability assessment in weak and stratified rock. For example, the most widely used rock-mass classification systems, namely those proposed by Bieniawski (1973, 1979, 1984) and Barton et.al.

(1974) may not be adequate, in their original form, for classifying the weak and stratified rock.

1.2. Scope of Thesis

The main objectives of the present study are as follows:

(i) to utilize the RMR and Q-systems for characterization of the weak and stratified rock formations encountered in Bigadiç Barox - Mine Region,

(ii) to introduce modifications, if and when necessary, for the ratings of the input parameters utilized in RMR and Q-systems, in order to better characterize the weak and stratified rock mass;

(iii) to compile field data for assessment of engineering properties of the rock-mass encountered in Bigadiç Colemanite-Mine Region.

1.3. Outline of the Thesis

In order to meet the objectives stated in Section 1.2, a detailed review of the literature associated with methods of mine-roof-stability assessment was conducted in Chapter II. The rock-mass classification systems were also reviewed and critically assessed in this Chapter. In thes study, various geotechnical studies including both field and laboratory investigations and laboratory tests were carried out aimed at providing input-data for rock-mass classification systems. These studies were presented in Chapter III. In Chapter IV, the rock mass encountered in Bigadiç Colemanite-Mine Region was classified based on RMR and Q-systems. The engineering parameters such as roof span, stand-up time, rock-load height, support pressure and other parameters were also discussed in Chapter IV. The modifications associated with some of the input parameters utilized in RMR and Q-systems were introduced in Chapter V. In addition, a comparison was made between the original and modified ratings (RMR and Q values) in terms of the output of these classification systems. Finally, the main conclusions associated with this study and the recommendations for future work were included in Chapter VI.

CHAPTER II

REVIEW OF THE LITERATURE

2.1. General

In this chapter, the main rock mass classification systems, utilized in mining and tunnelling are briefly reviewed. The emphasis will be given to Bieniawski's RMR and Barton's Q Systems since these classification systems will be the back-bone of the studies carried out in this thesis. The new developments associated with RMR and Q Systems are also included at the end of this chapter. The output of the RMR and Q-systems, on the other hand, be reviewed in Chapter 4, together with results obtained from in-situ investigations.

2.2. Rock Mass Classification Systems

Rock mass classifications being the backbone of the emprical design approach are widely employed in rock engineering. In fact, on many projects, the classification approach serves as the only pratical basis for the design of complex underground structures. Most of tunnels constructed at present make use of a number of classification system. The mostly used and the best known of these is Terzaghi's rock-load classification which was introduced over 40 years ago (Terzaghi,1946).

took cognizance of the new advances in rock-support technology, namely, rockbolts and shotcrete, as well as addressed different engineering projects: tunnels, chambers, mines, slopes and foundations. Today, there are many different rock-classification-systems in existence. The more common ones are presented in Table 2.1.

Table 2.1. Major Rock Mass Classification Systems

Classification System	Orginator and date	Country of Origin	Application Area
Rock loads	Terzaghi,1946	USA	Tunnels with steel supports
Stand-up time	Lauffer,1958	Australia	Tunneling
Rock quality designation	Deere, 1964	USA	Core loging tunneling
Intact rock strength	Deere,Miller,1966	USA	Communication
RSR concept	Wickham,et al,1972	USA	Tunneling
Geomechanics Classification (RMR System)	Bieniawski, 1973	S.Africa and USA	Tunnels, mines, foundations
Q-System	Barton et al.,1974	Norway	Tunneling,large chamber
Strength/block size	Franklin, 1975	Canada	Tunneling
Basic geotechnical classification	ISRM,1981	International	General

Rock mass classifications have been successfully applied throughout the world: in the United State (Deere, 1964, Wickham et.al.,1972, Bieniawski,1979), Canada (Coates, 1964, Franklin,1975), Western Europe (Lauffer,1958, Pacher et.al., 1974, Barton et.al.,1974), South Africa (Bieniawski, 1973, Oliver,1976, Laubscher,1975), Australia (Barton,1977, (Bäczynski,1980), New Zealand (Rutledge,1978), Japan (Ikeda, 1970), USSR (Protodyakonov,1974), some East European countries (Kidybinski,1979).

Recently, major advances were made in the use of rock mass classifications in coal mining (Bieniawski, et.al., 1980; Ghose and Raju,1981, Seegmiller,1983, Unal,1983, Thill,1984, Karmis and Kane,1984) and in hard rock (metal) mining (Cummings et.al.,1982, Kendorski et.al.,1983). In longwall mining, the rock mass classification approach has been utilized for assessment of roof spans and rock cavability (Chose and Dutta, 1987, Unrug and Szwilski,1980, Kidybinski,1979).

A rock mass classification has the following aims in an engineering application :

a. To divide a particular rock mass into groups of similar behaviour;

b. To provide a basis for understanding the characteristics of each group;

c. To yield quantitative data for engineering design; and

d. To provide a common basis for communications.

2.3. Bieniawski's Geomechanics Classification System (RMR - System)

The "Geomechanics Classification of Rock Masses" or "Rock Mass Rating-(RMR)" Concept was developed by Bieniawski in South Africa in 1973.

After a detailed study of classification concepts, structural regions in rock masses, and the parameters that are most significant in the behavior of rock masses, Bieniawski developed his rock mass rating (RMR) system. Later, he modified and extended the RMR concept (Bieniawski, 1974, 1976, 1979). This engineering classification,utilizes the following six parameters, all of which are either measurable in the field, or obtainable from borings.

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

The following explanations and terminology for RMR classification system are relavent. (Gotechnical log)

1. Strength of intact rock material

Bieniawski uses the classification of the uniaxial compressive strength of intact rock. Alternativelly, with the exception of very low strength rocks the point load index may be used as a measure of intact rock material strength.

2. Rock Quality Designation

The Rock Quality Designation (RQD) is defined as proportion of a NX borehole core (54 mm .dia.) drill-run which consist of intact lengths equal to or longer than 100 mm

$$RQD = 100 \times \frac{\Sigma \text{ lengths of core pieces} > 100 \text{ mm}}{\text{length of drill-run}} \qquad (2.1)$$

3. Spacing of Joints

The term joint is used to mean all discontinuities which may be joints, faults, bedding planes and other surfaces of weaknees.

4. Condition of Joints

This parameter accounts for the seperation (operture) of the joints, their continuity, the wall condition (soft or hard), wheathering of the rock wall and the presence of infiling material in the joints.

5. Ground Water Conditions

An attempt is made to account for the influence of ground water flow on the stability of underground excavations in terms of the observed rate of flow into to the excavation, the ratio of joint water pressure to major principal stresses or by some general qualitative observation of ground water conditions.

Since the various parameters are not equally important for the overall classification of rock mass, importance ratings are allocated to the different value ranges of the parameters, a higher rating indicating better rock mass conditions. To classify a rock mass, it is divided to a number of structural regions, i.e., zones in which certain geological features are more or less uniform within each region. For each region, the typical rather than the worst conditions are evaluated, and the importance ratings are assigned to each parameter according to a table provided for this purpose. The summation of five ratings yields the basic overall RMR for the structural region under consideration which then is adjusted due to the influence of the strike and dip of joints.

In case of tunnels and chambers, the output from the Geomechanics classification are:

(i) Five rock classes in groups of twenty reatings each.

(ii) Roof span versus stand-up time relationship

(iii) Support recommendations

(iv) Estimation of various engineering properties of rock mass (i.e., rock load height, modulus of deformation, internal friction angle)

To ensure the full structural stability a monitoring program is also recommended by RMR-system.

2.4. Barton's Q-system

The Q-system of rock mass quality was developed in Norway in 1974 by Barton, Lien, and Lunde of the Norwegian Geotechnical Institute. The Q system is based on a numerical assesment of the rock mass quality using six different parameters.

a. Rock quality designation (RQD)

b. Number of joint sets

c. Roughness of the most unfavarable joint or discontinuity

d. Degree of alteration of filling along the weakest joint

e. Water inflow

f. Stress conditions

The above six parameters are grouped into three quatients to give the overall rock mass quality as follows:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (2.2)$$

where

RQD = rock quality designation

J_n = joint set number

J_r = joint roughness number

J_a = joint alteration number

J_N = joint water reduction number

SRF = stress reduction factor

These six parameters can be reduced to only three parameters, each of which has a significant role on the stability of underground openings. There are:

1. block size (RQD/J_n)
2. interblock shear strength (J_r/J_a)- the friction angle of joints $\phi \approx \tan^{-1}$ (J_r/J_a); and
3. active stresses (J_w/SRF)

The following explanations and terminology for Q classification system are relavent.

1. Rock Quality Designation

When borecore is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay-free rock masses (Palmstrom,1974),

$$RQD = 115 - 33\ J_v\ \text{(approx.)}$$

where

J_v = Total number of joints per m^3

(RQD = 100 for $J_v < 4.5$)

But in this study RQD calculated as Deere's definition(1966).

2. Joint Set Number (J_s)

The parameter J_n representing the number of joint sets which will often be affected by foliation / schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in bore core due to these features, then it will be more appropriate to count them as "rondom joints" when evaluation J_n.

3. Joint Roughness Number (J_r)

Roughness or the nature of the asperities in the discontinuity surfaces is an important parameter characterizing the condition of discontinuities.

4. Joint Alteration Number (J_a)

This parameter represents the frictional characteristics of the joint walls or filling materials.

5. Joint Water Reduction Factor (J_w)

The parameter J_w is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stresses. Water may, in addition, cause softening and possible outwash in the case of clay-filled joints.

6. Stress Reduction Factor (SRF)

When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated
In such cases the strength of the intact
rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will than depend on the ratio rock-stress/rock-strength.

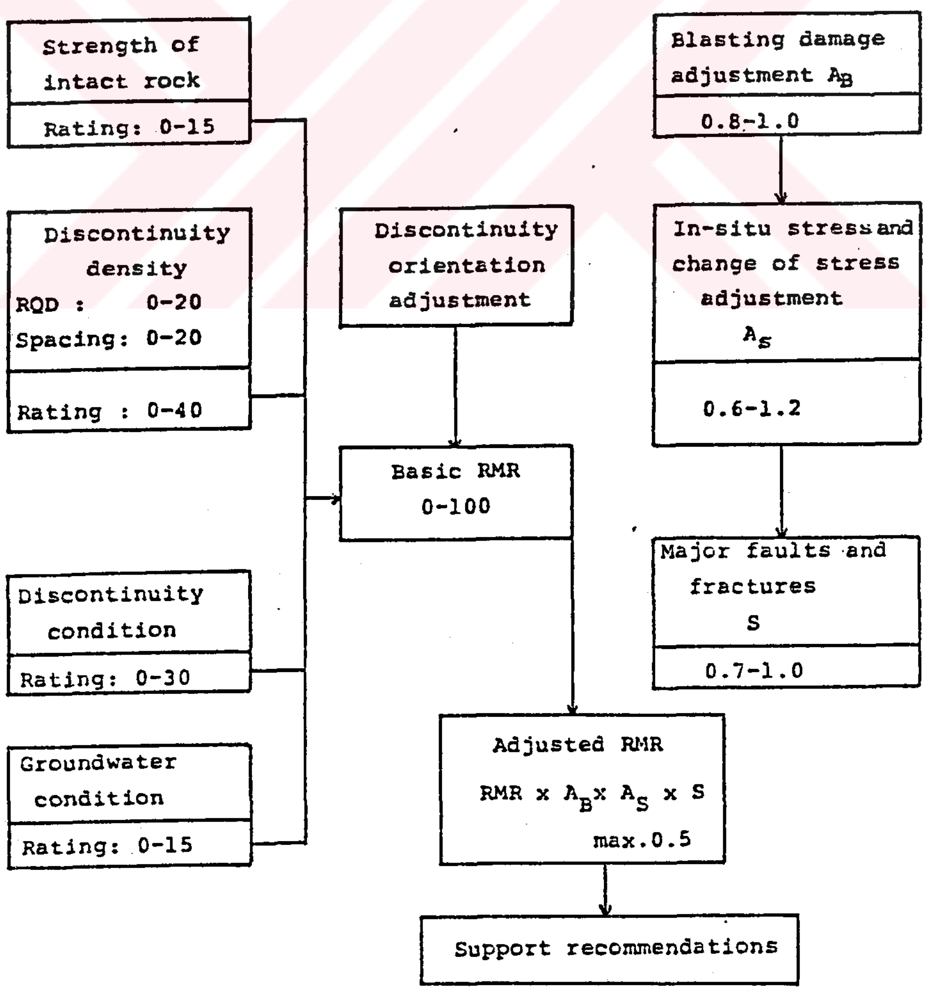
The possible Q values range between 0.001 (heavy squeezing ground) and 1000 (sound, unjointed rock) which covers nine rock classes of different intermediate ranges. The Q is related to tunnel support requirements by the equivalent dimensions of the excavation which is a function of both the size and purpose of the excavation. The index The index Q and the equivalent dimensions are related to 38 support categories. Appropriate permanent support, including rock bolt shotcrete and wire mesh is chosen by using support categories given (Barton et.al.,1974).

2.5. New Developments Associated with RMR and Q Systems

Engineering rock mass classifications have always been regarded as developing techniques. The Geomechanics Classification has been used extensively in mining, particularly in the United States. Initially, Laubscher and Taylor (1976) applied the Geomechanics Classification in asbestos mines in Africa specificaly to assess cavability of ore, While Ferguson (1979) extended this classification to mining tunnels and haulages. Since mining is a dynamic process, additional adjustments to the classification parameters were introduced, such as in-situ stresses, as shown in Table 2.2. Most recenty, the Geomechanics Classification was applied to coal mining in the United States (Bieniawski et.al.,1980, Newman,1981, Unal,1983) and in India (Ghose and Raju,1981) as well as to hard rock mining in the USA (Cummings et.al.,1982, Kendorski et al., 1983).

One of the objectives of this thesis is to modify the RMR and Q systems if and when necessary for better defining the weak rock masses. For this purpose, a number of parameters utilized by various researches have been reviewed in literature.

Table 2.2. Adjustments to the Geomechanics Classification-Mining Applications (after Bieniawski,1979)



2.5.1. Weathering

Laubscher and Taylor (1976) adjusted the basic ratings of the Geomechanics Classification to account for weathering, field and induced stresses, stress changes due to mining the orientation and type of excavation with respect to geological structure and effects of blasting. This was done for applications to hard rock mining. They initially proposed the adjustments to ratings by deducting or adding a value from an in-situ rating depending upon the conditions under which rock mass is encountered.. Later, they suggested the adjusted ratings to be a percentage of the basic rating (Laubscher and Taylor,1976; Fergusson,1979) Certain rock types wheather readily, which must be taken into consideration when decisions are made on the permanent support. The three properties that are affected by weathering are RQD, IRS (in-situ rock strength) and joint condition. The RQD percentage can be decreased by an increase in fractures as the rock weathers and the volume increases, an adjustment to 95% being possible. The IRS will decrease slightly if wheathering takes place along microstructures, a decrease to 96% being possible for the bulk of the rock. The main influence of weathering is on the condition of the joints, by alteration of the joint wallrock and/or the joint filling, an adjustment to 82% being possible. A total adjustment to 75% is possible and with certain rock types this figure could be even lower.

Weathering has a more dramatic effect on the strength of rock, which must be taken into account depending on the depth of the excavation and the effects of intrusion, foulting etc. The graph in Figure 2.1. can be used to estimate the corresponding strength reduction from a descriptive basis. Figure 2.1. have been recommended by Robertson(1970), and Worotnick and Derham(1970).

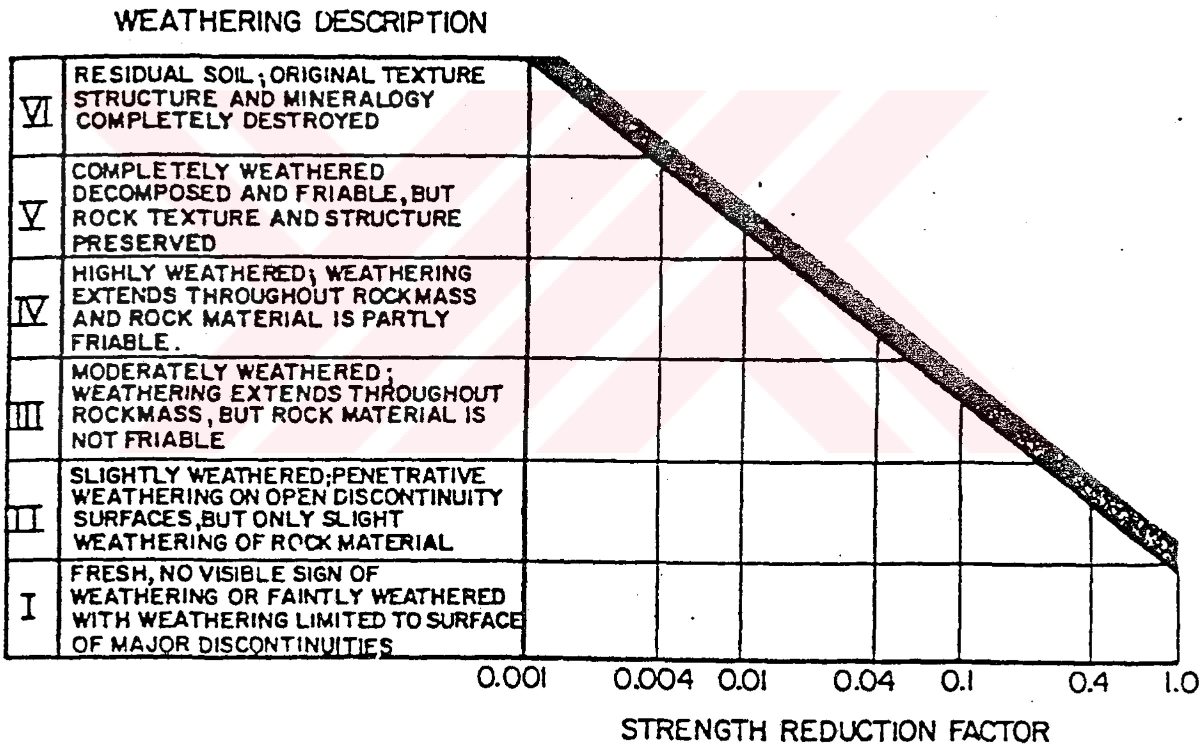


Figure 2.1 Strength reduction as a function of weathering. (after Rebortson,1970.)

2.5.2. Design Rock Mass Strength

Design rock mass strength (DRMS) have been calculated in the steps by Laubscher as follows :

a. Intact rock strength (IRS) is the uniaxial compressive strength of core specimens selected from solid core between joints or fractures. The IRS of a defined zone can be affected by the presence of strong and weak intact rock, which can occur in bedded deposits, deposits with varying mineralization and deposits which alteration bands, such as serpentinized dunites. An average value is assigned to the zone on the basis that the weak intact rock will have a greater influence on the average value. The relation is non-linear and the values can be read off on emprical chart presented in Figure 2.2. The rock mass strength cannot be higher than is intact rock strength.

Example

Strong rock IRS = 100 MPa

Weak rock IRS = 20 MPa

$$\frac{\text{Weak rock IRS}}{\text{Strong rock IRS}} \times 100 = 20\%$$

% Weak rock = 40 %

Average IRS = 48%

100 MPa IRS = 48 MPa

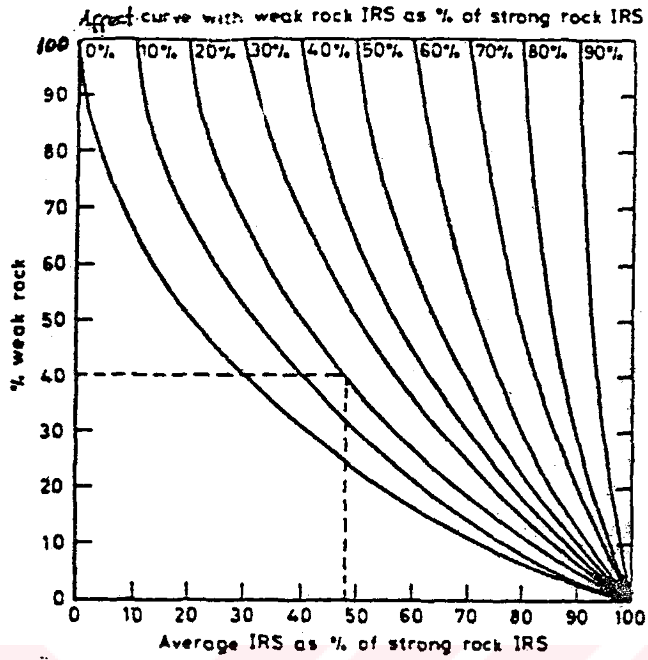


Figure 2.2. Determination of average IRS where rock mass contains weak and strong zones. (after, Laubscher,1976)

b. The procedure that is adopted to calculate the in-situ rock mass strength is as follows:

1) The IRS rating, B, is subtracted from the total rating, A, and therefore, the balance (RQD, joint spacing and joint condition Fig.2.3) will be a function of the remaining possible rating of 80. For example, a dunite with a rating of 60 and an IRS rating of 10 will have a reduction factor of 50/80, i.e. 0.625.

2) The IRS rating represents the strength, measured in mega pascals of the intact rock specimen, C , and this must be reduced to 80% of its value. The dunite with a rating of 10 has an IRS value of 100 and, therefore, the corrected strength is 80 MPa.

3) The corrected IRS value is multiplied by the reduction factor to give the in-situ RMS.

$$\frac{A-B}{80} \times C \times \frac{80}{100} = RMS$$

For the dunite example

$$\frac{60-10}{80} \times 100 \quad \frac{80}{100} = 50 \text{ MPa.}$$

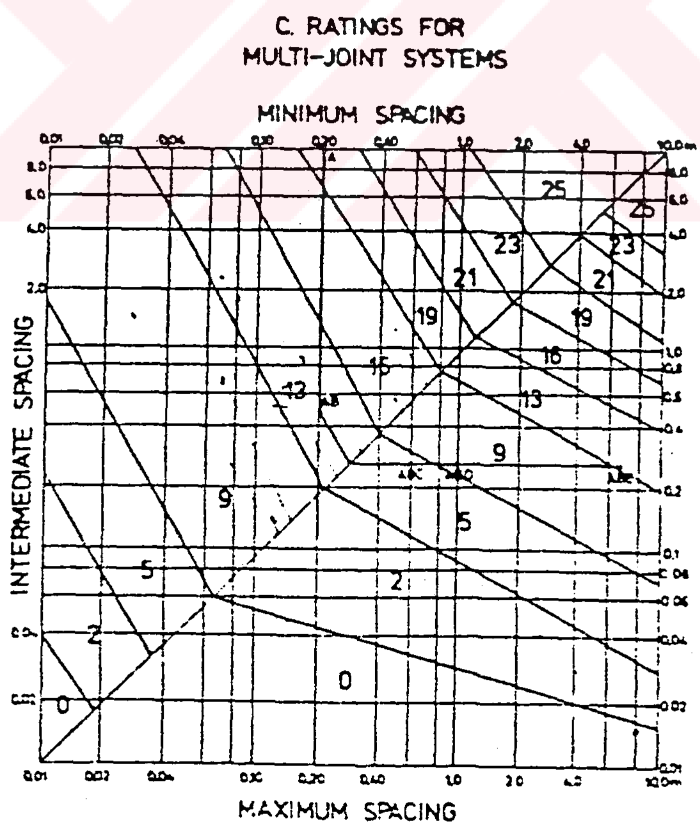


Figure 2.3. Geomechanics classification of jointed rock masses. (after Laubscher,1976)

c. The design rock mass strength (DRMS) is the unconfined rock mass strength in a specific mining environment. A mining operation exposes the rock surface and concern is with the stability of the zone that surrounds the excavation. The extent of this zone depends on the size of the excavation and, except with mass failure, instability propagates from the rock surface. The size of the rock block will generally define the first zone of instability. A combined adjustment for the effect of weathering the size and orientation of the rock surface with respect to points and the effect of blasting is applied to the RMS. For example

(Weathering = 90%)x(Joint Orientation = 80%)x (Blasting = 90%) = 65%

RMS = 50 MPa.

therefore

DRMS = 50x0.65 = 32.5 MPa

Whereas the percentage is emprical, the adjustment principle has been proved sound and, as such, forces the designer to take into account these important factors.

2.5.3. Influence of Strike and Dip Orientations

The size, shape and orientation of the excavation will affect the behaviour of the rock mass (laubscher,1976). For the purposes of support the classification rating should be reduced if the excavation is orientated in an unfavourable direction with respect to geological structures-particularly the weakest sets. The attitude of the joints and whether or not the bases of rock blocks are exposed have a significant bearing on the stability of the excavation. A block can be defined with the excavation surface as the base and varioeus. Intersecting joint combinations. If the orientation, size and shape of the excavation are such that block of ground will be exposed, on adjustment must be made. The magnitude of the adjustments depends on the attitude of the joints with respect to the vertical axis of the block, as gravity is the most significant force to be considered. During the development stage, the instability of the block depends on the number of joints that are incedined away from the vertical axis: the adjustments are shown in Table.2.3 .

Table 2.3. Orientation Adjustments for Blocks With Exposed Based (after Laubscher, 1976)

Number of joints	Number of faces inclined away from vertical and adjustment percentage				
	70%	75%	80%	85%	90%
3	3		2		
4	4	3		2	
5	5	4	3	2	1
6	6		4	3	2,1

The orientation of joints has a bearing on the covability of undercut rock masses. The same principles apply to the back of an undercut and adjustments should be made. The adjustments for the orientation of shear zones with development are for0-15^{o}, 76%; 15-45^{o}, 84% and 45-75^{o}, 92%.

Advance of the ends in the direction of the dip of the joints is preferable to development against the dip.

2.5.4. RQD and Spacing of Discontinuties

The Rock Quality Designation(RQD) is defined as the proportion of borehole core consisting of intact lengths equal to or longer than 4 inches (10 cm). To obtain the RQD value of a given rock it is not always possible to conduct a core drilling program. It is neither possible to drill along a rock surface. To determine the RQD of rock surfaces a tape is suspended along the surface and an assesment of the true fracture spacing is obtained. This then may be regarded as directly analogous to a borehole core.

Priest and Hudson(1976) presented a theorectical approach to continuty spacing and RQD based on the statistical distributions of spacing values that could distributions of spacing values that could occur along scanlines. The discontinuity spacings were considered in reference to the distances between points where discontinuities intersect a straight line through the rock mass. They assumed the discontinuity intersection points could be evenly spaced, clustered, random or some combination of these. The most probable type of discontinuty spacing distribution was proved to be the negative exponential discontinuity. They verified the validity of this assumption from the available experimental data. They found a linear relationship between the possible RQD value (RQD min) and the discontinuity frequencies as follows:

$$RQD_{min} = 100(1-0.1\lambda + 0.1/L) \qquad (2.3)$$

where:

L is the scanline length in meters and λ is the mean number of discontinuities permeter. If the scanline is long enough (compared to 0.1 m)

$$RQD_{min} = 100(1-0.1\lambda) \qquad (2.4)$$

They also found the following relationship between the theoretical maximum RQD,(RQD^*), and discontinuities frequency (λ), when the discontinuities spacing follow a negative exponential distribution:

$$RQD^* = 100\ e^{-0.1\lambda}(0.1\lambda + 1) \qquad (2.5)$$

On the other hand, the Geomechanics classification or rock masses regards six parameters of importance, including both RQD and discontinuity spacing. Since these two parameters are related to each other, one can put them together and assing a rating value for the combined effect. According to Priest and Hudson (1976), it is actually impossible to have an RQD less than the value predicted by Equation (2. 3). Thus, putting the two parameters together, gives the opportunity to check the obtained results from the field, and also to allocate to them a single classification rating value. Figure 2.4 a shows the maximum and minimum RQD values and RQD versus discontinuity spacing for randomly positioned discontinuities. The area between to possible values, is divided into sub-areas; single rating values are attributed to the conditions which are the direct summation of corresponding RQD and spacing rating values.

If it is found that the mean discontinuity spacing for a rock mass, for example, is 200 mm and the RQD is reported to be 35 %, Figure 2. 4 a reveals that for such a spacing, the minimum corresponding RQD is about 60. Therefore, these two reported values are not in agreement and these data should be cross checked.

If there is a large difference between the direct an indirect RQD values, the following points should be considered.

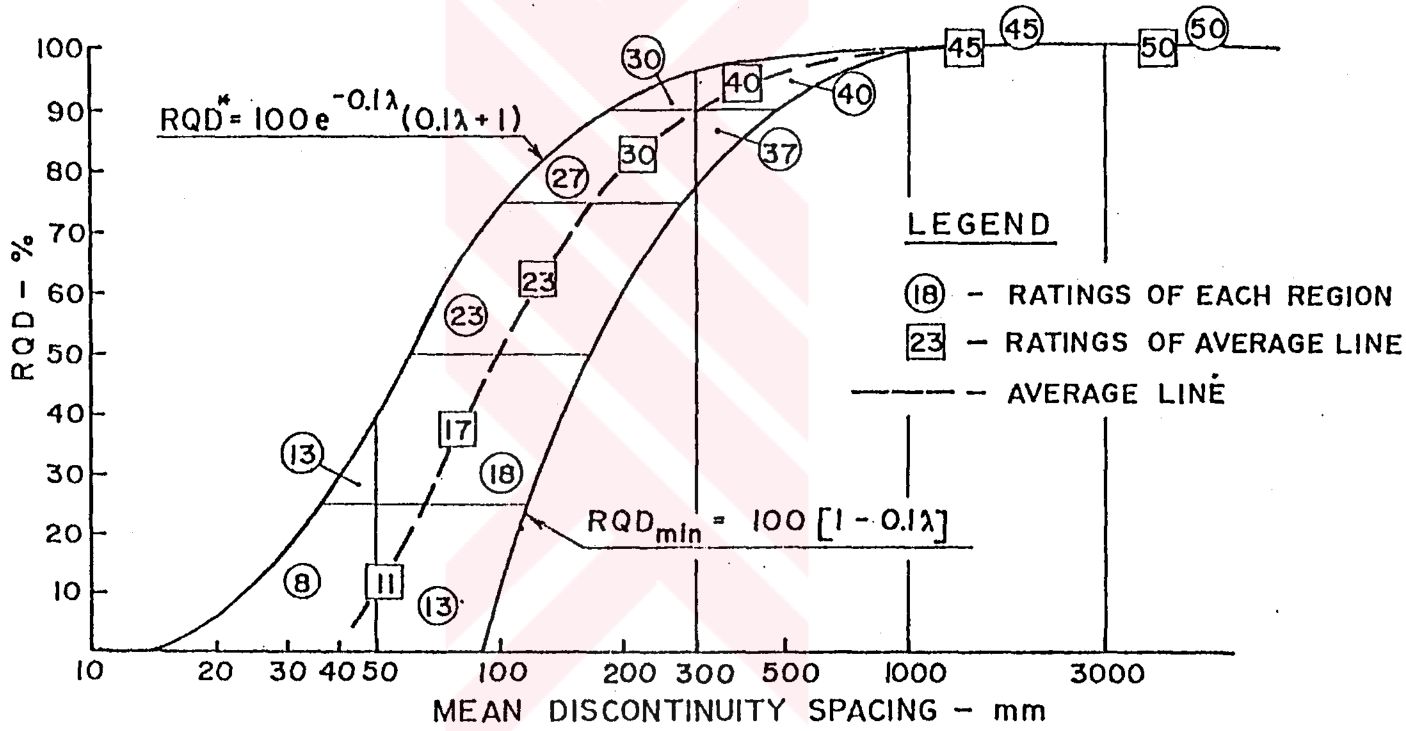


Figure 2.4.a. Maximum and minimum possible RQD values: RQD vs. Mean discontinuity spacing for randomly positioned discont inuities, and corresponding ratings (Bieniawski,1976) for different values of RQD and spacing.

- quality of the drilled cores directly related to the technique used and the degree of the operator's skill. Experience in determining the RQD of core is necessary before attempting indirect RQD assessment;
- fractures resulting from blasting may be misleading;
- some planes of weakness such as bedding planes do not cause a seperation when cored.

Figure 2.4 b has been provided by changing the values in accordance with the new rating system of the Geomechanics Classification (Bieniawski,1979). The dotted lines in Figures 2.4 a and 2.4 b are the average data with a single value. The usefullness of these lines is that a proper rating (given in squares) can be allocated to RQD and/or discontinuity spacing when only one of these two parameters of importance are known.

Figure 2.5 shows a graph of equation (2.5) relating RQD^* to λ together with the experimental data points presented earlier. Between values of $\lambda = 6/m$ and $\lambda = 16/m$ the relation between λ and RQD^* is approximately linear. The curve in Figure 2.5 has an inflection point P at $\lambda = 10/m$; at this point the curve has a slope equal to $dRQD^*/d\lambda/_{\lambda=10} = -10\ e^{-1} = -3.68$ m. The straight line drawn through P, tangential to the curve defined by Equation 2.5 is therefore described by the relationship shown in Equation 2.6.

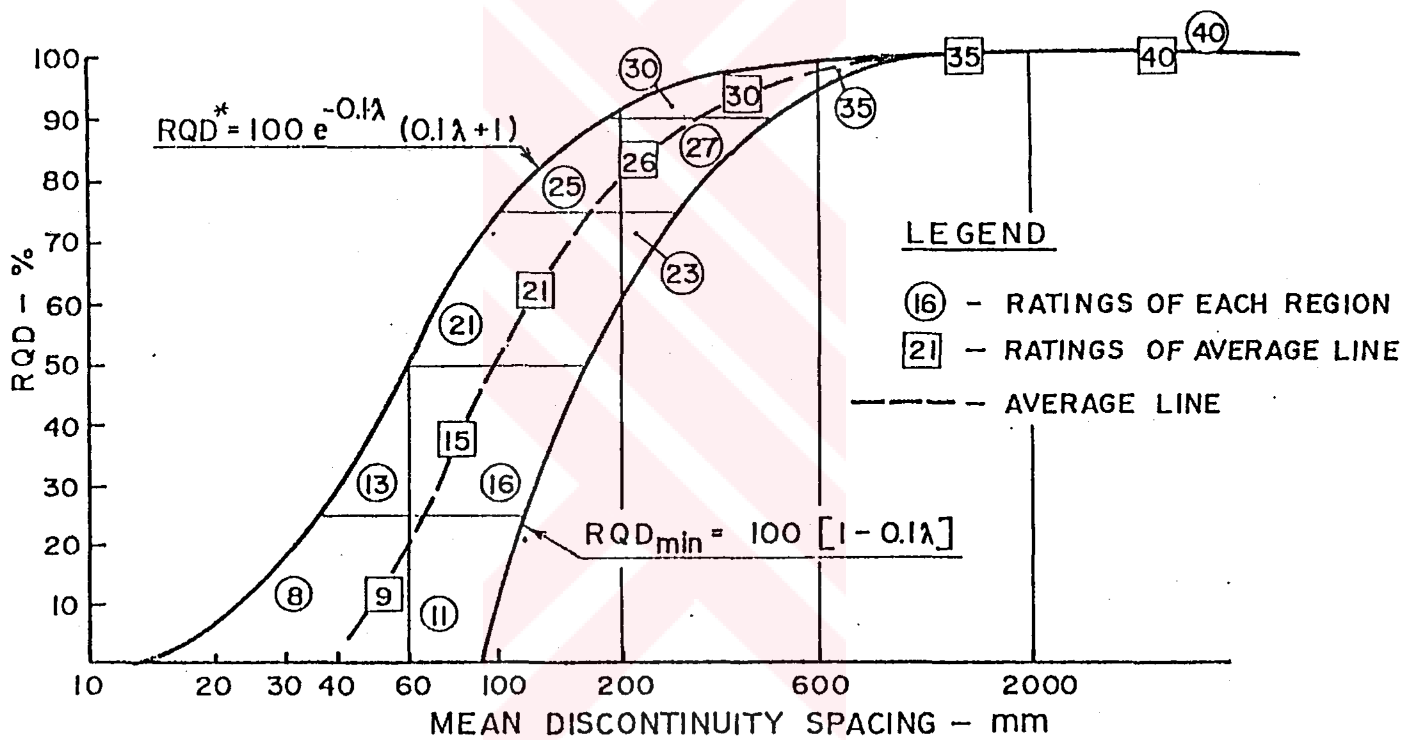


Figure 2.4b. Maximum and minimum possible RQD values: RQD vs. Mean discontinuity spacing for randomly positioned discontinuities, and corresponding ratings (Bieniawski, 1979A) for different values of RQD and spacing.

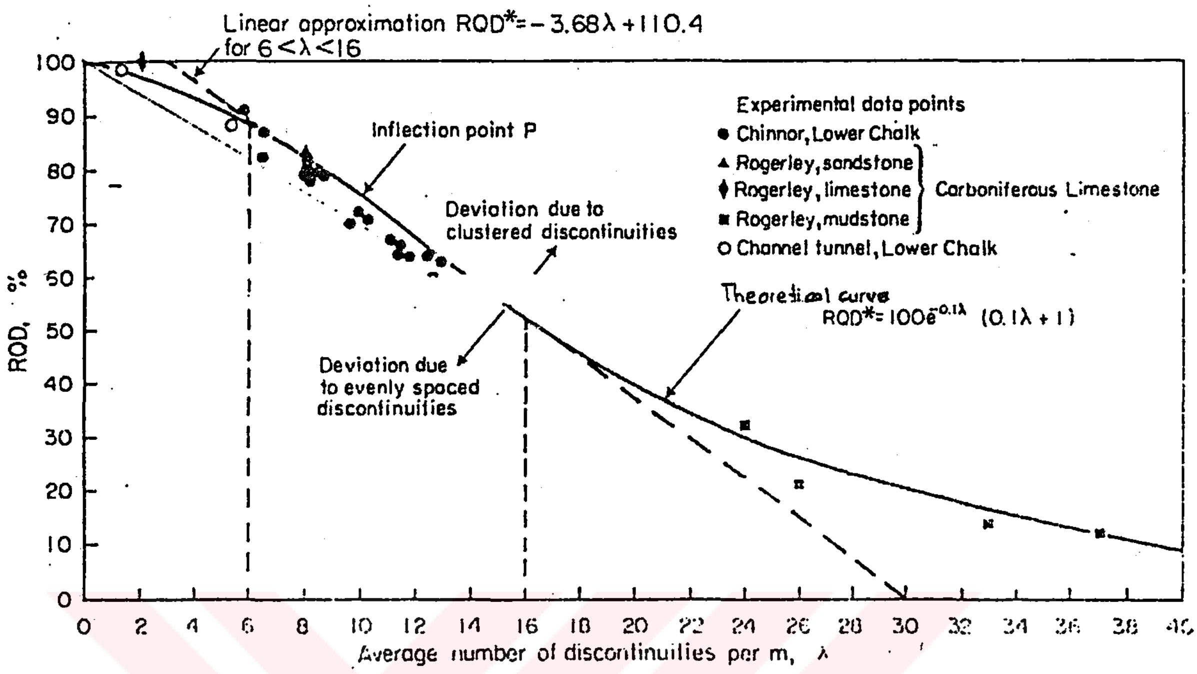


Figure 2.5. Relation between RQD and mean discontinuity. (after, Priest and Hudson ,1976)

$$RQD^{*} = -3.68\lambda + 110.4 \quad (2.6)$$

This line, shown in Figure 2.8, gives a good approximation of RQD between values of λ=6/m and λ= 16/m. Bieniawski (1973), by reference to Deere et al.,(1967) suggested that the relation between "fracture frequency" and RQD is linear; this statement is generally in agreement with Figure 2.5 within the stated range of λ .

A new method for obtaining data associated with classification parameters have been developed by Clarke and

Budavari (1981) based on in-situ observations. According to this method, the in-situ data is processed as follows:

(i) The number of joint sets were estimated with the aid of a stereographic projection;

(ii) The field survey data was then grouped into the respective sets as estimated in step (1);

(iii) The intersection angles between the survey line and the strike direction of each joint set were determined and the true joint spacings calculated after correcting for orientational bias(Terzaghi, 1965). The RQD values were also determined;

(iv) The influence of each joint set on the tunnel stability was estimated;

(v-viii) Data forms and calculation sheets were complied for each classification system, as for the borecore data.

In terms of RMR and Q classification systems, the standard data obtaining method and in-situ observation method have been used by Clarke and Budavari(1981) in Du Toitskloof, Tunnel, Delvers, Tunnel, and Bushkoppies Tunnel in South Africa. The results obtained from these observations have been summarized in the following paragraphs.

When the borecore and in-situ classification values are compared, it was concluded that both classification systems (RMR and Q),tend to indicate poorer rock mass conditions than in situ measurements.

The tendency for lower borecore than in-situ classification values is opposite to that indicate by results presented by Barton(1976) from a similar type of investigation Barton's borecore Q values were about twicey his in-situ values. This discrepancy appears to be related to the different rock conditions examined in each case. The present results are based on measurement from a variety of geological enviroments which included jointed rocks predominantly, although massive rocks were also examined. Barton's data were obtained from "quite massive biotite gneiss" only. This suggests that the relationship between the borecore and in-situ classification values may be linked to the rock mass condition, with lower borecore than in-situ values being associated with jointed rocks, and the opposite being the case in more massive varieties.

With regard to site investigations for underground excavations in rock, both the Geomechanics and Q classification Systems can be most useful. Their value must be seen in perspective and their limitations always recognized. Their best application would seem to be towards providing a general picture of the anticipated rock conditions, and an initial assessment of the likely support requirements in a planned underground excavation. However, the data should never be regarded as the final result in this respect. Since bore core results are generally expected to be directly applicable to the rock mass in the immediate vicinity of the borehole only, great care is necessary when attempting to extrapolate between boreholes. When possible, other investigatory techniques should also be used.

CHAPTER III

DATA COLLECTION AND PRESENTATION FOR CLASSIFICATION FROM BIGADIÇ - SIMAV MINE

3.1. General

In Bigadiç Borax Mines a series of Rock Mechanics studies were carried out by METU, Mining Engineering Department. These studies include the following (Paşamehmetoğlu, Ünal and Tutluoğlu,1986; Özel,1988; Gökay,1988).

(i) identification of geological formations,

(ii) underground and surface drilling studies,

(iii) investigation of laboratory tests,

(iv) classification of rock masses encountered in the area.

The investigation carried out and the analyses made in this thesis are based on the data obtained from the above mentioned studies.

3.2. Geographyand Description of the Mine

Etibank - Bigadiç Colemanite Mines are about 40 km from Balikesir and located 11 km North of Bigadiç. Balıkesir-

Bigadiç-İzmir highway is the main road in the region. Also the road between the mine and Bigadiç is in good condition in all seasons. The existence of large boron mineral reserves increases importance of the region in Turkey and also in the world. Simav mine is one of the largest underground mine in this region in terms of its annual production capacity.

3.3. Geology of the Bigadiç Region

At the Bigadiç region the mesozoic ophiolitic complex constitute the basemend. These basemend rocks are overlain unconformably by the volcanics and lacustrine volcano-sedimentary sequence of Neogene age.

In the basin Gündoğdu (1982; in Gündoğdu and Gökçen; 1983) distinguished three major lithological units starting from the bottom these include the metalimestones and metasandstones of the ophiolitie complex; based volcanics comprizing lower Miocene andesites and basalts, and lacustrine volcanosedimentary sequence of middle Miocene-Early Pliocene, The latter is referred to as the Bigadiç formations.

Within the Bigadiç formation Gündoğdu (1982; in Gündoğdu and Gökçen, 1983) distinguished Avşarbaşı, Değirmenli, Uzuntepe, Emirler and İskele members. The Avşarbaşı member overlie the Bazal volcanics uncorformably and consists of dolomicrites and siltstones. The Değirmenli member represents the second volcanic actwity of the region, it comprises continential and lacustrine deposits consisting of dacided-rhyodacidic tuffs and zeolites. The Uzuntepe member is largely imposed of and alternation of argillaceous and carbonaceons rocks. These rocks include the borate deposits The Emirler member represents the third stage volcanic activity of the region. These include almost 200m thick glassy tuffs. The İskele member is imposed of basalt bearing argillaceous and carbonaceous sequence.

3.4. Drilling Investigations

Drilling program carried out in the area included both surface and underground borehole drillings. The aim of the surface drilling was to identify the rock units that exist in Simav Mine Region and to determine the mechanical and physical properties of the rocks. In addition to that of surface boreholes a number of boreholes were also drilled in underground. The purpose of these drillings was to obtain in fact and representative cores from the clayey and laminated rock units existing between the ore-seams and weak rock mass.

3.4.1. Surface Boreholes

To obtain specimens for the laboratory tests and for rock-mass classification purposes a total of five borehole drillings were carried out in Simav-Mine Region. The boreholes were named as KM-1, KM-2, KM-3, KM-4 and KM-5.

3.4.2. Underground boreholes

In underground three boreholes, namely, YS-1, YS-2 and YS-3 were drilled.

In general the formations lying between the ore-seams mainly consist of limestone and marl which contains thin clay and tuff bands in varying thicknesses. The thicknesses of the seams and the interlayers are presented in Table 3.1.

Table 3.1. Thickness of the Seams and the Interlayers in Simav Mine

Formation	Thickness (m) Minimum	Maximum	Average
First seam (colemanite)	2.0	5.75	3.80
Between first and second seam	2.0	5.50	3.45
Second seam(Colemanite)	3.0	7.00	5.60
Between second and third seam	2.2	5.70	4.10
Third seam (Colemanite+ Ulexite	4.8	9.0	7.30
Between third and fourth seam	3.85	6.50	5.00
Fourth seam	3.50	6.50	5.00

3.5. Laboratory Investigation

A considerable number of laboratory tests were carried out on specimens, representing different formations including the ore zones. The specimens for laboratory tests were prepared by utilizing the cores obtained from various drill-holes (e.g. KM-1, 2, 3, 4, 5 and VS-1 and 2). A number of block obtained from different formations were also utilized for specimen preparation. The specimen preparation was carried out in the Rock Mechanics Laboratory of the Mining Engineering Department at M.E.T.U. in accordance with the standards suggested by the International Society of Rock Mechanics.

The main rock types encountered at Simav are shown in Table 3.2. For better identification of the rock formations and for easier communication the rock units were devided into three main groups based on their (i) physical appearance (ii) types and (iii) condition of containing clay or having incorporating other type of rocks. Details associated with this presentation are shown in Tables 3.3, 3.4 and 3.5.

The following laboratory tests were carried out:

1. Uniaxial Compressive Strength (σ_c)
2. Point - load Index ($I_{S(50)}$)
3. Slake - Durability Index (I_d)
4. Deformability (E,ν)
5. Shear Strength (τ)
6. Triaxial compression(σ_1, σ_3)
7. Indirect Tensile Strength (σ_t)
8. Density and Porosity
9. Clay analyses.

The results of the first six tests that were utilized in this thesis.

Table 3.2. The Main Rock Types at Simav-Mine Region

Rock types Cod. No:	The names of rock types
10a	Colemanite
10b	Ulexite
8	Limestone
4/8	Limestone alternating with claystone
5/8	Limeston with clay laminae
9a	Weathered limestone-with-claystone
9c	Highly - Weathered limestone
7	Tuffite
11	Tuff
12	Upper tuff
6a	Siltstone
13b	Claystone
14	Marl

In general the specimens were prepared in accordance with the methods suggested by the International Society for Rock Mechanics (ISRM, 1979A). The specimens were prepared from the cores obtained in the field. NX (54 mm) cores were used for uniaxial compressive strength

Table 3.3. Presentation of Rock Types According to Their Physical Appearence

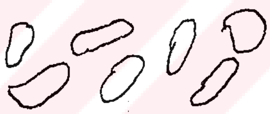
ROCK DESCRIPTION	APPEARANCE	DESCRIPTION CODE
Ground		0
Pebbly		1
Broken Cores		2
Broken and Weathered Cores		3
Laminated Cores		4
Thinly Laminated Cores		5

Table 3.4. Presentation of Rocks According to Their Types

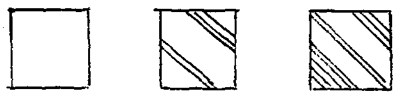
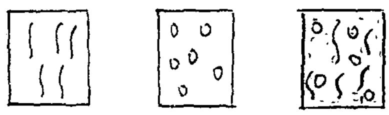
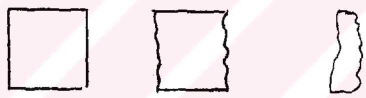
ROCK TYPE DESCRIPTION	VIEW	DESCRIPTION CODE
SILTSTONE a.Hard b.Sliced		6
TUFFITE		7
LIMESTONE		8
WEATHERED LIMESTONE a.Hard b.Porous c.Clay, Pebble and sand		9
CLAYSTONE a.Claystone b.Hard Clay c.Soft Clay		13

Table 3.5. Presentation of Rock Types According to Their Clay Content

ROCK TYPE DESCRIPTION	DESCRIPTION CODE
Clayey Colemanite	KK
Clayey Ulexite	KU
Limestone alternating with Claystone	4/8
Limestone alternating with Claystone interbedded with Tuff	4/8 $\overset{=}{\underset{=}{11}}$
Limestone alternating with Claystone interleveled with Tuff	4/8 ″11,
Weathered Limestone interbedded with Claystone	9 $\overset{=}{\underset{=}{13}}$
Limestone with Clay laminea and Claystone	- 5/8 - 13 -
Limestone-Laminated Marl	- 8 - 14 -

tests. In general, bedding planes are dipping $0^{\circ}-30^{\circ}$ with respect to the horizontal axis (Figure 3.1a)

A series of specimens were also taken from the blocks obtained in the field. The direction of coring, however, in this case was parallel to the bedding planes (Figure 3.1 b)

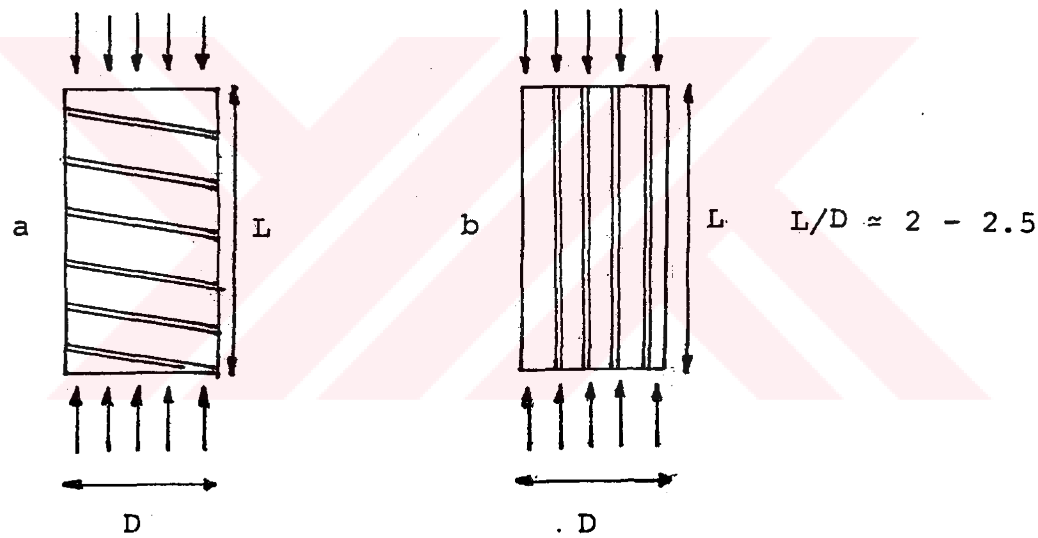


Figure 3.1. Orientation of bedding planes with respect to loading direction.

(a) in rock specimens prepaired from cores.

(b) in rock specimens prepaired from blocks.

The results of compressive strength tests are presented in Table 3.6. and 3.7.

Table 3.6. The Results of Uniaxial Compressive Strength Test Carried Out on Specimens Obtained from Drill Holes. (Paşamehmetoğlu, Ünal and Tutluoğlu,1986)

Rock Type	Number of Specimen	Uniaxial Compressive Strength (MPa)
(4/8) Limestone alternating with claystone	44	19.43 ± 7.49
(10a) Colemanite, clay bearing	8	7.82 ± 2.32
(10a) Colemanite	30	17.33 ± 10.38
(10b) Ulexite, clay bearing	6	5.16 ± 2.24
(10b) Ulexite	35	14.78 ± 6.31
(11) Tuff	20	30.85 ± 8.03
(7) Tuffite	18	7.13 ± 2.30
(12) Upper tuff	5	31.60 ± 7.64
(9abc) Weathered Limestone	41	5.36 ± 3.30
(8) Limestone	13	20.31 ± 5.34
(5/8) Limestone with clay laminae	5	17.12 ± 1.52

Table 3.7. The Results of Uniaxial Compressive Strength Test Carried Out on Specimen Obtained from Rock Blocks.(Paşamehmetoğlu, Ünal, Tutluoğlu, 1986)

Rock Type	Location	Number of specimen	Uniaxial Compressive Strength (MPa)
(4/8) Limestone alternating with claystone	Below the fourth seam	10	*12.26 ± 1.5
(10a) Colemanite Mossive clay bearing	From first and third seam	13 5 2	20.5 ± 12.1 *42.254 ± 7.64 *19.323
(10b) Ulexite	From third seam	10 4	17.5 ± 3.7 *13.99 ± 2.53
(11) Tuff	Below the first seam	15	*20.35 ± 2.42
(9abc) Weathered Limestone	Above the fourth seam	4 5	10 ± 3.6 *9.47 ± 0.53
(8) Limestone	Between the first and second seam	2	*38.787 - 57.299 (1)
(5/8) Limestone with clay laminae	Between the second and third seam	5	*11.89 ± 1.12

* Loading parallel to the bedding

(1) The result of two tests

The specimens which were too short to carry out uniaxial compressive strength tests were utilized for point - load strength index tests. The apparatus used has maximum pressure capacitiy of 54 kN. The point-load index tests were also carried out by following the methods suggested by International Society for Rock Mechanics (ISRM, 1972). The point-load strength index is calculated as the ratio of the applied load to the square of core diameter.

In carrying out point-load tests both the diametral (I_D) and the axial (I_A) tests were performed. In these tests the rock isotropy in horizontal and vertical directions were considered. The position of the bedding planes with respect to the direction of loading in axial and diametral tests are shown in Figures 3.2 and 3.3.

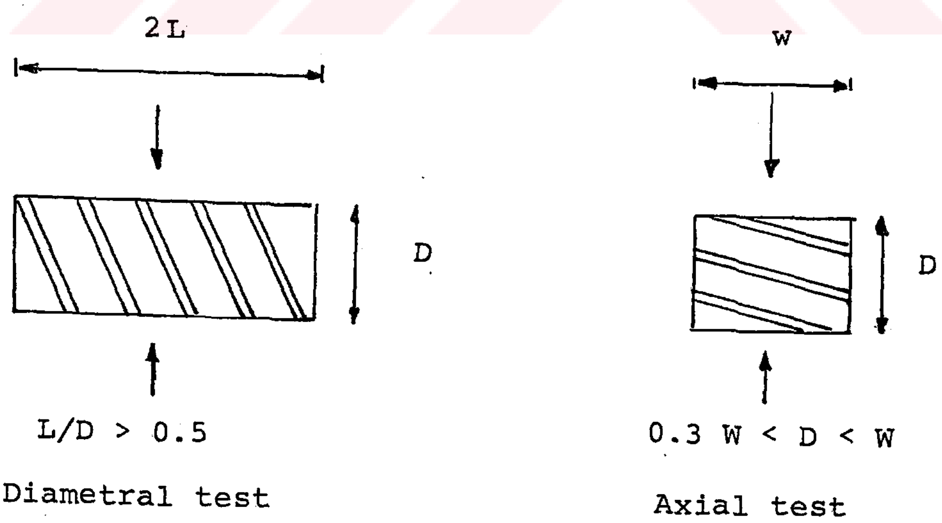


Figure 3.2. The position of bedding planes with respect to direction of loading in axial and diametral tests.

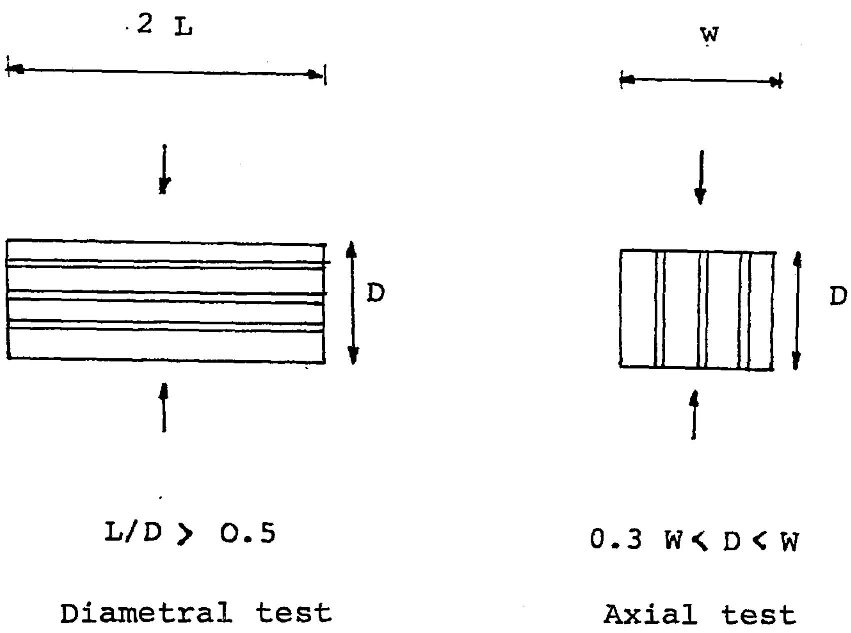


Figure 3.3. The position of bedding planes with respect to direction of loading in axial and diametral tests.

The results of point-load strength test are presented in Tables 3.8.

Shale and claystone,know to be affected by changing moisture conditions (Van Eechout and Peng, 1975), require testing for slake-durability to assess their weatherability A slake-durability index (Franklin and Chandia,1972) basically indicates the resistance of a material to deterioration when exposed to short term cycles of wetting and drying. This usually occurs in underground mines during rock exposure to atmosphere.

Table 3.8. The Results of Point-Load Strength Tests Carried out on Specimen Obtained from Drill Holes Rock Blocks

Rock Type	Number of Specimens	Diametrial Point Load Index (MPa)	Number of Specimens	Axial Point Load Index (MPa)
(10a) Colemanite	113 15	0.71 0.9*	75 17	1.15 0.99*
(10b) Ulexite	30 9	0.46 0.42*	14 14	0.81 0.89*
(8) Limestone	21 4	0.85 1.84*	9 4	1.52 2.12*
(4/8) Limestone alternating with claystone	152 9	0.54 0.51*	92 9	1.67 0.88*
(5/8) Limestone with clay laminae	44 12	0.30 0.74*	40 10	1.08 0.77*
(9abc) Weathered limestone (weak, rough surface)	139 6	0.126 0.69*	128 11	0.31 0.97*
(7) Tuffite	38 -	0.29 -	20	0.50 -
(11) Tuff	84 12	0.40 1.202*	62 17	0.63 1.89*
(12) Upper tuff	17 -	0.57 -	7	1.57 -
(6a) Siltstone	27 -	0.17 -	14	0.47 -
(13b) Claystone	22 -	0.123 -	15	0.27 -
(14) Marl	14 -	0.118 -	2	0.26 -

* Loading parallel to the bedding

The slake-durability tests were also calculated in accordance with the ISRM suggested methods. The standard slake-durability index is calculated as follows:

$$\text{Slake-Durability Index(first cycle)}\, I_{d-1} = \frac{B-D}{A-D} \times 100 \qquad (3.1)$$

$$\text{Slake-Durability Index(Second cycle)}\, I_{d-2} = \frac{C-D}{A-D} \times 100 \qquad (3.2)$$

where,

A is the weight of drum plus sample before first cycle;

B is the dry weight of drum plus retained sample; after first cycle;

C is thedry weight of drum plus retained sample after second cycle;

D is the weigth of dry and clean drum only.

The results of slake-durability tests are summarized in Table 3.9.

In order to give the readers a better feeling about the physical and mechanical properties of the rock types utilized, the results obtained from a number of laboratory tests are also given in Tables 3.10, 3.11, 3.12, 3.13, 3.14, 3.15.

Table 3.9. The Results of Slake Durability Tests Carried out on Specimens from Drill Holes (Paşamehmetoğlu, Ünal, Tutluoğlu,1986)

Rock Type		Number of tests	First cycle(%)	Number of tests	Second cycle(%)
(10a)	Colemanite	6	89	6	79
(10b)	Ulexite	5	56	5	25
(8)	Limestone	5	85	5	79
(4/8)	Limestone alterating with claystone	5	97	5	94
(9abc)	Weathered Limestone (weak, rough surface)	6	81	6	75
(7)	Tuffite	5	58	5	44
(11)	Tuff	6	80	6	65
(12)	Upper Tuff	5	99	5	96
(13b)	Claystone	5	44	5	28

Table 3.10 The Results of Deformability Tests Carried out on Specimens Obtained from Drill Holes (Paşamehmetoğlu, Ünal, Tutluoğlu,1986)

Rock Type	Number of Specimen	Modulus of Elasticity (MPa)	Pouson Ratio
(4/8) Limestone alterating with claystone	19	9.37±6.4	0.26±0.10
(8) Colemanite	7	47.20±17.04	0.18±0.06
(10b) Ulexite	9	20.47±15.54	0.17±0.05
(9abc) Weathered Limestone (weak rough surface	8	26.49±3.98	0.21±0.04
(7) Tuffite	6	1.97±0.47	0.14±0.18
(12) Upper tuff	2	6.56	0.21
(11) Tuff	11	7.73±3.63	0.23±0.11

Tables 3.11 The Results of Deformability Tests Carred out on Specimen Obtained from Rock Blocks (Paşamehmetoğlu, Ünal, Tutluoğlu,1986)

Rock Type	Number of Specimen	Modulus of Elasticity (MPa)	Poisson Ratio
(4/8) Limestone alterating with claystone	3	*8.75±2.13	*0.30±0.14
(9abc) Weathered Limestone (weak, rough surface)	4	8.34±2.94	0.20±0.01

* Loading parallelto the bedding.

Table 3.12. The Shear Strength Test Results

Rock Type	Member of Specimen	Cohesion C kPa	Friction Angle ϕ_p, ϕ_r	Normal Stiffness (MPa/m) k_n	Normal Stiffness (MPa/m) k_s	Relationship Between $\tau - \sigma$ (kPa)
(5/8) Limestone with clay laminae (Thinly laminated)	6	40	ϕ_p=18.5^o	320	1254	$\tau >$ 25 + 0.40σ , σ<250 $\tau >$ 70 + 0.24σ , σ<250
(4/8) Limestone alternating with claystone	3	150 20	ϕ_p=36.5^o ϕ_r=20^o	2651	5700	with irregularities: $\tau >$ 160 + 0.75σ slikensided: Peak values; $\tau_p >$ 40+0.55σ; σ < 625 $\tau_p >$ 180+0.38σ; σ > 625 Residual values $\tau_r >$ 80 + 0.36σ

Table 3.13. Triaxial Test Results (Applied Confining Pressure Series 0,4.9, and 14.7 MPa)

Rock Type	Number of Specimen	Relationship Between $\sigma_1 - \sigma_3$ (MPa)
(10a) Colemanite	4	σ_1 = 10.65 ± 4.89 σ_3
(10b) Ulexite	4	σ_1 = 14.28 ± 3.13 σ_3
(4/8) Limestone alterating with claystone	4	σ_1 = 21.66 ± 2.36 σ_3

Table 3.14. Indirect (Brezilian) Tensile Strength Test Results.

Rock Type	Number of Specimen	Indirect (Brezilian) Tensile Strength (MPa), T_o
(10a) Colemanite	62	1.39 ± 0.89
(10b) Ulexite	16	0.79 ± 0.45
(4/8) Limestone alternating with claystone	15	2.69 ± 1.88
(8) Limestone	19	2.027 ± 0.96

Table 3.15. Density and Porosity Test Results.

Rock Type	Number of Specimen	Porosity (%)	Dry Density (g/cm^3)	Saturated Density (g/cm^3)
(10a) Colemanite	30	2.22±1.3	2.16±0.12	2.23±0.17
(10b) Ulexite	33	-	1.81±0.11	-
(4/8) Limestone alternating with claystone	48	10.7±0.99	1.84±0.02	1.94±0.03

3.6. Discussion of Laboratory Test Results

The results, listed in Tables 3.16 and 3.17 indicate that the excountered lithologies are characterized by changing uniaxial compressive strength values lower than 25 MPa Rock with high uniaxial compressive strength values are intermediate tuff (11) and Upper tuff(12), and low strength values are alterated limestone (9abc) and tuffit (below 8MPa). Accordingto the ISRM (1981), the strength classifications of rocks are shown Table 3.16.

Table 3.16. The Strength Classification of Simav-Mine Rocks (ISRM, 1981)

Rock Type	Uniaxial Compressive Strength (MPa)	Designation
(4/8) Limestone-alterating with claystone	19.43 ± 7.49	Low
(10a) Colemanite	17.33 ± 10.38	Low
(10b) Ulexite	14.78 ± 6.31	Low
(11) Tuff	30.85 ± 8.03	Moderate
(7) Tuffite	7.13 ± 2.30	Low
(12) Upper tuff	31.60 ± 7.64	Moderate
(9abc) Weathered limestone	5.36 ± 3.30	Low
(8) Limestone	20.31 ± 5.34	Low
(5/8) Limestone with clay laminae	17.12 ± 1.52	Low

The results of point-load strength tests indicate that the following relationship with a correlation coefficient of 0.94, exist between the compressive strength and point-load index:

$$\sigma_c = 16.57\ I_{s(50)} + 2.13 \tag{3.3}$$

In these tests, the direction of loading was perpendicular to the bedding planes in case of compressive strength tests. On the other hand, the direction of loading was parallel and perpendicular in diameteral and axial point-load test respectively.

In the other series of tests the specimens prepared for uniaxial-compressive-strength tests were loaded parallel to bedding planes. During diametral and axial point-load tests the direction of loading was perpendicular and parallel to the bedding planes respectively. After these tests the following relationship was found:

$$\sigma_c = 19.60\ I_{s(50)} - 1.5 \tag{3.4}$$

The relationships shown in Equations 3.3. and 3.4. was determined considering the tests carried out on the point load and uniaxial compressive strength.

The anisotropy index (I_a) of various rock material was also determined based on point-load index tests. According to the test results, claytone has the highest anisotropy index (I_a = 3.6) while, intermediate tuff has the lowest (I_a = 1.61).

The results of slake-durability index tests indicate that the claystone and ulexite has a "low" slake-durability index (less then 30%). While the upper-tuff and limestone-laminated-with-claystone have a "high" slake-durability index (over 95%).

Colemanite has a high modulus of elasticity (47.28 GPa). However, limestone-laminated-claystone has a low modulus of elasticity (9.37 GPa). Also Poisson's ratio , for colemanite and limestone alternating with claystone are 0.18 and 0.26. The relationships between the σ_c - E and E/ γ - σ_c are shown in Figure 3.4 and 3.5.

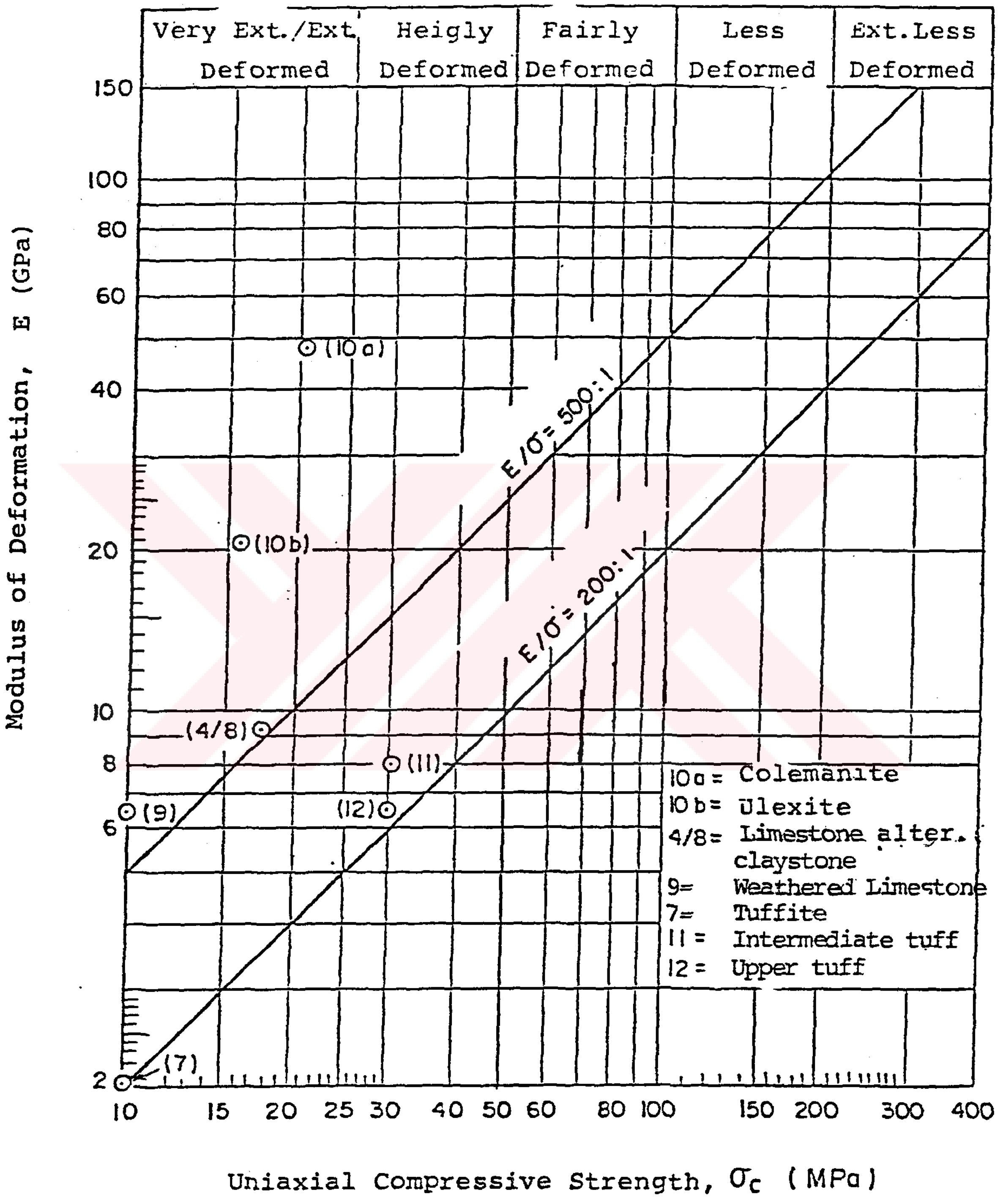


Figure 3.4. The Relationship Between the Modulus of Deformation (E) and Uniaxial Compressive Strength (σ_c).

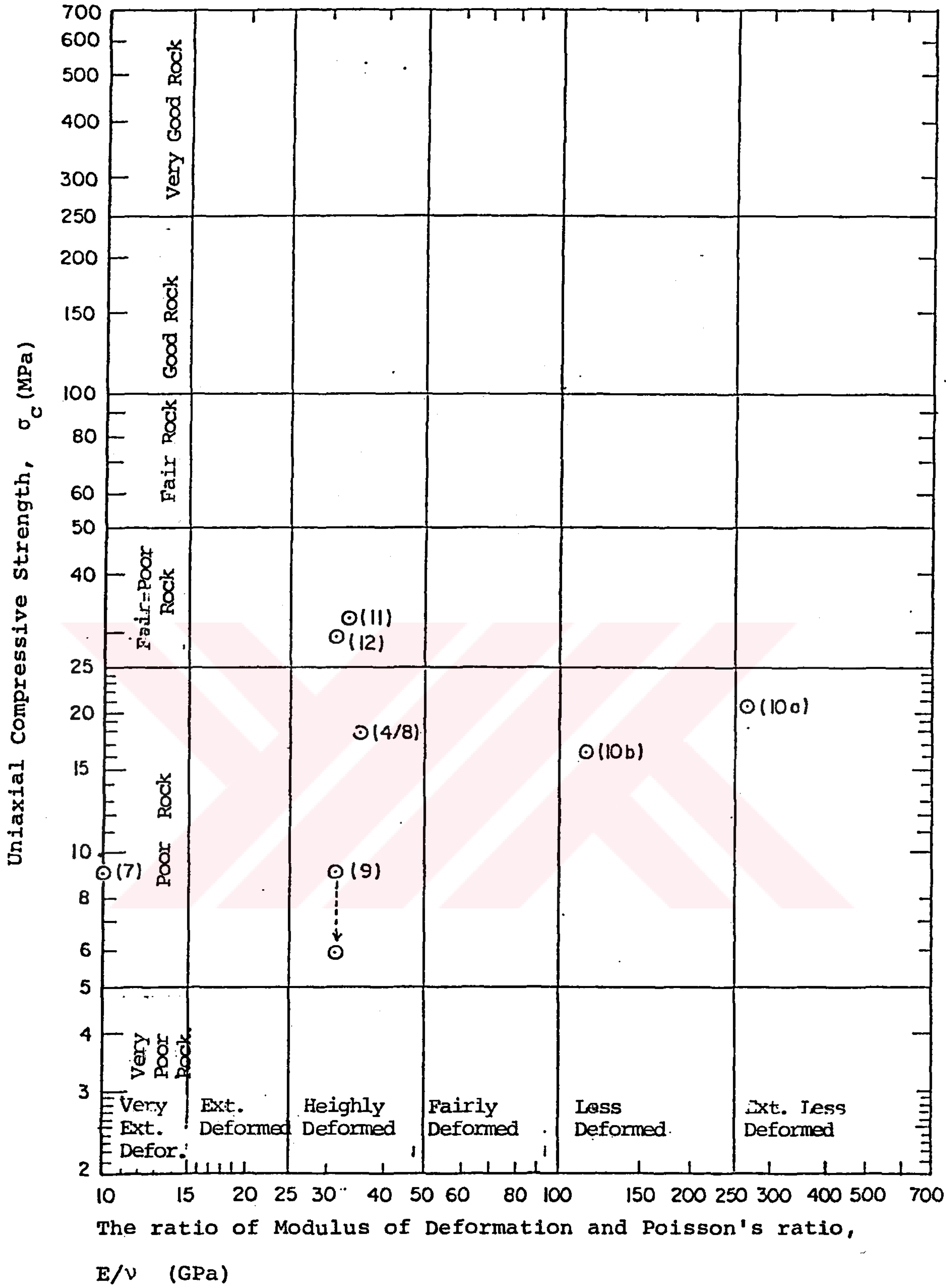


Figure 3.5. The relationship between the Uniaxial Compressive Strength (σ_c) and Modulus of Deformation (E)/ Poisson's ratio (ν)

CHAPTER IV

ROCK MASS CLASSIFICATION IN SIMAV-MINE REGION

The input information needed for design purposes generally includes geological characterization of the rock masses, evaluation of the virgin ground stresses and the mechanical properties which charecterize the rock mass in its natural state. Whatever procedures and techniques are employed to obtain the input parameters, it is necessary to emphasize that any such procedures and techniques should only be used if they can be fully justified for the purposes of a given project (Rafia ,1980). In other words, measurements and investigations should be carefully planned and matched with the purpose of a project, full justification being given for any investigations and tests performed.

Furthermore, determination of the input parameters for design should be so planned that as much quantative data as possible is obtained rather than relying on qualitative descriptions (Rafia ,1980).

In essence, the following three important messages emerge:

1. The eventual quality of the engineering design is directly proportional to the quality of the input parameters.

2. Any procedures or methods employed in providing the input data should be fully justified and carefully planned.

3. Quantitative rather than qualitative information is required for design purposes.

Determining the input parameters for rock engineering design will be of prime concern both to the design engineer and the engineering geologist. It is therefore essential that both these persons work closely as members of the same team. Moreover, two principles should be observed:

1. The design engineer should clearly state his requirements so that the engineering geologist can understand the need for specific input parameters.

2. The engineering geologist should provide the data in such a way that they can be employed directly for design purposes by the engineer.

The determination of the input data for design involving geological site characterization, ground stress conditions, ground water conditions and mechanical properties of rock masses is a wide subject which in itself could well be the topic of a textbook.

4.1. Geological and Geotechnical Logs Utilized in Rock Mass Classifications

The rock mass classification was carried out utilizing the cores obtained from five surface and two underground boreholes. The first step in classification was to divide the rock mass into a number of structural regions. These regions are geological zones of rock masses in which certain features are more or less uniform. Although rock masses are discontinuous in nature, they may nevertheless be uniform in regions when, for example, the type of rock or the spacings of discontinuities are the same throughout the region. In most cases, and for borehole evaluations, each type of formation was accepted as a new structural region. In addition, any distinct zones, (i.e containing clay, heavily fractured, highly weathered) within each formation were treated as a new structural region. It was also assumed that the boundaries of structural regions also coincide with such major geological features as faults and shear zones.

Once the structural regions have been delineated, the ratings for input parameters were determined and the associated information was filled in the geological and geotechnical borehole-log forms shown in Figures 4.1 and 4.2 respectively.

M.E.T.U. MINING ENGINEERING DEPARTMENT – GEOLOGICAL DRILLING LOG.					
Project				Drilling Location	
Consultant				Hole No.	
Manager				Logger	
Drilling machine		Coordinates	X : ; Y:	Controller	
Type of rig		Heigth (Z)		Starting Date	
Type of drill-bit.		Drilling Location	Scale:	Finishing Date	

Depth of Drill-run (m)	Length of Drill-run (m)	Diameter of (mm) Drill-hole Type of Drill bit	Water Table	Total Core Recovery (%) 20 40 60 80	Intact Core Recovery (%) 20 40 60 80	Rock Quality Designation 25 50 75 90	Joint Spacing (#/m) 2 4 8 16	Aperture	Hhgt from Sea Level (m)	Logging Symbol	Description of Formation	Formation Description Number

Figure 4.1. The form of geological drilling log.

M.E.T.Ü. MINING ENGINEERING DEPARTMENT– GEOTECHNICAL LOG.					
Project				Drilling Location	
Consultant				Hole No	
Manager				Logger	
Drilling Machine		Coordinates	X : ; Y:	Controller	
Type of rig		Height (Z)		Starting Date	
Type of drill-bit		Drilling Location		Finishing Date	

Depth of Drill-run (m)	Length of Drill-run (m)	Water Level	Joint Spacing (mm)	Apperture	Roughness	Continuity	Fracture Spacing	Filling Material	Joint alteration number (Ja)	RQD	Fc (MPa)
			20, 60, 200, 600, 2000, 6000	HA, AA, OA, CA, IA	— ; undulating ; irregular	1, 3, 10, 20	0.02, 0.25, 0.50, 2.5, 10.0	Type, Length, Strength, Apperture, Other rock		25, 50, 75, 90	

Figure 4.2. The form of geotechnical log.

4.2. Rock Mass Classification Based on Original RMR-System

In RMR-system, six input parameters are grouped into five ranges of values and importance ratings are allocated to the different value ranges of the parameters such that a higher rating indicates better rock mass conditions. Once the importance ratings are assigned to each parameter, they are summed to yield the basic rock-mass rating for structural unit under consideration. Finding the influence of the orientations of the main discontinuities, enables us to determine the adjusted (final) RMR value as the next step.

The results obtained from Geomechanics Classification applied to Simav Mine are presented in Table 4.1. During classification process a relatively short computer Program developed by Ünal (1985) was utilized.

4.3. Rock Mass Classification Based on Original Q-System.

The six parameters chosen to describe the rock mass quality Q are combined as described earlier in Section 2.4. The results of rock mass classifications carried out in Simav-Mine Region based on Q-System is presented in Table 4.2. The RMR, Q and RQD logs associated with Simav-Mine is given in Table 4.3.

Table 4.1. The Geomechanics Classification Results of Simav-Mine. (The RMR Values Represent the Weight Average Taken Based on Thricknesses of Layers).

Formations	Thickness (m)	Original RMR		Description of the Formations
1.LAYER	5.60	30		Weathered Limestone interbedded with Claystone and Tuff
2.LAYER	1.90	24		Limestone alternating with Claystone and repeated Limestone
3.LAYER	0.90	28		Limestone interbedded with Claystone and Tuff
1.SEAM	3.80	32		COLEMANITE
4.LAYER			16-46	Weathered Limestone interbedded with Claystone and Tuff
5.LAYER	3.45	24	12-32	Limestone alternating with Claystone and interbedded with Tuff
6.LAYER			12-32	Limestone alternating with Claystone and random COLEMANITE
2.SEAM	5.60	33		COLEMANITE
7.LAYER			19-40	Limestone alternating with Claystone and random COLEMANITE
8.LAYER	4.10	23	13-40	Limestone interbedded and Laminated Claystone
9.LAYER			18-33	Claystone and Limestone interbedded with Tuff

Table 4.1. (Continued).

<table>
<tr><th>Formations</th><th>Thickness (m)</th><th colspan="2">Original RMR</th><th>Description of the Formations</th></tr>
<tr><td>3.SEAM</td><td>7.30</td><td colspan="2">29</td><td>COLEMANITE</td></tr>
<tr><td>10.LAYER</td><td rowspan="3">5.00</td><td rowspan="3">28</td><td>13-24</td><td>Limestone alternating with Claystone and randomly interbedded Lst. and Clst.</td></tr>
<tr><td>11.LAYER</td><td>13-30</td><td>Limestone alternating with Claystone</td></tr>
<tr><td>12.LAYER</td><td>13-40</td><td>Limestone interbedded with hard Claystone</td></tr>
<tr><td>4.SEAM</td><td>5.00</td><td colspan="2">31</td><td>ULEXITE</td></tr>
<tr><td>13.LAYER</td><td rowspan="2">4.37</td><td rowspan="2">30</td><td>18-44</td><td>Limestone alternating with Claystone</td></tr>
<tr><td>14.LAYER</td><td>11-54</td><td>Claystone interbedded Limestone</td></tr>
<tr><td>5.SEAM</td><td>3.20</td><td colspan="2">35</td><td>COLEMANITE</td></tr>
<tr><td>15.LAYER</td><td>9.65</td><td colspan="2">37</td><td>Tuff,limestone interbedded with Claystone. Limestone alternating with Claystone</td></tr>
<tr><td>INTERMEDIATE TUFF</td><td>1.00</td><td colspan="2">56</td><td>Intermediate tuff</td></tr>
<tr><td>16.LAYER</td><td>5.45</td><td colspan="2">42</td><td>Tuff and weathered Limestone interbedded with Claystone</td></tr>
<tr><td>UPPER TUFF</td><td>2.80</td><td colspan="2">57</td><td>Upper Tuff</td></tr>
</table>

Table 4.2. The Q-Classification Results in Simav Mine Region. (The Q Values Represent the Weighted Average Taken Based on Thickness of Layers)

Formations	Thickness (m)	Original Q		Description of the Formations
1.LAYER	5.60	0.840		Weathered Limestone interbedded with Claystone and Tuff
2.LAYER	1.90	0.233		Limestone alternating with Claystone and repeated Limestone
3.LAYER	0.90	0.374		Limestone interbedded with Claystone and Tuff
1.SEAM	3.80	0.725		COLEMANITE
4.LAYER	3.45	0.213	0.15 0.98	Weathered Limestone interbedded with Claystone and Tuff
5.LAYER			0.42 0.51	Limestone alternating with Claystone and interbedded with Tuff
6.LAYER			0.02 0.31	Limestone alternating with Claystone and random COLEMANITE
2.SEAM	5.60	1.097		COLEMANITE
7.LAYER	4.10	0.471	0.096 0.482	Limestone alternating with Claystone and random COLEMANITE
8.LAYER			0.016 0.202	Limestone interbedded and Laminated Claystone
9.LAYER			0.018 0.317	Claystone and Limestone interbedded with Tuff

Table 4.2. (Continued)

Formations	Thickness (m)	Original Q		Description of the Formations
3.SEAM	7.30	0.325		COLEMANITE
10.LAYER	5.00	0.341	0.013 0.457	Limestone alternating with Claystone and randomly interbedded Lst. and Clst.
11.LAYER			0.015 0.45	Limestone alternating with Claystone
12.LAYER			0.045 0.603	Limestone interbedded with hard Claystone
4.SEAM	5.00	0.978		ULEXITE
13.LAYER	4.37	0.692	0.032 1.39	Limestone alternating with Claystone
14.LAYER			0.003 1.91	Claystone interbedded Limestone
5.SEAM	3.20	0.267		COLEMANITE
15.LAYER	9.65	0.963		Tuff,limestone interbedded with Claystone, Limestone alternating with Claystone
INTERMEDIATE TUFF	1.00	20.33		Intermediate tuff
16.LAYER	5.45	1.20		Tuff and weathered Limestone interbedded with Claystone
UPPER TUFF	2.80	21.54		Upper Tuff

Table 4.3. Summary of Classification Results in Simav-Mine Region (RMR, Q and RQD Rating Values Represent the Weighted Average Taken Based on Thicknesses of Layers).

Formations	Thickness (m)	Original RMR		Original Q		RQD		Description of the Formations
1.LAYER	5.60	30		0.840		23		Weathered Limestone interbedded with Claystone and Tuff
2.LAYER	1.90	24		0.233		2		Limestone alternating with Claystone and repeated Limestone
3.LAYER	0.90	28		0.374		10		Limestone interbedded with Claystone and Tuff
1.SEAM	3.80	32		0.725		24		COLEMANITE
4.LAYER			16-46		0.15 0.98		0-39	Weathered Limestone interbedded with Claystone and Tuff
5.LAYER	3.45	24	12-32	0.213	0.42 0.51	7	0-16	Limestone alternating with Claystone and interbedded with Tuff
6.LAYER			12-32		0.02 0.31		0-11	Limestone alternating with Claystone and random COLEMANITE
2.SEAM	5.60	33		1.097		24		COLEMANITE
7.LAYER			19-40		0.096 0.482		0-41	Limestone alternating with Claystone and random COLEMANITE
8.LAYER	4.10	23	13-40	0.471	0.016 0.202	12	0-36	Limestone interbedded and Laminated Claystone
9.LAYER			18-33		0.018 0.317		0-17	Claystone and Limestone interbedded with Tuff

Table 4.3. (Continued)

Formations	Thickness (m)	Original RMR		Original Q		RQD		Description of the Formations
3.SEAM	7.30	29		0.325		11		COLEMANITE
10.LAYER			13-24		0.013 0.457		0-18	Limestone alternating with Claystone and randomly interbedded Lst. and Clst.
11.LAYER	5.00	28	13-30	0.341	0.015 0.45	16	0-17	Limestone alternating with Claystone
12.LAYER			13-40		0.045 0.603		0-30	Limestone interbedded with hard Claystone
4.SEAM	5.00	31		0.978		22		ULEXITE
13.LAYER	4.37	30	18-44	0.692	0.032 1.39	29	0-75	Limestone alternating with Claystone
14.LAYER			11-54		0.003 1.91		0-43	Claystone interbedded Limestone
5.SEAM	3.20	35		0.267		8		COLEMANITE
15.LAYER	9.65	37		0.963		29		Tuff, limestone interbedded with Claystone, Limestone alternating with Claystone
INTERMEDIATE TUFF	1.00	56		20.33		74		Intermediate tuff
16.LAYER	5.45	42		1.20		48		Tuff and weathered Limestone interbedded with Claystone
UPPER TUFF	2.80	57		21.54		65		Upper Tuff

4.4. Discussion of the Results

The results obtained from RMR and Q-systems were compared based on the following:

1) Maximum span and stand-up time
2) Rock-load-height (h_t)
3) Support pressure (P)
4) Residual and peak friction angles (ϕ)
5) m and s material constants of the rock mass
6) Major and Minor principal stresses (σ_1 and σ_3)
7) Modulus of deformation (E_m)
8) Support requirements

Maximum Span and Stand-up Time

One of the outputs the RMR-system is the relationship between the stand-up time and unsupported-rock-span for various RMR values. The RMR-system also provides information on spans which "require no support" and the ones that will"collapse immediately". The Q-system also provides an equation from which the spans which will "not reqire any support" can be determined. The relationships associated with stand-up time, roof span and various rock mass quality indices are shown in Figures 4.3, 4.4 and 4.5.

According to Figure 4.3, if a mining practice requires an opening in a rock mass whose RMR = 35, the maximum

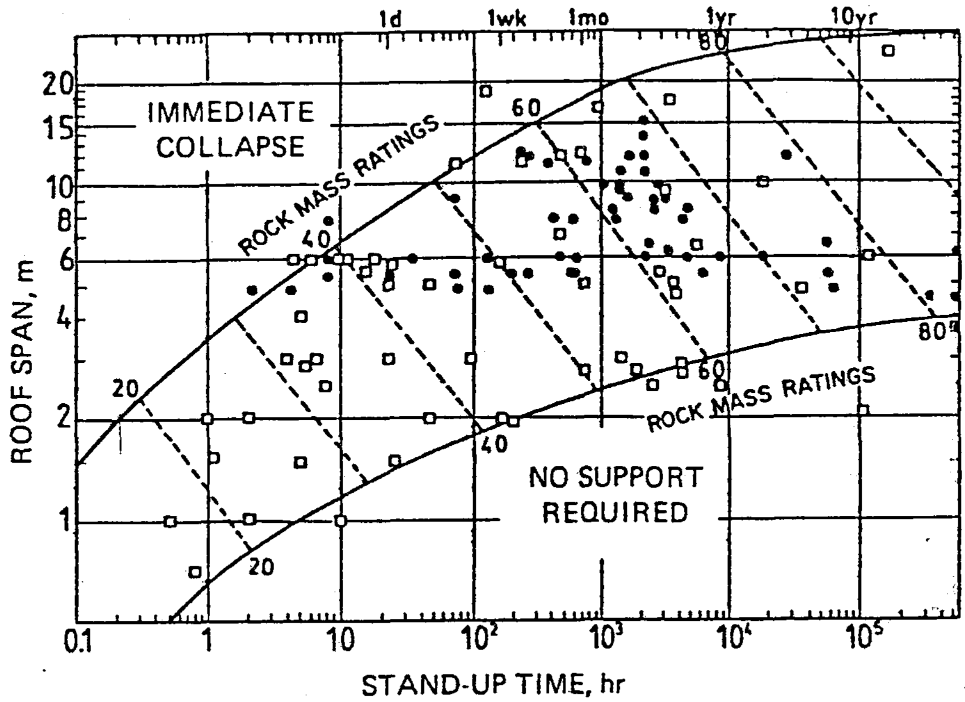
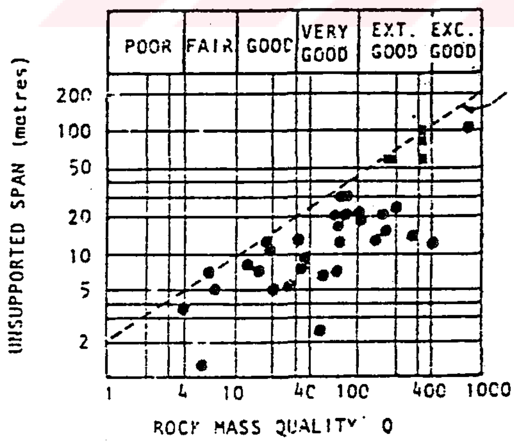


Figure 4.3. Relatinship between the stand-up time of an unsupported underground excavation span and the Geomechanics Classification proposed by Bieniawski (After Bieniawski, 1984)



• man-made excavations

▪ natural openings in limestone at Carlsbad, New Mexico.

Figure 4.4. Unsupported span versus rock mass quality(Q) relationship for man-made and natural unsupported excavations in different quality rock masses (After Barton,1974).

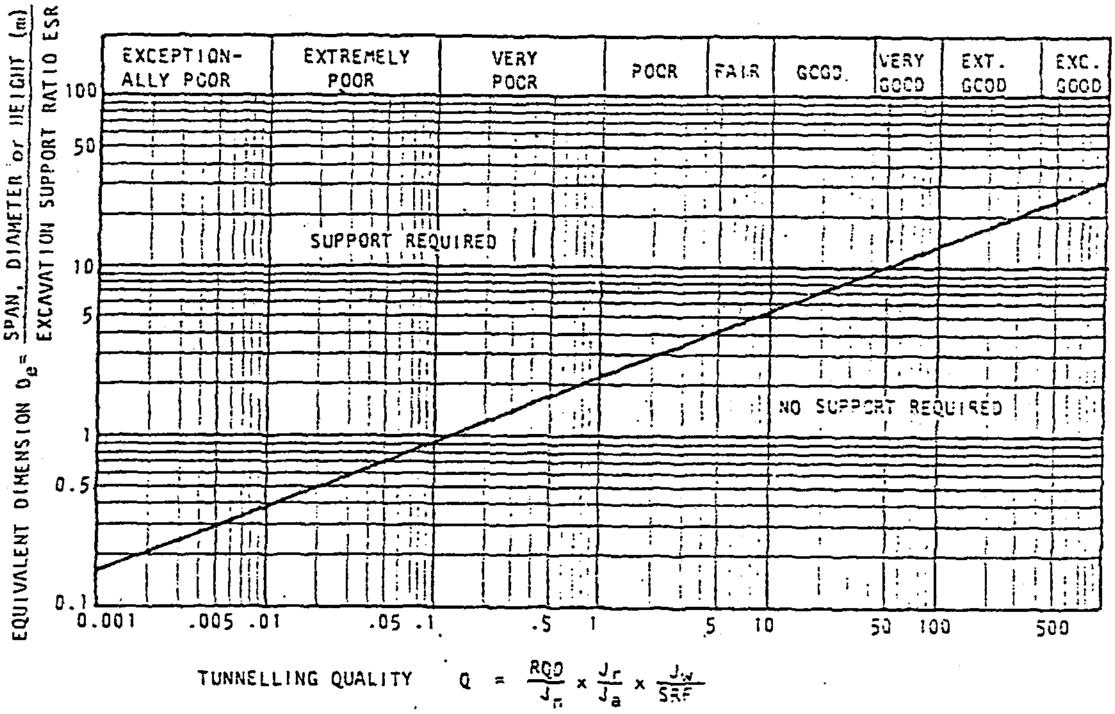


Figure 4.5. Recommended maximum unsupported excavation spans for different quality rock masses. (After Barton, 1974).

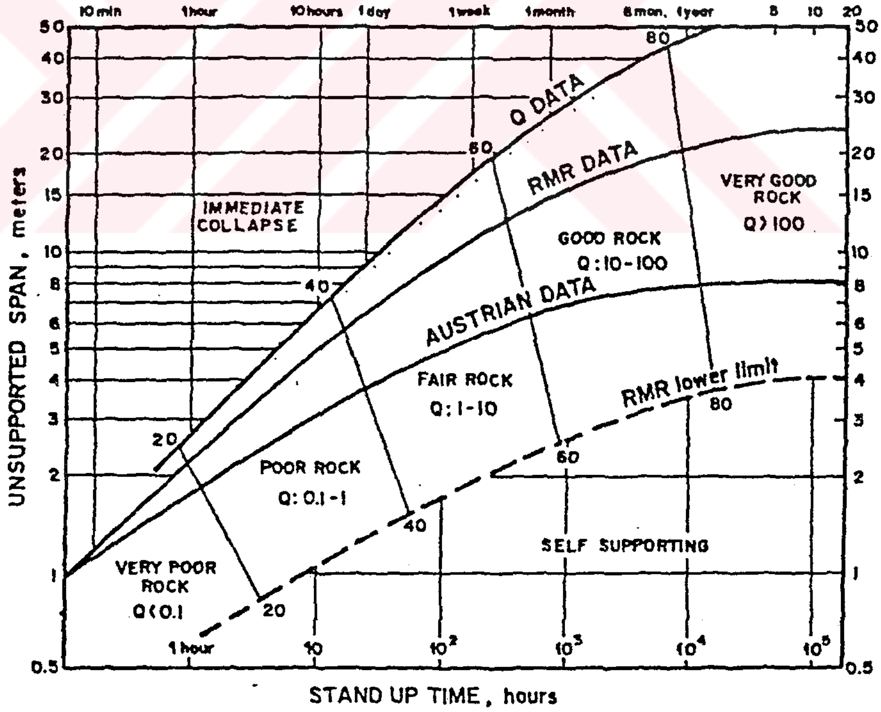


Figure 4.6. Comparison between stand-up times for unsupported excavation spans predicted by the Q-system. RMR and Austrian rock mass classification systems. Ratings are for the Geomechanics Classification (RMR). (After Bieniawski, 1984).

possible roof span is 5 m (read off from the interception of rating line of 35 with the upper heavy line). In this rock mass, it is also predicted that a roof, span 1.5 m or less will last unsupported infinitely (read off from the intersection of rating line of 35 with the bottom line in Figure 4.3).

In case of Q-system, the maximum limit for parmenent spans (in meters) can be obtained by using Figures 4.4 and 4.5 or Equations 4.1. and 4.2 given below:

$$B = 2(ESR)\ Q^{0.40} \tag{4.1}$$

$$B = 2\ Q^{0.66} \tag{4.2}$$

Where B is the maximum unsupported span, ESR is the excavation support ratio. ESR modifies the span or diameter of an opening, reflects the construction practice in that the degree of safety and support demanded by an excavation is determined by the purpose of the excavation, the presence of machinery, personel etc.

As pointed out by Barton, there is no way of knowing how close these excavations are to failure and hence it could be argued that Figure 4.4 will always provide a conservative estimate of unsupported excavation span. While this is certainly true, it must be remembered that before deciding upon a less conservative design it is worth spending a few moments contemplating the consequences of this.

A comparision between Scandinavian, South African and Austrian estimates of maximum unsupported span for different rock mass qualities has been made by Bieniawski (1979) and a part of his graphs is repreduced in Figure 4.6.

The estimated stand-up time, for various roof conditions, in Simav Mine are summarized in Tables 4.4 and 4.5.

In general the widths of the gate roadways excavated in underground mine were 2-2.5 meters. These roadways were supported by wooden post and beams. Underground investigations indicated that supports were always required in gateroads, in other words unsupported span of 2-2.5 meters were not possible. The classification results obtained from RMR system indicated that unsupported roof spans in Bigadic Mine could be excavated for 1.2-1.4 meters in seams. On the other hand, According to Q-system (Equation 4.2) excavation of unsupported gate-roads having spans of 2.04-3.32 meters were possible. These results obtained based on Q-system are contradictory to the observations made in underground mine. The Q-system on the other hand predicts better results with Equation 4.2.

The Rock - Load Height (h_t)

In order to define the support parameters, by use of emprical approaches it is particularly important to

Table 4.4. Estimated Maximum Unsupported Span and Stand- up Time for Different Roof Conditions of Bigadiç Mine (by RMR Classification)

Roadway	Rock Type	Roof Quality RMR	Description	Max. Unsupported span (m)	Stand-up Time
Above seam No 1	Weathered limestone int. with claystone and tuff	28.43	Poor Rock	1.1	9 hours
in seam No.1	Colemanite	32	Poor Rock	1.4	25 hours
Above seam No.2	Limestone alternating with claystone and tuff	24	Poor Rock	1.0	4 hours
in seam No.2	Colemanite	33	Poor Rock	1.4	30 hours
Above seam No.3	Limestone alternating with claystone	23	Poor Rock	0.9	3.5 hours
in seam No.3	Colemanite	29	Poor Rock	1.2	13 hours
Above seam No.4	Limestone alternating with claystone	28	Poor Rock	1.1	9 hours
in seam No.4	Ulexite	31	Poor Rock	1.3	20 hours
Above seam No.5	Limestone alternating with claystone	30	Poor Rock	1.25	16 hours
in seam No.5	Disintegrated Colemanite	35	Poor Rock	1.5	40 hours
in upper tuff	Upper tuff	57	Fair Rock	3.0	4 month

Table 4.5. Estimated Maximum Unsupported Span for Different Roof Conditions of Bigadiç Mine (by Q Classification, in Terms of ESR = 1.6)

Roadway	Rock Type	Roof Quality Q	Description	$B = 2 \times Q^{0.66}$ Max.unsupported span (m)	$B = 2\ ESR\ Q^{0.40}$ (m) Max.unsupported span (m)
Above seam No.1	Weathered limestone interbeded with claystone and tuff	0.65	Very Poor Rock	1.50	2.69
in seam No.1	Colemanite	0.61	Very Poor Rock	1.44	2.62
Above seam No.2	Limestone alternating with claystone and tuff	0.213	Very Poor Rock	0.72	1.78
in seam No.2	Colemanite	1.097	Poor Rock	2.13	3.32
Above seam No.3	Limestone alternating with claystone	0.471	Very Poor Rock	1.58	2.37
in seam No.3	Colemanite	0.325	Very Poor Rock	0.95	2.04
Above seam No.4	Limestone alternating with claystone	0.341	Very Poor Rock	0.98	2.1
in seam No.4	Ulexite	0.978	Very Poor Rock	1.97	3.17
Above seam No.5	Limestone alternating with claystone	0.692	Very Poor Rock	1.57	2.76
in seam No.5	Disintegrated Colemanite	0.460	Very Poor Rock	1.19	2.34
in upper tuff	Upper tuff	21.54	Good Rock	15.16	10.93

analyze the rock-load height that should be controlled by supports. The controlling mechanism however, depends on the type of support used, therefore, the rock loads should be treated differently.

An emprical equation, developed by Ünal (1983,1986) relate the rock-load height, (h_t) to a quantitative rock quality index (RMR), and roof span (B) as follows:

$$h_t = \frac{100-RMR}{100} B \text{ (m)} \qquad (4.3)$$

The rock-load heights expected on openings excavated in various formations are tabulated in Table 4.6.

Support Pressure

The relationship between the Q value and the parmenent support pressure can be calculated from Equations (4.4a) and 4.4b). If the number of joint sets is more than three:

$$P_s = 0.2 \; J_r^{-1} \; Q^{-1/3} \quad \text{(MPa)} \qquad (4.4a)$$

If the joint sets are less than three:

$$P_s = \frac{0.2}{3} J_r^{-1} \; J_n^{1/2} \; Q^{-1/3} \quad \text{(MPa)} \qquad (4.4b)$$

Table 4.6. The Rock-load Heights Calculated Based on RMR-System

Roadway	Rock Type	Roof Quality RMR	Description	Spans, B (m)		Rock load height h_t (m)	
				Unsupported Spans	Spand observed in underground	For unsupported Spans	For observed Spans
Above seam No.1	Alterated limenstone interbeded with claystone and tuff	28.43	Poor Rock	1.1	2.5	0.79	1.79
in seam No.1	Colemanite	32.0	Poor Rock	1.4	2.5	0.95	1.7
Above seam No.2	Limestone alterated with claystone and tuff	24.0	Poor Rock	1.0	2.5	0.74	1.9
in seam No.2	Colemanite	33.0	Poor Rock	1.4	2.5	0.94	1.68
Above seam No.3	Limestone laminated with claystone	23.0	Poor Rock	0.9	2.5	0.69	1.93
in seam No.3	Colemanite	29.0	Poor Rock	1.2	2.5	0-85	1.78
Above seam No.4	Limestone laminated with claystone	28.0	Poor Rock	1.1	2.5	0.79	1.8
in seam No.4	Ulexite	31.0	Poor Rock	1.1	2.5	0.897	1.73
Above seam No.5	Limestone laminated with claystone	30.0	Poor Rock	1.25	2.5	0.87	1.75
in seam No.5	Disintegrated Colemanite	35	Poor Rock	1.6	2.5	1.04	1.63
in upper tuff	Upper tuff	57	Fair Rock	3.0	4.3	1.29	1.35

Where P_s is the support load, Q is the rock-quality index, J_r is joint roughness index and J_n is joint sets number.

To calculate the rock pressure ,for RMR System, the expression shown in Equation 4.5 has been developed:

$$P = h_t \times \gamma \ (ton/m^2) \qquad (4.5)$$

In Equation (4.5) P is the rock pressure developing on supports in roadways, h_t is the rock-load height, and γ is the unit-weight of the rock.

The support pressures for various formations, calculated by Equations 4.4, 4.5 are tabulated in Tables 4. 7 , 4. 8.

Residual and Peak Friction Angles (ϕ)

Bieniawski's RMR-system and Barton's Q-seystem estimate friction angle in two different ways. According to Bieniawski's RMR rating, friction angle of the rock mass can be found from Table 4.9 .

In Barton's classification system the friction angle is obtained from Equation 4.7. (Barton,1974)

$$\phi = \tan^{-1} \left(\frac{J_r}{J_a}\right) \qquad (4.7)$$

Table 4.7. Support Pressures Estimated Based on Q-System.

Roadway	Rock Type	Rock Mases Qual Q	Joint Roughness Number J_r	Joint sets Number J_n	Support Pressure (kPa)
Above seam No.1	Weathered limenstone interbeded with claystone and tuff	0.65	3	9	77.0
in seam No.1	Colemanite	0.61	3	7	78.0
Above seam No.2	Limestone alternating with claystone and tuff	0.213	3	15	116.6
in seam No.2	Colemanite	1.097	3	9	64.6
Above seam No.3	Limestone alternating with claystone	0.471	2	15	128.0
in seam No.3	Colemanite	0.325	3	15	96.9
Above seam No.4	Limestone alternating with claystone	0.341	2	7	143.1
in seam No.4	Ulexite	0.978	3	9	67.0
Above seam No.5	Limestone alternating with claystone	0.692	2	2	26.7
in seam No.5	Disintegrated Colemanite	0.460	3	9	86.4
in upper tuff	Upper tuff	21.54	2	2	16.9

Table 4.8. Support Pressures Estimated Based on Ünal's Approach

Roadway	Rock Type	Rock Mass rating RMR	Unit weight of rock (kN/m^3)	Observed span (m)	According to the observed h_t (m)	Support According to the observed span (kPa)	Pressure According to the unsupported maximum span (kPa)
Above seam No.1	Weathered limestone: interbeded with claystone and tuff	28.43	23	2.5	1.79	4.12	18.2
in seam No.1	Colemanite	32.0	23	2.5	1.70	39.1	21.8
Above seam No.2	Limestone alternating with claystone and tuff	24.0	23	2.5	1.90	43.7	17.5
in seam No.2	Colemanite	33.0	23	2.5	1.68	30.6	21.6
Above seam No.3	Limestone alternating with claystone	23.0	23	2.5	1.93	44.4	15.9
on seam No.3	Colemanite	29.0	23	2.5	1.78	41.0	19.6
Above seam No.4	Limestone alternating with claystone	28.0	23	2.5	1.80	41.4	18.2
in seam No.4	Ulexite	31.0	23	2.5	1.73	39.8	20.6
Above seam No.5	Limestone alternating with claystone	30.0	23	2.5	1.75	40.2	20.0
in seam No.5	Disintegrated colemanite	35.0	23	2.5	1.63	37.5	24.0
in upper tuff	Upper tuff	57.0	23	2.5	1.85	42.5	29.7

Table 4.9. Estimation of Cohesion and Friction Angle of the Rock Mass Based on Rock Mass Classes (After Bieniawski, 1984).

Class No.	I	II	III	IV	V
Cohesion of the rock mass	>400 kPa	300-400kPa	200-300kPa	100-200kPa	<100kPa
Friction angle of the rock mass	$<45^{o}$	35^{o}-45^{o}	25^{o}-35^{o}	15^{o}-25^{o}	$<15^{o}$

Where ϕ is friction angle, J_a is joint alteration number and J_r is joint roughness number.

Recently, Sheorey(1985) recommends that the peak friction angle's should be obtained from the RMR-system (Table 4.9), but the residual friction angle's from the Q-system (Equation 4.7).

The peak values of recommended by Barton were found to be considerably on the higher side as far as this analysis was concerned. Bieniawski's values of the peak friction angle ϕ' were therefore chosen corresponding to the RMR calculated. The residual friction angle ϕ ,, was, however, taken from Barton's tables which give these values depending on the joint alteration number J_a, which Bieniawski's tables do not include them. In choosing the upper limit of ϕ_o from Barton's range it was decided that the minimum difference between ϕ' and ϕ_o should be 5^{o}, while the lower limit was considered acceptable (Sheorey, 1985).

m and s Material Constants for Rock Mass

Hoek and Brown (1980) proposed an emprical criterion of failure for rock mass strength as follows:

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \sqrt{m \frac{\sigma_3}{\sigma_c} + s} \qquad (4.8)$$

Where σ_1 is the major principal stress at failure, σ_3 is the applied minor principal stress, σ_c is the uniaxial compressive strength, and m and s are constants that depend on the properties of the rock and the extent to which it has been fractured by being subjected to σ_1 and σ_3. m and s values have been calculated by Hoek and Brown (1980) for various rock types. Priest and Brown (1983) also predict m and s values depending on RMR ratings obtained in the field.

The associated expression are shown Equations (4.9) and (4.10).

$$m = m_i \quad \exp \left| \frac{\text{RMR} - 95}{13.4} \right| \qquad (4.9)$$

$$s = \exp \left| \frac{\text{RMR} - 100}{6.3} \right| \qquad (4.10)$$

A comparison of m and s values as estimated by Hoek and Brown (1980) with the estimates of Priest and Brown (1983) is presented in Table 4.10.

Table 4.10. Comparison of m and s Values, As Obtained from Hoek and Brown (1980) and from Priest and Brown (1983)

Rock Type	RMR	Estimated of Priest and Brown m	s	According to Hoek and Brown's chart (for RMR) m	s
Colemanite (10a)	30.64	0.09	1.65×10^{-5}	0.086	2.4 10^{-5}
Ulexite (10b)	23.0	0.03	0.49×10^{-5}	0.028	0.74 10^{-5}
Limestone alternating with claystone (4/8)	37.0	0.043	4.5×10^{-5}	0.041	66.5 10^{-5}
Limestone (8)	21.0	0.0067	0.36×10^{-5}	0.0064	0.54 10^{-5}
Weathered limestone (9abc)	38.87	0.011	6.1×10^{-5}	0.010	8.7 10^{-5}
Tuffite (7)	52.43	0.115	52.5 10^{-5}	0.105	72.3 10^{-5}
Intermediate tuff (11)	35.89	0.036	3.8 10^{-5}	0.034	54 10^{-5}

Modulus of Deformation of Rocks (E_m)

The in-situ modulus of deformation (E_m) can be predicted by RMR-System (Bieniawski,1978) following the correlation given in Equation (4.11)

$$E_m = 2 \times RMR - 100 \qquad (4.11)$$

Where E_m is in-situ modulus of deformation in GPa and RMR > 50

Most recently based on the statistical evaluation of the field data, Serafim and Pereira (1983) proposed a new correlation as shown in Equation 4.12. The relationship suggested by Equations 4.11 and 4.12 are presented in Figure 4.7

$$E = 10^{\left(\frac{RMR-10}{40}\right)} \qquad (0 < RMR \leqslant 100) \qquad (4.12)$$

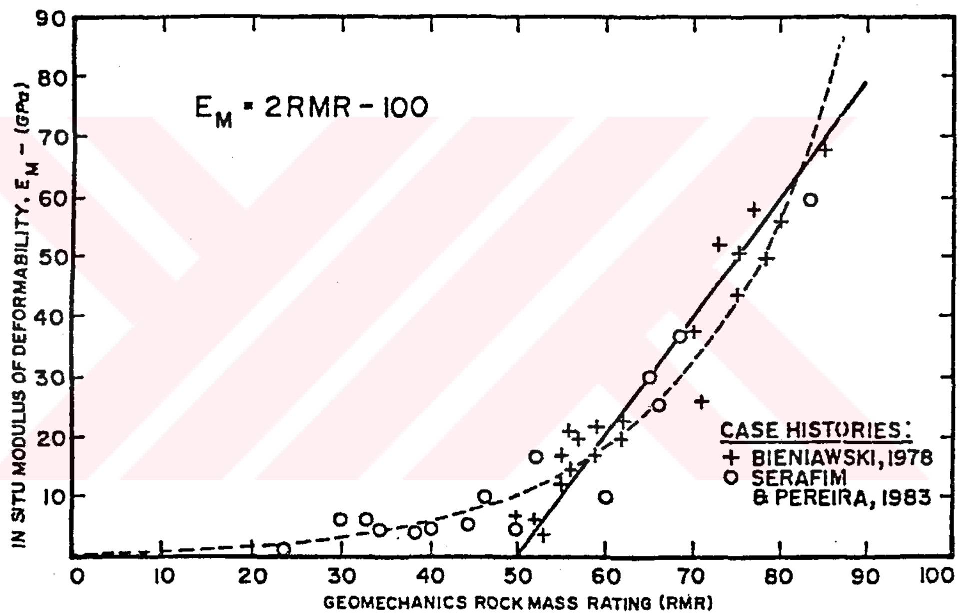


Figure 4.7. The relationship between the in-situ modulus of deformation and the RMR-System.

Table 4.11. Comparison of Modulus of Deformation as Predicted by Bieniawski (1978) and Serafim an Pereira

Rock Type	RMR-system RMR	Modulus of Deformation E_m (GPa)	
		Breniawski (1978)	Serafim and Pereira (1983)
Limestone alterating with claystone (4/8)	37		4.73
Colemanite (10a)	30.64		3.28
Ulexite (10b)	23.0		2.11
Weathered limestone (9abc)	38.87		5.27
Tuffite (7)	52.43	4.86	11.50
Tuff (11)	35.89		4.44

CHAPTER V

SUGGESTIONS FOR DETERMINATION OF RMR AND Q VALUES IN WEAK AND STRATIGIED ROCK MASSES

5.1. General

During this study, a total of 830 meters long core, obtained from seven boreholes, were examined for classification purposes. Visual examination of the core boxes, investigation carried out in the field, and observations made in Bigadiç underground mine indicated that, in general the rock mass was weak and stratified. For characterization of this rock mass, two of the well known classification systems, namely, RMR and Q-Systems were utilized.

During the process of classification, however, a number of serious difficulties were encountered in describing some of the rock-mass parameters. It was realized at this stage that, in their original form RMR and Q-Systems were insufficient to describe the weak and stratified rock masses for engineering purposes.

In this chapter the difficulties encountered in describing some of the rock-mass-classification parameters were discussed. The suggested modification and recommendations were also included.

5.2. Need for Modified Parameters for Characterization of Weak and Stratified Rock for RMR-System

During the process of RMR classification a number of serious difficulties were encauntered in describing some of the rock-mass parameters. These difficulties are associated with determination of "RQD", "Compressive Strength", "Effect of water in clay bearing strata", "Joint Condition" and "Joint spacing". These difficulties are briefly explained in the following paragraphs of section 5.2. The suggested modifications are described in Section 5.4.

5.2.1. Difficulties in Determining RQD

As suggested by Deere (1964), RQD is calculated as follows:

$$RQD = 100x \frac{\text{total length of core preces > 100 mm}}{\text{total length of a drill-run}}$$

According to this expression, regardless of the number of formations existing in a drill run, only the total length of the run is suggested to be taken into account. However, as shown in Figure 5.1. If half of the drill-run contained solid cores (RQD = 100) and the other half fractured weak and laminated cores (RQD = 0), the RQD of the total length would be 50 %. If we now assume that this weak and laminated strata (RQD 0) constitutes the immediate roof of a roadway, taking RQD as "50" could result in a considerable misinterpretation in engineering design-analyses. Recommendations on this subject are given in Section 5.4.1.

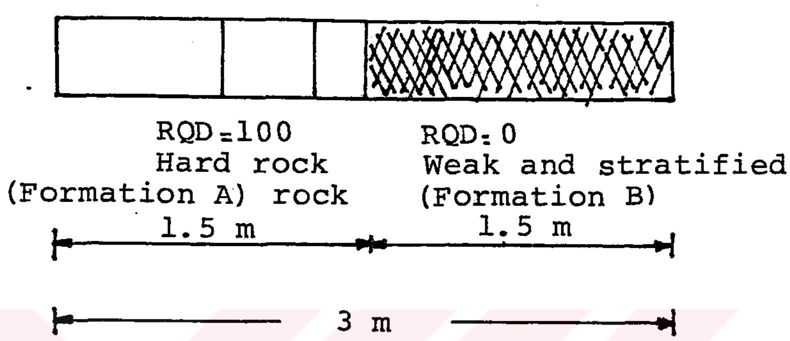


Figure 5.1. The positions of the formations in drill-run.

5.2.2. Difficulties in Determining Compressive Strength (σ_c)

For weak rocks (Uniaxial compressive strength less than 25 MPa), direct uniaxial compressive strength tests are recommended by Bieniawski. It become clear during these investigations that dermination of the direct uniaxial compressive strength of a weak and stratified rock mass through the total length of the formation was not possible. These was due to the difficulties in obtaining suitable cores at various levels for preparation of standard specimens required for laboratory testing. Obtaing only a few suitable cores within a thick but weak formation and assigning the strength of these cores for the total formation seems to be a wrong practice. The observations carried out on cores, obtained from considerable number of bore holes drilled in Bigadiç Region, indicated that, although the number of core pieces were not adequate for compressive strength tests they were certainly adequate for point-load strength tests. As a result of this study, diametral and axial point-load tests are recommended for obtaining strength index value. Following the procedures suggested the uniaxial compressive strength of weak rocks can be determined. The details are given Section 5.4.2.

5.2.3. Difficulties in Determining the Effect of Water on Clay Bearing Rock.

The strength of a rock mass decreases as the water content in that rock mass increases. The effect of water is even more pronounced in the existance of claystone or clay-bearing strata. In RMR-System, Bieniawski recommends a ground-water index without taken into account the damage caused in clay bearing strata due to the existing of underground water. In this study slake-durability tests were recommended to overcome this difficulty. Details on this subject are given in Section 5.4.3.

5.2.4. Difficulties in Describing the Joint Spacing (JS).

In the early stages of RMR calculations, bedding planes cracks, joints, and, fractures existing in the rock mass were assumed as plane of weaknesses. For example, as shown in Figure 5.2; the number of joints for formations A, B and C were counded as 9, greater than 16, and 2 respectively. Consideration of the above joint spacings, resulted in a lower value RMR value than the expected one. To overcome this problem, a modification was suggested as described in Section 5.4.4.

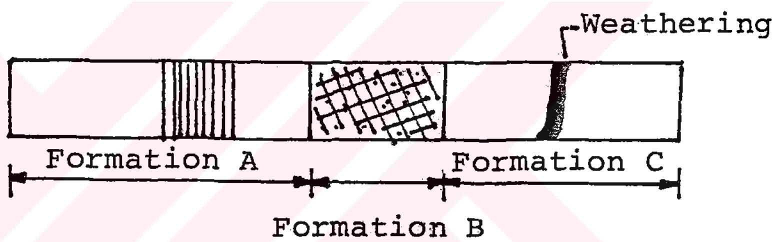


Figure 5.2. The spacing of joints in formations A,B and C.

5.2.5. Difficulties in Describing the Condition of Joints

In calculation of "Joint-Condition index", although the RMR-System considers a number parameters such as seperation, continuity, roughness, alteration and filling of joints, these parameters have not been weighed adequately. The details associated with joint-condition index, also including the suggestions made, are given in Section 5.4.5.

5.3. Need for Modified Parameters for Characterization of Weak and Stratified Rock for Q-System.

During the process of Q classification, a number of difficulties were encountered in describing some of the rock-mass parameters. These difficulties are associated with determination of "RQD", "Joint set number" and "Joint alteration number".

5.3.1. Difficulties in Determining RQD

One of the difficulties in determining RQD value was to find a suitable rating for RQD when this parameter was less than ten (RQD 10). According to Barton RQD should be taken as ten (10) for all RQD values which are less then then however, it is clear that there should be some difference between rocks having a RQD value of zero (0) and RQD value of ten (10)

The other difficulties given in Section 5.2.1 and modifications suggested in Section 5.4.1 also apply for the Q-System.

5.3.2. Difficulties in Determining Joint Set Number (J_n)

In formations where RQD and Intact Core Recovery (ICR) values were very high or very low, the joint set numbers could be clearly determined, however, in other cases, it was difficult to determine the joint set number. This was due to the man-made cracks created during drilling operations or during taking the cores out of the rig. A new approach was suggested to determine the joint set number from cores, as explained in Section 5.5.2.

5.3.3. Difficulties in Determining Joint Alteration Number (J_a)

Joint alteration number (J_a) in Q system have been divided into three sets, all of which consider filling parameter in determining the joint alteration number index. However, in some formations joints may not contain filling materials in them, as observed in some parts of the limestone formation and Upper tuff in Bigadiç Region. In this condition, determination of J_a parameter becomes difficult. Recommendations in this respect are given in Section 5.5.3.

5.4. Suggested Modification for RMR-System.

In this study a number of modifications have been suggested in determining RMR values for weak and stratified rock mass. These modifications are associated with determination includes, of "RQD", "Compressive Strength Index", "Weatherability", "Condition of Joint" and "Joint Spacing".

5.4.1. Rock Quality Designation (RQD)

In calculating RQD with the equation suggested by Deere only the total length of drill-run is considered as explained in Section 5.2.1. This could cause considerable misinterpretation in engineering analysis. For this reason, each formation should be treated seperately. In addition, RQD logging should be carried out soon after the cores obtained from boreholes. The standard procedures should be followed during brohole coring and in treating the cores until logging. If possible, the photographs of the core boxes should be taken as permenant records.

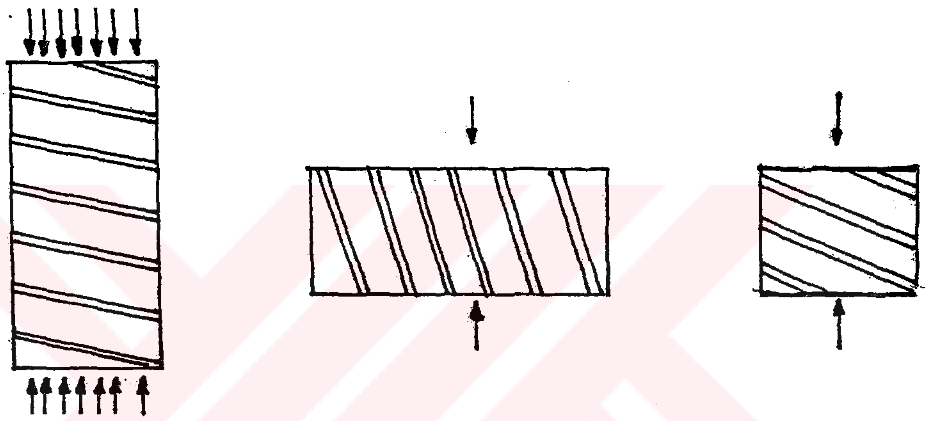
The RQD value can be predicted by Priest and Hudson(1976) considering the number of joints per meter. The associated equation is as follows:

$$RQD = 100\ e^{-0.1\lambda}(0.1\lambda + 1)$$

Where λ is mean number of discontinuities per meter.

5.4.2. Suggestions for Using Point-Load Index Values in Determining Compressive-Strength Index.

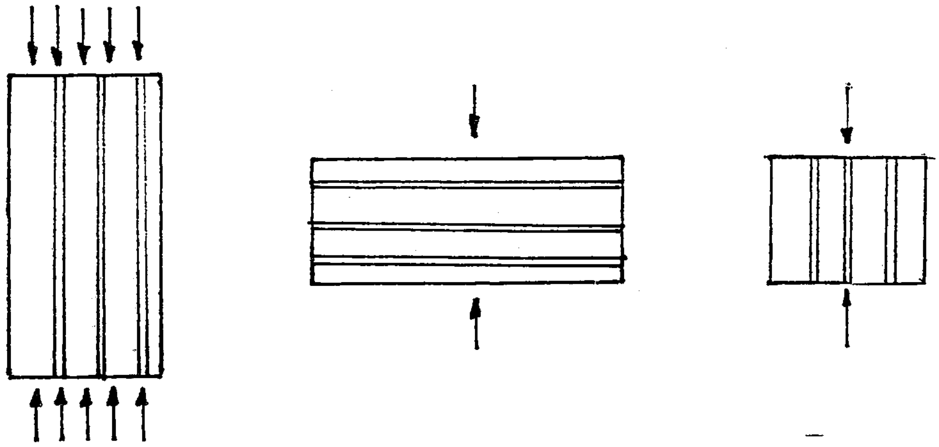
In this study, both the uniaxial-compressive strength and point-load index tests were carried out on different type of formations. During point-load index tests both the diametrial and the axial tests were considered. In general, two series of tests were performed. In the first type the direction of loading was more or less perpendicular to the bedding planes in case of uniaxial compressive strength tests. On the other hand, the direction of loading was "paralell" and "perpendicular" respectively in diametral and axial point-load index tests. The first series of tests, in our case, represents the actual orientation of the formations in underground. The direction of loading with respect to bedding planes during first series of tests are shown in Figure 5.3.



a) Uniaxial compressive strength test. b) Diametral point-load index test. c) Axial point-load index test.

Figure 5.3. Direction of loading with respect to bedding planes in uniaxial compressive strength and point-load index tests. (First series)

During the second series of tests the direction of loading was parallel to bedding planes in case of uniaxial compressive strength and axial point-load tests, while it was perpendicular to bedding planes in diameteral point-load index tests. The direction of loading with respect to bedding planes during second series of tests are shown in Figure 5.4.



a) Uniaxial compressive strength test. b) Diametral point-load index test. c) Axial point-load index test.

Figure 5.4. Direction of loading with respect to bedding planes in uniaxial compressive strength and point-load index tests (second series).

The results of first and second series of tests were evaluated statistically and the relationship between the compressive strength and point-load index have been investigated. The results obtained from these analyses are presented in Figures 5.5, 5.6, 5.7, 5.8, 5.9, and 5.10.

As shown in Figure 5.5. the best correlation coefficient was determined as 0.95 for both test types. The regression equations for these tests are as follows

$$(\sigma_c)_V = 16.57\ I_{s(50)} + 2.127 \quad (r=0.95) \qquad (5.1)$$

$$(\sigma_c)_H = 19.60\ I_{s(50)} - 1.50 \quad (r=0.95) \qquad (5.2)$$

Based on this study, it is suggested that the uniaxial compressive strength of rocks should be estimated by averaging the results obtained from both the diametrial and the axial tests. The direction of coring however should match the coring direction of the uniaxial compressive strength tests. This condition was indicated as dotted lines in Figure 5.5.

The compressive strength values obtained from various laboratory and point-load index tests are presented in Table 5.1.

The anisotropy index is defined as the ratio of mean $I_{s(50)}$ values measured perpendicular and parallel to planes of weakness, i.e. the ratio of greatest to least Point Load Strength Indices. I_a assumes values close to 1.0 for quasi-isotropic rocks and higher values when the rock is isotropic (ISRM, 1981).

As part of this study, the anisotropy index for various rock formations were also determined. During the calculations the results obtained from point load index Test Type I and Test-Type II were taken into account. However, the anisotropy index should be regarded as the "true" one, was difficult to decide. Anisotropy measured by diametral tests on cores drilled in two directions is favoured by the fact that the diametral test is the best controlled test, it is quite difficult to obtain cores in two directions.

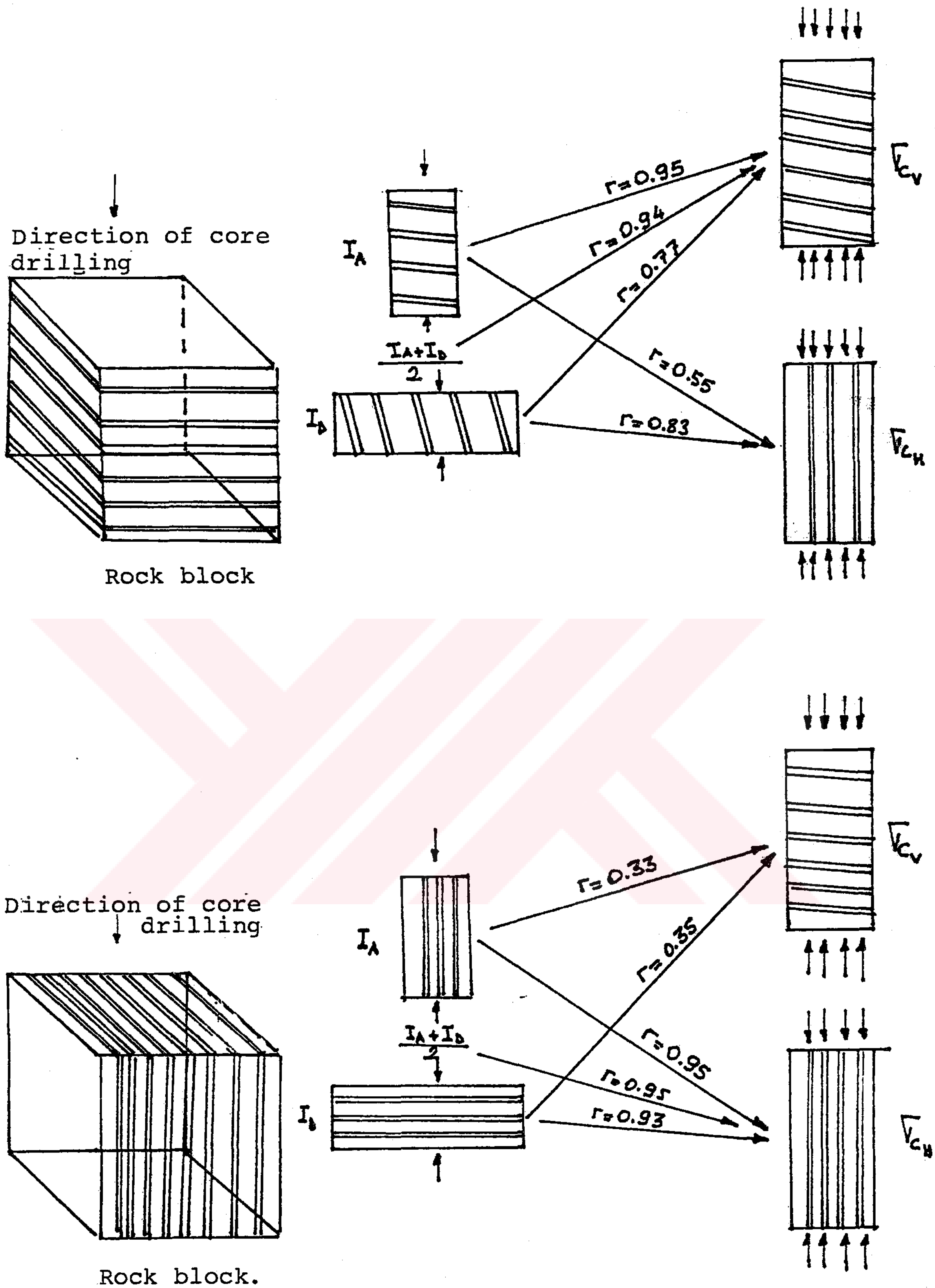


Figure 5.5. The correlation coefficients obtained from statistical evaluation of the compressive strength and point-load index test results

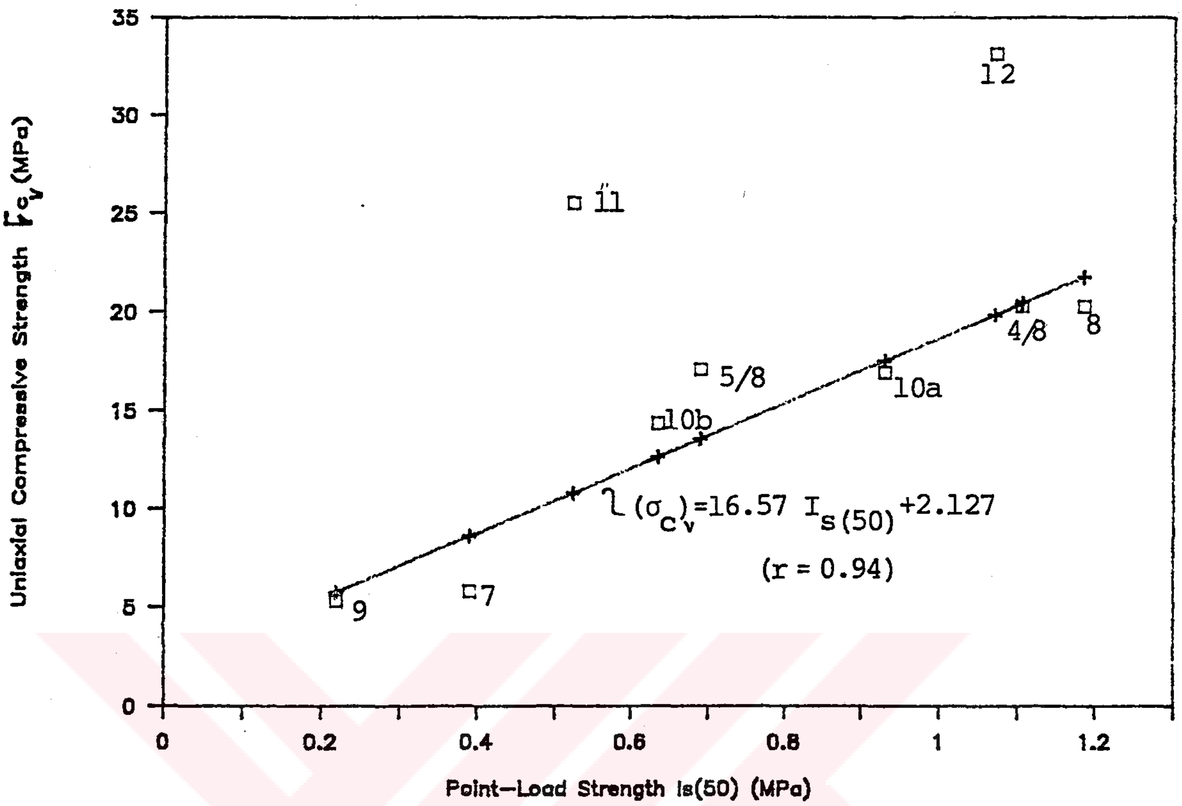


Figure 5.6. The relationship between the compressive strength $(\sigma_c)_v$ and point load index for test type-I

Note: 1) Data points associated with formation 11 and 12 (intermediate tuff and upper tuff). are not included.

2) If 11 and 12 included   r = 0.73.

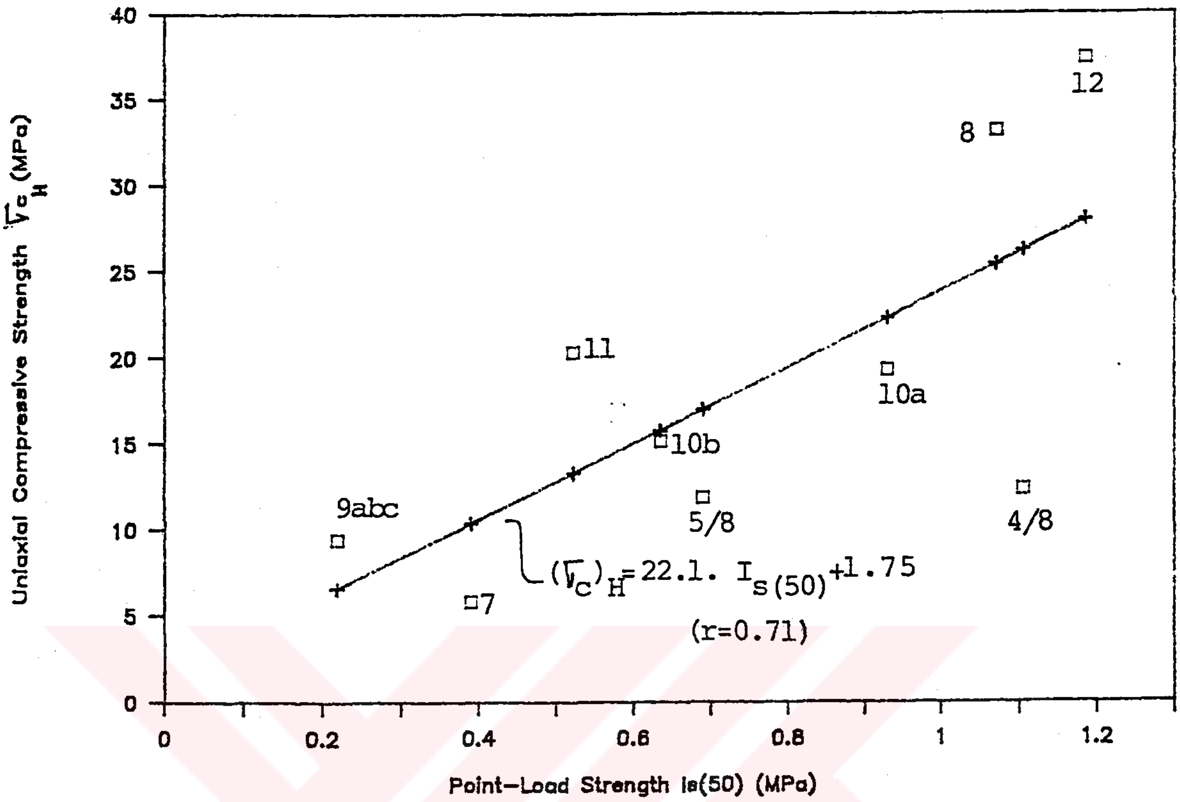


Figure 5.7. The relationship between the compressive strength $(\sigma_c)_H$ and point-load index for test type-I

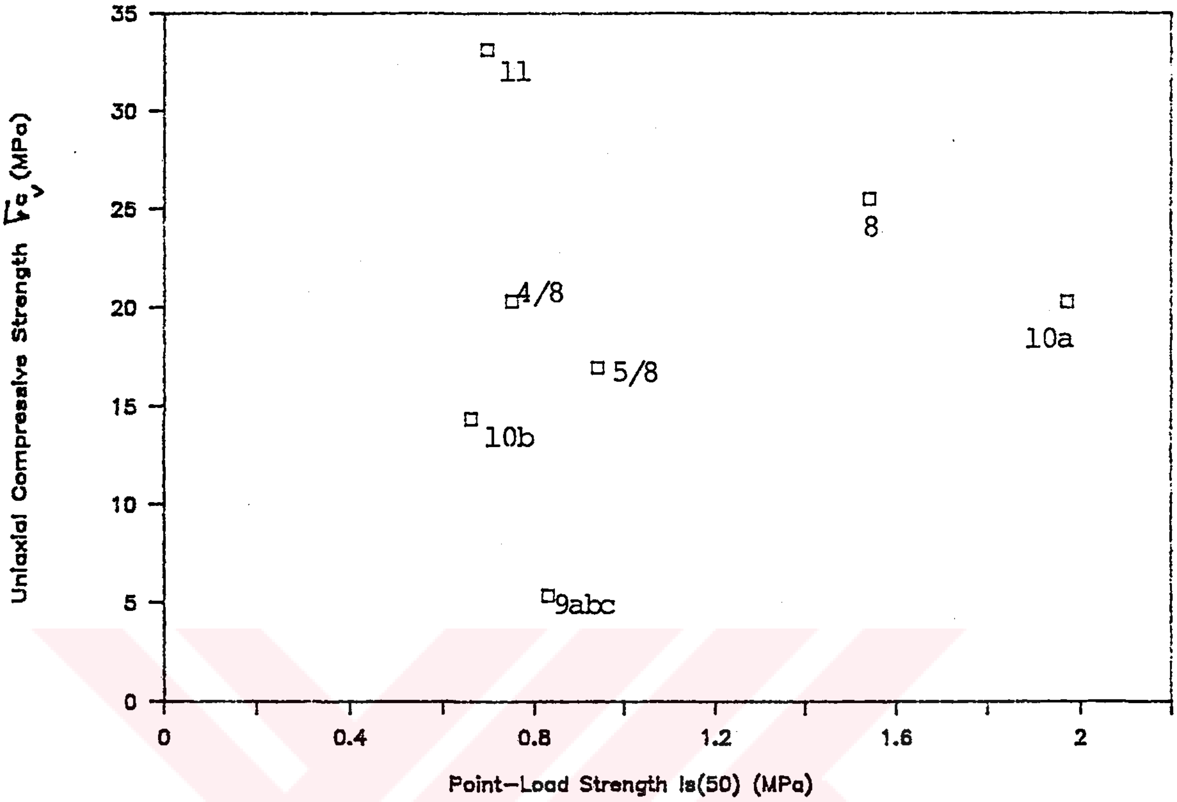


Figure 5. 8 . The relationship between the compressive strength $(\sigma_c)_v$ and point load index for test type-II

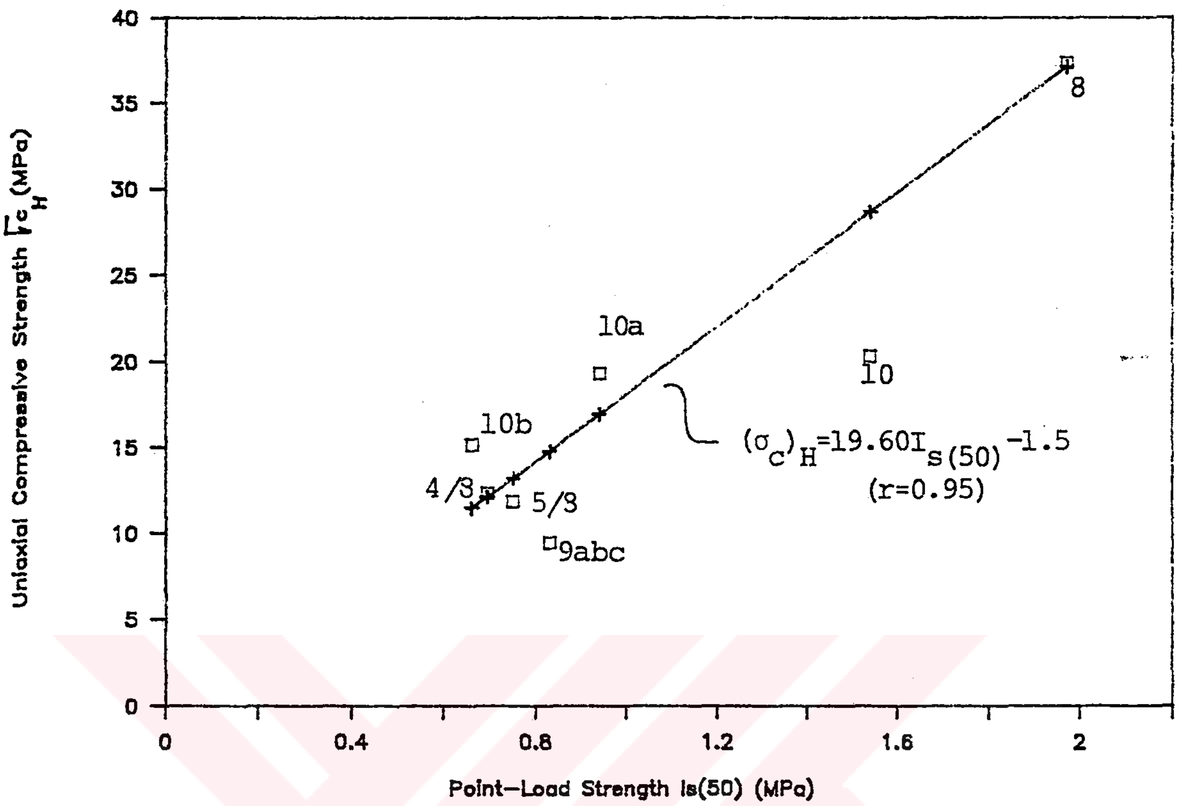


Figure 5. 9 . The relationship between the compressive strength $(\sigma_c)_H$ and point load index for test type-II

Note: 1) Data points associated 11(intermediate tuff) is not included

2) If 17 is included $r = 0.90$.

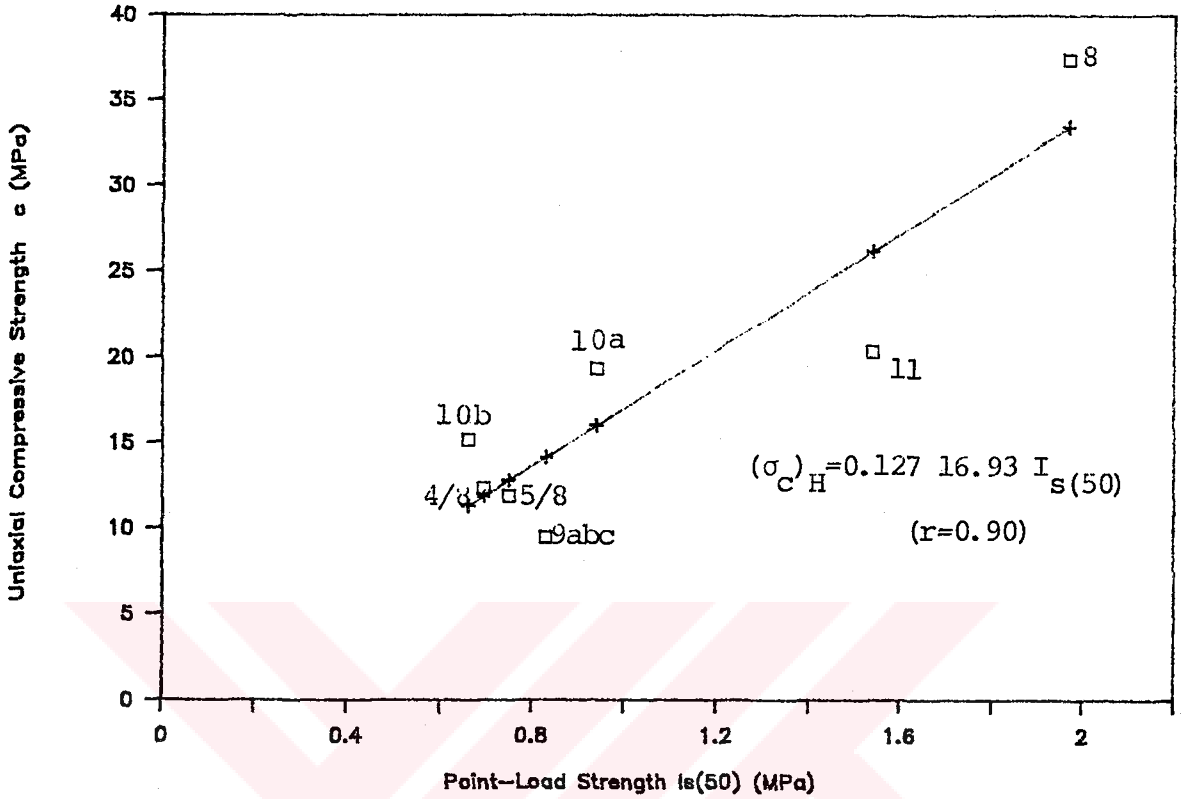


Figure 5.10. The relationship between the compressive strength $(\sigma_c)_H$ and point load index for test type-II.

Table 5.1. The compressive Strength Values Obtained from Laboratory and Point-Load Index Tests.

Rock Type	Actual $(\sigma_c)_v$ obtained from laboratory tests (MPa)	Actual $(\sigma_c)_{II}$ obtained from laboratory tests (MPa)	Actual $I_{s(50)}$ obtained from point-load test type - I (MPa)	obtained from point-load test type - II (MPa)	$(\sigma_c)_v$ From(4) (Equation) (MPa)	$(\sigma_c)_{II}$ From(5) (Equation) (MPa)
1	2	3	4	5	6	7
Colemanite (10a)	16.96(51)	19.323(2)	0.93(188)	0.94(17)	17.54	16.92
Ulexite (10b)	14.38(30)	15.18(4)	0.635(44)	0.66(23)	12.65	11.44
Limestone(8)	20.31(11)	37.38(2)	1.185(30)	1.97(8)	21.76	37.11
Wethered lmst.(9abc)	5.36(41	9.47(5)	0.218(267)	0.83(17)	5.74	14.77
Tuffite(7)	5.84(18)	-	0.39(58)	-	8.59	-
Int.tuff(11)	25.57(20)	20.35(15)	0.523(146)	1.54(29)	10.79	28.68
Upper tuff(12)	33.16(5)	-	1.07(24)	-	19.86	-
Limestone alterating with claystone (4/8)	20.33(47)	12.355(7)	1.055(244)	2.695(18)	20.45	12.12
Limestone with clay laminea (5/8)	17.12(5)	11.88(5)	0.69(84)	0.75(22)	13.56	13.2

* ( ) Number of tests performed

Favouring anisotropy measured by a combination of diametral and axial point-load tests on the same core is the fact that one then really knows that it is the same material that is tested in both direction. (Broch ,1983). Anisotropy index values recommendation by Broach and ISRM are presented in Table 5.2.

Table 5.2. Anisotropy Index Values for Test Types I and II anisotropy

Rock Type	ANISOTROPY INDEX VALUE OBTAINED FROM		
	Test Type I $I_a = \frac{I_A}{I_D}$	Test Type II $I_a = \frac{I_A}{I_D}$	$I_a = \frac{I_D}{I_D}$ $\frac{\text{(str. in the strongest dir)}}{\text{(str. in the weakest dir)}}$
Colemanite (10a)	1.62	1.098	1.27
Ulexite (10b)	1.76	2.10	0.92
Limestone (8)	1.78	1.15	2.15
Weathered limestone (9abc)	2.46	1.41	5.47
Tuffite (7)	1.71	-	-
Intermediate tuff (11)	1.61	1.57	3.0
Upper tuff (12)	2.75	-	
Limestone alternating with claystone (4/8)	3.11	1.73	0.95
Thinly laminated limestone with clay laminea (5/8)	3.6	1.047	2.45

Some rocks, for example shale, slate and schist, show directionally varying strength and deformation properties. Theoretically, anisotropic elastic materials can be treated as orthogonal complete materials. For anisotropic elastic materials the following relation is found between the elastic modulus and Poisson's ratio. (Lekhnitskii,1981).

$$\frac{E_1}{\nu_1} = \frac{E_{11}}{\nu_{11}} \tag{5.3}$$

Where ν_1 and E_1 are the Poisson's ratio and elastic modulus where the load is normal to the anisotropy plane; and ν_{11} and ν_{11} are for the case when load is applied parallel to the anisotropy plane.

Rocks have a different elastic modulus and Poisson's ratio in tension and compression. Ambartsumyan(1969) as reported by Jones (1975), has shown that isotropic materials with different moduli in tension and compression behave like ordinary anisotropic materials and the following relation is reported.

$$\frac{E_t}{\nu_t} = \frac{E_c}{\nu_c} \tag{5.4}$$

Where, ν_t, E_t are the tensile Poisson's ratio and elastic modulus; and ν_c, E_c are the compressive Poisson's ratio and elastic modulus.

Thus, E/ν is a useful compatative parameter for rocks, that also reflects any change in rock material and mass properties brought about by alteration, weathering or jointing (Turk and Dearman,1985).

The results obtained from the current study are presented in Figures 5.11. The results indicate quite high correlation coefficient for "ratio of Young's Modulus Poisson's ratio (E/r) - anisotropy index (I_a)" relationships.

In conclusion for weak and stratified rock Point-Load Strength tests are always recommended for classification purposes. However, conditions stated in this thesis should be considered. Ratings should be selected according to Point-Loid index values. The rating suggested by Bieniawski is quite sufficient.

5.4.3. Modificaton for Effect of Water in Clay-Bearing Strata

The slake-durability index has been developed for providing a quantitative measure of the relative breakdown from drying and wetting of argiellaceaus rocks. Durability tests predict the effects of weathering agents with short-term engineering significance. Of these, climatic slaking is undoubtedly the most widespread and therefore the most important. A slake-durability test predicts deterioration

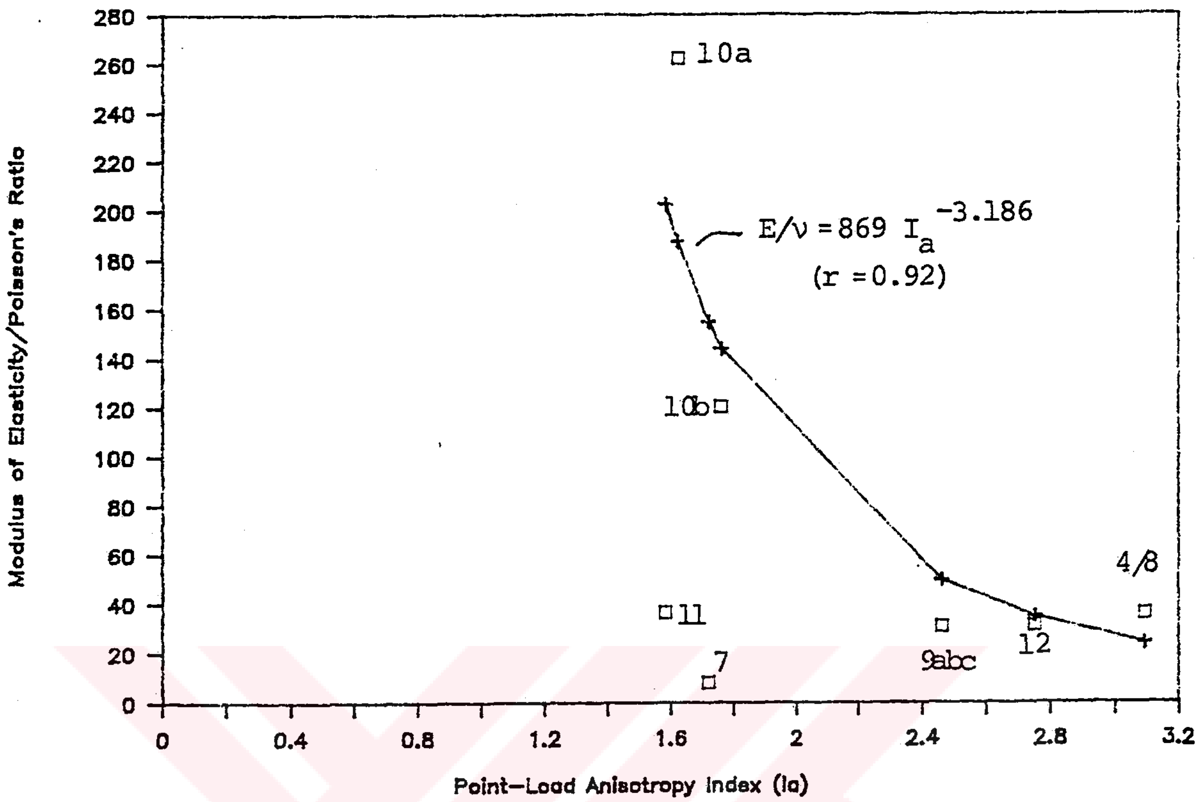


Figure 5.11. The relationship between the anisotropy index (J_a) and E/ν ratio in sedimentary rocks

Note: 1) Data points associated with formation 11 and 7 (intermediate tuff and tuffite) are not included.

2) If 11 and 7 is included r = 0.26.

due to climatic wetting and drying. The damage of clay-bearing rock due to effect of humidity can be expressed by the slake-durability index.

Among the parameters of the Geomechanics Classification, three of them, namely RQD, strength of the rock material and condition of joints are effected by the weathering process (Laubscher and Taylor,1976).

The RQD can be decreased by an increase in fractures as the rock deteriorates and the volume increases due to humidity effect. The strength of rock material will decrease, slightly in better quality rocks such as hard shale, and highly in weaker rocks such as claystone. The main influence of a deterioration due to wetting and drying process is on the condition of the discontinuities by alterations, either of the joint discontinuity, wall rock, and/or the joint filling. Increase in the width of the seperation between the wall joints resulted from a deterioration process, will decrease the quality of the rock mass(Rafia, 1980).

Utilizing the slake-durability test results, the relationship between a number of RMR parameters and slake-durability index (I_{d-2}) have been investigated. The associated equations and corresponding correclation coefficients are given in Table 5.3.

Table 5.3. Correlation Coefficient Values Between the Slake-Durability and RMR Parameters.

Regression Equation	Correlation Coefficient(r)
$\sigma_c = 0.0376\ Id_2 - 0.23$	$r = 0.83$
$RQD = 0.092\ Id_2 - 1.045$	$r = 0.66$

Where,

σ_c : Uniaxial Compressive Strength

JS : Joint Spacing

JC : Condition of Joint

RQD : Rock Quality Designation

Also, in the same conditions, correlation coefficients for RMR and Q classification systems have been determined as shown in Table 5.4.

Table 5.4. The Statistical Relationship Between the Slake-Durability Index and the RMR and Q Ratings

Regration Equation	Correlation Coefficient considering original RMR and Q values	Regration Equation	Correlation considering modified RMR and Q values
$RMR=0.231\ Id_2+12.98$	$r=0.67$	$RMR=0.593\ Id_2-8.05$	$r=0.87$
$Q=0.00886 \times e^{0.0564 Id_2}$	$r=0.81$	$Q=0.00113 \times e^{0.0679 Id_2}$	$r=0.86$

Evaluation of the statistical data indicate that the statistical relationship between the slake durability index (Id_2) and "RMR" and "Q" index values has the highest correlations coefficient. The effect of weathering on RQD, condition of joint (JC) and uniaxial compressive strength of the rock was also observed as explained by Laubscher and Taylor (1976). However, the relationship between Uniaxial Compressive Strength (σ_c) and Point-Load index ($I_{s(50)}$) indicated the highest correlation coefficient. (r=0.83). The relationship between the slake-durability index the parameters such as the σ_c, RQD, JS, JC, RMR and Q are shown in Figures 5.12, 5.13, 5.14, 5.15, 5.16 and 5.17. respectively.

In order to eliminate the inconsistency between the RMR and Q-systems (as will be shown later in this chapter) the weatherability has been thought as a new parameter combined with "underground-water condition" index in RMR-System. Wheatherability index can only be obtain from slake durability test results.

The details associated with the weatherability index and utilization of this index with underground water conditions are explained in Tables 5.5 and 5.6.

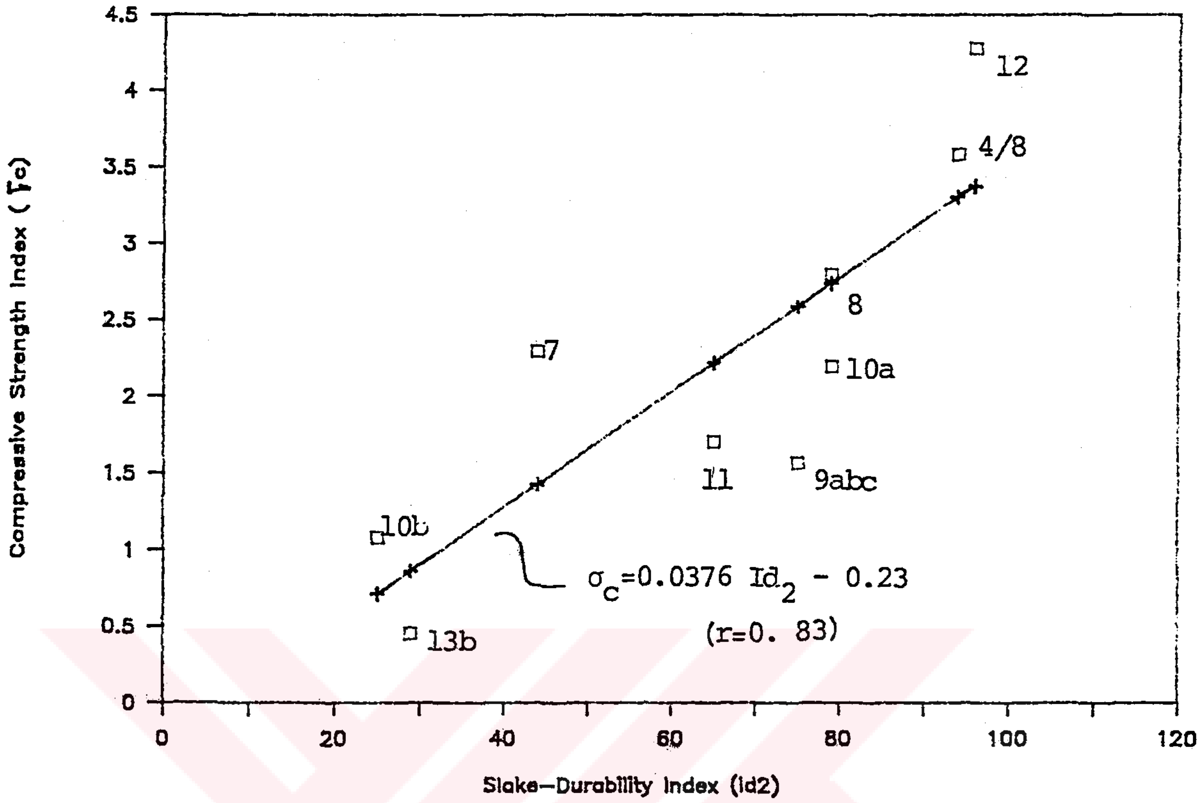


Figure 5.12. The relationship between the slake-durability index values and uniaxial compressive strength (σ_c) in RMR system

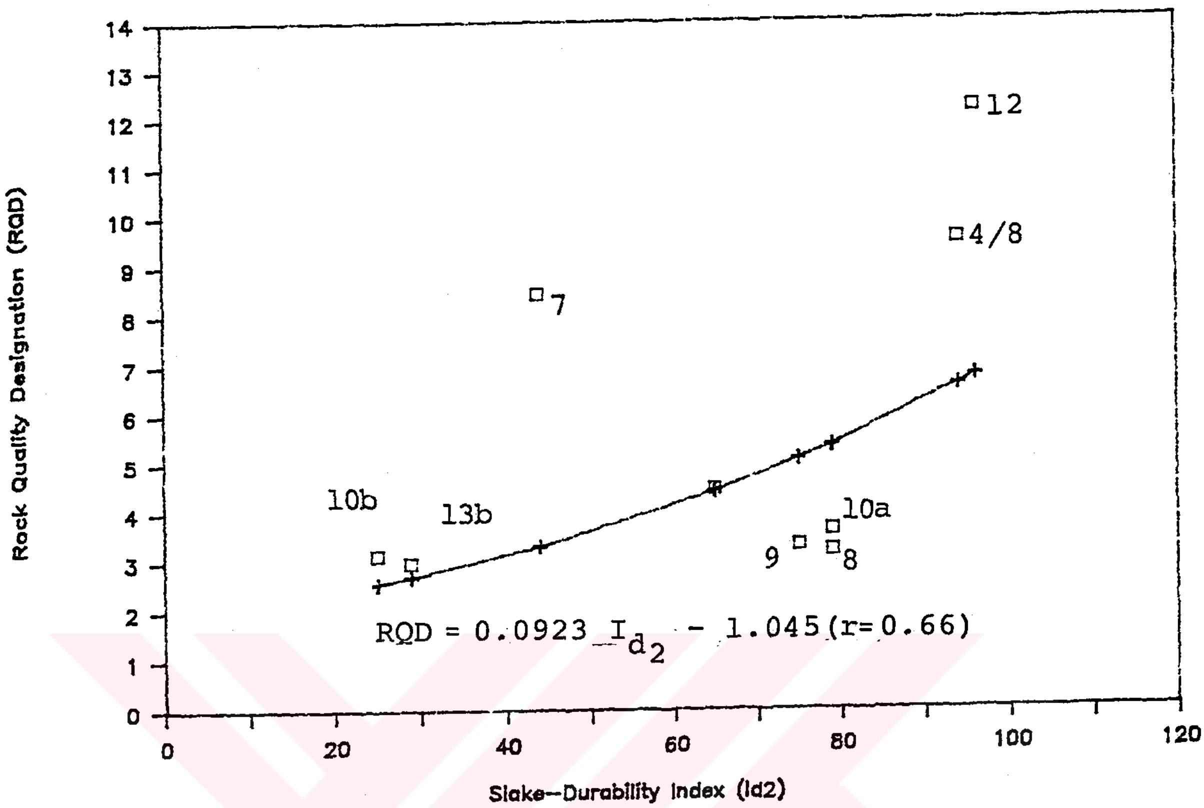


Figure 5.13 . The relationship between the slake-durability index values and RQD in RMR system.

Note: 1) Data points associated 7 (tuffite) is not included.

2) If 7 is included $r = 0.49$.

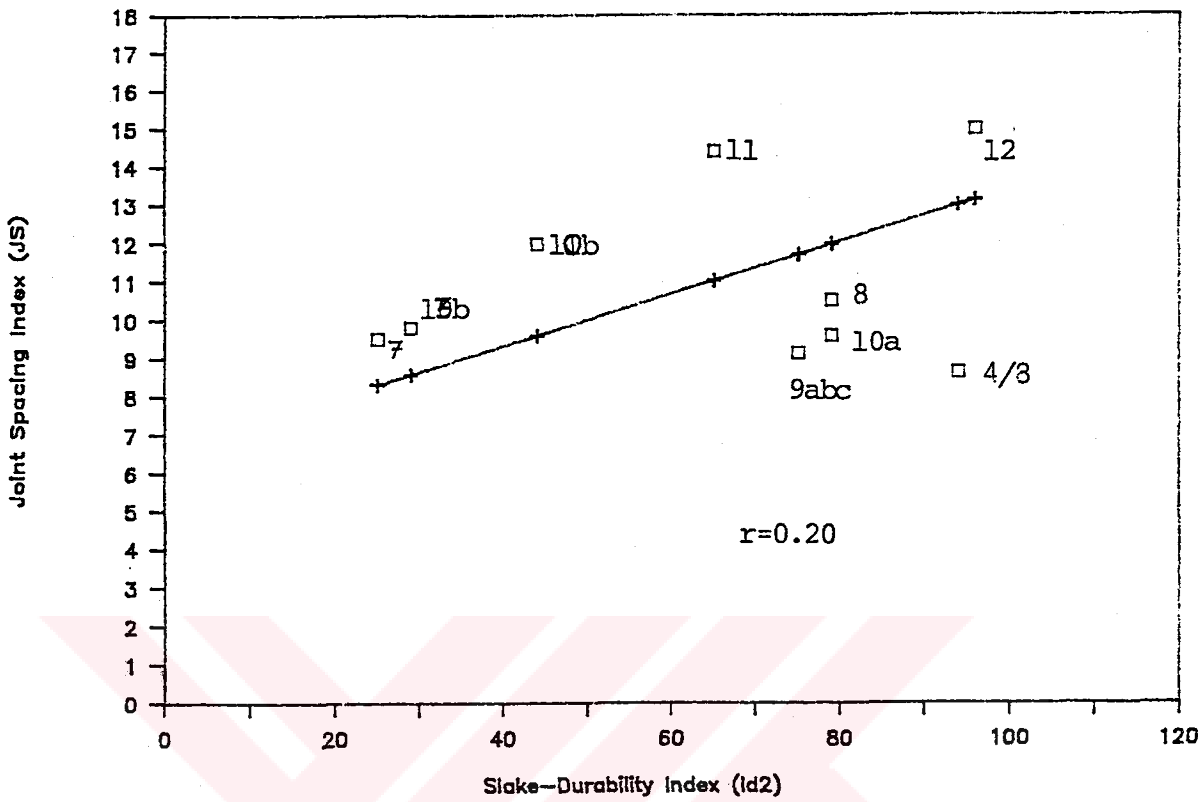


Figure 5.14. The relationship between the slake-durability index and joint spacing values as used in RMR system

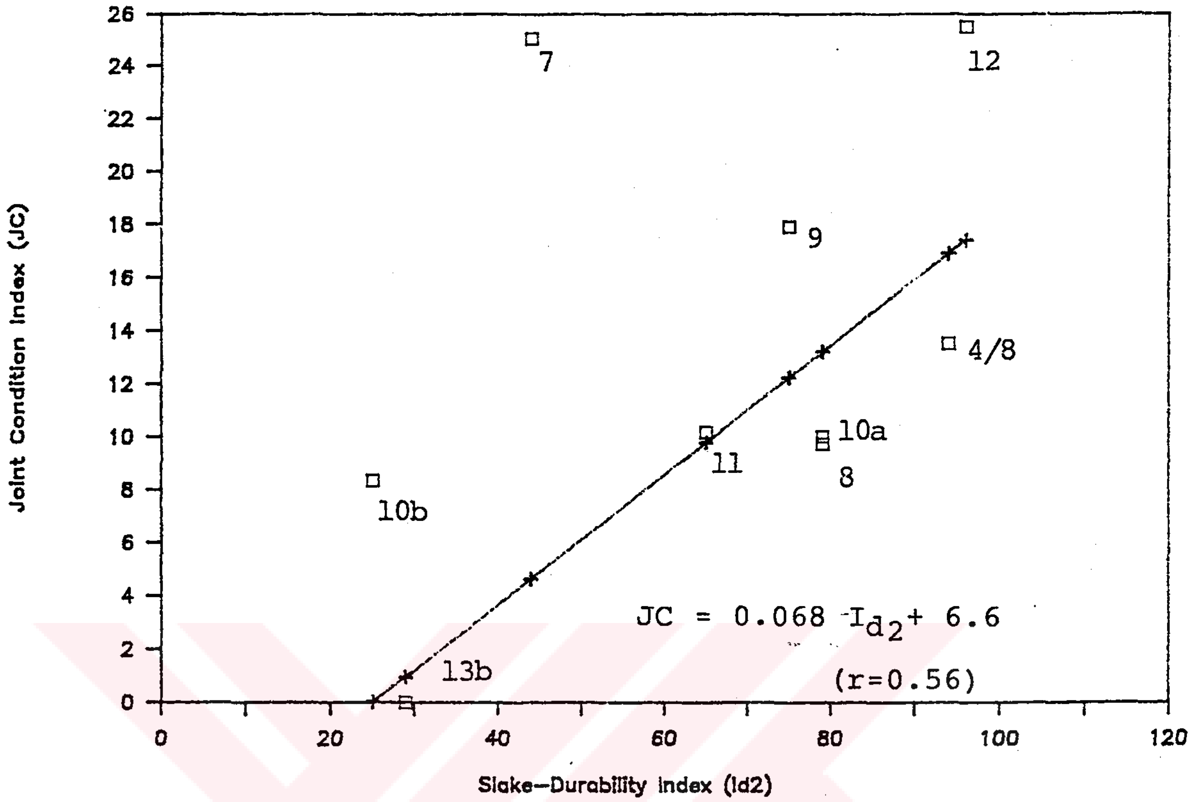


Figure 5.15. The relationship between the slake-durability index and joint condition values as used in RMR system

Note: 1) Data points associated with formation 7 (tuffite) is not included.

2) If 7 is included r = 0.44

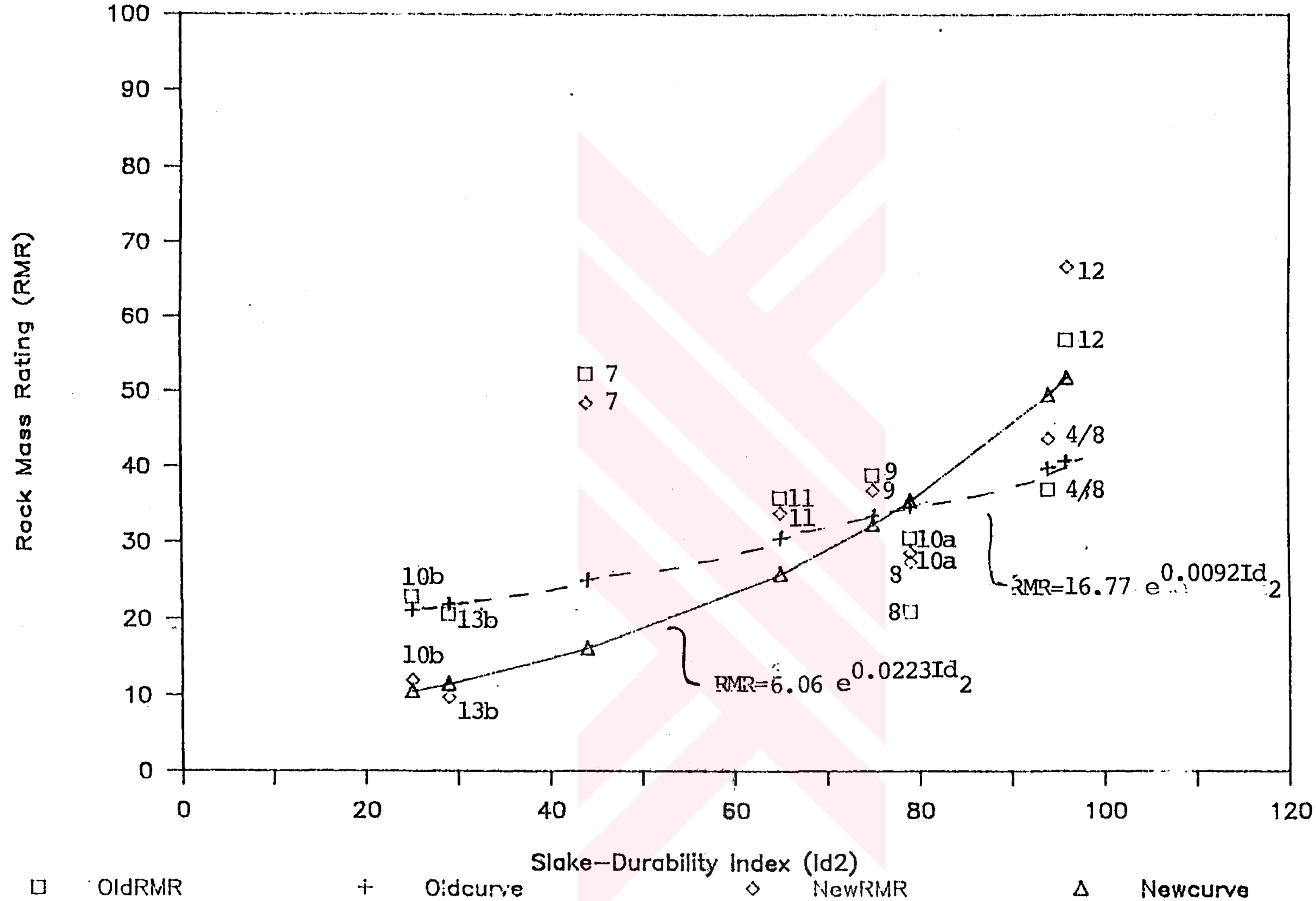


Figure 5.16 . The relationship between the slake-durability index and RMR values

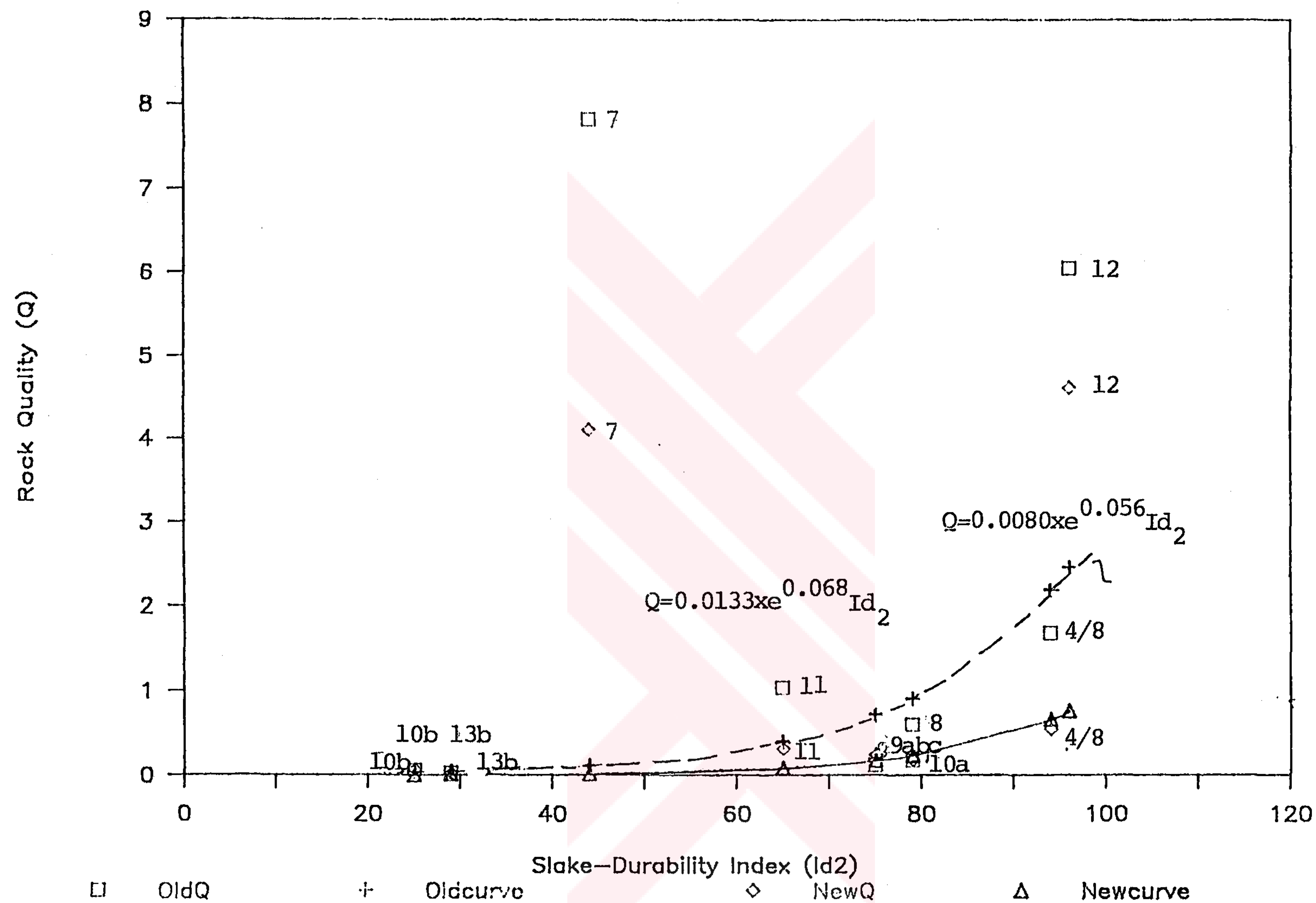


Figure 5.17. The relationship between the slake-durability index and Q values

Table 5.5. Rating Based on Slake-Durability Test Results

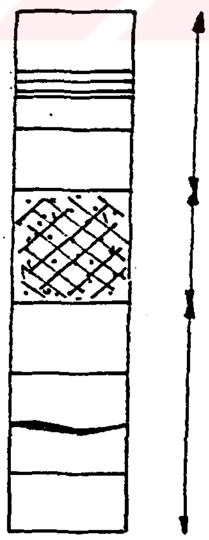
Slake durabilit(%)	Rating	Description
0-25	5	Rock mass is highly weathered
25-50	4	Rock mass is weathered
50-85	3	The filling meterials in joints are weathered
85-95	2	The consolidated filling is weathered weakly
95-100	1	Eventhough joint walls exist water can not influence those.

Table 5.6. Combined "Slake Durability" and "Underground Water Conditions" Ratings

Water Flow	Ratings None					<10 litres/min					10 - 25 litres/min					25 - 125 litres/min					> 125 litres/min				
Ratings	15					10					7					4					0				
Rating for slake durability	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
Rating for combined effect	15	15	15	15	15	15	14	10	5	0	12	8	4	0	-3	10	5	0	-3	-5	7	0	-3	-5	-7

5.4.4. Joint Spacing (JS)

From the cores, joint spacing determinations were become very difficult, especially in weak and stratified rocks. At the begining, of the current investigations, all of the cracks and fractures appearing on cores were assumed as planes of weakness and counted as joints, while determining the joint spacings. It was realized afterwards that, some of those cracks were man made or occured in core boxes afterwards. By careful analysis of the core boxes and photographs taken, this type of cracks were identified and disragarded during evaluations. Small fracture zones or fillings or weathering zones located within the same type of formation or seperating two different type of formations were counted as one plane of weakness each. This procedure is exhibited in Figure 5.18.



	Number of joints should not be counted as	Number of joints be counted as
Originally Intact core, but seperated in the core boxes	6	1
Shear zone or Highly bedded core piece or clay filling	16	1
Weathering zone.	4	2 (due to from weathering)

Figure 5.18. Determination of number of joints on cores.

5.4.5. Condition of Discontinuities (JC)

In the RMR-System, the terms separation, continuity, roughness, alteration and filling of joints have not been defined clearly. In determining the condition of discontinuities these parameters have not been weighed adequately either in this study, for evaluation of the parameter associated with the condition of discontinuities, the rating system suggested by Ünal (1985) was used. In order to calculate the index value for the condition of discontinuities (JC) Equation 5.5. was considered.

$$JC \quad A\ P\ (Y\ Z\ D) \tag{5.5}$$

Where, A is the weathering, P is the roughness, Y is the continuity, Z is the aperture and D is the filling. According to the rating systems Suggested by Ünal, the joint-condition parameters are grouped into various ranges. Finally importance ratings are allocated to different value-ranges of the parameters. The total rating range for JC is between 0-30 as also suggested by RMR-System.

5.5. Suggested Modifications in Q-System

The modifications suggested in Q-System are associated with determination of RQD, Joint Set Number(Jn) and Joint Alteration Number (Ja).

5.5.1. Rock Quality Designation (RQD)

The suggestion made for Rock Quality Designation (RQD) used in RMR dissucced (Section 5.4.1.) are also valid in Q-System.

5.5.2. Modification of Joint Set Number (In)

In weak and stratified rock mass, it is not easy to determine the number of joint sets directly from core boxes. This process becomes easier incase RQD and Intact Core Recovery (ICR) values are very high and/or very low. If RQD and ICR values are moderate, it is quite difficult to judge on the number of joint sets. In order to avoid this difficulty a new approach was suggested to determine the joint set number from cores as presented in Table 5.7.

5.5.3. Modification of Joint Alteration Number (Ja)

In determing the joint alteration number (Ja) in Q-System, the conditions have been catagorized into three groups. Most of these conditions include filling parameter in determining the joint alteration -number, except the first few conditions in Group 1 (rock wall contact). If, however, the filling material the Ja values are taken as 0.75 or 1 according to Barton (1974). Based on the experience gained in this study, the following recommendations can be made for the rock joints which

Table 5.7. The rating-index values for Joint Set Number in Weak and Stratified Rock.

ICR (%)	RQD (%)	Joint Set Number
90<ICR<100	RQD>90	2
75<ICR<90	25<RQD<90	3
	10<RQD<25	5
50<ICR<75	25<RQD<75	6
	10<RQD<25	7
10<ICR<50	25<RQD<50	9
	10<RQD<25	12
10<ICR<50	0<RQD<10	15
ICR<10	0<RQD<10	20

do not contain filling material:

- Unaltered joint walls Ja=1
- Slightly alterated joint walls Ja=3
- Alterated joint walls Ja=5

Alteration number of 0.75 (Ja 0.75) should not be used for weak and stratified rocks.

5.6. Comparision of Original and Modified RMR and Q Values

Based on the suggestions made in this thesis, the modified RMR and Q values were determined compeared. The original and modified ratings determined for various formations are presented in Table 5.8.

The regression equations and the associated correlation coefficients obtained from statistical evaluation of the data are given in Table 5.9.

The relationship between the RMR and Q ratings for the original and modified conditions are presented un Figures 5.19 and 5.20. As can been from Figures 5.19 and 5.20, the regression line in modified conditions is closer to the line suggest by Bieniawski as compared with the line obtained by original conditions. The regression coefficients (r=0.99 and r=0.94) in the modified conditions are higher than the original conditions (r=0.94 and 0.84).

Table 5.8. Comparision of Original and Modified RMR and Q Values.

Formations	Thickness (m)	In Original Classification			In Modified Classification			Description of the Formations
		RMR	Q	RMR Obtained from 9'lnQ + 44	RMR	Q	RMR Obtained from 9'lnQ + 44	
1.LAYER	5.60	30	0.840	42	34	0.54	38	Weathered Limestone interbedded with Claystone and Tuff
2.LAYER	1.90	24	0.233	31	26	0.1298	26	Limestone alternating with Claystone and repeated Limestone
3.LAYER	0.90	28	0.374	35	30	0.34	34	Limestone interbedded with Claystone and Tuff
1.SEAM	3.80	32	0.725	41	32	0.512	38	COLEMANITE
4.LAYER		16-46	0.15 0.98		7-41	0.018 0.44		Weathered Limestone interbedded with Claystone and Tuff
5.LAYER	3.45	24 12-32	0.213 0.42 0.51	30	23 12-23	0.176 0.03 0.125	28	Limestone alternating with Claystone and interbedded with Tuff
6.LAYER		12-32	0.02 0.31		14-33	0.016 0.297		Limestone alternating with Claystone and random COLEMANITE
2.SEAM	5.60	33	1.097	45	32	0.295	33	COLEMANITE

Table 5.8. (Continued)

Formations	Thickness (m)	In Original Classification RMR		Q		RMR Obtained from 9'lnQ + 44	In Modified Classification RMR		Q		RMR Obtained from 9'lnQ + 44	Description of the Formations
7.LAYER	4.10	23	19-40	0.471	0.096 0.482	37	23	8-42	0.180	0.012 0.79	29	Limestone alternating with Claystone and random COLEMANITE
8.LAYER			13-40		0.016 0.202			20-44		0.044 0.202		Limestone interbedded and Laminated Claystone
9.LAYER			18-33		0.018 0.317			9-18		0.015 0.045		Claystone and Limestone interbedded with Tuff
3.SEAM	7.30	29		0.325		34	23		0.120		25	COLEMANITE
10.LAYER	5.00	28	13-24	0.341	0.013 0.457	34	28	15-38	0.220	0.015 0.264	30	Limestone alternating with Claystone and randomly interbedded Lst. and Clst.
11.LAYER			13-30		0.015 0.45			-		- -		Limestone alternating with Claystone
12.LAYER			13-40		0.045 0.603			18-39		0.015 0.728		Limestone interbedded with hard Claystone
4.SEAM	5.00	31		0.978		44	29		0.210		30	ULEXITE

Table 5.8. (Continued)

<table>
<tr><th rowspan="2">Formations</th><th rowspan="2">Thickness (m)</th><th colspan="5">In Original Classification</th><th colspan="5">In Modified Classification</th><th rowspan="2">Description of the Formations</th></tr>
<tr><th colspan="2">RMR</th><th colspan="2">Q</th><th>RMR Obtained from 9'lnQ + 44</th><th colspan="2">RMR</th><th colspan="2">Q</th><th>RMR Obtained from 9'lnQ + 44</th></tr>
<tr><td>13.LAYER</td><td rowspan="2">4.37</td><td rowspan="2">30</td><td>18-44</td><td rowspan="2">0.692</td><td>0.032
1.39</td><td rowspan="2">41</td><td rowspan="2">25</td><td>5-44</td><td rowspan="2">0.190</td><td>0.017
1.40</td><td rowspan="2">29</td><td>Limestone alternating with Claystone</td></tr>
<tr><td>14.LAYER</td><td>11-54</td><td>0.003
1.91</td><td>11-37</td><td>0.003
0.51</td><td>Claystone interbedded Limestone</td></tr>
<tr><td>5.SEAM</td><td>3.20</td><td colspan="2">35</td><td colspan="2">0.267</td><td>32</td><td colspan="2">33</td><td colspan="2">0.330</td><td>34</td><td>COLEMANITE</td></tr>
<tr><td>15.LAYER</td><td>9.65</td><td colspan="2">37</td><td colspan="2">0.963</td><td>41</td><td colspan="2">-</td><td colspan="2">-</td><td>-</td><td>Tuff, limestone interbedded with Claystone,
Limestone alternating with Claystone</td></tr>
<tr><td>INTERMEDIATE TUFF</td><td>1.00</td><td colspan="2">56</td><td colspan="2">20.33</td><td>71</td><td colspan="2">66</td><td colspan="2">15.78</td><td>69</td><td>Intermediate tuff</td></tr>
<tr><td>16.LAYER</td><td>5.45</td><td colspan="2">42</td><td colspan="2">1.20</td><td>46</td><td colspan="2">-</td><td colspan="2">-</td><td>-</td><td>Tuff and weathered Limestone interbedded with Claystone</td></tr>
<tr><td>UPPER TUFF</td><td>2.80</td><td colspan="2">57</td><td colspan="2">21.54</td><td>72</td><td colspan="2">65</td><td colspan="2">18.50</td><td>65</td><td>Upper Tuff</td></tr>
</table>

Table 5.9. Regressions Equations and Correlation Coefficient Values Between the RMR and Q-Systems.

Data Points Considered	Regression Equations Original condition	Modified Condition
All data points obtained from five boreholes	$RMR=6.74\ell_nQ+34.29$ (r = 0.94)	$RMR=9.19\ell_nQ+41.28$ (r = 0.99)
Weighted average of each formatin	$RMR=5.88\ell_nQ+37$ (r = 0.84)	$RMR=7.47\ell_nQ+42.63$ (r = 0.94)

In the remaining part of this chapter, the outputs obtained from RMR and Q-Systems will be discussed based on the original and modified rating values. The outputs associated with the classification systems include the following:

1) Original and modified RMR and Q value and description of the rock mass;
2) Maximum unsupported span (B);
3) Unsupported span versus stand-up time relationship;
4) Rock-load height (h_t);
5) Support pressure (P);
6) Residual and peak friction angle (Ø);
7) m and s material constants ;
8) In-situ Modulus of Elasticity .

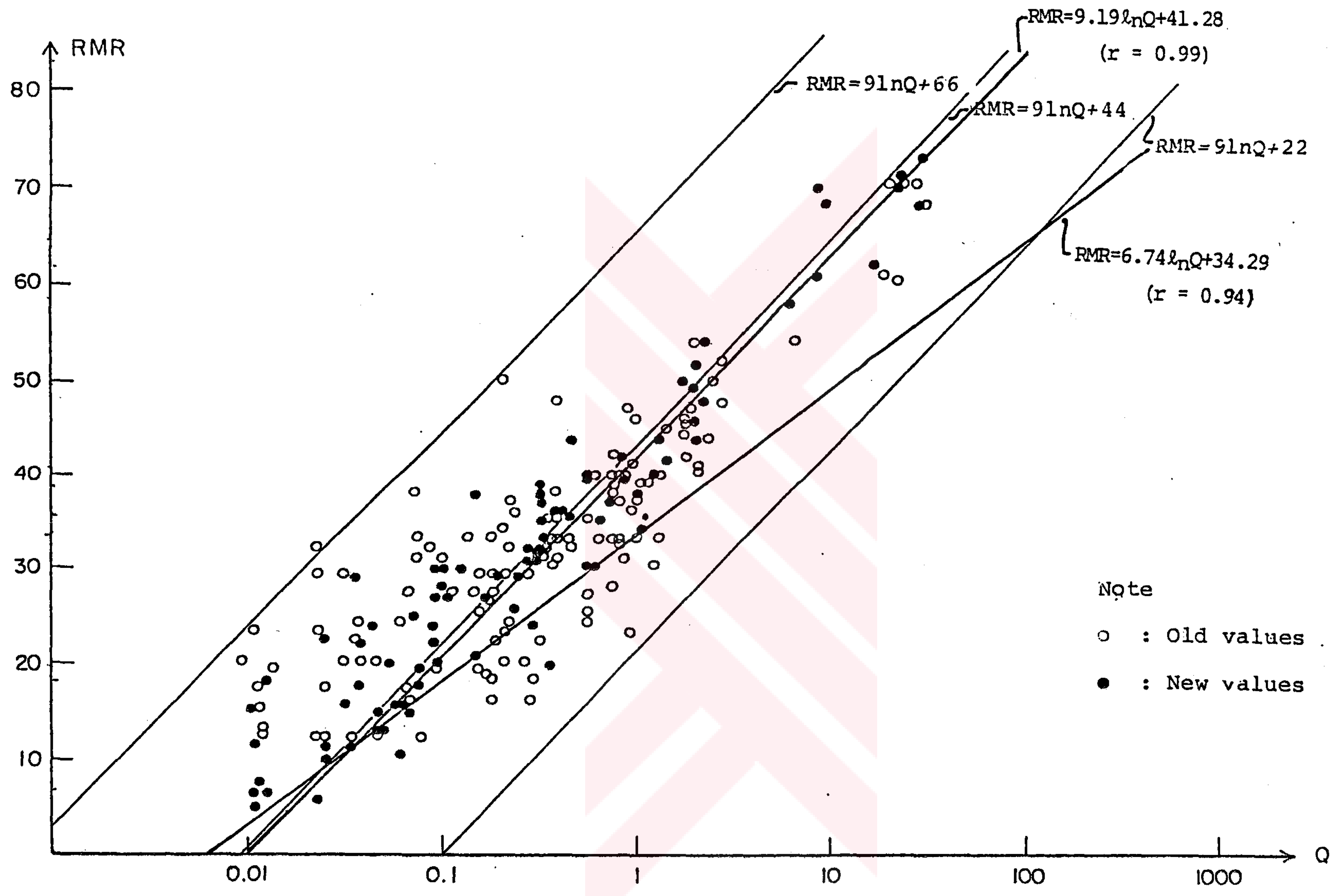


Figure 5.19. The relationship between the original and modified RMR and Q values (all data point obtained from boreholes are included).

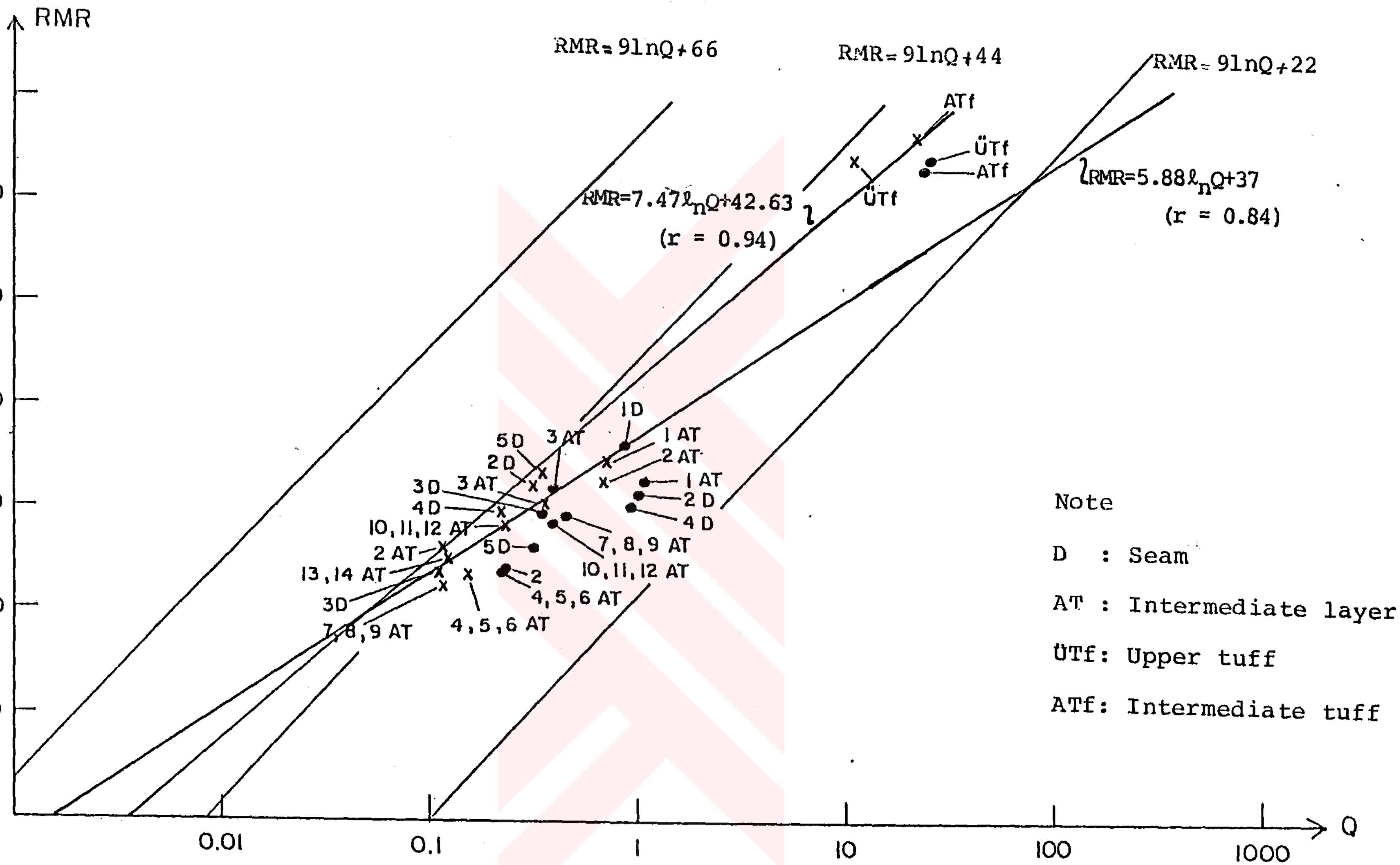


Figure 5.20. The relationship between the original and modified RMR and Q values (Weighted average of five boreholes are considered).

The output of the classification systems were compared in Tables 5.10, 5.11, 5.12, 5.13, 5.14, 5.15, 5.16, 5.17, 5.18,

As can be seen in Table 5.10 there were not much difference between the original and modified RMR values. Consequently the describtions of the formations remained to be the same except the last one (upper tuff) which was changed from fair rock to good rock. Also, in Q system descriptions of the formations remainedto the same except the 2nd seam which was changed from poor rock to very poor rock.

The results of maximum unsupported span values in both original and modified RMR and Q-systems are compared with are tabulated in Table 5.11 and 5.12.

For determination of maximum unsupported spans in RMR-System Figure 4.3, provided earlier, was utilized. For estimating the unsupported spans in Q-System. Equations 4.2 and 4.3 (see also Figures 4.4 and 4.6) were used.

In general, the width of the gate roadways excavated in underground mine were 2-2.5 meters. The roadways had been supported with wooden post and beams. Observations carried out in the underground mine indicated that leaving unsupported spans of 2-2.5 meters were not possible. The classification results obtained from the original and modified RMR-Systems indicated that

Table 5.10 Results of Comparison of RMR and Q-Systems

Roadway	Rock Type	Rock Mass Quality Index RMR				Rock Mass Quality Index Q			
		Original RMR	Description	Modified RMR	Description	Original Q	Description	Modified Q	Description
Above seam No.1	weathered limestone interbeded with claystone and tuff	28	Poor rock	33	Poor rock	0.65	Very Poor rock	0.49	Very poor rock
in seam No.1	Colemanite	32	"	32	"	0.61	"	0.454	"
Above seam No.2	Limestone alternating with claystone and tuff	24	"	23	"	0.213	"	0.166	"
in seam No.2	Colemanite	33	"	32	"	1.097	Poor rock	0.295	"
Above seam No.3	Limestone alternating with claystone	23	"	23	"	0.471	Very Poor rock	0.18	"
in seam No.3	Colemanite	29	"	24	"	0.325	"	0.12	"
Above seam No.4	Limestone alternating with claystone	28	Poor rock	28	Poor rock	0.341	"	0.22	Very poor rock
in seam No.4	Ulexite	31	"	29	"	0.978	"	0.21	"
Above seam No.5	Limestone alternating with claystone	30	"	25	"	0.692	"	0.19	"
in seam No.5	Disintegrated colemanite	35	"	33	"	0.460	"	0.33	"
in upper tuff	Upper tuff	57	Fair rock	66	Good rock	21.54	Good rock	18.5	Good rock

Table 5.11 Estimated Maximum Unsupported Span and Stand-up time for Different Roof Conditions at Bigadiç Mine (by RMR Classification System)

Roadway	Rock Type	Roof Quality Index RMR		Maximum Unsupported Span (B)		Stund-up time	
		Original RMR	Modified RMR	B, from Original RMR	B, from Modified RMR	Original RMR	Modified RMR
Aboveseam No.1	Weathered limestone interbeded with claystone and tuff	28	33	1.1	1.4	9hours	30 hours
in seam No.1	Colemanite	32	32	1.4	1.3	25 hours	20 hours
Above seam No.2	Limestone alternating with claystone and tuff	24	23	1.0	0.9	4 hours	3.5 hours
in seam No.2	Colemanite	33	32	1.4	1.4	30 hours	25 hours
Aboveseam No.3	Limestone alternating with claystone	23	33	0.9	0.9	3.5 hours	3.5 hours
in seam No.3	Colemanite	29	24	1.2	0.9	13 hours	4 hours
Aboveseam No.4	Limestone alternating with claystone	28	28	1.1	1.1	9 hours	9 hours
in seam No.4	Ulexite	31	29	1.3	1.2	20 hours	13 hours
Aboveseam No.5	Limestone alternating with claystone	30	25	1.25	1.05	16 hours	5.5 hours
in seam No.5	Disintegrated colemanite	35	33	1.6	1.4	40 hours	30 hours
in upper tuff	Upper tuff	57	66	2.1	2.8	4 month	15 month

Table 5.12. Estimated Maximum Unsupported Span for Different Roof Conditions of Bigadiç Mine (by Q Classification System)

Roadway	Rock Type	Roof Qualitiy Index Q		$B=2\ ESR\ Q^{0.40}$ (m)		$B=2Q^{0.66}$ (m)	
		Original Q	Modified Q	Original Q	Modified Q	Original Q	Modified Q
on seam No.1	Weathered limestone interbeded with claystone and tuff	0.65	0.49	2.69	2.40	1.50	1.25
in seam No.1	Colemanite	0.61	0.544	2.62	2.33	1.44	1.19
on seam No.2	Limestone alternating with cloystone and tuff	0.213	0.166	1.78	1.56	0.72	0.61
in seam No.2	Colemanite	1.097	0.295	3.32	1.96	2.13	0.89
on seam No.3	Limestone alternating with claystone	0.471	0.18	2.37	1.61	1.58	0.64
in seam No.3	Colemanite	0.325	0.12	2.04	1.37	0.95	0.50
on seam No.4	Limestone alternating with claystone	0.341	0.22	2.1	1.74	0.98	0.74
in seam No.4	Ulexite	0.978	0.21	3.17	1.71	1.97	0.71
on seam No.5	Limestone alternating with claystone	0.692	0.19	2.76	1.65	1.57	0.67
in seam No.5	Disintegrated Colemanite	0.460	0.33	2.34	2.05	1.19	0.96
in upper tuff	Upper tuff	21.54	18.5	10.93	10.28	15.16	13.72

unsupported roof spans in Bigadiç Mine could be excavated, up to 0.9-1.6 meters. On the other hand, according to the original and modified Q-Systems (Equation 4.2) excavation of unsupported gateroads having spans of 1.78-3.32 meters and 1.37-2.33 meters were possible. The results obtained especially from the original Q-System are contradictory to the observations made in underground mine. Based on this study, and underground observations it was concluded that in weak and stratified rocks, RMR-System predict more realistic values for unsupported spans.

The rock load height values for RMR system were obtained from Equation 4.3 suggested by Ünal (1983,1986). The result obtained in this study are presented in Table 5.13. THe rock load height values in original and modified conditions are between 1.63-1.93 and 1.68-1.93 respectively.

The support pressure expected on openings excavated in various formations, in both the original and modified RMR and Q-Systems, are tabulated in Tables 5.14 and 5.15. The support pressure values estimated by RMR-system were considered less than estimated by Q-system in both original and modified conditions.

Table 5.13. Rock-Load Heights Calculated Using the Original and Modified RMR Values

Roadway	Rock Type	Roof quality index RMR		Rock load height (m)	
		Original RMR	Modified RMR	h_t from Original RMR	h_t from Modified RMR
Aboveseam No.1	weatheredlimestone interbeded with claystone and tuff	28	33	1.8	1.67
in seam No.1	Colemanite	32	32	1.7	1.73
Aboveseam No.2	Limestone alternating with claystone and tuff	24	23	1.9	1.93
in seam No.2	Colemanite	33	32	1.68	1.70
Aboveseam No.3	Limestone alternating with claystone	23	23	1.93	1.93
in seam No.3	Colemanite	29	24	1.78	1.91
Aboveseam No.4	Limestone alternating with claystone	28	28	1.8	1.8
in seam No.4	Ulexite	31	29	1.73	1.78
Aboveseam No.5	Limestone alternating with claystone	30	25	1.75	1.87
in seam No.5	Disintegrated colemanite	35	33	1.63	1.68
in upper tuff	Upper tuff	57	66	1.85*	1.29*

* B = 4.3 m

Table 5.14. Support Pressures Estimated Based on Ünal's (1983) Equation.

Roadway	Rock Tpe	Rock Quality,Index RMR		Support Pressure P(kPa)	
		Original RMR	Modified RMR	Original RMR	Modified RMR
Above seam No.1	Weathered limestone interbeded with claystone and tuff	28.	33	41.4	38.3
in seam No.1	Colemanite	32	32	39.0	39.1
Above seam No.2	Limestone alternating with claystone and tuff	24	23	43.7	44.4
in seam No.2	Colemanite	33	32	38.6	39.1
Above seam No.3	Limestone alternating with claystone	23	23	44.4	44.4
in seam No.3	Colemanite	29	24	41.0	44.0
Above seam No.4	Limestone alternating with claystone	28	28	41.4	41.4
in seam No.4	Ulexite	31	29	39.8	41.0
Above seam No.5	Limestone alternating with claystone	30	25	40.2	43.0
in seam No.5	Disintegrated colemanite	35	33	37.5	38.6
in upper tuff	Upper tuff	57	66	42.5	29.7

Table 5.15. Support Pressures Estimated Based on Q-System.

Roadway	Rock Type	Rock Quality, Index Q		Support Pressure P (kPa)	
		Original RMR	Modified Q	Original Q	Modified Q
Above seam No.1	Weatheredlimestone interbeded with claystone and tuff	0.65	0.49	770	84.0
in seam No.1	Colemanite	0.61	0.454	78.0	87.0
Above seam No.2	Limestone alternating with claystone and tuff	0.213	0.166	111.6	121.0
in seam No.2	Colemanite	1.097	0.295	64.6	101.0
Above seam No.3	Limestone alternating with claystone	0.471	0.18	128.0	118.0
in seam No.3	Colemanite	0.325	0.12	96.9	135.0
Above seam No.4	Limestone alternating with claystone	0.341	0.22	143.1	110.0
in seam No.4	Ulexite	0.978	0.21	67.0	112.0
Above seam No.5	Limestone alternating with claystone	0.692	0.19	26.7	116.0
in seam No.5	Disintegrated colemanite	0.460	0.33	86.4	96.0
in upper tuff	Upper tuff	21.54	18.5	16.9*	25.0

* B = 4.3 m

Table 5.16. Compared of the Actual Results the with Estimated Results.

Rock type	Int. Friction angles (Ø)								Actual laboratory	
	Original		Midified		Original		Modified		Peak friction angle ϕ_p	Residual friction angle ϕ_r
	RMR	ϕ	RMR	ϕ	Q	ϕ	Q	ϕ		
Limestone alternating with claystone (4/8)	37.0*	23°	43.7*	27°	0.5	18.43°	0.648*	18.43°	29°	20°
Limestone with clay laminae (5/8)	25.9*	18°	20.9*	15°	0.069*	8.50°	0.069*	8.53	-	18.5

* These values are weighted average obtained from five boreholes.

The results of direct shear strength test carried out in the laboratory on Limestone with clay laminae and Limestone alternating with claystone are compared with the estimates of Barton and Bieniawski. The results calculated based on original and modified RMR and Q-Values are shown in Table 5.16.

The results shown in Table 5.16 indicate that the values of peak friction angle obtained from actual tests are very close the friction angles predicted by modified RMR-system. Equation 4.7 suggested by Barton considers joint alteration number (J_a) and joint roughness (J_r) which, Limestone alternating with claystone and Limenstone with clay laminae in both original and modified Q-system have same joint alteration number (J_a) and joint roughness (J_r) that's why the same internal friction angle values estimated by Barton are calculated. The residual friction angles obtained from actual laboratory tests are close to the friction angles recommended by Barton's. Also the same conclusion was stated by Sheorey (1985).

m and s values have been calculated by Hoek and Brown (1980) for various rock types. Priest and Brown (1983) have also predict m and s values depending on RMR ratings obtained in the field (Table 5.17).

Table 5.17. Comparison of m and s Values as Obtained from Hoek and Brown (1980) and from Priest and Brown (1983)

Rock Type	Rock Mass Quality RMR		Estimated of Priest and Brown From Original RMR		Estimated of Priest and Brown From Modified RMR		According to Hoek and Brown's Chart (for Original RMR)		According to Hoek and Browns's chart (for Modified RMR)	
	Original RMR	Modified RMR	m	s	m	s	m	s	m	s
Colemanite (10a)	30.64	28.64	0.09	1.65×10^{-5}	0.086	12×10^{-5}	0.086	2.4×10^{-5}	0.074	1.77×10^{-5}
Ulexite (10b)	23	12.0	0.03	0.49×10^{-5}	0.013	9.13×10^{-5}	0.028	0.74×10^{-5}	0.0127	0.132×10^{-5}
Limestone alternating with claystone (4/8)	37	43.69	0.043	4.5×10^{-5}	0.073	0.13×10^{-5}	0.041	6.5×10^{-5}	0.067	18.5×10^{-5}
Limestone (8)	21.0	27.4	0.0067	0.36×10^{-5}	0.011	1.0×10^{-5}	0.0064	0.54×10^{-5}	0.0103	1.45×10^{-5}
Weathered limestone (9abc)	38.87	36.87	0.011	6.1×10^{-5}	0.095	4.44×10^{-5}	0.010	8.7×10^{-5}	0.010	6.39×10^{-5}
Tuffite (7)	52.43	48.43	0.115	52.5×10^{-5}	0.085	27.8×10^{-5}	0.105	72.3×10^{-5}	0.078	38.8×10^{-5}
Intermediate tuff (11)	35.89	33.89	0.036	3.8×10^{-5}	0.03	2.8×10^{-5}	0.034	5.4×10^{-5}	0.029	4.01×10^{-5}

The results of Modulus of Deformation values in both original and modified RMR-System were compared in Table 5.18

Table 5.18. Comparision of Modulus of Deformation as Predicted by Bieniawski (1978) and Serafim and Pereira (1983).

Rock Type	Rock Quality Index RMR		Modulus of Deformation E_m(GPa) from Original RMR		Modulus of Deformation E_m(GPa) from Modified RMR	
	Original RMR	Motified RMR	Bieniawski (1978)	Serafim and Pereira (1983)	Bieniawski	Serafim and Pereira (1983)
limestone laminated with claystone (4/8)	37.0	43.0	-	4.73	-	6.68
Colemanite (10a)	30.64	28.64	-	3.28	-	2.92
Ulexite (10b)	23.0	12.0	-	2.11	-	1.12
Alterated limestone (9abc)	38.87	36.87	-	5.27	-	4.7
Tuffite (7)	52.48	48.43	4.86	11.50	-	9.13
Tuff (11)	35.89	33.89	-	4.44	-	3.96

CHAPTER VI

DISCUSSION AND CONCLUSIONS, AND RECOMMENDATIONS FOR THE FUTURE WORKS

6.1. Discussion and Conclusions

In this study, firstly, the ore and rock-units encountered in Bigadiç Region have been classified based on original RMR and Q-Systems. During this classification process a number of difficulties were encountered in defining some of the orginal input parameters. These difficulties were asociated with determination of RQD, uniaxial compressive strength, effect of clay and water, joint spacing, and joint conditions, in RMR-system, and with determination of joint alteration number and joint set number, in Q-system. Consequently it was found that RMR and Q-Systems, in their original forms, were insufficient to fully describe the weak and stratified rock formations. In this respect, and in the light of this study RMR and Q values were modified and results obtained were evaluated based on original and modified RMR values. The following main conclusions were derived:

1. In weak and stratified rocks the original Q-System predicts higher Q values.

2. In relatively strong rock (i.e tuff) there is not much difference between the original and modified Q-values.

3. In weak and stratified rock formations, there is not much difference between the original and modified RMR values.

4. In relatively strong rock mass (i.e. tuff) original RMR System predicts lower RMR values.

5. To overcome the difficulties, encountered during classifications, the following suggestions are made:

- In RMR-System:

(i) RQD :

a. In weak and stratified rock, the RQD values should be determined, as soon as, the cores taken from the corebarrels. Photographs of the cores should be taken as permenant records.

b. When a drill-run include more than one formation, then each formation should be considered seperately in RQD determination.

c. If cores are not seperated into pieces due to the existence of bedding planes appearing on that core, then it should be assumed as an intact core.

(ii) The uniaxial compressive strength of weak and stratified rocks can be determined by point-load tests. The mathematical average of the results obtained from both diametral and axial point-load tests are very close to the results obtained from direct uniaxial strength tests.

(iii) Weathering: Underground water affects the joint conditions. The effect of water becomes even more pronounced with existence of clay in the rock mass. In order to better describe this combined effect, the slake-durability tests should be carried out. Based on the results of these tests, new ratings should be developed as suggested in this thesis;

(iv) Joint spacing: If each of the cracks, beddings, fractures and shear zones obsereved in borehole cores are counted as plane of weakness, then a very low joint-spacings (high number of joints per meter) would result. This would also cause a calculation of a low RMR value. The above mentioned weakness zones, encountered in borehole cores should be considered as one plane of weakness each.

- In Q-System:

(i) ROD: The same procedures should be followed as suggested for RMR system.

(ii) Joint alteration number: In original classification this parameter is defined based on filling material. If filling material does not exist in the joint decision should be made based on joint walls;

(iii) Joint set number: Additional cracks on cores may develop during handling the cores; or due to weathering action. These effects are more pronounced in weak and stratified rocks. Precautions should be taken in handling cores. In deciding the joint set numbers, the intact core recovery and RQD values should be utilized as suggested in this thesis.

6. In this study, the following relationships between the compressive strength and point-load index were determined:

$$(\sigma_c)_v = 16.57\ I_{s(50)} + 2.13 \quad (r = 0.94,\ \text{for test type 1})$$

$$(\sigma_c)_h = 19.60\ I_{s(50)} - 1.5 \quad (r = 0.95,\ \text{for test type 2})$$

7. Based on the comparision of original and modified RMR and Q values, the following results were found:

(i) Rock-mass descriptions were not affected after modified classification. For example, poor rock-mass was still poor rock-mass after modification.

(ii) The regression equations for RMR and Q values were determined as follows:

$RMR = 5.83 \ln Q + 37$ ($r = 0.84$, in original system)

$RMR = 7.47 \ln Q + 42.63$ ($r=0.94$, in modified system)

(iii) The maximum unsupported spans calculated based on modified Q-Systems are more reasonable when compared with the actual observations made in Simav Underground Borax Mine. The results also indicate that the maximum unsupported span values determined by RMR-System are quite realistic.

(iv) The results of support pressures calculated based on RMR and Q systems indicate that, estimates of RMR-System are always less than the Q-System. The support pressures calculated from modified Q values are more reasonable as compared with the original Q values.

(v) m and s material properties in original and modified RMR-Systems were determined as estimated by Priest and Brown (1983) and from Hoek and Brown's chart (1980). The results indicated that, m values estimated by Priest and Brown are slightly greater than those estimated by Hoek and Brown. On the other hand, s values estimated by Priest and Brown were slightly less than the estimates of Hoek and Brown. It can be concluded that both methods provide the similar results.

(vi) Based on the internal friction angles predicted from Q and RMR Systems, and obtained from actual laboratory tests, it can be concluded that: the internall friction angles estimated by modified RMR values are close to the actual peak friction angles determined from laboratory tests. On the other hand, the internal friction angles estimated by Q-System are very close to the residual friction angle determined again from laboratory tests.

(vii) In this study, a relationship between the anisotropy index (I_a) and Modulus of deformability Poissons ratio (E/ν). The following results was found:

$$E/\nu \ \ 869\ I_a^{-3.186} \qquad (r=0.92)$$

6.2. Recommendations for Future Works

The results obtained in this study may only valid for weak and stratified formations encountered in Bigadiç-Simav Mine Region. For this reason, the following recommendations for the future works are made:

(i) The modified index values of RMR and Q-Systems should be determined in other locations and should be compared with the original index values of RMR and Q-Systems.

(ii) The results of classifications should be compared with field observations.

(iii) The load and convergence measurement in gateroadways and main haulageways should be carried out in order to determine the behaviour of these openings. This way a corelation could be found between the rock-mass quality and behaviour of the opening.

(iv) In RMR System, the modified water parameter depent on slake-durability test results and water flow. In this study, water flows of less than 10 litres per minute are used. In future works, the other rates of water flow should be determined.

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