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IMPROVED DESIGN OF TUNNEL SUPPORTS: VOLUME 5 - EMPIRICAL METHODS IN ROCK TUNNELING-- REVIEW AND RECOMMENDATIONS

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FINAL REPORT

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16. Abstract

Empirical methods that avoid the use of an explicit model by relating ground conditions to observed prototype behavior have played a major role in tunnel design. The main objective of Volume 5 is to provide the tunneling profession with a review of empirical methods, and also present some guidelines on what empirical methods are best suited for observational (adaptable) tunneling procedures.

Volume 5 is the final volume of the five volumes published on the Improved Design of Tunnel Supports. The Executive Summary of the five-volume report was published in December, 1979 (Report No. UMTA-MA-06-0100-79-15).

The remaining final reports published in this five-volume series are:

Volume 1 - Simplified Analysis for Ground-Structure Interaction in Tunneling
Volume 2 - Aspects of Yielding in Ground-Structure Interaction
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16. Abstract

This five-volume report includes the results of an extensive research effort by the Massachusetts Institute of Technology to improve the design methodologies available to tunnel designers. The purpose of this report is to provide the tunneling profession with improved practical tools in the technical or design area, which provide more accurate representations of the ground-structure interaction in tunneling. The design methods range from simple analytical and empirical methods to sophisticated finite element techniques as well as an evaluation of tunneling practices in Austria and Germany.

Volume 5 evaluates empirical methods in tunneling. Empirical methods that avoid the use of an explicit model by relating ground conditions to observed prototype behavior have played a major role in tunnel design. The main objective of this volume is to provide the tunneling profession with a review of empirical methods, and to also present some guidelines on what empirical methods are best suited for observational (adaptable) tunneling procedures.

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PREFACE

This report is the final volume of five publications (the Executive Summary of this five-volume report was published in December, 1979) which include the results of an extensive research effort by the Massachusetts Institute of Technology (MIT) to improve the design methodologies available to tunnel designers. The contract, DOT-TSC-1489, was funded by the U.S. Department of Transportation (DOT) and was sponsored by the Urban Mass Transportation Administration's (UMTA) Office of Rail and Construction Technology. The contract was monitored by the Transportation Systems Center (TSC) Construction and Engineering Branch.

The purpose of Volume 5 is to provide the tunneling profession with a review of empirical methods. It also presents some guidelines on empirical methods best suited for observational tunneling procedures.

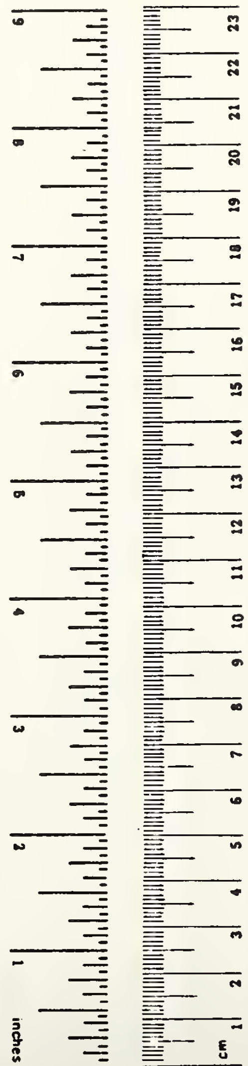
The review of the New Austrian Tunneling Method and the evaluation of ground support performance relations in squeezing rock was only possible with the help of our Austrian colleagues. In particular, we would like to mention Dr. M. John of Ingenieurgemeinschaft Lässer-Feizlmayr, Innsbruck, who has developed empirical performance predictions and provided us with data on the Arlbergtunnel. The permission of the Arlbergtunnel Authority, Innsbruck (Mr. H. Posch) is gratefully acknowledged. A similar amount of well-recorded and detailed data was made available by the Tauern Highway Authority, Salzburg (Messrs. G. Kollensperger and G. Dworschak), for which we would like to express our appreciation.

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

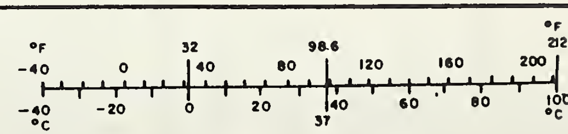
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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1. INTRODUCTION

Empirical methods have always been important in tunneling in contrast to many other branches of engineering. Beginning in the late 19th century when engineering entered the phase of formal modeling, this dichotomy could be seen to emerge even in the work of individuals. Culmann (1966), Ritter (1879), Kommerell (1913) were simultaneously developing analytical approaches to bridge and building design and empirical approaches to tunnel design. Of particular interest in this respect is Ritter's work, published a century ago (Ritter, 1879). He derived what may be considered a theoretical solution, however, being aware of the limitations he explicitely called for an empirical approach to be used in conjunction with it. The dichotomy between tunneling and other branches of civil engineering remains. Empirical or simple analytical methods are used at most in the preliminary design of bridges and buildings, while they are used in all phases of tunnel design and construction. Tunnels are built through a largely unknown environment, furthermore this environment is composed of different materials that vary over a wide range. In contrast other disciplines of civil engineering such as bridge or building construction use materials that are fairly well understood as are other influencing factors such as loads. Thus building construction lends itself much more to rigorous theoretical solutions than tunneling. In his inaugural address to the Prussian Academy of Sciences, Albert Einstein (1914) stated the following about theoretical and empirical methods:

> "The theorist's method involves his using as his foundation general postulates or "principles" from which he can deduce conclusions. His work thus falls into two parts. He must first discover his principles and then draw the conclusions which follow from them. For the second of these tasks he receives an admirable equipment at school. Once, therefore, he has performed the first task in some department, or for some complex of related phenomena, he is certain of success, provided his industry and intelligence are adequate. The first of these tasks, namely, that of establishing the principles which are to serve as the starting point of his deduction, is of an entirely different nature. Here there is no method capable of being learned and systematically applied so that it leads to the goal. The scientist has to worm these general principles out of nature by perceiving certain general features which permit of precise formulation, amidst large complexes of empirical facts.
>
> Once this formulation is successfully accomplished, inference follows on inference, often revealing relations which extend far beyond the province of the reality from which the principles were drawn. But as long as the principles capable of serving as starting points for the deduction remain undiscovered, the individual fact is of no use to the theorist; indeed he cannot even do anything with isolated empirical generalizations of more or less wide application. No, he has to persist in his helpless attitude towards the separate results of empirical research, until principles which he can make the basis of deduction reasoning have revealed themselves to him.........

> I have just now referred to a group of facts for the theoretical treatment of which the principles are lacking. But it may equally well happen that clearly formulated principles lead to conclusions which fall entirely, or almost entirely, outside the sphere of reality at present accessible to our experience. In that case it may need many years of empirical research to ascertain whether the theoretical principles correspond with reality."

In tunneling the principles that might serve as a starting point for the theoretical deduction remain still largely unknown and the empirical facts are often treated in a manner isolated from the underlying principles. Although not yet satisfactory from a theoretical point of view the consideration of empirical facts, the establishment and testing of empirical relations promises to bring us closer to the discovery of the underlying principles. In addition and this is the issue addressed here empirical methods are practically very attractive in view of the complexity of ground structure interaction in tunneling. In fact, the past decade has seen a resurgence of interest in empirical methods, and with it discussion and controversy. This discussion has centered on the suitability of various methods. In some applications identical results are obtained with differing methods and in some cases radically different ones. On certain occasions the discussion has focused more on the methodological rather than empirical aspects by taking issue with the appropriateness of particular parameters or the structure of some methods.

This report attempts to establish criteria for judging the suitability of empirical methods in tunnel design and construction in general and in observational tunnel design construction in particular. On this basis existing methods are reviewed and recommendations made for their use in observational tunnel design construction.

Specifically, the structure of empirical methods in tunneling is described first and basic requirements that an empirical method should satisfy are established (Section 2). Then a number of frequently used empirical methods are reviewed (Section 3). Each method is described by stating its principles and by showing in detail how it is intended to be applied, this is followed by a discussion of development and underlying philosophy, a critique, and ends with a summary. Also in Section 3 example applications are presented, case studies in which two or more of the methods have been used and their predictions compared with the actual requirements. The review of existing empirical methods are intended to show how well a particular method satisfies the previously extablished requirements. Section 4 parallels Section 2 in establishing requirements but now specifically for observational empirical methods considering their application both prior to construction and in the construction phase. In Section 5 it is shown how these requirements can be fulfilled or approximated through combinations or modifications of existing methods and the establishment of a well structured observational procedure. Conclusions are presented in Section 6.

It was very fortunate and it is gratefully acknowledged that many of the creators of modern empirical methods took the time to review a draft of this report and contribute with comments. Professor Z. Bieniawski, Dr. N. Barton, and Dr. R. Lien, Dr. D.U. Deere, and Dr. J.A. Franklin made corrections and comments regarding their methods. They also shared their thoughts on other methods and about the empirical approach in general. Depending on the wish of the particular contributor, his comments are identified as such or they have been incorporated by modification of the original text. The contributions by these leading tunnel designers were very helpful. They should not, however, be construed as an endorsement of the entire content of this report. There are areas where authors and creators, and creators amongst themselves disagree. The initiation of a discussion within this group seems to be a first positive result of this report.

2. STRUCTURE OF EMPIRICAL METHODS

2.1 INTRODUCTION

Both empirical and analytical approaches to tunnel design attempt to relate rock mass conditions to support requirements or construction procedures. Obviously, these relations must be specified among parameters which summarize the physical systems. Whereas, empirical approaches are developed without an explicit behavioral model, analytical approaches require one. Empirical approaches derive from a collection of prototype observations; analytical approaches derive from 'first principles'. Nevertheless, empirical approaches are also based on a behavioral model, even if that model is vague or only implicit.

Empirical and analytical methods serve the same purpose: to determine dimensions and quantities of tunnel support and possibly construction procedures. Empirical methods are used where there is insufficient information to establish an explicit model, when the parameter states of a model cannot be estimated, or when time and cost limitations prevent either. This means that empirical methods are primarily found in two applications:

1. Before construction ('limited' geologic information):
   - design of initial support, - determination of construction procedure, - preliminary design of final support.

2. During construction (limited time): - determination of (details of) initial support or adaptation of initial support, - determination of construction procedure, - design of final support.

In order to judge the suitability of empirical methods in tunnel design and construction in general and to examine if specific empirical methods are satisfactory, it is necessary to first determine the structure of empirical methods. This can then serve as a basis for establishing the requirements that such methods have to satisfy.

2.2 STRUCTURE OF EMPIRICAL METHODS

The structure of empirical methods in tunneling has never been considered in detail. Rather, they are simply stated to be correlations between rock mass conditions and support or construction. Although the predictions of empirical methods are quantitative, the procedure leading to them can be either quantitative or qualitative. This procedure is important in assessing the validity of the various techniques. As a first approximation a taxonomy of empirical methods, can be developed based on procedure (Figure 2.1). As can be seen, the primary distinction is whether geology is characterized quantitatively or qualitatively. This distinction is based on the predominant character of the method, since both quantitative and qualitative elements are present in every method. A truly quantitative description allows scaling on a physical scale (time, length).

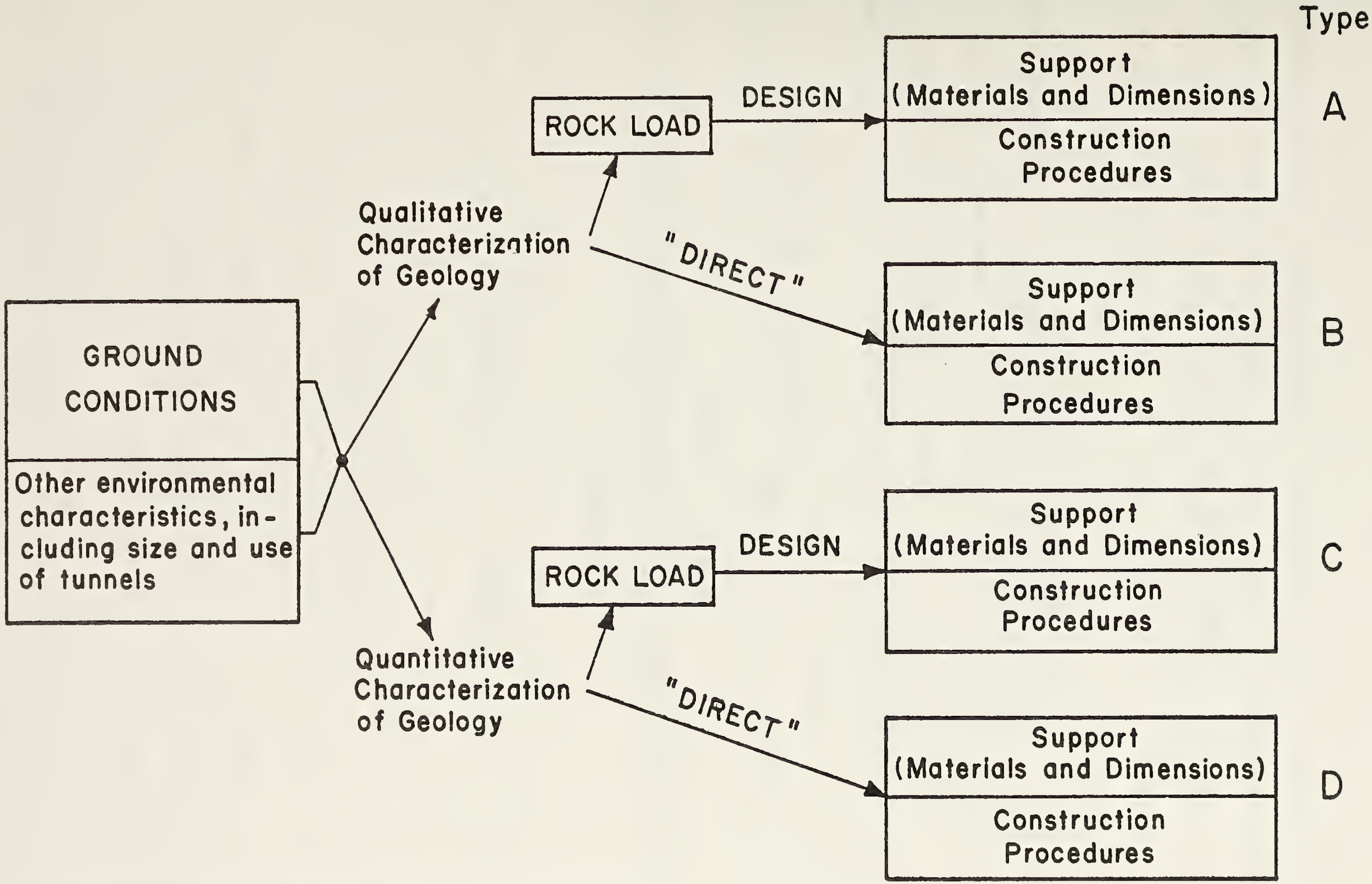

FIGURE 2.1 TAXONOMY OF EMPIRICAL METHODS

In a qualitative characterization a comparison is only possible on a ordinal scale i.e., one can say that condition A is better or worse than condition B or that it is between conditions B and C. However, it cannot be said to what degree A is better or worse than C.

Empirical methods with either qualitative or quantitative characterization can be further subdivided into methods that predict a rock load in a first step or methods that directly relate the characterization of the ground to support requirements and excavation procedure (Figure 2.1).

This taxonomy will be used to structure the above remarks.

Empirical Methods Type A - *Qualitative Indirect* or *Rock Load Methods*

Figure 2.2

With Type A methods the ground is divided into a number of (largely qualitative) classes. These are mostly based on geologic structure and water inflow. The rock load is tabulated by ground class, either directly as a load per unit area, or indirectly as a volume or rock per tunnel dimensions. For example:

Rock Volume = $a_i\ x + b_i y$ (Load = Rock Volume · Density)

x,y tunnel dimensions (e.g., height, width)

a_i, b_i constants for ground class;

The 'load' may be applied to the crown or to the crown and side walls (Figure 2.3).

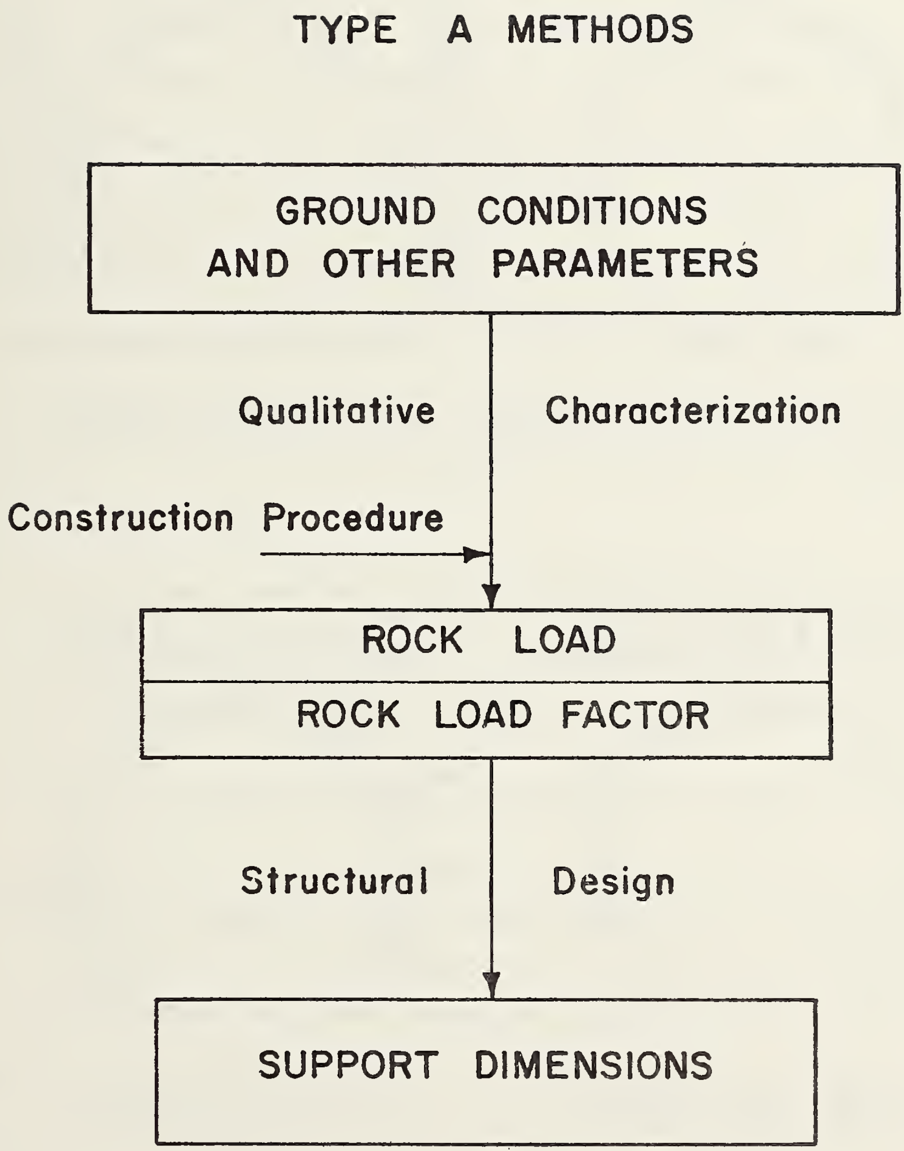

FIGURE 2.2 QUALITATIVE INDIRECT OR ROCKLOAD METHODS

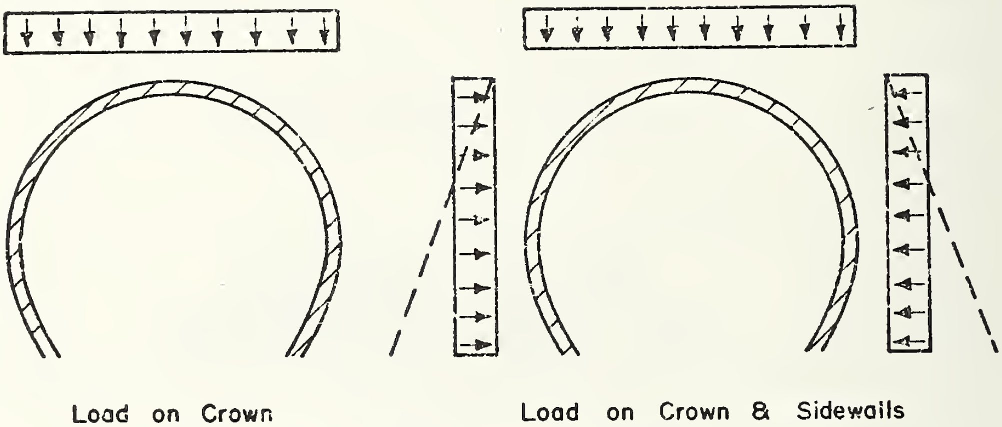

FIGURE 2.3 LOAD APPLICATION IN ROCKLOAD METHODS

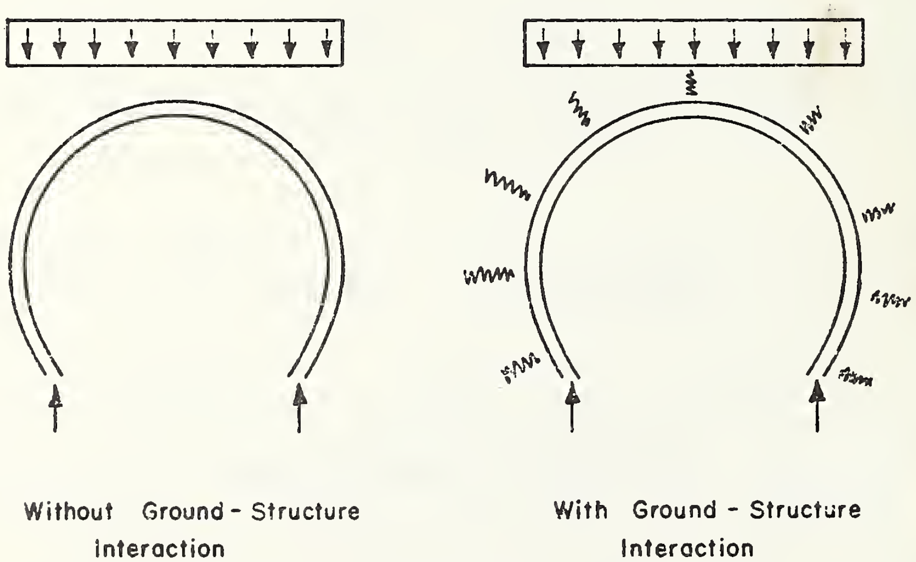

FIGURE 2.4 STRUCTURAL ANALYSIS IN ROCKLOAD METHODS

Once the load is determined it is applied in a standard structural analysis which may or may not consider ground structure interaction (Figure 2.4).

The best known Type A method is the "Terzaghi-Proctor and White approach" (Proctor and White, 1946). Proctor and White use a simple structural analysis with limited consideration of ground structure interaction to determine dimensions of steel sets. 'Rock Load' approaches analogous to Terzaghi's have been in existence for over 100 years and are probably the oldest widely applied tunnel design methods (Ritter, 1879; Kommerell, 1912; Bierbaumer, 1913).

Interestingly, the derivation of rock volumes is often quite sophisticated. Some methods take rock mass resistance into account and are to some extent 'quantitative'. Furthermore, in all of these methods load depends on opening size.

Empirical Methods Type B - *Qualitative Direct Method* (Figure 2.5)

Type B methods include those of Pacher et al. (1974) developed for the New Austrian Tunneling Method (NATM). The ground is categorized in qualitatively defined classes, similar to but more detailed than in Type A methods. The methods explicitly distinguish among types of discontinuities, lithologies, and stress states (overburden). A particular combination of construction procedure and support is then associated with each ground class (see Figure 2.6).

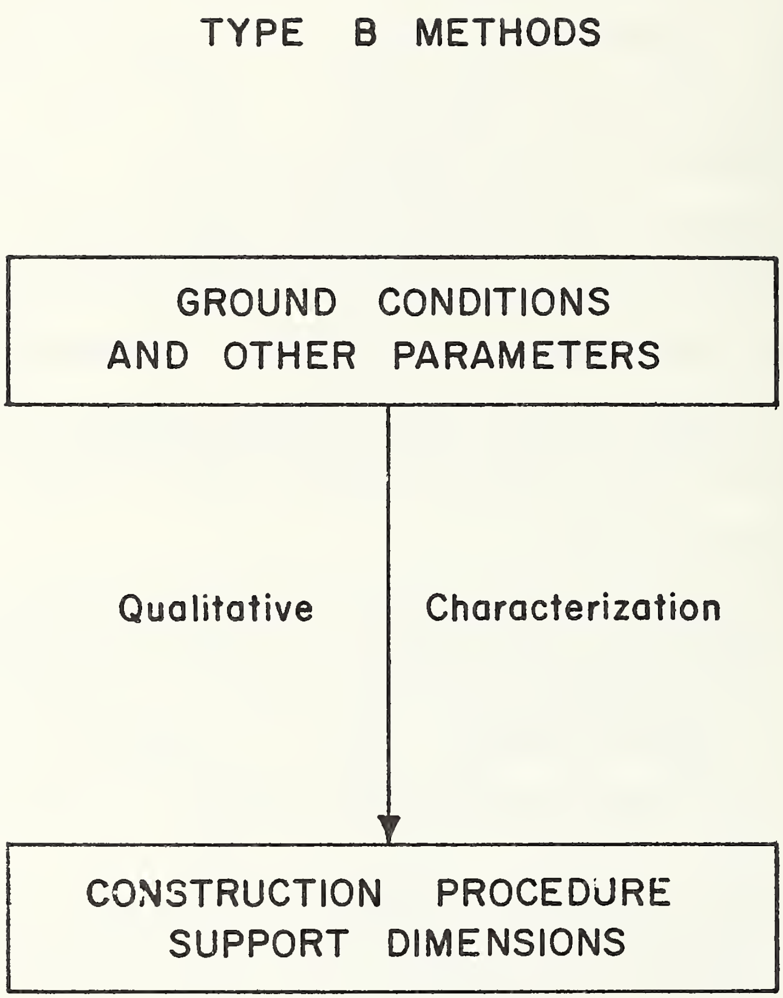

FIGURE 2.5 QUALITATIVE DIRECT METHODS

Class IV

GROUND CONDITIONS

Completely crushed and broken ground, chemically altered.
The overburden stress is large compared to the strength of the ground thus the ground will squeeze at the springlines and invert heave will occur. Water reduces the stability of the ground.

EXCAVATION METHOD

Heading and Benching Excavation (numbers on Fig.)
Roundlength = 1.5 to 2.0m
Face Support: shotcrete: 3 cm if necessary

SUPPORT

Placement in stages, immediately after excavation of a section:

1) shotcrete: 5cm
2) steelsets: 1.0 to 1.5m spaced
3) shotcrete: increased to 10cm
4) wirefabric
5) bolts: length: 4 to 9m, spaced 1.0 to 2.0m
6) shotcrete increased to 15cm

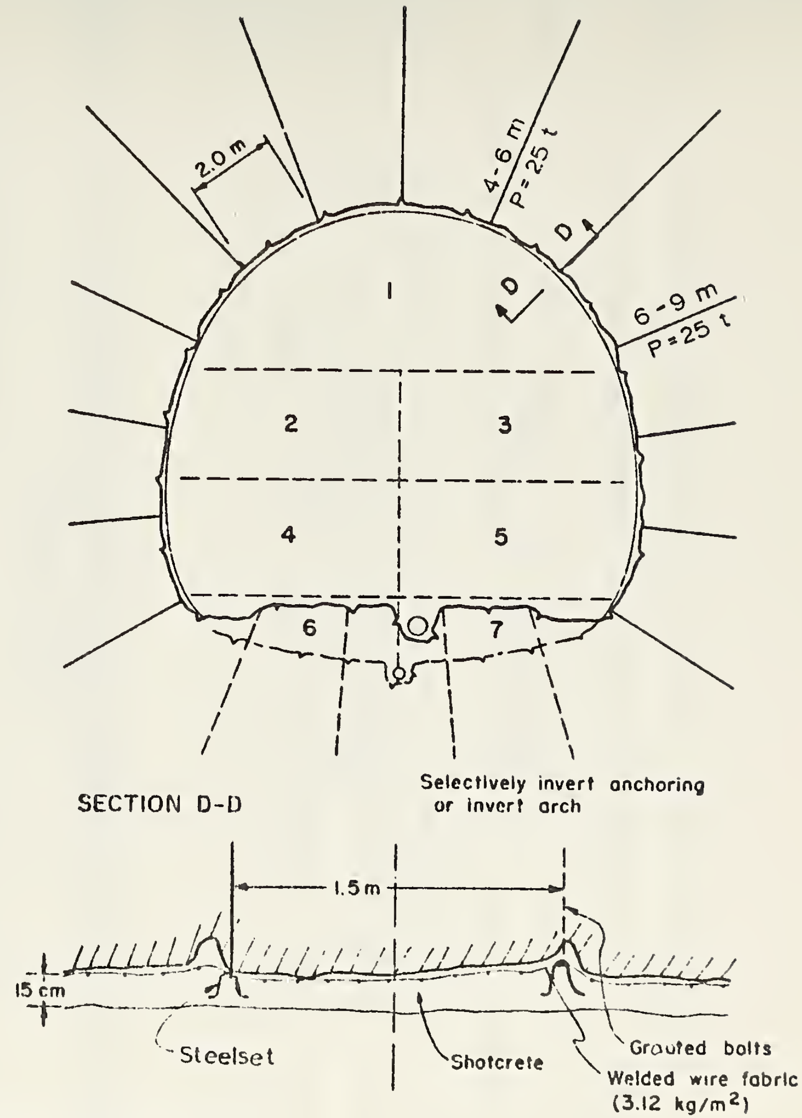

FIGURE 2.6 EXAMPLE OF NATM GROUND CLASS (AFTER JOHN,1976)

Generally, there are five to six ground classes in Type B methods. The geological definition of each class and the associated construction procedures are developed on a tunnel-by-tunnel basis. Thus, tunnel characteristics like stress state, opening shape, size are implicitly incorporated. The NATM is an adaptive method, in which details are developed for a particular application and modified during construction (if necessary).

Empirical Methods Type C - *Quantitative Rock Load Methods* (Figure 2.7)

In these methods the characterization of the ground is scaled quantitatively, and a Rock Load is related to the scale characterizing ground conditions. Rock load is applied to a structural design as already described for type A, qualitative rock load methods. The best known method of this type is Deere's et al. (1969) RQD - Rockload relation. Also in this category fall methods that consider the stability of wedges around a tunnel.

Empirical Methods Type D - *Quantitative Direct Method* (Figure 2.8)

In Type D methods ground (geology) is expressed quantitatively, and ground classes sometimes reflect opening size, shape, and use.

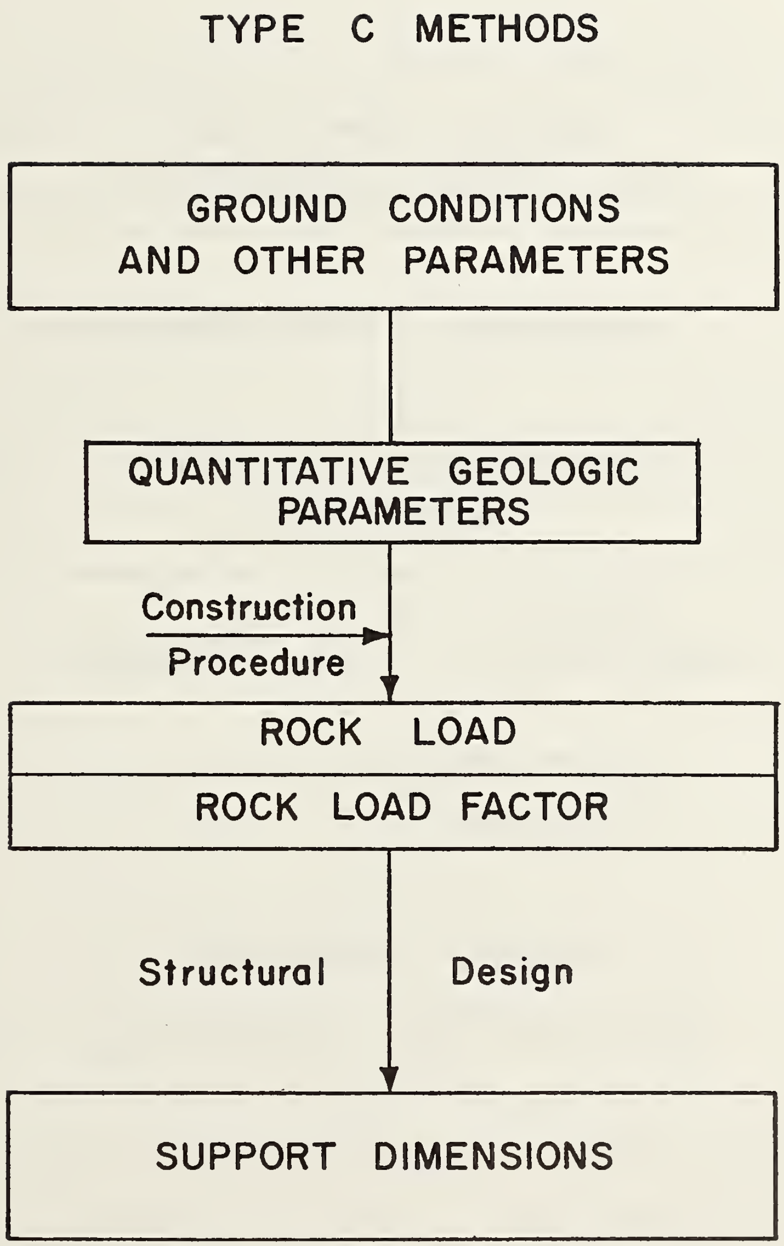

FIGURE 2.7 QUANTITATIVE ROCKLOAD METHODS

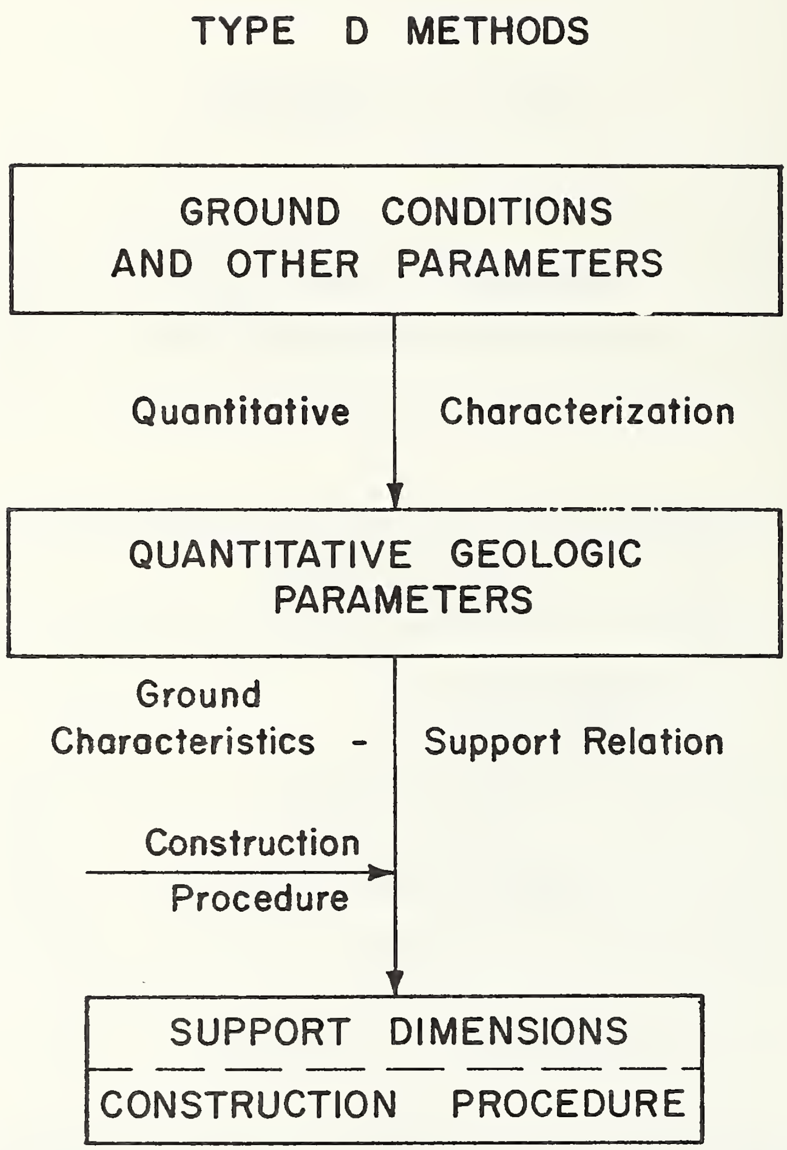

FIGURE 2.8 QUANTITATIVE DIRECT METHODS

The ground-support relation may consider construction procedure effects. Many recently proposed methods fall into this category.

For clarity, Type D methods are subdivided into those in which ground (geology) is expressed by a single parameter and those in which multiple parameters are used.

Methods Type D1 - *Single Parameter Quantitative*

The methods proposed by Lauffer (1958, 1960) and by Deere and his colleagues at the University of Illinois (Deere et al., 1968; Deere et al., 1969; Monsees, 1970; Merritt, 1972; Deere et al., 1974) are the best known of Type D1.

Lauffer published a case history on support requirements in a 5.3m diameter tunnel; the paper also includes his well known chart categorizing ground by its standup time for a particular size opening. He relates this characteristic to the support requirement in this particular tunnel.

Deere et al. (1968, 1969) categorize ground by RQD, giving support requirements as a function of RQD ranges, tunnel size, and excavation procedure. They also recommend the water conditions under which the method is appropriate. More recently, Deere et al. (1974) suggested three major categories for excavation and support requirements. These categories are based on RQD and qualitatively consider stress state, size effect, relative attitude of discontinuities, water pressure and inflow.

Method Type D2 - *Multiple Parameter Quantitative*

The methods by Barton (1974), Bieniawski (1979), Wickham et. al. (1974), Louis (1974), and Franklin (1976) fall into this category. Barton (1974, 1975, 1976) uses six parameters to quantify geology: frequency and type of discontinuities, water inflow, stress state, lithology, and intact rock properties. These parameters are combined into the scalar

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

RQD: Rock Quality Designation

J_n: Joint Set Number

J_r: Joint Roughness Number

J_a: Joint Alteration Number

J_w: Joint Water Number

SRF: Stress Reduction Factor

Support requirements are determined either from a graph and complemental tables or from an empirical support pressure formula. Opening dimensions and use are considered in the first approach.

Bieniawski (1979) again uses six parameter, basically similar to Barton's. The main differences with Bieniawski's method are that stress state is not explicitly considered, but discontinuity attitude is. Parameter values are summed yielding the so called Rock Mass Rating (RMR) which in turn is related to one of five ground classes. Each ground class is associated with a combination of support and construction procedures, somewhat similar to the NATM combinations.

Wickham et. al. (1974) describe regional geology (major lithology, faulting, strength), rock mass properties (spacing and attitude of joints) and water inflow with separate parameters. These are combined to give a so called Rock Structure Rating (RSR) which in turn is corelated to a Rib Ratio ($RR = \frac{8800}{RSR + 30} - 80$). The RR and opening size are related to spacings and type of steel sets, or to an equivalent Rock Load. Correction factors are given for Tunnel Boring Machine excavation, as is a formula for deriving rock bolt requirements from rock load.

Louis' (1974 a,b) method uses a combination of RQD and intact rock strength to arrive at a recommended excavation procedure (TBM, Drill and Blast). A companion classification, based on the two ratios 'fracture spacing/opening size' and 'intact compressive strength/state of stress', defines particular support - construction procedure combinations.

Franklin's (1970, 1975, 1976) method is similar to Louis', and was developed partly under contract to Louis at the Bureau of Geological and Mining Research (BRGM) in France.

2.3 CRITERIA FOR EMPIRICAL METHODS

Empirical methods should fulfill two sets of criteria, one which might be called user-requirements relating to practicality, economy, and safety; the other which might be called theoretical-requirements relating to correctness of derivation and appropriateness of methodology.

2.3.1 User Requirements

The user needs a generally applicable method requiring only limited special skills and easily measureable parameters, that results in an economic, safe opening. Specifically these criteria mean:

Economy and Safety. Supports and construction procedures should not be overly conservative nor should they fail. Ideally, the degree of safety should be known. For initial supports this means a factor of safety near 1; for final supports this means a design with a pre-determinable factor of safety (or probability of failure).

Safety mainly in the case of initial supports does not only concern structural failure but also excessive deformations. In squeezing ground for instance the support may fail without the tunnel collapsing but with increased deformations which may be entirely acceptable. These deformations should, however, not exceed some limit to prevent reexcavation, i.e., unfavorable economic consequences.

General Applicability and Robustness. The method should be applicable to a wide range of ground conditions*, opening sizes, and shapes and to different construction procedures and support types. If this is not the case, the range of applicability should be explicitly described.

Although some experience in tunnel design and construction may be a prerequisite, empirical methods should not require such skills that the user could just as easily have developed his own method. This particularly concerns subjective aspects of the method. After a few applications, the user should be able to easily and confidently make the required judgemental decisions. In particular, the method should be relatively insensitive to vagaries in judgement either by the same user or by different users, i.e., the method should be robust and repeatable.

Readily Determinable Parameters. Parameters are determined from boring logs, outcrops, maps, general knowledge of the area, and from observations in the tunnels. Some limited physical testing may also be used. Only a limited number of parameters can be determined from boreholes, outcrops and maps (particularly concerning conditions at tunnel grade). These explorations are usually made prior to construction and

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*Typical conditions possibly together with support type and construction procedure may lead to one of several typical behaviors ('loosening, squeezing, swelling').

time is thus usually not a limiting problem. In contrast, observations during construction in the tunnel detect many details, but time is often limited. Parameters that can be easily obtained from outcrops and boreholes or quickly observed (or measured) in the tunnel are desirable.

2.3.2 Requirements Regarding the Methodology and Derivation of Empirical Methods

While user requirements are concerned with applicability, theoretical requirements concern "basic correctness". As discussed, empirical methods do not rely on explicit formal models of ground conditions and support or construction. However, empirical relations are almost always based on some implicit model. The completeness and adequacy of underlying models are of paramount importance.

Model Accuracy. Figure 2.9 is a schematic characterization of ground-support inter-action. Here, ground is summarized by several influencing factors. The proportions and states (properties) of these factors can vary (Figure 2.10).

The jointing factor which is shown as an example represents the influence of joints i.e., the influence of the number, orientation, spacing, persistence, resistance of joints on the behavior of the ground around a tunnel. These detailed jointing characteristics "the properties of the

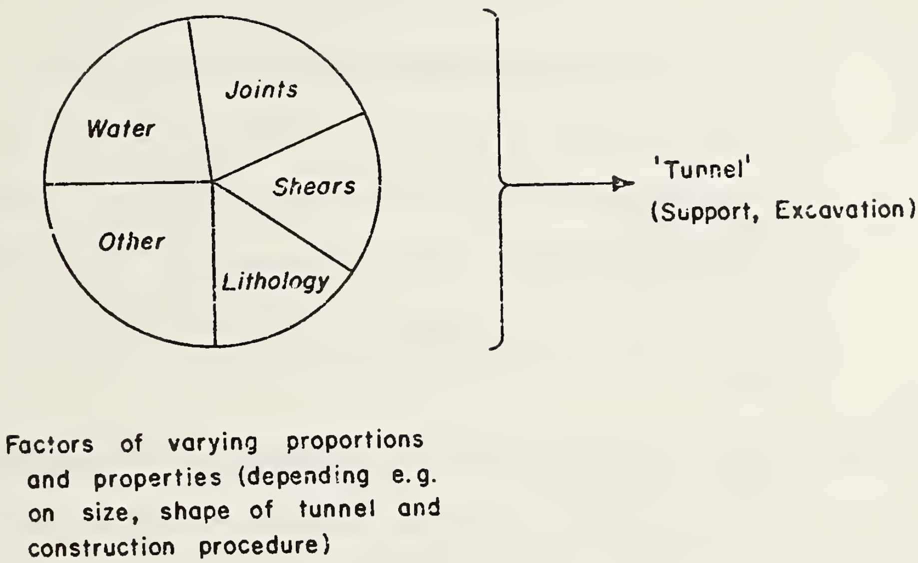

FIGURE 2.9 CHARACTERIZATION OF GROUND SUPPORT INTERACTION

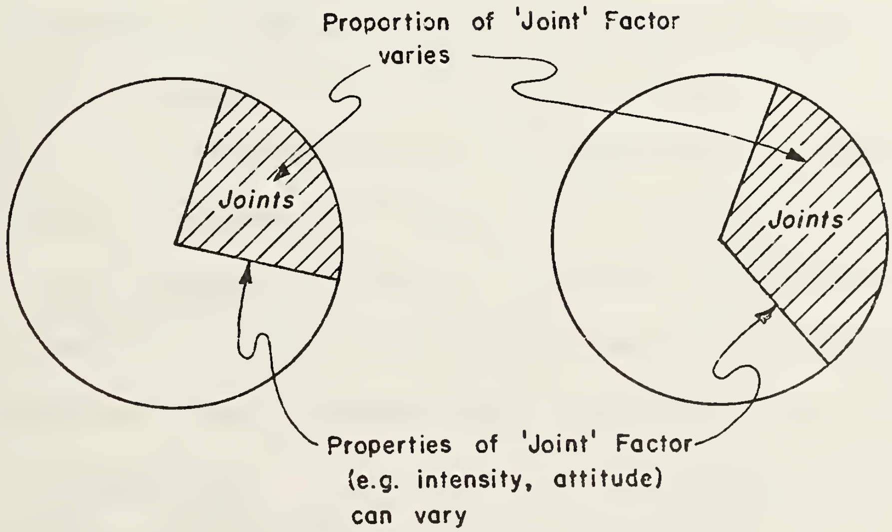

FIGURE 2.10 VARIATION OF PROPERTIES AND PROPORTIONS OF INFLUENCING FACTORS

jointing factor" can vary and with them the effect of the jointing factor on the tunnel. Also in some instances jointing may have a smaller effect relative to other factors e.g., in low strength rock the intact strength may become relatively more important. The jointing factor may also depend on the size of the tunnel.

Ideally, one would like to know which factors affect the tunnel, their relative importance, and their state (properties).

Empirical (and for that matter, analytical) methods should represent as exactly as possible these relative influences and states. Of course, this is usually not the case. Instead the ground is characterized by parameters that are not exactly congruent with the true factors. For example, RQD represents a part of the "jointing factor" and a part of the "intact rock factor". Further, the parameters may or may not be exhaustive or mutually exclusive (Figure 2.11). For example, "RQD" and "spacing" do not fully describe the "jointing factor", and to some extent they express the same property of the jointing factor.

Finally, even if parameters are mutually exclusive they may still be correlated. For example, joint spacing and water inflow rate.

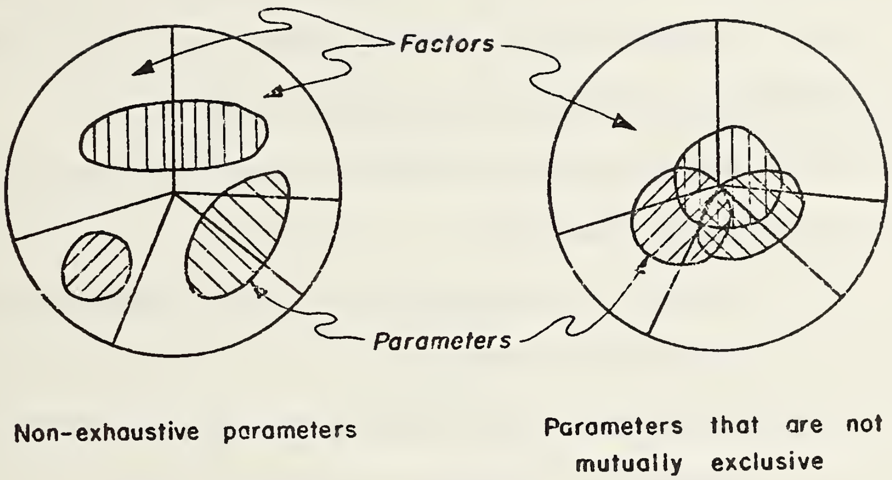

FIGURE 2.11 PARAMETERS REPRESENTING INFLUENCING FACTORS

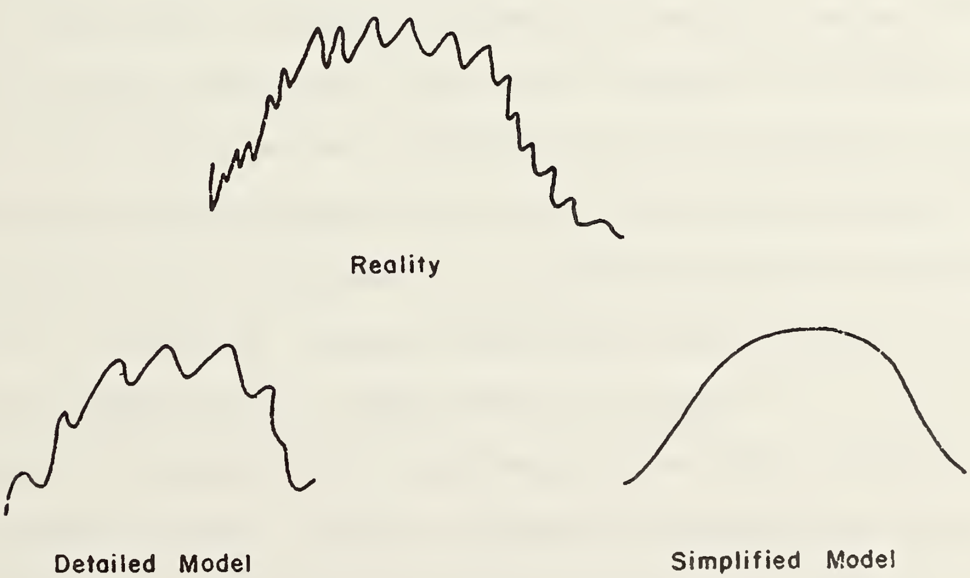

FIGURE 2.12 MODELS OF DIFFERENT DEGREES OF DETAIL REPRESENTING REALITY

The best representation of ground for tunneling would be by parameters that are congruent with the influencing factors. This is prevented by limitations that affect all geotechnical design approaches (Baecher, 1978):

- The underlying model(s) only partially represents reality.
- The ground and other in situ conditions are spatially variable.
- Techniques for determining geotechnical parameters introduce sampling and measurement errors (both random and biased).

Empirical methods relate tunnel features and processes to observed ground parameters, and thus involve three underlying models. One model represents geologic and geometric conditions of the ground, one represents interaction of the ground and tunnel, and one represents the connection of observations to actual geologic or geometric conditions of the ground. Again, these models are usually implicit.

It would be desirable to have models that represent the inherent variability of the ground and construction processes as closely as possible (Figure 2.12). However, there are trade-offs to be made in adopting a model. A detailed model is more accurate but requires more parameters than a simplified one. Since, the measurement and inference of parameters involves statistical uncertainty, the more interdependent parameters there are; the more uncertainty there is in inferences of each.

There is thus a trade-off between model uncertainty and parameter uncertainty; an increase of the number of parameters does not guarantee a better representation.*

Subjective Character of Empirical Methods. Because models are abstractions of reality and because parameter estimates involve uncertainties, no procedure for establishing support requirements can be "objective", they are all substantially subjective. This may not be obvious to the user. On one extreme is an empirical method using a subjective identification of ground parameters, which is subjectively related to support requirements. On the other extreme is an empirical method using a "measurement" of ground conditions, which is related by regression analysis to support requirements. The latter may be more consistent or repeatable and more explicit than the former, but both are fundamentally subjective. Nature is variable and the conditions of test cases must be summarized subjectively by the developer of an empirical method. Furthermore, a large number of implicit assumptions are required in formulating hypotheses for data analysis. Underlying models of an empirical method have a subjective character reflecting the developer's ideas. If the user does not recognize this,

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* This trade-off can be seen in the classic Coulomb (c,ϕ) description of granular materials. The model is a gross simplification of intricate particulate mechanics; however, the relative ease with which c,ϕ can be determined makes testing feasible and thus the amount of experimental work necessary to reduce parameter uncertainty. On the other hand a model considering the details of particle interaction would be more accurate but would require more parameters that are difficult to determine thus increasing parameter uncertainty.

errors may result. For instance in describing equivalent support pressure based on derivations from measured deflections or design dimensions (but not based on direct measurements) the model underlying the pressure derivations has a significant effect.

In going from one extreme to the other subjectivity (or discretion) by the user is simply replaced by that of the developer. One advantage of the more quantified methods, however, is that repeatability of factor estimates allows an intercomparison of results from one project to another. Nevertheless, the extension of regression analysis results to new situations requires a large number of unprovable assumptions taken on faith. The most important of these is that the cases to be predicted are homogeneous in all important ways with the calibrating cases.

Representative Modelling and Completeness. If an empirical method is based on cases in which a particular factor was important, generalization may be erroneous. For instance, the significance of geologic structure is recognized in all methods, but some ignore the attitude of the geologic structure relative to the opening. These cannot be generalized if the calibration cases were systematically biased with respect to attitude. Some methods rely mostly on tunnels in a low stress environment for calibration. Together with a particular discontinuity pattern this leads mostly to roof fall type instability. Such a method cannot be generalized to tunnels in squeezing or swelling rock. Thus a method should be able to distinguish between different types of mechanisms that influence support requirements. Most importantly, many methods do not take construction effects into account. Unless at least TBM vs. Drill and Blast is specified (or preferably details like partial or full face excavation, round lengths and support installation distance and sequence) generalization is difficult.

To summarize, empirical methods should attempt to satisfy the following in some ways incompatible objectives:

(1) They should promote economical yet safe designs.

(2) They should be generally applicable and robust. If they are not generally applicable the methods should clearly indicate their limits of applicability. The method must be insensitive to vagaries of use.

(3) The required parameters should be readily determinable without restrictions due to time, equipment, accessibility.

(4) The subjective character of empirical methods has to be recognized. Models are abstractions of reality and thus fundamentally subjective. A method that may be considered 'objective' by a user since it is based on measurable quantities is still fundamentally subjective.

(5) The model underlying the method whether explicit or implicit should be correct. It should thus consider all relevant factors and differentiate between various types of ground structure behavior.

3. REVIEW OF EXISTING EMPIRICAL METHODS

3.1 INTRODUCTION

In Section 2 the structure of empirical methods for tunnel design has been reviewed and requirements that an empirical method should fulfill have been established. In this chapter existing empirical methods will be reviewed employing these requirements as criteria. Since most of the criteria relate to the derivation of a method, an assessment of existing methods can only be made by detailed study. The methods will be reviewed individually in four groups following the taxonomy of Section 2. Type A methods - qualitative indirect or rock load methods - are discussed in Section 3.2, Type B methods - qualitative direct methods - are discussed in Section 3.3, Type C methods - quantitative indirect or rock load methods - are discussed in Section 3.4, Type D - quantitative direct methods - are discussed in Section 3.5. In Section 3.6 example applications, in Section 3.7 advantages and disadvantages of the methods, and in Section 3.8 conclusions are presented.

The format of discussion for each individual method is as follows: first in a section entitled 'methodology' the latest version of the method and recommended guidelines for its use are described. This is followed by a description of development and underlying philosophy, then a detailed critique is presented, finally closing with a summary of each method.

3.2 QUALITATIVE INDIRECT OR ROCK LOAD METHODS. (TYPE A)

With type A methods (Fig. 3.2.1) a number of largely qualitative classes, mostly based on geologic structure and water inflow, is related to a rock load. These rock loads are tabulated by ground class, either directly as a load per unit area or indirectly as a volume of rock per unit-length, the so-called rock load factor. The rock load or the volume may or may not be dependent on tunnel size. The predicted rock load is used to design a support system.

Rock load methods are the most widely used empirical methods for the design of tunnel supports. This may be due to the fact that they consist of two distinct steps, the first being the prediction of the rock load which can be accomplished by an engineering geologist or a geotechnical engineer and the second being a structural design which can be performed by a structural engineer.

However, the major reason for the popularity of the approach seems to lie in the involvement of structural engineers in their development. Structural engineers designed many tunnels at a time when structural engineering made the step from empiricism to employing analytical methods. The use of rock load approaches made it possible to at least design the support analytically. It is thus interesting to observe that many of the leading structural engineers of the period developed rock load approaches. (see below).

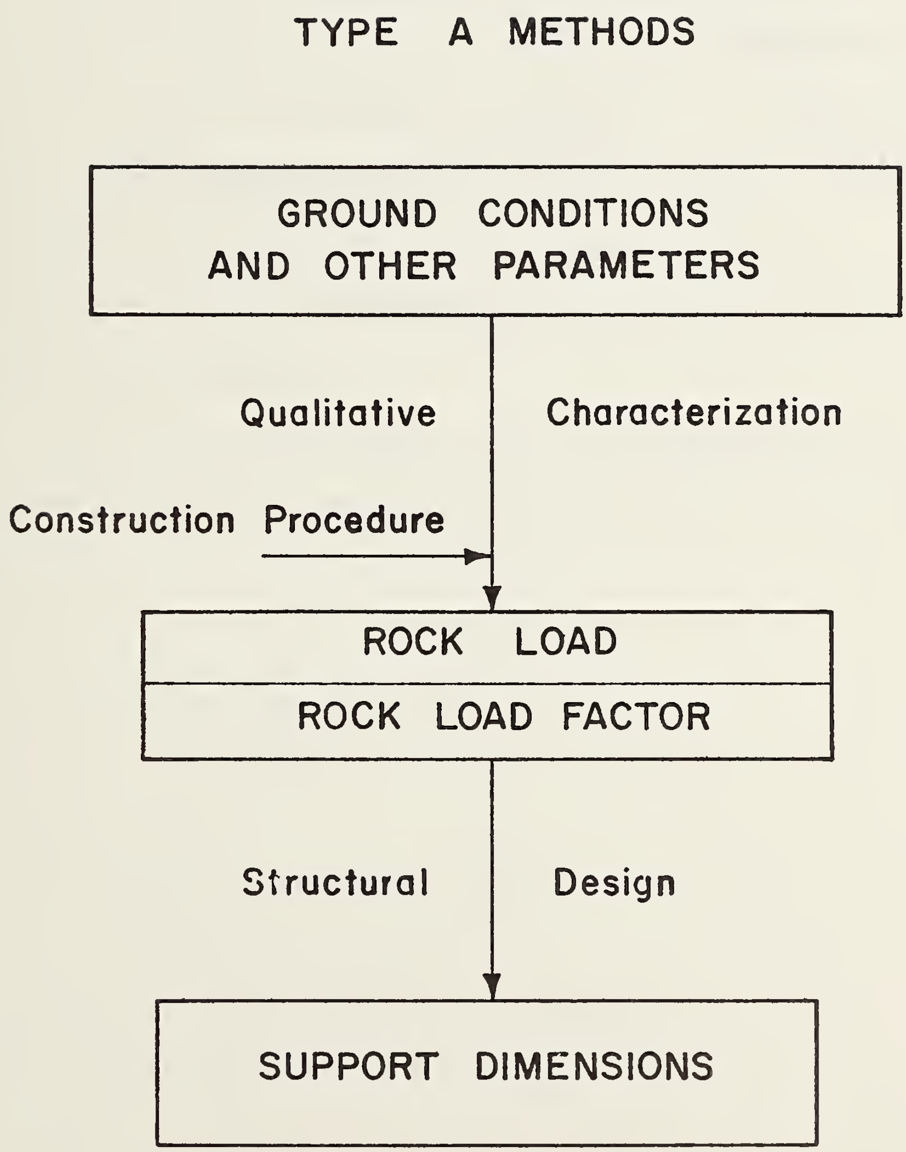

FIGURE 3.2.1 QUALITATIVE INDIRECT OR ROCKLOAD METHODS

Rock load methods that will be reviewed are those by:

- Ritter (1879)
- Kommerell (1912, 1940)
- Bierbaumer (1913)
- Terzaghi (1946)
- Stini (1950)

3.2.1 Ritters Method

Methodology and Application Guidelines

Ritter (1879) derived expressions for roof loads in tunnels with a flat roof (Fig. 3.2.2), and circular tunnels (Fig. 3.2.3). He also considered the stability of the side walls (Fig. 3.2.4). The volume of rock with unit weight γ (Figs. 3.2.2 and 3.2.3) is considered to be held in balance by a tensile unit force σ_t along the boundary ABC and by the force 'E' acting on the support.

For a flat roof (Fig. 3.2.2) the required average support pressure is (the derivation will be shown later under "development")

$$p = \gamma \left(\frac{1}{48} \frac{b^2}{\frac{\sigma_t}{\gamma}} - \frac{\sigma_t}{\gamma} \right)^{1)} \qquad \text{(Ri 1)}$$

p = required support pressure

b = span of flat roof

σ_t = tensile strength of ground

γ = unit weight of ground

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1) For cohesionless soil (σ_t=0) the support load reaches infinity in which case the support load should be taken as full overburden.

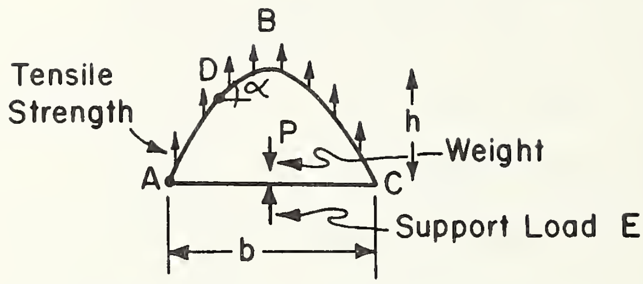

FIGURE 3.2.2 RITTER'S(1879) METHOD : FLAT ROOF

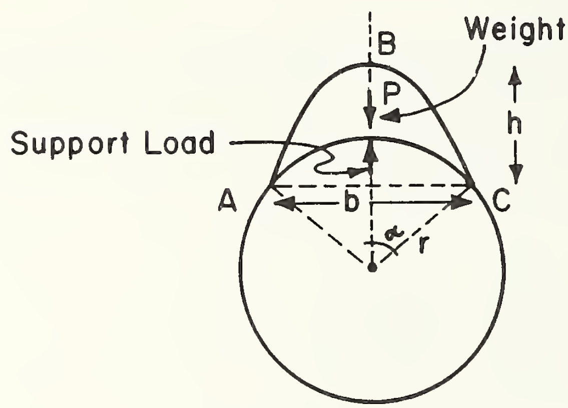

FIGURE 3.2.3 RITTER'S(1879) METHOD : CIRCULAR TUNNELS

No support is required when the following condition for the span is fulfilled.

$$b \leq 4\sqrt{3}\,\frac{\sigma_t}{\gamma} \cong 7\,\frac{\sigma_t}{\gamma} \qquad \text{(Ri 2)}$$

For a circular tunnel the average roof load is (Fig. 3.2.3)

$$p = \gamma\left(\frac{1}{12}\,\frac{r^2}{\frac{\sigma_t}{\gamma}} + \frac{2}{3}\,\frac{\sigma_t}{\gamma} - \frac{3}{4}\,\frac{r^2}{r+2\frac{\sigma_t}{\gamma}}\right) \qquad \text{(Ri 3)}$$

where r = radius of tunnel

No roof support is required for

$$r \leq 4\,\frac{\sigma_t}{\gamma} \qquad \text{(Ri 3a)}$$

The stability of a vertical tunnel wall (Fig. 3.2.4) is considered to be a function of the friction angle (ϕ) and the cohesive strength (c) normalized by the unit weight (γ) (for a detailed description see development). The values of a free standing tunnel wall acted upon only by local gravity is given by the values in Table 3.2.1.

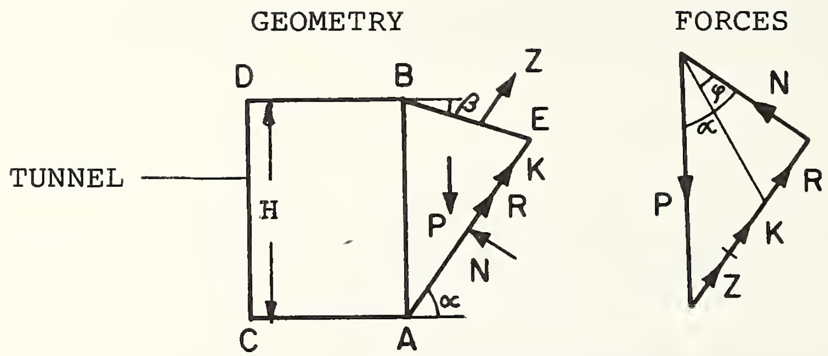

FIGURE 3.2.4 RITTER'S METHOD : WALL STABILITY

TABLE 3.2.1 WALL STABILITY AS A FUNCTION OF COHESION AND FRICTION ANGLE (AFTER RITTER, 1879)

Friction Angle ϕ	β	$\frac{\gamma \cdot H}{c}$
10°	16° 4'	7.9
15°	15°10'	8.4
20°	14°16'	9.0
25°	13°22'	9.6
30°	12°28'	10.3
35°	11°33'	11.1
40°	10°37'	12.1
45°	9°40'	13.2

Ritter assumed that cohesion equals tensile strength.

H = Height of Tunnel Wall

c = cohesion

γ = unit weight of the ground

Development and Underlying Philosophy

As mentioned before, the average support pressure p on a flat crown is the support force E divided by the span b. The support force E is the weight P of the rock that tends to drop out reduced by the tensile strength along the unknown boundary line (Fig. 3.2.2). The boundary curve of the rock mass is derived by determining the maximum support load E

$$E = P - \int_A^C \sigma_t \, ds \qquad \text{(Ri 4)}$$

This results in a parabola that passes through the edges of the crown with a height h that is dependent on the width b and the normalized tensile strength $z = \frac{\sigma_t}{\gamma}$

$$h = \frac{b^2}{16z} \qquad \text{(Ri 5)}$$

The load that has to be carried by the support is thus

$$E = P - \int_A^C \sigma_t \cdot ds = \gamma b \cdot \left(\frac{b^2}{48\frac{\sigma_t}{\gamma}} - \frac{\sigma_t}{\gamma} \right) \qquad \text{(Ri 7)}$$

By assuming that this force is uniformly distributed between A and C one obtains the average support load of Eq. Ri 1.

The support load for a circular tunnel is derived similarly with the assumptions shown in Fig. 3.2.3 which leads to the average support pressure in Eq. Ri 3.

Wall stability is estimated by considering a triangular wedge at the tunnel wall (Fig. 3.2.3) acted upon by its own weight P, a shear resistance on plane AE which consists of the frictional component R and the cohesive component K and a tensile resistance Z along plane BE. It is assumed that cohesion equals tensile strength. The force equilibrium (see Fig. 3.2.4) can be solved to result in

$$c = \frac{1}{2}\, h\, \frac{\sin 2\beta \, \cos(2\beta + \phi)}{\cos\phi \, (1 + 2\tan\beta)} \qquad \text{(Ri 6)}$$

The minimum of Eq. Ri 6 is found by differentiation with respect to β and setting the appropriate equation to zero

$$\tan(4\beta + \phi) = \frac{\cos^2\beta + 2\sin 2\beta \, \sin^2\beta}{\sin^2 2\beta} \qquad \text{(Ri 7)}$$

This equation (Ri 7) cannot be solved directly for β as a function of ϕ. Thus table 3.2.1 has been developed that gives β and $\frac{\gamma H_T}{c}$ as a function of ϕ.

Critique

In Ritter's method only 'local gravity forces' in the vicinity of the tunnel and not the entire stress field in the ground are considered. In particular for the stability of the wall only sliding out of wedges under their own weight is considered and not the possibility of the ground being overstressed (squeezing ground).

A second problematic aspect is the large roof load in cohesionless ground, the roof load in Ritter's formulation is only a function of the tensile strength and not the shear strength. For cohesionless material the predicted roof load may exceed the overburden in which case the overburden should be considered as the possible maximum. These points seem to limit the value of Ritter's method, however of great importance and still valid today are the points Ritter makes in his conclusions:

> "Not too much weight should be given to the above theoretical calculations since there were assumptions made that rarely are accurate in reality. The main purpose of this study was not to provide the tunneler with formulae which would enable him to compute the rock pressure in the interior of the earth without difficulties, this is as such impossible because there are many secondary conditions influencing the problem....Besides this theoretical approach there is also an empirical[1] approach, which, however also with difficulties may lead faster to practically useful results such an empirical approach will support and check the theoretical procedure."

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[1] An empirical approach means here that stresses should be monitored and then related to ground conditions.

Ritter continues and also mentions that besides weight, cohesion (tensile strength), shear strength, also the jointing and chemical alteration may influence the loads. Finally he calls for careful field measurements which had not been used up to that point in time.

Summary of Ritters Method

Ritter (1879) made one of the first attempts to derive the loads on a liner on a rational basis. His method may be considered to be quantitative in nature because it uses the tensile strength together with the dimensions of the opening to arrive at a rockload. It does however not consider the influence of shear strength, and stress state around the opening, facts that were not known at the time of the development. Ritter himself, recognized the limits of his method and called for actual performance monitoring.

3.2.2 Kommerell's Method (Kommerell, 1912, 1940)

Methodology and Application Guidelines.

A roof load is determined as a function of the settlement in the center of the crown a (see Fig. 3.2.5) and the loosening (volume increase) p of the ground. The height of the so called pressure ellipse (whose volume multiplied by the unit weight is the rock load) is

$$h = 100 \frac{a}{p(\%)} \tag{Ko 1}$$

The volume increase p of the ground may be determined in the field from recompaction tests of excavated material. The increase of the recompacted ground volume compared to the initial volume in place is the volume increase. Kommerell quotes typical values of volume increase (Table 3.2.2).

The value of the roof settlement has to be estimated either from experience or it has to be measured in the field.

For tunnels "with side pressure" (squeezing ground) Kommerell proposes to consider a wide ellipse. Two sliding planes are assumed at the base of the springlines inclined at $(45° + \phi/2)$ to the horizontal (Fig. 3.2.6) and acting through the intersection of springline and invert. The width of the pressure ellipse is determined by the intersection of the horizontal line through the crown with these sliding planes. The height of the pressure ellipse is again, a function of the roof settlement and the 'loosening' factor and is given by

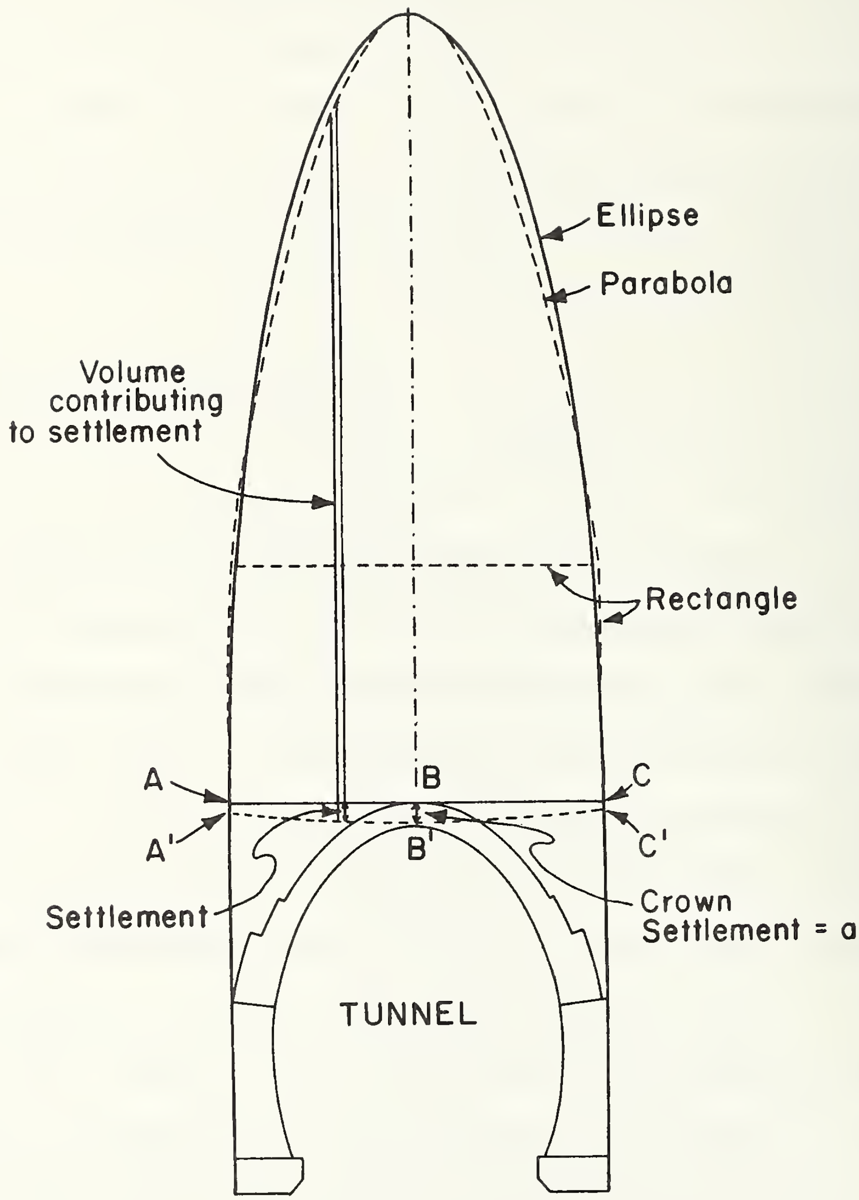

FIGURE 3.2.5 ROCKLOAD IN KOMMERELL'S METHOD
(AFTER KOMMERELL,1940)

TABLE 3.2.2 LOOSENING OF GROUND
(from Kommerell, 1940)

SOIL	LOOSENING p (%)
Light ground	1 to 3
Medium heavy ground	3 to 5
Firm ground (Shale, gravel with clay)	6 to 8
Firm ground	8 to 12
Rock	10 to 15

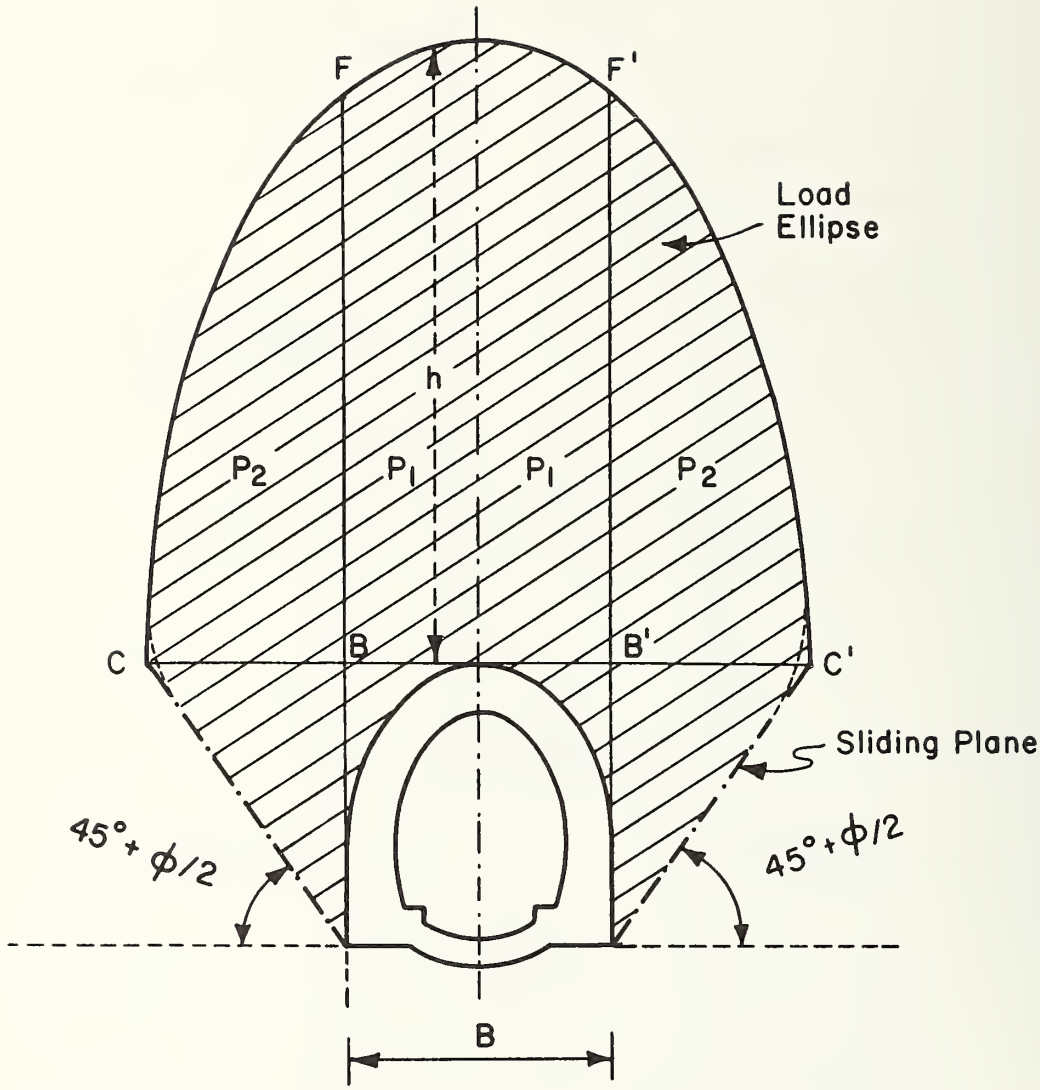

FIGURE 3.2.6 KOMMERELL'S(1940) METHOD IN SQUEEZING GROUND

equation Ko 1. The crown load is determined from the weight of the part of the pressure ellipse acting on the crown of the tunnel (Fig. 3.2.6). The side pressure of the springlines is considered with an earth pressure calculation by assuming a sliding wedge loaded by the part of the ellipse on top (P_2) of the wedge.

Development and Underlying Philosophy

Kommerell (1940) was an engineer with the German Railroads (Deutsche Reichsbahn) and was responsible for the construction of St. Bernard tunnel on the railroad line Metz-Vigy-Anzelingen in the years 1903 to 1906[1)]. A roof collapse occured which subsequently required reworking of a tunnel section and support redesign. This and other observations in this and other tunnels led Kommerell to the study of rock loads and the design of tunnel liners. For this purpose, Kommerell reviewed the literature. The load ellipse, however, is Kommerell's own idea.

Kommerell considers a horizontal plane through the crown of the tunnel (Fig. 3.2.5) initially passing through points A,B,C. During the construction the ground will settle (A',B',C'). The shape of the displaced line is assumed to be

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1) Kommerell was an engineer with the German Railroads (Deutsche Reichsbahn), however, the tunnel mentioned is now in France. During the period 1871-1918 Alsace-Lorraine was part of Germany.

a parabola and the settlement known in the three points. The volume of the loosened volume is determined by assuming that only the ground immediately above each point in the crown contributes uniquely to the loosening, i.e., the settlement of the crown is caused by a volume increase of the ground above each individual point. It is assumed that the shape of the settlement through the crown of the tunnel (wave A'-B'-C' in Fig. 3.2.5) is parabolic. The boundary of the volume contributing to the load is also bounded by a parabola (dashed lines in Fig. 3.2.5). Since the abutments also settle there will be also a load at the abutments and the total load figure is a combination of a rectangle and a parabola (Fig. 3.2.5). Finally, Kommerell makes the assumption that this combination of rectangle and parabola can be substituted by an ellipse. This ellipse has the advantage that only the settlement of the center point has to be known to determine the load figure instead of the settlement in the center of the crown and at the abutments.

Critique

Kommerell's method primarily represents the developer's experience in rather shallow tunnels. The method suggests that it is best to minimize the crown settlement in order to reduce the roof load, however, it does not discuss the fact that there still may be some roof support required even if the crown would not settle at all. In particular, it does not consider the beneficial effect (mobilization of shear strength) of deformation, but only a detrimental one ('loosening') this suggest that it has been developed from and for shallow tunnels. In addition, it is impossible to accurately monitor the crown settlement since some of the deformation occurs ahead of the actual excavation.

The determination of the actual volume increase of the ground is also somewhat problematic. Kommerell recommends to recompact the ground that has been excavated and compare the density of this recompacted material to the density of the in-situ ground. The volume increase (difference) between these densities yields the loosening (bulking) of the ground. However, a problem might arise in case of very loose ground which may yield a negative volume increase, and thus Kommerell's equation (Ko 1) would predict "negative" support load, i.e., no support would be required. Clearly such a case would actually require support.

Andreae (1956) discussed Kommerell's method only briefly, however, he calls the two quantities roof settlement "a" and loosening factor "p": "somewhat problematic".

Summary

Kommerell's method determines the rock load as a function of crown settlement and 'loosening' (=volume increase) of the ground. It is primarily applicable where rock loads in the crown occur. The rock load increases linearly with displacement which means that the method does not consider the benefical effect of deformation, only a detrimental one. The two parameters necessary to predict the roof load, the 'loosening factor p' and crown settlement 'a' are difficult to determine.

3.2.3 Terzaghi's (1946) Rock Load Approach

Methodology and Application Guidelines

Ground conditions are verbally described for nine ground classes (Terzaghi, 1946). A rock load is associated with each ground class which is a function of either only the width of the opening (B) or the sum ($B+H_t$) of width (B) and height (H_t) of the opening, with the exception of swelling ground where an absolute value is given. The nine ground classes, the ground descriptions and the corresponding rock loads are shown in Table 3.2.3. The support can then be designed with these rock loads.

In their text, that includes Terzaghi's rock load recommendations, Proctor and White (1946) describe a graphical procedure for steel set design and also provide a series of tables relative rock loads to type and spacing of steelset for a given tunnel diameter.

The rock load table should not be used without carefully studying the associated detailed description of the ground conditions by Terzaghi and to consider at least the footnotes that apply to several of the classes. Footnote 1 applies to ground classes 4 to 6: the rock load values given in Table 3.2.3 apply to the described ground conditions if the tunnel is located under the water table (possibly only temporarily); If the tunnel is located permanently above the water table the rock load can be reduced by 50%. Footnote 2 concerns the presence of shale in tunnel which may behave as

TABLE 3.2.3 TERZAGHI'S ROCKLOAD RECOMMENDATIONS (FROM TERZAGHI, 1946)

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 $(B + H_t)$.[1]

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

1. The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by fifty per cent.

2. Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shale may behave in the tunnel like squeezing or even swelling rock.

If a rock formation consists of a sequence of horizontal layers of sandstone or limestone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so-called shale and rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

squeezing or even swelling rock: Even if there are interlayers of shale between competent rock, these layers may squeeze and lead to settlement of the roof with subsequent increases in roof loads.

Development and Underlying Philosophy

Terzaghi's rock load recommendations have three different bases

- potential observed overbreak for massive to moderately jointed rock
- arching tests in sand for comparisons with field measurements
- actual field measurements i.e., the observation of crushed wooden blocks of known strength in timber supported tunnels in the eastern alps.

Terzaghi (1946) gives a detailed description of his way of deduction in Chapter 4 preceeding the table of rock loads in Chapter 5 of Proctor and White: "Rock Tunneling with Steel Supports".

The most important aspects of the three basic considerations by Terzaghi are summarized here.

Rock Loads based on overbreak. The rock load recommendations for ground classes 1 to 3 are based on considerations of potential overbreak. For these three classes Terzaghi studied the possible overbreak in tunnels. In Class

2, unweathered horizontally stratified rock, the stability depends on the spacing of the transverse joints. For widely spaced transverse joints with respect to the opening (see Fig. 3.2.7) a rectangular cross-section will be stable. For more closely spaced joints a dome is formed in the crown. However, if the required cross-section of the tunnel does not follow the natural arch, the rock that tends to drop out of the crown has to be supported, i.e., overbreak tends to occur.

Terzaghi lists the following factors as most important in influencing overbreak:

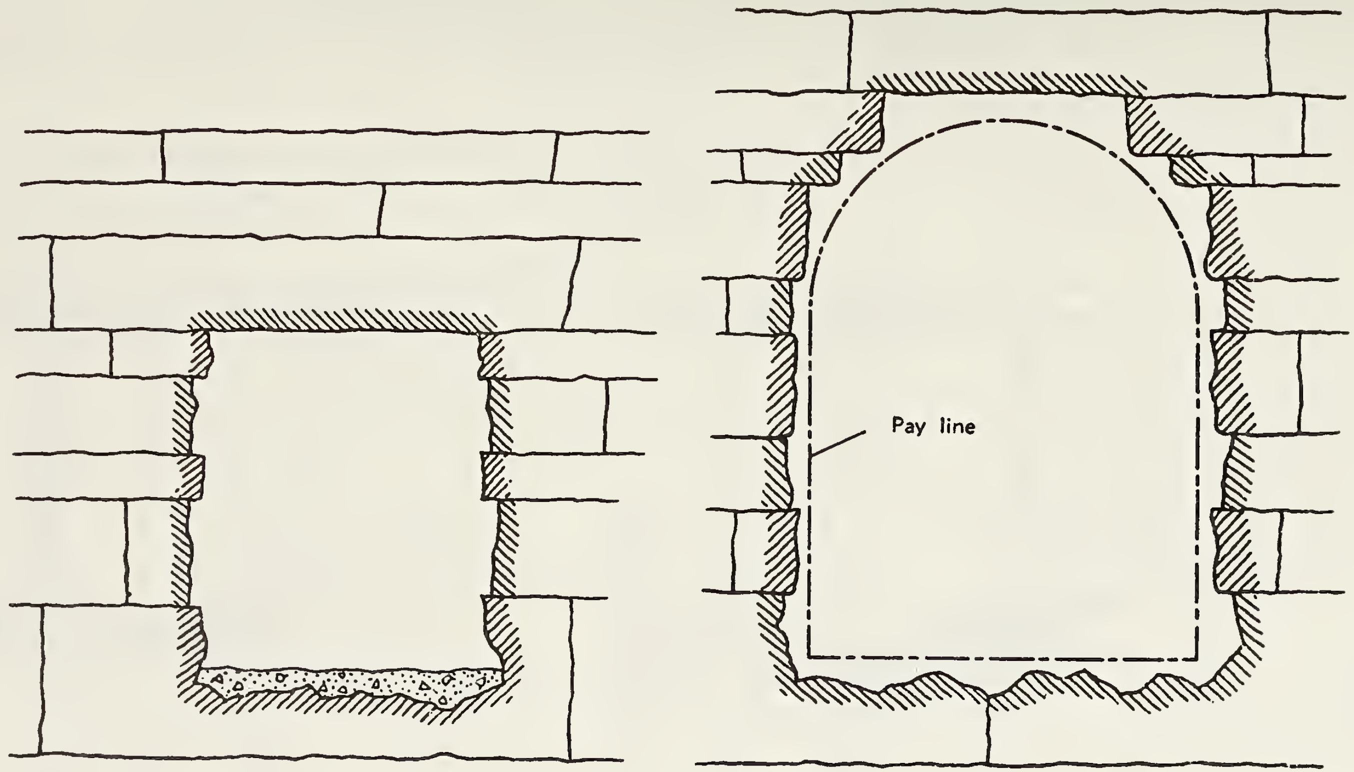
" . . 1) Spacing between joints.

2) Shattering effect of blasting on the rock located beyond the payline.

3) Distance between the working face and the roof support.

4) Length of time which elapses between the removal of the natural support of the roof and the installation of the artificial support. . ."

The effect of distance from face to the support is illustrated in Fig. 3.2.8. The closer the support is placed to the face the smaller the overbreak due to a three-dimensional dome action (Fig. 3.2.8 a,b). The maximum possible height of overbreak in horizontally stratified rock is probably 0.5 B (Fig. 3.2.9a). The recommended rock load thus includes the range from 0 to 0.5 B. The derivation for vertically stratified rock striking parallel to the tunnel is similar (Fig. 3.2.9b). In this case the maximum roof load as assessed



Bridge action in rocks with widely spaced transverse joints.

Bridge action in rocks with closely spaced transverse joints.

FIGURE 3.2.7 EFFECT OF TRANSVERSE JOINTING ON TUNNEL STABILITY (FROM TERZAGHI,1946)

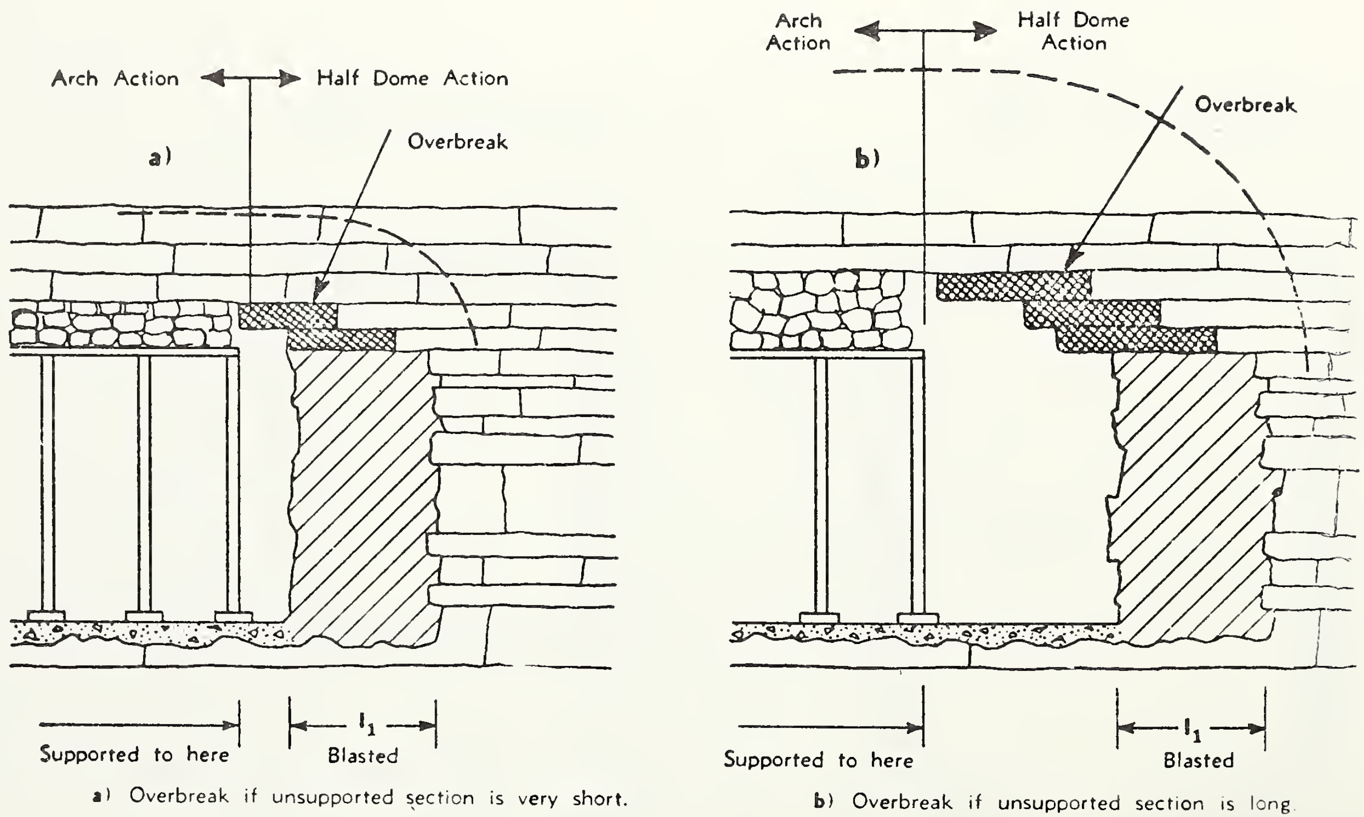


FIGURE 3.2.8 EFFEECT OF UNSUPPORTED LENGTH ON OVERBREAK
(FROM TERZAGHI,1946)

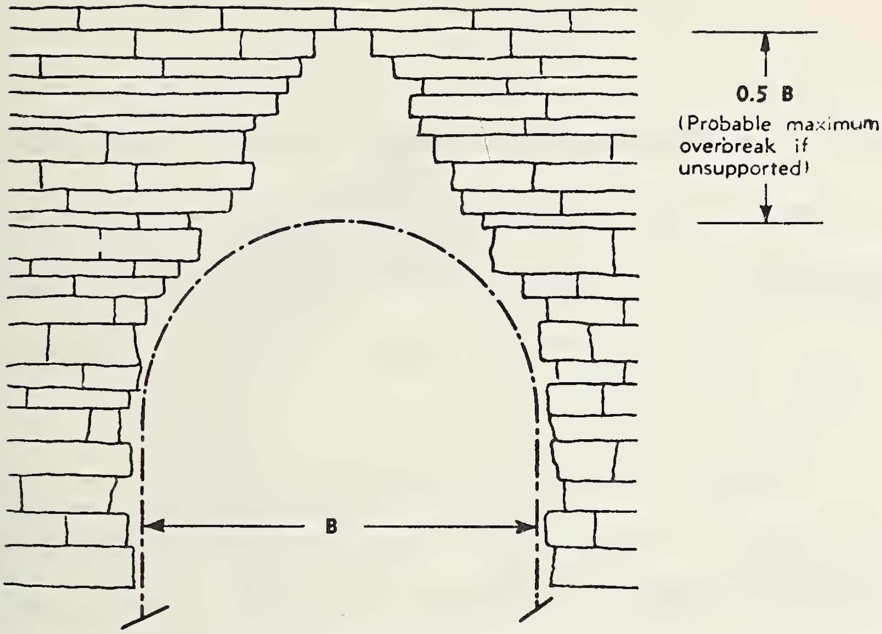


FIGURE 3.2.9a MAXIMUM PROBABLE OVERBREAK IN HORIZONTALLY STRATIFIED ROCK (FROM TERZAGHI,1946)

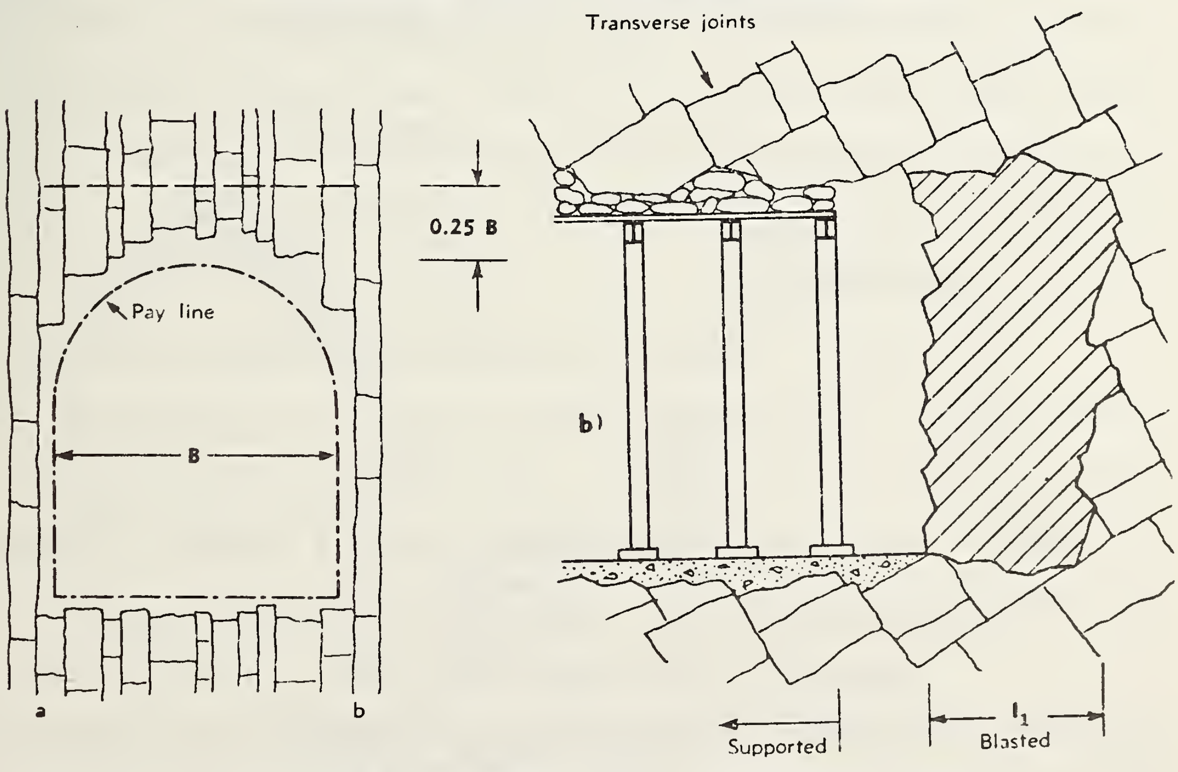


FIGURE 3.2.9b TUNNEL IN VERTICALLY STRATIFIED ROCK (FROM TERZAGHI,1946)

by Terzaghi is 0.25 B, leading to a range of rock load from 0 to 0.25 B. Terzaghi does not seperately consider rock strata striking perpendicular to the tunnel; for this case the values will, however, not exceed those for strata striking parallel to the tunnel.

For inclined strata Terzaghi states that the overbreak tends to produce a peaked roof, the maximum value of the roof load being in the order of 0.25 B for steep strata and 0.5 B for gently inclined strata. More important, however, is the consideration of the possibility of sliding wedges at the springlines (Fig. 3.2.10). For this case Terzaghi proposes to perform a graphic analysis; the effect of water has to be included in the friction angle (i.e., a "total" friction angle has to be considered).

Class 3 applies to moderately jointed, massive rocks, where in absence of support little or no overbreak may occur as shown in Fig. 3.2.11. With tightly blocked support Terzaghi expects thus smaller loads than the maximum overbreak and thus gives a range of rock load heights: 0 to 0.25 B.

<u>Tunnels in blocky and seamy rock</u>. The rock load recommendations for classes 4 to 6 are based on field measurements and for class 6 also on arching experiments. The field data used are those reported by Bierbaumer (1913) (measurements in timber supported tunnels) with some interpretation by Terzaghi;

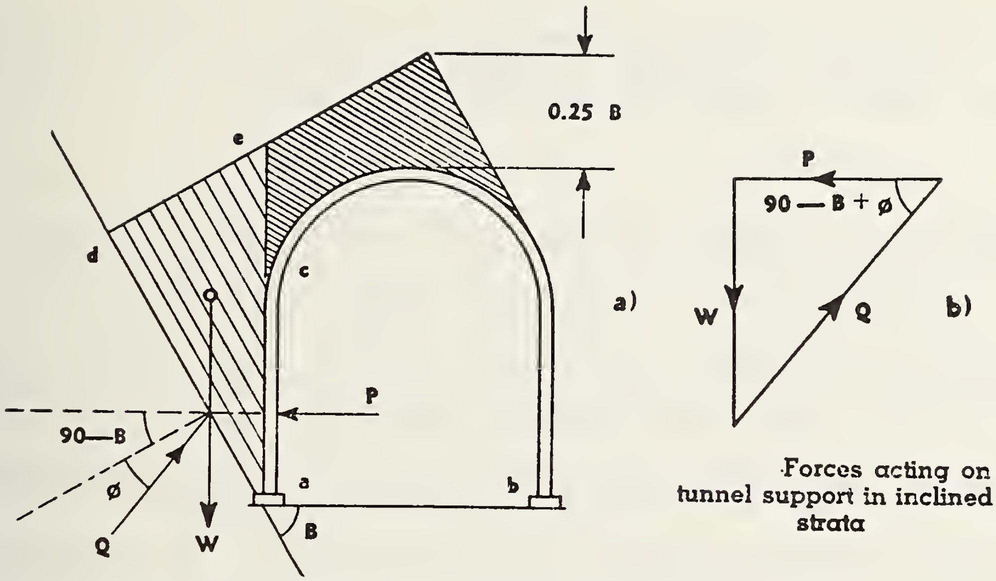


FIGURE 3.2.10 UNSTABLE WEDGES AT SPRINGLINES
(FROM TERZAGHI,1946)

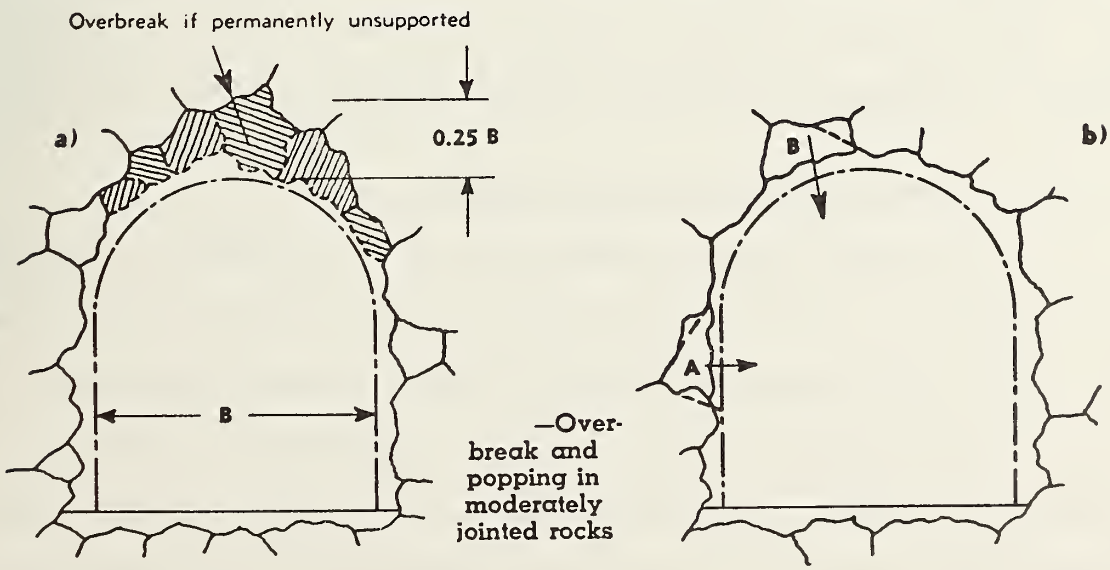


FIGURE 3.2.11 OVERBREAK IN MODERATELY JOINTED ROCK
(FROM TERZAGHI,1946)

Bierbaumer reports absolute values of pressure, Terzaghi has interpreted them as rock load factors.

Arching tests in sand (Terzaghi, 1936) have been reinterpreted by Terzaghi as rock load factors and are shown in Table 3.2.4 together rock loads for moderately blocky to very blocky and shattered rock (corresponding to classes 4 and 5 in Table 3.2.3). The above mentioned arching test included experiments on loose and dense sand above and below water table. The minimum values were reached for displacements (at trap door level) equal to 0.01 times the trap door width, the maximum values for settlement in the order of 0.1 times the trap door width. The detailed reasons for the increase of the ultimate loads by 15% beyond the initial loads could not be determined.

Tunnels in Squeezing and Swelling Rock

The rock load recommendations for support in squeezing ground (classes 7 and 8) conditions are also based on Bierbaumer's (1913) work. In Proctor and White (1946), Terzaghi has not referenced Bierbaumer, however, he uses Bierbaumer's table in the Chapter on tunnel geology in Redlich, Terzaghi, Kampe (1929). Bierbaumer has given his rock load recommendations as absolute load values in timber supported tunnels. Terzaghi has modified them and relates them to the dimensions of the tunnel.

TABLE 3.2.4 SUMMARY OF ARCHING TESTS AND COMPARISON TO FIELD MEASUREMENTS (TERZAGHI, 1946)

Comparison Between Rock Load (in feet) in Sand and in Blocky and Seamy Rock

Material		*Above water table*		*Below water table*[1]	
		$H_{p\,min}$	$H_{p\,max}$	$H_{p\,min}$	$H_{p\,max}$
Dense sand[2]	Initial	$0.27\ (B + H_t)$	$0.60\ (B + H_t)$	$0.54\ (B + H_t)$	$1.20\ (B + H_t)$
	Ultimate	$0.31\ (B + H_t)$	$0.69\ (B + H_t)$	$0.62\ (B + H_t)$	$1.38\ (B + H_t)$
Loose sand[2]	Initial	$0.47\ (B + H_t)$	$0.60\ (B + H_t)$	$0.94\ (B + H_t)$	$1.20\ (B + H_t)$
	Ultimate	$0.54\ (B + H_t)$	$0.69\ (B + H_t)$	$1.08\ (B + H_t)$	$1.38\ (B + H_t)$

Moderately blocky[3] $H_{p\,in} = 0$ increasing up to $H_{p\,ult} = 0.35\ (B + H_t)$

Very blocky and shattered $H_{p\,in} = .60\ (B + H_t)$ increasing up to $H_{p\,ult} = 1.10\ (B + H_t)$

1. Values are roughly equal to twice those for dry sand.
2. Values computed on basis of laboratory tests.
3. Values computed on the basis of the results of observations in railroad tunnels.

The rock load for swelling rock seems to be based on literature review, however, details could not be determined.

In addition to the rock load factors, Terzaghi gives recommendations for the construction of the tunnel support. For swelling rock Terzaghi proposes to leave some space behind the masonry which allows the rock to swell. Alternatively, Terzaghi proposes to use narrow flanged strong steel sets which would be able to sustain large pressures. Since the flanges of the steelsets would be small the rock could flow around the steelsets without damaging them (Fig. 3.2.12a). To prevent rock from falling into the tunnel lagging placed on the interior flange is proposed (Fig. 3.2.12b). Alternatively Terzaghi also recommends to place compressible spacers in the ribs in order to allow yielding. Finally, Terzaghi proposes to take load measurements in order to obtain a better understanding of the actual loads.

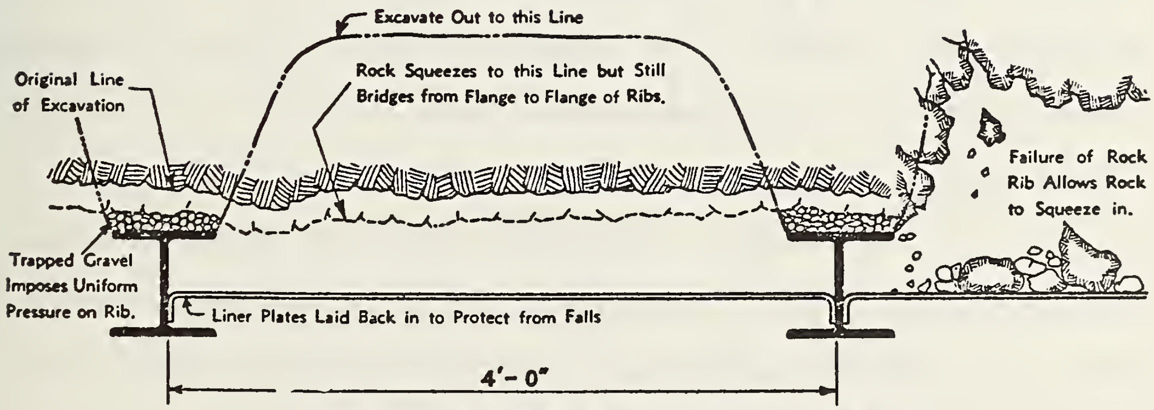
Critique

Terzaghi developed his rock load recommendations for steel supported tunnels based on

- considerations of overbreak
- arching tests
- Bierbaumer's results
  (field measurements in timber supported tunnels)

(a)

Flow of swelling rock around tunnel rib



Over-mining in stiff swelling ground to induce softening

In stiff swelling ground it is necessary to allow the ground to squeeze some undetermined amount to soften it. If the ground is not sufficiently soft to extrude between the ribs when the shrinkage provided in the crush lattices is used up, slots are excavated beyond the ribs, as shown here. This is repeated until extrusion between the ribs is established. The squeeze is then allowed to run its course before concreting.

(b)

FIGURE 3.2.12 GROUND SUPPORT BEHAVIOR IN SQUEEZING AND SWELLING ROCK (PROCTOR AND WHITE,1946)

Terzaghi quotes ranges of support in the rock load Table (table 3.2.3). In the text he discusses how he arrived at the tables and what factors lead to particular rock loads. Thus in applying the tables one should not only select the maximum value of rock load for the particular ground conditions (or possibly the average one) but rather one should also consider the effects of construction on the rock load.

For ground classes 1 through 6 Terzaghi proposes to tightly backpack the support and to place it early in order to prevent loosening and increases in loads. At the present time we cannot say whether this approach of minimizing displacements actually leads to minimum support requirements for classes one to six.

For classes 7 to 9 the rock load recommendation are based on observations in timber supported tunnels that were often excavated in sequence. (base drift, crown drift, slot to crown, widening; see Fig. 3.2.13) This sequence allowed some unknown amount of deformations to occur prior to excavating the full cross section. In addition the timber supports may be more deformable than steel supports which has also an effect on support loads. These phenomena are explained in Fig. 3.2.14.

If the excavation sequence allows for less deformation prior to the placement of the support than for the base cases used by Terzaghi in deriving his method, the load on the support will be larger.

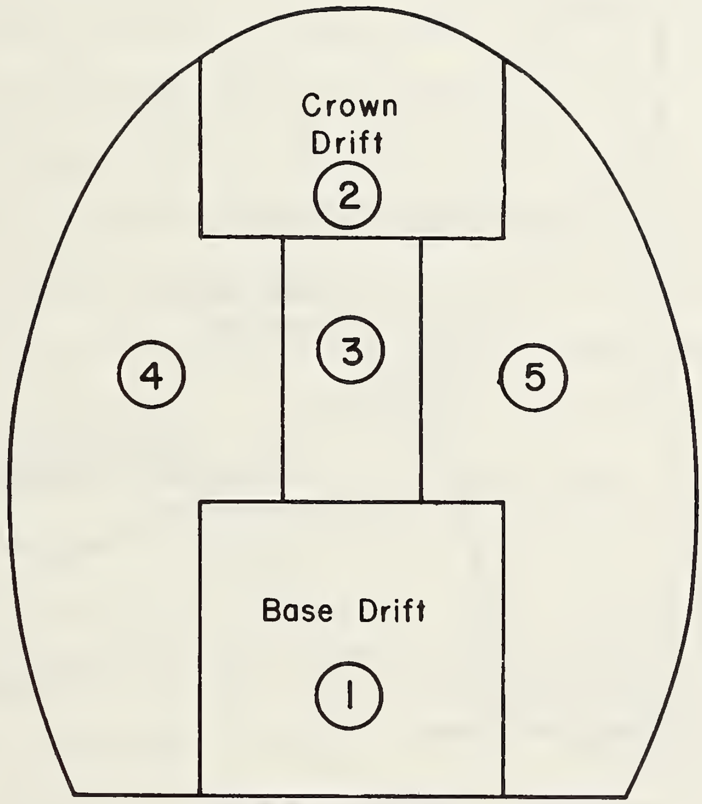


FIGURE 3.2.13 EXCAVATION SEQUENCE IN TIMBER SUPPORTED TUNNELS(AFTER ANDREAE,1948)

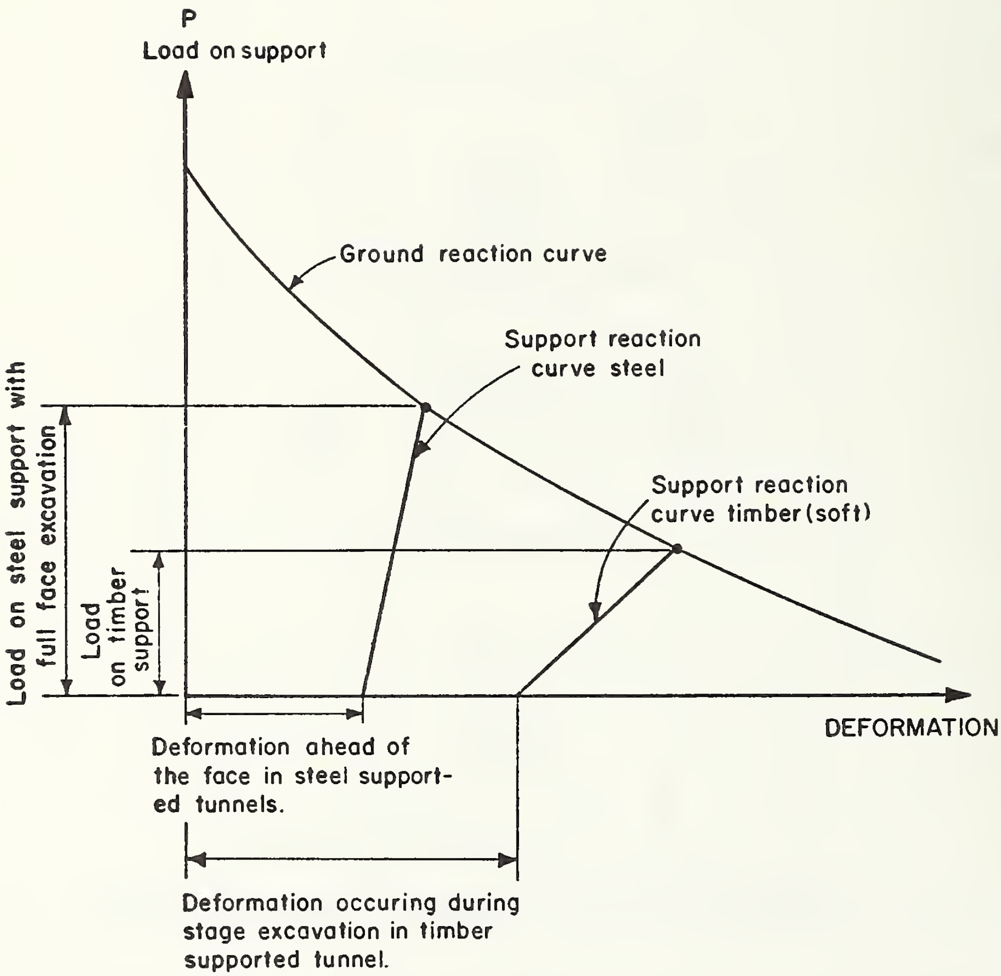


FIGURE 3.2.14 EFFECT OF EXCAVATION-SUPPORT PROCEDURE AND SUPPORT STIFFNESS ON SUPPORT LOADS

Summary

Terzaghi's method is a synthesis of work that was published before 1946 on tunnel construction. It is based on a detailed literature study which includes Kommerell's and Bierbaumer's method as well as many case studies. Additional recommendations are given based on considerations of possible overbreak and arching tests. The field measurements that were used by Terzaghi in the development have been reported earlier by Bierbaumer, Terzaghi used the results in a modified form.

Terzaghi considers nine classes that include a wide range of ground conditions, he differentiates between three basic mechanisms: loosening, squeezing, and swelling. In addition to rock load recommendation detailed construction procedures are described for steel set support. These recommendations were developed primarily from a literature survey which included primarily timber supported tunnels with particular excavation methods. The excavation support methods applied today are, however, different and the rock load recommendation may thus not be applicable to these newer methods. The deviation of rock load may be in either direction (larger or smaller).

3.2.4 Stini's (1950) Rockload Recommendations

Methodology and Application Guidelines

Stini (1950) gives rock load recommendations for the design of timbered tunnel supports. Ground conditions and ground behavior are verbally described for nine classes and related to a rock load (Table 3.2.5). The rock load values given in table 3.2.5 are valid for tunnels with width of 4 to 5m. For tunnels with smaller or larger width Stini recommends to decrease or increase the reference rock load by 10% for each one meter change of width. (e.g. for a tunnel of 2m width the rock load should be reduced by 30%, for a tunnel of 6m width increased by 10 to 20%.) This may also be expressed by an equation.

$$H_B = H_{Bo} \cdot \left(1 + \frac{B-Bo}{10}\right) \qquad \text{(St 1)}$$

B = actual span of tunnel in meters

Bo = span of 'standard' tunnel (Bo=4 to 5m)

H_B = rock load for tunnel of span B

H_{Bo} = rock load for 'standard' tunnel from table 3.2.5.

Note, that Stini does not give a fixed value for the 'standard' span but a range from 4 to 5 meters which leads to some further variations in Eq. St 1.

TABLE 3.2.5 STINI'S ROCKLOAD RECOMMENDATIONS (AFTER STINI, 1950)

HEIGHT OF ROCK PRESSURE (in meters) for tunnel with 4 to 5m span	GROUND DESCRIPTION	DETAILS OF GROUND BEHAVIOR	GROUND TYPES
0-0.5m	stable to very stable ground	Very little loosening along the circumference due to excavation work.	
0.5 to 1.0m	ground with satisfactory stability	Afterbreaks only caused by more or less unavoidable loosening caused by the excavation,becomes only important with time.	Mica Shists (rich in Mica) Schistose Gneisses.
1 to 2m	lightly afterbreaking ground	Small loads on support, only little loosening during excavation becomes more 'lively' after some time.	Heavily jointed quartzitic phyllites, chlorite schists, laminated calcitic mica schists rich in mica.
2 to 4m	medium afterbreaking ground	Ground becomes afterbreaking after initial stability.	Heavily jointed Dolomites in shear zones (Störungsstreifen).
4 to 10m	afterbreaking ground	The ground is quite stable after excavation, however, afterbreaks follow rapidly and strongly. Timbering is lightly loaded.	Clayshale, some thinly laminated brittle sandstones. 'Squeeze' Dolomites in shear zones.
10 to 15m	strongly afterbreaking ground	Loosening immediately after excavation, local rooffalls.	Thinly laminated marly/ clayey sandstones,phyllites rich in mica, some hard marl, laminated calcitic schists, lateral moraine (in the Zwenberg Tunnel up to 15m).
15 to 25m	lightly squeezing ground	The dense and strong timbering is intensely loaded.	Black schists, lightly laminated schists, quartzitic 'silk' schists rich in mica, hard rock with narrowly spaced clay filled discontinuities, rock in medium shear zones, many marlshales, humid clay ('dry' clay in Ratkonya Tunnel 14 to 19m), humid till (in the Dössener Tunnel: 20 to 25m).
25 to 40m	medium squeezing ground	The very dense and strong timbering is heavily loaded.	Brittle (mürbe) thinly laminated 'silk' schists, laminated schists (phyllites), soft marls, graphitic schists (black schist), wet clay (in the Ratkonya Tunnel: 27m).
40 to 60m	heavily squeezing ground	The timbering crushes even if it is very strong and dense.	Clay Shales, marls, large shear zones. Quetschgesteine.

Development and Underlying Philosophy

Stini has performed a careful literature review, in particular he cites Bierbaumer, and Kommerell along with many others. The rock load recommendations are based on this literature review and Stini's personal experience. Although Stini's (1950) recommendations were published after Terzaghi's (1946) Stini had developed them independently. Stini added Terzaghi's rock load recommendations in his text, however, he states that Proctor and White (Terzaghi) became only available after he had received the initial slip proofs of his (Stini's) book. Note that the five classes for the worse ground conditions correspond to the Bierbaumer's (1913) measurements, as does the description of the timber support. Thus Stini and Terzaghi seem to have used both the same source.

Critique

Stini's rock load recommendations give indications on what rock loads a timber support will experience. When applying these recommendations to other types of support one has to consider the fact that other types of support may be stiffer than timber. Each described state of loading of a timber support has an associated deformation. If, however, a stiffer support system is selected the loads that will be experienced by that support system may be different as illustrated in Fig. 3.2.15.

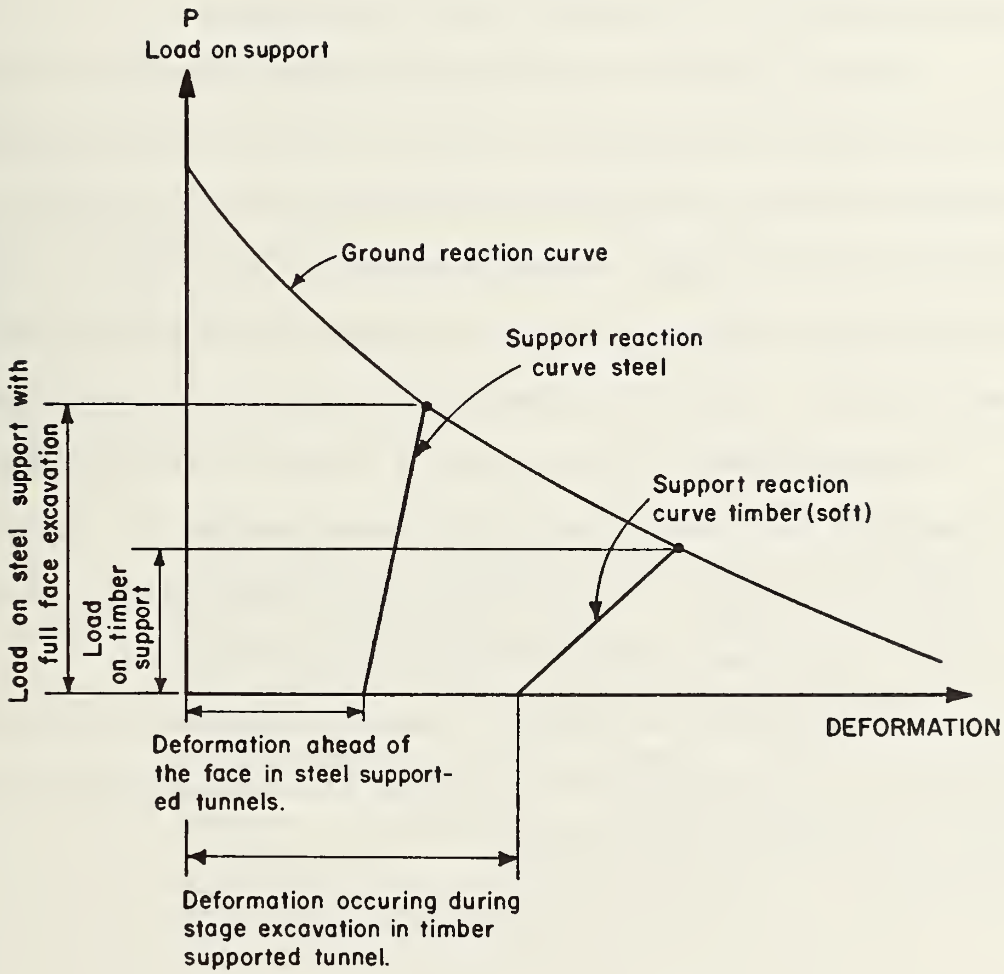


FIGURE 3.2.15 EFFECT OF EXCAVATION--SUPPORT PROCEDURE AND SUPPORT STIFFNESS ON SUPPORT LOADS

Summary of Stini's Method

Ground descriptions are verbally described for nine ground classes and related to a rock load.

Stini's method is based on a detailed literature survey and personal experience. Stini used also Bierbaumer's field measurements in developing his tables which explains that his recommendations are very similar to Terzaghi's. Stini reports the rock loads as absolute heights of rock and not as rock load factor. Also the effect of the size is not proportional to tunnel width. Stini gives a detailed description of rock types that fall in each category which makes the application by the uninitiated but geologically knowledgeable user possibly less difficult than Terzaghi's descriptions.

The method is based on timber supported tunnels thus care has to be taken when applying the recommendations for other construction procedures.

3.2.5 Bierbaumer's Method

Methodology and Application Guidelines

Bierbaumer (1913) published a treatise on the design of tunnel liners which is based on one hand on a detailed survey of literature up to that time and on the other hand on Bierbaumer's own field observation (he was an engineer with the Royal-Imperial Austrian Railroads). For the design of the final liner (masonry, quarry stones), Bierbaumer recommends to estimate the load on the final liner based on the observed loads on the initial timber support:

> At the turn of the century initial timber support followed by a masonry liner was the usual support configuration for railroad tunnels. Tunnels were excavated in stages starting with a timber-supported pilot tunnel which was followed by other timber-supported drifts until finally the full cross-section was excavated and supported by timber. The placement of the masonry liner followed while the timber support was removed in stages. With such a procedure the final liner is in effect placed at a certain distance from the heading of the pilot tunnel. This distance varied for different projects but was usually in the range of a hundred or more meters.

Bierbaumer recommends estimating the loads on the timber support in the full cross-section from the performance of the posts and cross-beams. Stresses in the posts can be estimated by observing how deep the posts are seated in the cross-beams (biting-in). Rock loads could be estimated and then the final liner designed based on these "observed stresses." Bierbaumer's stress estimates in the timber support are based on tests that he performed with timber connections (Table 3.2.7; Figure 3.2.16; see further comments in section on development). These observations (Table 3.2.7) are a substitute for direct load or stress measurements.

From his experience and a literature survey, Bierbaumer established the rock load recommendations as shown in Table 3.2.6, which consist of five ground classes. For each class, ground conditions, a range of initial and final rock loads, the type of (temporary) timber support and its state of loading, as well as recommendations for the final support (dimensions, type of masonry, invert support) are given. In the preconstruction phase the type of timber support may be estimated based on the ground description in Table 3.2.6. However during construction the behavior of the timber support should be observed, as discussed above, to determine the rock load which is then used to design the final liner (again using Table 3.2.6).

Bierbaumer points out that the largest probable rockload should be used to design the final liner. However it is not always possible to estimate the largest rockload based only on observations of the temporary timber support, particularly in case of swelling rock. Bierbaumer quotes the experience in the Bosruck-tunnel, which required only

TABLE 3.2.6 BIERBAUMER'S (1913) ROCKLOAD AND SUPPORT RECOMMENDATIONS

GROUND DESCRIPTION	INITIAL SUPPORT: ROCK LOAD (t/m^2) during excavations	INITIAL SUPPORT: ROCK LOAD (t/m^2) long-term	TEMPORARY TIMBER SUPPORT: type *)	TEMPORARY TIMBER SUPPORT: observed 'straining' **)	REMARKS FOR ROCK LOAD	FINAL LINER: TYPE OF CROSS-SECTION	THICKNESS OF LINER (centimeters), for single track tunnel: crown	for single track tunnel: abut-ment	for double track tunnel: crown	for double track tunnel: abut-ment	REMARKS FOR FINAL LINER
Rock more or less after breaking	0	8-12	wide and light	none or minor	Little 'Loosening' Pressure	Liner in quarry stone	50-60	70-80	50-80	80-100	No Invert
Cohesive gravel, very after-breaking rock 'Mild' rock mass with small over-burden	10	35	wide and strong	little	Larger 'Loosening' Pressure that are experi-enced during mining	for lightly squeezing ground Abutment: Quarry stone Crown: Strong arch in Quarry stones	70-80	100-120	90-100	130-150	Invert in quarry stone for cohesion-less ground
Cohesionless gravel, strong-ly afterbreak-ing rock (crown failures)	20-25	35	dense and strong	not excessive	Large 'Loosen-ing' pressures that are ex-perienced during mining. Probably dif-ficult to stablize.	for lightly squeezing ground Abutment: Quarry stone Crown: light arch in quarry blocks	60	90	80	120	Invert arch in quarry stone for cohesionless ground
'Mild' ground, larger over-burden, squeez-ing	35	50	very dense and strong	important	-	for heavily squeezing ground Crown: Rough rock blocks Abutment: Quarry stones	80	120	120	180	Invert arch in quarry stone
'Mild' ground very large overburden, heavily squeez-ing	50	120	very dense and as strong as pos-sible (hard timber)	until failure	-	for very heavily squeez-ing ground Crown: 'trimmed' rock blocks Abutment: Rough rock blocks	80-100	120-150	120-150	180-200	Invert arch in rough rock blocks

*) literally translated; wide refers to spacing, light and strong referes to strength of timber support
**) This is a translation of "inanspruchnahme" which could also be translated as state of loading. Since timber is quite compressible, a typical deformation is associated with each stress level.

minimal support during excavation but had to be reconstructed one year after completion in the zone where swelling ground was encountered. (Bierbaumer, however, does not give any recommendations on how tunnels in swelling ground should be designed.)

In addition to the basic design recommendations, Bierbaumer also comments on construction details that should be implemented if reconstruction work is to be avoided:

- Limit loosening of the ground, thus careful excavation and careful use of explosives, careful backpacking of lagging, do not leave fully excavated cross-section supported only by timber for extended periods of time, place masonry liner immediately after excavation to the full cross-section.
- Tight backpacking between masonry and rock is necessary, thus: remove timbering completely, backfill voids with masonry, place masonry tightly to ground.

Development and Underlying Philosophy

Bierbaumer's rock load recommendations are based on observations of temporary timber supports in tunnels. Timber posts in the temporary supported tunnel often buckle, thus it seems possible to estimate the loads in the post from their buckling strength and then to backfigure rock loads. The posts in the timber-supported tunnels rest on cross beams (timber blocks). The loads in the support can be estimated much faster and easier from observations of these timber blocks. Bierbaumer performed tests on two types of timber blocks (Figure 3.2.16). In both cases the outer pieces were round timbers (dry pine) of 10cm diameter, the center piece (timberblock) was a 10cm diameter round log or a 5cm rectangular block. For the center piece different types of timber were used (birch, oak, larch, pine). The results of the load tests did not show differences between the two configurations if the same types of wood were used. Significant differences were observed between center pieces in soft or hard wood. The average observed loads are shown in Table 3.2.7. Bierbaumer notes that the timbers were not more carefully prepared than they would be in actual tunneling. The load monitored in the center pieces for stage 3 (first sign of destruction) corresponds to the strength of the timber perpendicular to the fibers. With the results in Table 3.2.7 and observations in temporary supported tunnels, Bierbaumer derived the recommendations in Table 3.2.6.

Critique

Although support deformations are to some extent used to derive stresses and rock loads, only the latter are used in designing the final support. Since the timber supports buckle and also shorten due to the compression of the

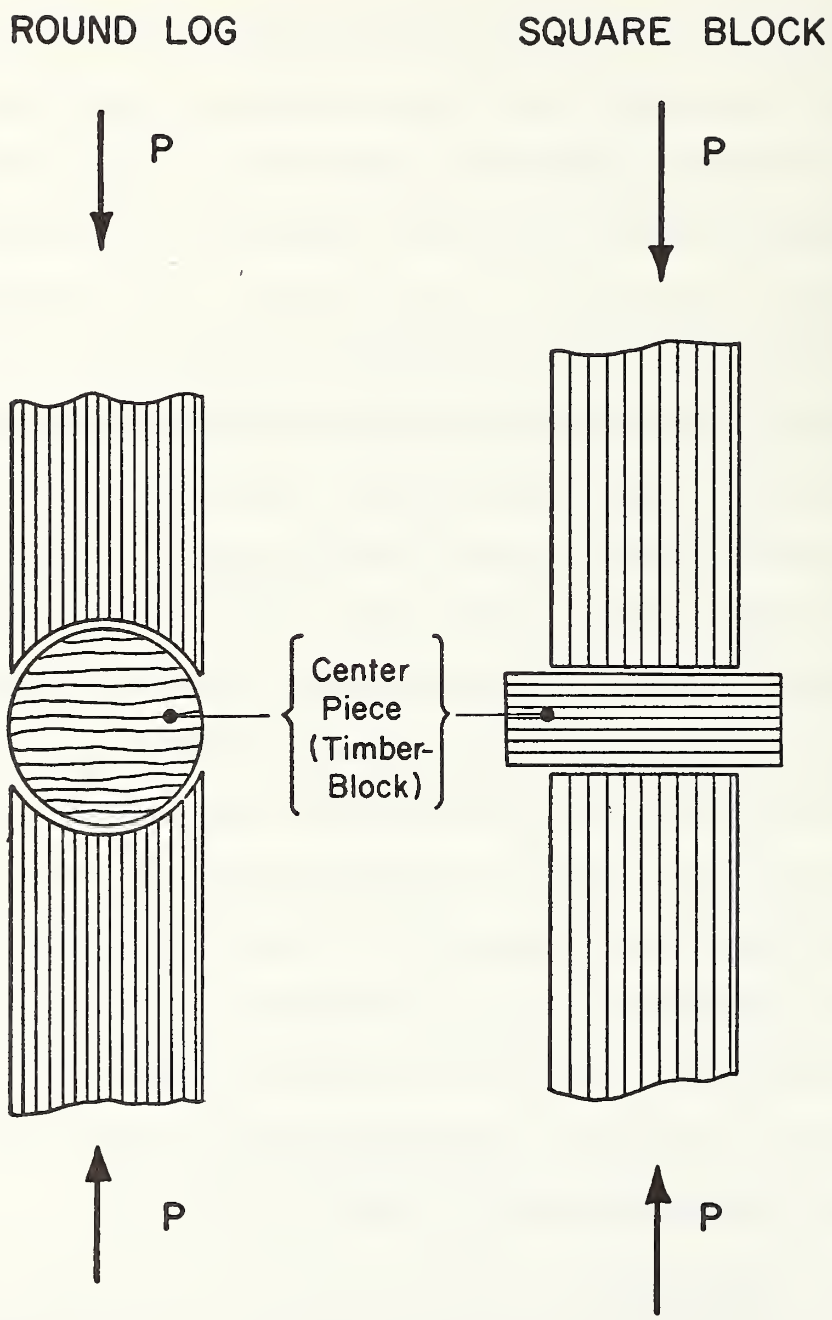


FIGURE 3.2.16 LOAD TESTS ON TIMBER BLOCKS
(AFTER BIERBAUMER, 1913)

TABLE 3.2.7 BIERBAUMER'S TESTS ON TIMBER BLOCKS (AFTER BIERBAUMER, 1913)

STAGE	OBSERVATIONS ON THE TIMBER BLOCKS	CENTER PIECE IN SOFT TIMBER (STRESS LEVEL)	CENTER PIECE IN HARD TIMBER (STRESS LEVEL)
1	Intimate contact of the timber surfaces without 'biting-in' of the surface.(1)	2.5 MPa ($25\ kg/cm^2$)	5 MPa ($50\ kg/cm^2$)
2	Strong 'biting-in' of the timber but without any signs of destruction of the center piece.(1)	5.0 MPa ($50\ kg/cm^2$)	10 MPa ($100\ kg/cm^2$)
3	First signs of destruction of the center piece (crushing and cracking).	7.5 MPa ($75\ kg/cm^2$)	15 MPa ($150\ kg/cm^2$)
4	General destruction of center piece, continuous cracks and seperations.	10 MPa ($100\ kg/cm^2$)	20 MPa ($200\ kg/cm^2$)

(1) 'biting-in' describes how deep the posts (see Fig. 3.2.16) are pressed into the cross beam, i.e., how deep the posts 'bite' into the cross beam.

blocks, a certain state of deformation of the ground support system is associated with each level of loading. These total deformations of the timber support are, however, unknown. Additional deformation of the ground may have occurred during the stagewise enlargement from a pilot tunnel to the full cross-section. Thus the total deformation may be substantial, but it is unknown. The rock load recommendations given by Bierbaumer are thus only applicable for a particular construction procedure where similar deformations occur (timber-supported tunnels and stagewise excavation).

Of greatest importance are Bierbaumer's recommendation to observe the actual performance of the temporary support and to design the final liner based on these observations (rock loads).

Summary

Bierbaumer observed the performance of timber-supported tunnels and used load tests on timber blocks to estimate the stress level in the support. By combining these observations he was able to backfigure the rock load. The rock load recommendations should only be used for a similar construction procedure, specifically that in stagewise timber-supported tunnels.

Bierbaumer proposes an observational procedure for the design of the final liner based on the observations of the temporary support. This observational approach has a definite advantage over a purely theoretical prediction using the tables.

3.3 QUALITATIVE DIRECT METHODS (TYPE B)

The ground conditions and/or the ground behavior with respect to tunneling are described verbally (mostly qualitatively) and are directly (i.e., not through a rock load) related to an excavation procedure and support requirements. This can be expressed graphically as shown in Fig. 3.3.1. Empirical relations of this type have been used during the construction of the Straight Creek Tunnel (now Eisenhower I - Tunnel) and the parallel Eisenhower II - Tunnel in Colorado (Section 3.3.1). One of the most advanced empirical methods of this type is the one used in conjunction with the New Austrian Tunneling Method (Section 3.3.2).

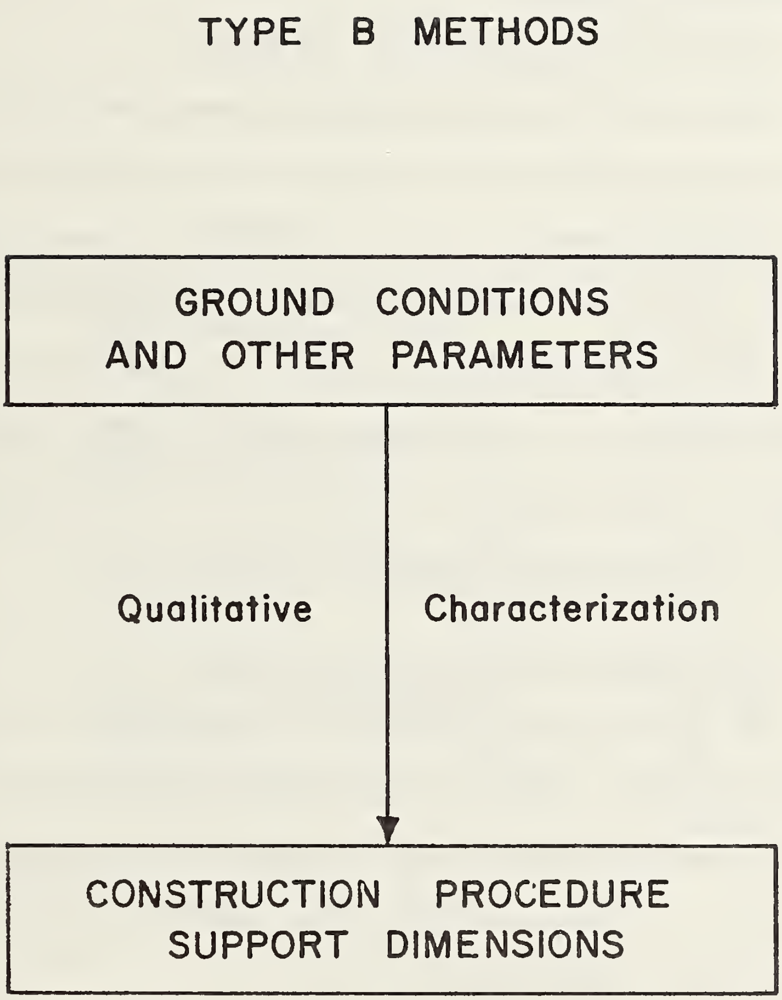


FIGURE 3.3.1 QUALITATIVE DIRECT METHODS

3.3.1 The Classification Used for the Straight Creek (Eisenhower) Tunnels

Methodology and Application Guidelines

The ground classification system used during the construction of the Eisenhower Tunnels distinguishes between four ground classes (Table 3.3.1). For each class ground conditions are described verbally and a construction-support procedure is assigned (see Fig. 3.3.2). In addition, the rock load is given for each class (Table 3.3.1). This rock load which is based on the Terzaghi classification was used in the design of the corresponding support systems.

Development and Underlying Philosophy

The procedure is based on Terzaghi's rock load approach (see Section 3.2.3). For each ground class a typical support system has been designed. Thus, during construction a support system is assigned directly, omitting the intermediate step of assigning a rock load.

To some extent particularly in the second bore an approach similar to the NATM (Section 3.3.2) although without incorporation of performance monitoring was used. Ground classes and thus support systems and excavation procedures were assigned by a team of contractor and owner representatives after careful observation of ground conditions.

TABLE 3.3.1 GROUNDCLASSIFICATION FOR EISENHOWER II TUNNEL (FROM KORBIN AND BREKKE, 1977)

Rock Class Descriptions and Predicted Rock Load

Rock Class	Description	Rock Load H_p in (meters)
Ia Ib	Massive to slightly blocky, having a joint spacing of greater than 1.0 feet with no alteration.	0 to .25B 0 - 12 (0 - 3.7)
IIa IIb	Moderately blocky and seamy having a joint spacing greater than 0.5 feet with little or no alteration.	.13 B to .35 (B + H_t) 6 - 25 (1.8 - 7.6)
IIIa IIIb	Very blocky and seamy, having a joint spacing less than 1.0 feet, moderately to highly altered with zones of moderate to intense shearing.	(.18 to 1.10) (B + H_t) 12 - 76 (3.7 - 23)
IVa	Squeezing (low to moderate) ground, highly crushed and altered, non-plastic abundant clay, joint spacing less than 0.5 ft.	(1.10 to 2.10) (B + H_t) 76 - 145 (23 - 44)
IVb	Squeezing (moderate to high) and swelling, plastic highly altered, mainly clay gouge.	(2.10 to 4.50) (B + H_t) 145 - 311 (44 - 95)

B, Width of Top Heading = 2 H_t, Height = 46 feet (14m).

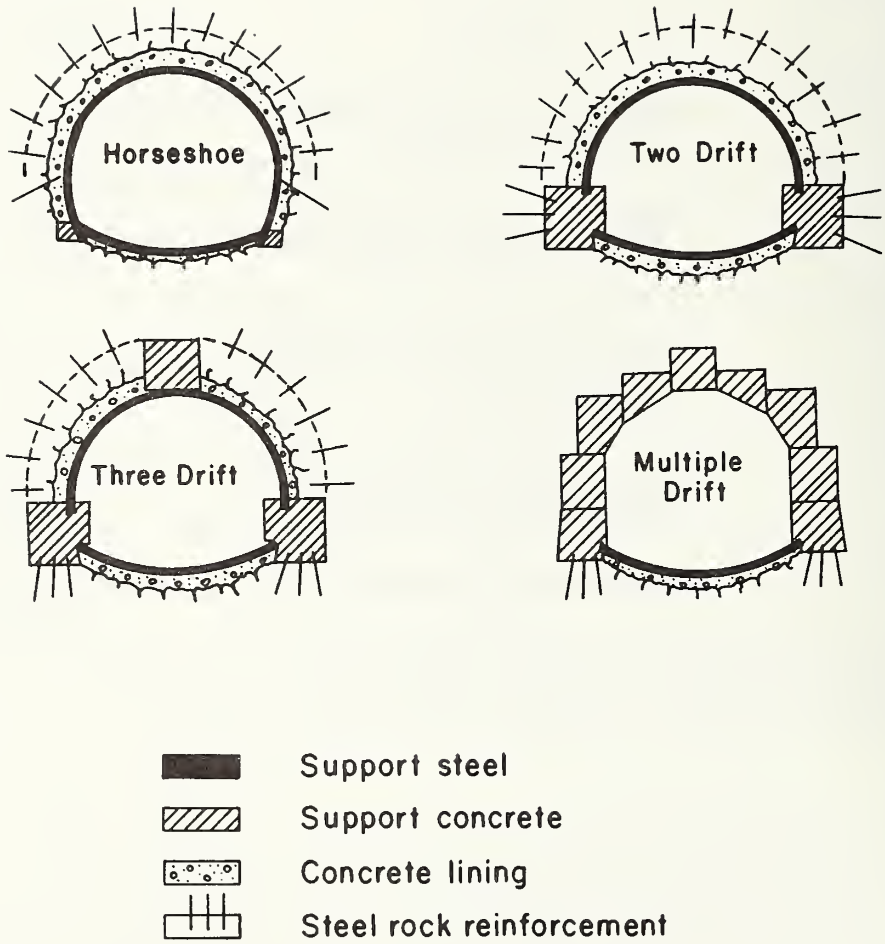


FIGURE 3.3.2 CONSTRUCTION-SUPPORT PROCEDURE FOR EISENHOWER II TUNNEL (FROM BROCHURE COLORADO DEPT. OF TRANSPORTATION,1976)

Critique

The ground classification for the Straight Creek Tunnel correlates ground conditions and support requirements directly. The support was designed based on Terzaghi's rock load tables. Also the experience from the first tunnel has been incorporated in the classification for the second tunnel.

This development makes it an ideal empirical method: oriented to a specific site and modified to the actual conditions. The problematic aspects lie thus not in the principles but in the design details. The support (partially due to the multiple drift excavation procedure) is substantially overdesigned as discussed by Einstein (1977). Although understandable in view of the "bad experience" in the first bore (see Hopper et. al., 1972) it reduces the value of an otherwise sound empirical approach.

Summary of Classification for Straight Creek Tunnel

The procedure relates verbally described ground conditions to four ground classes to which an excavation support procedure is associated.

The procedure is an ideal empirical method oriented to a specific site and modified to actual conditions. The disadvantage lies in the design details which led to substantial overdesign. The actual load recommendation should not be used in other cases.

3.3.2 NATM Ground Classification

Methodology and Application Guidelines

The ground classification procedure used with the New Austrian Tunneling Method (NATM) relates ground conditions, excavation procedure and support requirements. Ground behavior due to the tunneling process is described, and an excavation procedure and support requirements are assigned to typical behavioral classes. The classification procedure is adaptable. It is first adapted to each new project based on previous experience and a detailed geologic-geotechnical investigation. The classification is also adaptable during a single project based on performance monitoring. In other words, the NATM is an observational procedure.

The classification forms part of the contract and the detailed contract provisions influence the classification procedure.

A detailed description of the contractual arrangement can be found in Vol. 4 of this report series (Steiner, Einstein, Azzouz, 1978). Contractual details will thus only be mentioned here as far as they influence the classification procedure.

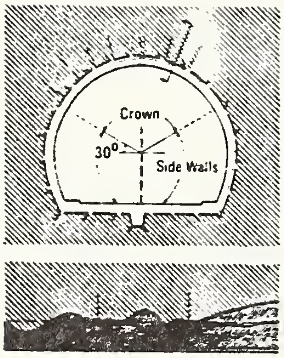
The ground classification used for the "Dalaas-tunnel" in the state of Vorarlberg (developed by M. John of ILF, Lässer-Feizlmayr, Consulting Engineer, Innsbruck, Austria) is shown in Table 3.3.2. This represents the most recent NATM classification. In Appendix A, earlier versions of classification systems have been compiled. It should be kept in mind, however, that a significant aspect of the NATM is its adaptability. A particular classification is thus only applicable to the one case for which it was developed and modified. The classification consists of a detailed description of conditions and the expected behavior of the ground as the opening is excavated. ('Geomechanical Indicator' in Table 3.3.2). The excavation procedure is described in detail, including round length, allowable cross-section and the method of excavation (e.g., type of blasting, excavation with hydraulic shovel). The support construction procedure is also prescribed, e.g., the sequence of placement of the individual support components. Support requirements are specified for the crown, the springlines, the invert and the face. In this newest version (Table 3.3.2) ranges of support quantities are quoted. Note that support ranges of adjacent classes overlap; this is possible with the contract provisions used for the Dalaastunnel: The classes reflect primarily 'excavation' classes, the actual support placed is determined based on performance monitoring. (The adaptation of the support to the actually monitored performance follows the principles of an observational method.)

TABLE 3.3.2 GROUND CLASSIFICATION FOR THE DALAAS-TUNNEL (AFTER JOHN, 1978)

CLASS	GROUND BEHAVIOR	GEOMECHANICAL INDICATORS	EXCAVATION				CONSTRUCTION PROCEDURE	SUPPORT PROCEDURE				
			SECTION	ROUND LENGTH	METHOD	STAND-UP TIME (Guidelines)		PRINCIPLE	CROWN	SPRINGLINES	INVERT	FACE
I	intact rock (free standing, = standfest)	The stresses around the opening are less than the rockmass strength; thus the ground is standing. Due to blasting seperations along discontinuities are possible. For high overburden danger of popping rock.	full face possible	no limit	smooth blasting	crown: weeks springlines: unlimited	Check Crown for loose rock. When Popping rock is present placement of support after each round	support against dropping rock blocks	Shotcrete: 0-5cm Bolts: Cap = 15t Length = 2 to 4m locally as needed	Bolts: Cap = 15t Length = 2 to 4m locally	NO	NO
II	lightly afterbreaking (nachbrüchig)	Tensile stresses in the crown or unfavorably oriented discontinuities together with blasting effects lead to seperations.	full face possible	3 to 5m	smooth blasting	crown: days springlines: weeks	The crown has to be supported after each round.	shortcrete support in crown bolted arch in crown	Shotcrete: 5-10cm with wirefabric $(3.12 kg/m^2)$ Bolts: Cap = 15t Length = 2 to 4m one per $4\text{-}6m^2$	Shotcrete: 0-5cm Bolts: Length = 2 to 4m locally	Bolts L=3.5m if necessary	
III (formerly IIIa)	afterbreaking to over-breaking	Tensile stresses in the crown lead to rooffalls that are favored by unfavorably oriented discontinuities. The stresses at the springlines do not exceed the mass strength. However, afterbreaking may occur along discontinuities (due to blasting).	full face with short round lengths	full face: 2 to 4m	smooth blasting	crown and springlines: several hours	Shotcrete after each round other support can be placed in stages.	combined shotcrete - bolted arch in crown and at springlines	Shotcrete: 5 to 15cm with wirefabric $(3.12 kg/m^2)$ Bolts: Cap = 15-25t Length = 3 to 5m, one per 3 to $5m^2$	Shotcrete: 5-10cm Bolts: 15 to 25t Length: 3 to 5m 1 per 3 to $5m^2$	Adapt Invert Support to local conditions.	Adapt Face Support to Local Conditions
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 $45m^2$)	full face: 2 to 3m heading: 2 to 4m	smooth blasting and local trimming with jack-hammer	crown and springlines: a few hours	Shotcrete after each round. The bolts in the heading have to be placed at least after each second round.	combined shotcrete - bolted arch in crown and springlines, if necessary closed invert	Shotcrete: 10 to 15cm with wirefabric $(3.12 kg/m^2)$ Bolts: fully grouted Cap = 25 tons Length = 4-6m one per $2\text{-}4m^2$	same as crown	Slab: 20 to 30cm	
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. $40m^2$)	heading: 1 to 3m benches 2 to 4m	smooth blasting or scraping or hydraulic excavator	crown and springline: very short free stand-up time	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: 15-20cm with wirefabric $(3.12 kg/m^2)$ Steelsets: TH21 Spaced: 0.8-2.0m Bolts: fully grouted Cap: 25t Length: 5 to 7m one per 1 to $3m^2$	same as crown but no linerplates necessary.	Invert Arch >40cm or Bolts L = 5 to 7m if necessary	Shotcrete 10cm in heading (if necessary) 3 to 7cm in bench
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 $25m^2$)	heading 0.5 to 1.5m benching 1 to 3m	scraping or hydraulic excavator	very limited stand-up time	as Class V	support ring of shotcrete with steelsets Incl. Invert Arch and densely bolted arch.	linerplates where necessary, Shotcrete: 20-25cm with wirefabric. steelsets TH21 0.5-1.5m Bolts: Cap:25t L=6-9m, 1 per $0.5\text{-}2.5m^2$	same as crown	Invert: >50cm Bolts: 6-9m long if necessary	Shotcrete 10cm and additional face breasting
VII	flowing		Requires special techniques e.g., chemical grouting, freezing, electro-osmosis									

Development and Underlying Philosophy

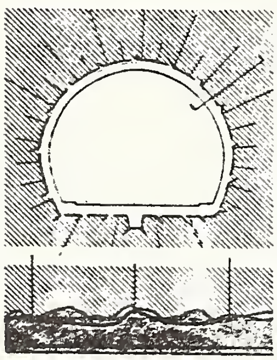
The development of the classification system used in Austrian Tunnel practice can be traced back at least to Lauffer (1958) and the construction of the Prutz-Imst hydro tunnel. Lauffer's classification system is itself based partly on Stini's (1950) rock load recommendations. Lauffer (1960) published a table of alternate support systems based on the experience at Prutz-Imst (for details see the description of Lauffer's method (Section 3.5.1)). Ground class alternates of timber, steelsets, rockbolt and shotcrete support for a 5.5m diameter tunnel were tabulated (see Appendix B). The classification system proposed by Lauffer was applied by Rabcewicz et. al. during the construction of the Schwaikheim Tunnel in Germany (Rabcewicz, 1965, 1969; Einstein et. al. 1977). A next step was the application of the procedure in the Tauern and Katschberg Tunnels. Five classes were considered (see Fig. 3.3.3). But these five classes did not include all ground conditions that were actually encountered in the Tauern tunnel and the contract had to be renegotiated which led to the creation of a new class SK. (SK = Sonderklasse = Special Class) with increased bolt support (Bolts of higher capacity and length of 6 to 9m). This change in classification was necessary due to the nature of the payment provisions for the Tauern tunnel. The placement of the support had to be included in the excavation unit price, in addition a bonus-malus clause was in effect for support quantities. (Support



QUALITY CLASS I

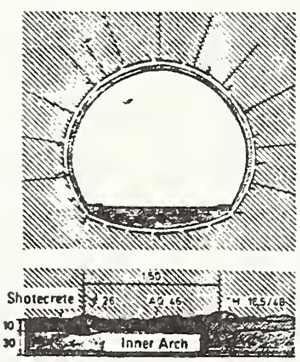
Exp. Anchor 26 mm dia., l = 1.5 m	12 m/m run
Steel Fabric Mat 3.12 kg/m^2	36 kg/m run

K = Crown U = Sidewall



QUALITY CLASS II

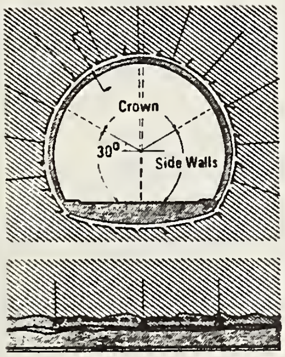
K	Exp. Anchor	26 mm dia., l = 3.5 m	9.8 m/m run
K+U	Exp. Anchor	26 mm dia., l = 1.5 m	9.0 m/m run
K+U	Steel Fabric Mat 3.12 kg/m^2 Crown + 50% of Sidewalls		56.2 kg/m run
K	Sealing of Notch Points with Shotcrete		12 m^2/m run



QUALITY CLASS III

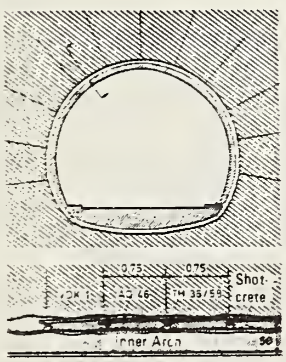
K+U	Perfo or SN Anchor	26 mm dia., l = 3.20 m	7.35 per m run = 23.5 m/m run
K+U	Steel Fabric Mat	3.12 kg/m^2	79.5 kg/m run
K	Tunnel Arch	TH 16.5/48	126 kg/m run
K+U	Shotcrete	d = 10 cm	25.5 m^2/m run

FIGURE 3.3.3 GROUNDCLASSES FOR THE TAUERN-TUNNEL (FROM POCHHACKER,1976)



QUALITY CLASS IV

Perfo or SN Anchor	26 mm dia., l = 4 m	11 per m run = 44 m/m run
Steel Fabric Mat	3.12 kg/m^2	79.5 kg/m run
Tunnel Arch	TH 25/58	462 kg/m run
Gallery Floorboards	34 kg/m^2	78 kg/m run
Shotcrete	d = 15 cm	25.5 m^2/m run
Face Breasting + Shotcrete	d = 3 cm	30 m^2/m run



QUALITY CLASS V

Perfo or SN Anchor	26 mm dia., l = 4 m	14.6 per m run = 58.4 m/m run
Steel Fabric Mat	3.12 kg/m^2	79.5 kg/m run
Tunnel Arch	TH 36/58	1223 kg/m run
Gallery Floorboards	34 kb/m^2	655 kg/m run
Shotcrete	d = 25 cm	26 m^2/m run
Face Breasting + Shotcrete	d = 3 cm	70 m^2/m run

FIGURE 3.3.3 GROUNDCLASSES FOR THE TAUERN-TUNNEL (FROM POCHHACKER,1976) (CONT.)

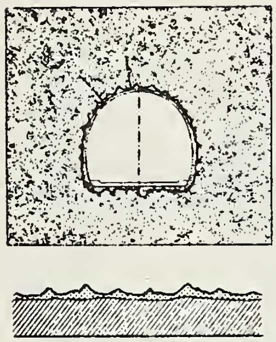
exceeding the standard quantities was paid at 75% only, whereas support which was not necessary was still paid at 25%; (for more details see Steiner, Einstein, Azzouz, 1979 = volume 4 of this series).

For the Arlberg tunnel the support quantities shown in Fig. 3.3.4 were initially included in the contract documents.

However, the actual performance was different from the anticipated one. The support quantities considerably exceeded the originally planned ones, in addition large convergences had to be tolerated in the tunnel. The ranges of support quantities for the Arlberg as they were actually used during construction are shown in Fig. 3.3.5. Some more details are presented in Appendix A, where classes IVa and Va correspond to the initially designed classes for the Arlberg tunnel, whereas classes IVb, IVc, and Va are actually placed supports in the western section of the Arlberg tunnel.

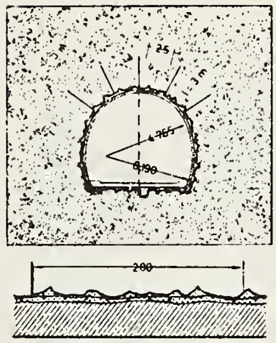
A further development of the NATM occurred in developing classification system for the Pfänder Tunnel (John, 1978) which is presented in Fig. 3.3.6 and 3.3.8. The Pfänder Tunnel was designed for drill and blast (Fig. 3.3.6) as well as excavation with a TBM (Fig. 3.3.7). A new class U has been created for the Pfänder Tunnel, this class is used near the portal, its main feature are longer bolts in the upper 45° degree sectors in order to avoid roof collapses, in addition the shotcrete thickness has been increased in this class.

CLASS I



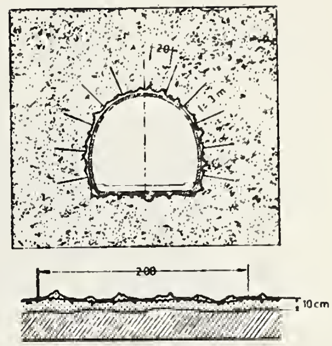
Shotcrete:0-5 cm
Bolts:locally
C=25 t, L=3m
OR
Bolts: 1 per 3m^2
L= 1m (locally 3m)
and wirefabric

CLASS II



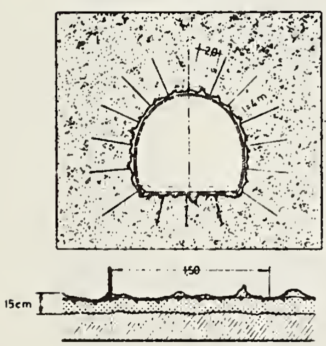
Shotcrete:5-10cm
Wirefabric
Bolts:1 per 4-6m^2
C=15 t, L=3m
fully grouted

CLASS III A



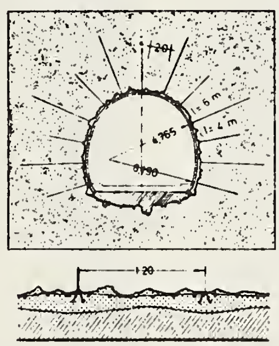
Shotcrete:10cm
Wirefabric
Bolts:1 per 4m^2
C=25 t, L=3m
fully grouted

CLASS IIIb



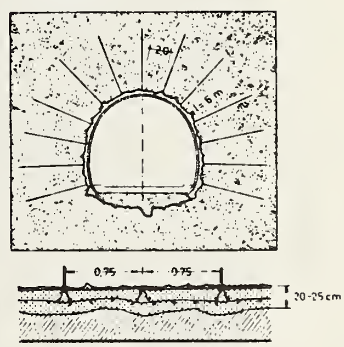
Shotcrete:15cm
Wirefabric
Bolts: 1 per 3m^2
C=25 t, L=4m

CLASS IV



Shotcrete:15-20cm
Wirefabric
Bolts:1 per 2-3m^2
C=25t,L=4 and 6m
Steelsets:14lb/ft
Spac: 1 to 1.5m
locally steel
lagging

CLASS V



Shotcrete:20-25cm
Wirefabric
Bolts:1 per 1.5 m^2
C=25t, L=6m
Steelsets: 19 lb/ft
Spac: 0.5 to 1m
possibly lagging

C=Capacity of Bolts; L=Length of Bolts
Wirefabric:weight per m^2 =3.12kg

FIGURE 3.3.4 GROUNDCLASSES FOR ARLBERG TUNNEL AS DESIGNED (INCLUDED IN BID DOCUMENTS),(AFTER JOHN,1976)

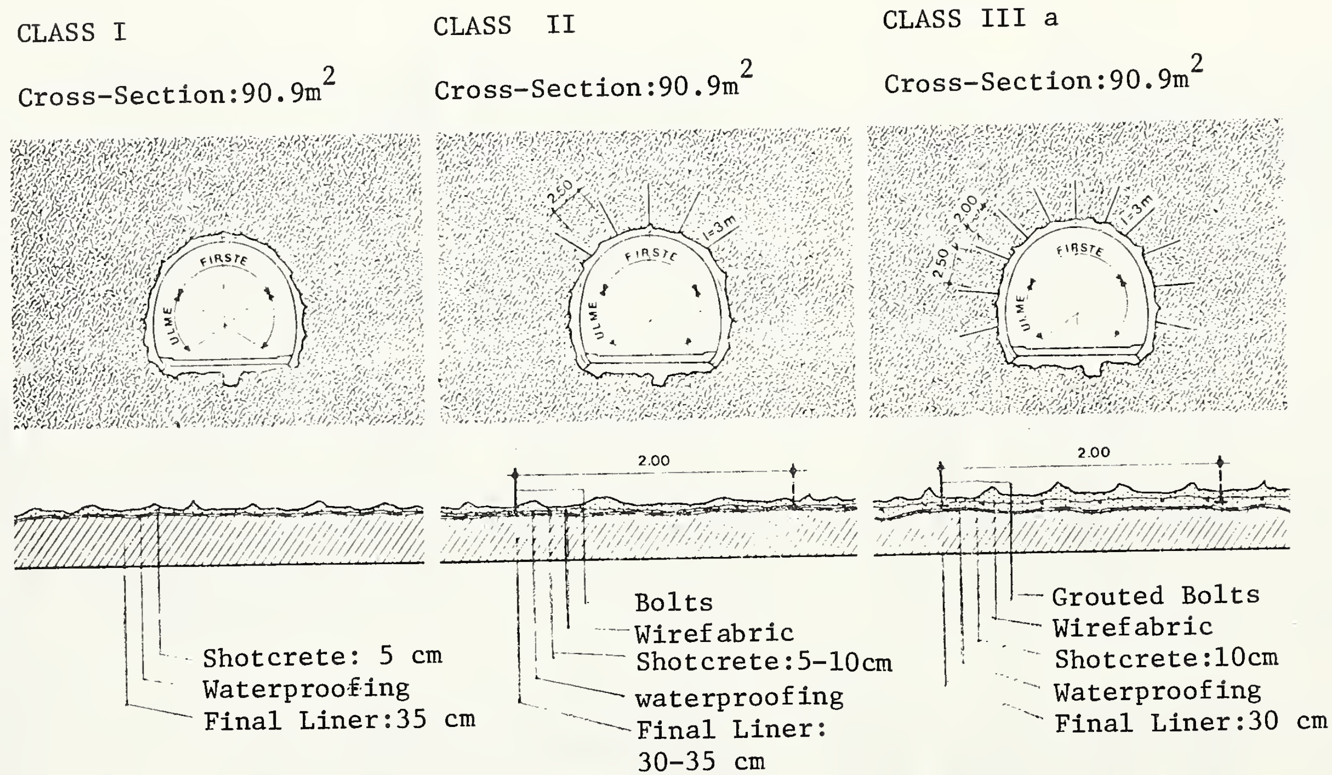


FIGURE 3.3.5 GROUNDCLASSES FOR ARLBERG TUNNEL, AS-BUILT
(AFTER LAESSER-FEIZLMAYR,1978)

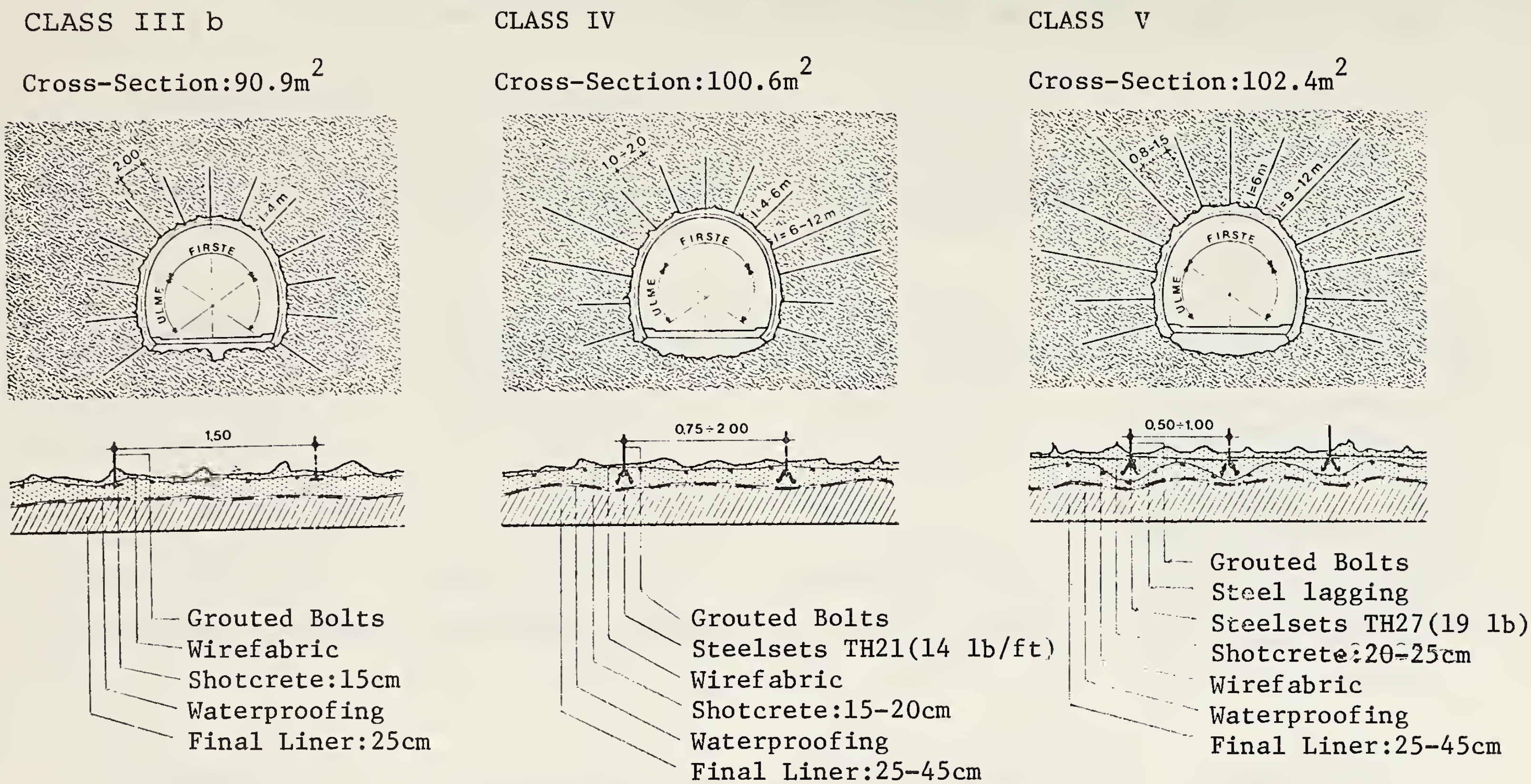
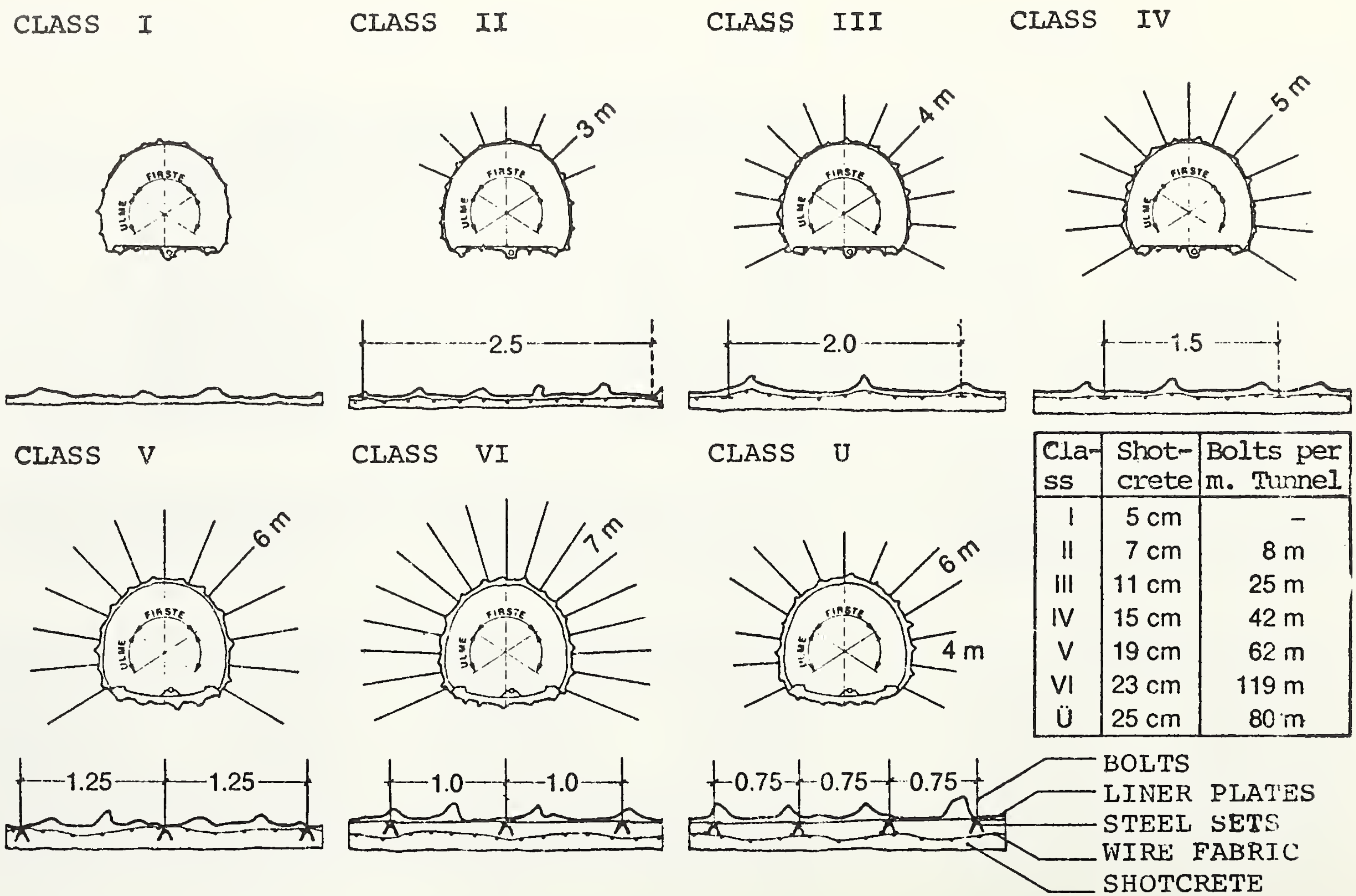


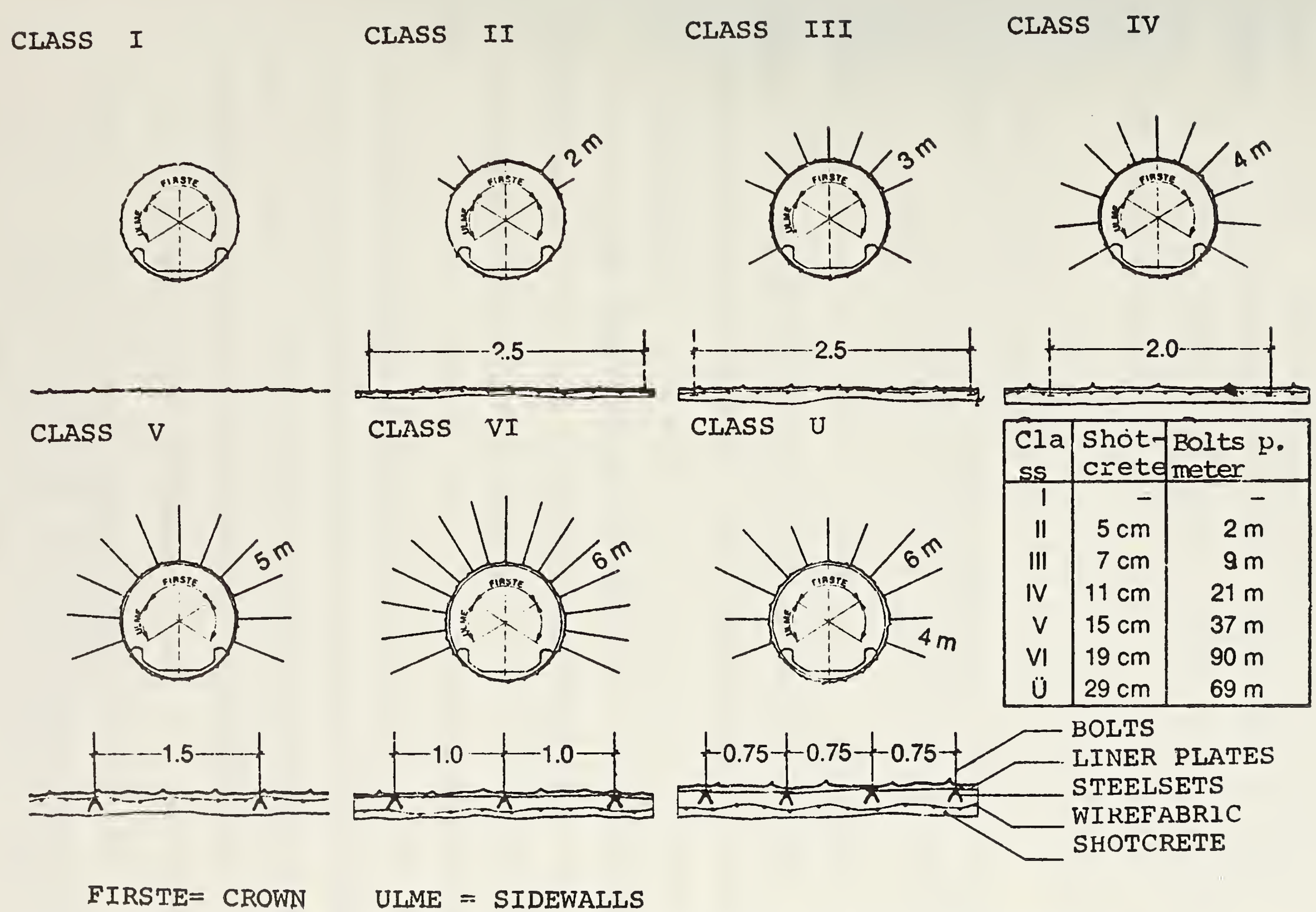
FIGURE 3.3.5 GROUNDCLASSES FOR ARLBERG TUNNEL, AS-BUILT (AFTER LAESSER AND FEIZLMAYR,1978) (CONT.)



Class	Shotcrete	Bolts per m. Tunnel
I	5 cm	–
II	7 cm	8 m
III	11 cm	25 m
IV	15 cm	42 m
V	19 cm	62 m
VI	23 cm	119 m
Ü	25 cm	80 m

FIRSTE = CROWN   ULME = SIDEWALLS

FIGURE 3.3.6 GROUNDCLASSES FOR CONVENTIONAL DRILL AND BLAST EXCAVATION FOR THE PFAENDER-TUNNEL (FROM JOHN, 1978)



Class	Shot-crete	Bolts p. meter
I	–	–
II	5 cm	2 m
III	7 cm	9 m
IV	11 cm	21 m
V	15 cm	37 m
VI	19 cm	90 m
Ü	29 cm	69 m

FIGURE 3.3.7 GROUNDCLASSES FOR MACHINE-EXCAVATION FOR THE PFAENDER TUNNEL(FROM JOHN,1978)

The classification for the TBM-excavation included also prescriptions on the placement of the support which had to be placed at some distance from the face regardless of the length of the machine. The Pfänder Tunnel is, however, built by drill and blast excavation and thus the classification for TBM-excavation cannot be verified.

The NATM classification procedure was further developed resulting in the most recent version the one used for the Dalaas Tunnel (Table 3.3.2) described before.

Critique

As mentioned earlier under methodology: The ground classification procedure used with the New Austrian Tunneling Method relates ground conditions, excavation procedure and support requirements. The ground is described behaviorally which allows to classify the ground in the field. The method is adaptable and it is usually adapted in the field based on monitored performance.

The classification procedure works only properly if the associated contractual issues are understood. Ground classification and construction contract are a single entity.

The procedure has, however, some technical limitations which make its use by outsiders difficult:

• The ground classification procedure as it stands right now was developed primarily from field observations. Experience that accumulated during the construction of one tunnel has been included in the classification procedure for another tunnel. Most of these tunnels have basically a width of approximately 10 to 12m. The method, however, has also been applied to larger tunnels and caverns, like the Tarbela Diversion tunnels (Golser, 1973), the Waldeck II cavern (Beton und Monierbau, 1976), as well as to smaller diameter hydropower tunnels.

There are not many published rules or methods that would allow the extrapolation to larger or smaller tunnels with the exception of Rabcewicz's shear body theory. Few results of observation in pilot tunnels and large chambers have been published which would allow a comparison and extrapolation.

The combinations of shotcrete, wirefabric, steelsets and rock bolts have been found empirically. The use of one of these support systems in a particular tunnel may be dictated to some extent by economics, the contract conditions and payment provisions of a contract. At present no rules exist as to how and if one or the other of the supports could be substituted by another technically equivalent support system. This limitation actually also holds for other classification methods.

Summary of the NATM Classification

Ground conditions in the tunnel are described behaviorally and related to an excavation and support procedure. Ranges of support quantities are assigned to these ground classes. The NATM classification is flexible and the excavation and support procedure is adapted based on performance monitoring. The method has been applied to a wide variety of ground conditions, primarily in the eastern Alps, but also elsewhere.

A direct implementation seems difficult since it requires previous field experience, for a prediction of ground conditions. However, its power lies in its adaptability during actual construction.

3.4 QUANTITATIVE INDIRECT (ROCK LOAD) RELATIONS - TYPE C

Ground conditions are described quantitatively (Fig. 3.4.1) and a rock load or rock load factor is related to this quantitative ground description. The support is designed by using the rock load for a structural design.

Empirical relations of this type are the RQD - Rock Load factor developed by Deere et. al. (1969) and Monsees (1970). Also in this category fall the wedge type analysis developed by Deere and his group (Deere et. al. 1971; Cording and Deere, 1972). The two analyses are discussed in parallel in the next section since both include some common features. (Other empirical relations developed by Deere and his colleagues at the University of Illinois are discussed in Section 3.5.2.)

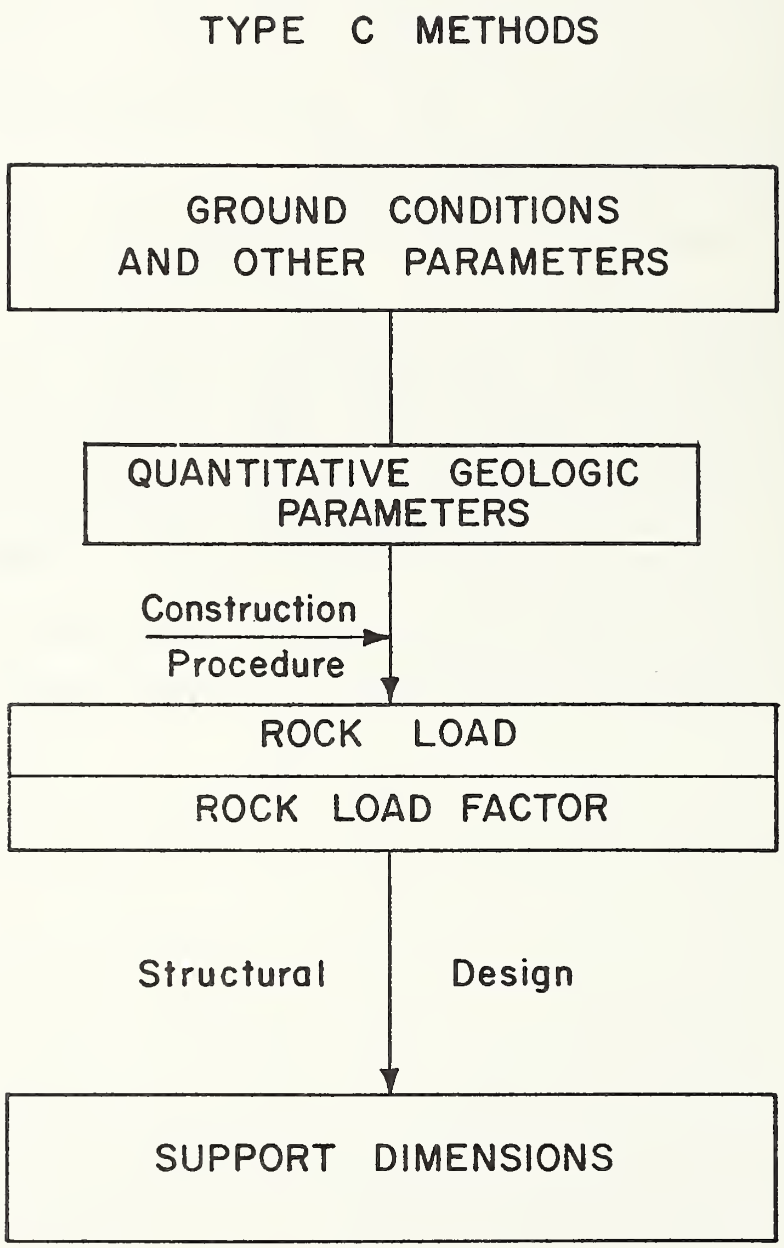


FIGURE 3.4.1 QUANTITATIVE INDIRECT (ROCKLOAD) METHODS

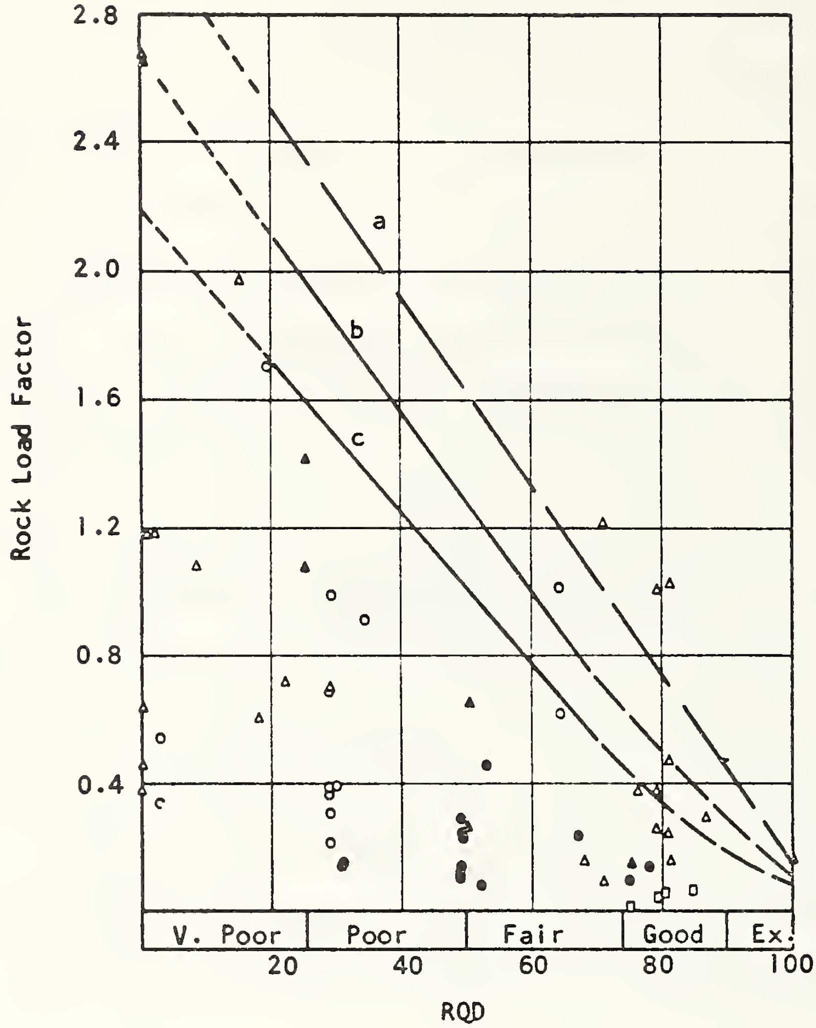
3.4.1 Relation Between RQD and Rock Load[1)]

Methodology and Application Guidelines

Parallel to the direct correlations between RQD and support Deere and his group developed methods that relate RQD to rock load or consider wedges dropping or sliding into the cavern. Based on the predicted rock load a support system can be designed. Fig. 3.4.2 shows the rock load factor recommendations based on RQD that were published in 1969. Curve 'a' is the relation between RQD and the Rock Load Factor for the average of Terzaghi's recommendation (a detailed explanation on how this curve was derived will be given under "Development and Underlying Philosophy"). Curve 'b' is the recommended load curve for a drill and blast tunnel based on field measurements. Curve 'c', which is 25% below curve 'b', is the recommended rock load curve for machine bored tunnels. These three curves should be used for steel set supported tunnels.

For larger chambers or tunnels in ground with widely spaced shear zones or other predominant discontinuities a design based on RQD may not be conservative. Thus a wedge type approach has been developed by Cording and Deere. (1972). which was further refined by Cording and Mahar (1974).

[1)] This procedure relates RQD to Rock Load, the more widely known direct RQD-Support relation is discussed in Section 3.5.2.



a - Average for Terzaghi's rock load factor
b - Recommended for steel sets, conventional tunneling
c - Recommended for steel sets, machine tunneling

FIGURE 3.4.2 RQD ROCK-LOAD FACTOR RELATION
(FROM DEERE ET AL.,1969)

This approach (Fig. 3.4.3) considers the actual shape of wedges and the shear strength along the discontinuities. (The table reflects primarily the experience from the Washington D.C. subway system.) The support design would use the weight of the wedges as input parameter.

Development and Underlying Philosophy

The methods proposed by Deere and his co-workers intend to rationalize the design of tunnel supports. There are two distinct types of methods that were developed to predict the support loads, one based on RQD, the other one based on a wedge analysis. In the following lines development of the RQD - Rock Load correlation will be described, followed by a description of the wedge methods.

The RQD - Rock Load correlation has been described in great detail by Monsees (1970). The 'estimated' correlation between RQD and Terzaghi's rock condition (from Monsees, 1970) is shown in Table 3.4.1. For each ground class given by Terzaghi a range of RQD has been assigned; also the RQD range is correlated to the range of "Terzaghi's rock load factors". Recall that Terzaghi quotes his rock load either as a function of the span B or the sum of span (B) and height (H_T) of the tunnel. Monsees has assumed that H_T=B, thus the rock load can be expressed solely as a function of the width B. The recommendations of Table 3.4.1 are plotted in Fig. 3.4.4.

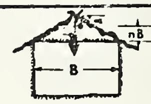
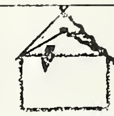
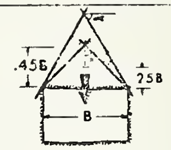
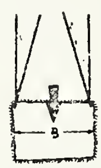
(α) DIP ANGLE	(θ) HALF ANGLE	(nB) HEIGHT of EQUIVALENT ROCK LOAD	MINIMUM CONDITION FOR FAILURE	
0° - 30°	90° - 60°	(0 - .15)B	Both planes wavy, offset	
30° - 45°	60° - 45°	(.15 - .25)B	One plane wavy or offset; One plane smooth to slightly wavy	
45° - 60°	45° - 30°	(.25 - .45)B	One plane sheared, continuous and planar; One plane slightly wavy	
60° - 75°	30° - 15°	(.45 - 1.0)B	Both planes sheared, continuous and planar	
75° - 90°	15° - 0°	> 1.0B	Low lateral stresses in arch; Surfaces planar, smooth, possibly open, or progressive failure aided by separation along low angle joints	

FIGURE 3.4.3 ROCK LOADS DUE TO DROPPING WEDGES IN TUNNEL CROWN (FROM CORDING AND MAHAR,1974)

TABLE 3.4.1 ESTIMATED CORRELATION BETWEEN RQD AND TERZAGHI'S ROCK CONDITION DESCRIPTION (FROM MONSEES, 1970)

ROCK CONDITION	RQD	RLF
Hard and intact	95-100	0
Hard, stratified, or schistose	90-100	0-0.50
Massive, moderately jointed	85-95	0.25-0.50
Moderately blocky and seamy	75-85	0.25-0.70
Very blocky and seamy	40-75	0.70-2.20
Completely crushed but chemically intact	0-25	2.20-3.00

Each of Terzaghi's classes is represented as a box on this plot. Curve 'b' on Fig. 3.4.4 links the centroids of each 'box' of Terzaghi's rock load recommendations and thus gives a recommended curve relating RQD and Terzaghi's rock load. Curve 'a' is the proposed envelope for the design of steel supports based on field measurements collected by Monsees. It represents a 90% envelope, i.e., 90% of the measurement points fall below this line.

An additional curve 'c' is shown in Fig. 3.4.2, for machine driven tunnels, this curve is 25% below the curve for conventionally driven tunnels.

For large rock chambers inconsistent trends were noted. It was found that the 'equivalent support pressure' i.e., the capacity of the bolts at yield expressed as a rock load factor was lower than rock loads in steelset supported tunnels (Fig. 3.4.5). Typical bolt support gave a rock load factor in the order of 0.1 to 0.25. This result possibly suggests a different support mechanism for rock bolts (dowelling effect).

Rock Loads Due to Unstable Wedges

An initial version of this approach was published by Cording et al. (1971) and later by Cording and Deere (1972). In this approach a wedge corresponding to the maximum tunnel

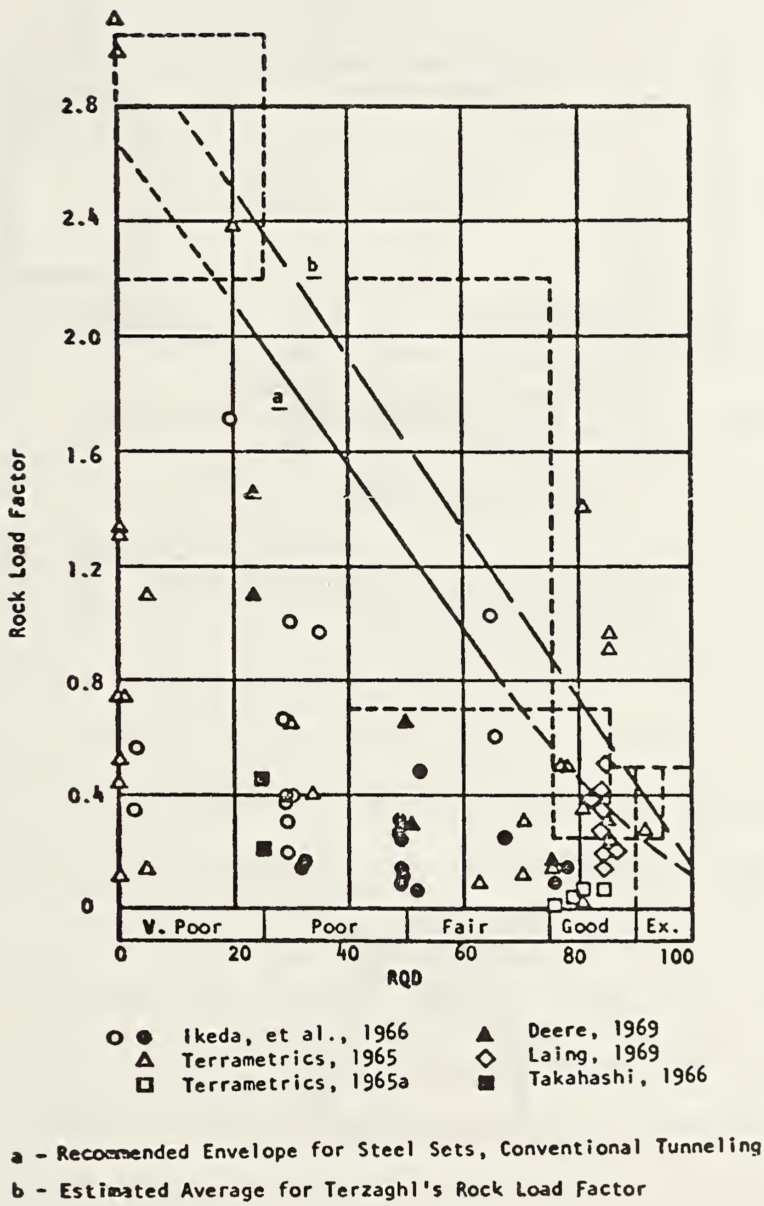


FIGURE 3.4.4 RELATION OF RQD TO TERZAGHI'S ROCKLOAD RECOMMENDATIONS AND FIELD MEASUREMENTS (FROM MONSEES,1970)

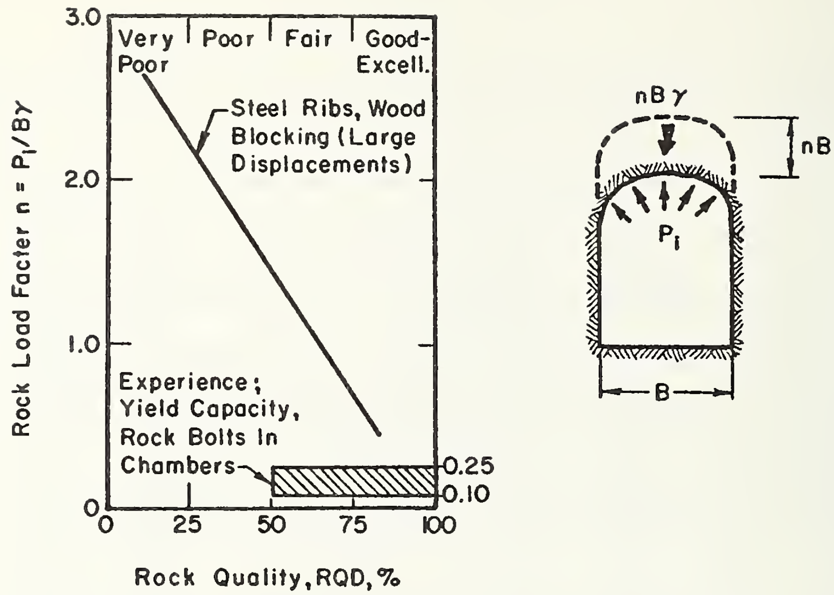


FIGURE 3.4.5 COMPARISON OF SUPPORT LOADS IN STEELSUPPORTED TUNNELS AND ROCK BOLTED CAVERNS (FROM CORDING AND DEERE,1972)

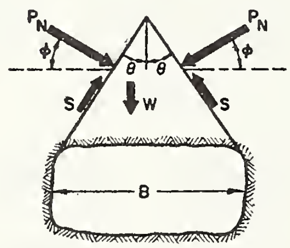


FIGURE 3.4.6 WEDGE STABILITY IN CROWN (FROM CORDING AND DEERE,1972)

width is considered (Fig. 3.4.6). The support pressure, p_i, required to keep the symmetrical wedge in place is given by

$$p_i = p_n \left(1 - \frac{\tan\phi}{\tan\theta}\right) + \frac{\gamma B}{4\tan\theta} \qquad (3.4.1)$$

p_n = stresses acting on sides of symmetrical wedge in place (see Fig. 3.4.6)

ϕ = friction angle on the sides of wedge

θ = half angle of wedge

B = width of tunnel

The required support pressure is a function of stresses on the wedge and the weight of the wedge. To analyze the stability of the wedge the normal stress on the wedge should be predicted along with ϕ, c and B. For opening half angles θ that are larger than the friction angle the wedge tends to drop out and the support has to carry the entire wedge. Note that once the wedge tends to drop the normal stress on the wedge p_n changes and ultimately reduces to zero thus, the required support capacity equals the weight of the wedge (provided no additional loosening takes place and leads to an increase in support loads i.e., more wedges falling out).

Cording and Deere make a further assumption: the wedge that can drop out is the largest one, i.e., $\theta = \phi$. With this assumption the required support pressure for a dropping wedge is:

$$p_i = \frac{\gamma B}{4\tan\phi} \tag{3.4.2}$$

thus the rock load factor is

$$\frac{p_i}{\gamma B} = \frac{1}{4\tan\phi} \tag{3.4.3}$$

This relation which considers a maximum wedge that drops out due gravity is shown in Fig. 3.4.7 together with experience in rock bolted chambers; the actually placed support in large chambers is less than predicted with equation 3.4.3. Equation 3.4.3 yields the maximum rock load for wedge type failure; if the actual discontinuity attitude and resistance were known one could use the less conservative equation 3.4.1. Cording and Deere (1972) discuss the selection of the appropriate rock load in detail and also the selection of rock bolt support (pattern) based on potential unstable wedges. (Fig. 3.4.8) in a tunnel of the Washington D.C. metro.

Critique of RQD - RLF Relations

Recall that the relation between RQD and rock load has been established as follows:

1) Terzaghi's rock load has been related by subjectively assigning ranges of RQD and Terzaghi's rock loads.

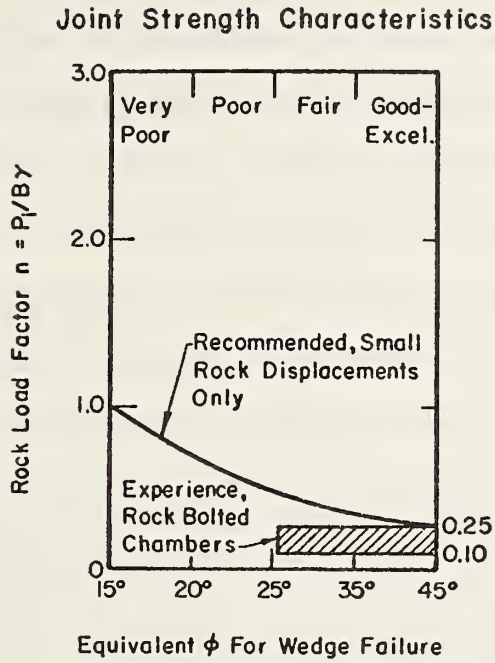


FIGURE 3.4.7 RELATION OF ROCKLOAD FACTOR TO FRICTION ANGLE ALONG DISCONTINUITIES (FROM CORDING AND DEERE,1972)

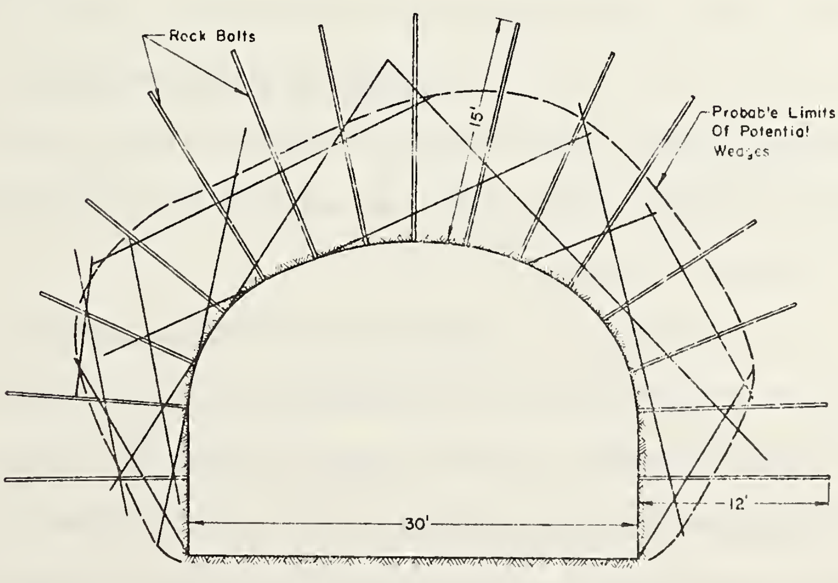


FIGURE 3.4.8 SELECTION OF BOLT PATTERN TO STABILIZE POTENTIALLY UNSTABLE WEDGES(FROM CORDING AND DEERE,1972)

2) Additional information in form of measured rock loads in conventionally built tunnels was used to modify the Terzaghi based results, i.e., to give more realistic results.

3) Finally for machine drilled tunnels a curve approximately 25% below the one for drill and blast tunnels has been chosen.

The relation between Terzaghi's rock load factor and RQD is shown in Table 3.5.1. Note that the rock loads given by Terzaghi cover continuous ranges, i.e., each range joins to the next one. In the assessment by Monsees, 1970, however, the ranges of RQD between very blocky and seamy rock (RQD = 40 to 75%) and completely crushed but chemically intact rock (RQD = 0 to 25%) are not joining. The rock load has not been defined for RQD = 25 to 40% in table 3.4.1, although the "average" curve, links the centers of the ranges (boxes) of RQD vs. Rock Load Factor.

A second point applied to the assessment of 'hard, stratified or schistose' rock for which a range of RQD = 90 to 100% has been assigned. Cecil (1970, 1975) has studied support requirements in Scandinavian tunnels, there he noted 'anomalous' cases, i.e., cases with low RQD (sometimes as low as RQD = 0) that stood unsupported. (Cecil's cases are listed in Section 3.5.2 in Table 3.5.7). These cases would be assessed as hard, stratified or schistose rock with Terzaghi's classification. By including Cecil's data, Deere's assessment of the rock load factor could be modified for 'stratified or schistose' rock to be RLF = 0.0 to 0.5 and RQD = 0 to 100%, tight and single joint set rather than RLF = 0 to 0.5 and RDQ = 90 to 100% (see Table 3.4.1).

Other anomalies noted by Cecil (1970) are cases with thin clay coatings and joint fillings in widely spaced joints, when instability was experienced for high RQD values (RQD > 75%). In these cases the filled discontinuity was the factor determinating stability. Those cases may be treated with the 'wedge' approach i.e., considering wedges that may become unstable.

Critique of Wedge Approach

An approach that considers the actual wedge geometry and the strength properties along the discontinuities, requires that

- the actual joint geometry
- the strength properties of the rock
- the stresses acting on the contact areas

are known in their approach. Cording and Deere neglect the in-situ stresses and only considered the weight of the wedge which is conservative.

The wedge approach has several advantages:

- it uses data that is available in the tunnel from geologic mapping,
- the computations are simple,
- in case of bolt support the pattern and the length of the bolts can easily be selected from a cross-sectional drawing.

Summary of Deere's Rock Load Methods

In the RQD-RLF method, RQD is related to a Rock Load Factor. Deere, et. al. explicitly state their assumption that RQD has to adequately represent ground conditions together with other limitations of their method.

In the wedge method the actual geometry of the opening and the structure of the geology (however, usually assuming conservative geologic conditions) is considered which allows to design a support system.

The advantages of the RQD method is the possibility to make a preliminary estimate based solely on easily obtainable information from a borehole. The method may, however, be overconservative in case only one joint set is present. The effect of the construction method is considered: for machine driven tunnels lower rock load factors are recommended. The wedge analysis has the additional advantage of considering actual discontinuity patterns.

The disadvantages are that RQD is not actually representative of all rock conditions. For the wedge analysis on the other hand the details may not be known during preliminary exploration; however, during construction the approach based on potentially unstable wedge may be used to adapt the support to the actual conditions.

3.5 QUANTITATIVE DIRECT METHODS

<u>Type - D - Methods</u>

Ground conditions are desxribed quantitatively either by a single parameter (Type D1) or by multiple parameters (Type D2). The parameters are then directly related to support requirement and excavation procedure (Fig. 3.5.1).

<u>Single parameter methods are:</u>

- Lauffer's stand-up time span relation (Sec. 3.5.1)
- RQD - Support relations by Deere and his group (Section 3.5.2)

<u>Multiparameter methods are:</u>

- Rock Structure Rating - RSR - Method by Wickham, et al. (Section 3.5.3)
- Rock Mass Rating - RMR - Method by Bieniawaki (Section 3.5.4)
- The Q-System - developed at NGI by Barton, et al. (Section 3.5.5)
- Louis' classification (Section 3.5.6)
- Franklin's classification (Section 3.5.7)

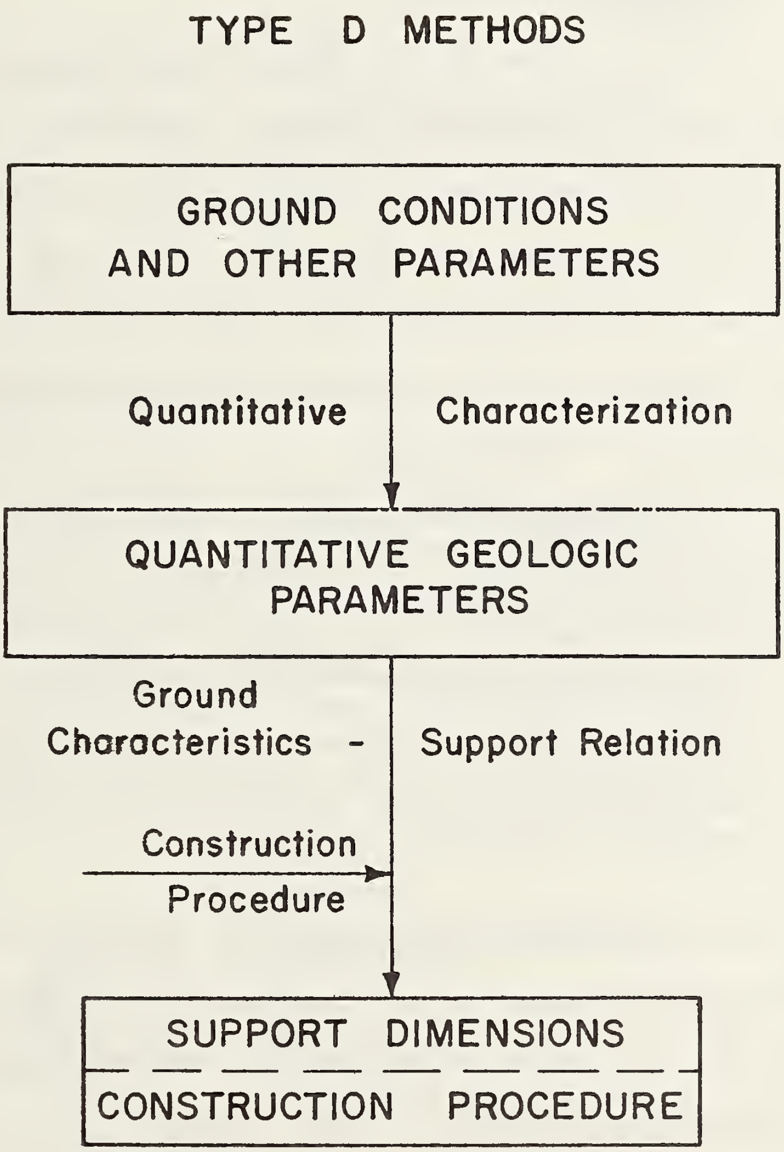


FIGURE 3.5.1 QUANTITATIVE DIRECT METHODS

3.5.1 Lauffer's Stand-up time -span- support relation

Methodology

Stand-up time is the time period during which an underground opening can remain unsupported without collapse or other serious disturbance. Ground conditions and the free (unsupported) span affect stand-up time has to be estimated, since an actual collapse should not occur. Detzlhofer (1974) describes in detail how the stand-up time can be estimated: (this is a partial, translation)

> ...."An estimate of the order of magnitude of the expected stand-up time, which is equal to the time until the support has to be placed, is possible. It will become easier to classify the ground after a time of adaptation to the actually encountered ground conditions, even though the actual stand-up time is unknown. Occassionally the acquired skill in classification may be verified with the occurence of a so-called 'inevitable' collapse ...."[1)]

As indicated stability of an opening or stand-up time is not only affected by ground conditions but also by the unsupported span ℓ^* which is defined as the smallest values of the three following distances: (Fig. 3.5.2)

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1) Detzlhofer gives this description how the stand-up time is estimated, but does not describe indicators that would announce collapses.

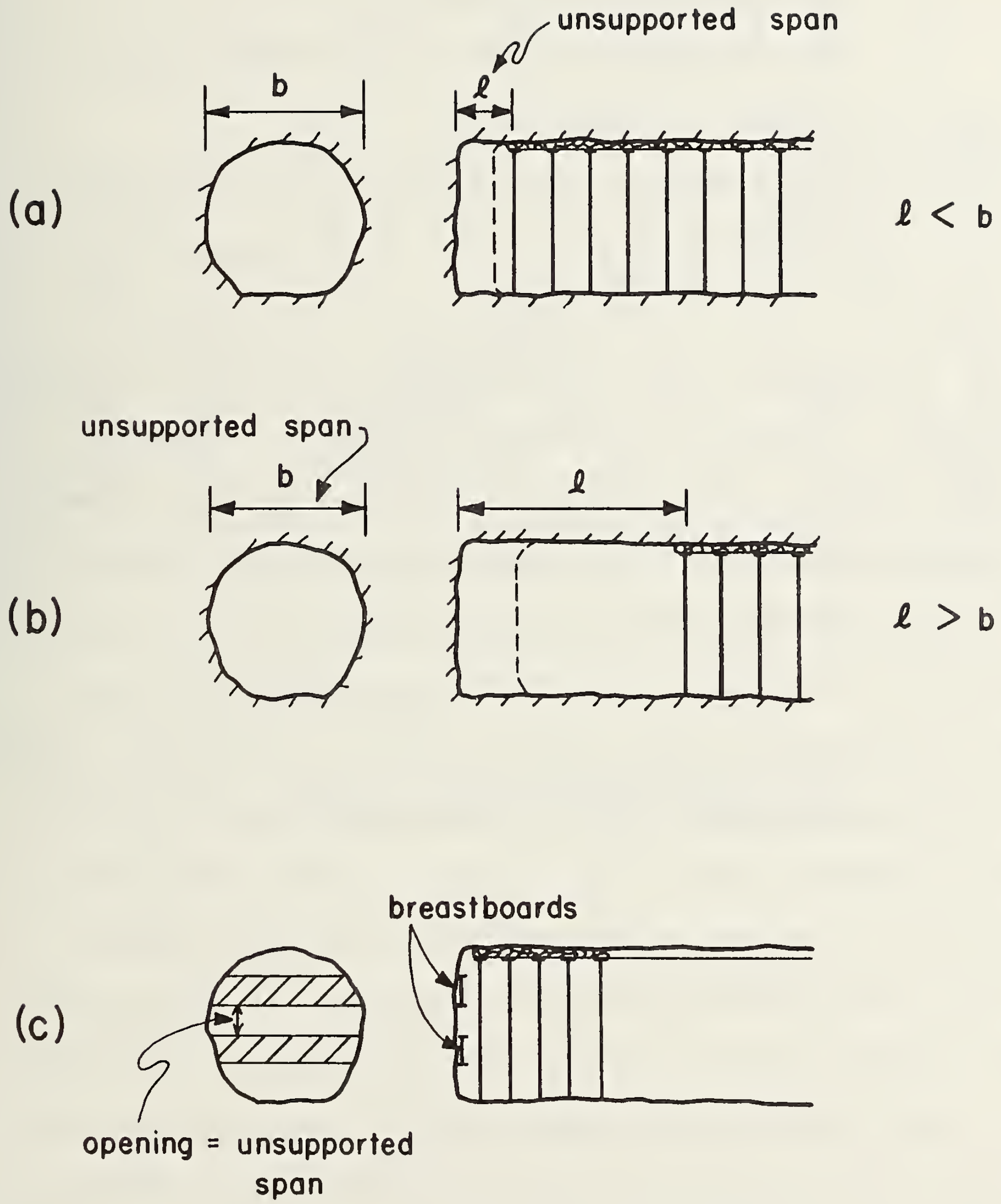


FIGURE 3.5.2 DEFINITION OF UNSUPPORTED SPAN
(AFTER LAUFFER, 1958)

(1) the tunnel span (diameter) (=b)

(2) the distance from the last placed support member to the face

(3) for breasted face the width of the opening at the face that can be left unsupported.

Unsupported span and stand-up time define a ground class (as shown in Fig. 3.5.3) which is in turn related to support requirements. The boundaries between ground classes are defined in Table 3.5.1. For each ground class Lauffer has given alternate support types: timber support, steel sets in combination with shotcrete, rock bolts in combination with wire mesh and shotcrete along (Table 3.5.2). A modified version of the Lauffer classification which is widely used has been published by Linder (1963) (Fig. 3.5.4), in this case shotcrete thickness is shown on the stand-up time span diagram (Fig. 3.5.4). The dimensions of the shotcrete are those given by Lauffer (1960) only the format of presentation is different.

The stand-up time span relation can only be tested in the tunnel and a prediction of the stand-up time is difficult. According to Spaun (1974) stand-up time resp. ground classes are predicted based on comparisons of the ground conditions for the new tunnel with experience gained in already built tunnels. The ground classes are estimated by direct comparison with experience gained in other tunnels.

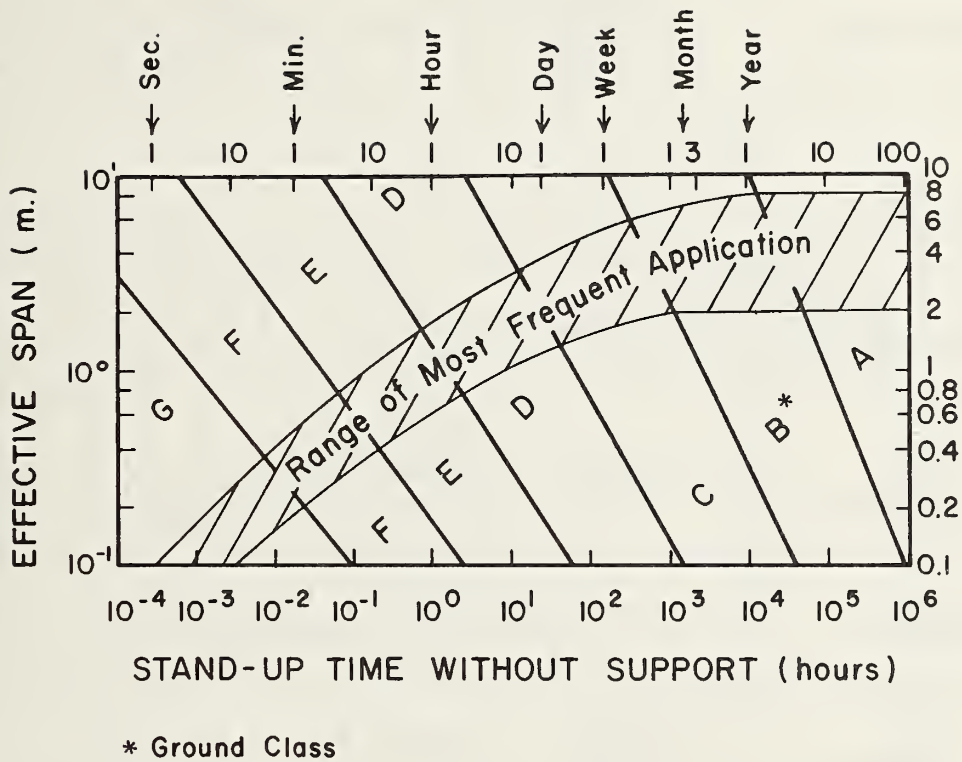


FIGURE 3.5.3 STAND-UP TIME UNSUPPORTED SPAN CLASSIFICATION (AFTER LAUFFER,1958)

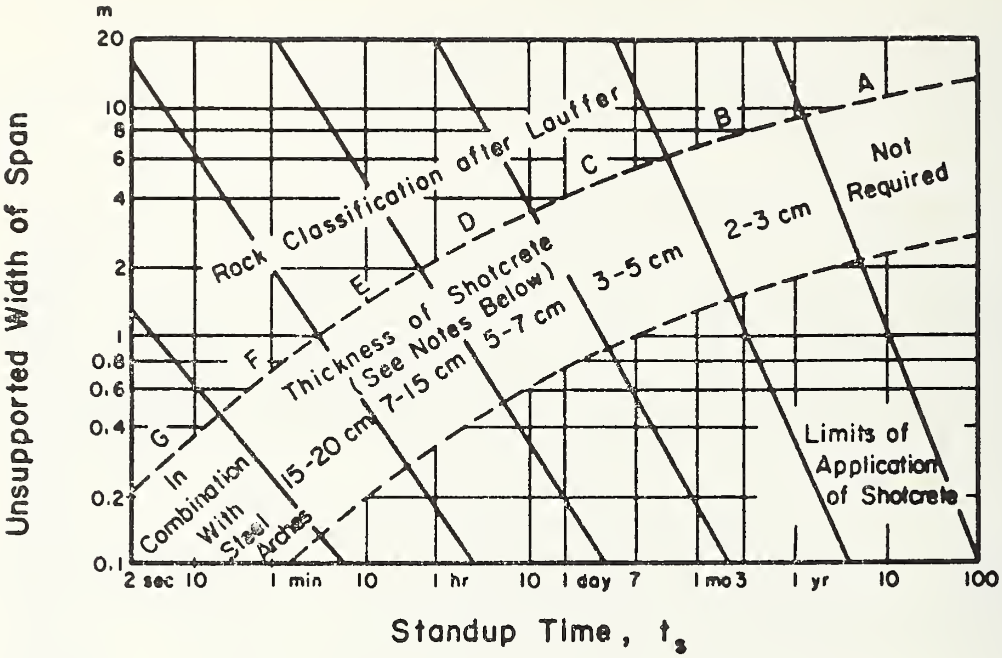
TABLE 3.5.1 TYPICAL EXAMPLE OF BOUNDARY LINES FOR STAND-UP TIME CLASSIFICATION AS A FUNCTION OF STAND-UP TIME t AND FREE SPAN ℓ^* (AFTER LAUFFER, 1958)

Ground Class	Examples for Stand-up Time and Span	Equations of the Assumed Boundary Lines (t in hours, ℓ^* in m)
A stable	20 years....4,0m	$t \times \ell^{1,0} = 1,0 \times 10^{5}$
B afterbreaking	6 months....4,0m	$t \times \ell^{1,2} = 2,5 \times 10^{3}$
C very afterbreaking	1 week......3,0m	$t \times \ell^{1,4} = 6,3 \times 10^{1}$
D breaking	5 hours.....1,5m	$t \times \ell^{1,6} = 1,6 \times 10^{0}$
E very breaking	20 mins.....0,8m	$t \times \ell^{1,8} = 4,0 \times 10^{-2}$
F squeezing	2 mins......0,4m	$t \times \ell^{2,0} = 1,0 \times 10^{-3}$
G very squeezing	10 secs.....0,15m	

TABLE 3.5.2 ALTERNATE SUPPORT RECOMMENDATIONS (AFTER LAUFFER, 1960)

Ground Class	Description	Timber Support	Stand-up time for unsupported span	Shotcrete	Rock Bolt with wedges (not grouted)	Steel Support Remaining in Tunnel
A	Standing	None	20 years 4.0m	No	No	Not necessary
B	After-breaking	Head Protection	6 months 4.0m	2 to 3cm in crown only	Bolts spaced 1.5 to 2.0m in crown with wirefabric	Application not economical
C	Highly afterbreak-ing	Crown Support	1 week 3.0m	3 to 5cm in crown only	Bolts spaced 1.0 to 1.5m in crown with wirefabric or 2cm shotcrete afterwards	Application not economical
D	Breaking	Light Timbering	5 h 1.5m	5 to 7cm pri-marily in crown with wirefabric	Bolts spaced 0.7 to 1.0m with wirefabric or 3cm shotcrete after placements of bolts	If necessary like class E
E	Very Breaking	Heavy Timbering	20 min. 0.8m	7 to 15cm with wirefabric	Only when bolt heads can be seated. Bolts spaced 0.5 to 1.2m with immediate shot-crete	Steel or con-crete lagging on steel sets
F	Squeezing	Forepole Timbering without face support	2 min. 0.4m	15 to 20cm with wirefabric and steel sets, if necessary face support with shotcrete	Cannot be bolted*)	Lagging on braced steel sets with ad-ditional shot-crete support afterwards
G	Heavily Squeezing	Forepole Timbering with face support	10 sec. 0.15m	Cannot be executed	Cannot be bolted*)	Lagging on braced steel sets with im-mediate shot-crete appli-cation

*) In classes F and G standard rock bolts (with wedges) cannot be used because they cannot be anchored (sliding, bearing capacity of wedges). Newer types of bolts (fully grouted bolts) may work, but were not available in the 1950's.



Notes:

(B) Alternatively rock bolts on 1.5-2 m spacing with wire net, occasionally reinforcement needed only in arch.

(C) Alternatively rock bolts on 1-1.5 m spacing with wire net, occasionally reinforcement needed only in arch.

(D) Shotcrete with wire net; alternatively rock bolts on 0.7-1m spacing with wire net and 3 cm shotcrete.

(E) Shotcrete with wire net; rock bolts on 0.5-1.2 m spacing with 3-5 cm shotcrete sometimes suitable; alternatively, steel arches with lagging.

(F) Shotcrete with wire net and steel arches; alternatively strutted steel arches with lagging and subsequent shotcrete.

(G) Shotcrete and strutted steel arches with lagging.

FIGURE 3.5.4 MODIFIED LAUFFER CLASSIFICATION, AFTER LINDER(1963), (FROM DEERE ET AL., 1969)

An approach which uses the strength of the ground mass and relates it to the stand-up time has been recently proposed by Vardar (1977). The procedure relates strain-rate dependent stress-strain properties of the 'rock mass', (which are intact properties to which a reduction factor has been applied) and elastic stresses to stand-up time. The procedure is based on many as yet unproven assumptions.

Development and Underlying Philosophy

The Lauffer stand-up time relation has been developed during the construction of the Prutz-Imst Hydropower tunnel in Austria. This tunnel and the parallel development of support practice is the development of empirical methods is of such importance that a more detailed description is given in Appendix B.

Initially Lauffer (1958) only published a chart relating the ground classes to stand-up time and unsupported span (Fig. 3.5.2), however, no support quantities were given. Support quantities were only quoted later by Lauffer (1960) reflecting the experience gained during the construction of the Prutz-Imst Hydropower tunnel.

Lauffer (1958) discusses also factors that have an influence on stand-up time.

The qualitative relations are shown in Fig. 3.5.5. The four factors influencing stand-up time discussed by Lauffer are:

- orientation of geologic structure (Fig. 3.5.5a)
- shape of tunnel cross-section (Fig. 3.5.5b)
- type of excavation (Fig. 3.5.5c)
- type of support procedure (Fig. 3.5.5d).

According to Fig. 3.5.5a, orientation of the discontinuities relative to the tunnel axis has an influence on stand-up time, discontinuities parallel to the tunnel lead to a decrease of the unsupported span compared to a discontinuities perpendicular to the tunnel. Fig. 3.5.5b shows that a circular cross-section is the most favorable in Lauffer's experience. In Fig. 3.5.5c it is shown that machine excavation has the most favorable effect, followed by blasting with low and high powder factors. With regard to support types (Fig. 3.5.5d) timbering is associated with the shortest unsupported span while shotcrete in combination with rock bolts makes the largest free span possible. Note that all these relations are qualitative and the formulation given in Lauffer's original article reflects his subjective assessment of the state of the art after the construction of the Prutz-Lust Power Tunnels (Appendix B). Lauffer credits Terzaghi, Stini and Rabcewicz with the introduction of the stand-up time.

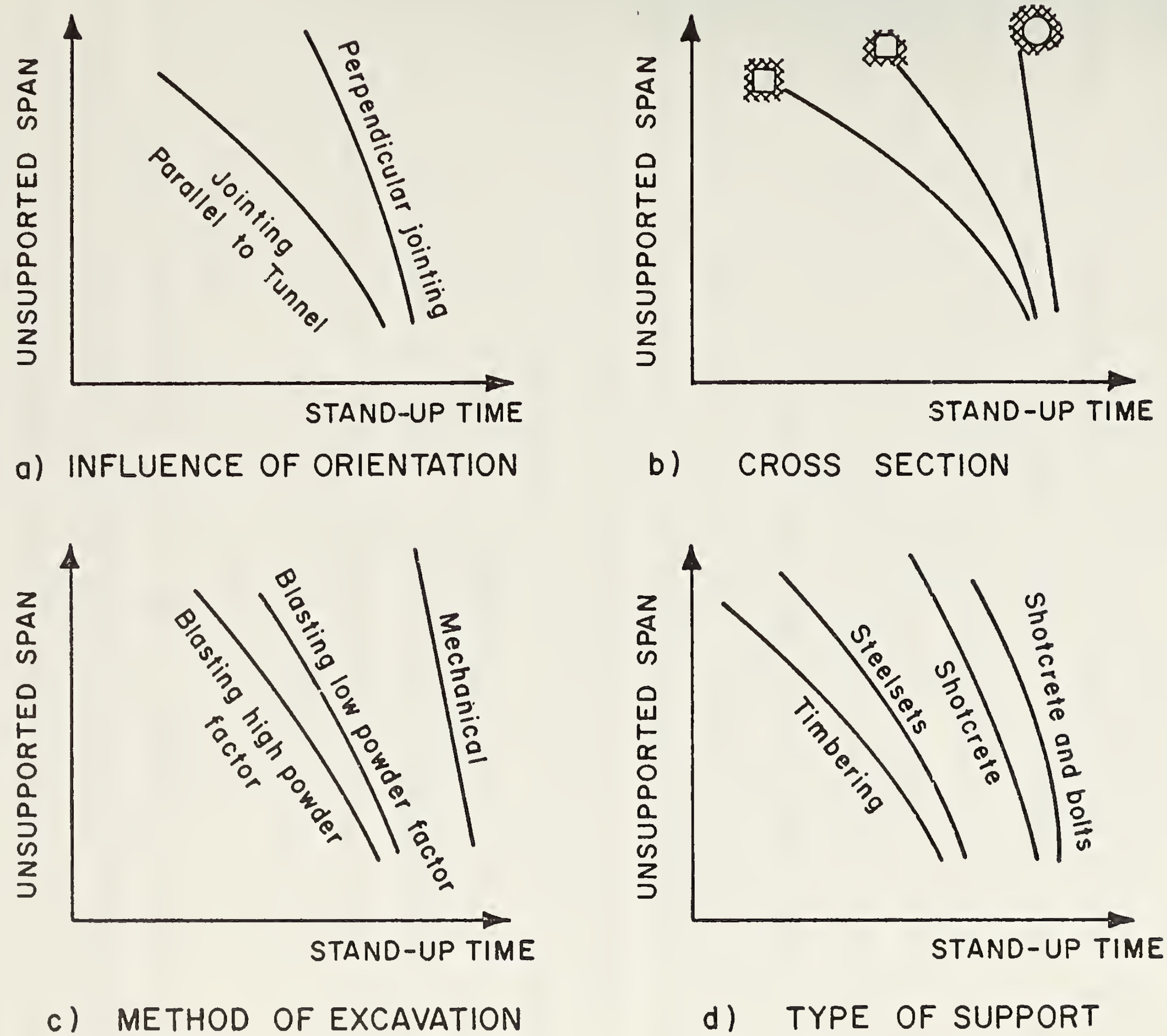


FIGURE 3.5.5 EFFECTS ON STAND-UP TIME SPAN RELATION(AFTER LAUFFER,1958)

Rabcewicz (1957) published a classification based on experience gained in Scandinavia which uses only stand-up time. Such a classification is sufficient as long as the round length and cross-section are constant. Stand-up time provides an estimate of the time which is available for the placement of support after the excavation. Further insight into Lauffer's method can be gained by detailed explanation of the stand-up time support chart (Fig. 3.5.3) which can also serve to eliminate some misinterpretation. The stippled band shows the region of the most frequent application. This band results from practical and economical limitations, the upper boundary of the band is given by the time required to place the support after the excavation of the round and this time must be shorter than the stand-up time. The lower boundary of the stippled band is given by the requirement that the free span should always be maximized such that the working area is as large as possible minimizing simultaneously the number of required excavation support cycles.

Lauffer has assumed boundary lines of different slopes between classes (Fig. 3.5.3, Table 3.5.1), he justifies flatter boundaries for worse ground conditions by the fact that the effect of unsupported span is more important in these cases.

Lauffer's papers has been modified by several authors, in particular Linder (1963). Linder combined the Lauffer (1958, 1960) stand-up diagram with Lauffer support recommendations (Lauffer 1960) into a single graph. This single figure found then its way into other publications and reports (e.g., Deere, et. al. 1969). (Fig. 3.5.4)

Detzlhofer (1974) writes that the Lauffer stand-up time chart reflects the experience at the Prutz-Imst Power tunnel of 5.3m diameter. Thus extrapolations to tunnels with larger diameters is difficult. During the construction of the Prutz-Imst pressure tunnel a pilot tunnel of $10m^2$ cross-section was driven. The performance was different in the pilot and the final tunnel illustrated with the cumulative distribution of encountered (Fig. 3.5.6) ground classes in the middle section Wenns. In the pilot tunnel approximately 70% of the ground was classified as ground class "a" and "b" (the "best" classes), however, in the final tunnel no classes "a" or "b" were encountered. This figure clearly illustrates the effect of tunnel size on tunnel support requirements. (More details of the Prutz-Imst Tunnel can be found in Appendix B.)

Critique

The Lauffer stand-up time - span relation is a report of a case study and should be taken as such. However, the concept of the stand-up time has always been used in tunnel construction since experienced tunnelers always tried to place the support before the tunnel collapsed and adapted the excavation support procedure accordingly. However, the excavation

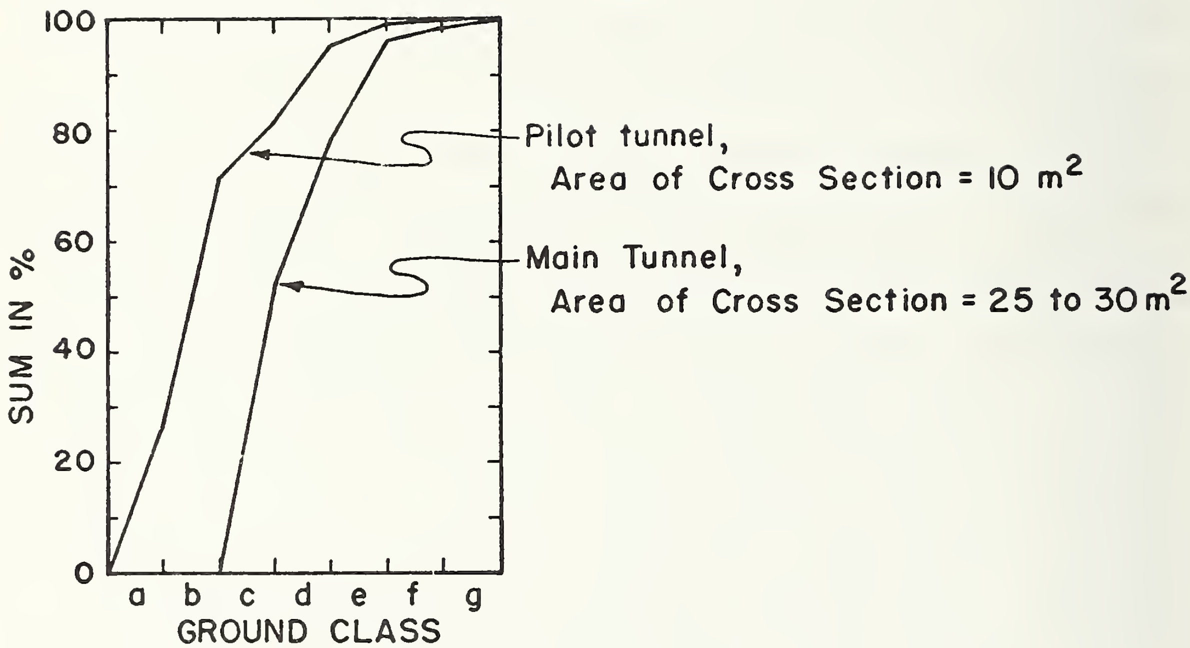


FIGURE 3.5.6 SIZE EFFECT ON GROUNDCLASSES OBSERVED IN SECTION WENNS OF THE PRUTZ-IMST TUNNEL (AFTER DETZLHOFER,1974)

support procedure may influence the stand-up time (which is assumed to be the stand-up time or virgin ground) as indicated by Lauffer himself in Fig. 3.5.5. This has been further demonstrated by many other authors e.g., by Korbin and Brekke (1975) in spiling reinforcement tests that showed increased stand-up time of ground supported by spiles. A classification which uses a procedure based alone on "stand-up time span" relations is thus usually not sufficient in tunneling.

Also, the definition of the unsupported span is not entirely consistent. The unsupported span is defined as the minimum of

(a) the tunnel span

(b) the distance from the last placed support member to the face

(c) for breasted faces: the width that can be left unsupported.

There is no ambiguity in definitions (a) and (b). The discrepancy arises when the support is placed up to the face: If the face does not require support, the effective span ℓ^* is zero; however, if there are breast boards required, the unsupported span (according to definition (c)) becomes larger than zero.

One other important note has to be made here. Lauffer (1958) writes his paper partly in the subjunctive and not as fact report. The stand-up time-unsupported span relation is to some extent a report about the experience from the Prutz-Imst Tunnel, however, to some extent it is also a proposal how ground could be classified based on stand-up time. Lauffer's stand-up time has been and is widely used in the German-speaking area, and resulted also in new developments. Berger (1969) wants to use the size of afterbreaks (i.e., the volume of rock that would fall out from the crown) as a classification criterion. For each ground class, Berger quotes a size of afterbreak.

However, to determine the size of the roofalls one would actually have to wait until they occur; this is not practical since this is neither economical nor very safe (excessive overbreak and subsequent backfill with concrete). Berger also criticizes the fact that Lauffer's classification does not consider the size of the tunnel for the design of the support, e.g., even if the excavation procedes by heading and benching the support has to be designed for the full size opening.

Koerner (1971) provides some critique from the point of view of the engineering geologist. The strongest criticism by Koerner concerns the validity of the stand-up time as <u>the</u> single parameter to describe ground behavior. Ground conditions should be considered explicitely like, bedding planes of the rock, jointing, groundwater and strength properties.

Koerner proposes to modify the stand-up time chart. In Lauffer's chart afterbreaking and squeezing ground are seperate classes. Koerner (Fig. 3.5.7) proposes to consider squeezing and afterbreaking ground in parallel and not seperately. Koerner states that squeezing ground is caused once the stresses around a tunnel exceed the strength of the rock mass which means that squeezing ground is also a function of the overburden and the strength, afterbreaking, however, is not necessarily dependent an overburden. Koerner thus postulates two mechanisms that may lead to the same stand-up time.

Despite all these limitations Lauffer's stand-up time chart has often been indiscriminantly included in recommendation for support design.

Summary of Lauffer's Method

Stand-up time and unsupported span are related to support requirements. Stand-up time characterizes ground behavior and is thus a useful classification parameter.

A method based on only two parameters may be considered advantageous. In reality stand-up time cannot be determined it has to be estimated. Other limitations of the method are:

- the Lauffer stand-up time span relation is a report of a case study and extrapolation is not simple.

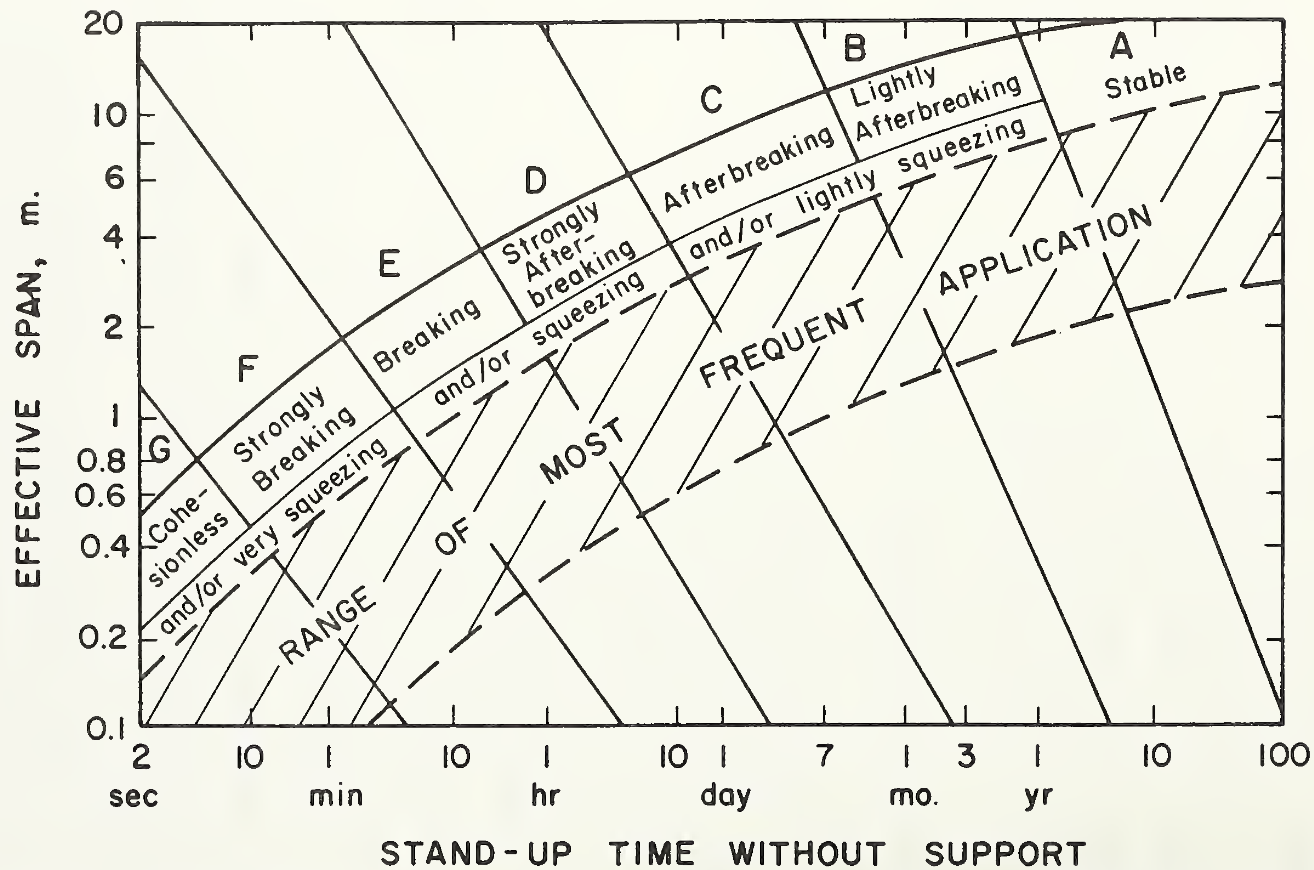


FIGURE 3.5.7 KOERNER'S MODIFICATION OF THE STAND-UP TIME UNSUPPORTED CLASSIFICATION (AFTER KOERNER,1971)

- Excavation support procedures may influence stand-up time.

3.5.2 Deere's Empirical Relations

Methodology and Application Guidelines

Deere and his co-workers at the University of Illinois have developed several related methods for the empirical design of tunnel support. They are:

1) RQD - Support relations (Deere, et al. 1969 a,b, Merritt, 1972);
2) RQD - Rockload, relations (Deere, et al. 1969a, Monsees, 1970);
3) Support capacity in large caverns (Cording et al. 1971);
4) Wedge Stability (Cording and Deere, 1972; Cording and Mahar, 1974);
5) Deere, Merritt, Cording (1974), 'descriptive' classification.

The Rock load methods, i.e., the RQD Rock load relation, the support capacity observed in large caverns, and the analysis of potentially unstable wedges were discussed in Sec. 3.4.1. Here the direct RQD - Support relation and the Deere, Merritt, Cording descriptive classification are discussed in parallel. The intended application and underlying philosophy of the Deere, Merritt, Cording descriptive classification is quite different from the RQD - Support relation, however, since its development was a logical outgrowth of the work by Deere and his colleagues it is discussed in this section as well.

RQD - Support Relation

The ground is described by RQD (Rock Quality Designation) which is a modified core recovery. RQD is the percentage of the length of intact core that is longer than 10cm (4 inches). (See also Appendix C.) Based on RQD the support can be estimated with Table Deere 3.5.3. Different support quantities are given for tunnels conventionally excavated by drilling and blasting and for machine bored tunnels. Support quantities distinguish between steel sets, rock bolts or shotcrete (sometimes a combination of these three support systems). The limitations of these RQD - support relations are clearly expressed in Table 3.5.4. The simplifying assumptions that are made are also listed. In particular these support recommendations should be only used for tunnels of 20 to 40 ft. diameter. Deere et al. clearly state that their recommendations reflect 1969 U.S. technology.

During actual construction the performance should be monitored and the ground support relations should be updated accordingly.

Descriptive Classification (Deere et al., 1974)

A modified verbal classification system has been published by Deere et al. (1974); they consider three major classes.

- Good tunneling ground.
- Average to difficult tunneling ground.
- Very difficult to hazardous tunneling ground.

TABLE 3.5.3 DEERE ET AL. RQD-SUPPORT RELATIONS (FROM DEERE ET AL., 1969)

GUIDELINES FOR SELECTION OF PRIMARY SUPPORT FOR 20-FT TO 40-FT TUNNELS IN ROCK

Rock Quality	Construction Method	Alternative Support Systems: Steel Sets			Rock Bolts[a] (Conditional use in poor and very poor rock)		Shotcrete[b] (Conditional use in poor and very poor rock)		
		Rock Load (B = Tunnel Width)	Weight of Sets	Spacing[c]	Spacing of Pattern Bolts	Additional Requirements and Anchorage Limitations[a]	Total Thickness: Crown	Total Thickness: Sides	Additional Support[b]
Excellent[d] RQD > 90	Boring machine	(0.0 to 0.2)B	Light	None to occasional	None to occasional	Rare	None to occasional local application	None	None
	Drilling and blasting	(0.0 to 0.3)B	Light	None to occasional	None to occasional	Rare	None to occasional local application 2 to 3 in.	None	None
Good[d] RQD = 75 to 90	Boring machine	(0.0 to 0.4)B	Light	Occasional to 5 to 6 ft	Occasional to 5 to 6 ft	Occasional mesh and straps	Local application 2 to 3 in.	None	None
	Drilling and blasting	(0.3 to 0.6)B	Light	5 to 6 ft	5 to 6 ft	Occasional mesh or straps	Local application 2 to 3 in.	None	None
Fair RQD = 50 to 75	Boring machine	(0.4 to 1.0)B	Light to medium	5 to 6 ft	4 to 6 ft	Mesh and straps as required	2 to 4 in.	None	Provide for rock bolts
	Drilling and blasting	(0.6 to 1.3)B	Light to medium	4 to 5 ft	3 to 5 ft	Mesh and straps as required	4 in. or more	4 in. or more	Provide for rock bolts
Poor RQD = 25 to 50	Boring machine	(1.0 to 1.6)B	Medium circular	3 to 4 ft	3 to 5 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.	4 to 6 in.	4 to 6 in.	Rock bolts as required (~4-6 ft cc.)
	Drilling and blasting	(1.3 to 2.0)B	Medium to heavy circular	2 to 4 ft	2 to 4 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.	6 in. or more	6 in. or more	Rock bolts as required (~4-6 ft cc.)
Very poor RQD < 25 (Excluding squeezing and swelling ground)	Boring machine	(1.6 to 2.2)B	Medium to heavy circular	2 ft	2 to 4 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section		Medium sets as required
	Drilling and blasting	(2.0 to 2.8)B	Heavy circular	2 ft	3 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section		Medium to heavy sets as required
Very poor, squeezing or swelling ground	Both methods	up to 250 ft	Very heavy circular	2 ft	2 to 3 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section		Heavy sets as required

Note: Table reflects 1969 technology in the United States. Groundwater conditions and the details of jointing and weathering should be considered in conjunction with these guidelines particularly in the poorer quality rock.

[a]Bolt diameter = 1 in., length = 1/3 to 1/4 tunnel width. It may be difficult or impossible to obtain anchorage with mechanically anchored rock bolts in poor and very poor rock. Grouted anchors may also be unsatisfactory in very wet tunnels.

[b]Because shotcrete experience is limited, only general guidelines are given for support in the poorer quality rock.

[c]Lagging requirements for steel sets will usually be minimal in excellent rock and will range from up to 25 percent in good rock to 100 percent in very poor rock.

[d]In good and excellent quality rock, the support requirement will in general be minimal but will be dependent on joint geometry, tunnel diameter, and relative orientations of joints and tunnel.

TABLE 3.5.4 LIMITATIONS OF RQD SUPPORT RELATIONS (AFTER DEERE ET AL., 1969)

The support recommendations listed in Table 3.5.3 are based on several simplifying assumptions, the most important of which are the following:

1. The RQD adequately describes the quality of the rock.
2. The support systems are installed as close to the face as possible; for steel sets and for rock bolts this would be about 2 to 4 ft., and for shotcrete essentially zero. Furthermore, it is assumed that the support systems are properly installed, i.e., lagging and blocking is tightly placed behind steel sets and rock bolts are properly tensioned.
3. The tunnel has a cross-section (either horse-shoe or circular) with the height approximately equal to the width.
4. The tunnel is approximately 20 to 40 ft. in width.
5. The natural stresses in the ground are low enough that stress concentrations around the periphery generally do not exceed the compressive strength of the rock.

For each of these three categories the ground and hydraulic conditions leading to this category are described in detail as are the consequences on construction. (See Table 3.5.5.)

Development and Underlying Philosophy

RQD - support relations have been developed by Deere, his students and collaborators at the University of Illinois. Studies on this subject have been conducted and are reported by:

Coon (1968)

Deere, Merritt, Coon (1968)

Deere, et al. (1969a, b)

Monsees (1970)

Cecil (1970, 1975)

Merritt (1972)

The original correlations were based on a limited amount of data. The initial work was performed by Coon (1968) which is also reported in Deere, Merritt, Coon (1968). The data base included a total of 14 tunnels; four of these tunnels studies were described in some details. In three of these four cases actually measured RQD's were available, in the fourth case RQD was estimated from joint spacing data and corrected for alteration of the rock (detailed explanation follows later in this section).

The first two cases consider two tunnels along Interstate 40 in North Carolina. The geology was similar with respect to rock type and core recovery. The first

TABLE 3.5.5a DEERE ET AL. (SAO PAOLO): DESCRIPTIVE CLASSIFICATION CATEGORY 1 (FROM DEERE ET AL, 1974)

Category 1 -- Good Tunneling Ground

The Category 1 rock condition needs little, if any, clarification. The rock is generally hard and only moderately jointed with joint surfaces tight and unaltered, and the joints discontinuous and irregular. Tunnels may be driven "bald-headed" (without any support) or with occasional rock bolts. Large-diameter tunnels (greater than 8m) and underground chambers will usually be pattern bolted.

The RQD values would be 90-100 (excellent rock); Terzaghi's classification would be hard and intact to massive, moderately jointed; and RSR values would probably be above 70 percent.

There are two geological conditions which could be adverse. One involves in-situ stress. The high quality rock of this category may in some areas be under considerable residual stress, perhaps from 100-750 kg/cm^2, and dangerous "popping" or "spalling" conditions may occur. Rock bolts and wire mesh or steel straps have been successfully used for mild popping. For more severe conditions, shotcrete and rock bolts or steel sets with close lagging have been required to protect the workmen.

The second possible adverse condition that may be encountered in this good category is the occurrence of very hard rock (orthoquartzite, metaquartizite, siliceous dolomite, quartz pegmatite dikes) which will make the cutter or bit costs of tunnel boring machines quite high and will reduce the rate of advance. Even drilling of blast holes for conventional drill-and-blast tunneling will be more expensive.

TABLF 3.5.5b DEERE ET AL DESCRIPTIVE CLASSIFICATION (SAO PAULO): CATEGORY 2 (page 1 of 3) (FROM DEERE ET AL, 1974)

Category 2 -- Average to Difficult Tunneling Ground

This second category constitutes the average rock conditions encountered in most tunnels. The rock is closely to moderatedly jointed with the joints often weathered or altered somewhat, and with slickensided joints, shear zones, and small to medium faults being quite common. Joints are often planar and continuous. RQD values would range from 25-90 through the designated categories of poor, fair, and good. In the Terzaghi classification the rock would typically range from blocky to very blocky and seamy with small zones of crushed or squeezing ground. The RSR value would generally be in the 40-60 range.

There are three important considerations in the design and construction of tunnels and chambers in this category: (1) the size effect; (2) the directional effect; and (3) the support appropriateness effect.

Size Effect. -- A small tunnel driven in blocky, seamy, and occasionally sheared rock may present few problems for support. Conventional rock bolts, thin shotcrete, or light steel ribs could be used as needed, or on a pattern basis. Advance rates would not be slowed by large collapses or by numerous fall-outs from the crown or face of the tunnel heading.

A large tunnel, however, in the same ground could experience severe stability problems where the shear zones or seams intersect the arch, high sidewalls, or the face. Large blocks could move into the opening either from the side or from the face that would not move in on a small opening. Similarly, the excavation of wide openings with shallow rock cover would be difficult, requiring multiple drifts and possibly pre-support umbrellas of rock bolts. A small opening in the same location would be routine construction. Similarly, in squeezing ground a small opening could be handled using steel ribs with invert struts whereas for a large opening multiple-drift excavation with stage concreting would be required.

Thus, whether a rock condition is "average" or "difficult" for tunneling (or even "very difficult and hazardous") depends to a considerable extent on the size of the tunnel.

Directional Effect. -- Most rock masses contain 3 or 4 sets of joints. These sets exert some influence on the size of tunnel muck, on the required explosives factor, on the overbreak and the shape of the excavated walls and roof, and, perhaps on the spacing of rock bolt support. However, the effects are "average" effects and it matters little as to the direction of tunnel driving. Overbreak will occur in about average amounts (20-30cms) and the excavated shape will be somewhat irregular. The amount of support required will be essentially independent of tunnel direction. (cont.)

TABLE 3.5.5b DEERE ET AL. DESCRIPTIVE CLASSIFICATION (SAO PAULO): CATEGORY 2 (page 2 of 3) (FROM DEERE ET AL., 1974)

However, where one set is dominant (and particularly where the joint surfaces are weakened by weathering or alteration) the directional effect becomes important. When driving parallel or sub-parallel to a predominant steep to vertical weakness, slabbing and overbreak on the sidewalls will occur. Also, large rock wedges may occur above the crown caused by the intersection of two of these steep joints. These wedges may loosen suddenly and fall out, or load the steel sets. When driving across such weakness in a perpendicular direction, however, little effect is noted unless the weak plane dip into the excavation in which case it is difficult to maintain a stable vertical face for drilling. When the weakness is sub-horizontal (0 to 30° dip), the roof will tend to break to the weakness. The overbreak may average 30-50cms. for both the vertical and horizontal cases.

Where the weakness is well developed and continuous such as for foliation shears, seams, or small faults, the directional problem attains even greater significance (Brekke and Howard, 1972; Deere, 1973). Quoting from the last reference,

> If the tunnels are sub-parallel to the foliation shear, then the problem is much more extensive as stability problems will first be encountered on one wall of the tunnel, then the roof, and finally across the other wall of the tunnel. Depending on the angle between the strike of the foliation shear and the direction of the tunnel the poor rock condition may continue for several hundred feet and hundreds of steel sets or large quantities of shotcrete and bolts might be required, as in the current water tunnel being driven in New York, or the subway tunnels and stations in Washington, D.C. ...

In summary, while the directional effect may not be too important in average isotropic conditions, it can be very important where strong anisotropic weaknesses exist.

Support Appropriateness Effect. -- For the Category 2 rock condition, "average to difficult", it has been noted that the degree of difficulty of tunneling depends to a considerable extent on the tunnel size and possibly on the direction of the tunnel. An equally significant factor is the appropriateness of the support used for the existing rock conditions.

The rock conditions can usually be handled safely with reasonable rates of advance and with reasonable costs if appropriate methods are used. Otherwise, fall-outs, heading cave-ins, injuries, slow advance rates, and contractural disputes may occur. (cont.)

TABLE 3.5.5b DEERE ET AL DESCRIPTIVE GROUNDCLASSIFICATION (SAO PAULO, 1974): CAGEGORY 2 (page 3 of 3) (FROM DEERE ET AL., 1974)

One common problem is where the design engineers consider that the rock conditions are reasonably good or at least of "fair" quality, and that the rock support will be mainly by rock bolts with the occasional use of steel ribs in the worst ground. During construction the contractor finds the rock generally to be quite blocky and seamy so that blocks of rock occasionally move and fall out during the drilling of the rock bolt holes. After this happens a couple of times or a worker is injured, the contractor contends that the ground is "steel ground" and he starts using heavy steel rib supports in large quantities with large cost overruns. The cheaper alternates of shotcrete and rock bolts, lighter steel ribs, or Bernold sheets could probably solve the problem but often are not part of the specified support items in the contract.

There is considerable organized effort at present to develop forms of contracts which could remove much of the risk from tunnel contracts and allow the design engineers and the contractor to work closely together throughout construction to achieve the most economical yet safe tunnel. Engineering geologists from both parties would certainly be involved in helping to select appropriate support and in projecting the observed geologic features ahead of the face for pre-planning.

TABLE 3.5.5c DEERE ET AL. DESCRIPTIVE CLASSIFICATION (SAO PAULO): CATEGORY 3 (FROM DEERE ET AL, 1974)

Category 3 -- Very Difficult to Hazardous Tunneling Ground

This third category of tunneling ground, fortunately, only occurs over a small percentage of the length of an average tunnel. Occasionally, it may make up a large percentage of a tunnel, in which case the time and cost of completing the tunnel may be very great -- often by a factor of two or more over the original estimate.

As noted previously, if the very difficult and hazardous condition is known in advance, the tunnel will still be difficult and costly to construct. If the condition is encountered unexpectedly, the results may be disastrous.

The geological conditions that lead to this category of tunneling ground include: (1) wet, very seamy, heavily jointed or sheared rock; (2) squeezing ground under large pressure; (3) high water pressures; (4) high water inflows; and (5) highly stressed rock.

The first four conditions may be associated with faults. The wet, very seamy, heavily jointed or sheared rock often occurs adjacent to fault zones. When encountered, it must be supported rapidly, including breast-boarding of the face, or cave-ins may be experienced. Supports must be adequate as loads tend to increase with time causing "heavy ground". Where clay or highly altered rock occurs on the fractures or in thick seams, the ground becomes squeezing ground and full-circule steel rib supports are required. The ground pressures with time may approach full-overburden pressure (although one-third to one-half of that is more likely; see Semple , Hendron, and Mesri (1973).

High water pressures equal to the full piezometric level of the groundwater table have been encountered in fault zones. This high water pressure can cause bursting of the roof, floor, or face -- even in hard rock. In sandy fault breccia or fault gouge it may cause piping and the sudden in-rush of masses of water and fault debris. It is also probable that part of the driving force in squeezing ground is caused by seepage forces of the high-pressure water being forced through the clayed gouge. Obviously, one of the best ways to treat a fault zone is to drill through it so as to pre-drain the high-pressure water on the far side to the maximum extent possible. Where several faults cross each other, the groundwater regime is often complex. The draining of one fault block by driving the tunnel into it, or by pre-drainage holes, may not lower the water pressure in the adjacent blocks because the faults act as dams and each fault block is essentially isolated hydraulically from its neighbor.

(cont.)

TABLE 3.5.5c DEERE ET AL DESCRIPTIVE CLASSIFICATION (SAO PAULO): CATEGORY 3 (CONT.) (FROM DEERE ET AL., 1974)

<u>High water inflows</u> may or may not be associated with high water pressures. Large inflows may be encountered in faults but alos in solution-riddled limestone or gypsum, and in lava flows (either in lava tubes or in flow breccia at the contacts of lava flows). A combination of high-capacity pumping and grouting may be required to handle the water. <u>Highly stressed rock</u> occurs in massive rock with few joints. In strong massive rocks such as granite or gneiss the stress in some cases may be so high that when peaked-up around the tunnel opening (by a stress concentration factor of 2 to 3) the new stress approaches the compressive strength of the rock. Slabbing occurs around the tunnel periphery by extension strain. Doming of the face may also take place. The slabbing is often very violent with large pieces being ejected at high velocity. This condition may range from a minor "popping" condition to a violent "rock bursting" condition. In the extreme condition the tunnel may be completely closed. The weaker rocks such as volcanic tuffs, sandstones, shales, and siltstones may also be stressed to their compressive strength (which would require a lower stress than for the granite or gneiss) and similar slabbing would occur almost coincident with the tunnel face advance. While not so violent as in the case of the strong rocks, and therefore not so dangerous, it is necessary to provide full-circle support of rock bolts, wire mesh, and shotcrete or circular steel ribs to support the failed rock and to reduce inward movement of the material.

tunnel did not experience any stability problems and stood unsupported for ten years (it was not used during this time due to a change in highway construction priorities). The second tunnel, however, built later required steel sets at 4 to 10 feet centers. Deere et al (1968) list several factors which differ for the two sites:

(1) different joint orientations at site 2, the joints strike subparallel to the tunnel.

(2) the joints at site 2 are more persistent.

(3) the joints in tunnel 2 showed more alteration, staining and clay seams.

(4) site 2 has more water.

(5) lithologies are similar however the proportions changed at site 1 quartzite is dominant with thin zones of slate and phyllite. At site 2 quartzite is reduced in favor of the other rocks.

(6) at site 1 the average RQD was 87 percent; at site 2 the average RQD was 29 percent.

In Deere's evaluation attention was primarily given to the rock quality designation; the intention was to examine if RQD could be used to correctly reflect the difference in rock conditions as it affected tunnel supports.

In the third case study a section of the Tehachapi Project was used. In Tunnel No. 3 a horizontal boring of 180 feet was drilled ahead of the tunnel. Joint fracture spacing and RQD were determined in the boring (Fig. 3.5.8) and plotted versus the actually placed support. The shear zone had a low RQD and was supported by steelsets and

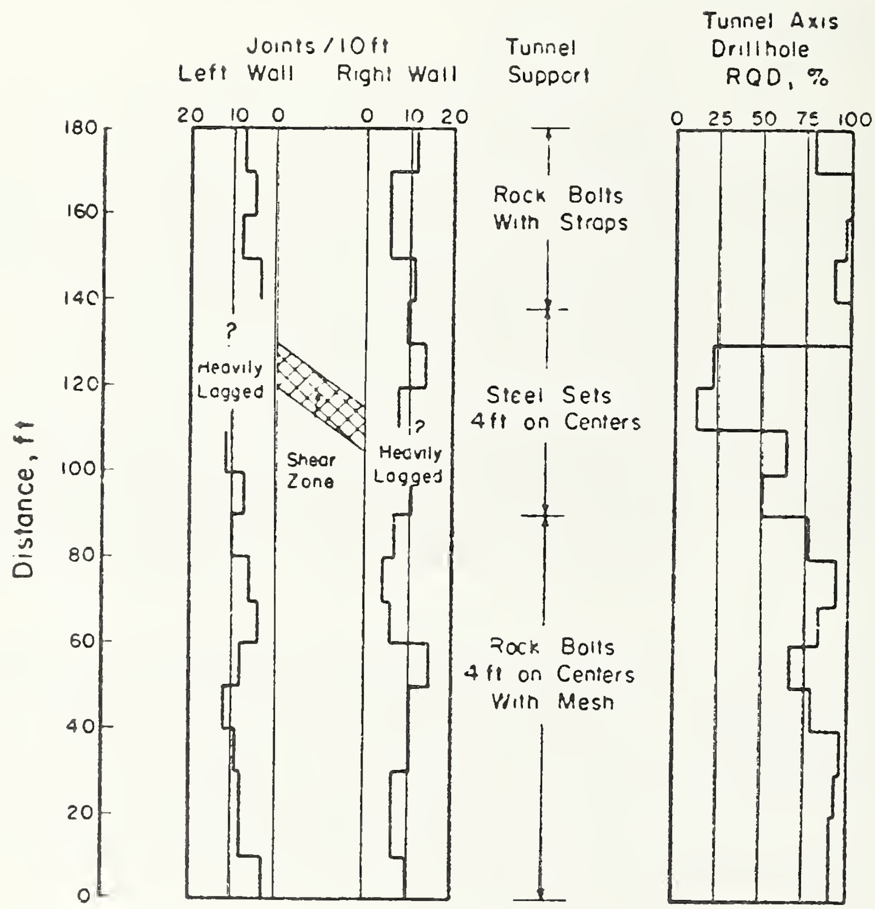
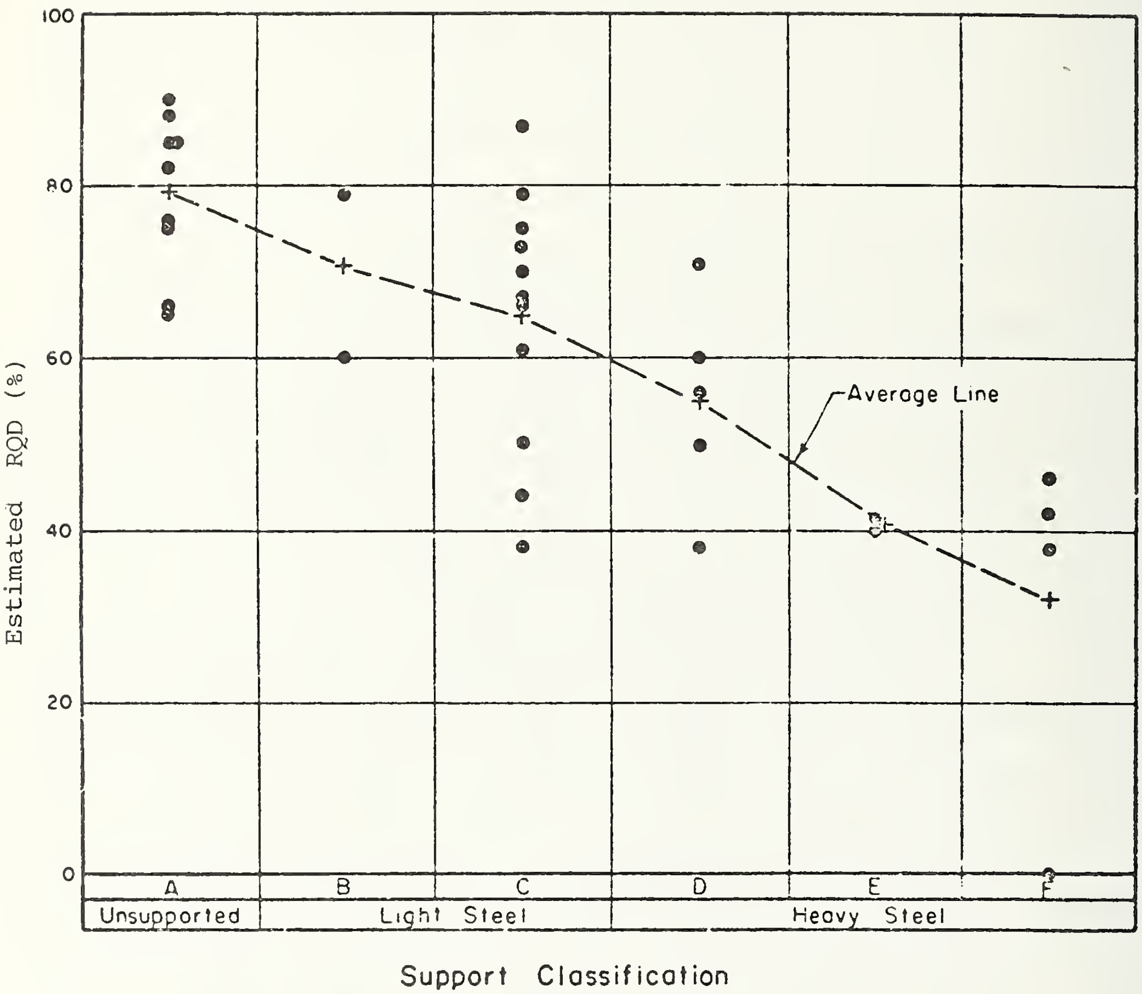


FIGURE 3.5.8 COMPARISON OF FRACTURE SPACING,RQD AND TUNNELSUPPORT IN THE TEHACHAPI TUNNEL NO. 3 (FROM DEERE ET AL,1968)

lagging. The zones on both sides of the shear zone showed high RQD (>75%) and rockbolts were used as support. Note that the fracture spacing is also shown in Fig. 3.5.8. The correlation between RQD and support requirements is good. Deere et al. (1968) state, "There seems to be no such correlation between joint frequency and support." They explain this by the fact that RQD was determined from a predrilled boring, whereas joint frequency was determined at the tunnel walls, and since the tunnel was heavily lagged in the vicinity of the shear zone, it may not have been possible to determine joint spacing or joint frequency.

Deere's fourth case study is the pilot tunnel for the Straight Creek Tunnel. Geologic data (joint frequency and alteration) and support data were obtained from a geologic report of the pilot tunnel. The joint frequency and alteration data were transformed by means of the correlations shown in Fig. 3.5.9 to RQD. Support was grouped into six classes; this and the RQD support relation is shown in Fig. 3.5.9. For each class there is considerable scatter and as a consequence, also the RQD ranges of adjacent classes overlap. The average values of RQD for the classes show, however, a consistent trend. The average values of RQD were plotted in Fig. 3.5.10.

As mentioned earlier a total of 14 case studies were available; the data for all cases are listed in Table 3.5.6. From these data a plot of RQD vs. opening width



RQD estimate based on reported average joint spacing and percent altered.

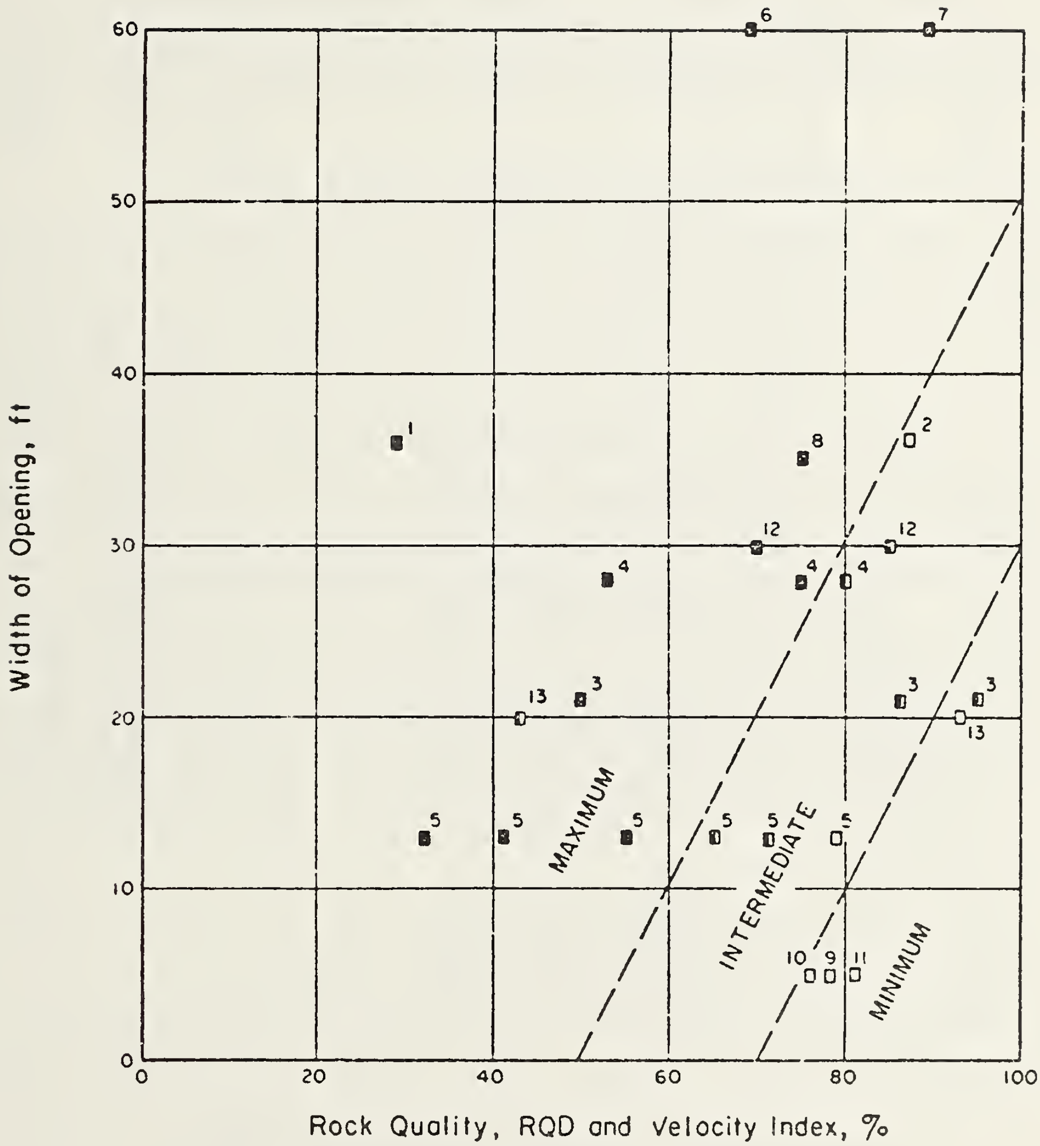
Assumed :

Joint Spacing	RQD
>1 ft	90
1 - 1/2	75
<1/2	50

RQD (est.) = RQD (jt. spacing) − % altered

Support Classification: A - Unsupported
B - 4 in. Ribs, 4-5' Spacing
C - 4 in. Ribs, 2-3' Spacing
D - 6 in. Ribs, 4-5' Spacing
E - 6 in. Ribs, 2-3' Spacing
F - 6 in. Ribs with Struts

FIGURE 3.5.9 COMPARISON OF ESTIMATED RQD TO SUPPORT CLASSIFICATION FOR THE STRAIGHT CREEK PILOT BORE (FROM DEERE ET AL, 1968)



- Unsupported or Occassional Rock Bolts
- Light Steel or Pattern Rock Bolts
- Heavy Steel or Pattern Rock Bolts (Long Bolts, Mesh)

Numbers refer to Table 3.5.6

FIGURE 3.5.10 ROCK QUALITY DESIGNATION (RQD) VERSUS TUNNEL WIDTH PLOT (FROM DEERE ET AL,1968)

TABLE 3.5.6 SUMMARY OF INITIAL CASE STUDIES FOR RQD SUPPORT RELATION (FROM DEERE ET AL, 1968)

Project	Width of Opening, ft.	Support	RQD or Velocity Index
1 Pigeon River No. 1	36	Unsupported	87
2 Pigeon River No. 2	36	8 in. WF 10 to 4 ft	29
3 Tehachapi Site 3	21		see Fig.3.5.8
4 Tehachapi Site 1	28	8 x 8 in. 6 ft o.c., and bolts 5 ft o.c.	54-80
5 Straight Creek	13	Heavy support to unsupported	see Fig.3.5.9
6 Cavity I, NTS	Hemisphere with radius of 60 ft	Top 32 ft - bolts, 3 ft o.c. Mid 24 ft - bolts, 3 ft o.c. Bot 16 ft - bolts, 6 ft o.c.	72 90
7 Cavity II, NTS	Hemisphere with radius of 60 ft	Top 32 ft - bolts, 3 ft o.c. Mid 24 ft - bolts, 3 ft o.c. Bot 16 ft - bolts, 6 ft o.c.	69
8 Cavity III, NTS	Hemisphere with radius of 35 ft	Top 24 ft - bolts, 3 ft o.c. Mid 16 ft - bolts, 3 ft o.c. Bot 8-16 ft - bolts, 6 ft o.c.	75
9 Adit at Two Forks	5		78
10 Adit at Yellowtail Dam	8		
11 Adits at Dworshak Dam	5		81
12 Diversion Tunnel at Dworshak Dam granite gneiss 960 ft	30	47% ribs, 53% bolts	Estimated 85
Schistose gneiss 760 ft	11	77% ribs, 23% bolts, ribs 5 ft o.c.	Estimated 70
13 Shaft, East Coast	20	Temporary timber and bolts - concrete lining w/ i 20 ft of face	44-93
14 Tunnel, NTS	10	Unsupported	75

has been obtained (Fig. 3.5.10). The support quantities have been divided into three ranges (minimum, intermediate, maximum).

This Figure seems to have served as a basis for the development of the support recommendations (Table 3.5.3). Note that in Table 3.5.3 also rock load factors are listed which correspond to those empirically derived for the RQD-Rock load correlation (see Section 3.4.1). The rock load factors for TBM's are 25% less than those for conventional drill and blast tunnels. The 'direct' case studies did not provide all the data for a complete table, in particular, no machine-bored tunnels were included, in the case studies. Thus rock loads were used to develop support recommendations for ranges of RQD. Details of the design are, however, not given.

Further work on the correlation of ground conditions expressed with RQD, to support requirements has been performed by Cecil (1970, 1975). The two publications by Cecil are identical in content, the first one is the Ph.D. thesis submitted to the University of Illinois, the second one is Proceedings No. 27 of the Swedish Geotechnical Institute.

Some of Cecil's results are reproduced here. Cecil studied a total of 97 tunnel sections in detail and attempted to correlate RQD, width and support requirements.

Fig. 3.5.11a shows a plot of RQD vs. width for all cases with the exception of cases where swelling clay gauge occured in joints and shear zones.

Fig. 3.5.11b compares the support requirements, derived by Cecil to those recommended by Coon's (1968). In Fig. 3.5.13 (begins on page 166) unsupported cases are plotted on the RQD - width diagram. Many of these cases actually plot in the area of maximum support. Cecil calls whese cases "anomalous". Detailed data on these tunnels are listed in Table 3.5.7. The common feature of these cases is that the ground has a single set of tight steeply dipping discontinuities. They strike primarily perpendicular to the tunnel but not in all cases (Table 3.5.7). Cecil summarized his findings on "anomalies" as follows:

> "The anomalies include: all rock conditions that contain softening clay materials, thin clay coatings, and single-sets of steeply dipping, closely spaced tight joints. The stability conditions and rock support associated with all of these geologic features may have absolutely no relationship to the rock quality designation. The first two conditions may lead to instability in rock with a high RQD value (>75%), whereas the second condition is often stable at very low (<20%) RQD values."

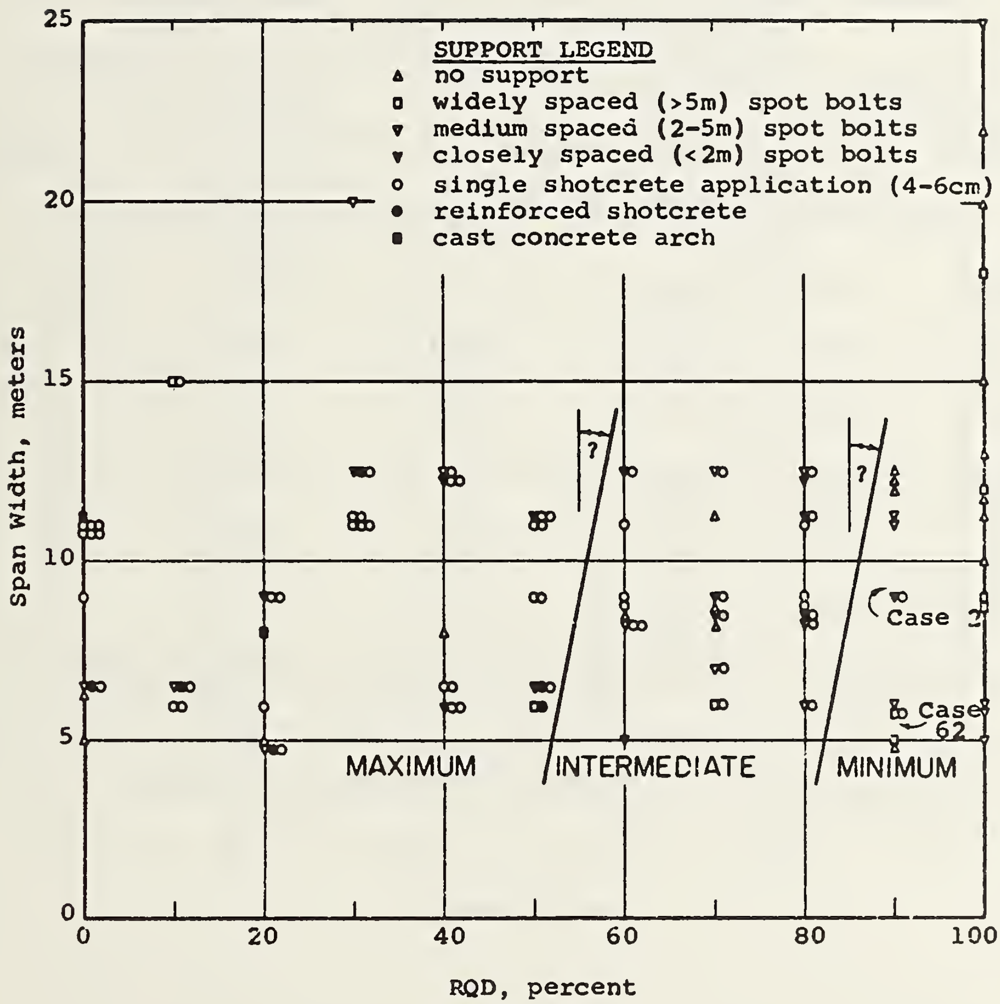


FIGURE 3.5.11a RQD TUNNEL WIDTH PLOT FOR CECIL'S CASE CASE STUDIES , CASES WITH CLAY GAUGE EXCLUDED (FROM CECIL,1975)

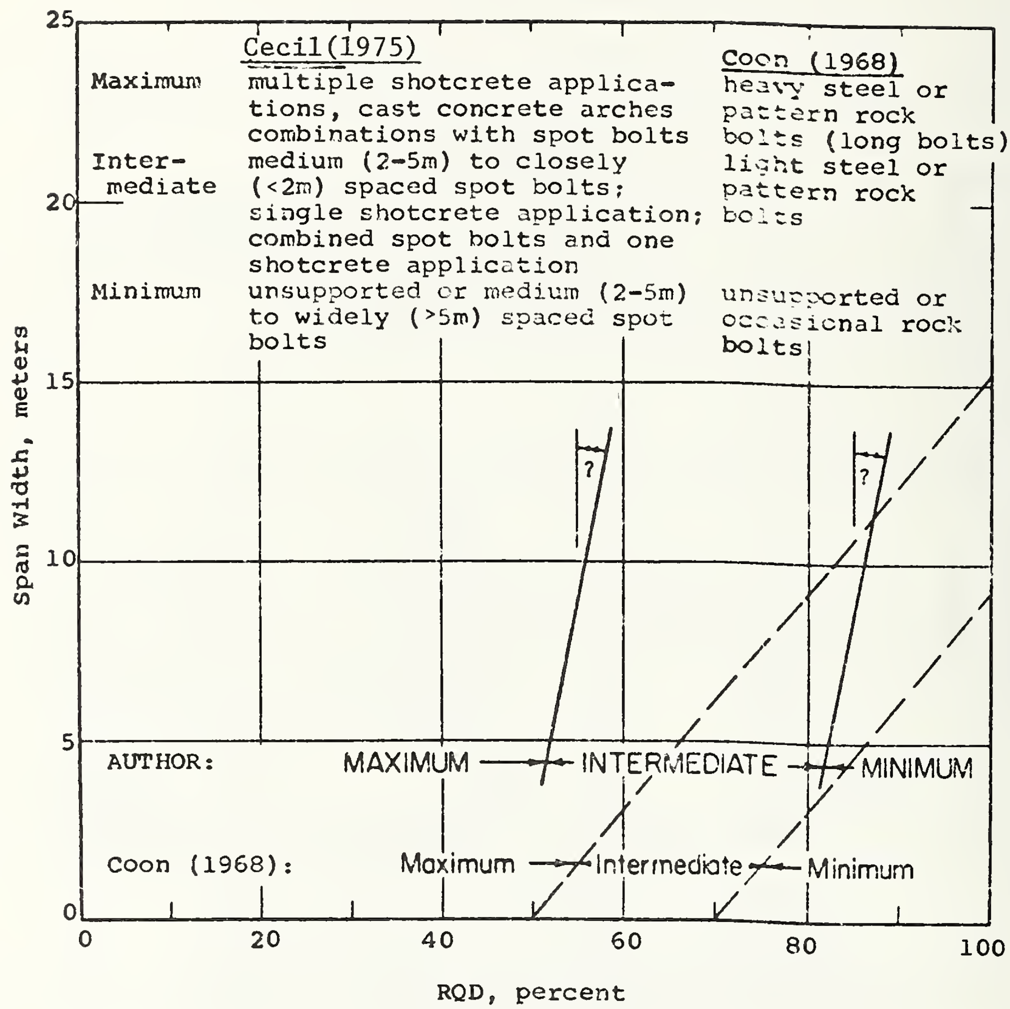


FIGURE 3.5.11b COMPARISON OF CECIL'S AND COON'S SUPPORT REGION BOUNDARIES (FROM CECIL, 1975)

TABLE 3.5.7 UNSUPPORTED TUNNELS WITH LOW RQD (FROM CECIL, 1975)

Case Number	RQD, percent	Strike of Major Discontinuities (from tunnel axis)	Dip of Major Discontinuities	Degree of Joint Discontinuity	Tightness of Discontinuity	Discontinuity Filling
6	60	0-30°	60-90°	Discon-tinuous	Tight	None
8	70	30-60°	30-60°	Discon-tinuous	Tight	None
20	70	0-30°	60-90°	Continuous	Tight	None
35	0	60-90°	60-90°	Discon/ Con	Tight	None
36	20	60-90°	60-90°	Discon/ Con	Tight	None
52	0	30-60°	30-60°	Con/ Discon	Tight	None
70	40	60-90°	60-90°	Continuous	Tight	None
83	70	60-90°	0-30° 60-90°	Discon/ Con	Tight	None

The RQD-support span relations were further advanced by Merritt (1972). Merritt considered about 60 case studies and plotted them on tunnel width RQD charts. (Fig. 3.5.13a and b). Merritt's study included steelset and bolt supported tunnels, but no shotcrete support. Merritt also notes the inapplicability of RQD if low strength shear zones with clay gouge are present. It is interesting to compare Coon's charts (Fig. 3.5.10) and Cecil's experience (Figs. 3.5.11 and 12) with Merritt's results (Figs. 3.5.13a, b), which show a continued development of the RQD-support relation and a continuous increase in the range of applicability, in particular the use of rock bolts for a wider range of RQD.

Deere and his colleagues explicitely recommend updating ground support relations by monitoring performance and incorporating this experience.

Descriptive Classification

As mentioned at the beginning and described in detail earlier, Deere, Merritt, and Cording developed the 'descriptive' classification. As an outgrowth of their experience in tunnel design and construction they saw the need for an overview type classification that could be applied initially in a project before proceeding with a more detailed empirical design method. The Deere, Merrit, Cording descriptive classification considers three principal categories of ground (Table 3.5.5).

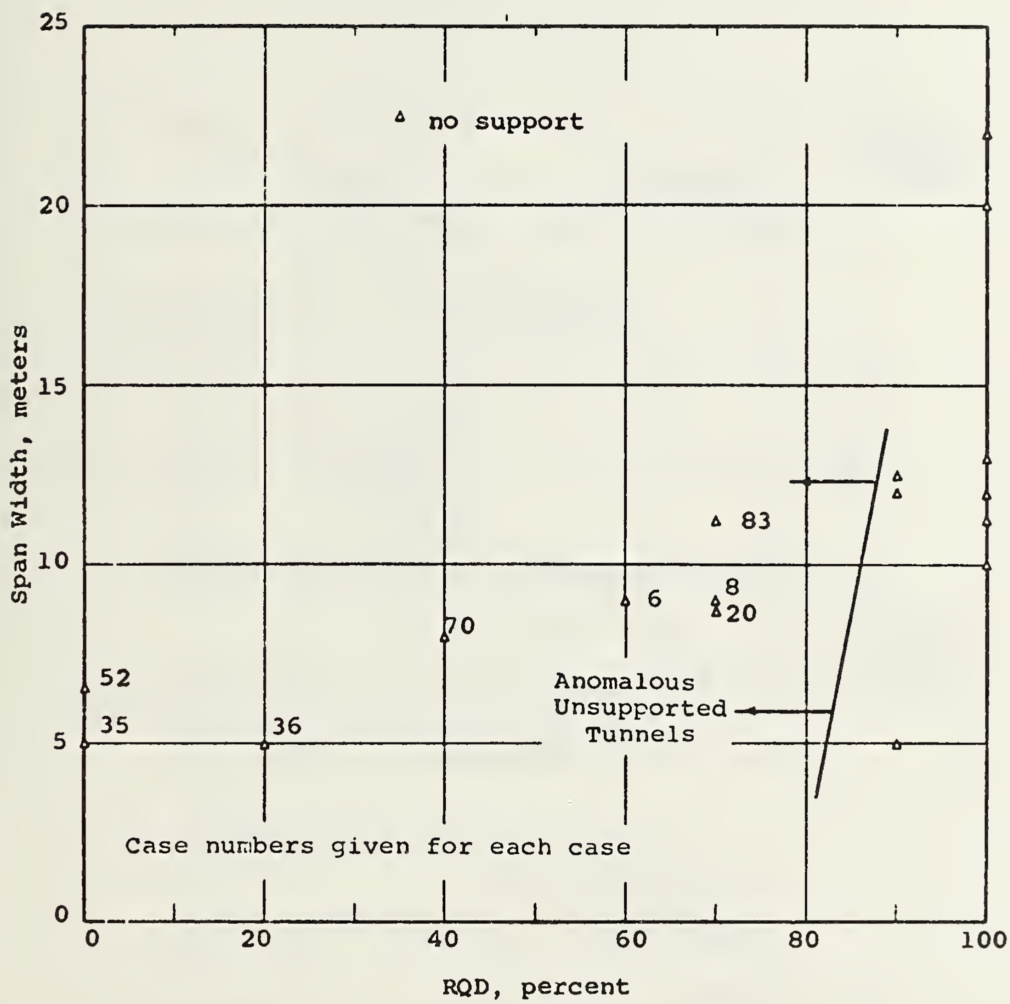
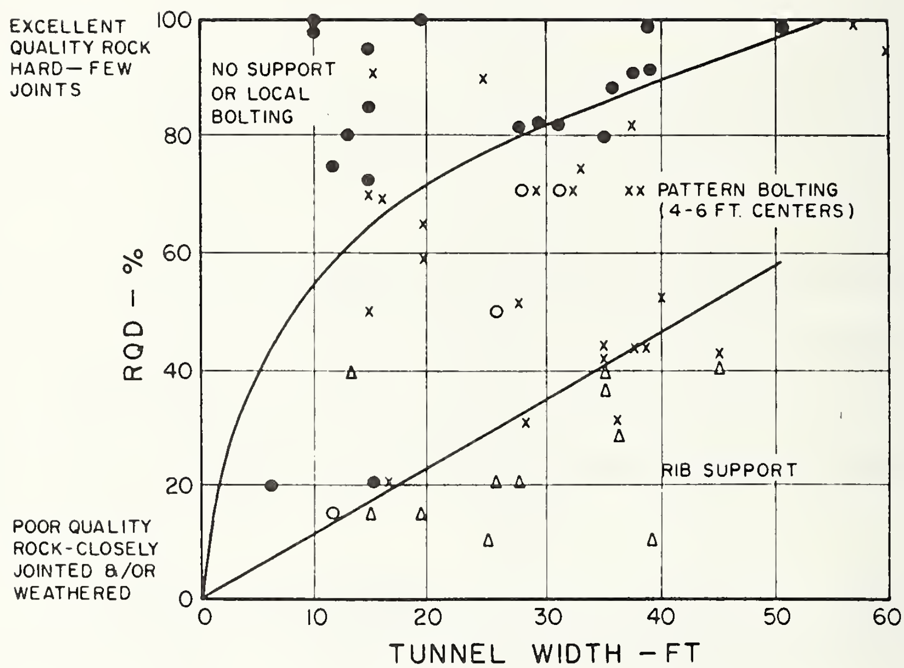


FIGURE 3.5.12 UNSUPPORTED CASES REPORTED BY CECIL(1975)

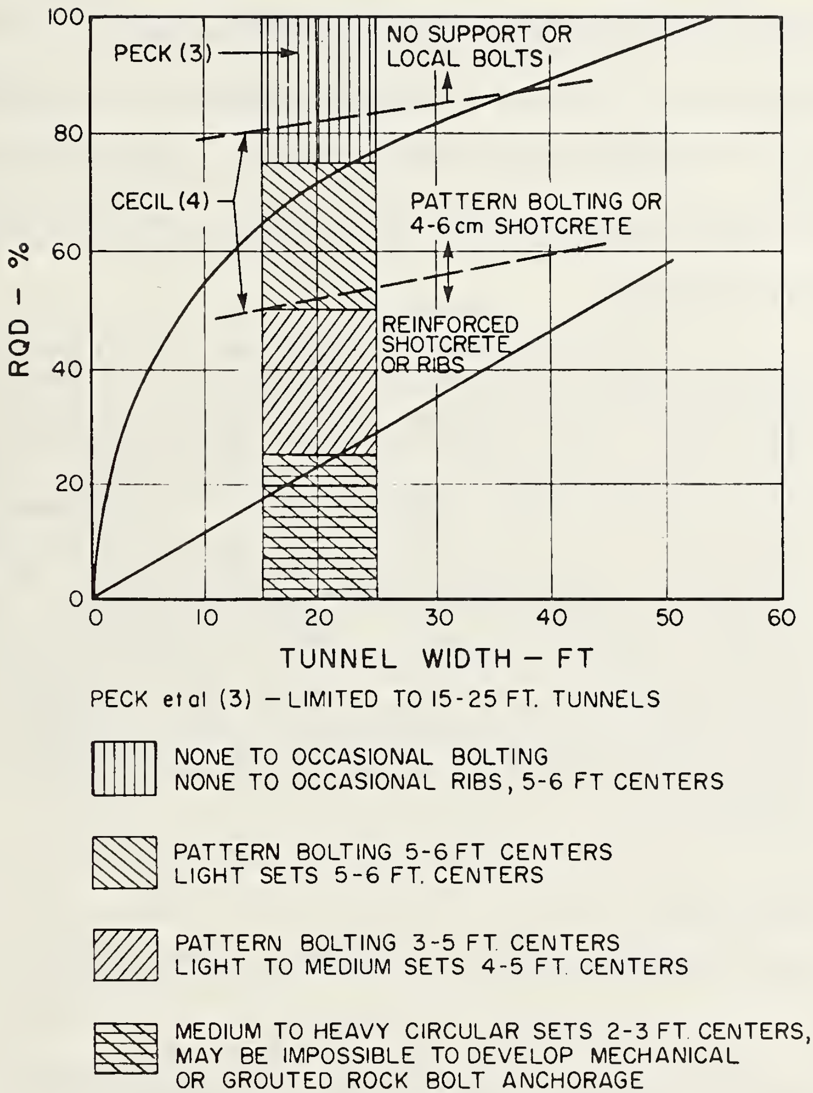


● NO SUPPORT
o OCCASIONAL BOLTS
x PATTERN BOLTING
Δ STEEL RIBS

NOTE

SUPPORT DATA FROM IGNEOUS AND METAMORPHIC ROCKS WHERE REAL ROCK PRESSURES OR SWELLING/SQUEEZING GROUND DID NOT EXIST

FIG 3.5.13a RQD SUPPORT RELATIONS FOR BOLT AND STEEL SET SUPPORTED TUNNELS (FROM MERRITT, 1972)



Cecil (4) = Cecil (1975)
Peck et al. (3) = Peck et al. (1969)

FIG. 3.5.13b COMPARISON OF RQD-SUPPORT RELATION
(FROM MERRITT, 1972)

Good Tunneling Ground

Average to Difficult Tunneling Ground

Very Difficult to Hazardous Tunneling Ground

For each of these three categories the ground and hydraulic conditions leading to this category are discussed as are the consequences on construction. The underlying philosophy and intended application is made clear by Deere's et al. (1974) statement:

> "...The three new categories of tunneling ground proposed herein (Category I - Good; Category II - Average to Difficult; and Category III - Very Difficult to Hazardous) provide a broad and simple grouping which would appear to have merit as general terms. Each category requires its own methods od construction and support...Most Promising Developments--It is believed that we will see the concurrent use of two systems: one a general system (Categories I, II, and III), and two, a more specific system such as the RSR or RQD modified to take into account the structured attitude and the surface characteristics of the joint surfaces..."

Critique

RQD support relations were a major step in relating information available from rock cores to anticipated tunneling characteristics. The use of the method increased rapidly and its present application is widespread.

Support predictions based on RQD are easy to perform and they give a first indication of the support requirements based on information gathered from borings.

However, RQD alone does not include all the factors that influence the stability and support requirements.

In particular, if more information on the ground is available, the entire information should be used. A reduction to a RQD may result in a loss of information.

The limitations of RQD based support prediction are clearly expressed by Deere, et al. and should be considered when applying the method (Table 3.5.4). For a detailed discussion on the meaning of RQD see also Appendix C.

The table of support recommendations is the subjective assessment by the developers of the method. This conclusion has been reached after studying Deere, et. al. (1969) and Monsees (1970) where the following identical statement was found:

> "Coon (1968) has shown that a qualitative relationship exists between the RQD and the support required for tunnels in rock. For steel sets or precast-concrete segments, it should be possible to design the supports to resist a load that is a function of the rock quality, the size of opening, and the construction technique employed. The tentatively recommended loads are given in Table 3.5.3. By relating the load to the RQD, rather than to Terzaghi's qualitative description of rock quality, the proposed system is less sensitive to variations in personal observations.
>
> For other tunnel support methods (such as rock bolts and shotcrete) it is necessary to consider the possible mechanisms of instability and the manner in which each of the support methods acts to maintain stability. This consideration, combined with an evaluation of the types and amounts of support that have proven to be successful, leads to guidelines (see Table 3.5.3) on which to base the design.
>
> The use of the recommendations in Table 3.5.3 will produce designs that are equal to or more economical than designs obtained by methods currently in use. It is desirable to

make observations of the tunnel behavior during and following the construction of the tunnel. Such observations will indicate tendencies toward instability which might occur in areas of particularly bad rock. Thus, potential problems can be detected early enough to take corrective action before a failure occurs."

These statements indicate that the creators of the RQD - Support relation attempted to make the predictions more objective than by other methods available at that time. However, they stronly recommend to perform observations in the tunnel to verify the adequacy of the support and to modify the support in case of unsatisfactory performance.

Cecil's (1970) findings should also be remembered as there are cases where the support requirements are not related to RQD. They include widely spaced clay filled zones or swelling clay zones where even for high RQD support may be necessary. On the other hand, for a single set of steeply dipping, closely-spaced tight joints no support may be required even with low RQD.

The 1974 Deere-Merritt-Cording 'descriptive' classification considers aspects of orientation, shear strength of the discontinuities, the influence of water conditions and construction procedure. Geologic features that lead to a parcular ground condition are discussed as are general construction procedures. The method is not intended to provide detailed support recommendations for which the RQD relations (or similar detailed methods) have to be used.

Summary of Deere's Direct Ground-Support Relations

Deere et al. relate RQD directly to support requirements. RQD can be easily determined from core borings. The method has proven useful and it is widely and successfully applied. The method differentiates between conventionally built and machine bored tunnels. Deere et al. recommend also to monitor the performance during construction of the tunnel and to adapt the support accordingly. The RQD has, however, some limitations, the same RQD may represent different rock structures with respect to joint spacing, persistence and orientation. If zones of low shear strength are encountered in a rock with a high RQD more support may be necessary than predicted only with the RQD-support relation.

The Deere-Merritt-Cording (1974) descriptive classification compliments the RQD-support relations. It is intended to be used in the preconstruction phase of a project by differentiating between three major ground categories. During construction it should be supplemented by a more detailed method.

3.5.3 The Rock Structure Rating - RSR - Method by Wickham, et. al.

Methodology and Application Guidelines

The Rock Structure Rating (RSR) Method employs three parameters A,B,C to assess ground conditions. (A "good" condition is rated with a high value). Parameter A (Table 3.5.8) assesses the general area geology and is a function of rock type, degree of decomposition and geological structure. Parameter B (Table 3.5.9) rates the joint pattern (joint spacing) and includes also the relative direction of the tunnel axis. Parameter C (Table 3.5.10) considers ground water and conditions of the discontinuity. The sum of the parameters A+B+C is the RSR rating for drill and blast tunnels. For tunnels driven with TBM the Rock structure rating is increased by the TBM-factor (Fig. 3.5.14). RSR is used to determine the rib ratio RR a support scale, i.e. a numerical value that corresponds to certain support dimensions and quantities. The relation between rock structure rating and rib ratio is by the following equation:

TABLE 3.5.8 ROCK STRUCTURE RATING: PARAMETER "A"
(FROM WICKHAM ET AL., 1974)

ROCK STRUCTURE RATING
PARAMETER "A"
GENERAL AREA GEOLOGY

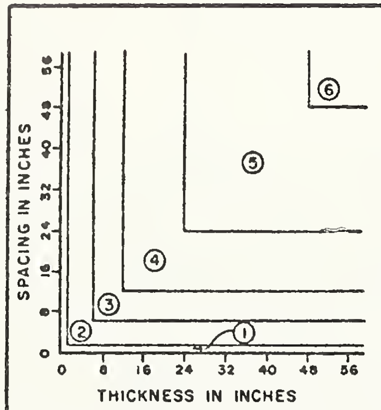
MAX. VALUE 30

BASIC ROCK TYPE

	HARD	MED.	SOFT	DECOMP.
IGNEOUS	1	2	3	4
METAMORPHIC	1	2	3	4
SEDIMENTARY	2	3	4	4

	GEOLOGICAL STRUCTURE			
	MASSIVE	SLIGHTLY FAULTED OR FOLDED	MODERATELY FAULTED OR FOLDED	INTENSELY FAULTED OR FOLDED
TYPE 1	30	22	15	9
TYPE 2	27	20	13	8
TYPE 3	24	18	12	7
TYPE 4	19	15	10	6

TABLE 3.5.9 ROCK STRUCTURE RATING: PARAMETER "B"
(FROM WICKHAM ET AL 1974)



ROCK STRUCTURE RATING PARAMETER "B" JOINT PATTERN DIRECTION OF DRIVE								MAX. VALUE 45
	STRIKE ⊥ TO AXIS					STRIKE ∥ TO AXIS		
	DIRECTION OF DRIVE					DIRECTION OF DRIVE		
	BOTH	WITH DIP		AGAINST DIP		BOTH		
	DIP OF PROMINENT JOINTS					DIP OF PROMINENT JOINTS		
	FLAT	DIPPING	VERTICAL	DIPPING	VERTICAL	FLAT	DIPPING	VERTICAL
(1) VERY CLOSELY JOINTED	9	11	13	10	12	9	9	7
(2) CLOSELY JOINTED	13	16	19	15	17	14	14	11
(3) MODERATELY JOINTED	23	24	28	19	22	23	23	19
(4) MODERATE TO BLOCKY	30	32	36	25	28	30	28	24
(5) BLOCKY TO MASSIVE	36	38	40	33	35	36	34	28
(6) MASSIVE	40	43	45	37	40	40	38	34

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

TABLE 3.5.10 ROCK STRUCTURE RATING: PARAMETER "C"
(FROM WICKHAM ET AL., 1974)

ROCK STRUCTURE RATING
PARAMETER "C"
GROUND WATER
JOINT CONDITION

MAX. VALUE 25

ANTICIPATED WATER INFLOW (GPM/1000')	SUM OF PARAMETERS A + B					
	13 - 44			45 - 75		
	JOINT CONDITION					
	GOOD	FAIR	POOR	GOOD	FAIR	POOR
NONE	22	18	12	25	22	18
SLIGHT (<200 gpm)	19	15	9	23	19	14
MODERATE (200-1000 gpm)	15	11	7	21	16	12
HEAVY (>1000 gpm)	10	8	6	18	14	10

Joint Condition: Good = Tight or Cemented; Fair = Slightly Weathered or Altered; Poor = Severely Weathered, Altered, or Open

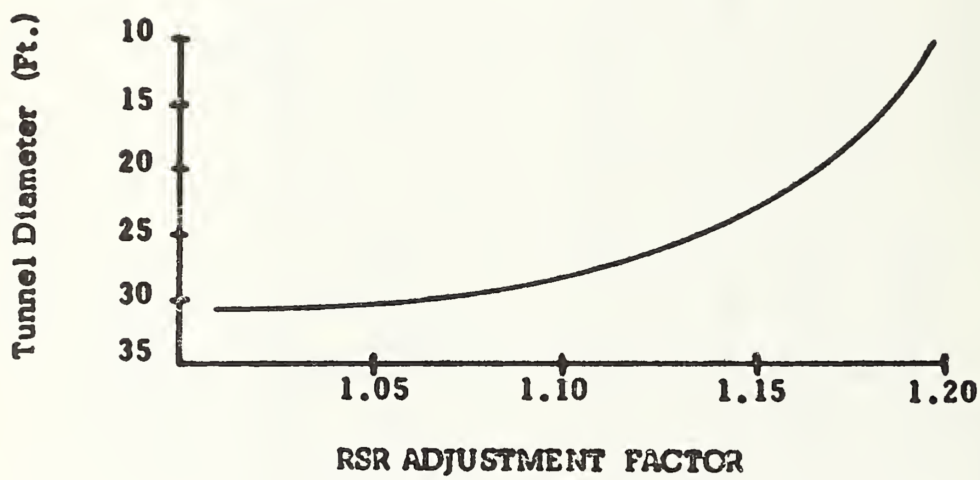


FIGURE 3.5.14 RSR ADJUSTMENT FACTOR FOR MACHINE-BORED TUNNELS (FROM WICKHAM ET AL.,1972)

$$RR = \frac{8800}{RSR + 30} - 80 \tag{Wi 1}$$

A rib ratio of 100 corresponds to a steel support which is designed for the long term load of a loose sand below the water table according to Terzaghi (1946). This rock load is

$$p = 1.38 \ (B + H) \ \gamma \tag{Wi 2}$$

B = width of tunnel

H = height of tunnel

γ = unit weight

The tunnel is assumed to be of equal width and height. The sand has a unit weight of 120 pcf. With the rock load p a load per feet of tunnel width is computed:

$$p_{(1 \text{ ft})} = 331 \ D^2 \tag{Wi 3}$$

with the rock load in Eq. Wi 3 and tables in Proctor and White (1946) the theoretical ("datum") spacing of steel sets (of various sizes) has been obtained for different sizes of tunnels (Table 3.5.11). The actual spacing of the steel sets is then determined:

$$\text{SPACING} = \text{DATUM SPACING} \times \frac{100}{RR} \tag{Wi 4}$$

Wickham, et. al. also derive an equation that directly relates RSR and rock load per unit area:

TABLE 3.5.11 REFERENCE ("DATUM") SPACING FOR A RIB RATIO = 100% (FROM WICKHAM ET AL., 1974)

	TUNNEL DIAMETER										
Rib Size	10'	12'	14'	16'	18'	20'	22'	24'	26'	28'	30'
4I7.7	1.16										
4H13.0	2.01	1.51	1.16	0.92							
6H15.5	3.19	2.37	1.81	1.42	1.14						
6H20		3.02	2.32	1.82	1.46	1.20					
6H25			2.86	2.25	1.81	1.48	1.23	1.04			
8WF31				3.24	2.61	2.14	1.78	1.51	1.29	1.11	
8WF40					3.37	2.76	2.30	1.95	1.67	1.44	1.25
8WF48						3.34	2.78	2.35	2.01	1.74	1.51
10WF49								2.59	2.22	1.91	1.67
12WF53										2.19	1.91
12WF65											2.35

$$W_r(\text{ksf}) = \frac{D(\text{ft})}{302}\left[\frac{8800}{RSR+30} - 80\right] \qquad \text{(Wi 5)}$$

where D = diameter of tunnel

which is based on the fact that a ribratio of one hundred corresponds to a rock load of 1.38 $\gamma(B+H)$ (for a complete derivation, see Wickham, et. al. 1972). Instead of Eq. Wi 5, Table 3.5.12 may be used relating RSR, W_r and diameter. It is also possible to estimate with the rockload W_r shotcrete and rockbolt supports. The design equation for bolt spacing in a square pattern is:

$$s = \sqrt{\frac{\text{Bolt Capacity}}{F \cdot W_r}} \qquad \text{(Wi 6)}$$

F = Factor of Safety

W_r = Rock Load

For shotcrete the empirical relation between rock load W_r (in ksf) and shotcrete thickness (in inches) proposed is:

$$t(\text{inches}) = 1 + \frac{W_r(\text{ksf})}{1.25} \qquad \text{(Wi 7)}$$

TABLE 3.5.12 CORRELATION OF ROCK STRUCTURE RATING TO ROCK LOAD AND TUNNEL DIAMETER (FROM WICKHAM ET AL., 1974)

TUNNEL DIAMETER (D)	(Wr) ROCK LOAD ON TUNNEL ARCH (K/sq.ft.)											
	0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
	CORRESPONDING VALUES OF ROCK STRUCTURE RATINGS (RSR)											
10'	62.5	49.9	40.2	32.7	21.6	13.8						
12'	65.0	53.7	44.7	37.5	26.6	18.7						
14'	66.9	56.6	48.3	41.4	30.8	22.9	16.8					
16'	68.3	59.0	51.2	44.7	34.4	26.6	20.4	15.5				
18'	69.5	61.0	53.7	47.6	37.6	29.9	23.8	18.8				
20'	70.4	62.5	55.7	49.9	40.2	32.7	26.6	21.6	17.4			
22'	71.3	63.9	57.5	51.9	42.7	35.3	29.3	24.3	20.1	16.4		
24'	72.0	65.0	59.0	53.7	44.7	37.5	31.5	26.6	22.3	18.7		
26'	72.6	66.1	60.3	55.3	46.7	39.6	33.8	28.8	24.6	20.9	17.7	
28'	73.0	66.9	61.5	56.6	48.3	41.4	35.7	30.8	26.6	22.9	19.7	16.8
30'	73.4	67.7	62.4	57.8	49.8	43.1	37.4	32.6	28.4	24.7	21.5	18.6

Development and Underlying Philosophy

The RSR method is the result of research performed by Jacobs Associates of San Francisco under a contract to the U.S. Bureau of Mines. A first report was published in 1972 (Wickham and Tiedemann, 1972a, 1972b), the research was extended and resulted in a modified version (Wickham and Tiedemann, 1974a,b). For the first study data from 33 tunnels were available for the second study additional tunnel data was included bringing the total number of tunnels to 53 (Table 3.5.13). Wickham, et. al. subdivided the tunnels into zones with similar ground conditions, thus leading to 134 sample sections for 1972 study and 187 sections for the 1974 report.

The tunnels considered are listed in Table 3.5.13, they vary considerably in size from 70 sq. ft. (9 x 9 ft., 2.7 x 2.7m = $8.4m^2$) to 1050 sq. ft. (36 ft. diam. ≅ 10.9m diam. ≙ $95m^2$). The development of the method required the determination of:

(1) of the Rib Ratio (RR)

(2) the Rock Structure Rating (RSR) with its 3 individual parameters A,B,C.

RR was determined for each individual case by relating the actual spacing to the "datum" spacing (RR = 100%) for the tunnel and steel set type. On the other hand the determination of A,B,C, resp. RSR was not as straight forward.

TABLE 3.5.13 DATA BASE OF THE ROCKSTRUCTURE RATING METHOD, 1974 VERSION (FROM WICKHAM ET AL., 1974) (page 1 of 3)

CASE HISTORY STUDY PROJECTS							
CASE HISTORY NO.	NAME OF TUNNEL	LOCATION	SIZE OF EXCAV. SECTS		TOTAL LENGTH L.F.	NO. OF STUDY SECTIONS	METHOD OF EXCAV.
			DIMENS.	SQ.FT.			
1	White Rock	Calif.	24x24 HS	480	24,000	2	D&B
2	Divide	Colo.	12x12 HS	130	28,000	1	D&B
3	Spring Creek No. 1	Calif.	22 Dia.	380	8,300	4	D&B
4	Spring Creek No. 2	Calif.	22 Dia.	380	4,500	3	D&B
5	Tecolote	Calif.	9x9 HS	70	33,500	12	D&B
6	Glendora	Calif.	20x20 HS	350	32,500	8	D&B
7	Canyon	Calif.	14x14 HS	180	54,000	8	D&B
8	Crystal Springs Bypass	Calif.	13x13 HS/ 13 Dia.	140	17,100	2	D&B/TBM
9	Azotea	N. Mex.	12 Dia.	110	66,000	2	TBM
10	Navajo No. 1	N. Mex.	20 Dia.	310	10,100	2	TBM
11	Navajo No. 2	N. Mex.	19x19 HS	330	25,280	2	D&B
12	Blanco	N. Mex.	11x11 HS/ 11 Dia.	90	45,600	2	D&B/TBM
13	Oso	Colo.	11x11 HS/ 11 Dia.	90	26,700	3	D&B/TBM
14	Starvation	Utah	9 Dia.	60	5,300	2	TBM
15	Water Hollow	Utah	13 Dia.	130	21,600	2	TBM
16	River Mountains	Nevada	12 Dia.	110	20,000	3	TBM
17	Clear Creek	Calif.	20x20 HS	350	56,600	3	D&B
18	Cascade Divide	Ore.	8 Dia.	50	2,100	1	D&B
19	Green Springs	Ore.	8 Dia.	50	4,800	1	D&B
20	Angeles	Calif.	34 Dia.	910	38,800	2	D&B

D&B - Drill and Blast
TBM - Tunnel Boring Machine

TABLE 3.5.13 DATA BASE OF THE ROCKSTRUCTURE RATING METHOD, 1974 VERSION (FROM WICKHAM ET AL., 1974) (page 2 of 3)

CASE HISTORY NO.	NAME OF TUNNEL	LOCATION	SIZE OF EXCAV. SECTS.		TOTAL LENGTH L.F.	NO. OF STUDY SECTIONS	METHOD OF EXCAV.
			DIMENS.	SQ.FT.			
21	Western Pacific, Nos. 1 thru 5	Calif.	22x30 HS	600	21,000	5	D&B
22	Castaic Dam Diversion	Calif.	24 H.Dia/ 33 H Dia.	400/900	3,600	2	D&B
23	Belden No. 1	Calif.	18.5 HS	310	23,600	8	D&B
24	Belden No. 2	Calif.	18.5 HS	310	9,600	5	D&B
25	Pit River No. 4	Calif.	23x22 HS	450	21,300	7	D&B
26	Poe Tunnel (*Partial)	Calif.	23x23 HS	470	*15,100	8	D&B
27	Camino	Calif.	14x15 HS	190	26,500	7	D&B
28	Loon Lake Tailrace	Calif.	18x18 HS	290	20,200	7	D&B
29	Jay Bird	Calif.	14x14 HS	180	21,000	3	D&B
30	Union Valley	Calif.	19x19 HS	320	4,500	2	D&B
31	Butt Valley	Calif.	17x16 HS	240	10,900	4	D&B
32	Caribou No. 2	Calif.	17x16 HS	240	8,700	4	D&B
33	Flathead	Mont.	22x30 HS	600	35,300	7	D&B
34	Berkeley Hills	Calif.	21x21 HS	370	16,200	9	D&B
35	Poe (partial)	Calif.	23x23 HS	470	17,600	5	D&B
36	Balboa Outlet	Calif.	16 Dia.	200	3,800	2	D&B
37	McCloud No. 1	Calif.	17x17 HS	260	11,200	5	D&B
38	McCloud No. 2	Calif.	17x17 HS	260	25,600	9	D&B
39	Lucky Peak Outlet	Idaho	23 Dia.	420	1,160	4	D&B

TABLE 3.5.13 DATA BASE OF THE ROCK STRUCTURE RATING METHOD, 1974 VERSION (FROM WICKHAM ET AL., 1974) (page 3 of 3)

CASE HISTORY NO.	NAME OF TUNNEL	LOCATION	SIZE OF EXCAV. SECTS.		TOTAL LENGTH L.F.	NO. OF STUDY SECTIONS	METHOD OF EXCAV.
			DIMENS.	SQ.FT.			
41	Cougar Dam Penstock	Oregon	20x20 HS	360	1,830	2	D&B
42	Pomme De Terre Outlet	Missouri	16 Dia.	200	432	1	D&B
43	Wilson Outlet	Kansas	18 Dia.	250	988	2	D&B
44	Danopolis Outlet	Missouri	17 Dia.	230	2,348	1	D&B
45	Littleville Outlet	Mass.	10x10 HS	90	374	1	D&B
46	N. Fork of Pound Resv.	Virginia	11 Dia.	90	690	1	D&B
47	Worcester Diversion	Mass.	21x21 HS	390	3,700	2	D&B
48	Tenkiller Ferry Outlet	Okla.	22 Dia.	380	557	1	D&B
49	Fort Randall Resv.	S. Dak.	26 Dia./ 36 Dia.	530/ 1,020	10,476	2	D&B*
50	Blue River Resv.	Oregon	24x24 HS	510	1,623	1	D&B
51	Hills Creek Sam	Oregon	27x27 HS/ 19 Dia.	650/ 250	1,150	2	D&B
52	Harold D. Roberts	Colorado	12x12 HS	130	122,900	12	D&B
53	Granduc	Brit. Col.	15x15 HS	200	54,500	4	D&B

*Kerf cut by coal saw

Several methods were used for the rating of the ground conditions and finally one method was selected. Wickham, et. al. (1972) comment:[1)]

> "As the study progressed, the original formats and assigned values were revised to more nearly reflect the data and findings of the research effort. RSR values as determined by the several methods were compared and subsequently correlated with actual ground support used in the respective tunnels. These comparisons and evaluation of results, in conjunction with other information obtained from case studies, were used in finalizing RSR method #2 which is proposed in this study."

A data pair (RR, RSR) was thus obtained for each of the tunnel sections and a regression was performed on the data resulting in the design equation.

There are differences between the 1972 and 1974 versions. The 1974 version included more data (53 tunnels instead of 33). The 1974 data required a change of the individual ratings (A,B,C) as dicussed below and of the regression equation.

The data considered in the 1972 version are shown in Fig. 3.5.15 and those of the 1974 version in Fig. 3.5.16. The regression of RR and RSR was given in 1972 by:

[1)] In the 1972 study Wickham, et. al. presented RSR method #1 and #2. Method #1 presents the initial classification attempts and included more parameters that were subsequently reduced to three (A,B,C) in method #2.

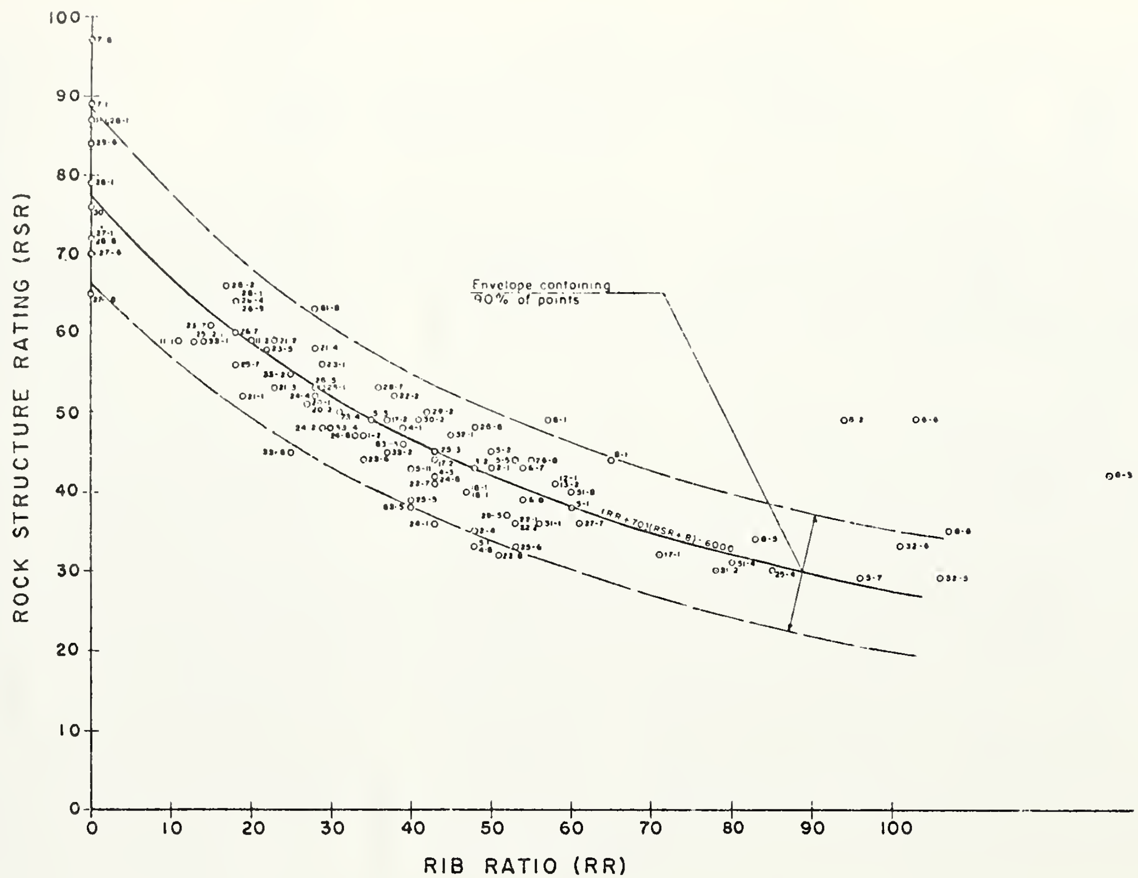


FIGURE 3.5.15 CORRELATION OF RSR AND RIBRATIO FOR 1972 VERSION
(FROM WICKHAM ET AL.,1972)

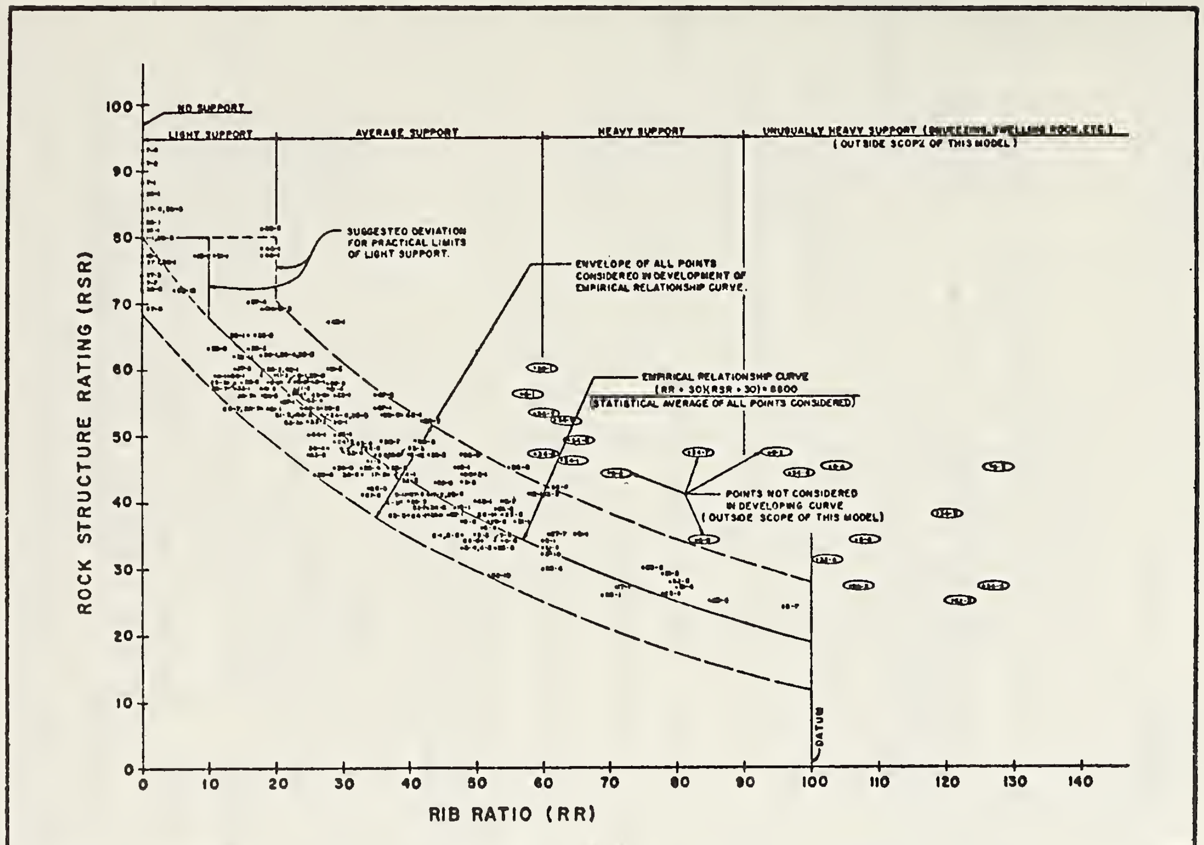


FIGURE 3.5.16 CORRELATION OF ROCK STRUCTURE RATING AND RIB RATIO FOR 1974 DATABASE (FROM WICKHAM ET AL, 1974)

$$RR = \frac{6000}{RSR + 8} - 70 \qquad \text{(Wi 8)}$$

and in 1974 by:

$$RR = \frac{8800}{RSR + 30} - 80 \qquad \text{(Wi 9)}$$

In Figs. 3.5.15 and 16 the 90% confidence band is shown for the 1972 regression resp. the envelope for the 1974 data set. Many points have been excluded from the 1974 regression analysis (Fig. 3.5.16) which are said to be outside the scope of the model. The criteria for exclusion are not stated in detail for each individual case. Some of the excluded data points are "oversupported" sections other data points represent squeezing ground conditions.

As mentioned before, the rating of the A,B,C parameters changed between the 1972 (Tables 3.5.14) and the 1974 (Tables 3.5.8 to 10) version. In particular the A - Parameter in the 1972 version considered only rock types and geologic structure. The 1974 version includes also the strength of the rock. The maximum rating of A was 30 in both versions. Parameter B in the initial version considered joint spacings from closely jointed (<0.5 ft.) to massive (>4.0 ft.) rock in the final version the range was extended very closely jointed rock (joint spacing <2 inches).

TABLE 3.5.14 RATING TABLES OF THE "1972" ROCK STRUCTURE RATING METHOD (FROM WICKHAM ET AL., 1972)

Rock Structure Rating--Parameter 'A'
General Area Geology

Basic Rock Type	Massive	Geologic Structure: Slightly Faulted or Folded	Moderately Faulted or Folded	Intensely Faulted or Folded
Igneous	30	26	15	10
Sedimentary	24	20	12	8
Metamorphic	27	22	14	9

Rock Structure Rating - Parameter 'B'
Joint Pattern-Direction of Drive

Average Joint Spacing Feet	Strike ⊥ to Axis					Strike ∥ to Axis		
	Direction of Drive							
	Both	With Dip		Against Dip		Both		
	Dip of Prominent Joints*							
	1	2	3	2	3	1	2	3
<.5 (Closely Jointed)	14	17	20	16	18	14	15	12
.5-1.0 (Moderately Jointed)	24	26	30	20	24	24	24	20
1.0-2.0 (Moderate to Blocky)	32	34	38	27	30	32	30	25
2.0-4.0 (Blocky to Massive)	40	42	44	36	39	40	37	30
>4.0 (Massive)	45	48	50	42	45	45	42	36

*1 = $<20^{\circ}$, 2 = 20°-50°, 3 = 50°-90°

Rock Structure Rating - Parameter 'C'
Ground Water, Joint Condition

Anticipated Water Inflow (gpm/1000 ft)	Sum of Parameters A + B: 20 - 45			46 - 80		
	Joint Condition *					
	1	2	3	1	2	3
None	18	15	10	20	18	14
Slight (200gpm)	17	12	7	19	15	10
Moderate (200-1000 gpm)	12	9	6	18	12	8
Heavy (1000 gpm)	8	6	5	14	10	6

*1 = tight or cemented, 2 = slightly weathered
3 = severely weathered or open.

Also the rating for the individual classes was adjusted in that the value for massive rock was reduced by 10% and the values for closely jointed rock were modified (see Table W.2 and Table 3.5.14). Parameter C was adjusted, the maximum value is now 25 instead of 20 (Table 3.5.10, 3.5.14).

The method has been tested by applying it to several case studies. Prebid information was used to predict the support which was then compared with the actually placed support. For details on this comparison the reader is referred to the report by Wickham, et. al. (1974).

Critique of the RSR - Method

Wickham, et. al. developed their method based on case studies of mainly steel supported tunnels: it may thus not be representative for other types of tunnel support.

The RSR - system allows one to seperate the determination of ground parameters into three distinct steps. The sum of these three ground parameters is the Rock Structure Rating RSR which is related to a rock load or the rib ratio (a measure for steel support).

Assessment of the parameters A,B,C: Parameter A requires a knowledgy of geology. Parameters B and C are measurable, however, difficulties arise if there is more than one joint set; in this case the user has to select the appropriate (average?) value for the parameters B and C (considering the fact that there are multiple joint sets).

The base cases are primarily steel supported tunnels, thus limiting the applicability. Furthermore, these tunnels were probably conservatively designed using the upper levels of Terzaghi's recommendations.

In developing their method, in particular the regression between RSR and RR, Wickham, et. al. exclude squeezing ground conditions and 'oversupported' tunnels. The detailed reasons for exclusion are not given. Also, if one is using the Rock Structure Rating Method as a predictive tool one cannot judge whether the actual ground conditions will be squeezing since there is no criterion given.

Summary of the RSR - Method

The RSR - Method rates ground conditions with three parameters. The sum of these three parameters (A rates general geology, B rates the joint pattern, C rates water inflow and conditions of discontinuities) is the Rock Structure Rating RSR. Support quantities are related to RSR by an equation which gives either a rock load or a rib ratio, a measure for steel set spacing.

The method is easily applicable, some experience may, however, be required for assessing parameter A. Problems may arise if there is more than one joint set since the method only considers one set explicitely in parameters B and C. The method excludes squeezing ground but no criterion is given on how to do this.

3.5.4 Bieniawski's Geomechanics Classification

Methodology and Application Guidelines

In the Geomechanics Classification procedure, ground conditions are described by the six key parameters (Table 3.5.15 A,B):

- strength of intact rock
- the Rock Quality Designation (RQD)
- spacing of joints
- orientation of joints
- condition of joints
- groundwater inflow.

Each of the six parameters is rated on a different numerical scale (Table 3.5.15). The sum of the six individual ratings (each individual rating will be discussed after the general description) yields the Rock Mass Rating, RMR with numerical values between 0 and 100. The particular RMR value is then used with the chart in figure 3.5.17a to derive the stand-up time of an unsupported span. Fig. 3.5.17a and Section D of Table 3.5.15 show also that five ground classes can be assigned to RMR ranges. For each ground class excavation and support procedures are specified as described in Table 3.5.16.

TABLE 3.5.15 GEOMECHANICS CLASSIFICATION, 1979 VERSION RATING TABLES (FROM BIENIAWSKI, 1979)

GEOMECHANICS CLASSIFICATION OF JOINTED ROCK MASSES

A. CLASS...:ATION PARAMETERS AND THEIR RATINGS

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

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

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

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

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

D. MEANING OF ROCK MASS CLASSES

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

Table 2.

The Effect of Discontinuity Strike and Dip Orientations in Tunneling.

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

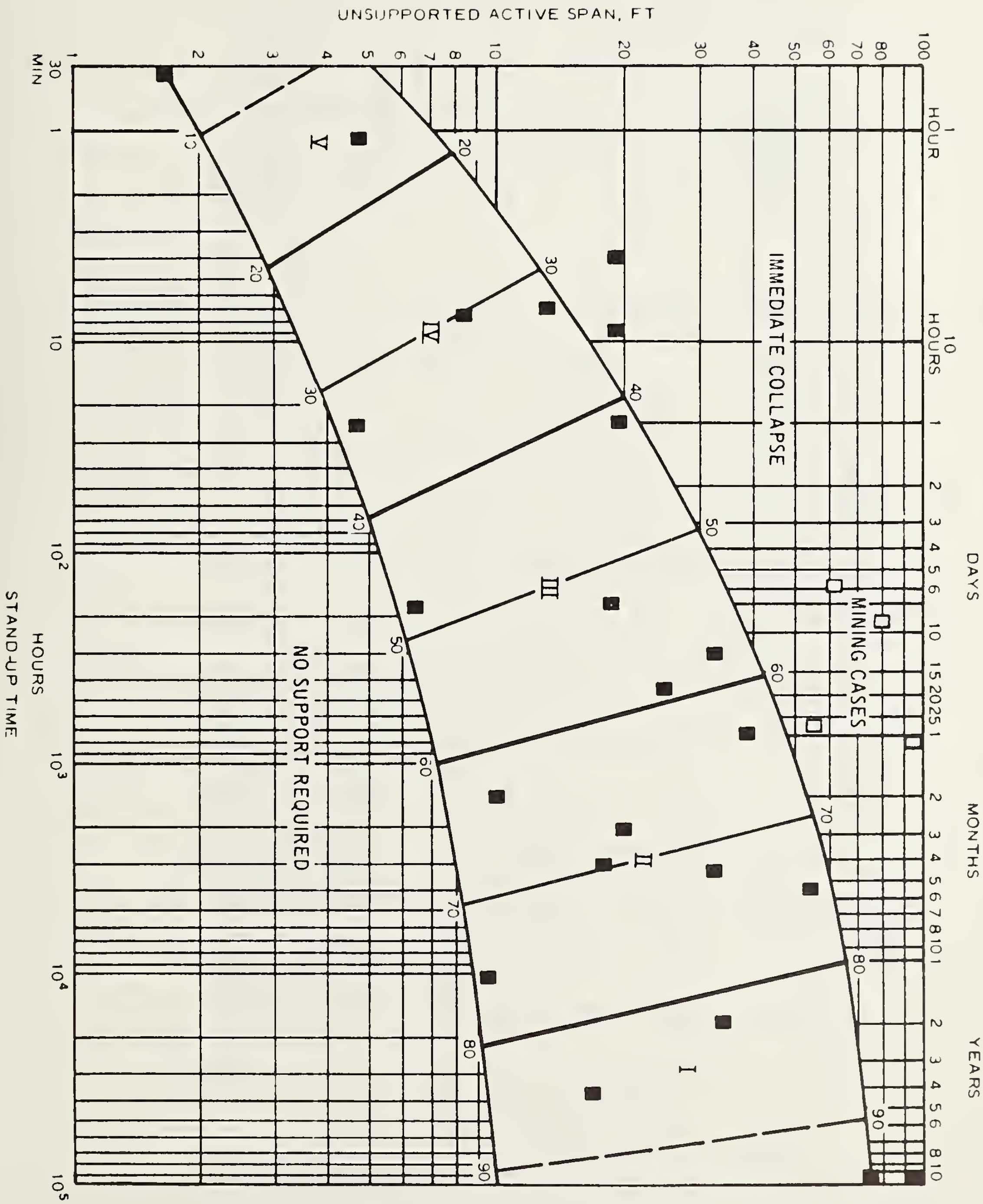


FIGURE 3.5.17a RELATION BETWEEN RMR AND STAND-UP TIME FOR UNSUPPORTED SPAN

TABLE 3.5.16 GEOMECHANICS CLASSIFICATION, SUPPORT-EXCAVATION RECOMMENDATION, 1979 VERSION (FROM BIENIAWSKI, 1979)

Geomechanics Classification Guide for Excavation and Support in Rock Tunnels.

SHAPE: HORSESHOE; WIDTH: 10 m; VERTICAL STRESS: BELOW 25 MPa; CONSTRUCTION: DRILLING AND BLASTING

Rock mass class	Excavation	Support		
		Rockbolts (20 mm dia., fully bonded)	Shotcrete	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 m - 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 walls 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. Instal 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.

(1) <u>Uniaxial strength</u>

Either the uniaxial strength or the point load index are determined, and rated on the scale in Table 3.5.15.A.1. The maximum rating is 15 for an unconfined compressive strength greater than 250 MPa (2000 kg/cm^2 = 28900 psi).

(2) <u>RQD</u>

RQD is assessed on a scale from 3 (RQD < 25%) to 20 (RQD = 90 to 100%).

(3) <u>Spacing of joints</u>

The third step involves the assessment of the joint spacing. The maximum rating is 30 for joints spaced more than 300mm, the minimum is 5 for joints spaced less than 50mm. Basically the rating has been developed for three joint sets. It may be advisable to use the chart in Fig. 3.5.17b to determine the rating for spacing in 'multi-joint systems'. This chart can be used for up to three joint sets, in the manner indicated on the chart; for more than three joint sets the three closest spaced sets are used.

(4) <u>Condition of joints</u>

The highest rating (30) is given to non-continuous joints in hard intact rock, the min-

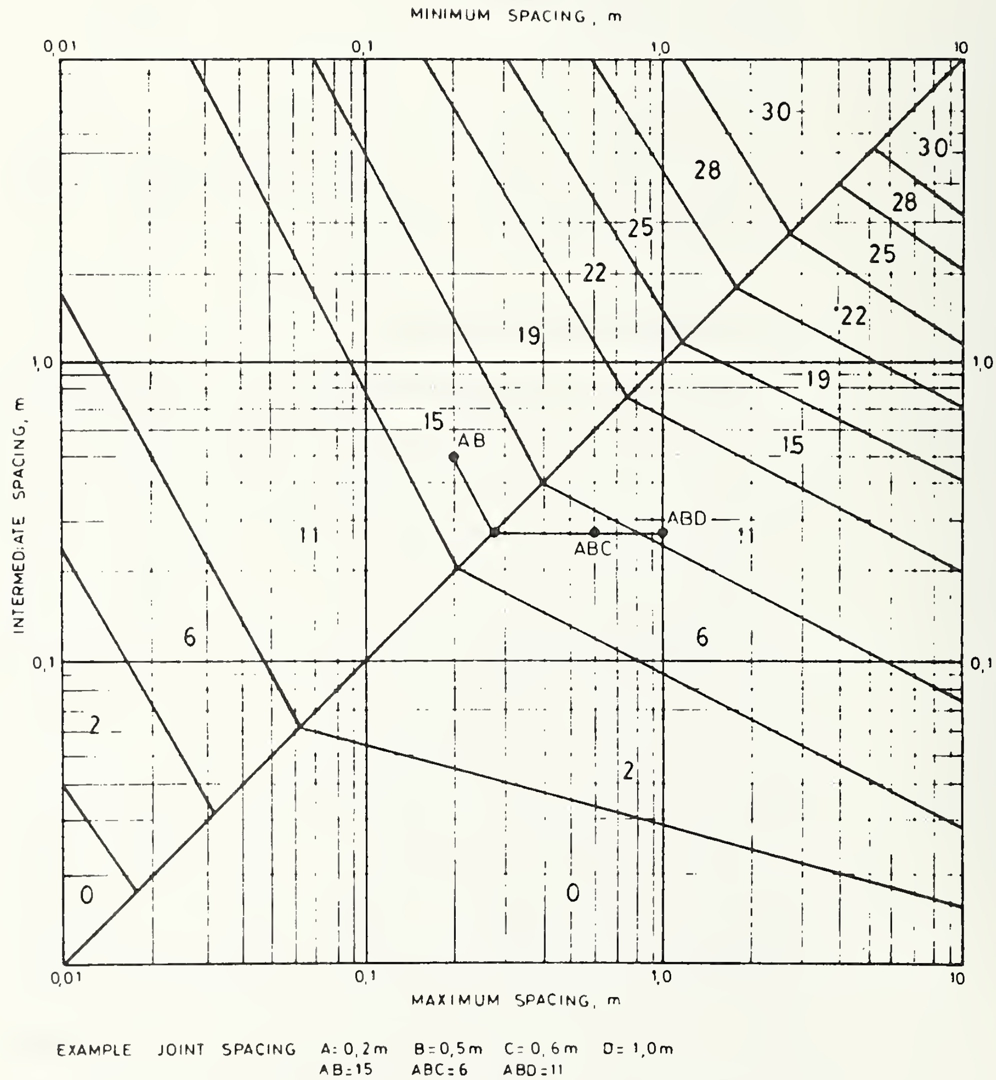


FIGURE 3.5.17b RATINGS FOR MULTIJOINT SYSTEM (FROM LAUBSCHER AND TAYLOR, 1976)

imum (0) to continuous joints with more than 5mm thickness with or without gouge.

(5) Water conditions

A completely dry tunnel is assessed with 15 whereas heavy water inflow and high water pressures with respect to major principal stress are assessed at zero (Table 3.5.15.A.5).

(6) Joint orientation

The different types of joint orientation and their ratings are shown in Table 3.5.15B. A favorable orientation is rated 0, a very unfavorable orientation - 12.

The sum of these six ratings yields the Rock Mass Rating (RMR) with which standup time for unsupported spans can be derived with Fig. 3.5.17a and with which the support and excavation procedure can be determined in Table 3.5.16. Note also that the chart in Fig. 3.5.17a contains case study data and thus an indication of the limits of application of the method.

Bieniawski (e.g. 1979) also recommends that the Geomechanics Classification should be used in conjunction with monitoring of tunnel performance.

Development and Underlying Philosophy

The Geomechanics Classification System was developed in five stages. The first version was published in the 'South African Civil Engineer' (Bieniawski, 1973) (Tables 3.5.17 and 18), followed by a second version in the Proceedings of the International Conference on Rock Mechanics in 1974 (Bieniawski, 1974a), (Tables 3.15.19 and 20). A third version appeared in the N.Z. - Australian Geomechanics Conference in 1975 (Bieniawski, 1975) (Tables 3.5.20 and 21). The rating table of the third version was actually originally published in the reply to the discussion of the first version in the South African Civil Engineer (Bieniawski, 1974b), (in this review the third version will be called the '1975' version). A fourth version was published in 1976 (Table 3.5.22) and finally a fifth one (Bieniawski, 1979).

The rating scales and some of the parameters considered have changed from version to version. These changes will be discussed as far as possible, since the different versions do not contain detailed reasons for the changes. One can assume however that with increased experience and more cases to which the method was applied, modifications became possible and necessary, a fact that has been confirmed by Bieniawski in his review comments.

Bieniawski (1973) characterizes the Geomechanics Classification System in the synopsis to have the following features:.....

TABLE 3.5.17 GEOMECHANICS CLASSIFICATION: 1973 RATING TABLES (FROM BIENIAWSKI, 1973)

Geomechanics Classification of jointed rock masses

Item	Class No and its description	1	2	3	4	5
		Very good	Good	Fair	Poor	Very poor
1	Rock quality RQD(%)	90–100	75–90	50–75	25–50	<25
2	Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Completely weathered
3	Intact rock strength, MPa	>200	100–200	50–100	25–50	<25
4	Spacing of joints	>3m	1–3m	0,3–1m	50–300mm	<50mm
5	Separation of joints	<0,1mm	<0,1mm	0,1–1mm	1–5mm	>5mm
6	Continuity of joints	Not continuous	Not continuous	Continuous no gouge	Continuous with gouge	Continuous with gouge
7	Ground water inflow (per 10m of adit)	None	None	Slight <25 litres/min	Moderate 25–125 litres/min	Heavy >125 litres/min
8	Strike and dip orientations	Very favourable	Favourable	Fair	Unfavourable	Very unfavourable

Importance ratings

A. INDIVIDUAL RATINGS FOR CLASSIFICATION PARAMETERS

Item	Parameter		Class 1	2	3	4	5
1	Rock quality RQD		16	14	12	7	3
2	Weathering		9	7	5	3	1
3	Intact rock strength		10	5	2	1	0
4	Spacing of joints		30	25	20	10	5
5	Separation of joints		5	5	4	3	1
6	Continuity of joints		5	5	3	0	0
7	Ground water		10	10	8	5	2
8	Strike and dip orientations	Tunnels	15	13	10	5	3
		Foundations	15	13	10	0	−10

B. TOTAL RATINGS FOR ROCK MASS CLASSES

Class No	1	2	3	4	5
Description of Class	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
Total rating	100 ← 90	90 ← 70	70 — 50	50 ← 25	< 25

TABLE 3.5.18 GEOMECHANICS CLASSIFICATION: 1973 EXCAVATION-SUPPORT RECOMMENDATIONS (FROM BIENIAWSKI, 1973)

Geomechanics Classification: Guidelines for selection of primary tunnel support
Tunnel sizes: 5-12 m: Construction by drilling and blasting

Rock mass class	Average stand-up time at unsupported span	ALTERNATIVE SUPPORT SYSTEMS						
		ROCKBOLTS*		SHOTCRETE			STEEL SETS	
		Spacing	Additional support	Crown	Sides	Additional support	Type	Spacing
1	10 years 5 m	GENERALLY NOT REQUIRED						
2	6 months 4 m	1,5–2,0 m	Occasional wire mesh in crown	50 mm	Nil	Nil	Uneconomic	
3	1 week 3 m	1,0–1,5 m	Wire mesh, plus 30 mm shotcrete in crown as required	100 mm	50 mm	Occasional wire mesh and rock bolts, if necessary	Light sets	1,5–2,0 m
4	5 hours 1,5 m	0,5–1,0 m	Wire mesh, plus 30–50 mm shotcrete in crown and sides	150 mm	100 mm	Wire mesh and 3 m rockbolts at 1,5 m spacing	Medium sets plus 50 mm shotcrete	0,7–1,5 m
5	10 min 0,5 m	Not recommended		200 mm	150 mm	Wire mesh, rockbolts and light steel sets. Seal face. Close invert.	Heavy sets with lagging, immediately 80 mm shotcrete	0,7 m

*Bolt diameter 25 mm, length ½ tunnel width. Resin bonded fully.

"1. Best aspects of previously used classification systems are incorporated.

2. It is based on properties of rock materials and rock masses.

3. It is functional (it can be applied to solutions of practical engineering problems).

4. Standard terms are employed.

5. A rating system is provided to weigh the relative importance of various classification parameters."

In the 1973 version Bieniawski uses 8 parameters (Table 3.5.17) to rate ground conditions, this was reduced to six parameters in the 1974 version, through consolidation of parameters "seperation", "continuity" of

TABLE 3.5.19 GEOMECHANICS CLASSIFICATION, 1974 VERSION (FROM BIENIAWSKI, 1974)

TABLE 3 : GEOMECHANICS CLASSIFICATION OF ROCK MASSES

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

1	Uniaxial compressive strength of intact rock	> 200 MPa	100 - 200 MPa	50 - 100 MPa	25 - 50 MPa	< 25 MPa
	Rating	10	5	2	1	0
2	Drill core quality RQD	90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25% or highly weathered
	Rating	20	17	14	8	3
3	Spacing of joints	> 3 m	1 - 3 m	0,3 - 1 m	50 - 300 mm	< 50 mm
	Rating	30	25	20	10	5
4	Strike and dip orientations of joints	Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
	Rating	15	13	10	6	3
5	Condition of joints	Very tight: separation < 0,1 mm Not continuous		Tight: < 1 mm and continuous No gouge	Open: 1 - 5 mm Continuous Gouge < 5 mm	Open' > 5 mm Continuous Gouge > 5 mm
	Rating	15		10	5	0
6	Ground water inflow (per 10 m of tunnel length)	None		< 25 litres/min	25 - 125 litres/min	> 125 litres/min
	Rating	10		8	5	2

B. ROCK MASS CLASSES AND THEIR RATINGS

Class No.	I	II	III	IV	V
Description of class	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
Total rating	100 ← 90	90 ← 70	70 ← 50	50 ← 25	< 25

C. MEANING OF ROCK MASS CLASSES IN TUNNELLING

Class No.	I	II	III	IV	V
Unsupported span	5 m	4 m	3 m	1,5 m	0,5 m
Average stand-up time	10 years	6 months	1 week	5 hours	10 minutes

TABLE 3.5.19 GEOMECHANICS CLASSIFICATION: 1974 VERSION (FROM BIENIAWSKI, 1974) (CONT.)

: THE EFFECT OF JOINT STRIKE AND DIP ORIENTATIONS IN TUNNELLING

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

GUIDE FOR SELECTION OF PRIMARY SUPPORT IN 5 m to 12 m DIAMETER TUNNELS AT SHALLOW DEPTH

Rock mass class	Alternative support systems for drilling and blasting construction		
	Mainly ROCKBOLTS*	Mainly SHOTCRETE	Mainly STEEL RIBS
I	GENERALLY NO SUPPORT IS REQUIRED		
II	Rockbolts spaced 1,5 to 2,0 m plus occasional wire mesh in crown	Shotcrete 50 mm in crown	Uneconomic
III	Rockbolts spaced 1,0 to 1,5 m plus wire mesh and 30 mm shotcrete in crown where required	Shotcrete 100 mm in crown and 50 mm in sides plus occasional wire mesh and rockbolts where required	Light sets spaced 1,5 m to 2 m
IV	Rockbolts spaced 0,5 to 1,0 m plus wire mesh and 30 - 50 mm shotcrete in crown and sides	Shotcrete 150 mm in crown and 100 mm in sides plus wire mesh and rockbolts, 3 m long spaced 1,5 m	Medium sets spaced 0,7 m to 1,5 m plus 50 mm shotcrete in crown
V	Not recommended	Shotcrete 200 mm in crown and 150 mm in sides plus wire mesh, rockbolts and light steel sets. Close invert.	Heavy sets spaced 0,7 m with lagging. Shotcrete 75 mm as soon as possible.

* Resin bonded bolts 20 mm diameter, length ½ tunnel width.

TABLE 3.5.20 GEOMECHANICS CLASSIFICATION: RATING TABLES OF THE 1975 VERSION (BIENIAWSKI, 1975)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

1	Strength of intact rock material	Point-load strength index	> 8 MPa	4 - 8 MPa	2 - 4 MPa	1 - 2 MPa	Use of uniaxial compressive test preferred		
		Uniaxial compressive strength	> 200 MPa	100 - 200 MPa	50 - 100 MPa	25 - 50 MPa	10-25 MPa	3-10 MPa	1-3 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		> 3 m	1 - 3 m	0,3 - 1 m	50 - 300 mm	< 50 mm		
	Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces. Not continuous No Separation Hard joint wall rock.	Slightly rough surfaces. Separation < 1 mm Hard joint wall rock	Slightly rough surfaces. Separation < 1 mm Soft joint wall rock	Slickensided surfaces OR Gouge < 5 mm thick OR Joints open 1-5 mm. Continuous joints	Soft gouge > 5 mm thick OR Joints open > 5 mm Continuous joints		
	Rating		25	20	12	6	0		
5	Ground water	Inflow per 10m tunnel length	None		< 25 litres/min	25 - 125 litres/min	> 125 litres/min		
		OR Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0		0,0 - 0,2	0,2 - 0,5	> 0,5		
		OR General conditions	Completely dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Rating		10		7	4	0		

B. ADJUSTMENT FOR JOINT ORIENTATIONS

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

C. ROCK MASS CLASSES AND THEIR RATINGS

Class No	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
Rating	100 ← 90	90 ← 70	70 ← 50	50 ← 25	< 25

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 5 m span	6 months for 4 m span	1 week for 3 m span	5 hours for 1.5 m span	10 minutes for 0.5 m span
Cohesion of the rock mass	> 300 kPa	200 - 300 kPa	150 - 200 kPa	100 - 150 kPa	< 100 kPa
Friction angle of the rock mass	45°	40° - 45°	35° - 40°	30° - 35°	< 30°
Caveability of ore	Very poor	Will not cave readily Large fragments	Fair	Will cave readily Good fragmentation	Very good

THE EFFECT OF JOINT STRIKE AND DIP ORIENTATIONS IN TUNNELLING

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

TABLE 3.5.21 GEOMECHANICS CLASSIFICATION: EXCAVATION AND SUPPORT RECOMMENDATIONS, 1975 VERSION (FROM BIENIAWSKI, 1975)

GUIDE FOR SELECTION OF PRIMARY SUPPORT IN HORSESHOE-SHAPED TUNNELS

WIDTH: 5 - 12 m, VERTICAL STRESS BELOW 30 MPa, CONSTRUCTION BY DRILLING AND BLASTING

Rock mass class	Excavation	Primary support		
		Rockbolts* (length for tunnel of 10 m width)	Shotcrete	Steel sets
I	Full face 3 m advance	Generally no support required except for occasional spot bolting		
II	Full face 1,0-1,5 m advance	Locally bolts in crown 2-3 m long, spaced 2-2,5 m with occasional wire mesh. Complete 20 m from face.	50 mm in crown as basis for waterproof	None
III	Top heading and bench 1,5-3 m advances in top heading	Systematic bolts 3-4 m long, spaced 1,5-2 m in crown and walls with wire mesh in crown. Complete 10 m from face.	50-100 mm in crown and 30 mm in sides	None
IV	Top heading and bench 1,0-1,5 m advance in top heading	Systematic bolts 4-5 m long, spaced 1-1,5 m in crown and walls with wire mesh. Complete 10 m from face	100-150 mm in crown and 100 mm in sides. Support to be installed as excavation proceeds	Occasional light ribs spaced 1,5 m where required
V	Multiple drifts 0,5-1 m advance in top heading	Systematic bolts 5-6 m long, spaced 1-1,5 m in crown and walls with wire mesh. Bolt invert. Complete 5 m from face	150-200 mm in crown, 150 mm in sides and 50 mm on face. Apply shotcrete as soon as possible after blasting	Heavy ribs spaced 0,75 m with steel lagging. Close invert

* 20 mm diameter, fully resin bonded, length ½ tunnel width

joints and "weathering" into a single parameter "condition of joints". In addition the rating of individual parameters has changed e.g., the rating of the maximum RQD (=90 to 100%) has increased from 16 to 20. Support recommendations for the 1973 and 1974 versions are identical and very similar to Lauffer's (1960) (compare Appendix B) recommendations. The support recommendations in later versions are very similar to the ground classes used in the Tauern tunnel.

The 1973 version triggered discussions by several individuals and groups. The discussions and Bieniawski's reply have been published in the July 1974 issue of the Civil

TABLE 3.5.22 GEOMECHANICS CLASSIFICATION: RATING TABLES 1976 VERSION (FROM BIENIAWSKI, 1976)

GEOMECHANICS CLASSIFICATION OF JOINTED ROCK MASSES

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point-load strength index	> 8 MPa	4 - 8 MPa	2 - 4 MPa	1 - 2 MPa	For this low range — uniaxial compressive test is preferred		
		Uniaxial compressive strength	> 200 MPa	100 - 200 MPa	50 - 100 MPa	25 - 50 MPa	10-25 MPa	3-10 MPa	1-3 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		> 3 m	1 - 3 m	0.3 - 1 m	50 - 300 mm	< 50 mm		
	Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces. Not continuous. No separation. Hard joint wall rock	Slightly rough surfaces. Separation < 1 mm. Hard joint wall rock	Slightly rough surfaces. Separation < 1 mm. Soft joint wall rock	Slickensided surfaces OR Gouge < 5 mm thick OR Joints open 1-5 mm. Continuous joints	Soft gouge > 5 mm thick OR Joints open > 5 mm. Continuous joints		
	Rating		25	20	12	6	0		
5	Ground water	Inflow per 10m tunnel length	None		< 25 litres/min	25 - 125 litres/min	> 125 litres/min		
		Ratio joint water pressure / major principal stress	OR 0		OR 0.0 - 0.2	OR 0.2 - 0.5	OR > 0.5		
		General conditions	OR Completely dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Rating		10		7	4	0		

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

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

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

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

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 5 m span	6 months for 4 m span	1 week for 3 m span	5 hours for 1.5 m span	10 minutes for 0.5 m span
Cohesion of the rock mass	> 300 kPa	200 - 300 kPa	150 - 200 kPa	100 - 150 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	40° - 45°	35° - 40°	30° - 35°	< 30°

TABLE III

THE EFFECT OF JOINT STRIKE AND DIP ORIENTATIONS IN TUNNELLING

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

Engineer in South Africa and seems to have led to the development of the 1975 version; the rating table was initially published in the reply by Bieniawski. This rating table incorporates changes that were proposed by the discussers. One of the most important changes is the rating of joint orientation. In the 1973, 1974 version favorable joint orientation was rated with a positive value (maximum = 15, minimum = 0). In the 1975 version, the favorable orientation was rated at zero, whereas the most unfavorable value received a negative rating (= -12 for Tunnels). Also, as mentioned ear lier the support - excavation procedure has been changed in the 1975 version. The new recommendations appear to be very similar to those used in the New Austrian Tunneling Method. These support requirements are now combinations of shotcrete, steel set and bolt support, in contrast to the earlier versions (based on Lauffer, 1960) that gave alternate support systems.

From the 1975 to 1976 version the class boundaries on the RMR (Rock Mass Rating) scale have changed. In the old version ground class V was assigned for RMR < 25, class IV for RMR = 25-50, class III for RMR = 50-70, class II for RMR = 70-90 and class I for RMR = 90-100. In the new version the classes are spaced evenly on the RMR scale with the boundaries at multiples of 20.

Recently, Bieniawski (1979) has published a fifth version of the Geomechanics Classification which makes it compatible with the suggested methods for rock mass description published by the International Society for Rock

Mechanics. There are, however, additional changes, in particular, the meaning of rock classes has been modified:

Stand-up time and span have been increased for all five classes (the actual support recommendation, remain, however, the same as in the 1976 version). At the same time the strength properties of the 'rock mass' were changed: in class I the cohesion was increased, the friction angle remained the same. In class V cohesion remained the same but the friction angle was reduced by a factor of two. In addition, Bieniawski (1979) presents correlations between RMR and the rock mass modulus E.

Critique

Bieniawski has developed his method in five steps. The revisions that led to these changes are not reported in detail. Some base cases are plotted in Fig. 3.5.17a and indicate the limits of applicability; however details on these cases are not given.

The rating of the joints (spacing and condition) does apply to rock masses having three joint sets. This restriction can be eliminated, however, by using the chart in Fig. 3.5.17b (as recommended by Bieniawski in his review comments) for cases with fewer joint sets.

As indicated above, the limit of applicability of the method is not fully known. The changes, however, suggest that method is not generally applicable. Also, in section 3.6 of this report several methods including Bieniawski's (1976) version have been applied to actually built cases.

Several of these cases were reported by Cecil (1970, 1975), a reference which was cited by Bieniawski in the publication of the first (1973) version. Some of Cecil's cases were actually unsupported, while, when applying Bieniawski's 1976 method considerable support was predicted.

Summary of the Geomechanics Classification

The method was developed in five stages and seems to be based on approaches by Deere (1969), Wickham, et. al. (1972) and Lauffer (1958, 1960) as well as information on NATM cases. Evidently the method was modified as more information became available.

The method considers six parameters:

- strength of intact rock
- rock quality designation
- attitude of discontinuities
- spacing of discontinuities
- condition of discontinuities
- water inflow

Each condition is rated on an individual scale. The sum of the six parameter ratings values yields the so-called Rock Mass Rating (RMR) which in turn is related to one of five ground classes. Each ground class is associated to a support and construction procedures, somewhat similar to the NATM classes. In addition Bieniawski strongly recommends to monitor tunnel performance in conjunction with the application of the Geomechanics Classification.

Details of the base cases for the method are not given thus the limit of applicability is known to some extent only.

The method does not require much user experience. The six parameters can be easily determined either during preliminary investigations or during construction.

Review Comments by Z.T. Bieniawski

In his review comments Bieniawski provided information on incorrect statements, he also expressed differences in opinion and gave general comments on empirical methods. Corrections have been directly made in the preceding text, his difference in opinion and general comments are given below.

1) Discrepancy of Opinion.

Changes Between the Five Versions: The reasons for the changes introduced to the Geomechanics Classification in my five versions were indeed increased experience and more case histories to which the method was applied plus my belief that rock mass classifications should be flexible enough to allow modifications once our knowledge of the rock mass behavior increases.

Changes of Class Boundaries: I have made the change of the RMR class boundaries at multiples of 20, as a result of my work with Laubscher on applying the Geomechanics Classification to mining (an extensive study) and after my visits to Australia, New Zealand, Norway and USA where I tested a number of case histories.

Robustness of Method (please note that this comment applies to Section   My many case studies show that for the same rock mass two experienced persons working independently can arrive at an RMR differing by no more than about 10.

2) Comments on Empirical Methods in General.

- I agree with all your five conclusions. In my case, I always use both the Geomechanics Classification and the Q-System. It may interest you that when I developed my classification I was not aware of Barton's work and he of mine. I started my classification research in 1972 during my visiting professorship with Leopold Miller at Karlsruhe University.

- I like very much the idea of combining the empirical (classification) methods with the observational approach. You may be interested to know that I have made some attempts in this respect in 1975 by monitoring the construction of the Overaal Tunnel which is described in the paper, "Monitoring the Behaviour of Rock Tunnels During Construction" by Bieniawski and Maschek (Civil Engineer in South Africa, Vol. 17, No. 10 - please see Figure 5). Since then, I have consulted on many tunneling projects including large underground chambers and, where possible, always insisted on supplementing rock mass classifications by monitoring.

● My latest thoughts on rock mass classifications are summarized in the paper which was presented to the annual meeting of the Transportation Research Board in Washington, D.C., 1980.

3.5.5 Barton's et al. Q-System

Methodology and Application Guidelines

The Q-System for ground classification has been developed by Barton, Lien and Lunde at the Norwegian Geotechnical Institute. (Barton, et. al., 1974; Barton, et. al., 1975; Barton, 1976) The system considers six parameters:

RQD = Rock Quality Designation

J_n = Joint Set Number

J_r = Joint Roughness Number

J_a = Joint Alternation Number

J_w = Joint Water Reduction Factor

SRF = Stress Reduction Factor

The six parameters are determined either directly (RQD) or with Tables 3.5.23 which give guidelines on how the numerical values of the parameters should be selected. These six parameters are combined into a composite index

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \qquad \text{(Ba 1)}$$

which represents rock quality.

In addition an equivalent dimension has to be obtained by dividing the width of the opening (for roof support) or the height (for wall support) by the excavation support ratio ESR. (Table 3.5.24)

TABLE 3.5.23 RATING TABLES FOR THE Q-SYSTEM (FROM BARTON ET AL., 1974) (page 1 of 4)

Descriptions and Ratings for the Parameters *RQD*, J_n, and J_r

1.	ROCK QUALITY DESIGNATION	(*RQD*)	
A.	Very poor	0—25	Note:
B.	Poor	25—50	(i) Where *RQD* is reported or measured as $\leqq 10$ (including 0) a nominal value of 10 is used to evaluate *Q* in Eq. (1)
C.	Fair	50—75	
D.	Good	75—90	
E.	Excellent	90—100	
			(ii) *RQD* intervals of 5, i. e. 100, 95, 90, etc. are sufficiently accurate
2.	JOINT SET NUMBER	(J_n)	
A.	Massive, no or few joints	0.5—1.0	
B.	One joint set	2	
C.	One joint set plus random	3	
D.	Two joint sets	4	
E.	Two joint sets plus random	6	
F.	Three joint sets	9	
G.	Three joint sets plus random	12	Note:
H.	Four or more joint sets, random, heavily jointed, "sugar cube", etc.	15	(i) For intersections use ($3.0 \times J_n$)
J.	Crushed rock, earthlike	20	(ii) For portals use ($2.0 \times J_n$)
3.	JOINT ROUGHNESS NUMBER	(J_r)	
	(a) *Rock wall contact* and (b) *Rock wall contact before 10 cms shear*		
A.	Discontinuous joints	4	Note:
B.	Rough or irregular, undulating	3	(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
C.	Smooth, undulating	2	
D.	Slickensided, undulating	1.5	
E.	Rough or irregular, planar	1.5	
F.	Smooth, planar	1.0	(ii) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are favourably orientated
G.	Slickensided, planar	0.5	
	(c) *No rock wall contact when sheared*		
H.	Zone containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	
J.	Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	

Descriptions and Ratings for the Parameters J_a and J_w

4.	JOINT ALTERATION NUMBER	(J_a)	φ_r (approx.)	
	(a) *Rock wall contact*			
A.	Tightly healed, hard, non-softening, impermeable filling i. e. quartz or epidote	0.75	(—)	Note: (i) Values of $(\varphi)_r$ are intended as an approximate guide to the mineralogical properties of the alteration products, if present
B.	Unaltered joint walls, surface staining only	1.0	(25^0—35^0)	
C.	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	(25^0—30^0)	
D.	Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20^0—25^0)	

TABLE 3.5.23 RATING TABLES FOR THE Q-SYSTEM (FROM BARTON ET AL., 1974) (page 2 of 4)

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 cms shear*		
F.	Sandy particles, clay-free disintegrated rock etc.	4.0	(25°—30°)
G.	Strongly over-consolidated, non-softening 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, < 5 mm in thickness). Value of J_a depends on percent of swelling clay-size particles, and access to water etc.	8.0—12.0	(6°—12°)
	(c) *No rock wall contact when sheared*		
K, L, M.	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6.0, 8.0 or 8.0—12.0	(6°—24°)
N.	Zones or bands of silty- or sandy clay, small clay fraction (non-softening)	5.0	
O, P, R.	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	10.0, 13.0 or 13.0—20.0	(6°—24°)

5.	JOINT WATER REDUCTION FACTOR	(J_w)	Approx. water pressure (kg/cm²)	
A.	Dry excavations or minor inflow, i. e. < 5 l/min. locally	1.0	< 1	Note:
B.	Medium inflow or pressure occasional outwash of joint fillings	0.66	1.0— 2.5	(i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5—10.0	(ii) Special problems caused by ice formation are not considered
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	

TABLE 3.5.23 RATING TABLES FOR THE Q-SYSTEM (FROM BARTON ET AL., 1974) (page 3 of 4)

Descriptions and Ratings for the Parameter *SRF*

6.	STRESS REDUCTION FACTOR			(*SRF*)
	(a) *Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated*			
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10.0
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*	σ_c/σ_1	σ_t/σ_1	
H.	Low stress, near surface	> 200	> 13	2.5
J.	Medium stress	200—10	13—0.66	1.0
K.	High stress, very tight structure (Usually favourable to stability, may be unfavourable to wall stability)	10—5	0.66—0.33	0.5—2.0
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
	(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

Note:

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

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

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

Notes on the Use of Pages 1, 2 and 3

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in pages 1, 2 and 3:

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

$$RQD = 115 - 3.3J_v \text{ (approx)} \tag{2}$$

where

J_v = total number of joints per m^3 (RQD = 100 for $J_v < 4.5$)

2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistocity, slatey cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in bore core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n in Table 1.

3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in a given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favorably orientated for stability, then a second, less favourably orientated joint set or discontinuity may sometimes be of more significance, and its higher value of (J_r/J_a) should be used when evaluating Q.

4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated (Pages 3, 6a). In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength (Pages 3, 6b). A strongly anisotropic stress field is unfavourable to stability and is roughly accounted for in note (ii), Page 3.

5. In general the compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the direction that is unfavorable for stability. This is especially important in the case of strongly anisotropic rocks. In addition, the test samples should be saturated if this condition is appropriate to present or future in situ conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

TABLE 3.5.24 EXCAVATION SUPPORT RATIO (FROM BARTON ET AL., 1974)

The Excavation Support Ratio (*ESR*) Appropriate to a Variety of Underground Excavations

	Type of excavation	*ESR*	No. of cases
A.	Temporary mine openings etc.	ca. 3—5?	(2)
B.	Vertical shafts: (i) circular section	ca. 2.5?	(0)
	(ii) rectangular/square section	ca. 2.0?	(0)
C.	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), pilot tunnels, drifts and headings for large excavations etc.	1.6	(83)
D.	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels, etc. (cylindrical caverns?)	1.3	(25)
E.	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections etc.	1.0	(79)
F.	Underground nuclear power stations, railway stations, sports and public facilities, factories etc.	ca. 0.8?	(2)

$$\text{Equivalent Dimension} = \frac{\text{Width (or Height)}}{\text{ESR}}$$

ESR is a function of the purpose of the opening, large caverns requiring high safety have small ESR, while temporary openings have high ESR. With Q and the equivalent dimension one enters into Fig. 3.5.18 which leads to a definition of a ground class depending on the 'box' into which the Q-equivalent dimension pair falls. No support is required for cases falling below the boxes. The support requirements for the 38 classes are listed in Table 3.5.25.

In some classes only one type of support is assigned. In other classes, however, a differentiation has to be made based on the ratios $\frac{RQD}{J_n}$, $\frac{J_r}{J_a}$ or $\frac{SPAN}{ESR}$. For some classes up to 4 subclasses are listed. In addition, a footnote is assigned to most of the classes and one has to decide whether the actual ground conditions comply with the details given in these footnotes.

Since support ranges are given, Barton et al. recommend in the 1975 version to obtain the specific support quantities for a particular case by linearly interpolating over the range of SPAN/ESR for this class.

Support may also be estimated from Q by means of two empirical support pressure equations,

$$P_{ROOF} = \left(\frac{2.0}{J_r}\right) Q^{-\frac{1}{3}} \qquad \text{(Ba 2)}$$

$$P_{ROOF} = \frac{2\, J_n^{\frac{1}{2}}}{3\, J_r} Q^{-\frac{1}{3}} \qquad \text{(Ba 3)}$$

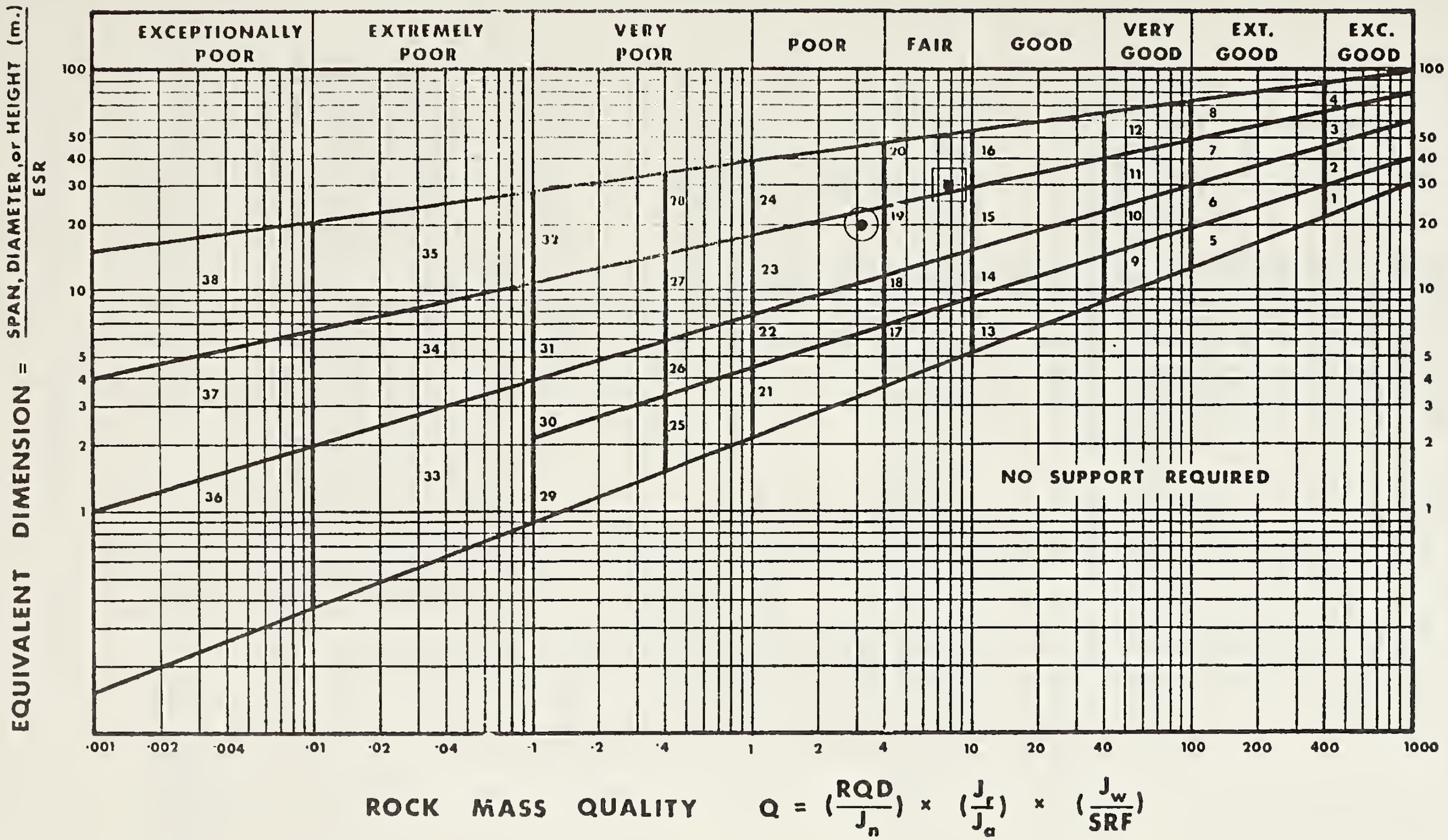


FIGURE 3.5.18 CLASSIFICATION DIAGRAM (FROM BARTON ET AL.,1974)

TABLE 3.5.25 SUPPORT RECOMMENDATIONS (FROM BARTON ET AL., 1974) (page 1 of 6)

. Support Measures for Rock Masses of "Exceptional", "Extremely Good", "Very Good", and "Good" Quality (Q range: 1000—10)

Support category	Q	Conditional factors RQD/ J_n	J_r/J_n	SPAN/ ESR (m)	P kg/cm² (approx.)	SPAN/ ESR (m)	Type of support	Note
1*	1000—400	—	—	—	< 0.01	20—40	sb (utg)	—
2*	1000—400	—	—	—	< 0.01	30—60	sb (utg)	—
3*	1000—400	—	—	—	< 0.01	46—80	sb (utg)	—
4*	1000—400	—	—	—	< 0.01	65—100	sb (utg)	—
5*	400—100	—	—	—	0.05	12—30	sb (utg)	—
6*	400—100	—	—	—	0.05	19—45	sb (utg)	—
7*	400—100	—	—	—	0.05	30—65	sb (utg)	—
8*	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*	100—40	≧ 30	—	—	0.25	23—48	B (tg) 2—3 m	—
		< 30	—	—			B (tg) 1.5—2 m + clm	—
12*	100—40	≧ 30	—	—	0.25	40—72	B (tg) 2—3 m	—
		< 30	—	—			B (tg) 1.5—2 m + clm	—
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—10cm	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—10cm	I, II, IV
16* See note XII	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—15cm	I, V, VI

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

The type of support to be used in categories 1 to 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 to 8.

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted

TABLE 3.5.25 SUPPORT RECOMMENDATIONS (FROM BARTON ET AL., 1974) (page 2 of 6)

Support Measures for Rock Masses of "Fair" and "Poor" Quality
(Q range: 10—1)

Support category	Q	Conditional factors RQD/J_n	J_r/J_a	SPAN/ ESR	P Kg/cm² (approx.)	SPAN/ ESR (m)	Type of support	Note
17	10—4	> 30	—	—	1.0	3.5—9	sb (utg)	I
		≧ 10, ≦ 30	—	—			B (utg) 1—1.5 m	I
		< 10	—	≧ 6 m			B (utg) 1—1.5 m + S 2—3 cm	I
		< 10	—	< 6 m			S 2—3 cm	I
18	10—4	> 5	—	≧ 10 m	1.0	7—15	B (tg) 1—1.5 m + clm	I, III
		> 5	—	< 10 m			B (utg) 1—1.5 m + clm	I
		≦ 5	—	≧ 10 m			B (tg) 1—1.5 m + S 2—3 cm	I, III
		≦ 5	—	< 10 m			B (utg) 1—1.5 m + S 2—3 cm	I
19	10—4	—	—	≧ 20 m	1.0	12—29	B (tg) 1—2 m + S (mr) 10—15 cm	I, II, IV
		—	—	< 20 m			B (tg) 1—1.5 m + S (mr) 5—10 cm	I, II
20* See note XII	10—4	—	—	≧ 35 m	1.0	24—52	B (tg) 1—2 m + S (mr) 20—25 cm	I, V, VI
		—	—	< 35 m			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) 1 m + S 2—3 cm	I
		< 12.5	≦ 0.75	—			S 2.5—5 cm	I
		—	> 0.75	—			B (utg) 1 m	I
22	4—1	> 10, < 30	> 1.0	—	1.5	4.5—11.5	B (utg) 1 m + 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) 1m	I
23	4—1	—	—	≧ 15 m	1.5	8—24	B (tg) 1—1.5 m + S (mr) 10—15 cm	I, II, IV, VII
		—	—	< 15 m			B (utg) 1—1.5 m + S (mr) 5—10 m	I
24* See note XII	4—1	—	—	≧ 30 m	1.5	18—46	B (tg) 1—1.5 m + S (mr) 15—30 cm	I, V, VI
		—	—	< 30 m			B (tg) 1—1.5 m + S (mr) 10—15 cm	I, II, IV

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

(tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; see Note XI)
S = shotcrete
(mr) = mesh reinforced
clm = chain link mesh
CCA = cast concrete arch
(sr) = steel reinforced

Bolt spacings are given in metres (m). Shotcrete, or cast concrete arch thickness is given in centimetres (cm).

TABLE 3.5.25 SUPPORT RECOMMENDATIONS (FROM BARTON ET AL., 1974) (page 3 of 6)

Support Measures for Rock Masses of "Very Poor" Quality (Q range: 1.0—0.1)

Support category	Q	Conditional factors RQD/J_n	J_r/J_a	$SPAN/ESR$ (m)	P kg/cm^2 (approx.)	$SPAN/ESR$ (m)	Type of support	Note
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 m	2.25	6—18	B (tg) 1 m + S (mr) 7.5—10 cm	I, IX
		—	—	< 12 m			B (utg) 1 m + S (mr) 5—7.5 cm	I, IX
		—	—	> 12 m			CCA 20—40 cm + B (tg) 1 m	VIII, X, XI
		—	—	< 12 m			S (mr) 10—20 cm + B (tg) 1 m	VIII, X, XI
28* See note XII	1.0—0.4	—	—	≧ 30 m	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 m			B (gt) 1 m + S (mr) 15—20 cm	I, II, IX
		—	—	—			CCA (sr) 30—100 cm + B (tg) 1 m	IV, VIII, X, XI

TABLE 3.5.25 SUPPORT RECOMMENDATIONS (FROM BARTON ET AL., 1974) (page 4 of 6)

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 See note XII	0.4—0.1	—	—	≧20 m	3.0	11—34	B (tg) 1 m + S (mr) 40—60 cm	II, IV, IX, XI
		—	—	< 20 m			B (tg) 1 m + S (mr) 20—40 cm	III, IV, IX, XI
		—	—	—			CCA (sr) 40—120 cm + B (tg) 1 m	IV, VIII, X, XI

* Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

TABLE 3.5.25 SUPPORT RECOMMENDATIONS (FROM BARTON ET AL., 1974) (page 5 of 6)

Support Measures for Rock Masses of "Extremely Poor" and "Exceptionally Poor" Quality
(Q range: 0.1—0.001)

Support category	Q	Conditional Factors RQD/J_n	J_r/J_a	$SPAN/ESR$ (m)	P Kg/cm^2 (approx.)	$SPAN/ESR$ (m)	Type of support	Note
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	0.1—0.01	—	—	≧15 m	6	6.5—28	B (tg) 1 m +S (mr) 30—100 cm	II, IX, XI
See note XII		—	—	≧15 m			CCA (sr) 60—200 cm +B (tg) 1 m	VIII, X, XI, II
		—	—	<15 m			B (tg) 1 m +S (mr) 20—75 cm	IX, XI, III
		—	—	<15 m			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	0.01—0.001	—	—	≧10 m	12	4.0—20	CCA (sr) 100—300 cm	IX
See note XIII		—	—	≧10 m			CCA (sr) 100—300 cm +B (tg) 1 m	VIII, X, II, XI
		—	—	<10 m			S (mr) 70—200 cm	IX
		—	—	<10 m			S (mr) 70—200 cm +B (tg) 1 m	VIII, X, III, XI

* Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

Supplementary Notes for Support Tables

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 some excavations, 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, for instance 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 its generally used as permanent support.

XI. According to the authors' 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).

Eq. Ba 2 is shown in Fig. 3.5.19, for different values of the Joint Roughness number J_r (diagonal lines). The cases that served as a basis for the equations are also shown in Fig. 3.5.19. The stippled band in Fig. 3.5.19 is Barton's et al subjective assessment of a relation between Q and support pressure, which represents the fact that for high Q no support is required.

For wall support requirements, Barton et al recommends to modify the Q-Parameter and then to use this modified Q Parameter to Eq. Ba 2 and Ba 3 or in Fig. 3.5.19: The modified Q are:

$$
\begin{aligned}
Q > 10 \quad &\rightarrow \quad Q_{wall} = 5Q \\
0.1 < Q < 10 \quad &\rightarrow \quad Q_{wall} = 2.5Q \\
Q < 0.1 \quad &\rightarrow \quad Q_{wall} = Q
\end{aligned}
$$

It should be noted that Barton (oral communication, 1979) and in his review comments does not recommend to use the Q - support presssure relation any more (see also critique).

Development and Underlying Philosophy

Barton's et al. method is based on approximately 200 case studies from the literature. Included are the 97 cases reported by Cecil (1970, 1975), large caverns studied by Cording, et al. (1971) and other cases reported in the literature. Cecil not only reported the cases but studied different classification schemes; however, he was unable to

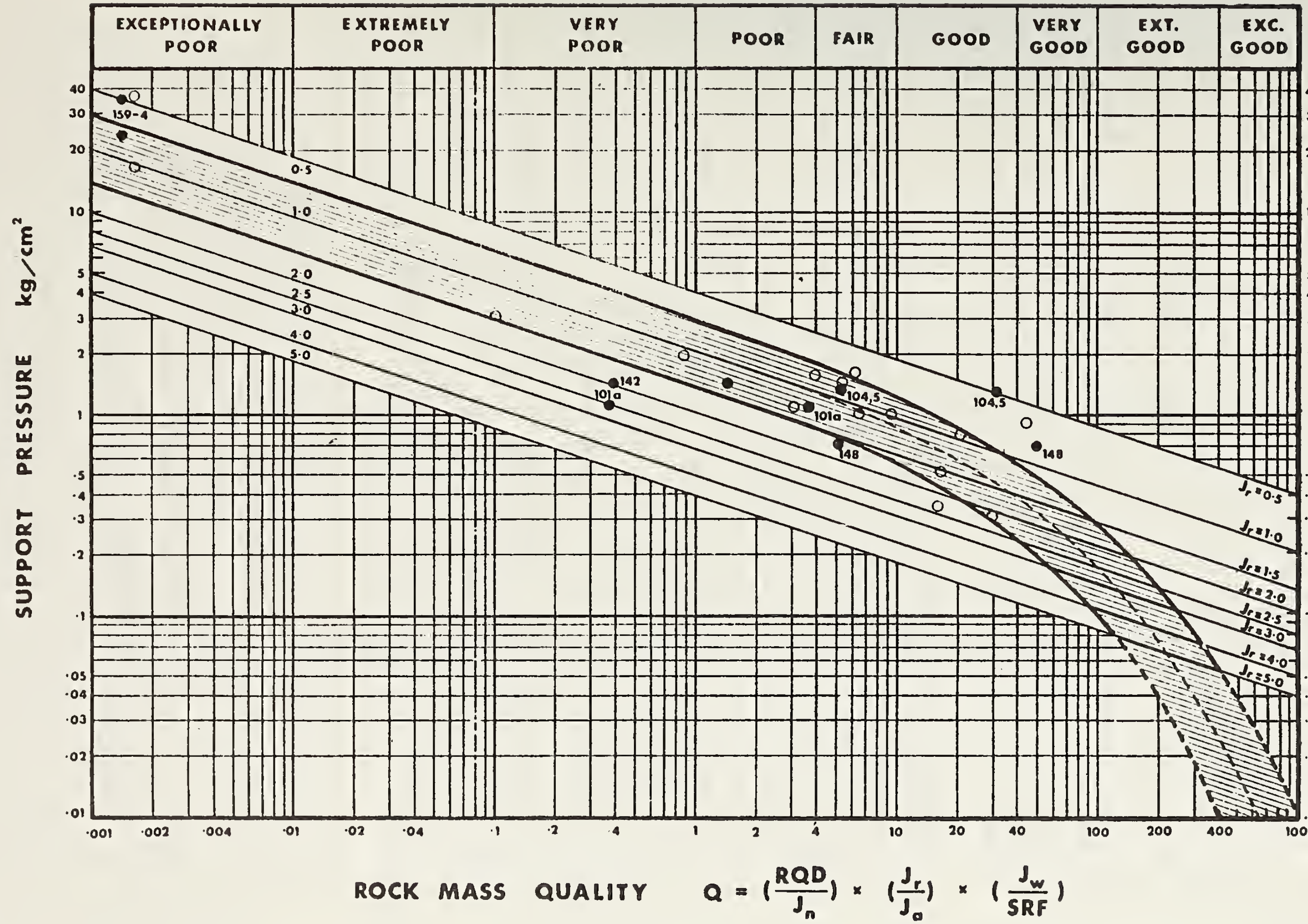


FIGURE 3.5.19 ROCK MASS QUALITY SUPPORT PRESSURE RELATION (FROM BARTON ET AL.,1974)

fit them conclusively to one scheme. Barton, et al. modified Cecil's cases by including more combined parameters. Cecil, e.g., considered primarily RQD - width plots. The modification started with the introduction of the joint number, then the joint roughness, J_r, and alteration number, J_a, finally the joint water factor, J_w, and the stress reduction factor, SRF, were added to arrive at a rock quality scale: Q. The ESR Scale was arrived at similarly by a systematic trial and error procedure. Combining the Q and ESR scale led to the case (box) classification by 3.5.18 case study.

The support pressure - Q classification procedure is based mostly on the study on large caverns reported by Deere, et al. (1971), where they considered the yield capacity of the rock bolts as support pressure. Other cases reported in the literature were also included. In some of these cases the support pressures were measured, in others it was a design pressure and sometimes it was backfigured using the 'hoop stress' formula to obtain the capacity of a shotcrete or concrete arch to which bolt capacity was added.

Another feature of the method is the physical meaning attributed to the ratios $\frac{RQD}{J_n}$, $\frac{J_r}{J_a}$. $\frac{RQD}{J_n}$ should be roughly equivalent to the block size and $\frac{J_r}{J_a}$ to the apparent shear strength between the blocks.

Critique

Barton's method is based on many case studies primarily from Norway and Sweden. Some base cases are reported in detail or they can be found in Cecil (1970) or Cording et al. (1971) while some others are not. The majority of the base cases is from Scandinavia and most of them have been reported by Cecil (1970, 1975). Cecil describes the geologic conditions; he states that the rocks are precambrian to siluriran. The method seems thus to be primarily applicable in similar ground conditions.

The method is based on six parameters (RQD, J_n, J_r, J_a, J_w, SRF). The determination of RQD is straightforward as is J_n. J_r and J_a require more experience and may be difficult to determine if the tunnel is outside the range of base cases (as experienced in the application of the method to the Tavern and Arlberg tunnels, see example applications in Section 3.6). The selection may also be problematic if multiple joint sets are encountered, i.e., one has to decide which joint set is the 'significant' one (see Appendix D). Orientation is only indirectly considered in the method. Barton et al. state (see Table 3.5.24) that depending on orientation, often not the joint set with the 'least strength' is the governing one for which J_r and J_a are to be determined, but a set with higher strength and less favorable orientation with regard to stability. However, no guidelines are given on how to decide on the critical combination.

Another potential problem lies in the large number of classes (boxes in Fig. 3.5.18) which may give the user too high a level of confidence in the accuracy of the support prediction.

Barton and Lien have reviewed this assessment of the Q-Method and do not agree with some of the statements made. Three specific comments are listed below, while their other more general review comments are given in Appendix D.

Note that the statement with which Barton and Lien take issue is typed in italics followed by their comments.

"During construction time constraints may limit the use of the method, not so much regarding measurement of the parameters but regarding ground assessment with the relatively complex set of tables and notes."

In fact time constraints are hardly a problem with the Q-system. We have found on numerous occasions that we can map the rock mass quality along a variable stretch of tunnel at a rate of up to several kilometers per day as routine. The length mapped, i.e., 1-3km, will of course depend on the geologic conditions. As with all methods there is obviously a time constraint in that the engineering geologist cannot be at several tunnel faces at one time. Poor quality rock concealed behind a layer of shotcrete (temporary support) creates a local problem for all ground classification systems that do not rely on monitoring.

"J_r and J_a require more experience and may be difficult to determine...if multiple joint sets are encountered, i.e., one has to decide which joint set is the 'significant' one (see Appendix D)."

Perhaps the authors would agree that the explicit instructions given by Barton et al. (1974) concerning which joint set or discontinuity to classify are at least easier to follow than the complete lack of joint set differentiation in other methods.

orientation and strength (compare also discussion in Appendix D). The fifth parameter J_w can be easily determined in most cases. SRF on the other hand may be difficult to determine particularly in the predictive phase where the detailed information required (see table 3.5.23) is often not available. Also SRF may have the same value in different types of ground. This does not matter as long as the other five parameters differentiate the ground. However in the case of swelling and squeezing ground (SRF = 5 to 15) the other parameters are not sufficient to produce such a differentiation while the consequences on support requirements may be substantially different.

During construction, time constraints may limit the use of the method, not so much regarding measurement of the parameters but regarding ground class assignment with the relatively detailed set of tables and notes in the adverse conditions of a tunnel.

A rough physical meaning has been assigned to the ratios RQD/J_n, J_r/J_a, J_w/SRF. RQD/J_n should be roughly equivalent to the block size in centimeters. The ratio J_r/J_a should be roughly equivalent to the apparent friction angle. The ratio J_w/SRF is considered to be equivalent to the "active stress". The limitations of these ratios are discussed by Barton et al. in the 1974 paper (potential users should refer to these complete discussions and not only to the summary statements in the 1975 and 1976 papers). There is a need for emphasizing a careful study of Barton's comments on these ratios. It is tempting to measure 'blocksize' and friction angle, since they appear to be more "objective" than J_n, J_r and J_a and to use them in the Q-relation. This has apparently happened in practice, (Blakey, 1979) and goes beyond Barton's et al. intention to use these relations as rough checks.

As in other methods the support pressure relations are not reliable which has also been recognized by Barton et al. (Barton, personal communication, 1979) and their use is no longer recommended. The reasons why these support pressure relations may be questionable are discussed in Appendix D.

The 'trail and error' procedure used in the development of the method is a clear indication of the subjective assessment used by the developers. The large number of cases and possible parameter combinations was examined by a systematic trial and error procedure. The decision on what parameters to combine and where to select limits is by definition subjective and makes it possible to incorporate the experience of the developers; see also discussion of Wickham et al. method, Section 3.5.3. Many of the base are listed and commented in detail, thus the user can judge the range of applicability. Caution has to be exercised using it for deviating cases. For squeezing rock the method leads to overly conservative support recommendations due to the Straight Creek tunnel base case.

"The 'trial and error' procedure used in the development of the Q-method is a clear indication of the subjective assessment used by the developers. The large number of cases and possible parameter combinations were examined by a systematic trial and error procedure. The decision on what parameters to combine and where to select limits is by definition subjective and makes it possible to incorporate the experience of the developer."

If the authors had followed the development of the Q-system closely, they would appreciate that the trial and error adjustment and readjustment of parameter ratings was an important factor in *reducing* the need for subjective judgements on our part. The large number of case records made it possible to generate the support recommendations quite objectively, in fact sometimes against our "better" judgement. (Example: the extreme support at Straight Creek.)

Also other specific comments by Barton and Lien relate to the assessment of discontinuity orientation and support pressure relations. Since both of these issues are discussed in detail in Appendix D, their respective comments will be given there.

Summary of Barton's Method

The method considers six parameters that describe ground conditions and are combined to yield a total rating Q of the ground. This together with the equivalent (the actual dimensions of the opening divided by the Excavation Support Ratio, ESR) defines a class which can be directly related to support quantities. The method gives detailed support recommendations for many conditions.

3.5.6 Louis' Method

Methodology and Application Guidelines

The empirical classification proposed by Louis (1974 a,b) considers

(a) the excavation-technique (e.g. TBM versus drill and blast)

(b) support-excavation procedure

it employs the following paramaters:

(a) joint spacing expressed by - RQD or by - median joint spacing

(b) the intact rock strength - unconfined compression test or by - point load test

(c) the overburden stress

(d) the diameter of the tunnel.

For the classification of excavation-technique Louis plots RQD vs. strength (either expressed by the unconfined compressive strength or by the point load index (Fig. 3.5.20). The boundaries between classes are shown on Fig. 3.5.20 and explained in Table 3.5.26.

The support excavation-procedure is determined with a second chart (Fig. 3.5.21). It considers the normalized fracture spacing i.e., the median joint spacing (M_f) divided by the tunnel diameter (D) and the normalized strength of the rock, i.e., the ratio of compressive strength (σ_c) to the maximum tangential elastic stress at the circumference of the opening ($3\gamma H$) (which is assumed to be three times the

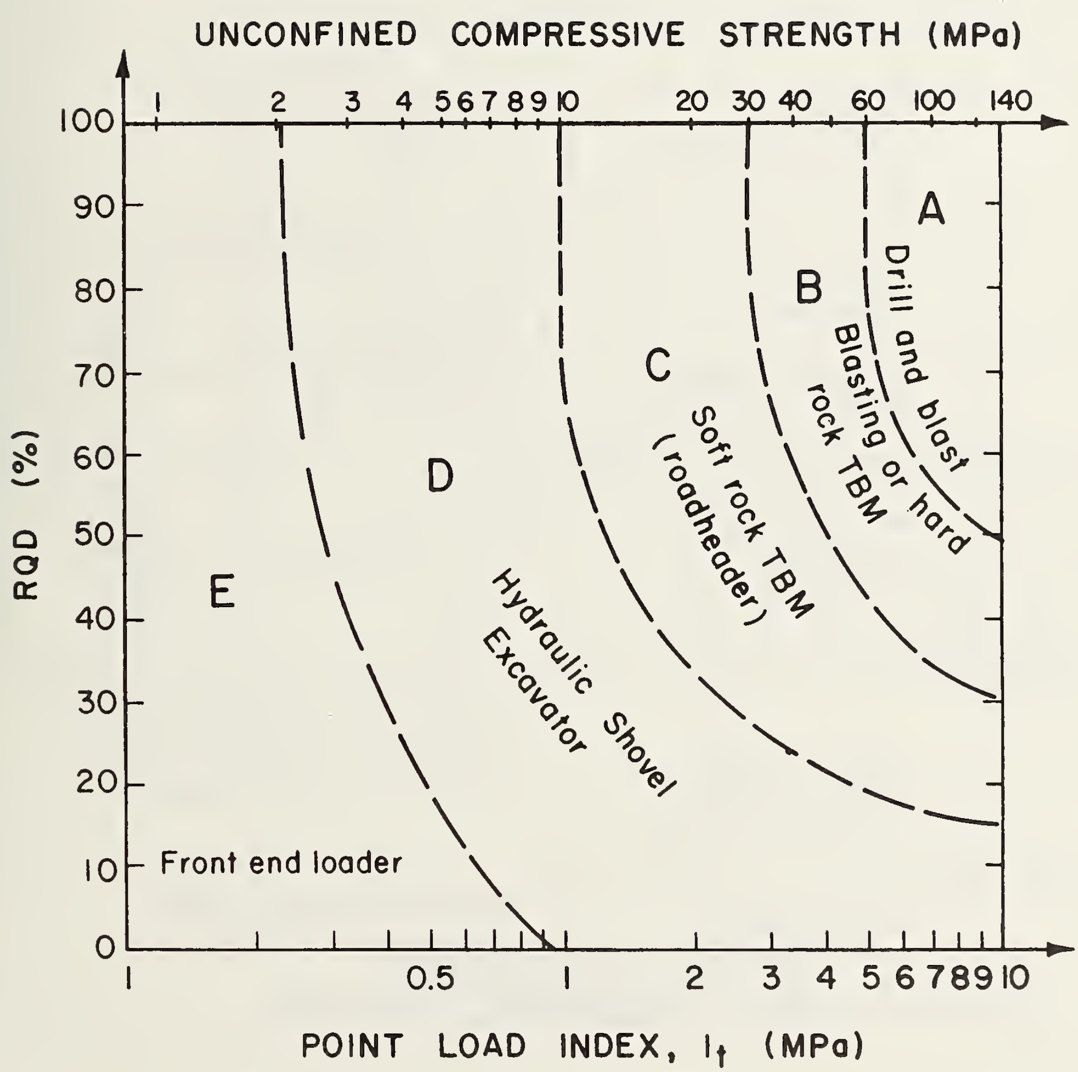


FIGURE 3.5.20 CLASSIFICATION FOR EXCAVATION
TECHNIQUE (AFTER LOUIS,1974)

TABLE 3.5.26 CLASSIFICATION WITH RESPECT TO EXCAVATION TECHNIQUE (AFTER LOUIS, 1974)

GROUND CLASS	GROUND CLASS	METHOD OF EXCAVATION
A	compact lightly fractured	Drill & Blast with pre-splitting or smooth wall blasting. Roundlength 2 to 3m.
B	compact but fractured rock mass medium strong	Drill & Blast with pre-splitting or smooth wall blasting. Roundlength = 2m Mechanical excavation with hard-rock TBM.
C	Mass fractured small mass strength	Tunnel boring machines for weak rock. Bench excavation by ripping, with or without loosening blasts.
D	Soft Rock with small strength or strongly fractured	Mechanical Excavation with road header or hydraulic excavator. Loosen harder zones with jack hammer.
E	Soft Ground or very fractured	Excavation with shovel excavator or front end loader.

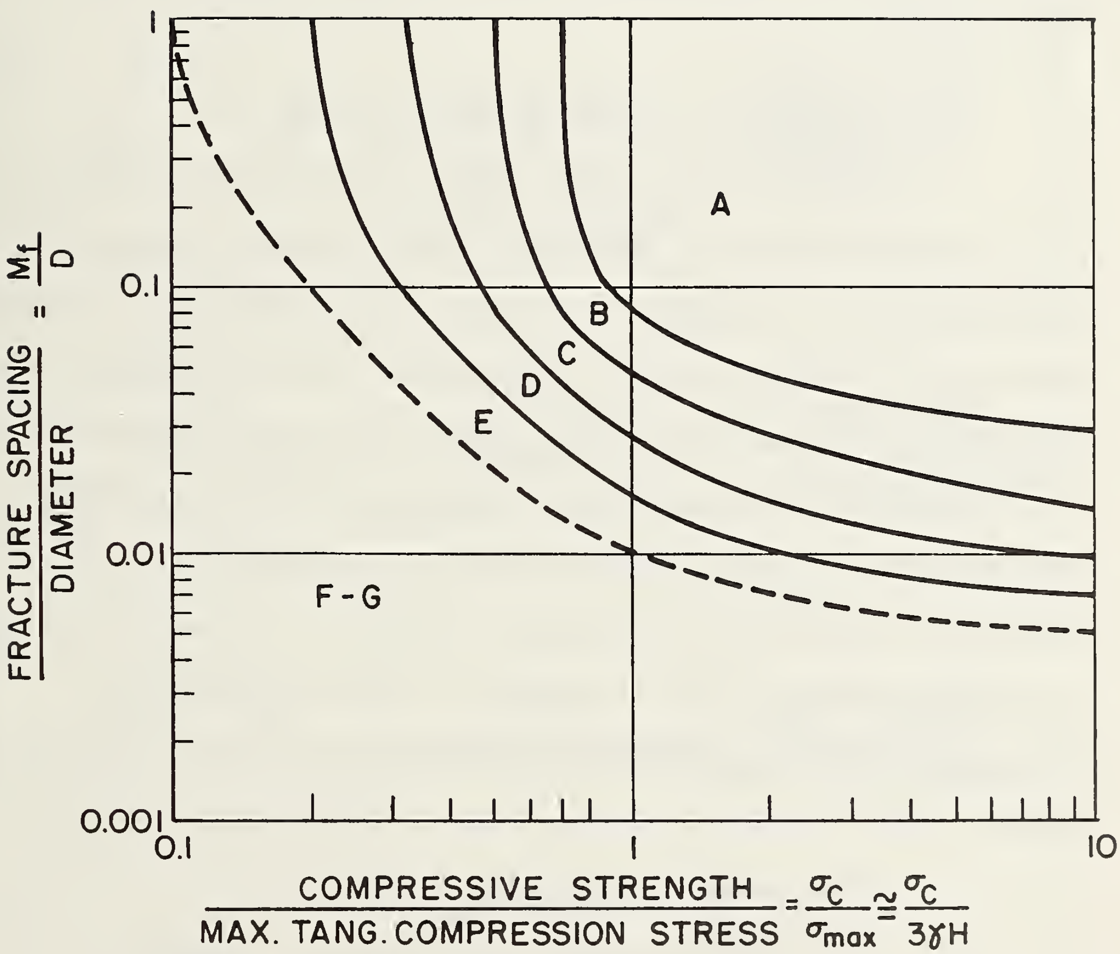


FIGURE 3.5.21 LOUIS' CLASSIFICATION CHART FOR EXCAVATION-SUPPORT METHOD (AFTER LOUIS,1974b)

overburden stress). A ground class is assigned to each pair of $(\frac{Mf}{D})$, $(\frac{\sigma_c}{3\gamma H})$ on the diagram (Fig. 3.5.21). The excavation support procedures for each ground class are described in Fig. 3.5.22.

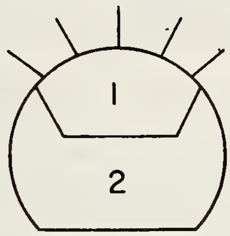
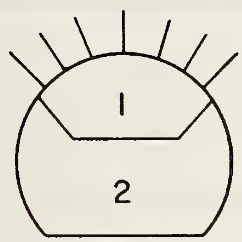
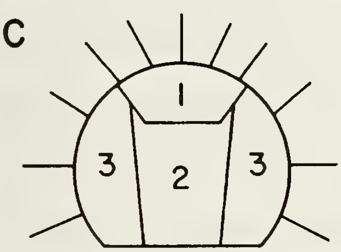
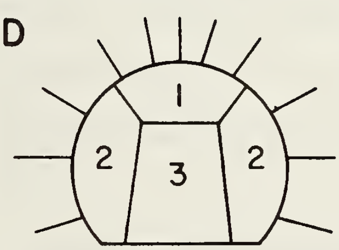
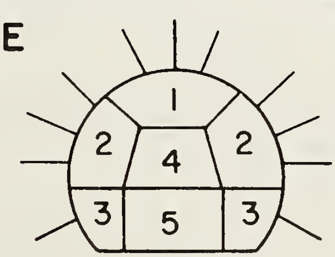
Development and Underlying Philosophy

The procedure was developed by Louis (1974, a,b) at the BRGM (Bureau de Recherche Geologique et Miniere = French Bureau of Mines and Geology) in connection with the design of a highway tunnel underneath the city center of Marseille. Two parallel dual-lane tunnels are planned, the first one will be constructed by enlarging an abandoned double-track railroad tunnel that led to the old port of Marseilles (Berille et Meneroud, 1974). In a second stage a parallel tunnel will be driven. The classification method was used to select between alignments for the second tunnel, the detailed procedure is, however, not given.

The method is based on:

(1) the strength and fracture spacing for selection of excavation-technique

(2) strength related to the 'acting' stress and block size with respect to tunnel dimensions for the selection of the support-excavation procedure.

Classes	Roundlength Excav.	Support [1]
A 	3 m drill and smooth blasting	1 bolt per 3 to 4 m^2
B 	1.5 to 3 m drill and smooth blasting	1 bolt per 8 m^2 15 cm shotcrete 1 to 2 layers of wirefabric
C 	1.5 to 2.5 m machine or very careful blasting	1 bolt per 5 m^2 18 cm shotcrete 2 layers of wirefabric
D 	1 to 2 m machine excavation	1 bolt per 4 m^2 21 cm shotcrete 2 layers of wirefabric
E 	0.8 to 1.5 m machine excavation	1 bolt per 3 m^2 light steelsets 24 cm shotcrete 3 layers of wirefabric

1) Length of bolts has not been specified by Louis

FIGURE 3.5.22 PROPOSED GROUNDCLASSES FOR MARSEILLE'S TUNNEL (AFTER LOUIS, 1974b)

Thus for the classification with respect to excavation-technique, RQD, a measure of jointing, and the intact rock strength were considered. For the support-excavation classification initially only RQD was considered to represent jointing and was plotted versus 'normalized strength'. Normalized strength is the unconfined compressive strength of the ground divided by three times the overburden stress $\frac{\sigma}{3\gamma H}$ (Three times the overburden stress corresponds to the elastic stress concentration at the circumference of a circular opening in an uniaxial stress field. Actually, there usually will be a horizontal stress thus the uniaxial stress field assumption is conservative.). The initially proposed RQD vs. $\frac{\sigma c}{3\gamma H}$ classification diagram is shown in Fig. 3.5.23. Later, Louis modified the procedure and used a 'normalized' block size, i.e., RQD was substituted by the median discontinuity spacing divided by the tunnel diameter.

Louis also proposed a procedure to consider water effects with the variables

- water pressure α
- permeability β
- swelling pressure. γ

Each variable is rated according to Table 3.5.27. The sum of the parameters $\alpha + \beta + \gamma$ equals ω which is used to determine the corresponding ground class.

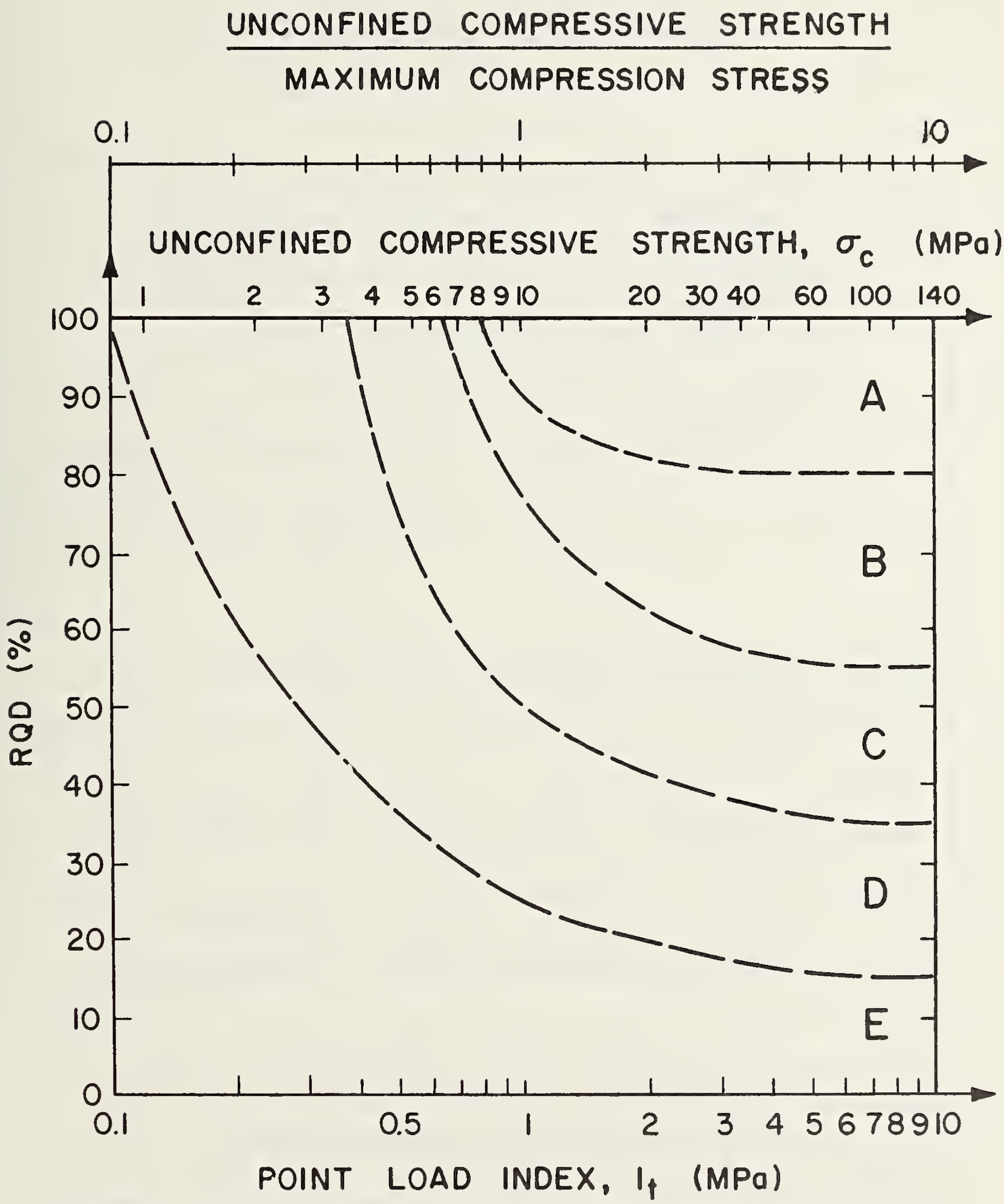


FIGURE 3.5.23 INITIAL GROUNDCLASSIFICATION CHART
FOR SUPPORT EXCAVATION METHOD
(AFTER LOUIS,1974a)

TABLE 3.5.27 CONSIDERATION OF WATER IN LOUIS' CLASSIFICATION (AFTER LOUIS, 1974)

RATING	HYDRAULIC HEAD ABOVE CROWN (meters) α	PERMEABILITY m/s β	SWELLING PRESSURE (in MPa) γ
0	No Water	$<10^{-8}$	No
	0	10^{-8}	0 MPa
1			
	1 Diameter $\cong$ 12m	10^{-6}	0.1 MPa
2			
	3 Diameter $\cong$ 36m	10^{-4}	0.5 MPa
3	> 36m	$>10^{-4}$	>0.5 MPa

$\alpha + \beta + \gamma$	0	1	2	3	4	5	6
Ground class	A	B	C	D		E	

Each ground will thus be classified in two ways, once with Figs. 3.5.21 and 22 for rock and stress conditions and once for water conditions (Table 3.5.27). Louis recommends to select the 'worse' ground class for design.

Louis (1973) also states that this method should only be used during preliminary phases of a project; and during construction, the performance of the tunnel should be monitored.

Critique

The procedure proposed by Louis considers actually measurable variables, i.e., joint spacing, the strength of intact rock and the overburden stress. The procedure does not consider the strength of discontinuities which may possibly be the governing factor (compared to intact strength). Also the method seems to consider only the spacing of a single joint set and not multiple joint sets.

In addition, other questions have to be raised.

The procedure was developed for a preliminary design and the selection between two possible alignments of the tunnel in Marseilles, however, the actual case studies that were used for the development are not given in detail.[1)] In particular it is not entirely clear how

[1)] Compare also the description of Franklin's method (Section 3.5.7), which gives some indications on the type of base cases.

the boundaries between classes (Fig. 3.5.21) were determined since the Marseille tunnel was only in the design stages during development of the method.

Summary of Louis Method

The method was developed for the preliminary design of a highway tunnel in Marseille. It considers

- strength of intact rock (unconfined compressive strength or point load index)
- spacing of joints (RQD, median spacing)
- tunnel diameter

All parameters are easily determinable during preliminary investigations and also during actual construction, but the method does not consider the strength of discontinuities.

Louis recommends to use performance monitoring during construction and to rely less on the classification chart for the actual support-excavation determination. A comparison classification based on RQD and strength may be used to determine the most suitable excavation technique.

3.5.7 Franklin's Classification

Methodology and Application Guidelines

Franklin's (1975, 1976) procedure is very similar to Louis'. Initial development of the method has been performed by Louis and Franklin (Franklin, 1979) working together. The format of the published versions is, however, somewhat different. Franklin considers the following parameters:

- unconfined compressive strength, which is usually determined by the point load strength test
- block size, earlier referred to as fracture spacing index
- the size of the opening
- the overburden stress

Each of the four parameters is plotted on an axis in Fig. 3.5.24. The data of an actual case are plotted on the chart and depending in which area of a particular quadrant the values fall different behavior resp. problems can be expected in the tunnel (Fig. 3.5.24).

For support estimation the charts shown in Fig. 3.5.25 are used. This particular chart applies to a tunnel of 7m diameter at approximately 80m depth (it is not 'normalized' as Louis').

A support-excavation procedure is selected with Fig. 3.5.25, the support quantities are determined with Fig. 3.5.26. Note that support quantities are given continuosly over the entire range of 'classes'.

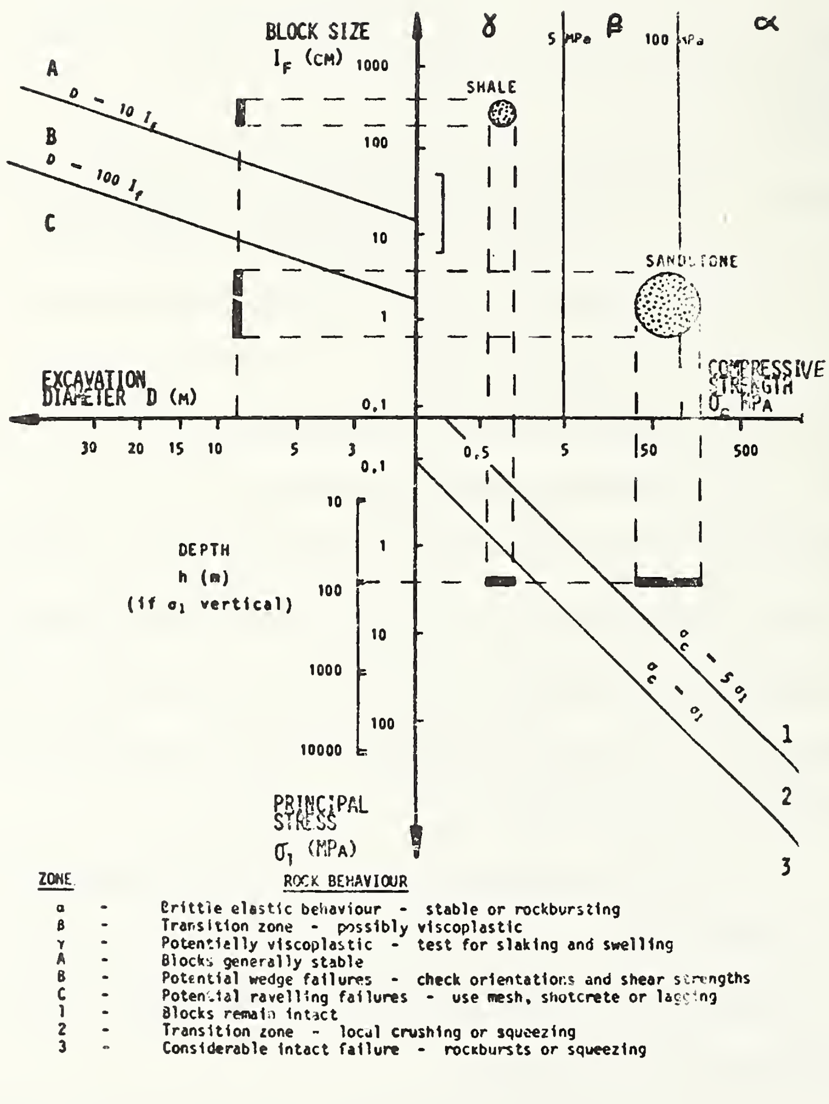
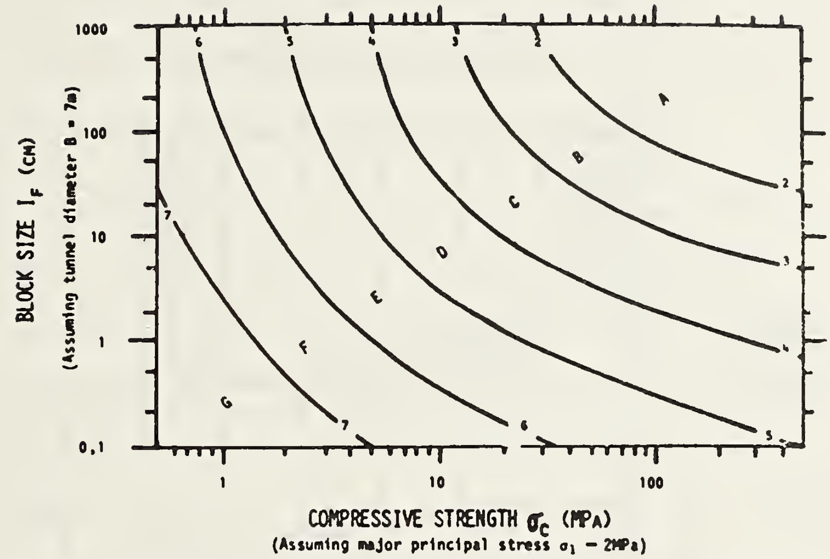
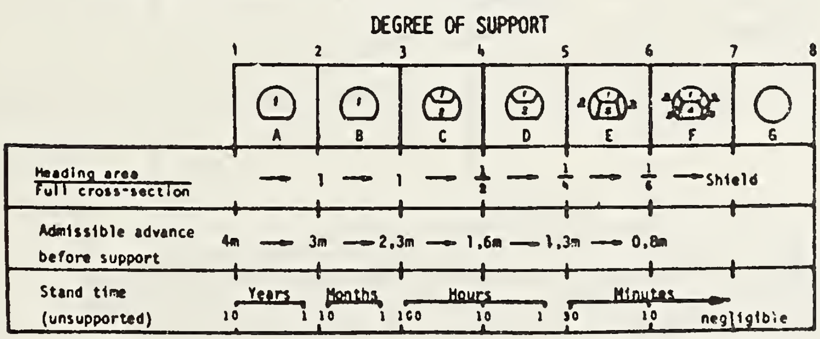


FIGURE 3.5.24 FRANKLIN'S DIAGRAM FOR PRELIMINARY EVALUATION OF TUNNEL STABILITY AND FAILURE MECHANISMS (FROM FRANKLIN,1976)

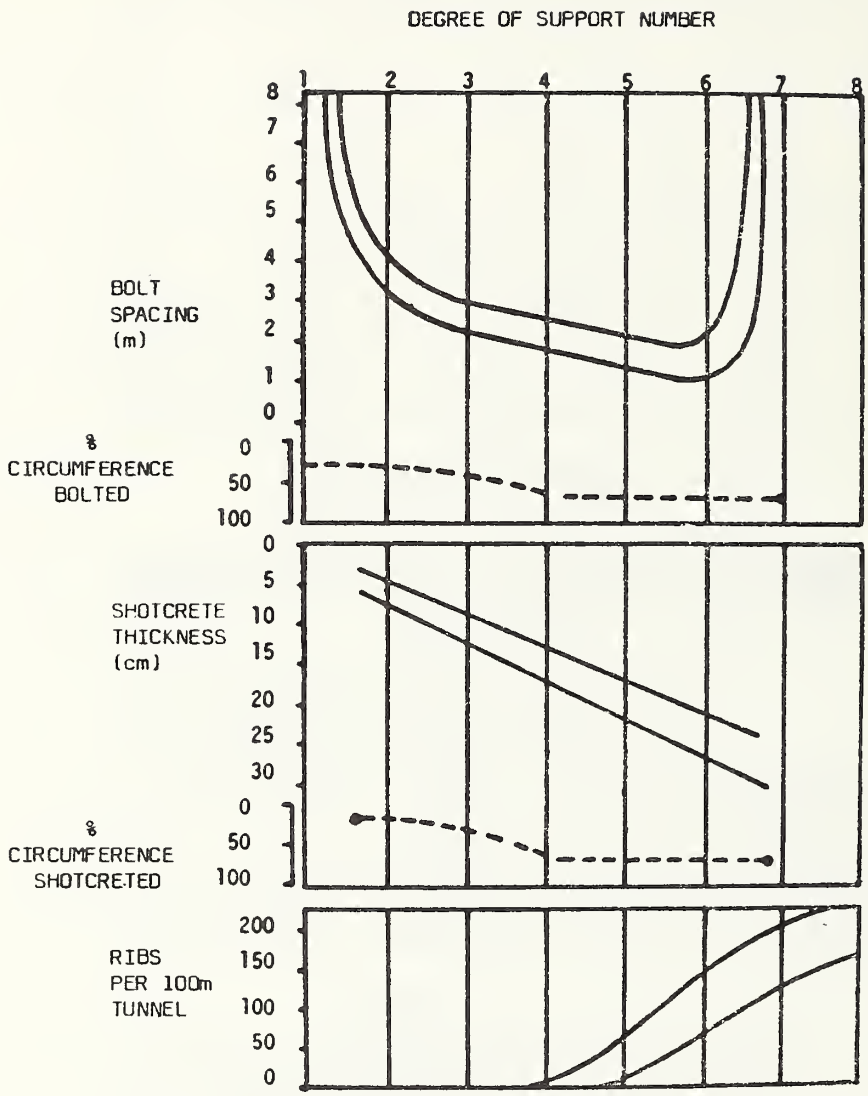


Contour numbers indicate the "degree of support" applicable to rock with given size -strength properties. Letters A-G indicate the "degree of support class". Interpolation can be used, for example degree of support = 2.5.



DERIVATION OF "DEGREE OF SUPPORT CONTOURS" (TOP), AND THEIR USE IN PREDICTING THE THREE MAIN VARIABLES OF EXCAVATION

FIGURE 3.5.25 FRANKLIN'S DEGREE OF SUPPORT CHARTS (FROM FRANKLIN,1976)



FOR DERIVATION OF "DEGREE OF SUPPORT NUMBER"
SEE FIGURE 3.5.25

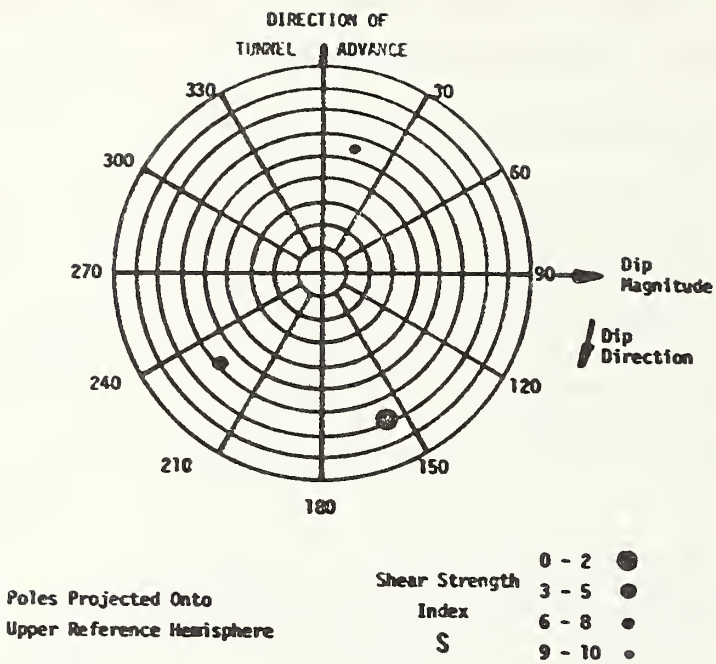
FIGURE 3.5.26 FRANKLIN'S SUPPORT RECOMMENDATIONS
(FROM FRANKLIN,1976)

In addition to the above described method for the design of support-excavation procedures, Franklin proposes a graphical procedure to study local instabilities at the face, using a stereonet (pole diagram). The discontinuities (joints, shears, faults) should be marked with a point whose size depends on the available shear resistance (Fig. 3.5.27). For discontinuities with low shear resistance a large dot is used. Depending on the location of the pole the severity of anticipated problems can be easily visualized.

Development and Underlying Philosophy

A complete picture on development and underlying philosophy can be gained from the review comments made by Franklin:

> "This classification was initially developed by Franklin as part of his PhD research programme, published as a general purpose rock mass classification in the 1970 thesis and in the proceedings of the 2nd Congress of the ISRM in 1970. The concepts were applied to tunneling by Louis and were developed further by Franklin under contract to Louis at the BRGM in France. Figures 3.5.24/25/26 were compiled from data obtained in a review of approximately 10 tunnel support case history reviews including those by Deere et al. (1967) and Terzaghi (1946). The previously employed rock classifications generally gave information on both size and strength, for example, RQD was converted to block size by a well established correlation formula. This allowed, for example, actual tunnel support data to be plotted as points on Figure 3.5.25, forming surprisingly narrow bands of variation particularly for the new Austrian Tunneling Method. Lines on this diagram represent envelopes to approximately 95 percent of the data. Contours



SHEAR STRENGTH INDEX 'S' = R + P + F (RANGE 0 - 10)

where :

ROUGHNESS INDEX R	=	0	(flat and polished)
	=	1	(flat)
	=	2	(amplitude 10 - 50 mm)
	=	3	(amplitude 50 - 100 mm)
	=	4	(amplitude 100 mm or more)
PERSISTENCE INDEX P	=	0	(100% continuous)
	=	1	(at least 50% continuous)
	=	2	(less than 50% continuous)
FILLING INDEX F	=	0	(at least 20 mm clay/mica/ graphite)
	=	1	(10 mm to 20 mm clay/mica/ graphite)
	=	2	(less than 10 mm clay/mica/ graphite)
	=	3	(at least 1 mm sandy material)
	=	4	(clean and hard surfaces)

FIGURE 3.5.27 FRANKLIN'S POLE-DIAGRAM FOR THE ASSESSMENT OF STABILITY PROBLEMS (FROM FRANKLIN,1976)

on the classification diagram (Fig. 3.5.25) were initially proposed and subsequently modified several times when the classification system was applied to the Rubira tunnel in Barcelona, Spain; Franklin was employed as a consultant on the project by BRGM. The same classification was applied successfully to a number of tunnels in France.

Versatility and simplicity were the main objects of this classification which was intended for applications in foundations, rock slopes and for the evaluation of construction materials as well as for use in tunneling applications. It seemed desirable to avoid several different types of rock classification for large projects where several applications applied. Block size and strength were selected as the main elements of most existing classifications, and the most important when attempting predictions of rock behaviour. It was considered that tunnel excavation and support conditions could be largely predicted on the basis of these two properties alone, but that supplementary parameters would essentially have to be considered, particularly those relating to joint orientations and characteristics. In practice, the 'degree of support number' was downgraded or upgraded arbitrarily, seldom by more than one unit, based on observations, for example, of joints and of groundwater inflow. The classification contours were developed for relatively shallow civil engineering tunnels of moderate diameter, to which the majority of the available data applied. An important restriction dictating the simplicity of the classification system was its use in 'quality control' of the rock at the advancing tunnel face, where a time of only approximately 10 minutes was available for observations between blasting and shotcreting operations."

Critique

The method considers intact strength, overburden, fracture spacing or block size and tunnel diameter. These parameters can be easily determined through field tests. Franklin in his review comments provided the following information on the limits of applicability:

> "The design curves are strictly applicable only to relatively shallow tunnels, of moderate diameter (30-300m depth and 4 to 10m diameter). For tunnels outside this range, the procedure has been to move the classification contours of Fig. 3.5.25 drawn on an overlay, in a direction at 45° to the size-strength axes, arriving at different degree of support numbers for the same rock conditions. The same procedure can be used to take into account factors such as shallow cover, excessive groundwater, inexperienced personnel, etc. which are difficult and probably undesirable to include in a classification of the rock per se. Extrapolations of this type, however, must be done on the basis of experience and judgement since there is little data available in these cases."

The method can be used as a 'quality control' tool during tunnel construction. It must be updated with experience gained on each individual project and judgement cannot be neglected.

On a more detailed level of critique, the following points might be raised:

- The method does not consider the resistance of joints.
- Difficulties may arise in determining block sizes if the fracture or joint spacing varies.

Summary

The method was derived from Franklin's PhD thesis and was further developed in a collaborative effort by Louis and Franklin. The method considers:

- block size or fracture spacing
- intact strength
- overburden stress
- tunnel diameter.

With these four parameters and two charts support quantities can be estimated. The method is strictly applicable to tunnels of 4 to 10m diameter at 30 to 300m depth.

Using judgement and experience other parameters (like orientation, water inflow) and other ranges of applicability can be included by shifting the class boundaries.

The input parameters for the method can be easily determined during preliminary phases and during actual construction. The method has the potential to be further expanded with increasing experience.

3.6 EXAMPLE APPLICATIONS

This section presents comparative applications of the empirical methods as discussed before made by the authors and others.

3.6.1 Example Applications by the Author

The methods by Barton (1974), Bieniawiski (1976), Wickham, et. al. (1974), Deere (1969) and Terzaghi (1946) have been applied to cases reported by Cecil (1970, 1975) and to sections of the Arlberg and Tauern tunnels. All cases are well documented. Cecil's cases form the basis of Barton's method and to some extent Bieniawiski's. The Arlberg and Tauern tunnels are NATM applications. To include the uncertainty or subjectivity in the interpretation of geology and other parameters, three parameter states were used for each case (worse, most likely, best) in the Arlberg and Tauern examples.

To put these case histories and the example applications into proper context, the following remark is necessary. The methods particularly those employing relatively few classes and reflecting a large number of base cases provide 'average' support recommendations. Some of the cases discussed here may be extreme cases (e.g., low RQD but very favorably oriented and tight joints, thus requiring no support). Some of the differences between example predictions and actually placed support may thereby be explained.

In Table 3.6.1 cases reported by Cecil that required no support are presented. The RQD varies from 0 to 100% in these cases, all these cases have a single, steeply dipping, tight joint set that in most cases strikes perpendicular to the tunnel axis.

With Terzaghi's classification this would be considered hard stratified or shistoze rock giving a rock load factor of 0 to 0.5B. Thus Terzaghi's method includes the correct prediction "no support requirements" within its range of rock loads. With Barton's direct relation also no support is predicted, unsurprising since Cecil's cases form part of Barton's data base. Note also that the support pressure predicted with Barton's Q-support pressure relation is approximately equal to the median rock load using Terzaghi's rock load approach.

Wickham, et al. predict rock loads that equal or exceed Terzaghi's maximum values (with the exception of case 35 that gives a lower value than Terzaghi). Since one can assume that Wickham, et al. base their method mostly on tunnels that were designed according to Proctor and White (Terzaghi), one is tempted to conclude, in contrast to widely held opinion, that Terzaghi's method is not necessarily conservative, but rather has been applied conservatively, in cases where no or light to medium support is necessary (the picture changes however for cases with heavy support, see Table 3.6.3, Arlberg and Tauern applications).

TABLE 3.6.1 EXAMPLE APPLICATIONS: UNSUPPORTED CASES REPORTED BY CECIL (1975)

Case	Rock Load (kPa) and Recommended Support (1)					(2) Comments
	Terzaghi	Barton	Wickham	Bieniawski	Deere	
Cecil 6 B= 9m H= 9m D=150m	0-112	38 No support	110 RR=22% 8WF40 1.25m	---- Class II: BO: locally in crown, L=3m, Sp=2.5m, with occasional wire mesh SC: 5cm in crown	135-290 BO: pattern in crown and walls, Sp=1-1.5m, OR SS: light to medium, Sp=1-1.5m, OR SC: $\geq$10cm on crown and walls	RQD used=60% RQD_v=0-100% RQD_a=80%
Cecil 8 B= 9m H= 9m D=140m	0-112	36 No support	111 RR=22% 8WF40 1.25m	---- Class III: BO: pattern in crown and walls, L=4m, Sp=1.5-2m, with wire mesh in crown SC: 5-10cm in crown and 3cm in walls	135-290 BO: pattern in crown and walls Sp=1-1.5m, OR SS: light to medium, Sp=1-1.5m, OR SC: $\geq$10cm on crown and walls	RQD_a=70%
Cecil 20 B= 9m H= 9m D=30m	0-112	36 No support	111 RP=22% 8WF40 1.25m	Class II: BO: locally in crown, L=3m, Sp=2.5m, with occasional wire mesh SC: 5cm in crown	135-290 BO: pattern in crown and walls, Sp=1-1.5m, OR SS: light to medium, Sp=1-1.5m, OR SC: $\geq$10cm on crown and walls	RQD used=70% (across joints) RQD_v=90% RQD_a=80%
Cecil 35 B= 12.5m H= 6.5m D=110 m	0-138	28 No support	124 RR=45% 10WF45 1.3m	Class III: BO: pattern in crown and walls, L=4m, Sp=1.5-2m, with wire mesh in crown SC: 5-10cm in crown and 3cm in walls	250-350 BO: pattern, Sp=1m, OR SS: heavy circular sets, Sp=0.6m, OR SC: $\geq$15cm comined with medium to heavy sets	RQD=0%
Cecil 36 B=12.5m H= 6.5m D=60 m	0-63	36 No support	124 RR=45% 10WF45 1.3m	Class III: BO: pattern in crown and walls, L=4m, Sp=1.5-2m, with wire mesh in crown SC: 5-10cm in crown and 3cm in walls	250-350 BO: pattern, Sp=1m, OR SS: heavy circular sets, Sp=0.6m, OR SC: $\geq$15cm comined with medium to heavy sets	RQD used=20% RQD_v=20-80%

BO = Bolts, SS = Steel Sets, SC = Shotcrete, L = Length (of bolts), Sp = Spacing (of bolts, or steel sets), for bolts square pattern is assumed, B= Width, H = Height, D= Depth,Overburden

(1) Support recommendations are given for the most likely condition

(2) RQD values are those reported by Cecil ; RQD_a = RQD along axis of tunnel, RQD_v = vertical RQD, RQD used = RQD used in classification.

TABLE 3.6.1 EXAMPLE APPLICATIONS: UNSUPPORTED CASES REPORTED BY CECIL (1975) (CONT.)

Case	Rock Load (kPa) and Recommended Support					Comments(1)
	Terzaghi	Barton	Wickham	Bieniawski	Deere	
Cecil 52 B= 6.5m H= 4.5m D=70 m	0-32	36 No support	101 RR=29% 8WF31 1.2m	Class III: BO: pattern in crown and walls, L=4m, Sp=1.5-2m, with wire mesh in crown SC: 5-10cm in crown and 3cm in walls	250-350 BO: pattern, Sp=1m, OR SS: heavy circular sets, Sp=0.6m, OR SC: $\geq$15cm comined with medium to heavy sets	RQD=0%
Cecil 70 B= 8 m H= 5.7m D=15 m	0-100	36 No support	149 RR=34% 8WF40 1.5m	--- Class III: BO: pattern in crown and walls, L=4m, Sp=1.5-2m, with wire mesh in crown SC: 5-10cm in crown and 3cm in walls	200-320 BO: pattern, Sp=0.6-1.3m, OR SS: medium to heavy, Sp=0.6-1.3m, OR SC: $\geq$15cm on crown and walls combined with bolts	RQD used=40% RQD=0-100%
Cecil 83 B=11.25m H= 8.3m D=60 m	0-70	71 No support	159 RR=26% 8WF40 0.9m	---- Class II: BO: locally in crown, L=3m, Sp=2.5m, with occasional wire mesh SC: 5cm in crown	135-290 BO: pattern in crown and walls Sp=1-1.5m, OR SS: light to medium, Sp=1-1.5m, OR SC: $\geq$10cm on crown and walls	RQD used=70% RQD=10-80%

BO = Bolts, SS = Steel Sets, SC = Shotcrete, L = Length (of bolts), Sp = Spacing (of bolts, or steel sets), for Bolts square pattern is assumed.

(1) RQD values are those reported by Cecil ; RQD_a = RQD along axis of tunnel, RQD_v = vertical RQD, RQD used = RQD used in classifcation.

Bieniawski's method predicts considerable support in all cases, which is astonishing since Cecil's cases were known to Bieniawski during the development of the first version (Bieniawski, 1973). Predictions with Deere's RQD method lead to predictions similar to Wickham's and Bieniawski's.

In Table 3.6.2 Cecil's cases requiring support are presented, he reports only support type but no detailed quantities. Thus these cases only serve to indicate variability among predictions and between different methods.

The predicted support quantities and the 'support pressure' or rock loads vary less than in the unsupported case.

Terzaghi's rock load recommendations and the associated support requirements lie now in the middle range of all predictions.

As expected, Barton's method agrees well with actually placed support quantities. With Wickham's et al. method supports are predicted that are in many cases more conservative than Terzaghi's. Also, predictions with Bieniawski's method are conservative compared to the actual placed support, the degree of conservatism, varies from case to case. Predictions with Deere's RQD-Relation are similar to Wickham and Bieniawski.

TABLE 3.6.2 EXAMPLE APPLICATIONS: SUPPORTED CASES REPORTED BY CECIL (1975)

Case	Actual Support	Rock Load (kPa) and Recommended Support					Comments[2]
		Terzaghi	Barton[1]	Wickham	Bieniawski	Deere	
Cecil 24 B=12.5m H= 6.5m D=60 m	Bolts, shotcrete and wire mesh	78-166 8WF31 0.6-1.3m	68-722 Class 27: BO: 1 per m^2 SC: 5-7.5cm	360-480 RR=53-71% 12WF65 0.53m-0.70m	----- Class III: BO: pattern in crown and walls, L=4m, with wire mesh in crown SC: 5-10cm in crown and 3cm in walls	188-406 BO: pattern in crown and walls, Sp=1-1.5m, OR SS: light to medium, Sp=1-1.5m, OR SC: ≥10cm on crown and walls	RQD used=60% RQD_v=0-80% RQD_a=0-100%
Cecil 28 B= 12.5m H= 6.5m D=100 m	Bolts and shotcrete	166-522 8WF40, 0.8m, to 12WF65, 0.4m	69 Class 27: BO: 1 per m^2 SC: 5-7.5cm	328-436 RR=48-64% 12WF65 0.6-0.75m	----- Class III: BO: pattern in crown and walls, L=4m, with wire mesh in crown SC: 5-10cm in crown and 3cm in walls	406-625 BO: pattern, Sp=0.6-1.3m, OR SS: medium to heavy, Sp=0.6-1.3m, OR SC: ≥15cm on crown and walls with bolts	RQD used=40% RQD_a=0-80%
Cecil 53 B= 6.5m H= 4.5m D=60 m	Bolts, shotcrete and wire mesh	96-303 6H25 0.45-1.4m	77 Class 25: BO: 1 per m^2 SC: 5cm	230-260 RR=64-74% 6H25 0.55-0.6m	----- Class IV: BO: pattern in crown and walls, L=4-5m, Sp=1-1.5m, with wire mesh SC: 10-15cm in crown and 10cm in walls SS: light ribs, Sp=1.5m	98-211 BO: pattern in crown and walls, Sp=1-1.5m, OR SS: light to medium, Sp=1-1.5m, OR SC: ≥10cm on crown and walls	RQD used=50% RQD_v=60% RQD_a=60%

BO = Bolts, SS = Steel Sets, SC = Shotcrete, L = Length (of bolts), Sp = Spacing (of bolts, or steel sets), for bolts square pattern is assumed, B= Width, H = Height, D= Depth, Overburden

(1) Support recommendations are given for the most likely condition

(2) RQD values are those reported by Cecil ; RQD_a = RQD along axis of tunnel, RQD_v = vertical RQD, RQD used = RQD used in classification.

TABLE 3.6.2 EXAMPLE APPLICATIONS: SUPPORTED CASES REPORTED BY CECIL (1975) (CONT.)

Case	Actual Support	Rock Load (kPa) and Recommended Support					Comments[2]
		Terzaghi	Barton[1]	Wickham	Bieniawski	Deere	
Cecil 57 B= 5.9 m H= 4.25m D=100 m	Bolts and two shotcrete applications	89-279 6H20 0.3-0.4m	350 Class 34: SC: 15-25cm	200-265 RR=62-82% 6H20 0.45-0.6m	----- Class IV: BO: pattern in crown and walls, L=4-5m, Sp=1-1.5m, with wire mesh SC: 10-15cm in crown and 10cm in walls SS: light ribs Sp=1.5m	192-295 BO: pattern, Sp=0.6-1.3m, OR SS: medium to heavy, Sp=0.6-1.3m, OR SC: $\geq$15cm on crown and walls with bolts	RQD used=40%
Cecil 58 B= 5.9 m H= 4.25m D=90 m	Bolts	89-279 6H20 0.3-0.4m	62-309 No support	47-75 RR=15-24% 6H20 1.5-2.4m	---- Class II: BO: locally in crown, L=3m, Sp=2.5m, with occasional wire mesh SC: 5cm in crown	0-44 None to occasional light support	RQD used=90% RQD_v=90%
Cecil 67 B= 5.9m H= 4.8m D=100 m	Shotcrete	0-74 None to 6H20 1.6m	130 Class 25: BO: 1 per m^2 SC: 5cm	200-250 RR=62-77% 6H20 0.5-0.6m	----- Class IV: BO: pattern in crown and walls, L=4-5m, Sp=1-1.5m, with wire mesh SC: 10-15cm in crown and 10cm in walls SS: light ribs Sp=1.5m	295-413 BO: pattern, Sp=0.66m SS: heavy circular, Sp=0.66m SC: $\geq$15cm combined with medium to heavy sets	RQD used=10% RQD_v=10-50% RQD_a=10-50%

BO = Bolts, SS = Steel Sets, SC = Shotcrete, L = Length (of bolts), Sp = Spacing (of bolts, or steel sets), for bolts square pattern is assumed, B= Width, H = Height, D= Depth, Overburden

(1) Support recommendations are given for the most likely condition

(2) RQD values are those reported by Cecil (27); RQD_a = RQD along axis of tunnel, RQD_v = vertical RQD, RQD used = RQD used in classification.

The Arlberg and Tauern case studies are presented in Table 3.6.3. All sections are located in squeezing ground. Performance in the Tauern and Arlberg tunnel has been monitored in detail. Convergences and contact stresses between ground and shotcrete and tangential stresses in the shotcrete have been measured in both tunnels. In the Arlberg tunnel also tangential stresses and convergences of the final liner were available. Note that the radial stresses vary over a considerable range. In addition, in the Tauern as well as the Arlberg tunnel longitudinal contraction slots were left in the shotcrete to allow for deformation. The load taken by the bolts is not or only partly included in the measured stresses. Thus, the relation of actually measured stresses (radial and tangential) to "support pressures" is not clear. This further illustrates the problematic aspects of support pressure relations even if they are based on 'measured stresses'.

With Terzaghi's method very heavy steel support is predicted, the steel sets have to be placed side by side.[1)] One additional point with respect to Terzaghi's rock load may be made. If one assumes that all actually placed support elements (reinforced shotcrete, bolts and steel sets) are loaded to full capacity a theoretical capacity of the support system against uniform outside pressure may be computed. This 'theoretical capacity' is lower than the "best

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[1)] Proctor and White's table for the design of steel sets includes 12WF65 as maximum size. However, here even higher capacity steel sets would be required.

TABLE 3.6.3a EXAMPLE APPLICATIONS: ARLBERG TUNNEL

CASE	ACTUAL SUPPORT AND PERFORMANCE		ROCK LOAD (kPa) AND RECOMMENDED SUPPORT				
			TERZAGHI	BARTON (**)	WICKHAM	BIENIAWSKI	DEERE
Arlberg ATW 50 B=11m H=11m D=40m	<u>Support</u> Shotcrete: 15cm Wirefabric: 3.12 kg/cm^2 Steelsets: TH 21, 1.5m Bolts per meter: 10x25tx4 to 6 m (1 Bolt per 2.5 m^2) Interior Liner <u>Performance</u> Initial Support: convergence = 50mm Interior (Final) Liner: Theoretical thickness = 25cm	Best	605 12WF65 0.5m	150 Class 31: SC: 10cm BO: 1 per m^2 <u>OR</u> C: 44cm BO: 1 per m^2	170 RR = 29% 12WF65 1m	- Class IV: SC: 10-15cm SS: 1.5m BO: 1 per 1-2.5 m^2, L=4-5m	- BO: 1 per 0.36-1.4m^2, <u>OR</u> SC: 15cm with bolts
		Most Likely		1160 Class 35: SC: 50cm BO: 1 per m^2 <u>OR</u> C: 100cm BO: 1 per m^2	410 RR = 69% 12WF65 0.7m	- Class V: SC: 15-20cm SS: 0.75m BO: 1 per 1-2m^2, L=5-6m	- BO: 1 per 0.36-1 m^2, <u>OR</u> SC: 15cm or more and closed invert with heavy steel sets
		Worst	1160 (*) Sets 12WF65 side by side, insufficient	1840 Class 38: SC: 83cm BO: 1 per m^2 <u>OR</u> C: 120cm BO: 1 per m^2	490 RR = 83% 12WF65 0.6m		
Arlberg ATW 350 B=11m H=11m D=140m	<u>Support</u> Shotcrete: 15cm Steelsets: TH21 Bolts per meter: 16x25tx6.5m (1 bolt per 1.5m^2) Interior Liner: 25cm(theo- <u>Performance</u> Initial Support: Convergence=100mm Interior (Final) Liner: Theoretical thickness = 25cm Tangential Stress= 1.4MPa Convergence=3mm	Best	605 12WF65 0.5m	89 Class 27: SC: 7.5cm BO: 1 per m^2,<u>OR</u> SC: 20cm BO: 1 per m^2	440 RR = 71% 12WF65 0.65m	- Class V: SC: 15-20cm SS: 0.75m BO: 1 per 1-2m^2, L=5-6m	- BO: 1 per 0.36-1 m^2, <u>OR</u> SC: 15cm with bolts
		Most Likely		620 Class 35: see ATW50	490 RR = 83% 12WF65 0.56m		- BO: 1 per 0.36-1 m^2 SC: 15cm and closed invert with heavy steelsets
		Worst	1160 (*) Sets 12WF65, side by side, insufficient	1050 Class 38: SC: 83cm BO: 1 per m^2 <u>OR</u> C: 120cm BO: 1 per m^2			

SC = Shotcrete SS = Steelsets BO = Bolts C = Concrete

B = Width H = Height D = Depth, Overburden

(*) The largest steelset in the "Proctor & White" tables is a 12WF65, a larger set may be designed. Here an actual design was not performed.

(**) In Barton's method a differentiation is made between the shear zones and squeezing ground leading to different support types in the same classes. The 'upper' dimensions are obtained when assessing it as 'shear zones', the lower ones as squeezing ground.

TABLE 3.6.3b EXAMPLE APPLICATIONS: TAUERN TUNNEL

CASE	ACTUAL SUPPORT AND PERFORMANCE		ROCK LOAD (kPa) AND RECOMMENDED SUPPORT				
			TERZAGHI	BARTON	WICKHAM	BIENIAWSKI	DEERE
Tauern TTN 1739 B=11m H=11m D=865m	Support Shotcrete: 15cm Steelsets: TH21,1.5m Bolts per meter: Initial: 13.4x25tx4m Additional: 5x25tx6m 2.6x25tx9m Performance Convergence = 270mm Radial stress between shotcrete and ground ≈ 0 to 1500 kPa	Best	1160 (*) Sets 12WF65, side by side, insufficient	43 Class 23: SC: 8cm BO: 1 per 1-2m^2	260 RR = 44% 12WF65 1.06m	Class IV: SC: 10-15cm SS: 1.5m BO: 1 per 1-2.5 m^2, L=4-5m	- BO: 1 per 0.4-1 m^2 SC: 15cm
		Most Likely		110 Class 31: C: 44cm BO:1 per m^2	410 RR = 71% 12WF65 0.66m	- Class V: SC: 15-20cm SS: 0.75m BO: 1 per 1-2m^2, L=5-6m	- SC: 15cm or more combined with closed invert with heavy steelsets
		Worst	2480 (*) Sets 12WF65, side by side, insufficient	4080 Class 38: C: concrete arch 120cm BO: 1 per m^2	430 RR = 72% 12WF65 0.65m		
Tauern TTN 1934 B=11m H=11m D=885m	Support Shotcrete: 15cm Steelsets: TH21,1m Bolts per meter 18x25tx6m 6x60tx6m Invert bolts: 4x25tx6m Performance: Convergence = 35mm Radial stress = 0-800 kPa	Best	1160 (*) Sets 12WF65, side by side, insufficient	173 Class 18: SC: 3cm BO: 1 per 1-2m^2	210 RR = 36% 12WF65 1.3m	Class IV: SC: 10-15cm SS: 1.5m BO: 1 per 1-2.5 m^2, L=4-5m	- BO: 1 per 0.4-1 m^2 SC: 15cm
		Most Likely		320 Class 35: C: 100cm BO: 1 per m^2	310 RR = 51% 12WF65 0.9m	- Class V: SC: 15-20cm SS: 0.75m BO: 1 per 1-2m^2, L=5-6m	- SC: 15cm or more combined with closed invert with heavy steelsets
		Worst	2480 (*) Sets 12WF65, side by side, insufficient	590 Class 35: C : 100cm BO: 1 per m^2	430 RR = 72% 12WF65 0.64m		
Tauern TTN 2196 B=11m H=11m D=885m	Support Shotcrete: 15cm Steelsets: TH21,1m Bolts per meter: 20.4x25tx6m 3.6x60tx6m Invert bolts: 4x25tx6m Performance Convergence = 110mm Radial stress=150-700kPa	Best	1160 (*) Sets 12WF65, side by side, insufficient	22 Class 22: SC: 7.5cm	260 RR = 44% 12WF65 1.06m	Class IV: SC: 10-15cm SS: 1.5m BO: 1 per 1-2.5 m^2, L=4-5m	- BO: 1 per 0.4-1 m^2 SC: 15cm
		Most Likely		212 Class 31: C: 44cm BO: 1 per m^2	390 RR = 67% 12WF65 0.7m	- Class V: SC: 15-20cm SS: 0.75m BO: 1 per 1-2m^2, L=5-6m	- SC: 15cm or more combined with closed invert with heavy steelsets
		Worst	2480 (*) Sets 12WF65, side by side, insufficient	1620 Class 38: C: concrete arch 120cm BO: 1 per m^2	410 RR = 69% 12WF65 0.6m		

SC = Shotcrete SS= Steelsets BO = Bolts C = Concrete

B = Width H = Height D = Depth, Overburden

(*) The largest steelset in the "Proctor & White" tables is a 12WF65, a larger set may be designed. Here an actual design was not performed.

case" rock load predicted with Terzaghi. However, in reality the shotcrete is not stressed to full capacity (contraction slots). Thus the rock load predicted with Terzaghi's method is even more conservative.

Predictions using Barton's method show large variations for both the direct ground-support and support pressure relations. For the assessment of the Arlberg- and Tauern tunnels Barton's et al. assessment of the Straight Creek Tunnel was studied in detail since the ground conditions in the Arlberg and Straight Creek tunnels are comparable (Einstein, 1977). In assessing J_n the question arose if it should be selected for completely crushed rock or the number of joints if shear zones are present. The "best" case reflects the number of joints and the worst case reflects the crushed rock. In the Arlberg tunnel cases the same Q was obtained using the SRF for shear zones and squeezing ground. Since the overburden is rather shallow, it could not be positively stated which would be the governing mechanism. The Q-system predicts different support types for two ground types, thus both are reproduced on Table 3.6.3a.

Bieniawski's support requirements come very close to the actually placed ones - unsurprising since the NATM cases are a part of his data base.

Wickham's et al. method predicts support which is substantially less than predicted with Terzaghi's rock load, however, a direct comparison is not possible. Their method is not too sensitive to changes in the assessment of the ground, the variation of the rock load never exceeds a factor of 3, as a distinct advantage. However, all these cases are in squeezing ground which is not treatable with the RSR number (as mentioned previously, they do not give a criterion how squeezing ground conditions should be determined and thus where their method is not applicable).

Deere's et al. method yields prediction in acceptable range.

3.6.2 Example Applications by Other Authors

Houghton (1976) used Wickham's et al., Barton's and Bieniawski's method in the Kielder test tunnels. He concluded that Bieniawski's method was easier to apply than Barton's. He found also that "classifications need to be interpreted in the context of the local geological environment" in other words, a subjective assessment is necessary. According to Houghton, the predictions with all classification systems were consistent. By correcting for the errors in Houghton's paper (Cockcroft, 1976) Barton's et al. method results in lower support quantitite than the others, but the actually placed support has not been compared to the predicted one.

Jethwa et. al. (1978) compared support pressures observed in a 5.6m diameter tunnel with predictions according to Barton and Terzaghi. The predicted ranges usually bracket the observed values from which they deviate between 50 to 100%.

C.M. Barton (1977) of the CSIRO in Australia compared empirical methods with actually placed support in mines (stopes and cross-cuts). Wickham's et. al. method predicts supports that are in agreement with the actually placed one. Bieniawski's RMR method predicts conservative stand-up times as well as conservative support quantities. Barton et. al. (1974) Q-system gave support predictions that agreed for the better ground conditions, however, in the fractured zones a very low Q was estimated which was considered not applicable.

A comprehensive study was performed in New Zealand by Rutledge and Preston (1978). Measured support loads were compared to predictions according to Barton (Fig. 3.6.1) and Wickham et. al. (Fig. 3.6.2). Both predictions had a large scatter and both overpredict support loads. Support loads smaller than 20 kPa were considered blocking loads by Rutledge et. al. Barton's method leads to overpredictions by a maximum factor of 35. However, there are also some cases where the actually measured rock load slightly exceeded the predictions (blocking loads excluded) with Barton's method. Wickham et. al. method overpredicts by a maximum factor of 15; no measured loads exceeded the predicted ones. (If the cases are excluded where only 'blocking loads' have been measured; rock loads up to approximately 20 kPa maximum.) Based on this finding, Rutledge and Preston propose, to modify Wickham's rock load equation (Fig. 3.6.3). This modification results in roughly half the rock load according to Wickham et. al. (1974).

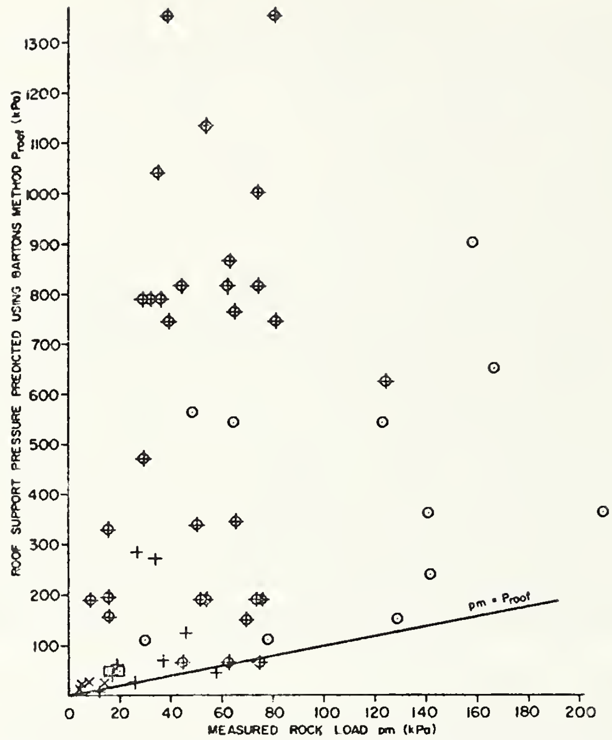


FIGURE 3.6.1 COMPARISON OF MONITORED AND PREDICTED ROCKLOADS WITH BARTON ET AL METHOD (FROM RUTLEDGE AND PRESTON,1978)

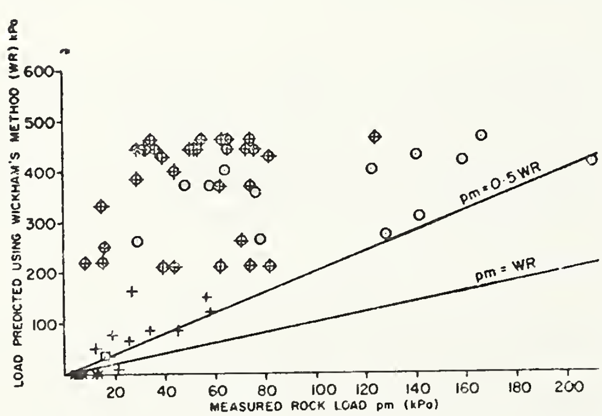


FIGURE 3.6.2 COMPARISON OF PREDICTED AND MONITORED ROCKLOADS WITH WICKHAM ET AL METHOD (FROM RUTLEDGE AND PRESTON,1978)

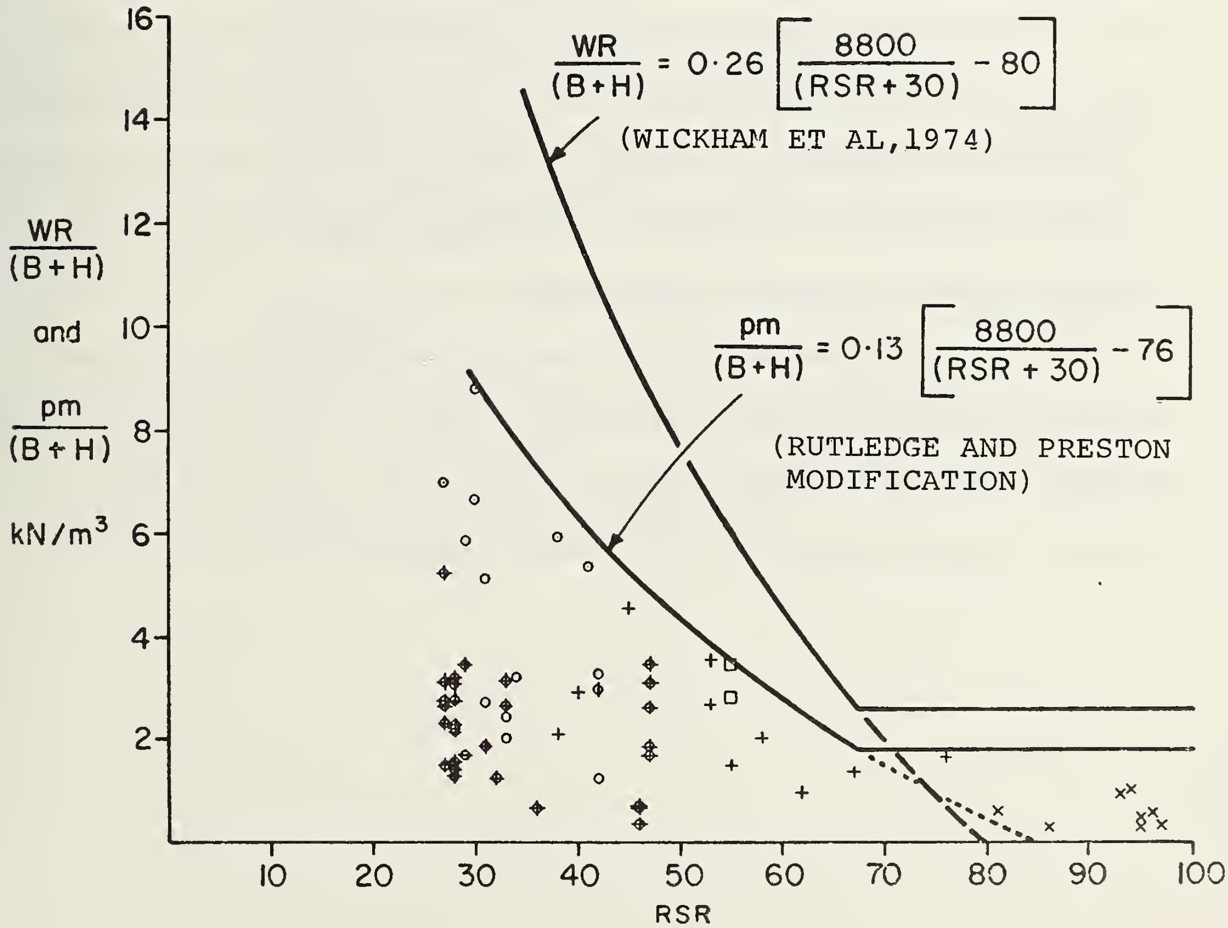


FIGURE 3.6.3 RUTLEDGE AND PRESTON'S PROPOSED MODIFICATION OF WICKHAM ET AL ROCK LOAD EQUATION (FROM RUTLEDGE AND PRESTON,1978)

3.6.3 Example Applications - Conclusions

The most obvious negative conclusions are the large variations of predicted support pressures within and between different methods. In some cases, some methods lead to overpredictions in other cases the same methods lead to more conservative values. (Moreover support pressures are often not representative due to the different 'mechanisms' for different types of support.) Methods that directly relate support quantities to ground conditions tend also to over-predict actual support requirement.

Methods were more easily applicable for cases within the range of the data base. None of the five employed methods gave consistent predictions. Thus, none of the methods can be judged to be better than another.

3.7 ADVANTAGES - DISADVANTAGES OF EXISTING EMPIRICAL METHODS

3.7.1 Introduction

As discussed in Section 2, empirical methods should fulfill the following requirements:

- Economy and Safety
- General Applicability and Robustness
- Practicality (Readily Determinable Parameter)
- Model Accuracy, Representative Modeling and Completeness
- Recognition of Subjective Character of Empirical Methods

It will now be shown how well these requirements are fulfilled by empirical methods in general and by the particular methods reviewed in preceding sections.

3.7.2 Economy and Safety

"Economy", as discussed here refers to the most economical support under the particular circumstances of a project. As extensively discussed in Volume 4 of this DOT-report series (Steiner et al., 1979), it is difficult to separate economy of the support from the overall economy in constructing a tunnel. A less economical tunnel support may still result in a more economical tunnel if other factors (e.g., excavation) are taken into account. The construction procedure including placement of tunnel support depends to a large extent on the entire design-construction process that also includes bidding and contracting practice. (See Steiner et al., 1979, Vol. 4 of this report series.)

Most empirical methods studied in this report do not require that the user consider the complex construction process in the support selection, rather they attempt to design tunnel supports in a somewhat isolated manner. However, the base cases from which the methods were developed and thus the corresponding support quantities may reflect the overall construction process.

As noted by Dowding et al. (1976), the selection of initial supports is often governed by factors having nothing to do with required capacity; for instance, material availability. Similar statements have been made by Cecil (1970, 1975).

From the point of view of safety there is an understandable tendency to be conservative, i.e., to place more support than absolutely necessary, resulting in overdesigned supports. Final supports are usually overdesigned by again being conservative, and often by not considering the effect of the initial support.[1)] All of this would be unimportant if the degree of overdesign were known, but it is not. Existing support both initial and final thus generally have capacities greater than needed.

As an example of such inherent conservatism consider the use of Terzaghi's rock load recommendations. Terzaghi quotes ranges of rock loads, the design of a particular case may, however, be based on the maximum load for the class, leading to overdesign. If such cases are then used to develop an empirical method, the conservative bias will be included. The method of Wickham et al. (1974) and the study by Rutledge et al. (1978) applying Wickham's et al. method seem to confirm this inherent overdesign (Section 3.6).

Although tending to overdesign, empirical methods are not necessarily safe. This is because the degree of overdesign is usually not known and because a method may not have been based on cases representative of the particular application (Section 3.7.3). Economy in support design is thus only achieved if:

[1)] The thickness of the final liner may also be governed by other factors; a certain thickness of concrete is required such that it can be placed behind a formwork. This theoretical thickness is further increased due to overbreak.

1) Methods are applied to situations which are similar to base cases, that were designed and constructed close to the possible optimum (e.g., a large number of base cases in a similar setting may give an indication of optimum design).

2) The support is adapted to ground conditions under consideration of the construction process.

In this sense (1) may be approached by the Q-System in the Scandinavian setting, (2) by the NATM.

3.7.3 General Applicability and Robustness

The detailed discussions and the example applications have shown that none of the methods is generally applicable. Also none of the methods is robust as demonstrated by the example applications in that slight parameter variations may lead to significantly different support requirements.

The limited applicability is inherent in the derivation of empirical methods, they rely on a limited number of base cases which cannot encompass all possibilities. Table 3.7.1 indicates which methods describe the base cases, the type of base cases and which methods explicitely mention limitations. In addition specific limitations in applicability and robustness of each method (where possible) will be discussed briefly.

Terzaghi's and Stini's method both have a reasonably wide range of application as far as geologic conditions are concerned. They are applicable for hard intact rock to heavily squeezing ground that was encountered primarily in the eastern Alps. However, both methods are based on interpretations of Bierbaumer's rock load measurements. Bierbaumer observed rock loads in timber supported tunnels that were usually built by excavating first a pilot drift which was followed by other drifts until the full cross-section was excavated. During the stagewise enlargement, displacements of unknown magnitude may have occurred. Therefore

TABLE 3.7.1 COMPARISON OF EMPIRICAL METHODS

METHOD	BASE CASE REPORTED	TYPE OF BASE CASES	LIMITATION MENTIONED 1)
Ritter	No	---	No
Kommerell	No	---	No
Bierbaumer	Yes	Tunnels in moderately jointed to squeezing rock; timber support and stagewise excavation.	No
Terzaghi	No	Tunnels in hard intact to squeezing ground; derived from timber supported tunnels.	Yes
Stini	Yes	Tunnels in hard intact to squeezing ground; timber supported, with stagewise excavation.	No
Straight Creek	Yes	Singular case with heavy steel support to multiple drifts backfilled with concrete.	No
NATM	Limited 2)	Hard intact to heavily squeezing ground with shotcrete, rockbolt and steelset support combinations.	Yes
Deere: RQD-Rockload	Limited	Measurements of Rockloads in steel supported tunnels.	Yes
Deere: Wedge	No	For the treatment of instable wedges.	Yes
Lauffer	No	Singular case study of 5.5m tunnel. (Base case not reported in the paper where the chart is shown.)	No
Deere: RQD	Limited	Primarily steel supported tunnels.	Yes
Deere: Sao Paulo	Limited	Hard intact to strongly jointed rock; support requirements described in general terms only.	Yes
Wickham	Yes	Steel set supported tunnels.	Yes
Bieniawski	Limited	---	Yes
Barton	Yes	Mostly tunnels in Scandinavia with shotcrete and/or rockbolt support; Precambrian to Silurian rocks.	Yes
Louis	No	---	Yes
Franklin	No	---	Yes

1) The degree to which limitations in these methods are mentioned varies, for specifics, see text.

2) The NATM classification is to some extent a condensed report of previously built cases that include many facets of tunnel construction, thus the base cases are implicitly reported.

these rock load recommendations are probably not entirely appropriate for the different procedures that are used today (full face excavation, bolting and shotcreting, possibly even steel ribs) and should thus be applied with the necessary caution.

Deere's RQD - Rock Load correlations suffer from the fact that similar RQD's may represent different combinations of discontinuity.

The NATM classification has been applied to a wide variety of ground conditions primarily in the eastern Alps. Most recent data come from two-lane highway tunnels. However the method has also been applied to smaller diameter (hydropower) tunnels and to large caverns (e.g., Waldeck II). NATM classes are developed for individual cases by the designers of the respective tunnels taking into consideration the expected ground conditions. If the method is correctly used ground classes are adapted as observations are made. In this sense the method is generally applicable, but cannot be transferred to new conditions by outsiders if conditions in particular dimensions are not the same. No explicit procedure now exists for altering support quantities and the excavation-support procedure for different dimensions.

Lauffer's stand-up time unsupported span classification has been developed from one case study with specific construction procedure, dimensions and ground conditions. Its applicability to other cases is thus questionable.

The direct RQD-support correlations are based on a limited set of data, the limits of the methods have been clearly expressed. The method considers different construction methods but not the detailed construction sequences. It is not robust as shown in the axample applications. (The wide variety of cases to which the method has been applied and the associated adaptations should make it possible to broaden its applicability.)

Deere's descriptive classification (Sao Paulo) considers RQD and other relevant ground parameters; it is similar to the NATM classifications and comes very close to being generally applicable. This classification is intended to be used as a first general classification step and leaves thus many decisions to the user. Deere et al. (1974) recomment to supplement it with a more detailed system like the direct RQD-support relations.

Wickham et al. state their method is not applicable in squeezing rock. However, it is not stated how squeezing ground can be identified in their classification system. The example applications have shown that Wickham's et al. method is not sensitive to small parameter variations, yet it is sensitive to larger variations. It is thus reasonably robust.

The applicability of Bieniawski's method is limited as shown by the example applications. The range of applicability cannot be determined since its base cases are unknown. The fact that the method has gone through five stages of development indicates that it has been continuously improved and its applicability extended. However, no detailed limits of applicability are given.

Barton's method is also limited, because it is primarily based on Scandinavian cases. However, in contrast to other methods most base cases are well documented. The user can get a good sense of the range of applicability by studying the original NGI report (1974) or Cecil's (1970, 1975) thesis. On the other hand the limitations of the method are not separately stated; the user has to assume that it is limited to the range of base cases. As shown by the example applications the method is sensitive to parameter variations.

Louis' and Franklin's methods are strictly applicable to tunnels of 4 to 10 in diameter at 30 to 300 in depth as mentioned by Franklin in his review comments. It was not possible to test their relation.

A final comment regarding all methods concerns the difference in applicability between 'direct support relations' and 'support pressure relations'.

Relations that directly relate ground conditions to and support requirements can by definition only be used with a similar support system. This difficulty is often mitigated by including correction factors for different support types. Support pressure relations attempt to circumvent this problem by relating support pressures (or rock loads) to ground conditions. The support pressures are then considered to be applicable for the design of any support system. This assumption neglects the fact that support itself affects support pressure, i.e., different supports in identical ground conditions are subject to different loads. Thus, only "support pressure relations" that include correction factors for different supports can in this sense be widely applicable.

In summary, no method is generally applicable. Some methods clearly show their derivation and provide a detailed description of the base cases thus providing information on the applicability range, while others do not and do not give the cases. No method, however, is accompanied by a complete listing of limitations and detailed limitation criteria. A careful background study by the user is thus required before application of any method.

3.7.4 Practicality, Readily Determinable Parameters

In each method one or several qualitative or quantitative parameters have to be observed or measured. The practicality of a method depends greatly on the ease with which such parameters can be determined within a given range of experience and given time frame.[1)]

Specific comments relating to individual methods are: Terzaghi's and Stini's methods rely on detailed ground descriptions that include some behavioral descriptions. Terzaghi's description is quite abstract, it thus requires more experience than Stini's whose description is more detailed and may be easier to use. For the design phase both methods rely on only a few parameters that may be estimated from surface geology or borehole information by an experienced user. During construction, a user can assign a ground class based on observed support behavior; this requires previous experience.

The NATM classification relies heavily on parameters that describe ground behavior, thus prediction of ground classes in the design phase requires an experienced designer. During construction, the parameters can easily (i.e., within the given time frame) determined by an experienced designer including also observed ground behavior, an uninitiated user may not be able to do this.

[1)] With respect to time frame one has to distinguish between the design and construction phases. During the design phase information is limited but time is not, during construction detailed information is available, however, time is limited.

RQD is easily determined for preliminary investigations from borings even by personnel not familiar with tunneling. During construction the user should not only rely on RQD but monitor the performance and update the RQD-support relations with this experience.

Deere, Merritt, Cording's 1974 classification is a general classification system with many parameters described in general terms only, thus requiring substantial experience. Deere et al., however, recommend to supplement this general system with a more detailed one (e.g., RQD relations) in the actual application.

Lauffer's stand-up time has to be estimated and may require substantial experience, in addition it may not be transferrable to other projects. During actual construction it may, however, be possible to develop a stand-up time relation for the particular tunnel as experience accrues.

Wickham's et al. method does not require much experience. During preliminary phases not all the information required may be available (e.g., water inflow). During the construction phase it seems possible to determine all three parameters within the limited time and the available information.

With Bieniawski's method little experience is required for parameter determination. During preliminary investigations not all parameters may be available and would thus have to be estimated. During construction time restraints may be a limiting factor.

With Barton's method, experience is required in assigning some of the parameters, in particular the stress reduction factor, and to some extent regarding the selection of J_r and J_a. During preliminary investigations not all parameters may be known and would thus have to be estimated. During construction, time constraints may limit the use of the method, not so much regarding measurement of the parameters but regarding ground assessment with the relatively complex set of tables and notes.

Louis' and Franklin's method rely on parameters that may be determined easily from boring and in tunnels with the exception of the major principal stress which has to be estimated. The methods are intended for use during preliminary investigations. During construction both propose to monitor the performance and to rely less on the classification although the parameters could be determined within a limited time frame. Franklin in his review comments stated that the parameters have been determined within a time frame of 10 min. between blasting and shotcreting operations in the Rubira tunnel.

In summary, some methods use parameters that can be easily obtained by a user with little experience, other methods rely heavily on experience, including some of the newer methods. The more detailed methods require information that is often not available during the design phases, while during the construction phase, time constraints may limit the determination of input parameters.

Naturally, the fact that experience is required is not a disadvantage in itself. The comments here are meant to indicate that even a person experienced in tunnel design and construction may need extensive working exposure in order to apply some methods while other methods would require less specific method oriented practice.

From the point of view of practicality, the RQD-Relations and Louis' and Franklin's methods probably rank highest.

3.7.5 Model Accuracy, Representative Modeling and Completeness

Model Accuracy

The accuracy of empirical methods depends on achieving an optimum between model detail and parameter uncertainty. Models can be made more detailed by increasing the number of parameters or by using different models for different applications. Some methods have only a few parameters while others include many parameters. Methods with more parameters may provide a more accurate model, but only if the parameters represent the influencing factors correctly, a point that will be discussed alter.

On the other hand parameter determination involves uncertainty, and for the usually limited set of data, parameter uncertainty may increase with the number of parameters. (If the parameters are based on independent data, however, there need be no increase in uncertainty.) Because the influencing factors or at least their relative contribution are not known at the present state of the art and because the available data for parameter determination are limited, it seems preferable to select a method with a small number of parameters. In other words, at the present time, it seems more appropriate to opt for reduced parameter uncertainty than for increased model complexity. With increasing knowledge of ground-structure behavior, a shift toward greater model accuracy will become possible. Naturally, if a method has been developed from and is applied to cases with a relatively narrow range of ground and support characteristics one may successfully use more complex methods at the present time.

Representative Modeling and Completeness

A model is representative if it includes the important influencing factors and represents them correctly; it is complete if it includes all influencing factors. A representative model does not have to be complete and a complete model does not have to be representative. The complexity of the problem makes it difficult to judge whether a method is either representative or complete. (As a matter of fact empirical methods have been developed because of this complexity.) For practical purposes it is important that a method be representative, but it need not be complete.

The representativeness of a model can be judged in three ways, one direct and two indirect:

- Comparison of predicted and measured support pressure.
- "Satisfactory" performance.
- All significant factors (as they are known at the particular state of the art) correctly considered.

A comparison of measured and predicted support pressures is appealing because hard information can be compared. Measurements do involve, however, an interpretation model of a complexity similar to the model underlying the empirical method. Measured support pressures are thus not easily verifiable.

For instance, some measurement interpretations assume superposition of capacities of different support systems (bolt capacity, hoop stress capacity for shotcrete and steelsets) to arrive at a 'support pressure'. This approach is questionable since in reality, different support components are stressed to different levels.

Satisfactory performance, the second way to judge representativeness is difficult to define particularly since most supports are overdesigned. Only a deformation criterion can provide information on satisfactory performance. However, deformation measurements are seldom made now (with the exception of NATM) and also a deformation criterion may be of limited usefulness because deformation is strongly related to the particular situation (ground conditions, overburden, support system, tunnel size). Experience with the NATM in squeezing ground (e.g., deformations after a certain tunnel advance) has shown that a deformation criterion can be established yet requires calibration over several hundred meters with similar geologic conditions. A collection of limit deformations (deformations related to satisfactory performance) could be developed from many cases where performance was monitored. With such a collection of limit deformations it would become possible to judge empirical methods.

Consideration of all significant factors is the third way to judge representativeness of a method. A representative method considers all the important factors as they are known at the present state of the art and weighs them also accordingly.

The following is a list of factors that a method should consider to be reasonably representative:

- Ground Conditions
- Construction Procedure

These factors can be further divided into more detailed ones:

Ground Conditions:

- Lithology
- Discontinuities:
  - Joints
  - Shear zones
- Hydrologic Conditions
- Stress State:
  - Overburden
  - Lateral Stresses

Construction Procedure:

- Type:
  - Drill and Blast
  - TBM
- Sequence:
  - Full Face
  - Heading and Bench
  - Pilot tunnel

The detailed description of each of the factors may involve several parameters, e.g., discontinuities require a description of orientation (strike and dip), spacing, type and characteristics of individual discontinuities (strength, waviness, persistence). The above list of important factors is by no means complete nor should it be considered to be representative for every application case. In the individual case other factors might have to be accounted for. The number of factors considered in a method may indicate its completeness, but not its representativeness. Representativeness of a method can only be judged by including the relative significance of the factors considered. If a method considers many factors of minor importance but omits few of major improtance it is not representative. In contrast a method that considers the major factors (even if few in number), but omits those of minor importance, is more representative.

A comparison of methods based on representativeness is difficult and would require the knowledge of the weight given to each factor. However, this weight is not known and it must be assigned by the user in the individual case.

In order to judge the representativeness of different methods, several factors that are considered important are listed in Table 3.7.2.

The table intends to assess whether, and to what extent, particular methods consider factors that <u>may</u> be representative. It can however, give only a first indication of which method may be more representative in the general case.

TABLE 3.7.2 CONSIDERATION OF FACTORS IN EMPIRICAL METHODS

METHOD	SECTION	CONSTRUCTION SEQUENCE 1)	JOINTS					SHEAR ZONES	STRESS STATE 2) (OVER-BURDEN)	INTACT STRENGTH 2)	WATER
			NUMBER OF SETS	SPACING OF SETS	RESISTANCE	ORIENTATION	INTERACTION OF SETS 3)				
Ritter	3.2.1	No	No	No	No	No	No	No	lim.	lim.	No
Kommerell	3.2.2	lim.	No	No	No	No	No	No	No	No	No
Bierbaumer	3.2.5	lim.	lim.	lim.	lim.	lim.	No	No	lim.	lim.	Yes
Terzaghi	3.2.3	lim.	lim.	lim.	lim.	Yes	Yes	lim.	lim.	lim.	Yes
Stini	3.2.4	lim.	lim.	lim.	lim.	lim.	No	lim.	lim.	lim.	Yes
Straight Creek	3.3.1	lim.	lim.	lim.	lim.	lim.	No	lim.	lim.	lim.	lim.
NATM	3.3.2	Yes	lim.	lim.	lim.	lim.	Yes	lim.	lim.	lim.	Yes
Deere: RQD-Load	3.4.1	lim.	lim.	Yes	lim.	No	lim.	No	No	lim.	lim.
Wedge	3.4.1	No	Yes	Yes	Yes	Yes	Yes	Yes	lim.	No	No
Lauffer	3.5.1	lim.	No	No	No	lim.	lim.	No	lim.	No	No
Deere RQD	3.5.2	lim.	lim.	Yes	lim.	No	lim.	No	No	lim.	lim.
Deere Merritt Cording	3.5.2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	lim.	Yes	Yes
Wickham	3.5.3	lim.	No	Yes	Yes	Yes	No	lim.	No	lim.	Yes
Bieniawski	3.5.4	Yes	Yes	Yes	Yes	Yes	No	lim.	lim.	Yes	Yes
Barton	3.5.5	No	Yes	Yes	Yes	lim.	lim.	Yes	Yes	lim.	Yes
Louis Franklin	3.5.6 } 3.5.7	Yes	No	Yes	No	No	No	No	Yes	Yes	Yes

1) Construction Effects include also the type of construction procedure that should be used.
2) Some methods do not consider stress state and intact strength explicitely, but both parameters are partly reflected in a combined parameter to distinguish such methods from those explicitely considering the factors, they are referred to as 'limited'.
3) Interaction of set means that two or more joint sets act together resulting in a more adverse behavior than any of the sets alone; it is considered if the joint sets are specifically characterized in the method.

lim. = limited

The relative weighting with which the factors should be considered is not generally known nor is it known how some of the methods assign such weights. What can be done with Table 3.7.2 is to judge representativeness of a method of one of the factors has a much greater weight than the others, as will be shown with two samples.

In some cases the <u>construction procedure</u> (sequence) may be important, particularly in difficult ground conditions. In such a case the factor construction procedure ought to be weighted accordingly. Methods that consider construction sequence in detail are:

- NATM
- Deere-Merritt-Cording
- Bieniawski
- Franklin/Louis

Yet even within these four classification systems the detailed level of discussion of the construction sequence varies. The NATM describes construction sequences in detail and uses the observed performance during the excavation support sequence as a classification criterion. Deere-Merritt-cording address the problems associated with constru-tion but do not use the construction sequence as classification criterion. In contrast, Bieniawski's method and Franklin/ Louis address the construction procedure in lesser detail.

In other cases, rock with intersecting shear zones might be encountered. In this case the detailed construction procedure may be of little interest but the support system has to be designed such as to stabilize potentially unstable rock blocks. In this case the wedge method would have to be chosen as the representative method.

From the above examples it is evident that no single method can be representative in all cases and also that a method has to be selected by the user on an individual basis.

3.7.6 Subjective Character of Methods

Because models are abstractions of reality and because parameter estimates involve uncertainties, no procedure for establishing support requirements can be "objective"; they are all substantially subjective. Subjective aspects in some methods are obvious to the user but not in others. In cases where the user has to make a subjective assessment, he will be aware of the subjectivity of the method. In other cases, however, the subjectivity has been shifted from the user to the creator of the method. This shifting of the subjective aspects can be best illustrated by the "extreme cases", Terzaghi's and Deere's (RQD) methods. With Terzaghi's method the user has to subjectively assign a ground class and rock load to a ground condition. With Deere's method the user has to subjectively assign a ground class and rock load to a ground condition. With Deere's method the user basically determines RQD, but the relation between RQD and rock load is to a large extent the subjective assessment of the developers. The other methods fall somewhere between these two extremes.

Reviewing the subjective character of all methods one finds further: Both Stini's and Terzaghi's methods involve also subjective input by the creator since they rely partly on Bierbaumer's measurements that they have reinterpreted. The NATM requires substantial subjective assessment from the user. The stand-up time estimate in Lauffer's method has to be made by the user, while the stand-up time span diagram is based on Lauffer's subjective assessment.

The original RQD relations have been shown before to represent the extreme case of creator subjectivity. The user's subjective input is required only as far as interpretation of borelogs and extrapolation between boreholes of concerned. The wedge method and particularly the Deere, Merritt,Cording 'descriptive classification' require substantial judgement on the part of the user.

In Wickham's method judgement on part of the user is necessary to determine the parameters. A further subjective decision by the user is the exclusion of squeezing ground conditions. An interesting aspect, which applies to most other methods, but has been specifically mentioned by Wickham et al. is the subjectivity in the development. The 'trial and error' procedure used to determine the scales and relative value of parameters is a prime example of creator judgement. They selected a limited set of the many possible combinations and made the final selection based on the best fit between predicted and actual support requirements. Since there are many combinations of parameter scales and relative ratings other combinations could have been as satisfactory; the particular selection involved their judgement.

In Bieniawski's method a subjective assessment on part of the user is necessary only in case of multiple sets of discontinuities. All other parameters are given on a measurable scale. In developing his method, Bieniawski used his judgement to assign the various scales and relative ratings of parameters. This process is particularly evident from the modifications leading from one version to the next.

In Barton's method several parameters require a subjective approach on part of the user. The selection of the 'significant' discontinuity for the determination of J_r and J_a is a subjective process, as is the determination of the stress reduction factor. Barton et al. employed a subjective procedure similar to that by Wickham et al. and Bieniawski to develop their method.

In Louis' and Franklin's method the parameters are measurable with the exception of the stress state which might have to be estimated. Subjective judgement on part of the user is required in shifting the class boundaries, if the method is applied outside the range of applicability.

All empirical methods are subjective, due to the inherent subjectivity in characterizing geology and construction processes. Subjectivity cannot be avoided and may only be shifted from one part of the method to another. On one extreme are methods where the user has to subjectively determine geologic parameters and possibly their relation to tunneling features and processes. The other extreme are methods where subjective decisions are made by the developer of the method or by the designer of the calibrating cases. The fact that the subjectively derived methods are verified by test cases does not make them objective.

Verification by a case means that the ground conditions of the test case fall within the range of the base cases. For another case that lies outside the base cases of the method the applied empirical method may fail and provide either non-conservative or overly conservative results which may however be unknown to the user. Alternatively the user might attempt to subjectively assess the degree of conservatism and to adjust the required support accordingly.

To summarize and restate, methods that seem to require only objective parameter state estimates do also incorporate subjectivity, but simply at other levels. It is possible that relying on the subjective assessment of an experienced creator of an empirical method is a better apporach than to the rely on the judgement of the particular user. No generally applicable statement would be appropriate as to which approach is better. In circumstances where the user has substantial experience relying on his own judgement may be better and vice versa. What is important however, is the fact that subjectivity enters the design decision at one or several levels.

3.8 CONCLUSIONS

The result of any 'tunnel design method' should be an optimum combination of support features and construction procedures for the particular ground conditions, opening dimensions and use. Due to the variety and variability of influencing factors and the interdependent character of support and construction, the problem is usually very complex. The use of analytical methods relying on prior creation of a model is thus limited. Empirical methods that relate ground conditions to tunnel design and contruction are a very appropriate tool substituting for or complementing analytical methods. However, the complexity of the problem affects empirical methods also -- none of the methods completely represents all the influencing factors. This has two consequences.

- A particular method can only provide accurate predictions for conditions similar to those of cases by which it was calibrated (base cases).
- Applying different methods to the same case will usually lead to different predictions.

No method is generally and consistently more accurate than others; however, optimum methods can be defined or developed for a limited range of applications. This has to be done for each individual method by comparing anticipated ground conditions and construction procedures to the base cases. Present methods are limited in their consideration of construction procedures and of some characteristics of geologic structure. This leads to the first conclusion:

The user has to carefully study the base cases or (as a minimum) the assumptions, comments on developments and limitations formulated by the developer of an empirical method. This should be done for every application.

The anticipated ground conditions in the application case have to be compared to the base cases. This requires that ground conditions of the application case are predicted as detailed and as accurately as possible. In no case should this prediction be limited to the individual parameters of a particular empirical method, rather the actual ground conditions should be predicted. It will then be possible to determine which method is most suitable for a particular application. Such a selection by comparison requires that the base cases are given which is not so for many methods (see Table 3.7.1). The comparison of anticipated ground conditions to those of the base cases may lead to the detection of deviations. These deviations may indicate that factors other than those considered in a particular empirical method (base cases) are significant.

This leads to the <u>second conclusion</u>:

The application of an empirical method requires a thorough, detailed consideration of ground conditions.

An accurate method should not only predict what has been *done* under similar conditions but what is *adequate*, i.e., the optimum support features and construction procedures. Empirical methods depend on base cases and since these were usually designed conservatively the empirical methods will also lead to overdesign. A method that is applied outside the range of the base cases may no longer be conservative and lead to unsafe supports, or vice versa.

This can only be corrected if the degree of overdesign in the base cases is known. This is often not possible at the present time, since it requires the knowledge of satisfactory (i.e, 'limiting') performance. In addition, the final support is often designed without taking into consideration the effect of the initial support, thus leading to additional overdesign.

The <u>third conclusion</u> is thus:

Present empirical methods frequently over-estimate support requirements; the degree of over-estimation is usually not known.

While predictions of support requirements (dimensions, materials) are often not accurate for the reasons discussed above, support pressure predictions are usually even less reliable. In some cases support pressures have been analytically backfigured from the design or are the design assumptions, while in other cases actually measured support pressures have been used. However, backfigured support pressures have often no similarity with measured ones. This may be due to the assumptions underlying the analysis or due to assumptions in the interpretation of measurements. Support pressures are appealing because they would allow to compare different support types on the same "scale". Support systems different from the base cases might thus be designed. However, different support systems may perform differently even in the same ground conditions resulting in different support pressures. If ground-support pressure relations are to be used then all factors of the ground structure system (e.g., stiffnesses and deformations ahead of support installation) have to be taken into account.

The fourth conclusion is therefore:

Ground-support pressure relations should not be used unless they are based on measurements or analogous observations that include all components of ground and structure (which is practically impossible).

All the methods consider only a limited number of factors. Even the methods with many parameters consider e.g. often only one set of discontinuities and one type of ground. (Only the Q-system considers multiple sets of discontinuities with the joint number and the Stress Reduction Factor but only one joint set is considered for the joint alteration and joint roughness factors). Thus even the more detailed methods are not complete.

However, if a method has been developed from and is applied to cases with a relatively narrow range of ground support characteristics one may successfully use more complex methods at the present time.

The <u>fifth conclusion</u> is thus:

Even more detailed methods are limited in their consideration of influencing factors. With increasing number of parameters the data base per parameter decreases. At present, methods with a small number of parameters and a large data base per parameter may provide more precise results than very complex methods do.

Another consideration is the practicality of parameter determination. Boreholes, outcrops and observations in the tunnel provide substantially different information. The information that can be gathered may be more or less than is required for a particular empirical method. Some methods use few parameters that can be easily determined in the field. Other methods require the use of relatively complex tables for the determination of the parameters. Since often only a limited time is available to select support during construction, methods that use information which can directly or simply be measured are preferable under such circumstances.

The sixth conclusion is thus:

The selection of an empirical method has to reflect the availability of information on parameters and the time limitations affecting information collection.

The inherent uncertainty and complexity of the tunneling problem makes empirical methods inevitably subjective. Subjective aspects may exist in the parameter determination by the user or in the formulation of the method by the developer or both. One relies thus in other words to varying degrees on one's experience or judgement and on that of the developer. Relying on someone else's judgement may be wise but does not make the method "objective". Subjective character is not unique to empirical methods. In a complex problem like tunneling analytical methods cannot be built on first principles alone but also involve many subjective hypotheses.

The seventh and last conclusion is:

All empirical methods are subjective whether this is obvious to the user or not.

The review of empirical methods and the conclusions drawn so far have shown advantages and limitations of individual methods but also strength and weaknesses of empirical methods in general. Ideally it would be desirable to compare all the methods and to recommend on a detailed basis which of the method(s) is or are best. From the preceding discussions it is clear that no such single positive statement can be made, depending on the particular application different methods will be best suited. What might be attempted however is to consider several characteristic conditions one of which may dominate a particular case and to compare for each of the scenarios which methods would be best suited. An important limitation of these comparsons has to be mentioned. It is assumed that only one characteristic aspect clearly predominates, the relative weight of other characteristics is not known simply the fact that they are significantly less important than the predominant one. If more than one or other characteristics are important then empirical methods have to be selected by studying individual methods considering limitations and base cases. Typical scenarios that one can consider are:

(1) Construction Procedure predominates.

(2) Narrow range of ground conditions and many base cases.

(3) Wide range of ground conditios and/or no similar base cases.

(4) Use by designer with little experience.

(5) Use by designer with substantial experience.

Scenario: Construction Procedure.

Methods that address the construction sequence to great detail are:

- NATM
- Deere-Merritt-Cording descriptive
- Bieniawski
- Franklin/Louis

The most detailed consideration of construction sequence is provided by the NATM. It actually goes so far as to use the construction sequence that should be used as a classification criterion. The Deere-Merritt-Cording classification gives a general overview coassification of problems encountered in tunneling but does not give such detailed construction procedures as the NATM. Bieniawski's and Franklin/Louis' method are both based on the NATM; with these methods combinations of ground parameters are related to a construction sequence.

Scenario: Narrow Range of ground conditions and many base cases.

In such a scenario the construction procedure will be similar throughout the tunnel(s) in such conditions. If many base cases are available one can also assume that design and construction over the period of years has reached an optimum. Such a scenario may therefore indicate relatively small overdesign. Methods that are well suited to such a scenario are:

- The Q-System, developed primarily from base cases in a Scandinavian setting, thus primarily applicable there or otherwise similar ground conditions.
- The wedge analysis method, developed by Deere et al. for the treatment of instable wedges. It has been used for the Washington, D.C. subway where rock with distinct shear zones was encountered.

Scenario: Wide range of ground conditions and/or no similar base cases.

Here a method with a wide data base from many cases may provide an initial "average" estimate of support requirements, the individual case may however deviate substantially from this average condition. In such a case it would thus be necessary to obtain an 'average' estimate and then during construction use an 'observational' procedure and to adapt support and update the ground support relations. Methods that fall into this category and also recommend the use of observations are:

- Deere's et al RQD-support relations which are based on a wide range of RQD and where the RQD-support relations have been continuously updated with new experience of the developers.
- Bieniawski's geomechanics classification which has been continuously updated by the developer by incorporating 'new' experience.

Scenario: Designer with little experience in tunnel design.

Possibility that a method is used by a designer with little experience in tunneling. Methods employing parameters that can be easily determined with existing knowledge, i.e., basic geotechnical experience, are thus preferred. Such a method should also report the limits of applicability.

- Franklin's and Louis' method fulfill these criteria since they consider easily determinable parameters.

For this scenario, one might add that if the designer is a structural engineer and thus used to 'design' loads as input parameters he may favor a rock load approach. However, rock loads are imprecise and depend not only on ground conditions but also on ground structure interaction and other effects. As has been stated earlier rock load approaches should not be used unless they take such effects into account.

Scenario: Tunnel designer with substantial experience

A designer having acquired a substantial amount of experience will be able to make judgements on support adaptation based on simple visual observations. On the other hand he will know when he might use a particular measurement to aid his judgement. The observations that are made vary naturally for different designers. Two examples of methods used by designers with substantial experience are the:

- NATM classification.
- Deere-Merritt-Cording's descriptive classification.

Note, however, that the NATM and Deere-Merrit-Cording's descriptive classification may also be useful to a less experienced designer since they represent also an attempt by their developers to transmit to a potential tunnel designer those aspects of tunneling that in their experience was important but cannot be expressed in measurable parameters.

4. REQUIREMENTS FOR EMPIRICAL OBSERVATIONAL METHODS

4.1 INTRODUCTION

The principles of the observational method are first recalled. The philosophy and concept of observational methods in geotechnical engineering have been applied by many leaders in the field, particularly Terzaghi, Casagrande and Peck. Peck (1969), in his Rankine lecture which brought the concept to a wide audience, summarized the method as follows:

> "In brief, the complete application of the method embodies the following ingredients.
>
> (a) Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.
>
> (b) Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. In this assessment geology often plays a major role.
>
> (c) Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions.
>
> (d) Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.
>
> (e) Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions.
>
> (f) Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.

(g) Measurement of quantities to be observed and evaluation of actual conditions.

(h) Modification of design to suit actual conditions.

The degree to which all these steps can be followed depends on the nature and complexity of the work. We can readily distinguish between projects, on the one hand, in which events have already set the stage for the observational method as being almost the only hope of success, and those, on the other hand, in which use of the method has been envisioned from the inception of the project. Applications of the first type are much the more familiar."

This report takes into consideration only observational methods of the second type; those which follow certain principles from the inception of the project. The details of such an observational procedure can be summarized as follows (Fig. 4.1): Geotechnical conditions are examined and a design, or several designs, for the most likely conditions are prepared, as well as contingency designs for deviating conditions. The construction method follows the design idea in that it is directed toward the most likely conditions and allows for adaptation based on the contingency designs and on monitored performance.

From the principles outlined above, two main goals for an empirical-observational method can be derived. Empirical relations are required during (1) the preconstruction phase, aimed at providing contingency designs, and (2) during the construction phase, aimed at achieving the required adaptations.

In the <u>preconstruction phase</u>, predictions should be made of the most likely conditions and the possible ranges of geology, and the ensuing excavation and support procedures and perfor-

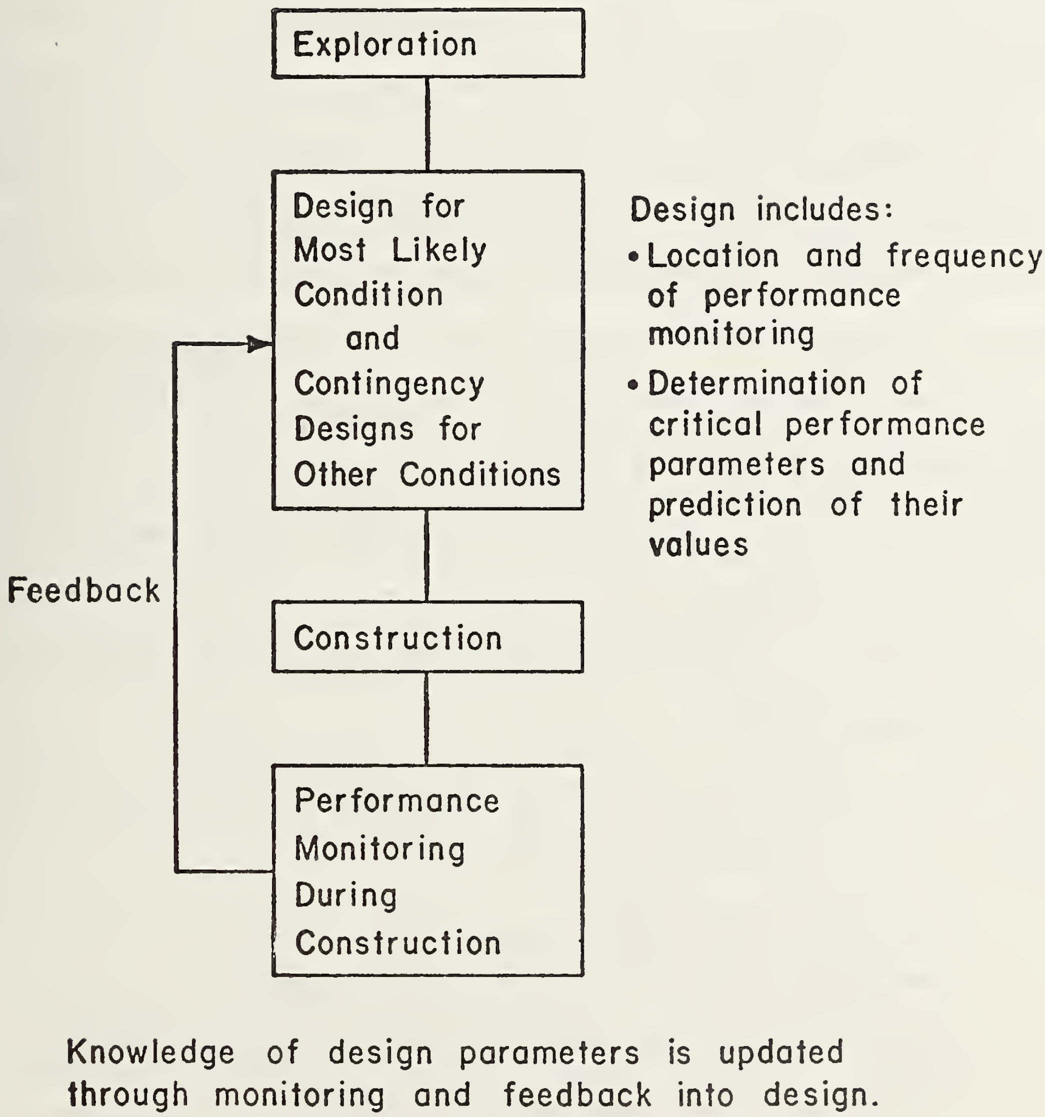


FIGURE 4.1 PRINCIPLE OF OBSERVATIONAL METHODS

mance. The information available during this phase is limited, but there are usually no time constraints imposed.

In the construction phase, the predictions of geologic/ geometric parameters, excavation-support and performance are narrowed down and updated. In this phase, more detailed information is available for many of the parameters. Relationships between geologic/geometric parameters, support and excavation procedures and performance can be updated with the information and experience gained during actual construction. A substantial body of information is available in the construction phase, but there may be time and accessibility constraints imposed on the gathering and use of this valuable information.

These two goals must be achieved through the use of empirical methods that fulfill the five principal requirements (specified in detail in Section 2) within the context of the limitations imposed by the preconstruction and construction phases of an observational method.

The requirements for empirical methods discussed in detail in Section 2 are briefly recalled:

- o Economy and Safety
- o General Applicability and Robustness
- o Practicality and Readily Determinable Parameters
- o Recognition of Subjective Character
- o Accurate and Representative Model

The character of observational methods and the particular goals will lead to specific detailed versions of these requirements, as will be discussed and proposed in this Section. The preconstruction phase is discussed in Section 4.2 and the construction phase in Section 4.3.

4.2 PRECONSTRUCTION PHASE

4.2.1 Goal

During the preconstruction phase, the most likely conditions and the possible range of conditions, including geologic, geometric, excavation-support and performance parameters should be predicted. The amount of information is limited, but time is not.

4.2.2 Problems

(a) Information that can be gathered is limited due to the linearity and planarity of the sources (boreholes, outcrops), while the information that is required should be three-dimensional and provide location, extent and shape of the geologic features. An extrapolation is thus necessary for geologic parameters with respect to geometry, location, extent and shape. Furthermore, the information is gathered at a location which is usually subject to different environmental factors than those at the tunnel level. Thus, material properties of the ground must be extrapolated to another 'environment' (e.g. weathering, previously undergone shearing, different stress state, hydrologic conditions, etc.).

> Examples: A surface outcrop is 'planar', and therefore, discontinuities parallel to the surface are not detected. This introduces a bias in the assessed discontinuity pattern.

Borings may detect discontinuities which are parallel to the surface. However, the orientation of the discontinuities with respect to the boring may be interpreted in several ways. An interpretation is not unambiguous!

The volume sampled by a boring is orders of magnitude smaller than the actual volume of the tunnel. The diameter of a boring is roughly two orders of magnitude smaller than that of a tunnel. Thus, the ratio of the volumes is five to six orders of magnitude (10^5 to 10^6) depending on the spacing of the borings.

(b) The relationships between not only geologic/geometric parameters on the tunnel level, but excavation-support procedure and performance must be predicted. These relations are not well understood. Performance and excavation-support procedure are not only influenced by geologic/geometric parameters, but by the excavation-support procedure as well. For the same geologic/geometric conditions, performance may strongly depend on the excavation-support procedure used.

Examples: Support and excavation are influenced by the type of support. Rockbolt and shotcrete act differently than steelset or timber supports. The excavation sequence has a strong influence on the support requirements and performance. If a pilot tunnel is driven and the tunnel is stagewise enlarged, the behavior will be different than a full-face excavation.

The Straight Creek Tunnel was designed using Terzaghi's rock load recommendations which are based on rock load measurements in stagewise excavated timber supported tunnels. A fullface shield tunnel was initially used in the most difficult zone and heavy steel sets were placed (14 WF 287) behind the shield. However, the shield had to be abandoned as a result of its getting stuck due to squeeze loads (Hopper et al., 1972). Subsequently a multiple drift excavation was chosen. The multiple drifts were excavated at the tunnel circumference and backfilled with concrete, thus allowing for little deformation, in contrast to the Arlberg Tunnel where large convergences were tolerated (Einstein, 1977).

The different actions of different support types were also clearly demonstrated in the tunnels and shafts of the Tarbela Tunnels. The initially planned steel-set support placed behind a shield was abandoned and replaced with shotcrete and steelsets blocked against it. In the transition zones between tunnel and shaft, the tunnel was supported by shotcrete (reinforced with wire fabric) and rockbolts, and caused considerably less difficulty (Rabcewicz, 1973; Golser, 1973; Hillis et al., 1976; Einstein et al., 1977).

(c) The parameters observed from exploration depend strongly on the sampling procedure and interpretation.

Example: RQD depends to a large extent on the drilling procedure, i.e., size of the core, type of core barrel, and details of machine operation. For borings smaller than NX, a correction factor should be applied.

Further, RQD depends on the orientation of the boring to the discontinuities (see Appendix C).

Together with the limitations mentioned previously, this produces the compound problem shown in Fig. 4.2.

In summary, the extrapolation to the real tunneling situation is difficult due to the following factors:

- The information is biased due to linearity, planarity, and different material properties observed.
- The sample is very limited in volume and extent.
- Sampling procedures may introduce a bias.
- Extrapolation difficulties (sample size, distance).
- Unknown relations between parameters representing influential factors and parameters reflecting performance.

These problems are basically the same as those encountered in any other empirical or analytical procedure. The difference

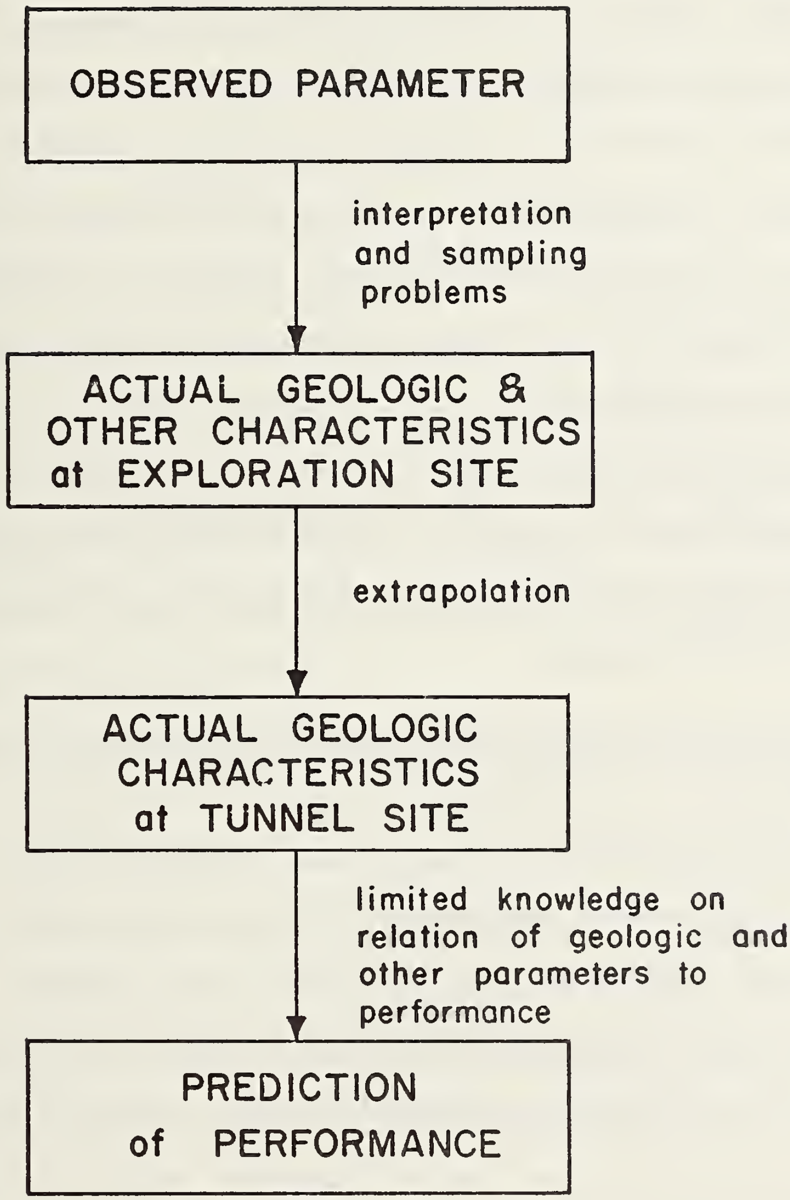


FIGURE 4.2 COMPOUND PROBLEM OF PERFORMANCE PREDICTION IN TUNNELING

is that on one hand updating is possible, but on the other hand updating is made easier if the same parameters are used in the preconstruction phase as in the construction phase. (In the construction phase, additional parameters may be determined, or the parameters previously used can be further differentiated. It is important to remember that during the preconstruction phase, time is usually unconstrained, making it possible to perform investigations that are time-consuming but that are important during construction, e.g. swelling tests.)

Based on the goal to predict contingency design and the problems outlined above, one can now determine the requirements that an empirical observational method should fulfill during the preconstruction phase.

4.2.3 Requirements for Empirical Method During Preconstruction Phase

(a) Select parameters such that a maximum amount of information can be gathered. Parameters may describe large regions with little information or small regions with detailed information. Parameters should be selected in order to achieve a maximum amount of information gathered for the tunnel under consideration.

> Example: A boring may yield a maximum amount of information for a very limited zone; however, it cannot be generalized. A seismic survey may yield information over a large area, but give no details. Depending on the geologic situation, the parameters

from a seismic survey may be completely meaningless for prediction of tunnel behavior. In other cases they may supplement boring data, and again in other cases they may provide more useful information than boring data.

(b) Select parameters such that ranges of possible behavior (most likely condition and extreme values) can be established.

(c) Select parameter(s) such that a change in tunnel performance is reflected in a change of parameter(s).

(d) Parameters must be compatible with parameters used during the construction phase. Otherwise, they cannot be easily updated during the construction phase and an additional interpretative step may be required.

Example: RQD, Core Recovery, may be easily monitored from a core log; however, it cannot be easily determined at the tunnel wall. An additional interpretative step is necessary (e.g., correlations of fracture spacing to RQD) to determine RQD.

(e) During the preconstruction phase there are usually no time constraints. Thus, parameters that are time consuming to determine but are required in the construction phase should be determined and prepared for adaptation and correlation. during the construction phase (for example, swelling tests should be performed during the preconstruction phase).

(f) Parameters selected should take into consideration the effect of construction and support procedures.

(g) Subjective assessment of parameters may be necessary. However, this should be done unequivocally and and consistently.

Example: RQD depends heavily on the drilling procedure. It may be attempted to correct RQD for 'poor quality drilling' procedures. This can only be done subjectively since the drilling process is very complex. The subjective assessment should, however, be performed consistently, which requires a detailed knowledge of both poor and good drilling procedures.

(h) Sampling bias cannot be eliminated, and a clearly-expressed estimate of this bias is necessary.

4.3 EMPIRICAL METHODS DURING CONSTRUCTION PHASE

4.3.1 Goals

During the construction phase, relationships between parameters describing ground and excavation-support procedures must be brought nearer to reality, i.e., they should describe the actual ground-excavation-support interaction more accurately. Specifically,

- The observed geologic and geometric characterizations must be utilized to make accurate predictions of excavation procedure and support requirements that can be implemented immediately.
- The monitored performance shortly after excavation (e.g. one round) can be used to make accurate predictions of satisfactory behavior (stability) of the initially supported tunnel (possibly for various stages

of the support-excavation procedure) and of the final support requirements.

4.3.2 Problems

(a) Many geologic-geometric, excavation-support and performance parameters can be observed. The most important question is: Which of these best represent the influencing factors? However, a determination of the influencing factors by elimination would require that a detailed record be kept of many parameters and subsequent elimination of those that have only a minor effect. To do this with a complete set of parameters is a nearly impossible task. Thus, the question is what the significant parameters are and how a limited set can be selected within the given restrictions of time and accessibility.

(b) What effect do construction-support procedures have? A set of parameters may be significant with regard to one particular procedure, but not with regard to another.

> Example: Water conditions may be important if a tunnel is excavated conventionally and supported by steelsets and lagging. In case of a shield, water conditions may be of minor importance.

(c) Experience from previously built tunnels may help to determine some parameters and their states because certain parameters that have been observed in other tunnels are generally significant. In addition, there will be other parameters, and different states of the generally valid parameters, that are significant in a given tunnel. However, the relative significance of each parameter (or change in state of the parameter) can only be determined by relating monitored performance, i.e. changes in performance, to changes in the parameter and parameter states. However, there still may be too many parameters to draw definite conclusions from the available performance and other observations, and thus an element of uncertainty remains.

Example: Water is considered to have a deteriorating effect on ground quality. However, the presence of water may be associated with other changes (e.g. more fracturing or more weathering), and therefore it is not entirely clear which effect is overriding, and these parallel changes possibly cannot be separated.

(d) In some sections of a tunnel a particular performance may be judged satisfactory. However, observance of the same performance in a section of the tunnel with different geologic-geometric conditions may or may not be judged satisfactory, due to a different underlying mechanism.

(e) Parameters monitored during construction must be compatible with parameters from the preconstruction phase.

(f) Even if the parameters are significant and the relationships between geologic/geometric parameters, excavation-support requirements and performance are established accurately, an additional problem exists due to the necessity for extrapolation: Excavation procedures must be predicted at least one round in advance. Further, the support may have to be adapted based on information gathered previously.

(g) Detailed parameters may be difficult and impractical to determine in the field. Constraints may be imposed by time limitations, accessibility, or a combination of both.

Example: The thickness of shear zones may be important, but this may not be determined due to an inability to gain access to the tunnel wall before support (shotcrete) is placed and the wall is covered (safety aspect).

Another type of constraint may be the type of excavation method used. With a full-face TBM, the ground conditions are visible only at the tunnel circumference at some distance from the face, and not at the face itself (although some inspections may be performed during shutdowns of the TBM).

(h) The performance of the tunnel is monitored for the purpose of relating specific performance measures to stability and performance predictions of initial and final support (including excavation method). Many parameters can be monitored; it is a question of which parameter and parameter states are significant.

Examples: In tunnels with 'loosening' behavior, only a small deformation can be tolerated. This deformation probably occurs vertically, and thus it is not necessary to monitor the horizontal convergence (e.g. DuPont Circle Station).

In contrast, in squeezing ground the horizontal convergence may be the most important parameter to be monitored (e.g. Arlberg).

In both cases it may take considerable experience, even in the particular tunnel, to establish criteria for satisfactory performance.

(i) Experience from past tunnels may be limited, due either to dissimilar ground conditions or different excavation-support procedures.

Example: Old railroad tunnels were excavated with pilot tunnels and a stagewise enlargement, and thus an unknown amount of deformation could have occurred prior to placement of the liner. Using new methods, the tunnel walls are immediately supported with shotcrete, and thus deformations can be observed in the completely excavated tunnel (Arlberg Railroad, 1880-83, vs. Highway-tunnel, 1974-78).

(j) Information derived from test sections may provide a better understanding of the underlying mechanisms. However, only a limited number of test sections can be built due to cost restrictions and limited accessibility. Further, test sections are often part of the actual tunnel, and the full excavation of these sections often occurs at too late a stage in tunnel construction. Test sections that are built prior to the actual tunnel may on the other hand not be representative.

Example: A pilot tunnel is driven over part of the future tunnel alignment. This tunnel is enlarged to full cross-sections at selected stations.

However, if the final tunnel is driven with full-face or a heading and bench procedure, the performance may differ so substantially that no correlation between test section and actual tunnel can be established.

(k) In contrast, performance in sections with different geologic/geometric parameters may make it possible to give a specific meaning to a measurement based on geologic/geometric and excavation-support parameters. In addition to permitting a better interpretation of measurement, such cases indicate which parameters are significant and should be monitored. Since such information is not often available, one usually must decide which parameters should be monitored.

(l) Practicality: It must be possible to monitor the parameters accurately and within time, cost and accessibilitv constraints.

Example: Theoretically, it may be necessary to monitor the performance (convergence) ahead of the face, which can be done by drilling ahead and installing horizontal deflectometers. However, the installation is impractical because it hinders construction work and is very costly.

SUMMARY

Selecting significant geologic parameters and appropriate empirical relations between geologic/geometric parameter states and support-excavation procedures depends on the performance observations.

At the same time, the interpretation of monitored performance depends on the actual combination of geologic/geometric characteristics and support-excavation procedure. The intricate interplay of geologic/geometric, excavation-support and performance parameters is difficult to establish and is subject to time and accessibility limitations.

In terms of the goal of adapting the excavation-support procedure to encountered geologic/geometric conditions based on monitored performance and the problems outlined above, it is possible to determine the requirements that an empirical-observational method should fulfill during the construction phase.

4.3.3 Requirements

(a) Establish a general set of relevant parameters and general relationships between geologic/geometric and excavation-support parameters.

(b) Establish rules for selecting a limited set of significant geologic/geometric parameters from the general set of relevant parameters.

(c) Parameters must include effects of excavation-support procedures.

(d) Geologic/geometric parameters must be limited on the basis of past experience, but should not exclude parameters that are significant to a given case, regardless of their past insignificance.

(e) Establish extrapolation procedures for geologic/geometric predictions and excavation-support relationships in the next round, or further ahead if necessary.

(f) Parameters observed during the construction phase must be compatible with parameters observed in the preconstruction phase.

(g) The parameters must be determinable within time and accessibility limitations.

(h) Establish remaining uncertainties in the parameters due to indeterminateness of the problem.

(i) Establish a general set of relevant parameters that should be monitored.

(j) Establish rules that allow selection of the significant performance parameters from the general set of relevant parameters. These parameters must be compatible with geologic/geometric parameters.

(k) Monitored performance cannot be transferred to other conditions without taking into consideration the geologic/geometric and excavation-support parameters. Rules which allow for the interpretation of the performance for a particular geologic condition must be established.

(l) It must be practical (i.e., in terms of accessibility and time) to monitor the performance parameters.

(m) The performance must be monitored in a consistent and repeatable manner. The measurement procedure must be described in detail.

(n) The intricate interaction of geologic/geometric, excavation-support and performance parameters must be considered in detail.

(o) Guidelines must be established, which combinations of parameters (geologic/geometric, excavation-support and performance) provide the significant information.

5. EMPIRICAL METHODS FOR OBSERVATIONAL TUNNEL DESIGN CONSTRUCTION PROCEDURES

5.1 INTRODUCTION

In this Section empirical observational methods will be proposed that are modifications of existing empirical methods or that are based on a combination of existing methods. By specifically addressing existing methods a more satisfactory result can be achieved than by creating a new method. Designers who have been using an existing method can continue to do so.

In Section 3 existing empirical methods were reviewed and it was concluded that none of the existing empirical methods is entirely satisfactory, which is not necessarily a disadvantage as long as a method can be updated and the predicted excavation-support modified during excavation. Naturally the NATM is the best example of an empirical method whose concept and practical application fully integrates updating. (For this reason it will not be discussed in this section.) However, it should be noted also that developers of other existing methods recommend that their methods be updated or at least the predicted support be adapted during construction. Deere et al. state that observations in the tunnel should be made, the support modified if necessary and the RQD-Support relation updated accordingly. Louis and Franklin also specifically call for support adaptation during construction and to update the ground support relations. Bieniawski recommends to adapt the support during construction. The following discussion relates to all empirical methods but the recommendations are to some extent already incorporated in those already mentioned.

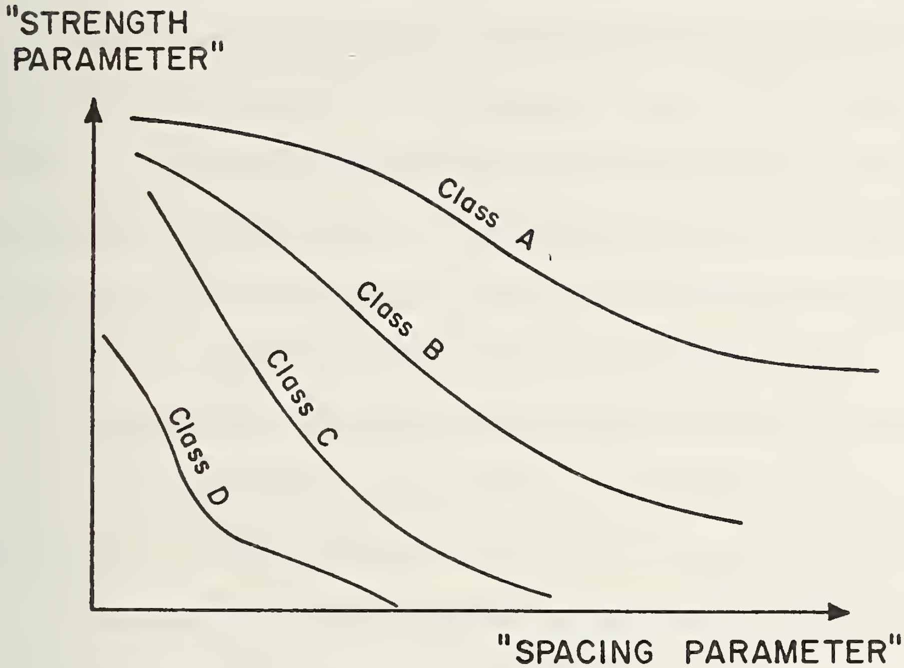
Existing methods that can be updated in an observational procedure should fulfill the requirements established in Section 4. In Section 5.2 modifications of existing empirical methods for use prior to construction are described, and in Section 5.3 the proposed approach during construction is discussed.

5.2 EMPIRICAL METHODS PRIOR TO CONSTRUCTION

The basic idea is to establish relations between parameters that describe geologic/geometric conditions, and between ground classes, excavation-support procedure and performance. The basic principle is illustrated in Fig. 5.1. Ground (and geologic/geometric) conditions are described by a "strength parameter" and a "spacing parameter". Ground classes are defined for certain combinations of these parameters. In each ground class, and depending on excavation-support procedure, different support quantities will have to be used.

Ground Classification

In the preconstruction phase a ground classification should be based on a few parameters due to limited information, and extrapolation to and along the tunnel zone is necessary. Also, the parameters must be determinable with different types of exploration.



	EXCAVATION PROC. 1		EXCAVATION PROC. 2	
	Support Type 1	Support Type 2	Support Type 1	Support Type 2
CLASS A	Q_{A11}, P_{A11}	Q_{A12}, P_{A12}	Q_{A21}, P_{A21}	Q_{A22}, P_{A22}
CLASS B	.	.	.	.
CLASS C	.	.	.	.
CLASS D	.	.	.	.
CLASS E	Q_{E11}, P_{E11}	Q_{E12}, P_{E12}	Q_{E21}, P_{E21}	Q_{E22}, P_{E22}

Q = QUANTITY OF SUPPORT

P = PERFORMANCE OF TUNNEL

FIGURE 5.1 PRINCIPLE OF GROUND CLASSIFICATION

On the other hand, they must represent the major effects of ground on tunnel performance. The two proposed parameters, "strength" and "spacing", each represent a group of geologic/ geometric characteristics that affect tunnel performance. The "strength" parameter includes persistence, waviness, tightness of discontinuities, effect of fillers, strength of intact rock, effects of water pressure, stress state, and possibly to some extent discontinuity orientation. The "spacing" parameter includes spacing of discontinuities, possibly the dimensions of the tunnel, and the discontinuity orientation effect. The parameters thus also fulfill the requirement that they can be related to the detailed information that becomes available during construction. However, as will be discussed when modifying existing empirical methods, the determination of the strength parameter may be difficult.

Parameters obtained from observations at the surface, in boreholes or other types of exploration must be extrapolated to and along the tunnel zone. This extrapolation is subject to inaccuracies and interpretations. The parameters are observed at a different location, and also include measurement errors and sampling bias. The task of extrapolation and interpretation is not simple, and involves a substantial amount of subjective judgment. Methods to do this in a consistent manner have been described by Lindner (1975), Einstein et al. (1978), Hogarth, (1975).

Ground Support (Excavation Procedure) Relations

Ground support and excavation procedure relations must fulfill the requirements established in Section 4, summarized below:

- o Available information must be optimally used.
- o Ranges of excavation support procedures and performance predictions must be given.
- o Only significant parameters must be used (a change in parameter must result in a change in performance.
- o The method must be compatible with that used in the construction phase.
- o Construction effects must be included.
- o A subjective assessment may be necessary, but must be performed consistently.
- o Uncertainties in the prediction must be expressed.
- o Sampling bias and measurement errors must be considered and corrected if possible.

Performance Prediction

Performance prediction is an important aspect of observational procedures because support and excavation are adapted on the basis of observed performance.

The performance estimates given in the preconstruction phase are the basis for updating the ground-support-performance relations, which are essential during construction. However, given the quantity and quality of information, such predictions will be relatively limited.

A satisfactory empirical method for the preconstruction phase of observational procedures will be achieved through:

(1) Modification and/or combination of existing methods such that they conform to ground classification systems introduced above. Modification and combination of these methods such that their ground support relations fulfill requirements established in Section 4.

(2) Development or application of existing performance predictions that can be used in conjunction with the ground classification methods (none of the existing empirical methods, except for the NATM, indlude performance predictions in the ground-support relations).

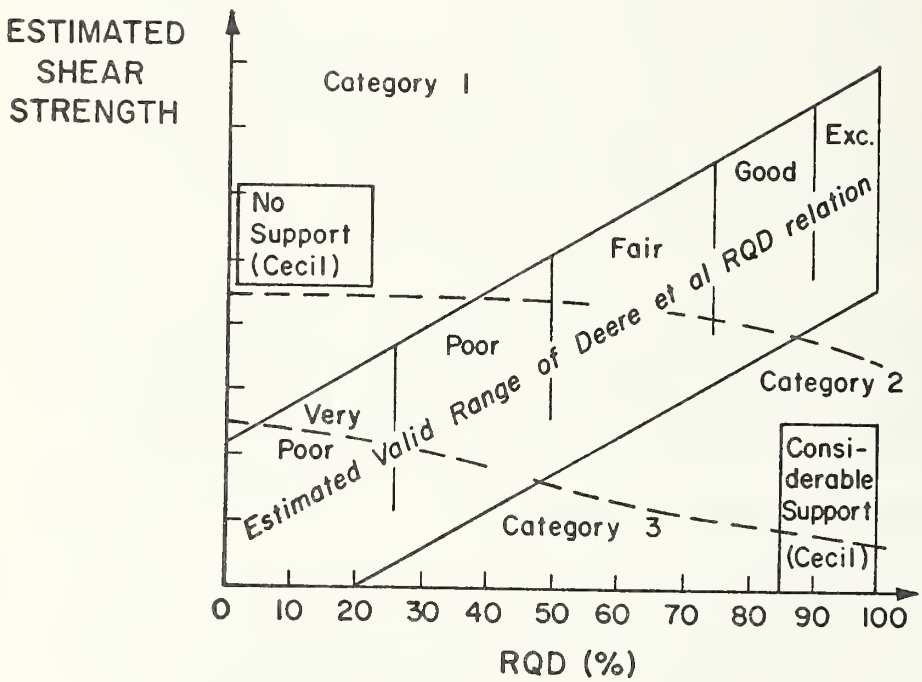
5.2.1 Modifications of Deere's RQD-Support Relation

Deere, Merritt, Cording propose to use the 'descriptive' classification in the initial phases of a project and to supplement it with the RQD-Support relations. The follwoing discussion intends to tie these two approaches and the wedge approach better together. In his research, Cecil (1970, 1975) has noted anomalous cases which did not fit into Deere's original RQD-Support relation. On one hand, tunnels in rock with a single set of tight discontinuities but low RQD (<20%) were stable, while on the other hand, in cases with high RQD

(> 85%) but low strength singular discontinuities, considerable support was required. The strength of the discontinuities is not reported in detail, but reasonable estimates for friction angles are: more than 45° (including asperities) for the tight discontinuities, and less than 25° for the clay-filled discontinuities. With this information and the original RQD ground classification, it is possible to establish a ground class chart as in Fig. 5.1. This is shown in Fig. 5.2, where the original RQD relation covers the diagonal region. Note that the "strength" parameter is an estimated shear strength (see Section 5.2.6). Although Cecil's descriptions (tight joints, clay-filled joints) can be related to the numerical values, an extension to other cases is highly judgmental.

In the preconstruction phase, the exact location of the boundaries in the RQD-Strength plot are not known, it must be established during the updating process. The general trend of the boundaries is, however, sloping from left to right.

As discussed in Section 3.4.1, Deere et al. propose to use a wedge approach if distinct low-strength shear zones are present. The wedge approach, however, considers absolute spacing of the discontinuities, and not RQD as "spacing parameter". In a first step, the ground classification chart (Fig. 5.2) will indicate whether a wedge-type approach may be useful. Support may then actually be "designed" using such a wedge approach, which is useful for low-strength discontinuities and high RQD. Note, how-



Categories 1 to 3 refer to the "descriptive classification" proposed by Deere et al. (1974) In the range of 'considerable support', i.e., high RQD but low strength, the wedge stability analysis may be useful.

FIGURE 5.2 MODIFICATION OF THE RQD CLASSIFICATION ever, that the wedge approach cannot be directly included in Fig. 5.2, because actual joint geometry is required.

The Deere-Merritt-Cording descriptive classification is intended to be used during preliminary phases of a project; it can also be roughly located in Fig. 5.2. Category 1 might be situated in the upper right-hand corner, with boundaries extending from left to right. Category 2 falls approximately in the middle of Fig. 5.2 (RQD = 25% to 90%), and Category 3 falls in the lower left-hand corner.

Fig. 5.2 serves as preliminary ground classification chart. After determination of tunneling conditions with the Deere-Merritt-Cording descriptive classification in the initial preconstruction phases, the support requirements may be determined using Deere's et al. original relations for the diagonal zone, with Cecil's corrections for tight single joint set and low RQD. For the other extreme (high RQD and low strength shear zones) the wedge analysis may be useful.

Comparing the modified RQD-Support relation to the requirements established in Section 4, one finds:

(a) Optimization of available information: RQD does often not represent the best "spacing parameter", and should be replaced by actual spacing, particularly in the wedge approach.

(b) Ranges of support are given for ranges of parameter.

(c) The parameters are significant.

(d) The method is only partially compatible since RQD cannot be measured directly in the tunnel during construction.

(e) Construction effects are considered to a limited degree only; support requirements are different for conventionally-excavated tunnels and machine-bored tunnels. However, no recommendations for excavation sequences are given.

(f) The strength determination for the modified relations is, to a large extent, subjective.

(g) Uncertainties are not explicitly accounted for.

(h) Sampling bias and measurement errors are present, e.g. RQD depends on drilling procedure and the orientation of the discontinuities with respect to the boring.

5.2.2 <u>The Modified RSR Concept</u>

Parameters A and C of the RSR concept represent strength properties of the rock mass, whereas parameter B represents joint spacing. Thus, if one plots the sum (A + C) = "strength parameter" (see Section 5.2.6 for aspects on strength parameter) versus B, equal RSR-Lines will be straight lines (Fig. 5.3), which represent boundaries of ground classes. The ground classification method used by Wickham et al. is well suited to the preconstruction phase. The description is in general geologic terms and considers ranges of geologic conditions, joint conditions and discontinuity spacings. The strength parameter reflects the strength of intact rock, but stress state is not considered.

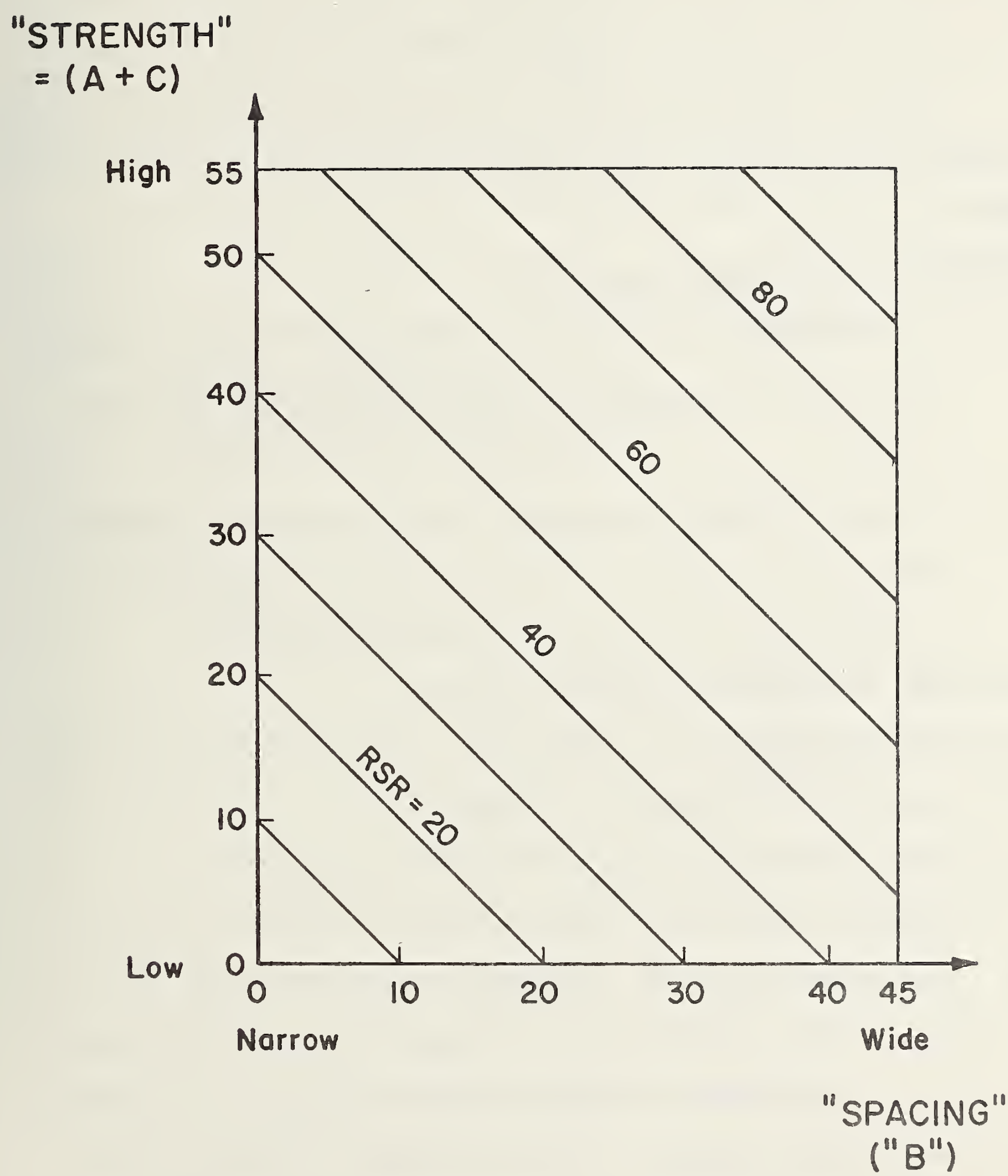


FIGURE 5.3 MODIFICATION OF WICKHAM ET AL. RSR METHOD

RSR is related to support requirements, as was shown in Section 3.5.3. In place of the original relation by Wickham et al., the modified relation by Rutledge and Preston (Section 3.6) may be used to obtain less conservative support requirements. However, even the results by Rutledge and Preston should be treated with caution, since they include only a limited amount of data.

Fulfillment of requirements:

(a) Optimization of available information - Most of the information available in the preconstruction phase is used.

(b) A single equation is given for the ground support relation; however, a range of support quantities is obtained for a range of ground conditions.

(c) The parameters are significant.

(d) The parameters are compatible with the construction phase and can easily be assessed.

(e) Construction effects are considered only to a limited extent by a TBM-correction factor, but no excavation-support procedures are prescribed for particular ranges of RSR (ground classes). At present, the method is applicable primarily to steelset supported tunnels, but other types of support systems may be related to RSR using the updating procedure.

(f) Subjective assessment - The parameters A, B and C must be extrapolated to and along the tunnel. Also, squeezing ground conditions (related to stress state) must be excluded by subjectively assessing the effect of stress state on ground conditions.

(g) Uncertainties are not explicitely considered.

5.2.3 The Modified Geomechanics Classification (Bieniawski)

The parameters of teh Bieniawski method can be grouped into "strength" and "apacing" parameters. The strength parameter includes the following parameters from Bieniawski:

- Intact strength
- Condition of discontinuities
- Groundwater conditions
- Orientation of joints

The spacing parameter includes:

- RQD
- Spacing of joints

A plot of spacing versus strength parameters (obtained by using the subparameters) shows that equal RMR-curves are straight lines (see Fig. 5.4). "Strength" is represented on a scale of 0 to 60, while "spacing" is represented on a scale of 0 to 40. However, the determination of these two "comprehensive" parameters must be performed by individually assessing each subparameter. The method therefore cannot yet be simplified to the desired level (compare Section 5.2.6 for a possible approach).

Ranges of RMR (ground classes) are related to excavation-support requirements (as shown in Section 3.5.4).

Fulfillment of requirements:

(a) Optimization of information - The subparameters make use of information, but the indirect determination of the comprehensive parameters is not desirable.

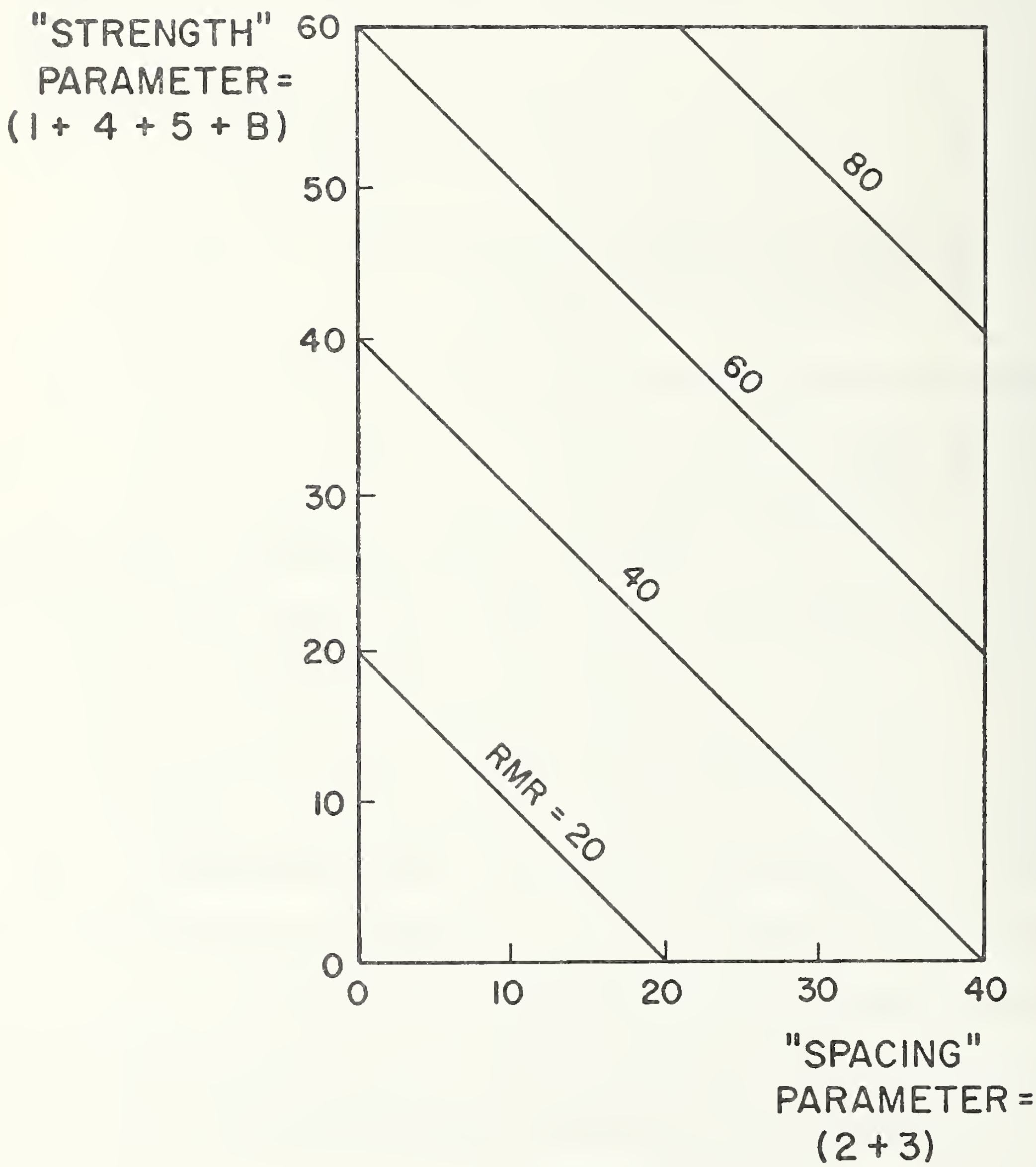


FIGURE 5.4 MODIFICATION OF BIENIAWSKI'S GEOMECHANICS CLASSIFICATION METHOD

(b) Ranges of support requirement are given.

(c) Parameters are significant.

(d) Compatibility - It may not be possible to determine all subparameters in the construction phase.

(e) Construction effects are considered in the ground-support relations.

(f) Subjective assessment is required in assessing the comprehensive parameters, even if the subparameters are actually available during the preconstruction phase.

(g) Uncertainties are not explicitly considered.

(h) Sampling bias and measurement errors affect the assessment of RQD and the condition of discontinuities (if based on borehole information).

5.2.4 The Modified Barton Q-System

In Barton's method, the ratio RQD/J_n can be combined to represent the spacing parameter, and J_r, J_a, J_w, SRF to $\frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$ as strength parameters. By plotting $\frac{RQD}{J_n}$ versus $\frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$, a strength-spacing plot is obtained. Equal Q-lines representing equal ground qualities are hyperbolas (Fig. 5.5). The spacing scale ranges from 0.5 to 200 and the strength scale from 1.25×10^{-4} to 5.33. As in the modified Bieniawski method, it is not possible at the present time to directly assess these two comprehensive parameters (it is still necessary to assess each of the six factors individually, and then combine them). However, with increased experience, the comprehensive parameters

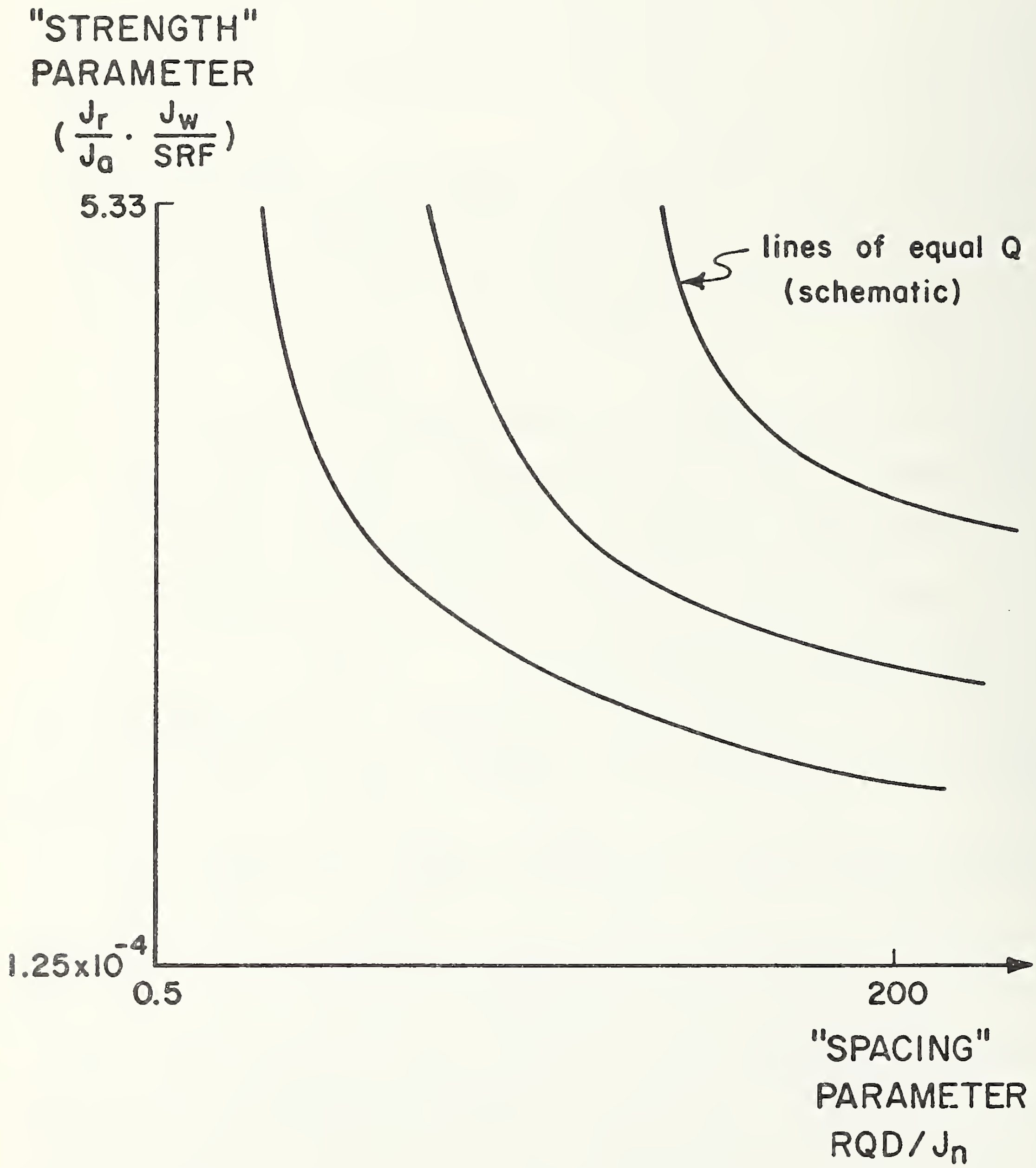


FIGURE 5.5 SCHEMATIC PRESENTATION OF THE MODIFIED BARTON CLASSIFICATION (Q-SYSTEM)

can be assessed directly. (Compare Section 5.2.6 for a possible direct assessment.)

Curves of equal Q are limits of ground classes. Ranges of Q can be related to support recommendations (see Section 3.5.5). However, construction effects are included to a limited extent only in the ground-support relations.

Fulfillment of requirements:

(a) Optimization of information - The method uses much information that may not actually be available in the preconstruction phase.

(b) Ranges - Variations in ground condition result in different ground classes.

(c) The parameters are significant.

(d) Compatibility with the construction phase is problematic; parameters are difficult to determine both in the preconstruction and construction phases.

(e) Construction effects are only considered to a very limited extent in the ground-support relations.

(f) Subjective assessment is required since many of the subparameters are not actually available during construction.

(g) Uncertainties are not explicitly considered in the method.

(h) Sampling bias and measurement errors may affect RQD (drilling procedure).

5.2.5 Louis' and Franklin's Methods

Both Louis' and Franklin's methods consider comprehensive "strength" and "discontinuity" parameters, and thus these methods do not have to be modified. The "strength" parameter includes intact strength of the rock normalized to overburden stress; the "discontinuity" parameter is a 'blocksize' normalized to tunnel diameter (see Fig. 5.6). The ground classes are related to support-excavation procedures (see Sections 3.5.6 and 3.5.7).

Fulfillment of requirements:

(a) Optimization of information - Not all the information available is used in the preconstruction phase.

(b) Ranges of ground parameters result in different ground classes.

(c) Parameters are significant.

(d) Parameters are compatible with construction phase.

(e) Construction effects are considered in the ground-support relations.

(f) Subjective assessment may be necessary.

(g) Uncertainties in ground-support relations are considered to a limited extent only in Louis' relation (Section 3.5.6). However, Franklin gives ranges of support (Section 3.5.7).

(h) Sampling bias and measurement errors may be present in the strength determination (e.g. selection of 'hard' rock pieces for strength determination.

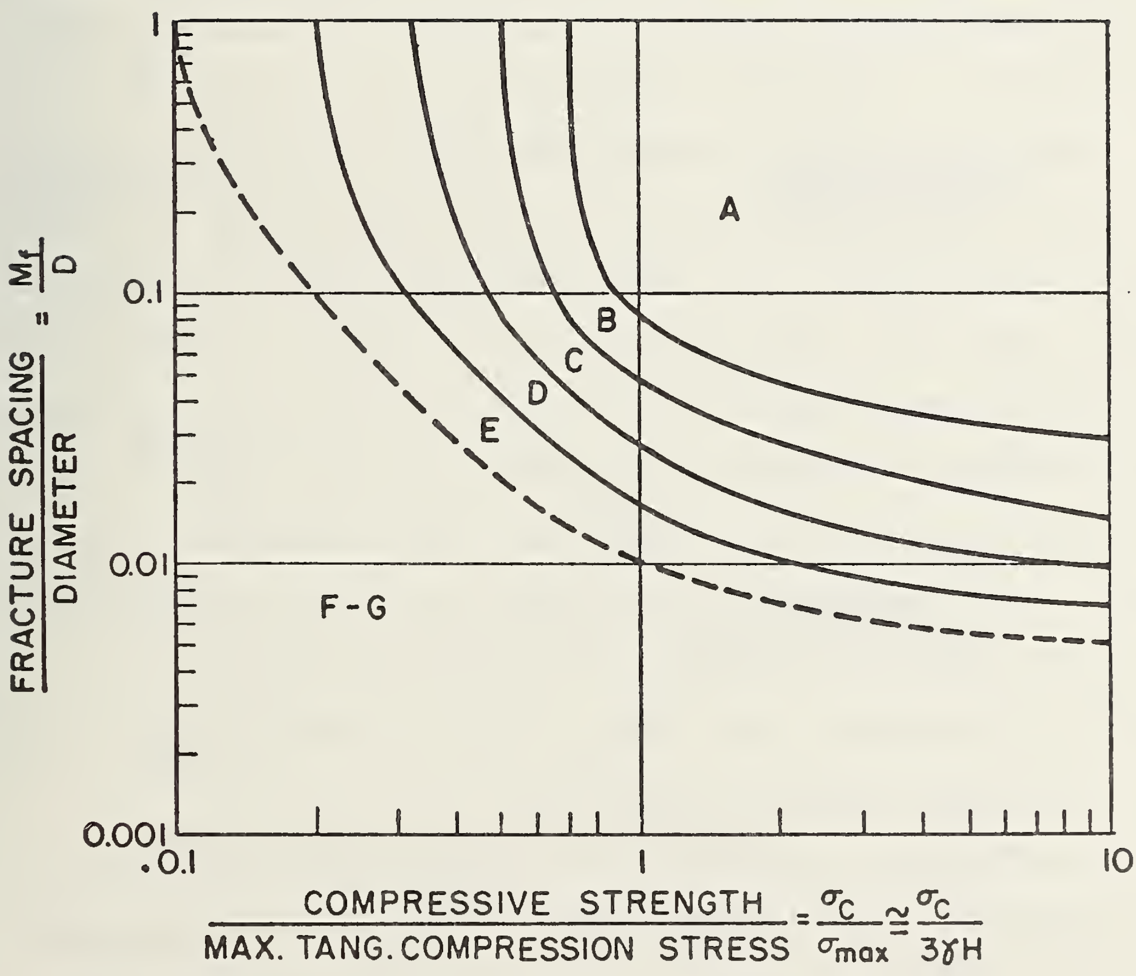


FIGURE 5.6 LOUIS' GROUND CLASSIFICATION CHART
(AFTER LOUIS, 1974b)

5.2.6 Assessment of the Comprehensive "Strength" Parameter

The comprehensive normalized rock mass strength parameter (Fig. 5.7) represents many factors including joint conditions, tightness, waviness, fillers, continuity, intact rock strength, water conditions and stress state. An assessment must consider all of these factors, and it is evident that a direct derivation of the comprehensive parameter is difficult. At present such a derivation is essentially subjective.

Some guidelines for this procedure can be given:

Each of the above-mentioned influencing factors is characterized by its extremes (i.e. fully persistent versus non-persistent joints; water: dry versus heavy inflow). Natural conditions are then subjectively associated with one of the extremes. The extremes of the comprehensive parameters are established by combining the extremes of the individual characteristics (subparameters). The assessment of intermediate states is more complex. They can be obtained through combination of different extremes (good, bad); however, the effect of the influencing factors will have to be subjectively weighted.

This procedure corresponds to the numerical derivation of the comprehensive parameters from subparameters that was shown in the description of Wickham et al., and Bieniawski's and Barton's methods. The procedure proposed here avoids some of the limitations of the numerical derivation (usually limited to a single joint set, exclusion of some factors such as stress state). In considering extremes of influencing factors only, it is less complex and more suitable to the preconstruction phase.

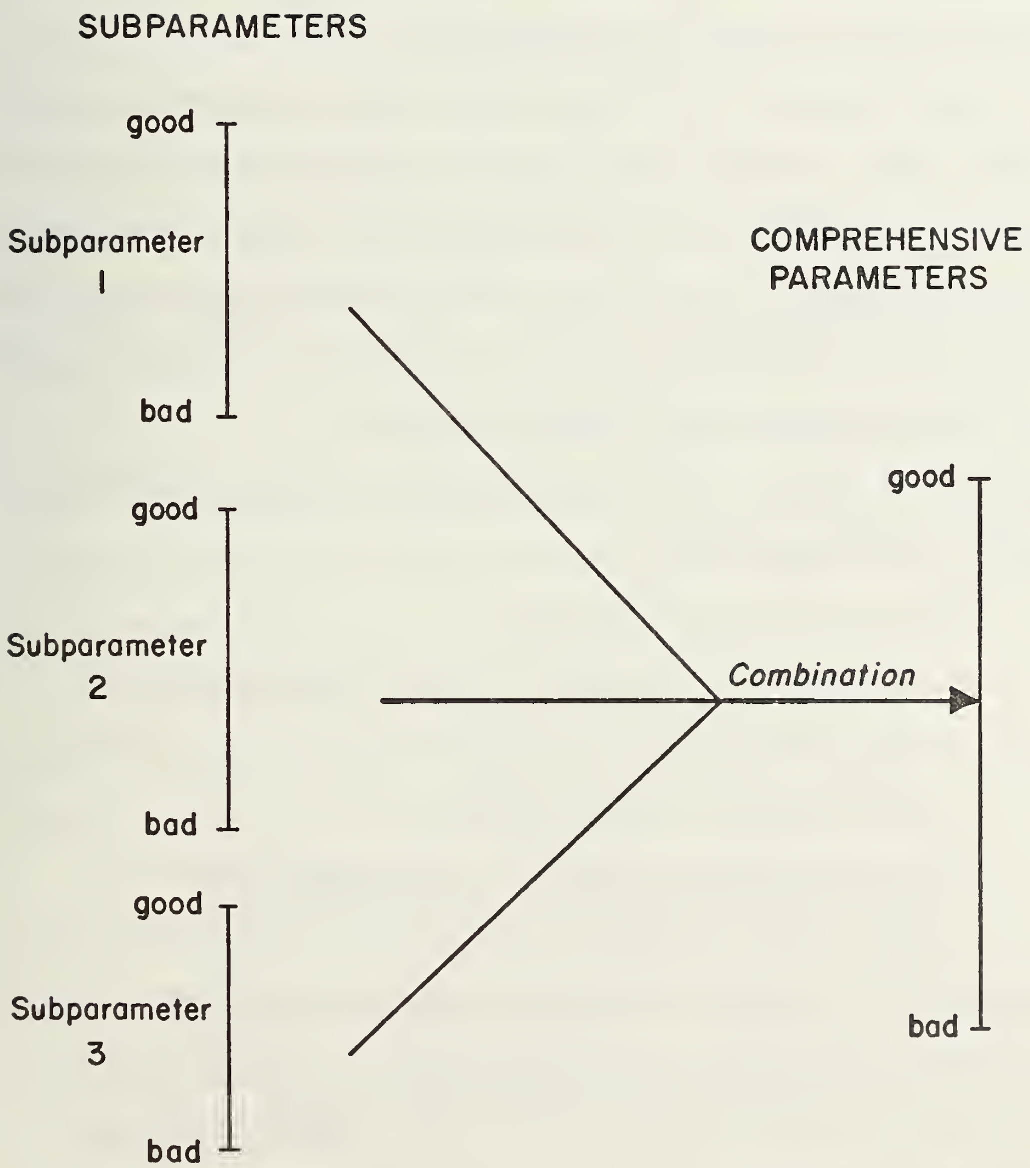


FIGURE 5.7 PRINCIPLE OF COMBINING SUBPARAMETERS INTO A COMPREHENSIVE PARAMETER

5.2.7 Prediction of Performance

In previous sections only ground classifications and relations between ground conditions and excavation-support requirements were discussed. In observational procedures it is equally important to predict performance. Not many empirical relations for the prediction of satisfactory performance exist. Results for large chambers and shallow tunnels have been reported by Cording et al. (1975), Cording and Mahar (1978). For squeezing ground, results by John (1979) are available which are supplemented by the results presented in Appendix E of this report (Analysis of the Arlberg and Tauern tunnel).

Cording and Mahar (1978) mention the following performance criteria for large chambers:

- Satisfactory performance
  - Displacements normal to the tunnel wall in the range of 2.5 to 7.5 mm (0.1 to 0.3 inch)
- Unsatisfactory performance
  - Displacements normal to the tunnel wall in the range of 12.5 to 75 mm (0.3 to 3.0 inches)

Unsatisfactory performance was primarily related to shear zones and major discontinuities that were not adequately supported. In such cases corrective measures had to be taken (additional bolt support).

For the tunnels of the Washington D.C. subway system, the observed displacements in "well-supported" tunnels (Cording and

Mahar, 1978) was on the order of 0.5 mm for small tunnels to 5 mm for larger excavations (no dimensions given). Displacements of 0.5 to 1.25 mm were found to be typical by Cording and Mahar (1978) for rock blocks of 1.2 to 1.8 meters in size, supported by 50 mm shotcrete and non-tensioned rock bolts (Cording et al., 1977). These displacements are attributed to separation along the joints which were required to tension the bolts.

For deep tunnels in squeezing ground, John (1979) has established relations dependent on the presence of shear zones. For shear zones parallel to the tunnel, convergences on the order of up to 700 mm (0.7 m) were observed for a tunnel 11 meters in diameter. In crosscuts of the same tunnel, i.e., where the shear zones strike perpendicular to the axis of the opening, observed convergences were substantially lower and reached only a few centimeters.

Data analyzed from the Arlberg and Tauern tunnels is presented in Appendix E. These tunnels are located in squeezing ground. The major findings of this analysis are:

(1) <u>The importance of lithology and petrography</u> - The effect of lithology could be established by comparing different rock types from the Tauern tunnel. Anhydrite showed the most favorable behavior, and performance decreased for the phyllites to the serpentine.

Effects of petrography can be illustrated by the performance of the phyllites. With increasing graphite content the performance worsened. Calcitic Chloritic Phyllites showed the most favorable performance, followed by Calcitic Graphitic Phyllites. Quartzitic Phyllites performed similarly to the Graphitic Phyllites.

(2) Effect of Water - Water inflow described as heavily dripping, lightly dripping and humid resulted in different performance, as illustrated by the data from the Arlberg Tunnel. The performance improved with decreased water inflow. Note that the data for different water conditions overlap considerably, and this finding is a general trend for the entire data set and not for singular points (see Appendix E).

(3) Effect of Shear Zones - The data analyzed from the Arlberg Tunnel (Appendix E) tends to indicate that a larger number of shear zones leads to a deterioration of performance.

Further, the orientation of the geologic structure with respect to the tunnel axis is important, as indicated by a comparison of the Tauern Tunnel (striking perpendicular to the axis of the tunnel) and the Arlberg Tunnel (striking parallel to the axis of the tunnel). The effect of orientation has been discussed in detail by John (1979) for the Arlberg Tunnel (discussed previously in this section).

(4) Large Scatter - The data analyzed shows a larger scatter for all the cases, with considerable overlap of adjacent "ground classes".

Summary - Preconstruction Phase

Empirical methods have to accomplish three objectives during the preconstruction phase. These are to provide:

(1) Ground classification

(2) Ground support relations

(3) Performance predictions

Every method has been modified to conform to a "standard" classification procedure which uses two "comprehensive" parameters: 'strength' and 'spacing'. For most methods it is still necessary at present to determine the subparameters that lead to the comprehensive parameter, or it is necessary to use substantial judgment in order to determine the parameters directly. This is particularly true for the strength parameter.

In each of the methods, ranges of 'strength' and 'discontinuity' parameters are related to a ground class and support requirement For Barton's, Bieniawski's and Wickhams's methods, these boundary curves are smooth mathematical curves on the 'strength' - 'spacing' plot. For the Franklin and Louis methods, the curves have been derived empirically.

Ground classes are related to support requirements using the original relations of the methods that are described in Section 3.

Performance can be predicted, but only to a limited extent.

5.3 CONSTRUCTION PHASE

5.3.1 Introduction

During the construction phase three basic tasks must be performed:

(1) Update relations between geologic geometric parameters and excavation-support procedure (ground-support excavation relations).

(2) Update performance predictions, i.e., relations between geologic geometric parameters and the excavation support procedure on one hand, and performance on the other hand.

(3) Predict excavation-support procedures and performance for the next round using the updated relations (1) and (2).

Updating ground-support (excavation) relations concerns two aspects: Updating of the ground conditions, and updating of the ground-support (excavation) relations. Ground (geologic/geometric) conditions may be different from the predicted ones. Even if they were exactly as predicted, the relation to support (excavation) may be incorrect, usually due to an additional factor which has not previously been considered (e.g. size of tunnel). (Note: Observed performance will be a criterion to judge ground-support relations.) In most cases neither the encountered ground conditions nor the actual ground-support relation will correspond to the predicted ones, and will have to be corrected through updating.

Updating of performance predictions consists of correcting the relations between ground condition, excavation-support procedure and performance that were established in the preconstruction phase. Basically, the relations between ground class and performance for a particular excavation procedure will be modified.

It is important to realize that updating in tasks 1 and 2 not only consists of the "passive" process of correcting the predicted conditions and relations to fit encountered ones, but the active updating process as well, i.e., the physical adaptation of excavation-support procedures.

Once reasonably updated ground-support and performance relations are available, it becomes possible to make predictions for the next excavation steps. It is actually the goal of the entire updating process to achieve accurate predictions of the excavation procedure and support required in the next round (excavation step) based on observations in previous rounds.

The three updating tasks and associated procedural recommendations will now be discussed in detail.

5.3.2 Ground Classification

The information that is available during the construction phase is more detailed than the information available during the preconstruction phase. Thus, the "comprehensive" or simplified parameters used in the preconstruction phase are replaced by more detailed parameters describing ground conditions. Since it is not known whether the parameters used in a particular empirical method accurately describe the significant factors, a more detailed record should be kept of the geologic/geometric parameters than the particular method requires. (A recommended record-keeping procedure will be described in Section 5.3.6.

The same procedure applies to record-keeping of excavation-support and performance data.)

The following parameters should be observed and recorded:

- Lithology (Rock Type)
- Schistosity
  - Type
  - Orientation
- Discontinuities
  - Joints
  - Shears, described by
  - Orientation
  - Spacing
  - Thickness
  - Strength
    - : Continuity (persistence)
    - : Filler
    - : Uneveness
- Water
  - Location
  - Rate of inflow
  - Change with time
- Behavior of ground during tunneling process
  - Overbreak
  - Formation of new fractures
  - Squeezing
  - Sliding of blocks

This information can be obtained and recorded in the process of normal geologic mapping.

Although desirable and highly recommended, mapping is often not done in practice, in which case the following information should be recorded at minimum:

- Lithology
- Description of Jointing (in general descriptive terms)
- Water Conditions
- Anomalies

The detailed information will then be transformed into the detailed parameters of the particular empirical method. In addition, combining the detailed parameters with the "comprehensive" ones used in the preconstruction phase will make a comparison between predicted and actual ground class possible. Ground class predictions for following sections of the tunnel can be updated accordingly.

However, another consideration enters into ground classification updating. The detailed parameters may not adequately describe a ground class. For instance, the appearance of foliation <u>surfaces</u> in a particular tunnel may be important. Rather than changing ground classes, this will lead to the creation of subclasses. However, the significance of additional characteristics, or in contrast, the insignificance of a particular parameter, can only be determined in conjunction with the updating of ground-support relations.

5.3.3 Updating Relations Between Ground Conditions and Excavation-Support Procedure

Analogous to ground class updating, a detailed record of the excavation support procedures is required. The record should consist of the following information:

Excavation - Support Sequence:

- Excavation method
  - Drill and blast: blasting pattern, powder factor
  - Machine bored
    : Details of machine operation
    : Thrust force
    : Power consumption
  - Heading and benching or full-face; length of heading and benching.
  - Invert (distance to face, construction sequence)
  - Work interruptions
- Placement sequence of support
  - Time frame, i.e., after each round or later
  - Sequence (placement of shotcrete in layers)
  - Details (e.g. contraction slots)

- Quantities of support
  - Thickness of shotcrete
  - Wire fabric (type, location, number of layers
  - Steelsets
    - : Type
    - : Spacing
    - : Details of installation
  - Lagging
    - : Type
    - : Backpacking
    - : Procedure
  - Bolts
    - : Type
    - : Grouted or not
    - : Resin grouted?
    - : Prestress
    - : Length
    - : Spacing
    - : Location
    - : Placement with respect to tunnel advance
  - Face support
    - : Breasting
    - : Shotcrete
  - Invert support (slab, bolts, arch)

- Support performance (visual observations)
  - Shotcrete
    - : Development of cracks
    - : Shear failures
  - Steelsets
    - : Buckling
    - : Punching failures of footings
  - Bolts
    - : Bending and tearing off of boltplates
    - : Load in bolts

Usual construction records do include most of this information, and the important matter is that of putting it together with the information on ground conditions in a timely manner. (As will be mentioned in Section 5.3.6, Record-Keeping, it is extremely important to record ground conditions and excavation-support procedures on the same sheet, or by some other method which makes comparisons readily obtainable.)

Updating starts with noting when changes in geologic/geometric (ground) conditions required changes in excavation-support procedures. At this point, one of the advantages of observational adaptable methods comes to bear, in that changes in ground conditions are clearly reflected by changes in excavation-support procedures (monitored performance can be used together with geologic/geometric information to decide on such changes - see Section 5.3.2).

The ground parameters (characteristics) whose changes in turn cause changes in excavation-support procedures are the significant parameters that must be included in the ground classification.

The significant parameters and their observed states are then listed together with the associated excavation-support method. This is the updated relation between ground conditions and excavation-support requirements (Fig. 5.8). This ground-support relation table is, in principle, updated throughout the tunnel. Updating ends when parameter changes do not cause excavation-support changes.

It is possible to again derive the "comprehensive" parameters by combining the detailed parameters and then establishing updated relations between ground class and excavation-support method. This may be useful for future applications, but unnecessary for the particular tunnel.

5.3.4 Updating Performance Predictions

The principle followed here is similar to that for updating of ground-support relations. Further, the records obtained for ground conditions and excavation-support are an integral part of the updating of performance relations. In addition, detailed performance records should be kept that consist of:

Monitored Performance

- Types of Instruments
  - Extensometers
  - Change of base section (convergence)
  - Geodetic survey

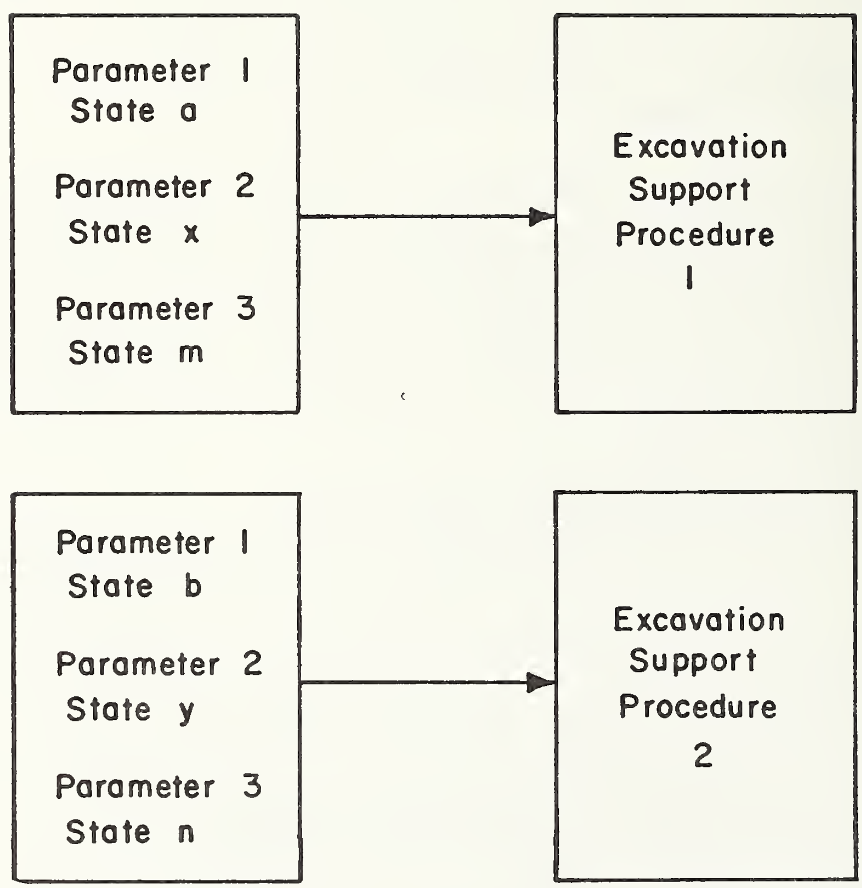


FIGURE 5.8 DETERMINATION OF SIGNIFICANT PARAMETERS

- Deflectometer
- Inclinometers
- Stress Cells (contact stresses, stresses in support elements)
- Bolt load cells

- Detailed Installation Procedure
  - Location in cross-section
  - Direction with respect to tunnel
  - Detailed placement sequence with respect to excavation-support sequence
  - Details of placement technique (e.g. drilling technique for boreholes, grouting procedure)
- Monitoring Procedure
  - Taking of initial reading (with respect to time frame and location in tunnel in comparison to excavation-support sequence)
  - Measurement sequence (intervals of measurements, also with respect to excavation-support sequence)
- Status of Instruments
  - Properly working
  - Malfunction (cause)
  - Damage

General Observations

- Performance of Ground
  - Formation of new fractures

- Squeezing
- Sliding of blocks
- Overbreak

- Support Performance
  - Shotcrete
    - : Development of cracks
    - : Shear failures
  - Steelsets
    - : Buckling
    - : Punching failures of footings
  - Bolts
    - : Bending and tearing off of boltplates
    - : Load in bolts
    - : Rupturing of bolts
  - Lagging
    - : Bending of lagging
    - : Squeezing through of rock

Note that the availability of this type of information depends upon the monitoring program and details of instrumentation, which will be described in Section 5.3.7. One can assume that observational methods will include the necessary monitoring program to provide the above-mentioned information. If this is not the case, updating of performance predictions will have to be reduced accordingly.

In the preconstruction phase relatively coarse performance predictions for the initial support have been made for expected combinations of ground condition and excavation-support methods. The listing of ground characteristics (parameters and their states) together with the excavation-support procedure actually used will be combined with monitored performance.(Fig. 5.9)

Performance parameters must be simplified and cannot include the entire set mentioned above. Again, parameters that change markedly will be considered significant and must be recorded. In addition, practicality must be considered (extensometers may be required in 'loosening' ground conditions; however, their installation near the face may hinder the advance. They may therefore not be a standard monitoring instrument, but only used in test sections. The updated listing of geometric/geologic parameters with the associated support-excavation characteristics will provide the empirical relations for the particular tunnel. (Again, updating basically continues throughout the tunnel until the relations are satisfactorily established.)

For future applications it may again be useful to transfer the observed parameters back into the "comprehensive" ones in order to obtain updated preconstruction relations.

Updating of performance predictions has until now been concerned only with initial supports. It is possible to extend the updating of performance predictions (and the adaptation of the structural dimensions and materials) to the final support. This requires the following records, which are usually obtained in a few test sections rather than continuously:

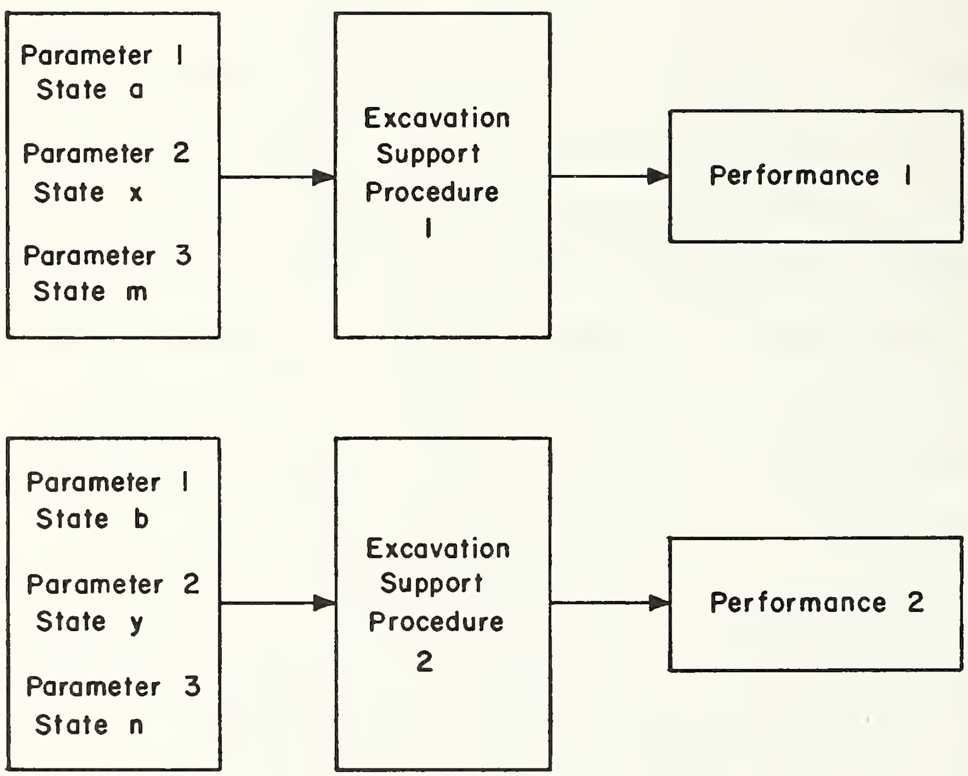


FIGURE 5.9 SCHEMATIC OF PERFORMANCE UPDATING

- Dimensions of Support
- Quality of Material
- Monitored Stresses - tangential in liner, contact stresses between initial and final support
- Monitored Deformations

This information is related to the ground characteristics, excavation-support methods and performance of initial support at the same site. Of main concern to this prediction is the rate of deformation of the initially supported tunnel at the time of the placement of final support. This is due to the fact that an observation-adaptation process in the true sense cannot be used; adaptation of the final support after placement is usually no longer possible. Deformation rates just prior to placement will be used to adapt final support requirements. This procedure has been used successfully at the Arlberg Tunnel where dimensions and thickness of the concrete liner were adapted on the basis of deformation rates (see Steiner, Einstein and Azzouz, 1979).

Seeber (1979) has used an analytical procedure to predict liner thickness on the basis of the expected residual convergence of an unsupported tunnel; however, he does not indicate how this residual convergence can be predicted.

5.3.5 Prediction of Excavation Support Procedures (and Performance) for the Next Excavation Step

The principal use of the updated ground support and performance relations in observational (adaptable) procedures is to predict the optimum excavation process and the support for the next round (support may be adapted to some extent after the round has been excavated; however, support may have to placed rapidly, particularly in unstable conditions, which requires a reasonably accurate prediction (see Section 4.3)).

The problem can basically be reduced to a prediction of ground conditions that will be encountered, and the associated prediction of excavation-support method and performance making use of the previously derived empirical relations. Ground conditions that change in a continuous manner can be predicted through trend observations. The detailed records on ground conditions described previously provide the necessary information. Not only the conditions encountered in a particular round will be recorded, but the changes from the conditions in the previous round as well (Fig. 5.10).

The following characteristics should be considered for the purpose of trend observation:

- Lithology
- Schistosity
- Jointing
- Shear Zones
- Water Conditions

Again, an updating procedure will be employed to make use of these observations. The characteristics (parameters) and their change from round i-1 to round i are considered significant if the characteristic causes a change in support excavation method.

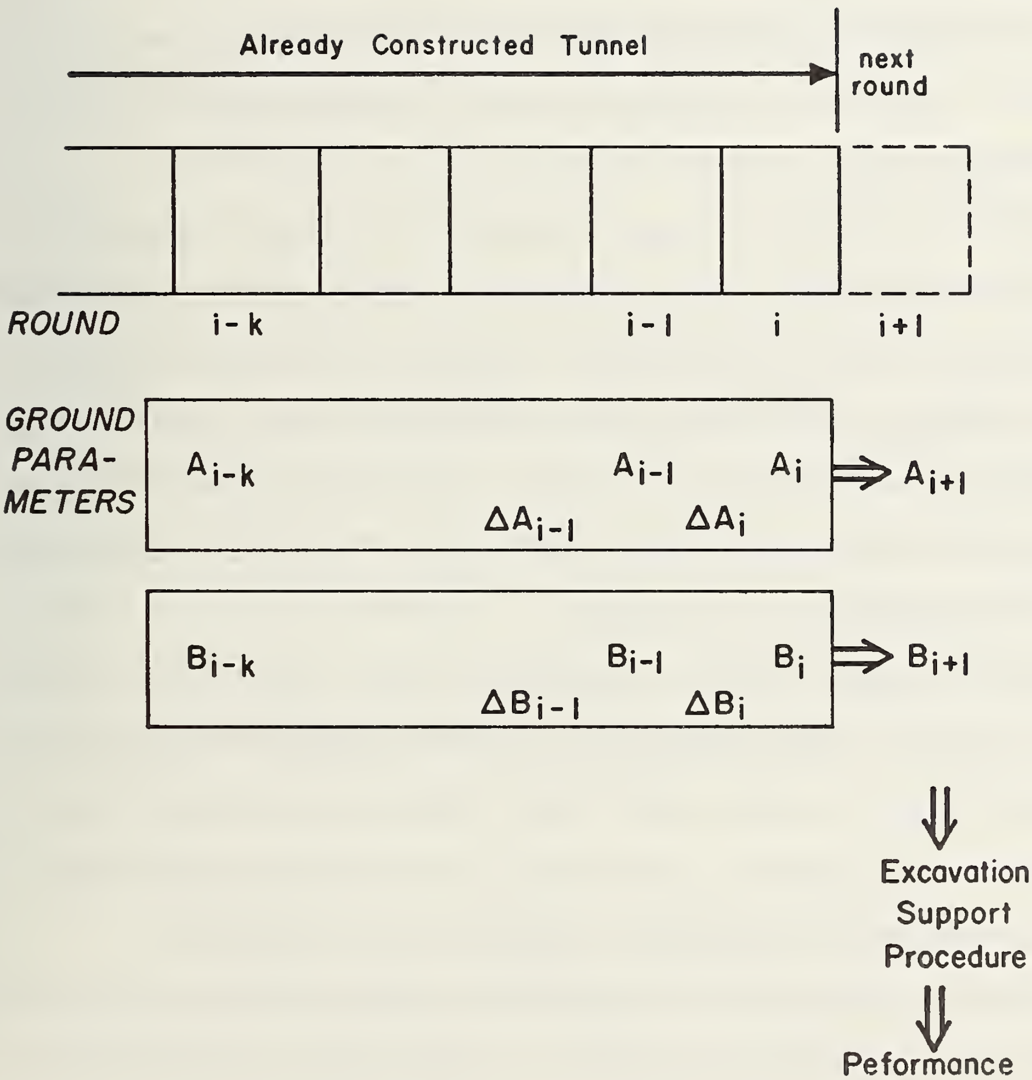


FIGURE 5.10 SCHEMATIC ILLUSTRATION OF PREDICTION FOR NEXT ROUND (PREDICTION AHEAD)

This is usually a subset of the significant ground parameters (see Section 5.3.2), but it may encompass the entire set. Experience with such parameter trends is listed and updated throughout the tunnel, and it provides the basis for predicting ground characteristics having continuous trends. The systematic procedure will have to be supplemented by subjective judgment of the geologist and other decision-making personnel (such as the project manager).

Singular features or abrupt changes can often be predicted by the same process, e.g. shear zones are often related to changes in weathering or jointing patterns. However, this may include very subtle changes that cannot be reflected in the observation procedure outlined. Thus, subjective assessment becomes critical. The judgment of experienced personnel cannot be replaced by even a very refined systematic procedure.

Finally, in the area of predicting ground conditions in the next round, certain features enter into the prediction that are not revealed by any trends (e.g. water dammed by an impervious fault zone). Only physical exploration ahead of the face can provide the required information (probehole in the face).

It has been mentioned at various points that detailed record-keeping and monitoring are required. This process must be dealt with in greater detail.

5.3.6 Record Keeping

In the updating procedure for ground information, information on excavation and support and performance data must be related to one another. Thus, all this information should be presented in a single comprehensive format, best achieved with an annotated map that includes all information on a single drawing. The parameters that should be included are those listed in Sections 5.3.3, 5.3.4, and 5.3.5. The documentation of the Tauern Tunnel (Fig. 5.11) is a good example of how this can be achieved.

5.3.7 Monitoring

A monitoring program must consider expected performance of the ground-support combination. "Loosening" behavior requires a different monitoring procedure than does squeezing ground. In "loosening" ground, little deformation may be required to cause a failure since loosening occurs primarily in the crown. Monitored performance must focus on small vertical movements in the crown. Cording et al. (1975) and Cording and Mahar (1978) recommend use of extensometers that are placed in the crown near the face to monitor these changes. In particular, they recommend extensometers installed in boreholes from the ground surface or from existing nearby excavation, so as to not hinder the excavation process.

For tunnels in squeezing ground conditions, the performance monitoring principles used in the Arlberg Tunnel may serve as an example. Two types of monitoring sections were used:

- o Standard monitoring section (Fig. 5.12)
- o Principal monitoring section (Fig. 5.13)

In the standard monitoring sections the horizontal convergence is monitored at one to three elevations (H1 to H3 in Fig. 5.12). In addition, settlements in the crown are monitored. In

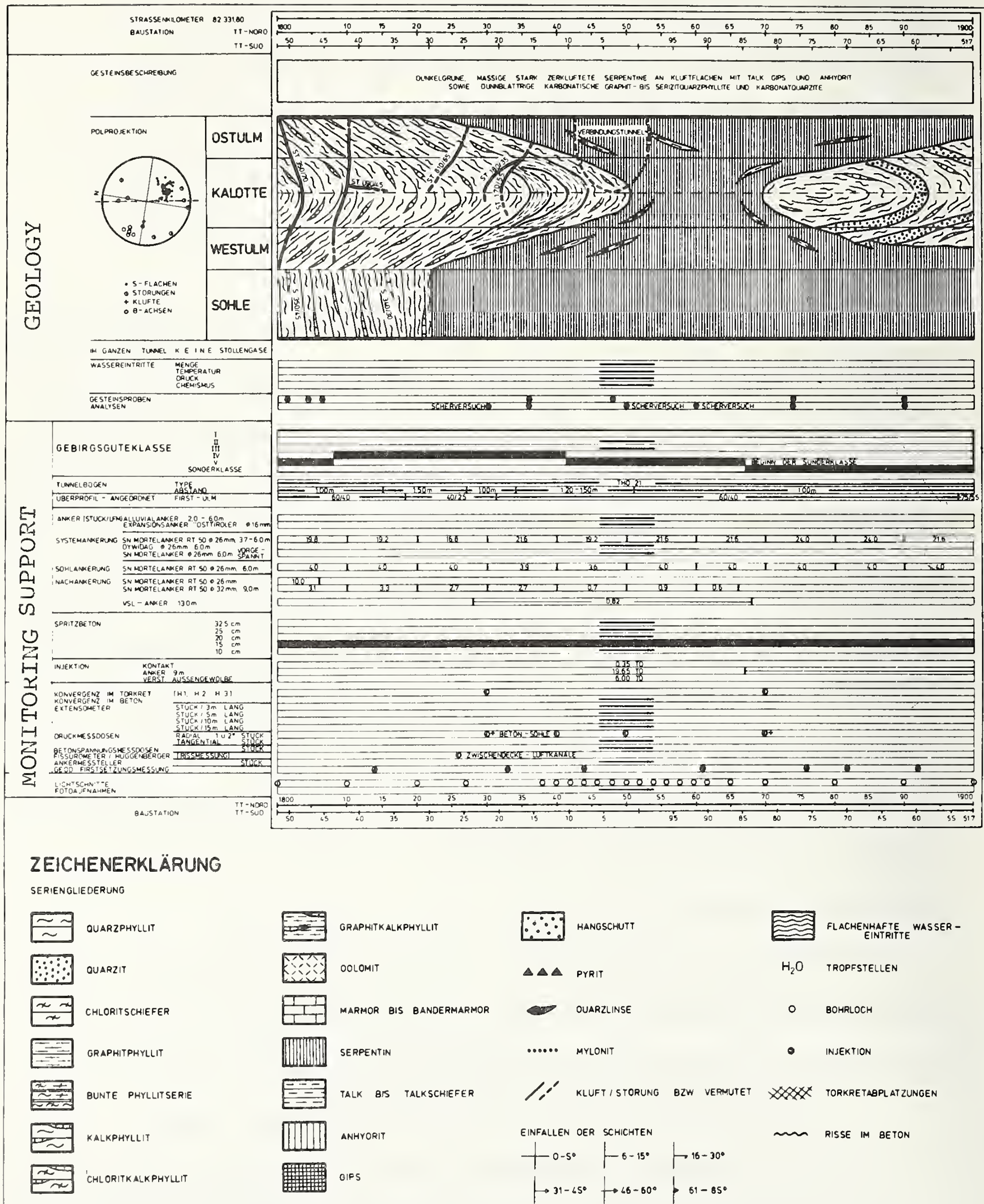
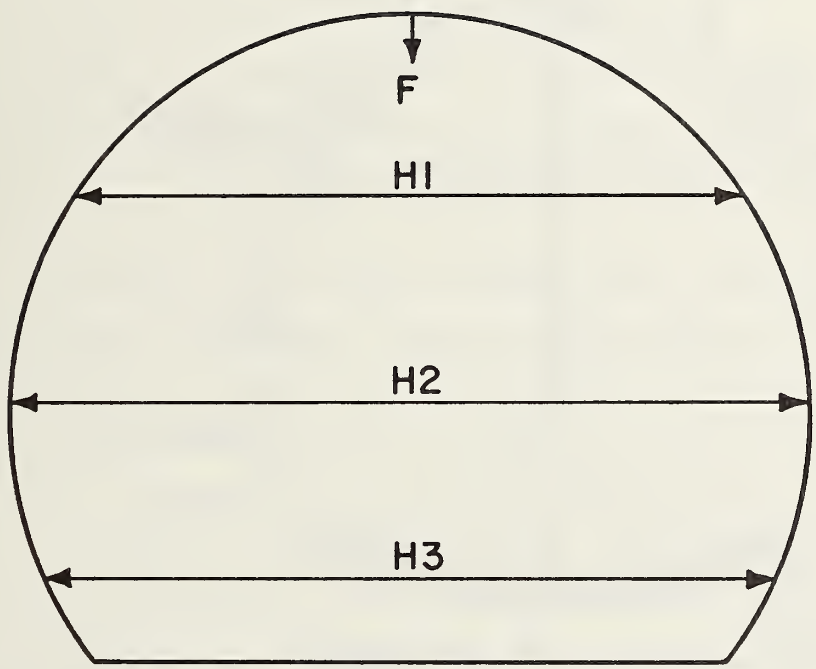


FIGURE 5.11 DOCUMENTATION OF TAUERN TUNNEL
(STATION KM 1.8 to 1.9)
(FROM TAUERNAUTOBAHN,1978)



H1, H2, H3 = Bases to monitor convergence

F = Crown settlement

FIGURE 5.12 STANDARD MONITORING CROSS SECTION

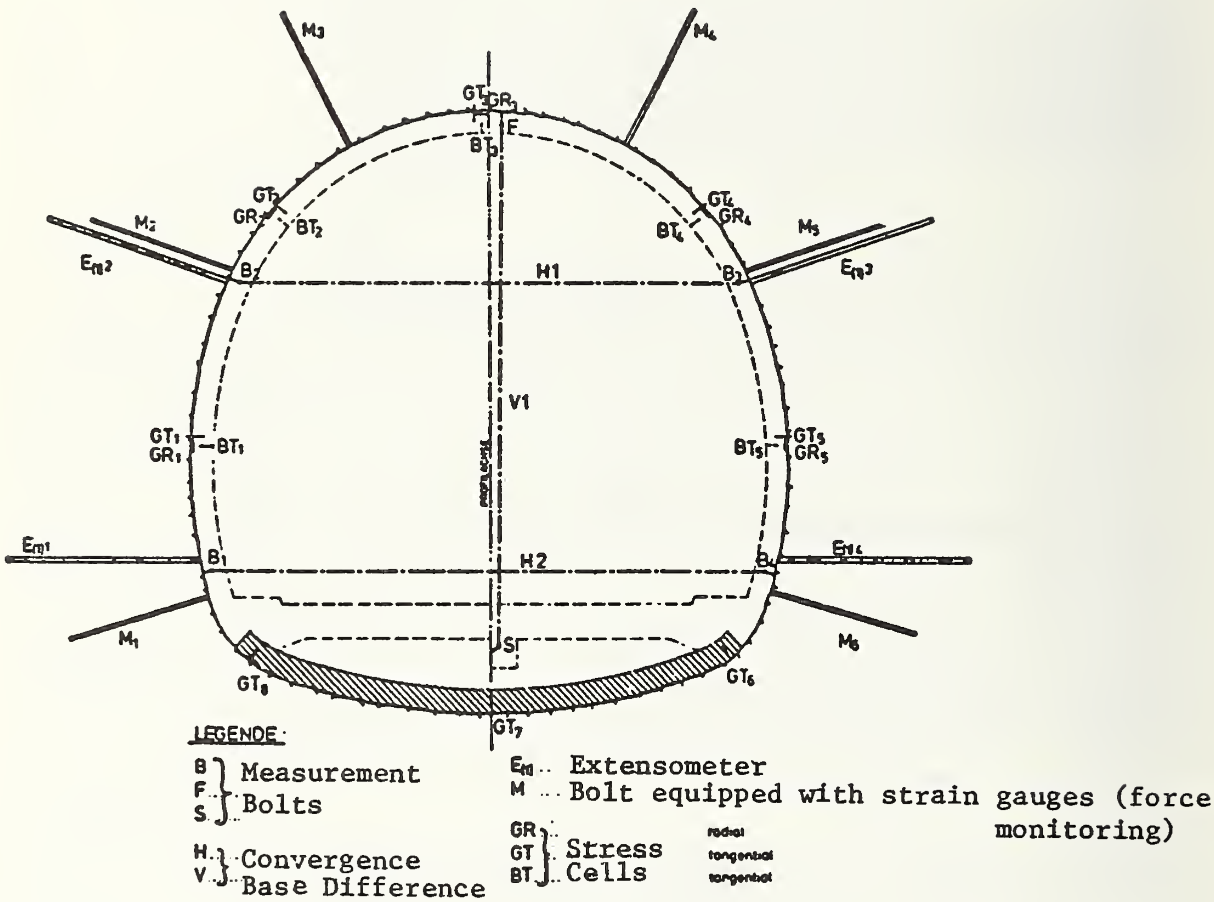


FIGURE 5.13 PRINCIPAL MONITORING CROSS-SECTION
(AFTER MAYRHAUSER,1976)

the principal monitoring cross-sections additional measurements are taken, including extensometers. Also, bolt forces are monitored by load cells or strain gauges (M1 to M6). Stress cells are placed between ground and shotcrete (GR) and tangentially in the shotcrete (GT). The performance of the final liner will also be monitored with stress cells (tangential and radial (BT)), as is the convergence of the final liner.

The spacing of the principal monitoring cross-sections varies. In the western section of the Arlberg Tunnel (Seeber, 1979) the spacing was reduced to 250 meters in the most unfavorable conditions. Standard monitoring sections were spaced as close as 10m. As ground conditions improved, the spacing of the principal monitoring sections was increased to approximately 1,000 meters.

In the Tauern Tunnel the spacing was also adapted to the adversity of the ground conditions and varied from 100 to 1,000 meters, the spacing of standard sections varied from 15 to 100m.

<u>Summary - Construction Phase</u>

During the construction phase the preconstruction predictions must be updated. This requires:

(1) Updating ground classification and ground support relations.

(2) Updating performance predictions.

(3) Predicting ahead for the next round based on relations obtained in (1) and (2).

For all three tasks it is important to keep a detailed record. Using this as a basis for systematic comparisons, it is possible to determine the significant parameters and to update ground-support performance relations, and to improve the capability to predict excavation-support requirements for the next round.

5.4 CONCLUSIONS

Modifications of existing empirical methods and updating procedures have been suggested in order to make these methods applicable to observational tunneling procedures. As mentioned before existing empirical methods fulfill the requirements for an observational approach to various degrees. The NATM basically follows the described procedure although largely qualitative. Since observational procedures are already incorporated it is not discussed here.

Deere, Merritt and Cording's 'descriptive' classification together with the RQD-support relation also fulfills many of the requirements for an observational empirical method. As a matter of fact the 'descriptive' classification was created just for the purpose of a preconstruction classification method in which a more detailed classification like Deere's et al. RQD-support relations could be incorporated during the later stages of the preconstruction phase and during construction, where also observations and updating is incorporated.

The Deere, Merritt, Cording 'descriptive' classification and the detailed RQD-support relations could be better tied together by introduction of a "strength" parameter. This modified RQD-support relation can be easily updated by redrawing class contours on the RQD-strength diagram.

The Franklin/Louis method is structured along the lines of "strength" and "spacing" parameters and can thus be used as it is in the preconstruction phase of an observational approach. The necessary updating during construction can also be easily accomplished by shifting the class contours that were determined prior to construction to fit the encountered conditions.

The methods by Barton et al., Bieniawski and Wickham et al. require a condensation of their multiparameter characterization in the preconstruction phase. It was shown how each of these methods could rely on a 'strength' and 'spacing' parameter and how these two parameters are related to teh detailed parameters that these methods use. This will make it possible to use either the condensed parameters or preferably the detailed parameters for updating during construction. Such an updating would basically lead to a shifting of class boundaries as is proposed for Deere's, Franklin's methods and as routinely done in the NATM. At the present time the condensed parameters, particularly the strength parameter can only be determined using substantial judgement. It is hoped that future work will provide a methodology and detailed guidelines on how to derive these parameters.

One can therefore conclude that in addition to the observational NATM, methods can be modified to follow an observational approach. Consistent updating has the great advantage that many of the previously mentioned limitations of empirical methods (Section 3) are no longer valid. Updating the empirical relations and adapting the tunneling process to fit encountered conditions brings tunnel design and construction closer to the desired optimum. The main purpose of this discussion and initial development was to show that empirical-observational approaches are possibel with all methods. Nevertheless, it can be stated that in addition to the NATM, Deere's et al. 'descriptive' classification combination with RQD-support relations and updating as well as the Franklin/Louis method come already very close to the recommended observational procedure.

6. CONCLUSIONS

6.1 INTRODUCTION

The complexity of ground and structural behavior, and particularly of ground-structure interaction in tunneling makes it difficult to understand the underlying principles. Consequently, behavioral models are only approximate, and analytical predictions which must be based on such models are either generally inaccurate or accurate only over very limited ranges of application. Empirical methods that avoid the use of an explicit model by relating ground conditions to observed prototype behavior have therefore played a major role in tunneling. However, there is considerable uncertainty among tunnel designers and contractors regarding the relative merit of many existing methods, particularly with the development of new methods over the past decade.

Comparative predictions made by different methods lead to substantially different results. Some methods require a minimum of easily determinable parameters, while others require a large set of relatively complex parameters. Also, some methods are based on qualitative descriptions of ground, while some are highly quantitative. Subjective assessment by the user may or may not be required. In addition, many methods attempt to relate ground conditions and support requirements without or with only minor consideration of tunnel construction procedures.

Since potential users may gain considerably from thoroughly knowing the applicability of various empirical methods, a systematic review of well known methods was conducted. The application methodology for each method was described as accurately as possible followed by a discussion of the development and underlying philosophy. Each method was compared to a set of criteria which made it possible to identify advantages and limitations. This comparison led to a set of general conclusions on the present state of the art of empirical tunnel design and construction. The conclusions were stated in Section 3, but will be reiterated in an abbreviated version in Section 6.2 below.

These conclusions show that in many applications empirical methods would gain considerably if support requirements were adapted during construction. Some empirical methods incorporate such an adaptation procedure through an observational approach, however, many do not, or not explicitely. It was therefore examined if and how existing empirical methods can be used in an observational approach, by first establishing the criteria that need to be fulfilled and then making suggestions on how the methods could be modified; the conclusions of this attempt are presented in Section 6.3.

6.2 PRESENT STATE OF EMPIRICAL METHODS IN TUNNEL DESIGN AND CONSTRUCTION

Empirical methods that relate ground conditions to tunnel support and excavation are appropriate tools substituting or complementing analytical methods. Yet the complexity of the problem affects empirical methods, as it does analytical ones, none of the methods represents all influencing factors.

Empirical methods are by definition developed from base cases. Two main consequences result:

- A particular method can only provide accurate predictions for conditions similar to those of cases by which it was calibrated (base cases).
- Applying different methods to the same case will usually lead to different predicitons.

In order to check the applicability of a method in a particular case the user has to study the base cases of every method he wishes to apply and compare them to the conditions of the application case. This should be done for every application, and requires a thorough and detailed consideration of ground conditions.

Present empirical methods frequently over-estimate support requirements, the degree of over-estimation is usually not known. On the other hand, a method applied outside the range of base cases may no longer be conservative and lead to unsafe supports. Ground support pressure relations or rock load approaches are even less reliable than direct ground support relations and should not be used unless not only ground conditions but construction procedure, support type and quantity are similar to the base case.

All methods consider only a limited set of parameters. Thus even more complicated methods are not complete. With increasing number of parameters the data base per parameter decreases leading to an increase in parameter uncertainty. Methods with a small number of parameters but a large data base (per parameter) may lead to more precise results than very complex methods do. (However, if a method has been developed from and is applied to cases with a relatively narrow range of ground support characteristics more complex methods may be successfully used at present.)

The selection of an empirical method has to reflect the availability of information on parameters and the time limitation affecting information collection.

The inherent uncertainty and complexity of the tunneling problem makes empirical methods inevitably subjective. Subjective aspects may exist in the parameter determination by the user, or in the formulation of the method by the developer, or both. Thus, the user must rely on his own experience and judgement, or that of the developer.

The latter may be wise but does not make the method 'objective'. Subjectivity is not unique to empirical methods. For a complex problem such as tunneling, analytical methods cannot be built on first principles alone, but must also involve subjective hypotheses.

The review of empirical methods and the conclusions drawn in Sections 2 and 3 have shown advantages and limitations of empirical methods which are summarized in Section 3.7, but also strength and weaknesses of empirical methods in generally discussed in Sections 3.7 and 3.8 and summarized above. Ideally it would be desirable to compare all the methods and to recommend which of the method(s) is or are best. This is, however, impossible because in different applications different methods will be best suited. What has been attempted however is to select several characteristic conditions, one of which may dominate a particular case and to compare for each of these scenarios which methods would be best suited. The limitation of this comparison is that only one characteristic predominates; the weight of the other factors is not known, simply the fact that they are significantly less important. If more than one characteristic predominates then empirical methods have to be selected by considering their limitations and base cases.

A comparison of the empirical methods has been performed by the following scenarios, each predominated by one characteristic condition.

(1) Construction Procedure dominates.

(2) Narrow range of ground conditions and many base cases.

(3) Wide range of ground conditions and/or

(4) Use by designer with little experience.

(5) Use by designer with substantial experience.

This comparison, which is described in detail in Section 3.8 and which is intended to identify the methods best suited to the particular scenario, has led to the following results:

If Construction Procedure has a significant effect, the ground conditions vary in a wide range and also include adverse ground conditions. For such a scenario the following methods seem best suited:

- NATM
- Deere-Merritt-Cording descriptive
- Bieniawski's Geomechanics Classification
- Franklin/Louis

If the ground conditions are confined to a narrow range and there are many base cases with similar conditions one can assume that over the period of years design and construction has reached an optimum. Such a scenario may therefore indicate relatively small overdesign. Methods that are well suited for such a scenario are:

- the Q-System developed from many Scandinavian cases
- the wedge analysis method

If ground conditions vary over a wide range and/or no similar base cases are available a method with a wide data base may provide an initial 'average' estimate of support requirements; the individual case may however deviate substantially from this average condition. In such a case the 'average' estimate has to be adapted to actual encountered ground conditions with an observational method and also the ground-support relations have to be updated accordingly. Methods that cover a wide range of ground conditions and also recommend the use of observations are:

- Deere et al. RQD-support relations
- Bieniawski's Geomechanics Classification.

A designer with little experience in tunneling selects a method with easily determinable geotechnical parameters, this might be:

- Louis or Franklin's method.

If the designer is a structural engineer with little experience he might favor a rock load method because with such a method standard structural design procedures could be used. Yet such a designer must be aware of the earlier stated limitations of rock load approaches.

A tunnel designer having acquired substantial experience will be able to make judgements on support requirements and adaptation based on simple visual observations. On the other hand he will know which particular measurements might be made that best aid his judgement. The details of such methods vary from individual designer to individual designer. Two examples of methods used by experienced designers are

- NATM classification
- Deere-Merritt-Cording's descriptive classification.

Note, however, that the Deere-Merritt-Cording descriptive classification and the NATM may also be useful to the less experienced designer since they represent also attempts by their developers to transmit to a potential tunnel designer those aspects of tunneling that in their experience was important but cannot be expressed in measurable parameters.

This comparative assessment of empirical methods should be considered as highlighting some of the conclusions drawn on advantages and limitations of particular methods and of empirical methods in general. For detailed comments on these advantages and limitations the reader is referred to Sections 3.7 and 3.8, and particularly to Sections 3.2 and 3.5, discussing the individual methods.

6.3 EMPIRICAL METHODS FOR OBSERVATIONAL PROCEDURES

The difficulty to accurately determine ground properties makes it often desirable to employ an observational approach in tunneling.

As stated several times in this report there are circumstances where empirical methods are based on many cases with relatively similar ground conditions and opening characteristics. The extensive experience may lead to empirical relations that are close to the optimum. Applying such an empirical method to a new case falling into this range will therefore lead to relatively accurate prediction and updating an observational approach is unnecessary. The user should, by following the recommended examination of base cases (conclusion 1, in Section 3.8) be capable of deciding whether an observational approach is warranted or not.

Since there will be many instances where observational approaches will be necessary, it was examined whether existing empirical methods could be applied in an observational approach and to suggest modification if necessary. This required the establishment of criteria that observational empirical methods have to fulfill.

Prior to construction, empirical relations between ground and support are required which are based on a few readily determinable parameters, parameters on which more information will become available during construction in order to make observational 'updating' possible. It is also necessary to explicitely assess the uncertainty in predicting the parameter states on the tunnel level. Further, approximate performance predictions must be possible prior to construction.

In the construction phase the appropriate excavation-support procedure including initial support requirements and the final support have to be predicted based on the actually encountered ground conditions. For this purpose the actual parameter states and their trends have to be determined. In addition, the previously established ground-support relations and ground-support performance relations must be updated during construction to represent actual behavior.

Suggestions were developed on how existing methods could be modified to satisfy the requirements of the preconstruction phase and the updating sequence during construction. (Naturally, the NATM is not discussed in this context since it already fulfills the requirements. Modified empirical methods should rely on two comprehensive parameters describing ground conditions. One parameter represents 'normalized rock mass strength' (which includes persistence, waviness, tightness of disconouities, effects of fillers, strength of intack rock, water pressure and stress state. The other parameter represents 'spacing' (including the individual sets of discontinuities, the dimensions of the tunnel and the effect of discontinuity orientation.

Deere, Merritt and Cording's 'descriptive' classification together with the RQD-support relation fulfills many of the requirements for an observational empirical method. As a matter of fact the 'descriptive' classification was created just for the purpose of a preconstruction classification supplemented by a more detailed classification like Deere's et al. RQD-support relations during the later stages of the preconstruction phase and during construction where also observations and updating are incorporated.

The Deere, Merritt, Cording 'descriptive' classification and the detailed RQD-support relations could be better tied together by introduction of a 'strength' parameter. This modified RQD support relation can then be easily updated by redrawing class contours on the RQD-strength diagram.

The Franklin/Louis method is structured along the lines of 'strength' and 'spacing' parameters and can thus be used as it is in the preconstruction phase of an observational approach. The necessary updating during construction can also be easily accomplished by shifting the class contours that were determined prior to construction, to fit the encountered conditions.

The methods by Barton et al., Bieniawski and Wickham et al. require a condensation of their multiparameter characterization in the preconstruction phase. It was shown how each of these methods could rely on a 'strength' and 'spacing' parameter and how these two parameters are related to the detailed parameters that these methods use.

This will make it possible to use either the condensed parameters or, preferably, the detailed parameters for updating during construction. Such an updating would basically lead to a shifting of class boundaries as proposed for Deere's, Franklin's methods and as routinely done in the NATM. At the present time the condensed parameters, particularly the strength parameter, can only be determined using substantial judgement. It is hoped that future work will provide a methodology and detailed guidelines on how to derive these parameters.

One can therefore conclude that in addition to the observational NATM, methods can be modified to follow an observational approach. Consistent updating has the great advantage that many of the previously mentioned limitations of empirical methods (Section 3) are no longer valid. Updating the empirical ground-support relations and adapting the tunneling process to fit encountered conditions brings tunnel design and construction closer to the desired optimum. The main purpose of this discussion and initial development was to show that empirical-observational approaches are possible with all methods. Nevertheless, it can be stated that in addition to the NATM, Deere's et al descriptive classification combination with RQD-support relations and updating, as well as the Franklin/Louis method come already very close to the recommended observational procedure.

Finally, and to put the entire discussion into context, one should note that the necessity for 'observational procedures' in tunneling has been stressed from a purely technical point of view. Yet implementing an observational procedure in practice has consequences that reach far beyond the technical area. Contractual arrangements and operational procedures have to be considered accordingly, a fact that has been discussed in detail by Steiner et al., 1979 (See Vol. 4 of this report series).

7. RECOMMENDATIONS FOR FURTHER RESEARCH

As mentioned before tunnel performance is influenced by a countless number of parameters. It is doubtful whether these parameters can ever be fully understood and whether they ever can be determined prior to construction. Observational empirical methods have been proposed that can be updated during construction. For the 'preconstruction phase' several empirical methods have been modified such that they are now based on two comprehensive parameters: "strength" and "spacing", which are a combination of subparameters. The modified existing methods have scales that are not easily comprehensible. Thus these parameters may be more closely studied such that they can be interpreted in a consistent way. Areas for further research are:

(1) The "spacing parameter" might be replaced by improved statistics of the spacing of several sets (instead of RQD). It may also be normalized to tunnel diameter.

(2) The "strength parameter" is a function of joint conditions, continuity, filling, intact rock strength, water conditions, stress state, and orientation. A more consistent interpretation

of the "strength parameter" may be achieved by using already known physical principles (e.g., effective stresses and pore pressure dissipation around a tunnel). By combining the parameters following soil and rock mechanics principles an improved understanding of ground behavior around a tunnel may be achieved. This does not mean that these relations would be no longer empirical, however, already established principles would be incorporated.

(3) Performance predictions are extremely limited and only little empirical data is available. Thus tools should be developed that allow the prediction of performance considering support excavation methods. This may be achieved by carefully collecting observations and interpreting them and developing relations.

Empirical Methods will always play an important role in tunnel design and construction. With careful development of these methods they will supplement analytical methods. Vice versa empirical methods can be made more consistent by using already established soil and rock mechanics principles. The two approaches do not have to exclude each other but should supplement each other.

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APPENDIX A

GROUND CLASSIFICATION FOR THE NEW AUSTRIAN TUNNELING METHOD

Ground classification used in conjunction with the New Austrian Tunneling Method (NATM) are compiled in this appendix.

The classification procedure considering six classes is applicable in both alpine transmountain tunnels and shallow tunnels. Each class may, however, be subdivided to suit specific conditions; in particular, different subclasses for transmountain and shallow tunnels are often used.

GROUND CLASS	GROUND CONDITIONS/DEPTH	EXCAVATION		STAND-UP TIME	SUPPORT	DEFORMATIONS	REMARKS	REFERENCES
		METHOD	ROUND LENGTH					
I a Fig. A-1 (left side)	Intact rock of high strength relative to the major principal stress (at any depth). (The soundness and strength of the rock is evidenced by blasthole traces at the circumference that are visible for at least 50% of their length.) Few widely spaced tight joints, well interlocked. Water does not influence the stability of the rock mass. Few afterbreaks of unfavorably positioned rock. For large overburden (or high principal stresses), the rock tends to spall.	Full face excavation. Smooth blasting with millisecond intervals.	The round length is not limited by rock conditions. It is limited by the equipment (drills) to 3 to 4 m.	Weeks in the crown. Unlimited at the springlines.	Welded wire fabric (1.78 kg/m^2) in the crown fixed with rockbolts (length = 1.5 m). Bolt spacing generally larger than 2 m; has to be adapted to secure single rock blocks, which might fall out. No time limit for placement of support, except for spalling rock where the bolts have to be placed immediately after excavation.	The rock mass reaches stability after excavation. Convergence measurements after excavation show no displacements.		John, M. (1976). Mayreder Zeitschrift (1975/1976). Pacher, F. (1976). Laabmayr (1977).
I b Fig. A-1 (right side)					Shotcrete (50 mm) in the crown may be substituted for the welded wire fabric and the bolts. Shotcrete is preferred in the case where waterproofing is necessary since it serves as a base for waterproofing seals. No time limit for placement of support, except for spalling rock where the bolts have to be placed immediately after excavation.			
II Fig. A-2	The rock is more jointed and fractured than in I, but is still unweathered. (The blasthole traces at the circumference are not visible after excavation.) The joints are more closely spaced, but tight; the rock loosens due to intersecting joint sets, fall-outs from the roof and the springlines occur a short time after excavation. Closely jointed rock, but tightly interlocked falls into this class even if the joints are filled with gouge of low strength, but are well interlocked due to small scale folding. No depth limit given. Ground water does not chemically alter the rock; water pressure may cause loosening. For small lateral stress ratios K_o, the tensile stresses in the crown exceed the tensile strength of the rock mass. The rocks in the crown will loosen and fall out (M. John, 1976). The stresses at the springline exeed the rock mass strength (M. John, 1976).	Full face excavation. Smooth blasting is required.	Maximum of 3 m.	Crown: several days. Springlines: several weeks.	Joint grooves are sealed with shotcrete immediately after each round. Welded wire fabric (1.78 kg/m^2) is placed in the crown and covered with approximately 5 to 10 cm shotcrete. Grouted, prestressed rockbolts of 15-ton capacity and bolt lengths of 3-3.5 m are placed in crown; longitudinal spacing of 2 m and circumferential spacing of 2.5 m. Additional bolting at the springline, if the joints are oriented unfavorably. Support has to be completed 20 to 40 m behind the face.	Convergence measurements show only slight deformations, however stability cannot be reached without support.	The distance at which the support has to be completed differs; e.g., Pacher and Laabmayr require 20 m, John 40 m.	Pacher (1976). Laabmayr (1977). John (1976).

TABLE A-1 GROUNDCLASSIFICATION FOR THE NATM

GROUND CLASS	GROUND CONDITIONS/DEPTH	EXCAVATION		STAND-UP TIME	SUPPORT	DEFORMATIONS	REMARKS	REFERENCES
		METHOD	ROUND LENGTH					
III a Fig. A-3	Heavily jointed rock, possibly jointed in several directions. The joints are not tight and filled with gouge. The joints are not or only to a small extent interlocked. The joints have little shearing resistance. Water does not alter the rock; clay and other water sensitive minerals are not abundant. For small lateral stress ratios (K_o), the tensile stresses in the crown exceed the "tensile strength" of the rock mass. For large K_o, the shear stresses exceed the shear strength along planes of discontinuities.	Full face excavation possible with shorter round length. Preferably partial face, heading and one bench. Smooth blasting is required. The circumferential blast holes have to be drilled 20 - 30 cm inside the final excavation line. The remaining rock at the circumference has to be trimmed with a jackhammer.	For full face, maximum 1.5 m. For partial face, maximum 3 m.	In the crown: several hours. At the springline: several days.	First a welded wire fabric (3.12 kg/m^2) and a first layer of shotcrete (5 cm) is placed. Then "ladders" welded from reinforcing bars, diameter 36 mm, spaced 150 mm and joined with bars each 250 mm are placed circumferentially in the crown (longitudinal spacing 1.5 m). Rockbolts (grouted and prestressed), capacity 25 tons, spacing longitudinally 1.5 m and circumferentially 2.0 - 2.5 m are placed in crown and at springlines The total shotcrete thickness is increased to 10 cm. For longer interruptions of the work (more than 12 h.), the face has to be supported with shotcrete (3 cm).		Ground Class III in the Arlberg tunnel does not require "ladders" or steel sets.	John, M. (1976) Laabmayr (1977) Pacher (1976) Mayreder (1975/76)
III b	Similar to IIIa; in addition, some squeezing from the sides occurs, which is caused by unfavorable orientation of joints, bedding planes, faults, shear zones or tectonic stresses.				Same as IIIa, except that the steel sets (welded reinforcing bars) are also placed down to the springlines and connected to the bolts. The final shotcrete thickness is increased to 15 cm. In the case of invert heave, an invert arch has to be placed no later than 30 days after excavation, which corresponds to a distance of approximately 30 m from the heading to invert closure.			Pacher (1976)
III c	Same as IIIa, but shallow overburden and mainly soils. (Deformation has to be limited to prevent settlements on the surface.) Soil with high strength, like highly overconsolidated clay, well cemented gravel and sand, but not yet approaching rock-like conditions. Possible zones of weaker material between cemented layers. Possibility of some ravelling and piping.	Heading and one bench with partial face TBM. It may be necessary to loosen rock by blasting.	The round length is 1.0 m for the heading and 4 m. for the bench.	In the crown: several hours. In the springline: several days.	First a welded wire fabric (3.12 kg/m^2) is placed. Light steel sets (21 kg/m) are placed each 1.0 to 1.5 m. Shotcrete thickness = 15 to 20 cm. Rockbolts (length = 3 m) at the springline (2 to 3 on each side) through steel sets; circumferential spacing 3 m. No face support is necessary No distance heading to invert closure has been reported.		Nürnberg (Hasenbuck) This class has been added by the authors of this report based on available information.	Beton and Monierbau (1976)

TABLE A-2 GROUNDCLASSIFICATION FOR THE NATM

GROUND CLASS	GROUND CONDITIONS/DEPTH	EXCAVATION		STAND-UP TIME	SUPPORT	DEFORMATIONS	REMARKS	REFERENCES
		METHOD	ROUND LENGTH					
IV a Fig.A-4	Squeezing ground conditions. Completely disturbed or broken rock. Soils with some cohesion, like clayey gravels, OC-clays, undisturbed silts. Water reduces the stability (ravelling, seepage pressure), especially as gouge filler may swell (this point has to be further clarified). Due to the low strength relative to the major principal stresses, squeezing at the springlines and invert heave occur.	Partial face excavation. A partial face TBM is preferred over blasting. In the heading, a central core supports the face. (This may be substituted by shotcrete supporting the face ($t = 3$ cm). The bench has to be excavated in stages.)	In the top heading 1.0 m to a maximum of 1.5 m.	Very short in the crown (½ hour). Several hours at the springlines.	After each excavation stage, the support has to be placed. 1) Shotcrete $t = 5$ cm. 2) Steel sets = 21 kg/m and spaced at 1.0 to 1.5 m (roundlength) with forepoling sheets. 3) Shotcrete increased to 10 cm. 4) Welded wire fabric (3.12 kg/m^2). 5) Bolts through steel sets (longitudinal spacing = 1.0 - 1.5 m; circumferential spacing = 2.0 m; length = 4.0 - 6.0 m. 6) Increase shotcrete to final thickness (15 cm). Step 6 can be completed after bench excavation. The invert has to be placed after completion of the bench excavation. The time from opening of the top heading to invert closure should not exceed 30 days, corresponding to a distance of 120 to 150 m in the Arlberg (Tauern approximately 100 m).	Measurement mainly avaialble for high overburden classes - see Class IV b.	Steel sets provide immediate support.	Pöchhacker (1974) John, M. (1976) Pöchhacker (1974)
IV b Fig.A-5	Same as IVa, but high tectonic stresses.	At Tauern and Arlberg, the heading (1) is excavated by blasting, the benching performed by crawler-type excavators. In the Arlberg tunnel, the heading (1) has a height of 4-4.5 m with no core supporting the face; the first bench (2 and 3) was excavated in 2 sections (2 and 3) at a distance of 60-100 m from the face. The second bench (4/5) was excavated at a distance of 40-60 m from the first one. Invert closure was thus about 120-150 m from the heading.			The excavation procedure is the same as for IVa, but the bolts are longer and spaced more closely: Length = 6 to 9 m (2/3 of the bolts have length 6 m; 1/3 have length 9 m). Spacing longitudinally = 1.2 m. Spacing circumferentially = 0.5 m. (No forepoling sheets are placed.) Compression slots (width = 20 cm) (Fig.A-12) in the sidewalls.	Convergence measurements from Arlberg tunnel: After heading, initial displacement rates = 20 mm/day; displacement after 10 days = 50 to 70 mm; displacement rates (10 to 30 days) = 0.5 to 1 mm/day in top heading only (mainly due to vibrations (blasting)). When bench excavation started (30 to 40 days after heading), displacement rates = 35 mm/day, reaching a total displacement of 270 to 330 mm (average 300 mm) 60 days after excavation of heading (Fig. I-4.4).	Special subclasses for Arlberg tunnel where rock was heavily squeezing, but excavation was not impeded.	Mayreder (1975/76) Pacher (1975) John, M. (1976)

TABLE A-3 GROUNDCLASSIFICATION FOR THE NATM

GROUND CLASS	GROUND CONDITIONS/DEPTH	EXCAVATION		STAND-UP TIME	SUPPORT	DEFORMATIONS	REMARKS	REFERENCES
		METHOD	ROUND LENGTH					
IV c Fig.A-6	Same as IVb.				Same as Class IVb, excepth spacing of steel sets = 1.0 m; longitudinal spacing of bolts = 0.5 m; circumferential spacing = 1.0 m. Length of bolts: 40% - L = 6.0 m 30% - L = 9.0 m 30% - L = 12.0 m	Same as IVb.	Same as IVb.	Same as IVb.
IV d Fig.A-7	Same as IVa. For shallow tunnels, mainly in soils. The soil has some cohesion, slight cementation in gravels and sands. Seepage starts to be important. Piping and blowouts can be controlled, however.	The face has to be divided into heading and bench. The face may be inclined to achieve the same purpose. At Grigny, the face was supported by a central core (2) and the excavation proceeded in stages as shown in Fig.A-7	0.5 m to 1.0 m.	In the crown: very short (½ h - 1 h). At face and springline: several hours.	In heading, vertical steel sets (spacing 0.5 to 1.0 m) and double layer of welded wire fabric (weight 4 kg/m^2). Shotcrete is then applied (thickness = 20 - 25 cm). The springlines are supported by inclined steel sets to accommodate the face. Three bolts (l = 3 m) placed through each set at the springlines (spacing = 2 m circumferentially)	From Bochum Subway. Surface settlement: 16 mm. Settlement of crown: 20 mm. (No convergence measurements available). The maximum rate of surface settlement is in the order of 3 mm/day.	At Bochum Subway, shotcrete = 25 cm. This class has been added by the authors of this report based on available information.	Beton-und Moinesban (1976) Jagsch et al. (1974) Egger (1974)

TABLE A-4 GROUNDCLASSIFICATION FOR THE NATM

GROUND CLASS	GROUND CONDITIONS/DEPTH	EXCAVATION		STAND-UP TIME	SUPPORT	DEFORMATIONS	REMARKS	REFERENCES
		METHOD	ROUND LENGTH					
V a Fig.A-8	Rock heavily squeezing. Very heavily fractured rock and soils which do not run or flow, but are loose blocky debris, gravel and sands with apparent cohesion. The strength of the material is small compared to the major principal stresses. Immediately after the excavation, the ground in the vicinity of the cavity starts to move significantly, followed by heavy squeezing.	Heading and bench excavation. Excavation with partial face TBM or with excavator or shovel. Overexcavation necessary to allow for deformations. (At Tauern and Arlberg, the heading is excavated by blasting, whereas the bench is excavated by a crawler-type loader.)	The round length is 0.5 - 1.0 m (maximum)	Crown: no stand-up time. Small to a few hours at the face and springlines.	The excavation requires careful forepoling. The spacing of the steel sets (21 to 27 kg/m) corresponds to the round length (0.5 to 1.0 m). Steel sets held against the rock by the bolts (see below). If possible the forepoling sheets (thickness = 3 mm) shall be spaced circumferentially as wide as possible to allow for contact of the shotcrete with the rock. After forepoling, all free surfaces have to be supported immediately with a layer of shotcrete at the circumference 10 - 15 cm, at the face 3 cm. Welded wire fabric (3.12 kg/m^2) is placed on the shotcrete. Bolts (length 4 - 6 m) with longitudinal spacing 1.0 m and circumferential spacing 2.0 m are placed. Voids behind forepoling sheets have to be grouted. After excavation of the bench, which is supported similarly to the top headings, additional bolts (length 6 - 9 m) are placed in the crown reducing the circumferential spacing to 1.5 m and the shotcrete is increased to 20 cm. If it is impossible to drill holes, the bolts have to be driven and grouted. After excavation of the bench, the invert has to be placed. The time from heading to invert closure should not exceed 30 days, which corresponds approximately to a distance of 100m from the top heading face to the invert closure.		Pacher stresses the need to divide rock class V into one for shallow and one for deep tunnels. For shallow tunnels, class V behavior is due to heavily jointed rock, whereas for deep tunnels, the fracturing is caused by the higher stresses. From the description of the excavation procedure at Tauern.	Pacher (personal communication, 1977). Pöchhacker (1974)
V b Fig.A-9	In addition to the above conditions, high stresses (tectonic stresses, large overburden).				No forepoling sheets are used. Surface is first covered with 5 cm of shotcrete, then welded wire fabric (3.12 kg/m^2) and steel sets (21 kg/cm^2) are placed (spacing = 0.75 m). Steel sets are held by the bolts (see below). Bolts of length 6 - 12 m are placed:	The displacements are in the same order as those given for class IVb.	Special class in Arlberg tunnel for very squeezing rock and high tectonic stresses. In the Arlberg, the bolts are no longer placed through steel sets, because they were sheared off due to differential movements.	John, M. (1976)

Ground Class Vb continued on the NEXT PAGE.

TABLE A-5 GROUNDCLASSIFICATION FOR THE NATM

GROUND CLASS	GROUND CONDITIONS/DEPTH	EXCAVATION		STAND-UP TIME	SUPPORT	DEFORMATIONS	REMARKS	REFERENCES
		METHOD	ROUND LENGTH					
V b (cont.)					35% - L = 6.0 m 40% - L = 9.0 m 25% - L = 12.0 m Longitudinal spacing = 0.375 m and circumferential spacing = 1.0 m.			
V c Fig.A-10	Same as Va. For shallower tunnels, mostly in soil, gravel, sands with some apparent cohesion. Seepage gradients are not too important; the groundwater table has been possibly lowered in advance of excavation.	Partial face excavation. Heading has to be opened by partial face TBM excavation or power shovel. Possibly central core (2) to support face. Bench has to be excavated in stages. At Grigny, a temporary invert in the top heading was placed. (Fig.A-10)	Round length only 0.5 m. With the use of forepoling, the round length may be increased.	No significant stand-up time in the crown.	The opened parta have to be supported immediately with shotcrete (3 - 5 cm), including the face and temporary invert. Welded wire fabric (double layer - 4 kg/m² at Grigny) and steel aets (spacing = 0.5 m) are placed and the shotcrete increased to 25 cm in the areas which form the circumference. Bolts (length = 4 - 6 m) through each set are longitudinally spaced at 0.5 m and circumferentially spaced at 1.0 - 1.5 m. For the case, no holes can be drilled; the bolts have to be driven and grouted. The support of the springlines is placed immediately after excavation with the same spacing as in the crown.		Requires very fast setting shotcrete. Used also at Grigny for less favorable sections. This class has been added by the authors of this report based on available information.	Laabmayr, SIA (1975). Egger (1974).
VI Fig-A-11	Flowing ground. Silty clay or sand with high water pressures. Or heavily jointed and sheared rock under high overburden with water pressure.	The excavation haa to be adapted to the support measures tor each individual case. The opened aectiona are amaller than the ones in class V.		None.	This class usually necessitates grouting (chemical) or freezing. Forepoling over the entire circumference ia necessary. (Forepoling aheets thicknesa = 3 mm). The face has to be supported either by shotcrete (3 cm) or by breast boards. Light ateel aets (21 kg/m) are needed for forepoling (spacing 0.5 m). The forepoling aheeta are covered with a firat layer of ahotcrete (10 cm). A first set of bolta (length 4.0 - 6.0 m) is placed with longitudinal spacing of 0.5 - 1.0 m and circumferential spacing of 1.0 m. As the excavation proceeda, a welded wire fabric ia placed and the thickness of the ahotcrete increased. Additionel bolts are pleced (length = 6 - 12 m) between the previously pleced ones. In eddition to a closed invert, it may be necessary to plece invert bolta. Contrection slots (see IVb) mey be neceseery	No typical deformation available.	This class contains all ground conditions which are worse than Claas V and the support has to be adapted from case to case. The support given is one possiblity. Shield not possible for large overburden.	John, M. (1976) Pacher (1976)

TABLE A-6 GROUNDCLASSIFICATION FOR THE NATM

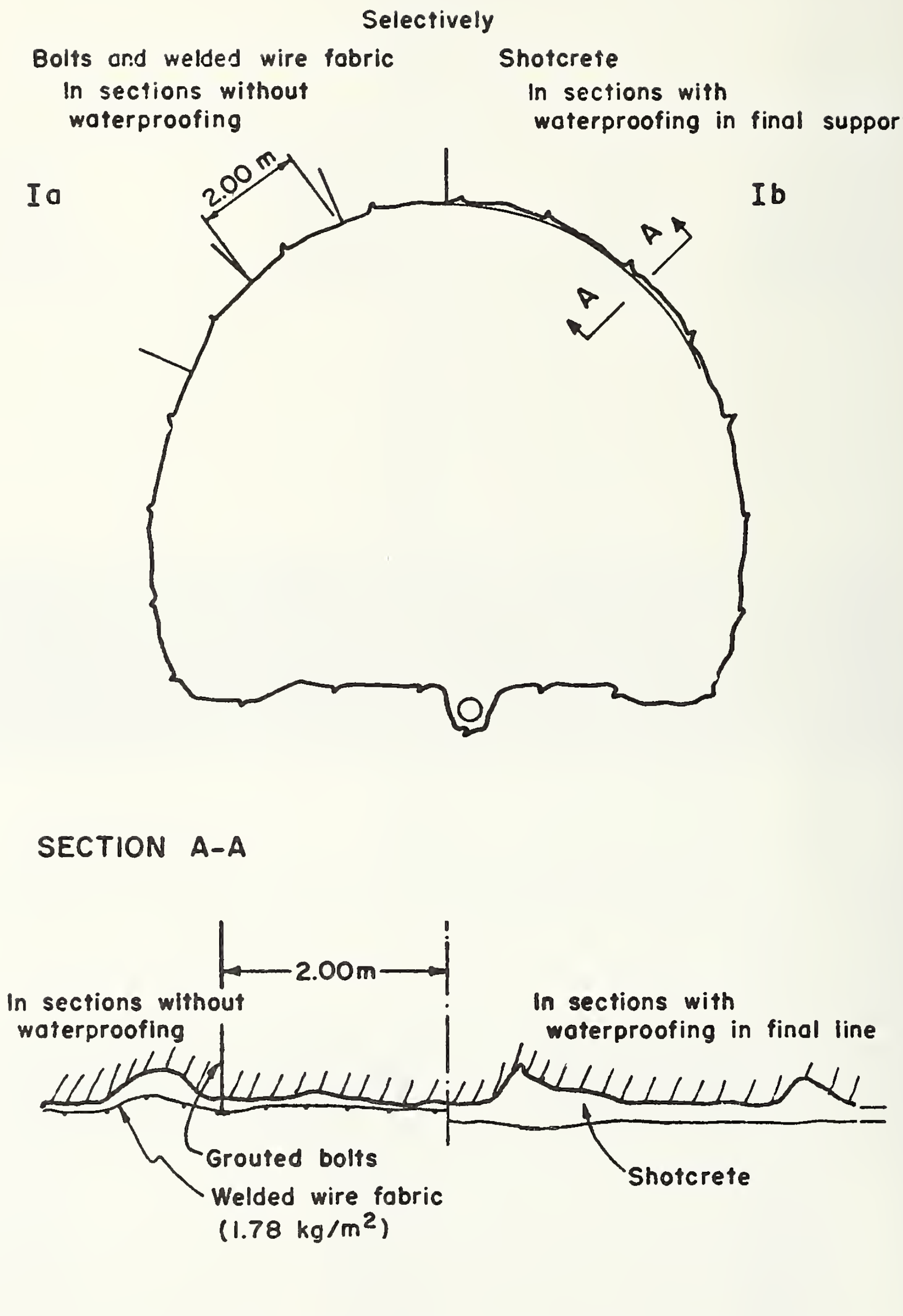
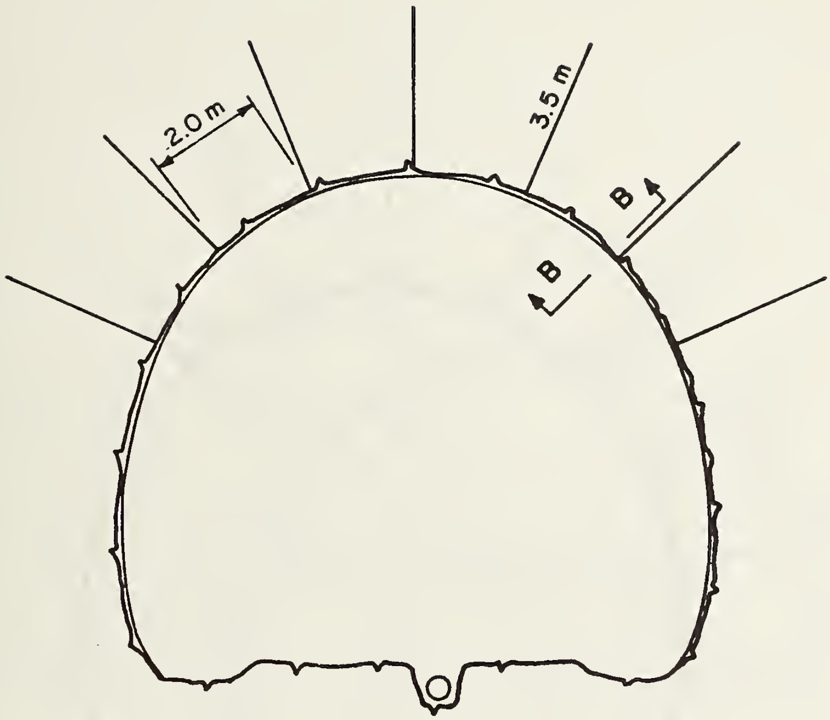


FIGURE A-1 SUPPORT FOR GROUNDCLASS I (AFTER JUDTMANN,1976)



SECTION B-B

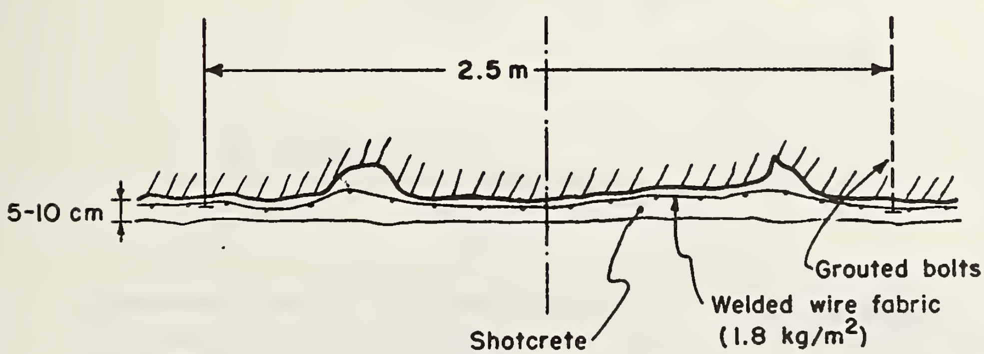


FIGURE A-2 SUPPORT FOR GROUNDCLASS II
(AFTER JUDTMANN, 1976)

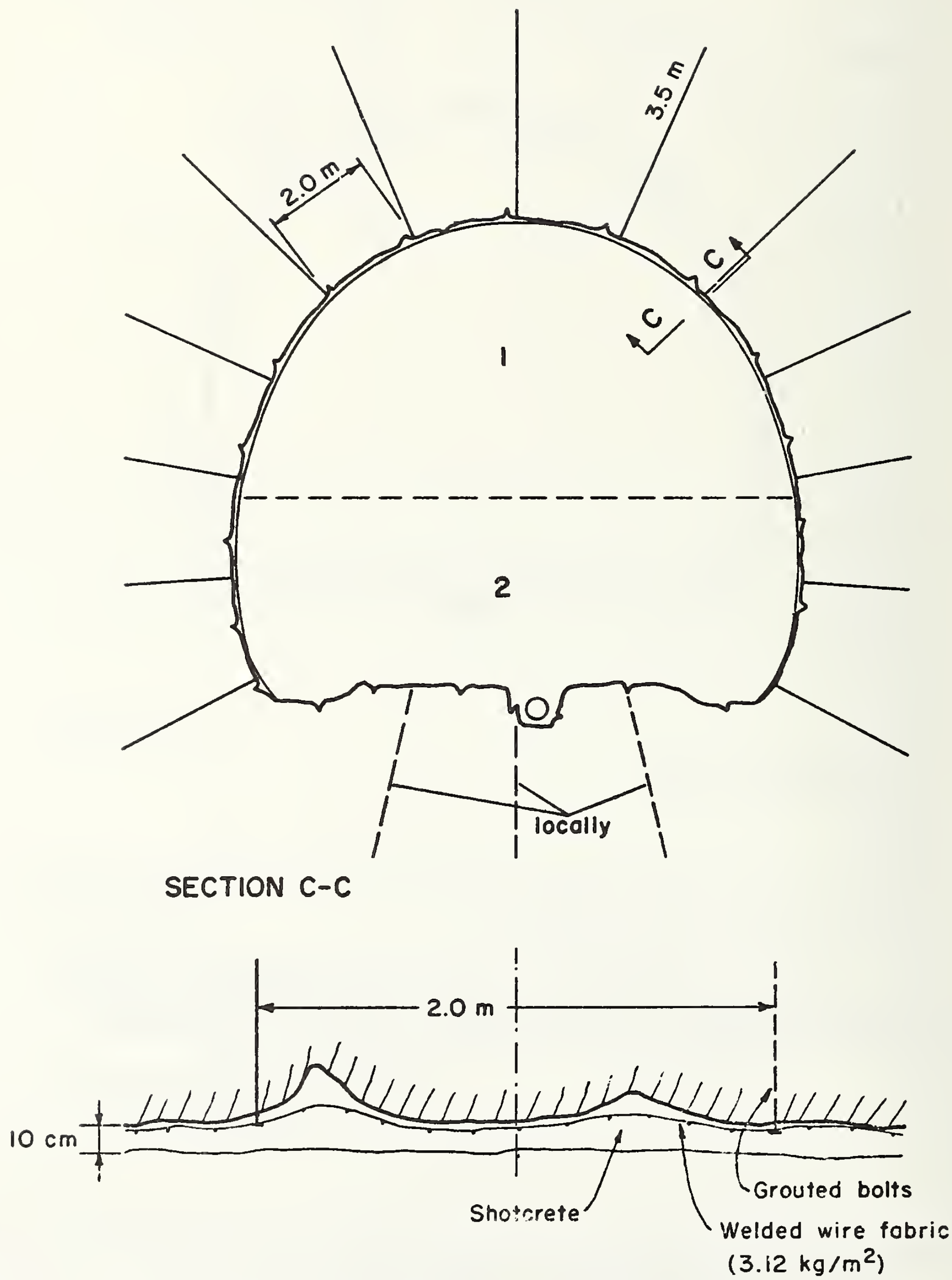


FIGURE A-3 EXCAVATION AND SUPPORT PROCEDURE
FOR GROUNDCLASS III
(AFTER JUDTMANN,1976)

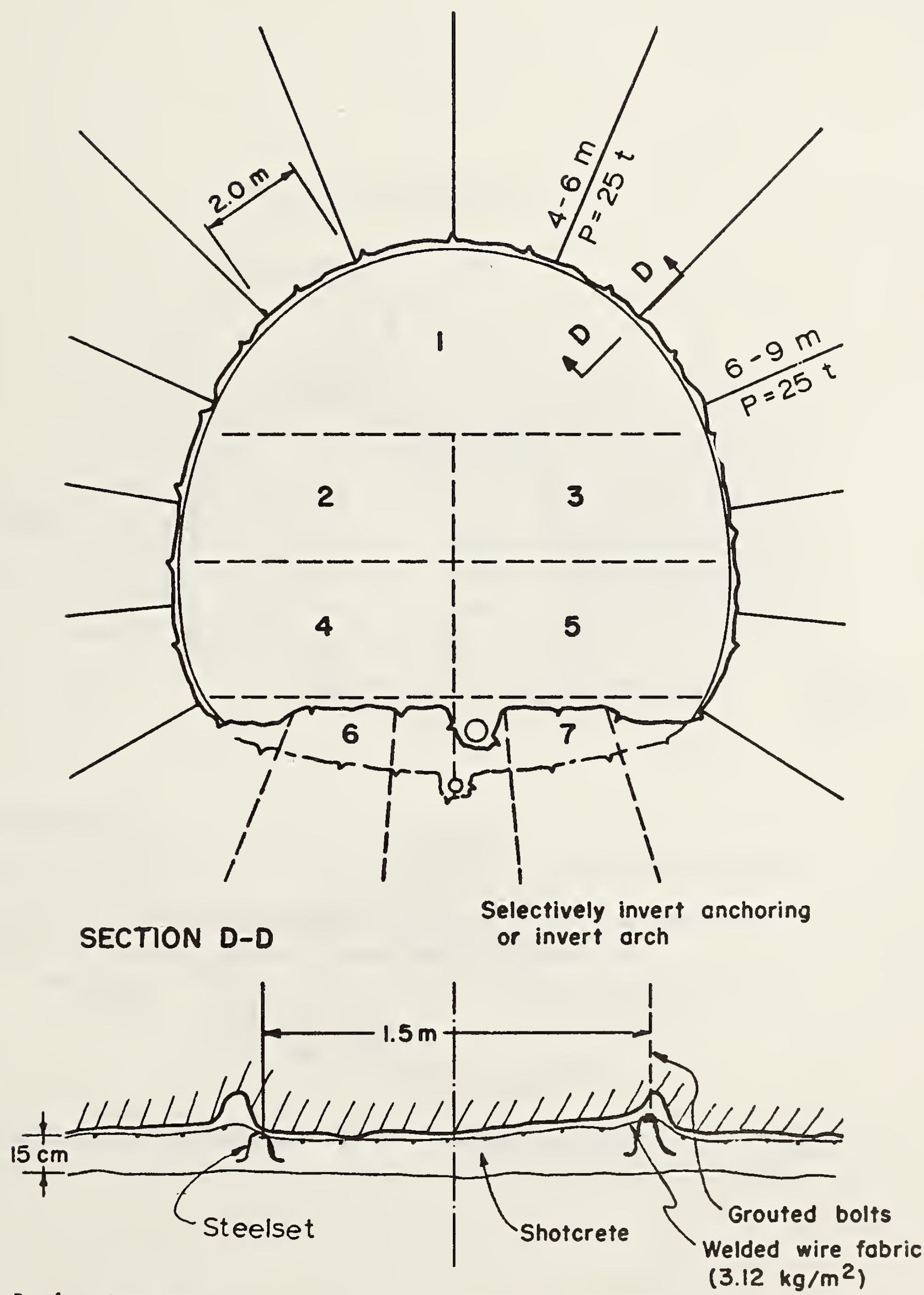


FIGURE A-4 GROUNDCLASS IV A (AFTER JUDTMANN,1976)

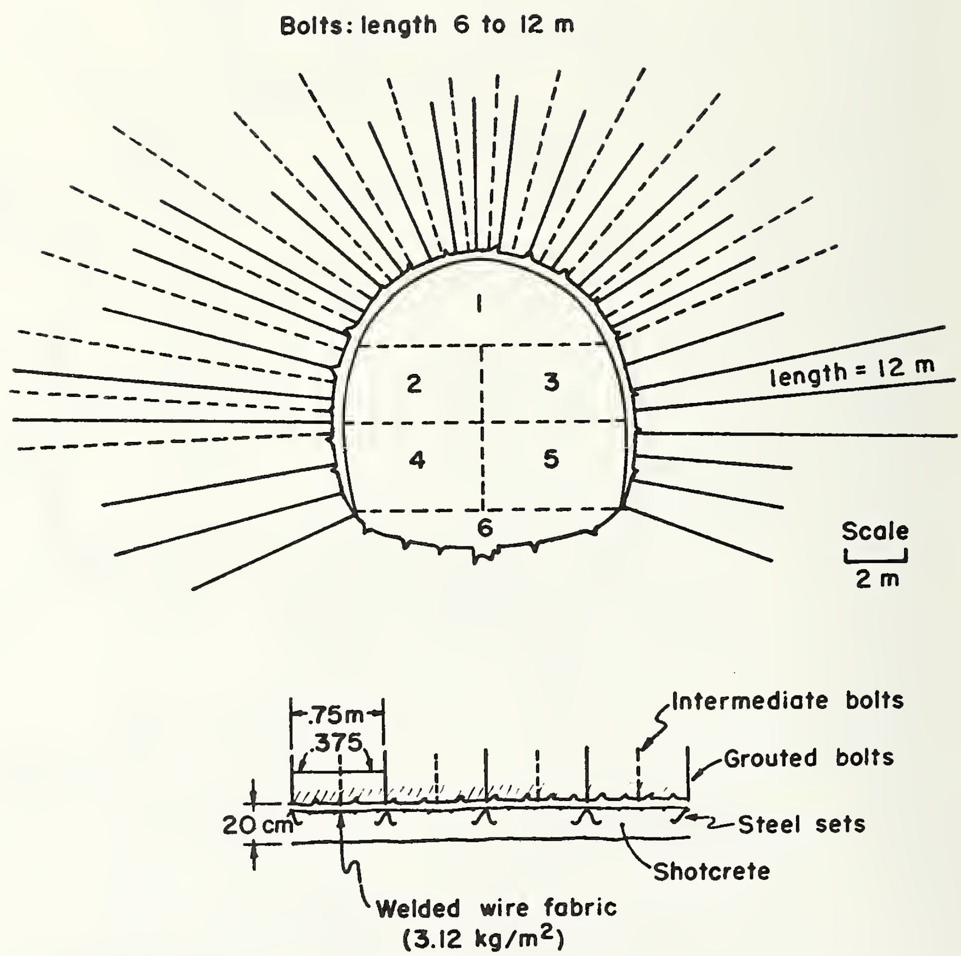


FIGURE A-5 GROUNDCLASS IV B (AFTER MAYREDER,1976)

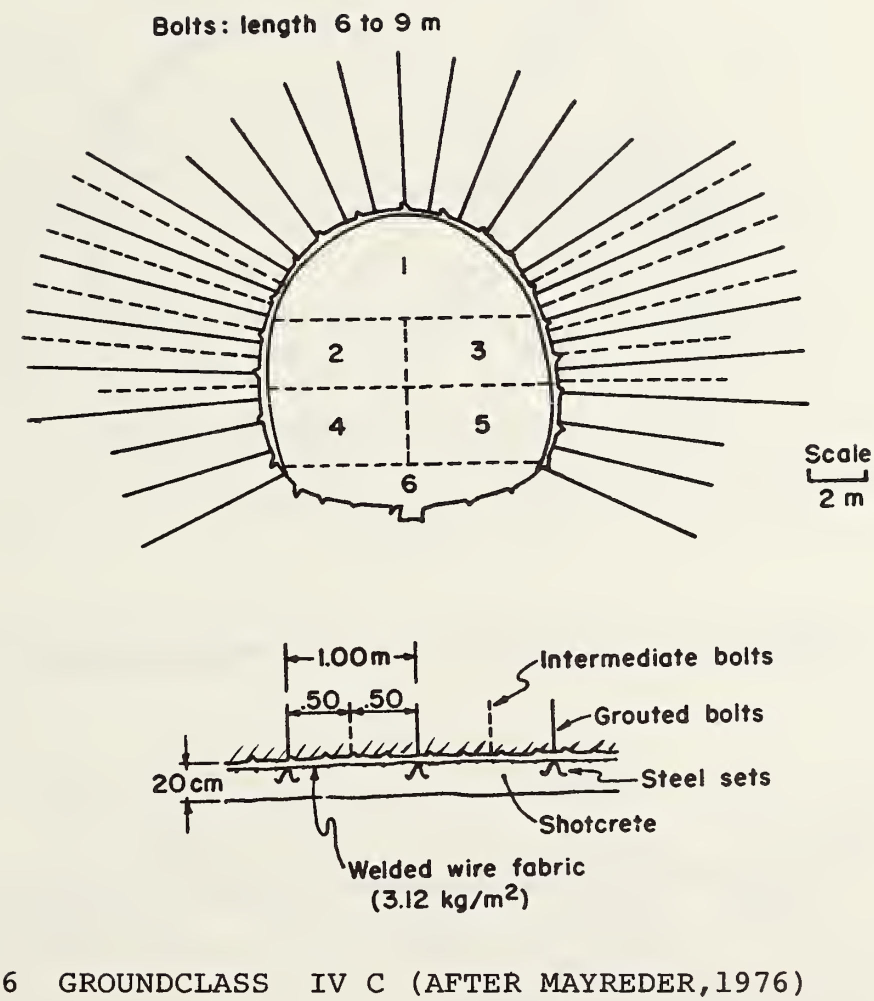


FIGURE A-6 GROUNDCLASS IV C (AFTER MAYREDER,1976)

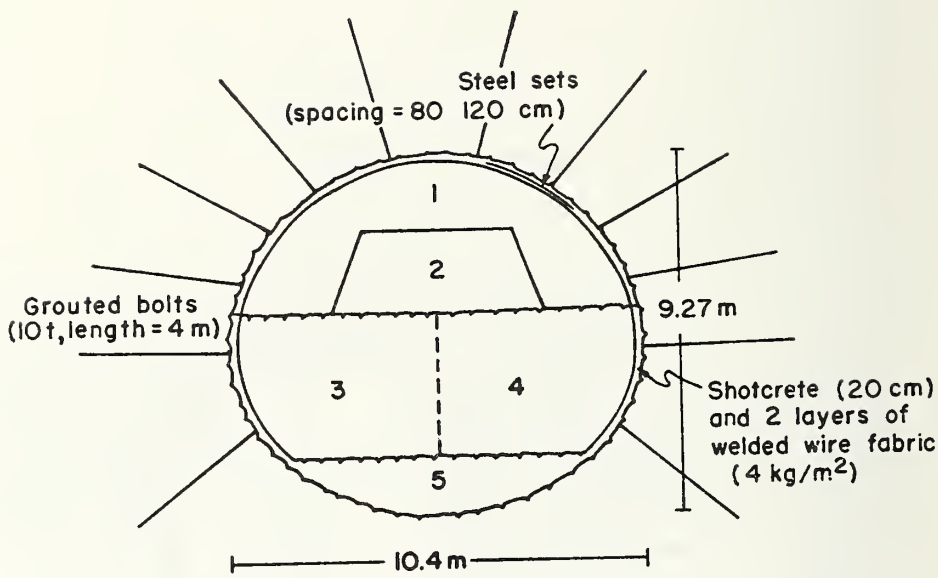


FIGURE A-7 GROUNDCLASS IV D (AFTER EGGER,1974)

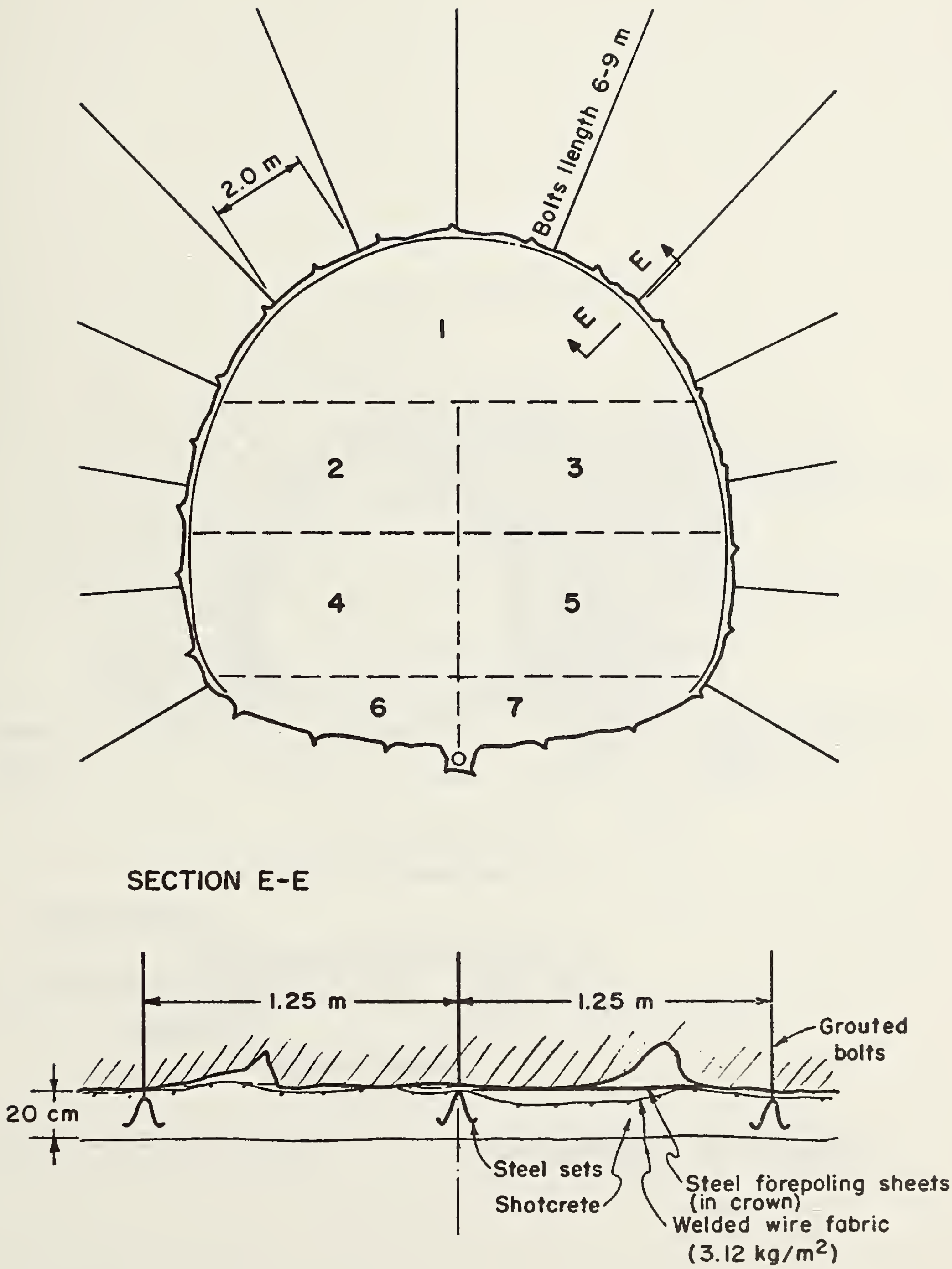


FIGURE A-8 GROUNDCLASS V A (AFTER JUDTMANN,1976)

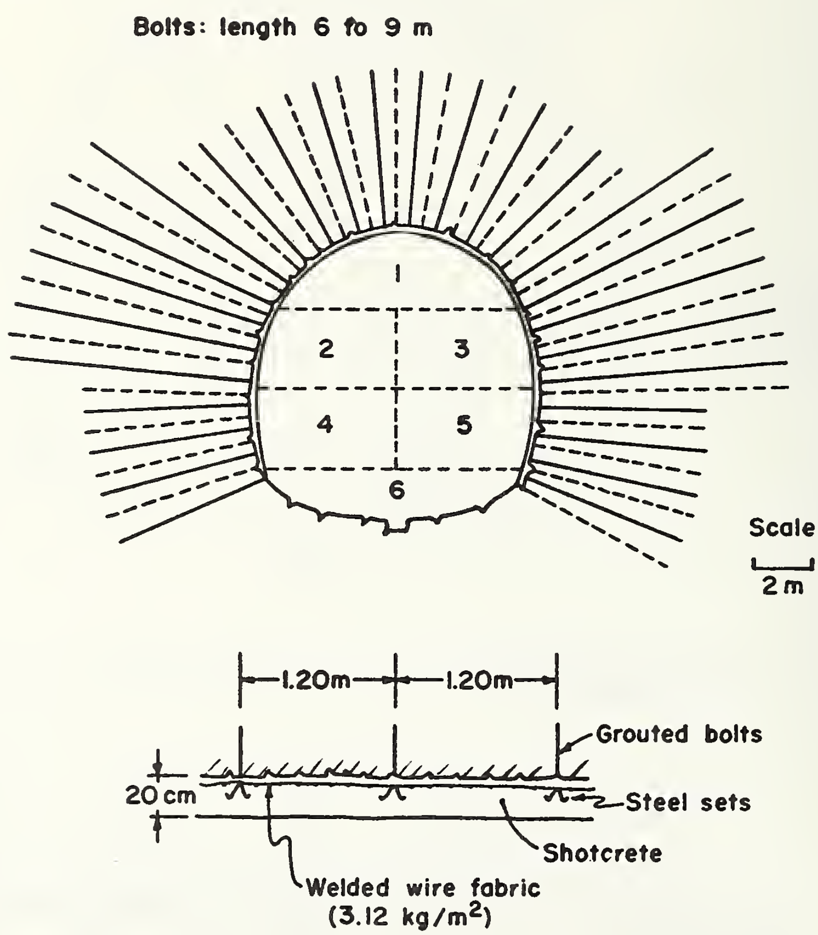


FIGURE A-9 GROUNDCLASS V B (AFTER MAYREDER,1976)

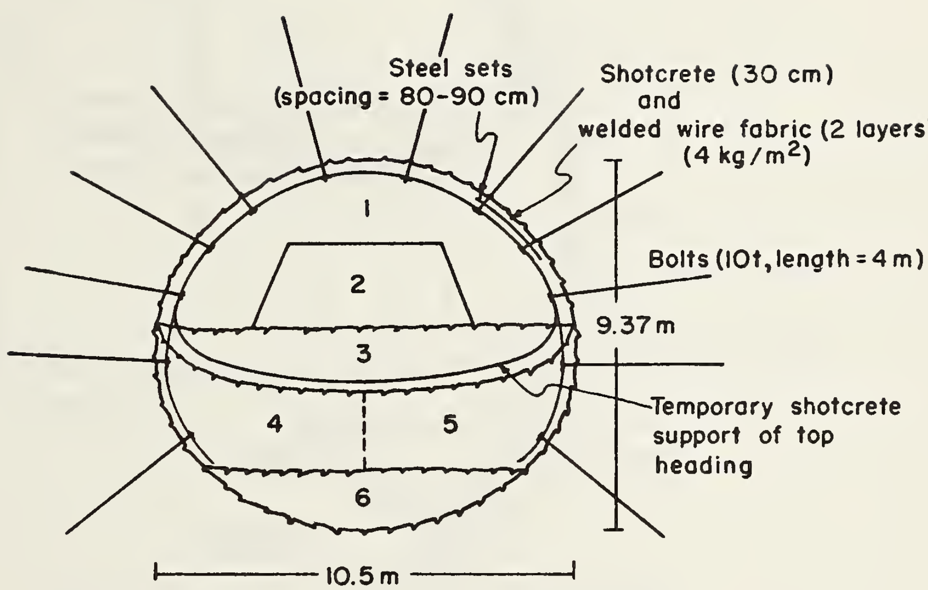


FIGURE A-10 GROUNDCLASS V C (AFTER EGGER, 1974)

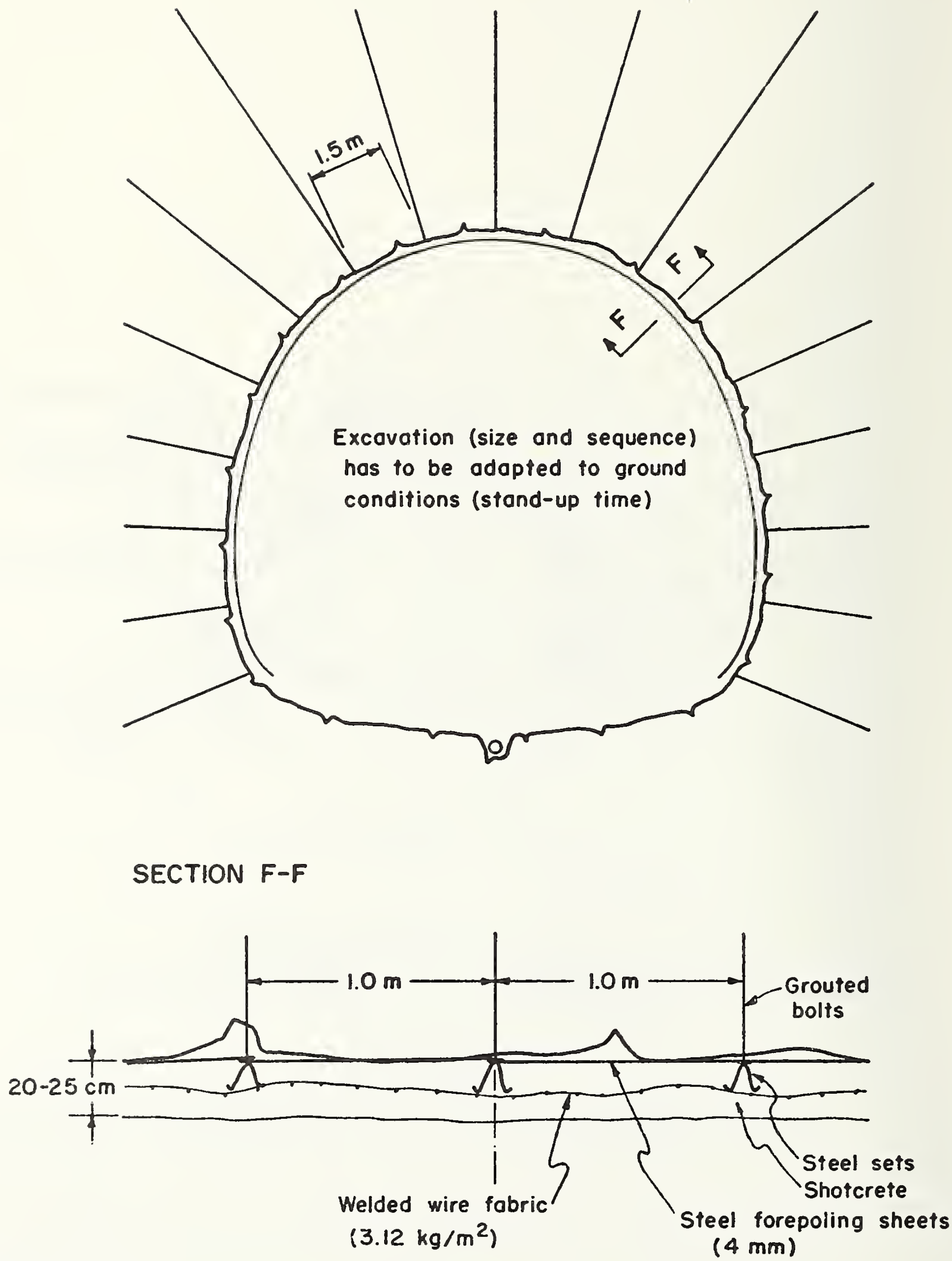


FIGURE A-11 GROUNDCLASS VI (AFTER JUDTMANN, 1976)

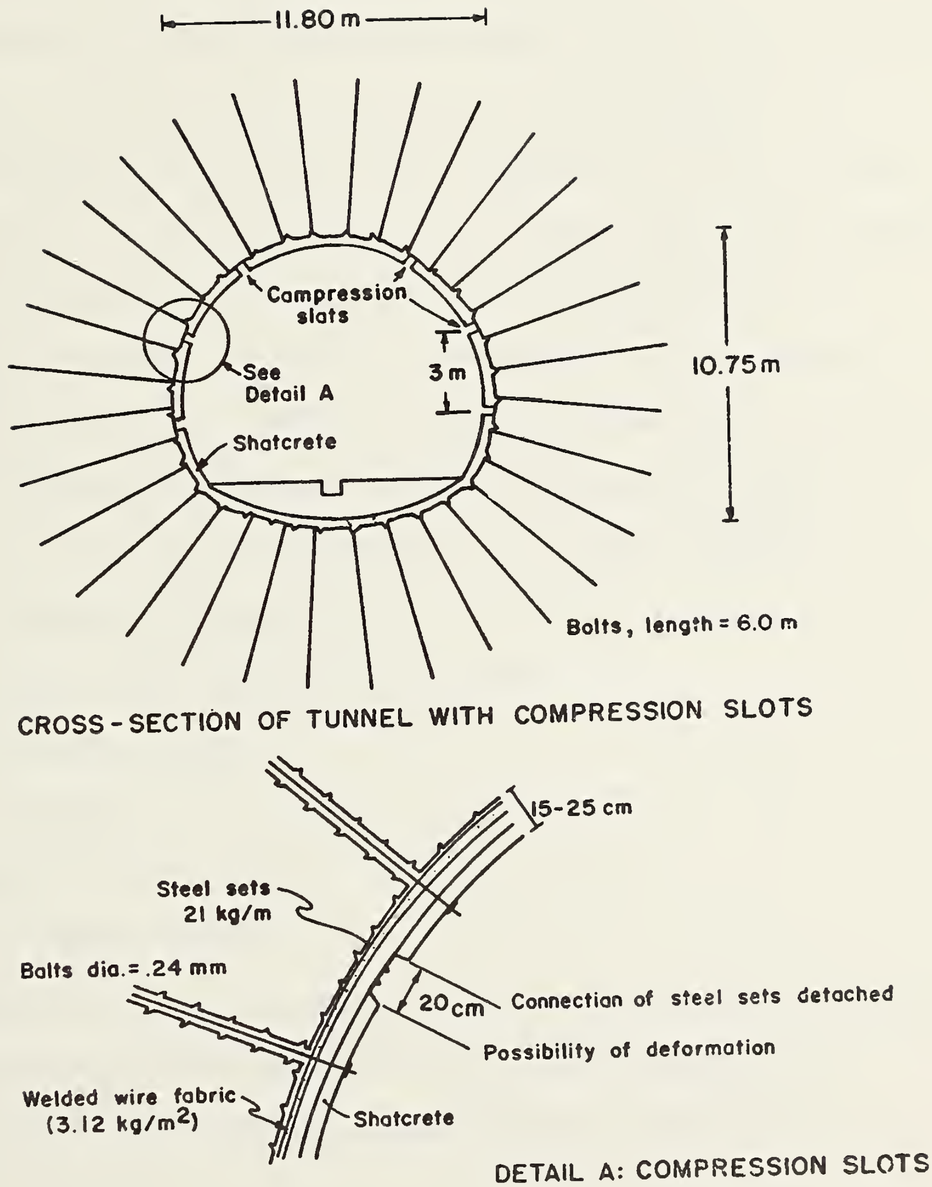


FIGURE A-12 PRINCIPLE OF COMPRESSION SLOTS
(AFTER POCHHACKER,1974)

APPENDIX B

THE PRUTZ-IMST HYDROPOWER TUNNEL

B.1 INTRODUCTION

The construction of the Prutz-Imst power tunnel is discussed in this appendix in some detail because it shows that with a proper contractual set-up, changes and adaptations can be made during construction. This can lead to a significant improvement of construction techniques that could not be achieved with a rigid contractual system. Also, the Prutz-Imst power tunnel is the case study on which Lauffer based his stand-up time span relation. The Prutz-Imst tunnel has been described from different viewpoints in several publications. This appendix is based on the following references:

Detzlhofer (1960, 1967, 1971, 1974)

Lauffer (1958, 1960)

Rotter (1960)

Schmidegg (1958, 1960)

Zaruba & Mencl (1976)

First, general aspects (Section B.2) are described, followed by geologic investigations (Section B.3), the construction procedures (Section B.4); finally, conclusions (Section B.5) are presented.

B.2 THE PRUTZ-IMST POWER SCHEME

The Prutz-Imst hydropower scheme of the Tiroler Wasserkraftwerke AG (TIWAG) uses the head of the river Inn in the province of Tyrol in the western part of Austria. A 40 km loop of the river Inn near Landeck with a head difference of 145 m is cut off by a 12.3 km long tunnel (Fig. B.1). From a hydraulic point of view, the tunnel can be subdivided into two sections, one with 5.1 m interior diameter from Runserau to Wenns, where the tributary of Pilzbach is fed into the tunnel, and from Wenns to Imst with a tunnel of 5.3 m interior diameter. The powerhouse is an underground cavern, a solution which was chosen after the initially planned "open-air" powerhouse proved to be too difficult to build due to very pervious subsoil. The cavern is located in limestone with the major discontinuities striking perpendicular to the axis. The geology of the cavern and the staged exploration procedure are described in detail by Zaruba and Mencl (1976).

A longitudinal section of the pressure tunnel with predicted and actually encountered geology is shown in Figure B.2. Note also that the groundclasses are shown for the pilot tunnel and the final tunnel, as well as the type of initial and final support.

At the time of its construction in the years 1953-1956, the pressure tunnel Prutz-Imst was the longest and largest tunnel that had been driven in Austria for some decades. Another factor that made this tunnel particularly challenging was the unfavorable ground conditions encountered.

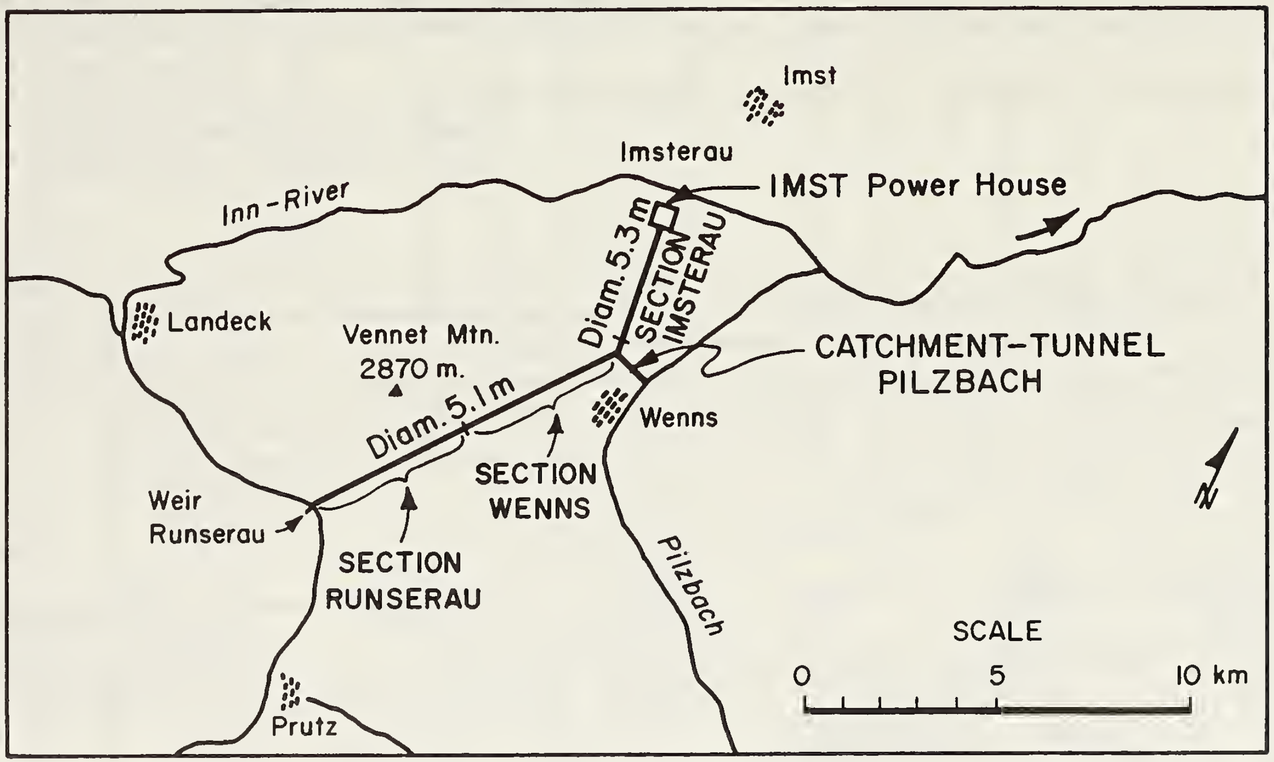
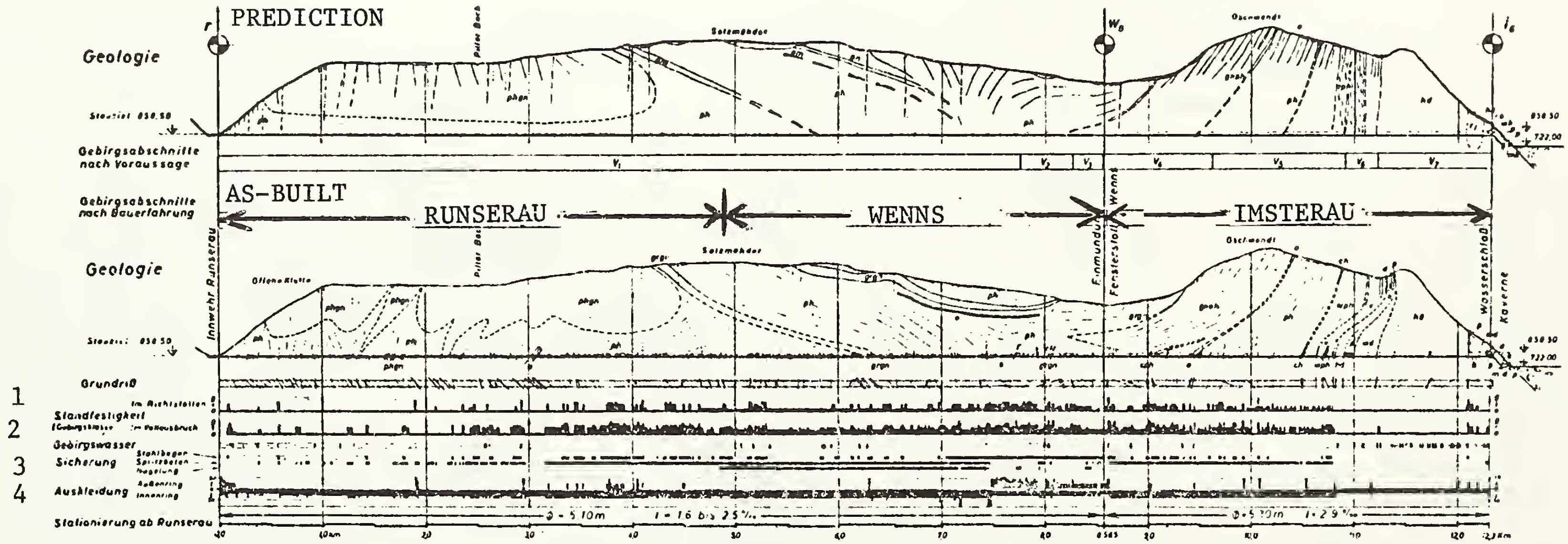


FIGURE B-1. MAP OF THE PRUTZ-IMST TUNNEL (AFTER LAUFFER, 1960)



1 GROUNDCLASSES IN PILOT TUNNEL
2 GROUNDCLASSES IN FINAL TUNNEL
3 SUPPORT (Stahlbogen= steelsets; Spritzbeton= shotcrete; Nagelung = rock bolts)
4 FINAL LINER
ph = phyllite quartzitic; phgn = phyllite gneiss ; gnph = gneissose phyllites
hd = dolomite; grgn = granite gneisses

FIGURE B-2 LONGITUDINAL SECTION PRUTZ-IMST TUNNEL (AFTER LAUFFER,1960)

B.3 GEOLOGIC INVESTIGATIONS

A thorough geologic investigation preceded bidding and construction. Aerial photographs were used, and new surface geologic maps were developed (Schmidegg, 1958, 1960). The maximum overburden is 900 m; thus, this thorough surface exploration could not provide a reliable prediction of the ground conditions. In order to reduce the uncertainty and to be able to anticipate difficult zones in the large tunnel, a pilot tunnel preceded the excavation of the final tunnel. The pilot tunnel with 6 to 10 m^2 cross-section (horseshoe: 2.5 by 2.5 m) was located in the invert (Fig. B.3) and preceded the excavation of the main tunnel by several hundred meters.

The major differences in geologic structure were that in section Runserau the phyllites were steeply dipping, in contrast to the middle section Wenns where they were dipping flatly. Consequently, greater difficulties were encountered in section Wenns. In particular, sudden rooffalls were encountered.

B.4 CONSTRUCTION PROCEDURE

The construction of the tunnel was performed in three sections. Each was let to a different contractor, resp. contractor's combine. The three sections were (Fig. B.1): the upstream section <u>Runserau</u>, 4.8 km long; the middle section <u>Wenns</u>, 3.8 km long; and the downstream section <u>Imsterau</u>, 3.7 km long. Since each section had only one point of attack and construction time was limited, the construction sequence had

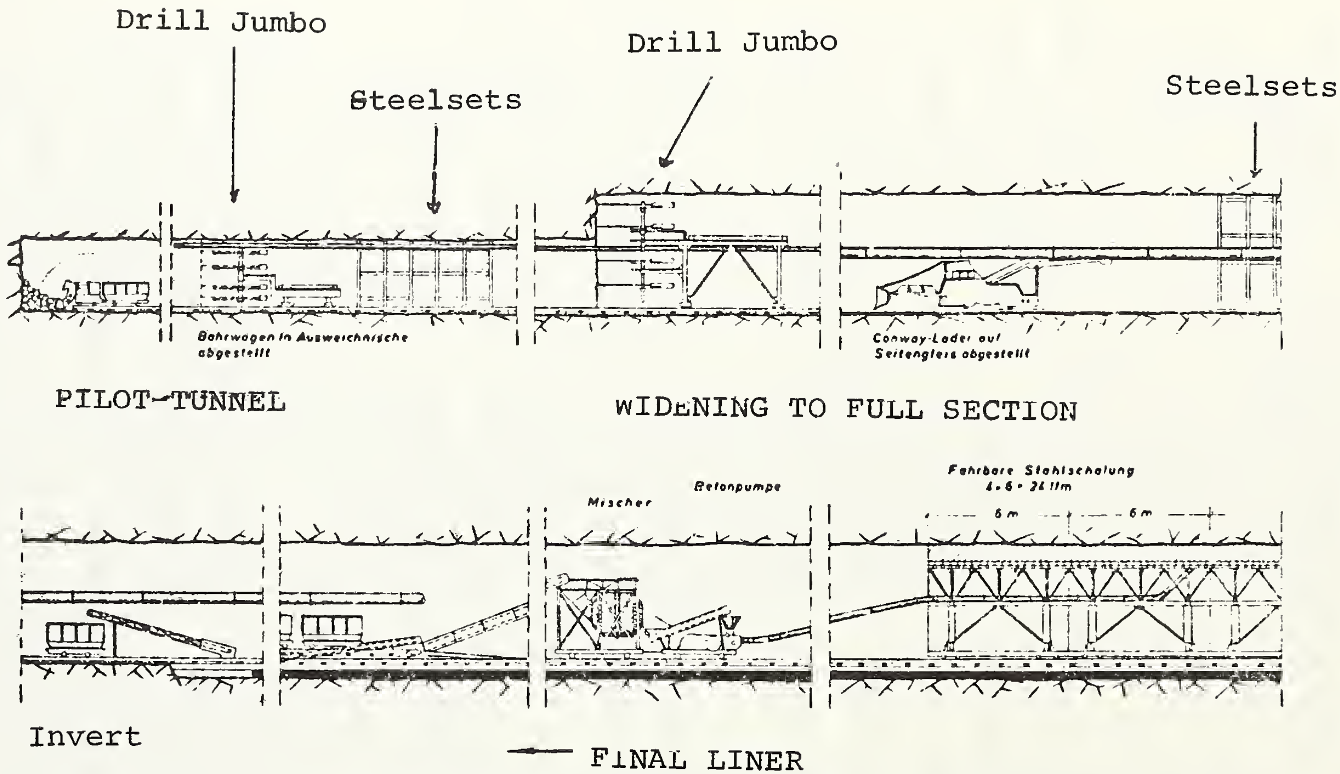


FIGURE B-3 CONSTRUCTION METHOD OF PRUTZ-IMST TUNNEL (SECTION RUNSERAU) (AFTER LAUFFER,1960) to be arranged such that parallel tasks were conducted simultaneously but separated in space: the pilot drift excavation preceded the widening to a full cross-section (that could theoretically be done at several points along the pilot tunnel); finally, the placement of the concrete liner followed.

Difficult zones caused fewer problems in the pilot tunnel and required less initial support than in the final tunnel, as indicated by the differing groundclasses in both tunnels (Figs. B.2 and B.7). The pilot tunnel was excavated with rounds of 1.5 to 2.0 m length, and the daily advance rate was 9 to 12 meters, preceding the final tunnel by several hundred meters. In all three sections the pilot tunnel was supported by steel sets.

Of particular interest are the support methods for the main tunnel that were used in the three sections. The support methods proposed in each section and initially used are presented in Table B.1. During the actual construction, the construction procedures changed in each section. The finally used procedures are presented in Table B.2. The changes in support-excavation procedure were due to:

(1) the adverse ground conditions that caused considerable difficulties;

(2) the development of new support procedures, in particular, shotcrete and rock bolts.

The development of the support procedures, which will be discussed below, had a significant impact on tunnel construction

TABLE B-1. INITIAL CONSTRUCTION PROCEDURE FOR PRUTZ-IMST TUNNEL (AFTER DETZLHOFER, 1960)

Groundclass		Rate of Advance (m/day)	Support in Section Runserau	Rate of Advance (m/day)	Support in Section Wenns	Rate of Advance (m/day)	Support in Section Imsterau
Free standing	a	7.5	None	7.0	None	8.0	None
Breaking	b	7.0	Steelsets with timber lagging removed before placement of concrete	7.0	Steelsets with timber lagging removed before placement of concrete	7.0	Steelsets with timber lagging (temporary support removed before placement of concrete)
Lightly afterbreaking	c	6.5	Steelsets with timber lagging. Removal of temporary support after some weeks and placement of shotcrete support.	6.0	Steelsets with timber lagging. Removal of temporary support after a few weeks. Exterior concrete arch in top heading with timber formwork over steelsets with footings at springline.	6.0	Steelsets with timber lagging. Removal of temporary support after several weeks, and replacement with shotcrete and rockbolts.
Medium afterbreaking	d	5.0	Steelsets and shotcrete left in final liner.	6.0		5.0	
Strongly afterbreaking	e	3.0	Timber Support. Exterior rings in concrete placed with timber formwork.	5.0	Excavation of the top heading with Kunz-Support System. Concrete or masonry arch in crown. Underpinning of arch from pilot tunnel: sections of 2 m length. Completion of exterior arches after excavation of bench (use of timber formwork).	3.5	Full exterior concrete rings on Kunz-Support System over the entire cross-section. No underpinning of heading.
Squeezing	f	2.0		4.0		2.5	
Squeezing with face support	g	1.5		2.0		1.5	

TABLE B-2. FINAL CONSTRUCTION PROCEDURE FOR PRUTZ-IMST TUNNEL AFTER DETZLHOFER, 1960)

Groundclass		Rate of Advance (m/day)	Support in Section Runserau	Rate of Advance (m/day)	Support in Section Wenns	Rate of Advance (m/day)	Support in Section Imsterau
Free standing	a	7.5	None	7.0	None	8.0	None
Breaking	b	7.0	Shotcrete after excavation.	7.0	Rockbolts (double wedge type) after each round, placed from platform car. Wirefabric as head protection. Shotcrete following in a short distance from platform car.	7.0	Shotcrete after excavation.
Lightly afterbreaking	c	6.5	Shotcrete after each round with wirefabric.	6.0		6.0	Shotcrete after each round with wirefabric and occasional bolts.
Medium afterbreaking	d	5.0		6.0		5.0	
Strongly afterbreaking	e	3.0		5.0	Steelsets with timber lagging removed at some distance from face, and replacement by shotcrete and rockbolts.	3.5	
Squeezing	f	2.0	Steelsets with wirefabric and covered with shotcrete, left in place.	4.0	Concrete board lagging on steelsets, covered with shotcrete (5-7 cm). Base of steelsets placed in concrete footing.	2.5	Steelsets and wirefabric covered with shotcrete and left in place.
Squeezing with face support	g	1.5		2.0		1.5	Exterior Ring on Kunz-Support System (was only necessary in waterbearing zone.

and can be considered to be a breakthrough in tunneling technology.

The support-excavation procedure for section Runserau will be described first, followed by a description of the procedures in section Wenns. The procedures used in section Imsterau are similar to those of section Runserau and are thus not discussed in detail.

Section Runserau

The excavation-support procedure is shown in Fig. B.3. In the pilot tunnel the support, if required, consisted of steelsets TH 21 (14 lbs/ft) and TH 27 (18 lbs/ft) with timber lagging. The support of the pilot tunnel was removed before the widening of the tunnel. In some sections of the pilot tunnel, shotcrete was used, which, however, was lost when the tunnel was widened. The widening to full cross-section was by "full face" excavation, i.e. the entire remaining face was excavated in one step. The round length was 1.8 to 2.0 m.

Initially, the final tunnel was supported like the pilot tunnel with steelsets and timber lagging. This temporary support was removed prior to placing the formwork and casting of the concrete liner. The ground was supported during a period of 2 to 4 months by this temporary support. During this time, the ground loosened around the circumference of the tunnel. Before the concrete could be placed, the loosened rock had to be removed; the volume of this loosened rock amounted to

3 to 4 m^3 per meter of tunnel, or 10% of the cross section. The loose rock and the problems associated with it led to changes in support procedures. However, this was not the only reason: Rotter (1960) reports that in the initial phases of section Runserau, three fatal accidents occurred in sections that were widened and were considered stable. (Rotter does not give any further details.)

The new support procedure was as follows:

(a) The circumference was covered with 7 cm shotcrete;

(b) Steelsets blocked with timber were placed inside the shotcrete;

(c) Prior to placement of the final concrete liner, the steelsets were removed;

(d) Formwork was placed and the final concrete liner was poured.

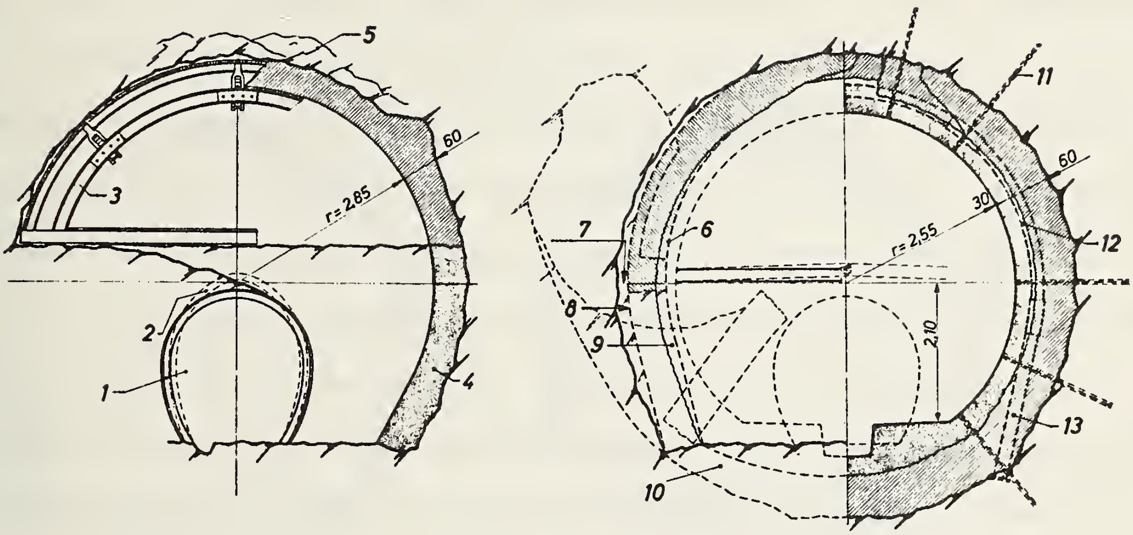
With this procedure, the steelsets could be removed without the danger of roof collapses before placement of the final liner. In some sections the temporary steelsets were actually omitted altogether. If necessary, i.e. in case the stand-up time was not sufficient, the next round was excavated in sections and shotcrete placed over the opened sections (no further details given).

A further development occurred in sections with the most unfavorable ground conditions where steelsets were imbedded in shotcrete.

Section Wenns

The original construction procedures for the middle section Wenns were somewhat different from those of section Runserau (Table B.1). For the support in the most difficult ground conditions (classes e, f, g), a Kunz Support System (Kunz'sche Rüstung) was proposed (Fib. B.4). With the Kunz-System, the top heading is excavated and supported first. The timber lagging rests with small posts on steelsets which are the framework for the concrete liner. The concrete of the top heading is poured and the timber lagging is removed as the concrete rises. The procedure is somewhat similar to the "Flying Arch Method". Note that the pilot tunnel remained in the lower part of the final tunnel while the excavation of the top heading proceeded. As a consequence, the steelsets in the pilot tunnel were squeezed at the springlines and lifted at the crown (dashed lines on Fig. B.4). After the heading was concreted, it was underpinned with pillars of two meter length (Fig. B.4). These pillars were constructed from the pilot tunnel. In some instances, however, the springlines of the tunnel moved in, which led the roof arch to settle and collapse at the springlines. For the better groundclasses (b, c, d) steelsets with timber lagging were originally planned as temporary support. Other problems in section Wenns were due to the horizontally foliated phyllites; this led to the occurrence of sudden unexpected rooffalls.

The substantially revised construction procedure was as follows: roof bolts provided the most satisfactory solution for



1 pilot tunnel
2 steelsets in pilot tunnel squeezed together
3 "Kunz" support system (Kunz'sche Ruestung)
4 underpinning of crown arch in 2 m wide slots from pilot tunnel
5 loosening
6 deformation at springlines
7 settlement
8 squeeze force at springlines
9 trimming
10 invert arch,in case of invert heave
11 grouted bolts
12 steelsets
13 steelsets in slots (underpinning)

FIGURE B-4 INITIAL CONSTRUCTION PROCEDURE IN PRUTZ-IMST TUNNEL FOR CLASSES f AND g (AFTER DETZLHOFER,1968) classes b and c; they were placed from a platform car that was advanced to the face after each round (Fig. B.6). Bolts (length = 1.8 m, spacing 1.2 to 1.5 m, bolt diameter = 24 mm, prestress = 3 to 5 tons) with wire fabric were placed in the crown down to the springline. Shotcrete was placed at some distance (no exact value given) from the platform car.

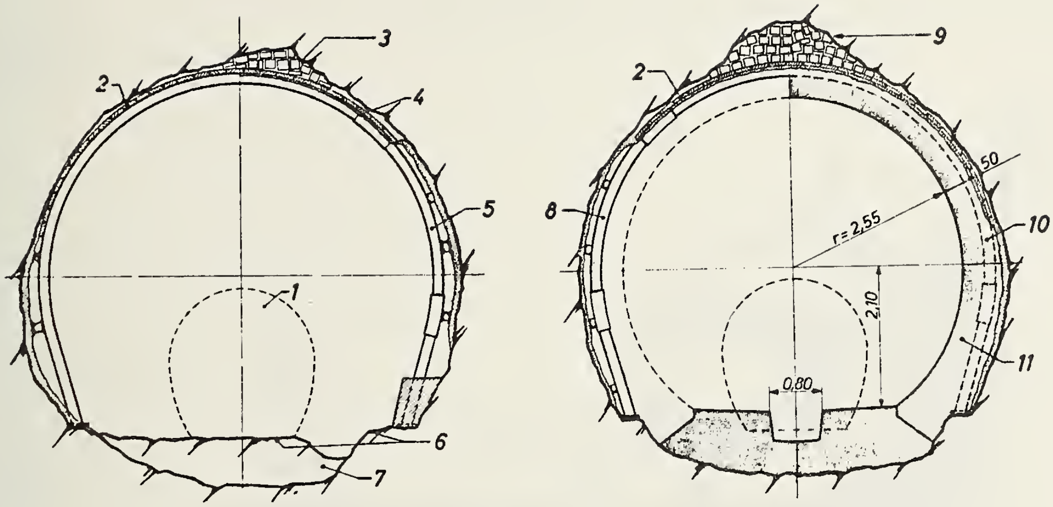
For classes d and e (heavily afterbreaking to squeezing ground) steelsets with timber lagging were used, which were replaced by shotcrete and rockbolts at a short distance from the face. This was necessary because the placement of shotcrete near the face would have slowed down the advance rate. In classes f and g, concrete board lagging was used on steelsets and the voids (overbreak) were backfilled with concrete blocks and grouted. Finally, a layer of shotcrete was placed over the concrete lagging. The footings of the steelsets were encased in concrete or shotcrete (Fig. B.5).

More details on the construction procedure can be found in Detzlhofer (1960).

B.5 CONCLUSIONS

The experience from the Prutz-Imst Tunnel demonstrated (1) improved construction procedures for tunnels; (2) a classification based on stand-up time; (3) that with proper contract procedures it is possible to modify excavation support methods to better suit the conditions encountered.

The improved support procedures consisted of shotcrete and



1 pilot tunnel
2 concrete board lagging
3 grouting of repacked voids
4 shotcrete
5 steelsets TH21 (14 lb/ft)
6 invert of pilot tunnel
7 invert of final tunnel
8 steelsets
9 voids backpacked with concrete blocks
10 steelsets left in final liner
11 final concrete liner

FIGURE B-5 CONSTRUCTION METHOD (FINAL) FOR CLASSES f AND g IN SECTION WENNS (AFTER DETZLHOFER 1968)

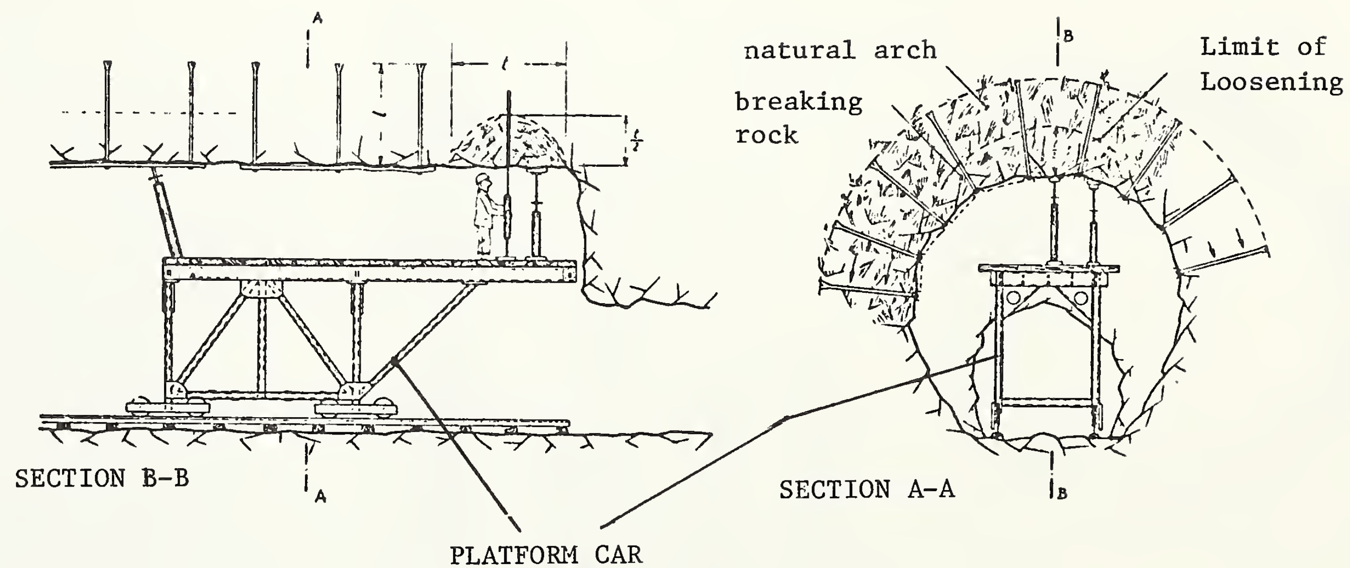


FIGURE B-6 ROCK BOLTING OF THE CROWN FROM PLATFORMCAR (FROM DETZLHOFER,1960) rockbolts that were introduced successfully.

The classification procedure proposed by Lauffer shows the importance of:

- stand-up time
- unsupported span
- shape of opening
- excavation method
- support type
- orientation of geologic structure

(See Section 3.5.1 for detailed figures.)

In addition, a size effect was found (Fig. B.7).

In summary, even if the Prutz-Imst Power Tunnel was built a quarter century ago, it still serves as a prime example of tunnel construction. The construction procedure was planned such that uncertainties could be reduced (pilot tunnel). The contract was formulated so that changes in the construction procedure could be made during actual construction. These changes led to innovative methods that can only be developed in the field.

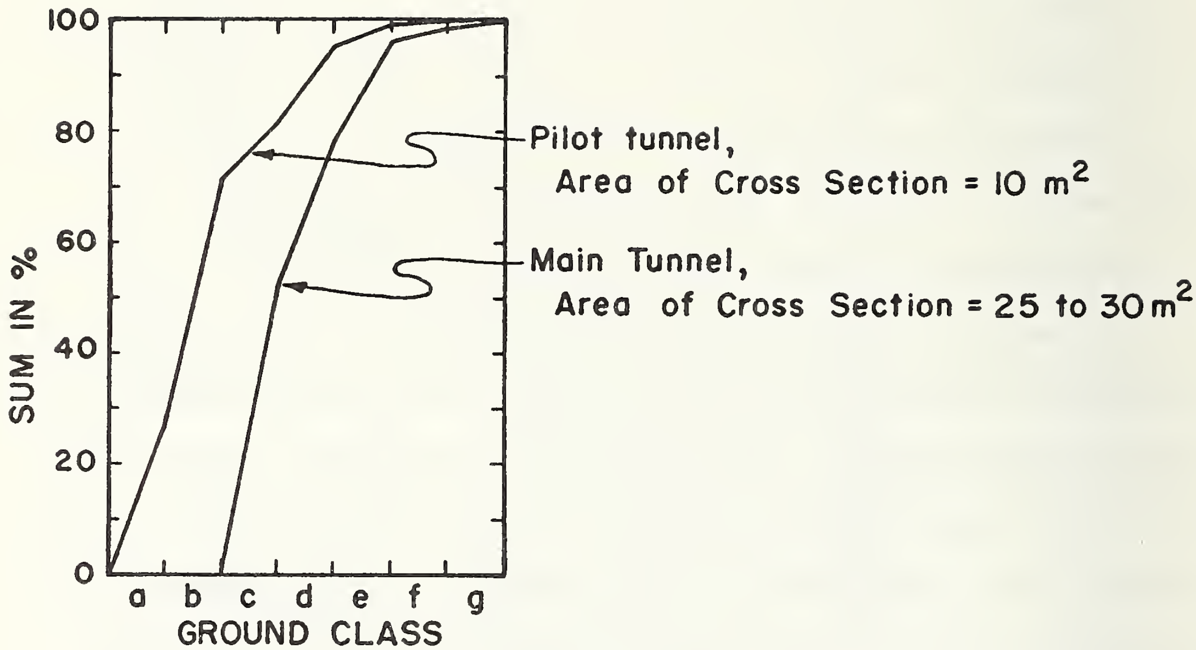


FIGURE B-7 SIZE EFFECT ON GROUNDCLASSES AS OBSERVED IN SECTION WENNS OF THE PRUTZ-IMST TUNNEL (AFTER DETZLHOFER,1974)

APPENDIX C

RELATIONS BETWEEN JOINT SPACING RESP.
JOINT FREQUENCY AND RQD

Rock quality designation, RQD, is a widely used index property for rock masses. RQD, which has been proposed by Deere (1963, 1967) is defined as the ratio of the sum of recovered core equal or longer than 10 cm or 4 inches. Deere (1967) defined RQD as follows:

"The RQD is a modified recovery percentage in which all pieces of sound core over 4 inches long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted . . . The RQD . . . evaluates fractures in the core caused by the drilling process, as well as natural fractures previously existing in the rock mass. For example, when the core hole penetrates a fault zone or a joint, additional breaks may occur which, although not natural fractures, are caused by the natural planes of weakness existing in the rock mass. These breaks should be included in estimating rock quality."

RQD is lower for higher fracture frequencies, regardless of drilling procedure, and thus has a clear advantage over unmodified core recovery. Deere's procedure includes breaks caused by the drilling process as fractures, since they are indicative of rock weakness. Such breaks generally occur, according to Deere along preexisting planes of weakness.

Although RQD is strictly defined as above, various methods have proposed to determine it from other than boreholes (outcrops, adits) and particularly to derive RQD from discountinuity (fracture, joint) spacing or other descriptions of the jointed rock. Deere et.al. (1969) published some of these correlations, they are shown in Fig. C-1. Deere used experimental data to relate RQD to fracture frequency. The resulting relationship is shown in Fig. C-2.

Note that in Fig. C-1 Deere shows a correlation for RQD based on joint spacing and fracture spacing. The same spacing results in a lower RQD for joints than for fractures. The same RQD is obtained in case joint spacing is twice fracture spacing. This difference may be attributed to the effect of persistence, fractures may be less persistent thus leading to larger core pieces and thus a higher RQD.

Another empirical relation for RQD has been developed by Palmstrom for clay free rocks and has been reported by Barton et. al. (1974).

$$RQD = 115 - 3.3\ J_v \qquad \text{(C-1)}$$

where J_v = total number of joints per cubic meter

$$RDQ = 100 \text{ for } J_v < 4.5$$

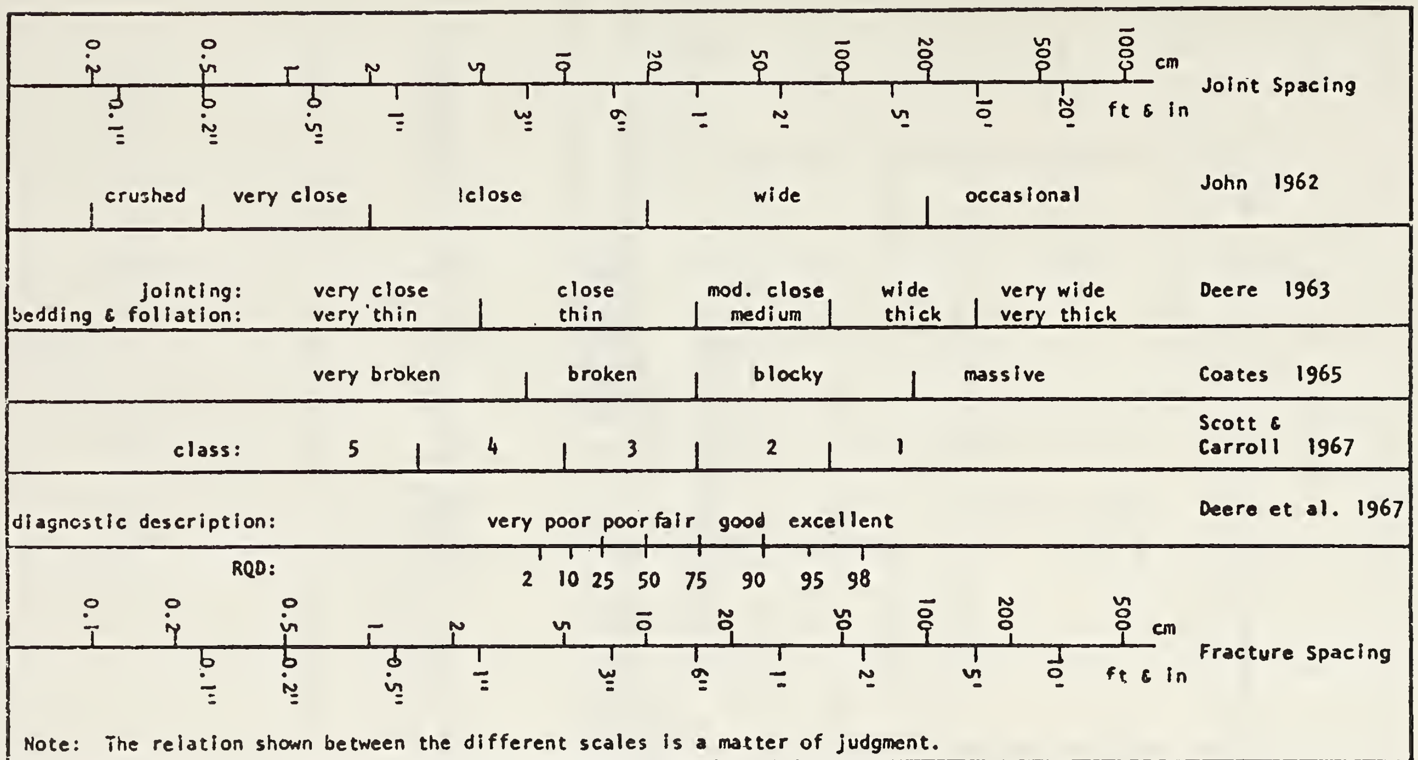


FIGURE C-1 CORRELATIONS OF ROCK QUALITY DESIGNATION TO OTHER TYPES OF JOINTING DESCRIPTION (FROM DEERE ET AL.,1969)

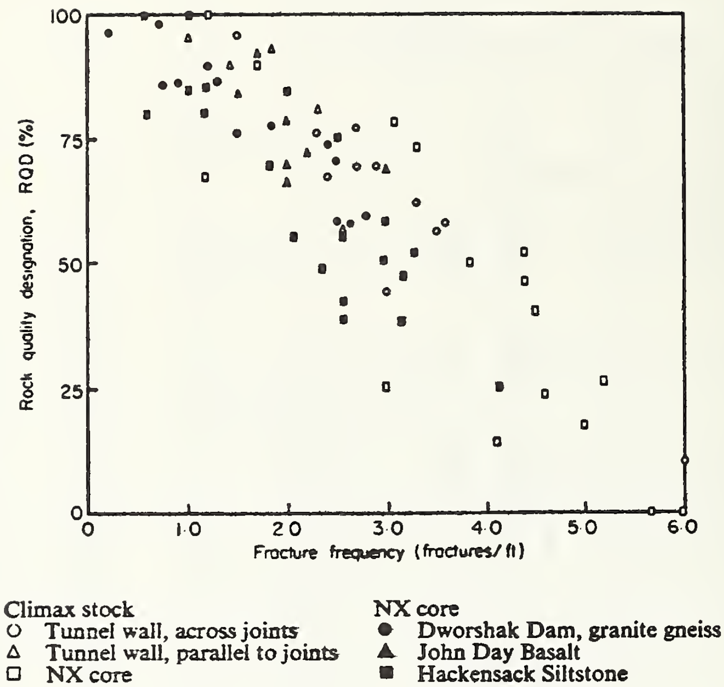


FIGURE C-2 EMPIRICAL RELATION OF RQD TO FRACTURE SPACING (FROM DEERE,1967)

Priest and Hudson (1976) developed an expression for RQD based on statistics for discontinuities perpendicular to a borehole, (Fig. C-3), their work has been extended by Dershowitz (1979) and will be further expanded in this appendix. Priest and Hudson assume that the discontinuity[1)] spacing is exponentially distributed.

$$f(x) = \lambda e^{-\lambda x} \qquad \text{(C-2)}$$

λ = average discontinuity frequency, number of discontinuities per unit length.

$m_{av} = \frac{1}{\lambda}$ = average discontinuity spacing.

By definition the RQD is the percentage of the total core length that exceeds a standard length t (t = 10 cm or 4 inches). The expected value of the RQD is

$$E(RQD) = 100 \int_t^L \lambda L f(x) dx/L \qquad \text{(C-3)}$$

By substituting the exponential distribution (Eq. C-2) in Eq. C-3

$$E(RQD) = 100 \lambda^2 \int_t^L x e^{-\lambda x} dx \qquad \text{(C-4)}$$

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1) In order to avoid confusion with statistical terms, the term discontinuity will be used for the remainder of this appendix.

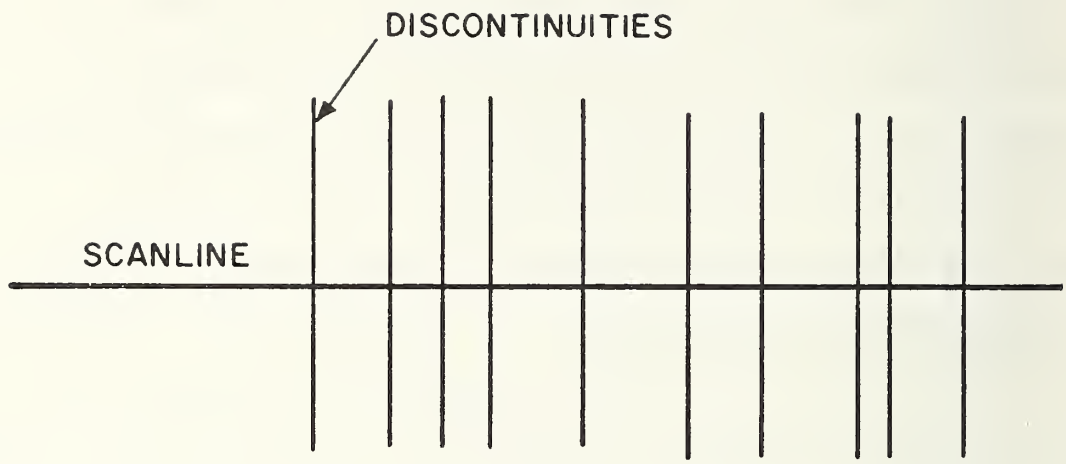


FIGURE C-3 DISCONTINUITIES PERPENDICULAR TO BORING

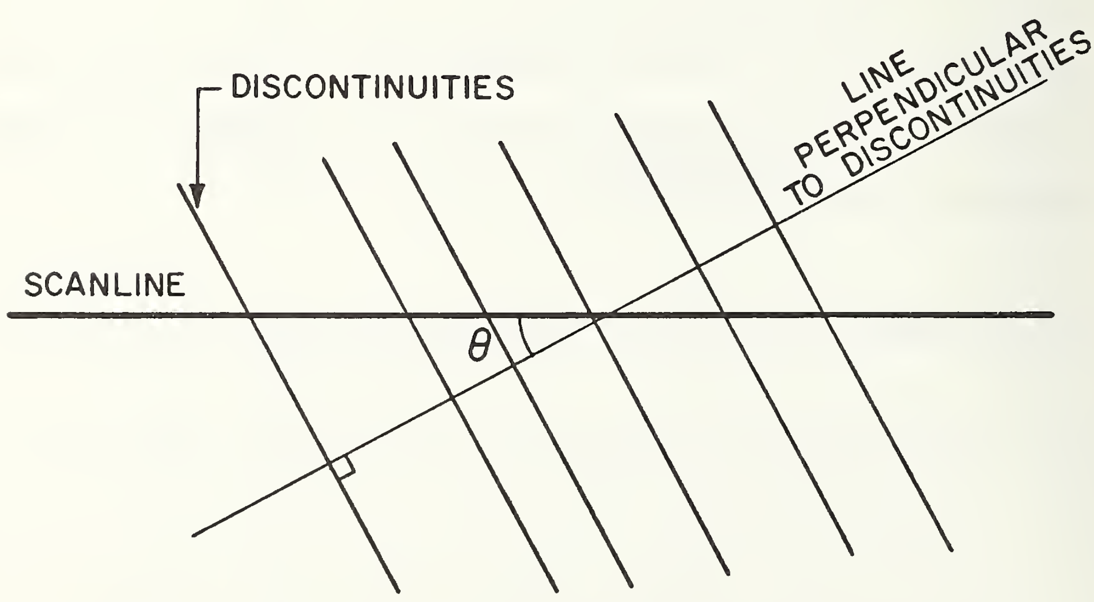


FIGURE C-4 DISCONTINUITIES INCLINED TO BORING

For long scanlines, terms with $e^{-\lambda L}$ can be ignored and thus

$$E[RQD] = 100\ e^{-\lambda t}\ (\lambda t + 1) \tag{C-5}$$

For a discontinuity frequency λ expressed as discontinuities per meter and the usual cut-off length of t=10 cm = 0.1 m

$$E[RQD] = 100\ e^{-0.1\lambda}\ (0.1\lambda + 1) \tag{C-6}$$

Dershowitz (1979) expanded the relation for a persistent joint set which is at an angle to the scanline (Fig. C-4). The average spacing perpendicular to the discontinuities is m_n, thus the frequency of discontinuities perpendicular to their planes is $\lambda_n = \frac{1}{m_n}$. The spacing along the scanline is

$m_s = \frac{m_n}{\cos\theta}$ thus the average frequency along the scanline (boring) is

$$\lambda_s = \lambda_n \cdot \cos\theta \tag{C-7}$$

by substituting λ_s into Eq. C-6

$$E[RQD] = 100 \cdot e^{-0.1\lambda \cdot \cos\theta}(0.1\lambda \cdot \cos\theta + 1) \quad \text{(C-8)}$$

Results of Eq. C-8 are shown in Fig. C-5. Note that the relation in Eq. C-7 holds only for discontinuities that are reasonably parallel, i.e., their orientation relative to the scanline is constant. The effect of nonparallel discontinuities has been studied by Dershowitz (1979), some of his results are shown in Fig. C-6. Dershowitz (1979) considered a Fisher distribution for the orientation (a spherical distribution). The value K is a measure for the scatter of the orientations, similar to the inverse of the coefficient of variation for distribution of a single variable.

For near parallel joints (high K) a strong dependence of RQD on angle between scanline and mean orientation can be seen. For more disperse orientation (low K) the expected value of RQD is less dependent on the angle between scanline and mean orientation of the discontinuities.

RQD is not only affected by the relative orientation but very strongly by the persistence of the discontinuities. A set of parallel discontinuities with equal persistence for each discontinuity is assumed perpendicular to the scanline (Fig. C-7). The persistence P is the ratio of the open area of a discontinuity to the total area of intact rock plus discontinuity (Eq. C-9)

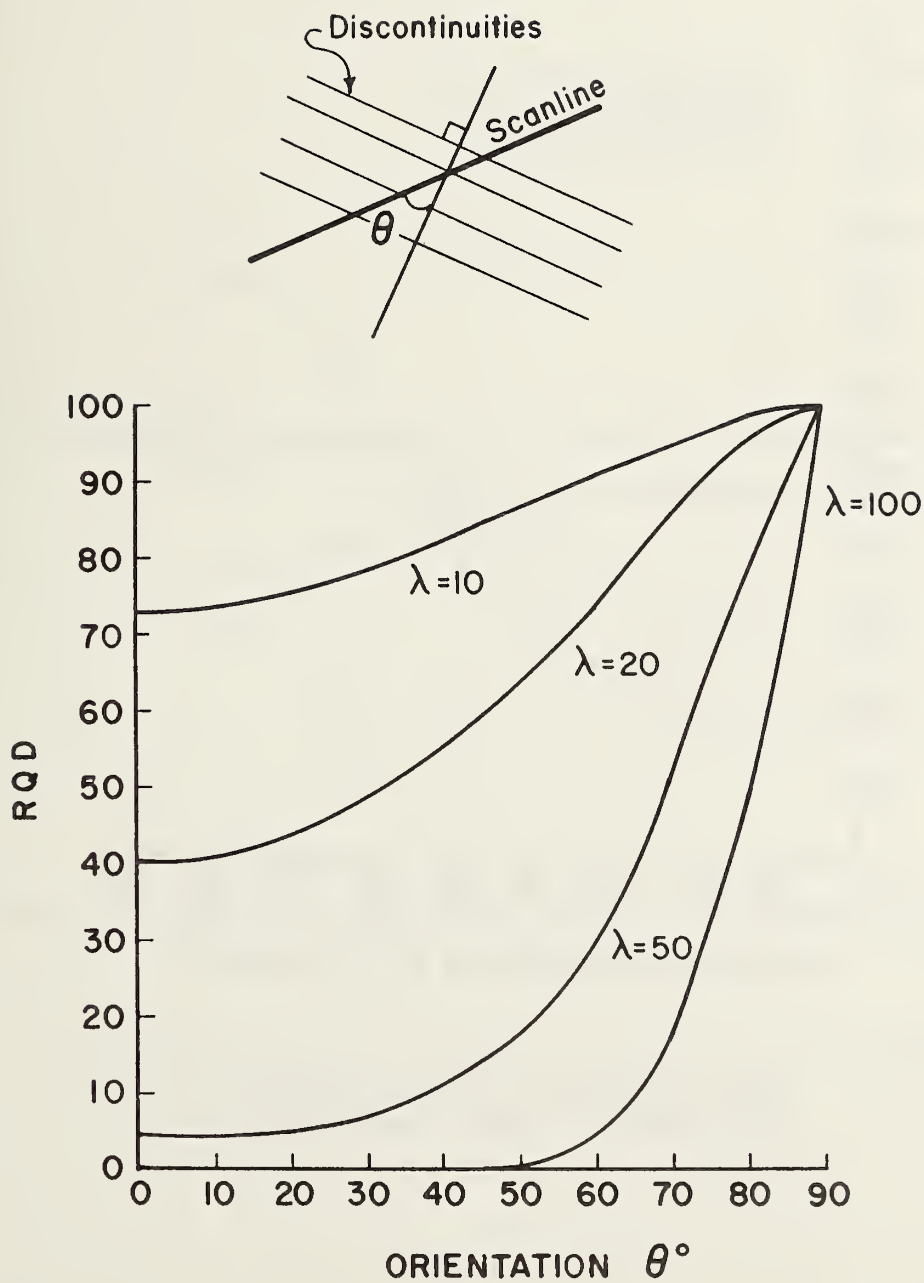


FIGURE C-5 RQD FOR PARALLEL DISCONTINUITIES AS A FUNCTION OF ORIENTATION

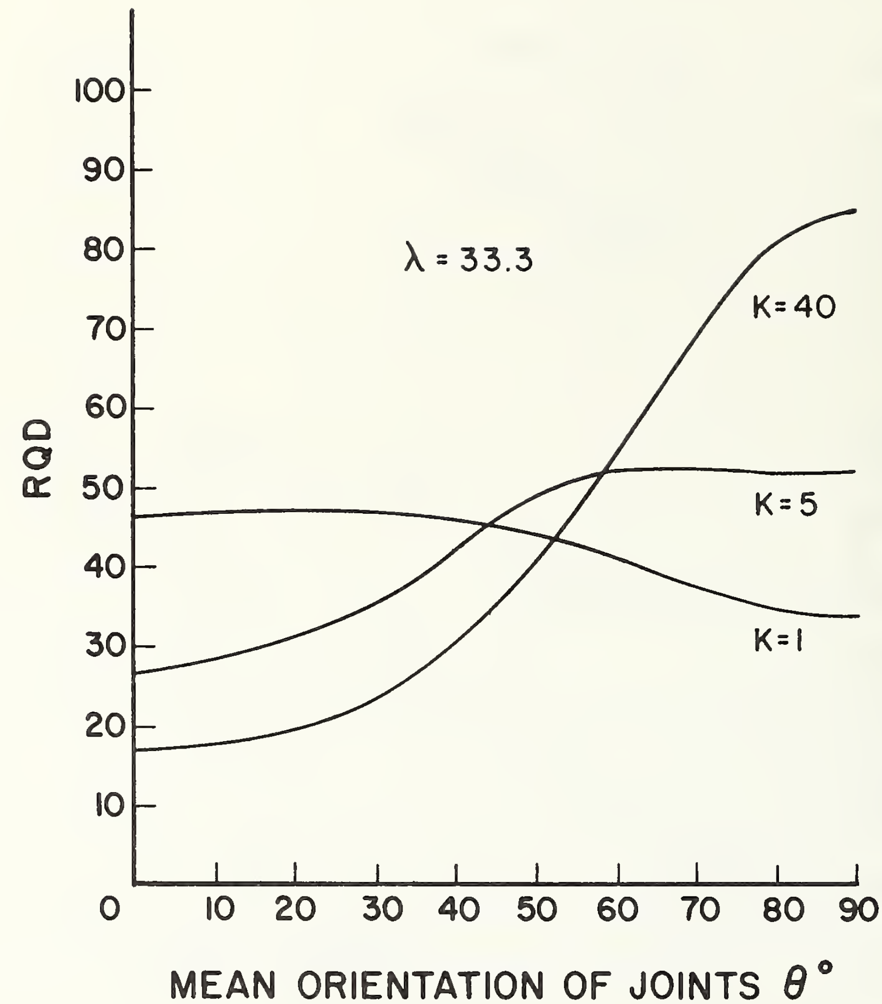


FIGURE C-6 RQD FOR NON-PARALLEL DISCONTINUITIES AS A FUNCTION OF MEAN ANGLE OF ORIENTATION (FROM DERSHOWITZ,1979)

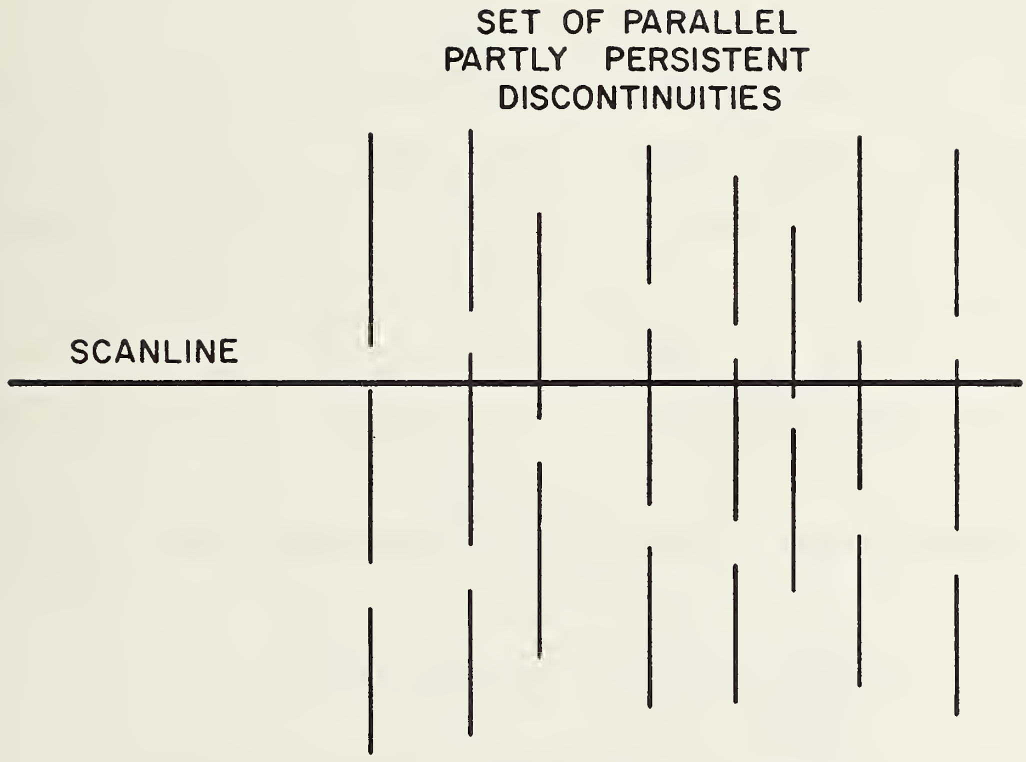


FIGURE C-7 PARTLY PERSISTENT SET OF DISCONTINUITIES

$$P = \frac{\text{Area of Discontinuity}}{\text{Total Area}} \tag{C-9}$$

The average discontinuity spacing is m_d thus the frequency of discontinuity planes is $\lambda_d = \frac{1}{m_d}$ (C-10)

Since the discontinuities are only partially persistent only persistent ones will be recorded along in scanline (boring). The frequency of recorded discontinuities along the scanline is

$$\lambda_r = P \cdot \lambda_d \tag{C-11}$$

By substituting λ_r in Eq. C-6 the expected value of RQD is:

$$E[RQD] = 100 \cdot e^{-P\lambda_d}(0.1\lambda P + 1) \tag{C-12}$$

Results of Eq. C-12 are shown in Fig. C-8.

Finally the expressions for RQD may be generalized for several sets of non-persistent discontinuities that are inclined to the scanline (Fig. C-9).

The total frequency of intersections with the scanline is

$$\lambda_s = \sum_{i=1}^{n} \lambda_i \, P_i \cos\theta i \tag{C-13}$$

where: n = number of discontinuity sets

λ_i = frequency of discontinuities in set i

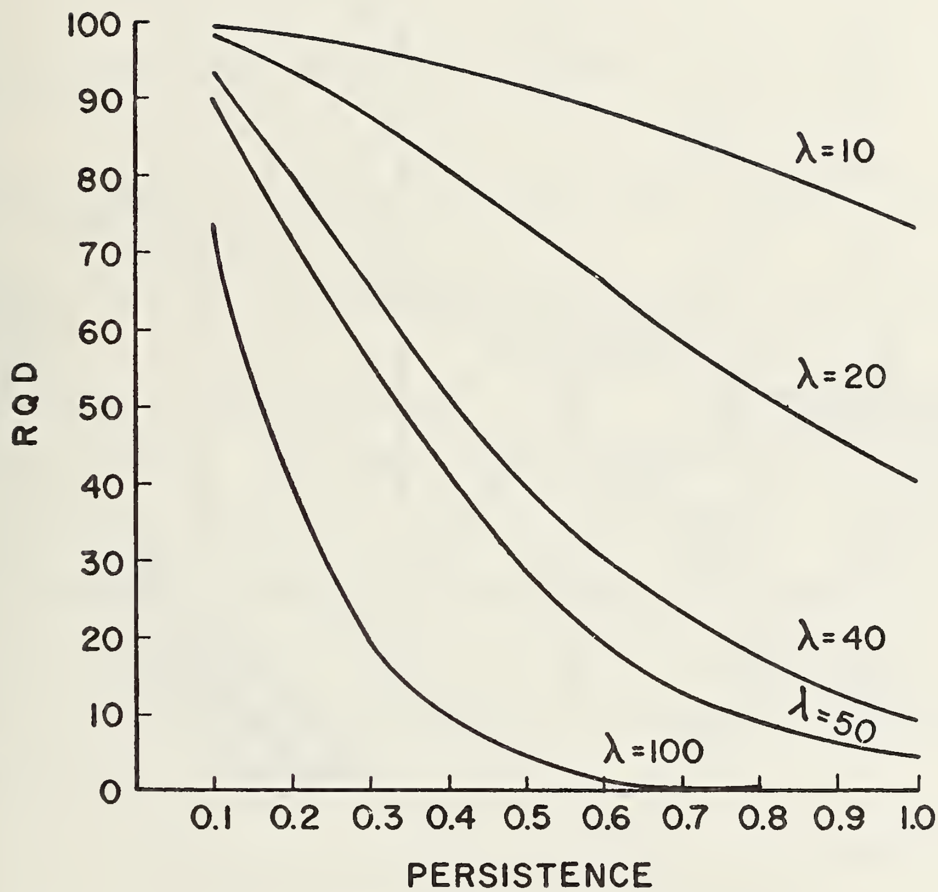


FIGURE C-8 EFFECT OF PERSISTENCE ON RQD

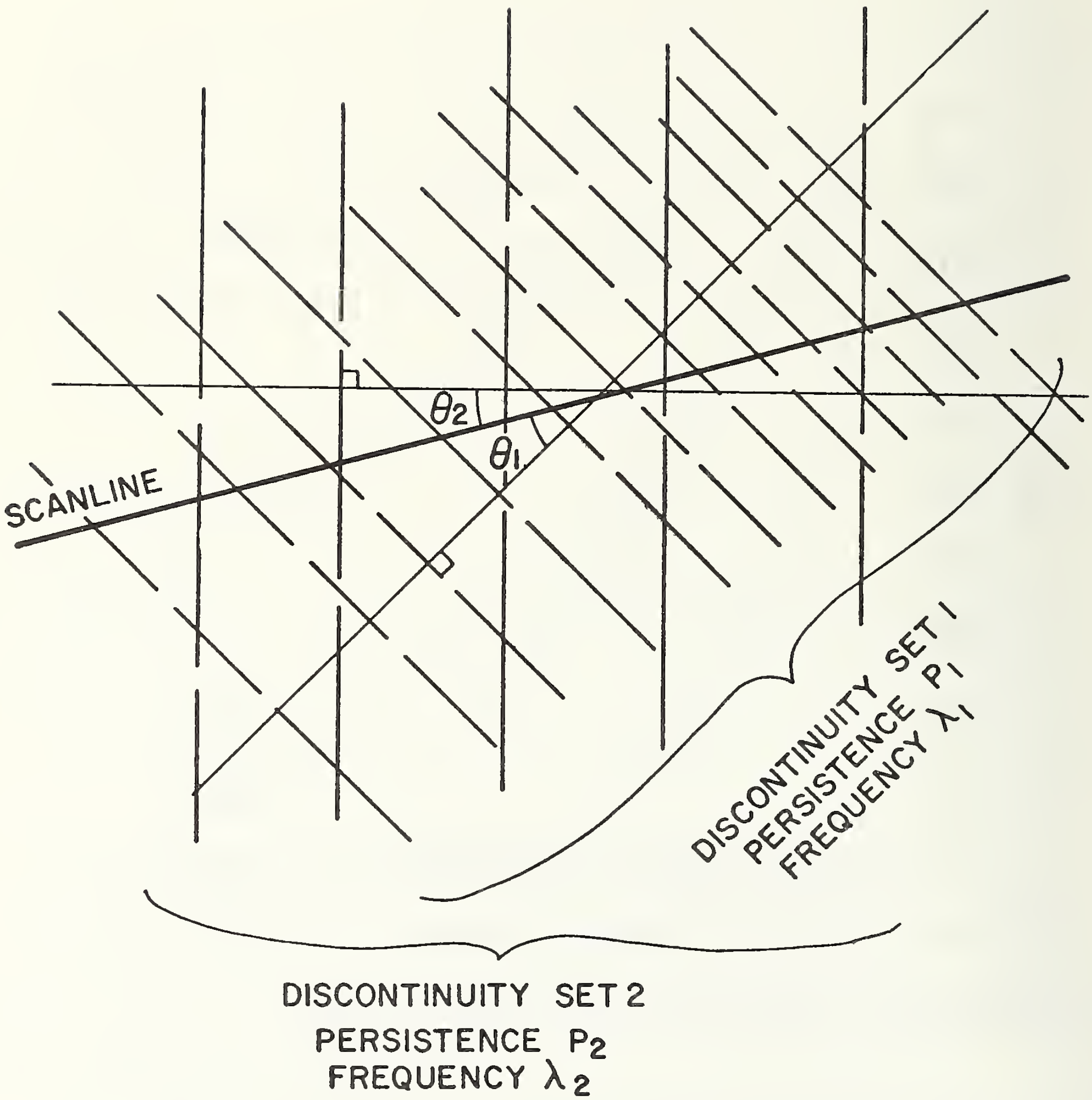


FIGURE C-9 TWO PARALLEL PARTLY PERSISTENT SETS OF DISCONTINUITIES AT ANGLE TO BORING

P_i = Persistence of discontinuities in set i

θ_i = angle between scanline and normal to discontinuity in set i.

By substituting Eq. C-13 into Eq. C-6 we obtain

$$E[RQD] = 100[0.1(\Sigma\lambda_i P_i \cos\theta_i)+1] \cdot e^{-0.1(\Sigma\lambda_i P_i \cos\theta_i)} \tag{C-14}$$

For all cases with equal λ_s (Eq. C-13) the predicted RQD will be the same. There are many possibilities of combining the discontinuity sets n, the spacing within an individual set and the persistence and orientation of a set. Thus one RQD value may present completely different rock masses. This does not even include other effects on RQD like weathering or filled discontinuities.

APPENDIX D

DETAILED DISCUSSION OF ASPECTS OF THE Q-METHOD AND REVIEW COMMENTS BY BARTON AND LIEN

D.1 INTRODUCTION

In this appendix, some aspects of Barton's et al. Q-System are discussed in detail, in particular:

- consideration of joint orientation (D.2)
- ground quality support pressure relations (D.3)

The review comments by Barton and Lien, except for some aspects that are directly related to Section 3.5.5 and were presented there, form Section D.4 of this appendix.

D.2 CONSIDERATION OF JOINT ORIENTATION

Orientation of joints and shears is only indirectly considered. The problem is not a lack of recognition of the influence of the joint orientation but how this should be considered in the Q-System. In the 1974 and 1975 papers, Barton et al. write:

> The parameter J_r and J_a (representing shear strength) should be relavant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favorably oriented for stability, then a second, less favorably oriented joint set or discontinuity may sometimes be of more significance, and its higher value of J_r/J_a should be used when evaluating

> Q. The value of (J_r/J_a) should in fact relate to the surface most likely to allow failure to initiate.

In the 1974 paper they also write:

> Undoubtedly, there are several other parameters which could be added to improve the accuracy of the classification system. One of these would be joint orientation. Although many case records included the necessary information on structural orientation in relation to excavation axis, it was not found to be the important general parameter that might be expected...If the joint orientation would have been included, the classification system would be less general, and its essential simplicity lost.

In the 1975 paper they also stated:

> Potential users of this classification method will have noted that the only mention of joint orientation is in Note 3* above. Most of the case records that were analysed included the necessary information on structural orientation relative to the excavation axes. However, the information was not found to be sufficiently important to justify the use of a seventh parameter.
>
> No doubt this was in some cases due to the fact that excavation axes were already oriented favorably with respect to weakness zones. It is certainly necessary to orient important excavations favorably with respect both to stress anisotropy and to weakness zones, as usually attempted. Most of the influence of orientation is automatically reflected in the value of Q since the parameters J_n, J_r, J_a, and SRF are indirectly weighted by "unfavorably oriented" features.

In the 1976 paper Barton et al. write:

> The exclusion of *orientation* as a separate parameter in the Q-system has been criticised quite widely, but possibly the basic philosophy of

---

*See Table 3.5.23.

the Q-system has not been fully appreciated by those concerned.

In all publications it has been emphasised that the parameters J_r *joint roughness number,* (Appendix: Table 3) and J_a *joint alteration number,* (Appendix: Table 4) should apply to the joint set or single discontinuity most likely to *allow failure to initiate.* The orientation of the feature relative to the excavation is implicit in these instructions. A practical example may be useful here. The Q-system was recently used for estimating the support requirements of a 19 meters span hydro power cavern and a parallel gate gallery of 3.5 meters span. A vertical narrow shear zone intersected the axis of both excavations, more or less perpendicularly. Besides other joint sets there was also a set of unfavourably orientated smooth, undulating joints dipping at about 50° from the downstream walls. The minimum value of J_r/J_a is obviously obtained from the shear zone. However, due to its favourable orientation this was ignored in the classification and the slightly higher value of J_r/J_a for the unfavourably orientated joints was considered more relevant. If the shear zone had been looser and clay-bearing, then clearly it would re-establish itself as the potential source of failure, and a lower Q-value and heavier support would result.

These comments show that the Q-system seems to recognize the significance of joint orientation, but that there may be a problem in how to practically consider it:

- It is recommended to select the J_r and J_a for the critically oriented discontinuity. However no guidelines are provided on which combinations of strength and orientation are the "significant" ones.
- Similarly, if several joint sets exist, all with the same J_r and J_a but different orientation, it is

not possible to include the effect of unfavorable orientation.

- If only one predominant joint set exists, orientation is difficult to consider with the Q-system. However, the orientation of such a single joint set may be crucial.

  This effect has been observed in the Arlberg- and Tauern tunnel (see also Appendix E and Steiner et al.,1979). In both tunnels performance of the main tunnel was substantially different from those of the cross-cuts that are oriented at right angles to the axis of the tunnel. In the Tauern tunnel, schistosity was striking perpendicular to the axis of the main tunnel (i.e. parallel to the cross-cut); in the Arlberg tunnel the shear zones were striking subparallel to the axis of the tunnel (perpendicular to the cross-cuts). In both cases performance was much worse if the geologic structure was subparallel to the axis of the opening.

D.3 GROUND QUALITY - SUPPORT PRESSURE RELATIONS

This discussion, although dealing only with Barton's et al. support pressure relation, should be viewed as a description of the problematic aspects of support pressure relations in general. It is therefore appropriate to start with the fact that Barton et al. (pers. comm., 1979 and following review comments, D.4) no longer recommend the use of support pressure equations.

Two equations to predict support pressures in the roof have been recommended:

$$p_{r(1)}(\mathrm{kPa}) = \frac{200}{J_r} Q^{-1/3} \quad \text{or} \quad p_{r(1)}(\mathrm{kg/cm^2}) = \frac{2}{J_r} Q^{-1/3}$$

and

$$p_{r(2)}(\mathrm{kPa}) = \frac{66.7}{J_r} J_n^{1/2} \cdot Q^{-1/3} \quad \text{or} \quad p_{r(2)}(\mathrm{kg/cm^2}) = \frac{2J_n^{1/2}}{3J_r} Q^{-1/3}$$

These relations were derived empirically from some of Barton's case studies.

The pairs of p and Q are shown in Fig. D.3. Note that there are solid black dots and open circles. The solid dots represent cases that were described in detail by Barton et al. in an appendix to the NGI-Report (Table D.2). The open circles refer to case studies that were less extensively reported (Table D.3).

<u>Determination of Support Pressure</u>

In some cases an actually monitored pressure between liner and ground is available. In other cases it has been backfigured from the capacity of rock bolts and tendons, and in a third group the pressure is a design pressure that was derived through analysis.

Among the first type is the Belchen tunnel in swelling rock (which will be discussed later). The second type includes primarily large underground chambers that were supported primarily by rockbolts and tiebacks (reported by Cording et al, 1971). The third type of pressure derivation was done for the Straight Creek Tunnel (Case 159) that will also be discussed later.

Determination of Q

For the cases represented by solid points, the actual assessment of Q and the basic parameters (RQD, J_n, J_r, J_a, J_w, SRF) has been reported by Barton. (In some of these cases actual ranges of parameters have been given; thus Q varies.)

Several regression analyses have been performed on the data (Tables D.2, D.3). Regressions 1 through 4 were performed on sets from the "black points" (Table D.2). Regressions 5 and 6 were performed on the entire data set (i.e. "black" and "open" points) (Tables D.1 and D.2). The detailed regressions performed are (Table D.3; Figs. D.1, D.2, D.3):

(1a) Regression on all "black points";

(1b) Regression on all "black points" (with Churchill Falls consisting of machine hall and surge chamber only counted once);

(2a) Regression on all "black points" without the Straight Creek Tunnel (159);

(2b) Regression on all "black points" without Straight Creek (Churchill Falls counted only once);

(3) Regression on all "black points" on maximum value of Q only;

(4) Regression on all "black points" on minimum value of Q only;

(5) Regression on all "black" and "open" data points (Churchill Falls 104, 105 only counted once);

(6) Regression on all "black" and "open" data points with the exception of the Straight Creek and Belchen tunnels.

When considering all the data, i.e. including extreme points, the regressions show high coefficients of correlation. The regression line is hinged to these extremes. However, the extremes are singular and the data for these cases may be uncertain, as will be discussed later. When the extreme cases are omitted, the regression line becomes flat and the slope of the line (double log plot), i.e. the exponent, becomes statistically insignificant (see Fig. D.2). There is actually approximately a 10% chance that the slope (the exponent) is positive, i.e. that support pressure would decrease with decreasing ground quality.

<u>Straight Creek (now Eisenhower I) Tunnel</u>

As discussed before, the extreme cases of the Straight Creek (now Eisenhower I) and Belchen tunnels have an overriding effect on the regression analysis. It is thus necessary to examine the accuracy of their p-Q values, which makes it possible to highlight some of the problematic aspects in backfiguring or measuring support pressures.

This support pressure value depends strongly on the assumed model which has many questionable features; in particular, it does not consider the effect of support (its stiffness and location of placement) which is usually of prime importance (see Vol. 1 and 2 of this series). The backfiguring of support pressures from design models introduces another problem in addition to the just discussed model accuracy: the designer is usually conservative. Unless this conservatism is explicitly formulated, design support pressures will include an unknown degree of conservatism.

The support pressure for this case is the design pressure for the main bore of the Straight Creek Tunnel. It has been derived by Hopper et al. (1972) through the following approach. It is assumed that the material in the plastic zone in the crown would loosen and drop out. This "loosening" load is superimposed to the support pressure required to limit the plastic zone to a particular size. With larger plastic zones, the "loosening" load increases at the same time the support to control the plastic zone decreases. Thus the sum of the "support" pressures shows a minimum for a certain plastic radius. This minimum is obtained with a graphical solution (curve e in Fig. D.4).

The ratio of tunnel radius to radius of plastic zone is 0.18 and the "support" pressure in the crown is 47 ksf =

23 kg/cm^2. The support pressure at the springlines (i.e. without "loosening" load) is 30 ksf = 14.7 kg/cm^2.

The Belchen Tunnel (case 173)

This is a case where support pressures have been measured mostly with stress cells at the rock support interface. The source of reference for the Belchen tunnel has not been mentioned by Barton; however, the most comprehensive source on the problems of the swelling rock in the Belchen tunnel is Grob (1972), who reported details of the field measurements and the results of laboratory tests (see Table D.5).

Barton's assessment of Q is shown in Table D.5 and will now be compared to the data reported by Grob.

RQD and J_n were assessed by Barton with RQD = 0 and J_n = 20 for gypsum, and with RQD = 40 and J_n = 4 for clayshale. Grob (1972) discusses the stability of the ground during excavation. The "Gipskeuper" (gypsum) was stable during excavation, whereas the stability of the "Opalinuston" (clayshale) varied because it was irregularly jointed. However, Barton's assessment of RQD/J_n (= $\frac{10}{20}$ for gypsum; = $\frac{40}{4}$ for clayshale) seems to run counter to the assessment by Grob. On the other hand, the gypsum undergoes much greater swelling deformation and produces higher support pressures. Stability during excavation and time-dependent pressure development are thus just the opposite of each other.

In view of these results, it seems to be somewhat questionable if swelling rock, at least rock of the kind encountered in the Belchen, can actually be handled by the Q method.

Conclusions on Support Pressure Relations

The necessity to rely on a model to backfigure support pressures or to interpret test results, the variability of parameters, and the problematic aspects of regression analysis make support pressure relations questionable. Instead of directly correlating ground parameters and support quantities (and excavation procedure) one tries to express support by "support pressure". This is an additional step and introduces additional inaccuracies mentioned above. The discussion of Barton's support pressure relations served to illustrate this, and as mentioned before, Barton et al. are recommending not to use these relations any more. The reason why the method was used to show those problematic aspects is not because it is any better or worse than others; rather, it is the only one whose detailed description of the base cases made such a detailed discussion possible.

TABLE D.1 DATA BASE FOR SUPPORTPRESSURE RELATION (DETAILED CASES = BLACK POINTS)

CASE	No.	Q	p	RQD	J_n	J_r	J_a	J_w	SRF
Veytaux	101a[1]	0.37	1.1	50	12	2	6	0.66	2.5
Veytaux	101a[2]	3.7	1.1	75	9	2	3	0.66	1.0
Churchill Falls	104[1]	5.2	1.3	95	9	1.5	4	0.66	0.5
Main Chamber	104[2]	31.6	1.3	95	9	1.5	1.0	1	0.5
Churchill Falls	105[1]	5.2	1.3	95	9	1.5	4	0.66	0.5
Surge Chamber	105[2]	31.6	1.3	95	9	1.5	1.0	1	0.5
Nevada I Test Site	141	1.5	1.4	90	3	1.5	2	1.0	15
Nevada II Test Site	142	0.39	1.4	70	9	1.5	2	1.0	15
Snowy Mountain	148[1]	5.3	0.7	85	6	1.5	4	0.5	0.5
Snowy Mountain	148[2]	50	0.7	100	4	2	1.0	0.5	0.5
Straight Creek	159-4	0.0014	24	0	20	1	8	0.33	15
Straight Creek	159-4	0.0014	37	0	20	1	8	0.33	15

TABLE D.2 DATA BASE FOR SUPPORT PRESSURE RELATION (OPEN CIRCLES)

Q	p (kg/cm^2)	Remarks
0.0017	16	Case 173, Belchen, Anhydrite
0.0017	36	Case 173, Belchen, Anhydrite
0.10	3	Case 173, Belchen, Clayshale
0.88	2.0	
3.1	1.1	
3.9	1.6	
5.2	1.4	
6.2	1.8	
6.3	1.0	
9.0	1.0	
21	0.8	
17	0.35	
18	0.5	
30	0.25	
45	0.9	

TABLE D.3 SUMMARY OF REGRESSION EQUATIONS

CASE	Regression for $p(kg/cm^2)$	r^2	Standard error of exponent	95% Bounds for exponent	Remarks
1a	$p = 2.08 \cdot Q^{-0.333}$	0.819	0.0495	-0.44 -0.22	Fig. D.2
1b	$p = 1.92 \cdot Q^{-0.355}$	0.848			Fig. D.3
2a	$p = 1.217 \cdot Q^{-0.0494}$	0.104	0.0512	-0.1675 +0.0687	Fig. D.2
2b	$p = 1.20 \cdot Q^{-0.0739}$	0.215			Fig. D.3
3	$p = 1.35 \cdot Q^{-0.0741}$	0.303	0.0562	-0.23016 +0.08108	Fig. D.2
4	$p = 1.21 \cdot Q^{-0.065}$	0.0996	0.09809	-0.337 +0.207	Fig. D.2
5	$p = 1.944 \cdot Q^{-0.367}$	0.870			Fig. D.4
6	$p = 1.412 \cdot Q^{-0.207}$	0.317			Fig. D.4

TABLE D.4 PARAMETERS FOR BELCHEN TUNNEL

CASE	GROUND	p (kg/cm^2)	Q	RQD	J_n	J_r	J_a	J_w*	SRF*	SPAN	ESR
173[1]	Gypsum Anhydrite	16	0.0017	10	20	1	20	0.66	10	11 m	1
173[2]		36	0.0017	10	20	1	20	0.66	10	11 m	1
173	Clayshale	3	0.10	40	4	1	20	1.0	5	11 m	1

* J_w and SRF have not been reported in detail by Barton, but the ratio J_w/SRF could be backfigured from Q and the other four parameters. The values selected for J_w and SRF correspond to the backfigured J_w/SRF.

TABLE D.5 SUMMARY OF STRENGTH PROPERTIES FROM LABORATORY TESTS FOR ROCK IN BELCHEN TUNNEL (AFTER GROB, 1972)

Name	Cohesion	ϕ
Gypsum Anhydrite & Marl (Gipskeuper)	65 to 170 $\frac{kg}{cm^2}$	35°
Clayshale (Opalinuston)	60 $\frac{kg}{cm^2}$	30°

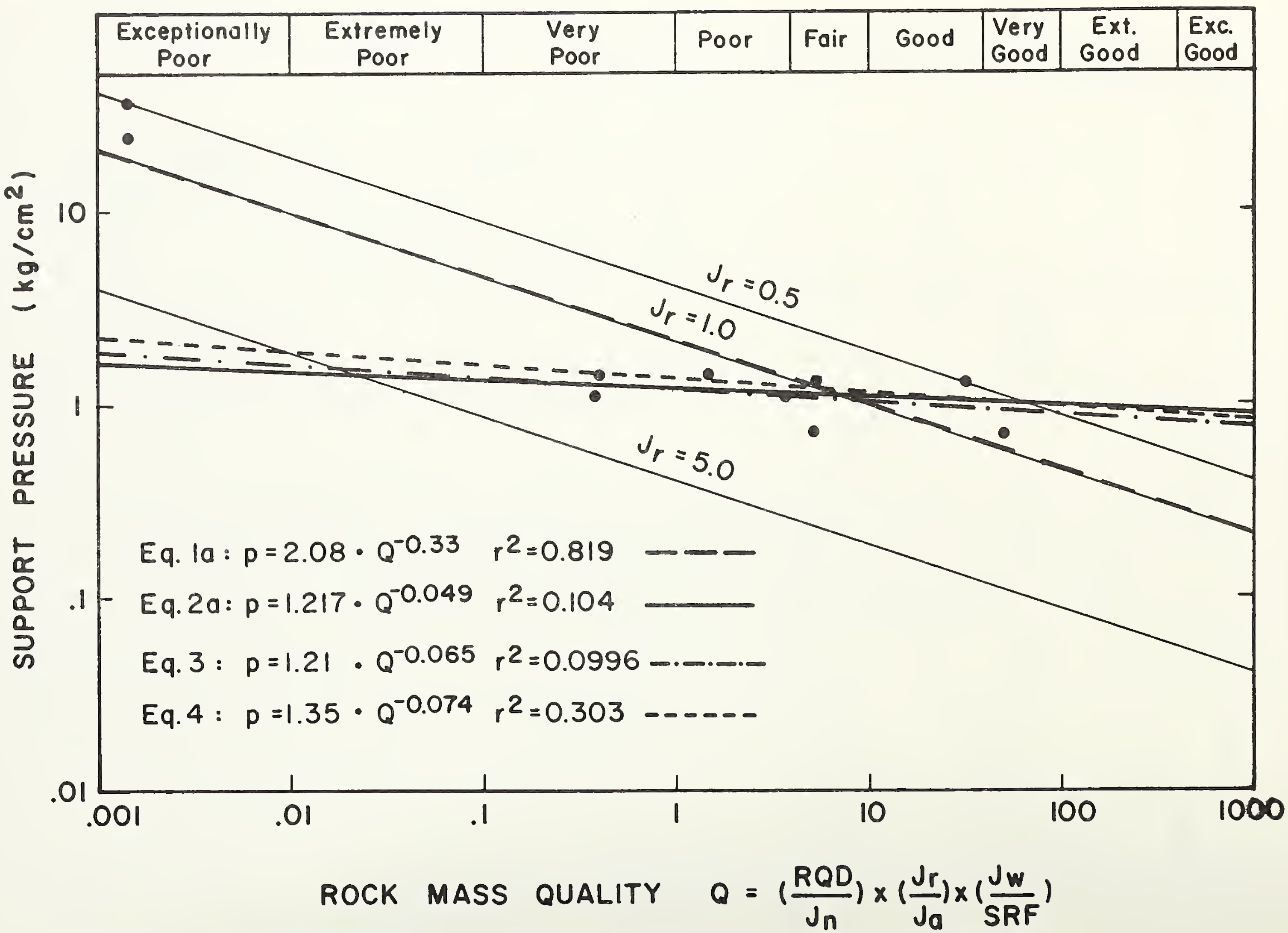


FIGURE D-1 REGRESSION ON DETAILED CASES ONLY (BLACK POINTS)

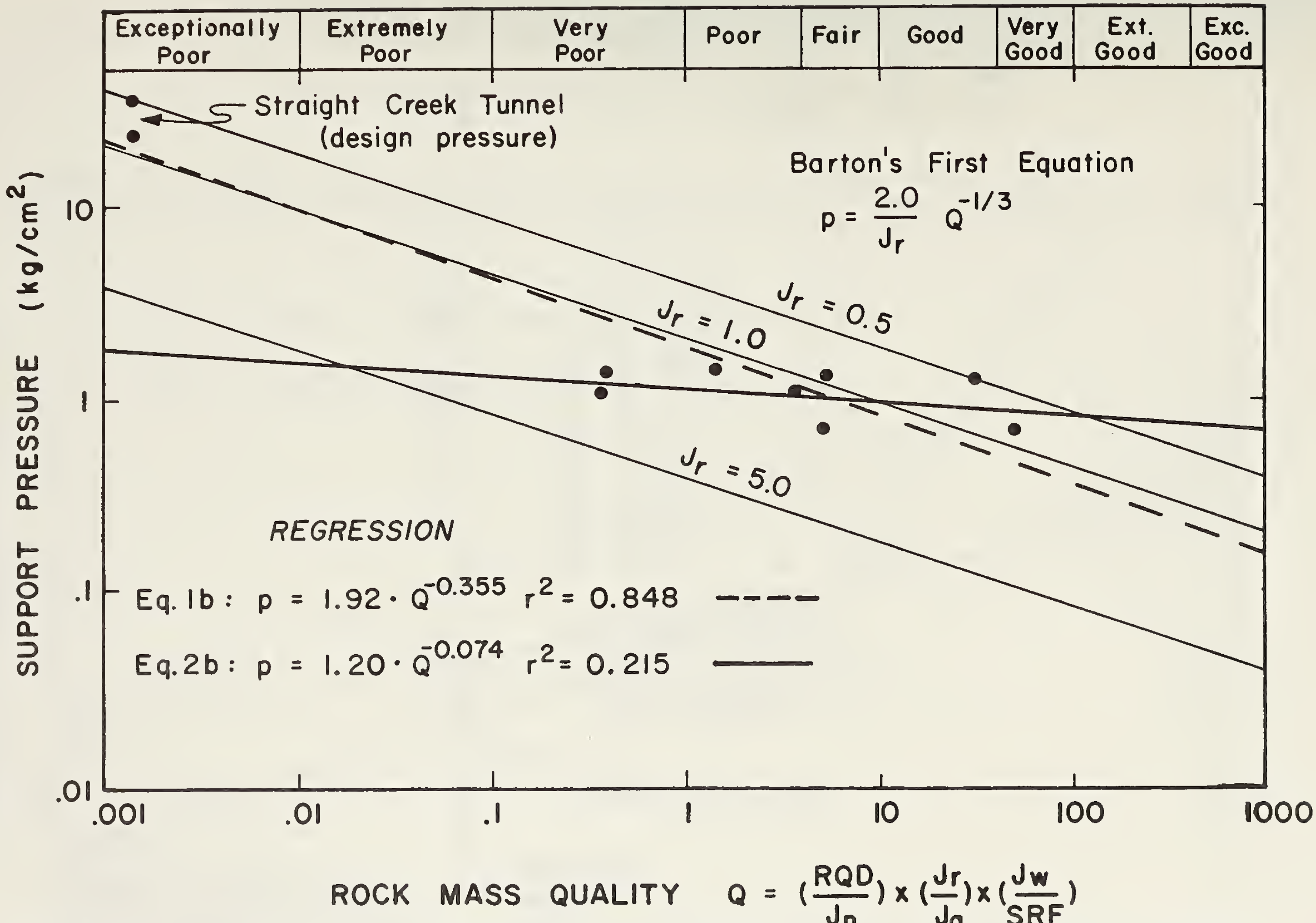


FIGURE D- 2 REGRESSION ON DETAILED CASE STUDIES AND CHURCHILL FALLS COUNTED ONLY ONCE

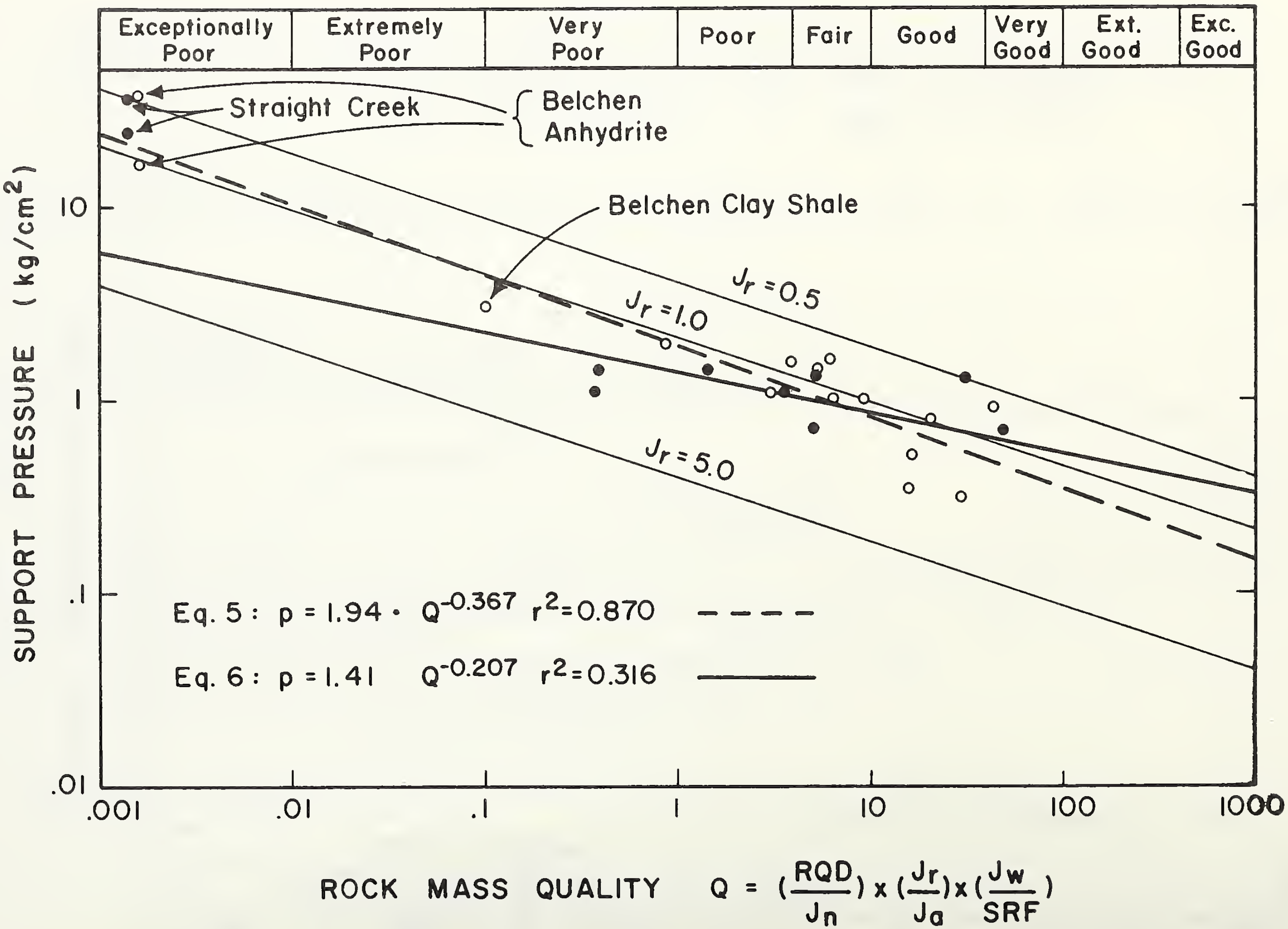


FIGURE D- 3 REGRESSION ON ALL DATA POINTS

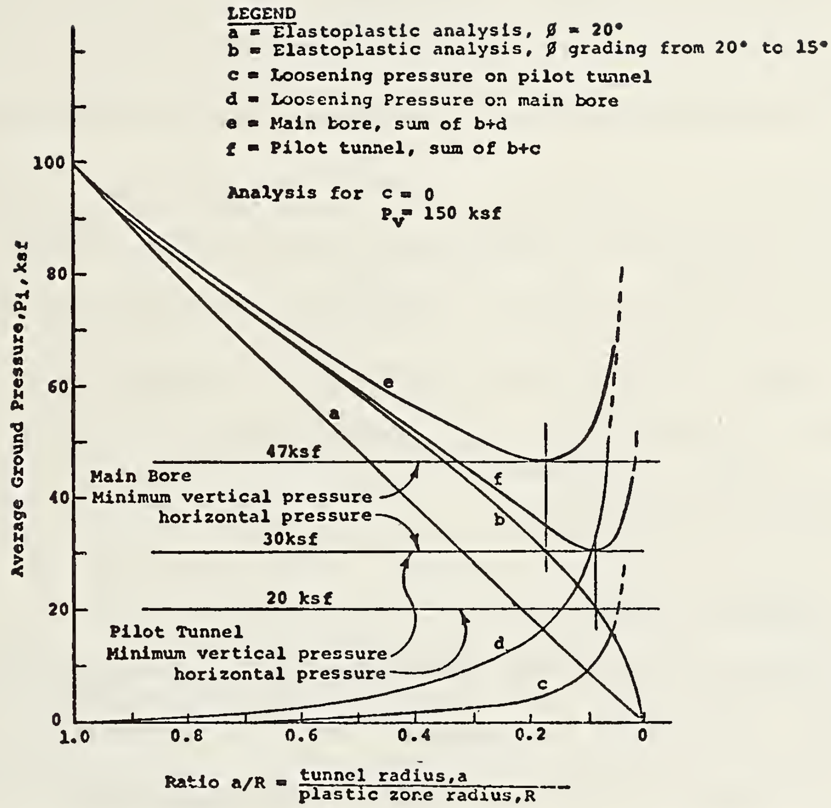


FIGURE D- 4 DESIGN USED FOR STRAIGHT CREEK TUNNEL
(FROM HOPPER ET AL.,1972)

D.4 REVIEW COMMENTS BY BARTON AND LIEN

The draft of this report was sent to all creators of modern empirical methods for comments. Barton's and Lien's comments contained some corrections which were directly incorporated in the text, as well as general and specific parts that provide additional information or show discrepancies with the authors' assessment. These are given below. The authors would again like to express their gratitude for the time and effort that Barton and Lien devoted to the review.

D.4.1 General Comments

> It seems to us that several of the methods reviewed were developed for diverse purposes, and it is therefore difficult to compare them in a consistent manner. It would perhaps have been wiser to group the methods as follows, before attempting a comparison:
>
> 1. Description of rock masses with respect to stability.
> 2. Relationships between rock mass descriptions and support (direct or via support pressure concepts).
> 3. Tunnel support methods based on deformation measurement.
>
> The final objective is of course that a tunnel is permanently stable once completed. However the different empirical methods contribute to this objective at different periods during a given project.
>
> There is also diversity in the philosophy towards classification. Methods such as Terzaghi's, Deere (Sao Paulo) and NATM rely on a verbal description as a basis for classifying the rock mass. Bieniawski's geomechanics classification, the Q-system and others are specifically numerical descriptions of the rock

mass. It is irrelevant to compare for example the Q-system with the NATM, which is a method of tunnel support based on deformation measurements. Nevertheless, because of the different "time of application" it could be quite useful to use the Q-system for an early classification of the rock mass to be followed by deformation measurements if conditions warranted such.

In most cases deformation measurement would not be warranted and the Q-system would be found perfectly adequate for final design decisions. Both methods would benefit from complementary application. The more detailed and specific numerical ratings for the rock mass obtained from the Q-system would improve the definition of (NATM) ground classes. At the same time, certain loosely defined parameter descriptions in the Q-system like "squeezing" (SRF = 5 - 20) and "swelling" (SRF = 5 - 15) could sometimes be better defined, and perhaps related to the measured deformation magnitudes. Such a "modification" we feel, would be much more valuable than trying to "simplify" ground classification into the two basic parameters, i.e. strength and spacing, as suggested by the authors.

D.4.2 Specific Comments

(A few other specific comments have been presented in

Section 3.5.5.)

Limited Knowledge of Parameters Prior to Construction

It must be true to say that a comprehensive method that gives all the correct answers before construction begins is an Utopian ideal. It is obviously necessary to update the preliminary rock mass descriptions once excavation begins. However, previous experience will have allowed one (as with most methods) to estimate the most likely range of tunnel-depth parameters, based on surface mapping and drill core analysis (if available).

Extrapolation

It is of course true that "Boreholes, outcrops and observations in the tunnel provide substantially different information." However, if the purpose of surface mapping and borecore analysis is to predict conditions at depth, as it should be, then quite a good degree of correspondence should be obtained by experienced users. From previous jobs they should have some knowledge of the way in which joint spacing usually increases, the way the number of joint sets may reduce, etc. It is normal practice to extrapolate the rock stress-strength parameters to tunnel depth - why not also the joint properties? Some detailed comments on this extrapolation process were given by Barton (1976).

Tendency of Empirical Methods to Over-Estimate Support Requirements

Approximately half the base cases incorporated in the Q-system are Scandinavian tunnels. The common propensity for final concrete linings regardless of rock quality in many countries with less experience in tunnel building, leads of course to over-designed base cases. This lining philosophy would not be economically viable in Scandinavia, where underground construction is so widespread. Support is specifically designed to correspond to the variable rockmass conditions. It is unwise of the authors to include the Q-system support recommendations in their universal over-design assumption. (A few of the extreme base cases are of course over-designed, due to the over-reaction to difficult conditions. They form only a small fraction of the 200 base cases.)

The authors' comparison of each method's support estimates with Cecil's (1970) *unsupported cases* shows that Bieniawski's (1976) method predicts considerable support in all cases, and Deere's (1969) method even larger amounts of support. The Q-system of course predicts no-support since Cecil's cases form the backbone of the method. Such discrepancies reflect the important differences in support philosophies between Scandinavia, South Africa and the U.S.A.

Consideration of Construction Sequence and Inclusion of Construction Effects

Few of the methods score "yes" ratings where *Construction effects* are concerned. The Q-system certainly has no special parameter that accounts for the relative effects of for example heavy blasting, smooth wall blasting, full face (TBM) excavation. However, their relative effects (on the zone of rock around a tunnel that requires support) are generally reflected in the Q-rating, if classification is performed in the tunnel. Here we are thinking of the effect particularly on RQD, where incipient joints and tight features that might otherwise be missed, are readily classified when too much energy has been expended in excavation, i.e. when blasting with excessive charges and lengths of rounds.

Explicit Statement of Limitations

As far as the Q-system is concerned, the limitations of the Q-system are implicit in the density of the base cases on the SPAN/ESR versus Q graphs. All the Q-system publications containing tables of recommended support (Barton et al. 1974, 1975) have the following footnotes under each table:

> *"Author's estimates of support. Insufficient case records available for reliable estimation of support."

The asterisk is placed against no less than 17 of the 38 support "boxes" (classes). The great majority of the 200 case records are distributed amongst the remaining 21 support "boxes". As can be seen from the distribution of case records, greatest reliability is found in the range of Q = 0.01 (extremely poor) up to Q = 100 (very good), though there are also a number of cases (8) of unsupported excavations with Q > 100 (extremely and exceptionally good).

The density of case records in this range of Q is sufficient to give reliable permanent support recommendations for values of SPAN/ESR (metres) in the range from 4 to 25 metres. In practice this means spans ranging from about 5 or 6 metres (ESR usually 1.6 or 1.3) up to 25 metres (ESR usually 1.0 - power station category).

Modified Empirical Methods for Observational Tunnel Design Construction Methods

The authors make some hypothetical recommendations for modifying existing empirical methods to make them "applicable in observational tunneling". However, the modifications are not practically proven, and appear of little help. The writers cannot for example share the opinion that the proposed comprehensive parameters of the "modified Barton Q-system" can be assessed directly with increased experience:

"strength parameter" $J_r/J_a \times J_w/SRF$

"spacing parameter" RQD/J_n

It is not possible to describe a complicated situation adequately with two parameters. In what way would a figure such as Figure 5.5 (p. 352) be used?

Support Pressure Discrepancies

The authors made a detailed regression analysis of the 27 cases used by Barton et al. (1974) in establishing their approximate support pressure - Q diagram. The authors find that "when considering all the data, i.e. including extreme points, the regressions show high coefficients of correlation. The regression line is hinged to these extremes." However they find that "when the extreme cases are omitted the regression line becomes flat."

We agree with the authors that the support pressure relations are not reliable - how could they be with relatively few base cases, and with the inherent interpretive step involved in analysing the actual pressures on the support? The authors have never used the relationships in design for this reason. In fact, they were presented in two of the publications due to the interest in comparing the Q-system with Terzaghi's support pressure predictions.

Part of the problem in obtaining support pressures from case records reported in the literature is that "support pressure" is variously defined. As pointed out by the authors, sometimes it is measured, sometimes it is a design pressure and sometimes it must be backfigured using the "hoop stress" formula. In addition there is the imponderable "support itself affects support pressure", i.e. different support methods in identical ground conditions are subject to different loads. Time of installation produces further variations.

Furthermore, the purpose of systematic bolting and shotcrete is to create a self-bearing arch of reinforced rockmass. It is difficult to compare this with the design of a concrete lining that must tolerate pressure from an unreinforced rock mass.

The writers have not favoured the use of support pressure relations because plenty of case records are available for estimating support requirements directly from the Q value. The infrequent use of continuous cast concrete linings, and the even less frequent use of steel sets in Scandinavia, make support pressure "design" formulae of limited interest.

On page 472 the authors question our classification of the Belchen tunnel - one of the extreme cases involving swelling gypsum. The writers assigned the values $RQD/J_n = 10/20$ since, being wise after the event, it was not necessary to make the mistake of misinterpreting the fact that the "Gipskeuper" was stable during excavation.

When the writers encounter a hydrothermally altered granite containing montmorillonite and note dry stable conditions during excavation, they nevertheless classify such ground with appropriate long term saturated swelling conditions in mind. "Crushed rock, earth-like" ($J_n = 20$) is the most relevant description of the long-term swelling conditions.

APPENDIX E

RESULTS OF ANALYSIS OF PERFORMANCE DATA

E.1 INTRODUCTION

Parallel to the review of existing empirical methods, data from several tunnel projects were analyzed in order to establish improved relations between ground conditions, excavation-support procedure, and performance. Detailed data was obtained from Tauernautobahn AG, Salzburg (Owner) for the Tauern (6.4 km) tunnel and from Ingenieurgemeinschaft Lässer-Feizlmayr, Innsbruck (Design Engineer) for the western section of the Arlberg tunnel (2.3 km). Both tunnels are located in squeezing ground and substantial convergences occurred. The two cases are described in detail in Steiner, Einstein, and Azzouz, 1979 (= Vol. 4 of this report series). In the following, the principle of data analysis is briefly reviewed, followed by a discussion of the results.

For the two tunnels a total of 303 performance monitoring cross-sections were available, as were detailed geologic maps and detailed support quantities and dimensions. This information has been transformed such that it can be specifically treated by computer. Data storage was accomplished interactively with the MIT Multics System followed by the data management using the Consistent and Janus programs.

E.2 DATA STORAGE

For each of the 303 cross sections geologic, support, and performance data were stored (approximately 50-100 per cross-section, i.e. a total of 15,000 to 30,000 data).

Geologic data included:

Primary Ground Type, i.e., the predominantly observed ground

Secondary Ground Type, i.e., ground of secondary dominance

Water (verbal, qualitative)

Degree of Jointing (verbal)

Number of Sets of Discontinuities (maximum = 3)

For each set of discontinuity:

- strike (relative to tunnel axis)
- dip (degrees)
- description (verbal)
- spacing (meters)
- thickness (centimeters)

Overburden (meters)

Support data included:

Thickness of Shotcrete (centimeters)

Type of Steelsets

Spacing of Steelsets (meters)

Four types of Bolts in crown and springlines:

- density (bolts per lineal meter of tunnel)
- length (meters)
- capacity (metric tons)

Invert Bolts:

- density (bolts per meter of tunnel)
- length (meters)
- capacity (metric tons)

Geometry of Tunnel:

- Width (meters)
- Height (meters)
- Radius in crown (meters)
- Radius of springlines (meters)
- Radius of invert (meters)

Performance data:

- Time of measurement (days)
- Convergences (Top, Lower) (millimeters)
- Crown settlement (millimeters)
- Convergence rates (millimeters/day)

E.3 DATA MANAGEMENT

As indicated in E.1 analyzing the data from the Tauern and Arlberg tunnels should lead to relations between ground conditions, support characteristics and observed peformance of support and tunnel. These relations should be practically useful; they should have predictive character and allow prediction of performance from ground conditions and support characteristics. The purpose of the data management is to select from the data set attributes that characterize best ground, support and performance. Such attributes can be:

- all the characteristics on which data are provided in the case records, i.e., the characteristics listed in E.2.

- a selected set of these characteristics.
- combination of these characteristics (e.g., primary ground type and water conditions could be combined to form one attribute).

Basically two types of approaches could be used to determine relevant attributes:

(a) perform data analyses with any possible combination of attributes and determine those that have the highest degree of statistical significance which are then considered to be most meaningful,

(b) obtain indications from inspection of the data, from other research or from preliminary analysis.

Approach (a) would mean that all possible combinations ought to be analyzed and the best relations found (i.e., statistically highly significant). Such a search for the statistically significant results may be misleading if the attribute states could be further differentiated or if more information would be available. Such pitfalls in statistical analysis have been discussed by deNeufville and Stafford (1971). Also this approach would result in a vast amount of plots, regression curves, statistics, making it difficult if not impossible to identify the most relevant attribute.

With approach (b) certain performance indications would be used as guidelines for the initial data analysis. With those indications an initial data analysis could be performed which would then lead to additional attributes. The procedure can be repeated. For example, during the preparation of the data from the Tauern tunnel it became evident that tunnel sections in Anhydrite performed better than adjacent sections in Serpentine. (Smaller convergences but similar support quantities and overburden.) Lithology seems thus a relevant attribute. Another example is water, which has an effect on support requirements and performance (Terzaghi, 1946). Thirdly, in the Arlberg tunnel the importance of shear zones was observed (John, 1976). Thus these indicators provide first relevant attributes that can be used to systematically inspect the data and isolate additional attributes. Approach (b) is also required given the fact that there are so many attributes and attribute states but only 303 cross-sections for which attributes are available. There are just not enough data for all relations.

The attributes that were gathered for each of the 303 cross-sections can be divided into three major groups: ground, support and performance attributes. From each of these groups attribute and attribute combinations were selected. The detailed selection process for each group of attributes is described below.

Ground Attributes

Only ground attributes that were available for all cross-sections were considered; these are:

- primary ground type
- secondary ground type
- water conditions
- foliation
- shear zones

The attribute primary ground type is a qualitative attribute; so are secondary ground type and water conditions. Foliation and shear zones are qualitatively and quantitatively described, the latter concerning orientation (strike, dip) and dimensions (spacing, thickness). However only the qualitative description is available for all cases; orientation varied often quite rapidly (over less than one tunnel diameter) and dimensions were frequently not available.

The first attribute considered was primary ground conditions. This leads to differences in observed performance, as expected from the preliminary inspection. To further differentiate ground conditions, other attributes are necessary. The second attribute is secondary ground conditions; it was combined with primary ground conditions to form the attribute primary-secondary ground conditions. This combined attribute has some particular properties. In the Arlberg tunnel primary and secondary ground conditions are highly correlated. If Garnetic Mica Schist was the primary ground condition, Feldspathic Gneiss was encountered as secondary ground and vice versa. In contrast in the

Tauern tunnel primary and secondary ground conditions are not correlated. Due to these peculiar properties the attribute secondary ground could not be used for further differentiation.

The third attribute, water conditions, is qualitatively described in the case records which were developed by the engineering geologist. The attribute 'water conditions' has been combined with 'primary ground conditions' into the attribute 'primary ground-water'. In the Arlberg tunnel water conditions vary from humid to heavily dripping in all primary ground types. In contrast in the Tauern tunnel only 'dry' water conditions were encountered. The combined attribute primary ground type-water could thus be used to further differentiate the cases in the Arlberg tunnel.

The fourth attribute, joint set 1, describes the foliation. It is highly correlated to primary ground. In the Arlberg tunnel the schistosity is subparallel to the axis of the tunnel. Mica Schist is thinly foliated and the Feldspathic Gneiss is laminated. In the Tauern tunnel schistosity strikes perpendicularly to the axis of the tunnel. The phyllites are thinly foliated and Anhydrite is massive. Foliation is thus an attribute that does not bring any further differentiation compared to primary ground type.

The fifth attribute, joint description 2, describes shear zones, which were a major factor in the Arlberg tunnel. The shear zones varied in number and size in each cross-section. In some cross-sections only one shear zone was encountered within the tunnel, in others there were two or more. Joint description 2 is thus a relevant attribute in the Arlberg tunnel. In contrast in the Tauern tunnel only local shear zones were mapped which did often not coincide with the monitoring section. Thus in the Tauern tunnel shear zones were not a significant factor.

More information on the shear zone was collected which is however not complete due to the following reasons. Orientation (strike and dip) of a shear zone varied over short distances (i.e., less than a tunnel diameter) and the selection of a representative orientation value was not possible. Thickness and spacing of shear zones were not available for all cross-sections.

Another ground attribute is overburden which is a numerical attribute on a ratio scale which was considered significant based on past experience and based on the mechanisms. Two other attributes listed in Section E.2,

Degree of jointing

Joint Set 3

were not considered because they are incomplete or only available in singular cases.

The approach selected was thus to define such ground attributes that lead to a sequential refinement but ended up with sufficient number of data per attribute combination, such that trends could be established. Practically, this meant that primary ground was the first attribute considered. No further refinements could be applied for the Tauern tunnel, because the water conditions were 'dry' in all cases and no shear zones (with the exception of local ones) were encountered. In the Arlberg tunnel the ground attributes could be further refined to take into account water conditions and shear zones. The combinations of ground attributes are shown in Table E.1.

Support Attributes

Support is characterized by quantity and capacity of different types of supports, i.e., steel sets, shotcrete and bolts. Each of the support elements contributes to the total support effect. The combination of the support is very complex; not all elements are stressed to the same level of capacity. Elements may retain their capacity even after reaching it, they are ideally plastic or strain hardening. Other support types (e.g., shotcrete, concrete) are often strain softening and lose their strength once they have reached their capacity. Also the different support elements are usually not strained to the same fraction of their capacity; thus a simple addition of their maximum capacity does probably not represent the combined effect of the elements.

In the Tauern and Arlberg tunnels, so called contraction slots were used (see Appendix A, Fig. A.12). This procedure eliminates an overstressing of shotcrete and steelsets since gaps are left in the shotcrete and the jonts of the steelsets are unlocked. The third support element, bolts, were stressed to their yield capacity as reported by John (1976). Performance was controlled by adapting bolt support (John 1976); i.e., the number of bolts and to some extent their length was adapted. Since bolts were stressed to their yield capacity and gaps left in the shotcrete (and steelsets) it is reasonable to consider, as a first approximation, the yield capacity of the bolts as support capacity. However, since often different bolt types were used the number of bolts was an insufficient descriptor of support capacity.

There are several ways to normalize support capacity; one could normalize it to a standard bolt size, i.e., the number of bolts of a standard bolt capacity. Another approach and the one adopted here is to use an equivalent bolt pressure which is defined as the bolt capacity divided by the tunnel surface area they support.

Performance Attributes

In many of the monitoring cross-sections the following three quantities (Fig. E.1a) were monitored during the excavation support phase, i.e., from initial excavation up to the time just prior to placement of the final liner:

- Convergence of the top chord (H1)
- Convergence of a lower chord (H2)
- Settlement of the crown.

The convergence measurements consider the relative displacement of two bolts placed in the tunnel wall. The settlement of the crown is monitored by precision levelling. The performance of the tunnel from initial excavation to placement of the final liner can most efficiently be monitored with the top convergence and crown settlement. Convergence measurements are easy to perform with a tape measure, and take little time. The precision levelling of the crown settlement is time consuming. For reasons of practicality often only the top convergence is measured during the entire construction period. The performance is monitored until deformation stabilizes or until the final liner is placed, after the deformation rate drops to a certain level. (This procedure has been used in the Arlberg tunnel and is extensively discussed by Steiner et al., 1979 = Vol. 4 of this report series.) This final deformation is thus characterizing the performance of the ground support system. Other performance attributes such as deformation rates might also serve to characterize support. Empirical relations between rates of convergence and final deformation have been established for the Arlberg tunnel by Mayrhauser (1976) and are also reported in Steiner et al., 1979 = Vol. 4 of this report. In the Arlberg and Tauern tunnels the final deformation was most completely monitored and was thus used as performance attribute.

E.4 CORRELATION ANALYSIS

E.4.1 Principle

An attempt was made to correlate attributes by varying ground attributes and keeping either support or performance attributes constant. The detailed procedure was to plot support attribute versus the performance attribute for a set of ground conditions and to compare these plots (Fig. E.lb). Specifically one first selects one ground attribute and obtains the performance-support plot for each state of this attribute. This can then be further refined by taking one attribute state of this attribute and combining it with another attribute with associated states.

Two major ground attributes were considered:

- primary ground (lithology)
- overburden

Overburden has been considered separately and was not further refined, because in some cases the overburden was nearly constant and also because this attribute is a continuous attribute in contrast to the other ground attributes that have discrete attribute states. (To study the effect of overburden one might have to consider ranges of overburden for otherwise similar ground attributes and not enough data was available for this.)

Primary Ground (Lithology) was the other important attribute considered. It has been refined by incorporating first water conditions and then shear zones. (See Fig. E.lb.)

For the above ground attributes performance, i.e., convergence (on the horizontal axis) and support pressure (vertical axis) were plotted. The data points may show different configurations, listed below.

a) Increase in convergence and decrease in support pressure.

b) Increase in convergence, increase in support pressure.

c) Variation of convergence with constant support pressure.

d) Constant convergence and variation of support pressure.

e) Variation of both convergence and support pressure with correlation.

Each of these configurations makes it possible to simultaneously examine how support and performance are affected by varying ground conditions. The relation that one would hope to find is an increase in convergence with decreasing support pressure. This might be expected by the fact that an initially stressed medium is unloaded to different levels and the resulting deformation observed. For unloading to lower levels (smaller support pressure) a larger convergence is thus expected and vice versa. In addition to the existence or non-existence of the abovementioned support-performance is itself indicative, large scatter or reverse trends may express the effect of additional factors.

The plots employ normalized support pressure, i.e., the ratio support pressure/overburden rather than support pressure as a scale on the vertical axis. As will be discussed below, overburden has an influence on performance. It would be practical if the overburden could be incorporated into the analysis. Since it is a ground attribute it would be preferable to combine it with another ground attribute, e.g., strength or stiffness of the rock. Such attributes are however not available. However, support pressure may be considered to indirectly represent strength of the rock since the support in these tunnels has been adapted based on observed performance. Normalized support pressure, i.e., the ratio of bolt pressure to overburden may thus be considered to indirectly represent strength normalized to overburden. (Another reason for normalizing support pressure is the fact that elastoplastic analytical solutions that are frequently used for the design of tunnels in squeezing rock employ normalized strength, i.e., strength stress state ratios.

E.4.2 Application of Correlation Analysis

The correlation analysis, whose principle has been derived above will now be applied first to the ground attribute overburden (without further refinement) and then to the ground attribute 'lithology' with refinement by including 'water conditions' and 'shear zones'. First the effect of overburden will be discussed below, followed by the discussion of the attribute lithology with its different states (see Table E.1). The refinement of the attribute 'lithology' with 'water' and 'shear zones' is discussed under each state of the attribute lithology.

E.4.2.1 Effect of Overburden

Sections of the Tauern tunnel can be used to demonstrate and discuss the effect of overburden. The Tauern tunnel (for a detailed discussion see Steiner et al., 1979, =Vol. 4 of this report) is a transmountain tunnel, near the portals the overburden is shallow but increases rapidly over the first few hundred meters of the tunnel to reach 600 to 900m in the central part. Near the southern portal ground conditions are similar to the central part, the rock types that were encountered are primarily different varieties of phyllites. (The section near the northern portal had different ground conditions from the remainder of the tunnel and can thus not be used for these comparisons.) In the section near the southern portal for different phyllites only shotcrete of 10cm thickness was required as initial support (no bolts or steelsets). The observed convergence in these sections was small. The range of overburden and of performance is given in Table E.2 for the different ground types. In the deeper lying sections of the Tauern tunnel convergence increased (Table E.1), and the support quantities had to be increased. Shotcrete was increased to 15cm, steelsets and rockbolts were added. In addition, to prevent 'shear failures' of the shotcrete, so called contraction slots were left in the shotcrete and the locks of the steelsets were left open (see Appendix A, Fig. A.12). Despite larger support quantities the convergence increased with increasing overburden, for otherwise similar ground conditions. It is thus evident that overburden has an effect on performance and that performance of a tunnel deteriorates with increasing overburden.

E.4.2.2 Correlation Analysis for the Ground Attribute Lithology

The ground attributes considered in this analysis are discussed below; for a listing of them, see Table E.1.

Mica Schist

The cases with mica schist (Fig. E.2), the Arlberg tunnel are the most numerous ones. Top convergence and normalized bolt pressure vary both by an order of magnitude with no apparent correlation between the two variables.

The mica schist cases have been further subdivided based on water conditions (Figs. E.3, E.6, E.9).

For mica schist with water dripping heavily (Fig. E.3) the range of the scatter is still large and no trend visible. For mica schist and water dripping lightly (Fig. E.6), the scatter is smaller and a trend of decreasing normalized bolt pressure with increasing convergence can be seen. This plot tends to confirm a dependence of performance on normalized support pressure. Note that in this case the overburden varied in a reasonably wide range (Table E.3). For mica schist and humid water conditions (Fig. E.9) convergence varies for essentially constant normalized boltpressure. Note that for this ground attribute state the overburden was essentially constant; thus one might conclude that the remaining scatter might be attributed to variations in other ground properties or the other unknown factors. It would have been desirable that one might obtain certain trend curves for each attribute state for comparison with each other. Yet this is not possible since most of the data is clustered. What can be attempted is a comparison of the means of normalized bolt pressure and convergence (see Table E.1). For similar average convergences, the average normalized support pressure for mica schist and water dripping heavily is larger than for mica schist and water dripping lightly. Thus more water has a deteriorating influence on performance. Such a definite distinction is not possible between lightly dripping and humid water conditions because support pressure decreases and convergence increases for humid conditions compared to dripping lightly.

The effect of shear zones could be studied for mica schist dripping heavily and dripping lightly. Two or more shear zones showed larger convergences for similar mean normalized support pressure compared to one shear zone. For humid water conditions, such a differentiation was not possible since all cases had only one shear zone.

Feldspathic Gneisses

Similar trends as for mica schists were observed for feldspathic gneisses (Figs. E.10 to 17). Comparing all feldspathic gneisses with all mica schist cases one finds that the average normalized support pressures are less and the average convergences are larger for the feldspathic gneisses. This does not necessarily indicate a difference in ground conditions, however, it may represent a learning effect. In the Arlberg tunnel first the section with mica schist was encountered before feldspathic gneiss. With increasing experience less support was placed and larger convergences were allowed. The effect of water conditions and shear zones is not as evident as for the mica schist. The differences in the means are small compared to the range of variation (Table E.1). A difference of the means of normalized support pressure and convergence can be seen between lightly (Fig. E.14) and heavily dripping (Fig. E.11). For heavily dripping water conditions, the 'mean' normalized support pressure and the 'mean' convergence are larger, thus indicating also a deteriorating influence of more water. For humid water conditions (Fig. E.15) no conclusions can be drawn. The results of the feldspathic gneisses serve also to illustrate that the scatter in the results may be attributed to variations in other ground properties, and additional factors other than overburden since the latter varies in a narrow range (Table E.3).

Serpentine (Fig. E.18)

For the three serpentine cases convergence increases with increasing bolt pressure; in this case this means both normalized and absolute bolt pressure, because the overburden is the same. The results support the conclusion on the effect of variability of ground conditions and the possible influence of other factors. Three different types of secondary ground were encountered. In the section with the largest convergence and support pressure only serpentine was encountered. In the case with intermediate performance, anhydrite and talc was observed as secondary ground, and in the case with smallest convergence anhydrite alone was the secondary ground type.

Anhydrite (Fig. E.19)

Anhydrite had convergences lower by an order of magnitude than serpentine; in addition the normalized support pressures were lower, a clear indication that anhydrite performs better than serpentine.

Phyllites (Figs. E.20 to E.24)

The mean convergences increased with the mean equivalent support pressure remaining constant with increasing graphite content; chloritic calcitic performed best followed by calcitic, by calcitic graphitic phyllites and finally by graphitic phyllites. Quartzitic phyllites performed like graphitic phyllites.

E.5 CONCLUSIONS

The correlation analysis of the data from the Tauern and Arlberg tunnel has shown that overburden has an effect on support and on convergence. The detailed analysis of normalized bolt pressure-convergence relations has shown that there are few clear relations. This may be caused by 1) the variability of the ground conditions, i.e., the ground of the same lithologic unit is variable in physical properties (stiffness, strength), or the importance of other unknown factors; 2) in many cases secondary ground is present, in addition to primary ground and the proportions of the ground types vary; 3) similarly ground water conditions vary within the same qualitative attribute state. (Furthermore water conditions is an attribute that has been subjectively assigned by the engineering geologist.)

Nevertheless some general trends could be established with available data that show the importance of:

- overburden
- lithology (and petrography)
- shear zones
- orientation of structure (comparison Arlberg-Tauern)
- variability of ground properties.

These factors are discussed in detail below.

<u>Effect of Overburden</u>

Overburden has an effect on performance and support requirements as experienced in the Tauern tunnel where near the southern portal (with smaller overburden) less support was required and less convergence was observed. Increasing overburden (for otherwise constant ground conditions) can lead to greater convergences and support pressures (see Section E.4.2.1).

Effects of Lithology and Petrography

The dependence of performance on lithology is illustrated by the performance of the Tauern tunnel (anhydrite, phyllites, serpentine). Anhydrite experienced the best performance with convergences in the range of ten millimeters whereas in serpentine convergences an order of magnitude larger were observed; in addition, the "normalized support pressure" was larger for the serpentine cases. The effect of petrography is nicely illustrated by the change in performance for the phyllites. Convergence increased with the same normalized bolt pressure from calcitic-chloritic phyllites to calcitic-graphitic phyllites to graphic phyllites, i.e., with increasing graphitic content.

Effect of Water Conditions

Water conditions could only be studied for the Arlberg tunnel (mica schist and feldpathic gneiss). Cases where heavily dripping water conditions (Fig. E.3) were encountered experienced similar average conditions as lightly dripping water conditions (Fig. E.6), yet the normalized bolt pressure was larger. This indicates that ground conditions are worse for the heavily dripping conditions than for lightly dripping ones.

Conditions that were classified as humid (Fig. E.9) show larger convergences but the support pressure was less than lightly dripping ones. It is, however, not possible to definitely conclude that this represents an increase in overall ground quality or whether ground quality stayed the same.

Effect of Shear Zones

A clear effect of the number of shear zones could only be found for mica schist combined with heavily dripping water conditions where the mean convergence for two or more shear zones was larger than for one shear zone with the mean normalized support pressure in a similar range. However, the other cases (mica schist, lightly dripping and feldspathic gneiss) did not indicate substantial differences.

Note however, that even for the mica schists with heavily dripping water conditions there is a substantial overlap of the ranges of convergence and convergence for one shear zone (Fig. E.5) and two or more shear zones (Fig. E.4).

Comparison Arlberg-Tauern

The results from the Arlberg tunnel (Figs. E.2 to E.17) and those from the Tauern tunnel (Figs. E.18, E.24) indicate that the convergences and normalized bolt pressures in the Tauern were smaller. This might be attributed to the drier conditions in the Tauern, the different lithology or the orientation of rock structure. In the Tauern the schistosity strikes essentially perpendicular to the tunnel axis with only local shear zones present (length a few meters and often oriented perpendicular to the axis of the tunnel). In the Arlberg tunnel the schistosity is striking subparallel to the tunnel with parallel shear zones. The influence of the structure-orientation can also be observed in crosscuts in both tunnels. Convergence in the crosscuts of the Arlberg tunnel were smaller than in the tunnel (John, 1979). In the Tauern tunnel, in one crosscut the convergence was substantially larger (Tauernautobahn, 1975) and was finally stabilized by reducing the cross-section of the crosscut and placing a thick liner (see Steiner, Einstein, and Azzouz, 1979, for details).

Thus one has to conclude that orientation of schistosity and/or shear zones has a significant effect on the performance. However, it is not possible to state to what degree the difference in performance between Tauern and Arlberg tunnel is caused by the different orientation of the structure and to what degree it is caused by the different lithology, and water conditions.

Variability of Ground Properties

All the results show a large scatter and considerable overlap between adjacent ground attribute states. These variations, which seem to be due to additional varying ground properties and other unknown factors. The variability will have consequences on analytical performance predictions. Such predictions have been attempted (e.g., Seeber and Keller, 1979), however one has to be cautious regarding the accuracy of such predictions in view of the results of these correlations. Some hints on the performance of an opening may be possible but not an accurate prediction.

E.6 CONSEQUENCES

Although some correlations between ground attributes on one hand and support and performance on the other hand can be identified; the main result of this analysis seems to be the significant scatter of support-performance relations even for relatively narrowly defined ground.

Since the Arlberg and the Tauern tunnel both were excavated with a very similar excavation-support procedure the scatter may only to a limited extent be attributed to variations in support mechanisms and construction sequences. The scatter must be attributed to a large extent to variations in additional ground properties and the effect of other unknown factors. This indicates that the prediction of support quantities and performance prior to the construction may be difficult. An 'observational' method is thus necessary, as is well known, this has been successfully done in the Tauern and Arlberg tunnel (Steiner et al., 1979 =Vol. 4 of this report series).

TABLE E-1. SUMMARY OF DATA ANALYSIS

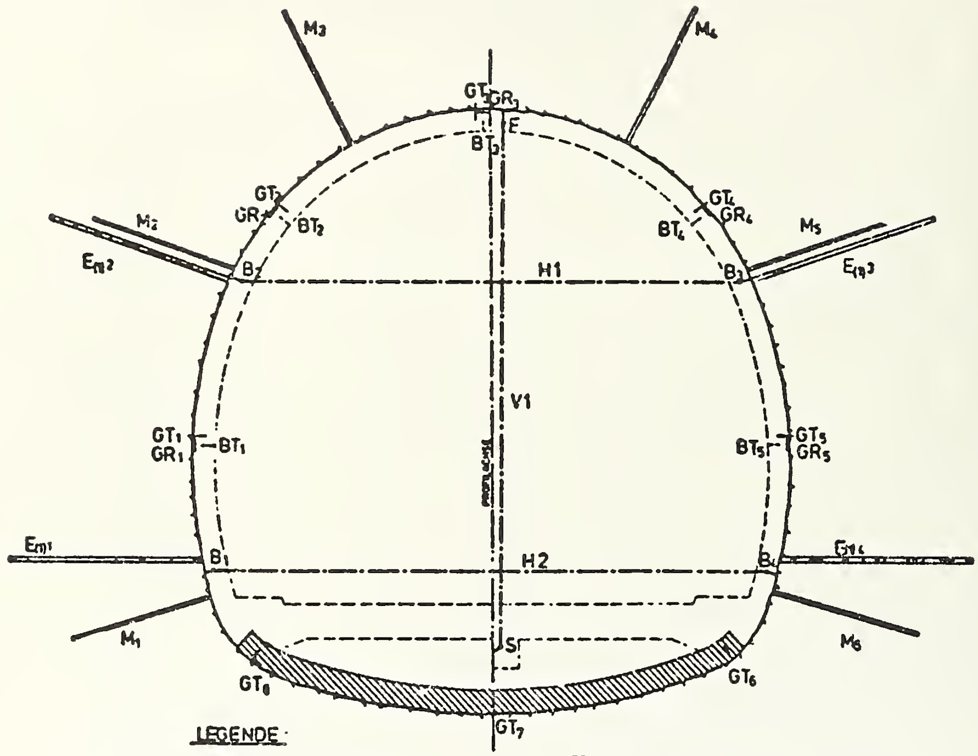
	GROUND TYPE	WATER	NO. OF SHEARS	NO. OF CASES	FIG.	CONVERGENCE (mm) MEAN	MIN	MAX	STANDARD DEVIATION	NORMALIZED EQUIVALENT BOLT PRESSURE MEAN	MIN	MAX	STANDARD DEVIATION
ARLBERG TUNNEL	Mica Schist with Garnet	ALL		133	E-2	282	79	690	120	0.029	0.011	0.090	0.016
		Heavily dripping	-	42	E-3	278	130	503	90	0.036	0.015	0.090	0.019
			≥2	20	E-4	316	156	503	88	0.035	0.018	0.085	0.018
			≤1	22	E-5	244	130	402	81	0.038	0.015	0.090	0.020
		Lightly dripping	ALL	77	E-6	257	79	558	107	0.027	0.014	0.077	0.012
			≥2	53	E-7	266	79	440	91	0.027	0.014	0.075	0.012
			≤1	24	E-8	238	97	558	135	0.026	0.015	0.077	0.013
		Humid	ALL ≤1	14	E-9	425	115	690	176	0.017	0.011	0.044	0.009
	Feldspathic Gneisses	ALL		39	E-10	449	162	674	141	0.017	0.012	0.032	0.005
		Heavily dripping	ALL	13	E-11	374	162	530	112	0.020	0.013	0.032	0.007
			≥2	9	E-12	384	206	518	98	0.020	0.013	0.032	0.007
			≤1	4	E-13	352	162	530	154	0.020	0.015	0.031	0.008
		Lightly dripping	ALL ≥2	10	E-14	355	184	519	124	0.017	0.013	0.020	0.002
		Humid	ALL	16	E-15	570	425	674	74	0.015	0.012	0.020	0.003
			≥2	6	E-16	572	425	655	106	0.016	0.013	0.020	0.003
			≤1	10	E-17	569	488	674	60	0.014	0.012	0.018	0.002
TAUERN TUNNEL	Serpentine			3	E-18	110	59	162	51.5	0.014	0.010	0.021	0.006
	Anhydrite (Massive)			6	E-19	9	5	16	3.9	0.006	0.002	0.016	0.005
	Phyllites: Calcitic			16	E-20	30	0	143.0	37	0.009	0.0	0.019	0.009
	Calcitic Chloritic			21	E-21	20	3	51	13	0.010	0.005	0.016	0.003
	Calcitic Graphitic			23	E-22	32	2	107	29.8	0.010	0.0	0.016	0.006
	Graphitic			22	E-23	76	2	277	57.6	0.012	0.0	0.018	0.003
	Quartzitic			14	E-24	84	9.5	228	79.5	0.015	0.003	0.029	0.007

TABLE E-2. STUDY SECTION WITH SMALL OVERBURDEN IN TAUERN TUNNEL

Ground Type	Cases	Overburden (meter)	Range of Convergence (mm)	Support Shotcrete
Phyllite: Calcitic	4	240 - 340	1.0 - 12.0	10cm
Phyllites: Graphitic Calcitic	2	140 - 210	2.0 - 5.0	10cm
Phyllites: Graphitic	1	70	2.0	10cm

TABLE E-3. RANGES OF OVERBURDEN

Ground Type		Water Condition	Range of overburden (meters) MIN	MAX	Remarks
Mica Schist with garnet		All	100	570	
		dripping heavily	100	560	
		dripping lightly	105	550	
		humid	565	570	
Feldspathic gneiss		All	560	595	
		dripping heavily	560	595	
		dripping lightly	565	595	
		humid	560	583	
Serpentine			870	872	
Anhydrite			590	609	
Phyllites:	Calcitic		413	654	5 cases with less than 340 m (Table E.2)
	Chloritic Calcitic		586	779	
	Calcitic Graphitic		427	743	2 cases with less than 210 m (Table E.2)
	Graphitic		740	980	One case with 70 m over-burden (Table E.2)
	Quartzitic		350	888	



LEGENDE:

B, F, S } Measurement Bolts

H. } Convergence
V. } Base Difference

E.. Extensometer
M ... Bolt equipped with strain gauges (force monitoring)

GR (radial), GT (tangential), BT (tangential) } Stress Cells

H 1 = BASE TO MONITOR TOP CONVERGENCE

H 2 = BASE TO MONITOR LOWER CONVERGENCE

F = POINT TO MONITOR CROWN SETTLEMENT

FIGURE E-1a MONITORING SECTION

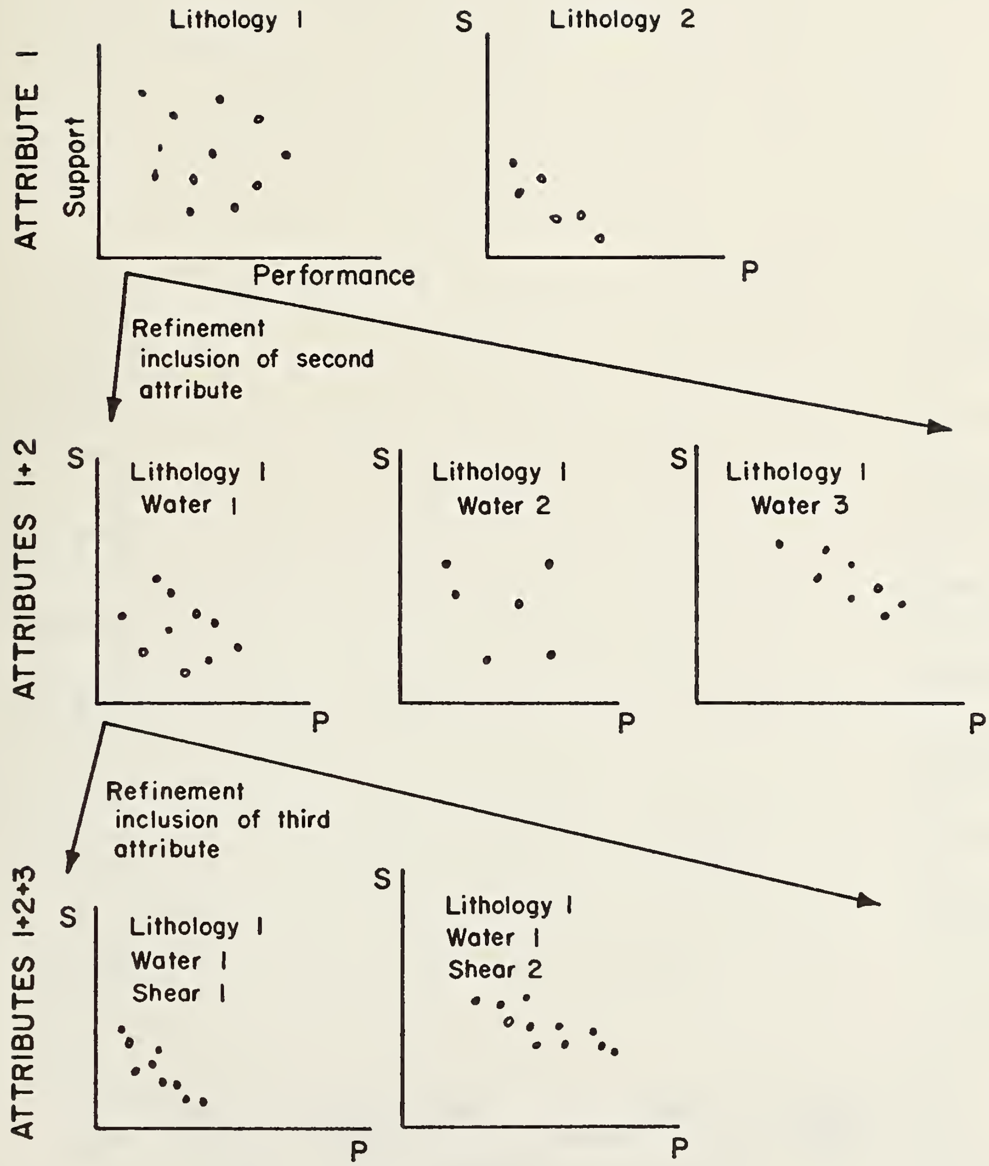


FIGURE E-1b PRINCIPLE OF GROUND ATTRIBUTE REFINEMENT

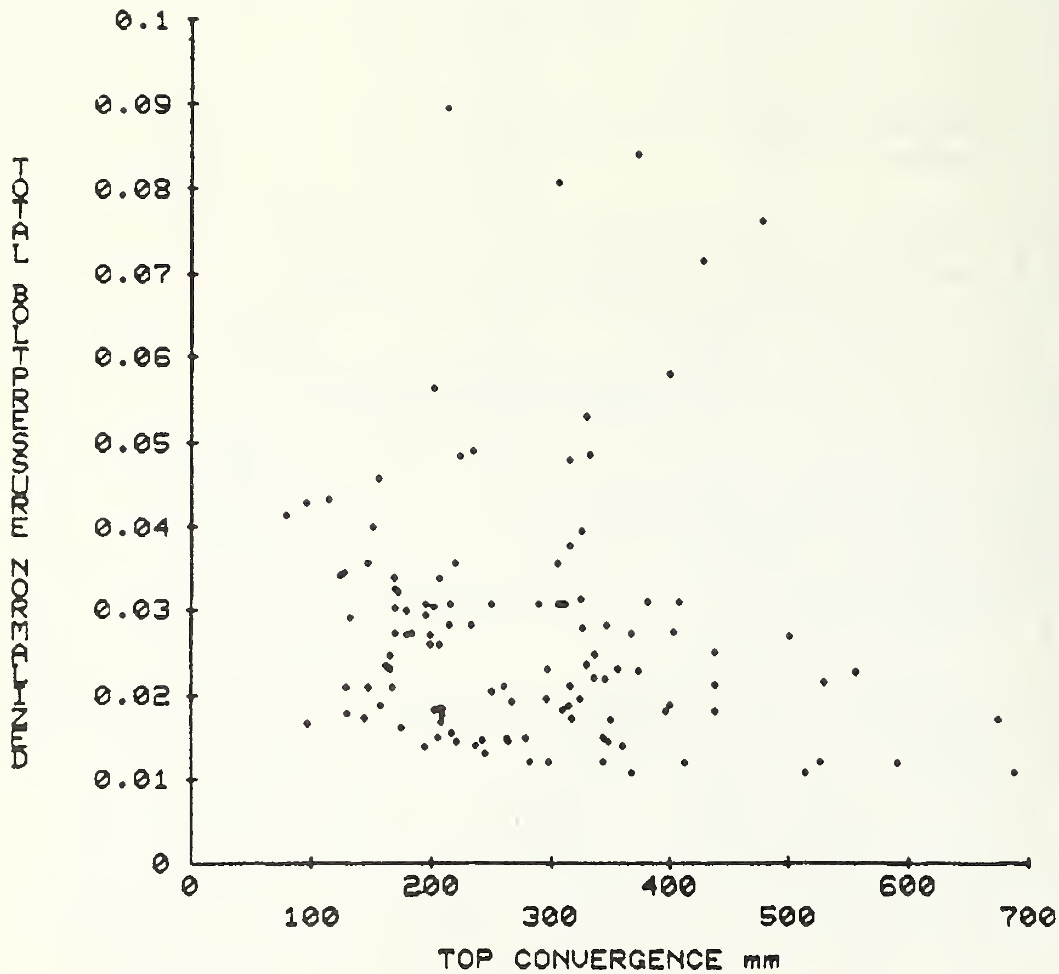


FIGURE E-2 ALL CASES WITH GARNETIC MICA-SCHIST

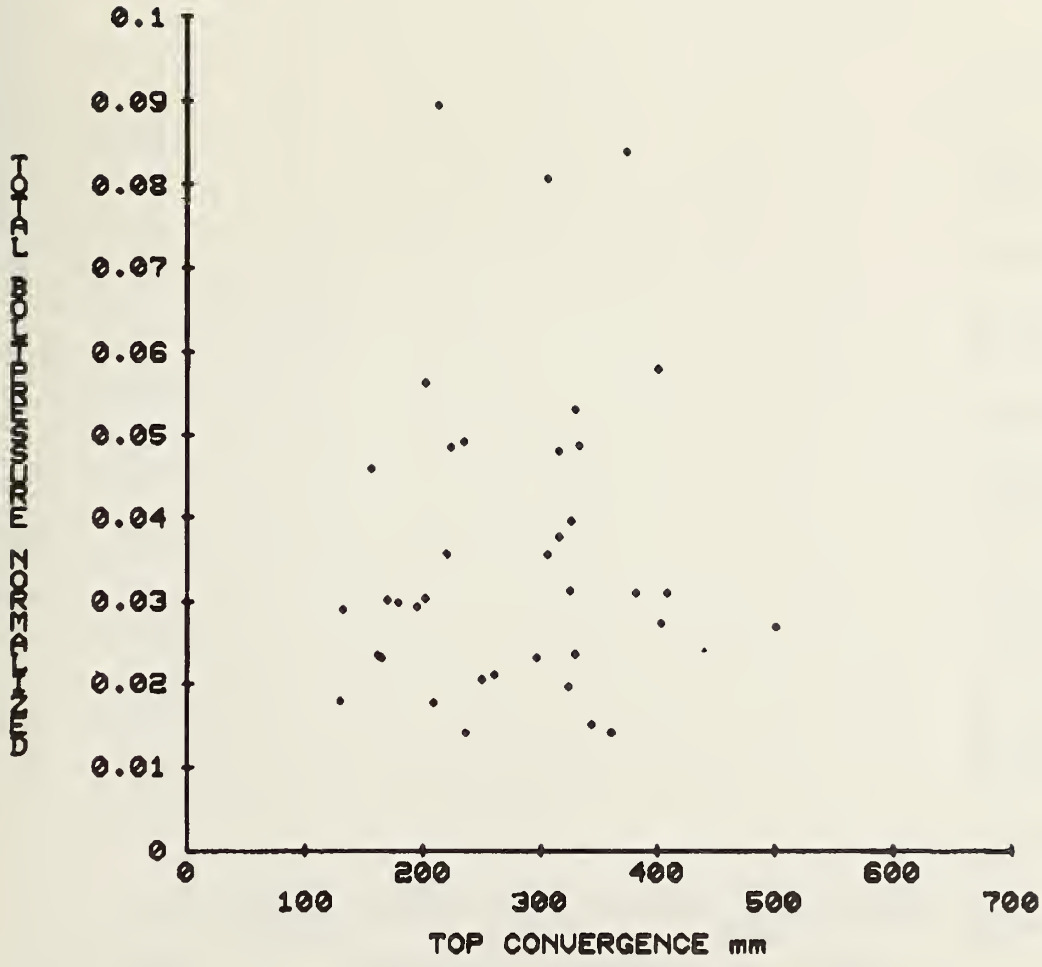


FIGURE E-3 GARNETIC MICA SCHISTS AND WATER DRIPPING HEAVILY

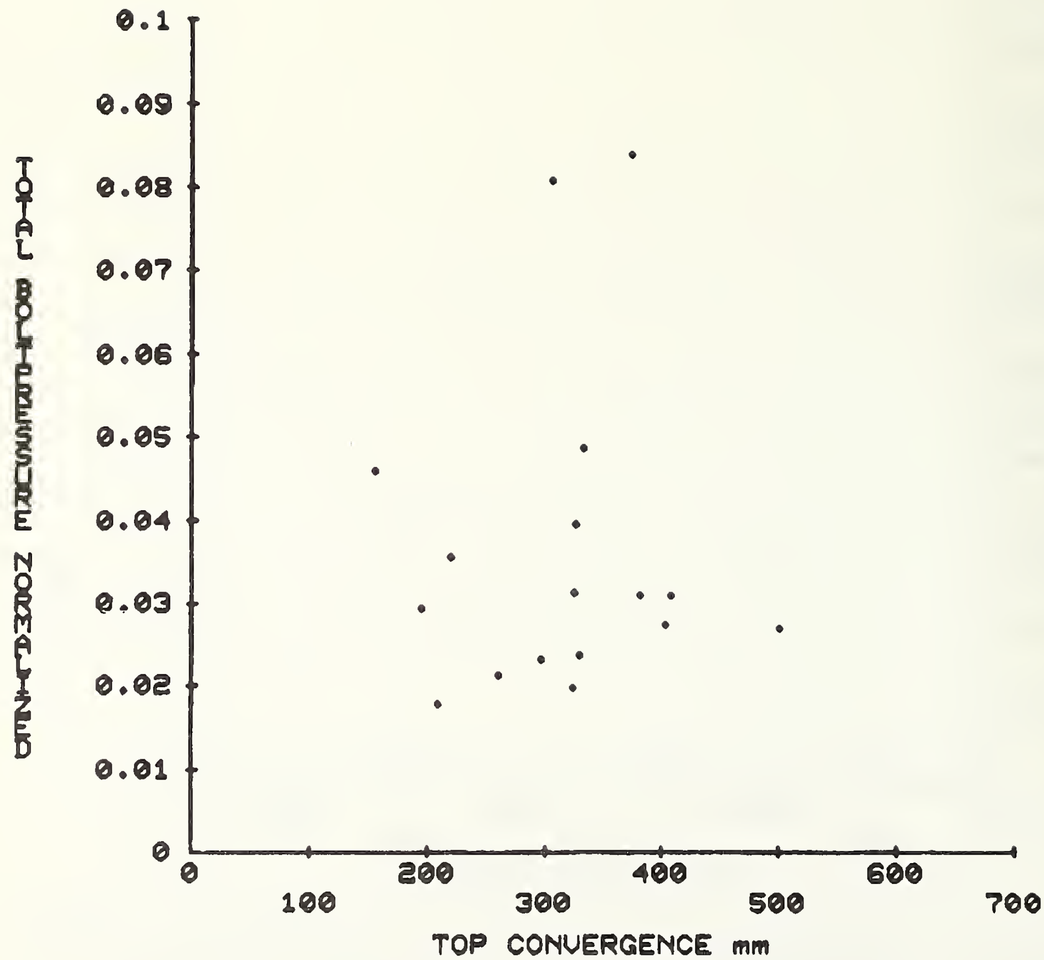


FIGURE E-4 GARNETIC MICA SCHISTS AND WATER DRIPPING HEAVILY AND TWO OR MORE SHEARZONES

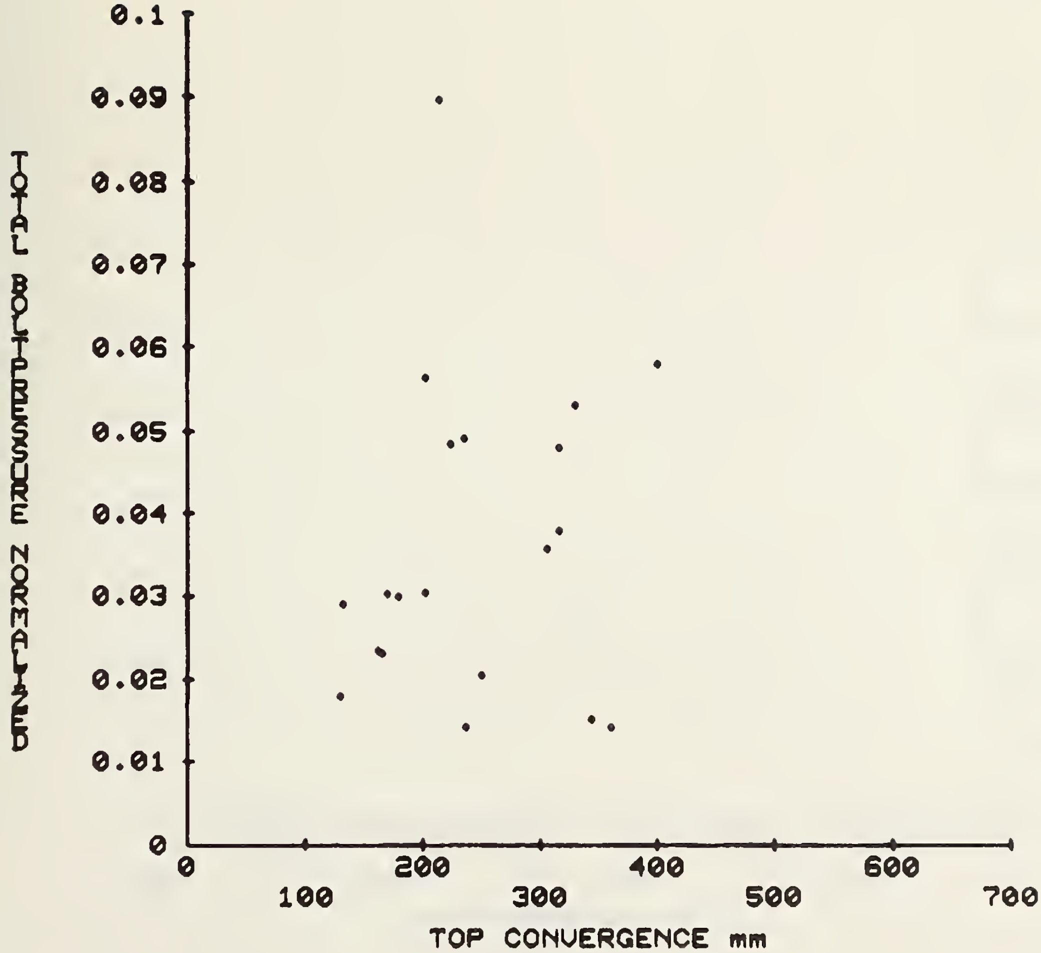


FIGURE E-5 GARNETIC MICA SCHIST AND WATER DRIPPING
HEAVILY AND ONE OR NO SHEARZONE

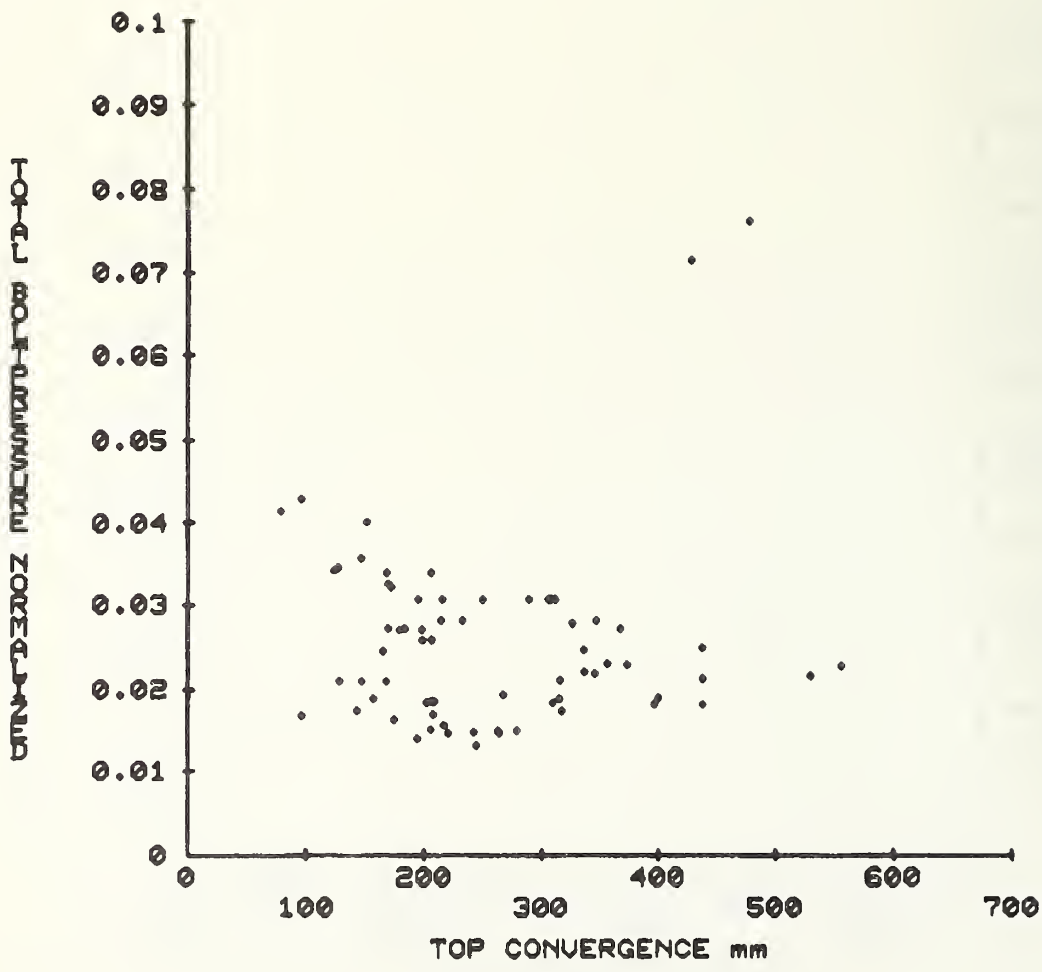


FIGURE E-6 MICA SCHIST WITH GARNET AND WATER DRIPPING LIGHTLY

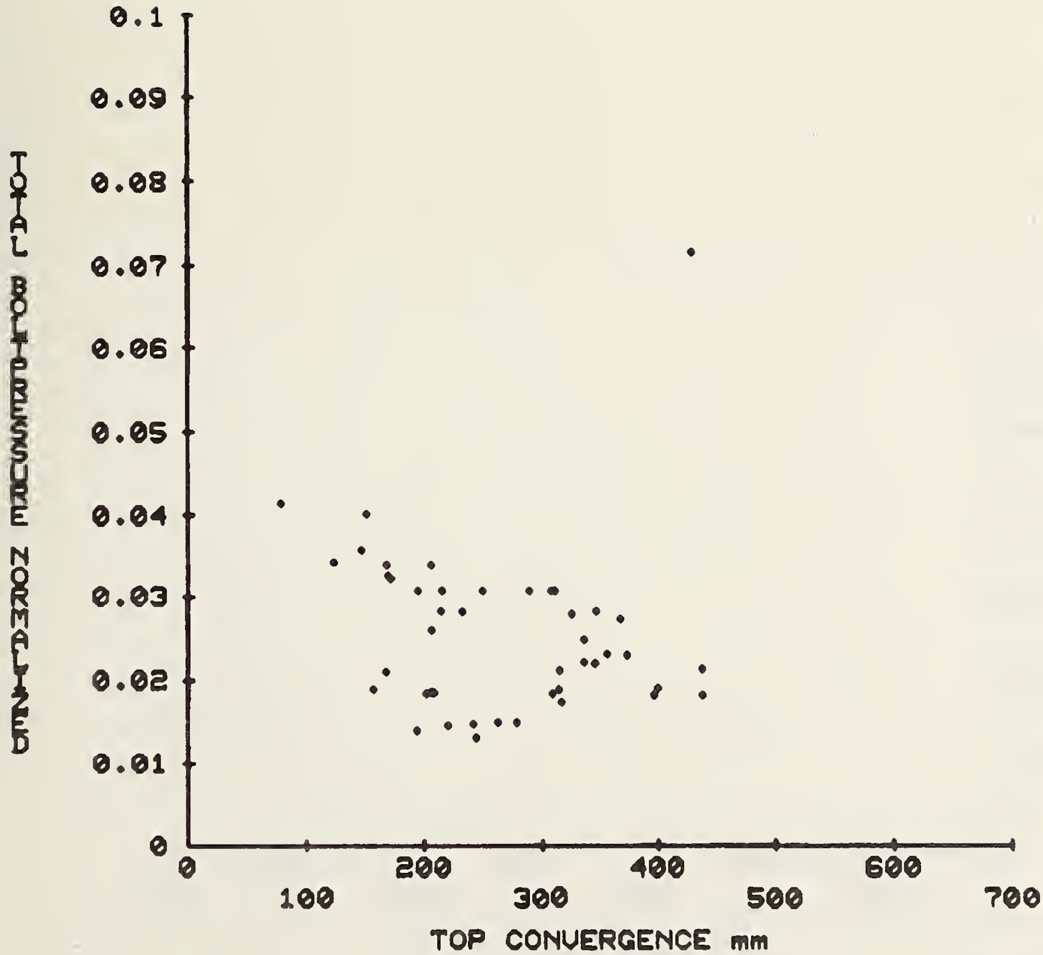


FIGURE E-7 MICA SCHIST WITH GARNET AND WATER DRIPPING LIGHTLY AND TWO OR MORE SHEAR ZONES

R

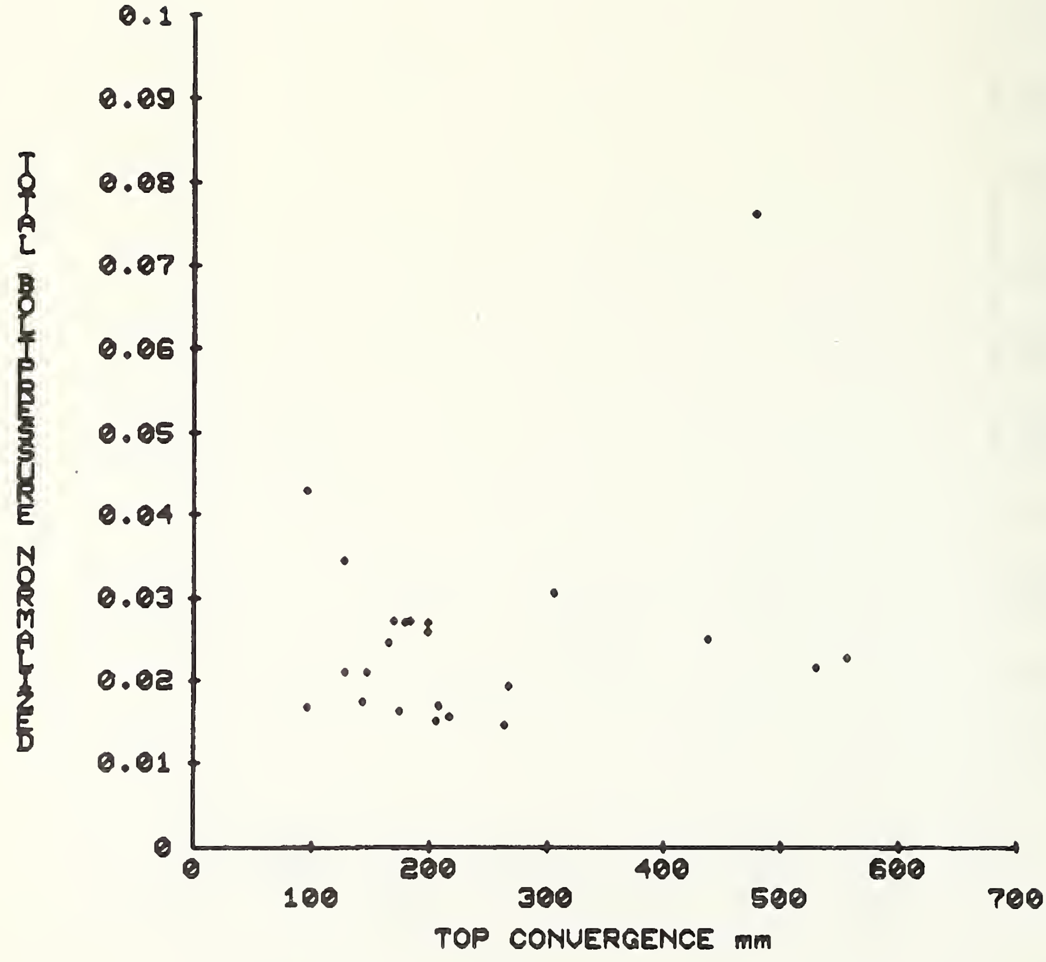


FIGURE E-8 MICA SCHIST WITH GARNET AND WATER DRIPPING LIGHTLY AND ONE OR NO SHEARZONE

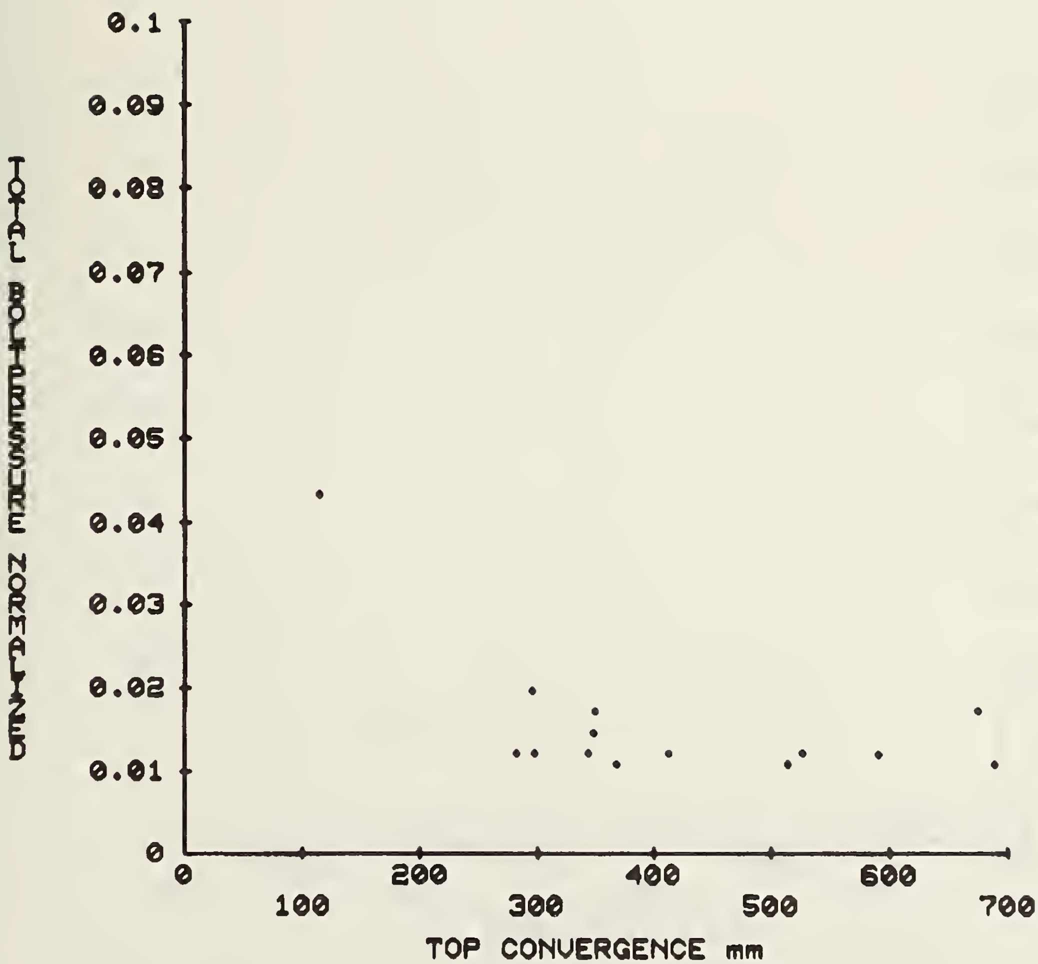


FIGURE E-9 MICA SCHIST WITH GARNET AND HUMID WATER CONDITIONS

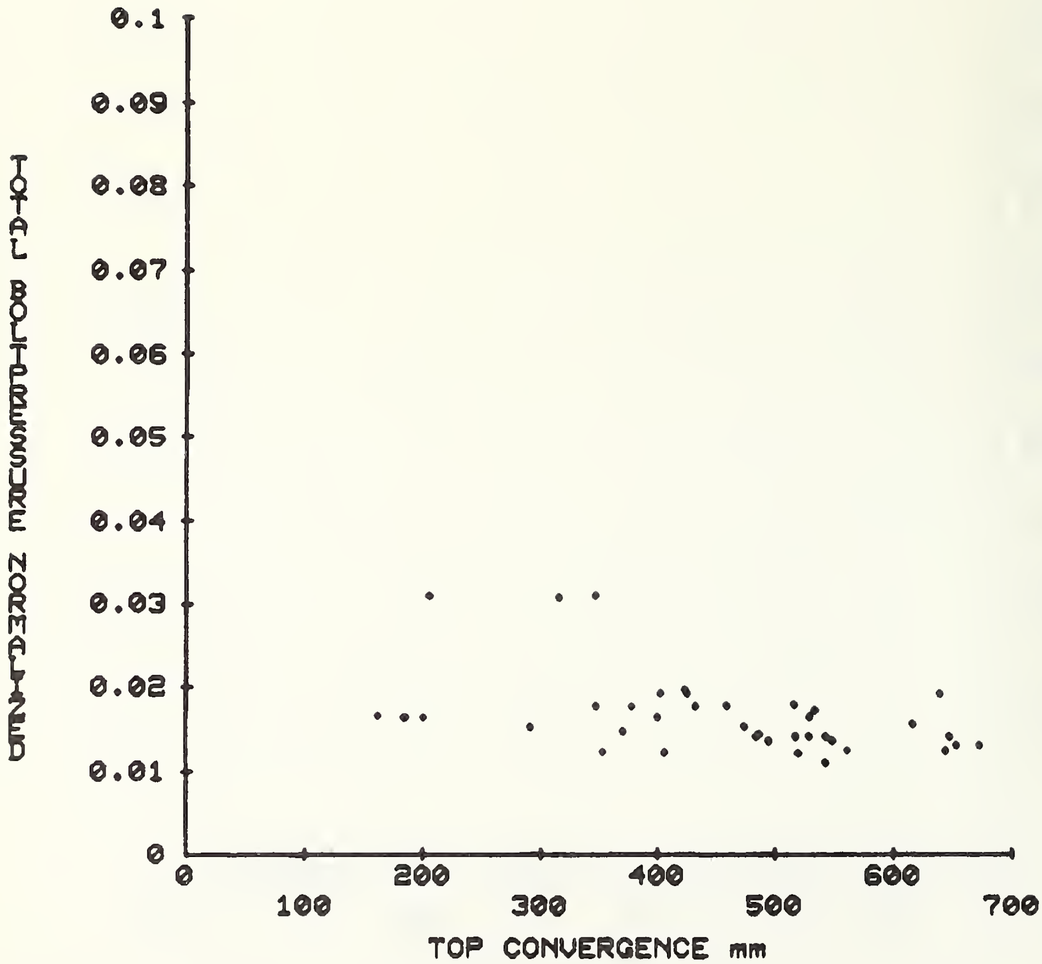


FIGURE E-10 ALL CASES WITH FELDSPATHIC GNEISS

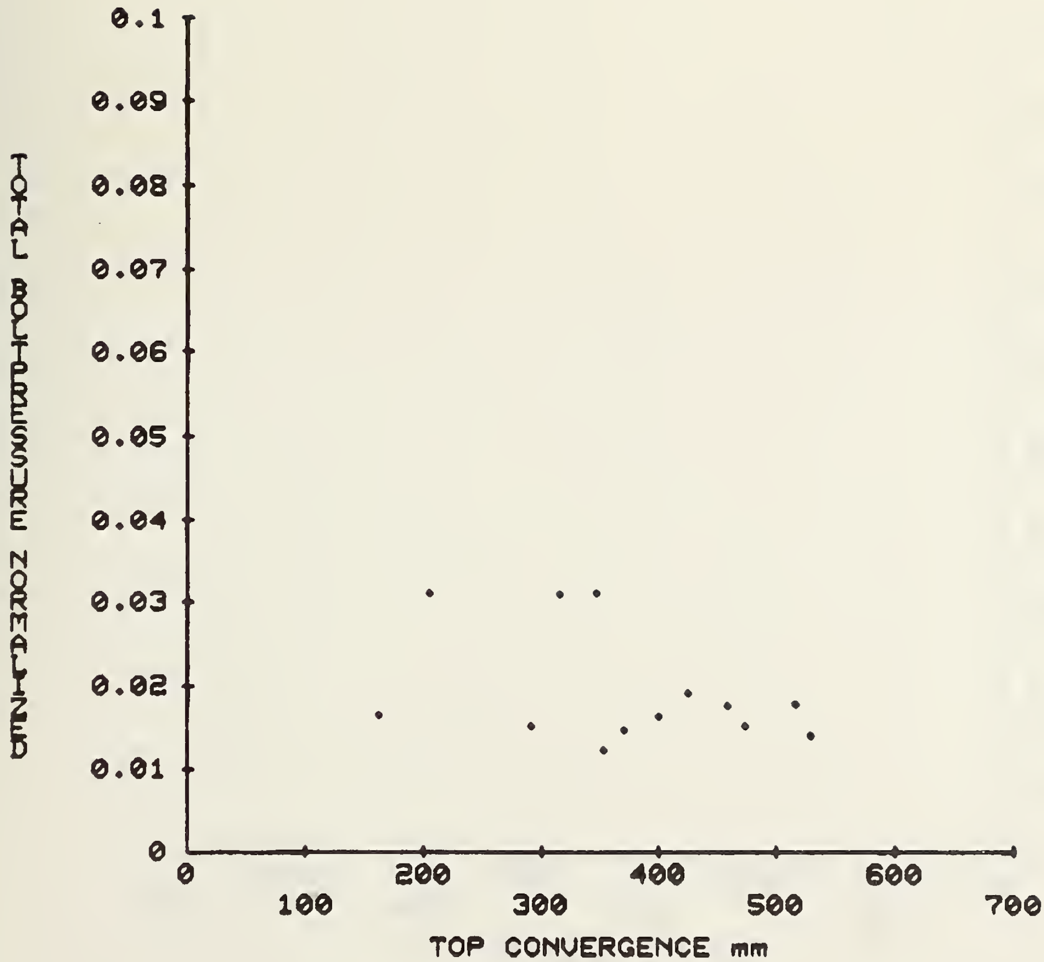


FIGURE E-11 FELDSPATHIC GNEISSES AND WATER DRIPPING HEAVILY

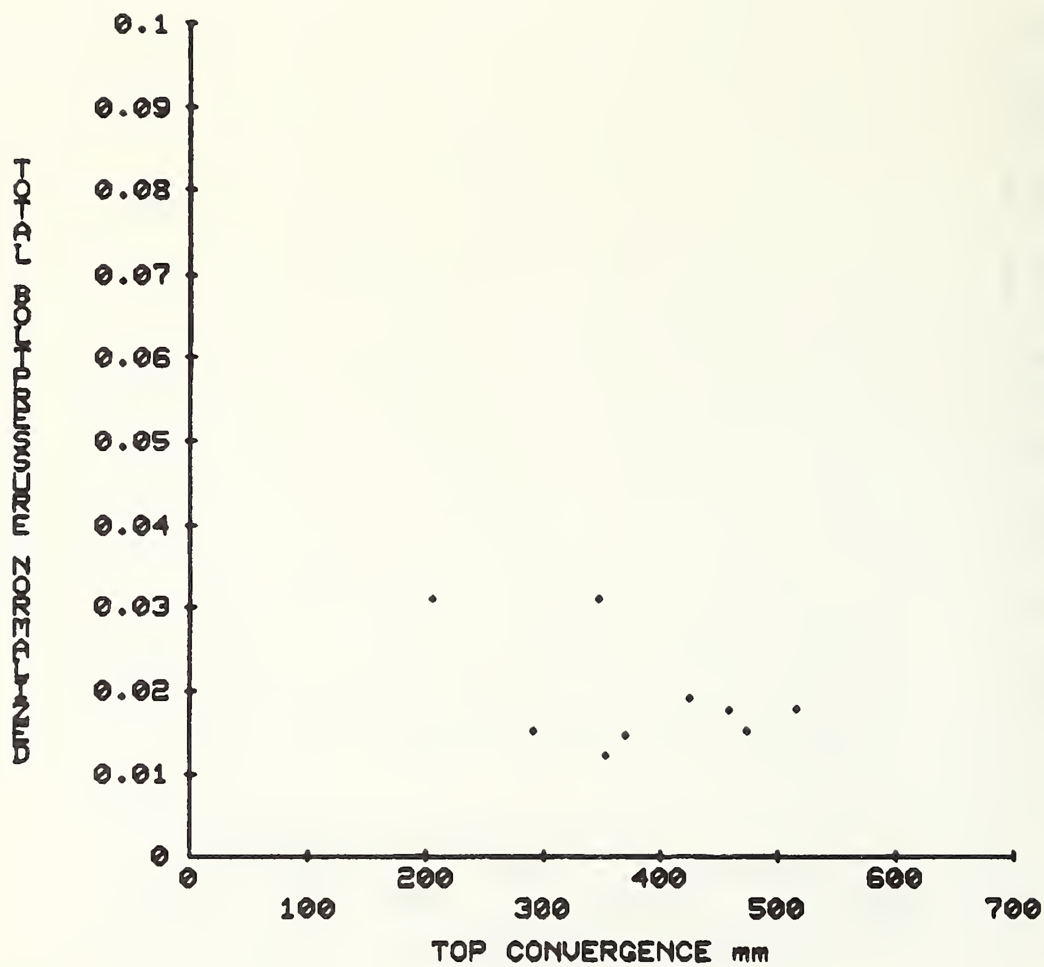


FIGURE E-12 FELDSPATHIC GNEISSES AND WATER DRIPPING HEAVILY AND TWO OR MORE SHEAR ZONES

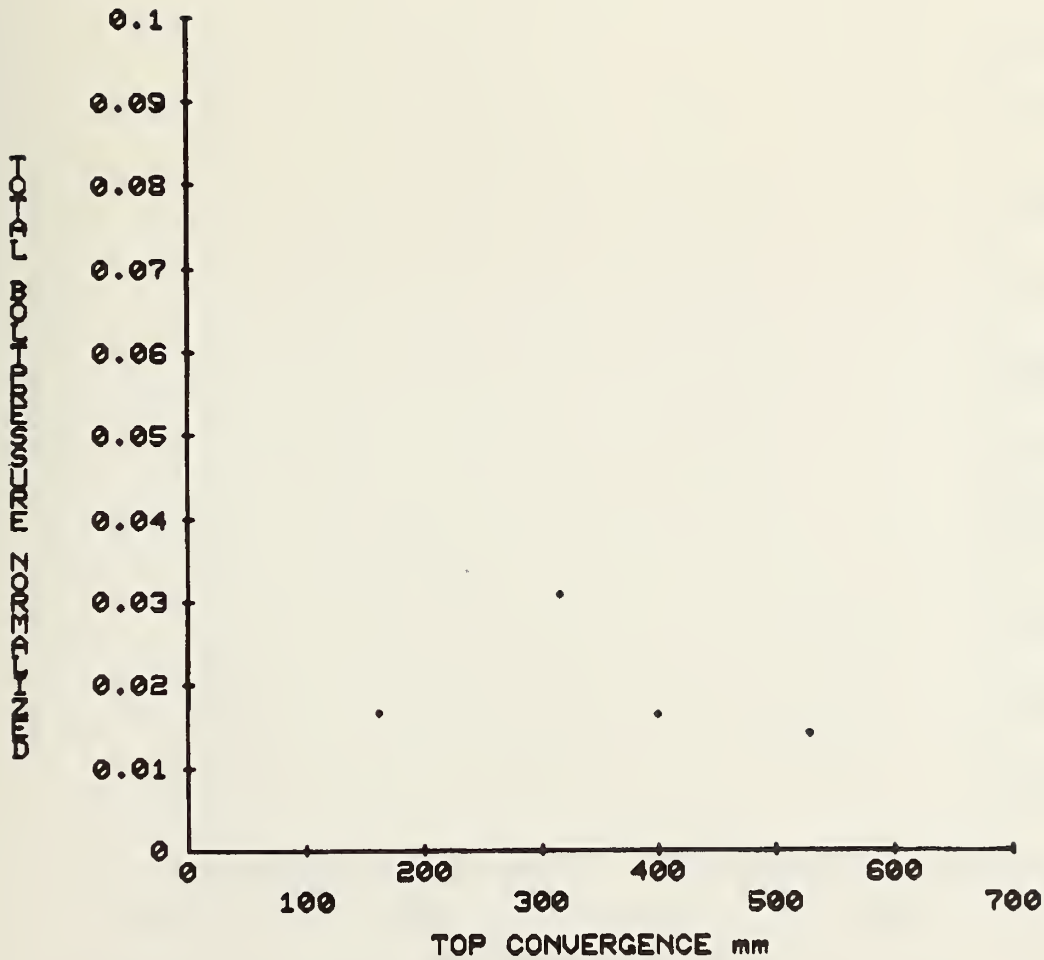


FIGURE E-13 FELDSPATHIC GNEISSES AND WATER DRIPPING HEAVILY AND ONE OR NO SHEARZONE

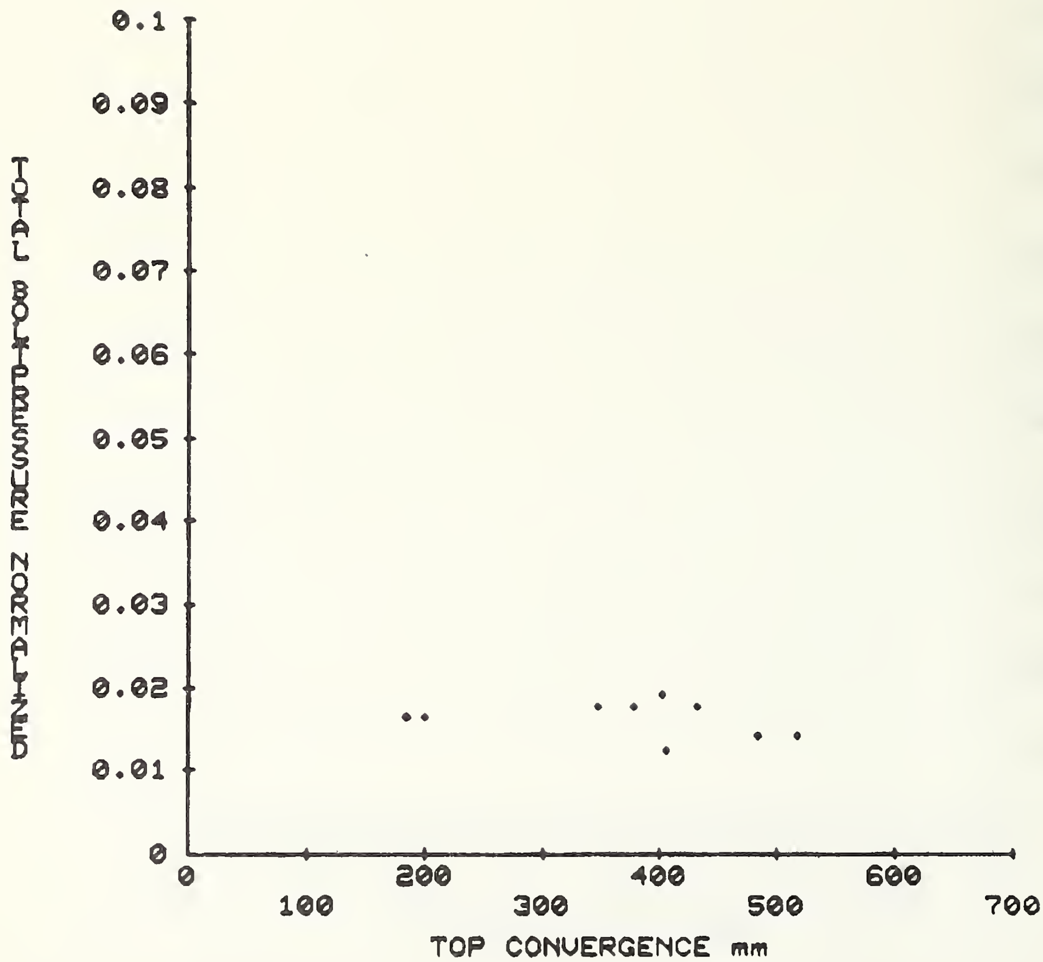


FIGURE E-14 FELDSPATHIC GNEISS AND WATER DRIPPING LIGHTLY

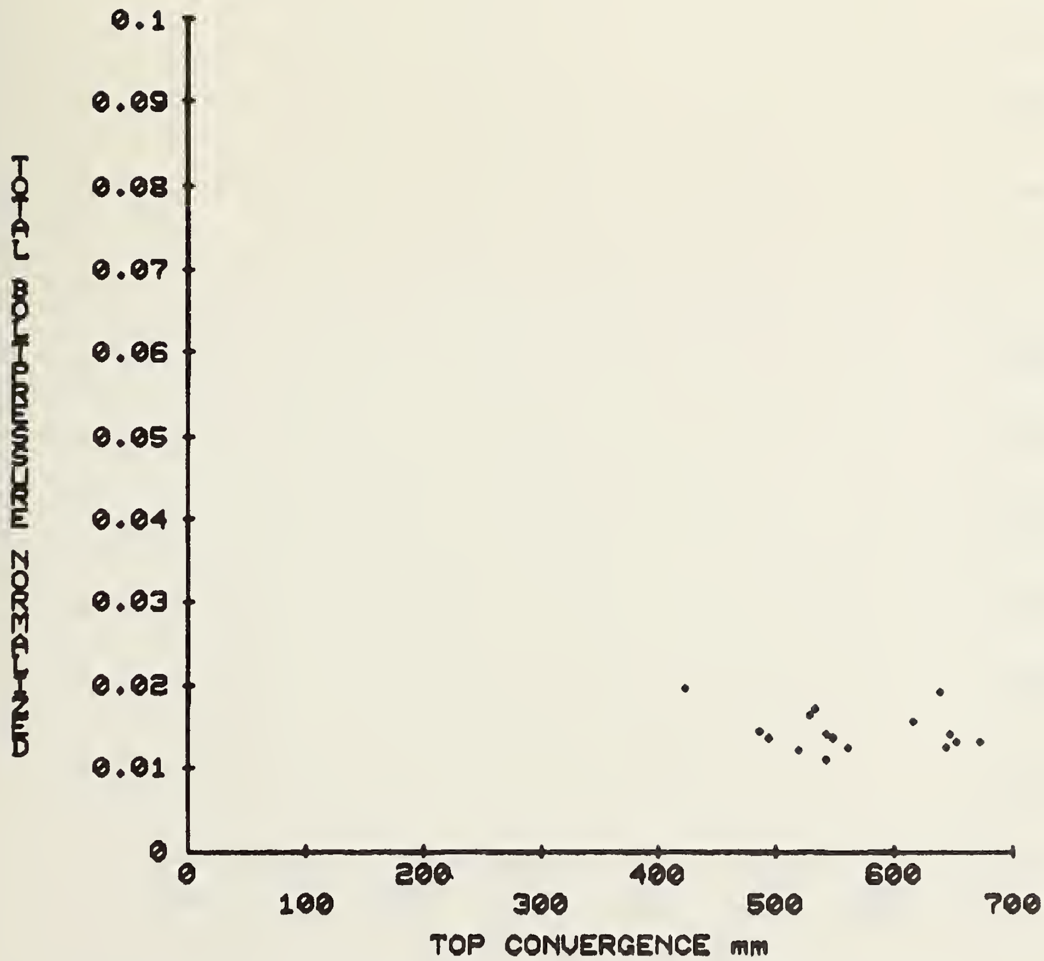


FIGURE E-15 FELDSPATHIC GNEISS AND HUMID WATER CONDITIONS

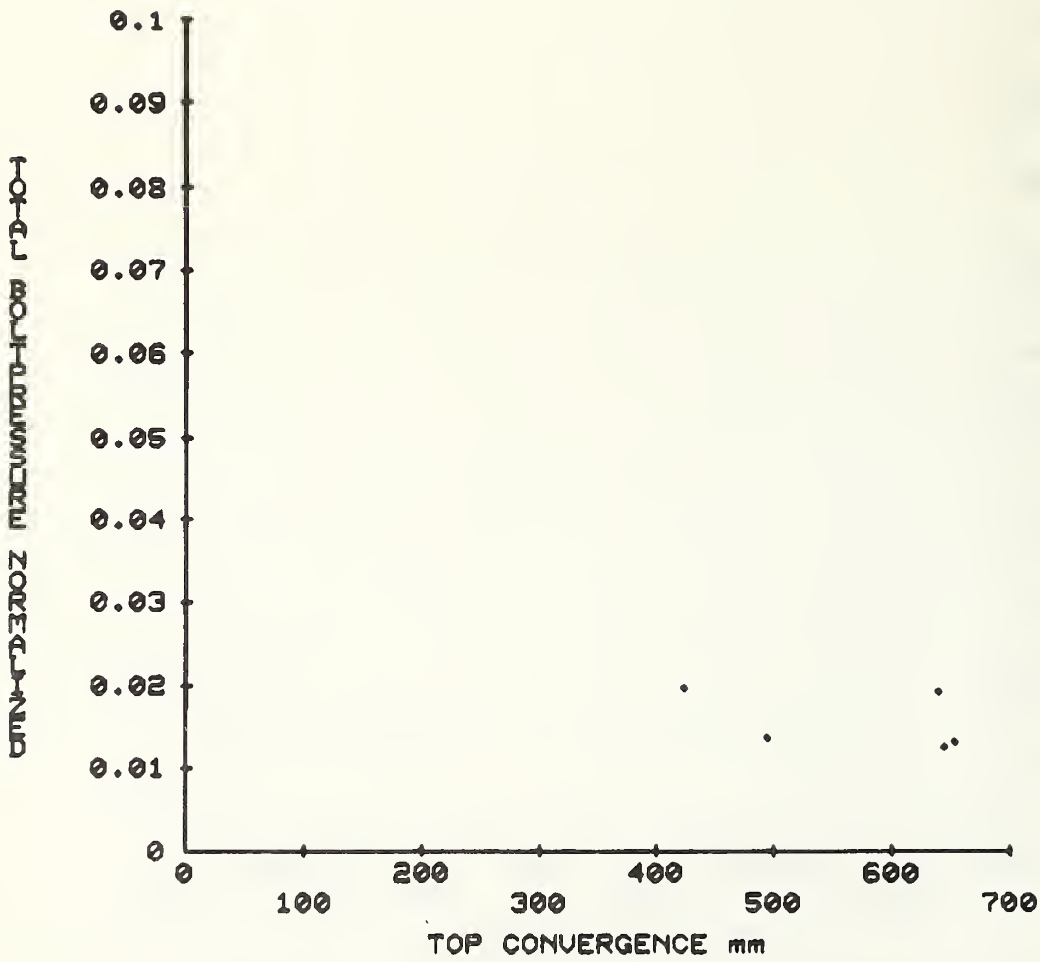


FIGURE E-16 FELDSPATHIC GNEISSES AND HUMID WATER CONDITIONS AND TWO OR MORE SHEARZONES

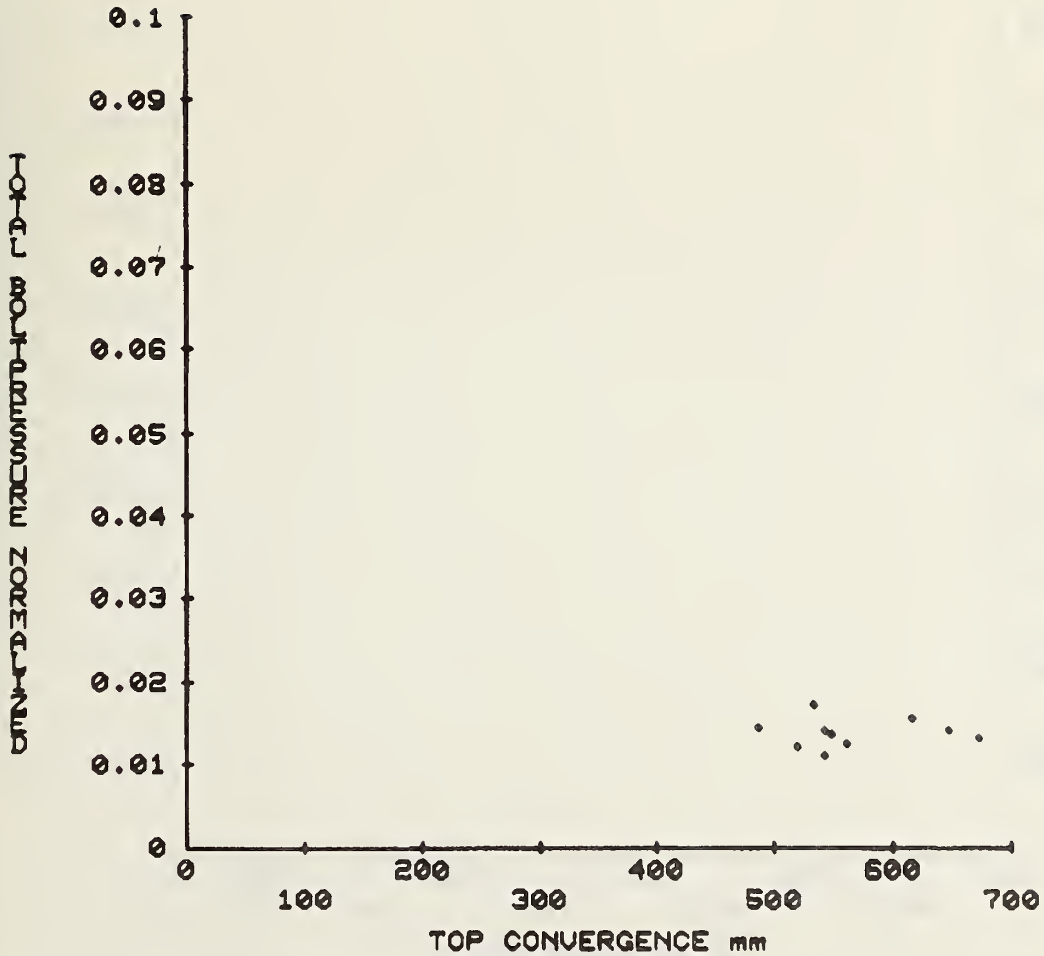


FIGURE E-17 FELDSPATHIC GNEISS AND HUMID WATER CONDITIONS AND ONE OR NO SHEAR ZONE

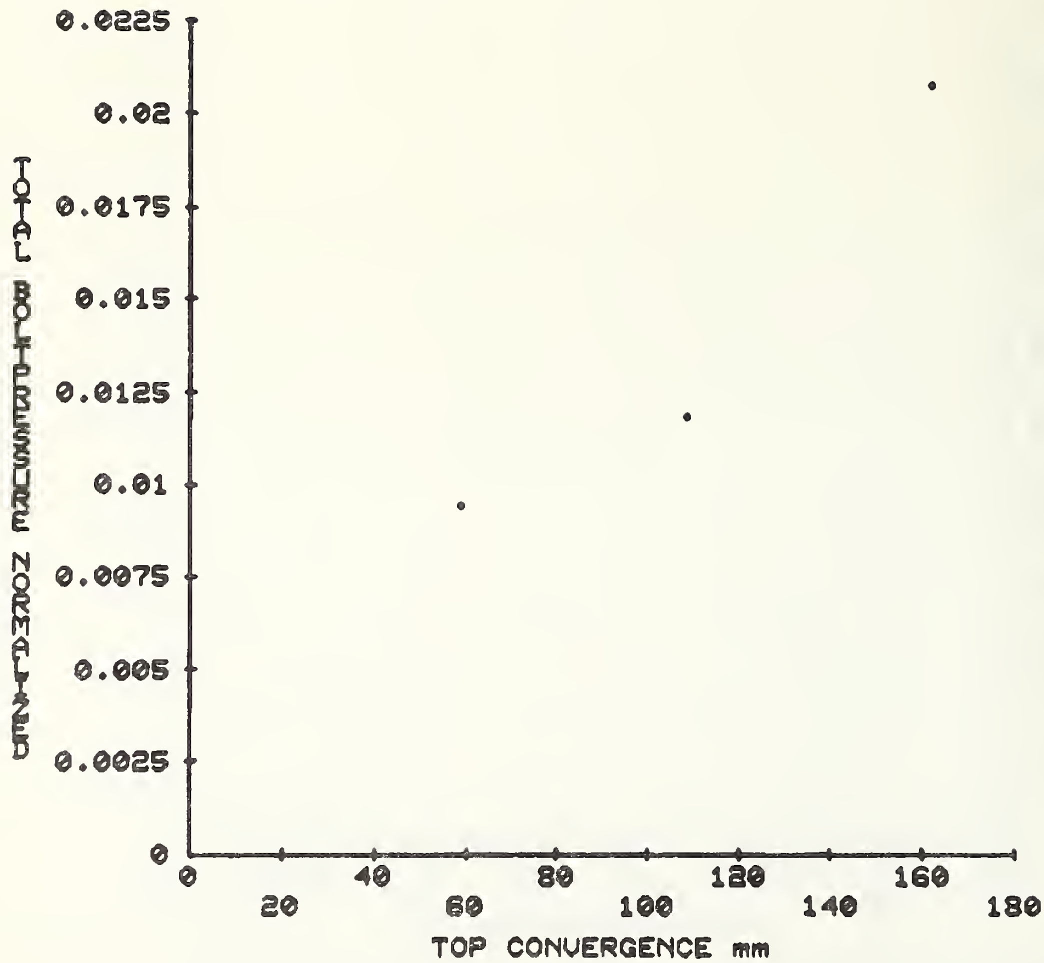


FIGURE E-18 ALL SERPENTINE CASES

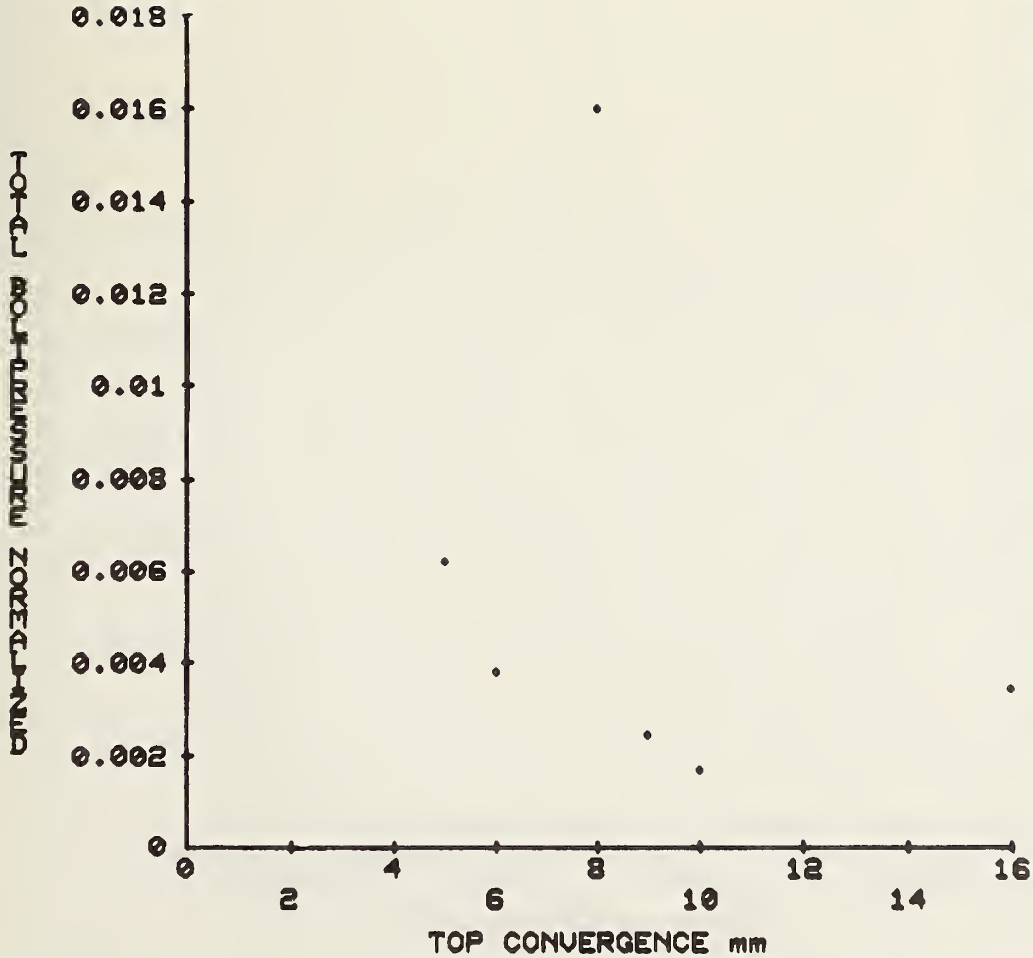


FIGURE E-19 ANHYDRITE CASES FROM TAUERN TUNNEL

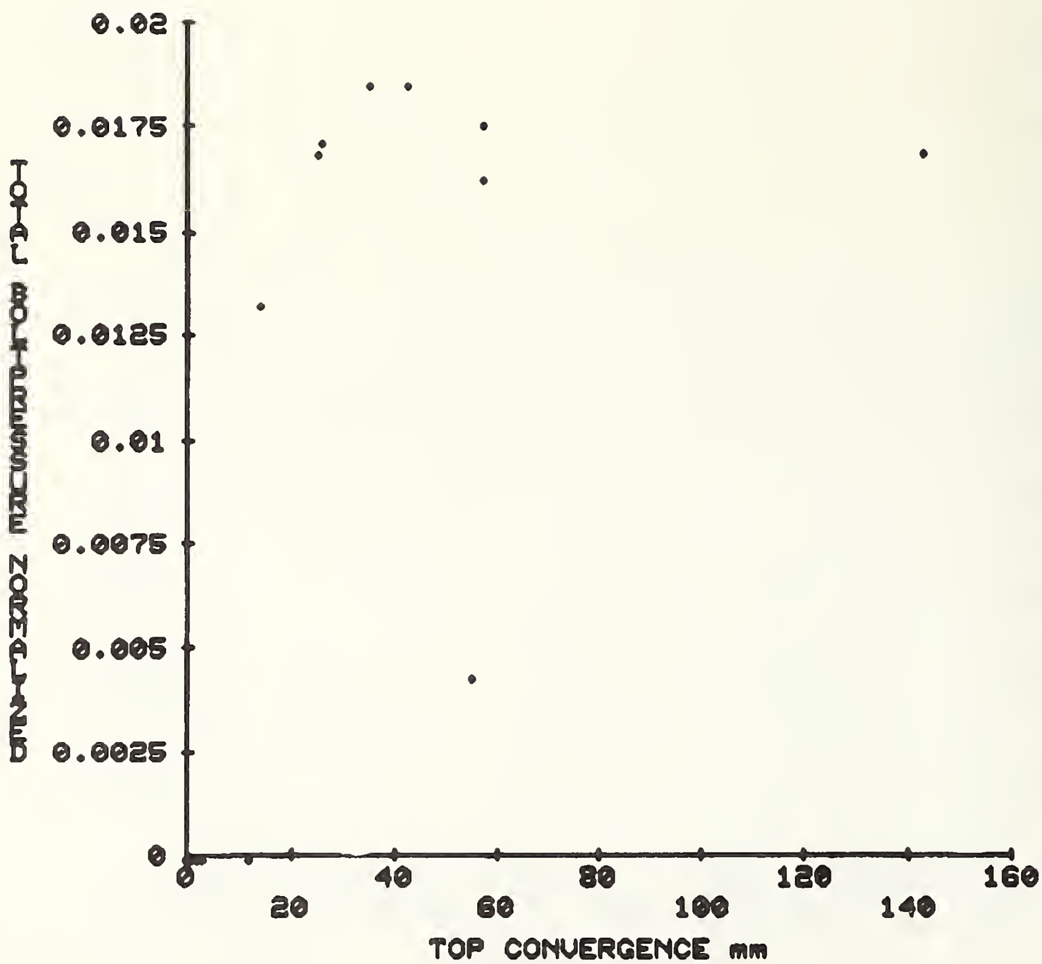


FIGURE E-20 PHYLLITES: CALCITIC (FROM TAUERN TUNNEL)

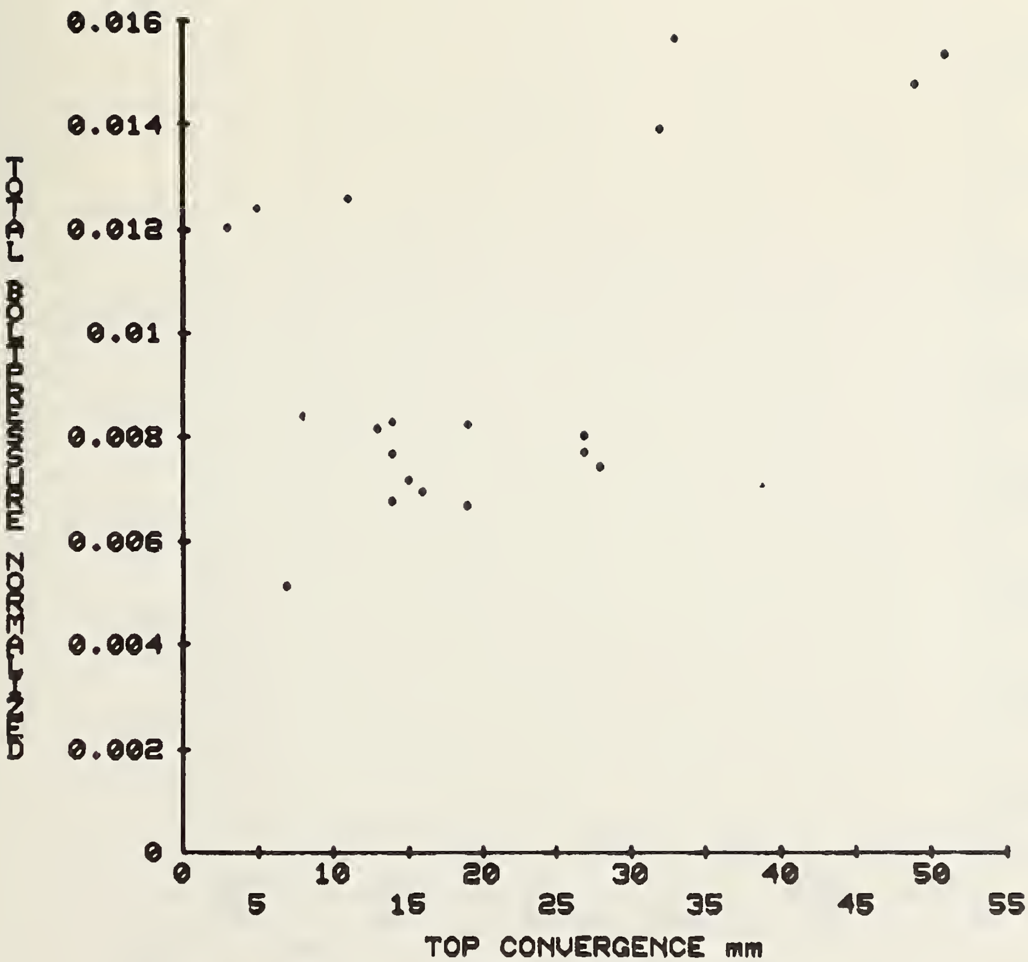


FIGURE E-21 PHYLLITES:CALCITIC-CHLORITIC
(FROM TAUERNTUNNEL)

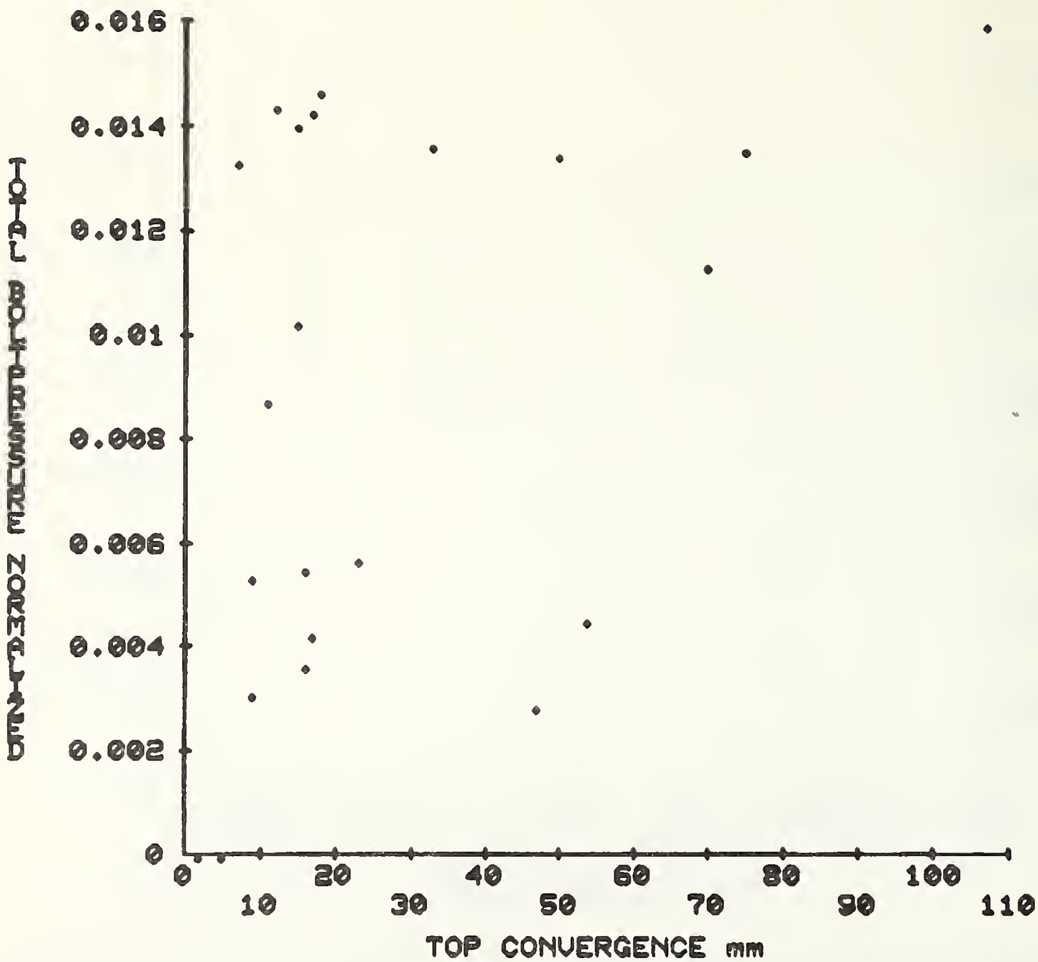


FIGURE E-22 PHYLLITES: CALCITIC-GRAPHITIC
(FROM TAUERN TUNNEL)

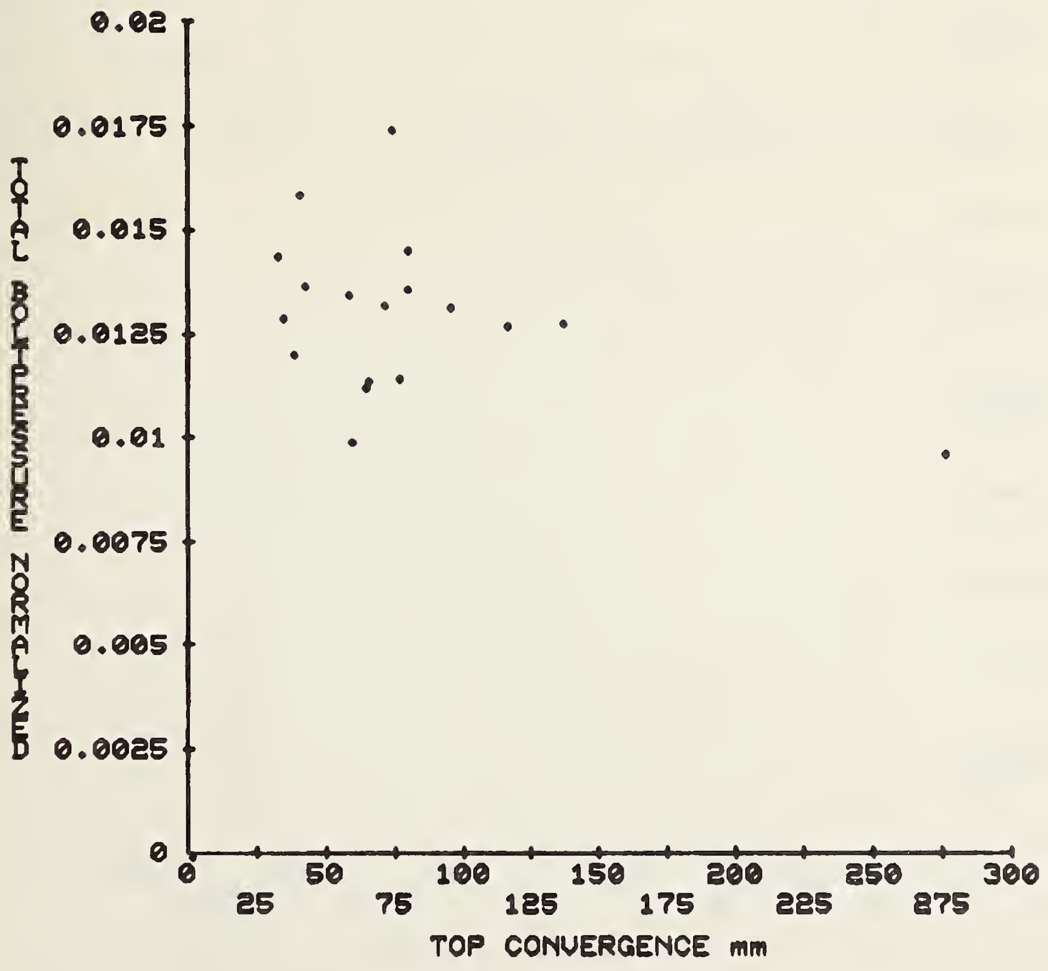


FIGURE E-23 PHYLLITES: GRAPHITIC (TAUERN TUNNEL)

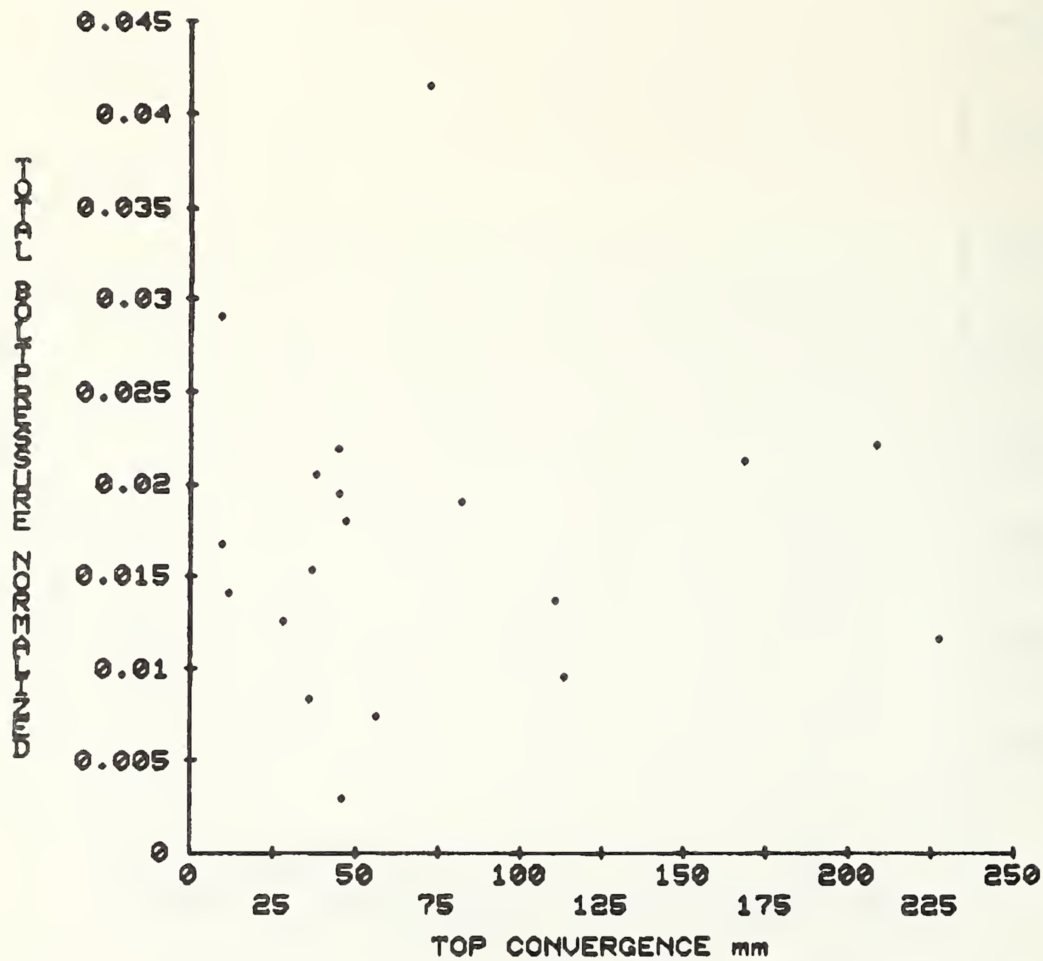


FIGURE E-24 PHYLLITES: QUARTZITIC (TAUERN TUNNEL)

APPENDIX F

REPORT OF NEW TECHNOLOGY

The work performed under this contract has led to the development of improved practical design tools to provide more accurate representations of the ground-structure interaction in tunneling. Provided in this volume, for the first time, is a review of available empirical design methods and guidelines best-suited for observational (adaptable) tunneling procedures.

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