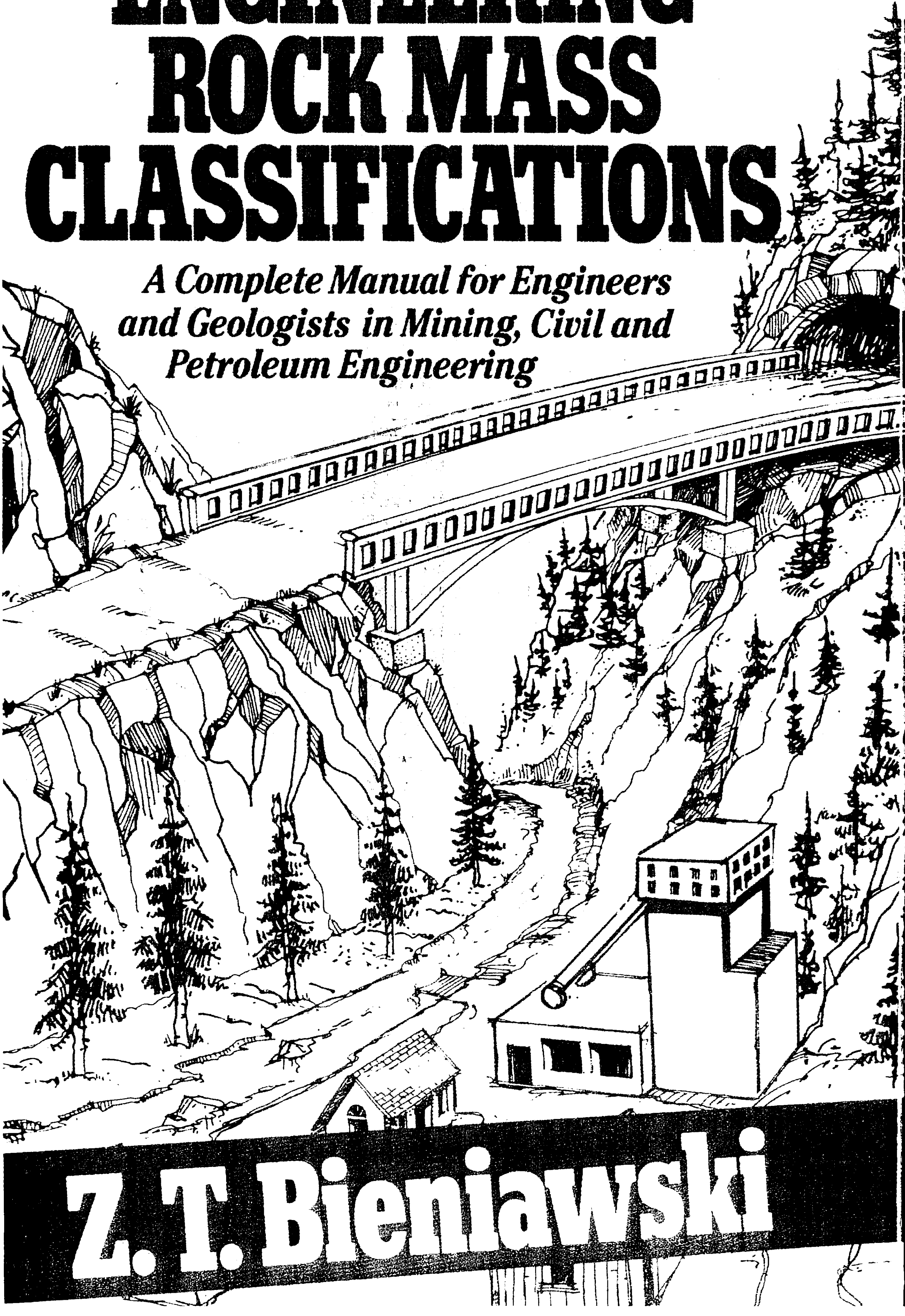
ENGINEERING ROCK MASS CLASSIFICATIONS

A Complete Manual for Engineers and Geologists in Mining, Civil and Petroleum Engineering



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A Complete Manual for Engineers and Geologists in Mining, Civil, and Petroleum Engineering

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WILEY

A WILEY-INTERSCIENCE PUBLICATION

John Wiley & Sons

New York / Chichester / Brisbane / Toronto / Singapore

Library of Congress Cataloging in Publication Data

Bieniawski, Z. T.

Engineering rock mass classifications: a complete manual for engineers and geologists in mining, civil, and petroleum engineering / Z. T. Bieniawski.

p. cm.

"A Wiley-Interscience publication."

Bibliography: p.

Includes index.

ISBN 0-471-60172-1

1. Geology, Economic. 2. Rocks—Classification. 3. Rock mechanics. I. Title.

TN260.B54 1989

624.1'5132—dc19 89-30956

CIP

10 9 8 7 6

Perseverantia omnia vincit

—this motto has guided my work

Contents

Preface

Rock mass classifications have emerged in the past 15 years as powerful design aids in civil engineering, mining, and geology, and more than 300 papers have been written on the subject. Yet, no comprehensive textbook dealing specifically with this topic exists. This book provides an in-depth treatment of the subject matter and aims to serve as an authoritative reference, consolidating otherwise widely scattered information. In addition, new, unpublished material and case histories have been included.

The subject of rock mass classifications is currently taught in over 1000 universities and colleges in the United States and abroad, to undergraduate and graduate students in geology, geological engineering, civil engineering, mining engineering, and petroleum engineering. The book presents not only the fundamental concepts of the various classification schemes but also critically appraises their practical applications in industrial projects.

This book is intended for engineers and geologists in industry, particularly consulting geotechnical engineers and engineering geologists, as well as for undergraduate students in engineering and graduate students in geology.

I remember fondly the many people who stimulated my thinking in the course of working over 15 years on the subject of rock mass classifications. I am particularly grateful to the late Professor Leopold Müller of Salzburg, who was instrumental in my developing the Geomechanics Classification and starting on this system during my visit to the Technical University of Karlsruhe, West Germany, in 1972. I am also grateful to my old colleague, Dr. Phillip J. N. Pells, now of the School of Civil Engineering at the University of Sydney, Australia, who made important contributions to my early work on rock mass classifications and is specifically acknowledged here because he never replies to my letters!

Many researchers and practicing engineers have made important contributions by modifying and improving my original RMR system (Geomechanics Classification). They are too numerous to identify here, but all are listed in Table 3.1 in the text. However, I would like to single out my former graduate students who, through their doctoral dissertations have significantly advanced the state of the art of rock mass classifications in mining. They are: Dr. David Newman, now assistant professor at the University of Kentucky; Dr. Erdal Unal, now associate professor at the Middle East University in Turkey; and Dr. Claudio Faria Santos from Brazil. Moreover, of my current doctoral candidates, Mr. Dwayne C. Kicker contributed by performing an up-to-date survey of rock mass classifications, and Dr. Glenn A. Nicholson, of the U.S. Army Corps of Engineers, developed an empirical constitutive relationship for rock mass based on rock mass classifications.

My friend Professor Dr. Georg Spaun of the Technical University of Munich was the source of many stimulating discussions and provided me with thrilling insights into engineering geology and the New Austrian Tunneling Method. Dr. Nick Barton of the Norwegian Geotechnical Institute was always helpful in exchanging ideas and permitting the use of the tables and figures concerning the Q-system. Finally, Professor Evert Hoek, now at the University of Toronto, was an inspiration over many years regarding the innovative use of rock mass classifications and their role in rock engineering design.

I would also like to acknowledge the assistance received during the preparation of the manuscript. The text was compiled at The Pennsylvania State University for a graduate course on geotechnical aspects of tunneling in rock. My research assistant, Dr. Claudio Faria Santos, prepared the microcomputer program for determining the rock mass rating and assisted in computerizing the data base of RMR case histories. My wife, Elizabeth, still remembering her graduate studies in librarianship, was most helpful in cross-referencing the text and the index. My secretary, Jessie Fowler, typed the manuscript and always remained cheerful in spite of endless corrections.

Z. T. BIENIAWSKI

University Park, Pennsylvania
June 1989

Engineering Rock Mass Classifications

1

Introduction

The origin of the science of classification goes back to the writings of the ancient Greeks but the process of classification, the recognition of similarities and the grouping of objects based thereon, dates back to primitive man.
—***Robert R. Sokal***

In his presidential address to the Classification Society, Professor Sokal not only provided a historical overview of the subject but also emphasized that classification is an important aspect of most sciences, with similar principles and procedures having been developed independently in many fields (Sokal, 1972).

The science of classification is called taxonomy, which deals with theoretical aspects of classification, including its basis, principles, procedures, and rules. A distinction should be made between classification and identification; classification is defined as the arrangement of objects into groups on the basis of their relationship, whereas identification means the allocation or assignment of additional unidentified objects to the correct class, once such classes have been established by prior classification.

1.1 FUNCTION OF CLASSIFICATIONS IN ENGINEERING

Classifications have played an indispensable role in engineering for centuries. For example, the leading classification society for shipping, Lloyd's Register of London, was established in 1760 when the first printed "register of ships" appeared. Particulars of ships were listed, with various classification symbols affixed, each denoting the condition of various parts of the ship structure or equipment. Today rigid standards are specified for ship construction and maintenance before a ship is insured, and these standards are laid down by the technical committee, composed of shipbuilders, marine engineers, and naval architects, that advises the classification society. Through a worldwide organization of surveyors, classifications are performed when a ship is built and when it is in operation; in essence, a classification society dictates the design and construction of every ship in the world more than 100 tons gross. It provides detailed specifications which must be met as the minimum standards. The American Bureau of Shipping, established in 1867, the Bureau Veritas of France, and the Registro Italiano Navale are other prominent classification societies, in addition to Lloyd's Register of Shipping.

In rock engineering, the first major classification system was proposed over 40 years ago for tunneling with steel supports (Terzaghi, 1946). Considering the three main design approaches for excavations in rock—analytical, observational, and empirical—as practiced in mining and civil engineering, rock mass classifications today form an integral part of the most predominant design approach, the empirical design methods. Indeed, on many underground construction and mining projects, rock mass classifications have provided the only systematic design aid in an otherwise haphazard "trial-and-error" procedure.

However, modern rock mass classifications have never been intended as the ultimate solution to design problems, but only a means toward this end. In fact, some 15 years ago, when work started on the major rock mass classification schemes in use today, the tunneling scene worldwide was often characterized by limited site investigation programs and even more limited, if any, design procedures. Any such procedures that were used then would hardly qualify nowadays as an engineering design process, such as that used systematically in other branches of engineering. Rock mass classifications were developed to create some order out of the chaos in site investigation procedures and to provide the desperately needed design aids. They were not intended to replace analytical studies, field observations, and measurements, nor engineering judgment.

1.2 ROCK CLASSIFICATIONS AS DESIGN AIDS

In essence, rock mass classifications are not to be taken as a substitute for engineering design. They should be applied intelligently and used in conjunction with observational methods and analytical studies to formulate an overall design rationale compatible with the design objectives and site geology. When used correctly and for the purpose for which they were intended, rock mass classifications can be powerful aids in design.

The objectives of rock mass classifications are therefore to

1. Identify the most significant parameters influencing the behavior of a rock mass.
2. Divide a particular rock mass formation into groups of similar behavior, that is, rock mass classes of varying quality.
3. Provide a basis for understanding the characteristics of each rock mass class.
4. Relate the experience of rock conditions at one site to the conditions and experience encountered at others.
5. Derive quantitative data and guidelines for engineering design.
6. Provide a common basis for communication between engineers and geologists.

The preceding items suggest the three main benefits of rock mass classifications:

1. Improving the quality of site investigations by calling for the minimum input data as classification parameters.
2. Providing quantitative information for design purposes.
3. Enabling better engineering judgment and more effective communication on a project.

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2

Role of Rock Mass Classifications in Site Characterization and Engineering Design

The mere formulation of a problem is far more often essential than its solution; to raise new questions, new possibilities, requires creative imagination and marks real advances in science.
—Albert Einstein

Unlike other engineering materials, rock presents the designer with unique problems. First of all, rock is a complex material varying widely in its properties, and in most mining as well as civil engineering situations, not one but a number of rock types will be present. Furthermore, a choice of rock materials is only available if there is a choice of alternative sites for a given project, although it may be possible, to some extent, to reinforce the rock surrounding the excavation. Most of all, the design engineer and geologist are confronted with rock as an assemblage of blocks of rock material separated by various types of discontinuities, such as joints, faults, bedding planes, and so on. This assemblage constitutes a rock mass. Consequently, the engineering properties of both intact rock and the rock mass must be considered.

2.1 ROCK AS AN ENGINEERING MATERIAL

The behavior of rock is best presented in a stress–strain curve, an example of which is given in Figure 2.1.

It will be noted that, initially, deformation increases approximately proportionally with increasing load. Eventually, a stress level is reached at which fracture is initiated, that is, minute cracks, which are present in almost any material, start to propagate. With increasing deformation, the crack propagation is stable, that is, if the stress increase is stopped, the crack propagation is also stopped. Further increasing the stress, however, leads to another stress level, called critical energy release, at which the crack propagation is unstable, that is, it continues even if the stress increase is stopped.

Next, the maximum load bearing capacity is reached. Called strength failure, this is in fact the strength of the rock material. Most rocks characterized by brittle fracture fail violently at this stage when tested in a conventional (soft) loading machine. In such a case, the specimen machine system collapses and strength failure coincides with rupture (i.e., complete disintegration of rock specimen). If, however, the stiffness of the testing machine is increased, the stress decreases with increasing strain. This stage is characterized by the negative slope of the stress–strain curve, and the material is now in a fractured state. This is important, since it shows that even cracked, fractured material can offer resistance to loads applied to it. An excavation may be such that it will not collapse even if the rock material surrounding such a structure has failed by exceeding its material strength. Thus, the rock surrounding an excavation may be fractured and the excavation still stable. Indeed, fractured rock may even be desirable, since it will not lead to sudden and violent strength failure. Practical applications of this concept to mining and tunneling and its significance for rock support considerations are dealt with in detail by Jaeger and Cook (1979).

Stress–strain curves serve as the source for obtaining the compressive or tensile strengths, the modulus of elasticity, and Poisson's ratio of rock materials. These properties of some common rock types can be found in Lama and Vukuturi (1978) and in Kulhawy (1975).

Laboratory testing methods are generally well established, and testing techniques have been recommended by the International Society for Rock Mechanics (ISRM) and the American Society for Testing and Materials

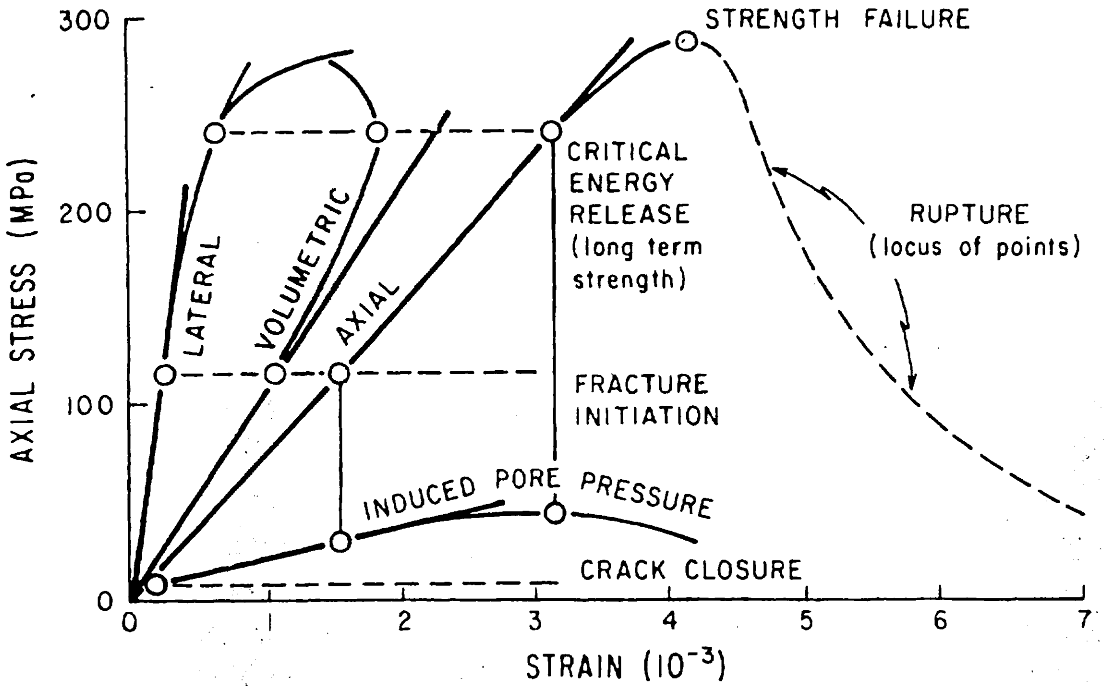


Figure 2.1 *Representation of brittle fracture mechanism for hard rock in uniaxial compression. (After Bieniawski, 1967.)*

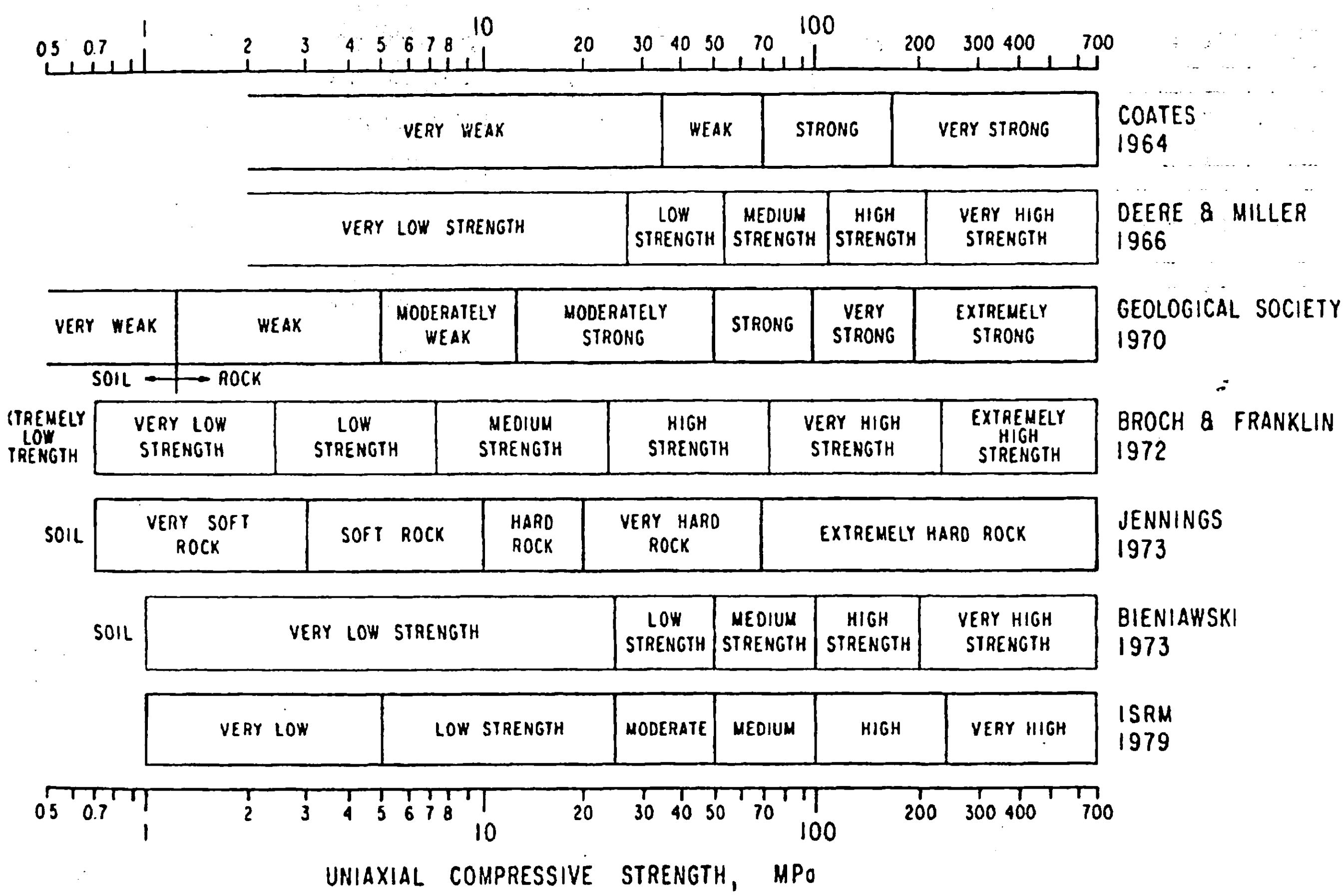


Figure 2.2 *Various strength classifications for intact rock.*

(ASTM). Detailed procedures for performing laboratory tests are available as ISRM Suggested Methods (1981b) or ASTM Standards (1987).

A number of classifications featuring rock material strength and modulus of elasticity have been proposed. The intact rock strength classifications are compared in Figure 2.2. The strength–modulus classification proposed by Deere and Miller (1966) is depicted in Figure 2.3, using sandstone as an example. This classification has been widely recognized as particularly convenient for use in the field of rock mechanics. Subsequently, the ISRM

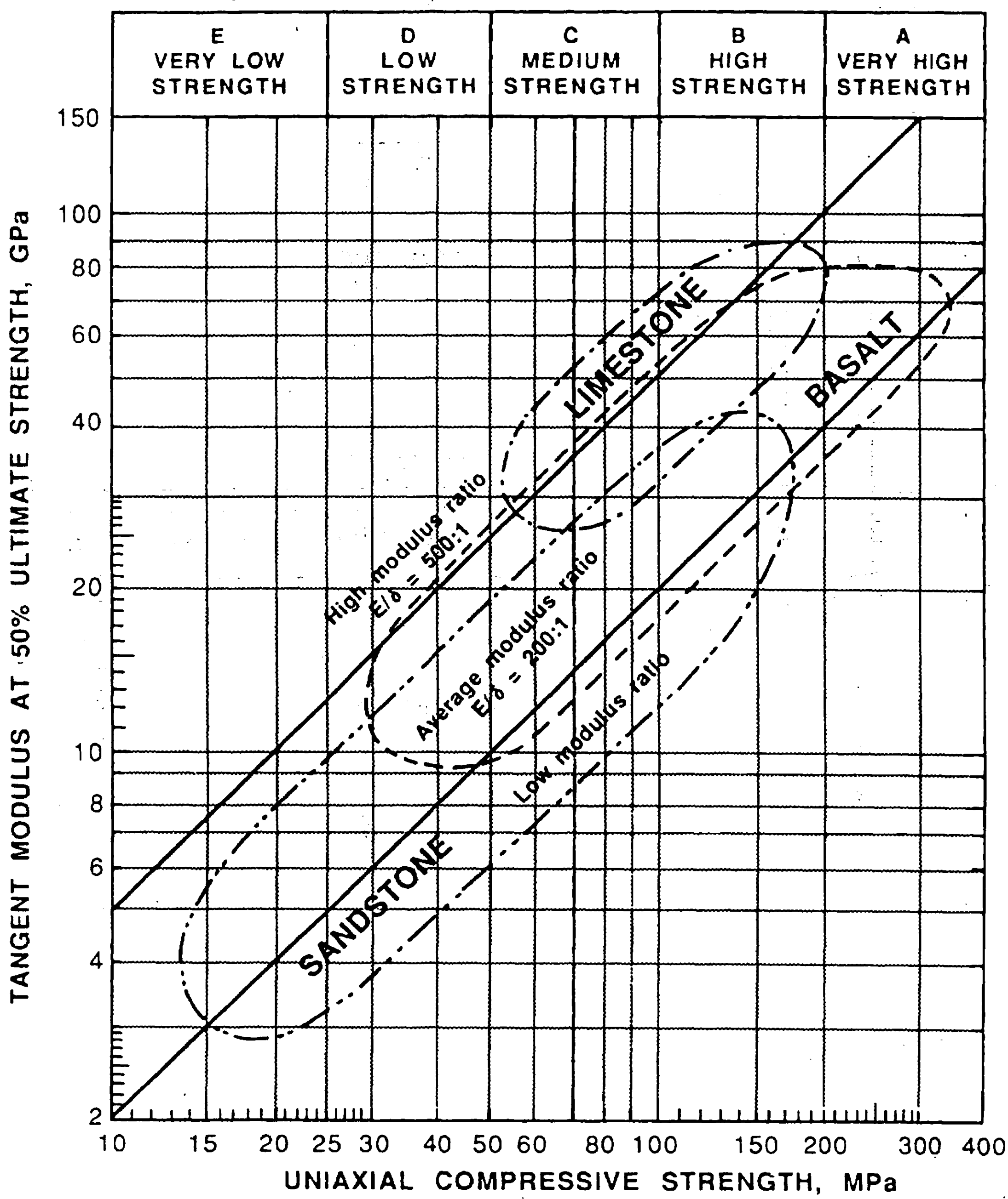


Figure 2.3 *Strength–deformation representation for three rock types. (After Deere and Miller, 1966.)*

Commission on Rock Classification has recommended different ranges of values for intact rock strength (ISRM, 1981b). The main reason for the ISRM ranges was the opinion that the Deere–Miller classification did not include differentiation in the strength in the range below 25 MPa. It should be also noted that this led to a recommendation that the convenient value of 1 MPa (145 psi) for the uniaxial compressive strength may be taken as the lowest strength limit for rock materials. Hence, the materials with a strength lower than 1 MPa should be considered as soils and described in accordance with soil mechanics practice.

The major limitation of the intact rock classifications is that they cannot provide quantitative data for engineering design purposes. Therefore, their main value lies in enabling better identification and communication during discussions of intact rock properties.

2.2 STRUCTURAL FEATURES OF ROCK MASSES

When the design engineer and the engineering geologist are confronted with rock, they must visualize the rock mass as an assemblage of intact rock blocks separated by different types of geological discontinuities. They must therefore consider the characteristics of both the intact material and the discontinuities.

The question immediately arises as to how the rock material is related to the rock mass. In answering this question, one must note, first of all, that the importance of the properties of intact rock material will be generally overshadowed by the properties of the discontinuities in the rock masses. However, this does not mean that the properties of the intact rock material should be disregarded when considering the behavior of jointed rock masses. After all, if discontinuities are widely spaced or if the intact rock is weak and altered, the properties of the intact rock may strongly influence the gross behavior of the rock mass. Furthermore, a sample of a rock material sometimes represents a small-scale model of the rock mass, since they both have gone through the same geological cycle. Nevertheless, in general, the properties of the discontinuities are of greater importance than the properties of the intact rock material.

An important issue in rock classifications is the selection of the parameters of greatest significance. There appears to be no single parameter or index that can fully and quantitatively describe a jointed rock mass for engineering purposes. Various parameters have different significance, and only if taken together can they describe a rock mass satisfactorily.

The strength of the rock material is included as a classification parameter in the majority of rock mass classification systems. It is a necessary parameter because the strength of the rock material constitutes the strength limit of the rock mass. The uniaxial compressive strength of rock material can be determined in the field indirectly by means of the point-load strength index (Franklin, 1970), so that one is not restricted to laboratory testing.

The second parameter most commonly employed is the rock quality designation (RQD). This is a quantitative index based on a modified core-recovery procedure which incorporates only sound pieces of core that are 100 mm or greater in length. The RQD is a measure of drill core quality or fracture frequency, and disregards the influence of joint tightness, orientation, continuity, and gouge (infilling). Consequently, the RQD does not fully describe a rock mass.

Other classification parameters used in current rock mass classifications are spacing of discontinuities, condition of discontinuities (roughness, continuity, separation, joint-wall weathering, infilling), orientation of discontinuities, groundwater conditions (inflow, pressure), and in-situ stresses.

An excellent discussion of the methods for quantitative description of discontinuities in rock masses can be found in ISRM (1981b).

It is accepted that in the case of surface excavations and those near-surface underground rock excavations that are controlled by the structural geological features, the following classification parameters are important: strength of intact rock material, spacing of discontinuities, condition of discontinuities, orientation of discontinuities, and groundwater conditions. In the case of deep underground excavations where the behavior of rock masses is stress-controlled, knowledge of the virgin stress field or the changes in stress can be of greater significance than the geological parameters. Most civil engineering projects, such as tunnels and subway chambers, fall into the first category of geologically controlled rock mass structures.

2.3 SITE CHARACTERIZATION PROCEDURES

Comprehensive site characterization guidelines were published by the International Association of Engineering Geology (1981a), the Construction Industry Research and Information Association (Weltman and Head, 1983), and the U.S. National Committee on Tunneling Technology (1984). This last reference was a very important contribution because its findings were based on a three-year case-history study of subsurface explorations for underground design and construction. The objective was to discover improvements in practices and procedures that could make geotechnical site investigation programs more effective. Based on 87 U.S. projects, it was recommended that

1. Expenditures for geotechnical site exploration should be 3% of estimated project cost.
2. The level of exploratory borings should be 1.5 linear ft of borehole per route ft of tunnel alignment.
3. Not only should all geologic reports be incorporated in the contract documents, but a "Geotechnical Design Report," compiled by the tunnel designers, should be included in the specifications.

The interaction of the various site characterization activities and the parameters needed for engineering design is demonstrated Table 2.1. It will be seen that the testing approaches are divided into categories of field testing and laboratory testing. Their purpose is to establish the needed design parameters characterizing the rock material, the rock mass, the in-situ stress field, and other conditions.

The first fact that must be recognized when planning a site investigation program is that there is no such thing as a standard site investigation (Hoek, 1982). This statement applies equally well to both stages of site characterization, namely, a preliminary site investigation and the detailed site characterization. The scope of the appropriate geological investigations is outlined in Figure 2.4.

The purpose of the initial site investigation is to establish the feasibility of the project. In essence, the initial site assessment involves the discovery, correlation, and analysis of such geological data as:

1. Rock types to be encountered.
2. Depth and character of the overburden.
3. Macroscopic scale discontinuities, such as major faults.
4. Groundwater conditions.
5. Special problems, such as weak ground or swelling rock.

The initial site assessment can utilize a number of sources of information, in particular

1. Available geological maps, published literature, and possibly, local knowledge.
2. Photogeological images (aerial and ground photographs) of the area.

The photogeological study is of special importance, and its benefits may even justify procuring new aerial photographs if those available are inadequate.

TABLE 2.1 Recommended Rock Mechanics Observations and Measurements for Site Characterization

Test	Property/Data				
	Rock Material	Rock Mass	In-Situ Stress Field	Modulus of Deformation	Empirical Design Data
			Laboratory Testing		
Uniaxial compression tests	Material strength, anisotropy			Elastic modulus, Poisson's ratio	
Triaxial compression tests	Friction and cohesion of rock material				m_i parameter
Density, porosity, water content, swelling	Density, porosity, slake durability				Weatherability and swelling parameters
			Field Testing		
Geotechnical surveys and integral sampling	Detailed engineering geological description of rock strata				Input data for engineering classifications of rock masses

Point-load test	Strength index from rock pieces				
Overcoring cells and small flat jacks			Magnitude and directions of stresses	Deformation parameters	
Plate bearing tests and borehole jacks		Effect of joints on strength of rock mass		Deformation parameters	
Seismic/sonic measurements	Sonic velocity data from laboratory rock			Longitudinal and shear wave velocities and dynamic moduli	
Convergence monitoring and borehole extensometers			Stress redistribution	Time-dependent rock mass movements around excavations	
Piezometers in boreholes		Water inflow, pressure, and permeability			
Rock bolt pullout tests					Rock support data: spacing, length, etc.

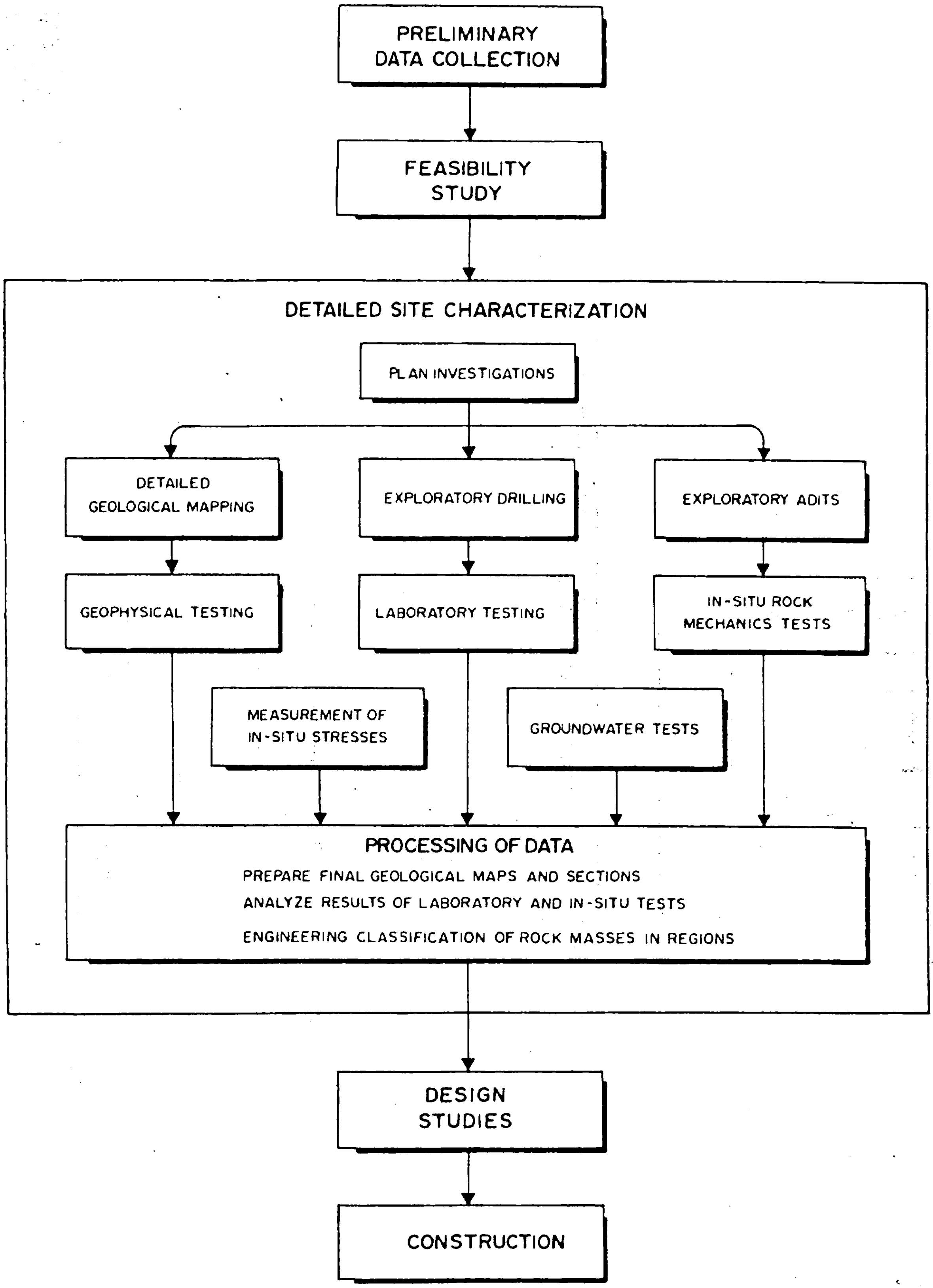


Figure 2.4 *Stages of a site characterization program.*

The benefits of the photogeological study include information on topography, drainage, lithology, geological structures, and discontinuities.

One of the purposes of the initial site exploration is to determine the regional geology of the vicinity of the project. This aspect is fully treated by Fisher and Banks (1978). While determination of the regional geology is based mainly on studies of reports, maps, and publications involving the geological history of the area as well as studies of information derived from local knowledge and aerial photography, some limited investigations may also be conducted. These would include mapping of the surface outcrops, physical exploration, and a limited program of drilling and groundwater investigations. Some laboratory tests on rock samples and index field tests on rock cores may also be performed. Based on these investigations, preliminary geological maps and sections showing favorable and unfavorable regions in the rock mass should be prepared. These maps and sections are important for planning the next stages of the site characterization program.

Where outcrops and geological structures are not easily deduced by either photogeological or ground investigations, geophysical methods may be used to locate large discontinuities such as faults. The most effective means of doing this would be by seismic or resistivity methods (Hoek and Brown, 1980).

Based on an initial site exploration, the final site characterization will be conducted once the feasibility of the project has been established. This stage of site characterization will include detailed exploratory drilling, geological mapping, geophysical surveys, and rock mechanics testing.

2.3.1 Drilling Investigations

The purpose of a drilling investigation is to

1. Confirm the geological interpretations.
2. Examine cores and boreholes to determine the quality and characteristics of the rock mass.
3. Study groundwater conditions.
4. Provide cores for rock mechanics testing and petrographic analyses.

As the object of drilling is to obtain rock cores for interpretation and testing, it is essential to obtain as near 100% core recovery as possible. To ensure a successful drilling operation, the following information should be remembered:

1. The cost of a drilling investigation for geotechnical purposes is much higher, sometimes by a factor of two, than the cost of drilling for mineral exploration purposes. Geotechnical drilling necessitates quality equipment and extra care, but it can provide high-quality information.

2. The drilling equipment should feature diamond core drilling facilities permitting core of at least NX size (54-mm dia) and featuring split double-tube core barrels to minimize drilling vibrations. Also included should be equipment for performing water pressure tests.

3. The purpose of the drilling investigation is to obtain not only the core logs but also the logging of the borehole itself. Hence, examination of the borehole walls by borehole cameras or by other systems should also be considered.

4. For meaningful interpretation of the orientation of the geological features, core orientation procedures may be employed during geotechnical drilling. A number of techniques are available for that purpose (Hoek and Brown, 1980). Boreholes should be angled so that vertical discontinuities can be sampled.

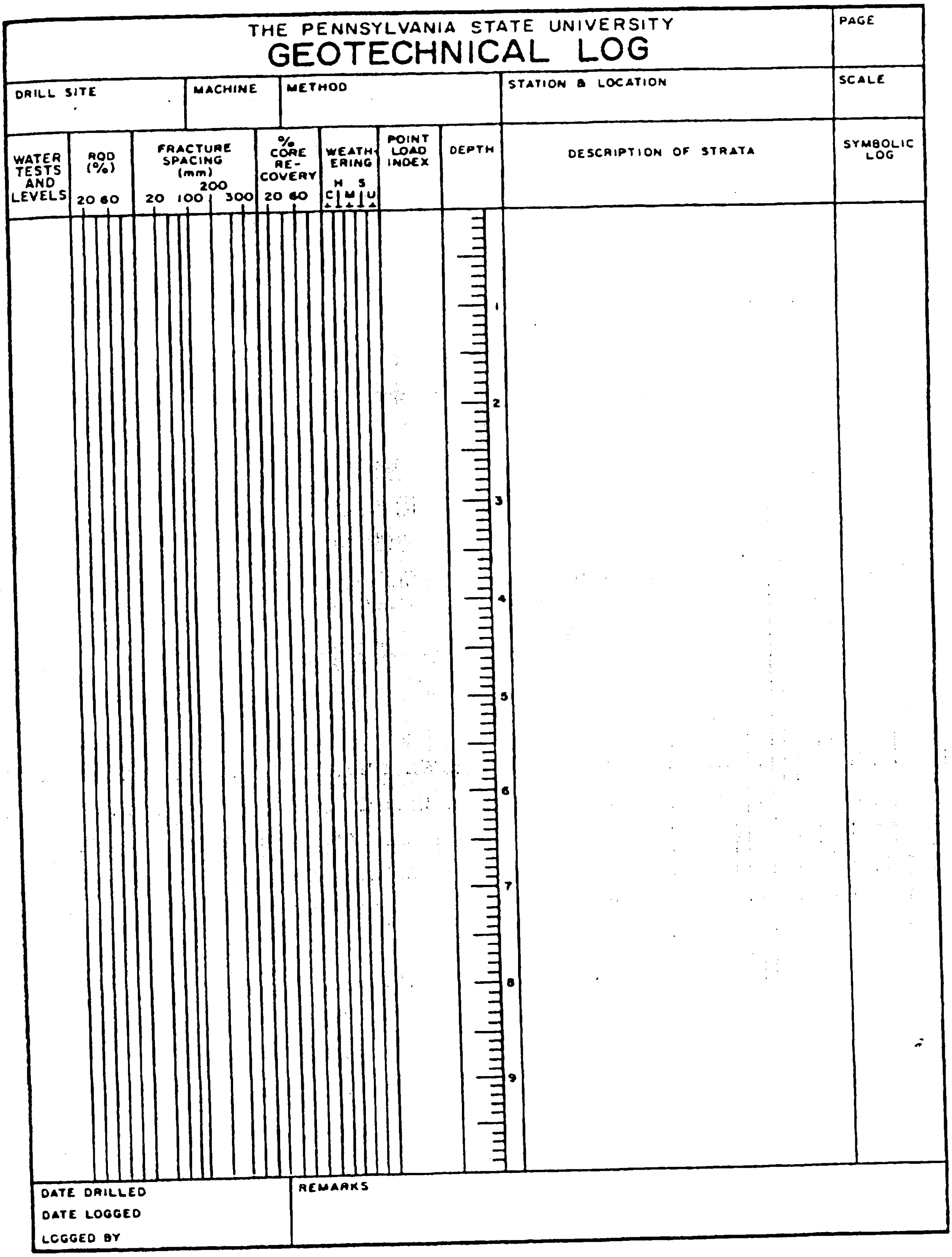
5. In cases where poor rock conditions are evident and yet 100% core recovery is desirable, a technique known as the integral sampling method may be employed (Rocha, 1967) in shallow holes.

6. Good care should be taken of the core recovered from the boreholes. This means that the cores should be photographed as soon as possible, carefully marked, placed in protective wrapping in the core boxes, and stored in properly provided storage sheds. Core samples removed for testing should be appropriately marked in the core boxes.

A systematic method should be used for geotechnical logging of the rock cores. There is a difference between a geological core log for general purposes and a geotechnical core log for engineering purposes. The geotechnical core log provides a format to record both the geological and engineering characteristics of the rock core and the results of any field tests. The log of core should systematically record all the information available from the core. An example of a geotechnical core log is given in Figure 2.5. It should be noted that there is no rigid standard format for a geotechnical core log, and the amount of detail used will depend on the actual purpose of the project.

2.3.2 Engineering Geological Mapping

The purpose of engineering geological mapping is to investigate the significant features of the rock mass, especially the discontinuities, such as naturally occurring joints. It is also important to determine the geological structure,



THE PENNSYLVANIA STATE UNIVERSITY
GEOTECHNICAL LOG
PAGE
DRILL SITE
MACHINE
METHOD
STATION & LOCATION
SCALE
WATER TESTS AND LEVELS
RQD (%) 20 60
FRACTURE SPACING (mm) 20 100 200 300
% CORE RE-COVERY 20 60
WEATH-ERING H S C M U
POINT LOAD INDEX
DEPTH 1 2 3 4 5 6 7 8 9
DESCRIPTION OF STRATA
SYMBOLIC LOG
DATE DRILLED
DATE LOGGED
LOGGED BY
REMARKS

Figure 2.5 *Geotechnical core log.*

especially in stratified rock which may have been subjected to faulting. Detailed procedures for engineering geological mapping have been described in a number of publications, notably by Dearman and Fookes (1974), Kendorski and Bischoff (1976), Dowding (1978), the International Association of Engineering Geology (1981b), and Compton (1985).

It should be noted that while engineering geological mapping is fairly frequently found on tunneling projects, this is not the case on mining projects. Engineering geological mapping in underground coal mines is a fairly recent innovation.

Finally, one should emphasize that one of the purposes of engineering geological mapping is to provide input data for a rock mass classification to be used at the site for estimating the stability of underground structures and support requirements. Clearly, engineering geological mapping will provide the most reliable input data for a rock mass classification although it is also possible to obtain reasonable data from interpretations of the borehole and core logs.

2.3.3 Geophysical Investigations

Geophysical techniques involving seismic refraction and reflection, electrical resistivity, and gravimetric and magnetic measurements form an accepted part of engineering-geological investigation procedures. Detailed descriptions of these methods, together with their applications, limitations, accuracy, and costs, may be found in many textbooks (see, specifically, Hoek and Brown, 1980). It should be emphasized that the results of geophysical surveys should always be checked by diamond drilling investigations. Alternatively, geophysical measurements may be used to provide geological information about regions of a rock mass positioned between two boreholes.

Geophysical investigations may be conducted either by surface geophysical investigations or by geophysical exploration in boreholes.

Of the geophysical techniques applicable to rock mechanics, the seismic refraction method is the most popular and useful for the purposes of rock mass characterization. This method may be used either on the surface or in boreholes.

The squared ratio between the longitudinal seismic wave velocity as measured in the field (V_F) and the sonic wave velocity as measured in the laboratory (V_L) has been used as an index of rock quality. The ratio is squared to make the velocity index equivalent to the ratio of the dynamic moduli. The difference in these two velocities is caused by the structural discontinuities in the rock mass. For a high-quality massive rock mass containing only a few joints, the velocity ratio (V_F/V_L) should approach unity. As the degree of jointing and fracturing becomes more severe, the

TABLE 2.2 Velocity Index and Rock Mass Quality[a]

Velocity Index $(V_F/V_L)^2$	Description of Rock Mass Quality
<0.2	Very poor
0.2–0.4	Poor
0.4–0.6	Fair
0.6–0.8	Good
0.0–1.0	Very good

[a]After Coon and Merritt (1970).

velocity ratio will be reduced to values lower than unity. Table 2.2 illustrates the relationship between the velocity index and rock mass quality (Coon and Merritt, 1970).

Attempts have been made to use the velocity index to estimate the ratio of the static modulus of the rock mass to the laboratory modulus of the rock material. However, Coon and Merritt (1970) concluded that the velocity index is not reliable for predicting directly in-situ rock mass deformability. This index has too many uncertainties because of the different sensitivities of the seismic and sonic waves as well as difficulties in generating and identifying elastic waves in rock masses and rock materials.

2.3.4 Geological Data Presentation

If determination of geological data for site characterization is a difficult problem, presentation of these data for engineering purposes is sometimes even more difficult. Communication between the engineering geologist and the design engineer would be greatly enhanced if the format for data presentation could be established in the early stages of an engineering project. The following suggestions are useful:

1. Borehole data should be presented in well-executed geotechnical logs.
2. Mapping data derived from joint surveys should be presented as spherical projections such as of the Schmidt or Wolff type (Goodman, 1976; Hoek and Brown, 1980).
3. A summary of all the geological data, including the groundwater conditions, should be entered in the input data sheets for rock mass classification purposes (see Fig. 2.6).
4. Longitudinal sections and cross sections of structural geology at the site should form an integral part of a geological report.
5. Consideration should be given to constructing a geological model of the site.

INPUT DATA FORM : GEOMECHANICS CLASSIFICATION (ROCK MASS RATING SYSTEM)

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

STRUCTURAL REGION	DEPTH, m	ROCK TYPE

STRENGTH OF INTACT ROCK MATERIAL

Designation	Uniaxial compressive strength, MPa	OR	Point-load strength index, MPa
Very High:	Over 250.........		>10..............
High:	100-250..........		4-10.............
Medium High:	50-100...........		0-4..............
Moderate:	25-50............		1-2..............
Low:	5-25.............		<1...............
Very Low:	1-5		

DRILL CORE QUALITY R.Q.D.

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

R.Q.D. - Rock Quality Designation

STRIKE AND DIP ORIENTATIONS

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

NOTE: Refer all directions to magnetic north.

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3	Set 4
Very wide:	Over 2 m	....	....	....	....
Wide	0.6 - 2 m	....	....	....	....
Moderate:	200 - 600 mm	....	....	....	....
Close:	60 - 200 mm	....	....	....	....
Very close:	< 60 mm	....	....	....	....

GROUND WATER

INFLOW per 10 m of tunnel length liters/minute ....
or
WATER PRESSURE kPa ....

GENERAL CONDITIONS (completely dry, damp, wet, dripping or flowing under low/medium or high pressure) ....

IN SITU STRESSES

CONDITION OF DISCONTINUITIES

		Set 1	Set 2	Set 3	Set 4
PERSISTENCE (CONTINUITY)					
Very low:	< 1 m	....	....	....	....
Low:	1 - 3 m	....	....	....	....
Medium:	3 - 10 m	....	....	....	....
High:	10 - 20 m	....	....	....	....
Very high:	> 20 m	....	....	....	....
SEPARATION (APERTURE)					
Very tight joints:	< 0.1 mm	....	....	....	....
Tight joints:	0.1 - 0.5 mm	....	....	....	....
Moderately open joints:	0.5 - 2.5 mm	....	....	....	....
Open joints:	2.5 - 10 mm	....	....	....	....
Very wide aperture:	> 10 mm	....	....	....	....
ROUGHNESS (state also if surfaces are stepped, undulating or planar)					
Very rough surfaces:		....	....	....	....
Rough surfaces:		....	....	....	....
Slightly rough surfaces:		....	....	....	....
Smooth surfaces:		....	....	....	....
Slickensided surfaces:		....	....	....	....
FILLING (GOUGE)					
Type:		....	....	....	....
Thickness:		....	....	....	....
Uniaxial compressive strength, MPa		....	....	....	....
Seepage:		....	....	....	....
WALL ROCK OF DISCONTINUITIES					
Unweathered		....	....	....	....
Slightly weathered		....	....	....	....
Moderately weathered		....	....	....	....
Highly weathered		....	....	....	....
Completely weathered		....	....	....	....
Residual soil		....	....	....	....

GENERAL REMARKS AND ADDITIONAL DATA

MAJOR FAULTS specify locality, nature and orientations.

NOTE: For definitions and methods consult ISRM document: *'Quantitative description of discontinuities in rock masses.'*

***Figure 2.6** Input data form for engineering classification of rock masses.*

2.4 INPUT DATA REQUIREMENTS: AN INTEGRAL APPROACH

Provision of reliable input data for engineering design of structures in rock is one of the most difficult tasks facing engineering geologists and design engineers. It is extremely important that the quality of the input data matches the sophistication of the design methods. It has been often contended that some design methods, such as numerical techniques, have outpaced our ability to provide the input data necessary for the application of these methods. Obviously, it must be realized that if incorrect input parameters are employed, incorrect design information will result.

The guidelines cited below are recommended as an integral approach to site characterization of rock masses:

Firstly, a detailed engineering geological assessment of the rock mass conditions and parameters is required.

Secondly, the stress field should be established by means of either an overcoring technique or small flat jacks. In the case where underground adits are not available for stress measurements by means of overcoring or small flat jacks, the hydrofacturing method may be employed in deep boreholes (Haimson, 1978).

Thirdly, seismic velocity geophysical surveys should be conducted to determine the continuity of the rock mass conditions throughout the area of the proposed engineering project.

Fourthly, diamond drilling of good-quality core of NX size (54-mm dia) should be undertaken so that the rock quality designation (RQD) can be established and samples can be selected for laboratory tests to determine the static strengths, moduli, and the sonic velocity on intact rock specimens.

The parameters needed for site characterization are summarized here for the convenience of the engineering geologists responsible for the collection of geological data for use in engineering design.

The first step is 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, the boundaries of structural regions will coincide with such major geological features as faults and shear zones.

Once the structural regions have been delineated, input parameters are established for each structural region and entered onto an input data sheet, an example of which is given in Figure 2.6.

Rock Quality Designation (RQD) This index is used as a classification parameter because, although not sufficient on its own for a full description of a rock mass, it has been found most useful in tunneling applications as a guide for selection of tunnel support. The RQD has been employed extensively in the United States and in Europe, and is a simple, inexpensive, and reproducible way to assess the quality of rock core (Deere et al., 1967).

This quantitative index is a modified core-recovery percentage which incorporates only those pieces of core that are 100 mm or greater in length. Shorter lengths of core are ignored, since they are considered to be due to close shearing, jointing, or weathering in the rock mass. It should be noted that the RQD disregards the influence of discontinuity tightness, orientation, continuity, and gouge (infilling) material.

For RQD determination, the International Society for Rock Mechanics (ISRM) recommends double-tube, N-sized core barrels (core dia. of 54 mm).

Spacing and Orientation of Discontinuities The spacing of discontinuities is the mean distance between the planes of weakness in the rock mass in the direction perpendicular to the discontinuity planes. The strike of discontinuities is generally recorded with reference to magnetic north. The dip angle is the angle between the horizontal and the joint plane taken in a direction in which the plane dips.

Condition of Discontinuities This parameter includes roughness of the discontinuity surfaces, their separation (distance between the surfaces), their length or continuity (persistence), weathering of the wall rock of the planes of weakness, and the infilling (gouge) material.

Roughness, or the nature of the asperities in the discontinuity surfaces, is an important parameter characterizing the condition of discontinuities. Asperities that occur on joint surfaces interlock, if the surfaces are clean and closed, and inhibit shear movement along the joint surface. Asperities usually have a base length and amplitude measured in millimeters and are readily apparent on a core-sized exposure of a discontinuity.

Separation, or the distance between the discontinuity surfaces, controls the extent to which the opposing surfaces can interlock as well as the amount of water that can flow through the discontinuity. In the absence of interlocking, the discontinuity filling (gouge) controls entirely the shear strength of the discontinuity. As the separation decreases, the asperities of the rock wall tend to become more interlocked, and both the filling and the rock material contribute to the discontinuity shear strength. The shear strength along a discontinuity is therefore dependent on the degree of separation, presence or absence of filling materials, roughness of the surface walls, and the nature of the filling material.

Continuity of discontinuities influences the extent to which the rock material and the discontinuities separately affect the behavior of the rock mass. In the case of underground excavations, a discontinuity is considered fully continuous if its length is greater than the dimension of the excavation. Consequently, for continuity assessment, the length of the discontinuity should be determined.

Weathering of the wall rock, that is, the rock constituting the discontinuity surfaces, is classified in accordance with the recommendations of the ISRM Committee on Rock Classification (1981b):

1. Unweathered/fresh. No visible signs of weathering are noted: rock fresh; crystals bright.
2. Slightly weathered rock. Discontinuities are stained or discolored and may contain a thin filling of altered material. Discoloration may extend into the rock from the discontinuity surfaces to a distance of up to 20% of the discontinuity spacing.
3. Moderately weathered rock. Slight discoloration extends from discontinuity planes for greater than 20% of the discontinuity spacing. Discontinuities may contain filling of altered material. Partial opening of grain boundaries may be observed.
4. Highly weathered rock. Discoloration extends throughout the rock, and the rock material is partly friable. The original texture of the rock has mainly been preserved, but separation of the grains has occurred.
5. Completely weathered rock. The rock is totally discolored and decomposed and in a friable condition. The external appearance is that of soil.

The *infilling (gouge)* has a twofold influence: a) depending on the thickness, the filling prevents the interlocking of the fracture asperities; and b) it possesses its own characteristic properties, that is, shear strength, permeability, and deformational characteristics. The following aspects should be described: type, thickness, continuity, and consistency.

Groundwater Conditions In the case of tunnels or mine drifts, the rate of inflow of groundwater in liters per minute per 10 meters of the excavation should be determined. Alternatively, general conditions can be described as completely dry, damp, wet, dripping, and flowing. If actual water pressure data are available, these should be stated and expressed in terms of the ratio of the water pressure to the major principal stress.

2.5 DESIGN METHODOLOGIES

The topic of design methodology as related to rock mass classifications is important for two reasons. Firstly, rock mass classifications are based on case histories and hence tend to perpetuate conservative practice unless they are seen as a design aid, requiring periodic updating. Secondly, they represent only one type of the design methods, an empirical one, which needs to be used in conjunction with other design methods. A good design methodology can ensure that rock mass classifications are used with the greatest effect and that they do not hamper but promote design innovation and state-of-the-art technology.

Various definitions of engineering design have been given (Bieniawski, 1984). In general, engineering design may be defined as that socioeconomic activity by which scientific, engineering, and behavioral principles, together with technical informaton and experience, are applied with skill, imagination, and judgment in the creation of functional economical, aesthetically pleasing, and environmentally acceptable devices, processes, or systems for the benefit of the society. The design process embraces all those activities and events that occur between the recognition of a social need or opportunity and the detailed specification of acceptable solution. The designer's responsibility continues during the manufacture or construction of the project and even beyond it.

The distinguishable stages of the engineering design process (Bieniawski, 1988) are

1. Recognition of a need.
2. Statement of the problem, identification of performance objectives, and design issues.
3. Collection of information.
4. Concept formulation in accordance with the design criteria: search for a method, theory, model, or hypothesis.
5. Analysis of solution components.
6. Synthesis to create detailed alternative solutions.
7. Evaluation of ideas and solutions.
8. Optimization.
9. Recommendation and communication.
10. Implementation.

Obert (1973) emphasized that, compared with the time that man has been mining underground, the concept of designing an underground opening is a relatively recent innovation. One reason for this is that the problem of designing a mine or a tunnel is different from that of designing a conventional structure such as a building or a bridge.

In a conventional engineering design, the external loads to be applied are first determined and a material is then prescribed with the appropriate strength and deformation characteristics, following which the structural geometry is selected. In rock mechanics, the designer deals with complex rock masses, and specific material properties cannot be prescribed to meet design requirements. Furthermore, the applied loads are not as important in rock masses as the forces resulting from the redistribution of the original stresses, that is, those existing before the excavation was made. Also, a number of possible failure modes can exist in a rock structure, so determination of the "material strength" is a major problem. Finally, the geometry of a structure in rock may depend on the configuration of the geological features. Hence, the design of an excavation in rock must include a thorough appraisal of the geological conditions and, especially, possible geological hazards.

In essence, rock mechanics design in mining and tunneling incorporates such aspects as planning the location of structures, determining their dimensions and shapes, their orientations and layout, excavation procedures (blasting or machine boring), support selection, and instrumentation. The rock mechanics engineer studies the original in-situ stresses, monitors the changes in stress due to mining or tunneling, determines rock properties, analyzes stresses, deformations, and water conditions (pressure and flow), and interprets instrumentation data.

Unfortunately, the application of improved geotechnical design concepts in mining and tunneling has not progressed at the same rate as for other engineering works. The result has been excessive safety factors in many aspects of underground projects. It is believed that an increasing demand for more realistic safety factors as well as the recognition of the money-saving potential of rock mechanics will lead to greater application of rock mechanics design in mining and tunneling. Nevertheless, while extensive research is being conducted in rock mechanics today, there still seems to be a major problem in "translating" the research findings into innovative and concise design procedures.

The design methods which are available for assessing the stability of mines and tunnels can be categorized as follows:

1. Analytical methods.
2. Observational methods.
3. Empirical methods.

Analytical design methods utilize the analyses of stresses and deformations around openings. They include such techniques as closed-form solutions, numerical methods (finite elements, finite difference, boundary elements, etc.), analog simulations (electrical and photoelastic), and physical modeling.

Observational design methods rely on actual monitoring of ground movement during excavation to detect measurable instability and on the analysis of ground–support interaction. Although considered separate methods, these observational approaches are the only way to check the results and predictions of the other methods.

Empirical design methods assess the stability of mines and tunnels by the use of statistical analyses of underground observations. Engineering rock mass classifications constitute the best-known empirical approach for assessing the stability of underground openings in rock (Goodman, 1980; Hoek and Brown, 1980).

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3

Early Rock Mass Classifications

Observation, not old age, brings wisdom.
—Plubilius Sententiae

Empirical design methods relate practical experience gained on previous projects to the conditions anticipated at a proposed site.

Rock mass classifications form the backbone of the empirical design approach and are widely employed in rock engineering. In fact, on many projects, the classification approach serves as the only practical basis for the design of complex underground structures. Most tunnels now constructed make use of some classification system. The most used and the best known of these is Terzaghi's rock load classification, which was introduced over 40 years ago (Terzaghi, 1946). Since then, this classification has been modified (Deere et al., 1970) and new rock classification systems have been proposed. These systems took cognizance of the new advances in rock support technology, namely, rockbolts and shotcrete, and addressed different engineering projects: tunnels, chambers, mines, slopes, and foundations. Today, so many different rock classification systems exist that it is useful to tabulate the more common ones, as shown in Table 3.1.

Rock mass classifications have been successfully applied throughout the world: in the United States (Deere et al., 1967; Wickham et al., 1972; Bieniawski, 1979), Canada (Coates, 1964; Franklin, 1976), western Europe

TABLE 3.1 Major Engineering Rock Mass Classifications Currently in Use

Name of Classification	Originator and Date	Country of Origin	Applications
1. Rock load	Terzaghi, 1946	USA	Tunnels with steel support
2. Stand-up time	Lauffer, 1958	Austria	Tunneling
3. NATM	Pacher et al., 1964	Austria	Tunneling
4. Rock quality designation	Deere et al., 1967	USA	Core logging, tunneling
5. RSR concept	Wickham et al., 1972	USA	Tunneling
6. **RMR system** (Geomechanics Classification)	Bieniawski, 1973 (last modified, 1979—USA)	South Africa	Tunnels, mines, slopes, foundations
RMR system extensions			
	Weaver, 1975	South Africa	Rippability
	Laubscher, 1977	South Africa	Mining
	Olivier, 1979	South Africa	Weatherability
	Ghose and Raju, 1981	India	Coal mining
	Moreno Tallon, 1982	Spain	Tunneling
	Kendorski et al., 1983	USA	Hard rock mining

	Nakao et al., 1983	Japan	Tunneling
	Serafim and Pereira, 1983	Portugal	Foundations
	Gonzalez de Vallejo, 1983	Spain	Tunneling
	Unal, 1983	USA	Roof bolting in coal mines
	Romana, 1985	Spain	Slope stability
	Newman, 1985	USA	Coal mining
	Sandbak, 1985	USA	Boreability
	Smith, 1986	USA	Dredgeability
	Venkateswarlu, 1986	India	Coal mining
	Robertson, 1988	Canada	Slope stability
7. **Q-system**	Barton et al., 1974	Norway	Tunnels, chambers
Q-system extensions	Kirsten, 1982	South Africa	Excavatability
	Kirsten, 1983	South Africa	Tunneling
8. Strength–size	Franklin, 1975	Canada	Tunneling
9. Basic geotechnical description	International Society for Rock Mechanics, 1981		General, communication
10. Unified classification	Williamson, 1984	USA	General, communication

(Lauffer, 1958; Pacher et al., 1974; Barton et al., 1974), South Africa (Bieniawski, 1973; Laubscher, 1977; Olivier, 1979), Australia (Baczynski, 1980), New Zealand (Rutledge, 1978), Japan (Nakao, 1983), India (Ghose and Raju, 1981), the USSR (Protodyakonov, 1974), and in Poland (Kidybinski, 1979).

Of the many rock mass classification systems in existence, six require special attention because they are most common, namely, those proposed by Terzaghi (1946), Lauffer (1958), Deere et al. (1967), Wickham et al. (1972), Bieniawski (1973), and Barton et al. (1974).

The rock load classification of Terzaghi (1946) was the first practical classification system introduced and has been dominant in the United States for over 35 years, proving very successful for tunneling with steel supports. Lauffer's classification (1958) was based on the work of Stini (1950) and was a considerable step forward in the art of tunneling since it introduced the concept of the stand-up time of the active span in a tunnel, which is highly relevant in determining the type and amount of tunnel support. The classification of Deere et al. (1967) introduced the rock quality designation (RQD) index, which is a simple and practical method of describing the quality of rock core from boreholes. The concept of rock structure rating (RSR), developed in the United States by Wickham et al. (1972, 1974), was the first system featuring classification ratings for weighing the relative importance of classification parameters. The Geomechanics Classification (RMR system), proposed by Bieniawski (1973), and the Q-system, proposed by Barton et al. (1974), were developed independently and both provide quantitative data for the selection of modern tunnel reinforcement measures such as rock bolts and shotcrete. The Q-system has been developed specifically for tunnels and chambers, whereas the Geomechanics Classification, although also initially developed for tunnels, has been applied to rock slopes and foundations, ground rippability assessment, and mining problems (Laubscher, 1977; Ghose and Raju, 1981; Kendorski et al., 1983).

3.1 ROCK LOAD CLASSIFICATION METHOD

Terzaghi (1946) formulated the first rational method of classification by evaluating rock loads appropriate to the design of steel sets. This was an important development because support by steel sets has been the most commonly used system for containing rock tunnel excavations during the past 50 years. It must be emphasized, however, that while this classification is appropriate for the purpose for which it was evolved, that is, for estimating

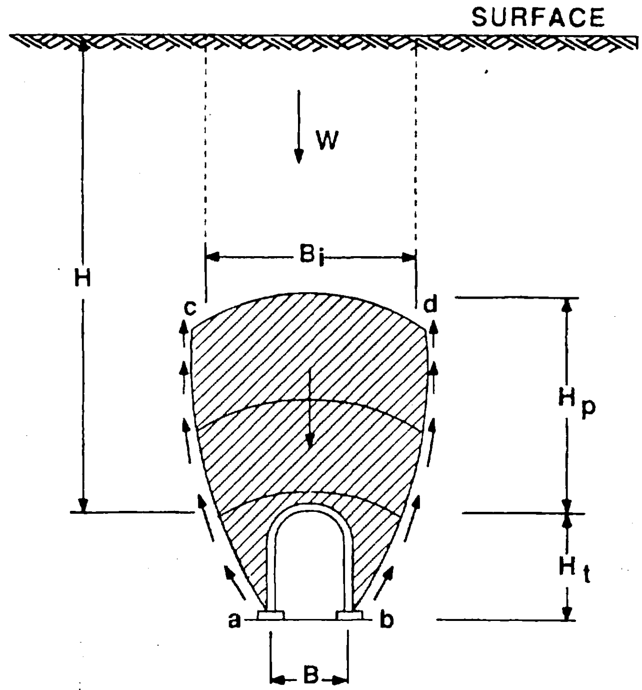


Figure 3.1 *The tunnel rock-load concept of Terzaghi (1946).*

rock loads for steel-arch supported tunnels, it is not as suitable for modern tunneling methods using shotcrete and rockbolts. After detailed studies, Cecil (1970) concluded that Terzaghi's classification was too general to permit an objective evaluation of rock quality and that it provided no quantitative information on the properties of rock masses.

The main features of Terzaghi's classification are depicted in Figure 3.1 and are listed in Tables 3.2 and 3.3. The rock load values in Table 3.2 apply to the described ground conditions if the tunnel is located under the water table. If the tunnel is located above the groundwater level, the rock loads for classes 4–6 can be reduced by 50%. An important revision of Terzaghi's rock load coefficients was presented by Rose (1982)—see Table 3.3—who showed that Terzaghi's rock conditions 4–6 should be reduced by 50% from their original rock load values because water table has little effect on the rock load (Brekke, 1968).

3.2 STAND-UP TIME CLASSIFICATION

The 1958 classification by Lauffer has its foundation in the earlier work on tunnel geology by Stini (1950), considered the father of the "Austrian School" of tunneling and rock mechanics. Stini emphasized the importance of structural

TABLE 3.2 Original Terzaghi's Rock Load Classification of 1946[a,b]

Rock Condition[c]	Rock Load H_p (ft)	Remarks
1. Hard and intact	Zero	Light lining required only if spalling or popping occurs.
2. Hard stratified or schistose	$0-0.5B$	Light support, mainly for protection against spalls. Load may change erratically from point to point
3. Massive, moderately jointed	$0-0.25B$	
4. Moderately blocky and seamy	$0.25B-0.35(B + H_t)$	No side pressure
5. Very blocky and seamy	$(0.35-1.10)(B + H_t)$	Little or no side pressure
6. Completely crushed	$1.10(B + H_t)$	Considerable side pressure Softening effects of seepage toward bottom of tunnel require either continuous support for lower ends of ribs or circular ribs
7. Squeezing rock, moderate depth	$(1.10-2.10)(B + H_t)$	Heavy side pressure, invert struts required. Circular ribs are recommended
8. Squeezing rock, great depth	$(2.10-4.50)(B + H_t)$	
9. Swelling rock	Up to 250 ft, irrespective of the value of $(B + H_t)$	Circular ribs are required. In extreme cases, use yielding support

[a] After Terzaghi (1946).

[b] Rock load H_p in feet on tunnel roof with width B (ft) and height H_t (ft) at depth of more than $1.5(B + H_t)$.

[c] *Definitions:*

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

Stratified rock consists of individual strata with little or no resistance against separation along the boundaries between strata. The strata may or may not be weakened by transverse joints. In such rock, the spalling condition is quite common.

Moderately jointed rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type, both spalling and popping conditions may be encountered.

Blocky and *seamy* rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support.

Crushed but chemically intact rock has the character of a crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.

Squeezing rock slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and submicroscopic particles of micaceous minerals or of clay minerals with a low swelling capacity.

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

TABLE 3.3 Terzaghi's Rock Load Classification Currently in Use[a,b]

Rock Condition	RQD	Rock Load H_p (ft)	Remarks
1. Hard and intact	95–100	Zero	Same as Terzaghi (1946)
2. Hard stratified or schistose	90–99	0–0.5 B	Same as Terzaghi (1946)
3. Massive, moderately jointed	85–95	0–0.25 B	Same as Terzaghi (1946)
4. Moderately blocky and seamy	75–85	0.25 B–0.20 $(B + H_t)$	Types 4, 5, and 6 reduced by about 50% from Terzaghi values because water table has little effect on rock load (Terzaghi, 1946; Brekke, 1968)
5. Very blocky and seamy	30–75	(0.20–0.60) $(B + H_t)$	
6. Completely crushed but chemically intact	3–30	(0.60–1.10) $(B + H_t)$	
6a. Sand and gravel	0–3	(1.10–1.40) $(B + H_t)$	
7. Squeezing rock, moderate depth	NA[c]	(1.10–2.10) $(B + H_t)$	Same as Terzaghi (1946)
8. Squeezing rock, great depth	NA[c]	(2.10–4.50) $(B + H_t)$	Same as Terzaghi (1946)
9. Swelling rock	NA[c]	Up to 250 ft irrespective of value of $(B + H_t)$	Same as Terzaghi (1946)

[a] As modified by Deere et al. (1970) and Rose (1982).
[b] Rock load H_p in feet of rock on roof of support in tunnel with width B (ft) and height H_t (ft) at depth of more than 1.5 $(B + H_t)$.
[c] Not applicable.

defects in rock masses. Lauffer proposed that the stand-up time for any active unsupported rock span is related to the various rock mass classes. An active unsupported span is the width of the tunnel or the distance from the face to the support if this is less than the tunnel width. The stand-up time is the period of time that a tunnel will stand unsupported after excavation. It should be noted that a number of factors may affect the stand-up time, such as orientation of tunnel axis, shape of cross section, excavation method, and support method. Lauffer's original classification is no longer used, since it has been modified a number of times by other Austrian engineers, notably by Pacher et al. (1974), leading to the development of the New Austrian Tunneling Method.

The main significance of the Lauffer–Pacher classification is that an increase in tunnel span leads to a major reduction in the stand-up time. This means, for example, that while a pilot tunnel having a small span may be successfully constructed full face in fair rock conditions, a large span opening in this same rock may prove impossible to support in terms of the stand-up time. Only with a system of smaller headings and benches or multiple drifts can a large cross-sectional tunnel be constructed in such rock conditions.

This classification introduced the stand-up time and the span as relevant parameters in determining the type and amount of tunnel support, and it has influenced the development of more recent rock mass classification systems.

3.3 ROCK QUALITY DESIGNATION (RQD) INDEX

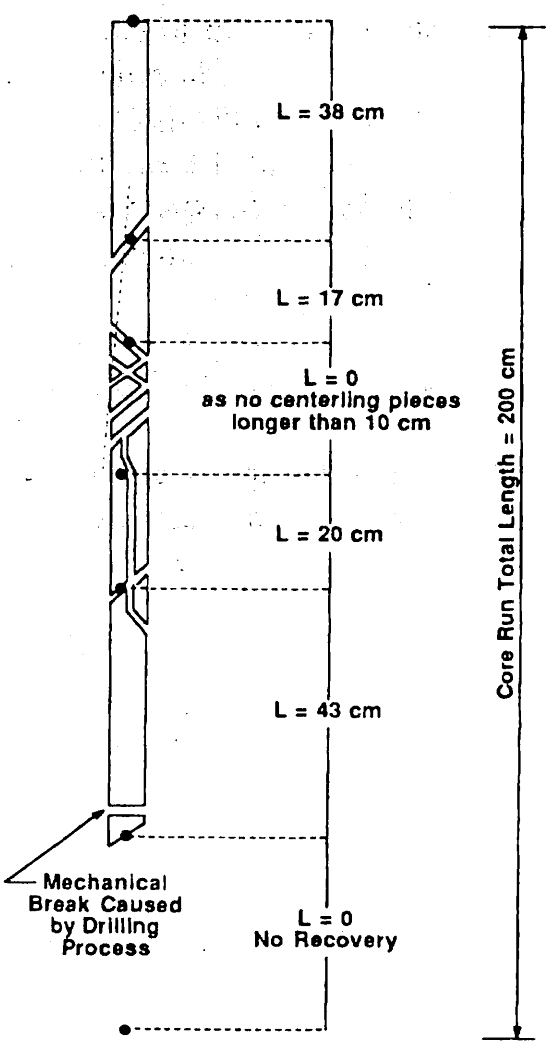
The rock quality designation (RQD) index was introduced over 20 years ago as an index of rock quality at a time when rock quality information was usually available only from the geologists' descriptions and the percentage of core recovery (Deere and Deere, 1988).

D. U. Deere developed that index in 1964, but it was not until 1967 that the concept was presented for the first time in a published form (Deere et al., 1967). The RQD is a modified core-recovery percentage which incorporates only sound pieces of core that are 100 mm (4 in.) or greater in length. This quantitative index has been widely used as a red flag to identify low-quality rock zones which deserve greater scrutiny and which may require additional borings or other exploratory work.

For RQD determination, the International Society for Rock Mechanics recommends a core size of at least NX diameter (54.7 mm) drilled with double-tube core barrels. The following relationship between the RQD index and the engineering quality of the rock was proposed by Deere (1968):

RQD (%)	Rock Quality
<25	Very poor
25–50	Poor
50–75	Fair
75–90	Good
90–100	Excellent

The correct procedure for measuring RQD is illustrated in Figure 3.2. It should be noted that the RQD percentage includes only the pieces of sound core over 100 mm (4 in.) long, which are summed and divided by the length of the core run. In this respect, pieces of core that are not hard and sound should not be counted even though they possess the requisite 100-mm length. Thus, highly weathered rock will receive zero RQD. Concerning the core



$$RQD = \frac{\sum \text{Length of Core Pieces} > 10 \text{ cm (4 in.)}}{\text{Total Core Run Length}} \times 100\%$$

$$RQD = \frac{38 + 17 + 20 + 43}{200} \times 100\%$$

RQD = 59% (FAIR)

Figure 3.2 *Procedure for measurement and calculation of rock quality designation. (After Deere, 1989.)*

run, the RQD calculations should be based on the actual drilling-run length used in the field, preferably no greater than 1.5 m (5 ft). The core length is measured along the centerline (see Fig. 3.2). The optimal core diameters are the NX size and NQ size (47.5 mm or 1.87 in.), but sizes between BQ and PQ with core diameters of 36.5 mm (1.44 in.) and 85 mm (3.35 in.) may be used provided careful drilling that does not cause core breakage by itself is utilized.

Cording and Deere (1972) attempted to relate the RQD index to Terzaghi's rock load factors and presented tables relating tunnel support and RQD. They found that Terzaghi's rock load concept should be limited to tunnels supported by steel sets, as it does not apply well to openings supported by rock bolts.

Merritt (1972) found that the RQD could be of considerable value in estimating support requirements for rock tunnels. He compared the support criteria based on his improved version, as a function of tunnel width and RQD, with those proposed by others. This is summarized in Table 3.4, compiled by Deere and Deere (1988).

TABLE 3.4 Comparison of RQD and Support Requirements for a 6-m (20-ft)-wide Tunnel[a]

	No Support or Local Bolts	Pattern Bolts	Steel Ribs
Deere et al. (1970)	RQD 75–100	RQD 50–75 (1.5–1.8-m spacing) RQD 25–50 (0.9–1.5-m spacing)	RQD 50–75 (light ribs on 1.5–1.8-m spacing as alternative to bolts) RQD 25–50 (light to medium ribs on 0.9–1.5-m spacing as alternative to bolts) RQD 0–25 (medium to heavy circular ribs on 0.6–0.9-m spacing)
Cecil (1970)	RQD 82–100	RQD 52–82 (alternatively, 40–60-mm shotcrete)	RQD 0–52 (ribs or reinforced shotcrete)
Merritt (1972)	RQD 72–100	RQD 23–72 (1.2–1.8-m spacing)	RQD 0–23

[a] Data interpolated from Merritt (1972) by Deere and Deere (1988).

Palmstrom (1982) has suggested that when core is unavailable the RQD may be estimated from the number of joints (discontinuities) per unit volume, in which the number of joints per meter for each joint set is added. The conversion for clay-free rock masses is

$$\text{RQD} = 115 - 3.3J_v \tag{3.1}$$

where J_v represents the total number of joints per cubic meter.

A secondary outcome of the RQD research in the late 1960s was the correlation of the RQD with the in-situ modulus of deformation, but this has not been used much in recent years (Deere and Deere, 1988).

Today, the RQD is used as a standard parameter in drill core logging and forms a basic element of the two major rock mass classification systems: the RMR system and the Q-system.

Although the RQD is a simple and inexpensive index, alone it is not sufficient to provide an adequate description of a rock mass because it disregards joint orientation, tightness, and gouge (infilling) material. Essentially, it is a practical parameter based on "a measurement of the percentage of 'good' rock (core) interval of a borehole" (Deere and Deere, 1988).

3.4 ROCK STRUCTURE RATING (RSR) CONCEPT

The RSR concept, a ground-support prediction model, was developed in the United States in 1972 by Wickham, Tiedemann, and Skinner. The concept presents a quantitative method for describing the quality of a rock mass and for selecting the appropriate ground support. It was the first complete rock mass classification system proposed since that introduced by Terzaghi in 1946.

The RSR concept was a step forward in a number of respects: first, it was a quantitative classification, unlike Terzaghi's qualitative one; second, it was a rock mass classification incorporating many parameters, unlike the RQD index, which is limited to core quality; third, it was a complete classification having an input and an output, unlike a Lauffer-type classification that relies on practical experience to decide on a rock mass class and which then gives an output in terms of the stand-up time and span.

The main contribution of the RSR concept was that it introduced a rating system for rock masses. This was the sum of the weighted values of the individual parameters considered in this classification system. In other words, the relative importance of the various classification parameters could be assessed. This rating system was determined on the basis of case histories as well as reviews of various books and technical papers dealing with different aspects of ground support in tunneling.

The RSR concept considered two general categories of factors influencing rock mass behavior in tunneling: geological parameters and construction parameters. The geologic parameters were a) rock type; b) joint pattern (average spacing of joints); c) joint orientations (dip and strike); d) type of discontinuities; e) major faults, shears, and folds; f) rock material properties; and g) weathering or alteration. Some of these factors were treated separately; others were considered collectively. The developers pointed out (Wickham et al., 1972) that in some instances it would be possible to define the above factors accurately, but in others, only general approximations could be made. The construction parameters were a) size of tunnel, b) direction of drive, and c) method of excavation.

All the above factors were grouped by Wickham, Tiedemann, and Skinner (1972) into three basic parameters, A, B, and C (Tables 3.5, 3.6, and 3.7, respectively), which in themselves were evaluations as to the relative effect of various geological factors on the support requirements. These three parameters are as follows:

1. *Parameter A:* General appraisal of a rock structure on the basis of
   a. Rock type origin (igneous, metamorphic, sedimentary).
   b. Rock hardness (hard, medium, soft, decomposed).
   c. Geologic structure (massive, slightly faulted/folded, moderately faulted/folded, intensely faulted/folded).
2. *Parameter B:* Effect of discontinuity pattern with respect to the direction of tunnel drive is on the basis of
   a. Joint spacing.
   b. Joint orientation (strike and dip).
   c. Direction of tunnel drive.
3. *Parameter C:* Effect of groundwater inflow based on
   a. Overall rock mass quality due to parameters A and B combined.
   b. Joint condition (good, fair, poor).
   c. Amount of water inflow (in gallons per minute per 1000 feet of the tunnel).

The RSR value of any tunnel section is obtained by summing the weighted numerical values determined for each parameter. Thus, RSR $= A + B + C$, with a maximum value of 100. The RSR reflects the quality of the rock mass with respect to its need for support. Since a lesser amount of support

Since 90% of the case-history tunnels were supported with steel ribs, the RR measure was chosen as the theoretical support (rib size and spacing). It was developed from Terzaghi's formula for determining roof loads in loose sand below the water table (datum condition). Using the tables provided in *Rock Tunneling with Steel Supports* (Terzaghi, 1946), the theoretical spacing required for the same size rib as used in a given case-study tunnel section was determined for the datum condition. The RR value is obtained by dividing this theoretical spacing by the actual spacing and multiplying the answer by 100. Thus, RR = 46 would mean that the section required only 46% of the support used for the datum condition. However, differently sized tunnels, although having the same RR, would require different weight or size of ribs for equivalent support. The RR for an unsupported tunnel would be zero; for a tunnel requiring the same support as the datum condition, it would be 100.

An empirical relationship was developed between RSR and RR values, namely

$$\begin{aligned} (RR + 80)(RSR + 30) &= 8800 \\ \text{or} \quad (RR + 70)(RSR + 8) &= 6000 \end{aligned} \tag{3.2}$$

It was concluded that rock structures with RSR values less than 19 would require heavy support, whereas those with ratings of 80 and over would be unsupported.

Since the RR basically defined an anticipated rock load by considering the load-carrying capacity of different sizes of steel ribs, the RSR values were also expressed in terms of unit rock loads for variously sized tunnels.

A total of 53 projects were evaluated, but since each tunnel was divided into typical geological sections, a total of 190 tunnel sections were analyzed. The RSR values were determined for each section, and actual support installations were obtained from as-built drawings.

The support was distributed as follows:

Sections with steel ribs:	147	(89.6%)
Sections with rock bolts:	14	(8.6%)
Sections with shotcrete:	3	(1.8%)
Total supported:	164	(100.0%)
Total unsupported:	26	
Total:	190 sections	

The RSR prediction model was developed primarily with respect to steel rib support. Insufficient data were available to correlate rock structures and rock bolt or shotcrete support. However, an appraisal of rock bolt requirements was made by considering rock loads with respect to the tensile strength of the bolt. The authors pointed out (Wickham et al., 1972) that this was a very general approach: it assumed that anchorage was adequate and that all bolts acted in tension only; it did not allow either for interaction between adjacent blocks or for an assumption of a compression arch formed by the bolts. In addition, the rock loads were developed for steel supported tunnels. Nevertheless, the following relation was given for 25-mm diameter rock bolts with a working load of 24,000 lb:

$$\text{Spacing (ft)} = \frac{24}{W} \tag{3.3}$$

where W is the rock load in 1000 lb/ft^2.

No correlation could be found between geologic condition and shotcrete requirements, so the following empirical relationship was suggested:

$$t = 1 + \frac{W}{1.25} \quad \text{or} \quad t = D\,\frac{65 - \text{RSR}}{150} \tag{3.4}$$

where t = shotcrete thickness, in.;
W = rock load, lb/ft^2;
D = tunnel diameter, ft.

Support requirement charts have been prepared that provide a means of determining typical ground-support systems based on RSR prediction as to the quality of the rock mass through which the tunnel is to be driven. Charts for 3-m-, 6-m-, 7-m-, and 10-m-diameter tunnels are available, an example being given in Figure 3.3. The three steel rib curves reflect typical sizes used for the particular tunnel size. The curves for rock bolts and shotcrete are dashed to emphasize that they are based on assumptions and not derived from case histories. The charts are applicable to either circular or horseshoe-shaped tunnels of comparable widths.

The RSR concept is a very useful method for selecting steel rib support for rock tunnels. As with any empirical approach, one should not apply the concept beyond the range of the sufficient and reliable data used for developing it. For this reason, the RSR concept is not recommended for selection of rock bolt and shotcrete support. It should be noted that although definitions of the classification parameters were not explicitly stated by the proposers, most of the input data needed would normally be included in a standard joint survey; however, the lack of definitions (e.g., "slightly faulted" or "folded" rock) may lead to some confusion.

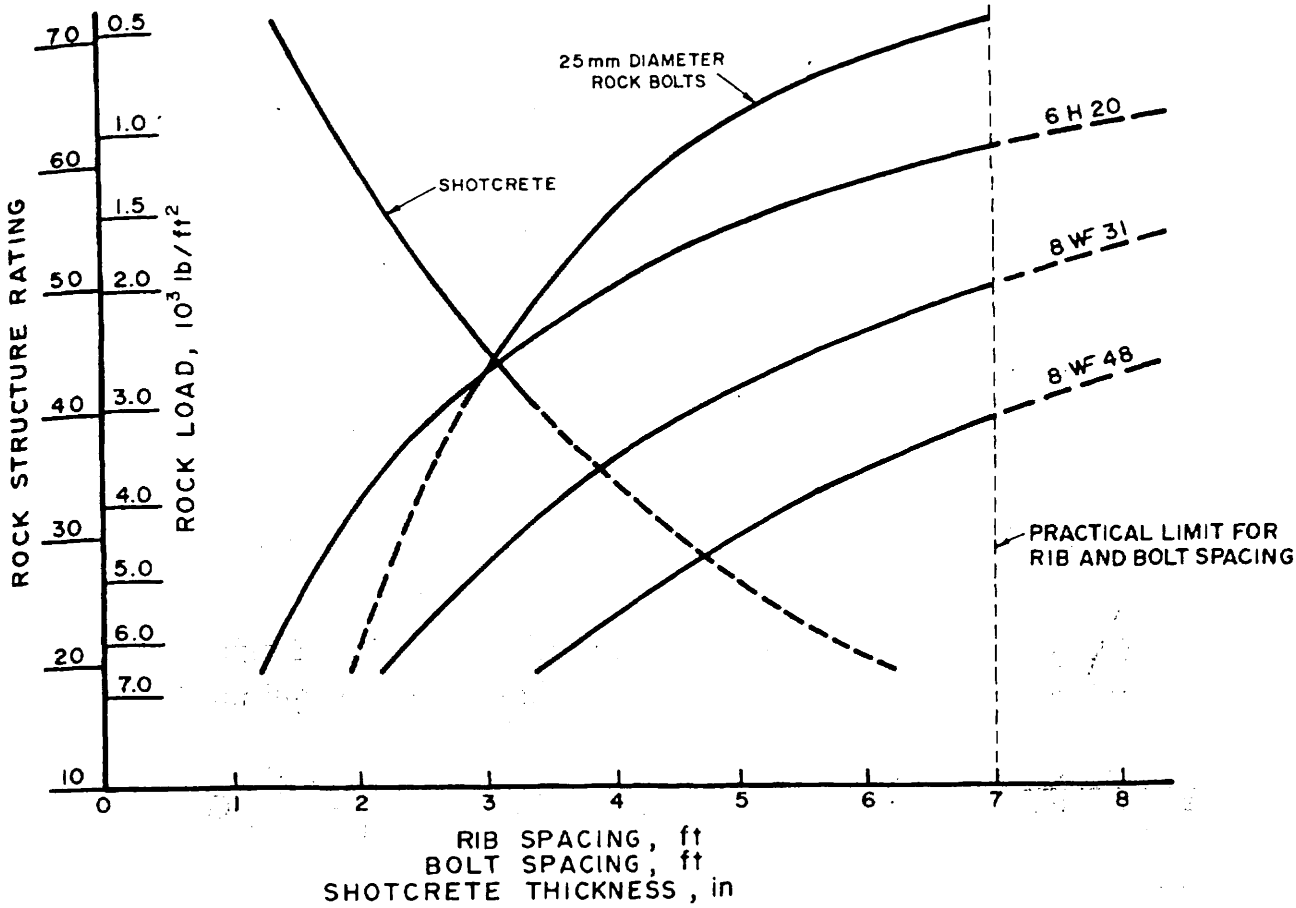


Figure 3.3 *RSR concept: support chart for a 24-ft- (7.3-m-) diameter tunnel. (After Wickham et al., 1972.)*

Sinha (1988) pointed out that while the RSR provides a rib ratio, to use this ratio one has to find Terzaghi's rock load and steel rib spacing and then reduce Terzaghi's rib spacing to correspond to the obtained rib ratio. It is not possible to prescribe the steel ribs or rock bolts without using the Terzaghi system. Thus according to Sinha (1988), the RSR concept may be viewed as an improvement of Terzaghi's method rather than an independent system.

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4

Geomechanics Classification (Rock Mass Rating System)

If you can measure what you are speaking about and express it in numbers, you know something about it.
—Lord Kelvin

The Rock Mass Rating (RMR) system, otherwise known as the Geomechanics Classification, was developed by the author during 1972–1973 (Bieniawski, 1973). It was modified over the years as more case histories became available and to conform with international standards and procedures (Bieniawski, 1979). Over the past 15 years, the RMR system has stood the test of time and benefited from extensions and applications by many authors throughout the world. These varied applications, amounting to 351 case histories (see Chap. 10), point to the acceptance of the system and its inherent ease of use and versatility in engineering practice, involving tunnels, chambers, mines, slopes, and foundations. Nevertheless, it is important that the RMR system is used for the purpose for which it was developed and not as the answer to all design problems.

Definition of the System Due to the RMR system having been modified several times, and since the method is interchangeably known as the Geomechanics Classification or the Rock Mass Rating system, it is important to state that the system has remained essentially the same in principle despite the changes. Thus, any modifications and extensions were the outgrowth of the same basic method and should not be misconstrued as new systems. To avoid any confusion, the following extensions of the system were valuable new applications but still a part of the same overall RMR system: mining applications, Laubscher (1977, 1984); rippability, Weaver (1975); hard rock mining, Kendorski et al. (1983); coal mining, Unal (1983), Newman and Bieniawski (1986); dam foundations, Serafim and Pereira (1983); tunneling, Gonzalez de Vallejo (1983); slope stability, Romana (1985); and Indian coal mines (Venkateswarlu, 1986).

Moreover, some users of the RMR system list their results as "CSIR rating" or talk of the "CSIR Geomechanical" system. This is incorrect and has never been used or endorsed by the author. The correct expressions are "Rock Mass Rating system" or the "RMR system," or the "Geomechanics Classification." While it is true that the author has worked for an organization whose initials are "CSIR," that organization did not develop the system, and indeed, most of the work on this system was performed after he left the CSIR some 12 years ago.

4.1 CLASSIFICATION PROCEDURES

The following six parameters are used to classify a rock mass using the RMR system (Geomechanics Classification):

1. Uniaxial compressive strength of rock material.
2. Rock quality designation (RQD).
3. Spacing of discontinuities.
4. Condition of discontinuities.
5. Groundwater conditions.
6. Orientation of discontinuities.

To apply the Geomechanics Classification, the rock mass is divided into a number of structural regions such that certain features are more or less uniform within each region. Although rock masses are discontinuous in nature, they may nevertheless be uniform in regions when, for example, the type of rock or the discontinuity spacings are the same throughout the region. In most cases, the boundaries of structural regions will coincide with major geological features such as faults, dykes, shear zones, and so on. After the structural regions have been identified, the classification parameters for each structural region are determined from measurements in the field and entered onto the input data sheet given in Figure 2.6.

The Geomechanics Classification is presented in Table 4.1.

In Section A of Table 4.1, five parameters are grouped into five ranges of values. Since the various parameters are not equally important for the overall classification of a rock mass, importance ratings are allocated to the different value ranges of the parameters, a higher rating indicating better rock mass conditions. The importance ratings are assigned to each parameter according to Section A of Table 4.1. In this respect, the average typical conditions are evaluated for each discontinuity set and the ratings are interpolated, using Classification Charts A–E. The charts are helpful for borderline cases and also remove an impression that abrupt changes in ratings occur between categories. Chart D is used if either RQD or discontinuity data are lacking. Based on the correlation data from Priest and Hudson (1976), the chart enables an estimate of the missing parameter. Furthermore, it should be noted that the importance ratings given for discontinuity spacings apply to rock masses having three sets of discontinuities. Thus, when only two sets of discontinuities are present, a conservative assessment is obtained. In this way, the number of discontinuity sets is considered indirectly. Laubscher (1977) presented a rating concept (see Chap. 8) for discontinuity spacings as a function of the number of joint sets. It can be shown that when less than three sets of discontinuities are present, the rating for discontinuity spacing may be increased by 30%.

After the importance ratings of the classification parameters are established, the ratings for the five parameters listed in Section A of Table 4.1 are summed to yield the basic (unadjusted for discontinuity orientations) RMR for the structural region under consideration.

The next step is to include the sixth parameter, namely, the influence of strike and dip orientation of discontinuities by adjusting the basic RMR according to Section B of Table 4.1. This step is treated separately because the influence of discontinuity orientations depends on the engineering applications, such as a tunnel, mine, slope, or foundation. It will be noted that the "value" of the parameter "discontinuity orientation" is not given in quantitative terms but by qualitative descriptions such as "favorable." To help decide whether strike and dip orientations are favorable or not in tunneling, reference should be made to Table 4.2, which is based on studies by Wickham et al. (1972). For slopes and foundations, the reader is referred to papers by Romana (1985) and by Bieniawski and Orr (1976), respectively.

The parameter "discontinuity orientation" reflects on the significance of the various discontinuity sets present in a rock mass. The main set, usually designated as set No. 1, controls the stability of an excavation; for example, in tunneling it will be the set whose strike is parallel to the tunnel axis. The

TABLE 4.1 The Rock Mass Rating System (Geomechanics Classification of Rock Masses)[a]

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point-load strength index (MPa)	>10	4–10	2–4	1–2	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength (MPa)	>250	100–250	50–100	25–50	5–25	1–5	<1
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD (%)		90–100	75–90	50–75	25–50	<25		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6–2 m	200–600 mm	60–200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered wall	Slickensided surfaces or Gouge < 5 mm thick or Separation 1–5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (L/min)	None	<10	10–25	25–125	>125		
			or	or	or	or	or		
		Ratio $\frac{\text{Joint water pressure}}{\text{Major principal stress}}$	0	<0.1	0.1–0.2	0.2–0.5	>0.5		
			or	or	or	or	or		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS

Strike and Dip Orientations of Discontinuities		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels and mines	0	−2	−5	−10	−12
	Foundations	0	−2	−7	−15	−25
	Slopes	0	−5	−25	−50	−60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

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

D. MEANING OF ROCK MASS CLASSES

Class no.	I	II	III	IV	V
Average stand-up time	20 yr for 15-m span	1 yr for 10-m span	1 wk for 5-m span	10 h for 2.5-m span	30 min for 1-m span
Cohesion of the rock mass (kPa)	>400	300−400	200−300	100−200	<100
Friction angle of the rock mass (deg)	>45	35−45	25−35	15−25	<15

[a] After Bieniawski (1979).

CHART A Ratings for Strength of Intact Rock

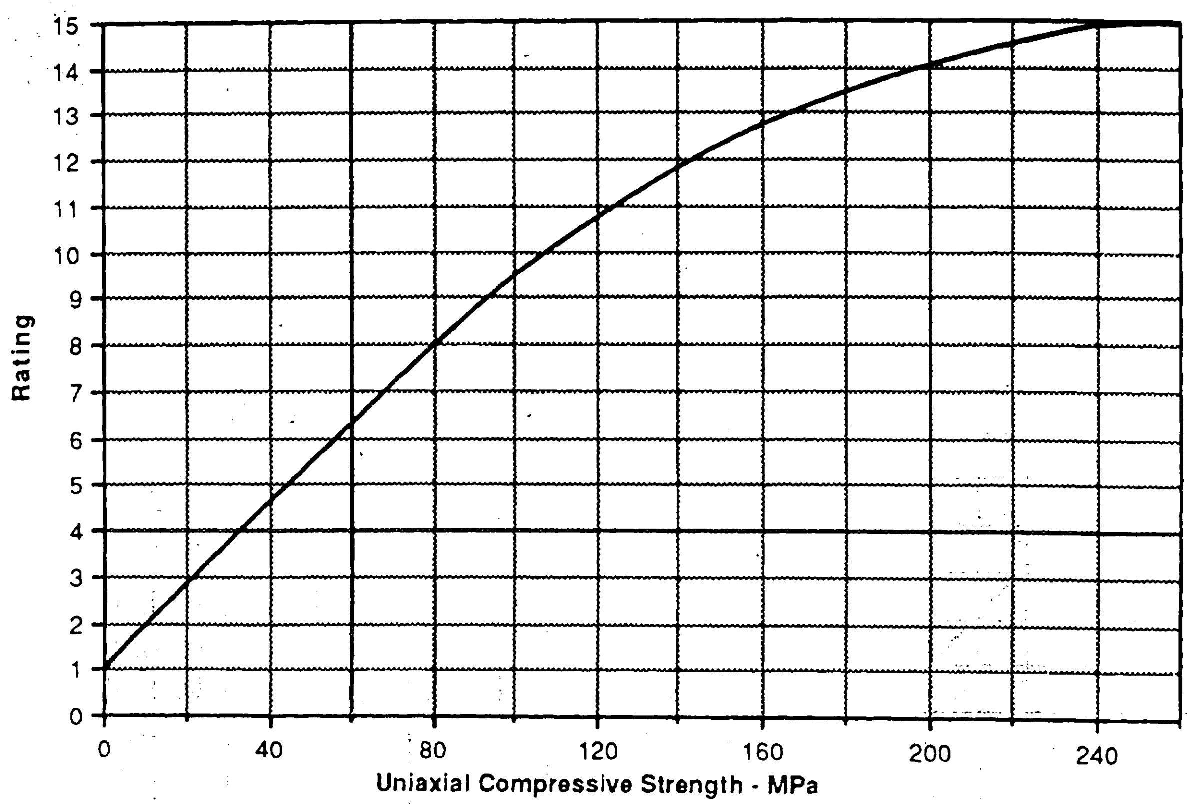


CHART B Ratings for RQD

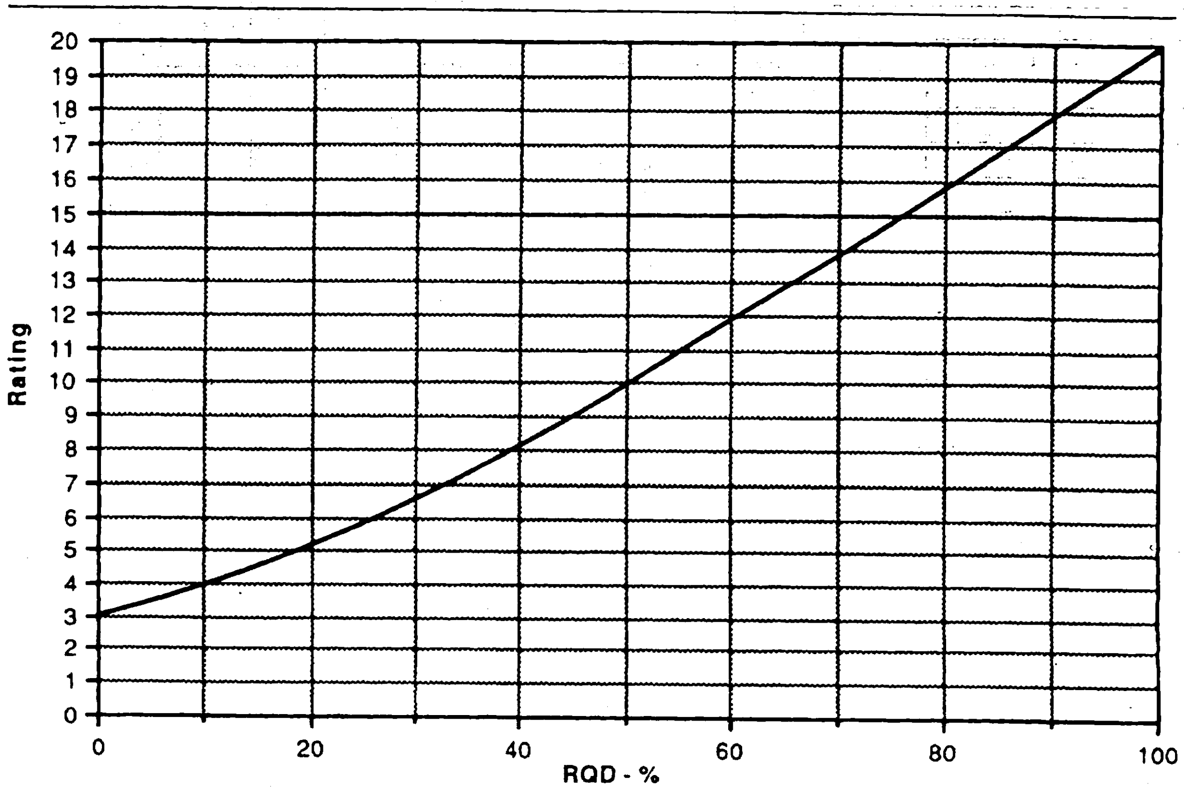


CHART C Ratings for Discontinuity Spacing

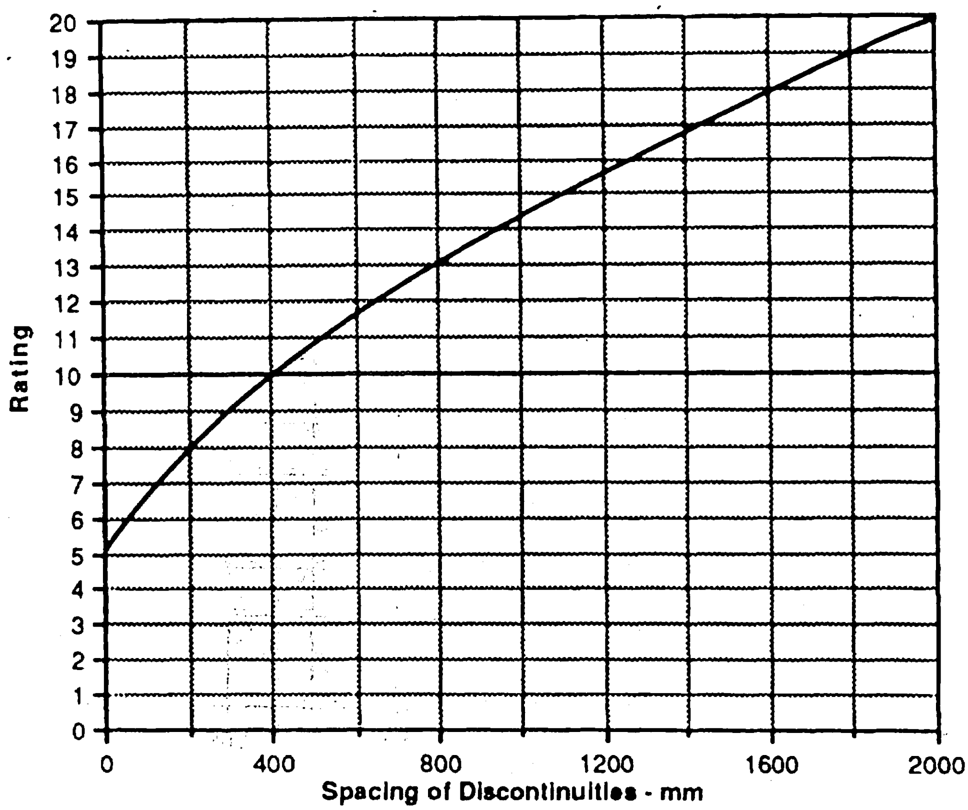


CHART D Chart for Correlation between RQD and Discontinuity Spacing

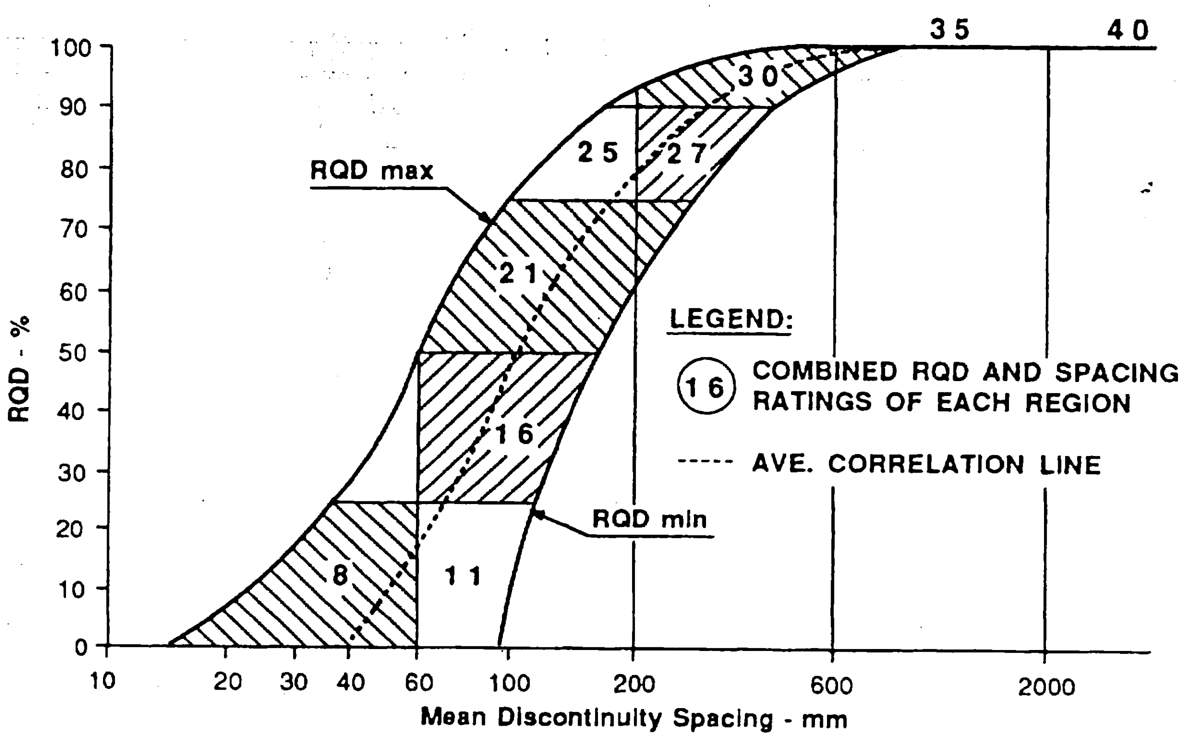


CHART E Guidelines for Classification of Discontinuity Conditions[a]

Parameter	Ratings				
Discontinuity length (persistence/continuity)	<1 m 6	1–3 m 4	3–10 m 2	10–20 m 1	>20 m 0
Separation (aperture)	None 6	<0.1 mm 5	0.1–1.0 mm 4	1–5 mm 1	>5 mm 0
Roughness	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
		Hard filling		Soft filling	
Infilling (gouge)	None 6	<5 mm 4	>5 mm 2	<5 mm 2	>5 mm 0
Weathering	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

[a] *Note:* Some conditions are mutually exclusive. For example, if infilling is present, it is irrelevant what the roughness may be, since its effect will be overshadowed by the influence of the gouge. In such cases, use Table 4.1 directly.

TABLE 4.2 Effect of Discontinuity Strike and Dip Orientations in Tunneling[a]

Strike Perpendicular to Tunnel Axis			
Drive with Dip		Drive against Dip	
Dip 45–90	Dip 20–45	Dip 45–90	Dip 20–45
Very favorable	Favorable	Fair	Unfavorable

Strike Parallel to Tunnel Axis		Irrespective of Strike
Dip 20–45	Dip 45–90	Dip 0–20
Fair	Very unfavorable	Fair

[a]Modified after Wickham et al. (1972).

summed-up ratings of the classification parameters for this discontinuity set will constitute the overall RMR. On the other hand, in situations where no one discontinuity set is dominant and of critical importance, or when estimating rock mass strength and deformability, the ratings from each discontinuity set are averaged for the appropriate individual classification parameter.

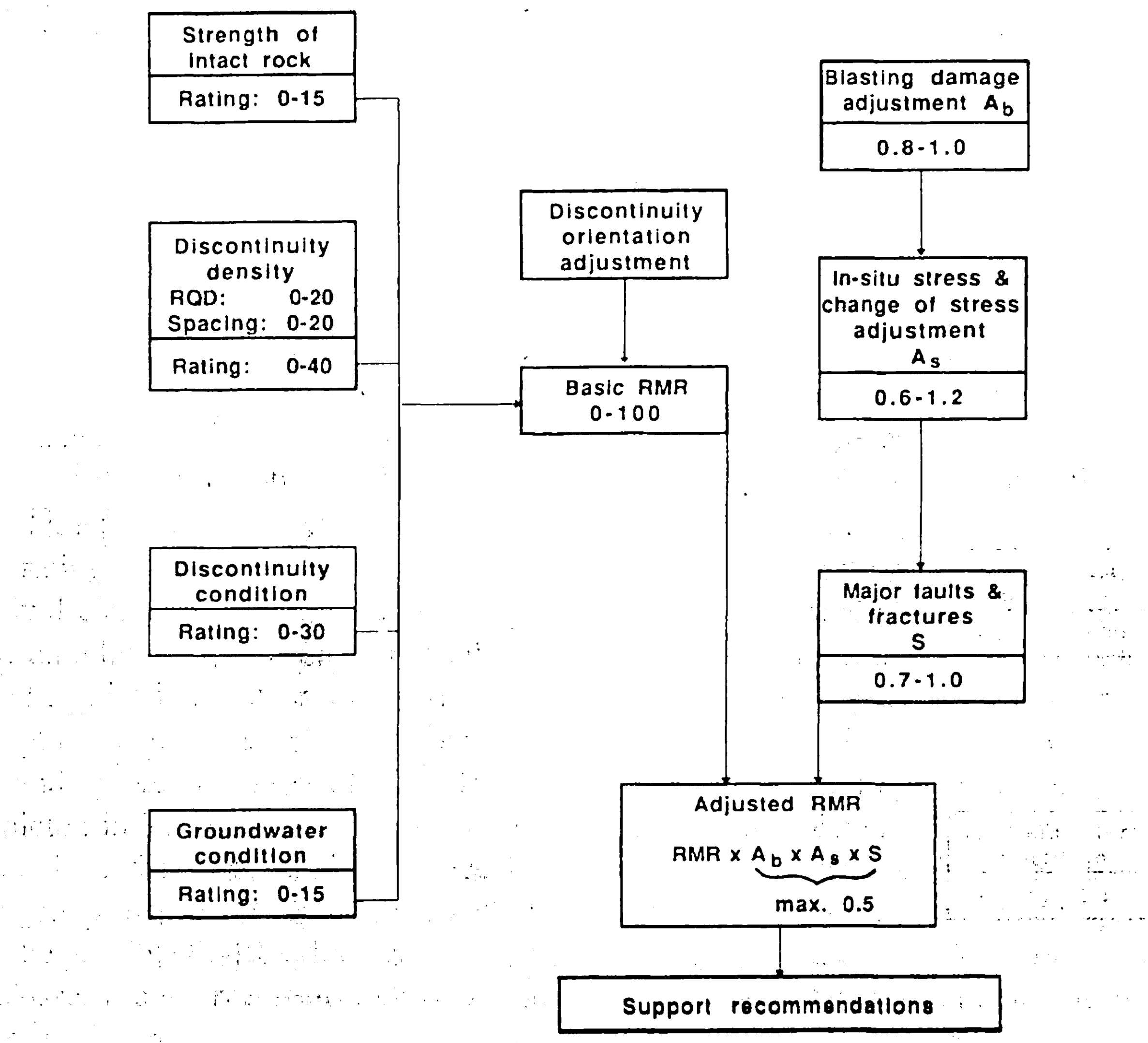
In the case of civil engineering projects, an adjustment for discontinuity orientations will generally suffice. For mining applications, other adjustments may be called for, such as the stress at depth or a change in stress; these have been discussed by Laubscher (1977) and by Kendorski et al. (1983). The procedure for these adjustments is depicted in Table 4.3.

After the adjustment for discontinuity orientations, the rock mass is classified according to Section C of Table 4.1, which groups the final (adjusted) RMR into five rock mass classes, the full range of the possible RMR values varying from zero to 100. Note that the rock mass classes are in groups of 20 ratings each. This concept of rating a rock mass out of a maximum value of 100 has a distinct advantage over an open-ended system in that it allows the engineer or geologist to get the sense of a relative quality, or the lack of it, of a given rock mass in terms of its maximum potential.

Next, Section D of Table 4.1 gives the practical meaning of each rock mass class by relating it to specific engineering problems. In the case of tunnels, chambers, and mines, the output from the Geomechanics Classification is the stand-up time and the maximum stable rock span for a given RMR, as shown in Figure 4.1.

When mixed-quality rock conditions are encountered at the excavated rock face, such as good-quality and poor-quality rock being present in one exposed area, it is essential to identify the "most critical condition" for the assessment of the rock strata. This means that the geological features most significant for stability purposes will have an overriding influence. For example, a fault or shear in high-quality rock face will play a dominant role irrespective of the high rock material strength in the surrounding strata.

TABLE 4.3 Adjustments to the Rock Mass Rating System for Mining Applications



It is recommended that when there are two or more clearly different zones in one rock face, one approach to adopt is to obtain RMR values for each zone and then compute the overall weighted value by the surface area corresponding to each zone in relation to the whole area, as well as by the influence that each zone has on the stability of the whole excavation.

The Geomechanics Classification provides guidelines for the selection of rock reinforcement for tunnels in accordance with Table 4.4. These guidelines depend on such factors as the depth below surface (in-situ stress), tunnel size and shape, and the method of excavation. Note that the support measures given in Table 4.4 represent the permanent and not the primary support. Table 4.4 is applicable to rock masses excavated using conventional drilling and blasting procedures.

Most recently, Lauffer (1988) presented a revised stand-up time diagram specifically for tunnel boring machine (TBM) excavation and superimposed it on the RMR diagram given in Figure 4.1. This is depicted in Figure 4.2, which is most useful because it demonstrates how the boundaries of RMR classes are shifted for TBM applications. Thus, an RMR adjustment can be made for machine-excavated rock masses.

Support load can be determined from the RMR system as proposed by Unal (1983):

$$P = \frac{100 - \text{RMR}}{100} \gamma B \tag{4.1}$$

where P = the support load, kN;
B = the tunnel width, m;
γ = the rock density, kg/m^3.

It must be emphasized that for all applications such as those involving the selection of rock reinforcement and determination of rock loads or rock mass strength and deformability, it is the actual RMR value that must be used and not the rock mass class, within which this RMR value falls. In

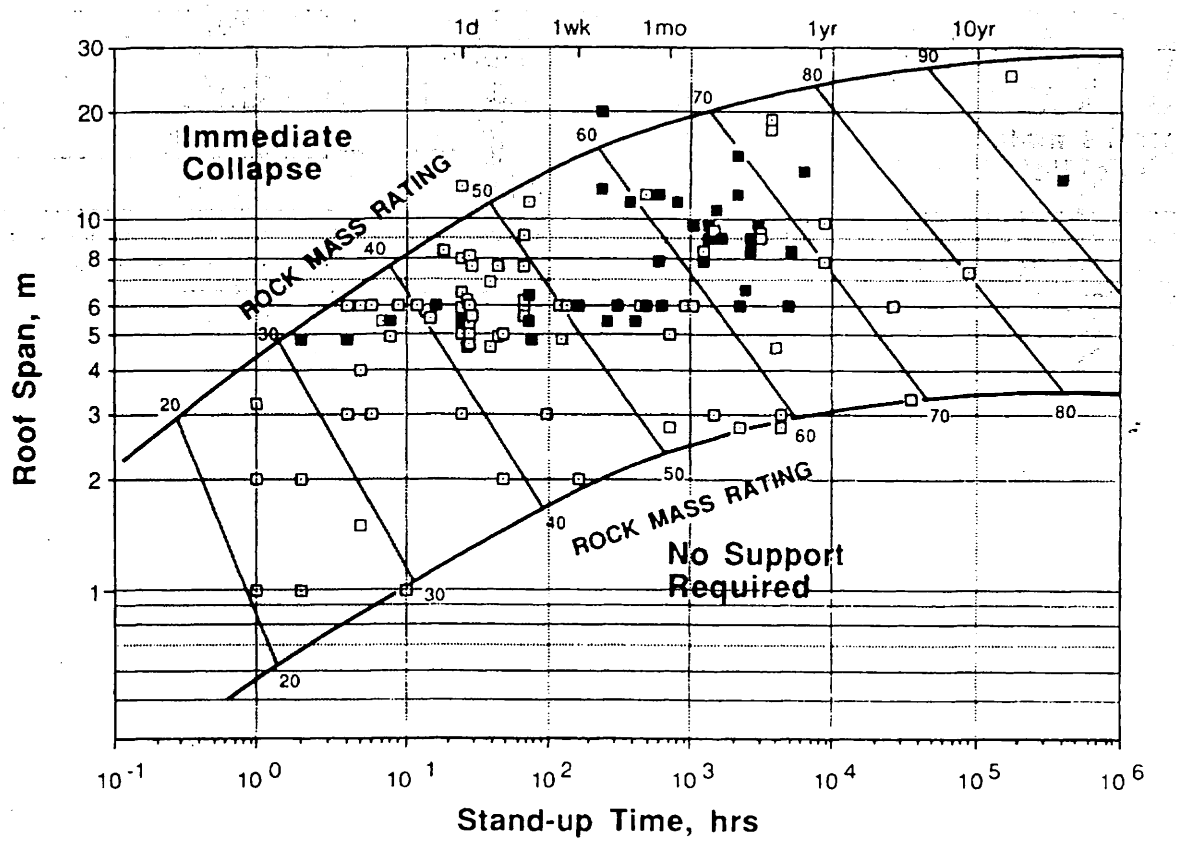


Figure 4.1 Relationship between the stand-up time and span for various rock mass classes, according to the Geomechanics Classification: output for tunneling and mining. The plotted data points represent roof falls studied: filled squares for mines, open squares for tunnels. The contour lines are limits of applicability.

TABLE 4.4 Guidelines for Excavation and Support of Rock Tunnels in Accordance with the Rock Mass Rating System[a]

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

[a] Shape: horseshoe; width: 10 m; vertical stress: <25 MPa; construction: drilling and blasting.

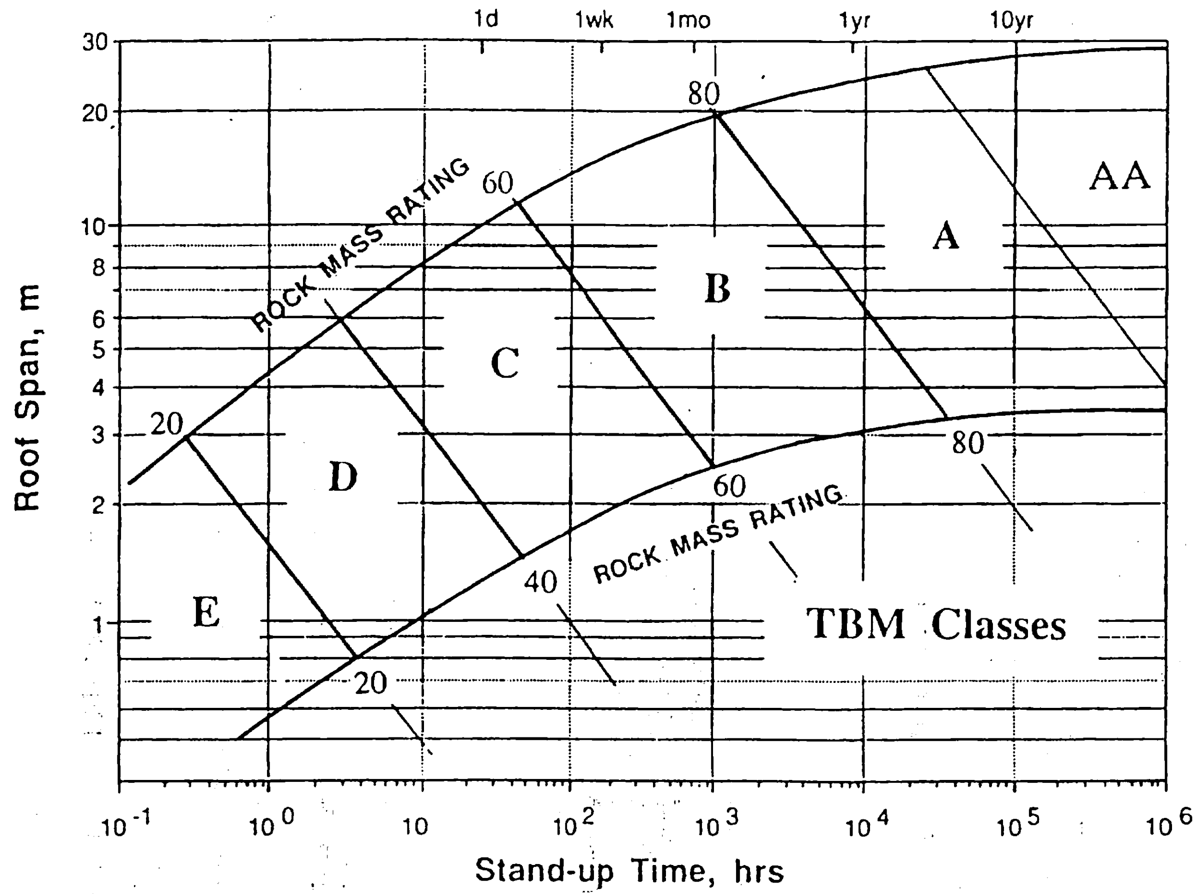


Figure 4.2 *Modified 1988 Lauffer diagram depicting boundaries of rock mass classes for TBM applications. (After Lauffer, 1988.)*

this way, the RMR system is very sensitive to individual parameters, because within one rock mass class, such as "good rock," there is much difference between RMR = 80 and RMR = 61.

Finally, note that the ranges in Table 4.1 follow the recommendations of the International Society of Rock Mechanics (ISRM) Commissions on Standardization and on Classification. The interested reader is referred to a document entitled *Suggested Methods for Quantitative Description of Discontinuities in Rock Masses* (ISRM, 1982).

4.2 APPLICATIONS

The Geomechanics Classification has found wide applications in various types of engineering projects, such as tunnels, slopes, foundations, and mines. Most of the applications have been in the field of tunneling (Bieniawski, 1984).

This classification system has been also used widely in mining, particularly in the United States, India, and Australia. Initially, Laubscher (1977) applied the Geomechanics Classification to asbestos mines in Africa. Most recently, the RMR system was applied to coal mining as well as to hard rock mining (Ghose and Raju, 1981; Abad et al., 1983; Unal, 1983; Kendorski et al., 1983; Newman, 1986; Venkateswarlu, 1986).

The Geomechanics Classification is also applicable to slopes (Romana, 1985) and to rock foundations (Bieniawski and Orr, 1976). This is a useful feature that can assist with the design of slopes near the tunnel portals as well as allow estimates of the deformability of rock foundations for such structures as bridges and dams.

In the case of rock foundations, knowledge of the modulus of deformability of rock masses is of prime importance. The Geomechanics Classification proved a useful method for estimating in-situ deformability of rock masses (Bieniawski, 1978). As shown in Figure 4.3, the following correlation was obtained:

$$E_M = 2\,\text{RMR} - 100 \tag{4.2}$$

where E_M is the in-situ modulus of deformation in GPa and RMR is >50.

More recently, Serafim and Pereira (1983) provided many results in the range RMR < 50 and proposed a new correlation:

$$E_M = 10^{(\text{RMR}-10)/40} \tag{4.3}$$

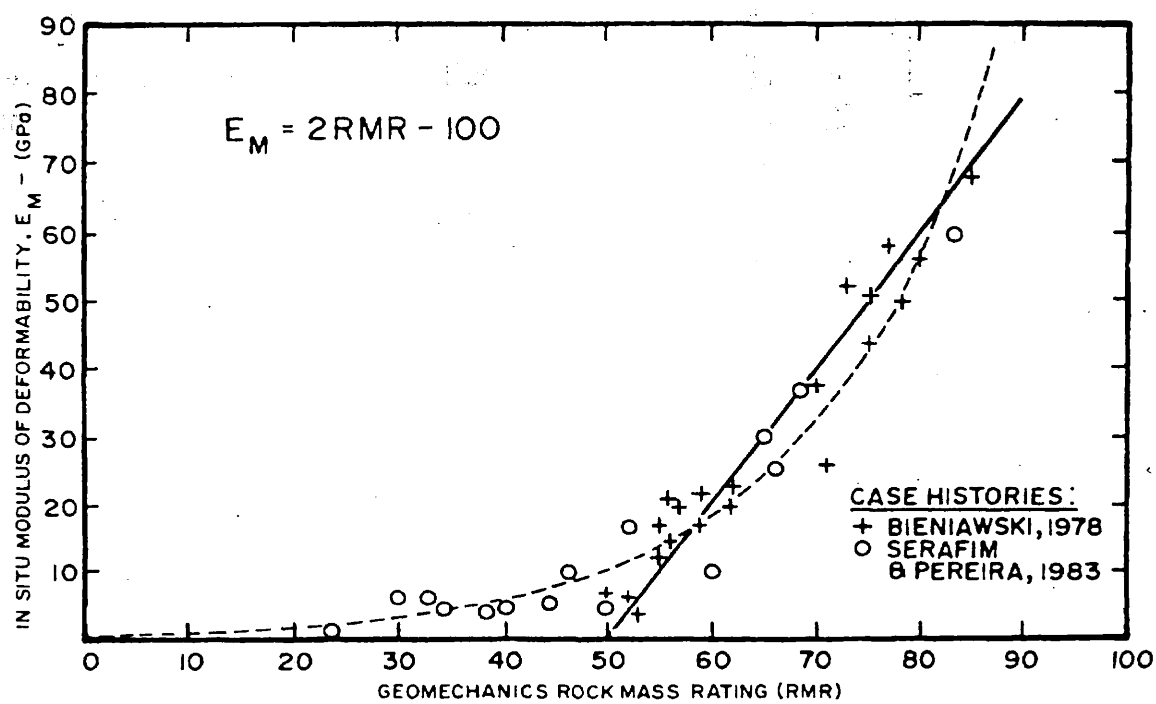


Figure 4.3 *Correlation between the in-situ modulus of deformation and RMR.*

In the case of slopes, the output is given in Section D of Table 4.1 as the cohesion and friction of the rock mass. Romana (1985) has applied the RMR system extensively for determination of rock slope stability.

Recently, Hoek and Brown (1980) proposed a method for estimating rock mass strength which makes use of the RMR classification. The criterion for rock mass strength is as follows:

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \sqrt{m\frac{\sigma_3}{\sigma_c} + s} \tag{4.4}$$

where σ_1 = the major principal stress at failure,
σ_3 = the applied minor principal stress,
σ_c = the uniaxial compressive strength of the rock material,
m and s = constants dependent on the properties of the rock and the extent to which it has been fractured by being subjected to σ_1 and σ_3.

For intact rock, $m = m_i$, which is determined from a fit of the above equation to triaxial test data from laboratory specimens, taking $s = 1$ for rock material. For rock masses, the constants m and s are related to the basic (unadjusted) RMR as follows (Hoek and Brown, 1988):

For Undisturbed Rock Masses (smooth-blasted or machine-bored excavations):

$$m = m_i \exp\left(\frac{\text{RMR} - 100}{28}\right) \tag{4.5}$$

$$s = \exp\left(\frac{\text{RMR} - 100}{9}\right) \tag{4.6}$$

For Disturbed Rock Masses (slopes or blast-damaged excavations):

$$m = m_i \exp\left(\frac{\text{RMR} - 100}{14}\right) \tag{4.7}$$

$$s = \exp\left(\frac{\text{RMR} - 100}{6}\right) \tag{4.8}$$

Moreno Tallon (1982) developed a series of correlations between tunnel deformation, RMR, and time, based on a case history in Spain. Unal (1983) proposed an "integrated approach" to roof stability assessment in coal mines by incorporating RMR with roof span, support pressure, time, and deformation. This is diagrammatically depicted in Figure 4.4. Finally, recent research by Nicholson and Bieniawski (1986), incorporating the RMR system, proposed an empirical constitutive relationship for rock masses.

4.2.1 Strengths and Limitations

The RMR system is very simple to use, and the classification parameters are easily obtained from either borehole data or underground mapping (Gonzalez de Vallejo, 1983; Cameron-Clark and Budavari, 1981; Nakao et al., 1983).

This classification method is applicable and adaptable to many different situations, including coal mining, hard rock mining, slope stability, foundation stability, and tunneling.

The RMR system is capable of being incorporated into theoretical concepts, as is evident in the work of Unal (1983), Moreno Tallon (1982), Hoek and Brown (1980), and Nicholson and Bieniawski (1986).

The Geomechanics Classification is adaptable for use in knowledge-based expert systems. With the introduction of fuzzy-set methodology applied to the Geomechanics Classification by Nguyen and Ashworth (1985) and by Fairhurst and Lin (1985), the subjectiveness, or fuzziness, inherent in a classification can be considered and incorporated into the expert system.

The output from the RMR classification method tends to be rather conservative, which can lead to overdesign of support systems. This aspect is best overcome by monitoring rock behavior during tunnel construction and adjusting rock classification predictions to local conditions. An example of this approach is the work of Kaiser et al. (1986), who found that the no-support limit given in Figure 4.1 was too conservative and proposed the following correction to adjust RMR (No Support) at the no-support limit for opening size effects:

$$\text{RMR (NS)} = 22 \ln \text{ED} + 25 \tag{4.9}$$

where ED is the equivalent dimension as defined by equation (5.2).

For the convenience of the user, a microcomputer program is listed in the Appendix for determination of the RMR and the resulting rock mass properties. An example of the output is included.

4.3 DATA BASE

The data base used for the development of a rock mass classification may indicate the range of its applicability. For example, the RMR system originally

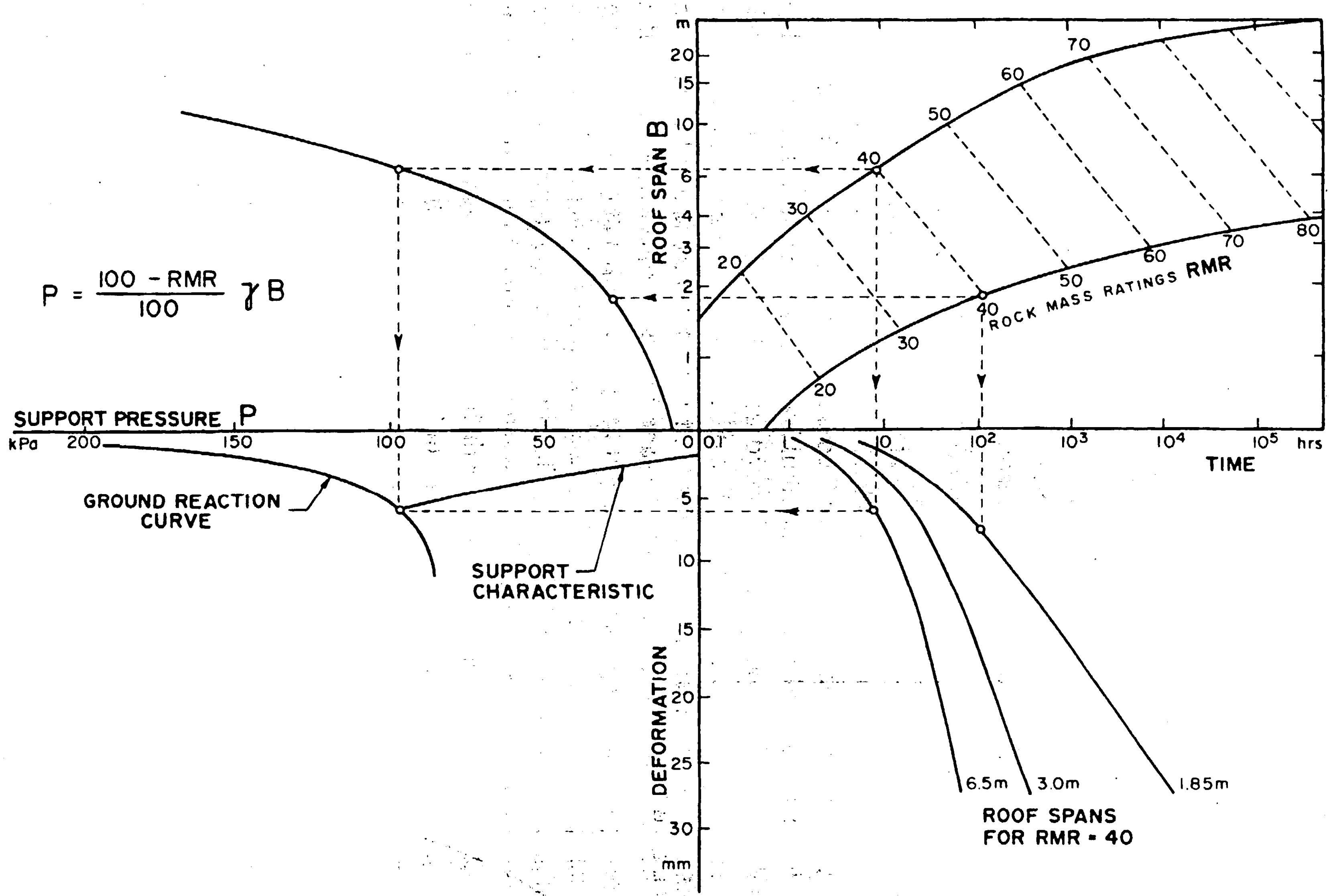


Figure 4.4 *Integration of RMR with support characteristics and roof deformation in coal mines. (After Unal, 1983.)*

involved 49 case histories (which were reanalyzed by Unal, 1983), followed by 62 case histories added by Newman and Bieniawski (1986) and a further 78 tunneling and mining case histories collected between 1984 and 1987. To date, according to the author's files, the RMR system has been used in 351 case histories (see Chap. 10). It was found that the system could be successfully used in rock formations not featured in the original case histories (Fowell and Johnson, 1982; Sandbak, 1985; Smith, 1986; Singh et al., 1986). At the same time, in some cases the system did not provide realistic results (Kaiser et al., 1986).

Nakao et al. (1983) made a significant contribution by performing a statistical reconsideration of the parameters for the Geomechanics Classification in order to apply the RMR system to Japanese geological conditions. In total, 152 tunnel cases were studied. It was found that the results of the parameter rating analysis "virtually agreed with that of the RMR concept."

Finally, the RMR classification—as any other—is not to be taken as a substitute for engineering design. This classification is only a part of the empirical design approach, one of the three main design approaches in rock engineering (empirical, observational, and analytical). It should be applied intelligently and used in conjunction with observational and analytical methods to formulate an overall design rationale compatible with the design objectives and site geology.

4.4 CORRELATIONS

A correlation was proposed between the RMR and the Q-index (Bieniawski, 1976) as well as between the RMR and the RSR (Rutledge and Preston, 1978). Based on 111 case histories analyzed for this purpose (involving 62 Scandinavian cases, 28 South African cases, and 21 case histories from the United States, Canada, Australia, and Europe), the following relationship was found for civil engineering tunnels (Bieniawski, 1976):

$$\mathrm{RMR} = 9 \ln Q + 44 \tag{4.10}$$

For mining tunnels, Abad et al. (1983) analyzed 187 coal mine roadways in Spain, arriving at this correlation:

$$\mathrm{RMR} = 10.5 \ln Q + 42 \tag{4.11}$$

Rutledge and Preston (1978) determined the following correlation from seven tunneling projects in New Zealand:

$$RSR = 0.77\ RMR + 12.4 \quad (4.12)$$

Moreno Tallon (1982) confirmed the above relationships on the basis of four tunneling projects in Spain. Jethwa et al. (1982) further substantiated the correlation by Bieniawski (1976) on the basis of 12 projects in India, whereas Trunk and Hönisch (1989) found an almost identical correlation to that given in equation (4.10) based on their study of tunnels in West Germany. For further discussion of these and other correlations, see Section 5.2.

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5

Q-System

Few things are created and perfected at the same time.
—Thomas Edison

The Q-system of rock mass classification was developed in Norway in 1974 by Barton, Lien, and Lunde, all of the Norwegian Geotechnical Institute. Its development represented a major contribution to the subject of rock mass classification for a number of reasons: the system was proposed on the basis of an analysis of 212 tunnel case histories from Scandinavia, it is a quantitative classification system, and it is an engineering system facilitating the design of tunnel supports.

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

1. RQD.
2. Number of joint sets.
3. Roughness of the most unfavorable joint or discontinuity.
4. Degree of alteration or filling along the weakest joint.
5. Water inflow.
6. Stress condition.

These six parameters are grouped into three quotients to give the overall rock mass quality Q as follows:

$$Q = \frac{\text{RQD}}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{\text{SRF}} \tag{5.1}$$

where RQD = rock quality designation,
J_n = joint set number,
J_r = joint roughness number,
J_a = joint alteration number,
J_w = joint water reduction number,
SRF = stress reduction factor.

The rock quality can range from $Q = 0.001$ to $Q = 1000$ on a logarithmic rock mass quality scale.

5.1 CLASSIFICATION PROCEDURES

Table 5.1 gives the numerical values of each of the classification parameters. They are interpreted as follows: The first two parameters represent the overall structure of the rock mass, and their quotient is a relative measure of the block size. The quotient of the third and the fourth parameters is said to be an indicator of the interblock shear strength (of the joints). The fifth parameter is a measure of water pressure, while the sixth parameter is a measure of a) loosening load in the case of shear zones and clay bearing rock, b) rock stress in competent rock, and c) squeezing and swelling loads in plastic incompetent rock. This sixth parameter is regarded as the "total stress" parameter. The quotient of the fifth and the sixth parameters describes the "active stress."

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

The Q value is related to tunnel support requirements by defining the equivalent dimensions of the excavation. This equivalent dimension, which is a function of both the size and the purpose of the excavation, is obtained by dividing the span, diameter, or the wall height of the excavation by a quantity called the excavation support ratio (ESR). Thus

$$\text{Equivalent dimension} = \frac{\text{span or height (m)}}{\text{ESR}} \tag{5.2}$$

The ESR is related to the use for which the excavation is intended and the degree of safety demanded, as shown below:

Excavation Category	ESR	No. of Cases
A. Temporary mine openings	3–5	2
B. Vertical shafts:		
Circular section	2.5	
Rectangular/square section	2.0	
C. Permanent mine openings, water tunnels for hydropower (excluding high-pressure penstocks), pilot tunnels, drifts, and headings for large excavations	1.6	83
D. Storage caverns, water treatment plants, minor highway and railroad tunnels, surge chambers, access tunnels	1.3	25
E. Power stations, major highway or railroad tunnels, civil defense chambers, portals, intersections	1.0	73
F. Underground nuclear power stations, railroad stations, factories	0.8	2

The relationship between the index Q and the equivalent dimension of an excavation determines the appropriate support measures, as depicted in Figure 5.1. Barton et al. (1974) provided the corresponding 38 support categories specifying the estimates of permanent support, as given in Tables 5.2–5.6. For temporary support determination, either Q is increased to $5Q$ or ESR is increased to 1.5 ESR.

It should be noted that the length of bolts is not specified in the support tables, but the bolt length L is determined from the equation

$$L = \frac{2 + 0.15B}{\text{ESR}} \tag{5.3}$$

where B is the excavation width.

The maximum unsupported span can be obtained as follows:

$$\text{Maximum span (unsupported)} = 2(\text{ESR})\, Q^{0.4} \tag{5.4}$$

The relationship between the Q value and the permanent support pressure P_{roof} is calculated from the following equation:

TABLE 5.1 Q-System Description and Ratings: Parameters RQD, J_n, J_r, J_a, SRF, and J_w[a]

Rock Quality Designation (RQD)

Very poor	0–25	*Note:*
Poor	25–50	(i) Where RQD is reported or measured as ⩽10 (including 0), a nominal value of 10 is used to evaluate Q in equation (5.1).
Fair	50–75	
Good	75–90	
Excellent	90–100	(ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate

Joint Set Number J_n

Massive, none or few joints	0.5–1.0	*Note:*
One joint set	2	(i) For intersections, use ($3.0 \times J_n$)
One joint set plus random	3	
Two joint sets	4	(ii) For portals, use ($2.0 \times J_n$)
Two joint sets plus random	6	
Three joint sets	9	
Three joint sets plus random	12	
Four or more joint sets, random, heavily jointed, "sugar cube," etc.	15	
Crushed rock, earthlike	20	

Joint Roughness Number J_r

(a) Rock wall contact and		*Note:*
(b) Rock wall contact before 10-cm shear		(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
Discontinuous joint	4	
Rough or irregular, undulating	3	

Smooth, undulating	2.0	*Note:*
Slickensided, undulating	1.5	(ii) $J_r = 0.5$ can be used for planar slickensided joints having lineation, provided the lineations are favorably oriented
Rough or irregular, planar	1.5	
Smooth, planar	1.0^b	
Slickensided, planar	0.5	(iii) Descriptions B to G refer to small-scale features and intermediate-scale features, in that order
(c) No rock wall contact when sheared		
Zone containing clay minerals thick enough to prevent rock wall contact	1.0^b	
Sandy, gravelly, or crushed zone thick enough to prevent rock wall contact	1.0^b	

Joint Alteration Number J_a

(a) Rock wall contact	J_a	ϕ_r (approx)
A. Tightly healed, hard, nonsoftening, impermeable filling, i.e., quartz or epidote	0.75	
B. Unaltered joint walls, surface staining only	1.0	25–35°
C. Slightly altered joint walls. Nonsoftening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25–30°
D. Silty or sandy clay coatings, small clay fraction (nonsoftening)	3.0	20–25°
E. Softening or low-friction clay mineral coatings, i.e., kaolinite, mica. Also chlorite, talc, gypsum, and graphite, etc., and small quantities of swelling clays (discontinuous coatings, 1–2 mm or less in thickness)	4.0	8–16°
(b) Rock wall contact before 10-cm shear		
F. Sandy particles, clay-free disintegrated rock, etc.	4.0	25–30°

(*Table continues on p. 78.*)

TABLE 5.1 *(Continued)*

	Joint Alteration Number J_a	
G. Strongly over-consolidated, nonsoftening clay mineral fillings (continuous, <5 mm in thickness)	6.0	16–24°
H. Medium or low over-consolidation, softening, clay mineral fillings. (continuous, <5 mm in thickness)	8.0	12–16°
J. Swelling clay fillings, i.e., montmorillonite (continuous, < mm in thickness). Value of J_a depends on percentage of swelling clay-sized particles, and access to water, etc.	8.0–12.0	6–12°
(c) No rock wall contact when sheared		
K. Zones or bands of disintegrated or crushed rock and clay (see G., H., J. for description of clay condition)	6.0, 8.0 or 8.0–12.0	6–24°
L. Zones or bands of silty or sandy clay, small clay fraction (nonsoftening)	5.0	
M. Thick, continuous zones or bands of clay (see G., H., J. for description of clay condition)	10.0, 13.0 or 13.0–20.0	6–24°

Note:

(i) Values of ϕ_r are intended as an approximate guide to the mineralogical properties of the alteration products, if present

Stress Reduction Factor (SRF)

	σ_c/σ_1	σ_t/σ_1	SRF	Note
(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				*Note:* (i) Reduce these SRF values by 25–50% if the relevant shear zones only influence but do not intersect the excavation
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10.0	
B. Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤50 m)			5.0	
C. Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m)			2.5	
D. Multiple-shear zones in competent rock (clay-free), loose surrounding rock (any depth)			7.5	
E. Single-shear zones in competent rock (clay-free) (depth of excavation ≤50 m)			5.0	
F. Single-shear zones in competent rock (clay-free) (depth of excavation >50 m)			2.5	
G. Loose open joints, heavily jointed or "sugar cube," etc. (any depth)			5.0	
(b) Competent rock, rock stress problems				
H. Low stress, near surface	>200	>13	2.5	(ii) For strongly anisotropic stress field (if measured): when $5 \leqslant \sigma_1/\sigma_3 \leqslant 10$, reduce σ_c and σ_t to 0.8 σ_c and 0.8 σ_t; when $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to
J. Medium stress	200–10	13–0.66	1.0	

(*Table continues on p. 80.*)

TABLE 5.1 (*Continued*)

Stress Reduction Factor (SRF)

K. High-stress, very tight structure (usually favorable to stability, may be unfavorable to wall stability	10–5	0.66–0.33	0.5–2.0	0.6 σ_c and 0.6 σ_t (where σ_c = unconfined compressive strength, σ_t = tensile strength (point load), σ_1 and σ_3 = major and minor principal stresses)
L. Mild rock burst (massive rock)	5–2.5	0.33–0.16	5–10	
M. Heavy rock burst (massive rock)	<2.5	<0.16	10–20	
(c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures				
N. Mild squeezing rock pressure			5–10	
O. Heavy squeezing rock pressure			10–20	(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H)
(d) Swelling rock; chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5–10	
R. Heavy swelling rock pressure			10–15	

Joint Water Reduction Factor J_w

	J_w	Approximate water pressure (kg/cm^2)
A. Dry excavations or minor inflow, i.e.,		
B. 5 L/min locally	1.0	<1
Medium inflow or pressure occasional outwash of joint fillings	0.66	1.0–2.5
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5–10.0
D. Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5–10.0
E. Exceptionally high inflow or water pressure at blasting, decaying with time	0.2–0.1	>10.0
F. Exceptionally high inflow or water pressure continuing without noticeable decay	0.1–0.05	>10.0

Note:

(i) Factors C–F are crude estimates. Increase J_w if drainage measures are installed

(ii) Special problems caused by ice formation are not considered

[a] After Barton et al. (1974).

[b] Nominal.

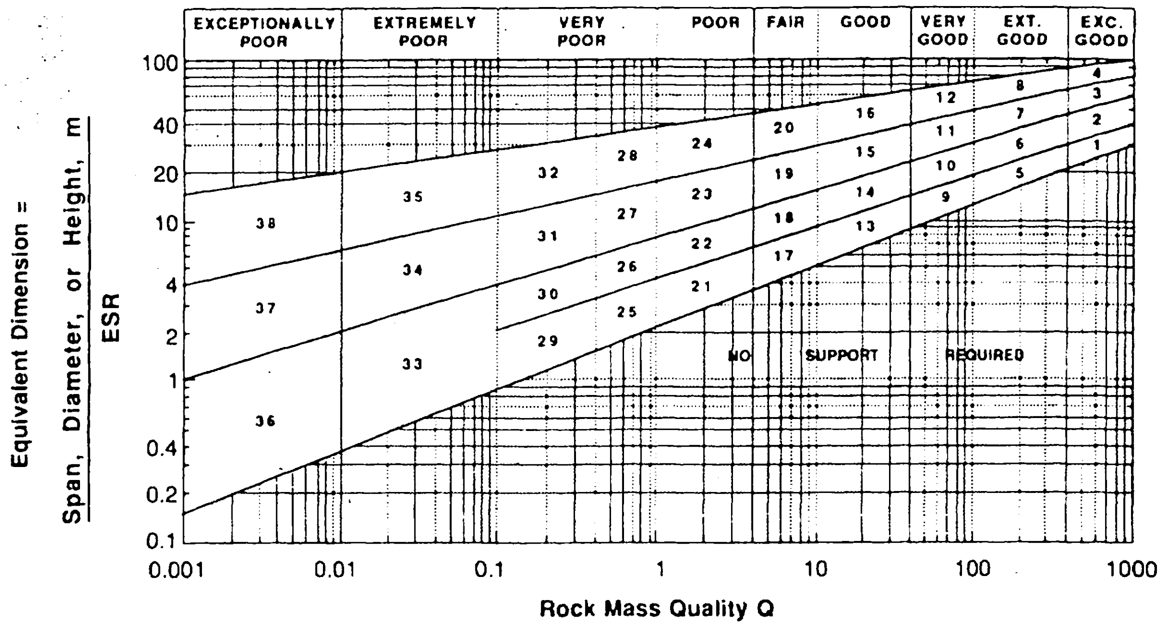

Figure 5.1 Q-system: equivalent dimension versus rock mass quality. (After Barton et al., 1974.)

$$P_{\text{roof}} = \frac{2.0}{J_r} Q^{-1/3} \tag{5.5}$$

If the number of joint sets is less than three, the equation is expressed as:

$$P_{\text{roof}} = \frac{2}{3} J_n^{1/2} J_r^{-1} Q^{-1/3} \tag{5.6}$$

Although the Q-system involves 9 rock mass classes and 38 support categories, it is not necessarily too complicated. Some users of the Q-system have pointed out that the open logarithmic scale of Q varying from 0.001 to 1000 can be a source of difficulty; it is easier to get a feeling for a quoted rock mass quality using a linear scale of up to 100. The numerical procedure may also give some users a misplaced sense of numerical precision—for example, when reporting Q values such as "11.53."

5.2 CORRELATIONS

As stated in Section 4.4, a correlation was developed between the Q-index and the RMR (Bieniawski, 1976) as well as between the Q-index and the RSR (Rutledge and Preston, 1978). A total of 111 case histories were analyzed for this purpose: 62 Scandinavian cases, 28 South African cases,

TABLE 5.2 Q-System: Support Measures for Q Range 10 to 1000[a]

Support Category	Q	Conditional Factors: RQD/J_n	Conditional Factors: J_r/J_n	Span/ESR (m)	P^b (kg/cm^2)	Span/ESR (m)	Type of Support	Notes (Table 5.6)
1[c]	1000–400				<0.01	20–40	sb (utg)	
2[c]	1000–400				<0.01	30–60	sb (utg)	
3[c]	1000–400				<0.01	46–80	sb (utg)	
4[c]	1000–400				<0.01	65–100	sb (utg)	
5[c]	400–100				0.05	12–30	sb (utg)	
6[c]	400–100				0.05	19–45	sb (utg)	
7[c]	400–100				0.05	30–65	sb (utg)	
8[c]	400–100				0.05	48–88	sb (utg)	
9	100–40	⩾20			0.25	8.5–19	sb (utg)	
		<20					B (utg) 2.5–3 m	
10	100–40	⩾30			0.25	14–30	B (utg) 2–3 m	
		<30					B (utg) 1.5–2 m + clm	
11[c]	100–40	⩾30			0.25	23–48	B (tg) 2–3m	
		<30					B (tg) 1.5–2 m + clm	
12[c]	100–40	⩾30			0.25	40–72	B (tg) 2–3m	
		<30					B (tg) 1.5–2 m + clm	

(*Table continues on p. 84.*)

TABLE 5.2 *(Continued)*

Support Category	Q	Conditional Factors		Span/ESR (m)	P[b] (kg/cm^2)	Span/ESR (m)	Type of Support	Notes (Table 5.6)
		RQD/J_n	J_r/J_n					
13	40–10	≥10	≥1.5		0.5	5–14	sb (utg)	I
		≥10	<1.5				B (utg) 1.5–2 m	I
		<10	≥1.5				B (utg) 1.5–2 m	I
		<10	<1.5				B (utg) 1.5–2 m + S 2–3 cm	I
14	40–10	≥10		≥15	0.5	9–23	B (tg) 1.5–2 m + clm	I, II
		<10		≥15			B (tg) 1.5–2 m + S (mr) 5–10 cm	I, II
				<15			B (utg) 1.5–2 m + clm	I, III
15	40–10	>10			0.5	15–40	B (tg) 1.5–2 m + clm	I, II, IV
		≤10					B (tg) 1.5–2 m + S (mr) 5–10 cm	I, II, IV
16[c,d]	40–10	>15			0.5	30–65	B (tg) 1.5–2 m + clm	I, V, VI
		≤15					B (tg) 1.5–2 m + S (mr) 10–15 cm	I, V, VI

[a] After Barton et al. (1974).
[b] Approx.
[c] Original authors' estimates of support. Insufficient case records available for reliable estimation of support requirements. The type of support to be used in categories 1–8 will depend on the blasting technique. Smooth-wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is >25 m. Future case records should differentiate categories 1–8. Key: sb = spot bolting; B = systematic bolting; (utg) = untensioned, grouted; (tg) tensioned (expanding-shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; S = shotcrete; (mr) = mesh-reinforced; clm = chain-link mesh; CCA = cast concrete arch; (sr) steel-reinforced. Bolt spacings are given in meters (m). Shotcrete or cast concrete arch thickness is given in centimeters (cm).
[d] See note XII in Table 5.6.

TABLE 5.3 Q-System: Support Measures for Q Range 1 to 10[a]

Support Category	Q	Conditional Factors		Span/ESR (m)	P^b (kg/cm^2)	Span/ESR (m)	Type of Support	Notes (Table 5.6)
		RQD/J_n	J_r/J_a					
17	10–4	>30			1.0	3.5–9	sb (utg)	I
		⩾10, ⩽30					B (utg) 1–1.5 m	I
		<10		⩾6			B (utg) 1–1.5 m + S 2–3 cm	I
		<10		<6			S 2–3 cm	I
18	10–4	>5		⩾10	1.0	7–15	B (tg) 1–1.5 m + clm	I, III
		>5		<10			B (utg) 1–1.5 m + clm	I
		⩽5		⩾10			B (tg) 1–1.5 m + S 2–3 cm	I, III
		⩽5		<10			B (utg) 1–1.5 m + S 2–3 cm	I
19	10–4			⩾20	1.0	12–29	B (tg) 1–2 m + S (mr) 10–15 cm	I, II, IV
				<20			B (tg) 1–1.5 m + S (mr) 5–10 cm	I, II
20[c]	10–4			⩾35	1.0	24–52	B (tg) 1–2 m + S (mr) 20–25 cm	I, V, VI
				<35			B (tg) 1–2 m + S (mr) 10–20 cm	I, II, IV
21	4–1	⩾12.5	⩽0.75		1.5	2.1–6.5	B (utg) 1m + S 2–3 cm	I
		<12.5	<0.75				S 2.5–5 cm	I
			>0.75				B (utg) 1m	I
22	4–1	>10, <30	>1.0		1.5	4.5–11.5	B (utg) 1m + clm	I
		⩽10	>1.0				S 2.5–7.5 cm	I
		<30	⩽1.0				B (utg) 1 m + S (mr) 2.5–5 cm	I
		⩾30					B (utg) 1 m	I
23	4–1			⩾15	1.5	8–24	B (tg) 1–1.5 m + S (mr) 10–15 cm	I, II, IV, VII
				<15			B (utg) 1–1.5 m + S (mr) 5–10 m	I
24[c,d]	4–1			⩾30	1.5	18–46	B (tg) 1–1.5 m + S (mr) 15–30 cm	I, V, VI
				<30			B (tg) 1–1.5 m + S (mr) 10–15 cm	I, II, IV

[a] After Barton et al. (1974).
[b] Approx.
[c] See note XII in Table 5.6.
[d] See footnote *c* in Table 5.2.

TABLE 5.4 Q-System: Support Measures for Q Range 0.1 to 1.0[a]

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_a	Span/ESR (m)	P^b (kg/cm^2)	Span/ESR (m)	Type of Support[c]	Notes (Table 5.6)
25	1.0–0.4	>10	>0.5		2.25	1.5–4.2	B (utg) 1 m + mr or clm	I
		≤10	>0.5				B (utg) 1 m + S (mr) 5 cm	I
			≤0.5				B (tg) 1 m + S (Mr) 5 cm	I
26	1.0–0.4				2.25	3.2–7.5	B (tg) 1 m + S (mr) 5–7.5 cm	VIII, X, XI
							B (utg) 1 m + S 2.5–5 cm	I, IX
27	1.0–0.4			≥12	2.25	6–18	B (tg) 1 m + S (mr) 7.5–10 cm	I, IX
				<12			B (utg) 1 m + S (mr) 5–7.5 cm	I, IX
				>12			CCA 20–40 cm + B (tg) 1 m	VIII, X, XI
				<12			S (mr) 10–20 cm + B (tg) 1 m	VIII, X, XI
28[d]	1.0–0.4			≥30	2.25	15–38	B (tg) 1 m + S (mr) 30–40 cm	I, IV, V, IX
				≥20, <30			B (tg) 1 m + S (mr) 20–30 cm	I, II, IV, IX
				<20			B (tg) 1m + S (mr) 15–20 cm	I, II, IX
							CCA (sr) 30–100 cm + B (tg) 1 m	IV, VIII, X, XI
29	0.4–0.1	>5	>0.25		3.0	1.0–3.1	B (utg) 1 m + S 2–3 cm	
		≤5	>0.25				B (utg) 1 m + S (mr) 5 cm	
			≤0.25				B (tg) 1 m + S (Mr) 5 cm	
30	0.4–0.1	≥5			3.0	2.2–6	B (tg) 1 m + S 2.5–5 cm	IX
		<5					S (mr) 5–7.5 cm	IX
							B (tg) 1 m + S (mr) 5–7.5 cm	VIII, X, XI
31	0.4–0.1	>4			3.0	4–14.5	B (tg) 1 m + S (mr) 5–12.5 cm	IX
		≤4, ≥1.5					S (mr) 7.5–25 cm	IX
		<1.5					CCA 20–40 cm + B (tg) 1 m	IX, XI
							CCA (sr) 30–50 cm + B (tg) 1 m	VIII, X, XI
32[d]	0.4–0.1			≥20	3.0	11–34	B (tg) 1 m + S (mr) 40–60 cm	II, IV, IX, XI
				<20			B (tg) 1 m + S (mr) 20–40 cm	III, IV, IX, XI

[a] After Barton et al. (1974).
[b] Approx.
[c] For key, refer to Table 5.2, footnote *c*.
[d] See note XII in Table 5.6.

TABLE 5.5 Q-System: Support Measures for Q Range 0.001 to 0.1[a]

Support Category	Q	Conditional Factors: RQD/J_n	Conditional Factors: J_r/J_a	Span/ESR (m)	P^b (kg/cm^2)	Span/ESR (m)	Type of Support[c]	Notes (Table 5.6)
33	0.1–0.01	≥2			6	1.0–3.9	B (tg) 1 m + S (mr) 2.5–5 cm	IX
		<2					S (mr) 5–10 cm	IX
							S (mr) 7.5–15 cm	VIII, X
34	0.1–0.01	≥2	≥0.25		6	2.0–11	B (tg) 1 m + S (mr) 5–7.5 cm	IX
		<2	≥0.25				S (mr) 7.5–15 cm	IX
			<0.25				S (mr) 15–25 cm	IX
							CCA (sr) 20–60 cm + B (tg) 1 m	VIII, X, XI
35[d]	0.1–0.01			≥15	6	6.2–28	B (tg) 1 m + S (mr) 30–100 cm	II, IX, XI
				≥15			CCA (sr) 60–200 cm + B (tg) 1 m	VIII, X, XI, II
				<15			B (tg) 1 m + S (mr) 20–75 cm	IX, XI, III
				<15			CCA (sr) 40–150 cm + B (tg) 1 m	VIII, X, XI, III
36	0.01–0.001				12	1.0–2.0	S (mr) 10–20 cm	IX
							S (mr) 10–20 cm + B (tg) 0.5–1.0 m	VIII, X, XI
37	0.01–0.001				12	1.0–6.5	S (mr) 20–60 cm	IX
							S (mr) 20–60 cm + B (tg) 0.5–1.0 m	VIII, X, XI
38[e]	0.01–0.001			≥10	12	4.0–20	CCA (sr) 100–300 cm	IX
				≥10			CCA (sr) 100–300 cm + B (tg) 1 m	VIII, X, II, XI
				<10			S (mr) 70–200 cm	IX
				<10			S (mr) 70–200 cm	VIII, X, III, XI

[a] After Barton et al. (1974).
[b] Approx.
[c] For key, refer to Table 5.2, footnote *c*.
[d] See note XII in Table 5.6.
[e] See note XIII in Table 5.6.

TABLE 5.6 Q-System: Support Measures—Supplementary Notes[a]

I.	For cases of heavy rock bursting or "popping," tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases.
II.	Several bolt lengths often used in same excavation, i.e., 3, 5, and 7 m.
III.	Several bolt lengths often used in same excavation, i.e., 2, 3, and 4 m.
IV.	Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2–4 m.
V.	Several bolt lengths often used in same excavation, i.e., 6, 8, and 10 m.
VI.	Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4–6 m.
VII.	Several of the older-generation power stations in this category employ systematic or spot bolting with areas of chain-link mesh, and a free-span concrete arch roof (25–40 cm) as permanent support.
VIII.	Cases involving swelling, e.g., montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.
IX.	Cases not involving swelling clay or squeezing rock.
X.	Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.
XI.	According to the authors' [Barton et al.] experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e., >1.5), possibly *combined with shotcrete. If* the rock mass is very heavily jointed or crushed (i.e., $RQD/J_n < 1.5$, for example, a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete (or shotcrete) arch to reduce the uneven loading on the concrete, but it may not be effective when $RQD/J_n < 1.5$, or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick-setting resin anchors in these extremely poor-quality rock masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.
XII.	For reasons of safety, the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 (span/ESR > 15 m only).
XIII.	Multiple drift method usually needed during excavation and support of arch, walls, and floor in cases of heavy squeezing. Category 38 (span/ESR > 10 m only).

[a] After Barton et al. (1974).

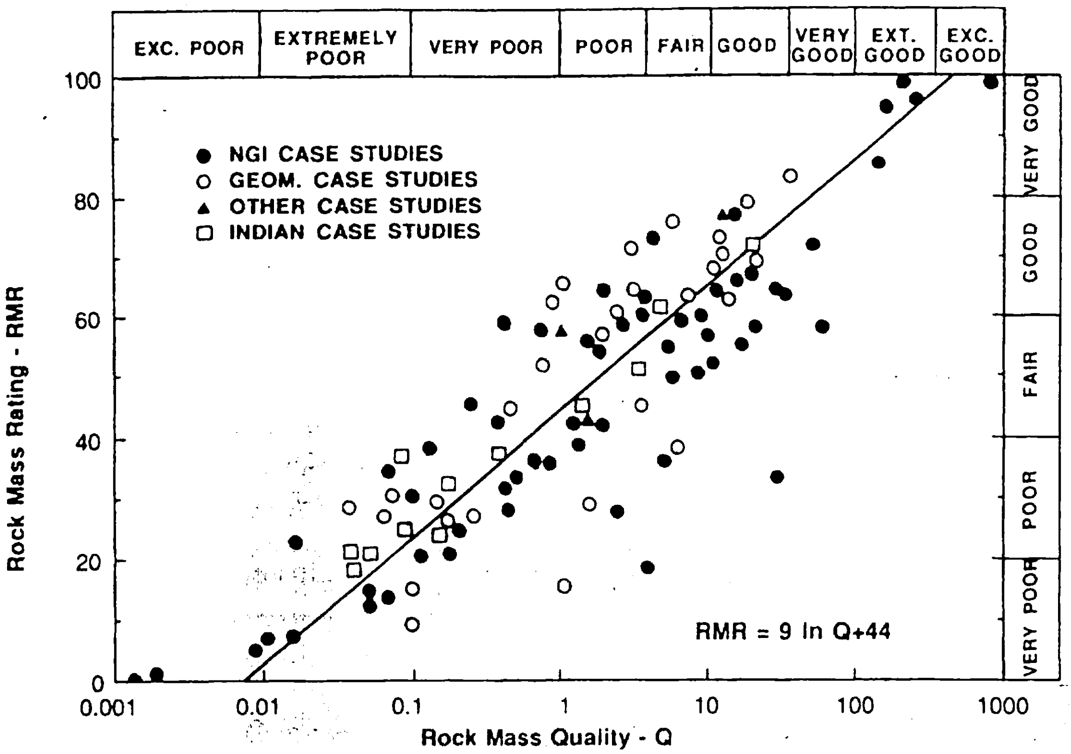

Figure 5.2 *Correlation between the RMR and the Q-index. (After Bieniawski, 1976 and Jethwa et al., 1982.)*

and 21 other case histories from the United States, Canada, Australia, and Europe. The results are plotted in Figure 5.2, from which it can be seen that the following relationship is applicable:

$$\text{RMR} = 9 \ln Q + 44 \tag{5.7}$$

The above correlation was further substantiated by Jethwa et al. (1982), whose case studies are also included in Figure 5.2. Further comparisons between the Q and the RMR systems are given by Barton (1988).

5.3 DATA BASE

Barton (1988) presented histograms of the 212 case records used to develop the Q-system. The majority of the cases are from Scandinavia (Sweden and Norway), including 97 cases reported by Cecil (1970).

The distribution of the rock types was as follows: 13 types of igneous rock, 26 types of metamorphic rock, and 11 types of sedimentary rocks. Hard rock was predominant, involving 48 cases of granite and 21 cases of gneiss.

The Q values covered the whole range of rock mass qualities; there were 40 cases with $Q = 10 - 40$, 45 cases with $Q = 4 - 10$, 36 cases with $Q = 1 - 4$, and 40 cases with $Q = 0.1 - 1.0$.

The predominant tunnel spans or diameters were 5–10 m (78 cases), and 10–15 m (59 cases). There were 40 cases of large caverns from hydroelectric projects, with spans of 15–30 m and wall heights of 30–60 m.

The excavation depths were commonly in the range of 50 to 250 m. However, 20 cases were in the range 250 to 500 m, and 51 cases involved depths less than 50 m.

Most case histories (180) were supported excavations; 32 of the 212 cases were permanently unsupported excavations. The predominant form of support was rock bolts, or combinations of rock bolts and shotcrete often mesh-reinforced.

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6

Other Classifications

Real difficulties can be overcome;
it is only the imaginary ones that are unconquerable.
—Somerset Maugham

Among the various modern rock mass classifications, the approach used by the New Austrian Tunneling Method and the strength–size classification of Franklin and Louis deserve special attention.

6.1 NATM CLASSIFICATION

The New Austrian Tunneling Method (NATM) features a qualitative ground classification system that must be considered within the overall context of the NATM. In essence, the NATM is an approach or philosophy integrating the principles of the behavior of rock masses under load and monitoring the performance of underground excavations during construction. The word "method" in the English translation is unfortunate, as it has led to some misunderstanding. The fact is that the NATM is not a set of specific excavation and support techniques. Many people believe that if shotcrete and rock bolts are used as support, then they are employing the New Austrian Tunneling Method. This is far from the truth. The NATM involves a combination of many established ways of excavation and tunneling, but the difference is the continual monitoring of the rock movement and the revision of support to obtain the most stable and economical lining. However, a number of other aspects are also pertinent in making the NATM more of a concept or philosophy than a method.

The New Austrian Tunneling Method was developed between 1957 and 1965 in Austria. It was given its name in Salzburg in 1962 to distinguish it from the traditional old Austrian tunneling approach. The main contributors to the development of the NATM were Ladislaus von Rabcewicz, Leopold Müller, and Franz Pacher.

Essentially, the NATM is a scientific empirical approach. It has evolved from practical experience and Rabcewicz called it "empirical dimensioning" (Rabcewicz, 1964). However, it has a theoretical basis involving the relationship between the stresses and deformations around tunnels (better known as the ground-reaction curve concept). Its early theoretical foundations were given by two Austrians, Fenner and Kastner. The method makes use of sophisticated in-situ instrumentation and monitoring, and interprets these measurements in a scientific manner.

As stated earlier, this method is often misunderstood, and recently a number of publications attempting to clarify these misconceptions have appeared in the international press; the more notable among them are those by Müller (1978), Golser (1979), Brown (1981), and Sauer (1988).

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

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

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

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

of the support is of vital importance. John (1980) provided a fine example of the use of instrumentation during the construction of the Arlberg Tunnel.

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

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

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

7. *Rock Mass Classification Determines Support Measures.* Payment for support is based on a rock mass classification after each drill and blast round. In some countries this is not acceptable contractually, and this is why the method has received limited attention in the United States. Figure 6.1 is an example of the main ground classes for rock tunnels and the corresponding support; these serve as the guidelines for tunnel reinforcement as well as for payment purposes.

The NATM calls for all parties involved in the design and construction of a tunneling project to accept and understand this approach and to cooperate in decision-making and the resolution of problems. The owner, the design engineer, and the contractor need to work as one team. The project should be staffed with well-trained field engineers (competent to interpret the observations and act on them) and with designers (or consultants) who visit the site frequently and are on call for difficult construction decisions. In Austria, only highly qualified contractors who can demonstrate their expertise in the use of shotcrete are employed.

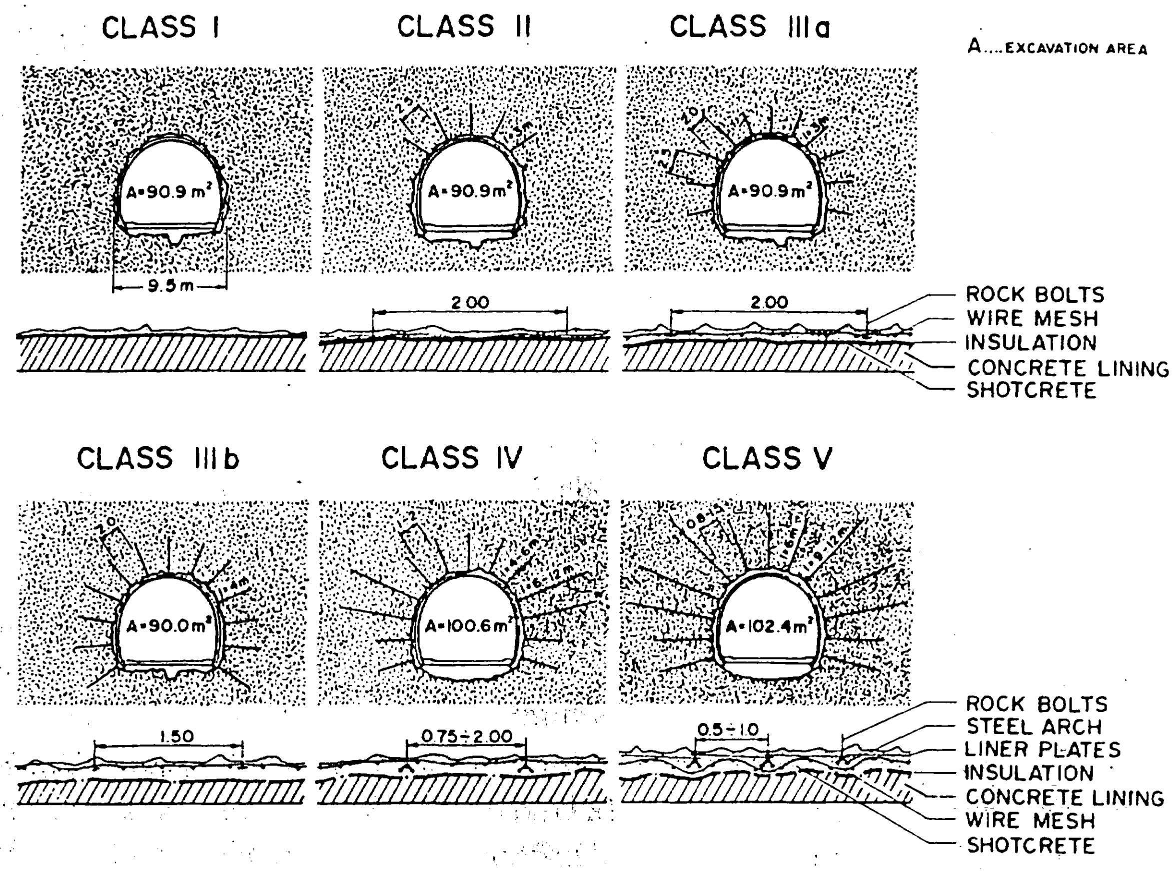

***Figure 6.1** Support measures according to the New Austrian Tunneling Method for the Arlberg Tunnel. (After John, 1980.)*

The European literature is full of descriptions involving successful applications of the New Austrian Tunneling Method, particularly in Austria, West Germany, France, and Switzerland (Sauer, 1988). However, its applications have also spread to other countries, such as Japan, India, Australia, Brazil, and, to a limited extent, the United States (Whitney and Butler, 1983).

In practice, the NATM Classification relates ground conditions, excavation procedure, and tunnel support requirements. The classification, which forms part of the contract, is adapted to a new project based on previous experience and a detailed geotechnical investigation. A particular classification is thus applicable only to the one case for which it was developed and modified. However, the system is highly adaptable and its development can be traced back to Lauffer (1958).

An example of the NATM Classification based on the work of John (1980) is given in Table 6.1. Note that the ground is described behaviorally and the rock mass is allocated a ground class in the field, based on field observations.

Accordingly, the rock mass is classified without a numerical quality rating; ground conditions are described qualitatively. The level of detail depends on the information available from site exploration. There are few published rules that allow the extrapolation to larger or smaller tunnels outside the typical range of 10–12-m width.

Austrian engineers (Brosch, 1986) believe that ground classification and contract conditions are inseparable and that a simple qualitative ground classification is preferable to one involving several parameters leading to an overall rock quality number. Clearly, this could lead to disputes, but since the contractor is paid on the basis of "as found" conditions, the conflict is minimized; if needed, an expert "Gutachter" (appraiser) is usually available to settle any disagreements by making a decision at the face.

6.2 SIZE–STRENGTH CLASSIFICATION

Franklin (1970, 1975) and Louis (1974) have developed a two-parameter classification procedure based on the strength of intact rock and the spacing of discontinuities in the rock mass, in relation to the size of the opening and the overburden stress. In fact, Franklin and Louis worked together on the initial development of this method, but subsequent investigations were reported only by Franklin (1986).

According to Franklin (1986), the "size–strength" approach to rock mass characterization has been found to be helpful in a variety of mining and civil engineering applications, both at the initial stages of planning and for the subsequent day-to-day design of underground excavations and ground control systems.

The concept of block size is analogous to that of grain size, but on a macroscopic scale. The rock mass is conceived as being made up of discrete intact blocks bounded by joints, and its behavior is being governed primarily by a combination of the size and the strength of a "typical" block.

In Figure 6.2, a plot of the size–strength classification is given, with broken and weak rock masses plotting toward the lower left of the diagram. Contours give a general-purpose rock quality index expressed as a decimal; for example, size–strength quality = 2.6.

If the rock unit to be classified is uniform in size and strength, it plots as a single point on the diagram. If the rock unit is variable, the scatter of measured values leads to the unit plotting as a zone. Apparently, the rock quality index may be correlated with the performance parameters relating to excavation and support requirements.

"Block size" is defined as the average "diameter" of a typical rock block in the unit to be classified; it is measured by observing an exposed rock face at the surface or underground, or rock core obtained by drilling (block size is closely related to RQD). Intact strength of the rock material may be estimated by using simple hammer and scratch tests or the point-load index

TABLE 6.1 Ground Classification for the NATM[a]

Class	Ground Behavior	Geomechanical Indicators	Excavation			
			Section	Round Length	Method	Stand-Up Time (Guidelines)
I	Intact rock (freestanding)	The stresses around the opening are less than the rock mass strength; thus, the ground is standing. Due to blasting, separations along discontinuities are possible. For high overburden danger of popping rock	Full face	No limit	Smooth blasting	Crown: weeks Springline: unlimited
II	Lightly afterbreaking	Tensile stresses in the crown or unfavorably oriented discontinuities together with blasting effects lead to separations	Full face	3–5 m	Smooth blasting	Crown: days Springline: weeks
III (formerly IIIa)	Afterbreaking to overbreaking	Tensile stresses in the crown lead to roof falls that are favored by unfavorably oriented discontinuities. The stresses at the springlines	Full face with short round lengths	Full face: 2–4 m	Smooth blasting	Crown and springline: Several hours

		do not exceed the mass strength. However, afterbreaking may occur along discontinuities (due to blasting)				
IV (formerly IIIb)	Afterbreaking to lightly squeezing	1) The rock mass strength is substantially reduced due to discontinuities, thus resulting in many afterbreaks; or 2) the rock mass strength is exceeded leading to light squeezing	Heading and benching (Heading max 45 m^2)	Full face: 2–3 m (heading 2–4 m)	Smooth blasting and local trimming with jackhammer	Crown and springline: a few hours
V	Heavily afterbreaking to squeezing	Due to low rock mass strength, squeezing ground conditions that are substantially influenced by the orientation of the discontinuities	Heading and benching (heading: max 40 m^2)	Heading: 1–3 m Bench: 2–4 m	Smooth blasting or scraping or hydraulic excavator	Crown and springline: very short free stand-up time
VI	Heavily squeezing	After opening the tunnel, squeezing ground is observed on all free surfaces; the discontinuities are of minor importance	Heading and benching (heading max 25 m^2)	Heading: 0.5–1.5 m bench: 1–3 m	Scraping or hydraulic excavator	Very limited stand-up time
VII	Flowing	Requires special techniques, e.g., chemical grouting, freezing, electroosmosis				

(*Table continues on p. 98.*)

Table 6.1 (*Continued*)

Class	Construction Procedure	Support Procedure: Principle	Crown	Springline	Invert	Face
I	Check crown for loose rock	Support against dropping rock blocks	Shotcrete: 0–5 cm			
	When popping rock is present placement of support after each round		Bolts: cap = 15 t Length = 2–4 m Locally as needed	Bolts: cap = 15 t Length = 2–4 m locally	No	No
II	Crown has to be supported after each round	Shotcrete support in crown	Shotcrete: 5–10 cm with wire fabric (3.12 kg/m^2)	Shotcrete: 0–5 cm	Bolts L = 3.5 m if necessary	
	Bolted arch in crown	Bolts: cap = 15 t Length = 2–4 m One per 4–6 m	Bolts: Length = 2–4 m locally			
III	Shotcrete after each round; other support can be placed in stages	Combined shotcrete—bolted round in crown and at springline	Shotcrete: 5–15 cm with wire fabric (3.12 kg/m^2) Bolts: cap = 15–25 t Length = 3–5 m	Shotcrete: 5–15 cm Bolts: 15–25 t Length: 3–5 m One per 3–5 m^2	Adapt invert support to local conditions	Adapt face support to local conditions
IV	Shotcrete after each round Bolts in the heading have to be placed at least after each second round	Combined shotcrete—bolted arch in crown and springline, if necessary closed invert	Shotcrete: 10–15 cm with wire fabric (3.12 kg/cm^2) Bolts: fully grouted Cap = 25 t Length = 4–6 m One per 2–4 m^2	Same as crown	Slab: 20–30 cm	
V	All opened sections have to be supported immediately after opening. All support placed after each round	Support ring of shotcrete with bolted arch and steel sets	Locally linerplates Shotcrete: 15–20 cm with wire fabric (3.12 kg/m^2). Steel sets: TH21 spaced: 0.8–2.0 m Bolts: fully grouted Cap = 25 t Length = 5–7 m One per 1–3 m	Same as crown but no linerplates necessary	Invert arch ≥40 cm or bolts L = 5–7 m if necessary	Shotcrete 10 cm in heading (if necessary) 3–7 cm in bench
VI	As Class V	Support ring of shotcrete with steel sets, including invert arch and densely bolted arch	Linerplates where necessary, shotcrete: 20–25 cm with wire fabric. Steel sets: TH21: 0.5–1.5 m Bolts: cap = 25 t L = 6–9 m One per 0.5–2.5 m^2	Same as crown	Invert: ≥50 cm Bolts: 6–9 m long if necessary	Shotcrete 10 cm and additional face breasting

[a] After John (1978); arrangement by Steiner and Einstein (1980).

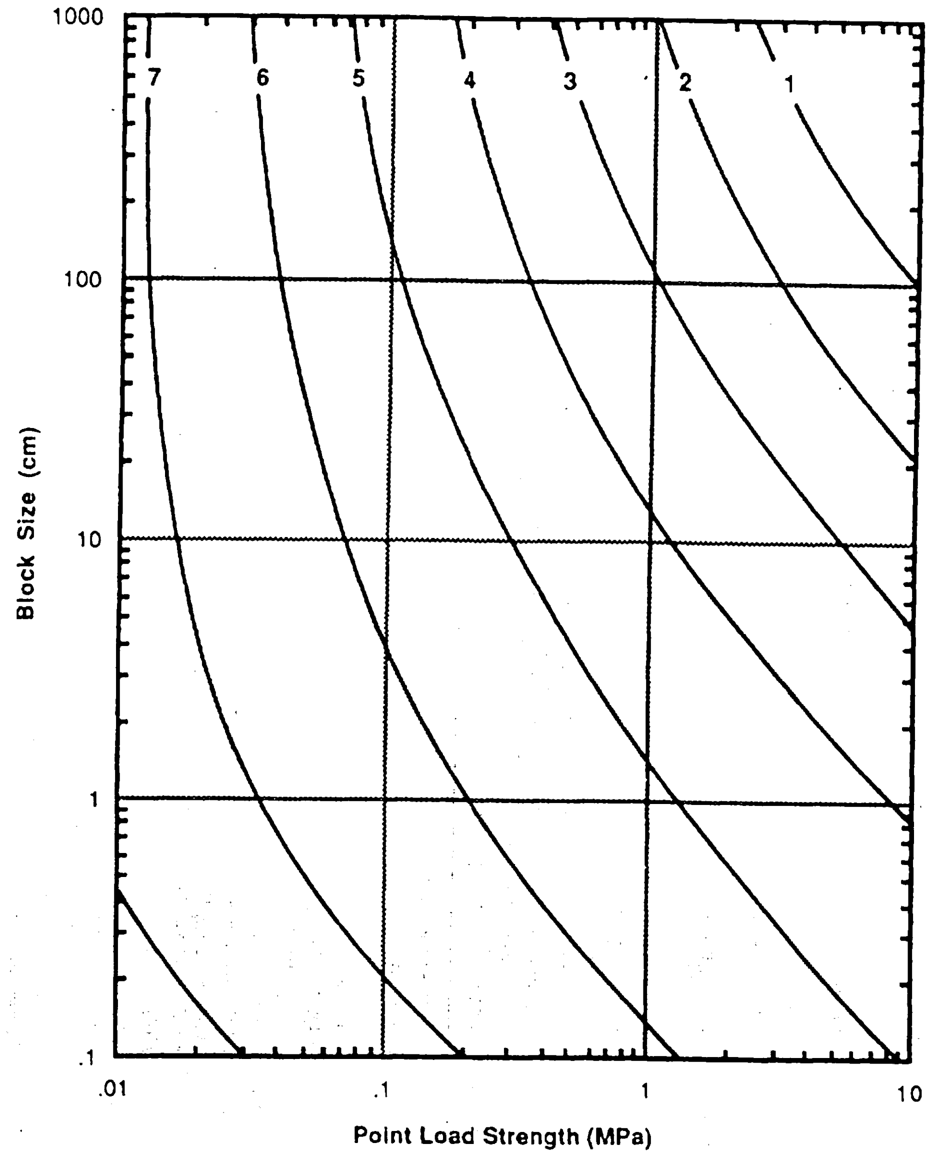

Figure 6.2 *Strength–size classification. (After Franklin, 1975.)*

test. Although measuring inaccuracies are inevitable both for block size and the intact strength determined in such fashions, this is not serious since the values are plotted on logarithmic scales in the classification diagram. Thus, an error of even 20% is usually insignificant.

Figure 6.3 shows a way of applying the concept of the size–strength classification to a preliminary evaluation of tunnel stability and failure mechanisms. In that figure, the zones of rock quality are first plotted in the upper-right quadrant according to the size–strength classification. Ratios of the excavation span to block size and of intact strength to the major principal stress are then examined.

The upper-left quadrant is used to examine the stability of blocks. When the ratio of block size to excavation size is greater than 0.1, blocks should

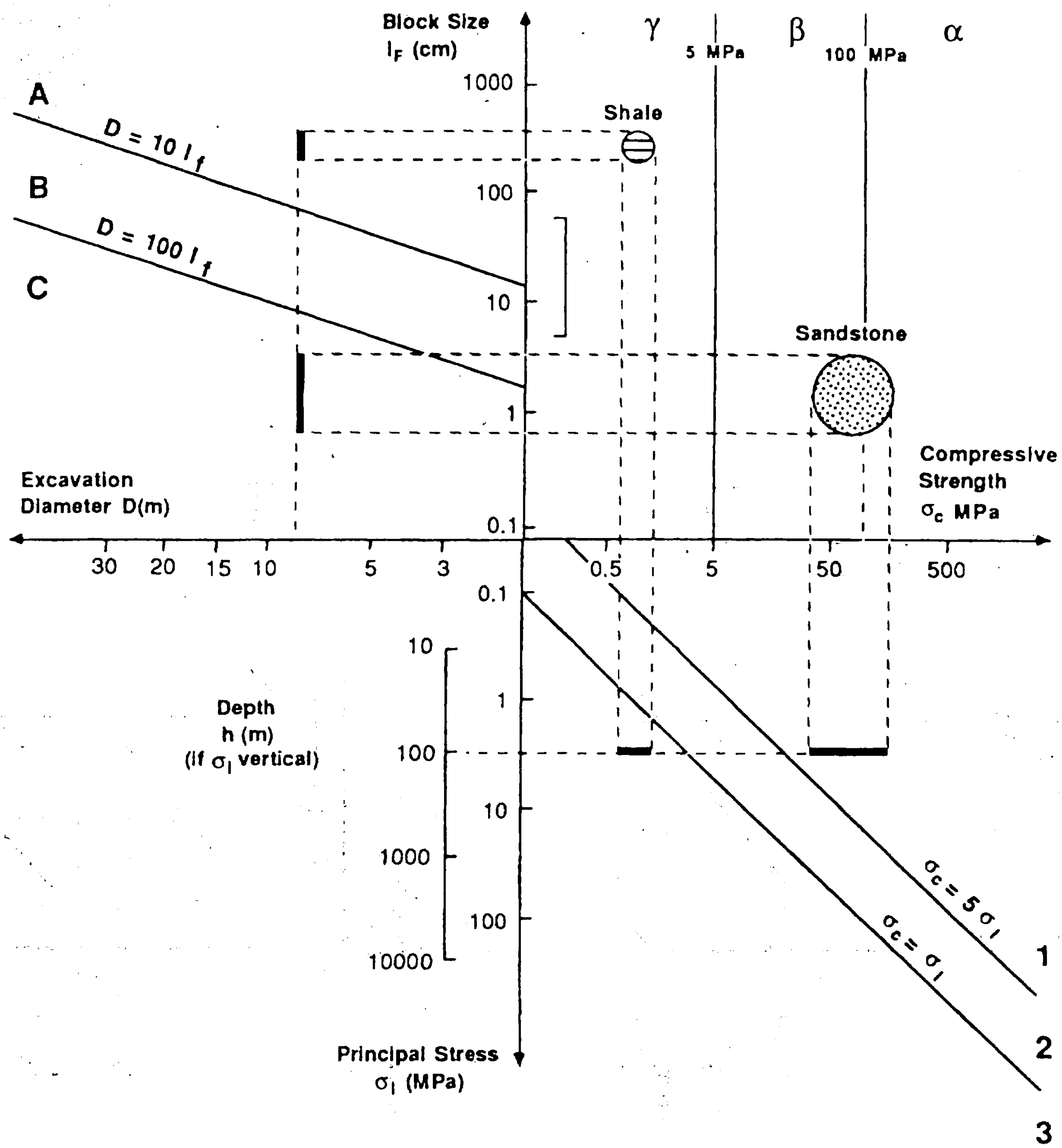

Figure 6.3 Diagram for preliminary evaluation of tunnel stability and potential failure mechanisms. (After Franklin, 1975.)

remain stable; when the ratio is less than 0.01, progressive raveling is likely. The lower-right quadrant provides information on the possibility of rock bursting or ground squeezing. When the strength–stress ratio is greater than 5, no fracture or flow is likely. When this ratio is less than 1, fracture or flow will occur depending on the ratio value: if the rock strength is low, failure will be by squeezing; if rock strength is high, failure will be by rock bursting.

Next, the excavation and support requirements are considered using empirical design procedures. Figure 6.4 enables selection of tunnel support by a variable combination of bolts, shotcrete, mesh, and ribs. The "degree of support number" plotted along the horizontal axis indicates an increasing intensity of support. Note that the size–strength contour numbers in Figure 6.2 correspond to those given in Figure 6.4.

6.3 ISRM CLASSIFICATION

The International Society for Rock Mechanics (ISRM, 1981) developed a general geotechnical description of rock masses aimed at characterizing and

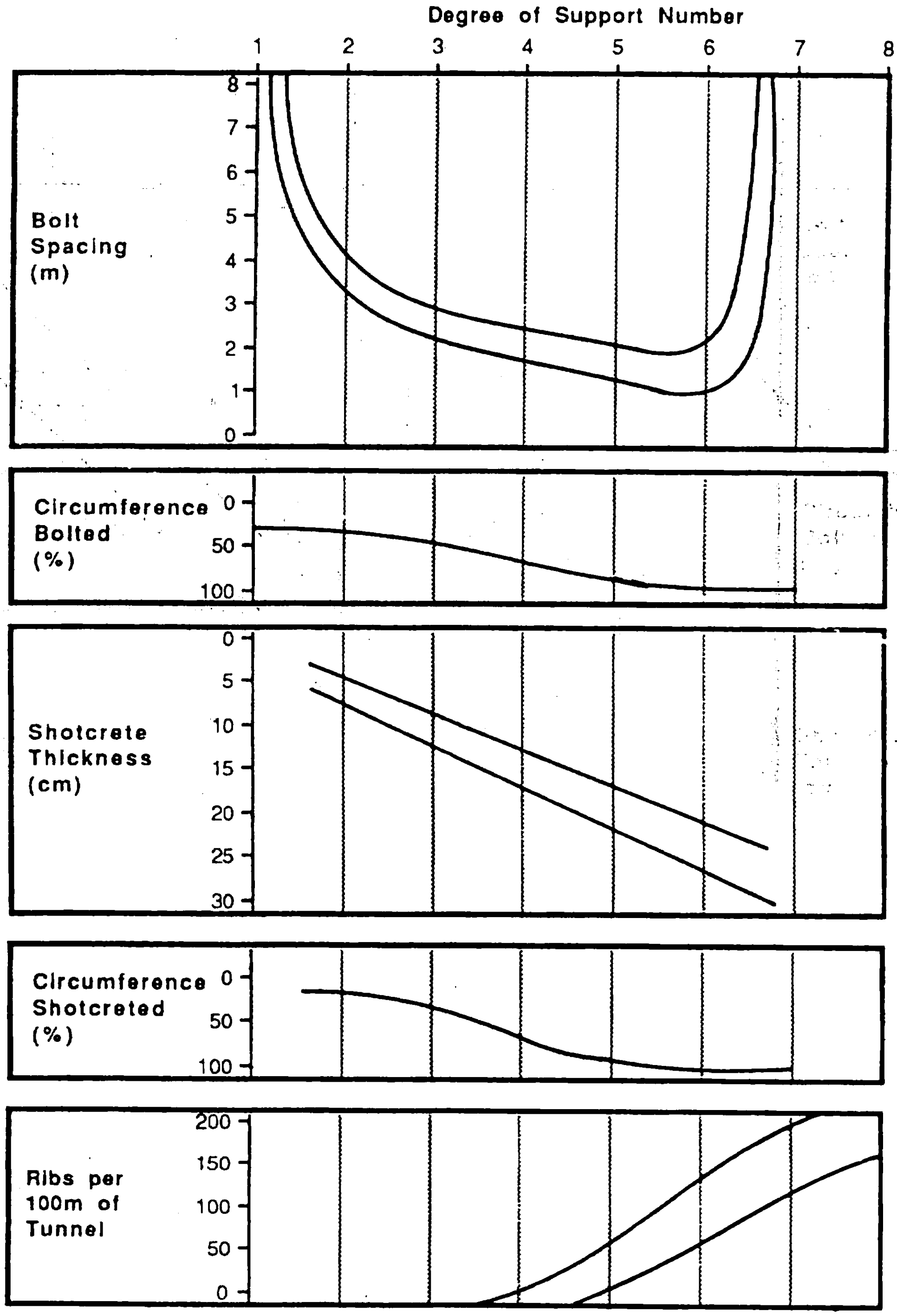

Figure 6.4 Relationship between "degree of support number" (derived from Fig. 6.2) and requirements for support quantities. (After Franklin, 1975.)

classifying, in simplified form, the various regions that constitute a given rock mass. The ISRM classification is not considered an exhaustive description and needs to be supplemented by additional, more detailed, information. Its value lies in presenting unambiguous terms as well as the standard interval limits for the parameters considered.

It was recommended that the following characteristics be taken into account when describing a rock mass:

1. Rock name, with a simplified geological description.
2. Two structural characteristics, namely, layer thickness and discontinuity spacing (fracture intercept).
3. Two mechanical characteristics, namely, the uniaxial compressive strength of the rock material and the angle of friction of the fractures.

The appropriate intervals of values and their descriptions are as follows:

DISCONTINUITY SPACING

Intervals (cm)	Terms
>200	Very wide
60–200	Wide
20–60	Moderate
6–20	Close
<6	Very close

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK MATERIAL

Intervals (MPa)	Terms
>200	Very high
60–200	High
20–60	Moderate
6–20	Low
<6	Very low

ANGLE OF FRICTION OF THE FRACTURES

Intervals (deg)	Terms
>45	Very high
35–45	High
25–35	Moderate
15–25	Low
<15	Very low

The standard intervals of parameters listed above have been incorporated in some rock mass classifications, for example, in the RMR system.

6.4 SPECIALIZED CLASSIFICATION APPROACHES

Important contributions have been made by many investigators who either modified the existing classifications or developed specialized classification approaches to meet a particular engineering application. The work of these contributors, who are listed in Table 3.1, is discussed in the following chapters under the appropriate applications.

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7

Applications in Tunneling

It is not who is right, but what is right,
that is of importance.
*—**Thomas Huxley***

The manner in which rock mass classifications are applied in tunneling is demonstrated in this chapter on the basis of three selected case histories; the first two involve tunnels, and the third a large chamber. Each of the projects deserves special attention from the point of view of rock mass classifications. One tunnel demonstrates the role of rock mass classifications in tunnel design specifications, while the other makes comparisons of classifications with monitoring data. The example of the chamber illustrates the effect of large spans. Additional relevant case histories are referenced.

7.1 PARK RIVER TUNNEL

Nicholson (1988) reviewed this informative case history, previously discussed by Engels et al. (1981), Blackey (1979), Bieniawski (1979), and Bieniawski et al. (1980).

The Park River Auxiliary Tunnel is a water-supply tunnel in the city of Hartford, Connecticut. Its function is flood control; it can divert the overflow of water from one river to another. The tunnel, whose inside diameter is 6.7 m, extends 2800 m between the intake and the outlet. It is excavated through shale and basalt rock at a maximum depth of 61 m below the surface. Located beneath a business district in the city, it is of an inverted siphon shape. The tunnel invert at the outlet is 15.9 m below the intake invert, with the tunnel slope at about 0.6%. A minimum rock thickness of approximately 15.3 m remains above the crown excavation at the outlet.

The bid prices for the tunnel ranged from $33.37 million for the drill and blast option to $23.25 million for machine boring with precast lining. The unit cost was $8303 per meter, based on tunnel boring machine (TBM) bid prices in 1978.

7.1.1 Tunnel Geology

Figure 7.1 shows a longitudinal geological section of the tunnel. The rocks along the alignment are primarily easterly dipping red shales/siltstones interrupted by a basalt dike and two fault zones.

Three major geological zones were distinguished along the tunnel route during preliminary investigations (Blackey, 1979):

1. Shale and basalt zones, constituting 88% of the tunnel.
2. Fractured rock zones (very blocky and seamy), between stations 23+10 and 31+10.
3. Two fault zones, one near station 57+50 and the other between stations 89+50 and 95+50.

Bedding and jointing are generally north/south, which is perpendicular to the tunnel axis (tunnel runs west to east). The bedding is generally dipping between 15° and 20°, whereas the joints are steeply dipping, between 70° and 90°. The joints in the shale have rough surfaces and many are very thin and healed with calcite.

Groundwater levels measured prior to construction of the tunnel indicated that the piezometric level in the bedrock was normally 47–58 m above the invert of the tunnel.

7.1.2 Geological Investigations

Site investigations included diamond core drilling, various tests in the boreholes, and a seismic survey. Tests in the boreholes featured borehole photography, water pressure testing, piezometer installation, observation wells, and pump tests.

Rock cores from 29 boreholes were used to determine the tunnel geology. Of these, 18 were NX (54-mm dia) and 11 were 100 mm in diameter. Ten

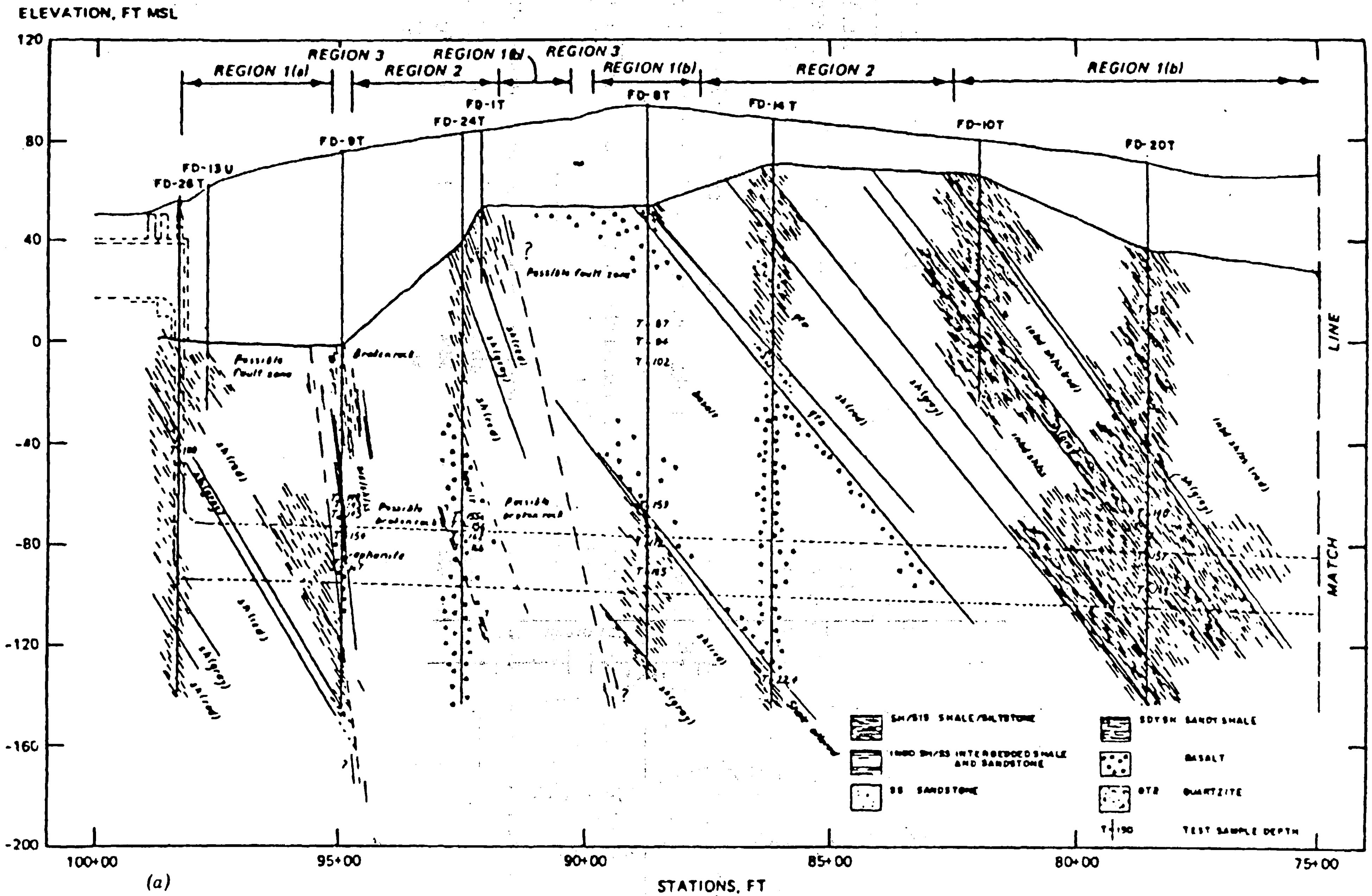
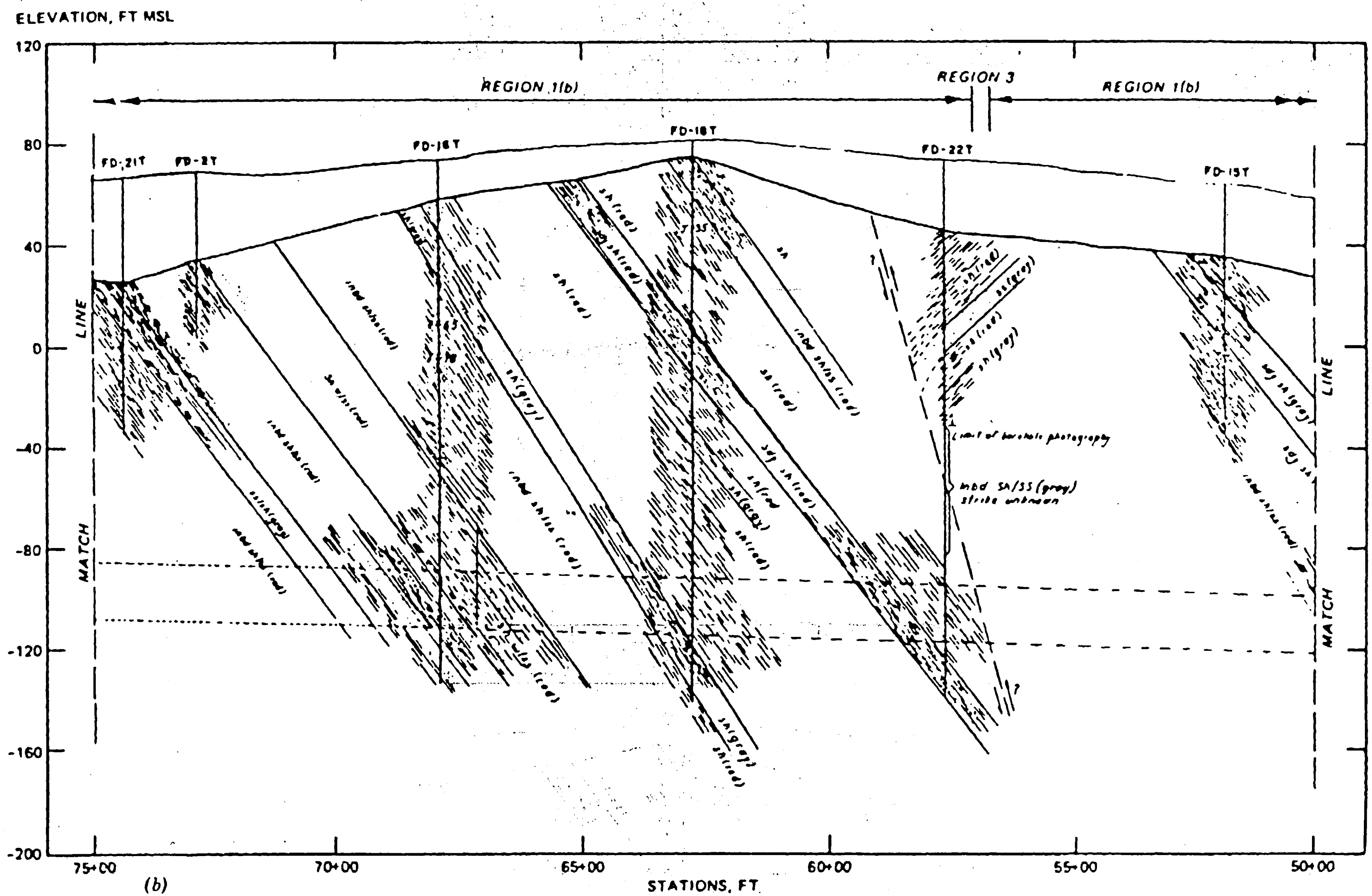
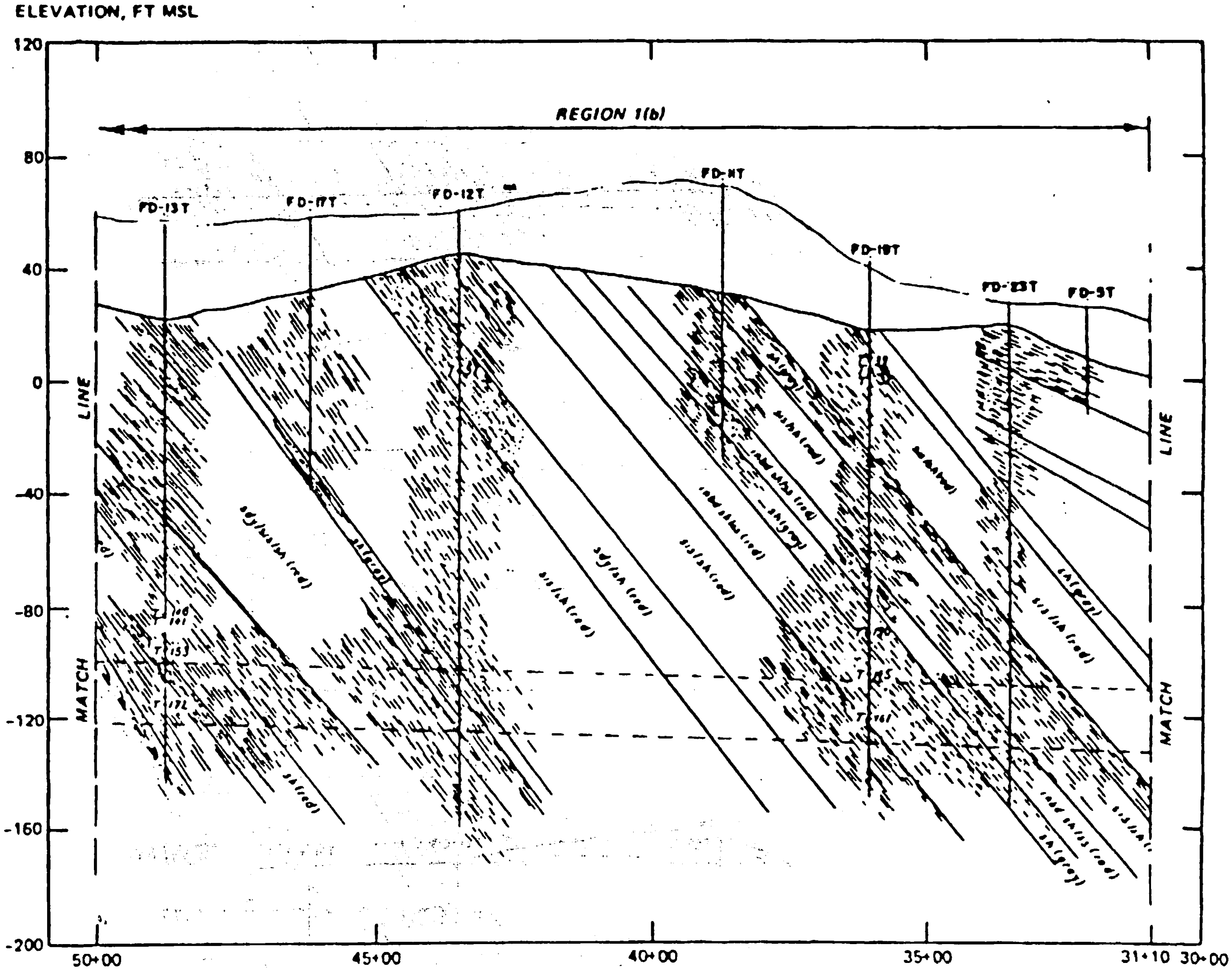


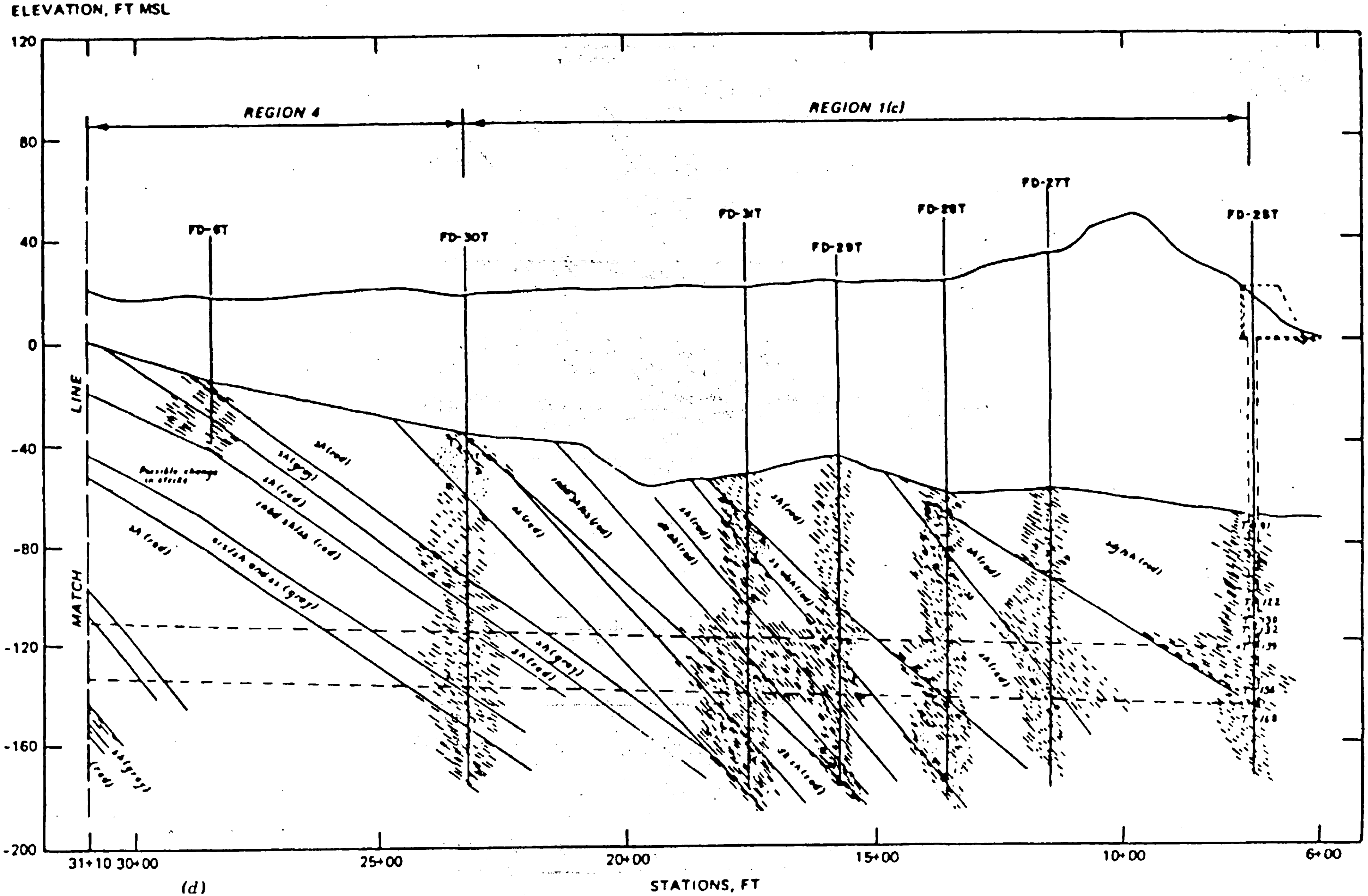
Figure 7.1a–d *Geologic profile of the Park River Tunnel. (Courtesy of the U.S. Army Corps of Engineers.)*


ELEVATION, FT MSL
120
80
40
0
-40
-80
-120
-160
-200
REGION 1(b)
REGION 3
REGION 1(b)
FD-21T
FD-2T
FD-16T
FD-18T
FD-22T
FD-15T
MATCH LINE
MATCH LINE
limit of borehole photography
inbd Sh/SS (grey)
strike unknown
75+00
70+00
65+00
60+00
55+00
50+00
(b)
STATIONS, FT



ELEVATION, FT MSL
REGION 1(b)
FD-13T
FD-17T
FD-12T
FD-11T
FD-19T
FD-23T
FD-5T
LINE
MATCH
120
80
40
0
-40
-80
-120
-160
-200
50+00
45+00
40+00
35+00
31+10
30+00
STATIONS, FT

(c)


ELEVATION, FT MSL
120
80
40
0
-40
-80
-120
-160
-200
REGION 4
REGION 1(c)
FD-6T
FD-30T
FD-31T
FD-29T
FD-28T
FD-27T
FD-25T
LINE
MATCH
31+10 30+00
25+00
20+00
15+00
10+00
6+00
STATIONS, FT

(d)

boreholes did not reach the tunnel level. All cores were photographed in the field immediately upon removal from the core barrel and logged, classified, and tested.

Borehole photography was employed in 15 boreholes to determine the discontinuity orientations and rock structure.

Core samples were selected from 21 localities within the tunnel, near the crown and within one-half diameter above the crown to determine the density, uniaxial compressive strength, triaxial strength, modulus of elasticity, Poisson's ratio, water content, swelling and slaking, sonic velocity, and joint strength. The results are given in Table 7.1.

In-situ stress measurements were conducted in vertical boreholes; out of 15 tests, only three yielded successful results. Eight tests could not be completed due to core breakage and four others failed—due to gage slipping and two to equipment malfunction. The measured horizontal stress was found to be 3.1 MPa ± 0.9 MPa. For the depth of 36.6 m, the vertical stress was calculated as 0.91 MPa. This gave the horizontal to vertical stress ratio as 3.4.

7.1.3 Input Data for Rock Mass Classifications

Input data to permit rock mass classifications have been compiled for all the structural regions anticipated along the tunnel route; in Figure 7.2, for example, they are depicted for the outlet region. It should be noted that all the data entered on the classification input sheets have been derived from the boreholes, including the information on discontinuity orientation and spacing. This was possible because borehole photography was employed for borehole logging, in addition to the usual core-logging procedures.

7.1.4 Tunnel Design Features

Three different tunnel sections were designed and offered as bid options:

1. Drill and blast with a reinforced, variable-thickness, cast-in-place liner designed to meet three ranges of rock loading.

TABLE 7.1 Summary of Rock Properties at the Park River Tunnel

Rock Material	No. of Tests	Uniaxial Compressive Strength (MPa)	No. of Tests	Modulus of Elasticity (GPa)
Shale	19	22.4–90.3 (av 53.4)	7	1.38–34.5 (av 14.5)
Basalt	11	38.2–94.8 (av 70.8)	9	6.14–68.9 (av 31.9)
Sandstone	2	64.5–65.8 (av 65.1)		

Name of project *Park River Tunnel*
Site of survey *Hartford, Conn.*
Conducted by *C. A. N.*
Date *July 15, 1978*

STRUCTURAL REGION	ROCK TYPE AND ORIGIN
13+10–7+10	*Shale with interbedded sandstone*

DRILL CORE QUALITY R.Q.D.*		
Excellent quality:	90 - 100%	
Good quality:	75 - 90%	
Fair quality:	50 - 75%	✓
Poor quality:	25 - 50%	
Very poor quality:	<25%	

*R.Q.D. : Rock Quality Designation

WALL ROCK OF DISCONTINUITIES	
Unweathered	
Slightly weathered	✓
Moderately weathered	
Highly weathered	
Completely weathered	
Residual soil	

GROUND WATER

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

STRENGTH OF INTACT ROCK MATERIAL

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

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3	Set 4
Very wide	Over 2 m				
Wide	0.6 - 2 m		✓		
Moderate	200 - 600 mm	✓			
Close	60 - 200 mm				
Very close	<60 mm				

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

STRIKE AND DIP ORIENTATIONS

Set 1	Strike *N23E* (average)	(from *N5E*	to *N35E*)	Dip *20* (angle)	*SE* (direction)
Set 2	Strike *N47E*	(from *N40E*	to *N60E*)	Dip *20*	*SE*
Set 3	Strike	(from	to)	Dip	
Set 4	Strike	(from	to)	Dip	

NOTE: Refer all directions to magnetic north

CONDITION OF DISCONTINUITIES

		Set 1	Set 2	Set 3	Set 4
PERSISTENCE (CONTINUITY)					
Very low:	<1 m				
Low:	1 - 3 m				
Medium:	3 - 10 m				
High:	10 - 20 m	✓	✓		
Very high:	> 20 m				
SEPARATION (APERTURE)					
Very tight joints	<0.1 mm				
Tight joints	0.1 - 0.5 mm				
Moderately open joints	0.5 - 2.5 mm	✓	✓		
Open joints:	2.5 - 10 mm				
Very wide aperture	> 10 mm				
ROUGHNESS (state also if surfaces are stepped, undulating or planar)					
Very rough surfaces					
Rough surfaces		✓	✓		
Slightly rough surfaces:					
Smooth surfaces					
Slickensided surfaces:					
FILLING (GOUGE)					
Type					
Thickness					
Uniaxial compressive strength, MPa					
Seepage					

MAJOR FAULTS OR FOLDS

Several small fracture zones were found in core logs. Zones range from 100 mm to 0.3 m in thickness and occur between sta. 16+00–13+50.

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

GENERAL REMARKS AND ADDITIONAL DATA

Random joints present

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

Figure 7.2 Input data sheet for structural region 1(c) of the Park River Tunnel.

2. Machine excavation with a reinforced cast-in-place lining.
3. Machine excavation with a reinforced precast lining.

Table 7.2 gives the recommended support and rock loads as based on the Terzaghi method.

The support recommendations were also prepared from other rock mass classification systems and are included in Table 7.3 (Bieniawski, 1979). The main conclusion to be drawn from this table is that the Terzaghi method, which recommends the most extensive support measures, clearly seems excessive by comparison with the recommendations of the other three classification systems. The reason for this is threefold. Firstly, the current permanent lining design does not account fully for the action of the temporary support, which in itself may be sufficient for the structural stability of the tunnel. Secondly, the original modifications of the Terzaghi method by Deere et al. (1970) were based on 1969 technology, which is now outdated. Thirdly, not enough use is made in the Terzaghi method of the ability of the rock to support itself. The Terzaghi method uses such qualitative rock mass descriptions as "blocky" and "seamy," which do not fully utilize all the quantitative information available from the site exploration program.

Tunnel instrumentation was planned to provide for design verification, future design applications, and monitoring of construction effects (Engels et al., 1981). Ten test sections at locations of different geologic conditions were selected in the tunnel. These sections consisted of extensometers (MPBXs) installed from the surface, as well as pore pressure transducers, rock bolt load cells, convergence points, and surface and embedded strain gages installed within the tunnel. Further, in-situ stress measurements were also considered. Since the precast liners were designed for the worst ground conditions (10% of the tunnel) but were utilized throughout the tunnel, they were in effect overdesigned for the major portion of the tunnel. The purpose of the instrumentation program was to validate design assumptions and to refine the calculations for future designs.

7.1.5 Construction

The greatest number of bids was made on the precast liner option, with five of the seven acceptable bids ranging in price from $23,248,185 to $28,551,497. The highest bid for the drill and blast option was $33,374,140 (Blackey, 1979).

The tunnel was advanced upgrade from the outlet shaft. Upon completion of the outlet shaft, approximately the first 72 m of the tunnel was advanced

TABLE 7.2 Park River Tunnel: Tunnel Design Rock Loads and Support Based on Terzaghi's Method

		Drill and Blast Construction: Diameter 26 ft			Machine Boring: Diameter 24 ft		
Rock Condition	Length of Zone (ft)	Rock Load (tsf)	Temporary Support	Permanent Lining	Rock Load (tsf)	Temporary Support	Permanent Lining
Best average quality: massive, moderately jointed RQD > 80	8000	1.1	11-ft bolts at 4½ ft, shotcrete 1 in. thick	Reinforced concrete 14 in thick plus 8-in. overbreak	0.5	10-ft bolts occasionally at 6 ft, shotcrete 2 in. if needed	Reinforced precast liner 9 in. thick, grouted
Worst average quality: very blocky, seamy RQD = 40	800	2.2	11-ft bolts at 2 ft, shotcrete 2 in. thick	Reinforced concrete 15 in. thick plus 8-in. overbreak	1.4	10-ft bolts at 3–5 ft, shotcrete 2 in. if needed	As above
Fault zones: completely crused RQD = 30	300	4.8	W8 steel beams at 2–4 ft, shotcrete 3 in. thick	Reinforced concrete 22 in. thick plus 8-in. overbreak	3.5	10-ft bolts at 3 ft, shotcrete 3 in. thick	As above

TABLE 7.3 **Park River Tunnel: Comparison of Support Recommendations**

	Support System			
Rock Conditions	Terzaghi's Method	RSR Concept	Geomechanics Classification	Q-System
Best average conditions: regions 1 and 2	Rock load: 1.1 tsf Reinforced concrete 14 in. thick plus 8-in. overbreak Temporary: 11-ft bolts at 4½ ft, shotcrete 1 in. thick	RSR = 76 Permanent: NA[a] Temporary: none	RMR = 72 Locally, rock bolts in roof 10 ft long at 8-ft spacing plus occasional mesh and shotcrete 2 in. thick	Rock load: 0.5 tsf $Q = 20$ Untensioned spot bolts 9 ft long spaced 5–6 ft. No shotcrete or mesh
Worst average conditions: sta. 23+00 to 31+00	Rock load: 2.2 tsf Reinforced concrete 15 in. thick plus 8-in. overbreak Temporary: 11-ft bolts at 2 ft, shotcrete 2 in. thick	RSR = 26 Permanent: NA[a] Temporary: 8W40 steel ribs at 2 ft	RMR = 37 Systematic bolts 12 ft long at 5-ft spacing with wire mesh plus shotcrete 5 in. thick	Rock load: 1.1 tsf $Q = 2.2$ Untensioned systematic bolts 9 ft long at 3-ft spacing plus shotcrete 1–2 in. thick Primary: spot bolts
Fault zones: region 3	Rock load: 4.8 tsf Reinforced concrete 22 in. thick plus 8-in. overbreak Temporary: steel ribs: W8 ring beams at 2–4 ft, shotcrete 3 in.	RSR = 23 Permanent: NA[a] Temporary: 8W40 steel ribs at 2 ft	RMR = 16 Steel ribs at 2½ ft, 15 ft with wire mesh plus shotcrete 8 in. thick	Rock load: 2.7 tsf $Q = 0.14$ Reinforced concrete 816 in. thick plus tensioned 9-ft bolts at 3 ft Primary: shotcrete 6–10 in. with mesh

[a] Not applicable.

using drill and blast excavation techniques to form a U-shaped chamber about 7.9 × 7.9 m in cross section. The roof of the tunnel in the drill and blast section of the project was supported with 3-m-long fully resin-grouted rock bolts installed on approximately 1.2–1.5-m centers and shotcreted.

After completion of the drill and blast section, the tunnel boring machine (TBM) was assembled in the excavated chamber, and the tunnel advance using the TBM began. The TBM was a fully shielded, rotary hard-rock machine manufactured by the Robbins Company of Seattle, Washington, which cut a 7.4-m diameter bore. The temporary support and final lining were provided by four-segment precast concrete liner rings that were erected in the tail shield of the TBM about 11–12 m behind the cutter face. Each of the four segments was 22.9 cm thick and about 1.8 m wide. A completed ring provided a finished inside diameter of 6.7 m. Circumferential sponge rubber O-rings were provided between rings, and neoprene pad gaskets and a hydraulic cement sealant were used between segments (Engels et al., 1981).

7.1.6 Examples of Classification Procedures

Item 1: Classification of Rock Mass Conditions

a. *Terzaghi:* "moderately blocky and seamy" (RQD = ~72%).

b. *RSR Concept:*

—Rock type: soft sedimentary rock;
—Slightly faulted and folded;
—Parameter $A = 15$;
—Spacing: moderate to blocky;
—Strike approximately perpendicular to tunnel axis, dip 0–20°;
—Parameter $B = 30$;
—Water inflow: moderate;
—Joint conditions: fair (moderately open, rough, and weathered);
—For: $A + B = 45$, parameter $C = 16$;
—Therefore: RSR = 15 + 30 + 16 = 61.

c. *Geomechanics Classification (RMR):*

—Intact rock strength, $\sigma_c = 50$ MPa
Rating = 4;
—Drill core quality, RQD = 55–58%; av 72%
Rating = 13;

—Spacing of discontinuities, range 50 mm to 0.9 m
Rating : 10;

—Condition of discontinuities; separation 0.8 mm to 1.1 mm, slightly weathered, rough surfaces
Rating: 25;

—Groundwater: dripping water, low pressure, flow 25–125 L/min
Rating 4;

—Basic RMR: 4 + 13 + 10 + 25 + 4 = 56 without adjustment for orientation of discontinuities;

—Discontinuity orientation: strike perpendicular to tunnel axis, dip 20°;
Fair orientation, adjustment: −5, adjusted RMR = 56 − 5 = 51;

—RMR = 51, represents Class III; fair rock mass.

d. *Q-System:*

—RQD = 72% (average);

—J_n = 6, two joint sets and random;

—J_r = 1.5, rough, planar joints;

—J_a = 1.0, unaltered joint walls, surface staining only;

—J_w = 0.5, possible large water inflow;

—SRF = 1.0, medium stress, $c/1 = 50/0.91 = 55$.

$Q = \text{RQD}/J_n \times J_r/J_a \times J_w/\text{SRF} = 9.0$ Fair rock mass.

Summary

Classification	Result
Terzaghi	Moderately blocky and seamy
RSR	61
RMR	51 Fair rock mass
Q	9.0 Fair rock mass

Item 2: Rock Loads

Drill and blast diameter: 7.4 m + 0.6 m overbreak = 8.0 m
Machine-bored diameter: 7.4 m
Shale density: 2660 kg/m^3 (166 lb/ft^3).

Method	Drill and Blast	TBM
Terzaghi	$h_p = 0.35C = 0.7B = 0.7 \times 8.0 = 5.6$ m Rock load $P = \gamma h_p = 0.146$ MPa (1.52 t/ft^2)	$h_p = 0.45B = 3.3$ m $P = 0.09$ MPa (0.9 t/ft^2)
RSR = 61	From Figure 3.3, $P = 0.067$ MPa (1.2 kip/ft^2)	TBM adjustment, RSR = 69.5, $P = 0.034$ MPa (0.7 kip/ft^2)
RMR = 51	$h_p = \frac{100 - 51}{100} B = 3.92$ m $P = \gamma h_t = 0.102$ MPa	TBM adjustment via conversion to RSR RMR = 74, $P =$ 0.049 MPa
$Q = 9$	$P = \frac{2.0}{J_r} Q^{-1/3} = \frac{2.0}{1.5}(9)^{-1/3} = 0.64$ kg/cm^2 $= 0.0628$ MPa or $P = \frac{2J_n^{1/2}}{3J_r} Q^{-1/3} = \frac{2\sqrt{6}}{31.5}(9)^{-1/3}$ $= 0.52$ kg/cm^2 $= 0.0513$ MPa	TBM adjustment via conversion to RSR $Q = 54$ $P = 0.0321$ MPa

Summary of Rock Loads in kPa (1 MPa = 1000kPa):

Method	Drill and Blast	TBM
Terzaghi	146	90
RSR	67	34
RMR	102	49
Q	63	32

Item 3: Self-supporting Span and Maximum Span: by RMR and Q Systems

Use Figure 4.1: span versus stand-up time

	RMR = 51	$Q = 9$ (ESR = 1.6)
Self-supporting span	2.4 m	
Maximum span	10.5 m	8 m [$D = 2(1.6) \times 9^{0.4}$]

Item 4: Stand-Up Time, Deformability and c, ϕ Values

For RMR = 51 and span = 8 m;

Stand-up time: approximately 70 h or 3 d;

Deformability, RMR = 56 (no adjustment for joint orientations);

$E = 2\ \text{RMR} - 100 = 12\ \text{GPa}(1.74 \times 106\ \text{psi});$
$c = 192\ \text{kPa};$
$\phi = 39^\circ$ (Table 4.1).

Item 5: Support Recommendations

Terzaghi: Drill and blast—light to medium steel sets spaced 1.5 m. Concrete lining.

RSR: Drill and blast—6H25 ribs on 2-m centers plus concrete lining.

RMR: Drill and blast—systematic bolts 3.5 m long spaced 1.5 m, shotcrete 50 to 100 mm in roof and 30 mm on walls, wire mesh in crown.

Q-System: Drill and blast—3 m long rock bolts spaced 1.5 m and 50 mm thick shotcrete.

Item 6: Tabulation of Results from Items 1–5

Item	Terzaghi	RSR	RMR	*Q*
Shale quality	Moderately blocky and seamy	61	51	9.0
Rock load height (m)	5.6	N/A[a]	3.9	N/A[a]
Rock load (kPa)	146	67	102	63
Stand-up time	N/A[a]	N/A[a]	3 d	N/A[a]
Support	Ribs at 1.5 m Concrete lining	Ribs at 2 m Concrete	3.5 m bolts at 1.5 m, shotcrete 50 to 100 mm, wire mesh	3 m bolts at 1.5 m, shotcrete 50 mm thick

[a] Not applicable.

7.2 OVERVAAL RAILROAD TUNNEL

Discussed by Davies (1976) and by Bieniawski and Maschek (1975), the Overvaal Tunnel is a good example how reliability of rock mass classifications can be cross-checked by in-situ monitoring of tunnel behavior during construction.

7.2.1 Geological Features of the Tunnel

The sedimentary rocks in the vicinity of the tunnel are essentially horizontally bedded sandstones and shales. A dolerite (diabase) sill of undetermined thickness has intruded these sedimentary rocks. Subsequent faulting of all the rock types has disturbed the structure to some extent.

The tunnel itself lies entirely in dolerite, which consists of feldspars, augite, and some accessory minerals. The rock material is hard to very hard and generally shows no weathering. The rock mass is extensively jointed. Difficult water conditions were encountered in some sections of the tunnel.

A longitudinal section of the geology of the tunnel is given in Figure 7.3. The geological investigations during the construction of the tunnel involved detailed joint surveys in the excavated portions of the tunnel and provided data on joint orientations, spacing, and condition as well as on groundwater conditions. Measurements of strike and dip of the main discontinuities were made throughout the length of the tunnel.

Rock quality designation (RQD) was determined from drill cores, and uniaxial compression strength tests on rock samples were made. Finally, thin sections for petrographic analyses were prepared and analyzed.

7.2.2 Rock Mass Conditions

Sixteen measuring stations were installed in representative or critical rock mass conditions in each heading. The rock mass conditions were determined

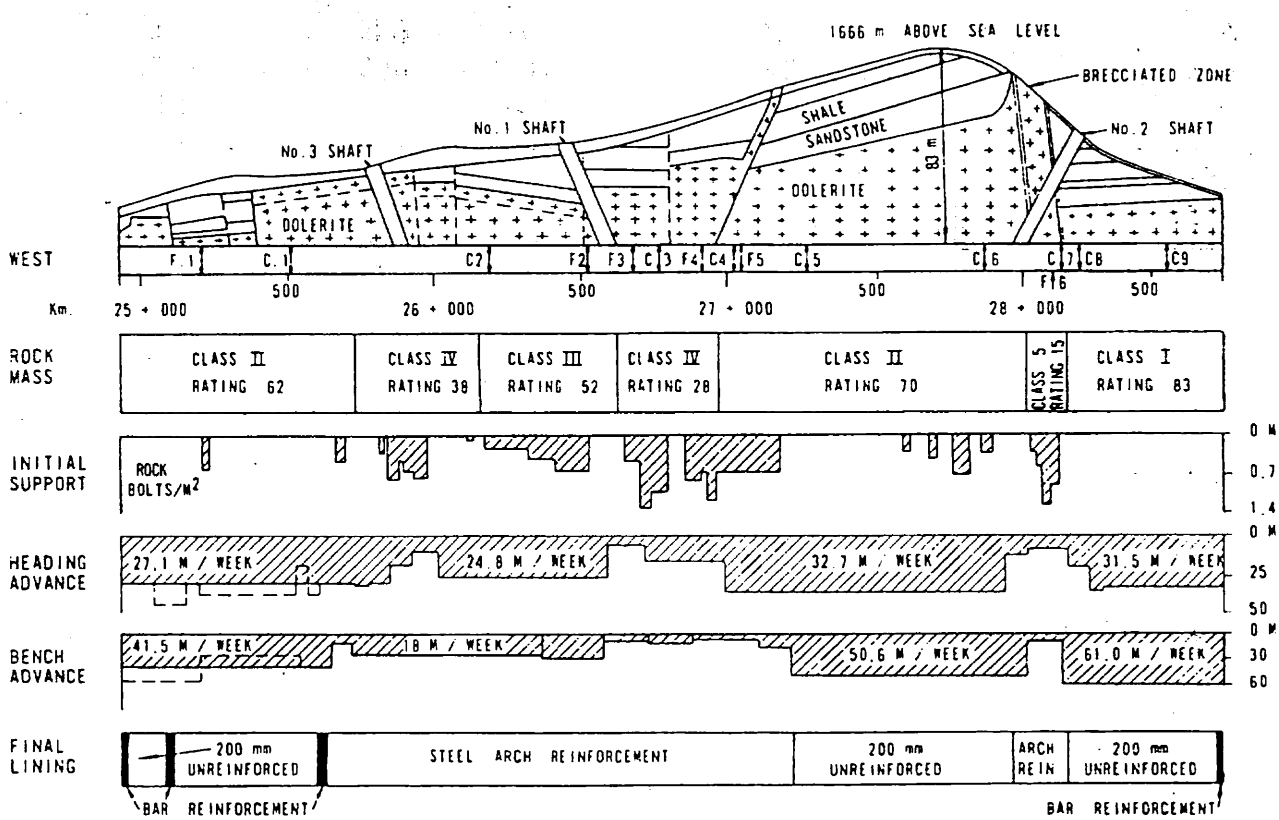


Figure 7.3 *Geotechnical data for a railroad tunnel. (After Davies, 1976.)*

in terms of the Geomechanics Classification, with each station being individually mapped. The rock mass classes and the classification ratings for each heading at the Overvaal Tunnel are shown in Figure 7.3.

7.2.3 Site Exploration

Geological exploration consisted of 18 boreholes supplemented by 16 percussion holes, together with resistivity and seismic surveys.

The boreholes showed that the complete tunnel would be in dolerite, with possibly three difficult sections. One of these was where the tunnel roof was close to the overlying sandstone contact and where shattered dolerite could be present. The second was where a zone of laminated dolerite was discovered, and the third was brecciated material associated with an intrusion along a fault line. The geological section along the tunnel is shown in Figure 7.3.

The borehole data were supplemented by resistivity surveys and by percussion holes drilled to clarify water problems. These additional methods also helped to locate, with greater accuracy, faults indicated by the seismic surveys.

A comparison of the support recommendations by six different classification systems is given in Table 7.4.

7.3 ASSESSMENT OF UNDERGROUND CONDITIONS FROM SURFACE ROCK EXPOSURES

Gonzalez de Vallejo (1983) presented an approach for classifying underground rock conditions based on surface rock exposure data. Using the Geomechanics Classification, he introduced corrections to the RMR ratings and demonstrated their use in tunnels and mines in Spain.

The classification procedure used in this approach is depicted in Tables 7.5 and 7.6, which are self-explanatory.

7.4 LARGE UNDERGROUND CHAMBERS

The value of rock mass classifications in the design of large underground chambers lies in their potential to identify possible instability problems and permit correlations of in-situ testing and monitoring data with rock mass quality for future uses. This may lead to estimates of rock mass deformability on the basis of rock mass classifications and may provide effective planning of the excavation sequence in trial enlargements.

TABLE 7.4 Comparison of Rock Mass Classifications Applied at the Overvaal Tunnel (Width: 5.5 m)

	Geomechanics Classification		Q-System	
Locality	Class	Support	Class	Support
H 6	I Very good rock RMR = 83	Occasional spot bolting	Good rock $Q = 33.0$	Spot bolting only
H 4	II Good rock RMR = 67	Locally, grouted bolts (20-mm dia) spaced 2–2.5 m, length 2.5 m plus mesh; shotcrete 50 mm thick if required	Good rock $Q = 12.5$	Systematic grouted bolts (20-mm dia) spaced 1–2 m, length 2.8 m
H 2	III Fair rock RMR = 52	Systematic grouted bolts spaced 1.5–2 m, length 3 m plus mesh and 100-mm-thick shotcrete	Fair rock $Q = 8.5$	Systematic grouted bolts spaced 1.5 m, length 2.8 m, and mesh
H 3	IV Poor rock RMR = 29	Systematic grouted bolts spaced 1–1.5 m, length 3 m, mesh plus 100–150-mm shotcrete (ribs at 1.5 m)	Poor rock $Q = 1.5$	Shotcrete only: 25–75 mm thick or bolts at 1 m, 20–30-mm shotcrete and mesh
H 5	V Very poor rock RMR = 15	Systematic grouted bolts spaced 0.7–1 m, length 3.5 m, 150–200-mm shotcrete and mesh plus medium steel ribs at 0.7 m. Closed invert	Extremely poor rock $Q = 0.09$	Shotcrete only: 75–100 mm thick or tensioned bolts at 1 m plus 50–75-mm shotcrete and mesh

	RSR Concept		RQD Index	
Locality	Class	Support	Class	Support
H 6	RSR = 68	Bolts 25-mm dia at 2 m (length not given)	Excellent RQD < 90	Occasional bolts only

H 4	RSR = 60	Bolts spaced 1–4 m, shotcrete 35–45 mm or medium ribs at 2 m	Good RQD: 75–90	Bolts 25-mm dia, 2–3 m long, spaced 1.5–1.8 m and some mesh or 50–75-mm shotcrete or light ribs
H 2	RSR = 57	Bolts spaced 1.2 m and 50-mm shotcrete or ribs 6H20 at 1.7 mm	Fair to good RQD: 50–90	Bolts 2–3 m long at 0.9–1 m plus mesh or 50–100-mm shotcrete or light/medium ribs at 1.5 m
H 3	RSR = 52	Bolts spaced 1 m and 75-mm shotcrete or ribs 6H20 at 1.2 m	Poor RQD: 25–50	Bolts 2–3 m long at 0.6–1.2 m with mesh or 150-mm shotcrete with bolts at 1.5 m or medium to heavy ribs
H 5	RSR = 25	NA[a]	Very poor RQD < 25	150-mm shotcrete all around plus medium to heavy circular ribs at 0.6-m centers with lagging

	NATM Classification		Size–Strength Classification	
Locality	Class	Support	Class	Support
H 6	I Stable	Bolts 26-mm dia, 1.5 m long, spaced 1.5 m in roof plus wire mesh.	A	50-mm shotcrete or 3-m-long bolts at 3.1 m
H 4	II Overbreaking	Bolts 2–3 m long spaced 2–2.5 m, shotcrete 50–100 mm with mesh	B	100-mm shotcrete with mesh and 3-m bolts at 2.8 m
H 2	III Fractured to very fractured	Perfo-bolts 26-mm dia, 34 m long, spaced 2 m plus 150-mm shotcrete plus wire mesh and steel arches TH16 spaced 1.5 m	C	150-mm shotcrete with mesh and 3-m bolts at 2.5 m
H 3	IV Stressed rock	Perfo-bolts 4 m long, spaced 1 m × 2 m and 200-mm shotcrete plus mesh and steel arches TH21 spaced 1 m. Concrete lining 300 mm	D	210-mm shotcrete with mesh and 3-m bolts at 2 m and steel ribs
H 5	V Very stressed rock	Perfo-bolts 4 m long spaced 1 m and 250-mm shotcrete plus mesh and steel arches TH29 spaced 0.75 m. Closed invert. Concrete lining 500 mm	E	240-mm shotcrete with mesh and 2-m bolts at 1.7 m, steel ribs at 1.2 m. Closed invert

[a] Not applicable.

TABLE 7.5 Geomechnics Classification from Surface Exposures[a]

Rock Quality Indexes	Range of Values				
1. Intact rock strength Point-load test (MPa) Uniaxial compressive strength (MPa) Rating	>8 >250 15	8–4 250–100 12	4–2 100–50 7	2–1 50–25 4	NA[b] 25–5 5–1 <1 2 1 0
2. *Spacing or RQD* Spacing (m) RQD (%) Rating	<2 100–90 20	2–0.6 90–75 17	0.6–0.2 75–50 13	0.2–0.06 30–25 8	<0.06 <25 3
3. *Conditions of discontinuities*[c] Rating	Very rough surfaces Not continuous joints No separation Hard joint wall 30	Slightly rough surfaces Not continuous joints Separation > 1 mm Hard joint wall 25	Slight rough surfaces Not continuous joints Separation 1 mm Soft or weathered joint walls 20	Slickensided surfaces Continuous joints Joints open 1–5 mm Gouge materials 10	Slickensided surfaces Continuous joints Joints open < 5 mm Gouge materials >5 mm thick 0

4. *Groundwater* Inflow per 10-m tunnel length (4 min)	None	<10	10–25	25–125	>125
General conditions	Dry	Slightly moist	Occasional seepage	Frequent seepage	Abundant seepage
Rating	15	10	7	4	0
5. *State of Stresses* Competence factor (vertical stress/intact strength)	<10	10–5	5–3	<3	
Rating	10	5	−5	−10	
Tectonic history	Zones near thrusts/faults of regional importance		Compression		Tension
Rating	−5		−2		0
Neotectonic activity	None or unknown		Assumed		Confirmed
Rating	0		5		−10
6. *Rock Mass Classes* Class number	I	II	III	IV	V
Rock quality	Very good	Good	Fair	Poor	Very poor
Rating	100–81	80–61	60–41	40–21	≤20

[a] After Gonzalez de Vallejo (1983).
[b] Not applicable.
[c] Adjustment for orientation as in Bieniawski (1979).

TABLE 7.6 Adjustment to Ratings for the Geomechanics Classification Based on Surface Data[a]

The Total Rating from Table 7.5 must be adjusted for the following factors:	
Excavation Methods	
Tunneling boring machines, continuous miner, cutter machines, roadheaders, etc.	+10
−Controlled blasting, presplitting, soft blasting, etc.	+5
Poor-quality blasting[b]	−10
Support Methods[c]	
Class I	0
Class II <10 d >10 d < 20 d >20 d	 5 −5 −20
Class III <2 d >2 d < 5 d >5 d < 10 d >10 d	 5 0 −5 −20
Class IV and V <8 h >8 h <24 h >24 h	 0 −10 −20
Distance to Adjacent Excavation[d]	
AEF < 2.5	−20
2.5 < AEF < 10	−10
AEF > 10	0
Portals, Accesses, and Areas with Small Overburden Thickness[e]	
PF > 5	−20
5 > PF > 10	−10
PF < 10	0

[a] After Gonzalez de Vallejo (1983).
[b] Conventional blasting: EMF = 0.
[c] Based on Bieniawski (1979) graphic representation of the stand-up-time and the unsupported span, the ratings are applied in relation to the maximum stand-up time.
[d] AEF is the adjacent excavation factor, defined as the ratio between the distance to an adjacent excavation, in meters, from the main excavation under design, and the span of that adjacent excavation, in meters.
[e] PF is the portal factor, defined as the ratio between the thickness of overburden and the span of the excavation, both in meters.

One of the best documented case histories available to the author is the Elandsberg Pumped Storage Scheme (Bieniawski, 1976; 1979). The role that rock mass classifications played in this project is described below.

Examination of rock conditions at Elandsberg by means of the Geomechanics Classifications revealed that the 22-m span needed for the 1000-MW underground power station fell outside the limits of accumulated experience (from the relevant case studies), even if the rock masses at Elandsberg were "good" to "very good" (Classes I and II, respectively). As the classification estimates (see Fig. 4.1) revealed "fair rock" (Class III) at best, only a full-sized trial test enlargement having a span of 22 m could reliably establish the feasibility of construction and the most suitable means of excavating and stabilizing such a large span.

7.4.1 Site Investigations

All the tests were conducted in the exploratory tunnels and enlargements. The rock strata within the site area consisted of vertically bedded graywacke which included minor amounts of phyllite. The geological conditions at the site were thoroughly explored both by over 1500 m of underground diamond drilling and long boreholes, diamond drilled from the surface, giving nearly 5000 m of core. Furthermore, detailed geological mapping and airphoto interpretation were also carried out. Groundwater conditions were assessed by a network of piezometers and by water pressure testing in boreholes. The graywacke rock was of good quality (RQD = 75–85%), while the phyllite was of fair quality (RQD = 65–75%). Apart from the vertical bedding foliation that represented the main jointing feature, three further joint sets were identified as well as minor faulting. Water inflows of between 70 and 250 L/min were recorded. The area is earthquake-prone, with earthquakes between 5.0 and 6.3 on the Richter scale registered recently. The Geomechanics Classification was used to assess the overall rock mass conditions. The graywacke rock mass was predominantly Class II (good rock), having an RMR = 66 to 87 (av: 75). The phyllite rock mass was of Class III (fair rock), with RMR = 43 to 60 (av: 57).

For cross-checking purposes, the graywacke rock mass was also classified using the RSR concept and the Q-system. It was found that the RSR = 62 (range: 60 to 68), whereas $Q = 30$ (range: 18 to 35).

During the investigations, the results of all the in-situ deformability tests were analyzed with reference to the Geomechanics Classification rock mass rating of the localities where the tests were conducted. The results are depicted in Figure 7.4.

Based on over 100 results from 37 in-situ tests, the following correlation was obtained:

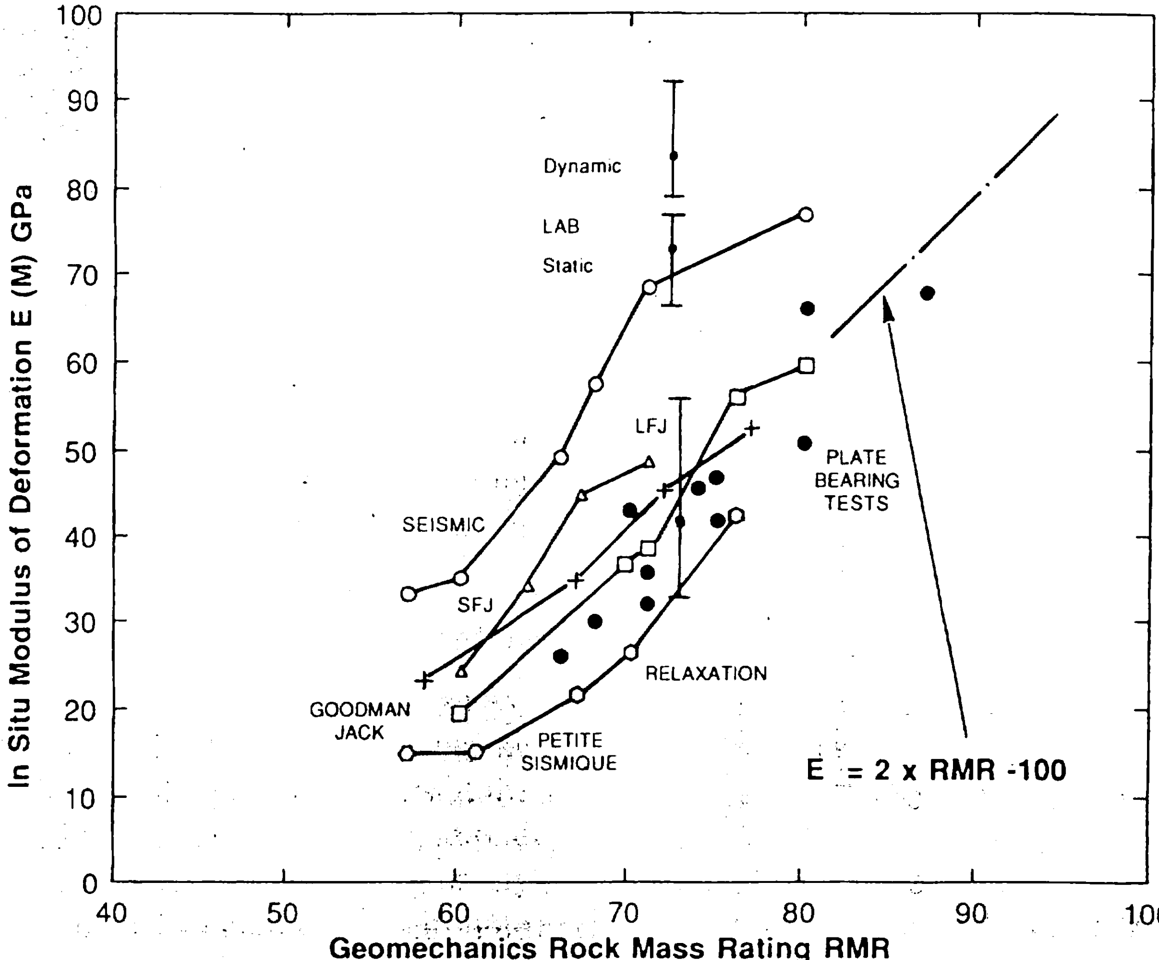


Figure 7.4 Experimental data relating RMR to in-situ modulus of deformation in the Elandsberg project. (After Bieniawski, 1979.)

$$E_M = 1.8\ \mathrm{RMR} - 88.4 \tag{7.1}$$

with a correlation coefficient of 0.8787 and a prediction error of 15.9%, which was defined as the difference between the observed value and the predicted value expressed as a percentage of the predicted value.

In view of the high correlation, the coefficients in the above equation were rounded off since the aim was to estimate the in-situ modulus for a preliminary assessment of rock mass deformability. This resulted in the following equation:

$$E_M = 2\ \mathrm{RMR} - 100 \tag{7.2}$$

This simple equation has a prediction error of 18.2%, which is sufficient for practical engineering purposes.

7.5 MAXIMUM SPANS AND SAFETY FACTORS FOR UNSUPPORTED EXCAVATIONS

Barton et al. (1980) discussed applications of the Q-system to estimating optimal cavern dimensions. An interesting aspect of the Q-system is its ability to recognize rock mass characteristics required for safe operation of permanently unsupported openings. A detailed analysis of all the available case records of unsupported exacavations revealed the following requirements:

General Requirements for Permanently Unsupported Openings

1. $J_n < 9$, $J_r > 1.0$, $J_w = 1.0$, SRF < 2.5.

Conditional Requirements

2. If RQD < 40, should have $J_n \leqslant 2$.
3. If $J_n = 9$, should have $J_r > 1.5$ and RQD > 90.
4. If $J_r = 1$, should have $J_n < 4$.
5. If SRF > 1, should have $J_r > 1.5$.
6. If span > 10 m, should have $J_n > 9$.
7. If span > 20 m, should have $J_n < 4$ and SRF < 1

Existing natural and man-made openings indicate that very large unsupported spans can be safely built and utilized if the rock mass is of sufficiently high quality. The case records that describe unsupported man-made excavations have spans ranging from 1.2 to 100 m. If there are only a limited number of discontinuous joints and the rock mass quality Q is up to 500 to 1000, the maximum unsupported span may only be limited by the ratio of rock stress/rock strength (Barton et al., 1980).

All the available case records of unsupported spans are plotted in Figure 7.5. The tentative curved envelope is the assumed maximum design span for man-made openings based on these available cases. The five square data points plotting above this curve were obtained from the huge natural openings of the Carlsbad limestone caverns in New Mexico. If the data for man-made and natural openings are combined, it is seen that the limiting envelope is approximately linear and can be represented by the following simple equation:

$$\text{Span} = 2Q^{0.66} \tag{7.3}$$

For design purposes, the suggested maximum design spans for different types of excavations are based on the curved envelope.

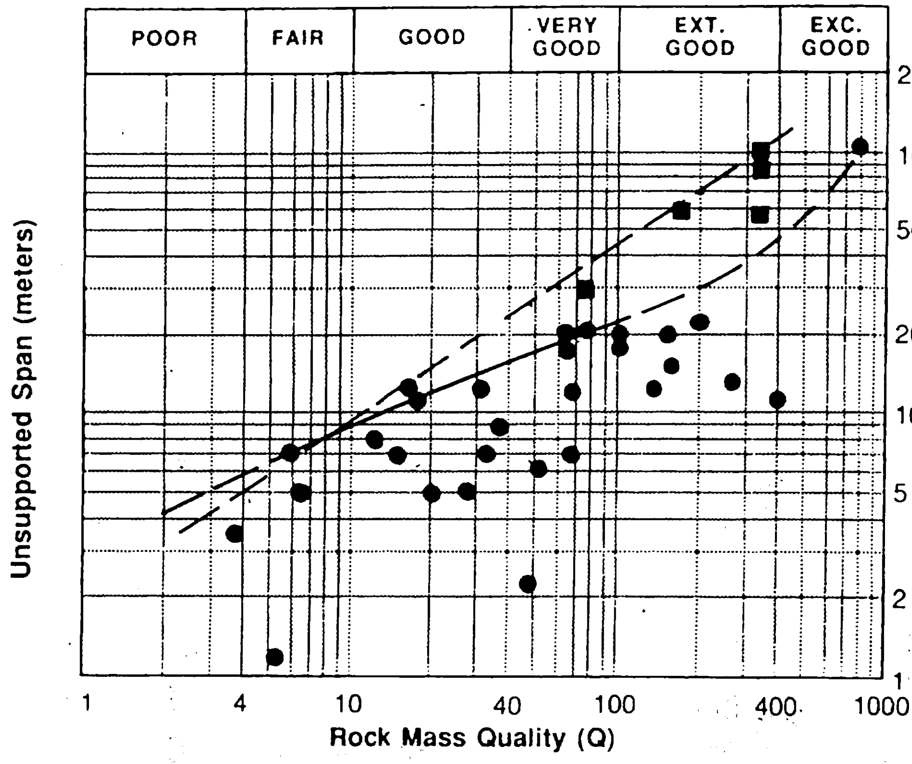


Figure 7.5 *Excavation span versus rock mass quality Q. Circles represent the man-made unsupported excavations reported in the literature. Squares represent natural openings from Carlsbad Caves, New Mexico. The curved envelope is an estimate of the maximum design span for permanently unsupported man-made openings. (After Barton et al., 1980).*

7.5.1 Estimating Support Requirements

To test the above correlation, nine locations were selected in and around a power station under construction (Barton et al., 1980). The roof arch was shotcreted at that time, though some 3–6 m of the walls were excavated and parts were not shotcreted. Both end walls were bare. Other unsupported locations were selected in the immediate vicinity of the powerhouse in an attempt to predict conditions likely to be encountered when the cavern height was increased to the maximum 31 m.

The six classification parameters of the Q-system were estimated and fell into three groups:

	RQD/J_n	J_r/J_a	J_w/SRF	Q
Best zones	98/4.3	1.7/1.0	1/1	39
Poorer zones	72/7	1.9/1.8	1/1	11
Worst zones	40/9	2/6	1/2.5	0.6

It was estimated that more than 90% of the excavated rock in the powerhouse (including roof and walls) would be of "best" quality, less than 10% of "poorer" quality, and probably only 1 or 2% of "worst" quality.

The mean ratings for the majority of the rock mass (best, $Q = 39$) were translated into the following descriptions:

1. RQD = 98 (excellent).
2. $J_n = 4.3$ (approx two joint sets).
3. $J_r = 1.7$ (rough-planar to smooth-undulating).
4. $J_a = 1.0$ (unaltered joints, surface staining).
5. $J_w = 1.0$ (dry excavations).
6. SRF = 1.0 (medium stress, no rock bursting).

The support recommendations based on the Q-system were as follows:

Best conditions:	ca 90%	$Q = 39$	Roof:	B 1.7 m center-to-center + clm
			Walls:	sb
Poorer conditions:	ca 10%	$Q = 11$	Roof:	B 1.5 m c/c + S(mr) 7 cm
			Walls:	B 1.6 m c/c + clm
Worst conditions:	1–2%	$Q = 0.6$	Roof:	B 1.0 m c/c + S(mr) 15 cm
			Walls:	B 1.2 m c/c + S(mr) 12 cm

where B = systematic bolting with given c/c spacing,
sb = spot bolts,
S(mr) = mesh-reinforced shotcrete,
clm = chain link mesh or steel bands.

The above recommendations for support, especially those for the majority of the rock mass ($Q = 39$), will obviously appear grossly inadequate in countries where a concrete lining has been a common feature of final tunnel support. However, it should be noted that the support recommendations obtained from the Q-system were based on the analysis of about 200 case records, 79 of them in the powerhouse category.

In Figure 7.5, it will be seen that $Q = 39$ (best) and the span of 19 m lie some 3–4 m above the maximum design span for permanently unsupported openings. Barton et al. (1980) observed that the recommended systematic bolting (spacing 1.7 m) and the steel banding (a single layer of shotcrete might be preferred for aesthetic reasons) seemed to be overdesign, considering that the joint spacing was 1–2 m and the existing joints relatively discontinuous. In addition, the mean ratings of the six rock mass parameters for the best-quality ($Q = 39$) rock satisfied all the conditional factors apparently needed for an excavation to be left permanently unsupported.

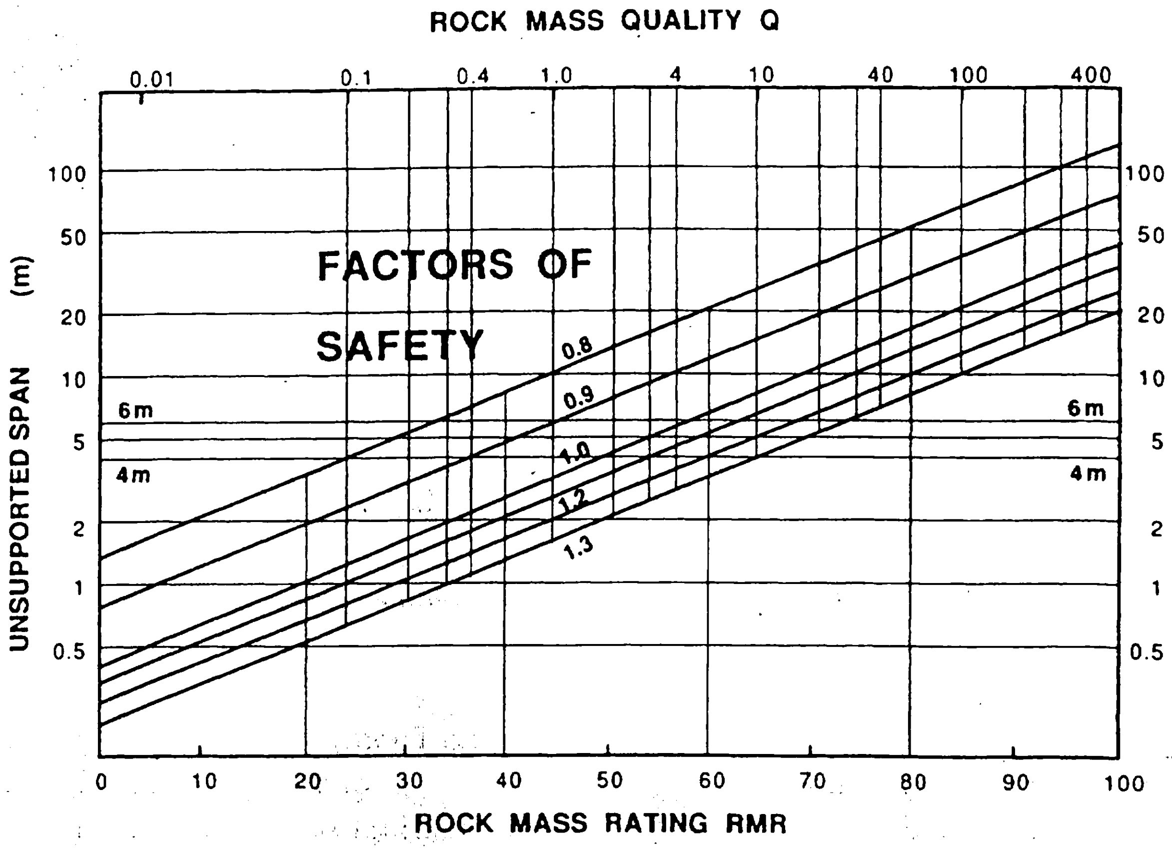


Figure 7.6 *Estimated factors of safety for unsupported underground excavations as a function of excavation span and rock mass quality. (Rearranged after Houghton and Stacey, 1980).*

7.5.2 Assessing Stability of Unsupported Excavations

Houghton and Stacey (1980) suggested a quantitative assessment, based on rock mass classification, for the factor of safety of unsupported excavations. This is depicted in Figure 7.6. They noted that due to different purposes of excavations, for civil engineering applications, factors of safety greater than 1.2 will be required when considering omission of support.

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8

Applications in Mining

It is not the things you don't know that get you into trouble.
It is the things you think you know for sure.
—Casimir Pulaski

Mining case histories featuring applications of rock mass classifications demonstrate their potential in the design of deep underground excavations, and hence the effects of high in-situ stresses. This is particularly true of hard-rock metal mining, which is generally performed at greater depth than coal mining. Nevertheless, coal mining applications are also informative due to the changing stress conditions imposed by abutment loadings such as experienced in longwall mining.

Significant contributions to mining applications of rock mass classifications were made by Laubscher (1977, 1984) and Cummings et al. (1982) for hard-rock mining, and by Unal (1983) and Venkateswarlu (1986) for coal mining. Other valuable work was performed by Brook and Dharmaratne (1985), Newman (1985), and Sandbak (1988).

8.1 HARD ROCK MINING: AFRICA

Laubscher (1977, 1984) modified the Geomechanics Classification developed by Bieniawski (1976, 1979) for mining applications involving asbestos mines in southern Africa. This modification featured a series of adjustments for

TABLE 8.1 Geomechanics Classification in Hard-rock Mining Applications: Basic Rock Mass Ratings[a]

Class	1 A	1 B	2 A	2 B	3 A	3 B	4 A	4 B	5 A	5 B
Rating	100–81		80–61		60–41		40–21		20–0	
Description	Very good		Good		Fair		Poor		Very poor	

	Parameter	Range of Values										
1	RQD	100–97		96–84	83–71	70–56	55–44	43–31	30–17	16–4	3–0	
	Rating (= RQD × 15/100)	15		14	12	10	8	6	4	2	0	
2	UCS[b] (MPa)	185	184–165	164–145	144–125	124–105	104–85	84–65	64–45	44–25	24–5	4–0
	Rating	20	18	16	14	12	10	8	6	4	2	0
3	Joint spacing	Refer to										
	Rating	Table 8.2										
4	Joint condition, including groundwater	Refer to Table 8.3										
	Rating	40 ——— 0										

[a] After Laubscher (1977).

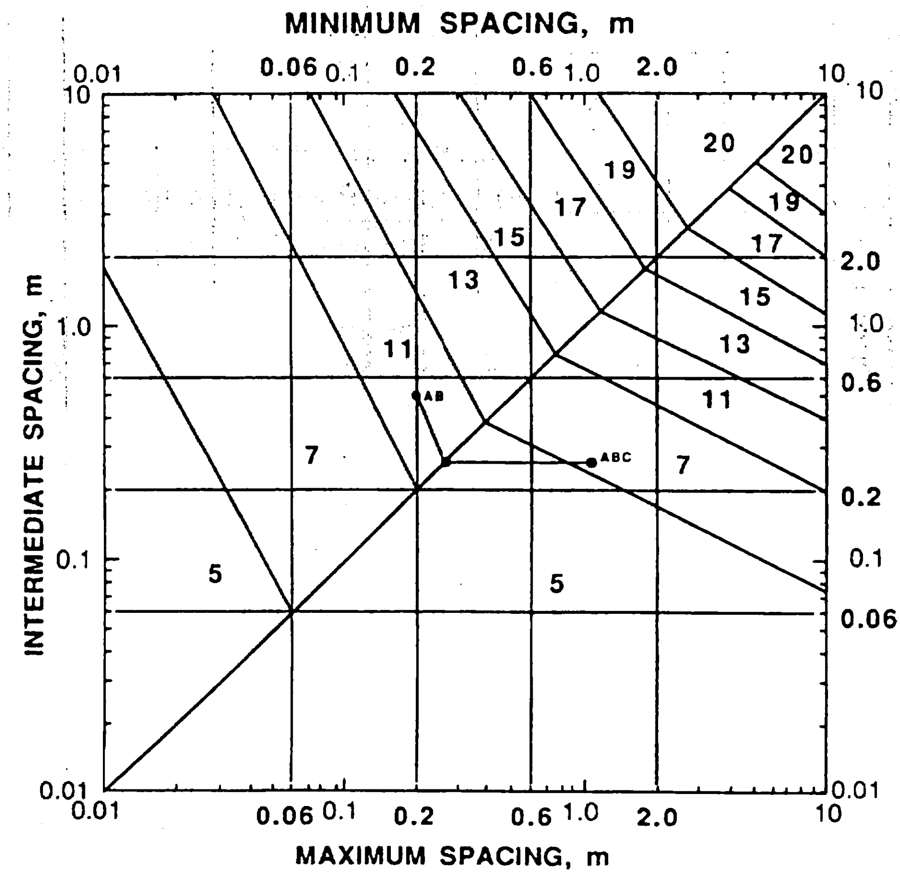
[b] Uniaxial compressive strength.

RMR values to accommodate the effects of the original (virgin) and induced stresses, changes in stress, as well as the effects of blasting and weathering. Full details are apparent from Tables 8.1–8.5:

Table 8.1: Basic rock mass ratings
Table 8.2: Ratings for multijoint systems
Table 8.3: Adjustments for joint condition and groundwater
Table 8.4: Total possible adjustments
Table 8.5: Support guide for mine drifts

Brook and Dharmaratne (1985) proposed further simplifications of Laubscher's modifications to the RMR system on the basis of mining and tunneling case histories in Sri Lanka. However, they found the log–log graph for joint spacing rating of multijoint systems was confusing and advocated a simpler representation; the graph, in Table 8.2, although again modified, would benefit from further improvements.

TABLE 8.2 Geomechanics Classification for Hard-Rock Mining Applications: Ratings for Multijoint Systems[a]



[a]Modified after Laubscher (1981) and Brook and Dharmaratne (1985).
[b]Example: joint spacing A = 0.2 m, B = 0.5 m, and C = 1.0 m; rating A = 15, AB = 11, and ABC = 7.

TABLE 8.3 Geomechanics Classification in Hard-Rock Mining Applications: Adjustments for Joint Condition and Groundwater[a]

Parameter	Description		Dry Condition	Wet Conditions		
				Moist	Moderate Pressure 25–125 l/min	Severe Pressure >125 l/min
A Joint expression (large-scale irregularities)	Wavy	Multidirectional	100	100	95	90
		Unidirectional	95 90	95 90	90 85	80 75
	Curved		89 80	85 75	80 70	70 60
	Straight		79 70	74 65	60	40
B Joint expression (small-scale irregularities or roughness)	Very rough		100	100	95	90
	Striated or rough		99 85	99 85	80	70
	Smooth		84 60	80 55	60	50
	Polished		59 50	50 40	30	20

C Joint-wall alteration zone	Stronger than wall rock		100	100	100	100
	No alteration		100	100	100	100
	Weaker than wall rock		75	70	65	60
D Joint filling	No fill—surface staining only		100	100	100	100
	Nonsoftening and sheared material (clay- or talc-free)	Coarse sheared	95	90	70	50
		Medium sheared	90	85	65	45
		Fine sheared	85	80	60	40
	Soft sheared material (e.g., talc)	Coarse sheared	70	65	40	20
		Medium sheared	65	60	35	15
		Fine sheared	60	55	30	10
	Gouge thickness < amplitude of irregularity		40	30	10	
	Gouge thickness > amplitude of irregularity		20	10	Flowing material 5	

[a]After Laubscher (1977).

TABLE 8.4 Geomechanics Classification in Hard-Rock Mining Applications: Total Possible Adjustments (in Percentages)[a]

Parameter	RQD	IRS[b]	Joint Spacing	Condition of Joints	Total
Weathering	95	96		82	75
Virgin and induced stresses				120–76	120–76
Changes in stress				120–60	120–60
Strike and dip orientation			70		70
Blasting	93			86	80

[a] After Laubscher (1977).
[b] IRS = intact rock strength.

TABLE 8.5 Geomechanics Classification in Hard-Rock Mining Applications: Support Guide for Mine Drifts[a,b]

Adjusted Classes	In-Situ Classes									
	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
1 and 2										
3A		a	a	a	a					
3B			b	b	b	b				
4A				c, d	c, d	c, d, e	d, e			
4B					g	f, g	f, g, j	h, f, j		
5A						i	i	h, i, j	h, j	
5B							k	k	l	l

[a] After Laubscher (1977).
[b] Key: a = generally no support but locally joint intersections might require bolting;
b = patterned grouted bolts at 1-m collar spacing;
c = patterned grouted bolts at 0.75-m collar spacing;
d = patterned grouted bolts at 1-m collar spacing and shotcrete 50 mm thick;
e = patterned grouted bolts at 1-m collar spacing and massive concrete 300 mm thick and only used if stress changes not excessive;
f = patterned grouted bolts at 0.75-m collar spacing and shotcrete 100 mm thick;
g = patterned grouted bolts at 0.75-m collar spacing with mesh-reinforced shotcrete 100 mm thick;
h = massive concrete 450 mm thick with patterned grouted bolts at 1-m spacing if stress changes are not excessive;
i = grouted bolts at 0.75-m collar spacing if reinforcing potential is present, and 100-mm reinforced shotcrete, and then yielding steel arches as a repair technique if stress changes are excessive;
j = stabilize with rope cover support and massive concrete 450 mm thick is stress changes not excessive;
k = stabilize with rope cover support followed by shotcrete to and including face if necessary, and then closely spaced yielding arches as a repair technique where stress changes are excessive;
l = avoid development in this ground, otherwise use support systems "j" or "k."

8.2 HARD ROCK MINING: USA

Cummings et al. (1982) and Kendorski et al. (1983) also modified the Geomechanics Classification (Bieniawski, 1979) for mining applications in U.S. block caving copper mines.

The MBR (modified basic RMR) system, depicted in Figure 8.1, uses the basic RMR approach of Bieniawski (1979) with some of the concepts of Laubscher (1977). Key differences lie in the arrangement of the initial rating terms and in the adjustment sequence. In the MBR system, the inputs are selected and arranged so that a rational rating is still possible using very preliminary geotechnical information from drill holes. The MBR is also a multistage adjustment; the output at each stage can be related to support for various mining conditions. The MBR rating is the result of the initial stage and is the simple sum of the element ratings.

The MBR is an indicator of rock mass competence, without regard to the type of opening constructed in it. This MBR value is used in the same fashion as the RMR for determining support requirements by consulting support charts or tables. The MBR recommendations are for isolated single tunnels that are not in areas geologically different from production areas.

The second stage is the assignment of numerical adjustments to the MBR that adapt it to the ore block development process. With regard to support, the principal differences between production drifts and civil tunnels (in development only) are the excavation techniques and the need for multiple, parallel openings. Unfavorable fracture orientation may also strongly influence stability. Input parameters relate to excavation (blasting) practice, geometry (vicinity, size, and orientation of openings), depth, and fracturing orientation.

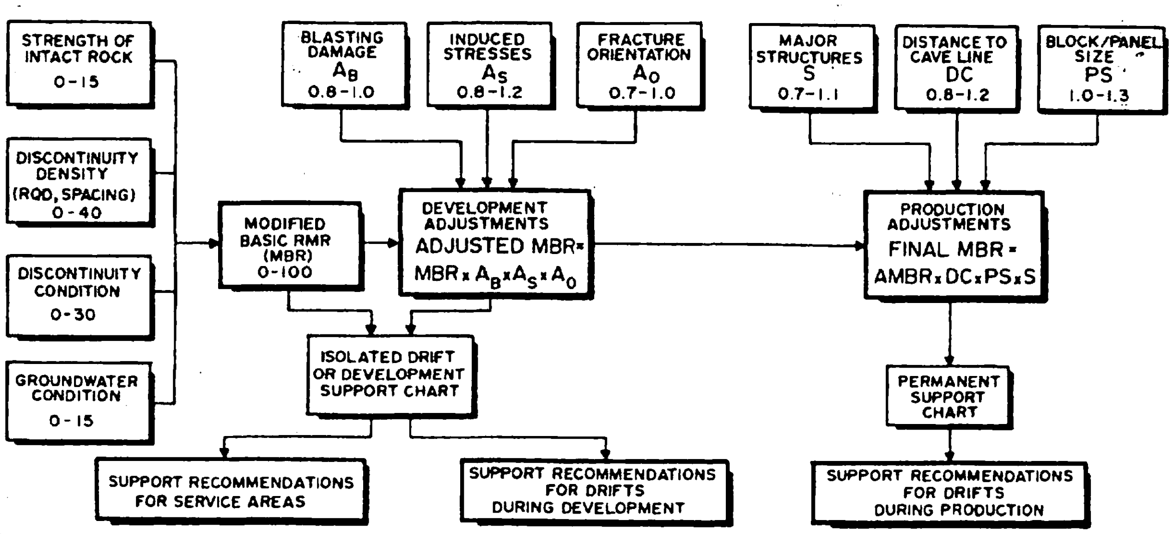


Figure 8.1 *The overall structure of the MBR system. (After Cummings et al., 1982.)*

The adjustment values are obtained from tables and charts, and the MBR is multiplied by the decimal adjustment to obtain the adjusted MBR. Drift support charts are consulted to give a range of supports for drift development (initial support). The user may select support according to the performance period desired, since lighter support will be adequate in some rock for short periods. The objective is to stabilize initially the opening during development so that the permanent support may use its full capacity to resist the abutment loading increment.

The third and last classification stage deals with the additional deformations due to abutment loadings. As stated before, caving deformations will also be accounted for if proper undercutting and draw control practices are followed. The most significant identified factors influencing abutment load are the location and orientation of the drift with respect to the caved volume, the size of the caved volume, the ability of the rock mass to withstand stress, the tendency of the lining to attract stress, and the role of any major structural trends that may serve to localize or transfer the abutment deformations. Input variables relate to block or panel size, undercutting sequence, level layout, MBR, and general structural geology of the area. The adjustment values, obtained from tables and graphs, are used as multipliers to the adjusted MBR and result in the final MBR. This value, together with an assessment of repair acceptability (depending on the type of opening) is correlated with recommendations for permanent support at intersections and in drift sections.

8.2.1 Approach

The first step in using the MBR system is the collection of representative data on geology and mining alternatives. Data sheets, such as those in Figures 8.2 and 8.3, are helpful in organizing these data.

Once the basic data have been assembled, the analysis proceeds according to the flow chart presented in Figure 8.1. Ratings are applied to the intact rock strength, discontinuity density and condition, and groundwater conditions.

Intact rock strength is rated according to Figure 8.4. The shaded region permits adjustment of ratings to allow for a natural sampling and testing bias.

The discontinuity density, which is related to blockiness and is the sum of ratings for RQD and discontinuity spacing, is depicted in Figure 8.5. If either type of data is lacking, it can be estimated through the use of Figure 8.6.

Table 8.6 is used for rating the discontinuity condition. The most representative conditions are assessed for this step. The degree or type of alteration can be a useful index for this as well.

MBR Input Data Sheet: Geological Data

Project Name ____________ Site of Survey ____________ By ____________ Date ____________

1. Geologic Region: ____________ Rock Type ____________ Location ____________

2. Compressive Strength: Average ____________ Range ____________ Method ____________ Comment ____________

3. Core Recovery: Interval ____________ Average ____________ Range ____________

4. RQD: Interval ____________ Average ____________ Range ____________

5. Discontinuity Spacing: Average ____________ Range ____________ Comment ____________

6. Discontinuity Condition

	Wall Roughness	Wall Separation	Joint Filling	Wall Weathering
Most Common				
Intermediate				
Least Common				
Consensus				

7. Water Condition Dry Damp Wet Dripping Flowing

8. Fracture Orientations

	Set 1	Set 2	Set 3	Set 4	Set 5
Strike					
Dip/Dir					
Rank					

9. Major Structures

	Strike	Dip	Dip Dir.	Width	Location/Comment
Name:					Location/Comment
Name:					
Name:					

10. Stress Field σ_1: Direction ____________ Magnitude ____________ Measured? ____________
σ_3: Direction ____________ Magnitude ____________ Measured? ____________

11. Source of Geological Data ____________

Figure 8.2 *Input form: geological data. (After Cummings et al., 1982.)*

MBR Input Data Sheet: Engineering Data

Project Name ______________ Site of Survey ______________ By __________ Date __________

1. Type of Drift(s) __________ 2. Orientation(s) __________ 3. Design Life __________

4. Design Dimensions Width __________ Width variation __________
   Height __________ Height variation __________

5. Drift Spacing (Horizontal) __________
   Other Openings Type __________ Size __________ Spacing __________

6. Extraction Ratio
   Multiple Openings: Excavated Area __________ Unexcavated __________ e_r __________
   Single Opening: 1.5 (width) __________ Excavated __________ Unexcavated __________ e_r __________

7. Distance below undercut - drift floor to undercut floor __________
   drift crown to undercut floor __________

8. Method of Excavation: Machine bored Controlled D & B Conventional D & B

9. Excavation conditions:
   Perimeter Hole Traces __________
   Rib or Crown Looseness __________
   New or Existing Cracks __________
   Overbreak & Barring-Down __________
   Other Criteria __________

10. Intersections, turnouts: Type __________ Location __________ Max. Span __________

11. Block Dimensions: Side __________ Orientation __________ End __________ Orientation __________

12. Cave Line Direction __________ Direction of Progress __________

13. Drift Location (in block, with respect to major structures and their dips, with respect to cave)
____
____

Figure 8.3 *Input form: engineering data. (After Cummings et al., 1982.)*

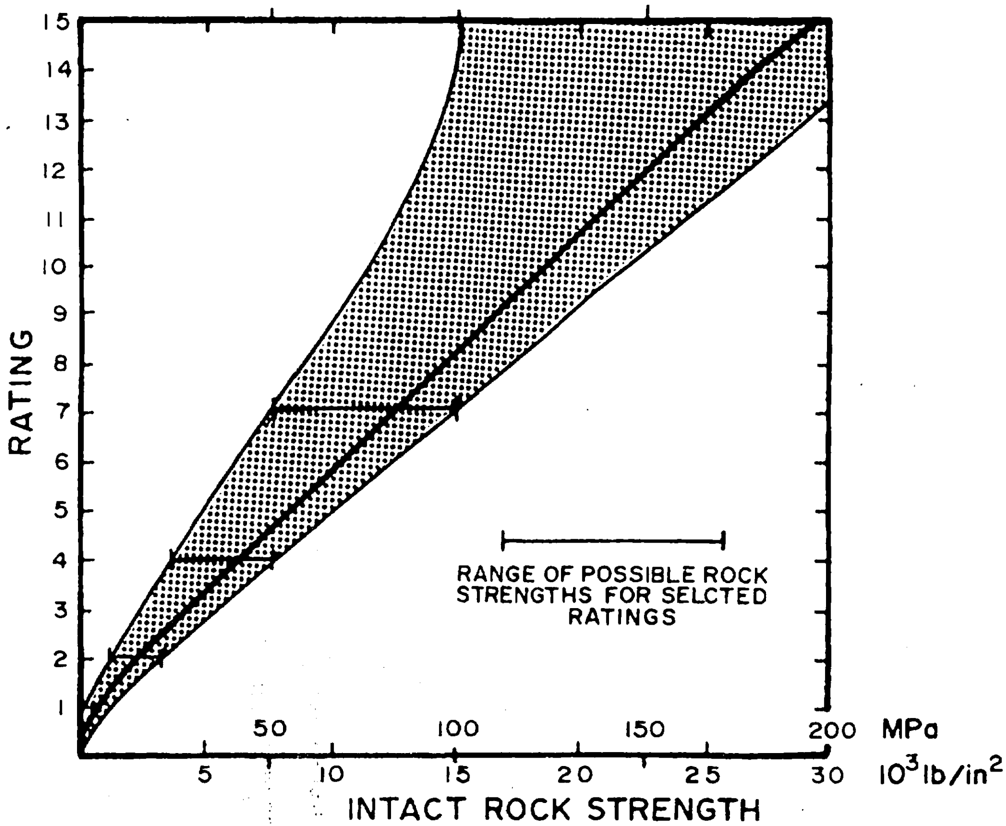


Figure 8.4 *Ratings for intact rock strength: MBR system. (After Cummings et al., 1982.)*

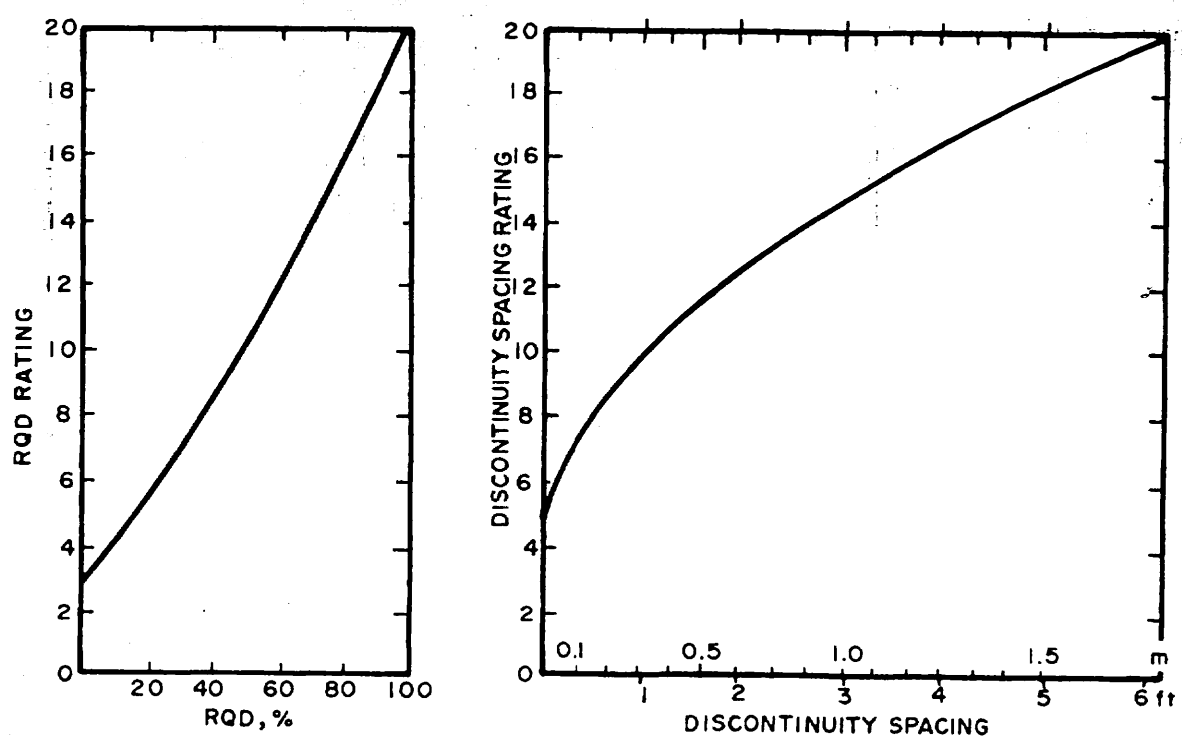


Figure 8.5 *Ratings for discontinuity density: MBR system. (After Cummings et al., 1982.)*

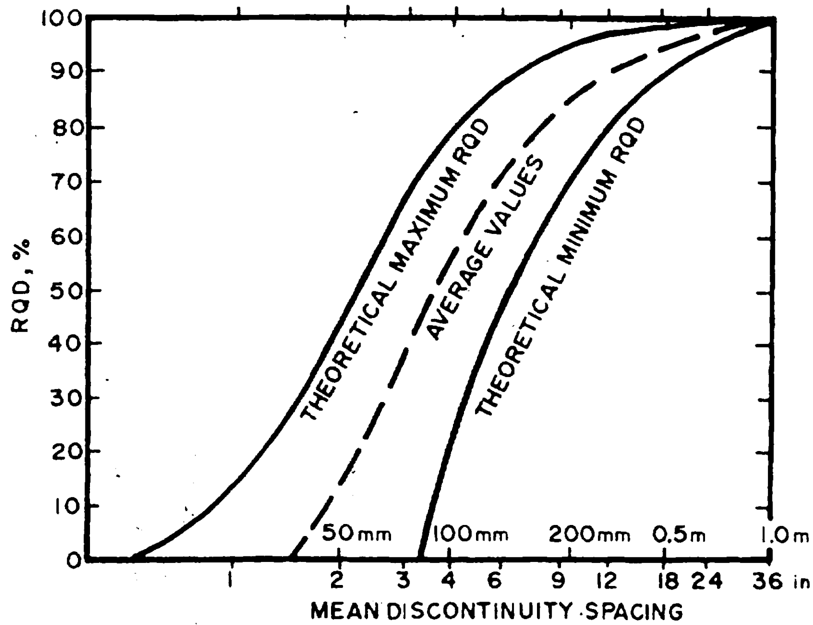


Figure 8.6 Theoretical relationship between RQD and discontinuity spacing. (After Priest and Hudson, 1976.)

The groundwater condition rating is determined from Table 8.7.

To obtain the MBR, the four ratings mentioned above are summed. The ranges of the input parameters are given in Figure 8.1. At this point, the development support chart, given in Figure 8.7, provides support for service areas away from production areas.

Having thus obtained the MBR and the applicable recommendations, the adjusted MBR is computed for development adjustments as follows.

TABLE 8.6 MBR System: Discontinuity Condition Ratings[a]

Description of Discontinuity					
Wall roughness[b]	VR	R–SR	SR	SM–SK	SM
Wall separation	None	Hairline	<1 mm	1–5 mm	>5 mm
Joint filling	None	None	Minor clay	Stiff clay, gouge	Soft clay, gouge
Wall weathering[c]	F	SL	SO	SO	VS
Rating	30	25	20	10	0

[a] After Cummings et al. (1982).

[b] Roughness: VR = very rough (coarse sandpaper),
R = rough (medium or fine sandpaper),
SR = smooth to slightly rough,
SM = smooth but not polished,
SK = slickensided, shiny.

[c] Weathering (alteration): F = fresh, unweathered, hard;
SL = slightly weathered, hard;
SO = softened, strongly weathered;
VS = very soft or decomposed.

TABLE 8.7 MBR System: Groundwater Condition Rating[a]

Condition	Rating
Dry	15
Dump	10
Wet	7
Dripping	4
Flowing	0

[a] After Cummings et al. (1982).

Firstly, the extraction ratio is computed for the mining layouts under study. For single drifts with multiple intersections or those that are otherwise affected by other openings, the extraction ratio may depend on the extent of the area considered. Only in such instances is the convention adopted that all openings within 1.5 drift diameters of each rib are considered in computing the extraction ratio. The ratio is computed at springline and therefore includes the horizontal planimetric area of the finger or transfer raises.

Blasting damage is next assessed according to the criteria of Table 8.8. Both the blasting damage adjustment A_b and the descriptive term (moderate, slight, severe, none) should be noted.

The induced stress adjustment A_s is then determined. The horizontal (σ_h) and vertical (σ_v) components of the stress field must be computed or estimated, and the adjustment A_s can then be read from Figure 8.8 for the appropriate effective extraction ratio, depth, and stress state. The extraction ratio is the area of rock, after development, being effective in carrying the load.

Next, the adjustment for fracture orientation A_o is computed. If drift exposures are available, Table 8.9 (top) is used. If no drift exposures exist but fracturing trends are known, Table 8.9 (bottom) can be used. The basis of Table 8.9 is that fractures perpendicular to the axis of the opening are more favorable than fractures parallel to it; that both development and support are facilitated by fractures that dip away from the heading rather than toward it; and that steep dips are preferable to shallow dips. If fracturing trends are not known but core is available for examination, fully interlocking core can be examined for the number of groups of discontinuities of similar inclinations in the core.

The three adjustments, A_s, A_b, A_o, are multiplied, yielding for most situations a decimal value between 0.45 and 1.0. The MBR is multiplied by this value or by 0.5, whichever is greater, to yield the adjusted MBR.

The development support chart in Figure 8.7 is then again consulted for support recommendations. It should be decided what degree of support

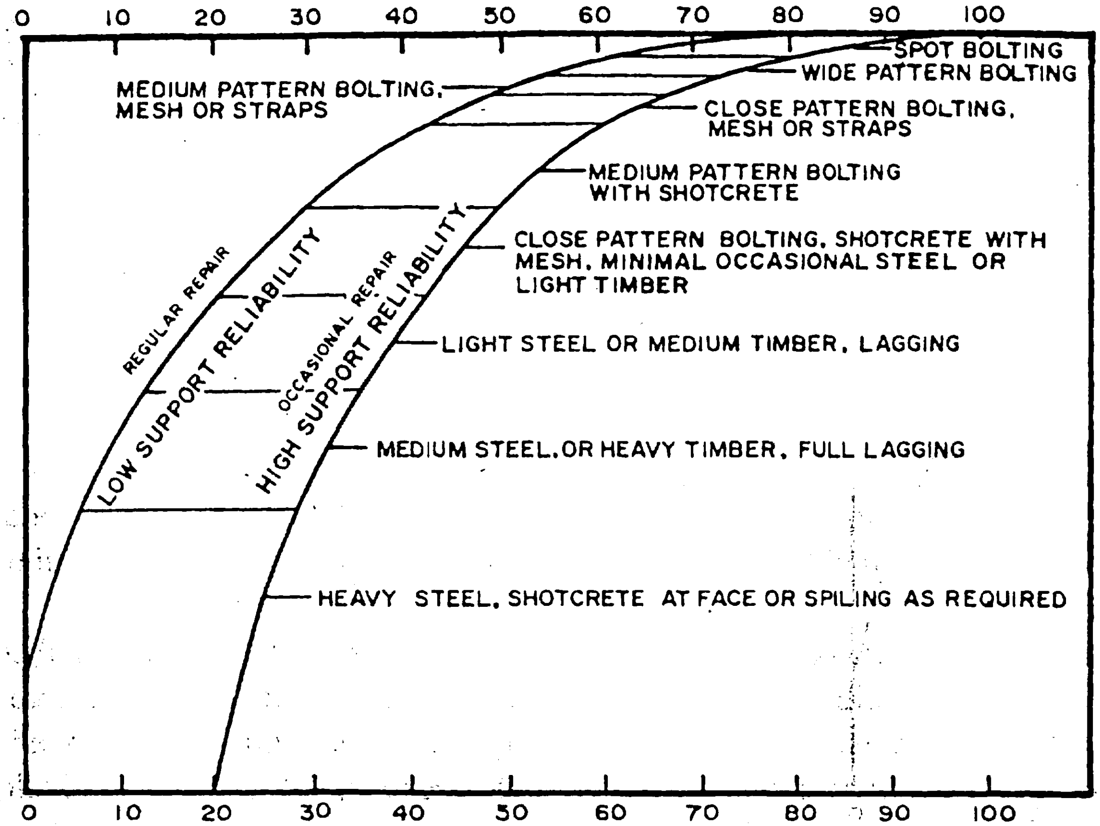


Figure 8.7 *MBR support chart for isolated or development drifts. (After Cummings et al., 1982.)*

Explanations of the Support Types

Spot Bolting: *Bolting to restrain limited areas or individual blocks of loose rock, primarily for safety.*

Wide Pattern Bolting: *Bolts spaced 1.5–1.8 m, or wider in very large openings.*

Medium Pattern Bolting, with or without Mesh or Straps: *Bolts spaced 0.9–1.5 m, 23-cm wide straps or 100-mm welded wire mesh.*

Close Pattern Bolting, Mesh, or Straps: *Bolt spacing less than 0.9 m, 100-mm welded wire mesh, 0.3-m straps, or chain link.*

Medium pattern bolting with shotcrete: *Bolts spaced 0.9–1.5 m and 80 mm (nominal) of shotcrete. Light mesh for wet rock to alleviate shotcrete adherence problems.*

Close Pattern Bolting, Shotcrete with Mesh, Minimal Occasional Steel, or Light Timber: *Bolt spacing less than 0.9 m with 100-mm welded wire mesh or chain link throughout, and nominal 100-mm of shotcrete. Localized conditions may require light wide-flange steel sets or timber sets.*

Light Steel, Medium Timber, Lagging: *Bolting as required for safety at the face—full contact (grouted or split set) bolts only. Light wide-flange steel sets or 0.25-m timber sets spaced 1.5 m, with full crown lagging and rib lagging in squeezing areas.*

Medium Steel, Heavy Timber, Full Lagging: *Medium wide-flange steel sets or 0.3-m timber sets spaced 1.5 m, fully lagged across the crown and ribs. Support to be installed as close to the face as possible.*

Heavy Steel, Shotcrete at Face or Spilling as Required: *Heavy wide-flange steel sets spaced 1.2 m, fully lagged on crown and ribs, carried directly to face. Spilling or shotcreting of face as necessary.*

General: *Bolting: bolts in spot bolting through close pattern bolting are considered to be 19 mm in diameter, fully grouted or resin-anchored standard rockbolts; mechanical anchors are acceptable in material of MBR > 60. Spilt-set use is at the discretion of the operator.*

TABLE 8.8 MBR System: Blasting Damage Adjustment A_b

Conditions/Method	Applicable Term	Adjustment A_b
1. Machine boring	No damage	1.0
2. Controlled blasting	Slight damage	0.94–0.97
3. Good conventional blasting	Moderate damage	0.90–0.94
4. Poor conventional blasting	Severe damage	0.90–0.80[a]
5. No experience in this rock	Moderate damage	0.90[b]

[a] Worst: 0.80.
[b] Nominal.

reliability is desired for development. It is recommended that the development support be selected so as to stabilize the opening for as long as it will take to bring the block into production.

Next, the final MBR is computed. In this third and last stage, the role of abutment loadings is accounted for. This is addressed through considerations of structural geology and mining geometry (production adjustments).

Faulted and shattered zones disrupt the mining-induced stress pattern and are dealt with through the adjustment for orientation of major structures, S (use Table 8.9). Although any zone of significantly less competence is eligible for adjustment, it is suggested that only the larger, nearby features are worthy of consideration. The limiting width–distance relationship will become clear for each mining property. Where information is too sparse or preliminary, it may be possible to characterize blocks of ground according to an expected or typical distribution of weakness zones.

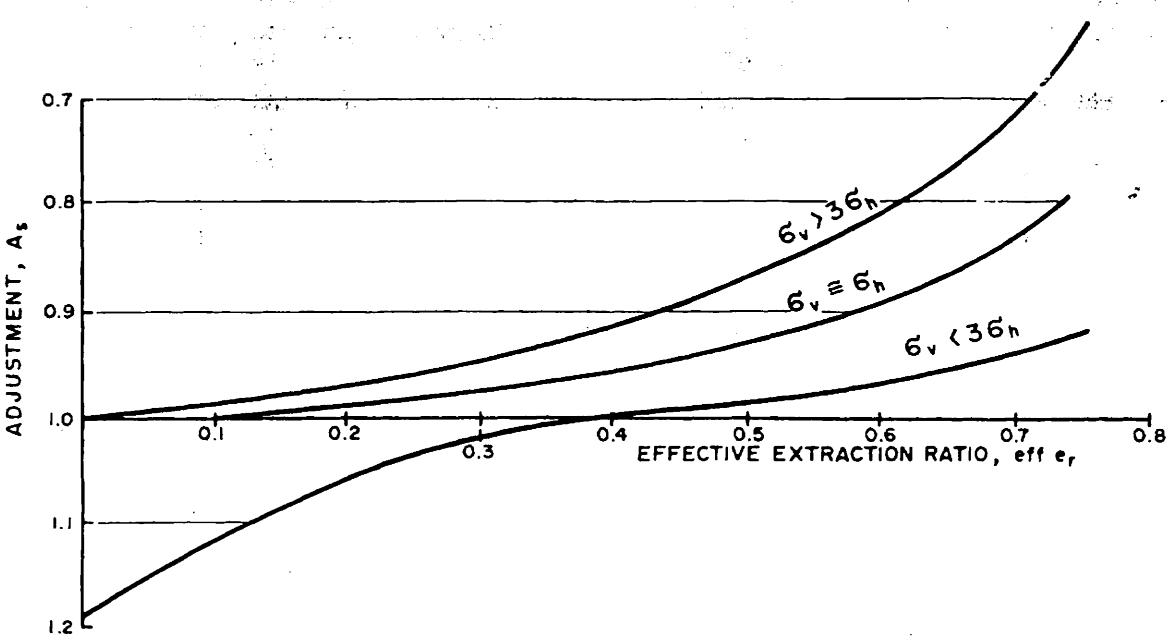


Figure 8.8 *Adjustment A_s for induced stresses due to multiple openings. (After Cummings et al., 1982.)*

TABLE 8.9 MBR System: Fracture Orientation Rating A_o[a]

Direct Observation in Drift

No. of Fractures Defining Block	No. of Nonvertical Faces: 1	2	3	4	5	6
3		0.95	0.80			
4		0.95	0.85	0.80		
5	1.0	0.95	0.90	0.85	0.80	
6	1.0	1.0	0.95	0.90	0.85	0.80

Indirect Observation of Fracture Statistics

Strike	Perpendicular						
Heading Direction	With Dip		Against Dip		Parallel		Flat Dip
Dip amount	45–90	20–45	45–90	20–45	45–90	20–45	0–20
Adjustment	1.0	0.95	0.90	0.85	0.80	0.90	0.85

[a] After Cummings et al. (1982).

The adjustment for the proximity to the cave line *DC* is computed from Figure 8.9. This rating refers to the point of closest approach of the cave area. In some cases, this means the vertical distance, and in others, the horizontal. The term reflects the dissipation of abutment load away from the point of application.

The block or panel size adjustment *PS* (see Fig. 8.10) reflects the relationship between magnitude of abutment stress and size of caved volume. Smaller panel or block sizes are associated with lower abutment load levels because the caved volume is smaller. Blocks larger than 60 m or so, as well as level-

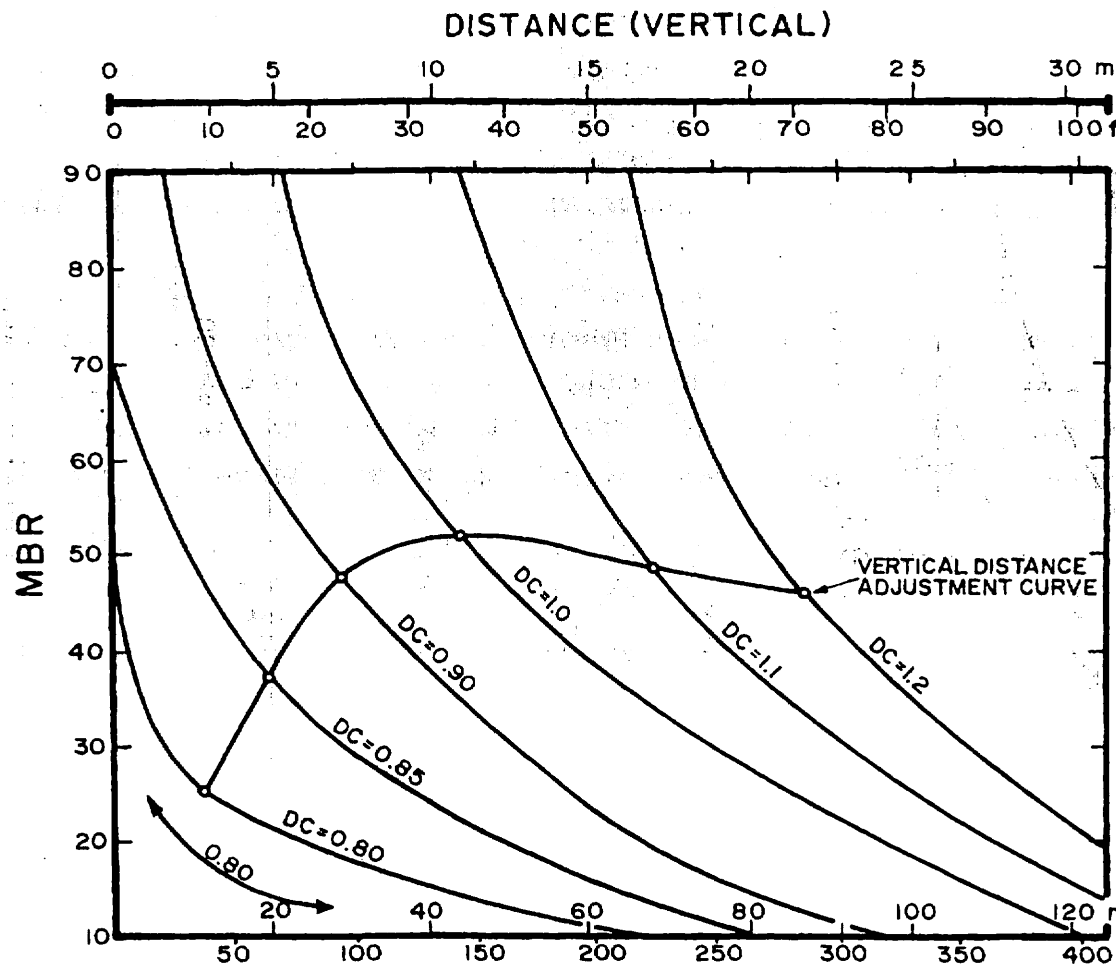


Figure 8.9 *Adjustment D_c for distance to cave line. For drifts beneath the caving area, the vertical distance is projected up to the single vertical distance adjustment curve; the rating is read by interpolating between the multiple curves. For workings horizontally removed from the caving area, the horizontal distance if projected up to the MBR value and the rating is interpolated at that point from the multiple curves. For working both beneath and to the side, ratings are computed both ways and the lowest value is taken. (After Cummings et al., 1982.)*

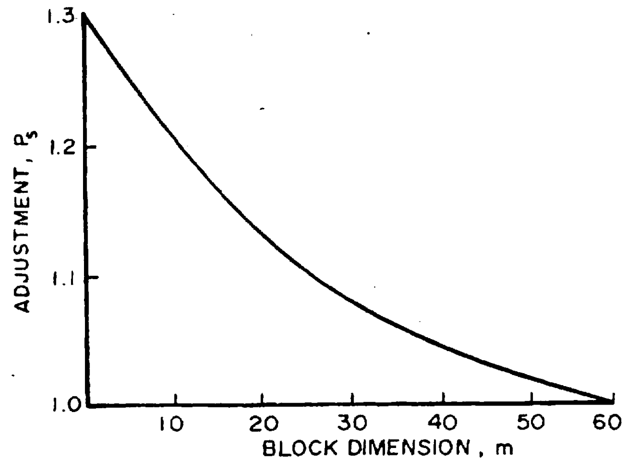


Figure 8.10 *Adjustment* P_s *for block/panel size. (After Cummings et al., 1982.)*

wide (mass) caving systems, receive an adjustment of 1.0. *PS* may also be applied to blocks that are partially undercut.

These three adjustment values, *S*, *DC*, and *PS*, are multiplied together and then multiplied by the adjusted MBR rating to yield the final MBR, which is used to obtain permanent drift support recommendations. The range of values for the product of these adjustment ratings is 0.56 to 1.7; there are no other restrictions on this range. In practice, the high end of this range will seldom be reached because small caving blocks are uncommon in present practice.

The recommended support is then arrived at for drift sections or intersections through Figure 8.11. For spans of more than 6 m, the rating scale for intersections is used.

The degree of acceptable repair refers to the occurrence of cracking, spalling, slabbing, or other unacceptable deformation of the lining, that requires a production interruption while repairs are made. Repairs necessitated by damage resulting from excessive secondary blasting, wear, and poor undercutting or draw control were not addressed in developing the support chart. A higher incidence of repair is tolerated in slusher or grizzly drifts than in fringe drifts of haulageways.

In selecting a support type based on final MBR, the user should have in mind the level of conservatism that was applied in selecting the development support. A high degree of support reliability in development will permit up to one repair category lighter support in production than might otherwise have been selected. For lower final MBR values, the support charts indicate a range of supports. This reflects the variability in conservatism among mine operators. Generally, the support used in such cases is the lightest in the range, although this depends on the acceptable amount of repair.

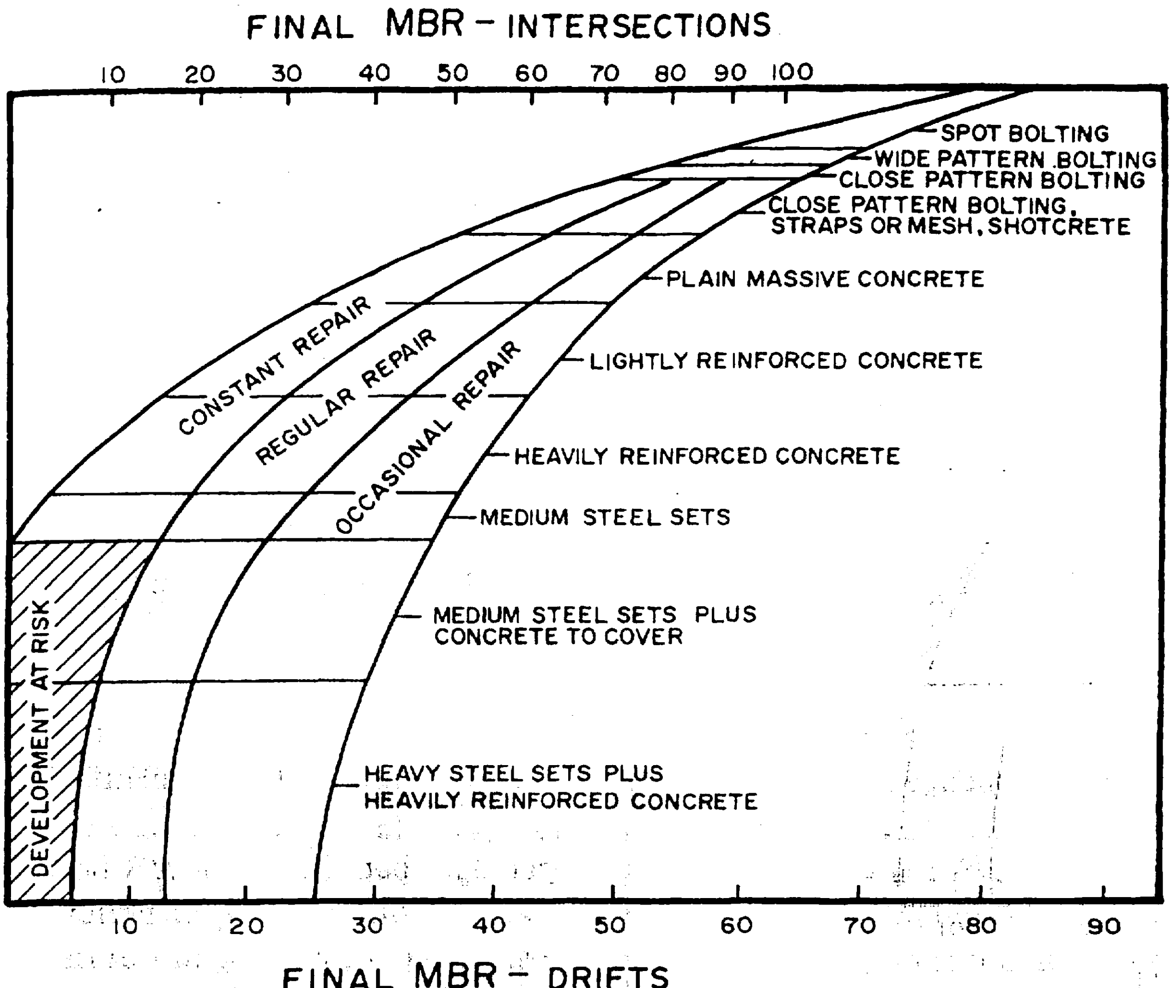


Figure 8.11 *MBR permanent support chart for production drifts. (After Cummings et al., 1982.)*

Explanation of the Support Types

Spot Bolting: *Bolting to restrain limited areas or individual blocks of loose rock, primarily for safety.*

Wide Pattern Bolting: *Bolts on spaced 1.2–1.8 m. May be wider in very large openings, when longer bolts are used.*

Close Pattern Bolting: *Bolts spaced less than 0.9–1.2 m (practical limit 0.6 m).*

Close Pattern Bolting, Mesh, or Straps, Shotcrete: *Close pattern bolts with welded mesh or chain link in raveling ground, and nominal 100 mm of shotcrete.*

Plain Massive Concrete: *Cast-in-place massive concrete lining, 0.3–0.45 m thick, may be applied over bolts or bolts and mesh when necessary. Prior shotcrete, if not damaged, may be considered part of this concrete thickness. Concrete should have a minimum* as-placed *28-d compressive strength of 21 MPa (3000 lb/in.^{2}.*

Lightly Reinforced Concrete: *Massive, cast-in-place concrete lining 0.3–0.45 m thick as above, lightly reinforced with rebars on 0.6-m centers or continuous heavy chain link. Reinforcement mainly in brows, crown, corners, and intersections.*

Heavily Reinforced Concrete: *Massive, cast-in-place concrete lining as above, heavily reinforced with rebars on 0.6-m centers or less, on ribs and crown.*

Medium Steel Sets: *Medium wide-flange steel sets on 1.2-m centers, fully lagged on the crown and ribs.*

Medium Steel Sets Plus Plain Concrete to Cover: *Medium wide-flange steel sets on 1.2-m centers, with plain, cast-in-place concrete (min strength 21 MPa) of sufficient thickness to cover the sets.*

Heavy Steel Sets Plus Heavily Reinforced Concrete: *Heavy wide-flange steel sets on 1.2-m centers, with minimum 0.3-m-thick heavily reinforced concrete throughout.*

General: *Concrete: it is assumed that proper concrete practice is observed: negligible aggregate segregation, full rock–concrete contact, adequate curing time.*

Chain link or steel sets, from development support, are considered reinforcement if the concrete between the sets is also reinforced.

8.2.2 Example

A mine, described in Table 8.10, uses a panel cave method with undercutting. Ore is developed and caved in a blockwise fashion. Undercut pillars are longholed and shot; there is no drift widening.

Slushers are used to move ore from the drawpoints to a drop point, through which it falls directly into ore cars in the haulage level. There are no transfer raises. Drift life is 1.5 yr or less, due to the relatively short ore columns (61 m or less). Slusher lanes are nominally 61 m in length, but may be less.

The type of drift considered in this example involves slusher lanes, which pose ongoing support problems. Key data for slusher drifts are given in Table 8.10.

Solution Determine the MBR for slusher drifts in altered porphyry. From data in Table 8.10, the ratings are as follows:

Intact Rock Strength. Av: 64 MPa. From Figure 8.4, rating = 5.

Discontinuity Density. Wide range of RQD, 39% (av). Discontinuities average spacing 18.3 cm (0.6 ft). May not be representative. From Figure 8.5, rating = 8 for RQD and rating = 10 for spacing. From Figure 8.6, a check: RQD = 39% relates to spacing of 15 cm. Thus

$$\text{Discontinuity density rating} = \text{RQD rating} + \text{Spacing rating} = 8 + 10 = 18 \tag{8.1}$$

Discontinuity Condition. Wall roughness: R–SR. Wall separation: "hairline" to less than 6 mm. Joint filling: none to minimal clay. Wall weathering: SL, some SO. From Table 8.6, rating = 25.

Groundwater Condition. Dry. From Table 8.7, rating = 15. Altered porphyry MBR:

$$\begin{aligned} \text{MBR} &= \text{intact rock strength rating} \\ &\quad + \text{discontinuity density rating} \\ &\quad + \text{discontinuity condition rating} \\ &\quad + \text{groundwater condition rating} \\ &= 5 + 18 + 25 + 15 \\ &= 63 \end{aligned} \tag{8.2}$$

Development Adjustments The application of development adjustments involves combining the engineering data with the geologic status.

Blasting Damage. Expect poor to fair conventional blasting. From Table 8.8, for "moderate damage," rating $A_b = 0.90$.

Induced Stresses. Slusher lanes are 3 m (10 ft) wide on 10.5-m (35-ft) centers. The 1.5 × 1.5-m finger raises are on 5.3-m (17.5-ft) centers, so the extraction ratio is given by

$$e_r = \frac{3 \times 5.3 = 2(1.5 \times 1.5)}{10.5 \times 5.3} = 0.37 \qquad (8.3)$$

From Kendorski et al. (1983), for a 3-m wide drift, a basic $e_r = 0.37$ and moderate blast damage generates an effective e_r of 0.51. This value reflects the area of rock remaining, after development, that is effective in accepting load.

In the absence of measurements, it may be expected $\sigma_1 = 1100$ psi (7.6 MPa). The horizontal stress is assumed to be $\sigma_1(\nu/1 - v) = \sigma_h = \sigma_3$, where ν is assumed to be 0.25 in the absence of measurements. Thus, $\sigma_v > 3\,\sigma_h$ and the top curve on Figure 8.8 is used. Thus, induced stresses rating $A_s = 0.88$.

Fracture Orientation. For altered porphyry, there are four fracture orientations. In order from most to least prevalent, the sets are (strike, dip, number of observations in set from Schmidt plot clusters)

1. NE, vertical, 56 observations.
2. WNW, steeply dipping NE, 53 observations.
3. NE, shallow or moderate dip SE, 32 observations.
4. NW, steeply dipping SW, 22 observations.

Slusher lane development is from NW to SE. Therefore, the sets are oriented as follows:

Set 1. Perpendicular
Vertical dip

Set 2. Parallel
Steep dip (45–90°)

TABLE 8.10 Geological and Engineering Data for the Design Problem

Geological Data

Project Name *Example Mine* Site of Survey *Upper Level* By *ABC* Date *2/25/88*

1. Geologic region: *Altered porphyry* Rock type *Alt. ppy, volcanic* Location *Block 1, access and slushers*
2. Compressive strength: Average *9,300 psi* Range *4,500–12,600* Method *Point-load* Comment *Many fractures-continuous*
3. Core recovery: Interval *80–300 ft* Average *83%* Range *66–100%*
4. RQD: Interval *-do-* Average *39%* Range *14–90%*
5. Discontinuity spacing: Average *0.6 ft* Range *0.2–1.6 ft* Comment *Locally 1.5–2 ft*
6. Discontinuity condition

	Wall roughness	Wall separation	Joint filling	Wall weathering
Most common	*Rough*	*Hairline*	*None*	*Slightly weathered*
Intermediate	*Slightly rough*	*< 1/4 in.*	*FeOx*	*Softened*
Least common	*Smooth*	*None*	*Clay*	*Severe*
Consensus	*R–SR*	*Hairline*	*Nonclay*	*SL*

7. Water condition Dry Damp Wet Dripping Flowing
8. Fracture orientations

	Set 1	Set 2	Set 3	Set 4	Set 5
Strike	*NE*	*WNW*	*NE*	*NW*	
Dip/direction	*Vert.*	*Str./NE*	*Mod./SE*	*Str/SW*	
Rank	*1(58)*	*2(53)*	*3(38)*	*4(88)*	

9. Major structures

	Strike	Dip	Dip dir.	Width	Location/comment
Name: *Fault zone*	*NE*	*Mod.*	*NW*	*+100 ft*	*SE Boundary, Block 1, distance = 80–100 ft*
Name:					
Name:					

10. Stress field

	Direction	Magnitude	Measured?
σ_1	*Vertical*	*1100 psi*	*No*
σ_3	*Horizontal*	*800 psi*	*No*

11. Source of geological data *Mainly core with limited underground exposures*

Engineering Data

Project Name *Example Mine* Site of Survey *Upper Level* By *ABC* Date *3/10/88*

1. Type of drift(s): *Slusher* 2. Orientation(s) *NW/SE* 3. Design life *About 1½ yr max.*

4. Design Dimension: Width *10 ft* Width variation *none*

   Height *11 ft* Height variation *none*

5. Drift spacing (horizontal) *35 ft center-to-center*

   Other openings Type *fingers* Size *5 ft × 5 ft* Spacing *17.5 ft center-to-center*

6. Extraction Ratio

   Multiple openings: Excavated area *225 ft^2* Unexcavated *388 ft^2* e_r *0.37*

   Single opening: 1.5 (width) ______ Excavated ______ Unexcavated ______ e_r ______

7. Distance below undercut: Drift floor to undercut floor *15 ft*

   Drift floor to undercut floor *4 ft*

8. Method of excavation: Machine-bored Controlled drilling and blasting

9. Excavation conditions

   Perimeter hole traces *Few seen. No blast holes remaining.*

   Rib or crown looseness *Ribs drummy in places. Crown tight after barring down.*

   New or existing cracks *Some new. Some old joints opened.*

   Overbreak and barring-down *O.B. = 1–2 ft. Barring: Same, not major.*

   Other criteria ______

10. Intersections, turnouts: Type *Intersection* Location *Access, vent* Max span *16 ft*

11. Block dimensions: Side *800 ft* Orientation *NW/SE* End *200 ft* Orientation *NE/SW*

12. Cave line Direction *ENE* Direction of programs *NNW*

13. Drift location (in block, with respect to major structures and their dips, with respect to cave). *Across block, beneath cave, fault zone structure across ends opposite slusher*

Set 3. Perpendicular, drive with dip
Moderate dip (20–45°)

Set 4. Parallel
Steep dip (45–90°)

From Table 8.9, the set ratings are: Set 1, 1.0; Set 2, 0.8; Set 3, 0.95, Set 4, 0.80. Weighting these according to the number of observations of each,

$$\frac{1.0 \times 56 + 0.8 \times 53 + 0.95 \times 32 + 0.8 \times 22}{56 + 53 + 32 + 22} = A_o = 0.90 \qquad (8.4)$$

Adjusted MBR Computation:

The adjustments are summarized as follows:

$$A_b = 0.90 \qquad A_s = 0.88 \qquad A_o = 0.90 \qquad (8.5)$$

Checking $A_b \times A_s \times A_o = 0.713$, which is greater than the 0.5 minimum value.

Thus, adjusted MBR = 0.713 (63) = 45, for altered porphyry in slusher drifts.

Production Adjustments. In this step, the adjusted MBR, which is related to development support, is further adjusted to allow for dynamic and transient deformations related to caving. It should be pointed out that adequate undercutting and draw control practice is assumed, so that loads developing during routine production remain below the peak abutment levels. There is no allowance in the MBR system for incomplete blasting of pillars during undercutting in which stubs are left, or for caving difficulties such as hangups or packed drawpoints, or other influences causing excessive weight to be thrown onto the drift support.

Major Structures. Since a fault zone exists in the vicinity of the cave area, this zone is considered a major structure. It is assumed that the fault zone was classified as a separate structure having an MBR of 37. The altered porphyry MBR is 63, as opposed to 37 for the fault zone, and this is a significant contrast. In reality, a zone of any width can be regarded as a major structure, so long as the zone is independently classifiable and of significant contrast in MBR value.

From Table 8.10, the fault zone is along the southeastern limit of Block 1; the strikes are generally northeast and the dips moderately northwest. The zone thickness is thought to be at least 30 m. Thus, $W = 30$ m.

The closest point of approach, of altered porphyry to the zone boundary, within the slusher lanes, is 24–30 m (80–100 ft).

The key information is thus:

Distance to the fault zone: 24 m = $0.8W$.

Fault strike versus heading direction: perpendicular.

Dip direction: toward the drifts. Dip amount: moderate.

From Table 8.9, the adjustment $S = 0.82$.

Distance to Cave Line. The closest point of approach is used. The sense of the distance, for slusher lanes, will be vertical, and amounts of the level separation. For small level separations, the height of the drift is significant.

The distance from the slusher drift crown to the undercut floor is 4.5 m − 3 m = 1.5 m (5 ft). The vertical distance adjustment curve in Figure 8.9 considers separations only as low as 3 m, so 3 m is used.

The adjustment $DC = 0.80$. Note that the MBR does not figure in this adjustment, where vertical distance is being considered.

Block Panel Size. The panel size dimension is taken perpendicular to the advancing cave line. The most unfavorable condition is selected, which in this base will be the maximum void opened up.

For a 61 × 61-m block, using diagonal retreat caving, the distance used will be well in excess of 61 m.

Adjustment $PS = 1.0$.

Final MBR Computation:

The adjustments are as follows:
Major structures $S = 0.82$
Cave line distance $DC = 0.80$
Block/panel size $PS = 1.0$

The final MBR is thus

$$\text{Adjusted MBR} \times S \times \text{DC} \times \text{PS} = 45 \times 0.82 \times 0.80 \times 1.0$$
$$= 29.52 = 30 \tag{8.6}$$

Support Recommendations

Isolated Drifts. From the support chart in Figure 8.7, it is readily seen that an isolated drift in altered porphyry (MBR = 63) would require rock bolts in either a wide or medium pattern; mesh may occasionally be required.

Development Support. From the same chart in Figure 8.7 and an adjusted MBR of 45, one would recommend close pattern bolting with mesh in better sections. Elsewhere, bolts and shotcrete, or occasional light steel, will be needed to stabilize the opening prior to final lining.

Production Support. The final MBR is 30. For slusher lanes, repair is fairly routine because of brow damage. The recommended support from Figure 8.11 corresponds to reinforced concrete over bolts or over bolts and mesh. For service life that is intended to be short, selection of lighter support may be feasible.

For intersections, additional concrete reinforcement should be provided.

8.3 COAL MINING: USA

Unal (1983, 1986) developed an empirical equation relating the rock load height h_t to the RMR from the Geomechanics Classification (Bieniawski, 1979) and to roof span B in coal mines as follows:

$$h_t = \frac{100 - \text{RMR}}{100} B \tag{8.7}$$

He showed that the roof bolt length can be estimated as one-half of the rock load height (h_t) and on this basis prepared a series of design charts for mechanically tensioned and resin grouted bolts for applications in U.S. coal mines. Examples of the charts are given. The key at the bottom of Chart 8.1 applies to Charts 8.2–8.5 as well.

8.3.1 Example

A roof strata is to be classified for a 6.1-m-wide coal mine entry to facilitate selection of mechanical or resin grouted rock bolts. The coal seam, the

CHART 8.1 Roof Support Design Chart #1 for Coal Mines

ENTRY WIDTH: 20-FT

ROOF ROCK CLASS	ROCK MASS RATING (RMR)	ROCK LOAD HEIGHT H_T (FT)	SUPPORT SPECIFICATIONS: MECHANICAL BOLTS	SUPPORT SPECIFICATIONS: RESIN BOLTS	ALTERNATE SUPPORT PATTERNS: MECHANICAL BOLTS/POSTS	ALTERNATE SUPPORT PATTERNS: RESIN BOLTS/POSTS	SPECIFICATIONS FOR POSTS
I VERY GOOD	90	2.0	L : 2.5' S : 5' x 5' G : 40 ϕ : 5/8" C : 6.2 tons			Not economical	
	80	4.0	L : 2.5" S : 5' x 4.5' G : 60 (40) ϕ : 3/4" (7/8") C : 11 tons	L : 2.5' S : 5' x 5' G : 60 ϕ : 3/4" C : 12 tons			
II GOOD	70	6.0	L : 3.0' S : 4' x 4' G : 60 ϕ : 3/4" C : 10 tons	L : 3.0' S : 5' x 5' G : 60 ϕ : 3/4" C : 18 tons			
	60	8.0	L : 4.0' S : 5' x 5' G : 60 ϕ : 5/8" C : 9 tons	L : 4.0' S : 5' x 5' G : 60 ϕ : 1" C : 23.7 tons			ϕ_p = 5.5" S_p = 10'
III FAIR	50	10.0	L : 5.0' S : 5' x 5' G : 40 ϕ : 3/4" C : 8 tons	L : 4.0' S : 5' x 4' G : 60 ϕ : 1" C : 23.7 tons			ϕ_p = 6.5" S_p = 10'
	40	12.0	L : 6.0' S : 5' x 5' G : 40 ϕ : 3/4" C : 7 tons	L : 4.0' S : 4' x 4' G : 60 ϕ : 1" C : 23.7 tons			ϕ_p = 6.5" S_p = 7.5'
IV POOR	30	14.0	L : 7.0' S : 5' x 5' G : 40 ϕ : 5/8" C : 6 tons	L : 5' S : 5' x 5' G : 60 ϕ : 3/4" C : 12 tons			ϕ_p = 5.5" S_p = 5'
	20	16.0	L : 8.0' S : 4' x 4.5' G : 40 ϕ : 5/8" C : 5 tons	L : 5' S : 5' x 5' G : 60 ϕ : 3/4" C : 12 tons	S_p = 4.5'	S_p = 5'	ϕ_p = 6.0"

L = bolt length ϕ = bolt diameter ϕ_p = post diameter
S = bolt spacing   C = bolt capacity S_p = post spacing
G = grade of steel

(1986), the stress adjustment multiplier is 0.9. Hence, the adjusted RMR value is calculated as $0.9(51 - 5) = 41$.

Thus, RMR = 41, and this value is used to select rock bolting parameters from Chart 8.1 or, better still, using the microcomputer program given in the Appendix. The computer graphics output from the program is also provided in the Appendix.

8.4 COAL MINING: INDIA

Venkateswarlu (1986) of the Central Mining Research Station (CMRS) of India modified the Geomechanics Classification (Bieniawski, 1979) for estimating roof conditions and support in Indian coal mines. The modification was called the CMRS Geomechanics Classification.

The Geomechanics Classification used in India is a simple and practical method of estimating roof conditions in a mine. The five classification parameters and their ratings are given in Table 8.11. Note that the point-load index I_{pl} obtained from an irregular piece of rock is converted to estimate the uniaxial compressive strength C_0 using the empirical relation: $C_0 = 14\ I_{pl}$. Weatherability is determined by the ISRM slake durability index test, with the first cycle taken for the classification. Groundwater seepage rate is measured by drilling a 1.8-m-long hole in the roof and collecting the water percolating through the hole. This water flow is expressed in mL/min. All the geological features are recorded through standard geotechnical mapping.

Based on the RMR obtained from Table 8.11, the support systems are selected from Table 8.12. An empirical relation has been established to estimate the rock load as follows:

$$\text{Rock load} = \text{span} \times \text{rock density} \times (1.7 - 0.037\ \text{RMR} + 0.0002\ \text{RMR}^2) \qquad (8.8)$$

This RMR classification system has so far been tried in 47 Indian coal mines. Majority of the roof strata experiencing ground control problems come under the category of RMR Class III (Fair) and Class IV (Poor).

8.4.1 Example

A coal mine in India has seam workings at a depth of about 140 m. The 3.5–5-m-thick seam is being developed by the room and pillar method. The seam is characterized as follows:

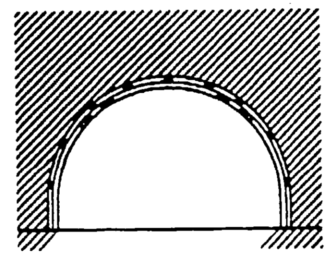
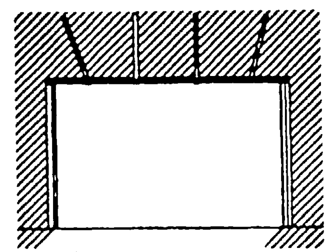
TABLE 8.11 Geomechanics Classification of CMRS, India: Ratings for Parameters[a]

Parameter		Range of Values				
1. Layer thickness	(cm)	<2.5	2.5–7.5	7.5–20	20–50	>50
	Rating	0–4	5–12	13–20	21–26	27–30
2. Structural features	Description	Highly disturbed with faults	Disturbed with numerous slips	Moderately disturbed	Slightly disturbed	Not disturbed
	Rating	0–4	5–10	11–16	17–21	22–25
3. Weatherability (I_{sd-1})	(%)	<60	60–85	85–97	97–99	>99
	Rating	0–3	4–8	9–13	14–17	18–20
4. Strength of the rock	(kg/cm^2)	<100	100–300	300–600	600–900	>900
	Rating	0–2	3–6	7–10	11–13	14–15
5. Groundwater flow	(mL/min)	>2000	2000–200	200–20	20–0	Dry
	Rating	0–1	2–4	5–7	8–9	10

RMR	0–10	10–20	20–30	30–40	40–50	50–60	60–70	70–80	80–90	90–100
CLASS	VA	VB	IVA	IVB	IIIA	IIIB	IIA	IIB	IA	IB
DESCRIPTION	VERY POOR		POOR		FAIR		GOOD		VERY GOOD	

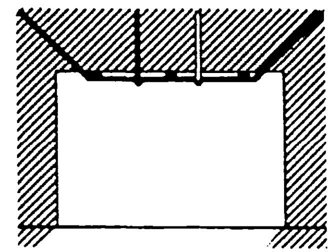
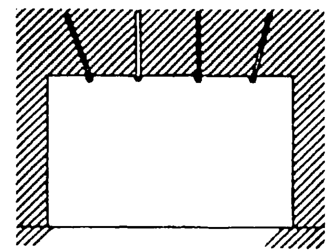
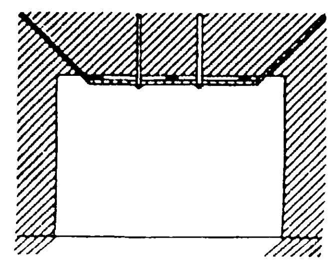
[a] After Venkateswarlu (1986).

TABLE 8.12 Design Guidelines for Roof Support in Indian Coal Mines: Roof Span 4.2–4.5 m (14–15 ft)[a]

		Permanent Openings (life more than 10 yr)	
RMR	Estimated Rock Load (t/m^2)	Support Description[b]	Recommended Supports
1	2	3	4
0–20	>10	Type A: yielding steel arches of 28 kg/m section	
20–30	7–10	Type B: full-column quick-setting grouted bolts with wire netting, W-straps and props; l = 1.8 m, S_b = S_r = 1.0 m or Type C: rigid steel arches; spacing 1.2 m	
30–40	5–7	Type D: resin bolting with W-strap and steel props (10 cm ϕ, 5-mm wall thickness); l = 1.8 m, S_b = 1.0 m, S_r = 1.2 m or Type E: brick walling (40 cm thick) with steel girders (200 × 100-mm section) at 1.2-m spacing, and concrete sleepers	

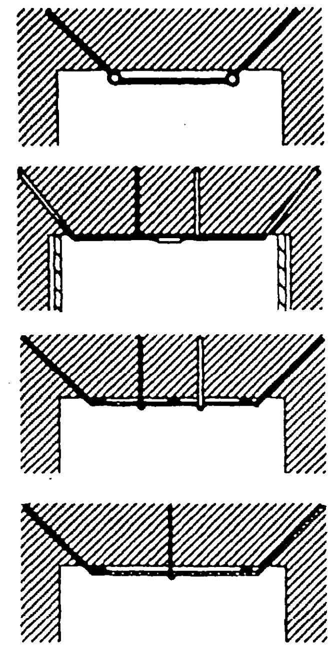
(*Table continues on p. 172.*)

TABLE 8.12 *(Continued)*

		Permanent Openings (life more than 10 yr)	
RMR	Estimated Rock Load (t/m²)	Support Description[b]	Recommended Supports
1	2	3	4
40–50	3–5	Type F: roof truss supplemented with grouted bolts and wooden sleepers (of treated timber): l = 1.5 m, S_b = 1.0 m, S_r = 1.2 m or Type G: rectangular steel supports (110 × 110-mm section) rigidly fixed at the ends with tie rods; timber lagging	
50–60	3–3	Type H: full-column cement grouted bolts; l = 1.5 m, S_b = S_r = 1.2 m or Type I: steel props on either side of gallery at 1.2-m spacing	
60–80	0.5–2	Type J: supports in disturbed zones wherever necessary (roof struss and bolting)	
80–100	<0.5	Generally supports not required	

Temporary Openings (life less than 10 yr)

RMR	Estimated Rock Load (t/m²)	General Supports Recommended[b]
0–20	>10	Type C: steel arches
20–30	7–10	Type K: roof truss using quick-setting grout (spacing 1.0 m) and wooden props (15 cm ϕ)
30–40	5–7	Type L: rope truss system (spacing 1.2 m) with bolting; l = 1.8 m, S_b = 1.0 m, S_r = 1.2 m
40–50	3–5	Type M: roof truss supplemented with rope dowelling and timber lagging; l = 1.5 m, S_b = 1.0 m, S_r = 1.2 m
50–60	2–3	Type N: roof truss with a single rope dowel; l = 1.5 m
60–80	0.5–2	Roof bolting in disturbed zones only
80–100	<0.5	Generally no supports



[a] After Venkatesvarlu (1986).

[b] Bolting parameters: bolt–dowel length l, bolt–dowel spacing S_b, row spacing S_r.

Main roof: massive sandstone
Immediate roof: carbonaceous shale—2.4 m
Top coal: 1.2 m
Main coal: 2.5 m

The span of the entries is 3.5 m and the mean density of the roof rocks is 2.0 g/cm^3. The parameter values and the allotted ratings for the two roof types of coal and shale are given in Table 8.13. The two RMR values have been combined by the weighted average method. An adjustment of 10% reduction is made to this combined RMR to account for the stresses induced by the overlying seam workings. The final RMR of 44.5 classifies the roof strata as Class IIIA (Fair Roof).

Roof support is selected on the basis of the above classification from Table 8.12. It can be seen that Class IIIA roofs require systematic roof

TABLE 8.13 Application of the Geomechanics Classification at an Indian Coal Mine

	Coal (1.2 m thick)		Shale (2.4 m thick)	
Parameter	Value	Rating	Value	Rating
1. Layer thickness	2.9 cm	4	6.8 cm	13
2. Rock strength	250–275 kg/cm^2	7	232 kg/cm^2	5
3. Weatherability				
Swelling strain	}	16	3.4% }	5
Slake durability	98.5% }		92.9% }	
4. Groundwater	Wet roof	9	Wet roof	9
5. Structural features	Cracks, minor cleats	18	Cracks, two sets of joints	
	RMR	*54*		*47*

$$\text{Weighted (combined) RMR} = \frac{(1.2 \times 54) + (2.4 \times 47)}{3.6} = 49.3$$

$$\text{Adjusted RMR} = 49.3 \times 0.9 = 44.5$$

$$\text{Rock load} = 3.5\text{ m} \times 2.007\text{ t/m}^2 \times [1.7 - (0.0037 \times 44.5) + (0.0002 \times (44.5)^2)] = 3.16\text{ t/m}^2$$

$$\text{Support load} = \frac{8 + (2 \times 6)}{1.0 \times 3.5} = 5.7\text{ t/m}^2$$

(Roof truss + 2 bolts; 1-m row spacing)
Safety factor = 1.8

support in combination with two grouted bolts. Spacing between the rows should be 1.0 m.

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9

Other Applications

Discoveries and inventions arise from observations of little things.
—Alexander Bell

Rock mass classifications have played a useful role in estimating the strength and deformability of rock masses and in assessing the stability of rock slopes. They were also shown to have special uses for serving as an index to rock rippability, dredgeability, excavatability, cuttability, and cavability.

9.1 ESTIMATING ROCK MASS STRENGTH

As discussed in Chapter 4, the empirical criterion proposed by Hoek and Brown (1980) enables estimation of the strength of rock masses using the expression (eq. 4.4)

$$\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \sqrt{m\,\frac{\sigma_3}{\sigma_c} + s} \qquad (9.1)$$

where σ_1 = the major principal stress at failure,
σ_3 = the applied minor principal stress,
σ_c = the uniaxial compressive strength of the rock material
m and s = constants dependent on the properties of the rock and

the extent to which it was fractured by being subjected to σ_1 and σ_3.

For intact rock material, $m = m_i$ is determined from a fit of the above equation to triaxial test data from laboratory specimens, taking $s = 1$. For rock masses, use is made of the RMR, as suggested by Hoek and Brown (1988):

When Rock Mass Is Undisturbed (e.g., carefully blasted or machine excavated rock):

$$m = m_i \exp\left(\frac{(\text{RMR} - 100)}{28}\right) \tag{9.2}$$

$$s = \exp\left(\frac{\text{RMR} - 100}{9}\right) \tag{9.3}$$

When Rock Mass Is Disturbed (as in slopes or blast-damaged rock):

$$m = m_i \exp\left(\frac{\text{RMR} - 100}{14}\right) \tag{9.4}$$

$$s = \exp\left(\frac{\text{RMR} - 100}{6}\right) \tag{9.5}$$

where RMR is the basic (unadjusted) rock mass rating from the Geomechanics Classification (Bieniawski, 1979).

The typical values for m and s for various rock types and corresponding to various RMR as well as Q values are listed in Table 9.1 (Hoek and Brown, 1988).

It has recently been suggested that the above Hoek–Brown criterion may underestimate the strength of highly interlocking rock masses such as those featuring high-strength basalt (Schmidt, 1987).

For weak rock masses, the latest contribution was made by Robertson (1988), who modified the RMR system (for ratings < 40) on the basis of back analysis from case histories involving pit slope failures in weak rock strata. This modification of the Geomechanics Classification is presented in Table 9.2, which shows that the maximum value for the groundwater parameter (15) has been added to the first parameter: strength of intact rock.

TABLE 9.1 The Hoek-Brown Failure Criterion: Relationship between *m* and *s* Constants and Rock Mass Quality[a]

APPROXIMATE RELATIONSHIP BETWEEN ROCK MASS QUALITY AND MATERIAL CONSTANTS						
Disturbed Rock Mass *m* and *s* Values						*Undisturbed Rock Mass m and s Values*
Empirical failure criterion $\sigma_1 = \sigma_3 + (m\sigma_c\sigma_3 + s\sigma_c^2)^{1/2}$ where σ_1 = major principal stress, σ_3 = minor principal stress, σ_c = uniaxial compressive strength of intact rock, *m* and *s* are empirical constants		CARBONATE ROCKS WITH WELL-DEVELOPED CRYSTAL CLEAVAGE *Dolomite, Limestone, and Marble*	LITHIFIED ARGILLACEOUS ROCKS *Mudstone, Siltstone, Shale, and Slate (Normal to Cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *Sandstone and Quartzite*	FINE-GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *Andesite, Dolerite, Diabase, and Rhyolite*	COARSE-GRAINED POLYMINERALLIC IGNEOUS AND METAMORPHIC CRYSTALLINE ROCKS *Amphibolite, Gabbro, Gneiss, Granite, Norite, and Quartz–Diorite*
INTACT ROCK SAMPLES						
Laboratory-size specimens free of discontinuities						
	m	7.00	10.00	15.00	17.00	25.00
RMR = 100	s	1.00	1.00	1.00	1.00	1.00
Q = 500	*m*	*7.00*	*10.00*	*15.00*	*17.00*	*25.00*
	s	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*

(*Table continues on p. 180.*)

TABLE 9.1 (*Continued*)

APPROXIMATE RELATIONSHIP BETWEEN ROCK MASS QUALITY AND MATERIAL CONSTANT

Disturbed Rock Mass m and s Values						*Undisturbed Rock Mass m and s Values*
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 1–3 m						
	m	2.40	3.43	5.14	5.82	8.56
RMR = 85	s	0.082	0.082	0.082	0.082	0.082
Q = 100	*m*	*4.10*	*5.85*	*8.78*	*9.95*	*14.63*
	s	*0.189*	*0.189*	*0.189*	*0.189*	*0.189*
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 1–3 m						
	m	0.575	0.821	1.231	1.395	2.052
RMR = 65	s	0.00293	0.00293	0.00293	0.00293	0.00293
Q = 10	*m*	*2.006*	*2.865*	*4.298*	*4.871*	*7.163*
	s	*0.0205*	*0.0205*	*0.0205*	*0.0205*	*0.0205*
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 0.3–1 m						
	m	0.128	0.183	0.275	0.311	0.458
RMR = 44	s	0.00009	0.00009	0.00009	0.00009	0.00009
Q = 1	*m*	*0.947*	*1.353*	*2.030*	*2.301*	*3.383*
	s	*0.00198*	*0.00198*	*0.00198*	*0.00198*	*0.00198*

POOR QUALITY ROCK MASS Numerous weathered joints at 30–500 mm with some gouge. Clean compacted waste rock						
	m	0.029	0.041	0.061	0.069	0.102
RMR = 23	s	0.000003	0.000003	0.000003	0.000003	0.000003
Q = 0.1	*m*	*0.447*	*0.639*	*0.959*	*1.087*	*1.598*
	s	*0.00019*	*0.00019*	*0.00019*	*0.00019*	*0.00019*
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced at less than 50 mm with gouge. Waste rock with fines						
RMR = 3	m	0.007	0.010	0.015	0.017	0.024
Q = 0.01	s	0.0000001	0.0000001	0.0000001	0.0000001	0.0000001
	m	*0.219*	*0.313*	*0.469*	*0.532*	*0.782*
	s	*0.00002*	*0.00002*	*0.0002*	*0.00002*	*0.00002*

[a] After Hoek and Brown (1988).

TABLE 9.2 Geomechanics Classification for Rock Slopes[a,b]

	Parameter		Ranges of Values										
1	Strength of intact rock material	Point-load strength index (MPa)	>10	4–10	2–4	1–2	For this low range, uniaxial compressive test is preferred						
		Uniaxial compressive strength (MPa)	R5 >250	R4 100–250	R3 50–100	R2 25–50	R1 5–25	R1 1–5	S5	S4	<1 S3	S2	S1
	Rating		30	27	22	19	17	15	10	6	2	1	0
2	Drill core quality RQD (%)		90–100	75–90	50–75	25–50	<25						
	Rating		20	17	13	8	3						
3	Spacing of discontinuities		>2 m	0.6–2 m	200–600 mm	60–200 mm	<60 mm						
	Rating		20	15	10	8	5						
4	Condition of discontinuities		Rock > R1 Very rough surfaces Not continuous No separation Unweathered wall rock	Rock > R1 Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Rock > R1 Slightly rough surfaces Separation < 1 mm Highly weathered walls	Rock ≥ R1 Slickensided surfaces or Gouge < 5 mm thick or Separation 1–5 mm Continuous	Rock < R1 Soft gouge > 5 mm thick or Separation > 5 mm Continuous						
	Rating		30	25	20	10							

[a] After Robertson (1988).

[b] Key:
- R1 = very weak rock
- R2 = weak rock
- R3 = medium strength rock
- R4 = strong rock
- R5 = very strong rock
- S1 = very soft soil
- S2 = soft soil
- S3 = firm soil
- S4 = stiff soil
- S5 = very stiff soil.

Using this approach, when RMR > 40, slope stability is determined by the orientation of and the strength along the discontinuities. Where the rating is less than 30, failure may occur through the rock mass at any orientation, and the rock mass strength is estimated from rating–strength correlation, as shown in Table 9.3. Robertson (1988) cautions that more case histories are needed before the correlation in Table 9.3 can be considered as typical.

Modifications to the shear strength estimates from the RMR values were also provided by Serafim and Pereira (1983) and are depicted in Table 9.4. Most recently, Trunk and Hönisch (1989) confirmed the friction angle estimates for rock masses, given in Section D of Table 4.1 ($\phi = 0.5$ RMR $+ 5$) and, on the basis of 40 case histories, suggested this refinement: $\phi = 0.5$ RMR $+ 8.3 \pm 7.2$.

Another approach to rock mass strength determination was proposed by Laubscher (1984). Using the Geomechanics Classification, the procedure is as follows:

1. The intact rock strength (IRS) rating is subtracted from the total rating RMR, and the balance is a function of the remaining possible rating of 85, since the maximum rating for the strength of intact rock is 15.
2. The IRS rating, which represents the strength σ_c, in MPa, of the rock material, must be reduced to 80% of its value since it is assumed that large (hard-rock) specimens have a strength equal to 80% of the standard core sample tested in the laboratory. This is a constant scaling factor. Thus

$$\frac{\text{RMR} - \text{IRS}}{85} \times \sigma_c \times \frac{80}{100} = \text{basic rock mass strength (BMRS)} \qquad (9.6)$$

TABLE 9.3 Geomechanics Classification for Rock Slopes: Strength Correlation[a]

		Strength Parameters			
	Rating	Island Copper Mine		Getchell Mine	
Rock Mass Class	(RMR)	C' (psi)	ϕ'	C' (psi)	ϕ'
IVa	35–40	12.5	40		
	30–35	10.5	36		
IVb	25–30	10.0	34	7.0	30
	20–25	20.0	30	7.0	26
Va	15–20	9.0	27.5	7.0	24
Vb	5–15	7.5	24	2.0	21

[a] After Robertson (1988).

TABLE 9.4 Geomechanics Classification for Rock Foundations: Shear Strength Data[a]

	Rock Mass Properties				
RMR	100–81	80–61	60–41	40–21	<20
Rock class	I	II	III	IV	V
Cohesion, kPa	>400	300–400	200–300	100–200	<100
Friction, deg	>45	35–45	25–35	15–25	<15
Modulus, GPa	>56	18–56	5.6–18	1.8–5.6	<1.8
	Shear Strength of Rock Material				
Cohesion, MPa	>25	15–25	8.5–15	4.5–8.5	<4.5
Friction, deg	>65	55–65	48–55	41–48	<41
	Frictional Shear Strength of Discontinuities, deg				
Rating for Condition of Discontinuities:	30	25	20	10	0
Completely dry	45	35	25	15	10
Damp	43	33	23	13	<10
Wet	41	31	21	11	<10
Dripping	39	29	19	10	<10
Flowing	37	27	17	<10	<10

[a] After Serafim and Pereira (1983).

3. The design rock mass strength is obtained by incorporating a variable reducing factor due to adjustments for weathering (90%), favorability of joint orientation (80%), and blasting effect (90%). Thus

$$\text{DRMS} = 90\% \times 80\% \times 90\% \text{ BRMS} = 0.65 \text{ BRMS} \qquad (9.7)$$

In addition, an averaging factor is employed where a rock mass contains weak and strong zones.

In another study, Stille et al. (1982) provided a direct correlation between the RMR and the uniaxial compressive strength of rock mass σ_{cM} on the basis of back-calculations featuring the finite element method and Swedish case histories. They suggested the following relationship:

RMR	100–81	80–61	60–41	40–21	<20
σ_{cM} (MPa)	30	12	5	2.5	0.5

Finally, Yudhbir (1983) suggested a rock mass criterion of the form discussed by Bieniawski (1974), namely

$$\frac{\sigma_1}{\sigma_c} = A + B\left(\frac{\sigma_1}{\sigma_c}\right)^{\alpha}. \tag{9.8}$$

where $\alpha = 0.75$ and A is a function of rock mass quality (note that $A = 1$ for intact rock), namely

$$A = \exp(0.0765\ \mathrm{RMR} - 7.65) = 0.0176Q^{0.65} \tag{9.9}$$

and B depends on rock type as determined by Bieniawski (1974) for these rock types:

Shale and limestone	$B = 2.0$
Siltstone and mudstone	$B = 3.0$
Quartzite, sandstone, and dolerite	$B = 4.5$
Very hard quartzite	$B = 4.5$
Norite and granite	$B = 5.0$

The above criterion requires experimental validation of the expression for parameter A.

9.2 ESTIMATING ROCK MASS MODULUS

The RMR from the Geomechanics Classification was related (Bieniawski, 1978) to the in-situ modulus of deformation in the manner shown in Figure 4.3.

The following relationship was obtained:

$$E_M = 2 \times \mathrm{RMR} - 100 \tag{9.10}$$

where E_M is the in-situ modulus of deformation in GPa and RMR > 50. For poorer-quality rock masses, Serafim and Pereira (1983) extended the above relationship in the range RMR < 50 as well as confirmed the equation. They also proposed this overall correlation:

$$E_M = 10^{(\mathrm{RMR} - 10)/40} \tag{9.11}$$

Using the well-known correlation RMR $= 9 \ln Q + 44$, Barton (1983) supplemented the data of Bieniawski (1978) with his own results and plotted the range of the measured values as depicted in Figure 9.1. He found a useful approximation:

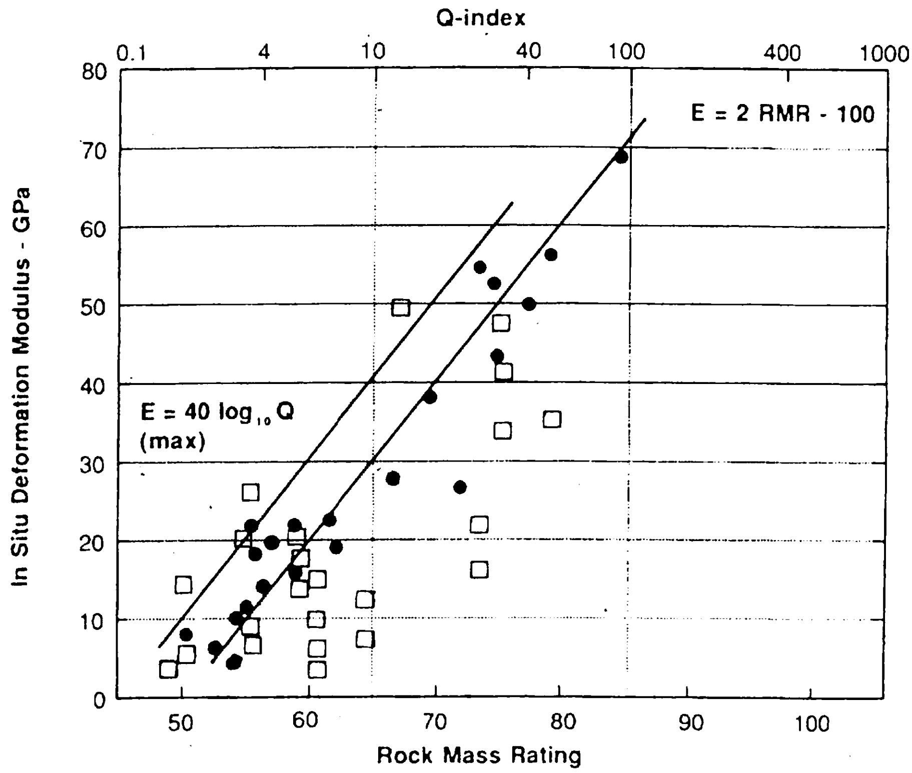


Figure 9.1 *Estimation of in-situ modulus of deformation from two classification methods: squares represent Q case histories, dots are RMR cases. (After Barton, 1983.)*

$$E_{\text{mean}} = 25 \log Q \text{ and } E_{\text{max}} = 40 \log Q \qquad (9.12)$$

($E_{\text{min}} = 10 \log Q$) and confirmed that careful double classification at a potential test site might eliminate the need for expensive tests or reduce their numbers.

9.3 ASSESSING ROCK SLOPE STABILITY

Romana (1985) made an important contribution in applying rock mass classifications to the assessment of the stability of rock slopes. He developed a factorial approach to rating adjustment for the discontinuity orientation parameter in the RMR system, based on field data. Recognizing that rock slope stability is governed by the behavior of the discontinuities and that in the original RMR system (Bieniawski, 1979) specific guidelines for favorability of joint orientations were lacking, his modification of the RMR system involved subtracting the newly proposed adjustment factors for discontinuity orientation and adding a new adjustment factor for the method of excavation. This approach is suitable for preliminary assessment of slope stability in rock, including very soft or heavily jointed rock masses.

The new adjustment rating for joints in rock slopes is a product of three factors:

F_1 reflects parallelism between the slope and the discontinuity strike.

F_2 refers to the discontinuity dip in the plane mode of failure.

F_3 relates to the relationship between the slope angle and the discontinuity dip.

The adjustment factor for the method of excavation F_4 depends on whether one deals with a natural slope or one excavated by presplitting, smooth blasting, mechanical excavation, or poor blasting.

The appropriate ratings are given in Table 9.5. The final calculation is of the form

$$\text{Adjusted RMR slope} = \text{RMR}_{\text{basic}} - (F_1 \times F_2 \times F_3) + F_4 \qquad (9.13)$$

Romana (1985) applied this procedure to 28 slopes with varying degrees of instability, including six completely failed ones, and found good agreement with stability assessment (rock mass quality) predicted by the RMR system. He listed all these case histories and stated that further work is under way on several other slopes.

9.4 SPECIAL USES

9.4.1 Rippability

This was the first excavation index to be evaluated by a rock mass classification approach. Based on the Geomechanics Classification, Weaver (1975) proposed a rippability rating chart as a guide for the case of excavation by tractor-mounted rippers of the Caterpillar type. In this approach, seismic velocity was a parameter selected to replace two standard parameters in the RMR system: the intact rock strength and the RQD.

Over a decade later, Smith (1986) modified the chart by Weaver (1975) by omitting seismic velocity, while Singh et al. (1986) discussed ground rippability in open cast mining operations and pointed out that the use of a single value of the seismic velocity can be a misleading parameter in the assessment of the rock rippability. The chart by Weaver (1975), and hence by Smith (1986), while based on many pertinent parameters, was considered of limited value because some parameters might not be easily quantified at the initial stage of design. Accordingly, an alternative rippability rating chart was suggested by Singh et al. (1986) and tested in a number of case histories

CHART 8.2 Roof Support Design Chart #2 for Coal Mines

ENTRY WIDTH: 18-FT

ROCK CLASS	ROCK MASS RATING (RMR)	ROCK LOAD HEIGHT H_T (FT)	SUPPORT SPECIFICATIONS: MECHANICAL BOLTS	SUPPORT SPECIFICATIONS: RESIN BOLTS	SPECIFICATIONS FOR POSTS
I VERY GOOD	90	1.8	L : 2.5' S : 5' x 5' G : 40 ϕ : 5/8" C : 6.2 tons		
	80	3.6	L : 2.5' S : 5' x 5' G : 60 (40) ϕ : 3/4" (7/8") C : 11 tons		
II GOOD	70	5.4	L : 3.0' S : 4.5' x 4' G : 60 ϕ : 3/4" C : 10 tons	L : 2.5' S : 4.5' x 5' G : 40 ϕ : 1" C : 15 tons	
	60	7.2	L : 4.0' S : 4.5' x 5' G : 60 ϕ : 5/8" C : 9 tons	L : 3.0' S : 4.5' x 4.5' G : 60 ϕ : 1" C : 18 tons	ϕ_p = 4.0" S_p = 7.5'
III FAIR	50	9.0	L : 5.0' S : 4.5' x 5' G : 40 ϕ : 3/4" C : 8 tons	L : 4.0' S : 4.5' x 5' G : 60 ϕ : 1" C : 23.7 tons	ϕ_p = 5.5" S_p = 10.0'
	40	10.8	L : 6.0' S : 4.5' x 5' G : 40 ϕ : 3/4" C : 7 tons	L : 4.0' S : 4.5' x 4' G : 60 ϕ : 1" C : 23.7 tons	ϕ_p = 5.0" S_p = 5.0'
IV POOR	30	12.6	L : 7.0' S : 4.5' x 4.5' G : 40 ϕ : 5/8" C : 6 tons	L : 4.0' S : 4' x 4' G : 60 ϕ : 1" C : 23.7 tons	ϕ_p = 5.0" S_p = 4.5'
	20	14.4	L : 8.0' S : 4.5' x 4' G : 40 ϕ : 5/8" C : 5 tons	L : 4.0' S : 4' x 4' G : 60 ϕ : 1-1/4" C : 28.74 tons	ϕ_p = 5.0" S_p = 4.0'

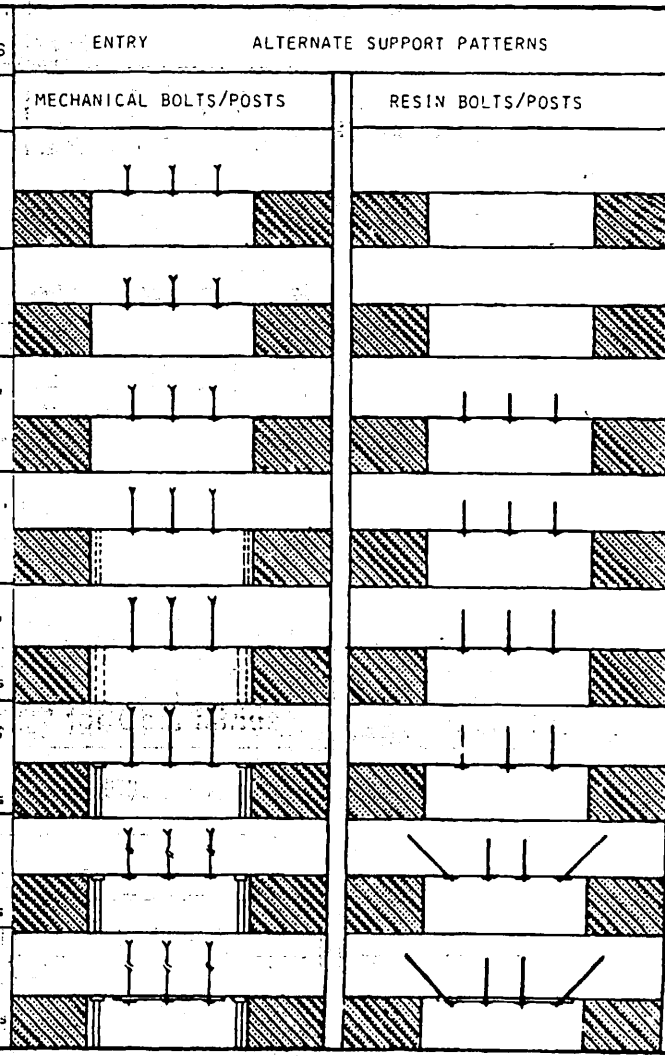


CHART 8.3 Roof Support Design Chart #3 for Coal Mines

ENTRY WIDTH: 16-FT

ROCK CLASS	ROCK MASS RATING (RMR)	ROCK LOAD HEIGHT RMR (FT)	SUPPORT SPECIFICATIONS: MECHANICAL BOLTS	SUPPORT SPECIFICATIONS: RESIN BOLTS	ALTERNATE SUPPORT - PATTERNS: MECHANICAL BOLTS/POSTS	ALTERNATE SUPPORT - PATTERNS: RESIN BOLTS/POSTS	SPECIFICATIONS FOR POSTS
I VERY GOOD	90	1.5	L : 2.5' S : 6' x 5' G : 40 φ : 5/8" C : 6.2 tons			Not economical	
	80	3.2	L : 2.5' S : 6' x 5' G : 60 (40) φ : 3/4" C : 11 tons			Not economical	
II GOOD	70	4.8	L : 3.0' S : 4' x 4.5' G : 60 (40) φ : 3/4" C : 10 tons	L : 2.5' S : 4' x 5' G : 60 φ : 3/4" C : 12 tons			
	60	6.4	L : 4.0' S : 4' x 5' G : 60 φ : 5/8" C : 9 tons	L : 3.0' S : 4' x 5' G : 40 φ : 1" C : 15.6 tons			ϕ_p = 4.0" S_p = 10'
III FAIR	50	8.0	L : 4.0' S : 4' x 5' G : 40 φ : 3/4" C : 8 tons	L : 3.0' S : 4' x 4.5' G : 60 φ : 1" C : 18 tons			ϕ_p = 5.0" S_p = 10'
	40	9.5	L : 5.0' S : 4' x 5' G : 40 φ : 3/4" C : 7 tons	L : 4.0' S : 4' x 5' G : 60 φ : 1" C : 23.7 tons			ϕ_p = 5.5" S_p = 10'
IV POOR	30	11.2	L : 6.0' S : 4' x 5' G : 40 φ : 5/8" C : 6 tons	L : 4.0' S : 4' x 4.5' G : 60 φ : 1" C : 23.7 tons			ϕ_p = 4.5" S_p = 5'
	20	12.8	L : 7.0' S : 4' x 5' G : 40 φ : 5/8" C : 5 tons	L : 4.0' S : 4' x 4' G : 60 φ : 1" C : 23.7 tons			ϕ_p = 5.0" S_p = 5'

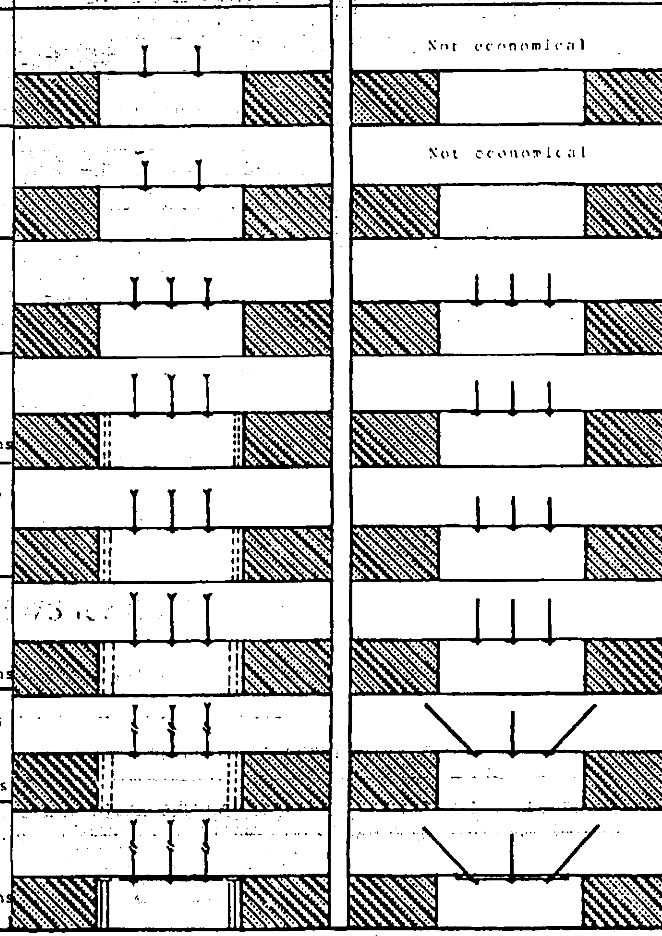
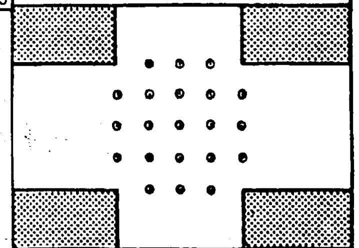
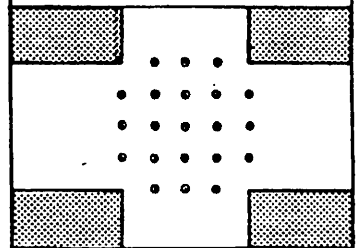
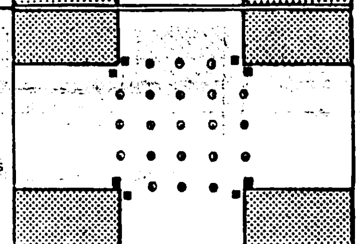
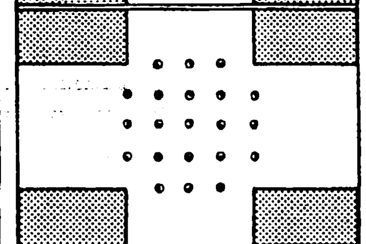
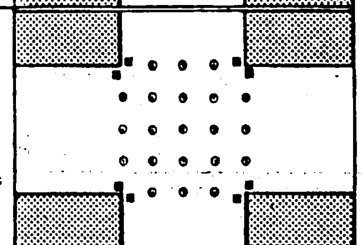
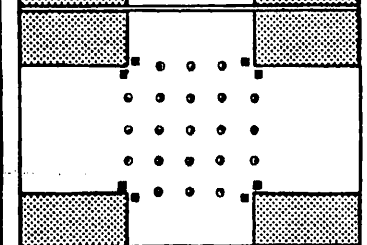
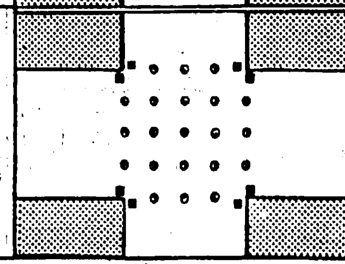
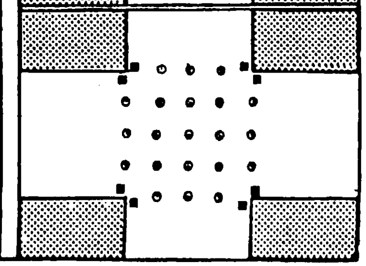


CHART 8.4 Roof Support Design Chart #4 for Coal Mines

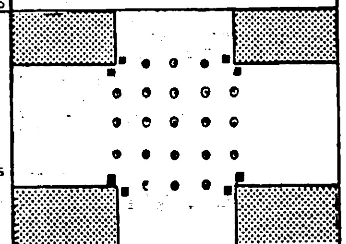
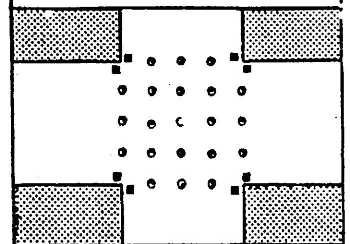
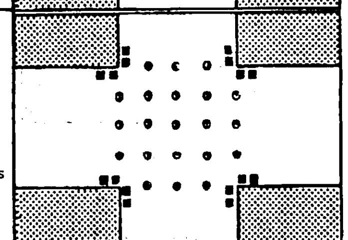
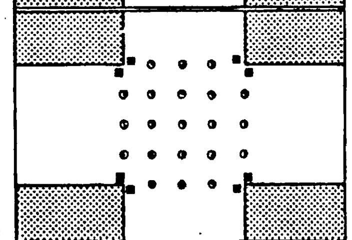
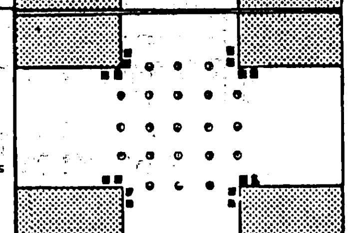
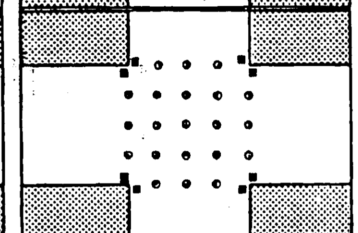
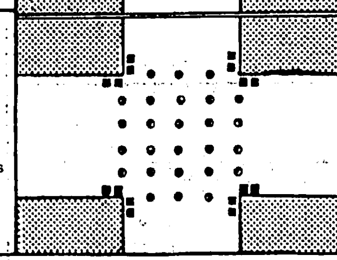
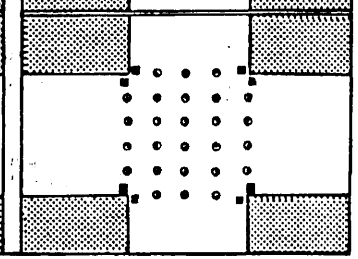
INTERSECTION OF TWO ENTRIES ENTRY WIDTH: 20-FT

ROCK CLASS	ROCK MASS RATING RMR	ROCK LOAD HEIGHT H_R (FT)	SUPPORT SPECIFICATIONS: MECHANICAL BOLTS/POSTS	SUPPORT SPECIFICATIONS: RESIN BOLTS/POSTS	ALTERNATE SUPPORT - PATTERNS: MECHANICAL BOLTS/POSTS	ALTERNATE SUPPORT - PATTERNS: RESIN BOLTS/POSTS
I VERY GOOD	90	2.8	L : 2.5' S : 5' x 5' G : 40 ϕ : 3/4" C : 8.8 tons	L*: 3.0'/3.0' S : 5' x 5' G : 40 ϕ : 3/4" C : 8.8 tons		
	80	5.7	L : 3.0' S : 5' x 5' G : 40 ϕ : 5/8" C : 6.2 tons ϕ_p: 4"	L*: 4.0'/4.0' S : 5' x 5' G : 60 ϕ : 1" C : 23.7 tons		
II GOOD	70	8.5	L : 5.0' S : 5' x 5' G : 40 ϕ : 3/4" C : 8.8 tons ϕ_p: 5"	L*: 3.0'/5.0' S : 5' x 5' G : 60 ϕ : 3/4" C : 13.2 tons ϕ_p: 4.5"		
	60	11.3	L : 6.0' S : 5' x 5' G : 40 ϕ : 3/4" C : 8.8 tons ϕ_p: 6"	L*: 3.5'/6.0' S : 5' x 5' G : 60 ϕ : 3/4" C : 13.2 tons ϕ_p: 5.5"		

L*: Resin Length/Bolt Length

CHART 8.5 Roof Support Design Chart #5 for Coal Mines

INTERSECTION OF TWO ENTRIES ENTRY WIDTH: 20-FT

ROCK CLASS	ROCK MASS RATING RMR	ROCK LOAD HEIGHT H_R (FT)	SUPPORT SPECIFICATIONS: MECHANICAL BOLTS/POSTS	SUPPORT SPECIFICATIONS: RESIN BOLTS/POSTS	ALTERNATE SUPPORT - PATTERNS: MECHANICAL BOLTS/POSTS	ALTERNATE SUPPORT - PATTERNS: RESIN BOLTS/POSTS
III FAIR	50	14.2	L : 7.0' S : 5' x 5' G : 40 ϕ : 5/8" C : 6.2 tons ϕ_p: 7"	L*: 3.5'/6.0' S : 5' x 5' G : 60 ϕ : 3/4" C : 13.2 tons ϕ_p: 6.5"		
	40	17.0	L : 9.0' S : 5' x 5' G : 40 ϕ : 5/8" C : 6.2 tons ϕ_p: 5.5"	L*: 3.5'/7.0' S : 5' x 5' G : 40 ϕ : 1" C : 15.8 tons ϕ_p: 7"		
IV POOR	30	19.8	L : 10.0' S : 5' x 5' G : 40 ϕ : 5/8" C : 6.0 tons ϕ_p: 6.0"	L*: 4.5'/8.0' S : 5' x 5' G : 60 ϕ : 1" C : 23.7 tons ϕ_p: 7"		
	20	22.6	L : 12.0' S : 5' x 4' G : 40 ϕ : 5/8" C : 5.0 tons ϕ_p: 6.5"	L*: 5.0'/8.0' S : 5' x 4' G : 60 ϕ : 1" C : 23.7 tons ϕ_p: 7"		

L*: Resin Length/Bolt Length

overlying and underlying strata, and the geometry of the opening are as follows:

Depth of Coal Seam Below Surface: 152 m

Stratigraphic Column:

Immediate roof: soft shale—4.5 m thick; average thickness of the layers in the roof 150 mm

Coal seam: 3.0 m thick

Floor: Fire clay

Tests have been carried out on the roof strata and the coal seam yielding the following property data:

Data	Coal	Soft Shale	Hard Shale
Thickness (m)	3.0	4.5	27.5
Unit weight (kN/m^3)	12.5	25.1	26.7
Compressive strength (MPa)	17.00	40.00	81.00
Roof strata conditions	Not applicable	Separation < 1 mm; slightly weathered, slightly rough surfaces; no infilling RQD ≈ 60%	
Groundwater conditions	Damp	Damp	Damp
In-situ stresses	Horizontal stress = 2.5 × (vertical stress)		

Solution Determination of the rock mass rating (RMR) for roof strata. In accordance with the Geomechanics Classification, the following ratings are obtained for the classification parameters:

Strength of intact rock (soft shale): 40 MPa	Rating = 5
Spacing of discontinuities: 150 mm	Rating = 7
Rock quality designation (RQD): 60%	Rating = 12
Condition of discontinuities: separation <1 mm, slightly weathered and slightly rough surfaces	Rating = 17
Groundwater conditions: damp throughout	Rating = 10
	Basic RMR = 51
Adjustment for discontinuity orientation: (horizontal bedding = fair orientation)	−5

Adjustment for in-situ state of stress: From the overburden depth, the vertical stress is 3.8 MPa and the horizontal stress (being 2.5 times this value) is 9.5 MPa. The ratio of the horizontal stress to the uniaxial compressive strength is 0.24. Using data from Newman and Bieniawski

TABLE 9.5 Modification of the Geomechanics Classification for Rock Slopes[a]

Bieniawski (1979) Ratings for RMR

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point-load strength index (MPa)	>10	4–10	2–4	1–2	For this low range, uniaxial compressive strength test is preferred		
		Uniaxial compressive strength (MPa)	>250	100–250	50–100	25–50	5–25	1–5	<1
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD (%)		90–100	75–90	50–75	25–50	<25		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6–2 m	200–600 mm	60–200 mm	<60 mm		
	Rating		20	15	10	8	5		

TABLE 9.5 (*Continued*)

Adjustment Rating for Methods of Excavation of Slopes

Method	Natural Slope	Presplitting	Smooth Blasting	Regular Blasting	Deficient Blasting
F_4	+15	+10	+8	0	−8

$SMR = RMR - (F_1 \times F_2 \times F_3) + F_4$

Tentative Description of SMR Classes

Class No.	V	IV	III	II	I
SMR	0–20	21–40	41–60	61–80	81–100
Description	Very poor	Poor	Fair	Good	Very good
Stability	Very unstable	Unstable	Partially stable	Stable	Fully Stable
Failures	Large planar or soil-like	Planar or large wedges	Some joints or many wedges	Some blocks	None
Support	Reexcavation	Extensive corrective	Systematic	Occasional	None

[a] By Romana (1985).

	Condition of discontinuities	Very rough surfaces. Not continuous. No separation. Unweathered wall rock	Slightly rough surfaces. Separation < 1 mm. Slightly weathered walls	Slightly rough surfaces. Separation < 1mm. Highly weathered walls	Slickensided surfaces. Or Gouge < 5 mm thick. Or Separation 1–5 mm Continuous	Soft gouge > 5 mm or Separation > 5 mm Continuous
4	Rating	30	25	20	10	0
	Groundwater in joint	Completely dry	Damp	Wet	Dripping	Flowing
5	Rating	15	10	7	4	0

Joint Adjustment Rating for Joints[b]

Case		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
P	$\lvert\alpha_j - \alpha_s\rvert$	>30°	30–20°	20–10°	10–5°	<5°
T	$\lvert\alpha_j - \alpha_s - 180°\rvert$					
P/T	F_1	0.15	0.40	0.70	0.85	1.00
P	$\lvert\beta_j\rvert$	<20°	20–30°	30–35°	35–45°	>45°
P	F_2	0.15	0.40	0.70	0.85	1.00
T	F_2	1	1	1	1	1
P	$\beta_j - \beta_s$	>10°	10–0°	0°	0°–(–10°)	<–10°
T	$\beta_j + \beta_s$	<110°	110–120°	>120°		
P/T	F_3	0	–6	–25	–50	–60

P = plane failure. α_s = slope dip direction. α_j = joint dip direction.
T = toppling failure. β_s = slope dip. β_j = joint dip.

(*Table continues on p. 190.*)

in Great Britain and Turkey. This chart is depicted in Table 9.6. based on a later publication (Singh et al., 1987) which demonstrated the application of this approach to the selection of rippers for surface coal mines.

9.4.2 Dredgeability

Dredgeability as applied to rock was defined by Smith (1987) as the ability to excavate rock underwater with respect to known or assumed equipment, methods, and in-situ characteristics. Dredging is a multimillion dollar operation in which breaking up or cutting the rock underwater requires an assessment of the rock mass quality in a similar way to rippability assessment. However, while the same parameters may be expected to govern, a given rock mass ripped underwater will usually be weaker than the same rock encountered in dry conditions due to the influence of water on the strength of rock.

Smith (1987) proposed an underwater rippability rating chart modifying the work of Weaver (1975), whose proposal, in turn, was based on the Geomechanics Classification. Smith's modification omitted not only the seismic velocity parameter used by Weaver, but also the joint continuity and joint gouge parameters, which, unlike for surface excavations, are not readily available in dredging applications. Table 9.7 depicts Smith's dredgeability chart, which, due to the above omissions, features the maximum underwater rippability (RW) rating of 65, compared with a maximum possible RMR of 100. This system provides a quantitative estimate of relative ripping difficulty, with the lower ratings corresponding to easier ripping and higher ratings to harder ripping or blasting. Since RW does not involve seismic velocity observations, it can be used as a means of independent comparison with the refraction method.

9.4.3 Excavatability

Excavatability, a term denoting ease of excavation, was extensively discussed by Kirsten (1982), who pointed out that seismic velocity was in general poorly correlated to the excavatability of a material because a whole range of the basic material characteristics that affect excavatability were not represented in the seismic velocity. Moreover, seismic velocity could not be determined to an accuracy better than about 20%, and it might have a variance of the order of 1000 m/s in apparently identical materials.

Kirsten proposed a classification system for excavation in natural materials in which the excavatability index N is given by

$$N = M_s \frac{\text{RQD}}{J_n} J_s \frac{J_r}{J_a} \tag{9.14}$$

TABLE 9.6 Rippability Classification Chart[a]

Parameters	Class 1	Class 2	Class 3	Class 4	Class 5
Uniaxial tensile strength (MPa)	<2	2–6	6–10	10–15	>15
Rating	0–3	3–7	7–11	11–14	14–17
Weathering	Complete	Highly	Moderate	Slight	None
Rating	0–2	2–6	6–10	10–14	14–18
Sound velocity (m/s)	400–1100	1100–1600	1600–1900	1900–2500	>2500
Rating	0–6	6–10	10–14	14–18	18–25
Abrasiveness	Very low	Low	Moderate	High	Extreme
Rating	0–5	5–9	9–13	13–18	18–22
Discontinuity spacing (m)	<0.06	0.06–0.3	0.3–1	1–2	>2
Rating	0–7	7–15	15–22	22–28	28–33
Total Rating	<30	30–50	50–70	70–90	>90
Ripping Assessment	Easy	Moderate	Difficult	Marginal	Blast
Recommended Dozer	Light duty	Medium duty	Heavy duty	Very heavy duty	

[a] After Singh (1987).

TABLE 9.7 Underwater Rippability (Dredgeability) Rating Chart[a]

Descriptive Classification	Rock Hardness[b] (MPa)	Rock Weathering	Orientation	Joint Spacing[c]
Very hard ripping or blasting	>70	Unweathered	Very favorable	$>3D$
Rating	10	10	15	30
Hard ripping	25–70	Slightly weathered	Unfavorable	D to $3D$
Rating	5	7	13	25
Average ripping	10–25	Weathered	Slightly unfavorable	$D/3$ to D
Rating	2	5	10	20
Easy ripping	3–10	Highly weathered	Favorable	$D/20$ to $D/3$
Rating	1	3	5	10
Very easy ripping	<3	Completely weathered	Very favorable	$<D/20$
Rating	0	1	3	5

[a] After Smith (1987).
[b] Corresponding to uniaxial compressive strength.
[c] Expressed as function of depth D.

where M_s = mass strength number, denoting the effort to excavate the material as if it were homogeneous, unjointed, and dry. Thus, M_s approximates the uniaxial compressive strength of rock in MPa;

RQD = rock quality designation (see Chap. 3);

J_n and J_r = the parameters from the Q-system (see Chap. 5);

J_s = relative ground structure number, representing the relative orientation of individual blocks to the direction ripping. For intact material, $J_s = 1.0$.

Once the excavatability index N is obtained from the above equation, it serves to classify the ease of excavation in rock as follows:

$1 < N < 10$	Easy ripping
$10 < N < 100$	Hard ripping
$100 < N < 1{,}000$	Very hard ripping
$1{,}000 < N < 10{,}000$	Extremely hard ripping/blasting
$N > 10{,}000$	Blasting

Ease of excavation was also studied by Abdullatif and Cruden (1983), who investigated methods of excavation featuring digging, ripping, and blasting at 23 sites and classified rock mass quality in terms of RMR and Q. Their findings are shown in Figure 9.2, which indicates quite distinct clusters of points for different methods of excavation. For example, it can be seen that rock mass can be dug up to an RMR of 30 and ripped up to an RMR of 60. Rock masses rated as "good" or better by the RMR system must be blasted.

There is also a distinct gap between the Q values of rock masses that can be dug, Q up to 0.14, and those requiring ripping, Q above 1.05. Abdullatif and Cruden (1983) observed that there was an overlap in Q values between 3.2 and 5.2 of rock masses that could be ripped and rock masses requiring blasting. They suggested that the reason the Q-system appears to present problems as a guide for excavatability of rock was that the active stress parameter J_w/SRF, while important in tunneling, shows little variation in rock masses at the surface.

9.4.4 Cuttability

Cuttability of rock is particularly important when using roadheaders—boom-type tunneling machines. According to Fowell and Johnson (1982), interpretation of borehole information at the site-investigation stage for predicting roadheader cutting rates was facilitated by the use of rock mass classifications.

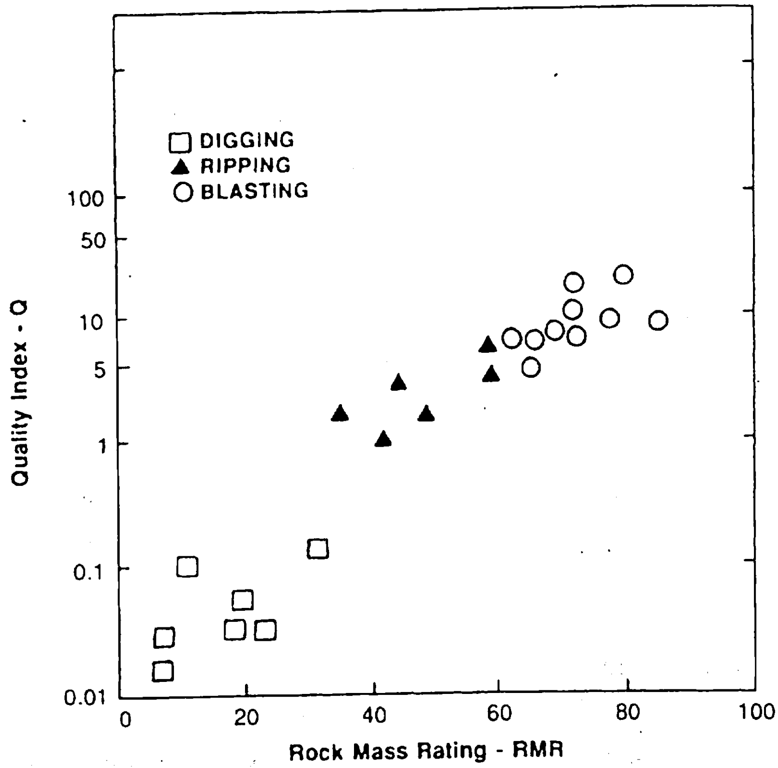


Figure 9.2 *Rock mass quality classification diagram (based on RMR and Q indexes) depicting various excavation methods on sites. (After Abdullatif and Cruden, 1983.)*

Based on 20 field results, Fowell and Johnson (1982) derived a relationship between the RMR values and the cutting rate in m^3/h for the heavyweight class of boom tunneling machines. The results are given in Figure 9.3, and the authors report that the only modification they made in the use of the Geomechanics Classification was in the rating for orientation, since, for excavation in general, an inverse relationship exists between support re-

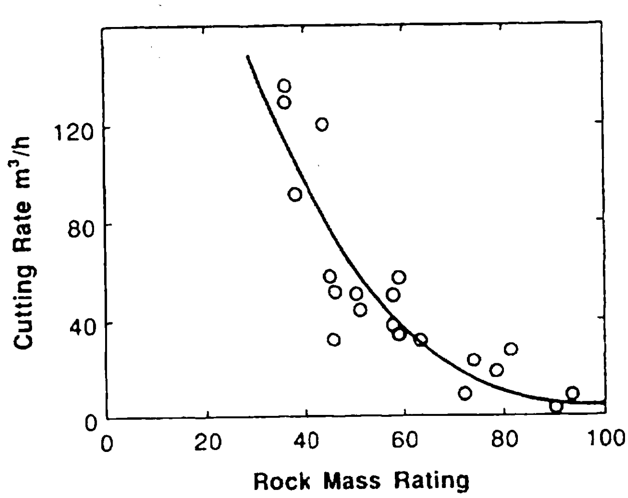


Figure 9.3 *Relationship between RMR and rock cutting rate. (After Fowell and Johnson, 1982.)*

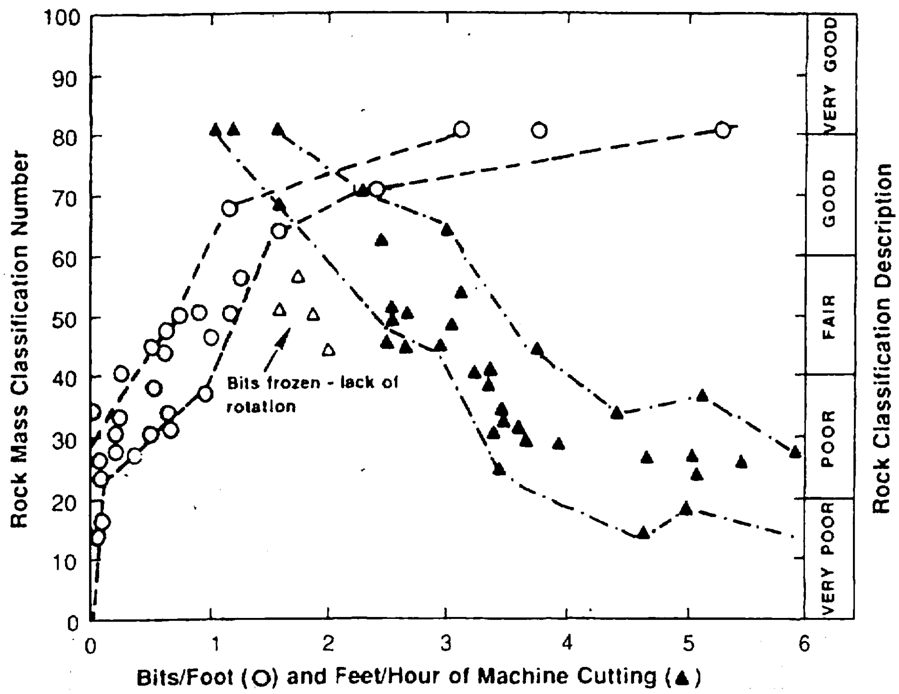


Figure 9.4 *Roadheader performance data, bits/ft and ft/h of machine cutting at San Manuel Mine in Arizona, 2375-ft level, P21A and P21B test. (After Sandbak, 1985.)*

quirements and ease of excavation. It can be concluded that the RMR system provided a remarkably consistent relationship with the roadheader cutting rate.

Sandbak (1985) also evaluated rock cutting performance by a roadheader relating it to the rock mass quality described in terms of the Geomechanics Classification. This was an extensive investigation conducted at the San Manuel copper mine in Arizona, and on the basis of 1430 ft (436 m) of drift excavation in variable rock conditions, the advance rates by the roadheader (DOSCO SL-120) were shown to be predictable from RMR values.

The results are given in Figure 9.4. It is apparent that the bits per foot rate and the feet per cutting hour rate can be effectively related to RMR values and rock mass classes. More recently, Stevens et al. (1987) presented RMR zoning plans of the San Manuel Mine, while Sandbak (1988) built on the success of the RMR-based evaluation of roadheader drift excavation and upgraded the approach to include it in the LHD (load–haul–dump) system design and in pillar sizing.

9.4.5 Cavability

Cavability of rock strata is an important aspect in longwall mining of coal as well as metal mining operations involving block caving.

TABLE 9.8 Cavability Estimates[a]

	RMR Class				
	1	2	3	4	5
Area undercut as "hydraulic radius"	NA[a]	30 m	20–30 m	8–20 m	<8 m
Cavability	Nil	Poor	Fair	Good	Very Good
Fragmentation	Nil	Large	Medium	Small	Very Good

[a] After Laubscher (1981).
[b] Not applicable.

Rock classifications have been used for this purpose (Laubscher, 1981; Bieniawski, 1987). Most recently, an important contribution was made by Ghose and Gupta (1988).

Laubscher (1981) used the Geomechanics Classification to assess cavability in asbestos mines and suggested a correlation between the RMR classes and caving as well as fragmentation characteristics. He also included estimates of the "hydraulic radius" in caving operations, which is defined as the caving area divided by the perimeter and serves to define the undercut area. The guidelines are summarized in Table 9.8.

Kidybinski (1982) and Unrug and Szwilski (1983) described a cavability classification used by coal mines in Poland. This classification is depicted in Table 9.9.

TABLE 9.9 Roof Cavability Classification Based on Polish Studies[a]

Roof Class	Roof Quality Index[b]	Allowable Area of Roof Exposure (m^2)
I Very weak	$L < 18$	1
II Little stable	$18 < L < 35$	1–2
III Medium stable	$35 < L < 60$	2–5
IV Stable	$60 < L < 130$	5–8
V Very strong	$L > 130$	>8

[a] After Kidybinski (1982) and Unrug and Szwilski (1983).
[b] Roof quality index $L = 0.016\,\sigma_M d$,

where σ_M = in-situ compressive strength of rock strata (kg/cm^2) = $\sigma_c K_1 K_2 K_3$,
σ_c = uniaxial compressive strength,
K_1 = 0.4 (coefficient of strength utilization),
K_2 = 0.7 (coefficient of creep),
K_3 = 50% (coefficient of moisture content),
d = mean thickness of roof strata layers (cm).

TABLE 9.10 Cavability Classification for Coal-Measure Strata[a]

Class	Cavability	Cavability Rating[b]	Caving Behavior
I	Extremely high	0–30	Very easy caving
II	High	31–45	Easy caving
III	Moderate	46–60	Moderately caving, poor in big blocks
IV	Low	61–70	Difficult caving, overhanging roof
V	Extremely low	71–100	Very difficult caving, large overhang

[a] After Ghose and Gupta (1988).
[b] Roof caving span $S = 0.87R - 10.1$, where R is the cavability value.

Cavability can also be evaluated by the RMR classification from the relationship between the rock stand-up time versus the unsupported span for the five rock mass classes, as shown in Figure 4.1.

Ghose and Gupta (1988) outlined a classification system for roof strata cavability using fuzzy-set methodology to assign ratings for four individual parameters: uniaxial compressive strength, average core size, thickness of roof beds, and depth below surface.

This classification model was applied to ten longwall case histories from Indian coal fields and resulted in the description given in Table 9.10.

9.5 IMPROVING COMMUNICATION: UNIFIED ROCK CLASSIFICATION SYSTEM

Williamson (1980, 1984) proposed the Unified (initially called "Uniform") Rock Classification System (URCS) as a reliable and rapid method of communicating detailed information about rock conditions for engineering purposes. The system has been used extensively by the Soil Conservation Service of the U.S. Department of Agriculture for classifying and describing information on rock materials (Kirkaldie et al., 1988).

The URCS consists of four physical properties: a) weathering, b) strength, c) discontinuities, and d) density. A general assessment of rock performance is then based on a grouping of these key elements to aid in making engineering judgments. These individual properties are estimated in the field with the use of a hand lens, a 1-lb (0.5-kg) ball peen hammer, a spring-loaded "fish" scale, and a bucket of water. Each property is divided into five ratings which convey uniform meaning to geologists, engineers, inspectors, and contractors as well as contract-appeal board members.

Subjective terminology, such as "slightly weathered, moderately hard, highly fractured, and lightweight," varies widely in meaning, depends on individual and professional experience, and cannot be quantified with any

TABLE 9.11 Unified Rock Classification System[a]

Degree of Weathering

Representative		Altered	Weathered			
			> Gravel Size		< Sand Size	
Micro fresh state (MFS) A	Visually fresh state (VFS) B	Stained state (STS) C	Partly decomposed state (PDS) D		Completely decomposed state (CDS) E	
Unit Weight Relative Absorption		Compare to Fresh State	Nonplastic	Plastic	Nonplastic	Plastic

Estimated Strength[b]

Reaction to Impact of 1 lb Ball Peen Hammer				Remolding[c]
"Rebounds" (elastic) (RQ) A	"Pits" (tensional) (PQ) B	"Dents" (compression) (DQ) C	"Craters" (shears) (CQ) D	Moldable (friable) (MQ) E
>15,000 psi >103 MPa	8,000–15,000 psi 55–103 MPa	3,000–8,000 psi 21–55 MPa	1,000–3,000 psi 7–21 MPa	<1,000 psi <7 MPa

(*Table continues on p. 200.*)

TABLE 9.11 *(Continued)*

Discontinuities

Very Low Permeability			May Transmit Water	
Solid (random breakage) (SRB)	Solid (preferred breakage) (SPB)	Solid (latent planes of separation) (LPS)	Nonintersecting open planes (2-D) D	Intersecting open planes (3-D) E
A	B	C	Attitude	Interlock

Unit Weight

Greater than 160 pcf 2.55 g/cm^3 A	150–160 pcf 2.40–2.55 g/cm^3 B	140–150 pcf 2.25–240 g/cm^3 C	130–140 pcf 2.10–2.25 g/cm^3 D	Less than 130 pcf 2.10 g/cm^3 E

Design Notation

Weathering — Weight

$\boxed{\text{A–E}}$ $\boxed{\text{A–E}}$ $\boxed{\text{A–E}}$ $\boxed{\text{A–E}}$

Strength — Discontinuity

[a] After Williamson (1980, 1984).

[b] Strength estimated by soil mechanics techniques.

[c] Approximate unconfined compressive strength.

reliability. The URCS is not intended to supplant the existing rock mass classifications but assists when descriptive terminology is ambiguous.

The URCS is depicted in Table 9.11.

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10

Case Histories Data Base

It is truth very certain that when it is in our power to determine what is true, we ought to follow what is most probable.
—René Descartes

The case histories used in the development and validation of the Geomechanics Classification (RMR system) are tabulated in this chapter. Originally, 49 case histories were investigated in 1973, followed by 62 coal mining case histories that were added by 1984 and a further 78 tunneling and mining case histories collected by 1987. To date, the RMR system has been used in 351 case histories.

To assist the readers in deciding whether their particular project site conditions fall within the range of data applicable to the RMR system, a summary of the case histories, featuring the principal data, is presented. Names of projects have been omitted at the owners' request. However, since this is abbreviated information, an example of the actual data sheet used in record keeping is shown in Figure 10.1. This data sheet is accompanied by the details of the geological conditions encountered and the support installed.

The tabulated RMR case histories are presented here in order of the RMR magnitude, from the highest to the lowest. However, all the records are stored using the MacWorks data base software for a Macintosh personal computer and can be retrieved and sorted by any item appearing in the heading of the tabulation (i.e., project type, span depth, etc.).

Accordingly, to demonstrate the RMR data base ranges, histograms are given in Figures 10.2–10.4 depicting the ranges of the RMR values, spans of excavations, and depths below surface applicable to the case histories on the basis of which the RMR system was developed.

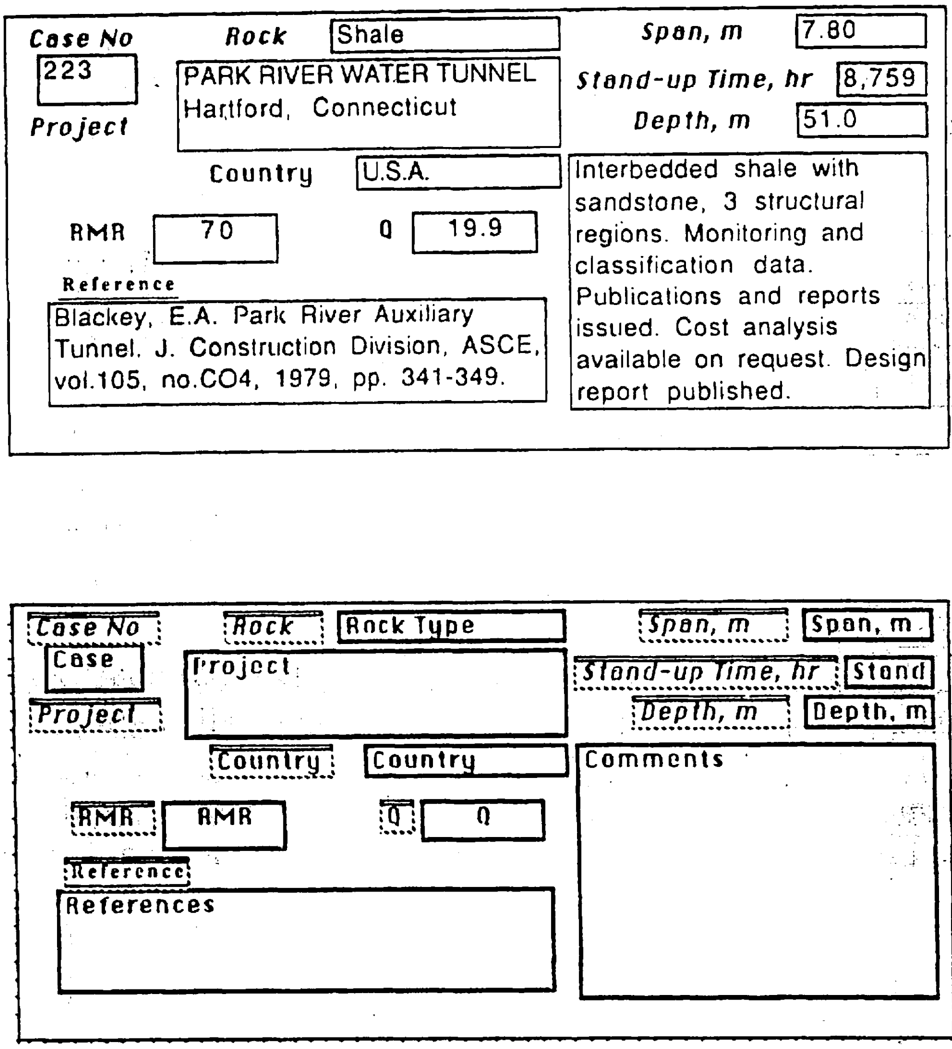

Figure 10.1 *A record-keeping form for RMR case histories.*

Listing of RMR Case Histories

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
2	gneiss	chamber	94	200.000	20.0		18
58	limestone	limestone mine	91		13.1	403000	686
256	lava	metal mine	88		16.8		1800
334	dunite	hard rock mine	86		25.0	175200	385
1	Shale	Railroad tunnel	86	103.000	7.4	87590	28
250	quartzite	metal mine	85		16.5		1494
284	siltstone	tunnel	85		6.0		200
99	dolerite	tunnel	83	33.300	5.5		51
264	gneiss	chamber	82		33.0		350
76	salt	chamber	82		44.0		338
283	sandstone	tunnel	82		6.0		200
248	dolerite	chamber	80		25.0		30
113	sandy shale	foundation	80	54.600	5.5		350
48	argillite	chamber	79		21.7		25
321	siltstone	tunnel	79		6.0		298
16	granite & gneiss	tunnel	78	16.700	14.6		442
251	quartzite	metal mine	78		12.0		2378
285	siltstone	tunnel	78		6.0		200
253	quartzite	metal mine	77		4.9		2650
57	dolomite	sewage tunnel	76		10.8		67
108	gneiss	tunnel	76	12.000	8.3		26
254	quartzite	metal mine	76		7.8		2100
238	sericite	metal mine	76	22.600	4.0		183
265	gneiss	chamber	75		29.0		171
245	greywacke	chamber	75		22.0		156
229	melaphyre	rock slope	75				191
103	metaphyre	tunnel	75	5.300	3.3	35040	217
252	quartzite	metal mine	75		9.8		2073
224	basalt	tunnel	74	11.250	7.8		61
3	granite	chamber	74	50.000	12.0		100
263	greywacke	chamber	74		33.5		300
181	quartzite	tunnel	74		4.6	3936	41
255	quartzite	metal mine	74		10.0		1700

Listing of RMR Case Histories (*Continued*)

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
263	greywacke	chamber	74		33.5		300
320	siltstone	tunnel	74		6.0		263
71	oil shale	oil shale mine	73	4.800	18.0		290
340	schist	chamber	73		12.0		150
52	siltstone	water tunnel	72		3.4		671
63	coal	coal mine	72		3.2		28
83	gneiss & granite	tunnel	72	11.300	15.5		70
182	dolerite	tunnel	72		5.0	720	29
209	granite	tunnel	72		12.0	480	70
210	gneiss	tunnel	72		3.0	1440	68
218	basalt	chamber	72	2.810	6.0		924
219	tuff	chamber	72	10.000	6.1		384
258	quartzite	metal mine	72		16.0		2092
260	gneiss	chamber	72		23.0		60
14	granite	chamber	71	50.000	23.5		335
104	dolerite	foundation	71	2.800			23
190	gneiss	tunnel	71		2.5		67
237	monzonite	metal mine	71	3.900	4.0		183
261	mudstone	chamber	71		13.7		152
319	sandstone	tunnel	71		6.0		210
337	argillite	chamber	71		21.5		25
97	dolerite	tunnel	70	12.500	5.5		48
128	shale	coal mine	70		15.3	2136	143
134	shale	coal mine	70		9.3	3000	152
139	shale	coal mine	70		9.9	2956	168
223	shale	tunnel	70	19.900	7.8	8759	51
236	porphyry	metal mine	70	5.000	4.3		214
247	gneiss	chamber	70		16.0		140
257	quartzite	metal mine	70		16.6		2750
259	granite	chamber	70		23.0		335
270	greywacke	tunnel	70		3.0		150
271	greywacke	tunnel	70	35.000	3.0		150
275	greywacke	tunnel	70		3.0		150

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
55	amphibolite	chamber	69		21.7		92
222	tuff	chamber	69	0.700	6.1		921
273	greywacke	tunnel	69		3.0		150
277	greywacke	tunnel	69		3.0		150
88	granite	chamber	68	20.000	10.0	8760	102
127	shale	coal mine	68		9.0	2568	154
133	shale	coal mine	68		6.6	2424	152
149	shale	coal mine	68		8.4	4944	171
156	shale	coal mine	68		9.0	3096	193
240	porphyry	metal mine	68	0.800	3.7		275
262	quartzite	chamber	68		22.0		200
329	monzonite	metal mine	68		20.0	240	706
345	schist	metal mine	68		6.0		76
126	shale	coal mine	67		12.0	2136	154
135	shale	coal mine	67		9.0	1632	160
140	shale	coal mine	67		7.8	1224	152
246	mudstone	chamber	67		16.3		150
61	sandstone	coal mine	66		4.5		150
138	shale	coal mine	66		10.8	1488	157
220	tuff	chamber	66	4.300	6.1		545
287	sandstone	tunnel	66		6.0		200
290	shale	tunnel	66		6.0		200
87	granite	chamber	65	16.900	19.0	3600	108
131	shale	coal mine	65		6.0	4824	152
137	shale	coal mine	65		9.6	1440	171
217	basalt	chamber	65	0.190	6.0		897
267	greywacke	tunnel	65		3.0		150
89	quartzite	tunnel	64	0.900	14.3		41
125	shale	coal mine	64		6.0	2160	154
136	shale	coal mine	64		8.4	2544	160
144	shale	coal mine	64		9.0	1320	156
146	shale	coal mine	64		9.9	1344	159
158	shale	coal mine	64		9.9	1032	125

Listing of RMR Case Histories (*Continued*)

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
289	shale	tunnel	64		10.0		225
15	gneiss	chamber	63	31.600	19.5		305
17	gneiss	chamber	63	31.600	24.7		299
62	shale	coal mine	63		3.5		92
101	dolerite	rock slope	63	3.000			73
124	shale	coal mine	63		11.4	792	154
152	shale	coal mine	63		12.0	600	122
227	dolerite	rock slope	63				31
286	sandstone	tunnel	63		6.0		200
348	sandstone	coal mine	63		3.6		54
77	sandstone	tunnel	62	0.800	15.0		200
79	sandstone	chamber	62	6.600	30.0		101
94	dolerite	tunnel	62	13.200	5.5		82
109	quartz-mica schist	tunnel	62	7.500	3.7		27
119	shale	coal mine	62		11.4	792	154
141	shale	coal mine	62		8.4	1200	137
192	gneiss	chamber	62		18.0	3600	108
202	mudstone	chamber	62		3.0	96	100
203	mudstone	chamber	62		2.0	168	100
208	sandstone	tunnel	62		11.4	72	200
241	porphyry	metal mine	62	1.300	3.7		330
288	shale	tunnel	62		10.0		225
49	gneiss	subway tunnel	61		6.1		19
59	quartzite	railroad tunnel	61		6.0		457
291	shale	tunnel	61		6.0		200
12	leptite	tunnel	60	35.000	9.0		9
92	diabase	tunnel	60	2.160	14.3		40
142	shale	coal mine	60		11.4	384	152
281	phyllite	tunnel	60		5.8		102
344	gneiss	tunnel	60		3.0		100
51	andesite	water tunnel	59		7.2		229
186	gneiss	tunnel	59		3.0	1440	67
189	gneiss	tunnel	59		2.8	2160	67

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
221	tuff	chamber	59	0.600	6.1		762
308	mudstone	tunnel	59		6.0	66	40
324	siltstone	tunnel	59		6.0		394
37	gneiss	tunnel	58	2.800	9.1		24
56	siltstone	shaft	58	0.400	3.7		684
130	shale	coal mine	58		7.8	600	152
143	shale	coal mine	58		12.3	240	156
188	granite	tunnel	58		2.8	4300	67
191	gneiss	tunnel	58		2.8	720	67
272	greywacke	tunnel	58		3.0		150
278	greywacke	tunnel	58		3.0		150
22	shale	coal mine	57		4.2		80
28	sandstone	coal mine	57		4.0		160
39	sandstone	coal mine	57		3.6		150
187	biotite	tunnel	57		3.0	4320	67
242	andesite	metal mine	57	0.300	3.7		275
282	phyllite	shaft	57		3.0		102
338	shale	coal mine	57		2.5		300
351	granite	tunnel	57	1.740	6.0	168	400
4	granite	tunnel	56	12.000	8.0		15
86	sandstone	tunnel	56	1.650	15.5		70
107	gneiss	tunnel	56	1.000	3.9		20
132	shale	coal mine	56		6.0	624	152
157	shale	coal mine	56		6.0	456	145
211	sandstone	tunnel	56		5.0	48	71
274	greywacke	tunnel	56		3.0		150
306	mudstone	tunnel	56		6.0	16	49
312	mudstone	tunnel	56		6.0	1056	8
323	siltstnoe	tunnel	56		6.0		365
326	mudstone	tunnel	56		6.0		173
327	mudstone	tunnel	56		6.0		98
346	shale	coal mine	56		3.8		810
34	shale	coal mine	55		4.5		60

Listing of RMR Case Histories (*Continued*)

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
35	shale	coal mine	55		4.5		40
36	shale	coal mine	55		3.2		30
44	tuff	chamber	55	1.500	30.5		401
216	basalt	chamber	55	2.330	6.0		924
243	fanglomerite	metal mine	55	0.200	3.7		330
293	shale	tunnel	55		6.0		200
295	shale	tunnel	55		6.0		200
20	gneiss	chamber	54	5.200	19.5		295
33	gneiss	chamber	54	5.200	24.7		300
42	granite	tunnel	54	1.900	14.6		442
47	coal	coal mine	54		3.5		141
151	shale	coal mine	54		5.4	408	146
279	greywacke	tunnel	54		3.0		150
292	shale	tunnel	54		6.0		200
296	shale	tunnel	54		6.0		200
32	shale	coal mine	53		3.2		20
43	sandstone	coal mine	53		4.2		390
50	sandstone	railroad tunnel	53		7.4		58
155	shale	coal mine	53		6.0	120	92
276	greywacke	tunnel	53		3.0		150
309	mudstone	tunnel	53		6.0	168	52
313	siltstone	tunnel	53		6.0	456	9
315	mudstone	tunnel	53		6.0	12	162
317	mudstone	tunnel	53		6.0	26280	159
322	siltstone	tunnel	53		6.0		310
325	siltstone	tunnel	53		6.0		211
328	mudstone	tunnel	53		6.0		56
84	granite	tunnel	52	0.690	15.5		65
95	dolerite	tunnel	52	10.000	5.5		56
212	granite	tunnel	52		5.0	24	69
231	porphyry	metal mine	52	1.250	3.7		755
336	argillite	chamber	52		21.5		25
341	sandstone	tunnel	52		6.0		210

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
21	coal & shale	coal mine	51		3.8		92
67	sandstone	coal mine	51		4.2		220
123	shale	coal mine	51		4.8	124	154
294	shale	tunnel	51		6.0		200
6	granite	tunnel	50	2.600	7.0		20
46	shale	coal mine	50		4.5		225
114	claystone	foundation	50	1.950	4.9		232
118	sandy shale	foundation	50	1.950	6.1		609
162	shale	coal mine	50		9.2	67	152
193	mudstone	tunnel	50		1.5	5	175
194	mudstone	tunnel	50		4.0	5	175
195	mudstone	tunnel	50		5.5	15	171
249	shale	chamber	50		20.0		45
311	mudstone	tunnel	50		6.0	6	69
314	mudstone	tunnel	50		6.0	312	103
349	sandstone	coal mine	50		3.6		375
18	gneiss	tunnel	49	5.300	6.1		330
19	gneiss	chamber	49	5.300	23.5		335
30	shale	coal mine	49		3.2		80
45	shale	coal mine	49		4.2		190
164	shale	coal mine	49		7.6	67	156
232	granite	metal mine	49	0.830	3.7		915
235	porphyry	metal mine	49	1.140	4.3		214
145	shale	coal mine	48		6.3	72	159
163	shale	coal mine	48		6.1	67	156
204	mudstone	chamber	48		3.0	24	100
205	mudstone	chamber	48		2.0	48	96
280	phyllite	tunnel	48		5.8		102
335	dunite	hard rock mine	48		2.8		295
69	siltstone	coal mine	47		3.2		300
153	shale	coal mine	47		5.4	72	145
161	shale	coal mine	47		5.6	67	152
298	shale	tunnel	47		10.0		225

Listing of RMR Case Histories *(Continued)*

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
310	mudstone	tunnel	47		6.0	6	61
347	coal	coal mine	47		4.2		145
25	coal	coal mine	46		4.2		386
160	shale	coal mine	46		8.4	18	171
180	shale	coal mine	46		8.1	28	154
199	mudstone	tunnel	46		6.0	9	90
60	gneiss & schist	tunnel	45	0.250	6.7		26
68	sandstone	coal mine	45		4.2		46
80	siltstone	chamber	45	3.300	30.0		100
115	shale	foundation	45	1.120	6.1		366
122	shale	coal mine	45		4.8	76	154
169	shale	coal mine	45		7.6	44	122
175	shale	coal mine	45		7.6	29	143
78	sandstone	tunnel	44	0.400	15.0		198
112	shale	foundation	44	1.000	5.5		195
166	shale	coal mine	44		6.9	39	175
168	shale	coal mine	44		4.9	44	122
170	shale	coal mine	44		4.9	44	122
176	shale	coal mine	44		5.6	29	154
215	basalt	chamber	44	0.100	6.0	897	897
297	shale	tunnel	44		10.0		225
316	siltstone	tunnel	44		6.0	4	131
318	mudstone	tunnel	44		6.0	28	178
333	porphyry	metal mine	44		4.5		706
350	sandstone	coal mine	44		4.2		510
40	shale	coal mine	43		3.1		335
129	shale	coal mine	43		5.4	24	175
147	shale	coal mine	43		5.4	264	152
165	shale	coal mine	43		4.6	39	157
167	shale	coal mine	43		4.6	39	175
172	shale	coal mine	43		6.1	26	160
174	shale	coal mine	43		5.6	29	143
177	shale	coal mine	43		5.3	28	154

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
201	mudstone	tunnel	43		6.0	480	54
38	sandstone	coal mine	42		3.5		104
64	limestone & schist	chamber	42	0.370	30.5		
111	mudstone & shale	chamber	42	1.670	13.7		101
179	shale	coal mine	42		5.0	28	154
268	greywacke	tunnel	42		3.0		150
66	siltstone	coal mine	41		4.2		18
171	shale	coal mine	41		4.6	26	160
173	shale	coal mine	41		4.6	26	143
178	shale	coal mine	41		4.7	28	154
299	shale	tunnel	41		10.0		225
307	siltstone	tunnel	41		6.0	132	43
54	granite	water tunnel	40		7.7		549
197	mudstone	tunnel	40		6.0	28	111
198	mudstone	tunnel	40		3.0	6	92
7	granite	tunnel	39	1.300	5.9	24	85
29	sandstone	coal mine	39		4.2		410
301	sandstone	tunnel	39		6.0		200
24	sandstone	coal mine	38		4.2		100
65	shale	coal mine	38		3.6		71
100	dolerite	tunnel	38	5.600	5.5		81
150	shale	coal mine	38		5.4	8	171
266	greywacke	tunnel	38	18.000	3.0		150
343	gneiss	tunnel	38		3.0		100
23	shale	coal mine	37		4.2		180
26	coal	coal mine	37		3.8		150
53	tuff	chamber	37	0.390	30.5		400
159	shale	coal mine	37		4.9	8	171
196	mudstone	tunnel	37		6.0	5	155
225	shale	tunnel	37	2.190	7.8		39
244	breccia	metal mine	37	0.030	3.7		330
269	greywacke	tunnel	37		3.0		150
339	sandstone	coal mine	37		2.5		400

Listing of RMR Case Histories *(Continued)*

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
27	coal & shale	coal mine	36		3.0		310
154	shale	coal mine	36		5.4	7	145
117	shale	foundation	35	0.370	6.1		152
121	shale	coal mine	35		4.8	4	154
302	siltstone	tunnel	35		6.0		200
12	mylonite	chamber	34	1.300	12.5	24	60
105	quartz-mica schist	tunnel	34	0.210	8.5		29
200	mudstone	tunnel	34		3.0	4	88
234	porphyry	metal mine	34	0.210	4.3		214
70	gneiss	tunnel	33	0.027	6.1		21
102	shale	rock slope	33	1.750	[illegible]		107
228	shale	rock slope	33		[illegible]		101
74	mudstone	coal mine	32		4.2		200
106	quartz-mica schist	tunnel	32	0.180	8.0		22
110	quartz-mica schist	chamber	32	0.180	93.0		21
300	sandstone	tunnel	32		6.0		200
120	shale	coal mine	31		4.8	2	154
239	breccia	metal mine	31	0.020	4.0		183
330	porphyry	metal mine	31		4.5		706
73	coal	coal mine	30		3.7		275
90	quartzite	tunnel	30	0.067	14.3		90
5	graywacke	tunnel	29	1.700	5.9		100
11	quartzite	headrace tunnel	29	0.180	8.0	24	200
96	dolerite	tunnel	29	1.470	5.5		26
206	mudstone	chamber	29		2.0	1	100
207	mudstone	chamber	29		1.0	2	100
31	shale	coal mine	28		3.2		30
91	quartzite	tunnel	28	0.033	14.3		41
93	quartzite	tunnel	28	0.067	14.3		39
304	siltstone	tunnel	28		6.0		200
81	siltstone	chamber	27	0.230	30.0		94
183	dolerite	tunnel	27		3.2	1	56
184	dolerite	tunnel	27		2.0	2	83

Case#	Rock Type	Project Type	RMR	Q	Span m	Stand up time hr	Depth m
185	dolerite	tunnel	27		1.0	10	48
85	breccia	tunnel	26	0.150	15.5		72
213	breccia	tunnel	26		2.0	1	70
332	porphyry	metal mine	26		4.5		706
10	schist	tailrace tunnel	24	0.170	9.0	0	110
41	shale	coal mine	24		3.6		250
303	breccia	tunnel	22		6.0		200
342	gneiss	tunnel	22		3.0		100
9	schist	chamber	21	0.100	6.5	24	50
233	porphyry	metal mine	21	0.020	4.3		214
230	breccia	metal mine	20	0.010	3.7		603
331	porphyry	metal mine	18		4.5		706
8	granite	tunnel	17	0.017	5.9	0	100
226	breccia	tunnel	16	0.140	7.8		46
98	dolerite	tunnel	15	0.090	5.5		47
116	coaly shale	foundation	10	0.040	6.1		152
305	breccia	tunnel	10		6.0		200
82	granite	tunnel	9	0.090	15.5		71
214	breccia	tunnel	9		1.0	1	68
72	granite	tunnel	8	0.011	14.6		442
75	granite	highway tunnel	8	0.001	14.6		399

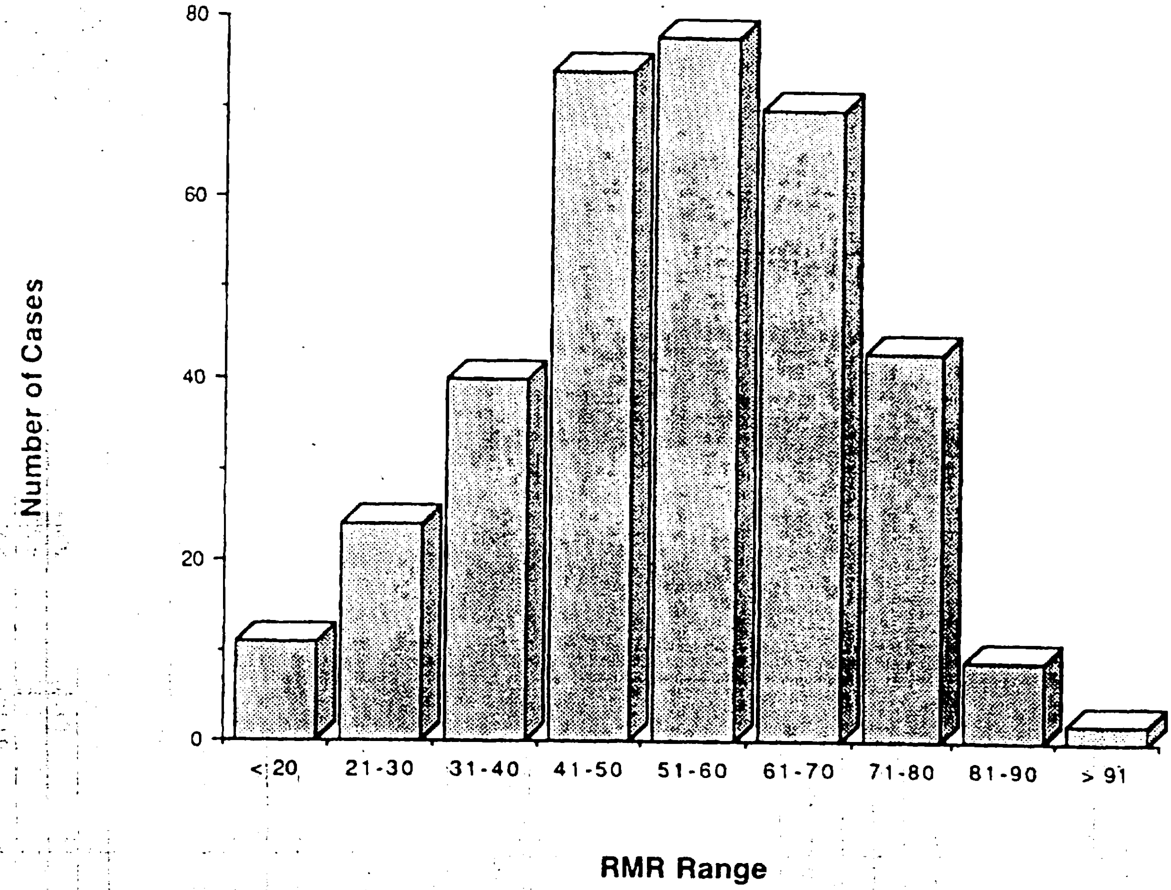

Figure 10.2 *Distribution of RMR values in the case histories studied.*

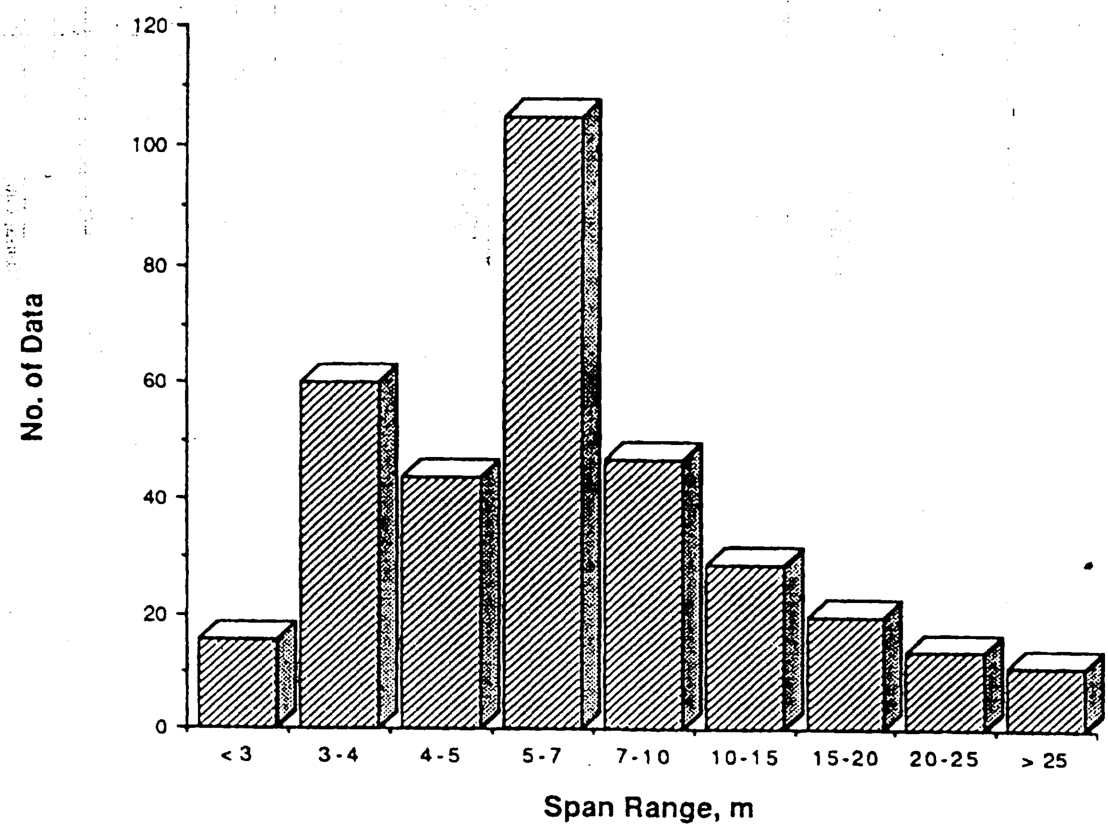

Figure 10.3 *The range of spans encountered in the RMR case histories.*

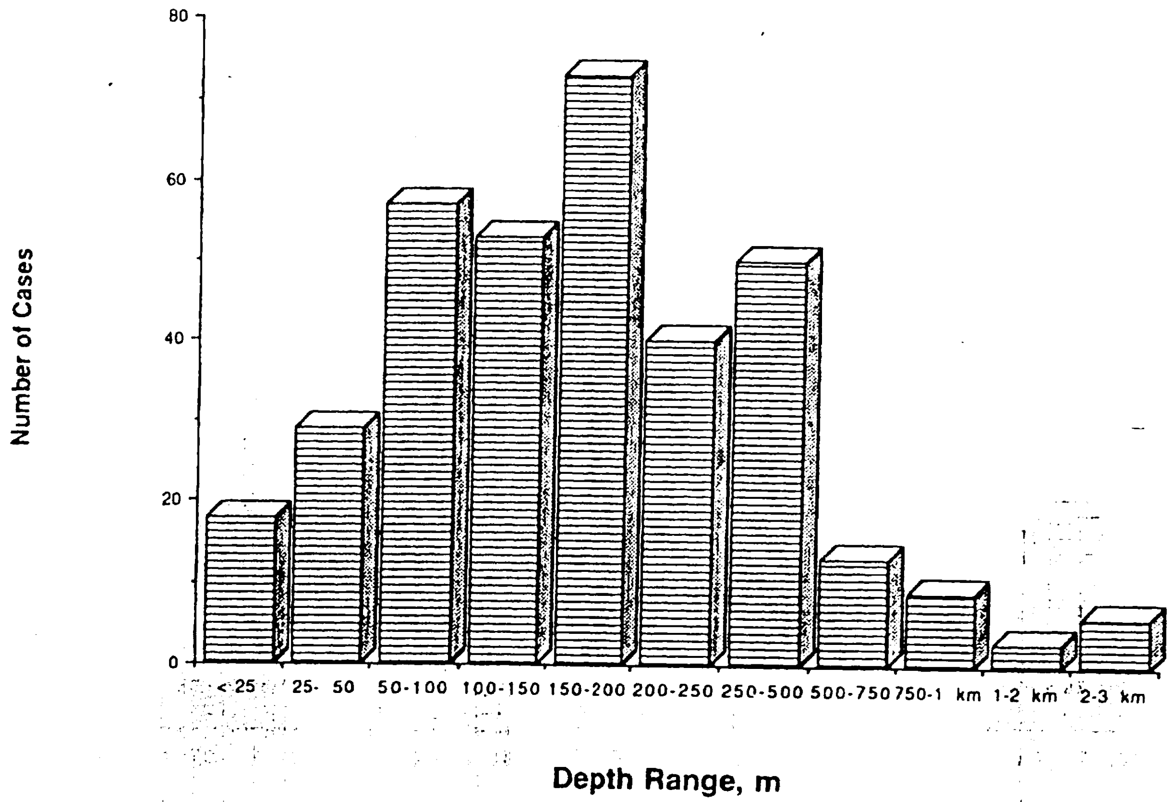

Figure 10.4 *The range of depths encountered in the RMR case histories.*

Appendix

Determination of the Rock Mass Rating: Output Example and Program Listing for Personal Computer

RMR

The Pennsylvania State University

Determination of the Rock Mass Rating
based on the Geomechanics Classification
of Bieniawski, 1979

Program written by Claudio Faria Santos
RMR System developed by Prof. Z. T. Bieniawski

Summer 1988

Do you wish a printed output of this program ?
- please answer "YES" OR "NO".

Questions:

What system of units are you going to use ?
- please answer M for metric or E for English customary units
? M

Enter the unit weight of the rock mass (in kN/cubic meter): ? 25

How many families of discontinuities are present in the rock mass ?
? 3

Which technique was used to determine the compressive strength
of intact rock in the laboratory (please answer 'P' for point load or
'U' for uniaxial compressive test) ?
? U

Enter the uniaxial compressive strength of the rock material (in MPa): ? 40

```
RMR

Enter the RQD: ? 60

Enter the discontinuity spacing (in meters): ? 0.150

Enter the discontinuity persistence (in meters): ? 10

Enter the separation between discontinuities (in mm): ? 0.125

Enter the condition of the joint surface
- please answer:
            'VR' for very rough
            'R' for rough
            'SR' for slightly rough
            'S' for smooth
            'SK' for slickensided
? SR

Enter the thickness of the joint infilling (in mm): ? 0.

Enter the weathering condition of the wall rock
- please answer:
            'UW' for unweathered
            'SW' for slightly weathered
            'MW' for moderately weathered
            'HW' for highly weathered
            'CW' for completely weathered
? SW

Enter the general groundwater condition
- please answer:
            'CD' for completely dry
            'DM' for damp
            'WT' for wet
            'DP' for dripping
            'FW' for flowing
? DM
```

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AMA

What is the effect of the strike and dip orientation
of the critical set of discontinuities  ?
- please answer:
            'VF' for very favorable
            'FV' for favorable
            'FR' for fair
            'UF' for unfavorable
            'VU' for very unfavorable
? FR

Estimate the weatherability of the rock mass ?
- please answer:
            'HR' for high resistance to weathering
            'MR' for intermediate resistance to weathering
            'LR' for low resistance to weathering
? LR
```

```
            Determination of RMR

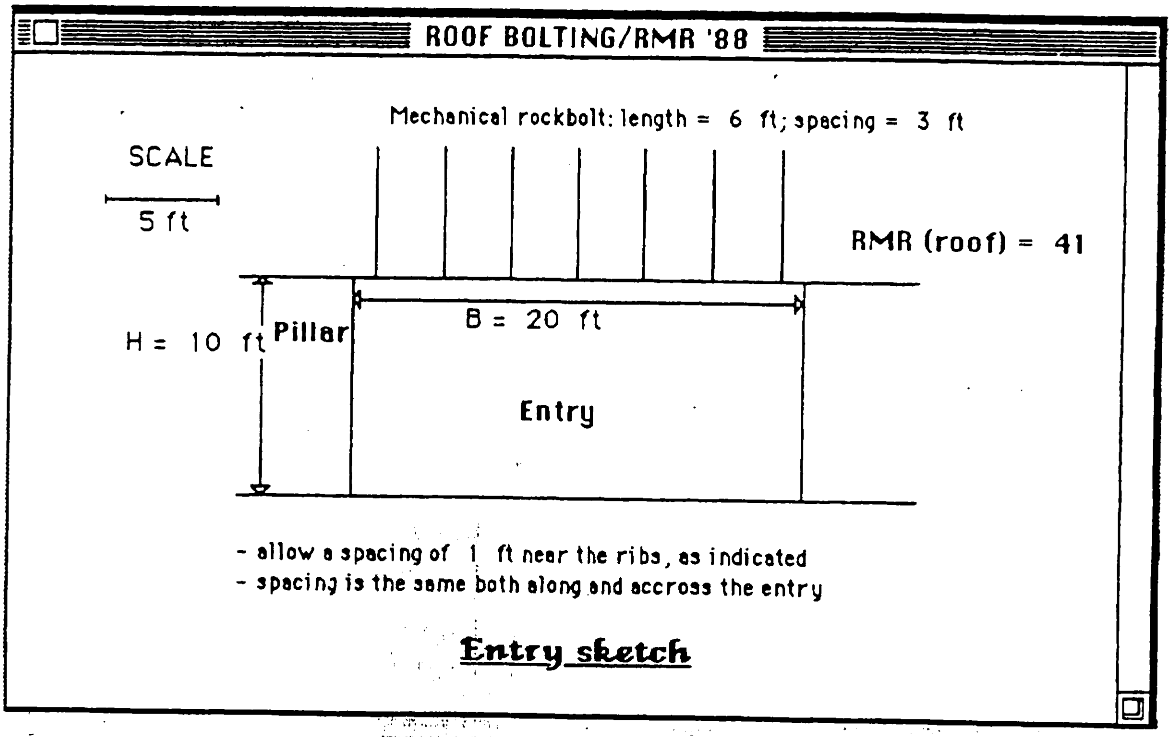
            Value of basic RMR:   50

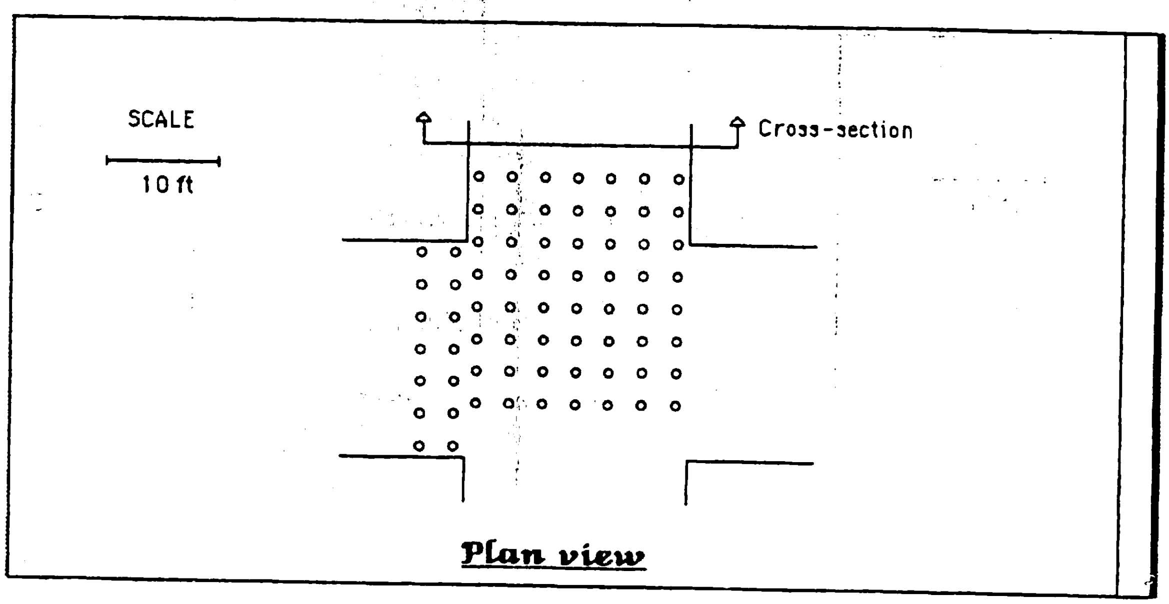
            Value of adjusted RMR:   41

            Value of RMR for dry conditions:   55


            Cohesion (kPa):  250

            Angle of internal friction:  30   degrees
```


ROOF BOLTING/RMR '88
Mechanical rockbolt: length = 6 ft; spacing = 3 ft
SCALE
5 ft
RMR (roof) = 41
H = 10 ft
Pillar
B = 20 ft
Entry
- allow a spacing of 1 ft near the ribs, as indicated
- spacing is the same both along and accross the entry
Entry sketch


SCALE
10 ft
Cross-section
Plan view

PROGRAM LISTING FOR PERSONAL COMPUTER

```
10 CLS
  DIM BL(2)
  PRINT
  CALL TEXTFONT (7)
  CALL TEXTSIZE (18)
  PRINT TAB (7) " The Pennsylvania State University"
  PRINT
  CALL TEXTFONT (5)
  CALL TEXTSIZE (14)
  PRINT TAB(11) "Determination of the Rock Mass Rating"
  PRINT TAB(10) "based on the Geomechanics Classification "
  PRINT TAB (21) "of Bieniawski, 1979"
  CALL TEXTFONT (0)
  CALL TEXTSIZE (12)
  PRINT
  PRINT
  PRINT TAB(11) "Program written by Dr. Claudio Faria Santos"
  PRINT TAB(10) "RMR System developed by Prof. Z. T. Bieniawski"
  PRINT
  CALL TEXTFONT (1)
  PRINT TAB(24) "August 1988"

  PRINT
  PRINT
  PRINT TAB (10) "Do you wish a printed output of this program ?"
  PRINT TAB (10) "- please answer "YES" OR "NO"."
  PRINT
100 INPUT PR$
  IF PR$="YES" THEN GOTO 2000
  IF PR$="NO" THEN GOTO 150
  PRINT
  PRINT TAB(10) "Please reenter the answer; use capital letters."
  GOTO 100

150 CLS
  CALL TEXTFACE (4)
  PRINT TAB(10) "Questions:"
  CALL TEXTFACE (0)
  PRINT
200 PRINT "What system of units are you going to use ?"
  PRINT "- please answer "M" for metric or "E" for U.S. customary units"
  INPUT SU$
  IF SU$="M" THEN GOTO 210
  IF SU$="E" THEN GOTO 220
  PRINT
  PRINT "- please reenter answer: "M" or "E" (use capital letters)"
  GOTO 200
210 PRINT
  INPUT "Enter the unit weight of the rock mass (in kN/cubic meter): ";GAMA
  PRINT
  GOTO 230
220 PRINT
  INPUT "Enter the unit weight of the rock mass (in pounds/cubic foot): ";PCF
  GAMA=PCF/6.363
  PRINT
230 PRINT "How many families of discontinuities are present in the rock mass ?"
  INPUT n
  PRINT
255 PRINT "Which technique was used to determine the compressive strength"
  PRINT "of intact rock in the laboratory (please answer 'P' for point load or"
  PRINT "'U' for uniaxial compressive test) ?"
  INPUT TT$
  PRINT
  IF TT$="P" THEN GOTO 265
```

```
  IF TT$="U" THEN GOTO 275
  PRINT "Please reenter the answer (P or U); use capital letters"
  GOTO 255
265 REM RMR question # 1a
  IF SU$="E" THEN GOTO 270
  INPUT "Enter the point load index (in MPa): "; PL
  GOTO 272
270 INPUT "Enter the point load index (in psi): "; PL
   PL=PL/145
272 PRINT
  SIGMA=24*PL
  GOTO 280
275 REM Question # 1b
  IF SU$="E" THEN GOTO 277
  INPUT "Enter the uniaxial compressive strength of the rock material (in MPa): ";SIGM
A
  PRINT
  GOTO 280
277 INPUT "Enter the uniaxial compressive strength of the rock material (in psi): ";SIG
MA
  SIGMA=SIGMA/145
  PRINT
280 INPUT "Enter the RQD: "; RQD
  PRINT
  REM RMR question # 3
  IF SU$="E" THEN GOTO 283
  INPUT "Enter the discontinuity spacing (in meters): "; SP
  PRINT
282 INPUT "Enter the discontinuity persistence (in meters): "; L
  PRINT
  INPUT "Enter the separation between discontinuities (in mm): "; ZETA
  PRINT
  GOTO 285
283 INPUT "Enter the discontinuity spacing (in feet): "; SP
  PRINT
  INPUT "Enter the discontinuity persistence (in feet): "; L
  PRINT
  INPUT "Enter the separation between discontinuities (in inches): "; ZETA
  CN=.305
  CV=25.4
  SP=SP/CN
  L=L/CN
  ZETA=ZETA/CV
  PRINT
285 PRINT "Enter the condition of the joint surface "
  PRINT "- please answer:"
  PRINT TAB(10) "'VR' for very rough"
  PRINT TAB(10) "'R' for rough"
  PRINT TAB(10) "'SR' for slightly rough"
  PRINT TAB(10) "'S' for smooth"
  PRINT TAB(10) "'SK' for slickensided"
  INPUT JR$
  PRINT
  IF JR$="VR" THEN GOTO 310
  IF JR$="R" THEN GOTO 320
  IF JR$="SR" THEN GOTO 330
  IF JR$="S" THEN GOTO 340
  IF JR$="SK" THEN GOTO 350
  PRINT "please reenter the answer (VR, R, SR, S or SK); use capital letters"
  GOTO 285
310 C4=6
  GOTO 355
320 C4=4.5
  GOTO 355
330 C4=3
  GOTO 355
340 C4=1.5
```

```
  GOTO 355
350 C4=0
355 IF SU$="E" THEN GOTO 360
INPUT "Enter the thickness of the joint infilling (in mm): "; T
  PRINT
  GOTO 365
360 INPUT "Enter the thickness of the joint infilling (in inches): "; T
  PRINT
  T=T/CV
365 PRINT "Enter the weathering condition of the wall rock "
  PRINT "- please answer:"
  PRINT TAB(10) "'UW' for unweathered"
  PRINT TAB(10) "'SW' for slightly weathered"
  PRINT TAB(10) "'MW' for moderately weathered"
  PRINT TAB(10) "'HW' for highly weathered"
  PRINT TAB(10) "'CW' for completely weathered"
  INPUT RW$
  PRINT
  IF RW$="UW" THEN GOTO 410
  IF RW$="SW" THEN GOTO 420
  IF RW$="MW" THEN GOTO 430
  IF RW$="HW" THEN GOTO 440
  IF RW$="CW" THEN GOTO 450
  PRINT "Please reenter the answer (UW, SW, MW, HW or CW); use capital letters"
  GOTO 365
410 E4=6
  GOTO 455
420 E4=4.5
  GOTO 455
430 E4=3
  GOTO 455
440 E4=1.5
  GOTO 455
450 E4=0
455 PRINT "Enter the general groundwater condition "
  PRINT "- please answer:"
  PRINT TAB(10) "'CD' for completely dry"
  PRINT TAB(10) "'DM' for damp"
  PRINT TAB(10) "'WT' for wet"
  PRINT TAB(10) "'DP' for dripping"
  PRINT TAB(10) "'FW' for flowing"
  INPUT GW$
  PRINT
  IF GW$="CD" THEN GOTO 510
  IF GW$="DM" THEN GOTO 520
  IF GW$="WT" THEN GOTO 530
  IF GW$="DP" THEN GOTO 540
  IF GW$="FW" THEN GOTO 550
  PRINT "Please reenter the answer (CD, DM, WT, DP or FW); use capital letters"
  GOTO 455
510 R5=15
  GOTO 555
520 R5=10
  GOTO 555
530 R5=7
  GOTO 555
540 R5=4
  GOTO 555
550 R5=0
555 PRINT "What is the effect of the strike and dip orientation "
  PRINT "of the critical set of discontinuities  ?"
  PRINT "- please answer:"
  PRINT TAB(10) "'VF' for very favorable"
  PRINT TAB(10) "'FV' for favorable"
  PRINT TAB(10) "'FR' for fair"
  PRINT TAB(10) "'UF' for unfavorable"
  PRINT TAB(10) "'VU' for very unfavorable"
```

```
   INPUT UF$
   PRINT
   IF UF$="VF" THEN GOTO 610
   IF UF$="FV" THEN GOTO 620
   IF UF$="FR" THEN GOTO 630
   IF UF$="UF" THEN GOTO 640
   IF UF$="VU" THEN GOTO 650
   PRINT "Please reenter the answer (VF, FV, FR, UF or UV); use capital letters"
   GOTO 555
610 ADJ=0
   GOTO 750
620 ADJ=2
   GOTO 750
630 ADJ=5
   GOTO 750
640 ADJ=10
   GOTO 750
650 ADJ=12
750 REM Determination of RMR:
   IF n>3 THEN LET F=1
   IF n=3 THEN LET F=1
   IF n=2 THEN LET F=1.33
   IF n=1 THEN LET F=1.33
   IF SIGMA>200 THEN LET R1=15:GOTO 800
   IF SIGMA<1 THEN LET R1=0:GOTO 800
   IF SIGMA<5 THEN LET R1=1:GOTO 800
   IF SIGMA<25 THEN LET R1=2:GOTO 800
    R1=1.4514+(.0684*SIGMA)
800 IF RQD>40 THEN GOTO 810
   IF RQD>25 THEN GOTO 820
   R2=3
   GOTO 825
810 R2=RQD/5
   GOTO 825
820  R2=(RQD/3)-(5+(1/3))
825 IF SP<.06 THEN LET R3=5:GOTO 850
    R3=14.6501*(SP^(.3587))
850 IF L<1 THEN LET A4=6:GOTO 870
   IF L>20 THEN A4=0:GOTO 870
   A4=6/L
870 IF ZETA<.1 THEN LET B4=6:GOTO 880
   IF ZETA>5 THEN LET B4=0:GOTO 880
   B4=.6/ZETA
880 IF T=0 THEN LET D4=6:GOTO 890
   IF T>5 THEN LET D4=0:GOTO 890
   D4=3
890 R4=A4+B4+C4+D4+E4
   BMR=R1+R2+R3+R4+R5
   URMR=BMR-R5+15
   URMR=URMR+.5
   URMR=INT(URMR)
   BMR=BMR+.5
   BMR=INT(BMR)
   IF BMR>100 THEN LET BMR=100
891 PRINT "Estimate the weatherability of the rock mass ?"
   PRINT "- please answer:
   PRINT TAB(10) "'HR' for high resistance to weathering"
   PRINT TAB(10) "'MR' for intermediate resistance to weathering"
   PRINT TAB(10) "'LR' for low resistance to weathering"
   INPUT QW$
   IF QW$="HR" THEN GOTO 892
   IF QW$="MR" THEN GOTO 892
   IF QW$="LR" THEN GOTO 893
   PRINT "Please reenter the answer (HR, MR or LR); use capital letters"
   PRINT
   GOTO 891
892 PRINT
```

```
   LET WY=1
   GOTO 895
893 LET WY=.9
895 PRINT "Is the value of the horizontal stresses known ?"
   PRINT "- please answer Y for "yes" or N for "no"
   PRINT
   INPUT YN$
   IF YN$="Y" THEN GOTO 896
   IF YN$="N" THEN GOTO 897
   PRINT "Please reenter the answer (Y or N); use capital letters"
   PRINT
   GOTO 895
896 IF SU$="E" THEN GOTO 898
   INPUT "Input the value of horizontal stresses (in MPa): ";HS
   PRINT
   GOTO 899
897 LET FLAG=1
   LET HC=1
   GOTO 900
898 INPUT "Input the value of horizontal stresses (in psi): ";HS
    HS=HS/145
   PRINT
899 LET Y=HS/SIGMA
   IF Y<.1 THEN LET HC=1: GOTO 900
   IF Y>.2 THEN LET HC=1: GOTO 900
   LET HC=.95
900 RMR=(BMR-ADJ)*WY*HC
  RMR=RMR+.5
  RMR=INT(RMR)

   CLS


   CALL TEXTFACE (4)
   PRINT TAB(10) "Determination of RMR"
   PRINT
   CALL TEXTFACE (0)
   PRINT TAB(10) "Value of basic RMR:  ";BMR

   PRINT
   PRINT TAB (10) "Value of adjusted RMR:  ";RMR

   PRINT
   PRINT TAB(10) "Value of RMR for dry conditions:  ";URMR

   PRINT
   REM Computation of c and ø:
   C=5*BMR
    FI=5+(BMR/2)
   PRINT
   IF SU$="E" THEN GOTO 950
   PRINT TAB(10) "Cohesion (kPa): ";C
   PRINT
   GOTO 955
950 CE=C*(.145)
  CE=CE+.5
  CE=INT(CE)
  PRINT TAB(10) "Cohesion (psi): ";CE
  PRINT
955 PRINT TAB(10) "Angle of internal friction: ";FI;" degrees"
   PRINT
   GOTO 19999
2000 CLS
   PRINT TAB(10) "WARNING:"
   PRINT TAB(10) "You need to have a line printer ("ImageWriter" or "
   PRINT TAB(10) "compatible) connected to your Macintosh. Make sure"
   PRINT TAB(10) "that the "Chooser" in the Apple Menu is set to right"
```

```
PRINT TAB(10) "printer."
FOR pause=1 TO 10000
NEXT pause

CLS
CALL TEXTFACE (4)
PRINT TAB(10) "Questions:"
LPRINT TAB(10) "Questions:"
CALL TEXTFACE (0)
LPRINT
PRINT
2200 PRINT "What system of units are you going to use ?"
LPRINT "What system of units are you going to use ?"
PRINT "- please answer M for metric or E for U.S. customary units"
LPRINT "- please answer M for metric or E for U.S. customary units"
INPUT SU$
LPRINT SU$
IF SU$="M" THEN GOTO 2210
IF SU$="E" THEN GOTO 2220
LPRINT
PRINT
PRINT "- please reenter answer: M or E (use capital letters)"
LPRINT "- please reenter answer: M or E (use capital letters)"
GOTO 2200
2210 PRINT
LPRINT
PRINT "Enter the unit weight of the rock mass (in kN/cubic meter): ";
INPUT GAMA
LPRINT "Enter the unit weight of the rock mass (in kN/cubic meter): ";
LPRINT GAMA
LPRINT
PRINT
GOTO 2230
2220 LPRINT
PRINT
PRINT "Enter the unit weight of the rock mass (in pounds/cubic foot): "
INPUT PCF
LPRINT "Enter the unit weight of the rock mass (in pounds/cubic foot): "
LPRINT PCF
GAMA=PCF/6.363
LPRINT
2230 PRINT "How many families of discontinuities are present in the rock mass ?"
LPRINT "How many families of discontinuities are present in the rock mass ?"
INPUT n
LPRINT n
PRINT
LPRINT
2255 PRINT "Which technique was used to determine the compressive strength"
PRINT "of intact rock in the laboratory (please answer 'P' for point load or"
PRINT "'U' for uniaxial compressive test) ?"
LPRINT "Which technique was used to determine the compressive strength"
LPRINT "of intact rock in the laboratory (please answer 'P' for point load or"
LPRINT "'U' for uniaxial compressive test) ?"
INPUT TT$
LPRINT TT$
LPRINT
PRINT
IF TT$="P" THEN GOTO 2265
IF TT$="U" THEN GOTO 2275
PRINT "Please reenter the answer (P or U); use capital letters"
LPRINT "Please reenter the answer (P or U); use capital letters"
GOTO 2255
2265 REM RMR question # 1a
IF SU$="E" THEN GOTO 2270
INPUT "Enter the point load index (in MPa): "; PL
LPRINT "Enter the point load index (in MPa): "; PL
GOTO 2272
```

```
2270 INPUT "Enter the point load index (in psi): "; PL
  LPRINT "Enter point load index (in psi): "; PL
   PL=PL/145
2272 PRINT
   SIGMA=24*PL
  GOTO 2280
2275 REM Question # 1b
  IF SU$="E" THEN GOTO 2277
  INPUT "Enter the uniaxial compressive strength of the rock material (in MPa): ";SIGM
A
  LPRINT "Enter the uniaxial compressive strength of the rock material (in MPa): ";SIG
MA
  PRINT
  LPRINT
  GOTO 2280
2277 INPUT "Enter the uniaxial compressive strength of the rock material (in psi): ";SI
GMA
  LPRINT "Enter the uniaxial compressive strength of the rock material (in psi): ";SIGM
A
   SIGMA=SIGMA/145
  PRINT
  LPRINT
2280 INPUT "Enter the RQD: "; RQD
  LPRINT "Enter the RQD: "; RQD
  PRINT
  LPRINT
  REM RMR question # 3
  IF SU$="E" THEN GOTO 2283
2282 INPUT "Enter  the discontinuity spacing (in meters):";SP
  LPRINT "Enter  the discontinuity spacing (in meters):";SP
  INPUT "Enter the discontinuity persistence (in meters): "; L
  LPRINT "Enter the discontinuity persistence (in meters): "; L
  PRINT
  LPRINT
  INPUT "Enter the separation between discontinuities (in mm): "; ZETA
  LPRINT :PRINT
  LPRINT "Enter the separation between discontinuities (in mm): "; ZETA
  LPRINT
  GOTO 2285
2283 INPUT "Enter the discontinuity spacing (in feet): "; SP
  LPRINT "Enter the discontinuity spacing (in feet): "; SP
  PRINT
  LPRINT
  INPUT "Enter the discontinuity persistence (in feet) "; L
  PRINT
  INPUT "Enter the separation between discontinuities (in inches): "; ZETA
  LPRINT "Enter the discontinuity persistence (in feet): "; L
  LPRINT
  LPRINT "Enter the separation between discontinuities (in inches): "; ZETA
  CN=.305
  CV=25.4
  SP=SP/CN
  L=L/CN
  ZETA=ZETA/CV
  PRINT
  LPRINT
2285 PRINT "Enter the condition of the joint surface "
  PRINT "- please answer:"
  PRINT TAB(10) "'VR' for very rough"
  PRINT TAB(10) "'R' for rough"
  PRINT TAB(10) "'SR' for slightly rough"
  PRINT TAB(10) "'S' for smooth"
  PRINT TAB(10) "'SK' for slickensided"
  INPUT JR$
  PRINT
  LPRINT "Enter the condition of the joint surface "
  LPRINT "- please answer:"
```

```
  LPRINT TAB(10) "'VR' for very rough"
  LPRINT TAB(10) "'R' for rough"
  LPRINT TAB(10) "'SR' for slightly rough"
  LPRINT TAB(10) "'S' for smooth"
  LPRINT TAB(10) "'SK' for slickensided"
  LPRINT JR$
  LPRINT
  IF JR$="VR" THEN GOTO 2310
  IF JR$="R" THEN GOTO 2320
  IF JR$="SR" THEN GOTO 2330
  IF JR$="S" THEN GOTO 2340
  IF JR$="SK" THEN GOTO 2350
  PRINT "please reenter the answer (VR, R, SR, S or SK); use capital letters"
  LPRINT "please reenter the answer (VR, R, SR, S or SK); use capital letters"
  GOTO 2285
2310 C4=6
  GOTO 2355
2320 C4=4.5
  GOTO 2355
2330 C4=3
  GOTO 2355
2340 C4=1.5
  GOTO 2355
2350 C4=0
2355 IF SU$="E" THEN GOTO 2360
  INPUT "Enter the thickness of the joint infilling (in mm): "; T
  PRINT
  LPRINT "Enter the thickness of the joint infilling (in mm): "; T
  LPRINT
  GOTO 2365
2360 INPUT "Enter the thickness of the joint infilling (in inches): "; T
  LPRINT "Enter the thickness of the joint infilling (in inches): "; T
  PRINT
  LPRINT
  T=T/CV
2365 PRINT "Enter the weathering condition of the wall rock "
  PRINT "- please answer:"
  PRINT TAB(10) "'UW' for unweathered"
  PRINT TAB(10) "'SW' for slightly weathered"
  PRINT TAB(10) "'MW' for moderately weathered"
  PRINT TAB(10) "'HW' for highly weathered"
  PRINT TAB(10) "'CW' for completely weathered"
  INPUT RW$
  PRINT
  LPRINT "Enter the weathering condition of the wall rock "
  LPRINT "- please answer:"
  LPRINT TAB(10) "'UW' for unweathered"
  LPRINT TAB(10) "'SW' for slightly weathered"
  LPRINT TAB(10) "'MW' for moderately weathered"
  LPRINT TAB(10) "'HW' for highly weathered"
  LPRINT TAB(10) "'CW' for completely weathered"
  LPRINT RW$
  LPRINT
  IF RW$="UW" THEN GOTO 2410
  IF RW$="SW" THEN GOTO 2420
  IF RW$="MW" THEN GOTO 2430
  IF RW$="HW" THEN GOTO 2440
  IF RW$="CW" THEN GOTO 2450
  PRINT "Please reenter the answer (UW, SW, MW, HW or CW); use capital letters"
  LPRINT "Please reenter the answer (UW, SW, MW, HW or CW); use capital letters"
  GOTO 2365
2410 E4=6
  GOTO 2455
2420 E4=4.5
  GOTO 2455
2430 E4=3
  GOTO 2455
```

```
2440 E4=1.5
  GOTO 2455
2450 E4=0
2455 PRINT "Enter the general groundwater condition "
  PRINT "- please answer:"
  PRINT TAB(10) "'CD' for completely dry"
  PRINT TAB(10) "'DM' for damp"
  PRINT TAB(10) "'WT' for wet"
  PRINT TAB(10) "'DP' for dripping"
  PRINT TAB(10) "'FW' for flowing"
  INPUT GW$
  PRINT
  LPRINT "Enter the general groundwater condition"
  LPRINT "- please answer:"
  LPRINT TAB(10) "'CD' for completely dry"
  LPRINT TAB(10) "'DM' for damp"
  LPRINT TAB(10) "'WT' for wet"
  LPRINT TAB(10) "'DP' for dripping"
  LPRINT TAB(10) "'FW' for flowing"
  LPRINT GW$
  LPRINT
  IF GW$="CD" THEN GOTO 2510
  IF GW$="DM" THEN GOTO 2520
  IF GW$="WT" THEN GOTO 2530
  IF GW$="DP" THEN GOTO 2540
  IF GW$="FW" THEN GOTO 2550
  PRINT "Please reenter the answer (CD, DM, WT, DP or FW); use capital letters"
  LPRINT "Please reenter the answer (CD, DM, WT, DP or FW); use capital letters"
  GOTO 2455
2510 R5=15
  GOTO 2555
2520 R5=10
  GOTO 2555
2530 R5=7
  GOTO 2555
2540 R5=4
  GOTO 2555
2550 R5=0
2555 PRINT "What is the strike and dip orientation "
  PRINT " of the critical set of discontinuities  ?"
  PRINT "- please answer:"
  PRINT TAB(10) "'VF' for very favorable"
  PRINT TAB(10) "'FV' for favorable"
  PRINT TAB(10) "'FR' for fair"
  PRINT TAB(10) "'UF' for unfavorable"
  PRINT TAB(10) "'VU' for very unfavorable"
  INPUT UF$
  PRINT
  LPRINT "What is the strike and dip orientation "
  LPRINT "of the critical set of discontinuities ?"
  LPRINT "- please answer:"
  LPRINT TAB(10) "'VF' for very favorable"
  LPRINT TAB(10) "'FV' for favorable"
  LPRINT TAB(10) "'FR' for fair"
  LPRINT TAB(10) "'UF' for unfavorable"
  LPRINT TAB(10) "'VU' for very unfavorable"
  LPRINT UF$
  LPRINT
  IF UF$="VF" THEN GOTO 2610
  IF UF$="FV" THEN GOTO 2620
  IF UF$="FR" THEN GOTO 2630
  IF UF$="UF" THEN GOTO 2640
  IF UF$="VU" THEN GOTO 2650
  PRINT "Please reenter the answer (VF, FV, FR, UF or UV); use capital letters"
  LPRINT "Please reenter the answer (VF, FV, FR, UF or UV); use capital letters"
  GOTO 2555
2610 ADJ=0
```

```
  GOTO 2750
2620 ADJ=2
  GOTO 2750
2630 ADJ=5
  GOTO 2750
2640 ADJ=10
  GOTO 2750
2650 ADJ=12
2750 REM Determination of RMR:
  IF n>3 THEN LET F=1
  IF n=3 THEN LET F=1
  IF n=2 THEN LET F=1.33
  IF n=1 THEN LET F=1.33
  IF SIGMA>200 THEN LET R1=15:GOTO 2800
  IF SIGMA<1 THEN LET R1=0:GOTO 2800
  IF SIGMA<5 THEN LET R1=1:GOTO 2800
  IF SIGMA<25 THEN LET R1=2:GOTO 2800
   R1=1.4514+(.0684*SIGMA)
2800 IF RQD>40 THEN GOTO 2810
  IF RQD>25 THEN GOTO 2820
  R2=3
  GOTO 2825
2810 R2=RQD/5
  GOTO 2825
2820  R2=(RQD/3)-(5+(1/3))
2825 IF SP<.06 THEN LET R3=5:GOTO 2850
   R3=14.6501*(SP^(.3587))
2850 IF L<1 THEN LET A4=6:GOTO 2870
  IF L>20 THEN A4=0:GOTO 2870
  A4=6/L
2870 IF ZETA<.1 THEN LET B4=6:GOTO 2880
  IF ZETA>5 THEN LET B4=0:GOTO 2880
  B4=.6/ZETA
2880 IF T=0 THEN LET D4=6:GOTO 2890
  IF T>5 THEN LET D4=0:GOTO 2890
  D4=3
2890 R4=A4+B4+C4+D4+E4
  BMR=R1+R2+R3+R4+R5
  URMR=BMR-R5+15
  URMR=URMR+.5
  URMR=INT(URMR)
  BMR=BMR+.5
  BMR=INT(BMR)
  IF BMR>100 THEN LET BMR=100
2891 PRINT "Estimate the weatherability of the roof strata "
  PRINT "- please answer:
  PRINT TAB(10) "'HR' for high resistance to weathering"
  PRINT TAB(10) "'MR' for intermediate resistance to weathering"
  PRINT TAB(10) "'LR' for low resistance to weathering"
  INPUT QW$
  LPRINT "Estimate the weatherability of the roof strata "
  LPRINT "- please answer:
  LPRINT TAB(10) "'HR' for high resistance to weathering"
  LPRINT TAB(10) "'MR' for intermediate resistance to weathering"
  LPRINT TAB(10) "'LR' for low resistance to weathering"
  LPRINT QW$
  IF QW$="HR" THEN GOTO 2892
  IF QW$="MR" THEN GOTO 2892
  IF QW$="LR" THEN GOTO 2893
  PRINT "Please reenter the answer (HR, MR or LR); use capital letters"
  PRINT
  LPRINT "Please reenter the answer (HR, MR or LR); use capital letters"
  LPRINT
  GOTO 2891
2892 PRINT
  LET WY=1
  GOTO 2895
```

```
2893 LET WY=.9
2895 PRINT "Is the value of the horizontal stresses known ?"
  PRINT "- please answer Y for yes or N for no
  PRINT
  INPUT YN$
  LPRINT "Is the value of the horizontal stresses known ?"
  LPRINT "- please answer Y for yes or N for no
  LPRINT
  LPRINT
  IF YN$="Y" THEN GOTO 2896
  IF YN$="N" THEN GOTO 2897
  PRINT "Please reenter the answer (Y or N); use capital letters"
  PRINT
  LPRINT "Please reenter the answer (Y or N); use capital letters"
  LPRINT
  GOTO 2895
2896 IF SU$="E" THEN GOTO 2898
  INPUT "Input the value of horizontal stresses (in MPa): ";HS
  PRINT
  LPRINT "Input the value of horizontal stresses (in MPa): ";HS
  LPRINT
  GOTO 2899
2897 LET FLAG=1
  LET HC=1
  GOTO 2900
2898 INPUT "Input the value of horizontal stresses (in psi): ";HS
  LPRINT "Input the value of horizontal stresses (in psi): ";HS
  HS=HS/145
  PRINT
  LPRINT
2899 LET Y=HS/SIGMA
  IF Y<.1 THEN LET HC=1: GOTO 2900
  IF Y>.2 THEN LET HC=1: GOTO 2900
  LET HC=.95
2900 RMR=(BMR-ADJ)*WY*HC
  RMR=RMR+.5
  RMR=INT(RMR)


  CALL TEXTFACE (4)
  PRINT TAB(10) "Determination of RMR"
  PRINT
  CALL TEXTFACE (0)
  PRINT TAB(10) "Value of basic RMR: ";BMR

  PRINT
  PRINT TAB (10) "Value of adjusted RMR: ";RMR

  PRINT
  PRINT TAB(10) "Value of RMR for dry conditions: ";URMR

  PRINT
  LPRINT
  CALL TEXTFACE (4)
  LPRINT TAB(10) "Determination of RMR"
  LPRINT
  CALL TEXTFACE (0)
  LPRINT TAB(10) "Value of basic RMR: ";BMR
  LPRINT
  LPRINT TAB (10) "Value of adjusted RMR: ";RMR
  LPRINT
  LPRINT TAB(10) "Value of RMR for dry conditions: "URMR
  LPRINT
  REM Computation of c and ø:
  C=5*BMR
  FI=5+(BMR/2)
  PRINT
  LPRINT
```

```
IF SU$="E" THEN GOTO 2950
PRINT TAB(10) "Cohesion (kPa): ";C
PRINT
LPRINT TAB(10) "Cohesion (kPa): ";C
LPRINT
GOTO 2955
2950 CE=C*(.145)
CE=CE+.5
CE=INT(CE)
PRINT TAB(10) "Cohesion (psi): ";CE
PRINT
LPRINT TAB(10) "Cohesion (psi): ";CE
LPRINT
2955 PRINT TAB(10) "Angle of internal friction: ";FI; " degrees"

LPRINT TAB(10) "Angle of internal friction: ";FI;" degrees"
PRINT :LPRINT
PRINT :LPRINT
PRINT

19999 PRINT "Do you want another RUN ";
INPUT RU$
IF RU$ = "YES" GOTO 10 ELSE GOTO 20000
20000 PRINT "*** END OF RUN ***"
FOR pause=1 TO 5000
NEXT pause
END
```

Bibliography

Being right is seldom enough.
Even the best ideas must be packaged and sold.
—Andrew Carnegie

This bibliography lists *in chronological order* all significant publications dealing with rock mass classifications. Although references are provided in this book at the end of each chapter, this bibliography also contains entries not referred to in the text but which are given here for completeness as well as for the convenience of those readers who wish to undertake a search of even the earliest references on the subject or are not sure of the author but remember the year of publication.

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