SWEDISH GEOTECHNICAL INSTITUTE

PROCEEDINGS
No. 27

CORRELATIONS OF ROCK BOLT-SHOTCRETE SUPPORT AND ROCK QUALITY PARAMETERS IN SCANDINAVIAN TUNNELS

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PREFACE

In its Reprints and Preliminary Reports No. 40, 1971 the Swedish Geotechnical Institute presented a paper by Mr Owen S. Cecil with the title "Correlation of Seismic Fraction Velocities and Rock Support Requirements in Swedish Tunnels". This paper describes part of a large work performed by the Author during 1966-68 when he was employed at the Institute. Parts of the work have also been published in IVA-rapport No. 4, 1968 ("Evaluation of visual rock classification systems for tunnel construction in Sweden") and in ASCE Civil Engineering 1970:1 ("Shotcrete support in rock tunnels in Scandinavia").

The main work comprises field studies of fourteen different underground rock construction projects in Sweden and Norway in order to provide an understanding of the nature and causes of instability in rock tunnels and rooms in Scandinavia. The material from these field studies has been used to evaluate the usefulness of visual rock classification systems for the assessment of the stability behavior and reinforcement requirements in underground rock construction in Sweden.

During 1968-70 Mr Cecil completed his work at the University of Illinois, U.S.A. with, e.g. laboratory model studies. The whole work resulted in a Ph.D. thesis with Professor Don Deere as adviser.

The field investigations were supported by grants from the Swedish Power Board (Statens Vattenfallsverk) and the Swedish Fortifications Administration (Kungl. Fortifikationsförvaltningen) and done in cooperation with, at that time called, the Rock Mechanics Committee of the Swedish Academy of Engineering Sciences (IVA).

The Author has kindly put his original figures etc to the Institute's disposal. The text is taken in its original form but retyped. The editorial work has been done by Mr Olle Holmquist and Mr Nils Flodin of the Institute.

The Institute thanks Dr Owen Cecil for his comprehensive work and believes that it will be of a great value for a better understanding of many problems in rock mechanics, especially those associated with tunnelling.

Stockholm, May 1975
SWEDISH GEOTECHNICAL INSTITUTE

CONTENTS

SUMMARY

Field observations at 14 civil engineering rock tunnel projects in Norway and Sweden have enabled empirical correlations to be drawn between three different rock quality parameters and the rock bolt-shotcrete supports used in loosening ground conditions. The three rock quality parameters used in the investigation are average discontinuity spacing, rock quality designation (RQD), and seismic velocity ratio. Numerical values of each of these three parameters have been related to the three support classifications of maximum (two or more shotcrete applications, frequently with closely spaced rock bolts), intermediate (one shotcrete application, frequently with medium to widely spaced bolts), and minimum (none or medium to widely spaced bolts). These correlations offer the most realistic approach to the selection of a shotcrete design that has heretofore been possible.

Laboratory model studies have been used to demonstrate the significance of joint orientation and tangential stress on the stability of an unsupported jointed medium. Both the failure mechanism and the mechanism of stabilization of an unsupported span have been described. The influence of intact material failures on the failure mechanism is particularly noteworthy.

Several simple rigid block analytical models have been used to demonstrate possible shotcrete-rock interactions. They point out the importance of the rock-shotcrete bond strength in determining the support capacity of a discontinuous shotcrete tunnel lining.

1. INTRODUCTION

1.1 Statement of Problem

When a practicing engineer undertakes a problem that is new to him, or one that is relatively new to the entire profession, one of his major concerns is to learn about the experience that others have had with the same problem. Until a subject or problem becomes of such widespread significance and interest that practices and experiences begin to appear in the professional literature, an engineer may have extreme difficulty in learning what is already known or what is being done by others in his particular area of interest.

The support of underground openings in rock, particularly through the application of shotcrete and rock bolts, is a subject that lies in that gray area of knowledge described in the preceeding paragraph. The use of shotcrete in the support of underground openings is no more than 15 years old in those parts of Europe where it was first introduced. There are no more than a dozen projects on the North American continent where shotcrete has been used extensively as underground support in rock tunnels. It is thus not too surprising that few engineers are well acquinted with its application in this area.

Although there are certain problems associated with the pure mechanical process of placing shotcrete on the walls and roof of a tunnel, the least understood problem in its application is the engineering design of shotcrete tunnel linings. There are three aspects to the development of engineering design procedures. First, it is necessary to understand the stability behavior of unsupported or unreinforced tunnels in order that the likely mode of failure is known and understood. Second, it is necessary to understand the rock-shotcrete interaction, that is, the manner in which shotcrete provides support in an unstable tunnel. Third, it is necessary to understand the conditions for which shotcrete has been used, or more precisely, the relationship between the behaviour of different shotcreted tunnels and the nature of the materials through which the tunnels are driven.

The purpose of this thesis is to provide insight into these three aspects of shotcrete design and to formulate preliminary design concepts based on practical experience. Although the results of the work are subject to modification as more experience is accumulated, they do provide a better insight into the problem and a more realistic treatment of practical experience than is presently available.

1.2 Approach

The approach taken to the stated problem involves three phases of investigation: (1) field observations in shotcreted tunnels; (2) laboratory model studies of unsupported openings in jointed rock; and (3) analytical models of the rock-shotcrete interaction.

Emphasis is necessarily placed on the field observations in order that real tunnel behavior can be understood and actual shotcrete practices can be observed. The largest part of the field observations consists of determinations of various rock quality parameters. Rock quality designation (RQD), average discontinuity spacing, and other rock mass structural properties were determined in over 90 individual cases. In a few cases, these data were supplemented with seismic refraction velocity measurements.

Although the value of borehole extensometer measurements for evaluation of the general stability of an opening in rock is recognized (Cording, 1968a), and would undoubtedly aid in the interpretation of the rock-shotcrete system behavior, the use of such instrumentation in Swedish and Norwegian tunnels is very seldom warranted because of the low percentage of tunnels that require support (less than 20%). Because of the very wide scattering of instability cases over many kilometers of stable, unsupported tunnel, the placement of instruments to measure rock mass properties and the behavior of the rock mass in unstable areas would have required much more detailed preliminary exploration for potentially unstable zones than is normally done. Furthermore, the placement of instruments in unstable zones would have required continual inspection of a tunnel face throughout the duration of a project. Rather than concentrate all attention in one tunnel and attempt to quantitatively monitor the behavior of a few instability cases, it was decided to study a large number of cases in many different tunnels and limit the observations to determinations of rock quality.

Although the field observations provide information that is indispensable in understanding the gross behavior of real tunnels, they do not provide a complete explanation of the machanical behavior of a tunnel and the mechanism of tunnel failure. It is essential that these facets of the problem be understood if the field observations are to be interpreted in the correct manner. For these reasons, a laboratory model of a simple, hypothetical unsupported span in jointed rock has been constructed. The model has made it possible to study qualitatively the effects of variations in different rock mass parameters on the stability of openings in jointed rock.

The analytical models of the rock-shotcrete interaction are simplified analyses that are intended to show the possible mechanisms by which shotcrete renders support in an unstable opening. They are not intended as design procedures. However, they do point out significant modes of behavior that heretofore have not been considered.

Because the primary aim of this thesis is to relate rock bolt-shotcrete tunnel supports to some simple measureable rock quality parameters, the methods used in the analysis of the field observations are necessarily of an empirical nature. An empirical approach is justified in consideration of the complexity of the problem and the lack of any practical theoretical approach. Rather than approach the problem from a theoretical viewpoint and attempt to consider all the factors that influence the support requirements in a shotcreted tunnel, the writer has chosen to pursue the empirical approach and attempt to explain any anomalous behavior or conditions for which the empirically derived relationships do not apply. The laboratory and analytical model studies have been very valuable in this respect.

It is believed that the approach outlined in the previous paragraphs can provide the most useful tools for design engineers that wish to use the experience of others in their work. This approach is even further justified in consideration of the fact that tunnels of all types are currently designed almost solely on the basis of experience. Because the application of shotcrete and rock bolts to underground support is currently a practice that is surrounded by much mystery and doubt, any systematic collection of experience would be a welcome contribution to most engineers.

1.3 Scope of Work

The scope of this thesis is limited to the behaviour of tunnels and the use of shotcrete in the rock conditions encountered on the Scandinavian peninsula. This area, particularly Sweden, deserves special consideration because of the wealth of experience that has been gained in the area of shotcrete and rock bolt support of tunnels in rock. Although the geologic conditions in Scandinavia are unique to only a few parts of the world, the experience accumulated in that area is broad enough that it can benefit others in different geologic areas, particularly if the tunnel behavior and experience are related to fundamental, measureable parameters, as is attempted in this thesis.

The field observations that constitute the greater part of the thesis were made in 14 different underground rock projects in Sweden and Norway. All of the observations were made during the construction or repair stage of tunneling when the possibilities for close inspection of rock conditions were best. Because construction practices have an influence on the behavior of tunnels, they are discussed in detail, and, along with several geologic factors, must be taken into consideration in any attempt to extrapolate the results of this thesis to other areas.

The correlations between tunnel support and rock quality parameters that have been derived from the field observations are by no means perfect, but they do indicate strong trends in collected experience and offer a much more realistic approach to shotcrete design than has heretofore been possible.

The field observations are presented and discussed in Chapter 2. Chapter 3 deals with the nature and causes of the observed instability. Material from both the laboratory model studies and the field observations is used in the discussion of this subject. Chapter 4 is devoted to the discussion of Swedish tunneling practices and their influence on tunnel stability. The role of shotcrete as a rock reinforcement is discussed with the aid of several simple analytical models. The empirical relationships between different rock quality parameters and the rock bolt-shotcrete supports used in tunnels are dealt with in Chapter 5. The potential for the practical application of the support-rock quality relationships is discussed and the needs for trial testing are explained.

2. FIELD STUDIES

2.1 Introduction

The field studies discussed in this chapter are the results of 15 months of field work in Swedish and Norwegian underground rock construction projects. In an attempt to familiarize himself with general Swedish underground rock construction practices and the specific nature and causes of instability and other tunneling problems, the writer undertook extensive inspection of a wide variety of projects during the period September 1966 - March 1968.

The observations were carried out during the construction or repair stage in a total of 14 different projects. The magnitude of the projects varies from short lengths (less than 1 km) of small-diameter water, sewer, and utility tunnels to complex hydroelectric schemes that include large underground machine halls and many kilometers of large-diameter water conveyance tunnels.

The projects include ten hydroelectric schemes, one railroad tunnel, one subway tunnel, one underground sewage treatment system, and a large wine and liquor storage facility. No mines were included in the studies.

The 14 projects and the types of tunnels for each project in which observations were made are listed in Table 2.1. The lengths given in the last column do not correspond to the total lengths of tunnels for each project, but rather only to the lengths available for inspection at the time of the field work. Considerable lengths of planned tunnel at a number of the projects could not be included in the studies, either because they had not yet been driven or because they had been driven and completely shotcreted so that observation of rock conditions was not possible.

A total of 13 underground rooms or chambers and about 67 kilometers of tunnel was inspected. The cross sectional areas of the inspected openings vary from seven square meters (75 sq ft) to 440 square meters (4840 sq ft). Span widths vary from 3.4 meters to 20 meters. Cross-sections of some of the underground openings are shown to scale in Figure 2.1. The depth of soil and rock cover in all of the projects is less than 300 m, and in most cases less than 100 m.

2.2 Geographic and Physiographic Setting

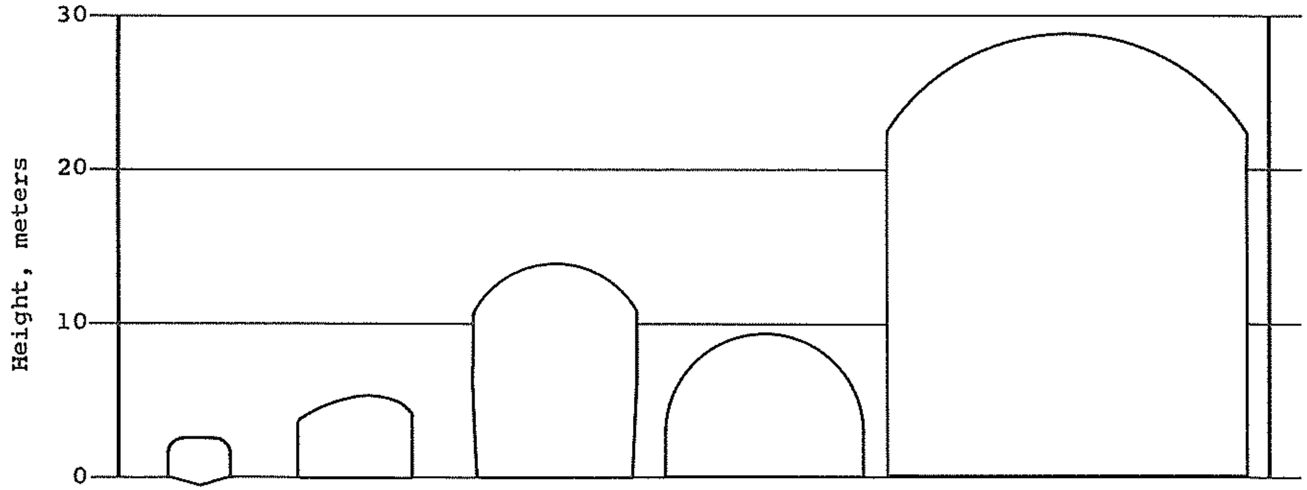
The locations of the projects are shown on the map in Figure 2.2. Eleven of the projects are located in Sweden and three in Norway. The numbers on the arrows correspond to the numbers in Table 2.1.

Three of the projects (9, 10, 11) lie in or around Stockholm in a physiographic region known as the Swedish central lowlands. The Lieråsen project near Drammen, Norway, is also located in a relatively low coastal area. The low mountainous terrain in that area, however, is in marked contrast to the flat plain around Stockholm. The other eleven projects are located on the main highland mass of the Scandinavian peninsula. Seven of the projects in this region are located on Swedish rivers that flow southeasterly to

TABLE 2.1

INSPECTED UNDERGROUND OPENINGS

Project	Type of Tunnel or Room	Cross-Sectional Area, m^2	Length, m
1. Seitevare Hydroelectric, Sweden	tailrace tunnel	98	5300
	machine hall	~ 130	40
	access tunnels	12-44	~ 1000
2. Vietas Hydroelectric, Sweden	headrace heading (Suorva)	68	3000
	headrace (Satisjaure)	80	600
	access tunnels	25-80	~ 1000
	tailrace tunnel	~ 100	500
3. Rendal Hydroelectric, Norway	headrace tunnel	43	3000
	access tunnels	50	1000
4. Sällsjö Hydroelectric, Sweden	tailrace tunnel	64	9000
5. Bergvattnet Hydroelectric, Sweden	collector tunnels	18	4000
	tailrace tunnel	30	5000
	access tunnels		1000
	machine hall	~ 60	20
6. Stensjöfallet Hydroelectric, Sweden	headrace tunnel	24	5000
	tailrace tunnel	24	3000
	access tunnels	20-24	1500
	machine hall	~ 100	30
7. Mo i Rana Hydroelectric, Norway	collector tunnels	19-39	10000
	machine hall	~ 300	~ 70
8. Lieråsen, Norway	double track railroad tunnel	60	5000
9. Käppala Sewage Treatment Works, Sweden	sedimentation chambers	116	1600
	collector tunnels	7	1000
	access tunnels	70	300
10. Stockholm Subway, Sweden	twin track subway tunnel	40	300
11. Årstadal, Sweden	underground storage rooms (two)	440	300
12. Rätan Hydroelectric, Sweden	tailrace tunnel	80	2000
	machine hall	300	60
	access tunnel	20	500
13. Letsi Hydroelectric, Sweden	access tunnels	~ 80	1000
14. Dabbsjö Hydroelectric, Sweden	intake and access tunnels	20-80	1000
			67120 m



Project	Käppala	Bergvattnet	Seitevare	Rätan	Årstadal
Span width	3.4m	6.5m	9m	11.25m	20m
Area	8m^2	30m^2	98m^2	80m^2	440m^2

Fig. 2.1 Cross-sections of some inspected openings

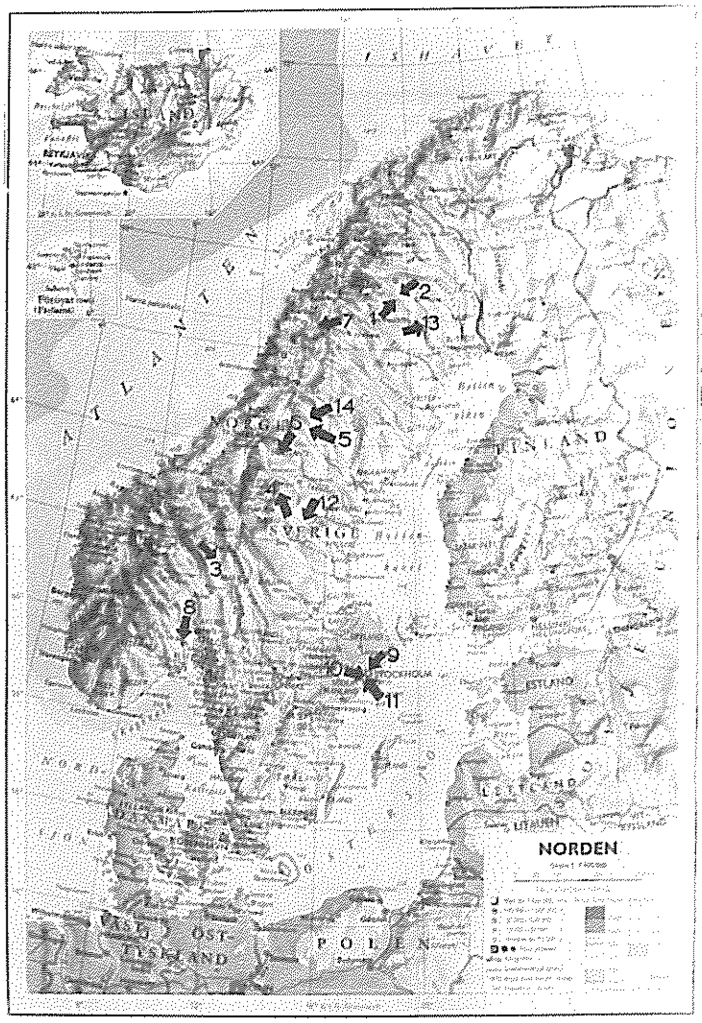


Fig. 2.2 Location map for projects

the Baltic Sea. Two of the projects (6, 7) are located on the so-called "keel", a ridgelike mountain chain in northern Scandinavia that corresponds approximately to the boundary between Norway and Sweden. The Rendal project is located in a graben valley that drains the east-central Norwegian mountain plateau.

The large concentration of projects in northwestern Sweden is the result of two factors. First, hydroelectric projects involve the greatest mileage of Swedish civil engineering tunnel construction and most of the remaining undeveloped river heads in Sweden are in the headwaters near the mountains. Second, the bedrock in and near the mountains of Sweden generally presents more tunnel stability difficulties than that of other parts of Sweden where construction was current during the writer's studies.

The three projects in Stockholm were included in the studies because of their representation of different bedrock conditions and their unusual size. The Norwegian projects were included because of their representation of different bedrock conditions and their severity of stability difficulties

2.3 Geologic Background

General

The behavior of a structure in rock depends to a great extent on the nature of the geologic environment, or more specifically on the properties of the surrounding rock mass.

Although geology alone usually does not provide quantitative information about the properties of a rock mass, it does convey very valuable qualitative information that many times implies a certain behavior spectrum and a specific set of possible problems. Furthermore, if it is desired to utilize past experience from particular construction projects, it is necessary to have a full understanding of the geologic environments of both the planned project and the previous project from which the experience is to be drawn. This section is an attempt to fill these needs.

The work in this thesis is admittedly confined to a rather narrow geologic environment. Only parts of Canada, the British Isles, Finland, and Russia have areas that are geologically similar to Sweden and Norway. However, there are many areas where the condition of the bedrock, irrespective of its age and history, is similar enough to warrant consideration for the application of Scandinavian tunneling experience. In this section the rock types and historical and structural geology are discussed. The material presented in Appendix B is oriented in more detail towards the specific geologic factors associated with instability and should aid considerably in determining the applicability of the reported results to other geologic invironments.

Bedrock Materials

The rock types encountered in the 14 projects include: gneiss, coarse-grained granite, fine-grained aplitic granite, diabase, amphibolite, diorite, several types of schist (graphite, mica, chlorite, alum), leptite, marble, quartzite, mylonite, sparagmite (metamorphosed arkose), metamorphosed greywacke, and metamorphosed claystone. An extremely wide variation in lithologies, even for one general rock type such as granite, was found in the projects and, together with varying grades of metamorphism, account for a wide range of intact rock properties.

Because the intact rock properties are of consequence in only two isolated cases, which were not included in the studies, no attempt was made to determine intact rock properties by means of laboratory testing. It is estimated that the range of compressive strengths for all the rock types is from about 10,000 psi for the weaker sparagmites and leptites to over 35,000 psi for certain diabases and quartzites. The significant property of these rocks, with the exception of the two previously mentioned cases, is that their strengths are high enough so that the rock mass behavior is governed primarily by the structural discontinuities of the mass.

Although most of the instability cases arose in sound, unaltered, jointed rock, some of the problems could be attributed to alterations of rock materials, particularly along joints and shear zones. Some extremely altered and weathered rocks with unconfined compressive strengths as low as 75 psi were encountered in limited amounts. However, the low compressive strength of these materials was not the main problem; the difficulties caused by them arose out of the physical disintegration and chemical changes that occurred upon exposure to air, water, and frost action. An amphibolite that was chemically altered to a moist sand-like condition and several granite and sparagmite masses whose feldspars were chemically altered to montmorillonitic clay products were among the more noteworthy examples of altered rock materials that caused tunnel stability difficulties. Alterations around joints and mechanical deterioration by shear movements and weakening by various joint fillings are common. The effects of such rock alterations will be discussed in detail in a later chapter.

Historical and Structural Geology

Simplified geologic maps of Sweden and Norway are shown in Figures 2.3 and 2.4. The bedrock of the Scandinavian peninsula can be classified into the following four major groups:

(a) Precambrian basement rocks (Swedish "urberg"- - "primitive rock") that form most of the eastern portion of the peninsula (Sweden) and a large portion of southern and northwestern Norway.
(b) Folded and thrusted rocks (most of which are Cambrian to Silurian in age) in the overthrusted mountain ranges that form the backbone or keel of the high Scandinavian plateau.
(c) Lesser amounts of Cambrian to Silurian sedimentary rocks that cover the southern tip of Sweden and parts of southern Norway, and
(d) a group of eruptive and sedimentary rocks of Permian age that lies in the vicinity of the Oslo fjord.

These four bedrock units are indicated on the geologic maps in Figures 2.3 and 2.4 The locations of the 14 projects are shown with black arrows.

Projects in the Precambrian Regions. The greatest part of the Swedish bedrock consists of Precambrian igneous and metamorphic rocks. Six of the Swedish projects (1, 9, 10, 11, 12, 13) are located entirely in the Precambrian bedrock. The three projects near Stockholm (9, 10, 11) are founded in Archean granites and gneisses that form part of the roots of the 2-billion-year-old Svecofennian geosynclinal mountain range, which of course has since been eroded away. Normal faulting is prominent in the Archean rocks in the Stockholm area.

The Rätan project (12) is founded in somewhat younger Proterozoic granites (both coarse- and fine-grained) and diabases. The amphibolite inclusions at Rätan are believed to be older than the Rätan granite.

Both the Letsi and Seitevare projects (1, 13) lie in Precambrian granites that are believed to be closely related to the same orogenic developments that formed the Stockholm granite and gneiss rocks. The leptite at the Seitevare project is an inclusion of older volcanic sediments that apparently was metamorphosed and partially melted during the formation of the granite.

Although the Precambrian rocks are generally very sound and unaltered and give few problems in underground construction, a few noteworthy examples of tunneling difficulties in the Swedish Precambrian do exist. Rock conditions at the Höljes project, parts of which are included in this work as a literature study, are about the most troublesome ever encountered in Swedish tunneling history. The rock at the project consists of hydrothermally altered sericite schists and amphibolite (Karlsson and Fryk, 1961). The Bergeforsens hydroelectric project, also located in the Precambrian bedrock, involved extensive reinforcement and rock treatment in connection with a network of volcanic dikes along which extensive hydrothermal alteration has taken place. Alterations of the alkaline and carbonate material in the dikes and the swelling of montmorillonitic alteration products in the surrounding gneissic granite country rock caused the greatest difficulties (Sällström, 1967).

The Rätan project included in this work passes through a zone of thrust faults in the Precambrian rock and is one of the most heavily reinforced tunnels in Swedish hydroelectric tunneling history. A less extensive thrust fault in the Precambrian bedrock is also responsible for the most troublesome conditions at the Seitevare project.

Projects in the Overthrusted Mountain Region. The

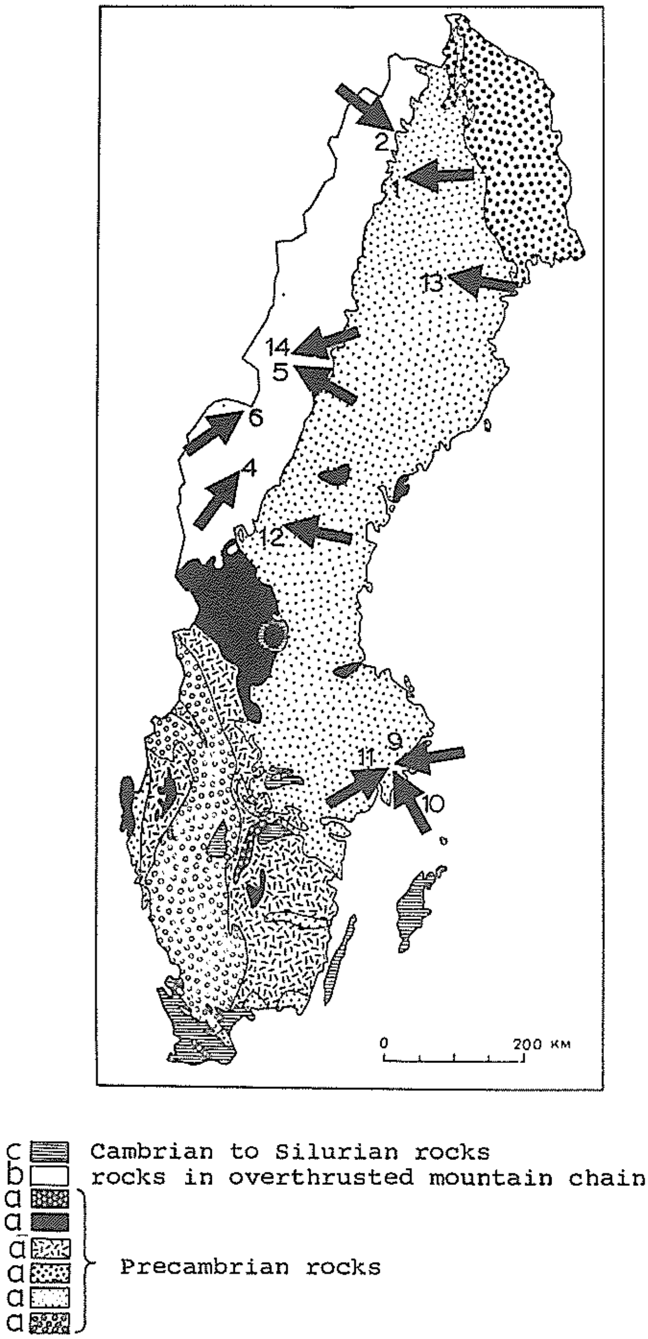


Fig. 2.3 Geologic map of Sweden
(after Lundegårdh et al., 1964)

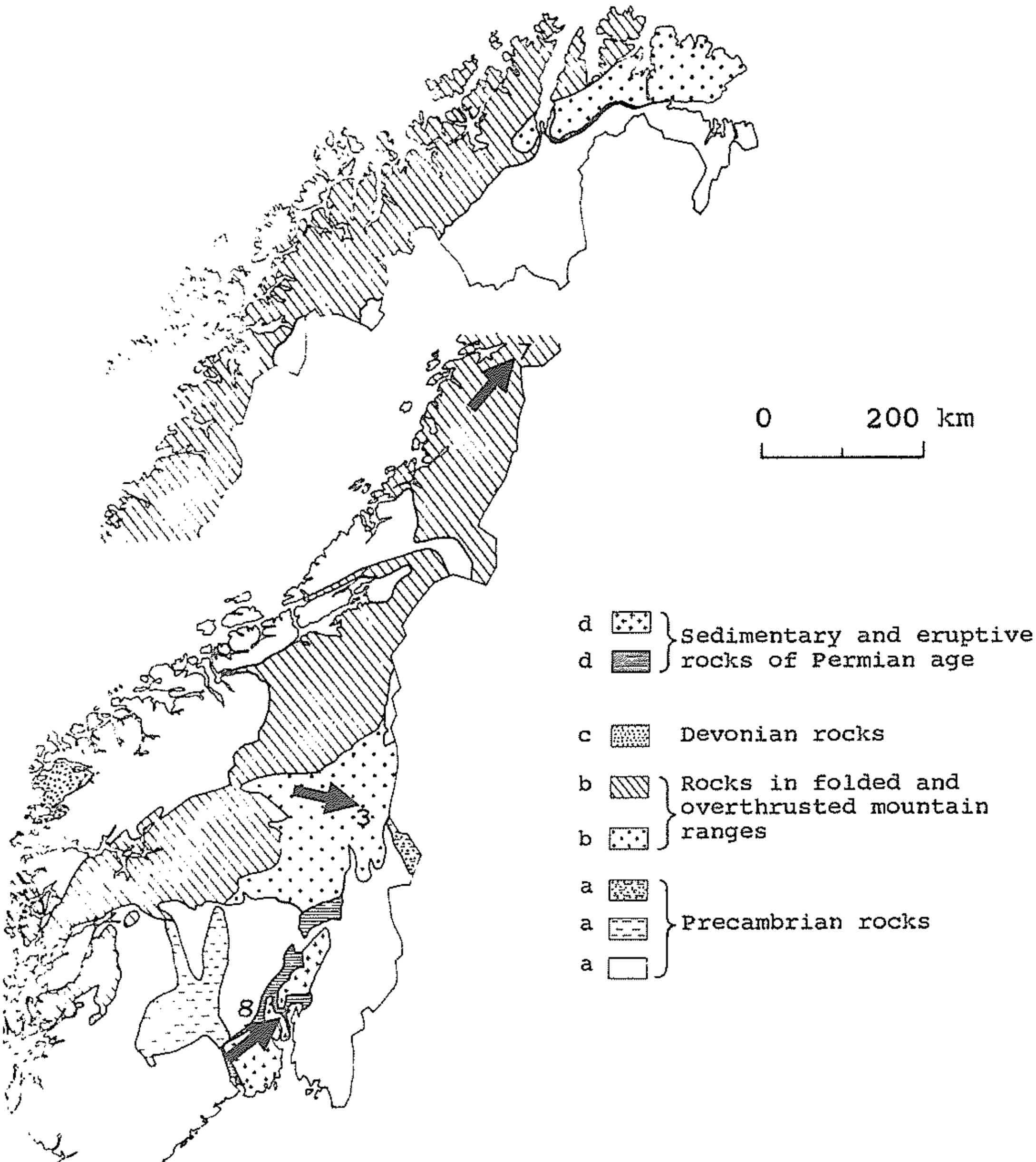


Fig. 2.4 Geologic map of Norway (after Selmer-Olsen, 1966)

mountain ranges along the northern Norwegian-Swedish border are a series of overthrusted nappes that have been displaced laterally as much as 100 km. (Lundegårdh et al., 1964). The extent of the overthrusting is indicated on the geologic maps of Sweden and Norway by unit b. The ages of these overthrusted rocks range from late Precambrian to Silurian.

Three distinct major overthrust sheets have been recognized in northwestern Sweden, but there exist many minor low-angle thrust faults within the major sheets and even in the Precambrian basement rock at the edges of the mountains. This faulting took place at the same time that the geosynclinal area along Norway's west coast was folded and uplifted. The period of tectonic activity is known geologically as the Caledonian orogeny, and is believed to have occurred at the close of the Silurian period. The tectonic activity that occurred at that time is responsible for the present structure of the Scandinavian peninsula.

The tailrace tunnel of the Sällsjö project (4) lies completely within the lower thrust sheet that is composed of Cambrian-Silurian schists and metagreywackes in this area. The tunnels at the Stensjöfallet project pass from the Precambrian basement granite (headrace, machine hall, portion of tailrace) through thin zones of quartzite and alum schist at an overthrust fault and into the metagreywacke overthrust sheet that is the same tectonic body as that in which the Sällsjö tunnel is located. The sparagmite mass in which the Rendal project (3) is located is a large plate that was overthrusted during the Caledonian, although the dominant local structure and topography are more strongly influenced by later normal faulting. The rock is of late Precambrian or early Cambrian age, the overthrusting is of Caledonian origin, and the normal faulting probably occurred during the same Permian faulting that disrupted the Oslo area.

The Bergvattnet project (5) is of particular geologic interest, as the tunnels at that site pass through parts of all three major overthrust sheets. The simplified geology is shown schematically in Figure 2.5. The Dabbsjö project, which was just started at the time of inspection, lies in the lower overthrust sheet.

Tunnels inspected at the Mo i Rana project (7) in Norway are located in schists in the highest mountains of the upper overthrust sheet, the so-called Rödingsfjäll nappe.

At the Vietas project (2) in northern Sweden the headrace tunnels pass through mylonite in the middle overthrust sheet, schist in the lower sheet, and quartzite in the Precambrian basement rock. The geology is shown schematically in Figure 2.6.

Oslo Eruptive Field. To the west and north of Oslo lies a relatively more recent (Permian) area of tectonic activity. The normal faulting of the Cambrian-Silurian bedrock in this area was accompanied by volcanic activity that was responsible for the formation of both extrusive and intrusive rocks (Selmer-Olsen, 1966). The Lieråsen project (8) in this region is located in the Drammen granite, which in recent construction has been found to contain frequent montmorillonitic feldspar alterations of the intact rock and along joints and shear zones (Huseby, 1966).

2.4 Field Study Procedures

The general procedure used in the field work was to first study the design drawings and preliminary geologic investigations in the engineering offices of either the project owner or designer. The purpose of this work was to become familiar with the general layout of the projects and to determine in which areas rock tunneling problems were most likely to be encountered. Personal communications with design and owner personnel were sometimes useful in this respect, as these people frequently had studied all of the preliminary investigation material and were in close contact with site personnel. Also of assistance were topographic sheets, seismic refraction surveys (for both bedrock surface contours and bedrock seismic velocities), geologic maps, and geologic reports describing field mapping of outcrops and diamond drill cores.

Site inspections were made at all of the projects listed in Table 2.1. Some of the projects were studied in more detail than others. The case reported for the subway in Stockholm represents about one-

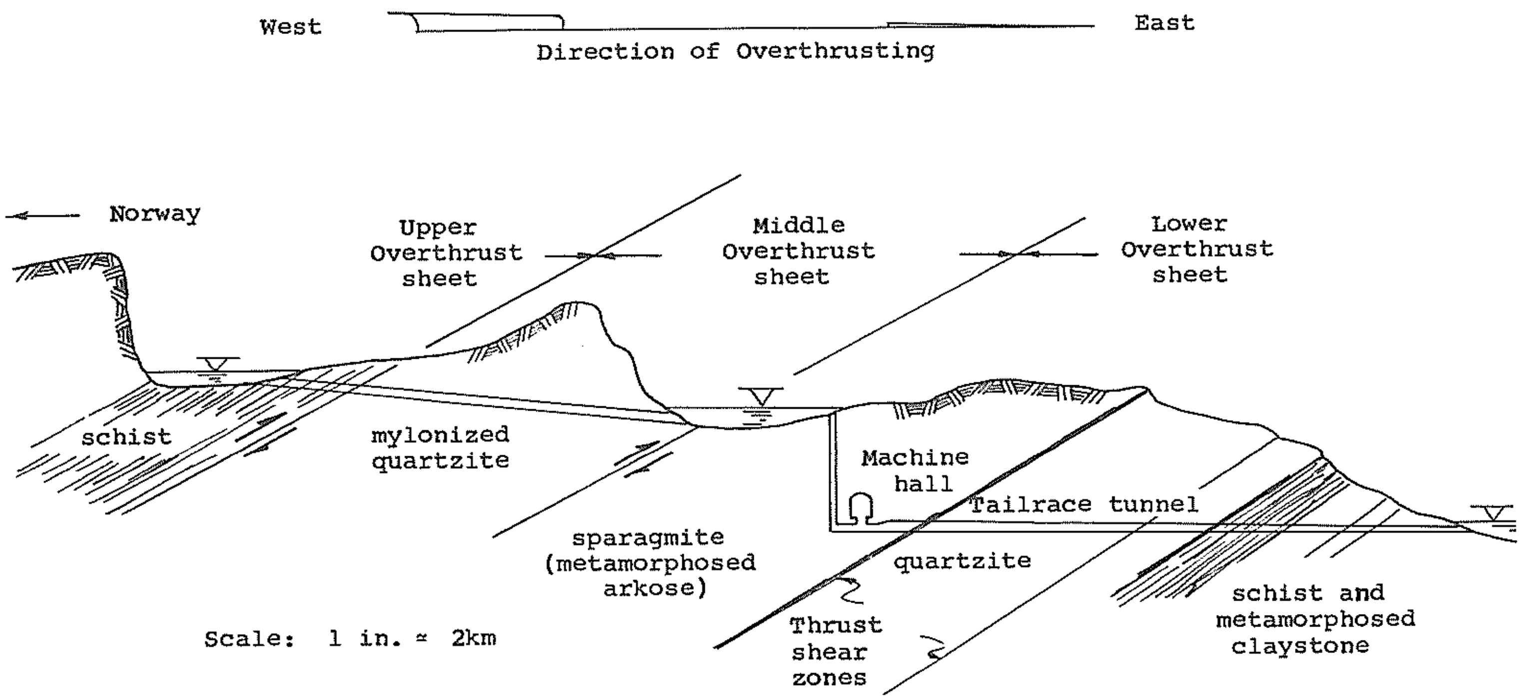


Fig. 2.5 Simplified schematic diagram of geology at Bergvattnet

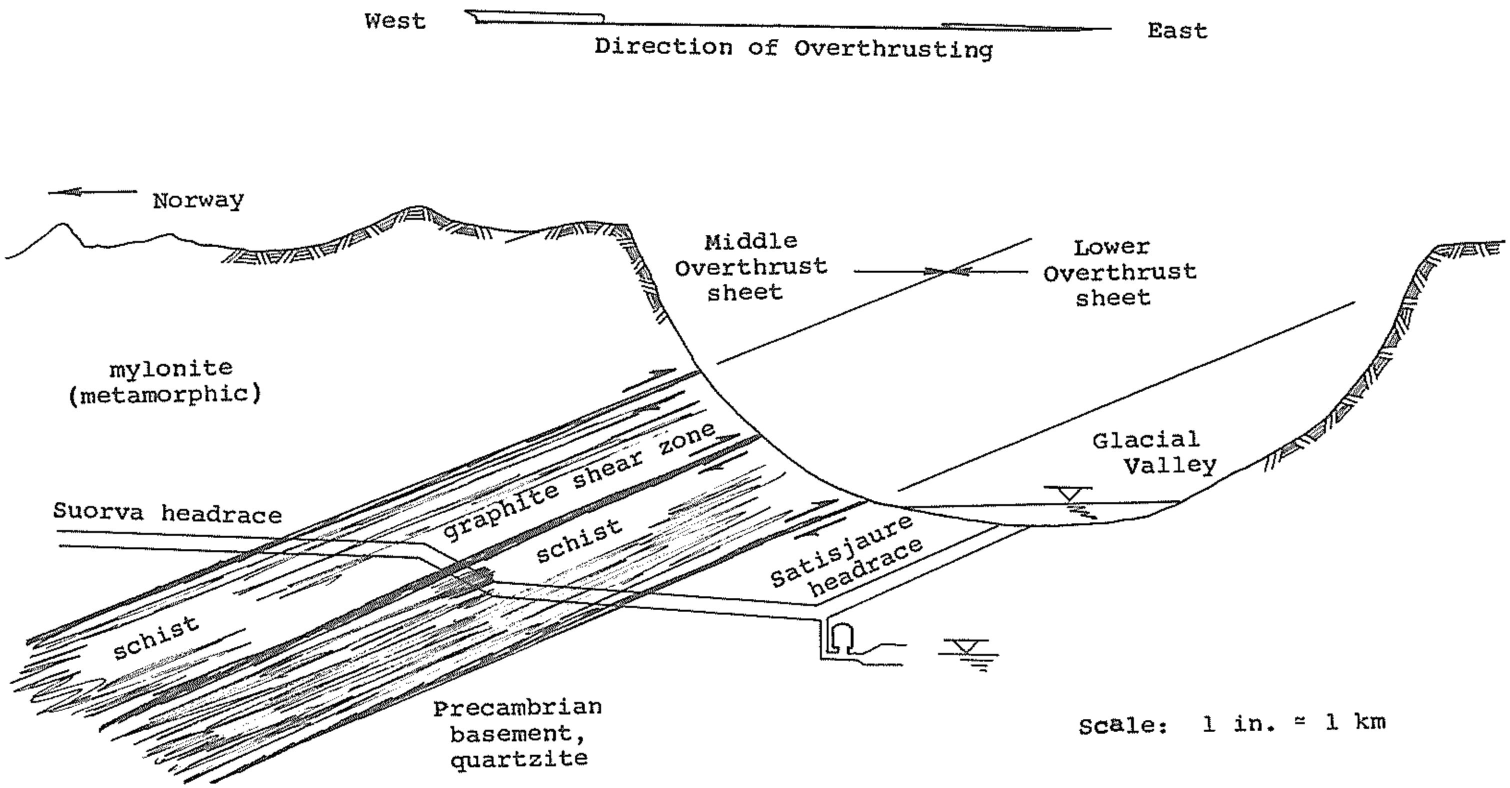


Fig. 2.6 Schematic diagram of geology of Vietas

hour observation and discussion with project personnel. Inspections of the Letsi project were limited to the access tunnels and involved only two hours. A total of about one month was devoted to observations at Rätan, and over three months at Seitevare, where the entire tailrace tunnel was inspected during the blasting of the bench.

At many of the projects several different inspection trips were made so that the tunnel headings could be observed in different rock conditions. In the poorest of rock conditions, which required shotcreting immediately after blasting, it was necessary to inspect the tunnels on a round-by-round basis so that the rock conditions could be observed and recorded prior to shotcreting.

At the first site inspections, after general project briefings and tours were completed, inquiries were directed specifically at tunneling methods, rock conditions encountered during tunneling, and reinforcement or support measures used in areas of unstable rock. Although a great deal of information often was obtained from project superintendents, tunneling foremen, and miners, actual inspection of the tunnels and headings provided the most information.

It was considered most desirable to make observations as soon as possible after blasting so that failures might be observed taking place and so that the unscaled and unsupported condition of the rock could be examined. Many of the observations were made from the muck pile, within 30 minutes after blasting. Other observations were made during scaling, bolting, shotcreting, mucking, drilling and up to several years after construction. At the projects that were under construction during the time of inspection it was the policy to inspect headings as soon as possible after blasting and to inspect other parts of the project during the drilling of the next round.

Conditions of instability in the form of roof falls, overbreak, wall slip-outs, and popping or slabbing rock were observed. The conditions around many stable tunnels were also recorded. An attempt was usually made to photograph the tunnel condition under observation. Sketches were also made. An attempt was made to obtain the following information for each observation:

a. Geometry of tunnel cross-section (width, height, area)
b. Nature of instability (roof fall, wall slip-outs, etc.)
c. Remedial measures
d. Rock type (occasionally Schmidt L-hammer hardness)
e. Factors responsible for condition (generally geologic structure-discontinuities, faults, etc.)
f. Overburden (rock and soil)
g. Average joint spacing and rock quality designation (RQD)
h. Ground water conditions
i. Local and regional structural geology or tectonic features.

An attempt was always made to relate the observed stability behavior to information available from preliminary investigations, particularly that from seismic refraction measurements and diamond drill cores. Unfortunately, the scarcity of the latter did not allow any correlations to be drawn between diamond drill core data and tunneling conditions. The correlations with seismic refraction measurements are discussed in a subsequent chapter.

An attempt was made to classify the rock at each observation according to some systematic procedure such as that proposed by Bergman (1965). Because no laboratory tests on rock cores were made, intact rock properties could not be classified. The information required for classification of the rock mass structure and discontinuity characteristics was recorded during observation and is presented in tabular form in the next section.

The rock quality designation (RQD) is a modified core recovery that is based indirectly on the number of fractures and the amount of softening or alteration in the rock mass (Deere et al., 1967). Instead of counting the fractures, an indirect measure is obtained by summing up the total length of core recovery but counting only those pieces of core that are 4 in. (10 cm) in length or longer, and which are hard and sound. It has been found that the RQD is a more sensitive and consistent indicator of generaly rock quality than is the gross core recovery percentage. The RQD method was originally introduced as a means of logging core information, but can be extended to logging of rock along any line or axis (Cording, 1968a). In sound rock it is only necessary to measure joint spacing, but in altered or soft rock it is also necessary to decide whether or not the rock material would be recovered in drilling. In the writer's field work an arbitrary criterion was used whereby any material that could be excavated by hand with a geology pick was discounted in the logging. The method has been used in the field studies to obtain rock quality designation values along and up tunnel walls in zones of unstable and stable rock. Values reported in these studies have been obtained from field measurements, measurements from photographs, and estimations made in the field. Logging "runs" were made over a distance equal to the width of the opening under investigation. For the actual measurements in the field it was sometimes found convenient to stretch a string over the desired run length and mark on it either every joint or discontinuity, or all rock blocks and non-recoverable material less than 4 inches in length, or all rock blocks greater than 4 inches in length, depending on the nature of the rock. In cases of very strongly jointed rock, it was found convenient to measure with a ruler all the rock blocks greater than 4 inches in width. After a number of measurements were made, it was found that estimations of RQD could be made to within about 10 percent of the actual value.

The orientation of the RQD runs was not the same for all cases, but rather was chosen for each case study, generally so as to cut normally across critical weak zones. Thus RQD runs were made both up and along tunnel walls as well as across tunnel faces. Because the effect of RQD anisotropy on any correlation between RQD and support requirements is likely to be large, estimates of RQD values for the vertical direction and parallel to the tunnel axis were made in addition to the "primary" RQD values determined across the primary weak zones responsible for instability. The vertical direction and the tunnel-axis direction are of particular interest because they are the two directions in which exploratory core borings are likely to be made prior to excavation.

In some cases it was possible to obtain seismic refraction velocity data, either from measurements in the tunnel or, more commonly, from ground surface refraction profiles. A series of ordinary seismic refraction profiles was shot in the Rätan tailrace tunnel for the purpose of determining the suitability of seismic refraction velocity as a measure or index of rock quality. This work is discussed in detail in Appendix A. The measurements in the Rätan tunnel led to the definition of the seismic velocity ratio as a measure of rock quality. This ratio is the ratio of the seismic velocity of a given rock condition to the seismic velocity of a very sound, unsupported condition in the same rock type. The seismic velocity ratio is similar to Deere's (1968) velocity ratio, which is the ratio of the compressional wave velocities of the in situ rock and of an intact specimen. Onodera (1963) was apparently the first investigator to propose such a quality index for in situ rock.

2.5 Field Observations

Presentation of Observations

In the 67 kilometers of tunnel and 13 rock chambers or rooms that were inspected, over 100 individual cases were studied in which some degree of support was required. The individual cases involved lengths of tunnel varying from several meters to several kilometers, and included not only loosening-type problems, but also high rock stress phenomena, such as spalling and popping rock.

At an early stage during the observations it became very apparent that the principal stability difficulty encountered in Swedish rock tunnels (in which most of the observations were made, and on which attention was most concentrated) is one of loosening instability. The phenomenon of loosening instability is the process that occurs in the roof of an unsupported opening in jointed rock as individual rock blocks slip and rotate under the action of gravity and the redistributed stresses around the crown of the opening. If a mass of blocks actually drops out of the roof of an opening, the height of the mass corresponds to Terzaghi's rock load (Terzaghi, 1946). The concept of loosening instability was apparently first discussed by Rabcewicz (Rabcewicz, 1944) as one of the three main types of pressure that act on tunnel linings (loosening pressure, genuine rock pressures, swelling pressure). Because of the general high quality of the Swedish bedrock and the general lack of any other tunneling problems, attention in the field studies was concentrated on the nature, causes, and treatment of loosening-type instability. It is for this specific problem that the field observations are presented in this section.

Observations made at 74 loosening instability cases where support was used, together with observations at 18 different unsupported tunnel sections, are given in Appendix B. These cases do not represent all of the inspected cases, but only those for which relatively complete observations were possible.

In addition to the cases actually observed by the writer, a few tunnel cases (Cases 93 - 97) are given in Appendix B that were available through personal communications with design and consulting engineers in various government and private agencies. These cases are very limited in number, as the required information was very seldom available. A specific attempt was made to gain information from large openings.

The information given for each case is presented according to the format shown in Table 2.2. Most of the items are self-explanatory. An indication of the nature of the instability (i.e., roof fall, wall slip-out) is given in item 6 together with a classification of the time-stability support conditions (given in parentheses). The relationships between time, stability, and support have been classified according to the conditions given in Table 2.3. The main purpose of this classification is to describe the time sequence of rock behaviour and support installation and the performance of the support. It is to be noted that only the last classification (H) involves failure of the supports.

The orientation in which the primary RQD values were taken and the method used to obtain the values (i.e., measurement or estimation) are given in item 11 of Table 2.2. As mentioned earlier, the RQD anisotropy is considered to be important. The RQD values in the vertical direction (RQD_v) and along the tunnel-axis direction (RQD_a) are also given in item 11. One of these values usually corresponds to the first, or primary, RQD value given. Where this is not the case, values of RQD_v or RQD_a were estimated. All RQD values are given to the nearest 10 percent.

TABLE 2.2

INFORMATION FORMAT FOR CASE HISTORIES IN APPENDIX B

1. Project location
2. Type of tunnel or room
3. W = width of opening, meters
4. H = height of opening, meters
5. A = cross-sectional area of opening, square meters
6. Nature of instability (stability classification)
7. L = Length of condition under consideration
8. Geologic features responsible for condition, rock type
9. Support or remedial measure
10. D = depth of overburden (soil and rock), meters
11. RQD, location, method; RQD_v; RQD_a
12. V = seismic velocity, m/sec*
13. SVR = seismic velocity ratio
14. Regional tectonics or major structural geology features
15. Ground water condition
16. Other notes

* Values given in parentheses are from projected ground surface data

TABLE 2.3

TIME-STABILITY-SUPPORT CLASSIFICATION FOR OBSERVED CASES

TIME-STABILITY-SUPPORT CLASSIFICATION FOR OBSERVED CASES

A	Stable at blasting, no anticipated falls, no support
B	Minor falls or overbreak at blasting, support not considered necessary for prevention of loosening
C	Stable at blasting, support in anticipation of loosening
D	Stable at blasting, unsupported, gradual deterioration and subsequent support
E	Falls at blasting, support in anticipation of progressive loosening
F	Falls at blasting, no support immediately after blasting, progressive loosening, support applied to prevent further loosening
G	Falls at blasting, support shortly after blasting to prevent or stop progressive loosening
H	Support shortly after blasting, failure of support thereafter, additional support

Seismic refraction velocity data are reported in items 12 and 13. The only data available for measurements in the tunnels are those from the Rätan project that are discussed in Appendix A. It was possible in a number of cases to project weak bedrock zones to the surface and correlate the projections with weak zones determined from surface refraction measurements. Where such projections and correlations have been made, the values for seismic velocity and seismic velocity ratio are shown in parentheses. The velocity values used to compute the seismic velocity ratios are shown.

The information necessary for a visual classification according to methods such as those proposed by Bergman (1965) and Coates (1964) is given in Table 2.4. The required information for all 92 field cases is given in Table B.1. The check list format is a combination of proposals by Deere (1963), Coates (1964), Bergman (1965), and Bjurström (1966-67) for the description and classification of rock mass characteristics. Some modifications and additions have been made by the writer. No attempt has been made to classify intact rock properties, as it is believed that none of the differences in observed behavior can be attributed to differences in intact rock strength. The classification of the rock material as sound or altered is considered adequate.

The classification of rock mass structure is that proposed by Bjurström for his studies in Swedish defense structures, and is an extension of Bergman's proposal. The five categories also include those suggested by Hagerman. The classification for average discontinuity spacing is after a proposal by Deere (1963) for both jointing and bedding. Discontinuity tightness is judged to be either tight or open. The designation tight or open is used in a very general sense. In general, if there exist one or two open joints that cause difficulty in a case that would otherwise be stable, the designation open is indicated.

The discontinuity type is mostly self-explanatory. If there is any indication of movement along discontinuities, the fourth category is indicated. It is clearly evident that more than one type of discontinuity may be present, as is indicated in many cases. Similarly, if evidence of shearing or movement exists along any one of the first three types of discontinuities, the fourth item is checked. The word "sköl" is a Swedish term for a shear zone that contains gouge and crushed, sheared material. A sköl is not necessarily a fault, along which net displacement has occurred. Typical "skölar" are seen in the photograph in Case 1, Appendix B.

In the check listing for discontinuity filling or coating, distinction is made between softening and non-softening clays. This distinction is not made on the basis of clay mineralogy, but rather on the basis of the behavior of the clay materials in the cases. The softening clays are those that undergo a reduction in strength with time that is caused by water absorption. In some of the cases (Rendal, Sällsjö, Mo i Rana, Lieråsen projects) identification of the clay mineral montmorillonite has been made through differential thermal analysis (Selmer-Olsen, 1968; Håland, 1967). Free swelling upon submersion in water of the dried and pulverized clay materials from the Sällsjö and Lieråsen projects was about 120 percent (Selmer-Olsen, 1968).

Because positive identification of clay minerals from all of the cases involving clay materials was not made, and because no systematic investigation of the laboratory behavior of the clay materials was undertaken, no attempt has been made to differentiate between those cases where actual swelling took place and those cases where the strength reduction was by softening at a small change in volume. All of these cases are collectively referred to as softening clay cases.

Those cases in which no time-dependent reduction in strength of the clay materials by softening occurred are termed non-softening clay cases. It is to be recognized that the behavior distinction between softening and non-softening clays is as dependent on the availability of water to the clay materials as it is on the clay minerals present. Some of the clay materials that are check listed in Table B.1 as non-softening cases probably would have been softening cases had water been present. Thus, the distrinction is merely one in the effect that the clay materials had on the stability of the tunnel. The significance of the softening clays lies primarily in their influence in allowing the surrounding rock to loosen to a

TABLE 2.4

CHECK LIST ROCK MASS CLASSIFICATION

Intact Rock Strength	
	Sound
	Altered or weathered
Rock Mass Structure	
	Massive, no or very few discontinuities
	One discontinuity set
	Two discontinuity sets
	Three discontinuity sets
	Random discontinuity (W), Crushed (C), or Earthlike (E)
Average Discontinuity Spacing	
	Less than 5 cm (2 in)
	5 cm - 30 cm (2 in - 1 ft)
	30 cm - 1 m (1 ft - 3 ft)
	1 m - 3 m (3 ft - 10 ft)
	Greater than 3 m (10 ft)
Discontinuity Tightness	
	Tight (T), Open (O)
Joint Continuity	
	Continuous (C), Discontinuous (D)
Discontinuity Type	
	Joint
	Bedding Plane
	Cleavage or schistosity
	Fault, shear, "sköl"
Discontinuity Filling or Coating	
	None
	Non-softening clay
	Softening clay
	Other low friction material
	Sandy or gravelly material, rock fragments
	Alteration along joints
Degree of Discontinuity Planeness (Intermediate Scale)	
	Plane
	Curved
	Irregular
Degree of Discontinuity Roughness	
	Slickensided
	Smooth
	Rough
Dip of Discontinuities	
	$0-30^{\circ}$
	$30-60^{\circ}$
	$60-90^{\circ}$
Strike of Discontinuities	
	$0-30^{\circ}$
	$30-60^{\circ}$
	$60-90^{\circ}$

greater degree than would occur if the clays did not soften with time and in the reduced frictional resistance of the softened materials.

Other low friction discontinuity filling and coating materials include graphite, chlorite, talc, and serpentine.

The degree of discontinuity roughness is considered very important, and is broadly divided into slickensided, smooth, and rough after Deere's (1963) recommendation. The dip and strike of discontinuities is classified very roughly as indicated. The strikes are given with respect to the tunnel axis and not the north direction as is conventionally done.

Description of Observed Stability Behavior

The time-stability-support classification given in Table 2.3 is the only attempt that has been made to classify the field observations. This classification is merely a description of the time sequence of support installation and tunnel behavior. No attempt has been made to classify or evaluate the degree of stability of the cases. The writer found that the degree of vagueness in defining stable and unstable openings in rock is very comparable to that in determining support requirements. For these reasons, the field cases are simply referred to as supported tunnels and unsupported tunnels. The word "stable" is used to indicate that no roof falls or wall falls were encountered and that no support was necessary to maintain the desired shape of the opening.

It must be emphasized that the support used in each of the cases is not necessarily the minimum required to present further loosening and enlargement of the opening. Nor does it correspond to any particular safety factor against progressive opening enlargement. The reinforcements and support are based solely on the judgement of the tunnel foreman and engineers who were responsible for the projects. It is very likely that some of the supported tunnels are greatly oversupported while others are very close to collapse. Furthermore, the methods of support are subject to large personal factors. In particular, the use of wire mesh reinforcement in shotcrete is a controversial item, a fact that is reflected in the pronounced absence of such reinforcement on some projects (Rätan, Seitevare) and its frequent use on other projects (Bergvattnet, Vietas).

All of the supported tunnels, and most of the unsupported tunnels, are instances where some degree of overbreak has occurred. The supported tunnels are instances where it was considered necessary to provide some degree of support to avoid further overbreak and progressive enlargement of the opening.

Some of the general characteristics of the unsupported and supported tunnels are given in Table 2.5. Examples from Appendix B are cited and photographs of some of the case examples are shown in Figures 2.7 - 2.16. An attempt was made to classify the supported tunnels according to the amount and shape of overbreak, roof falls, and wall slip-outs. However, it was found difficult to make clear distinctions among many of the cases, and an all inclusive classification system would necessarily involve many separate categories to fit all of the cases. The descriptions in Table 2.5 are not intended as a classification, but rather only several examples of repeatedly occurring behavior.

Significance of Observed Tunnel Behavior

From Table 2.5 and Appendix B it is seen that some of the supported tunnels are of relatively minor significance in consideration of the volume of overbreak and the amount of support used. However, many of the more extensive cases of overbreak and roof falls started as seemingly insignificant overbreak and enlarged through progressive block fallout.

The volume of the overbreak associated with individual cases, including that due to fallout varies from several cubic meters to 4000 cubic meters (Case 43). All but one of the cases involve less than 1000 cubic meters. No attempt has been made to estimate the volume of overbreak for each case.

The cost of the extra scaling, loading, hauling, and support work associated with the cited examples varies from a fraction to several times the cost of driving the tunnels through equally long sections of sound rock that requires no support.

TABLE 2.5

DESCRIPTIONS OF SOME TYPICAL OBSERVED FIELD CASES

Condition	Case Examples
No overbreak; no roof or wall fall-out; no support	21 (Fig. 2.7)
Minor overbreak; no roof or wall fallout; no support or only occasional spot rock bolting	36 (Fig. 2.8) 86, 92
Moderate overbreak; but no roof or wall fallout; support may be used for protection against small pieces of falling rock, but is frequently omitted; spot rock bolting common	6 (Fig. 2.9) 29, 78
Large overbreak, but no fallout; support same as in previous condition	12, 20, 64 17, 27 (Fig. 2.10)
Large overbreak and/or roof falls; support deemed necessary for prevention of progressive enlargement of opening	30, 32, 76, 86, 88 (Fig. 2.11)
Wedge-shaped roof falls along single geologic features; support necessary to either stop progressive loosening that is in progress or to prevent future loosening	47 (Fig. 2.12) 48-51, 53, 55, 90
Progressive roof falls in heavily fractured rock; formation of vault- or dome-shaped opening; support necessary to stop loosening that is in progress; fallout may exceed one half of tunnel area	33, 37 (Fig. 2.13) 39, 43, 56, 57
Large overbreak and fallout in tunnel walls that might undermine arch support	4, 10, 11, 13, 14, 15, 18 (Fig. 2.14)
Loss of corner at intersection of two tunnels	34 (Fig. 2.15) 75
Washing out of filling materials in faults or seams; loosening of rock mass	45, 54 (Fig. 2.16)

The total length of supports and reinforcements for the 92 observed cases amounts to about 5.2 km, or about 9 percent of the 57 kilometers of tunnel in which loosening problems were observed. This figure is lower than the 15 - 20 percent average that is frequently reported for Swedish civil engineering tunnels. The following reason are given for the difference:

(a) The writer was unable to inspect significant lengths of supported tunnels that were com-

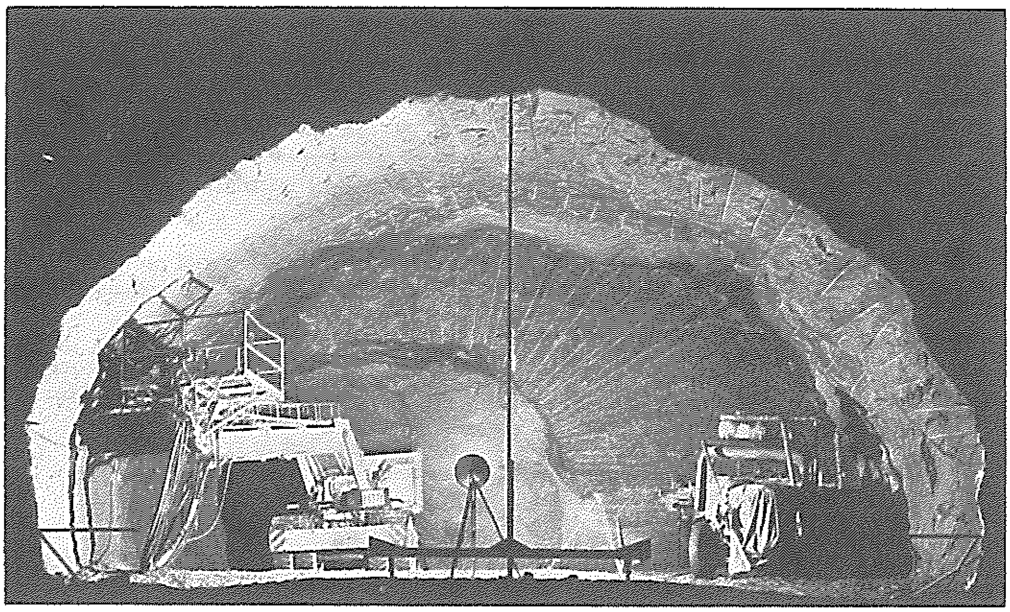


Fig. 2.7 Case 21. Presplit roof in Seitevare machine hall.
Example of no overbreak; no roof or wall fallout;
no support necessary

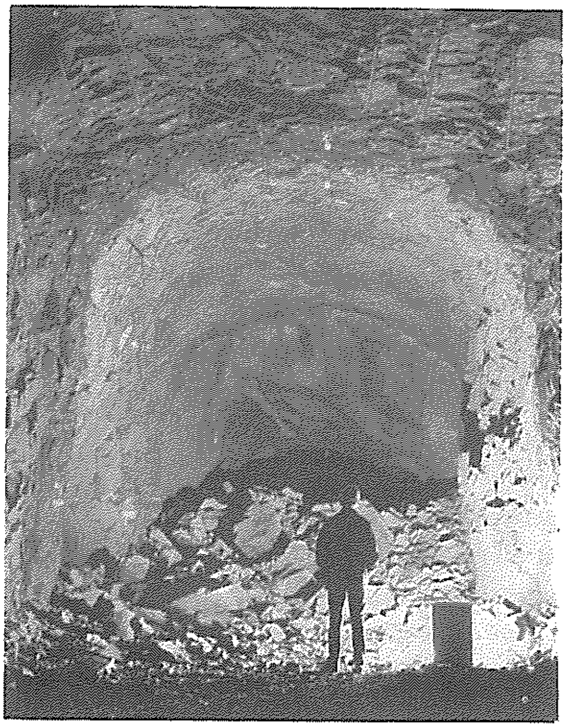


Fig. 2.8 Case 36. Access tunnel av Vietas.
Example of minor overbreak, but no roof or wall fallout;
no support or only occasional spot bolting

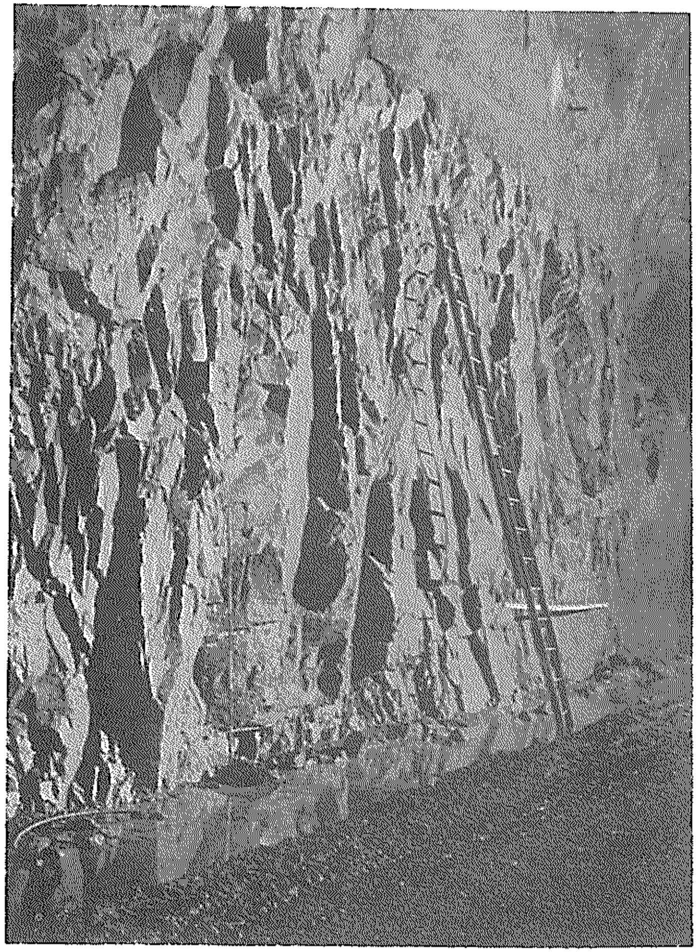


Fig. 2.9 Case 6. Seitevare tailrace tunnel. Example of moderate overbreak, but no fallout; support used only for protecting against small pieces of falling rock

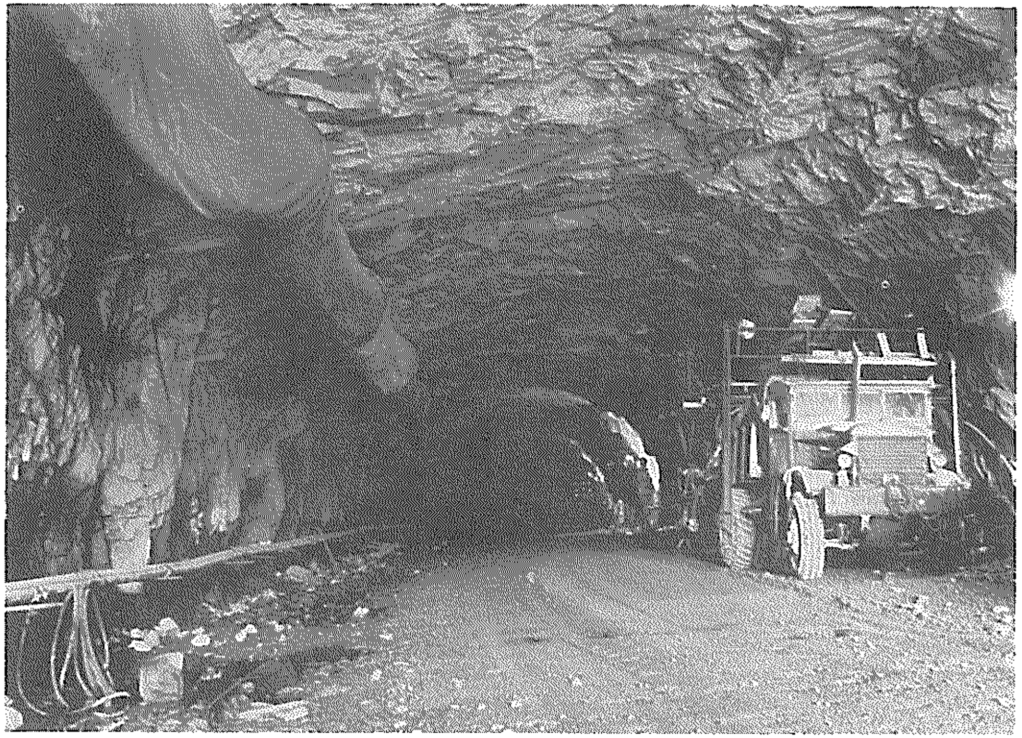


Fig. 2.10 Case 27. Suorva headrace heading at Vietas. Example of large overbreak, but no fallout; occasional spot bolting

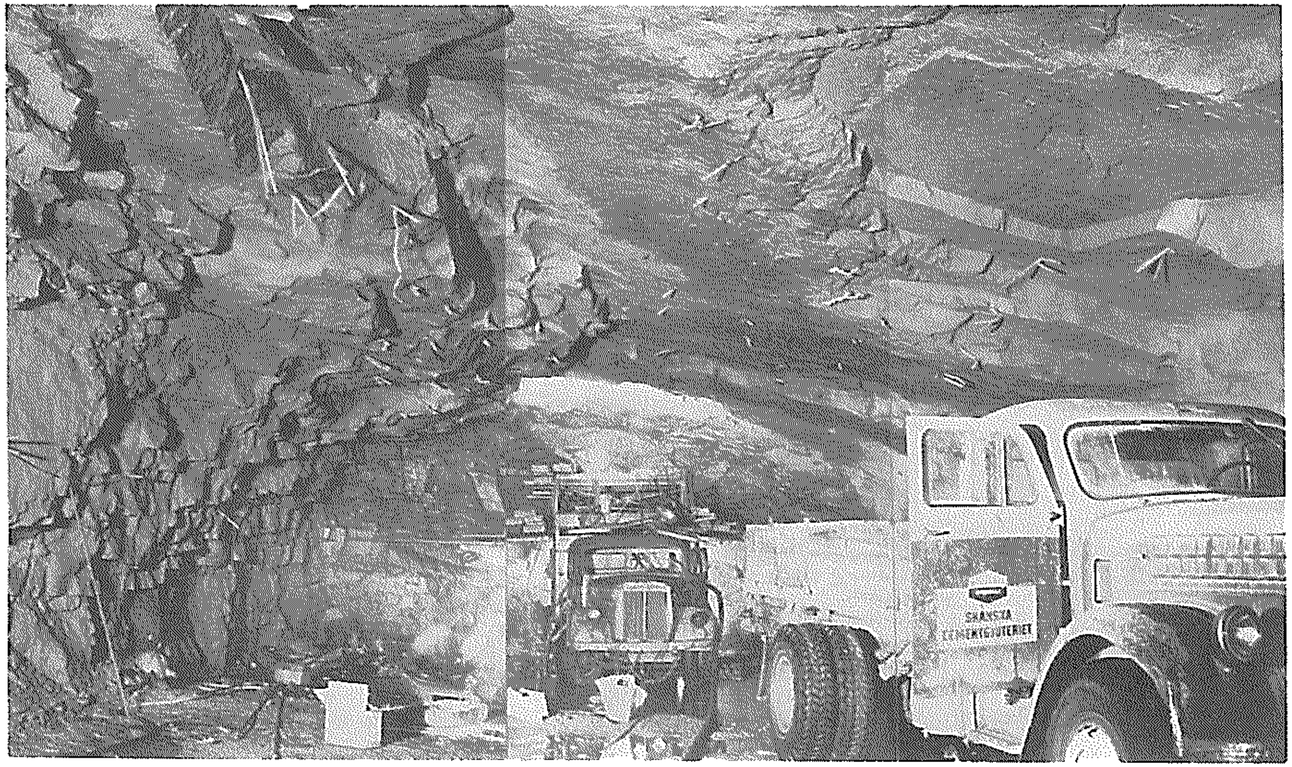


Fig. 2.11 Case 88. Heading for Rätan tailrace tunnel.
Example of large overbreak and roof falls; support necessary for prevention of progressive enlargement of opening

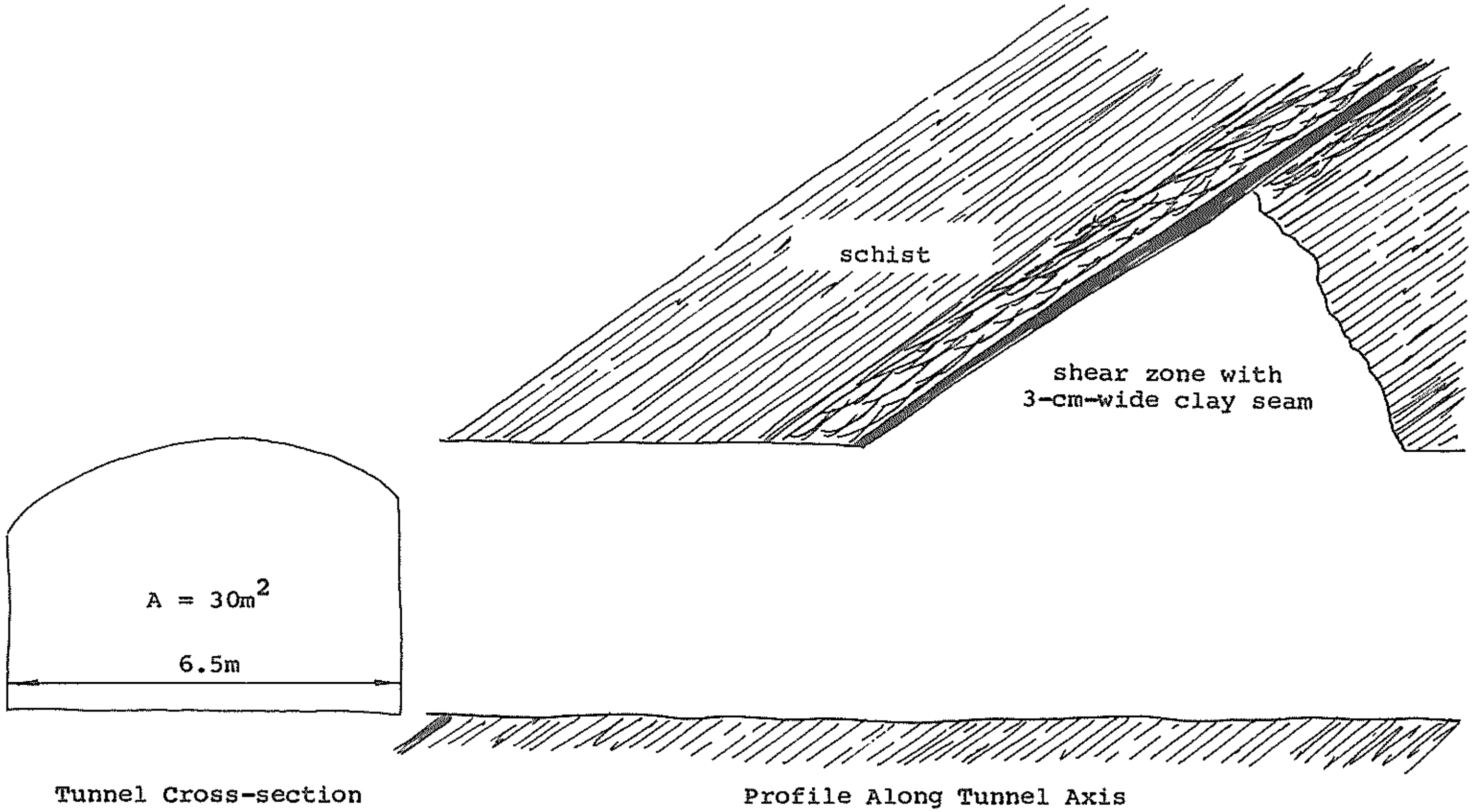


Fig. 2.12 Case 47. Bergvattnet tailrace tunnel.
Example of wedgeshaped roof fall along a single geologic feature; support necessary to either stop progressive loosening that is in progress or to prevent future loosening

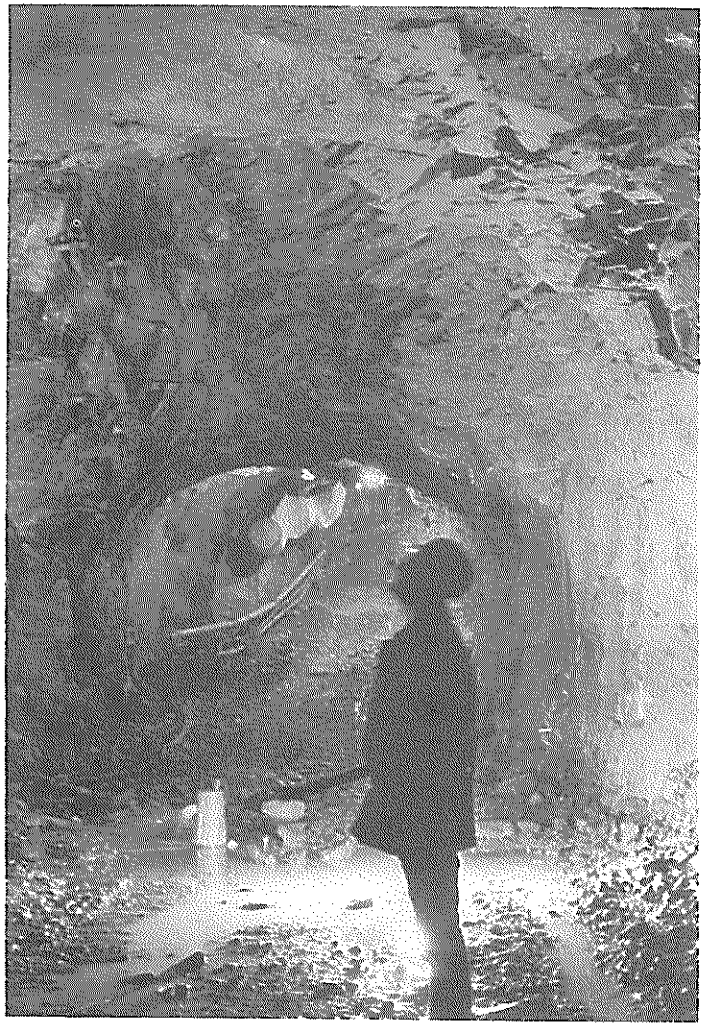


Fig. 2.13 Case 37. Access tunnel at Vietas.
Example of progressive roof falls in heavily fractured rock; formation of vault- or dome-shaped opening; support necessary to stop loosening; fallout exceeds one-half of tunnel area

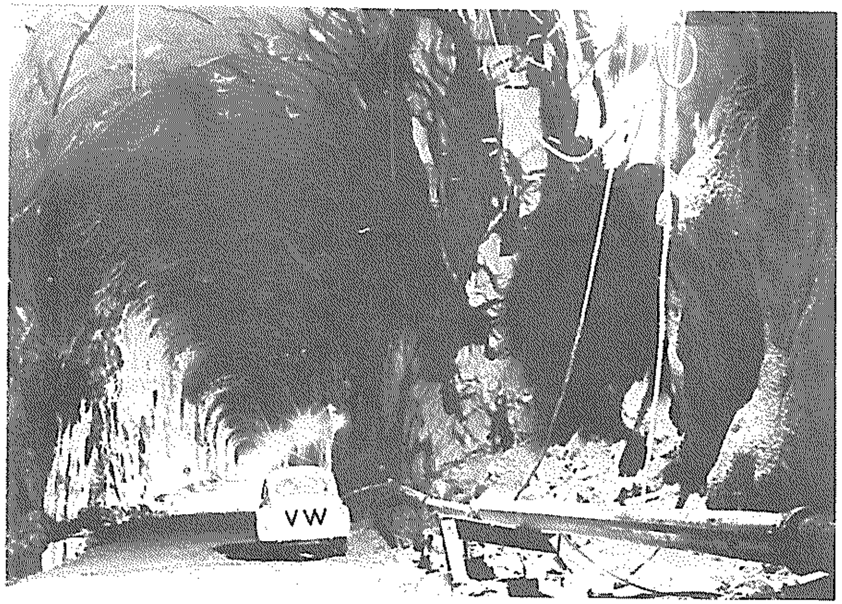


Fig. 2.14 Case 18. Seitevare tailrace tunnel.
Example of large overbreak and fallout in tunnel walls that might undermine arch support

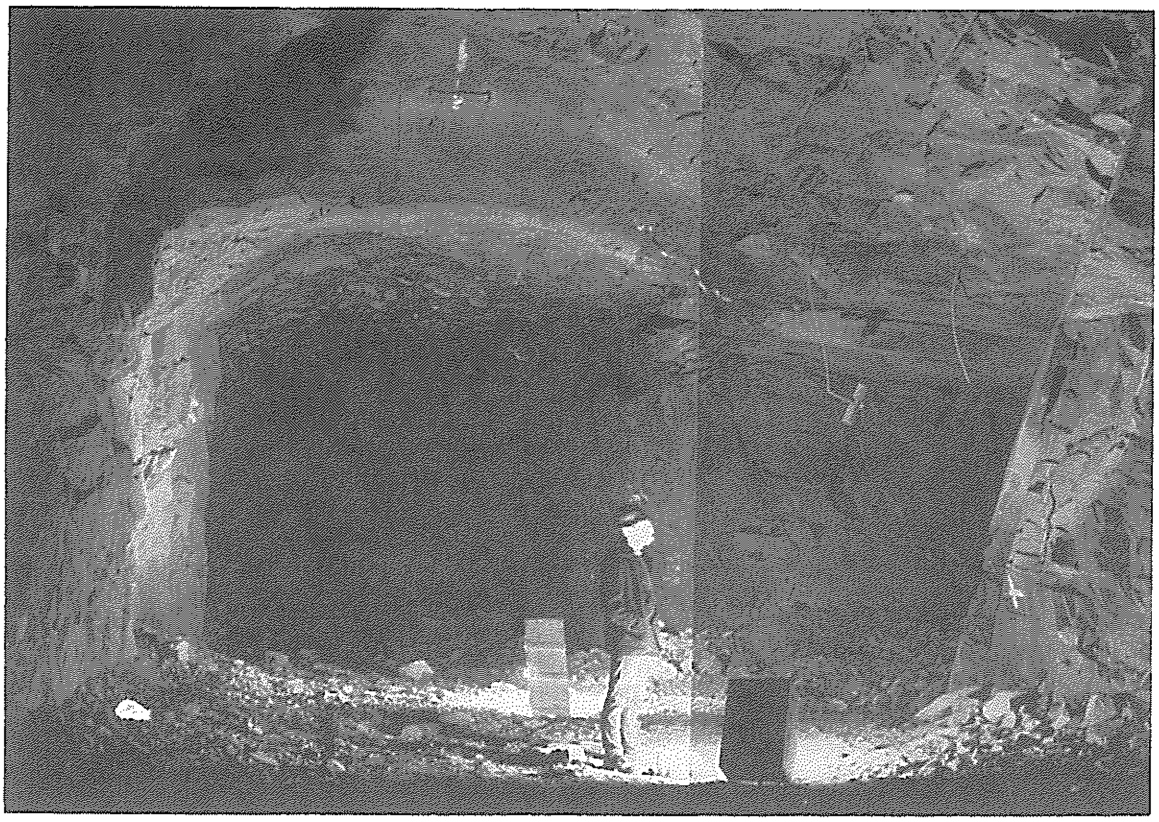


Fig. 2.15 Case 34. Access tunnel at Vietas.
Example of loss of corner at intersection of two tunnels

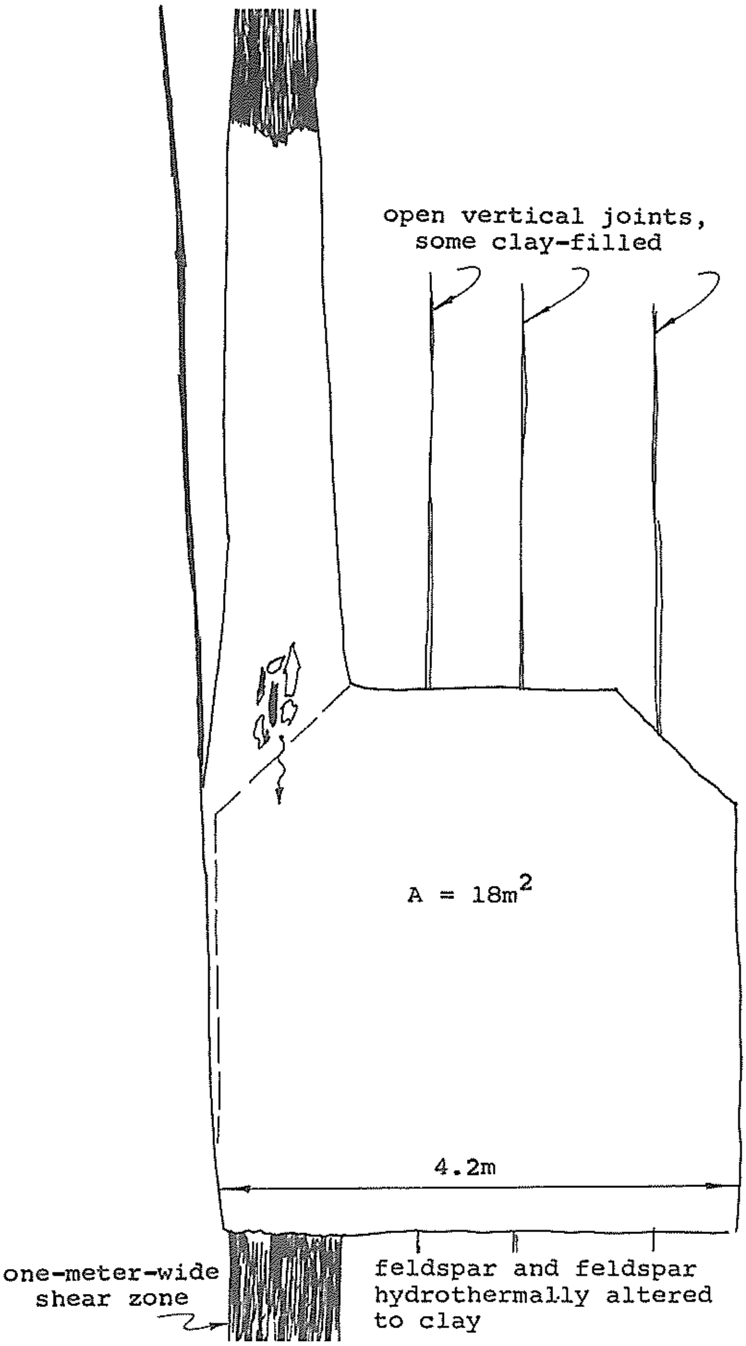


Fig. 2.16 Case 54. Collector tunnel at Bergvattnet.
Example of washing out of filling materials in faults and seams

pletely shotcreted.

(b) Much of the shotcrete application observed by the writer was not considered necessary for structural support purposes. Much shotcrete, particularly small aggregate mix ($< 3/8$ in), is used solely for protection against small pieces of falling rock. Furthermore, there frequently exist strong ties between those who determine tunnel support and the shotcrete contractor.

(c) Complete information was not obtained for all of the observed cases, and lighting difficulties very frequently prevented close observation.

There are two factors in the observed cases that deserve special consideration. First, those observations at tunnel portals and at tunnel intersections involve a free face that is not common to the observations made in straight tunnels. The latter represent plane-strain cases whereas portals and intersections approach a plane-stress condition. Hence, it would not be reasonable to mix these two kinds of observations in any attempts to correlate stability behavior or support requirements with rock mass properties or parameters.

Second, the type and amount of support used depends to a certain extent on the type of project, or more specifically, on the desirability of avoiding roof or wall fallout or other progressive tunnel deterioration. In transportation tunnels it is highly desirable to avoid any post-construction falls or tunnel deterioration. The same is true in machine halls, storage rooms, and other openings where post-construction falls of even the smallest magnitude are likely to cause damage or inconvenience. Thus, it is not too surprising that the support measures for these cases are more extensive than in openings where minor falls can be tolerated, such as in tailrace tunnels. Thus, the significance of a roof fall or potentially unstable area is not the same for all projects. Furthermore, the time at which such an area is supported depends on the nature of the opening. Areas that are accessible for repair or support work over long periods of time, such as access tunnels, generally are not given as close and rapid attention as areas that must be sealed off or otherwise made inaccessible a short time after construction. Both the portal or intersection cases and the cases where support is likely to be different because of the intended use of the tunnel are considered special cases in the following chapters and are designated with the letter "s". These cases are not included in the correlations that are presented and discussed in Chapter 5.

3. NATURE AND CAUSES OF LOOSENING INSTABILITY

3.1 Introduction

It is necessary to understand the nature and causes of loosening instability for the following reasons:

1/ Any theoretical analysis of design method for tunnels in jointed rock would require a knowledge of the mechanism of the mode of failure and the parameters that affect the behavior of the tunnel.

2/ Even if no valid theoretical methods were available for the design of support systems in tunnels subjected to loosening instability, a qualitative knowledge of their mechanical behavior might permit a better selection of the type and amount of support.

3/ An empirical approach to the design of tunnels and support systems in jointed rock is more likely to be fruitful if the behavior of the tunnel is understood, even if only qualitatively. In this way the most significant parameters can be chosen for correlation with the observed behavior.

Unfortunately it was not possible to observe very many roof falls taking place in the field. Most falls take place immediately or very shortly after blasting, and the process is so rapid that one only receives the impression of falling rock debris. For this reason, it was considered necessary to perform a laboratory model study. The model study, whose results are discussed in this chapter, provides the advantages of control and observation of the failure mechanism during all stages. The model study results and field observations are combined to give an explanation of the loosening stability behavior of jointed rock.

Finally, from the points of view of both design and construction, it is highly desirable to know which geologic and structural conditions are most commonly associated with loosening instability. All of these factors are discussed in detail in this chapter.

3.2 Existing Evidence and Hypotheses

The basic mechanism of loosening-type instability has been described by several writers. These postulated mechanisms will be described in detail, as all of them are directly applicable to the results of the model tests. They are all combined and supplemented with results from the tests to give an explanation for the loosening behavior of tunnels in jointed rocks.

Although several investigators (Rabcewicz, 1944; Terzaghi, 1946) have discussed loosening instability, Lang (1957, 1959, 1961, 1964) gives the simplest explanation of the basic failure mechanism associated with loosening. He defines the behavior as that due to "slipping", "separation" or both.

> "Slipping includes fracture by shear through the intact rock or by slipping on joint planes. In the latter case, not only must the surface friction on the joint be considered but also any interlocking between joint surfaces which would result in shear also playing a part."

> "Failure by a separation fracture can be caused in two ways, viz. tension or rotation. Failure by rotation is generally the direct result of the joint system in the rock. Under the combination of forces which may exist in the rock mass individual blocks are displaced. The displacement can be either a simple slipping or sliding, or in the more general case, the block is rotated under a moment, the joint opens and failure occurs by slipping or crushing or shearing at the corners of the block. This is the typical action of 'stoping' which is the familiar term used by tunnellers and miners to indicate a process of fall-out from the roof of an excavation (Lang, 1957, 1959)."

Lang further explains that

> ... it rarely occurs that failure is due to sliding on the joint only; it is brought about by sliding and rotation, or rotation under moment may by the prime cause. In either case sliding and rotation almost inevitably, in the case of confined blocks, cause the development of end forces on the block which have a stabilizing effect of the jointed material. The cause of failure is by rotation, as illustrated in Fig. 4b (Fig. 3.1a herein). The joint on the tensile side tends to open while the reaction between the two blocks on either side of the joint plane is transferred to a smaller area as the opening proceeds. Eventually, the joint fails by shearing or crushing of the material at the point or points about which rotation is taking place. If this point is confined (Fig. 4b) (Fig. 3.1a herein), then although the initial fracture may be a shear fracture at an angle to the joint surface, the material will still be capable of carrying load and is eventuellay crushed ... The case of a rough joint presents several interesting features, illustrated by Fig. 4c (Fig. 3.1b herein). A, C, D, B is a conventionalized rough joint. If the resultant forces of the two blocks are as indicated, then failure could occur by sliding after shearing off the asperities at C and D, or by rotation. In the latter case, the frictional forces developed on the faces C and D would tend to oppose the rotation (Lang, 1961)."

Although he gives no details, Lang clearly implies that the behavior of the roof of an excavation in jointed rock may be that of either an arch or a beam, depending on the reaction in the abutment zone. If only vertical reactions exist in the abutment zone, then beam action prevails. If both horizontal and vertical reaction components exist, then arch action can take place.

In summary, Lang has described qualitatively both the mechanical interaction and failure mechanism of individual blocks in a jointed rock mass. He also touches lightly on the overall gross behavior of openings in jointed rock, but gives no detailed explanation of this behavior.

Trollope (1966) has conducted simple model experiments of a trapezoidal opening in a mass of smooth, horizontally bedded, imbricated plastic cubes, as shown in Figure 3.2. Although his results are strictly qualitative in nature, they do illustrate vividly the loosening phenomenon in a block-jointed tunnel roof. Trollope's model studies demonstrate three noteworthy points:

1/ Roof fallout in an imbricated block-jointed medium (not subjected to lateral stress) occurs until a very stable triangular opening is established. The arching phenomenon responsible for this behavior results in two zones of material: a stable zone outside the triangular opening,

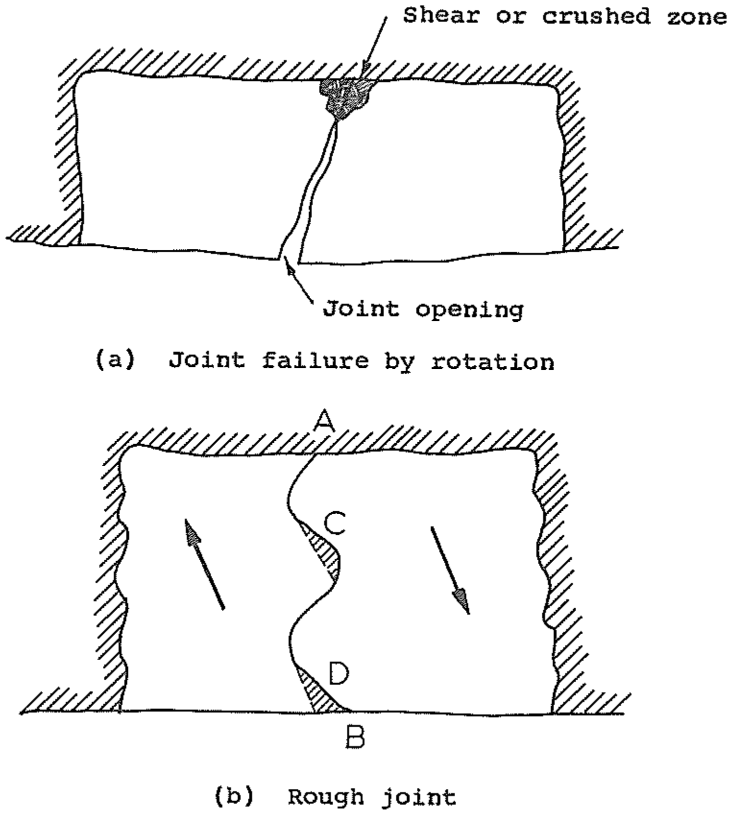


Fig. 3.1 Behavior of simple rock joints (after Lang, 1957)

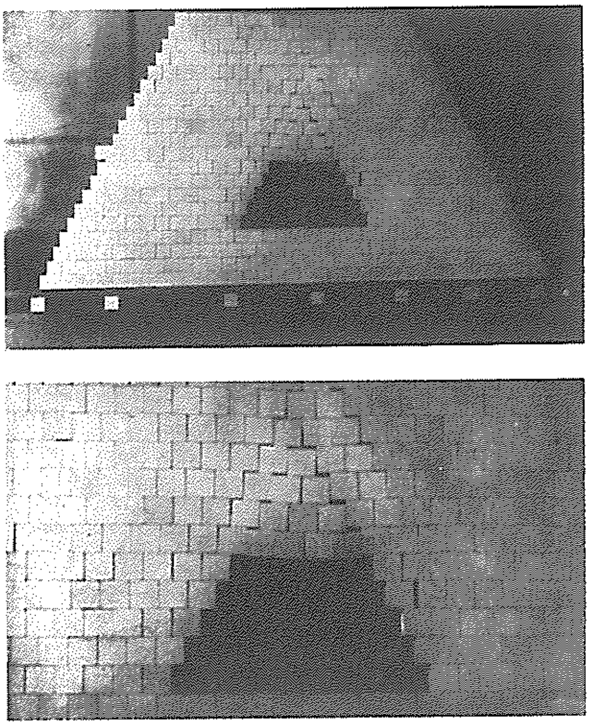


Fig. 3.2 Trollope's block-jointed model (after Trollope, 1966)

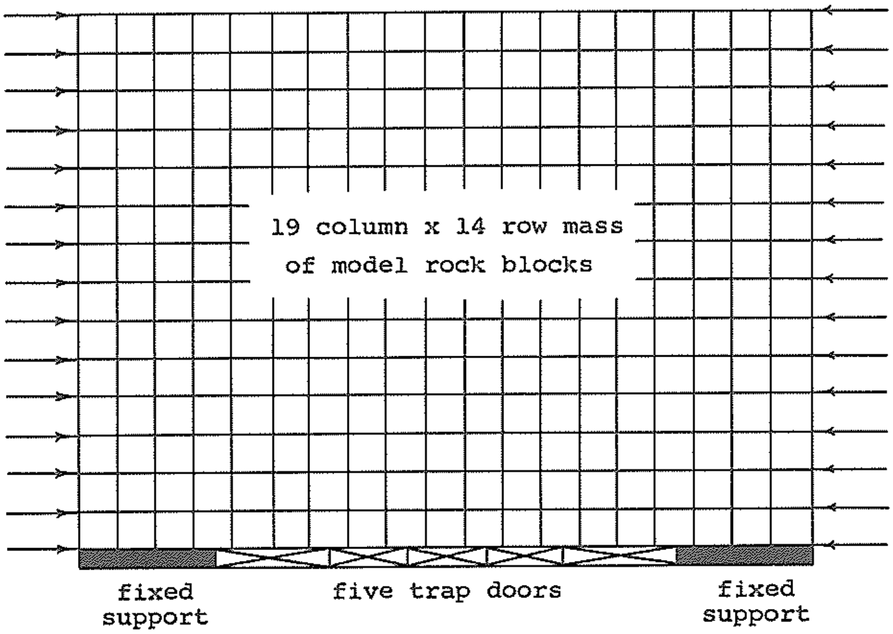


Fig. 3.3 Schematic diagram of model for study of loosening mechanism

and a suspended zone of loosened blocks.

2/ The extent of the suspended zone is not necessarily defined completely by the geometry of the opening, but is also influenced by the deformation or failure of material in the sides of the opening.

3/ For a block-jointed, horizontally-bedded mass, the fundamental unit in the suspended zone with respect to ultimate behavior is the simple row at the roof of the opening that is capable of supporting itself independently by interaction with the sides of the stable opening.

Trollope interprets the failure of a simple row of blocks in an unsupported tunnel roof subjected to gravity loading as a combined process of (a) a tendency for individual blocks to slide out in shear along vertical joints, and (b) bending of individual rows of blocks and the subsequent development of lateral thrusts that inhibit further slip along vertical joints. If stability is achieved by the combined process of slipping and bending, each row is ultimately relieved of any superincumbent load and is merely required to carry its own weight. Ultimate failure is conceived of as the development of plastic hinges at points of high compression. Because a rigorous solution for the stability of an arched mass of blocks is not available, Trollope suggests that the arching action and stability of a single row of blocks in a tunnel roof be treated hypothetically as a no-tension beam. The analysis is similar to that for an unreinforced brick beam that is available from previous work on masonry walls. Although the analysis does show the stabilizing effect of horizontal residual or tectonic stresses normal to joint planes and the effect of span width, the method has limited applicability because of the unknown stresses that act axially on the row of blocks and the unknown vertical central deflections of the roof span of an opening. Trollope's work does, however, provide a more mechanistic view of loosening instability than do Lang's descriptions of joint block failures.

Goodman, Taylor, and Brekke (1968) have taken what appears to be the most realistic analytical approach to the stability behavior of a jointed rock mass that heretofore has been attempted. Their finite element computer solution has been developed in such a way that individual joints are treated as no-tension linkage elements that can be assigned the same stiffness and frictional properties as real joints. The behavioral features of the method include intact rock failure in tension and shear, rotation of blocks, development of arches, and even to a certain extent, the collapse pattern of structures in jointed rock. The analysis has been checked for simple sliding and rotating joint configurations; Trollope's trapezoidal tunnel model; and stability conditions in several real tunnels subjected to dynamic loading. All of these trials have shown realistic results that compare favorably with known or observed behavior, and the method appears to be the most promising analytical tool for rationally analyzing general stability problems in jointed rock. The method has the unique advantage of being able to incorporate most of the important parameters that influence the mechanical behavior of a tunnel. The analysis by Goodman et al. (1968) of Trollope's trapezoidal opening model shows the same basic loosening mechanism described by Trollope, namely block rotation, and slip along vertical joints.

3.3 Mechanism of Loosening Instability

Laboratory Model

In the present investigation a model of a hypothetical unsupported opening in jointed rock was constructed for the purpose of investigating the mechanism of loosening-type instability and demonstrating the influence of several parameters on the stability of openings in jointed rock. The design and construction of the model and results from eight tests are presented in Appendix C.

The model apparatus consists of a load frame that supports and loads a mass of 266 2-1/2 in. x 2-1/2 in. model blocks, as shown in Figure 3.3. The mass of blocks is loaded by a lateral pressure that hypothetically represents the tangential stresses in the crown of a tunnel. The behavior of the mass of blocks is observed as the support is relieved by dropping the five trap doors beneath the center of the mass. The model is not intended to reproduce the behavior of a real tunnel in jointed rock, but only to provide a

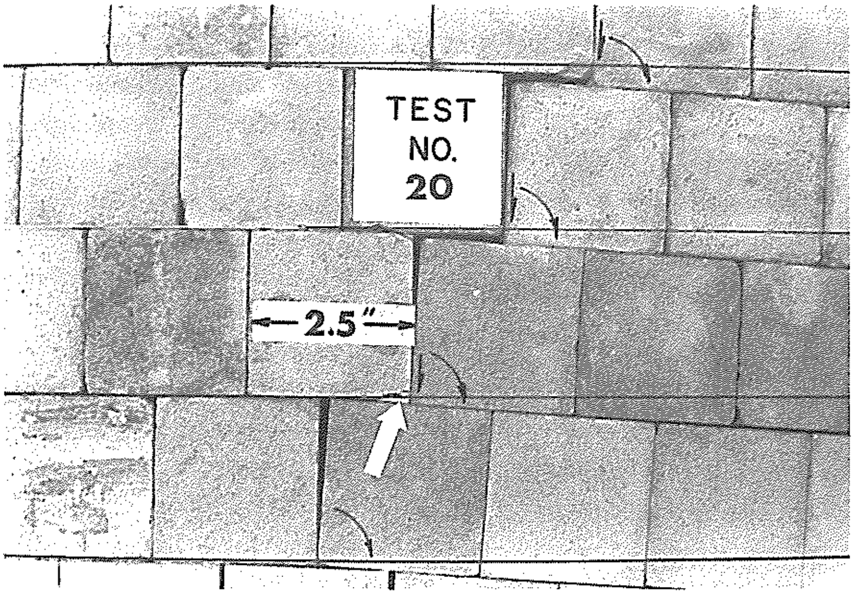


Fig. 3.4 Combined block slip and rotation

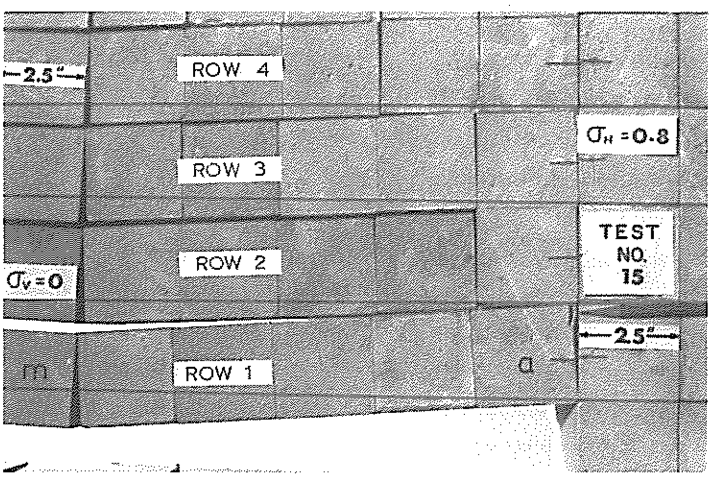


Fig. 3.5 Block row bending. Pencil points to corner tear caused by block rotation. Note slip of second and fourth rows

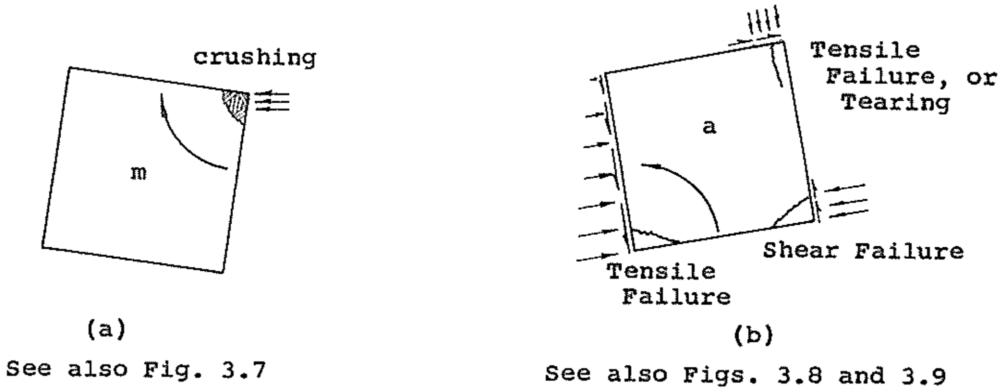


Fig. 3.6 Stress concentrations and failure patterns at block corners. Blocks lettered "m" and "a" correspond to blocks in bottom row of Fig. 3.5

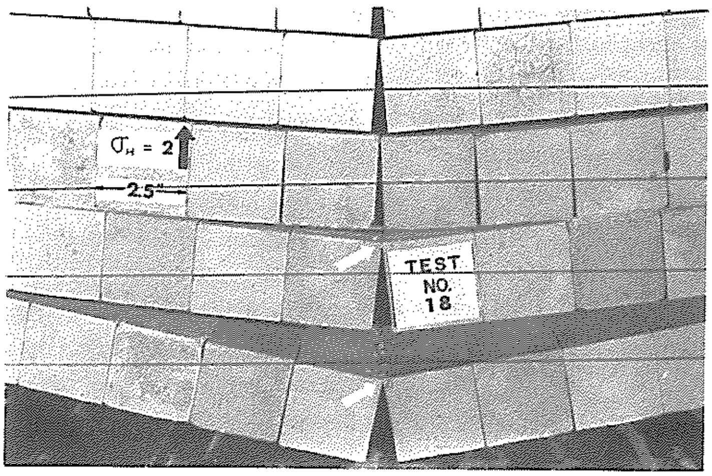


Fig. 3.7 Corner crushing associated with block rotation

mechanistic explanation of loosening in a block medium. Only the conclusions and a general interpretation of loosening instability are presented in this chapter.

The mechanical behavior of unsupported openings in a regularly jointed block medium is governed by the behavior of individual blocks as well as the interaction of larger numbers of blocks around the periphery of the opening. Because the gross behavior of a jointed mass is governed by interactions between all of the individual blocks, it is convenient to discuss the behavior of individual blocks prior to discussing the behavior of the block mass.

Behavior of Individual Blocks

As described by Lang (1959) and Trollope (1966), and shown in Figure 3.4, the behavior of individual blocks in an unsupported mass is characterized by both slipping and rotation.

Individual blocks slip because of a lack of resisting forces on their boundaries. The lack of resisting forces can be attributed to one or more of the following:

1/ A particularly low coefficient of friction along the boundaries of an unsupported block.

2/ An unfavorable orientation of the boundaries of a block or mass of blocks that prohibits development of resisting forces on its boundaries.

3/ A low component of stress normal to the boundaries are favorably oriented at steep angles to the periphery of the opening.

Individual blocks rotate under the action of their own weight and thrust reactions from adjacent blocks. The rotation of individual blocks usually occurs during the bending of an entire row of blocks, as shown in Figure 3.5. This action results in dilation of the block mass, as evidenced in the model tests by increases in lateral pressure during bending under conditions of lateral restraint.

Block rotations lead to stress concentrations at block corners, which may in turn cause corner failure by crushing, shearing, or tearing, as indicated in Figure 3.6. An example of corner crushing associated with block rotation is shown in Figure 3.7. Corner tearing or tensile failure associated with block slip is shown in Figure 3.8. A block corner that failed in shear is seen in Figure 3.9. A corner failure that may have occurred by combined shear and tension is seen in Figure 3.4 (white arrow).

Behavior of the Block Mass

Crushed block corners caused by block rotations may in themselves lead to block row failure, as seen in Figure 3.7. Tearing or tensile failure of block corners may result in reduced shearing resistance along vertical joints and subsequent block fallout, as illustrated in Figure 3.8.

In general, block slip may or may not result in fallout from the periphery of the opening. In either case, slip results in a redistribution of stresses within the block mass, and continues until a stabilized mass is achieved or until complete fallout occurs. If block rotations do not, or cannot, occur, slip leads to complete collapse of an opening without the development of a loosened or arched zone, as illustrated in Figure 3.10 for a block mass that contains two vertical teflon seams. If block rotations can occur, then initial slip may be arrested by stabilization through rotation and the development of arch action by lateral thrusting and shearing onto adjacent blocks, as shown schematically in Figure 3.11 and from model tests in Figures 3.12 and 3.13.

The fundamental behavior is governed by the successively smaller bending deformations of block rows above the free span. The blocks marked with X's in Figure 3.11 have become loosened and partially separated from the stable mass through slipping and rotating. The dome-shaped zone made up of these blocks is the loosened or suspended zone and corresponds to the mass of material that would be carried by artificial supports in a real tunnel. The horizontal and vertical thrusts that enable the unsupported mass to be stable are indicated with double arrows. Shearing forces are not shown, but are developed at all points of thrusting.
Collapse of the unsupported model span occurs pro-

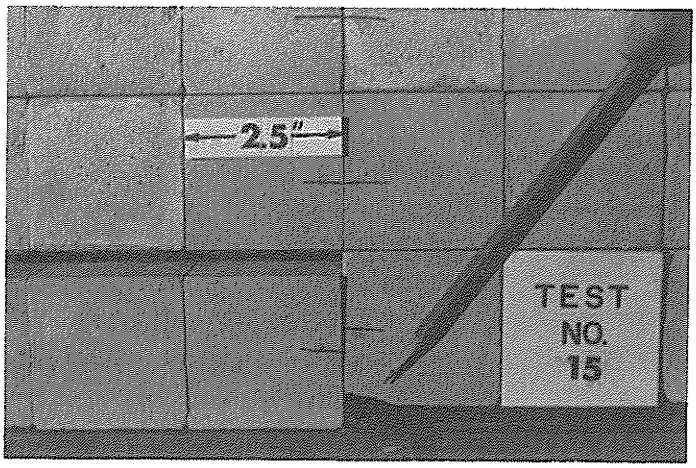


Fig. 3.8 Corner tearing or tensile failure associated with block slip

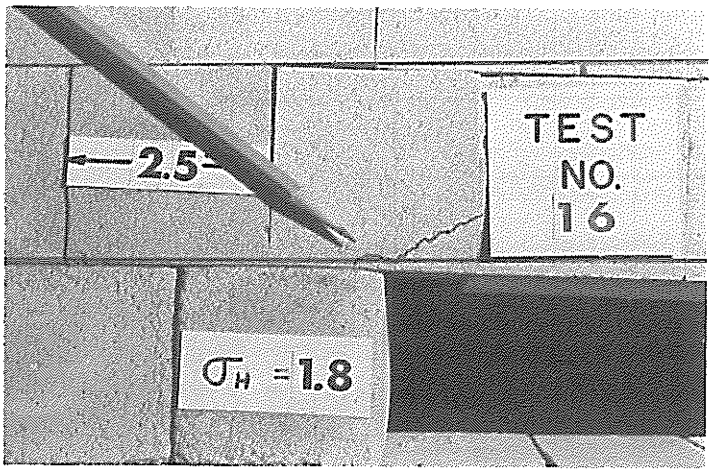


Fig. 3.9 Block corner failure in shear

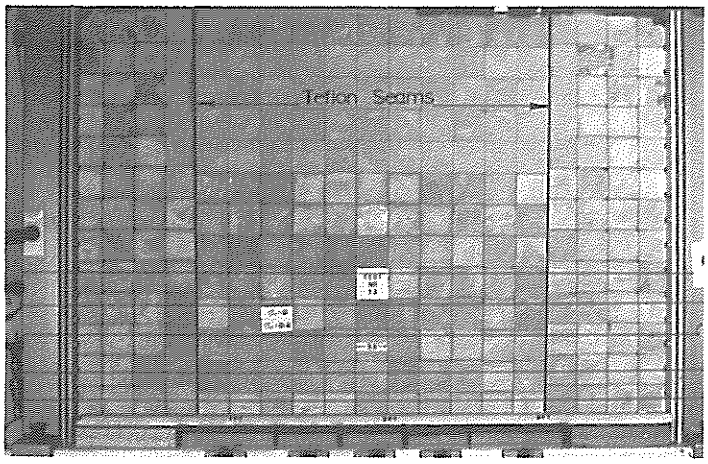


Fig. 3.10 Block mass failure by pure slip along two vertical teflon seams

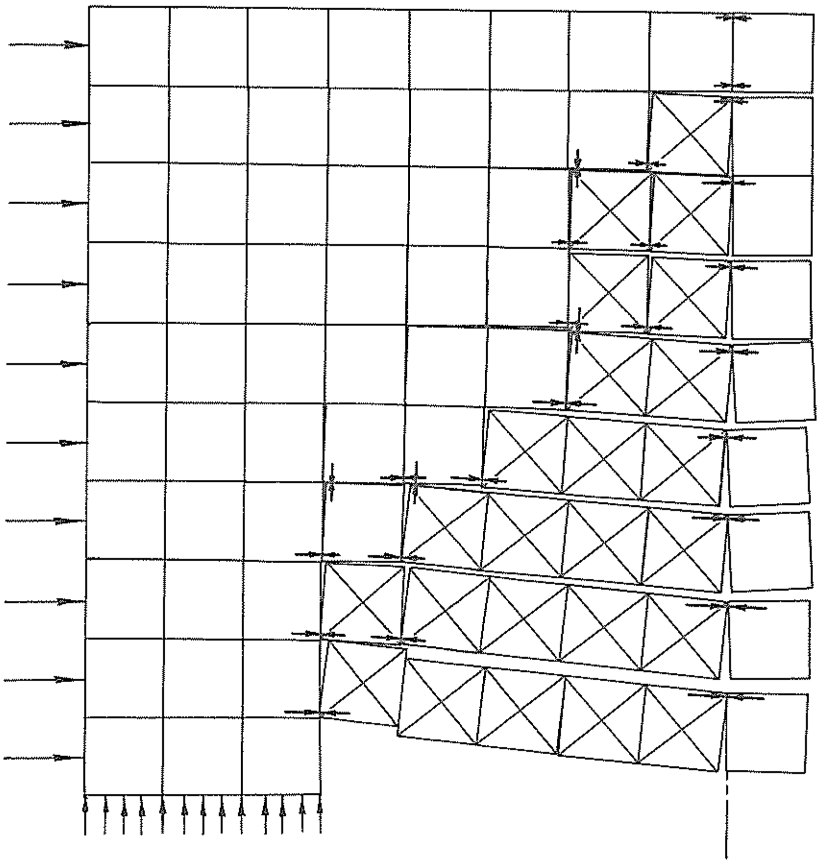


Fig. 3.11 Schematic diagram of arching in the loosened or suspended zone. Blocks with X's indicate loosened material

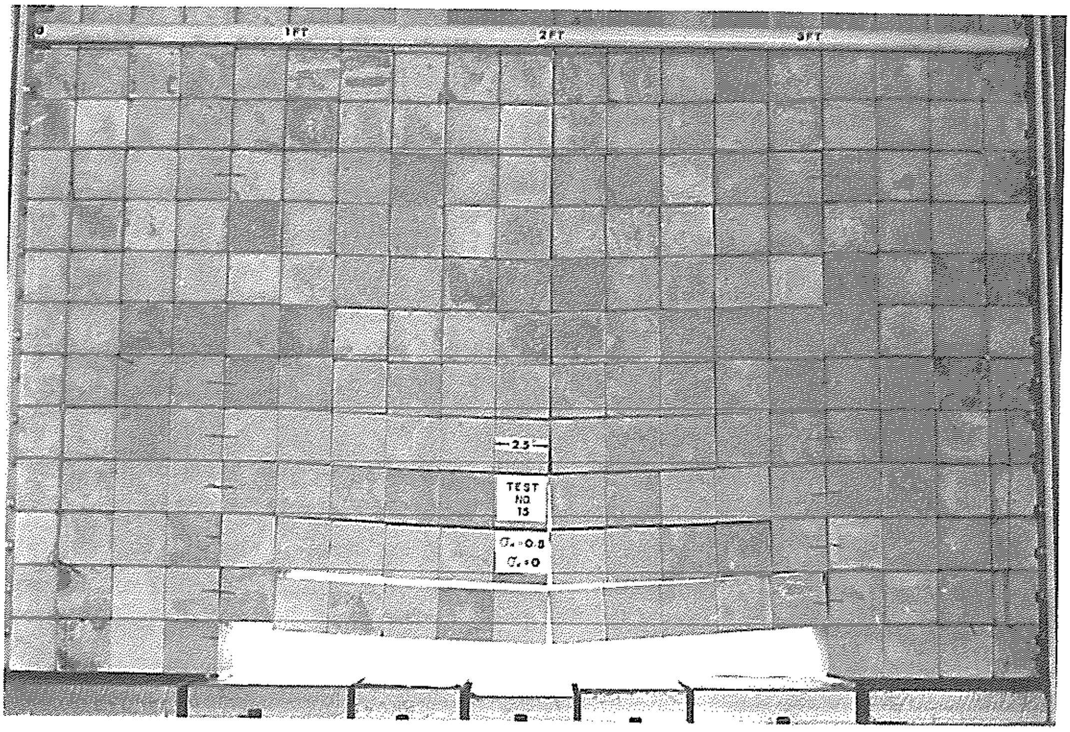


Fig. 3.12 Stabilization of an unsupported, non-imbricated block mass by arching into supported abutments. Note block slippage and rotation

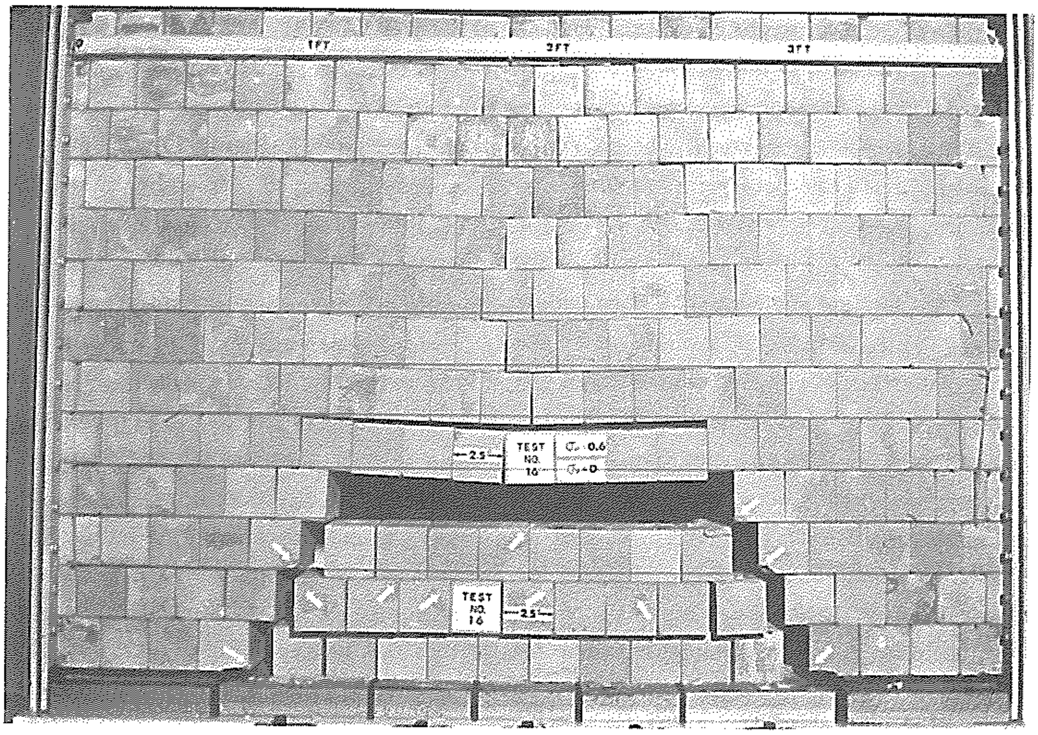


Fig. 3.13 Stabilization of an unsupported imbricated block mass by arching into supported abutments. Note block slippage and rotation

gressively as block corners fail or as individual blocks slip when the lateral pressures are reduced. That the mode of failure depends on the joint block configuration is seen a comparison of Figures 3.13 and 3.14. In an imbricated mass the fall of block rows from the unsupported span usually occurs as a result of block corner tearing, indicated by the white arrows along the edges of the abutments in Figure 3.13. All of the white arrows along the boundary of the stable mass indicate torn or tensile-failed block corners.

The collapse of the non-imbricated mass shown in Figure 3.14 occurs primarily by pure slip along block surfaces. Some tearing of block corners is associated with slip, as indicated by the three white arrows in the right hand portion of the block mass, but actual block row fall is by a sliding process.

In spite of obvious differences in mode of block fallout from the unsupported opening, the basic mechanism of arching is similar for both the non-imbricated and imbricated structures, and there does exist a strong resemblence in the shape of the loosened zone for both structures. It is interesting to note from Figures 3.12 and 3.13 that the vertical load on the bottom block row of the unsupported span reaches a value equal to the weight of a single row as the vertical deflection or sag of that row increases. This minimum vertical load is reached when the bottom row separates from the overlying rows. The gaps between such separated rows are referred to as Weber cavities (Denkhaus, 1964).

It is obvious that the load on any support placed under the lower span of Figure 3.12 or 3.13 would be a minimum if the placement of support were delayed until the Weber cavities formed. However, the inability to theoretically predict the exact displacement at which block row separation takes place, and the probable low safety factor against block row fallout that exists at deformations close to those required for block row separation preclude the use of such a "delayed support" philosophy in actual tunnel construction.

The mechanism of progressive self-stabilization through arching requires a certain degree of lateral restraint in order that shearing resistance can be developed between individual blocks. The availability of lateral restraint in a real tunnel depends on the stress distribution around the excavated opening and the strength and compressibility of the rock mass.

The arch action described in the previous paragraphs is similar to the mechanism observed by Trollope (1966) in his model studies and that obtained by Goodman (Goodman et al., 1968) in their computer analysis of Trollope's model. It is also somewhat similar to the mechanism described by Terzaghi (1943) for the behavior of sand above a yelding trapdoor.

Because an arching condition increases the vertical loads in the supported area adjacent to the unsupported mass, it follows that the strength of the abutment zones is significant, particularly for a well-developed suspended zone. According to Trollope (1966), "Deformation or failure of material in the sides of the opening can influence the suspended zone". As will be shown later, the role of the wall behavior of an opening depends to a great extent on the degree of loosening in the crown, which in turn is dependent on the magnitude of horizontal stress in the undisturbed rock.

<u>The Applicability of Deformations for a Stability Criterion</u>.

It would be desirable to be able to relate the degree of stability of a loosened rock mass to some measurable behavior. Because rock deformations are frequently measured in tunnels, this parameter is a very attractive one for consideration. The results from the model studies indicate that the magnitude of prefailure deformation normal to the periphery of the opening depends heavily on the joint structure and can vary from an imperceptible quantity for a rock structure in which the discontinuities are oriented at low angles ($<45^{\circ}$) to the perimeter of the opening (see Fig. C.12)* to as large as 1/20 of the span width for a rock structure in which the discontinuities are oriented parallel and perpendicular to the opening (see Figs. 3.12 and 3.13). It does not seem possible to establish any simple guidelines for safe or allowable deformations except to say that unfavorably oriented rock structures, such as that in Figure C.12, undergo very minute deformations prior to complete collapse whereas favorably oriented structures may undergo very large pre-failure deformations.

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* Fig. C.12 corresponds to figures in Appendix C.

Validity of Model Mechanism in Real Rock Masses

Although real rock masses are seldom jointed in the regular manner of the model, it is very likely that the behavior of any jointed medium is governed by the basic mechanisms fiscussed for the model. Deere, Peck, Monsees, and Schmidt (1969) discuss several basic behavior patterns for the rock-shotcrete interaction in jointed rock. Their suggested mechanism for progressive failure through rock block movements and rotations is not unlike the observations from the model tests.

The behavior of the suspended or loosened zone in the writer's model is very similar to the behavior hypothesized by Denkhaus (1964) for the fractured rock within the "voussoir" (French for brick) arch that forms as a result of intact rock failure in the crown of a tunnel or cavity, as shown in Figure 3.15. According to Denkhaus, the fractured rock mass in the dome "... tends to sag under its own weight until the irregularly shaped blocks of which it consists wedge between each other and against the solid boundary of the dome. Consequently the vertical downward forces of the weight of an individual block may be resolved into components causing thrust between the blocks and against the dome boundary and components tending to shear blocks out of the arch system formed by the thrust. The shear is resisted by the friction between the blocks which in turn is increased by the thrust so that the system remains stable."

Because very few cases in the field were observed during the actual failure process, it is only possible to hypothesize on the real failure mechanism. However, it is quite obvious that many roof falls take place strictly because of a lack of confinement at their boundaries, which in turn can be attributed to the orientation of the principal discontinuities.

Cases 10, 32, 58, and 86 are examples of such behavior. Rock block rotation and corner crushing are very unlikely in such cases. The wall slip-out in Cases 11, 13, and 15 similarly is a rather simple form of failure and can be explained purely on the basis of sliding along single discontinuities.

The significance of block corner failures is not readily apparent from any of the observed failures. It is practically impossible to determine if corners fail under in-situ rotation or when the blocks land on the tunnel floor. Because of the probable absence of block rotation in many of the roof and wall falls, it is unlikely that block corner failure had a large influence on the majority of observed cases. The high strengths of the rocks in which the observations were made can be considered a beneficial property, as they reduce the possibility for corner failure to occur. Because the stresses responsible for corner failure result from the dead weight of loosened rock material, the most likely occurrence of corner failure in any of the observed cases is in those instances where large volumes of loosened rock exist in the roof of a tunnel.

There is no reason not to believe that joint surface irregularities and asperites would not behave in essentially the same manner as block corners, i.e., they probably are points at which loads are concentrated when rotation and slippage occurs in the mass. As long as they remain intact, they contribute to the strength of the mass. Their failure could be expected to lead to loosening and possibly even failure of the mass.

Because it is impossible to determine the stress conditions in a rock mass by visual observation, the existence of true arching behavior in the field can only be implied. However, it is evident that the stability of many of the observed tunnels in heavily jointed rock is dependent on the existence of either high tangential crown stresses or significant arching action. No other explanation can be given for the stability of the tunnel crowns in the rock conditions shown in Cases 35, 52, 67 and 92. Because no signs of large rock movements, such as very open joints across the crown of a tunnel or sagging tunnel crowns, were observed, there is reason to believe that high tangential crown stresses account for the stable tunnels in jointed rock.

It is thus seen that the value of the model tests lies as much in understanding the behavior of stable tunnels as in understanding the mechanism of failure in unstable tunnels.

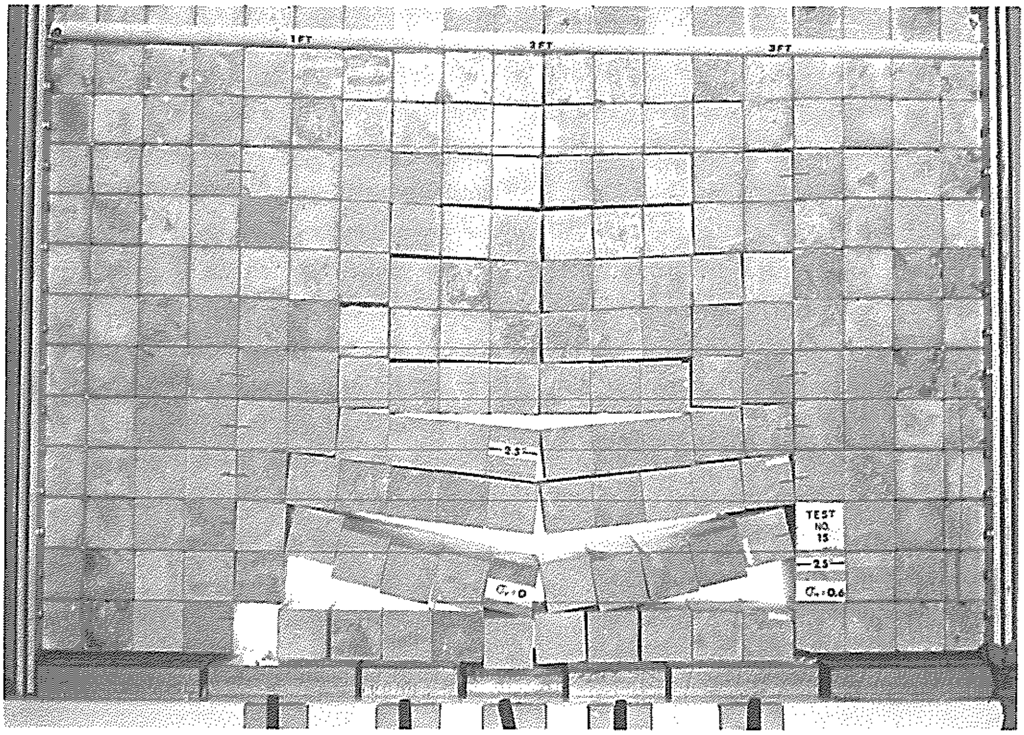


Fig. 3.14 Collapse of an unsupported non-imbricated block mass.
Note scaricity of corner failures; main failure mode is block slip

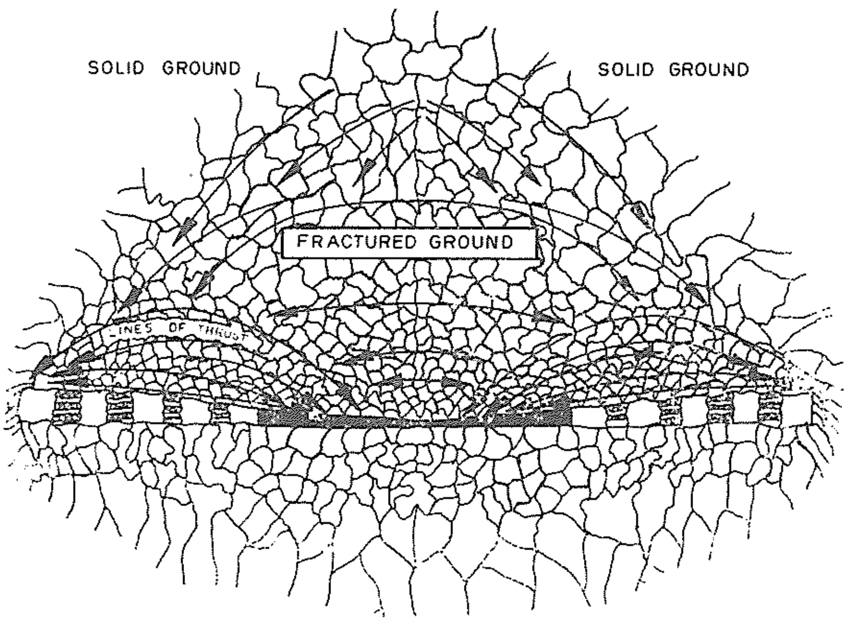


Fig. 3.15 Concept of "Voussoir" arch in mine stope
(after Denkhaus, 1964)

3.4 Factors that Influence Loosening Instability

From the model studies and from the field observations it is apparent that a very large numer of factors influence the stability of a tunnel in jointed rock. These factors include the following:

(1) Intact rock properties
(2) Rock discontinuity properties (including joint filling material)
(3) Groundwater
(4) Structural arrangement of geologic discontinuities and weak zones
(5) Magnitude of horizontal and vertical stress in undisturbed rock
(6) Shape of opening
(7) Ratio of joint spacing/span width
(8) Time-dependent variations of any of the above factors
(9) Tunneling techniques

Variations in any of these factors can change the stability of an opening.

Intact Rock Properties

The significance of intact rock properties is obvious from considerations of block slip and block rotation in the preceding section. Because of the possible importance of block corner failure in governing the stability behavior, the intact tensile and shear strength properties may be very significant. These parameters were not varied in the model tests, but it is imaginable that rock blocks with a very low shear and tensile strength would fail readily at corners and impair the development of arch action. Compressibility of the intact rock has not been discussed, but, within the limits of sound igneous and metamorphic rocks, probably has a small significance in governing rock mass stability behavior.

Evidence of the influence of intact rock properties on tunnel behavior under loosening conditions is not available from the field cases. None of the observed roof falls can be attributed to corner failures of the intact rock blocks. Although some broken rock block corners were observed in the rubble found beneath many of the observed falls, it is not certain whether these corner failures occurred as a result of blasting, in-situ crushing, shearing, or tearing associated with joint block movements in the loosened zone, or impact loading when the blocks hit the tunnel floor.

Because most of the unaltered intact rock materials encountered in the observations have very high strengths, the only likely occurrence of block corner failures in the observations is in those cases that involve altered rock material. However, the falls in all of those cases can be explained by factors other than intact block corner failure due to stress concentrations brought about by block rotation.

Because all discontinuous rock masses are not as regularly jointed as the model, it is not likely that block slip and rotation and corner failure in real rock will occur in a manner identical to that in the model. However, it is likely the same submodes of block corner failure (crushing, tearing, shearing) would occur in any jointed rock mass that contains sharp joint block corners, asperities along joints, or other points where high local stress concentrations may exceed the strength of the intact rock material.

Low intact rock strength may also affect tunnel stability by leading to failure of intact rock material during or after blasting. Such behavior is believed to be at least partly responsible for the wall fallout in Cases 4 and 11.

Rock Discontinuity Properties

Rock discontinuities include all the two dimensional, zero-tensile-strength features in a rock mass. The discontinuities encountered in the observed cases are summarized in Table 3.1. The properties of each of these features vary widely.

TABLE 3.1
TYPES OF DISCONTINUITIES IN FIELD CASES

Feature	Unsupported Cases[1]	Supported Cases[2]
Joint	14	62
Bedding plane	--	6
Cleavage or schistosity	2	1 10
Fault, shear, sköl	--	45
Combinations	1	47

1/ Cases in which support was not used
2/ Cases in which support was used

Rather than attempt to discuss the influence that each of the features has on tunnel stability, the properties and influence on stability of all discontinuities will be treated collectively in a discussion of joints.

Goodman et al., (1968) discuss extensively the influence of joints on tunnel stability. In their finite element analysis of jointed rock they found that the following parameters best characterize the "potential behavior" of joints: the unit stiffness across the joint; the unit stiffness along the joint; and the shear strength along the joint (described by a cohesion intercept and a friction angle). Analyses of the stress around a circular opening in several simple joint systems show the great difference in behavior when these three joint properties are varied.

Although no model tests were conducted by the writer in which the frictional resistance of all the blocks was varied, the tests conducted with the vertical and horizontal teflon seams (see Figs. C.8, C.9 and C.10) do indicate the effect of low joint friction on stability. The test results shown in Figures C.15 and C.16 indicate that vertical surfaces of low friction dractically reduce block row bending and induce premature slippage at relatively high values of lateral stress.

A single horizontal teflon seam (see Fig. C.17) significantly reduces the stability of block rows, as the shearing resistance along the horizontal joints, particularly near the abutment zones, effectively resists the bending of block rows. Loss of this resistance increases the stresses on block corners. This fact is evidenced by the crushed block corners seen in Figure C.17d that led to the collapse of the block rows located beneath the low friction seam.

It should also be mentioned here that the condition of joint block corners influences the development of arching action. Early model tests showed that an unsupported mass composed of blocks with rounded corners undergoes considerably greater deformation and block rotation prior to the development of a stable arched condition than does a mass composed of blocks with sharp, square corners. Furthermore, the suspended or loosened zone for a mass of rounded-corner blocks is larger than that for one of square-corner blocks.

The influence of low joint strength (caused by clay and other joint fillings) is plainly evident from the field cases. Cases 13, 33, and 76 are three instances where low joint friction was associated with wall and roof fallout. From Table B.1 in Appendix B it is seen that clay or other low friction joint filling or coating materials were present in 52 of the 74 observed cases in which support was used. Sandy or gravelly material or rock fragments were present in another 11 of the supported tunnels. Thus joint fillings of some type are present in 63 of the 74 observed, supported tunnels. Chemical alteration along joints occurred in 21 of the 74 cases and was responsible for one or more of the above named joint fillings.

Joint fillings not only generally reduce the friction along joints, but also usually give the rock mass a lower compressibility and, hence, result in greater deformations around an opening. They simultaneously lead to greater loosening than occurs when joints are unfilled and tight. Rock masses with filled joints are probably loose even before an opening is blasted in them. Relatively "open" joints were observed in 55 of the 74 supported tunnels and in only one of the 16 unsupported tunnels. Relatively "tight" joints occurred in 19 of the 74 supported tunnels and 15 or the 16 unsupported tunnels. It is thus plainly evident that a tight rock structure is a necessary, but not sufficient, condition for a stable opening.

The degree of discontinuity roughness for all of the cases has been visually classified. A summary of the observations is given in Table 3.2. The most significant conclusion that can be drawn from this data is that slickensided discontinuities are consistently related to supported tunnels. The positive correlation can be attributed not only to the low friction along slickensides that are frequently clay-coated, but also to the loose rock structure that usually results from the tectonic motions that develop slickensided surfaces.

TABLE 3.2

DEGREE OF DISCONTINUITY ROUGHNESS FOR ALL FIELD OBSERVATIONS

Description	Unsupported Tunnels	Supported Tunnels
Slickensided	--	45
Smooth	9	37
Rough	7	11

The continuity of joints and other rock discontinuities determines to a certain extent the influence that these features have on tunnel stability. The observed joint continuity is summarized in Table 3.3 Most of the supported tunnels involve continuous or through-going discontinuities. A significant portion of the unsupported tunnels involve discontinuous joints or combined sets of discontinuous and continuous joints.

Discontinuity planeness is summarized in Table 3.4. The only significance that can be attached to any particular degree of planeness is that most of the irregular discontinuities are undulating, slickensided shear planes that are frequently surrounded by loose rock that requires support. Furthermore, such slickensides normally contain clay joint fillings or coatings.

TABLE 3.3

DEGREE OF JOINT DISCONTINUITY

Continuity Description	Unsupported Tunnels	Supported Tunnels
Continuous	5	56
Discontinuous	4	4
Combined	7	14

TABLE 3.4

DEGREE OF DISCONTINUITY PLANENESS

Description	Unsupported Tunnels	Supported Tunnels
Plane	12	50
Curved	--	3
Irregular	4	24

Ground Water

Ground water can have an adverse effect on stability by reducing both the strength of rock materials (physical-chemical effects) and the strength of the rock mass (development of high pore pressures). The failure shown in Case 43 is an example where water significantly reduced the strength of a shear zone by causing swelling in montmorillonitic clay seams. Swelling and softening also result in reduced frictional resistance. Such behavior contributed to instability in most of the 16 cases in which softening clays were present.

Although no observed failures are specifically related to joint water pressures, it is very possible that such pressures contributed to instability in some of the rock shown in Cases 18, 43, 79 and 85.

Ground water may also have a purely hydraulic effect by washing out joint filling materials through piping action. This action contributed to the conditions shown in Case 45, 54, 56, and 57.

Structural Arrangement of Rock Discontinuities

The structural arrangement of rock discontinuities and weak zones includes both the number and orientation of discontinuity sets with respect to the periphery of the opening and the intersection geometry of discontinuities. These items account for the largest portion of the observed failures.

One very critical factor with respect to rock structure is the number of discontinuity sets in the rock mass. The structures observed in the field range from massive rock with no joints to completely crushed and earth-like materials. The distribution of structure types for all the field cases is shown in Table 3.5.

The term "random discontinuity" is used to denote one or more single, randomly oriented joints that do not belong to a set and intersect any regular sets that may exist. The condition of two discontinuity sets with a randon discontinuity is shown in Figure 3.16.

TABLE 3.5

ROCK MASS STRUCTURES IN FIELD OBSERVATIONS

Rock Mass Structure	Supported	Unsupported
Massive	--	4
One discontinuity set	14	10
One discontinuity set with random discontinuity	9	1
Two discontinuity sets	13	2
Two discontinuity sets with random discontinuity	20	--
Three discontinuity sets	10	--
Three discontinuity sets with random discontinuity, or more than three discontinuity sets	2	--
Crushed or earth-like rock	4	--

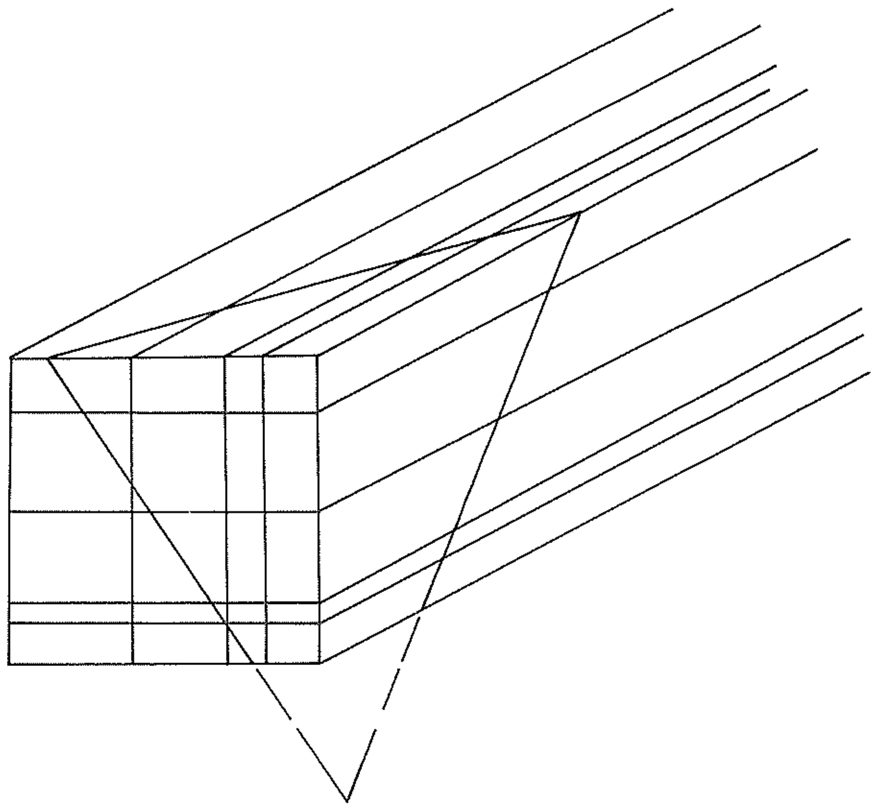


Fig. 3.16 Two discontinuity sets with a random discontinuity

It is readily apparent from Table 3.5 that cases involving multiple discontinuity sets are most often associated with support. Single sets of discontinuities are frequently of no cencern to the stability of an opening, as indicated in Cases 20 and 52. The relationship, or lack of relationship, between rock mass structure and the support used in the observed cases is discussed in Chapter 5.

Although the basic structure of the laboratory model was the same for all tests, several tests were conducted to show the effect of a random discontinuity on the basic two-dimensional joint structure used in all of the tests. The results shown in Figure C.19 indicate the very drastic effect that a single random joint may have on an otherwise stable block mass.

The orientation of discontinuities with respect to an opening determines to a large extent the likelihood of loosening and falls in the walls and roof of a tunnel. The dips and strikes (the latter with respect to the tunnel axis) of the discontinuities associated with the observed cases are summarized in Table 3.6. The following conclusions can be drawn from this table:

(a) Most of the observed cases involve single or multiple sets of discontinuities that dip in excess of 30°. Steeply dipping (60-90°) discontinuities are the most frequent single group.

(b) Of those discontinuities associated with unsupported tunnels, steeply dipping (60-90°) are the most common. Single sets of medium (30-60°) to steeply (60-90°) dipping joints are very frequently insignificant with respect to the stability of an opening, as evidenced by Cases 20 and 52. The model tests also demonstrate that steeply dipping joint sets (90°), and even random discontinuities that form a steep angle ($\geq 60^{\circ}$) with the opening, as in Figure C.18a, do not have any effect on the stability of an unsupported mass, provided the joints are tight and unfilled.

(c) The most common joints that cause tunnel wall instability are those that dip in excess of 30°. This fact is evidenced in Cases 10-15.

(d) Roof instability occurs over the entire range of discontinuity dip (0-90°). From both the model tests results shown in Figure C.19 and Cases 47-51 it is very obvious that low angle wedges of blocks will very likely fall from the periphery of an unsupported opening, as they are unable to transfer any confining stress across their boundaries.
The stabilizing effect of confining stress is clear from the model studies in which the discontinuities are oriented at steep angles ($>60^{\circ}$) to the opening periphery.

TABLE 3.6

ORIENTATIONS OF MAJOR DISCONTINUITY SETS IN OBSERVED CASES

Number of Cases with Indicated Support for Different Dips and Strikes

Support	Dips of Major Discontinuity Sets 0-30°	0-30°, 30-60°	30-60°	30-60°, 60-90°	0-30°, 60-90°	60-90°	All
None	1	--	3	--	2	10	--
Wall	--	--	2	3	4	5	--
Roof	5	1	10	7	12	9	1
Both	--	--	--	4	6	3	--

Support	Strike (from tunnel axis) of Major Discontinuity Sets 0-30°	0-30°, 30-60°	30-60°	30-60°, 60-90°	0-30°, 60-90°	60-90°	All
None	5	--	5	--	--	6	--
Wall	5	4	4	1	--	--	--
Roof	16	6	4	1	9	10	1
Both	--	2	1	4	2	2	2

(e) Single or multiple sets of discontinuities that dip less than 60^{o} are mot likely to be associated with roof instability. For any given dip in excess of 60^{o}, it is not possible to localize the most likely location on the tunnel periphery for instability to occur. Steeply dipping joints may cause no difficulty what-so-ever, or they may cause fallout in both the roof and walls of an opening.

(f) No particular strike or combination of strikes can be associated with any particular stability behavior. However, the chance that a discontinuity set striking sub-parallel (0-30^{o}) to the tunnel axis will lead to instability are greater than that for discontinuity sets that strike normal to the tunnel axis.

Magnitudes of Horizontal and Vertical Stresses

The magnitudes of the horizontal and vertical stresses in the undisturbed rock determine to a great extent the stress distribution around an opening of a given shape. Ratios of horizontal to vertical stress (K_o) that tend to create zones of tension in the crown of an opening are very unfavorable.

From theoretical analysis it can be shown that, for an opening of any given shape, an increase in the value of K_o increases in a compressive sense the stresses around the crown of an opening. Thus, high values of K_o tend to stabilize the crown if the jointing is steeply oriented with respect to the perimeter of the crown. The beneficial effect of high tangential crown stresses is one of added lateral restraint and greater resistance to block rotations and block row bending, less concentration of lateral thrusting at block corners, and a more even distribution of shearing stress along vertical joints.

The stabilizing effect of high stresses parallel to the periphery of an opening in a block mass whose joints are normal and parallel to the opening perimeter is obvious from a comparison of the two photographs in Figure 3.17a and 3.17 b.

The effect of lateral stress on the development of the loosened or suspended zone is seen in a comparison of Figures 3.18b and 3.19b. In Figure 3.18b the block mass is subjected to a lateral stress of 2.3 psi and there is no block rotation, block row bending, or developed loosened or suspended zone (shown schematically in a real tunnel in Fig. 3.18a). In Figure 3.19b the pressure had been dropped to 1.3 psi and as failure occurred, the mass tended to dilate under a condition of fixed lateral restraint, increasing the lateral pressure to 2.5 psi as the loosened or suspended zone developed.

The sequence of photographs in Appendix Figures C.13 and C.14 demonstrates further the influence of the confining stress on stability over the range of pressures associated with a completely stable unsupported span to a totally collapsed span.

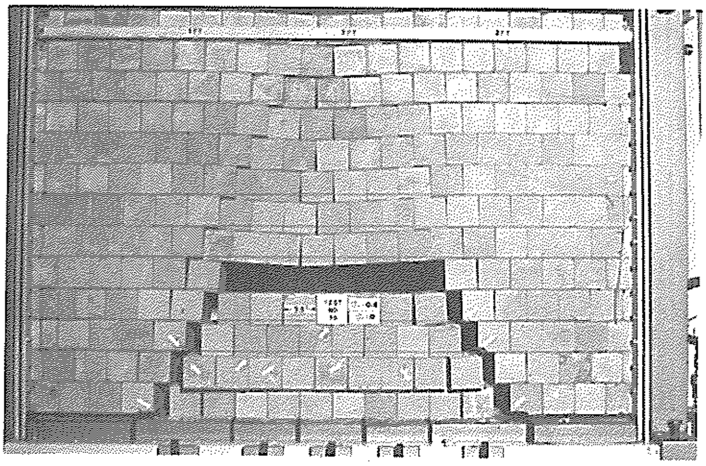
In-situ measurements of rock stresses around tunnels in Sweden have been made by two investigators (Hast, 1958, 1965; and Hiltscher, 1967, 1968). Hast's measurements have been made with his original wedged bore hole inclusion gage. Hiltscher employs overcoring of strain gages glued to the bottom of a drill hole, a modification of the South African doorstopper technique. Hast's measurements have been performed in holes drilled out from a tunnel, into the undisturbed rock. Hiltscher has also measured stresses away from the tunnel periphery, but some of his reported values are calculated from stresses measured at the periphery of the opening.

The results from the work of these investigators give, at best, a crude insight into the stresses around tunnels. The following conclusions can be drawn from the measurements made to date:

(1) Results are often widely scattered and contradictory.

(2) The stress distribution around a tunnel determined by measurements of strain at points in a drill hole is frequently very erratic, particularly in jointed rock.

(3) Reported values of the ratio of horizontal stress to vertical stress (K_o) in the undisturbed rock vary from 1 to 8. Most reported values fall in the range 1.5-3.5.



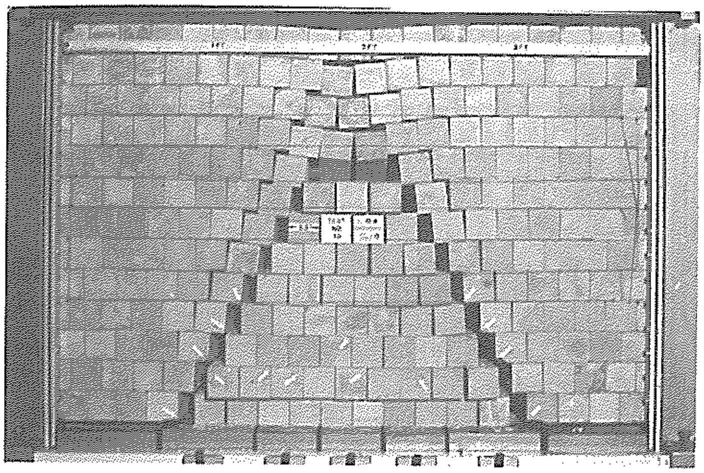
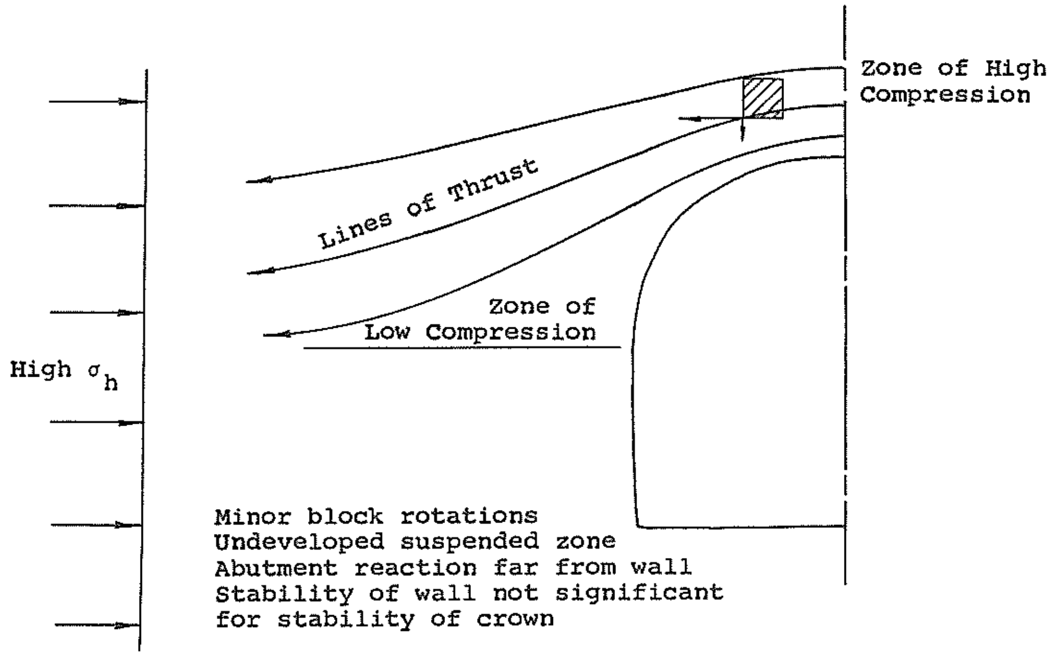
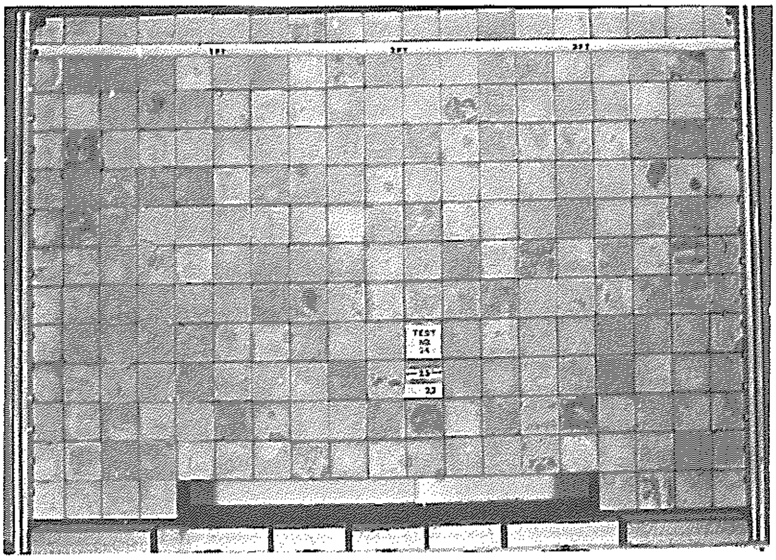


Fig. 3.17 Influence of lateral pressure on stability of an unsupported block mass

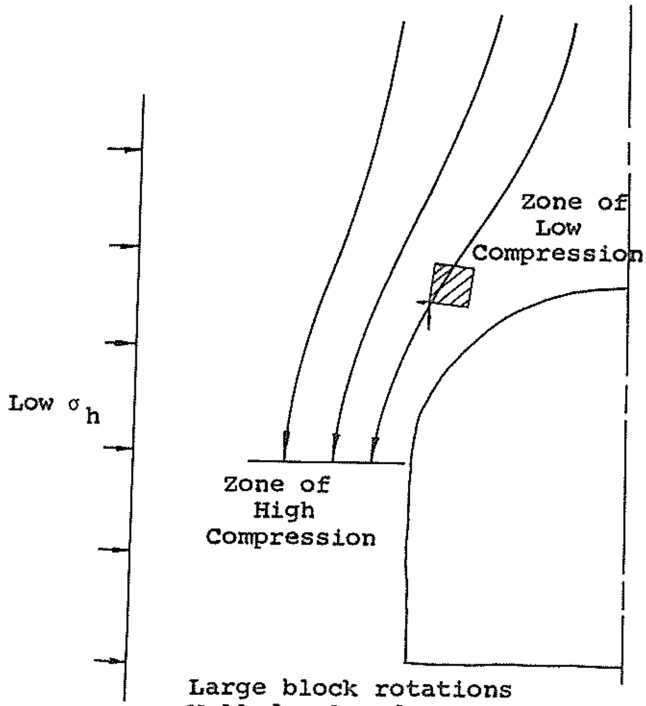


(a) Hypothesis for Real Tunnel



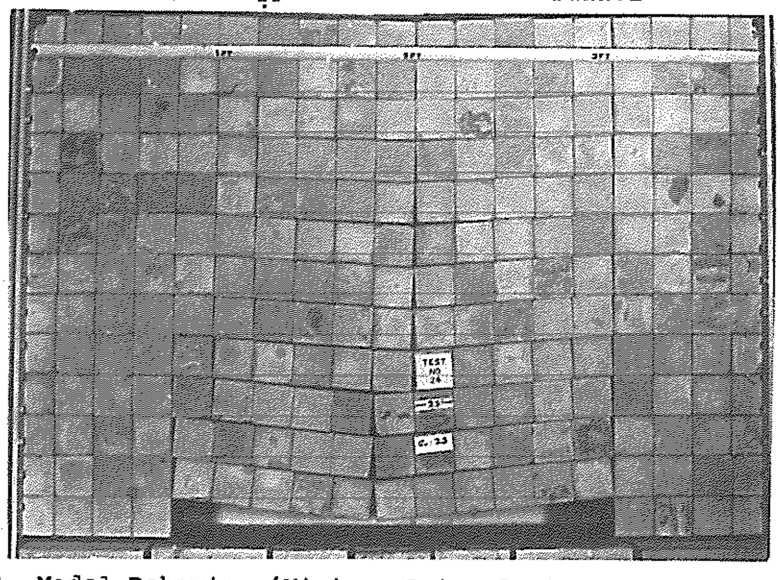
(b) Model Behavior (Minimum Lateral Stress = 2.3 psi)

Fig. 3.18 Tunnel crown behavior under high lateral stress (compare with Fig. 3.19)



Large block rotations
Well-developed suspended zone
Abutment reaction close to wall
Failure of Wall critical for crown stability

(a) Hypothesis for Real Tunnel



(b) Model Behavior (Minimum Lateral Stress = 1.3 psi)

Fig. 3.19 Tunnel crown behavior under low lateral stress (compare with Fig. 3.18)

(4) Very high stresses parallel to the walls and roof may exist around an opening in jointed rock. Hiltscher (1967) has reported tangential crown stresses of over 3000 psi at a distance of less than one meter from the periphery of a large headrace tunnel in heavily jointed rock (Vietas Hydroelectric project, Satisjaure head race tunnel, see Cases 31-37, Fig. 2.6). This magnitude of stress is very surprising in consideration that the rock cover at the measurement station is only 300 feet.

There are two significant geologic factors that are ample reasons to suspect that large horizontal stresses may exist in the Swedish bedrock, particularly near the mountain ranges in the northwestern part of the country.

(a) All of the mountain chains in northwestern Sweden are overthrust nappes. Most of the tunnel projects in northern and western Sweden are located near very major overthrusts.

(b) Geologists claim that the last continental glacier receded from northern Sweden about 8000 years ago (Lundegårdh et al., 1964). It is possible that there exist today in the bedrock of northern Sweden large residual horizontal stresses that have remained "locked" into the bedrock since the retreat of a 2-km-thick ice sheet only 8000 years ago. This possibility is substantiated by the relatively large land uplifts (10 mm/year) that are taking place at the present time in northern Sweden.

The reported stress measurement data on which conclusions (3) and (4) are based suggest that the behavior of some of the observed cases in the writer's work may be strongly influenced by high stresses around the opening, particularly in the crown. The instances of stability in jointed rock tunnels that are described in Cases 6, 20, 35, 36, 52, 70 and 83 can be explained very readily in terms of high compressive crown stresses.

Shape of Opening

The shape of an opening strongly influences the stress distribution around the opening. Shapes that result in tensile zones are particularly unstable. With a few exceptions the shapes of the tunnels studied do not deviate significantly from equidimensional openings, and no particular trends in stability behavior can be attributed to tunnel shape. The only consistently observed behavior associated with tunnel shape is the tendency for a rounded intrados in a tunnel in blocky rock to become squared as fallout occurs. This observation is illustrated in Cases 32, 33, 63, 64, and 65.

Joint Spacing/Span Width Ratio

The ratio of joint spacing to span width is important for two reasons. In the model studies it is shown that a small ratio is more conducive to large joint block rotations and vertical deformations, whereas large ratios permit only very minor pre-failure bending deformations (see Fig. C.20). In the latter case failure occurs by a punch shear collapse of a chimney-shaped block mass in contrast to the corner crushing and tearing that leads to collapse of a wider opening in the same medium.

The more significant effect of a large span width in a heavily jointed rock mass (low ratio of joint spacing/span width) is that there exists a greater chance that an unfavorably oriented discontinuity will be intersected (Cording, 1968b).

The relationships between tunnel span width, rock quality (as measured by RQD) average joint spacing, and required support measures are presented and discussed in Chapter 5.

There is no apparent relationship in the field data between tunnel span width and frequency of unstable behavior. Oddly enough, the behavior of tunnel walls seems to be more related to wall height than the behavior of tunnel crowns is to span width. High vertical tunnel walls (>4 m) very frequently pose stability problems where a span width of the same dimension in the same rock is stable. This observation can be made in Cases 4, 10, 11, 14 and 15. The obvious explanation for this behavior is that steeply dipping discontinuities (dip $> 45^{\circ}$), which are very common in the Swedish Precambrian bedrock, are generally of minor significance in the crown of an opening.

The fact that none of the observed wall failures led to crown failures indicates that abutment reactions due to arching probably do not exist in the immediate periphery of walls, as suggested schematically in Figure 3.19a, but rather are located at a large distance from the walls, as shown schematically in Figure 3.18a. Such behavior implies the existence of large horizontal stresses.

It is apparent from the field cases that the total volume of overbreak increases as tunnel size or span width increases. This is to be expected, even without consideration of joint spacing or rock quality.

Time-Dependent Variations

Time-dependent variations of any of the factors discussed in this section will change the stability of an opening. Changes in intact rock and rock joint properties with time are the most common changes encountered. Such changes are most likely to be caused by air or water deterioration. The creep behavior of a jointed rock mass is not well understood, but is a factor that could change the stability of an opening.

Significant changes in tunnel stability may occur as a result of readjustment of stresses in the walls and crown of a tunnel as the face is advanced. In addition to the adjustments that can be predicted from elastic theory, there are numerous geologic and construction factors that influence the time that is required for rock mass stabilization. Abel (1967) indicates that the following factors influence this time: (1) the percentage of alteration, (2) the relative water condition, (3) the rock type, (4) the relative degree of faulting and shearing, (5) the thickness of the nearest fault zone, and (6) the average tunnel advance rate during the period preceding stabilization.

The only time-dependent variations in stability that were observed in the field cases were those associated with water absorption and subsequent softening of some clay materials, as described in Cases 40-43, 45, 54, 71-73, 93, and 94. It is interesting to note that most of the cases in which supports actually failed (time-stability-support classification H) were cases that contain some form of softening clay.

Tunneling Practices

The effect of tunnel excavation techniques on stability will be discussed in the next chapter. It will be shown that a number of different construction factors, including the experience of the tunneling crew, have an influence, either directly or indirectly, on the stability of a tunnel.

4. SWEDISH TUNNELING PRACTICES AND THEIR INFLUENCE ON TUNNEL STABILITY

4.1 Introduction

The purpose of this chapter is to describe Swedish tunneling practices and their influence on the stability of tunnels. Although it is not possible to quantitatively evaluate the influence of these practices on the stability of a tunnel, the significant role that tunneling practices have in determining the stability behavior and support requirements of an opening in rock cannot be ignored. This is particularly true if an attempt is going to be made to extrapolate the Swedish experience reported in this work to tunnels in other countries that may be driven under completely different economic and constructional practices.

The factors in Swedish tunneling practice considered to be most influential in tunnel stability are discussed under the broad topics of general tunnel construction industry; tunneling methods; blasting techniques; and rock support and reinforcement techniques.

4.2 General Tunnel Construction Industry

The most significant aspects of the Swedish tunnel construction industry that influence the manner in which tunneling work is performed -- and indirectly the stability of rock openings -- are the broad experience accumulated in the past 30 years, the labor situation, and contracting practices.

Experience

There is no question as to the value of practical experience in tunneling. Because of the lack of any specific design procedures for rock tunnels and because of the impossibility of making reliable predictions of rock conditions in every round of a tunnel, the burden of coping with conditions as they are encountered in a tunnel during construction rests entirely on the shoulders of those people responsible for the construction of the tunnel. Matthias (1968) states that, "No matter what degree of control is provided in the (contract) specifications, the final decision on the amount of support required rests with the foreman and the miners and not with the design engineer." Time of support placement, as well as amount of support, and variations in blasting techniques to meet local geologic changes are also critical decisions that rest with the tunneling crew. Obviously, an experienced tunneling crew is of extreme value, particularly when adverse conditions are encountered.

Within the past 30 years rock tunneling in Sweden has become one of the largest and most skilled areas in construction. The many hundreds of kilometers of tunnels driven in connection with hydroelectric development, transportation, underground storage and industry, and civil and military defense have resulted in an accumulation of experience in tunneling that is equalled by only a few other countries in the world. Most significantly, this experience has been concentrated in the hands of a relatively small number of contractors, engineers, and miners. Probably no more than four to six contractors have been responsible for most of the tunnels driven by private groups. The Swedish State Power Board has constructed a very large portion of the hydroelectric developments. Thus, when a new tunnel project is started in Sweden, there is a very great likelihood that the people responsible for all phases of the work are very experienced. Such was the case in most of the tunnels in which the writer made observations.

Of equal importance to successful tunneling as the knowledge of past practices and experiences is a willingness among most Swedish tunneling people to accept new methods and practices. Their rapid adaptability to new technology has lead to significant innovations and improvements in rock drilling, blasting, support and other areas of less obvious importance, such as muck removal and underground safety. Smooth wall blasting and the ROBOT shotcrete equipment are two of the more significant contributions that the Swedes themselves have made to rock tunneling. As will be discussed later, both of these items influence the stability of tunnels.

The Influence of Labor

The value of a highly skilled and experienced mining crew has been pointed out in previous paragraphs. The skill and experience of Swedish tunnel labor is enhanced by several factors. Most importantly, Swedish laborers are very job-oriented, and their per capita output would shame their counterparts in many parts of the world. The successful driving of a tunnel, particularly the support of the tunnel, depends heavily on personal craftsmanship and attention to details, and hence on the abilities and attitudes of individual miners and their foremen. Because of such experience and job-oriented attitudes, set-up times and break-in times on new projects are a minimum.

Most Americans are very surprised when they walk to the heading of a large rock tunnel in Sweden and find only six men carrying out the drilling, blasting, and mucking cycles. The absence of labor union influences is apparent in other aspects of Swedish tunneling. Not only do the same miners frequently perform all the cycles of tunneling, and not only is their number limited to that necessary to do the work, but also there is a very small likelihood that work will be interfered with by strikes about some petty grievance. The smoothness of the tunneling operations and lack of potential disturbances due to strong union influences preclude many delays and interruptions that could be costly in terms of delayed support installation or other stability maintenance measures. Furthermore, labor unions do not specify which type of support may or may not be used on a job, and there is no resistance against the introduction of new, man-replacing devices and methods in rock tunneling. Thus, advances in tunnel technology that might improve the support of a tunnel are not hindered in any way by union action.

Contracting Practices

Contracting practices themselves would not appear to have any direct effect on the stability of an opening. However, the manner in which a contract is written may have an effect on the manner in which a tunnel is supported, which in turn has a direct effect on stability behavior of the tunnel. If a contractor receives a profitable unit cost for support, he is likely to use much more support than if he has underbid a job and is losing money. Or if a contractor is behind schedule and is not pressured by the owner to use support, his neglect or postponement of critical support installations may result in deterioration of a tunnel. Hence, it is important to understand the contracting practices and legal conditions under which tunnels are driven.

Tunnel construction in Sweden is carried out under three different types of contractor-owner relationships. Most of the projects of the Swedish State Power Board have been constructed by the Board itself, and do not involve private contractors. In very recent times somre very specilized types of work such as muck hauling and shotcreting have been let to private contractors. Private hydroelectric projects which constitute about 40 to 50 percent of the total hydroelectric development in Sweden, are usually constructed by private contractors. The two contractor-owner relationships in the privately constructed projects are those contracts based on a unit price agreement and those based on a cost-plus agreement. The unit price contracts are the most common type.

Owner Constructed. The primary feature of significance with regard to support and stability in projects that are constructed by the owners is that there exists no outside control or inspection of the work. Support needs are determined by the mining crew, the foreman, and the project tunneling engineer. Because the tunneling crew may receive an incentive pay on the basis of extra footage, thay are very likely to be most concerned about drilling, blasting, and mucking and may tend to neglect support work unless conditions are so bad that safety becomes a concern. The responsibility for supporting bad ground thus rests with the tunneling foreman and, more often, the project tunnel engineer. Whether a tunnel is over-supported or undersupported will thus depend to a large extent on the experience and interests of one or two men. Their decisions are based on a number of factors, but the time scheduling of support work is likely to be a critical factor. In the writer's experience, those projects in which time scheduling of tunneling operations was poor received the poorest attention to support work. Delays in operations due to equipment breakdown, ventilation difficulties, accidents, and other unscheduled events are frequently responsible for neglect of support work. Although such difficulties can arise on any project, even where a private contractor is performing the work, their effect on the stability of a tunnel is likely to be greatest where there is no outside pressure to maintain necessary support work.

Unit Price Contracts. When private contractors are engaged to construct a tunnel, the type of contract, in addition to the factors discussed in the preceeding paragraphs, can influence the support and hence the stability of a tunnel.

In Sweden, as in many other western European countries, the unit cost of different construction operations is placed above all other factors in engineering economy (Sandström, 1963). Contrary to common American practice where the aim is to get the tunnel completed as fast as possible, time of completion frequently is of minor significance in Swedish tunneling practice. "If a tunnel is scheduled for completion on a certain date, as determined by other economic considerations, it is started early enough to ensure that the completion date can be met with a minimum expenditure of labor, equipment, and explosives" (Sandström, 1963).

Although preliminary estimates for budgeting purposes are usually made of the amount of support likely to be needed in a tunnel, detailed breakdowns of unit costs for different support measures are the most important items in bid competition. Because bidding is usually closed, only the most desirable (i.e., the most experienced) contractors are asked to submit bids for a project, and support unit cost bids usually fall within predictable limits. Because of the experience in tunneling that most contractors have, there usually do not arise situations in which unit support costs are severely under- or over-estimated. However, the profit, or lack of profit that a contractor may be making on tunnel support may be reflected in the quantity that he uses in a tunnel.

In some of the projects in which the writer's observations were made it was quite obvious that high profits were being made on shotcrete. Such practice is conservative as far as stability of the tunnel is concerned. However, the strict control and inspection that is frequently carried out by the owner of a private hydroelectric development generally precludes drastic oversupport of an opening. More importantly, the round-by-round control exerted by the owner eliminates many instances where a contractor is likely to neglect support if he is not set up for shotcreting or rock bolting. Many times there is a tendency to neglect support when bad ground conditions are first encountered in a tunnel. The extent of the bad ground is generally not known, and the tunneling crew may well consider it desirable from the time standpoint to avoid mobilization of shotcreting equipment until a larger quantity of shotcrete can be applied at one time. There is always the hope that bad ground conditions may end abruptly after one or two rounds and that the support can be delayed until several other bad sections are encountered that require support.

Cost Plus Contracts. Very few of the projects observed by the writer were contracted on a cost plus basis. In those instances where this type of contract was used, the apparent tendencies were towards a neglect of support. However, this observation can probably be attributed more to a lack of experience or interest on the part of the tunnel engineer than on the type of contract. Contrary to the observations made by the writer, there is no reason to expect that a contractor operating on a cost plus basis would not tend to oversupport a tunnel, provided his men and equipment are conveniently available for the work. It is not likely that support work would ever be nonprofitable for a contractor who is operating on a cost plus agreement and, hence, if close owner control exists on a tunnel project, there should be no problem in obtaining the necessary support measures. Because of these considerations, a cost plus agreement should ideally yield the most desirable type of support contract from considerations of stability alone.

4.3 Tunneling Methods

In the drilled and blasted tunnels in which all of the writer's observations were made, variations in tunneling methods include variations in face attack methods (i.e., full face, heading and bench, etc.) and in the different operations of the tunneling cycle (drilling, blasting, support, mucking). The influences of various blasting and support techniques are treated in separate subsequent sections. The influence of face attack methods and miscellaneous tunnel cycle operations on stability are treated in this section.

Face Attack Methods

The method of face attack influences the stability of an opening in two ways. Of most importance is the span width of the progressive openings that are excavated. If a very wide span width is excavated by starting with a narrow opening and enlarging by stoping the side walls inward (Fig. 4.1) there exists a chance to install support before the full span width is excavated. Thus, loosening in the crown can be prevented, or at least reduced, in the final opening. Similarly, loosening in the side walls of the final opening can be reduced if support is placed in the walls of the initial opening (Fig. 4.2).

Of secondary, but also possibly large, significance in the method of face attack is the effect of different stress distributions that are associated with different shaped openings. It is very possible that the stress distribution around an initial pilot opening of an excavation is more vavorable to loosening than is the stress distribution that would arise if the opening were advanced full face. Such behavior depends strongly on the ratio of the horizontal to the vertical in-situ natural stresses (K_o) and the shapes of the openings.

The face attack method used in Swedish rock tunnels depends to a large extent on the size of the tunnel. Although full face advances have been used in tunnels up to 150 square meters in area, it is common to use multiple advances, such as heading and bench, for tunnels with an area in excess of 100 square meters.

The shape of the desired tunnel is also a factor that influences the face attack method. High, narrow

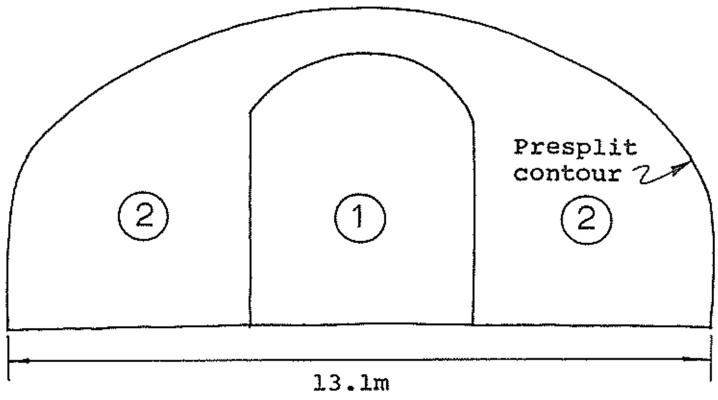


Fig. 4.1 Excavation by heading and side wall stoping for Seitevare machine hall

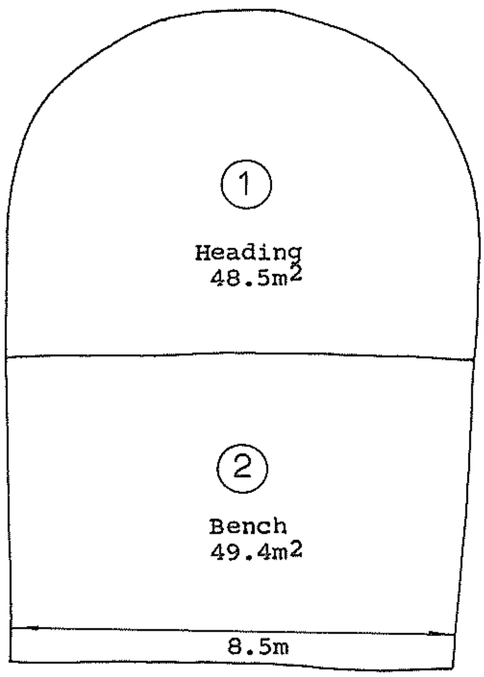


Fig. 4.2 Heading and bench blasting of the Seitevare tailrace tunnel

tunnels, such as the tailrace tunnel at the Seitevare project, are frequently driven by the heading and bench method. Practical reasons for using the heading and bench method in such a tunnel are as follows:

(a) Cheaper excavation by bench blasting

(b) Easier crown support installation from heading floor than from floor of completed opening

(c) Heading serves as pilot bore in unknown ground conditions.

The latter two factors may result in a better supported tunnel that is driven by the heading and bench method than would be obtained by driving full face. One undesirable factor that the writer observed in the Seitevare tunnel, driven by the heading and bench method as shown in Figure 4.2, was a tendency for the placement of support in the walls of the bench to lag the blasting of the bench much more than the placement of support in the heading lagged the blasting of the heading. A significant number of the unstable cases in the walls of that tunnel can be attributed to the delay in placement of support in the walls, following the blasting of the bench. Rock bolting and shotcreting of the heading were usually done on a roundwise basis whereas similar support in the bench was frequently delayed until falls or slips occurred.

The selection of a tunnel shape is very frequently based on the blasting methods considered desirable for an opening of the desired size and the maximum span width that is considered safe for the expected rock conditions. There was some question, for example, as to the relative merits of driving the Seitevare tunnel full face and heading and bench. The tunnel very easily could have been driven full face, but would have required a wider span width for the optimum blasting round, which in Sweden is usually considered to be a V-cut. The decision to use a heading and bench attack was based on the desirability to hold the span width to a minimum.

The tendency in Sweden is towards wider openings and full face attack, as such techniques enable larger drilling and mucking equipment to be used. Such a trend is probably warranted on the basis of the small differences in support needed for the different attacks. As discussed in the previous chapter, high vertical walls frequently cause as much stability difficulty as do wide spans. As will be seen from the discussions of field observations in the following chapter, there does not appear to be any strong relationship between span width and degree of support required for a given rock quality for the range in span widths of from seven to 13 meters. However, it can be expected that wider spans require support more frequently than do narrow spans in the same quality rock.

Face attack methods other than full face and heading and bench are not common in civil engineering projects in Sweden.

Other Factors

Several miscellaneous factors in the tunneling cycle can influence the stability of an opening. These factors include: mucking time, ventilation and smoke time, and scaling.

Mucking Time. The time required to muck out a round seemingly would have no influence on the stability of an opening. However, if support is placed after the mucking operation, the time required to muck out is of considerable importance, particularly if the standup time is less than the time required for mucking. The introduction of rubber-tired, front-end loaders and large (up to 40 ton) trucks in large tunnels (>80 square meters in area) has resulted in the reduction of mucking time by factors of 1/2 to 1/4 of that required by older electric, track-mounted shovels and small trucks. Where support work is carried out from the drilling platform, immediately after the mucking cycle, such differences in mucking time are very significant. All of the tunnels in the writer's observations were mucked with high speed equipment (mucking time: 1-3 hrs), and it is not believed that differences in muck time account for any significant differences in the field cases.

Ventilation and Smoke Time. Ventilation of the tunnel face and smoke time are two factors that affect tunnel stability in much the same way as mucking time. Good ventilation and short smoke times (15-30 minutes) obviously permit more rapid placement of support. Small differences in smoke time probably have no effect on stability, but the complete failure of a ventilation system or a very inefficient system may render working conditions at the face so bad that support work is neglected or given only minor attention, particularly if rock conditions are not so bad that work cannot proceed without support. Shotcreting and rock bolting are stenuous jobs that require good working conditions of light and air.

An extreme case of an instance where atmospheric conditions caused drastic slow-downs in all tunneling operations, particularly support placement, is shown in Figure 4.3. The normal visibility and atmospheric conditions at the tunnel portal are seen in Figure 4.3a. Under such conditions, support work is carried out unhindered and there are not likely to be delays that can be attributed to ventilation conditions. The slightly fogged condition seen in Figure 4.3b did not impair support work, but was responsible for a significant increase in mucking time because of the slower truck traffic that resulted from poor visibility. The effect of mucking time on stability has been discussed in the previous section. The conditions shown in Figure 4.3c resulted in a complete halt of all tunneling operations. At the time this condition arose the bench was being blasted out and the bench face was located in the rock conditions shown in Cases 11-15, 18 and 19. The deterioration of the tunnel walls in these cases was particularly noticeable during the time that attention was being concentrated on the ventilation difficulties. Because the visibility was so poor, deterioration of the wall rock could not be detected by passing by in a motor vehicle, and it was not until the ventilation problem was solved that support work was carried out.

Scaling. The scaling operation (barring down) that is normally part of the tunnel cycle when steel sets are used for supports can have a pronounced influence on stability. Several different ideas have been put forth as to the value of scaling. There is apparently no choice as to whether or not scaling must be done before steel sets are erected, as loose pieces of rock would certainly pose a threat to miners. When shotcrete is used there is a possibility to delete the scaling operation from the tunneling cycle, particularly if a remotely controlled shotcrete apparatus such as the Swedish ROBOT is used. Most strong advocates of shotcrete agree that scaling of all but the loosest of rock blocks should be omitted, as scaling only tends to loosen additional rock at the tunnel periphery. At least part of the fallout and overbreak in Cases 56 and 57 can be attributed to an attempt to scale the tunnel crown after blasting. The rock in these two cases is so fractured and loose that there is no limit to the amount of material that can be scaled from the walls and drown. In such cases, there is clearly nothing to be gained by intentionally enlarging the opening by scaling.

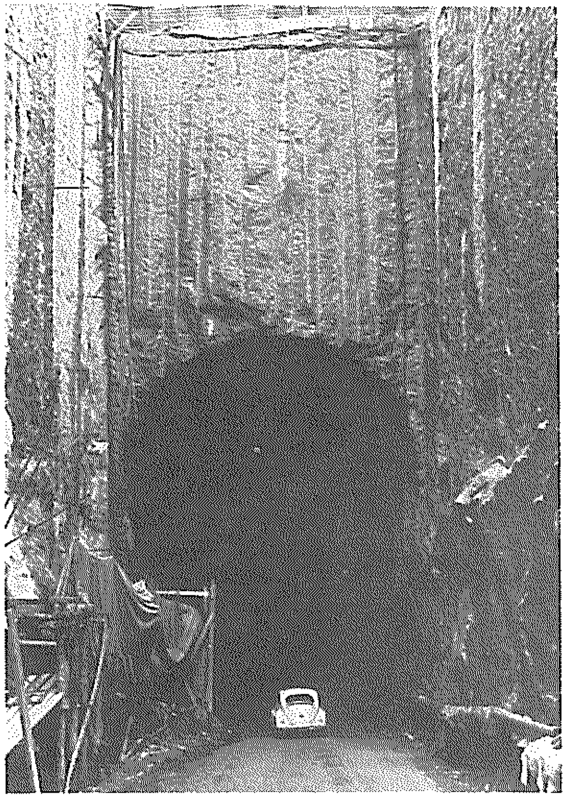
4.4 Blasting Techniques

General

The significance of blasting techniques on the stability of a tunnel in rock lies in the effect that blasting has on the rock around the periphery of the opening. Some blasting techniques leave the rock relatively intact and stable whereas others damage the rock severely by causing fracturing and loosening. The effects of different blasting techniques on the condition of the tunnel periphery rock is the subject of this section.

Swedish Method vs American Method

The so-called "Swedish method" of tunneling refers to the drilling and blasting techniques used in full face advances. Originally the "Swedish method" referred to the use of light, handheld jackleg rock drills, small diameter drill bits (32 mm), and lightweight mobile drill jumbos. More recently the handheld jacklegs are very frequently replaced by ladder-mounted, chain-fed drills that employ the same small-diameter drill bits driven by the same jacklegs. The counterparts of the so-called "American method" are heavy, hydraulically-controlled, boom-mounted drills on large, heavy, drill jumbos and large diameter (48 mm) drill bits. Large tunnel rounds drilled by the Swedish method are normally V or other wedge cuts whereas those drilled by the American method are more frequently parallel hole cuts. The common Swedish round requires more feet of drill hole but less weight of explosive per unit volume of rock excavated. The fundamental reason for the differences in the Swedish and American methods lies in the basic differences in the areas of largest potential savings in tunneling costs. In Swedish practice it has heretofore been possible to realize the

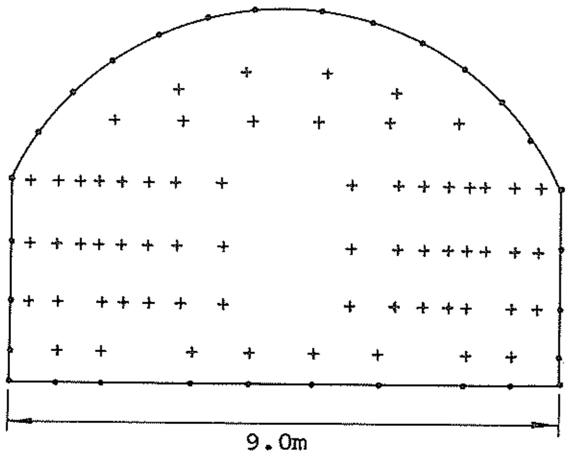


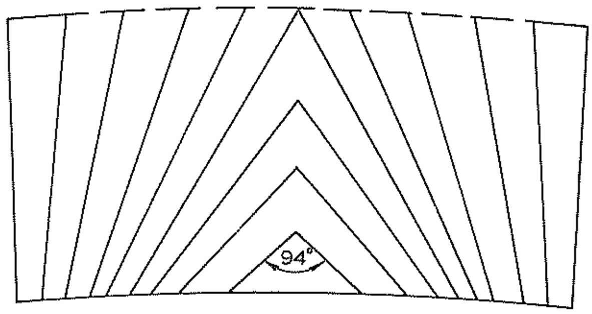
(a) Normal Air Condition.

(b) Slightly Fogged Air. Mucking time increased.

(c) Extremely Poor Visibility. Tunneling operations halted.

Fig. 4.3 Tunnel air conditions

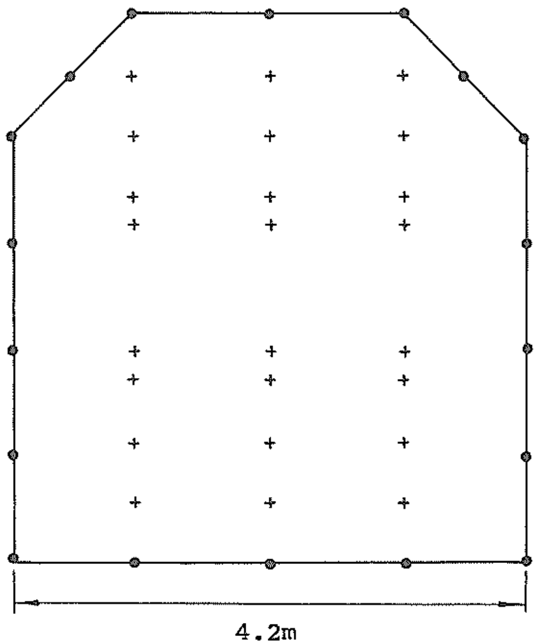


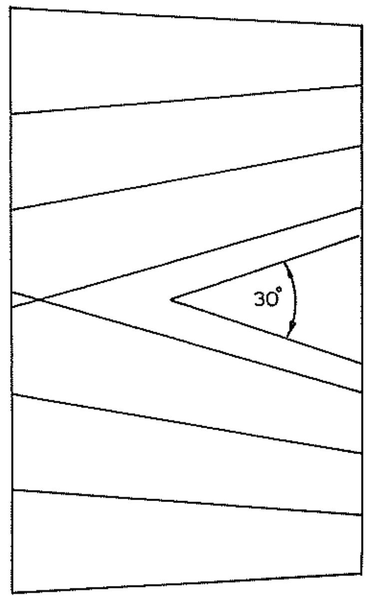


Plan Section Through Center of Tunnel

Area	48.5 m^2
Advance	4.2 m
Specific charge	1.4 kg/m^3 (2.3 lb/yd^3)
Contour hole spacing	0.9 m
Contour hole burden	0.9 m
Drill bit diameter	32 mm
Contour hole ignition	half second delay
Drilling factor	2.1 m/m^3 (5.3 ft/yd^3)

Fig. 4.4 Drilling pattern and blasting data for V-cut round for heading of Seitevare tailrace tunnel

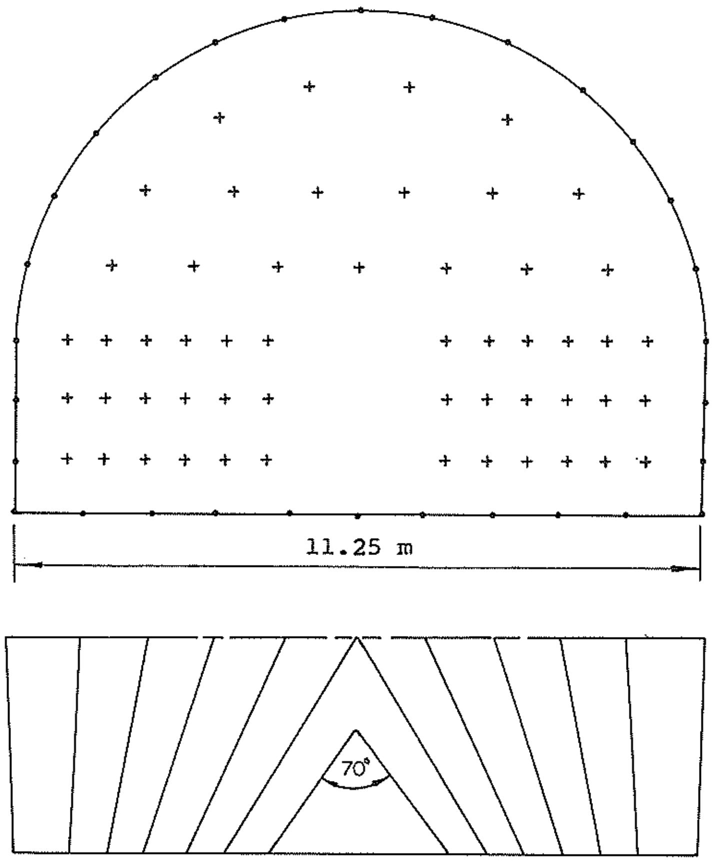




Vertical Section Through
Center of Tunnel

Area	17.9 m^2
Advance	2.7 m
Specific charge	1.91 kg/m^3 (3.2 lb/yd^3)
Contour hole spacing	1.1 m
Contour hole burden	0.5 m
Drill bit diameter	46 mm
Contour hole ignition	half second delay
Drilling factor	2.4 m/m^3 (6.0 ft/yd^3)

Fig. 4.5 Drilling pattern and blasting data for vertical V-cut round for collector tunnel at Bergvattnet hydroelectric project. Note large specific charge because of small angle (30^{o}) of V-cut and relatively large advance for a V-cut (65 % of tunnel width, compared to normal 50%; Langefors and Kihlström, 1963)



Plan Section Through Center of Tunnel

Area	80 m^2
Advance	3.4 m
Specific charge	1.0 kg/m^2 (1.7 lb/yd^3)
Contour hole spacing	1.3 m
Contour hole burden	1.3 m
Drill bit diameter	48 mm
Contour hole ignition	half second delay
Drilling factor	1.1 m/m^3 (2.8 ft/yd^3)

Fig. 4.6 Drilling pattern and blasting data for large-hole V-cut round for Rätan Tailrace tunnel. Note small advance (32% of tunnel width). Use of large Gardner-Denver drill jumbo did not permit longer advance at same angle of cut

greatest savings in reduced material costs, as labor is relatively inexpensive. In American practice, materials are relatively inexpensive compared to labor, and the greatest potential for reduced tunneling costs lies in the labor element. It is therefore more profitable to attempt to reduce the cost of labor by increasing the rate of advance than to attempt to cut material costs. This factor explains the use, in the American method, of large equipment that is capable of rapid face advance. Rising costs of labor in Scandinavia will probably eventually lead to conversion to American tunneling practices in that area.

From stability and support considerations, there exist several advantages to the Swedish method of tunneling. The light handheld jacklegs used in the Swedish method permit support work (rock bolting) to be carried out with the same equipment used for drilling the blasting round. Because the drill jumbo is very mobile and because the drillers are very close to the perimeter of the tunnel, the support work can be carried out from the jumbo while the face is being drilled. Because of this convenience, delays in support installation may be reduced.

One very distrinct advantage of the drilling jumbo or platform used in the Swedish method is the ease with which the crown can be inspected at any time during drilling. Much closer attention is thus given to the crown stability, as the nearness of the crown to the drillers makes rock falls seem much more of a threat than in the American method where the boom-mounted drills may be remotely controlled. The inspection of the crown and installation of crown support in the American method may, depending on the drill jumbo used, require an additional piece of equipment simply to provide access to the crown.

There are some claims that the Swedish method, involving angle cuts, small drill holes, and relatively lightly charged holes, is superior to the American method, involving parallel hole cuts, and large, relatively heavily charged holes, because of the generally smaller specific charges used. It is felt by some that the smaller charges do not damage the rock as severely as do heavily loaded parallel hole rounds. Unfortunately there exist no data to prove or disprove this hypothesis. Although the parallel hole rounds have typical specific charges that are higher than those of typical V-cut rounds (1.8-2.4 kg/m^3 or 3-4 lb/yd^3 for parallel hole rounds compared to 0.6-1.5 kg/m^3 or 1-2.4 lb/yd^3 for typical V-cut rounds in large tunnels), the largest part of the added charge in parallel hole rounds is in the cut, and it is not known whether or not this part of the round affects the rock at the periphery of the desired final opening. Because the perimeter or contour holes in both types of rounds can be designed identically, there exists the possibility that the wall rock in the completed tunnel will be identical for both types of rounds if the cut charges do no damage the wall rock.

Typical Rounds

Practically all of the tunnels in which observations were made were driven with V-cut rounds. Three typical rounds are shown in Figures 4.4-4.6. In all of the tunnels studied the specific charges (powder factors) vary from a minimum of 0.87 kg/m^3 (1.5 lb/yd^3) in the Satisjaure tunnel to 1.91 kg/m^3 (3.2 lb/yd^3) in the Bergvattnet collector tunnel. The latter is unusually high because of the large advance that was attempted with a V-cut round in that relatively small tunnel.

Of particular interest in all of the tunnels is the spacing of contour or perimeter holes. It will be noted from the rounds shown in Figures 4.4-4.6 that none of the hole spacings and ratios of hole spacing to burden conform to the suggestions given by Langefors and Kihlström (1963) for smooth wall blasting. Nor do the ignition patterns (half second delays in the perimeter holes) conform to those suggested by authorities (millisecond delays). In spite of improper hole spacing-burden relationships, the use of low density explosives in the perimeter holes is common practice and frequently gives good contours.

The extremely well-contoured crown of the Seitevare machine hall (Case 21) was obtained by presplitting a stoping round, as shown in Figure 4.1. Other attempts to presplit a full face round have not been successful (Skånska Cementgjuteriet, 1967).

The success of smooth wall blasting is obviously very dependent on the rock structure and, in particular, on the orientation of discontinuities with respect to the periphery of the opening. From Case 92 it is apparent that good results can be obtained in jointed rock when the discontinuities cross the tunnel at right angles. Similar results were obtained in Case 36 where a smooth wall stoping round was used to enlarge a small pilot drift. On the other hand, the flat-lying overthrust joints in Cases 88, 89, and 90 render successful smooth wall blasting very difficult. Although the hole spacings in these cases exceeded one meter, and the burden was no greater than the hole spacing, the same rounds in other parts of the tunnel gave very good results.

It is not unusual for smooth wall blasting to be effective in one part of the tunnel periphery and not in another. This is illustrated in Figure 4.7 where the crown displays a smooth contour but the intrados perimeter is relatively ragged. Such behavior may be explained by a number of factors, including joint orientation and curvature of the tunnel periphery. Anisotropy in the intact rock structure has a very large influence on the manner in which rock breaks during blasting. This fact has been known for many years in the Swedish quarry industry (Hagerman, 1943). It is well known in Sweden that gneisses can usually be smooth wall blasted better than granites (Lundegårdh, 1963).

Rock Damage Due to Blasting

Because the process of loosening at the periphery of a tunnel in jointed rock is a progressive process that necessarily starts when the first block at the tunnel perimeter falls out, the condition of the wall rock is of utmost importance. Damage of the wall rock that is caused by the charges in the perimeter holes of the blasting round will obviously be of great importance in determining the stability of the completed opening.

Very little is known about the real damage that is done to the rock around the perimeter of an opening when blasting takes place. It is probable that damage in the form of both cracking of intact rock and loosening of joint blocks and newly cracked blocks takes place. Hagerman (1966) claims that the cracked zone due to blasting extends only about one meter beyond the periphery of the opening.

Langefors (1965) discusses the extent of rock damage for different types of blasting. Ordinary blasting is thought to result in the formation of radial cracks and opening of existing joints to a distance of two meters from the periphery of the opening. According to Langefors (1965), instantaneous ignition of an entire round can produce a better contour, but results in crack formation and loosening to a distance of 10 meters beyond the edge of the opening. For this reason short delay ignition (millisecond delays) and instantaneous ignition with a time spread of greater than one or two milliseconds are recommended for smooth wall blasting (Brännfors, 1964; Langefors, 1965).

Langefors (1965) claims that presplitting with exact instantaneous ignition produces a loosened zone about two meters thick. Langefors distinguishes between the two-meter-thick zones produced in ordinary blasting and that produced in instantaneously ignited presplitting by the more pronounced cracking and loosening that occurs in the former. The best tunnel contour and least damage are said to occur when smooth wall blasting and presplitting are used with ignition times of less than 100 milliseconds and greater than several milliseconds. The damaged zone is believed to be less than 0.4 meters thick.

The extent of loosening given by Langefors is said to represent maximum damage in jointed rock. For intact rock, Langefors claims that no cracks are formed as a result of a correctly ignited smooth wall round.

In most full face blasting rounds for large tunnels, the zone of rock around the perimeter that is sometimes smooth wall blasted is usually broken with a more or less free burden which meands that the angle of breakage is greater than 90 degrees, as shown in Figure 4.8a. However, in bench blasting (Fig. 4.8b) there is a greater degree of fixation along the walls for the two side holes, and breakage is likely to occur by tearing or indirect tensile failure along the wall instead of by pure tension as occurs between two properly spaced perimeter holes in a smooth wall round. The tearing is frequently quite evident in the walls of a bench blasted round, as shown in Figure 4.9, and in jointed ground may

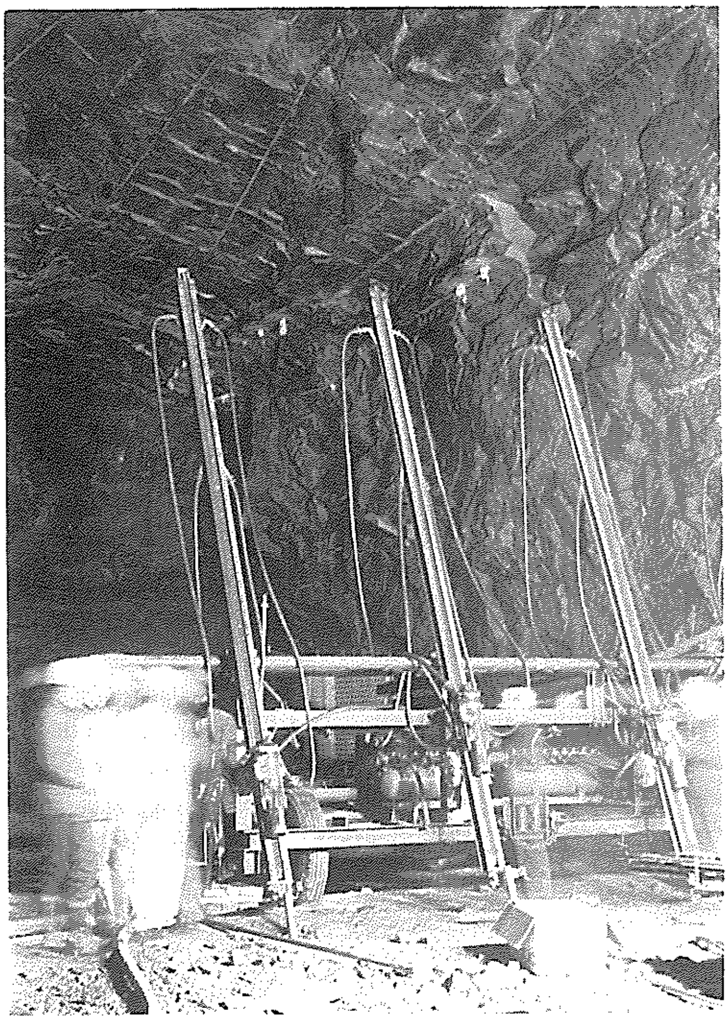


Fig. 4.7 Nonuniform contours from smooth wall blasting. Note smooth roof contour and rough intrados contour. Large-hole V-cut round in Svorva headrace tunnel, Vietas project. Perimeter hole spacing: 0.8-1.0 m. Perimeter hole burden: 1.0 m. Rock type: mylonite (metamorphic)

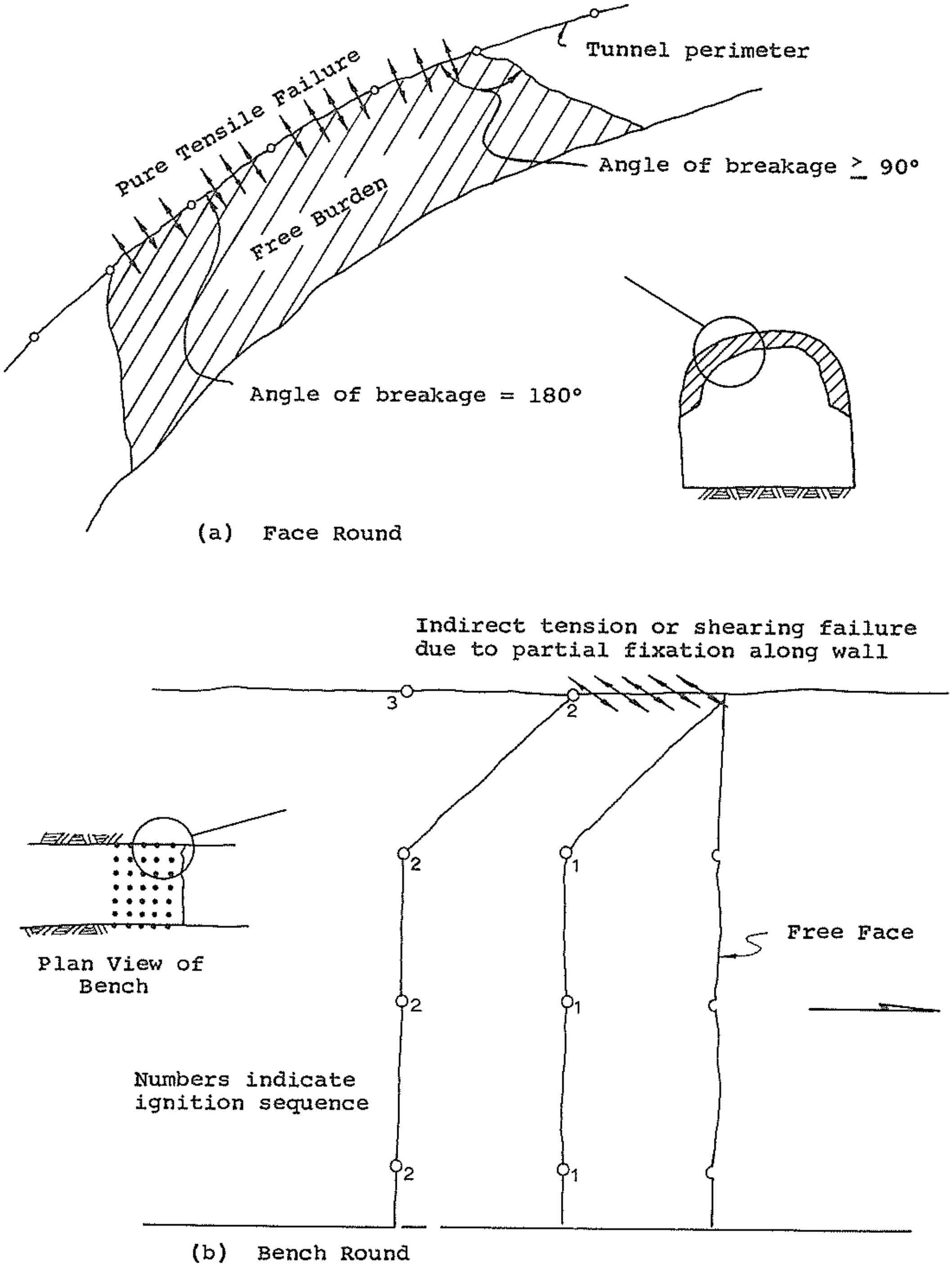


Fig. 4.8 Rock breakage in tunnel face round and in tunnel bench round

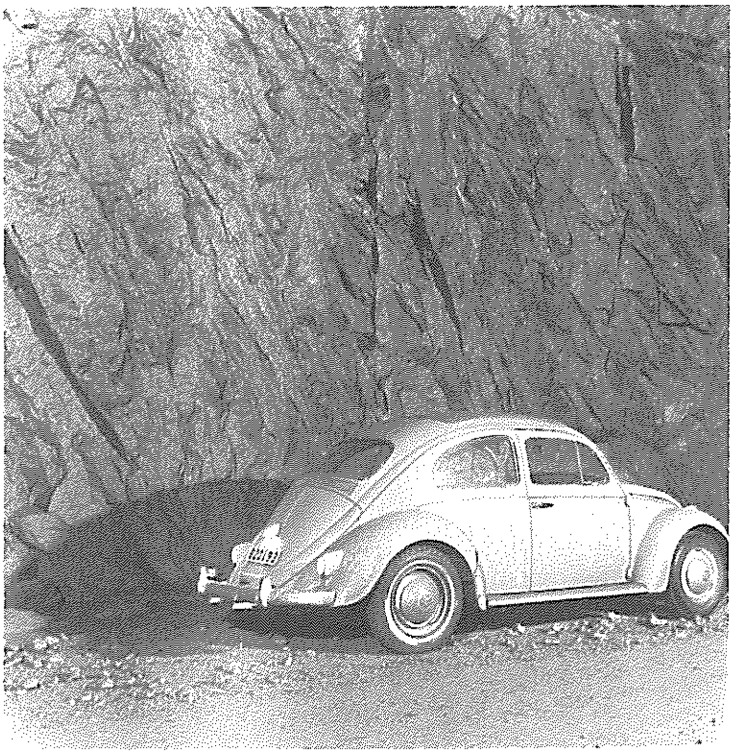


Fig. 4.9 Tearing of granite in bench-blasted side walls of Seitevare tailrace tunnel. Direction of blasting same as in Fig. 4.8 b

contribute to loosening of the side walls. Furthermore, gas pressures from the blast are directed more along the tunnel axis than in face blasting, and can result in loosening of jointed rock, as explained by Deere et al. (1969).

Although smooth wall and presplit rounds can be incorporated into bench blasting, such techniques are normally used in Sweden only for special structures, such as machine halls and large storage rooms.

There exist only meager quantitative data in Sweden about the extent of blast damage in the wall of a rock tunnel. Two sources of data that are of possible revelance are seismic measurements and stress measurements made in the walls and crown of a tunnel. Rahm (1965) reports both seismic and ultrasonic measurements that were made in the walls of small inspection galleries (span width = = 2 meters). Both the seismic sonde, used as down-the-hole apparatus, and the cross-hole ultrasonic measurements indicate a low velocity or destressed zone 0.25 to 1.0 meters thick around the tunnel periphery. Unfortunately no blasting techniques are reported for these tunnels. It is very likely, however, that no particular care in blasting was taken to maintain undamaged walls.

The seismic refraction measurements made in the Rätan tunnel that have been described in Appendix A give some additional clues as to the extent of destressing caused by blast damage and loosening movements in the wall rock. These measurements were made with geophone spacings as close as one meter, and should therefore reveal any thin low velocity zone. The thickness of the detectable low velocity zone along the tunnel floor varies from 1 to 1.4 meters. In consideration of the 0.5-1.0 meter thick layer of crushed rock and tunnel muck that lies on the floor, it must be concluded that the thickness of any extremely destressed or loosened zone along the bottom of the tunnel walls is less than one meter.

There are two possible interpretations for the Rätan tunnel measurements:

(1) It is quite possible that the seismically calculated low velocity layer is only the outermost layer of a thicker zone of destressed or blast-damaged rock whose degree of fracturing and looseness is too small to influence its seismic velocity. In such a case the destressed zone itself may act as a refracting medium if its velocity is the same as that of the undisturbed rock. The influence of such a destressed zone on the stability of a tunnel is not known, but it would not seem likely that such a mildly relieved zone would itself require any support.

(2) There exists also the possibility that the thickness of one meter given by the refraction measurements is the true thickness of the destressed or damaged zone at the point on the tunnel periphery where the measurements were made. Such a case would be consistent with the results reported by Rahm and discussed in the previous paragraph. Although Rahm's measurements were made with an entirely different technique and in tunnels with an area of less than one-tenth of the Rätan tunnel, they do indicate that no exceptionally deep rock damage or destressing has taken place. The fact that the thickness of the low velocity layers is of the same order of magnitude in these tunnels of different areas may indicate that the destressing is not as dependent on tunnel size as one might expect.

The fact that a relatively thin (5 to 10 cm) layer of shotcrete is very often sufficient to maintain roof stability in Swedish tunnels, even in extremely fractured rocks, would lead one to believe that the layer of heavily damaged and destressed rock at the tunnel periphery is not extensive.

Seismic refraction measurements have been used by others to determine the thickness of the destressed zone around tunnels. Scott et al. (1965) report a 1- to 7-foot-thick low velocity zone around the periphery of a large underground cavity in tuff. Scott et al. (1968) report a 5- to 10-foot-thick low velocity zone around tunnels in a granite stock. In the latter case it was not possible to estimate accurately the degree of destressing from the extent of fracturing on the tunnel walls.

Measurements of in-situ stress have been made in the walls and crown of several tunnels and rock rooms in Sweden. Although the absolute values of the results of these measurements are of uncertain value, the measurements are of value as an index or quantitative indication of the degree of destressing that exists at the periphery of an opening, provided measurements are also made of the stress at some distance from the edge of the opening.

The measurements of stress that have been reported by Hast (1958) are of limited value for the above-described purpose, as most of the measurements were made in mines where the local stress field is likely to be very different than that at some distance from a single tunnel. The results reported by Hiltscher (1967) for the Seitevare tailrace tunnel are more valuable in this respect. In four different sections in that tunnel measurements of strain on the bottom of a drill hole, due to overcoring, were made in vertical holes drilled up into the tunnel crown. All of the measurement stations were located in massive, sound, unsupported granite at depths of about 60-80 meters under the ground surface. Measurements were made in each hole at distances of 0.5-2.0 meters and at 9-12 meters from the perimeter of the opening. The results indicate that the magnitude of major principal stress at distances of from 0.5 to 2.0 meters from the tunnel perimeter varies from one to four times that at a distance of 9 to 12 meters from the perimeter. All of these data were obtained in an identical manner, and the appearance of the tunnel walls and roof at the measurement stations was very similar. It is thus obvious that the degree of loosening is very erratic and cannot be related solely to the appearance of the rock. The blasting data for this tunnel are shown in Figure 4.4. Although the stress measurements are not a direct measure of blas damage, they do indicate the absence of complete stress relief within the two-meter-wide zone around the tunnel.

4.5 Rock Support and Reinforcement Techniques

General

Rock support and reinforcement techniques influence the stability of a tunnel in that the manner in which they are installed, the time at which they are installed, and the mechanism by which they render support to an opening directly influence the loosening that may occur in an unstable opening. These topics are the subject of this section.

In Sweden, rock bolts and shotcrete are termed reinforcement rather than support. This distinction is made because it is believed that the primary function of rock bolts and shotcrete is to reinforce the rock around the perimeter of the opening rather than actually support loosened rock that may come to rest on the bolts or shotcrete. This idea of "helping" the rock to support itself has been expressed by many writers and there is no need to dwell upon the subject.

The shotcrete used in the cases discussed in this thesis is for the most part large aggregate (25 mm = = 1 in.) shotcrete applied by the dry mix process. Some of the applications used strictly for the purpose of protecting miners against small pieces of falling rock have a maximum aggregate size of 3/8 in. The rock bolts used in most of the projects are two- to four-meter-long bolts and are anchored either by Perfo sleeves or expansion shells. None of the bolts are grouted. The primary or temporary support or reinforcement of rock bolts and shotcrete that is used in most Swedish tunnels serves also as permanent or secondary support, or is at least an integral part of additional support applied at a later time.* The early application of reinforcement measures soon after blasting is perhaps the single most significant advantage of rock bolts and shotcrete over steel set and cast concrete supports. The time delays that are commonly associated with these latter support types most likely allow loosening to occur that frequently can be prevented when rock bolts and shotcrete are used. It is thus likely that rock bolts and shotcrete are not subjected to the same loads as are experienced by conventional supports. The rock-support or rock-reinforcement interaction is obviously different in the case of rock bolts and shotcrete than for steel sets and cast concrete arches.

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* Additional details of Swedish rock bolt and shotcrete practices are given by Brännfors (1964), Alberts (1965), and Cecil (1970).

Mechanism of Support

The mechanism by which shotcrete functions in stabilizing a tunnel in jointed rock has been discussed by many writers. Of the many hypotheses that can be found in the literature, those given by Alberts (1965) include the most likely support mechanisms. According to Alberts:

(1) Shotcrete is forced into open joints, fissures, seams, and irregularities in the rock surface and in this way serves the same binding function as mortar in a stone wall.

(2) Shotcrete hinders water seepage from joints and seams in the rock and thereby prevents piping of joint filling materials and air and water deterioration of the rock.

(3) Shotcrete's adhesion to the rock surface and its own shear strength provide a considerable resistance to the fall of loose blocks from the roof of a tunnel.

(4) A thicker shotcrete layer (20-30 cm) provides structural support, either as a closed ring or as an arch-type member.

Although a rigorous theoretical analysis of the rock-shotcrete interaction for a tunnel in jointed rock has not been attempted, several idealized models have been used for design and as demonstrations of the possible support mechanisms provided by shotcrete (Rotter, 1961; Linder, 1963; Rabcewicz, 1964-65). The analytical models fall into two general categories:

(1) those which treat the shotcrete as a structural-type mumber and assume the rock to behave as a continuum; and

(2) those which treat the rock-shotcrete interaction as a rigid block model of a single rock block in the crown of a tunnel.

In the discussion that follows, these two types of analyses are referred to as structural analyses and rigid block analyses. The assumptions for the role of the shotcrete in all of these simplified models correspond to one of the four items given by Alberts.

Structural Analyses. The most common structural-type analysis assumes that the rock behaves as a plastic continuum and that the shotcrete functions as a thin, closed ring. This assumption for the shotcrete support corresponds to item (4) of Alberts' hypothesis. The model has apparently been used with success in some rock conditions in the Albs (Rabcewicz, 1964, 1965; Rabcewicz, 1969). However, the behavior of tunnels in loosening ground in Scandinavia is very much different than the so-called genuine rock pressure phenomena encountered in the Alps, and the same concepts do not apply for the shotcrete-rock interaction. As a matter of fact, very few of the shotcrete applications observed by the writer for treatment of loosening instability were continuous ring- or arch-type constructions. Spotwise, discontinuous applications over only a part of the tunnel section are very common. Some typical discontinuous shotcrete applications are seen in Cases 3, 7, and 88. Examples of continuous applications are Cases 44, and 57. These two types of shotcrete supports are shown schematically in Figure 4.10.

Rigid Block Analysis. In loosening ground conditions, it is very likely that shotcrete provides support by one or more of the first three items given by Alberts, particularly if the application is discontinuous around the perimeter of the opening. Several authors (Rotter, 1961; Linder, 1963; Deere et al., 1969) have made simple calculations to show the capabilities of shotcrete in supporting various given volumes of rock. Because loosening instability is a progressive type of failure that starts when a single block, frequently termed the "key-stone". falls from the crown of the opening, analyses of the interaction between a shotcrete lining and a single rigid block of some arbitrary dimensions give some insight into the possible roles of shotcrete in supporting loosening rock conditions. It is normally assumed that stability is achieved if the key-stone can be held in place.

Several different rigid block models are shown in Figure 4.11. The type of loading and the resistance assumed to be provided by the shotcrete are given for each model. The models in (a) and (b) are intended to show the support contributed by the rock-shotcrete

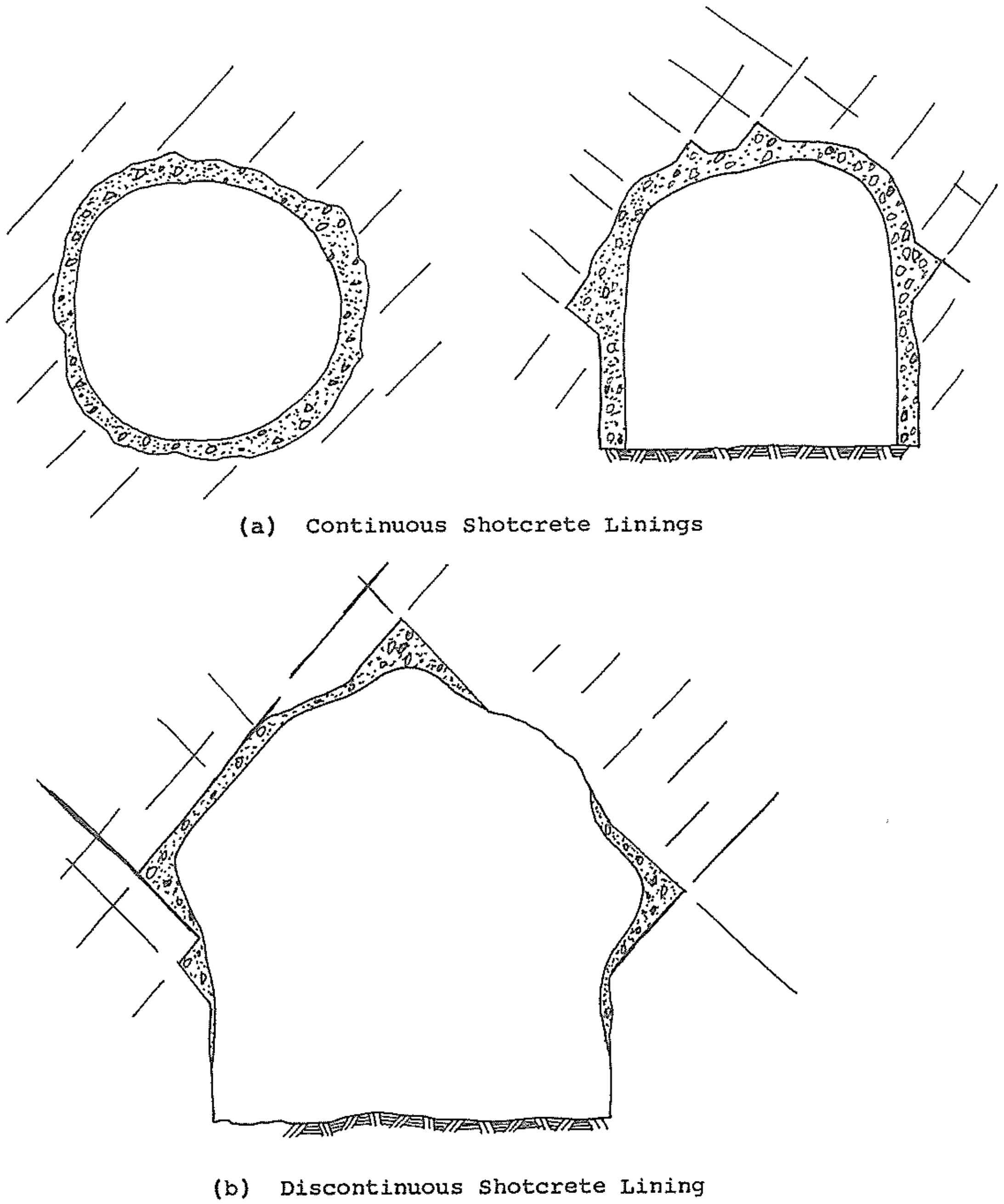


Fig. 4.10 Continuous and discontinuous shotcrete linings

bond. Those in (c), (d), and (e) show the support provided by the shearing resistance of shotcrete under different loading conditions. All of the models are intended to show the effect of localized mobilization of the shotcrete strength in a discontinuous application. With the exception of (a), all of the models are also possible modes of behavior in a continuous structural-type liner.

If a number of simplifying assumptions are made as to the forces acting on the blocks and the behavior of the shotcrete, it is possible to compute factors of safety against fallout of the blocks for each model in Figure 4.11. The computations are shown in Figure 4.12. The necessary assumptions are as follows:

(1) The blocks remain in place until the shotcrete can be applied. The standup time of the rock in relation to the time necessary to apply a shotcrete layer determines the validity of this assumption.

(2) Except in (e), the only force acting on the blocks is the dead weight of the block itself.

(3) In (a) and (b) the rock-shotcrete bond strength is fully mobilized over a perimeter strip whose width is equal to the thickness of the shotcrete. The real area over which the bond strength is mobilized depends on the stiffness of the shotcrete. For fully hardened shotcrete, the bond over the entire rock-shotcrete interface may be uniformly stressed.

(4) In (c), (d), and (e) the shearing resistance of the shotcrete is fully mobilized across the entire thickness of shotcrete.

In order to compare the relative stability factors for the different models, assumptions for shotcrete thickness ($t = 5$ cm), block dimensions ($a = 100$ cm), and the unit weight of the rock ($\gamma_r = 0.0027$ kg/cm^3) have been made. Each model analysis will be discussed separately and then the results of all the models will be compared.

The analysis shown in Figure 4.12a demonstrates the benefit that can be derived from a mere filling of the depressions and overbreak around the tunnel perimeter. This type of support corresponds to item (1) of Alberts' hypothesis. In the model it is assumed that the rock to both sides of the points "1" and "5" is stable through earlier shotcreting or other support. If the block W is to drop out of the crown, it must break the rock-shotcrete tensile bonds along 1-2 and 4-5 or else shear through the shotcrete. It is assumed that the full tensile bond strength across the rock-shotcrete contact is mobilized for a distance along 1-2 and 4-5 equal to the depth of the shotcrete overbreak filling, or about one half of the average size of the joint blocks. It is also assumed that the bond strength at a 45° angle to the rock-shotcrete interface (in the vertical direction) is equal to the tensile bond strength normal to the interface. This is probably a conservative assumption, as the bond strength in shear of a smooth rock-concrete interface is normally 1.5 times the direct tensile bond strength (Carmichael, 1970).

The expression for the factor of safety against fallout is seen to be directly proportional to the rock-shotcrete bond strength and indirectly proportional to the size of the block and the density of the rock. Although the validity of the assumptions used in the simple analysis is uncertain, the exercise does point out the benefit that may possibly be derived from a mere filling of depressions on the tunnel walls and roof. It should be pointed out at the same time, however, that it would be absolutely necessary for the surrounding rock to be stable in order to develop the resistance that is illustrated in the example. For this reason it would seem logical that shotcreting should be started at a point on the tunnel periphery that is stable, such as in the walls, and then carried up into the tunnel crown from both sides. In this manner the loads that may come onto the filled depressions can be thrust back into previously stabilized rock.

In Figure 4.12 (b) and (c) the stabilizing effect of a thin shotcrete layer on the fallout of a single block is considered. The possible modes of failure considered are a bond failure of the shotcrete along a strip of width equal to the shotcrete thickness around the block, and a shear failure through the shotcrete. The model in (c) apparently was first considered by Rotter (1961). This behavior corresponds to item (c)

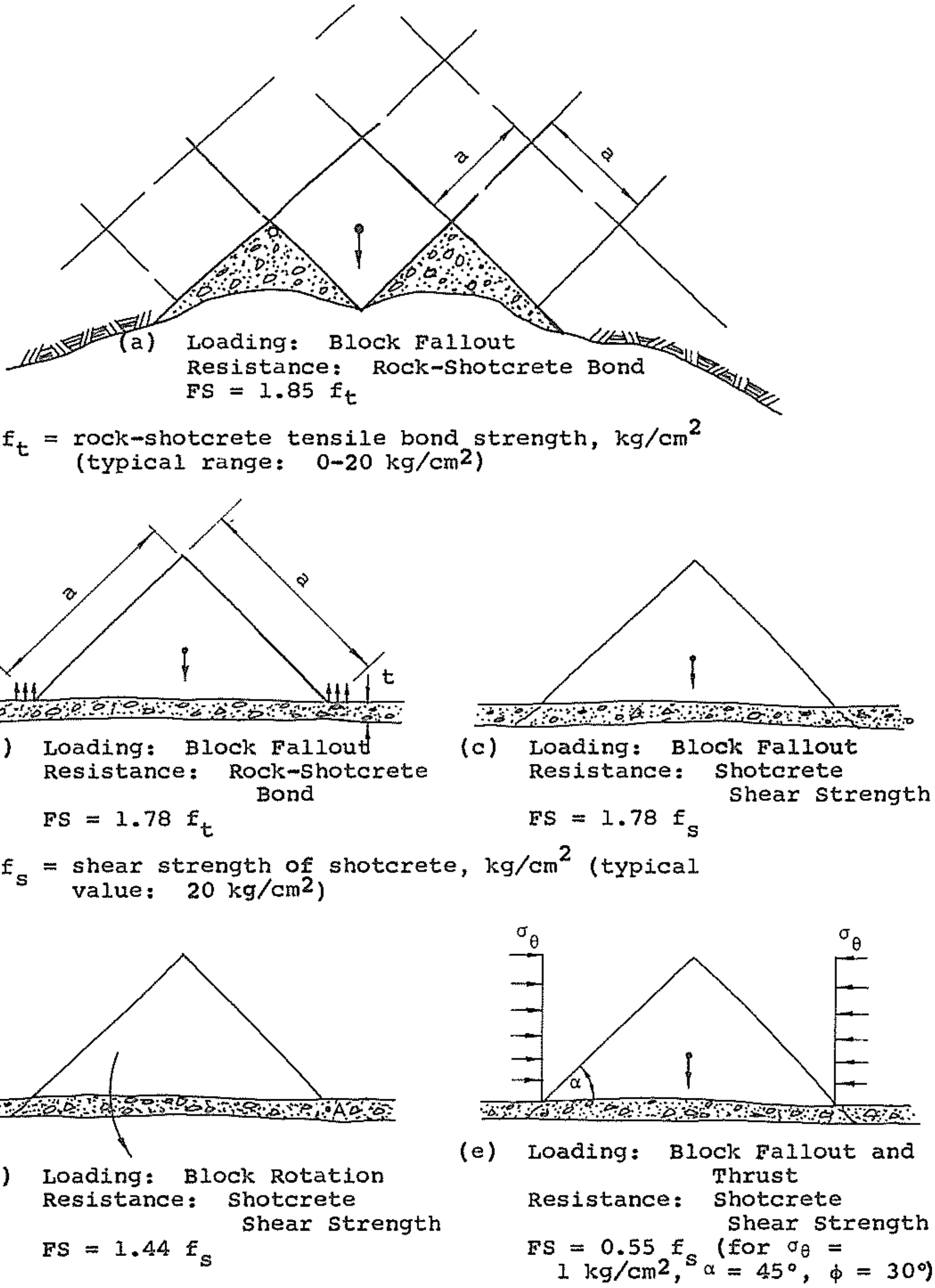
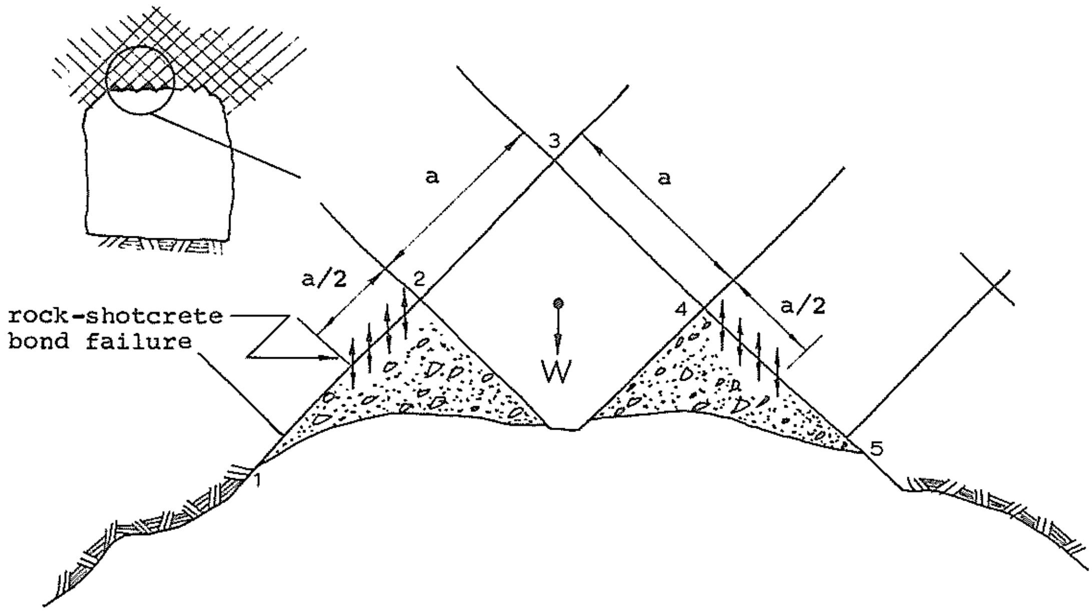


Fig. 4.11 Rigid block models for shotcrete tunnel linings. Factors of safety given for a = 100 cm, t = 5 cm, γ_r = 0.0027 kg/cm^3



Assume:

(a) unit weight shotcrete = unit weight rock

(b) no resistance to fallout on ends of block W

(c) bond strength in vertical direction is same as tensile bond normal to rock-shotcrete interface

DF = driving force = weight of wedge 1-3-5

$$= \frac{1}{2}\, 4a^3\, \gamma_r = 2a^3\, \gamma_r$$

RF = resisting force = $2\,(a \cdot \frac{a}{2})\, f_t = a^2\, f_t$

FS = factor of safety against fallout

$$FS = \frac{RF}{DF} = \frac{a^2 f_t}{2a^3\, \gamma_r} = \frac{f_t}{2a\, \gamma_r}$$

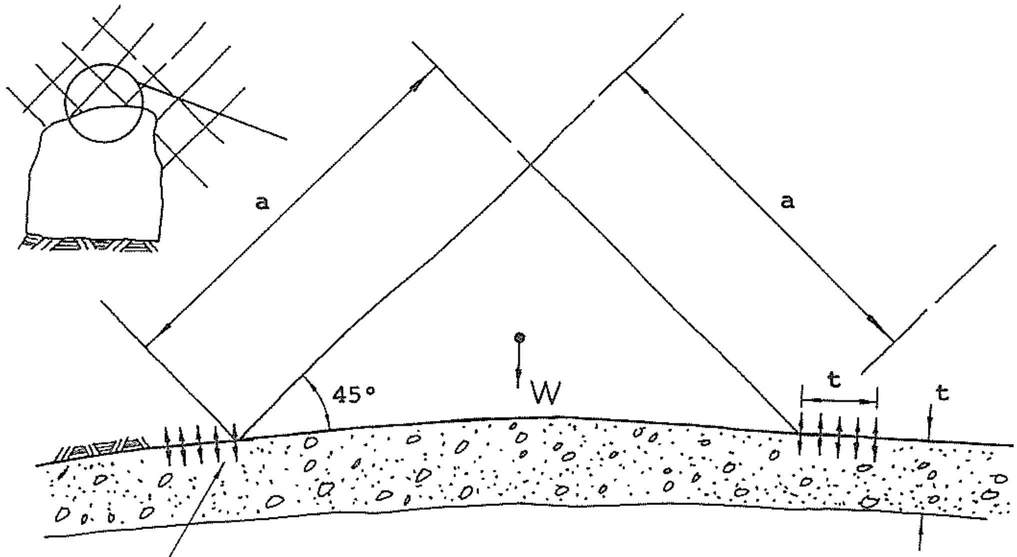
For $a = 100$ cm, $\gamma_r = 0.0027$ kg/cm^2

FS = $1.85\, f_t$ f_t expressed in kg/cm^2

range in values: 0-20 kg/cm^2

Fig. 4.12 Analyses of rigid block models

(a) Support rendered by overbreak filling: failure by rock-shotcrete bond failure



Assume rock-shotcrete tensile bond mobilized over perimeter strip of width equal to shotcrete thickness, t.

DF = driving force = weight of block = $\frac{1}{2} a^3 \gamma_r$

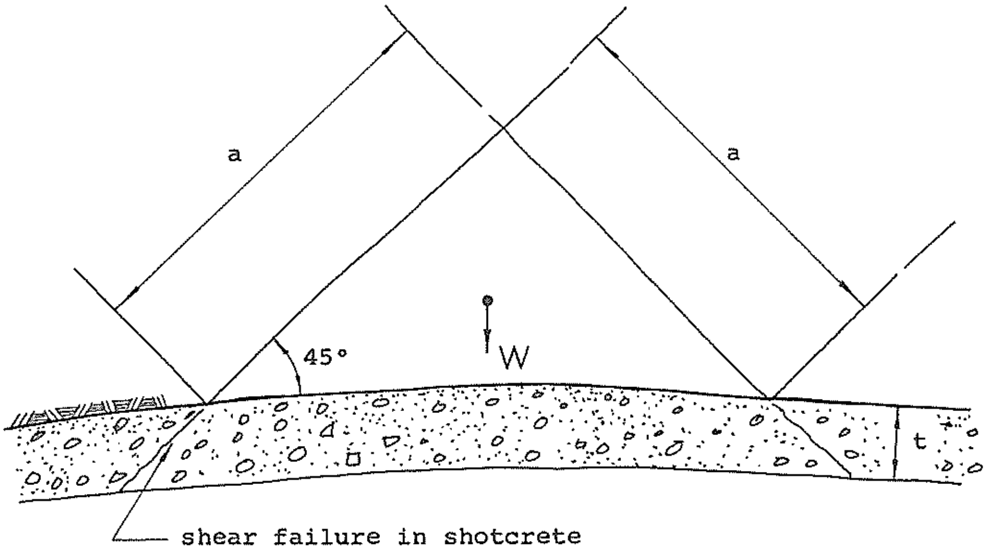
RF = resisting force = $2atf_t + 2 \times 1.4\ atf_t = 4.8\ atf_t$

$$SF = \frac{RF}{DF} = \frac{9.6\ tf_t}{a^2\ \gamma_r}$$

For $t = 5$ cm, $a = 100$ cm, $\gamma_r = 0.0027$ kg/cm^2

FS = $1.78\ f_t$

Fig. 4.12 (continued)
(b) Rigid block fallout: failure by rock-shotcrete tensile bond failure



$$DF = \frac{1}{2} a^3 \gamma_{rock}$$

$$RF = 2atf_s + 2 \times 1.4 \; atf_s = 4.8 \; atf_s$$

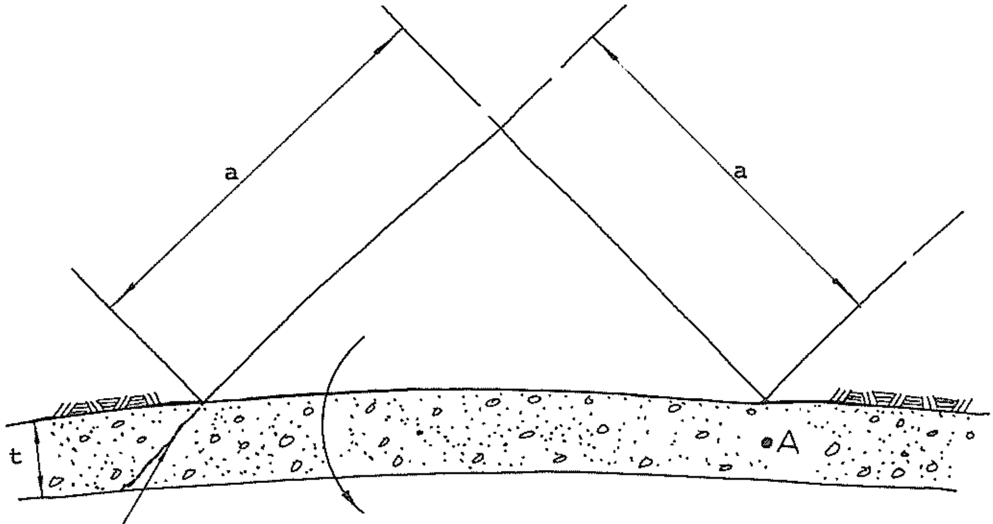
$$FS = \frac{RF}{DF} = \frac{9.6 \; tf_s}{a^2 \; \gamma_r}$$

For $t = 5$ cm, $a = 100$ cm, $\gamma_r = 0.0027$ kg/cm^2

$$FS = 1.78 \; f_s$$

Fig. 4.12 (continued)

(c) Rigid block fallout: failure by shearing through shotcrete



Assume triangular distribution of shearing resistance along front and back edges of block

$$DM = \text{driving moment} = \frac{1}{2}\, a^3\, \gamma_r \times 0.707\, a = 0.353\, a^4\, \gamma_r$$

$$RM = \text{resisting moment} = a \cdot t \cdot f_s \cdot 1.414\, a$$

$$+\ 2\,(t \times 1.414\, a \times \frac{f_s}{2})\ \frac{2}{3} \times 1.414\, a$$

$$RM = 1.414\, ta^2 f_s + 1.33\, ta^2 f_s = 2.74\, ta^2 f_s$$

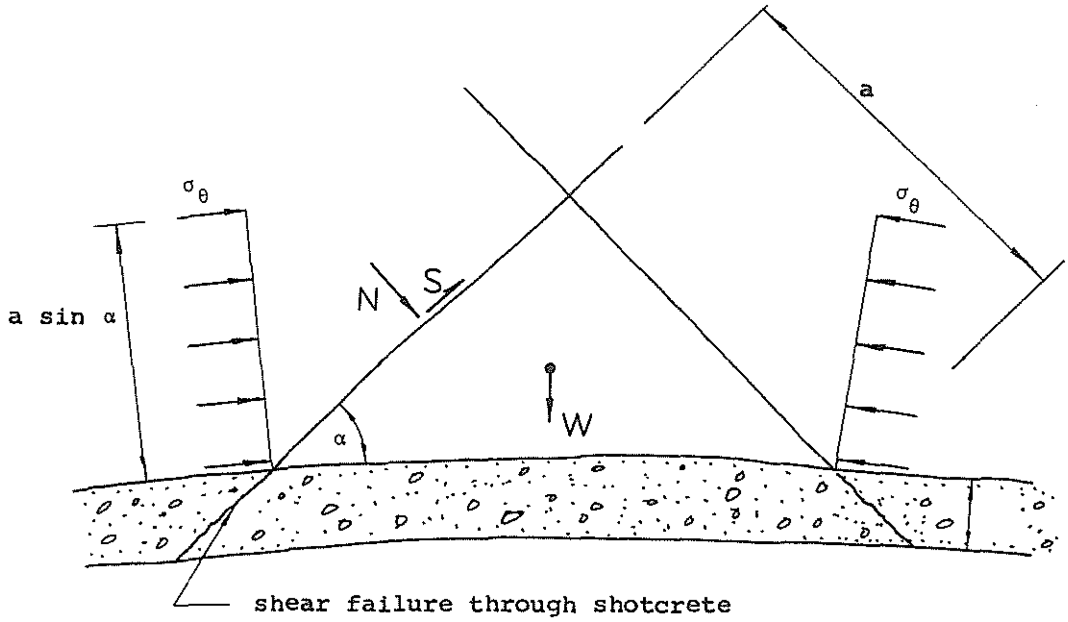
$$FS = \frac{RM}{DM} = \frac{2.74\, ta^2 f_s}{0.353\, a^4 \gamma_r} = 7.76\, \frac{tf_s}{a^2\, \gamma_r}$$

For $t = 5$ cm, $a = 100$ cm, $\gamma_r = 0.0027$ kg/cm^3

$$FS = 1.44\, f_s$$

Fig. 4.12 (continued)

(d) Rigid block rotation: failure by shearing through shotcrete



N = normal force on one side of wedge $= \sigma_\theta\ a^2 \sin^2 \alpha$

S = shearing resistance on one side of wedge $= N \tan \phi$

N_v = vertical component of normal force on wedge $= N \cos \alpha$

S_v = vertical component of shear force on wedge $= S \sin \alpha$

V = total vertical driving force on wedge $= W + 2\ N_v - 2\ S_v$

$$= \frac{1}{2}\ a^3\ \gamma_r + 2\ (N \cos \alpha - S \sin \alpha)$$

$$= \frac{1}{2}\ a^3\ \gamma_r + 2\ \sigma_\theta\ a^2 \sin^2 \alpha\ (\cos \alpha - \tan \phi \sin \alpha)$$

RF = resisting force in shotcrete $= 2\ atf_s + \dfrac{2\ atf_s}{\cos \alpha}$

$$FS = \frac{RF}{V} = \frac{2\ atf_s\ (1 + \frac{1}{\cos \alpha})}{\frac{1}{2}\ a^3\ \gamma_r + 2\ \sigma_\theta\ a^2 \sin^2 \alpha\ (\cos \alpha - \tan \phi \sin \alpha)}$$

For $\alpha = 45°$, $\phi = 30°$, $FS = \dfrac{9.6\ tf_s}{a^2\ \gamma_r + 0.6\ \sigma_\theta\ a}$

For $\sigma_\theta = 1\ kg/cm^2$ ($\sim$ 15 psi), $t = 5$ cm, $a = 100$ cm,

$\gamma_r = 0.0027\ kg/cm^2$, $FS = \dfrac{48.0}{27 + 60}\ f_s = 0.55\ f_s$

Fig. 4.12 (continued)

(e) Thrust on a rigid block: failure by shearing through shotcrete in Alberts' hypothesis. It is shown in the analysis of the models that the factor of safety against fallout has the same form for both modes of failure, but in (b) the factor of safety is dependent on the bond strength, f_t, and in (c) it is dependent on the shear strength of the shotcrete, f_s. It is thus seen that the portion of shear strength in the shotcrete that can be mobilized is no greater than the bond strength between the shotcrete and the rock. If the ultimate compressive strength of shotcrete is taken as $f'_c = 300$ kg/cm^2 (4200 psi) and the allowable shear stress as $4\sqrt{f'_c \text{ (psi)}} = 18$ kg/cm^2 (260 psi), then the bond strength of the rock-shotcrete interface must be at least 18 kg/cm^2 (260 psi) for the shear strength of the shotcrete to be mobilized. Rock-shotcrete tensile bond strengths of from 0 to 20 kg/cm^2 (300 psi) have been measured. The lower values are for clay and other mineral-coated surfaces and the higher values are for clean, rough granite surfaces. Thus, an extremely good bond is required to fully develop the shear strength of shotcrete. It is interesting to note that the factor of safety is inversely proportional to the square of the block dimension, a, in the models shown in (b) and (c). Although the analysis in (b) is based on the rather arbitrary assumption concerning the width of the strip over which the bond strength is mobilized, the calculations do point out the significance of the bond strength, a factor that has been ignored by many in computing the support capacity of shotcrete.

The analysis shown in Figure 4.12 (d) considers the rotation of a single rigid rock block about one of its corners. This analysis was considered by Deere et al. (1969), who showed that the safety factor against the failure by rotation is less than that by direct fallout, as also indicated in Figure 4.12 (d). Because the writer's model studies demonstrated very vividly the role of block rotations in loosening instability, the ability of shotcrete to resist block rotations as well as block slippage is particularly significant. Rock bolts aid considerably in this respect, even apart from their ability to apply a normal pressure to the rock surface. The benefit of untensioned grouted rebars in preventing rock block rotations is a factor that should not be overlooked in support design.

The analysis of a rigid block subjected to a uniform tangential compressive stress, σ_θ, is considered in Figure 4.12 (e). In this model the angle that the jointing makes with the tangential stress, α, and the shear strength along the rock joint, ϕ, are of major importance. For angles of ϕ greater than $(90 - \alpha)$, the tangential shear stress tends to stabilize the block, but for angles of ϕ less than $(90 - \alpha)$, the tangential stress tends to thrust the block into the opening. In order to compare this model with the others, an average tangential stress of 1 kg/cm^2 (15 psi) has been assumed to act in the rock.

The safety factors for each model analysis for the assumed values of a, t, and γ_r are shown in Figure 4.11. The rock-shotcrete tensile bond strength, f_t, and the shearing resistance of the shotcrete, f_s, are not assigned specific values, as these strengths vary greatly during the early age of the shotcrete. Although the absolute values of the factors of safety for the models shown in Figure 4.12 are not necessarily correct, the models do point out the relative value of shotcrete in supporting different types of loading by mobilization of different shotcrete strength parameters. The following conclusions can be drawn from the analyses:

(1) The mere filling of overbreak cavities and depressions on the rock surface may prevent progressive fallout by holding up single blocks, provided the surrounding rock is stable.

(2) If a shotcrete lining cannot carry thrust by closed-ring action or by bearing onto the tunnel floor, i.e., if it is a discontinuous application, then the rock-shotcrete bond strength limits the load-carrying capacity of the shotcrete.

(3) The stability of single rock blocks at the periphery of a shotcreted opening is strongly dependent on the size of the block that tends to move into the opening.

(4) More shotcrete is required for the prevention of block rotations than for the prevention of block fallout.

(5) Depending on the orientation of blocks with respect to the opening, thrusting from tangential

stresses may induce a more severe loading in the shotcrete than dead weight loading. It is to be expected that thrusting from large blocks of rock ($a > 2m$) could not be carried by shotcrete. Deere et al. (1969) reached the same conclusion. In such cases, the magnitude of the tangential stresses and the frictional resistance along rock joints are important in determining the load on the shotcrete.

(6) It is very likely that the actual role of shotcrete in supporting tunnels in jointed rock is a combination of the support mechanisms shown in Figure 4.11. Under different conditions different modes of support may predominate. For example, a six-inch continuous shotcrete application over the entire tunnel cross section could function as an overbreak-filling as well as a structural-type member capable of supporting a significant dead load weight. However, its sole function in a given tunnel section might be the prevention of rock block rotations.

"Design" of Tunnel Reinforcements and Supports in Sweden

The primary function of shotcrete in most Swedish tunnels is to serve as a deterrent to block fallout by pure slip along joints, block rotations, and block thrusting through the action of tangential rock stresses. However, because the predominant mode of support may change erratically over very short distances in a tunnel, and because several different modes of behavior may exist at one point, the design of shotcrete linings is not possible by the application of one of the simple model analyses of Figure 4.12. In fact, the sudden changes in geologic conditions that are common in most Swedish tunneling preclude the possibility of applying any general support design procedures that account for the local conditions which are encountered meter-by-meter in a tunnel. Although the rock bolt-shotcrete support system is very highly adaptable to a wide variety of rock conditions, the actual adaptation of the system to meet local conditions is a responsibility that lies in the hands of job site personnel, and not the design engineer.
In Sweden, reinforcement or support measures are evaluated by the tunnel foreman, miners, and tunnel engineer as the face advances. Representatives of the owner may also take part in the evaluation, but there are no computations or other systematic methods involved in the decision as to what type and how much support is to be applied. As a matter of fact, there do not even exist any guidelines or rules-of-thumb for application to tunnel supports. The rock quality-tunnel support relationships that are presented in the next chapter are attempts to provide needed information in this area of tunnel design.

5. EMPIRICAL RELATIONSHIPS BETWEEN ROCK QUALITY PARAMETERS AND TUNNEL SUPPORT

5.1 Introduction

Nature of Tunnel Stability Problem

It should now be clear that the tunnel stability problem is a complex one that involves many theoretical as well as constructional factors. Although theoretical numerical analyses of tunnel stability have been used for several real problems (Goodman et al., 1968) it is doubtful that it will ever be economically feasible to apply them to tunneling projects that involve long lengths of running tunnels and rapidly changing conditions. The quantity of information needed for theoretical analyses and the difficulty and expense involved in acquiring this information necessarily limit the use of such analyses to special situations. Similarly, because physical model studies are so expensive and require the same input data as an analytical model, their use is limited to special problems.

With the current geologic exploration and rock mechanics testing methods it is not economically feasible to attempt a pre-construction, or even post-construction rational analysis of the support requirements for a long tunnel through widely varying and rapidly changing rock conditions.

Empirical Approach

Empiricism is an attractive and frequently used approach to engineering problems that either do not lend themselves to rational solution or are more easily studied on the basis of observation and experimentation. The selection of tunnel supports fits this category very well. There is no question that tunnels today are designed almost solely on the basis of experience. All existing suggestions and guidelines for the design of rock tunnel supports are based on either past experience or pure guesses. Although some empirical correlations between rock quality and steel set supports have been derived from experience (Terzaghi, 1946; Deere et al., 1969; Monsees, 1970), there exist no such data for rock bolted and shotcreted rock tunnels. Because the application of shotcrete to underground support has generally been a very uncertain and mysterious process, even empirical rules would be welcome to the present state of the art. The purpose of this chapter is to derive empirical correlations between rock quality and support used in the observed field cases.

Any empirical relationship should be founded on those parameters that most significantly influence the behavior under consideration. In the case of tunnel stability behavior, rock quality is probably the most important single parameter that determines support requirements. If sound empirical relationships between rock quality and tunnel support are to be derived, it is necessary first to establish indices or classifications of rock material that are influenced by the parameters which most significantly influence tunnel stability. A review of some existing rock quality indices, parameters, and classifications is given in this chapter.

Time-Stability-Support Considerations

The classification of the observed cases according to the time-stability-support interaction given in Table 2.3 suggests that the time of placement of supports is an important factor that must be taken into consideration in any attempt to relate support to rock mass properties. Although this is indeed true, no attempt is made in this chapter to separate the various cases on the basis of time of support installation. The following reasons are given for not treating the data in this manner.

(a) Stand-up times for all the cases are not known.

(b) The supports used are not necessarily the same as those required; that is, the factor of safety against failure is not known for any of the cases.

It can be surmised that the supports used in many of the cases would have been less had they been installed immediately after blasting. The lumping together of all the cases as is done in this chapter, thus, can be expected to give relatively conservative correlations between rock quality and support.

5.2 Existing In-Situ Rock Classification Systems and Rock Quality Parameters

Review of Existing Systems and Parameters

In addition to the classification systems of Bergman, Coates, and Hagerman that have been mentioned earlier, there exist countless rock classification systems that have been developed by different people for different purposes. Some of the systems that are oriented towards tunnel support will be discussed in the following paragraphs.

Merritt (1968) reviewed a large number of in-situ rock classification systems and found that existing systems are based on either laboratory values of strength and deformation, or on some description of the jointing and weathering of the rock mass. It is obvious from the case studies presented in Chapter 3 that the stability behavior of tunnels in loosening ground is governed primarily by geologic discontinuities and alterations of the rock mass. Although a number of the classification systems described by Merritt take into consideration the spacing and condition of discontinuities and rock alteration, very few provide a quantitative measure of these features. In his extensive studies of various core recovery measures and geophysical measurements, Merritt found Deere's RQD (1964) and the velocity index (square of the ratio of in-situ to intact compressional wave velocities) to be the simplest and most reliable means of quantifying in-situ rock quality. His proposed engineering classification for in-situ rock is shown in Table 5.1. The descriptions and RQD divisions are the same as those given by Deere et al. (1967).

Coon (1968) has correlated the RQD and velocity index with in-situ rock mass deformability, rate of tunneling, and underground support requirements. The latter correlation was derived from tunnel projects where rock bolts and steel arches were used as support. The relationship that Coon found between RQD, span width, and support requirements is shown in Figure 5.1.

TABLE 5.1

MERRITT'S ENGINEERING CLASSIFICATION FOR IN-SITU ROCK (after Merritt, 1968)

RQD (percent)	Velocity index	Description
0-25	0-0.20	Very Poor
25-50	0.20-0.40	Poor
50-75	0.40-0.60	Fair
75-90	0.60-0.80	Good
90-100	0.80-1.0	Excellent

Although Coon did not describe the ground conditions at the projects from which his data were taken, it is assumed that loosening pressures were the principal source of instability.

As mentioned previously, seismic refraction data has been used very recently (Scott et al., 1968) to correlate rock quality and support requirements in the Straight Creek highway tunnel in Colorade. The correlations between seismic velocity, steel set spacing, and construction rate that were derived from the Straight Creek work are shown in Figure 5.2. Similar empirical correlations were found between seismic velocity and quantity of lagging and blocking and the height of the tension arch above the opening. These construction parameters have also been correlated with a numerical rock quality rating that is based on fracture spacing, percent mineral alteration, degree of faulting, foliation and schistosity, and rock type. This classification of rock quality is based on the local geologic conditions at Straight Creek and would require modification for application to other geologic conditions. The degree of personal judgements required in such a classification makes it undesirable for general use by different people.

Terzaghi's (1946) well known recommendations for rock loads on steel supports are based on a visual, verbal rock classification. The classification is not merely a physical description of the rock mass, but rather specifies an expected behavior of the rock mass when supported with conventional steel sets. The system thus may not be well suited for support with rock bolts and shotcrete.

Linder (1963) has applied Lauffer's (1958) rock classification system to tunnels supported with shotcrete and rock bolts. The recommended shotcrete thicknesses and alternate support systems are shown in Figure 5.3. This system also involves a very subjective classification of the rock and for this reason is not used in the analysis of the writer's field observations.

Hansági (1965, 1967) describes an arbitrary, empirical method for numerically evaluating the rock mass strength from drill core and unconfined compressive strength data. A rather complex statistical treatment of the data is used to evaluate a correction factor that is multiplied by the unconfined compressive strength of intact cores in order to arrive at a strength of the rock mass. Although Hansági (1967) has given support requirements for different values of his so-called "gefuge" factor, the method is considered unsuitable for two reasons: First, diamond drilling is necessary either along the tunnel axis or radially outward in the tunnel walls at four or five points on the tunnel section under consideration. Such drilling is never practiced in Swedish hydroelectric tunnels and openings. Second, and more important, the roof falls and other instability normally encountered in shallow (< 300 m tunnel construction in Sweden are the loosening-type of instability and are in no way related to the strength of the intact rock.

Conclusions

In-situ rock classifications and rock quality parameters or indices that offer attractive potential means of relating rock quality and support requirements fall into three general categories:

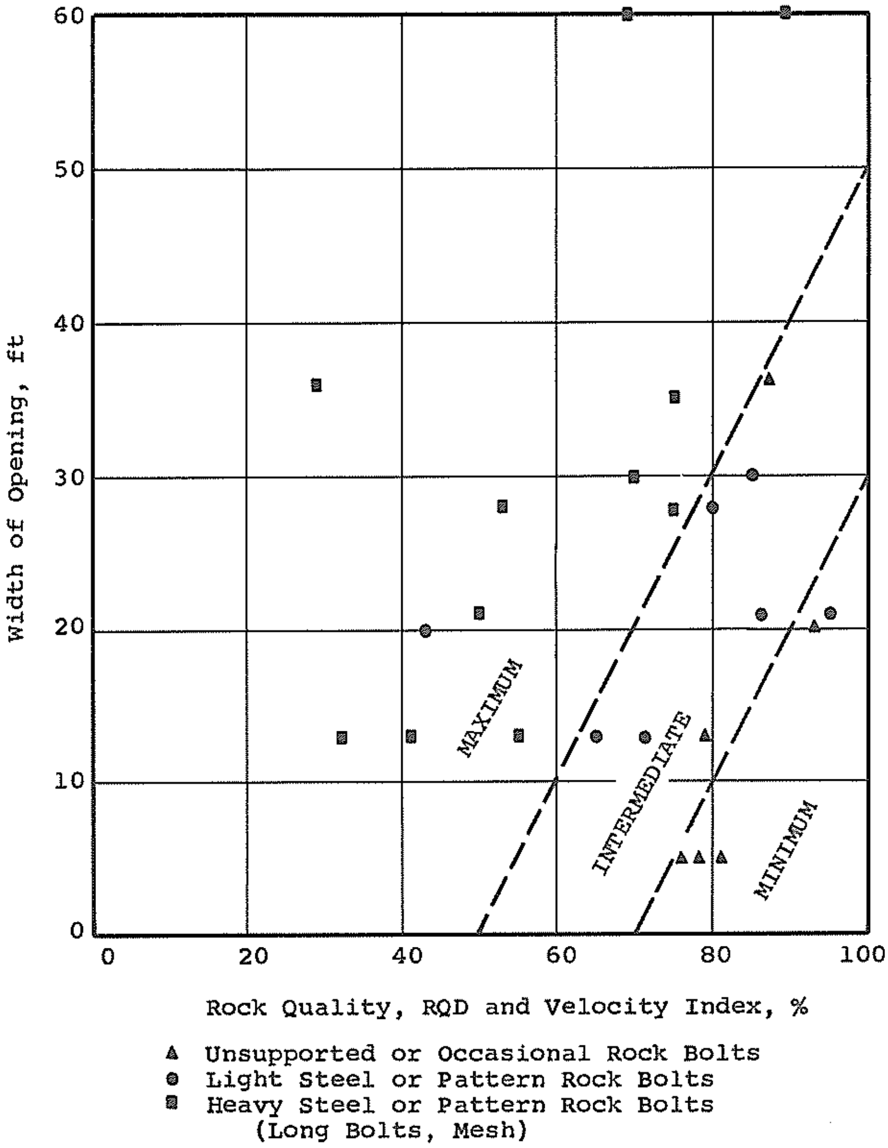


Fig. 5.1 Comparison of support requirements by width of opening and rock quality (after Coon, 1968)

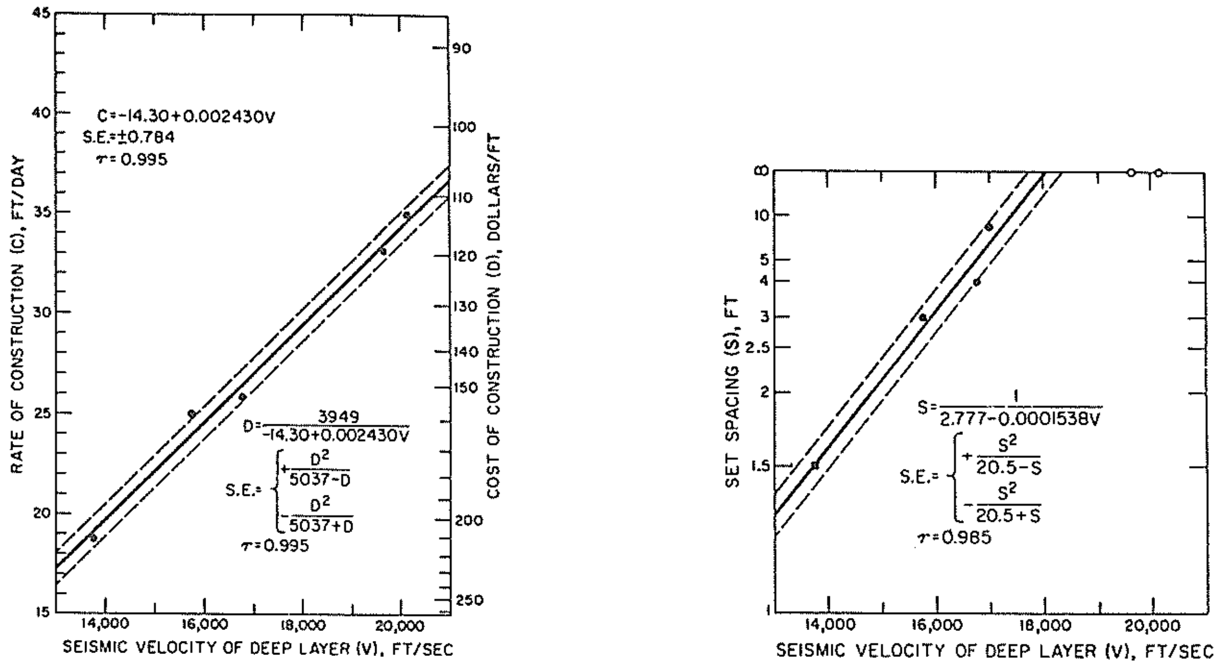


Fig. 5.2 Correlations of construction parameters and geophysical properties in straight creek tunnel (after Scott et al., 1968)

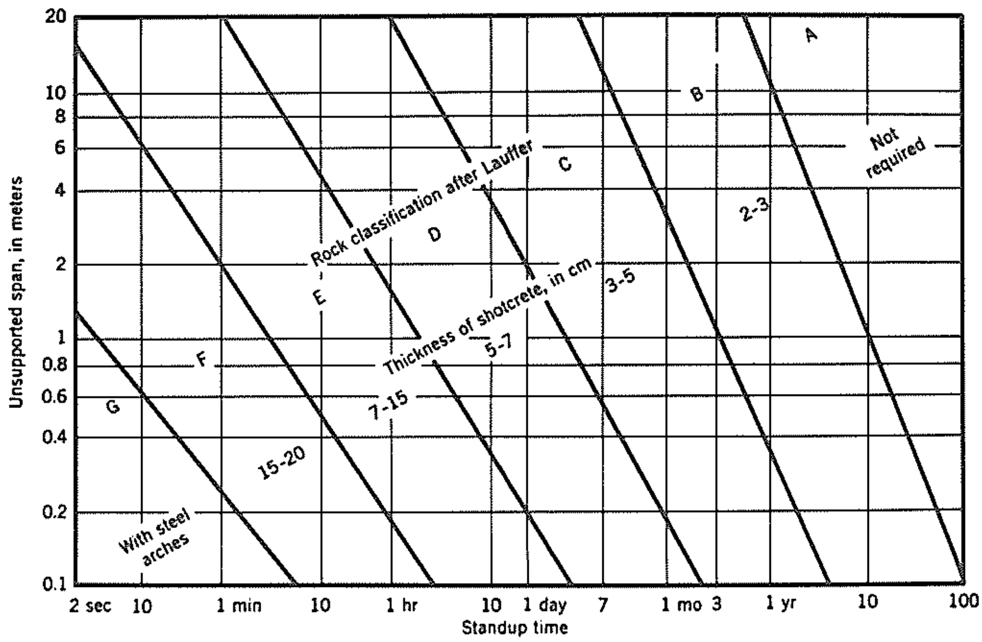


Fig. 5.3 Rock reinforcement with shotcrete according to Lauffer-Linder classification system (after Linder, 1963)

Reinforcements for rock classes

(A) No reinforcement required.

(B) 2-3 cm shotcrete; alternatively rock bolts on 1.5-2 m spacing with wire net, occasionally reinforcement needed only in arch.

(C) 3-5 cm shotcrete; alternatively rock bolts on 1-1.5 m spacing with wire net, occasionally reinforcement needed only in arch.

(D) 5-7 cm shotcrete with wire net; alternatively rock bolts on 0.7-1 m spacing with wire net and 3 cm shotcrete.

(E) 7-15 cm 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) 15-20 cm shotcrete with wire net and steel arches; alternatively strutted steel arches with lagging and subsequent shotcrete.

(G) shotcrete and strutted steel arches with lagging.

(1) purely descriptive, verbal classification systems, such as Terzaghi's or Bergman's.

(2) numerical systems based on relatively simple and inexpensive measurements, such as Deere's RQD.

(3) numerical systems based on elaborate, expensive measurements, such as seismic refraction velocities and other geophysical parameters.

In the following sections of this chapter attempts are made to use some form of each of these three classification types to relate rock quality to the supports used in the observed cases. The approach is admittedly a simplification of a very complex problem that involves many factors in addition to rock quality.

5.3 Relationships Between Tunnel Support and Visual, Verbal Rock Classification Systems

Selected System

The visual, verbal rock classification systems considered for the analysis of the field observations are those of Bergman (1965) and Terzaghi (1946). Hagerman's system (1966) is more or less incorporated into the modified and expanded Bergman classification given in Table B.1 of Appendix B. The factors listed in this table can be evaluated relatively easily and objectively.

Considerable difficulty was encountered in attempting to apply Terzaghi's rock classification system to the observed field cases. The rock conditions in a great many of the observed cases are not well described by any of the general terms, such as moderately blocky and seamy, used in Terzaghi's system. When the system was forcefully applied, it was found that the classification of many cases is arbitrary. Furthermore, the broad scope of some of the rock classes necessarily leads to heavy concentrations of widely different geology and tunneling conditions in the same rock class. For example, there is no doubt that Cases 26, 32, 33, 35 and 44 must be classified as very blocky and seamy rock according to Terzaghi's system. Yet the tunneling conditions in these cases are very different, principally because of differences in seemingly minor geologic details, such as clay fillings, discontinuity orientation, and tightness of joint structure. In conclusion, it was found that Terzaghi's system is too general to permit an objective evaluation of rock quality to be made. Seemingly minor geologic details, such as clay seams and thin shear zones, that can have a very large effect on tunnel stability, may not influence the classification of a rock mass in the Terzaghi system. As will be seen later, this problem is common to most measures of rock quality. The modified Bergman classification in Table B.1 is an all inclusive visual, verbal system that includes most of the descriptive terms used in Terzaghi's system. For these reasons, the latter was abandoned.

In order to attempt to correlate the modified Bergman check-list, verbal classification system with tunnel support requirements, it was necessary to choose a few select parameters from the system and compare these with the support actually used in the observed cases. The parameters chosen are rock mass structure and average joint spacing. The rock mass structure has been classified according to the designations given in Table 5.2. The classification is an attempt to grade different structures in increasing order of difficulty that they are likely to cause in a tunnel. The numbers assigned to the different rock mass structures are strictly for classification purposes and are not intended to convey any numerical degree of difficulty in tunneling. For example, the designation 6 used for three discontinuity sets does not mean that this rock is twice as difficult to mine or requires twice as much support as the rock with a classification of 3. The rock mass structure classifications have been made from the information given in Table B.1. Joint spacing classification has been made after Deere's (1963) recommendations. The rock mass structure classification, average joint spacing, span width, and support used are given in Table 5.3 for all of the observed cases. The notations "s" and "sc" after the case number indicate "special case" and "softening clay", respectively. These cases are not used in the correlations that follow.

TABLE 5.2

ROCK MASS STRUCTURE CLASSIFICATION

Classification	Description
1	Massive, no or few discontinuities
2	One discontinuity set
3	One discontinuity set and random discontinuity
4	Two discontinuity sets
5	Two discontinuity sets and random discontinuity
6	Three discontinuity sets
7	Three discontinuity sets and random discontinuity
8	Crushed rock or earth-like material

Because of the importance of span width in tunnel stability in jointed rock, an attempt has been made to take this factor into consideration in some of the correlations by dividing the average joint spacing of each case by the span width. This normalized average joint spacing is given in Table 5.3 as a percent of the span width. The actual values of average joint spacing used for the computations are 5 cm, 20 cm, 60 cm, 150 cm, and 300 cm for the respective average joint spacing classifications of < 5 cm, 5-30 cm, 30 cm-1 m, 1-3 m, and > 3 m.

TABLE 5.3

SUMMARY OF VISUAL CLASSIFICATION PARAMETERS FOR FIELD CASES

Case No. (1)	Span Width, m (2)	Rock Structure Classification (3)	Average Discontinuity Spacing (4)	$\frac{(4)}{(2)}$, % (5)	Rock Support[3] (6)
1 s[1]	9	4	1-3 m	16.7	▼●○
2	9	4	1-3 m	16.7	▼○
3	9	6	5-30 cm	2.2	○
4	9	2	5-30 cm	2.2	○○
5	9	2	5-30 cm	2.2	○
6	9	2	5-30 cm	2.2	△
7	9	3	5-30 cm	2.2	▼○
8	9	2	5-30 cm	2.2	△
9	9	6	5-30 cm	2.2	○
10	9	6	5-30 cm	2.2	○
11	9	2	5-30 cm	2.2	▼○○
12	9	3	30 cm-1 m	6.7	□
13	9	5	1-3 m	16.7	▼○
14	9	5	30 cm-1 m	6.7	▼○
15	9	5	1-3 m	16.7	▼○
16	9	7	5-30 cm	2.2	▽○○
17	9	2	1-3 m	16.7	□
18 sc[2]	9	8	<5 cm	0.56	○○○○
19 sc	9	8	5-30 cm	2.2	○○○○
20	9	2	5-30 cm	2.2	△

Notes:

1 "s" indicates special case

2 "sc" indicates softening clay

3 Support Legend

- △ no support
- □ widely spaced (> 5 m) spot bolts
- ▽ medium spaced (2-5 m) spot bolts
- ▼ closely spaced (< 2 m) spot bolts
- ○ single shotcrete application (4-6 cm)
- ■ cast concrete arch
- ● reinforced shotcrete

Symbols used in combination, i.e. ▽○○ indicates medium spaced spot bolts and two shotcrete applications.

TABLE 5.3, Continued

Case No (1)	Span Width, m (2)	Rock Structure Classification (3)	Average Discontinuity Spacing (4)	$\frac{(4)}{(2)}$, % (5)	Rock Support[3] (6)
21	13	1	> 3 m	23.1	△
22 s	4	5	1-3 m	37.5	▼
23	12.5	3	5-30 cm	1.6	▼●○
24	12.5	5	5-30 cm	1.6	▼●
25	12.5	5	5-30 cm	1.6	▼○
26	12.5	4	30 cm-1 m	4.8	▽○
27	12.5	4	30 cm-1 m	4.8	△
28	12.5	5	5-30 cm	1.6	▽○
29	12.5	2	1-3 m	12.0	△
30	15	3	<5 cm	0.33	□○
31	12	4	30 cm-1 m	5.0	□
32	12.5	6	30 cm-1 m	4.8	▼
33	12.5	6	5-30 cm	1.6	▼○○
34 s	5	4	5-30 cm	4.0	▽
35	5	3	<5 cm	1.0	△
36	5	2	5-30 cm	4.0	△
37	5	5	5-30 cm	4.0	▽●○
38 s	5	5	5-30 cm	4.0	▼
39	8	6	5 cm	0.625	■
40 sc	8	5	5-30 cm	2.5	■
41 sc	8	2	5-30 cm	2.5	■
42 sc	8	5	5-30 cm	2.5	■, ○○○
43 sc	9	2	5-30 cm	2.23	■
44	9	5	<5 cm	0.56	○
45 sc	9	4	1-3 m	16.7	□○○
46 s	7	4	5-30 cm	2.9	▽●
47 sc	6.5	2	<5 cm	0.77	▽●○
48	6.5	2	5-30 cm	3.1	▽●○
49 sc	6.5	3	5-30 cm	3.1	○○
50	6.5	2	5-30 cm	3.1	○○
51	6.5	2	< 5 cm	0.77	▽●○
52	6.5	2	< 5 cm	0.77	△
53	6.5	6	5-30 cm	3.1	▼●○
54 sc	4.2	3	30 cm-1 m	1.43	▽●○
55 sc	4.2	2	5-30 cm	4.77	▼●
56	5.9	8	5-30 cm	3.4	○○
57	5.9	8	5-30 cm	3.4	▼○○
58	5.9	4	1-3 m	25.3	▽
59	5.9	5	5-30 cm	3.4	□○
60	5.9	2	5-30 cm	3.4	▽○
61 sc	5.9	5	30 cm -1 m	10.2	□○
62	5.9	5	30 cm -1 m	10.2	□○
63	5.9	6	1-3 m	25.3	▽
64	5	6	1-3 m	30.0	▽
65	5.9	6	1-3 m	25.3	▽
66	7	4	5-30 cm	2.86	▽○
67	5.9	2	5-30 cm	3.4	○
68	10	1	> 3 m		△
69 sc	8	4	1-3 m	18.9	▽■
70	8	2	5-30 cm	2.5	△
71 sc	9.5	5	5-30 cm	2.11	■
72 sc	9.5	5	1-3 m	15.8	■
73 sc	9.5	3	1-3 m	15.8	■
74	12	2	1-3 m	12.5	△
75 s	12	2	1-3 m	12.5	▽
76 s	8	5	1-3 m	18.8	▼■
77	20	1	> 3 m	15.0	△
78	5	2	30 cm -1 m	12.0	△
79	11.25	8	< 5 cm	0.45	○○○, ■
80	11.25	3	5-30 cm	1.78	○○

TABLE 5.3, Continued

Case No. (1)	Span Width, m (2)	Rock Structure Classification (3)	Average Discontinuity Spacing (4)	$\frac{(4)}{(2)}$, % (5)	Rock Support[3] (6)
81	11.25	5	5-30 cm	1.78	CO
82	11.25	6	30 cm -1 m	5.3	▼O
83	11.25	4	5-30 cm	1.78	△
84	11.25	7	5-30 cm	1.78	C
85	11.25	8	5-30 cm	1.78	COO
86	11.25	4	1-3 m	13.33	▽
87	11.25	1	> 3 m	26.7	△
88	11.25	4	5-30 cm	1.78	▼OO
89	11.25	3	30 cm -1 m	5.34	C
90	11.25	2	30 cm -1 m	5.34	▽
91	12	2	5-30 cm	1.67	△
92	20	3	5-30 cm	1.0	▽
93 sc	12.6	---	---	---	CCCC, ■
94 sc	12.6	---	---	---	CCCC, ■
95	22	---	---	---	△
96	15	---	---	---	△
97	18	---	---	---	□

Relationship Among Tunnel Support, Span Width, and Average Discontinuity Spacing

The support used in all of the cases, excluding the special cases, is shown in Figure 5.4 as a function of average joint spacing and span width. The following observations can be made from this plot:

(a) There were no observed cases in which support was used in rock with an average discontinuity spacing greater than three meters.

(b) Unsupported tunnels exist in rock masses of a wide range of average joint spacings.

(c) Unstable tunnels in rock whose average discontinuity spacing is greater than 30 cm are usually supported by either rock bolts alone, a single shotcrete spplication, or combined rock bolts and a single shotcrete application. Multiple shotcrete applications or heavier tunnel supports were used in only five of the thirty-two cases in which the average discontinuity spacing was greater than 30 cm.

(d) 32 of the 57 cases (about 56 %) in rock whose average discontinuity spacing is less than 30 cm were treated with maximum support measures (multiple shotcrete applications, sometimes in combinations with rock bolts; cast concrete arches).

(e) No relationship exists between average discontinuity spacing and the specific support used in the observed cases.

(f) The frequency of close rock bolt spacings (< 2 m) increases significantly for cases in which the average discontinuity spacing is less than 30 cm.

(g) No relationship exists between support used and span width for the observed cases. However, it is obvious that for average discontinuity spacings of less than 30 cm, heavier supports are most common for all span widths.

The distribution of supported and unsupported tunnels according to average discontinuity spacing is shown in the lower portion of Figure 5.4.

The tunnel support chart shown in Figure 5.5 shows the same data in Figure 5.4, less those cases that involve some form of softening clay. The conclusions

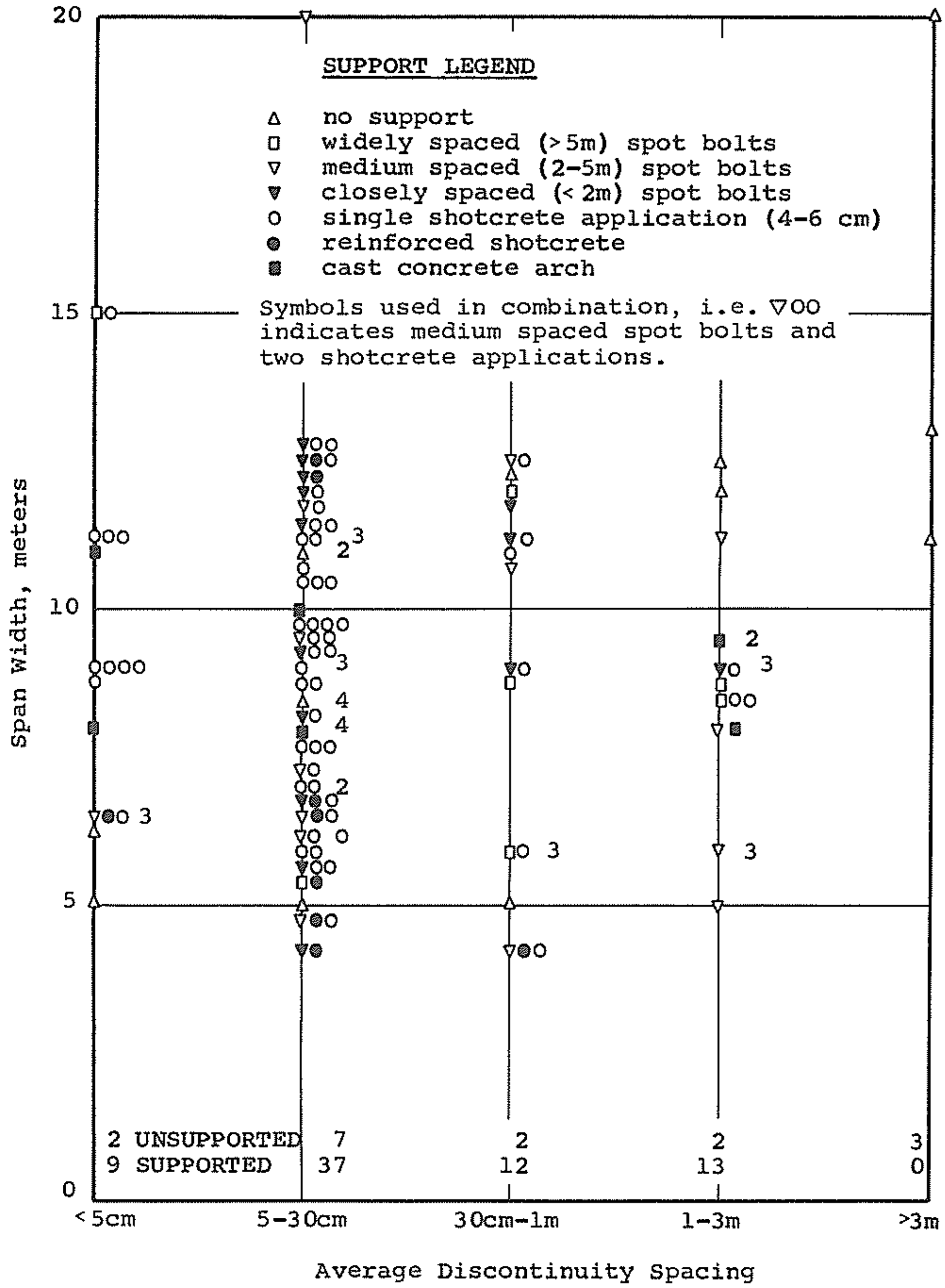


Fig. 5.4 Tunnel support chart based on span width and average discontinuity spacing - all cases

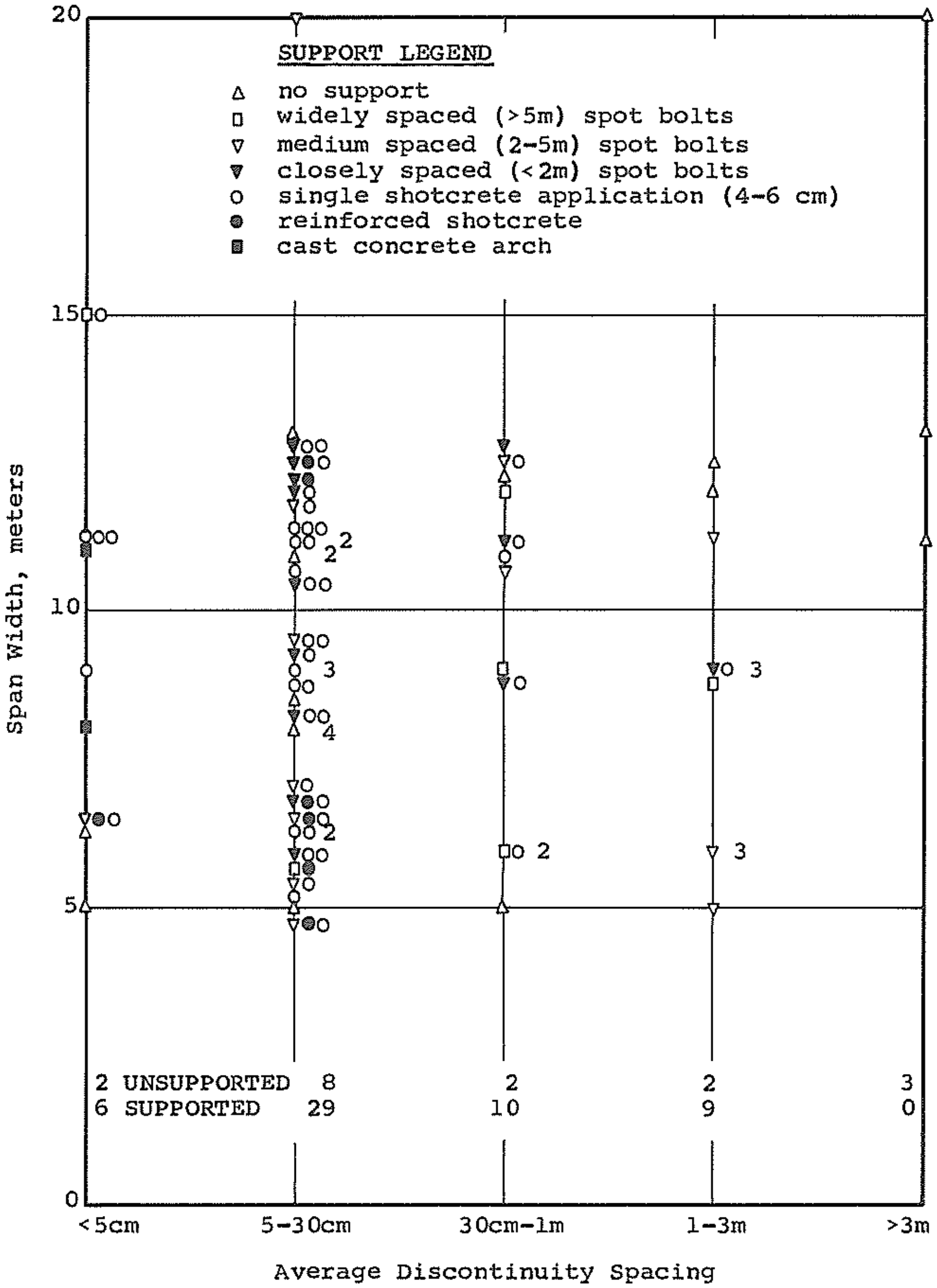


Fig. 5.5 Tunnel support chart based on span width and average discontinuity spacing - softening clay cases excluded

that can be drawn from this chart are similar to those drawn from Figure 5.4 with the following exceptions:

(a) All of the unstable cases in rock masses having an average discontinuity spacing greater than 30 cm were supported either by rock bolts alone or combinations of rock bolts and a single shotcrete application. No multiple shotcrete applications were used in this range.

(b) Only about 39 percent of the cases with an average discontinuity spacing of less than 30 cm were treated with maximum support measures.

Even when the softening clay cases are disregarded, it is not possible to draw any relationship between the specific support used and average discontinuity spacing. A very general, and very conservative, relationship that might be drawn from Figure 5.5 is given in Table 5.4. It should be realized that this relationship is only a set of upper bounds on the support that has been used for the different cases. Strict adherence to its use would obviously lead to over-conservative selection of support. The frequent occurrence of stable, unsupported tunnels in rock whose discontinuity spacing is less than 30 cm will be discussed in detail in the next section. It serves to point out here that close and very close joint spacing are not always synonymous with instability.

TABLE 5.4
RELATIONSHIP BETWEEN TUNNEL SUPPORT AND AVERAGE DISCONTINUITY SPACING

Average Discontinuity Spacing	Tunnel Support
> 1 m	None to widely spaced spot bolts (> 5 m)
30 cm-1 m	Medium to closely spaced bolts with one shotcrete application
< 30 cm	Multiple shotcrete applications, frequently with closely spaced (< 2 m) bolts

The distribution of supported softening clay cases is shown in Figure 5.6. It can be observed in this chart that no relationship exists between average discontinuity spacing, span width, and support used for these cases.

Relationships Among Tunnel Support, Span Width, and Rock Mass Structure

Tunnel support charts based on span width and rock mass structure are shown in Figure 5.7 (for all cases excluding the softening clay cases) and Figure 5.8 (for softening clay cases only). The following conclusions can be drawn from these two figures:

(a) Maximum support was generally used for crushed or earthlike rock (structure classification 8).

(b) Most of the stable cases in which no support was used have either no or only one set of discontinuities (classifications 1 and 2). All cases in which the rock mass structure classification was 5 or greater were supported to some degree. This limit is to be expected, as classification 5 is the lowest class in which three dimensional joint block are present (two discontinuity sets with random discontinuity).

(c) Absolutely no relationship exists between rock mass structure classification and support used. Maximum support was used throughout the range of rock mass structure classes. Not even the vaguest of trends exist with respect to support used and span width when the data are plotted as shown in Figure 5.7.

(d) The occurrence of softening clay materials is in no way related to the rock mass structure.

5.4 Relationships Between Tunnel Support and Rock Quality Designation (RQD

General

The Rock Quality Designation (RQD) offers a very simple and quantitative method to classify rock.

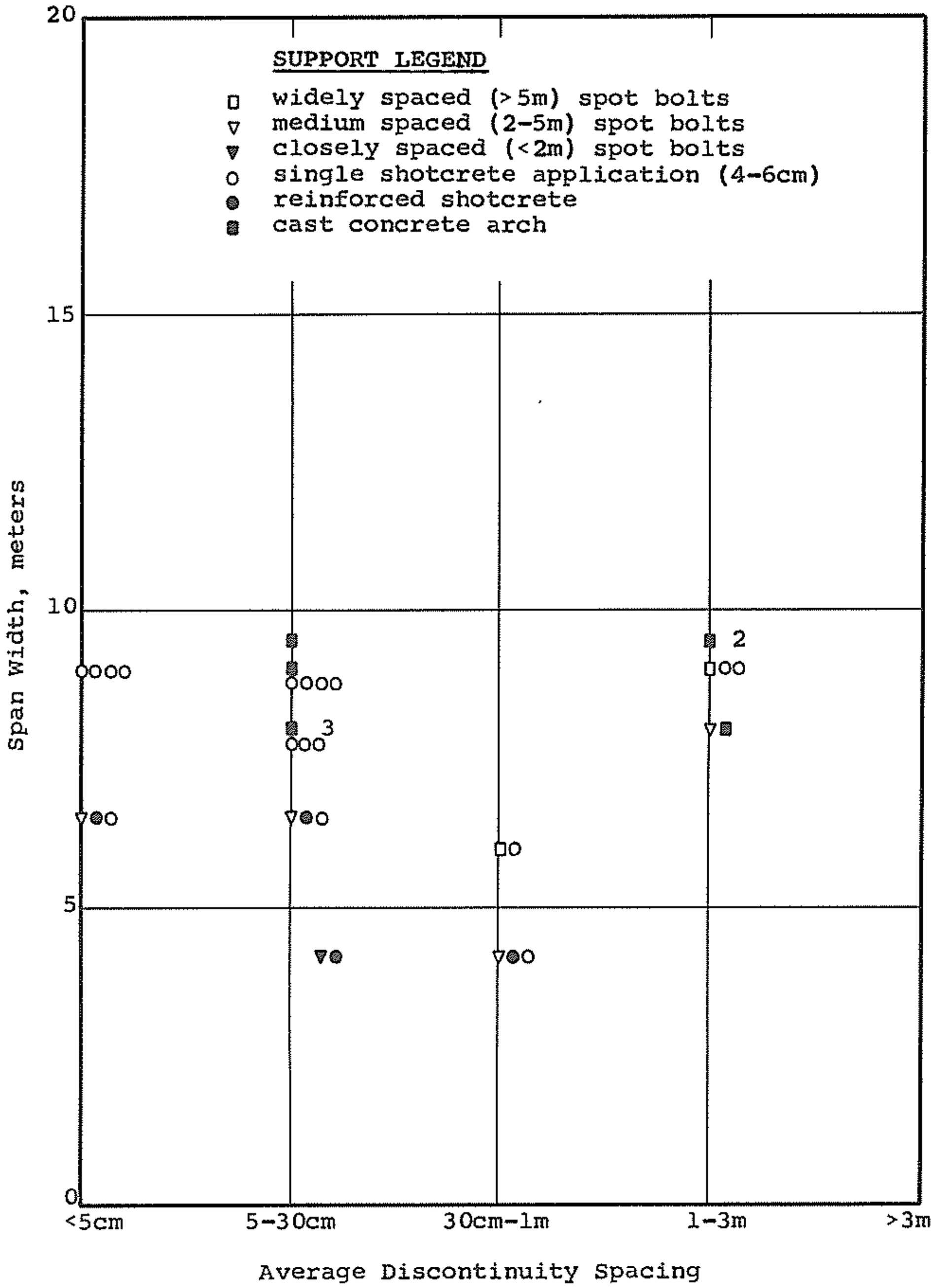


Fig. 5.6 Tunnel support chart based on span width and average discontinuity spacing - softening clays only

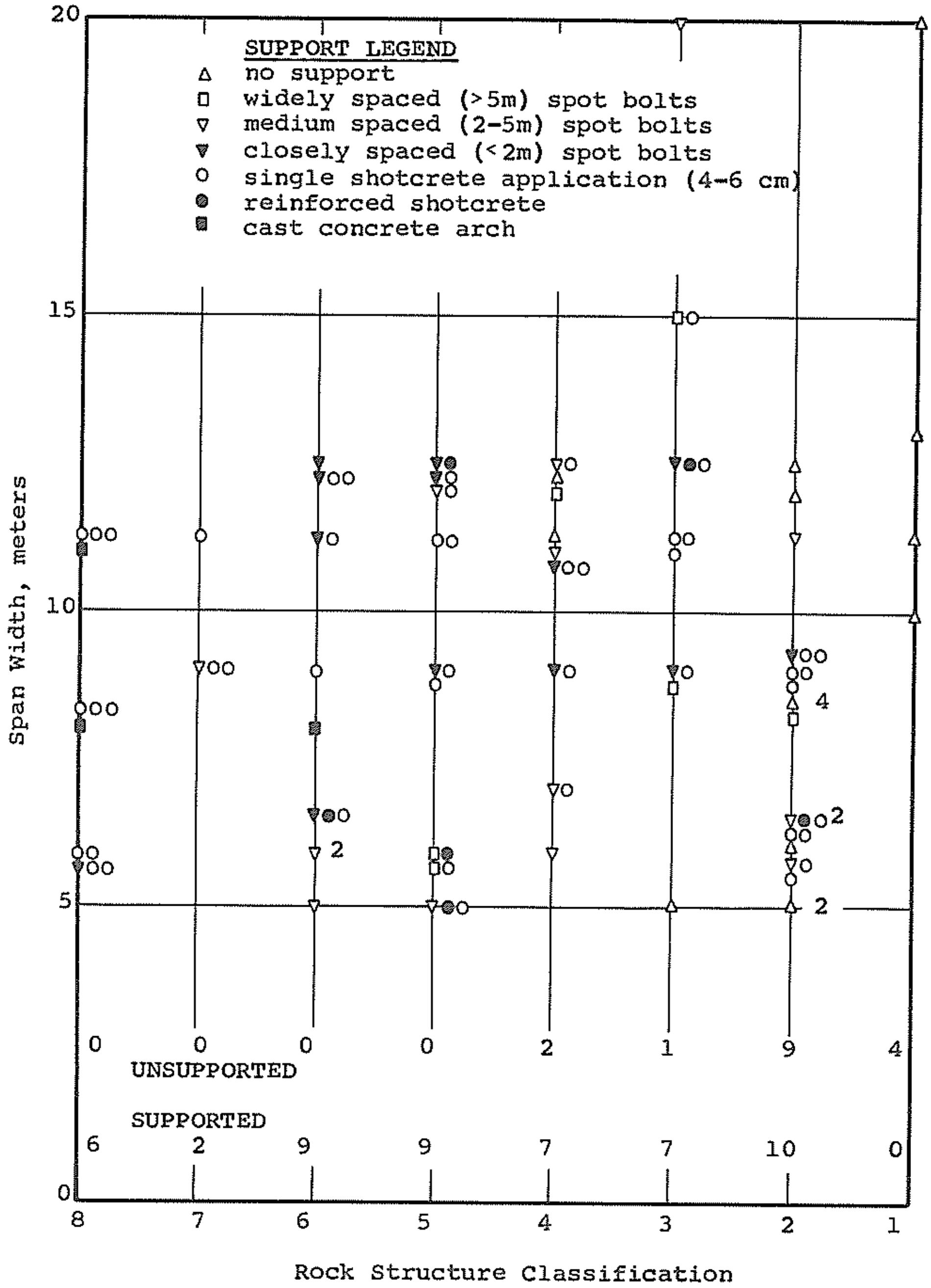


Fig. 5.7 Tunnel support chart based on span width and rock mass structure - softening clay cases excluded

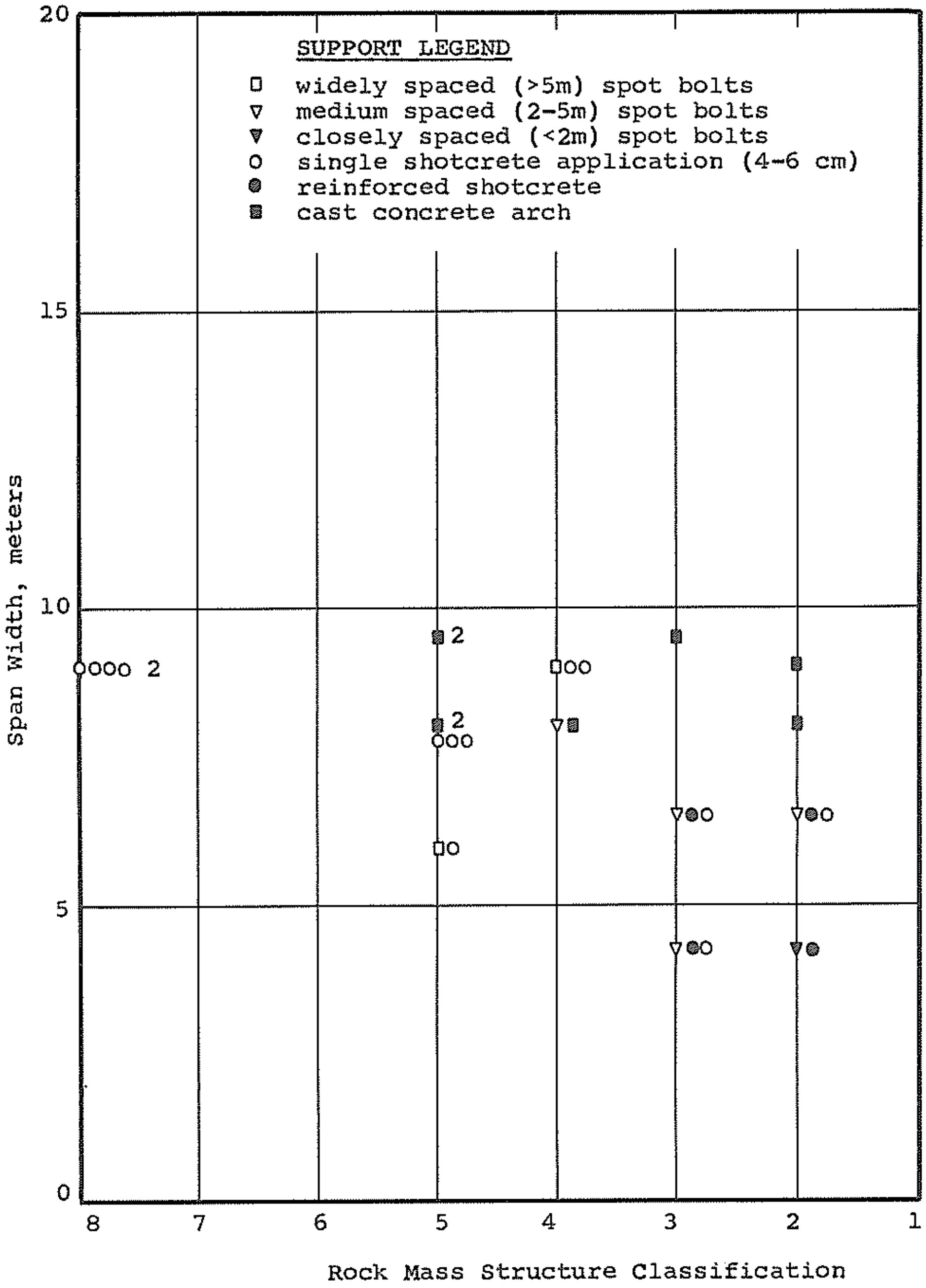


Fig. 5.8 Tunnel support chart based on span width and rock mass structure - softening clay cases

It has the advantage of being a continuous, numerical classification in contrast to the group-type classifications discussed in the previous section. The relationships between RQD and support used in the field observations are the subject of this section.

RQD values, span width, seismic velocity ratio, and support for all of the field cases are summarized in Table 5.5. The primary RQD value is listed under the "other RQD" column when it does not correspond to either the vertical or centerline RQD value. The primary RQD value is underlined in those cases where it corresponds to a specific direction. Seismic velocity ratios shown in parentheses are those from projected ground surface data.

Relationship Among Tunnel Support, Span Width and Primary RQD

All Cases. A tunnel support chart based on primary RQD and span width is shown in Figure 5.9. The data include softening clay cases. The limits shown for the different degrees of support are not well-defined, as there is a large scattering and overlapping of cases, particularly in the intermediate range where several cast-concrete arch and multiple shotcrete applications (maximum support) cases exist.

Statistics for the distribution of the cases in Figure 5.9 according to support classification are given in Table 5.6. It is seen, for example, that the maximum support boundary is actually an envelope for 80 percent of the cases in which maximum support was used. Fifteen percent of the cases in which maximum support was used fall into the intermediate classification and five percent into the minimum classification.

These figures indicate the strength, or inclusiveness of the different support classifications. Such a table will be shown for all succeeding support charts. The slope of the boundaries between the support categories will be discussed in a later paragraph.

Softening Clay Cases. The primary RQD-span width tunnel support chart for only the softening clay cases is shown in Figure 5.10. There is obviously no grouping of cases and it can be concluded that for rock that contains softening clay and requires support in a tunnel, there is no relationship of rock mass structure, joint spacing, or RQD to the support that is used.

Exclusion of Softening Clay Cases. The primary RQD-span width tunnel support chart for all cases except the softening clay cases is shown in Figure 5.11. The same boundaries are shown on this chart as on the chart that includes all the cases. It is seen that the boundaries are somewhat more delineating when the softening clay cases are excluded. The data in Table 5.7 indicate this fact. It is of particular interest to note that the maximum support classification envelops 95 percent of the cases in which maximum support was used. Although the intermediate class envelops only 71 percent of the cases in which intermediate support was used, only 8 percent of the intermediately supported cases fall into the minimum support area of the chart. The delineated areas are thus reasonably conservative in indicating the support that might be predicted for a given RQD and span width.

Unsupported Tunnels. The unsupported tunnels (cases in which no support was used) that occur throughout the range of support are shown separately in Figure 5.12. The causes of the anomalous unsupported tunnels at RQD values of less than 90 percent have been discussed previously. The structural discontinuity data for these cases are summarized in Table 5.8. The following conclusions may be drawn concerning these eight unsupported tunnels:

(a) Most of the tunnels have only one discontinuity set.

(b) The strike of the discontinuities is of no apparent significance.

(c) Most of the tunnels have steeply dipping discontinuities.

(d) Most of the tunnels have either completely discontinuous joint (discontinuity) sets or some continuous and some discontinuous joints.

(e) All of the tunnels have tight, unfilled discontinuities.

TABLE 5.5

SUMMARY OF ROCK QUALITY DESIGNATION VALUES AND SEISMIC VELOCITY RATIOS FOR FIELD CASES

Case No.	Span Width, m	RQD_v	RQD_a	Other RQD	Seismic Velocity Ratio[3]	Rock Support[4]
1 s[1]	9	0-100	70	--	(0.63)	▼●○
2	9	0-100	90	--	(0.89)	▼○
3	9	60	70	--	--	○
4	9	50	50	--	--	○○
5	9	80	80	--	--	○
6	9	0-100	80	60	--	△
7	9	70	70	--	--	▼○
8	9	70	70	--	--	△
9	9	80	80	--	--	○
10	9	60	70	60	--	○
11	9	40	40	20	(0.76)	▼○○
12	9	100	100	--	--	□
13	9	80	80	--	(0.62 - 0.80)	▼○
14	9	70	70	--	(0.62 - 0.80)	▼○
15	9	80	80	--	(0.62 - 0.80)	▼○
16	9	70	60	--	--	▽○○
17	9	100	100	--	--	□
18 sc[2]	9	10	0-80	--	(0.48 - 0.77)	○○○○
19 sc	9	30	10-100	--	(0.48 - 0.77)	○○○○
20	9	90	80	70	--	△
21	13	100	100	--	--	△
22 s	4	100	100	--	--	▼
23	12.5	0-80	0	30	--	▼●○
24	12.5	0-80	0-100	60	--	▼●
25	12.5	70	10-90	--	--	▼○
26	12.5	80	40-100	--	--	▽○
27	12.5	90	50-100	--	--	△
28	12.5	40	0-80	--	--	▽○
29	12.5	90	90	--	--	△
30	15	10	0-20	--	--	□○
31	12	100	100	--	--	□
32	12.5	80	90-100	--	--	▼
33	12.5	40	0-90	--	--	▼○○
34 s	5	90	90	--	--	▽
35	5	0	0	--	--	△
36	5	20-80	20	--	--	△
37	5	20-40	20	--	--	▽○○
38 s	5	60	0-100	--	--	▼
39	8	20	20	--	--	■
40 sc	8	40-100	50	--	--	■

Notes:

1 "s" indicates special case

2 "sc" indicates softening clay

3 Values of seismic velocity ratio given in parentheses are taken from projected ground surface data

4 Support Legend

- △ no support
- □ widely spaced (> 5 m) spot bolts
- ▽ medium spaced (2-5 m) spot bolts
- ▼ closely spaced (< 2 m) spot bolts
- ○ single shotcrete application (4-6 cm)
- ● reinforced shotcrete
- ■ cast concrete arch

TABLE 5.5, Continued

Case No.	Span Width, m	RQD_v	RQD_a	Other RQD	Seismic Velocity Ratio[3]	Rock Support[4]
41 sc	8	60	10-90	--	--	■
42 sc	8	0-90	40	--	--	■, ○○○
43 sc	9	20	20	--	--	■
44	9	0	0	--	--	○
45 sc	9	90	90	--	--	□○○
46 s	7	0-80	30	30	--	▽●
47 sc	6.5	0	0	--	(.74)	▽●○
48	6.5	10	10	--	--	▽●○
49 sc	6.5	0-50	0-50	30	--	○○
50	6.5	60	40	--	--	○○
51	6.5	0	0	--	--	▽●○
52	6.5	0	0	--	--	△
53	6.5	60	60	50	--	▼●○
54 sc	4.2	0-100	0-100	70	--	▽●○
55 sc	4.2	80	80	80	--	▼●
56	5.9	0-30	10	--	--	○○
57	5.9	40	40	--	--	▼○○
58	5.9	90	100	--	--	▽
59	5.9	10-100	50	--	--	□●
60	5.9	80	80	--	--	▽○
61	5.9	30-100	70	--	--	□○
62	5.9	90	90	--	--	□○
63	5.9	100	100	--	--	▽
64	5	100	100	--	--	▽
65	5.9	60-100	90	--	(0.70)	▽
66	7	70	10-100	--	--	▽○
67	5.9	10-50	10-50	20	--	○
68	10	100	100	--	--	△
69 sc	8	90	90	--	--	▽■
70	8	0-100	40	--	--	△
71 sc	9.5	0-100	30	--	--	■
72 sc	9.5	0-100	70	--	--	■
73 sc	9.5	0-100	80	--	--	■
74	12	100	100	--	--	△
75 s	12	100	100	--	--	▽
76 s	8	0-90	0-90	70	--	▼■
77	20	100	100	--	--	△
78	5	90	80-100	--	--	△
79	11.25	0	0	--	--	○○○, ■
80	11.25	0-100	60	--	(0.68)	○○
81	11.25	20-60	30	--	0.63(0.67)	○○
82	11.25	90	80	--	0.86	▼○
83	11.25	10-80	70	--	1.0 (0.73)	△
84	11.25	60	60	--	0.79(0.54)	○
85	11.25	0-30	0-30	--	0.58-0.73 (0.73)	○○○
86	11.25	90	90	--	0.96	▽
87	11.25	100	100	--	1.0	△
88	11.25	50	60	--	--	▼○○
89	11.25	80	90-100	--	--	○
90	11.25	90	90	--	--	▽
91	12	0-100	90	--	--	△
92	20	0-100	30	--	--	▽
93 sc	12.6	0	--	--	(0.50-0.64)	○○○○, ■
94 sc	12.6	20	--	--	(0.50-0.64)	○○○○, ■
95	22	100	100	--	--	△
96	15	100	100	--	--	△
97	18	100	100	--	--	□

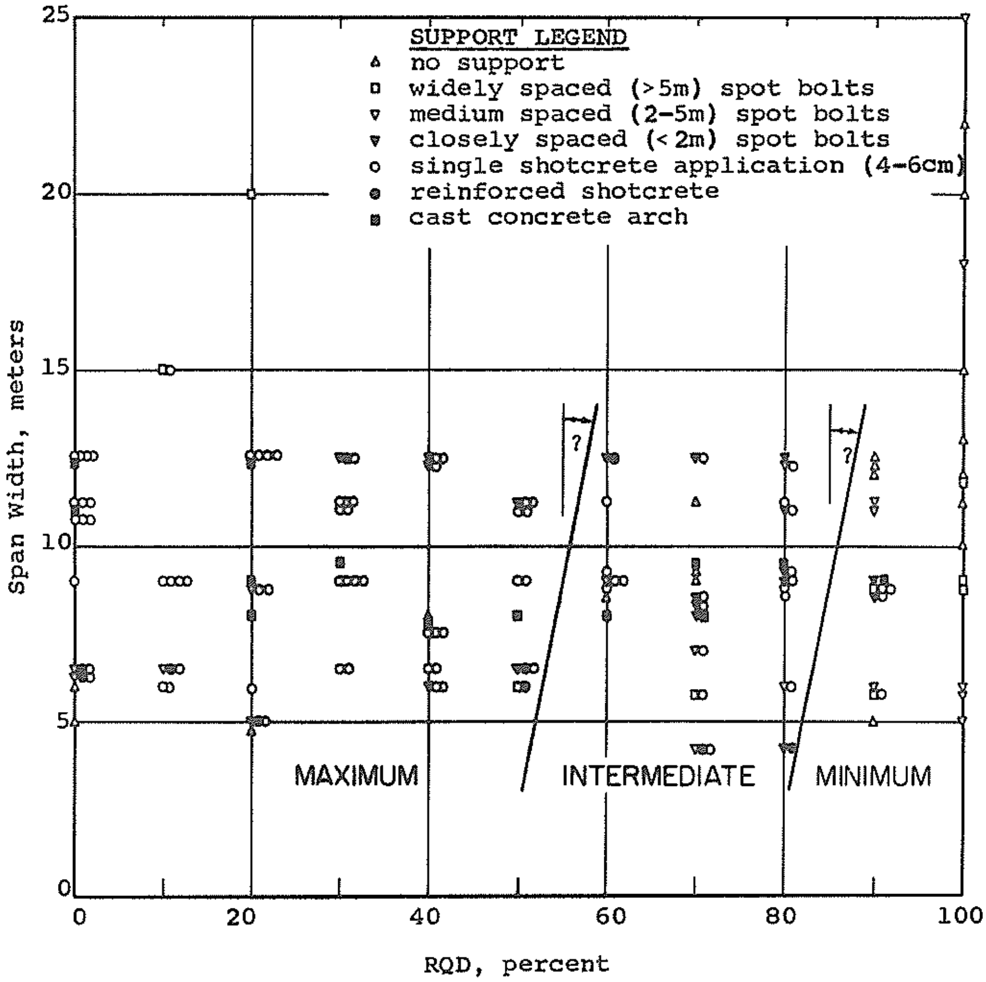


Fig. 5.9 Tunnel support chart based on primary RQD and span width - all cases

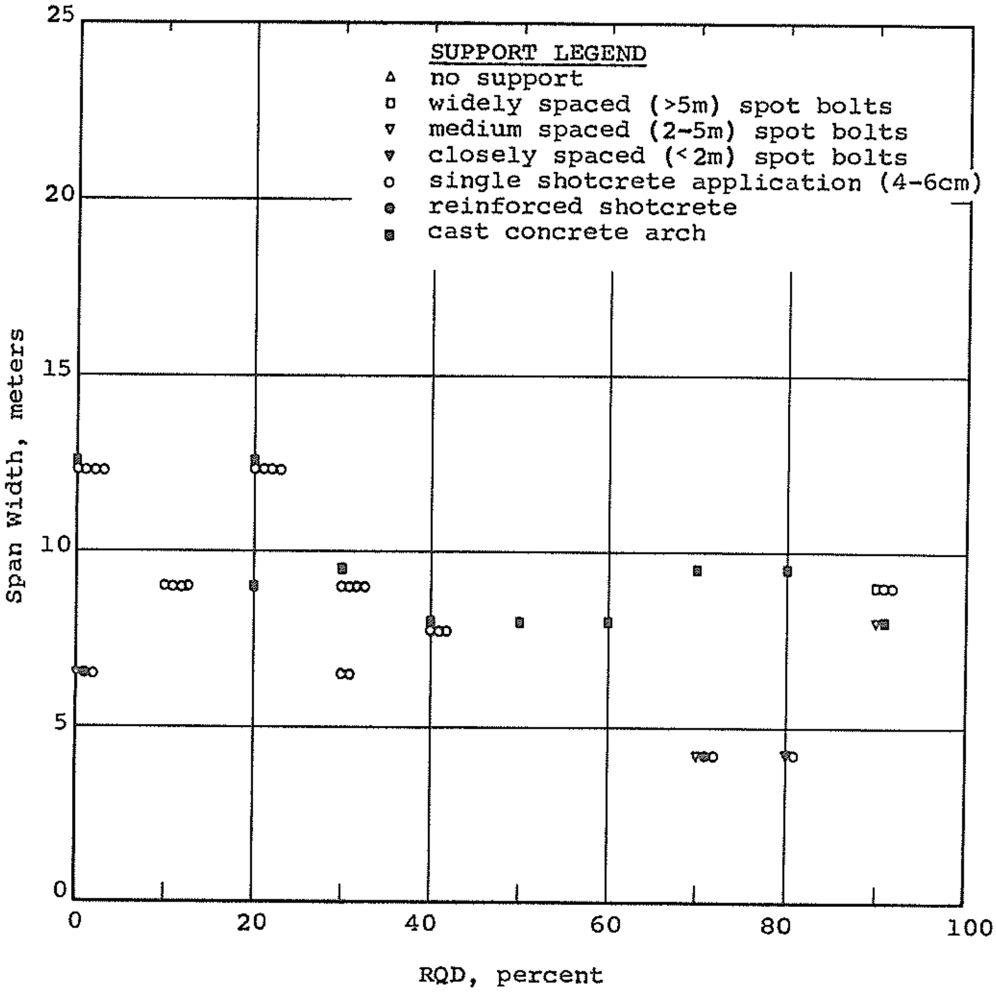


Fig. 5.10 Tunnel support chart based on primary RQD and span width - softening clay cases

TABLE 5.6

DISTRIBUTION OF CASES IN FIGURE 5.9 ACCORDING TO ACTUAL SUPPORT USED

Actual Support Used	Classification on Support Chart		
	Maximum	Intermediate	Minimum
Maximum (40 cases)	32/40 = 80%	6/40 = 15%	2/40 = 5%
Intermediate (25 cases)	5/25 = 20%	18/25 = 72%	2/25 = 8%
Minimum (32 cases)	5/32 = 15.5%	5/32 = 15.5%	22/32 = 69%

TABLE 5.7

DISTRIBUTION OF CASES IN FIGURE 5.11 ACCORDING TO ACTUAL SUPPORT USED

Actual Support Used	Classification on Support Chart		
	Maximum	Intermediate	Minimum
Maximum (20 cases)	19/20 = 95%	1/20 = 5%	0%
Intermediate (24 cases)	5/24 = 21%	17/24 = 71%	2/24 = 8%
Minimum (34 cases)	5/34 = 14%	6/34 = 18%	23/34 = 68%

The tangential skin stress at the periphery of the tunnels is of obvious significance. These stresses must be of sufficient magnitude to hold the discontinuous mass tightly together and prevent fallout. As pointed out in Chapter 3, there is evidence to suggest that the origin of the high crown skin stresses lies in high horizontal virgin rock stresses.

Discussion. The slope of the line dividing the different support categories in Figure 5.11 is uncertain. The unsupported and lightly supported tunnel spans greater than 12.5 meters indicate that the intermediate-minimum support boundary may not intersect the 100 percent RQD axis in the range of the chart. More cases in very wide (> 12.5 m) spans are needed to establish the boundaries in this range. It would be very unwise to extrapolate the boundaries shown in Figure 5.11.

A comparison of the support boundaries in Figure 5.11 and those suggested by Coon (1968) is shown in Figure 5.13. The differences in the locations of these boundaries can be attributed to two factors.

The quality of the rock is probably the single most important factor that accounts for the differences in the support boundaries. The rock in which the author's observations were made is for the most part sound and unaltered, whereas much of Coon's data is from tunnel projects where the rock is chemically altered. Low RQD values in the author's observations are most frequently the result of very close jointing in sound, unaltered rock. Low RQD values in Coon's data are very frequently the result of rock alteration and weathering along shear zones.

It is very likely that a tunnel in jointed sound, unaltered rock with an RQD of 50 percent will require less support than the same size tunnel in altered rock with an RQD of 50 percent. This conclusion can be reached from considerations of the probable movements and loosening associated with each of the two rock conditions described above. As shown in the model study, it is entirely possible for a jointed

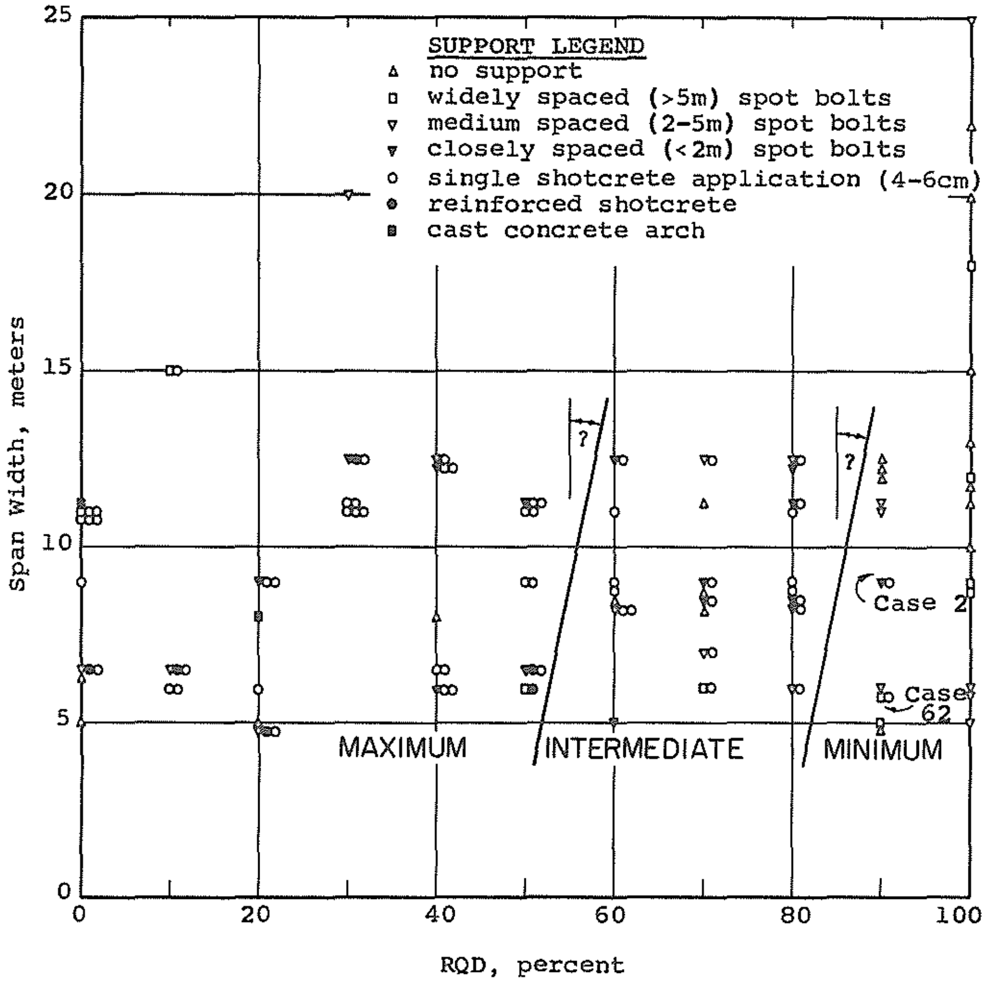


Fig. 5.11 Tunnel support chart based on primary RQD and span width - softening clay cases excluded

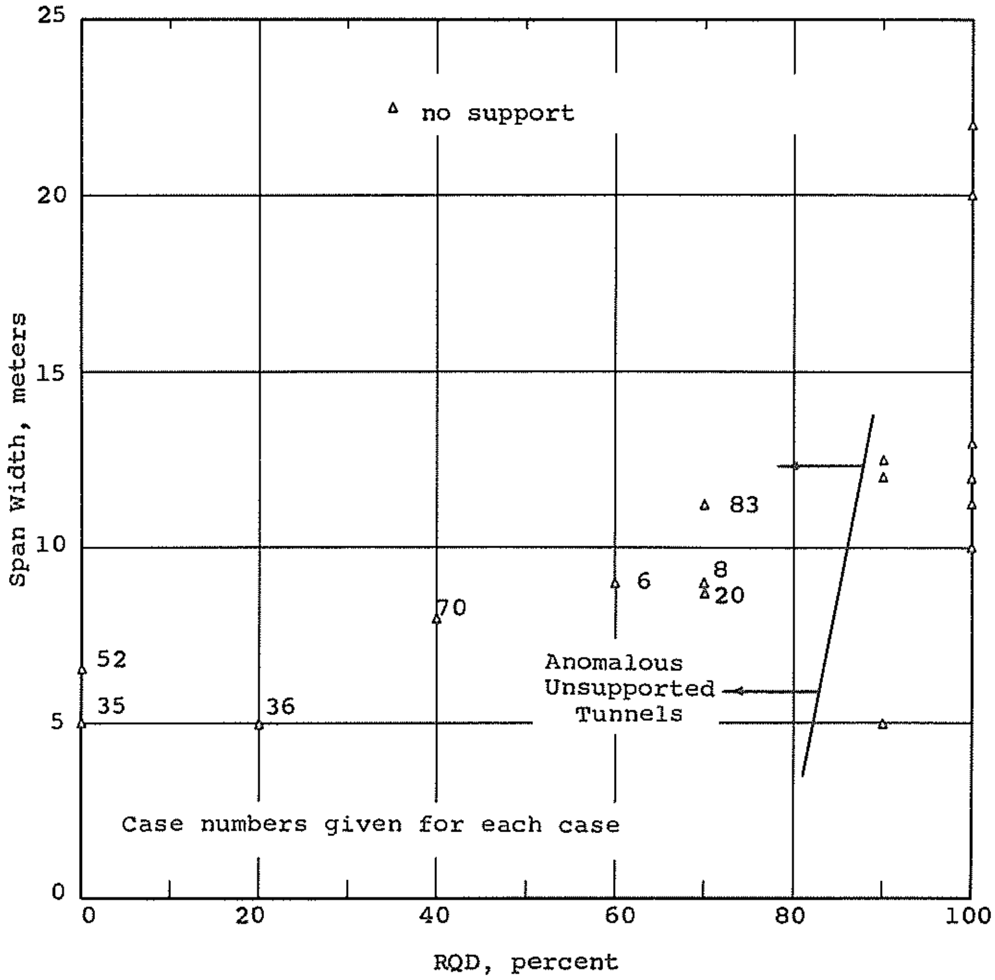


Fig. 5.12 Tunnel support chart based on primary RQD and span width - unsupported tunnels only

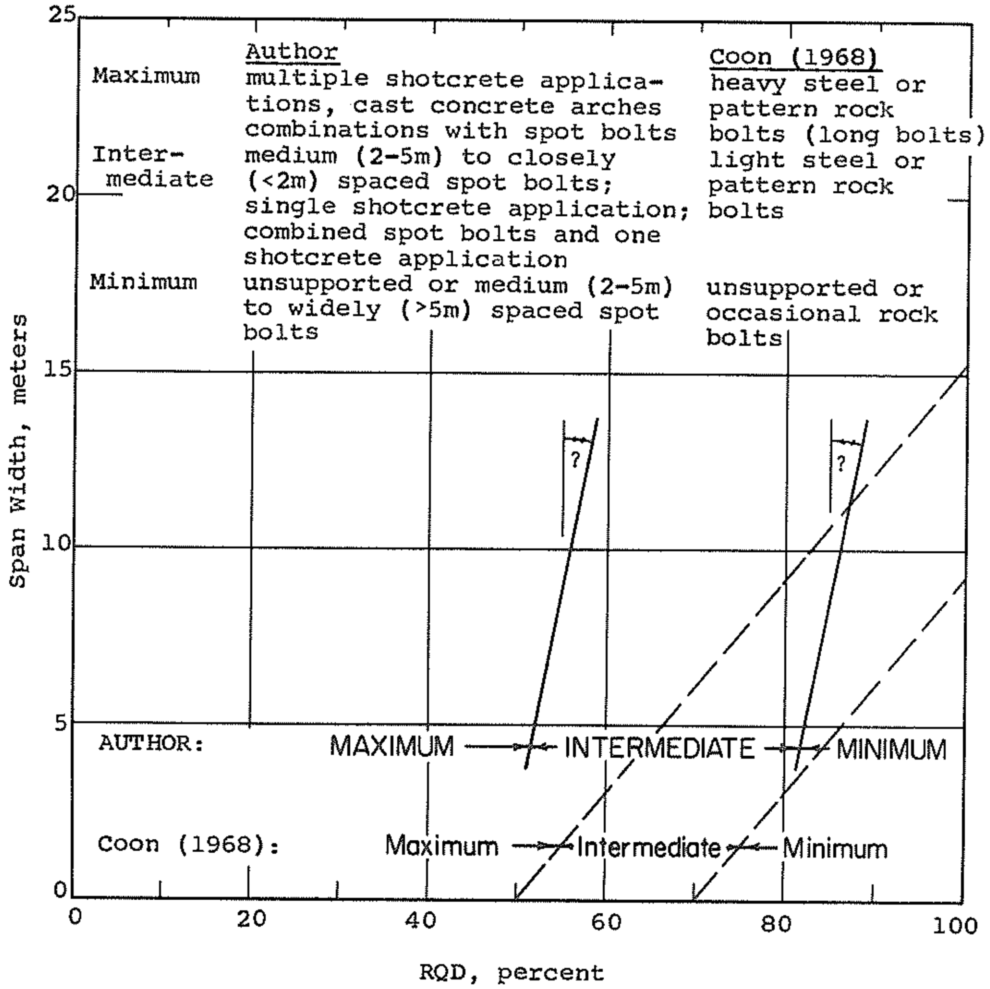


Fig. 5.13 Comparison of Author's and Coon's support chart boundaries

TABLE 5.8

SUMMARY OF STRUCTURAL DATA FOR UNSUPPORTED TUNNELS

Case Number	RQD, percent	Rock Structure Classification	Strike of Major Discontinuities (from tunnel axis)	Dip of Major Discontinuities	Degree of Joint Discontinuity	Tightness of Discontinuity	Discontinuity Filling
6	60	2	0-30°	60-90°	Discontinuous	Tight	None
8	70	2	30-60°	30-60°	Discontinuous	Tight	None
20	70	2	0-30°	60-90°	Continuous	Tight	None
35	0	3	60-90°	60-90°	Discon/Con	Tight	None
36	20	2	60-90°	60-90°	Discon/Con	Tight	None
52	0	2	30-60°	30-60°	Con/Discon	Tight	None
70	40	2	60-90°	60-90°	Continuous	Tight	None
83	70	4	60-90°	0-30° 60-90°	Discon/Con	Tight	None

rock mass to maintain the stresses that occur around an excavated opening, provided they are oriented at favorable angles to the principal direction of loading. Loosening can thus be prevented and the joints may not have any effect on the stability of the opening.

Altered rock, on the other hand, may have a loose structure even before an opening is excavated in the medium. When an opening is excavated in such a medium, the rock is unable to transfer or maintain the redistributed stresses at the periphery of the opening, tangential skin stresses migrate away from the opening and loosening occurs.

The same behavior takes place in rock that is entirely intact, but that contains soft joint fillings, alterations along joints, or open loosely fitting, slickensided discontinuities. Many of the intermediately supported cases with relatively high RQD values exhibit such behavior. Cases 2 and 62, that are intermediately supported and yet have RQD values of 90 percent (see Fig. 5.11), are examples.

A second and equally significant factor that may account for some of the differences in location of support category boundaries in Figure 5.13 is the method of tunnel construction, particularly the method of rock support. The cases that Coon reports were supported mainly with steel sets and conventional support methods, whereas all of the author's reported cases were treated with shotcrete. Many of the latter were supported shortly after blasting, even before mucking out. It is fairly well accepted that, in loosening ground conditions, the minimum rock support required to maintain a stable opening is that which is applied as soon after blasting as possible. Furthermore, most tunneling experts agree that a close fit between the supports and the rock will also minimize the quantity of support required or, at least, minimize the loads that are transferred to any given support system. Shotcrete appears to be superior to steel sets in both of these aspects; that is, it can be installed much more quickly than steel ribs and also provides a much better "fit" to the rock surface, and in this way, more effectively restrains loosening.

As pointed out in the last chapter, blasting is also a construction variable that may affect the degree of support required in a tunnel. Coon does not describe the blasting methods used in his case histories. The blasting rounds used in the writer's case tunnels, although not conventional smooth wall rounds, frequently gave relatively smooth tunnel contours compared to those that might be obtained in ordinary blasting.

It is obvious from an examination of Table 5.5 that the majority of observed cases are anisotropic in their RQD values. Because the practical application of any correlations between rock quality or rock classification and tunnel support would necessarily involve core borings or other unindirectional exploration, it is of interest to know the anisotropy of any such correlations.

Relationship Among Tunnel Support, Span Width and Vertical RQD (RQD_v)

A tunnel support chart based on the RQD along a vertical axis and span width is shown in Figure 5.14. Several observations can be made from this chart:

(a) A considerable greater degree of dispersion exists than in the primary RQD chart (Fig. 5.11).

(b) Boundaries between different support categories are poorly defined, particularly between the maximum and intermediate support categories. However, the range in probable boundary location includes the more well-defined boundary suggested in Figure 5.11.

(c) Considerably more points are found on the RQD_v chart than on the primary RQD chart, particularly at the extreme RQD values of 0 and 100 percent. The reason is that vertical RQD values for some of the cases in which the primary weak zones are vertically oriented were estimated to vary from 0 to 100 percent. The exact value depends on the vertical line in the weak zone along which the RQD is measured or estimated. In such cases where a range in RQD_v values have been estimated, both values have been plotted. The same situation applies for the centerline RQD values that will be discussed subsequently.

Because of the scattering of data in Figure 5.14, the support class boundaries are not always conservative envelopes for the cases. For example, the data in Table 5.9 indicate that 12 percent of the maximum supported cases fall into intermediate and minimum support areas of the chart and 17 percent of the intermediately supported cases fall into the minimum support area.

Relationship Among Tunnel Support, Span Width, and Centerline RQD (RQD_a)

A tunnel support chart based on the RQD along the tunnel axis (RQD_a) and span width is shown in Figure 5.15. As in the vertical RQD chart, there is considerably greater dispersion than in the primary RQD support chart. The greater number of data points on the chart in Figure 5.15 is explained by the same reasons given in the previous paragraphs for the vertical RQD chart.

In spite of the very wide dispersion of data in Figure 5.15 two support boundaries are shown. However, the boundaries are much less definitive than for the primary RQD support chart. The data in Table 5.10 indicate that, although the maximum support cases are well confined to the maximum support category, the intermediately supported cases are distributed throughout the chart. Particularly significant is the 24 percent that occurs in the minimum support area.

Summary of Anomalies

The anomalies associated with the primary RQD-support relationship of Figure 5.11 are common to all of the RQD support charts. They include: all rock conditions that contain softening clay materials, thin clay-coatings and joint fillings in widely spaced joints, and single sets of steeply dipping, closely-spaced tight joints. The stability conditions and rock support associated with all of these geologic features may have absolutely no relationship to the rock quality designation. The first two conditions may lead to instability in rock with a high RQD value ($>75\%$), whereas the third condition is often stable at very low ($<20\%$) RQD values.

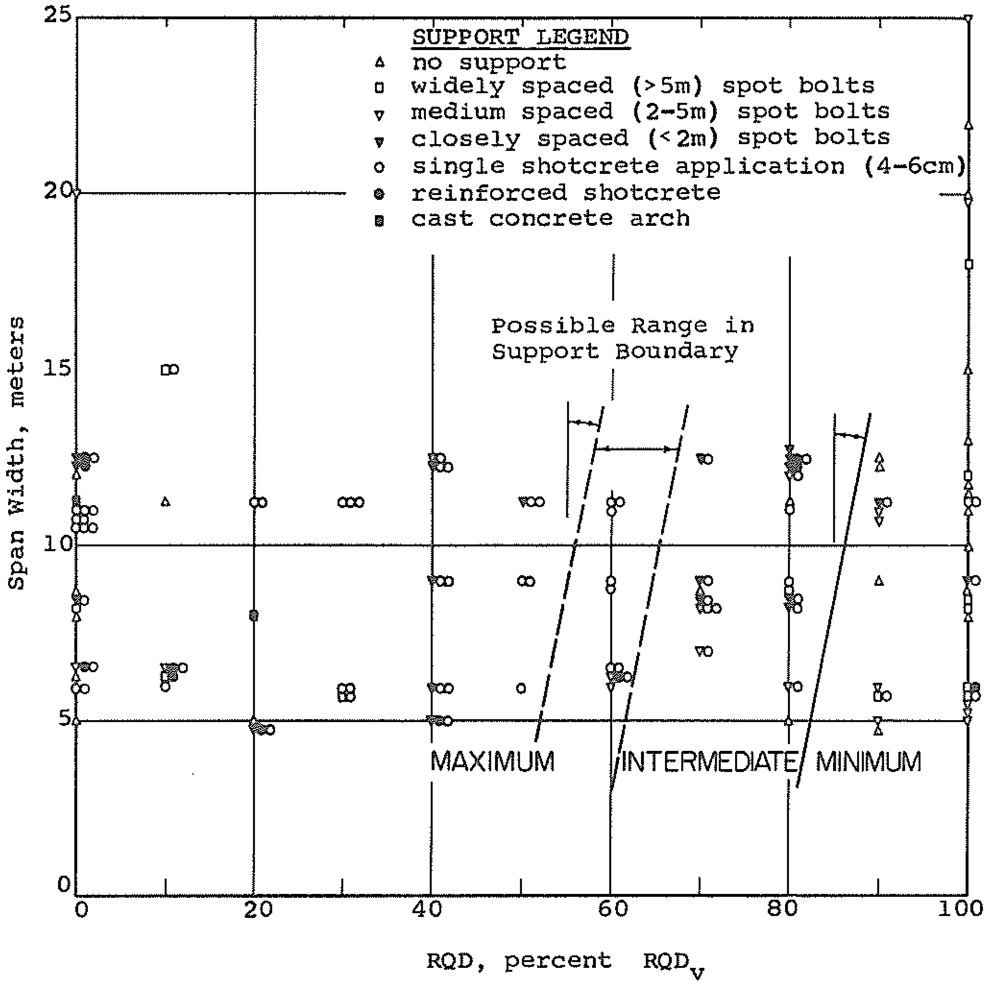


Fig. 5.14 Tunnel support chart based on vertical RQD (RQD_V) and span width - softening clay cases excluded

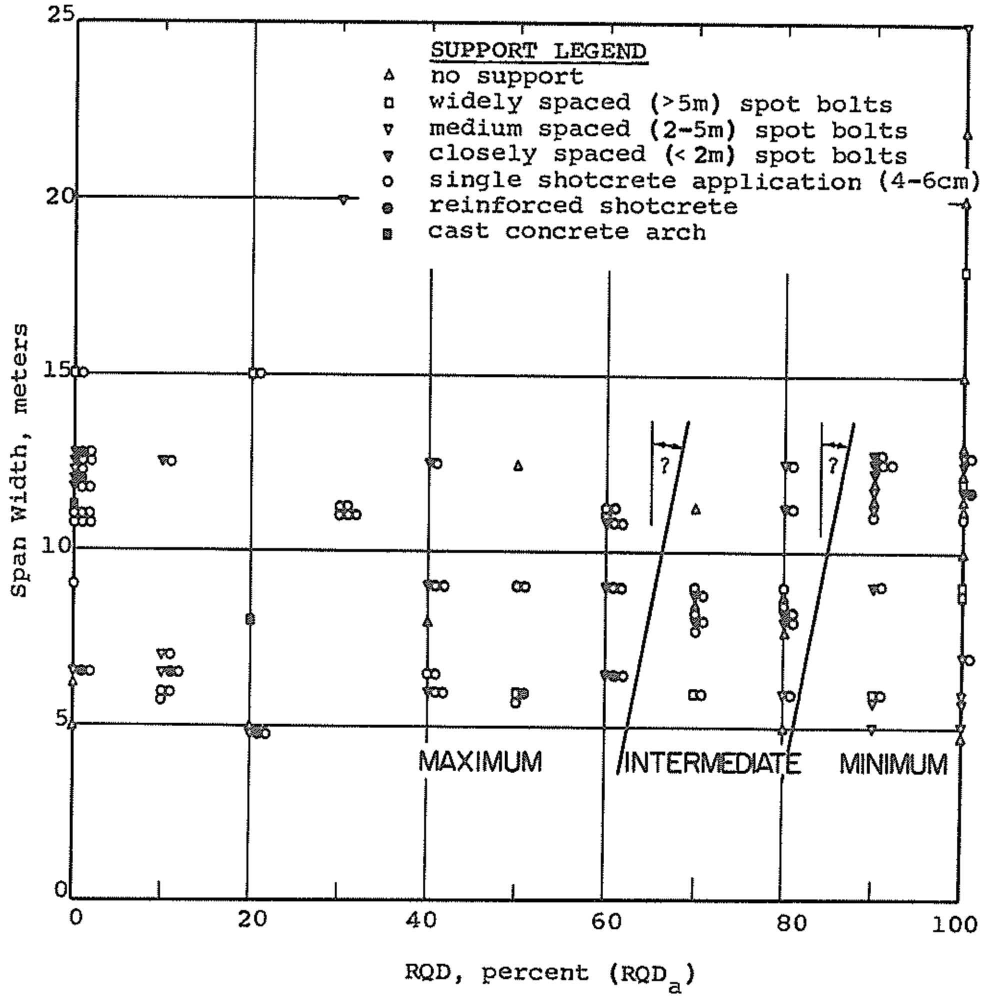


Fig. 5.15 Tunnel support chart based on centerline RQD (RQD$_a$) and span width - softening clay cases excluded

TABLE 5.9

DISTRIBUTION OF CASES IN FIGURE 5.14 ACCORDING TO ACTUAL SUPPORT USED AND SUPPORT CLASSES

Actual Support Used	Classifications on Support Chart		
	Maximum	Intermediate	Minimum
Maximum (25 cases)	22/25 = 88%	2/25 = 8%	1/25 = 4%
Intermediate (29 cases)	12/29 = 41.5%	12/29 = 41.5%	5/29 = 17%
Minimum (39 cases)	8/39 = 21%	4/39 = 10%	27/39 = 69%

TABLE 5.10

DISTRIBUTION OF CASES IN FIGURE 5.15 ACCORDING TO ACTUAL SUPPORT USED

Actual Support Used	Classification on Support Chart		
	Maximum	Intermediate	Minimum
Maximum (22 cases)	21/22 = 96%	0%	1/22 = 4%
Intermediate (33 cases)	12/33 = 36%	13/33 = 40%	8/33 = 24%
Minimum (36 cases)	6/36 = 17%	5/36 = 14%	25/36 = 69%

5.5 Relationships Between Tunnel Support and Seismic Velocity Ratio

General

The available seismic refraction data include the Rätan tunnel measurements and data from surface surveys at the Rätan, Bergvattnet, and Seitevare projects. The seismic velocity ratios that are available for all of the cases are given in Table 5.5. Tunnel seismic velocity data are available for only about seven of the observed cases in Rätan. Unfortunately, most of the tunnel along which the refraction lines were run was shotcreted. However, very complete geologic and construction logs of the entire tunnel were available and it was possible to correlate seismic velocities with both bedrock details and support measured used. This information has been presented in Table A.3.

Seismic velocity ratios based on ground surface data are given in parentheses in Table 5.5. These ratios have been computed by dividing the velocity of the weak zone that apparently correlates with the tunnel weak zone by the velocity of the surrounding rock. Examples are shown in Figures A.1 and A.2.

Relationships Between Tunnel Support and Tunnel Seismic Velocity Ratio

A support chart for the tunnel seismic data at the Rätan project is shown in Figure 5.16. In spite of the limited number of data points, there appear to be three areas in which the support used is characterized by the same groupings of maximum, intermediate, and minimum as exhibited by the RQD charts. The approximate support classification boundaries are as shown in Table 5.11. The maximum support designation is very conservative, as only seven of the 14 cases with a seismic velocity ratio of less than 0.8 were actually treated with maximum support measures. The other seven were supported with intermediate measures.

Merritt's velocity index (Merritt, 1968) is somewhat comparable to the square of the seismic velocity ratio. A rock support distribution chart based on the square of the seismic velocity ratio is shown in Figure 5.17. Three support categories are again apparent, as shown in Table 5.12. However, the scattering of cases with different support measures in the maximum category is similar to that in Figure 5.16, and the squaring of the seismic velocity ratio does not appear to offer any advantages over the first power seismic velocity ratio chart other than a wider numerical band for the intermediate support category.

TABLE 5.11

RELATIONSHIP BETWEEN TUNNEL SUPPORT AND SEISMIC VELOCITY RATIO

Seismic Velocity Ratio	Tunnel Support
> 0.90	minimum
0.8–0.9	intermediate
< 0.8	maximum

TABLE 5.12

RELATIONSHIP BETWEEN TUNNEL SUPPORT AND SQUARE OF SEISMIC VELOCITY RATIO

$(SVR)^2$	Tunnel Support
> 0.8	minimum
0.6–0.8	intermediate
< 0.6	maximum

Anomalous Behavior

Anomalies can be expected to occur in the correlation of seismic velocities and support, just as they do in the RQD correlations. Although no swelling clays were encountered in the Rätan project, it is reasonable to assume that the behavior of such materials would not correlate any better with seismic velocity than it does with RQD or average discontinuity spacing. Similarly, clay and other weak or low friction joint fillings that can cause instability in a rock mass with widely spaced joints may not have any effect on seismic velocity. On the other hand, one or two open joints that may not have any effect on the stability of an opening can drastically lower the seismic velocity and give the impression of low quality rock. The possibility that these conditions may exist must be considered in the interpretation of detailed seismic refraction records.

Relationship Between Tunnel Support and Bedrock Surface Seismic Velocity Ratio

A rock support chart based on span width and ground surface seismic velocity ratios at the Rätan, Seitevare, and Bergvattnet projects is shown in Figure 5.18. The chart also includes several cases from the Höljes project that was studied as a literature case. All the cases, including the softening clay cases, are shown on the chart. The only apparent support boundary is the one shown at 0.80-0.82 that divides the unsupported and supported cases. Chart areas for the different degrees and types of support are not apparent. The lack of such detailed correlations can partly be attributed to the differences in relative qualities of weak rock zones at the surface and at the depth of the tunnel. The relative qualities of different weak rock zones at the bedrock surface, as measured by the refraction velocities, may not be the same as the relative qualities of the same zones in the tunnel. In particular, zones of very closely, vertically jointed rock that frequently are tight and insignificant as far as stability is concerned may have a low bedrock surface velocity that is not much different than that for a very loose weak zone that requires maximum support in a tunnel.

5.6 Relationships Between Different Rock Quality-Tunnel Support Correlations

In the previous three sections it has been shown that there exist crude correlations between three different rock quality indices or classifications and rock support used in the case studies (Figs. 5.5, 5.11, and 5.16). The average joint spacing, primary RQD, and seismic velocity ratio have given the best correlations between support used and single rock quality parameters. In this section the interrelationships of these three measures of rock quality are examined.

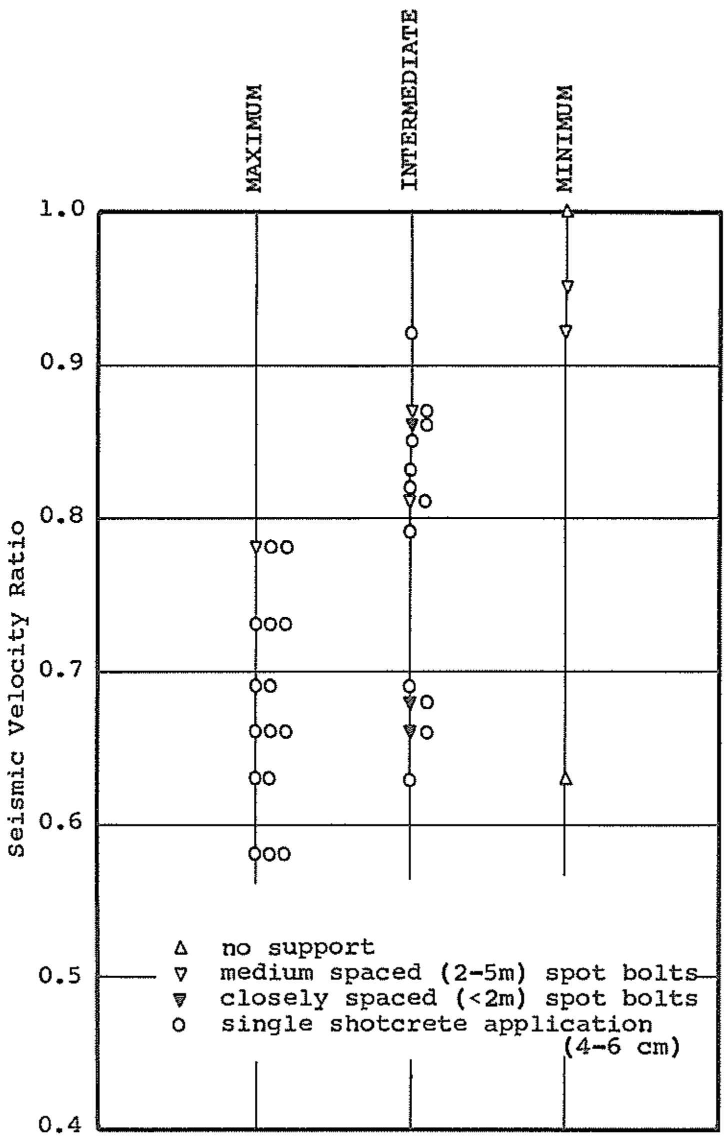


Fig. 5.16 Tunnel support chart based on tunnel seismic velocity ratios at Rätan project (span width = 11.25 m)

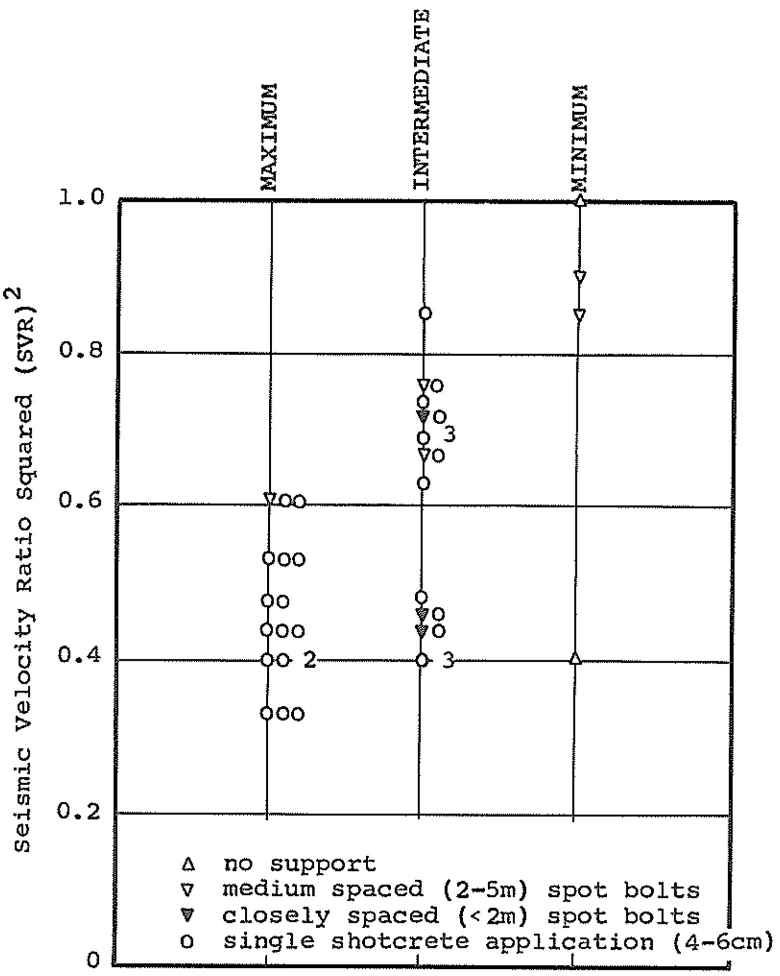


Fig. 5.17 Tunnel support chart based on square of tunnel seismic velocity ratios at Rätan project (span width = 11.25 m)

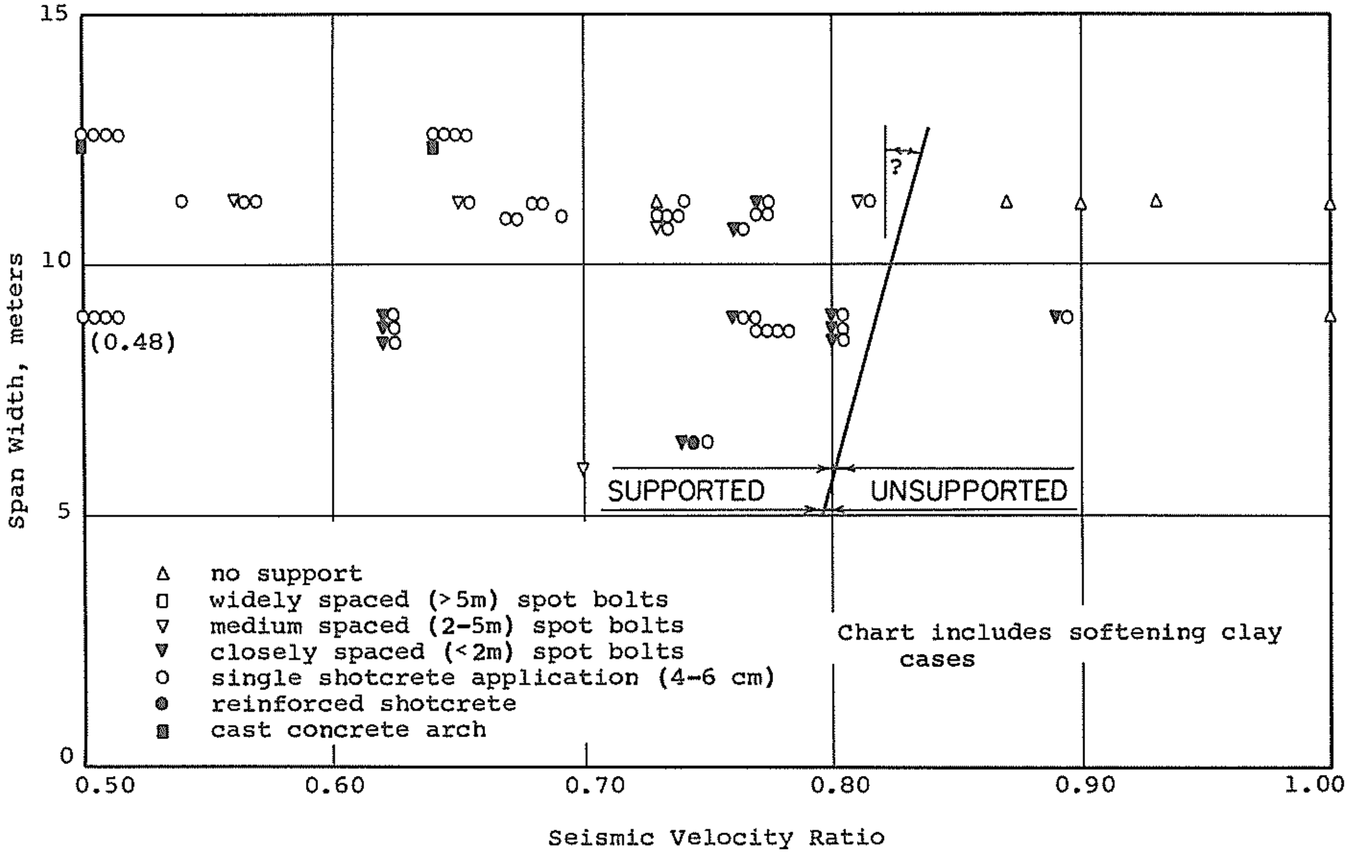


Fig. 5.18 Tunnel support chart based on span width and ground surface seismic velocities at Rätan, Seitevare, and Bergvattnet projects

Softening clay cases are excluded from all of the plots in the following paragraphs. All RQD values used are primary values.

RQD vs Average Discontinuity Spacing

A support chart based on RQD and average discontinuity spacing is shown in Figure 5.19. The following observations can be made from this chart:

(1) All cases having an average discontinuity spacing greater than 30 cm were treated with intermediate or minimum support measures, regardless of their RQD value.

(2) With only one exception in 41, all cases having an RQD greater than 50 were treated with intermediate to minimum support measures, regardless of their average discontinuity spacing.

Because these two conclusions could have been reached independently from Figures 5.5 and 5.11 and because there does not appear to be any unique relationship between RQD and average discontinuity spacing, the chart in Figure 5.19 does not aid in the attempt to correlate support and rock quality. Its only value is for indicating the superiority of a graded measure of rock quality, such as the RQD, to a group or step-type classification of joint spacing. Although an actual average discontinuity or joint spacing could be computed and used as a continuous-type measure of rock quality, the work involved in such a process would be considerably greater than making RQD measurements, and it is doubtful that a correlation based on the former would be any more positive than that based on RQD.

RQD vs Seismic Velocity Ratio

Those few cases at the Rätan project where observations were made of the rock along the tunnel refraction lines are shown in Figure 5.20 on a plot of seismic velocity ratio and primary RQD. Although sufficient data do not exist to allow a perfect relationship to be established, there does exist a pronounced trend towards a linear relationship between seismic velocity ratio and RQD. Very coincidentally, the RQD values of the cases in Figure 5.20 are centerline values; that is, the direction of the RQD determinations is along the tunnel axis in the same direction as the refraction lines. It is to be expected that these two directional properties of the rock mass would not correlate in perpendicular directions.

The same data that appear in Figure 5.20 are shown in Figure 5.21 in a plot of the square of the seismic velocity ratio and the RQD. There is less scatter in this plot and the apparent relationship may be similar to that found by Merritt (1968) and shown on the chart as a solid line. The data do not fit as well the relationship assumed by Deere, Hendron, Patton and Cording (1967).

An attempt has been made to correlate the square of the seismic velocity ratio and average discontinuity spacing, but the poor distribution of data points over the range of average discontinuity spacings did not show any trends.

It may be concluded that RQD and seismic velocity are the only two measures of rock quality that correlate well both with support and with each other.

5.7 Application of Rock Quality-Tunnel Support Relationships

Summary of Correlations

The results from the previous sections in this chapter indicate that reasonable and potentially useful correlations exist between average discontinuity spacing, RQD, seismic velocity ratio, and support used in tunnels. The relationships for all three rock quality indices are summarized in Table 5.13. The guidelines given in this table apply to span widths of less than about 13 meters.

The numbers given in parentheses in Table 5.13 indicate the percent of cases in the different support classes in which more or less support was used than indicated on the chart. For example, in the primary RQD correlation 25 percent of the cases that lie in the area designated intermediate support on Figure 5.11 actually were treated with minimum support,

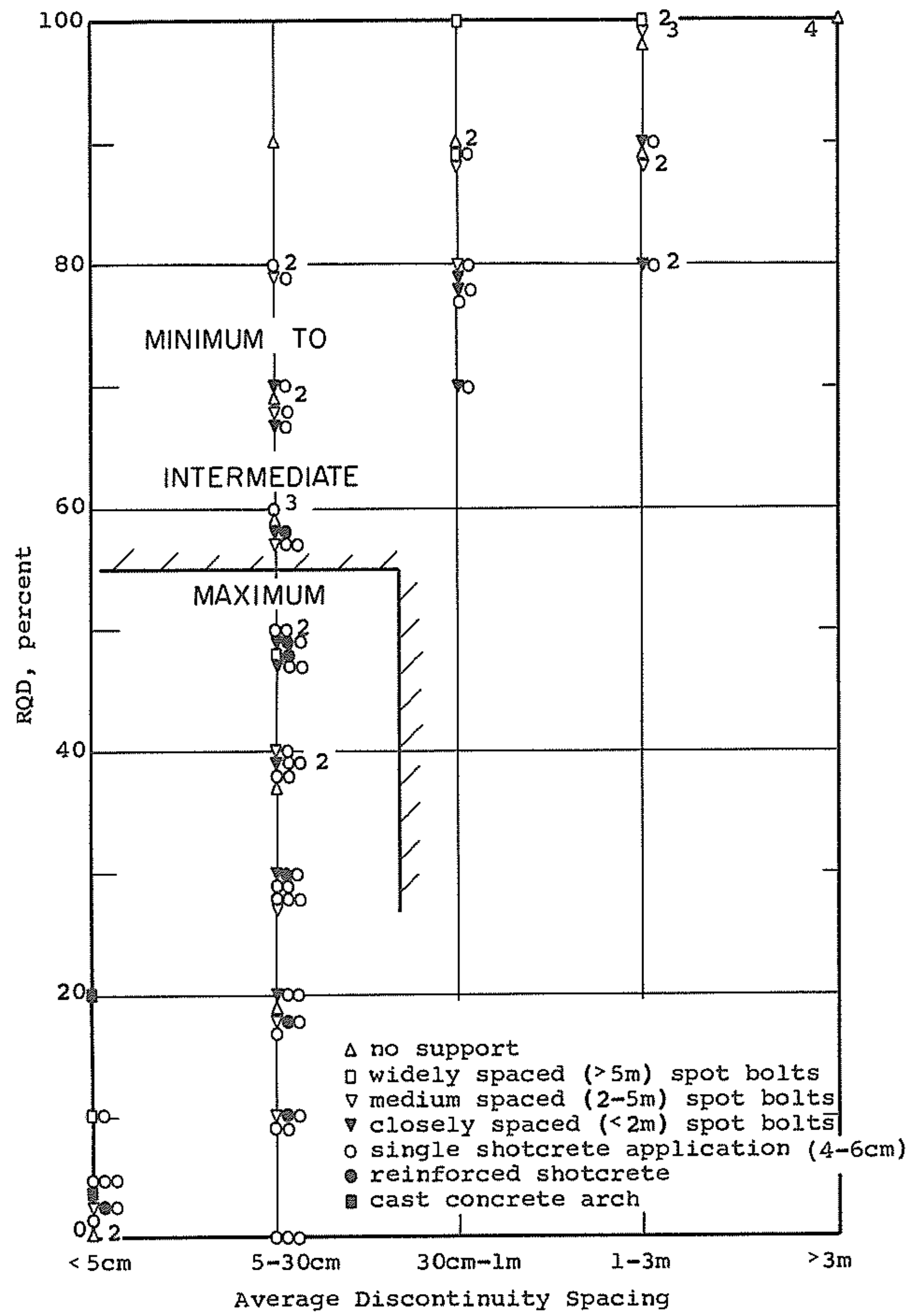


Fig. 5.19 Tunnel support chart based on RQD and average discontinuity spacing

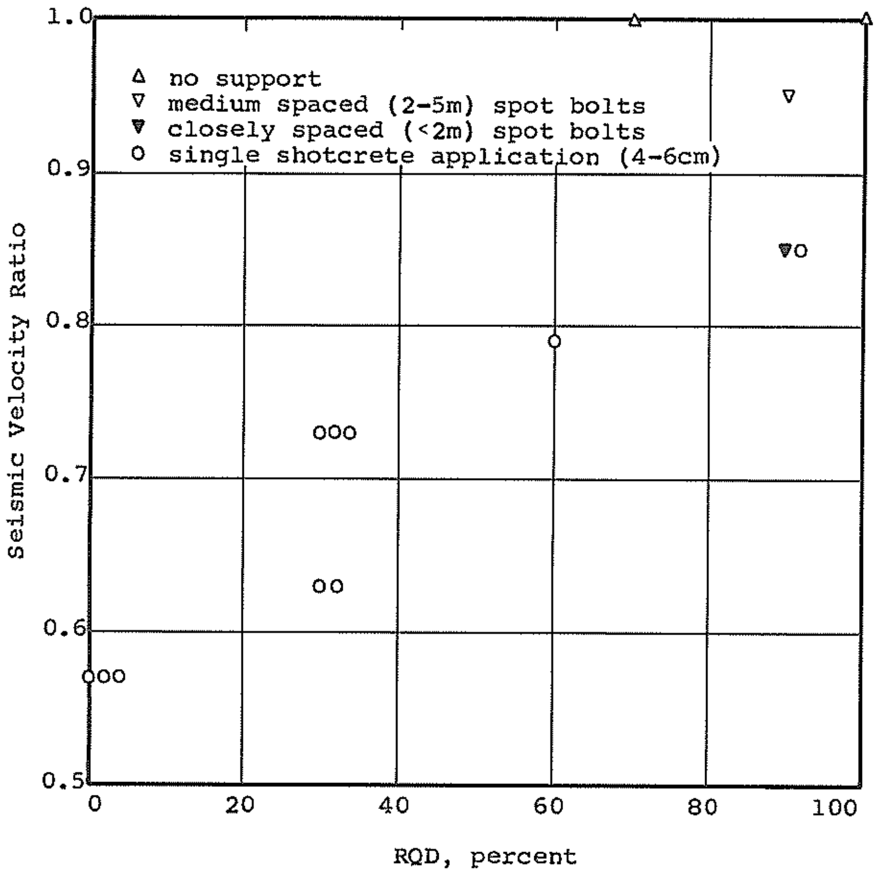


Fig. 5.20 Tunnel support chart based on tunnel seismic velocity ratio and RQD

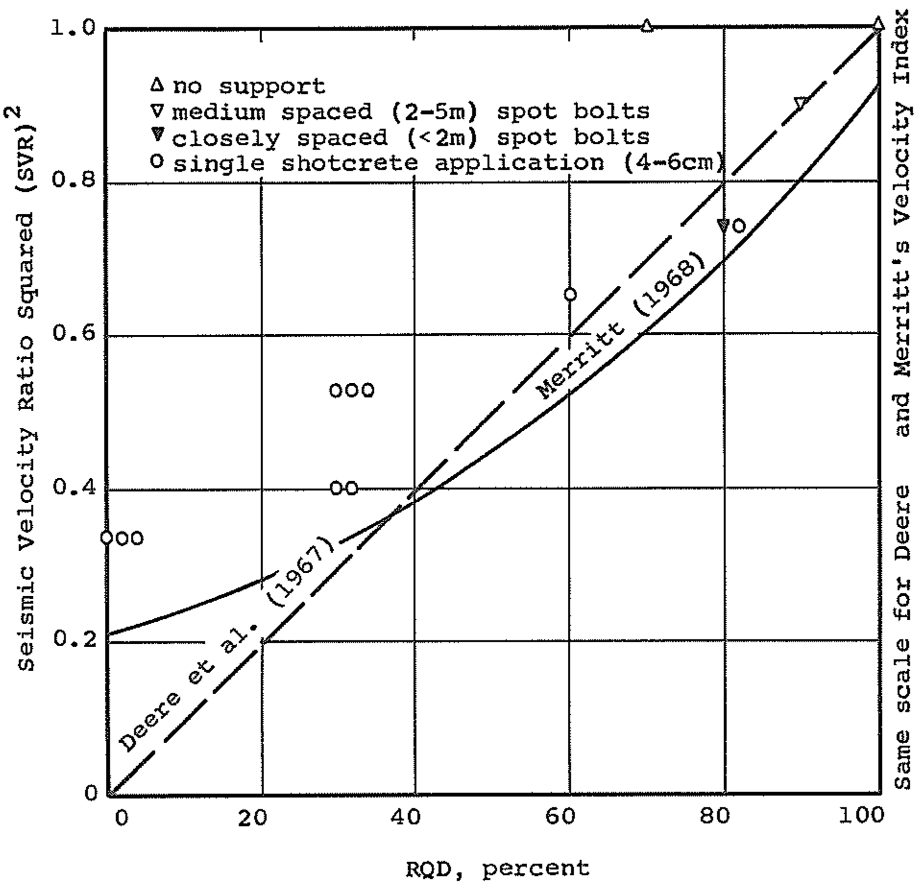


Fig. 5.21 Tunnel support chart based on square of seismic velocity ratio and RQD

and 4 percent with maximum support. The percentages in parentheses thus give an indication of the reliability of the different measures of rock quality in making a prediction of rock support for a tunnel. It can be concluded from Table 5.13 that the correlation based on primary RQD gives the most positive indication of support that is likely to be used.

It should be emphasized again that the support shown in all of the charts is that which has been used and may not correspond to an actual support requirement, a factor that is difficult, if not impossible, to evaluate even after a tunnel has been excavated. The charts are a collection of experience.

Application to Predictions of Rock Support Requirements

The results summarized in the preceding paragraphs have been derived from observations in tunnels. Such observations are obviously necessary in the derivation of any empirical relationship. However, for the relationships to be of most value in actual practice, it is necessary that the rock quality parameters be evaluated prior to excavation by inexpensive and reliable means.

Average discontinuity spacing and RQD are measures of rock quality that can be evaluated through diamond core drilling and, hence, have a potential use in evaluating support needs in an underground excavation. Although diamond drilling is not an inexpensive method of exploration and is not always reliable, it is presently the most commonly used exploration tool.

Merritt (1968), Coon (1968), and others have used the RQD in NX core borings to correlate rock quality and engineering properties of rock masses. The diameter of the core obviously influences the apparent RQD that is determined from a core. Deere (1968) recommends the use of NX size (2-1/8-in. diameter) and larger cores. Smaller size cores (1 1/4-in. and 1-5/8-in. diameter) are generally used in exploratory drilling in Sweden. Some modification of the RQD, such as that used by Heuzé (1970), should be used for these smaller size cores.

Because the location and orientation of weak bedrock zones generally are not known prior to exploration, it would not be possible to apply the primary RQD support relationship of Figure 5.11. However, the vertical and centerline RQD relationships of Figures 5.14 and 5.15 are potentially useful guidelines for estimations of support, as they correspond to the two most likely directions of exploratory drilling. The anomalies and uncertainties associated with the correlations must be accepted as inherent weaknesses, but are tolerable in view of the lack of any other such correlation.

The average discontinuity spacing-support correlation suffers the same weakness as the primary RQD relationship in that a diamond core may not necessarily yield the information necessary for its determination. However, if borings are oriented at angles of greater than 45 degrees to the direction of major discontinuity sets, it should be possible to determine their average spacing and, hence, apply the conclusions drawn from Figure 5.5.

The determination of bedrock seismic velocities by ground surface refraction measurements is a common preliminary investigation for many underground projects in Sweden. The support chart presented in Figure 5.18 should serve as a rough guideline for the determination of likely bedrock weak zones that will require support. However, to use the more detailed information derived from Figure 5.16 it would be necessary to determine seismic velocities at the location where the tunnel is to be driven. At the present time such determinations are possible from the ground surface only by such expensive methods as in-hole and cross-hole surveys. These methods cannot be considered routine preliminary investigations for long sections of running tunnels.

The greatest possibility that exists for the application of the data from Figure 5.16 to preliminary rock support estimations lies in the development of better and cheaper seismic methods. Merritt (1968) has suggested that an in-hole geophone that can be lowered to any depth in a shallow boring and wedged against the rock would be a suitable piece of seismic equipment for civil engineering projects. Such equipment might be suitable for holes drilled ahead of a

TABLE 5.13

SUMMARY OF ROCK QUALITY - TUNNEL SUPPORT CORRELATIONS

Measure of Rock Quality	Boundaries for Indicated Support		
	MAXIMUM	INTERMEDIATE	MINIMUM
Average discontinuity spacing (Fig. 5.7)	< 30 cm	30 cm - 1 m	> 1 m
	(22%)[1]	(17%)	(--)
	(--)	(--)	(22%)[2]
RQD, primary (Fig. 5.11)	<60 %	60 - 80 %	> 80 %
	(35%)	(25%)	(--)
	(--)	(4%)	(8%)
RQD, vertical (Fig. 5.14)	<70 %	70 - 80 %	>80 %
	(48%)	(22%)	(--)
	(--)	(11%)	(18%)
RQD, centerline (Fig. 5.15)	<70 %	70 - 80 %	> 80 %
	(46%)	(28%)	(--)
	(--)	(0%)	(27%)
Seismic Velocity Ratio (Fig. 5.16)	<0.8	0.8 - 0.9	> 0.9
	(53%)	(0%)	(--)
	(--)	(0%)	(33%)

1) Percent of cases in which less support was used.

2) Percent of cases in which more support was used.

working face as well as from the ground surface.

Need for Trial Testing

Swedish Geologic Environment. Trial testing of the rock quality-support correlations is necessary before applications can be made with any confidence. Such testing would serve not only to check the applicability of the relationships derived from the tunnel observations to preliminary investigation material, but also to determine the economic feasibility of using the methods. Because of extreme variations in rock conditions over short distances, it may not be feasible to use vertical drill holes to predict conditions along a horizontal tunnel, particularly if the main weak zones are vertical. Thus, borings inclined or parallel to the tunnel are to be preferred in areas where conditions are expected to change quickly.

It may be expected that RQD values determined from a tunnel wall may not correspond to those determined from a drill core. Partially healed joints and poorly defined schistosity that may open up during blasting, but not in drilling, can account for differences. The relationships derived from the tunnel observations thus may not be valid for RQD values or average discontinuity spacings determined from core borings. However, the possibility of such relationships existing is very strong, and it remains to correlate boring data with tunneling experience to see if the relationship is different.

The seismic velocity ratio support chart in Figure 5.16 is based on a very limited amount of data.

Before efforts are actually made to apply the results of this work, additional testing is necessary to support or disprove the trends indicated. Additional measurements in tunnels are warranted where there is the possibility of determining seismic velocities in a preliminary investigation program.

Other Geologic Environments. All of the preceeding comments have concerned the geologic environment in which all of the field observations were made. The question as to the applicability of the results of this chapter to different geologic environments is one that can be answered only through observations in tunnels and trial application in other geologic conditions. However, for reasons discussed earlier it can be expected that rock quality-support correlations will be different for different geologic environments. The degree of rock alteration and weathering is likely to change the relationships significantly. It is suggested that some measure or index of alteration be established to supplement the RQD in conditions where alteration is frequent.

The magnitude and direction of the natural ground stresses are two factors that may also have a significant effect on rock quality-support correlations. Thus, if correlations in different geologic environments are to be attempted, an attempt should be made to gather information (both qualitative and quantitative) about the in-situ rock stresses.

It is doubtful that ground surface seismic refraction measurements would be useful for evaluating rock quality in conditions that differ very much from those in which the writer made his observations. The requirements for the successful, clear determination of narrow (<20 m) bedrock weak zones from ground surface refraction surveys are a distrinct, abrupt change in material properties at the soil-bedrock interface and a thin overburden cover, preferably less than 50 meters. Glaciated terrain whose soil cover is thin are ideally suited.

It is quite possible that even the seismic velocity ratio-support relationships derived from the Swedish tunnel measurements would not be applicable to other geologic conditions. The principle of the seismic velocity ratio-support correlation, however, may prove to be applicable to a wider variety of geologic conditions, but again, trial testing is necessary.

6. CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

6.1 Conclusions

On the basis of the laboratory model studies and field observations described in the previous chapter, the conclusions listed below can be drawn.

Laboratory Model Studies

1. The loosening failure mechanism in a joint block mass is governed by the interaction of individual block rotation and slip movements. The mode of behavior of groups of blocks depends strongly on the joint block configuration and orientation. Block rows may interact and behave as multi-hinged arches. Groups of blocks bounded by low angle discontinuities may act as one rigid block.

2. Loosening failures may result from pure slip along discontinuity boundaries or from block corner failures. Block corners may fail by tearing, shearing, or crushing. The model studies only point out the possibility of such failures occurring. The significance of such failures in real tunnels in loosening ground depends on the manner in which loosening occurs. Corner failures are not expected to be of large significance unless large volumes of rock are involved in the loosened zone or ground arch that forms above the opening.

3. Unsupported jointed block masses may remain stable under the action of high compressive stresses that effectively confine the discontinuous mass and that prevent blocks from slipping, rotating, and subsequent loosening. Even if the confining stresses are relatively low, block rotations and slippage may lead to a stable, arched zone of loosened blocks above the unsupported span. These two mechanisms of stabilization of a jointed mass are possible explanations for the field observations of stable tunnels in heavily

jointed rock. The likelihood that large horizontal stresses exist in much of the Swedish bedrock suggests that the latter mechanism is the most probable explanation of the observed field behavior.

4. Joint friction and discontinuity orientation may drastically affect both the mode of failure in a loosening block mass and the stress level at which failure occurs. It is quite obvious that very minor geologic details such as single random discontinuities oriented at a low angle (30^{o}) to the perimeter of the opening and single clay-coated joints can cause failure in an otherwise stable mass.

Field Studies

1. Field observations in Swedish and Norwegian tunnels have enabled reasonable and potentially useful correlations to be drawn between three rock quality parameters (average discontinuity spacing, RQD, and seismic velocity ratio) and the shotcrete and rock bolt supports used in large (60 sq. meters) tunnels. The correlations are summarized in Table 6.1. All of these relationships are generally very conservative envelopes that bracket the experience collected from the field observations.

2. Anomalies in the relationships given in Table 6.1 exist for rock that contains softening clay material and for rock that contains a single set of close, tight, clean fractures. The former may cause difficulty regardless of the average discontinuity spacing, and the latter may not be associated with any need for support, even when the average discontinuity spacing is less than 10 cm, or four inches.

3. The relationships derived by the author for shotcreted and rock bolted tunnels are significantly different from similar relationships found by Coon for American tunnels supported by steel sets. Differences in the relationships are attributed to differences in geologic conditions, support methods, and other construction techniques such as blasting methods. It is also very likely that seemingly insignificant construction factors, such as mucking time, labor relations, and contracting practices may have an indirect effect on the stability of tunnels.

4. Although attempts to derive relationships between rock mass structure and support were not successful, they do show some interesting trends in rock mass stability behavior. Any rock structure containing three dimensional joint blocks (structural units) requires support of some kind. Many one dimensional structures, regardless of their RQD or average discontinuity spacings do not require support, even in large tunnels.

5. In consideration of the large number of factors that influence the stability of an opening in jointed rock, it is amazing that correlations as good as those in Chapter 5 even exist.

TABLE 6.1

SUMMARY OF RELATIONSHIPS BETWEEN TUNNEL SUPPORT AND ROCK QUALITY PARAMETERS

Rock Quality Parameter	Limit for Support Boundary		
	Maximum	Intermediate	Minimum
Average Discontinuity Spacing	< 30 cm	30 cm - 1 m	> 1 m
RQD	< 60%	60 - 80%	> 80%
Seismic Velocity Ratio	< 0.8	0.8 - 0.9	> 0.9

6.2 Recommendations for Future Work

Mechanism of Loosening Instability

The actual mechanism of loosening instability in tunnels is adequately understood on a qualitative basis. The results from the tests described in this investigation have indicated the significance of certain rock mass parameters, but they have not given quantitative information about the effect of these parameters on stability. It would be desirable to make such quantitative studies so that the relative importance of different parameters could be studied and so that real tunnel conditions could be simulated. However, because a large centrifugally loaded model would be necessary to obtain quantitatively meaningful results for the loosening-type instability problem, it is felt that expected results from good laboratory model studies are not warranted by the expense.

Goodman's (1968) finite element study offers a more attractive means of studying loosening instability. Parameter studies that involve variations in input data for intact rock and rock mass properties, opening geometry, and free field stresses could probably be done at a much lower cost than reliable laboratory model studies. Furthermore, computer studies should enable a much more comprehensive parameter study to be made. Although the absolute magnitudes of the results (factors of safety against fallout, for example) may not be absolutely correct, variations in data input should enable conclusions to be reached about the relative quantitative effect of various properties and rock mass parameters on loosening instability. In particular, it would be very valuable to understand the effect of variations in intact rock properties on loosening stability, provided the mechanism of block rotations is built into the computer model, as Goodman has done. All of the possible modes of block corner failure should be capable of occurring in the computer model.

Field Studies

1. Before any of the derived support-rock quality correlations can be applied with confidence, it is necessary that they be checked with the information obtained from preliminary investigations. The RQD and average discontinuity spacing relationships should be checked from the information derived from core borings and actual tunneling conditions.

2. The seismic velocity ratio relationship derived from the Rätan tunnel measurements is not yet substantiated by sufficient data to warrant consideration for application to preliminary investigation material. Additional tunnel refraction measurements and comparisons with supports are needed in order to establish the possibility of using this property as an indicator of necessary tunnel support. However, it is doubtful that relationships between seismic velocity and tunnel support can be derived that are better than those established on the basis of the RQD. The continuation of the writer's tunnel seismic refraction work is not justified unless some new, quick and inexpensive method can be devised for measuring seismic velocities in the rock through which the tunnel is to be driven. The most promising possibility is the use of a simple velocity or other geophysical probe in a rapidly drilled hole in the tunnel face or inclined from the ground surface.

3. For the ordinary seismic refraction measurements that are carried out on the ground surface, the relationship shown in Figure 5.18 should be useful for a rough indication of the behavior of the rock in a tunnel. It is suggested that additional data be added to the chart when they are available. The limiting seismic velocity ratio of 0.8 that has been found as an approximate dividing line between supported and unsupported rock may well prove to be different for different geologic conditions.

4. There is a definite need to search for relationships between rock quality and tunnel support requirements in other geologic environments. Studies similar to the field work done by the writer should be carried out for other tunnels in different geologic environments. This type of research is very inexpensive and, although the results are of limited value until they can be applied to predicting tunneling conditions from preliminary investigation material, the studies are necessary if tunneling experience is to be utilized to the fullest extent.

5. When attempts are made to derive rock quality-

support relationships in other geologic environments, some means should be devised to account for the degree of weathering and alteration of the rock. Because altered rock may have a more detrimental effect on tunnel stability than sound, heavily jointed rock, there must be some method to distinguish between the two when making numerical estimations of rock quality.

6. The support charts shown in Chapter 5 relate the rock quality to the supports used in different tunnels and in no way give any indication of the factor of safety against failure. It would be very valuable to collect and plot support and rock quality data for cases in which some quantitative information about loads in the support is available. This is simple enough to do for steel sets where total loads in the support can be measured. However, for rock bolted and shotcreted tunnels there are no simple measurements that can be made. One possibility is to collect data for some arbitrary measure of load at some arbitrary point in the support. These data, together with rock quality data and an indication of the opening geometry (width, height), might give more meaningful correlations than those presented in Chapter 5. Furthermore, there is a definite need for data in which shotcrete applications have failed.

7. The simple analyses for the rock-shotcrete interaction that have been made in Chapter 4 are based on assumptions of uncertain validity. Specifically, the role of the rock-shotcrete bond is not well understood. Field and laboratory measurements of shotcrete bond strength are needed to gain an insight into the possible support that can be derived from a mechanism such as that described in Figure 4.12a. Field measurements and observations in shotcreted tunnels should be directed towards an understanding of the load-transfer mechanism or interaction between shotcrete and rock. Very detailed measurements of strains in shotcrete linings may help to clarify the interaction, although it is likely that the stress variations and gradients in shotcrete linings, particularly at re-entrant angles and near the rock surface, are very erratic. Finally, field observations, together with the results from laboratory and field tests of the bond strength between rock and shotcrete, could be combined with a theoretical analysis of the interaction that would possibly give a more accurate picture than the simple models used heretofore.

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REFERENCES

ABEL, J.F., 1967. "Tunnel Mechanics", Quarterly, Colorade School of Mines, V. 62, No. 2.

ALBERTS, C., 1965. "Bergförstärkning genom betongsprutning och injektering", Bergmekanik, Ingeniörsvetenskapsakademiens meddelande 142, Stockholm, pp. 231-240.

BERGMAN, S.G.A., 1965. "Funktionell Bergklassificering", Bergmekanik, Ingeniörsvetenskapsakademiens meddelande 142, Stockholm, pp. 115-123.

BJURSTRÖM, S., 1966-67. Personal communication, Stockholm.

BRÄNNFORS, S., 1964. Bergsprängningsteknik, Svenska Bokförlaget/Bonniers, Stockholm.

CARMICHAEL, A., 1970. Personal communication, Urbana.

CECIL, O.S., 1970. "Shotcrete Support in Rock Tunnels in Scandinavia", Civil Engineering, January, pp. 74-79.

COATES, D.F., 1964. "Classification of Rocks for Rock Mechanics", International Journal of Rock Mechanics and Mining Sciences, Vol. 1, pp. 421-429.

COON, R.F., 1968. "Correlation of Engineering Behavior with the Classification of In-Situ Rock", Ph.D. Thesis, Department of Geology, University of Illinois.

CORDING, E.J., 1968a. "The Stability During Construction of Three Large Underground Openings in Rock", Ph.D. Thesis, Department of Civil Engineering, University of Illinois, Urbana. Published as Technical Report No. 1-813 by U.S. Army Engineer Waterways Experiment Station, Vicksburg.

CORDING, E.J., 1968b. "Stability of Large Underground Openings at the Nevada Test Site", preprint of paper presented at Second Space-Age Conference of the American Society of Civil Engineers, Los Angeles, 25, April.

DEERE, D.U., 1963. "Technical Description of Rock Cores for Engineering Purposes", Rock Mechanics and Engineering Geology, Vol. I/1, pp. 16-22.

DEERE, D.U., HENDRON, A.J., PATTON, F.D., and E.J. CORDING, 1967. "Design of Surface and Near-Surface Construction in Rock", Proceedings of the Eight Symposium on Rock Mechanics, The American Institute of Mining, Metallurgical and Petroleum Engineers, Inc., New York, pp. 237-302.

DEERE, D.U., 1968. "Geological Considerations", Chapter One in Rock Mechanics in Engineering Practice, edited by K.G. Stagg and O.C. Zienkiewicz, John Wiley and Sons, New York, pp. 1-20.

DEERE, D.U., PECK, R.B., MONSEES, J.E., and B. SCHMIDT, 1969. "Design of Tunnel Liners and Support Systems", Final Report to Office of High Speed Ground Transportation, U.S. Department of Transportation, Washington, D.C.

DENKHAUS, H.G., 1964. "Critical Review of Strata Movement Theories and Their Application to Practical Problems", Journal of the South African Institute of Mining and Metallurgy, Vol. 64, pp. 310-332.

GOODMAN, R.E., TAYLOR, R.L., and T.L. BREKKE, 1968. "A Model for the Mechanics of Jointed Rock", Journal of the Soil Mechanics and Foundation Divison, American Society of Civil Engineers, SM 3, Paper No. 5937.

HAGERMAN, T.H., 1954. "Om Svenska Bergarter och Deras Provning För Konstruktionsändamål", Statens Provningsanstalt, Meddelande 85, Stockholm.

HAGERMAN, T.H., 1966. "Rock Bodies and Particular Zones in Rock, The Geological Structure as a Factor in Rock Stability", Proceedings of the First Congress of the International Society of Rock Mechanics, Lisbon, Vol. 1, pp. 159-161.

HANSÁGI, I., 1965. "Numerical Determination of Mechanical Properties of Rock and of Rock Masses", International Journal of Rock Mechanics and Mining Sciences, Vol. 2, pp. 219-223.

HANSÁGI, I., 1967. "Berghållfasthetsbestämning Genom Diamantborrning", Ingeniörsvetenskapsakademien Bergmekanikkommitten Diskussionsmöte, 3 March, Stockholm.

HASSELSTRÖM, B., 1951. "Seismisk refraktionsmätning - en rekognosceringsmetod för bestämning av jorddjup," Teknisk Ukeblad, Nr. 27, Oslo.

HASSELSTRÖM, B., TAHM, L. & K.A. SCHERMAN, 1964. "Methods for the Determination of the Physical and Mechanical Properties of Rock," Eighth International Congress on Large Dams, Question 28, Edinburg.

HAST, N., 1958. "The Measurement of Rock Pressure in Mines," Sveriges Geologiska Undersökning Årsbok 52, Nr. 3, Stockholm.

HAST, N., 1965. "Spänningstillståndet i den fasta jordskorpans övre del," Bergmekanik, Ingeniörsvetenskapsakademiens meddelande 142, Stockholm, pp. 13-23.

HEUER, R.E., 1968. "Geomechanical Model Study of the Behavior of Underground Openings in Rock Subjected to Static Loads," Interim Report, Department of Civil Engineering, University of Illinois, Urbana.

HEUZÉ, F.E., 1970. "The Design of 'Room and Pilar' Structures in Competent Jointed Rock. Example: The Crestmore Mine, California," Doctor of Engineering Dissertation, Graduate Division, University of California, Berkeley.

HILTSCHER, R., 1967. "Mätning av Bergtryck i Tunneltak," Ingeniörsvetenskapsakademien Bergmekanikkommittén Diskussionsmöte, 3 March, Stockholm, pp. 61-75.

HILTSCHER, R., 1968. "Borrhålsbotten-Friborrningsmetoden," Ingeniörsvetenskapsakademien Bergmekanikkommittén Diskussionsmöte, 9 February, Stockholm, pp. 67-75.

HUSEBY, F.C.A., 1966. "Lieråsen Tunnel, Strukturgeologiske og Ingeniørgeologiske Undersøkelser," Tekniske Tidsskrift for Norges Statsbaner, Nr. 3, Oslo.

HÅLAND, G., 1967. "Inndrift og Sikringstiltak i Lieråstunnelen," Teknisk Tidsskrift for Norges Statsbaner, Nr. 1, May, Oslo, pp. 1-12.

KARLSSON, L. & J.O. FRYK, 1961. "Reinforcement Work in Tailrace Tunnel for Höljes Hydroelectric Power Station," Väg- och Vattenbyggaren, Nr. 4, Stockholm.

LANG, T.A., 1957. "Rock Behavior and Rock Bolt Support in Large Excavations," Symposium on Underground Power Stations, American Society of Civil Engineering, October Convention, New York.

LANG, T.A., 1959. "Underground Experience in the Snowy Mountains - Australia," Proceedings of the Second Protective Construction Symposium (Deep Underground Construction), RAND Corporation, Santa Monica.

LANG, T.A., 1961. "Theory and Practice of Rock Bolting," American Institute of Mining, Metallurgical and Petroleum Engineers, Inc., Mining Transactions, Vol. 220.

LANG, T.A., 1964. "Rock Mechanics Considerations in Design and Construction," Proceedings of the Sixth Symposium on Rock Mechanics, University of Missouri, Rolla.

LANGEFORS, U., 1965. "Berghållfasthetens beroende av Sprängningen," Bergmekanik, Ingeniörsvetenskapsadademien meddelande 142, Stockholm, pp. 210-217.

LANGEFORS, U. & B. KIHLSTRÖM, 1963. The Modern Technique of Rock Blasting, Almqvist and Wiksell, Stockholm.

LAUFFER, H., 1958. "Gebirgsklassifizierung für den Stollenbau," Geologie und Bauwessen, 24, H.1.

LINDER, R., 1963. "Spritzbeton im Felshohlraumbau," Die Bauteknik, 10/1963.

LUNDEGÅRDH, P.H., 1963. "Projektering av Rum och Tunnlar i Berg," Sveriges Geologiska Undersökning, Serie C, Nr. 590, Stockholm.

LUNDEGÅRDH, P.H., LUNDQVIST, J., and M. LINDSTRÖM, 1964. Berg och Jord i Sverige, Almqvist and Wiksell, Stockholm.

MATTHIAS, ?., 1968. Personal files of D.U. Deere from consulting meetings for Washington, D.C. subway.

MERRITT, A.H., 1968. "Engineering Classification for In-Situ Rock," Ph.D. Thesis, Department of Geology, University of Illinois, Urbana.

MONSEES, J.E., 1970. "Design of Support Systems for Tunnels in Rock," Ph.D. Thesis, Department of Civil Engineering, University of Illinois, Urbana.

ONODERA, T.F., 1963. "Dynamic Investigation of Foundation Rocks In-Situ," Proceedings of the Fifth Symposium on Rock Mechanics, Pergamon Press, New York, pp. 517-533.

PATTON, F.D., 1966. "Multiple Modes of Shear Failure in Rock and Related Material," Ph.D. Thesis, Department of Geology, University of Illinois, Urbana.

RABCEWICZ, L. v., 1944. Gebirgsdruck und Tunnelbau, Springer-Verlag, Vienna, Austria.

RABCEWICZ, L. v., 1964-65. "The New Austrian Tunneling Method," Water Power, November, pp. 453-457; December, pp. 511-516; January, pp. 19-25.

RABCEWICZ, L. v., 1969. "Stability of Tunnels Under Rock Load," Water Power, June, pp. 225-229; July, pp. 266-273; August, pp. 297-302.

RAHM, L., 1965. "Deformations- och Ljudhastighetsmätningar i Bergtunnlar," Bergmekanik, Ingeniörsvetenskapsakademiens meddelande 142, Stockholm, pp. 178-183.

ROSENBLAD, J.L., 1968. "Development of a Rock-Like Model Material," Preprint of paper for the Tenth Symposium on Rock Mechanics, University of Texas, Austin.

ROTTER, E., 1961. "Spritzbeton und seine praktische Anwendung im Untertagebau," Berg- und Hüttenmännische Monatshefte, Vol. 5/6, May-June, Vienna, pp. 152-165.

SANDSTRÖM, G., 1963. The History of Tunneling, Barrie and Rockliff, London, 427 pp.

SCHERMAN, K.A., 1959. "Förundersökningar av Berg," Bergsprängning, Ingeniörsvetenskaps-akademiens FKO-meddelande Nr. 30, Stockholm, pp. 9-20.

SCOTT, J.H., CARROLL, R.D. & D.R. CUNNINGHAM, 1965. "Seismic Studies in Pile Driver Tunnels, Area 15, Nevada Test Site, Nevada," Technical Letter: Pile Driver -1, United States Department of the Interior, Geological Survey.

SCOTT, J.H., LEE, F.T., CARROLL, R.D. & C.S. ROBINSON, 1968. "The Relationship of Geophysical Measurements to Engineering and Construction Parameters in the Straight Creek Tunnel Pilot Bore, Colorado," International Journal of Rock Mechanics and Mining Sciences, Vol. 5, Nr. 1, pp. 1-30.

SELMER-OLSEN, R., 1966. Alminnelig Geologi og Ingeniør Geologi, Tapir forlag, Trondheim, 409 pp.

SELMER-OLSEN, R., 1968. Personal communication, Trondheim.

SKÅNSKA CEMENTGJUTERIET, 1967. "Försök med Metoden Förspräckning-Robotsprutning vid Åsens Kraftverkstunnel," Internal circular, AB Skånska Cementgjuteriet, Gothenburg.

SÄLLSTRÖM, S., 1967. "Berggrundsproblem vid Bergeforsens Kraftverk," "Väg- och vatten-byggaren, Vol. 3.

TERZAGHI, K.T., 1936. "Stress Distribution in Dry and in Saturated Sand above a Yielding Trap-door," Proceedings of the First International Conference on Soil Mechanics, Cambridge, Massachusetts, Vol. 1, pp. 307-311.

TERZAGHI, K.T., 1943. Theoretical Soil Mechanics, John Wiley and Sons, New York, pp. 66-76.

TERZAGHI, K.T., 1946. "Rock Loads and Defects on Tunnel Supports," Section I in Rock Tunneling with Steel Supports, Proctor, R.V., and T.L. White, The Commercial Shearing and Stamping Company, Youngstown, Ohio.

TROLLOPE, D.H., 1966. "The Stability of Trapezoidal Openings in Rock Masses," Rock Mechanics and Engineering Geology, Vol. IV/3.

APPENDIX A

SEISMIC REFRACTION MEASUREMENTS

A.1 General

The usefulness of seismic refraction measurements for the planning of underground construction in rock has been known in Sweden for at least twenty-five years. The earliest applications were primarily for the determination of the depth to bedrock, or the thickness of soil cover (Hasselström, 1951). Refinements in the interpretation technique led to further application of the refraction method for the accurate determination of bedrock velocities. Since 1959 the method has been used very successfully in Sweden for the localization of weak zones at the bedrock surface (Scherman, 1959; Hasselström et al., 1964). Two examples of cases where low seismic velocity zones determined from ground surface refraction measurements correlate well with difficult tunneling conditions are shown in Figures A.1 and A.2.

The seismic refraction technique has been used in the pilot bore of the Straight Creek highway tunnel in the Rocky Mountains, Colorado, USA, to establish relationships between rock quality and various construction parameters, such as rates of advance and spacings of steel sets (Scott et al., 1968). The results from these studies show that the longitudinal seismic velocity as determined from the seismic refraction technique provides a reliable numerical index of rock quality that can be related to the economic and engineering aspects of tunneling. It was concluded from the Straight Creek studies that, if such measurements should prove to be valid for indexing rock quality for a wide variety of geologic conditions, then it would be worthwhile to direct efforts towards the development of methods to measure seismic velocities in feeler holes that could be driven out ahead of the working face during the driving of the tunnel. This method of investigation might prove to be particularly suited to mechanical "mole" tunneling.

A potential use in Sweden of such seismic velocity-tunneling relationships as those developed in the Straight Creek tunnel also lies in their application to velocities determined by conventional ground surface refraction measurements. In order to test the applicability of seismic velocity data as a measure of rock quality for Swedish bedrock and tunneling contitions, an investigation was made in the tailrace tunnel at the Rätan hydroelectric project in Jämtland, northern Sweden. Ground surface seismic refraction measurements over the entire tunnel line had been made during the preliminary investigation stage at Rätan. These investigations were complemented with a total length of 1010 meters of seismic refraction profile that was shot in the tunnel during the construction period. The purpose of these latter measurements was the following:

1. To indicate the most reasonable interpretation of the velocities provided by surface refraction measurements.

2. To determine the significance of seismic velocity as a measure of rock quality with respect to different tunnel support requirements in loosening ground conditions.

3. To investigate the extent of the destressed zone at the periphery of the tunnel.

A.2 Tunnel Measurement Technique at Rätan

The tunnel measurements at Rätan were carried out by Craelius Terratest AB (formerly AB Elektriska Malmletning) in essentially the same manner as the ground surface measurements that were made by the same company during the preliminary investigations. Descriptions of the principle of seismic refraction measurement are available elsewhere (Hasselström, 1951; Scherman, 1959) and will not be given here. Some of the technical details of the Rätan tunnel measurements are given below.

A geophone spacing of 5 meters was used for most of the measurements in the tunnel. Shorter sections of profile were shot with 2.5- and 1-meter spacings to investigate in more detail the extent of the destressed zone at the tunnel periphery.
The geophones were of the ordinary spring suspension

LEGEND:

DBH diamond drill hole

cast concrete support

shotcrete support (50 cm thick)

shotcrete support (20 cm thick)

low velocity zone on bedrock surface

NOTES:

1. Ground surface, bedrock surface, and seismic velocities have been projected from a seismic refraction profile located 100 meters from tunnel.
2. Bedrock seismic velocities given in m/sec.

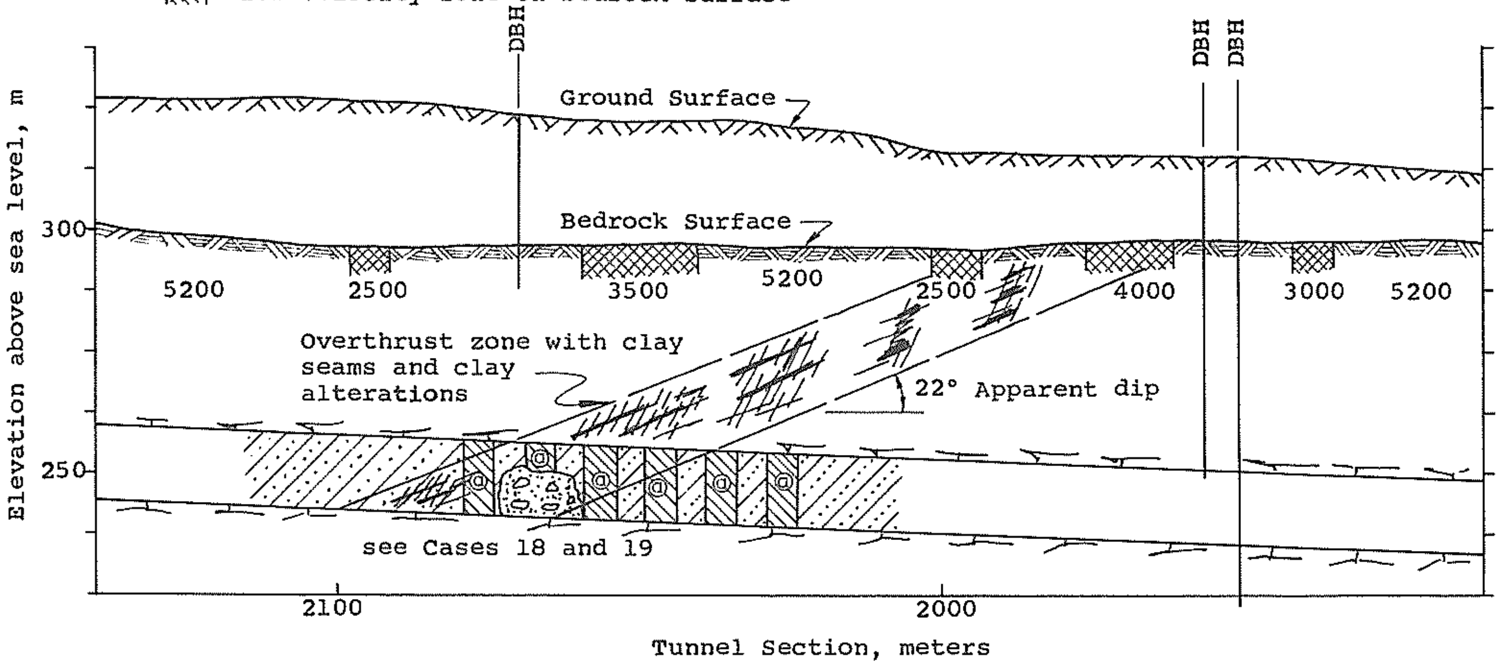


Fig. A.1 Example of Low Bedrock Surface Velocities Related to Poor Tunneling Conditions, Seitevare Hydroelectric Project

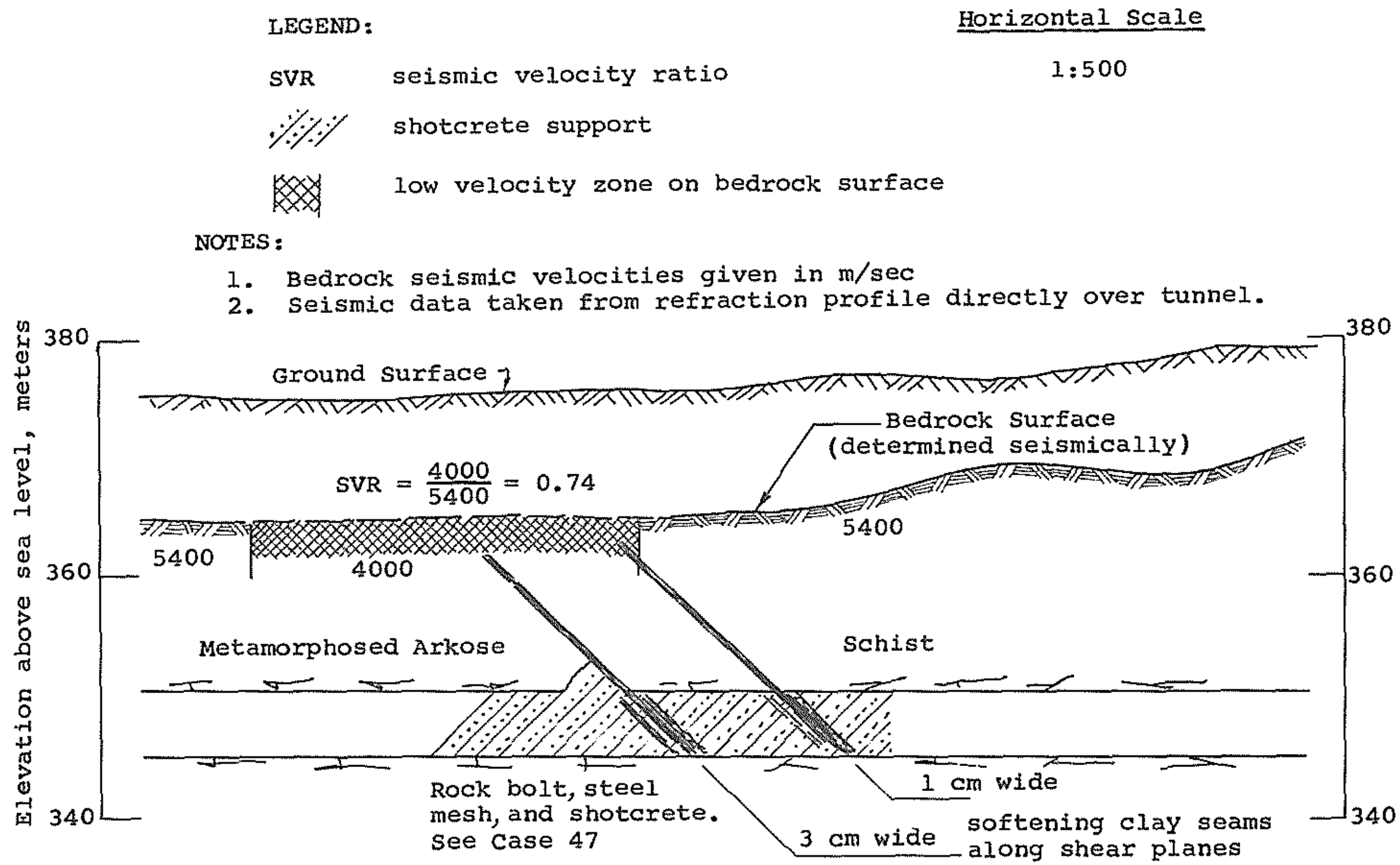


Fig. A.2 Example of Low Bedrock Surface Velocities Related to Poor Tunneling Conditions, Bergvattnet Hydroelectric Project moving coil type (Hall-Sears manufacture) with frequency response in the 5 -300 cps range. Recording was done photographically at paper speeds of 2.5-5 m/sec.

The set-up for shot points and geophones is shown in Figure A.3. Shot holes were drilled one-meter deep into the corner of the tunnel wall and floor. A one-to-two-inch stub of dynamite was used for the shot charge. The geophones were originally screwed into 18-inch long steel dowels that could be firmly implanted in the tunnel floor, but it was later found easier and equally effective to screw the pickups into 4-inch diameter, 1-inch thick steel plates and place the latter directly on the tunnel floor.

A normal shot line consisted of 20 geophones on 5-meter spacings. Shot points were located at both end points, at 20 meter intervals within the geophone spread, and at 25 meters and 50 meters from each end of the spread.

Clean records were obtained for most of the entire profile length of 1010 meters. The only disturbances were experienced in a diabase and amphibolite inclusion that apparently conducted leak currents from the tower of a 20,000 volt high tension line that crosses the tunnel at the ground surface above that station. It was generally not possible to detect shear wave arrivals.

A.3 Results from the Rätan Measurements

Intact Rock Seismic Velocities

The tunnel refraction profiles at Rätan included measurements in the following rock types: coarse-grained granite; fine-grained aplitic granite; amphibolite; and diabase. Because of the very detailed round-by-round geologic mapping that was done in the tunnel, it has been possible to correlate the seismic velocities with rock types, support quantities, and in a few cases, rock quality classifications. Some Schmidt N-hammer hardness values were also measured.

The four different rock types, the maximum seismic velocities in these different rock types, and Schmidt N-hammer hardness values are given in Table A.1. After the seismic measurements had been completed and analyzed it became evident that three different hardnesses of coarse-grained granite exist along the profiles. The 6000 m/sec rock corresponds to the granite in which the Schmidt N-hammer hardness of 55 was obtained. Both the velocity and the hardness values are the maximum values for the corresponding rock types. The rock conditions to which these properties correspond are those that do not require any support to maintain the desired tunnel shape. The rock is jointed in some cases, but very tight. The approximate relationship between seismic velocity and Schmidt N-hammer hardness is shown in Figure A.4.

One very pronounced feature in the tunnel data is the increase in longitudinal seismic velocity over that obtained from ground surface measurements in the same rock. These differences are illustrated in Table A.2 in which it is seen that the tunnel velocities are up to 17 percent higher than the surface velocities for high quality rock and up to 38 percent higher for low quality rock. These differences can be attributed to two factors:

(1) An overall loosening of rock at the bedrock surface that has occurred after glacial retreat. Weak zones at the bedrock surface probably are considerably more relieved and loosened than sounder rock.

(2) A tightening or increase of confining stress with depth that effectively holds jointed rock together.

It is thus obvious that direct comparisons of bedrock surface and tunnel measurements cannot be made.

Normalization of Field Data

The velocity range along all of the tunnel profiles of from 3700 m/sec to 6400 m/sec is alone an indication of the variation in rock quality in the tunnel. However, to make any valid comparisons of the velocities in different sections of the tunnel for different supported conditions, it is first necessary to normalize all the velocities so that differences due to intact rock

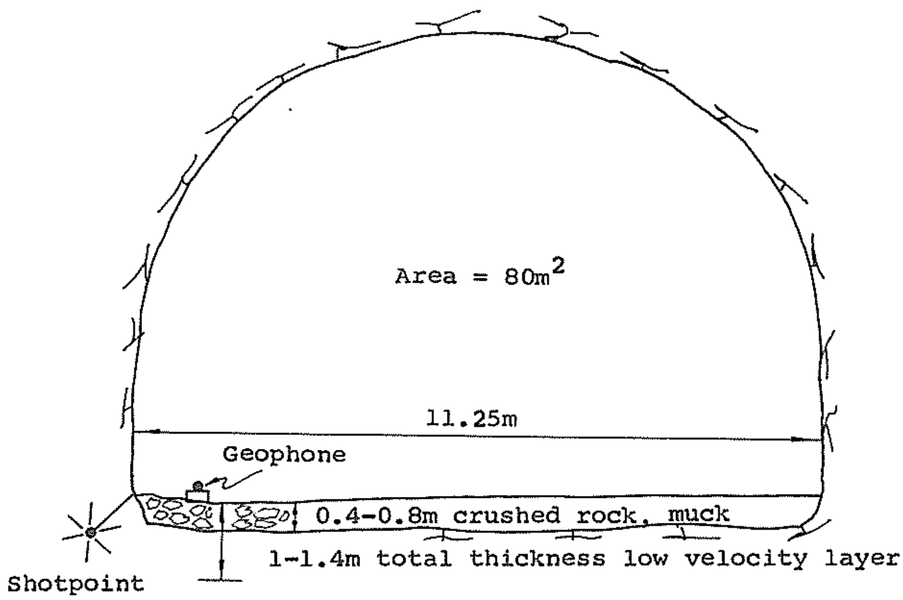


Fig. A.3 Geophone Set-Up for Tunnel Refraction Measurements at Rätan

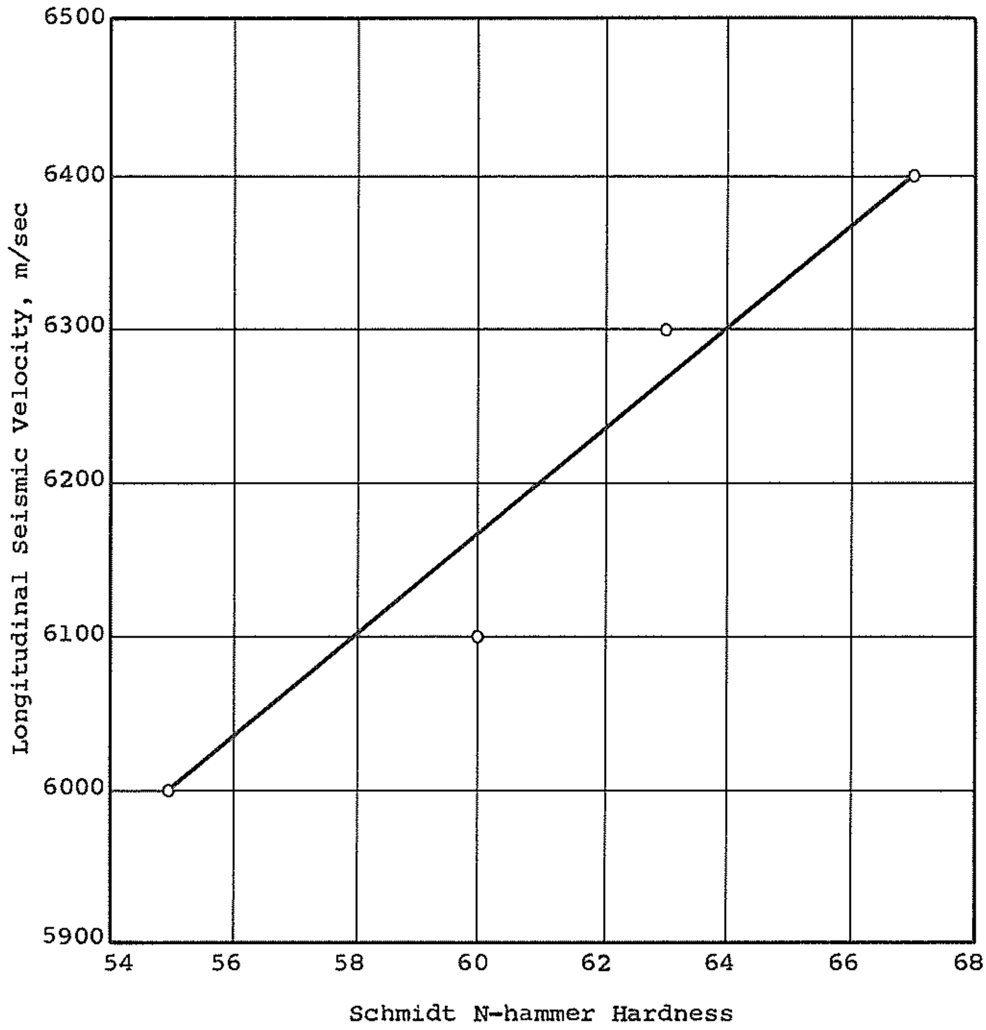


Fig. A.4 Relationship Between Schmidt N-hammer Hardness and Longitudinal Seismic Velocity properties are eliminated.

The tunnel seismic velocity for a given rock condition in a given rock type has been normalized by dividing by the maximum seismic velocity (as given in Table A.1) for that particular rock type. The ratio so obtained has been termed the seismic velocity ratio (SVR). This idea of establishing a "base velocity" for each rock type is similar to that used by Lakshmanan (1966). Deere's velocity ratio is a similar tool for normalizing the field seismic velocity (Deere et al., 1967). Onodera (1963) was apparently the first investigator to propose such a quality index for in-situ rock.

TABLE A.1

MAXIMUM SEISMIC VELOCITIES AND REBOUND HARDNESSES FOR RÄTAN ROCK TYPES

Rock Type		Velocity of High Quality Rock m/sec	(ft/sec)	Schmidt N-hammer hardness
Coarse-grained Granite (1)	a	5500	(18,000)	--
	b	6000	(19,700)	55
	c	6400	(21,000)	--
Fine-grained Aplitic Granite (2)		6400	(21,000)	67
Amphibolite (3)		6100	(20,000)	60
Diabase (4)		6300	(20,700)	63

The seismic velocity ratio is a normalized indicator of rock quality, that is, a combined measure of the following items:

1. Degree of jointing
2. Degree of alteration
3. Degree of openness of joints, or looseness of rock structure.

Presentation of Rätan Tunnel Data

The seismic velocities for the unsupported rock conditions along the tunnel refraction profiles are shown in Table A.3. The conditions are grouped according to the different rock types and maximum velocities. Where it has been possible to project weak zones in the tunnel to low velocity bedrock surface zones, the bedrock surface velocities are shown in parentheses, together with the adjacent high velocity rock at the bedrock surface. The tunnel seismic velocity ratios are computed as described previously. The bedrock surface seismic velocity ratios are computed from the velocity values shown in parenthesis. It is assumed that the rock immediately adjacent to low velocity zones is the same type as that in the low velocity zone. The support used in tunneling driving is shown according to the legend.

A.4 Other Seismic Data

Ground surface seismic refraction measurements also were made in the preliminary investigation stage of the Bergvattnet and Seitevare projects. The techniques used in these investigations are the same as those used in the Rätan project and, because the measurements and interpretations were made by the same company, the results from all of the investigations may be studied collectively.

Because the rock cover in the Rätan, Bergvattnet, and Seitevare projects is relatively thin (less than 100 meters), and because major bedrock weakness zones are usually planar and continuous, it is very frequently possible to correlate zones of poor rock in a tunnel with low velocity zones on the bedrock surface. Such correlations have been made for a number of the tunnel observations at Bergvattnet and Seitevare in addition to those at Rätan. The correlated bedrock surface velocities for the former two projects are presented in section 2.5. Correlations between support quantities and seismic velocity data are made in Chapter 5. Information from the tunnel seismic measurements pertaining to the extent of the destressed zone along the refraction profiles is presented and discussed in Chapter 4.

TABLE A.2

COMPARISON OF THE RELATIVE MAGNITUDES OF TUNNEL VELOCITIES AND SURFACE VELOCITIES

Rock Type	Tunnel Velocity m/sec (1)	Surface Velocity m/sec (2)	Ratio $\frac{(1)}{(2)}$
High Quality Rock			
Granite	6000	5300-5500 5400	1.11
Granite	6400	5500-5700 5600	1.14
Diabase	6300	5400	1.17
Low Quality Rock			
Granite	5000	3500-4000 3800	1.32
Diabase	4000	2900	1.38
Fine-grained Granite	4000-4650 4300	3700-3800 3750	1.15
Fine-grained Granite	4000	3500	1.15

Note: The ratios (1)/(2) have been computed from the average underscored velocities.

TABLE A.3

SEISMIC VELOCITY DATA AND SUPPORT REQUIREMENTS FOR ROCK CONDITIONS ALONG TUNNEL SEISMIC PROFILES

Rock Type	Rock Condition	Seismic Velocity m/sec	SVR tunnel	SVR surface	Support*
(1a) Coarse-grained Granite	shear zone with altered seam, water-bearing	4500	0.82	--	O
	blocky rock, some altered seams	4500	0.82	--	▽O
	shear zone with some open, water-bearing joints	4500	0.82	--	O
(1b) Coarse-grained Granite V_{max} = 6000 m/sec	blocky, loose rock around a one-meter wide diabase vein, water-bearing	(4000/5200)	--	0.77	▼O
	intersecting diabase and quartz veins in jointed rock	(3800/5200)	--	0.73	▽O
	open, water-bearing seam in massive rock	4500	0.75	--	△
(1c) Coarse-grained Granite V_{max} = 6400 m/sec	two 50-cm wide vertical clay seams in blocky ground	5900	0.92	--	O
	one-meter-wide, flat-lying sheared, altered seam in large to small blocky rock, partly altered; some open, water-bearing fissures, some seams gravel-filled due to "Rappakivi" alteration	5000	0.78	0.56 - 0.74	▽OO
	two-meter-wide altered diabase vein in a shear zone, prominent structure is horizontal	5600	0.87	--	▽O
	shear zone with a one-meter-wide diabase vein and a talc seam	(4500/5600)	--	0.81	▽O
	one open, water-bearing joint in massive rock	4500	0.70	--	△

*Support Legend

△ no support
□ widely spaced (>5m) spot bolts
▽ medium spaced (2-5m) spot bolts
▼ closely spaced (<2m) spot bolts
O single shotcrete application (4-6 cm)
● reinforced shotcrete
■ cast concrete arch

Symbols used in combination, i.e. ▽OO indicates medium spaced spot bolts and two shotcrete applications.

TABLE A.3. Continued

Rock Type	Rock Condition	Seismic Velocity m/sec	SVR tunnel	SVR surface	Support*
(2) Fine-grained Aplitic Granite V_{max} = 6400 m/sec	shear zone, very heavily fractured rock in center of zone	(3400-5000)	--	0.68	○○, ○○○
	an open, water-bearing seam (one meter wide)	4000	0.63	--	△
	one to two-meter-wide altered seam, water-bearing	4000	0.63	--	○
	shear zone, see Case 81 Appendix B, RQD = 30%	4000 (3500-5200)	0.63	0.67	○○
	shear zone	4400 (3600-5200)	0.69	0.69	○
	closely spaced vertical joints near amphibolite contact	4000	0.63	--	○
	closely spaced vertical joints some water-bearing	4000	0.63	--	○
	shear zone, sugar-cube rock structure, some clay-filled joints	4400	0.69	--	○○
	thrust zone, see Case 85, Appendix B, RQD = 30%	4200-4650 (3800/5200)	0.66 - 0.73	0.73	○○○
	thrust zone, intersecting diabase veins and seams of altered granite similar to Case 85	3700 (3700-4900)	0.58	0.75	○○○
	blocky rock, see Case 82 Appendix B, RQD = 80%	5500	0.86	--	▼○
	closely spaced vertical joints, tight RQD = 80% see Case 83, Appendix B	6400	1.00	0.73	△
(3) Amphibolite V_{max} = 6100 m/sec	very blocky rock, see Case 86, Appendix B, RQD = 90-100%	5800	0.95	--	▽
	very blocky and loose rock around a talc-serpentine seam	4000	0.66	--	▼○
(4) Diabase V_{max} = 6300 m/sec	one-meter-wide fractured and altered vein; surrounding rock blocky and loose	(3400)	--	0.65	▽○
	large blocky rock	5800	0.92	--	▽
	shear zone, see Case 84 Appendix B, RQD = 60%	5000 (2900-5400)	0.79	0.54	○

TABLE A.3. Continued

Rock Type	Rock Condition	Seismic Velocity m/sec	SVR tunnel	SVR surface	Support*
Continued (4)	large blocky rock near shear zone	5400	0.86	--	O
	shear zone	4000	0.63	0.77	OO
	very blocky and loose rock around a chlorite-talc seam	4300 (3800/5000)	0.68	0.76	▼O
	shear zone, very blocky	5200	0.83	--	O

APPENDIX B

FIELD OBSERVATIONS IN NORWAY AND SWEDEN

INFORMATION FORMAT FOR CASE HISTORIES

1. Project, location
2. Type of tunnel or room
3. W = width of opening, meters
4. H = height of opening, meters
5. A = area of opening, square meters
6. Nature of instability (roof fall, wall slip-out, overbreak, etc.); (stability classification, see legend below)
7. L = length of condition under consideration, meters
8. Geologic features responsible for condition, rock type
9. Support or remedial measure
10. D = depth of overburden (soil and rock), meters
11. RQD, location, method; RQD_v; RQD_a
12. V = seismic velocity, m/sec;*
13. SVR = seismic velocity ratio
14. Regional tectonics or major structural geology features
15. Ground water condition
16. Other notes

LEGEND FOR TIME-STABILITY-SUPPORT CLASSIFICATION

A stable at blasting, no anticipated falls, no support

B minor falls or overbreak at blasting, support not considered necessary for prevention of loosening

C stable at blasting, support in anticipation of loosening

D stable at blasting, unsupported, gradual deterioration and subsequent support

E falls at blasting, support in anticipation of progressive loosening

F falls at blasting, no support immediately after blasting, progressive loosening, support applied to prevent further loosening

G falls at blasting, support shortly after blasting to prevent or stop progressive loosening

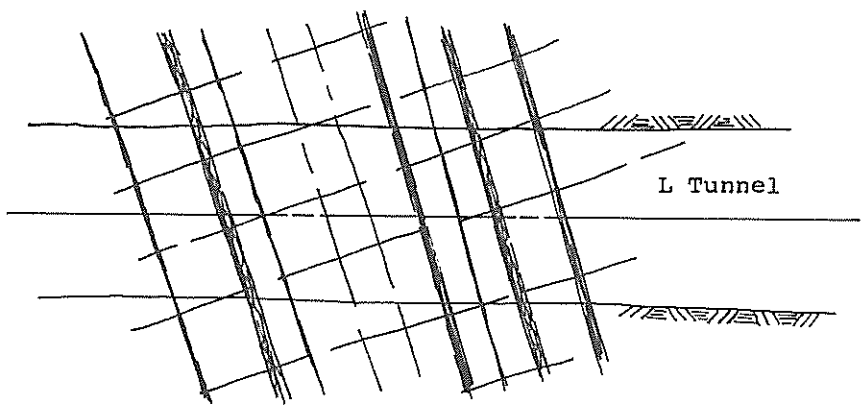
H support shortly after blasting, failure of support thereafter, additional support

---

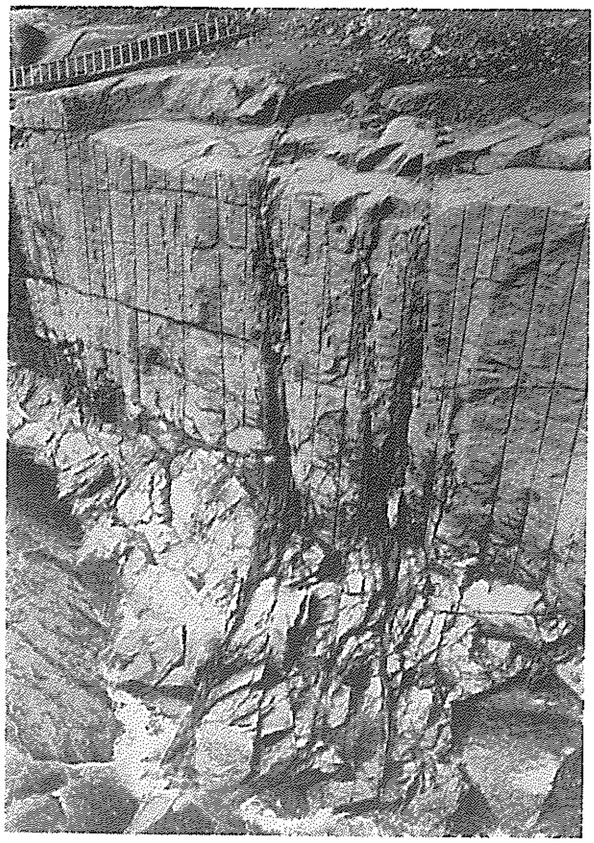
*Values given in parentheses have been taken from projected ground surface data.

Case 1

1. Seitevare Hydroelectric, northern Sweden
2. draft tube
3. W = 9m
4. H = 9m
5. A = $98m^2$
6. anticipated roof falls if unsupported; (C)
7. L = 30m
8. seven narrow (20-50cm) vertical shear zones of fractured and altered granite; some cross joints (see sketch)
9. 6-8cm shotcrete over entire area; locally 15cm reinforced shotcrete for treatment of individual shear zones; spot bolting between zones
10. D = 190m
11. RQD = 70% along wall, estimated from observation of similar features in nearby rock cut (see photo) and from geologic mapping of zone in tunnel; RQD_v = 0-100%
12. (V = 3000-3300 m/sec)
13. (SVR = 3150/5000 = 0.63)
14. project site located about 10km from major overthrust mountain range; several major local vertical shear zones within 200m
15. minor flows (1-3 lit/min)
16. special case: extremely important to avoid roof or wall deterioration in draft tube



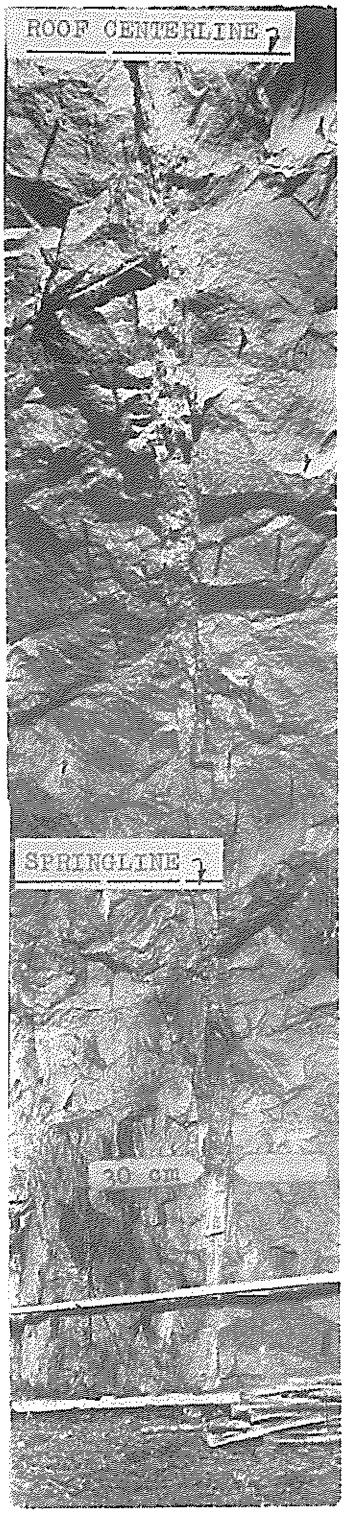
Plan of Tunnel



Case 1. Similar Shear Zones in a Nearby Cut

Case 2

1. same as Case 1
2. tailrace tunnel
3. same as Case 1
4. same as Case 1
5. same as Case 1
6. anticipated roof falls if unsupported; (C)
7. L = 5m
8. one 30-cm-wide vertical shear zone in granite, bounded by sound, blocky rock; located 20m from Case 1
9. spot rock bolting, shotcrete
10. D = 190m
11. RQD = 90% along wall, measured; RQD_V = 0-100%
12. (V = 4300-4600m/sec)
13. (SVR = 4450/5000 = 0.89)
14. same as Case 1
15. less than 1 lit/min flow



Case 3

1–5. same as Case 2

6. overbreak in walls at blasting (see photo); (E)

7. L = 20m

8. intersection of three non-orthogonal joint sets in leptite

9. one shotcrete application

10. D = 200m

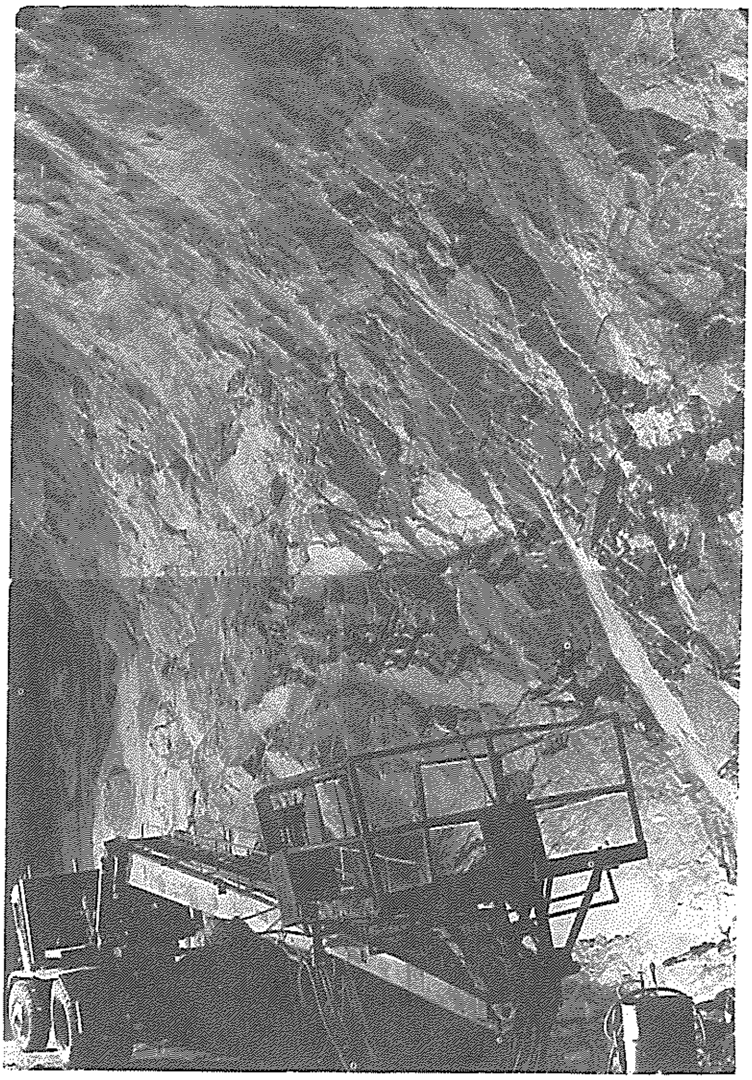
11. RQD = 60%, up wall, measured, RQD_a = 70%

12. ————

13. ————

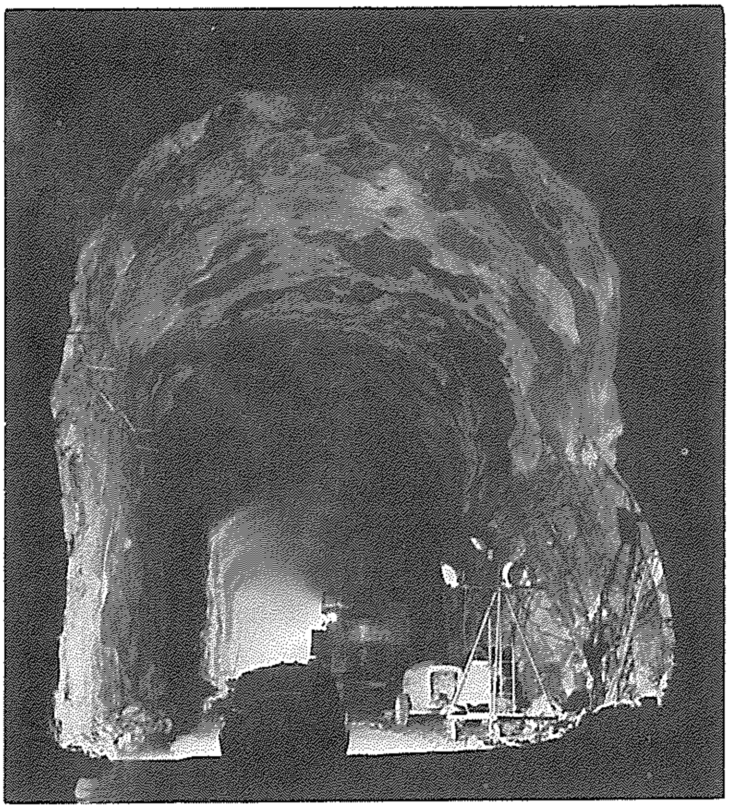
14. same as Case 1

15. no water



Case 4

1-5.	same as Case 2
6.	progressive large overbreak after blasting (see photo); (F)
7.	L = 30m
8.	clay seams and slickensides along sheared leptite bedding planes
9.	two shotcrete applications to stop loosening
10.	D = 170m
11.	RQD = 50% vertical, measured from core photo; RQD_a = 50%
12.	-----
13.	-----
14.	major normal and thrust shearing in area
15.	minor water seepage through shotcrete
16.	notice stable roof, unstable wall



Case 5

1-5. same as Case 2

6. overbreak, minor falls, washing out of fine rock flour; (D)

7. L = 10m

8. locally and erratically disintegrated rock along leptite bedding (see photo)

9. one shotcrete application

10. D = 150m

11. RQD = 80% up wall, measured and estimated; RQD_a = 80%

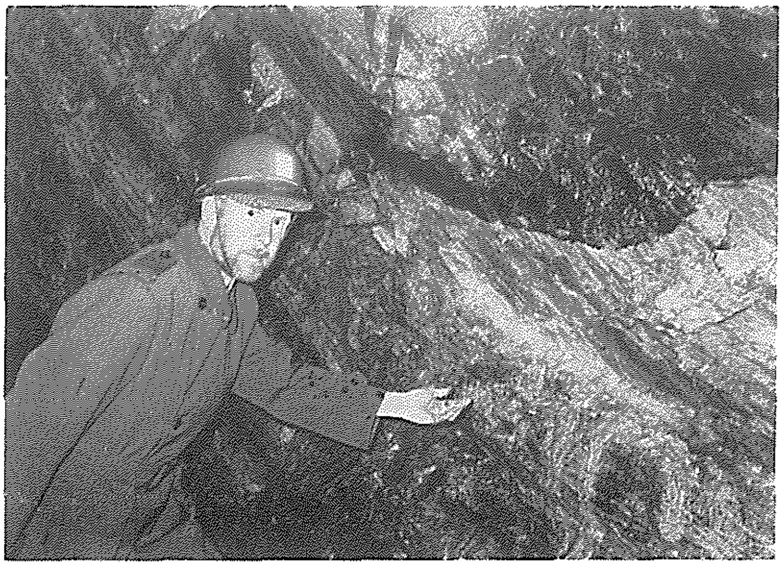
12. ----

13. ----

14. same as Case 4

15. less than 5 lit/min

16. very small water inflows washing fines from joints



Case 6

1-5. same as Case 2

6. minor overbreak, shotcrete used as a protection against small pieces of falling rock; (B)

7. L = 20m

8. closely, vertically jointed leptite (see photo)

9. none necessary from stability standpoint

10. D = 150m

11. RQD = 60% across jointing, measured; RQD_v = 0-100%; RQD_a = 80%

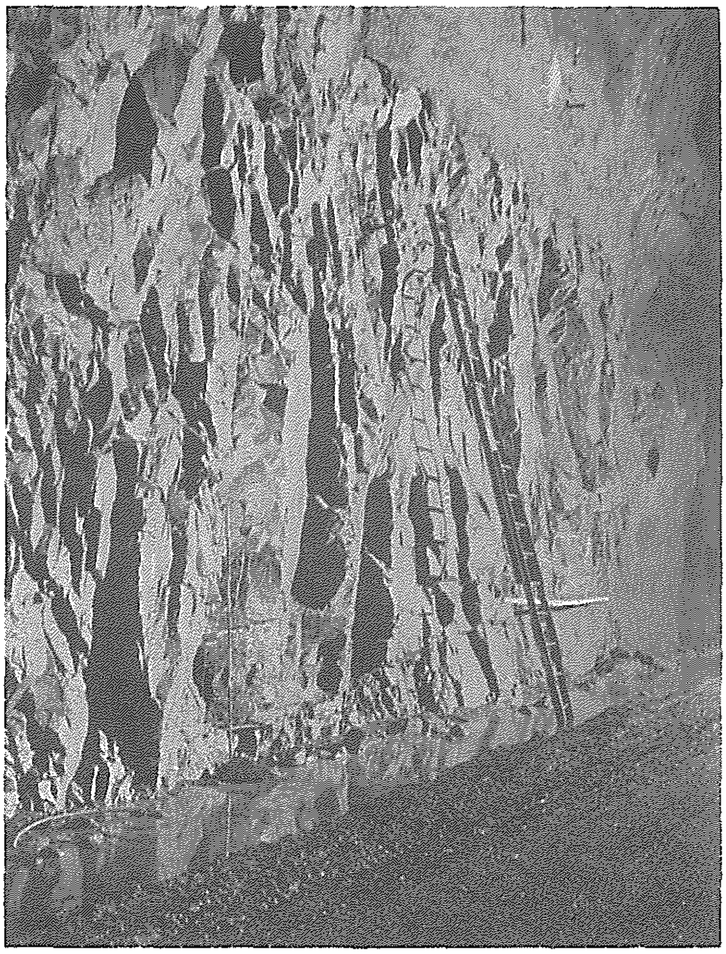
12. ----

13. ----

14. same as Case 4

15. no water

16. absence of roof falls is somewhat remarkable



Case 7

1-5. same as Case 2

6. large overbreak in wall and intrados, possible loss of arch abutment (see photo); (F)

7. L = 10m

8. intersection of a regular, closely-spaced set of diagonal joints and one vertical clay-filled fissure, leptite rock

9. rock bolts and one shotcrete application

10. D = 140m

11. RQD = 70% up wall, estimated; RQD_a = 70%

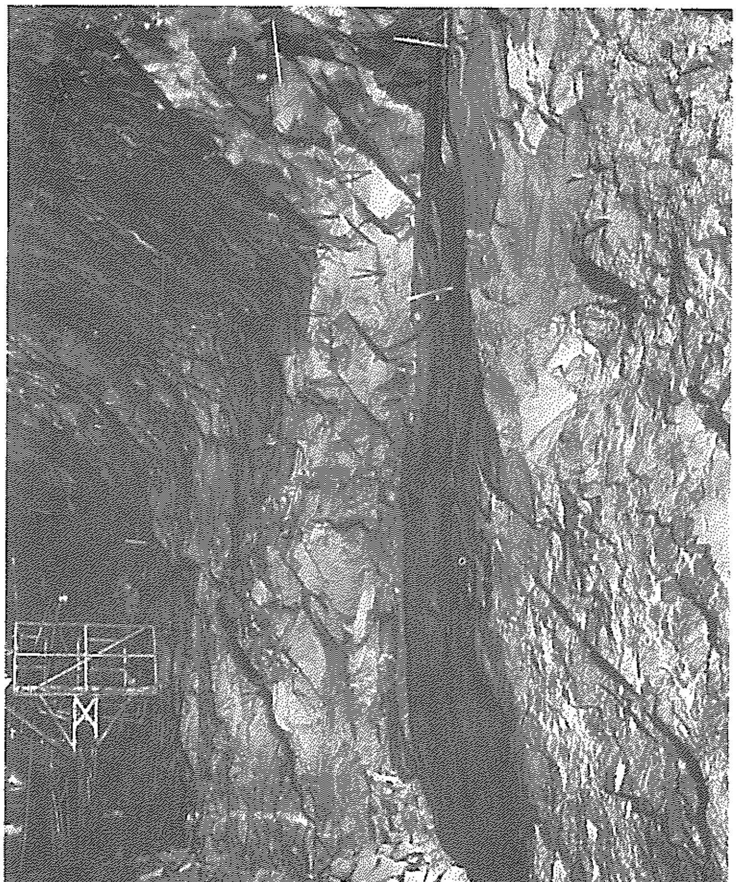
12. ----

13. ----

14. same as Case 4

15. insignificant water inflows

16. no falls or overbreak in roof



Case 8

1-5. same as Case 2

6. none; (B)

7. L = 50m

8. closely spaced set of tight diagonal joints in leptite; same as Case 7 but no intersecting joints

9. none necessary from stability considerations

10. D = 140m

11. RQD = 70% up wall, estimated; RQD_a = 70%

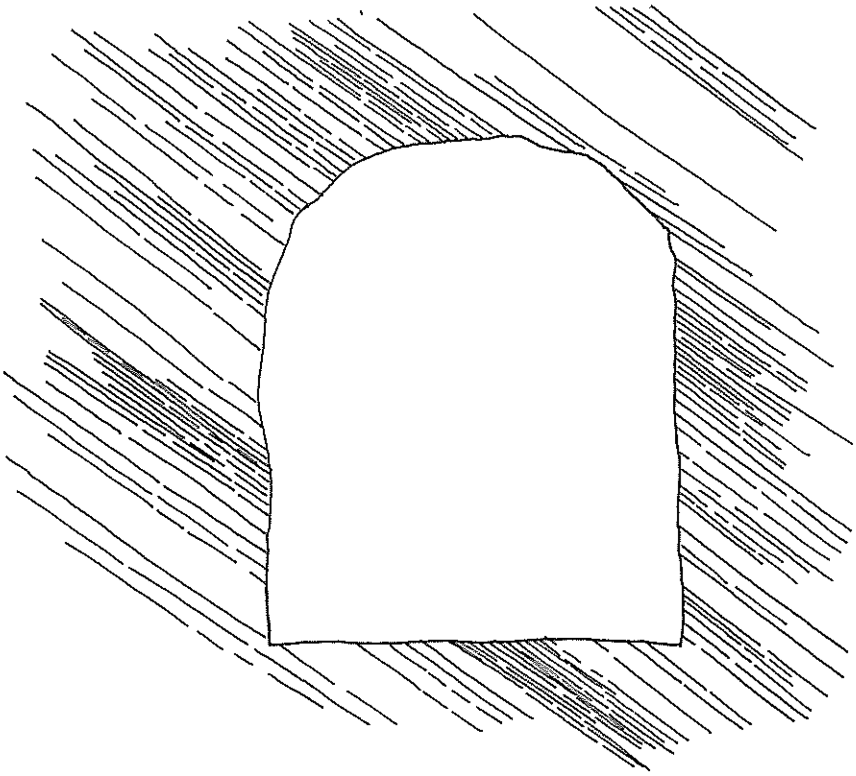
12. ----

13. ----

14. same as Case 4

15. none

16. note significance of minor geologic detail (single vertical joint) that differentiates behavior in Case 7 from that in Case 8



Case 9

1-5. same as Case 2

6. overbreak in intrados (see photo); (F)

7. L = 20m

8. small blocky structure in leptite

9. one shotcrete application

10. D = 140m

11. RQD = 80% up wall, estimated; RQD_a = 80%

12. ----

13. ----

14. same as Case 4

15. water inflow less than 1 lit/min

16. roof is stable at this section

Case 10

1-5. same as Case 2

6. overbreak and slip-outs in walls, likely loss of intrados and roof if unsupported; (F)

7. L = 15,

8. blocky to slabby structure in leptite caused by three nearly orthogonal, intersecting joint sets (see photo)

9. one shotcrete application

10. D = 120m

11. RQD = 60% across major joint set, measured; RQD_v = 60%; RQD_a = 70%

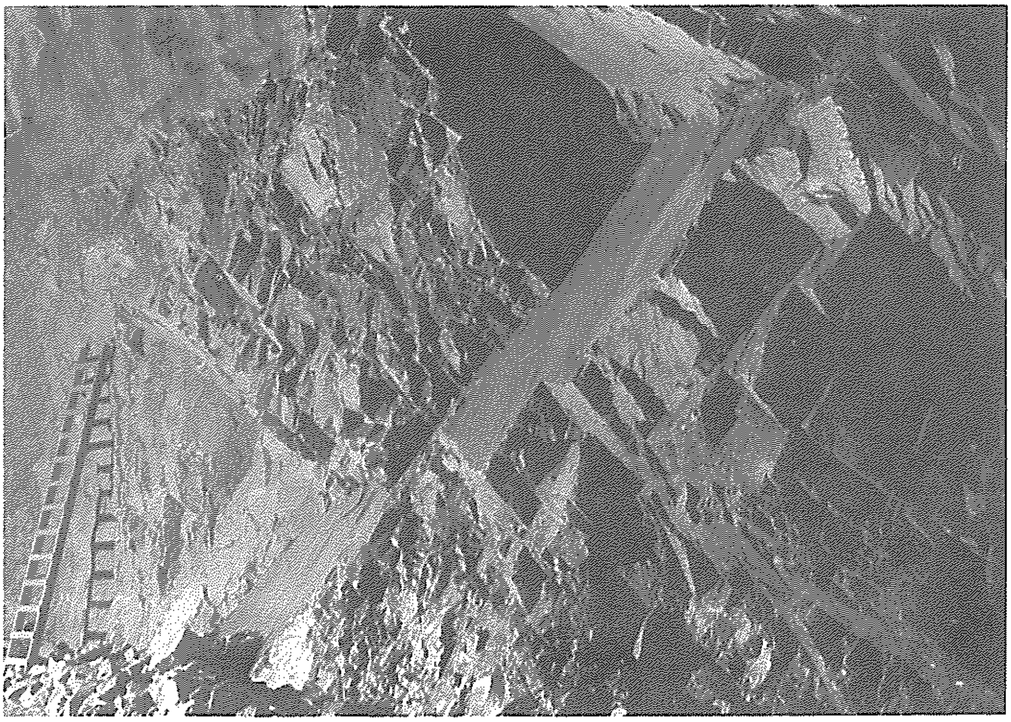
12. ----

13. ----

14. same as Case 4

15. none

16. no roof falls; major joint along which most of wall slip-out took place is clay-coated



Case 11

1-5. same as Case 2

6. wall slip out, possible undermining of arch support and subsequent collapse of crown; (D)

7. L = 15m

8. sliding took place along a 3-cm-thick clay seam that occurs in a shear zone parallel to the bedding in the leptite

9. rock bolts, two shotcrete applications

10. D = 120m

11. RQD = 20% across shear zone, estimated; RQD_v = 40%; RQD_a = 40%

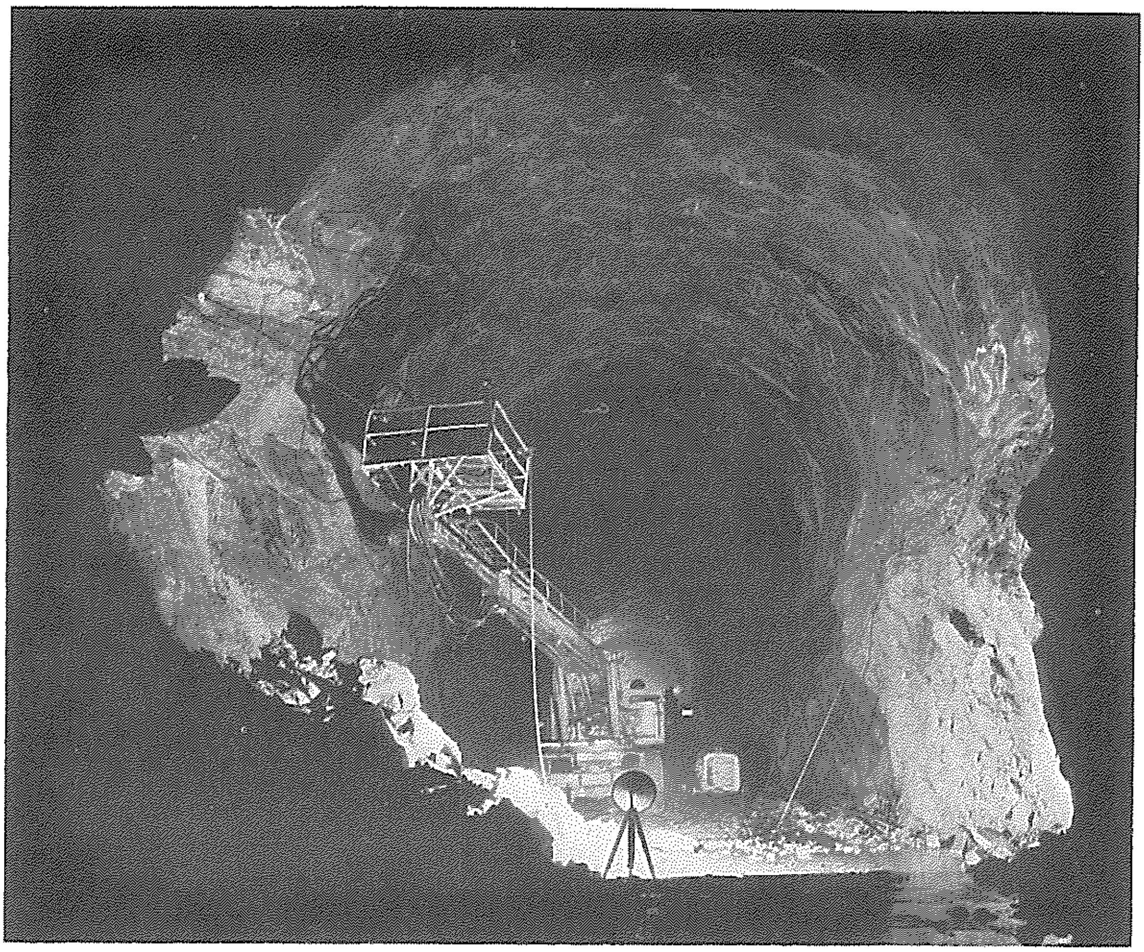
12. (V = 3800m/sec)

13. (SVR = 3800/5000 = 0.76)

14. same as Case 4

15. no water

16. feature has not caused any difficulty in roof



Case 11

Case 12

1-5. same as Case 2

6. overbreak in walls and intrados, stable roof; (D)

7. L = 150m

8. irregular, undulating, vertical, clay-coated joints, occasionally intersecting; granite

9. spot rock bolting

10. D = 60m

11. RQD = 100% in all directions, observed

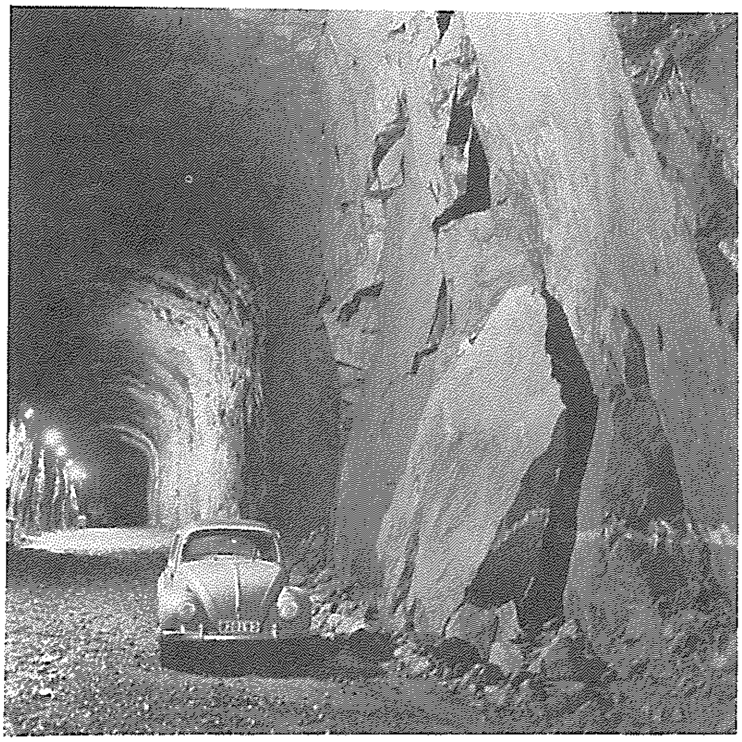
12. ----

13. ----

14. same as Case 4

15. no water

16. added cost in scaling, breaking large blocks and mucking



Case 13

1-5. same as Case 2

6. large overbreak in walls and intrados, some roof falls; support necessary to prevent loss of abutment reaction (see photo); (F)

7. L = 100m

8. irregularly oriented, widely spaced, intersecting, clay-coated joints; loose rock structure; many slickensides (black arrows); granite, weak rock type, occasionally breaks across intact rock because of alterations

9. rock bolting and shotcrete, locally two applications

10. D = 100m

11. RQD = 80% along wall, measured; RQD_V = 80%

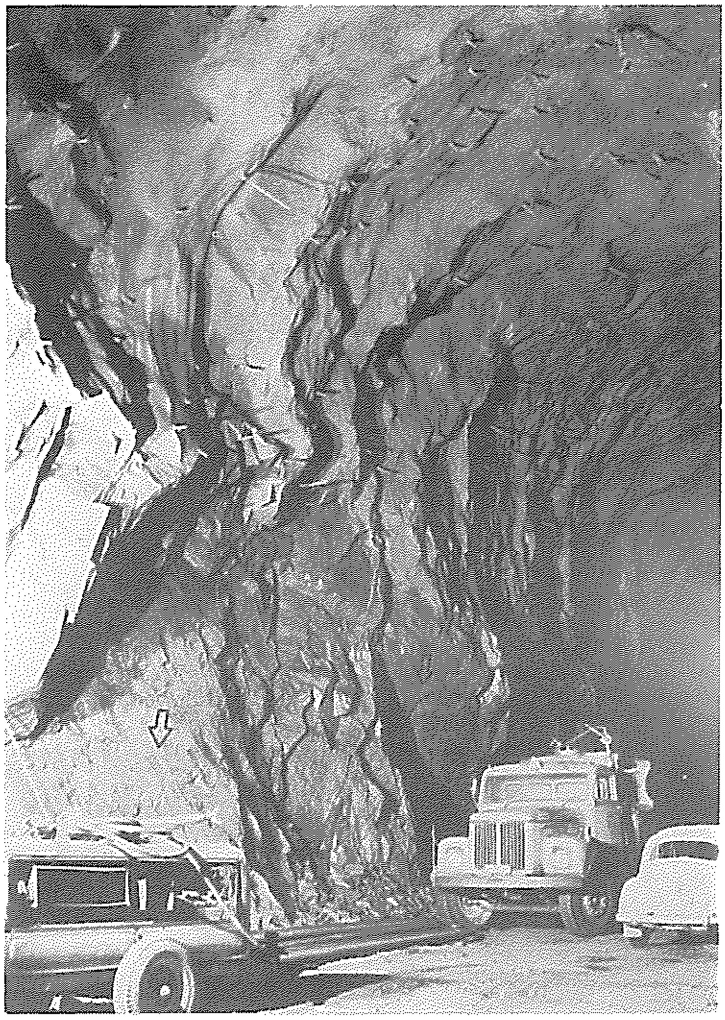
12. (V = 3100-4000 m/sec)

13. (SVR = 3100-4000 m/sec)

14. both vertical and low-angle shear zones in near vicinity, evidence of movement (slickensides) on most joint surfaces

15. some flows to 50 lit/min

16. difficult to predict behavior of tunnel, as failure often occurs across intact rock at weak, partially altered zones



Case 13

Case 14

1-5. same as Case 2

6. wall, roof, and intrados falls, large overbreak; (F)

7. L = 50m

8. intersecting vertical and 45° dipping joint systems, many clay-coated, slickensided joint surfaces; frequent breakage across intact rock; similar to rock in Case 13, weak, large-grained granite with some altered feldspars (see photo)

9. rock bolting; locally two shotcrete applications

10. D = 100m

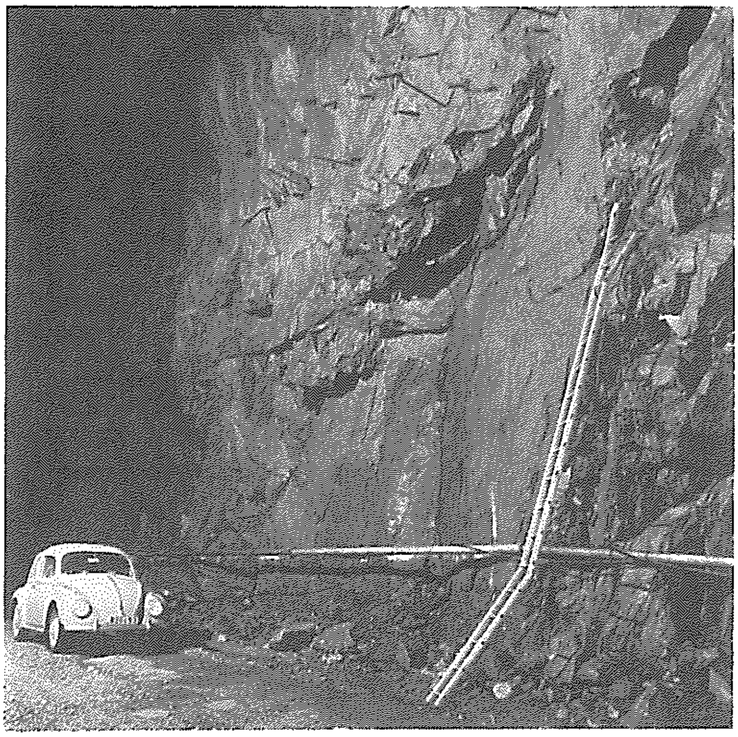
11. RQD = 70% up and along walls, measured

12. same as Case 13

13. same as Case 13

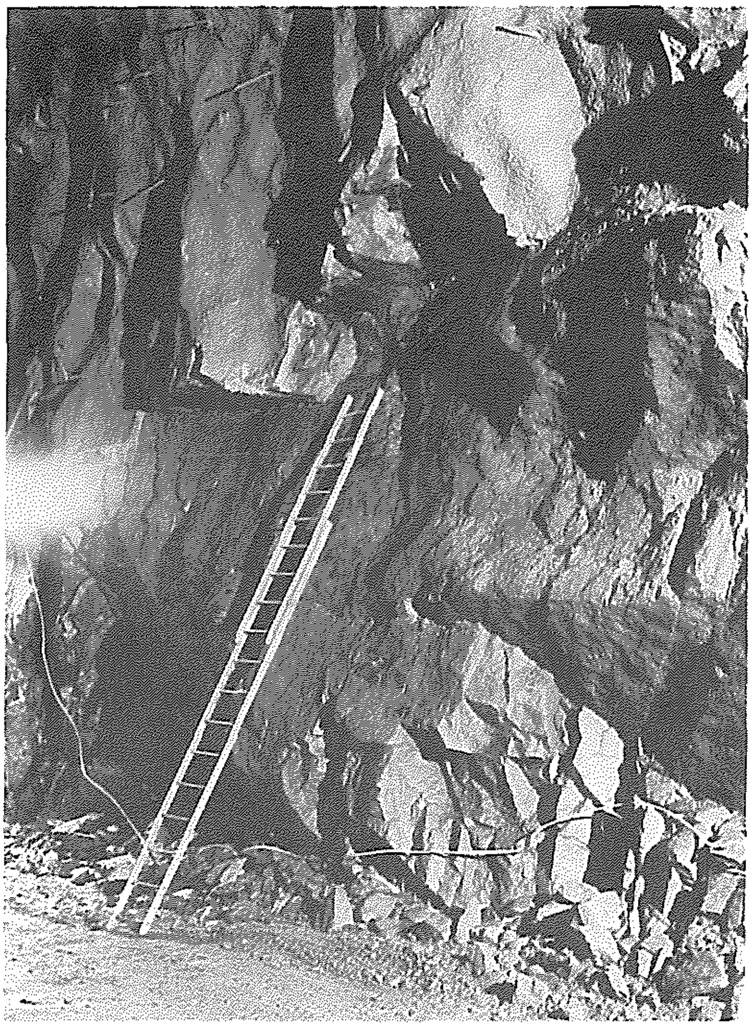
14. same as Case 13

15. minor inflows (<10 lit/min)



Case 15

1-5.	same as Case 2
6.	wall, roof, and intrados falls; large overbreak; (F)
7.	L = 60m
8.	same as Case 14, not as many joints
9.	rock bolting and one shotcrete application
10.	D = 100m
11.	RQD = 80% up and along walls, measured
12-14.	same as Case 14
15.	minor inflows



Case 16

1-5. same as Case 2

6. large overbreak in walls and intrados, very likely progressive fallout into crown if unsupported (see photo); (E)

7. $L = 20m$

8. three non-orthogonal intersecting joint sets in leptite

9. rock bolting, two shotcrete applications

10. $D = 120m$

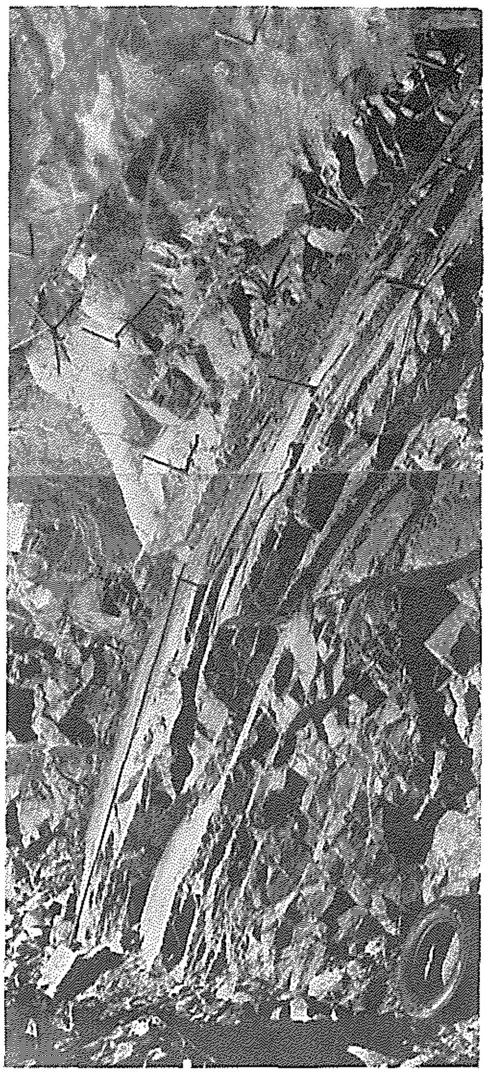
11. RQD = 60% along wall, estimated; RQD_v = 70%

12. ----

13. ----

14. same as Case 4

15. no water



Case 17

1-5. same as Case 2

6. moderate overbreak in walls, but no potentially unstable rock; (C)

7. L = 120m

8. set of steeply dipping, widely spaced, tight joints in granite

9. spot bolting; shotcrete for protection against small pieces of falling rock; rock in foreground is from scaling

10. D = 120m

11. RQD = 100% in all directions, estimated

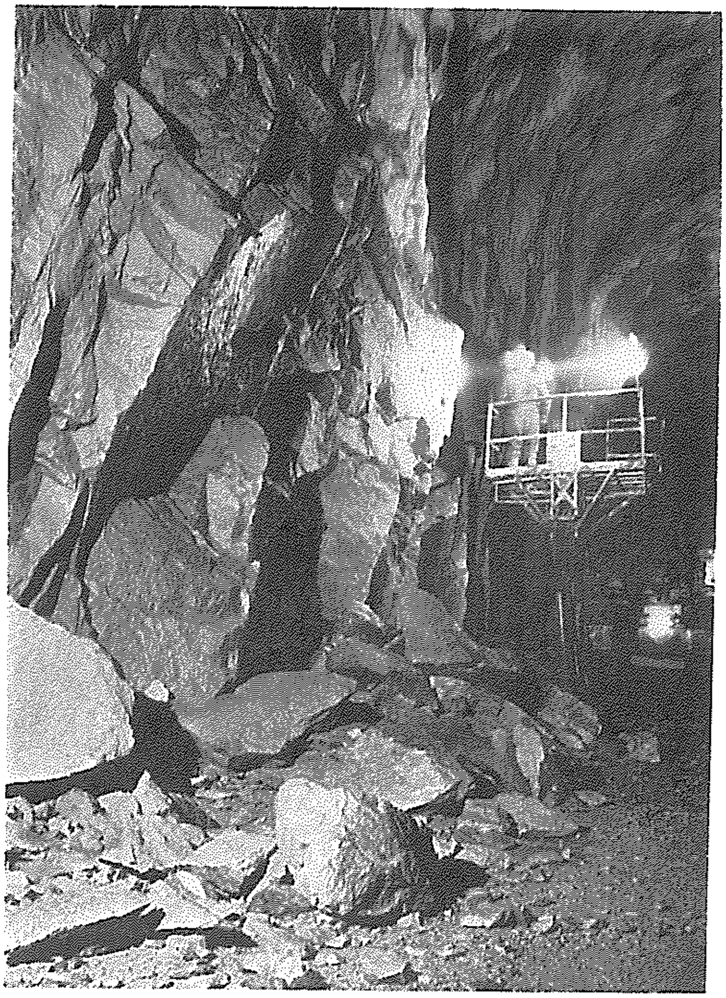
12. ----

13. ----

14. overthrusting and vertical faulting in near vicinity

15. minor ground water

16. considerable added tunneling cost in scaling



Case 18

1-5. same as Case 2

6. disintegration of wall rock, earth-like fallout in walls, possible undermining of arch abutments (see Fig. 2.14); (H)

7. L = 70m

8. strongly crushed, sheared and altered granite in an overthrust fault; feldspars hydrothermally altered to clay (see photo Case 19)

9. initially 5-10cm lightly reinforced shotcrete after blasting; after failure of initial support, 20-30cm unreinforced shotcrete and five unreinforced shotcrete arches (5m wide x 50cm thick) on 10m spacings. Some cast concrete abutments in worst areas, as in photo.

10. D = 50m

11. RQD = 10% up walls, measured; RQD_a = 0-80%

12. (V = 2500-4000 m/sec)

13. (SVR = 2500/5200 - 4000/5200 = 0.48-0.77)

14. overthrust zone in Precambrian rock; otherwise same as Case 1

15. minor water inflows froze in tunnel walls during winter; freeze-thaw action in summer months loosened entire overthrust zone; large portions of tunnel walls could be excavated with a hand spade after zone thawed during summer

Case 19

1-5.	same as Case 2
6-10.	same as Case 18
11.	RQD = 30% up wall, measured; RQD_a = 10-100%
12-15.	same as Case 18
16.	note failed shotcrete; ice (January) on walls. Rock was very solid and stable during winter months

Case 20

1–5. same as Case 2

6. minor overbreak in walls and roof; tunnel stable without support (see photo); (B)

7. L = 30m

8. closely spaced, tight vertical joints in leptite

9. none

10. D = 30m

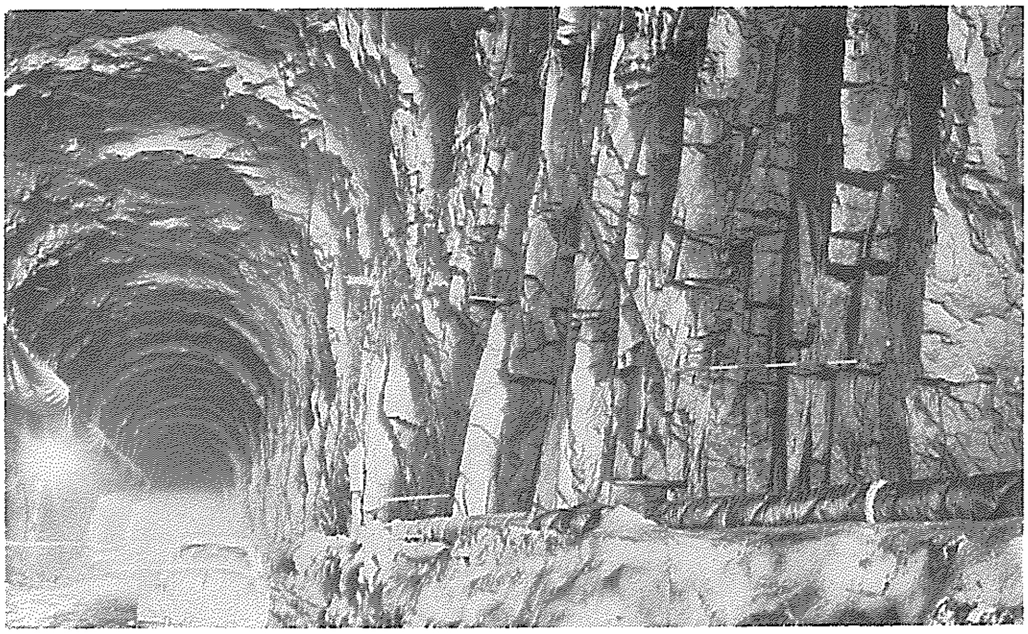
11. RQD = 70% across joints, measured; RQD_v = 90%; RQD_a = 80%

12. ----

13. ----

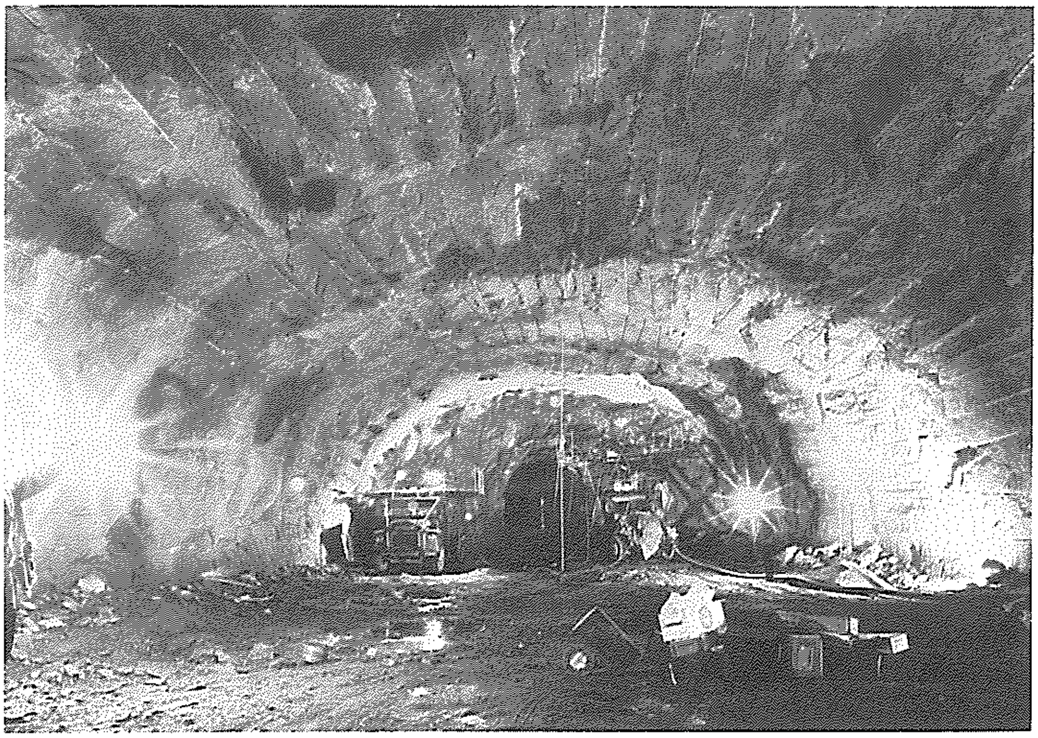
14. overthrusting and vertical shearing in area

15. no water



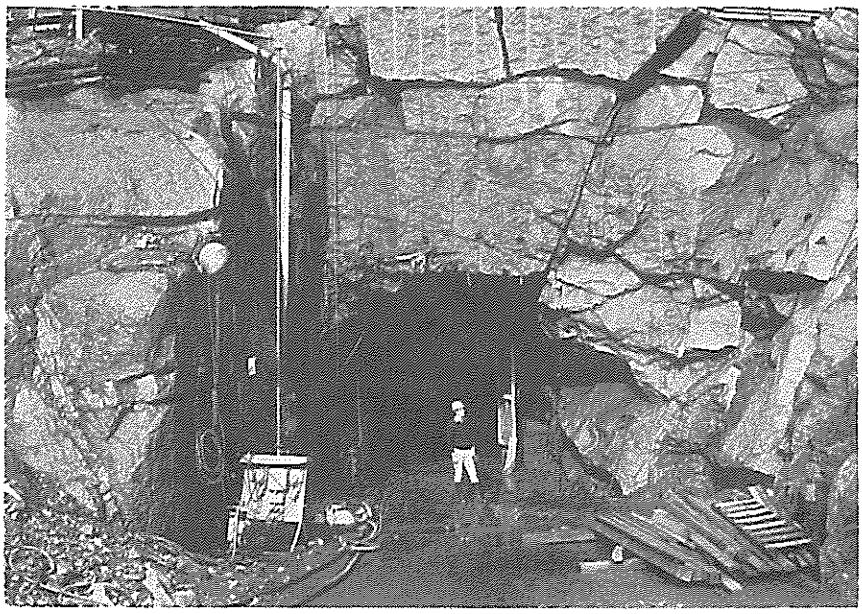
Case 21

1. same as Case 1
2. machine hall
3. W = 13m
4. H = 11m
5. A = $130m^2$
6. stable, no overbreak, no roof falls; (A)
7. L = 40m
8. massive granite, a few widely spaced vertical joints
9. none
10. D = 170m
11. RQD = 100% all directions, observed
12. ----
13. ----
14. same as Case 1
15. minor inflows (<5 lit/min) along some open joints very serious in spite of small flows, as water leaked through gunite roof coating and had to be collected and drained to protect machinery



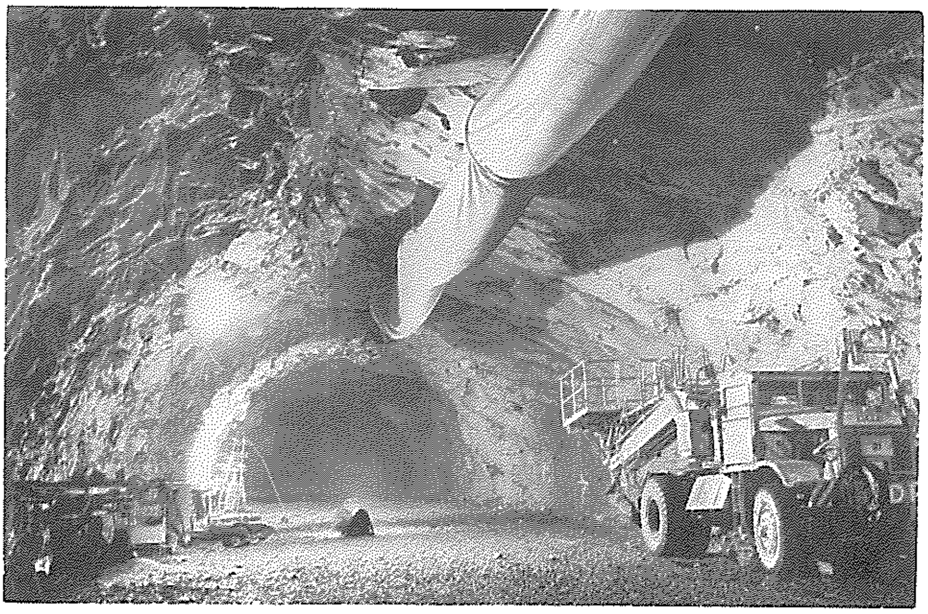
Case 22

1. same as Case 1
2. access tunnel, portal area
3. W = 4m
4. H = 3m
5. A = $12m^2$
6. block falls near portal (see photo); (E)
7. L = 15m
8. open, sheeting joints in granite intersected by a few steeply dipping open joints
9. rock bolts
10. D = 5-10m
11. RQD = 100% all directions, estimated
12. ----
13. ----
14. same as Case 1
15. minor flows along bedrock surface



Case 23

1. Vietas Hydroelectric, northern Sweden
2. heading for headrace tunnel, Suorva line
3. W = 12.5m
4. H = 6.5m
5. A = 68m^2
6. wedge-shaped roof fall (see photo); (F)
7. L = 50m
8. shear zone in mylonite; crushed rock and clay joint fillings in zone; intersecting 30° dipping joints
9. rock bolts, wire mesh, shotcrete
10. D = 150m
11. RQD = 30% across shear zone, estimated; RQD_v = 0-80%; RQD_a = 0%
12. ----
13. ----
14. tunnel lies in a major overthrust sheet (see Fig. 2.6)
15. very minor inflows



Case 24

1-5. same as Case 23

6. wedge-shaped roof fall (see sketch); (F)

7. L = 60m

8. one meter-wide shear zone in mylonite; crushed mylonite and clay seams and joint fillings; shear zone intersected by flat-lying joint set

9. rock bolts, shotcrete, wire mesh

10. D = 60m

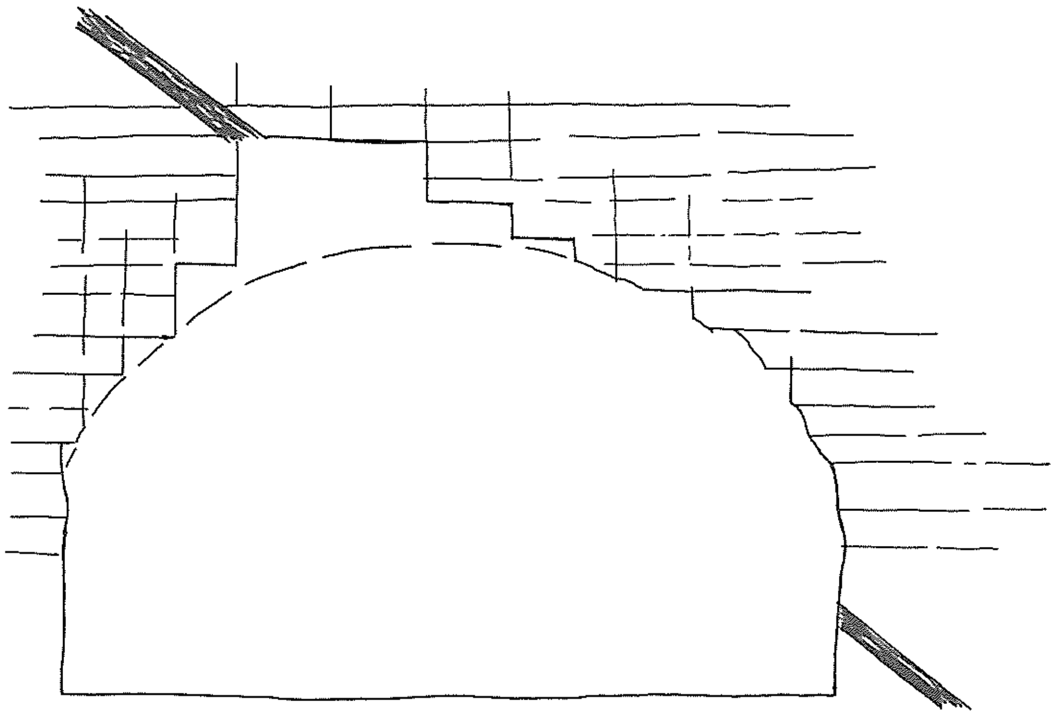
11. RQD = 60% across shear zone, estimated; RQD_v = 0-80%; RQD_a = 0-100%

12. ----

13. ----

14. same as Case 23

15. minor inflows (<3 lit/min)



Case 25

1-5. same as Case 23

6. large overbreak in intrados, some large roof falls (see photo); (F)

7. L = 100m

8. closely spaced horizontal joint set in mylonite, intersected by widely spaced vertical joints; some blast damage and breakage across intact rock

9. rock bolts, shotcrete

10. D = 80m

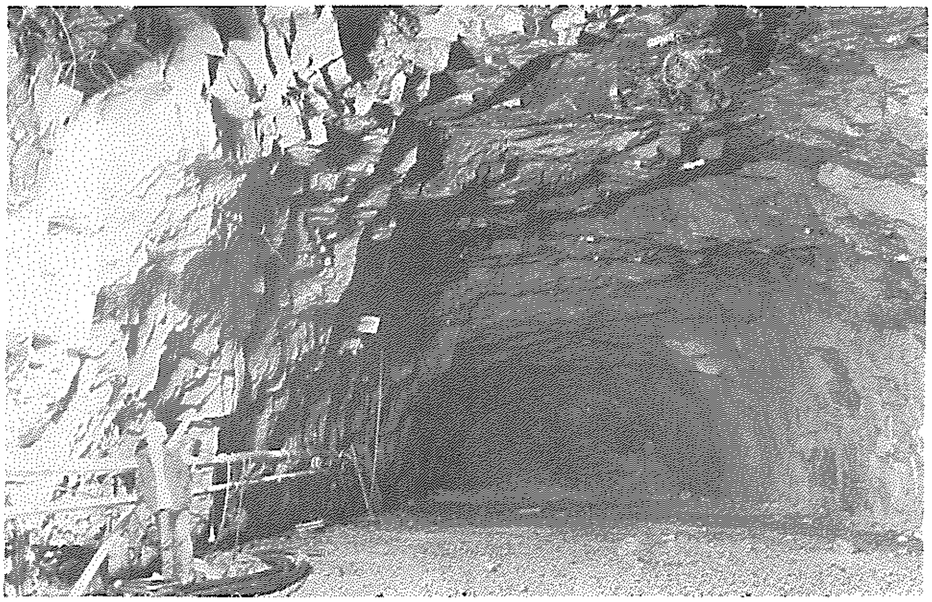
11. RQD = 70% up walls, measured; RQD_a = 10-90%

12. ----

13. ----

14. same as Case 23

15. no water



Case 26

1-5. same as Case 23

6. large overbreak in roof and walls, some roof falls (see photo); (G)

7. L = 50m

8. intersecting vertical and 30-50° dipping joint sets (see photo); mylonite (metamorphosed granite)

9. rock bolting, eventually shotcrete

10. D = 60m

11. RQD = 80% along wall, estimated; RQD_v = 40-100%

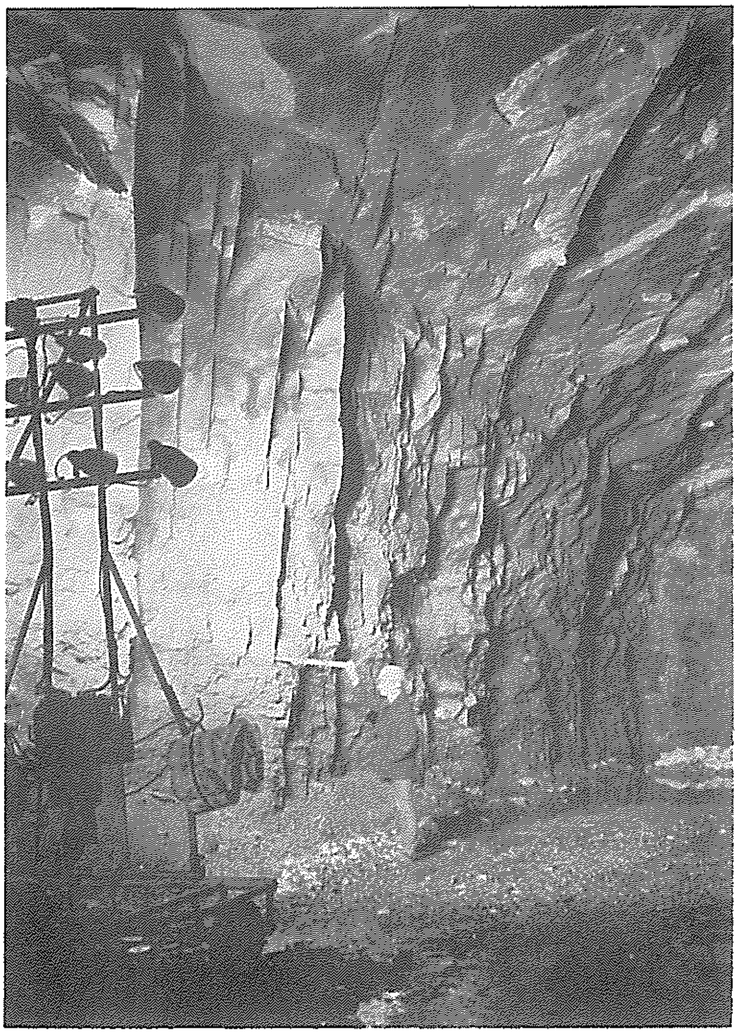
12. ----

13. ----

14. same as Case 23

15. insignificant inflows

16. blasting has opened up small block structure in rock that may be healed jointing or cleavage



Case 27

1-5. same as Case 23

6. large overbreak, but no roof falls; (B)

7. L = 200m

8. intersecting diagonal joint set and flat-lying schistosity planes; schist (see Fig. 2.10)

9. none

10. D = 90m

11. RQD = 90% up walls, estimated; RQD_a = 50-100%

12. ----

13. ----

14. tunnel lies in a major overthrust sheet (see Fig. 2.6)

15. no inflows

Case 28

1-5. same as Case 23

6. wedge-shaped roof fall, moderate overbreak in entire section (see sketch); (F)

7. L = 100m

8. smooth, undulating, slickensided, graphite-coated shear surfaces intersecting schistosity; some slickensided schistosity planes; schist

9. rock bolts, shotcrete

10. D = 100m

11. RQD = 40% up wall, estimated; RQD_a = 0-80%

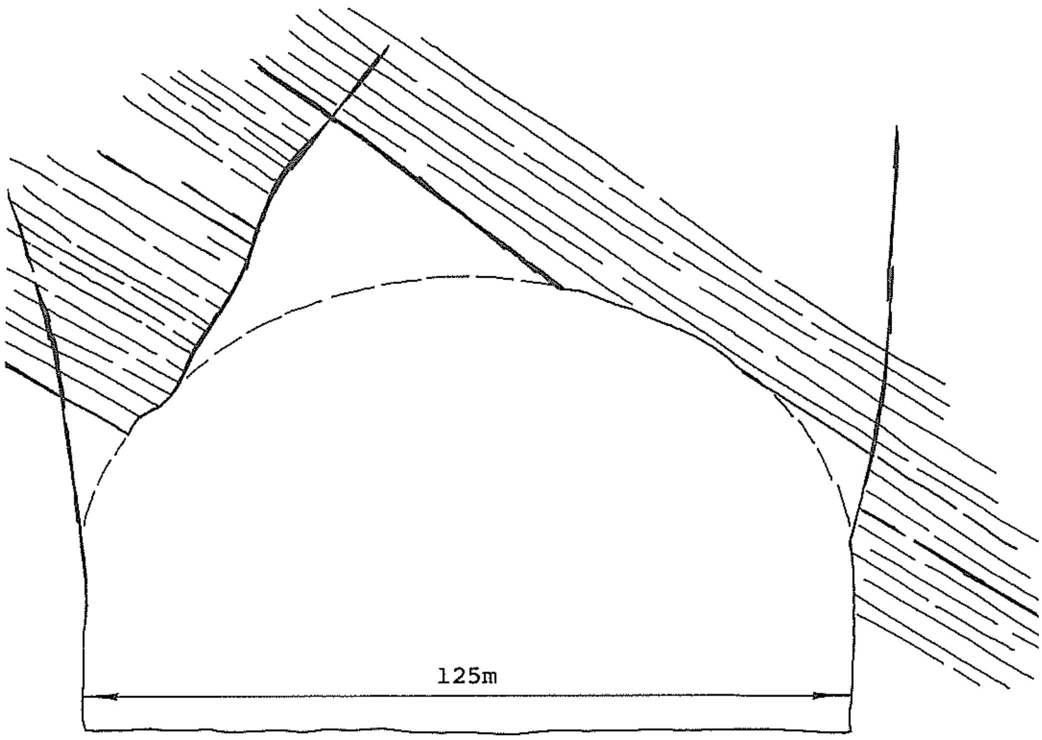
12. ----

13. ----

14. same as Case 27

15. no inflows

16. smooth wall blasting not very effective in this rock



Case 29

1-5. same as Case 23

6. stable, moderate overbreak; (B)

7. L = 200m

8. massive schist, one discontinuous joint set and occasional continuous random joints

9. none

10. D = 80m

11. RQD = 90% all directions, estimated

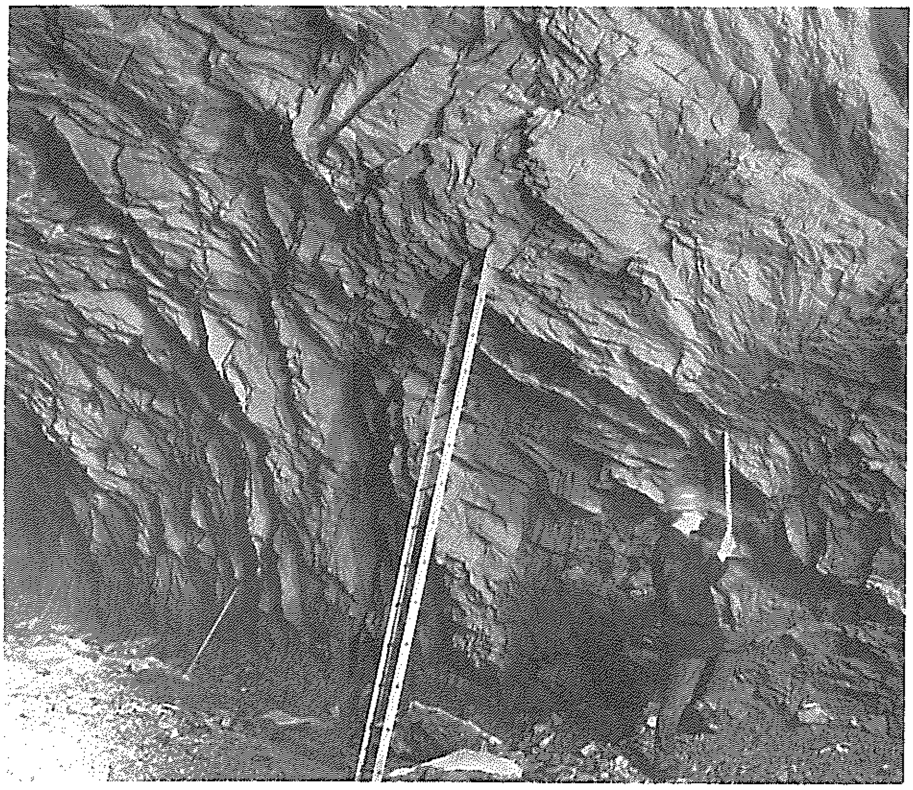
12. ----

13. ----

14. same as Case 27

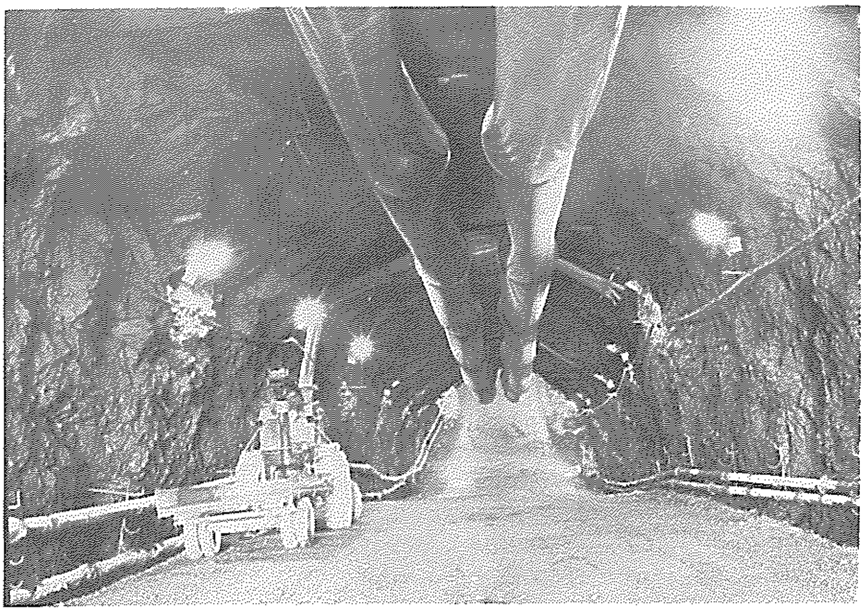
15. no water

16. RQD determinations difficult because of schistosity



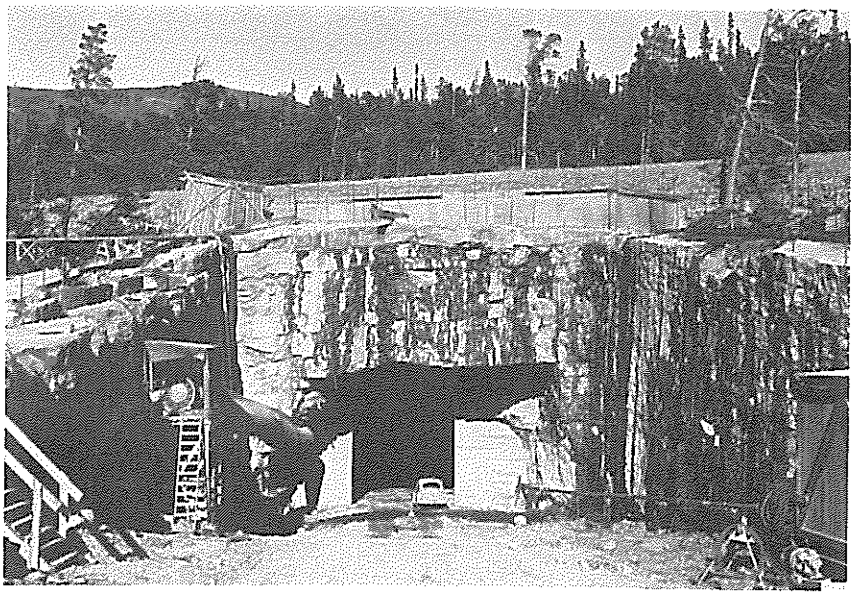
Case 30

1. same as Case 23
2. access tunnel, near portal
3. W = 15m
4. H = 9m
5. A = $110m^2$
6. roof falls up to $20m^3$ (see photo); (G)
7. L = 50m
8. thinly laminated schist, very loose structure
9. rock bolts, eventually shotcrete
10. D = 10-50m
11. RQD = 10% up wall, estimated; RQD_a = 0-20%
12. ----
13. ----
14. same as Case 27; actual overthrust zone is in roof at portal, only 5m above roof in photo
15. minor water inflows



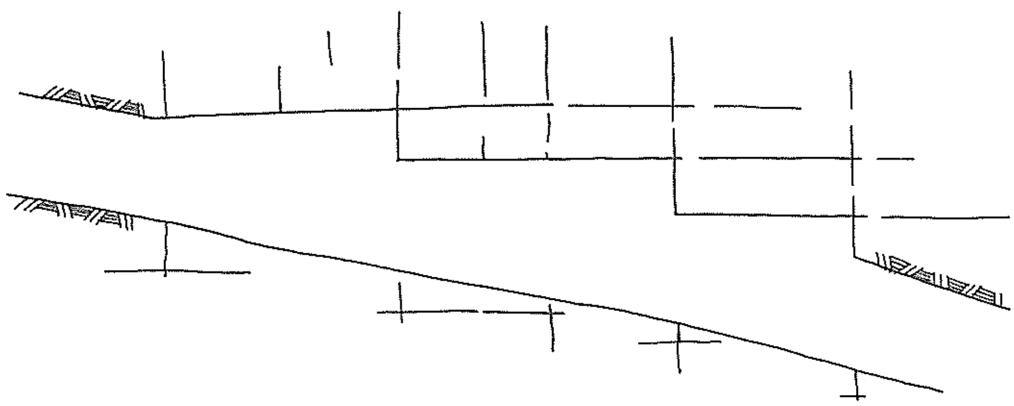
Case 31

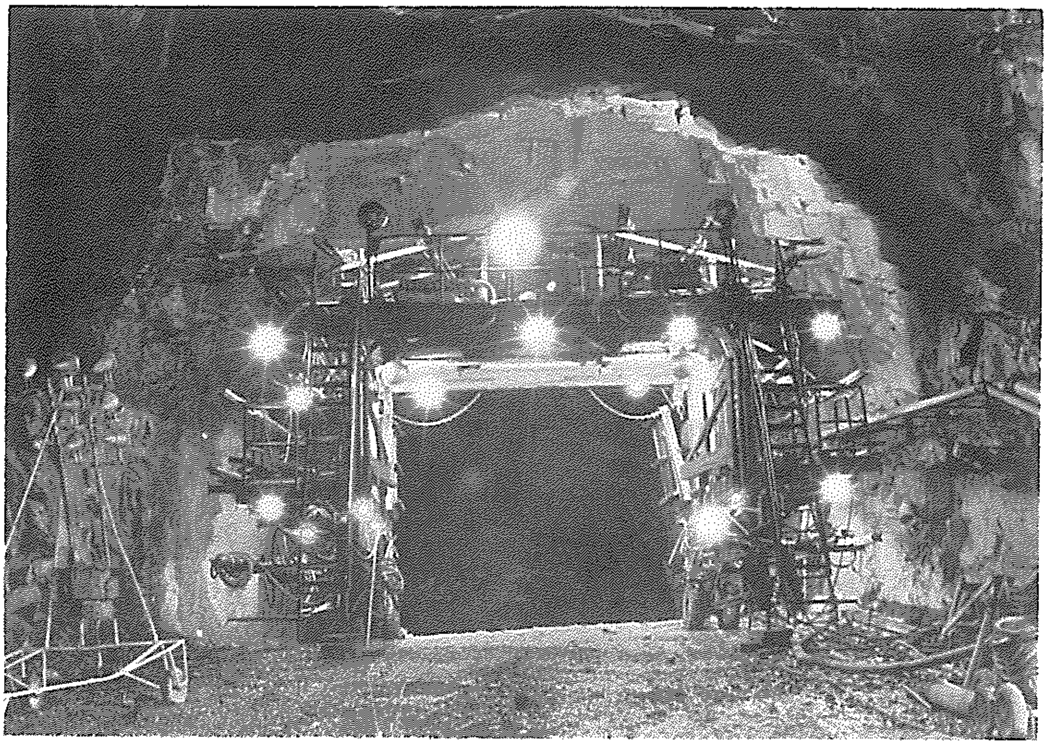
1. same as Case 23
2. heading for tailrace tunnel
3. W = 12m
4. H = 9m
5. A = $100m^2$
6. moderate overbreak in intrados; (E)
7. L = 100m
8. intersecting horizontal sheeting joints and widely spaced vertical joints in quartzite
9. rock bolting
10. D = 10-30m
11. RQD = 100% all directions, observed
12. ----
13. ----
14. tunnel located beneath major overthrust mountain range (see Fig. 2.6)
15. minor inflows

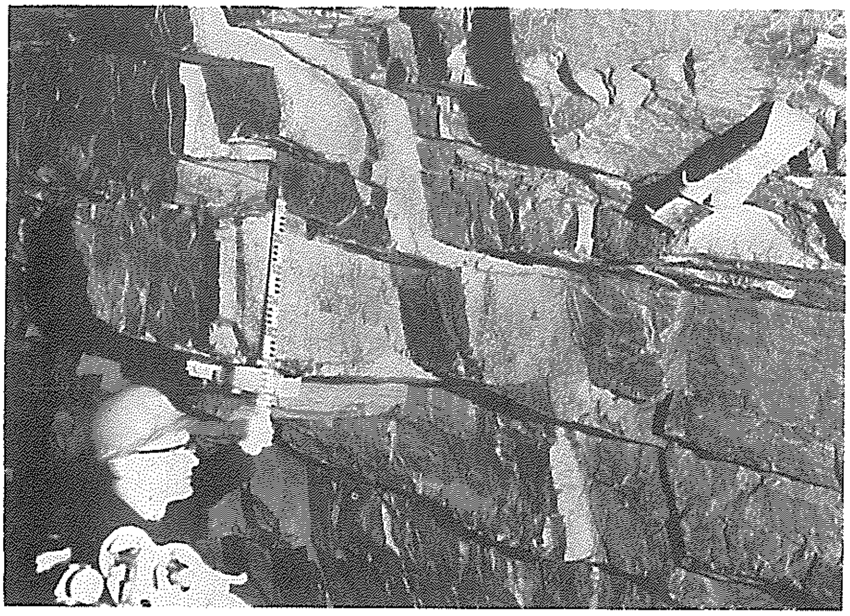


Case 32

1. same as Case 23
2. headrace tunnel, Satisjaure line
3. $W = 12.5m$
4. $H = 7.5m$
5. $A = 80m^2$
6. large overbreak above springline (see photos); (E)
7. $L = 100m$
8. flat-lying overthrust joints intersect two sets of vertical joints, structure is large to small blocky; quartzite
9. rock bolts
10. $D = 50-80m$
11. RQD = 80% up wall, measured; $RQD_a = 90-100\%$
12. ----
13. ----
14. same as Case 31; tunnel located very near a prominent thrust fault
15. no water
16. most of roof fallout caused by tunnel orientation with respect to joint structure (see sketch)







Case 32

Case 33

1-5. same as Case 32

6. large overbreak and large roof falls, tunnel area increased by up to 50 percent because of falls and overbreak; (G)

7. L = 100m

8. very strongly fractured quartzite, prominent horizontal overthrust shear zones and joints; clay-filled seams up to 3cm thick; many clay-filled joints.

9. rock bolts, two shotcrete applications

10. D = 50-80m

11. RQD = 40% up wall, measured; RQD_a = 0-90%

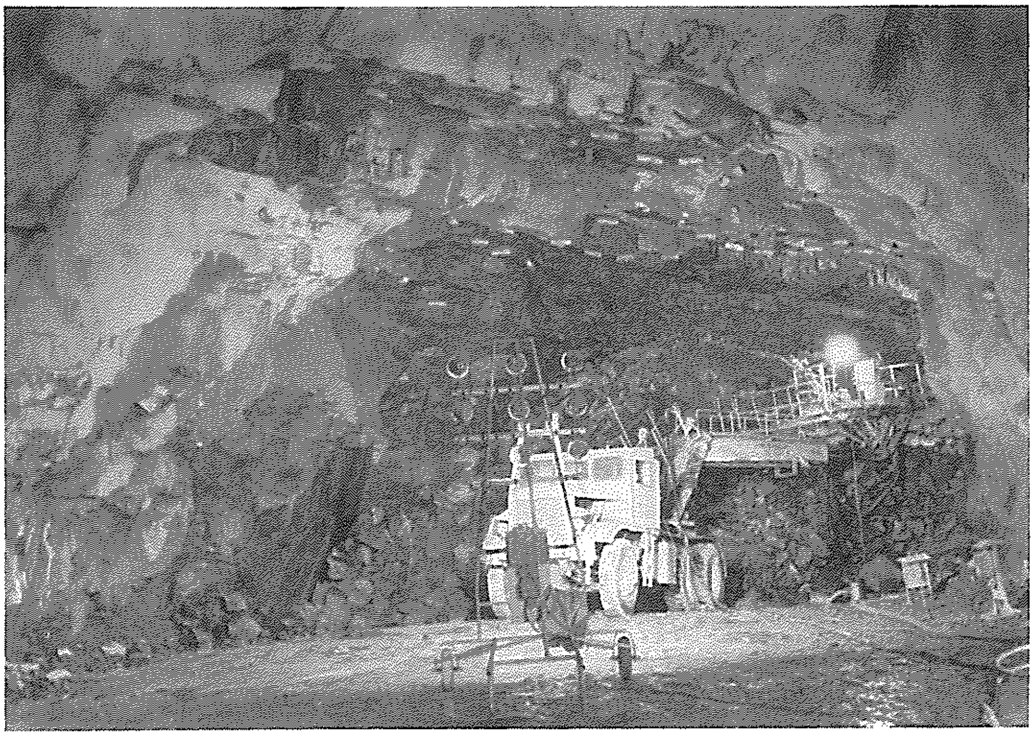
12. ----

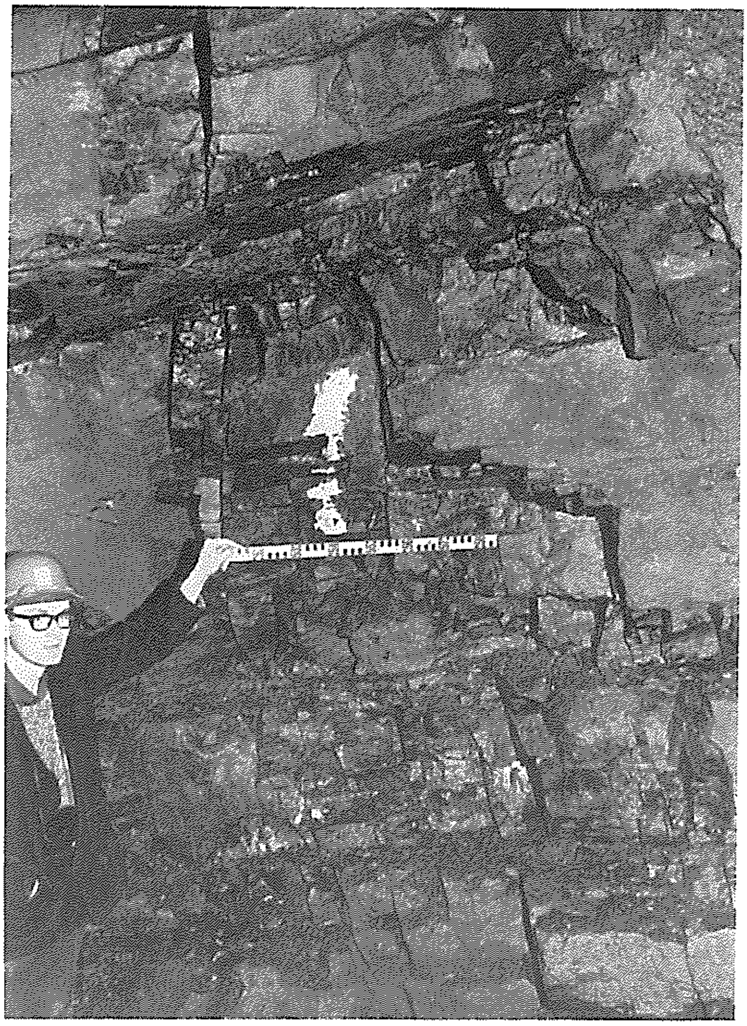
13. ----

14. location in a prominent overthrust shear zone, just under two major overthrust sheets (see Fig. 2.6)

15. minor water

16. bolting and shotcreting before mucking out





Case 33 Closeup of Rock

Case 34

1. same as Case 23
2. drifts in machine hall area
3. W = 5m
4. H = 5m
5. A = $25m^2$
6. fallout at corner of intersection (see Fig. 2.15); (H)
7. L = 5m
8. two intersecting joint sets in quartzite
9. rock bolts before and after failure
10. D = 70m
11. RQD = 90% along wall, measured; RQD_v = 90%
12. ----
13. ----
14. same as Case 32
15. no water

Case 35

1-5. same as Case 34

6. stable section, minor overbreak, no falls; (A)

7. L = 10m

8. strongly sheared quartzite; vertically oriented joints (see photo); very tight structure

9. none

10. D = 110m

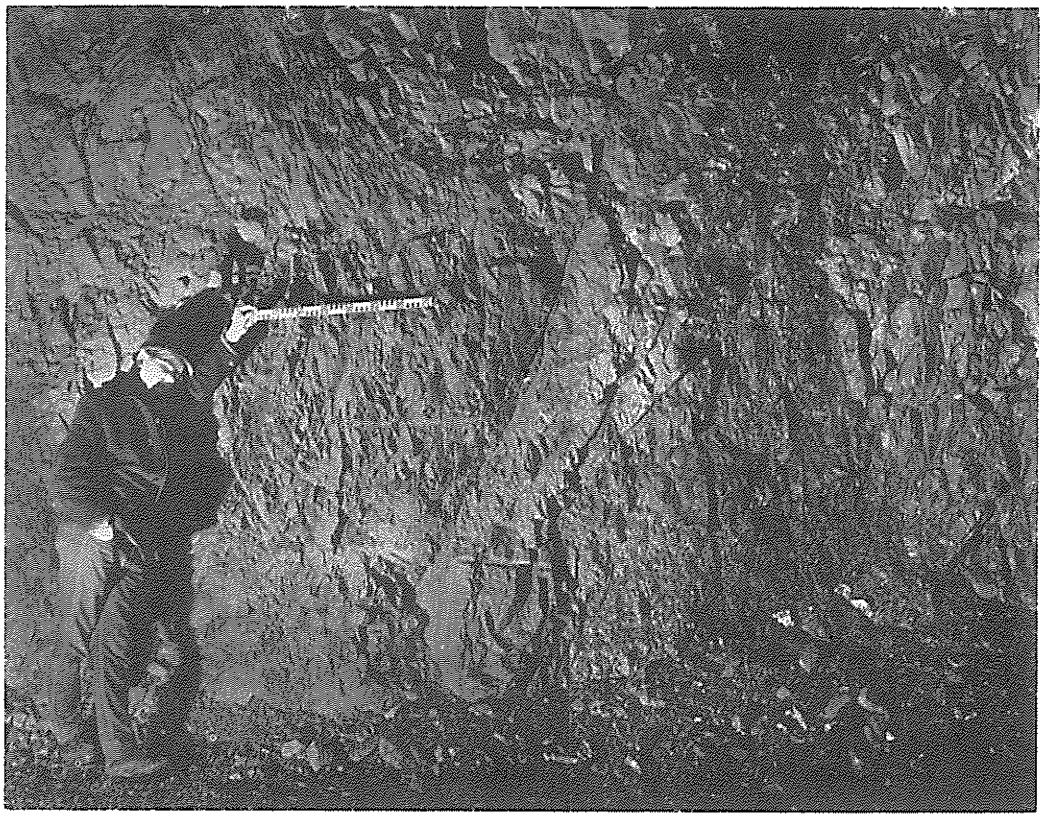
11. RQD = 0% along wall, measured; RQD_v = 0%

12. ----

13. ----

14. same as Case 32

15. minor water inflows



Case 36

1-5. same as Case 34

6. stable section; minor overbreak (see Fig. 2.8); (A)

7. L = 10m

8. strongly fractured quartzite; same shear zone as in Case 35

9. none

10. D = 60m

11. RQD = 20% along wall, measured; RQD_v = 20-80%

12. ----

13. ----

14. same as Case 32

15. no water

Case 37

1-5. same as Case 34

6. vault-shaped crown formed by progressive roof falls (see Fig. 2.13); (E)

7. L = 20m

8. set of vertical joints in quartzite intersected by several random clay-filled joints and a set of closely spaced diagonal joints

9. rock bolts, wire mesh, two shotcrete applications

10. D = 110m

11. RQD = 20% along wall, estimated; RQD_v = 20-40%

12. ----

13. ----

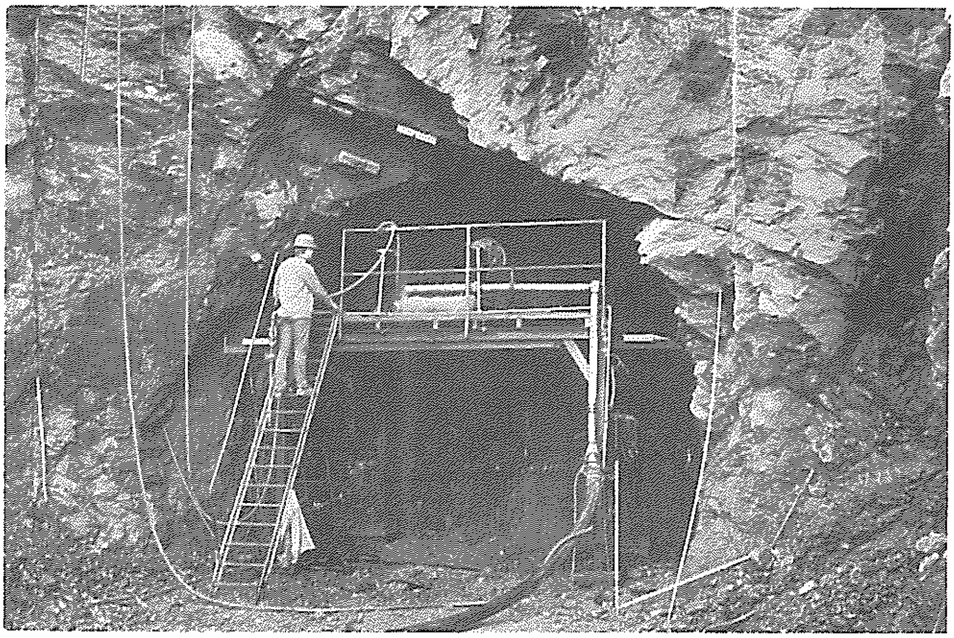
14. same as Case 32

15. minor water inflows

16. location only 20m from Case 35

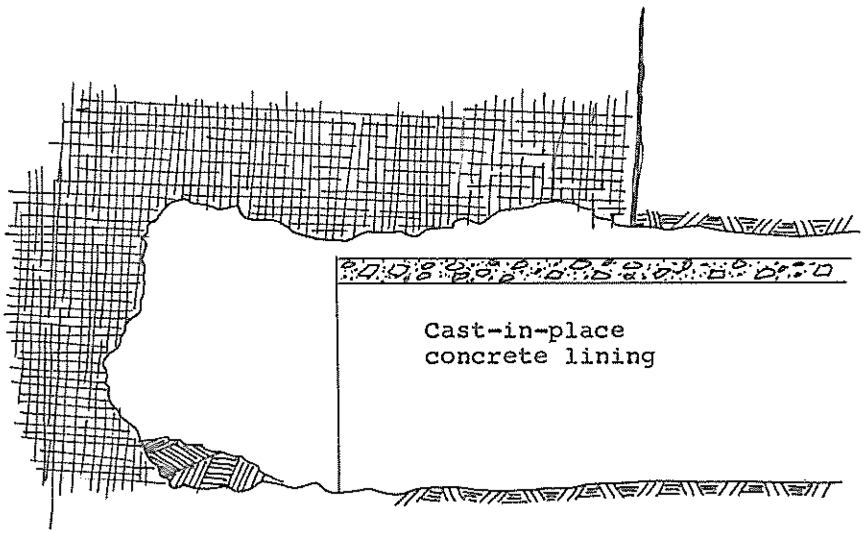
Case 38

1. Vietas Hydroelectric, northern Sweden
2. access tunnel near portal
3. $W = 5m$
4. $H = 5m$
5. $A = 20m^2$
6. roof falls at portal (see photo); (G)
7. $L = 5m$
8. intersection of bedding planes and vertical joints in dolomite
9. rock bolts
10. $D = 10m$
11. RQD = 60% up wall, estimated; RQD_a = 0-100%
12. ----
13. ----
14. same as Case 27
15. no water
16. RQD determinations difficult because of healed bedding in dolomite



Case 39

1. Rendal Hydroelectric, central Norway
2. headrace tunnel
3. W = 8m
4. H = 6m
5. A = $43m^2$
6. major roof falls, progressive formation of dome- and vault-shaped crown; falls from face (see sketch); (G)
7. L = 50m
8. shear zone in quartzite; "sugar cube" rock structure
9. cast concrete arch immediately after mucking
10. D = 200m
11. RQD = 20% all directions, estimated
12. ----
13. ----
14. major regional normal faulting is responsible for horst-graben topography in area
15. 5-10 lit/min water inflows



Case 40

1-5. same as Case 39

6. large overbreak and roof falls; (D)

7. L = 150m

8. narrow (<10cm) vertical shear zones in metamorphosed arkose that contain montmorillonitic clay; surrounding rock is blocky

9. temporary support with shotcrete, permanent support with cast concrete arches

10. D = 200m

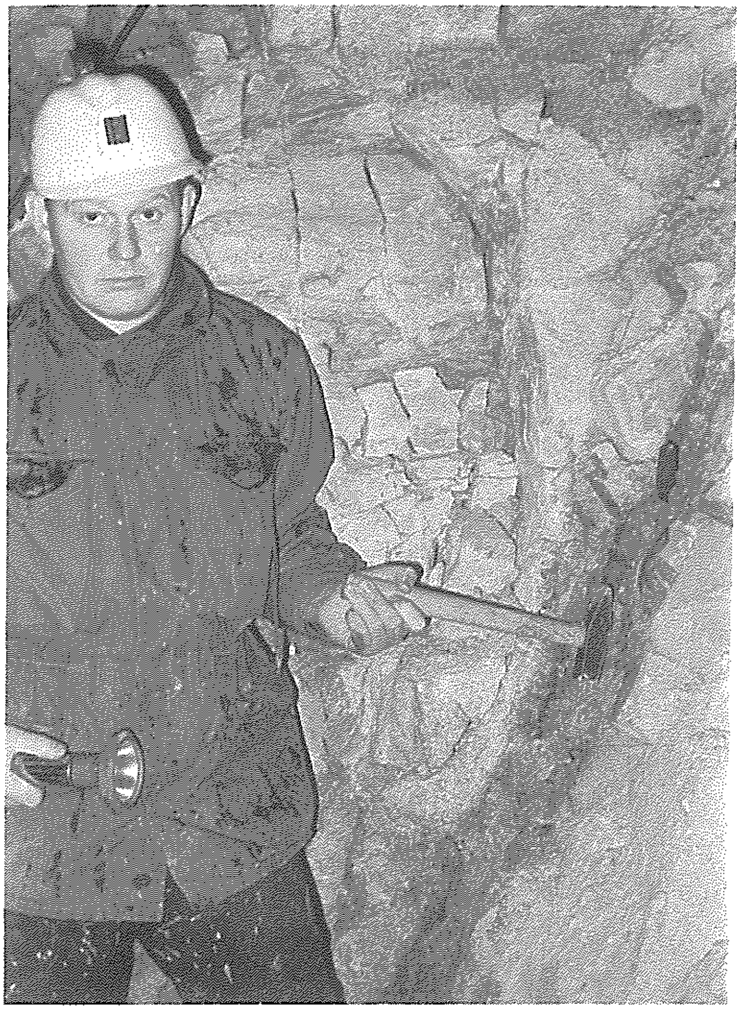
11. RQD = 50% along wall, measured; RQD_v = 40-100%

12. ----

13. ----

14. same as Case 39

15. minor inflows have led to swelling of clay



Case 41

1-5. same as Case 39

6. progressive loosening of walls and crown caused by swelling clay; (H)

7. L = 50m

8. horizontal bedding planes in metamorphosed arkose partially filled with montmorillonite swelling clay; some vertical joints also clay-filled and coated (see photo)

9. shotcrete failure in adhesion to clay-coated rock surface; permanent support with cast concrete arches

10. D = 150m

11. RQD = 60% up wall, estimated; RQD_a = 10-90%

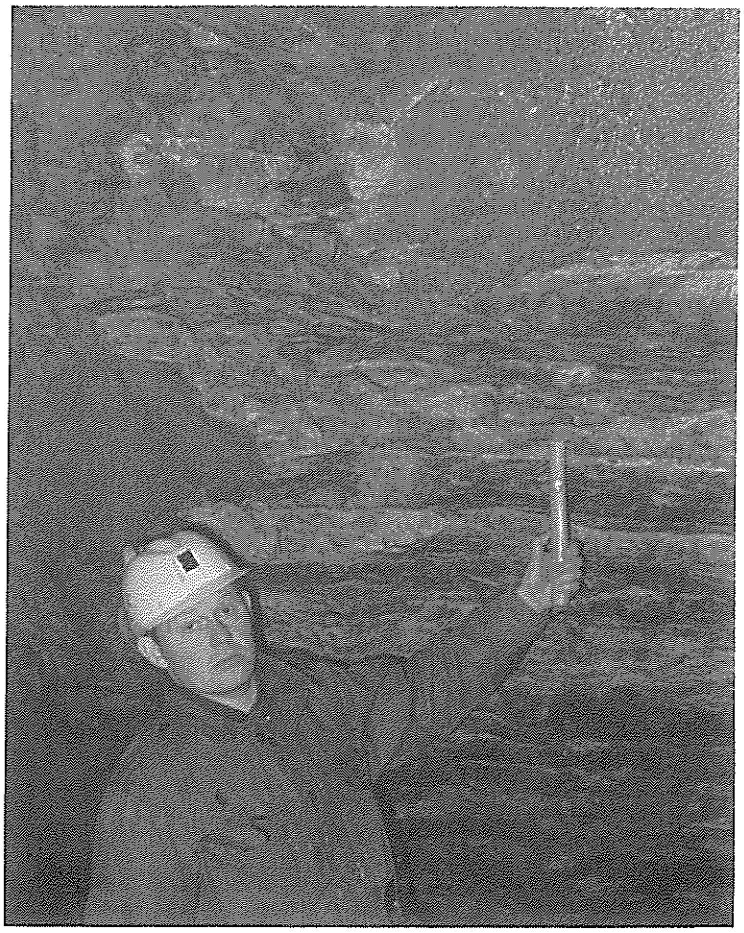
12. ----

13. ----

14. same as Case 39

15. about 5 lit/min inflow

16. white material in photo to which shotcrete does not adhere is clay



Case 42

1-5. same as Case 39

6. roof falls; progressive loosening in crown, walls, and face as montmorillonitic clay seams absorb water and swell; (E)

7. L = 80m

8. vertical and horizontal clay-filled fissures in the vicinity of a vertical shear zone in metamorphosed arkose; similar to Case 40

9. temporary support with one shotcrete application ; permanent support with cast concrete arches; experimentation with two to three shotcrete applications for permanent support.

10. D = 200m

11. RQD = 40% along wall, estimated; RQD_V = 0-90%

12. ----

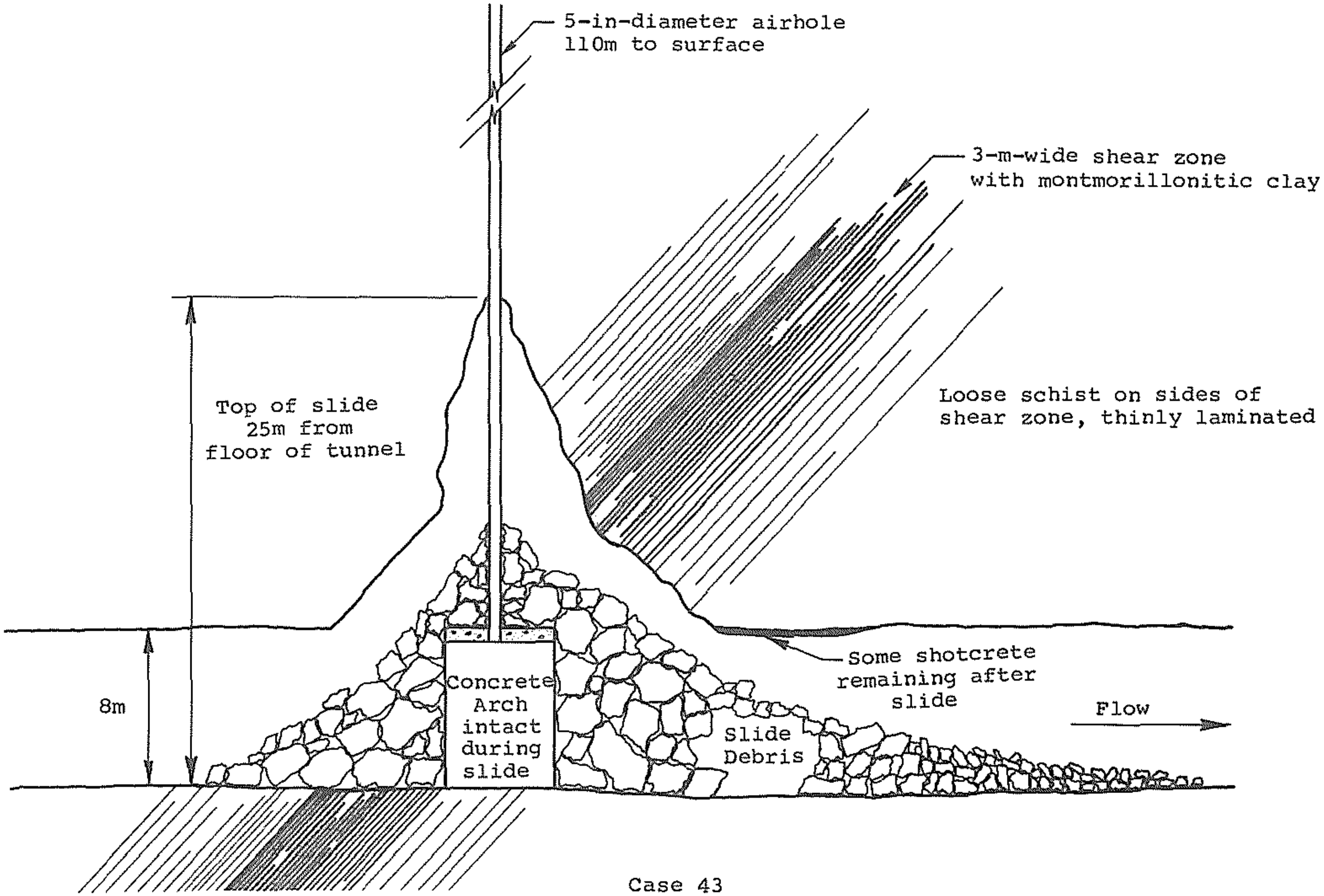
13. ----

14. same as Case 39

15. up to 5 lit/min inflow

Case 43

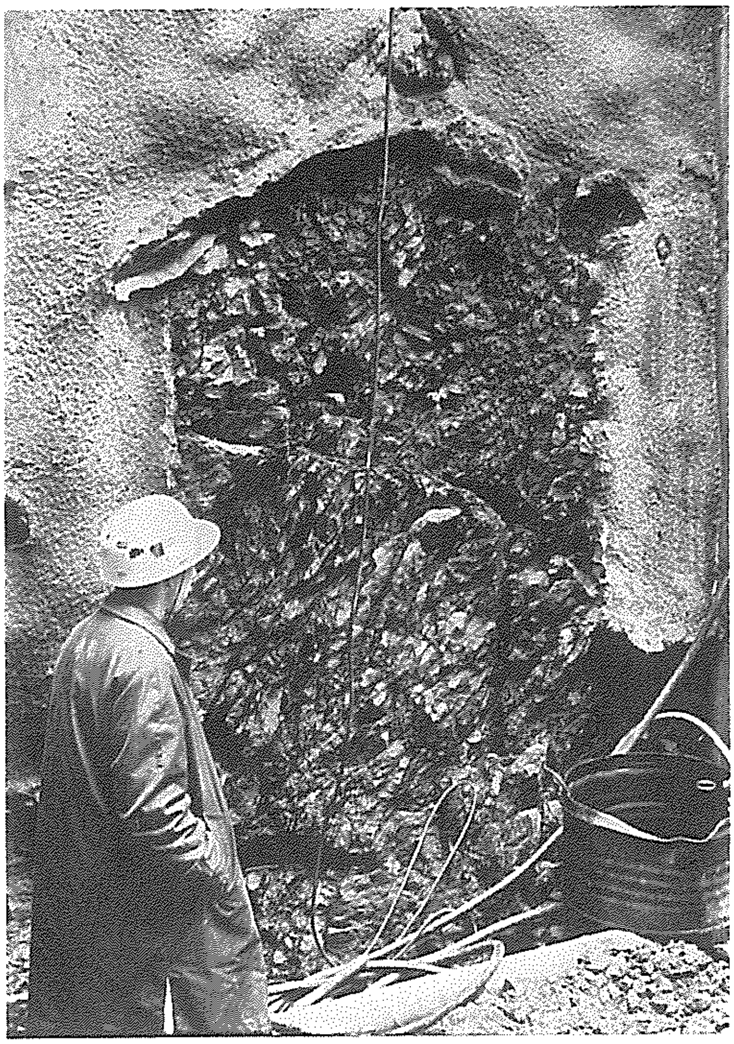
1. Sällsjö Hydroelectric, northern Sweden
2. tailrace tunnel
3. W = 9m
4. H = 8m
5. A = 64m^2
6. complete collapse of tunnel during operation of power plant (see sketch); vault-shaped crown opening; (H)
7. L = 25m
8. three-meter-wide shear zone in thinly-laminated schist; swelling montmorillonitic clay seam in shear zone, some chlorite joint coatings
9. original thin (6-8cm) shotcrete failed; permanent support after failure with cast concrete arches
10. D = 110m
11. RQD = 20% all directions, estimated
12. ----
13. ----
14. tunnel located in an overthrust sheet
15. ground water seepage into zone along a cased de-air hole may have contributed to swelling of clay
16. economically the most catastrophic case


5-in-diameter airhole
110m to surface
3-m-wide shear zone
with montmorillonitic clay
Top of slide
25m from
floor of tunnel
Loose schist on sides of
shear zone, thinly laminated
Some shotcrete
remaining after
slide
Concrete
Arch
intact
during
slide
8m
Slide
Debris
Flow

Case 43

Case 44

1-5. same as Case 43, near portal
6. very small overbreak; some small roof falls; (C)
7. L = 100m
8. thinly-laminated schist, similar to that shown in photo
9. one shotcrete application
10. D = 30m
11. RQD = 0% all directions, observed
12. ----
13. ----
14. same as Case 43
15. insignificant inflows



Case 45

1-5. same as Case 43

6. very large risk for large roof falls, possible collapse of opening. Stability due only to tensile strength across schistosity (see sketch); (D)

7. L = 30m

8. 3-15cm-thick clay-filled shear seam in schist partially washed out; clay is montmorillonitic; some washing out of seam may have occurred during draining of tunnel for repair

9. none prior to tunnel drainage, eventual rock bolts and shotcrete

10. D = 100m

11. RQD = 90% up wall, estimated; RQD_a = 90%

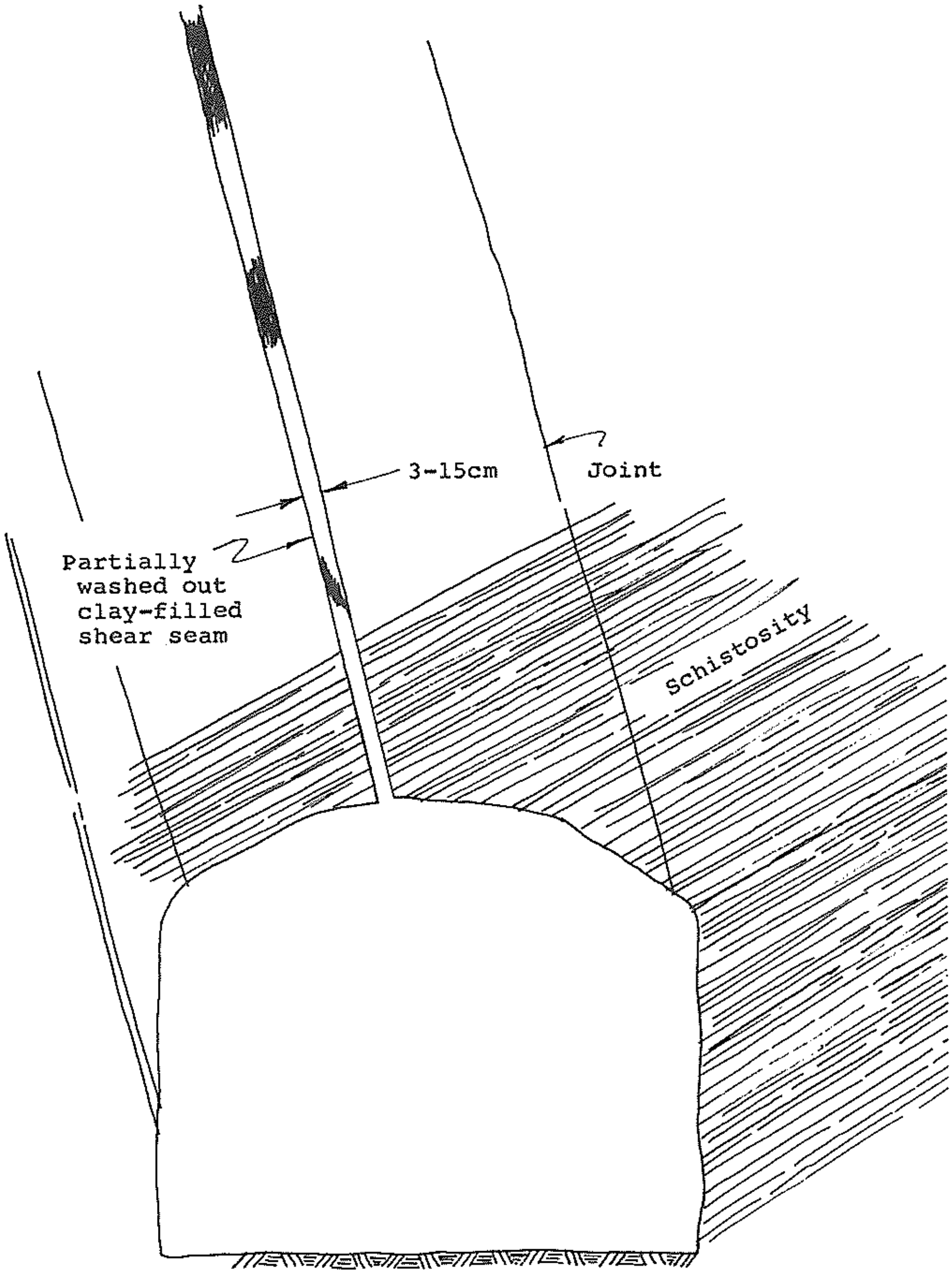
12. ----

13. ----

14. same as Case 43

15. no water

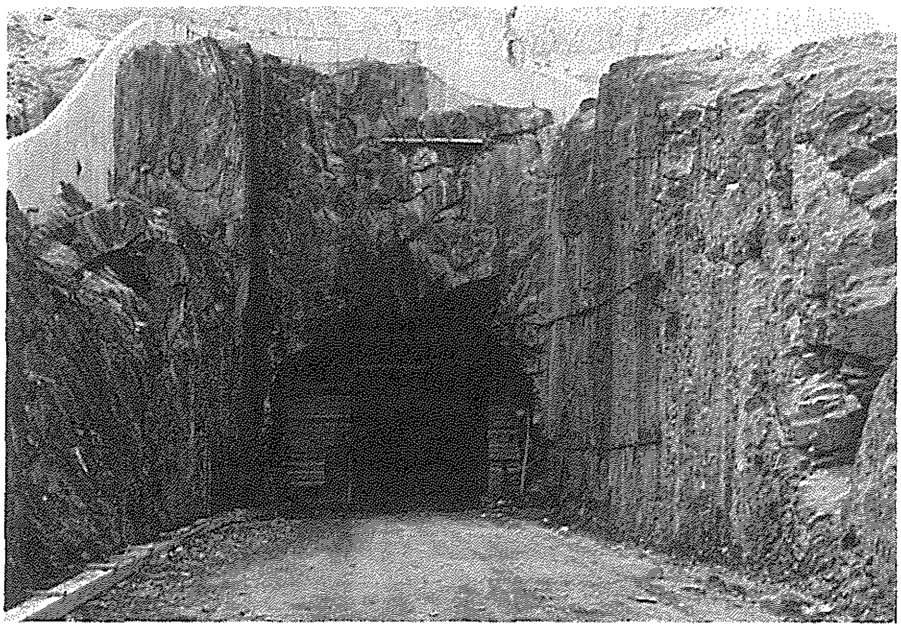
16. RQD estimations difficult because of uncertain cohesion across schistosity


3-15cm
Joint
Partially
washed out
clay-filled
shear seam
Schistosity

Case 45

Case 46

1. Sällsjö Hydroelectric, northern Sweden
2. access tunnel, portal area
3. W = 7m
4. H = 6m
5. A = $35m^2$
6. block falls in roof (see photo); (G)
7. L = 10m
8. intersecting cross joints and schistosity planes in schist (see photo)
9. rock bolts, wire mesh, eventually shotcrete
10. D = 7-10m
11. RQD = 30% across roof, estimated; RQD_v = 0-80%; RQD_a = 30%
12. ----
13. ----
14. same as Case 43
15. minor water



Case 47

1. Bergvattnet Hydroelectric, northern Sweden
2. tailrace tunnel
3. W = 6.5m
4. H = 4.5m
5. A = $30m^2$
6. wedge-shaped roof fall (see Fig. 2.12); (D)
7. L = 15m
8. three-cm-wide softening clay seam in a one-meter-wide overthrust shear zone in thinly laminated schist; clay seam partly washed out; crushed rock on both sides of clay seam
9. rock bolts; wire mesh, shotcrete
10. D = 30m
11. RQD = 0% along wall, measured; RQD_v = 0%
12. (V = 4000m/sec)
13. (SVR = 4000/5400 = 0.74)
14. tunnel is located in an overthrust sheet (see Fig. 2.5)
15. minor water inflows have partially washed out clay and fine materials in shear zone, causing loosening in surrounding rock

Case 48

1-5. same as Case 47

6. wedge-shaped roof fall; similar to Case 47; (D)

7. L = 15m

8. fall occurred at an overthrust shear zone in schist along which exists a 3-cm-thick clay and graphite seam. Shear zone is 50-100cm thick and contains smooth, slickensided, graphite-coated joint surfaces.

9. rock bolts, wire mesh, two shotcrete applications

10. D = 50m

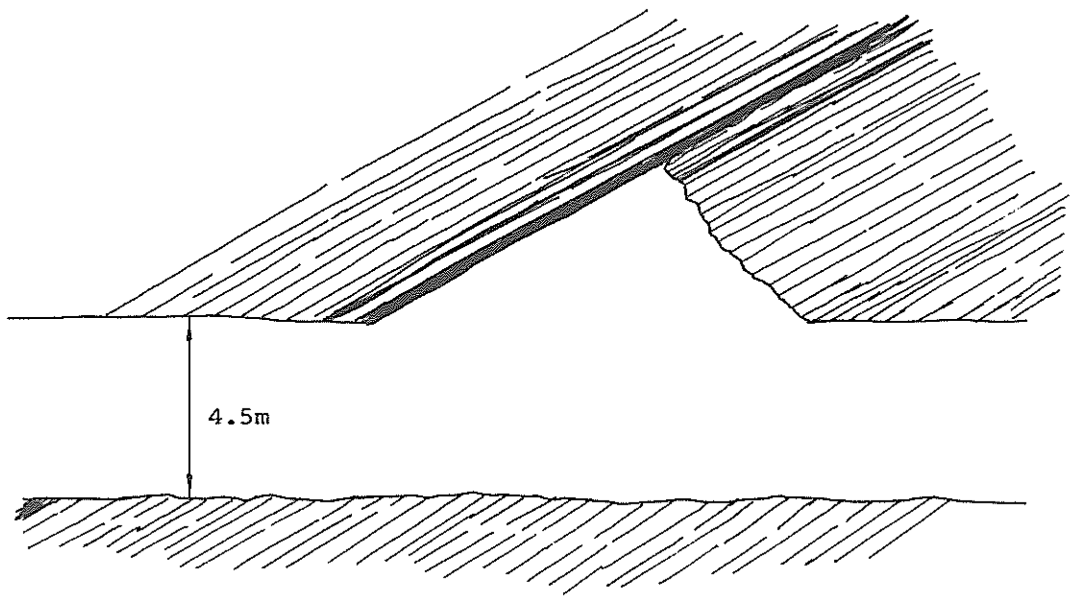
11. RQD = 10% along wall, measured; RQD_V = 10%

12. ----

13. ----

14. same as Case 47

15. insignificant inflows



Case 49

1-5. same as Case 47

6. roof fall and progressive washing out of shear zone materials; see sketch; (F)

7. L = 20m

8. vertical shear zone in schist, crushed rock in a 10cm-wide matrix of softening clay; loose schist surrounding zone

9. two shotcrete applications

10. D = 30m

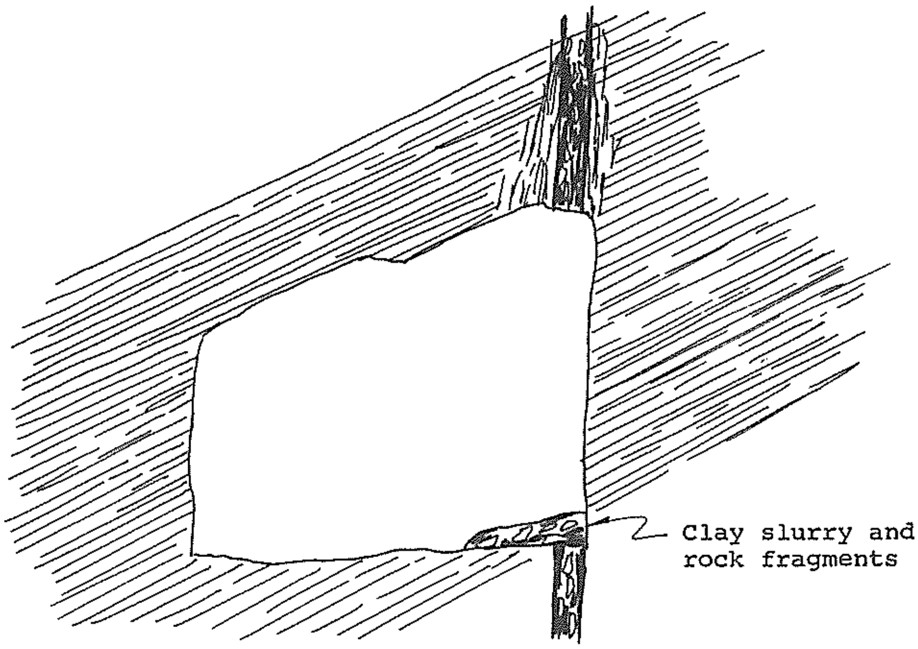
11. RQD = 30% across zone, estimated; RQD_v = 0-50%; RQD_a = 0-50%

12. (V = 4000m/sec)

13. (SVR = 4000/5400 = 0.74)

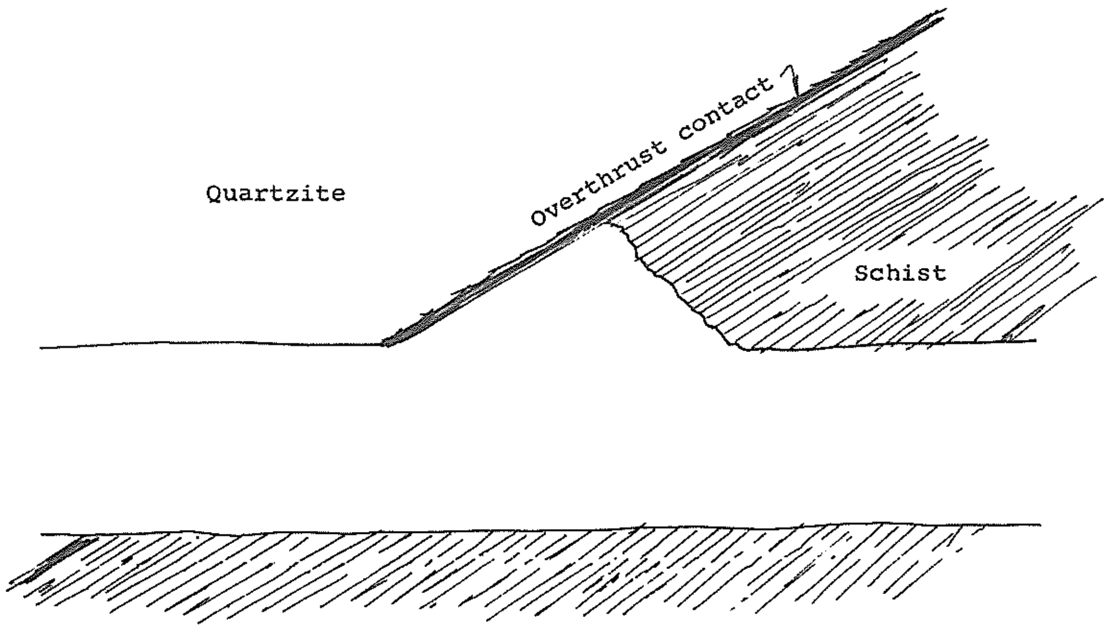
14. same as Case 47

15. same as Case 47



Case 50

1-5.	same as Case 47
6.	wedge-shaped fallout, similar to Case 47, but smaller volume; (D)
7.	L = 15m
8.	overthrust contact between schist and quartzite; clay-coated slickensides in 30cm-wide thrust zone
9.	two shotcrete applications
10.	D = 70m
11.	RQD = 40% along wall, measured; RQD_v = 60%
12.	----
13.	----
14.	same as Case 47; zone is actual overthrust fault
15.	insignificant inflows



Case 51

1-5. same as case 47

6. wedge-shaped roof fall (see sketch); (D)

7. L = 15m

8. very closely jointed metamorphosed claystone, frequent slickensides and clay-coated joint surfaces, loose structure (see sketch)

9. rock bolts, wire net, shotcrete

10. D = 60m

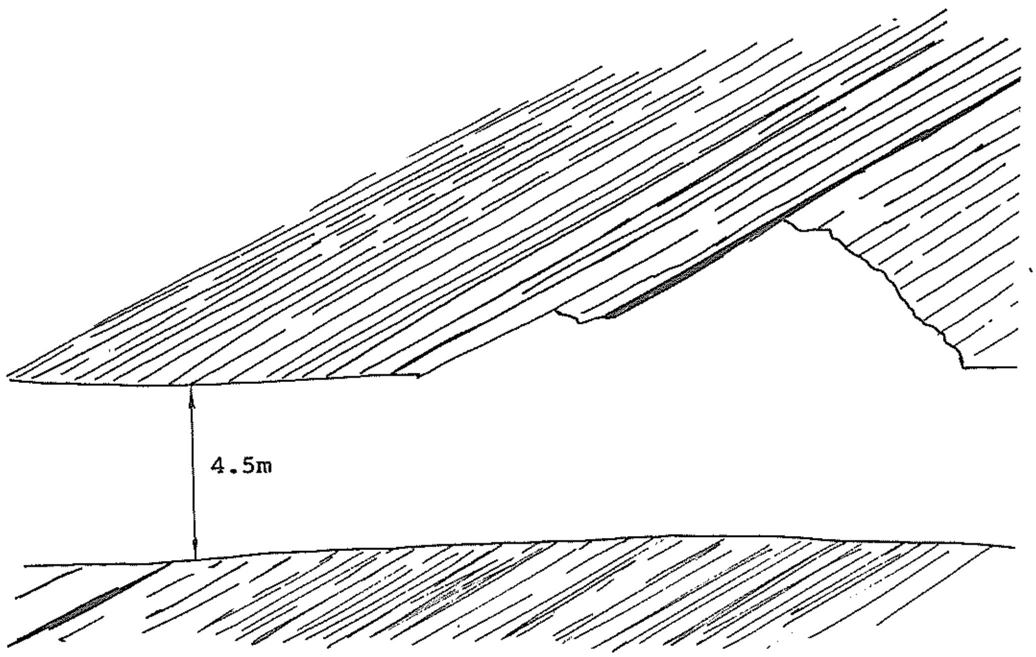
11. RQD = 0% all directions, observed

12. ----

13. ----

14. same as Case 47

15. no water



Case 52

1-5. same as Case 47

6. stable section, minor overbreak, no falls; (A)

7. L = 50m

8. very closely jointed metamorphosed claystone (see photo); similar to rock in Case 51, but contains no slickensides, structure is very tight

9. no structural support required; light shotcrete or gunite for protection against small pieces of falling rock

10. D = 70m

11. RQD = 0% all directions, measured

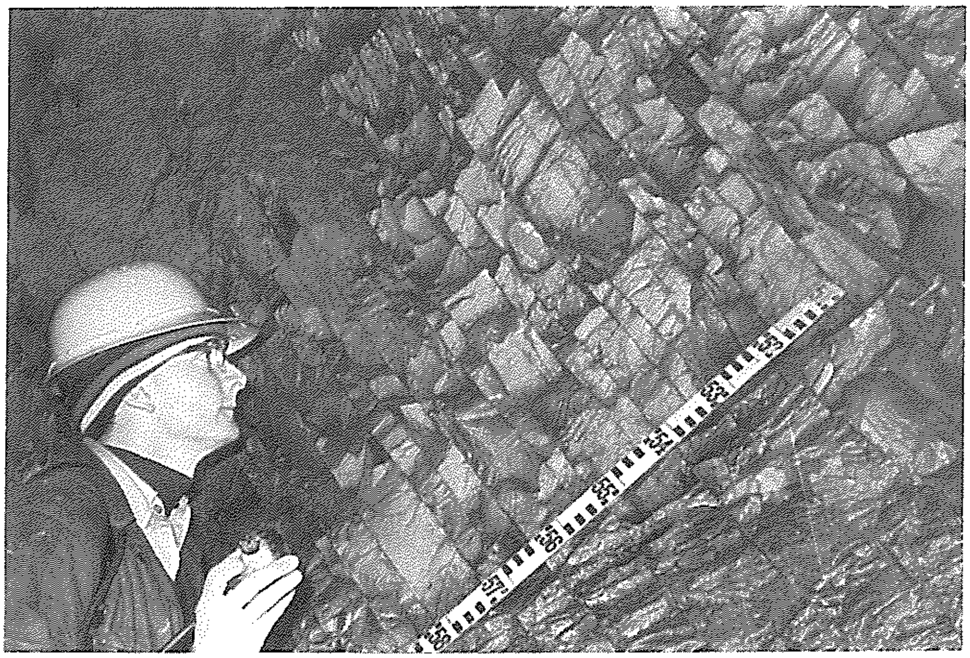
12. ----

13. ----

14. same as Case 47

15. insignificant inflows

16. location about 200m from Case 51



Case 53

1-5. same as Case 47

6. large fallout in roof (see sketch); (D)

7. L = 20m

8. interaction of shear zone and secondary jointing in schist (see sketch)

9. rock bolts, wire mesh, shotcrete

10. D = 60m

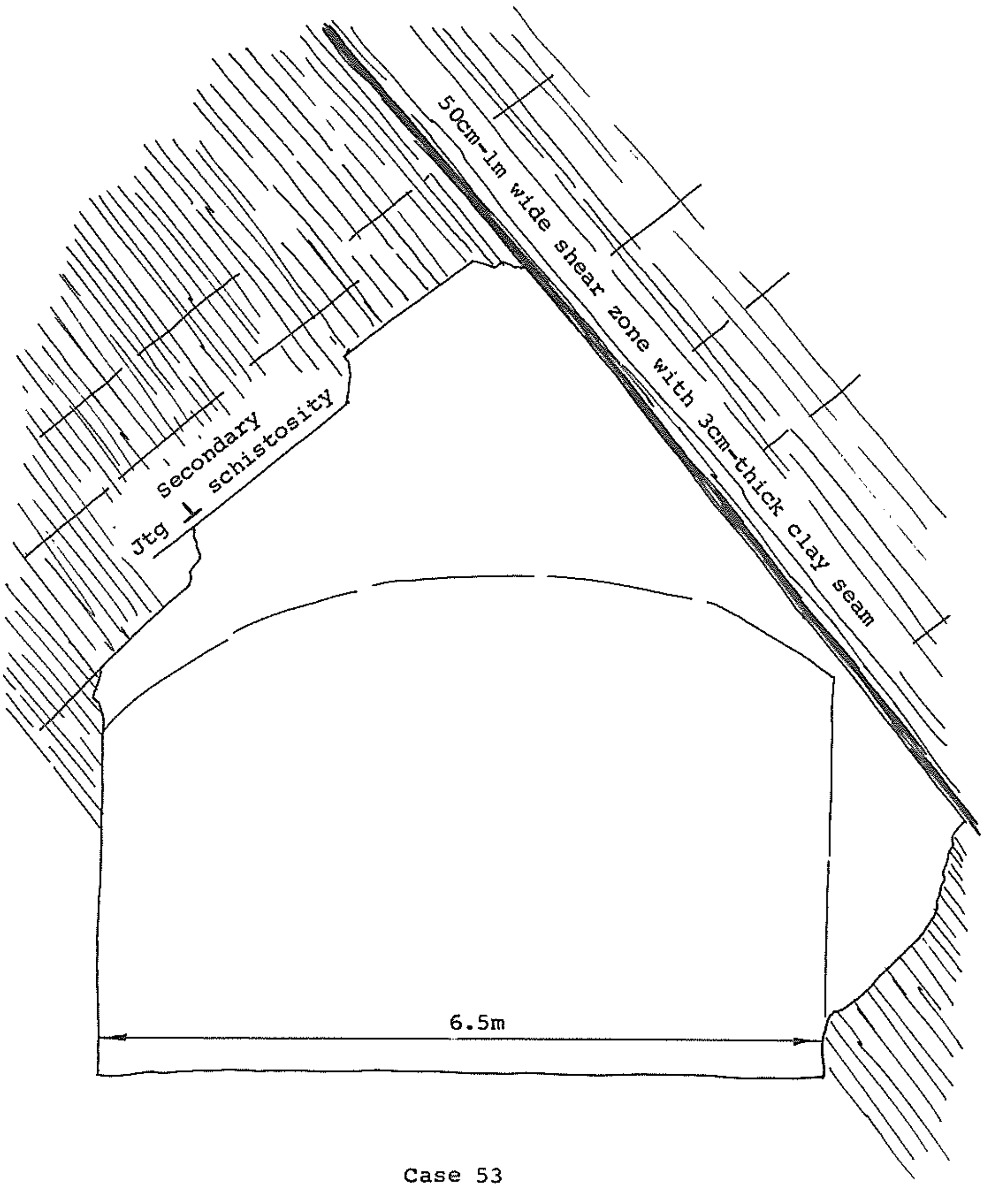
11. RQD = 50% across shear zone, measured; RQD_v = 60%; RQD_a = 60%

12. ----

13. ----

14. same as Case 47

15. no water


50cm-1m wide shear zone with 3cm-thick clay seam
Secondary
Jtg ⊥ schistosity
6.5m

Case 53

Case 54

1. same as Case 47
2. collector tunnel
3. W = 4.2m
4. H = 4.5m
5. A = $18m^2$
6. vertical chimney-shaped void in roof formed by washout of shear zone materials (see Fig. 2.16); loosened rock in crown, likely collapse of section if unsupported; (F)
7. L = 50m
8. one-meter-wide shear zone in quartzite; zone filled with intact feldspar and feldspar hydrothermally altered to clay
9. rock bolts, wire mesh, two shotcrete applications
10. D = 60m
11. RQD = 70% across tunnel, measured; RQD_v = 0-100%; RQD_a = 0-100%
12. ----
13. ----
14. zone is located in a major overthrust sheet (see Fig. 2.5)
15. large water and water-clay slurry inflows after blasting caused chimney-shaped opening to form. Clay continued to run out at a lesser rate after blasting

Case 55

1-5. same as Case 54

6. wedge-shaped roof fall (see sketch); (H)

7. L = 20m

8. overthrust shear zone in quartzite; clayey, sandy joint fillings in joints parallel to zone

9. rock bolts, wire mesh, shotcrete

10. D = 90m

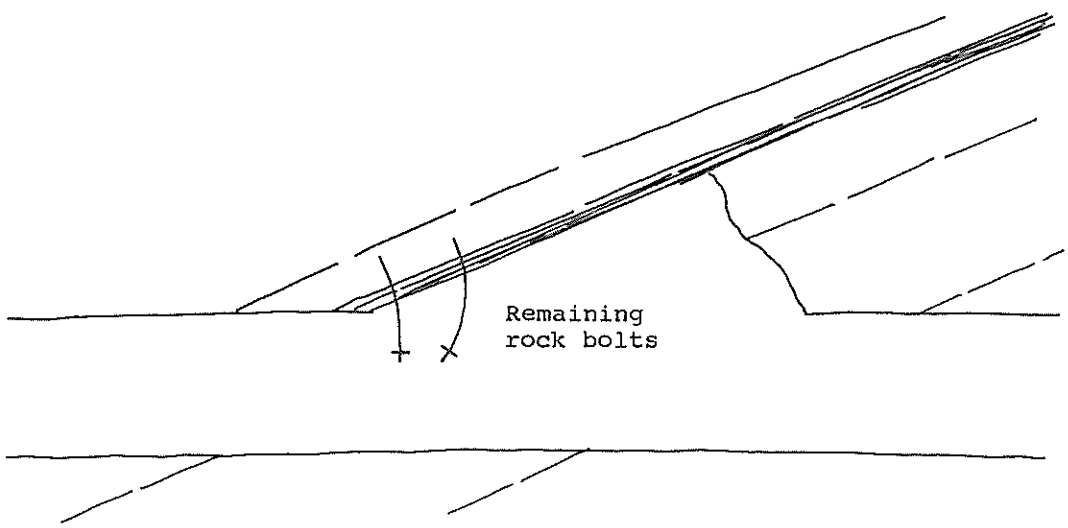
11. RQD = 80% across shear zone, estimated; RQD_v = 80%, RQD_a = 80%

12. ----

13. ----

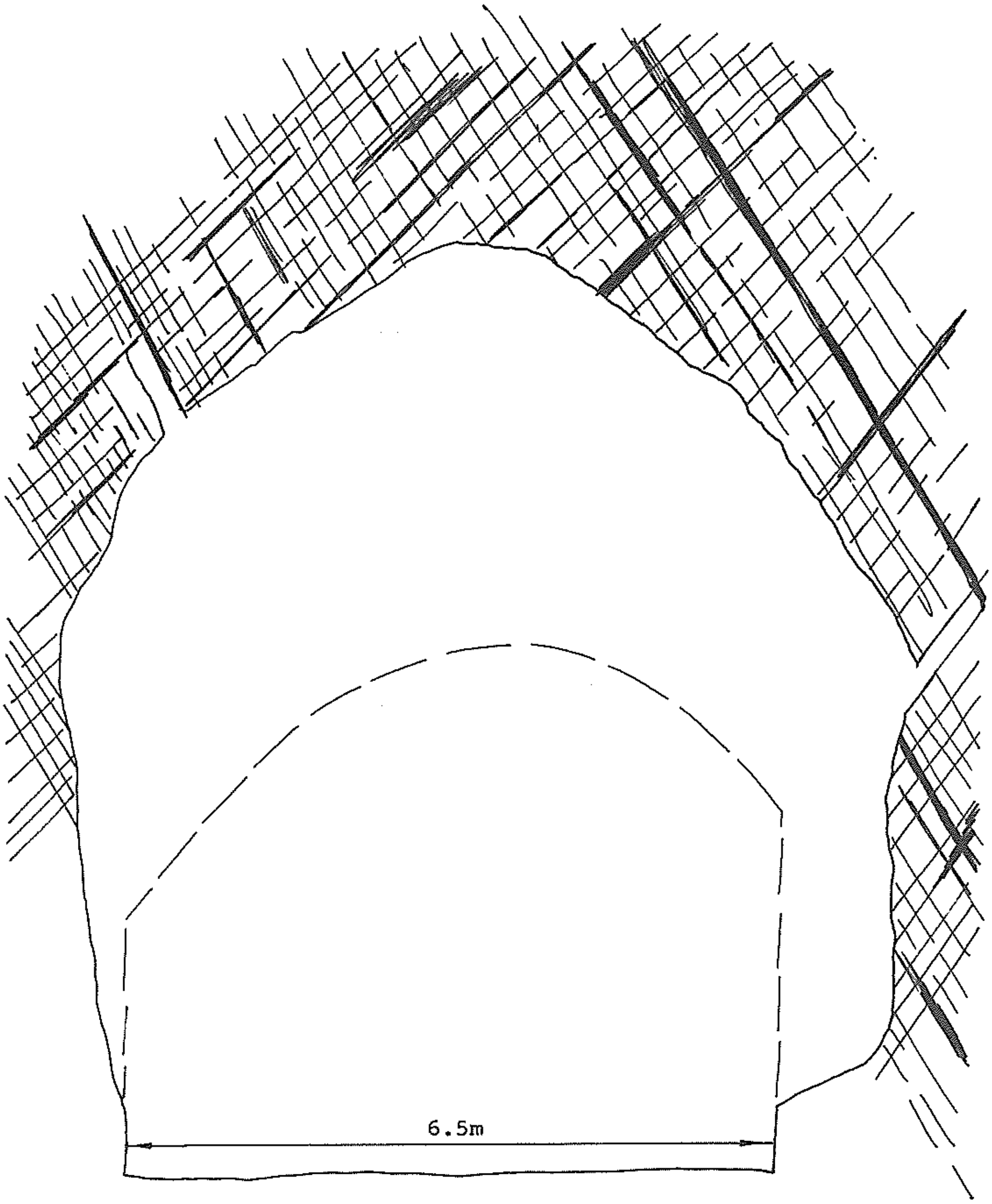
14. location in same major overthrust sheet as Case 54

15. initial inflows washed some softening clay joint fillings out



Case 56

1. Stensjöfallet Hydroelectric, northern Sweden
2. headrace tunnel
3. $W = 5.9m$
4. $H = 4.25m$
5. $A = 24m^2$
6. progressive roof fallout to form a large vault-shaped opening (see sketch); (F)
7. $L = 20m$
8. 10-meter-wide vertical shear zone in granite; rock is crushed and frequently altered to an earthy gravel; some remnant joint surfaces coated with clay; zone has probably been strongly affected by hydrothermal action; rock adjacent to zone is blocky and loose
9. no support immediately after blasting; eventually supported with two shotcrete applications.
10. $D = 100m$
11. RQD = 10% average along 20m length of wall, measured; RQD_v = 0-30%
12. ----
13. ----
14. tunnel is located within 10km of a major overthrust sheet; locally, vertical shear zones occur together with low angle shear zones
15. large water inflows after blasting carried fault zone debris into tunnel, left open voids up to 1m wide.
16. initial attempts to scale roof provoked falls and formation of vault; shotcrete immediately after blasting may have prevented large overbreak



Case 56

Case 57

1-5. same as Case 56

6. similar to Case 56, but fallout less serious and confined to right side of tunnel section as shown in sketch; (F)

7. L = 10m

8. vertical shear zone (50cm-1m wide) in granite contains crushed and altered material that was partially washed out, leaving open voids around shotcreted roof. Roof subsequently caved and progressively enlarged section. Occasional clay-filled fissures. Blocky and loose rock in adjacent walls and roof.

9. rock bolts, two shotcrete applications

10. D = 100m

11. RQD = 40% along and up wall, estimated

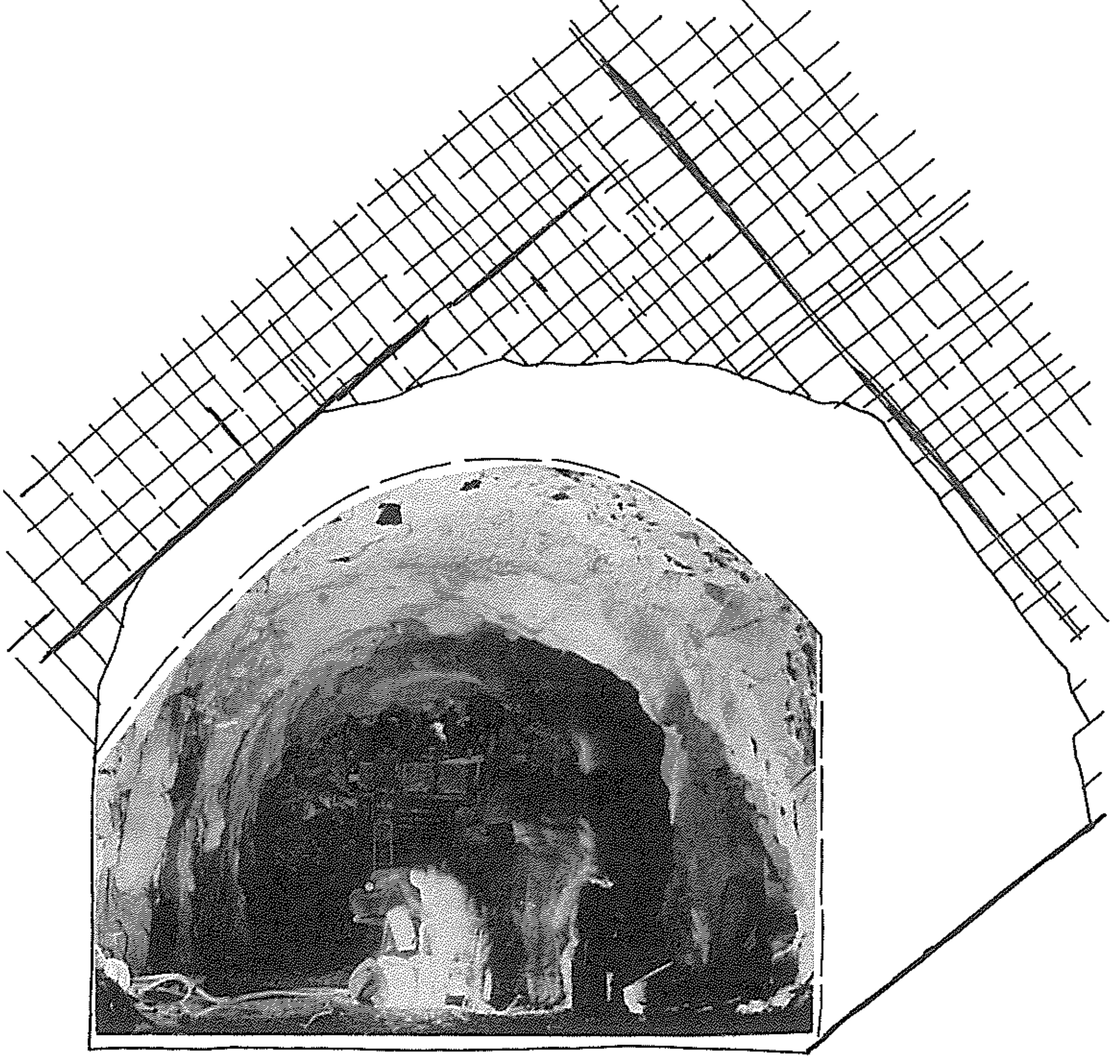
12. ----

13. ----

14. same as Case 56

15. same as Case 56

16. some scaling after blasting, before bolting and shotcreting; subsequent shotcreting after washing out occurred



Case 57

Case 58

1-5. same as Case 56

6. block falls in roof (see sketch); (E)

7. L = 20m

8. widely spaced (2-3m) inclined joint sets in granite, very large blocky structure; because of movements along a horizontal shear in wall, rock structure is loose

9. rock bolts

10. D = 90m

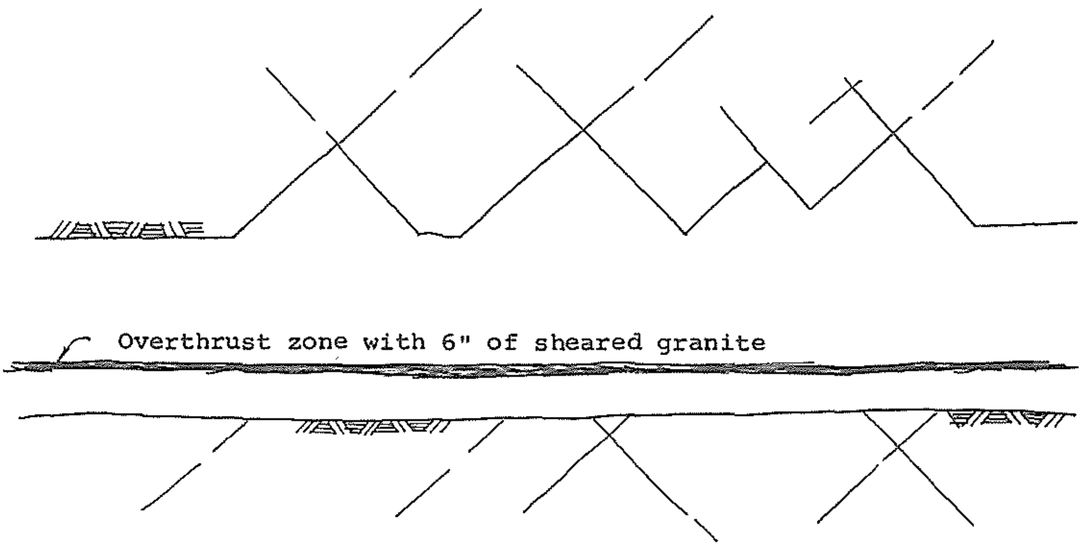
11. RQD = 100% along wall, estimated; RQD_v = 90%

12. ----

13. ----

14. same as Case 56

15. insignificant flows



Profile Along Tunnel Axis

Case 59

1-5. same as Case 56

6. overbreak and roof falls (see photo); (E)

7. L = 10m

8. 50cm-wide vertical shear seam with clay-filled joints and some altered granite; adjoining rock is partially altered to a very loose, earth-like material; on edge of zone rock is blocky

9. rock bolts, wire mesh, shotcrete

10. D = 95m

11. RQD = 50% along wall, estimated; RQD_V = 10-100%

12. ----

13. ----

14. same as Case 56

15. no water

Case 60

1-5. same as Case 56

6. wedge-shaped roof fall (see sketch); (D)

7. L = 20m

8. 1-meter-wide zone of sheared granite with clay seam; slide boundary is a thin (<1cm) clay seam and thinly sheared material that lie in contact with massive rock

9. rock bolts, shotcrete

10. D = 85m

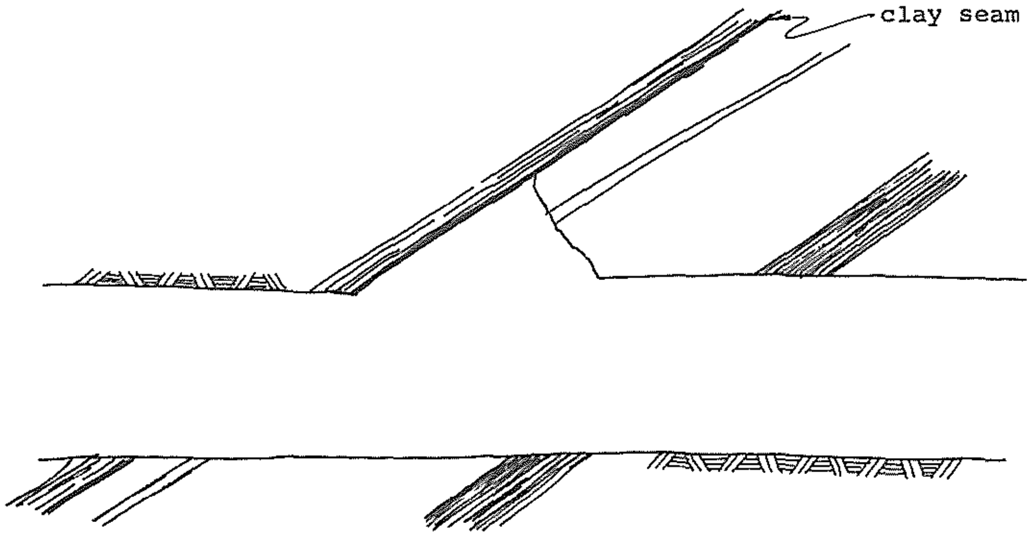
11. RQD = 80% along wall, measured; RQD_v = 80%

12. ----

13. ----

14. same as Case 56

15. insignificant inflows



Case 61

1-5. same as Case 56

6. overbreak and roof falls; (E)

7. L = 15m

8. steeply dipping shear zone, completely altered to clay products, surrounding granite blocky; clay is montmorillonitic and swelling could take place and lead to loosening of large volumes of rock

9. rock bolts, shotcrete

10. D = 70m

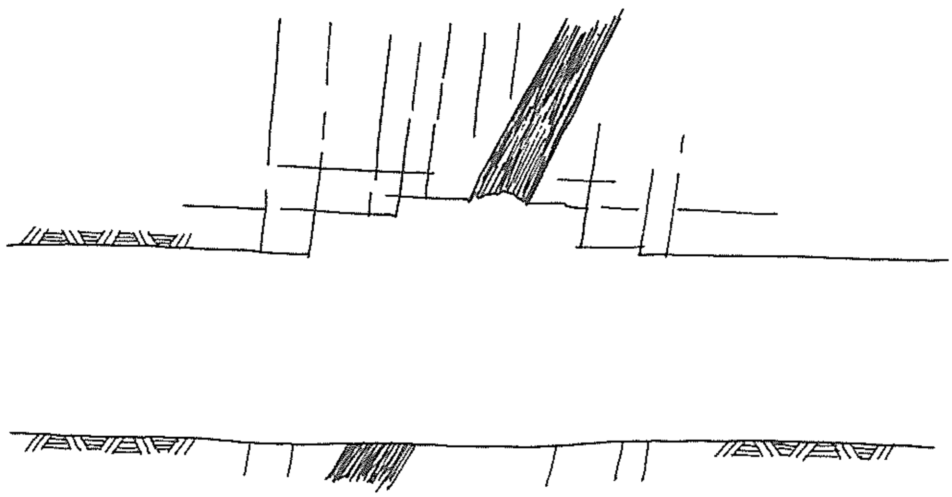
11. RQD = 70% average along wall, measured; RQD_v = 30-100%

12. ----

13. ----

14. same as Case 56; zone in this case shows evidence of much vertical shearing

15. insignificant water



Case 62

1-5. same as Case 56

6. overbreak, some roof falls up to several cubic meters, (F)

7. L = 30m

8. very loose large blocky and seamy granite in the vicinity of a few vertical altered seams with clay and sand fillings; some discontinuous diagonal fissures

9. rock bolts, shotcrete

10. D = 40m

11. RQD = 90% along wall, estimated about same vertically

12. ----

13. ----

14. same as Case 56

15. minor inflows

Case 63

1-5. same as Case 56

6. overbreak in intrados and crown; (E)

7. L = 50m

8. three intersecting joint sets in granite; wide joint spacing

9. rock bolts

10. D = 80m

11. RQD = 100% up and along wall, measured

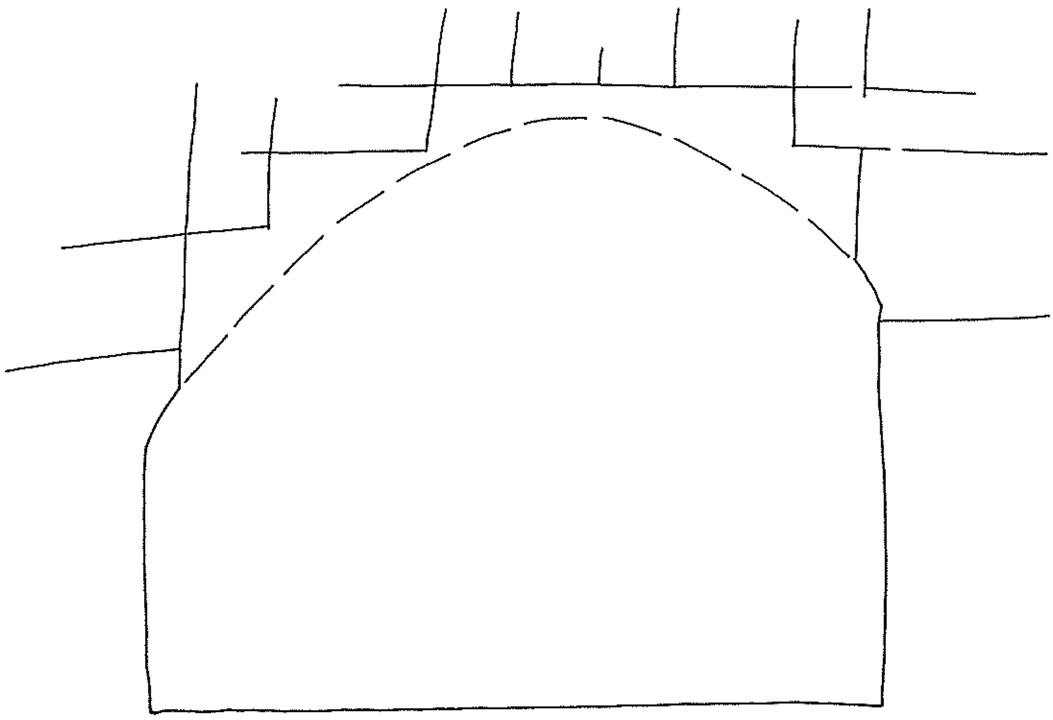
12. ----

13. ----

14. same as Case 56

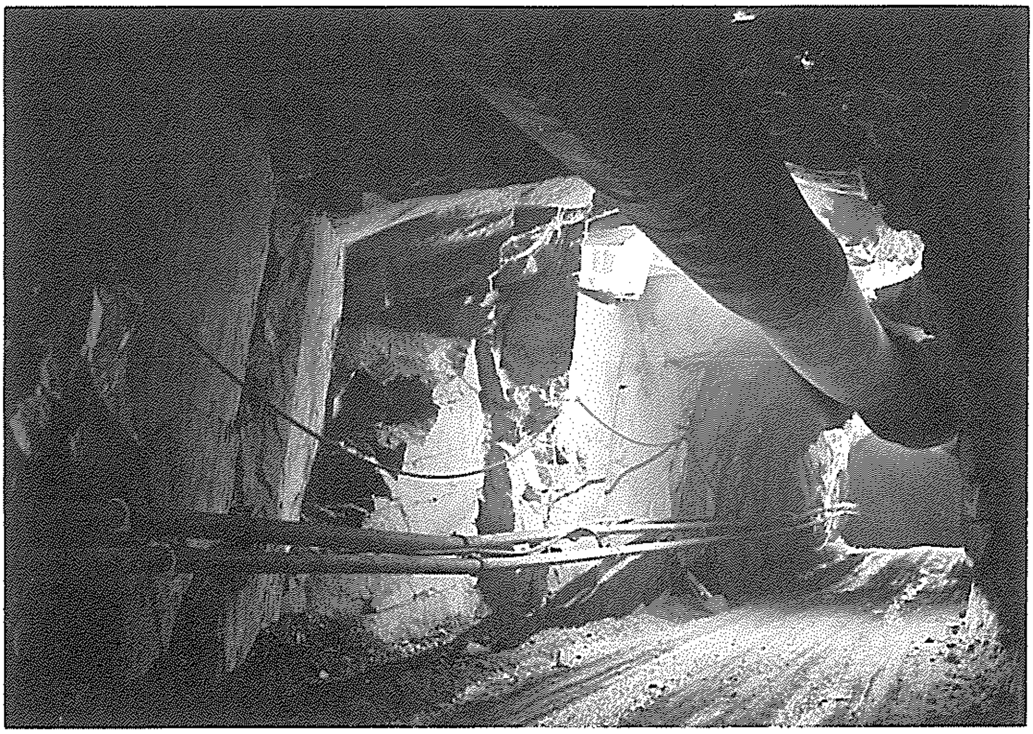
15. no water

16. progressive overbreak would probably eventually stabilize section



Case 64

1. Stensjöfallet Hydroelectric, northern Sweden
2. access tunnel, Stora Stensjön
3. $W = 5m$
4. $H = 5m$
5. $A = 20m^2$
6. large overbreak in entire section (see photo); (E)
7. $L = 50m$
8. same blocky granite as in Case 63; horizontal shearing joints and vertical joints
9. spot rock bolting
10. $D = 15m$
11. RQD = 100% all directions, measured
12. ----
13. ----
14. same as Case 56
15. no inflows



Case 65

1-5. same as Case 56

6. overbreak to form a square section; (E)

7. L = 20m

8. intersecting joint sets in granite; blocky structure; some joints filled with clay- and sand-size material, all joints open, loose rock structure

9. rock bolts

10. D = 40m

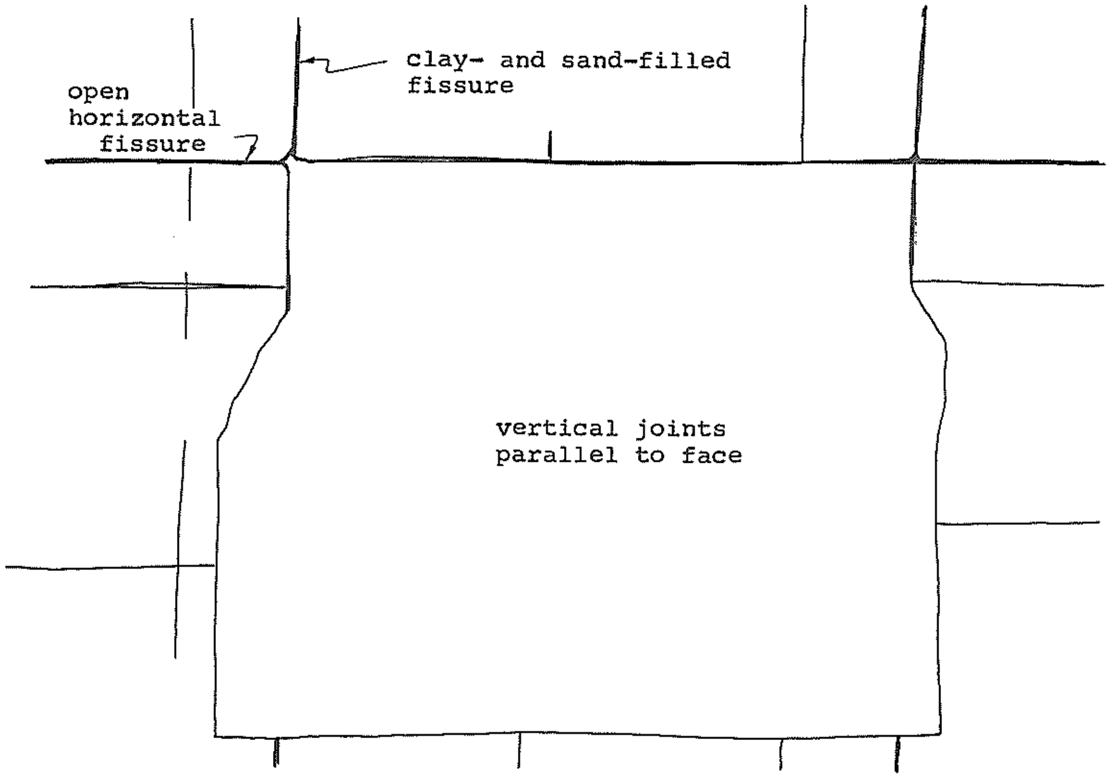
11. RQD = 90% along wall, measured; RQD_V = 60-100%

12. (V = 3500 m/sec)

13. (SVR = 3500/5000 = 0.70)

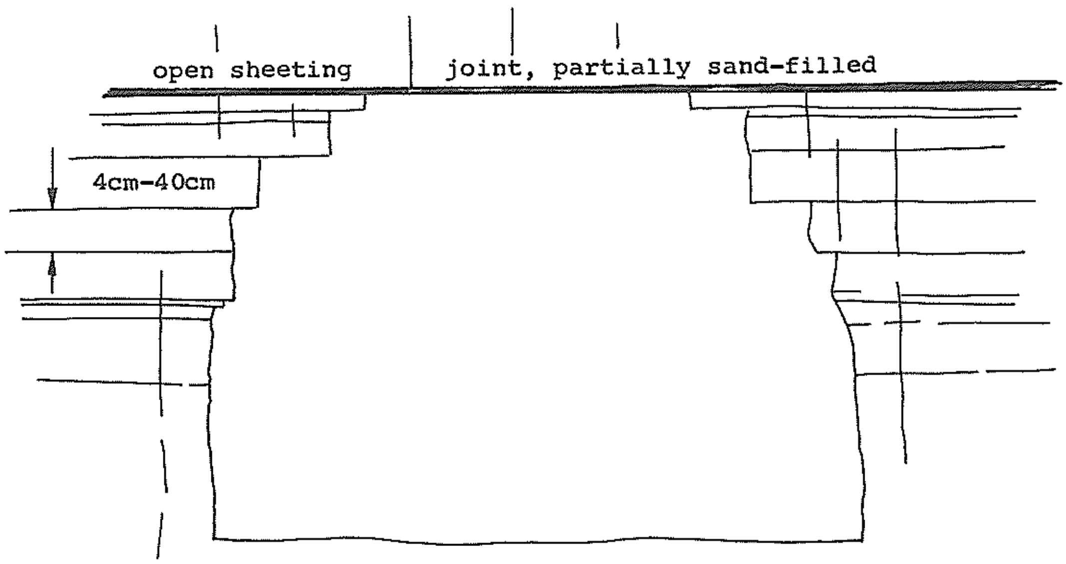
14. same as Case 56

15. minor water inflows (<1 lit/min)



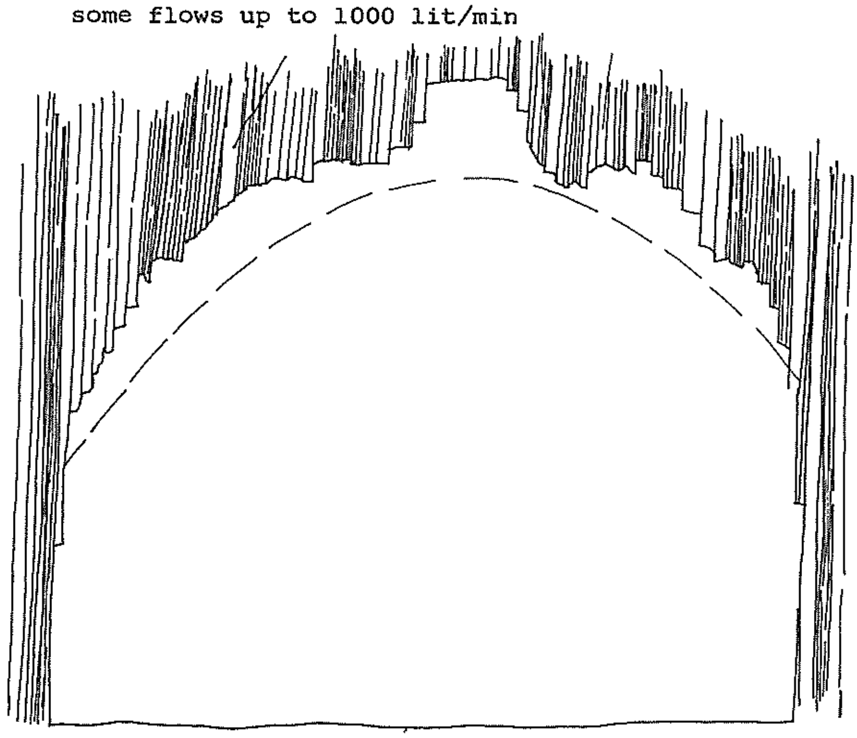
Case 66

1. Stensjöfallet Hydroelectric, northern Sweden
2. access tunnel, Lilla Stensjön
3. W = 7m
4. H = 4.5m
5. A = $24m^2$
6. overbreak above springline; (E)
7. L = 80m
8. horizontal sheeting joints in granite partially filled with sand-size material
9. rock bolts, shotcrete
10. D = 15-20m
11. RQD = 70% up wall, measured; RQD_a = 10-100%
12. ----
13. ----
14. same as Case 56
15. insignificant water



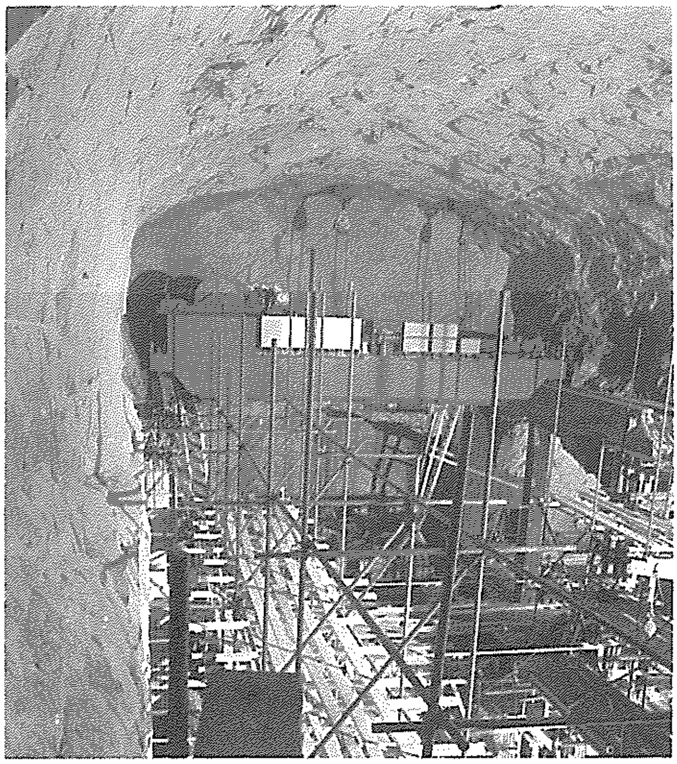
Case 67

1. Stensjöfallet Hydroelectric, northern Sweden
2. tailrace tunnel
3. W = 5.9m
4. H = 4.8m
5. $A = 24m^2$
6. large overbreak in intrados, some roof falls; (E)
7. L = 50m
8. close vertical jointing cutting across schistose rock structure; schistose metagreywacke
9. shotcrete
10. D = 100m
11. RQD = 20% across tunnel, estimated; RQD_v = 10-50%; RQD_a = 10-50%
12. ----
13. ----
14. tunnel is in an overthrust sheet
15. some flows up to 1000 lit/min



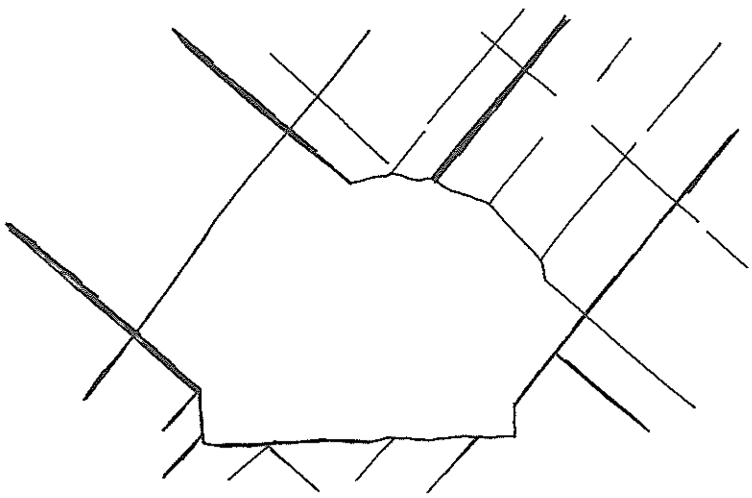
Case 68

1. Stensjöfallet Hydroelectric, northern Sweden
2. machine hall
3. W = 10m
4. H = 15m
5. A = $140m^2$
6. stable, insignificant overbreak; (A)
7. L = 90m
8. massive granite no joints
9. no structural support required
10. D = 300m
11. RQD = 100% all directions, observed
12. ----
13. ----
14. same as Case 50
15. no water



Case 69

1. Mo i Rana Hydroelectric, northern Norway
2. collector tunnel (Akersvatnet)
3. $W = 8m$
4. $H = 5.7m$
5. $A = 39m^2$
6. roof falls, loosening of large rock blocks in crown; (D)
7. $L = 25m$
8. large blocky marble; montmorillonitic clay joint fillings up to 1cm wide have swelled, started to run out of joints
9. rock bolts, cast concrete arches
10. $D = 15m$
11. RQD = 90% up wall, estimated; RQD_a = 90%
12. ----
13. ----
14. location in a major overthrust sheet
15. minor amounts of water have enabled swelling of montmorillonite and have partially washed out joint fillings



Case 70

1-5. same as Case 69

6. stable, minor overbreak, no roof falls; (A)

7. L = 10m

8. strongly sheared granite, joint spacing 5-25 cm, very tight vertical structure

9. none

10. D = 15m

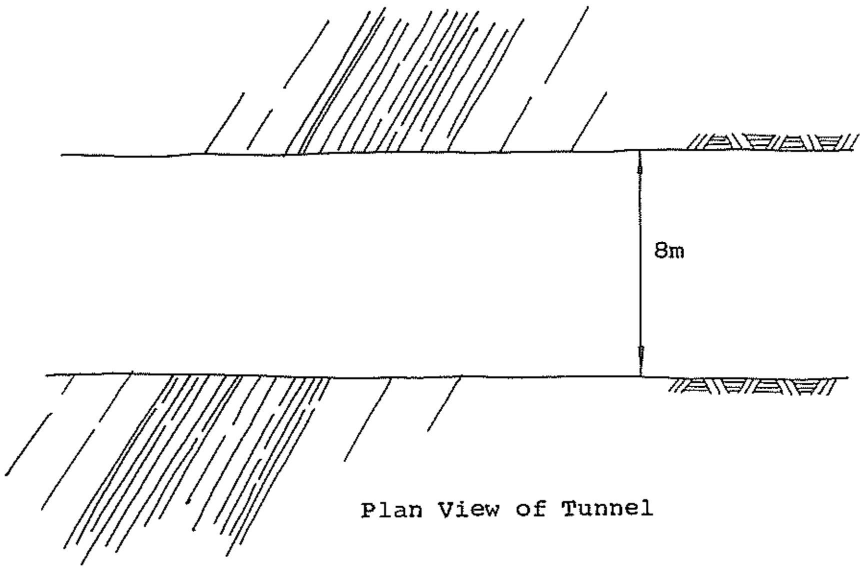
11. RQD = 40% along wall, measured; RQD_V = 0-100%

12. ----

13. ----

14. same as Case 69, severe cases of slabbing only 500 m from this location

15. insignificant water



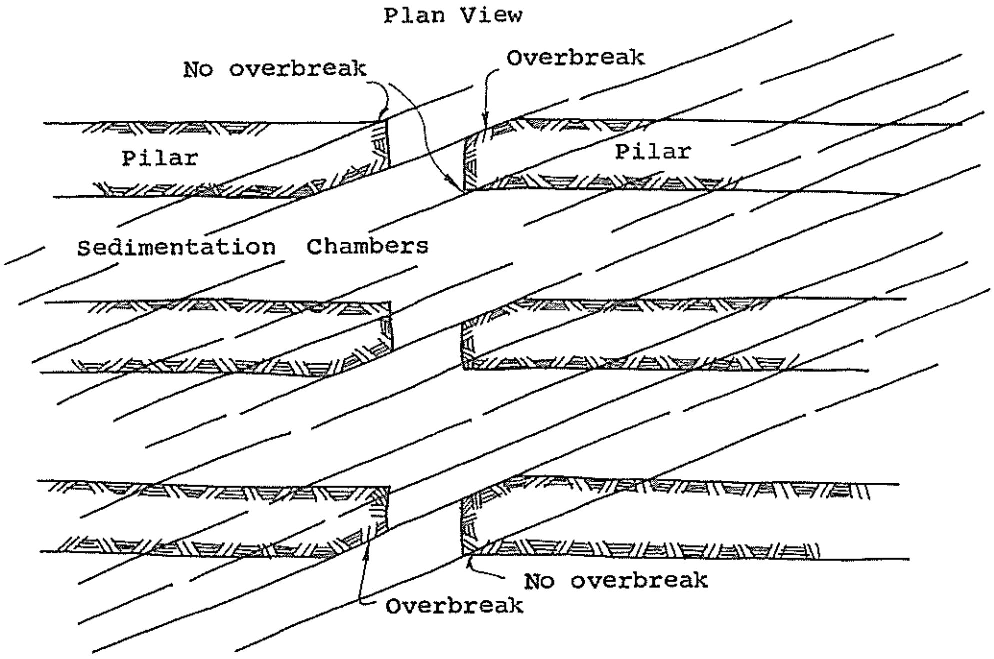
Plan View of Tunnel

Cases 71, 72 and 73

1. Lieråsen, Norway
2. twin track railroad tunnel
3. W = 9.5m
4. H = 6.5m
5. A = $60m^2$
6. overbreak and roof falls; (E)
7. L = total about 2 km
8. intersecting montmorillonite-filled joints and shears in granite; some hydrothermally altered rock containing montmorillonite
9. cast concrete arches and multiple shotcrete applications
10. D = up to several hundred meters
11. at three different locations RQD values along the wall were estimated to be 30, 70 and 80% RQD_v values range from 0 to 100%
12. ----
13. ----
14. major normal faulting in area
15. some moderate inflows
16. heavy applications of shotcrete (25 cm) cracked shortly after application, additional shotcrete (10 cm) applied to obtain complete stabilization

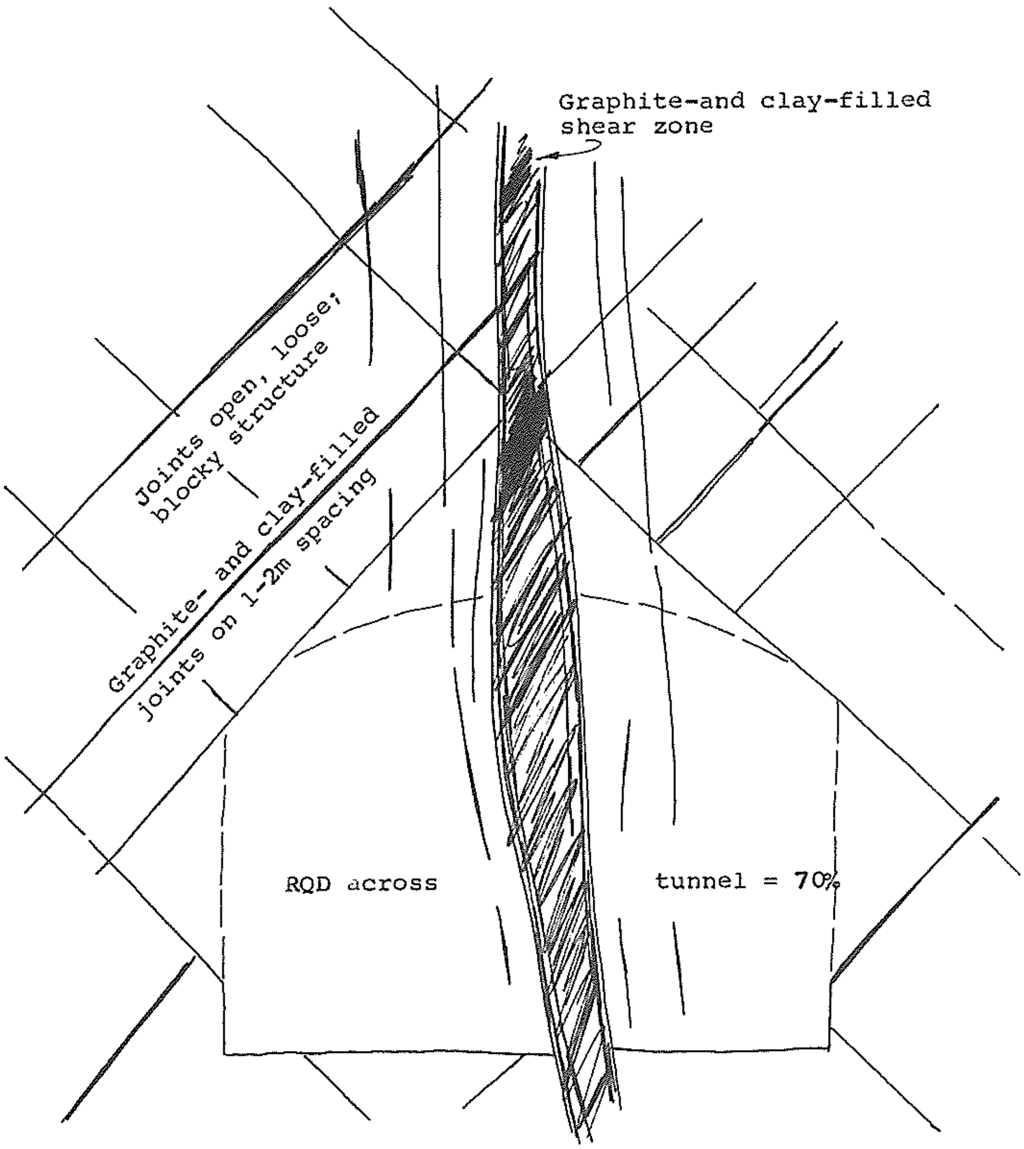
Cases 74 and 75

1. Käppala Waste Water Treatment Plant
2. sedimentation chambers
3. W = 12m
4. H = 12.5m
5. A = 116m^2
6. no overbreak in chambers; overbreak at intersections (see sketch); (A, E)
7. L = about 2 km
8. massive granite; widely spaced, tight vertical joints
9. none in chambers; bolts at intersections
10. D $\leq$ 100m
11. RQD = 100% all directions, observed
12. ----
13. ----
14. major normal faulting in vicinity
15. insignificant inflows
16. note importance of corner location in determining overbreak



Case 76

1. Stockholm subway
2. double track train tunnel
3. W = 8m
4. H = 6m
5. A = $40m^2$
6. overbreak in entire section, roof falls (see sketch); (G)
7. L = 20m
8. loosened granite around a graphite- and clay-filled vertical shear zone; joints in surrounding rock are open, partially clay-filled
9. rock bolts, cast concrete arch
10. D = 20-30m
11. RQD = 70% across tunnel face, measured; RQD_a = 0-90%; RQD_v = 0-90%
12. ----
13. ----
14. major normal faulting in area
15. insignificant inflows

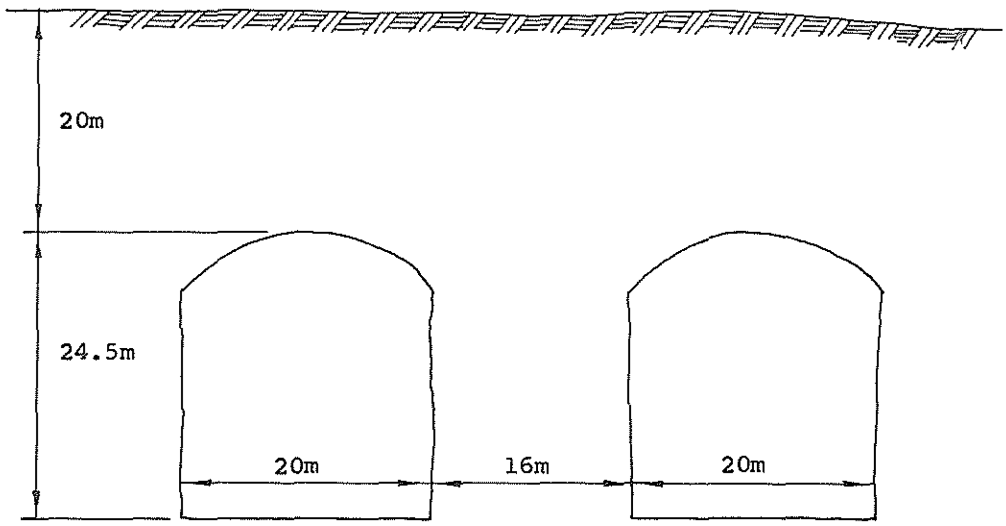


Graphite-and clay-filled shear zone
Joints open, loose; blocky structure
Graphite- and clay-filled joints on 1-2m spacing
RQD across tunnel = 70%

Case 76

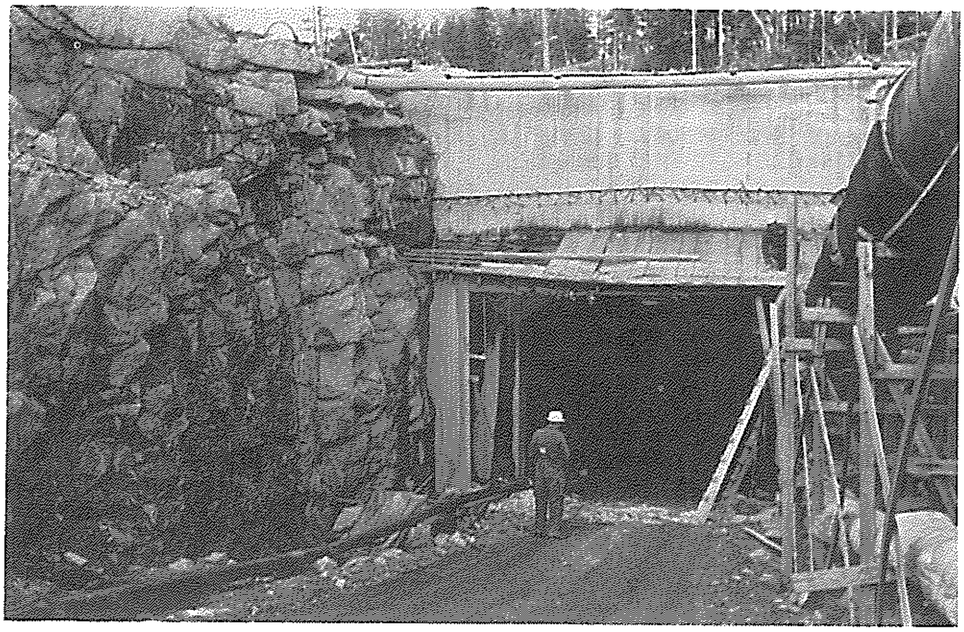
Case 77

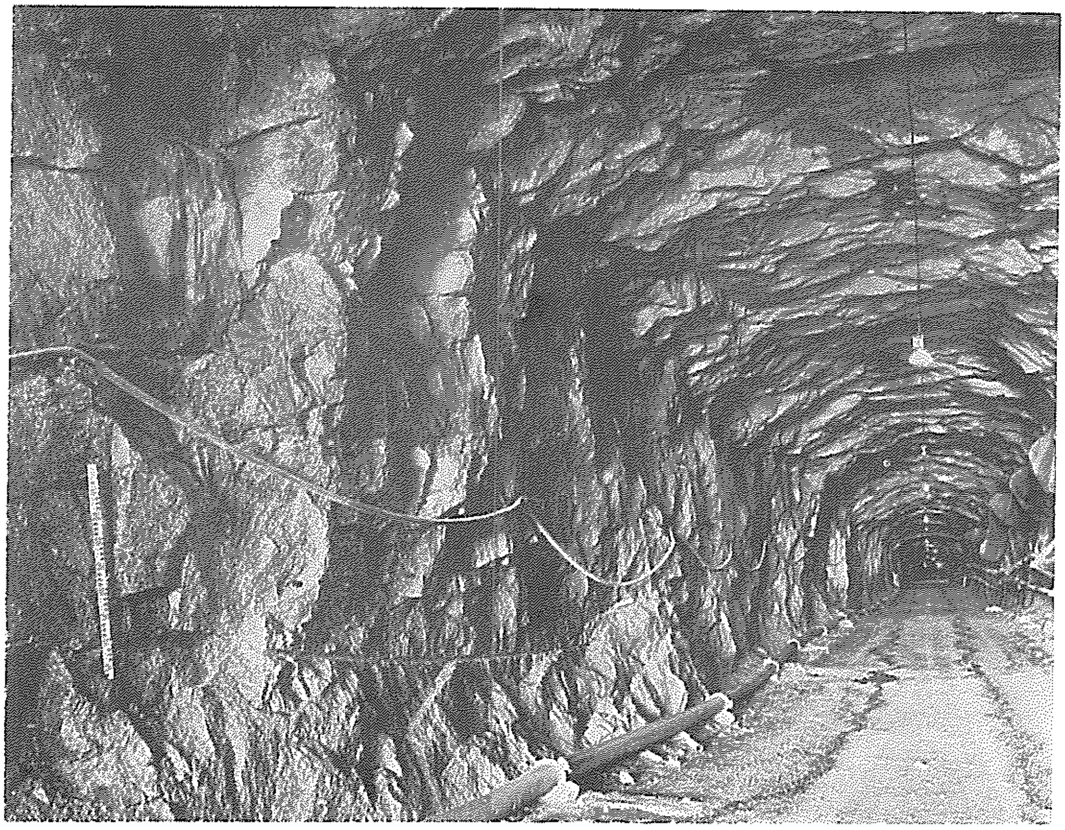
1. Årstadal, Stockholm
2. underground wine and liquor storage rooms
3. W = 20m
4. H = 24.5m
5. A ≈ $440m^2$
6. minor overbreak, no falls or slides; (A)
7. L = 300m
8. massive gneiss, few joints
9. none, only 50 spot bolts in about 300m of chamber, including several intersections
10. D = 18m
11. RQD = 100% all directions, observed
12. ----
13. ----
14. major normal faulting in vicinity
15. insignificant inflow
16. Note thin cover over these large rooms



Case 78

1. Råtan Hydroelectric, central Sweden
2. access tunnel
3. $W = 5m$
4. $H = 4.5m$
5. $A \approx 20m^2$
6. moderate overbreak; flat roof because of rock structure (see photo); (B)
7. $L = 30m$
8. sheeting in granite, joint spacing 10cm-50cm
9. no support
10. $D = 5-10m$
11. RQD = 90% up wall, measured; RQD_a = 80-100%
12. ----
13. ----
14. overthrust faulting in area
15. insignificant inflows





Case 78

Case 79

1. Råtan Hydroelectric, central Sweden
2. tailrace tunnel
3. $W = 11.25m$
4. $H = 8.3m$
5. $A = 80m^2$
6. progressive collapse of roof at and near portal, failure of initial single shotcrete application; (H)
7. $L = 20m$
8. altered and disintegrated amphibolite, consistency of a moist sand, unconfined strength of a 5 cm cube = 5 kg/cm^2 (75 psi)
9. 20 cm shotcrete or cast concrete arch
10. $D = 5\text{-}20m$
11. RQD = 0, all directions, observed
12. ----
13. ----
14. major overthrusting in area
15. insignificant water flows

Case 80

1-5. same as Case 79

6. roof falls, major overbreak; (E)

7. L = 20m

8. shear zone in fine-grained granite; vertical jointing (5-40 cm spacing), some clay-coated joints (similar to rock above board fence in photo)

9. two shotcrete applications

10. D = 30m

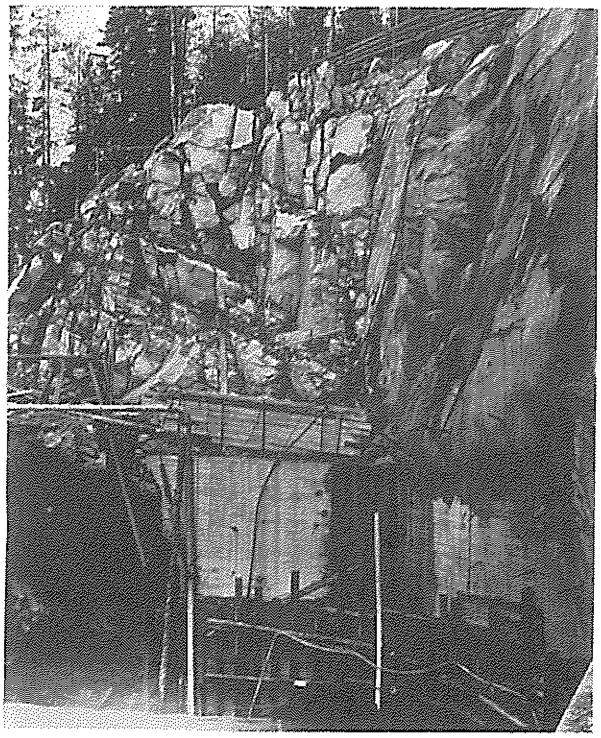
11. RQD = 50% along wall, estimated; RQD_V = 0-100%

12. (V = 3400m/sec)

13. (SVR = 3400/5000 = 0.68)

14. same as Case 79

15. minor inflows



Case 81

1-5. same as Case 79

6. overbreak, minor falls in roof; (E)

7. L = 10m

8. strongly crushed and sheared fine-grained granite; some altered material; rock around zone is penetrated by a network of fine cracks that are filled with rock flour (see photo)

9. two shotcrete applications

10. D = 60m

11. RQD = 30% along wall, measured; RQD_V = 20-60%

12. V = 4000m/sec; (V = 3500m/sec)

13. SVR = 4000/6400 = 0.63; (SVR = 3500/5200 = 0.67)

14. same as Case 79

15. some open, water-bearing joints; small amounts of joint filling washed out

Case 82

1-5. same as Case 79

6. large anticipated roof and wall falls if unsupported; (C)

7. L = 30m

8. large blocky fine-grained granite; three intersecting joint sets

9. rock bolts, shotcrete (one application)

10. D = 60m

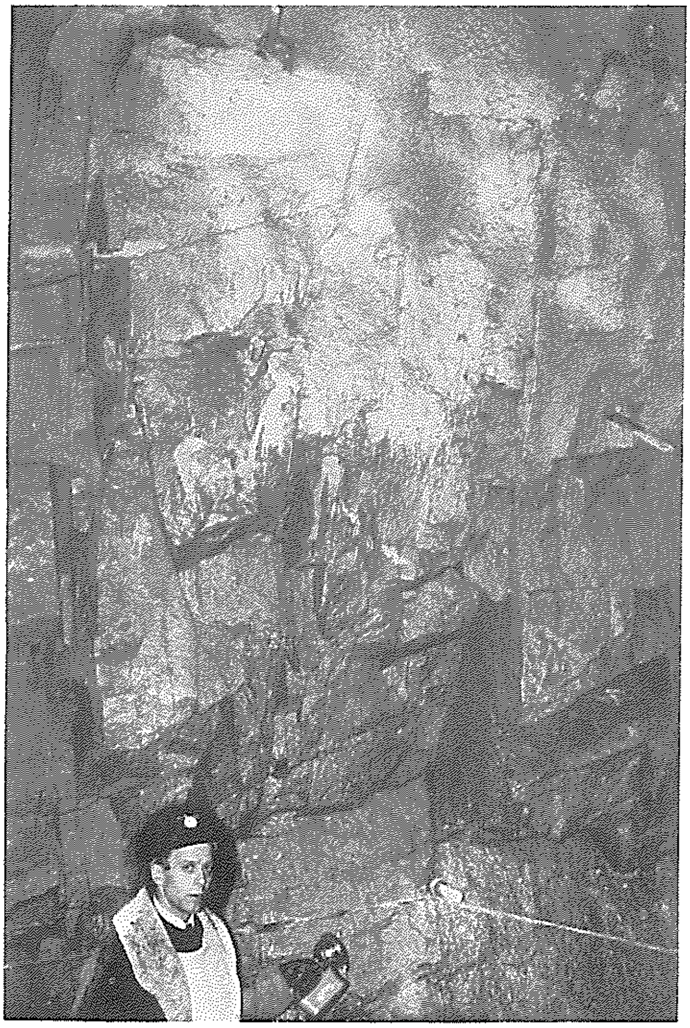
11. RQD = 80% along wall, measured; RQD_v = 90%

12. V = 5500m/sec

13. SVR = 5500/6400 = 0.86

14. same as Case 79

15. minor water inflow along open joints



Case 83

1-5. same as Case 79

6. minor overbreak, no falls; (A)

7. L = 20m

8. vertically jointed fine-grained granite; joint spacing 4-100 cm; some flat-lying intersecting joints - blocky to slabby, tight structure (see photo)

9. none

10. D = 60m

11. RQD = 70% along wall, measured; RQD_V = 10-80%

12. V = 6400m/sec; (V = 3800m/sec)

13. SVR = 1; (SVR = 3800/5200 = 0.73)

14. same as Case 79

15. very minor inflows

Case 84

1-5. same as Case 79

6. large overbreak, some roof falls; (E)

7. L = 20m

8. shear zone in diabase, small block structure

9. shotcrete

10. D = 70m

11. RQD = 60% along wall, measured; RQD_V = 60%

12. V = 5000 m/sec; (V = 2900m/sec)

13. SVR = 5000/6300 = 0.79; (SVR = 2900/5400 = 0.54)

14. same as Case 79

15. some open water-bearing joints

Case 85

1-5. same as Case 79

6. major roof falls, face and walls unstable; (G)

7. L = 130m

8. heavily sheared rock with frequent clay-filled joints and some altered rock; structure generally dips 45° along the axis of the tunnel; rock type is alternating layers of diabase and fine-grained aplitic granite

9. shotcrete before mucking out, additional shotcrete after mucking

10. D = 60m

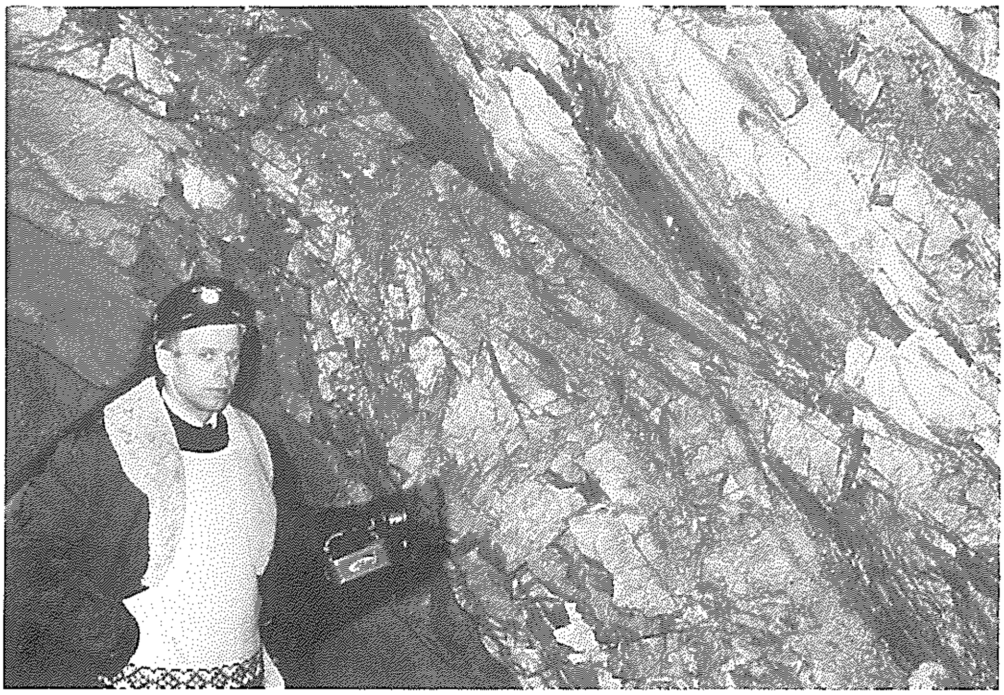
11. RQD = 0-30% along and up walls, measured

12. V = 3700-4650m/sec; (V = 3800m/sec)

13. SVR = 3700/6400-4650/6400 = 0.58-0.73; (SVR = 3800/5200 = 0.73)

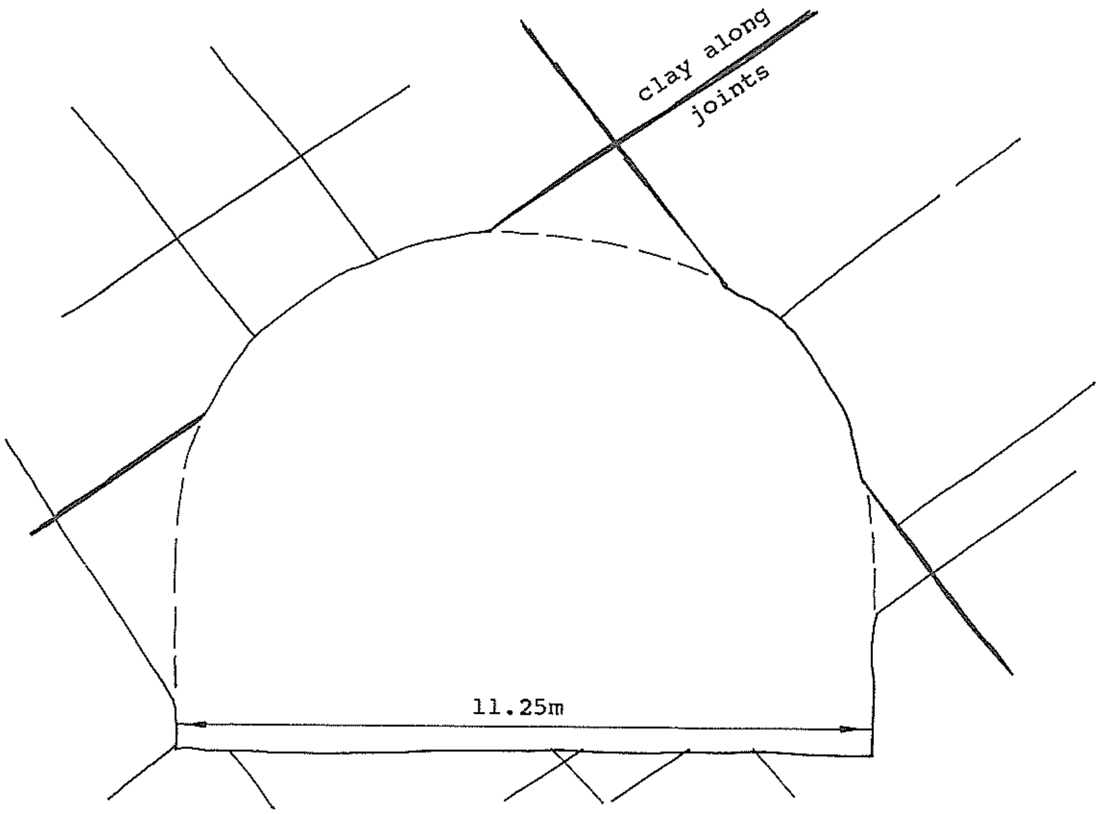
14. unit is an overthrust fault zone

15. some open water bearing joint sets



Case 86

1-5. same as Case 79
6. overbreak; (C)
7. L = 100m
8. large blocky amphibolite; a few widely spaced joints, some clay-filled joints
9. rock bolts
10. D = 60m
11. RQD = 90% up wall, estimated; RQD_a = 90%
12. V = 5800m/sec
13. SVR = 5800/6100 = 0.95
14. same as Case 79
15. minor inflows



Case 87

1-5. same as Case 79

6. no overbreak or falls; (A)

7. L = 200m

8. massive, sound, intact granite; very few joints

9. none

10. D = 50m

11. RQD = 100% all directions; observed

12. V = 6000m/sec

13. SVR = 1

14. located 200m from overthrust fault

15. no water

Case 88

1. Rätan Hydroelectric, central Sweden

2. heading for tailrace tunnel

3. W = 11.25m

4. H = 5.5m

5. A = $55m^2$

6. overbreak in entire section, marginal stability in intrados (see Fig. 2.11); (G)

7. L = 40m

8. overthrust fault zone; horizontally sheared rock; many joints partly or entirely filled with clayey and sandy material; very loose structure

9. rock bolts, two shotcrete applications

10. D = 60m

11. RQD = 50% up wall, measured; RQD_a = 60%

12. ----

13. ----

14. overthrust fault zone

15. some artesian flows in vertical drill holes in floor

Case 89

1-5. same as Case 88

6. overbreak above springline; (E)

7. L = 70m

8. overthrust fault zone, same as Case 88, but wider joint spacing, joints generally tight, few clay- or sand-filled joints

9. one shotcrete application

10. D = 60m

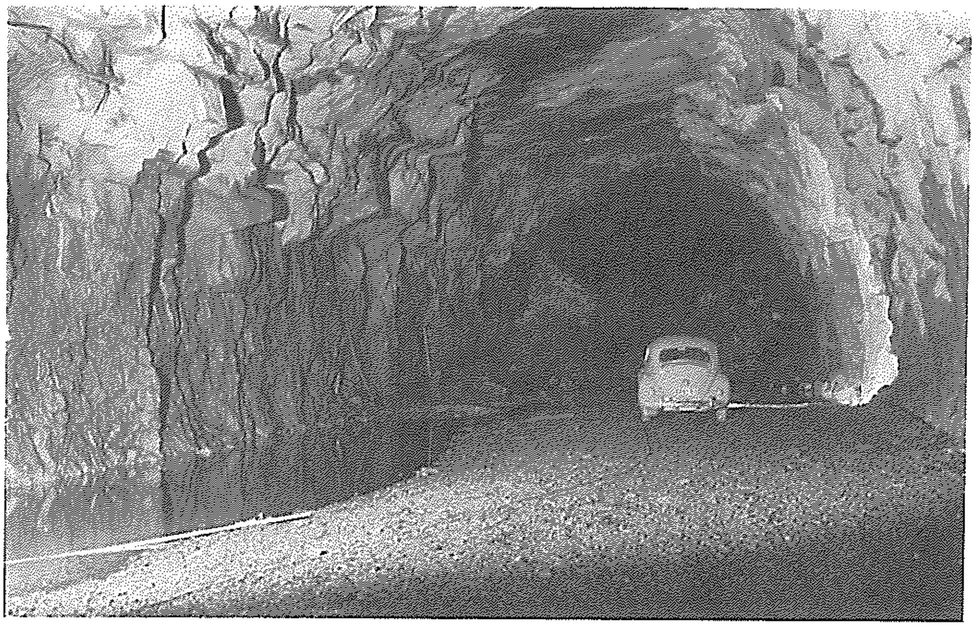
11. RQD = 80% up wall, measured; RQD_a = 90-100%

12. ----

13. ----

14. same overthrust as Case 88 but less shearing movement along joints

15. no water



Case 90

1-5. same as Case 88

6. overbreak and falls in roof (see photo); (E)

7. L = 100m

8. overthrust shear jointing, some clay-filled joints

9. rock bolts

10. D = 60m

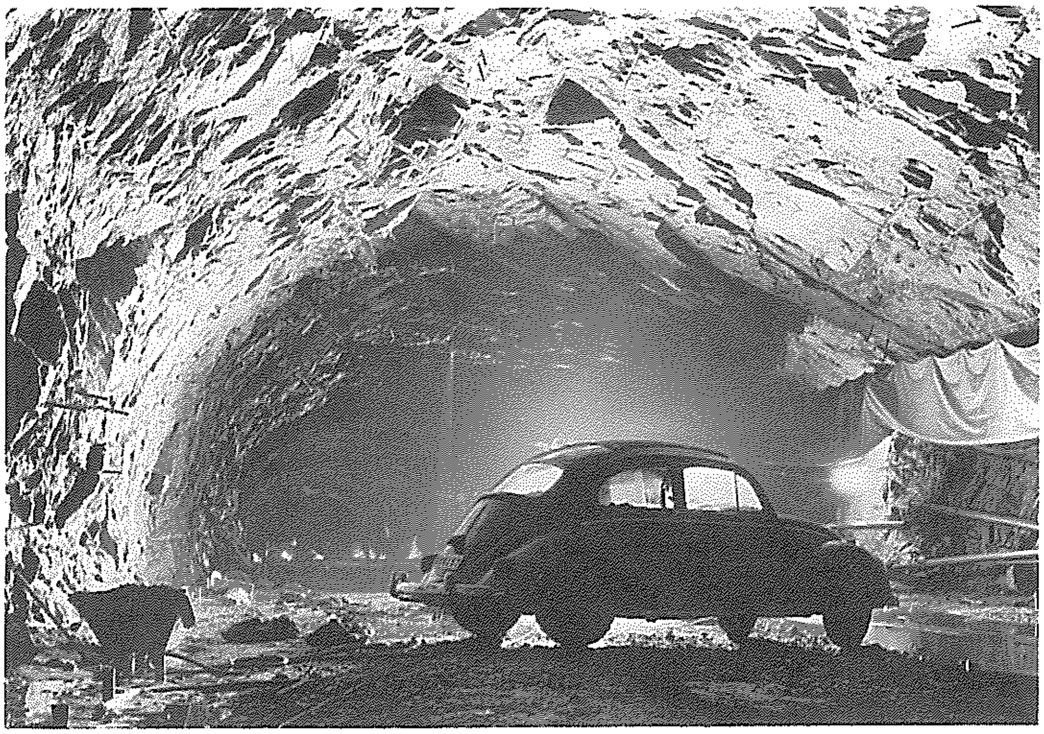
11. RQD = 90% up wall, measured; RQD_a = 90%

12. ----

13. ----

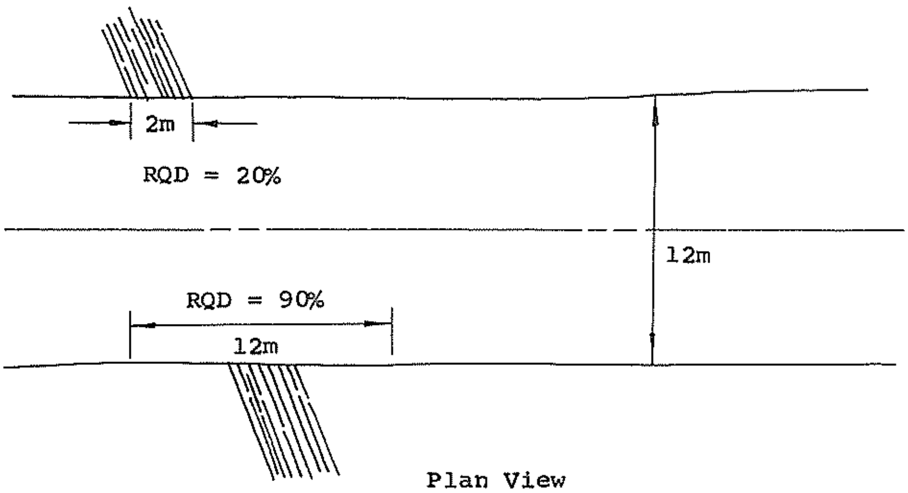
14. overthrust zone

15. moderate water inflows (to 100 lit/min)
artesian condition in drill holes in floor



Case 91

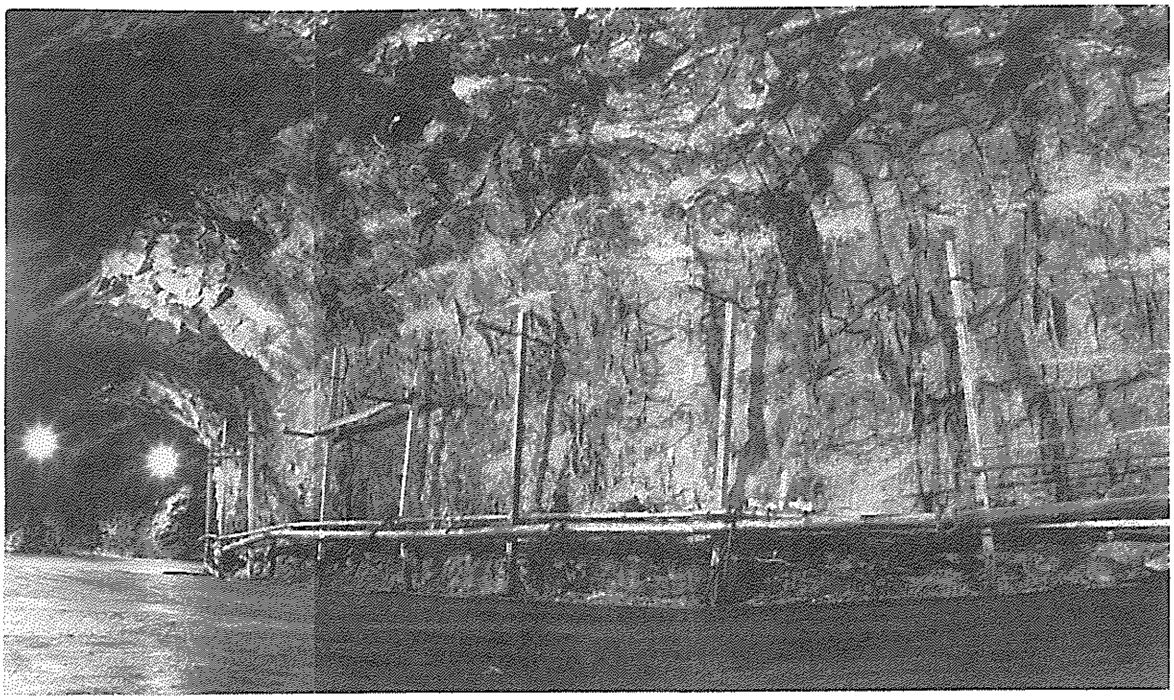
1. Rätan Hydroelectric, central Sweden
2. surge chamber
3. W = 12m
4. H = 5m
5. A ≈ $50m^2$
6. minor overbreak, no roof falls; (A)
7. L = 12m
8. close vertical jointing over a 2-meter section, very tight, granite
9. none
10. D = 60m
11. RQD = 20% along 2-meter zone, measured
    RQD = 90% along 12-meter length of opening;
    RQD_v = 0-100%
12. ----
13. ----
14. same as Case 79
15. insignificant inflow



Plan View

Case 92

1. Letsi Hydroelectric, northern Sweden
2. intersection of access tunnels to tailrace tunnel and machine hall
3. W = 20m
4. H = 9m
5. A $150m^2$
6. minor overbreak, no falls; (C)
7. L = 20m
8. very closely spaced vertical joints in granite, intersected in roof by a few widely spaced diagonal and flat-lying joints, structure very tight (see photo)
9. spot rock bolting
10. D = 100m
11. RQD = 30%, along wall, measured; RQD_v = 0-100%
12. ----
13. ----
14. normal and thrust faulting in vicinity
15. insignificant water inflows



Cases 93 and 94

1. Höljes Hydroelectric, central Sweden
2. tailrace tunnel
3. W = 12.6m
4. H = 13.4m
5. A = 140m^2
6. extensive overbreak and roof falls
7. L = over one mile total
8. heavily fractured and altered rock; frequent clay seams up to 10 cm thick; kaolinite and montmorillonite alteration products; clay zones at contacts between different rock types; clay-coated and graphite-coated joint surfaces; up to several meters of clay products encountered in drill holes; amphibolite, diabase quartz porphyry, sericite alterations.
9. heavy shotcreting (up to 25 cm) and cast concrete arches
10. D $\leq$ 100m
11. RQD values of 0 to 20% have been estimated from drill logs and descriptions from geologist's notes
12. (V = 2500-3600m/sec)
13. (SVR = 2500/5000-3600/5000 = 0.5-0.64)
14. previous strong tectonic activity in this area, overthrusting and normal faulting
15. frequent inflows worsened stability by washing material out of wide, filled joints and seams
16. not observed by author, notes and descriptions from geologist's notes.

Case 95

1. Subway, Stockholm, Sweden
2. subway stations
3. W = 22m
4. H = 8m
5. A ≃ $160m^2$
6. stable
7. L = 100m
8. massive granite and gneiss, very few joints
9. no structural support required
10. D = 10-50m
11. RQD = 100% all directions, implied from personal communications
12. ----
13. ----
14. major normal faulting in region
15. ----
16. not observed by author

Case 96

1. Swedish Baltic Coast
2. underground naval storage chambers
3. W = 15m
4. H = 30m
5. A ≈ $400m^2$
6. stable
7. L = up to 500m
8. massive granite and gneiss
9. no structural support required for static loading
10. D = 30m
11. RQD = 100% all directions, implied from personal communications
12. ----
13. ----
14. normal faulting in region
15. ----
16. not observed by author

Case 97

1. underground rooms, large cities in Sweden
2. defense chambers, air raid shelters
3. W = 18m
4. H = 6m
5. A = 90-100m^2
6. minor roof falls and overbreak
7. L = 1000m
8. massive gneiss, occasional intersecting joints
9. rock bolting
10. D = 20m
11. RQD = 100% all directions, implied from personal communications
12. ----
13. ----
14. ----
15. ----
16. not observed by author

TABLE B.1

CHECK LIST ROCK MASS CLASSIFICATION FOR FIELD CASES

	Case Number
	Intact Rock Strength
1	Sound
2	Altered or weathered
	Rock Mass Structure
3	Massive, no or very few discontinuities
4	One discontinuity set
5	Two discontinuity sets
6	Three discontinuity sets
7	Random discontinuity (W), Crushed (C), or Earthlike (E)
	Average Discontinuity Spacing
8	Less than 5 cm (2 in.)
9	5 cm - 30 cm (2 in. - 1 ft)
10	30 cm - 1 m (1 ft - 3 ft)
11	1 m - 3 m (3 ft - 10 ft)
12	Greater than 3 m (10 ft)
13	Discontinuity Tightness Tight (T), Open (O)
14	Joint Continuity Continuous (C), Discontinuous (D)
	Discontinuity Type
15	Joint
16	Bedding Plane
17	Cleavage or schistosity
18	Fault, shear "sköl"
	Discontinuity Filling or Coating
19	None
20	Non-softening clay
21	Softening clay
22	Other low friction material
23	Sandy or gravelly material, rock fragments
24	Alteration along joints
	Degree of Discontinuity Planeness (Intermediate Scale)
25	Plane
26	Curved
27	Irregular
	Degree of Discontinuity Roughness
28	Slickensided
29	Smooth
30	Rough
	Dip of Discontinuities
31	$0 - 30^{o}$
32	$30 - 60^{o}$
33	$60 - 90^{o}$
	Strike of Discontinuities
34	$0 - 30^{o}$
35	$30 - 60^{o}$
36	$60 - 90^{o}$
37	Primary RQD, percent
38	Instability None (N), Wall (W), Roof (R), Both (B)

TABLE B.1. Continued

Case Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1	x	x	x			x	x	x	x	x		x				x	x			x
2				x	x						x		x	x	x			x	x	
3																				
4				x	x	x	x	x			x	x					x			x
5	x	x											x	x	x				x	
6			x						x	x						x				
7							W					W	W	W	W	W		E	W/E	
8																		x		
9			x	x	x	x	x	x	x	x	x					x			x	x
10												x		x						
11	x	x											x		x		x			
12																				
13	O	O	T	O	O	T	T	T	T	T	O	T	O	O	O	T	T	O	O	T
14	C	C	C/D	C	C	D	C/D	D	D	C/D	C	C	C/D	C	C	C/D	D	C	C	C
15	x	x	x			x	x	x	x	x		x	x	x	x	x	x	x	x	x
16				x	x						x									
17																				
18	x	x									x		x	x	x			x	x	
19			x			x		x	x	x						x	x			x
20	x	x		x			x				x	x	x	x	x					
21																		x	x	
22																				
23					x													x	x	
24	x	x		x	x						x									
25	x	x	x		x		x	x	x	x	x						x			x
26				x												x				
27						x						x	x	x	x			x	x	
28	x	x		x			x				x	x	x	x	x			x	x	
29			x		x	x	x	x	x	x		x				x		x	x	x
30	x	x															x			
31														x	x	x				
32			x				x	x		x								x	x	
33	x	x	x	x	x	x	x		x	x	x	x	x	x	x	x	x			x
34	x	x	x	x		x	x		x	x		x	x	x	x		x			x
35			x				x	x		x	x		x			x		x	x	
36	x	x			x				x							x				
37	70	90	60	50	80	60	70	70	80	60	20	100	80	70	80	60	100	10	30	70
38	R	R	W	W	B	N	W	N	R	W	W	W	W	W	W	W	W	W	W	N

TABLE B.1. Continued

Case Number	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
1	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
2																				
3	x																			
4			x						x	x					x	x				
5		x		x	x	x	x	x			x			x			x	x		x
6												x	x						x	
7		W	W	W	W			W		W					W		W	W		W
8										x					x				x	
9			x	x	x			x					x	x		x	x	x		x
10						x	x				x	x								
11		x							x											
12	x																			
13	T	O	O	O	T	T	T	O	T	O	T	T	O	T	T	T	O	O	O	O
14	C	C	C	C/D	C/D	C/D	C/D	C	D	C	C	C/D	C/D	C	C/D	C/D	C	C	C	C
15		x	x	x	x	x	x				x	x	x	x	x	x	x	x	x	x
16																		x		
17							x	x	x	x										
18			x	x	x			x		x			x						x	x
19	x					x	x		x	x	x	x		x	x	x		x	x	
20			x	x									x				x			
21																				x
22								x												
23			x	x	x															
24		x																		x
25	x			x		x	x				x	x	x	x				x	x	x
26			x																	
27		x	x		x			x	x	x					x	x	x	x		
28			x	x	x			x		x			x				x			
29			x	x	x	x	x			x	x	x	x	x	x	x		x	x	x
30	x	x							x									x		
31		x	x	x	x		x			x	x	x	x	x				x	x	
32				x		x		x	x								x			
33	x	x	x		x	x	x	x			x	x	x	x	x	x	x	x	x	x
34		x	x	x	x	x	x	x		x	x	x	x				x	x	x	
35	x				x	x			x					x			x	x		x
36											x	x	x		x	x			x	x
37	100	100	30	60	70	80	90	40	90	10	100	80	40	90	0	20	20	60	20	50
38	N	R	R	R	R	R	N	R	N	R	R	R	R	W	N	N	R	R	B	B

TABLE B.1. Continued

Case Number	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60
1			x	x	x	x	x	x	x	x	x	x	x	x	x			x	x	x
2	x	x														x	x		x	
3																				
4	x		x				x	x	x	x	x	x		x	x					x
5		x		x	x	x							x					x	x	
6																				
7		W		W					W					W		C/E	C/E		E	
8				x			x				x	x								
9	x	x	x			x		x	x	x			x		x	x	x		x	x
10														x						
11					x													x		
12																				
13	O	O	O	T	O	O	O	O	O	O	O	T	O	O	O	O	O	O	O	O
14	C	C	C	C	C	C	C	C	C	C	C	C/D	C/D	C	C	C	C	D	C	C
15		x		x	x	x					x	x	x	x	x	x	x	x	x	x
16	x	x																		
17				x	x	x	x	x	x	x			x							
18		x	x		x		x	x	x	x	x		x	x	x	x	x		x	x
19				x		x						x						x		
20								x		x	x		x			x	x		x	x
21	x	x	x		x		x		x					x	x					
22								x												
23															x	x	x			
24	x	x												x		x	x		x	
25		x	x		x	x	x	x	x	x	x	x	x	x	x			x	x	x
26																				
27	x	x		x												x	x			
28		x	x				x	x	x	x	x		x		x	x	x		x	x
29	x			x	x	x						x		x						
30																		x		
31	x	x							x						x					
32			x		x	x	x	x		x	x	x	x			x	x	x		x
33		x		x	x	x								x		x	x		x	
34	x	x			x									x	x	x	x	x		
35	x		x	x		x						x	x			x	x			
36		x		x			x	x	x	x	x					x	x		x	x
37	60	40	20	0	90	30	0	10	30	40	0	0	50	70	80	10	40	100	50	80
38	R	R	R	R	R	R	R	R	R	R	R	N	R	R	R	B	B	R	R	R

TABLE B.1. Continued

Case Number	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
1	x	x	x	x	x	x	x	x	x	x				x	x	x	x	x		x
2											x	x	x						x	
3								x									x			
4							x			x			x	x	x			x		x
5	x	x				x			x		x	x				x				
6			x	x	x															
7	W	W									W	W	W			W			E	W
8																			x	
9						x	x			x										x
10	x	x									x							x		
11			x	x	x				x			x	x	x	x	x				
12																	x			
13	O	O	T	T	O	O	T		O	T	O	O	O	T	T	O	T	O		O
14	C/D	C/D	C	C	C	C	C	-	C	C	C	C	C	C	C	C	C/D	C	-	C
15	x	x	x	x	x	x	x		x	x	x	x	x	x	x	x	x	x		x
16																				
17																				
18	x	x			x				x		x	x	x			x				x
19			x	x			x			x				x	x		x	x		
20		x			x											x				x
21	x								x		x	x	x							
22					x											x				
23							x													
24		x							x		x	x	x							
25	x		x	x	x	x	x		x	x				x	x	x	x	x		x
26																				
27		x									x	x	x			x				
28	x	x			x				x		x	x	x			x				x
29	x		x	x			x							x	x					x
30						x				x							x	x		
31		x	x	x	x	x					x	x	x					x		
32																x				
33	x	x	x	x	x	x	x		x	x	x	x	x	x	x	x	x			x
34		x	x	x	x	x	x		x					x	x	x				
35				x					x		x	x	x			x	x			x
36	x	x			x					x	x	x	x					x		
37	70	90	100	100	90	70	20	100	90	40	30	70	80	100	100	70	100	90	0	50
38	R	R	R	R	R	R	R	N	B	N	B	B	B	N	W	B	N	N	B	R

TABLE B.1. Continued

Case Number	81	82	83	84	85	86	87	88	89	90	91	92
1	x	x	x	x		x	x	x	x	x	x	x
2					x							
3							x					
4									x	x	x	
5	x		x			x		x				x
6		x		x								
7	W			W	C/W				W			
8												
9	x		x	x	x			x			x	x
10		x							x	x		
11						x						
12							x					
13	O	T	T	O	O	O	T	O	T	O	T	T
14	C	C	C/D	D	C	C	D	C	C	C	D	C/D
15	x	x	x	x	x	x	x	x	x	x	x	x
16												
17												
18	x				x	x		x		x		
19		x	x	x			x				x	x
20	x				x	x		x	x	x		
21												
22												
23	x											
24	x				x			x				
25	x	x	x	x		x	x	x	x	x	x	x
26												
27					x							
28					x	x		x				
29	x	x	x	x	x				x			
30					x	x	x			x	x	x
31	x	x	x					x	x			
32	x			x	x	x				x		x
33	x	x	x	x	x		x				x	x
34		x		x				x	x	x		
35				x		x						
36	x	x	x	x	x		x				x	x
37	30	80	70	60	30	90	100	50	80	90	90	30
38	R	B	N	R	B	B	N	R	R	R	N	R

APPENDIX C

LABORATORY MODEL TEST

C.1 Purpose and Approach

The purpose of the laboratory model study was twofold:
(1) to investigate the machanism of loosening-type failures around unsupported openings in jointed rock, and
(2) to demonstrate the influence of joint configuration, joint orientation, joint spacing/span width, joint friction, and horizontal stress on the stability of an opening in jointed rock.

Rather than attempt to model a specific tunnel, subjected to the idealized boundary conditions that are thought to exist around a real tunnel, it was decided to investigate the general behavior of an unsupported mass of jointed rock, subjected only to loosening type failures. The very simple model that has been constructed represents a hypothetical unsupported span in jointed rock. The information that is obtained from the model is strictly of a qualitative nature and no attempt is made to scale the results to the tunnels observed in the field.

C.2 Previous Work

Experimental studies of loosening-type instability behavior in rock tunnels are very limited and have involved mainly relatively simple laboratory models of hypothetical tunnels in jointed rock.

The simplest form of tunnel model in jointed rock, subjected only to gravity loading where failure is by loosening, that has been used is the type of model built of individual blocks of non-rock-like material such as sugar cubes or toy blocks. These models generally demonstrate only qualitatively, but often very effectively, the nature of loosening failures. Although sugar-cube-type models have been used for class room and lecture demonstrations, only a few such models have been reported in the literature. Trollope's plastic cube model (Trollope, 1966) very vividly demonstrates loosening in jointed media, but does not show the influence of such rock mass parameters as joint friction, joint orientation, and state of natural stress on the stability of an opening in jointed rock.

Lang (1964) reports the use of several simple Lucite block models to show qualitatively the application of rock bolts for support in the roof and abutment of an underground power house excavation. These experiments included a capacibity for rock mass loading but, like Trollope's model, they were not intended to demonstrate the influence of varying rock mass parameters on stability. Nor do either Trollope's or Lang's model scale any of the important rock properties, such as strength or density.

C.3 Hypothetical Model for Loosening-Type Behavior

Because the purpose of the model study was to investigate the general behavior of an unsupported mass of jointed rock, subjected only to loosening type failures, it was decided to design a very simple model that represents an unsupported tunnel in jointed rock only in a hypothetical way.

The basic model that has been designed and constructed is shown schematically in Figure C.1.
A 14 row x 19 column mass of model rock blocks is supported on its bottom edge by fixed supports at the ends of the span and by trap doors over its center 11 columns. Uniform lateral and vertical pressures are applied over the sides and top om the mass.

The behavior of the unsupported mass is observed when the trap doors under the center of the span are dropped. If failure does not occur immediately, the lateral pressure is reduced until failure occurs. The lateral pressure, σ_h, in the model is intended to correspond to the tangential stress in the roof of a tunnel.

The principle of the model is similar to that of Terzaghi's trap door experiments in sand (Terzaghi, 1936). However, the vertical movement of the trap door supports in the rock block model cannot be controlled as was done in Terzaghi's experiments

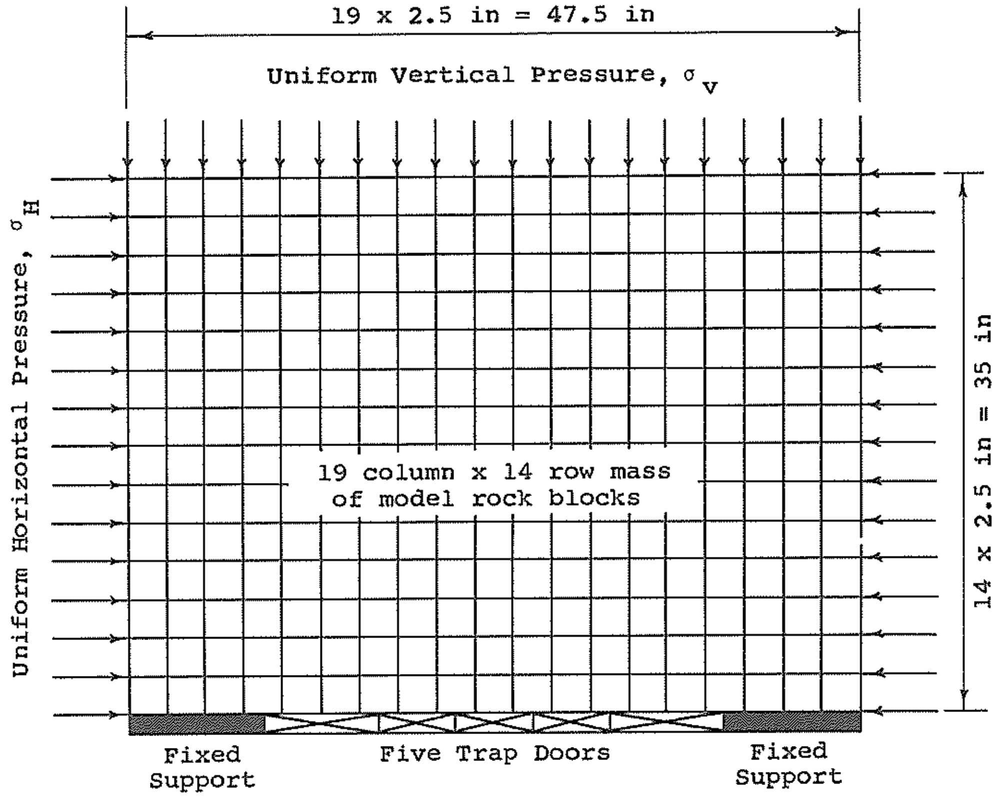


Fig. C.1 Schematic Diagram of Model

with sand. Thus the only conditions of vertical support in the block model experiments are supported and unsupported. The trap doors are not completely removed, however, and as failure progresses upwards in the block mass, a partial degree of support is acquired, as will be shown later.

The primary information desired from the model tests was the stability condition of the unsupported block mass (stable or unstable) and the mechanism of failure. It was not the purpose to obtain quantitative information about the stress-strain distribution in the block mass.

The geometry and boundary conditions of the model do not represent those of a tunnel in an infinite medium, and hence it is to be expected that the stress distribution and resultant behavior of the rock mass do not correspond precisely to those around a real tunnel. However, it is felt that the same parameters govern the behavior of both models and the effects of variations of these parameters are the same for both.

Another model, in which a tunnel is excavated in the center of a large mass of blocks, was considered. However the large size of block mass required for a reasonably small joint spacing/span width ratio (less than 1/5-1/10) prohibits practical construction.

A plane stress loading condition was selected for the model because of the relative simplicity of loading the model and the ease of viewing progressive failure as the loading is changed.

C.4 Selection of a Rock-Like Model Material

Because the behavior of tunnels in jointed rock at shallow depths is controlled mainly by joints and other geologic discontinuities, and because intact rock properties have a seemingly minor influence on loosening type failures, it was originally considered sufficient to construct a model that would simulate only the physical geologic structure of a rock mass. To this end, a number of different block-type objects were considered for the model, such as small model bricks, toy building blocks, short pieces of square steel or aluminum bar, and sugar cubes. However, after further consideration of the possible influence of intact rock properties on loosening behavior, such as crushing of highly stressed corners, and after consideration of the non-rock-like properties of the previously considered block objects (particularly joint shear strength), it was decided that an attempt should be made to use simulated rock blocks whose intact mechanical properties resemble those of a rock-like material.

It was very fortunate that two very extensive investitations into rock-like model materials had just been completed at the time the author began his model study. A material developed at the University of Illinois (Heuer, 1966) and another at the Missouri River Division Laboratory of the Corps of Engineers (Rosenblad, 1967) have properties that are reasonable good simulations of some rock-types. The choice of model materials was narrowed to these two materials, not because they closely resemble the rock types from the field studies, but rather because they are the only known materials that model all the significant mechanical properties of real rock, particularly the angle of internal shearing resistance. The physical and mechanical properties of the Heuer and Rosenblad model materials, together with the same properties for a typical sound granite, are shown in Table C.1.

Rosenblad's model material was chosen for the writer's block model study because the technique of casting this material into very convenient block sizes (2.5 in. x 2.5 in. x 6 in.) had already been developed at the same institution where the writer was working. At the time of initiation of the writer's work, Heuer's material had been used only in large pieces for study of intact rock behavior around a tunnel, whereas Rosenblad had specifically developed his technique for the study of jointed rock masses. Since that time, Heuer's model material has been used in jointed rock model studies and would be just as suitable for the type of studies undertaken by the writer, although the casting technique used by Rosenblad produces more uniform block dimensions than the sawing technique used in the continuation of Heuer's work.

Rosenblad's model material consists of the following

TABLE C.1

PHYSICAL AND MECHANICAL PROPERTIES OF TWO ROCK-LIKE MODEL MATERIALS

Property	Heuer Material	Rosenblad Material	Typical Values for Sound Granite
Density, pcf	117	120	160
Unconfined compressive strength, q_u, psi	555	610	25,000
Tangent modulus of elasticity at 50 percent of q_u, psi	0.60×10^6- 1.37×10^6	1.5×10^6	7×10^6
Poisson's ratio, ν	0.25	0.15	0.15
Tensile strength	$0.06\, q_u$	$0.14\, q_u$	$0.069\, q_u$
Strain at failure in q_u test, %	0.055-0.15	0.085	0.35
Angle of internal friction for intact material (triaxial tests)	32.5^o	49.5^o 26.5^{o*} (200 psi)	55^o
Angle of shearing resistance for cast surface (direct shear)	--	39.5^o	31^o (Smooth Natural Joints)
Angle of shearing resistance for sawed surface	--	31.5^o	28^o

* Two component Mohr envelope, normal pressure (psi) at intersection of two straight line components is given in parentheses.

components (percentages by weight):

Valley, Nebraska river sand	76%
Hydrocal B-11 plaster	10%
Water	14%

A concrete stick vibrator is used to vibrate the mix into plexiglass molds.

C.5 Model Blocks

It was originally hoped that Rosenblad's model blocks could be used directly for the writer's investigations, but it was discovered that the variation in size of these blocks was as great as 1/8 in. When stacked into a large mass of 100 blocks or more a total misalignment of up to 1/2 in. was not uncommon, and could be avoided only with the utmost care in selection of blocks. Because the model of interest to the author would involve over 200 blocks in more than 10 rows, and because a uniform fit of the blocks was considered very important, it was decided to construct new block molds that would be capable of producing very uniform, square blocks.

Five, five-gang molds were very carefully milled from plexiglass. It was found that the nonuniformity of thickness of stock plexiglass plate was the greatest cause of the nonuniformity in Rosenblad's blocks. All of the plexiglass plate used for the writer's mold was milled to close tolerance ($\pm$ 0.003 in.) and the molds were purposely made small (five-gang) so that control of block size would be easier. The finished average cross-sectional edge dimensions for the cast blocks vary by no more than 0.008 in.

The surface frictional characteristics of the cast blocks that have been used by Rosenblad are given in

Table C.1. Because Rosenblad's direct shear tests were conducted at relatively high normal pressures (≥40 psi) and because the expected block failures in the writer's model would probably occur at very low normal pressures, a number of simple, sliding block experiments were conducted on an inclined plane. The average value of the angle of sliding friction found from 45 tests for the cast block surfaces is 29°. The same figure for sawed block surfaces is 34°. For the same properties, Rosenblad got 39.5° and 31.5° in direct shear tests and 41° and 34° in triaxial tests on jointed specimens. Rosenblad attributes the lower values of his "thin-skin-removed" (same as the writer's sawed surface) tests to the roller action of individual sand grains on the joint surfaces.

In the writer's tests, the higher friction angles for the sawed surfaces, in comparison to the cast surfaces, are attributed to a Patton "i" effect at low normal stresses, that is, individual sand grains act as asperities. In the writer's inclined plane tests on cast surfaces, there are no sand grains to act as asperities.

C.6 Model Apparatus

The model apparatus consists of a load frame that supports and loads the mass of 266 model rock blocks as shown in Figure C.1. An overall view of the apparatus is shown in Figure C.2. Front and end view drawings of the apparatus are shown in Figure C.3. The pressure boxes that apply the uniform pressure to the sides and top of the mass of blocks are built on to the two steel channels that form the vertical legs of the apparatus.

A detailed cross-sectional view of the pressure boxes is shown in Figure C.4. The pressure boxes are filled with water and activated through a partially water-filled tank to which air pressure is applied. The pressure boxes have been tested to pressures of 30 psi. Most of the tests, however, have been conducted at much lower pressures where the pressure gradient due to the column of water in the vertical pressure boxes strongly influences the resultant pressure on the side of the model.

A detailed cross-sectional view of the trap door assembly is shown in Figure C.5. The fixed supports and trap doors along the bottom of the apparatus are covered with teflon sheeting. Another layer of teflon sheeting and a 1/4 in.-thick rubber pad are placed across the supports and trap doors before the first row of blocks is placed in the apparatus. The purpose of the two teflon layers is to reduce friction between the bottom row of blocks and the test frame. The purpose of the rubber pad is to provide a uniform bearing for the blocks. Two teflon sheets are also placed between the edges of the block mass and the vertical pressure boxes.

Most of the tests have been conducted at pressures of less than 10 psi. Unfortunately, it is not possible to measure strains in the model at such low stress levels. It is believed, however, that the loading system employed gives as uniform a loading on the edges of the model as can be obtained. The 1/16-in.-thick rubber membrane of the pressure box deforms very easily under low pressures (less than 0.5 psi) and is able to "follow" the deformation of the block mass.

C.7 Model Test Configurations

A number of different test configurations, including several with "excavated" openings in the middle of the block mass, were tried. The most consistent results were obtained with the trap door arrangement in which an unsupported span width of 11 blocks represents an unsupported opening in jointed rock.

Tests were conducted initially by applying both horizontal and vertical pressures to the perimeter of the block mass. However it was found that the vertical pressure tends to cause punch-type shear failures along the outermost vertical joints of the unsupported span. When tests were conducted without an application of vertical pressure, a completely different behavior was observed, namely a bending of the block rows, accompanied by very large pre-failure deformations. Because the bending usually extended to the upper row of blocks, it was concluded that the application of a uniform vertical pressure on the top row of blocks is an unrealistic boundary condition for those cases where large bending deformation occur.

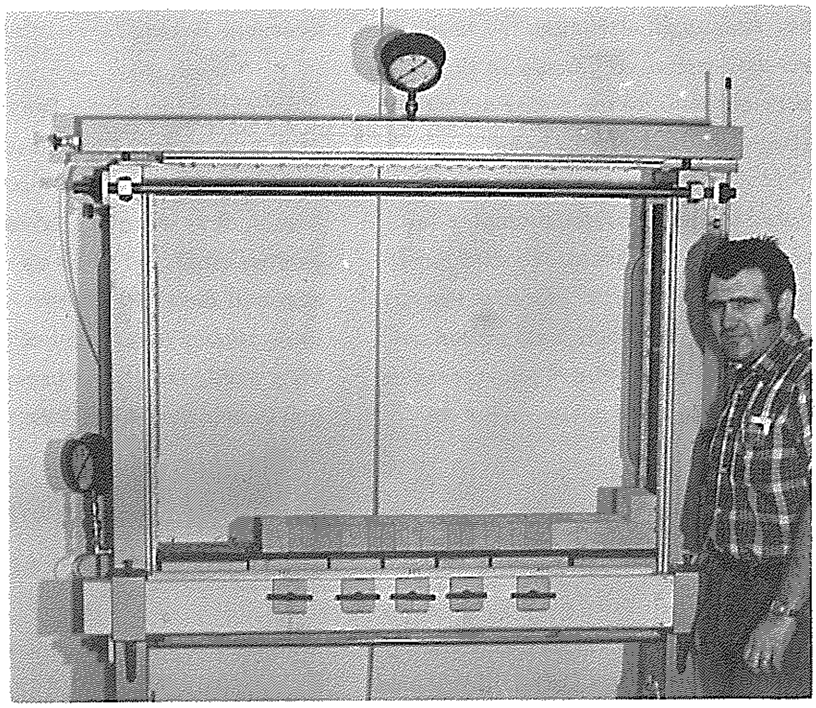


Fig. C.2 Model Loading Apparatus

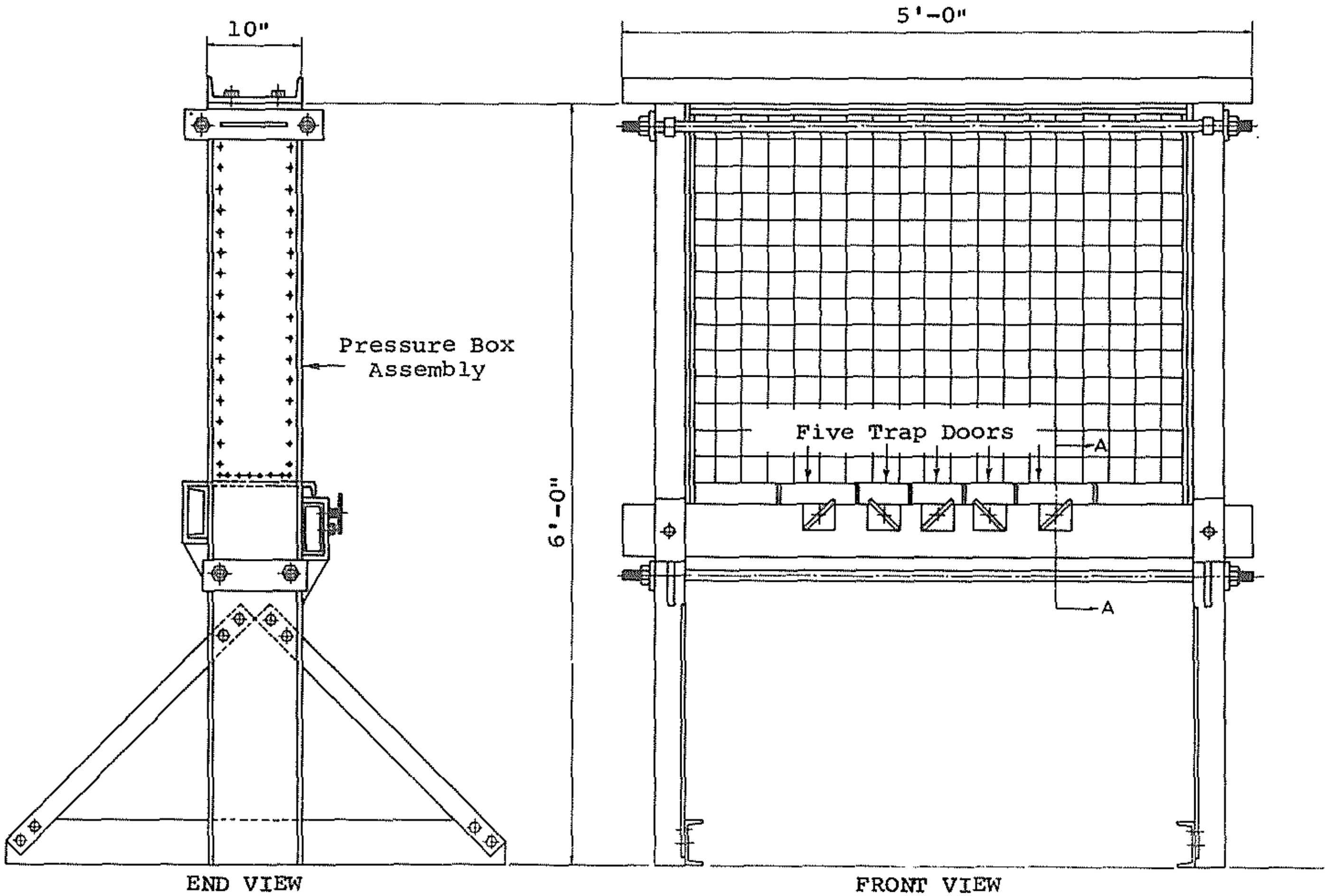


Fig. C.3 Front and End Views of Model Loading Apparatus

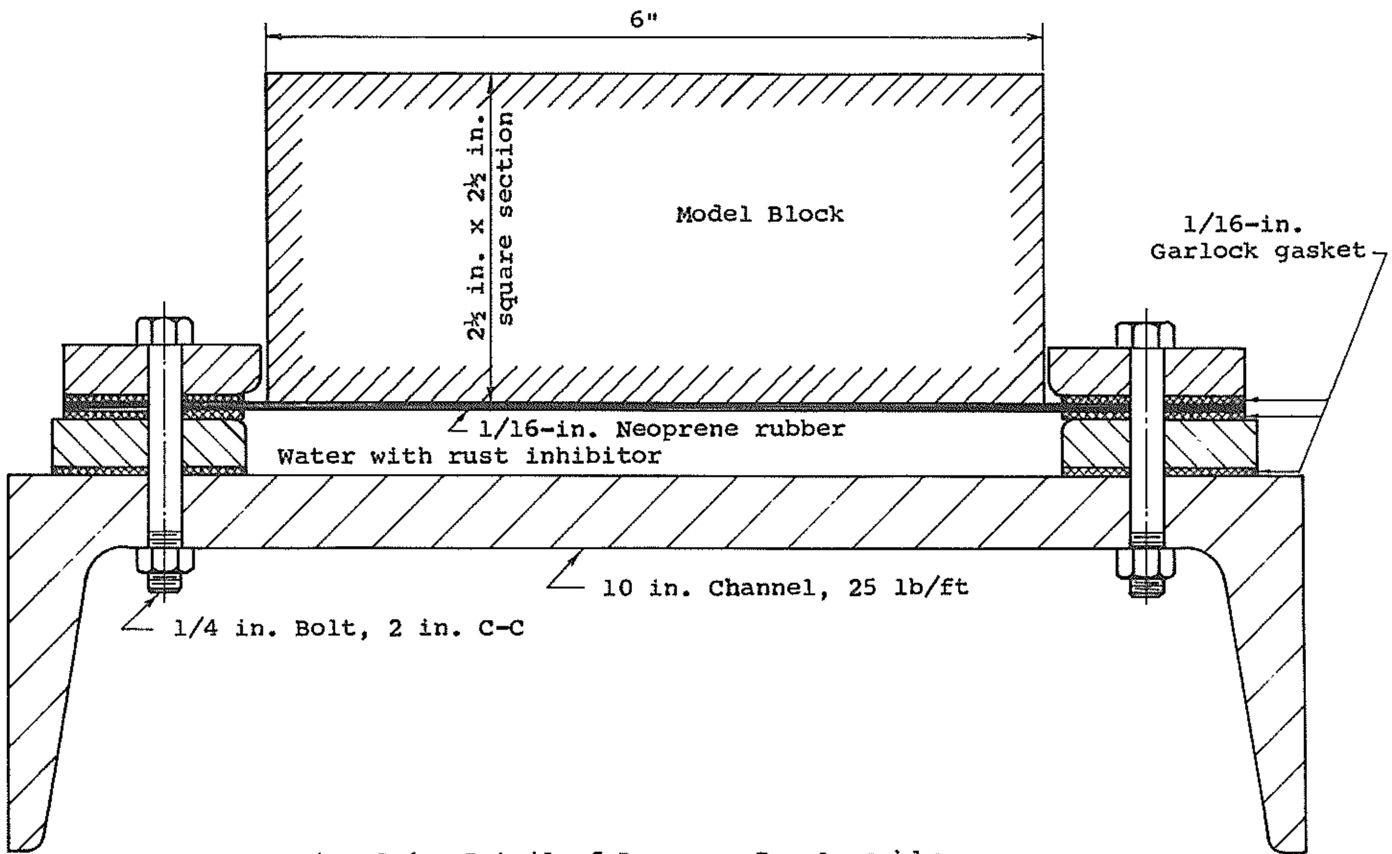


Fig. C.4 Detail of Pressure Box Assembly

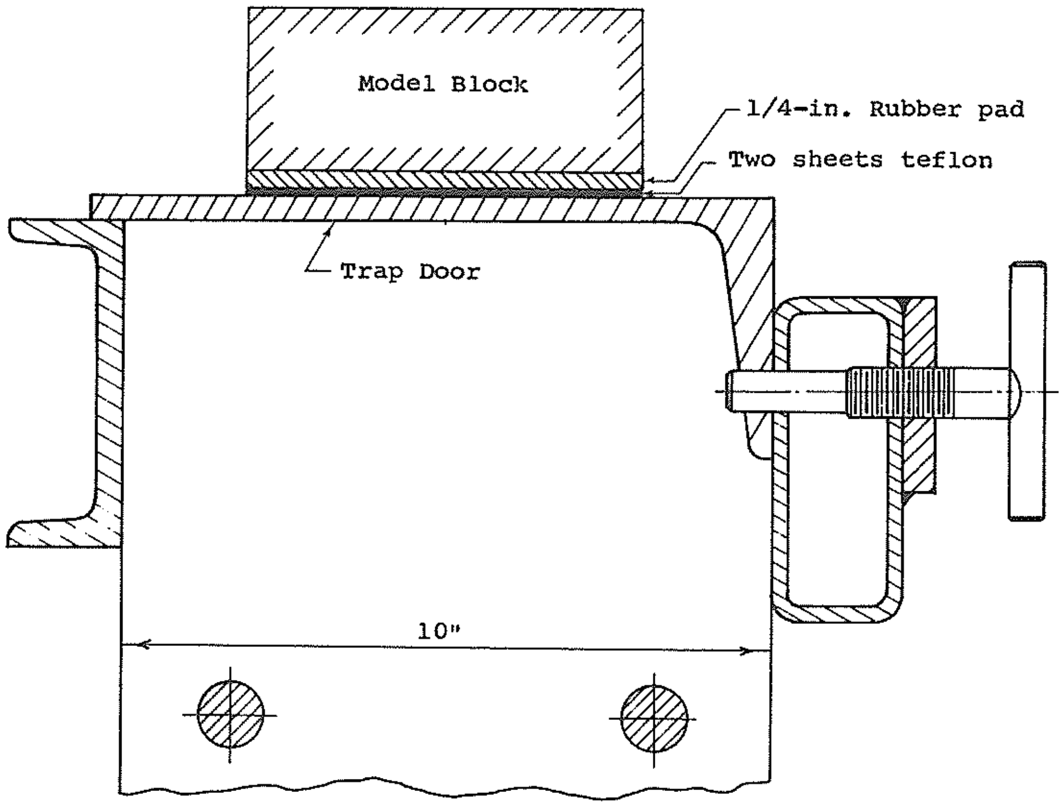


Fig. C.5 Detail of Trap Door Assembly
Section A-A, Fig. C.3

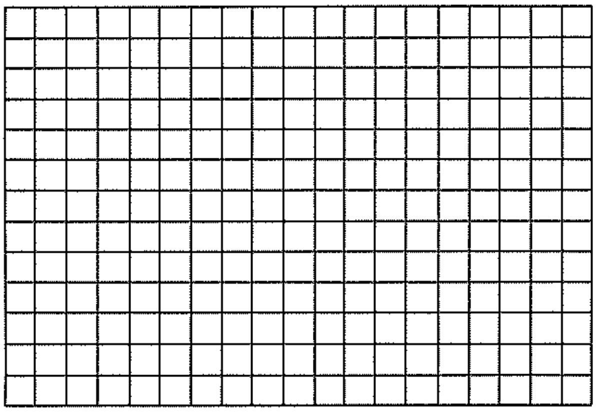


Fig. C.6 Non-Imbricated Block Mass

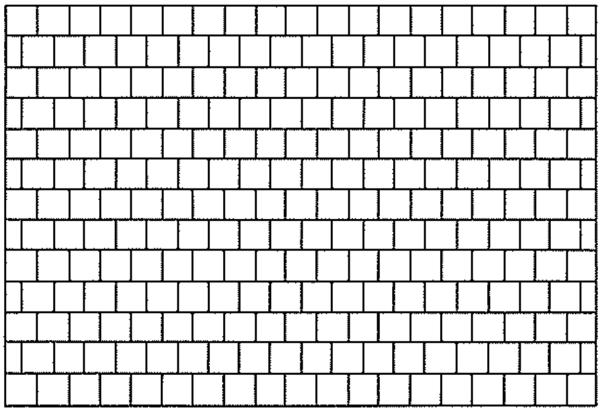


Fig. C.7 Imbricated Block Mass

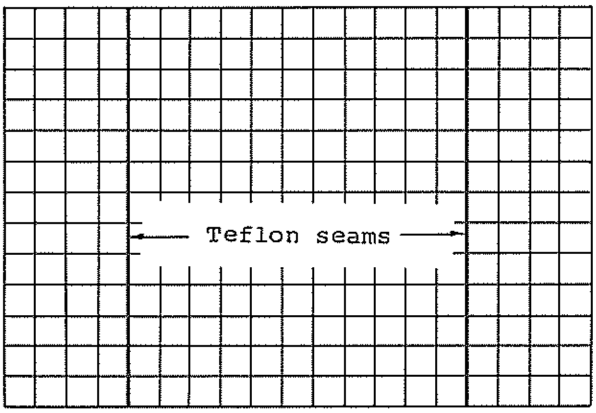


Fig. C.8 Two Low-Friction Vertical Seams in a Non-Imbricated Block Mass

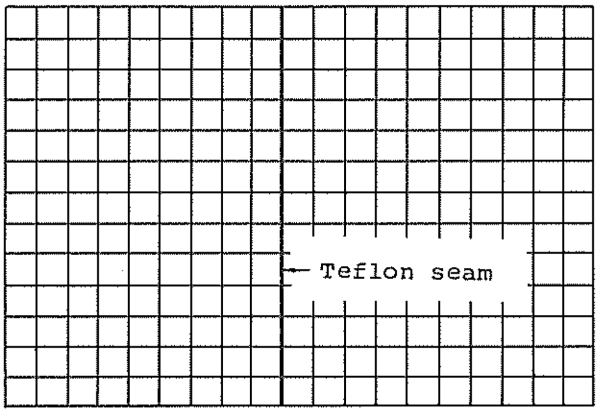


Fig. C.9 One Low-Friction Vertical Seam in a Non-Imbricated Block Mass

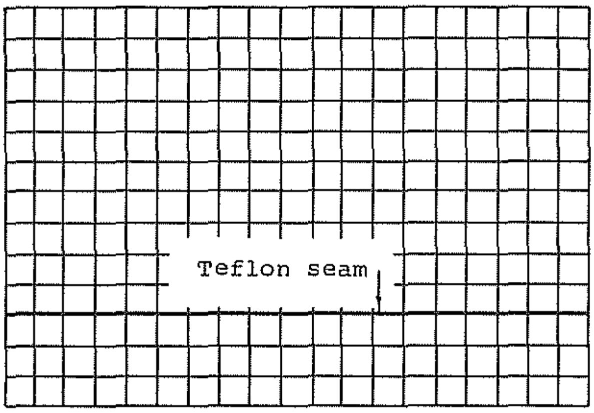


Fig. C.10 One Low-Friction Horizontal Seam in a Non-Imbricated Block Mass

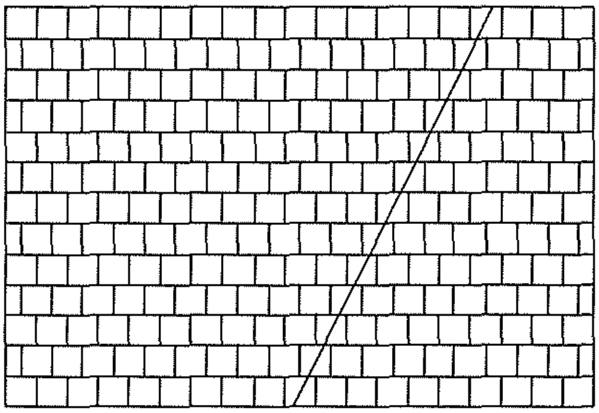


Fig. C.11 60° Joint in an Imbricated Block Mass

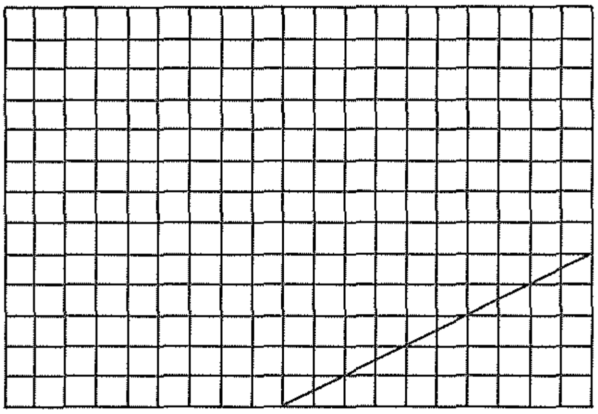


Fig. C.12 30° Joint in a Non-Imbricated Block Mass

Thus it is readily apparent that the conditions at the top edge of the model do not represent the conditions at an infinite distance from the edge of an opening. The final test procedure adopted was to apply a large horizontal pressure to the block mass, drop all five trap doors, and reduce the horizontal pressure until failure occurred.

All observations during the tests were recorded photographically. A grid of horizontal black lines was used to observe vertical deformations. No measurements of any kind were attempted, as the sole purpose of the model tests was to observe the failure mechanism and the effects of various parameter changes on the behavior of the model.

Tests were conducted with the block configurations shown in Figures C.6 - C.12. The first two configurations shown in Figures C.6 and C.7 are intended to yield information on two basic joint block configurations, namely non-imbricated and imbricated joint blocks. The configurations in Figures C.8, C.9, and C.10 are designed to show the effect of low joint friction on rock mass stability. The configurations of Figures C.11 and C.12 are designed to show the effect of non-orthogonally-oriented joints on the stability of a regularly, orthogonally jointed rock mass.

Because the vertical pressure is equal to the weight of the overlying blocks in all of the tests, variations of the horizontal pressure, σ_h, in each of the tests enabled the effect of the principal stress ratio, K_o, to be studied for each configuration.

C.8 Model Test Results

A total of 24 model tests was conducted. About one half of these tests were conducted for the purpose of testing the apparatus and determining the most suitable test procedures. A number of block configurations were tested several times to determine the reproducibility of the model behavior. Only the results from eight of the tests, representing those block configurations shown in Figures C.6 - C.12, will be presented and used in the discussion. Two tests, a five-block span and an eleven-block span, were run on the configuration shown in Figure C.6.

All of the test results are discussed with the aid of photographs. A complete set of about 20 photographs was taken for each test, but for brevity purposes only the most significant photographs for the test configurations are shown. The photographs for tests corresponding to the block configurations in Figures C.6 - - C.12 are shown in Figures C.13 - C.19 respectively. Results from the five-block span test corresponding to the plain, non-imbricated configuration of Figure C.6 are shown in Figure C.20.

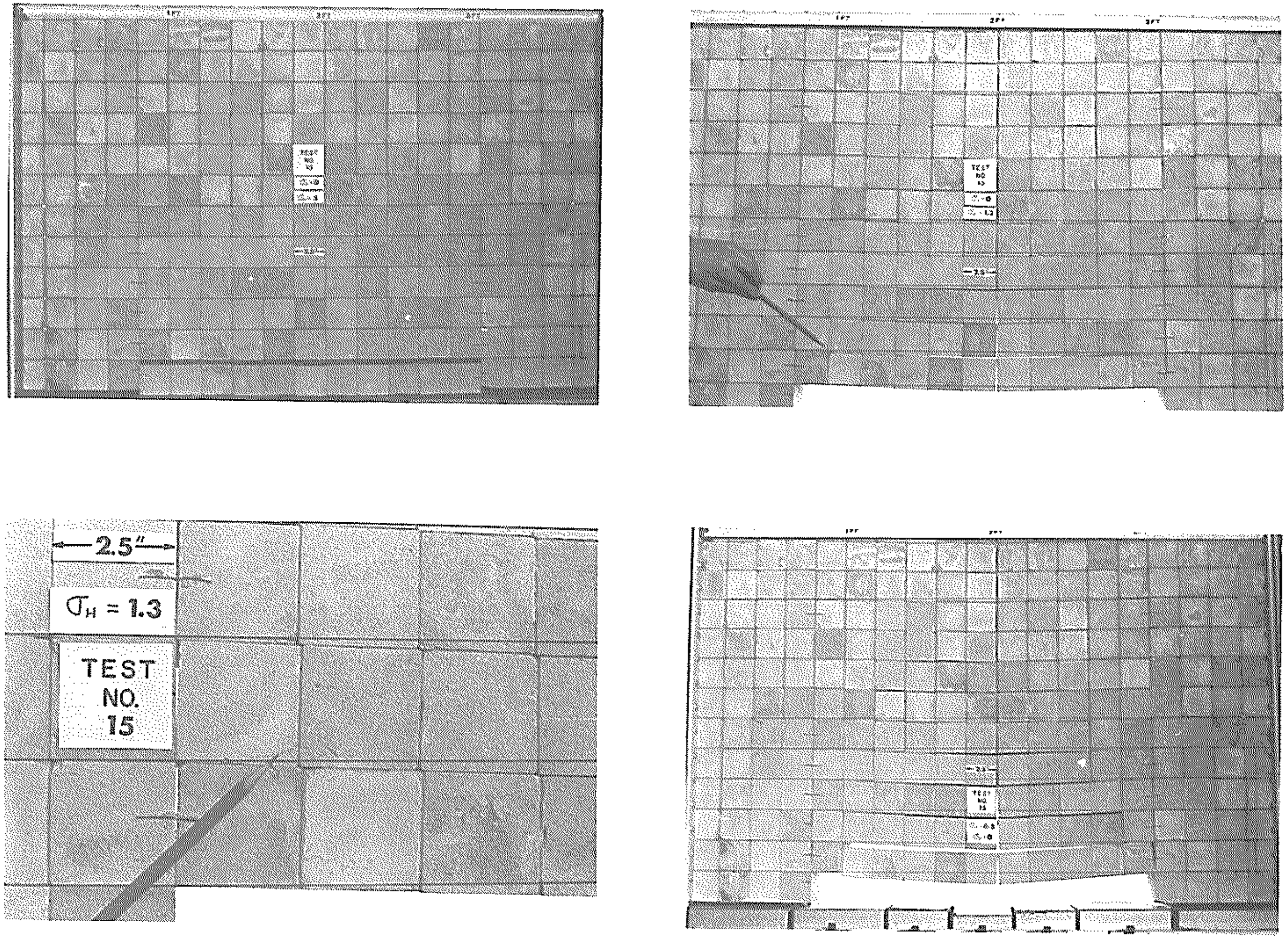
C.9 Discussion of Test Results

Only the results of tests on plain non-imbricated and imbricated block masses (Figs. C.6 and C.7) are discussed in detail in this appendix. The purpose is to describe the stability behavior of the model. An attempt is made in Chapter 3 to combine the results from all the model tests and the field observations and arrive at a general mechanism for the loosening instability behavior of unsupported openings in rock.

The photographs in Figures C.13 and C.14 show that the failure mechanism of unsupported spans in the model occurs progressively as the horizontal pressure is reduced. The behavior of the bottom row of blocks is generally very erratic, as the horizontal stresses along this row are not uniformly distributed. The bottom row generally fails by dropping out before significant bending of the row occurs.

Although it was not the intention in designing the model, the bottom-row of blocks corresponds somewhat to the destressed zone around a tunnel. The significant behavior of the model, the arching phenomenon, does not take place until the bottommost block row has slipped out. In most of the model tests the bottom row was removed after it had slipped out. This enabled complete freedom of movement of block row No. 1 (see Fig. C.13e), that is slipout of this row occurred before the vertical deformation due to bending was prevented by the presence of the trap doors.

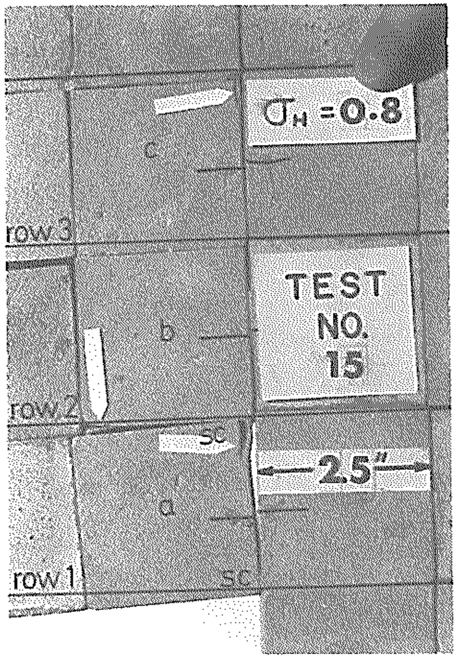
After the bottom row slips out, extremely large de-



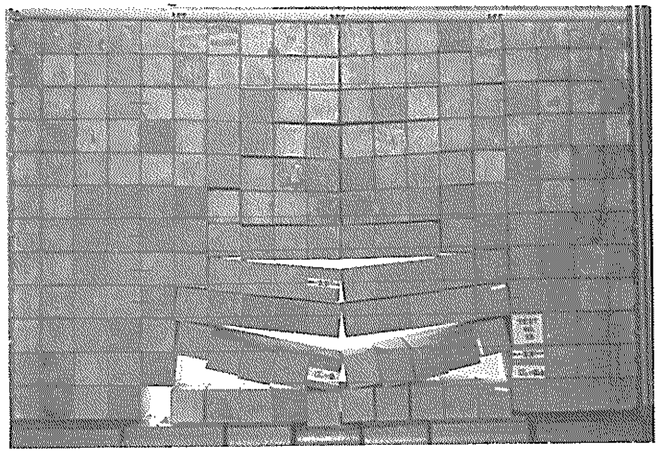
(c) σ_h = 1.7 psi; corner shearing caused by block rotation.

(d) σ_h = 1.2 psi; combined slip and rotation in a stable arched span.

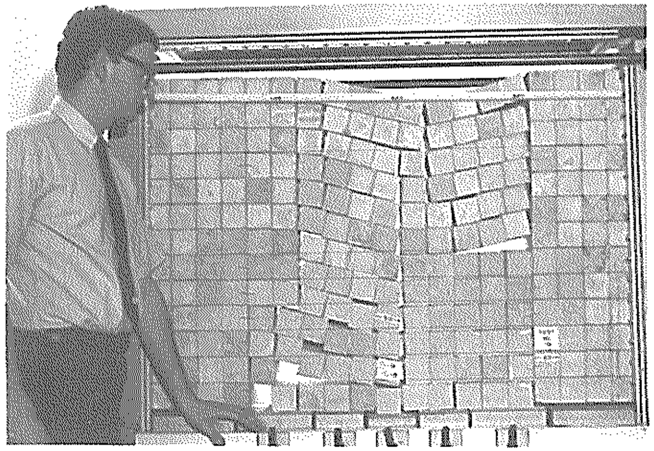
Fig. C.13 Loosening Failure in a Non-Imbricated Block Model



(e) σ_h = 1.2 psi; corner tearing caused by block rotation. Failure of corner in block "b" may have been caused by combined slip and rotation.

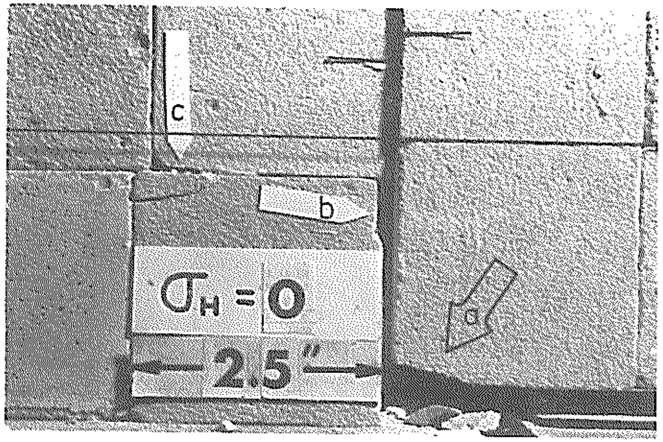


(f) σ_h = 1.0 psi; dome-shaped mass of loosened blocks above a partially supported span.

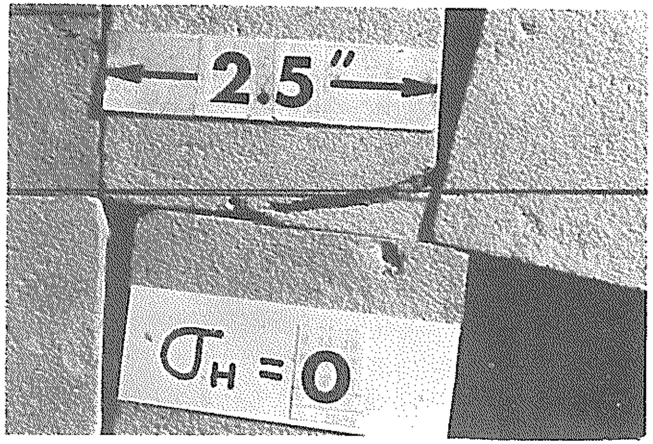


(g) σ_h = 0 psi; ultimate failure.

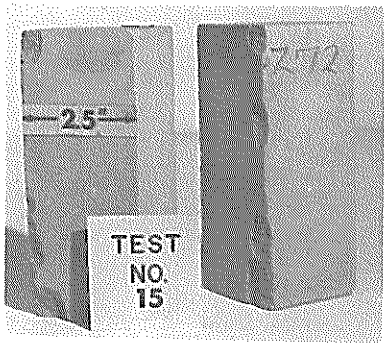
Fig. C.13 (Continued)



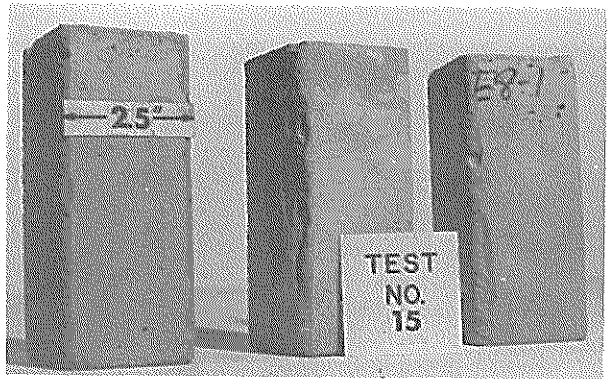
(h) Block slip caused by failure of corner in tension (black arrow at "a"). Arrow at "b" indicates tensile failure during rotation. Arrow "c" indicates shear failure.



(i) Combined tension and shear failure of corner caused by block slip and rotation.

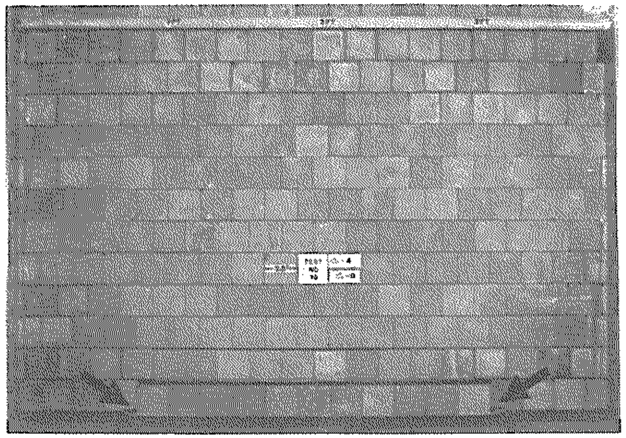


(j) Corner tearing caused by block rotation.

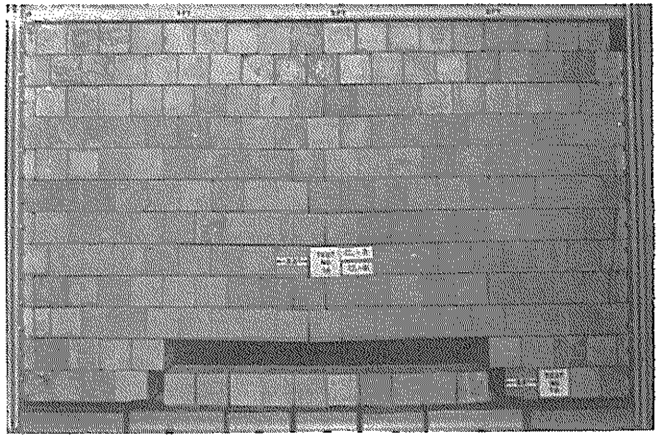


(k) Corner crushing caused by rotation.

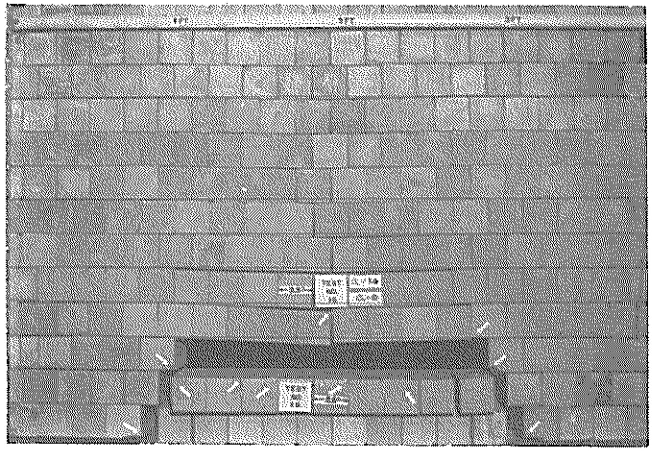
Fig. C.13 (continued)



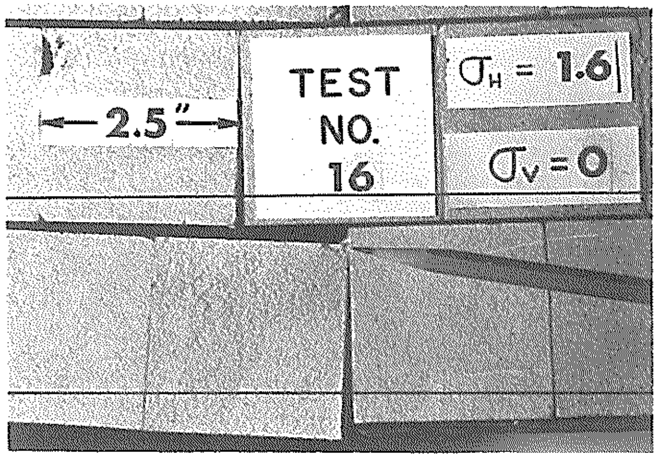
(a) σ_h = 4 psi; block row slippage caused by corner tearing (arrows).



(b) σ_h = 2 psi.

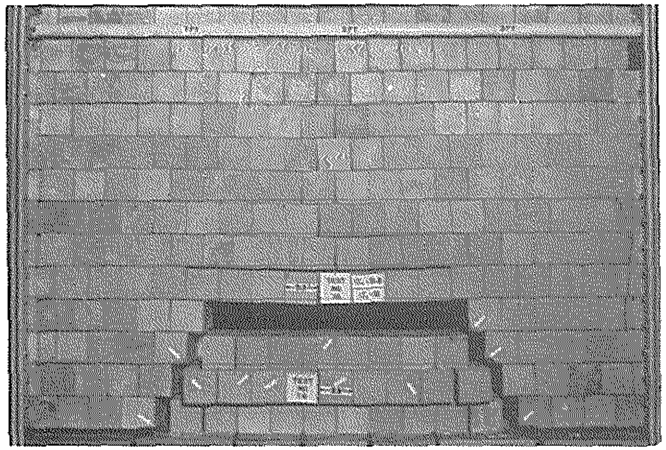


(c) σ_h = 1.6 psi; arrows at boundary of opening indicate torn corners.



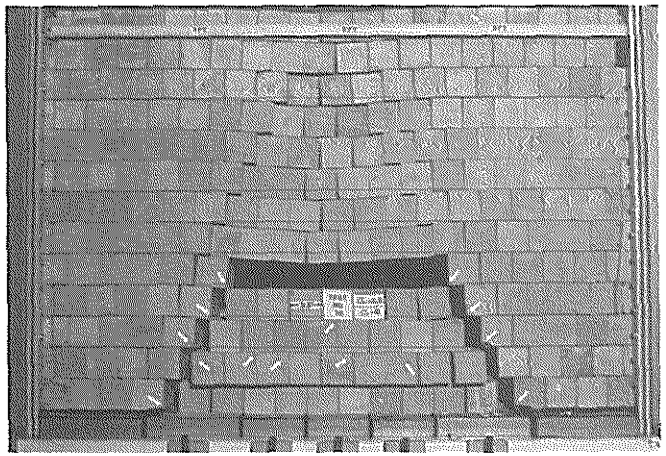
(d) σ_h = 1.6 psi; block slip at crushed corner in midspan of row.

Fig. C.14 Loosening Failure in an Imbricated Model

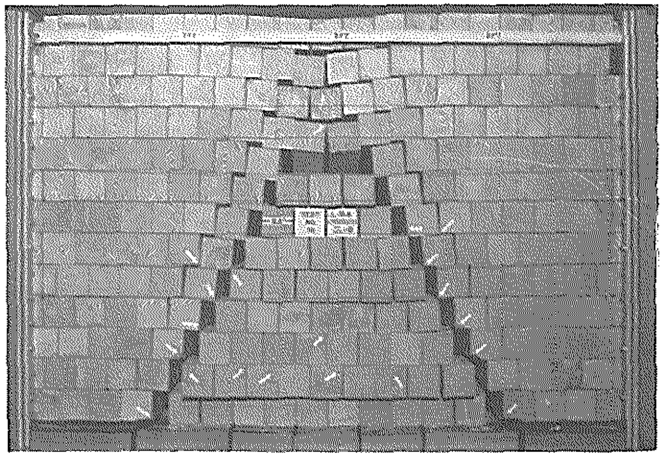


(e) σ_h = 1.2 psi.

(f) σ_h = 0.9 psi.

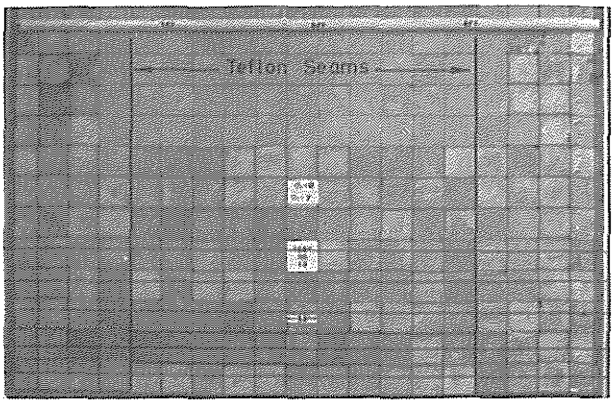


(g) σ_h = 0.9 psi.

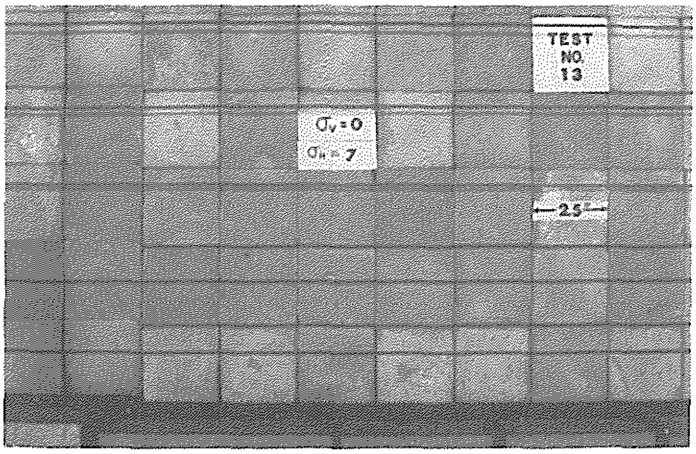


(h) σ_h = 0.4 psi.

Fig. C.14 (Continued)

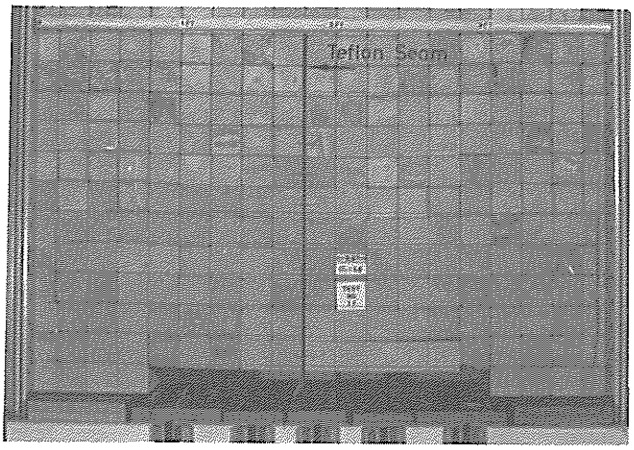


(a) σ_h = 7 psi; failure by pure slip along low friction seams; note absence of block rotation and block row bending.

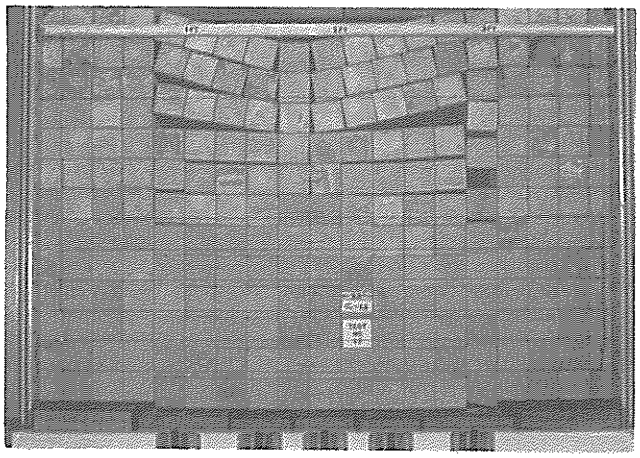


(b) σ_h = 7 psi; closeup; note absence of block corner failures.

Fig. C.15 Loosening Failure in a Non-Imbricated Block Mass with Two Low-Friction Vertical Seams.

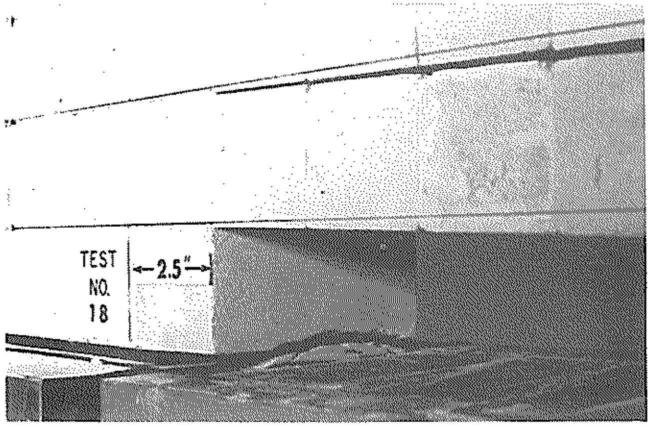


(a) σ_h = 1.5 psi; note slip of unsupported mass.

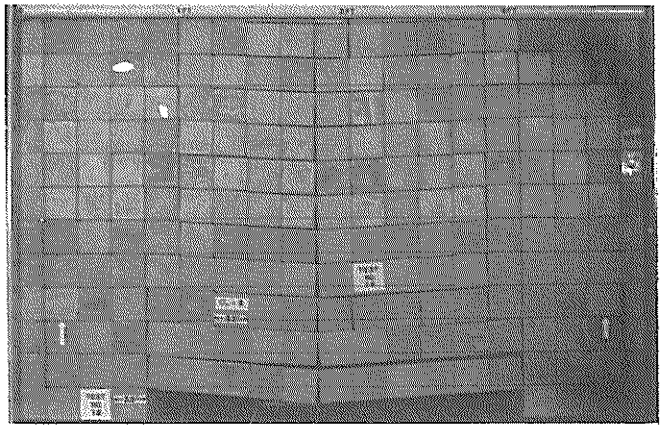


(b) σ_h = 1.5 psi; complete collapse of mass after minor block row bending.

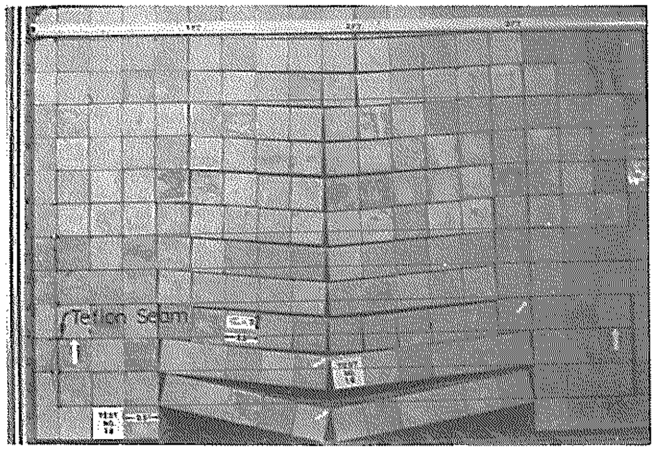
Fig. C.16 Loosening Failure in a Non-Imbricated Block Mass with a Single Low-Friction Vertical Seam



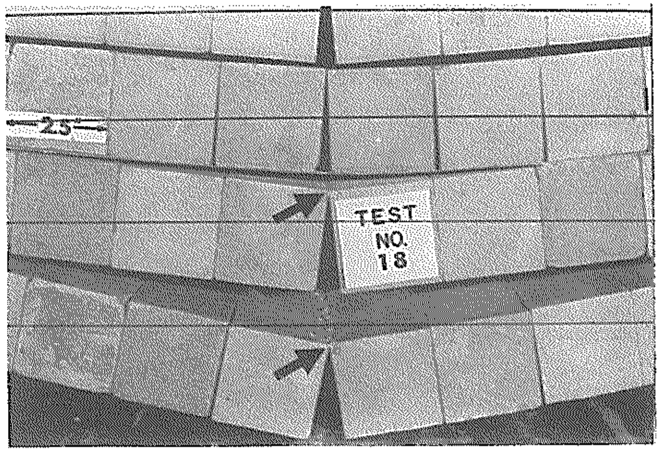
(a) σ_h = 2.5 psi; block row fallout caused by corner tensile failure.



(b) σ_h = 2 psi.

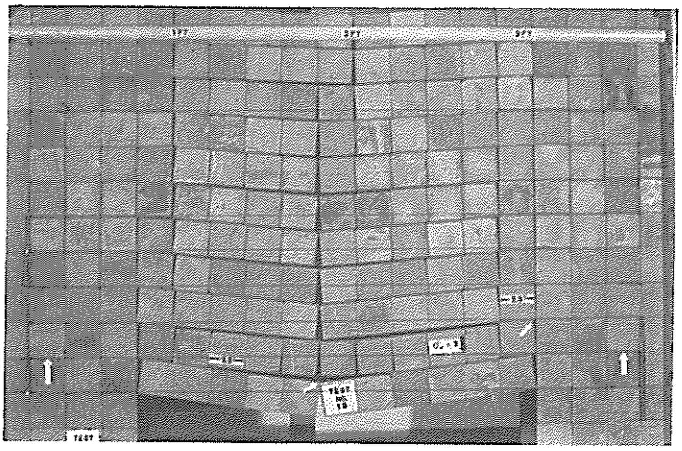


(c) σ_h = 2 psi; block row failure caused by rotation and corner crushing.

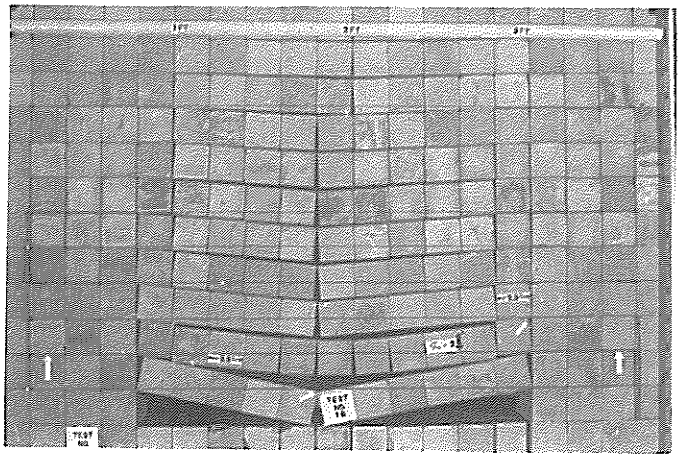


(d) σ_h = 2 psi; closeup of crushed block corner.

Fig. C.17 Loosening Failure in a Non-Imbricated Block Mass with a Single Low-Friction Horizontal Seam

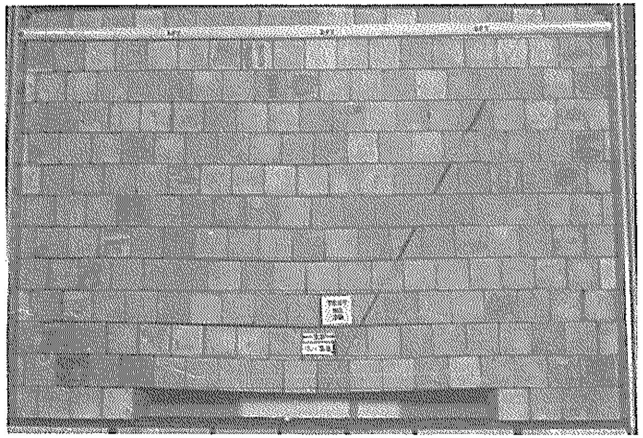


(e) σ_h = 2 psi; increasing bending deformation under a condition of lateral restraint.

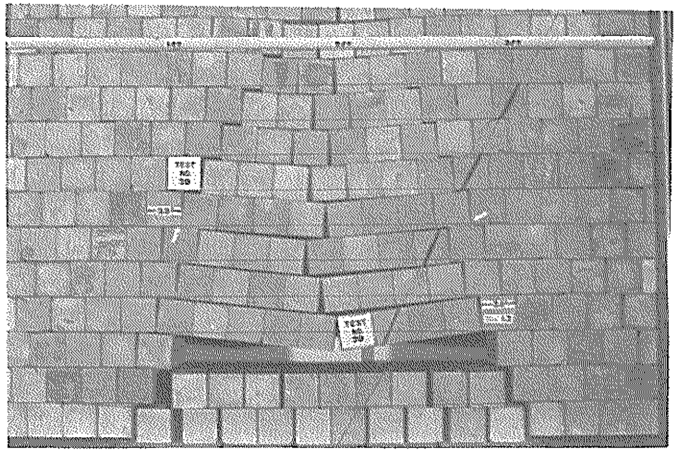


(f) σ_h = 2 psi; fully developed loosened zone.

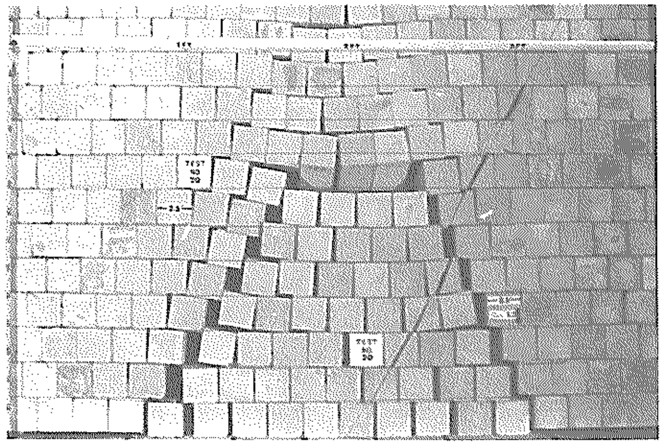
Fig. C.17 (continued)



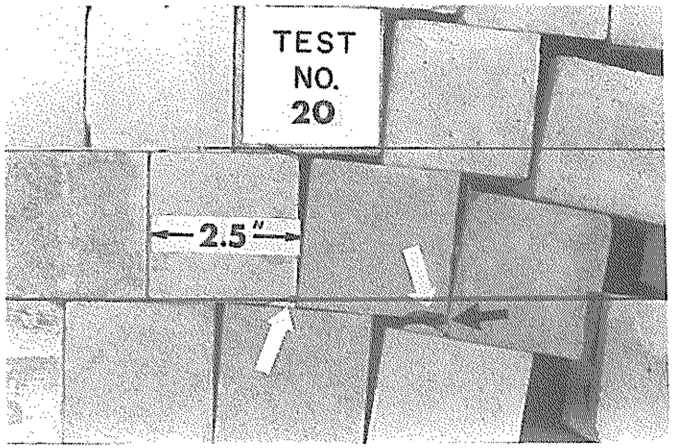
(a) σ_h = 2.5 psi.



(b) σ_h = 1.2 psi; note that 60° joint does not affect failure mechanism.



(c) σ_h = 1.1 psi; compare with same failure mode in Fig. C.14h.



(d) σ_h = 1.1 psi; block row slip caused by corner tear (black arrow).

Fig. C.18 Loosening Failure in an Imbricated Block Mass Intersected by a Steep (60° dip) Joint. Note similarity to Behavior in Fig. C.14.

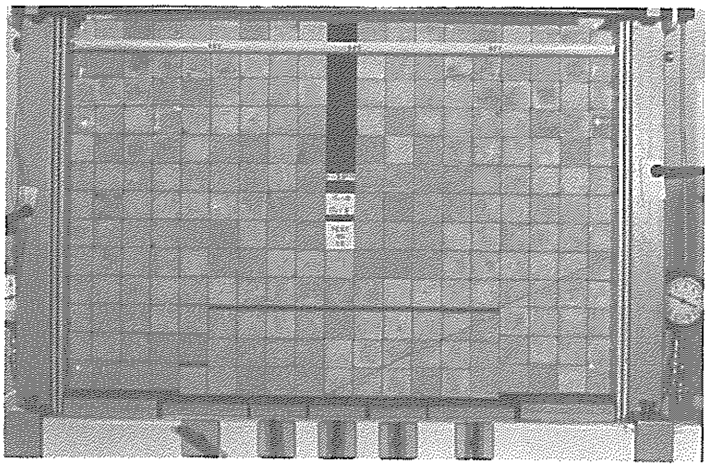


Fig. C.19 Loosening Failure in a Non-Imbricated Block Mass Intersected by a Flat (30° dip) Joint. Mass unstable under initial lateral stress of 5 psi, block fallout by pure slip, no block rotations.

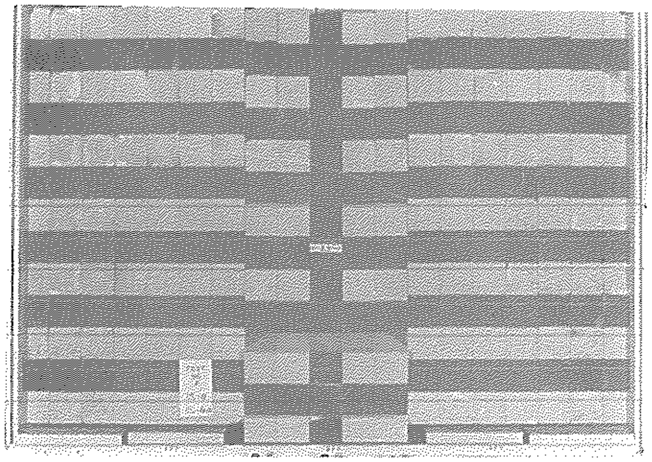


Fig. C.20 Loosening Failure in a Non-Imbricated Block Mass, Narrow Span Width. Note absence of bending, failure by punch shear.

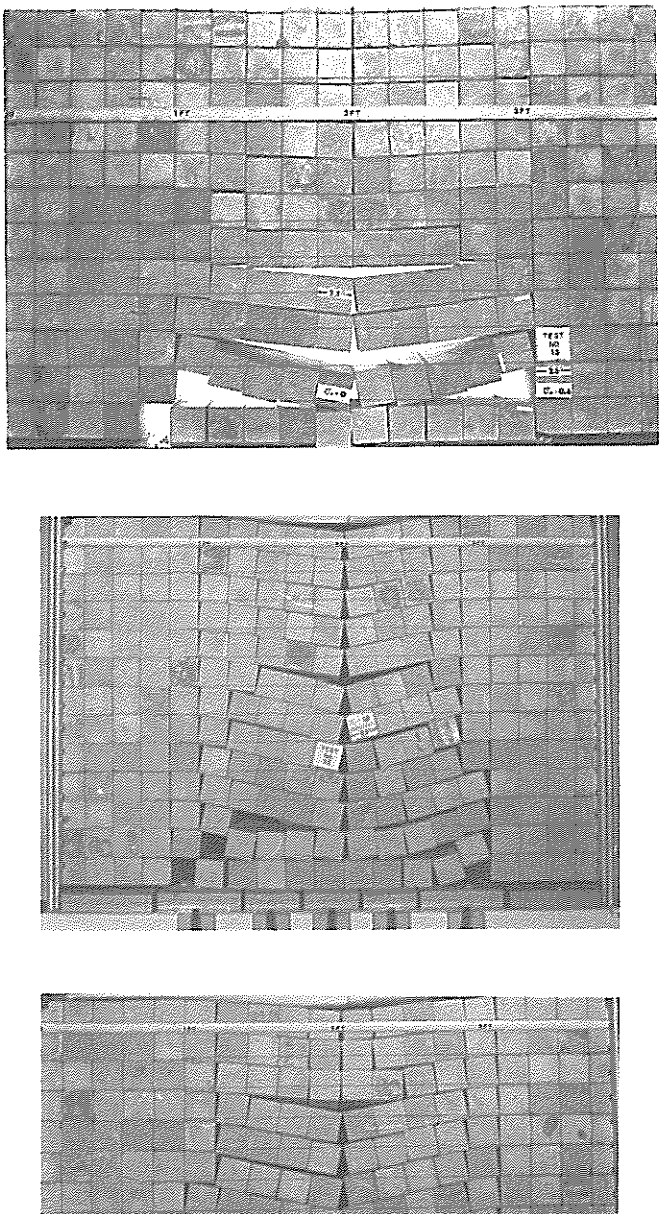


Fig. C.21 Failure Patterns for Three Identical Block Configurations formations of the overlying rows, on the order of 1/20 of the span width, occur prior to failure, as seen in Figures C.13d, C.14g, C.17b, C.17e, and C.18b. These large vertical deformations are accompanied by significant shearing deformations between rows of blocks, as illustrated in Figures C.13c and C.17e. These shearing deformations tend to imbricate a non-imbricated block mass and have a very significant effect on its stability behavior.

The bending behavior of block rows is not uniform. In some cases, such as in Figure C.13f, block rows hinge only at their midpoints. In other instances, such as in Figure C.14g, the block rows behave as a simply supported, multi-hinged arch.

Block row failure, either by slip or block corner failure, occurs as the deformation of block rows increases. The block corner failures that have been observed are of three different types.

Local corner crushing occurs at the pivot points of large block rotations, as seen in Figures C.13k, C.14d, and C.17d. The two end blocks in Figure C.13e show the corner conditions of blocks that undergo such behavior. The upper corners of blocks at the centers of block rows commonly exhibit such failure.

Tensile or tearing fractures of block corners occur very frequently on the blocks located at the ends of block rows, as seen in Figures C.13e, C.13j and C.17a. These failures frequently lead to block slip and are easily recognized as they open up under continuing displacement.

Shear fractures occasionally occur at the corners of blocks at the ends of block rows, as seen in Figures C.13c, C.13h, and C.13i. These failures are not always discernible from tensile failures.

All three types of block corner failure are significant in determining the behavior of the model block mass. They all are accompanied by deformation and subsequent redistribution of block corner contact stresses. As block corners fail the magnitude of the contact stress is reduced and results in a reduction of shearing resistance at the block contacts.
That the behavior of block corners is important should be obvious from close inspection of the blocks in the center and at the ends of block rows. The block rotations at these points are mot pronounced and, as the bending deformations increase, the weight of an entire block row is transferred to the surrounding mass through only four contact points, two at each end of a block row as marked by "sc" in Figure C.13e. The stress distributions on the edges of a mid-span block and an end block are shown schematically in Figure 3.6.

If block slip occurs, such as in the case of row 2 in Figure C.13e, half of the weight of the block row is either supported at only one point, in the same row, or else in stransferred to the underlying row. Increased corner stresses can also arise if one corner fails in tension, as in the upper corners of blocks "a" and "c", and additional horizontal thrust load is transferred to the corner of the pivot point at the center of the block row. Thus, if the shearing resistance along horizontal joints is low, as in the test shown in Figure C.17, large thrust loads are transferred through the contact point at the center of the row and increase the likelihood of failure at that point (see Fig. C.17d). It is thus obvious that the intact strength properties of the block material are very important. Both the shear strength (particularly in unconfined compression) and the tensile strength of the intact material determine the behavior of block corners.

The actual fallout of blocks from the model occurs either as a result of pure slip along a block surface or as a result of the tensile failure of block corners, as seen in Figures C.17a and C.18d.

Block fallout in the imbricated model usually occurs as a result of corner tearing of the ends of unsupported block rows, as illustrated by Figures C.14f and C.14g. The white arrows at the boundary of the stable mass indicate corner tearing. Occasionally failure is by block slip at the crushed corner in the middle of the span, as illustrated in Figure C.14d.

Block fallout in the non-imbricated model occurs either as a result of corner crushing brought about by large block rotations (as in Fig. C.17d), or by pure slip along vertical joints (as in Fig. C.13f). Slip always occurs at the end of a block row, adjacent to the blocks marked "a", "b", and "c" in Figure C.13e. The following explanation is given for this behavior. The end blocks (a, b, and c) rotate and wedge themselves against the adjacent supported mass. When the unsupported mass is stable, the rotation of these end blocks is the key to the development of arching action through lateral thrusting. This wedging of block "a" against the suroounding block serves as the abutment reaction for the block row labeled "row 1" in Figure C.13e, and tends to keep block "b" in its original position. Thus the free span in row 2 is shortened by one block at each end. This process tends to continue upward in the block mass and leads to shorter and shorter block rows, and a dome-shaped zone of loosened blocks, as seen in Figure C.13f. Each block row acts as a multi-hinged arch. Shear, or slip, ultimately occurs at the ends of the block rows, adjacent to the rotated block (marked "a" in Fig. C.13e) where the shear force is largest.

Because of the progressive shortening of block row spans with increasing height above the opening, and because of the "self-imbrication" caused by shear displacements between individual block rows, the continued slipping of blocks progressively leads to a dome-shaped zone of failed blocks, provided complete block slip is prevented. The stable block configuration just prior to complete collapse, as seen in Figure C.13f, resembles a stable arched opening in moist sand.

The remanent, dome-shaped collapse structures of three identical models are shown in Figure C.21. Had the model height been greater, the collapsed structure probably would have more closely resembled the peaked collapse geometry of the imbricated model, shown in Figure C.14h.

It is significant to mention here that the behavior of the model is not reproducible in all respects. The general failure patterns for any one particular block configuration are similar, but the magnitude of horizontal stress at which any one block row begins to slip varies considerably. Similarly, arching in the non-imbricated model does not always occur identically in the same block configuration. The three different failures in three identical non-imbricated models shown for comparison in Figure C.21 verify this fact. The photographs were taken at slightly different stages in collapse, but the rough outlines of the ultimate loosened zone are visible in all three photographs. The differences in failure geometry and lateral pressure at any particular stage of failure can be attributed to very minor variations in block "fit" in the model that are impossible to control. The presence of individual grains of sand on the surface of a block can determine the manner in which block rows behave.

The variation in results from the inclined plane tests for the determination of the angle of sliding friction indicate the possibility for large variations in any behavior that depends on the surface frictional characteristics of the blocks. In the total of 45 inclined plane tests conducted on the cast block surfaces, values of the sliding friction angle vary from 26 to 32 degrees.

C.10 Similitude Considerations

Although it was not the purpose of this model study to derive data that can be scaled to prototype conditions, it is necessary that the various model parameters obey certain scaling laws if the model behavior is to be considered realistic. For the loosening failure model that has been constructed body forces (i.e. gravity) are of utmost importance and must be modeled in the same scale as the surface forces. Heuer (1968) has shown that the scale factors of density (K_ρ), length (K_L), gravitational acceleration (K_g), stress (K_σ) and unconfined compressive strength (Kq_u) must be related according to the expression

$$K_\rho K_L K_g = K_\sigma = Kq_u$$

For the model material used in this study, the unit weight, ρg, is 120 pcf and the unconfined compressive strength, q_u, is 610 psi. For an assumed prototype granite rock with a unit weight of 160 pcf and an unconfined compressive strength of 25,000 psi, $K_\rho K_g = \frac{120}{160} = 0.75$ and $Kq_u = 610/25,000 =$ $= 0.0244$. From the relationship

$$K_\rho K_L K_g = Kq_u$$

$$K_L = \frac{0.0244}{0.75} = 0.0325$$

Thus, for a strong granite, the 27.5-inch unsupported model span represents a 70-foot prototype span at a depth of 84 feet in a rock mass whose joint spacing is 6.4 feet.

Although it is quite obvious that the behavior of the model does not reproduce that of a tunnel at depth in a rock mass of infinite extent, it is very likely that the failure mechanism and the influence of various rock mass-tunnel parameters are similar for both cases.

If it would be desired to model the behavior of a span at a depth where no effects from the ground surface exist, it would be necessary to use a model of at least twice the height of that used in the writer's studies.

LIST OF PROCEEDINGS OF THE SWEDISH GEOTECHNICAL INSTITUTE

No.	Title	Year	Sw. Crs. Price
1.	Soil Sampler with Metal Foils. Device for Taking Undisturbed Samples of Very Great Length. *W. Kjellman, T. Kallstenius* and *O. Wager*	1950	10:—
2.	The Vane Borer. An Apparatus for Determining the Shear Strength of Clay Soils Directly in the Ground. *Lyman Cadling* and *Sten Odenstad*	1950	10:—
3.	Device and Procedure for Loading Tests on Piles. *W. Kjellman* and *Y. Liljedahl*	1951	Out of print
4.	The Landslide at Sköttorp on the Lidan River, February 2, 1946. *Sten Odenstad*	1951	10:—
5.	The Landslide at Surte on the Göta River, September 29, 1950. *Bernt Jakobson*	1952	15:—
6.	A New Geotechnical Classification System. *W. Kjellman, L. Cadling* and *N. Flodin*	1953	Out of print
7.	Some Side-Intake Soil Samplers for Sand and Gravel. *Torsten Kallstenius*	1953	10:—
8.	Influence of Sampler Type and Testing Method on Shear Strength of Clay Samples. *Bernt Jakobson*	1954	10:—
9.	Some Relations between Stress and Strain in Coarse-Grained Cohesionless Materials. *W. Kjellman* and *B. Jakobson*	1955	10:—
10.	Accurate Measurement of Settlements. *W. Kjellman, T. Kallstenius* and *Y. Liljedahl*	1955	10:—
11.	Influence of Organic Matter on Differential Thermal Analysis of Clays. *Lennart Silfverberg*	1955	10:—
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13.	Pore Water Pressure Measurement in Field Investigations. *Torsten Kallstenius* and *Alf Wallgren*	1956	10:—
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	Earth Pressure from Friction Soils. A Report on Half Scale Tests. *Arne Rinkert.*		
	Measurements in Grain Silos during Filling and Emptying. *Werner Bergau.*		
18.	Consolidation of Clay, with Special Reference to Influence of Vertical Sand Drains. A Study Made in Connection with Full-Scale Investigations at Skå-Edeby. *Sven Hansbo*	1960	20:—
19.	Standard Piston Sampling. A Report by the Swedish Committee on Piston Sampling	1961	10:—
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21.	Studies on Clay Samples Taken with Standard Piston Sampler. *Torsten Kallstenius*	1963	25:—
22.	Salt in Swedish Clays and its Importance for Quick Clay Formation. Results from some Field and Laboratory Studies. *Rolf Söderblom*	1969	25:—
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No. 24.	Clay Microstructure. A Study of the Microstructure of Soft Clays with Special Reference to Their Physical Properties. *Roland Pusch*	1970	30: —*
25.	Negative Skin Friction on Long Piles Driven in Clay. I. Results of a Full Scale Investigation on Instrumented Piles. II. General Views and Design Recommendations. *Bengt H. Fellenius*	1971	30: —
26.	Organic Matter in Swedish Clays and its Importance for Quick Clay Formation. *Rolf Söderblom*	1974	25: —
27.	Correlations of Rock Bolt-Shotcrete Support and Rock Quality Parameters in Scandinavian Tunnels. *Owen S. Cecil*	1975	40: —

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