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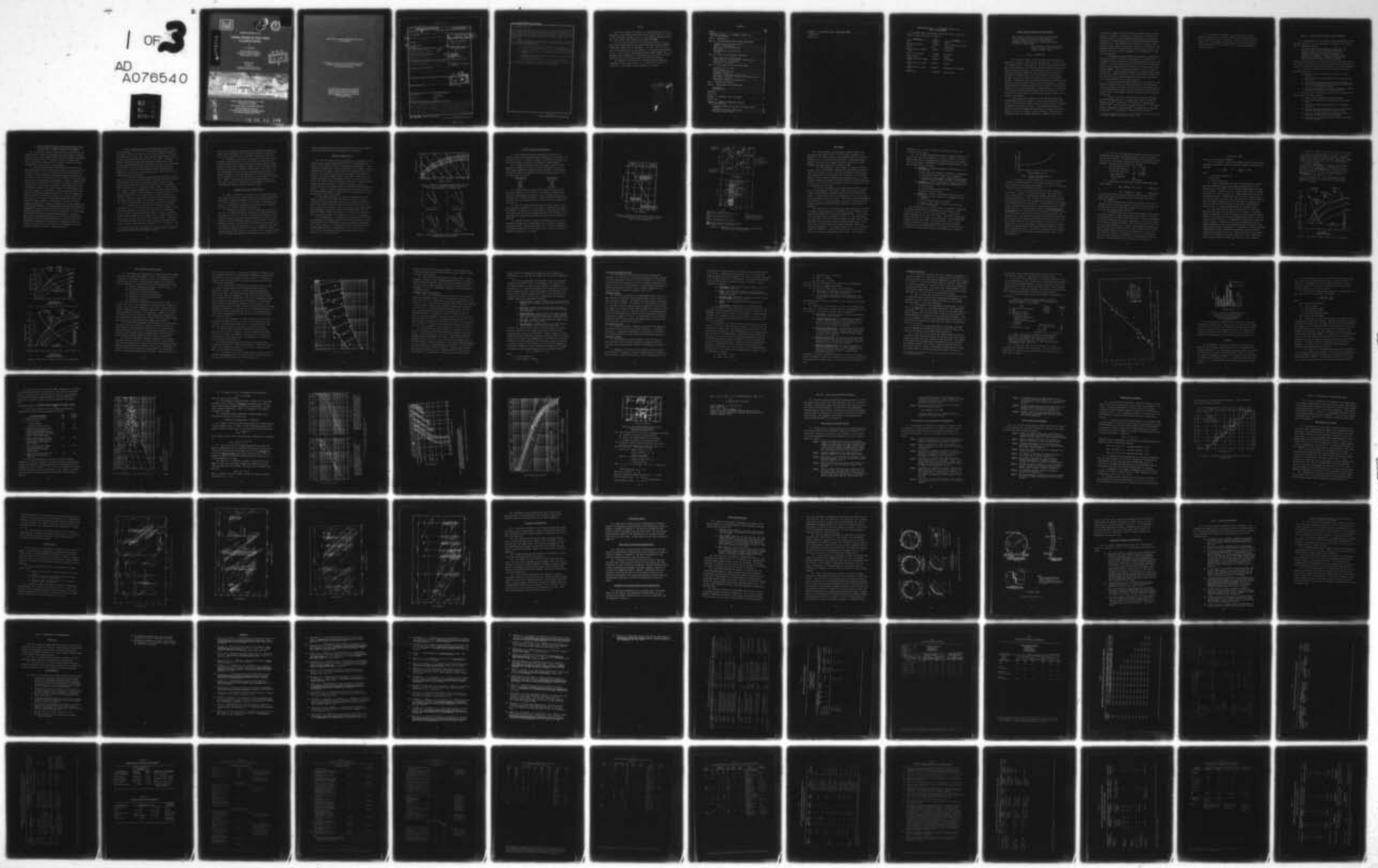

TECHNICAL REPORT GL-79-19

TUNNEL DESIGN BY ROCK MASS CLASSIFICATIONS

by

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report discusses tunnel design procedures based on various rock mass classification systems. A comparison is made between the tunnel support design based on the classical Terzaghi rock load method and the support selection based on the RSR Concept, the Geomechanics Classification, and the Q-System. These classification systems are described in detail and guidelines are given for step-by-step application of the three methods. Using an actual tunnel case history, an evaluation is made of the current design practice by comparing it

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20. ABSTRACT (Continued).

with the design approaches involving the three rock mass classification systems. It is concluded that the current design practice may lead to overdesign of support, and recommendations are made for improved procedures that would ensure the construction of safe and more economical rock tunnels. Finally, a few areas are identified where more research would benefit the current tunnel design practice.

In order to accomplish the main purpose of this report, namely to evaluate tunnel design practices with respect to rock mass classification systems, the following scope of work was defined:

a. Review existing classification systems in rock engineering.

b. Provide a user's guide for the most useful classification systems.

c. Evaluate design practices on the basis of a selected tunnel case history.

d. Identify practical steps leading to improved design of safe and more economical tunnels.

e. Recommend research requirements needing immediate attention.

The above scope of work was accomplished during this study, and the procedures, results, and discussions are presented in this report.

PREFACE

This report contains the results of an investigation by Prof Z. T. Bieniawski of Pennsylvania State University, University Park, Pa. Funds for this study were provided by the U. S. Army Engineer Waterways Experiment Station (WES) under Purchase Order DACW39-78-M-3714.

The study was performed in FY 78 under the direction of Dr. D. C. Banks, Chief, Engineering Geology and Rock Mechanics Division (EG&RMD), Geotechnical Laboratory (GL), and Messrs. J. P. Sale and R. G. Ahlvin, Chief and Assistant Chief, respectively, GL. The contract was monitored by Mr. J. S. Huie, Chief, Rock Mechanics Branch (RMB), EG&RMD. Mr. G. A. Nicholson, RMB, assisted with the geological data collection and interpretation for the case history study of the Park River Tunnel.

The Commanders and Directors of the WES during this study and preparation of this report were COL J. L. Cannon, CE, and COL N. P. Conover, CE. The Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
gallons per minute	3.785412	cubic decimetres per minute
inches	25.4	millimetres
kips (force) per square foot	47.88026	kilopascals
miles (U. S. statute)	1.609344	kilometres
pounds (force)	4.448222	newtons
pounds (force) per square inch	6.894757	kilopascals
pounds (force) per square foot	47.88026	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square foot	0.09290304	square metres

TUNNEL DESIGN BY ROCK MASS CLASSIFICATIONS

"The origin of the science of classification goes back to the writings of the ancient Greeks; however, the process of classification — the recognition of similarities and the grouping of objects based thereon — dates to primitive man."

Prof. Robert R. Socal — Presidential Address to the U. S. Classification Society (Chicago, 1972).

PART I: INTRODUCTION

1. The design of tunnels in rock currently utilizes three main approaches: analytical, observational, and empirical. In view of the very complex nature of rock masses and the difficulties encountered with their characterization, the analytical approach is the least used in the present engineering practice. The reason for it does not lie in the analytical techniques themselves, since some have been developed to a high degree of sophistication, but in the inability to furnish the necessary input data as the ground conditions are rarely adequately explored. Consequently, such analytical techniques as the finite element method, the boundary element method, closed form mathematical solutions, photoelasticity or analogue simulation are only useful for assessing the influence of the various parameters or processes (but one at a time) and for comparing alternative design schemes; they are the methods of the future not as yet acceptable as the practical engineering means for the design of rock tunnels.

2. The observational approach, of which the New Austrian Tunneling Method is the best example, is based on observations and monitoring of tunnel behavior during construction and selecting or modifying the support as the project proceeds. This represents essentially a "build as you go" philosophy since the support is adjusted during construction to meet the changes in ground conditions. This approach is nevertheless based on a sound premise that a flexible tunnel lining,

utilizing the inherent ability of the rock to support itself, is preferable to a rigid one. In practice, a combination of rockbolts and shotcrete is used to prevent excessive loosening in the rock mass but allowing it to deform sufficiently to develop arching and self-support characteristics. The problem with this approach is, however, that it requires special contractual provisions: these may be suitable for the European practice for which they were evolved over many years of trial and error, but are not easily adaptable to the established U. S. contracting procedures.

3. The empirical approach relates the experience encountered at previous projects to the conditions anticipated at a proposed site. If an empirical design is backed by a systematic approach to ground classification, it can effectively utilize the valuable practical experience gained at many projects, which is so helpful to exercising one's engineering judgment. This is particularly important since, to quote a recent paper:[1] "A good engineering design is a balanced design in which all the factors which interact, even those which cannot be quantified, are taken into account; the responsibility of the design engineers is not to compute accurately but to judge soundly."

4. Rock mass classifications, which thus form the backbone of the empirical design approach, are widely employed in rock tunneling and most of the tunnels constructed at present in the United States make use of some classification system. The most extensively used and the best known of these is the Terzaghi classification which was introduced over 30 years ago.[2]

5. In fact, rock mass classifications have been successfully applied throughout the world: in the United States,[2-6] Canada,[7-8] Western Europe,[9-12] South Africa,[13-16] Australia,[17] New Zealand,[18] Japan,[19] USSR,[20] and in some East European countries.[21,22] Some classification systems were applied not only to tunneling but also to rock foundations,[23,24] rock slopes,[25,*] and even mining problems.[16]

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* Personal communication with K. W. John, 1978.

6. The purpose of this report is to evaluate tunnel design practices with respect to rock mass classification systems and particularly those which have been introduced in the recent years, have been tried out on a large number of tunneling projects, and have offered a practical and acceptable alternative to the classical Terzaghi classification of 1946.

PART II: CLASSIFICATION SYSTEMS IN ROCK ENGINEERING

7. A statement made in 1972 during the First Rapid Excavation and Tunneling Conference[5] is still appropriate for summarizing the present state of tunneling technology:

> "Predicting support requirements for tunnels has, for many years, been based on observation, experience and personal judgment of those involved in tunnel construction. Barring an unforseen breakthrough in geophysical techniques for making tunnel sites investigations, the prediction of support requirements for future tunnels will require the same approach."

Rock mass classifications can, if fulfilling certain conditions, effectively combine the findings from observation, experience, and engineering judgment for providing a quantitative assessment of rock mass conditions.

8. A rock mass classification has the following purposes in a tunneling application:

- a. Divide a particular rock mass into groups of similar behavior.
- b. Provide a basis for understanding the characteristics of each group.
- c. Facilitate the planning and the design of excavations in rock by yielding quantitative data required for the solution of real engineering problems.
- d. Provide a common basis for effective communication among all persons concerned with a tunneling project.

9. These aims can be fulfilled by ensuring that a classification system has the following attributes:

- a. Simple, easily remembered, and understandable.
- b. Each term clear and the terminology used widely acceptable.
- c. Only the most significant properties of rock masses included.
- d. Based on measurable parameters that can be determined by relevant tests quickly and cheaply in the field.
- e. Based on a rating system that can weigh the relative importance of the classification parameters.
- f. Functional by providing quantitative data for the design of tunnel support.

g. General enough so that the same rock mass will possess the same basic classification regardless whether it is being used for a tunnel, a slope, or a foundation.

10. To date many rock mass classification systems have been proposed, the better known of these being the classifications by Terzaghi (1946),[2] Lauffer (1958),[9] Deere (1964),[3] Wickham, Tiedemann, and Skinner (1972),[5] Bieniawski (1973),[13] and Barton, Lien, and Lunde (1974).[12] These classification systems will be discussed in detail while other classifications can be found in the references.

11. The six classifications named above were selected for detailed discussion because of their special features and contributions to the subject matter. Thus, the classical rock load classification of Terzaghi,[2] the first practical classification system introduced, has been dominant in the United States for over 30 years and has proved very successful for tunneling with steel supports. Lauffer's classification[9] based on work of Stini[26] was a considerable step forward in the art of tunneling since it introduced the concept of the stand-up time of the active span in a tunnel that is most relevant for determination of the type and the amount of tunnel support. Deere's classification[3] introduced the rock quality designation (RQD) index, which is a simple and practical method of describing the quality of rock core from borings. The concept of rock structure rating (RSR), developed in the United States by Wickham, Tiedemann, and Skinner,[5,6] was the first system assigning classification ratings for weighing the relative importance of classification parameters. The Geomechanics Classification proposed by Bieniawski[13] and the Q-System proposed by Barton, Lien, and Lunde[12] were developed independently (in 1973 and 1974, respectively), and both these classifications provide quantitative data enabling the selection of modern tunnel reinforcement measures such as rockbolts and shotcrete. The Q-System has been developed specifically for tunnels, while the Geomechanics Classification, although also initially developed for tunnels, has been applied to rock slopes and foundations, ground rippability assessment, as well as to mining problems.[23]

12. Some comparisons have been made between the various classification systems.[17,18,23,27,28,29] One detailed comparison was made by the author[23] during the construction of a railroad tunnel,[30] which was 18 ft* wide and 2.4 miles long. This tunnel was characterized by highly variable rock conditions--from very poor to very good. In addition, a one-year tunnel-monitoring program featuring 16 measuring stations enabled correlation between the classification ratings of rock conditions with the amount of rock movement, the rate of face advance, and the support used. This project thus afforded an ideal opportunity for comparison of the various classification systems. The results of this comparison are given in Table 1.

13. It is widely believed that the design of underground excavations is, to a large extent, the design of underground support systems.[28] This means that since rock mass classifications are used as tunnel design methods, they must be evaluated with respect to the guidelines that they provide for the selection of tunnel support. In this connection, however, it must be remembered that tunnel support may be regarded as the primary support (otherwise known as the temporary support) or the permanent support (usually concrete lining). Primary support (e.g., rockbolts, shotcrete, or steel ribs) is invariably installed closely to the tunnel face shortly after the excavation is completed. Its purpose is to ensure tunnel stability until the concrete lining is installed.

14. It should not be overlooked that the primary support may probably be able to carry all the load ever acting on the tunnel. After all, modern supports do not deteriorate easily and the traditional concept of the temporary and permanent support is losing its meaning. In some European countries, for example, Austria, Germany, Sweden, and Norway, only one kind of support is understood, generally a combination of rockbolts and shotcrete, and concrete linings are considered unnecessary if tunnel monitoring shows stabilization of rock movements. This

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

is the case for highway and railroad tunnels, while water tunnels may feature concrete linings, not for structural stability reasons but to reduce surface friction and to prevent water leakage into the rock.

15. Consequently, the use of the concept of the primary and the permanent supports may well lead to overdesign of tunnels since the so-called primary support may be all that is necessary and the concrete lining only serves as an expensive cosmetic feature acting psychologically to bolster public confidence in the safety of the tunnel. The only justification for placing concrete lining may be that since the current knowledge of rock tunnel engineering is still incomplete, a radical departure from the customary methods of design may not be advisable. However, the possibility of tunnel overdesign should not be overlooked, and methods of minimizing this possibility, without jeopardizing tunnel safety, should be constantly sought.

Terzaghi's Rock Load Classification

16. Since the purpose of this report is to evaluate other than the Terzaghi classification system and since his classification is fully treated both in Proctor and White's book[2] and in EM 1110-2-2901,[31] it will not be repeated here. However, for the sake of completeness and because of its historical importance, and main features of Terzaghi's rock load classification are given in Appendix A.

17. Terzaghi's contribution lies in formulating, over 30 years ago, the first rational method of evaluating rock loads appropriate to the design of steel sets. This was an important development, because support by steel sets has been the most commonly used system for containing rock tunnel excavations during the past 50 years. It must be emphasized, however, that while this classification is appropriate for the purpose for which it was evolved, i.e., for estimating rock loads for steel-arch supported tunnels, it is not so suitable for modern tunneling methods using shotcrete and rockbolts. After detailed studies, Cecil[32] concluded that Terzaghi's classification was too general to

permit an objective evaluation of rock quality and that it provided no quantitative information on the properties of rock masses.

Lauffer's Classification

18. The 1958 classification by Lauffer[9] has its foundation in the earlier work on tunnel geology by Stini,[26] who is considered as the father of the "Austrian School" of tunneling and rock mechanics. Stini emphasized the importance of structural defects in rock masses. Lauffer proposed that the stand-up time for any active unsupported rock span is related to the various rock mass classes as shown in the diagram in Figure 1. An active unsupported span is the width of the tunnel or the distance from the face to the support if this is less than the tunnel width. The stand-up time is the period of time that a tunnel will stand unsupported after excavation. It should be noted that a number of factors may affect the stand-up time, as illustrated diagrammatically in Figure 2. Lauffer's original classification is no longer used since it has been modified a number of times by other Austrian engineers, notably von Rabcewicz, Gosler, and Pacher.[10]

19. The main significance of Lauffer's classification is that Figure 1 shows how an increase in a tunnel span leads to a drastic reduction in the stand-up time. This means, for example, that while a pilot tunnel having a small span may be successfully constructed full face in fair rock conditions, a large span opening in this same rock may prove impossible to support in terms of the stand-up time. Only a system of smaller headings and benches or multiple drifts can enable a large cross-section tunnel to be constructed in such rock conditions.

20. A disadvantage of a Lauffer-type classification is that these two parameters, the stand-up time and the span, are difficult to establish and rather much is demanded of practical experience. Nevertheless, this concept introduced the stand-up time and the span as the two most relevant parameters for the determination of the type and amount of tunnel support, and this has influenced the development of more recent rock mass classification systems.[13]

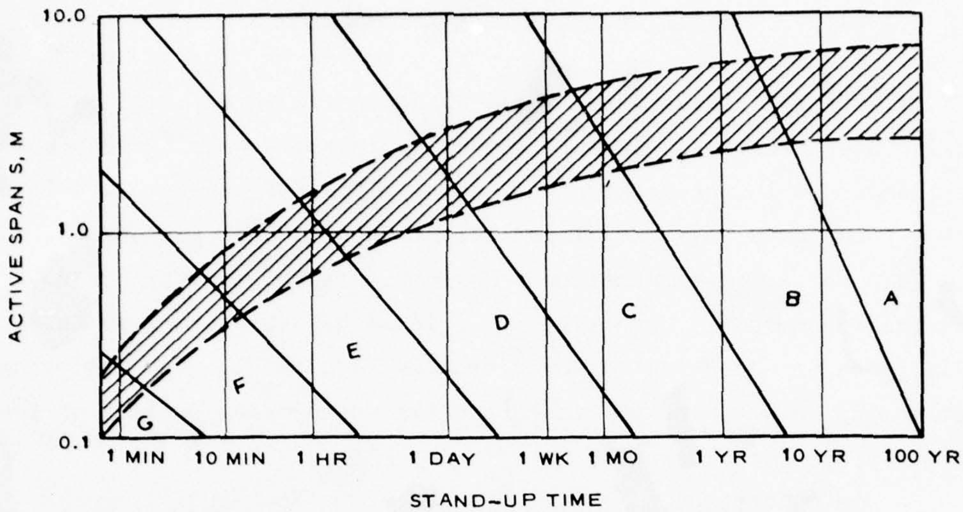

Figure 1. Lauffer's relationship between active span and stand-up time for different classes of rock mass: A - very good rock, G - very poor rock

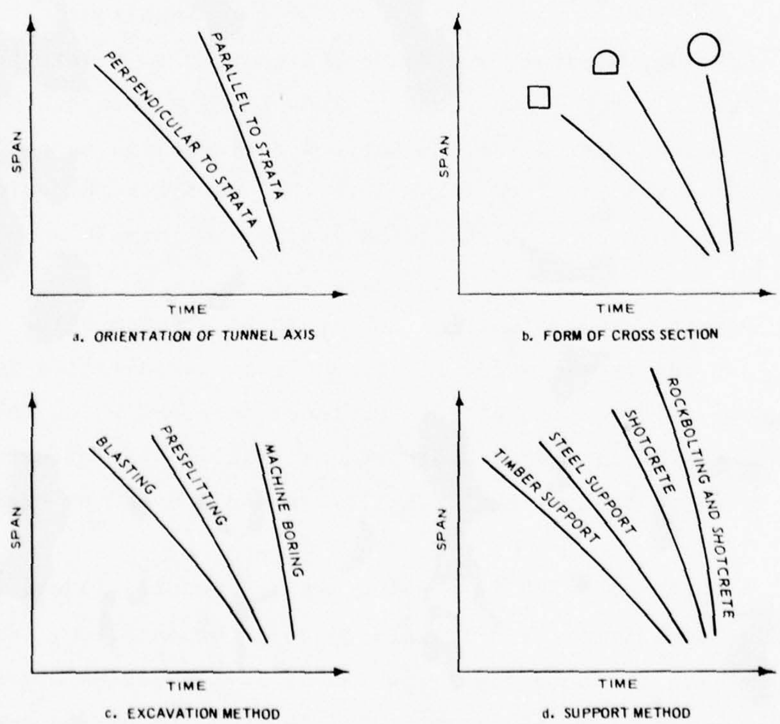

Figure 2. Factors influencing rock mass stability during tunneling (schematically after Lauffer[9])

Deere's Rock Quality Designation

21. Deere[3] proposed in 1964 a quantitative index based on a modified core recovery procedure which incorporates only those pieces of core that are 4 in. or greater in length. This RQD has been widely used and has been found very useful for selection of tunnel support.[4]

22. For RQD determination, the International Society for Rock Mechanics recommends a core size of at least NX diameter (2.16 in.) drilled with double-barrel diamond drilling equipment. The following relationship between the RQD index and the engineering quality of the rock was proposed by Deere:[3]

RQD, percent	Rock Quality
< 25	Very Poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

23. Cording, Hendron, and Deere[33] attempted to relate the RQD index to Terzaghi's rock load factor. They found a reasonable correlation for steel-supported tunnels but not for openings supported by rockbolts, as is evident from Figure 3. This supports the opinion that Terzaghi's rock load concept should be limited to tunnels supported by steel sets.[34]

24. Merrit[35] found that the RQD could be of much value in estimating support requirements for rock tunnels as demonstrated in Figure 4. He pointed out a limitation of the RQD index in areas where the joints contain thin clay fillings or weathered material. The influence of clay seams and fault gouge on tunnel stability was discussed by Brekke and Howard.[36]

25. Although the RQD is a quick and inexpensive index, it has considerable limitations by disregarding joint orientation, tightness, and gouge material. Consequently, while it is a practical parameter for core quality estimation, it is not sufficient on its own to provide an adequate description of a rock mass.

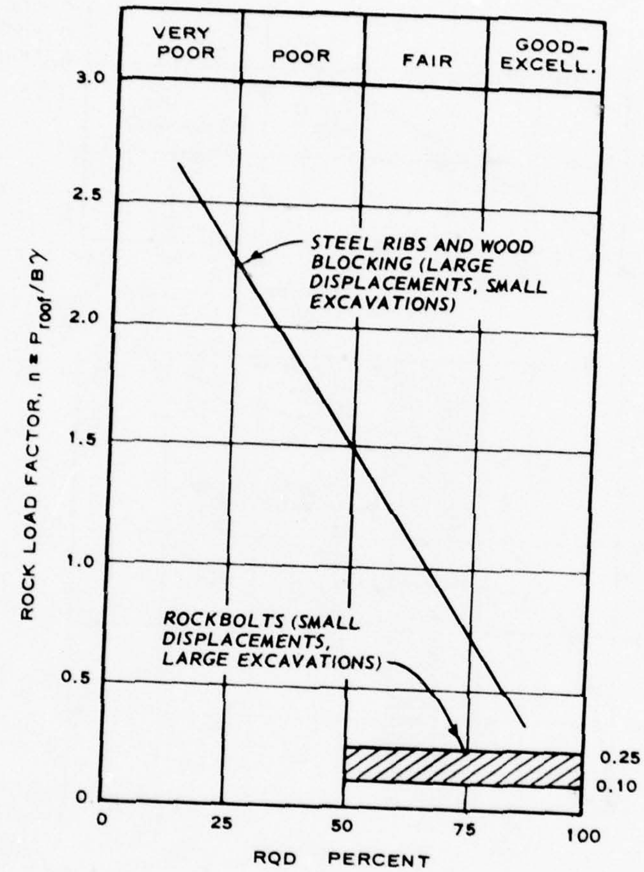

Figure 3. Comparison of roof support designs for steel rib-supported tunnels and for rock-bolted caverns (after Cording and Deere[34])

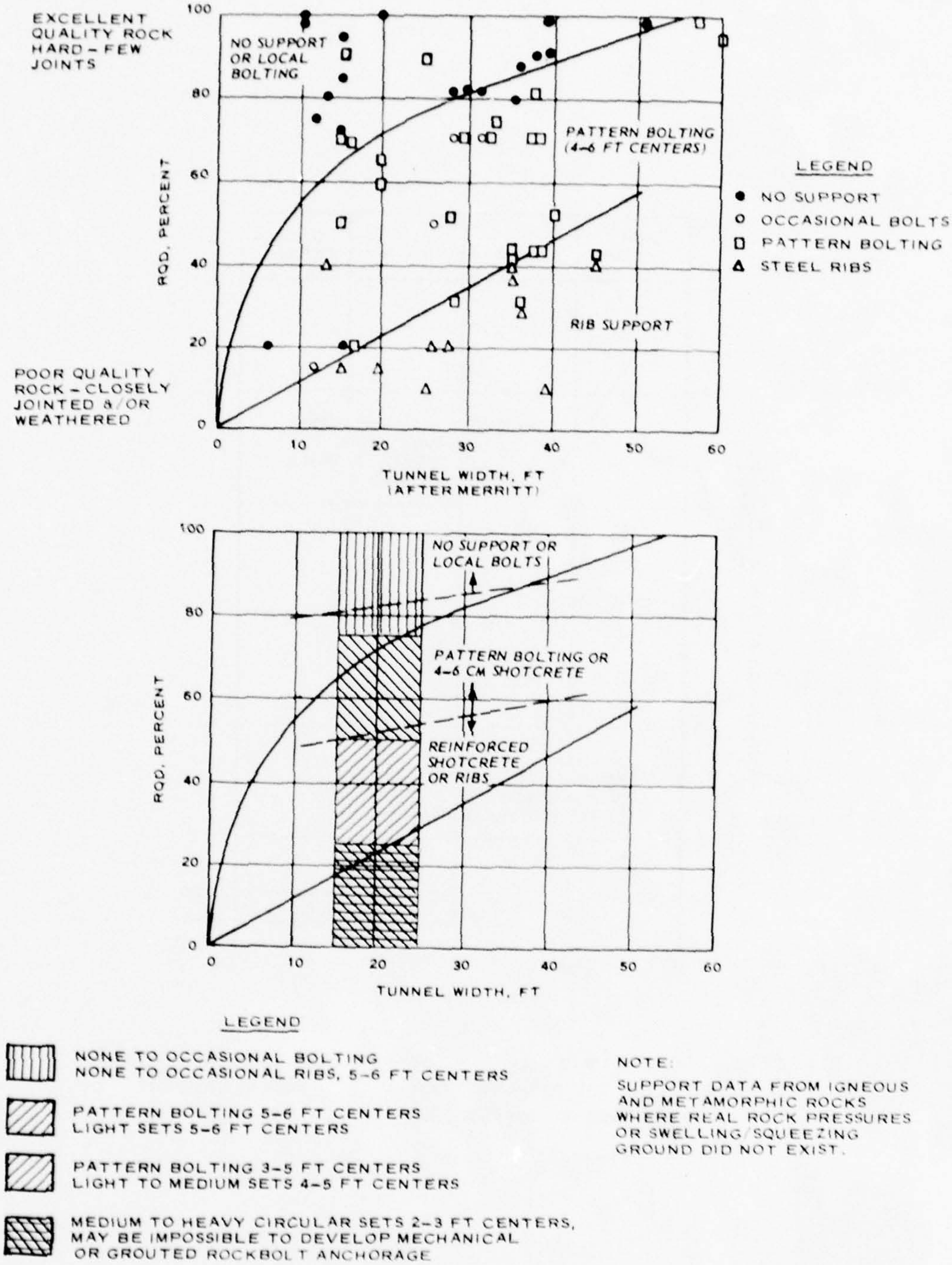

Figure 4. Comparison of rock quality support criteria from various sources (after Merritt[35])

RSR Concept

26. The RSR Concept, a ground support prediction model, was developed in the United States in 1972 by Wickham, Tiedemann, and Skinner.[5,6] The concept presents a quantitative method for describing the quality of a rock mass and for selecting the appropriate ground support. It was the first complete rock mass classification system proposed since that introduced by Terzaghi in 1946.

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

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

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

parameters were: (a) size of tunnel, (b) direction of drive, and (c) method of excavation.

30. All the above factors were grouped by Wickham, Tiedemann, and Skinner[5] into three basic parameters, A, B, and C (Tables 2, 3, and 4, respectively), which in themselves were evaluations as to the relative effect on the support requirements of various geological factors. These three parameters were as follows:

a. Parameter A. General appraisal of rock structure is on the basis of:

   (1) Rock type origin (igneous, metamorphic, sedimentary).

   (2) Rock hardness (hard, medium, soft, decomposed).

   (3) Geologic structure (massive, slightly faulted/folded, moderately faulted/folded, intensely faulted/folded).

b. Parameter B. Effect of discontinuity pattern with respect to the direction of tunnel drive is on the basis of:

   (1) Joint spacing.

   (2) Joint orientation (strike and dip).

   (3) Direction of tunnel drive.

c. Parameter C. Effect of groundwater inflow is based on:

   (1) Overall rock mass quality due to parameters A and B combined.

   (2) Joint condition (good, fair, poor).

   (3) Amount of water inflow (in gallons per minute per foot of the tunnel).

31. The RSR value of any tunnel section is obtained by summarizing the weighted numerical values determined for each parameter. This reflects the quality of the rock mass with respect to its need for support regardless of the size of the tunnel. The relation between RSR values and tunnel size is taken into consideration in the determination of respective rib ratios (RR), as discussed below. Since a lesser amount of support was expected for machine-bored tunnels than when excavated by drill and blast methods, it was suggested that RSR values be adjusted for machine-bored tunnels in the manner given in Figure 5.

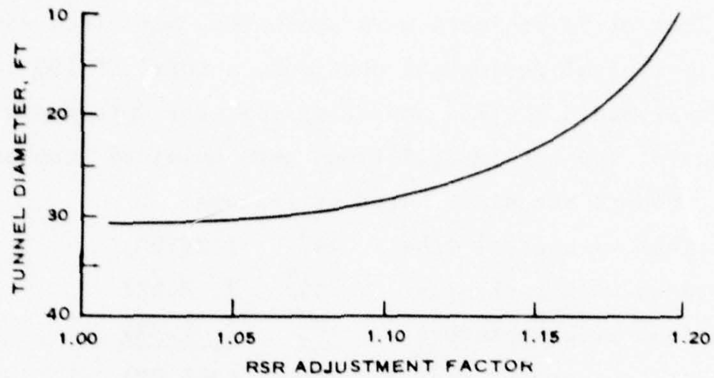

Figure 5. RSR concept-adjustment for machine tunneling

32. It should be noted that Tables 2, 3, and 4 are reproduced not from the original reference[5] but from a paper[6] published two years later, because the RSR ratings were changed in 1974 and the latter paper represents the latest information available.

33. In order to correlate RSR values with actual support installations, a concept of the RR was introduced. The purpose was to have a common basis for correlating RSR determinations with actual or required installations. Since 90 percent of the case history tunnels were supported with steel ribs, the RR measure was chosen as the theoretical support (rib size and spacing). It was developed from Terzaghi's formula for determining roof loads in loose sand below the water table (datum condition). Using the tables provided in Rock Tunneling with Steel Supports,[2] the theoretical spacing required for the same size rib as used in a given case study tunnel section was determined for the datum condition. The RR value is obtained by dividing this theoretical spacing by the actual spacing and multiplying the answer by 100. Thus, RR = 46 would mean that the section required only 46 percent of the support used for the datum condition. However, different size tunnels, although having the same RR would require different weight or size of ribs for equivalent support. The RR for an unsupported tunnel would be zero and would be 100 for a tunnel requiring the same support as the datum condition.

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

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

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

$$(RR + 80)(RSR + 30) = 8800 \qquad \text{(Reference 6)}$$

or

$$(RR + 70)(RSR + 8) = 6000 \qquad \text{(Reference 5)}$$

It was concluded[6] that rock structures with RSR values less than 19 would require heavy support while those with ratings of 80 and over would be unsupported.

36. Since the RR basically defined an anticipated rock load by considering the load-carrying capacity of different sizes of steel ribs, the RSR values were also expressed in terms of unit rock loads for various sized tunnels as given in Table 5.

37. The RSR prediction model was developed primarily with respect to steel rib support.[6] Insufficient data were available to correlate rock structures and rockbolt or shotcrete support. However, an appraisal of rockbolt requirements was made by considering rock loads with respect to the tensile strength of the bolt. The authors pointed out[5] that this was a very general approach: it assumed that anchorage was adequate and that all bolts acted in tension only; it did not allow either for interaction between adjacent blocks or for an assumption of a compression arch formed by the bolts. In addition, the rock loads were developed for steel supported tunnels. Nevertheless, the following relation was given for 1-in.-diam rockbolts with a working load of 24,000 lb:

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

where W is the rock load in 1000 psf.

38. No correlation could be found between geologic prediction and shotcrete requirements, so that the following empirical relationship was suggested:

$$t = 1 + \frac{W}{1.25} \quad \text{or} \quad t = \frac{D}{150}(65 - RSR)$$

where

t = shotcrete thickness, in.

W = rock load

D = tunnel diameter, ft

39. Support requirement charts have been prepared that provide a means of determining typical ground support systems based on a RSR prediction as to the quality of rock structure through which the tunnel is to be driven. Charts for 10-, 20-, and 24-ft-diam tunnels are shown in Figures 6, 7, and 8, respectively. Similar charts could be used for other tunnel sizes. The three steel rib curves reflect typical sizes used for the particular tunnel size. The curves for rockbolts and shotcrete are dashed to emphasize that they are based on assumptions and were not derived from case histories. The charts are applicable to either circular or horseshoe-shaped tunnels of comparable widths.

40. The author believes that the RSR Concept is a very useful method for selecting steel rib support for rock tunnels. As with any empirical approaches, one should not apply a concept beyond the range of sufficient and reliable data used for developing the concept. For this reason, the RSR Concept is not recommended for selection of rockbolt and shotcrete support. However, because of its usefulness for steel rib support determination, the author prepared an input data sheet for this classification system (see Appendix B). It should be noted that although the definitions of the classification parameters were not explicitly stated by the proposers,[5] most of the input data needed will be normally included in a standard joint survey; however, the lack of definitions (e.g., slightly faulted or folded rock) may lead to some confusion.

41. A practical example using the RSR Concept is as follows:

Consider a 20-ft-diam tunnel to be driven in a slightly faulted strata featuring medium hard granite. The joint spacing is 2 ft and the joints are open. The estimated water inflow is 250 gal/min per 1000 ft of the tunnel length. The tunnel will be driven against a dip of 45 deg and perpendicular to the jointing.

Solution: From Table 2: For igneous rock of medium hardness (basic rock type 2) in slightly faulted rock, parameter A = 20. From Table 3: For moderate to blocky jointing, with strike perpendicular to the tunnel axis and with a drive against the dip of 45 deg, parameter B = 25. From Table 4: For A + B = 45, poor joint condition and moderate water flow, parameter C = 12.

Thus: RSR = A + B + C = 57. From Figure 7, the support requirements for a 20-ft-diam tunnel with RSR = 57 (estimated rock load 1.5 kips/sq ft) will be 6H20 steel ribs at 6-ft spacing.

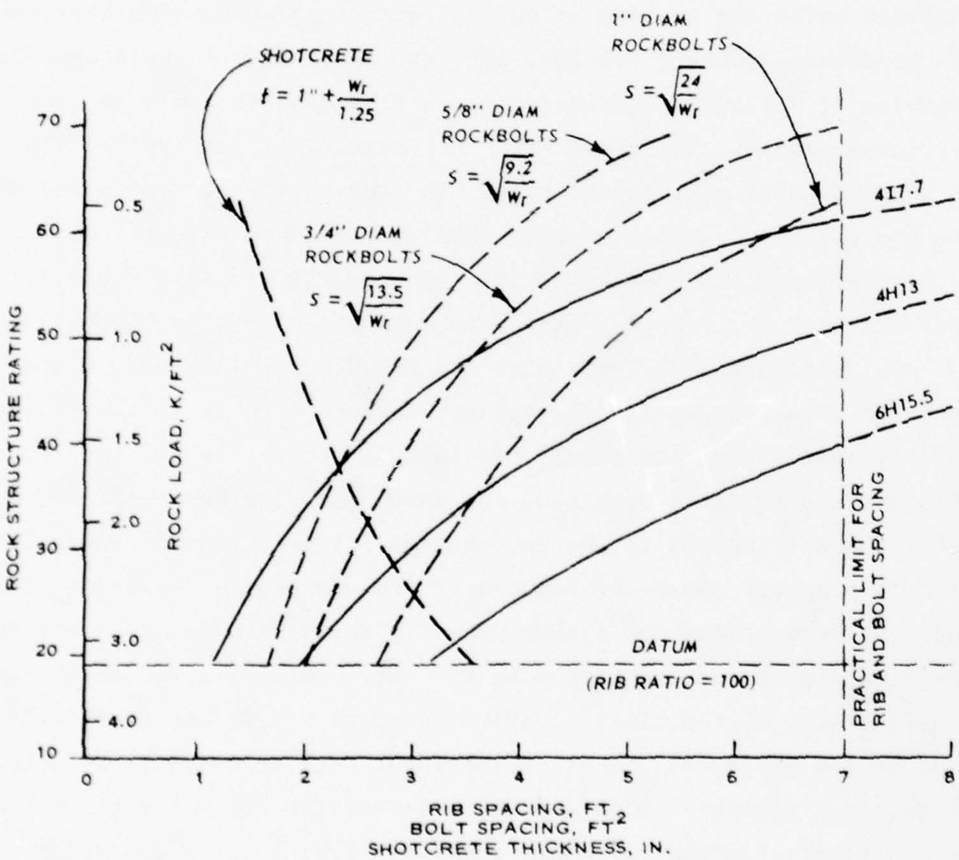

Figure 6. RSR concept - support chart for 10-ft-diam tunnel

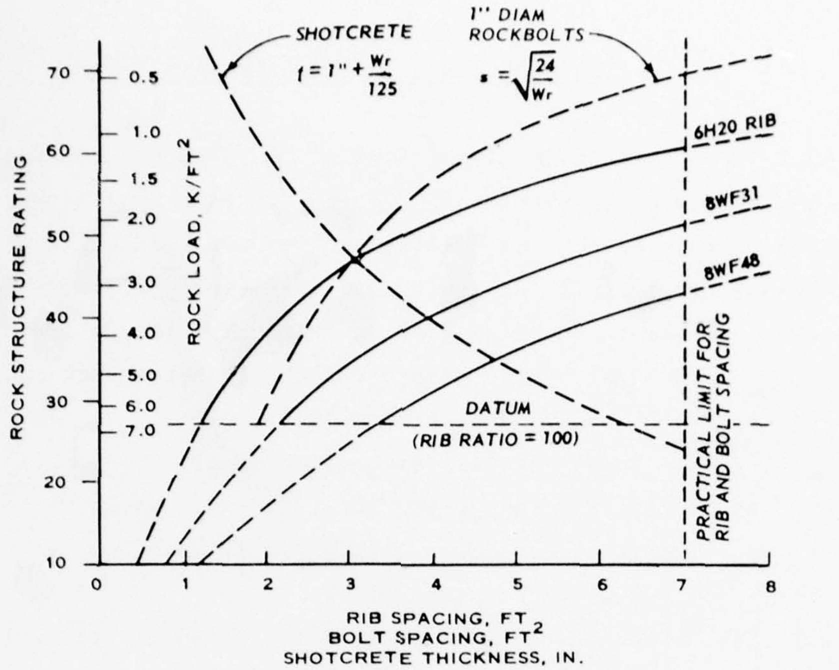

Figure 7. RSR concept - support chart for 20-ft-diam tunnel

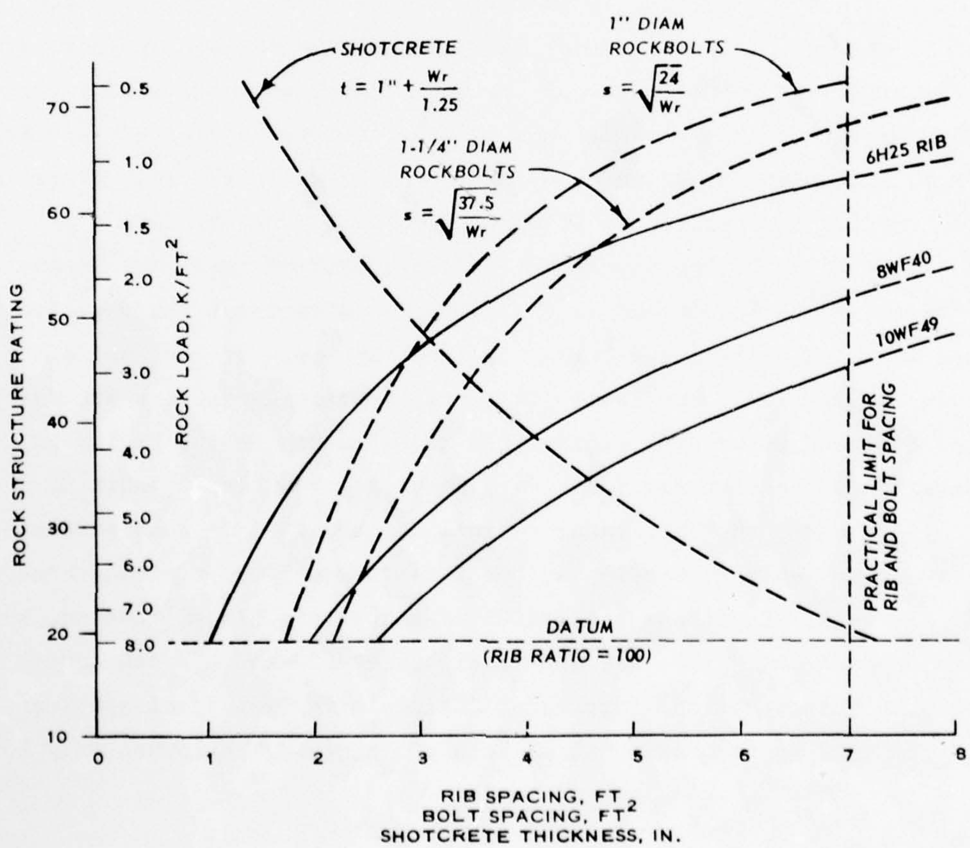

Figure 8. RSR concept - support chart for 24-ft-diam tunnel

The Geomechanics Classification

42. The Geomechanics Classification or the Rock Mass Rating (RMR) System was developed by Bieniawski[13] in 1973. This engineering classification of rock masses, especially evolved for rock engineering applications, utilizes the following six parameters, all of which not only are measurable in the field but can also be obtained from borings:

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

43. The Geomechanics Classification is presented in Table 6. In Section A of Table 6, five parameters are grouped into five ranges of values. Since the various parameters are not equally important for the overall classification of a rock mass, importance ratings are allocated to the different value ranges of the parameters, a higher rating indicating better rock mass conditions. These ratings were determined from 49 case histories investigated by the author[23] while the initial ratings were based on the studies by Wickham, Tiedemann, and Skinner.[5]

44. To apply the Geomechanics Classification, the rock mass along the tunnel route is divided into a number of structural regions, i.e., zones in which certain geological features are more or less uniform within each region. The above six classification parameters are determined for each structural region from measurements in the field and entered onto the standard input data sheet as shown in Appendix B.

45. Next, the importance ratings are assigned to each parameter according to Table 6, Section A. In this respect, the typical rather than the worst conditions are evaluated since this classification, being based on case histories, has a built-in safety factor. Furthermore, it should be noted that the importance ratings given for joint spacings apply to rock masses having three sets of joints. Thus, when only two

sets of joints are present, a conservative assessment is obtained. Once the importance ratings of the classification parameters are established, the ratings for the five parameters listed in Section A of Table 6 are summed to yield the basic overall rock mass rating for the structural region under consideration.

46. At this stage, the influence of the strike and dip of joints is included by adjusting the basic rock mass rating according to Section B of Table 6. This step is treated separately because the influence of joint orientation depends upon engineering application, e.g., tunnel, slope, or foundation. It will be noted that the "value" of the parameter "joint orientation" is not given in quantitative terms but by qualitative descriptions such as "favourable." To facilitate a decision whether strike and dip orientations are favourable or not, reference should be made to Table 7, which is based on studies by Wickham, Tiedemann, and Skinner.[5] In the case of civil engineering projects, an adjustment for joint orientations will suffice. For mining applications, other adjustments may be called for such as the stress at depth or a change in stress.[23]

47. After the adjustment for joint orientations, the rock mass is classified according to Section C of Table 6, which groups the final (adjusted) rock mass ratings (RMR) into five rock mass classes. Note that the rock mass classes are in groups of twenty ratings each.

48. Next, Section D of Table 6 gives the practical meaning of each rock mass class by relating it to specific engineering problems. In the case of tunnels and chambers, the output from the Geomechanics Classification is the stand-up time of an unsupported rock span for a given rock mass rating (Figure 9).

49. Longer stand-up times can be achieved by selecting rock reinforcement measures in accordance with Table 8. They depend on such factors as the depth below surface (in situ stress), tunnel size and shape, and the method of excavation.

50. It should be noted that the support measures given in Table 8 represent the permanent and not the primary support. Hence, additional concrete lining is not required for structural purposes.

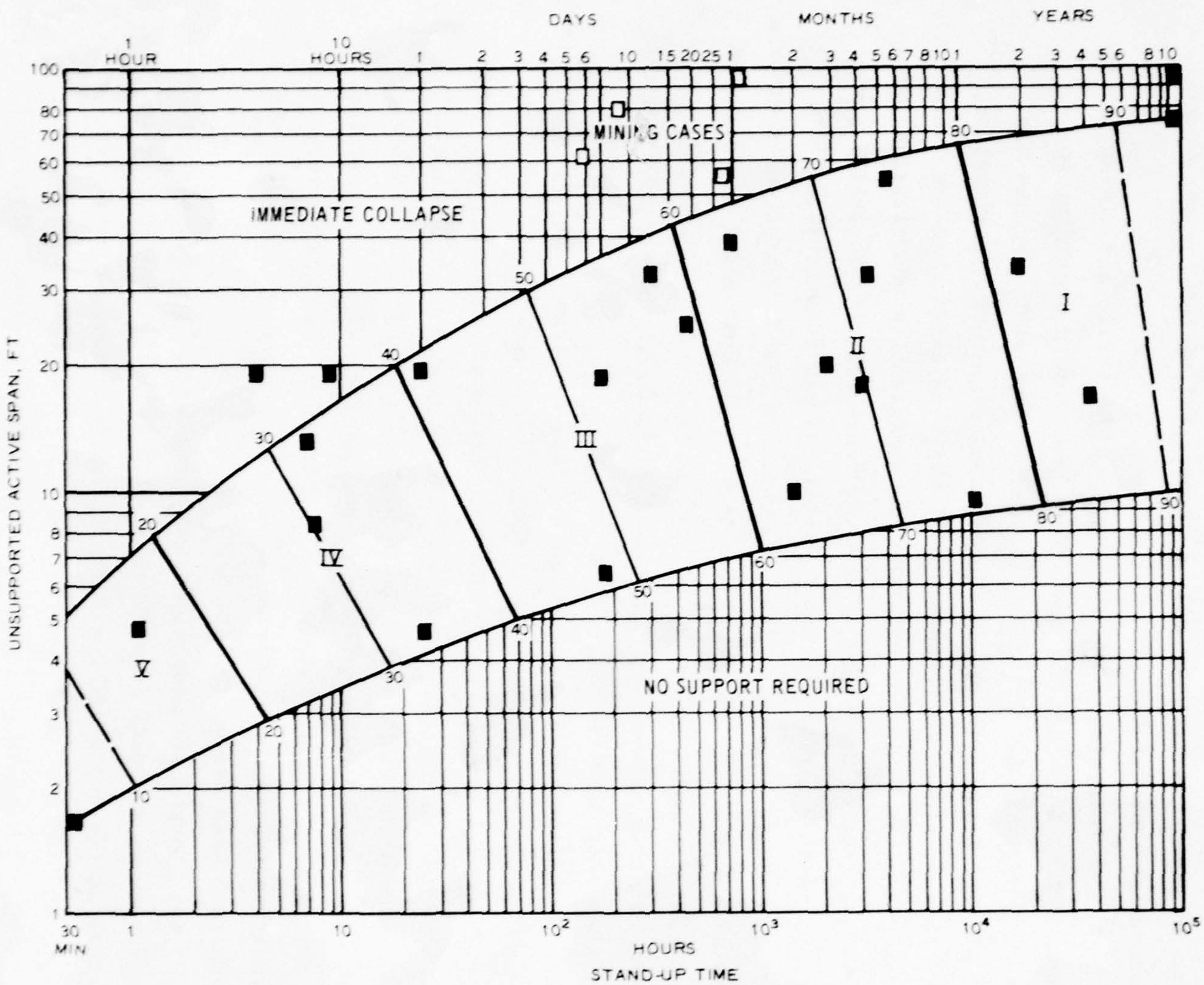

Figure 9. Geomechanics Classification - output of stand-up time versus unsupported span

However, to ensure full structural stability it is recommended that tunnel monitoring during construction should provide a check on stabilization of rock movements.

51. The Geomechanics Classification recognizes that no single parameter or index can fully and quantitatively describe a jointed rock mass for tunneling purposes. Various factors have different significance, and only if taken together can they describe satisfactorily a rock mass. Each of the six parameters employed in this classification is discussed below.

Strength of intact rock material

52. There is a general agreement that knowledge of the uniaxial compressive strength of intact rock is necessary for classifying a rock mass. After all, if the discontinuities are widely spaced and the rock material is weak, the rock material properties will influence the behavior of the rock mass. Under the same confining pressure, the strength of the rock material constitutes the highest strength limit of the rock mass. The rock material strength is also important if the use of tunneling machines is contemplated. Finally, a sample of the rock material represents sometimes a small-scale model of the rock mass since they have both been subjected to the same geological processes. It is believed that the engineering classification of intact rock, proposed by Deere and Miller,[37] is particularly realistic and convenient for use in the field of rock mechanics. This classification is given in Table 9.

53. The uniaxial compressive strength of rock material is determined in accordance with the standard laboratory procedures, but for the purpose of rock classification, the use of the well-known, point-load strength index is recommended. The reason is that the index can be determined in the field on rock core retrieved from borings and the core does not require any special preparation. Using simple portable equipment, a piece of drill core is compressed between two points. The core fails as a result of fracture across its diameter. The point-load strength index is calculated as the ratio of the applied load to the square of core diameter. A close correlation exists (to within ~20 percent)[38]

between the uniaxial compressive strength (σ) and the point-load strength index I_s such that for standard NX core (2.16-in. diameter), $\sigma \doteq 24\ I_s$.

54. In rock engineering, the information on the rock material strength is preferable to that on rock hardness. The reason is that rock hardness, which is defined as the resistance to indentation or scratching, is not a quantitive parameter and is subjective to a geologist's personal opinion. It has been employed in the past before the advant of the point-load strength index that can now assess the rock strength in the field. For the sake of completeness, the following hardness classification was used in the past:

a. Very soft rock. Material crumbles under firm blow with a sharp end of a geological pick and can be peeled off with a knife.

b. Soft rock. Material can be scraped and peeled with a knife; indentations 1/16 to 1/8 in. show in the specimen with firm blows.

c. Medium hard rock. Material cannot be scraped or peeled with a knife; hand-held specimen can be broken with the hammer end of a geological pick with a single firm blow.

d. Hard rock. Hand-held specimen breaks with hammer end of pick under more than one blow.

e. Very hard rock. Specimen requires many blows with geological pick to break through intact material.

It can be seen from the above that for the lower ranges up to medium hard rock, hardness can be assessed from visual inspection and by scratching with a knife and striking with a hammer. However, for rock having the uniaxial compressive strength of more than 3500 psi, hardness classification ceases to be meaningful due to the difficulty of distinguishing by the "scratchability test" the various degrees of hardness. In any case, hardness is only indirectly related to rock strength, the relationship between the uniaxial compressive strength and the product of hardness and density being expressed in the following formula:[39]

$$\log \sigma_c = 0.00014\ \gamma\ R + 3.16$$

where

γ = dry unit weight, pcf

R = Schmidt hardness (L-hammer)

Rock quality designation (RQD)

55. This index has already been discussed in paragraphs 21 through 25. It is used as a classification parameter, because although it is not sufficient on its own for a full description of a rock mass, the RQD index has been found most useful in tunneling applications as a guide for selection of tunnel support, has been employed extensively in the United States and in Europe, and is a simple, inexpensive, and reproducible way to assess the quality of rock core.[34]

Spacing of joints

56. The term joint means all discontinuities present in the rock mass that may be technically joints, bedding planes, minor faults, or other surfaces of weakness. The behavior of joints governs the behavior of a rock mass as a whole. The presence of joints reduces the strength of a rock mass, and their spacing governs the degree of such reduction. For example, a rock material with a high strength, but intensely jointed, will yield a weak rock mass. Spacing of joints is a separate parameter, because the RQD index does not lend itself for assessing the spacing of joints from a single set of cores. A classification of joint spacings proposed by Deere[39] is most widely used and has been incorporated into the Geomechanics Classification (Table 10).

Orientation of joints

57. Studies by Wickham, Tiedemann, and Skinner[5] have emphasized the effect of joint orientations on tunnel stability. In accordance with Table 7, a qualitative assessment of favourability is preferred to more elaborate systems for joint orientation and inclination effects.

Condition of joints

58. This parameter includes roughness of the joint surfaces, their continuity, their opening or separation (distance between the surfaces), the infilling (gouge) material, and weathering of the wall rock.

59. Roughness or the nature of the asperities in the discontinuity surfaces is an important parameter characterizing the condition of discontinuities. Asperities that occur on joint surfaces interlock, if

the surfaces are clean and closed, and inhibit shear movement along the joint surface. Roughness asperities usually have a base length and amplitude measured in terms of tenths of an inch and are readily apparent on a core-sized exposure of a discontinuity. The applicable descriptive terms are defined below (it should be stated if surface are stepped, undulating, or planar):

a. Very rough. Near vertical steps and ridges occur on the discontinuity surface.

b. Rough. Some ridge and side-angle steps are evident; asperities are clearly visible; and discontinuity surface feels very abrasive.

c. Slightly rough. Asperities on the discontinuity surfaces are distinguishable and can be felt.

d. Smooth. Surface appears smooth and feels so to the touch.

e. Slickensided. Visual evidence of polishing exists.

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

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

a. Very tight: < 0.1 mm.

b. Tight: 0.1-0.5 mm.

c. Moderately open: 0.5-2.5 mm.

d. Open: 2.5-10 mm.

e. Very wide: 10-25 mm.

Note that where the separation is more than 25 mm, the discontinuity should be described as a major discontinuity.

62. The infilling (gouge) has a two-fold influence:

a. Depending on the thickness, the filling prevents the interlocking of the fracture asperities.

b. It possesses its own characteristic properties, i.e., shear strength, permeability, and deformational characteristics.

The following aspects should be described: type, thickness, continuity, and consistency.

63. Weathering of the wall rock, i.e., the rock constituting the joint walls, is classified in accordance with the recommendations of the Task Committee of the American Society of Civil Engineers:[40]

a. Unweathered. No visible signs are noted of weathering; rock fresh; crystals bright.

b. Slightly weathered rock. Discontinuities are stained or discolored and may contain a thin filling of altered material. Discoloration may extend into the rock from the discontinuity surfaces to a distance of up to 20 percent of the discontinuity spacing.

c. Moderately weathered rock. Slight discoloration extends from discontinuity planes for greater than 20 percent of the discontinuity spacing. Discontinuities may contain filling of altered material. Partial opening of grain boundaries may be observed.

d. Highly weathered rock. Discoloration extends throughout the rock, and the rock material is partly friable. The original texture of the rock has mainly been preserved, but separation of the grains has occurred.

e. Completely weathered rock. The rock is totally discolored and decomposed and in a friable condition. The external appearance is that of soil. Internally, the rock texture is partly preserved, but grains have completely separated.

It should be noted that the boundary between rock and soil is defined in terms of the uniaxial compressive strength and not in terms of weathering. A material with the strength equal to or above 150 psi is considered as rock.

Groundwater conditions

64. In the case of tunnels, the rate of inflow of groundwater in gallons per minute per 1000 ft of the tunnel should be determined,[5] or a general condition can be described as completely dry, damp, wet, dripping, and flowing. If actual water pressure data are available, these should be stated and expressed in terms of the ratio of the water pressure to the major principal stress. The latter can be either measured or determined from the depth below surface, i.e., the vertical stress increases with depth at 1.1 psi per foot of the depth below surface.

65. The rock mass along the tunnel route is divided into a number of structural regions, and the above classification parameters are determined for each structural region and entered onto the standard input data sheet, as enclosed in Appendix B.

66. The advantage of the Geomechanics Classification is that it is not only applicable to rock tunnels but also to rock foundations[24] and slopes.[25,26] This is a very useful feature that can assist with the design of slopes near the tunnel portals as well as allow estimates of the deformability of foundations for such structures as bridges. After all, for a highway or railroad route involving tunnels and bridges, the output from the Geomechanics Classification for slopes and foundations will be very useful.

67. In the case of rock foundations, the rock mass rating RMR from the Geomechanics Classification has been related[24] to the in situ modulus of deformation in the manner shown in Figure 10.

68. In the case of rock slopes, the output is given in Section D of Table 6 as the cohesion and friction of the rock mass. These output values were based on the data compiled by Hoek and Bray.[41] The validity of the output from the Geomechanics Classification to the rock slopes was tested by Steffen[25] and by John.* Steffen analyzed 35 slopes of which 20 had failed. He used the Geomechanics Classification to obtain the average values of cohesion and friction and then calculated the

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* See footnote, page 5.

safety factor based on slope design charts by Hoek and Bray.[41] The results given in Figure 11 show definite statistical trends.

69. In spite of its versatility, the Geomechanics Classification is not considered sufficient to deal with all tunnel stability problems.[13] Like with other empirical methods, it should be backed by a monitoring program during the tunnel construction. The purpose of such a program would be to check on the rock conditions predicted by the classification and to evaluate the behavior of the adopted support measures.

70. A practical example using the Geomechanics Classification is as follows:

> Consider a slightly weathered quartzite in which a 20-ft-span tunnel is to be driven. The following classification parameters were determined:

Item	Value	Rating
1. Strength of rock material	22,000 psi	12
2. RQD	80-90%	17
3. Spacing of joints	1-3 ft	20
4. Condition of joints: continuous joints slightly rough surfaces separation <1 mm highly weathered rock wall no gouge		12
5. Groundwater	Moderate inflow	7
	Basic rock mass value	68
6. Orientation of joints	Fair	-5
	Final RMR	63

Rock Mass Class: II - good rock

> Output: From Figure 9, for RMR = 63 and unsupported span = 20 ft, the stand-up time will be about 1 month. From Table 8, recommended tunnel support is rockbolts in crown 10 ft long, spaced at 8 ft with shotcrete 2 in. thick and wire mesh. From Figure 10, the rock mass modulus is estimated as 3.7×10^6 psi.

71. It is important that the chart in Figure 9 is correctly applied for the selection of the output data. For this purpose, the actual RMR's are used that are represented by the series of near parallel lines in Figure 9.

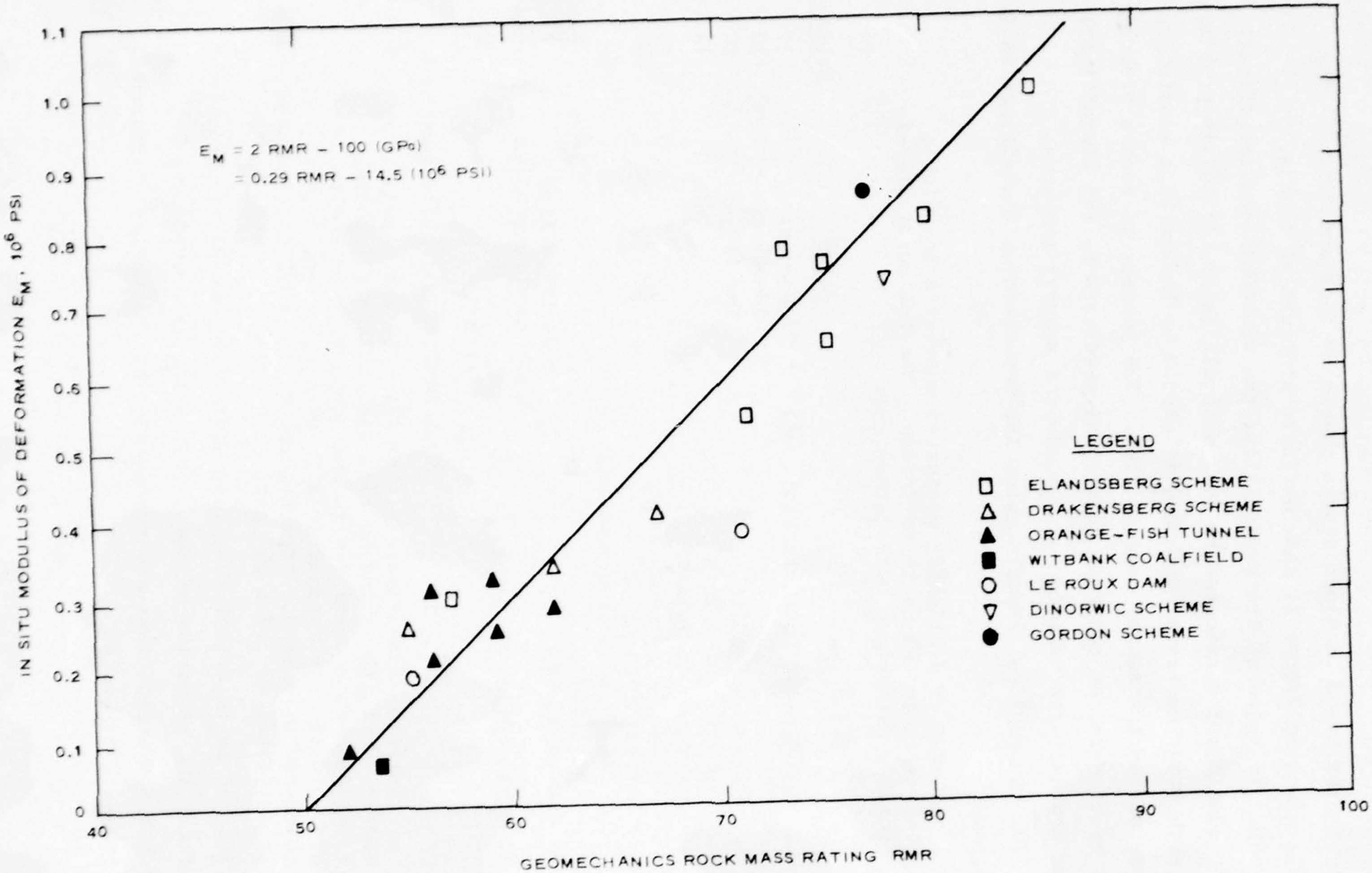


Figure 10. Relationship between in situ modulus and rock mass rating

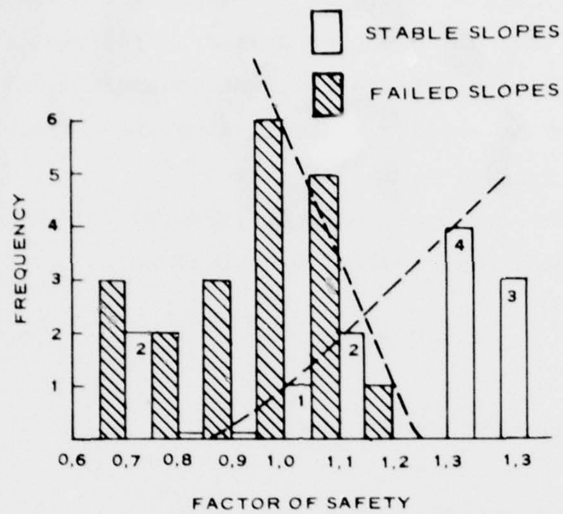


Figure 11. Frequency distribution of slope stability as predicted by Hoek's design charts for the geomechanics system strength parameters (after Steffen[25])

72. The intercept of an RMR line with the desired tunnel span determines the stand-up time. Alternatively, the intercept of an RMR line with the top boundary line determines the maximum span possible in a given rock mass; any larger span would result in the immediate roof collapse. An intercept of the RMR line with the lower boundary line determine the maximum span that can stand unsupported indefinitely.

Q-System

73. The Q-System of rock mass classification was developed in Norway in 1974 by Barton, Lien, and Lunde, all of the Norwegian Geotechnical Institute.[12] Its development represented a major contribution to the subject of rock mass classifications for a number of reasons: the system was proposed on the basis of an analysis of some 200 tunnel case histories from Scandinavia,[42] it is a quantitative classification system, and it is an engineering system enabling the design of tunnel supports.

74. The Q-System is based on a numerical assessment of the rock mass quality using six different parameters: (a) RQD, (b) number of joint sets, (c) roughness of the most unfavourable joint or discontinuity, (d) degree of alteration or filling along the weakest joint, (e) water inflow, and (f) stress condition.

75. The above six parameters are grouped into three quotients to give the overall rock mass quality Q as follows:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

where

RQD = rock quality designation

J_n = joint set number

J_r = joint roughness number

J_a = joint alteration number

J_w = joint water reduction number

SRF = stress reduction number

76. In Tables 11 - 13, the numerical values of each of the above parameters are interpreted as follows. The first two parameters represent the overall structure of the rock mass, and their quotient is claimed to be a measure of the relative block size. The quotient of the third and the fourth parameters is said to be related to the interblock shear strength (of the joints). The fifth parameter is a measure of water pressure, while the sixth parameter is a measure of: (a) loosening load in the case of shear zones and clay bearing rock, (b) rock stress in competent rock, and (c) squeezing and swelling loads in plastic incompetent rock. This sixth parameter is regarded as the "total stress" parameter. The quotient of the fifth and the sixth parameters is regarded as describing the "active stress."

77. The proposers[12] of the Q-System believed that the parameters, J_n, J_r, and J_a, played a more important role than joint orientation, and if joint orientation had been included, the classification would have been less general. However, the orientation is implicit in the parameters J_r and J_a, because they apply to the most unfavourable joints.

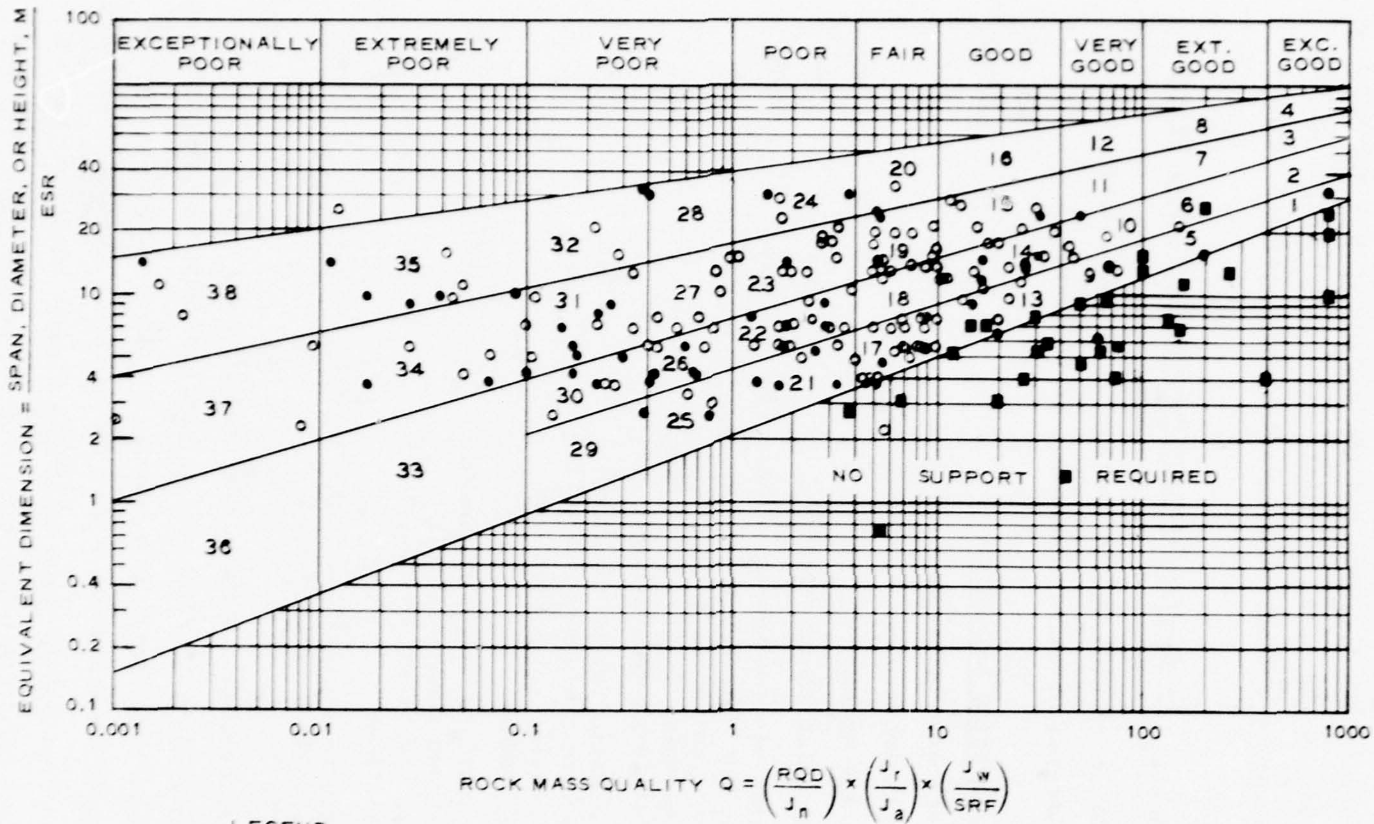
78. The Q is related to the tunnel support requirements by defining the equivalent dimensions of the excavation. This equivalent dimension, which is a function of both the size and the purpose of the excavation, is obtained by dividing the span, diameter, or the wall height of the excavation by a quantity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

$$\text{Equivalent dimension} = \frac{\text{Excavation span, diameter, or height, metres}}{\text{ESR}}$$

79. The ESR is related to the use for which the excavation is intended and the degree of safety demanded, as follows:

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

80. The relationship between the index Q and the equivalent dimension is illustrated in Figure 12 in which 38 support categories are shown by box numbering. Support measures that are appropriate to each category are listed in Tables 14 - 18. Since it was decided that bolting and shotcrete support deserves most attention, case histories featuring steel rib support, concrete arch roofs, and precast linings have been ignored.



LEGEND

- ○ CASES COLLECTED BY BARTON ET AL[12]
- ● CASES COLLECTED BY CECIL[32]
- ■ UNSUPPORTED EXCAVATIONS

Figure 12. Q-System - equivalent dimension versus rock mass quality (after Barton[43])

81. The length of bolts L is determined from the equation:

$$L = 2 + 0.15\ B/ESR$$

where B is the excavation width.

82. The 38 support categories listed in Tables 14 - 17 have been specified to give estimates of permanent roof support since they were based on roof support methods quoted in the case histories. For temporary support determination, either Q is increased to 5Q or ESR is increased to 1.5 ESR.

83. The maximum limit for permanent unsupported spans can be obtained as follows (see also Figure 13):

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

84. Figure 14 shows the relationship between the rock mass quality Q and the stand-up time. In Figure 15, the relationship between Q and permanent support pressure P_{roof} is plotted from the following equation:

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

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

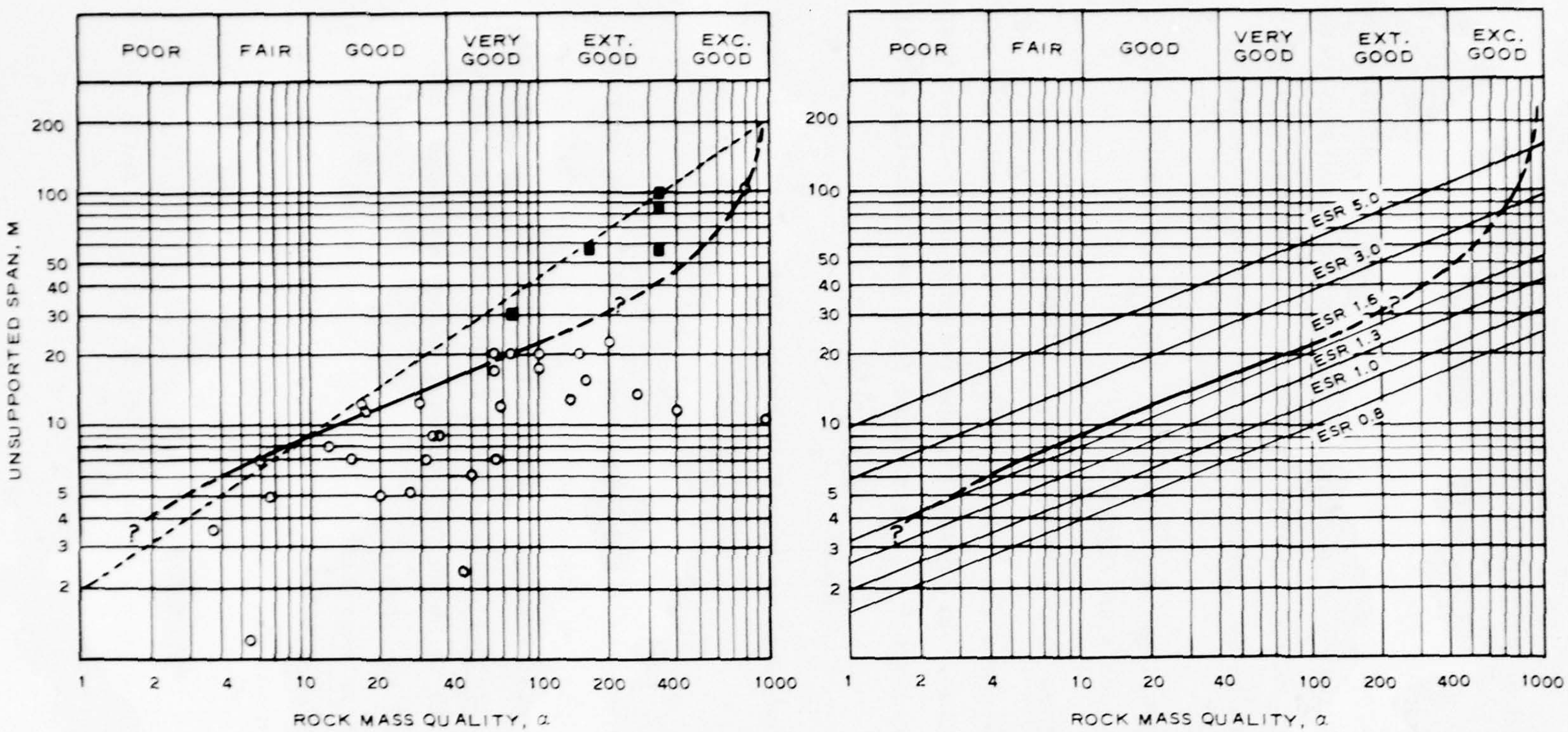
$$P_{roof} = \frac{2}{3} J_n^{1/2}\ J_r^{-1}\ Q^{-1/3}$$

85. The proposers of the Q-System emphasized[12] that while the support recommendations for the large-scale excavations would generally incorporate thicker shotcrete and longer bolts, the bolt spacing and theoretical support pressure would remain roughly the same. This is supported by Figure 16 in which roof support pressures range from 5 to 20 psi independent of the span.

86. When core is unavailable, the RQD is estimated[12] from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. The conversion for clay-free rock masses is

$$RQD = 115 - 3.3\ J_v$$

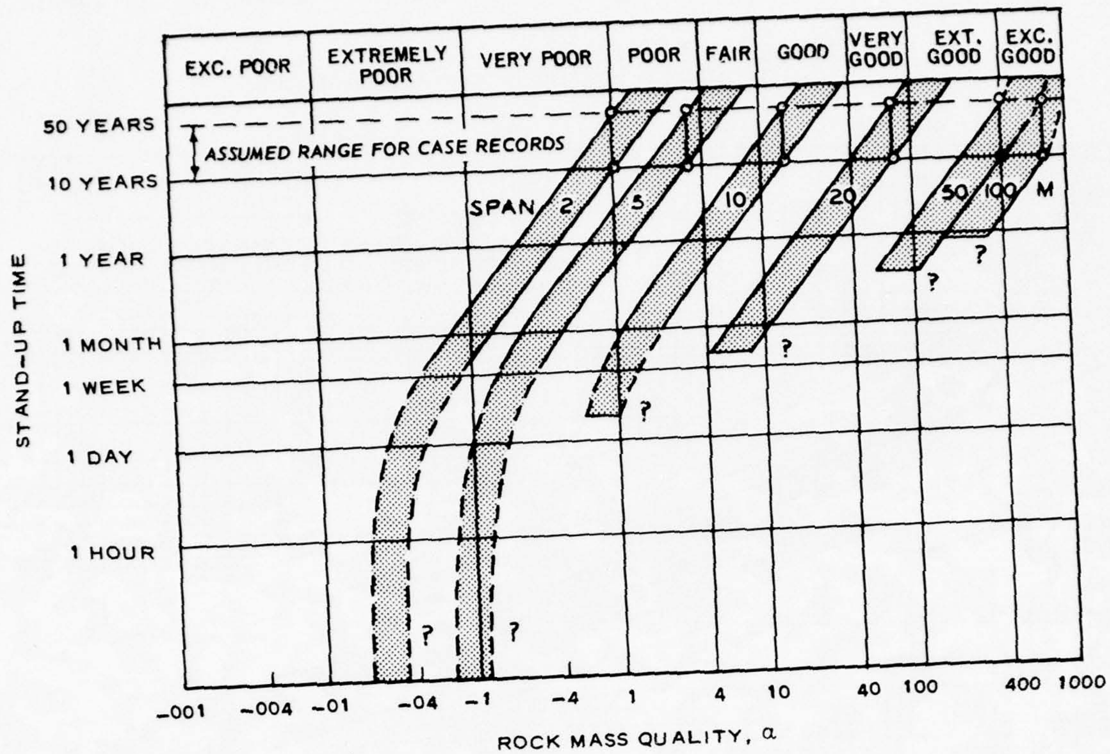
where J_v represents the total number of joints per cubic metre (RQD = 100 percent for $J_v < 4,5$).



a. CIRCLES REPRESENT THE MAN-MADE UNSUPPORTED EXCAVATIONS REPORTED IN THE LITERATURE. SQUARES REPRESENT SOME NATURAL OPENINGS FROM CARLSBAD, NEW MEXICO. THE CURVED ENVELOPE IS AN ESTIMATE OF THE MAXIMUM DESIGN SPAN FOR PERMANENTLY UNSUPPORTED MAN-MADE OPENINGS.

b. SUGGESTED DESIGN LIMITS FOR THE VARIOUS TYPES OF EXCAVATIONS LISTED IN PARA. 79. ESR VALUES GREATER THAN 1-6 APPLY TO TEMPORARY OPENINGS, FOR WHICH LESS STRINGENT REQUIREMENTS CONCERNING STAND-UP TIMES ARE ACCEPTABLE. SUPPORT MAY BE REQUIRED IF SPANS IN EXCESS OF THE RELEVANT DESIGN LIMITS ARE EXCAVATED.

Figure 13. Q-System - unsupported span versus rock mass quality (after Barton[43])



NOTE: THE ENVELOPES REPRESENT A PRELIMINARY ATTEMPT AT PREDICTING HOW MUCH THE STAND-UP TIME REDUCES WHEN THE SPAN OF AN UNSUPPORTED EXCAVATION IS INCREASED BEYOND THE MAXIMUM DESIGN SPAN (FIGURE 13).

Figure 14. Q-System — stand-up time versus rock mass quality

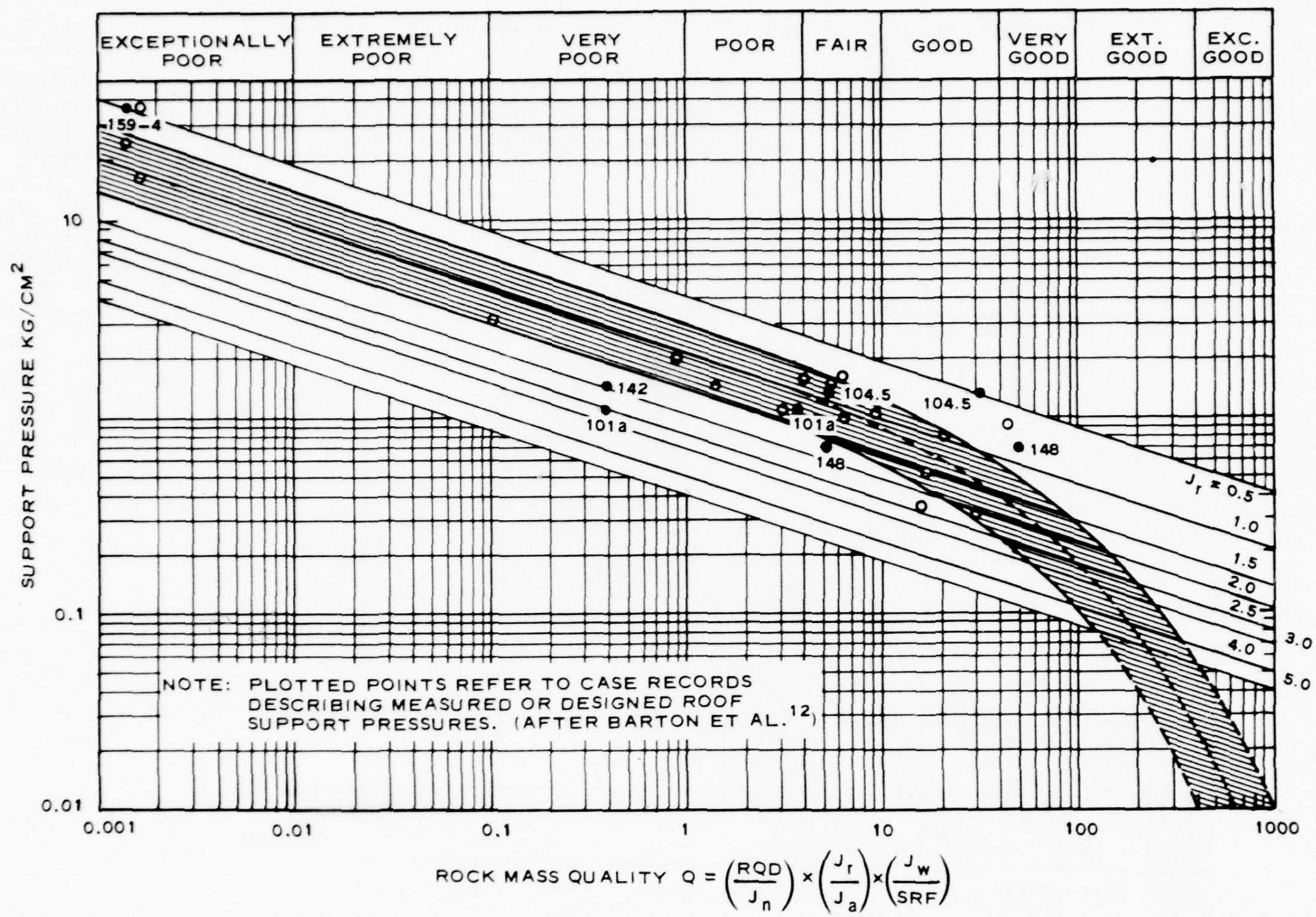


Figure 15. Q-System – support pressure versus rock mass quality

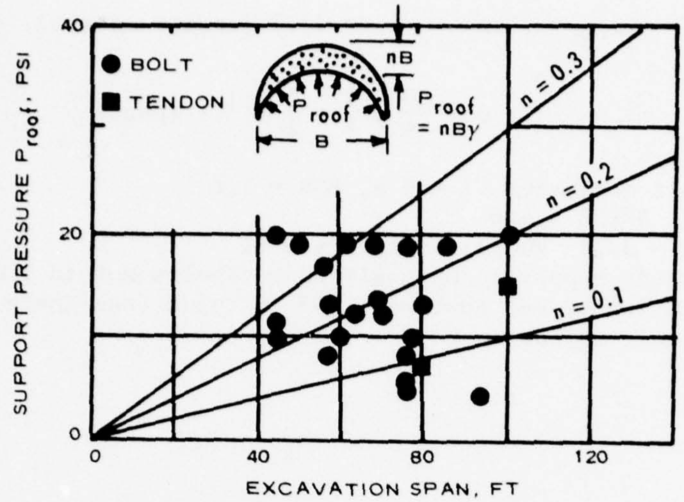


Figure 16. Design support pressures for roofs of large caverns (after Cording, Tiedemann, and Skinner[33])

87. The following steps are involved in applying the Q-System:

<u>a</u>. Classify the relevant rock mass quality.

<u>b</u>. Choose the optimum dimensions of excavation.

<u>c</u>. Estimate the appropriate permanent support.

88. A practical example using the Q-System is as follows:

Consider a water tunnel of 9-m (29.5 ft) span in a phyllite rock mass. The following is known:

Joint set 1: smooth, planar $J_r = 1.0$
chlorite coatings $J_a = 4.0$
15 joints per metre

Joint Set 2: smooth, undulating $J_r = 2$
slightly altered walls $J_a = 2$
5 joints per metre

Thus: $J_v = 15 + 5 = 20$ and $RQD = 115 - 3.3\ J_v = 50$ percent
$J_n = 4$
most unfavourable $J_r/J_a = 1/4$

Minor water inflows: $J_w = 1.0$

Uniaxial compressive strength of phyllite: $\sigma_c = 40$ MPa

Major principal stress: $\sigma_1 = 3$ MPa } Virgin stresses
Minor prinicpal stress: $\sigma_3 = 1$ MPa }

Thus: $\sigma_1/\sigma_3 = 3$ and $\sigma_c/\sigma_1 = 13.3$ (medium stress), SRF = 1.0

$$Q = \frac{50}{4} \times \frac{1}{4} \times \frac{1}{1} = 3.1 \text{ (poor)}$$

Support estimate: B = 9 m, ESR = 1.6
Thus: B/ESR = 4.6
For Q = 3.1: support category = 21
Permanent support: untensioned rockbolts spaced 1 m, bolt length 2.9 m, and shotcrete 2-3 cm thick (see Table 18, note 1)
Temporary support: none

PART III: GUIDE TO CLASSIFICATION PROCEDURES

89. The main rock mass classification systems currently in use in the design of rock tunnels were fully described in Part II. Apart from Terzaghi's classification, three other rock mass classification systems were shown to be most promising: the RSR Concept, the Geomechanics Classification, and the Q-System. Accordingly, the step-by-step design procedures will be summarized in this section for these three classification systems. For Terzaghi's classification, full guidelines are given in EM 1110-2-2901[31] and in Appendix A.

User's Guide for the RSR Concept

90. The RSR Concept, a ground support prediction model developed in the United States in 1973 by Wickham, Tiedemann, and Skinner,[5,6] is particularly suitable for selection of steel support for rock tunnels. It requires determination of the three parameters A, B, and C listed in Tables 2, 3, and 4.

Step 1. Divide the proposed tunnel route into geological regions, such that each region would be geologically similar and would require one type of support; i.e., it will not be economical to change tunnel support until rock mass conditions change distinctly, that is, a new structural region can be distinguished.

Step 2. Complete classification input data worksheet, as given in Appendix B, for each structural region.

Step 3. From Tables 2 to 4, determine the individual classification parameters A, B, and C and their sum, which gives the RSR = A + B + C.

Step 4. Adjust the RSR value in accordance with Figure 5 if the tunnel is to be excavated by a tunnel boring machine.

Step 5. Select a support requirement chart appropriate for the tunnel size, e.g., the chart for 10-, 20-, and 24-ft-diam tunnels in Figures 6, 7, and 8, respectively. These charts are applicable to both circular and horseshoe-shaped tunnels. From the selected chart, determine the rib type and spacing corresponding to the RSR value. Ignore curves for rockbolt and shotcrete support since they are not based on sufficient case history data.

Step 6. Estimate the rock load from Table 5 and the theoretical RR from the formula:

$$(RR + 80)(RSR + 30) = 8800$$

The values obtained are for comparison purposes between the structural regions.

User's Guide for the Geomechanics Classification

91. The Geomechanics Classification, which was developed in 1973 by Bieniawski,[13] enables determination of the RMR, the tunnel maximum unsupported span, the stand-up time, the support requirements, the in situ rock mass modulus, and the cohesion and friction of the rock masses.

Step 1. Divide the proposed tunnel route into structural regions, such that each region would be geologically similar and would require one type of support.

Step 2. Complete classification input data worksheet, as given in Appendix B, for each structural region (see paragraph 44).

Step 3. From Table 6, determine the ratings of the six individual classification parameters and the overall RMR value, following the procedure outlined in paragraphs 42 through 46 and 52 through 65.

Step 4. From Figure 9, determine the maximum unsupported rock span possible for a given RMR. If this span is smaller than the span of the proposed tunnel, the heading and bench or multidrift construction should be adopted (see paragraphs 71 and 72).

Step 5. From Figure 9, determine the stand-up time for the proposed tunnel span. If the tunnel falls below the lower limit line, no support will be required. If the stand-up time is not sufficient for the life of the tunnel, the appropriate support measures must be selected.

Step 6. From Table 8, select the appropriate tunnel support measures and note that these represent the permanent support.

Step 7. If foundation design is contemplated for nearby structures, select from Figure 10 the in situ modulus of deformation of the rock mass (see paragraphs 66 and 67).

Step 8. If the rock slopes near the tunnel portals are to be designed, select from Section D of Table 6 the cohesion and friction data (see paragraph 68).

Step 9. Consider a monitoring program during the tunnel construction for sections requiring special attention (see paragraph 69).

User's Guide for the Q-System

92. The rock mass quality Q-System, which was developed in Norway in 1974 by Barton, Lien, and Lunde,[12] enables the design of rock support in tunnels and large underground chambers.

Step 1. Divide the proposed tunnel route into structural regions, such that each region would be geologically similar and would require one type of support category.

Step 2. Complete classification input data worksheet, as given in Appendix B, for each structural region.

Step 3. Determine the ratings of the six classification parameters from Tables 11, 12, and 13 and calculate the Q value (see paragraph 75).

Step 4. Select the excavation category from paragraph 79 and allocate the ESR.

Step 5. From Figure 12, determine the support category for the Q value and the tunnel span/ESR ratio.

Step 6. From Tables 14 through 18, select the support measures appropriate to the support category. Calculate the length of rockbolts from paragraph 81.

Step 7. The selected support measures are for the permanent support. Should it be required to determine the primary support measures, consult paragraph 82.

Step 8. For comparison purposes, determine the support pressure from paragraph 85.

Step 9. For record purposes, from Figures 13 and 14, estimate the possible maximum unsupported span and the stand-up time.

Comparison of Procedures

93. For convenience of application, practical examples for using each of the three classification systems are given in paragraphs 41, 70, and 88. A detailed discussion of a selected case history, giving comparisons between Terzaghi's approach and the three classifications, follows in Part IV. It is appropriate, however, to consider here if any relationships or comparisons exist between the three classification systems.

94. A correlation has been attempted between the Geomechanics RMR and the Q-value.[23] A total of 111 case histories were analyzed involving 68 Scandinavian cases, 28 South African cases, and 21 other documented case histories from the United States, Canada, Australia, and Europe. The results are plotted in Figure 17 from which it will be seen that the following relationship is applicable:

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

Rutledge[18] recently determined in New Zealand the following correlations between the three classification systems:

$$RMR = 13.5 \log Q + 43 \quad \text{(standard deviation = 9.4)}$$

$$RSR = 0.77\ RMR + 12.4 \quad \text{(standard deviation = 8.9)}$$

$$RSR = 13.3 \log Q + 46.5 \quad \text{(standard deviation = 7.0)}$$

95. A comparison of the stand-up time and the maximum unsupported span, as shown in Figures 9, 13, and 14, reveals that the Geomechanics Classification is more conservative than the Q-System, which is a reflection of the different tunneling practice in Scandinavia based on the generally excellent rock and the long experience in tunneling.

96. A comparison of the support recommendations by six different classification systems is given in Table 1. Other comparisons are made in References 17, 18, 23, 27, 28, and 29.

97. Although the above comparisons are interesting and useful, it is believed that one should not necessarily rely on any one classification system but should conduct a sensitivity analysis and cross-check

the findings of one classification with another. This could enable a better "feel" for the rock mass.

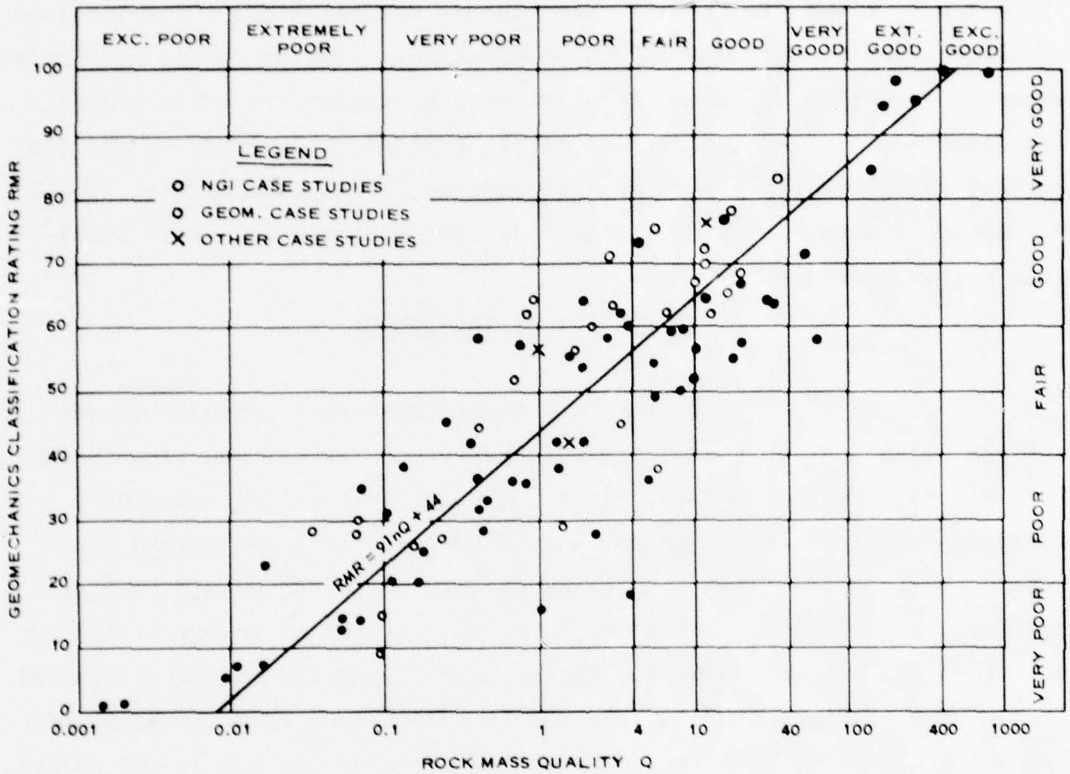


Figure 17. Correlation between Geomechanics Classification and Q-System

PART IV: CASE HISTORY OF THE PARK RIVER TUNNEL

98. In order to demonstrate the potential of the tunnel design by rock mass classifications a case history was selected. This involved the Park River Tunnel in Hartford, Connecticut, a water tunnel currently under construction by the U. S. Army Corps of Engineers. This project was selected, because the details of the geological exploration and the current design practice were well documented,[44] and even in situ stress measurements were conducted.[45] In addition, borehole logs were available for examination.

Description of the Tunnel

99. The function of the Park River (auxiliary conduit) Tunnel will be to conduct approximately one quarter of the maximum flow in the Park River to the Connecticut River. The completed tunnel will have a 22-ft inside diameter and extend some 9100 ft between the intake and outlet shafts. It will be excavated through shale and basalt rock at a maximum depth of 200 ft below the surface. The tunnel invert at the outlet shaft is 52 ft below the intake invert with the tunnel sloping at a rate of approximately 7 in. per 100 ft. A minimum rock thickness of approximately 50 ft will remain above the crown excavation at the outlet.

100. The 22-ft-diam tunnel will be machine bored and lined throughout with precast reinforced concrete segments 9 in. thick. For drill and blast construction, the initial design specified the minimum thickness of a cast-in-place reinforced concrete liner as 14 in. (Plate 9A-21 of Reference 44) with additional 8 in. being allowed to the excavation pay line. Thus, the minimum expected concrete thickness would be 22 in. giving the nominal excavation size of 25.7 ft. This nominal excavation size would increase to 27.7 ft where heavy structural support was expected with the concrete liner stipulated as 22 in. thick.

101. Temporary rock support was prescribed for the entire length of the tunnel in the case of the construction by drilling and blasting. Typical support patterns (for 88 percent of the tunnel) would be

1-1/8-in.-diam rock anchors (rockbolts fully resin bonded but not tensioned), 11 ft long, spaced 4-1/2 ft with shotcrete 1 in. thick without wire mesh. In poor ground condition, the bolt spacing is between 2 and 4 ft with shotcrete 2 in. thick. In two fault zones, expected to be approximately 300 ft long, structural W8 steel ring beams at 3 ft will be used.

102. The anticipated cost of the tunnel is $17.0 million for machine boring or $1880 per foot, based on bid prices. If conventional drill and blast construction were used, the cost would have been $27.8 million (including the shafts).

Tunnel Geology

103. In Figure 18, a longitudinal geological section of tunnel is shown. The rocks along the alignment are primarily easterly dipping Triassic sandy red shales/siltstones interrupted by a zone of basalt flows and some limited rock types near the basalt. Bedding is distinct and often regular to the extent that many marker beds correlated between boreholes. Descriptions of the various rock types are given in Table C1, Appendix C.

104. Three main geological zones were distinguished along the tunnel route:[44]

a. Shale and basalt zones, constituting 88 percent of the tunnel.

b. Fractured rock zone (very blocky and seamy), between sta 23 + 10 and 31 + 10 (800 ft).

c. Two fault zones, one near sta 57 + 50 and the other between sta 89 + 50 and 95 + 50.

105. Bedding and jointing are generally north to south which is perpendicular to the tunnel axis (tunnel will run west to east). The bedding is generally dipping between 10 and 20 deg while the joints are steeply dipping between 70 and 90 deg. Joints in the shale have rough surfaces, and many are very thin and healed with calcite.

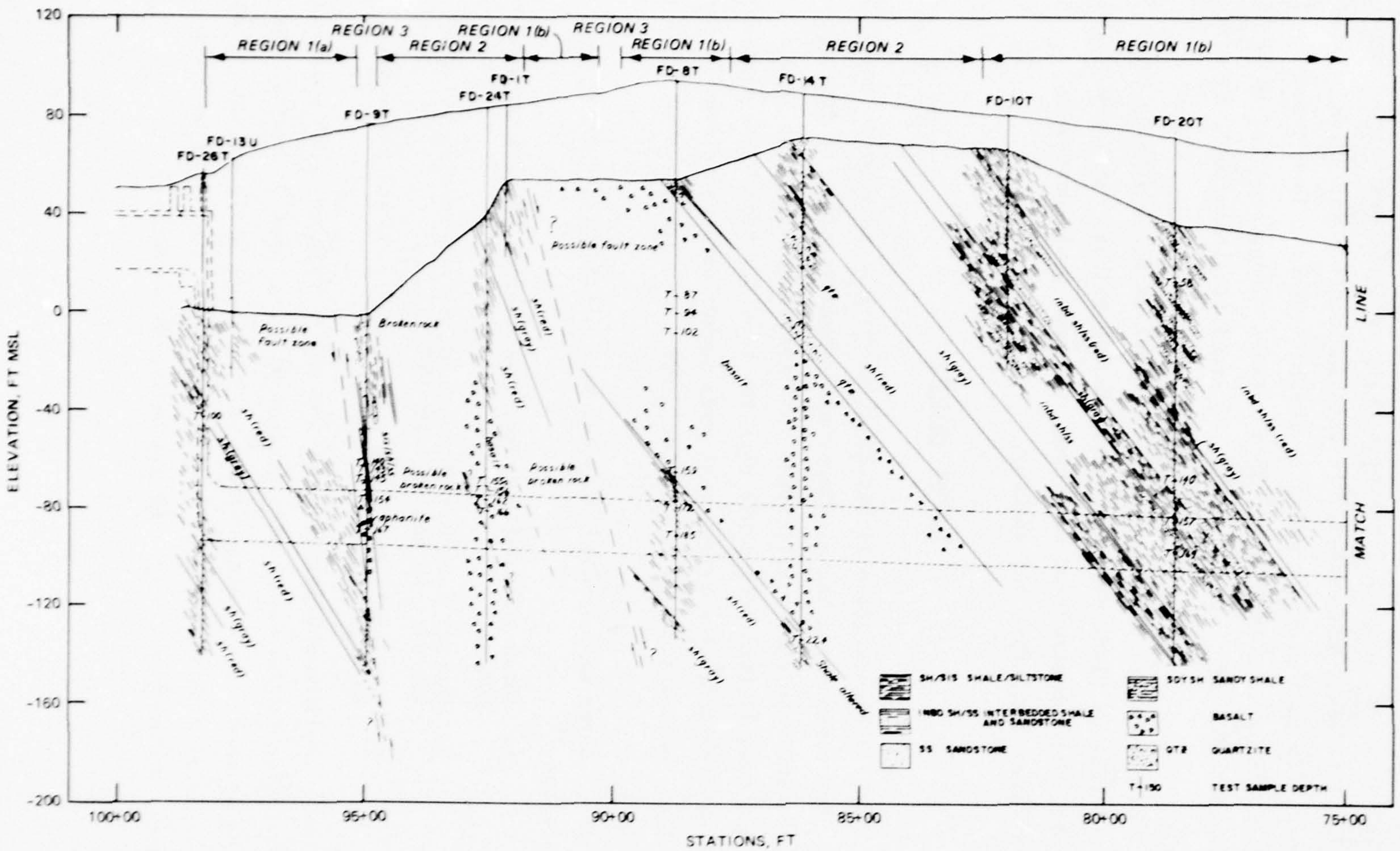


Figure 18. Geologic profile of Park River Tunnel (sheet 1 of 4)

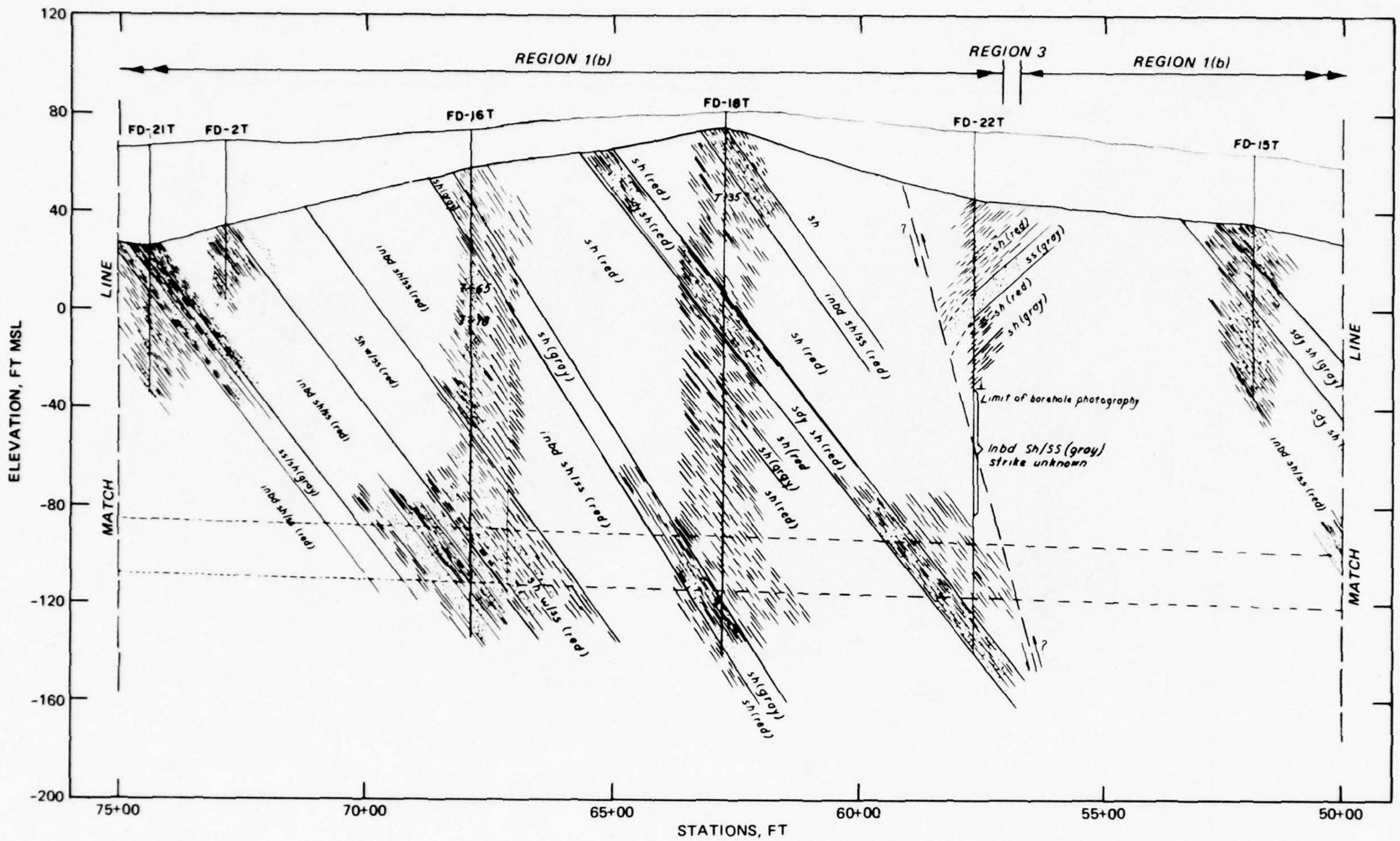


Figure 18 (sheet 2 of 4)

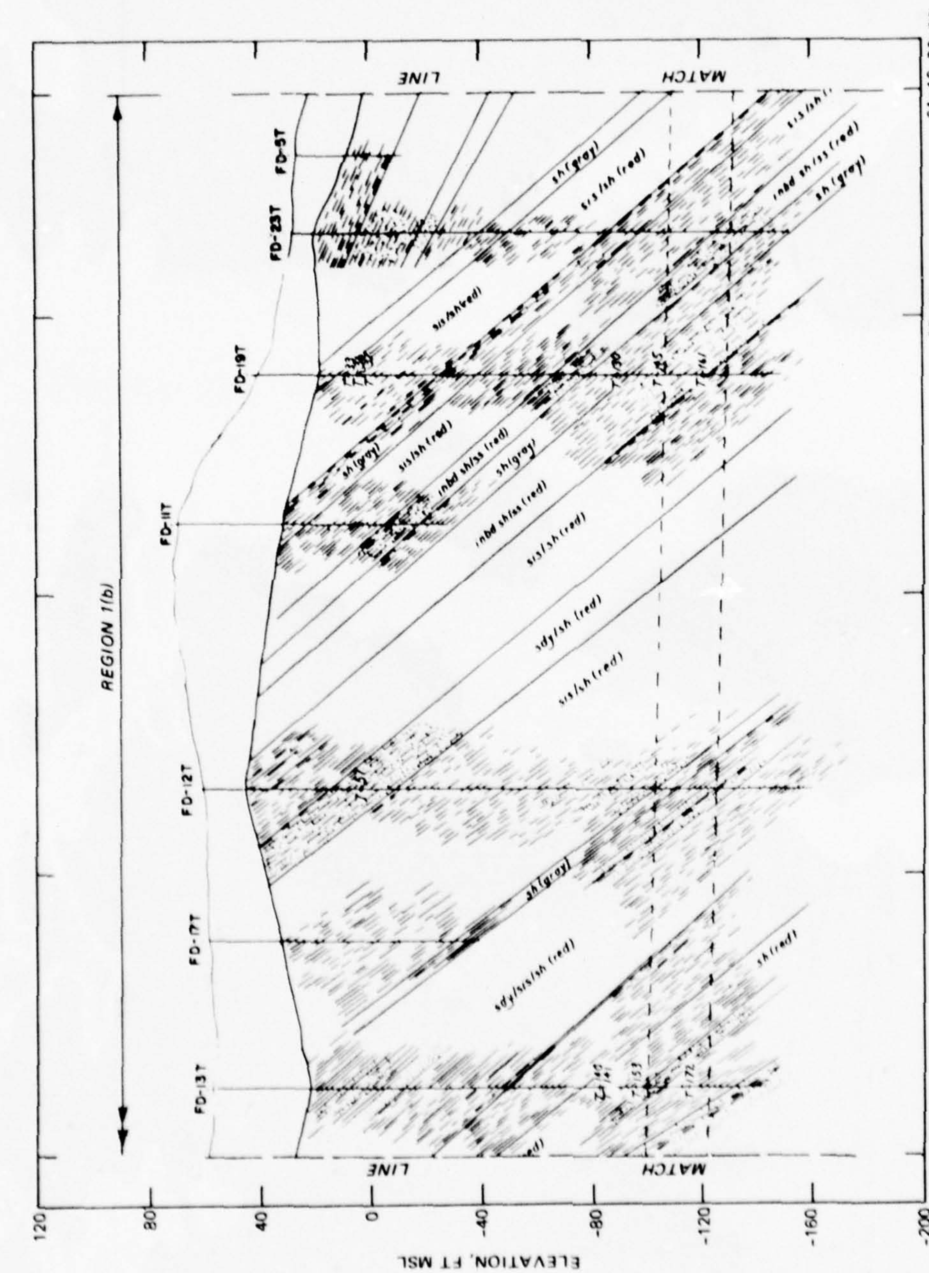


Figure 18 (sheet 3 of 4)

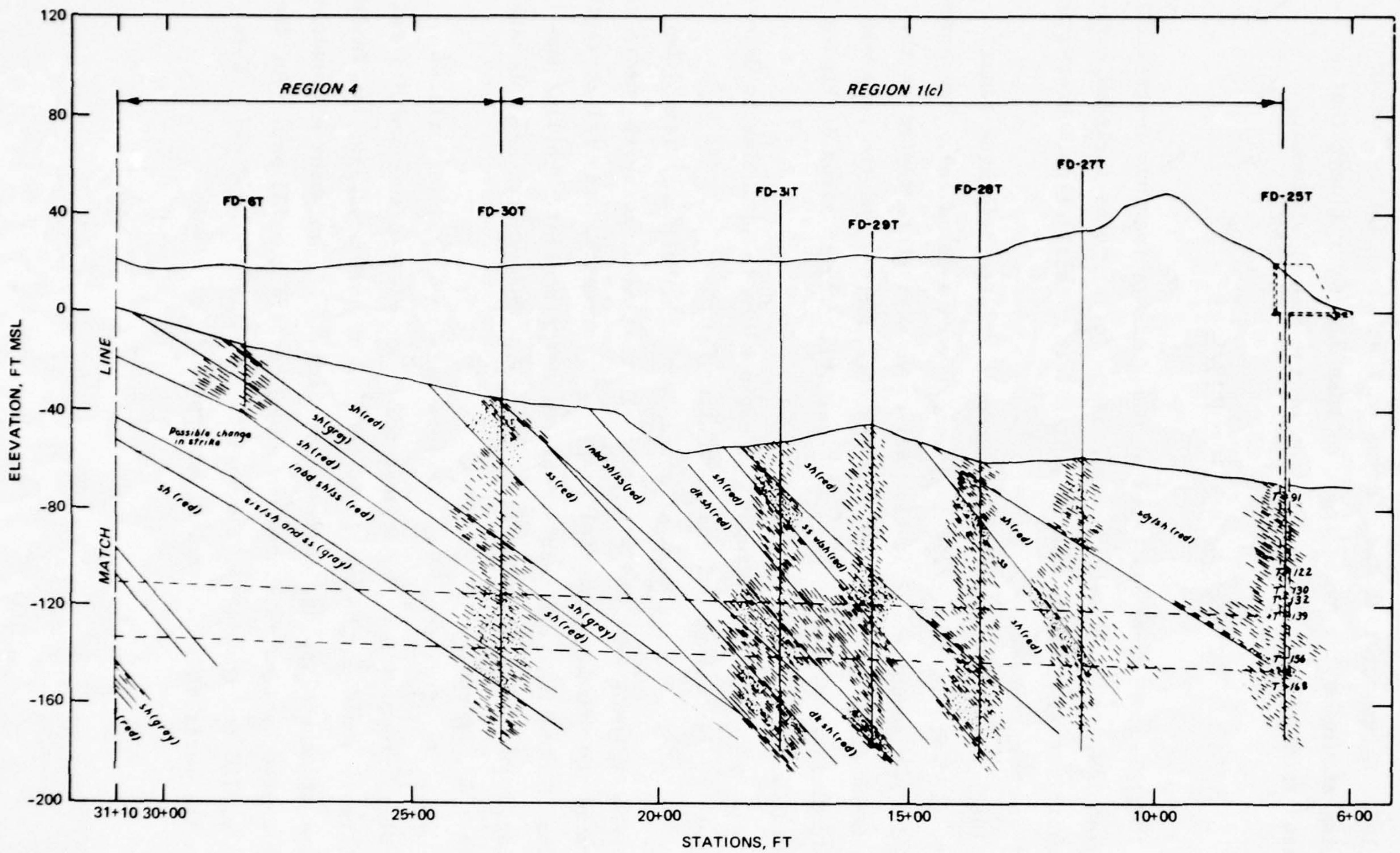


Figure 18 (sheet 4 of 4)

106. Groundwater studies indicated that water inflow during tunneling should be low averaging less than 3 gpm per linear foot of untreated tunnel for the major portion of the tunnel alignment.

Geological Investigations

107. Explorations consisted of core borings, various tests within the boreholes, and a seismic survey. Tests in boreholes included borehole photography, pressure testing, piezometer installation, observation wells, and pump tests.

108. Rock cores from 29 borings were used to determine tunnel geology (18 were NX diam (2.16 in.) and 11 were 4-in. diam). Ten boreholes did not reach tunnel level. All cores were photographed in the field immediately upon removal from the core barrel, and the core was logged, classified, and tested. Typical drill log is given in Figure C1, Appendix C.

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

110. Core samples were selected from 21 localities within the tunnel, near the crown, and within one-half diameter above the crown to determine the density, uniaxial compressive strength, triaxial strength, modulus of elasticity, Poisson's ration, water content, swelling and slaking, sonic velocity, and joint strength. The results are tabulated in Table C2, Appendix C.

111. In situ stress measurements were conducted in vertical boreholes[45] involving 15 tests, but only four yielded successful results. Eight tests could not be completed because of core breakage; two failed because of gage slipping, and two more because of equipment malfunction. The measured horizontal stress was found to be 452 ± 133 psi. For the depth of 120 ft, the vertical stress is calculated as 132 psi. This gives the horizontal to vertical stress ratio as 3:42.

Bieniawski's Report

112. Input data to enable rock mass classification by the RSR Concept, the Geomechanics Classification, and the Q-System are listed in Figures C2 through C4, Appendix C. The data are presented for each structural region anticipated along the tunnel route. The best average ground condition (Table 23) was subdivided into two separate regions, basalt zones and shale zones. Station limits for each zone are shown in Figure 18.

Input Data for Rock Mass Classifications

113. Input data to enable rock mass classifications by the RSR Concept, the Geomechanics Classification, and the Q-System are listed in Figures C2 through C7, Appendix C. The data are presented for each structural region anticipated along the tunnel route.

114. It should be noted that all the data entered on the classification input sheets have been derived from the borings, including information on joint orientation and spacing. This was possible because borehole photography was employed for borehole logging in addition to the usual core logging procedures. However, considerable effort was required in extracting the data from the geological report for the classification purposes since engineering geological information was not systematically summarized in the form of classification input work sheets.

Assessment of Rock Mass Conditions by Classifications

115. Rock mass classifications in accordance with the Terzaghi Method, the RSR Concept, the Geomechanics Classification, and the Q-System are performed in Tables 19, 20, 21, and 22, respectively, and are summarized in Table 23.

Tunnel Design Features

116. Based on the geological information, the design of the tunnel recognizes the following features, with reference to the geological profile in Figure 18:

a. Nominal support (8000 ft): good rock, best average conditions, RQD > 80 percent, water inflow 1 gpm per foot of tunnel.

b. Heavy support (800 ft): sta 23 + 10 to 31 + 10. The tunnel intersects an area of thin rock cover and thick overburden, and rock conditions at tunnel grade are described as very blocky and seamy. The rock is not tight, dipping 7 to 14 deg, and water inflows of 4 gpm per foot of tunnel are anticipated.

c. Steel support in fault zones (300 ft): sta 93 + 50 to 95 + 50 and 56 + 00 to 57 + 00. Broken rock is assumed due to faulting, dipping between 20 and 60 deg, and a low RQD of 30 percent. Pressure tests showed water inflows of 15-20 gpm per foot of tunnel.

117. The above rock conditions are summarized in Table 19. The designers believe (Reference 44, p. 21) that the actual conditions will exceed the best average condition in most of the tunnel. If machine excavation is employed, the rock load factors are expected to be reduced by as much as 50 percent in the major portion of the tunnel.

118. Excavation conditions are expected to depend on the construction method selected. Control of water inflow and slaking for conventional excavation will be provided by shotcrete without mesh, but no shotcrete is anticipated if the construction is by tunnel machine boring with precast lining. The grouted lining will provide the necessary control for reducing water inflow and any spalling near the face. In any case, only relatively low water inflow was indicated by pressure and pump test data.

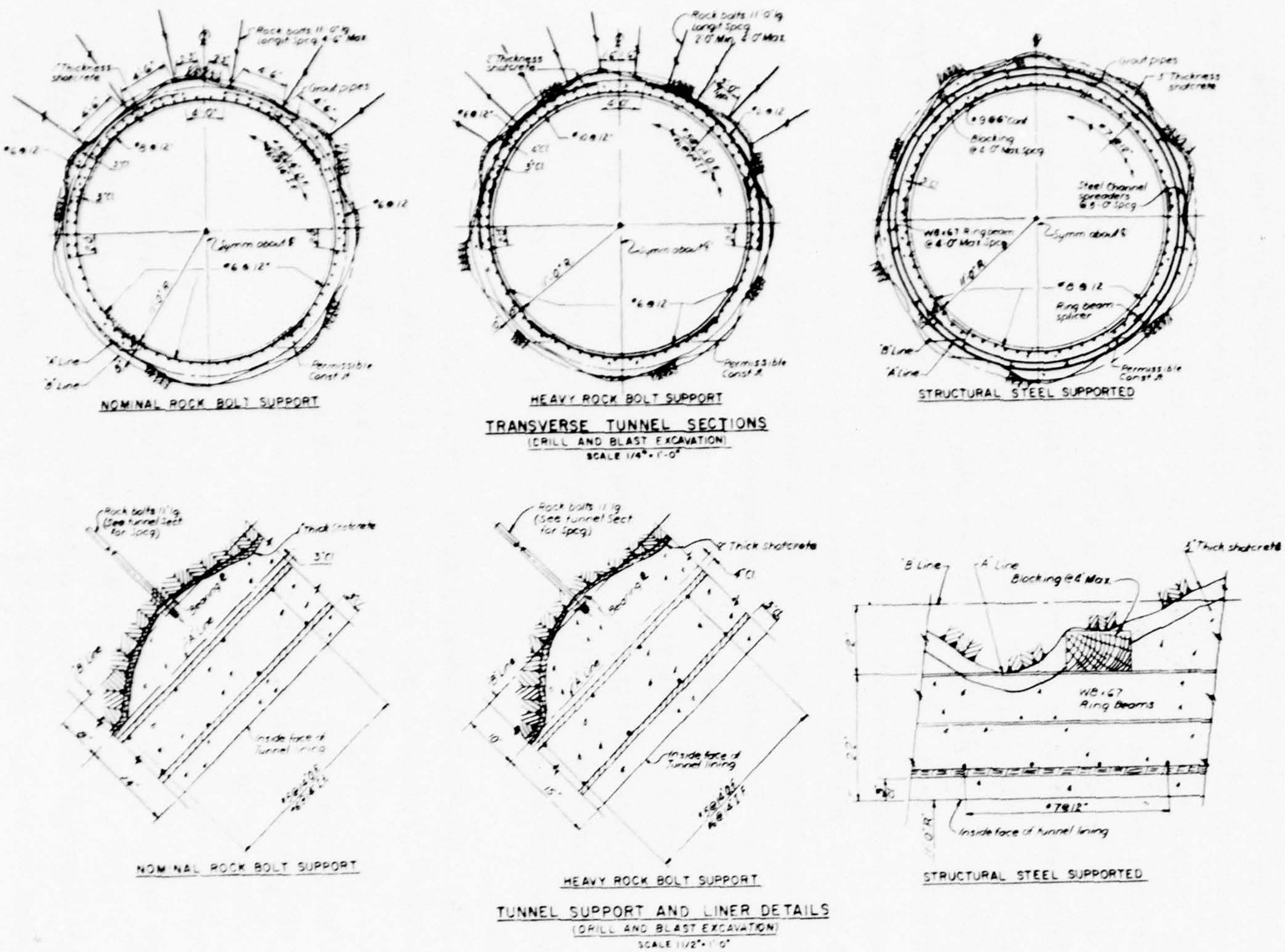
119. Geologic conditions at tunnel grade are considered suitable for machine boring of the tunnel accompanied by precast tunnel lining. Because of immediate installation of the lining, the tunnel would drain less water under the city since a drill and blast tunnel will stand for

up to one year before a permanent lining is installed. Machine excavation would also cause less vibrations. The anticipated cost of machine excavation with precast segments is $17.0 million while the cost of conventional tunneling would be $27.8 million (including the shafts). With respect to Figure 19, note that payment for concrete is up to line "B" for conventional tunneling but only up to line "A" (the minimum excavation line) for machine tunneling.

120. The envisaged tunnel designs for each of the three ground conditions are shown in Figure 19. The details of the recommended primary (temporary) support and the final lining for drill and blast construction are presented in Figure 19a. The basic design was based on the Terzaghi Method. Temporary rock supports will be required only for the cast-in-place alternative and will provide the primary rock support for up to one year prior to placement of the permanent lining. For machine tunneling, this will not be necessary (Figure 19b).

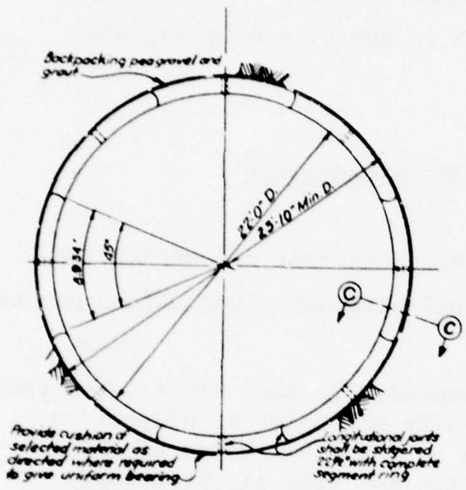
121. As the tunnel will be completely full when in operation, the design of the tunnel liner assumed a pressure of 15 psi for contact grouting, which would ensure that the liner remains in compression under net internal load conditions. Grouting will be applied to the full ring. For purposes of analyzing stresses in the concrete liners, a coefficient of subgrade reaction of 1000 kci (580 pcf) for the rock was assumed.

122. Tunnel instrumentation is planned to provide for design verification, future design applications, and monitoring of construction effects. Ten test sections at locations based on differing geologic or design conditions will be installed throughout the length of the tunnel. These test sections will have instruments tailored to the test areas but will consist of 10 extensometers (MPBX's) installed from the surface and pore pressure transducers, rockbolt load cells, convergence points, and surface and embedded strain gages installed within the tunnel. Furthermore, in situ stresses will be determined using the overcoring technique. The test sections have been arranged to provide the greatest amount of data based on the planned construction schedule of a TBM with precast

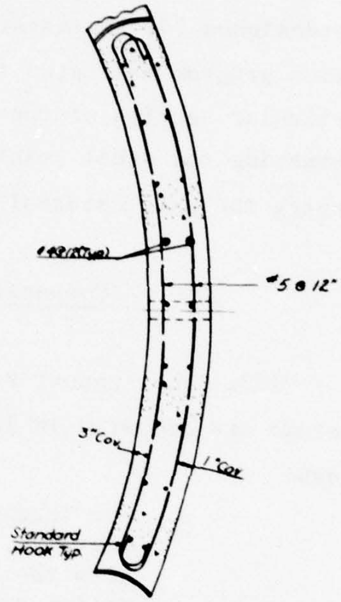


a. CAST-IN-PLACE CONCRETE

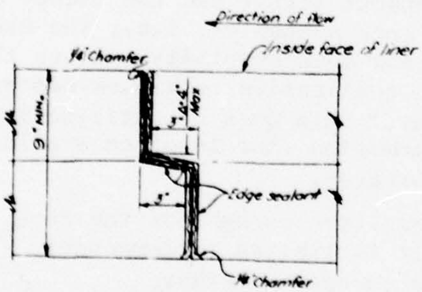
Figure 19. Details of tunnel support (sheet 1 of 2)



TRANSVERSE TUNNEL SECTION
PRECAST LINER ALTERNATE
SCALE 1/4" = 1'-0"



REINFORCEMENT DETAILS
PRECAST LINER
SCALE 1" = 1'-0"



SECTION C-C
TRANSVERSE JOINT DETAIL
SCALE 3" = 1'-0"

NOTES:

1. Design of precast segments based on a compressive strength of 5000 pounds per square inch at 28 days.
2. The grouting procedure will have to be carefully monitored during the construction phase to insure uniform pressures throughout the cross section.

b. PRECAST LINER

Figure 19 (sheet 2 of 2)

lining. Since the precast segments are designed for the worst ground conditions but are utilized throughout the tunnel, they are in effect overdesigned for the major portion of the tunnel. If the instrumentation program indicates that higher strength units are needed for a particular section of the tunnel, the design could be modified by increasing the steel reinforcement, which is now at a minimum, and keeping the same external shape.

Comparison of Support Recommendations

123. The support recommendations based on four classification systems are compared in Table 23. The following main conclusions may be drawn:

a. The Terzaghi Method recommends the most extensive support measures, which seem clearly excessive by comparison with the recommendations by the other three classification systems. The reason for this is three-fold: (1) the current permanent lining design does not account fully for the action of the temporary support, which in itself may be sufficient for the structural stability of the tunnel; (2) the original recommendations by Deere et al.[4] were based on the 1969 technology, which is now much outdated; and (3) not enough use is made of the ability of the rock to support itself and the recent progress in the field of rock mechanics, i.e., the use of monitoring to assess rock mass stability. Since the Terzaghi Method uses such qualitative rock mass descriptions as "blocky and seamy," this does not utilize fully all the quantitative information that is often available from a site exploration program.

b. The RSR Concept is not sensitive enough for the rock conditions encountered; it is limited to temporary support only and for steel support design.

c. Both the Geomechanics Classification and the Q-System give fairly similar recommendations, and any differences in support prediction by these two methods will enable the designer to exercise a better engineering judgment.

d. The final concrete lining for drill and blast construction could possibly be reduced by 6 in., which would result in savings of $2 million ($650,000 per 2 in. of concrete). Since a monitoring program is planned, this recommendation would not be hazardous to the tunnel safety.

PART V: RESEARCH REQUIREMENTS

124. The present study has revealed a number of aspects in the present tunnel design practice, which could benefit from further research. It is believed that improved tunnel design procedures, for the construction of safe and more economical rock tunnels, would result in the following areas:

a. If a better and more systematic engineering geological description of the rock mass conditions is provided, e.g., in accordance with the input data sheets listed in Appendix B.

b. If there is a better communication and understanding among all the persons concerned with a tunneling project.

c. If the current tunnel design practice, which is based on the revised Terzaghi Method[34], is supplemented by the methods advocated by the more modern rock mass classification systems, such as the Geomechanics Classification, the Q-System, and the RSR Concept. These classification systems make full use of the quantitative data from site investigations. No one classification system should necessarily be singled out to the exclusion of the others; instead a cross-check of the results should be aimed for.

d. If the action of the temporary support (otherwise known as the primary support) is fully incorporated into the design of the permanent lining, the thickness and the reinforcement of the latter could be greatly reduced without endangering the safety of the tunnel.

e. If during the tunnel construction a more comprehensive tunnel-monitoring program could be incorporated, similar to the procedures generally envisaged for the so-called New Austrian Tunneling Method (NATM), not only the adopted design could be verified but a safe and more economical tunnel construction would be ensured.

f. If the reinforced concrete linings are replaced by shotcrete and mesh linings in the case of rock tunnels, other than possibly water conduits. However, even water tunnels are sometimes left unsupported.[46]

g. If more research is conducted into the stand-up time of unsupported as well as variously supported rock spans, more confidence could be placed in the predictions from the rock mass classification systems.

h. If more carefully documented tunnel case histories are compiled featuring comparisons between support designs based on different methods, better understanding of design concepts will be achieved.

125. Some of the above requirements deserve further elaboration. Thus, item a above means that sometimes even when a well-planned geological investigation has been conducted, the data presentation is not well compiled so that much additional time is needed by the rock engineer to extract the parameters needed for design. The use of the worksheets given in Appendix B would greatly simplify the input data collection.

126. For a better communication on a tunneling project, a training program is called for to ensure that the geologists understand the engineers' requirements and that the engineers make it clear as to what is needed and why for design purposes.

127. The NATM technique has a number of possible interpretations and constitutes a study on its own. It should be reviewed in detail and compared with the current tunnel design procedures.

128. The concept of the temporary and permanent support appears quite outdated in view of the current rock engineering technology and its use leads to the overdesign of tunnels. The concept could be reexamined without endangering tunnel safety, because any reduction in tunnel support can be backed by a suitable rock monitoring program.[47]

129. The relationship between the stand-up time and the rock span requires verification from actual case histories in the United States, and a research program directed to this aspect would make a great contribution in the field of rock tunneling. In the Corps of Engineers tunnel research program, there is a mechanism for how this could be achieved since Work Unit No. 31560 calls for preparation of tunnel design revisions by September 1981.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

130. The current rock tunnel design practices do not utilize the latest rock mass classification systems. These systems, such as the RSR Concept, the Geomechanics Classification, and the Q-System, offer a realistic and valuable alternative to the tunnel design procedures based on the Terzaghi (steel support) Method.

131. There is a need for more research in a number of areas of rock tunnel design, and some recommendations are given below.

132. Case histories are not easy to compile due to the lack of sufficient information, both concerning the geology and the design, and yet they constitute a most valuable source of practical knowledge.

Recommendations

132. Based on this study, the following recommendations are made:

a. The current tunnel design practices should be supplemented by the approaches advocated by such rock mass classification systems as the Geomechanics Classification, the Q-System, and the RSR Concept. Tunnel support recommendations by all these systems should be systematically compared on all tunneling projects.

b. Engineering geological description of rock masses for tunneling purposes should be compiled in accordance with the data worksheets given in Appendix B. This would greatly facilitate a more effective documentation of tunnel case histories.

c. A training program for engineering geologists and tunnel engineers should be initiated to ensure a better communication on tunneling projects.

d. The principles and potential of the NATM, as the prime example of an observational tunnel design approach, should be investigated as a systematic study and compared with the other design approaches.

e. Research should be initiated into three areas:

(1) The interaction of the temporary and permanent support measures.

(2) The relationship between the stand-up time and unsupported, as well as supported, rock spans.

(3) Systematic documentation of tunnel case histories for comparison of rock conditions, support design, and construction experience.

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Table 1

Comparison of Rock Mass Classifications Applied at the Overall Tunnel (Width 5,5 m)

Locality	Geomechanics Classification (Bieniawski, 1973[3])		NGI Classification: Q-System (Barton, 1974[12])		RSR Classification (Wickham, 1972[5])	
	Class	Support	Class	Support	Class	Support
H 6	I Very good rock RMR = 83	Occasional spot bolting.	Good rock Q = 33,0	Spot bolting only	RSR = 68	Bolts 25 mm dia. at 2 m (length not given)
H 4	II Good rock RMR = 67	Locally, grouted bolts (20 mm dia.) spaced 2-2,5 m, length 2,5 m plus mesh; shotcrete 50 mm thick if req.	Good rock Q = 12,5	Systematic grouted bolts (20 mm dia.) spaced 1 m - 2 m; length 2,8 m.	RSR = 60	Bolts spaced 1,4 m, shotcrete 35-45 mm or medium ribs at 2 m
H 2	III Fair rock RMR = 52	Systematic grouted bolts spaced 1,5-2 m, length 3 m plus mesh and 100 mm thick shotcrete.	Fair rock Q = 8,5	Systematic grouted bolts spaced 1,5 m, length 2,8 m; and mesh	RSR = 57	Bolts spaced 1,2 m and 50 mm shotcrete or ribs 6H20 at 1,7 mm
H 3	IV Poor rock RMR = 29	Systematic grouted bolts spaced 1-1,5 m, length 3 m, mesh plus 100-150 mm shotcrete (ribs at 1,5 m).	Poor rock Q = 1,5	Shotcrete only: 25-75 mm thick or bolts at 1 m, 20-30 mm shotcrete and mesh.	RSR = 52	Bolts spaced 1 m and 75 mm shotcrete or ribs 6H20 at 1,2 m.
H 5	V Very poor rock RMR = 15	Systematic grouted bolts spaced 0,7-1 m, length 3,5 m, 150-200 mm shotcrete and mesh plus medium steel ribs at 0,7 m. Closed invert.	Extremely poor rock Q = 0,09	Shotcrete only: 75-100 mm thick or tensioned bolts at 1 m plus 50-75 mm shotcrete and mesh.	RSR = 25	N/A*
	RQD Classification (Deere, 1969[2])		Austrian Classification (Rabcewicz/Pacher, 1974[10])		French Classification (Louis, 1974[11])	
H 6	Excellent RQD > 90	Occasional bolts only.	I Stable	Bolts 26 mm dia., 1,5 m long spaced 1,5 m in roof plus wire mesh.	A	50-mm shotcrete or 3 m long bolts at 3,1 m.
H 4	Good RQD: 75-90	Bolts 25-mm dia., 2 m-3 m long spaced 1,5-1,8 m and some mesh or 50-75 shotcrete or light ribs.	II Over-breaking	Bolts 2-3 m long spaced 2-2,5 m, shotcrete 50-100 mm with mesh.	B	100 mm shotcrete with mesh and 3 m bolts at 2,8 m.
H 2	Fair to good RQD: 50-90	Bolts 2 m-3 m long at 0,9-1 m plus mesh or 50-100 mm shotcrete or light/medium ribs at 1,5 m.	III Fractured to very fractured	Perfo-bolts 26 mm dia., 3-4 m long spaced 2 m plus 150 mm shotcrete plus wire mesh and steel arches TH16 spaced 1,5 m.	C	150 mm shotcrete with mesh and 3 m bolts at 2,5 m.
H 3	Poor RQD: 25-50	Bolts 2 m-3 long at 0,6-1,2 m with mesh or 150 mm shotcrete with bolts at 1,5 m or medium to heavy ribs.	IV Stressed rock	Perfo-bolts 4 m long, spaced 1 m by 2 m plus 200 mm shotcrete plus mesh plus steel arches TH21 spaced 1 m. Concrete lining 300 mm.	D	210 mm shotcrete with mesh and 3 m bolts at 2 m and steel ribs.
H 5	Very poor RQD < 25	150 mm shotcrete all around plus medium to heavy circular ribs at 0,6 m centres with lagging.	V Very stressed rock	Perfo-bolts 4 m long spaced 1 m plus 250 mm shotcrete plus mesh and steel arches TH29 spaced 0,75 m. Closed invert. Concrete lining 500 mm.	E	240 mm shotcrete with mesh and 3 m bolts at 1,7 m; steel ribs at 1,2 m. Closed invert.

* Not applicable.

Table 2

Rock Structure Rating - Parameter A

Rock Structure Rating
Parameter "A"
General Area Geology

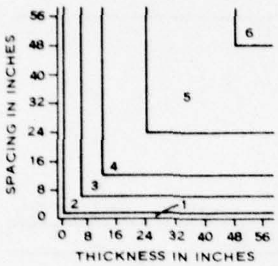
Max. Value 30

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

Rock Structure Rating - Parameter B



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

* Dip: flat - 0 to 20 deg; dipping - 20 to 50 deg; and vertical - 50 to 90 deg.

Table 4

Rock Structure Rating - Parameter C

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.

Table 5

Correlation of Rock Structure Rating to Rock Load and Tunnel Diameter

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

Table 6

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 10 m tunnel length	None		<25 litres/min	25 - 125 litres/min	>125 litres/min		
		OR			OR	OR	OR		
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0		0.0 - 0.2	0.2 - 0.5	>0.5		
		OR			OR	OR	OR		
		General conditions	Completely dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems		
	Rating		10		7	4	0		

B. Rating Adjustment for Joint Orientations

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
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 7

<u>Effect of Joint Strike and Dip Orientations in Tunneling</u>

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 favorable	Favorable	Fair	Unfavorable	Very unfavorable	Fair	Unfavorable

Table 8

Geomechanics Classification Guide for Excavation and Support of Rock Tunnels

(Tunnel Widths: 20-40 ft, Construction: Drilling and Blasting)

Rock Mass Class	Excavation	Support		
		Rockbolts* (Length: 1/3 to 1/2 Tunnel Width)	Shotcrete	Steel Sets
Very good rock I RMR: 81-100	Full face. 10 ft - advance	Generally no support required except for occasional spot bolting		
Good rock II RMR: 61-80	Full face. 3-5 ft advance Complete support 60 ft from face.	Locally bolts in roof 10 ft long, spaced 8 ft with occasional wire mesh.	2 in. in roof where required.	None
Fair rock III RMR: 41-60	Top heading and bench 5-10 ft advance in top heading. Commence support after each blast. Complete support 20 ft from face.	Systematic bolts 12 ft long, spaced 5-6 ft in roof and walls with wire mesh in crown.	2 to 4 in. in roof and 1 in. on walls.	None
Poor rock IV RMR: 21-40	Top heading and bench 3-5 ft advance in top heading. Install support concurrently with excavation.	Systematic bolts 12-15 ft long, spaced 3-5 ft in roof and walls with wire mesh.	4 to 6 in. in roof and 4 in. on walls.	Light to medium ribs spaced 5 ft where required.
Very poor rock V RMR: <20	Multiple drifts. 1.5-3 ft advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 15-20 ft long, spaced 3-5 ft in roof and walls with wire mesh. Bolt invert.	6 to 8 in. in roof, 6 in. on walls and 2 in. on face.	Medium to heavy ribs spaced 2 ft 6 in. with steel lagging and forepoling if required. Close invert.

* Length of bolts specified here is applicable to tunnels 30 ft wide.

Table 9

Classification of Intact Rock Strength[37]

Description	Uniaxial Compressive Strength lbf/in²	Uniaxial Compressive Strength MPa	Examples of Rock Types
Very low strength	150-3500	1-25	Chalk, rocksalt.
Low strength	3500-7500	25-50	Coal, siltstone, schist.
Medium strength	7500-15000	50-100	Sandstone, slate, shale.
High strength	15000-30000	100-200	Marble, granite, gneiss.
Very high strength	>30000	>200	Quartzite, dolerite, gabbro, basalt.

Table 10

Classification for Joint Spacing[3]

Description	Spacing of Joints		Rock Mass Grading
Very wide	> 3m	> 10ft	Solid
Wide	1m to 3m	3ft to 10ft	Massive
Moderately close	0.3m to 1m	1ft to 3ft	Blocky/seamy
Close	50mm to 300mm	2in to 1ft	Fractured
Very close	< 50mm	< 2in	Crushed and shattered

Table 11

Q-System: Description and Ratings - RQD, J_n, and J_r[12]

Rock Quality Designation (RQD)

Very poor................	0-25	Note:
Poor.....................	25-50	(i) Where RQD is reported or measured as $\leq$ 10 (including 0) a nominal value of 10 is used to evaluate Q in Eq. (1).
Fair.....................	50-75	
Good.....................	75-90	
Excellent................	90-100	(ii) RQD intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.

Joint Set Number (J_n)

Massive, no or few joints	0.5-1.0	Note:
One joint set............	2	(i) For intersections use ($3.0 \times J_n$)
One joint set plus random	3	
Two joint sets...........	4	(ii) For portals use ($2.0 \times J_n$)
Two joint sets plus random..................	6	
Three joint sets.........	9	
Three joint sets plus random..................	12	
Four or more joint sets, random, heavily jointed, "sugar cube", etc........	15	
Crushed rock, earthlike..	20	

Joint Roughness Number (J_r)

(a) Rock wall contact and (b) Rock wall contact before 10 cms shear		Note: (i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
Discontinuous joints.....	4	
Rough or irregular, undulating...............	3	
Smooth, undulating.......	2	Note:
Slickensided, undulating	1.5	(ii) J_r = 0.5 can be used for planar slickensided joints having lineation, provided the lineations are favorably orientated.
Rough or irregular, planar..................	1.5	
Smooth, planar...........	1.0	
Slickensided, planar.....	0.5	(iii) Descriptions B to G refer to small scale features and intermediate scale features, in that order.
(c) No rock wall contact when sheared		
Zone containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	
Sandy, gravelly or crushed zone thick enough to prevent rock wall contact.................	1.0 (nominal)	

Table 12

Q-System: Description and Ratings - J_a [12]

Joint Alteration Number

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

Note:

(i) Values of $(\phi)_r$ are intended as an approximate guide to the mineralogical properties of the alteration products, if present.

Table 13

Q-System: Description and Ratings - SRF and J_w [12]

Stress Reduction Factor

		σ_c/σ_1	σ_t/σ_1	(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.			
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 favorable to stability, may be unfavorable 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 $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to $0.8\ \sigma_c$ and $0.8\ \sigma_t$; when $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\ \sigma_c$ and $0.6\ \sigma_t$ where: σ_c = unconfined compression strength, 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).

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
B.	Medium inflow or pressure occasional outwash of joint fillings............................	0.66	1.0-2.5
C.	Large inflow or high pressure in competent rock with unfilled joints..........................	0.5	2.5-10.0
D.	Large inflow or high pressure, considerable outwash of joint fillings.....................	0.33	2.5-10.0
E.	Exceptionally high inflow or water pressure at blasting, decaying with time...............	0.2-0.1	>10.0
F.	Exceptionally high inflow or water pressure continuing without noticeable decay...........	0.1-0.05	>10.0

Note:

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

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

Table 14

Q-System: Support Measures for Rock Masses of "Exceptional," "Extremely Good," "Very Good," and "Good" Quality (Q Range: 1000-10)[12]

Support Category	Q	Conditional Factors: RQD/J_n	Conditional Factors: J_r/J_n	SPAN/ESR (m)	P kg/cm^2 (approx.)	SPAN/ESR (m)	Type of Support	Note (Table 18)
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) 5-10 cm	I, II
		--	--	<15			B (utg) 1.5-2 m +clm	I, III
15	40-10	>10	--	--	0.5	15-40	B (tg) 1.5-2 m +clm	I, II, IV
		≤10	--	--			B (tg) 1.5-2 m +S (mr) 5-10 cm	I, II, IV
16* 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) 10-15 cm	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 14-17: sb = spot bolting; B = systematic bolting; (utg) = untensioned, grouted; (tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses; 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 15

Q-System: Support Measures for Rock Masses of "Fair" and "Poor" Quality

(Q Range: 10-1)[12]

Support Category	Q	Conditional Factors RQD/J_n	Conditional Factors J_r/J_a	SPAN/ ESR	P Kg/cm^2 (approx.)	SPAN/ ESR (m)	Type of Support	Note (Table 18)
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) 1 m	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.

Table 16

Q-System: Support Measures for Rock Masses of "Very Poor" Quality (Q Range: 1.0-0.1)[12]

Support Category	Q	Conditional Factors RQD/J_n	Conditional Factors J_r/J_a	SPAN/ESR (m)	P kg/cm^2 (approx.)	SPAN/ESR (m)	Type of Support	Note (Table 18)
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 (tg) 1 m +S (mr) 15-20 cm	I, II, IX
		--	--	--			CCA (sr) 30-100 cm +B (tg) 1 m	IV, VIII, X, XI
29*	0.4-0.1	>5	>0.25	--	3.0	1.0-3.1	B (utg) 1 m + S 2-3 cm	--
		≤5	>0.25	--			B (utg) 1 m + S (mr) 5 cm	--
		--	≤0.25	--			B (tg) 1 m + S (mr) 5 cm	--
30	0.4-0.1	≥5	--	--	3.0	2.2-6	B (tg) 1 m + S 2.5-5 cm	IX
		<5	--	--			S (mr) 5-7.5 cm	IX
		--	--	--			B (tg) 1 m +S (mr) 5-7.5 cm	VIII, X, XI
31	0.4-0.1	>4	--	--	3.0	4-14.5	B (tg) 1 m +S (mr) 5-12.5 cm	IX
		≤4, ≥1.5	--	--			S (mr) 7.5-25 cm	IX
		<1.5	--	--			CCA 20-40 cm +B (tg) 1 m	IX, XI
		--	--	--			CCA (sr) 30-50 cm +B (tg) 1 m	VIII, X, XI
32 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) 1m	IV, VIII, X, XI

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

Table 17

Q-System: Support Measures for Rock Masses of "Extremely Poor" and "Exceptionally Poor" Quality (Q Range: 0.1-0.001)[12]

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 (Table 18)
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.

Table 18

Q-System: Supplementary Notes for Support Tables[12]

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 is 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).

Table 19

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

		Drill and Blast Construction			Machine Boring			
Rock Condition	Length of zone ft	Rock Load tsf	Temporary Support	Permanent Lining	Rock Load tsf	Temporary Support	Permanent Lining	Water Inflow gpm
Best average quality: massive, moderately jointed RQD > 80	8000	0.5 Bg = 1.1	11 ft bolts at 4-1/2-ft shotcrete 1 in. thick	Reinforced concrete 14 in. thick plus 8 in. overbreak	0.5	10 ft bolts occasionally at 6 ft shotcrete 2 in. if needed	Reinforced precast line 9 in. thick grouted	1
Worst average quality: very blocky, seamy RQD = 40	800	0.5 × 2Bg = 2.2	11 ft bolts at 2 ft shotcrete 2 in. thick	Reinforced concrete 15 in. thick plus 8 in. overbreak	1.4	10 ft bolts at 3 to 5 ft shotcrete 2 in. if needed	As above	4
Fault zones: completely crushed RQD = 30	300	1.1 × 2Bg = 4.8	W8 steel beams at 2 to 4 ft shotcrete 3 in. thick	Reinforced concrete 22 in. thick plus 8 in. overbreak	3.5	10 ft bolts at 3 ft shotcrete 3 in. thick	As above	50

Note: Density g = 116 - 171 pcf (avg 169).

Table 20

Rock Mass Classifications for the Park River Tunnel

in Accordance with the RSR Concept

Parameters and Regions	Best Average Conditions Region 1	Best Average Conditions Region 2	Worst Average Conditions Sta 23+00 to 31+00	Fault Zones Region 3
Parameter A: (Table 2)	18 Rock type 3 (shale) slightly faulted	30 Rock type 2 (basalt) massive	7 Type 3 (shale) intensely faulted (very seamy)	7 (6)
Parameter B (Table 3)	Set 1: 36; Set 2: 34; Set 3: 38	Set 1: 38; Set 2: 43	13	10
Parameter C (Table 4)	22	22	6	6
RSR = A + B + C	76 74 78	90 95	26	23
Rock load for 26-ft tunnel, ksf	<0.5	Off scale	~7.0	~7.0
Rib type and spacing	None	None	8W40 at 2 ft 10W49 at 3 ft	8W40 at 2 ft

Note: For input data see Appendix C.

Table 21

Rock Mass Classifications for the Park River Tunnel in Accordance with the Geomechanics Classification

Parameter and Region	Best Average Conditions Region 1	Best Average Conditions Region 2	Worst Average Conditions Sta 23+00 to 31+00	Fault Zones Region 3
Intact rock strength	7	7	7	7
RQD	20	20	13	4
Joint spacing	20	20	10	5
Joint condition	20	22	10	6
Groundwater	8	10	7	4
In situ rating	75	79	47	26
Joint orientation	-5	-5	-10	-10
RMR	Good rock 70	Good rock 74	Poor rock 37	Very poor rock 16
Maximum span and stand-up time	55 ft at 2-1/2 months or 26 ft at 4 months	26 ft at 6 months	18 ft at 12 hr	5 ft at 1/2 hr
Support	Locally bolts in roof 10 ft long at 8 ft plus occasional mesh, shotcrete 2 in. thick		Systematic bolts 12 ft long at 5 ft shotcrete, 5 in. wire mesh	Ribs at 2-1/2 ft bolts 15 ft long at 3 ft shotcrete, 8 in. wire mesh

Note: For input data sheets see Appendix C.

Table 22

Rock Mass Classifications for the Park River Tunnel

in Accordance with the Q-System

Parameter	Best Average Conditions: Region 1	Best Average Conditions: Region 2	Worst Average Conditions Sta 23+00 to 31+00	Fault Zones: Region 3
RQD	80	90	40	(17) 28 (35)
J_n	6	12	9	15 (3)
J_r	1.5	1.5	1.5	1.5
J_a	1.0	1.0	2.0	4.0
J_w	1.0	1.0	0.66	0.5
SRF	1.0	1.0	1.0	2.5
Q	Good rock 19.99	Good rock 11.25	Poor rock 2.19	Very poor 0.139
Rock load in roof	0.5 tsf	0.59 tsf	1.02 tsf	(1.85) 2.70 tsf
	Category 13		Category 22	Category 31
Permanent support	Untensioned spot bolts 9 ft long, spaced 5 to 6 ft. No shotcrete		Untensioned 9 ft bolts, spaced 3 ft plus shotcrete 1-2 in. thick	Reinforced concrete 8-16 in. thick plus tensioned 9 ft bolts at 3 ft
			Category 13	
Temporary support	None		9 ft bolts at 6 ft	Shotcrete 6-10 in. thick with steel mesh

Note: For input data see Appendix C.

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Table 23

Comparison of Support Recommendations for the Park River Tunnel

	Support System			
Rock Conditions	Terzaghi's Method	RSR Concept	Geomechanics Classification	Q-System
Best average conditions: Regions 1 and 2	Rock load: 1.1 tsf Reinforced concrete 14 in. thick plus 8 in. overbreak Temporary: 11 ft bolts at 4-1/2 ft, shotcrete 1 in. thick	Permanent: N/A Temporary: None	Locally rockbolts in roof 10 ft long at 8 ft spacing plus occasional mesh and shotcrete 2 in. thick	Rock load: 0.5 tsf Untensioned spot bolts 9 ft long spaced 5-6 ft. No shotcrete or mesh
Worst average conditions: Sta 23+00 to 31+00	Rock load: 2.2 tsf Reinforced concrete 15 in. thick plus 8 in. overbreak Temporary: 11 ft bolts at 2 ft, shotcrete 2 in. thick	Permanent: N/A Temporary: 8W40 steel ribs at 2 ft	Systematic bolts 12 ft long at 5 ft spacing with wire mesh plus shot-crete 5 in. thick	Rock load: 1.1 tsf Untensioned systematic 9 ft long bolts at 3 ft spacing plus shotcrete 1-2 in. thick. Primary: spot bolting
Fault zones: Region 3	Rock load: 4.8 tsf Reinforced concrete 22 in. thick plus 8 in. overbreak Temporary: steel ribs: W8 ring beams at 2-4 ft, shotcrete 3 in.	Permanent: N/A Temporary: 8W40 steel ribs at 2 ft	Steel ribs at 2-1/2 ft, 15 ft bolts at 3 ft with wire mesh plus shotcrete 8 in. thick	Rock load: 2.7 tsf Reinforced concrete 8-16 in. thick plus tensioned 9 ft bolts at 3 ft. Primary: shotcrete 6-10 in. with mesh

APPENDIX A: TERZAGHI'S ROCK LOAD TABLES

Table A1

Terzaghi's Rock Load Classification for Steel Arch-Supported Tunnels[2]

(Rock Load H_p in Feet of Rock on Roof of Support in Tunnel With Width B (feet) and Height H_t (feet) at a Depth of More Than $1.5(B + H_t)$)*

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.**	0 to 0.5B	Light support, mainly for protection against spalls. Load may change erratically from point to point.
3. Massive, moderately jointed.	0 to 0.25B	(same as 2)
4. Moderately blocky and seamy.	$0.25B$ 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 effects 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 $4.50)(B + H_t)$	(same as 7)
9. Swelling rock.	Up to 250 feet, irrespective of the value of $(B + H_t)$	Circular ribs are required. In extreme cases use yielding support.

* 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 percent.

** 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 a 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 the rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such formations, the roof pressure may be as heavy as in very blocky and seamy rock.

Table A2

Rock Loads and Classification[4]

Fracture Spacing (cm)	Spacing (ft, in)	RQD (%)	Rock Condition	Rock Load, H_p Initial	Rock Load, H_p Final		Remarks
	2'	98	1. Hard and Intact	0	0	Generally no side pressure. Erratic load changes from point to point.	Lining only is spalling or popping
50	1'	95	2. Hard Stratified or Schistose	0	0.25B		Spalling common
		90	3. Massive, moderately jointed	0	0.5B		Side Pressure if strata inclined, some spalling
20	6"	75	4. Moderately blocky and seamy	0	0.25B to 0.35C		
10	4"	50	5. Very blocky, seamy and shattered	0 to 0.6C	0.35C to 1.1C		Little or no side pressure
5	2"	25, 10, 2	6. Completely crushed		1.1C		Considerable side pressure. If seepage, continuous support.
	1"		7. Gravel and sand	0.54C to 1.2C	0.62C to 1.38C		Dense
2				0.94C to 1.2C	1.08C to 1.38C		Side pressure $P_h = 0.3\gamma\,(0.5H_t + H_p)$ Loose
Weak and coherent			8. Squeezing, moderate depth		1.1C to 2.1C		Heavy side pressure. Continuous support required.
			9. Squeezing, great depth		2.1C to 4.5C		
			10. Swelling		up to 250'		Use circular support. In extreme cases: yielding support.

Notes:
1) For rock classes 4, 5, 6, 7, when above ground water level, reduce loads by 50%.
2) For sands (7), H_{pmin} is for small movements (-0.01C to 0.02C) H_{pmax} for large width movements (-0.15C).
3) B is tunnel width, $C = B + H_t$ = width + height of tunnel (in feet). For circular tunnel, $C = 2B = 2H_t$.
4) γ = density of medium, lbs/ft^3.

Table A3

Support Recommendations for Tunnels in Rock (20- to 40-ft Diameter) Based on RQD[4]

Rock Quality	Tunneling Method	Alternative Support Systems: Steel Sets[2)]	Rockbolts[3)]	Shotcrete
EXCELLENT[1)] RQD > 90	A. Boring Machine	None to occ. light set. Rock load (0.0-0.2)B.	None to occasional	None to occ. local application
	B. Conventional	None to occ. light set. Rock load (0.0-0.3)B.	None to occasional	None to occ. local application 2 in. to 3 in.
GOOD[1)] 75 < RQD < 90	A. Boring Machine	Occ. light sets to pattern on 5-ft to 6-ft ctr. Rock load (0.0 to 0.4)B.	Occasional to pattern on 5-ft to 6-ft centers	None to occ. local application 2 in. to 3 in.
	B. Conventional	Light sets, 5-ft to 6-ft ctr. Rock load (0.3 to 0.6)B.	Pattern, 5-ft to 6-ft centers	Occ. local application 2 in. to 3 in.
FAIR 50 < RQD < 75	A. Boring Machine	Light to medium sets, 5-ft to 6-ft ctr. Rock load (0.4-1.0)B.	Pattern, 4-ft to 6-ft ctr.	2 in. to 4 in. on crown
	B. Conventional	Light to medium sets, 4-ft to 5-ft ctr. Rock load (0.6-1.3)B.	Pattern 3-ft to 5-ft ctr.	4 in. or more crown and sides
POOR[2)] 25 < RQD < 50	A. Boring Machine	Medium circular sets on 3-ft to 4-ft ctr. Rock load (1.0-1.6)B.	Pattern, 3-ft to 5-ft ctr.	4 in. to 6 in. on crown and sides. Combine with bolts.
	B. Conventional	Medium to heavy sets on 2-ft to 4-ft ctr. Rock load (1.3-2.0)B.	Pattern, 2-ft to 4-ft ctr.	6 in. or more on crown and sides. Combine with bolts.
VERY POOR[3)] RQD < 25 (Excluding squeezing or swelling ground.)	A. Boring Machine	Medium to heavy circular sets on 2-ft ctr. Rock load (1.6 to 2.2)B.	Pattern, 2-ft to 4-ft ctr.	6 in. or more on whole section. Combine with medium sets.
	B. Conventional	Heavy circular sets on 2-ft ctr. Rock load (2.0 to 2.8)B.	Pattern, 3-ft center.	6 in. or more on whole section. Combine with medium to heavy sets.
VERY POOR[4)] (Squeezing or swelling.)	A. Boring Machine	Very heavy circular sets on 2-ft ctr. Rock load up to 250-ft.	Pattern, 2-ft to 3-ft ctr.	6 in. or more on whole section. Combine with heavy sets.
	B. Conventional	Very heavy circular sets on 2-ft ctr. Rock load up to 250-ft.	Pattern, 2-ft to 3-ft ctr.	6 in. or more on whole section. Combine with heavy sets.

Notes:
1) In good and excellent quality rock, the support requirement will be, in general, minimal but will be dependent upon joint geometry, tunnel diameter, and relative orientations of joints and tunnel.
2) Lagging requirements will usually be zero in excellent rock and will range from up to 25% in good rock to 100% in very poor rock.
3) Mesh requirements usually will be zero in excellent rock and will range from occasional mesh (or straps) in good rock to 100% mesh in very poor rock.
4) B = tunnel width.

APPENDIX B: SUMMARY OF PROCEDURES FOR ROCK MASS CLASSIFICATIONS

1. The procedures for rock mass classifications are summarized here for the convenience of the engineering geologists responsible for the collection of geological data.

Geomechanics Classification--Rock Mass Rating (RMR) System

2. This engineering classification of rock masses, especially evolved for rock tunneling applications, utilizes the following six parameters, all of which are determined in the field:

a. Uniaxial compressive strength of intact rock material.

b. Rock quality designation (RQD).

c. Spacing of discontinuities.

d. Condition of discontinuities.

e. Orientation of discontinuities.

f. Groundwater conditions.

The rock mass along the tunnel route is divided into a number of structural regions, and the above six classification parameters are determined for each structural region and entered onto the standard input data sheet (Figure B1). The following explanations and terminology are relevant.

Structural regions

3. These regions are geological zones of rock masses in which certain features are more or less uniform. Although rock masses are discontinuous in nature, they may nevertheless be uniform in regions when, for example, the type of rock or the spacings of discontinuities are the same throughout the region. In most cases, the boundaries of structural regions will coincide with such major geological features as faults and shear zones.

Discontinuities

4. This term means all discontinuities in the rock mass, which may be technically joints, bedding planes, minor faults, or other surfaces of weakness. It excludes major faults that will be considered as structural regions of their own.

CLASSIFICATION INPUT DATA WORKSHEET: GEOMECHANICS CLASSIFICATION OF ROCK MASSES

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

STRUCTURAL REGION

No. ____________
Sta. ____________
Sta. ____________
Sta. ____________
Sta. ____________

ROCK TYPE AND ORIGIN

DRILL CORE QUALITY R.Q.D.

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

WALL ROCK OF DISCONTINUITIES

Unweathered ....
Slightly weathered ....
Moderately weathered ....
Highly weathered ....
Completely weathered ....

GROUNDWATER

INFLOW per 1000 ft of tunnel length gal/min ....

or

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

....

STRENGTH OF INTACT ROCK MATERIAL

	Uniaxial compressive strength, psi	
Very high:	Over 32,000	....
High:	16,000 - 32,000	....
Medium:	8,000 - 16,000	....
Low:	4,000 - 8,000	....
Very low:	150 - 4,000	....

CONDITION OF DISCONTINUITIES

		Set 1	Set 2	Set 3
CONTINUITY				
Very low:	< 3 ft	....	....	....
Low:	3-10 ft	....	....	....
Medium:	10-30 ft	....	....	....
High:	> 30 ft	....	....	....
SEPARATION				
Tight joints:	< 0.01	....	....	....
Moderately open joints:	0.01-0.1 in.	....	....	....
Open joints:	0.1-0.5 in.	....	....	....
Range in.:		....	....	....
ROUGHNESS				
Very rough surfaces:		....	....	....
Rough surfaces:		....	....	....
Slightly rough surfaces:		....	....	....
Smooth surfaces:		....	....	....
Slickensided surfaces:		....	....	....
FILLING (GOUGE)				
Type:		....	....	....
Thickness:		....	....	....
Consistency:		....	....	....

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3
Very wide:	Over 10 ft	....	....	....
Wide:	3-10 ft	....	....	....
Moderate:	1-3 ft	....	....	....
Close:	2 in.-1 ft	....	....	....
Very close:	< 2 in.	....	....	....
Range		....	....	....

MAJOR FAULTS OR FOLDS

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

STRIKE AND DIP ORIENTATIONS

				DIRECTION
Set 1	Strike:	(from .... to ....)	Dip:	....
Set 2	Strike:	(from .... to ....)	Dip:	....
Set 3	Strike:	(from .... to ....)	Dip:	....

GENERAL REMARKS AND ADDITIONAL DATA

The geologist should supply any further information which he considers relevant.

Figure B1. Standard input data sheet

Intact rock strength

5. The uniaxial compressive strength of rock material is determined in accordance with the standard laboratory procedures, but for the purpose of rock classification, the use of the well-known, point-load strength index is recommended. The reason is that the index can be determined in the field on rock core retrieved from borings and the core does not require any specimen preparation. Using simple portable equipment, a piece of drill core is compressed between two points. The core fails as a result of fracture across its diameter. The point-load strength index is calculated as the ratio of the applied load to the square of core diameter. A close correlation exists (to within ~20 percent) between the uniaxial compressive strength and the point-load strength index I_s such that for standard NX core (2.16-in. diam), $\sigma = 24 I_s$.

Rock quality designation (RQD)

6. This quantitative index is based on a modified core recovery procedure, which incorporates only those pieces of core that are 4 in. or greater in length. Shorter lengths of core are ignored as they are considered to be due to close shearing, jointing, or weathering in the rock mass. It should be noted that the RQD disregards the influence of discontinuity tightness, orientation, continuity, and gouge material. Consequently, while it is an essential parameter for core description, it is not the sufficient parameter for the full description of a rock mass.

7. For RQD determination, the International Society for Rock Mechanics recommends double-tube, N-size core barrels (core diameter of 2.16 in.). The accepted divisions of RQD values are as follows:

RQD, percent	Core Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
< 25	Very poor

Spacing and orientation of discontinuities

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

Condition of discontinuities

9. This parameter includes roughness of the discontinuity surfaces, their separation (distance between the surfaces), their length or continuity (persistence), weathering of the wall rock of the planes of weakness, and the infilling (gouge) material. The Task Committee of the American Society of Civil Engineers set up the following weathering classification, which should be used:

a. Unweathered. No visible signs are noted of weathering; rock fresh; crystals bright.

b. Slightly weathered rock. Discontinuities are stained or discolored and may contain a thin filling of altered material. Discoloration may extend into the rock from the discontinuity surfaces to a distance of up to 20 percent of the discontinuity spacing.

c. Moderately weathered rock. Slight discoloration extends from discontinuity planes for a distance greater than 20 percent of the discontinuity spacing. Discontinuities may contain filling of altered material. Partial opening of grain boundaries may be observed.

d. Highly weathered rock. Discoloration extends throughout the rock, and the rock material is partly friable. The original texture of the rock has mainly been preserved, but separation of the grains has occurred.

e. Completely weathered rock. The rock is totally discolored and decomposed and in a friable condition. The external appearance is that of soil. Internally, the rock texture is partly preserved, but the grains have completely separated.

It should be noted that the boundary between rock and soil is defined in terms of the uniaxial compressive strength and not in terms of weathering. A material with the strength equal to or above 150 psi is considered as rock.

10. Furthermore, in rock engineering, the information on the rock material strength is preferable to that on rock hardness. The reason is that rock hardness, which is defined as the resistance to indentation or scratching, is not a quantitive parameter and is subjective to a geologist's personal opinion. It has been employed in the past before the advent of the point-load strength index that can now assess the rock strength in the field. For the sake of completeness, the following hardness classification was used in the past:

- a. Very soft rock. Material crumbles under firm blow with a sharp end of a geological pick and can be peeled off with a knife.
- b. Soft rock. Material can be scraped and peeled with a knife; indentations 1/16 to 1/8 in. show in the specimen with firm blows.
- c. Medium hard rock. Material cannot be scraped or peeled with a knife; hand-held specimen can be broken with the hammer end of a geological pick with a single firm blow.
- d. Hard rock. Hand-held specimen breaks with hammer end of pick under more than one blow.
- e. Very hard rock. Specimen requires many blows with geological pick to break through intact material.

It can be seen from the above that for the lower ranges up to medium hard rock, hardness can be assessed from visual inspection and by scratching with a knife and striking with a hammer. However, for rock having the uniaxial compressive strength of more than 3500 psi, hardness classification ceases to be meaningful due to the difficulty of distinguishing by the "scratchability test" the various degrees of hardness. In any case, hardness is only indirectly related to rock strength, the relationship being between the uniaxial compressive strength and the product of hardness and density expressed in the following formula:

$$\log \sigma_c = 0.00014 \gamma R + 316$$

where

γ = dry unit weight, pcf

R = Schmidt hardness (L-hammer)

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

of discontinuities. Asperities that occur on discontinuity surfaces interlock, if the surfaces are clean and closed, and inhibit shear movement along the discontinuity surface. This restraint on movement is of two types. Small high-angle asperities are sheared off during shear displacement and effectively increase the peak shear strength of the fracture. Such asperities are termed roughness. Large, low-angle asperities cannot be sheared off and "ride" over one another during shear displacement, changing the initial direction of shear displacement. Such large asperities are termed waviness and cannot be reliably measured in core.

12. Roughness asperities usually have a base length and amplitude measured in terms of tenths of an inch and are readily apparent on a core-sized exposure of a discontinuity. The applicable descriptive terms are defined below (state also if surfaces are stepped, undulating, or planar):

a. Very rough. Near vertical steps and ridges occur on the discontinuity surface.

b. Rough. Some ridge and side-angle steps are evident; asperities are clearly visible; and discontinuity surface feels very abrasive.

c. Slightly rough. Asperities on the discontinuity surfaces are distinguishable and can be felt.

d. Smooth. Surface appears smooth and feels so to the touch.

e. Slickensided. Visual evidence of polishing exists.

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

the filling material. The description of the separation of the discontinuity surfaces is given in millimetres as follows:

- a. Very tight: < 0.1 mm.
- b. Tight: 0.1-0.5 mm.
- c. Moderately open: 0.5-2.5 mm.
- d. Open: 2.5-10 mm.
- e. Very wide: 10-25 mm.

Note that where the separation is more than 25 mm, the discontinuity should be described as a major discontinuity.

14. The infilling (gouge) has a two-fold influence:

- a. Depending on the thickness, the filling prevents the interlocking of the fracture asperities.
- b. It possesses its own characteristic properties, i.e., shear strength, permeability, and deformational characteristics.

The following aspects should be described: type, thickness, continuity, and consistency.

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

Groundwater conditions

16. In the case of tunnels, the rate of inflow of groundwater in gallons per minute per 1000 ft of the tunnel should be determined,[5] or a general condition can be described as completely dry, damp, wet, dripping, and flowing. If actual water pressure data are available, these should be stated and expressed in terms of the ratio of the water pressure to the major principal stress. The latter can be either measured or determined from the depth below surface, i.e., the vertical stress increases with depth at 1.1 psi per foot of the depth below surface.

Rock Structure Rating - RSR Concept

17. The RSR Concept, developed in the United States in 1972 by Wickham, Tiedemann, and Skinner,[5,6] is based on the following three parameters:

a. Parameter A. General appraisal of rock structure is based on:

(1) Rock type origin.

(2) Rock hardness.

(3) Geological structure.

b. Parameter B. Discontinuity pattern with respect to the direction of tunnel drive is based on:

(1) Joint spacing.

(2) Joint orientation (strike and dip).

(3) Direction of tunnel drive.

c. Parameter C. Effect of groundwater inflow is based on:

(1) Overall quality of rock due to parameters A and B combined.

(2) Condition of joint surfaces.

(3) Amount of water inflow (in gallons per minute per foot of the tunnel).

Although the definitions of the above parameters were not explicitly stated by the proposers, most of the data needed are normally included in a standard joint survey. However, it is recognized that the lack of the definitions may lead to some confusion. An input data worksheet for the RSR Concept is shown in Figure B2.

Q-System for Tunnel Support

18. The Q-System, which was developed in Norway in 1974 by Barton, Lien, and Lunde,[12] determines the rock mass quality - termed Q - as a function of six parameters: (a) RQD, (b) number of joint sets, (c) roughness of the weakest joints, (d) degree of alteration or filling along the weakest joints, (e) water inflow or pressure, and (f) rock stress condition. These six parameters are grouped into three quotients.

19. The first two parameters represent the overall structure of the rock mass, and their quotient is claimed to be a crude measure of the relative block size. The quotient of the third and fourth parameters is said to be related to the shear strength of the joints. The fifth parameter is a measure of water pressure, while the sixth parameter is a measure of: (a) loosening load in the case of shear zones and clay-bearing rock, (b) rock stress in competent rock, and (c) squeezing and swelling loads in plastic imcompetent rock. This sixth parameter is regarded as the "total stress" parameter. The quotient of the fifth and sixth parameters is regarded as describing the "active stress." An input data worksheet for the Q-System is shown in Figure B3.

CLASSIFICATION INPUT DATA WORKSHEET: RSR CONCEPT FOR TUNNEL SUPPORT

Project Name: ____________________

Site of Survey: ____________________

Conducted By: ____________________

Date: ____________________

Basic rock type: ....

Igneous ☐ Metamorphic ☐

Sedimentary ☐

Hardness

Hard ☐ Medium ☐ Soft ☐

Decomposed ☐

Geological structure

Massive ☐

Slightly faulted or folded ☐

Moderately faulted or folded ☐

Intensely faulted or folded ☐

Water inflow per 1000 ft of tunnel

.... gal/min.

Joint condition

	Set No. 1	2	3
Tight or cemented	☐	☐	☐
Slightly weathered or altered	☐	☐	☐
Severely weathered, altered or open	☐	☐	☐

Structural Region: ____________________

Sta. ____________________

Sta. ____________________

Sta. ____________________

Sta. ____________________

Joint spacing

		Set No. 1	2	3
Very closely jointed:	< 2 in.	☐	☐	☐
Closely jointed:	2-6 in.	☐	☐	☐
Moderately jointed:	6 in. - 1 ft	☐	☐	☐
Moderate to blocky:	1-2 ft	☐	☐	☐
Blocky to massive:	2-4 ft	☐	☐	☐
Massive:	> 4 ft	☐	☐	☐
Range in ft:		__	__	__

Joint orientation

	Set 1	Set 2	Set 3
Strike w.r.t. magnetic north	.....	.....	.....

Strike ⊥ to tunnel axis ☐ Set No. ____

Strike || to tunnel axis ☐ Set No. ____

Dip orientation

	1	2	3
Dip: 0-20 deg	☐	☐	☐
20-50 deg	☐	☐	☐
50-90 deg	☐	☐	☐
Direction:	__	__	__

Tunnel drive: with dip ☐

against dip ☐

Figure B2. Input data worksheet for the RSR Concept

CLASSIFICATION INPUT DATA WORKSHEET

Q-SYSTEM

Project Name:______________________ Conducted by:______________________

Site of Survey:______________________ Date:______________________

Structural Region:______________________ Rock Type:______________________

Sta.______________________

Sta.______________________

Sta.______________________

Sta.______________________

JOINT SETS

Massive rock, no or few joints	
No. of joint sets present	
Additional random joints exist	
Rock heavily fractured	
Crushed rock	

ROCK QUALITY DESIGNATION

Average RQD = __________%

Range = ________________%

WATER CONDITIONS

Dry or minor inflow	
Medium inflow	
Large inflow, unfilled joints	
Large inflow, filling washed out	
Exceptional transient inflow	
Exceptional continuous inflow	
Approx. water pressure: lb/sq in.	

ROUGHNESS OF JOINTS

Rough or irregular	
Smooth	
Slickensided	
Undulating	
Planar	
Not continuous	
Wall rock contact	
No wall contact	

STRESS CONDITIONS

Low stress, near surface	
Med. stress: σ_c/σ_1 = 10-200	
High stress: σ_c/σ_1 = 5-10	
Weakness zones with clay	
Shear zones	
Squeezing rock	
Swelling rock	
Stress values if determined: $\sigma_{vert.}$ $\sigma_{horz.}$	

FILLING AND WALL ALTERATION

Tightly healed joints				
Unaltered, staining only				
Slightly altered				
Silty or sandy coatings				
Clay coatings				
Sand or crushed rock filling				
Stiff clay	<5mm		>5mm	
Soft clay	<5mm		>5mm	
Swelling clay	<5mm		>5mm	

GENERAL

Uniaxial strength of rock material

Tensile:__________psi

Compressive:__________psi

Strike and dip orientation of the weakest joints

Average strike __________ Average dip __________

Dip direction __________

Figure B3. Input data worksheet for the Q-System

APPENDIX C: CASE HISTORY DATA:

PARK RIVER TUNNEL

Table C1

Description of Rock Types

Red Shale/Siltstone: The dominant rock type is reddish-brown shale/siltstone. The shale contains sandy phases and is interbedded with gray shales and thin sandstones. It is thin bedded and calcareous. Calcite fills the open-bedding planes, joints, and fractures. The shales are usually well cemented and moderately hard, but some zones are classified as soft and weak. The sandy phases are mostly competent and hard to very hard. Shale samples from near the intake exhibited a slaking-like action when submerged. This is attributed to stress relief by coring. Bedding strikes roughly north-south and generally dips 10 to 20 deg to the east but with local variations.

Gray-Black Shales: Gray and sometimes black shales are interbedded with the red shales. They are thin-bedded and similarly oriented. The beds are thinner than the red beds and were used as markers to correlate between boreholes. Gray shales are calcareous, moderately hard to soft and are similar in physical properties to the red shales.

Sandstones: Thin whitish to gray calcareous sandstone beds are common within the shales. Many sandy zones appear to correlate between boreholes and were used as markers. The beds are hard but sometimes show some solution activity and localized concentrated jointing. Variations include a coarse red sandstone (arkose) and a thin zone of interbedded volcanic sandstone and shale that were encountered in only two boreholes, but in no other borings.

Basalts: Basalt flows near the intake shaft are oriented consistent with the local stratigraphy although structural modifications are apparent. They are usually gray and olive gray (locally black), slightly vesicular and nonvesicular, calcareous, hard, and contain headed hairline fractures throughout. Localized broken and weathered zones occur.

Aphanite: This gray fine-grained to glassy rock type occurs in borehole FD-9T between the depths 137 and 188 feet. Its origin is uncertain and it occurs in zone with unresolved structural discontinuities. It is hard to very hard but also contains numerous irregular healed hairline fractures. Some zones may be slightly weathered and less dense.

Table C2

Summary of Rock Properties

Property		Red Shale	Gray Shale	Basalt	Aphanite	Red Sandstone
Specific gravity (dry)	No. of tests	25	4	14	3	2
	Range	2.58-2.72	2.61-2.73	2.68-2.87	2.46-2.62	2.58-2.73
	Average	2.66	2.66	2.74	2.54	2.66
Unit weight (pcf)	No. of tests	25	4	14	3	2
	Range	161-169.7	162.9-170.4	167.2-175.3	153.5-163.5	161-170.4
	Average	166	166	172.2	158.5	165.7
Uniaxial compressive strength (psi)	No. of tests	19	4	11	3	2
	Range	3,242-13,100	4,329-14,740	5,540-13,740	2,700-6,660	9,350-9,536
	Average	7,752	8,556	10,263	4,090	9,443
Modulus of elasticity E(psi × 10^6)	No. of tests	7	1	9	1	--
	Range	0.2-5.0	2.5	0.89-10.0	3.0	
	Average	2.1		4.62		

BORE HOLE PHOTO LOG (An example)			BORING NO. FD-8-T
NAME Park River Tunnel		LOCATION Hartford, Connecticut	
DATE PHOTOGRAPHED Nov 27-38, 1975	IRIS SETTING 5.6 and 4.0	CONDITION OF BORING Good	
DEPTH PHOTOGRAPHED 35.0 to 220.0'	WATER DEPTH Flowing at Surface	WATER CONDITION Clear	
FEET CASING (In Photo) 35.0-39.0'	FEET CONCRETE (In Photo) None		FEET ROCK (In Photo) 39.0-220.0'

DEPTH RANGE	DESCRIPTION
45.5-46.2	Jt., Str. N 45 °E, dip 80 °NW, 1/8" at top to 1/32" at bottom, healed with white material (smooth), planar, terminates at bedding Jt. at bottom
45.2-46.3	Gray-green rock
46.2	Bedding Jt., Str. N-S, dip 15 °E, 1/16" partly open, rough, planar
46.3-160.0	Dark gray rock containing numerous small irregular white inclusions At 51 feet rock gradually changes to dark blue-gray color
53.6	Jt. Str. N 70 °E, dip 20 °SE, 1/32-1/16" partly open, stained, rough, planar
53.9-54.1	Jt., Str. N 20 °W, dip 30 °NE, 1/32-1/16" partly open, stained, rough, planar
54.3-54.7	Jt., Str. N 30 °W, dip 50 °NE, hairline-1/32", healed with white material, rough and irregular
56.2-56.3	Jt., Str. about N-S, dip 45 °W, 1/32", healed with white material, rough, irregular, discontinuous
56.7-57.9	Jt., Str. N 30 °E, dip 80 °NW, hairline-1/32", healed with white material, rough, planar, discontinuous
58.4-59.3	Jt., Str. N 10 °E, dip 75 °W, 1/32-1/16" healed with white material, rough, planar
59.1	Jt., Str. N-S, dip 10 °E, 1/16" healed with white material, rough, irregular
59.0-59.5	Jt., Str. N 10 °E, dip 75 °W, 1/16" healed with white material, rough, planar, discontinuous
60.7-61.5	3 Jts., Str. N 10 °E, dip 75 °W, 1/32-1/16" healed with white material

Figure C1. Typical drill log

CLASSIFICATION INPUT DATA WORKSHEET: GEOMECHANICS CLASSIFICATION OF ROCK MASSES

Name of project: Park River Tunnel
Site of survey: Hartford, Conn.
Conducted by: G. A. Nicholson
Date: ____________

STRUCTURAL REGION

No. Subregion 1(a)
Sta. 98+10-95+20
Sta. ____________
Sta. ____________
Sta. ____________

ROCK TYPE AND ORIGIN

Shale

DRILL CORE QUALITY R.Q.D.

Excellent	90 - 100%	
Good	75 - 90%	
Fair	50 - 75%	55 avg.
Poor	25 - 50%	
Very poor	< 25%	
Range %	20-90%	

WALL ROCK OF DISCONTINUITIES

Unweathered	
Slightly weathered	✓
Moderately weathered	✓
Highly weathered	✓
Completely weathered	

GROUNDWATER

INFLOW per 1000 ft of tunnel length gal/min 170

or

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

Low

STRENGTH OF INTACT ROCK MATERIAL

	Uniaxial compressive strength, psi	
Very high:	Over 32,000	
High:	16,000 - 32,000	
Medium:	8,000 - 16,000	✓ (assumed)
Low:	4,000 - 8,000	
Very low:	150 - 4,000	

CONDITION OF DISCONTINUITIES

CONTINUITY		Set 1	Set 2	Set 3
Very low:	< 3 ft			
Low:	3-10 ft			
Medium:	10-30 ft			
High:	> 30 ft	✓	✓	
SEPARATION				
Tight joints:	< 0.01			
Moderately open joints:	0.01-0.1 in.	1/8-1/16	1/4-1/16	1/8-1/16
Open joints:	0.1-0.5 in.			
Range in.:				
ROUGHNESS				
Very rough surfaces:		✓	✓	✓
Rough surfaces:				
Slightly rough surfaces:				
Smooth surfaces:				
Slickensided surfaces:				
FILLING (GOUGE)				
Type:				
Thickness:		N/A		
Consistency:				

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3
Very wide:	Over 10 ft			
Wide:	3-10 ft			
Moderate:	1-3 ft	✓	✓	✓
Close:	2 in.-1 ft			
Very close:	< 2 in.			
Range (ft)		0.2-8.9	1.2-5.2	0.3-11.5

MAJOR FAULTS OR FOLDS

One known fault zone in this area between sta 95+20 to 94+70±. All major faults are classified as Region 3.

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

STRIKE AND DIP ORIENTATIONS

	Strike	From	To	Dip	Direction
Set 1	N65E	N50E	N80E	35	NW
Set 2	E-W	E-W	N80W	40	N to NE
Set 3	N20E	N15E	N25E	68	NW

GENERAL REMARKS AND ADDITIONAL DATA

Random joints are present. Inputs data for this sheet are average values obtained from available downhole photo logs of this region.

The geologist should supply any further information which he considers relevant.

Figure C2. Data input worksheets, Subregion 1(a) (sheet 1 fo 3)

CLASSIFICATION INPUT DATA WORKSHEET: RSR CONCEPT FOR TUNNEL SUPPORT

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Conducted By: G. A. Nicholson

Date: ________

Structural Region: Subregion 1(a)

Sta. 98+10-95+20

Sta. ________

Sta. ________

Sta. ________

Basic rock type: ...Shale...............

Igneous [ ] Metamorphic [ ]

Sedimentary [✓]

Hardness

Hard [ ] Medium [✓] Soft [ ]

Decomposed [ ]

Geological structure

Massive [ ]

Slightly faulted or folded [✓]

Moderately faulted or folded [ ]

Intensely faulted or folded [ ]

Water inflow per 1000 ft of tunnel

...170....... gal/min.

Joint condition

	Set No. 1	2	3
Tight or cemented	[ ]	[ ]	[ ]
Slightly weathered or altered	[✓]	[✓]	[✓]
Severely weathered, altered or open	[ ]	[ ]	[ ]

Tunnel part of this region.

Joint spacing

		Set No. 1	2	3
Very closely jointed:	< 2 in.	[ ]	[ ]	[ ]
Closely jointed:	2-6 in.	[ ]	[ ]	[ ]
Moderately jointed:	6 in. - 1 ft	[ ]	[ ]	[ ]
Moderate to blocky:	1-2 ft	[ ]	[ ]	[ ]
Blocky to massive:	2-4 ft	[✓]	[✓]	[✓]
Massive:	> 4 ft	[ ]	[ ]	[ ]
Range in ft:		0.2-6.9	1.2-5.2	0.3-11.5

Joint orientation

	Set 1	Set 2	Set 3
Strike w.r.t. magnetic north	N65E.	E-W..	N20E.

Strike ⊥ to tunnel axis [ ] Set No. 3

Strike || to tunnel axis [ ] Set No. 1 & 2

Dip orientation

		1	2	3
Dip:	0-20 deg	[ ]	[ ]	[ ]
	20-50 deg	[✓]	[✓]	[ ]
	50-90 deg	[ ]	[ ]	[✓]
Direction:		NW	N-NE	NE

Tunnel drive: with dip [✓] against dip [ ] with respect to prominent joint set No. 1 and also set No. 3

Figure C2 (sheet 2 of 3)

CLASSIFICATION INPUT DATA WORKSHEET

Q-SYSTEM

Project Name: Park River Tunnel

Conducted by: G. A. Nicholson

Site of Survey: Hartford, Conn.

Date:

Structural Region: Subregion 1(a)

Rock Type: Shale

Sta. 98+10-95+20

Sta.

Sta.

Sta.

JOINT SETS

Massive rock, no or few joints	
No. of joint sets present	3
Additional random joints exist	yes
Rock heavily fractured	
Crushed rock	

ROCK QUALITY DESIGNATION

Average RQD = 55 %

Range = 20-90 %

WATER CONDITIONS

Dry or minor inflow	
Medium inflow	✓
Large inflow, unfilled joints	
Large inflow, filling washed out	
Exceptional transient inflow	
Exceptional continuous inflow	
Approx. water pressure: 40 lb/sq in.	

ROUGHNESS OF JOINTS

Rough or irregular	✓
Smooth	
Slickensided	
Undulating	
Planar	
Not continuous	
Wall rock contact	
No wall contact	

STRESS CONDITIONS

Low stress, near surface	
Med. stress: σ_c/σ_1 = 10-200	✓
High stress: σ_c/σ_1 = 5-10	
Weakness zones with clay	
Shear zones	
Squeezing rock	
Swelling rock	
Stress values if determined: $\sigma_{vert.}$ N/A $\sigma_{horz.}$ 450 ± 132 psi	

FILLING AND WALL ALTERATION

Tightly healed joints				
Unaltered, staining only				✓
Slightly altered				✓
Silty or sandy coatings				
Clay coatings				
Sand or crushed rock filling				
Stiff clay	<5mm		>5mm	
Soft clay	<5mm		>5mm	
Swelling clay	<5mm		>5mm	

GENERAL

Uniaxial strength of rock material

Tensile: N/A psi

Compressive: 8000 psi

Strike and dip orientation of the weakest joints

Average strike E-W Average dip 40

Dip direction N to NE

* Set No. 2 has largest joint openings.

Figure C2 (sheet 3 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: GEOMECHANICS CLASSIFICATION OF ROCK MASSES

Name of project: Park River Tunnel
Site of survey: Hartford, Conn.
Conducted by: G. A. Nicholson
Date: ______

STRUCTURAL REGION

No. Subregion 1(b)
Sta. 91+70-90+25
Sta. 89+85-88+30
Sta. 82+50-57+10
Sta. 56+60-31+10

ROCK TYPE AND ORIGIN

Shale and/or shale and sandstone interbeds

DRILL CORE QUALITY R.Q.D.

Excellent	90 - 100%	
Good	75 - 90%	80 avg.
Fair	50 - 75%	
Poor	25 - 50%	
Very poor	< 25%	
Range %	20-100%	

WALL ROCK OF DISCONTINUITIES

Unweathered	
Slightly weathered	✓
Moderately weathered	
Highly weathered	
Completely weathered	

GROUNDWATER

INFLOW per 1000 ft of tunnel length: gal/min 670

or

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

STRENGTH OF INTACT ROCK MATERIAL

	Uniaxial compressive strength, psi	14,700 (max)
Very high	Over 32,000	
High	16,000 - 32,000	
Medium	8,000 - 16,000	8900 avg
Low	4,000 - 8,000	
Very low	150 - 4,000	(3200 min)

CONDITION OF DISCONTINUITIES

		Set 1	Set 2	Set 3
CONTINUITY				
Very low	< 3 ft			
Low	3-10 ft			
Medium	10-30 ft			
High	> 30 ft	✓	✓	
SEPARATION				
Tight joints	< 0.01			
Moderately open joints	0.01-0.1 in.	1/32-1/16	1/32-1/16	
Open joints	0.1-0.5 in.		(1/8-1/4)	
Range in.				
ROUGHNESS				
Very rough surfaces				
Rough surfaces		✓	✓	
Slightly rough surfaces				
Smooth surfaces				
Slickensided surfaces				
FILLING (GOUGE)				
Type				
Thickness		N/A		
Consistency				

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3
Very wide	Over 10 ft			
Wide	3-10 ft			
Moderate	1-3 ft	✓	✓	
Close	2 in.-1 ft			
Very close	< 2 in.			
Range (ft)		0.1-24.0	1.4-9.5	

MAJOR FAULTS OR FOLDS

Two known and one possible fault zones in this area. The known faults are at sta 95+25 to 94+75 and 57+00 to 56+50±. The possible fault is at sta 90+30 to 89+90±. All major faults are classified as Region 3.

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

STRIKE AND DIP ORIENTATIONS

	Strike	(from	to)	Dip	DIRECTION
Set 1	N10E	N10W	N50E	22	SE
Set 2	N45E	N30E	N70E	75	NW
Set 3					

GENERAL REMARKS AND ADDITIONAL DATA

Input data for this sheet are average values obtained from available downhole photo logs of this region. **Random** joints are present in addition to set 1 & 2. Set 1 is the predominant set.

The geologist should supply any further information which he considers relevant.

Figure C3. Data input worksheets, Subregion 1(b) (sheet 1 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: RSR CONCEPT FOR TUNNEL SUPPORT

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Conducted By: G. A. Nicholson

Date: ____

Basic rock type: ... Shale ...

Igneous [ ] Metamorphic [ ]

Sedimentary [√]

Hardness

Hard [ ] Medium [√] Soft [ ]

Decomposed [ ]

Geological structure

Massive [ ]

Slightly faulted or folded [√]

Moderately faulted or folded [ ]

Intensely faulted or folded [ ]

Water inflow per 1000 ft of tunnel

......670... gal/min.

Joint condition

	Set No. 1	2	3
Tight or cemented	[ ]	[ ]	[ ]
Slightly weathered or altered	[√]	[√]	[ ]
Severely weathered, altered or open	[ ]	[ ]	[ ]

Structural Region: Subregion 1(b)

Sta. 91+70-90+25

Sta. 89+85-88+30

Sta. 82+50-57+10

Sta. 56+60-31+10

Joint spacing

		Set No. 1	2	3
Very closely jointed:	< 2 in.	[ ]	[ ]	[ ]
Closely jointed:	2-6 in.	[ ]	[ ]	[ ]
Moderately jointed:	6 in. - 1 ft	[ ]	[ ]	[ ]
Moderate to blocky:	1-2 ft	[ ]	[ ]	[ ]
Blocky to massive:	2-4 ft	[√]	[√]	[ ]
Massive:	> 4 ft	[ ]	[ ]	[ ]
Range in ft:		0.1-24.0	1.4-9.5	—

Joint orientation

	Set 1	Set 2	Set 3
Strike w.r.t. magnetic north	N10E	N45E	.....

Strike ⊥ to tunnel axis [ ] Set No. 1

Strike || to tunnel axis [ ] Set No. 2

Dip orientation

		1	2	3
Dip:	0-20 deg	[√]	[ ]	[ ]
	20-50 deg	[ ]	[ ]	[ ]
	50-90 deg	[ ]	[√]	[ ]
Direction:		SE	NW	__

Tunnel drive: with dip [2]

against dip [1]

Figure C3 (sheet 2 of 3)

CLASSIFICATION INPUT DATA WORKSHEET

Q-SYSTEM

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Structural Region: Subregion 1(b)

Sta. 91+70-90+25
Sta. 89+85-88+30
Sta. 82+50-57+10
Sta. 56+60-31+10

Conducted by: G. A. Nicholson

Date:

Rock Type: shale and/or shale and sandstone interbeds

ROCK QUALITY DESIGNATION

Average RQD = 80 %
Range = 20-100 %

JOINT SETS

Massive rock, no or few joints	
No. of joint sets present	2
Additional random joints exist	yes
Rock heavily fractured	
Crushed rock	

ROUGHNESS OF JOINTS

Rough or irregular	✓
Smooth	
Slickensided	
Undulating	
Planar	✓
Not continuous	
Wall rock contact	
No wall contact	

WATER CONDITIONS

Dry or minor inflow	
Medium inflow	✓
Large inflow, unfilled joints	
Large inflow, filling washed out	
Exceptional transient inflow	
Exceptional continuous inflow	
Approx. water pressure: lb/sq in.	

FILLING AND WALL ALTERATION

Tightly healed joints				✓
Unaltered, staining only				✓
Slightly altered				
Silty or sandy coatings				
Clay coatings				
Sand or crushed rock filling				
Stiff clay	<5mm		>5mm	
Soft clay	<5mm		>5mm	
Swelling clay	<5mm		>5mm	

STRESS CONDITIONS

Low stress, near surface	
Med. stress: σ_c/σ_1 = 10-200	✓
High stress: σ_c/σ_1 = 5-10	
Weakness zones with clay	
Shear zones	
Squeezing rock	
Swelling rock	
Stress values if determined: $\sigma_{vert.}$ 450 ± 132 psi $\sigma_{horz.}$ N/A	

In situ stress measured

GENERAL

Uniaxial strength of rock material

Tensile: N/A psi

Compressive: 8900 psi (avg)

Strike and dip orientation of the weakest joints

Average strike N10E Average dip 22

Dip direction SE

Figure C3 (sheet 3 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: GEOMECHANICS CLASSIFICATION OF ROCK MASSES

Name of project: Park River Tunnel
Site of survey: Hartford, Conn.
Conducted by: G. A. Nicholson
Date: ____________

STRUCTURAL REGION
No. Subregion 1(c)
Sta. 23+10-7+10+
Sta. ________
Sta. ________
Sta. ________

ROCK TYPE AND ORIGIN
Shale with interbedded sandstone

DRILL CORE QUALITY R.Q.D.

Excellent	90 - 100%	....
Good	75 - 90%	....
Fair:	50 - 75%	72 avg.
Poor:	25 - 50%	....
Very poor:	< 25%	....
Range %:	30-100%	....

WALL ROCK OF DISCONTINUITIES

Unweathered	....
Slightly weathered	✓
Moderately weathered	....
Highly weathered	....
Completely weathered	....

GROUNDWATER

INFLOW per 1000 ft of tunnel length — gal/min 890

or

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

STRENGTH OF INTACT ROCK MATERIAL

	Uniaxial compressive strength, psi	
Very high:	Over 32,000	....
High:	16,000 - 32,000	....
Medium:	8,000 - 16,000	....
Low:	4,000 - 8,000	✓(assumed)
Very low:	150 - 4,000	....

CONDITION OF DISCONTINUITIES

		Set 1	Set 2	Set 3
CONTINUITY				
Very low:	< 3 ft	....	....	....
Low:	3-10 ft	....	....	....
Medium:	10-30 ft	....	....	....
High:	> 30 ft	✓	✓	....
SEPARATION				
Tight joints:	< 0.01			
Moderately open joints:	0.01-0.1 in.	1/32-1/16	1/32-1/16	....
Open joints:	0.1-0.5 in.	....	....	....
Range in.:		....	....	....
ROUGHNESS				
Very rough surfaces:		....	....	....
Rough surfaces:		✓	✓	....
Slightly rough surfaces:		....	....	....
Smooth surfaces:		....	....	....
Slickensided surfaces:		....	....	....
FILLING (GOUGE)				
Type:		fractured rock	....	....
Thickness:		2"-1'	....	....
Consistency:		....	....	....

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3
Very wide:	Over 10 ft	....	....	....
Wide:	3-10 ft	....	✓	....
Moderate:	1-3 ft	✓	....	....
Close:	2 in.-1 ft	....	....	....
Very close:	< 2 in.	....	....	....
Range (ft)		1-2	2-13	....

MAJOR FAULTS OR FOLDS

Several small fracture zones were found in core legs. Zones range from 2 in. to 1 ft thick & consist of fractured rock & black material. Strike & dip of zones range from N70E-N25W & 40NE to 40SE. Zones probably occur between sta 16+00-13+50.

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

STRIKE AND DIP ORIENTATIONS

	Strike	From	To	Dip	Direction
Set 1	N23E	N5E	N35E	20	SE
Set 2	N47E	N40E	N60E	20	SE
Set 3	....	....	....	....	....

GENERAL REMARKS AND ADDITIONAL DATA

Random joints are present.

The geologist should supply any further information which he considers relevant.

Figure C4. Data input worksheets, Subregion 1(c) (sheet 1 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: RSR CONCEPT FOR TUNNEL SUPPORT

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Conducted By: G. A. Nicholson

Date: ____

Basic rock type: Shale with interbedded sandstone

Igneous [ ] Metamorphic [ ]

Sedimentary [√]

Hardness

Hard [ ] Medium [ ] Soft [√]

Decomposed [ ]

Geological structure

Massive [ ]

Slightly faulted or folded [√]

Moderately faulted or folded [ ]

Intensely faulted or folded [ ]

Water inflow per 1000 ft of tunnel

900 gal/min.

Joint condition

	Set No. 1	2	3
Tight or cemented	[ ]	[ ]	[ ]
Slightly weathered or altered	[√]	[√]	[ ]
Severely weathered, altered or open	[ ]	[ ]	[ ]

Structural Region: Subregion 1(c)

Sta. 23+10-7+10+

Sta. ____

Sta. ____

Sta. ____

Joint spacing

		Set No. 1	2	3
Very closely jointed:	< 2 in.	[ ]	[ ]	[ ]
Closely jointed:	2-6 in.	[ ]	[ ]	[ ]
Moderately jointed:	6 in. - 1 ft	[ ]	[ ]	[ ]
Moderate to blocky:	1-2 ft	[√]	[ ]	[ ]
Blocky to massive:	2-4 ft	[ ]	[ ]	[ ]
Massive:	> 4 ft	[ ]	[√]	[ ]
Range in ft:		1-2	2-13	

Joint orientation

	Set 1	Set 2	Set 3
Strike w.r.t. magnetic north	N23E.	N47E..	.....

Strike ⊥ to tunnel axis [√] Set No. 1

Strike || to tunnel axis [ ] Set No. ____

Dip orientation

		1	2	3
Dip:	0-20 deg	[√]	[√]	[ ]
	20-50 deg	[ ]	[ ]	[ ]
	50-90 deg	[ ]	[ ]	[ ]
Direction:		SE	SE	__

Tunnel drive: with dip [ ]

against dip [√] 1 & 2

Figure C4 (sheet 2 of 3)

CLASSIFICATION INPUT DATA WORKSHEET

Q-SYSTEM

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Structural Region: Subregion 1(c)

Sta. 23+10-7+10+

Sta. ____

Sta. ____

Sta. ____

Conducted by: G. A. Nicholson

Date: ____

Rock Type: ____

JOINT SETS

Massive rock, no or few joints	
No. of joint sets present	2
Additional random joints exist	yes
Rock heavily fractured	
Crushed rock	

ROCK QUALITY DESIGNATION

Average RQD = 72 %

Range = 30-100 %

WATER CONDITIONS

Dry or minor inflow	
Medium inflow	✓
Large inflow, unfilled joints	
Large inflow, filling washed out	
Exceptional transient inflow	
Exceptional continuous inflow	
Approx. water pressure: 50 lb/sq in.	

ROUGHNESS OF JOINTS

Rough or irregular	✓
Smooth	
Slickensided	
Undulating	
Planar	✓
Not continuous	
Wall rock contact	
No wall contact	

STRESS CONDITIONS

Low stress, near surface	
Med. stress: σ_c/σ_1 = 10-200	✓
High stress: σ_c/σ_1 = 5-10	
Weakness zones with clay	
Shear zones	
Squeezing rock	
Swelling rock	
Stress values if determined: $\sigma_{vert.}$ 450 ± 132 psi $\sigma_{horz.}$ N/A	

FILLING AND WALL ALTERATION

Tightly healed joints				✓
Unaltered, staining only				✓
Slightly altered				
Silty or sandy coatings				
Clay coatings				
Sand or crushed rock filling				
Stiff clay	<5mm		>5mm	
Soft clay	<5mm		>5mm	
Swelling clay	<5mm		>5mm	

GENERAL

Uniaxial strength of rock material

Tensile: N/A psi

Compressive: 4000-8000 psi (assumed)

Strike and dip orientation of the weakest joints

Average strike N23E Average dip 20

Dip direction SE

Figure C4 (sheet 3 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: GEOMECHANICS CLASSIFICATION OF ROCK MASSES

Name of project: Park River Tunnel
Site of survey: Hartford, Conn.
Conducted by: G. A. Nicholson
Date: ______

STRUCTURAL REGION

No. 2
Sta. 94+70–91+70
Sta. 88+30–82+50
Sta. ______
Sta. ______

ROCK TYPE AND ORIGIN

Basalt

DRILL CORE QUALITY R.Q.D.

Excellent	90 - 100%	91 avg.
Good	75 - 90%	
Fair	50 - 75%	
Poor	25 - 50%	
Very poor	< 25%	
Range %		60–100%

WALL ROCK OF DISCONTINUITIES

Unweathered	
Slightly weathered	✓
Moderately weathered	
Highly weathered	
Completely weathered	

GROUNDWATER

INFLOW per 1000 ft of tunnel length — gal/min 90

or

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

STRENGTH OF INTACT ROCK MATERIAL

	Uniaxial compressive strength, psi	
Very high:	Over 32,000	
High:	16,000 - 32,000	
Medium:	8,000 - 16,000	
Low:	4,000 - 8,000	
Very low:	150 - 4,000	

CONDITION OF DISCONTINUITIES

CONTINUITY		Set 1	Set 2	Set 3
Very low:	< 3 ft			
Low:	3-10 ft			
Medium:	10-30 ft			
High:	> 30 ft	✓	✓	
SEPARATION				
Tight joints:	< 0.01			
Moderately open joints:	0.01-0.1 in.	1/16	1/32–1/16	
Open joints:	0.1-0.5 in.	(1/4–1/8)	(1/4–1/8)	
Range in.:				
ROUGHNESS				
Very rough surfaces:				
Rough surfaces:		✓	✓	
Slightly rough surfaces:				
Smooth surfaces:				
Slickensided surfaces:				
FILLING (GOUGE)				
Type:				
Thickness:		N/A		
Consistency:				

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3
Very wide:	Over 10 ft			
Wide:	3-10 ft		✓	
Moderate:	1-3 ft	✓		
Close:	2 in.-1 ft			
Very close:	< 2 in.			
Range (ft)		0.4–3.5	0.3–8.2	

MAJOR FAULTS OR FOLDS

The west end boundary (sta 94+70) of this region consists of a known major fault. All major faults are classified as Region 3.

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

STRIKE AND DIP ORIENTATIONS

	Strike	(from	to)	Dip	DIRECTION
Set 1	N10E	N-S	N20E	65	W-NW
Set 2	N20W	N10W	N30W	70	SW
Set 3					

GENERAL REMARKS AND ADDITIONAL DATA

Random joints are present.

The geologist should supply any further information which he considers relevant.

Figure C5. Data input worksheets, Region 2 (sheet 1 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: RSR CONCEPT FOR TUNNEL SUPPORT

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Conducted By: G. A. Nicholson

Date: ____

Basic rock type:...Basalt............

Igneous [✓] Metamorphic [ ]

Sedimentary [ ]

Hardness

Hard [ ] Medium [✓] Soft [ ]

Decomposed [ ]

Geological structure

Massive [✓]

Slightly faulted or folded [ ]

Moderately faulted or folded [ ]

Intensely faulted or folded [ ]

Water inflow per 1000 ft of tunnel

......100.... gal/min.

Joint condition

	Set No. 1	2	3
Tight or cemented	[ ]	[ ]	[ ]
Slightly weathered or altered	[✓]	[✓]	[ ]
Severely weathered, altered or open	[ ]	[ ]	[ ]

Structural Region: 2

Sta. 94+70-91+70

Sta. 88+30-82+50

Sta. ____

Sta. ____

Joint spacing

		Set No. 1	2	3
Very closely jointed:	< 2 in.	[ ]	[ ]	[ ]
Closely jointed:	2-6 in.	[ ]	[ ]	[ ]
Moderately jointed:	6 in. - 1 ft	[ ]	[ ]	[ ]
Moderate to blocky:	1-2 ft	[ ]	[ ]	[ ]
Blocky to massive:	2-4 ft	[✓]	[ ]	[ ]
Massive:	> 4 ft	[ ]	[✓]	[ ]
Range in ft:		0.4-3.5	0.3-8.2	—

Joint orientation

	Set 1	Set 2	Set 3
Strike w.r.t. magnetic north	N10E.	N20W.	.....

Strike ⊥ to tunnel axis [ ] Set No. 1 & 2

Strike || to tunnel axis [ ] Set No. ____

Dip orientation

		1	2	3
Dip:	0-20 deg	[ ]	[ ]	[ ]
	20-50 deg	[ ]	[ ]	[ ]
	50-90 deg	[✓]	[✓]	[ ]
Direction:		W-NW	SW	—

Tunnel drive: with dip [ ] 1 & 2

against dip [ ]

Figure C5 (sheet 2 of 3)

CLASSIFICATION INPUT DATA WORKSHEET

Q-SYSTEM

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Structural Region: 2

Sta. 94+70-91+70

Sta. 88+30-82+50

Sta.

Sta.

Conducted by: G. A. Nicholson

Date:

Rock Type: Basalt

JOINT SETS

Massive rock, no or few joints	
No. of joint sets present	2
Additional random joints exist	yes
Rock heavily fractured	
Crushed rock	

ROCK QUALITY DESIGNATION

Average RQD = 90 %

Range = 60-100 %

WATER CONDITIONS

Dry or minor inflow	✓
Medium inflow	
Large inflow, unfilled joints	
Large inflow, filling washed out	
Exceptional transient inflow	
Exceptional continuous inflow	
Approx. water pressure: 50 lb/sq in.	

ROUGHNESS OF JOINTS

Rough or irregular	✓
Smooth	
Slickensided	
Undulating	
Planar	
Not continuous	
Wall rock contact	
No wall contact	

STRESS CONDITIONS

Low stress, near surface	
Med. stress: σ_c/σ_1 = 10-200	✓
High stress: σ_c/σ_1 = 5-10	
Weakness zones with clay	
Shear zones	
Squeezing rock	
Swelling rock	
Stress values if determined: 450 ± $\sigma_{vert.}$ 132 psi $\sigma_{horz.}$ N/A	

FILLING AND WALL ALTERATION

Tightly healed joints				
Unaltered, staining only				✓
Slightly altered				
Silty or sandy coatings				
Clay coatings				
Sand or crushed rock filling				
Stiff clay	<5mm		>5mm	
Soft clay	<5mm		>5mm	
Swelling clay	<5mm		>5mm	

GENERAL

Uniaxial strength of rock material

Tensile: N/A **psi**

Compressive: 10,000+ **psi**

Strike and dip orientation of the weakest joints

Average strike N10E **Average dip** 65

Dip direction N-NW

Figure C5 (sheet 3 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: GEOMECHANICS CLASSIFICATION OF ROCK MASSES

Name of project: Park River Tunnel
Site of survey: Hartford, Conn.
Conducted by: G. A. Nicholson
Date: ____

STRUCTURAL REGION

No. 3
Sta. 95+20-94+70
Sta. 90+25-89+85
Sta. 57+10-56+60
Sta. ____

ROCK TYPE AND ORIGIN

Fault zones in basalt-sh interface and sh and/or ss/sh interbeds.

DRILL CORE QUALITY R.Q.D.

Excellent:	90 - 100%	....
Good:	75 - 90%	....
Fair:	50 - 75%	....
Poor:	25 - 50%	....
Very poor:	< 25%	17&18 (avg)*
Range %:	1-35%	....

WALL ROCK OF DISCONTINUITIES

Unweathered	....
Slightly weathered	....
Moderately weathered	✓
Highly weathered	....
Completely weathered	....

GROUNDWATER

Inflow per 1000 ft of tunnel length gal/min 1400

or

General conditions (completely dry, damp, wet, dripping or flowing under low, medium or high pressure):

Heavy

STRENGTH OF INTACT ROCK MATERIAL

	Uniaxial compressive strength, psi	
Very high:	Over 32,000	....
High:	16,000 - 32,000	....
Medium:	8,000 - 16,000	8.4-10K
Low:	4,000 - 8,000	....
Very low:	150 - 4,000	....

CONDITION OF DISCONTINUITIES

		Set 1	Set 2	Set 3
CONTINUITY				
Very low:	< 3 ft	....	....	....
Low:	3-10 ft	....	....	....
Medium:	10-30 ft	✓	....	....
High:	> 30 ft	....	....	....
SEPARATION				
Tight joints:	< 0.01	....	....	....
Moderately open joints:	0.01-0.1 in.	....	....	....
Open joints:	0.1-0.5 in.	✓ (est.)	....	....
Range in.:		....	....	....
ROUGHNESS				
Very rough surfaces:		....	....	....
Rough surfaces:		....	....	....
Slightly rough surfaces:		....	....	....
Smooth surfaces:		....	....	....
Slickensided surfaces:		✓	....	....
FILLING (GOUGE)				
Type:		Crushed rock & black material		
Thickness:		....	....	....
Consistency:		....	....	....

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3
Very wide:	Over 10 ft	....	....	....
Wide:	3-10 ft	....	....	....
Moderate:	1-3 ft	....	....	....
Close:	2 in.-1 ft	....	....	....
Very close:	< 2 in.	✓ (est.)	....	....
Range		....	....	....

MAJOR FAULTS OR FOLDS

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

STRIKE AND DIP ORIENTATIONS

	Strike		Dip	Direction
Set 1	Strike: Random	(from .... to ....)	Dip:	....
Set 2	Strike:	(from .... to ....)	Dip:	....
Set 3	Strike:	(from .... to ....)	Dip:	....

GENERAL REMARKS AND ADDITIONAL DATA

This region consists of fault zones.

The geologist should supply any further information which he considers relevant.

* Average for faults at sta 95+20± to 94+70± and 57+10 to 56+60± (FD-9T and FD-22T), respectively.

Figure C6. Data input worksheets, Region 3 (sheet 1 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: RSR CONCEPT FOR TUNNEL SUPPORT

Project Name: Park River Tunnel

Site of Survey: Harford, Conn.

Conducted By: G. A. Nicholson

Date: ____

Basic rock type: Basalt-shale interface with sh and/or ss/sh interbeds

Igneous [ ] Metamorphic [ ]

Sedimentary [✓]

Hardness

Hard [ ] Medium [✓] Soft [ ]

Decomposed [ ]

Geological structure

Massive [ ]

Slightly faulted or folded [ ]

Moderately faulted or folded [ ]

Intensely faulted or folded [✓]

Water inflow per 1000 ft of tunnel

.......1400.. gal/min.

Joint condition

	Set No. 1	2	3
Tight or cemented	[ ]	[ ]	[ ]
Slightly weathered or altered	[ ]	[ ]	[ ]
Severely weathered, altered or open	[✓]	[ ]	[ ]

Structural Region: 3

Sta. 95+20-94+70

Sta. 90+25-89+85

Sta. 57+10-56+60

Sta. ____

Joint spacing

		Set No. 1	2	3
Very closely jointed:	< 2 in.	[✓]	[ ]	[ ] (est.)
Closely jointed:	2-6 in.	[ ]	[ ]	[ ]
Moderately jointed:	6 in. - 1 ft	[ ]	[ ]	[ ]
Moderate to blocky:	1-2 ft	[ ]	[ ]	[ ]
Blocky to massive:	2-4 ft	[ ]	[ ]	[ ]
Massive:	> 4 ft	[ ]	[ ]	[ ]
Range in ft:		N/A	__	__

Joint orientation

	Set 1	Set 2	Set 3
Strike w.r.t. magnetic north	N/A.	.....	.....

Strike ⊥ to tunnel axis [ ] Set No. ____ N/A

Strike || to tunnel axis [ ] Set No. ____

Dip orientation

	1	2	3	
Dip: 0-20 deg	[ ]	[ ]	[ ]	
20-50 deg	[ ]	[ ]	[ ]	N/A
50-90 deg	[ ]	[ ]	[ ]	
Direction:	__	__	__	

Tunnel drive: with dip [ ] N/A

against dip [ ]

Figure C6 (sheet 2 of 3)

CLASSIFICATION INPUT DATA WORKSHEET

Q-SYSTEM

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Structural Region: 3

Sta. 95+20-94+70

Sta. 90+25-89+85

Sta. 57+10-56+60

Sta.

Conducted by: G. A. Nicholson

Date:

Rock Type: Basalt interface and sh and/or ss/sh interbeds

ROCK QUALITY DESIGNATION

Average RQD = 17-28%

Range = 1-35%

JOINT SETS

Massive rock, no or few joints	
No. of joint sets present	
Additional random joints exist	
Rock heavily fractured	✓
Crushed rock	

ROUGHNESS OF JOINTS

Rough or irregular	
Smooth	
Slickensided	✓
Undulating	
Planar	✓
Not continuous	
Wall rock contact	
No wall contact	

WATER CONDITIONS

Dry or minor inflow	
Medium inflow	
Large inflow, unfilled joints	
Large inflow, filling washed out	✓
Exceptional transient inflow	
Exceptional continuous inflow	
Approx. water pressure: 55 lb/sq in.	

FILLING AND WALL ALTERATION

Tightly healed joints				
Unaltered, staining only				
Slightly altered				
Silty or sandy coatings				✓
Clay coatings				
Sand or crushed rock filling				
Stiff clay	<5mm		>5mm	
Soft clay	<5mm		>5mm	
Swelling clay	<5mm		>5mm	

STRESS CONDITIONS

Low stress, near surface	
Med. stress: σ_c/σ_1 = 10-200	
High stress: σ_c/σ_1 = 5-10	
Weakness zones with clay	
Shear zones (fault zone)	✓
Squeezing rock	
Swelling rock	
Stress values if determined: $\sigma_{vert.}$ $\sigma_{horz.}$	

GENERAL

Uniaxial strength of rock material

Tensile: N/A psi

Compressive: 8.4-10K psi

Strike and dip orientation of the weakest joints

Average strike N/A , Average dip N/A

Dip direction N/A

Figure C6 (sheet 3 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: GEOMECHANICS CLASSIFICATION OF ROCK MASSES

Name of project: Park River Tunnel
Site of survey: Hartford, Conn.
Conducted by: G. A. Nicholson
Date: ____________

STRUCTURAL REGION

No. 4
Sta. 31+10-23+10
Sta. ________
Sta. ________
Sta. ________

ROCK TYPE AND ORIGIN

Shale with interbedded sandstone.

DRILL CORE QUALITY R.Q.D.

Excellent	90 - 100%	....
Good	75 - 90%	....
Fair	50 - 75%	....
Poor	25 - 50%	40+DM 9
Very poor	< 25%	....
Range %	20-100%	....

WALL ROCK OF DISCONTINUITIES

Unweathered	....
Slightly weathered	✓
Moderately weathered	....
Highly weathered	....
Completely weathered	....

GROUNDWATER

INFLOW per 1000 ft of tunnel length gal/min 1160

or

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

STRENGTH OF INTACT ROCK MATERIAL

	Uniaxial compressive strength, psi	
Very high:	Over 32,000	......
High:	16,000 - 32,000	......
Medium:	8,000 - 16,000	8300
Low:	4,000 - 8,000	......
Very low:	150 - 4,000	......

CONDITION OF DISCONTINUITIES

CONTINUITY		Set 1	Set 2	Set 3
Very low:	< 3 ft	....	....	....
Low:	3-10 ft	....	....	....
Medium:	10-30 ft	....	....	....
High:	> 30 ft	....	....	....
SEPARATION				
Tight joints:	< 0.01	....	....	....
Moderately open joints:	0.01-0.1 in.	1/32-1/16	1/32-1/16	....
Open joints:	0.1-0.5 in.	....	....	....
Range in.:		....	....	....
ROUGHNESS				
Very rough surfaces:		....	....	....
Rough surfaces:		✓	✓	....
Slightly rough surfaces:		....	....	....
Smooth surfaces:		....	....	....
Slickensided surfaces:		....	....	....
FILLING (GOUGE)				
Type:		....	....	....
Thickness:		N/A	N/A	....
Consistency:		....	....	....

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3
Very wide:	Over 10 ft	....	....	....
Wide:	3-10 ft	✓	✓	✓ ← Assumed
Moderate:	1-3 ft	....	....	....
Close:	2 in.-1 ft	....	....	....
Very close:	< 2 in.	....	....	....
Range (ft)		N/A	....	....

MAJOR FAULTS OR FOLDS

N/A

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

STRIKE AND DIP ORIENTATIONS

	Strike	from	to	Dip	DIRECTION
Set 1	N23E	N15E	N35E	10-20°SE	....
Set 2	N75W	N60E	N85E	60-75 NW	....
Set 3	....	....	....	....	....

GENERAL REMARKS AND ADDITIONAL DATA

This region is representive of the worst average rock conditions presented in DM-9. Available borehole data does not support the severity of conditions reported.

The geologist should supply any further information which he considers relevant.

Figure C7. Data input worksheets, Region 4 (sheet 1 of 3)

CLASSIFICATION INPUT DATA WORKSHEET: RSR CONCEPT FOR TUNNEL SUPPORT

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Conducted By: G. A. Nicholson

Date: ____

Structural Region: 4

Sta. 31+10-23+10

Sta. ____

Sta. ____

Sta. ____

Basic rock type: shale w/interbedded sandstone

Igneous [ ] Metamorphic [ ]

Sedimentary [√]

Hardness

Hard [ ] Medium [√] Soft [ ]

Decomposed [ ]

Geological structure

Massive [ ]

Slightly faulted or folded [√]

Moderately faulted or folded [ ]

Intensely faulted or folded [ ]

Water inflow per 1000 ft of tunnel

....1200.... gal/min.

Joint condition

	Set No. 1	2	3
Tight or cemented	[ ]	[ ]	[ ]
Slightly weathered or altered	[√]	[√]	[√]
Severely weathered, altered or open	[ ]	[ ]	[ ]

Joint spacing

		Set No. 1	2	3
Very closely jointed:	< 2 in.	[ ]	[ ]	[ ]
Closely jointed:	2-6 in.	[ ]	[ ]	[ ]
Moderately jointed:	6 in. - 1 ft	[ ]	[ ]	[ ]
Moderate to blocky:	1-2 ft	[√]	[√]	[√]
Blocky to massive:	2-4 ft	[ ]	[ ]	[ ]
Massive:	> 4 ft	[ ]	[ ]	[ ]
Range in ft:		N/A	__	__

Joint orientation

	Set 1	Set 2	Set 3
Strike w.r.t. magnetic north	N23E.	N75W.	N-S..

Strike ⊥ to tunnel axis [ ] Set No. 1 & 3

Strike || to tunnel axis [ ] Set No. 2

Dip orientation

		1	2	3
Dip:	0-20 deg	[√]	[ ]	[ ]
	20-50 deg	[ ]	[ ]	[ ]
	50-90 deg	[ ]	[√]	[√]
Direction:		SE	NW	SW

Tunnel drive: with dip [ ]

against dip [ ]

Figure C7 (sheet 2 of 3)

CLASSIFICATION INPUT DATA WORKSHEET

Q-SYSTEM

Project Name: Park River Tunnel

Site of Survey: Hartford, Conn.

Structural Region: 4

Sta. 31+10-23+10

Sta.

Sta.

Sta.

Conducted by: G. A. Nicholson

Date:

Rock Type: Shale with interbedded sandstone

JOINT SETS

Massive rock, no or few joints	
No. of joint sets present	2
Additional random joints exist	✓
Rock heavily fractured	
Crushed rock	

ROCK QUALITY DESIGNATION

Average RQD = 40%

Range = 20-100%

WATER CONDITIONS

Dry or minor inflow	
Medium inflow	
Large inflow, unfilled joints	✓
Large inflow, filling washed out	
Exceptional transient inflow	
Exceptional continuous inflow	
Approx. water pressure: lb/sq in.	

ROUGHNESS OF JOINTS

Rough or irregular	✓
Smooth	
Slickensided	
Undulating	
Planar	✓
Not continuous	
Wall rock contact	
No wall contact	

STRESS CONDITIONS

Low stress, near surface	
Med. stress: σ_c/σ_1 = 10-200	✓
High stress: σ_c/σ_1 = 5-10	
Weakness zones with clay	
Shear zones	
Squeezing rock	
Swelling rock	
Stress values if determined: $\sigma_{vert.}$ N/A $\sigma_{horz.}$ 450 ± 132 psi	

FILLING AND WALL ALTERATION

Tightly healed joints				✓
Unaltered, staining only				✓
Slightly altered				
Silty or sandy coatings				
Clay coatings				
Sand or crushed rock filling				
Stiff clay	<5mm		>5mm	
Soft clay	<5mm		>5mm	
Swelling clay	<5mm		>5mm	

GENERAL

Uniaxial strength of rock material

Tensile: N/A psi

Compressive: 8300 psi

Strike and dip orientation of the weakest joints

Average strike N23E Average dip 15

Dip direction SE

Figure C7 (sheet 3 of 3)

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Bieniawski, Z T
Tunnel design by rock mass classifications / by Z. T. Bieniawski, Pennsylvania State University, Department of Mineral Engineering, University Park, Pa. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1979.
71, [60] p. : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; GL-79-19)
Prepared for Office, Chief of Engineers, U. S. Army, Washington, D. C., under Purchase Order No. DACW39-78-M-3114.
References: p. 67-71.

1. Classifications. 2. Construction. 3. Design. 4. Engineering geology. 5. Park River project. 6. Rock classification. 7. Rock masses. 8. Rock mechanics. 9. Rocks. 10. Tunnels. I. Pennsylvania. State University. Dept. of Mineral Engineering. II. United States. Army. Corps of Engineers. III. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; GL-79-19.
TA7.W34 no.GL-79-19

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AD-A076 540 PENNSYLVANIA STATE UNIV UNIVERSITY PARK DEPT OF MINE--ETC F/G 13/13
TUNNEL DESIGN BY ROCK MASS CLASSIFICATIONS.(U)
SEP 79 Z T BIENIAWSKI DACW39-78-M-3114
UNCLASSIFIED WES/TR/GL-79-19 NL

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INFORMATION

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

IN REPLY REFER TO: WESGA

22 January 1979

AD-A076 540

Errata Sheet

No. 1

TUNNEL DESIGN BY ROCK MASS CLASSIFICATIONS

Technical Report GL-79-19

September 1979

1. On page 2, CONTENTS, line 21: Delete Bieniawski's Report . . . 57.
2. Page 57: Replace this page with the inclosed corrected page.

Input Data for Rock Mass Classifications

112. Input data to enable rock mass classification by the RSR Concept, the Geomechanics Classification, and the Q-System are listed in Figures C2 through C4, Appendix C. The data are presented for each structural region anticipated along the tunnel route. The best average ground condition (Table 23) was subdivided into two separate regions, basalt zones and shale zones. Station limits for each zone are shown in Figure 18.

113. Paragraph deleted.

114. It should be noted that all the data entered on the classification input sheets have been derived from the borings, including information on joint orientation and spacing. This was possible because borehole photography was employed for borehole logging in addition to the usual core logging procedures. However, considerable effort was required in extracting the data from the geological report for the classification purposes since engineering geological information was not systematically summarized in the form of classification input work sheets.

Assessment of Rock Mass Conditions by Classifications

115. Rock mass classifications in accordance with the Terzaghi Method, the RSR Concept, the Geomechanics Classification, and the Q-System are performed in Tables 19, 20, 21, and 22, respectively, and are summarized in Table 23.