DESIGN OF TUNNEL LINERS AND SUPPORT SYSTEMS

INTERIM REPORT

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by

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Design of tunnel liners and support systems.

PREFACE

This is the interim technical report submitted in accordance with the terms of contract No. 3-0152 between the U.S. Department of Transportation and the University of Illinois. The report is also a preliminary version of the first part of the final report for this project. As the research under the project continues and the second part of the final report is written, some revisions or deletions in the material discussed in these initial four chapters may be necessary or desirable.

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DESIGN OF TUNNEL LINERS AND SUPPORT SYSTEMS

CHAPTER 1

INTRODUCTION

1.1 Statement of Problem

It has been conservatively estimated that more than $35 billion will be spent on excavation for underground facilities in the United States during the next 20 years. Mining oriented projects will add $34 billion to this figure during the same period.

The construction of transportation tunnels is a slow, expensive and often dangerous undertaking. A large portion of the effort and expense is devoted to the lining of tunnels. Estimates for a number of different types of linings and tunneling procedures indicate that the cost of the lining material alone is of the order of 30 to 40 percent of the total cost of the tunnel and may even exceed 50 percent (Mayo et al, 1968). A survey of recent bids on tunnel projects of various kinds shows that the cost of lining material and installment may range from 10 to 20 percent of the total cost for machined tunnels in competent rock to more than 50 percent of the total cost for blasted tunnels in poor rock. It is thus clear that even minor improvements in the design and construction of linings can result in large savings.

Tunnel linings in the United States are at present designed primarily by simplified empirical methods. These methods usually have some root in theory but have been modified and influenced by intuition and experience so that this root is well disguised and easily overlooked. As a result, tunnel lining design is more of an art than a science.

It is doubtful that the art of tunnel lining design can ever be developed to a science comparable to structural design. The structural engineer can specify both the configuration and the properties of the materials making up the structure and the design usually can be accomplished by a direct application of theory. Theoretical solutions similar to those for structural design are available for stresses and deformations in lined and unlined tunnels in idealized materials. The tunnel designer, however, must work with an existing material that does not have the properties of the idealized material. In addition, the properties of this existing material are seldom accurately known and may change significantly within short distances. As a result, the mechanisms and concepts of material behavior assumed in the theoretical solutions are often not satisfactory.

Improvements in tunnel lining design methods are obviously needed. It is believed that they can only be achieved by an increased awareness of the mechanisms and modes of behavior of the system composed of tunnel and surrounding medium. It is also believed that this awareness can best be obtained by a study of the observed behavior of tunnels in the field. The purpose of this report is to propose new tunnel lining design methods based on a union of theoretical considerations and empirical knowledge from field observations, and on the basis of which improved and more economical tunnel linings can be developed.

1.2 Scope and Organization of Investigation

To improve design concepts for tunnel liners this investigation considers the problem in its entirety. Tunnels in all types of materials, constructed with various procedures, and supported by various types of liners are considered. The report consists of two parts.

The first part of the report starts with a consideration of the fundamental concepts of tunnel behavior and the effects of the construction process on the time-dependent equilibruim conditions of the tunnel-medium system. Then, the existing theories and practices of tunnel lining design are reviewed and critically examined. A general classification system for geologic materials is presented. In this system a general relation is drawn between geologic materials, the problems and behavior of tunnels in those materials, and the general design approach (soil mechanics, rock mechanics, or continuum mechanics) to be used. Classification systems for specific and specialized use are suggested.

Does Itaza have to go further?

In the second part of the report the behavior of individual lined and unlined tunnels is considered. Field observations of deformations and stresses in specific tunnels are examined and evaluated. Recommendations are given for improved tunnel lining design procedures and construction techniques. These recommendations are based on a fusion of theoretical considerations and empirical observations of behavior. Where possible, these recommendations include innovations in tunnel construction techniques and materials.

The report closes with recommendations for additional research to close the gaps in knowledge pointed out by the study.

CHAPTER 2

FUNDAMENTALS OF TUNNEL SUPPORT DESIGN

2.1 Outline of Problem

This investigation is a study of tunnel liners under a large variety of conditions. It includes deep and shallow tunnels in media from the softest soil to the most competent rock, and it considers the influence of discontinuities, non-uniformities, and environmental factors of many kinds. The breadth of the study is intended to permit establishing the basic concepts and criteria common to all tunnels, and to aid in separating and classifying problems in accordance with particular conditions.

A tunnel support system is often visualized as a liner or lining in more or less continuous contact with the medium around its circumference. This view is too limited. Other types of supports, such as rock bolts, may fulfill the necessary requirements. Often, the medium is perfectly able to support itself.

The support system finally provided in the finished tunnel is commonly termed the permanent support. On the other hand, supporting agents of various kinds are often installed before the construction of the permanent support. These are termed temporary supports. In this report, the general term "tunnel support system" includes both the permanent and temporary supports. The distinction is often of little significance unless the terms are related to specific construction procedures.

This study of tunnel supports consists of two basic elements: 1) a careful evaluation of what is desired of a tunnel support system under the varying conditions to which it will be subjected during its lifetime, and 2) an investigation to determine how each of these requirements can be met. As a rule the conventional support systems serve several purposes and may serve different purposes at different times. The direction of the most important loads, and the distribution of stresses and deformations around the tunnel may vary from the early construction stage to the final permanent stage.

By separating the varying functions and requirements of a tunnel support system, the suitability of current or new systems can be judged. The intended use of the final product establishes the requirements and tolerances for the permanent support. The most important requirements of the permanent structure are related to permanent stability, water control, and deformations after initiation of transportation service. Requirements such as immediate retainment of the medium dictate the need for and type of temporary support. The need for immediate retainment is greatly influenced by the character of the medium and the methods of excavation and construction. Both the transitory and the permanent requirements are influenced by environmental factors.

In the following sections, concepts and requirements of tunnel behavior are presented as a basis for a general approach to the design and construction of tunnel supports.

2.2 Concepts of Tunnel Behavior

Excavation of a tunnel opening and the subsequent construction of supports change the stress conditions for the tunnel and the surrounding medium. These changes may be continuous or in stages. A comprehension of the deformations associated with these changes is necessary for an understanding of the behavior of tunnel supports.

Changes of Equilibrium During Construction

The state of the medium before the excavation of a tunnel cavity is one of equilibrium in a gravity field. The process of tunneling evokes new equilibrium conditions which will change during the various stages of tunneling and construction of supports until a final equilibrium is reached. In this final equilibrium, all changes in strain and stress around the tunnel opening cease and the hydraulic conditions are restored to static (or dynamic) equilibrium.

A region of changing stresses, characterized by increased vertical pressure, travels ahead of the advancing face of the tunnel. Changes of equilibrium conditions are also felt at a considerable distance behind the face. The distribution of stresses has a three dimensional character at a point near the face, but approaches a two-dimensional state as the face advances. The rate at which the two-dimensional state is approached is influenced by the rate of advance of the face in relation to the time-dependent behavior of the medium.

The continuous or frequent changes in the conditions for stress equilibrium cannot take place without deformations in the medium. If supports are employed, these will deform as well. There is always an immediate deformation response to a change in equilibrium conditions, and commonly there is an additional, time-dependent response. In a waterbearing medium, the excavation of the tunnel changes the pore water pressures around the opening, and flow of water is induced. In fine grained materials with a low permeability, the establishment of hydrostatic or hydrodynamic equilibrium is not immediate. The associated time-dependent changes in effective intergranular pressures in the medium then lead to time-dependent deformations.

Time lags may also be associated with visco-elastic or visco-plastic phenomena such as creep in the medium itself or along joint planes in the medium. Whatever the cause of the time lags, their most important effect is that a final equilibrium for a set of boundary conditions often is not reached before new changes in boundary conditions occur.

Tunnel construction not only changes the equilibrium conditions but in many cases the medium itself. Blasting commonly reduces the strength of the rock around the opening; shoving by a closed or nearly closed shield disturbs and may remold the soil. Indeed, disturbing the material in the immediate vicinity of the opening is hardly avoidable. Where a tunnel is advanced without blasting in a medium which requires little or no immediate support, however, the disturbance may be minimal.

The Influence of the Support System on Equilibrium Conditions

Most tunnel openings are supported at some stage of construction. The behavior of a tunnel opening and a support system is dependent on the time and manner of the placement of the support and its deformational characteristics.

The reasons for providing support are manifold. Sometimes support is required for the immediate stability of the opening. It may be furnished even before excavation, for example by air pressure, forepoling or ground improvements. Under these circumstances the interaction between the medium and the supporting agent commences during or before excavation. When a shield is used for immediate support, a lining is erected inside the shield, and the annular void cleared by the shove of the shield is at least partly filled with pea-gravel and grout. The lining may be intended as a permanent support consisting, for example, of cast iron. It may alternatively be a relatively flexible one in which a stiffer permanent lining may later be constructed. In this event,at least three different equilibrium conditions must be considered.

Where there is need for long-term but not immediate support, the support may be constructed at some distance behind the face. A partial relaxation with associated movements may then take place before the support interacts with the medium.

Often a liner is erected and expanded into contact with the medium. The expansion induces a prestress in both the liner and the medium and influences subsequent deformations.

Even where instability or callapse of the opening is not imminent, support may still be required for various reasons, usually to control or limit deformations. Large deformations may lead to undesired settlements of the ground surface or to interference with other structures. Such deformation must be restrained at a suitably early stage. Deformations of a soil or rock mass commonly result in an undesirable reduction in strength and coherence of the medium. In a jointed or weak rock the material above the opening tends to loosen and may sooner or later exert considerable loads on the support. These loads are reduced if loosening is prevented by suitable support.

Although the initial stability may be satisfactory, conditions may be such that final equilibrium cannot be reached without support. This may occur in a jointed rock subject to progressive loosening, in creeping or swelling materials, and in materials whose strength decreases with time. Except in such creeping materials as salts, these long term phenomena are associated with volume changes.

It is impossible and undesirable to avoid deformations in the soil or rock altogether. Some movement is necessary to obtain a favorable distribution of loading between the medium and the support system. In each instance, the engineer must determine how much movement is beneficial to the behavior of the tunnel, and at what movements the effects will become detrimental. The engineer's conclusions regarding these matters determine whether and where restraints are to be applied to the tunnel walls. His conclusions also determine the character and magnitude of those restraints. In tunnels in hard rock the beneficial movements take place almost immediately, and subsequent movements are likely to lead to loosening and additional loading. Hence, in this case rapid construction of supports is usually desirable.

It is apparent that many factors determine whether and where a support system should be constructed for structural reasons alone. The choice of whether and where supports are actually employed is influenced by additional factors such as the psychological well-being of the workers, or the economy that might be achieved by adopting a uniform construction procedure throughout the same tunnel even though the properties of the medium vary.

No matter what the reason for using restraints, the loads to which a support will be subjected depend on the stage of equilibruim prevailing at the time the support is introduced. Thus, if final equilibrium has been reached before support is provided, the support does not receive loads from the medium at all. On the other hand, when support is furnished before final equilibrium has been established, new boundary conditions are superimposed on the conditions existing when the support is constructed. The new final conditions depend on the time the support was provided and involve the interaction between the support and the medium. If a stiff support could be installed in the medium before excavation by an imaginary process that did not in any way disturb the remaining material, it would be subjected to stresses resembling those of the in-situ condition existing before the excavation. However, the at least temporary reduction of the radial stresses to atmospheric pressure (or to the air pressure in the tunnel), as well as many other activities, generally introduce such deformations into the medium that the stresses acting on the tunnel support bear little or no resemblance to the initial stresses in the medium.

Procedures for the analysis and design of tunnel supports are necessarily simplified, but they should be based on the considerations of equilibrium and deformations briefly outlined above. In addition, a number of factors which are not directly related to the interaction between a support system and the medium are significant in the actual design of supports. Such factors, which are dealt with in the following section, sometimes even override considerations of structural interaction.

2.3 Constructional and Environmental Factors

The preceding section dealt with the influence on the behavior and requirements of a tunnel support system of excavation procedures, of the disturbance of the medium, and of the structural interaction between the medium and the support. Other factors significant in tunnel support design may be separated into constructional details, structural non-uniformities and environmental factors.

Effects of Constructional Details and Structural Non-Uniformities

One of the most important requirements is providing thrust for the propulsion of a shield or a tunnel excavator. In competent media this thrust can be developed by jacking friction blocks against the tunnel walls or by means of a pilot anchor, but in less competent media the thrust is almost always obtained by jacking against that part of the lining already erected. The stresses in the liner caused by this thrust are considerable and commonly determine the design of the liner.

Construction procedures are not generally uniform, even in a uniform medium. The most important constructional non-uniformities are associated with the quality of the contact between the tunnel support and the medium. Irregular graveling and grouting of an annular space between a liner and the medium, or non-uniform blocking behind a steel rib may cause considerable non-uniformity of stresses and even stability problems.

Structural non-uniformities, i.e., disruptions of the structural uniformity of the tunnel, have considerable impact on the feasibility and economy of a tunneling support scheme. Structural non-uniformities include, for example, enlargements of the opening for stations, crossswalks connecting twin tunnels, ventilation shafts, and emergency exits. These features may entail disruptions of the uniformity of the construction procedures, and they also give rise to constructional and design problems in themselves, especially since they often are of irregular, non-symmetrical shape and are of different rigidities than the main tunnel. Where the construction involves the opening of a wall or the roof of an already finished tunnel, new and more complex equilibrium conditions are introduced which may call for special stability considerations. Whereas a structurally uniform tunnel may often require only nominal lining, the savings inherent in a refined design may not be possible because of the necessity for the frequent changes in section at and near the non-uniformities.

Environmental Factors

Sources of loading and deformation not in direct contact or relation with the tunnel may also pose problems of importance. These sources include structures existing before the tunnel was built, and structures added at a later date. The problems are of three types. They include the effect of tunneling operation on existing structures such as nearby buildings, deep basements, or adjacent tunnels. They also include the influence of such structures on the tunnel excavation and the behavior of the support system. Most important, however, they are concerned with the effect of future construction which will alter the equilibrium conditions of the tunnel after the start of the transportation service. At this time requirements for the behavior of the tunnel may be strict and deformations associated with equilibrium changes may be critical.

Traffic tunnels are often constructed in pairs, and sometimes even three or more tunnels are driven close together. The interaction between a tunnel already driven and one being driven is of considerable importance. Considerations of this interaction may determine major features of the design as well as details of construction. In a weak deformable medium, the driving of a second tunnel in a region where stresses have already been increased by a previous tunnel may be much more difficult than driving the first tunnel. Furthermore, the first tunnel may experience substantial deformations and its support system may be subjected to large additional loads on account of driving the second tunnel. The vibrations and the disturbance of rock due to blasting dictate a minimum distance between adjacent tunnels excavated by blasting in rock.

Alteration of the hydrological environment because of the construction of the tunnel also deserves attention. The alteration may be temporary or permanent, depending on the water-tightness of the tunnel. Lowering of the groundwater table increases the effective stresses with resulting settlements, most of which are irreversible. An engineering decision must usually be made as to whether the lining should be made watertight (and the ground water table restored to its original position), or should be made to act as a drain. The first decision implies that the lining must eventually withstand the hydrostatic pressure; the second requires that the drains will always be functional.

Simplifications for Design Purposes

The foregoing discussion demonstrates that a great many factors other than the interaction between the tunnel support system and the medium are involved in the engineering decisions pertaining to the design and construction of the tunnel supports. Often one or several of these factors actually control the design. However, current design procedures consider these factors only indirectly. The relevant factors may be considered in some detail, but the final result of the considerations is a simplified design procedure applicable to a length of tunnel which may involve a variety of media and a variety of conditions. A suitable safety factor is applied to the most unfavorable conditions expected. Restrictions on future construction on the right-of-way are sometimes established as precautions against unreasonable inroads on the factor of safety.

The simplified design procedures do not reflect the complex problems that may have influenced their development. They generally treat the lining, temporary or permanent, as a structure subjected under static conditions to given loadings. Occasionally, in the more advanced versions of the procedures, the loadings are taken as dependent on the deformation. The general applicability of such design procedures is severely limited. Hence, the procedures must be reevaluated each time a tunnel is driven under even slightly different conditions.

2.4 Importance of Classification of Geologic Media

The preceding sections may be regarded as an informal classification of the problems associated with the design and construction of tunnel supports. Different types of problems result from the changing equilibrium conditions, from the disturbance of the surrounding medium, from the interaction between the supports and the medium, and from structural, constructional, and environmental factors. Any one of these problems is influenced by the character of the medium through which the tunnel passes. Indeed, the character of the medium is probably the most important of all the variables. A rational approach to the design of support systems must from the outset include an evaluation and classification of this variable.

A classification of geologic materials with respect to tunneling should delineate the types and severity of problems connected with tunnel support, and it should indicate which characteristics of the medium should be included in investigations and evaluations. With its aid, the appropriate theories and analyses can be chosen for the design procedures, and suitable construction methods can be applied.

A wholly new system or method for classification of geologic materials is not considered necessary. The fields of soil and rock mechanics are already crowded with such systems. Instead, an attempt has been made in this study to unify already accepted terminology and classification systems so that the considerable experience with these systems can be fully utilized.

The most important characteristic separating soil and rock is the relative importance of discontinuities. As a rule, the behavior of rock is dictated by the spacing and nature of discontinuities such as joints, whereas soil can commonly be assumed continuous and homogeneous. These considerations suggest that a basic parameter for classification should be related to the characteristic size of a unit such as the grain size of a soil or the joint spacing of a rock.

A useful classification based on inherent characteristics of the material is presented in Chapter 4. A classification of a general and conceptual character is first developed. Specialized subsystems, useful for diagnosing particular problems and applicable within particular ranges of characteristics of the medium, are then developed on the basis of an accumulation of experience. In previous classifications only verbal descriptions of different rock classes have commonly been used. It is a purpose of this investigation to substitute numbers for qualitative descriptions.

2.5 The Relevance of Empirical Evidence

Although a classification of geologic media is important, it is of little use by itself. To be useful it must be related to problems of interest to engineers, and it must be substantiated by field evidence. This field evidence consists of observations, both qualitative and quantitative, of the actual behavior of tunnel openings of various geometries, constructed in different ways in different media. Quantitative observations in tunnels can be classified generally as measurements of stress or strain (pressure or deformation).

It is difficult to measure stresses in a tunnel liner and more difficult to measure those in the surrounding medium. Any stresses that are measured are not total stresses but only the changes in stresses from those that existed when the instrument was installed. Since stresses are obtained from strain observations, they are subject to more or less inaccurate assumptions concerning the stress-strain behavior of the materials.

In liners the measured stress changes may closely represent the actual stresses because the stresses in the lining immediately after installation are probably close to zero. The installation of the instrument in the liner causes only a small disturbance of the actual stresses carried by the liner. By contrast, the stresses in the medium at the time of installation of the liner are, in general, not known and the very act of installing the instrument causes further unknown changes in the stress field. It is more difficult to measure stresses at a point in the medium remote from the tunnel than it is to measure the relative displacement of that same point with respect to the tunnel wall at any fixed point.

Because of these problems, there is a tendency to devote most quantitative observations to measurements of stresses in the liner and of displacements of selected points in the medium. Measurements of the changes in the overall inside dimensions of the lining are also often made.

Empirical evidence of deformational behavior is more reliable (and more common) than measurements of stress or load. This evidence can be used to investigate correlations of the behavior of tunnel openings and support systems with properties of the medium, and to improve general understanding of the mechanisms involved in the interaction between the tunnel and the medium.

Under any circumstances the measurements taken are strictly applicable only to the particular tunneling procedures used and to the properties of the medium where the measurements were taken. The results of measurements taken in a blasted tunnel, for example, are strongly affected by the loosening and fracturing of the surrounding medium; a disturbance which may be entirely non-existent in a bored tunnel in the same medium. Thus, though case histories are highly informative, and, indeed, essential, considerable care must be taken in extending past experience to present application, especially if novel procedures are involved.

2.6 Basic Approach to Design of Tunnel Support Systems

The previous sections have outlined the features that must be taken into account in the design and construction of tunnel support systems. An investigation seeking to improve the capability of the engineer in this respect must not neglect any of these features. Some fundamental principles can be derived from them on the basis of the preceding discussions.

It is apparent that economy can be achieved by taking advantage of the capability of the medium to support itself. By proper choice of a support system and of the stage or stages of construction at which it is installed, the loads on the support can be minimized and the medium can be made to take a major portion of the load. Thus, the support system should be regarded primarily as a reinforcement or restraint that helps the medium to support itself.

It is easily demonstrated that this concept of helping the medium to help itself is reasonable and possible. The principal action of rock bolts, for instance, is to arrest movements towards the opening and thereby to increase the capacity of the rock to carry tangential stresses. The increase is achieved by transferring forces so far away from the opening that they can be dissipated safely. The principal load-carrying agent is, in fact, the rock itself. Another example is the support of an opening by shotcrete immediately after excavation. Shotcreting hinders movements and loosening of the rock making it capable of carrying more load than is possible with conventional supports which generally allow loosening.

Control of the movement of the medium is important. While too large movements may lead to loosening or weakening of the medium and may render it incapable of sustaining loads, too small movements may in some instances not allow the medium to carry the loads of which it is capable. An approach based on control of deformations is likely to be the key to the successful design and construction of tunnel supports.

The preceding section pointed out that empirical evidence of deformational behavior is more reliable than measurements of stress or pressure. Design methods based on a deformational approach are generally of a less sensitive character than those based on more or less arbitrary assumptions of loading. This situation further illustrates the validity and potential of a deformational approach.

The condition and properties of the medium will to a considerable extent dictate when and how deformations should be controlled. The response of the support to loading is also significant. A support may be chosen which will allow the medium to adjust itself a specified or controlled amount. For given conditions there should exist an optimum combination of flexibility or rigidity, and adjustability. It is a purpose of this investigation to improve the understanding of these mechanisms and provide guidelines for design on this basis.

The foregoing considerations apply to a particular tunnel opening under given conditions. It may not be worth-while to expend great effort to arrive at the most suitable support system for all the conditions of a given tunnel alignment. The reliability of engineering predictions and the likelihood of hazardous situations depend on the uniformity of the soil or rock medium. Yet, it is impracticable to obtain complete information about the subsurface conditions. Therefore, when conditions are known to be non-uniform, three different approaches are commonly used: 1) A tunneling scheme may be adopted which is safe and as economical as possible under a variety of conditions including the worst that can reasonably be foreseen; this method is conservative. 2) The tunneling scheme may be designed such that most unpleasant surprises can be handled safely and changing conditions can be accommodated by minor modifications; this method is more economical provided there are not too many surprises. 3) Advance investigations may be made by borings from the tunnel face, by pilot drifts, or by other detailed exploration, so that safety precautions and modifications can be made ready for application as the changed conditions are exposed; this method is safe but the cost of the explorations may exceed the possible savings that can be realized on account of the knowledge obtained from them.

Where conditions are known to be uniform, a specialized, though inflexible, method of tunnel construction may be suitable and economical.

A balance must be found between the uniformity of the medium, the amount of advance information to be obtained about the medium, and the ability of the tunnel support procedures to be modified in accordance with the variability of the medium. It is hoped that this investigation will facilitate the engineering decisions leading to this balance.

CHAPTER 3

EXISTING THEORIES, DESIGN METHODS, AND PRACTICES

3.1 Introduction

In recent years, the number of tunnels has greatly increased. Many new theories and methods of tunnel lining design have been advanced. However, tunnel lining design is still more of an art than a science.

This art has produced tunnel linings which, in general, have shown satisfactory performance. The degree of conservatism in the design of these tunnel linings is unknown but could be very significant. Improvements in the art are not possible without first gaining an understanding of its present level of development. The following sections include a critical summary of existing theories, methods, and practices of tunnel lining design. Their uses and misuses are emphasized, and the ideal conditions under which they may be applied are discussed. Where possible, the error resulting from applying them under non-ideal conditions is indicated.

Karoly Szechy presents in his book, "The Art of Tunnelling" (1966), a large collection of theories and methods of tunnel lining design. Although Szechy's selection and comments are rather uncritical, his book is a compact source of additional information about the subject.

In many studies of tunnel behavior, only the medium itself is considered. In these studies, the soil or rock is considered a continuum and static states of stress and deformation around unlined openings are investigated. By various approximations, the effects of time and of a supporting liner may be added.

Another approach largely disregards the be considers a lining structure loaded by more or less may, with some complication, be assumed deformation the loads on the structural liner, a large number of empirical theories may be used, including arching ar and methods based on a continuum approach.

Current practice in tunnel lining design t estimate of soil or rock load and the design of the independent processes. These two aspects of the des responsibility of different groups.

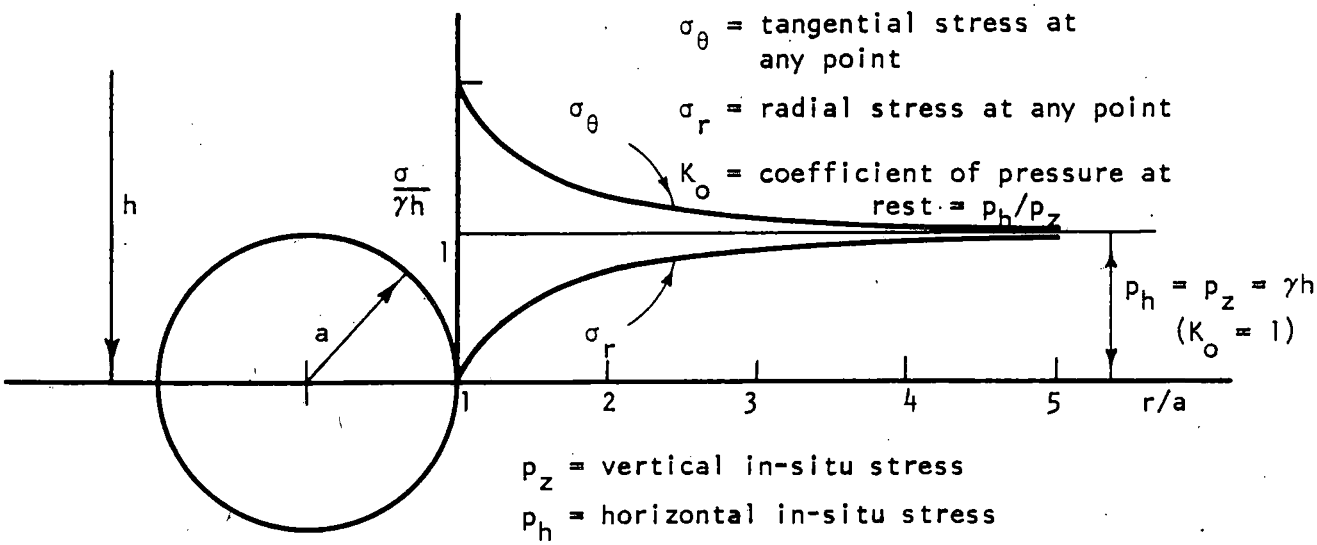
3.2 Analyses Based on Continuum Approach

In general terms, the ground in which a tu can be considered as either a continuum or a discont report, a continuum is a material in which macroscopic physical properties such as stress or density can be u models to define the mechanical behavior of the materi may be approximated by a continuum, tunnel behavior with theories of continuum mechanics. Continuum app to tunnels include elastic analyses, elasto-plastic tests.

Elastic Analyses

The theory of elasticity may be used to fi around a hole in an elastic, isotropic, and uniform are available for unlined circular openings in a pla for any combination of horizontal and vertical loadings. The use of the theory of elasticity permits solutions that include the rock weight and arbitrary boundary conditions. These solutions, such as Mindlin's (1940), Schmied's (Szechy, 1966), and Savin's (1961), result in equations for the tangential stress, σ_θ, and the radial stress, σ_r, around the opening, and the radius of the zone in which the stresses are significantly influenced by the opening. The equations are rather elaborate and include a number of constants to be determined by the boundary conditions. In most cases a simplified solution, Kirsch's solution (Timoshenko and Goodier, 1951), which disregards the influence of the proximity of the surface, will suffice. The results of this simplified solution, which assumes a homogeneous stress field, are given in Appendix 1.

Figure 3.1 indicates the type of stress distribution obtained with these equations when $K_o = 1$, i.e., $p_h = p_z$. In subsequent sections the modifications in the highly stressed zone near the opening resulting from plastic behavior will be considered.



a = radius of the circle

r = distance to the point under consideration

h = depth of cover

These solutions cannot be applied indiscriminately since neither soil nor rock is an elastic, isotropic, and homogeneous material. For a few special cases, such as machine driven tunnels in some rocks, elasticity solutions may be used with quite acceptable accuracy. However, stability problems seldom occur in such rocks. For other media, and especially when the medium is weakened by the tunneling operation, the elastic solutions are unacceptable for the prediction of stress-strain behavior. Even in such cases, however, Cording (1968) found that the elastic solutions may accurately predict the small, approximately elastic, displacements which occur immediately upon removal of rock support by excavation. He used a finite element method to analyze stresses and deformations around openings of irregular shapes and with K_o values different from unity. The deformation results were in reasonable accordance with measurements of immediate deformations taken in cavities at the Nevada test site. Displacements associated with instability and plastic behavior cannot be predicted by these solutions.

Elastic solutions can be found for shapes other than circular and for problems involving elastic linings in elastic media. Although the principles are the same for the solution of these problems as for the problem of a circular opening in an elastic medium, the actual execution of the solution is much more involved. Elasticity problems are well suited to computer solution. By the use of a computer the effects of variations in the parameters and complicated configurations and boundary conditions can be studied.

Elasto-plastic Analyses

One of the first steps in attempting to improve the applicability of theoretical solutions to computing stresses around unlined tunnels is to introduce elasto-plastic properties for the materials. The simplest elasto-plastic solutions assume the horizontal in-situ stress (p_h) equal to the vertical in-situ stress ($p_z = z\gamma$), i.e., $K_o = 1$. The failure or yield criterion used is either Tresca's criterion (corresponding to φ=0) or a Mohr-Coulomb criterion (corresponding to a c-φ material). These solutions are given in Appendix 1.

An annular plastic zone is developed around an unlined tunnel when the stress level p_z exceeds the unconfined compression strength of the material, $q_u = 2c$ for a frictionless material, or $q_u = \frac{2\ c\ \cos\ \varphi}{1 - \sin\ \varphi}$ for a frictional material. When an internal pressure p_i is applied to the tunnel walls, for example by a lining or by air pressure, the stress level required for the development of a plastic region is increased, as indicated by the formulae given in Appendix 1. For a cohesionless material (c=0), $q_u = 0$, and the opening is unstable without an internal pressure.

The radius of the plastic zone depends on the stress level p_z, the internal pressure p_i, and the strength constants c and φ in the manner indicated in Figs. 3.2 and 3.3. It is apparent that both the constants c and φ have a great influence on the radius of the plastic zone.

Examples of stress distributions around an opening in a frictional material are shown in Fig. 3.4. It is interesting to note that the radius of the plastic zone increases as the ratio p_z/p_i increases.

When K_o is not equal to unity, or where the tunnel is so shallow that the effect of the proximity of the ground surface cannot be disregarded, these results cannot be used directly. However, the elastic solutions may be used to suggest at least when and where a plastic zone will be indicated. Whenever the maximum tangential stress at a point on the tunnel wall exceeds

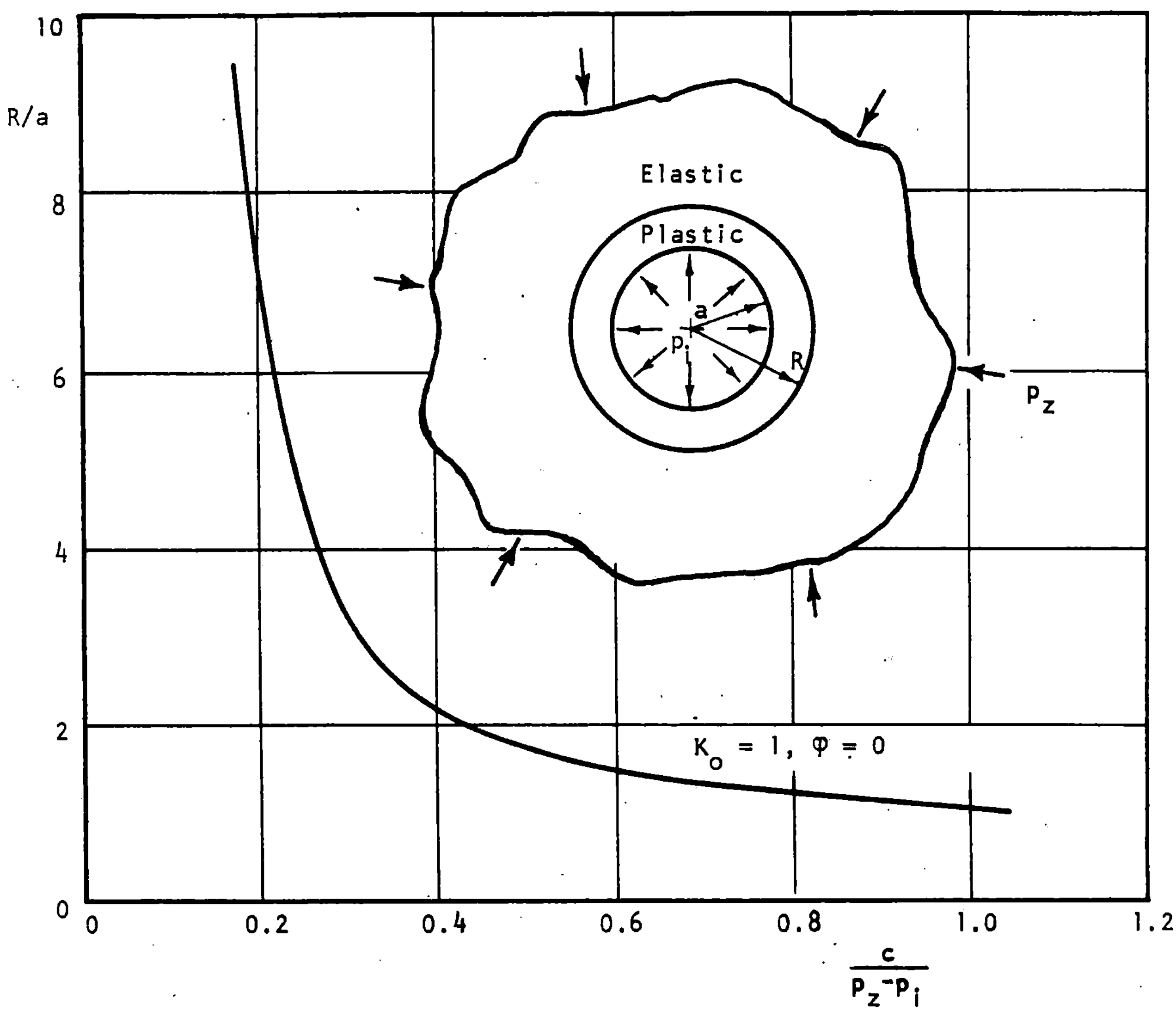


Fig. 3.2. Relation Between the Radius of the Plastic Zone, R, One-half the Unconfined, c, the Overburden Pressure, p_z, and the Pressure in the Opening Interior, p_i.

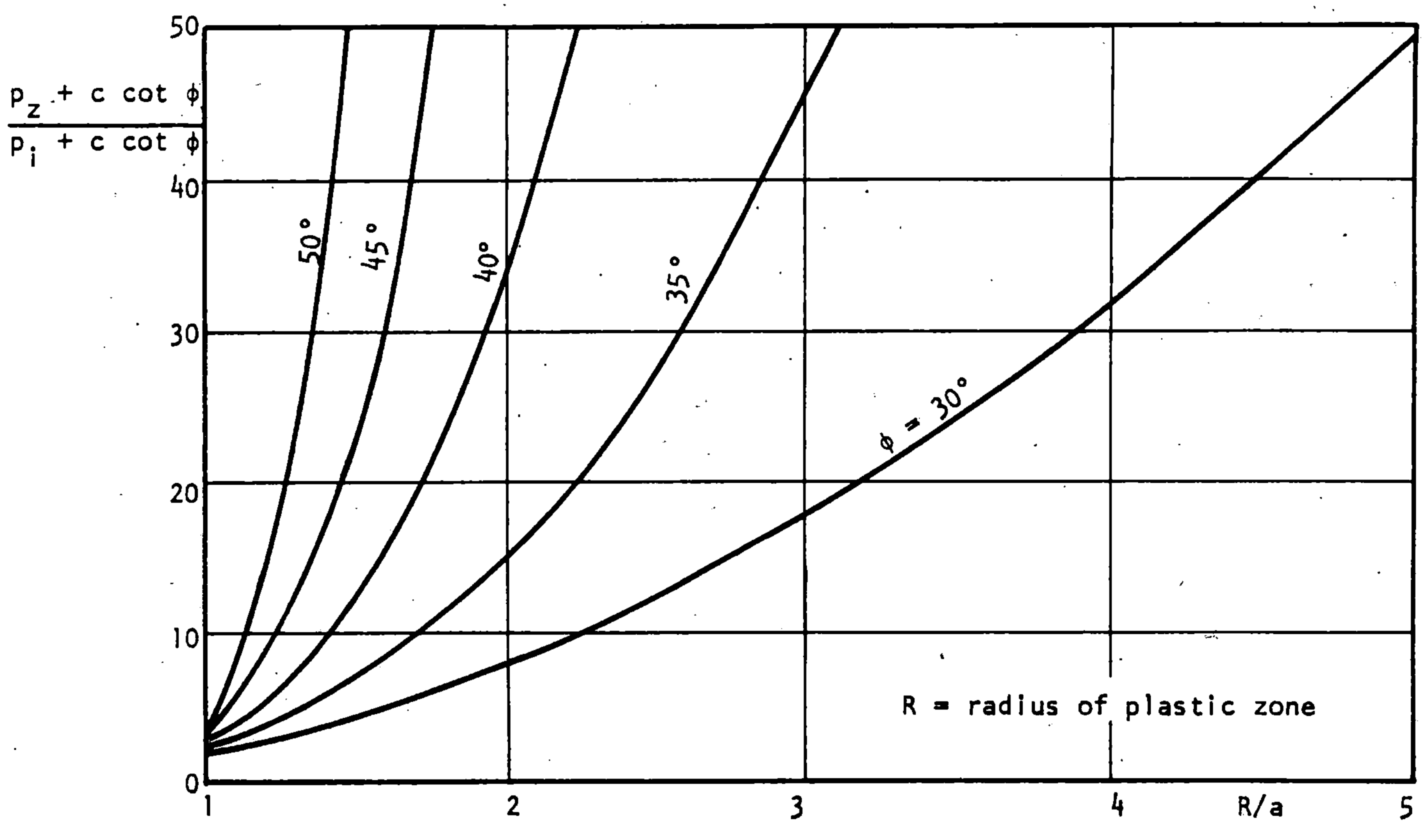


Fig. 3.3. Radius of Plastic Zone vs $\frac{p_z + c \cot \phi}{p_i + c \cot \phi}$

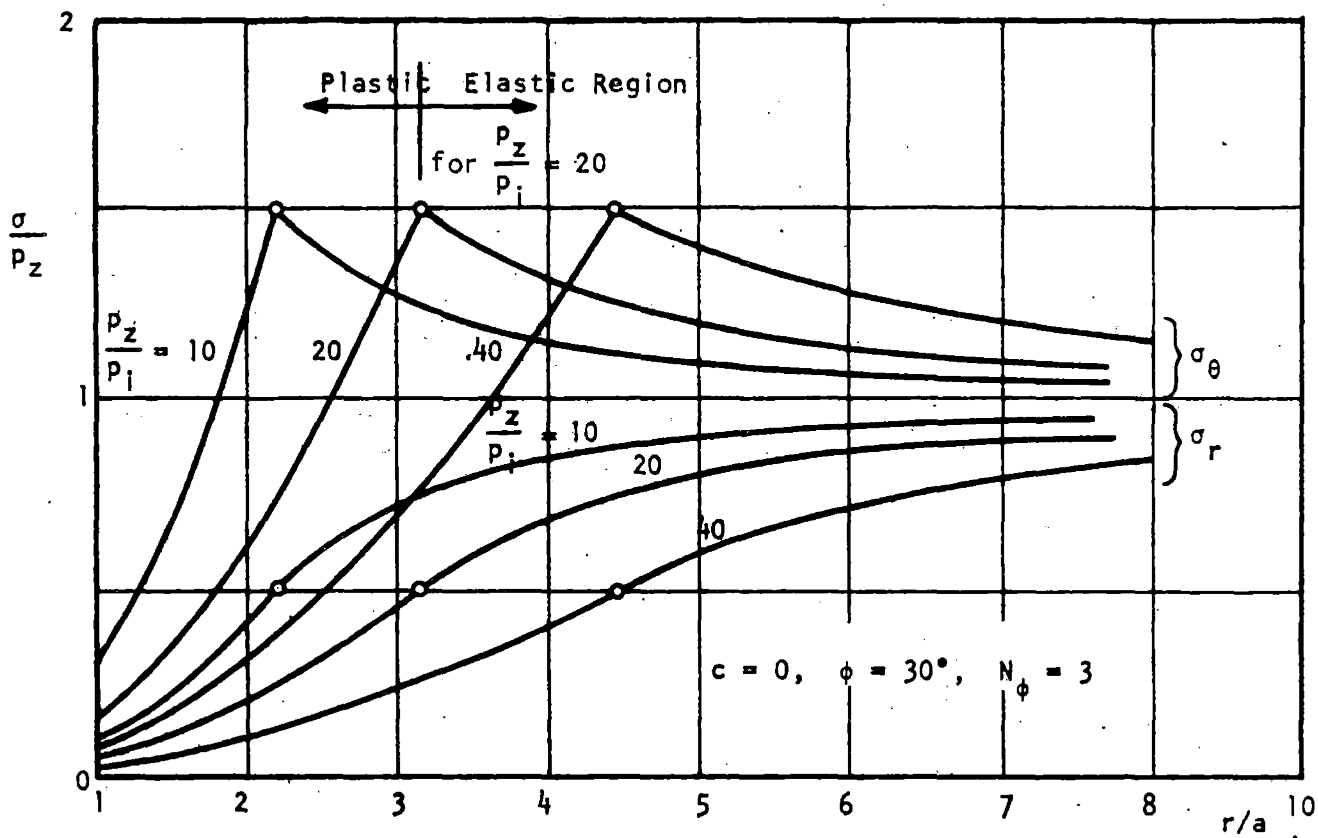


Fig. 3.4. Distribution of Stresses Around a Circular Tunnel in Elasto-Plastic Material

q_u, the material will yield and a plastic zone will develop. If the plastic zone is small, its extent can be found approximately by comparing the stresses found by the elastic solution with the yield stress of the material. Wherever the yield stress is exceeded, the material is in a plastic state. This method, of course, quickly becomes inaccurate with increasing size of the plastic zone, because it disregards the stress redistribution caused by the plastic behavior of the material.

Deformations of the tunnel walls can be computed on the basis of both the elastic and the elasto-plastic analyses. These deformation estimates are of ratherlimited interest: the elastic deformations are immediate and take place before a liner can be placed; the elasto-plastic deformations are ultimate deformations and do not include time-dependent behavior.

Another method of studying the plastic behavior of rock around an underground opening shows considerable promise. By means of computers, problems with a large variety of boundary conditions (for example, non-circular openings) and any initial stress condition (K_o not equal to unity) can be studied.

Reyes (1966) used a finite element method to study non-circular openings in a frictional material with several values of K_o. This analysis is based on a generalized von Mises yield criterion. In principal stress space this criterion plots as a surface having the shape of a right circular cone with its axis along the space diagonal. As long as the point representing the principal stresses at any instant remains inside this surface, strains are elastic. At the surface, yielding begins. Movement of the point along the yield surface causes both elastic and plastic strain.

Reyes' flow rule predicts an increase in volume with yielding. This limits the applicability of the theory to dilating materials and even then the predicted rate of dilation is much larger than observed by tests. The flow rule may be a reasonable assumption as long as the plastic region is small, but with additional yielding the results become unreliable. The deformations computed by this method are less reliable than the stresses.

Reyes found that for $K_o = 0.25$ and 0.40 the plastic zones did not surround the circular cavities studied in detail but propagated along 45-degree directions. This is in accordance with predictions on the basis of elastic analyses (Fenner, reported by Szechy, 1966). Reyes' results have not been verified by field measurements.

Model Tests

Laboratory models can be used to evaluate and improve the theoretical solutions and to improve the engineer's judgment. Photoelastic experiments permit the study of stress conditions around openings of any shape in a plane stress field. Comparisons of photoelastic results for a circular opening with the corresponding theoretical results have shown good agreement. This agreement encourages belief in both the theory for circular openings and the photoelastic results for other shapes of openings. As a result, photoelastic analyses for complicated geometries, i.e., horseshoe and multiple tunnels, are valid representations of the elastic stresses.

Photoelasticity is, in fact, simply an elastic method of solution that produces visual results. Though it is usually applied to two-dimensional problems, techniques for three-dimensional problems exist. These techniques involve special procedures for stress-freezing models and taking slices for analysis. Because of their complexity and because most tunnel problems are primarily two-dimensional, these techniques are seldom applied to tunnel investigations.

Other experimental investigations are conducted using blocks of man-made rock, such as sand cemented by gypsum or plaster-of-Paris. Tunnels are machined into the unstressed blocks, the model loaded, and its behavior measured. One such series, loaded in plane stress, is being conducted by the Omaha District of the U. S. Army Corps of Engineers. A second series, loaded in plane strain, is being conducted at the University of Illinois (sponsored by the Waterways Experiment Station). These experiments study both the elastic and the plastic behavior of tunnels under a variety of conditions. It is expected that the results of these experiments will provide valuable evidence of the behavior of tunnels in a continuum.

Discussion

The continuum approach to tunnel analysis should be applied directly only when the medium around the tunnel truly meets the assumptions made for the continuum. This may be the case in massive, unfractured rock or in some uniform clay or sand deposits. Even when the medium may be considered continuous, the necessary assumptions concerning the stress-strain relationship for the material will render the analyses inaccurate.

In a discontinuum, the theories discussed above may be used as the first step of an analysis. However, the engineer will find it necessary to evaluate the assumptions of the theory in light of his understanding of the actual behavior of the material. Blind application of the theories without this evaluation could lead to serious mistakes.

As computers become more advanced, it is certain that they will be applied repeatedly to the study of tunnel behavior. Without this valuable tool, many analyses could not be used because of the time involved in solving complicated equations manually. The computer can solve the equations very quickly. Thus, a far greater variety of configurations, stress conditions, and material properties can be included in the analyses. The results can be of great value for developing an appreciation of the effects of changes in these variables.

Model tests in the laboratory are useful in two respects. First, they provide a check of theoretical solutions. The tunnel configuration and the medium around the tunnel are made to match the theoretical assumptions. Test results then provide evidence of the applicability of the theory. Secondly, model tests can be used as an independent tool. Models can be built for problems that cannot be solved easily by theoretical solutions. Scale factors can then be used to project the test results up to the prototype. These tests require a good knowledge of and adherence to the laws of similitude.

Limitations in manufacturing a material to model the properties of the medium in the field, in changing the properties of the material, or in creating the proper stress conditions often impair the usefulness of model tests.

In materials that are not perfectly elastic, the loading techniques used in laboratory tests may lead to error. Most tests are conducted by loading a block in which a tunnel was drilled prior to loading. Under field conditions, the tunnel is excavated in a mass that is already under stress. Thus, even when the boundary conditions have apparently been duplicated, a different loading path has been followed in the laboratory than in the field.

Plasticity theory predicts that the behavior is influenced by the loading path.

Before any elastic or elasto-plastic theory can be accepted for predicting the stress-strain behavior of the medium around a tunnel it is necessary to show by field or laboratory measurements that the results are reasonable. Some field measurements, such as Cording's work outlined above, have indicated reasonable agreement with theory in the elastic range. The number of such measurements, however, is not sufficient. Additional measurements are needed for tunnels in a wide variety of geologic conditions and for a variety of construction and support techniques. In some cases these measurements will permit the correlation of theory with observed field behavior. In other cases they will lead to the development of empirical or semi-empirical guidelines for design when the theories are not directly applicable.

3.3 Liners as Structural Units Subjected to External Loads

The preceding section dealt primarily with the behavior of unlined openings. This section, in contrast, deals with methods of lining design which largely neglect the behavior of the surrounding material and substitute forces on a structural liner for the action of the medium. If the external forces on a lining are known, the moments, shear forces, and thrusts can be determined at any point in the lining by structural analyses. Irregular lining shapes also can be studied in this manner.

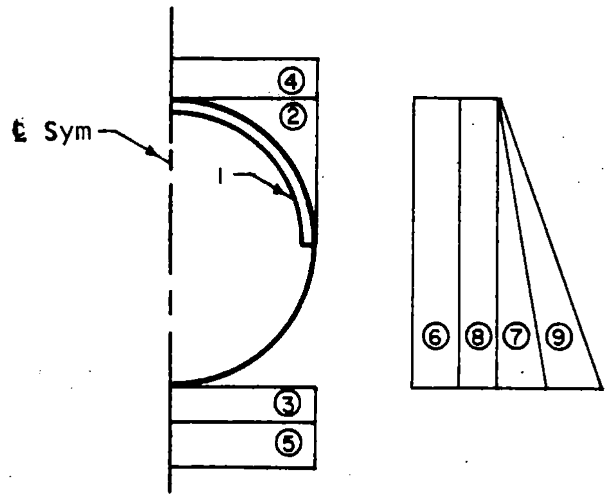
The major difficulty with these methods is the estimation of the external loads. These loads are often chosen as arbitrary, though possibly reasonable, loads which are independent of the deformation of the lining. The method is sometimes improved by the selection of external loads which vary with the deformation.

Hypothetical Constant Loads on Liners

The hypothetical constant loads used for the design of liners are usually based on simplified hypothetical mechanisms of behavior of the soil or rock around the liner. The loads may be expressed, for example, as functions of the friction angle and unit weight of the soil but do not really represent actual soil behavior. One of the first hypothetical loading schemes for a circular tunnel section was proposed by Hewett and Johannesson (1922). It is summarized in Fig. 3.5. The engineer must estimate the value of α, the angle of repose, use α to find c (the active earth pressure coefficient) and 1/c (the passive earth pressure coefficient) and then decide if the earth pressure is active, passive, or somewhere between the two.

This loading system is then applied to an elastic, elliptical ring and general equations for thrust, shear, and moment in the liner are derived. As a final step these equations are specialized for such conditions as "wet ground," "dry ground," and "firm water-bearing ground." It should be noted that interaction between the tunnel lining and the medium can be included to some extent in the selection of the value of K_o. The authors include a figure of c vs. shortening of the vertical axis for one tunnel.

Spangler (1960) has conducted a number of studies of the loadings on buried, flexible conduits such as corrugated-metal pipe highway culverts. He assumes a pressure distribution as shown in Fig. 3.6. He bases the vertical load on Marston's theory, which is an arching theory in principle similar to Terzaghi's (1943), and the horizontal pressure is taken as the product of the modulus of passive resistance of the fill and one-half the horizontal deflection of the pipe. The conduits studied by Spangler were installed in open cuts or



Loads

1. The weight of the upper half of the tunnel.

2. The weight of the earth within the area marked 2.

3. A uniform upward force balancing 1 and 2.

4. The weight of the loading above the top of the tunnel.

5. A uniform upward reaction balancing 4.

6. The horizontal pressure due to the water above the top of the tunnel.

7. The horizontal pressure due to the water from top to bottom of the tunnel.

8. The horizontal pressure due to the earth above the top of the tunnel equal to the product of the weight of earth (bouyant unit weight if submerged) above the top of the tunnel and the factor K, where

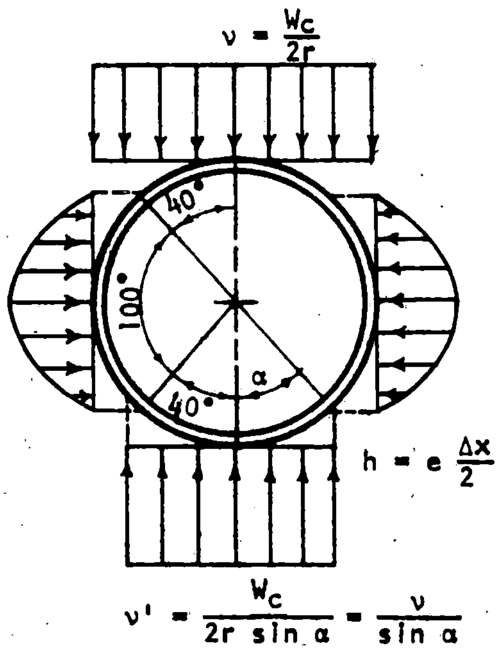
$$C < K < 1/C \quad .$$

9. The horizontal pressure due to the earth between the top and the bottom of the tunnel. At any point, the pressure is the product of the weight of soil between that point and the top of the tunnel and the factor K. Soil weighed as in 8.

Note: $C = \dfrac{1 - \sin \alpha}{1 + \sin \alpha} = 1/3$ to 1 $\qquad \alpha$ = angle of repose of ground.

Fig. 3.5. Design Loads for Tunnel in Soil
(After Hewett and Johannesson, 1922)

on the ground surface and then backfilled. Therefore, his assumed stress distribution has only limited application to driven tunnels.



Δx = horiz. deflection of the pipe, in inches, same as vert. deflection

W_c = vert. load per unit length of pipe, pounds per linear inch

r = mean radius of the pipe, in inches

e = modulus of passive resistance of the enveloping soil, in pounds per square inch per inch

Fig. 3.6. Assumed Pressure Distribution on Flexible Pipe (After Spangler, 1960)

Many other methods of estimating loads on tunnels for use with this or a similar method have been proposed. Some of these (for example by Maillart, Jaky, and Balla) are discussed by Szechy (1966). None of these methods can be said to properly represent actual soil or rock behavior.

Deformation-Dependent Loads

The loads discussed above are, in general, not dependent on deformations, except that in some cases the engineer may choose load factors compatible with expected deformations. The theories discussed in this section are based on the assumption of a direct one-to-one relationship between deformation and load (Winkler theory, see for example, Terzaghi, 1955).

The Muller-Breslau principle states that the deflected shape of a structure represents the influence line for a function such as stress, shear, moment, or reaction component if the function is allowed to act through a unit distance (McCormac, 1960). Anders Bull (1944) uses this principle to derive equations for the moment, shear, and thrust at any point in a tunnel liner due to concentrated loads around the periphery of the tunnel. He then approximates the distributed loads on the tunnel by replacing them with concentrated loads at a spacing equal to one-sixteenth of the periphery. He suggests that the vertical active pressure in a sandy soil be taken as the overburden pressure and the horizontal active pressure be taken as one-third of that.

Expressions for the radial and tangential deflections at any point due to concentrated loads on the periphery are found. Two sets of equations for the same deflection are then written: one set in terms of the soil reactions (passive soil forces) and the settlement of the tunnel and the other set in terms of the active forces and the soil reactions. These two sets of equations are then equated and the resulting simultaneous equations solved for the soil reactions. Finally, the shear, moment and thrust in the liner are found by the summation of those due to the active forces and those due to the soil reactions.

Bull presents the results of applications of his theory to actual tunnel designs and concludes that the liner stresses are lower than those previously presumed and that the liner thickness can be substantially reduced.

A method for statical computations of tunnel lining behavior based on approximating the structure by a rod polygon is discussed by Wissmann (1968). The medium behavior is assumed to be given by Winkler's subgrade reaction number which is simplified by introducing single springs or rods at the nodes of the framework. These springs are usually considered to act only in compression and are distributed sufficiently closely to each other to give an acceptable approximation of the soil-structure interaction.

Because of the large numbers of springs required, hand solution of the many simultaneous equations in Wissmann's solution is not possible and a computer is used. The coefficient of subgrade reaction is assumed linear although it is recognized that it actually depends on several factors. These factors include the type of soil, the properties of the soil, the thickness of the soil layers, the size and shape of the contact area, and the magnitude of the load.

Since a computer is used for the solution it is possible to include a more accurate approximation of the subgrade reaction coefficient. This may be accomplished by replacing the non-linear stress-strain curve with a train of polygons rather than a single mean slope. Wissmann does not believe this refinement is necessary.

Wissmann usually assumes that the crown pressure and side pressures are at-rest pressures. To these are added active invert pressures, passive invert pressures created by the vertical settlements, and water pressures. Other loads, such as traffic loads on the soil surface or in the tunnel and dynamic loads caused by explosions, may also be considered. It is claimed that the final results from this method are no worse than others in the literature and that the utility of the rod system used is virtually limitless while most other methods are for specially shaped tunnel profiles.

The Winkler theory assumes that the subgrade reaction coefficient is independent of the magnitude of pressure under the footing and has the same value for every point of the surface of contact between the structure and the soil. As discussed by Terzaghi (1955) the subgrade reaction coefficient actually varies with the size, shape and depth of the surface of contact. Thus the field or laboratory tests used to evaluate the subgrade reaction coefficient must be planned and evaluated with care if Wissmann's results are to be reasonable. It is believed that these inherent drawbacks in the Winkler theory, and the disregard of the effect of construction methods, make the apparent accuracy of Wissmann's approach rather illusory.

Use of Computer Analysis

The use of computers has been mentioned in Section 3.2 and in previous paragraphs of this section. This tool has allowed engineers to investigate many problems that previously required too much computation time. It has similarly permitted a more thorough investigation of the effects of variations in the values assumed for the major parameters of a problem.

As a result of the great growth in the application of computers to the solution of engineering problems, nearly every computer center has on file, or has access to, ready-made programs that can be used by a tunnel designer. Wissmann's approximation of a liner by a polygon of connected rods, for example, can readily be solved by existing programs such as structural engineering program STRESS developed at the Massachusetts Institute of Technology. The designer must program the coordinates of the node points, the stress-strain properties of the rods, and the properties of the springs used to approximate the surrounding medium and the loading.

Computers are necessary for finite element solutions, such as Reyes' (1966). For some of these solutions existing programs can be used with little modification. If necessary, new programs can be written. In nearly all such applications of the computer, the major problem is not one of adapting or writing a program, but of determining the properties or range of properties to be assumed for the materials.

Discussion

As a rule, considering liners as structural units subjected to arbitrary external loads leads to solutions that can be handled readily, in some instances by means of a computer. Like the continuum approach, some of these solutions can provide information on the effect of variations in assumed medium properties and in lining configuration. By studying a number of such variations it is possible for an engineer to improve his appreciation of the importance of the variables and thus improve his judgment.

Like the continuum approach, the hypothetical constant load approach has shortcomings. The uninitiated may tend to place too much confidence in the numbers generated by the solution rather than study them as guides.

If the hypothetical constant load approach is to be used as a design tool, the engineer must, in nearly all cases, assume either the earth pressure coefficient or the subgrade modulus. Because no universally acceptable method of determining either of these factors exists for the wide range of tunnel media encountered, it is possible to arrive at several "correct" values for the factors.

Both the hypothetical constant loads approach and the subgrade modulus techniques were developed primarily for soft ground tunneling.

It is doubtful that subgrade modulus techniques have application to rock tunnels, but the rock load concepts discussed in Section 3.5 have many traits in common with the hypothetical constant loads approach.

The hypothetical constant load and the subgrade modulus theories generally recommend that the vertical load on the lining be taken as the full weight of the overburden. However, the effect of redistribution of stresses above and around the tunnel, can be introduced into these methods. The following section discusses some of the basic concepts of arching or stress redistribtuion that have been applied to tunnel lining design.

3.4 Arching Concepts

The concept of arching as applied to tunnels assumes that some of the material directly above the tunnel is supported by the lining. The lining deflects downwards as a result of this load. A shearing force is developed between the stationary material at the sides of the tunnel and the material above the tunnel which tends to move with the lining. These shearing forces transfer part of the load from above the tunnel to the stationary material at the sides. This transfer of load from a yielding mass to adjoining stationary material is called arching.

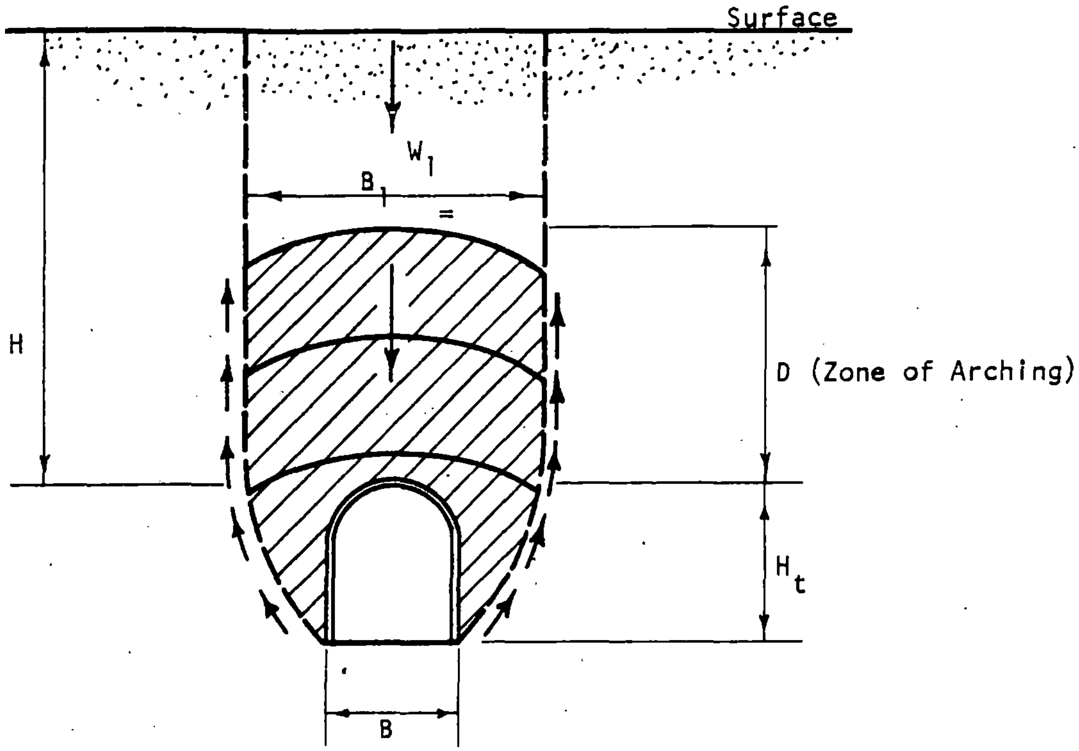
Terzaghi (1943) has a detailed discussion of arching above a trap door covered with a granular material. He reports that the concept was considered by several previous investigators. According to Terzaghi's theory, the arching or stress redistribution is approximated by the transfer of load by shear across imaginary vertical planes drawn from the sides of the trap door to the surface of the material.

In 1946, Terzaghi extended the arching concept to the determination of loads on tunnel liners in crushed rock and sand. He approximated the ground arch illustrated by Fig. 3.7 (a) with the simplified one shown in Fig. 3.7 (b). A major portion of the overburden load is transferred by friction onto the material at the sides of the tunnel. The balance of the material, represented by an equivalent height H_p, is carried by the roof support. Trap door experiments in dry and flooded sand led to the recommendations for H_p given in Appendix II (Terzaghi, 1946).

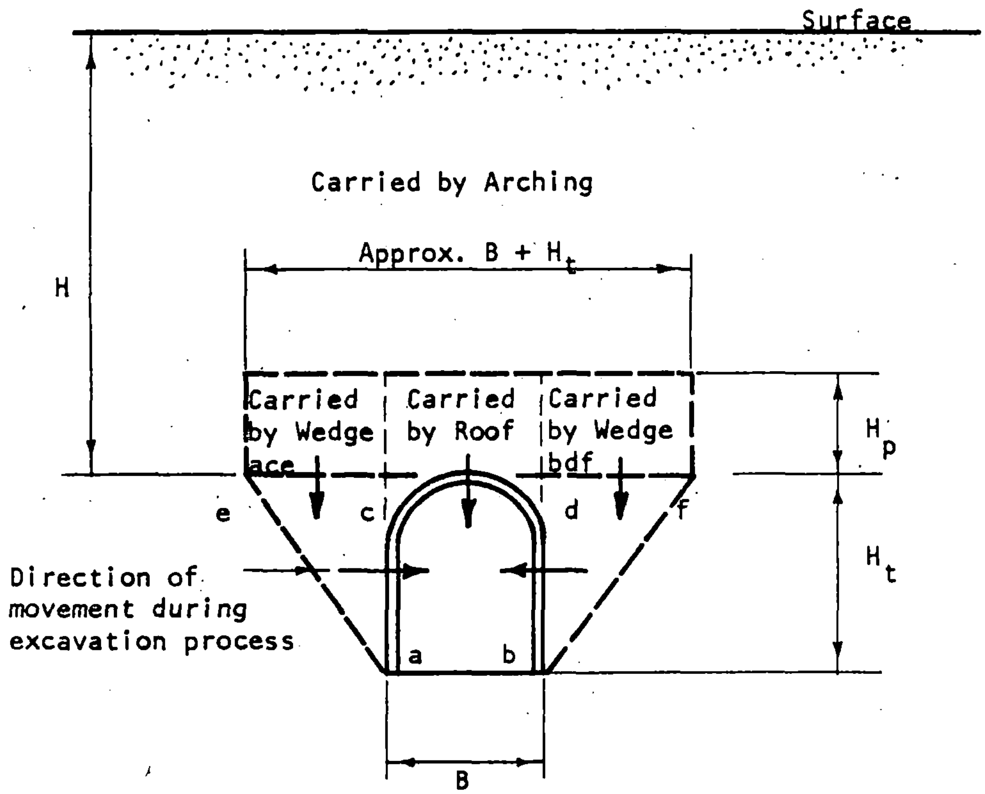
Arching has been studied by many investigators. The theories vary in the choice of simplifying assumptions. Except for the theories that are based on the theory of elasticity, all the theories introduce assumptions concerning both the shape of sliding bodies above the trap door and the horizontal and vertical stress distributions. None of the theories is truly rigorous. However, the results of even a rigorous plastic theory would be of doubtful value in tunneling because movements rarely are large enough to completely activate the plastic zones.

3.5 Rock Load Concepts

The loading concepts discussed in the previous sections are usually applied to tunnels in soil. This section deals with the concepts usually applied in rock. Some massive, unjointed, and undisturbed rock masses might behave as continua while the behavior of others might be approximated using concepts of deformation-dependent loading. Field observations, however, indicate that rock does not behave in strict accord with any of the previously discussed theories. Therefore several investigators have developed



a. Ground Arch



b. Assumed Support Loading

Fig. 3.7. Roof Load in Crushed Rock and Sand (After Terzaghi, 1946)

rock load concepts that are based almost completely on field observations of rock behavior modified or supported by only enough simplified theory to permit the engineer to develop design values.

Terzaghi's Rock Load Theory

Terzaghi (1946) defines rock load as the height of the mass of rock which tends to drop out of the roof of a tunnel. His values of rock loads are represented by ranges since there are no well defined boundaries between the factors that determine the rock load. He relates certain rock classes to certain recommended design load ranges on the lining structure. His descriptions of the rock classes are qualitiative and are given in Appendix II together with the recommended loadings.

Appendix II in addition summarizes similar loading recommendations developed by Bierbaumer (1913) and Stini (1950). These recommendations are discussed to some extent by Szechy (1966). Szechy also discusses rock load theories by Kommerell, Ritter, Bierbaumer, and Protodyakonov. These theories all assume that the supports are required to carry the weight of rock in a natural arch above the tunnel. The arch is assumed to be bounded by either a parabola or a half ellipse. These theories require more mathematical manipulation than Terzaghi's theory and their results would appear to be no more reliable.

In addition to the rock load, Terzaghi recognized that the bridge action period (stand-up time), defined as the time the roof will remain essentially stable and undeformed after exposure, was of great interest to the tunnel engineer. The construction technique must be so selected that support is installed before this period expires. During this period there is a progressive loosening or disintegration of the structure of the rock around the opening. If allowed to continue, the rock will loosen and fall out until the cavity is filled or a stable ground arch is formed. If support is installed prior to the end of the bridge action period the net load on the supports may be less than the ultimate rock load since the complete formation of the ground arch will be inhibited by the supports. Appendix II describes attempts by Terzaghi (1948) and Lauffer (1958) to assign numbers to the bridge action period for various types of rock.

Loosening Pressure, Genuine Rock Pressure, and Swelling Pressure

Rabcewicz (1944) and Stini (1950) discuss rock pressures in three main categories: loosening pressure, genuine rock pressure, and swelling pressure. The first of these is essentially the same as Terzaghi's rock load as previously discussed.

Genuine rock pressure is associated with the creation of plastic regions around the opening (see p. 24). The excavation of a tunnel causes an increase of the tangential stresses in the material close to the wall of the tunnel. At the same time the radial stress is removed, and the rock is no longer confined. Thus, the rock at the wall surface is stressed in an unfavorable manner, and it will fail or yield when the concentrated tangential stress exceeds the unconfined compression strength. Depending on the character of the rock, this failure or yield manifests itself in a variety of manners. If the rock is ductile (salt under certain conditions, clays, some clay-shales) squeezing is initiated and the material encroaches upon the opening at a rate determined by the rock properties and the degree of overstressing. At the same time, stresses are redistributed, and the plastic zone increases in size. In some cases a stable condition is eventually reached. If the rock is highly overstressed, the squeeze may continue until the opening has vanished. By applying an internal support, the plastic zone is reduced, and stability can be achieved. The load on this internal support, the so-called genuine rock pressure, is dependent upon the flexibility of the support, and the stage of development of the plastic zone when the support was constructed.

If the material is brittle, the rock will slab or pop at the location of the highest stress concentration if the compressive strength is exceeded. This may occur in hard rocks at great depths. Again, the development of the plastic or failed zone will continue until a stable plastic zone is created. Figure 3.3 shows the strong influence of the internal pressure p_i on the extent of the plastic zone. This figure indicates that a weak support often will keep the material from failing, an observation supported by experience in deep mines. The magnitude of the necessary internal pressure p_i, the genuine rock pressure, is highly influenced by the time the support is installed. A delay in support installation may permit the rock to move sufficiently to reduce the friction angle φ to its lower, residual value. As shown in Fig. 3.3, the internal pressure p_i must be increased to maintain a constant plastic zone radius as φ decreases.

The considerations and the formulae given in Section 3.3 and Appendix I may predict the occurrence of plastic behavior and genuine rock pressure. The magnitudes of loads and deformations, on the other hand, cannot at present be predicted with a satisfactory accuracy. The types of plastic behavior, the rate of deformation, and the influence of internal support on the genuine rock pressure will be treated further in the second part of this report.

The third type of rock pressure under this theory, swelling pressure, occurs in some rocks as well as in soil. It is associated with an increase in water content of the material. Swelling may be associated with squeezing, or, in contrast to squeezing, swelling may occur without the development of a plastic zone. The pressure exerted on an internal support by swelling ground depends on when the support is installed and how much additional rock movement the support allows.

Method of Relative Yield

Lane (1957) considered blocks bounded by vertical planes through the tunnel springlines in an attempt to study the arching phenomenon over the tunnel. When he compared his results with measurements taken at Garrison Dam, Lane was not satisfied with the agreement. A more desirable agreement was found with the method of relative yield.

In the method of relative yield the arching concept is modified to include the relative stiffnesses of the rock and the tunnel. Different deformation moduli are applied to the rock, a flexible lining, or a stiff lining. The lower boundary at the invert is assumed to settle uniformly and significant differential deflections are assumed to occur over the height of the elastic blocks.

Rear Abutment Load

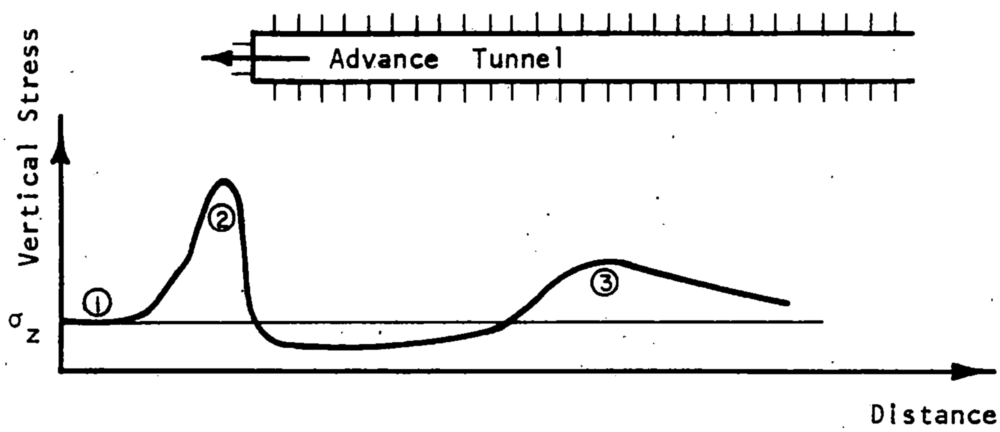
The results of measurements taken in the Straight Creek pilot tunnel have been discussed in terms of the rear abutment load concept (Terrametrics, 1965). This concept apparently originated among mining engineers to explain phenomena experienced in long wall mining.

At the face of the tunnel the rock load is pictured as being supported by the two side walls and a longitudinal cantilever beam that is fixed by the material behind the face. With the advance of the tunnel, a given point on the cantilever deflects as the effective length of the cantilever (from the point to the face) increases. Supports installed a short distance behind the face are loaded as they begin to pick up the load from this deflecting cantilever. Eventually, the supports pick up sufficient load to bring the system to equilibrium. This equilibrium load is the rear abutment load. Figure 3.8 illustrates the rear abutment load coneept. The use of this concept for design purposes is based primarily on experience.

Discussion

Rock load concepts are based on actual field observations and result in ranges of values for the load carried by the supports. Using experience and the available exploratory data, the engineer can select the best design load for the given site. These concepts have been used successfully to design rock support systems for many tunnels.

One objection to the rock load concept is its failure to predict accurately the behavior of the materials in the field from a knowledge of the condition of the rock. Most supports are installed early enough that the rock in the roof does not loosen all the way back to a stable arch. The supports are thus not called upont to support the complete weight of rock that would fall out if unsupported. The rock load, therefore, depends on the time supports are installed and the amount of additional loosening that takes place after installation of the supports. This again depends on the type and quality of the support.



Notes

1. σ_z = Background Vertical Stress
2. Stress concentration ahead of destressed tunnel face
3. Rear Abutment Load

Fig. 3.8. Illustration of Stress Concentration Ahead of Tunnel Face and Rear Abutment Load

The addition of genuine rock and swelling pressures to the rock load (loosening pressure) results in a more general concept. Unfortunately the practice in the United States has been to attempt to predict only the rock load. In Europe some attempts have been made to include genuine rock and swelling pressures.

Lane's relative yield theory and the rear abutment load concept both were developed for a particular set of conditions. While they appear to be reasonable for their respective conditions, they may not be sufficiently general to be applied elsewhere.

3.6 Stability of Tunnel Liners

Buckling has not usually been considered a problem for linings for near-surface tunnels. A simple, partly empirical equation for the buckling of a thin-walled cylinder of radius a with a modulus of inertia of the wall I per unit length is

$$p_{cr} = 2 \frac{k_s EI}{a^3} ,$$

where k_s (lbs/sq in.) is the modulus of reaction, E is the modulus of elasticity of the lining material, and p_{cr} is the uniform all-around pressure creating instability. Calculations with this equation indicate that buckling is not a probable mode of failure for reinforced concrete cylinders buried in soil (Hendron et al., 1968). Therefore, buckling of reinforced concrete cylinders will not be considered further in this study.

Additional calculations with the equation given above and field experience indicate that for thin-walled liners, buckling rather than yielding by ring compression may be the most critical. The liners most commonly used are believed to be too thick for this to be true. Innovations in the field of lining construction, however, may lead to thinner linings for which buckling is critical.

3.7 Analyses for Multiple Tunnels

Multiple tunnels have usually been studied by superimposing elastic or elasto-plastic theoretical solutions or by performing photoelastic experiments with multiple holes. Superposition of solutions is exact for linearly elastic materials and can be applied to cases where the medium can be assumed to behave elastically. For any other case, the solutions should be considered for their value as tools in the development of judgment. Photoelasticity provides visual study of the same phenomenon.

Indescriminate use of superposition of elasto-plastic solutions can lead to considerable inaccuracy. The behavior of elasto-plastic materials is path-dependent. Thus, superposition of elasto-plastic solutions can be recommended as a first approximation only. Such superposition of solutions should not be used for design.

Field evidence from Garrison Dam (Lane, 1957) indicates that superposition does not work in the clay-shale at that site. Lane found that the middle tunnel (of three) experienced different stresses when it was driven between two existing tunnels than it did when driven first followed by the other two tunnels. These tunnels were spaced approximately two diameters center to center.

Coates and McRorie (1962) discuss field measurements taken on adjacent tunnels in brecciated rock 650 ft below the surface. For these tunnels, which were 10 ft in diameter and were spaced 20 ft (two diameters) center to center, they found that the vertical load arose from the weight of loose material that accumulated on top of the sets. (In a discussion to the paper Osler noted that Terzaghi's rock load concepts could be used to predict the measured loads on the sets.) The authors observe that the spacing between these tunnels is apparently sufficiently great that the set loads were not affected by the adjacent tunnels. This observation contradicts Lane's results, and indicates the great influence of the geologic conditions and construction techniques on the interaction between neighboring tunnels.

Studies by Riley (1964) and Agarwal and Boshkov (1967) are typical of the three-dimensional photoelasticity investigations of intersecting or adjacent tunnels. Riley's experiments were conducted to determine the stress distributions associated with tee, cross, and right angle tunnel intersections. The intersections were located in a uniform, uniaxial stress field, and the stress concentrations were studied using stress freezing and slicing techniques. He found that tensile stresses near the intersection were practically unchanged from those in the tunnel away from the intersection. Compressive stresses were approximately 60% higher near the intersection. The high stress region was localized and decayed within a distance of one diameter of the intersection.

Agarwal and Boshkov used similar techniques to study horizontal tunnels separated by a vertical distance K and with an angle α between the projections of the centerlines. Stress concentration factors were determined for selected points on the tunnel periphery and up to six radii away from the tunnel for selected values of K and α. They found that the stress concentration factors, compared to those for a single tunnel, were significantly altered within a distance of approximately two radii from the point at which the tunnels diverge from each other. The maximum stress concentration factors were shown to be as high as 250 percent of those for a single tunnel.

3.8 Design Methods in Common Use

The theories available for tunnel liner analysis and design have been discussed in general terms in previous sections of this chapter. The application of these theories to the actual design of tunnel linings has not been demonstrated. The following paragraphs will discuss the background for, and give examples of, design methods in general use. It will become obvious that the design methods actually used often bear little resemblance to theory and that the influence of theory on the design methods is indirect.

Reasons for Simplified Design Methods

Design methods for tunnel liners should be formulated so they can be used by engineers who have an awareness of the potential problems arising from the non-ideal behavior of soil and rock but who do not fully understand this behavior. Typically such an engineer is a structural engineer. He is capable of designing a tunnel lining when told, for example, that the vertical load is equal to the overburden pressure and the horizontal load is some portion of the vertical. He may have little capability or motivation for determining the actual conditions. For a much more complicated theory of soil behavior he would have even less interest and knowledge.

The design method chosen should be consistent with the accuracy with which the properties of the medium are known. With computer solutions it is easy to fall into the habit of expressing the answers to several significant digits. Material properties cannot be determined accurately by explorations, field test, or lab tests because of the variability of geologic media. Complicating refinements to any theory can hardly be justified if they would lead the designer to believing that his results are more accurate than the material properties.

A second and related factor also influences the complexity of the design method. Even if the material properties could be precisely determined, it is usually necessary to make some simplifying assumptions when the properties are used. For example a curved load-deflection curve from a load test is approximated by a straight line to develop a subgrade reaction number. Or, even more critically, the load test is conducted with a small bearing plate and the resulting subgrade modulus is assumed to apply to the behavior of the much larger volume of loaded soil around the tunnel. Such assumptions automatically introduce errors.

Finally, a design method should include a minimum number of variables. By a reduction in the number of variables, the design method becomes applicable to a larger range of conditions. Thus, a simplified design method may be desirable where a long tunnel system must pass through a variety of conditions.

Examples of Current Practice

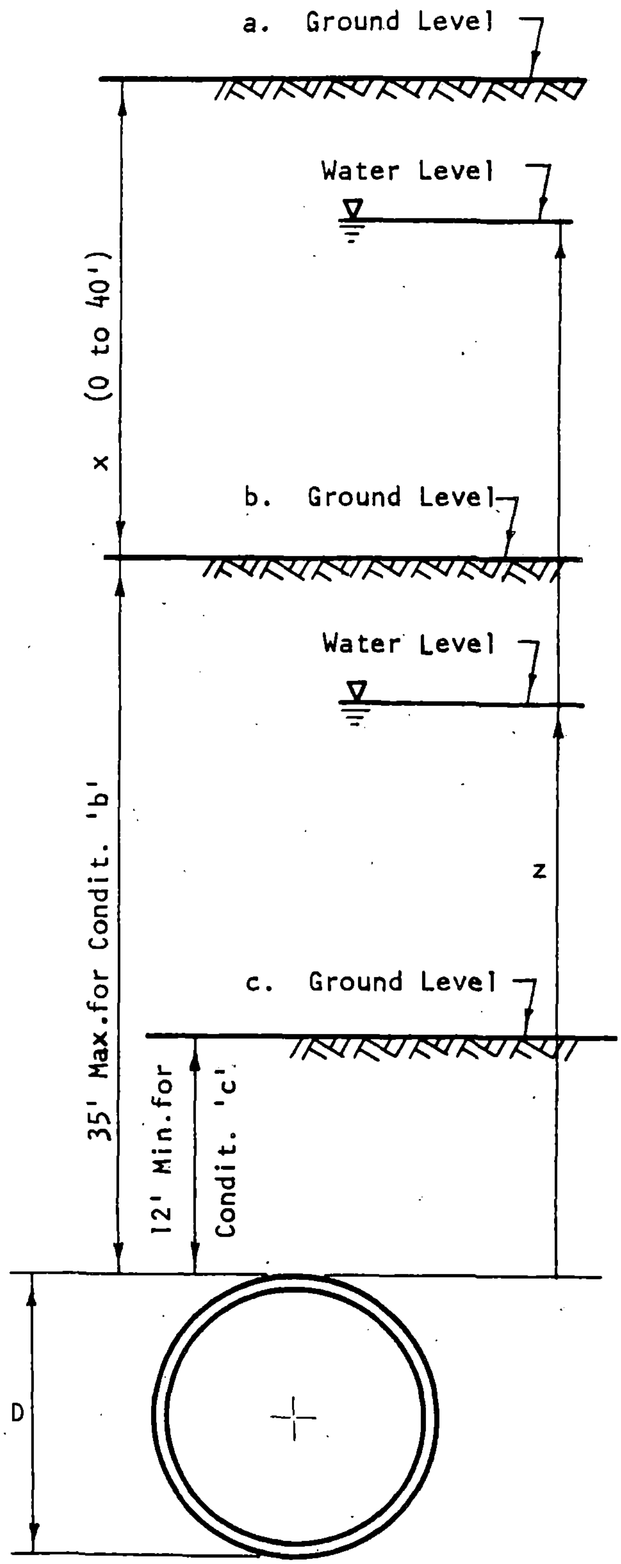
The preceeding section mentioned a few of the reasons for making design methods as simple as possible. In this section, recent tunnel projects will be reviewed to gain a better appreciation of the types of design methods actually used. Tunnel designers do not often publish the details of their assumptions and simplifications so these examples can be obtained only by personal contacts.

The San Francisco Bay Area Rapid Transit (BART) tunnels are predominantly in water-bearing granular materials but soft clay is encountered along certain lengths of the tunnels. The consultants for the BART system believed that the tunnel lining would offer little resistance to the unbalanced ground forces on the horizontal and vertical planes. They believed that the ground and the lining would distort together until largely uniform forces acted on the circumference of the lining. Their design recommendations, therefore, included a combination of radial loads related to the overburden and stresses in the liner caused by its distortion.

The recommended design loads for single tunnels are shown in Figs. 3.9 and 3.10. Arching was considered to begin being important in granular materials at a depth of burial to the crown of about 26 ft (1.5 times the diameter) but it does not appear explicitly in the equations until the depth of burial is 35 ft. Beyond 35 ft of cover, only one-half of the overburden was considered to load the tunnel. No arching was considered in soft clay. Figures 3.11 and 3.12 show the recommendation for including the effects of interaction between adjacent tunnels.

The consultants also presented a set of guidelines for assessing the influence of future construction of small structures over the tunnels. In general, structures that would neither physically interfere with the tunnels nor increase the loading on the tunnels beyond maximum design loads are permitted.

Using the equation and guidelines given above three basic liner classes were developed for the system. For a given portion of the system, the designer must check the loads on that section and then select the appropriate liner class.



a. For depth to crown from 35 ft to 75 ft (over 35' arching is considered). and ground water z above crown

$$P_r = (35 + \frac{x}{2})\,\gamma_t + (z + D)\,\gamma_w$$

plus

Bending forces resulting from axial diameter changes of + 5/8''

b. For depth to crown from 12 ft to 35 ft and ground water at z above crown.

$$P_r = 35\,\gamma_t + (z + D)\,\gamma_w$$

plus

Bending forces resulting from axial diameter changes of 1/2''

c. For minimum depth to crown of 12 ft

$$P_r = (12 + D/2)\,\gamma_t + (z + D)\,\gamma_w$$

plus

Bending forces as in b

Notes:

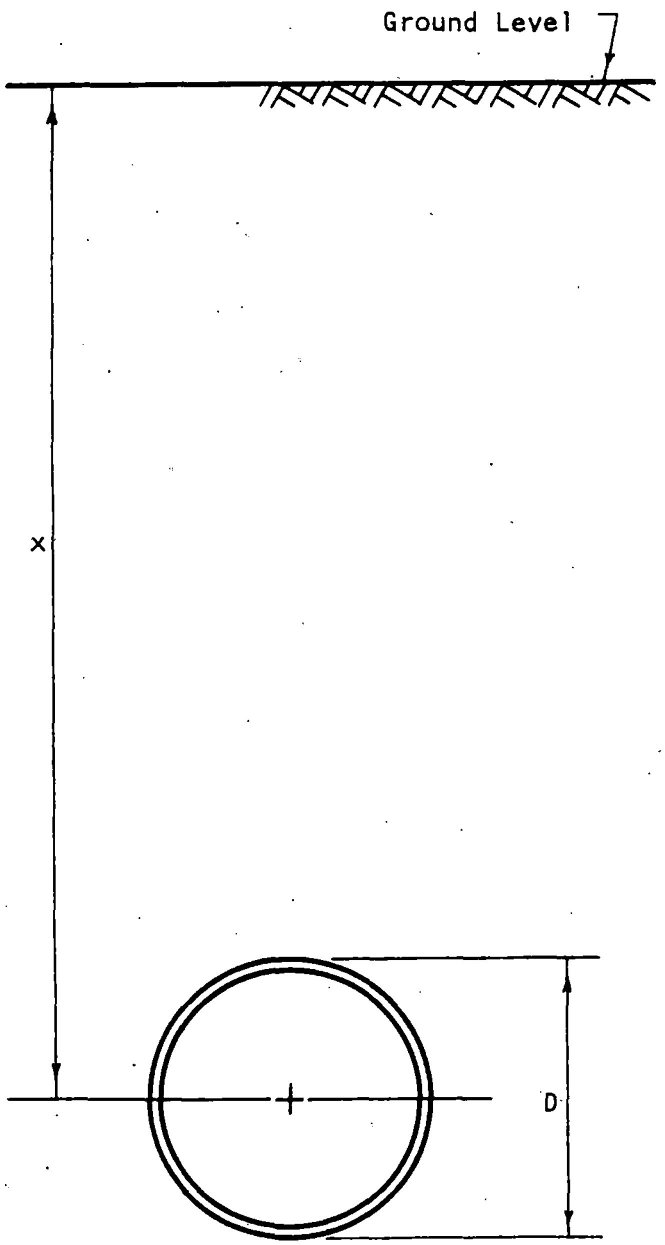
γ_t = Unit weight of soil adjusted for bouyancy

γ_w = Unit weight of water

P_r = Radial load, PSF, applied uniformly around circumference

All dimensions are in feet

Fig. 3.9. BART Ground Pressures for Tunnel in Sand or Predominantly Granular Ground

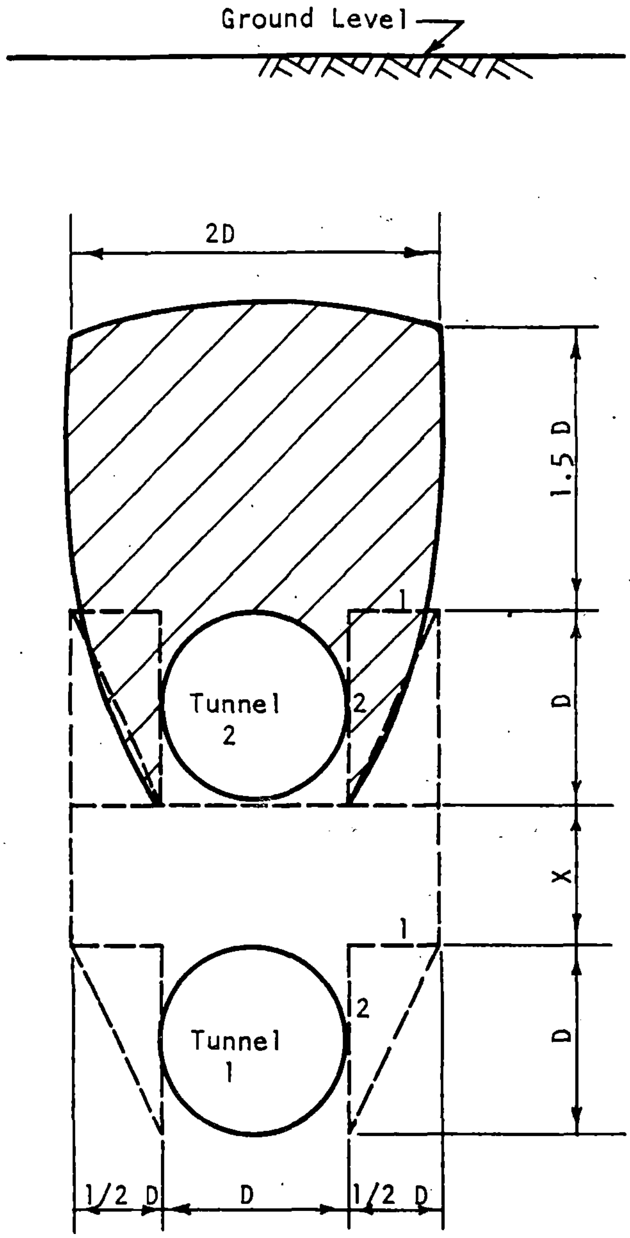


$P_r = x_1 \gamma_v$ = Radial load
(uniform around circumference).

Vertical axis lengthens
= Horizontal axis shortens
= 7/8"

γ_v = Unit Weight of Soil

Fig. 3.10. BART Ground Pressures for Tunnel in Soft Plastic Clay



Tunnel 1:

The pressure on the lower tunnel may be approximated by

$$P_e = 1.5\ \gamma_t\ D + \gamma_w\ X$$

with a practical limit of:

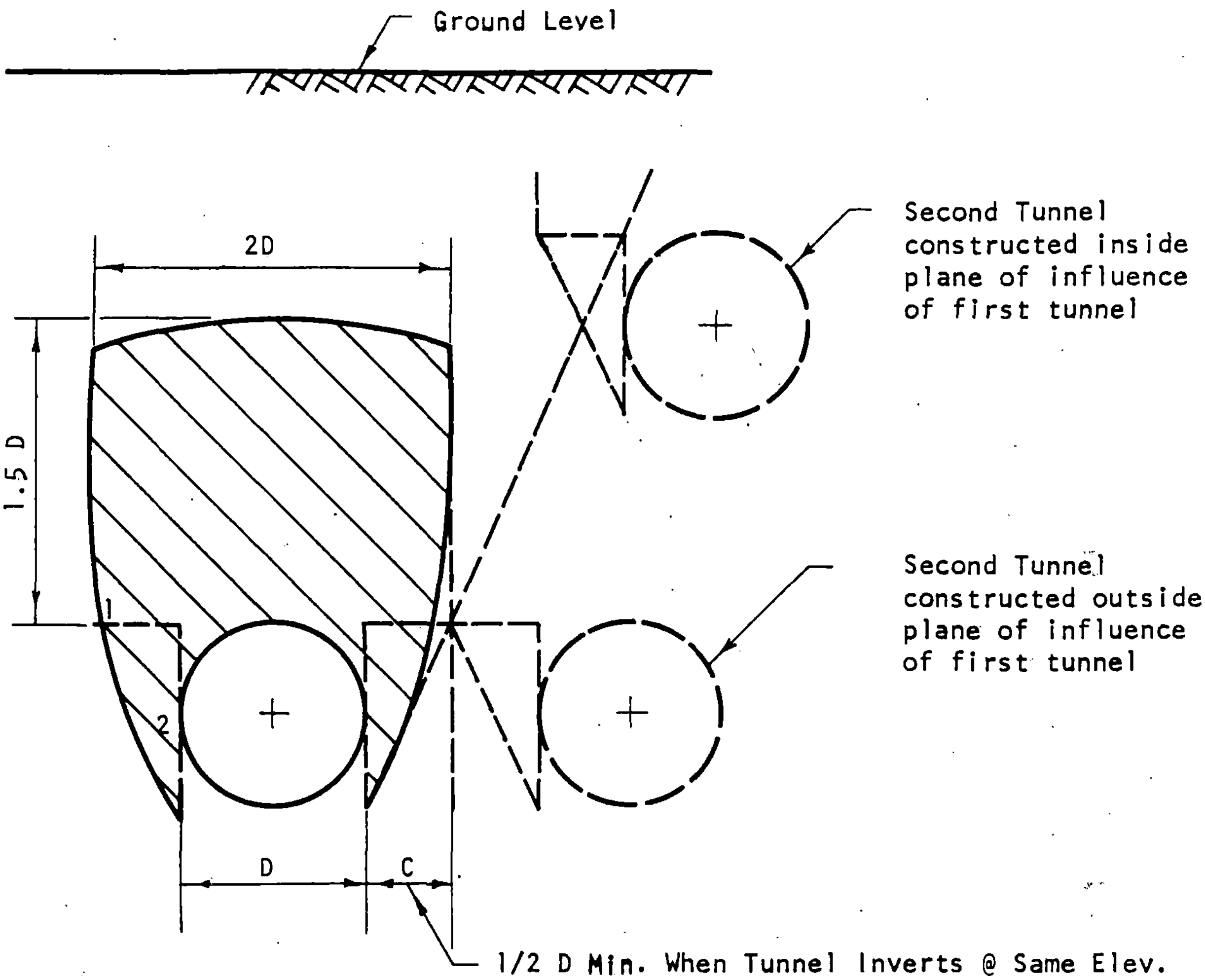
$$3.0\ \gamma_t\ D$$

The loading of the soil of the level occupied by the upper tunnel may be considered off-set by the bouyancy of the upper tunnel.

Tunnel 2:

Loading same as condition (b) or (c) of Fig. 3.9. Usually no arching effect available because soil has been disturbed by lower level tunneling.

Fig. 3.11. Effects of Vertically Adjacent Tunnel (BART)



Case (a) Tunnels Horizontally Adjacent

1) If clearance between tunnels "C" > D, additional $P_r = 0$
2) If C is 1/2 D, additional pressure should be $.5\gamma_w D$
3) Tunnels should not be driven closer than C = 1/2 D

Case (b) Tunnels Offset Both Horizontally and Vertically

1) When horizontal clearance between tunnels is equal to or greater than one-half of the vertical distance between the inverts of the tunnel plus one tunnel diameter, the additional pressure on the lower tunnel is zero ($P_r = 0$) (Plane of Influence assumed at 2v on 1 h)
2) When the horizontal clearance between tunnels is less than in (b)1 above, the additional pressure P_r should be evaluated between Fig. 3.11 and Case (a)(2) above.

Fig. 3.12. Effects of Adjacent Tunnels (BART)

The design manual for the Washington subway recommends a ratio of horizontal pressure to vertical overburden pressure of 0.875 for the design of reinforced concrete rigid earth tunnel sections. With this ratio the liner sections are to be designed as rigid structures using the working stress design method. Using ratios of 0.45 and 1.00, the sections are then to be checked using ultimate strength design methods and a maximum concrete stress of 85 percent of the 28-day strength.

Wherever possible, the following guide was used in establishing the minimum rock cover for the Washington subway:

Maximum Excavated Width	Minimum Sound Rock Cover
20 ft	10 ft
35 ft	15 ft
70 ft	30 ft

With rock cover of these amounts, the rock was believed capable of supporting itself and the overburden with no more than rock bolt reinforcement.

For any condition in either soil or rock not covered by the above paragraphs the design manual instructs the designer to follow the recommendations of the Soils Consultant.

Szechy (1966) gives a summary of the design specifications for the Lisbon Subway. The vertical and horizontal loads on Lisbon Subway tunnels driven in sand and silt were estimated according to Terzaghi's recommendations (see Appendix II). Loads for tunnels in rock were also estimated as recommended by Terzaghi. In clays the vertical pressure was taken as the overburden pressure minus an amount depending on the effective shear strength of the material. Horizontal pressures in clay were estimated as 0.5 to 0.7 times the vertical pressure.

Vertical design loads recommended for the Budapest Subway were also based on Terzaghi's theory. Details and examples of the application of Terzaghi's arching theories to this subway project are found in Szechy (1966).

Soviet standard specifications for the design of underground and motorway tunnels are quite detailed. Vertical rock pressures may be assumed from a table or from a rock load concept based on Protodyakonov's theory. These specifications are also summarized in Szechy.

The recent design specifications discussed in this section illustrate an important point: the process of developing design methods are repeated for every new project, but the results seldom represent innovations or generalizations. The original design of the Chicago subway tunnels serves as an example. This design followed the existing practice for sewers in that city (Terzaghi, 1942). The vertical pressure on the top of the tunnel was taken equal to the overburden and the vertical pressure on the base was taken as the sum of the pressure on the top and the weight of the arch. Side pressures were assumed equal to the pressure of a fluid with a density of 1/3 to 2/3 of the actual weight of the soil. These concepts are very similar to those used four decades later in the Washington and the Lisbon subways, although the factors are different.

Contrast with Refined Analyses

Preceeding sections have discussed in varying detail a number of the theories and methods that have been developed for tunnel design and have reviewed a few examples of current practices. Even a cursory comparison between the theories and the practices leads to the conclusion that there is only slight resemblance. Such a comparison does not tell the whole story.

The BART design loading consists of a modified overburden load applied radially plus bending stresses from specified diameter changes. On deeper analysis, the following observations can be made:

1. The overburden and water pressure loads applied radially represent a simplification of the hypothetical loads on liner concept.
2. Arching is included for depths of burial to the crown greater than 26 ft.
3. The inclusion of bending stresses due to diameter changes indicates the recognition that the liner will deform to develop its loading and that the final stresses in the liner are a function of those deformations.

The Washington Subway Manual of Design Criteria requires a very simple loading for rigid linings in soil: a vertical loading equal to the overburden and a horizontal loading equal to a specified fraction of the vertical one. Sound rock is considered self supporting under specified conditions. The recommended loading for rigid linings in soil is an arbitrary external load applied to a liner but the deliberations and experience that led to its adoption are not apparent. Neither is supporting evidence given for the recommendation that sound rock is self supporting under some conditions. In these design criteria, the designer is given a set of rules to follow but he is given nothing to improve his understanding of the behavior of the system composed of tunnel and surrounding medium.

3.9 Rock Reinforcement Methods

Any support system which ties or holds the rock mass together and helps the rock to support itself will be labeled rock improvement. Occasionally rock improvement may be applied before excavation but it is usually applied shortly after excavation. The following subsections discuss the three principal classes of rock improvement: rock bolts, grouting, and shotcreting.

Rock Bolts

All of the tunnel lining design methods discussed to this point tacitly assume that the lining is essentially continuous around the periphery. For example, it may consist of steel sets or reinforced concrete. During the last 20 years, however, rock bolts have been used with increasing frequency. Rock bolts are discontinuous, i.e., they are installed only at discrete points, though a regular pattern may be used. They are most frequently used in tunnels in relatively good rock, but may be successful in some tunnels in rock that would be classified as poor. The design of rock bolted tunnels is quite unlike any of the lining design techniques previously discussed.

Rock bolts make their greatest contribution when they are installed very close behind the face. They should take maximum advantage of the natural strength of the rock and thus help the rock support itself. In some instances in-situ stresses are high enough to cause the rock to flow plastically. Initially rock bolts may not be able to resist this plastic flow. As the plastic zone increases in size, the interior normal stress required for stability decreases. At some point, rock bolts can apply this stress and stabilize the opening. No other movement or relaxation of the rock which leads to sliding, rotating, or lowering of the normal stress on potential sliding surfaces should be allowed.

The selection of rock bolts for a given application has not yet reached the stage where it is considered proper to speak of designing rock bolt systems. More appropriately, rock bolt systems can be said to be selected. The following paragraphs will outline the most common methods of selecting rock bolt systems. It is believed that these methods will continue to change and become more reliable as the use of rock bolts increases. In the subsections to follow, six of the most common rock bolt selection methods will be grouped into three groups: experience, rock support, and rock reinforcement. For any given project, it is necessary to consider all of the possible modes of behavior or mechanism that may occur. Then, one or more of the methods is applied and the most critical case is used as the basis for selecting the rock bolts. The anchorage selection is perhaps less well defined than the selection of the rock bolt system itself. Some designers rely solely on experience. Others select the anchorage using broad guidelines similar to those proposed by Rabcewicz (1957). Such guidelines indicate a cement or epoxy grout for rock bolt anchorage in very hard or in weak rock or shale, and expansion anchors in hard or medium hard rock.

Selection Based on Experience. -- Experience provides rules or guidelines that will often result in a reasonable rock bolt system. Designs based on any of the other hypotheses should be compared with these guidelines which may be stated simply:

$L = (1/3 \text{ to } 1/2)\ B$ usually 6 ft to 8 ft

$S = 5$ ft to 8 ft

where L is the bolt length

S is the bolt spacing

and B is the tunnel width

The pattern for the rock bolts should be regular and should be established by the designer. Patterns are specified for complete sections of the tunnel. Miners may not understand the concepts of rock bolt behavior and the project nearly always will progress more smoothly if the miners are not expected to make frequent changes in the rock bolt pattern to meet localized conditions within a section of the tunnel. Thus the pattern is designed for the most critical conditions expected within that tunnel section.

Selection Based on Rock Support. -- The selection of a rock bolt design may be based on concepts of rock support. For such concepts the opening is considered basically stable but it is possible that a bed or joint block may fall out of the roof of the tunnel, or the sidewall may spall locally. The size, type, and pattern of bolting is selected by engineering judgment based on the best geologic information available and on experience under similar conditions.

Rock bolts to be placed in a horizontally stratified rock are selected and spaced so that their combined design strength is equal to the dead weight of the section of the strata that would tend to fall.

They are made long enough to ensure sufficient anchorage in deeper, more confined, and stronger strata. To hold joint blocks in suspension the selection is similar except that more experience and judgment are needed to select the most critical size, location, and orientation of the potentially unstable joint block. Often a trial and error approach is used in which several possible joint block configurations are investigated and the most critical one used for design. Spalling can be prevented by applying a normal stress at the surface. The magnitude of this stress and thus the size and spacing of the bolts required to furnish it are often unknown so that a trial and error installation is often used.

Selection Based on Rock Reinforcement. -- The purpose of the rock bolt in another group of rock reinforcement hypotheses is to confine the rock so that it will become at least a part of the total structure supporting the opening. Included in this group are the concept of beam building and arch building and the Mohr Coulomb shear strength hypothesis.

The beam building concept assumes that the rock bolts can be designed to form a laminated structural beam of adjacent beds instead of merely suspending a falling bed as in the previous discussion. The rock bolts are assumed to increase the friction and thereby the horizontal shear between the beds, hence forcing them to act as a beam.

In a tunnel that is capable of supporting itself the stable condition of the rock in the roof often is observed to be an arch. This arch does not usually correspond to the shape of the original excavation and it is formed by a progressive failure in the rock. Reinforcement of the rock with bolts, if done early enough, will stop these failures and lead to the formation of a stable arch of the shape originally excavated. This is the arch building hypothesis of rock bolting.

The Mohr-Coulomb shear strength hypothesis is based on the application to rock of the familiar failure or yield condition:

$$\sigma_\theta = 2c \quad N_\varphi + \sigma_r N_\varphi = q_u + \sigma_r N_\varphi \quad ;$$

where $N_\varphi = \frac{1 - \sin\varphi}{1 + \sin\varphi}$, σ_θ is the rock stress parallel and σ_r is the rock stress normal to the tunnel wall, and q_u is the unconfined compressive strength. Near a tunnel wall σ_r is usually zero, but rock bolts can be used to apply a normal stress at the wall and to increase it to some non-zero value. If the unconfined compressive strength is comparatively low the additional strength from the term $\sigma_r N_\varphi$ can be very significant. The effect of the rock bolt in this theory is identical to the effect of an internal liner exerting a uniform force p_i on the tunnel walls. The analyses given in 3.3 and Appendix I are applicable in this case.

Shotcrete

Shotcrete is the newest and most promising type of tunnel reinforcement. Recent experience at several locations around the world has shown that shotcrete and its various adaptations can save large sums of money in tunnel construction.

The impedance to the widespread application of shotcrete for tunnel reinforcement at this time consists of two parts: 1) a skepticism among tunnel designers and builders over the ability of shotcrete to support an underground opening, and 2) a lack of qualitative criteria on which the design of shotcrete can be based. These two factors are in some cases interrelated. Like rock bolts, the application of shotcrete has not yet reached the stage where it is possible to speak of the "design" of shotcrete. The selection of a shotcrete tunnel lining is a crude and often completely undefined process. The remaining discussion is devoted to the various concepts that are presently used to explain the function of shotcrete, and to arrive at a selection of a shotcrete lining.

The contribution of shotcrete to the reinforcement of an opening is in principle similar to that of rock bolts. That is, the shotcrete, through its confining effect on the rock at the periphery of the opening, causes the rock to act as a self-supporting arch which is capable of sustaining at least part of the loads in the stress redistribution zone. As with rock bolts, the greatest benefit from shotcrete can be realized if the shotcrete is applied soon after excavation. Some geologic conditions, notably various swelling materials and some popping rock, have not been stabilized by immediate shotcrete application. However, nonswelling types of ground are most easily controlled by applying shotcrete within one or two hours after excavation. Rapid application provides rapid confinement of the rock and thereby hinders the development of loosening pressures. It also seals the rock against the deleterious effects of air and water.

The above mentioned functions of shotcrete can be considered "confining" functions. Because of the unknown mechanical interaction between the shotcrete and the tunnel wall, there is little chance that this component of the shotcrete's reinforcing effect can be analyzed rationally. Shotcrete, can, however, deliver another reinforcement component, that of a structural member.

Depending on the deformational behavior of the shotcreted tunnel and on the geometry of the shotcrete application, the shotcrete itself may function as an arch, a thin plate, or a closed circular ring. The interplay of the two functions of shotcrete is not well-defined, and analysis must necessarily be limited to crude approximations.

The approximations most commonly made for an analysis of the structural behavior of shotcrete involve two assumptions: an assumption of the mechanism of load development in the rock, and an assumption of the load carrying machanism in the shotcrete. Various simplified models have been used for each assumption.

The simplest shotcrete analyses have considered loosening pressures and have assumed shotcrete failure by diagonal shear. Rotter (1961) uses such an analysis to compute a safety factor against diagonal shear through a thin reinforced shotcrete plate that is loaded with the dead weight of a pyramidal rock block. The same analogy is often used in Sweden.

Other analytical models have been developed to fit various shotcrete-tunnel behavior mechanisms. In the Alps, for example, pressure phenomena associated with squeezing ground conditions sometimes cause a type of tunnel failure that is most adequately treated with continuum solutions. Rabcewicz (1964, 1965) uses equations such as those given in Appendix I for plastic behavior to determine the so-called "skin resistance" necessary to prevent loosening. It is then assumed that a shotcrete lining can provide the necessary skin resistance through the action of a yielding cylindrical liner. This approach has been used to justify the use of shotcrete for sections of the Washington, D. C., Subway. Its validity depends on two factors: 1) the ability of the shotcrete to yield sufficiently to develop the plastic zone at the tunnel wall, but at the same time maintain its structural integrity so that excessive loosening does not occur; 2) the closeness with which the analytical model fits the real rock behavior. Both of these factors have been more or less verified for conditions at several tunnel sites in the Alps, but the applicability of these factors to conditions at other sites is uncertain.

Local experience in different geologic environments has produced a number of rule-of-thumb guidelines for the selection of shotcrete reinforcement. Unlike the methods described in the previous paragraphs, the rules established from local experience have no theoretical basis. Classification systems of different varieties are used to tie local experience to the geologic environment.

Linder (1963), for example, has superimposed a set of shotcrete rules on Lauffer's classification system as shown in Fig. 3.13. The problem with such a relationship is that the rock classification is very subjective and leaves a wide margin for personal judgment.

Hansagi (1965, 1967) uses a classification based on joint spacing and unconfined compressive strength to relate the material to the reinforcement requirements in underground openings at the Kiruna iron mines in northern Sweden. This system, although quantitative in nature, has not been applied to a wide enough variety of rock conditions to be of universal value.

Although most shotcrete applications have been in rock tunnels, shotcrete has recently been used successfully in a Milano, Italy, subway tunnel in running sand and gravel (Chase, 1968). In this tunnel, the arch, the face, and muckpile must be shotcreted to hold the ground. Steel reinforcing bar spiling is driven ahead of the face and the ground is then

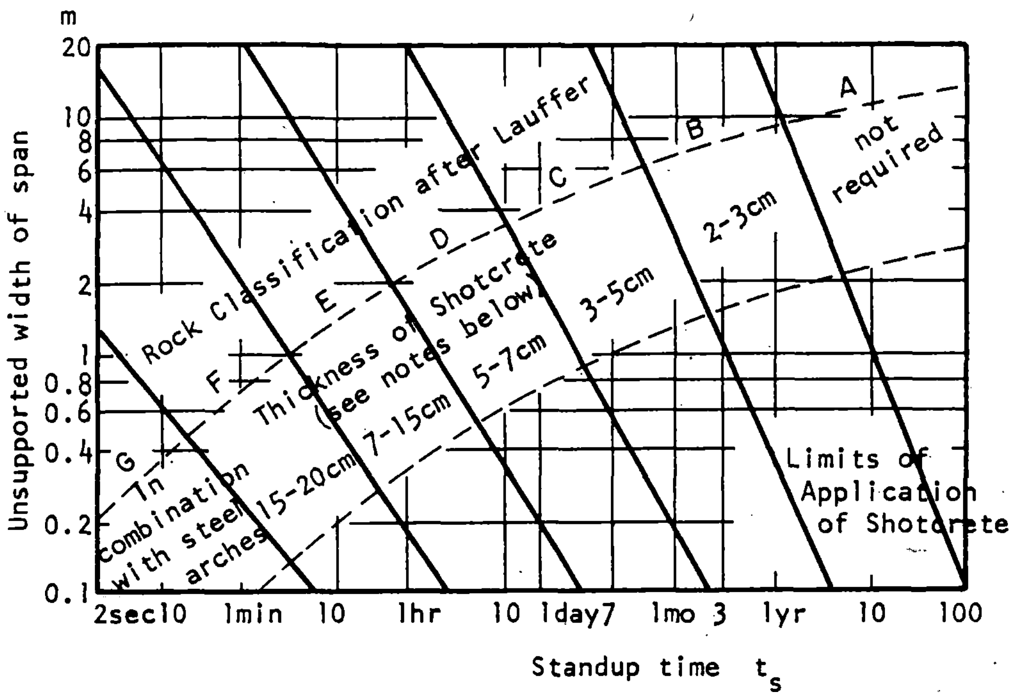


Fig. 3.13. Rock Reinforcement with Shotcrete (After Linder, 1963)

Notes:

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

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

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

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

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

(G) Shotcrete and strutted steel arches with lagging.

removed a few shovels full at a time. The nozzleman sprays the ground as soon as it is exposed. Permanent support consists of arches consisting of reinforcing bar trusses embedded in shotcrete and a continuous layer of six inch thick shotcrete.

The construction and lining procedure used on this subway was not designed, it evolved. Settlements have not occurred above the shotcreted subway. Above a nearby shield driven subway, settlements of three-quarters of an inch have occurred.

Grout

Cement and chemical grouts are usually considered only for the control of the flow of underground water. In recent years, a few attempts have been made to use resin grouts to bind soil or broken or faulted rock in an artificial matrix and thereby to increase the strength of the material. Thus strengthened, the material would contribute to its own support.

In the NORAD facility an epoxy resin grout was used to strengthen the rock at the intersection of two tunnels (Erickson, 1964). The grout was injected in specially drilled grout holes and through plastic tubes next to rock bolts. Rock movements were stopped that previously had not been stopped with rock bolts alone. A series of laboratory tests was performed to evaluate the effectiveness of the grout. These tests indicated that the grout did bond across the joints and that sufficient strength could be developed to justify the use of this expensive grout.

By cementing layers of material together it is possible to use grout for beam building. McLean (1964) reports such an application in the Homestake mine. With the use of resin grout, beams were formed with three to four times the strength of the original material.

Other uses of grout in strengthening rock have been reported by Goss and Coolbaugh (1961) and McLean and McKay (1964). In the first of these, holes were drilled ahead of the face and the material grouted. Goss and Coolbaugh report that this technique resulted in a four-fold increase in tunnel advance rate and a 50 percent decrease in lost time in bad ground. In the second application, McLean and McKay found that a combination of rock bolts and resin provided a superior support system compared to steel liner at 17 percent of the cost of the steel liner.

These examples of the successful application of grout as a medium reinforcement material indicate that the concept does work. From the experience gained it is possible to evaluate the suitability of various types of grout for the stabilization of a given medium. Nearly all successful applications have been at least partly of an experimental nature. Grout can be effective in wet materials but is ineffective if the joints or fragments have a clay coating. At present, no basis for a rational design of a reinforcement scheme using grout exists. The writers believe that Erickson (1964) best summarizes the situation in his statement that the problems of "evaluating the results and in putting numbers on the amount of benefication are intriguing and still remain to be solved."

3.10 Concluding Remarks

This chapter has discussed a number of elaborate tunnel lining design theories, but it also has shown that most actual designs of tunnel liners are based on very simplified design methods. It has been indicated that most of these simplified methods have their root in one or more of the elaborate methods.

The discussions, experience, and deliberations that convert the elaborate methods and ideas into a workable design tool are seldom included in design manuals. Without them, the designer is forced to work from blind faith and thereby does not often gain an understanding of the actual behavior of the medium around the tunnel or of the very real shortcomings of his design assumptions.

The design methods developed for particular tunnel projects, such as the BART projects, are not innovations or generalizations. They are simplified best estimates suited to particular conditions, and are not generally applicable. According to current practice, the process of developing methods of design must be repeated for every new project.

It is believed that major new and elaborate design methods are not needed. Rather, an improved appreciation of the critical assumptions and limitations of existing ones, as applied to actual field conditions, is very badly needed. In addition, it is important that the effect of present and future construction techniques, lining materials, and exploration techniques (many of which were not in existence when some of the theories were developed) be considered.

Design methods should reflect observations made in previously constructed tunnels. For example, Terzaghi (1942), observed that the tunnel tubes in the Chicago subway were acted on by fairly uniformly distributed external pressures after conditions became stable. He concluded that the bending moments in a cylindrical tube with a circular cross section are small and that the permanent lining should be as thin as compatible with construction requirements.

Field observations and records of tunnel behavior are evaluated in the second part of this report. On the basis of this evaluation, improved methods of interpreting and applying existing design methods are recommended. The recommendations permit a more logical selection of the design assumptions and theories to be used and a better appreciation of the limitations and possible errors to be expected in the designs. Emphasis is placed on gaining an understanding of the actual behavior of the medium around the tunnel and correlating material properties and behavior.

CHAPTER 4

CLASSIFICATION OF GEOLOGIC MATERIALS FOR TUNNELING PURPOSES

4.1 Concepts of Classification

An engineering classification of geologic materials serves many purposes. The most important purpose is to facilitate the recognition of problems associated with a construction project; a specific class of problems is associated with a specific class of geologic materials. The classification should also delineate which parameters are significant for design and construction problems, and indicate which theories or procedures are useful and applicable to a given case. The relative importance of the parameters, and the validity of the theories, will vary from one class of geologic materials to another. Classification is the first step towards an actual prediction of the behavior of the material in a given situation.

General and Specialized Classifications

Classification can be viewed in several steps or on several levels of specialization. One may consider a primary or _general_ classification, a secondary or _diagnostic_ classification, and a tertiary or _specialized_ classification.

The ideal primary or _general_ classification is independent of its application to a particular type of problem or structure (Coates, 1964). It depends only on the material (rock or soil mass). The direct applicability of such a classification is limited because of its general nature. However, if the classification parameters are chosen with its possible uses in mind, the classification will aid in understanding concepts of material behavior, and form a rational basis for specialized classifications.

The secondary level of classification is more specialized and may be termed a diagnostic classification. Its purpose is to recognize problems and to define the important parameters. It is developed by relating the primary classification to the problems under consideration and is, thus, dependent on its contemplated use. In tunneling, for example, the diagnostic classification of a material such as jointed rock must consider the size of the tunnel opening.

The specialization may be advanced further. The third step reaches into actual design and construction considerations. Such specialized classifications may predict the blasting behavior, rippability or drillability of a material, or furnish a prediction of rock load or soil pressure. This is the final stage of classification.

A Unified Classification of Soils and Rocks

In this report a system of classifications applicable to all geologic materials is developed. There are many reasons for this development. Modern methods of tunnel excavation and support in soils and rocks display many similarities: shotcrete has been used as temporary and permanent stabilization of the face and the circumference of the tunnels in rocks as well as in soils; mechanized excavators for soil and rock are in many respects similar; shields are being used in rocks as well as in media commonly classified as soils; and many types of linings have been used for soils as well as for rocks. Although the behavior of massive rock and sand is highly dissimilar, there are many overlapping characteristics of soils and rocks: concepts valid for stiff clays extend naturally into shales and slates; varying degrees of alteration or cementation can form any intermediate characteristics between solid rocks and unindurated soils. It is significant that the greatest problems in rock tunneling are frequently associated with zones where the rock approaches the character of soil.

Tradition in the United States has separated the art of tunneling into two distinctly different professions: soft ground tunneling and rock tunneling. The boundary between these professions is rather arbitrary. The distinction is not nearly so great in other countries; in Continental Europe, for example, soft ground is commonly considered just another rock, which is softer and creates a specific class of problems.

In addition to bridging the gaps between classification systems applicable separately to soils and rocks, a unified classification system will facilitate the coordination of soil and rock mechanics research. Furthermore, it will inspire development of unified concepts applicable to the basic problems involved in the design and construction of underground openings.

To avoid confusion in a field which is already crowded with classification methods and systems, the proposed system is tied to basic characteristics of geologic materials, and currently available and accepted classification systems will be utilized. Thus, no new terms or parameters are introduced.

Correlation with Existing Classifications

During the years, experience has been accumulated and associated with various classification systems. It is often difficult to evaluate and utilize this experience, either because the description of the classes is insufficient and qualitative or because the systems are applicable only to particular geologic regions or to particular rock types. In this report an attempt is made to draw out this experience and incorporate it in a classification of more general utility.

Most developers of classification systems for tunnel purposes have taken a behavioristic approach, that is, they have observed actual behavior of the materials in tunnels and applied descriptive terms to this behavior. Then they have correlated their descriptive classes with selected parameters of the rock or soil medium (Bierbaumer, 1913; Stini, 1950; and others). However, it is obvious that the classification cannot wait until the material behavior is experienced. It is possible to develop a clearer and more useful system by accurately describing by meaningful numbers rocks or soils as they are actually found and analyzed by advance investigations. By accumulated experience and theoretical considerations these descriptions can then be converted into predictions of tunnel behavior and design values.

The attempt to classify geologic materials may be compared to Casagrande's (1948) development of the Unified Soil Classification, which includes the primary and secondary stages as outlined in the preceding paragraphs. The Unified Soil Classifiction has not been applied to tunneling purposes in a diagnostic manner, and it may not be suited directly to this purpose. However, certain features of it will be used in this report.

The classification systems may be considered as a tool to facilitate diagnostic and design work. The writers agree with Casagrande (1948) that the experienced engineer who really understands geologic materials can apply principles of geotechnics without a formal classification. Unfortunately experienced tunnel engineers are few, and inexperienced engineers are often given large responsibilities. In addition, the designers of tunnel support systems are usually structural engineers with little knowledge of the important aspects of geologic materials, and the data on which they base their designs are collected by geotechnical engineers who often have little appreciation of structural concepts. The problem of communication between these groups of engineers is major and real. The classifications serve to point out important parameters and problems and constitute effective means of communication.

4.2 Characteristic Particle Size as a Primary Classification Parameter

One of the basic features that distinguish soil and rock from other materials encountered in engineering is the significance of the discontinuities. Since all rocks contain fractures and planes of weakness, both soil and rock may be considered particulate materials and as such, discontinuous. However, if viewed on the proper scale, discontinuous materials may sometimes be considered continuous for practical purposes. All materials are discontinuous on the atomic or molecular level but some may act as perfect continua on a practical level.

Continuity of Geologic Materials

Under ideal conditions, a uniform and continuous material would exert uniformly distributed loads on a tunnel support system, and deformations would be symmetrical with respect to a vertical plane through the longitudinal centerline of the tunnel. A discontinuous material, on the other hand, is usually not uniform and may have strongly directional properties. A discontinuous material, therefore, often deforms non-uniformly and exerts erratic loads on tunnel supports. Whereas the stability of a tunnel opening in a continuous material can be related to the intrinsic strength and deformation properties of the material, stability in a discontinuous material depends primarily on the character and spacing of the discontinuities.

It is important to determine whether a material should be considered continuous or discontinuous in a particular case. Accordingly, the type of behavior of the material may be predicted, suitable theories and methods of design may be employed, and the reliability of the predictions and the safety factor for the support may be estimated.

If a sample of a particulate material, sand or jointed rock, is only a few times larger than the size of individual fragments, the sample cannot be considered continuous on a statistical basis. If the sample size is many times the size of the individual fragments, the effect of the discontinuities is statistically levelled out, and the sample may be considered continuous.

For a tunnel driven through a jointed or particulate material, a characteristic dimension, such as the tunnel diameter, may be compared with the sizes of the individual fragments or the joint spacing of the material. In this manner, particle or fragment size, or joint spacing, Other features enhance the usefulness of a characteristic particle size (or spacing) as a classification tool. The permeability of soil to water and air is closely related to the grain size. Cohesion or coherence increases with decreasing grain size, and the importance of time-dependent behavior is closely related to the grain size. The importance of gravity forces and the volumetric stability are related to grain size in a more secondary manner. In addition the mineralogy of the individual grains is often closely related to the grain size.

If the action of the medium around a tunnel opening is considered, the following interesting observation can be made: when the particle size is very small (clay size) the medium acts as a coherent continuum. With sand and gravel sizes the medium is a non-coherent continuum. For a certain range of joint spacings, depending on the size of the tunnel opening, a rock medium acts as a discontinuum and the character and geometry of the discontinuities are of major importance. But when the joint spacing becomes very large, the medium again acts as a coherent continuum. We have, in fact, closed the circle and noted the similarity between the action of a clay on one side and massive rock on the other side. This similarity is further emphasized by the gradational change in character from soft clay to hard clays, to massive shales and to massive rocks of any constituency. This observation leads to the proposal of a circular general classification chart such as shown in Fig. 4.1.

The general classification may be related to the applicability of various types of theories and the major controlling factors of tunnel behavior. The boundaries shown on the chart are approximate. Fig. 4.1 is discussed further in subsequent sections.

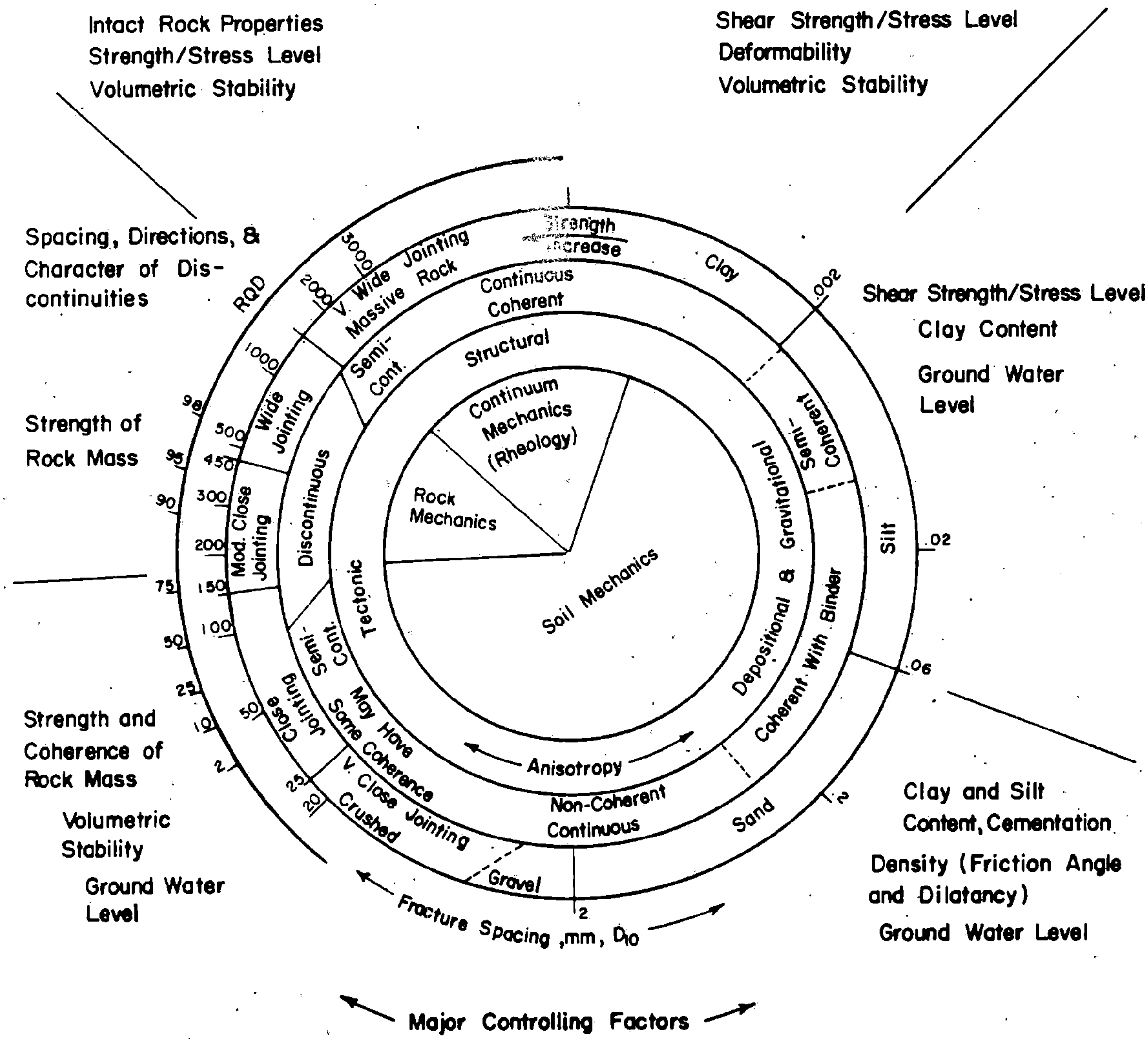


Fig. 4.1. Unified Classification of Geologic Materials

Characteristic Parameters of Soils and Rocks

The average particle size of a medium is not necessarily the best indicator of the behavior of the medium. A clayey sand may have an average grain size in the sand range, but its behavior may be more characteristic of a clay. The behavior of a jointed rock mass with great variation in joint spacings may be controlled by the smaller joint blocks rather than the average block size.

The finer grain size fraction of a soil is generally of disproportionate importance, in particular for the permeability of the soil. Hazen (1892), and many others have taken this into account by using the equivalent diameter, D_{10}*, of the 10 percent fraction, rather than the mean diameter for evaluating permeability. Indications are that D_{10} may be indicative of strength and deformation properties as well. Therefore, D_{10} has been adopted as a primary classification parameter for soils. The grouping and description terms are those used in the Unified Soil Classification.

The uniformity of the soil obviously is an important factor determining its behavior. This factor is not considered directly in this primary classification diagram, although it can be considered by the choice of an appropriate representative grain size.

A complete basic classification of a rock mass should include the following important characteristics:

1) Spacing, regularity, and character of discontinuities;
2) Properties of the intact rock;
3) Degree of alteration.

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* 10 percent of the weight of a sample passes a sieve with openings of a width D_{10}.

These characteristics determine the behavior of a jointed rock mass. Except when the rock is highly altered or is very weak, the overriding characteristic for the problem of opening stability is the first one, the character of the discontinuities. In addition it is commonly found that a high degree of fracturing exists together with a high degree of alteration. It is, then, suitable to use parameters related to the character of the discontinuities as a basis for both the primary and the secondary classification systems.

The terms "joint spacing" and "average joint spacing" have often been used in the description of rock masses. Strictly speaking, these terms refer to distances between joints in particular sets of parallel or nearly parallel joints. When joints are observed along a bore hole or a tunnel wall, the measured spacings or frequencies are not true joint spacings or frequencies. Joints of different sets intersect the wall at varying angles, and the average spacing or frequency measured is a distorted and compound measure of all the joint sets. In addition, fractures are included which do not necessarily belong to any joint set but may, for example, be partings along weak bedding or schistosity planes. Often, however, the term "joint spacing" is used for the spacing of fractures along a bore hole or a tunnel wall, and it is frequently difficult to determine whether a "joint spacing" recorded in the literature is a true joint spacing or a fracture spacing. In this report the term "joint spacing" is reserved for the strict meaning of the term, and "fracture spacing" refers to spacings of all fractures encountered along a bore hole or a tunnel wall. "Fracture frequency" is the number of fractures per ft.

Where several joint systems are present, the average fracture spacing is generally $\frac{2}{3}$ to $\frac{1}{3}$ of the average joint spacing of any one of the joint systems. For correlation purposes it is considered sufficiently accurate to use a ratio of 1/2 between fracture spacing and joint spacing. It must be realized, however, that this ratio may be considerably in error. If, for example, there is only one dominant joint system, the ratio would be closer to unity. The ratio also depends on the orientation of the bore hole or tunnel wall relative to the directions of the joints.

Fig. 4.2 shows several attempts to classify rock according to the spacing of the joints or fractures. John's classification has the pleasing trait that it is related to numbers which are an extension of the Unified Soil Classification (2,20,200 cm). Unfortunately John's classification is very insensitive to rock mechanics problems. Deere's classification breaks up the scale in smaller sections, compatible with the relative importance of the joint spacings. It is significant that where Deere calls a spacing "moderately close", John calls it "wide." For purposes of correlating experience, a few other systems are shown. Whereas John considers joint spacings in the strict sense of the word, there is some doubt as to the definition of "joint spacing" in the remainder of the systems given here. It is possible, therefore, that these scales should properly be placed somewhat to the right of the position shown.

It is usually difficult to obtain an accurate estimate of the joint spacing or the fracture spacing on the basis of exploratory borings. It is, therefore, desirable to use a supplementary parameter which is easier to obtain. For years the core recovery has been taken as an indicator of the soundness of the rock. The utility of this parameter is limited because of extraneous factors influencing the core recovery. The modified

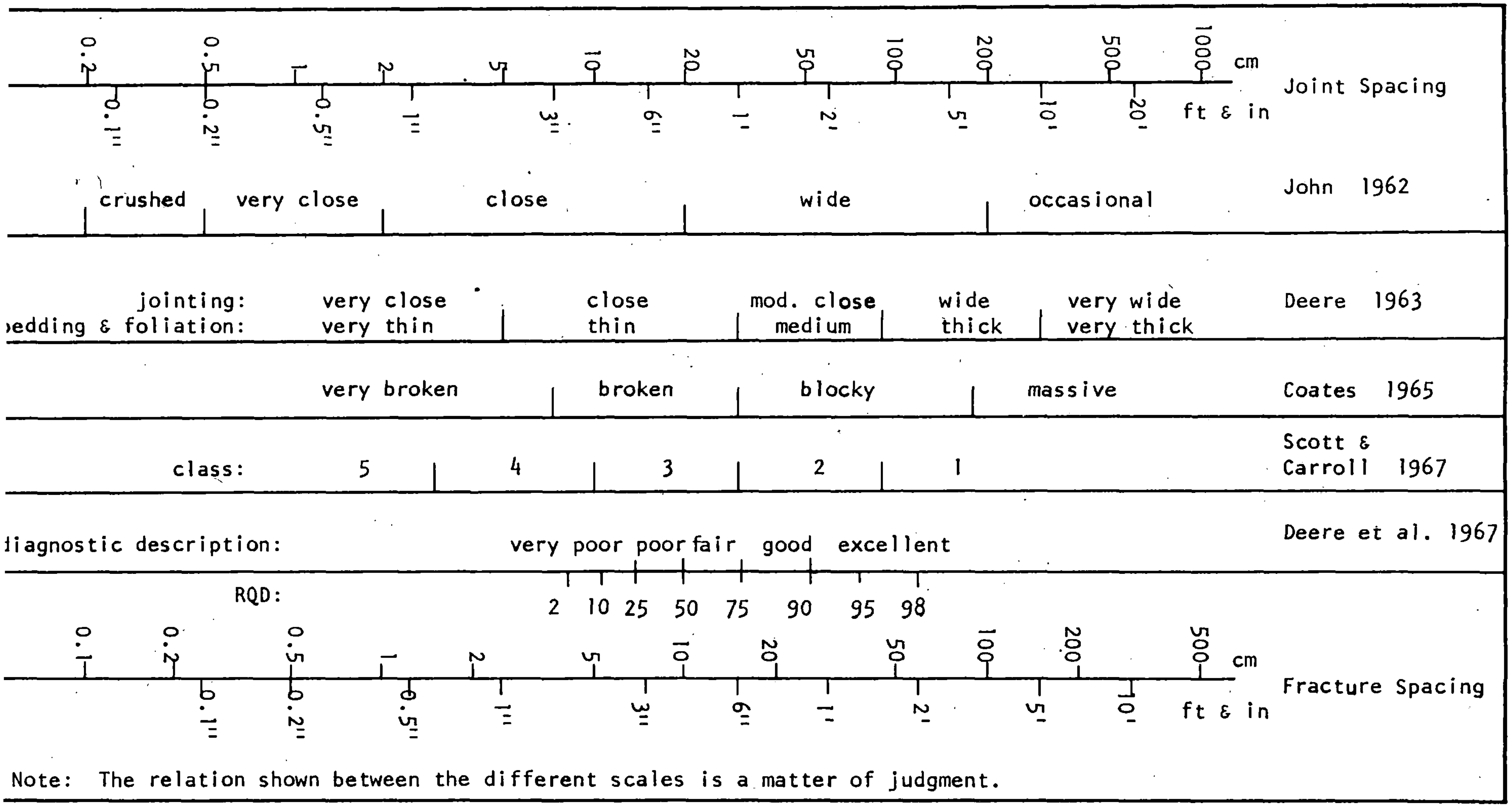

Fig. 4.2. Correlation of Joint Spacing Classifications

core recovery (Rock Quality Designation - RQD) eliminates some of the uncertainities in the core recovery classification (Deere et al., 1967; Merritt, 1968).

Rock Quality Designation (RQD)

Considerable experience with the Rock Quality Designation (RQD), developed at the University of Illinois, has now been accumulated. For example, Coon (1968) correlated RQD with actual tunnel support in a number of tunnels. RQD is at present used, for example, by the California Department of Water Resources, and Chicago Metropolitan Sanitary District. The RQD has been correlated with fracture frequency (Deere et al., 1967) and is included in Fig. 4.2. The relation between fracture spacing and joint spacing is not accurate, as mentioned previously; the choice of a ratio 1/2 between fracture spacing and joint spacing is a matter of jugment. In addition, RQD is also influenced somewhat by the degree of alteration and other weakness factors. Thus, the location of RQD on the scale of joint spacing must not be taken as an accurate correlation, and may vary from one rock type to another.

In the primary classifcation diagram, Fig. 4.1, the average fracture spacing is taken as a parameter for the jointed rock, and RQD is indicated on the diagram.

The Direct Use of the Primary Classification

The primary classifcation is of limited direct utility for design purposes. It does, however, outline some important features of tunnel behavior. It indicates for which materials coherent or non-coherent behavior may be expected and it brackets the region of discontinuous behavior. This permits a preliminary judgment of which theories may be applied in a given case, as indicated on Fig. 4.1, and which material parameters to use in analyses.

The term "coherence" requires some comment. It is preferred over "cohesion" as a descriptive term for geologic materials. It describes the behavior of the material during and after construction and is a material quality that may be influenced by the conditions during construction. Clean sands, though not possessing real cohesion, may display coherence caused, for example, by moisture in relatively fine sands, or by a slight amount of cementation which is often not discovered by sampling the material. Characteristics of weakly cemented sands have been treated by Kieslinger (1962), who also used the term "coherence" (Zusammenhang). In some materials coherence can be demonstrated by a compressive strength in unconfined compression, or by a tensile strength. Materials with real cohesion also have a coherence. Some materials which do not have a tensile strength or a real cohesion may under certain circumstances display a coherence which is closely related to the dilational character of the material near failure. This coherence is of a frictional character. Coherence may be induced by various methods of restraint or prestressing.

Fig. 4.1 indicates that the material around an opening may be assumed continuous for a large variety of geologic materials. For most jointed rocks, however, this assumption is a poor one. The boundaries of the range of joint spacings for which the material must be considered discontinuous are important. The size required for a sample of a discontinuous material to be considered continuous is a matter of judgment. John (1962) suggests that a sample of about 10 times the average size of the single units of the discontinuum may be considered a uniform continuum. It is clear that this will depend to a great extent on the uniformity of the unit sizes in the material or the uniformity of the spacings of the discontinuities. As mentioned, it is likely that a spacing less than the average joint spacing is characteristic of tunnel behavior. As an empirical limit of the range of discontinuous rock masses, therefore, the ratio of particle size or joint spacing to tunnel diameter of 1 to 50 has been chosen. If the average joint spacing is less than $\frac{1}{50}$ of the tunnel, the material may in most cases be assumed to act as a continuum.

On the other hand, if the joint spacing is large in comparison with the size of the tunnel opening, the rock behavior will be largely independent of the joints. This might be expected to occur when the joint spacing is greater than about one third of the size of the opening.

Using the approximate ratio of 1/2 between fracture spacing and joint spacing, it is found that a rock should be considered discontinuous when the ratio of fracture spacing to tunnel diameter is between the approximate limits of 1/5 and 1/100. For a range outside these limits, the rock may be considered continuous, though possibly anisotropic. Fig. 4.1 shows these approximate boundaries when the tunnel opening is about 20 ft wide.

In a jointed rock the directions of the joints usually show considerable regularity, although the spacings may be irregular. Thus, jointed rocks usually display anisotropic behavior, even when the joint spacings are narrow. Anisotropic properties can be expected also of materials that may be considered uniform.

In the following section and in subsequent chapters, the primary classification is utilized to develop classification subsystems and to delineate the influence of the different parameters on the behavior of the materials in tunnels.

4.3 Distinguishing Features of Tunnel Behavior

To develop the diagnostic and analytical phases of a classification of geologic materials for tunnel purposes, it is necessary to consider the actual mechanisms of behavior of the media in tunnels. This will define the important parameters and problems that should be considered. Some aspects of tunnel behavior have already been discussed in Chapters 2 and 3.

Utility of Behavioristic Approach

A behavioristic approach consists of a description of tunnel behavior under a variety of conditions and a subsequent correlation with soil and rock properties. The engineer needs information that will help him predict the behavior of a tunnel given certain properties of the medium. This information must first be developed by a behavioristic approach.

The most comprehensive behavioristic classification of soils and soft rocks was developed by Terzaghi (1950). A detailed description of this work is given in Appendix II. According to this classification, one soil type may belong in 2 or 3 different groups, and each group contains a number of different soils in varying conditions.

An important parameter relating a soil type to a behavioristic group is the hydraulic condition. Thus, a fine sand may be running if dry, cohesive-raveling if moist, or flowing if below the ground water table. Fig. 4.3 shows the relation of Terzaghi's classification to the primary classification and includes the effect of the hydraulic condition.

The great direct effect of the hydraulic condition is felt by the continuous, non-coherent materials. For materials with even a small amount of coherence, the effect is smaller and other characteristics

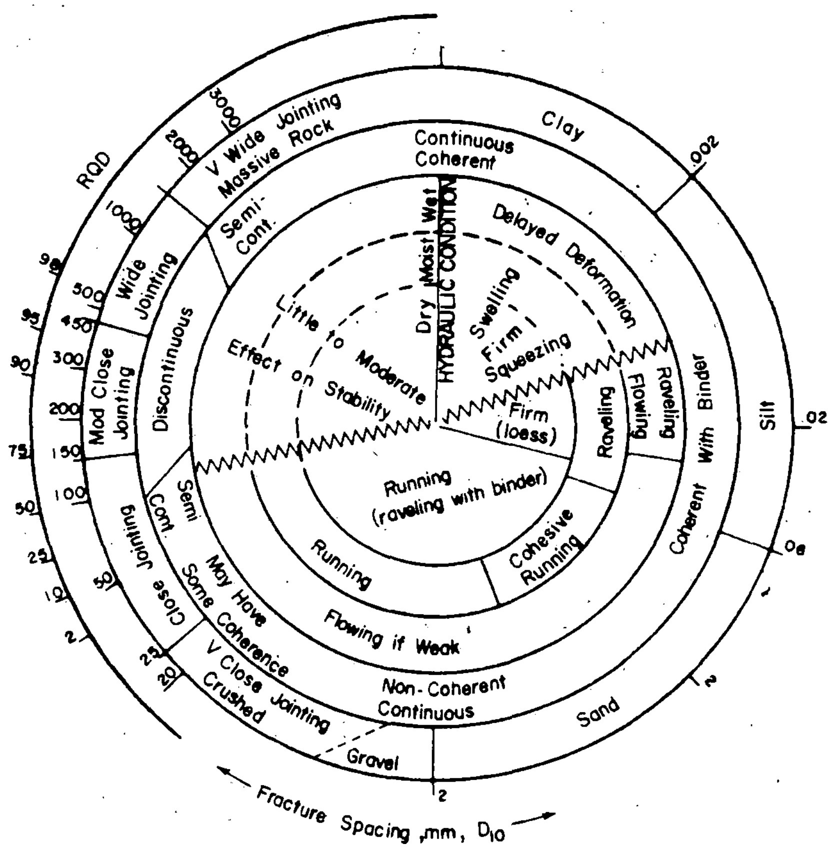

Fig. 4.3. Relationship Between Terzaghi's Behavioristic Classification and the Primary Classification, Including the Effect of the Hydraulic Condition

Using Terzaghi's terms (Terzaghi, 1950), non-coherent behavior is characterized by loosening, slowly or rapidly raveling, running, and flowing; semi-coherent or temporarily coherent behavior by cohesive raveling; and coherent behavior by squeezing, swelling, or being stable.

Significance of Shear Strength and Induration

The most significant parameter for continuous, coherent materials is usually the shear strength, or rather the ratio of the shear strength to the overburden pressure. It will be shown later that there is a direct relationship between this ratio and the stability of the opening. For small values of the ratio, the opening is unstable and must be supported. For higher values, the opening may be reduced by squeezing in an amount largely determined by the ratio. As the ratio increases, the opening becomes more stable, although some high plasticity clays or clay-shales may exhibit swelling. If the shear strength is high, for example in a massive shale where fissures are widely spaced, the ratio will indicate whether popping may be expected.

The significance of the shear strength, or the ratio of shear strength to overburden pressure, is partly direct, relating stresses and strengths around the opening. Moderate or high shear strengths are also indirect indicators of consolidation and induration and, thus, of problems related to these features. The amount of consolidation or induration, together with a measure of the plasticity of the material, indicates the swelling potential, and influences other things, such as possible initial stress states. These features will be discussed in detail in a later chapter.

As the shear strength and the degree of induration increase, the massive and non-fissile material grades into a fissile material, and the likelihood of encountering joints and other discontinuities increases. When the discontinuities render the strength of the rock mass appreciably lower than the strength of the intact rock, the intact strength loses most of its importance, and the amount and character of the fissuring control the behavior. In a soft rock, such as clay-shale, a high degree of fissuring does not necessarily mean that the fissuring controls the behavior. Such fissures are often discontinuous so that the strength of the rock mass is not appreciably lower than the intact strength, and other features, such as swelling or squeezing control the behavior.

Although the mineralogy and the petrography of a rock in itself are of limited importance except in a few cases, the classification chart given in Fig. 4.4 is of some use. It is a specialized primary classification of sedimentary rocks, indicating the boundaries of different types of behavior, the non-coherent and the coherent types. For most of the rocks, beyond a certain shear strength, the behavior of the material in a given situation is governed by the discontinuities.

The thought processes leading to Fig. 4.4 may be reversed to indicate the effect of fracturing and alteration on an indurated material. The final product of the processes of fracturing and alteration would depend both on the relative importance of the two processes and on the parent material. Igneous rocks may be included in this reverse diagram as well.

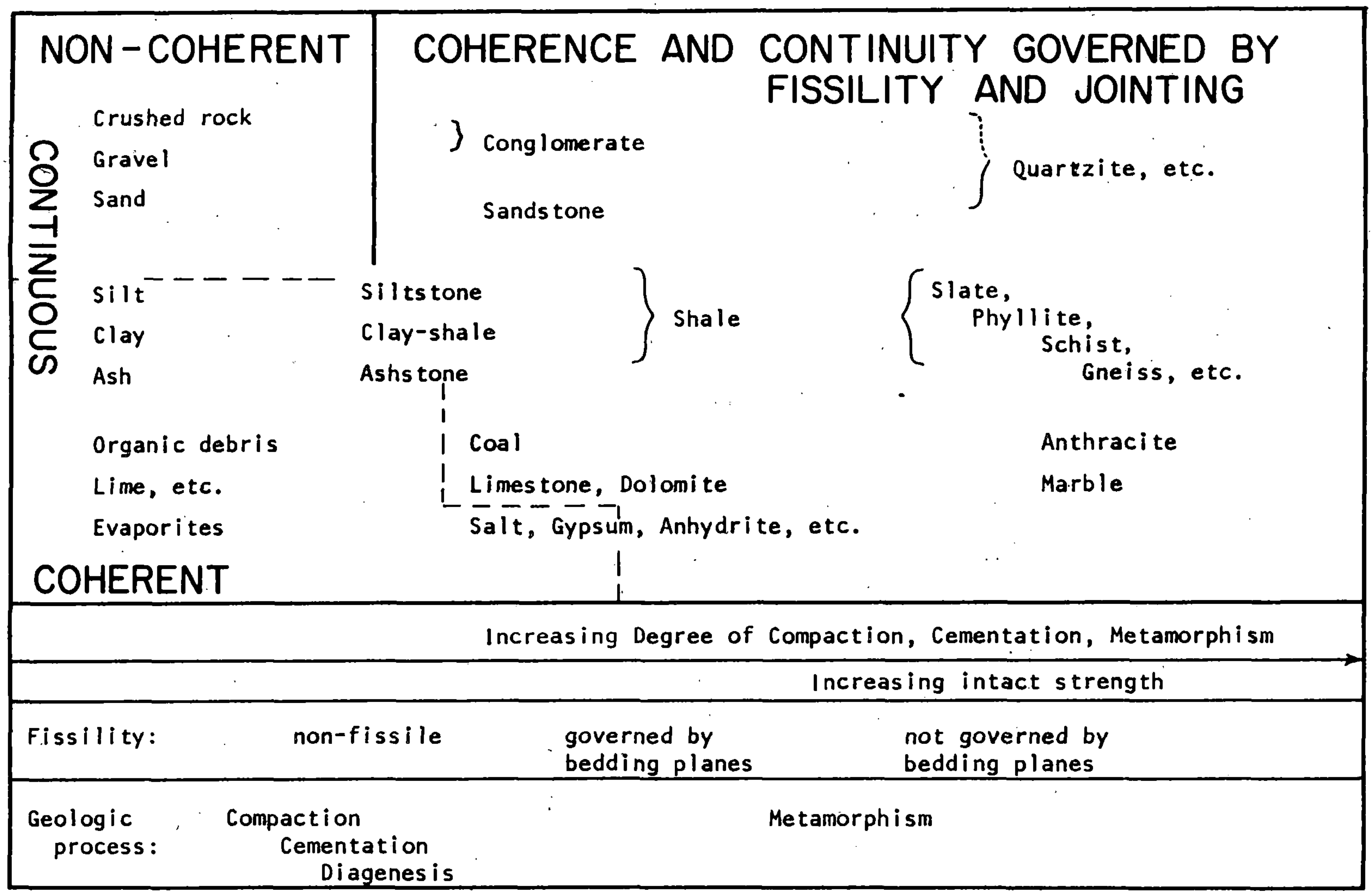

Fig. 4.4. Primary Classification of Sedimentary Rocks

Significance of Gravity Forces

Where the shear strength is a primary controlling factor of the behavior of a tunnel, gravity forces enter the problem only as the source of the overburden pressure. Usually the orientation of the tunnel opening in relation to the direction of the gravity forces is of minor importance. In non-coherent materials, however, the action of the gravity forces is more direct and the conditions above the crown of the tunnel are significantly different from those below the invert.

If the material is continuous, the mechanism of the behavior is largely independent of the size of the opening, but deformations are roughly proportional to the size. If, on the other hand, the material is discontinuous, the mechanism may vary with the size of the opening. With a joint spacing of, for example, one ft, an opening with a diameter of 1/2 ft would behave as an opening in a nearly perfect continuum. With an opening diameter of 5 ft, the behavior would approach that of a discontinuum, and would be governed by the fractures. If the opening is large enough, the fractures would still govern the behavior but the spacing relative to the size of the opening would be so small that the material could be assumed to act as a continuous non-coherent material. An indication of this change of mode of behavior is the variation of the stand-up time with the unsupported length of an opening (see Appendix II). Depending on the general quality of the rock, Lauffer (1958) indicates a decrease in stand-up time proportional to the unsupported length to a power varying between -1 and -2. Terzaghi (1948) indicates somewhat similar relationships. The changing behavior of a discontinuous mass is also reflected in the recommended values of rock loads (Terzaghi, 1946; Bierbaumer, 1913; see Appendix II), which increase linearly with the size of the opening.

These effects are directly related to the pull of gravity, which is, indeed, the major source of the loading of any tunnel support system and a controlling factor for the behavior of a tunnel opening. In a discontinuous mass the gravity effect is direct. The rock load concept is based on the requirement that a certain height of rock must be carried by the support system. In non-coherent continuous masses, arching concepts and stress concentrations are related directly to the pull of gravity, and theories of subsiding blocks and wedges are based on this direct effect. However, if the material is coherent, the effect of gravity is more indirect. Here, the controlling factor is the stress level as indicated by the overburden pressure in relation to the shear strength. In expansive or swelling rocks, the direct gravity effects may lose their significance altogether.

4.4 Use and Expansion of Classification

The previous sections have described some of the important features of tunnel behavior as related to properties of the medium. The treatment is incomplete and serves mainly to outline the concepts and utility of the classification of geologic materials and to point out the most significant parameters for tunnel behavior. In the remainder of this report these concepts will be followed and amplified. It is intended that the concepts be developed to serve as tools applicable to the exploratory stages of a tunnel project as well as to design and construction of the project.

Geologic Explorations for Tunnels

One of the greatest difficulties in dealing with geologic materials is their variability. Within short distances, a tunnel project may encounter the most diverse conditions. It is nearly impossible to uncover all the important variations by present-day exploration techniques. A large number of case histories attest to the frequency with which unexpected conditions occur, often with disastrous results.

Although a good classification system will enable the engineer to predict the behavior of a tunnel accurately in a particular medium, it cannot reduce the risk of encountering unpleasant surprises. This can be done only by increased exploration of the tunnel alignment. An ideal way to secure information about conditions along the tunnel alignment is to drill exploratory borings in front of the advancing face. Experience shows, however, that this is a very expensive way of securing information. Furthermore the information is needed at the design stages before bidding. Such procedures make tunneling safer and may indeed help the contractor make money, but do not reduce expenses for the owner. A method is needed which will give adequate information about the variability of the geologic materials along the total alignment by inexpensive advance investigations.

Use of Results from Geologic Explorations

Armed with a general knowledge of the geological conditions along the tunnel alignment, the engineer is able to design an exploratory program which will furnish the proper parameters for the design of the tunnel. As the exploration goes on, the program should be modified according to the findings of the explorations in such a manner that the significant parameters for the prevailing conditions can be extracted. If the exploration uncovers unexpected conditions it may be necessary to modify the exploratory technique to investigate other or additional significant factors.

With the parameters of the project (size and shape of tunnel, depth and overburden pressure) and the results of explorations, the engineer will be able to classify the behavior of the material surrounding the tunnel in a general manner (i.e., coherent or non-coherent, continuous or discontinuous types of behavior) and evaluate the possible significance of the overburden pressure and gravity forces. This classification and evaluation involves the primary classification chart, Fig. 4.1.

If the behavior will be essentially coherent, the next step is a study of the shear strength in relation to the overburden pressure and the plasticity to determine whether the behavior will be stable, squeezing, or swelling, or whether popping is likely, and to design a suitable support system, if necessary. For this purpose the engineer will consult charts or tables.

If, on the other hand, the parameters indicate discontinuous behavior, the character of the discontinuities must be studied further, the stand-up time evaluated, and construction procedures adopted which will reduce the amount of loosening of the rock. Rock loads will be estimated. This study may also be made with the aid of charts or tables.

In a continuous but non-coherent material, the hydraulic conditions attain great importance. The location of the ground water table, the permeability, D_{10}, and the degree of cementation determine whether the material will be flowing, running or raveling. The type and degree of severity of the problem determines the remedial measures and supports required. In addition, the amount of water flowing into the tunnel may be estimated.

Various additional factors, such as adverse anisotropy of the initial state of stress, may be evaluated where necessary, and additional features such as feasibility of various excavation methods may be correlated with the classification system.

Finally it is possible to relate expenses and rates of progress to a classification system. This has, for example, been attempted by the California Department of Water Resources (ENR, 1959) and Coon (1968). This, however, is not a topic for this investigation.

It is the opinion of the writers that the use of a reasonably uniform and systematic classification of geologic materials as a basis for the evaluation of all types of problems connected with tunneling will simplify the concepts of tunnel behavior and make them easier to apply. Furthermore, it will provide the necessary framework for the accumulation of experience. The remainder of this report is devoted mainly to the filling in of this framework.

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APPENDIX I

ELASTIC AND PLASTIC STRESS DISTRIBUTION AROUND CIRCULAR UNDERGROUND OPENINGS

I.1 Introduction

This appendix summarizes the results of elastic and perfectly plastic theories applied to the problem of stress distribution around unlined circular underground openings. The idealized materials considered include elastic material and plastic "φ=0" and "c-φ" materials. The theories assume that the opening is situated deep enough that the effect of the proximity of the earth surface can be neglected.

The complete derivations of these results are not given, and the equations are included in this report primarily for easy reference.

I.2 Elastic Stress Distribution

The problem is considered a plane strain problem, the material is elastic with Poisson's ratio ν. Kirsch's solution (see for example Terzaghi and Richart, 1952) disregards the influence of the depth of the opening. However, Mindlin's more comprehensive solution (1940), which considers the depth, shows that the approximation is very good for depths more than about four diameters.

If the overburden pressure at the depth of the tunnel axis is $p_z = \gamma h$, and the ratio of the horizontal pressure to the vertical pressure is $K_o = p_h/p_z$, the stresses in the medium are given by:

$$\sigma_r = \frac{1}{2} p_z \left[(1+K_o)\left(1-\left(\frac{a}{r}\right)^2\right) + (1-K_o)\left(1+3\left(\frac{a}{r}\right)^4 - 4\left(\frac{a}{r}\right)^2 \cos 2\theta\right)\right] \qquad (I\text{-}1)$$

$$\sigma_\theta = \frac{1}{2} p_z \left[(1+K_o)\left(1+\left(\frac{a}{r}\right)^2\right) - (1-K_o)\left(1+3\left(\frac{a}{r}\right)^4\right) \cos 2\theta\right] \qquad (I\text{-}2)$$

$$\tau_{r\theta} = (K_o-1)\left(1-3\left(\frac{a}{r}\right)^4 + 2\left(\frac{a}{r}\right)^2\right) \sin 2\theta \qquad (I\text{-}3)$$

$$\sigma_y = \nu(\sigma_r + \sigma_\theta) \qquad (I\text{-}4)$$

The notations are indicated on Fig. I-1. σ_y is the stress in the direction parallel to the tunnel axis. For K_o=1, the formulae are simplified to

$$\sigma_r = p_z \left(1-\left(\frac{a}{r}\right)^2\right) \qquad (I\text{-}1a)$$

$$\sigma_\theta = p_z \left(1+\left(\frac{a}{r}\right)^2\right) \qquad (I\text{-}2a)$$

$$\tau_{r\theta} = 0 \qquad (I\text{-}3a)$$

$$\sigma_y = 2\nu p_z \qquad (I\text{-}4a)$$

If a uniform internal pressure p_i is applied to the walls of the opening, the formulae for K_o=1 are modified to

$$\sigma_r = p_z \left(1-\left(\frac{a}{r}\right)^2\right) + p_i \left(\frac{a}{r}\right)^2 \qquad (I\text{-}1b)$$

$$\sigma_\theta = p_z \left(1+\left(\frac{a}{r}\right)^2\right) - p_i \left(\frac{a}{r}\right)^2 \qquad (I\text{-}2b)$$

The formulae for $\tau_{r\theta}$ and σ_y remain unchanged.

Equations I-1a and I-2a are illustrated on Fig. 3.1 in the main body of the report.

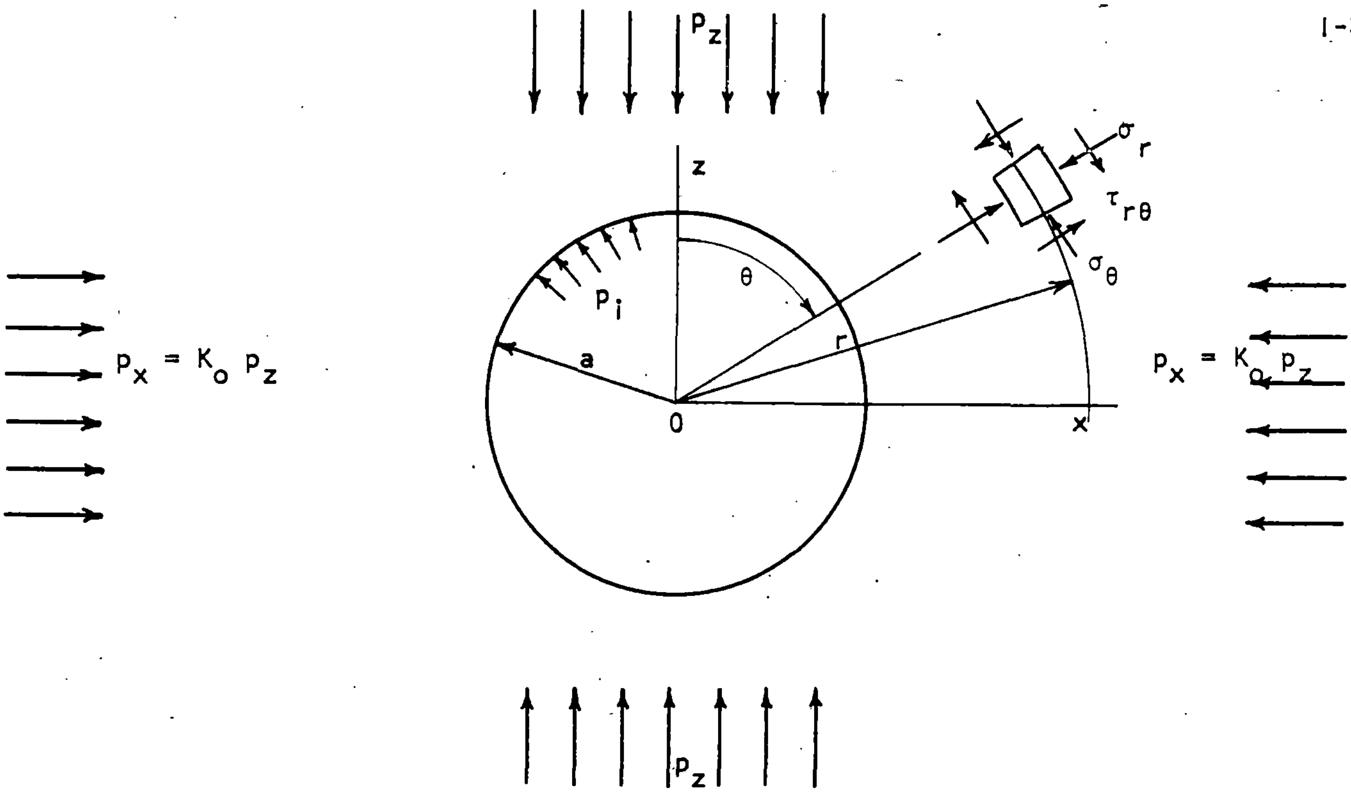


Fig. 1-1: Notations

The maximum and minimum tangential stresses for an unlined opening occur at the crown (and invert) and at the springlines (for θ=0 and $\frac{\pi}{2}$). The stresses at these points are given by

$$\sigma_\theta = p_z\ (3\ K_o - 1) \quad \text{at crown and invert,} \tag{1-5}$$

$$\sigma_\theta = p_z\ (3 - K_o) \quad \text{at the springlines.} \tag{1-6}$$

For $K_o = 1$, the stresses are uniform around the opening (independent of θ) and the tangential stress at the wall is $2p_z$. For $K_o < \frac{1}{3}$ and $K_o > 3$, tensile stresses occur either at the crown and invert or at the springlines.

1.3 Plastic Stress Distribution, φ=0

A plastic material is considered, where yielding will occur when

$$\sigma_1 - \sigma_3 = 2c, \tag{1-7}$$

where c is the undrained shear strength or one half the unconfined compressive strength, q_u. When the stress difference is less than 2c, the material is considered elastic. The stress field is a uniform compression stress, $p_z = \gamma H = p_h$, i.e., $K_o = 1$, and the influence of depth is disregarded. An internal pressure p_i is applied to the walls of the opening.

For $p_z - p_i \leq c$, no plastic zone will develop because the stress difference is everywhere less than 2c. For $p_c - p_i > c$, a plastic zone of radius R will develop, where

$$R = a \exp\left(\frac{p_z - p_i}{2c} - \frac{1}{2}\right) \tag{I-8}$$

This equation is illustrated on Fig. 3.2 in the main body of the report. In the plastic zone, for $a \leq r \leq R$, the stresses are

$$\sigma_r = p_i + 2c \ln \frac{r}{a} \tag{I-9}$$

$$\sigma_\theta = \sigma_r + 2c = p_i + 2c \left(1 + \ln \frac{r}{a}\right) \tag{I-10}$$

$$\sigma_y = \frac{1}{2}(\sigma_r + \sigma_\theta) = p_i + c \left(1 + 2 \ln \frac{r}{a}\right) \tag{I-11}$$

In the elastic zone, for $r \geq R$, the stresses are

$$\sigma_r = p_z - c \left(\frac{a}{r}\right)^2 \exp\left(\frac{p_z - p_i}{c} - 1\right) \tag{I-12}$$

$$\sigma_\theta = p_z + c \left(\frac{a}{r}\right)^2 \exp\left(\frac{p_z - p_i}{c} - 1\right) \tag{I-13}$$

$$\sigma_y = 2 \nu p_z \tag{I-14}$$

In the plastic zone the volume is assumed constant, so that $\nu = \frac{1}{2}$. The shear stress $\tau_{r\theta}$ is zero everywhere because of symmetry. The notations are given in Fig. I-1. Note that for $c = p_z - p_i$, R = a and the formulae

1-12 and 1-13 are identical to the elastic formulae for $K_o = 1$. The radial stress at the boundary of the plastic zone is $\sigma_R = p_z - c$, independent of p_i.

A derivation of these formulae for $p_i = 0$ is given, for example, by Obert and Duvall (1967).

1.4 Plastic Stress Distribution, c-φ Material

A material is now considered which will yield when

$$\sigma_1 = \sigma_3 \frac{1 + \sin \varphi}{1 - \sin \varphi} + \frac{2c \cos \varphi}{1 - \sin \varphi} . \qquad (1\text{-}15)$$

This is the Mohr-Coulomb yield criterion. c is the cohesion intercept on a τ-σ diagram and φ the friction angle, as determined by triaxial tests. The notations and the assumptions are, otherwise, as in 1-3, for example, $K_o = 1$ and the material is assumed to maintain its volume in the plastic zone, i.e., $\nu = \frac{1}{2}$.

For $p_z \leq \frac{p_i + c \cos \varphi}{1 - \sin \varphi}$ there is no plastic zone, and the elastic formulae are valid. For larger values of p_z, an annular plastic zone with radius R is developed, where

$$R = a \left[(1 - \sin \varphi) \frac{p_z + c \cot \varphi}{p_i + c \cot \varphi} \right]^{\frac{1 - \sin \varphi}{2 \sin \varphi}} \qquad (1\text{-}16)$$

In the plastic zone, $a \leq r \leq R$, the stresses are:

$$\sigma_r = - c \cot \varphi + (p_i + \cot \varphi) \left(\frac{r}{a}\right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} \qquad (1\text{-}17)$$

$$\sigma_\theta = \frac{2 c \cos \varphi}{1 - \sin \varphi} + \frac{1 + \sin \varphi}{1 - \sin \varphi} \sigma_r \qquad (1\text{-}18)$$

$$\sigma_y = \frac{1}{2} (\sigma_r + \sigma_\theta) = \frac{c \cos \varphi}{1 - \sin \varphi} + \frac{1}{1 - \sin \varphi} \sigma_r \quad (1\text{-}19)$$

At the boundary of the plastic zone, the radial stress is

$$\sigma_R = p_z (1 - \sin \varphi) - c \cos \varphi \quad (1\text{-}20)$$

and the tangential stress is

$$\sigma_\theta = p_z (1 + \sin \varphi) + c \cos \varphi . \quad (1\text{-}21)$$

In the elastic zone, $r \geq R$, the stresses can be found by equations I-1b, I-2b, I-3a, and I-4a by substituting R for a and σ_R for p_i.

Examples of the variation of the radial and tangential stresses with distance from the opening are given on Fig. 3.4. Equation I-16 is illustrated on Fig. 3.3.

The equations above appear to have been derived more or less independently by Kastner (1949), Fenner (1938), Sirieys (1964), and Bray (1967). Their respective equations, however, take forms that bear little resemblance to those given here or to each other.

The formulae may be simplified by setting either p_i or c equal to zero. However, the solution is not possible for both p_i and c equal to zero, because the opening is unstable in this case. For $\varphi=0$, the equations in Section I.3 apply.

When the material in the plastic zone remains coherent, as for example in a clay, the strength properties of the material are not altered greatly, and the formulae given above apply. For more brittle materials, however, the transformation into a plastic material leads to fracturing, and the material may lose strength. Thus, in some cases, though a cohesion c is assumed for the material in the elastic region, it may be reasonable to drop this cohesion in the plastic zone. In that case, the radius of the plastic zone is

$$R = a \left[(1 - \sin \varphi) \frac{p_z - \frac{c \cos \varphi}{1 - \sin \varphi}}{p_i} \right]^{\frac{1 - \sin \varphi}{2 \sin \varphi}} . \qquad \text{(I-22)}$$

While the tangential stress on the elastic side of the elastic-plastic boundary is still given by Equation I-21, the tangential stress just inside the plastic zone is less by the amount $\frac{2\ c \cos \varphi}{1 - \sin \varphi}$.

There are to the writers' knowledge no closed solutions available for values of K_o other than one, or which take into account the influence of the depth. However, work by Fenner (1938) indicates that the change in pressure with depth would increase the radius of the plastic zone upwards and reduce it downwards, so that the plastic zone resembles an ellipse with the cavity in its lower focal point.

It is possible to obtain an idea about the configuration of a plastic zone for K_o not equal to one by regarding the material elastic and finding the zones where the shear strength of the material is exceeded. This method, of course, is not accurate because the plastic zones cause stress redistribution, so that usually the plastic zones would be estimated too small. It is possible, however, on the basis of elastic stress distribution to make an accurate prediction of the stress level that will initiate plastic behavior at a point on the tunnel wall. Whenever the tangential stress at the free tunnel wall reaches a value of $q_u = 2c$ (or $\frac{2\ c \cos \varphi}{1 - \sin \varphi}$ for a frictional material), plastic behavior is initiated.

It is possible to obtain an estimate of the elastic deformations of the tunnel walls for all values of K_o (Obert and Duvall, 1967). The plastic deformations can be estimated by a method given by Bray (1967).

APPENDIX II

SOIL AND ROCK CLASSIFICATIONS, ROCK LOADS, AND STAND-UP TIME

II.1 Introduction

The concepts of rock load and stand-up time are discussed in Chapter 3. This Appendix lists some of the more important classifications of rock on the basis of the rock load concept, together with recommended loadings for design. In addition, Terzaghi's behavioristic classification of soil for tunneling purposes is presented.

II.2 Soil and Rock Classifications

The rock load concept, that the support system carries a certain height, H_p, of loosened rock, is strictly applicable only to rocks where this loosening can occur and is the primary contribution to the load on the support. This is usually the case in jointed rocks which are non-coherent to begin with or which lose some coherence with time. Where loosening is prevented, the rock load or loosening pressure is partly or completely eliminated.

In squeezing and swelling rock, loosening may take place but the important mechanism of behavior is of a coherent type. In sands and gravels the effects of arching and stress redistribution are pronounced and the rock load height must be considered an equivalent height.

Table II-1 shows five different rock classification systems. They have been related to each other in an approximate manner by inference from the descriptions and examples given in the references. The classes

Table II-1

Fracture Spacing — cm: 100, 50, 20, 10, 5, 2; ft-in: 2', 1', 6", 4", 2", 1"; RQD %: 98, 95, 90, 75, 50, 25, 10, 2

Weak and coherent (classes 8–10 / 7–9 / F–G and below)

TERZAGHI (1946)

	Rock Load, H_p Initial	Rock Load, H_p Final	Remarks
1 Hard and intact	0	0	Lining only if spalling or popping (Generally no side pressure)
2 Hard stratified or schistose	0	0.25 B	Spalling common (Generally no side pressure; Erratic load changes from point to point)
3 Massive, moderately jointed	0	0.5 B	Side pressure if strata inclined, some spalling (Generally no side pressure; Erratic load changes from point to point)
4 Moderately blocky and seamy	0	0.25 B to 0.35 C	(Generally no side pressure; Erratic load changes from point to point)
5 Very blocky, seamy and shattered	0 to 0.6 C	0.35 C to 1.1 C	Little or no side pressure
6 Completely crushed		1.1 C	Considerable side pressure. If seepage, continuous support.
7 Gravel and sand	0.54 C to 1.2 C	0.62 C to 1.38 C	Dense Side pressure $P_h = 0.3\gamma(0.5H_t + H_p)$
	0.94 C to 1.2 C	1.08 C to 1.38 C	Loose
8 Squeezing, moderate depth		1.1 C to 2.1 C	Heavy side pressure. Continuous support required
9 Squeezing, great depth		2.1 C to 4.5 C	
10 Swelling		up to 250' (80 m)	Use circular support, in extreme cases: yielding support

TERZAGHI (1946)
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 (~0.01 C to 0.02 C), H_{pmax} for large movements (~0.15 C).
3) B is tunnel width, C = B + H_t = width + height of tunnel. For circular tunnel, H_t = 0.

STINI (1950)

	Rock Load, H_p m	Remarks
1 Stable	0-0.5	Very little loosening
2 Nearly stable	0.5-1	Few rock falls from loosening, with time
3 Lightly broken	1-2	Loosening with time
4 Medium broken	2-4	Immediately stable, break-up after few months
5 Broken	4-10	Immediately fairly stable, later rapid break-up
6 Very broken	10-15	Loosens during excavation, local roof falls
7 Lightly squeezing	15-25	High pressures
8 Moderately squeezing	25-40	
9 Heavy squeezing	40-60	Very high pressures

STINI (1950)
Note: Loads are for 5 meter wide tunnel. For L meter wide tunnel:
$H_p = H_{p5m}(0.5 + 0.1\ L)$

BIERBAUMER (1913) and others

	Rock Load, H_p m Initial/Final	Side Pressure m Initial/Final	Invert Pressure m	
Lightly broken	0/3-4	0/0	0	Little Loosening
Very broken	3/11-13	0/1	1-2	Loosening with time
Extremely broken	5-10/11-15	2-4/2-6	4	Roof falls, loosening at time of excavation
Soft, squeezing or flowing, moderate depth	10-13/15-25	4/4	6	
Soft, heavy squeezing, great depth	15-25/47-75	8/6	12	

BIERBAUMER (1913) and others
(ref. BENDEL (1948))
Note: Originally loads were given in t/m^2.

LAUFFER (1958)

A Stable
B Unstable after long time
C Unstable after short time
D Broken
E Very broken
F Squeezing
G Heavy squeezing

LAUFFER (1958)
Note: This classification is correlated with standup time, see text

RABCEWICZ (1957)

Sound
Sound, stratified or schistose (some fissures?)
Strongly fissured
Fully mechanically disturbed
Gravel and sand
Pseudo-sound rock (properties change with time)
Some squeezing (genuine rock pressures), small overburden
Heavy squeezing, Large overburden
Swelling
Silt, clay

RABCEWICZ (1957)
Note: This classification has been used for evaluating feasibility of rock bolt types, see ref.

of jointed rocks have been correlated with average fracture spacings and the RQD. This correlation was performed primarily on the basis of work carried out at the University of Illinois (Deere et al., 1967; Coon, 1968). The correlation is approximate only, and can never be made accurate. Modification may, however, be desired as additional data and experience is obtained. Rabcewicz's classification has been related to the feasibility of various rock bolting schemes (Rabcewicz, 1957).

The description of the different rock types given by the different authors are qualitative, and in some instances typical examples of rocks falling in one group or another are given. As an important example of rock type descriptions, Terzaghi's qualitative classification is given below. This appears to be the most explicit description of rock classes. The description is slightly modified from Terzaghi's original, partly on the basis of an amplification by the California Department of Water Resources (ENR., Dec. 17, 1959).

Description of Rock Condition:

Intact rock has no joints or cracks. Breakage occurs across sound rock or is controlled by damage to the rock caused by the excavation. Spalling or popping may occur.

Massive, moderately jointed rock contains joints and cracks but individual blocks are interlocking. Spalling or popping may occur. Vertical walls need no support.

Stratified or schistose rock consists of individual strata which may separate along boundaries. If the strata are more than about 5' thick, the rock is better classified as massive or intact. If transverse joints are as close as or closer than the distance between strata boundaries, the rock is classified in the preceding group or one of the following groups. Spalling is common.

Blocky and seamy rock consists of chemically intact or nearly intact rock fragments, separated from each other by joints or other discontinuities and imperfectly interlocked. Vertical walls may require support. When individual blocks are larger than two feet, the rock is called moderately blocky and seamy; when blocks are smaller than two feet, the rock is called very blocky and seamy.

Crushed rock consists of chemically intact fragments that may be partly recemented. Individual particles are of gravel size or smaller. This material behaves much like gravel or sand.

Note that spalling refers to the falling out of individual blocks, primarily as a result of damage during excavation. Popping is the violent detachment of rock slabs from sides or roof, and is caused primarily by the overstressing of competent rock.

Terzaghi's classification of soils (and altered or weak rocks) is basically behavioristic. The version given below has been modified to include some current concepts.

Description of Soil or Rock Behavior:

Firm rock or soil is a material which will stand unsupported in a tunnel for several days or longer. The term includes a great variety of materials: sands and sand-gravels with clay binder, stiff unfissured clays at moderate depths, and massive rock.

Squeezing soil or rock advances slowly into the tunnel without perceptible volume increase. A pre-requisite for squeeze is an overstress of the material close to the tunnel opening, hence, for a given material the overburden pressure is an important parameter. Examples of squeezing media are soft to medium clays at moderate depth and clay-shales at greater depth. Salt may be considered a squeezing or creeping material at considerable depth.

Swelling soil or rock advances into the tunnel chiefly by expansion. The mechanism may be compared to the expansion of a soil in a consolidometer when load is removed. Although squeezing is often associated with swelling, overstressing of the material is not necessary for swelling to occur. Clays and clay-shales with a high plasticity index generally have a high swelling capacity.

Raveling soil or rock indicates a material which gradually breaks up into chunks, flakes, or angular fragments. The process is time dependent and materials may be classified by the rate of disintegration as slowly or rapidly raveling. For a material to be raveling it must be moderately coherent and friable or discontinuous. Examples are fine moist sand, sands and sand-gravels with some binder, stiff fissured clays, and jointed rocks.

Running ground indicates a material which will invade the tunnel until a stable slope is formed at the face. Stand-up time is zero or nearly zero. Examples are clean medium to coarse sands and gravels above ground water level. Material intermediate between running and raveling ground is termed cohesive-running.

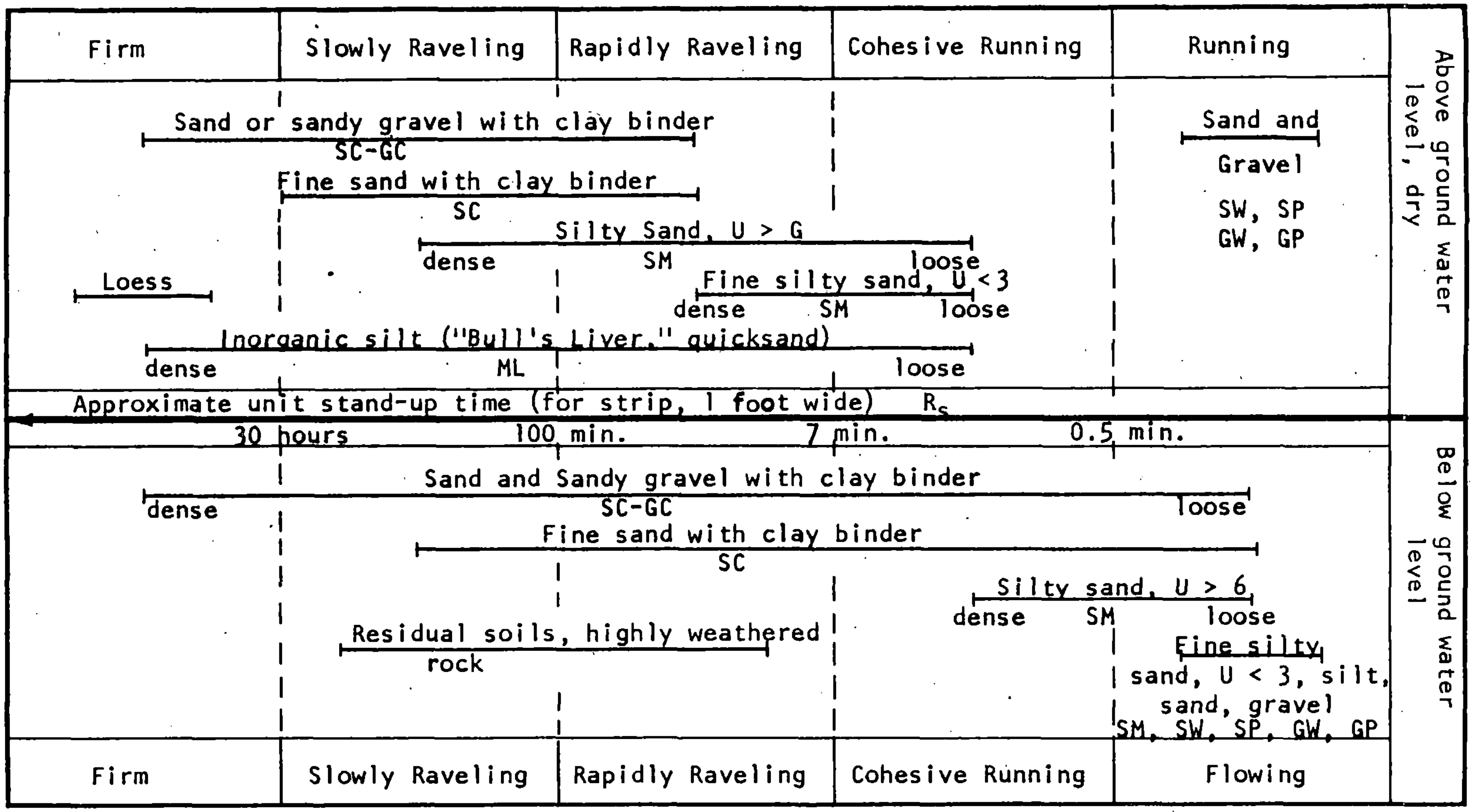
Flowing ground is a material in which water and solids together invade the tunnel from all sides, including the bottom. It is encountered in free-air tunnels below the ground water table in materials with little or no coherence. Organic silt may be flowing or squeezing.

The classifications above were developed for application primarily to the excavation of tunnels. The condition of the excavation and the methods used in the excavation process influence the action of the immediate support system. They may actually control the selection of the design and the construction procedure for the immediate support system.

Fig. II-1 shows the behavioristic classification of various soil types. The behavioristic classification is shown in relation to the primary classification on Fig. 4.3 in the main report.

II.3 The Dependence of Rock Load and Stand-Up Time on Tunnel Size

Terzaghi (1946) and Stini (1950) present linear relationships between the rock load and the size of the tunnel. These relationships are shown on Fig. II-2. The agreement between them is fairly good, especially for tunnel diameters of the order of 5 meters. Bierbaumer



Note: 1) Air loss (in tunneling under compressed air) and water inflow is governed by the permeability, largely a function of D_{10}.
2) Behavior below ground water table under suitable air pressure is approximately the same as above ground water level.
3) Loose is here defined by N < 10 (standard penetration test), dense by N > 30.
4) Descriptive terms of materials according to the Unified Soil Classification.
5) Behavior may be somewhat better than shown above ground water level, if material is moist and fine or silty.

Fig. II-1. Behavioristic Classification of Various Soils

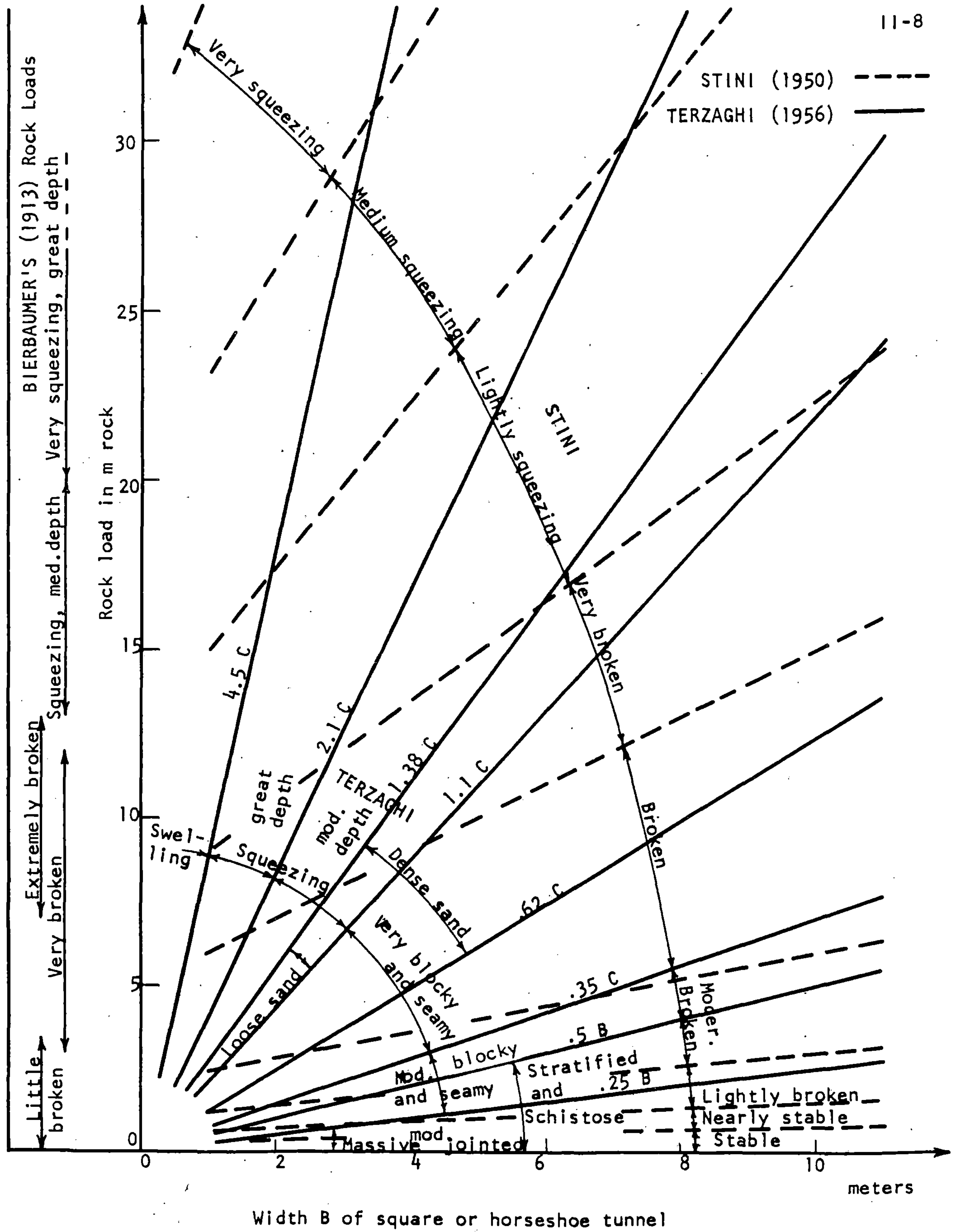


Fig. II-2. Rock Loads as a Function of Tunnel Size

(Bendel, 1948) does not give a relationship between tunnel size and rock load, but the agreement is reasonable if tunnel diameters of the order of 5 meters are considered.

The time dependence of the behavior of rock is indicated by initial and final values in Table II-1. Often the rock is stable or will exert only a small load, the initial load, immediately after the excavation. Continuous loosening and plastic deformation will increase the load on the supports to the final value in a matter of days or months. For sands and gravels Terzaghi indicates an increase in load of 15 pervent, but for jointed or crushed rocks the increase can be much greater because the initial load may be small. The increase in load with time is greatly influenced by the amount of damage caused by the excavation process and by the amount of loosening allowed. Thus, modern methods of tunneling and support which disturb the rock less and allow less loosening will result in smaller rock loads than those given here, which are based on conventional tunneling procedures of 20 years ago.

The time dependent behavior is also manifested by the stand-up time or bridge action period (Terzaghi, 1950 and 1946), which is defined as the time elapsing between the excavation and the first perceptible movement of the roof. The stand-up time is a function of the rock type, the unsupported length or width of the tunnel, and the amount of disturbance of the material during excavation. Lauffer correlated his rock classes A to G with stand-up time and unsupported width. For a given rock the stand-up time t_s is given as a function of L, the unsupported length, by

$$t_s = \text{constant} \times L^{-(1+\alpha)}$$

where α varies from 0 for class A to 1 for class G rock. The relationships are given in Table II-2 and in Fig. II-3.

Table II-2

Rock Class	Example t_b	L_m	Equation
A	20 years	4	$t_s = 1 \times 10^5 \; L^{-1}$
B	6 months	4	$t_s = 2.5 \times 10^3 \; L^{-1.2}$
C	1 week	3	$t_s = 6.3 \times 10 \; L^{-1.4}$
D	5 hours	1.5	$t_s = 1.6 \; L^{-1.6}$
E	20 min.	0.8	$t_s = 4 \times 10^{-2} \; L^{-1.8}$
F	2 min.	0.4	$t_s = 1 \times 10^{-3} \; L^{-2}$
G	10 sec.	0.15	

t_s in hours, L in m

Terzaghi (1948) indicates that for a raveling material the stand-up time t_s of an unsupported strip is inversely proportional to its width L, and that the stand-up time for a square opening with side L is of the order of 50 percent longer than for a strip of width L. For an elongate opening, the relationship between t_s and L is

$$t_s = R_s \, L^{-1},$$

where R_s is the stand-up time for a one foot wide strip. The following classification of running and raveling ground is given:

Running: $R_s = 0$

Cohesive running: $R_s < 7$ min

Rapid raveling: $R_s = 7$ min to 100 min

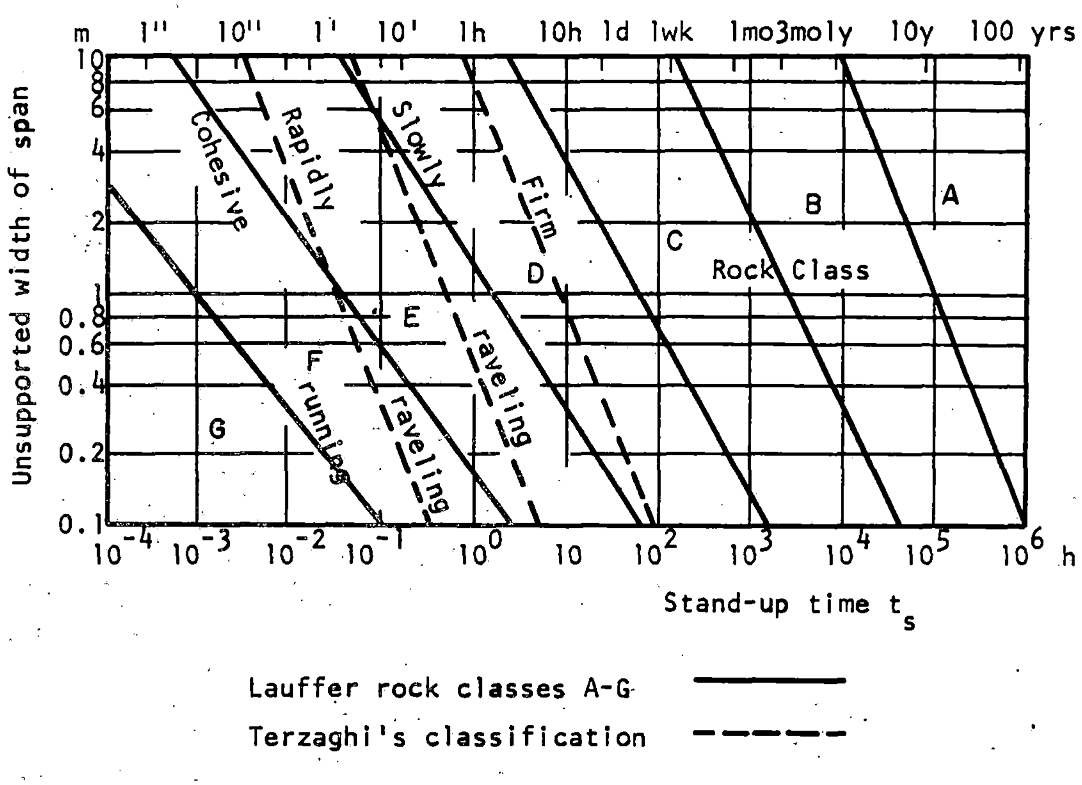
Slow raveling: $R_s = 100$ min to 30 hours

Firm: $R_s > 30$ hours

Terzaghi's and Lauffer's numbers cannot be compared directly because the power of L is different. If, however, they are compared at the same values of L, say L = 5 to 10 feet, it is found that, roughly speaking, Lauffer's class F would be cohesive running, class E rapid to slow raveling, class D slow raveling to firm, and class C firm. Terzaghi's classifications are also indicated on Fig. II-3.

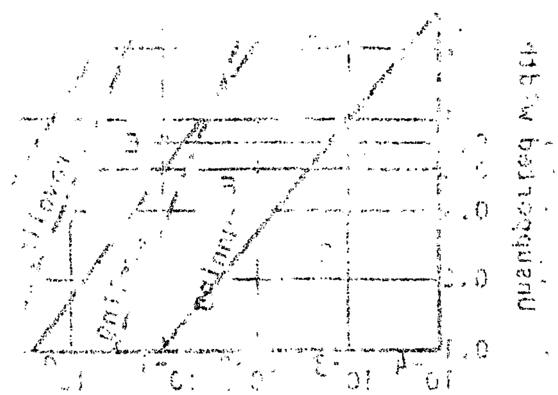
There are several objections to Lauffer's classification. There does not appear to be any justification for the varying powers of L. The utility of the classification is limited and doubtful for the high-quality rocks, where stand-up time may in effect be infinity.

The term stand-up time was applied by Terzaghi primarily to non-coherent rocks. If the rock is coherent the term loses most of its significance. Although movements of the roof may be perceived after a short time, this does not necessarily indicate an immediate stability problem. The most important behavioristic character of a coherent (squeezing or swelling) material is the rate of deformation. This is roughly proportional to the size of the opening and controlled by the ratio of the shear strength to the stress level, and the swelling potential.



Modified from Lauffer (1958)

Fig. 11-3. Stand-up Time as a Function of Rock Class and Unsupported Width of Tunnel Roof.



Laufer ... Lauffer ...

Terzaghi ...

Fig. 11-3. Standup Time ...
Unsupported ...