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PROVED SUBSURFACE INVESTIGATION FOR GHWAY TUNNEL DESIGN AND CONSTRUCTION

H v. 1

Vol. 1. Subsurface Investigation System Planning

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DEPARTMENT OF TRANSPORTATION • UNITED STATES OF AMERICA •

May 1974

Final Report

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16. Abstract

This volume covers that study portion which evaluated current subsurface investigation techniques, investigated interactions of the subsurface investigation system elements, produced a value analysis model for indicating the relative value of achieving new levels of performance in a subsurface investigation system, and determined the feasibility of horizontal long-hole drilling. Volume 2 describes the development of new acoustic techniques for subsurface investigation in soils.

A critical analysis of known subsurface investigative techniques was made. Summaries are organized into groupings for comparison. Techniques not fully utilized or needing additional development are identified.

Correlations were made of subsurface conditions as they affect the major specific design consideration and the ability of individual techniques to identify specific subsurface conditions and to operate in different geologic environments.

A preliminary value analysis model for use in optimizing a subsurface investigation system is developed and its use illustrated.

The subsurface investigation systems used for highway tunnels is examined to identify the operations or function involved. Several potentially feasible subsurface investigation systems are synthesized according to particular geologic environments.

The drilling of long horizontal holes is found to be technically feasible, but improvements in penetration rates, guidance and control techniques, and geologic sensing are needed to make them economically competitive.

This is the first of two volumes. Volume 2 is published as FHWA-RD-74-30, subtitle: New Acoustic Techniques Suitable for Use in Soil.

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UNITED STATES GOVERNMENT

Memorandum

DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

DATE: OCT 3 0 1974

In reply refer to: HRS-11

SUBJECT: Transmittal of Research Report No. FHWA-RD-74-29 and 30. "Improved Subsurface Investigation for Highway Tunnel Design and Construction" Volumes 1 and 2.

FROM : Director
Office of Research
Washington, D.C. 20590

TO : Regional Federal Highway Administrators
Regions 1, 3-10 and Regional Engineer, Region 15

Sufficient copies of this report are being distributed to provide two copies for each Regional Office, one copy for each Division Office and the standard number of copies for each State Highway or Transportation Department. Direct distribution is being made to the Division Offices.

This report will be of interest to tunneling and soils engineers responsible for planning and executing site investigation programs for highway tunnels and heavy structures. Highway research engineers may find a particular interest in the second volume of the study that presents some novel ideas in the application and capabilities of acoustic sensing techniques in soil environments for site investigation purposes.

The study has accomplished the following objectives:

- review of the state of the art;
- investigation of the interaction of the subsurface investigation subsystem elements and the tunneling system;
- development of a value analysis model for planning rational site investigation programs;
- determination of the feasibility of long range horizontal drilling for exploration purposes.
- development of new acoustic techniques suitable for use in soil, in particular to map the soil-rock interface between bore holes.

A limited number of additional copies of this report are available for official use from the Bridge Structures Group (HRS-11), Structures and Applied Mechanics Division, Office of Research. Additional copies for the public are available from the National Technical Information Service (NTIS), Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22151. A small charge will be imposed for each copy ordered from the NTIS.

Charles F. Scheffey

Charles F. Scheffey

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Section 1

Introduction

BACKGROUND

The expansion of existing surface areas to accommodate all functions of our modern society is not an acceptable alternative to surface saturation and visual clutter, for this very expansion is done at the expense of physical proximity. Multi-level use of land and the mingling of facilities seems to be a more feasible alternative. If the land is to be used in a truly three-dimensional manner, greater use of underground space is implicit and there is already a growing interest in the transportation field to make use of the underground for moving people and goods.

The congestion on the Nation's highways, especially in urban areas, is obvious to all users. The right-of-way problems and increasing volumes of traffic in cities will encourage the increased use of underground space. Unfortunately, the high construction cost of underground transporation facilities is severely restraining their use. The comparison of relative cost in Table 1 shows that highway tunnels in urban areas are significantly more costly than surface roadways.[1] To realize the

Table 1. Relative construction costs of urban roadways (excluding land cost, etc.)

Type of construction	Index Number Average	Index Number Range
At ground level	1	0.7 to 1.3
Depressed and open	1.5	-
Elevated on embankment	2	-
Elevated and retained	3	-
Depressed and retained	5.5	-
Elevated viaduct	7.5	4 to 10
Bored tunnel in good condition	13	10 to 16
Cut-and-cover tunnel	14	5 to 30
Immersed tube tunnel	25	-
Bored tunnel under river	50	18 to 65

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[1] "Underground Motorways for Urban Areas." Tunnels and Tunneling, Vol. 3 No. 4 (July-August, 1971), pp 277-278.

full potential of underground space these tunneling costs must be reduced by a substantial amount.

A determination of the details of subsurface conditions is more important to reducing the total cost of a tunnel than any other detail of design. While present day exploration methods yield good qualitative information on subsurface conditions, the information so obtained is merely an indication of conditions encountered at the single location where the exploration took place. Usually time and budget constraints limit the number of such small-scope explorations which are practical and feasible for each particular project. Ground conditions adjacent to and between these explorations cannot be defined with precision and accuracy, thereby introducing uncertainties in predicting the material through which the tunnel must pass.

The Federal Highway Administration, Department of Transportation (DOT), as a part of its Tunneling Technology for Future Highways Program, is attempting to provide a new dimension of space for transportation by improving tunnel design and technology, reducing costs, and increasing rate of construction of tunnels. A major effort in this program is directed toward developing improved means for determining subsurface conditions at tunnel sites. This present report is part of that effort.

THE TECHNICAL PROBLEM

The following is the technical problem as it was identified in DOT's Request for Proposal for this work.

It is practically impossible to justify increasing the number of vertical exploratory borings beyond certain limits. Therefore, to obtain a complete description of subsurface conditions, two alternatives are available as follows:

1. To perform vertical borings and to use an indirect technique which permits the interpolation of data between boreholes with absolute certainty.

2. To use a horizontal penetration technique along arbitrary, preselected paths, covering distances of the order of 1 mile, having a remote guidance control, and providing information about materials encountered.

In any case, various direct as well as indirect subsurface exploration techniques need to be integrated into systems to optimize operations applicable to different geological and environmental conditions.

Each integrated system must be defined in such a manner that it provides satisfactory information for solving the following problems:

1. Choice of excavation technique;
2. Estimation of stability and safety conditions; and
3. Establishment of lining requirements.

This information is needed for every tunnel. However, different methods at significantly different costs may be used in each of the three above-stated problem areas. In the first problem area, the investigation must be continuous over the length of the tunnel and only major differences in materials must be recognized. In the two remaining problem areas, a few, high-quality, representative samples must be obtained. Therefore, it is most important to define the integrated subsurface exploration systems in view of the objectives.

Among indirect techniques already in use for subsurface investigation, acoustic methods appear to have a potential for further technical development. However, in applying acoustic techniques to soil, more problems are encountered than in rock.

The acoustic properties of a heterogeneous medium consisting primarily of soil vary over a wide range. This fact causes difficulties in interpretation of three-dimensional wave propagation phenomena. Among the technical problems to be solved for a soil medium is the coupling of the signal source and the receiver to the soil. Boreholes may offer a solution to this problem but the presence of casing material can complicate the interpretation of results.

Present acoustic techniques in soil investigations are based on questionable assumptions and rely on either transverse or reflecting waves. However, boreholes may make it possible to combine several techniques.

SCOPE OF THE STUDY

The study effort was concerned with the evaluation of current practice and the development and theoretical analysis of new and improved subsurface investigation techniques for highway tunnels. The following qualifications apply to the research:

1. The environment is both urban and rural.
2. The zone of interest is between the ground surface and a depth of 500 feet.

3. The material is both soil and rock.

In the conduct of the research effort we were required to perform, as a minimum, the following tasks:

Task A - Conduct critical in-depth analysis of all known techniques that are used or that can be used for the determination of subsurface conditions for highway tunnels in an urban or rural environment to depths of 500 feet.

1. Past experience, records, and geological maps
2. Indirect techniques
3. Direct techniques
4. Analyze current tunneling design practice with respect to the efficient application of the information obtained from different techniques for economical selection of the excavation technique and design. Provide a preliminary mathematical model for value analysis of subsurface investigation.

Task B - Identify integrated subsurface exploration systems for highway tunnels for specific geological conditions and environmental factors. Analyze the interaction of the components and subsystems and suggest alternate systems where applicable.

Task C - Establish the feasibility of using remotely guided horizontal penetration techniques to provide information about ground conditions and material properties encountered along arbitrary, preselected paths to a depth of 1 mile.

Task D - Develop new acoustic techniques for use in soil that can be used to determine with certainty the location of any rock between boreholes. Investigate the potential of acoustic techniques for predicting variations of soil properties between boreholes.

1. a. Describe the properties of the sonic spectrum in soils.

   b. Investigate the feasibility of the application of acoustic techniques for detecting the exact location of changes in materials between boreholes in soil.

   c. Explore possible ways to improve the coupling of the signal source and the receiver to the soil.

2. Perform a detailed theoretical analysis of the propositions and prove that the new techniques are sound and worthy of further considerations.

3. Develop preliminary drawings of proposed instrumentation based on the new ideas. Include, as a minimum, descriptions, preliminary specifications, and the necessary explanation so that the

instruments can be visualized.

4. Develop a specific, detailed work plan for proving the ideas and instruments under laboratory and field conditions.

The above Tasks identified as A, B, and C comprised 60 percent of the study effort and were carried out by Fenix & Scisson, Inc. (F & S). These Tasks are reported upon in this Volume 1. The Task identified as D comprised 40 percent of the study effort and was carried out by Telcom, Inc. under subcontract to F & S. Task D is reported upon in Volume 2.

FINDINGS

The major findings in each significant area of this study are as follows:

SUBSURFACE EXPLORATION SYSTEMS ANALYSIS

Many techniques are presently available for obtaining information about the subsurface. In this study, only those which we considered as possibly having application for use in site investigations for highway tunnels located within 500 feet of the ground surface were analyzed. Some of those techniques which we thus included in our investigations, however, were found to have limited application for our purpose with their present state of development. This group of techniques included passive microwave, ultraviolet, luminescence, and passive microwave radiometry remote sensing methods; and gravitational and temperature borehole logging methods.

There were other techniques studied which were proven methods, but found to be little used and with poor potential. This group of techniques included such methods as magnetic, electromagnetic, gravity, and radiometric surface geophysical; spontaneous potential, gamma ray, and neutron borehole logs; laboratory unconfined compressive strength tests for soil; and laboratory uniaxial shear and tensile strength, triaxial compressive and shear strength, and creep tests for rock.

Those techniques studied that were found to be little used although they were proven and appear to have good potential included color photography, infrared photography, and side-looking airborne radar remote sensing methods.

Those techniques considered as being proven but little used and with good potential in only one or two of the major tunneling problem areas included visual or photographic borehole logging and conventional resistivity, microresistivity, focusing electrode, induction, formation density and caliper borehole logs.

We also found from our analysis of techniques that the following ones were among those only partially developed but they have good potential for possible additional use after they have become more fully developed: Thermal infrared, multispectral photography, multispectral scanning and infrared radiometry remote sensing methods; acoustic holography; horizontal drilling; and sonic borehole logging. Although much work has already been done on developing instruments for measuring the in situ state of stress, we found that a considerable amount still remains to be done before the gathering of really reliable information about the state of stress becomes economical, and still in some cases, technically feasible.

During the course of our investigation we found that there are almost as many possible listings for the specific kinds of information wanted to design a tunnel as there are tunnel designers. In general, however, all wanted to know the ground type, its structural defects (some to a greater degree than others), its physical and engineering properties (some in more detail than others), and what the ground water conditions were. The reason for this wide variance in kinds and amounts of information wanted by tunnel designers we found to be apparently due to the fact that each uses slightly different methods for their design and each attached different amounts of importance to each type of information (i.e. the fracture spacing, the compressive strength, the existing state of stress, etc.).

SUBSURFACE EXPLORATION SYSTEMS SYNTHESIS

After studying case histories for different highway tunnels, it became apparent that there is no common investigation system used. It was found that although data on the particular techniques used in the investigation was available, information on how the data obtained during the investigation was utilized for the design and to what extent was scarce. We also found very few instances where the actual conditions encountered were compared to those predicted from the site investigation. Our analysis of the available data, however, resulted in the conclusion that although the subsurface investigation techniques now available would have been sufficient to provide the information needed in most cases, a high percentage of the major cost overruns experienced in tunneling projects can be, in many cases, directly traceable to either insufficient subsurface information, incorrect interpretation of the information available, or management's decision to not follow the investigator's recommendations.

DESIGNING AN OPTIMUM SUBSURFACE EXPLORATION SYSTEM

Every tunneling project has its own best subsurface investigation system which is dependent upon the geologic environment existing at the tunnel site and the kind and amount of information wanted by the tunnel designers. Although we can always expect, at least in the foreseeable future, to have some degree of uncertainty involved with predicting ground conditions at a tunnel site, we found that there has been little apparent attempt to make a concentrated effort to reduce this uncertainty except by an increased use of the now commonly used techniques such as drilling closer-spaced boreholes and/or the taking of more samples for laboratory tests.

Rarely is any kind of systematic approach used to design a subsurface investigation system for highway tunnels. We found that this is due to several reasons, some of which are:

1. A simple easily understood investigation system useable under any set of conditions is not available and would be virtually impossible to design.
2. An insufficient amount of time and/or money is allotted for the subsurface investigation program to allow the use of what may be considered as better techniques by the investigation team for obtaining the desired information, but management usually does not want to spend either the necessary money or time.
3. The investigation teams do not want to use any techniques with which they are unfamiliar.

During our study we found that a relatively simple method for optimizing the subsurface investigation system could probably be developed and so, as the first step in this direction, we formulated a preliminary value analysis model.

In attempting to make estimates of future costs and performances of systems and equipment, we found that the diversity of tunnel specifications made correlations of available data for making reliable estimates of future costs and performance difficult (if not impossible). The most suitable method for this purpose was found to be a series of generalized equations which can be used to determine cost variance resulting from changed subsurface conditions. These equations are easily varied for different localities and escalatable for use at some future time.

HORIZONTAL LONG HOLE DRILLING

We found from our analysis of the current state-of-the-art for horizontal drilling, that the drilling of long horizontal holes is technically feasible. However, it may not be economically feasible when extended over distances of one mile with present techniques unless the ground conditions are favorable. We were able to formulate six different systems for drilling horizontal holes up to a mile in length that were technically feasible. To make horizontal drilling of long holes more competitive with other exploration techniques: increased penetration rates, improved guidance and directional control techniques, and improved and new sensing equipment for geologic investigation useable in a horizontal hole and compatible with the drilling equipment is needed.

NEW ACOUSTIC TECHNIQUES SUITABLE FOR USE IN SOIL

This area of the study considered the development of new acoustic techniques suitable for use in soil. Primary consideration was given to determining the location of any rock that may be present between boreholes. A secondary objective was to investigate the potential of acoustic techniques for predicting variations of properties between boreholes.

In pursuit of these objectives, the properties of the sonic spectrum in soil were investigated with application to locating discontinuities. A review of potential techniques for improving coupling to soils was also conducted.

As a result of these investigations, a pulse compression technique similar to one used in radar was judged to be the most promising technique for enhancement of range and resolution. This technique utilizes a swept frequency transmission and temporal filtering of the received signal. It is concluded that the pulse compression technique is suitable and practical for further development.

This portion of the study was performed by Telcom, Inc. and is reported upon in Volume 2.

Section 2

Subsurface Exploration Systems Analysis

SUBSURFACE CONDITIONS

The general subsurface conditions considered in this study are:

1. Urban or rural environment
2. Maximum depth of 500 feet
3. Soft ground, mixed ground, or rock
4. With or without high water table
5. Normal and extremely unfavorable conditions.

In simple terms, we considered every subsurface condition down to 500 feet. Our intent was to determine the impact of various subsurface conditions upon the design and construction of highway tunnels, specifically:

1. Choice of excavation technique
2. Design of ground support
3. Evaluation of hydraulic conditions
4. Evaluation of safety hazards
5. Evaluation of potential damage to other man-made structures
6. Design of final lining.

Our approach was to devise a subsurface classification system which can be used to correlate conditions with the major tunnel design and construction considerations. We considered the commonly used classification systems but none would fit our specific needs. Therefore, we devised a system which we call the Tunnel and Subsurface Conditions Classification System.

COMMONLY USED CLASSIFICATION SYSTEMS

The different classification systems reported in the literature are, for the most part, based upon either origin, particle size, texture, structure, composition, distinct properties, behavior in particular circumstances, or some combination of these factors. Most commonly used classifications are based on geological environments. Table 2 is one example. A similar but more general example which correlates major problems with different geological environments is given in Figure 1.

Table 2. Classification of geological environments.

Soils	Residual	Mechanically weathered	igneous metamorphic	
		Chemically weathered	sandstone limestone shale	
	Transported	River	headwater valley plains alluvial fans	
		Glacier	till or unstratified drift	
			stratified drift	meltwater river meltwater lake
			ice contact	
		Lake	shore and delta zones central portion	
		Swamp		
		Estuary		
		Ocean		
		Tidal marshes and flats		
		Wind	dune sands loess	
		Landslides		
	Organic			
	Pumice and Ash			
	Precipitals or Evaporates			
Rocks	Igneous	Composition	acid intermediate basic	
		Depth of Formation	intrusive	plutonic hypabyssal
			extrusive	flow pyroclastic
	Sedimentary	Mechanical or Clastic		
		Chemical		
		Organic		
	Metamorphic	Cataclastic		
		Pyrometasomatic		
		Dynamometamorphic		

ASSOCIATED POTENTIAL PROBLEMS

GEOLOGICAL ENVIRONMENTS

Above the water table

		Water Inflow	Face Stability	Ground Pressure	Surface Settlement	Ground Vibration	Change in Excavation and/or Support Methods
Soil	Residual			●			●
	Alluvial		●		●		●
	Glacial		●		●		●
	Loessial						
	Organic		●		●		
	Volcanic		●		●		
	Precipitates & Evaporites						
Rock	Igneous; Intrusive			●		●	
	Extrusive					●	
	Sedimentary; Conglomerate						
	Siltstone, Sandstone						
	Shale, Mudstone		●	●			●
	Limestone, Dolomite			●	●		●
	Evaporites						
	Metamorphic			●		●	

Below the water table

		Water Inflow	Face Stability	Ground Pressure	Surface Settlement	Ground Vibration	Change in Excavation and/or Support Methods
Soil	Residual	●	●	●	●		●
	Alluvial	●	●		●		●
	Glacial	●	●		●		●
	Loessial	●	●		●		
	Organic	●	●		●		
	Volcanic	●	●		●		
	Precipitates & Evaporites	●	●		●		
Rock	Igneous; Intrusive	●		●		●	
	Extrusive	●				●	
	Sedimentary; Conglomerate	●					
	Siltstone, Sandstone	●					
	Shale, Mudstone	●	●	●	●		●
	Limestone, Dolomite	●		●	●		●
	Evaporites	●					
	Metamorphic	●		●		●	

Figure 1. Potential problems associated with geological environments.

The Unified Soil Classification System (Table 3) is the soil classification system most widely used by engineers in the United States. This system provides the most fundamental information about the physical properties of soil. Two other soil classification systems often used in the United States are the American Association of State Highway Officials (AASHO) System (Table 4) and the Federal Aviation Agency (FAA) System (Table 5). These systems, however, do not enjoy anywhere near the popularity of the Unified System.

A rock classification system commonly used is one for determining the kind and amount of support required in tunnels. This system, by Terzaghi,[2] is behavioristic in nature and divides all rocks into the following seven categories: intact rock, stratified rock, moderately jointed rock, blocky and seamy rock, crushed rock, squeezing rock, and swelling rock. Similar classifications have been proposed by numerous other authors, but all are similar although different descriptive terms are used.

A classification system that has been used for describing the rock surrounding an underground opening in rock mechanics studies is as follows:[3]

1. Competent rock, rock that will sustain an opening without artificial support.
    a. Massive-elastic, homogeneous and isotropic.
    b. Bedded-elastic, homogeneous and isotropic beds with the individual bed thicknesses less than the span of the opening and having little cohesion between the beds.
    c. Massive-plastic, rock that will flow under low stresses.
2. Incompetent rock, rock which requires artificial supports in order to sustain an opening.

This system is used primarily for studies on the structural stability of underground openings.

Several methods have been proposed for describing the quality of the rock structure. The most commonly used is the "Rock Quality Designation" (RQD) method.[4] The RQD is a modified core recovery

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[2]Terzaghi, Karl. "Introduction to Tunnel Geology" in Rock Tunneling with Steel Supports by Robert V. Proctor and Thomas L. White. Commercial Shearing and Stamping Co. Youngstown, Ohio. 1946.

[3]Coates, D. F. Rock Mechanics Principles. Mines Branch Monograph 874. Dept. of Energy, Mines and Resources; Mines Branch. Ottawa, Canada. 1965.

[4]Deere, D. U., A. J. Hendron, Jr., F. D. Patton, and E. J. Cording. "Design of Surface and Near-Surface Construction in Rock." Proceedings of 8th Symposium on Rock Mechanics (1966). AIME, New York. 1967. pp. 237-302.

Table 3. Unified soil classification system.

Primary Divisions			Group Symbol	Secondary Divisions	Laboratory Classification Criteria		Supplementary Criteria for Visual Identification
Coarse-grained soils. (More than half of material is larger than No. 200 sieve size.)	Gravels. (More than half of the coarse fraction is larger than No. 4 sieve size.)	Clean gravels. (Less than 5% of material is smaller than No. 200 sieve size.)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.	C_u greater than 4. C_c between 1 and 3.		Wide range in grain size and substantial amounts of all intermediate particle sizes.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.	Not meeting all gradation requirements for GW.		Predominantly one size or a range of sizes with some intermediate sizes missing.
		Gravels with fines. (More than 12% of material is smaller than No. 200 sieve size.)	GM	Silty gravels, and gravel-sand-silt mixtures, which may be poorly graded.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line, with PI between 4 and 7 is boarderline case. GM-GC	Non-plastic fines or fines of low plasticity.
			GC	Clayey gravels, and gravel-sand-clay mixtures, which may be poorly graded.	Atterberg limits above "A" line, with PI greater than 7.		Plastic fines.
	Sands. (More than half of the coarse fraction is smaller than No. 4 sieve size.)	Clean sands. (Less than 5% of material is smaller than No. 200 sieve size.)	SW	Well graded sands, gravelly sands, little or no fines.	C_u greater than 6. C_c between 1 and 3.		Wide range in grain sizes and substantial amounts of all intermediate particle sizes.
			SP	Poorly graded sands, gravelly sands, little or no fines.	Not meeting all gradation requirements for SW		Predominately one size or a range of sizes with some intermediate sizes missing.
		Sands with fines. (More than 12% of material is smaller than No. 200 sieve size.)	SM	Silty sands, and sand-silt mixtures, which may be poorly graded.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line with PI between 4 and 7 is boarderline case. SM-SC	Non-plastic fines or fines of low plasticity.
			SC	Clayey sands, and sand-clay mixtures, which may be poorly graded.	Atterberg limits above "A" line, with PI greater than 7.		Plastic fines.

Table 3. Unified soil classification system (continued).

Primary Divisions		Group Symbol	Secondary Divisions	Laboratory Classification Criteria		Supplementary Criteria for Visual Identification: Dry Strength	Reaction to Shaking	Toughness near Plastic Limit
Fine grained soils. More than half of material is smaller than No. 200 Sieve size.	Silts and clays. (Liquid limit is less than 50.)	ML	Inorganic silts, clayey silts, rock flour, silty very fine sands.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line with PI between 4 and 7 is borderline case ML-CL	None to slight	Quick to slow	None
		CL	Inorganic clays of low to medium plasticity; silty, sandy or gravelly clays.	Atterberg limits above "A" line, with PI greater than 7.		Medium to high	None to very slow	Medium
		OL	Organic silts and organic silt-clays of low plasticity.	Atterberg limits below "A" line.		Slight to medium	Slow	Slight
	Silts and clays. (Liquid limit greater than 50.)	MH	Inorganic silts, clayey silts, elastic silts, micaceous or diatomaceous silty or fine sandy soils.	Atterberg limits below "A" line.		Slight to medium	Slow to none	Slight to medium
		CH	Inorganic clays of high plasticity, fat clays.	Atterberg limits above "A" line.		High to very high	None	High
		OH	Organic clays and silty clays of medium to high plasticity.	Atterberg limits below "A" line.		Medium to high	None to very slow	Slight to medium
Highly organic soils		Pt	Peat, meadow mat, highly organic soils.	High ignition loss, LL and PI decrease after drying.		Organic color and odor, spongy feel, frequently fibrous texture.		

Table 4. AASHO soil classification system.

General Classification	Granular Materials (35% or less passing No. 200)							Silt Clay Materials (More than 35% passing No. 200)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5: A-7-6
Sieve analysis Percent passing: No. 10 No. 40 No. 200	50 max 30 max 15 max	- 50 max 25 max	- 51 min 10 max	- - 35 max	- - 35 max	- - 35 max	- - 35 max	- - 36 min	- - 36 min	- - 36 min	- - 36 min
Characteristics of fraction passing No. 40 Liquid limit Plasticity index	- 6 max		- NP	40 max 10 max	41 min 10 max	40 max 11 min	41 min 11 min	40 max 10 max	41 min 10 max	40 max 11 min	41 min *
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine Sand	Silty or clayer gravel and sand				Silty soils		Clayey soils	

*Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

Table 5. FAA soil classification system.

	Soil Group	Mechanical Analysis - Percent				Liquid Limit	Plasticity Index
		>No. 10 Sieve	<No. 10 Sieve				
			>No. 60	<No. 60 and >No. 270	<No. 270		
Granular	E-1	0 to 45	40+	60-	15-	25-	6-
	E-2	0 to 45	15+	85-	25-	25-	6-
	E-3	0 to 45	---	---	25-	25-	6-
	E-4	0 to 45	---	---	35-	35-	10-
	E-5	0 to 45	---	---	45-	40-	15-
Fine Grained	E-6	0 to 55	---	---	45+	40-	10-
	E-7	0 to 55	---	---	45+	50-	10 to 30
	E-8	0 to 55	---	---	45+	60-	15 to 40
	E-9	0 to 55	---	---	45+	40+	30-
	E-10	0 to 55	---	---	45+	70-	20 to 50
	E-11	0 to 55	---	---	45+	80-	30+
	E-12	0 to 55	---	---	45+	80+	---
	E-13	Muck and peat-field examination					

classification based on counting only those sections of sound rock in the core that are longer than four inches. The RQD is a continuous variable expressed as a percent and can vary from 0 to 100. The correlation between RQD and rock quality description is

RQD	Rock Quality Description
0 to 25%	Very Poor
25 to 50%	Poor
50 to 75%	Fair
75 to 90%	Good
90 to 100%	Excellent

Figure 2 shows an example of determining the RQD from a core run.

A system similar to the RQD method, the "Core Index Number",[5] has not found as much acceptance as the RQD. The Core Index Number is the sum of the 0.1 percent core loss, the 0.1 percent broken core (smaller

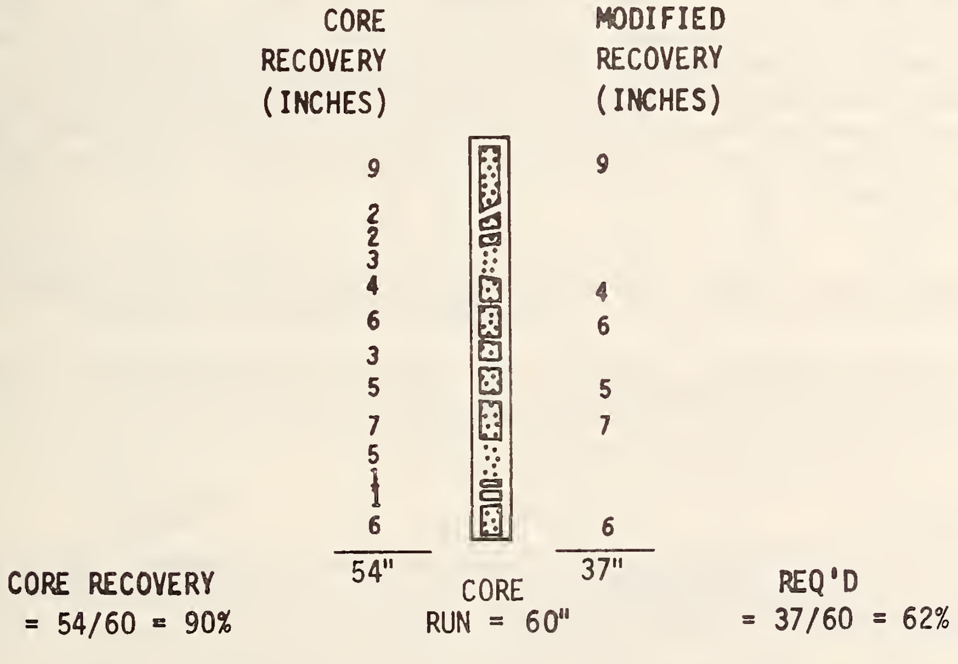

Figure 2. Modified core recovery as an indicator of rock quality.

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[5]Deere, D. U., R. B. Peck, J. E. Monsees, and B. Schmidt. Design of Tunnel Liners and Support Systems. Report for U. S. Dept. of Transportation, OHSGT, Contract 3-0152. Univ. of Ill. February, 1969.

than 3-inch pieces), and the joint frequency. There is almost perfect correlation between the Core Index Number and the RQD for a given site, indicating they can be used interchangeably.[6]

Another concept proposed for describing rock structure quality is the "Rock Structure Rating" (RSR).[7] The RSR method rates the relative effect of three different parameters on ground support requirements, each with respect to several geologic factors and to each other where applicable. Parameter A is a general appraisal of the rock structure, parameter B relates the joint pattern and direction of tunnel driving, and parameter C is a general evaluation of the groundwater inflow effect on support requirements (see Table 6). The RSR value for a particular geologic section is the numerical sum of the three parameters for that section.

A classification of intact rock that is based on laboratory-determined values of mechanical properties is often used.[8] This classification is based on the uniaxial compressive strength and the modulus of elasticity for the rock. The compressive strength used is that determined for specimens having a length to diameter ratio of at least two. The modulus used is the tangent modulus taken at a stress level equivalent to one-half the ultimate strength of the rock. The rock is then classified by both strength (uniaxial compressive strength) and modulus ratio (ratio of modulus of elasticity to the uniaxial compressive strength) as shown in Table 7. A classification plot may also be used. Figure 3 shows the base graph used for this purpose. The rocks are then classified by both strength and modulus ratio such as AM, BL, BH, CM, etc. where:

$$\text{the modulus ratio} = \frac{\text{tangent modulus at 50\% ultimate strength}}{\text{uniaxial compressive strength}}.$$

Probably the most commonly used classification of ground conditions for tunneling is the "Tunnelman's Ground Classification" which is based

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[6]Deere, D. U., R. B. Peck, J. E. Monsees, and B. Schmidt. Design of Tunnel Liners and Support Systems. Report for U.S. Dept. of Transportation, OHSGT, Contract 3-0152. Univ. of Ill. February, 1969.

[7]Wickman, George E., Henry R. Tiedemann, and Eugene H. Skinner. "Support Determinations Based on Geologic Predictions." Proceedings of North American Rapid Excavation and Tunneling Conference (Chicago, 1972). AIME, New York. 1972. pp. 43-64.

[8]Stagg, K. G. and O. C. Zienkiewicz, editors. Rock Mechanics in Engineering Practice. John Wiley & Sons, New York. 1968.

Table 6. Rock structure ratings.

Parameter 'A', General Area Geology

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

Parameter 'B', Joint Pattern and Direction of Drive

Average Joint Spacing Feet	Dip of Prominent Joints							
	Strike Perpendicular to Axis					Strike Parallel to Axis		
	Direction of Drive							
	Both	With Dip		Against Dip		Both		
	0°-20°	20°-50°	50°-90°	20°-50°	50°-90°	0°-20°	20°-50°	50°-90°
< 0.5	14	17	20	16	18	14	15	12
0.5-1.0	24	26	30	20	24	24	24	20
1.0-2.0	32	34	38	27	30	32	30	25
2.0-4.0	40	42	44	36	39	40	37	30
> 4.0	45	48	50	42	45	45	42	36

Parameter 'C', Groundwater and Joint Condition

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

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

Table 7. Engineering classification for intact rock.

Class	Description	Uniaxial Compressive Strength (psi)	Modulus Ratio
A	Very high strength	Over 32,000	
B	High strength	16,000-32,000	
C	Medium strength	8,000-16,000	
D	Low strength	4,000- 8,000	
E	Very low strength	Less than 4,000	
H	High modulus ratio		Over 500
M	Average (medium) ratio		200 - 500
L	Low modulus ratio		Less than 200

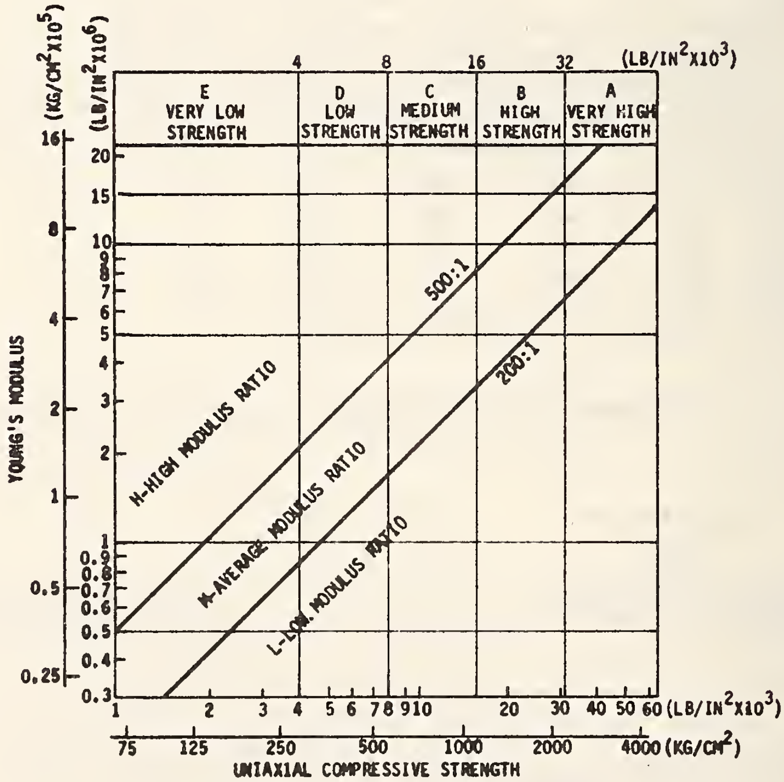

Figure 3. Engineering classification for intact rock.

on Terzaghi's principal categories.[9] Following is a brief description for each of the ground condition classes in an expanded version of this classification system.

1. Intact Rock contains neither joints nor hairline cracks. Breakage occurs across sound rock or is controlled by damage to the rock resulting from excavation. Spalling or popping conditions may occur.

2. Massive, Moderately Jointed Rock contains joints and hairline cracks, but the individual blocks have grown together or are so intimately interlocked that vertical walls need no support. Spalling or popping conditions may be encountered.

3. Stratified or Schistose Rock consists of individual strata with little or no resistance to separation along the boundaries between strata. The strata may or may not be weakened by transverse joints. Spalling is quite common.

4. Moderately Blocky and Seamy Rock consists of chemically intact or nearly intact rock fragments that are entirely separated from each other and imperfectly interlocked. Individual blocks are larger than two feet across. Vertical walls may need support.

5. Very Blocky and Seamy Rock consists of chemically intact or nearly intact rock fragments that are entirely separated from each other and imperfectly interlocked. Individual blocks are less than two feet across. Vertical walls probably need support.

6. Firm Soil and Altered (or Weak) Rock will stand unsupported for several days or longer.

7. Crushed but Chemically Intact Rock consists of fragments that may be partly recemented. Individual particles are of gravel size or smaller (3 inches).

8. Slow Raveling Soil and Altered (or Weak) Rock gradually breaks up into chunks, flakes or angular fragments after a few hours to a few days. The material must be moderately coherent and friable or discontinuous.

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[9] Terzaghi, Karl. "Introduction to Tunnel Geology" in Rock Tunneling with Steel Supports by Robert V. Proctor and Thomas L. White. Commercial Shearing and Stamping Co., Youngstown, Ohio. 1946.

9. Fast Raveling Soil and Altered (or Weak) Rock gradually breaks up into chunks, flakes or angular fragments within a few hours. The material must be moderately coherent and friable or discontinuous.

10. Cohesive Running Ground will invade or run into the tunnel until a stable slope is formed at the heading (slope angle of about 34 degrees to the horizontal) after a brief period of raveling.

11. Running Ground will invade, or run into, the tunnel until a stable slope is formed at the heading (slope angle of about 34 degrees to the horizontal) immediately or almost immediately after being exposed.

12. Squeezing Soil and Altered (or Weak) Rock advances slowly into the tunnel with fracturing and without perceptible volume increase or increase in water content. A prerequisite for squeezing is an overstress of the material in the immediate vicinity of the tunnel opening.

13. Swelling Soil and Altered (or Weak) Rock advances slowly into the tunnel with a considerable volume increase in the ground surrounding the tunnel. Overstressing of the material is not necessary.

14. Very Soft Squeezing Ground advances rapidly into the tunnel in a plastic flow.

15. Flowing Ground is a material in which water and solids together invade the tunnel from all sides, including the bottom, like a viscous liquid. If the flow is not stopped, then it continues until the tunnel is completely filled.

16. Bouldery Ground contains boulders within a matrix of gravel, sand, silt, clay or combination thereof. The material will probably be loosely consolidated or just lightly cemented.

Another method of classifying different ground types use a series of deformation models.[10] The purpose of this system is for idealizing various ground types for theoretical studies in ground dynamics. Some of the models used are:

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[10] Coates, D. F. Rock Mechanics Principles. Mines Branch Monograph 874. Dept. of Energy, Mines and Resources; Mines Branch. Ottawa, Canada. 1965.

1. Linear Elastic Body. A body having a straight line, reversible stress-strain curve.

2. Curvilinear Elastic Body. A body having a curved, reversible stress-strain curve.

3. Bilinear Elastic Body. A body having a reversible stress-strain curve consisting of two straight lines.

4. Elasto-Plastic Body. A body having a stress-strain curve composed of a sloping straight line connected with a horizontal line indicating plastic deformation.

5. Plasto-Elastic Body. A body having a stress-strain curve composed of an initial horizontal line connected to a sloping straight line.

6. Visco-Elastic Body. A body whose deformation varies with both stress level and duration, and is fully recoverable.

7. Visco-Plastic Body. A body whose deformation varies with both stress level and duration, and is not fully recoverable.

8. Locking Medium. A mass having a stress-strain curve composed of an initial, horizontal straight line connected to a vertical straight line. This system is useful for advanced stress distribution studies. However, the concepts involved in this classification system are well advanced of any simple testing techniques that could be used to supply the empirical information required.

TUNNEL AND SUBSURFACE CONDITIONS CLASSIFICATION SYSTEM

Table 8 presents the classification system prepared for use in this study. This system is essentially a listing of the major tunnel and subsurface conditions ranked by class.

The matrix in Figure 4 was prepared to correlate the relationships between subsurface conditions and the major design and construction consideration. More detailed matrix correlations were then prepared (Figures 5 through 10) to show how the subsurface conditions relate to decision making in major tunnel design and construction considerations.

Table 8. Tunnel and subsurface conditions classification system.

MAJOR DIVISIONS	SECONDARY DIVISIONS	CLASS
Tunnel Characteristics	Length	Under 2 miles Over 2 miles
	Size (Cross-Sectional Area)	Under 500 square feet 500 to 1,000 square feet Over 1,000 square feet
	Shape	Circular Oval or Elliptical Horseshoe or Arched Rectangular
	Location	Urban Rural
	Thickness of Cover	100 to 500 feet 50 to 100 feet Under 50 feet
	Proximity to Man-Made Structures	Over 200 feet 50 to 200 feet Under 50 feet Intersects Tunnel
	Excavation Method	Conventional Drill and Blast Boring Machine Full Face Sweeping Head Shield Compressed Air Cut and Cover
	Support Type	None Rock Bolts Unbolted Concrete Segments Poured-in-Place Concrete Shotcrete Steel Sets Steel Ribs and Lagging or Liner Plates Bolted Steel, Cast Iron or Concrete Segments
Ground Type		Hard Ground (Rock)

Table 8. Tunnel and subsurface conditions classification system (continued).

MAJOR DIVISIONS	SECONDARY DIVISIONS		CLASS
Ground Type			Soft Ground (Soil or Rock) Mixed Ground Bouldery Ground
Structural Features (Discontinuities or Anisotropic Layers)	Type and Width	Fractures, Joints, Contacts, Bedding Planes	Under 1 inch
		Faults, Layers, Veins, Dikes, Shear Zones, Open Cavities	Under 10 feet 10 to 25 feet Over 25 feet
	Spacing		Over 10 feet 3 to 10 feet 1 to 3 feet 4 inches to 1 foot Under 4 inches
	Orientation (Strike and Dip)		40° to 90° from Tunnel Axis 10° to 40° from Tunnel Axis 0° to 10° from Tunnel Axis
	Tightness		Tightly Bonded Partly Bonded Open
	Filling Material		Insoluble, Nonswelling, Nonerodible, Tight Crushed Rock Fragments or Sand, Poor Cohesion None Soluble or Erodible Lubricative Clay, Chlorite, Talc, Graphite, Serpentine Swelling Clay
	Surface Character		Rough Smooth Irregular Planar
Ground Conditions	Degree of Anisotropy		Isotropic Moderately Anisotropic Strongly Anisotropic

Table 8. Tunnel and subsurface conditions classification system (continued).

MAJOR DIVISIONS	SECONDARY DIVISIONS	CLASS
Ground Conditions (cont.)	State of Stress	Low Intermediate High
	Unweathered Bedrock Surface	Above Tunnel Roof, Over 1.1 (Tunnel Width & Height) 0.5 Tunnel Width to 1.1 (Tunnel Width & Height) Under 0.5 Tunnel Width Within Tunnel Cross-Section At or Below Tunnel Floor
	Groundwater Volume Rate Inflow	Very Low - No Pumping Required Low - Requires Pumping High - Requires Remedial Action
	Degree of Decomposition or Alteration	Low Moderate High
	Permeability	Impervious Pervious Highly Pervious
	Water Table Depth	At or Below Tunnel Floor Above Tunnel Floor At or Near Ground Surface
	Ground Water Composition	Slightly Corrosive Moderately Corrosive Highly Corrosive
	Moisture Content	Dry or Moist Wet Saturated
	Susceptibility to Earthquakes	Unlikely to Occur Some Susceptibility Highly Susceptible
	Dangerous Gases Present	

Table 8. Tunnel and subsurface conditions classification system (continued).

MAJOR DIVISIONS	SECONDARY DIVISIONS	CLASS
Physical-Mechanical Properties (cont.)	Solubility of Components	Insoluble Soluble Very Soluble
	Density	Low Intermediate High
	Modulus of Rupture	High Intermediate Low
	Porosity	Low Intermediate High
	Creep	Extremely Low Rate Low Rate High Rate
	Variability	None Moderate Extreme

CONDITIONS		CHOICE OF EXCAVATION TECHNIQUE	DETERMINATION OF SUPPORT REQUIREMENTS	EVALUATION OF POTENTIAL WATER PROBLEMS	EVALUATION OF SAFETY HAZARDS	EVALUATION OF POSSIBLE DAMAGE TO OTHER MAN-MADE STRUCTURES	DETERMINATION OF LINING REQUIREMENTS
TUNNEL CHARACTERISTICS	LENGTH	•					
	SIZE (CROSS-SECTIONAL AREA)	•	•	•	•	•	•
	SHAPE	•	•		•	•	•
	LOCATION	•	•			•	•
	THICKNESS OF COVER	•	•		•	•	•
	PROXIMITY TO MAN-MADE STRUCTURES	•	•		•	•	
	EXCAVATION METHOD		•		•	•	•
	SUPPORT TYPE				•		•
GROUND TYPE	SOIL, ROCK OR MIXED	•	•	•	•	•	•
STRUCTURAL FEATURES (DISCONTINUITIES OR ANISOTROPIC LAYERS)	TYPE & WIDTH	•	•	•	•		•
	SPACING	•	•	•	•		•
	ORIENTATION (STRIKE-DIP)	•	•	•	•		•
	TIGHTNESS	•	•	•	•		•
	FILLING MATERIAL	•	•	•	•		•
	SURFACE CHARACTER		•	•	•		•
GROUND CONDITIONS	DEGREE OF ANISOTROPY	•	•	•	•		•
	STATE OF STRESS	•	•		•	•	•
	UNWEATHERED BEDROCK SURFACE	•	•		•	•	•
	GROUNDWATER VOLUME RATE INFLOW	•	•	•	•		•
	DEGREE OF DECOMPOSITION OR ALTERATION	•	•		•		•
	PERMEABILITY			•		•	•
	WATER TABLE DEPTH			•	•	•	•
	GROUND WATER COMPOSITION		•	•	•		•
	MOISTURE CONTENT			•			•
	SUSCEPTIBILITY TO EARTHQUAKES		•		•		•
	DANGEROUS GASES PRESENT				•		
	UNCOMFORTABLE GROUND TEMPERATURES				•		
PHYSICAL-MECHANICAL PROPERTIES	COMPRESSIVE STRENGTH (S&R)	•	•				
	MODULUS OF ELASTICITY (R)		•				
	POISSON'S RATIO (S&R)		•				
	CONSISTENCY (S)	•	•		•		•
	HARDNESS (DRILLABILITY) (R)	•					
	SWELLING CHARACTERISTICS (S&R)		•		•	•	•
	COHESION (S&R)	•	•		•	•	•
	ABRASIVENESS (S&R)	•					
	SHEAR STRENGTH (S&R)		•		•		•
	WEATHERING RESISTANCE (R)		•		•		•
	SOLUBILITY OF COMPONENTS (R)		•	•	•	•	•
	DENSITY (S&R)		•				
	MODULUS OF RUPTURE (R)		•		•		
	POROSITY (S&R)			•			
	CREEP (R)		•				•
VARIABILITY		•	•	•	•	•	•

Figure 4. Tunnel and subsurface conditions important to major design considerations.

CONDITIONS			CONVENTIONAL DRILL & BLAST	FULL FACE BORING MACHINE	SWEEPING HEAD BORING MACHINE	SHIELD	COMPRESSED AIR	CUT & COVER
TUNNEL CHARACTERISTICS	LENGTH	UNDER 2 MILES	•			•	•	•
		OVER 2 MILES	•	•	•	•	•	•
	SIZE (CROSS-SECTIONAL AREA)	UNDER 500 SQUARE FEET	•	•	•	•	•	•
		500 TO 1000 SQUARE FEET	•	•	•	•	•	•
		OVER 1000 SQUARE FEET	•				•	•
	SHAPE	CIRCULAR	•	•	•	•	•	
		OVAL OR ELLIPTICAL	•		•		•	
		HORSESHOE OR ARCHED	•		•		•	•
		RECTANGULAR	•				•	•
	LOCATION	URBAN	•	•	•	•	•	•
		RURAL	•	•	•	•	•	•
	THICKNESS OF COVER	100 TO 500 FEET	•	•	•	•	•	
		50 TO 100 FEET	•	•	•	•	•	•
		UNDER 50 FEET						•
	PROXIMITY TO MAN-MADE STRUCTURES	OVER 200 FEET	•	•	•	•	•	•
		50 TO 200 FEET	•	•	•	•	•	•
		UNDER 50 FEET	•	•	•	•	•	•
		INTERSECTS TUNNEL	•					•
GROUND TYPE	SOIL, ROCK OR MIXED	HARD GROUND (ROCK)	•	•	•			•
		SOFT GROUND (SOIL OR ROCK)		•	•	•	•	•
		MIXED GROUND	•				•	•
		BOULDERY GROUND	•					•
	TYPE & WIDTH: FRACTURES, JOINTS, CONTACTS, BEDDING PLANES	UNDER 1 INCH	•	•	•			•
	TYPE & WIDTH: FAULTS, LAYERS, VEINS, DIKES, SHEAR ZONES, OPEN CAVITIES	UNDER 10 FEET	•	•	•	•		•
		10 TO 25 FEET	•	•	•	•		•
		OVER 25 FEET	•	•	•	•		•
	SPACING	OVER 10 FEET	•	•	•	•		•
		3 TO 10 FEET	•	•	•	•		•
		1 TO 3 FEET	•			•		•
		4 INCHES TO 1 FOOT	•			•		•
		UNDER 4 INCHES	•			•		•
STRUCTURAL FEATURES (DISCONTINUITIES OR ANISOTROPIC LAYERS)	ORIENTATION (STRIKE & DIP)	40° TO 90° FROM TUNNEL AXIS	•	•	•	•		•
		10° TO 40° FROM TUNNEL AXIS	•	•	•	•		•
		0° TO 10° FROM TUNNEL AXIS	•			•		•
	TIGHTNESS	TIGHTLY BONDED	•	•	•	•	•	•
		PARTLY BONDED	•	•	•	•		•
		OPEN	•	•	•	•		•
	FILLING MATERIAL	INSOLUBLE, NONSWELLING, NONERODIBLE, TIGHT	•	•	•	•	•	•
		CRUSHED ROCK FRAGMENTS OR SAND, POOR COHESION	•			•		•
		NONE	•	•	•	•		•
		SOLUBLE OR ERODIBLE	•	•	•	•		•
		LUBRICATIVE CLAY, CHLORITE, TALC, GRAPHITE, SERPENTINE	•	•	•	•	•	•
		SWELLING CLAY				•	•	•

Figure 5. Tunnel and subsurface conditions influencing choice of excavation technique.

CONDITIONS			NONE	ROCK BOLTS	UNBOLTED CONCRETE SEGMENTS	POURED-IN-PLACE CONCRETE	SHOTCRETE	STEEL SETS	STEEL RIBS AND LAGGING OR LINER PLATES	BOLTED STEEL, CAST IRON, OR CONCRETE SEGMENTS
TUNNEL CHARACTERISTICS	SIZE (CROSS-SECTIONAL AREA)	UNDER 500 SQUARE FEET	●	●	●	●	●	●	●	●
		500 TO 1000 SQUARE FEET	●	●	●	●	●	●	●	●
		OVER 1000 SQUARE FEET	●	●	●	●	●	●	●	●
	SHAPE	CIRCULAR	●	●	●	●	●	●	●	●
		OVAL OR ELLIPTICAL	●	●	●	●	●	●	●	●
		HORSESHOE OR ARCHED	●	●	●	●	●	●	●	●
		RECTANGULAR	●	●		●	●	●	●	●
	LOCATION	URBAN		●	●	●	●	●	●	●
		RURAL	●	●	●	●	●	●	●	●
	THICKNESS OF COVER	100 TO 500 FEET	●	●	●	●	●	●	●	●
		50 TO 100 FEET	●	●	●	●	●	●	●	●
		UNDER 50 FEET			●	●	●	●	●	●
	PROXIMITY TO MAN-MADE STRUCTURES	OVER 200 FEET	●	●	●	●	●	●	●	●
		50 TO 200 FEET	●	●	●	●	●	●	●	●
		UNDER 50 FEET	●	●	●	●	●	●	●	●
		INTERSECTS TUNNEL	●	●	●	●	●	●	●	●
	EXCAVATION METHOD	CONVENTIONAL DRILL & BLAST	●	●		●	●	●	●	
		BORING MACHINE-FULL FACE	●	●	●	●	●	●		
		-SWEEPING HEAD	●	●		●	●	●		
		SHIELD							●	●
		COMPRESSED AIR			●	●			●	●
		CUT & COVER			●	●		●	●	●
GROUND TYPE	SOIL, ROCK OR MIXED	HARD GROUND (ROCK)	●	●		●	●	●		
		SOFT GROUND (SOIL OR ROCK)			●	●	●		●	●
		MIXED GROUND			●	●	●	●	●	●
		BOULDERY GROUND			●	●	●		●	●
STRUCTURAL FEATURES (DISCONTINUITIES OR ANISOTROPIC LAYERS)	TYPE & WIDTH: FRACTURES, JOINTS, CONTACTS, BEDDING PLANES	UNDER 1 INCH	●	●	●	●	●	●	●	●
	TYPE & WIDTH: FAULTS, LAYERS, VEINS, DIKES, SHEAR ZONES, OPEN CAVITIES	UNDER 10 FEET			●	●	●	●	●	●
		10 TO 25 FEET			●	●	●	●	●	●
		OVER 25 FEET			●	●	●	●	●	●
	SPACING	OVER 10 FEET	●	●	●	●	●	●	●	●
		3 TO 10 FEET		●	●	●	●	●	●	●
		1 TO 3 FEET			●	●	●	●	●	●
		4 INCHES TO 1 FOOT			●	●	●	●	●	●
		UNDER 4 INCHES			●			●	●	●
	ORIENTATION (STRIKE & DIP)	40° TO 90° FROM TUNNEL AXIS	●	●	●	●	●	●	●	●
		10° TO 40° FROM TUNNEL AXIS		●	●	●	●	●	●	●
		0° TO 10° FROM TUNNEL AXIS		●	●	●	●	●	●	●
	TIGHTNESS	TIGHTLY BONDED	●	●	●	●	●	●	●	●
		PARTLY BONDED		●	●	●	●	●	●	●
		OPEN			●	●	●	●	●	●
	FILLING MATERIAL	INSOLUBLE, NONSWELLING, NONERODIBLE, TIGHT	●	●	●	●	●	●	●	●
		CRUSHED ROCK FRAGMENTS OR SAND, POOR COHESION			●	●	●	●	●	●
		NONE			●	●	●	●	●	●

Figure 6. Tunnel and subsurface conditions influencing choice of support method.

Conditions			None	Rock bolts	Unbolted concrete segments	Poured-in-place concrete	Shotcrete	Steel sets	Steel ribs and lagging or liner plates	Bolted steel, cast iron, or concrete segments
STRUCTURAL FEATURES (CONT.)	FILLING MATERIAL (CONT.)	SOLUBLE OR ERODIBLE			•	•	•	•	•	•
		LUBRICATIVE CLAY, CHLORITE, TALC, GRAPHITE, SERPENTINE			•	•	•	•	•	•
		SWELLING CLAY				•		•	•	•
	SURFACE CHARACTER	ROUGH	•	•	•	•	•	•	•	•
		SMOOTH		•	•	•	•	•	•	•
		IRREGULAR	•	•	•	•	•	•	•	•
		PLANAR		•	•	•	•	•	•	•
GROUND CONDITIONS	DEGREE OF ANISOTROPY	ISOTROPIC	•	•	•	•	•	•	•	•
		MODERATELY ANISOTROPIC	•	•	•	•	•	•	•	•
		STRONGLY ANISOTROPIC	•	•	•	•	•	•	•	•
	STATE OF STRESS	LOW	•	•	•	•	•	•	•	•
		INTERMEDIATE		•	•	•	•	•	•	•
		HIGH				•		•	•	•
	UNWEATHERED BEDROCK SURFACE	ABOVE TUNNEL ROOF, OVER 1.1 (TUNNEL WIDTH & HEIGHT)	•	•	•	•	•	•	•	•
		0.5 (TUNNEL WIDTH) TO 1.1 (TUNNEL WIDTH & HEIGHT)		•	•	•	•	•	•	•
		UNDER 0.5 (TUNNEL WIDTH)			•	•	•	•	•	•
		WITHIN TUNNEL CROSS-SECTION			•	•	•	•	•	•
		AT OR BELOW TUNNEL FLOOR			•	•	•	•	•	•
	GROUND WATER VOLUME RATE INFLOW	VERY LOW-NO PUMPING REQUIRED	•	•	•	•	•	•	•	•
		LOW-REQUIRES PUMPING	•	•	•	•	•	•	•	•
		HIGH-REQUIRES REMEDIAL ACTION		•		•		•	•	•
	DEGREE OF DECOMPOSITION OR ALTERATION	LOW	•	•	•	•	•	•	•	•
		MODERATE		•	•	•	•	•	•	•
		HIGH			•	•	•	•	•	•
	GROUND WATER COMPOSITION	SLIGHTLY CORROSIVE	•	•	•	•	•	•	•	•
		MODERATELY CORROSIVE	•	•	•	•	•	•	•	•
		HIGHLY CORROSIVE	•	•	•	•	•	•	•	•
	SUSCEPTIBILITY TO EARTHQUAKES	UNLIKELY TO OCCUR	•	•	•	•	•	•	•	•
		SOME SUSCEPTIBILITY		•		•	•	•	•	•
		HIGHLY SUSCEPTIBLE				•		•	•	•
PHYSICAL-MECHANICAL PROPERTIES	COMPRESSIVE STRENGTH	VERY HIGH (OVER 32,000 PSI)	•	•	•	•	•	•	•	
		HIGH (16,000 TO 32,000 PSI)	•	•	•	•	•	•	•	
		MEDIUM (4000 TO 16,000 PSI)	•	•	•	•	•	•	•	
		LOW (2000 TO 4000 PSI)		•	•	•	•	•	•	•
		VERY LOW (UNDER 2000 PSI)		•	•	•	•	•	•	•
	MODULES OF ELASTICITY	HIGH (OVER 5×10^6 PSI)	•	•	•	•	•	•	•	
		INTERMEDIATE (1 TO 5×10^6 PSI)		•	•	•	•	•	•	
		LOW (UNDER 1×10^6 PSI)			•	•	•	•	•	•
	POISSON'S RATIO	LOW (UNDER 0.20)	•	•	•	•	•	•	•	
		INTERMEDIATE (0.20 TO 0.30)		•	•	•	•	•	•	
		HIGH (OVER 0.30)			•	•	•	•	•	•

Figure 6. Tunnel and subsurface conditions influencing choice of support method (continued).

CONDITIONS			NONE	ROCK BOLTS	UNBOLTED CONCRETE SEGMENTS	POURED-IN-PLACE CONCRETE	SHOTCRETE	STEEL SETS	STEEL RIBS AND LAGGING OR LINER PLATES	BOLTED STEEL, CAST IRON, OR CONCRETE SEGMENTS
PHYSICAL-MECHANICAL PROPERTIES (CONT.)	CONSISTENCY	HARD			•	•	•		•	
		FIRM OR STIFF			•	•	•		•	
		SOFT							•	•
	SWELLING CHARACTERISTICS	NONSUSCEPTIBLE	•	•	•	•	•	•	•	•
		SOME SUSCEPTIBILITY				•			•	•
		VERY SUSCEPTIBLE								•
	COHESION	VERY COHESIVE	•	•	•	•	•	•		
		SOME COHESION					•		•	•
		NONCOHESIVE								•
	SHEAR STRENGTH	HIGH	•	•	•	•	•	•		
		INTERMEDIATE			•	•	•	•	•	•
		LOW					•		•	•
	WEATHERING RESISTANCE	HIGH	•	•	•	•	•	•	•	•
		INTERMEDIATE		•	•	•	•	•	•	•
		LOW			•	•	•		•	•
	SOLUBILITY OF COMPONENTS	INSOLUBLE	•	•	•	•	•	•	•	•
		SOLUBLE		•	•	•	•	•	•	•
		VERY SOLUBLE			•	•	•		•	•
	DENSITY	LOW	•	•	•	•	•	•	•	•
		INTERMEDIATE	•	•	•	•	•	•	•	•
		HIGH	•	•	•	•	•	•	•	•
	MODULUS OF RUPTURE	HIGH	•	•			•			
		INTERMEDIATE		•	•	•	•	•	•	•
		LOW			•	•	•	•	•	•
	CREEP	EXTREMELY LOW RATE	•	•	•	•	•	•	•	•
		LOW RATE	•	•		•		•	•	•
		HIGH RATE	•	•				•	•	•
VARIABILITY		NONE	•	•	•	•	•	•	•	•
		MODERATE		•	•	•	•	•	•	•
		EXTREME					•	•	•	•

Figure 6. Tunnel and subsurface conditions influencing choice of support method (continued).

CONDITIONS				PUMPING REQUIREMENTS	CORROSION PROBLEMS	POSSIBLE WASHOUTS
TUNNEL CHARACTERISTICS	SIZE (CROSS-SECTIONAL AREA)		UNDER 500 SQUARE FEET	●		●
			500 TO 1000 SQUARE FEET	●		●
			OVER 1000 SQUARE FEET	●		●
GROUND TYPE	SOIL, ROCK OR MIXED		HARD GROUND (ROCK)			
			SOFT GROUND (SOIL OR ROCK)			●
			MIXED GROUND			●
			BOULDERY GROUND			●
STRUCTURAL FEATURES (DISCONTINUITIES OR ANISOTROPIC LAYERS)	TYPE & WIDTH	FRACTURES, JOINTS, CONTACTS, BEDDING PLANES	UNDER 1 INCH	●		
		FAULTS, LAYERS, VEINS, DIKES, SHEAR ZONES, OPEN CAVITIES	UNDER 10 FEET	●		●
			10 TO 25 FEET	●		●
			OVER 25 FEET	●		●
	SPACING		OVER 10 FEET	●		
			3 TO 10 FEET	●		
			1 TO 3 FEET	●		●
			4 INCHES TO 1 FOOT	●		●
			UNDER 4 INCHES	●		●
	ORIENTATION (STRIKE & DIP)		40° TO 90° FROM TUNNEL AXIS	●	●	●
			10° TO 40° FROM TUNNEL AXIS	●	●	●
			0° TO 10° FROM TUNNEL AXIS	●	●	●
	TIGHTNESS		TIGHTLY BONDED			
			PARTLY BONDED	●		●
			OPEN	●		●
	FILLING MATERIAL		INSOLUBLE, NONSWELLING, NONERODIBLE, TIGHT			
			CRUSHED ROCK FRAGMENTS OR SAND, POOR COHESION	●		●
			NONE	●		●
			SOLUBLE OR ERODIBLE	●		●
			LUBRICATIVE CLAY, CHLORITE, TALC, GRAPHITE, SERPENTINE			
			SWELLING CLAY			●
GROUND CONDITIONS	DEGREE OF ANISOTROPY		ISOTROPIC			
			MODERATELY ANISOTROPIC	●		●
			STRONGLY ANISOTROPIC	●		●
	GROUNDWATER VOLUME RATE INFLOW		VERY LOW-NO PUMPING REQUIRED			
			LOW-REQUIRES PUMPING	●		●
			HIGH-REQUIRES REMEDIAL ACTION	●		●
	PERMEABILITY		IMPERVIOUS $K < 0.0001$ CM/SEC			
			PERVIOUS	●	●	●
			HIGHLY PERVIOUS $K > 0.1$ CM/SEC	●	●	●
	WATER TABLE DEPTH		AT OR BELOW TUNNEL FLOOR			
			ABOVE TUNNEL FLOOR	●	●	●
			AT OR NEAR GROUND SURFACE	●	●	●

Figure 7. Tunnel and subsurface conditions influencing evaluation of potential water problems.

CONDITIONS			PUMPING REQUIREMENTS	CORROSION PROBLEMS	POSSIBLE WASHOUTS
GROUND CONDITIONS (CONT.)	GROUND WATER COMPOSITION	SLIGHTLY CORROSIVE	•	•	
		MODERATELY CORROSIVE	•	•	•
		HIGHLY CORROSIVE	•	•	•
	MOISTURE CONTENT	DRY OR MOIST			
		WET	•		
		SATURATED	•		•
PHYSICAL-MECHANICAL PROPERTIES	SOLUBILITY OF COMPONENTS	INSOLUBLE			
		SOLUBLE			•
		VERY SOLUBLE			•
	POROSITY	LOW	•		
		INTERMEDIATE	•		
		HIGH	•		
VARIABILITY		NONE	•	•	•
		MODERATE	•	•	•
		EXTREME	•	•	•

Figure 7. Tunnel and subsurface conditions influencing evaluation of potential water problems (continued).

CONDITIONS			GROUND FALLS	DUST	WATER INFLOW	HEAT AND/OR HUMIDITY	DANGEROUS GAS INFLOW	INTERFERENCE WITH NEARBY ACTIVITIES
TUNNEL CHARACTERISTICS	SIZE (CROSS-SECTIONAL AREA)	UNDER 500 SQUARE FEET	•	•				•
		500 TO 1000 SQUARE FEET	•	•				
		OVER 1000 SQUARE FEET	•	•				
	SHAPE	CIRCULAR	•					•
		OVAL OR ELLIPTICAL	•					•
		HORSESHOE OR ARCHED	•					
		RECTANGULAR	•					
	THICKNESS OF COVER	100 TO 500 FEET	•					
		50 TO 100 FEET	•					
		UNDER 50 FEET	•					•
	PROXIMITY TO MAN-MADE STRUCTURES	OVER 200 FEET						
		50 TO 200 FEET						
		UNDER 50 FEET	•					•
		INTERSECTS TUNNEL	•					•
	EXCAVATION METHOD	CONVENTIONAL DRILL & BLAST	•	•				
		BORING MACHINE-FULL FACE	•	•	•	•		•
		-SWEEPING HEAD	•	•	•	•		•
		SHIELD	•		•			
		COMPRESSED AIR	•		•	•		•
		CUT & COVER	•					
	SUPPORT TYPE	NONE	•					
		ROCK BOLTS	•					•
		UNBOLTED CONCRETE SEGMENTS						•
		POURED-IN-PLACE CONCRETE						•
		SHOTCRETE	•					•
		STEEL SETS	•					•
		STEEL RIBS & LAGGING OR LINER PLATES						•
		BOLTED STEEL, CAST IRON OR CONCRETE SEGMENTS						•
GROUND TYPE	SOIL, ROCK OR MIXED	HARD GROUND (ROCK)	•	•				
		SOFT GROUND (SOIL OR ROCK)	•	•	•			
		MIXED GROUND	•	•	•			•
		BOULDERY GROUND	•	•	•			
STRUCTURAL FEATURES (DISCONTINUITIES OR ANISOTROPIC LAYERS)	TYPE & WIDTH: FRACTURES, JOINTS, CONTACTS, BEDDING PLANES	UNDER 1 INCH	•					
	TYPE & WIDTH: FAULTS, LAYERS, VEINS, DIKES, SHEAR ZONES, OPEN CAVITIES	UNDER 10 FEET	•		•			
		10 TO 25 FEET	•		•			
		OVER 25 FEET			•			
	SPACING	OVER 10 FEET	•					
		3 TO 10 FEET	•					
		1 TO 3 FEET	•					
		4 INCHES TO 1 FOOT	•					
		UNDER 4 INCHES	•					
	ORIENTATION (STRIKE & DIP)	40° TO 90° FROM TUNNEL AXIS	•					
		10° TO 40° FROM TUNNEL AXIS	•					
		0° TO 10° FROM TUNNEL AXIS	•					

Figure 8. Tunnel and subsurface conditions influencing evaluating of safety hazards.

CONDITIONS			GROUND FALLS	DUST	WATER INFLOW	HEAT AND/OR HUMIDITY	DANGEROUS GAS INFLOW	INTERFERENCE WITH NEARBY ACTIVITIES
STRUCTURAL FEATURES (CONT)	TIGHTNESS	TIGHTLY BONDED	•		•			
		PARTLY BONDED	•		•			
		OPEN	•		•			
	FILLING MATERIAL	INSOLUBLE, NONSWELLING, NONERODIBLE, TIGHT	•					
		CRUSHED ROCK FRAGMENTS OR SAND, POOR COHESION	•		•			
		NONE	•		•			
		SOLUBLE OR ERODIBLE	•		•			
		LUBRICATIVE CLAY, CHLORITE, TALC, GRAPHITE, SERPENTINE	•					
		SWELLING CLAY	•					
	SURFACE CHARACTER	ROUGH	•					
		SMOOTH	•					
		IRREGULAR	•					
		PLANAR	•					
GROUND CONDITIONS	DEGREE OF ANISOTROPY	ISOTROPIC	•					
		MODERATELY ANISOTROPIC	•					
		STRONGLY ANISOTROPIC	•					
	STATE OF STRESS	LOW	•					
		INTERMEDIATE	•					
		HIGH	•					
	UNWEATHERED BEDROCK SURFACE	ABOVE TUNNEL ROOF, OVER 1.1 (TUNNEL WIDTH & HEIGHT)	•					
		0.5 (TUNNEL WIDTH) TO 1.1 (TUNNEL WIDTH & HEIGHT)	•					
		UNDER 0.5 (TUNNEL WIDTH)	•					
		WITHIN TUNNEL CROSS-SECTION	•					
		AT OR BELOW TUNNEL FLOOR	•					
	GROUNDWATER VOLUME RATE INFLOW	VERY LOW-NO PUMPING REQUIRED			•	•		
		LOW-REQUIRES REMEDIAL ACTION			•	•		
		HIGH-REQUIRES REMEDIAL ACTION			•	•		•
	DEGREE OF DECOMPOSITION OR ALTERATION	LOW	•					
		MODERATE	•					
		HIGH	•					
	WATER TABLE DEPTH	AT OR BELOW TUNNEL FLOOR			•			
		ABOVE TUNNEL FLOOR			•			
		AT OR NEAR GROUND SURFACE			•			
	GROUND WATER COMPOSITION	SLIGHTLY CORROSIVE			•			
		MODERATELY CORROSIVE			•			
		HIGHLY CORROSIVE			•			
	SUSCEPTIBILITY TO EARTHQUAKES	UNLIKELY TO OCCUR	•		•		•	
		SOME SUSCEPTIBILITY	•		•		•	
		HIGHLY SUSCEPTIBLE	•		•		•	
	DANGEROUS GASES PRESENT						•	
	UNCOMFORTABLE GROUND TEMP.					•		

Figure 8. Tunnel and subsurface conditions influencing evaluating of safety hazards (continued).

CONDITIONS			GROUND FALLS	DUST	WATER INFLOW	HEAT AND/OR HUMIDITY	DANGEROUS GAS INFLOW	INTERFERENCE WITH NEARBY ACTIVITIES
PHYSICAL-MECHANICAL PROPERTIES	CONSISTENCY	HARD	•					
		FIRM OR STIFF	•					
		SOFT	•					
	SWELLING CHARACTERISTICS	NONSUSCEPTIBLE	•					
		SOME SUSCEPTIBILITY	•					
		VERY SUSCEPTIBLE	•					
	COHESION	VERY COHESIVE	•					
		SOME COHESION	•					
		NONCOHESIVE	•					
	SHEAR STRENGTH	HIGH	•					
		INTERMEDIATE	•					
		LOW	•					
	WEATHERING RESISTANCE	HIGH	•					
		INTERMEDIATE	•					
		LOW	•					
	SOLUBILITY OF COMPONENTS	INSOLUBLE	•					
		SOLUBLE	•					
		VERY SOLUBLE	•					
	MODULUS OF RUPTURE	HIGH	•					
		INTERMEDIATE	•					
		LOW	•					
VARIABILITY		NONE	•	•	•		•	•
		MODERATE	•	•	•		•	•
		EXTREME	•	•	•		•	•

Figure 8. Tunnel and subsurface conditions influencing evaluating of safety hazards (continued).

CONDITIONS			RUPTURE	SETTLEMENT	VIBRATION
TUNNEL CHARACTERISTICS	SIZE (CROSS-SECTIONAL AREA)	UNDER 500 SQUARE FEET		•	
		500 TO 1000 SQUARE FEET		•	
		OVER 1000 SQUARE FEET		•	
	SHAPE	CIRCULAR		•	
		OVAL OR ELLIPTICAL		•	
		HORSESHOE OR ARCHED		•	
		RECTANGULAR		•	
	LOCATION	URBAN	•	•	•
		RURAL			
	THICKNESS OF COVER	100 TO 500 FEET		•	
		50 TO 100 FEET		•	•
		UNDER 50 FEET		•	•
	PROXIMITY TO MAN-MADE STRUCTURES	OVER 200 FEET			
		50 TO 200 FEET		•	•
		UNDER 50 FEET	•	•	•
		INTERSECTS TUNNEL	•	•	•
	EXCAVATION METHOD	CONVENTIONAL DRILL & BLAST	•		•
		BORING MACHINE-FULL FACE			
		-SWEEPING FACE			
		SHIELD		•	
		COMPRESSED AIR	•	•	
		CUT AND COVER	•		
GROUND TYPE	SOIL, ROCK OR MIXED	HARD GROUND (ROCK)			•
		SOFT GROUND (SOIL OR ROCK)		•	
		MIXED GROUND		•	
		BOULDERY GROUND		•	
GROUND CONDITIONS	STATE OF STRESS	LOW			
		INTERMEDIATE		•	
		HIGH		•	
	UNWEATHERED BEDROCK SURFACE	ABOVE TUNNEL ROOF, OVER 1.1 (TUNNEL WIDTH & HEIGHT)			
		0.5 (TUNNEL WIDTH) TO 1.1 (TUNNEL WIDTH & HEIGHT)		•	
		UNDER 0.5 (TUNNEL WIDTH)		•	
		WITHIN TUNNEL CROSS-SECTION		•	
		AT OR BELOW TUNNEL FLOOR		•	
	PERMEABILITY	IMPERVIOUS K < 0.0001 CM/SEC			
		PERVIOUS		•	
		HIGHLY PERVIOUS K > 0.1 CM/SEC		•	
	WATER TABLE DEPTH	AT OR BELOW TUNNEL FLOOR			
		ABOVE TUNNEL FLOOR		•	
		AT OR NEAR GROUND SURFACE		•	
PHYSICAL-MECHANICAL PROPERTIES	SWELLING CHARACTERISTICS	NONSUSCEPTIBLE			
		SOME SUSCEPTIBILITY	•	•	
		VERY SUSCEPTIBLE	•	•	

Figure 9. Tunnel and subsurface conditions influencing evaluation of possible damage to other man-made structures.

CONDITIONS			RUPTURE	SETTLEMENT	VIBRATION
PHYSICAL-MECHANICAL PROPERTIES (CONT.)	COHESION	VERY COHESIVE			
		SOME COHESION			
		NONCOHESIVE		•	
	SOLUBILITY OF COMPONENTS	INSOLUBLE			
		SOLUBLE		•	
		VERY SOLUBLE		•	
VARIABILITY		NONE	•	•	
		MODERATE	•	•	
		EXTREME	•	•	

Figure 9. Tunnel and subsurface conditions influencing evaluation of possible damage to other man-made structures (continued).

CONDITIONS				NONE	NOMINAL OR LIGHT	MEDIUM	HEAVY
TUNNEL CHARACTERISTICS	SIZE (CROSS-SECTIONAL AREA)		UNDER 500 SQUARE FEET	•	•	•	•
			500 TO 1000 SQUARE FEET	•	•	•	•
			OVER 1000 SQUARE FEET	•	•	•	•
	SHAPE		CIRCULAR	•	•	•	•
			OVAL OR ELLIPTICAL	•	•	•	•
			HORSESHOE OR ARCHED	•	•	•	•
			RECTANGULAR		•	•	•
	LOCATION		URBAN		•	•	•
			RURAL	•	•	•	•
	THICKNESS OF COVER		100 TO 500 FEET	•	•	•	•
			50 TO 100 FEET		•	•	•
			UNDER 50 FEET		•	•	•
	EXCAVATION METHOD		CONVENTIONAL DRILL & BLAST	•	•	•	•
			BORING MACHINE-FULL FACE	•	•	•	
			-SWEEPING HEAD	•	•	•	
			SHIELD			•	•
			COMPRESSED AIR			•	•
			CUT AND COVER		•	•	•
	SUPPORT TYPE		NONE	•	•		
			ROCK BOLTS	•	•	•	•
			UNBOLTED CONCRETE SEGMENTS	•	•		
			POURED-IN-PLACE CONCRETE	•			
			SHOTCRETE	•	•		
			STEEL SETS	•	•	•	•
			STEEL RIBS & LAGGING OR LINER PLATES	•	•		
			BOLTED STEEL CAST IRON OR CONCRETE SEGMENTS	•	•		
GROUND TYPE	SOIL, ROCK OR MIXED		HARD GROUND (ROCK)	•	•	•	•
			SOFT GROUND (SOIL OR ROCK)			•	•
			MIXED GROUND			•	•
			BOULDERY GROUND			•	•
STRUCTURAL FEATURES (DISCONTINUITIES OR ANISOTROPIC LAYERS)	TYPE & WIDTH	FRACTURES, JOINTS, CONTACTS, BEDDING PLANES	UNDER 1 INCH	•	•	•	•
		FAULTS, LAYERS, VEINS, DIKES SHEAR ZONES, OPEN CAVITIES	UNDER 10 FEET		•	•	•
			10 TO 25 FEET		•	•	•
			OVER 25 FEET			•	•
	SPACING		OVER 10 FEET	•	•	•	•
			3 TO 10 FEET		•	•	•
			1 TO 3 FEET		•	•	•
			4 INCHES TO 1 FOOT			•	•
			UNDER 4 INCHES				•
	ORIENTATION (STRIKE & DIP)		40° TO 90° FROM TUNNEL AXIS	•	•	•	•
			10° TO 40° FROM TUNNEL AXIS		•	•	•
			0° TO 10° FROM TUNNEL AXIS		•	•	•
	TIGHTNESS		TIGHTLY BONDED	•	•	•	•
			PARTLY BONDED		•	•	•
			OPEN			•	•

Figure 10. Tunnel and subsurface conditions influencing determination of lining requirements.

CONDITIONS			NONE	NOMINAL OR LIGHT	MEDIUM	HEAVY
STRUCTURAL FEATURES (CONT.)	FILLING MATERIAL	INSOLUBLE, NONSWELLING, NONERODIBLE, TIGHT	•	•	•	•
		CRUSHED ROCK FRAGMENTS OR SAND, POOR COHESION			•	•
		NONE			•	•
		SOLUBLE OR ERODIBLE			•	•
		LUBRICATIVE CLAY, CHLORITE, TALC, GRAPHITE, SERPENTINE			•	•
		SWELLING CLAY				•
	SURFACE CHARACTER	ROUGH	•	•	•	•
		SMOOTH		•	•	•
		IRREGULAR	•	•	•	•
		PLANAR		•	•	•
GROUND CONDITIONS	DEGREE OF ANISOTROPY	ISOTROPIC	•	•	•	•
		MODERATELY ANISOTROPIC	•	•	•	•
		STRONGLY ANISOTROPIC	•	•	•	•
	STATE OF STRESS	LOW	•	•	•	•
		INTERMEDIATE		•	•	•
		HIGH			•	•
	UNWEATHERED BEDROCK SURFACE	ABOVE TUNNEL ROOF, OVER 1.1 (TUNNEL WIDTH & HEIGHT)	•	•	•	•
		0.5 (TUNNEL WIDTH) TO 1.1 (TUNNEL WIDTH & HEIGHT)		•	•	•
		UNDER 0.5 (TUNNEL WIDTH)		•	•	•
		WITHIN TUNNEL CROSS-SECTION			•	•
		AT OR BELOW TUNNEL FLOOR			•	•
	GROUNDWATER VOLUME RATE INFLOW	VERY LOW-NO PUMPING REQUIRED		•	•	•
		LOW-REQUIRES PUMPING			•	•
		HIGH-REQUIRES REMEDIAL ACTION			•	•
	DEGREE OF DECOMPOSITION OR ALTERATION	LOW	•	•	•	•
		MODERATE		•	•	•
		HIGH			•	•
	PERMEABILITY	IMPERVIOUS $K < 0.0001$ CM/SEC	•	•	•	•
		PERVIOUS		•	•	•
		HIGHLY PERVIOUS $K > 0.1$ CM/SEC			•	•
	WATER TABLE DEPTH	AT OR BELOW TUNNEL FLOOR	•	•	•	•
		ABOVE TUNNEL FLOOR		•	•	•
		AT OR NEAR GROUND SURFACE		•	•	•
	GROUND WATER COMPOSITION	SLIGHTLY CORROSIVE		•	•	•
		MODERATELY CORROSIVE		•	•	•
		HIGHLY CORROSIVE			•	•
	MOISTURE CONTENT	DRY OR MOIST	•	•	•	•
		WET		•	•	•
		SATURATED			•	•
	SUSCEPTIBILITY TO EARTHQUAKES	UNLIKELY TO OCCUR	•	•	•	•
		SOME SUSCEPTIBILITY		•	•	•
		HIGHLY SUSCEPTIBLE			•	•

Figure 10. Tunnel and subsurface conditions influencing determination of lining requirements (continued).

CONDITIONS			NONE	NOMINAL OR LIGHT	MEDIUM	HEAVY
PHYSICAL-MECHANICAL PROPERTIES	CONSISTENCY	HARD		•	•	•
		FIRM OR STIFF		•	•	•
		SOFT			•	•
	SWELLING CHARACTERISTICS	NONSUSCEPTIBLE	•	•	•	•
		SOME SUSCEPTIBILITY		•	•	•
		VERY SUSCEPTIBLE			•	•
	COHESION	VERY COHESIVE	•	•	•	•
		SOME COHESION		•	•	•
		NONCOHESIVE			•	•
	SHEAR STRENGTH	HIGH	•	•	•	•
		INTERMEDIATE		•	•	•
		LOW			•	•
	WEATHERING RESISTANCE	HIGH	•	•	•	•
		INTERMEDIATE		•	•	•
		LOW			•	•
	SOLUBILITY OF COMPONENTS	INSOLUBLE	•	•	•	•
		SOLUBLE			•	•
		VERY SOLUBLE			•	•
	CREEP	EXTREMELY LOW RATE	•	•	•	•
		LOW RATE	•	•	•	•
		HIGH RATE	•	•	•	•
VARIABILITY		NONE	•	•	•	•
		MODERATE		•	•	•
		EXTREME			•	•

Figure 10. Tunnel and subsurface conditions influencing determination of lining requirements (continued).

To illustrate the use of these matrixes for preliminary decision making in tunnel design, we prepared an example with the use of an assumed tunnel configuration and location. Figure 11 shows the assumed general geological conditions and location for this example tunnel. Table 9 gives the values which were assumed for each of the specific design and construction requirements. Using Table 9 in conjunction with Figures 5 through 10, results in preliminary decisions for each of the major design and construction considerations (Table 10).

TECHNIQUES FOR DETERMINING SUBSURFACE CONDITIONS

The purpose of a subsurface investigation is to provide information about subsurface conditions. The quantity and quality of information provided depends upon the intended use of the information, available resources, the applied subsurface investigation techniques, and the accuracy of data interpretation.

PERSONNEL QUALIFICATIONS AND EXPERIENCE

Of primary importance to a tunnel site evaluation is the individual selected to direct the investigation, interpret the evidence, and then present the findings in a concise and usable form to those responsible for design and construction. Ideally, this person should be an Engineering Geologist or Geological Engineer with extensive, practical field experience in reconnaissance and detailed preexcavation studies for tunnels and other major construction sites. He should have a thorough understanding of the pertinent and proven geological, geophysical, and soil and rock mechanics methods as well as being familiar with the newly evolving techniques which could be useful. He should also possess the managerial ability to plan a systematic, but flexible, investigation program and direct the exploration team.

Perhaps the most valuable experience which can qualify the tunnel site exploration manager is a background of having followed tunnel construction projects from their inception, into subsurface investigations, and on through their excavation, with an emphasis being placed on comparing the actual conditions encountered against those predicted. This experience of performing explorations, making geologic projections, and then seeing them proven or disproven cannot be duplicated by any amount of "book learning."

The experienced manager, with the help of qualified assistants, is able to sift and weigh the geological, geophysical, and laboratory test

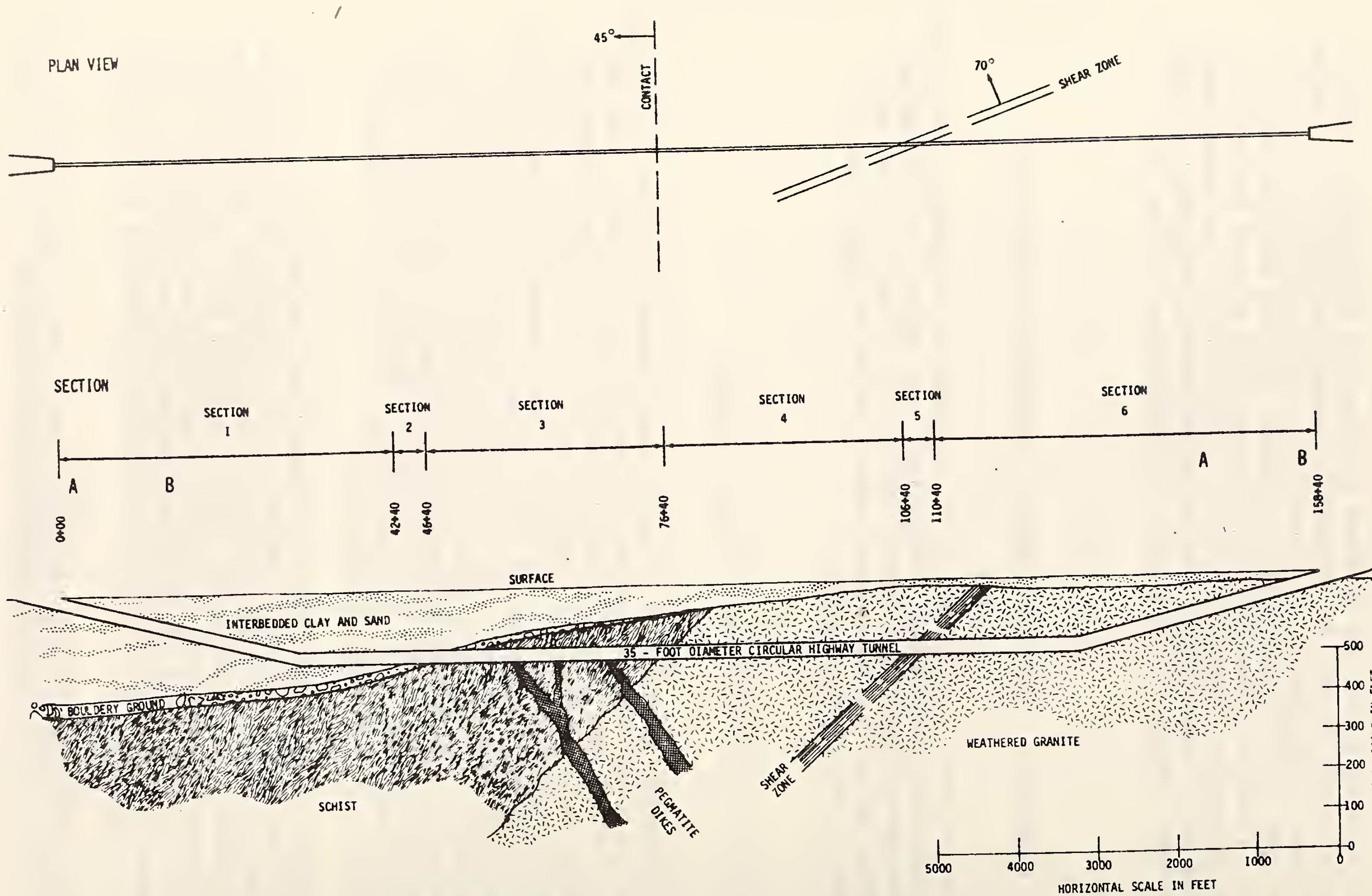

Figure 11. Assumed geologic conditions for example tunnel.

Table 9. Assumed tunnel and subsurface conditions for example tunnel.

CONDITIONS		SECTION 1	SECTION 2	SECTION 3	SECTION 4	SECTION 5	SECTION 6
TUNNEL CHARACTERISTICS	LENGTH	4240'	400'	3000'	3000'	400'	4800'
	SIZE	35' DIAM	35' DIAM	35' DIAM	35' DIAM	35' DIAM	35' DIAM
	SHAPE	RECTANGULAR-CIRCULAR	CIRCULAR	CIRCULAR	CIRCULAR	CIRCULAR	CIRCULAR
	LOCATION	URBAN	URBAN	URBAN	URBAN	URBAN	URBAN
	THICKNESS OF COVER	0-150'	150'	150'	150'	150'	150'
	PROXIMITY TO MAN-MADE STRUCTURES	INTERSECTING	30'-80'	40'-80'	80'-130'	140'	40'-140'
GROUND TYPE	SOIL, ROCK OR MIXED	SOFT GROUND (SOIL)	BOULDERY GROUND	HARD GROUND (ROCK)	HARD GROUND (ROCK)	HARD GROUND (ROCK)	HARD GROUND (ROCK)
STRUCTURAL FEATURES (DISCONTINUITIES OR ANISOTROPIC LAYERS)	TYPE & WIDTH	BEDDING	NONE	JOINTS / DIKES	JOINTS < 1"	SHEAR PLANES	JOINTS < 1"
	SPACING	1'-4'	-	3'-10' / >10'	3'-10'	4"-1'	3'-10'
	ORIENTATION (STRIKE-DIP)	FLAT	-	60°-60° / 90°-55°	20°-45° & 90°-45°	20°-45°	20°-45° & 90°-45°
	TIGHTNESS	PARTLY BONDED	-	PARTLY BONDED / TIGHTLY BONDED	OPEN	PARTLY BONDED	OPEN
	FILLING MATERIAL	-	-	CHLORITE / PEGMATITE	NONE	INERT GOUGE	NONE
	SURFACE CHARACTER	SMOOTH PLANAR	-	SMOOTH PLANAR / IRREGULAR NONPLANAR	ROUGH PLANAR	ROUGH PLANAR	ROUGH PLANAR
GROUND CONDITIONS	DEGREE OF ANISOTROPY	STRONGLY ANISOTROPIC	ISOTROPIC	STRONGLY ANISOTROPIC	ISOTROPIC	MOD. ANISOTROPIC	ISOTROPIC
	STATE OF STRESS	LOW	LOW	LOW	LOW	LOW	LOW
	UNWEATHERED BEDROCK SURFACE	BELOW TUNNEL FLOOR	BELOW TUNNEL X-SECT. TO WITHIN TUNNEL	ABOVE TUNNEL X-SECT.	ABOVE TUNNEL X-SECT.	ABOVE TUNNEL X-SECT.	WITHIN TUNNEL TO ABOVE TUNNEL X-SECT.
	GROUNDWATER VOLUME RATE INFLOW	LOW	HIGH	VERY LOW	LOW	HIGH	LOW
	DEGREE OF DECOMPOSITION	LOW	LOW	MODERATE	MODERATE	HIGH	MODERATE
	PERMEABILITY	PERVIOUS	HIGHLY PERVIOUS	IMPERVIOUS	PERVIOUS	HIGHLY PERVIOUS	PERVIOUS
	WATER TABLE DEPTH	NEAR GROUND SURFACE	NEAR GROUND SURFACE	NEAR GROUND SURFACE	NEAR GROUND SURFACE	NEAR GROUND SURFACE	NEAR GROUND SURFACE
	GROUND WATER COMPOSITION	SLIGHTLY CORROSIVE	SLIGHTLY CORROSIVE	SLIGHTLY CORROSIVE	SLIGHTLY CORROSIVE	SLIGHTLY CORROSIVE	SLIGHTLY CORROSIVE
	MOISTURE CONTENT	SATURATED	SATURATED	DRY OR MOIST	SATURATED	SATURATED	SATURATED
	SUSCEPTIBILITY TO EARTHQUAKES	UNLIKELY TO OCCUR	UNLIKELY TO OCCUR	UNLIKELY TO OCCUR	UNLIKELY TO OCCUR	UNLIKELY TO OCCUR	UNLIKELY TO OCCUR
	DANGEROUS GAS	NONE	NONE	NONE	NONE	NONE	NONE
	GROUND TEMPERATURE	MODERATE	MODERATE	MODERATE	MODERATE	MODERATE	MODERATE
PHYSICAL-MECHANICAL PROPERTIES	COMPRESSIVE STRENGTH	VERY LOW	VERY LOW	MEDIUM	MEDIUM	LOW	MEDIUM
	MODULUS OF ELASTICITY	NOT APPLICABLE	NOT APPLICABLE	INTERMEDIATE	INTERMEDIATE	LOW	INTERMEDIATE
	POISSON'S RATIO	HIGH	HIGH	INTERMEDIATE	INTERMEDIATE	HIGH	INTERMEDIATE
	CONSISTENCY	FIRM	SOFT	NOT APPLICABLE	NOT APPLICABLE	NOT APPLICABLE	NOT APPLICABLE
	HARDNESS (DRILLABILITY)	SOFT	MIXED HARD & SOFT	MEDIUM	MEDIUM	SOFT	MEDIUM
	SWELLING CHARACTERISTICS	NONSUSCEPTIBLE	NONSUSCEPTIBLE	NONSUSCEPTIBLE	NONSUSCEPTIBLE	NONSUSCEPTIBLE	NONSUSCEPTIBLE
	COHESION	SOME COHESION	SOME COHESION	VERY COHESIVE	VERY COHESIVE	SOME COHESION	VERY COHESIVE
	ABRASIVENESS	MEDIUM	HIGH	HIGH	HIGH	MEDIUM	HIGH
	SHEAR STRENGTH	INTERMEDIATE	LOW	INTERMEDIATE	HIGH	LOW	HIGH
	WEATHERING RESISTANCE	INTERMEDIATE	LOW	HIGH	HIGH	LOW	HIGH
	SOLUBILITY OF COMPONENTS	INSOLUBLE	INSOLUBLE	INSOLUBLE	INSOLUBLE	INSOLUBLE	INSOLUBLE
	DENSITY	INTERMEDIATE	INTERMEDIATE	INTERMEDIATE	INTERMEDIATE	INTERMEDIATE	INTERMEDIATE
	MODULUS OF RUPTURE	NOT APPLICABLE	NOT APPLICABLE	INTERMEDIATE	INTERMEDIATE	LOW	INTERMEDIATE
	POROSITY	HIGH	HIGH	LOW	LOW	LOW	LOW
	CREEP	NOT APPLICABLE	NOT APPLICABLE	EXTREMELY LOW RATE	EXTREMELY LOW RATE	EXTREMELY LOW RATE	EXTREMELY LOW RATE
VARIABILITY		MODERATE	NONE	MODERATE	MODERATE	NONE	MODERATE

Table 10. Preliminary analysis of design and construction considerations for example tunnel.

GEOLOGIC SEGMENT		APPLICABLE EXCAVATION TECHNIQUES	APPLICABLE SUPPORT METHODS	EVALUATION OF POTENTIAL WATER PROBLEMS	EVALUATION OF SAFETY HAZARDS	EVALUATION OF POSSIBLE DAMAGE TO OTHER MAN-MADE STRUCTURES	EXPECTED LINING REQUIREMENTS
SECTION 1	A	CUT AND COVER	POURED-IN-PLACE CONCRETE, STEEL SETS AND LAGGING DR LINER PLATES, BOLTED STEEL, CAST IRON, OR CONCRETE SEGMENTS.	MODERATE PUMPING REQUIRED	NORMAL	DISRUPTION OF STREETS, SEWERS, AND UTILITY LINES	MEDIUM
	B	SHIELD	UNBOLTED CONCRETE SEGMENTS, POURED-IN-PLACE CONCRETE, STEEL SETS AND LAGGING OR LINER PLATES, BOLTED STEEL, CAST IRON, DR CONCRETE SEGMENTS.	MODERATE PUMPING REQUIRED	NORMAL	INSIGNIFICANT	MEDIUM
SECTION 2		CONVENTIONAL DRILL AND BLAST AND/OR HAND MINING	UNBOLTED CONCRETE SEGMENTS, POURED-IN-PLACE CONCRETE, STEEL SETS AND LAGGING OR LINER PLATES, BOLTED STEEL, CAST IRON, OR CONCRETE SEGMENTS.	MAJOR PUMPING AND REMEDIAL ACTION REQUIRED	ABOVE NORMAL ROCK FALL AND ACTIVITY INTERFERENCE HAZARDS	RUNNING GROUND MAY CAUSE SUBSIDENCE AND CONSEQUENT SETTLING DF BUILDINGS AND RUPTURED SEWER LINES	MEDIUM TO HEAVY
SECTION 3		FULL FACE BORING MACHINE, SWEEPING HEAD BORING MACHINE, CONVENTIONAL DRILL & BLAST.	ROCK BOLTS, SHOTCRETE, OR STEEL SETS	VERY MINOR PUMPING REQUIRED	NORMAL	INSIGNIFICANT	LIGHT
SECTION 4		FULL FACE BORING MACHINE, SWEEPING HEAD BORING MACHINE, CONVENTIONAL DRILL & BLAST.	ROCK BOLTS, SHOTCRETE, OR STEEL SETS	MINOR PUMPING REQUIRED	NORMAL	INSIGNIFICANT	LIGHT
SECTION 5		CONVENTIONAL DRILL & BLAST	UNBOLTED CONCRETE SEGMENTS, POURED-IN-PLACE CONCRETE, STEEL SETS AND LAGGING DR LINER PLATES, BOLTED STEEL, CAST IRON, OR CONCRETE SEGMENTS	MODERATE TO MAJOR PUMPING AND REMEDIAL ACTION REQUIRED	ABOVE NORMAL ROCK FALL AND ACTIVITY INTERFERENCE HAZARDS	CAVING GROUND MAY CAUSE SUBSIDENCE AND CONSEQUENT SETTLING DF BUILDINGS AND RUPTURED SEWER LINES	MEDIUM TO HEAVY
SECTION 6	A	FULL FACE BORING MACHINE, SWEEPING HEAD BORING MACHINE, CONVENTIONAL DRILL & BLAST.	ROCK BOLTS, SHOTCRETE, OR STEEL SETS	MINOR PUMPING REQUIRED	NORMAL	INSIGNIFICANT	LIGHT
	B	CUT & COVER	POURED-IN-PLACE CONCRETE, STEEL SETS AND LAGGING OR LINER PLATES, BOLTED STEEL, CAST IRON, DR CONCRETE SEGMENTS	MODERATE PUMPING REQUIRED	NORMAL	DISRUPTION DF STREETS, SEWERS, AND UTILITY LINES	MEDIUM

data developed, discounting misleading information, combining the important factors, and then applying an intuitive, mature judgement to the development of the principal end product of the investigation: a geological section along the most feasible tunnel route, displaying and describing all of the important ground conditions expected to be encountered.

In addition to a competent manager, the exploration team should include experienced technical specialists from the fields of engineering geology, engineering geophysics, and soil and/or rock mechanics as needed. Preferably, a high percentage of these field personnel will have accumulated substantial experience from similar work in areas having geologic conditions similar to those expected at the assigned site, such as thick alluvium, sedimentary rocks, or crystalline igneous rocks. The group should be completed with members having predominant experience in geologic environments other than that of the assigned site so that they may contribute useful ideas from somewhat different points of view, and with recent graduates who are familiar with the new techniques being taught, such as the use of computers for efficient data reduction. All personnel should obtain a clear understanding of the reasons for conducting each step of the site investigation program along with the degree of detail and precision required in each operation.

TUNNEL SITE INVESTIGATION

A tunnel site investigation program is usually conducted in several stages. In many cases, the difference between successive stages will only be in the amount of detailed information obtained. The same exploration techniques may, therefore, be used in more than one of the investigative stages, but with progressively more detailed information being obtained in the latter stages.

The first step in any subsurface investigation for a tunnel is the office study. This includes a search for and then examination of all available material which might contain information about the area of the proposed tunnel site. Such material would include such items as previous reports, the literature, maps, and photographs.

After the office study, the next step usually taken is a surface geologic reconnaissance. This should entail a visit to the site where the information gathered in the office study can be visually confirmed and additional major geologic features, recent changes in surface features, and accessibility for further exploration work determined. The area that is reconnoitered should encompass a sufficient distance on either side of the proposed tunnel alignment to enable the observer to view all of the features which could be of importance to the tunnel.

A detailed geologic mapping of the surface along the tunnel line is the next step taken. The emphasis of this step should be on the engineering properties of the ground material and its structural features. The width of the area mapped will depend upon the overburden condition and geologic structure of the area. In addition to the surface map, geologic sections which extend down to the proposed tunnel horizon are prepared. Depending upon the amount of information obtained during the office study, the reconnaissance and detailed mapping steps may be done at the same time.

In most instances, especially where the geology is complex, projections of surface features down to tunnel grade is not sufficiently accurate for a suitable tunnel design. One or more supplementary methods must then be used to obtain additional information so that the projections of the subsurface conditions existing along the tunnel alignment can be made with sufficient accuracy. Several possibilities are available for obtaining this necessary supplementary or additional information.

Geophysical surveys are one means of obtaining information about the ground mass conditions. The various geophysical techniques will furnish data relatively quick at a reasonable cost, but their effectiveness is dependent upon proper correlations. When properly interpreted, geophysical surveys can help in selecting the best borehole locations as well as furnishing information about geologic conditions between outcrops and/or boreholes. In some cases, they can also provide useful data on a physical property such as the in situ determination of the seismic modulus of elasticity.

Drill holes can provide samples for geologic classification and laboratory test. The hole can also be used to observe in situ conditions by using borehole TV, cameras, or periscopes. Borehole logging tools can also be used in the drill hole to obtain additional information concerning the ground conditions. Various in situ tests may also occasionally be conducted in the drill holes.

In some areas of complex geology, the exploration methods already mentioned may still not furnish the desired amount of information. In such cases, excavations such as trenches, adits, or pilot tunnels may be considered. These excavations are excellent for direct observation and determination of ground conditions along the tunnel alignment. Excavations such as pilot tunnels and adits are expensive and usually only the larger projects can afford them. Tradeoffs are considered where such excavations might have auxiliary use such as for drainage or ventilation during construction of the main tunnel.

Laboratory tests on sample specimens are used to determine various physical and mechanical properties of the materials expected to be encountered in the tunnel.

COMPARISON OF TECHNIQUES

A substantial amount of data was collected during the course of this study for identification and study of the various subsurface exploration techniques now available for use in an exploration system applicable to highway tunnel site investigations. Detailed descriptions were made for each of the techniques considered to have possible application to highway tunnel site investigations. These detailed descriptions are presented in the appendixes.

To facilitate comparisons, data on each technique has been summarized on a short form containing 17 major comparison items.

To further facilitate comparisons, the various techniques were classified into several groupings. The groupings used and the techniques included within each are:

A. Preliminary Geological Investigations
   1. Information search
   2. Preliminary site inspection

B. Geological Mapping
   1. Remote sensing
   2. Direct surface mapping

C. Geophysical Surveys
   1. Seismic
   2. Electrical resistivity
   3. Magnetic
   4. Electromagnetic
   5. Gravity
   6. Radiometric
   7. Acoustic holography

D. Sampling
   1. Surface
   2. Subsurface
      a. Drilling equipment
      b. Disturbed sampling methods
      c. Undisturbed sampling methods

E. Borehole Logging
   1. Visual or photographic
   2. Spontaneous potential
   3. Conventional resistivity
   4. Microresistivity
   5. Focusing electrode
   6. Induction
   7. Gamma ray
   8. Neutron
   9. Formation density
   10. Sonic
   11. Gravitational

12. Caliper
13. Temperature

F. Exploration Excavations and In Situ Testing
   1. Trenches and pilot tunnels
   2. In situ testing
      a. Soil shearing strength
      b. State of stress
      c. In situ permeability

G. Laboratory Testing
   1. Soil
      a. Gradation
      b. Moisture content
      c. Unit weight and porosity
      d. Consistency
      e. Shearing strength
      f. Unconfined compressive strength
      g. Permeability
      h. Swelling
   2. Rock
      a. Identification
      b. Specific gravity and porosity
      c. Hardness and abrasion resistance
      d. Weathering resistance
      e. Uniaxial compressive strength
      f. Uniaxial shear strength
      g. Uniaxial tensile strength
      h. Uniaxial flexural strength
      i. Triaxial compressive and shear strengths
      j. Static elastic constants
      k. Dynamic elastic constants
      l. Permeability
      m. Creep

The detailed descriptions for each of the above listed subsurface exploration techniques appears in the appendixes at the end of this report as follows:

Preliminary geological investigations in Appendix A,
Geological mapping in Appendix B,
Geophysical surveys in Appendix C,
Sampling in Appendix D,
Borehole logging in Appendix E,
Exploration excavations and in situ testing in Appendix F, and
Laboratory testing in Appendix G.

The short forms completed on each of the techniques were also grouped together in the same classifications as above. These groupings of techniques are given in Tables 11 through 16. These tables can be used to obtain a comparison between techniques.

Table 11. Comparison of subsurface exploration techniques, preliminary studies and geologic mapping.

CONDITIONS	PRELIMINARY STUDIES		GEOLOGIC MAPPING	
	INFORMATION SEARCH	PRELIMINARY SITE INSPECTION	BY REMOTE SENSING	DIRECT SURFACE MAPPING
1. SITE SELECTION	RECONNAISSANCE.	RECONNAISSANCE.	RECONNAISSANCE.	RECONNAISSANCE AND DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	DETERMINE GENERAL GEOLOGICAL CONDITIONS OF AREA TO PLAN WHAT INFORMATION AND FUTURE EXPLORATION WORK IS REQUIRED.	DETERMINE EXISTING STATUS OF AREA, VIEW GENERAL GEOLOGICAL CONDITIONS FIRST HAND.	PREPARATION OF TOPOGRAPHIC MAPS, IDENTIFICATION OF GEOLOGIC STRUCTURES AND SOIL AND ROCK TYPES, AND DETERMINATION OF HYDROLOGIC CONDITIONS.	DETERMINE GEOLOGIC STRUCTURE OF AREA, LOCATE SURFACE STRUCTURAL FEATURES, MAKE PICTORIAL REPRESENTATION OF AREA GEOLOGY, PREDICT SUBSURFACE CONDITIONS ALONG TUNNEL ALLIGNMENT.
3. GEOLOGICAL ENVIRONMENT	ANY.	ANY.	ANY.	ANY.
4. PRINCIPLE USED	REVIEW OF ALL PREVIOUSLY RECORDED PERTINENT INFORMATION AVAILABLE ABOUT AREA.	VISUAL OBSERVATION.	DIFFERENT MATERIALS EXHIBIT DIFFERENT ELECTROMAGNETIC CHARACTERISTICS.	MEASUREMENT OF EXPOSED GEOLOGICAL FEATURES.
5. QUANTITIES MEASURED	ALL PERTINENT DATA OBTAINED.	MAJOR EXPOSED GEOLOGICAL AND SURFACE FEATURES ROUGHLY OR APPROXIMATELY NOTED.	ENERGY EMITTED AND REFLECTED FROM THE GROUND SURFACE AT WAVELENGTHS THROUGHOUT THE ELECTROMAGNETIC SPECTRUM.	ORIENTATION AND SIZE OF EXPOSED GEOLOGICAL STRUCTURAL FEATURES, ORIENTATION OF GEOLOGIC CONTACTS.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	TENTATIVE CONCLUSIONS BASED UPON AMOUNT AND QUALITY OF INFORMATION OBTAINED.	FORMULATE DETAILED PLAN FOR FUTURE EXPLORATION WORK.	RELATIVE ELEVATIONS FROM PHOTOS. IN OTHER METHODS QUANTITIES COMPUTED ARE INTERMEDIATE FOR COMPARISONS, FINAL RESULTS ARE IDENTIFICATION OF MATERIALS AND FEATURES.	LATERAL AND VERTICAL PROJECTION OF EXPOSED STRUCTURAL FEATURES, AND SOIL AND ROCK UNITS.
7. COVERAGE	ALL POTENTIAL TUNNEL ROUTES	LINEAR BAND ALONG PROPOSED TUNNEL ALLIGNMENT.	DEPENDS UPON HEIGHT OF SENSING PLATFORM, AND LENS OR TYPE OF EQUIPMENT USED.	STRIP ALONG TUNNEL ALIGNMENT SUFFICIENTLY WIDE TO INCLUDE FEATURES NECESSARY TO ALLOW INTELLIGENT PROJECTION OF GEOLOGY TO TUNNEL LEVEL.
8. EFFECTIVE DEPTH	NOT APPLICABLE	SURFICIAL.	MOSTLY SURFICIAL BUT A FEW METHODS HAVE VARYING DEPTH PENETRATION.	SURFICIAL DIRECTLY
9. LIMITATIONS	USUALLY ONLY FURNISHES GENERAL INFORMATION	INFORMATION OBTAINED IS NOT IN MUCH DETAIL AND MAY BE MISINTERPRETED.	GROUND CHECKING REQUIRED FOR POSITIVE MATERIAL IDENTIFICATION.	ONLY DEPICTS SURFACE EXPOSURE OF FEATURES WITH ACCURACY. MAP SCALE USED DETERMINES MINIMUM SIZE OF FEATURES SHOWN.
10. SENSITIVITY	DEPENDS UPON THOROUGHNESS OF INFORMATION OBTAINED	DEPENDS UPON COMPLEXITY OF GEOLOGY IN AREA.	VARIABLE FROM METHOD TO METHOD.	DEPENDS UPON COMPLEXITY OF GEOLOGY IN AREA AND EXTENT OF EXPOSURE FOR STRUCTURAL FEATURES.
11. POSSIBLE ERRORS IN INTERPRETATION	NOT ALL PERTINENT FACTS MAY BE FURNISHED.	SUBSURFACE CONDITIONS CAN NOT BE SEEN. COMPLEXITY OF GEOLOGY MAY NOT BE APPARENT.	INTERPRETATION MAY BE ATTEMPTED BEYOND THE DISCRIMINATION CAPABILITY OF A PARTICULAR METHOD. MISINTERPRETATION ALSO RESULTS FROM INEXPERIENCE.	INSUFFICIENT AREA MAPPED. NOT ALL PERTINENT FEATURES MAPPED IN SUFFICIENT DETAIL. LIMITED OUTCROPS MAY CAUSE MISINTERPRETATION.
12. PRINCIPAL INSTRUMENTS USED	MAPS, REPORTS, PHOTOGRAPHS, RECORDS, PROFESSIONAL PUBLICATIONS.	BRUNTON COMPASS, ROCK PICK, HAND LENS, CAMERA.	CAMERAS, OPTICAL-MECHANICAL SCANNERS, PASSIVE-MICROWAVE SCANNERS, RADAR, AND INFRARED AND MICROWAVE RADIOMETERS.	PLANE TABLE AND ALIDADE, COMPASS, TRANSIT, CAMERA.
13. ENERGY SOURCES	NOT APPLICABLE.	NOT APPLICABLE.	BATTERIES AND POWER FROM AIRCRAFT SYSTEMS.	NOT APPLICABLE.
14. CREW SIZE REQUIRED	ONE OR MORE.	ONE OR MORE.	2 TO 10.	2 TO 4.
15. TIME REQUIRED AND WORK ACCOMPLISHED	DEPENDS UPON SIZE AND COMPLEXITY OF AREA STUDIED AND UPON QUANTITY OF REFERENCE MATERIAL AVAILABLE.	GENERALLY LESS THAN ONE MAN-MONTH.	DATA ACQUISITION CAN COVER SEVERAL HUNDRED SQUARE MILES PER DAY. DATA REDUCTION AND INTERPRETATION ACCOUNT FOR THE MAJOR PORTION OF TIME CONSUMED.	DEPENDS UPON AMOUNT OF EXPOSURES, COMPLEXITY OF GEOLOGY, SCALE OF MAPPING AND WEATHER.
16. COST	GENERALLY \$1,000-\$10,000.	GENERALLY \$1,000- \$5,000.	DATA ACQUISITION FOR 15 SQUARE MILE AREA VARIES FROM \$1,000-\$15,000. INTERPRETATION OFTEN COSTS AS MUCH AS ACQUISITION.	DEPENDS UPON AMOUNT OF EXPOSURES, COMPLEXITY OF GEOLOGY, SCALE OF MAPPING AND WEATHER.
17. POTENTIAL AREAS OF IMPROVEMENT	MORE DOCUMENTATION OF PREVIOUS WORK FOR EASY RETRIVAL BY OTHERS.	INSURE INVESTIGATOR HAS PROPER TRAINING AND EXPERIENCE.	BETTER DISCRIMINATION BETWEEN EARTH MATERIALS, INCREASED DEPTH PENETRATION OF INVESTIGATION.	MORE THOROUGH ATTENTION PAID TO DETAIL OF STRUCTURAL FEATURES, ESPECIALLY DISCONTINUITIES.

Table 12/. Comparison of subsurface exploration techniques, geophysical.

CONDITIONS	GEOPHYSICAL			
	SEISMIC REFRACTION	SEISMIC REFLECTION	ELECTRICAL RESISTIVITY	MAGNETIC
1. SITE SELECTION STAGE	RECONNAISSANCE AND DETAILED INVESTIGATION.	RECONNAISSANCE AND DETAILED INVESTIGATION.	RECONNAISSANCE AND DETAILED INVESTIGATION.	RECONNAISSANCE.
2. ENGINEERING APPLICATION	DETERMINE DEPTH TO BEDROCK, WATER TABLE, AND UNWEATHERED ROCK. DETECT AND TRACE GEOLOGIC STRUCTURES AND BURIED CHANNELS. DETERMINE S-WAVE AND P-WAVE VELOCITIES FOR DERIVING ROCK PROPERTIES.	DETERMINE DEPTH TO VARIOUS STRATA AND CONTINUITY OF ROCK LAYERS. LOCATE DISCONTINUITIES. PROVIOE DATA ON STRATIGRAPHY.	DETERMINE DEPTH TO BEDROCK, VARIOUS STRATA AND WATER TABLE. LOCATE FAULT ZONES.	OETECT FAULTS AND CONTACTS. OUTLINE INTRUSIVE DIKES. LOCATE BURIED PIPELINES. OCCASIONALLY DETERMINE DEPTH OF WEATHERING AND DISTRIBUTION OF ALTERATION.
3. GEOLOGICAL ENVIRONMENT	ANY	PRIMARILY SEDIMENTARY ROCKS.	ANY, BUT PRIMARILY FOR OVERBURDEN.	ANY, BUT PRIMARILY IGNEOUS.
4. PRINCIPLE USED	DIFFERENCE IN SEISMIC-WAVE VELOCITIES THROUGH MEDIA OF DIFFERENT DENSITIES.	DIFFERENCE IN SEISMIC-WAVE VELOCITIES THROUGH MEDIA OF DIFFERENT DENSITIES.	DIFFERENCES IN ELECTRICAL RESISTIVITY OR CONDUCTIVITY OF INDIVIDUAL STRATA.	OIFFERENCES IN MAGNETIC FIELO INTENSITIES BETWEEN READING STATIONS.
5. QUANTITIES MEASURED	SEISMIC-WAVE TRAVEL TIME BETWEEN ENERGY SOURCE AND GEOPHONES, DISTANCE BETWEEN ENERGY SOURCE AND GEOPHONES AND DISTANCE BETWEEN GEOPHONES. ACCURATE TO ± 0.0005 TO 0.002 SECONDS = ± 5 TO 25 FEET.	SEISMIC-WAVE TRAVEL TIME BETWEEN ENERGY SOURCE AND GEOPHONES, DISTANCE BETWEEN ENERGY SOURCE AND GEOPHONES, AND DISTANCE BETWEEN GEOPHONES. ACCURATE TO ± 0.0005 TO 0.002 SECONDS = ± 5 TO 25 FEET DECREASING WITH DEPTH.	CURRENT INPUT, POTENTIAL DIFFERENCE, AND ELECTRODE SPACING. MEASURES FROM 0.003 TO 10,000 OHMS WITH ACCURACY OF ± 0.2 OHMS GENERALLY.	MAGNETIC FIELD INTENSITY. ACCURATE TO ± 1 GAMMA FOR TOTAL FIELD, 2.5 TO 10 GAMMA FOR VERTICAL FIELD, AND ± 10 GAMMA FOR HORIZONTAL FIELD.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	SEISMIC-WAVE (USUALLY P-WAVE) VELOCITIES AND DEPTHS TO REFRACTING STRATA.	SEISMIC-WAVE VELOCITIES AND DEPTHS TO REFLECTING SURFACES.	APPARENT RESISTIVITY.	NONE DIRECTLY, BUT MAGNETIC EFFECT HAS SHAPE IN PLAN INDICATIVE OF DEPTH TO CAUSING BOOY.
7. COVERAGE	LINEAR AT ANY DESIRED HORIZONTAL SPACING	LINEAR AT ANY DESIRED HORIZONTAL SPACING.	LINEAR, BOTH LATERALLY AND VERTICALLY.	POINT.
8. EFFECTIVE DEPTH	UP TO 500 ± FEET. GREATER DEPTH REQUIRES EXCESSIVE HORIZONTAL SPREAD.	GREATER THAN 500 ± FEET.	UP TO 3000+ FEET BUT USUALLY LESS THAN 100 FEET. DEPENDS UPON ELECTRODE SPACING AND POWER INPUT.	NOT SELECTIVE, BUT FIELD STRENGTH DECREASES AS SQUARE OF THE DISTANCE.
9. LIMITATIONS	VERTICAL VELOCITY CALIBRATION REQUIRED FOR DEPTH DETERMINATIONS. SEISMIC-WAVE VELOCITIES OF SUCCESSIVE STRATA MUST INCREASE WITH DEPTH. SOME BOREHOLES REQUIRED FOR CORRELATION.	VERTICAL VELOCITY CALIBRATION REQUIRED FOR DEPTH DETERMINATION. SOME BOREHOLES REQUIRED FOR CORRELATION.	A HIGH RESISTIVITY CONTRAST BETWEEN THE MATERIALS BEING LOCATED IS REQUIRED. STRATEGICALLY LOCATED BOREHOLES REQUIRED FOR REFERENCE.	DOES NOT PROVIDE DIRECT EVIOENCE OF ROCK GEOMETRY. DIFFERENCE IN MAGNETIC INTENSITIES FOR THE MATERIALS BEING LOCATED IS REQUIRED. CAN NOT BE USED NEAR EXTRANEOUS MAGNETIC MATERIALS.
10. SENSITIVITY	POOR RESULTS FOR STEEPLY DIPPING STRATA. THIN STRATA LAYERS MAY BE MISSED.	THIN STRATA LAYERS MAY BE MISSED.	RESULTS OFTEN AMBIGUOUS. IS A FUNCTION OF THE CURRENT SOURCE POWER CAPABILITY, SENSITIVITY OF THE MEASURING CIRCUIT, AND THE MOISTURE CONTENT AND/OR THE SALINITY OR FREE ION CONTENT OF THE CONATE MOISTURE.	IS A FUNCTION OF A MATERIALS MAGNETIC SUSCEPTIBILITY AND THE VARIATION WITH TIME OF THE EARTH'S NATURAL MAGNETIC FIELD. GROUND SURVEY REAOINGS ARE 25 TO 50 TIMES MORE ACCURATE THAN AIRBORNE READINGS.
11. POSSIBLE ERRORS IN INTERPRETATION	CAN NOT DISTINGUISH BETWEEN STRATA HAVING SIMILAR SEISMIC-WAVE VELOCITIES. LATERAL VARIATIONS IN STRATA COMPOSITION.	VARIATIONS IN STRATA COMPOSITION. READINGS CAN BE OBSCURED BY EARLIER ARRIVAL OF REFRACTED WAVES FROM OVERLYING STRATA.	VARIATIONS IN STRATA COMPOSITION.	INSUFFICIENT KNOWLEDGE OF THE MAGNETIC SUSCEPTIBILITY FOR THE VARIOUS ROCK UNITS AND OF THE REMANENT MAGNETIZATION PRESENT.
12. PRINCIPAL INSTRUMENTS USED	ELECTROMAGNETIC, VARIABLE RELUCTANCE, CAPACITATIVE, OR PIEZOELECTRIC GEOPHONES: AMPLIFIERS; AND OSCILLOGRAPH OR RECORDING UNIT.	ELECTROMAGNETIC, VARIABLE RELUCTANCE, CAPACITATIVE, OR PIEZOELECTRIC GEOPHONES: AMPLIFIERS; AND OSCILLOGRAPH OR RECORDING UNIT.	ELECTRICAL RESISTIVITY METER, MILLIAMPMETER, POTENTIAL MEASURING DEVICE.	DIP NEEDLE, MAGNETIC BALANCE (SCHMIDT MAGNETOMETER), FLUXGATE MAGNETOMETER, OR PROTON MAGNETOMETER.
13. ENERGY SOURCES	EXPLOSIVES, IMPACTOR, OR VIBRATORS.	EXPLOSIVES, IMPACTORS, OR VIBRATORS.	REVERSIBLE DIRECT OR LOW-FREQUENCY ALTERNATING ELECTRICAL CURRENT USUALLY SUPPLIED BY HEAVY-DUTY, DRY-CELL BATTERIES.	NATURAL MAGNETIC FORCES.
14. CREW SIZE REQUIRED	2 FOR SINGLE CHANNEL, 2 TO 7 FOR MULTI-CHANNEL, 15 TO 20 FOR VIBROSEIS.	2 FOR SINGLE CHANNEL, 15 TO 20 FOR VIBROSEIS.	A GEOPHYSICIST PLUS 1 TO 4 ASSISTANTS FOR MOVING ELECTRODES.	2 TO 3.
15. TIME REQUIRED AND WORK ACCOMPLISHED	5 TO 20 DEPTH DETERMINATIONS PER DAY WITH DEPTHS TO 500 FEET.	3 TO 15 DEPTH DETERMINATIONS PER DAY WITH DEPTHS GREATER THAN 500 FEET.	5 TO 20 DEPTH DETERMINATIONS PER DAY WITH DEPTHS TO 500 FEET.	WITH MAGNETIC BALANCE, SEVERAL READINGS PER HOUR. WITH FLUXGATE, SURVEYED AS RAPIDLY AS ONE CAN WALK. WITH PROTON, 3 TO 10 SECONDS PER STATIONARY READING.
16. COST	$20 TO $60 PER DEPTH DETERMINATION WITH PORTABLE EQUIPMENT, $100 TO $150 PER DEPTH DETERMINATION WITH VIBROSEIS.	$50 TO $60 PER DEPTH DETERMINATION WITH PORTABLE EQUIPMENT, $100 TO $150 PER DEPTH DETERMINATION WITH VIBROSEIS.	ABOUT $500 PER PROFILE-MILE.	ABOUT $100 PER PROFILE-MILE.
17. POTENTIAL AREAS OF IMPROVEMENT	DATA PROCESSING AND INTERPRETATION.	DATA PROCESSING AND INTERPRETATION.	DATA PROCESSING AND INTERPRETATION.	DATA PROCESSING AND INTERPRETATION.

Table 12. Comparison of subsurface exploration techniques, geophysical (continued).

CONDITIONS	GEOPHYSICAL			
	ELECTROMAGNETIC	GRAVITY	RADIOMETRIC	ACOUSTICAL HOLOGRAPHY
1. SITE SELECTION STAGE	RECONNAISSANCE.	RECONNAISSANCE.	RECONNAISSANCE.	RECONNAISSANCE.
2. ENGINEERING APPLICATION	LOCATE AQUIFERS, BURIED PIPELINES, UTILITY LINES. OUTLINE INTRUSIVE DIKES. DETERMINE DEPTH TO BEDROCK.	MEASURE LATERAL CHANGES IN ROCK TYPES. DETERMINE DEPTH TO BEDROCK. LOCATE SOLUTION CHANNELS AND CAVITIES, HELP DETERMINE ALLUVIAL AQUIFER POROSITY.	LOCATE AREAS OF ABNORMALLY HIGH RADIATION. DISTINGUISH ROCKS HAVING DIFFERENT RADIOACTIVITY. CAN YIELD DATA ON SHALE CONSTITUENCY.	LOCATE DISCONTINUITIES IN GEOLOGIC STRUCTURE.
3. GEOLOGICAL ENVIRONMENT	ANY.	ANY	ANY.	ANY, BUT GENERALLY ROCK.
4. PRINCIPLE USED	DETECTION OF SECONDARY MAGNETIC FIELD OCCURRING IN PARTICULAR MATERIAL AS RESULT OF ARTIFICALLY-GENERATED, ALTERNATING-ELECTROMAGNETIC FIELD.	DIFFERENCES IN DENSITIES OF SUBSURFACE MATERIALS AS INDICATED BY LATERAL CHANGES IN THE EARTH'S GRAVITATIONAL FIELD.	DETECTION OF GAMMA-RAY RADIATION.	OPTICAL DIFFERENCES BETWEEN ACOUSTIC AND CARRIER WAVE INTENSITIES.
5. QUANTITIES MEASURED	AMPLITUDE AND PHASE ANGLE, OR PORTIONS IN- AND OUT-OF-PHASE WITH THE SOURCE CURRENT, OF SECONDARY FIELD COMPONENTS RELATIVE TO SOURCE. OR DIRECTION OF SECONDARY FIELD AT RECEIVER (DIP ANGLE PROCEDURE).	FORCE OF GRAVITY IN GRAVITY UNITS. ACCURATE TO ± 0.0000001 GALS.	INTENSITY OF RADIATION, OR RATE OF GAMMA RAY PRODUCTION, IN MILLIROENTGENS PER HOUR AT UP TO 4000+ COUNTS PER SECOND.	AMPLITUDE OF GENERATED AND RECEIVED SIGNALS, ELAPSED TIME BETWEEN TIME OF GENERATION AND RECEIPT OF ECHO.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	READINGS ARE PLOTTED AND RESULTING CURVES INTERPRETED.	NONE DIRECTLY, BUT READINGS ARE CORRECTED FOR INSTRUMENT DRIFT, LATTITUDE AND ELEVATION CHANGES FROM A DATUM, AND TOPOGRAPHIC RELIEF.	NONE DIRECTLY, BUT VARIANCE ABOVE NORMAL BACKGROUND COUNT IS EVALUATED.	DISTANCE BETWEEN SOURCE AND DISCONTINUITY.
7. COVERAGE	POINT.	SPHERICAL AROUND A POINT.	POINT.	LINEAR.
8. EFFECTIVE DEPTH	UP TO 150 FEET WITH MOVING-SOURCE AND TO 1000 FEET WITH FIXED-SOURCE EQUIPMENT.	UP TO 3000+ FEET, BUT SIGNAL INTENSITY DECREASES AS SQUARE OF THE DEPTH.	SURFICIAL.	UP TO 500+ FEET, BUT SIGNAL INTENSITY DECREASES AS SQUARE OF THE DISTANCE.
9. LIMITATIONS	HAS RESTRICTED APPLICATIONS. DEPENDS UPON THE METHOD (MOVING OR FIXED SOURCE) USED. DEPENDS UPON THE PRESENCE OF A CONDUCTIVE MATERIAL.	DOES NOT PROVIDE A DIRECT MEASUREMENT OF ROCK GEOMETRY. REQUIRES KNOWING DENSITIES OF ROCK TYPES PRESENT FOR INTERPRETATION OF READINGS.	ONLY MEASURES SURFICIAL MANIFESTATIONS. DOES NOT DISCRIMINATE BETWEEN SOURCES OF RADIATION.	METHOD IS STILL IN AN EXPERIMENTAL STAGE OF DEVELOPMENT.
10. SENSITIVITY	RESULTS OFTEN AMBIGUOUS. IS A FUNCTION OF THE CURRENT SOURCE POWER CAPABILITY, SENSITIVITY OF THE MEASURING DEVICE, FREQUENCY OF THE INPUT POWER, AND SEPARATION DISTANCE BETWEEN TRANSMITTER AND RECEIVER.	DEPENDS UPON DENSITY CONTRASTS BETWEEN GEOLOGIC UNITS IN A HORIZONTAL DIRECTION.	DEPENDS UPON SIZE AND QUALITY OF RADIATION SOURCE AND DEGREE OF RADIOACTIVITY OF THE OVERBURDEN.	RESULTS CAN BE AMBIGUOUS. RESOLUTION DEPENDS UPON DISTANCE BETWEEN ANOMALY AND INSTRUMENT PACKAGE. DETECTION DEPENDS UPON ANOMALY SIZE AND DISTANCE AWAY FROM THE INSTRUMENT PACKAGE.
11. POSSIBLE ERRORS IN INTERPRETATION	IMPROPER COMPARISON OF FIELD DATA CURVES WITH REFERENCE CURVES. INACCURATE PLOTTING OF CURVES.	POSITION AT READING POINTS NOT DETERMINED WITH SUFFICIENT ACCURACY.	LOCAL GEOLOGY IMPROPERLY EVALUATED. GROUND WATER LEACHING EFFECTS NOT CONSIDERED SUFFICIENTLY.	CAN NOT DETERMINE THE TYPE OF DISCONTINUITY.
12. PRINCIPAL INSTRUMENTS USED	INDUCTION MAGNETOMETER, AMPLIFIER, HEADPHONES, AND INCLINOMETER FOR MEASURING DIP ANGLE.	SWINGING PENDULUM (GENERALLY USED FOR LABORATORY-TYPE DETERMINATIONS), GRAVITY METER (MOST WIDELY USED FIELD INSTRUMENT), OR TORSION BALANCE.	SCINTILLOMETER OR GAMMA-RAY SPECTROMETER.	TRANSMITTING AND RECEIVING TRANSDUCERS, ELECTRONIC SIGNAL PROCESSOR, AND RECONSTRUCTION DISPLAY DEVICE.
13. ENERGY SOURCES	PASSIVE USES RADIATION FROM VLF STATIONS. ACTIVE USES PORTABLE TRANSMITTER GENERATING ALTERNATING ELECTRICAL CURRENT.	EARTH'S GRAVITATIONAL FIELD.	NATURAL RADIOACTIVITY.	VIBRATOR OR EXPLOSIVES.
14. CREW SIZE REQUIRED	ONE FOR PASSIVE POWER SOURCE. 2 TO 4 WHEN USING A TRANSMITTER.	ONE GEOPHYSICIST PLUS TWO OR THREE, 3- OR 4-MAN, SURVEY CREWS.	ONE.	ONE GEOPHYSICIST PLUS 2 TO 10 ASSISTANTS.
15. TIME REQUIRED AND WORK ACCOMPLISHED	CREW OF 2 OR 3 CAN TRAVERSE 3 TO 6 MILES PER DAY TAKING READINGS AT 75- TO 100-FOOT INTERVALS. FIXED SOURCE EQUIPMENT REQUIRES AN ADDITIONAL 0.1 TO 0.5 THE TOTAL FIELD TIME FOR SETTING UP THE SOURCE.	SWINGING PENDULUM, 30+ MINUTES PER DETERMINATION. GRAVITY METER, 50+ READINGS PER DAY. TORSION BALANCE, ONLY A FEW READINGS PER DAY.	AS RAPIDLY AS TERRAIN CAN BE TRAVERSED.	NOT ESTABLISHED STILL IN EXPERIMENTAL STAGE OF DEVELOPMENT.
16. COST	ABOUT $75 TO $150 PER LINE MILE.	$300 TO $1,000 PER PROFILE MILE FOR SURVEYING AND GRAVITY READINGS.	$15 TO $30 PER LINE-MILE FOR A GROUND SURVEY.	HIGH, BUT NOT ESTABLISHED. STILL IN EXPERIMENTAL STAGE OF DEVELOPMENT.
17. POTENTIAL AREAS OF IMPROVEMENT	DATA PROCESSING AND INTERPRETATION.	DATA PROCESSING AND INTERPRETATION.	DATA PROCESSING AND INTERPRETATION.	RESOLUTION AND RECONSTRUCTION OF SUBSURFACE FEATURE IMAGES.

Table 13. Comparison of subsurface exploration techniques, sampling.

CONDITIONS	SAMPLING			
	SURFACE	DRILLING EQUIPMENT	DISTURBED SAMPLES	UNDISTURBED SOIL SAMPLES, OPEN TUBE
1. SITE SELECTION STAGE	RECONNAISSANCE.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	IDENTIFICATION OF MATERIAL AND OBTAIN SAMPLES FOR LABORATORY TESTING.	OBTAIN ACCESS FOR SAMPLING. ALLOW USE OF BOREHOLE LOGGING TECHNIQUES. FIELD OBSERVATION OF IN-SITU CONDITIONS. OBTAIN PROFILE OF SUBSURFACE MATERIAL.	IDENTIFICATION AND CLASSIFICATION OF MATERIAL. OBTAIN SAMPLES FOR LABORATORY TESTING.	OBTAIN SAMPLES HAVING MINIMAL DISTURBANCE FOR LABORATORY TESTING.
3. GEOLOGICAL ENVIRONMENT	ANY.	SOIL FOR AUGER DRILL AND WASH BORING. SOIL AND SOFT ROCK FOR CHURN DRILL. SOIL AND ROCK FOR ROTARY DRILL.	ANY.	FRIABLE, PARTIALLY DRY SILTS AND CLAYS.
4. PRINCIPLE USED	VISUAL OBSERVATION.	VARIOUS DRILLING TECHNIQUES, CORING AND NON-CORING.	SAMPLER DRIVEN INTO GROUND AND THEN WITHDRAWN WITH SAMPLE FOR SOILS. COLLECTION OF DRILL CUTTINGS.	SAMPLER PUSHED INTO GROUND AND THEN WITHDRAWN WITH SAMPLE.
5. QUANTITIES MEASURED	COMPOSITION, TEXTURE, STRUCTURE, WEATHERING, ALTERATION, FOLIATION, POROSITY, HARDNESS, INDURATION, CEMENTATION.	DRILL BIT ROTATIONAL SPEED (IF APPLICABLE) AND ADVANCE RATE, CIRCULATING FLUID DENSITY, FLOW RATE AND GAIN OR LOSS, DEPTH DRILLED, WEIGHT OR PRESSURE APPLIED TO THE BIT, BIT WEAR.	FORCE REQUIRED TO DRIVE SAMPLER IN SOILS. SIZE OF CUTTINGS.	FORCE REQUIRED TO PUSH SAMPLER.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	PHYSICAL AND MECHANICAL PROPERTIES IF SAMPLES ARE LABORATORY TESTED.	DRILLING RATE, SPECIFIC ENERGY REQUIRED TO DRILL, COMPRESSIVE STRENGTH OF ROCK INDIRECTLY.	NONE, BUT AN INDICATION OF GROUNDS PENETRATION RESISTANCE IS OBTAINED.	NONE, BUT AN INDICATION OF GROUNDS PENETRATION RESISTANCE IS OBTAINED.
7. COVERAGE	POINT.	LINEAR WITH DEPTH.	LINEAR ALONG THE SAMPLER AXIS.	LINEAR ALONG THE SAMPLER AXIS.
8. EFFECTIVE DEPTH	SURFICIAL.	UP TO 30,000+ FEET FOR ROTARY, TO BEDROCK FOR CHURN, TO 200 FEET FOR AUGER, AND TO 100 FEET FOR WASH BORING.	TO BEDROCK FOR SOILS. UP TO 30,000+ FEET FOR ROCK.	SURFICIAL.
9. LIMITATIONS	GIVES PRELIMINARY AND APPROXIMATE INFORMATION ONLY. ROCK SAMPLE AVAILABILITY RESTRICTED TO OUTCROPS AND SURFACE FLOAT. GEOCHEMICAL SAMPLING RESTRICTED TO AREAS COVERED BY RESIDUAL SOIL.	ACCESSIBILITY TO DRILLING SITE. EACH METHOD NOT APPLICABLE TO ALL CONDITIONS.	SOIL CAN NOT CONTAIN APPRECIABLE AMOUNTS OF LARGE-SIZE GRAVEL AND NEEDS SUFFICIENT COHESION TO REMAIN IN THE SAMPLER DURING WITHDRAWAL.	UNSUITABLE FOR SOILS TOO HARD TO PERMIT SMOOTH PENETRATION AND SOILS CONTAINING GRAVEL THAT WILL DAMAGE THE TUBE OR SAMPLE.
10. SENSITIVITY	DEPENDS UPON QUALITY OF SAMPLES OBTAINED.	DEPENDS UPON OPERATORS SKILL AND EXPERIENCE.	DEPENDS ON DEPTH FROM WHICH SAMPLE IS OBTAINED AND STABILITY OF BOREHOLE WALL.	DEPENDS ON CORE TAKEN TO AVOID SAMPLE DISTURBANCE
11. POSSIBLE ERRORS IN INTERPRETATION	SAMPLES MAY NOT BE INDICATIVE OF THE MATERIAL IN SITU AND/OR AT DEPTH.	CAN DETECT ONLY MAJOR CHANGES IN STRATIGRAPHY AND CONDITIONS.	SAMPLE CONTAMINATED BY MATERIAL FROM ANOTHER STRATUM.	HOLE NOT PROPERLY CLEANED BEFORE TAKING SAMPLE
12. PRINCIPAL INSTRUMENTS USED	HAND LENS AND MICROSCOPE, PICKS, HAMMERS, SHOVELS.	ROTARY, CHURN (OR PERCUSSION), OR AUGER DRILLS; WASH BORING.	AUGER DRILLS AND OPEN-DRIVE, DISPLACEMENT, OR CABLE-TOOL SAMPLERS FOR SOILS.	SHELBY TUBE.
13. ENERGY SOURCES	NOT APPLICABLE.	GASOLINE OR DIESEL ENGINE, COMPRESSED AIR, HYDRAULIC, ELECTRICAL POWER, PUMPS.	DRILLING RIG DRIVE MECHANISM, HYDRAULIC OR MECHANICAL JACKS, HAMMERS.	MOST SATISFACTORY METHOD FOR PUSHING SAMPLER IS WITH DRIVE MECHANISM PREFERABLY HYDRAULIC) ON DRILL RIG.
14. CREW SIZE REQUIRED	ONE.	COMMONLY 2 OR 3.	COMMONLY 2.	COMMONLY 2.
15. TIME REQUIRED AND WORK ACCOMPLISHED	AS RAPIDLY AS TERRAIN CAN BE TRAVERSED, SAMPLES COLLECTED, AND OBSERVATIONS MADE AND RECORDED.	100 TO 600 FEET PER DAY IN SOIL, 20 TO 300 FEET PER DAY IN ROCK.	DEPENDS ON DEPTH OF SAMPLING AND/OR DRILLING RATE.	10 TO 30 MINUTES PER SAMPLE.
16. COST	MINIMAL.	\$3 TO \$15 PER FOOT.	UP TO \$15 PER SAMPLE.	ABOUT \$15 PER SAMPLE.
17. POTENTIAL AREAS OF IMPROVEMENT	OBTAIN BETTER REPRESENTATIVE SAMPLES.	INCREASED PENETRATION RATES.	REDUCE SAMPLE CONTAMINATION.	OBTAIN BETTER REPRESENTATIVE SAMPLES.

Table 13. Comparison of subsurface exploration techniques, sampling (continued).

CONDITIONS	UNDISTURBED SAMPLING		
	SOIL SAMPLES, PISTON TUBE.	SOIL SAMPLES, ROTARY CORE	ROCK SAMPLES, CORING
1. SITE SELECTION	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	OBTAIN SAMPLES HAVING MINIMAL DISTURBANCE FOR LABORATORY TESTING.	OBTAIN SAMPLES HAVING MINIMAL DISTURBANCE FOR LABORATORY TESTING.	OBTAIN SAMPLES FOR LABORATORY TESTING. OBSERVATION OF IN-SITU ROCK CONDITIONS.
3. GEOLOGICAL ENVIRONMENT	SOILS.	HARD COHESIVE SOILS.	ROCK.
4. PRINCIPLE USED	SAMPLER PUSHED INTO GROUND AND THEN WITHDRAWN WITH SAMPLE	ANNULAR-TYPE HOLE DRILLED WITH RECOVERY OF RESULTING INNER CORE.	ANNULAR-TYPE HOLE DRILLED WITH RECOVERY OF RESULTING INNER CORE.
5. QUANTITIES MEASURED	FORCE REQUIRED TO PUSH SAMPLER.	DRILL BIT ROTATIONAL SPEED AND ADVANCE RATE; CIRCULATING FLUID DENSITY, FLOW RATE, AND LOSS OR GAIN; PERCENTAGE AND CHARACTERISTICS OF CORE RECOVERED; DEPTH DRILLED.	DRILL BIT ROTATIONAL SPEED, PRESSURE APPLIED, WEAR, AND ADVANCE RATE; CIRCULATING FLUID DENSITY, FLOW RATE, AND LOSS OR GAIN; PERCENTAGE AND CHARACTERISTICS OF CORE RECOVERED; DEPTH DRILLED.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	NONE, BUT AN INDICATION OF GROUNDS PENETRATION RESISTANCE IS OBTAINED.	EMPIRICAL DESIGNATIONS FOR GROUND CLASSIFICATION FROM CORE RECOVERY, CONDITION, AND DEPTH.	EMPIRICAL DESIGNATIONS FOR GROUND CLASSIFICATION FROM CORE RECOVERY, CONDITION, AND DEPTH.
7. COVERAGE	LINEAR ALONG THE SAMPLER AXIS.	LINEAR ALONG THE BOREHOLE AXIS.	LINEAR ALONG THE BOREHOLE AXIS.
8. EFFECTIVE DEPTH	TO BEDROCK.	TO BEDROCK.	UP TO 30,000+ FEET.
9. LIMITATIONS	SOIL CAN NOT CONTAIN APPRECIABLE AMOUNTS OF GRAVEL AND NEEDS SUFFICIENT COHESION TO REMAIN IN SAMPLER DURING WITHDRAWAL.	GROUND MUST BE COMPETENT WITHOUT VOIDS OR EXCESSIVE FRACTURING.	BADLY FRACTURED GROUND REQUIRES GROUTING, CASING, OR COSTLY DRILLING FLUID ADDITIVES.
10. SENSITIVITY	REQUIRES A SMOOTH CONTINUOUS PUSH AT A UNIFORM FAST RATE.	DEPENDS UPON THE ROTATIONAL SPEED AND ADVANCE RATE OF THE DRILL BIT, THE CIRCULATING FLUID TYPE AND FLOW RATE USED, AND THE AMOUNT OF CORE RECOVERY.	DEPENDS UPON KNOWLEDGE OF HOLE DIRECTION AND AMOUNT OF CORE RECOVERY.
11. POSSIBLE ERRORS IN INTERPRETATION	HOLE NOT PROPERLY CLEANED BEFORE TAKING SAMPLE.	MAY REACH WRONG CONCLUSIONS FOR REASONS OF LOW CORE RECOVERY.	MAY REACH WRONG CONCLUSIONS FOR REASONS OF LOW CORE RECOVERY. IMPROPER ORIENTATION OF CORE.
12. PRINCIPAL INSTRUMENTS USED	FIXED PISTON (SUCH AS HVORSLEV-TYPE, HONG, HYDRAULIC PISTON, OR SWEDISH FOIL SAMPLER), RETRACTABLE PISTON, FREE PISTON.	SINGLE- AND DOUBLE-TUBE CORE BARRELS.	SINGLE- AND DOUBLE-TUBE CORE BARRELS.
13. ENERGY SOURCES	MOST SATISFACTORY METHOD FOR PUSHING SAMPLER IS WITH DRIVE MECHANISM-PREFERABLY HYDRAULIC, ON DRILL RIG.	DRILLING RIG.	DRILLING RIG.
14. CREW SIZE REQUIRED	COMMONLY 2.	2 TO 3.	2 TO 3.
15. TIME REQUIRED AND WORK ACCOMPLISHED	10 TO 30 MINUTES PER SAMPLE.	50 TO 200 FEET PER 8-HOUR SHIFT.	20 TO 100 FEET PER 8-HOUR SHIFT.
16. COST	ABOUT $15 PER SAMPLE.	$4 TO $5 PER FOOT DIRECT DRILLING COST	$5 TO $15 PER FOOT DIRECT DRILLING COST.
17. POTENTIAL AREAS OF IMPROVEMENT	OBTAIN BETTER REPRESENTATIVE SAMPLES.	INCREASE PENETRATION RATES AND CORE RECOVERY.	INCREASE PENETRATION RATES AND CORE RECOVERY

Table 14. Comparison of subsurface exploration techniques, borehole logging.

CONDITIONS	BOREHOLE LOGGING.			
	VISUAL OR PHOTOGRAPHIC	SPONTANEOUS POTENTIAL	CONVENTIONAL RESISTIVITY	MICRORESISTIVITY
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	VISUAL EXAMINATION OF BOREHOLE WALLS, DETERMINE ORIENTATION OF GEOLOGIC DISCONTINUITIES.	DETECT AND LOCATE BOUNDARIES OF PER-PERMEABLE BEDS. CORRELATE STRATI-GRAPHY BETWEEN BOREHOLES.	DETERMINE DEPTH TO BEDROCK, THICKNESS AND TYPE OF SEDIMENTARY LAYERS, PRESENCE AND SALINITY OF GROUND WATER, ROCK PER-MEABILITY. CORRELATE STRATIGRAPHY BETWEEN BOREHOLES.	DELINEATE PERMEABLE FORMATIONS.
3. GEOLOGICAL ENVIRONMENT	ANY.	ANY, BUT PRIMARILY SEDIMENTARY.	ANY, BUT PRIMARILY SEDIMENTARY.	ANY, BUT PRIMARILY SEDIMENTARY.
4. PRINCIPLE USED	VISUAL OBSERVATION, TELEVISION AND STILL PHOTOGRAPHY.	POTENTIAL DIFFERENCE BETWEEN TWO ELECTRODES.	DIFFERENCE IN ELECTRICAL RESISTIVITY OR CONDUCTIVITY OF STRATA.	DIFFERENCE IN ELECTRICAL RESISTIVITY BETWEEN MUD CAKE AND FORMATION.
5. QUANTITIES MEASURED	ATTITUDES OF DISCONTINUITIES AND BEDDING PLANES.	DIFFERENCE IN POTENTIAL BETWEEN MOVING ELECTRODE IN BOREHOLE AND FIXED ELEC-TRODE ON GROUND SURFACE. DEPTH TO MOVING ELECTRODE.	CURRENT INPUT AND POTENTIAL DIFFERENCE BETWEEN TWO ELECTRODES AT FIXED SPACING WITHIN BOREHOLE. DEPTH TO READING POINT.	RESISTIVITY OF MUD CAKE AND FORMATION INVADED BY THE DRILLING MUD. DEPTH TO POINT OF READING.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	NOT APPLICABLE.	PLOT OF THE SP CURVE WHICH GIVES LOCATION AND THICKNESS OF PERMEABLE BEDS.	APPARENT RESISTIVITY IN OHM-METERS. PLOT OF DEPTH VERSUS RESISTIVITY READ-INGS GIVES LOCATION AND THICKNESS OF DIFFERENT STRATA.	PLOT OF DEPTH VERSUS RESISTIVITY READINGS.
7. COVERAGE	CIRCULAR ABOUT A POINT AND LINEAR ALONG THE BOREHOLE AXIS.	ROUGHLY SPHERICAL.	ROUGHLY SPHERICAL.	ROUGHLY SPHERICAL.
8. EFFECTIVE DEPTH	PERISCOPE TO 100+ FEET, TELEVISION TO 4000 FEET, FILM-TYPE CAMERA TO 8000 FEET.	NOT APPLICABLE	DEPENDS ON ELECTRODE SPACING, BUT GEN-ERALLY LESS THAN 20 FEET OUT FROM BORE-HOLE.	VERY SHALLOW OUT FROM BOREHOLE WALL, LESS THAN 6 INCHES.
9. LIMITATIONS	HUMIDITY MAY FOG THE LENS, MUDDY WATER CAN RESTRICT VIEWING.	BOREHOLE MUST CONTAIN A CONDUCTIVE FLUID. IS NOT SATISFACTORY FOR USE IN UNCASED HOLES.	BOREHOLE MUST CONTAIN A CONDUCTIVE FLUID. RESISTIVITY CONTRAST BETWEEN FORMATIONS REQUIRED. RESULTS OFTEN AMBIGUOUS. ONLY INDUCTION RESISTIV-ITY CAN BE RUN IN HIGHLY SALINE HOLE FLUIDS.	BOREHOLE MUST CONTAIN A CONDUCTIVE DRILLING MUD. HAS A SMALL, LIMITED DEPTH OF PENETRATION. POROSITIES MUST BE BETWEEN 12 AND 17%.
10. SENSITIVITY	DEPENDS UPON CLARITY OF BOREHOLE AT-MOSPHERE AND CLEANLINESS OF BOREHOLE WALLS.	DEPENDS UPON INTENSITY OF ELECTRICAL CURRENT USED, THICKNESS OF PERMEABLE BEDS, DIAMETER OF BOREHOLE, AND IN-DIVIDUAL RESISTIVITIES OF THE VARIOUS FORMATIONS, MUD FILTRATE AND BOREHOLE FLUID.	IS FUNCTION OF POWER SOURCE CAPABILITY, SENSITIVITY OF MEASURING CIRCUIT, ELECTRODE SPACING, AND MUD CAKE THICK-NESS.	IS FUNCTION OF POWER SOURCE CAPABILITY, SENSITIVITY OF MEASURING CIRCUIT, CHAR-ACTER AND THICKNESS OF MUD CAKE, AND CHARACTER OF BOREHOLE WALL SURFACE.
11. POSSIBLE ERRORS IN INTERPRETATION	ORIENTATION OF CAMERA NOT KNOWN COR-RECTLY.	DIFFERENCES IN RESISTIVITY AND THICK-NESS OF SUCCESSIVE BEDS AFFECT INTER-PRETATION. ANOTHER LOGGING TECHNIQUE USUALLY NECESSARY TO AID IN INTERPRE-TATION.	TYPICAL CURVE SHAPES MUST BE WELL KNOWN. EXTENT OF DRILLING MUD INVASION AND BED THICKNESS AFFECT INTERPRETATION.	EXTENT OF DRILLING MUD INVASION AND BED THICKNESS AFFECT INTERPRETATION.
12. PRINCIPAL INSTRUMENTS USED	PERISCOPE, TELEVISION, FILM-TYPE CAMERA.	RECORDING GALVANOMETER.	NORMAL, LATERAL, AND FOCUSED LATERAL DEVICES.	MICROLOG
13. ENERGY SOURCES	ELECTRICAL CURRENT.	ELECTRICAL CURRENT	ELECTRICAL CURRENT.	ELECTRICAL CURRENT.
14. CREW SIZE REQUIRED	ONE OPERATING TECHNICIAN.	1 OR 2.	1 OR 2.	1 OR 2.
15. TIME REQUIRED AND WORK ACCOMPLISHED	DEPENDS ON DEPTH AND METHOD USED.	ABOUT 15 MINUTES FOR A 500-FOOT HOLE, PLUS SET-UP.	ABOUT 15 MINUTES FOR A 500-FOOT HOLE, PLUS SET-UP.	ABOUT 30 MINUTES FOR A 500-FOOT HOLE PLUS SET-UP.
16. COST	$150 TO $175 PER DAY FOR CAMERA AND $175 TO $200 PER DAY FOR TECHNICIAN	$600 TO $700 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.	$600 TO $700 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.	$600 TO $700 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.
17. POTENTIAL AREAS OF IMPROVEMENT	GREATER AVAILABILITY AND BETTER CONTROL.	PORTABILITY FOR RUGGED TERRAIN.	PORTABILITY FOR RUGGED TERRAIN AND ADAPT FOR HORIZONTAL HOLES.	ADAPT FOR HORIZONTAL HOLES AND MINIATURIZATION.

Table 14. Comparison of subsurface exploration techniques, borehole logging (continued).

CONDITIONS	BOREHOLE LOGGING			
	FOCUSING ELECTRODE	INDUCTION	GAMMA RAY	NEUTRON
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	DETECT AND LOCATE BOUNDARIES OF FORMATIONS HAVING MODERATE-TO-SMALL BED THICKNESSES.	DETERMINE SEQUENCE AND THICKNESS OF VARIOUS STRATA. LOCATE FORMATION BOUNDARIES.	DEFINE SHALE BEDS. DETECT AND EVALUATE RADIOACTIVE MINERALS. CORRELATE STRATIGRAPHY BETWEEN BOREHOLES.	DELINEATE POROUS FORMATIONS AND DETERMINE THEIR POROSITY. DETECT GAS. LITHOLOGY INTERPRETATION.
3. GEOLOGICAL ENVIRONMENT	ANY, BUT PRIMARILY SEDIMENTARY.	ANY, BUT PRIMARILY SEDIMENTARY.	ANY.	ANY, BUT PRIMARILY SEDIMENTARY.
4. PRINCIPLE USED	DIFFERENCE IN ELECTRICAL RESISTIVITY OF STRATA.	DIFFERENCE IN ELECTRICAL CONDUCTIVITY OF STRATA AS DETERMINED FROM CURRENT FLOW INDUCED BY A GENERATED, ALTERNATING MAGNETIC FIELD.	MEASUREMENT OF NATURAL RADIOACTIVITY.	COUNTING OF EITHER NEUTRONS AT A FIXED DISTANCE FROM THEIR SOURCE OR THE GAMMA RAYS EMITTED BY ATOMS CAPTURING THE RELEASED NEUTRONS.
5. QUANTITIES MEASURED	CURRENT INPUT AND POTENTIAL DIFFERENCE BETWEEN MOVING ELECTRODE IN BOREHOLE AND FIXED ELECTRODE ON GROUND SURFACE. DEPTH TO MOVING ELECTRODE.	CURRENT INPUT, INDUCED VOLTAGE.	COUNT OF GAMMA RAYS REACHING DETECTORS. LOGGING SPEED OF INSTRUMENT. DEPTH TO POINT OF READING.	COUNT OF NEUTRONS AND/OR GAMMA RAYS REACHING DETECTORS. LOGGING SPEED OF INSTRUMENT. DEPTH TO POINT OF READING.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	PLOT OF DEPTH VERSUS RESISTIVITY READINGS.	PLOT OF DEPTH VERSUS CONDUCTIVITY.	PLOT OF DEPTH VERSUS GAMMA RAY COUNT.	PLOT OF NEUTRON AND/OR GAMMA RAY COUNT AND POROSITY VERSUS DEPTH.
7. COVERAGE	ROUGHLY SPHERICAL.	ROUGHLY SPHERICAL.	CIRCULAR ABOUT A POINT.	DISTANCE BETWEEN SOURCE AND DETECTOR ALONG BOREHOLE AXIS.
8. EFFECTIVE DEPTH	SHALLOW OUT FROM BOREHOLE WALL, ONE INCH TO 3 FEET.	LESS THAN 11 FEET OUT FROM BOREHOLE WALL.	SHALLOW OUT FROM BOREHOLE WALL, UP TO ABOUT ONE FOOT.	SHALLOW OUT FROM BOREHOLE WALL, UP TO ABOUT ONE FOOT.
9. LIMITATIONS	BOREHOLE MUST CONTAIN A CONDUCTIVE FLUID. RESISTIVITY CONTRAST BETWEEN FORMATIONS REQUIRED.	RESISTIVITY OF FORMATIONS SHOULD BE LESS THAN 100 OHM-METERS AND THE BOREHOLE CONDUCTIVITY SHOULD BE LOW. CAN BE USED IN AIR.	DOES NOT DISCRIMINATE BETWEEN RADIATION SOURCES.	CANNOT DIFFERENTIATE BETWEEN A GAS AND LOW POROSITY.
10. SENSITIVITY	IS FUNCTION OF MUD CAKE AND INVADED ZONE THICKNESSES, AND BOREHOLE DIAMETER.	DEPENDS UPON RELATIVE CONDUCTIVITY OF THE VARIOUS FORMATIONS.	DEPENDS UPON DISTANCE TO RADIATION SOURCE.	INSTRUMENT RESPONSE REFLECTS MOSTLY AMOUNT OF HYDROGEN IN THE FORMATION AND THUS INDICATES PRIMARILY THE LIQUID-FILLED POROSITY.
11. POSSIBLE ERRORS IN INTERPRETATION	EXTENT OF DRILLING MUD INVASION AFFECTS INTERPRETATION.	TOOL NOT PROPERLY CALIBRATED.	PRESENCE OF ELEMENTS HAVING HIGH ATOMIC NUMBERS WILL CHANGE THE ABSORPTIVE CHARACTERISTICS OF THE FORMATIONS.	PRESENCE OF HYDROCARBONS AND/OR CLAY IN THE FORMATIONS CAN AFFECT INTERPRETATION.
12. PRINCIPAL INSTRUMENTS USED	LATEROLOG, MICROLATEROLOG, PROXIMITY, AND SPHERICALLY FOCUSED LOGS.	COAXIAL TRANSMITTER AND RECEIVER COILS WITH A GALVANOMETER.	SCINTILLATION COUNTER.	GNT, SNP (SIDEWALL NEUTRON POROSITY) OR CNL (COMPENSATED NEUTRON LOG) TOOLS.
13. ENERGY SOURCES	ELECTRICAL CURRENT.	HIGH-FREQUENCY ALTERNATING CURRENT.	NATURAL RADIOACTIVITY.	PLUTONIUM-BERYLLIUM, AMERICIUM-BERYLLIUM, OR RADIUM-BERYLLIUM FOR THE RADIOACTIVE SOURCE.
14. CREW SIZE REQUIRED	1 OR 2	1 OR 2	1 OR 2.	1 OR 2.
15. TIME REQUIRED AND WORK ACCOMPLISHED	ABOUT 30 MINUTES FOR A 500-FOOT HOLE, PLUS SET-UP.	ABOUT 15 MINUTES FOR A 500-FOOT HOLE PLUS SET-UP.	ABOUT 30 MINUTES FOR A 500-FOOT HOLE PLUS SET-UP.	ABOUT 30 MINUTES FOR A 500-FOOT HOLE PLUS SET-UP.
16. COST	\$600 TO \$700 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.	\$600 TO \$700 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.	\$500 TO \$700 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.	\$350 TO \$550 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.
17. POTENTIAL AREAS OF IMPROVEMENT	ADAPT FOR USE IN HORIZONTAL HOLES.	ADAPT FOR USE IN HORIZONTAL HOLES AND MINIATURIZATION.	ADAPT FOR USE IN HORIZONTAL HOLES.	PORTABILITY FOR RUGGED TERRAIN.

Table 14. Comparison of subsurface exploration techniques, borehole logging (continued).

CONDITIONS	BOREHOLE LOGGING			
	FORMATION DENSITY	SONIC	GRAVITATIONAL	CALIPER
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	DETERMINE POROSITY. DETECT GAS. EVALUATE COMPLEX LITHOLOGIES AND SHALEY SANDS. IDENTIFY MINERALS IN EVAPORITE DEPOSITS.	DETERMINE POROSITY WHEN THE LITHOLOGY IS KNOWN. CORRELATE LITHOLOGY BETWEEN BOREHOLES. AID TO INTERPRETING SEISMIC RECORDS AND DETERMINING ELASTIC CONSTANTS.	DETERMINE POROSITY.	DETERMINE GEOMETRY AND VOLUME OF BOREHOLE. INDICATE GEOLOGIC STRUCTURE. CORRECTION OF RAW DATA FROM OTHER LOGGING METHODS.
3. GEOLOGICAL ENVIRONMENT	ANY, BUT PRIMARILY SEDIMENTARY.	ANY.	ANY.	ANY.
4. PRINCIPLE USED	COUNTING OF GAMMA RAYS AT A FIXED DISTANCE FROM A SOURCE.	VELOCITY OF A SOUND WAVE THROUGH A FORMATION.	MEASUREMENT OF EARTH'S GRAVITATIONAL FIELD.	PHYSICAL MEASUREMENT OF BOREHOLE DIAMETERS.
5. QUANTITIES MEASURED	COUNT OF GAMMA RAYS REACHING DETECTORS. LOGGING SPEED OF INSTRUMENT. DEPTH TO POINT OF READING.	TRAVEL TIME FOR A COMPRESSIONAL SOUND WAVE TO TRAVERSE ONE FOOT OF A FORMATION AND TRAVERSING TIME OF INSTRUMENT THROUGH BOREHOLE.	CHANGES IN GRAVITATIONAL ATTRACTION, DEPTH TO POINT OF READING.	DIAMETER OF BOREHOLE IN SEVERAL DIRECTIONS, DEPTH TO POINT OF READING.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	PLOT OF GAMMA RAY COUNT AND APPARENT BULK DENSITY VERSUS DEPTH.	PLOT OF SOUND WAVE AND INSTRUMENT TRAVEL TIMES VERSUS DEPTH. CALCULATION OF FORMATION POROSITY AND ELASTIC CONSTANTS FOR GROUND	AVERAGE BULK DENSITY OF THE GROUND.	VOLUME AND SHAPE OF BOREHOLE.
7. COVERAGE	DISTANCE BETWEEN SOURCE AND DETECTOR ALONG BOREHOLE AXIS.	LINEAR DISTANCE BETWEEN SONIC RECEIVERS ALONG BOREHOLE AXIS.	CIRCULAR ABOUT A POINT.	CIRCULAR ABOUT A POINT.
8. EFFECTIVE DEPTH	AT BOREHOLE WALL	AT BOREHOLE WALL.	UP TO 500 ± FEET OUT FROM THE BOREHOLE WALL.	TO BOREHOLE WALL.
9. LIMITATIONS	INSTRUMENT MUST BE CALIBRATED. CALIPER LOG ALSO REQUIRED.	CALIPER AND DENSITY LOGS ALSO REQUIRED	REQUIRES A DIFFERENCE IN ELEVATION BETWEEN READING POINTS.	CAN NOT DETECT LOCAL PROTRUSIONS AND CAVITIES NOR AMOUNT OF HOLE DEVIATION.
10. SENSITIVITY	DEPENDS UPON THE CONTACT BETWEEN INSTRUMENT AND BOREHOLE WALL.	DEPENDS UPON RELATIVE SONIC CHARACTERISTICS OF FORMATIONS. THE PRESENCE OF SHALE, GAS, AND/OR FRACTURES MAY OBSCURE READINGS.	DEPENDS UPON DIFFERENCE IN GROUND MASS BETWEEN TWO DEPTHS.	DEPENDS UPON LOGGING SPEED AND TRUE ROUNDNESS OF BOREHOLE.
11. POSSIBLE ERRORS IN INTERPRETATION	PRESENCE OF HYDROCARBONS AND SHALE OR CLAY IN THE FORMATIONS CAN AFFECT INTERPRETATION.	LITHOLOGY NOT KNOWN IN SUFFICIENT DETAIL.	PRESENCE OF HIGH DENSITY OBJECT CAN AFFECT READINGS.	WALL ENLARGEMENTS MAY OCCUR BETWEEN CONTACT POINTS OF ARMS WITH BOREHOLE WALL AND THUS GO UNDETECTED.
12. PRINCIPAL INSTRUMENTS USED	FORMATION DENSITY COMPENSATED TOOL HAVING A GAMMA-RAY SOURCE AND TWO DETECTORS (ONE FOR SHORT SPACING AND ONE FOR LONG SPACING).	COMPENSATED VELOCITY TYPE HAVING TWO MAGNETOSTRICTION-TYPE TRANSMITTING AND TWO PIEZOELECTRIC RECEIVING TRANSDUCERS. 3-D VELOCITY TYPE HAVING ONE TRANSMITTING AND ONE RECEIVING TRANSDUCER.	BOREHOLE GRAVIMETER.	CALIPER TOOL.
13. ENERGY SOURCES	GAMMA-RAY RADIOACTIVE SOURCE.	ELECTRICAL CURRENT.	EARTH'S GRAVITATIONAL FIELD.	ELECTRICAL POWER TO OPERATE CALIPER ARMS.
14. CREW SIZE REQUIRED	1 OR 2.	1 OR 2.	1 OR 2.	1 OR 2.
15. TIME REQUIRED AND WORK ACCOMPLISHED	ABOUT 30 MINUTES FOR A 500-FOOT HOLE PLUS SET-UP.	ABOUT 30 MINUTES FOR A 500-FOOT HOLE PLUS SET-UP.	ABOUT 30 MINUTES FOR A 500-FOOT HOLE PLUS SET-UP.	ABOUT 30 MINUTES FOR A 500-FOOT HOLE PLUS SET-UP.
16. COST	$450 TO $550 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.	$400 to $500 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.	$400 TO $600 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.	$400 TO $600 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.
17. POTENTIAL AREAS OF IMPROVEMENT	ADAPT FOR USE IN HORIZONTAL HOLES AND MINIATURIZATION.	ADAPT FOR USE IN HORIZONTAL HOLES AND MINIATURIZATION	ADAPT FOR USE IN HORIZONTAL HOLES.	ADAPT FOR USE IN HORIZONTAL HOLES.

Table 14. Comparison of subsurface exploration techniques, borehole logging (continued).

CONDITIONS	BOREHOLE LOGGING
	TEMPERATURE
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	DETERMINE GROUND TEMPERATURES AND GEOTHERMAL GRADIENT.
3. GEOLOGICAL ENVIRONMENT	ANY.
4. PRINCIPLE USED	CHANGE IN RESISTANCE OF AN ELECTRICAL CIRCUIT DUE TO A TEMPERATURE CHANGE.
5. QUANTITIES MEASURED	FLUID TEMPERATURE WITHIN BOREHOLE. DEPTH TO POINT OF READING.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	GROUND TEMPERATURE AT PARTICULAR DEPTHS AND GEOTHERMAL GRADIENT.
7. COVERAGE	POINT.
8. EFFECTIVE DEPTH	WITHIN BOREHOLE.
9. LIMITATIONS	BOREHOLE MUST CONTAIN A LIQUID.
10. SENSITIVITY	DEPENDS UPON THE ELAPSED TIME BETWEEN STOPPING LIQUID CIRCULATION AND TAKING READING, AND HEAT CONDUCTIVITY OF THE LIQUID.
11. POSSIBLE ERRORS IN INTERPRETATION	AMOUNT OF HEAT FLOW BETWEEN THE GROUND AND BOREHOLE LIQUID NOT EVALUATED CORRECTLY.
12. PRINCIPAL INSTRUMENTS USED	THERMISTOR.
13. ENERGY SOURCES	ELECTRICAL CURRENT.
14. CREW SIZE REQUIRED	1 OR 2.
15. TIME REQUIRED AND WORK ACCOMPLISHED	ABOUT 45 MINUTES FOR A 500-FOOT HOLE PLUS SET-UP.
16. COST	$300 TO $400 PER 500-FOOT HOLE DIRECT COST WHEN RUN WITH ANOTHER LOG.
17. POTENTIAL AREAS OF IMPROVEMENT	ADAPT FOR USE IN HORIZONTAL HOLES.

Table 15. Comparison of subsurface exploration techniques, trenches and pilot exploration tunnels and in situ testing.

CONDITIONS	TRENCHES AND PILOT EXPLORATION TUNNELS	IN SITU TESTING		
		SOIL SHEARING STRENGTH	STATE OF STRESS	GROUND WATER HYDROLOGY
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATIONS.	DETAILED INVESTIGATIONS.
2. ENGINEERING APPLICATION	OBTAIN SAMPLES FOR LABORATORY TESTING. OBTAIN FIRST-HAND INFORMATION ON POSSIBLE EXCAVATION DIFFICULTIES AND SUPPORT REQUIREMENTS. DETAILED EXAMINATION OF GROUND STRUCTURAL FRACTURES AND CONDITIONS.	DETERMINE THE IN SITU COHESIVE SHEARING STRENGTH OF SOIL.	DETERMINE MAGNITUDE AND DIRECTIONS OF THE PRINCIPAL GROUND STRESSES.	DETERMINE ABILITY OF A FORMATION TO CONDUCT FREE WATER, ESTIMATE QUANTITY OF WATER THAT MAY FLOW INTO TUNNEL.
3. GEOLOGICAL ENVIRONMENT	ANY.	SOIL.	ANY, BUT PRIMARILY ROCK.	ANY.
4. PRINCIPLE USED	NOT APPLICABLE.	MEASUREMENT OF TORSIONAL FORCE REQUIRED TO SHEAR A CYLINDRICAL SOIL SURFACE.	OVERCORING OF STRAIN GAGE LOCATION.	MEASUREMENT OF WATER FLOW AMOUNT THROUGH A SECTION OF GROUND.
5. QUANTITIES MEASURED	MAGNITUDE AND DIRECTIONS OF PRINCIPLE GROUND STRESSES, DISCONTINUITY ORIENTATION AND FREQUENCY, WATER INFLOW, GROUND TEMPERATURE.	APPLIED TORQUE, ANGULAR ROTATION, VANE GEOMETRY, AND FRICTIONAL RESISTANCE OF TESTING DEVICE COMPONENTS.	CHANGE IN STRAIN.	VOLUME OF WATER PUMPED, WATER LEVELS IN OBSERVATION WELLS, DISTANCES BETWEEN WELLS, AND AQUIFER THICKNESS (IF IT IS A CONFINED AQUIFER) OR CHANGE IN WATER LEVEL AND ELAPSED TEST TIME.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	REQUIREMENTS FOR SUPPORT, PUMPING, AND LINING IN MAIN TUNNEL.	INDICATED SHEAR STRENGTH.	STRESS RELIEF THAT CAUSED THE STRAIN CHANGE OBSERVED.	OVERALL COEFFICIENT OF PERMEABILITY FOR THE GROUND IN TEST AREA.
7. COVERAGE	LINEAR.	POINT.	POINT.	SPHERICAL AROUND A POINT.
8. EFFECTIVE DEPTH	TRENCHES TO 20 ± FEET. TUNNELS AT ANY DEPTH.	TO BEDROCK, DEPENDS UPON TORQUE STRENGTH OF RODS.	UP TO 200 FEET.	TO ANY PROPOSED TUNNEL DEPTH.
9. LIMITATIONS	TRENCHES LIMITED TO PORTAL AREAS. TUNNELS ARE COSTLY IN BOTH TIME AND MONEY.	NOT SUITABLE FOR STIFF CLAYS WHERE STRENGTH IS CONTROLLED BY FRACTURES OR SLICKINSIDES.	ONLY INDICATES STRESS RELIEF OCCURRING AFTER STRAIN GAGE PLACEMENT.	ONLY CONSIDERS STEADY-STATE FLOW CONDITIONS. APPLICABLE OVER A LIMITED VERTICAL GROUND INTERVAL.
10. SENSITIVITY	EXCELLENT ABILITY TO DETECT CHANGES IN GROUND CONDITIONS.	ERRATIC RESULTS OBTAINED IN SOFT SOILS CONTAINING GRAVEL, ETC.	DEPENDS ON CARE TAKEN TO OBTAIN READINGS.	GIVES BETTER ESTIMATION OF IN SITU CONDITIONS THAN DOES LABORATORY TESTS. DOES NOT INDICATE EFFECTS OF LONG PERIOD OR SEASONAL FLUCTUATIONS.
11. POSSIBLE ERRORS IN INTERPRETATION	INACCURATE EXTRAPOLATION OF DATA TO LARGER TUNNEL SIZE.	SOIL MAY HAVE BEEN REMOLDED PRIOR TO TESTING. RESULTS ARE ONLY INDICATIVE.	CAN ONLY ESTIMATE ORIGINAL STRESS EXISTING BEFORE ACCESS MADE FOR GAGE PLACEMENT.	WATER TABLE IS ASSUMED TO BE HORIZONTAL AND AQUIFER THICKNESS IS TAKEN AS RELATIVELY CONSTANT. DOES NOT DIFFERENTIATE BETWEEN INDIVIDUAL FLOW ROUTES. VERY SLIGHT MUDDING OF HOLE CAN ALTER RESULTS.
12. PRINCIPAL INSTRUMENTS USED	STRAIN GAGES, VISUAL OBSERVATIONS.	VANE SHEAR DEVICE.	STRAIN GAGES OR INCLUSION STRESS METER.	PUMP AND WATER FLOW RATE METER FOR PUMPING TESTS. WATER LEVEL DIFFERENCE INDICATOR AND TIMER FOR OPEN-END TESTS.
13. ENERGY SOURCES	NOT APPLICABLE.	TORQUE APPLIED EITHER BY HAND OR AN ELECTRIC MOTOR THROUGH A GEAR TRAIN.	NOT APPLICABLE.	ELECTRIC MOTORS, MOST GENERALLY, TO OPERATE PUMPS.
14. CREW SIZE REQUIRED	1 TO 3 FOR READINGS PLUS AN EXCAVATION CREW.	1 OR 2.	2 OR 3.	2 OR MORE.
15. TIME REQUIRED AND WORK ACCOMPLISHED	5 TO 50 FEET ADVANCE PER 24-HOUR DAY FOR 10-TO 12-FOOT TUNNEL.	5 TO 15 MINUTES PER TEST.	4+ HOURS PER TEST.	2 TO 6 HOURS PER TEST USUALLY.
16. COST	VARIABLE UPON TUNNEL SIZE AND GROUND CONDITIONS ENCOUNTERED.	ABOUT $5 PER TEST.	$100 TO $500 PER TEST.	VARIABLE UPON METHOD USED AND LENGTH OF TEST.
17. POTENTIAL AREAS OF IMPROVEMENT	FASTER ADVANCE RATES.	DEVELOP IMPROVED TECHNIQUES.	DEVELOP IMPROVED TECHNIQUES.	DEVELOP IMPROVED TECHNIQUES.

Table 16. Comparison of subsurface exploration techniques, laboratory testing.

CONDITIONS	LABORATORY TESTING OF SOIL			
	GRADATION	MOISTURE CONTENT	UNIT WEIGHT	POROSITY
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	FOR CLASSIFICATION OF SOIL MATERIALS.	FOR CLASSIFICATION. AID IN EVALUATION OF POTENTIAL WATER PROBLEMS.	USED IN ENGINEERING DESIGN.	USED IN ENGINEERING DESIGN. INDICATION OF POSSIBLE PERMEABILITY FOR SOIL MASS.
3. GEOLOGICAL ENVIRONMENT	SOIL.	SOIL.	SOIL.	SOIL.
4. PRINCIPLE USED	MATERIAL PASSING THRU SCREENS. VELOCITY OF FREELY FALLING SPHERES.	DRYING OF SPECIMEN TO DETERMINE AMOUNT OF MOISTURE REMOVED.	RATIO OF A SAMPLES WEIGHT TO ITS VOLUME.	RATIO OF VOID VOLUME TO TOTAL VOLUME OF SPECIMEN.
5. QUANTITIES MEASURED	WEIGHT OF MATERIAL RETAINED ON SCREEN. HYDROMETER READINGS AT SPECIFIC TIME INTERVALS, WEIGHT OF DRIED SPECIMEN.	WEIGHT AND VOLUME OF MOIST SPECIMEN, OVEN-DRIED WEIGHT OF SPECIMEN, AND WEIGHT OF EQUAL VOLUME OF WATER TO SPECIMEN.	WEIGHT AND VOLUME OF MOIST SPECIMEN AND OVEN DRIED WEIGHT OF SAME SPECIMEN, AND SPECIFIC GRAVITY OF THE SOLIDS.	WEIGHT AND VOLUME OF MOIST SPECIMEN AND OVEN DRIED WEIGHT OF SAME SPECIMEN, AND SPECIFIC GRAVITY OF THE SOLIDS.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	PERCENTAGE WEIGHT OF MATERIAL BETWEEN SCREEN SIZES. EFFECTIVE DIAMETER OF SMALLER SIZES FROM HYDROMETER TEST. PLOT OF GRAIN SIZE DISTRIBUTION.	WATER CONTENT (IS ON A WEIGHT BASIS) OR THE SPECIFIC GRAVITY OF THE SOLIDS AND THEN THE DEGREE OF SATURATION (IS ON A VOLUME BASIS).	UNIT WEIGHTS OF SOLIDS, UNIT DRY WEIGHT OF SOIL, UNIT WET WEIGHT OF SOIL.	POROSITY AND/OR VOID RATIO.
7. COVERAGE	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
8. EFFECTIVE DEPTH	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
9. LIMITATIONS	MATERIAL MUST BE FREE OF FOREIGN MATTER.	SPECIMEN MUST CONTAIN ORIGINAL MOISTURE.	NOT APPLICABLE.	NOT APPLICABLE.
10. SENSITIVITY	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.
11. POSSIBLE ERRORS IN INTERPRETATION	SPECIMEN MAY NOT BE REPRESENTATIVE.	SPECIMEN MAY NOT BE REPRESENTATIVE.	SPECIMEN MAY NOT BE REPRESENTATIVE.	SPECIMEN MAY NOT BE REPRESENTATIVE.
12. PRINCIPAL INSTRUMENTS USED	SET OF SIEVES. HYDROMETER.	OVEN, ANALYTIC OR BEAM BALANCE, VOLUMETRIC SPECIMEN CUTTER.	ANALYTIC OR BEAM BALANCE, CONTAINERS.	ANALYTIC OR BEAM BALANCE, CONTAINERS.
13. ENERGY SOURCES	HEAT FOR DRYING OF SPECIMEN.	HEAT FOR DRYING OF MOIST SPECIMEN.	HEAT FOR DRYING OF SPECIMENS.	HEAT FOR DRYING OF SPECIMENS.
14. CREW SIZE REQUIRED	ONE	ONE.	ONE.	ONE
15. TIME REQUIRED AND WORK ACCOMPLISHED	8 TO 12 TESTS PER 8-HOUR SHIFT.	3 TO 5 TESTS PER 8-HOUR SHIFT	0 TO 12 TESTS PER 8-HOUR SHIFT.	8 TO 12 TESTS PER 8-HOUR SHIFT.
16. COST	ABOUT $20 PER TEST.	$45 TO $65 PER TEST.	ABOUT $20 PER TEST.	ABOUT $15 PER TEST.
17. POTENTIAL AREAS OF IMPROVEMENT	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED METHODS	DEVELOP IMPROVED METHODS.

Table 16. Comparison of subsurface exploration techniques, laboratory testing (continued).

CONDITIONS	LABORATORY TESTING OF SOIL			
	CONSISTENCY	SHEARING STRENGTH	UNCONFINED COMPRESSIVE STRENGTH	PERMEABILITY
1. SITE SELECTION	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	DETERMINE DEGREE OF FIRMNESS FOR A FINE-GRAINED SOIL FOR CLASSIFICATION.	DETERMINE STABILITY OF SOIL.	DETERMINE COMPRESSIVE STRENGTH OF SOIL. AS AN INDIRECT METHOD OF DETERMINING SOIL SHEARING STRENGTH.	DETERMINE ABILITY OF A SOIL TO CONDUCT FREE WATER UNDER A GIVEN HYDRAULIC GRADIENT.
3. GEOLOGICAL ENVIRONMENT	CLAY AND SILT.	SOIL.	SOIL.	SOIL.
4. PRINCIPLE USED	CHANGE IN STRENGTH WITH CHANGE IN MOISTURE CONTENT.	MEASUREMENT OF FORCE THAT CAUSES A SPECIMEN TO FAIL.	UNIAXIAL LOADING TO SPECIMEN FAILURE.	MEASURING THE AMOUNT OF WATER FLOW FORCED THROUGH A SPECIMEN.
5. QUANTITIES MEASURED	SPECIMEN VOLUME, WEIGHT AT EACH CONSISTENCY-LIMIT CONDITIONS, AND VOLUME AND WEIGHT OF SPECIMEN WHEN OVEN-DRIED.	NORMAL AND SHEARING FORCES IN DIRECT SHEAR TEST. NORMAL AND CONFINING PRESSURES, AND PORE PRESSURE IF APPLICABLE, IN TRIAXIAL TEST. SPECIMEN SIZE AND VOLUME CHANGE DURING SHEARING IN BOTH.	AXIAL APPLIED FORCE AND VERTICAL DEFORMATION OF THE SPECIMEN AT INCREMENTAL PERIODS DURING TEST.	WATER VISCOSITY, LENGTH AND CROSS-SECTIONAL AREA OF SPECIMEN, ELAPSED TEST TIME, AND FLOW QUANTITY AND HEAD LOSS FOR CONSTANT-HEAD TEST. OR INSIDE CROSS-SECTIONAL AREA OF STANDPIPE AND DIFFERENCE IN STANDPIPE WATER LEVEL DURING TEST FOR FALLING-HEAD TEST.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	WATER CONTENT AT EACH CONSISTENCY LIMIT, AND CONSISTENCY INDEXES.	ANGLE OF INTERNAL FRICTION AND COHESION INTERCEPT FROM MOHR ENVELOPE DIAGRAM OF TEST RESULTS. MODULUS OF ELASTICITY FROM SLOPE OF STRESS STRAIN CURVE.	AXIAL STRAIN AND COMPRESSIVE STRESS. USUALLY PLOT OF STRESS VERSUS STRAIN IS MADE.	COEFFICIENT OF PERMEABILITY FOR LAMINAR FLOW.
7. COVERAGE	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
8. EFFECTIVE DEPTH	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
9. LIMITATIONS	ONLY SUITABLE FOR FINE-GRAINED SOILS (MINUS 40 MESH).	IN SITU CONDITIONS ARE ONLY APPROXIMATED IN TRIAXIAL TEST. STATE-OF-STRESS ONLY DETERMINED AT FAILURE IN DIRECT SHEAR TEST.	SOIL MUST HAVE SOME COHERENCE.	RESULTS ONLY VALID FOR PARTICULAR SOIL TESTED. ONLY CONSIDERS STEADY-STATE FLOW CONDITIONS.
10. SENSITIVITY	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.
11. POSSIBLE ERRORS IN INTERPRETATION	SPECIMEN ALLOWED TO DRY BEFORE TESTING CAN GIVE ERRONEOUS RESULTS. SPECIMEN SLIPS RATHER THAN FLOWS IN LIQUID LIMIT TEST.	IN SOFT SOILS, SAMPLE DISTURBANCE FREQUENTLY OVER-COMPENSATED BY TRIAXIAL TESTS.	SINGLE SPECIMEN RESULT MAY VARY WIDELY FROM AVERAGE STRENGTH.	TEST PROCEDURES MAY NOT BE INDICATIVE OF ACTUAL IN SITU CONDITIONS.
12. PRINCIPAL INSTRUMENTS USED	LIQUID LIMIT DEVICE, SHRINKAGE DISH.	DIRECT SHEAR DEVICE, TRIAXIAL COMPRESSION DEVICE.	UNCONFINED COMPRESSION DEVICE.	CONSTANT-HEAD AND FALLING-HEAD TEST DEVICES.
13. ENERGY SOURCES	HEAT FOR DRYING OF SPECIMENS.	MECHANICAL JACK OR HYDRAULIC FLUID PRESSURED BY PUMP OR HAND LEVER.	MECHANICAL JACK OR HYDRAULIC FLUID PRESSURED BY PUMP OR HAND LEVER.	WATER HEAD.
14. CREW SIZE REQUIRED	ONE	1 OR 2	1 OR 2	ONE.
15. TIME REQUIRED AND WORK ACCOMPLISHED	3 TO 5 DETERMINATIONS FOR EACH CONSISTENCY LIMIT AND INDEX PER 8-HOUR SHIFT.	5 TO 8 DIRECT SHEAR AND 2 TO 3 TRIAXIAL SHEAR TESTS PER 8-HOUR SHIFT.	1 TO 3 HOURS PER TEST.	3 TO 6 TESTS PER 8-HOUR SHIFT.
16. COST	ABOUT $30 PER TEST.	$10 TO $15 PER TEST FOR DIRECT SHEAR AND ABOUT $35 FOR TRIAXIAL SHEAR.	ABOUT $40 PER TEST.	ABOUT $75 PER TEST.
17. POTENTIAL AREAS OF IMPROVEMENT	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED METHODS.

Table 16. Comparison of subsurface exploration techniques, laboratory testing (continued).

CONDITIONS	LABORATORY TESTING OF SOIL	LABORATORY TESTING OF ROCK		
	SWELLING	IDENTIFICATION	SPECIFIC GRAVITY	APPARENT POROSITY
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	DETERMINE THE SWELLING CHARACTERISTICS OF A CLAY.	FOR CLASSIFICATION OF ROCK TYPES.	USED IN ENGINEERING DESIGN.	USED IN ENGINEERING DESIGN.
3. GEOLOGICAL ENVIRONMENT	CLAYS.	ROCK.	ROCK.	ROCK.
4. PRINCIPLE USED	CHANGE IN VOLUME AND EXPANSION PRESSURE OF SPECIMEN WHEN WATER IS ADDED.	UNAIDED VISUAL INSPECTION, MICROSCOPIC EXAMINATION, CHEMICAL AND SPECTROGRAPHIC ANALYSIS, X-RAY DIFFRACTION, DIFFERENTIAL THERMAL ANALYSIS.	RATIO OF THE SPECIMEN MASS TO THE MASS OF AN EQUAL VOLUME OF WATER AT A SPECIFIED TEMPERATURE.	RATIO OF PORE SPACE VOLUME TO THE TOTAL VOLUME OF SPECIMEN.
5. QUANTITIES MEASURED	DIFFERENCE IN SAMPLE VOLUME AT DRY AND SATURATED CONDITIONS. PRESSURE EXERTED BY SAMPLE WHEN IT IS WETTED.	MINERAL SIZE, SHAPE, ARRANGEMENT AND COMPOSITION. DEGREE OF COHERENCE.	METHOD 1 - DRY WEIGHT, WET WEIGHT IN AIR, WET WEIGHT IN WATER. METHOD 2 - DRY WEIGHT, BULK VOLUME. METHOD 3 - DRY WEIGHT, APPARENT VOLUME (FROM SIZE MEASUREMENTS).	METHOD 1 - DRY WEIGHT, WET WEIGHT, APPARENT VOLUME (FROM SIZE MEASUREMENTS). METHOD 2 - APPARENT VOLUME (BY PYCNOMETER), APPARENT VOLUME (FROM SIZE MEASUREMENTS).
6. QUANTITIES COMPUTED FROM MEASUREMENTS	CHANGE IN VOLUME AND/OR EXPANSION PRESSURE WHEN WETTED.	NOT APPLICABLE.	SPECIFIC GRAVITY IN GRAMS PER CUBIC CENTIMETER.	APPARENT POROSITY IN PERCENT.
7. COVERAGE	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
8. EFFECTIVE DEPTH	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
9. LIMITATIONS	GIVES APPROXIMATE INFORMATION ONLY.	MUST HAVE UNDISTURBED SAMPLE FOR SOME OBSERVATIONS.	NOT APPLICABLE.	NOT APPLICABLE.
10. SENSITIVITY	DEPENDS ON ADHERENCE TO RECOMMENDED TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON COMPLEXITY OF COMPOSITION AND TEXTURE.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.
11. POSSIBLE ERRORS IN INTERPRETATION	TEST PROCEDURES NOT INDICATIVE OF ACTUAL IN SITU CONDITIONS.	SAMPLE MAY NOT BE REPRESENTATIVE.	SPECIMEN MAY NOT BE REPRESENTATIVE.	SAMPLE MAY NOT BE REPRESENTATIVE.
12. PRINCIPAL INSTRUMENTS USED	SIEVE SCREENS AND GRADUATED CYLINDER. CONSOLIDOMETER.	HAND LENS, PETROGRAPHIC, BINOCULAR, OR ELECTRON MICROSCOPE, X-RAY DIFFRACTOMETER, DIFFERENTIAL THERMAL ANALYZER, SPECTROGRAPH, QUALITATIVE CHEMICAL LABORATORY EQUIPMENT.	OVEN, ANALYTIC OR BEAM BALANCE, PYCNOMETER, MICROMETER.	OVEN, ANALYTIC BALANCE, PYCNOMETER, MICROMETER.
13. ENERGY SOURCES	GRAVITY.	ELECTRICAL CURRENT.	HEAT FOR DRYING OF SPECIMEN.	HEAT FOR DRYING OF SPECIMEN.
14. CREW SIZE REQUIRED	ONE.	ONE.	ONE.	ONE.
15. TIME REQUIRED AND WORK ACCOMPLISHED	USUALLY OVER A 24-HOUR TEST PERIOD.	DEPENDS ON EXPERIENCE OF EXAMINER AND COMPLEXITY OF SAMPLE MINERALOGY.	20 TO 60 DETERMINATIONS PER 8-HOUR SHIFT	20 TO 60 DETERMINATIONS PER 8-HOUR SHIFT
16. COST	ABOUT $20 PER TEST	VARIABLE UPON METHODS USED.	ABOUT $10 PER TEST.	ABOUT $10 PER TEST.
17. POTENTIAL AREAS OF IMPROVEMENT	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED METHODS.

Table 16. Comparison of subsurface exploration techniques, laboratory testing (continued).

CONDITIONS	LABORATORY TESTING OF ROCK			
	HARDNESS	ABRASION RESISTANCE	WEATHERING RESISTANCE	UNIAXIAL COMPRESSIVE STRENGTH
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	USED IN DETERMINATION OF "DRILL-ABILITY".	USED IN SELECTION OF MUCK HANDLING AND PUMPING EQUIPMENT.	ESTIMATION OF LONG-TERM STABILITY FOR TUNNEL WALLS.	USED FOR SELECTION OF EXCAVATION METHOD, DETERMINATION OF SUPPORT REQUIREMENTS.
3. GEOLOGICAL ENVIRONMENT	ROCK.	ROCK.	ROCK.	ROCK.
4. PRINCIPLE USED	REBOUND OF POINTED HAMMER OR PISTON, SCRATCHING, CRUSHING, MINIATURE-DRILLED HOLES.	ABRADING OF MATERIAL BY ACCELERATED WEAR.	AIR-WATER SLAKING TESTS, SWELLING TESTS, LEACHING TESTS.	UNIAXIAL LOADING TO COMPRESSIONAL FAILURE.
5. QUANTITIES MEASURED	HEIGHT OF HAMMER OR PISTON REBOUND, AMOUNT OF MATERIAL CRUSHED TO A 0.5 MM SIZE AND HAMMER BLOWS REQUIRED.	AMOUNT OF MATERIAL ABRADED DURING TEST.	RESISTANCE TO SLAKING, SOLUTION, OR EROSION DETERMINED. IN SWELLING TEST DETERMINE DIFFERENCE IN SAMPLE VOLUME AT DRY AND SATURATED CONDITIONS AND PRESSURE EXERTED BY SAMPLE WHEN WETTED.	SPECIMEN LENGTH AND DIAMETER, COMPRESSIVE LOAD REQUIRED TO CAUSE FAILURE.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	RELATIVE HARDNESS.	RELATIVE ABRASIVENESS.	ESTIMATED TIME FOR ROCK TO DETERIORATE.	UNIAXIAL COMPRESSIVE STRENGTH IN PSI.
7. COVERAGE	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
8. EFFECTIVE DEPTH	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
9. LIMITATIONS	DEPENDS ON PROPERTIES OF INDIVIDUAL GRAINS AND NATURE OF GRAIN BONDING.	DEPENDS ON PROPERTIES OF INDIVIDUAL GRAINS AND NATURE OF GRAIN BONDING.	TEST NOT QUANTITATIVE.	INTACT, FRACTURE-FREE SPECIMENS REQUIRED.
10. SENSITIVITY	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON LENGTH OF TEST PERIOD.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.
11. POSSIBLE ERRORS IN INTERPRETATION	SPECIMEN MAY NOT BE INDICATIVE OF IN SITU MASS. RESULTS ARE ONLY RELATIVE.	RESULTS ARE ONLY RELATIVE.	DIFFICULT TO SIMULATE LONG TERM WEATHERING EFFECTS IN SHORT TIME PERIOD.	SINGLE SPECIMEN MAY VARY WIDELY FROM AVERAGE STRENGTH.
12. PRINCIPAL INSTRUMENTS USED	SHORE SCLEROSCOPE, SCHMIDT IMPACT HAMMER, PROTODYAKONOV DROP TESTER AND VOLUMETER, MICROBIT DRILL.	DORRY ABRASIVE MACHINE, PADDLE TYPE MACHINE, LOS ANGELES ABRASION MACHINE.	SWELLING TEST - SIEVE SCREENS, GRADUATED CYLINDER, CONSOLIDOMETER.	DIAMOND CORE DRILL, DIAMOND SAW, SURFACE GRINDER, POLISHING LAP, CALIPER, HYDRAULIC PRESS.
13. ENERGY SOURCES	GRAVITY, SPRINGS.	ELECTRICAL POWER.	NOT APPLICABLE.	HYDRAULIC FLUID PRESSURED BY PUMP OR HAND LEVER.
14. CREW SIZE REQUIRED	ONE.	ONE.	ONE.	ONE.
15. TIME REQUIRED AND WORK ACCOMPLISHED	ABOUT 25 SAMPLES PER 8-HOUR SHIFT USING SCLEROSCOPE OR IMPACT HAMMER.	4 TO 8 TESTS PER 8-HOUR SHIFT.	SLAKING AND LEACHING TESTS MAY BE CONTINUED FOR SEVERAL WEEKS.	ABOUT 15 SAMPLES PER 8-HOUR SHIFT.
16. COST	ABOUT $10 PER TEST FOR SCLEROSCOPE.	ABOUT $70 PER TEST FOR LOS ANGELES TEST.	ABOUT $5 PER TEST.	$20 TO $30 PER TEST.
17. POTENTIAL AREAS OF IMPROVEMENT	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED METHODS.	DEVELOP A STANDARD TEST METHOD.	DEVELOP IMPROVED METHODS.

Table 16. Comparison of subsurface exploration techniques, laboratory testing (continued).

CONDITIONS	LABORATORY TESTING OF ROCKS			
	UNIAXIAL SHEAR STRENGTH	UNIAXIAL TENSILE STRENGTH	UNIAXIAL FLEXURAL STRENGTH	TRIAXIAL COMPRESSIVE & SHEAR STRENGTH
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	LIMITED.	LIMITED.	USED IN DESIGN OF GROUND SUPPORT.	USED IN SELECTION OF EXCAVATION METHOD.
3. GEOLOGICAL ENVIRONMENT	ROCK.	ROCK, ESPECIALLY LAYERED ROCK.	ROCK, ESPECIALLY LAYERED ROCK.	ROCK.
4. PRINCIPLE USED	LOADING TO SHEARING FAILURE.	DIRECT, INDIRECT, RING, AND POINT LOAD TESTS USED.	LOADING TO BENDING OR FLEXURAL FAILURE.	UNIAXIAL LOADING, WITH CONFINING PRESSURE, TO SPECIMEN FAILURE.
5. QUANTITIES MEASURED	CROSS-SECTIONAL AREA, THICKNESS OR DIAMETER OF SPECIMEN, FORCE OR TORQUE REQUIRED TO CAUSE FAILURE, RADIUS OF PUNCH. MEASUREMENTS NEEDED DEPEND ON TEST TECHNIQUE.	SPECIMEN LENGTH AND DIAMETER, TENSILE LOAD CAUSING FAILURE IN INDIRECT, RING AND POINT LOAD TESTS.	SPECIMEN DIAMETER OR THICKNESS AND WIDTH, LENGTH BETWEEN BEARING EDGES OF LOWER PLATE, APPLIED LOAD AT FAILURE.	SPECIMEN LENGTH AND DIAMETER, CONFINING PRESSURE, COMPRESSIVE LOAD REQUIRED TO CAUSE FAILURE, STRAIN RATE, DEFORMATION, INCLINATION OF FAILURE PLANE.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	UNIAXIAL SHEAR STRENGTH IN PSI.	UNIAXIAL TENSILE STRENGTH IN PSI.	UNIAXIAL FLEXURAL STRENGTH IN PSI. MODULUS OF RUPTURE.	CONSTRUCT A STRESS DIFFERENCE VS AXIAL STRAIN CURVE. CONSTRUCT MOHR STRESS CIRCLES ON ARITHMETIC PLOT WITH SHEAR STRESSES AS ORDINATES AND NORMAL STRESSES AS ABSCISSAS. DRAW MOHR ENVELOPE TANGENT TO MOHR CIRCLES. DETERMINE ANGLE OF SHEARING RESISTANCE AND THE COHESION INTERCEPT.
7. COVERAGE	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
8. EFFECTIVE DEPTH	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
9. LIMITATIONS	INTACT, FRACTURE-FREE SPECIMEN REQUIRED.	INTACT, FRACTURE-FREE SPECIMENS REQUIRED.	INTACT, FRACTURE-FREE SPECIMENS REQUIRED.	INTACT, FRACTURE-FREE SPECIMENS REQUIRED.
10. SENSITIVITY	DEPENDS ON ADHERENCE TO RECOMMENDED TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO RECOMMENDED TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.
11. POSSIBLE ERRORS IN INTERPRETATION	IN SINGLE, DOUBLE, AND PUNCH TESTS, MAJOR SHEAR STRESS BUILDUPS OCCUR ALONG SHEAR OR PUNCH AND DIE EDGES, CAUSING FRACTURE FAILURE IN DIRECTION DIFFERENT FROM THAT OF MAXIMUM SHEAR STRESS.	SPECIMEN CONTAINING UNDETECTED STRUCTURAL FLAWS CAN GIVE ERRONEOUS RESULTS.	SPECIMEN CONTAINING UNDETECTED STRUCTURAL FLAWS CAN GIVE ERRONEOUS RESULTS.	SPECIMEN CONTAINING UNDETECTED STRUCTURAL FLAWS CAN GIVE ERRONEOUS RESULTS.
12. PRINCIPAL INSTRUMENTS USED	DIAMOND CORE DRILL, DIAMOND SAW, SURFACE GRINDER, POLISHING LAP, CALIPER, HYDRAULIC PRESS.	LOADING DEVICE, METAL CAPS, CEMENTING MATERIAL, DIAMOND CORE DRILL, DIAMOND SAW, SURFACE GRINDER, CALIPER.	DIAMOND CORE DRILL, DIAMOND SAW, CALIPER, LOADING DEVICE, LOAD-APPLYING AND SUPPORT BLOCKS.	DIAMOND CORE DRILL, DIAMOND SAW, SURFACE GRINDER, POLISHING LAP, CALIPER, LOADING DEVICE, PRESSURE-MAINTAINING DEVICE, TRIAXIAL COMPRESSION CHAMBER, DEFORMATION MEASURING DEVICE, FLEXIBLE MEMBRANE.
13. ENERGY SOURCES	HYDRAULIC FLUID PRESSURED BY PUMP OR HAND LEVER.	HYDRAULIC FLUID PRESSURED BY PUMP OR HAND LEVER.	SCREW JACK, HYDRAULIC FLUID PRESSURED BY PUMP OR HAND LEVER.	HYDRAULIC FLUID PRESSURED BY PUMP OR HAND LEVER.
14. CREW SIZE REQUIRED	ONE.	ONE.	ONE	1 OR 2.
15. TIME REQUIRED AND WORK ACCOMPLISHED	ABOUT 15 SAMPLES PER 8-HOUR SHIFT.	ABOUT 15 SAMPLES PER 8-HOUR SHIFT.	ABOUT 15 SAMPLES PER 8-HOUR SHIFT.	ABOUT 4 SAMPLES PER 8-HOUR SHIFT.
16. COST	$10 TO $20 PER TEST.	ABOUT $10 PER TEST FOR BRAZILIAN, ABOUT $40 FOR DIRECT.	ABOUT $25 PER TEST.	ABOUT $50 PER TEST.
17. POTENTIAL AREAS OF IMPROVEMENT	DEVELOP A STANDARD TEST METHOD.	DEVELOP IMPROVED METHODS.	DEVELOP A STANDARD TEST METHOD.	DEVELOP IMPROVED METHODS.

Table 16. Comparison of subsurface exploration techniques, laboratory testing (continued).

CONDITIONS	LABORATORY TESTING OF ROCK			
	STATIC ELASTIC CONSTANTS	DYNAMIC ELASTIC CONSTANTS	PRIMARY PERMEABILITY	CREEP
1. SITE SELECTION STAGE	DETAILED INVESTIGATION.	DETAILED INVESTIGATION.	DETAILED INVESTIGATION	DETAILED INVESTIGATION.
2. ENGINEERING APPLICATION	USED IN ESTIMATION OF ROCK STABILITY.	USED IN ESTIMATION OF ROCK STABILITY	DETERMINE ABILITY OF A ROCK TO CONDUCT FREE WATER UNDER A GIVEN HYDRAULIC GRADIENT.	USED FOR DETERMINING LONG-TERM ROCK DEFORMATION.
3. GEOLOGICAL ENVIRONMENT	ROCK.	ROCK.	ROCK.	ROCK.
4. PRINCIPLE USED	STRESS-STRAIN RELATIONSHIPS.	DYNAMIC MODULI DETERMINED FROM RESONANT FREQUENCY OR ULTRASONIC PULSE VELOCITIES.	MEASURING THE AMOUNT OF WATER OR AIR FLOW FORCED THROUGH A SPECIMEN.	DETERMINATION OF LONG-TERM CREEP RATE UNDER CONSTANT LOAD.
5. QUANTITIES MEASURED	AXIAL COMPRESSIVE OR TENSILE STRESS, VERTICAL AND LATERAL STRAIN.	RESONANT FREQUENCY METHOD - LONGITUDINAL, TRANSVERSE, AND TORSIONAL FREQUENCIES OF VIBRATING PRISMS OR CYLINDERS, SPECIMEN WEIGHT, LENGTH, AND DIAMETER OF CROSS-SECTIONAL AREA. ULTRASONIC PULSE METHOD - PULSE VELOCITIES OF COMPRESSION AND SHEAR WAVES, SPECIMEN WEIGHT, VOLUME, DENSITY, APPARENT POROSITY, AND DEGREE OF SATURATION.	SAME AS SOIL PERMEABILITY TEST WHEN USING WATER. INLET AND OUTLET PRESSURES AND FLOW RATE WHEN USING GAS.	SPECIMEN DEFORMATION AS A FUNCTION OF TIME, DIMENSIONS OF SPECIMEN, APPLIED LOAD.
6. QUANTITIES COMPUTED FROM MEASUREMENTS	MODULUS OF ELASTICITY (YOUNG'S MODULUS), POISSON'S RATIO, BULK MODULUS, MODULUS OF RIGIDITY, LAME'S CONSTANT.	DYNAMIC MODULUS OF ELASTICITY (DYNAMIC YOUNG'S MODULUS), DYNAMIC POISSON'S RATIO, DYNAMIC MODULUS OF RIGIDITY, DYNAMIC BULK MODULUS, DYNAMIC LAME'S CONSTANT.	COEFFICIENT OF PERMEABILITY FOR LAMINAR FLOW.	STRAIN VS TIME CREEP CURVE PLOTTED.
7. COVERAGE	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
8. EFFECTIVE DEPTH	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.	NOT APPLICABLE.
9. LIMITATIONS	INTACT, FRACTURE-FREE SPECIMENS REQUIRED.	INTACT, FRACTURE-FREE SPECIMENS REQUIRED.	RESULTS ONLY VALID FOR PARTICULAR ROCK TESTED. ONLY CONSIDERS STEADY-STATE FLOW CONDITIONS.	CONSIDERABLE LENGTH OF TIME REQUIRED FOR TEST.
10. SENSITIVITY	DEPENDS ON ADHERENCE TO RECOMMENDED TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO RECOMMENDED TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO STANDARD TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.	DEPENDS ON ADHERENCE TO RECOMMENDED TEST PROCEDURES AND ACCURACY OF MEASUREMENTS.
11. POSSIBLE ERRORS IN INTERPRETATION	SPECIMEN MAY NOT BE INDICATIVE OF IN SITU MASS.	SPECIMEN MAY NOT BE INDICATIVE OF IN SITU MASS.	TEST PROCEDURES MAY NOT BE INDICATIVE OF ACTUAL IN SITU CONDITIONS.	RESULTS MAINLY QUALITATIVE.
12. PRINCIPAL INSTRUMENTS USED	DIAMOND CORE DRILL, DIAMOND SAW, SURFACE GRINDER, LOADING DEVICE, CLAMPING DEVICE WITH STRAIN GAUGES.	DIAMOND CORE DRILL, DIAMOND SAW, SURFACE GRINDER, BALANCE, CALIPER. RESONANT FREQUENCY METHOD - DRIVING CIRCUIT, PICKUP CIRCUIT, SPECIMEN SUPPORT. ULTRASONIC PULSE METHOD - PULSE GENERATOR UNIT, TRANSDUCERS, PREAMPLIFIER, DISPLAY AND TIMING UNIT.	CONSTANT-HEAD AND FALLING HEAD WATER TEST DEVICES. AIR PERMEAMETER.	DIAMOND CORE DRILL, DIAMOND SAW, SURFACE GRINDER, POLISHING LAP, CALIPER, LOADING DEVICE, STRAIN-MEASURING DEVICES.
13. ENERGY SOURCES	HYDRAULIC FLUID PRESSURED BY PUMP OR HAND LEVER.	ELECTRICAL CURRENT.	WATER HEAD OR COMPRESSED AIR PRESSURE.	GRAVITY.
14. CREW SIZE REQUIRED	ONE.	ONE.	ONE.	ONE.
15. TIME REQUIRED AND WORK ACCOMPLISHED	4 TO 6 TESTS PER 8-HOUR SHIFT.	8 TO 12 TEST PER 8-HOUR SHIFT FOR YOUNG'S MODULUS AND POISSON'S RATIO.	6 TO 10 TESTS PER 8-HOUR SHIFT.	TESTS MAY EXTEND OVER MANY WEEKS.
16. COST	ABOUT $75 PER TEST FOR YOUNG'S MODULUS AND POISSON'S RATIO.	ABOUT $50 PER TEST FOR YOUNG'S MODULUS AND POISSON'S RATIO.	ABOUT $30 PER TEST.	VARIABLE UPON LENGTH OF TEST PERIOD.
17. POTENTIAL AREAS OF IMPROVEMENT	DEVELOP STANDARD TEST METHODS.	DEVELOP STANDARD TEST METHODS.	DEVELOP IMPROVED METHODS.	DEVELOP IMPROVED ACCELERATED METHODS.

To avoid premature elimination of potentially useful techniques, no attempt was made to screen these techniques until we reached the system synthesis stage of our study.

TUNNEL DESIGN REQUIREMENTS

The optimum design of a structure requires a complete and thorough knowledge of the materials that are to be used in its construction. In the case of a tunnel, this means the characteristics of the materials through which the tunnel is to be driven must be known in sufficient detail to allow an accurate prediction of the ground behavior in the tunnel vicinity, both during construction and throughout the life of the tunnel after construction is completed. The accuracy with which the ground conditions at the tunnel location can be predicted will govern the number and magnitude of the unforeseen problems that will be encountered, both during and after construction of the tunnel.

The type of geologic and other site information generally required to adequately design a highway tunnel is listed in Table 17.

TUNNELING PROBLEMS

"Problem - (1) A perplexing question that demands a solution or (2) a situation that causes perplexity or puts one in a predicament." If a subsurface investigation for a tunnel is designed to provide a solution to the perplexing question of adequate design and economic construction, then it follows that sufficient subsurface information, accurately interpreted and properly applied should avoid "a situation that causes perplexity or puts one in a predicament." Such situations related to subsurface conditions can be grouped into six major tunneling conditions encountered which are in excess of those predicted.

1. Changes in ground conditions
2. Water inflow
3. Face instability
4. Ground pressure
5. Settlement of the ground surface
6. Ground vibration caused by blasting

The accurate prediction of these six conditions is vital to the "adequate" design and "economic" construction of a highway tunnel.

Table 17. Information required for highway tunnel design.

TUNNEL CHARACTERISTICS	Location		
	Physical Characteristics		Size Shape Length Alignment
	Cover Thickness		
	Proximity to Other Man-Made Structures		Type Location Orientation
REGIONAL AND LOCAL GEOLOGY	Structural Features	Folds Faults Shear zones Unconformities Contacts Bedding Jointing Fractures Intrusions Solution channels Bedrock surface	Strike Dip Extent Size Spacing Type Kind Frequency Uniformity Surface characteristics Filling material characteristics
	Susceptibility To Ground Movement	Earthquakes Slides Avalanches Slumping	
	Natural Hazards	Dangerous gases	Pressure Rate of release
		Ground temperature	Magnitude Gradient
GROUND CONDITIONS	Soil	Composition Texture Structure	
		Index properties	Particles size distribution shape surface area Density Porosity Moisture content absorbed capillary Consistency Cohesion

Table 17. Information required for highway tunnel design (continued).

GROUND CONDITIONS	Soil	Engineering properties	Permeability constant head variable head Compressibility indexes Consolidation Shearing strength Compressive strength Modulus of elasticity Penetration resistance Swelling characteristics Poisson's ratio
	Rock	Types and composition	Mineralogy Crystal structure Grain size Bonding Degree of decomposition or alteration Cementation
		Texture	Amorphous Aphanitic Crystalline Granular Elastic Porphyritic Banded Foliated Glassy
		Degree of Anisotropy and Homogeneity	
		Index properties	Density Porosity Moisture content

Table 17. Information required for highway tunnel design (continued).

GROUND CONDITIONS	Rock	Engineering properties	Compressive strength unconfined triaxial Tensile strength Shearing strength unconfined triaxial Flexural strength Modulus of elasticity Poisson's ratio Impact resistance Hardness Abrasive resistance Weathering resistance Permeability Solubility Swelling characteristics Creep characteristics Heat conductivity Thermal expansion characteristics
		Stresses	Residual ground stresses Stress relief characteristics
HYDROLOGICAL CONDITIONS	Surface Drainage		
	Ground water		Location Composition Rainfall and seasonal effects Storage capabilities Storage coefficient Transmissibility External uses Barrier boundary locations Recharge sources Pressure head Extent Volume flow rates

CHANGES IN GROUND CONDITIONS

In tunnel construction, the nature of the material to be excavated affects all construction operations, including drilling speed, mucking speed, amount of supports, overbreak, and water handling. If a properly designed and conducted subsurface investigation permits an accurate description of anticipated ground conditions then there is no excuse for a problem occurring as a result of changes in ground conditions during construction.

EXCESS WATER INFLOW

The amount of water inflow encountered in a tunnel will depend upon such factors as:

1. Permeability (both primary and secondary) of the ground mass surrounding the tunnel.
2. Hydraulic gradient causing the water to flow.
3. Size of the flow paths exposed to the water inflow.
4. Amount of recharge to the water-flow source.
5. Storage capacity of the water-flow source.
6. Extent and degree of influence exerted by the tunnel upon the ground water.
7. Fluctuations and variability of these different factors.

Darcy's equation is the simplest, and probably the most commonly used, expression for obtaining an estimate as to the water inflow expected in a tunnel. Simply stated, Darcy's equation is

$$Q = kAi$$

where Q is the quantity of water flowing through a cross-sectional area A of a material having a coefficient of permeability of k and with a hydraulic gradient of i.

The Darcy equation assumes laminar flow under steady state conditions. There will be some cases where the water flow through the ground mass is confined to specific paths or routes while in other instances the water may enter a tunnel from the entire surrounding ground. It is, therefore, imperative that the boundaries of possible water flow routes be determined so that a reasonable estimate of the possible water inflow volumes can be made from Darcy's equation. The construction of a flow net is helpful for visualizing the flow within a watercourse, especially when a tunnel intersects and/or is contained within a confined aquifer.

FACE STABILITY

The stability of the tunnel face is a major concern on many projects. Face stability is most frequently associated with soft ground tunnels but can also apply to some rock tunnels. It will often dictate the construction procedures used. The stability of a tunnel face will depend upon at least the following six factors:

1. Shearing strength of the ground mass.
2. Overburden pressure.
3. Depth-to-width ratio for the tunnel.
4. Stress-strain characteristics of the ground mass.
5. Time-dependent strength loss and delayed deformation of the ground.
6. Construction procedures used.

Different kinds of instability may occur depending upon ground cohesion and permeability. In low permeability clays, the initial stability is largely controlled by the undrained shear strength of the clay. After the tunnel face has been exposed for some extended period, the stability will then be controlled by the effective (or drained) shear strength. When the ground has a high permeability so that rapid drainage occurs then the stability is controlled from the outset by the effective shear strength.

In predicting face stability, the procedure usually followed is to consider a tunnel face in clay as having a sufficient short-time stability when the ratio of the overburden pressure to the undrained shear strength is less than six.[11] As the face is allowed to stand, however, the porewater pressure dissipates which then usually leads to a volume expansion with a resulting decrease in face material strength. Tests are being conducted to develop other stability parameters in addition to the undrained shear strength such as the liquidity index.[12]

Ground water conditions are the most important single factor for the stability of a tunnel face in a noncoherent material.[11] Above the water table, a noncohesive material will run into the tunnel until a stable slope is formed with a slope angle equal to the friction angle of the material in a loose state. With some cohesion, however, the

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[11] Deere, D.U., R.B. Peck, J.E. Monsees, and B. Schmidt. Design of Tunnel Liners and Support Systems. Report for U.S. Dept. of Transportation OHSGT, Contract 3-0152. Univ. of Ill. February, 1969.

[12] Attewell, P.B. and J.B. Bodin. "Development of Stability Ratios for Tunnels Driven in Clay." Tunnels and Tunneling, Vol. 3 No. 3 (May-June, 1971), pp. 195-198.

face will stand unsupported over some limited tunnel height dependent upon the amount of cohesion. Below the water table, the stability will depend upon the ability of the material cohesion to withstand the seepage forces of the water draining into the tunnel. It is usually very difficult to estimate the cohesion that is available to withstand these seepage forces because, although reliable techniques are available to estimate the face stability, the accuracy of the data upon which the estimate is based is usually insufficiently accurate.

GROUND PRESSURE

Many theories abound in the literature concerning the determination of ground pressures and design of tunnel supports and lining. Most of these theories however, are restrictive in application and are seldom used. In actual tunneling practice the anticipated ground pressures are usually estimated from empirical expressions. If the tunnel is located near the ground surface, the full overburden pressure is usually used as the ground load. For deeper tunnels, the empirical formulas most often used are those developed by Terzaghi.[13] In general terms, Terzaghi's formulas can be expressed as

$$H = nw \text{ and } H = N(w+h)$$

where H is the ground load in feet of ground for a tunnel having a width of w feet, a height of h feet, and an overburden thickness of at least $1.5(w+h)$ feet. The values for the coefficients n and N depend upon the condition of the ground material.

The Tunnelmans' Ground Classification (which is described in the subsection of this report entitled Subsurface Conditions) is used to distinguish between the different ground conditions for Terzaghi's formulas. Table 18 gives the value ranges for the coefficients n and N as recommended by Terzaghi for use in his equation.

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[13] Terzaghi, Karl. "Introduction to Tunnel Geology" in Rock Tunneling with Steel Supports by Robert V. Proctor and Thomas L. White. Commercial Shearing and Stamping Co., Youngstown, Ohio. 1946.

Table 18. Coefficients for use in Terzaghi rock-load formulas.

	Rock Conditions	Coefficients
1.	Intact rock	$n = N = 0$
2.	Stratified or schistose rock	$n = 0$ to 0.5
3.	Massive, moderately jointed rock	$n = 0$ to 0.25
4.	Moderately blocky and seamy rock	$n = 0.25$ to $N = 0.35$
5.	Very block and seamy rock	$n = 0.35$ to 1.10
6.	Cohesionless material such as sand	$N = 0.60$
7.	Crushed but chemically intact rock	$N = 1.10$
8.	Squeezing rock at moderate depth	$N = 1.10$ to 2.10
9.	Squeezing rock at great depth	$N = 2.10$ to 4.50
10.	Swelling rock	Up to 250 feet irrespective of w and h

In table 18, the tunnel roof is assumed to be below the water table. If instead it is located above the water table, then the coefficient values for rock conditions 4, 5, and 7 can be reduced by 50 percent. The division between rock conditions 4 and 5 is usually taken at an individual block size of two feet, that is if the individual blocks are smaller than two feet, the rock is classified as being very blocky and seamy. Crushed rock (class 7) may be partly recemented with the individual particles being of a gravel size or smaller.

The stand-up time or bridge-action period for an underground opening is extremely important in a tunneling project. The stand-up time is defined as the elapsed time between excavations and perceptible movement of the tunnel roof. It is a function of the rock type, the amount of disturbance imparted to the material during excavation, and the active span of the tunnel which is the greater between the unsupported length or width of the tunnel opening. Two classifications are in general use, one by Lauffer and the other by Terzaghi.

Lauffer used seven rock classes and related the length of time that an opening can stand unsupported to the active span by means of a graph which is divided into the different rock classes.[14] The general equation used by Lauffer is:

$$t = \text{constant} \times L^{-(1+f)}$$

[14] Deere, D.U., R.B. Peck, J.E. Monsees, and B. Schmidt. Design of Tunnel Liners and Support Systems. Report for U.S. Dept. of Transportation, OHSGT, Contract 3-0152. Univ. of Ill. February, 1969.

where t is the stand-up time in hours,

L is the active span in meters, and

f is a constant that varies according to the rock class.

The equations which he used to separate the different rock classes were:

$t = 100000L^{-1}$ for separating rock classes A and B

$t = 2500L^{-1.2}$ for separating rock classes B and C

$t = 63L^{-1.4}$ for separating rock classes C and D

$t = 1.6L^{-1.6}$ for separating rock classes D and E

$t = 0.04L^{-1.8}$ for separating rock classes E and F

$t = 0.001L^{-2}$ for separating rock classes F and G

Terzaghi applied his classification primarily to noncoherent materials. If the material is coherent, then the term stand-up time loses most of its significance. Terzaghi indicated that for a noncoherent material, the stand-up time was inversely proportional to its active span and that the stand-up time for a square opening with a side (L) was in the order of 50 percent longer than for a strip of (L) width.[15] For an elongated opening, the relationship between stand-up time and active span is

$$t = \frac{T}{L}$$

where T is the stand-up time for a one-foot wide strip.

The following classifications can be given using the Tunnelmans' Ground Classification terms for soils or altered (weak) rock:

For firm material, R is greater than 30 hours.
For slowly raveling material, R is between 100 minutes and 30 hours.
For fast raveling material, R is between 7 and 100 minutes.
For cohesive running material, R is between 30 seconds and 7 minutes.
For running or flowing ground, R is less than 30 seconds.

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[15] Deere, D.U., R.B. Pack, J.E. Monsees, and B. Schmidt. Design of Tunnel Liners and Support Systems. Report for U.S. Dept. of Transportation, OHSGT, Contract 3-0152. Univ. of Ill. February, 1969.

Terzaghi's classifications can also be shown as a graph which would be very similar to Lauffer's, but having steeper division lines between classes.

SETTLEMENT OF THE GROUND SURFACE

Settlement of the ground surface over a tunnel will usually result from two main causes; consolidation as a result of water table lowering and plastic flow associated with ground movement into the tunnel. Consolidation of the soil material above the tunnel will result if compresion of the overburden is allowed. This may occur if water is removed from within the overburden because it allows the weight of the upper portion of the overburden to compress the lower portion from which the water has been removed. Where initially the hydrostatic pressure of the water helped to support the weight of the overlying material, once the water is removed then, if the ground structure is too weak to support the overlying material by itself, consolidation and thus settlement will occur. The amount of settlement that will result can be estimated from[16]

$$y = Y\left(\frac{\Delta e}{1-e}\right)$$

where y is the settlement resulting from the water table being lowered,

Y is the distance the water table is lowered,

e is the original void ratio of the ground in the layer from which the water is removed, and

Δe is the change in the void ratio from the original void ratio.

The settlement that must most often be anticipated with a tunnel, however, is plastic flow and it is usually encountered by a shield driven tunnel. This settlement can be associated with the following main causes:[17]

1. Ground movement toward the working face.
2. Inflow of material with groundwater entering the tunnel at unprotected locations.

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[16] Hough, B. K. Basic Soils Engineering. Ronald Press, New York, 1957.

[17] Hansmire, William H. and Edward J. Cording. "Performance of a Soft Ground Tunnel on the Washington Metro." Proceedings of North American Rapid Excavation and Tunneling Conference (Chicago, 1972). AIME, New York. 1972. pp 371-389.

3. Inadequate filling of voids behind the shield and outside the liner.
4. Incomplete expansion of the liner.
5. Liner deflection.

These ground movements may be accentuated by yawing and diving or nosing of the shield as well as by the necessity for negotiating curves. The volume of ground loss resulting from these various causes can be calculated knowing the geometry of the shield, its operating characteristics, the lining geometry and deflection characteristics, and the construction procedures followed. This ground loss will usually, but not always appear as a settlement of the ground surface. The volume of settlement may be less than the ground loss into the tunnel because of expansion of the ground. The ground loss volume can be roughly approximated as being about one percent of the tunnel volume for good conditions and workmanship.[18] The literature indicates that the shape of the surface settlement volume will approximate a Gaussian (or normal) probability curve with the width of the depression zone being about equal to the tunnel width plus three times the cover thickness and tunnel height.

GROUND VIBRATION

Ground vibration resulting from blasting can, if excessive, cause damage to other structures in the area. From the various investigations on this phenomenon reported in the literature, it has been concluded that if any ground vibration near a structure has a peak particle velocity greater than two inches per second, then there is a fair chance that damage may occur to the structure. This limit has been generally accepted as the safe vibration criteria and is so stated in many codes. The data collected from the many tests conducted for these investigations result in a general propagation equation which is universally accepted. This equation is of the form

$$v = \gamma \left(\frac{D}{W^{\alpha}}\right)^{-\beta}$$

where v is the peak particle velocity in inches per second,
D is the distance from source to detector in feet,
W is explosive weight in pounds per delay, and

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[18] Peck, R. B., A. V. Henron, Jr., and B. Mohraz. "State of the Art of Soft-Ground Tunneling." Proceedings of North American Rapid Excavation and Tunneling Conference (Chicago, 1972). AIME, New York. 1972. pp. 259-286.

γ, α, and β are constants for a particular site which depend upon the ground conditions at the site and thus must be determined for each site. Usually the exponent (α) is taken to have a value of 1/2 although there are indications in the literature that for some cases it is 1/3. The exponent (β) is the slope of the regression line when particle velocity is plotted versus the distance between source and detector on a log graph.

Section 3

Subsurface Exploration Systems Synthesis

In this portion of the study, the preexcavation subsurface investigation systems in current use for highway tunnel design and construction were examined and analyzed, then possible improved systems containing both proven methods and partially developed techniques were formulated to provide stimulus for improvement of this difficult task. Our approach to the synthesis of new investigation systems followed a three-step path to conclusion:

1. Analysis of currently used systems to determine areas of inadequacy.
2. Subdivision of the total subsurface investigation systems showing the present status of all its elements.
3. Synthesis of new systems having potential for improvement over existing systems.

CURRENTLY USED SUBSURFACE INVESTIGATION SYSTEMS

A necessary first step toward the development of improved subsurface investigation systems is to determine and analyze those systems currently in use.

A review of subsurface exploration systems applied in the U. S. during the past 15 years in preexcavation investigations of highway tunnel routes shows that no standard approach has been adopted, but rather a wide variety of systems has been used. Differences in exploration systems are due primarily to differing geologic environments, variations in complexity of geology, state of the information available at the beginning, and personal preferences and habits of the investigators, as well as time and budget constraints imposed by management.

The only procedures found to be common to all highway tunnel investigations are the preliminary information search and preliminary site inspection. The follow-up, more detailed, portions of the investigation diverge into various patterns. Almost all systems utilize vertical exploratory boreholes to some extent, but the use of surface geophysics, borehole geophysical logging and other in situ tests, laboratory tests, and pilot tunnels differ markedly from project to project. During the past 10 years an increasing acceptance and use of surface geophysical methods has been noted.

On the following pages, brief descriptions of representative exploration case studies are presented for five highway tunnels scattered throughout the U. S.

CARLIN CANYON TUNNELS[19,20]

LOCATION	- Elko County, Nevada
NUMBER OF TUNNELS	- Twin bores, driven 1972-1973
SEPARATION	- 50 feet
LENGTH	- 1360 feet and 1430 feet
SIZE	- 41 feet wide by 31 feet high
SHAPE	- Horseshoe
OBSTRUCTION	- Hill
MAXIMUM THICKNESS OF COVER	- 460 feet
GENERAL GEOLOGY	- Jointed fractured sedimentary rocks including limestone, shale, chert pebble conglomerate, sandstone, siltstone, and clay shale. Formations strike roughly 45° from the tunnel axis and dip 50° to 80°. Interior rock was predicted to be moderately blocky and jointed and that in portal areas to be blocky, fractured and seamy.
ENVIRONMENT	- Rural
EXCAVATION METHOD	- Drill and blast

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[19] Clair A. Hill & Associates. Geologic and Foundation Report for Carlin Canyon Tunnels near Carlin, Elko County, Nevada. Undated.

[20] State of Nevada, Department of Highways. Plan and Profile of Proposed State Highway, Lander-Eureka County Line to 1 Mile East Junction U.S. 93 (Wells), Eureka and Elko Counties. 1972.

EXPLORATION SEQUENCE

A preexcavation investigation conducted by the consulting firm of Clair A. Hill & Associates, March to April 1964, consisted of:

1. Preliminary geologic information search.

2. Preliminary site inspections.

3. Field work (sequence not determined).

   a. Eight boreholes were drilled in portal areas. Split spoon samples were taken in unconsolidated sediment and diamond cores cut from bedrock. Samples were examined, logged and boxed for bidder inspection.

   b. Reconnaissance geologic mapping was done in the vicinity.

   c. Detailed geologic mapping was done in a Southern Pacific railroad tunnel located approximately 250 feet south of, and roughly parallel to, the closest proposed highway tunnel.

   d. Geology mapped in the railroad tunnel was projected to the proposed highway bores.

   e. A refraction seismic survey was conducted at portal areas to determine the depth to bedrock, to assist in determination of safe allowable design loads for tunnel footings, and to allow forecasting of expected rock loads. Seven one-spread traverses using a total of 28 shot points were completed.

EXPLORATION COSTS - Not available

CODY TUNNELS[21]

LOCATION - Park County, Wyoming

NUMBER OF TUNNELS - No. 1 - 3224 feet, No. 2 - 150 feet, No. 3 - 150 feet

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[21] Sherman, William F. "Engineering Geology of Cody Highway Tunnels, Park County, Wyoming." Geological Society of American, Engineering Geology Case Histories No. 4. 1963. pp. 27-32.

SIZE - 32 feet wide (?) by 23 feet high (?)

SHAPE - Horseshoe

OBSTRUCTION - Hill

MAXIMUM THICKNESS OF COVER - Not available

GENERAL GEOLOGY - Moderately jointed and faulted Precambrian rocks comprised of granodiorite and minor pegmatites and hornblende-mica schist.

ENVIRONMENT - Rural

EXCAVATION METHOD - Drill and Blast

EXPLORATION SEQUENCE

1. Preliminary geologic information search.

2. Preliminary site inspections.

3. Field work.

   a. An 8-foot by 8-foot pilot tunnel was driven to explore the tunnel No. 1 route.

   b. The pilot tunnel was mapped geologically.

APPROXIMATE EXPLORATION COSTS (Partial)

PILOT TUNNEL - $263,000

COLLIER TUNNEL[22]

LOCATION - Del Norte County, California

NUMBER OF TUNNELS - One, driven 1961 to 1963

LENGTH - 1800 feet

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[22]State of California, Division of Highways, a 4-page internal report and two maps dated 1960.

SIZE - 34.5 feet wide by 38 feet high

SHAPE - Horseshoe

OBSTRUCTION - Hill

MAXIMUM THICKNESS OF COVER - 430 feet (approximately)

GENERAL GEOLOGY - Highly folded, partly metamorphased sedimentary rocks. Rocks include thinly bedded slaty shale, siltstone, and sandstone striking subparallel to the tunnel axis and dipping 45° to 90°, averaging 75°. The rock was classified as very blocky and seamy.

ENVIRONMENT - Rural

EXCAVATION METHOD - Drill and blast

EXPLORATION SEQUENCE

The preexcavation investigation conducted by the Engineering Geology Section of the Bridge Department of the Division of Highways, State of California during the periods August to December, 1959 and February, 1960 consisted of:

1. Preliminary geologic information search.

2. Preliminary site inspections.

3. Field Work.

    a. Examination of visible rock outcrops in the vicinity of the proposed tunnel.

    b. Rotary borings and soil tube tests. Six holes were drilled.

EXPLORATION COSTS - Not available

EAST RIVER MOUNTAIN TUNNELS[23, 24, 25, 26]

LOCATION	- Bland County, Virginia and Mercer County, West Virginia
NUMBER OF TUNNELS	- Twin bores, driven 1970 to 1973
SEPARATION	- 35+ feet
SIZE	- 34 feet wide by 34 feet high
SHAPE	- Horseshoe
OBSTRUCTION	- Hill
MAXIMUM THICKNESS OF COVER	- 1000+ feet
GENERAL GEOLOGY	- The tunnel was driven nearly normal to the strike through a tilted sedimentary rock sequence which forms the northwest limb of a northeast trending syncline. Rocks penetrated include predominantly limestone along with significant thicknesses of sandstone, quartzite, quartzitic sandstone, siltstone, and shale. Formational dip averages 28° southeast through much of the tunnel, with increases to near vertical near the north end.

The principal problem area was the south portal where incompetent and wet, highly weathered sandstone and shale on the dip slope

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[23]Baker, Michael, Jr., Inc. East River Mountain Tunnels, Portal Location Study, South Portal Area, Mercer County, West Virginia, Bland County, Virginia. January, 1967.

[24]Baker, Michael, Jr., Inc. East River Mountain Tunnels, Final Geologic Report, Mercer County, West Virginia, Bland County, Virginia. January, 1967.

[25]Murphy, Eugene G. "East River Mountain Highway Tunnel in U. S. A." Tunnels and Tunneling, Vol 2 No. 6 (November-December, 1970) pp. 367-368.

[26]Smith, James D. Private Communication. October, 1973.

were costly to penetrate. Some solution cavities encountered in limestone required grouting and extra attention, but did not present major difficulties.

ENVIRONMENT	- Rural
EXCAVATION METHOD	- Drill and blast, except for the soft ground methods required at the south portals.

EXPLORATION SEQUENCE

Preexcavation investigations were carried out by the consulting firm of Michael Baker, Jr., Inc. during the period 1964 to 1966.

1. Preliminary geologic information search.

2. Preliminary site inspections.

3. Field work.

    a. Surface geologic mapping.

    b. Preliminary NX core drilling program was carried out and consisted of 14 vertical holes and one angle hole to depths of 120 feet to 800 feet. Holes were drilled along proposed tunnel alignment with greatest concentration placed in portal areas.

    c. Final core drilling program consisted of 10 vertical holes and one angle hole to depths of 110 feet to 451 feet. Of these, five were NX size and six were 6-inch diameter. Heaviest concentration was in the problem south portal area.

    d. Laboratory tests were run on core samples obtained from the 6-inch holes. Consolidated-undrained triaxial compression tests and classification tests were performed on most holes. For one hole; core gradation, Atterberg Limits, shearing resistance, and unconfined compression tests were used.

APPROXIMATE EXPLORATION COSTS (Partial)

PRELIMINARY CORE DRILLING	- $35,000
FINAL CORE DRILLING	55,000
TOTAL	$90,000

STRAIGHT CREEK TUNNEL[27 through 32]

LOCATION	- Colorado
NUMBER OF TUNNELS	- Twin bores, one completed 1973, second planned.
SEPARATION	- To be 120 feet at portals, increasing to 250 feet near mid-point.
LENGTH	- 8380 feet
SIZE	- 45 feet by 45 feet
SHAPE	- Horseshoe
OBSTRUCTION	- Mountain range
MAXIMUM THICKNESS OF COVER	- 1500 feet (approximately)
GENERAL GEOLOGY	- Extensively faulted and locally altered crystalline rock consisting of 75 percent Precambrian granite, 24 percent Precambrian gneiss, and 1 percent Tertiary (?) diorite dikes.

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[27] "Pilot Bore is Laboratory for Twin Road Tunnels." Engineering News-Record. August 13, 1964. pp. 38-39.

[28] Robinson, C. S. and F. T. Lee. "Geologic Research at the Straight Creek Tunnel Site, Colorado." Highway Research Record No. 57. 1964. pp. 18-34.

[29] Hurr, Theodore R. and David B. Richards. "Ground Water Engineering of the Straight Creek Tunnel (Pilot Bore), Colorado." Engineering Geology, Vol 3 No. 2 (July, 1966), pp. 80-90.

[30] Robinson, C. S. and F. T. Lee. "Results of Geologic Research at the Straight Creek Tunnel (Pilot Bore), Colorado." Highway Research Record No. 185. 1967. pp. 9-19.

[31] Scott, James H. and Roderick D. Carroll. "Surface and Underground Geophysical Studies at Straight Creek Tunnel Site, Colorado." Highway Research Record No. 185. 1967. pp. 20-35

[32] Hopper, R. C., T. A. Lang and A. A. Matthews. "Construction of Straight Creek Tunnel, Colorado." Proceedings of North American Rapid Excavation and Tunneling Conference (Chicago, 1972). 1972. pp. 501-538.

ENVIRONMENT - Rural

EXCAVATION METHOD - Drill and blast

EXPLORATION SEQUENCE

1. Late 1930's - Early planning for a vehicular tunnel through the Continental Divide was begun.

2. Nov. 1941 - May 1943 - An exploratory tunnel was driven 5483 feet. This site was subsequently abandoned.

3. After World War II - Interest revived and several widely spaced routes through the Continental Divide were considered.

4. 1955 - Four vertical core holes were drilled along the final selected route which is some 1.5 miles from the original pilot tunnel.

5. 1960 - A location study showed that an 11-mile road distance savings would result in using the site selected.

6. Summer, 1962 - U.S. Gelogical Survey mapped surface geology over six square miles along tunnel route at 1:12,000 scale with particular attention paid to percentage of rock types and attitudes of foliation, faults, and joints.

7. 1962 - 1966 - Two additional core holes were drilled.

8. 1962 - 1966 - Drill holes were logged geophysically, including resistivity, gamma ray, and gamma ray density.

9. 1962 - 1966 - A seismic survey was carried out by the USGS along the tunnel alignment and on right angle traverses to determine seismic velocities.

10. 1962 - 1966 - Another USGS seismic survey was carried out at the east protal area to determine thickness of surface material.

11. 1962 - 1966 - Borehole seismic methods were used to determine velocity layering.

12. 1962 - 1966 - A surface electrical resistivity survey was performed by the USGS along part of the tunnel route.

13. 1962 - 1966 - Laboratory testing of surface and drill hole samples supervised by the USGS included the mineralogy, porosity, grain density, dry bulk density,

saturated bulk density, determination of elastic properties by static and dynamic methods, triaxial compressive and shear strength tests, and the measurement of swelling properties of fault gouge clay.

14. 1963 - 1964 - A pilot tunnel, approximately 11 feet by 11 feet was driven 8284 feet.

15. 1963 - 1964 - Geophysical investigations including resistivity and seismic velocity measurements were made in the pilot tunnel.

16. 1963 - 1964 - The pilot tunnel was mapped geologically for rock types, degree of alteration, and attitudes of foliation, faults, shear zones, and joints. Walls were mapped at 1:600 scale throughout and at 1:60 scale along sections tested geophysically. Geologic maps of the tunnel face were prepared at 800 stations. Points of groundwater inflow were noted.

17. 1963 - 1964 - Pilot tunnel wallrock was sampled for various laboratory tests. Systematic samples were taken for petrographic analysis. Samples of altered wallrock and gouge were obtained for mineralogic and swelling pressure tests. Linear chip samples were cut from 5 feet on either side of rock mechanics instrumentation stations for grain size and mineral analysis. Blocks of up to one-foot size were taken for elastic property tests.

18. 1963 - 1964 - Systematic rock mechanics instrumentation was installed by a contract organization at 41 stations on 200-foot intervals. Instruments used were load cells, multiple position borehole extensometers, and bar extensometers which yielded information on rock mass behavior and development of rock loads on supports.

19. 1963 - 1963 - Pilot tunnel groundwater studies included recording of flow rates at the portal, chemical tests for mineral content, specific conductance measurements at many points, and monitoring of the water level in a surface drill hole.

APPROXIMATE EXPLORATION COSTS

GEOLOGIC STUDIES $ 100,000

CORE DRILLING	103,000
ROCK MECHANICS	59,000
PILOT TUNNEL	1,422,000
TOTAL	$1,684,000

ANALYSIS OF PRESENT SYSTEMS

Because of the limited extent of this study, we were unable to determine if there were specific inadequacies in the particular investigation systems used. A lack of sufficient available information also prevented us from being able to suggest possible alternative systems which would have furnished the same or more information as was furnished by the systems used, but possibly at less cost. However, some generalities can be made from the data we did obtain and examine.

A high percentage of highway tunnels are completed with significant cost overruns, much of which can be traced to inadequate preexcavation exploration or to faulty interpretation of exploration data. In retrospect, it appears that most of those tunnel projects which experienced overruns due to encounters with unexpected adverse conditions could have benefited from the application of more systematic and thorough exploratory programs. Undoubtedly, in some cases the individuals directing the exploration realized a need for additional work, but lacked funds and/or time due to conditions fixed by management.

In some other projects, sufficient information was apparently available from the site exploration program, but improper interpretation and/or extrapolation of this data was responsible for the problems that were encountered during the construction phase. There have also been cases where either management or the contractor chose, for their own reasons, not to accept the recommendations of the site investigators and as a consequence, problems were encountered during construction.

When tunneling problems occur as a result of insufficient, misinterpreted or misapplied geologic information, you enter into that problem area called, "a situation that causes perplexity or puts one in a predicament." Unfortunately for us, very few individuals connected with a tunneling project can agree as to the specific reasons for an unexpected problem occurring. This is apparently a very sensitive area and we found it difficult to obtain documentation on particular projects which would give us an indication of the specific deficiency which allows a tunneling problem to occur unexpectedly.

A review of difficulties encountered is some of the case study tunnels, and in numerous others examined more briefly, has led us to the conclusion that the most frequent, and usually the most costly problems are those of ground stability within the tunnel. In addition, these problems sometimes also cause detrimental surface subsidence, mainly where shallow soft ground tunnels are involved.

The second-ranking principal problem area appears to be that of excessive groundwater inflows. Adverse geologic surprises affecting the excavation method are less common and less costly than the foregoing mentioned areas. Thus, ground stability and hydrologic conditions emerge as those features most difficult to evaluate with existing subsurface investigation technology.

The value analysis model described elsewhere in this report can serve not only as a guide to optimization of a subsurface investigation but also can furnish the exploration manager with an effective tool for convincing management of the risks being faced if thorough exploration is not allowed.

An attempt was made in this study to analyze the various case studies obtained by application of the value analysis model developed. Our effort was not fruitful, however, because more specific information than we were able to obtain is needed to develop meaningful results. The individual, or group of individuals, intimately connected with the exploration program for a particular tunnel project would be able to obtain worthwhile results with the value analysis model, although its primary purpose is for planning new site investigations rather than for post mortem evaluation of past projects.

SYSTEM SUBDIVISION

For analysis and comparison of existing systems and to synthesize potentially feasible and improved new systems, the total investigation procedure was broken down into subsystems corresponding to their principal objectives. Subsystems were selected by grouping the six major areas of tunnel design considerations shown earlier on Figure 4 into three main objectives:

1. Investigation For Excavation Method (from choice of excavation techniques).

2. Investigation of Ground Stability (a combination of the determinations of support and lining requirements).

3. Investigation of Hydrologic Conditions (from evaluation of potential water problems).

The other two areas of tunnel design considerations shown on Figure 4, evaluation of safety hazards and evaluation of possible damage to other man-made structures, are related to all of the above three major subsystems, but are not in themselves major targets of exploration.

The three selected subsystems, while covering quite different basic areas of investigation, are nevertheless all interrelated. For example, the primary cause of a ground stability problem may be the weak rock strength due to close fracture spacing in a shear zone, but this problem may be considerably worsened by the effects of a high water inflow.

Components of the subsystems are the varied subsurface exploration techniques identified, described, and analyzed elsewhere in this report. Full descriptions of these methods are included in the appendixes and condensed short-form versions were presented in Section 2. The entire list of techniques is included in Figure 12 which is a matrix showing the subdivision of the exploration system and status of the present and possible future applicability of each technique with respect to each of the three major subsystem objectives. This correlation was then screened to remove those techniques which have little present or predicted future application for highway tunnel site investigations. Those techniques which remain from this screening - those presently useful and those which appear to offer promise of becoming more useful in the future are presented in Figure 13.

SYNTHESIS OF POTENTIALLY IMPROVED SYSTEMS

COMPONENT AREAS HAVING GREATEST IMPROVEMENT POSSIBILITIES

In addition to the well proven and highly used investigation methods, those incompletely developed and underused techniques also shown on Figure 13 are worthy of further attention as some of them could be of substantial benefit if properly applied.

A study of problems encountered in tunnels revealed that the greatest and most common difficulties are found in the ground stability subsystem, followed by the hydrologic conditions subsystem and that problems in the excavation method subsystem are of a lesser overall magnitude. Therefore, it appears that the greatest opportunity for improvement of tunnel site investigation systems would be to improve existing methods and develop new methods to better understand ground stability and hydrologic conditions.

COMPONENTS (SUBSURFACE EXPLORATION TECHNIQUES)				INVESTIGATION FOR EXCAVATION METHOO	INVESTIGATION OF GROUNO STABILITY	INVESTIGATION OF HYOROLOGIC CONOITIONS
LABORATORY TESTING	ROCK		CREEP		X	
LABORATORY TESTING	ROCK		PERMEABILITY			●
LABORATORY TESTING	ROCK		OYNAMIC ELASTIC CONSTANTS		●	
LABORATORY TESTING	ROCK		STATIC ELASTIC CONSTANTS		●	
LABORATORY TESTING	ROCK	TRI-AXIAL	SHEAR STRENGTH		X	
LABORATORY TESTING	ROCK	TRI-AXIAL	COMPRESSIVE STRENGTH		X	
LABORATORY TESTING	ROCK	UNIAXIAL	FLEXURAL STRENGTH		●	
LABORATORY TESTING	ROCK	UNIAXIAL	TENSILE STRENGTH		X	
LABORATORY TESTING	ROCK	UNIAXIAL	SHEAR STRENGTH		X	
LABORATORY TESTING	ROCK	UNIAXIAL	COMPRESSIVE STRENGTH	●	●	
LABORATORY TESTING	ROCK		WEATHERING RESISTANCE		●	
LABORATORY TESTING	ROCK		ABRASION RESISTANCE	●		
LABORATORY TESTING	ROCK		HARDNESS	●		
LABORATORY TESTING	ROCK		POROSITY			●
LABORATORY TESTING	ROCK		SPECIFIC GRAVITY		●	
LABORATORY TESTING	ROCK		IDENTIFICATION	●	●	●
LABORATORY TESTING	SOIL		SWELLING		○	
LABORATORY TESTING	SOIL		PERMEABILITY			●
LABORATORY TESTING	SOIL		UNCONFINEO COMP. STR.	X		
LABORATORY TESTING	SOIL		SHEARING STRENGTH		●	
LABORATORY TESTING	SOIL		CONSISTENCY	●	●	
LABORATORY TESTING	SOIL		POROSITY			●
LABORATORY TESTING	SOIL		UNIT WEIGHT		●	
LABORATORY TESTING	SOIL		MOISTURE CONTENT			●
LABORATORY TESTING	SOIL		GRAOATION	●	●	
IN SITU TESTING			IN SITU PERMEABILITY		○	○
IN SITU TESTING			STATE OF STRESS	○	○	
IN SITU TESTING			SOIL SHEARING STRENGTH		○	
			TRENCHES & PILOT TUNNELS	●	●	●
BOREHOLE LOGGING			TEMPERATURE			
BOREHOLE LOGGING			CALIPER	□	□	
BOREHOLE LOGGING			GRAVITATIONAL			
BOREHOLE LOGGING			SONIC	○	○	X
BOREHOLE LOGGING	NUCLEAR LOGS		FORMATION DENSITY	□	□	X
BOREHOLE LOGGING	NUCLEAR LOGS		NEUTRON	X	X	X
BOREHOLE LOGGING	NUCLEAR LOGS		GAMMA RAY	X	X	X
BOREHOLE LOGGING	ELECTRICAL LOGS		INOUCTION	□	□	X
BOREHOLE LOGGING	ELECTRICAL LOGS		FOCUSING ELECTRODE	□	□	X
BOREHOLE LOGGING	ELECTRICAL LOGS		MICRORESISTIVITY	□	□	X
BOREHOLE LOGGING	ELECTRICAL LOGS		CONVENTIONAL RESISTIVITY	□	□	X
BOREHOLE LOGGING	ELECTRICAL LOGS		SPONTANEOUS POTENTIAL	X	X	X
BOREHOLE LOGGING			VISUAL OR PHOTOGRAPHIC	□	□	X
SAMPLING	SUBSURFACE	SAMPLING, UNDIST.	CORING	●	●	●
SAMPLING	SUBSURFACE	SAMPLING, UNDIST.	PISTON TUBE	●	●	●
SAMPLING	SUBSURFACE	SAMPLING, UNDIST.	OPEN TUBE	●	●	●
SAMPLING	SUBSURFACE	SAMPLING	OISTURBEO SAMPLING	●	●	●
SAMPLING	SUBSURFACE	DRILLING, VERTICAL	WASH BORING	X	X	X
SAMPLING	SUBSURFACE	DRILLING, VERTICAL	AUGER	X	X	X
SAMPLING	SUBSURFACE	DRILLING, VERTICAL	CHURN OR PERCUSSION	X	X	X
SAMPLING	SUBSURFACE	DRILLING, VERTICAL	ROTARY	●	●	●
SAMPLING	SUBSURFACE	DRILLING	HORIZONTAL	○	○	○
SAMPLING			SURFACE SAMPLING	●	●	●
GEOPHYSICAL SURVEYS			ACQUSTIC HOLOGRAPHY	○	○	□
GEOPHYSICAL SURVEYS			RAOIOMETRIC	X	X	X
GEOPHYSICAL SURVEYS			GRAVITY	X	X	X
GEOPHYSICAL SURVEYS			ELECTROMAGNETIC	X	X	X
GEOPHYSICAL SURVEYS			MAGNETIC	X	X	X
GEOPHYSICAL SURVEYS			ELECTRICAL RESISTIVITY	●	●	●
GEOPHYSICAL SURVEYS			SEISMIC REFRACTION	●	●	●
GEOPHYSICAL SURVEYS			SEISMIC REFRACTION	●	●	●
GEOLOGICAL MAPPING			OIRECT SURFACE MAPPING	●	●	●
GEOLOGICAL MAPPING	REMOTE SENSING		PASSIVE MICROWAVE RAOIOMETRY			
GEOLOGICAL MAPPING	REMOTE SENSING		INFRAREO RAOIOMETRY	○	○	○
GEOLOGICAL MAPPING	REMOTE SENSING		LUMINESCENCE			
GEOLOGICAL MAPPING	REMOTE SENSING		ULTRAVIOLET			
GEOLOGICAL MAPPING	REMOTE SENSING		PASSIVE MICROWAVE			
GEOLOGICAL MAPPING	REMOTE SENSING		SIDE-LOOKING AIRBORNE RADAR	□	□	□
GEOLOGICAL MAPPING	REMOTE SENSING		MULTISPECTRAL SCANNING	○	○	○
GEOLOGICAL MAPPING	REMOTE SENSING		MULTISPECTRAL PHOTOGRAPHY	○	○	○
GEOLOGICAL MAPPING	REMOTE SENSING		THERMAL INFRARED	□	○	○
GEOLOGICAL MAPPING	REMOTE SENSING		INFRAREO PHOTOGRAPHY	□	□	□
GEOLOGICAL MAPPING	REMOTE SENSING		BLACK & WHITE PHOTOGRAPHY	●	●	●
GEOLOGICAL MAPPING	REMOTE SENSING		COLOR PHOTOGRAPHY	□	□	□
			PRELIMINARY SITE INSPECTION	●	●	●
			INFORMATION SEARCH	●	●	●
SUBSYSTEMS				INVESTIGATION FOR EXCAVATION METHOO	INVESTIGATION OF GROUNO STABILITY	INVESTIGATION OF HYOROLOGIC CONOITIONS
SYSTEM				SUBSURFACE INVESTIGATION		

PRESENT STATUS OF EXPLORATION TECHNIQUES
WITH REGARD TO TUNNEL SITE INVESTIGATION

- ● PROVEN, COMMONLY USED
- ○ PARTIALLY OEVELOPED, LITTLE USED, GOOO POTENTIAL FOR IMPROVEMENT ANO INCREASED USE
- □ PROVEN, LITTLE USED OR UNUSED, GOOD POTENTIAL FOR INCREASED USE
- X PROVEN, LITTLE USED OR UNUSED, POOR POTENTIAL FOR INCREASED USE
- BLANK NO OR VERY LIMITED APPLICATION

Figure 12. Evaluation of subsystem components for subsurface investigation system.

System	Subsystems			
		Investigation for Excavation Method	Investigation of Ground Stability	Investigation of Hydrologic Conditions
Subsurface Investigation				

Components (Subsurface Exploration Techniques)				Investigation for Excavation Method	Investigation of Ground Stability	Investigation of Hydrologic Conditions
			Information Search	●	●	●
			Preliminary Site Inspection	●	●	●
Geological Mapping	Remote Sensing		Color Photography	□	□	□
Geological Mapping	Remote Sensing		Black & White Photography	●	●	●
Geological Mapping	Remote Sensing		Infrared Photography	□	□	□
Geological Mapping	Remote Sensing		Thermal Infrared	○	○	○
Geological Mapping	Remote Sensing		Multispectral Photography	○	○	○
Geological Mapping	Remote Sensing		Multispectral Scanning	□	○	○
Geological Mapping	Remote Sensing		Side-Looking Airborne Radar	□	□	□
Geological Mapping	Remote Sensing		Infrared Radiometry	○	○	○
Geological Mapping			Direct Surface Mapping	●	●	●
Geophysical Surveys			Seismic Refraction	●	●	●
Geophysical Surveys			Seismic Reflection	●	●	●
Geophysical Surveys			Electrical Resistivity	●	●	●
Geophysical Surveys			Acoustic Holography	○	○	○
Sampling			Surface Sampling	●	●	●
Sampling	Subsurface	Drilling	Horizontal	○	○	○
Sampling	Subsurface	Drilling	Vertical, Rotary	●	●	●
Sampling	Subsurface	Sampling	Disturbed Sampling	●	●	●
Sampling	Subsurface	Sampling, Undist.	Open Tube	●	●	●
Sampling	Subsurface	Sampling, Undist.	Piston Tube	●	●	●
Sampling	Subsurface	Sampling, Undist.	Coring	●	●	●
Borehole Logging			Visual or Photographic	□	□	
Borehole Logging	Electrical Logs		Conventional Resistivity	□	□	
Borehole Logging	Electrical Logs		Microresistivity	□	□	
Borehole Logging	Electrical Logs		Focusing Electrode	□	□	
Borehole Logging	Electrical Logs		Induction	□	□	
Borehole Logging			Formation Density	□	□	
Borehole Logging			Sonic	○	○	
Borehole Logging			Caliper	□	□	
			Trenches & Pilot Tunnels	●	●	●
In Situ Testing			Soil Shearing Strength		○	
In Situ Testing			State of Stress	○	○	
In Situ Testing			In Situ Permeability		○	○
Laboratory Testing	Soil		Gradation	●	●	
Laboratory Testing	Soil		Moisture Content			●
Laboratory Testing	Soil		Unit Weight		●	
Laboratory Testing	Soil		Porosity			●
Laboratory Testing	Soil		Consistency	●	●	
Laboratory Testing	Soil		Shearing Strength		●	
Laboratory Testing	Soil		Permeability			●
Laboratory Testing	Soil		Swelling		○	
Laboratory Testing	Rock		Identification	●	●	●
Laboratory Testing	Rock		Specific Gravity		●	
Laboratory Testing	Rock		Porosity			●
Laboratory Testing	Rock		Hardness	●		
Laboratory Testing	Rock		Abrasion Resistance	●		
Laboratory Testing	Rock		Weathering Resistance		●	
Laboratory Testing	Rock		Compressive Strength	●	●	
Laboratory Testing	Rock		Flexural Strength		●	
Laboratory Testing	Rock		Static Elastic Constants		●	
Laboratory Testing	Rock		Dynamic Elastic Constants		●	
Laboratory Testing	Rock		Permeability			●

PRESENT STATUS OF EXPLORATION TECHNIQUES WITH REGARD TO TUNNEL SITE INVESTIGATION

● PROVEN, COMMONLY USED
○ PARTIALLY DEVELOPED, LITTLE USED, GOOD POTENTIAL FOR IMPROVEMENT AND INCREASED USE
□ PROVEN, LITTLE USED OR UNUSED, GOOD POTENTIAL FOR INCREASED USE

Figure 13. Subsystem components for subsurface investigation system proven and commonly used or having good potential.

Figure 13 was prepared to show that the greatest possibilities for improvement in site investigation techniques appear to exist in the following areas:

- Remote sensing reconnaissance - Improvement of existing techniques to permit better recognition of structural features and soil and rock types at surface and for development of others having greater depth penetration will be quite valuable in the areas of ground stability and hydrologic conditions. More extensive use should be made of color photography as a replacement of conventional black and white photography. Research is expected to improve the capabilities of multispectral photography and scanning, thermal infrared, and infrared radiometry. Sidelooking airborne radar imaging is quite valuable and will certainly be used more if the cost can be reduced for coverage of small areas.

- Seismic refraction and reflection, and borehole seismic techniques - Opportunities exist for increased adaptation of equipment and interpretation using computer processing from the recognized leader in this field, the petroleum exploration industry. Improvement in these techniques would enable a better understanding being made in all three major subsystem areas and in particular for ground stability and hydrologic conditions.

- Acoustic holography - Development of this technique into a standard exploration method for seismically "seeing" into the earth to detect voids and major structural discontinuities would represent a major accomplishment. Again an improvement in acoustic holography would benefit all three major subsystem areas, but particularly ground stability.

- Horizontal drilling - Very little use has been made of long horizontal exploratory holes which could be considered as miniature "pilot tunnels" if drilled along the proposed tunnel alignment. Besides returning information on ground type and conditions, horizontal holes provide some opportunity for advance remedial action by drainage or grouting of highly permeable water-bearing zones. The use of horizontal drilling becomes more cost beneficial with increased tunnel depth. Near the surface closely spaced vertical holes, perhaps coupled with seismic surveying, would provide sufficient information quicker and at lower cost. For the deeper depths, however, horizontal drilling in conjunction with borehole geophysical logging techniques, when suitably developed, could furnish better information than is obtained from any other method now in use except for pilot tunnels. Those major areas needing improvement to make horizontal long-hole drilling more practical for use in subsurface investigation are better guidance and control methods and faster penetration rates.

- Borehole logging - This is another area where the engineering geologic field has excellent opportunity for advancement through adaptation of logging methods developed for petroleum exploration and production purposes. The methods most likely able to contribute significantly to the determination of rock lithology and its engineering properties are the sonic logs (including 3-D velocity and acoustic waveform), density, electrical resistivity, and caliper logs. Tools should be developed for use in both vertical and horizontal holes. The information obtained from borehole logging is important for all three major subsystem areas.

- Pilot Tunnels - As long as other investigation techniques continue to leave strong doubts about subsurface conditions, it is expected that the use of pilot tunnels will increase because of the great financial risks of adverse geologic surprises.

- State of stress - While this is a major factor in ground stability, techniques for its determination are only partially developed and partially reliable. Continued research is expected to result in substantial improvements in this vital area. More reliable and detailed information about the in situ state-of-stress conditions is needed. Therefore, more effort should be expended in this direction.

- In situ permeability - The determination of hydrologic conditions is another difficult area where general improvement is needed. It is expected that the use of in situ permeability testing in boreholes will increase.

It should be noted that the foregoing list of areas of tunnel site investigation where substantial improvement is required, and expected, is comprised completely of field investigation techniques and that all except remote sensing are tests performed directly on the soil or rock material mass in situ.

Laboratory tests on disturbed and undisturbed soil samples and intact rock core samples have a very definite usefulness to the site investigation, but their limitations and order of importance must be recognized. Knowledge of the in situ conditions with inherent inhomogenities and discontinuities across the span of a highway tunnel are of considerably more vital importance to tunnel designers than detailed quantitative data about an intact specimen which is bound to be unrepresentative of the mass. Thus it is expected that laboratory tests will be improved and standardized to the general benefit of site investigators, but no opportunities for major breakthroughs in the field with regard to the major design requirements are presently obvious.

Components of the subsurface investigation system which will continue to be very important but which are not expected to show near future dramatic improvement or increases in use are the preliminary information search and site inspection, surface geologic mapping, electrical resistivity surveying and core drilling. Computer storage and retrieval of geologic data will facilitate the information search. Some increase of core drilling will probably result if more thorough investigations are permitted in the future.

SYNTHESIZED NEW SYSTEMS

Utilizing the exploration techniques shown on Figure 12, ten different potentially feasible subsurface investigation systems for highway tunnels have been synthesized. These sythesized systems are for different assumed combinations of location environment and generalized ground type. The location environments used were urban and rural. The generalized ground types used were soil, mixed (soil and rock), rock having a simple geologic structure, and rock having a complex geologic structure. Figure 14 shows the suggested sequence for each of the ten systems that were synthesized in this study. It should be noted that several of the components in the various synthesized systems have a present status of being only partially developed, as was shown as Figure 13. However, in our synthesizing there was no distinction made as to the present status of the individual exploration techniques.

Many subsurface investigation systems other than those shown on Figure 14 could also be synthesized for each of the condition combinations assumed. Those synthesized systems given here, however, are considered to be the ones that can furnish the greatest amount of information about the subsurface conditions under the greatest range of applications. These systems thus show the best potential for improvements over existing systems.

Application of the value analysis model developed in this study, although desirable, is not possible for evaluating the different synthesized subsurface investigation systems. This is because a specific site must be used with the model for comparing the different systems. For any particular site, there would be an optimum system, but for two or more different sites there can be a different optimum system for each.

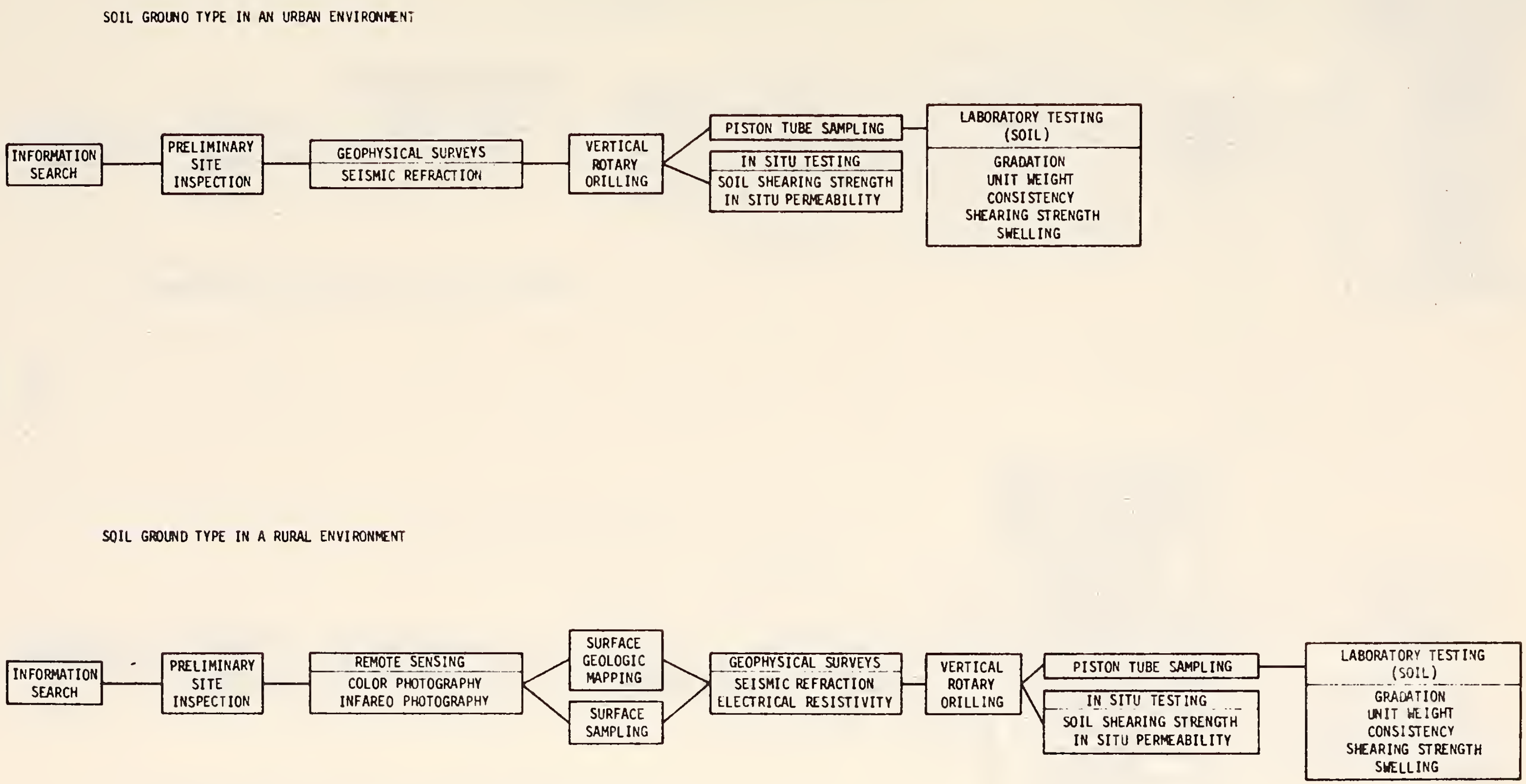


Figure 14. Synthesized new subsurface investigation systems.

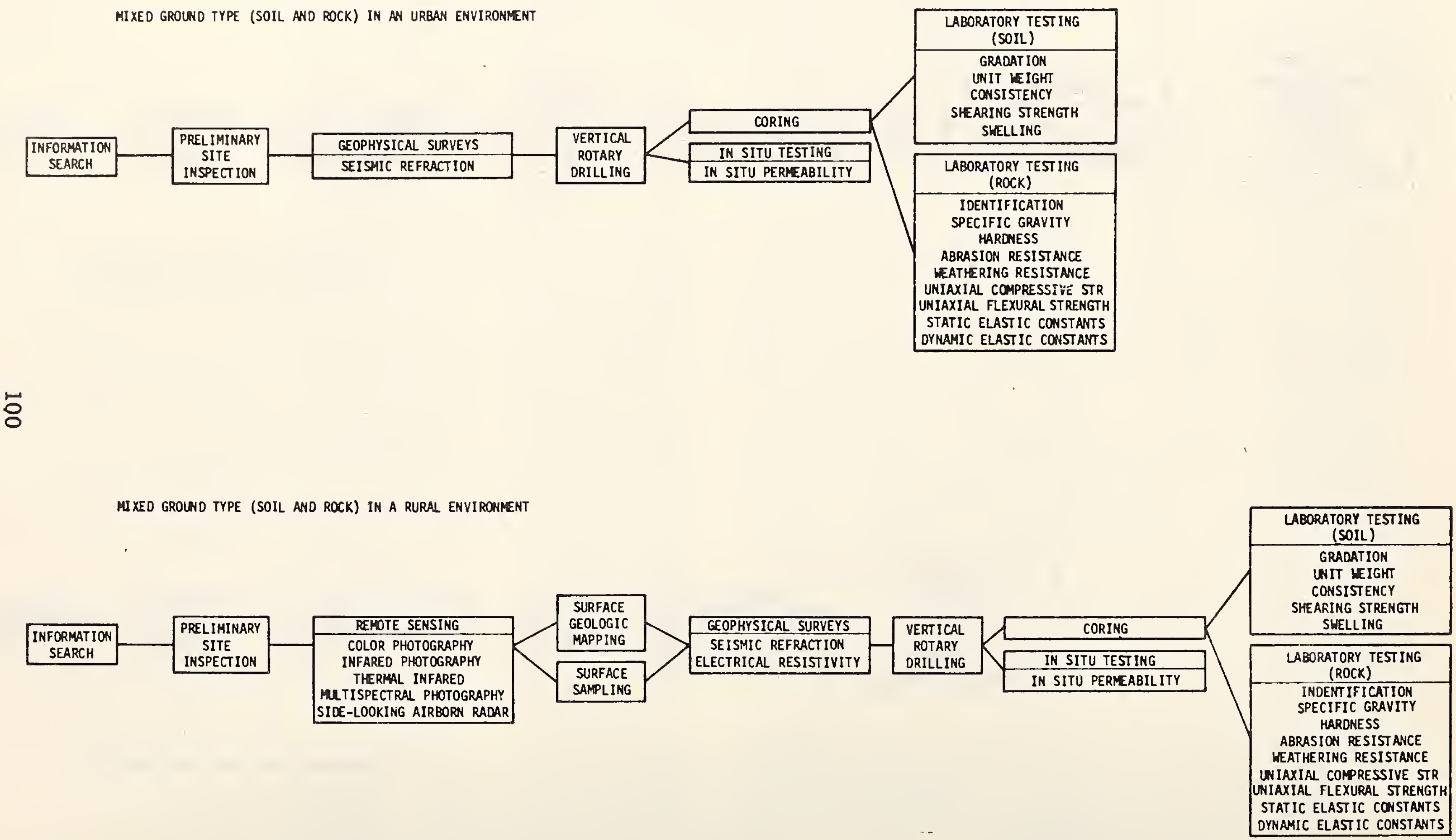


Figure 14. Synthesized new subsurface investigation systems (continued).

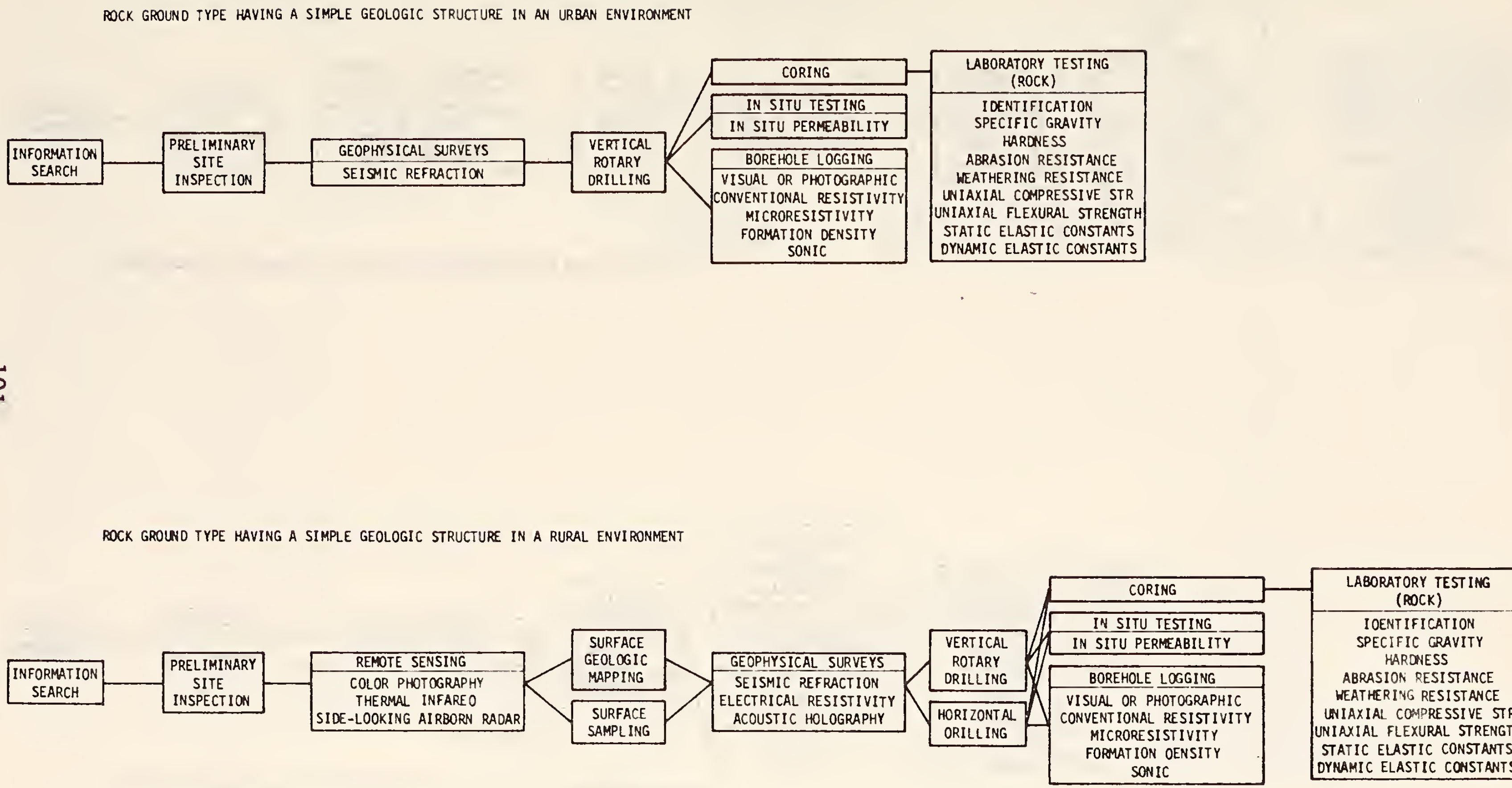
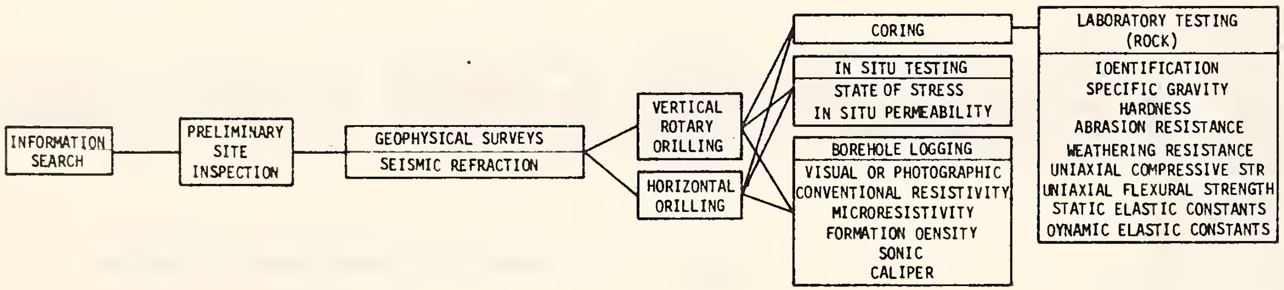


Figure 14. Synthesized new subsurface investigation systems (continued).

ROCK GROUND TYPE HAVING A COMPLEX GEOLOGIC STRUCTURE IN AN URBAN ENVIRONMENT



ROCK GROUND TYPE HAVING A COMPLEX GEOLOGIC STRUCTURE IN A RURAL ENVIRONMENT

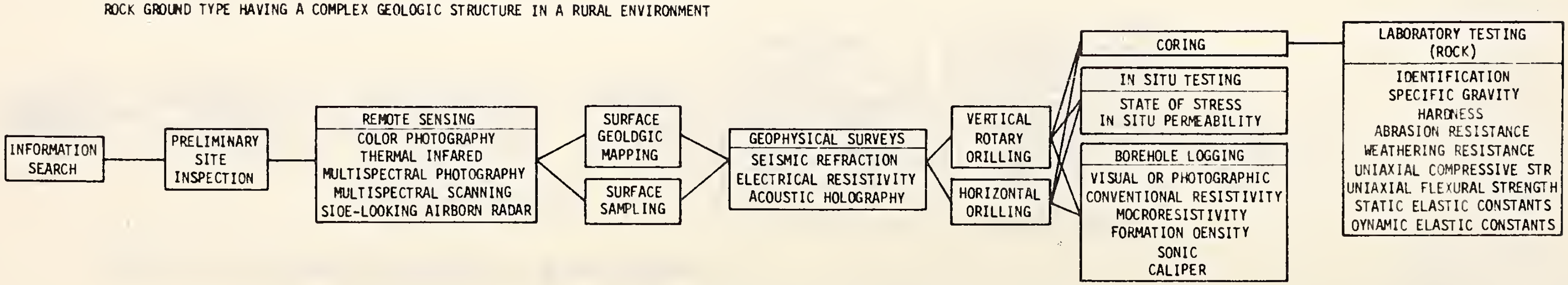
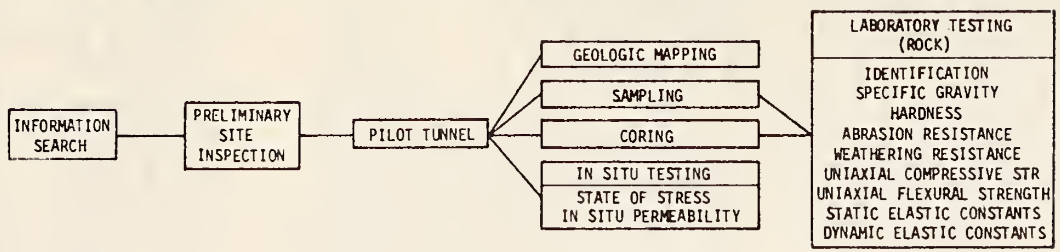


Figure 14. Synthesized new subsurface investigation systems (continued).

ROCK GROUND TYPE HAVING A COMPLEX GEOLOGIC STRUCTURE IN AN URBAN ENVIRONMENT



ROCK GROUND TYPE HAVING A COMPLEX GEOLOGIC STRUCTURE IN A RURAL ENVIRONMENT

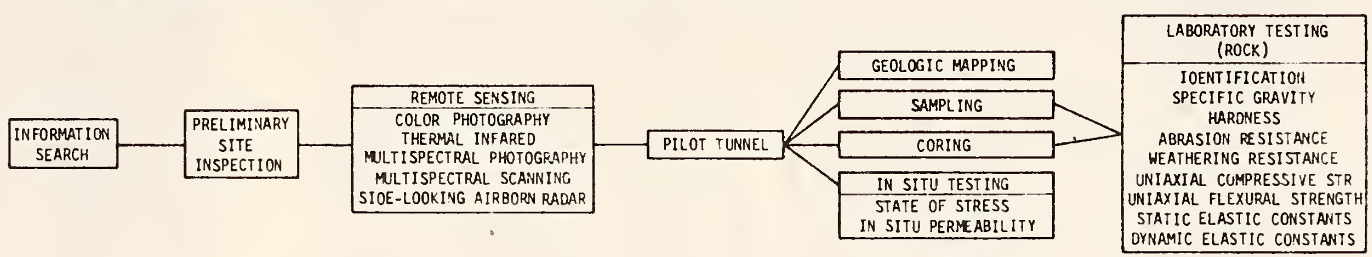


Figure 14. Synthesized new subsurface investigation systems (continued).

SECTION 4

DESIGNING AN OPTIMUM SUBSURFACE EXPLORATION SYSTEM

There is a broad range of subsurface exploration techniques from which to design a subsurface exploration system suitable for determining the subsurface conditions important to the design and construction of a highway tunnel. Before a selection of applicable techniques can be made, however, the abilities of the different subsurface exploration techniques to identify the various subsurface conditions under different environments must be known. Using the data collected earlier in this study, correlations have been made showing the ability of different subsurface exploration techniques to identify various subsurface conditions and also the ability of each technique to function in different environments. Figure 15 shows the correlation of technique ability to identify subsurface conditions and Figure 16 shows the correlation of each technique's ability to function in different environments. Using these correlations, it is possible to determine which subsurface exploration techniques can be used to obtain information about a particular subsurface condition.

A random selection from among the various exploration techniques found to be applicable from these correlations could result in a subsurface exploration system. The resulting system, however, would not necessarily be the optimum. To provide for an optimum system design, a preliminary decision procedure was developed in this study for selecting the best combination of components for a subsurface exploration system. Proper use of the procedure should result in the design of a subsurface exploration system that can provide maximum benefit for the money spent.

VALUE ANALYSIS MODEL

To select the best subsurface exploration system for a particular tunnel site, it is necessary to determine the value of the information expected to be obtained by each of the candidate exploration techniques. Following is a brief narrative of the analysis used in this study for determining the value of information expected from use of the various candidate exploration techniques.

Having an insufficient amount of knowledge about the subsurface conditions along a tunnel alignment may result in numerous problems. A problem is defined as an adverse event that is dependent upon the subsurface conditions and which occurs at some specific location unexpectedly or with a magnitude not anticipated. In some instances,

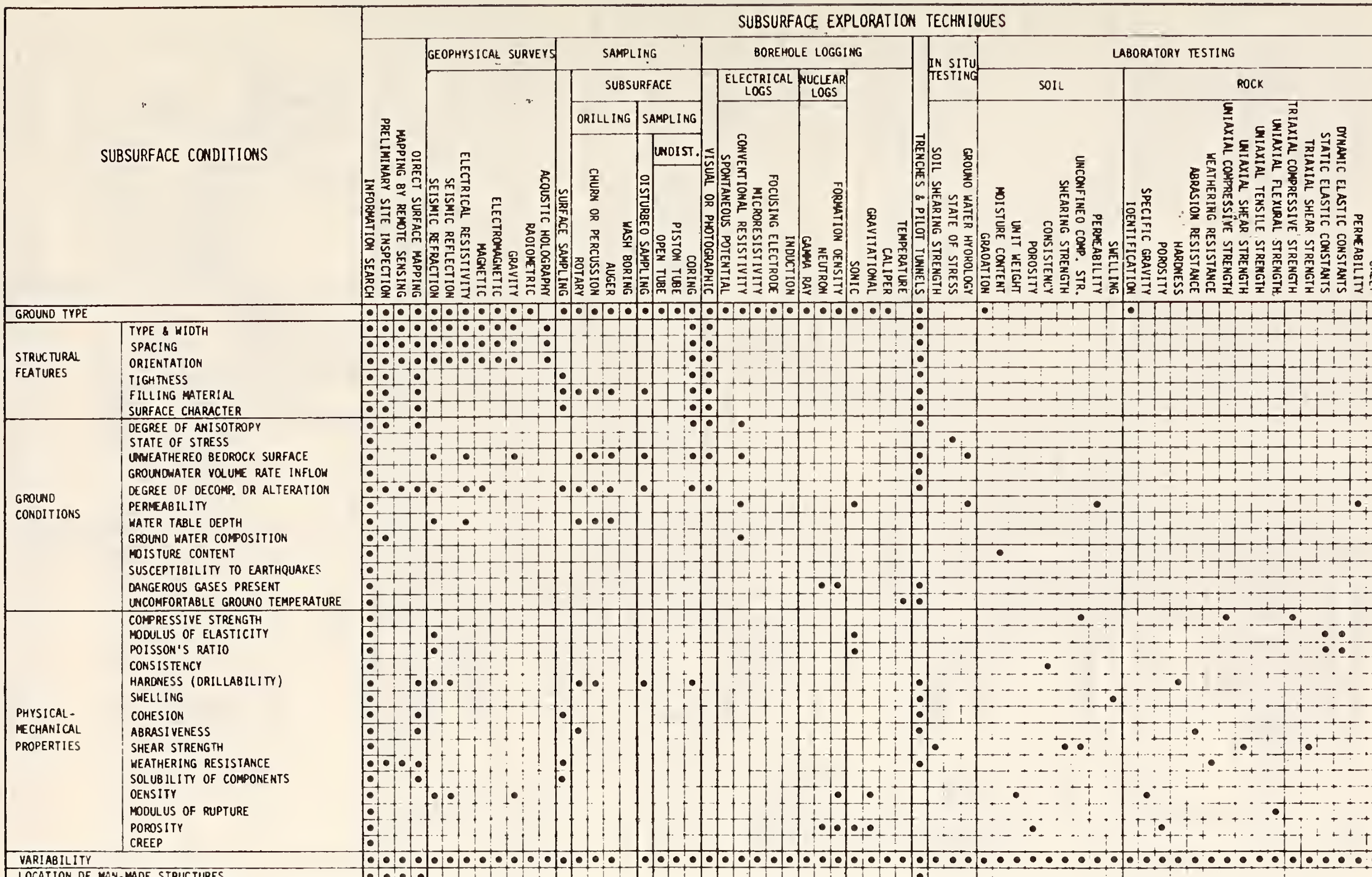


Figure 15. Subsurface conditions identifiable by subsurface exploration techniques.

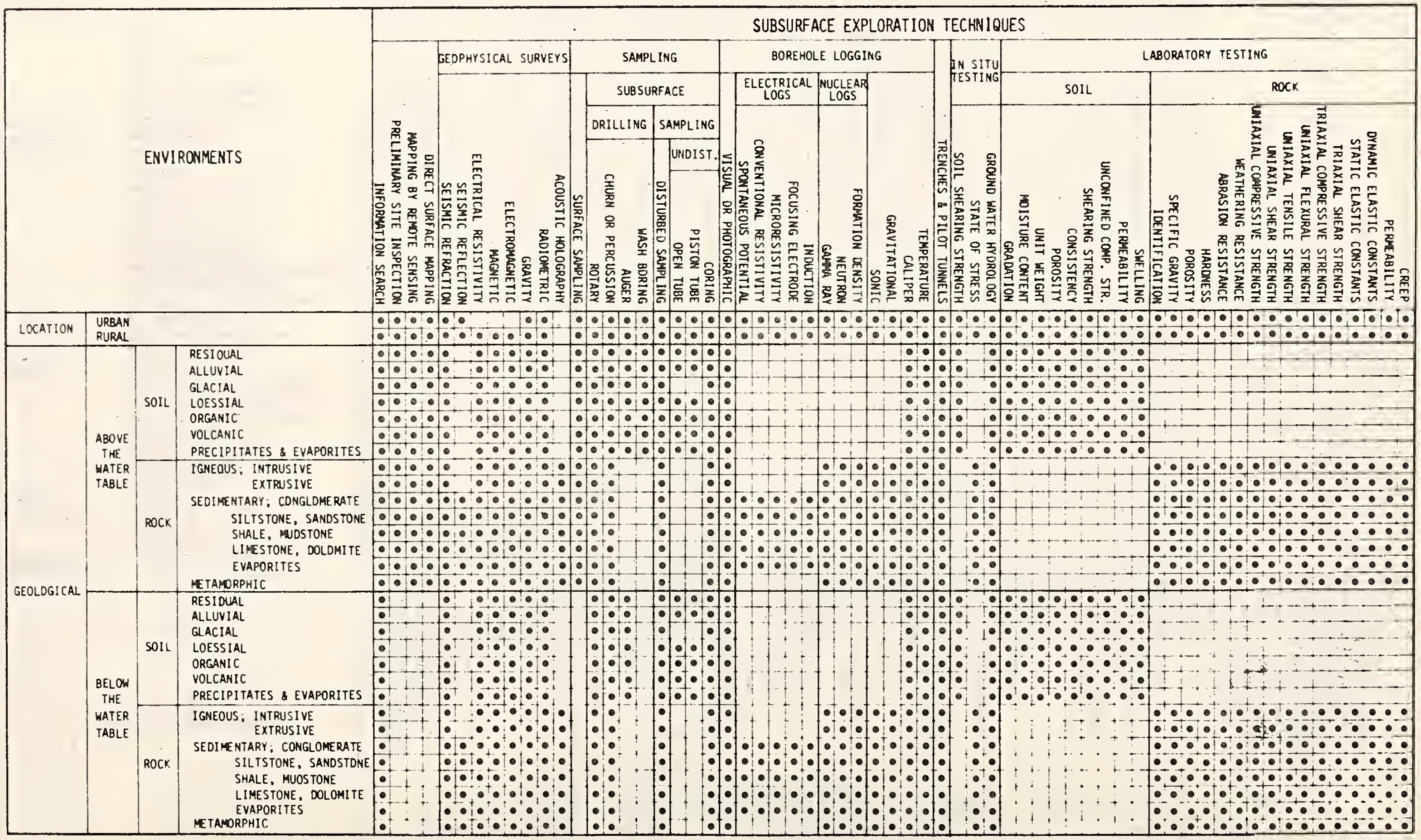


Figure 16. Applicable environments for subsurface exploration techniques.

several problems may result from a single combination of subsurface conditions. If any problems occur, there will be some amount of cost consequence that results and which is dependent upon the severity of the problems. This cost consequence will include such items as the cost of necessary remedial action, delays, change in tunneling methods, damage claims, etc.

Depending upon the complexity of the geological environment and the completeness of information known about the subsurface conditions in that environment, there will be some probability of occurrence for each possible problem. The completeness of information can be expressed by the level of confidence that one has about the possible consequences that may result. The level of confidence can thus be defined as the quantity and quality of the information that is known relative to that desired. The probability of occurrence is defined as a measure of the degree of belief that one has about the likelihood of an event happening. The risk value associated with each problem will then be the product of the cost consequence and the probability of occurrence for that problem.

The use of a suitable subsurface exploration technique or combination of techniques will furnish some amount of information about one or more subsurface conditions. The information thus obtained will increase the level of confidence and consequently reduce the probability of occurrence, and likewise the risk value, associated with the various problems. The value of the information obtained by a candidate subsurface exploration system will then be the total reduction in risk values for the different problems that can be attributed to the increased amount of knowledge obtained by the candidate system.

This analysis was then the basis used for developing the preliminary value analysis model. A step-wise procedure was adopted for the model because of the interrelationship that exists on every tunneling project between the different subsurface conditions which may occur, the various possible problems that could result and those subsurface exploration techniques which could be used to obtain additional information about the subsurface conditions. Figure 17 is a flow chart showing the order of steps to be performed with the value analysis model. The individual steps are described as follows:

1. Divide the tunnel into distinct segments of arbitrary lengths. The parameters representing the various subsurface candidates within each segment should all be consistent over the segment length even though the information about each may be incomplete. The individual segment lengths should, therefore, be determined by the user according to the information then available concerning the subsurface conditions.

2. Select a tunnel segment and evaluate the available information concerning the subsurface conditions within that

1. Divide tunnel into distinct segments.

2. Select a segment and evaluate it for potential problems. Determine what problems may occur.

3. Select one of the potential problems.

4. Identify the subsurface conditions affecting this problem.

5. Rate the relative percentage importance of each condition to this problem.

6. Define the confidence level held for each condition.

7. Estimate the probability of this problem occurring.

8. Estimate the cost consequence to the project if this problem occurs.

9. Calculate the risk value for this problem.

10. Apportion this risk value among those subsurface conditions pertinent to this problem.

11. Is there another potential problem that may occur in this segment? Yes

No

12. Is there another tunnel segment to be considered? Yes

No

Figure 17. Value analysis model flow diagram.

13. Sum the apportioned risk values from all of the problems for each subsurface condition.

14. Identify those conditions requiring more information.

15. Determine which subsurface exploration methods are suitable for obtaining this desired information.

16. Select one of the potential problems.

17. Select an applicable subsurface exploration method.

18. Estimate the percentage increase in confidence level expected for each applicable subsurface condition.

19. Calculate the expected resulting reduction in apportioned risk value for each subsurface condition.

20. Is there another subsurface exploration method applicable for this potential problem? Yes

No

21. Is there another potential problem to be considered? Yes

No

Figure 17. Value analysis model flow diagram (continued).

22. Select one of the applicable subsurface exploration methods.

23. Sum the apportioned risk value reductions from every problem for each subsurface condition.

24. Sum the apportioned risk value reductions from every condition for the method being considered.

25. Determine the cost of using the method being considered.

26. Calculate the ratio of total risk value reduction to application cost for the method being considered.

27. Is there another subsurface exploration method to be considered? Yes

No

28. Compare the cost benefit ratios calculated for each of the subsurface exploration methods. Are any of these ratios > 1? No

To Step 41

Yes

Figure 17. Value analysis model flow diagram (continued).

29. Formulate candidate systems from among the various subsurface exploration methods.

30. Select one of the candidate subsurface exploration systems.

31. Sum the risk value reductions from each subsurface condition for each method in the system being considered.

32. Sum the risk value reductions from each method for the candidate system under consideration.

33. Determine the cost of using the system under consideration.

34. Calculate the ratio of total risk value reduction to application cost for the system under consideration.

35. Is there another candidate subsurface exploration system to be considered? Yes

No

36. Compare the cost benefit ratios calculated for each of the candidate subsurface exploration systems. Are any of these ratios > 1? Yes

No

37. Have all possible candidate subsurface exploration systems been formulated? No

Yes

38. Select that system (or method) having the greatest cost benefit ratio.

Figure 17. Value analysis model flow diagram (continued).

39. Is the selected system's (or method's) cost benefit ratio of sufficient magnitude? No

Yes

40. Use the selected subsuface exploration system or method to acquire additional information.

41. Stop analysis or repeat it for the next exploration stage.

From Step 28

Figure 17. Value analysis model flow diagram (continued).

segment with regard to potential problems. Determine what particular problems may occur, remembering that there may be several with some possibly occurring together.

3. Select one of the potential problems that may occur in the tunnel segment under consideration.

4. Identify those subsurface conditions upon which this potential problem is dependent.

5. Rate the relative percentage importance (b) for each of these subsurface conditions to the potential problem under consideration. For each particular case, the user must decide these relative importances on the basis of the affect that each subsurface condition has upon the potential problem.

6. Define the level of confidence now held (d) about the known information for each of these subsurface conditions. A scale of 0 to 1, with the value of 1 being the maximum level desired, should be used.

7. Estimate the probability (P) that the potential problem under consideration may occur. This estimate will be some decimal number between 0 and 1 assigned by the user according to his judgement concerning the likelihood of the potential problem occurring. A value of 0 will mean that there is, in the user's judgement, no likelihood of the potential problem occurring while a value of 1 will denote that the potential problem will occur with certainty. The user should make the probability estimate from his expectations of the consequences which experience and knowledge indicate could occur based upon that information available concerning the subsurface conditions and geological environment.

8. Estimate the cost consequence (C) to the project if the potential problem under consideration occurs. This would be that additional cost, both direct and indirect, incurred by the project as the result of the potential problem occurring. If two or more problems are associated, that is, dependent upon each other or possibly occurring together, then extreme care must be exercised in assigning values to such items as delays which could be chargeable to more than just one of the problems in amounts that can vary considerably according to the particular circumstances actually encountered on the project. Different users, and even a single user, may use different methods for this proration depending upon the circumstances of the particular case under investigation.

Therefore, the particular method used for accomplishing this step should be documented for future reference by the user in each case considered.

9. Calculate the risk value (R) associated with the potential problem under consideration. This risk value will be the product of the probability (from step 7) and the cost consequence (from step 8) for this particular problem, or R = PC. The risk value is that dollar value the user places on the potential problem based upon available information that he has at this time.

10. Apportion this calculated risk value among those subsurface conditions upon which the potential problem is dependent. This apportionment is done according to both the relative importance (from step 5) and the level of confidence held about the known information (from step 6) for each of these subsurface conditions. Use the equation

$$r = \frac{aR}{A}$$

where r is the apportioned risk value,

a is the apportionment factor for a particular subsurface condition = $b(1-d)$, and

A is the sum of the apportionment factors for a particular problem =

$$A = a_1 + a_2 + a_3 + \ldots. + a_{n-1} + a_n = \Sigma a.$$

The apportioned risk values will then be the value of the information that the user has available at this time concerning each respective subsurface condition.

11. Is there another potential problem that may occur in the tunnel segment under consideration? If the answer is yes, return to step 3. If the answer is no, continue on to step 12.

12. Is there another tunnel segment to be considered? If the answer is yes, return to step 2. If the answer is no, continue to step 13.

13. For each and every applicable subsurface condition, sum the apportioned risk values (from step 10) from all of the respective potential problems that are dependent in some manner upon that condition. All of the tunnel segments should be included for this summation. The total obtained for each subsurface condition will then represent the total

value of the information that the user has available concerning that condition. Use the expression

$$E = r_I + r_{II} + r_{III} + \dots + r_{N-I} + r_N = \Sigma r$$

where E is the sum of the apportioned risk values from all potential problems for a particular subsurface condition.

14. By a review of the results from step 13, identify those primary subsurface conditions about which more information is desired. Those conditions will be those that have the higher apportioned risk value totals.

15. Determine which subsurface exploration methods can be used to obtain additional information about these subsurface conditions. This additional information must be capable of raising the confidence levels held concerning the information about the subsurface conditions. This raise in confidence levels will then, in turn, reduce the risk values connected with these subsurface conditions.

16. Select one of the potential problems that may occur.

17. Select one of the subsurface exploration methods that will furnish additional information about one or more of the subsurface conditions upon which the potential problem under consideration is dependent.

18. Estimate the percentage increase in the level of confidence that is expected (f) for each applicable subsurface condition by application of the selected subsurface exploration method. This percentage increase will be equal to 100 times the difference between the expected resulting level of confidence and the present level of confidence, all divided by the present confidence level.

19. Calculate the reduction in the apportioned risk value for each applicable subsurface condition (G) due to the expected increase in confidence level from

$$G = \frac{bdrf}{100.0a}$$

This risk value reduction will then be the value of the information that is expected from the resulting increased confidence level obtained by use of the subsurface exploration method under consideration?

20. Is there another subsurface exploration method applicable for this potential problem under consideration?

If the answer is yes, return to step 17. If the answer is no, continue on to step 21.

21. Is there another potential problem to be considered? If the answer is yes, return to step 16. If the answer is no, continue on to step 22.

22. Select one of the applicable subsurface exploration methods that could be considered as a possible component for a subsurface exploration system.

23. For each and every subsurface condition applicable to the subsurface exploration method under consideration, sum the expected risk value reductions (from step 19) for all of the respective potential problems that are dependent in some manner upon that condition using

$$S = G_I + G_{II} + G_{III} + \dots + G_{N-I} + G_N = \Sigma G.$$

The total (S) obtained for each subsurface condition will then represent the total value in risk value reductions which the user expects concerning that condition by application of the method under consideration.

24. Sum the individual total risk value reductions from the various subsurface conditions (from step 23) to obtain the total risk value reduction expected by application of the subsurface exploration method under consideration (T) using

$$T = S_1 + S_2 + S_3 + \dots + S_{n-1} + S_n = \Sigma S.$$

25. Determine the cost (h) of using the subsurface exploration method under consideration. This cost may have to be estimated from past experience, but the user can usually obtain it more accurately from quotations by different service vendors.

26. Calculate the ratio of the total risk value reduction expected (from step 24) to the cost of application (from step 25) for the subsurface exploration method under consideration, or

$$L = \frac{T}{h}.$$

This calculated ratio (L) will be the cost benefit ratio for the method being considered by itself.

27. Is there another applicable subsurface exploration method to be considered? If the answer is yes, return to step 22. If the answer is no, continue to step 28.

28. Compare the cost benefit ratios calculated for each of the applicable subsurface exploration methods considered. Do any of these methods have a cost benefit ratio equal to or greater than one? If the answer is yes, continue to step 29. If the answer is no, then further exploration is not justified so continue to step 41.

29. From among those subsurface exploration methods having the greatest cost benefit ratios, formulate as many practical combinations of methods as deemed necessary to obtain candidate subsurface exploration systems that will furnish some amount of information about the more important subsurface conditions.

30. Select one candidate subsurface exploration system for consideration.

31. For each and every subsurface exploration method within the system being considered, sum the individual total risk value reductions from the various applicable subsurface conditions (from step 23) to obtain the total risk value reduction expected by application of the respective methods using

$$U = \Sigma S.$$

If a particular subsurface condition is applicable to more than one of the methods in the system under consideration, disregard all of the risk value reductions for that condition except for the greatest one and use that reduction value with only that method to which it pertains.

32. Sum the individual total risk value reductions from the various subsurface exploration methods (from step 31) to obtain the total risk value reduction expected by application of the system under consideration (W) using

$$W = U_1 + U_2 + U_3 + \ldots + U_{n-1} + U_n = \Sigma U.$$

33. Determine the cost (H) for using the candidate subsurface exploration system under consideration.

34. Calculate the ratio of the total risk value reduction expected (from step 32) to the cost of application (from step 33) for the candidate subsurface exploration system under consideration, or

$$M = \frac{W}{H}.$$

This calculated ratio (M) will be the cost benefit ratio for the candidate system being considered.

35. Is there another candidate subsurface exploration system to be considered? If the answer is yes, return to step 30. If the answer is no, continue to step 36.

36. Compare the cost benefit ratios calculated for each of the candidate subsurface exploration systems considered. Do any of these systems have a cost benefit ratio equal or greater than one? If the answer is yes, continue to step 38. If the answer is no, then none of the candidate systems that were formulated are justified, so continue to step 37.

37. Have all possible candidate subsurface exploration systems been formulated from among those methods having individual cost benefit ratios equal to or greater than one (from step 28)? If the answer is yes, then there is no possible candidate system justified, so continue to step 38. If the answer is no, return to step 29.

38. Select that candidate subsurface exploration system or individual subsurface exploration method (if no candidate system has a cost benefit ratio sufficiently larger than one) having the greatest cost benefit ratio.

39. Although the selected candidate system or method is justified because its cost benefit ratio is greater than one, is the cost benefit ratio of sufficient magnitude to the user? If the answer is yes, continue to step 40. If the answer is no, continue to step 41.

40. Use the selected subsurface exploration system or method to acquire additional information about the subsurface conditions.

41. Stop the analysis. If additional information has been obtained about the subsurface conditions, then repeat the analysis for the next stage of exploration.

If there is a limited amount of time available in which to perform the desired subsurface exploration then only those subsurface exploration methods and systems that can be utilized within that time frame should be considered in the analysis. Likewise, if there is a limit to the amount of funds available to the user for the subsurface exploration, then only those methods and systems which do not exceed this cost limitation should be considered.

When selecting the various candidate methods and systems, care must be taken to insure that the selected candidates are practical. For instance, when using seismic geophysical methods, some boreholes may be required for correlation purposes, and some sampling methods must be used to obtain the necessary test specimens required when considering laboratory tests. Therefore, the various subsurface exploration methods which are dependent upon each other must be considered together in combination because of their association.

The preliminary value analysis model developed in this study can enable a designer of a subsurface exploration program to make the best choice from among various possible alternate subsurface exploration systems. It can also be used to indicate to the user, or to someone else by the user, the relative value of achieving new levels of exploration information or performance. This preliminary model is quite general in nature and thus should be usable to some advantage at any stage of the subsurface exploration process for any size of project having any complexity of geological environment and subsurface conditions.

DESIGN OF AN OPTIMUM SYSTEM

Forms (Figures 18, 19, and 20) have been developed for use with the value analysis model. These forms will aid the user in applying the model to a particular project. The use of these forms can be best illustrated by an example.

> Let us assume there are three potential problems which we can label as I, II, and III along a tunnel that at this stage has been considered as having only one segment. It is further assumed that problem I depends upon the subsurface conditions A, B, C, and D while problem II depends upon subsurface conditions B, D, E, and F and problem III depends upon subsurface conditions A, C, E, and G. It is also assumed that the subsurface exploration methods a, b, c, d, e, and f can be used to obtain additional information about one or more of these subsurface conditions as follows:
>
> method a for conditions A, B, and C
> method b for conditions A, D, and E
> method c for conditions B and D
> method d for conditions C and G
> method e for condition F
> method f for condition G
>
> A set of the forms used with the value analysis model have been filled out for this example (Figures 21 through 25). The values that would be estimated by the user for the other parameter needed (in steps 5, 6, 7, 8, 18, 25, and 33 of the model) were assumed for this example as shown on the completed example forms.

PROBLEM. ______________________ IN TUNNEL SEGMENT: ______________

PROBABILITY OF OCCURANCE (P) = ______________ COST CONSEQUENCE (C) = ______________ BY: ________ DATE: ________

RISK VALUE (R) = PxC = ______________________

SUBSURFACE CONDITIONS	RELATIVE % OF IMPORTANCE b	LEVEL OF CONFIDENCE HELD d	APPORTION-MENT FACTOR a	APPORTIONED RISK VALUE r	APPLICABLE SUBSURFACE EXPLORATION METHODS							
					% INCREASE IN CONFIDENCE LEVEL f	REDUCTION IN APPORTIONED RISK VALUE G	% INCREASE IN CONFIDENCE LEVEL f	REDUCTION IN APPORTIONED RISK VALUE G	% INCREASE IN CONFIDENCE LEVEL f	REDUCTION IN APPORTIONED RISK VALUE G	% INCREASE IN CONFIDENCE LEVEL f	REDUCTION IN APPORTIONED RISK VALUE G
	100.0	A =										

FORM FST-234-1

EQUATIONS: $a = b(1.0-d)$, $A = a_1 + a_2 + a_3 + \ldots + a_{n-1} + a_n = \sum a$, $r = \frac{aR}{A}$, & $G = \frac{bdrf}{100.0a}$

Figure 18. Value analysis model, Form 1.

SUBSURFACE CONDITIONS	$E = \Sigma r$	SUM OF APPORTIONED RISK VALUE REDUCTIONS FROM RESPECTIVE POTENTIAL PROBLEMS (S) FOR APPLICABLE SUBSURFACE EXPLORATION METHODS							
T =									
METHOD COST (h) =									
COST BENEFIT RATIO (L) =									

FORM FST-234-2

EQUATIONS : $E = r_I + r_{II} + r_{III} + \ldots + r_{N-I} + r_N = \Sigma r$, $S = G_I + G_{II} + G_{III} + \ldots + G_{N-I} + G_N = \Sigma G$, $T = S_1 + S_2 + S_3 + \ldots + S_{n-1} + S_n = \Sigma S$, & $L = \frac{T}{h}$

Figure 19. Value analysis model, Form 2.

EXPLORATION METHODS	SUM OF APPORTIONED RISK VALUE REDUCTIONS FROM RESPECTIVE SUBSURFACE CONDITIONS (U) FOR APPLICABLE SUBSURFACE EXPLORATION SYSTEMS							
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
W =								
SYSTEM COST (H) =								
COST BENEFIT RATIO (M) =								

FORM FST-234-3

EQUATIONS: $U = S_1 + S_2 + S_3 + \ldots + S_{n-1} + S_n = \Sigma S$, $W = U_1 + U_2 + U_3 + \ldots + U_{n-1} + U_n = \Sigma U$, & $M = \frac{W}{H}$

Figure 20. Value analysis model, Form 3.

PROBLEM: I IN TUNNEL SEGMENT: ______

PROBABILITY OF OCCURANCE (P) = 0.3 COST CONSEQUENCE (C) = $ 10,000 BY: ______ DATE: ______

RISK VALUE (R) = PxC = $ 3,000

SUBSURFACE CONDITIONS	RELATIVE % OF IMPORTANCE	LEVEL OF CONFIDENCE HELD	APPORTION-MENT FACTOR	APPORTIONED RISK VALVE	APPLICABLE SUBSURFACE EXPLORATION METHODS a: % INCREASE IN CONFIDENCE LEVEL	a: REDUCTION IN APPORTIONED RISK VALUE	b: % INCREASE IN CONFIDENCE LEVEL	b: REDUCTION IN APPORTIONED RISK VALUE	c: % INCREASE IN CONFIDENCE LEVEL	c: REDUCTION IN APPORTIONED RISK VALUE	d: % INCREASE IN CONFIDENCE LEVEL	d: REDUCTION IN APPORTIONED RISK VALUE
	b	d	a	r	f	G	f	G	f	G	f	G
A	40	0.2	32	$1548.39	50	$193.55	25	$96.77				
B	30	0.5	15	725.81	10	72.58			20	$145.16		
C	20	0.4	12	580.64	15	58.06					25	$96.77
D	10	0.7	3	145.16			35	118.55	20	67.74		
	100.0	A =	62									

FORM FST-234-1

EQUATIONS: $a = b(1.0-d)$, $A = a_1 + a_2 + a_3 + \ldots + a_{n-1} + a_n = \sum a$, $r = \frac{aR}{A}$, & $G = \frac{bdrf}{100.0a}$

Figure 21. Value analysis example problem, Form 1a.

PROBLEM: II IN TUNNEL SEGMENT: ______

PROBABILITY OF OCCURANCE (P) = 0.4 COST CONSEQUENCE (C) = $15,000 BY: ______ DATE: ______

RISK VALUE (R) = PxC = $6,000

SUBSURFACE CONDITIONS	RELATIVE % OF IMPORTANCE b	LEVEL OF CONFIDENCE HELD d	APPORTION-MENT FACTOR a	APPORTIONED RISK VALUE r	APPLICABLE SUBSURFACE EXPLORATION METHODS a: % INCREASE IN CONFIDENCE LEVEL f	a: REDUCTION IN APPORTIONED RISK VALUE G	b: % INCREASE IN CONFIDENCE LEVEL f	b: REDUCTION IN APPORTIONED RISK VALUE G	c: % INCREASE IN CONFIDENCE LEVEL f	c: REDUCTION IN APPORTIONED RISK VALUE G	e: % INCREASE IN CONFIDENCE LEVEL f	e: REDUCTION IN APPORTIONED RISK VALUE G
B	20	0.5	10	$1132.08	10	$113.21			20	$226.42		
D	40	0.7	12	1358.49			35	$1109.43	20	633.96		
E	25	0.3	17.5	1981.13			10	84.91				
F	15	0.1	13.5	1528.30							45	$76.42
	100.0		A = 53									

FORM FST-234-1

EQUATIONS: $a = b(1.0-d)$, $A = a_1 + a_2 + a_3 + \ldots + a_{n-1} + a_n = \sum a$, $r = \frac{aR}{A}$, & $G = \frac{bdrf}{100.0a}$

Figure 22. Value analysis example problem, Form 1b.

PROBLEM: III IN TUNNEL SEGMENT: ____

PROBABILITY OF OCCURANCE (P) = 0.8 COST CONSEQUENCE (C) = $8000 BY: ____ DATE: ____

RISK VALUE (R) = PxC = $6400

SUBSURFACE CONDITIONS	RELATIVE % OF IMPORTANCE	LEVEL OF CONFIDENCE HELD	APPORTION-MENT FACTOR	APPORTIONED RISK VALUE	APPLICABLE SUBSURFACE EXPLORATION METHODS a: % INCREASE IN CONFIDENCE LEVEL	a: REDUCTION IN APPORTIONED RISK VALUE	b: % INCREASE IN CONFIDENCE LEVEL	b: REDUCTION IN APPORTIONED RISK VALUE	d: % INCREASE IN CONFIDENCE LEVEL	d: REDUCTION IN APPORTIONED RISK VALUE	f: % INCREASE IN CONFIDENCE LEVEL	f: REDUCTION IN APPORTIONED RISK VALUE
	b	d	a	r	f	G	f	G	f	G	f	G
A	35	0.2	28	$2560.00	50	$320.00	25	$160.00				
C	15	0.4	9	822.86	15	82.29			25	$137.14		
E	45	0.3	31.5	2880.00			10	123.43				
G	5	0.7	1.5	137.14					20	64.00	40	$128.00
	100.0		A = 70									

FORM FST-234-1

EQUATIONS: $a = b(1.0-d)$, $A = a_1 + a_2 + a_3 + \dots + a_{n-1} + a_n = \sum a$, $r = \frac{aR}{A}$, & $G = \frac{bdrf}{100.0a}$

Figure 23. Value analysis example problem, Form 1c.

SUBSURFACE CONDITIONS	$E = \Sigma r$	SUM OF APPORTIONED RISK VALUE REDUCTIONS FROM RESPECTIVE POTENTIAL PROBLEMS (S) FOR APPLICABLE SUBSURFACE EXPLORATION METHODS							
		1	2	3	4	5	6		
A	$4108.39	$513.55	$256.77						
B	1857.89	185.79		$371.58					
C	1403.50	880.92			$233.91				
D	1503.65		1227.98	701.70					
E	4861.13		208.34						
F	1528.30					$76.42			
G	137.14				64.00		$128.00		
T =		$1580.26	$1693.09	$1073.28	$297.91	$76.42	$128.00		
METHOD COST (h) =		$ 950	$1100	$ 725	$350	$125	$425		
COST BENEFIT RATIO (L) =		1.66	1.54	1.48	0.85	0.61	0.30		

FORM FST-234-2
EQUATIONS: $E = r_I + r_{II} + r_{III} + \ldots + r_{N-I} + r_N + \Sigma r$, $S = G_I + G_{II} + G_{III} + \ldots + G_{N-I} + G_N = \Sigma G$, $T = S_1 + S_2 + S_3 + \ldots + S_{n-1} + S_n = \Sigma S$, & $L = \frac{T}{h}$

Figure 24. Value analysis example problem, Form 2.

EXPLORATION METHODS	SUM OF APPORTIONED RISK VALUE REDUCTIONS FROM RESPECTIVE SUBSURFACE CONDITIONS (U) FOR APPLICABLE SUBSURFACE EXPLORATION SYSTEMS							
	1	2	3	4	5	6		
a	$1580.26	$1580.26	$1394.47	$1394.47				
b	1436.32	1436.32	1436.32	1436.32	$1693.09	$1693.09		
c			371.58	371.58	371.58	371.58		
d	64.00		64.00		297.91	233.91		
e	76.42	76.42	76.42	76.42	76.42	76.42		
f		128.00		128.00		128.00		
W =	$3157.00	$3221.00	$3342.79	$3406.79	$2439.00	$2503.00		
SYSTEM COST (H) =	$2525	$2600	$3250	$3325	$2300	$2725		
COST BENEFIT RATIO (M) =	1.25	1.24	1.03	1.02	1.06	0.92		

FORM FST-234-3
EQUATIONS:

$U = S_1 + S_2 + S_3 + \ldots + S_{n-1} + S_n = \Sigma S$, $W = U_1 + U_2 + U_3 + \ldots + U_{n-1} + U_n = \Sigma U$, & $M = \frac{W}{H}$

Figure 25. Value analysis example problem, Form 3.

Generalized equations are presented in this subsection which can be used to determine how tunnel costs vary with changing ground conditions. These cost equations were prepared for use with the value analysis model presented earlier in this report.

We reviewed tunneling costs information from a number of sources in preparing these cost equations. Unfortunately, statistics on tunneling and related cost data are not readily available, especially data on tunnels of the size required for highways. The best available sources and some of their limitations relative to this present study are described in the following paragraphs.

The State of California published a 1959 report[33] that presents a procedure for estimating tunnel costs. This procedure involves the use of a series of curves having a cost base of January, 1957 and were developed from the records of 99 tunnel projects. The data was extrapolated up to 28-foot diameter tunnels from 24-foot maximum diameter tunnels. Steel sets were the only ground support considered in this California report.

A Fenix & Scisson, Inc. 1968 report[34] contains a graph of total cost versus tunnel diameter for 33 tunnels. This graph shows a cost envelope, but no breakdown of costs are presented.

A 1969 report by Pack[35] and others at the University of Illinois, contains some cost data on tunnels in both soil and rock. But the data was primarily for ground support and lining and was almost entirely for an 18-foot tunnel size.

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[33] "Procedure for Estimating Costs of Tunnel Construction." Appendix C to Bulletin No. 78, Investigation of Alternate Aqueduct Systems to Serve Southern California. State of California Department of Water Resources, September, 1959.

[34] Fenix & Scisson, Inc. "Conventional Tunneling Methods." Vol. III of Feasibility of Flame-Jet Tunneling. Report G-910560-10 for U.S. Department of Transportation, OHSGT, Contract 7-35126 by United Aircraft Research Laboratories. East Hartford, Conn. May, 1968.

[35] Peck, R.B., D.U. Deere, J.E. Monsees, H. W. Parker, and B. Schmidt. Some Design Considerations in the Selection of Underground Support Systems. Report for U.S. Dept. of Transportation, OHSGT and UMTA, Contract 3-0152. University of Illinois. November, 1969.

In another Fenix & Scisson, Inc. report,[36] published in 1970, costs for 59 soft-ground tunnels were presented and analyzed. Average percentage breakdowns of cost for the various component operations were presented.

The data contained in the 1970 Fenix & Scisson, Inc. report has been used to obtain the graph shown in Figure 26. The 1970 costs in the report were esculated to 1973 dollars for Figure 36 by using the Engineering News Record construction cost index.

Harza Engineering Company conducted a study[37] in 1970 in which they developed a series of cost estimating equations from various tunnel cost data. These equations were obtained from empirical fits of cost curves to the cost matrices that were developed during their study.

In a 1971 paper by Lyons[38] the following ranges for a relative proportions of labor, equipment, and materials likely to be encountered by soft-ground tunneling in Great Britain was given:

Labor, 26 to 55 percent
Equipment, 7 to 28 percent
Materials, 21 to 50 percent

Although these values were for British tunnels, they show that there is a wide variance in the cost components and this is the same everywhere.

Figure 26 and the above value ranges, as well as each of the references cited in this subsection all indicate that there is a wide variance in tunnel costs. This wide variance results from the great variability in conditions and information known about these conditions between different tunnel projects. Because of this variability, we did not find it practical to prepare charts which attempt to project tunnel costs for this study.

The Harza Engineering Company report equations were modified for use in this present report to indicate approximate costs in 1973 dollars for the major cost elements of tunneling. Appropriate unit price indexes for the Chicago area were obtained from the Engineering News Record and used to determine the 1973 dollar values.

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[36]Fenix & Scisson, Inc. A Systems Study of Soft Ground Tunneling. Report DOT-FRA-OHSGT-231 for U.S. Dept. of Transportation, OHSGT and UMTA, Contract 9-0034. Tulsa, Oklahoma. May, 1970.

[37]Wheby, Frank T. and Edward M. Cikanek. A Computer Program for Estimating Cost of Hard Rock Tunnelling (COHART). Report for U. S. Department of Transportation, OHSGT and UMTA, Contract 9-00003. Harza Engineering Co., Chicago. May, 1970.

[38]"Underground Motorways for Urban Areas." Tunnels and Tunnelling, Vol. 3 No. 4 (July-August, 1971), pp 277-278

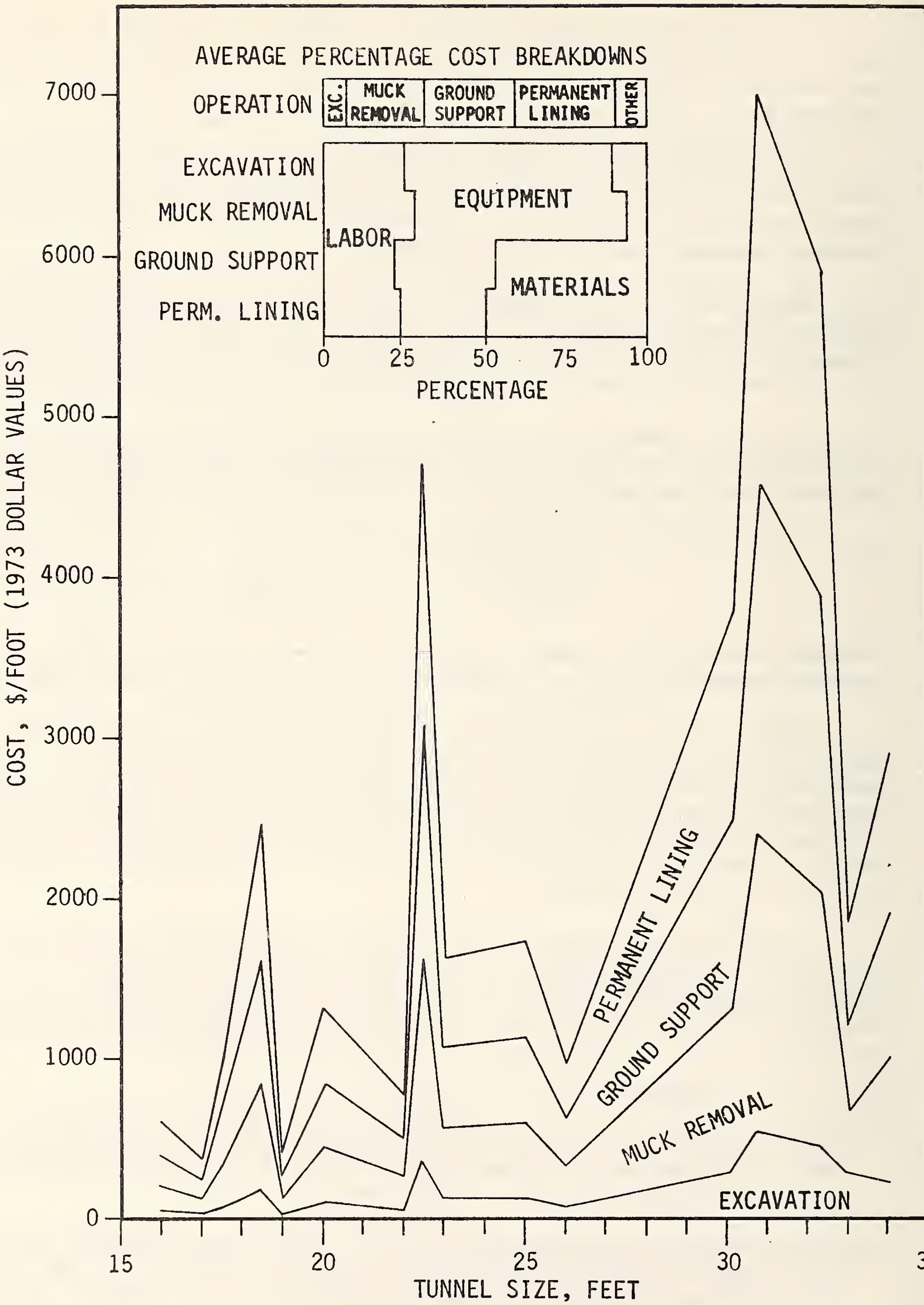

Figure 26. Cost breakdowns for soft ground tunnels based on data from 31 tunnels.

For our equations, we divided tunneling operations into four major components - excavation, muck removal, ground support, and permanent lining. Costs for those components were divided into elements of labor, equipment, and materials.

We considered the three tunnel shapes shown in Figure 27. We have assumed that:

1. The tunnel is driven from portals and by conventional drill and blast methods only, except for the circular shape for which a boring machine was also considered.
2. Rail haulage and truck haulage are the two muck removal systems.
3. Steel sets and rock bolts are the ground support methods.
4. Formed concrete is the permanent lining.

We present our tunnel cost equations in Tables 19 through 22. The symbols used for the equations in these tables are defined here.

B = shape factor for tunnel shape based on cross-sectional area (circular = 0.785, horseshoe = 0.893, and modified horseshoe = 0.425)
D = characteristic nominal excavation dimension of tunnel, feet
e = base of natural logarithms, = 2.71828 ....
F = factor used for different excavation methods and for tunnel shapes
h = average slope for much haulage, absolute value of decimal
J = unit volume of lining, cubic yards per foot of tunnel length
L = length of tunnel, miles
M = muck loading rate, cubic yards per hour
Q = groundwater inflow at tunnel face, gallons per minute
R = rock quality designation, percent
S = unconfined compressive strength of rock, pounds per square inch
t = thickness of lining, feet
T = blocking and logging used with steel sets, mfbm per foot of tunnel length
U = weight of steel set supports, pounds per foot of tunnel length
V = weight of rock bolt supports, pounds per foot of tunnel length
W = weight of wire mesh and straps used with rock bolts, pound per foot of tunnel length
x = factor used for different muck loading rates (3.50 when $M \leq$ 100 cubic yards per hour; 6.6 when M >100 and< 300 cubic yards per hour; 12.90 when $M \geq 300$ cubic yards per hour)
y = factor used for different excavation methods
Z = factor used for different tunnel shapes with truck haulage
$5.0 - 0.0001(70 - D)^2$ for circular or horseshoe shape
$4.5 - \frac{(D - 50)^4}{44,000}$ for a modified horseshoe shape

z = factor used for different tunnel shapes (circular = 2.2, horseshoe = 2.0, modified horseshoe = 1.6)

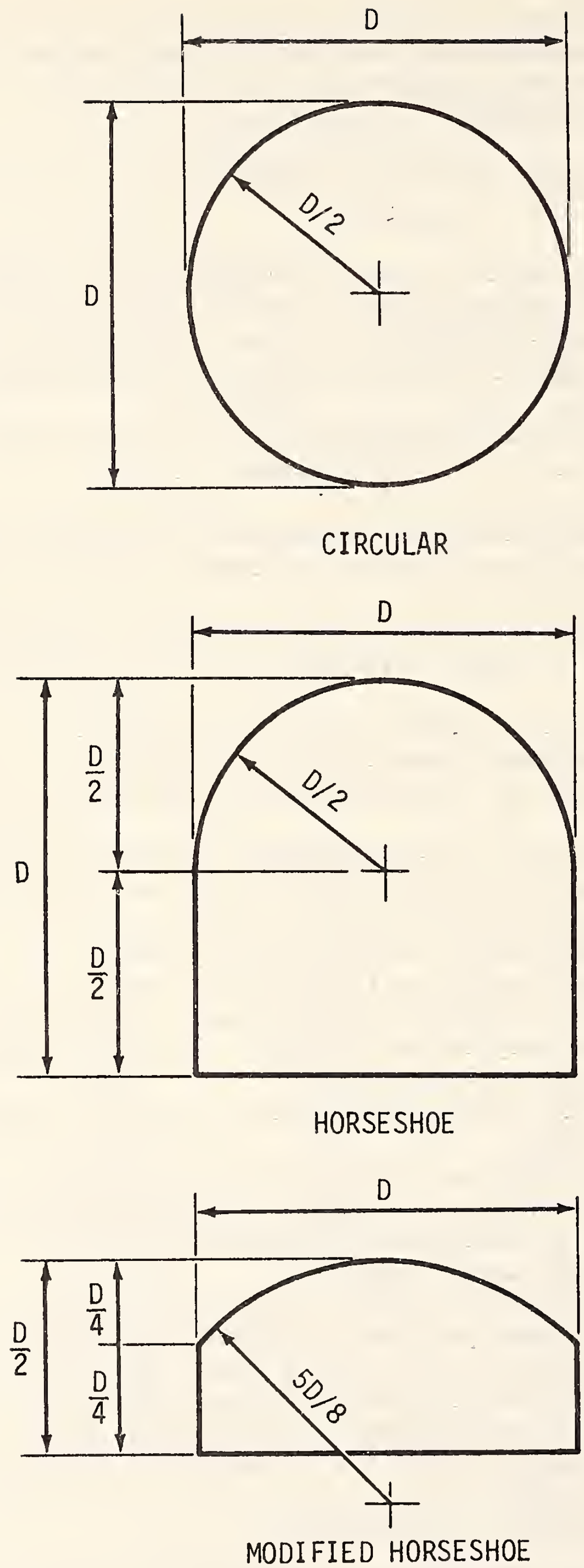

Figure 27. Tunnel shapes used for tunnel cost equations.

Table 19. Cost estimating equations for excavation.

Conventional Drill and Blast

$$A = \frac{0.8RD^{2.25} + 1000}{B(1.0 + 0.0000025S)D^{1.5}e^{0.035D}} \times \frac{Q + 1500}{5.0Q + 1500}$$

Circular Shape

Labor

$$C = \left[0.07(D + 40)^2 - 90.70\right] \times \frac{24 + 0.45L}{A} \times \frac{55.59}{35.72}$$

Equipment

$$C = \left[0.046(D + 15)^2 + x\right] \times \frac{24}{A} \times \frac{131.4 \times 122.1}{134.8}$$

Materials

$$C = \left\{\frac{B \times D^2}{2000}\left[S^{0.5} + 1.1156(7.1 - 0.01R)^2\right] + \frac{0.11(D + 10)^2 - 25}{A}\right\} \times \frac{466.94}{350.00}$$

Horseshoe Shape

Labor

$$C = \left[0.08(D + 40)^2 - 90.70\right] \times \frac{24 + 0.45L}{A} \times \frac{55.59}{35.72}$$

Equipment

$$C = \left[0.05(D + 15)^2 + 4 + x\right] \times \frac{24}{A} \times \frac{131.4 \times 122.1}{134.8}$$

Materials

$$C = \left\{\frac{B \times D^2}{2000}\left[S^{0.5} + 1.11556(7.1 - 0.01R)^2\right] + \frac{0.12(D + 10)^2 - 22}{A}\right\} \times \frac{466.94}{350.00}$$

Table 19. Cost estimating equations for excavation (continued).

Modified Horseshoe Shape

Labor $$C = \left[0.10D^2 + 69.30\right] \times \frac{24 + 0.45L}{A} \times \frac{55.59}{35.72}$$

Equipment $$C = \left[0.04(D + 5)^2 + 17 + x\right] \times \frac{24}{A} \times \frac{131.4 \times 122.1}{134.8}$$

Materials $$C = \left\{\frac{B \times D^2}{2000}\left[S^{0.5} + 1.11556(7.1 - 0.01R)^2\right] + \frac{0.13(D - 5)^2 + 14}{A}\right\} \times \frac{466.94}{350.00}$$

Bored Circular Shape $$A = \frac{50000(100 + R)}{S \times D} \times \frac{Q + 1500}{5.0Q + 1500}$$

Labor $$C = \left[\frac{50000 + 152(D + 12)^2}{5280L} + (0.021D^2 + 55) \times \frac{24 + 0.45L}{A}\right] \times \frac{55.59}{35.72}$$

Equipment $$C = \left[\frac{25000 + 234(D - 10)^2}{5280L} + \frac{1.152D^2}{A}\right] \times \frac{131.4 \times 122.1}{134.8}$$

Materials $$C = \left[\frac{D^2}{211.2L} + \left(5000 + S + \frac{120000}{A}\right) \times \frac{D^2}{220000}\right] \times \frac{466.94}{350.00}$$

Table 20. Cost estimating equations for muck removal.

Rail Haulage

Labor

$$C = \left\{\frac{0.6M}{1100}\left[(300 - A)\left(\frac{L}{2}\right)^{0.33} + \frac{A}{2}\left(1.0 + \frac{2.3L}{12}\right) + 150\right] + \left[21\left(\frac{L}{2}\right)^{0.82} + \frac{A}{12}\right]\right\}\frac{24 + 0.45L}{A} \times \frac{55.59}{35.72}$$

Equipment

$$C = \left\{\frac{16.128M}{1100}\left[(300 - A)\left(\frac{L}{2}\right)^{0.33} + \frac{A}{2}\left(1.0 + \frac{2.3L}{12}\right) + 150\right] + 0.7L\right\}\frac{24}{A} \times \frac{131.4 \times 122.1}{134.8}$$

Materials

$$C = 0.015BL\left[0.167(7.1 - 0.01R)D\right]^2 \times \frac{466.94}{350.00}$$

For conventional drill and blast

$$C = 0.011775LD^2 \times \frac{466.94}{350.00}$$

For bored circular

Truck Haulage

Labor

$$C = \left[\frac{0.462M}{z}\left(\frac{L}{2}\right)^{0.7} + 7.1\right] \times \frac{24 + 0.45L}{A} \times \frac{55.59}{35.72}$$

Equipment

$$C = (1.5 + 3.1z) \times \frac{0.6M}{z} \times \left(\frac{L}{2}\right)^{0.7} \times \frac{24}{A} \times \frac{131.4 \times 122.1}{134.8}$$

Materials

$$C = 0.025BL\left[0.167(7.1 - 0.01R)D\right]^2 \times (1.0 + 77h) \times \frac{466.94}{350.00}$$

For conventional drill and blast

$$C = 0.019625LD^2 \times \frac{466.94}{350.00}$$

For bored circular

Use the appropriate A values from Table

Table 21. Cost estimating equations for ground support.

Steel Sets

Circular Shape $$U = F \times e^{\left[3.7 + (0.7D - 6)^{0.5} - 2(0.01R + 0.2)^{2}\right]}$$

with $F = 0.56(0.01R - 0.2)^{2} + 1.22$
For conventional drill and blast

$F = 1.0$
For bored circular

Horseshoe Shape $$U = e^{\left[0.8 + (0.85D - 7)^{0.5} + 0.3(100 - R)^{0.5}\right]}$$

Modified Horseshoe Shape $$U = e^{\left[0.82 + (0.92D - 6)^{0.5} + 0.22(100 - R)^{0.5}\right]}$$

Blocking and Lagging for Steel Sets $$T = F\left[0.0018 + 0.0056(1.17 - 0.01R)^{2}\right]$$

with $F = 1.0$
For conventional drill and blast circular

$F = 1.0 + 0.012D$
For conventional drill and blast horseshoe

$F = 1.0 + 0.003D$
For conventional drill and blast modified horseshoe

$F = 0.81$
For bored circular

Labor $$C = y(0.001U \times A + 0.4D) \times \frac{24 + 0.45L}{A} \times \frac{55.59}{35.72}$$

Equipment $$C = y(6.0 + 0.0002U \times A) \times \frac{24}{A} \times \frac{131.4 \times 122.1}{134.8}$$

Materials $$C = \left(\frac{36y}{A} + 0.096U + 150T\right) \times \frac{466.94}{350.00}$$

with $y = 1.1$
For conventional drill and blast

$y = 1.0$
For bored circular

Table 21. Cost estimating equations for ground support (continued).

Rock Bolts $$V = \frac{8.56 \times F \times D \times (100 - R)^2 \times e^{0.026D}}{(152 - R)^2}$$

with $F = 1.0$
For conventional drill and blast circular and horseshoe

$F = 1.6$
For conventional drill and blast modified horseshoe

$F = 0.56$
For bored circular

Wire Mesh and Straps for Rock Bolts $W = 3.3 \times F \times D \times (1.0-0.01R)$

with $F = 1.0$
For circular and horseshoe straps

$F = 0.74$
For modified horseshoe strap

Labor $$C = 0.02V \times A \times \frac{24 + 0.45L}{A} \times \frac{55.59}{35.72}$$

Equipment $$C = 1.76 \times \frac{24}{A} \times \frac{131.4 \times 122.1}{134.8} \quad \text{When } D \leq 16$$

$$C = 9.35 \times \frac{24}{A} \times \frac{131.4 \times 122.1}{134.8} \quad \text{When } D > 16 \text{ and } V \times A \leq 1000$$

$$C = \left[9.35 + 0.0023(V \times A - 1000)\right] \times \frac{24}{A} \times \frac{131.4 \times 122.1}{138.4} \quad \text{When } D > 16 \text{ and } V \times A > 1000$$

Materials $$C = \left[0.32W + \left(0.23 + \frac{S}{530000}\right) \times V\right] \times \frac{466.94}{350.00}$$

Use the appropriate A values from Table

Table 22. Cost estimating equations for permanent linings

Formed Concrete Lining Using Reuseable Hinged Steel Forms

$$J = \frac{0.167b \times D \times (7.1 - 0.01R) \times t}{27}$$

With Rail Transport

Labor

$$C = \left\{\frac{24}{A}(35 + 0.016M) + \left[0.07(J + 26)^2 + 1.5(0.21L + 1.5) \times J + 9\left(\frac{27J}{b \times t} - 2t\right) + 21\left(\frac{L}{2}\right)^{0.82} - 12\right] \times \frac{24 + 0.45L}{A}\right\}\frac{55.59}{35.72}$$

Equipment

$$C = \left\{30 + \left[1.58(0.21L + 1.5) + 1.9\right] \times J + 0.7L + 0.03M + \left(\frac{27J}{b \times t} - 2t\right) z\right\}\frac{24}{A} \times \frac{131.4 \times 122.1}{134.8}$$

Materials

$$C = \left[0.018\left(\frac{27J}{b \times t} - 2t\right) + 13J\right] \times \frac{466.94}{350.00}$$

With Truck Transport

Labor

$$C = \left\{\frac{24}{A}(35 + 0.016M) + \left[0.07(J + 26)^2 + 7.7(0.14L + 0.48) \times \frac{J}{Z} + 9\left(\frac{27J}{b \times t} - 2t\right) - 4.9\right] \times \frac{24 + 0.45L}{A}\right\}\frac{55.59}{35.72}$$

Equipment

$$C = \left\{30 + (1.5 + 3.1Z) \times \frac{J}{Z} \times (0.14L + 0.48) + 0.03M + \left(\frac{27J}{b \times t} - 2t\right) z\right\}\frac{24}{A} \times \frac{131.4 \times 122.1}{134.8}$$

Materials

$$C = \left[0.18\left(\frac{27J}{b \times t} - 2t\right) + 13J\right] \times \frac{466.94}{350.00}$$

Use the appropriate A values from Table 19.

Section 5
Horizontal Long Hole Drilling

STATE-OF-THE-ART

TUNNEL EXPLORATION DRILLING

Long horizontal exploratory holes have seldom been drilled for public-works projects such as highway tunnels and water diversion tunnels. The Bureau of Reclamation has stated that long horizontal holes are not drilled, principally due to their relatively high cost, although short 40- to 50-foot, horizontal exploratory holes are sometimes drilled using conventional diamond-coring or rotary-percussion drilling equipment.[39]

The State of California Department of Water Resources has constructed many long, water diversion tunnels. Apparently, the deepest horizontal exploratory hole drilled on any of these projects was a 200-foot hole drilled in hard rock with a Mission down-the-hole Hammerdrill.

About 1955, 3-inch diameter, horizontal exploratory holes were drilled from each end of a planned Pennsylvania Turnpike tunnel. One hole was about 1,700 feet long and the other about 1,800 feet long. The holes were drilled using conventional, diamond core-drilling techniques. The directional control was reported to be fair to good.

In Colorado during recent years, small diameter, horizontal exploratory holes have been drilled at least 1,000 feet deep for the Straight Creek Highway tunnel job and 700 to 800 feet deep for the Vail Pass Highway tunnel. These holes were all drilled using wireline diamond core-drilling methods. Directional control was reported to be fair to good.

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[39] Morris, J. Personal communication. Section Head, Tunnels and Underground Construction, U.S. Bureau of Reclamation, Denver, Colorado. August 15, 1973.

U.S. ATOMIC ENERGY COMMISSION (AEC)

Since about 1965, the AEC at Mercury, Nevada, has been drilling small-diameter, long horizontal exploratory holes prior to tunnel excavation. The holes are all drilled in soft- to medium-hard volcanic rocks. The longest hole drilled to date - about 3,700 feet - was drilled using conventional, wireline diamond-coring techniques and a 20-foot long, NX core barrel. The drill used was an air-powered, hydraulic core drilling rig having a 2-foot feed stroke. All holes are core drilled 100 percent.

Approximately twenty horizontal holes have been drilled at the Nevada Test Site to an average depth of 2,000 feet. Directional control has varied from fair to very good. Several 2,600-foot holes terminated within 3 to 6 feet of a 10-foot by 10-foot target. Vertical deviation is controlled by changing the drilling assembly configuration and varying the thrust and rotational speed. The holes are surveyed with a Sperry-Sun single shot and multishot survey instrument.

JAPANESE UNDERSEA SEIKAN TUNNEL

The Japanese appear to have drilled the world's longest small-diameter horizontal exploratory holes. The holes are being used to explore ahead of the Seikan Tunnel which connects the islands of Japan. The tunnel is being driven in soft- to medium-hard volcanic rocks. The longest hole appears to have been at least 5,300 feet in length and was completed in late 1971 or early 1972. This hole was drilled using conventional, rotary drilling techniques and 6 3/4-inch three-cone roller bits.

The drill rig used is 400-horsepower electric-hydraulic. The overall length of the drill rig is 40 feet and has a 18.5-foot feed stroke. This rig was designed for drilling 16,000-foot horizontal holes using a 4 1/2-inch, external-upset, drill pipe.

Directional control appears to have varied from fair to good. A five-inch Dyna-Drill mud motor with a one-degree bent sub was used between 3,325 feet and 3,895 feet to control the vertical and horizontal deviation of the hole. The Dyna-Drill was used to deviate the hole down and to the right. Vertical deviation was controlled by varying the stabilization, thrust and rotational speed, in addition to using the Dyna-Drill with a one-degree bent sub. Horizontal deviation was controlled with stabilization and the use of a Dyna-Drill with a one-degree bent sub.

The hole was surveyed every 10 meters with either a Sperry-Sun EC inclinometer or a magnetic multishot pump-down survey instrument. These survey devices are pumped down the inside of the drill rods and retrieved on a wireline. also tested was a drill pipe having a pulse inclinometer to indicate the inclination angle. No details are available regarding its operations or performance.

Prior to drilling the 5,300-foot long hole, the Japanese had rotary drilled many other long holes to distances of at least 1,350 feet.

The Japanese experimented with several down-the-hole motors. The intent was to use the motors to extend the length of drilling from 2,000+ meters to as much as 5,000 meters (16,500 feet). In addition, the down-the-hole motors, when used with a bent sub, could be used to deviate the drill hole back on target. The types of mud motors evaluated were the positive displacement Dyna-Drill and a turbine or vane-type Turbo-Drill. An electrical powered Electro-Drill was also tested. Test results available are quite sketchy, but since a Dyna-Drill was used during the drilling of the 5,300-foot hole it could be inferred that the Japanese considered the Dyna-Drill to be the most effective.

The Japanese did some work with the use of a down-the-hole seismic device which would give some indication of rock structure. No information was available regarding the results of this testing.

U.S. BUREAU OF MINES

In 1942 the U.S. Bureau of Mines (USBM) drilled two long, 3-inch diameter, horizontal holes in a Pennsylvania oil-bearing sandstone.[40] One hole was 2,334 feet deep and the other was 2,255 feet deep. They were drilled horizontally from a room excavated at the bottom of a mine shaft about 420 feet below the surface. These holes were diamond core-drilled most of the way using conventional techniques. Wireline coring had not been developed. Directional control of the holes was apparently not very good. Inclinations were measured with a hydrofluoric acid bottle inclinometer and bearing angles were evidently not measured. The vertical deviation of the hole was controlled by changing the configuration of the drilling assembly and by varying thrust and rotational speed.

For several years, the USBM has been interested in the use of long, small-diameter horizontal holes drilled in coal seams for purposes of methane relief. Between 1969 and 1970, the USBM drilled many horizontal holes in coal, the longest being 503 feet drilled in a 7-foot high coal seam. These holes were all drilled using an air-hydraulic rotary drill and three-bladed drag bits.

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[40] Elder, C.W. Jr. Horizontal Drilling for Oil in Pennsylvania. U.S. Bureau of Mines Report of Investigations 3779. September, 1944.

In 1972, Fenix & Scisson, Inc., under a USBM R&D contract, drilled a 3 1/2-inch diameter, 1,100-foot horizontal hole in a 5-foot thick coal seam located in an old strip mine.[41] This hole was drilled using a gasoline-driven, hydraulic rotary drill rig having an 11-foot feed stroke. It was rotary drilled using a 3 1/2-inch three-cone roller bit. Directional control was good to very good. Vertical deviation was controlled by varying the configuration of the drilling assembly and by changing thrust and rotational speed. The hole was surveyed with a Sperry-Sun magnetic multishot survey instrument. Another 3 1/2-inch horizontal hole, drilled a total length of 1,034 feet, was accurately guided to its target with the aid of a newly designed in-hole, cableless telemetry survey system.

The cableless telemetry survey system was an integral part of the drilling assembly. It is a unique directional drilling guidance survey method that indicates the bit position without its having to be pulled from the hole. The system performs three basic functions: (1) down-hole directional sensing, (2) transfer and processing of the telemetry information, (3) display of information on the driller's console and by teletype. This information is used to directionally guide the drill bit to a preselected target. Directional control with this system was excellent.

Also in 1972, Fenix & Scisson, Inc. and its subcontractor, Sprague and Henwood, Inc., under a different contract sponsored by the USBM, rotary drilled several 3-inch diameter horizontal holes in a 7-foot thick coal seam at the bottom of a large diameter drilled shaft 840 feet deep. The longest horizontal hole was 850 feet deep. These holes were drilled using an air-driven, hydraulic rotary drilling rig. Three-bladed drag bits were used. Directional control was good to very good. Vertical deviation was controlled by changing the drilling assembly configuration and by varying thrust and rotational speed. The holes were surveyed with a Tro-Pari, mechanical single shot survey instrument.

NATIONAL COAL BOARD, ENGLAND

Between 1957 and 1958, the English National Coal Board developed methods for drilling 3- to 4-inch horizontal holes in narrow coal seams

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[41] Rommel, Robert R. and Larry A. Rives. Advanced Techniques for Drilling 1,000-ft. Small Diameter Horizontal Holes in a Coal Seam. Report for U.S. Bureau of Mines, Contract H0111355, Vol. 1, Fenix & Scisson, Inc., Tulsa, Okla. March, 1973.

for distances up to 450 feet.[42] These holes could have been drilled much deeper by increasing the size of the drill rig. The holes were drilled using conventional rotary drilling techniques and three- and four-bladed drag bits. An electric hydraulic drill rig was used. Directional control was reported to be good to very good. Vertical deviation was controlled by changing the drilling assembly configuration, by varying the thrust and rotational speed, and by using slotting techniques.

DOWNHOLE PERCUSSION DRILLING

In 1972, a technique for drilling long small diameter horizontal holes in medium-hard to very hard rock was conceived and tested by Jacobs Associates of San Francisco, California.[43] This project was under a contract sponsored by the Advanced Research Projects Agency. The drilling technique utilizes an air-powered, downhole percussion drill having independent up-hole rotation and a new method for handling up to 1,000 feet of drill pipe in one piece. Special tools were designed that would insert or withdraw the drill pipe at over 200 feet per minute. Three holes were drilled in granite rocks, the deepest being 862 feet deep and 4 inches in diameter. This appears to be a promising technique for rapidly drilling long horizontal holes in hard to very hard rocks such as granite. Directional control was not good, however, and additional work needs to be done in this area. The holes were surveyed with a magnetic multishot survey instrument.

CURVED HOLE DRILLING

In the last couple of years, several small diameter holes have been directionally drilled beneath a river channel in the Sacramento Valley of California using a 1 3/4-inch Dyna-Drill mud motor. These holes were drilled from one side of the river to the other in soil and were in excess of 600 feet long. Directional control was reported to be good.

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[42] Baxter, J.S. "Drilling of Long Boreholes in Coal." Colliery Engineering, (December, 1959), pp 520-525.

[43] Williamson, T.N. Research in Long Hole Exploratory Drilling for Rapid Excavation Underground. Report for U.S. Bureau of Mines, Contract H020020. Jacobs Associates, San Francisco. October, 1972.

In 1973, Calvert Western Exploration Company and its consultant, Fenix & Scisson, Inc., under contract to the USBM, successfully drilled a 3-inch diameter curved hole from the ground surface to a coal seam 777 feet below surface. The rock drilled consisted of a mixed series of horizontally bedded sedimentary rocks. The hole was then extended another 410 feet horizontally within the 7-foot thick coal seam. The total hole length was 1,730 feet. The hole was drilled using a 3-inch diameter diamond plug bit and a 1 3/4-inch Dyna-Drill mud motor having a 45-minute bent housing.

Directional control was good to excellent. The vertical and horizontal deviation was controlled by changing the Dyna-Drill configuration and by varying its downhole orientation. The hole was surveyed with a Sperry-Sun, magnetic multishot survey instrument.

This hole is unique in that:

- It was drilled all of the way using a 3 1/4-inch Dyna-Drill which is a positive displacement mud motor.
- This is the first time a curved hole has been drilled from the surface to a coal seam and then extended horizontally within the coal seam.

This drilling technique, or a modification to it, appears to have considerable potential as a means of drilling long horizontal exploratory holes at depths as great as 500 feet below the surface. The holes could be collared from a surface set-up, then curved so that they would become horizontal at the proposed tunnel elevation.

REMOTE GUIDANCE SYSTEMS

In addition to the cableless telemetry survey device mentioned earlier (which was developed by Telcom, Inc.), Sperry-Sun Well Surveying Company has developed a steering survey tool. This is a remote in-hole survey device for use in small or large diameter, vertical or horizontal drill holes. The survey data from the steering tool is transmitted to the surface through an electrical conductor. The steering tool is pumped down the inside of the drill pipe and retrieved on a wireline. This survey instrument will record the bearing angle, vertical angle, axial orientation angle (tool face), and the bottom hole temperature.

Since this remote guidance tool requires an electrical conductor cable to transmit survey data to the surface, it cannot be left in the drill string when the drill pipe is rotating. It would be possible to leave it in the drill string, however, when used in conjunction with a downhole motor where the drill pipe does not rotate. In this case, the readout cable could run to the surface between the outside of the drill pipe and the wall of the borehole.

Another remote guidance system that has been described in the literature required a second target hole. A homing device is placed in the target hole and the horizontal hole is then drilled toward the homing device. (U.S. Patents #3,285,350 and #3,406,766.) This guidance system has been field tested between two vertical holes for a maximum distance of 630 feet. It was stated the system would be accurate up to 1,050 feet. This system uses a magnetic field and patented sensor to determine directions. The disadvantages of the system are its relatively short range, the need for a second borehole, and the need for sending the signals to the surface on a wireline.

NEW PENETRATION TECHNIQUES

There are several relatively new, or contemplated, penetration techniques that appear to have some potential for drilling long, horizontal holes.

CONTINUOUS EJECTED CORING[44]

This drilling technique uses reverse circulation of the drilling fluid and a drill pipe consisting of two concentric pipes. The drilling fluid decends in the annulus between the two drill pipes and returns in the center pipe, bringing the cuttings and cores with it to the surface. It is claimed that this drilling method has proved quite effective when drilling through loose and caving zones. Good recovery of cores that are weak, friable, or plastic is claimed. So far as is known, this technique has not as yet been adapted to drilling horizontal holes. To date, several drilling contractors have successfully drilled many vertical or near vertical holes using this system.

RETRIEVABLE BIT DRILLING

This technique has been developed and tested by both Japanese and U.S. drilling equipment companies. This system consists of a bit-mounted inner tube that passes through the drill pipe. After the bit has

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[44] Henderson, Homer I. "The Continuous-Core Drilling Rig in the Exploration Program." Eleventh Symposium on Exploration Drilling. Quarterly of the Colorado School of Mines, Vol. 58, No. 4 (October, 1963), pp 137-150.

been pumped down, it is secured in place for drilling by a latching mechanism. Retractable teeth on the bit tube swing out to cut a hole that clears the drill pipe. The worn bit is recovered by pumping down on overshot device on a wireline. The overshot clamps onto the bit tube and the bit is recovered with the wireline. The potential advantages of this system is that it would greatly reduce the time required to change a worn bit in a deep hole. Also, if required, the drilling pipe could be used as casing. To date, this technique has been tested on vertical holes, but it is not known if attempts to drill horizontal holes have been made. It may be possible to core drill selected intervals by running an oversize wireline coring string inside the drill pipe and coring a smaller pilot hole.

HYDRAULIC JET BIT DRILLING

The University of Missouri at Rolla has been working on the development of a hydraulic jet bit for small diameter holes. Jet bit drilling has potential in that it could result in long life, be capable of rapidly penetrating very hard rocks, and have directional drilling capabilities. Petroleum drilling companies have been experimenting with large diameter roller core jet bits for drilling vertical holes for several years. Apparently no horizontal holes have been drilled with a jet bit drilling system to date.

SUBTERRENE ROCK MELTING PROBE

The Los Alamos Scientific Laboratory (LASL) of the University of California, under sponsorship from the National Science Foundation and U.S. Atomic Energy Commission (AEC), is working on the development of a high-powered rock melting probe. They have developed a prototype which can make a 2-inch hole 50 to 60 feet horizontally in rock. A 2-inch diameter hole was advanced at about 5 feet per hour. An advantage of the subterrene drill is that it can penetrate any material from loose soil to very hard rock. In addition, it leaves a lining of molten and solidified glass as it excavates. The glass lining could be quite useful in maintaining hole stability when penetrating loose unconsolidated soils. LASL is planning to develop a directional subterrene drill which would contain its own in-hole directional sensing package. One disadvantage of this system could be the difficulty of running geophysical logs in a glass-lined hole.

IN-HOLE THRUSTERS AND IN-HOLE MOTORS

The Continental Oil Company (CONOCO), Ponca City, Oklahoma, has for several years been evaluating the use of in-hole thrusters and motors for accurately drilling long horizontal holes in coal seams.[45] They have field tested several types of hydraulic in-hole motors including the Dyna-Drill and hydraulic motors of their own design. The in-hole thruster is a hydraulic, self-advancing device which provides thrust to the bit and is also capable of vertical and horizontal directional control. In addition, CONOCO has been working on the development of an in-hole orientation sensing package and in-hole nuclear shale sensing package. To date, CONOCO has drilled horizontal holes in coal up to about 800 feet deep.

COMPARISON OF HORIZONTAL PENETRATION TECHNIQUES

The state-of-the-art discussion of the previous subsection demonstrated that drilling of the longer length horizontal holes is far from being a common occurrence. To determine which horizontal penetration techniques have potential for penetrating the ground along an arbitrary preselected path over distances of approximately one mile, a comparative analysis of the various available techniques was made. Following the selection of those potentially best systems, the feasibility for application of horizontal penetration techniques to subsurface investigation is discussed and established.

COMPARATIVE ANALYSIS

The comparative analysis of horizontal penetration techniques has been made using a system type of analysis. The horizontal penetration system is divided into its major subsystems, each of which has several possible methods or components available that may enable it to perform its function satisfactorily. The various components of each subsystem are then compared with those of the other subsystems to determine their compatability with each other. The potentially best system concepts for penetrating the ground horizontally along an arbitrary preselected path approximately one mile long will then result from this analysis.

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[45]Dahl, H. D. Personal Communication. Director, Mining and Minerals Division, Continental Oil Co., Ponca City, Okla. August 15 and September 19, 1973.

The major subsystems of the horizontal penetration system will be:

1. Material Disintegration.
2. Material Removal.
3. Energy Transmission Sources for Thrust, Rotation, and Travel In and Out of the Hole.
4. Guidance and Measurement of Deviation.
5. Geological and Hydrological Investigation.

MATERIAL DISINTEGRATION SUBSYSTEM

There are numerous rock and soil penetration techniques available today for vertical hole drilling that can be adapted to horizontal hole drilling. In addition, there are numerous penetration techniques that have only been laboratory tested and there are others which are completely theoretical but which could possibly be adapted to boring horizontal holes in soil and rock.

The various rock disintegration methods are first described and the primary factors which determine the ability of the disintegration subsystems to penetrate horizontally one mile are then defined. The disintegration components are then compared against the various factors in order to screen out those disintegration components which do not appear to have merit.

Description of Rock Disintegration Methods--The disintegration of rock by mechanically induced stresses includes these methods which employ the direct application of a force to exceed the strength of the rock.

Rotary bits are used to perform practically all of the drilling in oil field operations, as well as a large percentage of the exploration and blast holes drilled in mining operations. These bits are rotated by means of a connecting drill pipe and they disintegrate the rock by either a crushing or a milling action. The force required to exceed the strength of the rock being drilled is supplied by the dead weight of the drill string and/or thrust that is applied hydraulically.

Rotary bit drilling is a very versatile method that is adaptable to a wide range of surface and subsurface conditions. It is an efficient method of rock disintegration in terms of energy input to the rock for the volume of rock disintegrated, and it is compatible with both wet and dry environments as well as with a wide range of material removal systems. The main limitation of the rotary bit drilling principle is its relatively low capability for delivering energy to the bit.[46]

Tooth bits are used for drilling soft to medium hard rock which usually has less than a 30,000 psi compressive strength. The design and fabrication of these bits can be varied for drilling particular rock types. These bits are the ones most widely used for oil field drilling.

Button bits (e.g. carbide insert bits) are used to drill hard rock which has a compressive strength in excess of 30,000 psi.

Drag bits are used to a limited extent for oil field drilling in soft unconsolidated material, but they are used extensively in blast-hole drilling. This bit has a flat bottom profile and removes material by a planing action. It requires a large amount of drill-pipe torque when drilling consolidated material.

Disc or kerf bits have been used mostly on tunneling machines and, to a lesser extent, for large diameter vertical hole drilling in soft and medium strength rock. At present, disc bits are not suitable for drilling small diameter holes. Their greatest application is on tunneling machines in soft material.

Diamond plug or full face bits consist of diamonds set into a solid metal matrix. They have a long life, but usually provide a slower drilling rate than either the tooth or button bits. Diamond plug bits are used in the oil fields to drill very hard and abrasive rock or for drilling at great depths (below 10,000 to 12,000 feet) when the time consumed in changing bits exceeds the life of tooth or button bits.

Carbide plug bits are solid body bits with small tungsten carbide cubes or chips set into a bonding matrix. They are made in a variety of shapes for specialized work and are often used for milling metal from a hole. The slow penetration rate achieved with this bit does not make it competitive with the other bit types for drilling rock.

Auger bits are used for boring soft materials such as soils. The auger tends to pull itself ahead while at the same time dislodging the soil. This technique has only been used to bore relatively short holes not exceeding several hundred feet.

Earth screws differ from the auger in that it compresses the soft material into the hole walls as it is rotated rather than dislodging

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[46] Ash, J. L., R. L. Gatliff, W. W. Grovenburg, G. C. Mathis, and J. A. Walker. Vertical Hole Development Study. Report for U.S. Atomic Energy Commission, Nevada Operations Office. Las Vegas, Nevada. October, 1971.

and transporting it outside the hole. Earth screws have only been used to bore short horizontal holes.[47]

Rock screws are as yet undeveloped. Ingersoll Rand has been testing a long tapered cutting head having cutting elements mounted on a spiral so that they will pull the rock screw into the hole as it advances, thus reducing thrust requirements.[48] Cuttings would be removed from the hole by either air or liquid. This technique will probably be limited to relatively short horizontal holes.

Zublin bits have been used to a limited extent for drilling oil field holes in soft rock. They combine the action of a rotary cone and a drag bit to achieve a planing action without the excessive drill-pipe torque that is required for a drag bit.

The Conical Borer, designed by Foster-Miller for penetrating high strength rocks such as granite, requires a smaller diameter pilot hole. The conical reamer requires considerably less thrust to ream hard rock at a given penetration rate than does a conventional three-cone roller bit.[48]

Hydraulic jets at pressures up to 5,000 psi have been used successfully by the Russians for mining coal. Hydraulic jets using the higher pressures (15,000 to 20,000 psi) which are required to disintegrate harder rock have yet to be developed for commerical use. However, substantial experimental work has been done with this method.

The retractable rotary bit is a relatively recent development in which the bit tube and bit are pumped down the inside of the drill pipe. Upon reaching bottom, the bit tube latches into the end of the drill pipe and several cutter arms extend to enlarge the hole to clear the pipe. The bit itself can be either a roller cone, drag, or diamond plug type.

Coring, which drills only the periphery of the borehole, is common practice in exploratory work. This technique has been used for drillling very hard to very soft materials. Conventional diamond coring requires making a round trip in and out of the hole to recover the core, which can vary from 5 to 20 feet or more in length. Wireline diamond coring is faster than convention coring, especially as the hole gets deeper, because the core barrel is retrived on a wireline and then pumped back to the bottom of the hole.

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[47]Paone, James, William E. Bruce, and Roger J. Morrell. Horizontal Boring Techniques: A State-of-the-Art Study. U.S. Bureau of Mines Information Circular 8392. September, 1968.

[48]Olson, James J. and Thomas C. Atchison. "Research and Development--Key to Advances for Rapid Excavation in Hard Rock." Proceedings of North American Rapid Excavation and Tunneling Conference (Chicago, 1972). AIME, New York. 1972. pp. 1393-1441.

Continuous coring is also faster than conventional coring and frequently has a higher overall penetration rate than wireline coring. Continuous coring works particularly well when drilling in lost circulation or caving zones. This coring method requires a dual pipe string. The core is broken off in 6-inch or shorter lengths behind the bit and pumped to the surface by the drilling fluid.

Steel shot or calyx coring has been used in the past to core-drill large diameter vertical holes and shafts to relatively shallow depths. This method utilized small steel balls, circulated with the drilling fluid, as the grinding medium. At present, this technique does not appear suitable for drilling long horizontal holes in rock and soil.

The air or hydraulic hammer is a reciprocating tool in which an anvil activated by compressed air strikes the back of the bit. Percussion drilling tools which use air pressure, or hydraulic pressure, to drive a reciprocating bit against the rock have been used for drilling small diameter holes in hard rock at shallow depths. Out-of-hole percussion drills have a maximum depth limitation of around 300 to 400 feet. In-hole air-driven percussion drills would probably be limited to a maximum depth of about 3,000 feet.

Magnetostriction tools use a pulsating magnetic field which causes magnetostrictive materials to vary the length of the tool at the same frequency as that of the magnetic field. The resultant vibration is then transmitted directly to the bit to assist in rock failure. This technique has been successfully field tested.

The solenoid, hammer-type percussion tool has solenoid coils which raise and lower a solenoid armature inside the coils. The solenoid armature terminates in a hammer at its lower end. The changing field causes the armature-hammer combination to repeatedly strike an anvil which is attached to a rock bit. This technique has not been successfully field tested.

The eccentric-weight percussion tool uses drilling mud as an activator. The mud is pumped down the drill string and through an axial-flow mud turbine located within the bottomhole apparatus. The output shaft of the mud turbine is attached to eccentric weights and although there is no lateral unbalance, there is vertical unbalance. The lower end of the case containing the eccentric weights is attached to a long section of drill collars which transmit motion directly to the bit, and the resultant vibratory energy is thus transmitted to the rock. This technique has not been successfully field tested.

The explosive drilling method utilizes explosives in capsule or liquid form which are delivered to the bottom of the hole through a drill pipe. The Russians have extensively tested explosive drilling. One model using capsule explosives developed 68 horsepower. The bottom point of the drilling fluid circulation had to be some distance above the bottom of the hole to prevent dilution and washing away of the explosive, and this resulted in poor bottomhole cleaning. Explosive drills are not greatly affected by rock strength. However, they are least effective in clay and weak rocks that yield plastically. The high cost of explosive charges in capsule form, emplacement problems, rock removal problems, and the problem of keeping the hole in gauge combine to make this method unattractive.

The compression of rock to create a hole in the earth is in the conceptual stage only. It has been visualized as a continuous penetrator with a conical nose. The penetrator would be thrust ahead through the rock, crushing the rock ahead of it and pushing it aside. Data from impact tests indicate that thrusts of 2 to 10 million pounds would be required to push an 8-inch diameter penetrator through sedimentary rock of average strength. This method is not practical at this time.

Drilling by stress relief is still only a concept. This method requires the rock to be exposed to a vacuum environment. Rock disintegration would then occur as the stresses trapped within the rock formation are released. This method would be restricted to rock that is not subject to plastic flow.

Implosions to drill a hole has been visualized as pumping hermetically sealed air capsules to the bottom of the hole and breaking them against the rock. The low power output of implosion drills would seriously limit their application. For example, an implosion drill pumping 1,000 capsules each hour (4-inch diameter capsules) into a 10,000-foot deep hole filled with water would have a power output of only 6 horsepower. The high cost of the capsules and the low horsepower delivered to the bottom of the hole makes this method impractical.

Sonics or ultrasonics, and vibration are combined into one technique to remove rock by abrasion and cavitation. These drills use magnetostrictive or electrostrictive cores to vibrate emitters at frequencies of 20,000 to 30,000 cycles per second. The ultrasonic drill has primary application for cutting ceramics and hard alloys. The Russians have experimented with sonic drills operating in the 20 to 30 kilocycles per second frequency range. The high specific energy requirements and low power output are present disadvantages of this method.

Vibration, using compressed air as the source of energy, has been successfully used to bore shallow horizontal holes in soft, homogeneous soils. As the pneumatic borer advances in the hole, it compacts the soil, thus eliminating the material removal problem. At present, this method is only feasible for boring relatively shallow horizontal holes.

Spark percussion or electro-hydraulic drills use high energy sparks to break and remove rock from the hole. Sparks are produced by high energy capacitors which can produce pressure pulses in excess of 100,000 psi. Spark drill tests have been conducted in air and in gas environments. However, information is not available for any tests conducted in liquids with a hydrostatic head. This tool is limited by its power output.

Some rocks spall or eject chips from their surface due to high thermal stresses when they are rapidly heated. The thermal stresses are brought about by differences in the thermal expansion of the constituent minerals, heating of liquid or gaseous inclusions, phase changes in minerals, removal of water of crystallization, and chemical reactions. Although thermal spalling is an effective method of drilling highly spallable rocks, this method of causing rock disintegration has limited application because only a few rocks will spall.

Flame drills use a high temperature combustion source to spall rock. Two types of downhole apparatus are used. One utilizes a drill pipe rotated from the surface in a conventional manner while the other utilizes a bottomhole apparatus tied to a wireline and operated in much the same fashion as a cable tool bit. Both types require oxygen and a fuel, as well as water, to be delivered to the bottomhole apparatus. The oxygen and fuel are burned in a chamber which is cooled by the water. Mechanical reamers have been used to insure that a gauge hole is drilled and to assist in removing the spalled rock.

Flame drilling has been used commercially in the United States for several years to drill blast holes in granite and taconite. The use of flame drills has been restricted to depths of 50 to 150 feet in an atmospheric environment. Although these drills use a cheap fuel and deliver a large amount of power to the rock, only under extremely difficult drilling conditions can the high fuel consumption be economically justified.

Electric drills use the same basic concept as flame drills to disintegrate rock, except the localized high temperatures are produced by electric energy. For one model, a bit is used as an electrode. The bit is rotated to maintain a gauge hole and is cooled by air injection. This model has been successfully used for drilling taconite in a dry environment. An experimental model uses two electrodes enshrouded by a high pressure gas-filled chamber (for protection from liquid in the hole) and a feed-off mechanism to advance the electrodes as they are consumed. Development work on this model has become inactive because of the electrode consumption problem, the necessity for shielding the electrodes, and the problem of conducting electrical power down hole. This method is not considered practical nor economical for drilling long horizontal small diameter holes.

A thermal drill called Terra Jetter heats the rock to between 500° and 1000°F. with superheated steam and then sprays liquid nitrogen at -210°F. against the heated rock to thermally shock and pulverize it. No drilling tests have been reported for this drill.

Cryogenics as a drilling method is still a concept. For this method, some medium would be used to cool the rock to create thermal stresses and cause spalling. It does not have the potential for creating either the temperature differentials or the assoicated thermal stresses which are possible with heating. Cryogenics have been used for wall stabilization in shaft construction during mining operations and might be used in the same manner to assist in mechanical drilling, but used alone it is an impractical drilling method.

Most rocks fuse at temperatures between 2000° and 4000°F. The devices that are capable of fusing holes in rocks can also produce spalling when they encounter spallable rocks since spalling occurs at a lower temperature than does fusion. Fusion devices have the advantage of being able to fuse holes in any rock, but they also have the disadvantage of requiring very high energy (50,000 to 60,000 foot-pounds per cubic inch) levels.

Electric fusion has been successfully used to bore shallow holes in rock and soil. The most successful electric fusion device to date was developed by the Los Alamos Scientific Laboratory (LASL) at Sandia, New Mexico. This boring device, called a subterrene, generates sufficient heat to melt all types of rock and soil. Depending on the material penetrated, it may or may not be necessary to remove material from the hole. In soil and soft rocks, all of the molten material is displaced into the wall of the hole by densification. In hard, dense rocks such as granite, it is necessary to remove the molten rock, and possibly a core, from the hole. Continued research and development could result in this technique becoming a practical penetration method.

Electron beam devices use a high electrical potential to generate a source of electrons which are focused into a beam by means of a bias grid and a focus lens. The directional electron beam is focused against a rock where the beam dissipates its kinetic energy into the shallow surface layer of the rock. The power concentration of the electron beam is great enough to fuse holes in any rock. With the use of the electron beam, the melt-cut mode of operation is not restricted by rock characteristics such as hardness, texture, and thermal properties. Early devices required a vacuum environment to prevent gas molecules from scattering the beam, but a newer model can operate at atmospheric pressure. Electron beams have been used to drill small holes in rubies and other minerals, but large scale drilling tests have not been reported in the literature. The large energy requirements and the required vacuum or atmospheric-pressure environment makes this method impractical for boring long horizontal holes.

Lasers produce coherent light beams with power concentrations in excess of one billion kilowatts per square centimeter which can fuse holes in any rock. The power output of one of the largest crystal lasers produces an average power output of only 50 watts. However, gas lasers of 10 to 20 kilowatts are being developed. The power of gas lasers is directly proportional to their length. Lasers would have a low rate of penetration because of their high energy requirements (50,000 to 60,000 foot-pounds per cubic inch) and low power output. This fact coupled with the delicacy and large size of its required components precludes using this method for boring long horizontal holes.

The use of atomic energy to fuse or vaporize rocks is still in the conceptual stage. It is not anticipated that the state-of-the-art will be sufficiently developed within the next five years to consider nuclear energy as a means of drilling long horizontal holes in rock and soil.

Chemical drilling has been performed in the laboratory using fluorine and other highly reactive chemicals to drill through sandstone, limestone, and granite. Rock softeners in the form of "hardness reducer" type chemicals have also been tried in laboratory tests to increase drilling rates. Increases in drilling rates have been observed, but the reason is not fully understood. The difficulty of handling large quantities of highly reactive chemicals, toxic gaseous-by-products, and the high cost of the chemicals themselves make this method economically unattractive.

Factors to be Considered--The following are the major factors to be considered for the rock disintegration subsystem to penetrate soils and rocks over horizontal distances up to one mile.

<u>Hardness of Material Penetrated</u>--The material disintegration subsystem must satisfactorily penetrate the material in which the hole is to be located. The compressive strength of the material has been used for this factor consideration.

<u>Is Currently an Operational Technique</u>--This factor indicates which material disintegration methods have advanced beyond the laboratory stage and been field tested in vertical or horizontal holes.

<u>Longest Horizontal Hole Drilled to Date</u>--This information indicates the maximum horizontal depth drilled by each penetration technique.

<u>Circulation Medium Required</u>--The circulation medium required to transport the cuttings, if any, to the surface must be known. This factor indicates which mediums, or environments are necessary for satisfactory use of each penetration technique.

<u>Estimated Maximum Depth Capability</u>--Here, based on current available knowledge, we indicate the maximum horizontal depth each disintegration technique has for penetrating soil and rock.

Acceptable Deviation Control--This factor shows which material disintegration techniques appear to be capable of sufficient vertical and horizontal control to accurately penetrate horizontal distances of one mile.

Type of Sample Recovered--This information is useful in selecting a penetration technique so this factor indicates the type of samples which can be obtained with each penetration technique. For instance, if it is necessary to obtain a core for providing satisfactory geological information then it is obvious that a coring technique will have to be used. If coring could be conducted on an intermittent basis, then some other material disintegration process can serve as the primary penetration technique.

Can Cope With Groundwater Inflow--Anticipated groundwater inflows will influence what penetration technique is selected. This factor indicates which penetration techniques can be used when groundwater inflows are encountered.

Compatible With Remote Guidance or Geophysical Systems--These factors show which penetration techniques can be used in conjunction with remote guidance and geophysical systems.

Future Potential for One-Mile Long Holes--There are several horizontal penetration techniques which are definitely limited in their ability to drill horizontal holes up to one mile long. This factor classifies the various penetration techniques as to their potential for drilling horizontal holes to depths of one mile.

Can Cope With Moderate Lost Circulation Problems--If lost circulation zones are encountered, it will be necessary to bridge them off with the aid of lost circulation materials added to the circulation medium. It may also be necessary to inject chemicals or cement grouts. This factor is used to show which penetration techniques are usable when moderate lost circulation problems are encountered.

Screening of Material Disintegration Components--The comparison made of the various penetration techniques against the major factors described above is shown in Figure 28. Careful interpretation of Figure 28 shows that numerous components of the rock disintegration subsystem can be immediately eliminated from further consideration. The reasons used for elimination include:

MATERIAL DISINTEGRATION COMPONENTS	HARDNESS OF MATERIAL PENETRATED (PSI)				IS CURRENTLY AN OPERATIONAL TECHNIQUE	LONGEST HORIZONTAL HOLE DRILLED TO DATE (FT)				CIRCULATION MEDIUM REQUIRED			ESTIMATED MAXIMUM DEPTH CAPABILITY (FT)				ACCEPTABLE DEVIATION CONTROL	TYPE OF SAMPLE RECOVERED			CAN COPE WITH GROUND WATER INFLOW	COMPATIBLE WITH REMOTE GUIDANCE SYSTEM	COMPATIBLE WITH REMOTE GEOPHYSICAL SYSTEM	HAS FUTURE POTENTIAL FOR ONE MILE LONG HOLES	CAN COPE WITH MODERATE LOST CIRCULATION PROBLEMS
	TO 8000	8000 TO 16000	16000 TO 32000	32000 PLUS		0 TO 500	500 TO 1000	1000 TO 3000	3000 TO 5500	AIR	WATER	MUD	0 TO 500	500 TO 1000	1000 TO 3000	3000 TO 5500		SLUDGE	CHIP	CORE					
A. ROTARY DRILLING CONVENTIAL NON-CORING																									
1. TOOTHED ROLLING CUTTER		•			•					•	•	•				•	•	•	•		•	•	•	•	•
2. BUTTON ROLLING CUTTER			•		•		UNKNOWN				•	•				•	•	•			•	•	•	•	•
3. BLADED DRAG BITS	•				•		•			•	•	•				•	•	•	•		•	•	•	•	•
4. DISC OR KERF CUTTERS	•						UNKNOWN			•	•	•		TOO LARGE FOR HOLE				•	•		•	•	•		•
5. PLUG BITS, (DIAMOND, CARBIDE)				•	•	•					•	•			•		•	•			•	•	•		•
6. AUGER	•				•	•							•						•			•	•		•
7. EARTH SCREW	•				•	•								•					•			•	•		•
8. ROCK SCREW		•					UNKNOWN							•					•		•	•			•
9. ZUBLIN BIT		•								•	•	•		UNKNOWN				•	•		•	•	•		•
10. CONICAL BORER			•								•	•		UNKNOWN			•	•			•	•	•	•	•
11. HYDRAULIC JET BIT				•							•	•				•	•	•	•		•	•	•	•	•
12. RETRACTABLE DRILL BIT				•	•		UNKNOWN				•	•		UNKNOWN			•	•	•		•			•	•
B. ROTARY DRILLING CORING																									
1. CONVENTIONAL				•	•			•			•	•			•		•	•		•	•	•	•		•
2. WIRELINE				•	•				•		•	•				•	•	•		•	•			•	•
3. CONTINUOUS				•	•		UNKNOWN				•	•				•	•	•	•	•	•			•	•
4. STEEL SHOT		•									•	•			•			•		•	•				

Figure 28. Evaluation of material disintegration components.

MATERIAL DISINTEGRATION COMPONENTS	FACTORS TO CONSIDER																								
	HARDNESS OF MATERIAL PENETRATED (PSI)				IS CURRENTLY AN OPERATIONAL TECHNIQUE	LONGEST HORIZONTAL HOLE DRILLED TO DATE (FT)				CIRCULATION MEDIUM REQUIRED			ESTIMATED MAXIMUM DEPTH CAPABILITY (FT)				ACCEPTABLE DEVIATION CONTROL	TYPE OF SAMPLE RECOVERED			CAN COPE WITH GROUND WATER INFLOW	COMPATIBLE WITH REMOTE GUIDANCE SYSTEM	COMPATIBLE WITH REMOTE GEOPHYSICAL SYSTEM	HAS FUTURE POTENTIAL FOR ONE MILE LONG HOLES	CAN COPE WITH MODERATE LOST CIRCULATION PROBLEMS
	TO 8000	8000 TO 16000	16000 TO 32000	32000 PLUS		0 TO 500	500 TO 1000	1000 TO 3000	3000 TO 5500	AIR	WATER	MUD	0 TO 500	500 TO 1000	1000 TO 3000	3000 TO 5500		SLUDGE	CHIP	CORE					
C. ROTARY DRILLING, PERCUSSION																									
1. AIR HAMMER, OUT-OF-HOLE				•	•	•				•			•						•						
2. AIR HAMMER, IN-HOLE				•	•		•			•					•				•			•	•		
3. HYDRAULIC HAMMER, IN-HOLE				•							•	•				•		•	•		•	•	•	•	•
4. MAGNETOSTRICTION HAMMER, IN-HOLE		UNKNOWN								•	•	•				•		•	•		•	•	•	•	•
5. SOLENOID HAMMER, IN-HOLE		UNKNOWN								•	•	•				•		•	•		•	•	•	•	•
6. ECCENTRIC-WEIGHT HAMMER, IN-HOLE		UNKNOWN								•	•	•				•		•	•		•	•	•	•	•
D. EXPLOSIVE DRILLING				•							UNKNOWN			UNKNOWN			UNKNOWN		•		UNKNOWN			UNKNOWN	
E. COMPRESSION PENETRATOR		UNKNOWN									NONE								NONE		•	•	•		
F. STRESS RELIEF BY VACUUM										•									•		UNKNOWN	•	•		UNKNOWN
G. IMPLOSION DRILLING										•	•	•							•		UNKNOWN				UNKNOWN
H. SONICS OR ULTRASONICS		UNKNOWN								•	•	•		UNKNOWN			UNKNOWN		•			•	•	UNKNOWN	•
I. VIBRATION, AIR-POWERED DRILLING	•				•	•					NONE		•						NONE			•	•		•
J. ELECTRO-HYDRAULIC OR SPARK DRILLING				•						•	UNKNOWN			UNKNOWN			UNKNOWN		•		UNKNOWN	•	•	UNKNOWN	UNKNOWN
K. FLAME SPALLING				•	•					•			•						•			•	•		•
L. ELECTRIC SPALLING				•						•					•				•			•	•		•
M. THERMAL STEAM				•						•					•				•			•	•		•
N. CRYOGENIC DRILLING				•							UNKNOWN			UNKNOWN			UNKNOWN		•			•	•		•
O. ELECTRIC FUSION SUBTERRENE				•						•	NONE OR						•		NONE OR •	•		•	•		•
P. ELECTRON AND LASER BEAM, FUSION AND VAPORIZATION				•						•	NONE OR						UNKNOWN		NONE OR •			•	•		•
Q. NUCLEAR REACTOR FUSION				•						•	NONE OR						•		NONE OR •	•		•	•		•
R. CHEMICAL ACID				•							UNKNOWN							•	•			•	•		UNKNOWN
S. CHEMICAL ROCK SOFTENERS				•							UNKNOWN			UNKNOWN				•	•			•	•		UNKNOWN
T. STEEL PROJECTILES				•															•					UNKNOWN	UNKNOWN

Figure 28. Evaluation of material disintegration components (continued).

- The component has not yet been fully developed nor will it be an operational technique in the near future.
- The component is unable to penetrate horizontally for one mile.
- It is not possible to control the horizontal and vertical deviation satisfactorily.
- It cannot be used in the presence of groundwater inflows.
- The disintegration component cannot at this time be scaled down to drill a small diameter hole.
- The disintegration component has too slow an advance rate to be economically feasible.

The rock disintegration components eliminated from further consideration in this study were:

Disc or Kerf Cutters
Auger
Earth Screw
Rock Screw
Zublin Bits
Conical Borer
Conventional Core Drilling
Steel Shot Core Drilling
Air Hammer, Out-of-Hole
Air Hammer, In-Hole
Hydraulic Hammer, In-Hole
Magnetostriction Hammer, In-Hole
Solenoid Hammer, In-Hole
Eccentric Weight Hammer, In-Hole
Explosive Drilling
Compression Penetrator
Stress Relief by Vacuum
Implosion Drilling
Sonic or Ultrasonic Drilling
Vibration, Air-Powered Drilling
Electro-Hydraulic or Spark Drilling
Flame Spalling
Electric Spalling
Thermal Steam
Cryogenic Drilling
Electric Fusion "Subterrene"
Electron & Laser Beam Fusient Vaporization
Nuclear Reactor Fusion
Chemical Acid
Chemical Rock Softeners
Steel Projectiles

Of these rock disintegration components which have been dropped, the following show some promise for possible development into a long hole drilling technique. These are:

Conical Borer
Hydraulic Hammer, In-Hole
Magnetostrictive Hammer, In-Hole
Solenoid Hammer, In-Hole
Eccentric Weight Hammer, In-Hole
Sonic or Ultrasonic Drilling
Electro-Hydraulic or Spark Drilling
Electric Fusion "Subterrene"

The remaining rock disintegration components, which can be considered viable candidates for a successful horizontal penetration system, are listed below. Based on an evaluation of all of the pertinent factors, these surviving components of the rock disintegration subsystem should stand a good chance of becoming a part of an effective penetration system in the near future.

1. Rotary Drilling, Conventional Non-Coring
   - Tooth Rolling Cutter
   - Button Rolling Cutter
   - Bladed Drag Bit
   - Plug Bit
   - Hydraulic Jet Bit
   - Retractable Drill Bit
2. Rotary Drilling, Coring
   - Wireline
   - Continuous Core, Dual String

MATERIAL REMOVAL SUBSYSTEM

The relative merits of all known methods for removal of the material fragments created by the excavation subsystem were considered and evaluated. These transporting methods were grouped into the following six basic categories:

1. Pneumatic
   - High Pressure Direct (Air Lift)
   - Vacuum
   - Foam
2. Hydraulic
   - Water
   - Mud
   - Other Liquid
3. Mechanical
   - Augers
   - Core, Conventional
4. Flotation
5. Explosive

6. Downhole Disposal
    Compression
    Fusion
    Vaporization
    Hydraulic Fracturing

Pneumatic--The pneumatic method is used for removing material from all sizes of boreholes where only very small quantities of water are encountered. This method depends on the velocity of air to remove cuttings from the bit area and transport them out of the hole. The pressure differential required to move the cuttings out of the hole can be obtained by either compressing air and forcing it through a circulating system, or by pulling a vacuum on the return loop of the circulating system.

Hydraulic--This method is used extensively for removing cuttings from drilled holes. It depends on the velocity of a liquid medium to lift the cuttings from the bottom of the hole and then carry them out of the hole. The pressure differential necessary to produce an adequate velocity for removing the cuttings can be obtained in a number of ways. Direct positive displacement pumping, compressed air, vacuum, and an unbalanced liquid column are all common practices.

Mechanical--Mechanical methods of removing cuttings from a small diameter, one mile long, horizontal borehole would be quite limited in effectiveness. Augers are limited to relatively short, straight holes. Conventional diamond coring has much greater depth capabilities, but is economically impractical due to the time consumed by the numerous trips required for core removal. On the other hand, wireline core removal techniques could be economically practical for material removal. The wireline coring method combines hydraulic removal of the cuttings with mechanical removal of the resultant core through the core barrel.

Flotation--This process depends on a differential specific gravity between the material to be moved and a carrying fluid. Because the undesirable effects of a weighted circulating system exceed the advantages that could be obtained, this concept is not practical.

Explosive--For rock removal, explosives require the use of violent expansive forces. This concept is not practical due to the equipment requirements and personnel safety.

Downhole Disposal--As a means of rock disposal, downhole disposal requires that the material displaced from the borehole remain under ground. At present, this concept is not practical for long horizontal boreholes.

The material removal method that is ultimately selected is largely dependent on what rock disintegration method is used and on the expected geological conditions to be encountered. The rock disintegration component(s) selected are in turn a compromise that has been reached after considering all of the obstacles to penetration which must be overcome.

After carefully reviewing the literature, the following components of the material removal subsystem can be eliminated from further consideration for the near future:

High Pressure Direct (Air Lift)
Vacuum
Augers
Core, Conventional
Flotation
Explosive
Compression
Fusion
Vaporization
Hydraulic Fracturing.

Considerable research and development is going to be required to develop any of these material removal methods into workable methods. Of these, the fusion method appears to have the most potential. In relatively porous soil and rocks, it has been found possible to dispose of the molten rock from a high temperature subterrene by densification of the molten material into the wall of the hole.

The following material removal components were, therefore, selected as the best possible available choices for use in the near future:

1. Pneumatic
   - Foam
2. Hydraulic
   - Water
   - Mud
   - Other Liquid
3. Hydraulic - Pneumatic
   - Air Assist
4. Hydraulic - Mechanical
   - Core, Wireline.

Thus, there are only four major material removal components at present that can be considered as candidates for a successful horizontal penetration system. These are:

1. Pneumatic
2. Hydraulic
3. Hydraulic - Pneumatic
4. Hydraulic - Mechanical

Figure 29 shows the compatability between the surviving candidate rock disintegration methods and the possible material removal methods as regards horizontal exploratory holes bored one mile long. Figure 29 indicates that only the hydraulic material removal components are compatible with all of the candidate material disintegration components. This is true for small diameter, one mile long horizontal holes. A literature review indicates that high pressure compressed air is not effective in small diameter holes deeper than 3,000 feet. The use of foam would, therefore, probably not be effective in holes much deeper than 3,000 feet. Foam and air assist is not compatible with coring or the use of the jet bit. The air assist method would probably have its greatest application in removing cuttings from long horizontal holes which are collared from a near vertical, surface set-up. The core recovery material removal methods are, of course, only applicable to core drilling techniques. The coring methods, however, may be required when drilling through zones susceptible to caving or lost circulation, or through very hard rock.

ENERGY TRANSMISSION SUBSYSTEM

Every penetration technique requires an energy source to provide thrust and/or rotation to the soil or rock disintegration device. In addition, a means of transferring this energy to the rock disintegration device must be provided. Also, a means must be provided to move the penetration device in and out the borehole.

All of the known and available modes for transferring thrust and torque (if required) to the rock penetration system were investigated. Consideration was also given to methods of moving the rock penetration tools in and out of the borehole. The various energy transmission methods were grouped into two main categories as follows:

1. Out-of-Hole Power Source
   - Rigid drill pipe, single string
   - Rigid drill pipe, dual string
2. In-Hole Power Source
   - Percussion drill, rigid drill pipe with independent out-of-hole rotation
   - Mud motor, rigid drill pipe, independent out-of-hole thrust
     - Positive displacement (Dyna-Drill)
     - Turbine (Turbo-Drill)
   - Electric motor, rigid drill pipe, independent out-of-hole thrust (Electro-Drill)
   - In-hole, self-advancing, thruster providing thrust to in-hole motor.

	Material Removal Components						
	Pneumatic	Hydraulic				Hydraulic Pneumatic	Hydraulic Mechanical
Material Disintegration Components	Foam	Water	Mud	Other Liquid	Core Continuous	Air Assist	Core Wireline
Toothed Rolling Cutter		●	●	●		●	
Button Rolling Cutter	●	●	●	●		●	
Bladed Drag Bits	●	●	●	●		●	
Plug Bits	●	●	●	●		●	
Hydraulic Jet Bit		●	●	●			
Retractable Drill Bits	●	●	●	●		●	
Wireline Coring		●	●	●			●
Continuous Coring		●	●	●	●		

Figure 29. Compatibility of material disintegration and material removal subsystem components.

Conventional sources of energy are up-hole power units. Conventional means of transferring this energy are rigid steel drill pipes. Several types of in-hole motors are newer sources of energy for providing rotation. At least one type of in-hole, self-advancing, hydraulic thruster is available for providing thrust to the in-hole motor in lieu of a rigid drill pipe and out-of-hole source of thrust.

The in-hole motors can provide rotation to the rotary rock disintegration device, while the thrust must be provided by the up-hole power source through a rigid drill pipe. The recent advent of an in-hole hydraulic thruster has provided a potentially new technique for applying thrust to an in-hole motor. This in-hole thruster removes the requirement for a rigid drill string as the thruster is remotely operated through flexible hydraulic hoses from the surface.

Rotary percussion drilling techniques require either an up-hole power source to provide the percussive force through the drill string to the bit or an in-hole percussion device. The in-hole percussion device requires independent up-hole rotation through the rigid drill string.

The method chosen to provide thrust, rotation or move the material disintegration device into and out of the borehole will depend on the penetration techniques selected. It may be found expedient to use a combination of several power sources while drilling a long horizontal hole. For instance, conventional rotary drilling, using a three-cone roller bit may be the primary penetration technique used. Then, in order to correct adverse hole deviation, a downhole motor and bent sub may be used. Or, an in-hole motor powered rock disintegration device may be used to extend a long horizontal hole in order to overcome the high torques required for rotary drilling.

Figure 30 shows the interrelationship between the candidate material disintegration subsystem components and the candidate energy transmission subsystem components. The rotational and thrust forces applied to the majority of the candidate rock disintegration subsystem components is through a rigid drill pipe with the power source on the surface.

There are still some limitations on the use of in-hole motors. Past experience, mostly from drilling vertical boreholes, has shown that maintenance on the in-hole motors can be a severe problem. Also, there could be some problems with cuttings removal in a deep horizontal hole as the drill rods would not rotate and thus assist in removing the cuttings from the hole. The problem of removing cuttings from the borehole also is a potential source of difficulty for the in-hole thruster. The trailing hydraulic hoses and power lines could act as traps for the cuttings, and possibly make it difficult to pull the hydraulic thruster and rock disintegration device from the borehole.

	Energy Transmission Components				
	Out-of-Hole Power Source		In-Hole Power Source		
Material Dis-integration Components	Rigid Drill Pipe, Single String	Rigid Drill Pipe, Dual String	Mud Motor, Rigid Drill Pipe, Out-of-Hole Thrust	Electric Motor, Rigid Drill Pipe, Out-of-Hole Thrust	In-Hole Thruster, with In-Hole Motor
Toothed Rolling Cutter	●	●	●	●	●
Button Rolling Cutter	●	●	●	●	
Bladed Drag Bits	●	●	●	●	●
Plug Bits	●	●	●	●	●
Hydraulic Jet Bit	●				
Retractable Drill Bits	●				
Wireline Coring	●				
Continuous Coring		●			

Figure 30. Compatibility of material disintegration and energy transmission subsystem components.

A reliable and accurate borehole guidance technique must be used to drill a long horizontal exploratory hole along a predetermined path to a target. The path the borehole follows must be closely controlled so that geological and geophysical data obtained from the hole will be representative of conditions to be encountered along the proposed tunnel alignment.

Over the years, numerous methods of measuring borehole inclination and bearing and axial orientation of the drilling assembly have been developed. Most of these early survey methods were designed to work in vertical or near vertical holes, but in the last few years several have been developed for use in holes whose inclination angle was horizontal or above the horizontal.

The borehole guidance or survey techniques that are currently available can provide one or more of the following directional data: (1) inclination only; (2) horizontal bearing angle only; (3) inclination and bearing; (4) inclination, bearing, and axial orientation of the drilling assembly.

To accurately directionally drill horizontal holes as long as a mile, it is necessary that the guidance technique used be capable of recording borehole inclination and bearing and axial orientation of the drilling assembly. The inclination angle, bearing angle and borehole depth can be used to calculate and plot horizontal and vertical projections of the borehole trajectory. The axial orientation of the drilling assembly must be known to accurately guide the borehole.

Figure 31 is a comparison of a few of the borehole survey techniques used and the survey data they will record. Included are a couple of new techniques that have not been extensively field tested. All of the survey methods that have been developed and used have not been listed. In general, only the more common methods are noted. Survey techniques which will only work in a vertical or inclined hole are not described.

Many survey devices will only produce one reading per survey. These are called single shot devices. Others can record many hundreds of readings photographically on film or electronically using a cable or cableless readout. These are often referred to as multishot or continuous readout survey instruments.

Many of the survey devices can be pumped down the inside of the drill pipe and retrieved on a small diameter wireline. Others must be run in and out of the hole on the end of the drill pipe. The telemetry survey instrument is run into the hole as an integral part of the drilling assembly. In the future, it may be possible to run the survey steering tool in the hole as a part of the drilling assembly when it is

BOREHOLE GUIDANCE TECHNIQUE	INFORMATION OBTAINED		
	INCLINATION	BEARING	AXIAL ORIENTATION
Hydrofluoric Acid Etch Tube	●		
Photographic Inclinometer Devices	●		
Electrical Readout Inclinometer Devices	●		
Kiruna Method	●		
Manometer	●		
Gelatin Tube		●	
Radioactive Paint		●	
Mechanical Locking Devices		●	
Mechanical Gimbal-Mounted Device	●	●	
Maas Compass	●	●	
Steering Tool Device		●	●
Single Shot Gimbal-Mounted Compass	●	●	●
Multishot Gimbal-Mounted Compass	●	●	●
Survey Steering Tool	●	●	●
Telemetry Device	●	●	●
Gyroscopic Device	●	●	●

Figure 31. Survey data obtained by borehole guidance techniques.

used with an in-hole mud motor. In this case, the electrical conductor would be placed in the annulus between the drill pipe and rock.

Whenever a magnetic compass is used, it must be located in a section of nonmagnetic drill pipe. This also applies to the survey steering tool and the telemetry survey device.

Today, most borehole survey instruments can be purchased or rented and operated by the drilling contractor. However, regarding the gyroscope, survey steering tool and when fully developed, probably the telemetry survey tool, these instruments can only be rented and must be operated by qualified service personnel.

Description of Guidance Methods--The following is a brief description of the various borehole survey devices listed in Figure 31.

<u>Inclinometers</u>--The hydrofluoric acid etch tube consists of a small glass tube partly filled with hydrofluoric acid.[49] The tube is run into the hole and brought to rest. After 15 to 20 minutes it is withdrawn. The line etched on the wall of the glass tube is then used to determine the vertical angle of the hole.

There are numerous photographic inclinometer devices that are either single or multishot. These devices take advantage of the force of gravity, and work on mechanical principals which are not affected by magnetism and therefore can be run inside steel pipe or casing. Sperry-Sun Well Surveying Company and Eastman Industries, Inc. manufacture reliable inclinometers.

Existing electrical readout inclinometer devices are complicated and the data must be sent to the surface through an electrical conductor.

The Kiruna inclinometer is another chemical method of determining inclination. With this technique, metallic copper deposits out onto a small strip of iron from a copper solution. The line formed by the top of the copper deposit is then used to determine the inclination angle.

The manometer method can only be used for horizontal holes that continue to rise above the level of the collar of the hole.[50] The

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[49] Paone, James, William E. Bruce, and Roger J. Morrell. <u>Horizontal Boring Technology: A State-of-the-Art Study</u>. U.S. Bureau of Mines Information Circular 8392. September, 1968.

[50] Baxter, J. S. "Drilling of Long Boreholes in Coal." <u>Colliery Engineering</u> (December, 1959), pp. 520-525.

hydrostatic head of the drilling fluid inside the drill pipe is measured on a pressure gauge or manometer. This method is limited in that it will not indicate the magnitude of the vertical angle and it will not work when the hole falls below the horizontal.

Directional Devices Measuring Bearing Only--The melted gelatin method consists of a small glass tube filled with liquid gelatin and containing a small open, floating magnetic compass.[51] The device is run into the hole and after a period of time at rest the gelatin congeals and fixes the compass needle in position.

Another method utilizes radioactive paint on the tip of a compass needle. The compass needle reading is picked up on photographic film.

A third device mechanically locks the compass needle in position after a timed interval.

Directional Devices Measuring Bearing and Inclination--A mechanical, gimbal-mounted single shot survey device, the Tro-Pari, has been available for some time. This device consists of a plumb bob and compass mechanism which uses a time lock to lock the compass and inclinometer after a predetermined time. The compass readings are not reliable in a ferromagnetic environment.

The Maas compass is a combination of the hydrofluoric acid etch tube and the gelatin tube described above.[52] Using this single shot device, both the bearing and inclination can be recorded at the same time.

There are several steering tool devices such as manufactured by Sperry-Sun Well Surveying Company and Scientific Drilling Controls, Inc. These instruments can continuously record the bearing angle and the axial orientation of the drilling assembly. These techniques were developed as an aid in maintaining the orientation of in-hole deviation tools such as the mud motor Dyna-Drill. A disadvantage of these techniques is that they currently require an electrical conductor to send the data from the survey device in the hole to the surface readout equipment. In the future, it may be possible to send the survey data to the surface by utilizing a cableless telemetry signal system.

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[51] Paone, James, William E. Bruce, and Roger J. Morrell. Horizontal Boring Technology: A State-of-the-Art Study. U.S. Bureau of Mines Information Circular 8392. September, 1968.

[52] Fenix & Scisson, Inc. A Systems Study of Soft Ground Tunneling. Report DOT-FRA-OHSGT-231 for U. S. Dept. of Transportation, OHSGT and UMTA, Contract 9-0034. Tulsa, Oklahoma. May, 1970.

Directional Devices Measuring Bearing, Inclination And Axial Orientation--Probably the most common borehole survey instrument used today is the gimbal-mounted magnetic compass. A compass is gimbal-mounted on a moveable pedestal which acts as a pivot. The survey instrument can be designed to photographically record one shot or up to a thousand shots on film. In order to record magnetic bearing, the compass angle unit must be positioned in a section of non-magnetic drill pipe near the bit.

The survey steering tool is similar to the steering tool device described above except it will also record the inclination angles in addition to the bearing and axial orientation. This device also sends the data to the surface through an electrical conductor.

The telemetry survey device is a unique cableless survey tool that is located in the drill string, near the bit.[53] Whenever survey data is required, drilling is stopped and the survey information is sent to the surface through the drill pipe using a telemetry signal. The signal is computer processed and in less than one minute the location of the end of the hole is known. Telcom, Inc., McLean, Virginia has built and successfully tested a prototype in-hole telemetry survey instrument under subcontract to Fenix & Scisson, Inc.

To date, there are no known gyroscopic borehole survey instruments manufactured that will operate in a small diameter horizontal hole.[54] Slim hole gyrocompasses will only operate in holes whose inclination angle is about 70 degrees off vertical. The data can be recorded continuously on the surface by sending the signal through an electrical conductor. It can also be built as a photographic single or multishot device. A main advantage of the gyroscopic instruments is that they will operate in a magnetic environment.

Screening Of Subsystem Components--All of the borehole guidance subsystem components except for the following have been eliminated from further consideration as a viable candidate for directional control. The remaining components include:

Multishot gimbal-mounted magnetic compass
Survey steering tool device
Telemetry borehole survey device
Gyroscopic borehole survey device

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[53] Rubin, L. A. Survey System Design and Fabrication. Report for U.S. Bureau of Mines, Contract H0111355, Vol. 2, Telcom, Inc., McLean, Virginia. March, 1973.

[54] Eye, Aim, PMS Catalogue. Scientific Drilling Controls, Inc., Newport Beach, California. 1972-1973.

The other borehole guidance subsystems shown in Table 31 were eliminated due to one or more of the following reasons:

- They will not simultaneously record inclination angle, bearing angle and axial orientation of the drilling assembly.
- They are too time consuming to be practical.
- Their accuracy is not acceptable.
- Their reliability is not acceptable.

In order to accurately and rapidly drill a one mile long horizontal hole it will be necessary to survey often and plot the vertical and horizontal trajectory of the hole. Therefore, it is imperative that the survey method used be a multishot or continuous readout type to reduce the number of times the survey instrument must be used.

The magnetic compass multishot instrument is still the least expensive, most reliable, and fastest method available for running directional surveys in long horizontal boreholes. A gyroscopic multishot survey instrument, when developed for small diameter horizontal boreholes, would have a considerable advantage in a ferromagnetic environment over the other three magnetic survey instruments remaining. The telemetry survey instrument is still in the developmental stage, but holds considerable promise for future development. The survey steering tool can be a great aid when attempting to maintain the orientation of an in-hole mud motor during directional drilling work.

Figure 32 is a comparison of the various material disintegration components with the guidance components. Investigation of the matrix shows that neither the survey steering tool or the telemetry survey tool is compatible with the wireline or continuous coring penetration techniques. In addition, the survey steering tool is not compatible with the non-coring rotating drilling techniques. This is due to the need for having an electrical conductor from the steering tool to the surface.

The horizontal borehole guidance components are compatible with the remaining material disintegration components.

GEOLOGICAL INVESTIGATION SUBSYSTEM

The purpose of the horizontal borehole is to provide meaningful geological, hydrological, and other engineering data regarding the material penetrated. The information thus obtained can then be used to help in the design and selection of construction procedures and equipment for a highway tunnel.

Existing and contemplated geophysical techniques of borehole logging, when used in a horizontal borehole, could be quite useful in enabling the geologist and engineer to better predict subsurface conditions along a

MATERIAL DISINTEGRATION COMPONENTS		GUIDANCE COMPONENTS: MAGNETIC COMPASS MULTISHOT	SURVEY STEERING TOOL	TELEMETRY	GYROSCOPIC DEVICE
Coring	Wireline	●			●
	Continuous	●			●
Non-Coring Rotating Drill String	Toothed Rolling Cutter	●		●	●
	Button Rolling Cutter	●		●	●
	Bladed Drag Bits	●		●	●
	Plug Bits	●		●	●
	Hydraulic Jet Bit	●		●	●
	Retractable Drill Bit	●			●
Non-Coring Stationary Drill String	Toothed Rolling Cutter	●	●	●	●
	Button Rolling Cutter	●	●	●	●
	Bladed Drag Bits	●	●	●	●
	Plug Bits	●	●	●	●
	Hydraulic Jet Bit	●	●	●	●

Figure 32. Compatibility of material disintegration and guidance subsystem components.

proposed tunnel alignment. This geophysical information will supplement the geological data obtained by more direct means such as cores, chip samples, caliper surveys, changes in penetration rates and so on.

There are several problems which must be overcome to obtain geophysical information from a horizontal borehole. These include:

1. Getting the logging tool into the borehole. If the logging tool can be made an integral part of the drilling assembly, this problem is eliminated. Otherwise, it may be necessary to push the logging device into the hole with the drill pipe. If a wireline coring technique is used, it may be possible to pump the logging device down inside the drill pipe and out the end of the core barrel.

2. If the drilling technique requires the hole to be cased with steel casing due to unstable hole conditions, this could greatly reduce the number of geophysical logging techniques that could be used. At present some logging techniques such as acoustic methods will not work satisfactorily in a cased hole.

3. If a remote borehole geophysical logging device were developed to be used as an integral part of the drill string, it would not be possible to use it with a wireline or continuous coring technique or a replaceable bit technique of boring the hole.

4. Many existing geophysical logging techniques require that the hole be full of fluid. This will present problems for holes which are collared from a horizontal set-up.

A considerable amount of information can be obtained regarding the material penetrated by the horizontal borehole. This geological data can be obtained by either direct or indirect methods. The more direct methods include:

1. Core samples which can be tested in order to determine the material physical characteristics.
2. Chip and sludge samples.
3. Changes in penetration rates at a given thrust and revolutions per minute (rpm) which indicate changes in material hardness.
4. Changes in torque at a given thrust and rpm which can indicate changes in material properties.
5. Changes required in thrust at a given rpm in order to maintain a constant penetration rate.
6. Changes in color of drilling fluid returns.
7. Reduction or increase in volume of drilling fluid returns.

The indirect methods of obtaining geological information on the material penetrated include the various geophysical and other borehole logging techniques. Many geophysical logging methods that have been developed for logging vertical holes could be adapted for use in horizontal holes.

At present, there are few geophysical methods of borehole logging that will accurately determine the important physical soil and rock characteristics required for the design of underground excavations. Therefore, research to develop new and improved geophysical borehole logging techniques for tunnel design is required. This research should be directed towards the development of new and improved techniques for determining in situ geological, physical, and hydrological characteristics of the material penetrated by the horizontal borehole.

Figure 33 shows the types of information that can be obtained by several of the geophysical logging techniques which are available. Most of these logging techniques have been extensively used in vertical holes.

Geophysical logging of horizontal boreholes has been a relatively uncommon occurrence to date. Those which we have knowledge of include acoustic mapping, electromagnetic, sonar velocity, resistivity, natural gamma, and mechanical caliper logs.

SELECTION OF BEST POTENTIAL SYSTEMS

Analysis of the data presented in Figures 28, 29, 30, and 32 shows that the rotary drilling techniques, using either water or mud, are the prime candidates to consider at this time for drilling one mile long horizontal holes. Eventually, if R & D efforts are successful, other novel techniques may be developed to the point that they can be effective, but until that time comes, the rotary drilling techniques, both non-coring and coring, offer the best chances for successfully penetrating soils and rocks horizontally for distances of one mile.

The following are examples of rotary drilling assemblies that have been or could be used to drill long horizontal holes in soil and rock. To date, only assemblies A & B have been used to successfully directionally drill a straight horizontal hole longer than 3,000 feet and only assembly A has been used to drill a horizontal hole one mile long.

I. Existing Equipment and Technology Developed
   A. Standard rotary non-coring assembly
      1. Rotary bit designed for type and hardness of material to be penetrated
      2. Stabilization designed for maximum horizontal and vertical directional control
      3. Non-magnetic survey assembly
      4. Rigid drill pipe to surface
   B. Standard rotary wireline diamond coring assembly
      1. Rotary diamond coring bit
      2. Wireline coring assembly
      3. Reaming shells and stabilizers designed for maximum vertical and horizontal directional control

Borehole Logging Technique	Information Obtained												
	Elastic Properties	Fractures	Lithology	Structure	Porosity	Permeability	Water Content	Fluid Movement	Borehole Diameter	Formation Damage	Mineral Identification	Rock Deformation	Material Deformation
3-D Velocity	●	●	●	●	●						●	●	
Acoustical Imaging		●	●	●						●			
Seisviewer	●	●	●	●						●			
Velocity			●		●					●	●	●	
Wave Amplitude	●		●								●	●	
Gamma Ray			●	●							●		
Neutron			●		●		●				●		
Density (gamma-gamma)	●		●		●						●		
Nuclear Magnetism					●	●							
Single Point Resistivity			●			●							
Induction			●	●	●	●	●	●					
Spontaneous Potential			●					●					●
Borehole Gravimeter	●				●								
Micrology		●	●		●		●						

Figure 33. Horizontal borehole logging techniques and anticipated results.

Borehole Logging Technique	Information Obtained												
	Elastic Properties	Fractures	Lithology	Structure	Porosity	Permeability	Water Content	Fluid Movement	Borehole Diameter	Formation Damage	Mineral Identification	Rock Deformation	Material Deformation
SP Dipmeter				●						●			
Guard and Laterologs							●	●		●			
Caliper	●								●	●			
Borehole Television and Camera		●		●						●			
Borehole Sonar									●	●			
In Situ Extensometer	●												●
In Situ Pore Pressure					●	●	●						
In Situ Stress	●												●

Figure 33. Horizontal borehole logging techniques and anticipated results (continued).

4. Non-magnetic survey assembly
5. Rigid drill pipe to surface

C. Continuous coring assembly
1. Rotary diamond coring bit
2. Reaming shells and stabilizers designed for maximum vertical and horizontal directional control
3. Non-magnetic survey assembly
4. Rigid dual wall pipe to surface for continuous ejected core

D. In-hole motor
1. Rotary bit designed for hardness of material to be penetrated
2. In-hole, positive displacement mud motor with or without bent sub or bent housing
3. Optional stabilization
4. Non-magnetic survey assembly
5. Rigid drill pipe to surface

II. Equipment and Technology Not Fully Developed

A. In-hole motor with in-hole thruster
1. Rotary bit suitable for material penetrated
2. In-hole, positive displacement mud motor, with or without bent sub or bent housing
3. In-hole, self-advancing, hydraulic thruster (not fully developed)
4. Non-magnetic survey assembly or:
5. Remote guidance system, cableless telemetry
6. Hydraulic hoses and electrical conductors extending to surface

B. Retractable drill bit
1. Rotary, retractable two step drill bit designed for hardness of material to be penetrated. Bit is pumped down inside of drill pipe and retrieved on a wireline.
2. Stabilization designed for maximum horizontal and vertical control
3. Non-magnetic survey assembly
4. Rigid drill pipe to surface (drill pipe can also serve as casing if required)

The equipment and technology exists today for drilling small diameter horizontal holes up to one mile long in materials having compressive strengths up to 8,000 psi. The Japanese have rotary drilled at least one horizontal hole over a mile long in volcanic rocks. They used a conventional non-coring three-cone roller bit. Another horizontal hole 3,700 feet long was drilled at Mercury, Nevada, in volcanic rocks using wireline core drilling equipment.

There are several limiting factors which make it difficult to drill a long, straight, small diameter horizontal hole. They include:

1. Directional control is difficult. This is largely due to the influence of gravity on the horizontal drilling assembly. Generally, the influence of gravity tends to cause the hole to deviate downward. When rotary drilling, the clockwise rotation tends to deviate the hole to the right.

2. Penetration rates are low, and costs per foot are high, especially in the harder rocks. Due to space limitations, the drilling equipment is usually small when drilling from underground set-ups. This can result in not having enough horsepower to do the job right. Short lengths of drill pipe are generally handled and the drill rig feed strokes are usually short. This increases the total drilling time.

3. There is a great shortage of people who have the training and experience required to accurately drill long horizontal holes.

4. It is difficult to correct the adverse deviation of a drill hole once it has occurred. The relatively recent introduction of the in-hole motor with a bent sub as a hole deviation device has helped solve this problem.

5. When penetrating the harder materials, torque requirements may become excessive due to the higher thrusts required. This could result in drill string failures due to metal fatigue. As the thrust is increased, the drill pipe has more of a tendency to bow and develop bends. This results in frequent and hard contact between the drill pipe and wall of the borehole. As the hole becomes deeper, drag could increase until it exceeded the available thrust and torque. It is possible that drag could increase to the point where no amount of available thrust could force the drill pipe into the hole and still be able to cut the hard rock. Under those conditions, it would be necessary to use an in-hole thruster in conjunction with an in-hole motor.

There are several new techniques under development that could result in making it easier, faster, and less expensive to drill long horizontal boreholes in soil and rocks. Among these are:

1. Remote, cableless guidance or survey systems. Survey data is sent to surface through the drill string, providing continuous and instantaneous information on the borehole trajectory. A considerable savings would be realized in survey time and lost drilling time. Adverse

hole deviations could be detected before they became difficult to correct. Such a system has already been field tested with favorable results.

2. The use of an in-hole mud motor, having a bent housing and/or bent sub, to correct adverse horizontal hole deviation. This technique has had very little use, but appears to be a promising method for changing the direction of a horizontal hole.

3. The development of drill rigs having longer feed strokes. This will reduce drill rod handling time.

4. The continuous core drilling technique has not yet been used to drill long holes in either soil or rock. Based on vertical hole drilling experience, this method appears to be a promising technique for drilling lost circulation zones and friable, weak, plastic, or unconsolidated sand zones.

5. The development of a directional jet bit for drilling small diameter horizontal holes. The development of a jet bit utilizing high pressure water jets to cut soil and rock could greatly speed up penetration rates. In addition, this same bit could be used to change hole direction, as required, by utilizing a side tracking jet. The combination of a successful directional jet bit and a cableless in-hole guidance system would greatly increase the overall penetration rates.

6. The retractable drill bit has been developed and used by the Japanese and at least one U. S. manufacturer of oil field drilling tools. The bit assembly is pumped down through the drill pipe. The worn bit is recovered with an overshot clamping device attached to a wireline which is also pumped down the drill pipe. This method provides for the possible casing of the borehole with the drill pipe. The bit can consist of a roller cone bit, a bladed bit or a diamond or carbide plug bit. Test drilling to date has been in vertical holes, but the technique should work in horizontal holes as well.

7. The fusion of rock and soil utilizing a high temperature probe. The Los Alamos Scientific Laboratory has been working on the development of such a device called "Subterrene." In porous material, no material need be removed from the hole. In dense material, it would be necessary to remove a rock core and/or rock "wool" cuttings. It may be possible to equip the subterrene with its own self-contained guidance package. In

addition, directional control could be obtained by differential heating of the high temperature probe. There are many problems which must be overcome with this system, however, before it will be possible to penetrate horizontally one mile.

FEASIBILITY OF DRILLING HORIZONTAL LONG HOLES

Today it is technically feasible, using existing technologies, to accurately drill a small diameter, horizontal hole one mile long in soil and rock. Depending on the value of the information to be gained and the actual cost of drilling, the hole may or may not be cost prohibitive. There are several factors which determine the feasibility of accurately drilling a small diameter horizontal hole one mile long, using remote guidance, and providing information about the geological conditions encountered. For instance:

1. The equipment selected must be adequately sized to do the job. It must be capable of providing sufficient thrust, torque, and retractive forces to the drilling assembly and bit to penetrate the soil or rock.

2. The proper drilling tools must be provided. Too often this is not the case.

3. It must be possible to penetrate the rock or soil with the penetration tools and equipment selected. For instance, it would not be possible to penetrate granite rocks with a small diameter three-cone roller bit using the bits and equipment available today. The bearings in the small diameter roller bits are not capable of surviving the very high thrusts that would be required to exceed the rocks' compressive strength.

4. It must be possible to maintain an open hole at all times. The drilling technique selected must be capable of keeping the hole open and in a stable condition. The material penetrated must be competent enough to remain open. If it is necessary to penetrate soft, unstable material, this factor will greatly influence the choice of drilling technique.

5. Satisfactory solutions to groundwater problems must be devised. Large inflows of groundwater could be a problem. When drilling from an underground set-up, it would probably be necessary to stop drilling and grout off the water inflow.

6. Satisfactorily coping with severe lost circulation problems. Circulation would have to be rapidly restored or it could lead to a stuck drill string.

7. Adequate cuttings removal from the hole is a must. If the cuttings are not rapidly removed from the hole, it could result in a stuck drill string.

8. It must be possible to control the vertical and horizontal direction of the borehole. In addition, it must be possible to correct adverse hole deviation by deflecting the hole back to its original trajectory.

9. The penetration rate must be great enough that the total cost per foot is not prohibitive.

10. The drilling crew must have sufficient experience and technical expertise to know how to drill such a hole and how to cope with any problem that might arise.

11. The drilling crew must be capable of using alternate drilling techniques as changing ground conditions dictate. Two or more drilling techniques may have to be used in a single long hole.

12. The correct circulating medium must be used which is compatible with the drilling technique and hole conditions. For instance, if a drilling mud is required, the success of the operation may depend on the fact that the correctly engineered mud is being used.

13. The maximum velocity of the circulating medium must be compatible with the material being penetrated. That is, high velocities may severely erode the walls of a hole drilled in a soft material.

14. If a remote guidance system is to be used, it will not be possible to use a wireline coring or continuous coring technique. The guidance system, installed in the drill string, will prevent passage of the core. Therefore, if it is necessary to use these coring techniques to complete a hole, it will not be possible to use a remote guidance system. Also, if it is necessary to use a remote guidance system having a cable readout to the surface, it will not be possible to rotate the drill pipe. In this case, an in-hole motor would have to be used.

There are many variable factors which will influence the cost of drilling horizontal directional holes. For instance:

1. Labor cost
2. Survey time required
3. Time spent deviating the hole back on course
4. Material hardness/penetration rate
5. Drilling problems - lost circulation, fishing jobs, etc.
6. Direction of bedding
7. Experience of drilling crew
8. Total number of holes at that site
9. How often bits must be replaced

Because of these many variable factors, drilling costs can and will vary considerably. Therefore, it is impossible to accurately predict the actual cost of drilling a long horizontal hole without knowing exactly where it will be drilled. Even then the estimate could be in error by a considerable margin.

To have some indication of the costs incurred for drilling long horizontal holes, some rough estimates and approximate costs encountered with a few previously drilled holes have been obtained. This data is given in Table 23. Figure 34 is a graph showing these representative drilling costs on a basis of cost-per-foot versus hole length.

Table 23. Drilling data for drilled horizontal long holes.

HOLE SIZE & LOCATION	DRILLING TECHNIQUE USED	DEPTH (FEET)	MATERIAL HARDNESS (PSI)		TIME REQUIRED TO DRILL (FEET PER DAY, THREE SHIFTS.)	COST PER FOOT ($)	REMARKS
			0 TO 10,000	10,000 TO 50,000			
3 inch, Mercury, Nevada	wireline diamond core	3,700 deepest 2,000	2000 to 5000		100 for 3,000-foot hole. 120 for up to 1,000+ feet.	30 overall average for 3,000 to 3,500 feet.	Includes survey and wedging.
3 inch, U.S.A. Estimator #1	wireline diamond core	to 5,000	yes	yes	1,000=75 2,000=60 3,000=51 4,000=36 5,000=24 (Conservative)	30+ overall for 4,000-foot hole. Estimated 40 for everything.	Cost per ft. is basic cost; does not include such items as mud, survey, directional control, mobilization, bad ground, grouting, etc.
3 inch, U.S.A. Estimator #2	wireline diamond core	to 5,000	yes	yes		1,000 ft.=6 to 10 3,000 ft.=6.50 to 12.00 5,000 ft. = 15 to 20	Would not include mud, survey, directional control, mobilization, etc.
3 inch, U.S.A. Estimator #3	wireline diamond core	to 5,000	yes	yes	Estimated time to drill 5,000-ft. hole = 100 days or 2,400 hrs.	0 to 1,000 ft., NX=14, BX=13 1,000 to 2,000 ft., NX=16, BX=14 2,000 to 3,000 ft., NX=18, BX=15 3,000 to 4,000 ft., NX=21, BX=18 4,000 to 5,000 ft., NX=26, BX-21	Estimated drilling costs; does not include time for extensive survey or directional work.
3½ inch, Ohio	rotary with 3-cone roller bit	1,100	600 to 2,000		600 ±	2.50 to 4.50	Drilling 1,000 ft. horizontal holes in coal seam.
3 inch, Pennsylvania	1 3/4 inch Dyna-drill and diamond plug bit-curved hole from surface	1,730	yes		30± overall average Estimated 40 feet per day, under good conditions	30 to 35	This estimate includes a lot of downtime, several days without drilling.

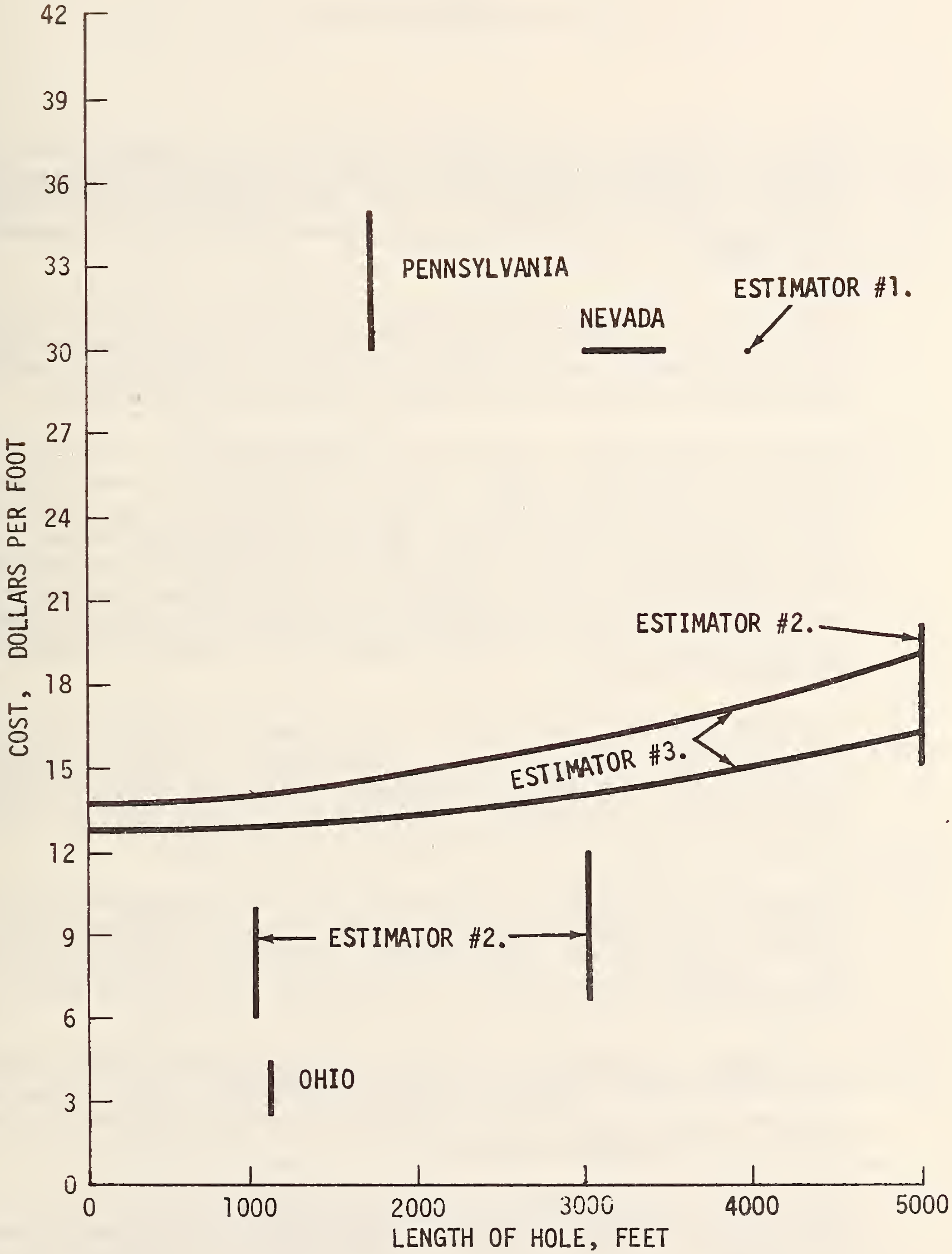

Figure 34. Representative drilling costs for horizontal holes. See Table 23 for individual hole descriptions.

Section 6

Conclusions

1. Many of the major cost overruns encountered on highway tunnel projects are directly traceable to either insufficient geologic information, incorrect interpretation of the information, or a decision by management to not follow the recommendations of the investigation team.

2. One act more than any other single action which can improve the design and proper implementation of subsurface investigation systems is the systematic and scientific analysis of subsurface conditions prediction as compared with actual conditions encountered.

3. Available subsurface investigative techniques are, in most cases, adequate for highway tunnel site investigations if used properly and to the necessary extent. However, money and time constraints can, and usually does, prevent their effective utilization. Having an inadequate amount of money and/or time available for the subsurface investigation can preclude the use of the most suitable investigative techniques and may even prevent the proper or maximum utilization of those techniques which can be used with the money and time allotted.

4. Proven subsurface investigative techniques not being fully utilized include remote sensing by color photography, infrared photography, and side-looking radar; and borehole logging by visual or photographic means, conventional resistivity, microresistivity, focusing electrode, induction, and formation density logs.

5. Subsurface investigative techniques not fully developed, but which appear to have good potential include thermal infrared, multispectral photography, multispectral scanning, and infrared radiometry remote sensing; acoustic holography; and sonic borehole logging in soils.

6. A systematic approach to planning the optimum subsurface investigation system for a highway tunnel is rarely used. The value analysis model developed in this study can be used to optimize a subsurface investigation system.

7. The drilling of horizontal holes for distances up to one mile is technically feasible. However, improvements are needed in penetration rates, guidance and directional control, and geologic surveying equipment to make this technique economically competitive with other investigative techniques.

Section 7

Recommendations

The following recommendations are suggested for achieving better subsurface investigations in connection with highway tunneling projects.

1. On each tunneling project, encourage the gathering of sufficient information to allow the proper design, and then make available to the industry a thorough documentation of the project which should include:
   a. A complete record of all investigative work done in the order accomplished.
   b. Interpretations made and how they were modified by later information.
   c. A complete record of the construction including details on any problems encountered and how they were solved.
   d. A complete and critical comparison of the actual subsurface conditions encountered with those predicted including, in retrospect, what steps should have been taken, or information gathered, to have accomplished the job more efficiently.
2. Improve the performance of present day subsurface investigative techniques in order to reduce the time and cost required for their use. The investigative techniques in this category would include surface seismic refraction and reflection, and conventional resistivity methods; rotary drilling; and undisturbed sampling techniques.
3. Make greater use of those subsurface investigative techniques now proven, but not fully utilized, such as: remote sensing by color photography, infrared photography, and side-looking radar; and borehole logging by visual or photographic means, conventional resistivity, microresistivity, focusing electrode, induction, and formation density logs.
4. Develop those subsurface investigative techniques now unproven, but having good potential such as: thermal infrared, multispectral photography, multispectral scanning, and infrared radiometry remote sensing; acoustic holography; and sonic borehole logging in soils.
5. Implement, for each tunneling project, use of the value analysis model developed in this study to insure the most optimum subsurface investigation system is formulated.
6. Improve the penetration rates and guidance and directional control techniques for the horizontal drilling of long holes. Also, improve the present sensing equipment, and develop new sensing equipment, for geologic investigation from a horizontal hole.

Appendix A

Preliminary Geological Investigations

The engineering geology investigation of a proposed tunnel route begins in the office with an intensive search for all available pertinent information on the assigned site and its surroundings and for case histories of previous tunnel construction in similar geologic environments.

The status of existing geologic knowledge in the proposed tunnel area must be determined and digested before an intelligent plan of further investigation can be developed. For example, if aerial photographs and large scale geologic maps are already available, these efforts may not have to be duplicated. The field investigation can then be started in a more advanced stage than if the site were located in an unmapped area.

Besides accumulating all available information on the lithology and structural geology of the site, the exploration group should learn all possible about the terrain, accessibility, climate, groundwater, existing surface and subsurface construction, and any oil and gas and mining exploration or production activity.

After, or during, the initial intensive information search, one or more preliminary site inspections of the proposed tunnel route must be made to determine the existing status of the area, to examine first hand the geology which has been read about, and to formulate a systematic plan of attack for the more detailed geologic, geophysical, hydrologic, and rock mechanics investigation which is to follow.

At this early stage, a number of possible tunnel routes may be under consideration and the impressions resulting from the initial combined field and office study form the basis for selecting the most favorable route or routes which are to be studied in greater detail.

Following is a list of the principal sources which should be checked in this comprehensive search.

U.S. GEOLOGICAL SURVEY (USGS)

USGS publications, open-file reports and maps are the single most comprehensive source of geologic and hydrologic information in the country. This material includes:

Geologic index maps - Maps compiled for each state showing coverage and sources of most published geologic maps.

Folios of the geologic atlas of U. S. - Maps and descriptions of bedrock and surface materials for many urban and seacoast areas.

Geologic quadrangle maps - This series succeeds the older geologic folios and is concentrated on areas not previously mapped in detail.

Geophysical investigations - Mostly aeromagnetic maps, some gravity, radiometric, and geologic maps.

Water supply papers - Information on groundwater resources in specific areas, usually including description of subsurface conditions.

Hydrologic investigations atlases - Information on water resources, floods, geology, and engineering soils in specific localities.

Topographic maps - Covering entire country at various scales. Considerable geologic information on rock types, and structure may be deduced from study of land forms and drainage patterns.

Aerial photographs - Copies of photographs ranging in scale from 1:15,000 to 1:85,000 are available for many areas in sizes from 9-inch by 9-inch (contact print) to 36-inch by 36-inch (enlargements). Aerial mosaics covering more limited areas are also available for sale.

Other USGS publications - Bulletins, professional papers, circulars, annual reports, monographs, miscellaneous geologic investigation maps, mineral resource maps and charts, and special geologic maps contain general physical geology emphasizing mineral and petroleum resources.

U.S. BUREAU OF MINES (USBM)

The USBM has a variety of publications and open-file reports dealing with the development of mineral deposits (including geologic summaries) and rock mechanics.

U.S. DEPARTMENT OF AGRICULTURE (USDA), SOIL CONSERVATION SERVICE

USDA maps and reports describe surface soils of many counties in agricultural terms. Physical geology is also summarized. The USDA also has aerial photographs of many areas of the country available.

U.S. COAST AND GEODETIC SURVEY

Charts of coastal areas are available showing soundings of sea bottom and topographic and cultural features adjacent to the coast or waterways.

U.S. ARMY CORPS OF ENGINEERS

Considerable geologic work carried out in the Mississippi River Valley has been reported on. Other geological studies have been carried out as part of dam site investigations.

U.S. BUREAU OF RECLAMATION

Geological studies have been made in dam site, tunnel and waterway investigations.

NATIONAL REFERRAL CENTER FOR SCIENCE AND TECHNOLOGY, LIBRARY OF CONGRESS

The Directory of Information Resources in the United States: Water lists and describes many organizations dealing with water.

GEOLOGICAL SOCIETY OF AMERICA

The Geological Society of America also has several useful sources of information including:

> Bibliography and Index of Geology - Very useful monthly listing of geologic publications indexed by subject and area.
>
> Memoirs, monthly bulletins, engineering geology case histories, guidebooks, and special papers - Reports cover specialized geological subjects and detailed investigations of local geology, often including detailed geologic maps.
>
> Geologic maps - General small scale geologic maps of the country and various alluvial deposit maps.

STATE GEOLOGICAL SURVEY PUBLICATIONS

Many detailed geological maps and reports on specific areas are available.

PROFESSIONAL PERIODICAL JOURNALS

The following is a partial listing of geological, geophysical, and petroleum and mineral exploration and development related publications which contain useful information on subsurface conditions in the United States:

> Bulletin of the Association of Engineering Geologists
> Engineering Geology, An International Journal
> American Association of Petroleum Geologists Bulletin
> Proceedings and Transactions of American Society of Civil Engineers
> Geophysics
> Economic Geology
> Mining Engineering
> Engineering and Mining Journal.

SYMPOSIUMS, CONFERENCES, GUIDEBOOKS

Numerous publications containing papers presenting specific subsurface information have been produced by national, regional, state, and local organizations. Included in this lengthy list are:

Proceedings of annual highway geology symposiums
Highway Research Record
Proceedings of annual soil mechanics and rock mechanics symposiums
Proceedings of tunneling and rapid excavation conferences
Proceedings of annual soils engineering symposiums

UNIVERSITY PUBLICATIONS

Detailed geology of local areas is often described.

UNIVERSITY THESES (MASTERS AND Ph.D)

Intensive geological studies are made on specific areas, sometimes several states distant from the university being attended. Xerox and/or microfilm copies of all PhD thesis written in the U.S. are available at a nominal charge from University Microfilms, Inc. in Ann Arbor, Michigan.

GEOPHYSICAL-GEOLOGICAL DATA COMPANIES

Large quantities of geophysical and some geological data are available for sale by a few organizations. The bulk of this material is seismic, magnetic, gravity, and photogeologic survey information which has been accumulated in the search for oil and gas.

COMPANY FILES

A great amount of specific information on underground conditions has been collected by consulting geoscience service organizations, contractors, underground construction firms, and mineral, petroleum, groundwater, and geothermal exploration organizations. This material is often confidential and very difficult to learn of or obtain, but it is certainly worth an attempt. The trading of information is sometimes successful.

LOCAL GOVERNMENT, INDUSTRY AND CONSULTANTS

Discussion with geologists, geophysicists, and engineers who have spent years investigating the subsurface in the area of interest for various reasons are excellent sources of information in the study of a proposed tunnel route. Particularly qualified individuals may contribute substantially as consultants in a site investigation project.

CASE HISTORIES

Completed and active tunnel projects should be researched, with particular attention paid to those driven through geologic environments having similarities to that of the assigned tunnel route. Comparisons of investigation methods used and conditions predicted against actual conditions encountered should improve the quality of judgement developed through an individual's personal experience. Besides published case history papers, direct personal contact with key personnel involved in the past projects can prove very useful.

Appendix B
Geologic Mapping

Geologic maps are a basic and vital ingredient to preexcavation tunnel site investigations. Properly prepared, geologic maps give an indication of the distribution, nature, and structure of the soil and rock units which must be excavated and supported.

Most geologic maps are prepared as either plan or section views. Since maps are planar views, both plans and sections are necessary as a minimum for visualizing three dimensional subsurface geologic features. Block diagrams or three dimensional models are sometimes used to enhance the interpretation of subsurface geology.

The most common geologic plan maps illustrate either surface geology or bedrock geology showing the upper surface of rock formations as if soil overburden were removed. Geologic plan maps are prepared at various scales and show great variations in the amount of detail recorded. Mapping requirements depend upon the stage at which the investigation happens to be and the scale and quality of existing geologic maps. Remote sensing techniques are valuable aids to both reconnaissance and detailed surface geologic mapping.

Principal end products of geologic mapping and subsurface exploration are vertical and sometimes right angle cross sections through the proposed tunnel alignment depicting the investigation team's prediction of the positions, types, and conditions of subsurface material. These sections are constructed from various types of information including the downward projection of soil and rock units mapped at surface, results of geophysical surveys, and drill hole data.

The most valuable, and by far the most costly, exploration operation, the driving of a pilot tunnel, best exposes the rock for geologic examination and mapping. Geologic features important to tunnel design should be carefully mapped in great detail in these bores. Special techniques have been developed for mapping the geology around the entire cylindrical surface of circular machine-driven tunnels.

Finally, detailed geologic maps should be prepared for all completed highway tunnels before the geology is hidden behind linings. Besides the regular geologic plan maps and longitudinal sections, geologic maps of the advancing tunnel face at intervals are worthwhile. These maps of finished tunnels when compared with advance geologic predictions, demonstrate the adequacy of the preexcavation investigation and can be used to improve the experience of the investigators, serve as valuable records for tunnel maintenance, and will be useful should a parallel tunnel be desired.

REMOTE SENSING

Aerial exploration techniques, commonly referred to as remote sensing, involve the application of a wide variety of sensing systems which measure energy emitted and reflected by the land surface from a distance without direct contact. Remote sensing is primarily a reconnaissance exploration tool serving as an aid to geologic mapping, enabling large areas to be surveyed rapidly and economically by instruments borne by aircraft or earth-orbital satellite.

For tunnel site investigations, remote sensing methods are very useful for detection of the patterns of important regional structural features such as faults and folds, surface drainage patterns affecting hydrology of the area, and surface cultural features. They also provide the images required for photogrammetric production of topographic maps. Skilled interpretation including improving computer processing of data allows some measure of understanding of the subsurface conditions to be derived from the various techniques, but they cannot eliminate the need for detailed on-site surface and subsurface investigation. The imaginative use of selected remote sensing techniques can be extremely useful in planning the detailed on-site work and may reduce the amount of on-site work required.

Substantial advances in remote sensing survey methods and interpretation of value to tunnel site investigations are likely within the next few years. Remote sensing capability has been most highly developed by branches of the military, and it is likely that operational but presently highly classified methods would be useful in civilian engineering studies.

Figure B-1 shows the subdivided electromagnetic spectrum and the principal sensing systems used to detect energy of the different wave lengths.[55]

Electromagnetic sensors may be classified as imaging or nonimaging. Imaging sensors include cameras, optical-electromechanical scanners, passive-microwave scanners, and radar. These sensors record on photographic film two-dimensional pictures of radiation received directly by camera or indirectly by photographing the cathode ray tube or crater lamp patterns from scanners or radar. Nonimaging sensors include infrared and microwave radiometers, laser and microwave profilers, radiofrequency sounding instruments, gamma-ray spectrometers, and electromagnetic induction devices. The latter two are described later in this

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[55] Jackson, Philip, et al. Tunnel Site Selection By Remote Sensing Techniques, study for A.R.P.A. of U.S. Department of Defense. Willow Run Laboratories, University of Michigan. September, 1972.

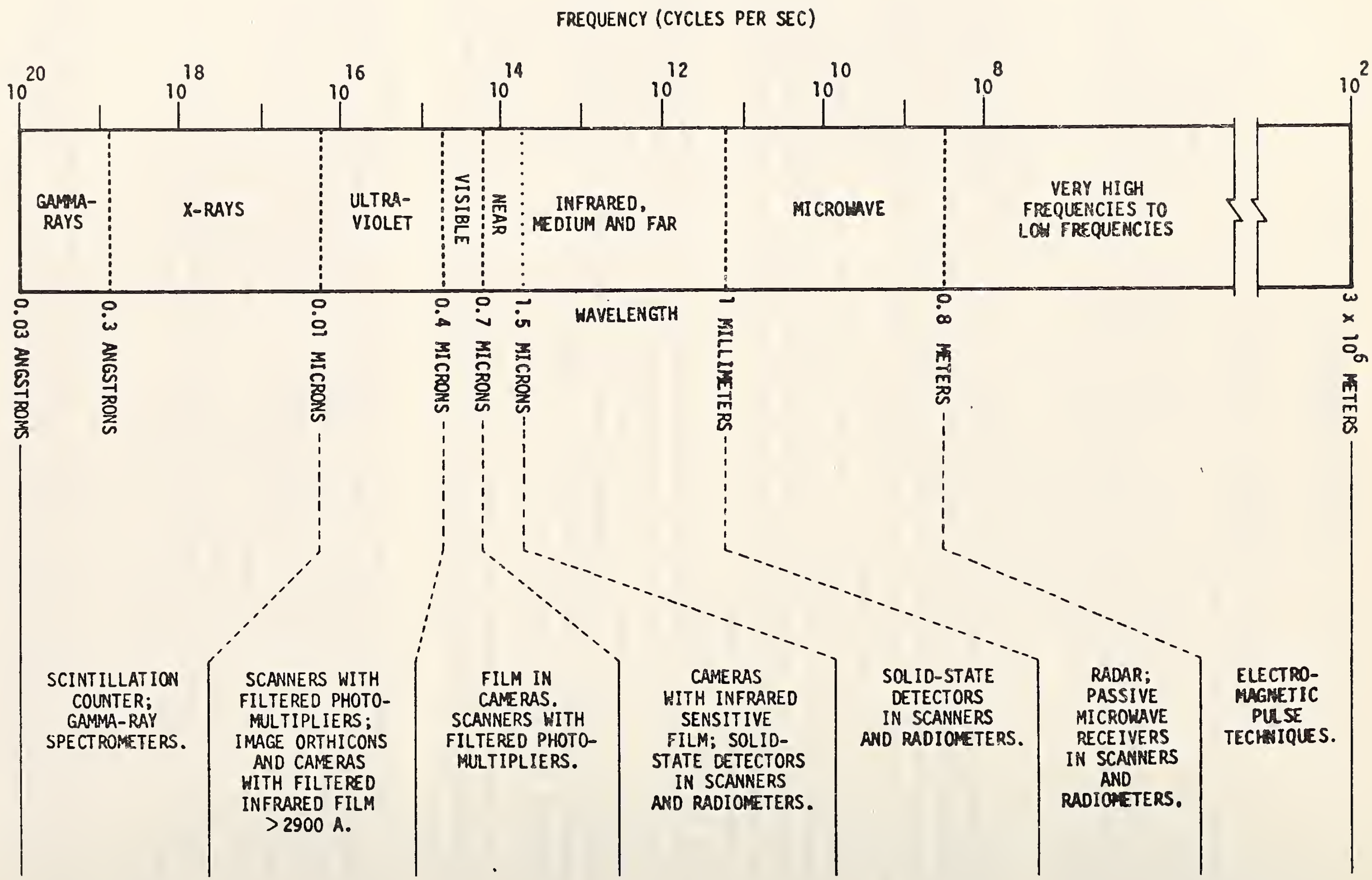

Figure B-1. Electromagnetic spectrum with principal sensing systems.

report in the section on geophysical surveys. This group of sensors are used to produce one-dimensional records on chart paper of radiation received.

AIRBORNE REMOTE SENSING

Table B-1 shows the more important airborne remote sensing methods which have proven to be useful in highway tunnel site investigations or which may find some application in the future.

Table B-1. Remote sensing methods for tunnel site investigation.

STATUS OF METHODS	REMOTE SENSING METHOD	
	IMAGING	NONIMAGING
Proven, relative low cost, widely used.	Panchromatic black & white photography	
Proven, relative low cost, less commonly used.	Color photography Color infrared photography Black & white infrared photography	
Being improved, higher cost than conventional photography, little used to date but expected to increase, promising for special problems, available commercially.	Thermal infrared Multispectral photography Multispectral optical-mechanical scanning Side-looking airborne radar	Infrared radiometry Infrared spectral radiometry
Little used to date, possible application to specialized problems, limited availability.	Passive microwave Ultraviolet Luminescence	Passive microwave radiometry

PANCHROMATIC BLACK & WHITE PHOTOGRAPHY

To date, this has been by far the most widely used remote sensing method for tunnel site investigation. Black and white, and to a much lesser extent color, photography is recognized as being the most rapid and economic means of preparing topographic base maps necessary for engineering site studies including the plotting of reconnaissance and detailed geologic data.

Black and white photography of a 15-square mile area should cost approximately $1,500 at a 1:6000 scale. Photogeologic interpretation of prints covering this size area should cost $1,500 - $2,500.

COLOR PHOTOGRAPHY

Conventional color photography continues to be widely thought of as the best single remote sensing method available for tunnel site investigation. Considerable useful information about geologic structure, soil and rock types, slope stability, and hydrologic conditions can be gained from study of color prints. A relatively large number of people are experienced in photogeologic interpretation of both color and black and white photography using field spot checks for verification. Color photography thus far has been used less than black and white, but its use is increasing as many features are more easily distinguished with color.

Detailed color photography of a 15-square mile area which would be sufficient for most highway tunnels would cost approximately $2,000 at 1:6000 scale with about half the cost due to flying and half due to the photography. Geologic interpretation cost would be similar to that for black and white photography.

COLOR AND BLACK & WHITE INFRARED PHOTOGRAPHY

Infrared photography records reflected radiation on film in the same manner as conventional photography and is also generally used during daylight. Some features, especially differences in vegetation which may be clues to soil types and moisture content, may stand out with greater clarity than with conventional photography.

Costs are comparable to conventional color photography. If infrared were recorded simultaneously with regular color, the unit cost of each would be reduced accordingly.

THERMAL INFRARED IMAGERY

This is a heat measuring method which records emitted radiation to determine apparent surface temperatures through the use of a scanning radiometer. Infrared radiation (IR) with wavelengths between 8 and 13 microns is the portion most commonly recorded by this method. Radiation recorded on infrared-sensistive film indicates warmer areas as white or light gray and colder areas as gray to black. Often this type of survey is run just before dawn and again after the surface has been warmed by the sun.

Thermal infrared imagery is especially useful for detecting anomalies near surface areas with high water content such as shattered fault zones or saturated soils subject to landsliding. Recently developed image-ratioing techniques allow discrimination between rocks of substantially differing silicate content and between those of differing ferric oxide content.[56]

The cost of performing this type of survey over a 15-square mile area would likely be $5,000 - $10,000.

INFRARED RADIOMETRY

This nonimaging method is useful for repeated measurements of surface temperatures. The data obtained requires less processing than that produced by infrared imagery to obtain similar useful information. However, this method is quite sensitive to the vegetation present, the season of the year, the amount of precipitation that has fallen, and in some instances even the time of day the data is obtained.

INFRARED SPECTRAL RADIOMETRY

Apparent surface temperature measurements are taken through the wavelength range of 1 to 16 microns with spectrometers, interferometers, and radiometers. Libraries of spectral signatures are being produced and stored in computers for determination of rock composition by comparison.

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[56] Jackson, Philip, et al. Tunnel Site Selection By Remote Sensing Techniques, study for A.R.P.A. of U.S. Department of Defense. Willow Run Laboratories, University of Michigan. September, 1972.

MULTISPECTRAL PHOTOGRAPHY

Multiple cameras (up to 9) are used to record energy reflected from the ground surface within separate, specific wavelength bands. The advantage of this system is that tonal contrasts between different materials are sharper in different segments of the spectrum, thus increasing the opportunity for identifying geologic features of interest. The cost of this system is substantially higher than conventional photography. This is due to the use of several films and the resulting large mass of data which requires some type of computer processing for effective use.

MULTISPECTRAL (MULTICHANNEL) OPTICAL-MECHANICAL SCANNING

This system permits data acquisition through a broader portion of the spectrum than photographic systems. Eighteen or more channels may be used to record energy having wavelengths between 0.3 and 16 microns.

Producing a much greater volume of data than multispectral photography, the system requires even more extensive computer processing which increases its cost. Three types of computer processing or filtering found to be useful are:

1. Single-channel quantization and level slicing,
2. Two-channel ratioing, and
3. Multiple-channel statistical pattern recognition.

SIDE-LOOKING AIRBORNE RADAR (SLAR)

This recently developed method has proven to be an excellent means of mapping structural geologic features and may provide information on rock types and geomorphic features such as eskers, terraces, and old stream channels.

The system transmits radio-frequency energy along a narrow beam, then passes energy reflected back from the terrain through a cathode ray tube where the images are recorded by passing a continuous strip of film across the tube. Cloudy weather presents no difficulty to the system.

SLAR is principally a tool for regional mapping of large areas in the range of hundreds of square miles and would be quite costly to apply to a small tunnel site investigation. Cost for an area of 5,000 square miles or greater is in the \$5 - \$10 per square mile range.

Reportedly, some 15 percent of the United States has already been covered by some form of SLAR survey and much of this data is available to government agencies at no cost from the U.S. Geological Survey and the Air Force.

PASSIVE MICROWAVE

Images record radio-frequency energy emitted from the earth which may serve as an indicator of differing soil moisture contents. The system has not been thoroughly developed and presently would not be applicable to tunnel site investigations.

PASSIVE MICROWAVE RADIOMETRY

Some success in detection of subsurface voids has been reported with this technique which measures apparent temperatures at different radio frequencies. The longer wavelength ranges allow deeper penetration of temperature determination.

ULTRAVIOLET

Differentiation between some rock types (especially acidic and basic) may be possible in the ultraviolet band as reflectance contrasts are often greater than in the visible light range.[57] The system is not now applicable to tunnel site investigations.

LUMINESCENCE

Differentiation between some certain rock types such as limestone and dolomite might be possible by illuminating the ground surface with ultraviolet light and recording resulting luminescence in the ultraviolet, visible, or infrared wavelengths.[57] The system is not presently applicable to tunnel site investigations.

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[57]Fischer, William A. "Examples of Remote Sensing Applications to Engineering." Highway Research Board Special Report 102. 1969. pp. 13-21.

SATELLITE-BORNE REMOTE SENSING

The National Aeronautics and Space Administration's Earth Resources Technology Satellite-1 (ERTS-1) launched into earth orbit in July, 1972, contains advanced remote sensing equipment which has mapped a large portion of the earth in strips at periodic intervals.

The system obtains small scale imagery of some 10,000 square miles per view with two separate subsystems. The multispectral scanner subsystem (MSS) detects light energy in four separate spectral bands, including infrared. The return beam vidicon (RBV) subsystem uses three cameras, each viewing the same terrain but recording light from different spectral bands which when superimposed result in a full color image. Data recorded is transmitted to the ground for use.

ERTS-1 data may be of limited use to particular tunnel site studies as major structural features can be distinguished, but generally the small scale obliterates detail which may be vital to the study. Accordingly, it is necessary to obtain some type of more useful, larger scale, conventional airborne photography.

ERTS-1 color composite pictures of much of the United States are now available for purchase from the U.S. Geological Survey's EROS Data Center in Sioux Falls, South Dakota, in 10-inch by 10-inch contact color prints and transparencies or enlargements to 40-inch by 40-inch at nominal cost.

A second satellite, ERTS-2, launched in the summer of 1973, includes a thermal measuring band to increase the capabilities of the system.

DIRECT SURFACE MAPPING

Surface geologic mapping commonly progresses through two or three stages. During the preliminary site selection stage, it is usually adequate to produce small scale maps by photogeologic interpretation of aerial photographs and other remote sensing data in conjunction with field spot checking and limited rapid reconnaissance field mapping. The objectives of this reconnaissance mapping are to: (1) identify principal soil and rock units and prominent structural features and trends, (2) enable preliminary selection of one or more promising tunnel alignments, and (3) provide a base for planning subsequent investigations.

The second stage, a feasibility study to prove the acceptability of a particular route or to enable final selection of one of the alternative choices, requires a more detailed study including the preparation of improved larger scale geologic maps and probably some subsurface investigations such as geophysical surveys and drilling.

After the definite tunnel route has been selected, a third stage consisting of additional, more thorough subsurface investigations will be carried out and detailed surface geologic mapping is completed if not already finished during the second stage. In some cases, the tunnel alignment is fixed by planners in the beginning and the entire investigation could be subdivided into two stages, reconnaissance and detail.

The geology of the complete tunnel route should be mapped with coverage extending an appropriate distance beyond the proposed tunnel route to include any features which might have bearing on the interpretation of the subsurface picture.

Map scales used during the various stages are dependent upon the objectives of the particular stage of investigation and the complexity and amounts of exposed geology. Reconnaissance maps may be produced at 1:12,000 or 1:6,000 scales. Detailed mapping might be performed at scales on the order of 1:2,400 or 1:1,200. It is often advisable to map selected specific areas of high interest in even much greater detail, sometimes even down to a scale of 1:120 or 1:60 to show important structural relationships such as joint patterns.

Ideally, detailed geology is plotted in the field on air photos or an accurate topographic base map. Air photos allow the mapper to position himself with regard to recognizable surface features. Advance surveying and marking of grids on the ground surface also greatly facilitates geologic mapping. The long-established systems of plane-table mapping remain very useful and require a minimum crew of an instrument man to operate the alidade and a geologist to make observations.

During mapping, areas of rock outcrops should be outlined on the map. Actual observations of contacts, faults, etc. can be made only in these areas. It is the geologist's duty to interpolate geological features through covered areas between outcrops whenever possible. However, the distinction between observed geology and inferred geology must be clearly made and strongly emphasized to all who have occasion to use the maps. Mapped geology should be drawn with solid heavy lines and inferred geology should be drawn with dashed or dotted lines sometimes using question marks to indicate the degree of confidence in these projections. It is often useful to develop two separate maps in the office, one showing outcrop areas and actual mapped geology, and the other showing both mapped and inferred gology.

Detailed geologic mapping should identify the various soil and rock units present and record the positions and attitudes of contacts and of major and minor structural discontinuities. All geologic features important to engineering design of the tunnel and any indications of problem areas should be carefully studied and recorded during mapping including the taking of abundant descriptive notes, photographs and appropriate samples to supplement the map itself. Items of prime importance are the width, spacing, orientation, tightness, kind and type of filling material, and surface character of discontinuities such as faults, shear zones, and joints. Other features to be noted include composition, texture, cohesion, rock hardness, bedding and foliation characteristics, and the degree and type of alteration and weathering.

Major faults and folds may be projected downward with good accuracy at times, but unprojectable features which may occur in large number, such as minor faults and joints, should be studied and recorded in a statistical manner, compiling the data in the form of strike frequency diagrams and contour diagrams.

Appendix C

Geophysical Surveys

A relatively few basic laws of physics, such as Newton's Law, Ohm's Law, and Snell's Law, form the basis for most geophysical investigation techniques. All of these methods involve measurement of some physical property of the underlying soil and rock, in which a change in measured value indicates the possibility of some change in the soil or rock. Thus, changes in measured values, or "anomalies" as they are often called, interest the tunnel site investigator because they may represent structural or lithologic changes within the earth. The principal effort in geophysical investigations is the interpretation of the significance of these anomalies.

The major techniques of geophysical investigation can be broken down into general categories including (1) measurement of differences in the reaction of earth materials to artificially induced force fields produced by electrical or seismic impulses introduced into the materials at or near the earth's surface, (2) measurement of variations of the earth's magnetic and gravitational fields, and (3) measurement of electrical currents spontaneously generated by geologic bodies.

Both surface and subsurface techniques are used in geophysical investigations. Subsurface geophysical investigation tools are directed toward the interpretation of data obtained from boreholes and are discussed in Appendix E.

The principal methods which have been used in surface geophysical investigations are these:

1. Seismic
2. Electrical
3. Magnetic
4. Gravity
5. Radioactive

These methods are discussed in this Appendix C.

SURFACE SEISMIC METHODS

USES

Surface seismic geophysical methods are used to determine the following for subsurface soil and rock units.

- Depth
- Thickness
- Attitude
- Nature
- Distribution
- Structure
- Geologic discontinuities

In general, seismic methods are most applicable in those areas underlain by prominently stratified sedimentary rock or covered by substantial thicknesses of alluvial material. Where bedrock geology is very complex or is comprised of igneous or metamorphic rocks at or near the surface, the application of seismic techniques is minimal. Subsurface strata, to be differentiated by the use of seismic methods, must have contrasting wave velocity characteristics.

As is the case with many geophysical exploration techniques, the two seismic techniques - refraction and reflection - were first developed to a high degree by the petroleum exploration industry and later adapted for use in engineering applications.

Refraction seismic is the most important and widely used surface geophysical exploration method for investigating subsurface conditions to evaluate tunnel and other major construction sites. The reflection seismic method is used to a much lesser extent, but improvements in equipment and interpretation techniques are increasing its use in engineering applications.

PRINCIPLES

Seismic methods are very similar to optics in that both deal with a type of energy propagated in the form of waves. Studies of both involve the quantities of velocity, frequency, intensity, phase, direction, travel time, wave length, absorption, refraction, and reflection. In seismic studies travel time, refraction, and reflection are of principal importance; very little quantitative use is made of direction, frequency, and intensity.

Seismic procedures involve the input of artificially produced seismic energy into the earth by means of explosives, hammer impacts, or vibrators. This energy travels outward from the source through the subsurface in an approximately spherical wavefront. Precise travel time is measured for the period required by these energy waves to go directly through surface material, or to go downward to refracting or reflecting horizons and bounce back up to the ground surface. At the ground surface, the energy waves are sensed by detectors, known as geophones, which are spaced at measured, regular intervals from the seismic energy source (Figure C-1). To record the direct waves, the distance between the energy source and the detectors must be much less than the depth to the upper refracting horizon. Travel time versus distance relationships are used to determine wave velocities in the various strata. The wave velocities are in turn used to calculate depths to subsurface refracting or reflecting horizons.

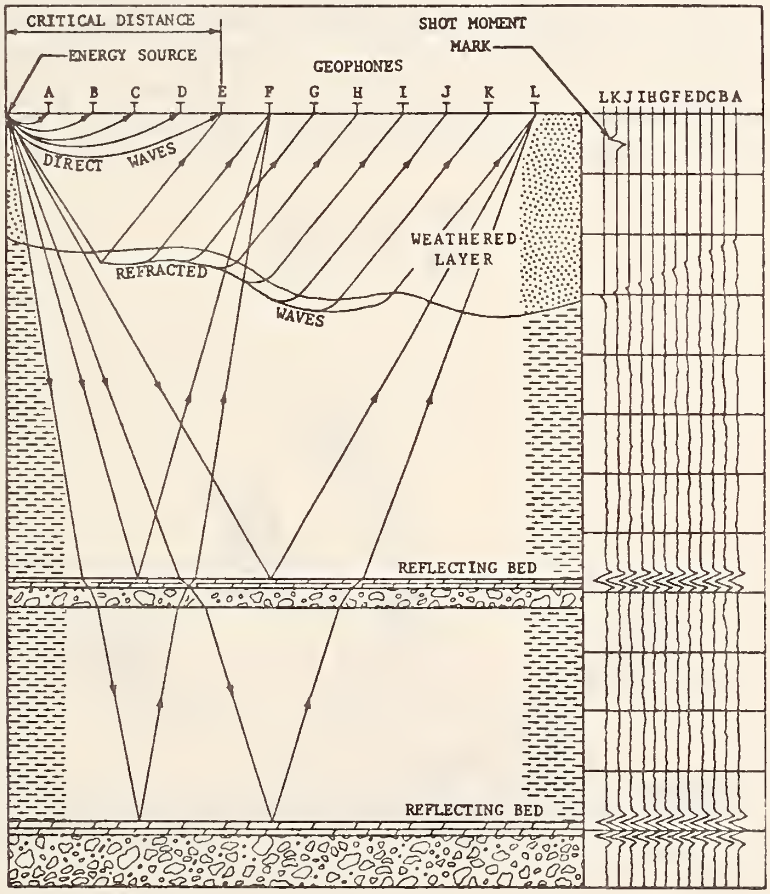

Figure C-1. Seismic-wave paths showing direct, refracted, and reflected waves and recorded seismogram arrivals (Schematic).

An artificially-created seismic shock produces several types of elastic waves, including the longitudinal (compressional), transverse (shear), Rayleigh, and Love types.

The longitudinal wave is the one of primary interest for depth determinations in present seismic surveying techniques as it is the fastest wave and thus the first to reach the detection points. Transverse and Rayleigh wave velocities may be used in determinations of in situ elastic properties of near surface materials. Love waves are never recorded unless the detectors are designed to respond to horizontal ground motion.

In a homogeneous medium, seismic waves spread outward from a point source in expanding spheres. Huygen's principal states that every point on a wavefront is the source of a new wave that travels out from it in spheres also. If the spherical waves have a sufficiently large radius they can be considered as planes. Perpendicular lines to these planes, called wave paths or rays can be used to represent the waves.

Figure C-2 illustrates the principle of reflection at an interface between two elastic media having different longitudinal velocities, transverse velocities, and densities. The angle of reflection of the longitudinal wave is equal to the angle of incidence with both being measured from the normal to the interface. The angle of reflection of the transverse wave can be determined from the expression:

$$\frac{\text{Sin } r_T}{\text{Sin } \xi} = \frac{V_{T1}}{V_{L1}}$$

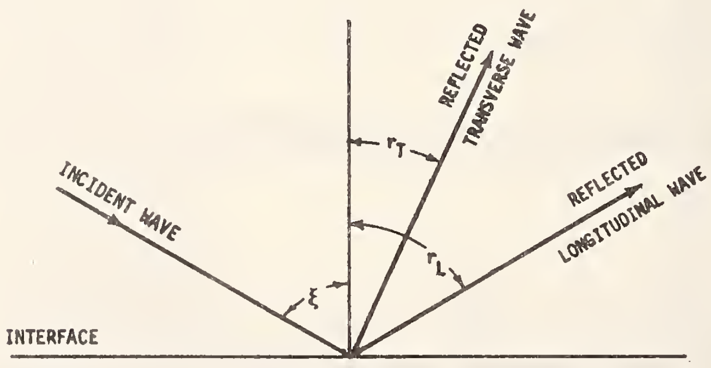

Figure C-2. Reflection of plane elastic waves at interface.

where ξ is the angle of incidence,

r_T is the angle of reflection of a transverse wave,

V_{T1} is the transverse velocity in the upper medium, and

V_{L1} is the longitudinal velocity in the upper medium.

Snell's law which explains the refraction of light rays, applies to seismic refraction as well. Figure C-3 shows the bending which takes place when elastic waves cross the interface between two elastic media having differing properties. According to Snell's law, the angle of refraction which a longitudinal wave makes with the normal to the interface may be computed from the following relationship:

$$\frac{\operatorname{Sin} R_L}{\operatorname{Sin} \xi} = \frac{V_{L2}}{V_{L1}}$$

where R_L is the angle of refraction of a longitudinal wave,

V_{L2} is the longitudinal velocity in the lower medium, and

ξ and V_{L1} are as previously defined.

The relationship used to compute the angle of refraction for a transverse wave is:

$$\frac{\operatorname{Sin} R_T}{\operatorname{Sin} \xi} = \frac{V_{T2}}{V_{T1}}$$

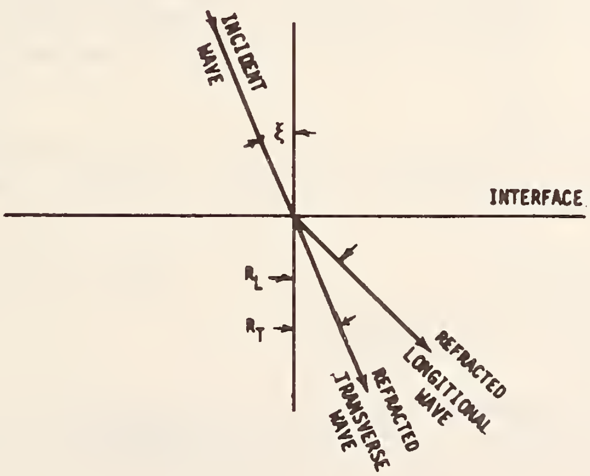

Figure C-3. Refraction of plane elastic waves across interface.

where R_T is the angle of refraction of a transverse wave,

V_{T2} is the transverse velocity in the lower medium, and

ξ and V_{T1} are as previously defined.

The critical angle of incidence for longitudinal refraction is important in the refraction seismic exploration method and is calculated from

$$\text{Sin } \xi_C = \frac{V_{L1}}{V_{L2}}$$

where ξ_C is the critical angle of incidence.

For any angle of incidence greater than this critical angle, the longitudinal wave is totally reflected so no refraction occurs. When the angle of incidence is less than the critical angle, the longitudinal wave may be both reflected and refracted. The critical angle of incidence occurs when the longitudinal angle of refraction is 90^{o}. Under this circumstance the refracted wave does not penetrate the lower medium, but instead travels along the interface and returns to the surface detectors at the critical angle.

VELOCITY

Precise determinations of longitudinal wave velocities are essential for accurate depth calculations in seismic exploration. The velocity of wave propagation through a subsurface material depends upon its combined physical-mechanical properties such as the modulus of elasticity, Poisson's ratio, density, texture, porosity, and composition.

In sedimentary rocks, wave velocity varies with depth, lithology, geographic location, structural position, and age. Generally, velocity increases with increases in

- Density
- Degree of consolidation
- Degree of metamorphism
- Depth
- Geologic age
- Lime content in sandstones or shales
- Interstitial fluid content of unconsolidated rocks
- Degree of cementation, and with
- Decreasing porosity and decreasing interstitial content of consolidated rocks.

Most sedimentary rocks are anisotropic and exhibit higher velocities along bedding planes than normal to them. Commonly the velocity in shales parallel to stratification is about 10 percent greater than at right angles to it; in some cases it is 50 percent greater.[58]

Foliated metamorphic rocks are also anisotropic, allowing higher velocities parallel to foliation than normal to it.

Mineralogic composition and crystalline structure affect the wave velocities through igneous and nonfoliated metamorphic rocks. Velocity increases with a decrease in silica content and with an increase in the size of mineral grains.

Table C-1 shows velocity ranges for longitudinal seismic waves through some typical soils and rocks.

Table C-1. Selected velocities of seismic longitudinal waves in subsurface materials.

	Velocity (ft/sec)
Alluvium	1,500 - 6,500
Clay	3,500 - 9,000
Loess	1,000 - 1,000
Sand	500 - 6,500
Glacial till	1,500 - 5,500
Granitic rocks	13,000 - 20,000
Gabbro, diabase, basalt	16,500 - 22,000
Sandstone, shale	4,500 - 15,000
Limestone	
soft	5,500 - 14,000
hard	9,000 - 21,000
crystalline	18,500 - 21,000
Dolomite	16,000 - 20,000
Anhydrite, gypsum, salt	11,500 - 18,000
Slate	11,500 - 14,500
Schist and gneiss	11,500 - 24,500
Water	4,800

Longitudinal and transverse wave velocities may be determined from the following relationships:

[58] LeRoy, L. W. and Harry M. Crain, editors. Subsurface Geologic Methods. Colorado School of Mines, Golden, Colo. 1949.

$$V_L = \sqrt{\frac{B + (4/3)G}{\rho}}$$

$$V_T = \sqrt{\frac{G}{\rho}}$$

where V_L is the longitudinal wave velocity,

V_T is the transverse wave velocity,

B is the bulk modulus,

G is the rigidity (or shear) modulus, and

ρ is the bulk density.

Poisson's ratio may be determined from these wave velocities using the expression:

$$\mu = \frac{1}{2}\left[1 - \frac{V_T^2}{V_L^2 - V_T^2}\right]$$

where μ is Poisson's ratio.

SEISMIC INSTRUMENTS

Seismic equipment used in engineering investigation is classified as either multichannel or one-channel. Earlier instruments were mechanical but now almost all in use are electrical. Operation of either type is fairly simple, but their proper application and interpretation require a skilled geophysicist having a knowledge of local geology or a geologist with extensive seismic exploration experience.

MULTICHANNEL EQUIPMENT

Portable multichannel equipment most commonly used in refraction surveys for engineering applications consists of an amplifying and recording unit, usually powered with batteries, and a multiple array of equally-spaced geophones which receive the refracted energy from the ground usually induced by means of explosives or some type of impactor. A number of the systems available use 12 geophones while others operate with 24. Geophone spacing in engineering applications is usually 10 to 50 feet. (In oil exploration which covers larger areas and greater depths than engineering studies, some systems use 48 geophones.)

Geophones, also termed detectors or seismometers, respond to the vertical component of the ground's motion resulting from the seismic waves generated by the exploration crew. The geophones convert these seismic oscillations into electrical impulses. The electromagnetic geophone, the simplest and most widely used type,[59] consists of a coil rigidly attached to the case and extending inside a spring-attached magnet. Movement of the ground causes relative motion between the coil and magnet which in turn produces an electromotive force that is proportional to the velocity of the motion.

Three other types of geophones in use are: the variable reluctance, the capacitive, and the piezoelectric (pressure) types.

Electric impulses from the geophones are transmitted to amplifiers. Output from the amplifiers is transmitted to an oscillograph which transforms the electric currents by means of galvanometers into electric oscillations.[59] These oscillations are recorded on photographic paper which is developed in the field or on direct print recording paper which needs no chemical processing.

The seismic records or seismograms are produced on paper marked by vertical lines which represent 0.01 second per division, allowing readings to be made in milliseconds. Oscillating lines in a horizontal direction show the vibrations picked up by each geophone (see Figure C-1). The instant of the explosion or hammer impact is shown on one line or trace and the instant of first wave arrivals at each geophone is determined visually from inspection of the seismogram.

Complete sets of portable multichannel equipment, including the power supply, weigh from 100 to 500 pounds. Various systems commonly used in engineering applications may be purchased within a price range of $5,000 to $20,000.

Multichannel equipment is usually used when the depth to be investigated exceeds 50 to 75 feet or in areas of complex geology. Stam[60] reports that a 12-channel system requires a crew of 7 to 10 men for efficient operation and that in routine seismic profiling an average of 5 to 15 depth determination may be made per day with depths ranging up to 500 feet. Paterson[61] states that a 2-man crew can obtain 15 to 20 determinations per day with multichannel equipment at a depth range of 0 to 50 feet.

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[59]Dobrin, Milton R. Introduction to Geophysical Prospecting. 2nd ed. McGraw-Hill, New York. 1960.

[60]Stam, J. C. "Modern Developments in Shallow Seismic Refraction Techniques." Geophysics, Vol. 27 No. 2 (April, 1962). pp 198-212.

[61]Paterson, Norman R. "Portable Facsimile Seismograph - The Equipment and Its Application." Mining in Canada, (Dec., 1967-Jan., 1968). Reprint.

Irving[62] reported a cost summary for all seismic work performed during 1962 by the Bureau of Soil Mechanics of New York State using pickup truck-mounted 12-channel equipment (Table 26). Average cost per depth determination was $40.50. These figures include the cost of preparing outcrop maps and geological reports considered essential to the seismic investigations.

Table C-2. Summary of cost analysis (1962).

Factor		Value
Production		
No. of determinations (avg. of 2 shots)		2,489
No. of ELF (equiv. linear feet of drilling req'd to determine depth.)		138,570
No. of determinations/day/party		5
No. of ELF/day/party		278
No. of stations/day/party		10
Unit cost		
Per determination	$ 40.50	
Per ELF	$ 0.728	
To maintain each party/working day	$ 202.00	
Total cost	$100,895.00	

Less commonly used in engineering application is the more expensive heavy duty petroleum exploration type seismograph equipment. Mossman and Heim[63] describe the successful application of the VIBROSEIS® system to depths of 600 feet on a major tunnel exploration program within the city of Chicago. Refraction methods were employed to determine the depth to bedrock which ranged down to 150 feet and reflection methods indicated the top of an important dolomite some 600 feet deep.

This major survey provided subsurface geologic information over a 920-square mile area with over 6,700 observation points determined

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[62]Irving, Francis R., "Seismic Surveying Methods, Equipment and Costs in New York State." Highway Research Record No. 81. (1965). pp 2-8.

[63]Mossman, R. W. and George E. Heim. "Seismic Exploration Applied to Underground Excavation Problems." Proceedings of North American Rapid Excavation and Tunneling Conference (Chicago, 1972). AIME, New York. 1972. pp 169-192.

along 420 line miles at an approximate cost of $127 per observation point.

The VIBROSEIS system inputs energy into the earth, at a controlled frequency and amplitude, for a few seconds by means of hydraulic vibrators. It uses 20 electromechanical detectors which pick up refracted and reflected waves and send corresponding electrical signals to amplifying and recording instruments. This current's phase, frequency, and amplitude were sampled at intervals of one to four milliseconds and recorded on digital tape for subsequent computer data processing.

The system requires two to four vibrators, a recording truck, two trucks for hauling and distribution of the seismometers and connecting cables, a survey pickup, personnel carrier, vibrator service truck, and permit and supervisor's cars.

Advantages of the VIBROSEIS system are:

1. Ability to operate within heavily populated urban areas with a minimum of disturbance.
2. Frequency, amplitude and duration of energy input are controllable.
3. Less affected by external "noise" than other systems.

The principal disadvantages are:

1. High cost compared to the smaller scale systems.
2. Lack of portability which limits areas of use.

ONE-CHANNEL EQUIPMENT

Standard single-channel seismographs are generally simpler and cheaper than multichannel systems but perform a similar task: measurement of the travel time for the first arrival wave to go from the shot or impact point to the geophone. Standard one-channel equipment includes a timer, a geophone (same as used with multichannel), and a sledge hammer or impactor. Some models display wave arrivals on a cathode-ray image tube which may be photographed for a permanent record. Others give direct digital readings of travel time.

Another system eliminates the requirement for a field seismograph by using a special tape recorder, a geophone, and a tamper. The recorder contains a digital integrating circuit and computer-type 4-track tape to record the field surveys. Playback and interpretation may then be carried out in the office.

The one-channel system, usually operated with a two-man crew, requires 12 shot, or hammer, points to determine the same amount of information obtainable from a single shot tested with a 12-channel system. In operating with one-channel equipment, the single geophone is usually kept in one location and the impact point is moved outward point by point. Sometimes a common impact point is used instead and the geophone is moved outward.

Typical one-channel systems weigh only 50 to 100 pounds complete and cost between $2,000 and $4,000.

One-channel equipment is generally most applicable in geologically uncomplicated areas to depths of 50 to 75 feet. One-channel is usually faster than multichannel equipment for shallow work where the hammer or impactor can be used. The approximate limitation on the use of hammers is a 50 feet depth penetration and a 200-foot hammer to geophone spacing.[64] The use of explosives will increase the effective depth penetration of one-channel systems to at least 150 feet but at the changeover point, from hammer to explosives, it becomes more advantageous to switch to multichannel equipment.

A recently developed one-channel system called the Facsimile Seismograph is also now in use. The Facsimile Seismograph records not only the first cycle of the first-arrival seismic wave, but also the following cycles and later seismic waves. Each wave arrival produces a mark on electrosensitive facsimile paper allowing depth calculations to be made directly on the paper.

Advantages of this type of instrument are the permanent record produced and the recording of seismic pulses even during the presence of random noise. The instrument may also be used for shallow reflection surveying.

A complete Facsimile system weighs less than 100 pounds and costs between $3,500 and $5,000. It is best suited to shallow 40- to 75-foot depth work, but may be applied to depths of 100 to 150 feet using explosives.

Paterson[65] states that a two-man Facsimile seismograph crew can complete an average of 20 to 25 determinations per day in the depth range of 0 to 50 feet.

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[64] Stam, J. C. "Modern Development in Shallow Seismic Refraction Techniques." Geophysics, Vol. 27 No. 2 (April, 1962) pp 198-212

[65] Paterson, Norman R. "Portable Facsimile Seismograph - The Equipment and Its Application." Mining in Canada, (Dec.,1967-Jan.,1968). Reprint.

ACCURACY

In general, travel times may be recorded to greater accuracy with one-channel equipment. Recording accuracy varies from 1/4 to 2 milliseconds, depending on the equipment used.[66] Depth determination accuracies also vary with the complexity of the geology.

Hall[67] reports that seismic depth determination accuracies of ±10 percent down to 75 feet and ±15 percent down to 100 feet have been obtained.

Stam[68] references a 1951 paper by B. Hasselstrom which relates that in 59 cases using multichannel equipment, the average inaccuracy was less than 5 percent for depths varying between 6 and 300 feet.

Bigelow[69] provided a comparison (Table C-3) of actual depths determined by wash boring versus refraction seismic-determined depths

Table C-3. Comparison of seismic and wash-boring depth determination.

Seismic Error (feet)	Number of Locations
0 to 2.5	33
2.6 to 5.0	9
5.1 to 10.0	7
10.0	2

[66] Paterson, Norman R. "Portable Facsimile Seismograph - The Equipment and Its Application." Mining in Canada. (December, 1967 - January, 1968). Reprint.

[67] Hall, Geoffrey R. "Seismic Surveying Techniques." The Military Engineer, No. 396 (July-August, 1968). pp 269-270.

[68] Stam, J. C. "Modern Developments in Shallow Seismic Refraction Techniquies." Geophysics, Vol. 27 No. 2 (April, 1962). pp 198-212.

[69] Bigelow, Norman, Jr. "Seismograph Operations by Maine State Highway Commission." Highway Research Record No. 81 1965. pp 9-15.

for 51 points using multichannel equipment in highway investigation in Maine. The depths investigated were not given, but it is assumed that most were less than 50 or 75 feet.

Mossman and Heim[70] report that a comparison of the major VIBRO-SEIS survey conducted in Chicago with borehole information showed that depth to bedrock (down to 150 feet) was determined with an average error of 6 feet and the surface of the dolomite (at approximately 600 feet) was picked with an average error of less than 25 feet.

Reflection methods generally produce a higher degree of accuracy than refraction methods in depth determinations.

REFRACTION FIELD MEASUREMENTS

The common types of refraction surveys are termed spot determinations, continuous profiling, and fan shooting. The nature of the problem determines the technique and equipment used.

SPOT DETERMINATIONS

Single depth determinations may be made with either multi- or one-channel equipment at scattered locations when only general or limited information on the subsurface layering is sought or in areas of very simple geology. Errors may result with this method in the case of dipping bedrock.

CONTINUOUS PROFILING

When the geology is complex and when a high degree of detail is required, several depth determinaitons are carried out on a straight line with either multi- or one-channel equipment so that cross sections may be prepared. This profiling is the most common technique applied. Usually with multichannel equipment, the shots and geophones are laid out on a long line, shots are made at equally-spaced intervals, and the

[70]Mossman, R. W. and George E. Heim. "Seismic Exploration Applied to Underground Excavation Problems." Proceedings of North American Rapid Excavation and Tunneling Conference (Chicago, 1972). AIME, New York. 1972. pp 169-192.

geophone arrays are shifted ahead in equally-spaced moves. Shots are fixed and recorded at opposite directions from the geophone array (see Figure C-4). Shots A and A', which are fired successively, are picked up by geophones 1A to 12A; etc.

PROBLEMS

The refraction seismic method is generally usable only where each successively deeper layer has a higher velocity than the one above it. Thus a major limitation of this method is its inability to produce much information on slow velocity layers which lie beneath high velocity layers. Also seismic methods are based on the assumption that individual layers exhibit uniform velocity characteristics laterally. So when layers are encountered which change composition substantially across short horizontal distances, difficulties and errors in depth determination may result. In most areas a minimum amount of drilling is required to permit the reliable interpretation of seismic data.

In the situation where the surface is flat and the bedrock surface is sloping, seismic surveying and interpretation become more difficult than in the simple horizontal case. It is best to lay out profile lines normal to the strike of the bedrock surface and to run the lines in both directions to avoid errors.

Where the ground surface is sloping and the bedrock is either horizontal or sloping, survey lines should be planned to follow the contour of the land to avoid the necessity of extra readings and calculations.

Accuracy will be poorer and more depth points must be determined in areas having bedrock with an irregular surface, in contrast to those areas having bedrock with a planar surface.

Longitudinal wave velocity usually ranges from 1,000 to 2,500 feet per second in loose dry soils. As the longitudinal velocity in water is 4,800 feet per second the water table is sometimes mistaken for a

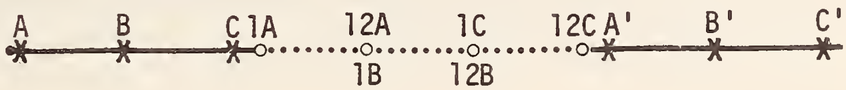

Figure C-4. A typical refraction profile.

firm soil stratum underlying a loose soil stratum. This emphasizes the requirement for drilled holes to assure proper interpretation.

FAN SHOOTING

Fan shooting is often used for tracing buried stream channels. With the one-channel system the geophone is set up at a point known to be over the channel and a series of shocks are produced along an arc at points equidistant from the detector. The same problem may be solved using multichannel equipment with the shot point.

REFLECTION FIELD MEASUREMENTS

Reflection seismic methods are used to a very limited extent in tunnel site investigations and other engineering applications. These methods are widely used in petroleum exploration, however, to determine depths to strata from 2,000 to 30,000 feet or more.

The reflection methods use explosives, vibrators, or impactors to input energy into the earth, then record and make use of the energy which is bounced off the surfaces of reflecting strata such as limestones and dolomites and picked up at the ground surface by geophones.

Of the equipment discussed previously, that most commonly used in reflection work includes the one-channel Facsimile seismograph and the multichannel VIBROSEIS system, both of which have the capability of recording the later arriving reflection waves as well as the early arriving refraction waves.

For tunnel investigations to depths greater than 500 feet, reflection methods may be used in place of, or in combination with, refraction methods. Advantages of the reflection methods include greater accuracy, a depth penetration of 2 to 3 times that of refraction at the same source to geophone spacing, and the lack of velocity inversion or hidden layer problems, allowing work on frozen ground or ice.[71]

Modern reflection surveying is often carried out by the popular technique known as the Common Depth Point (CDP) method. Other methods are correlation shooting and dip shooting.

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[71]Paterson, Norman R. "Portable Facsimile Seismograph - The Equipment and Its Application." Mining in Canada, (Dec., 1967 - Jan., 1968). Reprint.

Several reflection measurements are made to a single subsurface point from varying distances (Figure C-5)

where *G*1 is the first geophone location and
*S*1 is the first energy source location, etc.

These measurements are corrected as necessary and added together. This statistical averaging improves the quality of the information and reduces the effect of external noise and inaccurate individual readings. A digital computer is required for efficient processing of data.

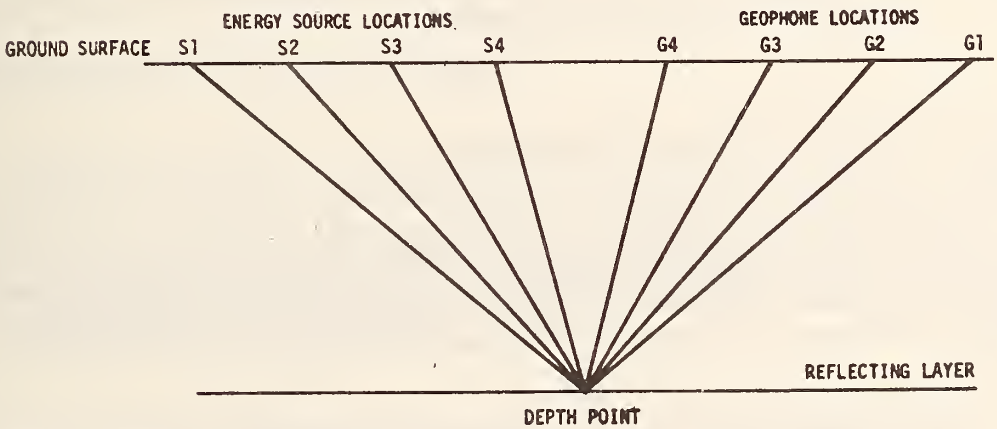

Figure C-5. Common depth point reflection paths. (Schematic).

CORRELATION SHOOTING

Where reflecting beds are persistent and easily identifiable so that there is no problem of correlation, the usual arrangement is to align all detectors on one side of the shot point. Shot distances vary from 200 to 2,000 feet to the middle receiver and receiver intervals are from 20 to 100 feet. The method gives high accuracy and maximum detail but at high cost. Small scale geologic structures of a size to be of interest in petroleum exploration may be located.

DIP SHOOTING

In areas of difficult correlation or where dips greater than a few degrees are present, the dip shooting method may be used. One of the following procedures may be followed:[72]

1. Shoot twice in the same hole, with receivers placed first on one side, then on the other.
2. Place receivers in one line, shoot first on one side, then on the other side.
3. Shoot once in one hole with half of the receivers on each side.
4. Shoot with receivers on one side and determine dip using absolute times and time gradient.

SITE INVESTIGATION APPLICATIONS

Both refraction and reflection seismic methods may be carried out in both rural and urban environments, including water covered areas. Because of "noise" problems in urban areas more time and patience is usually required than for a corresponding survey in an unpopulated area.

The principal applications of seismic methods to tunnel site investigations are:

- Determination of overburden thickness - configuration of bedrock surface
- Determination of depth of weathering

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[72] Heiland, C. A. Geophysical Exploration. Prentice-Hall, New York. 1946.

- Determination of depth to water table
- Location and tracing of buried channels
- Determination of elastic properties of subsurface materials
- Determination of geologic structural features - major faults, folds and cavities
- Location of different rock types
- Determination of rippability of rock
- Reconnaissance mapping for planning core drilling
- Location of sand and gravel deposits for construction purposes

DETERMINATION OF OVERBURDEN THICKNESS - CONFIGURATION OF BEDROCK SURFACE

The greatest amount of engineering seismic work to date has been directed at this application. Detailed knowledge of the bedrock surface is extremely vital if a tunnel is to be driven on or near the interface between alluvium and bedrock. Even in conditions where the principal portion of a tunnel is to pass through rock it may be necessary to learn alluvium thickness at the portal areas and highway approaches. Refraction methods should be used for this application.

DETERMINATION OF DEPTH OF WEATHERING

Refraction methods are often able to detect the depth of strong weathering in bedrock. A standard example would be the simple three-layer case of alluvium (slow velocity), weathered bedrock (slow to moderate velocity), and fresh bedrock (high velocity). Since weathering is often gradational the time travel curves may not show a sharp break, but instead may bend gradually.

DETERMINATION OF DEPTH TO WATER TABLE

Refraction seismic methods are usually capable of determining the depth to the upper surface of groundwater in porous alluvial material, but would seldom determine the water table if it occurs within the bedrock.

LOCATION AND TRACING OF BURIED CHANNELS

When tunnels are to be driven through alluvium or at its contact with bedrock, groundwater information is especially important. Undetected gravel-filled channels might allow major water inflows during the advance of tunnel workings which could cause major caving and pumping problems if intersected unexpectedly. The refraction fan shooting technique is most applicable to channel tracing.

DETERMINATION OF ELASTIC PROPERTIES OF SUBSURFACE MATERIALS

Longitudinal (compressional) and transverse (shear) wave velocities of subsurface materials determined by seismic methods may be used to calculate the value of Poisson's ratio in situ, and if the density is known the in situ modulus of elasticity (Young's modulus) may also be calculated. One-channel Facsimile seisographs or other systems equipped for recording later arrival waves would be best able to collect the necessary data for this type of application.

DETERMINATION OF GEOLOGIC STRUCTURAL FEATURES - MAJOR FAULTS, FOLDS AND CAVITIES

Both refraction and reflection seismic methods may provide indications of major geologic discontinuities such as faults, folds, or open cavities. Faulted sedimentary rocks with displacements greater than 25 feet might be detectable, but similar features cutting crystalline rocks would probably go unnoticed unless a paralleling fractured zone of sufficient width were present in which case velocity differences indicating the feature might be found.

LOCATION OF DIFFERENT ROCK TYPES

Rocks with different wave velocities and having a steeply dipping contact could be delineated by seismic methods - for example, a thick diabase dike intersecting sedimentary formations.

DETERMINATION OF RIPPABILITY OF ROCK

Refraction seismic methods are routinely applied to preexcavation analysis of near surface rock which is to be removed, such as cuts required at tunnel portals and highway approaches. Velocity determinations permit generally reliable predictions to be made on whether the material is rippable or will require drilling and blasting. Figure C-6 is an example of a chart showing velocity ranges through which common soil and rock material may be readily loosened with heavy duty tractor-ripper combinations.[73] These charts are available from various construction equipment manufacturers for their products.

RECONNAISSANCE MAPPING FOR PLANNING CORE DRILLING

Core drilling is one of the most necessary and usually the most expensive technique employed in tunnel site evaluations. Since only a limited number of holes can be afforded, it is imperative that all of them be placed where they can contribute a maximum of vital information. It is inadvisable to blindly drill holes on a regular pattern--instead, a maximum number should be used to test the more complex, least understood areas, leaving fewer holes for investigating the more thoroughly known areas. In this connection low-cost reconnaissance seismic surveys can be very helpful in providing subsurface geological information valuable in planning the drilling program.

LOCATION OF SAND AND GRAVEL DEPOSITS FOR CONSTRUCTION PURPOSES

Although not a primary factor in tunnel site evaluation, one consideration in a complete study is the location of sand and gravel deposits which may be required for concrete used at the portals and in linings. Refraction seismic surveys are often successful in providing information on such deposits.

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[73]Juergens, R. E. "Sub-soil Surveys Eliminate Pre-Bid Guesswork." Construction Methods and Equipment (September, 1962). Reprint.

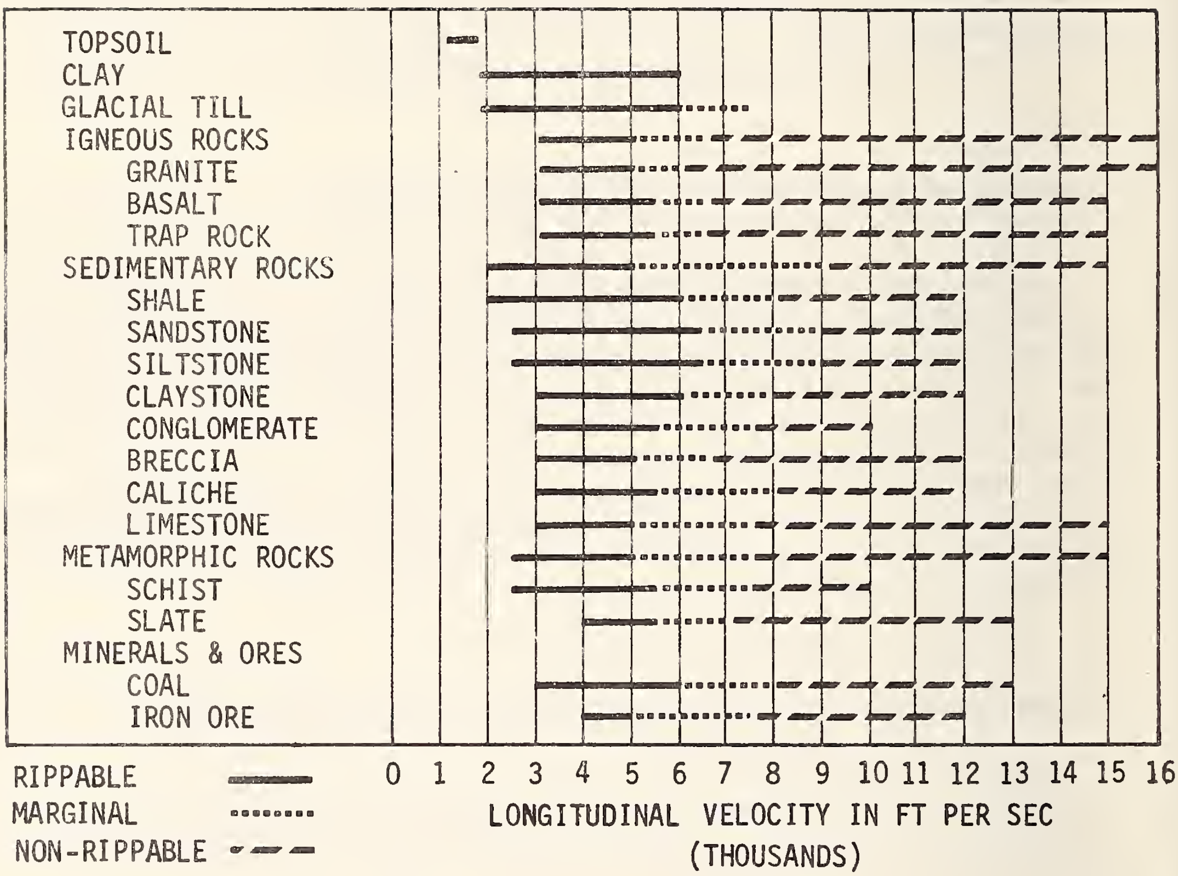

Figure C-6. Rippability of subsurface materials related to longitudinal seismic velocity.

SEISMIC METHODS AND THE ELECTRICAL RESISTIVITY METHODS

The refraction seismic method presently appears to be the most applicable and most commonly used surface geophysical exploration tool for subsurface investigation of tunnel routes and other engineering sites. It is closely followed in overall popularity and value by the electrical resistivity method. The reflection seismic method could probably be ranked third in total importance in this regard. Of course for any given situation this ranking may be quite different. To a large extent seismic and resistivity methods have overlapping applications, but in other areas each is able to supply unique information so while they could be considered competitive for some applications, in others they are complementary and used together to good advantage.

An interesting comparison of applications between an early model one-channel refraction seismograph and an electrical resistivity system was made by Lawson, Foster, and Mitchell.[74] Table C-4 summarizes this comparison.

Table C-4. Relative merits and limitations of resistivity and refraction seismic systems.

Qualification	Electrical Resistivity	Seismic Refraction
Locates bedrock surface	Usually	Yes
Distinguishes between hard and soft sandstones	No	Yes
Detects soft layers beneath hard layers	Usually	No
Is affected by wind, traffic, etc.	No	Yes
Is affected by fences, buried cables, and overhead wires	Sometimes	Seldom
Differentiates boulder zones from bedrock	Usually	Usually
Locates water table		
(a) in soil	With difficulty	Usually
(b) in rock	Yes	Very difficult

[74]Lawson, C. E., W. R. Foster, and R. E. Mitchell. "Geophysical Equipment Usage in the Wisconsin Highway Commission Organization." <u>Highway Research Record No. 81</u>. (1965). pp 42-47.

Table C-4. Relative merits and limitations of resistivity and refraction seismic systems (continued).

Qualification	Electrical Resistivity	Seismic Refraction
Aids in determining rippability of bedrock	No	Yes
Usable in water saturated areas	Yes	No
Usable in frozen ground	Yes	No
Operates under wet or damp conditions	Yes	No
Locates suitable quarry rock	Possible	Yes
Locates suitable gravel deposits	Yes	Sometimes
Affected by uneven ground surface and uneven subsurface layers	Yes	Yes
Requires experience operator	Definitely	Definitely

BOREHOLE SEISMIC METHODS

Seismic investigations in boreholes can be used to determine important engineering properties of rocks at depth. An important use of this seismic method is in the determination of average wave velocities in various rock units. This information is vital to an accurate interpretation of subsurface geology from seismic records.

Comparisons of velocities obtained from field and laboratory seismic methods give some indication of the degree of jointing, faulting or other irregularities in the bedrock between shot point and detector. Seismic wave velocities through in situ rock are usually slower than those observed in core specimens because of joints, faults, porous, or altered zones in the rock in place or due to the presence of previously undetected low velocity rocks of another type.

The fundamental relationship between seismic velocity and the principal elastic constants is:

$$V_L^2 = \frac{E(1-\mu)}{(1-2\mu)(1+\mu)}$$

where V_L is the longitudinal (compressional) wave velocity,

ρ is the density,

E is the modulus of elasticity (Young's modulus), and

μ is Poisson's ratio.

The most common size of core hole (NX - approximate 3-inch diameter) drilled for subsurface investigations to a depth of 500 feet is sufficiently large for placement of seismic shots or geophone arrays.

Figure C-7 shows various arrangements of shot points and detector arrays commonly used in borehole-to-surface and borehole-to-borehole seismic velocity investigations. Standard seismic equipment is used in these studies.

ELECTRICAL RESISTIVITY METHODS

A surface electrical resistivity survey is a procedure for determining depths to geological interfaces wherein separations of electrodes in an array are increased by increments. A plot of observed apparent resistivity versus electrode separation, when compared with similar plots for theoretically computed cases, yields estimates of the depths to the interfaces and the resistivities of the strata.

The principal applications of resistivity surveys for tunnel site studies are:

1. Determination of overburden thickness - Configuration of bedrock surface.
2. Location and configuration of groundwater concentrations.
3. Determination of physical soil and rock characteristics.
4. Determination of geologic structural features - Major faults and folds.

PRINCIPLES

The following diagrams show idealized representations of current flow through a homogeneous subsurface stratum (Figure C-8) and through a two-layer section (Figure C-9).

The electrical resistivity of soil or rock formations limits the amount of current flowing through the material when an electrical potential is applied. Resistivity is commonly measured in units of ohm-square meters per meter which reduces to ohm-meters. True resistivity can be measured only in homogeneous ground. Compositional variations almost always present within the range of the instruments in actual practice results in readings which are termed apparent resistivities.

Resistivities of soil and rock vary considerably depending upon

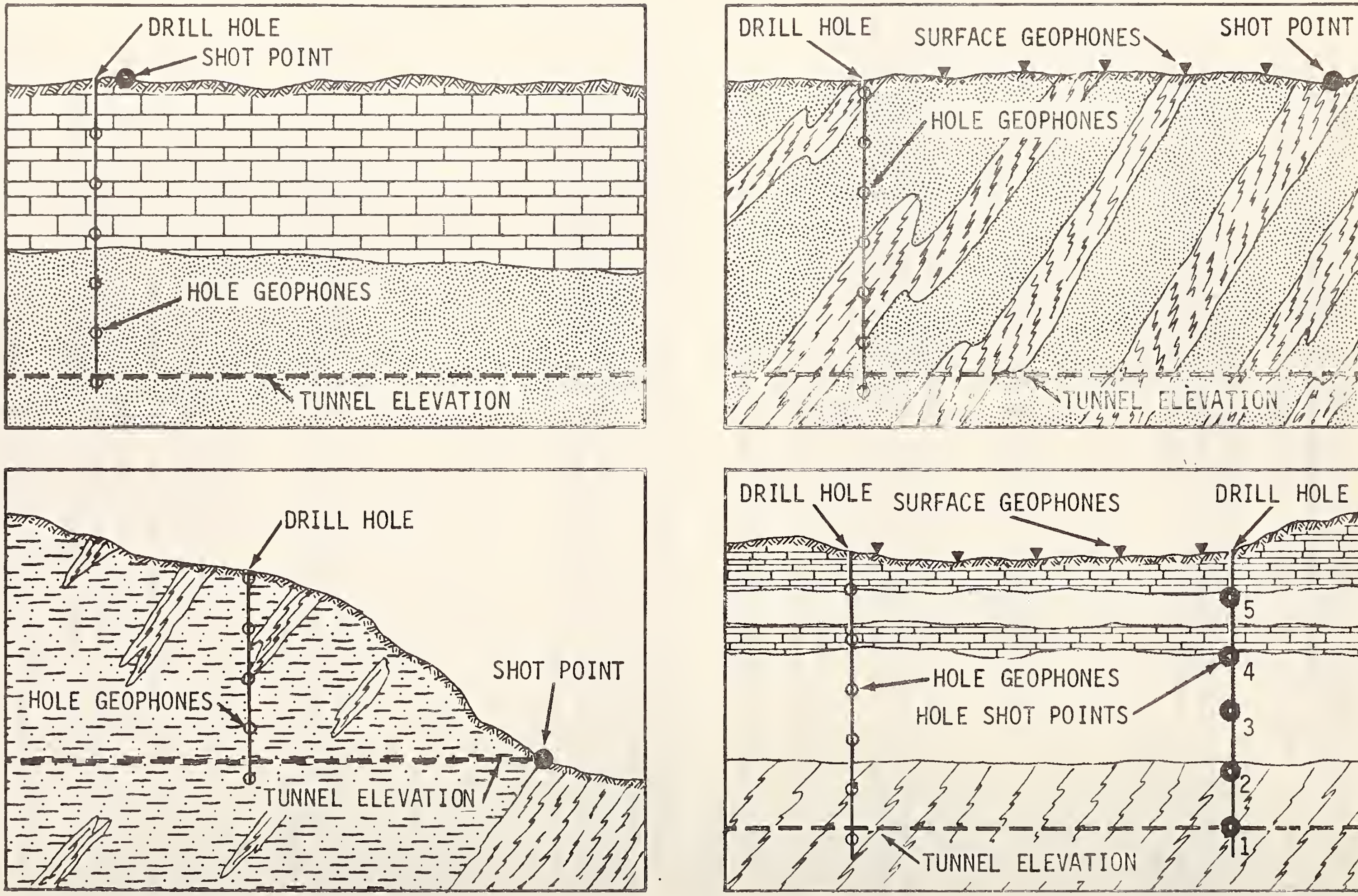

Figure C-7. Some possible arrangements for seismic investigation utilizing drill holes (Schematic.)

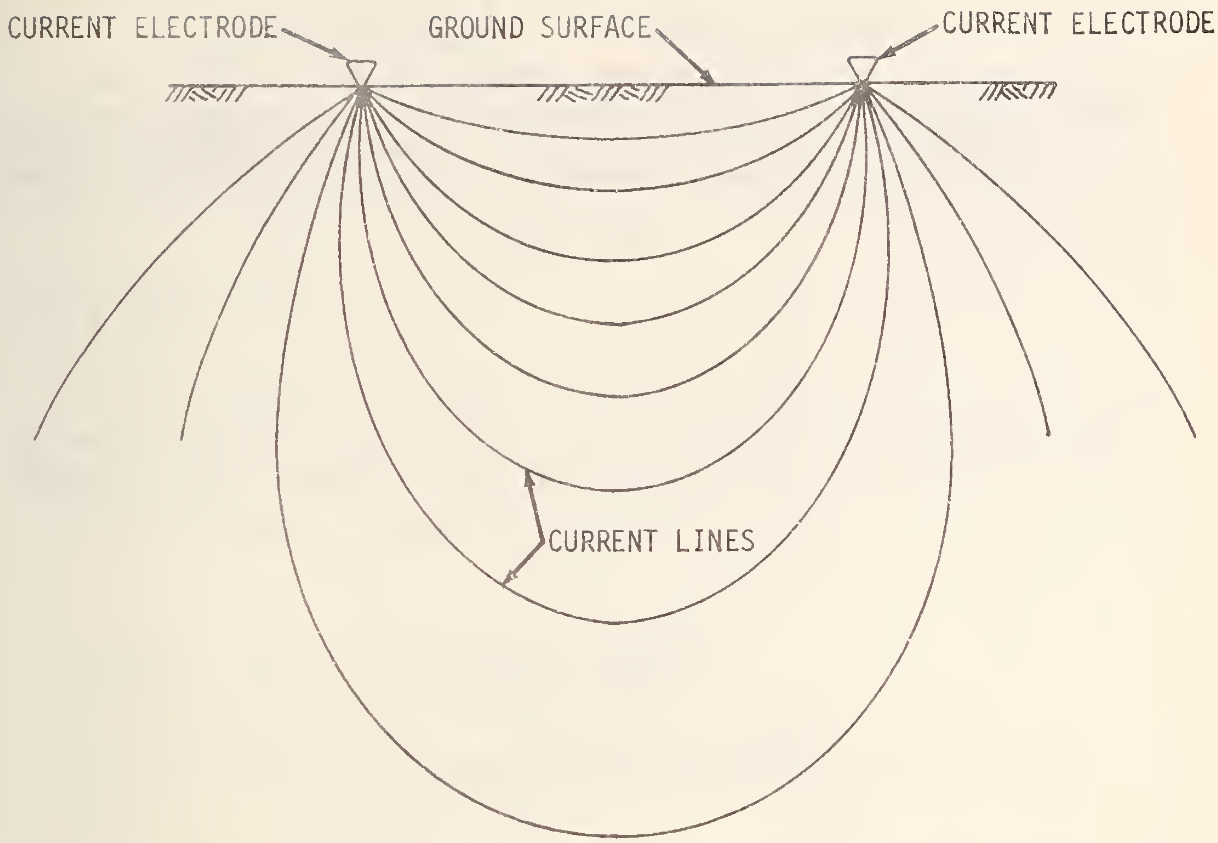

Figure C-8. Lines of current flow in a homogeneous subsurface stratum. (Schematic.)

type of material, density, porosity, water content, salinity, and temperature. Gisco[75] reports that the median resistivity of rocks within 500 feet of the surface is 150 ohm-meters. According to Todd[76] igneous and metamorphic rocks show values from 100 to 100,000,000 ohm-meters whereas sedimentary and unconsolidated rocks have values of 10 to 10,000 ohm-meters.

Electrical resistivity survey equipment consists of a power source (reversible direct current or low frequency alternating current are used to minimize polarization effects), a milliameter for measuring the current applied, a potential measuring device much as a direct current vacuum tube voltmeter or a potentiometer, reels, wire, metallic

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[75] Gisco General Catalog. Geophysical Instrument & Supply Co., Denver, Colorado. 1972.

[76] Todd, David K. Ground Water Hydrology. John Wiley and Sons, New York. 1959.

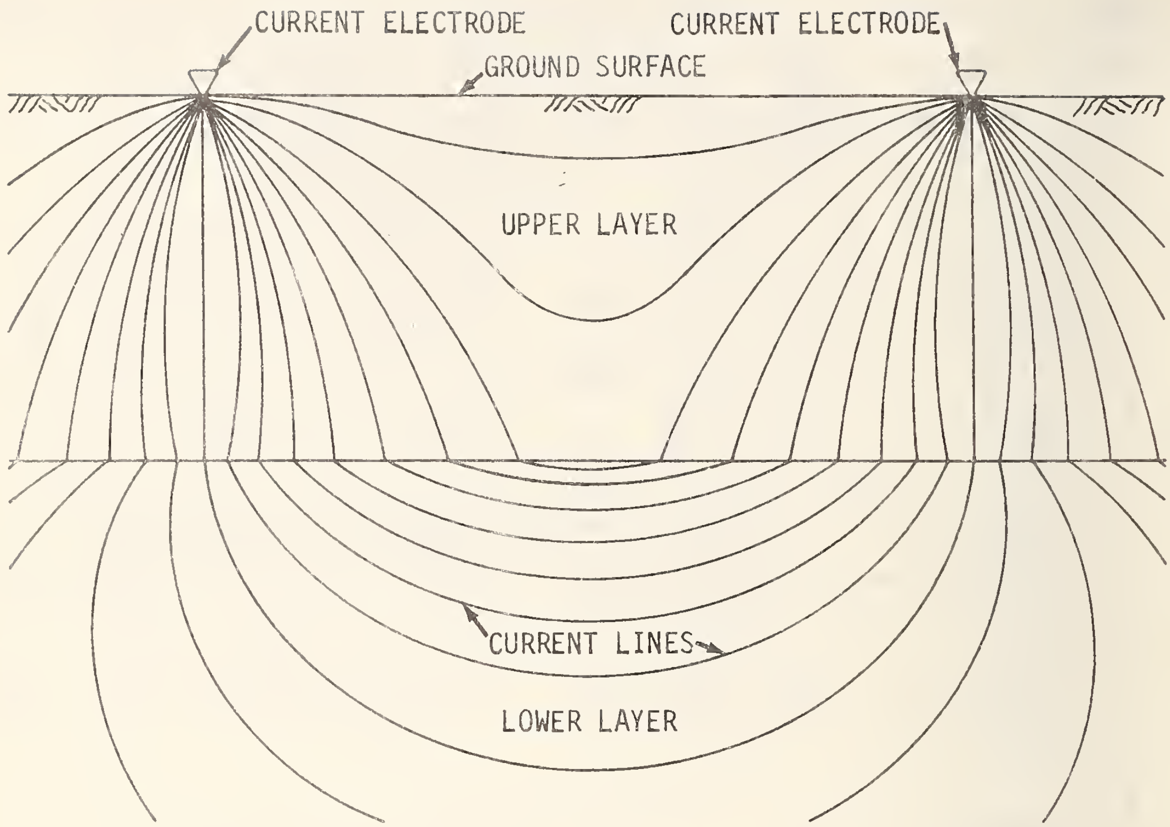

Figure C-9. Lines of current flow in a layered section. (Conductivity of lower medium is substantially greater than that of upper.) (Schematic.)

stake current electrodes, and nonpolarizing porous cup potential electrodes.

Most resistivity surveys utilize four electrodes placed along a straight line in various spacing arrangements. The depth of survey measurement is controlled by the spacing of the electrodes.

Variations in earth resistivity may be determined both laterally and vertically. When studying the lateral change in resistivity, for example, across facies changes in alluvium or across dikes or high-angle fault zones, a fixed electrode separation is maintained and the array is moved as a unit along the traverse line. This method of measuring resistivity to a constant depth is known as horizontal profiling. When studying the change in resistivity with depth, the electrode spacing is gradually increased outward from a central point along the traverse line, a technique known as vertical sounding or depth probing.

METHODS

Many different electrode arrays have been tried. The array used is governed by the nature of the problem, personal preference, and size of crew available. The most commonly used electrode patterns are the Wenner, Schlumberger, Modified Schlumberger, and Dipole arrays.

WENNER ARRAY

In this widely-used method, the four electrodes are spaced at equal distances along a straight line. The outside two are used to transmit current to the ground and the inside two are used to measure voltage.

For the Wenner electrode configuration (Figure C-10),

$$PA = (2\pi a)\frac{V}{I}$$

where PA is apparent resistivity,

a is the distance between adjacent electrodes

V is the voltage difference between the potential electrodes, and

I is the applied current.

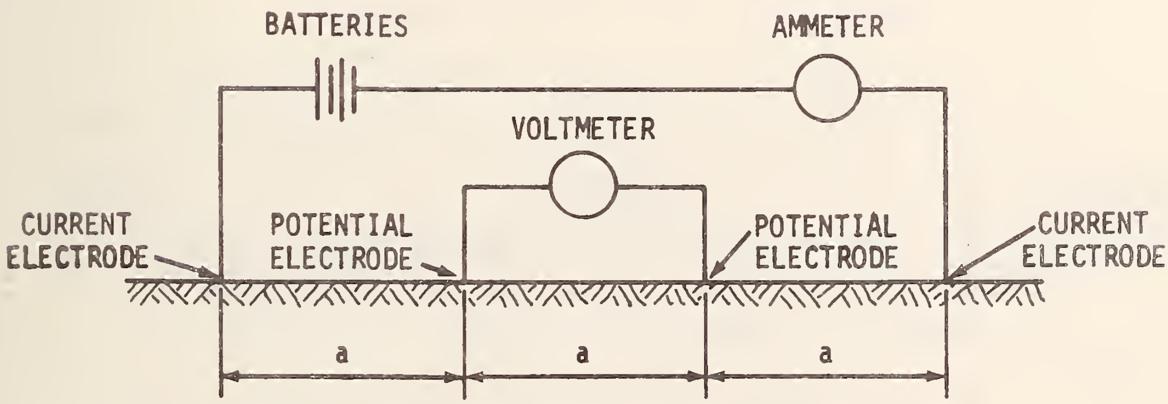

Figure C-10. Wenner Array. (Schematic.)

For depth sounding, all four electrodes are moved outward from the center point, keeping adjacent ones equidistant. The depth surveyed is roughly equivalent to the spacing between adjacent electrodes.

The sending and receiving instruments are located in a common position where observations are made by the geophysicist. When the electrode separation is large four assistants are required, one at each electrode.

SCHLUMBERGER ARRAY

In this method all four electrodes are placed symmetrically along a straight line but the inner two, used to measure voltage, are spaced less than one-fifth the distance between one of them and an outer, current injecting electrode (Figure C-11).

With the standard Schlumberger method,

$$PA = \left(\pi \frac{L^2 - b^2}{4b}\right) \frac{V}{I}$$

where L and b are the current and potential electrode spacings, respectively.

To make depth soundings the outer electrodes are moved outward in steps while the inner electrodes are kept stationary unless the voltage observed between them becomes too small to measure. In addition to the geophysicist, two assistants are required, one for each moving electrode.

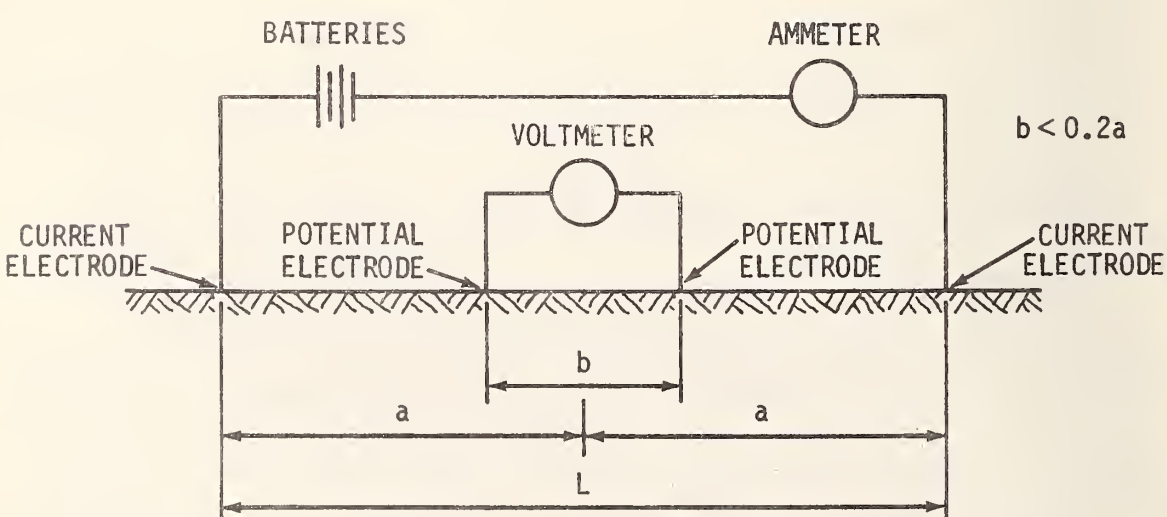

Figure C-11. Schlumberger Array. (Schematic.)

MODIFIED SCHLUMBERGER ARRAY

In this method one of the current electrodes of a Schlumberger array is moved far out along the survey line to perhaps two or three times the distance for which surveying is desired. The other current electrode is then moved outward from the measuring electrodes step by step.

The principal advantage of this method is that only a two-man crew is required, a geophysicist and one assistant for the single moving electrode.

DIPOLE ARRAY

In this method the two current electrodes are separated from the two measuring electrodes (Figure C-12). The electrode spacing and line orientation are arbitrary. This array is useful for deep investigations where it is not possible to orient all of the electrodes in a straight line. However, measurements made with irregular spacings and orientations are very difficult to interpret.

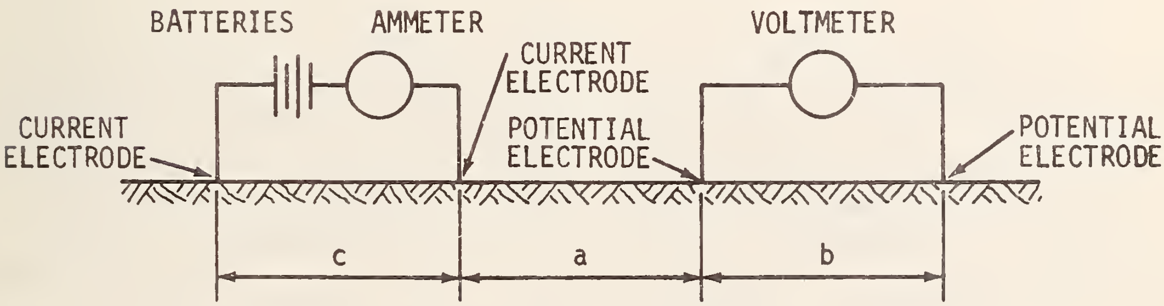

Figure C-12. Dipole Array. (Schematic.)

The electrical resistivity method has a number of engineering geological applications related to tunnel site investigations and is also widely used in other civil engineering studies and groundwater exploration, and to a lesser extent in the fields of mineral and oil exploration. This method is limited, however, to use in rural areas away from buried pipelines, cables, railroads, and wire fences which disturb the earth's electrical field. Electrical resistivity methods are often used in conjunction with seismic refraction surveys because geologic conditions undetectable by one method are often resolved with the other. These two systems are the most commonly used surface geophysical techniques in civil engineering site investigation.

DETERMINATION OF OVERBURDEN THICKNESS - CONFIGURATION OF BEDROCK SURFACE

This is probably the most common application of resistivity surveys in site investigations and is particularly important if a tunnel is to be driven on or near the contact between alluvium and bedrock. Where tunnels are to be driven principally through rock, the depth to bedrock is important, especially at portal areas. Properly applied electrical resistivity surveys can commonly determine overburden thickness within accuracies of 10 to 15 percent at most points, but occasional measurements can be extremely erroneous.

LOCATION AND CONFIGURATION OF GROUNDWATER CONCENTRATIONS

Knowledge of groundwater conditions is vital to the planning of tunnel excavations below the water table. Groundwater flow into a tunnel may present dewatering and tunnel stability problems. Mineralized water may damage tunnel lining materials. Lowering of the water table due to tunnel excavation may have an adverse effect by causing domestic and irrigation wells to go dry. Among the hydrologic factors which should be considered for soil and rock overlying a proposed tunnel route are water table levels, porosity, permeability, water composition, pressure, hydraulic gradient, discharge, and recharge.

Electrical resistivity and surface-potential are the most useful surface geophysical methods available for defining shallow groundwater concentrations. The use of resistivity methods to locate groundwater concentrations is best applied to gentle-dipping aquifers such as porous sandstone or gravel. The method is much less effective in study of strongly deformed sediments or crystalline rocks. In relatively porous strata the resistivity is governed more by the amount and composition of contained water than by the nature of the rock itself. Figure C-13 shows relative resistivities arising from a variety of groundwater conditions.

Surface electrical resistivity surveying should be considered only a first step toward evaluating groundwater conditions with regard to their effect on tunnel construction. Considerable further groundwater information should be sought from test holes drilled along the proposed tunnel route through various types of borehole geophysical logging (especially electrical methods), borehole pressure tests, pumping tests, and laboratory analyses.

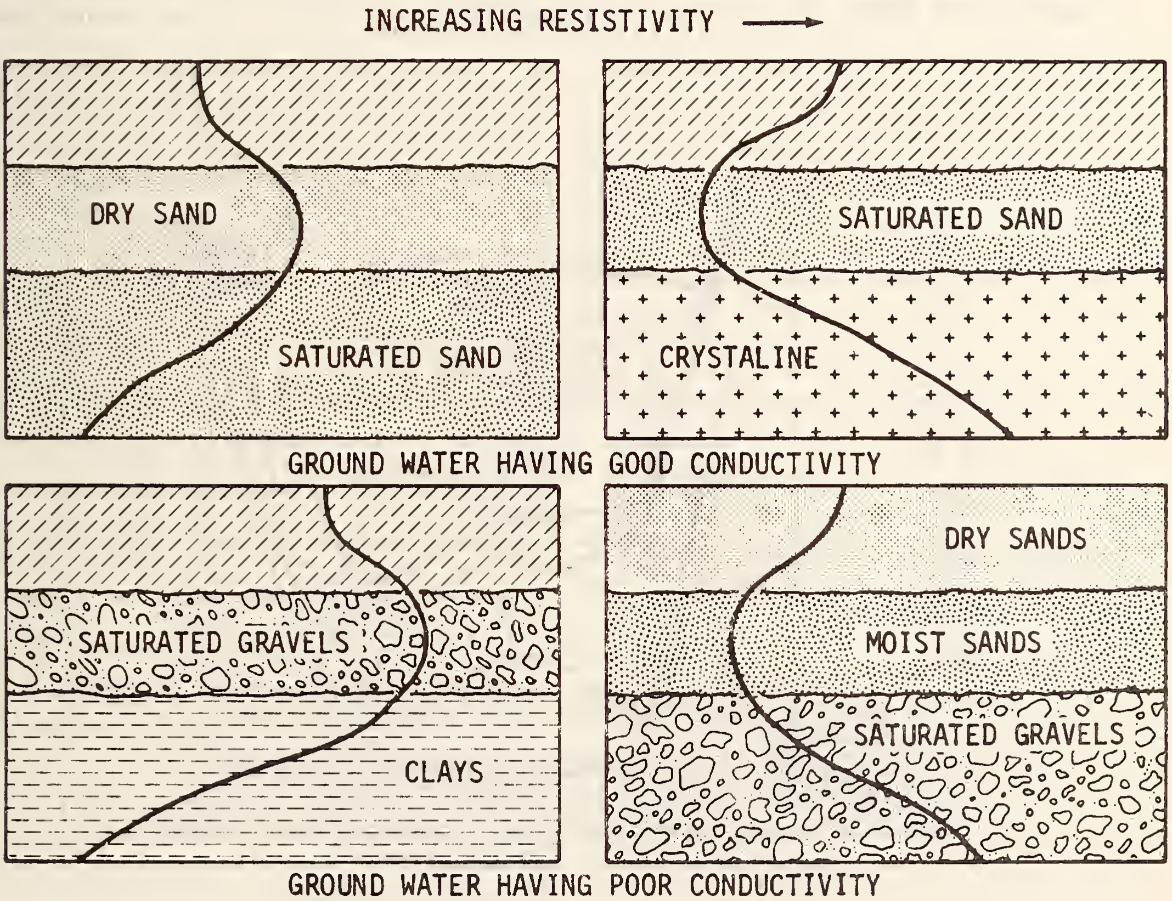

Figure C-13. Various resistivity indications on conductive and non-conductive groundwaters.

Used in correlation with detailed drill hole logs, considerable indirect information about the subsurface material may be deduced from electrical resistivity data. The principal characteristics determinable are relative degree of consolidation and porosity. Highly fractured zones saturated with water may be delineated.

DETERMINATION OF GEOLOGIC STRUCTURAL FEATURES - MAJOR FAULTS AND FOLDS

Prominent geologic structures observed by overburden may be discerned through the observation of differences in electrical resistivity. Figure C-14 shows an example of the effects of a major fault zone which

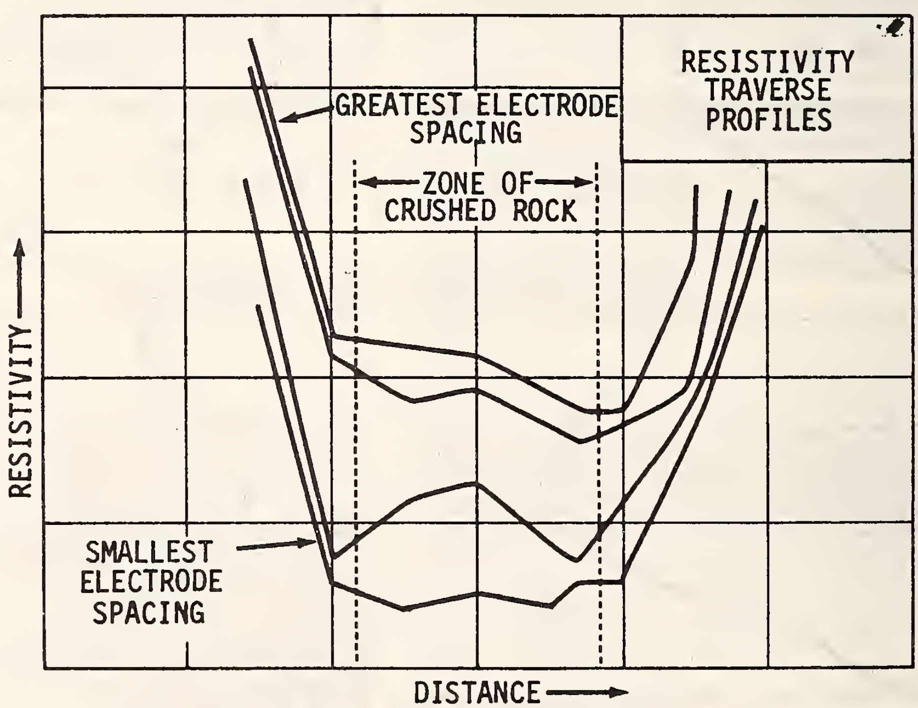

Figure C-14. Location of fault zone by resistivity mapping.

contains much more water than the solid unfractured granite. Figure C-15 shows the resistivity effects due to a major fold.

LOCATION OF SAND AND GRAVEL DEPOSITS FOR CONSTRUCTION PURPOSES

Sand and gravel are required for concrete used in tunnel portals and interior linings. The presence or absence of deposits nearby is seldom considered a factor which would cause either approval or cancellation of a proposed tunnel, however it is important in estimating cost. Exploration for sand and gravel is commonly carried out in the vicinities of tunnels with surface electrical resistivity methods in the hope that local deposits can be found to avoid the high cost of transportation from distant points. An example of this application is shown in Figure C-16.

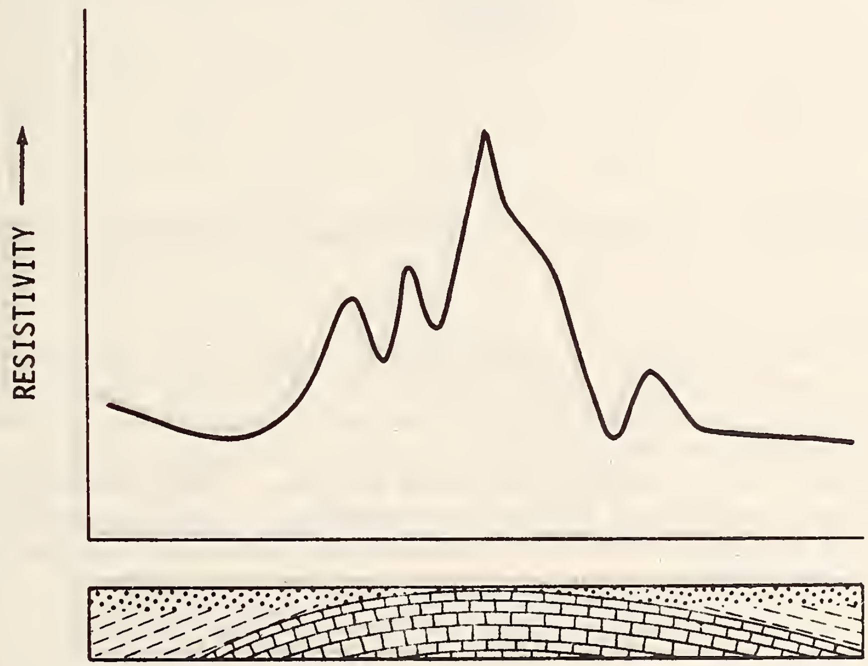

Figure C-15. Location of buried anticline by resistivity mapping.

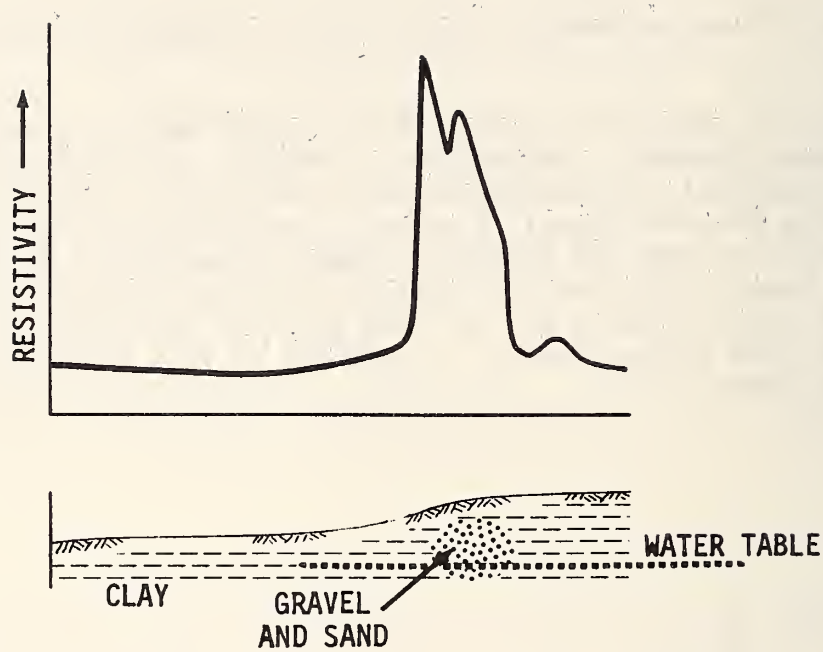

Figure C-16. Location of gravel lense by resistivity mapping.

Subsurface information sought in the applications discussed, especially in geologically complex areas, is often best developed through the combined use of both electrical resistivity and seismic refraction surveying. These two methods are the most widely used surface geophysical methods in civil engineering site studies. The two methods are complementary, evaluating different properties. Used together, a more complete and accurate picture of subsurface conditions can be obtained than would result from either method used alone.

INTERPRETATION

As with all geophysical exploration methods, electrical resistivity techniques are most effectively applied and interpreted by thoroughly experienced geophysicists in close cooperation with experienced geologists familiar with the geology of the area being studied. Based on past experience and a preliminary knowledge of geologic conditions at hand, the geophysicist selects the proper equipment, electrode arrays, and orientation and number of survey lines to determine the required information. A number of strategically located test holes, carefully logged geologically, are vital to a geophysicist's interpretation of resistivity data as the method does not produce precise depth or condition information. A close-spaced grid of test holes would supply the most thorough and exact information but is prohibitively expensive and time consuming. Therefore each unique situation is best evaluated by some optimum mixture of geophysical techniques and reference drill holes.

EQUIPMENT

Electrical resistivity surveys have been utilized for several decades and many sophisticated interpretation techniques have been developed, some of which utilize computer processing of data.

Several manufacturers produce electrical resistivity equipment, each commonly offering several units having different depth capacities and sensitivities. Equipment sensitivity is related to both the power capability of the current source and the sensitivity of the measuring instruments. A useful comparison between sets of equipment may be made by calculating the ratio of the minimum voltage that can be measured with the desired precision to the maximum current that can be supplied to the ground. The maximum current figure to be used should be the smaller of either the maximum current which can be handled by the equipment or the maximum voltage provided in the current supply divided by a nominal contact resistance value, such as 150 ohms. The smaller the ratio, the more sensitive the equipment. Different applications require equipment having different sensitivities, for example:

Application	Sensitivity Ratio: $\frac{\text{Minimum Voltage}}{\text{Maximum Current}}$
Exploration for gravel deposits or ground water to a maximum depth of 50 feet.	0.1 - 1 ohm
Studies of overburden thickness and determination of geologic structure down to 1,000 feet.	.001 ohm
Deep surveys for locating crystalline basement rock below 10,000 to 15,000 feet of sedimentary rock.	.000001 ohm

The proper equipment must be selected for each application. Highly sensitive equipment should not be used if unneeded as its use will greatly reduce the number of measurements which can be made in a day. According to Gisco,[77] a 2-man crew using relatively low sensitivity equipment for studying shallow gravel deposits can make depth soundings to 50 feet at 10 to 40 locations during a 10-hour day. Using more sensitive equipment, the number of readings would be decreased by a factor of 2 to 10 with no improvement in the quality of results.

Electrical resistivity surveys usually employ heavy duty dry cell batteries as a source of direct current to the current electrodes.

MAGNETIC METHODS

Magnetic geophysical methods for investigating subsurface conditions involve measurement of the magnetic field over an area of interest, comparing differences in intensity from place to place. Magnetic measurements with instruments called magnetometers are most commonly carried out by airborne methods although detailed surveys are often made on the surface and occasionally readings are taken in boreholes. Magnetic surveys have not been used extensively in tunnel site investigations, but in many cases valuable preliminary information could be gained from airborne and/or surface methods which are widely used in mineral and petroleum exploration.

Principal applications of magnetic surveying to tunnel site investigations include:

1. Determination of major structural features (faults, folds).
2. Location of different rock types.
3. Determination of depth of weathering and distribution of alteration.

PRINCIPLES

The earth's magnetic field resembles that of a bar magnet with its long axis oriented close to the earth's axis of rotation. The earth may be considered as a relatively uniformly polarized sphere with the magnetic field being vertical and directed inward at the north magnetic pole and vertical, directed outward at the south magnetic

[77] Gisco General Catalog. Geophysical Instrument & Supply Co., Denver, Colorado. 1972.

pole. At the equator the field is horizontal, pointing northward (Figure C-17). The magnetic field at any point is a vector which may be resolved into horizontal and vertical components.

The natural magnetic force field has both direction and intensity at every point on the earth's surface. Direction is a factor of location on the earth and intensity is a summation of the earth's magnetic field and the field due to local rock formations and mineral deposits. Since the earth's magnetic field is about the same over a limited area, differences in measured intensity are an indication of differences in composition of underlying alluvium and rock.

The unit of measurement for magnetic intensity is the gauss which is numerically equal to 1 dyne or 1 oersted. Since the total magnetic field of the earth is normally about 1/2 oersted, a smaller unit termed a gamma, defined as 10^{-5} oersted, is the commonly used unit of intensity for field work. The earth's magnetic field, measured at the surface, normally has an intensity of 35,000 to 70,000 gammas.

The magnetic intensity at any specific point is continually changing due to several factors. Certain changes are cyclical and others appear to be random. Some of the changes appear to be related to sun spot activity; variations of several hundred gammas are common during magnetic storms. It is inadvisable to attempt magnetic surveys during such occasions. Depending upon the sensitivity of a magnetic survey, corrections must be made for some of the intensity changes taking place. The change in intensity with time at a specific point is monitored by taking readings at intervals with a base instrument or by returning the field instrument for periodic checks.

Since variations in the magnetism of subsurface materials are the features sought by magnetic surveying, an explanation of the magnetization of rocks is vital to an understanding of survey results.

Rocks and minerals are classified according to their behavior when placed in a magnetic field. The most common group are paramagnetic, materials which tend to align their long dimensions in the direction of the field. Most igneous rocks belong to this group. Diamagnetic materials such as quartz, graphite, rock salt, gypsum, and anhydrite tend to align their long dimensions across the field. Sulphur, calcite, and other materials are often observed to be nonmagnetic, that is their paramagnetic effects equal their diamagnetic effects. In the earth's magnetic field, paramagnetic materials tend to increase the normal field, diamagnetic materials tend to decrease it, and nonmagnetic materials exert no effect.

The most important magnetic property of rocks is magnetic susceptibility which is a measure of rock's ability to produce their own magnetic fields. Susceptibility depends upon the strength of the field

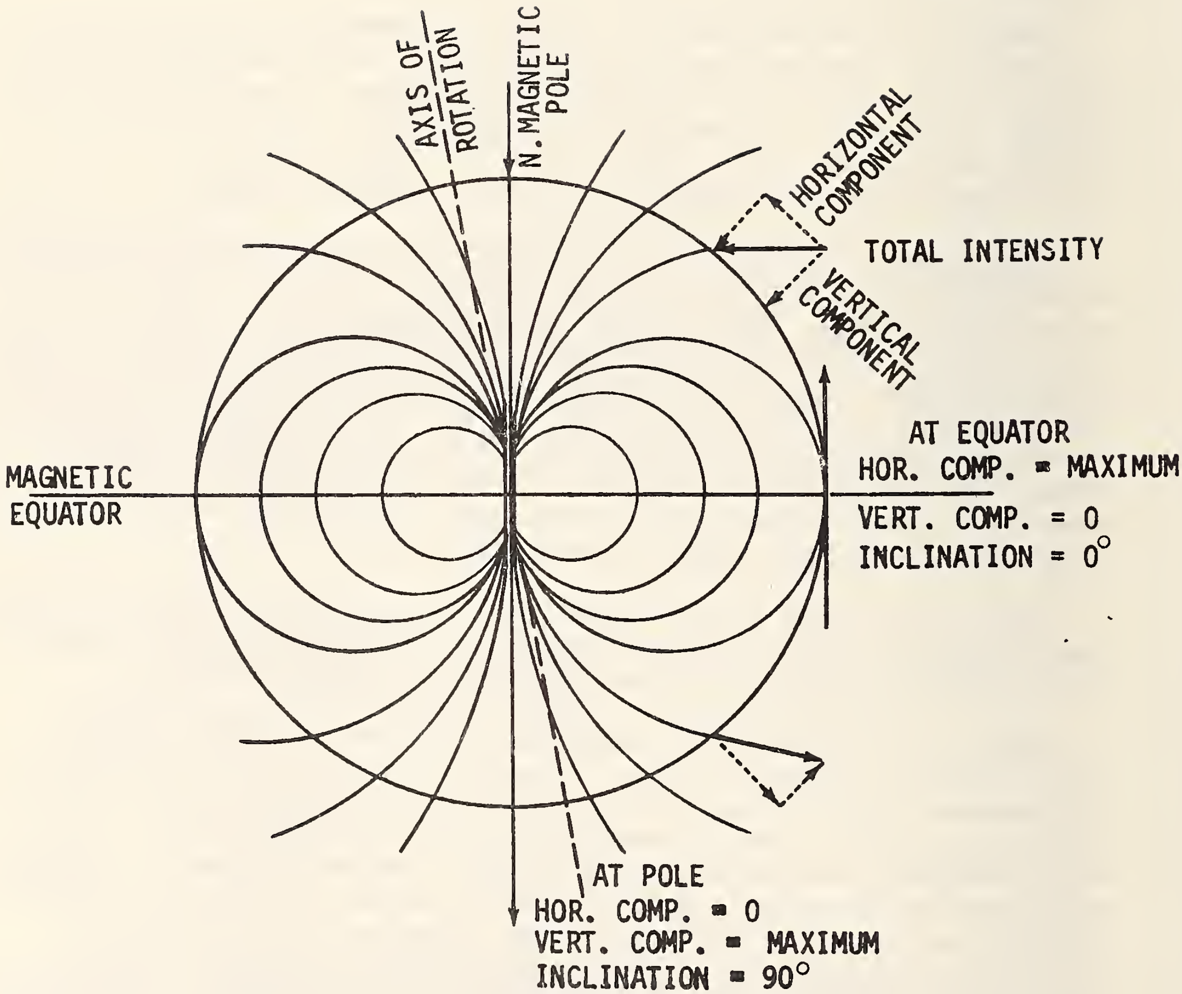

Figure C-17. Earth's magnetic field. (Schematic.)

in which the measurement is made. A few elements, such as iron, nickel, and cobalt, as well as minerals and rocks containing them, produce magnetic fields in the absence of an external field and are called ferromagnetic. The principal cause for local differences in magnetic intensity is the variation of magnetite content in subsurface alluvium and rock. Magnetic susceptibilities of the major rock types in electromagnetic c.g.s. (centimeter-gram-second) units are given in Table C-5.

Table C-5. Laboratory measured magnetic susceptibilities of surface samples and cores. (After Dobrin)

	Magnetic susceptibility (c.g.s. units) x 10^6	
Rock Type	Average	Range
Dolomite	8	0 to 75
Limestone	25	2 to 300
Sandstone	30	0 to 1,700
Shale	50	5 to 1,500
Metamorphic	350	0 to 5,800
Acid Igneous	650	3 to 6,500
Basic Igneous	2,600	40 to 9,700

Commonly, magnetic susceptibilities of various rock units outcropping in an area to be surveyed are determined in advance to aid in the planning of the survey. Different rock units must have contrasting susceptibilities if the magnetic survey is to successfully distinguish between them. Information on susceptibilities is very useful for selecting the proper magnetometer for a particular job. When susceptibility differences are great a magnetometer with low sensitivity will suffice, but if susceptibilities are close a more sensitive magnetometer is required.

Instruments for measuring susceptibility of in situ rocks and others which test the values of core samples are available for $1,000 to $1,500.

In addition to magnetite content, another factor responsible for magnetization of rocks is geologic history including metamorphism, tectonic movements, and lightning. Two types of magnetization are recognized:

- Remanent or permanent magnetization - due to magnetite content and geologic history
- Induced magnetization - caused by the earth's present magnetic field

The interpretation of magnetic survey data is usually done under the assumption that the magnetization indicated is induced by the earth's present magnetic field. The degree of magnetization, or polarization, of rocks for induced magnetization is the product of their susceptibility and the earth's field.

While the component of remanent magnetization may actually be quite significant in certain rocks, its direction is not consistent enough to cause many problems in interpretation.

MAGNETIC INSTRUMENTS

Several types of instruments have been developed for measuring magnetic fields. The most common types measure the intensity of the vertical component of the vector; others measure the total intensity of the field. Selection between these two principal methods is mainly a matter of personal preference. Vertical measurements are most common in ground surveys and total intensity measurements are generally obtained in airborne work.

Following are brief descriptions of four of the more well-known types of magnetic field measuring instruments for airborne and ground use.

One of the first magnetic surveying instruments was the dip needle, a simple compass needle free to move in a vertical plane. Anomalies of 1,000 gammas or more could be detected by these insensitive instruments which were used in ground mineral exploration surveys.

A greatly improved instrument, the magnetic field balance, often referred to as the Schmidt-type magnetometer, then became the most important type for reconnaissance and precision ground surveys, and is still used to a small extent. Instruments of this type can be used to measure changes in the magnetic field with an accuracy of 5 to 10 gammas. Most common are those which measure the vertical field although horizontal types have also been used.

Vertical magnetic balances utilize a horizontal bar magnet mounted on a quartz knife edge. Changes in the magnetic field cause the magnet to rotate an amount proportional to the intensity of the field. No external power is required for its operation. Disadvantages of using this instrument are that it must be mechanically delicate for high precision and it must be motionless to take readings. Models for surface use with advertised accuracies of ±3 gammas are available for $3,000 to $5,000. Less sensitive instruments are available at lower prices.

The first instrument built for mobile use was the fluxgate magnetometer which is used in both airborne and ground surveys. In this type, usually two strips of highly magnetically permeable metal are wound with coils carrying alternating current. These coils are within a magnetizing and measuring coil carrying direct current. Distortion of the waveform of the alternating current is proportional to changes in the magnetic field. Thus the field is measured electronically. Fluxgate magnetometers have similar sensitivities to good magnetic balances and are available in the same price ranges.

Proton precession magnetometers are a popular, recently developed type suitable for both airborne and ground use. They are based upon a complex gyromagnetic effect. A bottle of hydrogen-rich material (usually water) is encased by a coil of wire through which direct current is passed. The magnetic field produced attempts to align all the hydrogen atoms in the same direction, but since they act as individual gyroscopes they do not become aligned and instead rotate at a frequency proportional to the particular external field strength. Direct current is pulsed through the wire and the frequency is measured between pulses. Some of these instruments have sensitivities to ± 1 gamma. Equipment of this type for ground use is priced in the range \$2,500 to \$5,000.

FIELD MEASUREMENTS

Airborne magnetic surveying in mineral and petroleum exploration accounts for a much greater volume of expenditure than does ground surveying. Airborne measurements are commonly carried out with fixed wing aircraft or helicopters at elevations of 1,000 feet above the surface with line spacings of 1/4 to 1 mile. Normal airborne coverage is about 500 line-miles per day.

The major advantage of airborne magnetic surveying is its relatively low cost per unit area considering reasonably large areas. However, tunnel sites would be classed as very small areas so unit-area cost for airborne surveys would be much higher than usual due to mobilization costs and minimum fees. One factor which reduces costs for airborne methods is the simultaneous recording of several types of geophysical data when applicable. Magnetic, electromagnetic, and radioactivity surveys may be conducted concurrently from a single aircraft. A disadvantage is the reduction of accuracy. While accuracies of 1 to 2 gammas may be obtained on the ground, airborne readings are seldom better than ± 50 gammas.[78]

[78] Parasnis, D. S. Mining Geophysics. Elsevier Publishing Co., New York. 1966.

DETERMINATION OF MAJOR STRUCTURAL FEATURES

When rocks having substantially different magnetic susceptibilities are brought into contact by major fault offsets, these structures may be easily detected by a magnetic survey even though concealed by overburden.

Fold axis, particularly in metamorphic rocks, may be determined from magnetic surveys when layers of differing magnetic susceptibility are present. Magnetic susceptibilities of most sedimentary rocks are quite low so magnetics would not be expected to yield such information on folds within them.

LOCATION OF DIFFERENT ROCK TYPES

Much can be learned about the relative positions of differing rock types in normal contact with each other, such as sedimentary-igneous, sedimentary-metamorphic, igneous-metamorphic and sometimes igneous-igneous and metamorphic-metamorphic contacts, even when obscured by overburden (see Figure C-18).

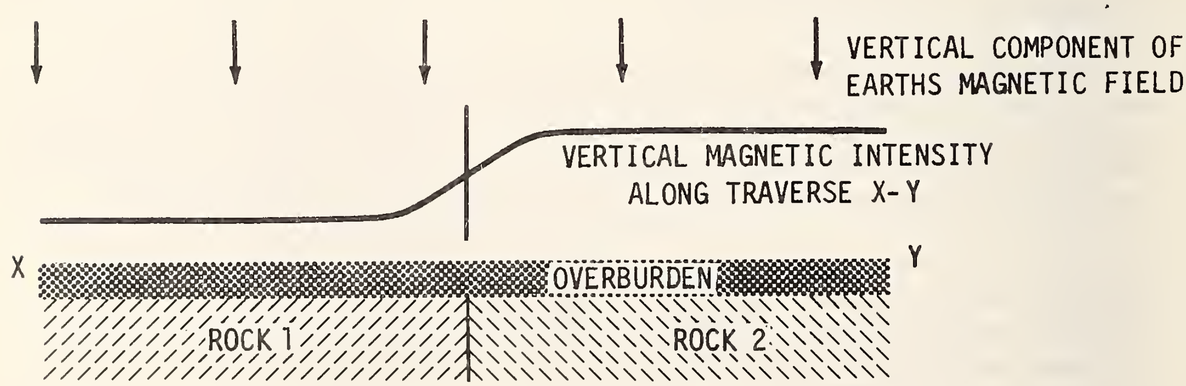

Figure C-18. Variation in vertical intensity of earth's magnetic field across contact between two rock types.

In areas covered by sedimentary rocks, magnetic surveys reveal considerable information about the configuration and depth to magnetic basement crystalline rocks.

DETERMINATION OF DEPTH OF WEATHERING AND DISTRIBUTION OF ALTERATION

Occasionally the depth of weathering and location of zones of hypothermal alteration may be determined in crystalline rocks where magnetic minerals have been altered and replaced by hydrothermal solutions and weathering processes.

INTERPRETATION

The planning of magnetic surveys is best done by qualified geophysicists in cooperation with geologists having intimate knowledge of structural trends or lineations in the area to be surveyed. Where such trends are present it is best to orient magnetic survey lines normal to them to obtain maximum information.

Broad interpretation of the results of magnetic surveys may be quite simple in some cases, but generally complexities and subtleties can only be deciphered by experienced geophysicists, often in cooperation with geologists. Elaborate computer systems utilizing digitized airborne magnetic data are in use, falling of course within the fields of specialists.

In limited areas of the country, aeromagnetic quadrangle maps produced for or by government agencies already exist and would be worthy of study if a tunnel is planned within any of these areas.

Ground magnetic surveying with a two- or three-man crew costs about $150 per profile mile. The mileage covered per day is variable depending upon the precision required, spacing of reading points, type of magnetometer used, weather, and terrain. Using Schmidt type magnetometers which must be read while stationary, several readings per hour may be taken. Using continuous reading instruments such as fluxgate magnetometers, ground may be surveyed as rapidly as one walks. Proton magnetometers allow ground readings to be taken rapidly, usually requiring only 3 to 10 seconds per stationary reading. They may be vehicle-mounted for mobile surveying using recording systems.

ELECTROMAGNETIC METHODS

Electromagnetic methods have been most highly developed by mineral exploration groups searching for massive sulphide deposits in the Canadian shield. These methods are also somewhat useful for geological bedrock mapping in covered areas and may indicate some geologic structures. Electromagnetics have been applied to petroleum exploration on a limited scale, but very little or no use of the methods has been reported or is indicated in the field of engineering site investigations.

Electromagnetic methods are not recommended for tunnel site exploration because of their very limited value in this regard. Other geophysical methods can usually be employed to obtain more useful data. Although seldom used in engineering work these methods do have the capabilities of providing some information in the following areas of related interest:

1. Geologic mapping.
2. Determination of major geologic structural features.
3. Aquifer location.
4. Location of buried utilities and pipelines.

PRINCIPLES

In electromagnetic methods, a magnetic field is generated by putting an alternating current through a loop of wire or through a long straight wire grounded at both ends. This generated magnetic field causes induced or eddy currents to flow in closed loops within conductive earth materials. The eddy currents created generate their own magnetic fields so the total magnetic field generated at a given point consists of the primary or normal field due to the applied current and a secondary or anomalous field due to the eddy currents produced in conducting soil or rock. The resultant magnetic field is commonly measured with an induction magnetometer using a coil of wire as a receiver.

METHODS

Reconnaissance electromagnetic exploration is usually conducted by airborne or "semiairborne" methods and detailed follow-up work is performed with portable equipment on the ground. Electromagnetic borehole logging is sometimes used to search for nearby conductive bodies not penetrated by the hole.

A wide variety of techniques and equipment are available but discussions of these are kept brief in this report because of their minimal application to tunnel site exploration. Electromagnetic methods may be broadly classified into two main types:

1. Fixed source - moving receiver methods.
2. Moving source and receiver methods.

FIXED SOURCE - MOVING RECEIVER METHODS

These methods use a transmitting setup in a fixed location and a receiving loop which is moved around to explore the nearby area. Some of the more important methods in use are:

Turam Method--The primary alternating field is established by a current passing through either a long straight grounded cable or a cable loop laid out on the ground parallel to the long dimension of a suspected conductive body. Measurements are made along observation lines perpendicular to the cable layout with two staffs or coils connected to a compensator-amplifier unit and maintained at a fixed distance apart (usually 50 to 100 feet). The phase difference, field strength ratio, and direction and inclination of the electromagnetic field are measured and indicated directly.

The related Turair semiairborne method uses a large transmitting loop (for example 2 miles x 2 miles) on the ground as the primary source. The changes of phase and amplitude are measured along parallel profiles across this source using two coplanar horizontal (vertical component) or two coaxial coils (horizontal component) mounted in a "bird" towed by a helicopter perpendicular to the formational strike if such is known. An area of 30 square miles or 250 line miles can be covered from one transmitting loop during one day of operation.

VLF Method--The receiving instrument measures the vertical in-phase component (tangent of the tilt angle of the polarization ellipsoid) and the vertical out-of-phase (quadrature) component (short axis of polarization ellipsoid compared to long axis).

Dip Angle Method--The receiving loop with inclinometer is used to measure the angle between the plane of the loop where minimum signal is heard and the horizontal plane, this angle being the tilt of the major axis of the ellipse of polarization.

MOVING SOURCE AND RECEIVER METHODS

Both the transmitter and the receiver are moved about in these methods, usually with a fixed separation maintained between the two. The line between source and receiver may be oriented either parallel or perpendicular to the direction of movement. Most airborne and some surface electromagnetic methods are included in this category. Three commonly used transmitting and receiving coils are:[79]

1. Coils horizontal and coplanar.
2. Coils vertical and coplanar.
3. Coils vertical and coaxial.

Parameters commonly measured include:

1. Out-of-phase components (also known as quadrature or imaginary components).
2. In-phase (real) components.
3. Amplitude.
4. Horizontal intensity.
5. Dip angles.
6. Transient response.

Magnetic and/or radiometric readings are often taken concurrently with airborne electromagnetic surveys.

SITE INVESTIGATION APPLICATIONS

GEOLOGIC MAPPING

Electromagnetic surveying can be used as an aid to bedrock geologic mapping in alluvium covered areas, especially where materials of contrasting conductive nature are present.

DETERMINATION OF MAJOR GEOLOGIC STRUCTURAL FEATURES

Folds, shear zones, faults and altered zones may be detected

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[79] Paranis, D. S. Mining Geophysics. Elsevier Publishing Co., New York 1966.

when these features are either more conductive or less conductive than the surrounding material.

AQUIFER LOCATION

Due to differences in conductivity characteristics of aquifer and nonaquifer material as well as differences between fresh water aquifers and those saturated with brackish water, the various electromagnetic methods can be useful in identifying some of these conditions in conjunction with other observations.

LOCATION OF BURIED UTILITIES AND PIPELINE

Buried metallic lines and other structures are detectable by electromagnetic methods.

EQUIPMENT

All methods use some type of transmitting and receiving systems.

Turam equipment includes a transmitting system comprised of a portable motor-generator set and a primary cable layout and a receiving system comprised of a compensator amplifier, two receiver staffs or coils, a connecting cable, a staff clinometer, and stethoscope headphones. A crew of two or three is used.

VLF methods take advantage of the uniform horizontal fields generated by powerful very low frequency military radio transmitter located around the world. Both ground and airborne systems are in use. The only equipment required for ground surveys is a lighter than 5-pound receiving instrument which is powered by small batteries and operated by one man.

Dip angle methods use a source loop placed in a vertical plane and a receiving instrument consisting of a single receiving loop equipped with an amplifier, headphones, and an inclinometer.

Moving source and receiver methods use a variety of transmitting and receiving coils and connecting cables.

Generally the depth of electromagnetic survey penetration does not exceed half of the distance between source and receiver. Fixed source systems allow prospecting to depths of at least 1,000 feet. Moving source systems are usually limited to penetrations of 100 to 150 feet.

Electromagnetic field data are interpreted by comparing field curves with standard curves developed from measurements made on small scale models.

Ground electromagnetic crews can be expected to traverse 3 to 6 miles per day with moving source equipment, taking readings at intervals of 75 to 100 feet when traverse lines have been surveyed and cleared in advance. Using fixed source equipment, the surveying rate is about the same after the source cable has been laid. Source deployment is reported to require from 1/10 of the total field time using a vertical loop source up to 1/2 the total field time using a long wire source.

Airborne electromagnetic surveys conducted with fixed wing aircraft or helicopter are commonly carried out at speeds of 60 to 120 miles per hour and 100 to 1,000 line miles of surveying per day can be accomplished.[80]

A 1962 cost comparison[88] showed that ground electromagnetic surveys in Canada cost $70 to $100 per line mile with line cuttings in wooded areas being an additional $50 to $75 expense per line mile. Airborne electromagnetic surveys, usually flown together with a magnetometer, cost $15 to $50 per line mile. The higher costs result from the use of a helicopter.

GRAVITY METHODS

The gravity geophysical method detects and measures lateral changes in the earth's gravitational pull that are associated with near-surface variations in rock density.

Gravity measuring has been perfected in petroleum exploration and used to a limited extent in mineral exploration, but it is seldom used in civil engineering applications such as tunnel site investigations. Under certain geologic conditions gravity surveying may be a useful reconnaissance tool for preliminary tunnel positioning studies. Measurements are usually made on surface. Much less common has been the

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[80] Pemberton, Roger H. and H. O. Seigal. "Canadian Geophysical Technology, Applications and Limitations Overseas". Mining in Canada (September 1968). Reprint.

use of gravity meters in shipborne, airborne, and borehole surveys.

Although seldom used because other geophysical methods generally supply more useful information, the gravity method could be applied to tunnel site investigations for:

1. Determination of major geologic structural features.
2. Detection of intrusive features.
3. Determination of depth to bedrock.
4. Determination of alluvial aquifer porosity.

PRINCIPLES

Gravitational surveying utilizes Newton's law of gravitational attraction which states that the force of attraction between two masses is proportional to their product and inversely proportional to the square of the distance between them (when dimensions of the masses are very small compared to the distance between the centers of the masses). Expressed mathematically,

$$F = \frac{\delta M_1 M_2}{d^2}$$

where F is the force of attraction,

δ is the universal gravitational constant determined by laboratory methods,

M_1 and M_2 are two masses, and

d is the distance between the mass centers.

In the c.g.s. (centimeter-gram-second) system, δ is 6.670×10^{-8}. This is the force in dynes that is exerted between two masses of one gram each having centers one centimeter apart.

An object weight is due to the gravitational attraction of the earth directed approximately toward the center of the earth. Gravity may be expressed as a force per unit mass (in units of dynes per gram) or in terms of the equivalent acceleration (with units of centimeters per second per second). The approximate average value of gravity at the earth's surface is 980 dynes per gram or 980 centimeters per second per second, which is commonly referred to as 980 gals. In field work the "milligal" (0.001 gal) is the commonly used unit of measurement. Sometimes the "gravity unit" (0.0001 gal) is used.

The force of gravity is a vector quantity having both magnitude and direction, and which may be resolved into vertical and horizontal components. The horizontal component is very small compared to the vertical (Figure C-19).

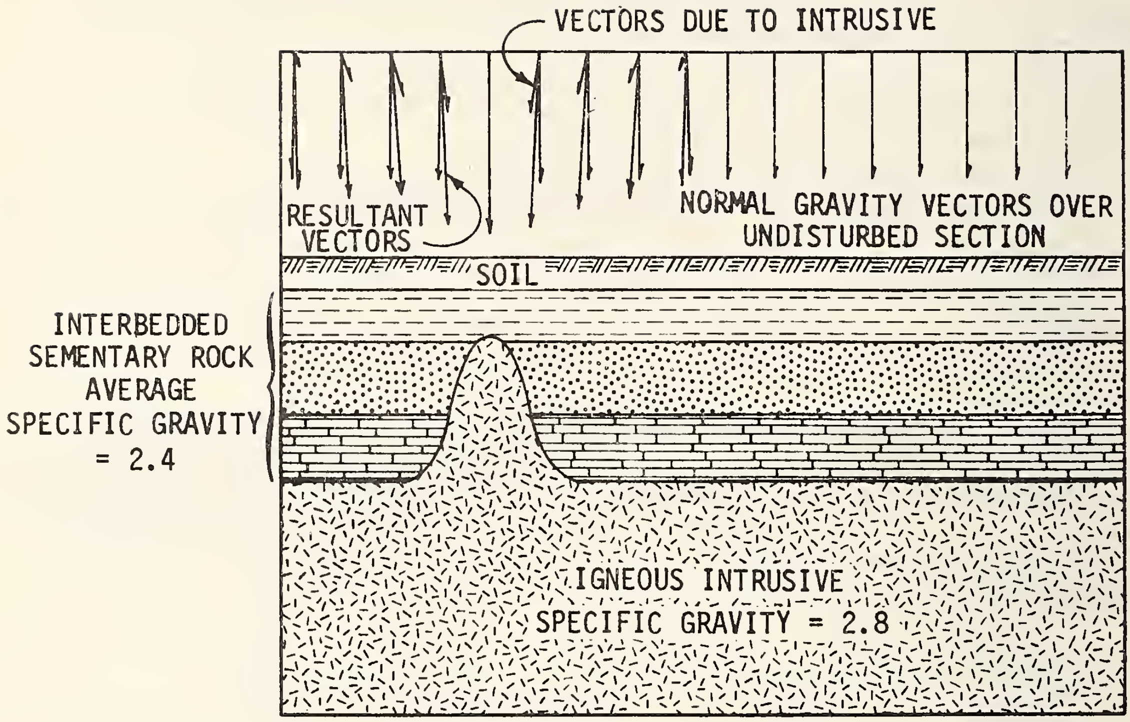

Figure C-19. Gravity vectors over normal and disturbed areas.

Gravity surveys determine areas having anomalously high or low gravitational attraction which depend on density contrasts between geologic units in a horizontal direction. Important anomalous differences in gravity surveys are often only 1 to 10 milligals. A difference of 50 milligals is considered major.

The more geological knowledge that is available on a surveyed area, the more competently the gravity data can be interpreted. Since density differences in subsurface material give rise to irregular gravity patterns, knowledge of the densities for the rock types present is vital to an understanding of these patterns. Table C-6 gives the densities of some common rocks.

Table C-6. Densities of some common rock types.

Rock Type	Density (gm/cm^3) Average	Range
Sandstone	2.3	1.6 to 2.8
Shale	2.4	1.8 to 2.5
Limestone	2.5	1.9 to 2.9
Acid Igneous	2.6	2.3 to 3.1
Dolomite	2.7	2.4 to 2.9
Metamorphic	2.8	2.4 to 3.1
Basic Igneous	2.8	2.1 to 3.2

Density data may be acquired by a variety of methods, but complete information is seldom available. The following sources have been described by Nettleton:[81]

1. Cores - good density measurements may be obtained on consolidated rocks.

2. Drill cuttings - values obtained tend to be too high as the larger usable chips usually come from the harder sections penetrated.

3. Gamma-gamma density logs - run in drill holes, give good quality data for consolidated sediments but less reliable data for unconsolidated sediments.

4. Seismic velocity logs - as there is an approximate relation between seismic wave speed and density, approximate density values may be obtained.

5. Borehole gravity meter - very good density information may be obtained but since instruments are difficult to make and operate, very little of this work has been done.

6. Surface sampling by gravity meter - a profile of close-spaced gravity readings is taken across a topographic undulation, then the density of surface material is determined by trial and error computations.

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[81] Nettleton, L. L. "Elementary Gravity and Magnetics for Geologists and Seismologists". Society of Exploration Geophysicists, Monograph Series, No. 1. 1971.

EQUIPMENT

Numerous laboratory and field instruments have been developed for measuring the force of gravity. The principal types of instruments are:

1. Swinging pendulum.
2. Torsion balance.
3. Gravity meter (gravimeter).

SWINGING PENDULUM

This instrument is generally used for laboratory type determinations of absolute gravity (to within 1 milligal) or lateral variations in gravity (to within 0.1 milligal). Gravitational acceleration is calculated from measurements involving the time of oscillation and length of the pendulum. This process is quite time consuming, usually requiring 30 minutes or more per determination.

TORSION BALANCE

Torsion balances utilize various arrangements of weights attached to a bar which is suspended by a torsion wire and free to rotate in a horizontal plane around the wire. These instruments, capable of only a few readings per day, measure gravity gradient (rate of change of the vertical component of gravity with horizontal distance) and curvature.

GRAVITY METER

Gravity meters measure small variations in the vertical component of the gravity vector and the numerous models available are the most widely used field instruments now in service. Generally 50 or more readings per day may be completed with light weight (25 to 45 pounds), portable instruments. While the many gravimeters employ different refinements, all models are essentially precision weighing devices in which the force of gravity acting on a mass is offset by the tension in a spring.

Direct-reading instruments record the final resting position of the mass and null-type instruments utilize a restoring force to return the mass to a given scale position. LeRoy and Crain[82] illustrate the extreme sensitivity required in gravimetric measurements with the example that if a scale were built to accommodate 3,000 tons of coal (corresponding to the force of gravity of the earth), the addition or subtraction of one ounce of coal (corresponding to about 0.01 milligal) would be measurable.

FIELD MEASUREMENTS AND DATA REDUCTIONS

Spacing and alignment of gravity measurement points varies according to the problem or application. Reconnaissance structural surveys in oil exploration sometimes use a 1-mile regular-grid spacing. Detailed surveys over prospective areas in mineral exploration have utilized spacings of 100 feet or less.

The elevations and geographical positions of all gravity measurement points must be accurately determined. Surveying to establish this control usually comprises the major item of cost in a gravity project. Elevation precision to the nearest 0.1 foot is usually desirable. Reconnaissance gravity work can be expected to cost $300 to $1,000 per profile mile for surveying and gravity readings depending upon station spacing and terrain. One geophysicist with a gravimeter would probably require two or three survey crews (each made up of three or four men) continuously surveying points, and a single survey crew preparing points in advance.

To begin a survey, a gravimeter reading is taken first at a base station and then additional stations are occupied. After one-half to two hours, depending upon the precision required, the base station is reoccupied. The difference between the two readings taken there, known as instrument drift, is proportioned out as corrections to the readings taken at the other stations. Several other corrections must also be applied to the gravity readings:

1. Latitude corrections - made to remove the effect of gravity increase toward the earth's poles due to the fact that the earth's equatorial radius is greater than its polar radius.

2. Elevation corrections - necessary because the closer a station is to the center of the earth, the greater the force of gravity. This correction is made in two steps using sea level or some other datum as a reference elevation:
   a. Free air correction - 0.09406 milligals per foot is added

[82]LeRoy, L. W. and Harry M. Crain, editors <u>Subsurface Geologic Methods.</u> Colorado School of Mines, Golden, Colo. 1949.

to station readings taken above the datum and subtracted from readings taken below the datum. This reduces readings to a common datum.

b. Bouguer correction - (0.01276)x(density of local rock) milligals per foot is applied with the opposite sign of the free air correction. This accounts for the mass of material between the datum plane and the observation plane.

3. Terrain correction - made in areas having substantial local topographic relief, but is not necessary in flat areas.

After all corrections have been applied to the measured data a contoured Bouguer anomaly map is usually drawn. Various methods of anomaly separation are commonly applied to Bouguer data to separate the smooth, deep-seated regional gravity effects from the shallow residual effects due to the crustal geologic units of interest, resulting in a contoured residual anomaly map.

The residual anomaly map is interpreted with the help of knowledge of the local geology. Various complex mathematical manipulations, often using digital computers, are applied in interpreting the survey results. Unfortunately, gravity anomalies are ambiguous. A variety of subsurface conditions can result in an anomaly of the same dimensions and magnitude (see Figure C-20).

SITE INVESTIGATION APPLICATIONS

DETERMINATION OF MAJOR GEOLOGIC STRUCTURAL FEATURES

Major geologic structures such as folds and faults are detectable through gravity surveying when rocks of substantially differing densities are displaced (see Figure C-21).

DETECTION OF INTRUSIVE FEATURES

The presence and configuration of intrusive rock masses are often detectable by gravity surveying. Two examples are gravity lows associated with salt domes which penetrate higher density sedimentary rocks and gravity highs where irregular-shaped high-density igneous rock has intruded into sedimentary rocks.

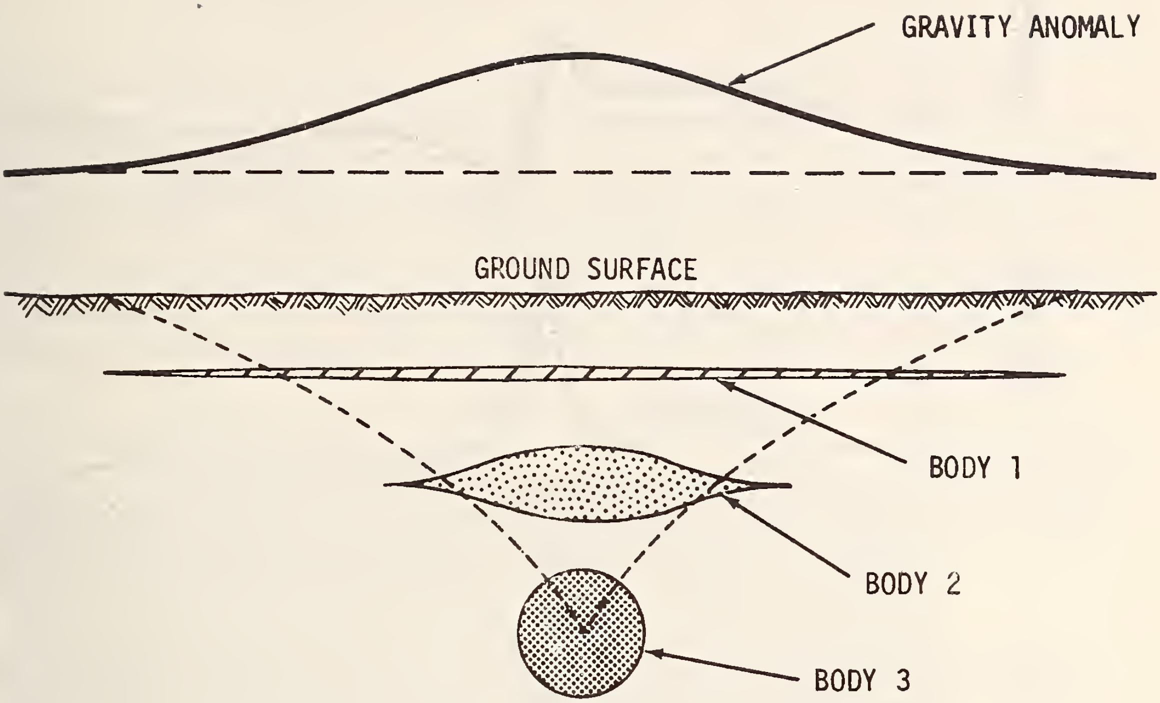

Figure C-20. Cone of anomaly sources. Shallower and broader bodies can result in the same anomaly as the deeper sphere.

Interpretation of gravity surveys of areas having an overburden of unconsolidated sediments resting on uniform density crystalline bedrock may provide depth-to-bedrock data. Results can be obtained within a 10 to 15 percent accuracy, similar to those produced by electrical resistivity surveys, but the latter is most often used as it is applicable to a wider range of geologic conditions. Sumner and Burnett[83] describe the use of a gravity survey to determine bedrock lithology at a building site in California.

[83] Sumner, John R. and John A. Burnett. "Use of Precision Gravity Survey to Determine Bedrock." Journal of the Geotechnical Engineering Division. Proceedings of the ASCE, Vol. 100 No. GT1 (January, 1974). pp 53-60.

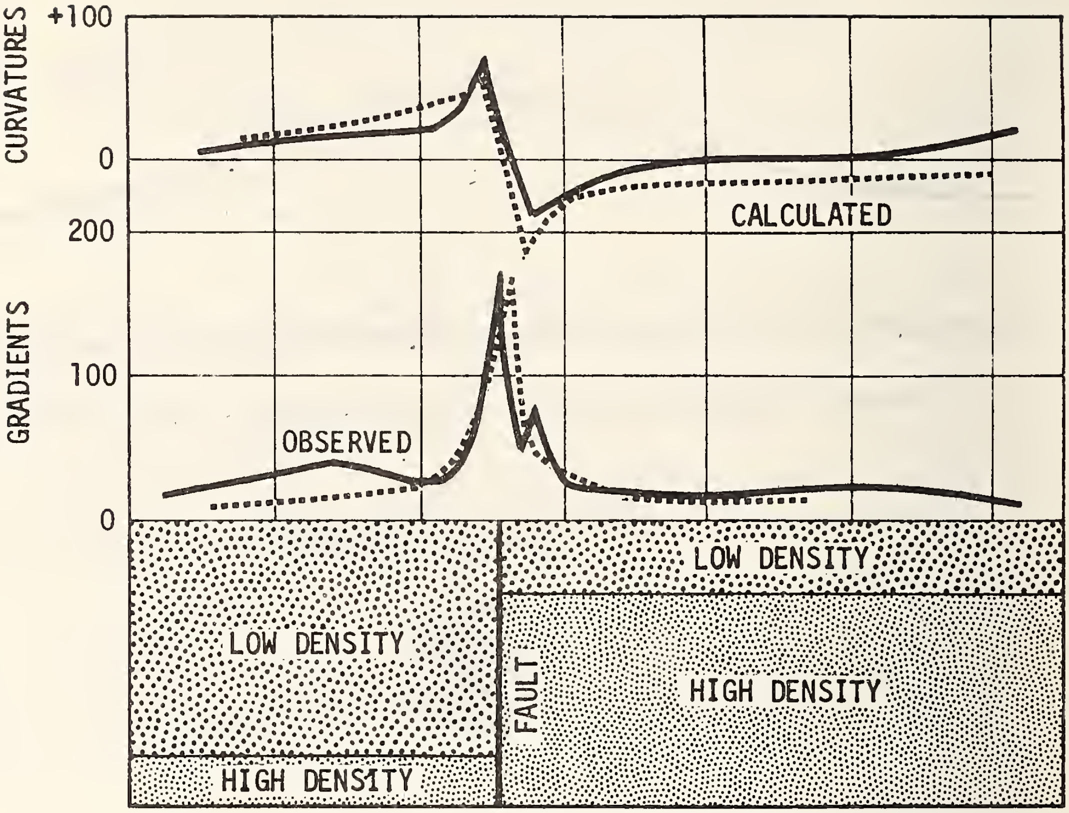

Figure C-21. Location of fault by gravity mapping.

DETERMINATION OF ALLUVIAL AQUIFER POROSITY

When depth to bedrock is known with certainty, detailed gravity surveying in conjunction with laboratory grain density measurements may be used to compute the average porosity of an unconsolidated near-surface aquifer. Eaton, Mastin, and Murphy[84] describe the use of this method at a California location.

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[84] Eaton, Gordon P., Neill W. Martin, and Michael A. Murphy. "Application of Gravity Measurements to Some Problems in Engineering Geology". Engineering Geology, Vol. 1 No. 2 (July, 1964). pp 6-21.

RADIOMETRIC METHODS

Radioactive investigation methods involve the measurement of natural radiation emitted by spontaneously disintegrating or decaying elements occurring in surface and near surface alluvium and rocks. The principal application of radioactivity surveys is in prospecting for uranium deposits, and less commonly, for petroleum, thorium, or potassium-rich mineral deposits. These methods are rarely used in civil engineering or engineering geological studies. However, if radiometric data is available it can be useful in conjunction with other geophysical information for geological mapping in certain geologic environs. Radioactivity is measured by airborne, surface, and borehole methods.

The principal values of radiometric data in the evaluation of proposed tunnel routes is its use as an aid to surface geological mapping. Different rock types having differing radioactivity can be distinguished. It is unlikely that radiometric methods will be used to any appreciable extent in engineering geological site evaluations as other methods provide more information.

Large areas of the United States, especially in the west, have been surveyed radiometrically in the search for uranium and petroleum. However, most of this data is of a company-confidential nature and would be very difficult to obtain.

PRINCIPLES

Naturally occurring elements with an atomic number greater than 83, and a few with atomic numbers below 83, are naturally radioactive. Uranium, thorium, one isotope of potassium (potassium 40) and certain of their decay products are the principal sources of radiation emitted by soil and rock.

The most important types of natural radioactivity emitted during the decay process are:

1. Alpha radiation (positive helium nuclei),
2. Beta radiation (negative electrons), and
3. Gamma radiation (electromagnetic energy with no mass or electrical charge).

Field radiometric instruments measure gamma radiation which has a greater ability to penetrate rock, water and air than do alpha and beta radiation, both of which have mass and electric charges. Gamma rays are electromagnetic radiation of the same nature and velocity as light and X-rays, but having much greater energy and higher frequencies.

On decaying, a radioactive atom, termed the parent, disintegrates to form a product, called the daughter, emitting one or more gamma rays, or sometimes none, in the process. The gamma rays emitted have a characteristic wave length or energy.

Soils and rocks contain varying amounts of radioactive uranium, throium, and potassium minerals. In general, the highest radioactive mineral contents are found in acidic igneous rocks, arkosic sandstones, and black shales. Intrusive and extrusive basic igneous rocks and most sedimentary rocks have low contents. Metamorphic rocks have varying amounts depending on the nature of the parent rocks and subsequent mineralization. Table C-7 lists the common radioactive element content of igneous and sedimentary rocks.

Table C-7. Potassium-40, Thorium, and Uranium content in Igneous and Sedimentary Rocks

	Igneous Rocks		Sedimentary Rocks		
	Basaltic	Granitic	Shales	Sandstones	Carbonates
Potassium					
Range	0.2 - 2.0	2.0 - 6.0	1.6 - 4.2	0.7 - 3.8	0.0 - 2.0
Average	0.8	3.0	2.7	1.1	0.3
Thorium					
Range	0.5 - 10.0	1.0 - 25.0	8.0 - 18.0	0.7 - 2.0	0.1 - 7.0
Average	4.0	12.0	12.0	1.7	1.7
Uranium					
Range	0.2 - 4.0	1.0 - 7.0	1.5 - 5.5	0.2 - 0.6	0.1 - 9.0
Average	1.0	3.0	3.7	0.5	2.2

INSTRUMENTS

The principal portable field radiation detection instruments are:

1. Geiger-muller counter.
2. Scintillometer.
3. Gamma-ray spectrometer.

GEIGER-MULLER COUNTER

This was probably the earliest successful instrument used in surface uranium prospecting. This simple, inexpensive tool measures gamma radiation from all sources without having the ability to distinguish between sources and has a low sensitivity or efficiency of one percent or less, so it is useful for testing samples but not for reconnaissance. These counters usually consist of thin-walled glass or aluminum tubes enclosing a negatively charged metal cathode and a center wire anode, and filled with an inert gas such as argon mixed with approximately 10 percent of a quenching agent such as ethyl alcohol. Gamma rays penetrating the tube strike atoms which are then ionized and discharge pulses at the electrodes that are made observable by the use of a flashing light or audible click. Most later Geiger counters utilized ratemeters with needle deflections showing intensity of radiation in milliroentgens per hour.

SCINTILLOMETER

This improved instrument is based on the phenomenon that a flash of visible light is produced when a gamma ray strikes substances known as phosphors. The most commonly used phosphors are crystals of sodium iodide, often improved by the addition of an activator such as thallium. The flashes of light or scintillations strike the photosensitive cathode of a photomultiplier tube generating an electric pulse which is measured to indicate milliroentgens per hour.

Efficiencies of scintillometers are reported to be 50 to 90 percent, far better than the Geiger counter, so they are very good reconnaissance instruments for airborne or surface work. Scintillometers, however, do not discriminate between sources of radiation.

GAMMA-RAY SPECTROMETER

The more recently developed gamma-ray spectrometers represent a major advance over the scintillometer because they have the ability to distinguish between the three major natural sources of radioactivity - Uranium 238, Thorium, 232, and Potassium 40 - and to determine what percentage of total radiation is contributed by each. Gamma-ray spectrometers also utilize phosphor crystals to pick up radiation, but in addition have pulse height analyzers which allow measurement of radiation amounts from the different sources. This gives a much better idea of the geology of the terrain being traversed.

Two principal types of spectrometers - integral and differential - are in use. The integral or single threshold type discriminates by accepting only gamma rays of an intensity higher than a preset variable threshold level. The differential or window type utilizes two threshold levels and measures only that radiation having energy levels between the two thresholds.

FIELD MEASUREMENTS

At present, the major utilization of field radiation detection instruments is the airborne use of gamma-ray spectrometers in the search for uranium. Radiometric surveys are usually flown at as low an altitude as possible, often at 200 to 500 feet, terrain permitting. Sometimes airborne radiometric surveying is carried out simultaneously with magnetic and electromagnetic surveying, in which case the flight pattern is governed by what appears ideal for the particular method considered most useful of the group. Ground surveys are now commonly used to check the points of interest located by aerial surveys.

ACOUSTIC HOLOGRAPHY

Acoustic holography is a recent development in the long utilized field of acoustic imaging. Experimental work is underway to develop applications for this method of "seeing" inside solid objects and producing useful images of the interior composition. Among the applications being studied are seismic holography, nondestructive testing, medical diagnostics, ultrasonic microcopy, and underwater viewing.

Of particular interest to engineering geologists involved in tunnel site evaluation is the possible application of seismic holography for detecting subsurface voids and highly fractured and faulted areas. Although presently in the experimental stage, promising test results indicate that techniques of this type may be developed into practical exploration tools.

PRINCIPAL

In acoustic holography an object is struck with an acoustic wave and that wave scattered from the object is added to a carrier wave. The interference pattern resulting from the mixture of these two waves is then converted to a photographic transparency on which opaque areas represent low intensity values and transparent areas represent high intensity values. This transparency is termed a two-dimensional spatial transform or an acoustic hologram of the object.

When the acoustic hologram is properly illuminated with laser light an inverse transform is produced. The emerging light waves contain the same information as the original carrier waves. In this manner holograms can be produced to reconstruct three-dimensional visual images of the object studied. Different cross-sectional views of the object may be obtained.

METHODS

Seismic holography is an extension of acoustic holography in which visual images of underground features are produced through the use of seismic waves. Very long wavelength radiation of 10 to 300 meters at low frequencies of 10 to 1,000 Hertz (cycles per second) is input into the earth with vibrators or explosives and multiple arrays of detectors pick up the waves returning to the ground surface.

EQUIPMENT

Equipment required for seismic holography is the same as that used in other acoustic holographic applications except that it is much larger as the seismic energy source must input kilowatts of energy into the earth. The major items required are a transmitting source, a receiving array, electronic signal processing, and a reconstruction display device. Seismic data obtained in the field may be digitized and added to a carrier wave synthetically by computer to produce a hologram of a subsurface area.

SITE INVESTIGATION APPLICATIONS

An interesting U.S. Bureau of Mines seismic holographic experiment[85] carried out to detect an explosively created shattered zone at an average depth of 80 feet in an oil shale formation was moderately successful. However, two problem areas shown to exist by this experiment

[85] Fitzpatrick, G. L., H. R. Nicholls and R. D. Munson. An Experiment in Seismic Holography. U.S. Bureau of Mines, Report of Investigation 7607. 1972.

with its phase-amplitude modulated equipment were:

1. The optical resolution of the system was sufficient theoretically, but the definition of the images was unquestionably poor and the object outline could not be seen.

2. The contrast of phase objects was low compared with more reflective objects.

The experimenters did conclude, however, that:

1. Seismic holography might be useful to determine the size and general shape of underground objects.

2. A lenless Fourier - transfer hologram arrangement seems to be especially suitable for making holograms because the background radiation does not appreciably interfere with the reconstructed images, thus allowing easier scaling.

3. The seismic energy source for making holograms does not need to be a narrow band.

4. Using a synthetic reference wave of arbitrary frequency and position provided indirectly by computer program, without altering the field data in any way, allows generation of an indefinite number of holograms from the same data.

5. A phase-only hologram might be better suited than a phase-amplitude modulated hologram for seismic applications, especially when the holograms are constructed sequentially and an explosive is used as the energy source.

Acoustic holographic techniques show promise for obtaining valuable information, but additional research and development is necessary to make this method a practical tool for use in tunnel site investigations.

Appendix D
Sampling

SURFACE SAMPLING

Surface sampling carried out for tunnel site investigations is of two principal types:

1. Outcrop or float material sampling.
2. Geochemical soil or water sampling.

OUTCROP

Representative samples of all the various rock types exposed in place or indicated by loose pieces of float along the proposed tunnel route are taken in the field for further examination and study in the office and laboratory. When available, these surface rock samples are extremely valuable as they provide strong indications of the type and character of material to be encountered when a tunnel penetrates their downward projections. Even though surface exposures may be weathered to varying degrees, much insight into subsurface conditions is gained from their examination. The cost of obtaining surface samples is negligible compared to the cost of those obtained from boreholes.

The locations of all samples should be noted and recorded. Descriptions should be written and photographs taken of the outcrops at the time of sampling, noting fracture patterns, stratigraphic position of the sample, and any other evidence which may be of help in predicting subsurface conditions. Preparatory to some tests, the orientation of a sample is carefully determined and marked.

Representative surface samples should be studied both by inspection cf hard specimens and by microscopic examination to determine the composition, texture, foliation, porosity, and degrees of hardness, induration, cementation, weathering, and hydrothermal alteration. Various laboratory tests should be performed on intact specimens to determine the important physical and mechanical properties. In some cases chemical analysis may be useful. Tests may also be carried out on joint filling material if it has been preserved at surface.

A sample collection of rock types to be penetrated by the tunnel can serve as a very useful visual aid in discussions when the exploration team is presenting their findings to management or design engineers.

GEOCHEMICAL

Where areas covered by residual soils lack bedrock exposure in outcrops, systematic geochemical sampling and analysis may be used to advantage for determining the types of subsurface rocks present and their approximate areal extent. Samples should be taken from the same horizon in the soil profile to minimize variations common to varying degrees of weathering.

Total composition, concentrations of characteristic elements or minerals, or recognizable composition ratios may, in favorable condition, aid in distinguishing bedrock types.

Analyses of spring water may furnish additional clues to the nature of bedrock in covered areas.

SUBSURFACE SAMPLING

DRILLING EQUIPMENT

The usual practice for obtaining soil or rock samples for laboratory testing is to drill holes to the desired sampling depth. Information on the quality of the subsurface material may be obtained not only from the samples, but also from field observations of the resistance encountered in advancing the hole.

There is no single type of drill rig that is capable of taking every type of sample in every type of subsurface material. Next to economics, the geological conditions exert perhaps the most influence upon the selection of the exploration equipment and the extent of its use.

Bore or drill holes can be vertical, oblique or horizontal. The danger of a hole caving increases with (1) depth and diameter of the hole, (2) the presence of cohesionless or highly fractured ground, and (3) the presence of ground water.

Bore holes may be stabilized by filling them with either a drilling fluid, such as water or drilling mud, or by installing casing. Water stabilizes the hole wall by hydrostatic pressure. Drilling mud is a slurry prepared by mixing commercially available muds with water to give a readily pumpable fluid that exhibits suitable viscosity, good gel characteristics and low filtration qualities. Generally, an increased gel strength and viscosity is necessary in highly permeable soils. The mud stabilizes the hole walls by coating them with a relatively impervious film and by a hydrostatic pressure. In some cases, it is necessary for the unit weight of the mud to approach the in situ weight of the ground material in order to keep the hole open. The most expensive, but safest, method of stabilizing the hole wall is by the use of casing or pipe. Generally, casing is used only when drilling mud would increase the water content of the ground to be sampled or when shallow, wet, and/or caving stratum must be sealed off. Flush-joint or flush-coupled casing is the most satisfactory. The flush internal surface of the casing prevents hanging and jarring of a sampler while the flush external surface allows easier penetration and withdrawal of the casing.

The three basic types of drill rigs used are: (1) the rotary, or core-drill, (2) the churn or cable-tool rig, and (3) the auger rig. Rotary drill rigs can be truck, trailer, skid, and/or post mounted. In the rotary rig the motor is connected to a drill head (or table) which rotates a drill rod having a bit at the end. The bit is then advanced forward by gravity feed and/or pressure from the drill rig drive mechanism as required to obtain a suitable penetration rate. The drilling fluid is usually pumped down through the drill rod string and back up the hole to a recirculating sump or pit. The bit rotation, rate of advance, and rate of circulating fluid are all adjusted to produce cuttings small enough to be removed from the hole at the same rate as the bit penetrates. Air may be used as the drilling fluid in some instances. The rotary drilling method, with a drilling fluid for removal of the cuttings, is recommended whenever possible. A drilling fluid must not be used, however, when wanting undisturbed samples in dry soil.

Churn drills may be truck or trailer mounted and are usually powered by independent motors or engines. The advance with a churn drill is accomplished by alternately raising and dropping a chisel-shaped bit that is attached to a heavy weight or drill bar suspended from a steel cable. Churn drills are advantageous in advancing a borehole through boulder or rubble zones. They are not suitable for advancing boreholes used to obtain undisturbed samples. Casing is usually required with churn drilling and it is normally driven with the drill driving mechanism.

Percussion drills can be classified with churn drills and may be truck, trailer, wagon, or post mounted. They are almost always operated by compressed air. Percussion drills perform well in brittle or friable material, but they are not satisfactory for use in undisturbed soil sampling operations.

An auger drill is advanced by a rotational movement in combination with the application of pressure. Auger drilling does not require a drilling fluid or wash water. Above the water table, auger drilling leaves a perfectly dry and relatively clean hole. The auger should not be forced into the ground so rapidly as to displace the soil downward or outward and cause soil disturbance. Sufficient pressure should be applied, however, to make the auger penetrate as rapidly as it cuts. Under no conditions should an auger be allowed to overfill before it is withdrawn. Excess penetration disturbs the soil at the hole bottom and an overfilled auger acts as a piston while being withdrawn, thus tending to pull in the walls and bottom of the hole.

Auger drilling is developing into the most common method of soils exploration for depths up to about 200 feet. The speed of operation for most soil types is greater than that for other drilling methods. Determining the ground water level is easier with auger drilling than with the other drilling methods. Casing is usually not necessary with auger drilling except in cohesionless soils and sometimes below the water table. The disadvantage of auger drilling is the extreme difficulty or impossibility of drilling through fluid soils, boulders, and hard strata located at considerable depth.

Augers may be either hand operated for the softer soils at shallow depths or power driven for the harder material and deeper depths. There are several types of augers available. A barrel type auger consists of a barrel having curved blades at its bottom end. The barrel may be either solid, hinged, or split. The hinged and split augers are especially adapted for removal of sticky soil, but they are not strong enough for use in hard or stony ground.

Helical augers are probably the most widely used type of auger and may be either short flight or continuous flight. They are sometimes called worm-type augers. Helical augers are not very effective in stiff, cohesive soil, weathered rock, or soil containing coarse gravel and boulders. They will, however, handle occasional cobbles or boulders up to a size that is slightly less than the vertical distance between the blades. The continuous flight augers may have either a hollow or solid stem. In the hollow stem, the end of the auger has a removable plug to prevent soil squeezing up into the hollow stem during the drilling operation. Soil samples can then be taken through the hollow stem upon removal of the plug. The diameter of a continuous flight auger is necessarily limited due to the large torque required for turning a long string of augers inside a borehole. A short flight auger connected to strong, torque resistant drilling rods does not have the same limitation. The advantage of a continuous flight auger is that the soil is automatically brought up to the ground surface while advancing the borehole.

A successful combination of the rotary and percussion principals is seen in rotary buckets or bucket augers. A bucket auger consists of a relatively short barrel or bucket that is open at the top and provided with some variety of teeth arrangement at the bottom. Different varieties of teeth are available for drilling in different types of ground (Figure D-1). Bucket augers that have hinged bottoms, commonly called drop-bottom buckets, are well suited for rapid removal of the material from the bucket. For drilling cohesionless or very wet and soft cohesive ground, the openings in the bucket bottom are covered with rubber or steel flaps to prevent the loss of material. The progress with bucket augers is not as fast as with helical augers, but it is still quite rapid provided large boulders are not encountered.

The simplest method for advancing a borehole is with the displacement sampler. This is done by driving the sampler that has a plug locked at the bottom into the ground. This is a crude method and should not be considered acceptable except for preliminary reconaissance because undisturbed samples cannot be obtained with it. Displacement causes disturbance below the depth of penetration for a distance equal to about three times the sampler diameter.

The technique known as wash boring is another relatively simple method of drilling in soils. Casing is driven into the washable material by repeated blows of a drop hammer. After the casing has been driven to a reasonable depth, the ground is washed out from inside the casing. This is done by the chopping and twisting action of a washbit. The cuttings are removed from the hole by circulating water, which is also

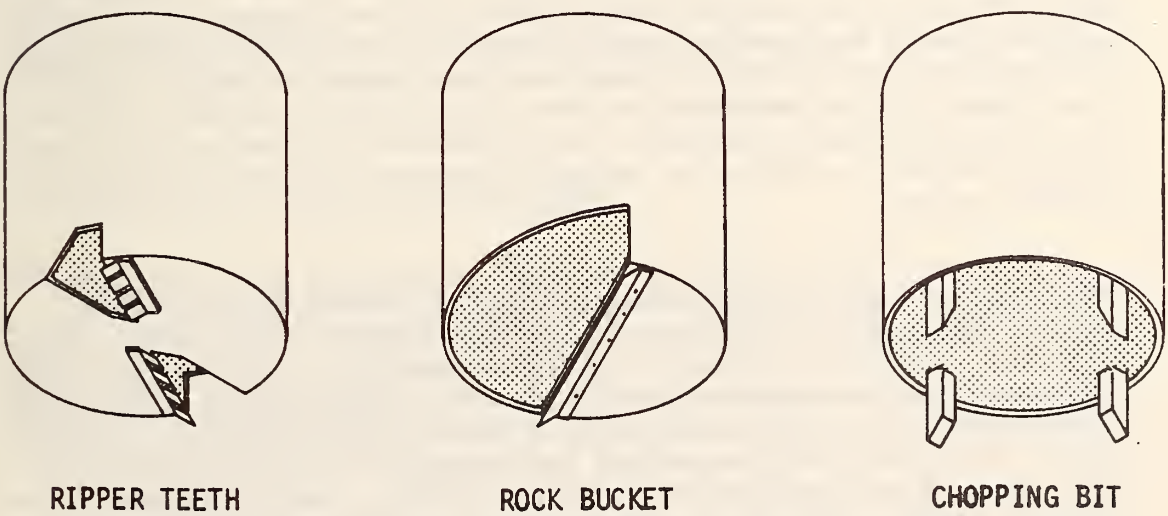

Figure D-1. Typical bucket augers.

used for jetting through the end of the washbit. From the soils engineering point of view, the wash-boring method of drilling has more disadvantages than advantages. The procedure is slow, the chances of sample disturbances are great, and occasionally the information obtained on the changes in soil layers between sampling intervals is unreliable. The method is, however, relatively common, especially in those areas of extensive cohesive soil deposits. The main advantage for wash-boring is the limited amount of equipment needed.

DISTURBED SAMPLING TECHNIQUES

In a disturbed sample, the material must be representative of the ground mass from which it is obtained. A disturbed sample contains all of the constituents of a particular stratum, but with an altered structure. Disturbed soil samples used for laboratory testing must not be contaminated by material from other strata.

The cuttings from wash boring will give some, though incomplete, information about the general character of the soil and the sequence of the soil strata. Such identification has to be regarded with caution, however, because the wash water tends to wash the fines out of the soil and the cuttings may indicate a coarser soil than is actually present. Similarly, large gravel may be broken up into fine chips under the bit and not appear in the cuttings.

Augers may be used for obtaining disturbed samples in all soil types except for cohesionless soils that will not stay in the auger and/or those soils in which the borehole will not remain open. Augers are best suited for sampling above the water table. When determining the depth from which the cuttings come when using continuous flight power augers, it is necessary to take into account the speed with which the cuttings travel up the hole. Soil samples returned to the ground surface during the advance of an auger, however, will be of a questionable value.

The use of drive samplers may also be used for obtaining disturbed soil samples. An open drive sampler may be used in stiff cohesive soils. This sampler consists of a head, a barrel, and a cutting shoe. It may be driven by pushing, jacking, or hammering. Pushing is the preferred method because it results in a continuous motion of the sampler. Jacking produces an intermittent slow motion of the sampler. Hammering is the least desirable method for driving the sampler because of its resulting intermittent fast sampler motion. The impact and vibrations of the hammering action contribute considerably to damage of the sample structure. The hammering procedure for driving the sampler is still much used, however, because the blow count taken during the hammering provides a rough, but easily obtainable and very tangible, characteristic of the soil in place. The limitations of the blow count should be clearly understood, however.

A split-spoon or split-tube sampler is used in the same manner as the open-drive sampler. The split-tube sampler is preferred over the open drive sampler because the two halves can be separated to observe soil stratification. When assembled, the two halves of the split-spoon, or split-tube, sampler are held together by the head at one end and the shoe at the other end. These samplers are satisfactory for use in all types of soil provided the appropriate sample retainers are used.

The Memphis and Porter samplers are retractable-plug displacement samplers. With the plug locked at the bottom of the sampler, the sampler is driven to the desired depth, the plug is retracted, and the sampler is then driven to obtain the sample. These samplers are intended for manual operation and thus are limited for use to a depth of about 35 feet. These samplers have a barrel, a cutting shoe, and extension pipes having the same dimensions as the barrel.

The New Orleans sampler is a type of cable-tool sampler. Cable-tool samplers perform best in cased holes in which drilling fluid is not used. A cable-tool sampler is driven by means of a heavy wire-line hammer attached to the head of the sampler. They may or may not contain a split tube or liner. Although the natural stratification of the soil is preserved, the in situ thickness or density may not be.

Free-piston samplers (as described in the section on undisturbed sampling techniques) may be used to obtain disturbed samples from most soil types if they are driven rather than pushed during the sampling operation.

In rock drilling, the disturbed samples obtained will consist entirely of the cuttings recovered from the drilling operation.

UNDISTURBED SAMPLING TECHNIQUES

Undisturbed samples are taken primarily for laboratory strength tests and in those cases where the properties of the material in place have to be studied. A completely undisturbed soil sample cannot be obtained. Careful application of the recommended techniques will, however, provide samples having a minimal disturbance that are suitable for laboratory "undisturbed sample" tests.

To obtain an undisturbed soil sample, a clean open borehole of sufficient diameter must be drilled to the desired sampling depth. The diameter of the borehole should be as small as practicable. A borehole diameter that is 3/8 to 5/8 of an inch greater than the outside diameter of the sampler or casing, if casing is used, should be sufficient for clearances.

Numerous field observations and careful longtime research has led to the conclusion that samplers for taking undisturbed soil samples should be thin-walled. The sample tube wall should be as thin as practicable, but still strong enough to prevent buckling of the tube during sampling. Conventionally, a thin-wall sampler has a sampling tube wall thickness less than 2.5 percent of its diameter.

It is obvious that the amount of soil displaced by a sampler should be as small as possible in comparison with the volume of the material extracted. In other words, the ratio of the volume of the soil displaced by the sampler to the total volume of the sample extracted should be minimal. This ratio is known as the "area ratio" which is expressed as a percentage and is defined as follows:

$$\text{Area ratio} = 100.0 \left[\left(\frac{D}{d} \right)^2 - 1.0 \right]$$

where D is the outside diameter of the sample tube, and

d is the inside diameter of the sample tube.

The area ratio must not be greater than 13 percent and preferably is less than 10 percent. A small area ratio value obviously means that the damage caused to the sample in the provess of sampling is relatively small.

The ratio of the final sample length to the soil column length from which it has been extracted is called the recovery ratio. This shows the recovery ratio relationship which is:

$$\text{Recovery ratio} = \frac{L}{H}$$

where L is the final sample length, and

H is the soil column length from which the sample is obtained.

The size of the sample required for testing dictates the minimum acceptable sample tube diameter. Generally, a 5-inch ID tube is used to sample cohesive soils and a 3-inch ID tube is used in cohesionless soils. Sample lengths of 24 to 30 inches are usually sufficient. Frequently, however, larger samples are desired for more accurate laboratory results. The ratio of length to diameter for the sampler is very important because friction on the sides of the sampler can cause compression of the sample during driving of the sampler. As the penetration of a sampler increases, the accumulated friction along the perimeter of the sample causes a compressive stress in the soil immediately below the sampler. This stress is found from:

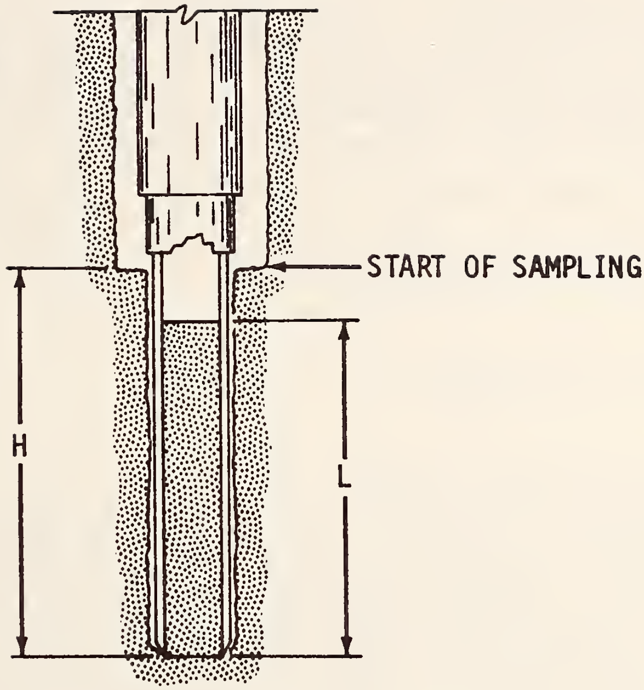

Figure D-2. Recovery ratio.

$$\text{Compressive stress} = \frac{\pi dL}{0.25\pi d^2} = \frac{4L}{d} \text{ times the average frictional stress.}$$

where d is the sample diameter and
L is the sample length.

To minimize the compressive stress, the maximum sample length to diameter ratio should be 10 to 15 for most soil. For wet silts and clays which develop little friction, the sample length to diameter ratio may be as much as 20.

The sample tube should have a smooth, sharp cutting edge free of dents and nicks. The cutting edge should be formed to cut a sample that is 0.5 to 1.5 percent smaller than the inside diameter of the sample tube. The clearance required varies with the character of the soil being sampled, but it should be kept to the minimum required for maximum sample recovery. Sticky cohesive soils require the greatest clearance.

Cold-drawn seamless steel tubing provides the most practical and satisfactory sample tubes. However, welded-and-drawn-over-the-mandrel steel tubing is available with dimensional and roundness tolerances satisfactory for sample tubes. Generally, other tubing with a welded seam is unsatisfactory. Brass and stainless steel tubing are satisfactory when acceptable tolerances can be maintained, but the extra cost can not usually be justified. Steel tubes should be cleaned and coated with laquer or epoxy resin to prevent rust and form a smooth surface to reduce frictional resistance on the tube during sampling.

A variety of samplers are available for use in obtaining undisturbed soil samples. Basically all are variations of either a push-tube type or a rotary core barrel.

PUSH-TUBE TYPE SAMPLERS

Push-tube type samplers should not be driven to obtain undisturbed samples because the vibration resulting from the driving action causes disturbance in the sample. Soils that are too hard to permit smooth penetration of a sampling tube and soils containing gravel that will damage the tube and sample require the use of a core-barrel type sampler to obtain undisturbed samples. The sample should be taken as soon as possible after advancing the hole to minimize swelling and/or plastic deformation of the soil being sampled. A clean open hole with a minimum amount of disturbed material at the bottom is essential for obtaining satisfactory undisturbed soil samples.

A piston sampler is one type of push-tube sampler. There are three kinds of piston samplers: stationary or fixed, retractable, and free. Fixed piston samplers are satisfactorily used to obtain undisturbed samples of very soft to stiff clays, silts, and sands that do not contain appreciable amounts of gravel but with sufficient cohesive material for cohesion both above and below the water table. After the borehole has been advanced and cleaned, the assembled sampler with the piston assembly locked flush with the cutting edge of the sampling tube is lowered to the bottom of the hole. The piston assembly is then released from the sampling tube and held stationary to prevent further movement. During the time of actual sampling, the piston assembly remains at this level. The piston moves with respect to the sampler only during the actual push. It helps pull the sample into the tube and retains it after the push is completed. A vacuum breaker rod is provided to ease removal of the sampler head after the sample has been withdrawn.

In the "Hvorslev-type" of fixed piston sampler the piston assembly is held stationary by a clamp to a piston rod extension which passes through the sampler head and inside the drill rod to surface. A cone lock in the sampler is used to hold the piston to the drill rod after advancing the sampler and prevent downward movement of the piston during withdrawal of the sampler. This sampler is used in both cased and uncased holes.

In the "Hong" fixed piston sampler, which can be used in cased holes only, the piston is held stationary by an outer barrel which surrounds the sample tube and seats on a special casing drive shoe. The need for a separate set of piston extension rods is eliminated. A ratchet system holds the piston in place. In both the "Hvorslev-type" and "Hong" samplers the sample tube is advanced beyond the piston by pushing the drill rod.

Another type of fixed piston sampler is a hydraulically operated piston sampler designed for use in uncased holes that are stabalized with drilling mud. It is suitable for undisturbed sampling in soft to medium clays and silts. In these samplers, the piston is attached to the main head of the sampler and thereby to the bottom of the drill rod string. The sampler tube is advanced by pumping water or drilling mud through the drill rod. This hydraulic pressure is confined on top of the sampler head by an outer tube until it builds up sufficiently to push the sample tube down past the piston. The drill rod string, holding the piston, is held stationary by clamping its upper end to the drill rig and it thus also furnished the reaction for the hydraulic drive pressure. This reaction requires stiff, large diameter drill rods for satisfactory operation in deep holes to prevent rod bending and a consequent upward movement of the piston. One double tube sampler is available in which the drive is actuated by a buildup of hydraulic pressure in the drill rod string and sampler head, that at peak pressure causes a shear pin in the sampler head to fail. The pressure built up between the main sampler head and the sample tube head then forces the sample tube to advance in one fast, continuous push. A port in the lower end of the piston rod allows the pressure to vent when the sample tube head passes the port ending the drive.

A fourth kind of fixed piston sampler is the Swedish foil sampler, developed to obtain long, continuous undisturbed samples in soft cohesive soils. The sample is progressively encased in thin axial strips of steel foil to reduce the friction between the sample and the sampler barrel as the sampler is advanced. The sampler consists of a cutter head attached to the lower end of a barrel. The upper portion of the cutter head is double walled and contains a magazine for holding 16 rolls of the foil strips. The foil strips are pulled from the magazine at the same rate the sampler penetrates the soil. The foil strips form an almost continuous liner that remains stationary with respect to the soil sample. These strips slide against the inside of the sampler barrel, minimizing the friction between the sampler wall and soil sample. The sampler barrels are constructed in 8.2-foot long sections connected by split couplings. Additional sections can be added to extend the length of the sampler.

The effect of penetration rate and/or pushing interruptions are relatively unimportant with the foil sampler. The sampler is supported by adhesion and friction between the sample and foil strips. Although soil friction inside the foil sampler is practically nonexistent, frictional forces between the ground and outside of the sampler can become great enough to prohibit pushing the sampler. Two innovations are available to reduce this exterior friction: (1) a water jet attached just above the cutter edge, and (2) a rotary outer barrel with cutter teeth and the capability of circulating a drilling fluid for use in harder soils. Continuous samples as long as 50 feet have been obtained with one push of the foil sampler.

In samplers having a retractable piston, the piston is withdrawn to the top of the sampler just before the start of actual sampling and then held in position during sampling. An example of such a sampler is the California sampler. After completion of the drive, the piston is raised a slight additional amount. The California sampler takes three to five 12-inch-long consecutive samples, thus covering a continuous sampling range of up to five feet.

The free piston type sampler has a piston that is free to move with the top of the sample during the actual sampling operation. The piston is locked into position at the bottom of the sampler barrel for lowering the sampler to the desired sampling depth. When the sampler reaches the sampling depth, the piston is unlocked so that it then rests on the sample entering the tube. A clamp is used for locking the piston rod to prevent downward movement of the piston during withdrawal of the sampler after obtaining the sample. A free piston type sampler may be used in stiff clays or partially dry silts and clays.

Another kind of push-tube type sampler is the open, or "Shelby" type, sampler. An open sampler consists of seamless steel tubing attached to a sampler head that contains a check valve or vents for the escape of air or water. They may be used in friable, partially dry silts and clays. Open samplers, however, are not recommended for obtaining undisturbed samples from boreholes. Vibration of the sample and/or vacuum created under the sample during withdrawal frequently result in the loss of the sample. The check valve is supposed to prevent hydrostatic pressure from forcing the sample out of the tube during withdrawal, but it is not always reliable or effective.

The most satisfactory method for pushing an undisturbed sampler is with the drive mechanism (preferably hydraulic) on the drill rig. Some other methods for advancing the sampler are pushing by hand, hand operated mechanical or hydraulic jacking, and hydraulic piston sampling. Usually an undisturbed sampler is pushed by hand only in test pit sampling where short diameter, short thin wall samplers are used. Hand operated mechanical or hydraulic jacks that are normally used in field operations produce a slow, erratic penetration rate and cause vibrations in the drill rod string, creating disturbance of the sample. This method should, therefore, be avoided whenever possible and used only in shallow holes in medium dense, sandy soil or fine grained soil with a low sensitivity ratio (ratio of undisturbed to remolded unconfined compressive strengths).

The full advance of the sampler should be made in one smooth, continuous push at a uniform rate. The penetration rate of an undisturbed sampler greatly affects the degree of disturbance of the sample obtained. A fast, continuous penetration is required to prevent buildup of friction between the sampling tube and the soil. Penetration rates of 0.16 to 1.0 feet per second should be used. The penetration rate should be as smooth as possible and the sampler should not be rotated during the drive. If the drive is interrupted for any reason, it should not be restarted. Restarting the drive will result in an increased penetration resistance, sample disturbance, and a decrease in the total recovery ratio. In cohesionless soils below the water table, vacuum caused by the piston can cause piping of the sample if the length of drive is excessive.

After completion of the sampling push in fine grained soils, the withdrawal operation should be delayed for about 10 minutes to allow the adhesion and/or friction between the sample and the sampling tube to increase and assist in holding the sample in the tube. The sampler should be withdrawn slowly and uniformly without rotation and with a minimum amount of shock and vibration. Fast withdrawal tends to create a vacuum below the sampler and causes disturbance and/or loss of the sample. If drilling fluid is present in the borehole additional fluid should be added as the sampler is removed to keep the borehole full at all times.

Ordinary sampling methods in cohesionless soils usually result in loss of the sample when the sampler is withdrawn. An attempt to prevent this loss by a purely mechanical method has been made in the Sprague and Henwood Main type sampler with flaps or flap doors. The sample in this apparatus is still partly disturbed though when passing by the entrance flap. A more efficient way is by the impregnation of the sample with chemicals, asphalt, or cement grout. The chemicals are injected through a pipe welded to the outside of the sampler barrel and leading to a groove in the sampler shoe to form a plug at the bottom of the sample.

The lower portion of cohesionless samples may also be stabilized by freezing. This method is intended primarily for sampling saturated sand and silt and normally requires casing. There is no conclusive evidence as to which sampler type gives the best results in the freezing method, but apparently fixed piston samplers are preferable. The sampler is lowered down the hole clamped to an annular auger that keeps the sampler centrally located in the hole. After the sampler has been forced into the ground the annular auger is withdrawn and replaced by an annular freezing chamber. The main objective of undisturbed sampling in cohesionless soils, such as sand, is somewhat different than that in cohesive soils where samples for compressive and shear tests are needed. In the case of sand, the primary importance is knowing whether the material is dense or loose and its moisture content.

Undisturbed soil samples of cohesive soils should be removed from the sampling tube as soon as possible after the sample is withdrawn from the borehole. A delay in removing cohesive samples from the sampling tube allows adhesion and friction between the sample and tube to build up, resulting in a greater than normal sample disturbance by ejection. The sample should be removed from the sampling tube by pushing the sample out through the top of the tube by means of a sample jack. The sample should be extruded from the tube in one smooth, uniform stroke of the jack to minimize disturbance of the sample during ejection. Hydraulically activated sample jacks are the most satisfactory method for extruding soil samples from the tube. When hydraulic pressure is not readily available, then mechanical sample jacks can be used. Pneumatically activated sample jacks are not satisfactory for extruding undisturbed samples. The initial pressure required to overcome the frictional forces between the sample and the tube is usually considerably greater than that required to extrude the sample, so the expansion of the air resulting from this pressure drop ejects the sample too rapidly which sometimes results in serious sample disturbance.

Undisturbed soil samples should be processed and handled carefully to insure that they remain in an undisturbed condition. Cohesive soil samples should be sealed to prevent loss of moisture and shrinkage. Cohesionless soil samples should be confined in the tubes to preserve their in situ density, but they must be drained to prevent disruption of the structure due to liquification.

ROTARY CORE BARREL SAMPLES

Core barrel samples are generally taken in hard cohesive soils (materials relatively insensitive to sampling disturbance) or rocks. The percentage of core recovery is an indication of soundness and degree of weathering of rock. Core sections having a low recovery should be carefully examined to estimate the condition of the missing sections and also the reasons for the low recovery.

There are both single tube and double tube core barrels used in core boring. In the latter type, the inner tube retains the core and usually does not rotate with the outer tube. The rotating tubes in both types of core barrels have drill bits at the cutting ends. The actual cutting devices may be either permanently attached to the bit, fixed to the bit but exchangeable or refaced when worn, or fed between the bit and the rock as chilled shot. The measures taken to avoid core loss consist of (1) relieving air or water pressure above the sample, for example, by a ball check valve that automatically opens if the pressure exceeds a certain limit and (2) using a core catcher or core spring that permits the core to move upward but prevents it from falling downward out of the core barrel.

Single tube barrels are used in sound rock or in large diameter holes in all kinds or rock. If the material to be sampled is soil or fissured or soft rock then the core should be protected from the erosive action of the drilling water. A double tube barrel is recommended for such cases. The inner tube should be provided with a liner if soil cores are to be taken. The liner containing the soil sample is then removed from the sampler, sealed, and sent to the laboratory. The presence of hard particles such as gravel or rock fragments may seriously disturb or even destroy a soft sample as they are ground against the sample by the drilling operation.

Core barrels having internal or bottom discharge bits set with tungsten carbide teeth are suitable for obtaining undisturbed samples of stiff to hard soils. The bit teeth should be set at 20 to 30 degrees with respect to the radius to cause a slicing action and force the cuttings and drilling fluid away from the core. The inner tube is attached to the head assembly of the core barrel by a ball bearing type swivel which thus allows the inner tube to remain relatively stationary while the outer barrel rotates to cut the core. The cuttings are removed by circulating the drilling fluid between the inner and outer tubes. Standard double tube core barrel sizes are 2 3/4 by 3 7/8 inches, 4 by 5 1/2 inches and 6 by 7 3/4 inches. Samples up to 20 feet long can be obtained.

The "Denison" sampler is a double tube core barrel designed to sample coarse sand, gravel and gravelly soils, and clays and silts that are too hard to sample with a push type of sampler. The "Denison" sampler can be fitted with inner tube shoes of various lengths that will allow the inner tube to be extended for up to 3 inches beyond the outer barrel. The long shoe is used to sample easily erodible material. Standard "Denison" samplers are available for 2-foot lengths of 4- and 6-inch diameter cores.

The "Pitcher" sampler is a variation of the "Denison" sampler and can be used to sample the same types of soil. The inner barrel is spring loaded to telescope in and out of the outer cutting barrel as the hardness of the material being sampled varies. This feature thus eliminates the need for various lengths of inner barrel shoes. In extremely soft soil the inner tube can lead the cutter bit by as much as 6 inches. The spring loaded inner tube thus automatically adjusts the relative positions of the cutting edges to suit the material being sampled. Standard "Pitcher" samplers are available for 3- to 5-foot lengths in nominal core sizes of 3-, 4- and 6-inch diameter.

A special sampler, not commercially available, was developed by the U.S. Army Engineer Waterways Experiment Station incorporating the principle of the "Denison" sampler to obtain samples of hard or gravelly soils and rock. This sampler was designed to provide an undisturbed sample retained in a 5-inch ID steel tube that could then be cut later into sections for testing without removal of the sample from the tube. Two outer barrel cutter bit arrangements enable the inner tube cutting edge to lead or follow the outer barrel cutter bit. The inner tube adapter is provided with a series of spacers to enable varying the degree of the two relative positions. The core size obtained is a nominal 5-inch diameter by 2.5 feet long.

In core barrel sampling, the core barrel is lowered to within a few feet of the borehole bottom and circulation of the drilling fluid begun. After any excess cuttings that may have settled to the bottom are thus removed, the core barrel is lowered to the hole bottom, rotated, and then forced downward at a uniform rate. The rotational speed and advance rate is adjusted to ensure continuous penetration by steady cutting of the bit. If the advance rate is too rapid the bit will become plugged and grind away the core. If the bit advance is too slow or intermittent the core will be exposed to excessive erosion and torsional stresses. For most soil coring, a rotational speed of 50 to 150 rpm is satisfactory. The drilling fluid pressure and flow rate must also be controlled for cuttings removal. An excessive pressure and flow will erode the core while too low a pressure and flow will allow the cuttings to enter the core barrel along with the core and plug the bit. If the soil is easily eroded and the sharpened inner tube shoe will penetrate the soil under the pressure to be exerted on the core barrel, then the inner tube shoe should extend below the cutting teeth of the outer barrel. If the inner tube shoe will not penetrate the soil, then a shorter shoe having its cutting edge even with or slightly above the cutter teeth of the bit on the outer barrel should be used.

Several other samplers that are suitable for obtaining undisturbed soil samples are commercially available. All have their limitations and are to be used in specific soil types and under certain limiting conditions. They are, for the most part, variations of the thin wall fixed or free piston sampler and/or the core barrel.

Undisturbed rock samples are obtained by coring. These rock cores generally come in four sizes (see Table D-1). The AX and BX cores are the most commonly used sizes. It is advisable to use as large a core diameter as economy will permit. The larger the diameter, the more accurate the geological observations will be on fracturing and jointing and the smaller the core loss will usually be in fractured rock. Diamond bits are the most commonly used.

Table D-1. Standard sizes, in inches, for casings, rods, core barrels, and holes.

Size symbol		Casing OD	Casing bit OD	Core barrel bit OD	Drill rod OD	Approx. diameter of core hole	Approx. diameter of core
Casing, core barrel	Drill rod						
EX	E	1 13/16	1 27/32	1 7/16	1 5/16	1 1/2	7/8
AX	A	2 1/4	2 5/16	1 27/32	1 5/8	1 7/8	1 3/16
BX	B	2 7/8	2 15/16	2 5/16	1 29/16	2 3/8	1 5/8
NX	N	3 1/2	3 9/16	2 15/16	2 3/8	3	2 1/8

The orientation of geologic structure such as bedding, fracturing, foliation, jointing, and shears can be critical in the design of tunnels. Oriented core is one method by which the orientation of geologic structure can be determined; often it will be the only sure method. The orienting core barrel is similar to conventional diamond core. The two principal differences are: three triangular hardened knives or scribes mounted in the shoe at the bottom of the core barrel and a survey instrument attached to the top of the barrel and aligned with the orienting knife. The survey instrument contains a compass-angle device, a multishot camera, and a clock mechanism. The entire instrument assembly is enclosed in a nonmagnetic drill collar. The knives cut three continuous grooves into the core as it enters the barrel. One of these grooves is the orienting groove. The knife that cuts this orienting groove has a fixed alignment with a lug that appears on the survey instruments' photograph of the compass. At predetermined intervals, the drilling is stopped and the time and depth are recorded and a photograph taken. After the coring run is completed, the film is developed and the drift of the hole and reference groove orientation for any desired depth can be determined. The cost for oriented core service is about $225 to $275 per day plus $100 per day and expenses for a service engineer if one is required.

By using a dual drill string and having the drilling fluid go down the hole within the annular space between the dual pipes and then return to surface within the inner pipe, it is possible to have the circulating fluid bring the cores continuously up to surface. This method thus eliminates the need for removing the drill string from the hole to recover the core. In this method, the cuttings are also obtained in addition to the cores and broken core pieces. A nonrotating core breaker is used to break off the core in conviently handled sizes. This method is commercially available as the "Con-Cor" drill and normally requires a two-man crew. A geologist is used to examine the cores.

Appendix E

Borehole logging

A valuable method of analyzing the subsurface conditions is by examination of the various logging records which are available:

1. Driller's log
2. Cuttings log
3. Photographic log
4. Electrical logs (and variations)
5. Radioactivity logs
6. Sonic log
7. Gravitational log
8. Caliper log
9. Temperature log

The use of all of these formation evaluation tools would not be practical or economical. A careful study of the advantages of each technique and the applicabilty of these techniques in specific cases, along with the amount and type of information required, will determine which method or combination of methods can be most effectively used in a specific operation.

Figure E-1 lists several logging techniques which could be used to predict subsurface conditions for tunneling operations.

DRILLER'S LOG

This is probably the oldest logging method and consists of the driller's interpretation of the formations he has encountered during his tour of duty. The value of a driller's logs varies from almost worthless to very valuable, depending upon the experience and methods used by the driller in preparing the logs. A good driller's log is prepared after carefully observing drilling rate, cuttings, and action of the drilling tools. With the experience most drillers have accumulated, a valuable log can be compiled if it is based on overall consideration of these factors.

Borehole logs		Basic physical measurement	Information obtained from basic measurements: Quantitative												Geological and engineering																				
			BOREHOLE DIAMETER	POROSITY MAGNITUDE	PERMEABILITY	WATER SATURATION %	WATER RESISTIVITY	WATER SALINITY	GAMMA RAY INTENSITY	YOUNG'S MODULUS	BULK MODULUS	SHEAR MODULUS	POISSON'S RATIO	FORMATION PRESSURE	CORRELATION	FORMATION DIPS	GEOLOGIC STRUCTURE	DEPOSITION ENVIRONMENT	GEOLOGIC HISTORY	SEDIMENTATION STUDIES	FORMATION LITHOLOGY	MINERAL IDENTIFICATION	DIAGENESIS OF ROCKS	ROCK ANISOTROPY	ROCK MECHANICS	LOCATING WATER RESOURCES	GAS DETECTION	FLUIDS DISTRIBUTION	FLUID MOVEMENT	FLUID PRESSURES	HYDRODYNAMIC STUDIES	POROSITY TYPE	ROCK DEFORMATION	FORMATION DAMAGE	PERMEABILITY - QUALITATIVE
PHOTOGRAPHIC	BOREHOLE TELEVISION	CONTINUOUS ORIENTED PHOTOGRAPHS														•				•			•											•	
	BOREHOLE CAMERA	PHOTOGRAPHS														•				•			•											•	
POTENTIAL	SPONTANEOUS POTENTIAL	NATURAL EARTH POTENTIAL					•	•						•	•	•		•	•	•	•					•	•		•	•	•		•		•
RESISTIVITY	SHORT NORMALS	SHALLOW ZONE RESISTIVITIES				•		•						•	•			•		•				•			•	•	•	•				•	•
	LATERALS	DEEP ZONE RESISTIVITIES		•	•	•	•	•						•	•					•				•		•	•	•	•	•	•			•	•
	GUARD AND LATEROLOG	MODERATE DEPTH RESISTIVITIES				•	•	•							•					•				•		•	•	•	•		•			•	•
	INDUCTION	DEEP ZONE RESISTIVITIES		•	•	•	•								•	•	•		•	•				•		•	•	•	•	•	•		•	•	•
	MICROLOG	FLUSHED ZONE RESISTIVITIES				•									•					•							•	•				•		•	•
	MICROLATEROLOG	FLUSHED ZONE RESISTIVITIES				•									•					•							•	•				•		•	•
	PROXIMITY	FLUSHED ZONE RESISTIVITIES				•									•		•			•							•	•				•		•	•
NUCLEAR	GAMMA RAY	NATURAL RADIOACTIVITY							•						•		•	•	•	•	•	•	•												•
	NEUTRON	NEUTRON CAPTURE		•			•	•						•	•					•	•	•	•			•	•	•		•	•	•			•
	DENSITY	ELECTRON DENSITY		•			•	•						•	•			•	•	•	•	•	•			•	•	•		•	•	•			•
SONIC	VELOCITY	ACOUSTIC VELOCITY		•			•	•						•	•					•	•	•	•		•	•	•	•	•	•	•	•	•	•	
	WAVE AMPLITUDE	AMPLITUDE OF ACOUSTIC WAVE													•						•	•	•		•							•	•		•
	3D VARIABLE DENSITY	FULL ACOUSTIC WAVE DISPLAY		•						•	•	•	•								•	•	•		•							•	•		•
GRAVITATIONAL		GRAVITY INTENSITY		•													•																		
MECHANICAL	CALIPER	HOLE DIAMETER - GEOMETRY	•												•	•																		•	
TEMPERATURE		FORMATION TEMPERATURE												•	•		•		•		•					•			•	•	•				

Figure E-1. Information obtained from some principal geophysical borehole logs (after Birdwell Div. of Seismograph Service Corp.).

CUTTINGS LOG

This logging method is covered under sampling elsewhere in this report, but it is appropriate to mention it here because of its relationship to other logging methods.

The cuttings which have been circulated to the surface or recovered by bailing in cable-tool operations provide a continuous record of the formations encountered. The analysis of cuttings from a rotary-drilled hole requires considerable experience, as a great percentage of the cuttings recovered at a given depth may have been retained in the circulating system from other depths or may have fallen off the wall from some point up the hole.

PHOTOGRAPHIC LOG

Three types of borehole viewing and photographic devices have been developed: the periscope, film-type camera, and television camera. These instruments are very useful for obtaining information that might otherwise not be available from expensive boreholes drilled in tunnel site investigations. Their principal applications are:

1. Examination of borehole walls through intervals of poor or no core recovery (such as gravel, boulders, soft or shattered strata or open voids).
2. Strike and dip orientation of geologic discontinuities such as faults, joints, veins, and formation contacts.
3. Examination of drilling problems such as broken drill rods.

Limitations in the use of these instruments includes:

1. Humidity above the water level in a hole which may fog the lens (heaters or moisture-absorbing devices may be used to alleviate this problem).
2. Below the water level, muddy water will restrict the view (flocculants will often help to clarify).
3. Mud cake on borehole walls will obscure geologic details (must remove by washing-swabbing techniques).
4. Caving holes may prevent entry of the camera or cause its loss.

Where only intermittent use of borehole cameras is required, it is common practice to contract service organizations for these examinations as the purchase price of the equipment required is commonly high.

Borehole cameras may be used in vertical or inclined holes. Changes and improvements are continually being made by the manufacturers.

PERISCOPE

Periscopic viewing devices consisting of a battery powered light and a mirror and lens system enclosed in sectional tubing have been used to depths of about 100 feet in boreholes with diameters greater than 2-1/4 inches. The attachment of a camera at surface allows color or black and white photography.

FILM TYPE CAMERA

Film type cameras in tubes produce photographs of borehole walls on either color or black and white film ranging in size from 16 to 35 mm. These film images are superior to those obtained with TV cameras.

At least four U. S. manufacturers (Seiscor in Tulsa, Ok.; AV Electronics, Inc. in Fresno, Ca.; Republic Research in St. Paul, Mn.; and Pennsylvania Drilling Co. in Pittsburg, Pa.) produce a variety of borehole cameras for use in drill holes to at least 8000-foot depths. Both "down-the-hole" and "right-angle" viewing models using different mirrors, prisms, and lenses are available including one which photographs stereoscopic pairs (for 3-dimensional viewing) and another with a conical mirror for 360 degree coverage. Lighting is generally supplied by a strobe flash unit.

The smallest diameter instrument available requires a hole diameter of 3 inches or greater while other cameras require hole sizes of 6 inches or greater.

At least 30 minutes is required to photograph a point of interest, remove the camera from the hole, and develop the film for viewing.

Sales price of the smallest diameter unit available (for use in 3-inch or larger diameter hole) including power converter was $12,000 in 1965. In addition, a gasoline driven electric generator ($325), a projector printing device ($1500) and a field viewer ($375) were advertised as accessories.

TELEVISION CAMERA

Tube-enclosed miniature television cameras electronically transmit an image of the borehole wall by closed circuit TV to a viewing monitor on surface. This image may be preserved on video tape or features of interest may be recorded by photographing the TV screen with a still camera.

At least two U. S. (AV Electronics, Inc. in Fresno, Ca. and Oceanographic Engineering in San Diego, Ca.) and one German (Eastman International Co. in Hanover) manufacturer produce a limited variety of borehole TV cameras for use in 3-inch or larger diameter drill holes to a maximum depth of 4,000 feet. Incandescent lamps or neon cells are used for lighting. Compact cameras are generally more costly than the larger diameter models, so in some cases it may be more economical to ream the hole to a larger diameter prior to photography.

A vehicle mounted TV system costs approximately $30,000. Other systems are more readily portable and thus usable in rough terrain.

Borehole inspection with portable television systems is presently available from service organizations at a rental cost of approximately $150 per day for the equipment and $150 to $175 per day plus expenses for an operating technician.

SPONTANEOUS POTENTIAL (SP) LOGS

PRINCIPLE

The spontaneous potential method is the only electrical method which uses a natural field, that is, one supplied by *spontaneous* electro-chemical phenomena. All other electrical methods use artificially created electric fields.

The SP curve on an electric log shows spontaneous potentials at different depths in the borehole and represents small electromotive forces caused by infiltration (by the drill mud) of the reservoir rocks or possibly by an electro-chemical reaction between mud and reservoir fluid. It is used to indicate the porosity and permeability of the rocks penetrated: a more or less straight line on the curve corresponds to shales and the maxima to the left corresponds to permeable strata (Figure E-2).

The recording of spontaneous potential logs is calibrated in millivolts and is made according to a simple technique. An electrode located

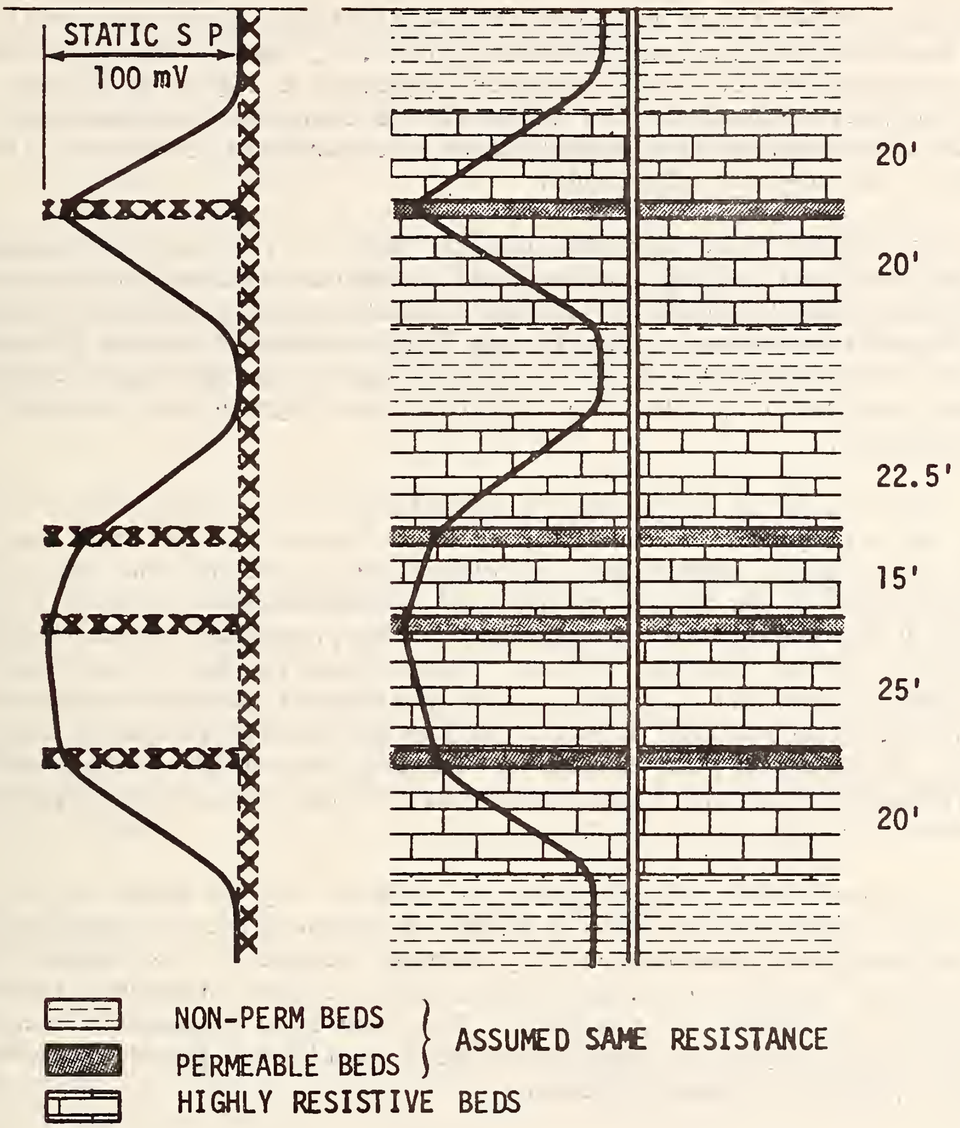

Figure E-2. SP phenomena in highly resistive formations. (Schematic.)

at the end of an insulated cable, is moved up or down in the mud filling the drill hole. The cable passes over a calibrated sheave, and is wound on a winch. Contact with the insulated conductor is established through a slip-ring collector connected to one terminal of a recording galvanometer. The other terminal of the galvanometer is connected to a potentiometric circuit, and then to another electrode that is usually placed in the mud pit or attached to the casing of the hole.

The movement of the sensitive paper or film in the recorder is synchronized with the movement of the electrode along the drill hole. The depth scales used in recording are 1, 2, and 5 inches per 100 feet of electrode motion. The recorder registers a log on which the abscissas are proportional to the depth of the downhole electrode and the ordinates represent the potential of the downhole electrode with reference to the up-hole electrode.

The drill holes in which the SP logs are recorded are usually filled with mud having a water base. The mud density is such that at each depth the hydrostatic pressure in the hole is greater than that in the formation and as a result, the fluid contained in the permeable beds cannot contaminate the mud. Also, the mud is in constant circulation during the drilling operation prior to the logging and, therefore, is homogeneous.

The recording galvanometer measures all the differences of potential appearing between the electrodes. However, experience has shown that under usual conditions, provided proper precautions are taken, the deflections on the SP log correspond to phenomena occurring at the contacts between the beds themselves. These phenomena produce an electric current, called the "SP current," which uses the mud as its return path. (The expression "SP current" may seem rather illogical as SP stands for spontaneous potential; however, it has definitely passed in to common use.) In so doing, it creates in the mud, by ohmic effect, potential differences which can be measured and plotted versus corresponding depths to constitute the SP log.

In electrical logging practice, the SP log is shown on the left hand side track of the record where it can be easily correlated and interpreted with the resistivity curves located to the right. Usually, the SP log consists of a base line, more or less straight, having excursions or "peaks" to the left. The base line frequently has been found to correspond to impervious beds, while the peaks are usually found opposite permeable strata.

USES

Uses of the SP curve are:

1. To detect permeable beds. It will not measure the value of permeability, however.
2. To define the boundaries of zones of permeability, and of individual beds.
3. To calculate the resistivity of interstitial or formation water.
4. To correlate geologic sections from one drill hole to another.

5. As an indicator of lithology where detailed knowledge of the stratigraphy of an area is available.
6. To approximate the amount of shale in a sand, and in favorable circumstances, as a useful indication of the respective proportions of permeable and impervious beds in a given interval.

The instrumentation for the SP log normally is incorporated with an electrode system which measures resistivity of formations to the passage of electric currents through them. The entire system is then run into drill holes on one cable. The resulting electrical survey of the hole is called an ES log or Induction-Electrical log.

COST

The cost of using such electric logging devices will be dependent on the type of service required. As a third party service, 500-foot drill holes can be logged as described in the preceding paragraphs by major logging companies for a direct cost of $600 to $700 per hole. It would be possible to purchase a portable unit or units with which to do the work, in which case savings would be perhaps one third to one half. A one or two man crew would be needed, depending on the type of unit used.

THE SP DIPMETER

Dipmeters are used in a borehole to determine the strike and dip of formations by utilizing lithologic contrasts, such as bedding planes, between different stratified formations. The Dipmeter utilizes three SP electrodes situated 120° apart in a horizontal plane with the electrodes oriented to magnetic north by a downhole compass. A borehole photoclinometer is then used in conjuntion with this assembly to obtain the deviation of the borehole from vertical. Logs recorded over several hundred feet of formation contacts are then correlated to obtain formation dip and attitude.

CONVENTIONAL RESISTIVITY LOGS

PRINCIPLE

Resistivity measurements on the earth's surface have been used to explore for coal, minerals, domes and shallow subsurface structures for a long time. The specific resistivity curves of the borehole electrical log represent a measurement in which an electrical current is passed between two electrodes in the conductive fluid of a borehole and the potential difference between two adjacent potential electrodes is observed. This potential, after suitable corrections for borehole influences, is then related through Ohm's Law to the resistivity of the formations near the measuring electrodes.

Normal symmetrical curves are obtained by having a current electrode at the recording depth and the return current electrode at a distance at least ten times the distance from the potential electrode to the nearest current electrode. Depth of investigation into formations adjacent to the borehole is controlled by the potential-current electrode spacing. The conventional Electric log includes a short 16-inch spacing for shallow investigation and a longer 64-inch spacing for deeper investigation. Electrode spacings have varied throughout the years and from one geologic province to another, however, depending upon the problems existing at the particular time and location.

The Lateral non-symmetrical curve is obtained by having two fairly closely spaced potential electrodes at an appreciable distance, usually 19 feet or more, from the nearest current electrode. The lateral depth of investigation is approximately the potential-current electrode spacing. The Lateral curve obtains a resistivity reading of the rock usually unaffected by mud filtrate invasion, thus permitting a quantitative measurement of the percent of hydrocarbon and water in the rock pore spaces. The Archie water saturation equation was originated in 1938 and still is the basis for approximately seventy percent of the formation evaluation methods for fluid saturation determinations. This equation is:

$$S_w = \left(\frac{R_w}{\eta^m R_t}\right)^{\frac{1}{n}}$$

where S_w is the fraction of water in the pore space,

R_w is the formation water resistivity,

η is the fractional porosity,

m is a cementation exponent obtained from core measurements and logging methods,

R_t is the true resistivity from a Lateral or other resistivity device, and

n is the saturation exponent, generally taken to have a value of 2.

The Lateral curve also is useful for delineation of thin resistive formation breaks and for geological correlation in exploration. The Lateral and Normal resistivity curves have been largely replaced by more sophisticated focused current logs, but the resistivity curves of the Electric log have been responsible for finding millions of barrels of oil and trillions of cubic feet of gas overlooked by all other borehole sampling and exploration methods.

CONVENTIONAL RESISTIVITY DEVICES

In the Normal Device (Figure E-3), a current of constant intensity is passed between electrodes A and B. The resultant potential difference is measured between electrodes M and N. Electrodes A and M are on a sonde and electrodes B and N are, theoretically, located an infinite distance away. In practice, B is the cable armor, and N is an electrode on the bridle (the insulation-covered lower end of the cable), far removed from electrodes A and M. The distance AM is called the spacing (16 inches for the Short Normal and 64 inches for the Medium Normal) and the point of inscription for the measurement is midway between A and M.

In the basic Lateral Device (Figure E-4), a constant current is passed between electrodes A and B, and the potential difference between electrodes M and N, located on two concentric spherical equipotential surfaces centered on A, is measured. Thus the voltage measured is proportional to the potential gradient between M and N. The point of inscription is midway between M and N. The spacing between A and the measure point is 18 feet 8 inches. (The sonde used in practice differs from that shown in Figure E-4 in that the positions of the current and measuring electrodes are interchanged; however, this reciprocal sonde records the same resistivity values as the basic sonde described above. Also, all electrodes are in the borehole, with N located 50 feet 10 inches above M.)

Generally speaking, the longer the spacing, the deeper the device investigates into the formation. Thus, of the ES resistivity logs, the 18-foot 8-inch Lateral has the deepest investigation and the 16-inch Normal the shallowest.

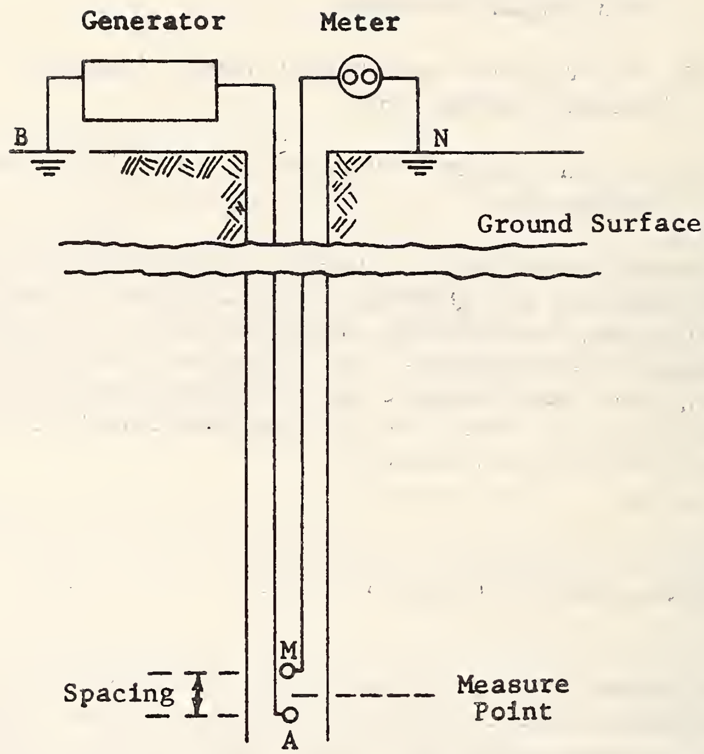

Figure E-3. Basic arrangement of Normal Device.

Resistivity logs are normally incorporated with other logging systems such as SP or Gamma Ray curves with the entire system being run into a drill hole on one cable.

The direct cost of the resistivity log will vary from $600 to $700 when conducted by a major logging company or a third party service. This price will vary depending on the distance of travel for the logging truck and crew.

MICRORESISTIVITY LOGS

Microresistivity devices are used to measure resistivity of the flushed zone of formations penetrated by a drilled hole and to delineate permeable beds by detecting the presence of mud cake.

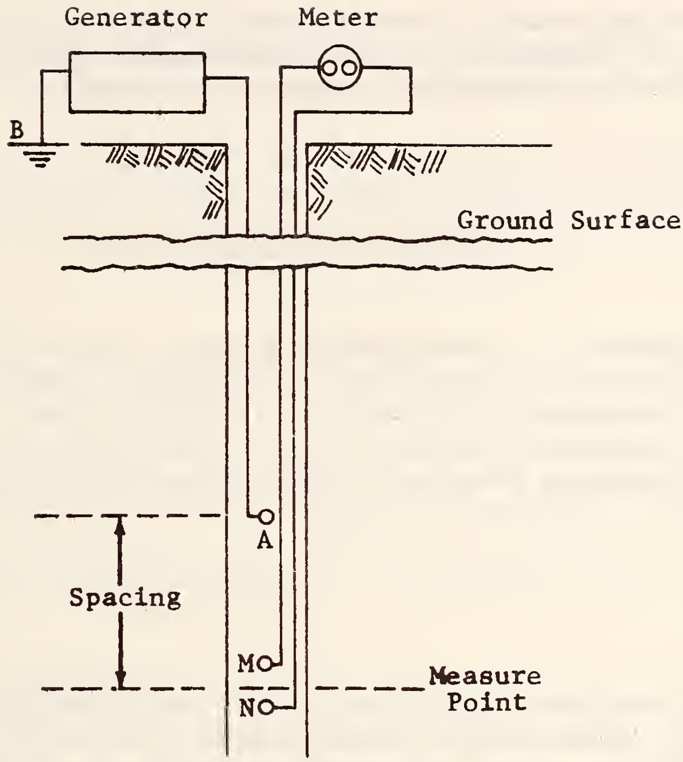

Figure E-4. Basic arrangement of Lateral Device.

PRINCIPLE

By means of arms and springs, a rubber pad is pressed against the hole wall. In the face of the pad are inserted three small electrodes in line, spaced one inch apart. With these electrodes, a 1-inch by 1-inch microinverse and a 2-inch micronormal are recorded simultaneously.

As drilling mud filters into the permeable formations, mud solids accumulate on the hole wall forming a mud cake. The resistivity of the mud cake is about equal to or slightly greater than the resistivity of the mud. It is usually considerably smaller than the resistivity of the invaded zone near the borehole.

The two-inch micronormal has a greater depth of investigation than the 1-inch by 1-inch microinverse. It is, therefore, less influenced by the mud cake, and reads a higher resistivity, producing "positive" curve separation.

USES

The Microlog permits a very accurate delineation of permeable beds in all types of formations. It can also provide satisfactory resistivity and porosity information under favorable conditions.

FOCUSING-ELECTRODE LOGS

The responses of conventional electrical logging (ES) systems can be greatly affected by the borehole and adjacent formations. These influences are minimized by a family of resistivity tools which use focusing currents to control the path taken by the measure current. These currents are supplied from special electrodes on the sondes.

EQUIPMENT

The focusing-electrode tools include the Laterologs and Spherically Focused logs. These tools are much superior to the ES devices for salt muds and/or highly resistive formations and for large resistivity contrasts with adjacent beds. They are much better for resolution of thin to moderately thick beds. Focusing-electrode systems are available with deep, medium, and shallow depths of investigation.

APPLICATION

Resistivity devices using the focusing electrode principle meet certain logging requirements better than other types now available. These requirements are in general as follows:

1. To take measurements leading to determination of the true resistivity in conditions for which the Induction tools are not well suited; i.e., true resistivity values in excess of 100 ohmmeters, and/or mud resistivities of the same order or lower than those of the formation waters.
2. To provide correlation and flushed-zone resistivity determinations in conjunction with deeper-reading resistivity devices.

COST

Focusing-electrode logs are usually run with a Gamma Ray survey, although they may also be run in association with the SP log. All logging systems are run into the drill hole to be surveyed on one cable.

The direct cost of focusing device resistivity surveys conducted in holes whose depth is 500 feet or less will vary from $600 to $700 when run by major logging companies or a third party service.

INDUCTION LOGS

The Induction log was developed to measure formation resistivity in boreholes containing oil-base muds. Electrode devices do not work in these non-conductive muds and attempts to use wall-scratcher electrodes proved unsatisfactory. Experience soon demonstrated that the induction tools had many advantages over the conventional ES for logging wells drilled with water-base muds.

Induction logging devices are focused to minimize the influence of the borehole and of the surrounding formations. They are also designed for deep investigation and reduction of the influence of the invaded zone.

PRINCIPLE

Practical Induction sondes include a system of several transmitter and receiver coils. However, the principle can be understood by considering a sonde with only one transmitter coil and one receiver coil.

High frequency alternating current of constant intensity is sent through the transmitter coil. The alternating magnetic field thus created induces secondary currents in the formations. These currents flow in circular ground-loop paths coaxial with the transmitter coil, and they, in turn, create magnetic fields which induce signals in the receiver coil. The receiver signals are essentially proportional to the conductivity of the formations. Any signal produced by direct coupling of transmitter and receiver coils is balanced out by the measuring circuits.

The Induction log operates to advantage when the borehole fluid is an insulator - even air; but the tool will also work very well when the borehole contains conductive mud, provided the mud is not too salty, the formations not too resistive, and the borehole diameter is not too large.

COST

The Induction log can be used most effectively in holes filled with moderately conductive drilling muds, non-conductive muds, and in empty holes. Vertical focusing is good, making possible reliable evaluation of beds down to about 3½ feet.

The direct cost of Induction surveys will vary from $600 to $700 in most areas of the United States where conducted by major logging companies. Distance traveled by logging truck and crew may affect this price somewhat.

GAMMA RAY LOGS

PRINCIPLE

The Gamma Ray log is a measurement of the natural radioactivity of the formations. In sedimentary formations, the Gamma Ray log normally reflects the shale content of the formations because the radioactive elements tend to concentrate in clays and shales. Clean formations usually have a very low level of radioactivity, unless radioactive contaminants such as volcanic ash or granite wash are present, or when the formation waters contain dissolved potassium salts.

The Gamma Ray log can be recorded in cased wells. It is frequently used as a substitute for the SP log in cased holes where the SP is unavailable or in open holes where the SP is unsatisfactory. In both cases, it is useful for the location of the non-shaly beds and for correlation.

APPLICATIONS OF THE GAMMA RAY LOG

The Gamma Ray log is used:

1. To define shale beds when the SP curve is rounded (in very resistive formations) or flat, or when the SP curve cannot

be recorded (non-conductive muds, empty holes, or cased holes).
2. To determine the proportion of shale and, in some regions, can be used quantitatively as an indicator of shale content.
3. To detect and evaluate radioactive minerals, such as potash or uranium ore.
4. To delineate non-radioactive minerals including coal beds.
5. To correlate beds in cased holes.
6. Sometimes in connection with radioactive tracer operations.

COST

The Gamma Ray log normally is incorporated and run with other logging systems, such as the neutron, acoustic, density, or resistivity surveys. The entire system is then run into a drill hole on one cable.

The cost of the Gamma Ray log is usually incorporated with the cost of associated logging systems. As a third party service, a 500-foot deep drill hole can be logged by major logging companies for a direct cost of $500 to $700. This price will vary depending on distance of travel for the logging truck and crew.

NEUTRON LOGS

Neutron logs are used principally for delineation of porous formations and determination of their porosity. They respond primarily to the amount of hydrogen present in the formation. Thus, in *clean* formations whose pores are filled with water or oil, the Neutron log reflects the amount of liquid-filled porosity.

Gas zones can often be identified by comparing the Neutron log with another porosity log or a core analysis. A combination of the Neutron log with one or two other porosity logs yields even more accurate porosity values and lithology identification, including evaluation of shale content.

PRINCIPLE

With the Neutron logging tool, high energy neutrons are continuously emitted from a radioactive source which is mounted in the sonde. These neutrons collide with nuclei of the formation materials in what may be thought of as elastic "billiard-ball" type collisions. With each collision, a neutron loses some of its energy.

The amount of energy that is lost per collision depends on the relative mass of the nucleus with which the neutron collides. The greatest energy loss occurs when the neutron strikes a nucleus of practically equal mass, i.e., a hydrogen nucleus. Collisions with heavy nuclei do not slow the neutron down very much; and thus, the slowing-down of neutrons depends largely on the amount of hydrogen in the formation.

Within a few microseconds of emission, the neutrons have been slowed down by successive collisions to thermal velocities corresponding to energies of around .025 electron volts. They then diffuse randomly, without losing any more energy, until they are captured by the nuclei of atoms such as chlorine, hydrogen, silicon, etc.

The capturing nucleus becomes intensely excited and emits a high energy gamma ray of capture. Depending on the type of Neutron logging tool, either these capture gamma rays or the neutrons themselves are counted by a detector in the sonde.

When the hydrogen concentration of the material surrounding the neutron source is large, most of the neutrons are slowed down and captured within a short distance of the source. Likewise, if the hydrogen concentration is small, the neutrons travel further from the source before being captured. Therefore, the counting rate at the detector (with the source-detector spacings commonly used) will increase for a decreased hydrogen concentration, and vice versa.

EQUIPMENT

Neutron logging tools in use include the GNT series, the newer SNP (Sidewall Neutron Porosity), and the CNL (Compensated Neutron Log). They use plutonium-beryllium, radium-beryllium, or americium-beryllium sources to provide neutrons with initial energies of several million electron volts.

The GNT is a non-directional device, employing a detector that is sensitive to both high energy capture gamma rays and thermal neutrons. It can be run in either cased or uncased holes. Porosities obtained from GNT logs run in cased holes, however, are less accurate because of the uncertainties arising from the casing weight and position, the presence of cement behind the casing, etc. Several source-detector spacings are available so that the best spacing can be used for the particular borehole conditions and porosity range. The GNT is run off hole-center to minimize borehole effects.

In the SNP, the neutron source and detector are mounted on a skid which is applied to the borehole wall. The neutron detector is a proportional counter, shielded so that only neutrons having energies above about 0.4 electron volts are detected.

The SNP has several advantages:

1. Because it is a sidewall device, borehole effects are minimized.
2. Because epithermal neutrons are measured, the perturbing effects of strong thermal neutron absorbers (such as chlorine and boron) in the formation waters and matrix are minimized.
3. Most of the corrections required are performed automatically in the panel.

The CNL is a mandrel-type tool especially designed for use in combination with any of several other tools to provide a simultaneous Neutron log. The CNL is a dual-spacing, thermal-neutron-detection instrument. The ratio of counting rates from the two detectors is processed by surface equipment to produce a linearly-scaled recording of neutron porosity index. A 16-curie source produces neutrons at four times the rate of a standard logging source, greatly reducing statistical variation. The use of longer source-to-detector spacings increases the depth of investigation. A comparison of CNL with SNP logs run in the same wells shows that the CNL has a radial depth of investigation appreciably greater than that of the SNP. The effects of borehole parameters are greatly reduced by taking the ratio of two counting rates which are similarly affected by these perturbations. The CNL can be run in liquid-filled holes, either cased or uncased, but cannot be used in gas-filled holes.

Most Neutron logging sondes in current use are designed so that both Gamma Ray and Neutron logs can be run simultaneously. When the GNT or CNL is run in cased hole, a casing collar log can be also run.

GNT-type tools are available with 3 5/8-inch and 1 11/16-inch diameters; there is also a special high-temperature tool with a diameter of 2 5/8 inches.

The SNP equipment is designed for operation in uncased holes, either liquid filled or empty. The minimum hole diameter in which this tool can be used is 5 inches. A Caliper curve is recorded simultaneously with the SNP Neutron.

The CNL is available in 3 3/8-inch and 1 11/16-inch diameters. Only the larger tool is combinable with other tools at the present time.

USES

Determination of porosity is one of the most important uses of Neutron logs. Corrections for lithology and borehole parameters are necessary for accurate porosity determinations.

The SNP is specifically designed for use in open holes, and provides porosity readings having minimum borehole effect. It is the only effective Neutron tool for use in gas-filled holes.

The CNL is designed for use in combination with other open-hole or cased-hole tools. The compensation feature greatly reduces the effects of borehole parameters.

The Neutron log is used in combination with other porosity logs for lithology interpretation.

COST

The cost of Neutron logging systems conducted by major logging companies in drill holes 500 feet or less in depth in association with at least one other log will vary according to the type of system used. Neutron loggin as first developed will have a direct cost of about $350, while Sidewall Neutron Porosity loggin should average about $475. Compensated Neutron logging costs will be somewhat higher at about $550.

FORMATION DENSITY LOGS

The Formation Density log is useful as a porosity-logging tool. Other uses of density measurements include identification of minerals in evaporite deposits, detection of gas, and evaluation of shaly sands and complex lithologies.

PRINCIPLE

A radioactive source, applied to the hole wall in a shielded sidewall skid, emits medium-energy gamma rays into the formations. These gamma rays may be thought of as high-velocity particles which collide with the electrons in the formation. At each collision, a gamma ray loses some, but not all, of its energy to the electron and then continues with diminished energy. This type of interaction is known as Compton scattering. The logging source and detector of the logging tool are so designed that the tool response is predominantly due to this phenomenon. The scattered gamma rays reaching the detector, at a fixed distance from the source, are counted as an indication of formation density.

The number of Compton-scattering collisions is related directly to the number of electrons in the formation. Consequently, the response of the Density tool is determined essentially by the electron density (number of electrons per cubic centimeter) of the formation. Electron density is related to the true bulk density, in grams per cubic centimeter, which in turn depends on the density of the rock matrix material, the formation porosity, and the density of the fluids filling the pores.

EQUIPMENT

In order to minimize the influence of the mud column, the source and the detector, mounted on a skid, are shielded. The openings of the shields are applied against the wall of the borehole by means of an eccentering arm. The force exerted is substantially greater than in the case of a microsonde, and the skid has a plow-shaped leading edge. Therefore, it is able to cut through soft mud cakes usually encountered at medium and shallow depths. However, some mud cake may remain in place between the skid and the formation at greater depths when mud cakes are hard. Any mud cake or mud remaining between the tool and the formation is "seen" as part of the formation by the tool and must be accounted for.

A correction is needed when the contact between the skid and the formations is not perfect (due to mud cake or roughness of the borehole walls). In unfavorable cases, this correction can be fairly large. If only one detector is used, the correction is not easy to determine, as it depends on the thickness, the weight, and even the composition of the mud cake or mud interposed between the skid and the formations.

COST

The direct cost of the Formation Density log for hole depths of 500 feet or less will be $450 to $550 when service is performed by a major logging company in conjunction with one or more other surveys.

SONIC LOGS

The Sonic log is a recording, versus depth, of the time, *delta t*, required for a compressional sound wave to traverse one foot of formation. Known as the interval transit time, *delta t* is the reciprocal of the velocity of the compressional sound wave. The interval transit time for a given formation depends upon its lithology and porosity. Its dependence upon porosity, when the lithology is known, makes the Sonic log very useful as a porosity log. Integrated acoustic transit times are helpful in interpreting seismic records.

EQUIPMENT

The Sonic tools in current use are of the BHC (borehole compensated) type. This type sonde substantially reduces spurious effects at hole-size changes as well as errors due to sonde tilt.

The BHC system uses one transmitter above and one transmitter below two pairs of receivers. When one of the transmitters is pulsed, the sound wave generated enters the formation and the elapsed time between detection of the first arrival at the two corresponding receivers is measured.

The BHC transmitters are pulsed alternately, and values are read on alternate pairs of receivers. The *delta t* values from the two sets of receivers are averaged automatically by a computer at the surface. The computer also integrates the transit-time readings to obtain total travel times.

With the BHC system, the quality of the measurement is very good (accurate calibration, no borehole effect, and excellent vertical definition). The relationship between *delta t* and porosity is somewhat complex, but good values are found for formations containing intergranular porosity.

USES

Uses of the Sonic log are:

1. For correlation in many cases where other logs give poor results. Some lithologies are identified by the magnitude of the *delta t* reading.

2. In combination with other porosity logs to evaluate shaly sands, determine formation lithology, and determine the amount of secondary porosity.
3. The integrated travel time from a Sonic log is useful in seismic interpretation.
4. Overpressured formations can be studied from *delta* t data.

COST

The Sonic log is always run with a hole caliper survey and is generally one of several other logging systems utilized in association with it. When it is run with at least one other survey by a major logging company, the direct cost should vary from $400 to $500 in holes 500 feet in depth.

OTHER SONIC WAVE MEASUREMENTS

A complete analysis of an elastic wave propagated through a porous medium cut by a borehole shows that a multitude of formation and borehole parameters affect the amplitude and shape of the wave. However, as long as some of the parameters are fixed or predictable, elastic wave characteristics can be related to unknown parameters of interest. In regular Acoustic *delta t* logging, for example, the velocity of one part of the wave is related to formation porosity. Various other wave characteristics have been empirically related to other formation parameters.

The flexibility of the BHC equipment permits the recording of Acoustic logs other than *delta t* logs. These include the Amplitude log for fracture detection, Cement Bond log, and Variable Density log (or 3-D Velocity log).

Sonic Amplitude logs have been used to locate fracture systems in hard formations. Both Compressional and Shear Amplitude logs are recorded. Generally, the compressional wave has been found to be attenuated more by vertical and high-angle fractures, while the shear wave seems to be more sensitive to horizontal and low-angle fractures. The occurrence of fractures is generally limited to carbonate rocks and hard sandstones. The amplitude of the sonic wave is large unless the rock is fractured. Over a fractured zone it may be reduced by a factor of as much as ten or twenty.

Interpretation of the Sonic Amplitude log for fracture detection is qualitative and empirical, largely based on comparison of the logs with core and drilling data. Bedding planes and thin shale streaks, as well as "healed" fractures, may give the same response as open fractures, so considerable care is necessary in the interpretation. Tool centering is very important.

Another approach to fracture location uses two porosity logs, a Sonic log, and either a Density or a Neutron log. In this method, it is assumed that the Sonic *delta t* ignores the fractures, responding only to the matrix porosity. The Density or Neutron log, on the other hand, reads the total porosity. Therefore, any difference between the two logs (in a clean formation of known lithology) is interpreted as fracture porosity. Where this method is used, lower limits are usually set on the minimum total porosity and minimum fracture porosity necessary to make a commercial well.

The 3-D Velocity logging system utilizes transmitting and receiving transducers placed a known distance from each other. The transmitter generates pulses and the receiving transducer detects any pressure waves reaching it in a borehole and converts these pressure waves to electric signals which are transmitted, along with a synchronized pulse, to the surface recording equipment. The total wave train, including pressure, shear, and boundary waves, can be displayed as intensity modulation, or variable density, by a specially designed camera with a fibre optic face. This information, in addition to time and depth lines, is recorded on film which moves past the face of the fibre optic tube synchronized with the depth measuring device. The length of the sweep can be adjusted to display selected portions of the received wave trains ranging from 250 microseconds to 25 milliseconds.

Computer programs are applied to the raw data obtained from the 3-D Velocity log and the Density log to compute the corrected compressional and shear wave velocities; the Shear, Bulk, and Young's moduli; and Poisson's ratio. These elastic moduli, computed from theoretical relations developed for homogeneous, isotropic, and elastic materials, indicate good agreement with other sources of measurements.

In subsurface engineering problems associated with the science of rock mechanics, the 3-D Velocity log can provide in situ measurements with sufficient accuracy to be of practical field application. The in situ dynamic properties obtained from the 3-D Velocity log have been used to investigate rock mechanics problems involved with dam construction, tunnels and canals, and for foundation studies for buildings and nuclear power plants. The 3-D Velocity log utilized to investigate the fracturing and rate of subsidence of the roof over caverns.

The hole-to-hole technique is also applied with the 3-D Velocity log to investigate the fractures and bedding planes existing between two boreholes. In this technique, the transmitter is in one hole and the receiver is moved upward in the other hole. The distance over which an acoustical signal can be transmitted hole-to-hole is approximately 200 feet in competent rock media.

The boundary wave (Tube or Stonely wave) propagation can be used to compute the bulk density of the formation. Boundary waves are elastic waves that occur in a borehole because of the existence of a fluid-rock boundary (the borehole wall) that is cylindrical.[86] The propagation characteristics of these waves depend upon the ratio of the wavelengths to the hole diameter. If the wavelengths are small in relation to the hole diameter, the waves are called Stonely waves and their velocity depends upon the wavelength and the elastic moduli. When the wavelengths become greater than five times the hole diameter the wave velocity becomes independant of the wavelength and these waves are then called Tube waves.

The following Lamb's equation indicates the relation of elastic wave velocities to the ratio of the densities of borehole fluid and borehole wall rock for Tube waves.

$$\rho_b = \frac{\rho_f}{(V_f + V_t)(V_f - V_t)} \left(\frac{V_f\, V_t}{V_s}\right)^2$$

where ρ_b is the formation density,

ρ_f is the borehole fluid density,

V_f is the fluid velocity,

V_s is the shear velocity, and

V_t is the Tube wave velocity.

The relation between the density ratio and elastic wave velocities is much more complicated in the case of the Stonely waves which are dispersive.

In petroleum engineering problems involving exploration, production, and reservior studies, the 3-D Velocity log is used for amplitude investigation and cement bonding evaluation, and to determine rock porosities, fracture locations, and gas and water contact zones. The 3-D Velocity log can be operated in cased and uncased holes. The 3-D Velocity log is the only Acoustic log that detects cement bonding to both, or to either, the formation and/or casing. Conventional amplitude logs do not indicate quality of cement bonding to formations. Recently some researchers have utilized the 3-D velocity log for bulk lithology evaluations in carbonates and evaporites.

[86] Geyer, R. L. and J. I. Myung. The 3-D Velocity Log, A Tool for In Situ Determination of the Elastic Moduli of Rocks. Birdwell Division, Seismograph Service Corp., Tulsa, Oklahoma. 1973.

By timing the interval that occurs between seeing a flash of lightning and then hearing the associated thunder, it is possible to calculate the distance from the observer to the region in which the lightning occurred; the distance being the product of the velocity for sound in air and the elapsed time. The region of the lightning occurrence can be located with fair accuracy because the visual data shows the direction and the sound derived data shows the distance. Without this usual data, we would know neither the elapsed time from the generation of the sound to its arrival at a location nor the direction of the region in which the sound was generated.

The need for visual data may be eliminated by generating the sound in a controlled manner and then measuring the time interval from the time of generation to the time an echo is received from objects of interest. The pulse-echo method of locating objects, or flaws, in materials has been applied extensively utilizing well developed techniques of sonar and ultrasonic nondestructive testing.

The technology of acoustical imaging and mapping is advancing rapidly. One new development is the application of holography to acoustical imaging.

Just as the mechanical disturbances generated by an earthquake travel for thousands of miles through the earth, the sound generated by an acoustical transducer will penetrate rock for a considerable distance. The acoustic transducers are used to convert electrical signals into mechanical, i.e. acoustic or sound, energy which propagates through the rock. As illustrated in Figure E-5, these acoustical signals are reflected from anomalies in the surrounding rock. The reflected sound energy is converted to electrical signals by the same transducer or a second transducer mounted close to it. Both the amplitude of the received signal and the elapsed time between the time of generation and the time the echo is received are accurately measured. This information is then further processed to produce the raw data needed for mapping anomalies in the rock. It should be emphasized that what is really achieved is a mapping of the acoustical interfaces of the surrounding volume. However, this mapping may be interpreted to indicate fault structures and mineralized zones.

The ability to carry out this work rests in part upon the availability of high-power transducers small enough for insertion into holes as small as 1 7/8 inches in diameter. The transducer will thus fit in an AX-size diamond drill hole and may be used in larger size holes. These transducers must also be sensitive enough to detect echoes from anomalies.

There are two types of loss which limit the range that can be

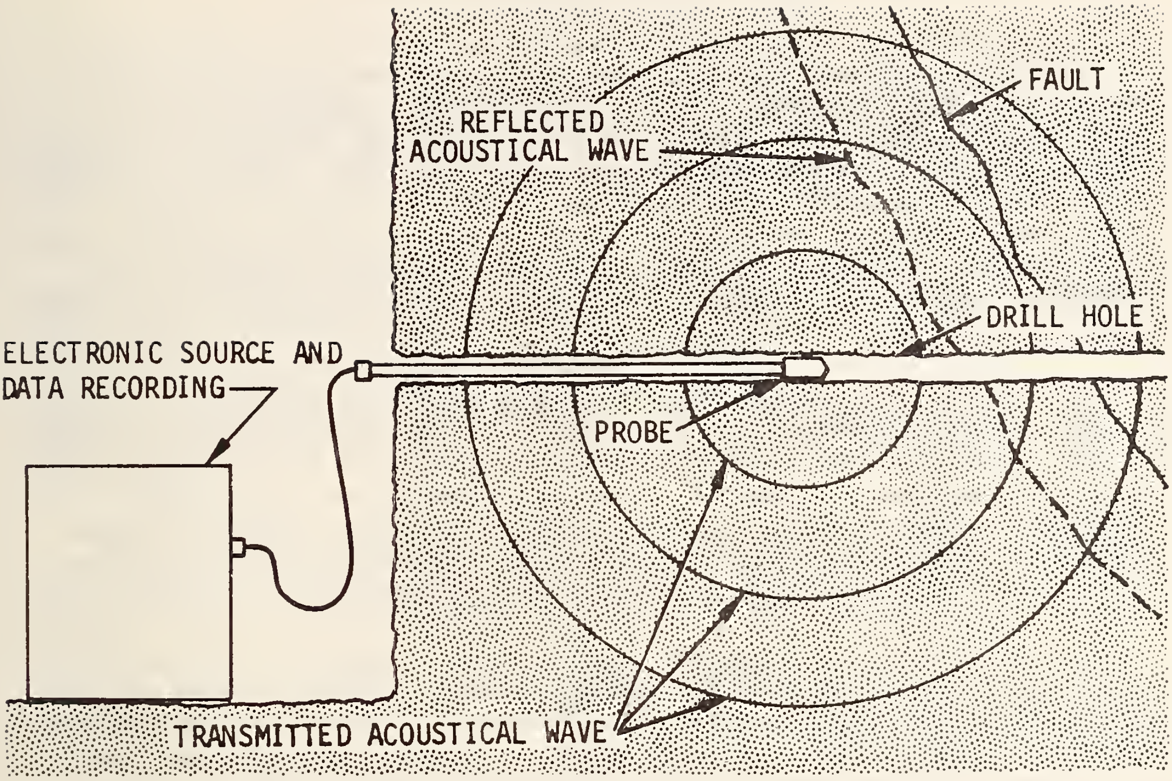

Figure E-5. Operation of acoustical prospecting from drill holes.

achieved with this system. The first type of loss is due to the fact that at greater distances the acoustical energy is spread out over a larger surface. A reflecting object of any given size represents a smaller fraction of this surface as its distance from the source increases. Thus, the reflected energy decreases inversely as the square of the distance from the probe. This inverse square relationship assumes that the reflecting object is a large planer surface. Losses are much more severe, and the resulting loss of energy varies as the inverse fourth power of the range, if the object is a small diffuse reflector.

The second source of energy loss is the absorption of acoustical energy in the rock. This absorbed energy will be transformed into heat and will, therefore, not be available to be detected as a mechanical disturbance. The absorption of energy in rock is strongly dependent upon the frequency of the acoustical energy. Higher frequencies are absorbed much more strongly than the lower frequencies. There is a trade-off, however, which prevents going to very low frequencies since at these frequencies the resolution becomes very poor. That is, reflecting objects are located with much less accuracy.

The location of acoustical anomalies in rock would be very simple if it was only necessary to send out a pulse and measure the elapsed time for all return echoes. Unfortunately, the situation is not that simple. First, such time intervals merely tell the distance to the reflecting anomaly and do not indicate the direction. Secondly, there is a confusion in the return echoes since almost always there are several reflecting objects within range. There would be some reduction in complexity if it were possible to sound out the distances to the reflecting objects from three different drill holes located around the object.

The objective, however, is to provide extensive information about the surrounding geologic structure from a single drill hole and thereby greatly reduce the number of drill holes required to fully describe a given fault system. It may be possible to accomplish this and to establish complete orientation and mapping information from a single drill hole through the special design of an acoustical probe together with a novel method of data collection and processing.

The program for mapping the region around a 400-foot section of diamond drill hole involves the taking of data at several hundred points and includes some 2,000 or more bits of data. So much data is hard to analyze by manual methods and the need for computer analysis arises. One company has developed a Fortran program to handle this analysis. The output of this program purportedly indicates the fault planes in the region surrounding the diamond drill hole, each fault plane being given its strike and dip values. Computer analysis, following the dictates of the Fortran program, then provides the structure angle and cross sectional thickness of the fault plane or mineralized zone. This analysis also provides the information necessary to calculate the strike and dip of the detected anomalies. In effect, the computer analysis of the collected data reconstructs a three-dimensional representation of the structures which must exist to produce the particular pattern of data collected.

The data is collected in the following manner. At each probe position distance into the drill hole, there are reflections from fault planes lying at several distances measured from the position of the probe. At each probe position, the amplitude and the elapsed time of each reflected acoustical signal is recorded. The signal elapsed time is converted to distance and the raw data, consisting of distance and reflected signal amplitude, is then analyzed by computer.

GRAVITATIONAL LOGS

The Borehole Gravimeter is a relatively new tool that measures gravity, and thus bulk density and porosity approximately 500 feet laterally. Conventional borehole porosity logging devices only obtain measurements from a few inches to a few feet out from the borehole.

The Borehole Gravimeter is a high precision, large volume, bulk density tool which provides measurements that are relatively unaffected by mud cakes, invaded zones, washouts, or casing. This device is particularly useful for measuring low porosities, such as fractured hydrocarbon-bearing rocks, and for obtaining fluid density behind casing. It is an ideal tool for bulk averaging the porosity of vugular and fractured rocks and all heterogeneous formations. It distinguishes gas from oil or water.

The difference in gravitational attraction between any two depths in a borehole is a function of the free air gradient and the mass of material between the two depths as summarized in the following equation:

$$G_1 - G_2 = \Delta G = F\Delta Z - 4\pi K\rho\Delta Z$$

where F is the free air gradient = 0.09406 milligals per foot,

$4\pi K\rho$ is twice the Bouguer gradient = 0.02551ρ milligals per foot,

ρ is the average bulk density between the two depths, and

ΔZ is the difference in elevation between the two depths.

Then

$$\Delta G = 0.09406\Delta Z - 0.02551\rho\Delta Z$$

or solving the equation for bulk density,

$$\rho = 3.687 - 39.20 \frac{\Delta G}{\Delta Z}$$

The downhole gravimeter measures ΔG while ΔZ is measured by depth meters. The measurement is unique in that it requires no calibration, as is required in most geophysical logging methods, and a very large mass of rock is measured.

CALIPER LOGS

Caliper logs are used to measure the diameter, volume, and geometry of boreholes. They are sometimes used for subsurface formation correlation based on the differential "washing out" or caving of diverse formations. Very few boreholes are truly round, but instead tend to be elliptical in cross section with the long axis trending parallel to the regional strike of certain formations under stress and folding. Thus, Caliper logs in conjunction with Dipmeter surveys have become an exploration tool. Caliper logs are necessary to all logging programs for the possible correction of raw logging data for borehole effects, particularly for the non-focused logs. They will also provide an approximate estimate of mud cake thickness.

Caliper logging tools are either one, two, three, four, or six arm in construction. The Birdwell Division of Seismograph Service Corporation has developed huge Six-Arm Caliper and Density tools especially for use in large diameter holes drilled by the Atomic Energy Commission in Nevada.

The Caliper tool is a highly sensitive borehole diameter measuring device. In practice, it is run with a Gamma Ray detector to provide precise depth correlation between the Caliper log and other logs run in the same hole. One Caliper tool has three independently operated measuring arms which ride the wall of the borehole and can detect variations as small as 1/4 inch in diameter. The arms are extended and retracted as desired from the surface by sending current to a small electric motor in the tool. This feature allows repeated runs over a section of the hole without returning to the surface.

The Birdwell Six-Arm Caliper may be programmed to record three different diameters 120° apart, as well as an average diameter. Because of the independent action of each arm, the diameters recorded are those of the circles described by the tips of the three pairs of arms regardless of their radial position in relation to the center of the tool. These diameters can be recorded with a choice of 1-, 2-, or 4-inch scales per horizontal inch of record with all scales shown in their correct depth relationship.

When hole volumes are desired at the time of logging, a special surface panel is used to obtain them. This panel is capable of taking two input signals and providing an integrated output signal. One input signal is the depth factor and the other input signal is the radius squared of the average diameter as recorded with the Six-Arm Caliper.

TEMPERATURE LOGS

Temperature logs record the temperature of the fluid in the borehole, which reflects thermal relationships between the formations and their fluids. Initially, they were employed to log formations through the differential thermal properties of adjacent formations. Present uses include the location of cement behind casings, the determination of rock permeability in formations flowing gas into empty boreholes, and the location of permeability and fluid movements following hydraulic fracturing of low porosity formations. In recent years, temperature surveys have been used in regional exploration for structures such as salt domes and to study the movement of subsurface formation waters in areas under hydrodynamics.

In the temperature logging tool, a thermistor that is very sensitive to changes in temperature is incorporated in the electrical circuit. A small temperature change causes a large change in electrical resistance that is recorded at the surface recorder. The thermistor is mechanically mounted so that the effect of mounting and the protective material around it has a minimum effect on the temperature change sensed by the thermistor. These resistance changes are calibrated in degrees Fahrenheit and may be recorded at one degree per inch on the chart, two degrees per inch, or five degrees per inch.

Computerized Differential Temperature Surveys were introduced a few years ago to amplify the small temperature anomalies between adjacent depth levels. Magnetic tape is employed and the log represents a continuous analog plot of differential temperatures versus depth.

NEEDED IMPROVEMENTS IN BOREHOLE LOGGING

One of the major disadvantages of the previously described logging techniques is that they do not provide detailed information for design at any great distance from the borehole. The only major exception is the acoustical imaging technique which can provide dip, strike, and location of faults, joints, and facies changes for distances up to 50 or 100 feet in rock depending on its hardness. In soils, however, the range of this technique is greatly limited. Another problem with these logging techniques is that most of these logs must be run in an open, fluid filled hole.

Access to work being done on new and comtemplated borehole logging techniques is quite difficult to come by. The various petroleum, mining and service companies are reluctant to talk about proprietary information which is still under development. For this reason, it is quite difficult to present here a complete list of new and contemplated geophysical borehole logging techniques.

A relatively new logging method has been developed that used an acoustic mapping technique which can be run in a single horizontal borehole.[87] This technique is only effective in rock and must be run in an open hole. The geophysical data is computer processed and the results indicate the strike, dip, and relative position of faults, joints, bedding planes, and other major structural features. Holosonics is the major developer of this geophysical logging system.

Relatively recent work by the U. S. Geological Survey (USGS), Denver, Colorado, has shown that reliable estimates of the in situ dynamic moduli of rock can be made from data provided by full wave sonic logs.[88, 89] The USGS found that the dynamic Young's, Shear, and Bulk modulus; shear velocity; shear and compressional characteristic impedance; and amplitude and energy reflection coefficients may be reliably estimated on the basis of the compressional wave velocities of the rocks investigated.

Birdwell, a division of Seismograph Service Corporation, has also found that their 3-D Velocity logging system will provide in situ measurements of dynamic rock moduli to be of practical field use.[90]

FUTURE TRENDS

Future trends in logging research and development in the areas of acoustics, neutron activation, and nuclear research are outlined below.

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[87] Brenden, B. B., V. I. Neeley, and G. F. Garlick. Acoustical Prospecting From Diamond Drill Holes, presented at the Colorado Mining Association 73rd National Western Mining Conference, Denver, Colorado. February, 13-14, 1970.

[88] Carroll, R. D. "The Determination of the Acoustic Parameters of Volcanic Rocks From Compressional Velocity Measurements." International Journal Rock Mechanics and Mining Science, Vol. 6. 1969. pp. 577-579.

[89] Carroll, R. D. and J. E. Paul. The Estimation of In Situ Acoustic Properties of Volcanic Rocks from Compressional Velocity Measurements--Central Nevada. Central Nevada Report No. 39, USGS-474-69. U. S. Geologic Survey, Denver, Colorado. 1970.

[90] Geyer, R. L. and J. I. Myung. The 3-D Velocity Log, A Tool for In Situ Determination of the Elastic Moduli of Rocks. Birdwell Division, Seismograph Service Corp., Tulsa, Oklahoma. 1973.

ACOUSTICS

Promising trends in acoustic studies in borehole measurements include:

- Attenuation
- Borehole scanning
- Multiple-wave forms from different modes of source operation
- Variable frequency sources
- Variable intensity recording
- Pulse amplitude
- Shape and rise time
- Tape signals to generate all possible recorded curves
- Computer data processing and interpretation techniques.

Potential applications from these studies will include:

- Defining lithology
- Determination of rock structure from elastic parameters
- Detection and evaluation of fractured and regular zones
- Improved generation and detection of compressional, shear, and other wave types
- Improved generation and detection of an acoustic "picture" or visual record of the formation around the borehole
- The realistic determination of porosity, permeability, and formation saturation.

NUCLEAR

Continuing research in radiation physics as applied towards borehole logging include attempts to:

- Increase detector selectivity and efficiency
- Increase neutron generator source life and neutron output
- Provide accurate in-hole spectra recording and on site spectral analysis
- Improve detection and data handling techniques
- Define formation chemical effects on neutron capture
- Model neutron die-away as a means of simulating neutron capture mathematically
- Devise an effective through-casing porosity device.

A few of the potential applications resulting from the improved nuclear devices or interpretation techniques could include:

- Lithology from elemental analysis

- Formation fluid content
- Compensated through-pipe porosity measurements
- Mineral identification
- Data regarding the rock and soil structure.

NUCLEAR RESONANCE

It has been reported that nuclear resonance measurement has considerable versatility for future applications. Future applications could include the measurement of:

- Permeability
- Formation fluid type and content
- Lithology
- Indications of pore size.

At present, this technique requires more powerful nuclear sources, imporved signal detection, and new data processing techniques.

Many of these logs can and should be run together in the borehole. The logging time saved would greatly reduce logging costs. In addition, the computer evaluation of logs can provide large time and cost savings. Three areas of concern regarding improvements in computerized logging are data transmission-processing systems, log digitizing, and computer programs for log evaluation.

OTHER FUTURE DEVELOPMENTS

Several organizations are working on new and improved borehole logging techniques, but, as stated earlier, details are difficult to obtain. Holosonics of Richland, Washington, is working on an in-hole acoustical holography viewer. It is claimed that with this device it would be possible to "see" several feet into the material penetrated. A real-time television-type picture of the material structure could be obtained. The data would be recorded for future playback and computer processing.

Telcom, Inc. of McLean, Virginia, is working on several proprietary borehole logging systems. These systems would utilize Telcom's proprietary "cableless" telemetry link for transmitting geophysical data from the in-hole sensor to the out-of-hole processing equipment.

Appendix F

Exploration Excavations And In Situ Testing

TRENCHES AND PILOT EXPLORATION TUNNELS

Trenches and pilot tunnels are used in site investigations when geologic information is required in greater detail than surface inspection, borehole samples, or indirect subsurface investigation techniques can provide.

TRENCHES

Trenches, test pits, and adits are used to investigate subsurface conditions to relatively shallow depths and generally penetrate soil or weathered rock. These excavations are most commonly used at tunnel portal areas where support problems often occur, but may also be used to advantage elsewhere along the tunnel route.

At portal areas; trenches, test pits, or adits are usually used to determine depth and character of the overburden or weathered bedrock, to provide openings for sampling, and to determine hydrologic conditions and planes of weakness which may cause slope stability problems. Along the inner portion of the tunnel alignment, the main applications of these excavations is to expose bedrock for geological mapping and sampling to thinly covered areas.

Trenching is relatively inexpensive, but seldom employed to depths greater than 20 feet. Trenches may be cut by hand methods, conventional earth excavating equipment, or hydraulicking with a stream of water. Sloping or stepped sides are used in deep trenches for safety.

Test pits or shafts and horizontal adits are more costly than trenches but may be used to reach depths of several hundred feet.

Methods of obtaining undisturbed soil samples from the floors of trenches or test pits are described in the Appendix on Sampling Methods.

PILOT TUNNELS

The relatively high construction cost of large highway and other vehicular tunnels makes it imperative that subsurface conditions be determined in advance. Geologic surprises encountered during construction can result in delays which may increase total cost by millions or even tens of millions of dollars.

A small pilot tunnel driven in advance of the final highway tunnel is by far the best exploration method presently available but also the costliest. Included within this designation are true pilot tunnels excavated within the final tunnel cross-sectional area as well as those driven close to the final alignment, either at a different elevation or to one side.

All highway pilot tunnels completed to date in the U.S. have been in rock. Because of costly support requirements, the concept of pilot tunneling is not readily adaptable to soft ground conditions. Insofar as we can determine, all past highway pilot tunnels in the U.S. have been driven by conventional drill and blast methods. However, there is no reason why boring machines cannot be used for this purpose in the future, provided the rock can be cut at comparable cost and squeezing ground does not exist.

If a pilot tunnel is decided upon, then the use of other exploration methods is likely to be considerably reduced. The main factor tending to cause favoring of a drill and blast method over the boring machine method for a pilot tunnel is that a drill and blast method is better able to cope with variable tunneling conditions, including passing through difficult ground which may not have been identified in advance.

Pilot tunnels allow a thorough investigation of subsurface properties and conditions first hand, including instrumentation to determine the in situ state of stress which is of vital importance to an understanding of the rock mass stability. Detailed geologic mapping of the pilot tunnel with emphasis on structural discontinuities is imperative. Short core holes can be drilled outward to check geologic conditions at the perimeter of the final tunnel in complex areas where additional information outside the pilot tunnel appears necessary. Abundant samples for any desired laboratory tests are readily available from the pilot tunnel face and walls or from short core holes. All samples taken should be marked to show their orientation. While pilot tunnels presently provide the maximum amount and quality of the vital geologic data required, all problems are not automatically solved. The prediction of stability of the full-size tunnel by extrapolation from stress and stability observation in the pilot tunnel is not an easy matter.

Pilot tunnels are also used to determine hydrologic conditions of the area penetrated. If at all possible, the tunnel should be driven slightly up grade to allow groundwater to flow out by gravity and thus avoid, or reduce, pumping.

If the pilot tunnel is driven within the outline of the final bore, it will reduce the cost of mining the full-size bore. For example:

1. Ground conditions may be improved by advance drainage of saturated zones and by grouting and rock bolting.
2. The pilot tunnel provides a free face for blasting which will greatly reduce explosives requirements if the full size tunnel is completed by drill and blast methods.
3. The pilot tunnel provides excellent access for ventilation and water and power lines.

Pilot tunnels are commonly driven with cross sectional areas in the range of 100 to 175 square feet. Past experience has proven that maximum tunnel advance rates are attained in headings having an equivalent unlined diameter between 10 and 12 feet and that costs per linear foor of tunnel remain about the same up to this size. Above 12 feet, costs begin to increase substantially. Also, the 10- to 12-foot size has adequate cross sectional area for ventilation and other utility lines. Excellent diagrams showing cost breakdowns and average advance rates compiled from 99 tunnels which can be used to determine the economic advisability of the 10- to 12-foot size are presented in Appendix C of the California State Department of Water Resources Bulletin No. 78.[91]

Two highway tunnel projects that utilized pilot tunnels are the Straight Creek project in Colorado and the Cody No. 1 tunnel in Wyoming. The Straight Creek pilot tunnel, driven in 1963-64 approximately 11 feet by 11 feet in size and 8,284 feet long, reportedly cost $1,422,000, or $172 per foot. The Cody No. 1 pilot tunnel, driven prior to 1960 at approximately 8 feet by 8 feet in size and 3,224 feet long, cost $263,000, or $82 per foot.

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[91] "Procedure for Estimating Costs of Tunnel Construction." Appendix C to Bulletin No. 78, Investigation of Alternate Aqueduct Systems to Serve Southern California. State of California Department of Water Resources, September, 1959.

SOIL SHEARING STRENGTH

The in situ testing of the shearing strength for soil is sometimes performed when it is not possible to obtain high quality undisturbed samples for laboratory testing.

A vane method for measuring the shear strength of soils in the field was developed in Sweden in about 1948. In its simplest form, the device consists of a four-winged vane that is fixed to a vertical rod. The vane may be either sunk within a casing, as in sampling, or driven into the ground directly if soil conditions permit. The rod of the vane is rotated from the surface at a rate of about 0.1 degree per second. Both hand-operated and motor-driven devices are available for rotating the rod. However, the hand-applied torque methods are usually unreliable. The rate of torque application is very important. The applied torsional force required to cause a cylindrical surface to be sheared by the vane is measured. This force will be the maximum torque required to rotate the vane during the test. After the maximum torque required is determined, the vane is rapidly rotated for a minimum of 10 revolutions and then the slowly applied torque procedure is repeated. The maximum torque reading then obtained will allow determination of the remolded strength.

The shear strength of the soil is calculated from the expression

$$S = \frac{T-f}{K}$$

where S is the shear strength,

T is the measured maximum torque,

K is a constant depending upon the dimensions and shape of the vane, and

f is the frictional resistance between the torque rods and the ground.

The vane shear test is used only to determine the apparent cohesive strength of the soil on the same basis that it is determined in undrained laboratory shear tests. With the vane shear test, the angle of friction of the soil is assumed to be zero during the test.

An instrument has been developed at Iowa State University that measures both the cohesion and internal friction by testing the wall of a bored hole.[92] This instrument is called the Borehole Direct-Shear Test Device. The instrument consists primarily of an expandable cylindrical head with two opposed, horizontally grooved plates. To run the test, the instrument is inserted into a trimmed borehole and the plates are then expanded hydraulically against the hole walls with the expansion pressure being controlled and noted on a pressure gauge.

The shear head is then pulled vertically until the soil shears. By measuring the pulling force and dividing by the plate size, the soil shearing strength per unit area is obtained. The pulling force is then temporarily relaxed and a higher expansion pressure applied. A second pull is then made to shear the soil. Four or five repetitions of expansion and pull are usually made. The results are plotted and given a linear plot which is the Mohr-Coulomb strength envelope. Figure F-1 shows a typical plot of test results. The intercept of this linear plot with the

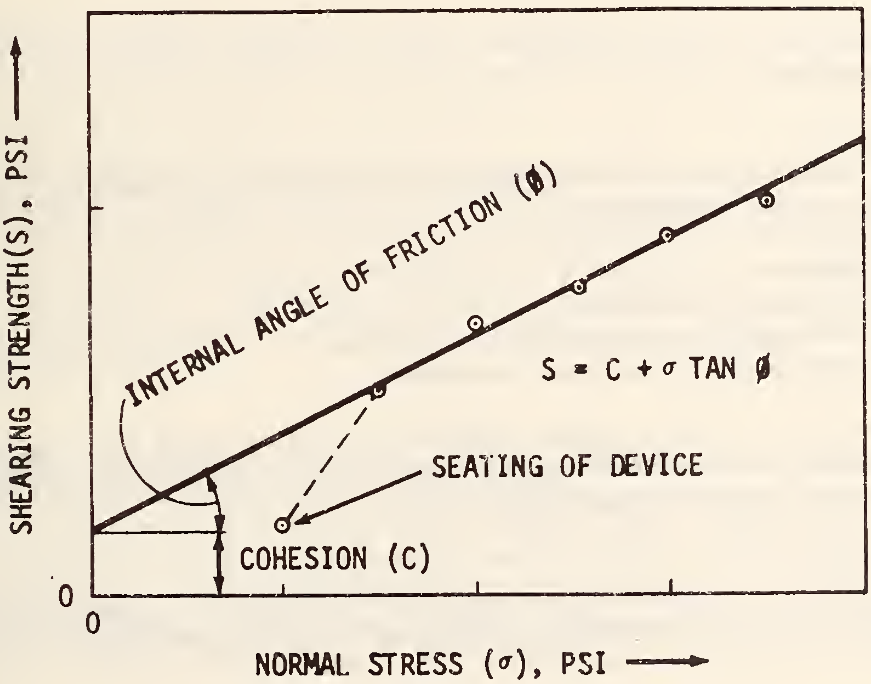

Figure F-1. Typical test results from a Borehole Direct-Shear Device.

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[92] Lohnes, R. A. and R. L. Handy. "Test Method for In Situ Soils". The Military Engineer, No. 393 (January-February, 1968) pp. 30-32.

shearing strength axis will be the cohesion value and the slope of the line will be the internal friction value.

Appendix D of the Corp of Engineers, Engineer Manual EM 1110-2-1907 (31 March 1972), Soil Sampling, describes the detailed procedures for field vane shear tests.

STATE OF STRESS

In situ state of stress is one of the most important subsurface conditions influencing ground stability in highway tunnel construction, but remains one of the most difficult to determine. Tunnel stability is dependent upon the size, shape, and orientation of the opening and the cohesion and strength of the subsurface material in relation to the stress forces acting upon it.

STRESS COMPONENTS

The existing subsurface state of stress is the resultant of several independent stress components:[93]

1. Gravity
2. Thermal gradients
3. Currently active tectonic processes
4. Locked in or residual stresses.

In addition, a secondary stress component results from the penetration of the existing stress field by an exploration hole and emplacement of some type of stress measurement device.

Gravity Components--The stress due to gravity is comprised of vertical and horizontal components which may be computed in the following manner:[93]

$$\sigma_v = \rho g h$$

where σ_v is the vertical stress due to gravity,

ρ is the soil or rock density,

[93] Hoskins, E. R. and E. H. Oshier. "Development of Deep Hole Stress Measurement Device". New Horizons in Rock Mechanics, Proceedings of 14th Symposium on Rock Mechanics (1972). ASCE, New York, 1973. pp. 299-310

g is the gravitational acceleration, and

h is the depth below the ground surface;

and

$$\sigma_H = \sigma_V \left(\frac{\mu}{1-\mu} \right)$$

where σ_H is the horizontal stress due to gravity, and

μ is Poisson's ratio.

Since Poisson's ratio for most rocks is between 0.2 and 0.33, the ratio of horizontal to vertical stress due to gravity should be between 0.25 and 0.5. However, in situ measurements[94] have indicated stress ratios of 0.5 to 0.8 for hard rock, 0.8 to 1.0 for soft rock such as shale and salt, and there have been some cases of ratios greater than 1. Thus this latter equation for the horizontal stress should only be used for estimating the lower limit of the horizontal stress. The actual horizontal stress will usually be greater than indicated by this equation.

Thermal Gradient Component--This component may be calculated if the geothermal gradient and certain physical constants are known.

Active Tectonic Component--Detailed structural mapping of the most recent fault and fracture and joint patterns may give clues to the direction of this component, but its reliable determination can only be obtained from in situ measurements in combination with the residual component. At present it is not possible to separate these two components which together are the most important components as they may cause unusual difficulties.

Residual Component--This component is defined as the stress field remaining in the absence of thermal gradients or external loads. This is compaction, crystallization, or tectonic stress induced in a rock at some point in its geologic history and stored to the present time. Some knowledge of this component may be obtained from geologic mapping of structural features. In situ measurements are used to determine the combined magnitude and direction of residual and active tectonic stresses.

Component Due to Measurement Technique--This secondary component is determined from calculations based on experimental work with particular methods.

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[94] Obert, Leonard and Wilbur I. Duvall. Rock Mechanics and the Design of Structures in Rock. John Wiley & Sons, New York. 1967.

In general, stress conditions found between surface and a depth of 500 feet are moderate, but occasionally areas having high residual stress may be encountered within this depth range.

In situ state of stress is a condition of the rock mass rather than of the rock material; therefore, it must be measured in place. Tunnel route investigations may be required to determine the state of stress under one of two conditions:

1. When no pilot tunnel is driven, measurements can only be made remotely from surface in exploratory boreholes.

2. When a pilot tunnel is driven, instrumentation of that bore will produce the most reliable stress data.

Boreholes--Determination of stress in boreholes drilled from surface to depths of several hundred feet is by far the more difficult of the two situations. The following methods have been used in boreholes for this purpose although the application of some has thus far been restricted to shallow holes:

1. Borehole deformation cell
2. Borehole inclusion stress meter
3. Borehole strain gauge
4. Hydraulic fracturing
5. Seismic wave propagation
6. Observation of core discing.

<u>Borehole Deformation Cell</u>--This technique involves placement in the hole of a tool fitted with strain gauges, followed by overcoring of the section of hole containing the gauges, and then determination of the resulting diametral changes due to relief of the stress. The method is reported to be useful only in shallow holes, less than 40 feet deep. Deformation must be assumed to change linearly with stress reduction.

<u>Borehole Inclusion Stress Meter</u>--Useful only in shallow holes, this technique utilizes inclusion stress meters of either high or low modulus material cemented in a borehole. Using typical plug material such as glass, any changes in the secondary principal stress in the rock will cause a proportional change of stress in the same direction in the inclusion. In some methods the inclusion is observed by polariscope without overcoring, in others observation of stress patterns is made with a polariscope and overcoring for stress relief is performed.

Borehole Strain Gauge--Probably the most widely used and accepted, this technique utilizes one of a variety of types of strain gauge devices attached to either the end or sides of a borehole. Stress relief is obtained by overcoring or drilling a small hole in the center of strain gauge elements which have been bonded to the borehole surface. The strain resulting from the stress relief is measured and used for calculating the stress. This procedure is similar to that of the deformation cell method except overcoring or center drilling is carried out in the same hole without requiring overcoring at a larger hole diameter. This method is also most easily applied to shallow holes but some success has been reported to depths of 200 feet[95] and equipment is being designed for use to depths of 7,500 feet.[96]

Hydraulic Fracturing--Hydraulic fracturing in boreholes can be used to obtain information on stress conditions within drilling depths. Based upon the use of oilfield fracturing techniques, a section of the borehole is isolated at the desired depth with packers and this interval pressured up with fluid until the borehole wall is ruptured. The attitude of the fracture created and the pressure at which the fracture occurred may in some cases provide useful stress information, especially when the drill hole parallels the direction of one of the principal stresses as would be expected when a vertical hole penetrates horizontal sedimentary rocks.

Some experimental laboratory work has been done and numerous references on oilfield fracturing are available, but thus far this is not a well established technique for determining stress in rocks being investigated for highway tunnels.

Seismic Wave Propagation--This method involves the measurement of the velocity of longitudinal seismic waves traveling between two points within an in situ rock mass and comparing this with the velocity-stress relationship determined for the same rock on intact specimens in the laboratory. The method would appear to have some value for rocks which exhibit increased velocities at higher stresses, but would be of no use in cases where the velocity-stress relationship remains nearly constant over a wide range of stress conditions.

One complication with this method arises from the fact that laboratory velocity-stress relationships are determined on intact samples

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[95]Fairhurst, C. Methods of Determining In-Situ Rock Stresses At Great Depths. Technical Report No. 1-68, prepared for the Missouri River Div., U.S. Army Corps of Engineers. December, 1967.

[96]Hoskins, E. R. and E. H. Oshier. "Development of Deep Hole Stress Measurement Device". New Horizons in Rock Mechanics, Proceedings of 14th Symposium of Rock Mechanics (1972). ASCE, New York. 1973 pp. 299-310.

whereas the in situ velocity is slowed by varying degrees due to varying degrees of jointing and fracturing.

Observation of Core Discing--Drill cores cut from rock which is under greater than normal stress may split into thin discs, often having axial dimensions less than the core diameter. The stress required to cause discing in a particular rock may be determined by triaxial laboratory test methods.[97] The occurrence of core discing thus serves as a warning indicator of high stress conditions. While it is not possible to determine the exact stress, at least a minimum value may be determined; that is, the existing stress is known to be equal to or greater than the minimum stress required to cause discing.

Pilot Tunnel--The stress field may be far more easily examined from within as is the case when one has access into the rock via a pilot tunnel. Strain and rock load measurements taken in a pilot tunnel enable the investigator to better predict the conditions which will be in effect when the full sized bore is excavated.

The in situ stress field is disrupted by the pilot tunnel excavation. Therefore strain readings taken from within boreholes drilled several feet into the tunnel walls produce more useful and reliable stress information than readings from techniques performed on the tunnel walls or at depths only a foot or two into the rock. The walls of a pilot tunnel driven with a tunneling machine will be much less disturbed than those in a drill and blast tunnel.

Instruments used for stress determinations in pilot tunnels may be classified[98] as:

1. Rock surface instruments
   a. Strain rosettes
   b. Optical strain gauges

2. Shallow emplaced instruments
   a. Flat jacks
   b. Strain relief slots
   c. Borehole deformation cells

3. Deep emplaced instruments - Borehole Extensometers

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[97] Obert, Leonard and Wilbur I. Duvall. Rock Mechanics and the Design of Structures in Rock. John Wiley & Sons, New York. 1967.

[98] Hartmann, Burt E. Rock Mechanics Instrumentation for Tunnel Construction. Terrametrics, Inc., Golden, Colorado. 1967.

Rock surface instruments determine stress condition at the surface which has been disrupted by the excavation process and therefore produce data of limited usefulness.

Strain Rosettes--These strain measuring devices measure changes in strain in three or more directions simultaneously. Stress on the test area is relieved by overcoring or drilling a surrounding ring of holes.

Optical Strain Gauges--These low cost photoelastic patches, when bonded to the rock surface, give some indication of changes in stress condition on the rock surface.

Shallow emplaced instruments measure stress generally within 5 feet of the rock surface which has been destressed somewhat by excavation, so do not provide true information on the undisturbed in situ stress condition.

Flat Jacks--Flat jack instruments are flat, rectangular, or circular hollow steel devices which are grouted into slots cut normal to the rock surface. The jack is expanded hydraulically to force the sides of the slot apart to the positions they occupied before the slot was cut. The pressure required to restore the slot to its original position is assumed to be equivalent to the residual stress.

Strain Relief Slots--Strain measurements are taken across slots cut by drilling a series of connecting holes.

Borehole Deformation Cells--This stress relief overcoring technique which produces measurable diametral strain changes was described above in the group of methods used from the surface. As this method is sometimes applied at depths over 5 feet from the tunnel wall it could also be included in the next category of deep emplaced instruments.

Deep emplaced instruments provide the best information now obtainable for predicting the rock mass behavior in the full size tunnel.

Borehole Extensometers--Both single and multiple position instruments may be used to indicate strain occurring between fixed points in boreholes drilled in the pilot tunnel roof and walls. Valuable data is obtained by monitoring these instruments at periodic time intervals and also recording the distance between the instrument stations and the advancing tunnel face at the times of reading. The size and shape of destressed zones and of zones of tension or compression may be outlined with confidence. These instruments may be used in boreholes up to several hundred feet long but are usually used within a range of a few tens of feet from the pilot tunnel roof or walls.

Values Obtained--While various methods are available for determining in situ stress from the surface or from pilot tunnels, this field is far from being an exact science. Since values obtained by one method often vary considerably from values obtained by other methods, the investigator must realize that even the most carefully determined data must be considered as crudely approximate. Stress data, while only approximate, is nonetheless quite valuable to highway tunnel designers and a concerted effort should always be made to arrive at a good understanding of existing stress conditions before commencing an underground project as costly as a highway tunnel.

GROUNDWATER HYDROLOGY

Thorough knowledge of the groundwater conditions is necessary for intelligent engineering design and subsequent construction of a tunnel. Excessive groundwater inflows constitute one of the major problems often encountered in tunneling operations. Besides pumping problems, inflowing water often intensifies stability problems with slopes at portal areas and in incompetent ground within the tunnel. This is due to lubrication of discontinuities, erosion, or decomposition which cause acceleration of ground movement into the excavated workings.

Determination of the existence and magnitude of water problems prior to the excavation investigation is often very difficult, especially in highly fractured or complex geologic areas. Nevertheless, a thorough effort must be made to determine subsurface hydrologic conditions in advance. As is the case with other major tunneling problems, the cost consequence is much greater when the problem occurs unexpectedly than when it has been anticipated.

INVESTIGATION

Hydrologic study of a proposed tunnel site should be started early in the investigation and continued throughout. During the preliminary stage, all literature on the local groundwater situation should be obtained, pertinent state and federal agencies contacted, topographic and geologic maps and air photographs studied, and field reconnaissance carried out to determine climatic conditions, drainage patterns, and subsurface water conditions. All existing wells should be located and if possible their water levels determined and monitored over as long a period as possible. Selected pumping tests should be performed in these wells and in exploratory holes drilled during the investigation. An extensive body of literature (including many U.S. Geological Survey Water Supply Papers, several U.S. Bureau of Reclamation reports, many articles in various technical publications, and textbooks) is available on various pumping test methods and the interpretation of results.

Quantitative estimates may be possible for primary permeability and yield of sedimentary soil and rock aquifers, but generally secondary fracture type permeability in rock is quite irregular and unpredictable and therefore does not lend itself to ready determination by standard groundwater calculation methods.

Loss or gain of drilling fluids should be noted and recorded during exploratory core drilling. Thorough geologic logging of core recovered may reveal the fracture patterns causing secondary permeability. One of the most valuable tests for obtaining information about the permeability of rock units intersected by drill holes is the formation pressure test, or pump-in test. Usually a straddle packer assembly is used to isolate a particular section over some desired length of the drill hole and then water is pumped into the rock at constant or different pressures for timed intervals. Differences in flow rates at different pressures may allow an approximation to be made of the groundwater pressure which is being overcome.

Groundwater at the tunnel level approaches the hydrostatic head pressure of 0.433 psi per foot-of-depth below the surface of any body of groundwater having continuous vertical permeability throughout.

To date, most exploratory drilling has been vertical from the ground surface. In situ hydrologic tests in these holes may or may not be representative of all the intervening ground between them. The use of a horizontal exploratory hole along the tunnel alignment in conjunction with some vertical holes to improve the three-dimensional picture would give extremely valuable continuous information on hydrologic conditions and would even provide some opportunity for advance remedial action by grouting. Exploration and grouting with long horizontal holes to one-mile lengths are being performed in a long undersea railway tunnel in Japan.

PRINCIPLES

Groundwater hydrology or geohydrology is the science of evaluating the distribution, mobility, and quality of groundwater. The principles of groundwater hydraulics govern the movement of water through permeable soil and rock. Scientific in situ analysis of groundwater hydraulics is made by conducting pumping tests of wells and applying equations which have been derived from particular boundary conditions. The principal properties of soil and rock affecting groundwater hydraulics are porosity and permeability.

Porosity of soil or rock is the ratio of the volume of contained void spaces to the total volume of the mass, expressed either as a decimal fraction or percentage. It may be subdivided into:

1. Primary porosity, the void space created when the soil or rock formed in its present state.
2. Secondary porosity, the void space created by joints, faults solution holes, and openings along bedding or foliation planes.

Permeability of soil or rock is a measure of its ability to transmit fluid, such as water, under a pressure gradient. Likewise it may be expressed as either:

1. Primary permeability, that created during formation of the material.
2. Secondary permeability, that due to fracturing, faulting, folding, or solution.

Basic equations for evaluating the flow rates and amounts of groundwater available for flow into tunnel workings include those for determining the coefficient of permeability, intrinsic permeability, hydraulic conductivity, transmissivity, specific yield, specific retention, and storage coefficient.

Permeability has been commonly expressed in terms of the coefficient of permeability which is one item in Darcy's equation for flow of fluids through porous media:

$$Q = \frac{kAh}{L}$$

where Q is the quantity of water flowing,

k is the coefficient of permeability,

A is the cross sectional area involved, and

$\frac{h}{L}$ is the hydraulic gradient (ratio of head loss by friction, h, to distance, L, in the direction of flow).

The coefficient of permeability is often expressed in terms of the Darcy unit which gives the flow in cubic centimeters per second of a fluid having unit viscosity, through a cross sectional area of one square centimeter under a pressure gradient of one atmosphere per centimeter. More commonly used in the petroleum industry is the term millidarcy which is 0.001 Darcy.

Groundwater investigators often prefer to express the coefficient in terms of the Meinger unit, given in gallons per day per square foot of cross sectional area. Thus in the above form of Darcy's equation,

Q is given in gallons of water per day at $60^{\circ}F$,

A is in square feet,

h is in feet of water, and

L is in feet.

Unconsolidated surficial deposits such as stream or glacial sediments may exhibit Meinger primary permeabilities of up to 20 gallons per day per square foot.[99]

Primary permeability is usually of minor significance in consolidated rock with the exceptions of some lava flows and very permeable sandstone aquifers which may permit serious water inflows into tunnel workings. Primary permeabilities are generally less than 0.1 gallons per day per square foot in crystalline igneous and metamorphic rocks and fine-grained sedimentary rocks such as shales and mudstone.[100] Significant permeabilities of greater than 1 to 10 gallons per day per square foot are uncommon.

Primary permeability may be determined by both laboratory (see Appendix G) and in situ well pressure pumping tests.

Secondary permeability, mainly a property of fractured rock rather than soil, may be very high and is the principal cause of severe water problems in rock tunnels. This property must be determined by in situ well-pressure pumping tests as laboratory test methods on intact specimens are of no value for this purpose.

Because permeability is a property of the medium independent of the properties of the fluid, the U.S. Geological Survey (USGS) is adopting the term "intrinsic permeability" which is determined as follows:[101]

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[99] Goodman, R.E., D. G. Moye, A. Van Schalkwyk, and I. Javandel. "Ground Water Inflows During Tunnel Driving." Engineering Geology, Vol. 2 No.1. 1965. pp. 39-56.

[100] Wahlstrom, Ernest E. Tunneling in Rock. Elsevier Scientific Publishing Company, Amsterdam. 1973.

[101] Lohman, S.W. Ground-Water Hydraulics. U.S. Geological Survey Professional Paper 708. 1972.

$$k = -\frac{q\nu}{g\left(\frac{dh}{dl}\right)} = -\frac{q\nu}{\frac{de}{dl}}$$

where k is the intrinsic permeability in square meters or square micrometers,

q is the rate of flow per unit area or $\frac{Q}{A}$ in meters per second,

ν is the kinematic viscosity in square meters per second,

g is the acceleration of gravity,

$\frac{dh}{dl}$ is the gradient or unit change in head per unit length of flow with h and l in meters, and

$\frac{de}{dl}$ is the potential gradient or unit change in potential per unit length of flow with e measured in Joules per kilogram and l in meters.

In addition, the USGS is adopting the term hydraulic conductivity in consistent units to replace the coefficient of permeability which was expressed in inconsistent units of gallons per day per square foot. A medium has a hydraulic conductivity of unit length per unit time when it transmits a unit volume of groundwater at the prevailing viscosity through a unit corss-sectional area, measured at right angles to the direction of flow, in unit time and under a unit hydraulic gradient. Thus, hydraulic conductivity is expressed by:

$$K = -\frac{q}{\frac{dh}{dl}}$$ in either feet per day or meters per day

where q and $\frac{dh}{dl}$ are as defined above.

Transmissivity is the rate at which water of the prevailing kinematic viscosity is transmitted through a unit width of the aquifer under a unit hydraulic gradient.[102] This quantity is expressed as:

$$T = Kb$$

where T is the transmissivity in square feet per day or square meters per day,

K is as devined above, and

b is the thickness of the aquifer in feet or meters.

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[102] Lohman, S.W. Ground-Water Hydraulics. U.S. Geological Survey Professional Paper 708. 1972.

Another term frequently used in quantitative groundwater study is specific yield which is the water yielded from water bearing material by gravity drainage. Meinzer[103] has defined specific yield or soil or rock as the ratio of the volume of water which, after being saturated, it will yield by gravity to its own volume. The expression is:

$$S_y = \frac{v_g}{V}$$

where S_y is the specific yield as a decimal fraction,

v_g is the volume of water drained by gravity, and

V is the total volume.

Specific retention of soil or rock is defined as the ratio of the volume of water which, after being saturated, it will retain against the pull of gravity to its own volume.[104] This quantity is determined by:

$$S_r = \frac{v_r}{V} = \eta - S_y$$

where S_r is the specific retention as a decimal fraction,

v_r is the volume of water retained against gravity,

η is the porosity as a decimal fraction, and

V and S_y are as defined above.

Storage coefficient is the volume of water an aquifer is capable of releasing from or taking into storage per unit surface area of the aquifer per unit change in head. Theiss[115] produced the following equation for confined aquifers:

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[103] Meinzer, O. E. Outline of Ground-Water Hydrology, with Definitions. U.S. Geological Survey Water-Supply Paper 494. 1923.

[104] Meinzer, O. E. Outline of Ground-Water Hydrology, with Definitions. U.S. Geological Survey Water-Supply Paper 494. 1923.

[105] Theiss, C. V. "The Significance and Nature of the Cone of Depression in Ground-Water Bodies." Economic Geology, Vol. 33 No. 8. 1938. pp. 889-902

$$s = \frac{Q}{4\pi T} \int_{\frac{r^2 S}{4Tt}}^{\infty} \left(\frac{e^{-u}}{u}\right) du$$

where s is the drawdown,

Q is the constant discharge rate from a well,

T is the transmissivity,

r is the distance from the discharging well to the point of observation,

S is the storage coefficient,

t is the elapsed time since discharge began, and

u is the variable of integration.

The storage coefficient of unconfined aquifers is about equal to the specific yield, generally ranging from 0.1 to 0.3, but that on confined aquifers is much smaller, ranging from 0.00001 to 0.001.[106]

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[106]Lohman, S.W. Ground-Water Hydraulics. U. S. Geological Survey Professional Paper 708. 1972.

Appendix G

Laboratory Testing

A knowledge of the properties of rock and soil is required for an adequate design of excavated openings. Rock and soil composition and character are often not homogeneous over substantial areas - they vary horizontally and vertically and contain a variety of defects or discontinutities such as faults, fractures, and geologic contacts. It would be ideal to conduct test work in situ on a large scale so that the characteristics and behaviors measured would be truly representative of a large mass equal to or greater than the size of underground openings desired. Thus far only a limited number of in situ tests have been developed so the major amount of testing is carried out in the laboratory on small core and other rock samples which are commonly intact specimens, free of the above mentioned discontinuities. Tests readily performed on these specimens yield a great amount of valuable information, but it is difficult to translate this data from defect-free material to the in situ conditions of the larger scale on which excavations are made. In situ tests are usually conducted within tunnels and other underground openings; very few are carried out from surface during tunnel site investigations.

Since rock mechanics is a relatively new field in which productive research is continually being advanced by a variety of organizations, very little standardization of laboratory test procedures has been accomplished in the United States. Additional research and eventual evolvement of recognized standard test methods is necessary so that results of widely separated projects can be reported on a common basis and the experience of past projects can best be applied to future problems.

LABORATORY TESTING OF SOILS

GRADATION

Soil particles vary widely in their shape. There are two major shape distinctions generally recognized to be important in engineering. One is where the three dimensions of a particle are of the same order of magnitude while the other shape configuration is platelike. The latter shape is characteristic of the clays which are usually the more unstable. Those particles which have a bulky shape (all three dimensions of the same order of magnitude) will primarily be the gravels, sands, and silts. The shape and structure of those particles larger than about 0.05 millimeters in diameter can be distinguished with the naked eye. For sizes smaller than 0.05 millimeters, a microscopic examination is necessary. If a microscopic study is not possible, then it is necessary to make deductions on the particle shapes and soil structure based on knowledge of other soil characteristics.

It is seldom of practical interest in engineering to make precise measurements of the diameters of individual particles in a soil mass. Rather, it is usually sufficient to determine the relative amounts of material that contain particles either within certain size limits or with particles larger or smaller than one particular size of special significance.

The particle size distribution of coarse-grained soils is determined directly by a sieve analysis, while that for fine-grained soils is determined indirectly by hydrometer analysis, and that for mixed soils is determined by a combined sieve and hydrometer analysis.

A sieve analysis consists of passing a sample of the soil through a set of sieves which are shaken in a prescribed manner, and then weighing the amount of material that is retained on each sieve; sieves are wire screens having openings of standard sizes. The percentage of material by weight retained on each of the sieves is computed from:

$$\text{Percent retained} = \frac{\text{weight retained on sieve x 100}}{\text{total weight of the sample}}$$

The hydrometer analysis is based on Stokes' law which relates the terminal velocity of a sphere falling freely through a fluid to its diameter. It is assumed that Stokes' law can be applied to a mass of dispersed soil particles of various sizes and shapes. The hydrometer is used to determine the percentage of dispersed soil particles remaining in suspension at a given time. The maximum grain size equivalent to a spherical particle is computed for each hydrometer reading using Stokes' law. A dispersing agent is used to prevent flocculation of the fine size soil particles. A nomograph is usually used to solve Stokes' equation for the particle diameter corresponding to a particular hydrometer reading. To compute the percent of particle diameters smaller than that corresponding to a given hydrometer reading, the following formulas are used:

For a hydrometer calibrated in specific gravity,

$$\begin{matrix}\text{Percent finer}\\ \text{by weight}\end{matrix} = \frac{G_s\,(h_s - C_d + C_t) \text{ x } 100}{W_s\,(G_s - 1)}$$

For a hydrometer calibrated in grams per liter,

$$\text{Percent finer by weight} = \frac{(h_s - C_d + C_t) \times 100}{W_s}$$

where G_s is the specific gravity of solids,

W_s is the oven dry weight of the soil sample, and

$h_s - C_d + C_t$ is the calibrated hydrometer reading minus dispersing agent correction plus temperature correction algebraically.

A combined analysis is used for materials containing particles finer than 200 mesh when the size analysis of the -200 mesh fraction is of interest. A sieve analysis is used on the +200 mesh material and a hydrometer test is made on the -200 mesh material.

The data obtained from the sieve analysis and hydrometer tests are presented in either a tabular or graphical form with the latter being the one most often used. The graphical presentation shows the gradation of the material and is made on semi-logarithmic paper. The particle diameters are plotted on the logarithmic scale and the percent finer by weight fractions are plotted on the arithmetic scale.

CONSISTENCY

Consistency is a soil condition which indicates the degree of firmness for a fine-grained soil. The Atterberg limits are used to define the separate states of consistency - the liquid, the plastic, the semisolid, and the solid states. These limits define the change in strength of a fine-grained soil with changes in its moisture content and are expressed as the water content of the soil as it passes from one state of consistency to another.

When a soil drying from the liquid state reaches a point at which it ceases behaving as a liquid and begins to acquire the properties of a plastic, it is said to have reached its liquid limit. As the soil is further dried, it reaches the point at which it begins behaving as a semisolid rather than as a plastic and it is then said to have reached its plastic limit. Eventually as the soil continues to lose moisture it reaches the point at which shrinkage ceases and this point is called the shrinkage limit. The consistency indexes are the differences between specified limits. The plasticity index indicates the range of water contents within which the soil has plastic properties and it is the difference between the liquid and plastic limits. The shrinkage index, which is not used as generally as the plasticity index, is the difference between the plastic and shrinkage limits.

In practice, only the finer soils or fine fractions of coarser soils are described by their consistency. These soils are those whose condition is markedly affected by changes in moisture content such as the clays and silts. The addition of water to these soils increases the thickness of the film of absorbed water covering each soil particle. Increasing the water film thickness permits the individual particles to slide past one another more readily.

The consistency limits tests are made only on minus 40 mesh material. The soil specimens used for liquid and plastic limit determinations are not dried prior to testing because drying may alter the soil by causing the particles to subdivide or agglomerate, by driving off absorbed moisture which is not regained upon rewetting, or by effecting a chemical change in any organic material contained in the soil. These effects can significantly change the limits, especially the liquid limit. Distilled water is used in the tests to minimize any possibility of ion exchange between the soil and any impurities present in water.

The liquid limit of a soil is defined as the water content, expressed as a percentage of the weight of the oven-dried soil, at which two halves of a standard soil cake separated by a groove of standard dimensions will flow together for a distance of 1/2 inch when the cup containing the soil cake is dropped 25 times for a distance of 1 centimeter at the rate of 2 drops per second. The groove is closed by a flow of the soil and not by slippage between the soil and the cup.

A standard liquid limit device and grooving tool are used in making the liquid limit tests. A soil specimen is thoroughly mixed with water to form a uniform paste. A portion of this soil paste is then placed into the cup of the liquid limit device and leveled off to a depth of about 1 centimeter. The soil cake thus formed is then divided in half by use of the grooving tool until a clean, sharp groove of the proper dimension is formed. The cup is connected to the device and the number of blows required to close the groove for half an inch is noted. The water content of this sample is then calculated as described in the section on "Moisture Content". The remaining portion of the soil sample is dried somewhat to reduce its water content, another sample is taken, and a liquid limit test run. This procedure is repeated three or four times. For each test, a different number of blows are required to close the groove. A plot of the water content on an arithmetic scale versus the number of blows on a logarithmic scale is then made. This is called the flow line and should approximate a straight line. The liquid limit is then the intersection of this line and the 25-blow line on the graph.

A simplified procedure called the one-point method for determining the liquid limit has been investigated.[107] This method is based on the experience that the slope of the liquid limit flow lines for soils within a given geological environment is essentially a constant. The liquid limit can be determined from just one individual test provided the slope constant has been previously established. In the one-point liquid limit test, the liquid limit is calculated from the expression

$$\text{Liquid Limit} = W_N \left(\frac{N}{25}\right)^{\tan\beta}$$

where W_N is the water content of the soil in which N number of blows closes the groove and

$\tan\beta$ is the slope of the flow line.

The value of 0.121 for $\tan\beta$ is generally used, but it may not be correct in many cases.

The plastic limit of a soil is defined as the water content, expressed as a percentage of the weight of the oven-dried soil, at which the soil begins to crumble when rolled into a 1/8-inch diameter thread. This is done by taking an ellipsoidal-shaped sample of the soil and rolling it between the root of the fingers and a flat smooth surface at a rate of between 80 and 90 strokes per minute. When the diameter of the thread being formed become 1/8 of an inch without crumbling the thread is folded and kneaded into an ellipsoid again and the rolling process repeated. This procedure is continued until the soil has dried to the point where the 1/8-inch diameter thread will break into numerous pieces. The water content of the sample at this point is the plastic limit.

The shrinkage limit finds considerable application in testing water-sensitive, expandable clays. The shrinkage limit of a soil is defined as the water content, expressed as a percentage of the weight of the oven-dried soil, at which a further loss in moisture will not cause a volume decrease. As part of the shrinkage limit test, the shrinkage ratio and linear shrinkage are also usually determined. The shrinkage ratio is the ratio between a given volume change and the corresponding change in water content above the shrinkage limit. The linear shrinkage is the decrease in one dimension of a soil mass, expressed as a percentage of the original dimension, when the water content is reduced from a given value to the shrinkage limit.

For the shrinkage tests, a soil specimen is mixed with sufficient water to fill the voids and to allow the soil to be readily worked into the shrinkage dish without any air bubbles being present. The inside surface of the shrinkage dish is coated with petroleum jelly to prevent sticking of the soil and it is then filled with the prepared soil specimen and struck off even with the rim of the dish. The filled dish is weighed and the soil pat is allowed to air-dry until a definite color change occurs.

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[107] Dept. of the Army, Corp. of Engineers. Laboratory Soils Testing. Engineering Manual EM1110-2-1906. November 30, 1970.

it is then oven-dried to a constant weight. The volume of the shrinkage dish is measured by determining the amount of mercury it will hold. This volume is also the volume of the wet soil pat. The volume of the oven-dried soil pat is determined from its displacement in mercury. Having determined the various weights and volumes, the following calculations are made:

$$\text{Shrinkage Limit} = SL = \left(\frac{W_1 - W_2 - V + V_s}{W_2 - W_c} \right) \times 100$$

$$\text{Shrinkage Ratio} = SR = \frac{W_2 - W_c}{V_s}$$

$$\text{Linear Shrinkage} = L_s \left[1 - \sqrt[3]{\frac{100}{R(w - SL) + 100}} \right] \times 100$$

where SL is the shrinkage limit,

W_1 is the weight of the shrinkage dish and wet soil pat,

W_2 is the weight of the shrinkage dish and oven-dried soil pat,

W_c is the weight of the shrinkage dish,

V is the volume of the wet soil pat,

V_s is the volume of the oven-dried soil pat,

SR is the shrinkage ratio,

L_s is the linear shrinkage, and

w is the given water content value.

About 1948, the stiffening limit concept was introduced in Norway for extrasensitive clays.[108] Such clays are apparently stable at water contents greater than the conventional liquid limit, but they can be suddenly liquified by factors which are not well understood. The stiffening limit as determined in a laboratory is defined as the minimum water content at which a thoroughly stirred clay suspension still flows under its own weight in a standard 11-millimeter diameter test tube after exactly 1 minute of rest.

Some possible errors that could cause an inaccurate determination of the consistency indexes are:

1. The specimens are not representative.
2. The specimens are improperly prepared.
3. The water contents inaccurately determined.
4. The testing equipment is improperly constructed or adjusted.
5. Incorrect test procedures are followed.

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[108] Krynine, Dimitri P. and William R. Judd. Principles of Engineering Geology and Geotechnics. McGraw-Hill, New York. 1957.

ASTM Standards D423-66, D424-59 (1971), and D427-61 (1967) along with Appendixes III, III A, and III B of the Corp of Engineers, Engineer Manual EM 1110-2-1906 (30 November 1970), Laboratory Soils Testing, give complete descriptions of the procedures to be followed in determining the various consistency limits for soil.

MOISTURE CONTENT

Knowledge of the degree of saturation for a soil is extremely useful because this characteristic influences such soil properties as permeability, shear strength, and compressibility.

The moisture content of soil may be expressed in two different ways. One is on a weight basis which is commonly known as the water content. The other is on a volume basis which is called the degree of saturation.

The water content of a soil is defined as the ratio, expressed as a percentage, of the weight of water in a given mass of the soil to the weight of the solid particles in the same soil mass. The procedure used for calculating the water content is to determine the weight of the water that is removed from a given moist soil mass by drying it to a constant weight in an oven controlled at 110 ± 5° C (230 ± 9° F). The weight of the soil remaining after oven drying is then used as the weight of the solid particles. The water content in percent is then calculated from

$$w = \left(\frac{W_1 - W_2}{W_2 - W_c}\right) \times 100 = \frac{W_w \times 100}{W_s}$$

where w is the water content in percent,

W_1 is the weight of the container and moist soil,

W_2 is the weight of the container and oven-dried soil,

W_c is the weight of the container,

W_w is the weight of water in the soil mass, and

W_s is the weight of the soil particles in the same soil mass.

The amount of material to be used in the water content determination will usually depend on the maximum size of particles, the amount of material available, and the requirement that the specimen be representative of the material. Some possible errors that could cause an inaccurate water content determination are:

1. The specimen is not representative.

2. The specimen is too small. As a rule, the larger the specimen the more accurate the determination because of the larger weights involved.
3. Moisture lost before weighing the wet specimen.
4. Incorrect oven temperature or a variation of temperatures within the oven.
5. Specimen removed from the oven before a constant oven dried weight is obtained.
6. Moisture gain before weighing the oven-dried specimen.
7. Weighing the oven-dried specimen before it has cooled.
8. Using an incorrect container weight.

The degree of saturation of a soil is defined as the ratio expressed as a percentage of the volume of water in a given mass of the soil to the volume of voids in the same soil mass. In order to calculate the degree of saturation, the weight and volume of the wet soil specimen, the weight of the same specimen after oven drying, and the specific gravity of the solids must be known. The weights needed are determined in the same manner as for determining the water content of the soil. The volume of the wet specimen is determined either by linear measurement (the volumetric method) or by measurement of the volume, or weight, of the water displaced by the specimen (the displacement method).

The volumetric method for determining the volume of the soil specimen consists of computing the total volume from linear measurements of a regularly shaped mass. One method of doing this is by using a calibrated ring-shaped specimen cutter to obtain a cylindrical specimen. The specimen cutter is pushed into the soil sample, which is progressively trimmed in advance of the cutter, until a test specimen is obtained. By knowing the volumetric volume of the specimen cutter, the volume of the sample is known.

The displacement method for determining the volume of the soil specimen consists in measuring the volume or weight, and then calculating the volume of water displaced by the soil specimen. This method is particularly adaptable to irregularly shaped specimens. The sample is wax coated during its submergence in water to prevent additional moisture being added to the specimen. The volume of the wax coating must be accounted for in the calculations for determining the volume of the wet specimen.

The specific gravity of a soil is defined as the ratio of the weight in air of a given volume of solid soil particles at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature. A representative soil sample is mixed with water to form a slurry which is then placed into a flask. The flask is then evacuated of air and filled with water to a calibrated volume and the filled flask is weighed. Having previously determined the weight of the flask containing the calibrated volume of plain water and after determining the weight of the dry soil particles, the specific gravity of solids may be calculated from

$$G_s = \frac{KW_s}{W_s + W_3 - W_4}$$

where G_s is the specific gravity of solids,

K is a correction factor based on the density of water at 20° C (68° F),

W_s is the weight of the soil particles,

W_3 is the weight of the flask plus water at the test temperature, and

W_4 is the weight of the flask plus water plus soil particles at the test temperature.

Having determined the weight and volume of the wet soil specimen, the weight of the same specimen after oven drying, and the specific gravity of the solids, the degree of saturation can be calculated from

$$S = \frac{G_s \left(W_t - W_s\right) \times 100}{62.4\ G_s V_t - W_s}$$

where S is the degree of saturation,

W_t is the weight of the wet specimen,

W_s is the oven-dried weight of the specimen,

G_s is the specific gravity of solids, and

V_t is the volume of the wet specimen.

Some possible errors that could cause inaccurate saturation determinations, in addition to those previously given for water content determination are:

1. For volume determination
   a. Imprecise measurements
   b. Voids on the specimen formed by excess trimming
   c. Voids on surface of the specimen not filled by wax or air bubbles formed beneath the wax

2. For specific gravity determination
   a. Imprecise weighings
   b. The temperature of the flask and contents not being uniform
   c. Incomplete removal of the entrapped air from the soil suspension

d. A dirty flask or excess moisture on the outside or inside of the flask
e. The water meniscus not being coincidental with the flask calibration mark
f. The use of water containing dissolved solids.

ASTM Standard D2216-71 and Appendix I of the Corp of Engineers, Engineer Manual EM 1110-2-1906 (30 November 1970), Laboratory Soils Testing, gives complete descriptions of the procedures to be followed in determining the moisture content of soils. Appendix II of EM 1110-2-1906 describes the volumetric displacement methods for determining the volume of a wet specimen as well as the procedure for calculating the degree of saturation. ASTM Standard D854-58 (1972) and Appendix IV of EM 1110-2-1906 describe the determination of the specific gravity of solids for a soil.

UNIT WEIGHT AND POROSITY

The unit weight and porosity (or void ratio) measurement of a soil are commonly used in conjunction with engineering design. There are three different unit weights that are commonly used:

1. The unit weight of solids
2. The unit dry weight of soil
3. The unit wet weight of soil

The porosity and void ratio are indications of the soil density in terms of relative volumes and although they are not equivalent, they perform similar functions. These various terms are defined as follows:

The unit weight of solids is the weight of the solid particles per unit volume of the solid particles in the same soil mass.

The unit dry weight of soil is the weight of the soil particles per unit of total volume of soil mass.

The unit net weight of soil is the weight of the wet or moist soil mass per unit of total volume of soil mass.

The porosity is the ratio, expressed as a percentage, of the volume of voids to the total volume of the soil mass.

The void ratio is the ratio of the volume of voids to the volume of solid particles in a given soil mass.

The quantities that must be known to calculate the above terms are the weight and volume of the wet specimen, the weight of the same specimen after oven drying, and the specific gravity of the solids. These quantities are determined as described in the section on "Moisture Content." Once these quantities have been determined, then the following formulas are used.

$$\gamma_s = \frac{W_s}{V_s} = 62.4G_s$$

$$\gamma_d = \frac{W_s}{V_t}$$

$$\gamma_m = \frac{W_t}{V_t}$$

$$e = \frac{V_v}{V_s} = \frac{62.4G_sV_t - W_s}{W_s} = \frac{\gamma_s - \gamma_d}{\gamma_d} = \frac{n}{1 - n}$$

$$\eta = \frac{V_v}{V_t} = \frac{62.4G_sV_t - W_s}{62.4G_vV_t} = \frac{\gamma_s - \gamma_d}{\gamma_s} = \frac{e}{1 + e}$$

where γ_s is the unit weight of solids,

W_s is the weight of the solid particles,

V_s is the volume of the solid particles,

G_s is the specific gravity of solids,

γ_d is the unit dry weight of the soil mass,

V_t is the total volume of the soil mass,

γ_m is the unit wet weight of the soil mass,

W_t is the weight of the wet or moist soil mass,

e is the void ratio

V_v is the volume of voids, and

η is the porosity.

SHEARING STRENGTH

The shearing strength of soil is often the principal factor governing the behavior of soil under loading. The stability of soil in cuts, openings, and embankment slopes depends directly on its shearing strength or resistance to sliding. The shearing strength of soils is variable within one particular soil as well as from one soil to another. It is often necessary to not only determine a soil's shearing strength for a given set of conditions, but also to be able to identify those factors which may cause variations in the soil strength.

Shear strength is defined as the resistance to deformation by continuous shear displacements of material particles or masses caused by the action of a tengential stress. The Coulomb Formula is used to show the state of shear equilibrium in a soil mass. This formula can be stated in general terms as:

$$S = C + (P) Tan\, \phi$$

where S is the maximum shear stress or shearing strength

C is the cohesion

P is the stress normal to the surface of failure

ϕ is the angle of internal friction

This equation is usually applied to the surface in the soil on which failure is thought likely. Then the stress P is the stress normal to that surface and S is the maximum shearing stress which can be sustained. The normal stress P may be either the total or the effective stress depending upon the considerations of the test and its application. The values of C and ϕ are chosen appropriate to the initial consolidation state of the soil and the drainage conditions under which it is sheared.

One of the most important hypotheses in soil mechanics is that the deformation of the soil structure is controlled by the effective stress. This effective stress can not be measured directly. However, it is equivalent to the difference between the total applied normal stress and the hydrostatic pressure of the pore water, both of which are real and measurable in many circumstances. The effective stress thus represents that portion of the total stress which is absorbed or balanced by the soil structure.

The major part of the shear resistance developed by cohesionless materials is due to friction between the solid particles on either side of the slip plane. The coefficient of friction between two solids is approximately a constant which depends upon the physical characteristics of the solid surfaces as affected by surface roughness and the composition of the solids. During shear there will be structural changes in the surfaces which thus change the shearing strength along the failure plane.

In a cohesive soil, the values of C and ϕ will be relatively constant with respect to effective stresses and under drained conditions. However, these values will not be constant for undrained conditions.

Instead they will depend upon the shearing rate, drainage opportunities, changes in consistency due to consolidation, structural disturbance, and many other factors. The evaluation of these terms can thus be an extremely difficult and complicated task. The pore pressure development as related to shearing rate is not well understood which sometimes results in expected failure in cohesive soil masses.

There are several common methods of shear testing. Some methods can be used with either cohesive or cohesionless soils while others are used with just one type of soil. In the direct shear test - probably the most commonly used although it overestimates the shear strength - a normal load is used to keep the sample in place while a directly applied tangential force tends to separate one portion of the sample from the other. Several types of containers are used. A two-piece rectangular box (Figure G-1) is the most commonly used type of container. Another type consists of rings which can be subjected to either single or double shear (Figure G-2). A third type shears out a cylindrical section from within a cylinder (Figure G-3). The specimen in this third container type undergoes more uniform strains and maintains a more nearly constant area during shear than in the others, but its complexity and the difficulty of specimen preparation offset this advantage.

The shear force can be applied either by increasing the force at a given rate and measuring the resulting displacements (stress-controlled loading units), or by moving the segments of the container apart at a given rate and measuring the resulting force (strain-controlled loading units). Stress-controlled units are preferred for running shear tests at a very low rate. A strain-controlled test is a little easier to perform because the loading rate in stress control must usually be manually regulated. Due to the difficulties in controlling drainage of the soil specimen during a direct shear test, only the drained test method in which complete consolidation is permitted under each increment of normal and shear stress is normally used. Generally, a minimum of three specimens, each under a different normal stress, are tested to establish the relationship between shear strength and normal stress.

In a torsional shear test, a column of soil is subjected to a twisting moment. The moment is normally applied through a disc at the top or bottom. The disc usually has ribs to help prevent slippage between it and the soil. Lateral pressure can be applied to the specimen if desired. The main advantage of the torsional shear test is that the cross section of the sample remains more nearly constant during shear than it does in either the direct or triaxial tests. This advantage, however, is more than offset by the fact that the shear displacements vary as the specimen radius, thus exaggerating progressive failure. Torsional shear tests are more common in Europe than in the United States.

The triaxial compression test is used to measure the shearing

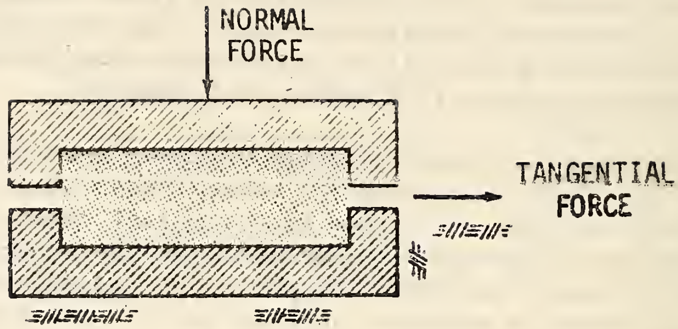

Figure G-1. Direct shear box for single shear. (Schematic.)

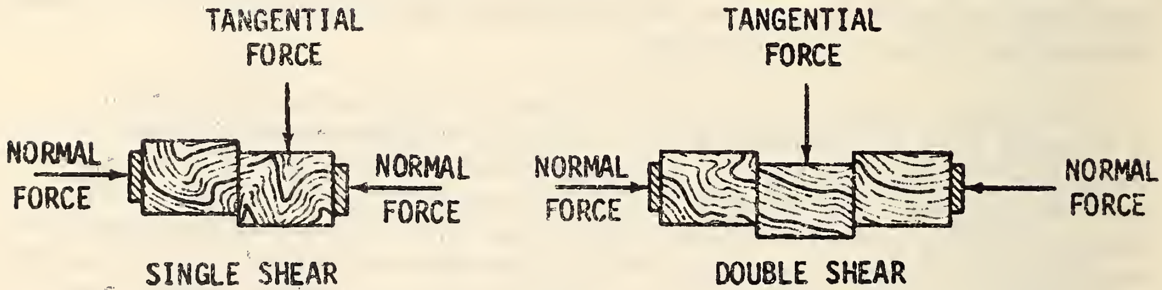

Figure G-2. Direct shear rings. (Schematic.)

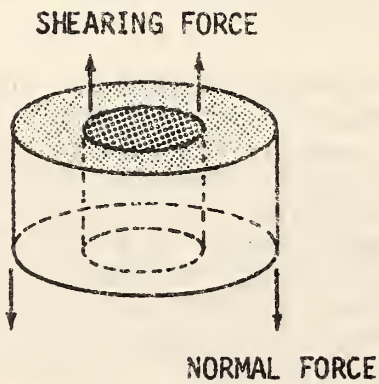

Figure G-3. Direct shear of annular specimen. (Schematic.)

strength of a soil under controlled drainage conditions. In the basic triaxial test, a cylindrical specimen that has been enclosed within a rubber membrane is placed into a triaxial testing chamber, subjected to a confining fluid pressure, and then axially loaded to failure. Connections at the ends of the specimen are used to control drainage of the pore water from the specimen. Usually, a minimum of three specimens, each under a different confining pressure, are tested to establish the relationship between shear strength and normal stress. The test is called triaxial because the three principal stresses are known and controlled.

There are three types of basic triaxial compression tests; the unconsolidated-undrained, the consolidated-undrained, and the consolidated-drained which are referred to as the Q, R. and S tests respectively. The type of test used is selected to closely simulate the anticipated field conditions. Both controlled-strain and controlled-stress types of loading devices are used with triaxial tests.

In the Q test, the water content of the test specimen is not allowed to change during either the application of the confining pressure or the axial loading of the specimen to failure. The Q test is usually applicable only to those soils which are not free-draining, that is, to those soils having a permeability less than 0.0010 centimeters per second.

In the R test, complete consolidation of the test specimen is permitted under the confining pressure and then the specimen is axially loaded to failure while holding the water content constant. The test specimens for the R test must as a general rule be completely saturated before being axially loaded.

In the S test, complete consolidation of the test specimen is permitted under both the confining pressure and the axial loading. Consequently, no excess pore pressure exists at the time of failure. The axial loading is applied slowly enough to allow the water to drain from the specimen in the S test.

The advantages of the triaxial tests as compared to the direct shear test are:

1. The progressive effects of failure are smaller.
2. The measurement of specimen volume changes during shearing are more accurate.
3. The complete state of stress is known at all stages of the test, whereas only the stresses at failure are known in the direct shear test.
4. The triaxial machine is the more adaptable to special requirements.

The advantages of the direct shear test are:

1. The direct shear machine is simpler and faster to operate.

2. A thinner test specimen is used, thus facilitating drainage of the pore water from a saturated sample.

The results of both direct shear and triaxial tests are usually presented as Mohr stress circle diagrams showing shear stresses versus the applied principal or normal stresses.

Appendixes IX and IX A of the Corp of Engineers, Engineer Manual EM 1110-2-1906 (30 November 1970), Laboratory Soils Testing and ASTM Standard D3080-72 give detailed procedures for direct shear tests. Appendix X and ASTM Standard D2850-70 describe triaxial tests.

UNCONFINED COMPRESSIVE STRENGTH

The unconfined compression test measures the compressive strength of a cylinder of soil which has no lateral support. It is applicable only to those soils which have some coherence, that is those soils that retain intrinsic strength after the removal of confining pressure. Dry or crumbly soils, fissured or varved materials, sands, and silts cannot be suitably tested in unconfined compression.

In the unconfined compression test, a laterally unsupported cylindrical specimen is subjected to a gradually increased axial compression load until failure occurs. The axial load may be applied either by the controlled strain procedure, where the stress is applied to produce a predetermined strain rate, or by the controlled stress procedure, in which the stress is applied in predetermined load increments.

The specimen is placed in the testing machine with its vertical axis as near to center of the loading plates as possible. The specimen is then loaded until cracks are definitely developed within the specimen or the stress-strain curve is well past its peak. Load readings and vertical deflection readings, as measured by a proving ring, are recorded periodically during the course of the test. The unconfined test specimens should have a length to diameter ratio between 1.5 and 3.0. If at all possible, the angle between the cracks formed and the horizontal should be measured and recorded. After the test, the water content of the specimen is determined. The results of an unconfined compressive test can be presented in a summary table and/or by a stress-strain curve.

Where the unconfined compressive strength is also obtained after remolding, the sensitivity ratio is also calculated and reported. The sensitivity ratio is calculated from

$$\text{Sensitivity ratio} = \frac{\text{undisturbed compressive strength}}{\text{remolded compressive strength}}$$

A value greater than 3 or 4 for the sensitivity ratio is considered high.

The shear stress is usually taken as one-half the compressive stress when an unconfined compression test is used to determine the shear strength of a soil.

Some errors that cause an inaccurate determination of the unconfined compressive strength and thus make such tests unreliable for determining shear strength are:

1. Test not appropriate to the type of soil being tested.
2. Specimen prepared improperly.
3. Loss in the initial water content.
4. Rate of strain or loading rate too fast.

Appendix XI of the Corp of Engineers, Engineer Manual EM 1110-2-1906 (30 November 1970), Laboratory Soils Testing and ASTM Standard D2166-66 (1972) give detailed procedures for conducting the unconfined compression test.

PERMEABILITY

Permeability is a property of soil which denotes its ability to conduct free (not sorbed) water under a given hydraulic gradient. It is defined as the rate of water discharge under laminar flow conditions at a temperature of 20° C (68° F) through a unit cross sectional area of a soil medium under a given unit hydraulic gradient. The coefficient of permeability thus has the dimensions of velocity.

The permeability of a soil depends on many factors, the most important ones being (1) the size and shape of the soil grains, (2) the geometry of the pores, (3) the void ratio of the soil, (4) the properties of the percolating fluid, particularly its viscosity which is in turn sensitive to temperature, and (5) the degree of saturation.

For an estimation of the velocity of flow, the coefficient of permeability should be determined experimentally. There are two main types of laboratory tests used to determine the coefficient of permeability. These are the constant-head and the falling-head tests. In the constant-head test, the head of water is kept constant throughout the test while in the falling-head test, the head of water is allowed to fall. The constant-head test is used primarily for coarse-grained soils with permeability values greater than about 0.0010 centimeters per second and the falling-head test is used principally for the less pervious soils having permeability values less than 0.0010 centimeters per second. The apparatus used for permeability testing may vary considerably in detail depending primarily on the character and condition of the sample being tested.

The formulas that are usually used for calculating the coefficient of permeability for these laboratory tests are as follows:

For constant-head tests

$$k_{20} = \frac{\epsilon QL}{hAt}$$

For falling-head tests

$$k_{20} = 2.303 \;\frac{\epsilon aL}{At}\; Log\left(\frac{h_o}{h_f}\right) = \frac{\epsilon aL}{At}\; ln\left(\frac{h_o}{h_f}\right)$$

where k_{20} is the coefficient of permeability at 20° C,

ϵ is the viscosity of water at the test temperature,

Q is the quantity of flow,

L is the length of specimen over which the head loss is measured,

h is the head loss over length L,

A is the cross sectional area of the specimen,

t is the elapsed time of test,

a is the inside cross sectional area of the standpipe,

Log is the common logarithm,

ln is the natural logarithm,

h_o is the height of water in the standpipe above the discharge level at the test start, and

h_f is the height of water in the standpipe above the discharge level at the test end.

The coefficient of permeability in laboratory tests is computed on the basis of Darcy's law which is

$$q = \frac{Q}{t} = kiA$$

where q is the rate of discharge or the quantity of flow Q per unit of time t,

k is the coefficient of permeability,

i is the hydraulic gradient, and

A is the cross sectional area of the specimen.

Darcy's law is limited to laminar flow conditions with a complete saturation of the voids. In turbulent flow, the flow is not proportional to the first power of the hydraulic gradient. Under incomplete saturation conditions, the flow is in a transient state and time dependent. All permeability tests, both field and laboratory are performed after the material becomes saturated and a constant flow rate is established.

Appendix VII of the Corp of Engineers, Engineer Manual EM 1110-2-1906 (30 November 1970), Laboratory Soils Testing, describes several laboratory techniques for determining the coefficient of permeability. The ASTM Special Technical Publication 479, "Special Procedures for Testing Soil and Rock for Engineering Purposes" Section V, also contains several suggested methods for permeability tests.

SWELLING

There are three basic causes of soil swelling. One is a combination of elastic rebound, and sometimes moisture content recovery, in a compressed soil mass after the compressive force has been removed. This cause is conspicuous in a consolidation test and is common for any material, particularly a saturated clay. The other two causes depend on the property of some clays to intensively attract water and hold it with a resulting overall increase in volume. These are the frost-sensitive and the water-sensitive expandable clays. The latter are the most important cause of swelling encountered by highway tunnels in the continental U.S.

The expandability (or expansivity) of clay is primarily dependent upon the amount of montmorillonite and some types of illite present. The basic cause of swelling is the attraction and sorption of water by the expanding clay lattices. A contributing factor is the relief of the capillary pressure resulting from thickening of the capillary films. The capillary films are automatically enlarged during wetting of the soil and the acting compressive stress is relieved which then permits further opening of the expandable lattices.

The swelling in clays is associated with the sorption of moisture from either a liquid that comes in contact with the clay or from ambient humid air. Generally, the swelling will be gradual and last for a long time until a limit is attained. Laboratory tests have shown that the water intake is greatest at the beginning of the swelling process and then proceeds at a decreasing rate.

Expandable minerals may be identified by petrographic tests such as microscope analysis, X-ray diffraction, and differential thermal analysis. In ordinary engineering practice, however, these methods are usually not economically feasible. The simple tests briefly described in the following paragraphs are used for most engineering practice.

In the free-swell test, ten cubic centimeters of the minus 40 mesh size fraction of clay is slowly added to 100 cubic centimeters of water in a graduate. After 24 hours, the volume of settled and expanded material is read in terms of the cylinder graduations. The results are reported as a percentage swell value. The free-swell test alone cannot depict the expandability of a clay sufficiently and so, the following various tests are performed in more important cases.

A high colloidal content indicates only a possibility of expandable colloids being present. In those regions where montmorillonite-type clays are found, however, the probability of the presence of expandable colloids is high. In such regions, combinations of a low shrinkage limit and a high plasticity index indicate probable swelling. The low shrinkage limit value indicates that swelling may start at a low moisture content. The high plasticity index value indicates that the clay is capable of sorbing a large amount of water without cracking or bursting at high moisture contents.

Load expansion tests are generally performed in consolidometers or similar devices used in the study of consolidation and compressibility of soils. Basically, a consolidometer is a ring in which the clay sample is placed between two porous stones (Figure G-4). The lower stone

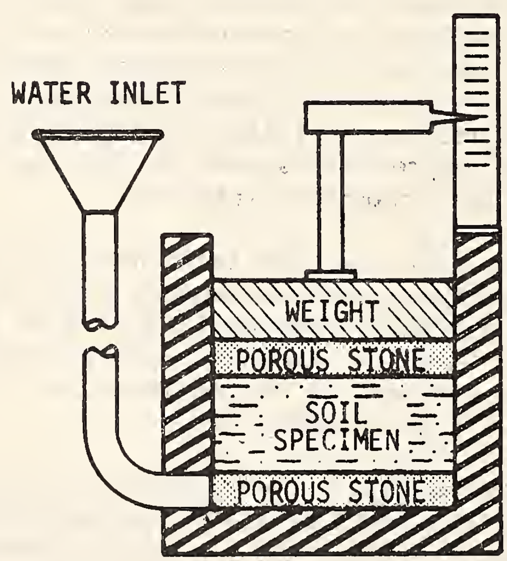

Figure G-4. Consolidometer for swelling test. (Schematic.)

is immovable and the upward movements of the upper stone are recorded. Usually, two identical samples are cut from natural (or remolded) material and placed in consolidometers. After the samples have dried to at least the shrinkage limit, the volume of one of the samples is determined by immersion in mercury. A unit load is applied to the top surface of the other sample which is then wetted from below. As it swells its thickness is measured. In this way the volume change of the given material from its natural (or remolded) condition to the air-dried and saturated conditions, respectively, is determined.

Consolidometer tests may be used to determine the expansion pressure of a clay by preventing the vertical expansion of laterally confined specimens and measuring the pressure developed.

One technical difficulty in testing expansive soils in containers is the tendency for the sample to swell unevenly, more at the center and less at the periphery because of the wall friction. Generally, properly conducted tests of expansive soils in containers are quite time consuming, especially for clays having a low permeability.

LABORATORY TESTING OF ROCK

IDENTIFICATION

Rock identification during tunnel site investigations is accomplished principally by visual examination. Identification forms the basis for delineating the subsurface material into formations, or rock units, which differ from one another in composition, texture, origin, degrees of induration, alteration, and weathering, and other physical properties and features.

Representative samples of the various rock units expected to be penetrated by the highway tunnel should be selected from surface exposures, borehole cores, or pilot tunnels and submitted for a more thorough petrographic study and in some cases chemical and other laboratory analyses of components. A detailed examination of these samples enables the engineering geologist to better plan the types and amounts of physical-mechanical laboratory tests which he will recommend be performed and contributes significantly to a more complete understanding of the results of these tests.

Components of most crystalline rocks may be identified with high powered, polarizing microscopes by studying thin sections using transmitted light or polished sections using reflected light. Another microscopic technique involves observation of mineral grains immersed in oils having different refractive indices.

Cryptocrystalline (crystals too fine grained for microscope indentification), amorphous noncrystalline, and glassy rocks may require chemical analysis for positive identification.

Clay minerals warrant detailed study if present in appreciable quantities since montmorillomite and some types of illite may be quite deleterious to rock strength and stability due to their nature of swelling and disintegration upon contact with water. Principal laboratory techniques for clay mineral identification are use of the X-ray diffractometer, differential thermal analysis, and observation by electron microscope.

SPECIFIC GRAVITY AND POROSITY

APPARENT SPECIFIC GRAVITY

Specific gravity is the ratio of the mass of a specimen to the mass of an equal volume of water at a specified temperature. Several methods have been used for determining this property.

ASTM Designation C97-36 specifies the following procedure for measuring the apparent specific gravity of natural building stone.

$$\text{Apparent specific gravity} = \frac{W_s}{W_t - W_w}$$

where W_s is the weight of the oven-dried specimen,

W_t is the weight of the wet specimen after water immersion, and

W_w is the weight of the specimen suspended in water.

Another method for low-porosity rocks (under one percent porosity) involves measurement of the volume of the specimen, usually by pycnometer, and obtaining its dry weight. Then

$$\text{Apparent specific gravity} = \frac{\text{Dry Weight}}{\text{Bulk Volume}}$$

This method is applied equally well to regular or irregular shaped specimens.

A method used for rocks known to have a porosity greater than one percent involves preparation of a near-perfect cylinder which is dried to certain specifications, weighed, and then accurately measured with a micrometer to the nearest one-thousandth inch. Then

$$\text{Apparent specific gravity} = \frac{W_s}{V_A}$$

where W_s is the dry weight and

V_A is the apparent volume as calculated from measurements.

APPARENT POROSITY

The apparent porosity of a rock specimen is the ratio of the volume of surface and surface-connected pore space to the apparent volume of the specimen. Two methods of the determination are described. Obert, Windes, and Duval[109] recommend the following calculation:

$$\text{Apparent porosity (percent)} = \left(\frac{W_t - W_s}{\rho_w V_A}\right) \times 100$$

where W_t is the weight of the water-saturated specimen,

W_s is the weight of the oven-dried specimen,

V_A is the external volume calculated from dimension measurements, and

ρ_w is the density of water.

Paone and Bruce[110] describe a different method.

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[109] Obert, Leonard, S. L. Windes, and Wilbur I. Duvall. Standardized Tests for Determining the Physical Properties of Mine Rock. U. S. Bureau of Mines Report Investigation 3891. 1946.

[110] Paone, James and W. E. Bruce. Drillability Studies, U. S. Bureau of Mines Report Investigation 6324. 1963.

Apparent porosity (percent) =

$$\left(\frac{V_A - V_p}{V_A}\right) \times 100$$

where V_A is the apparent volume calculated from dimension measurements and

V_p is the volume obtained by pycnometer.

HARDNESS AND ABRASION RESISTANCE

Rock hardness, along with compressive strength and degrees of fracturing, induration, and coherence between grains indicate the "drillability" of a rock unit. These properties and conditions must be evaluated by the design engineer for his recommendation on the tunneling method to be used. If a tunnel boring machine is selected, the proper bit type, number, and spacing are also dependent upon this information.

The general term hardness, as applied to rocks, is quite vague as a number of properties including toughness, strength, elasticity, brittleness, and ductility are involved. Rocks are combinations of minerals so hardness and other physical and mechanical properties are functions of the properties of individual grains and of the nature of the bonding between the grains.

Several different types of hardness tests have been developed including those which measure impact rebound hardness and the resistance to scratching, crushing, or abrasion. A presently popular technique using microbits is a useful indicator of "drillability."

IMPACT REBOUND HARDNESS

Two types of instruments - the Shore Scleroscope and the Schmidt impact hammer - are used to determine numerical rebound hardness values on flat surface specimens.

The Shore Scleroscope indicates hardness in scale divisions ranging from 0 to 120 using a diamond pointed hammer dropped from a fixed height which rebounds to a height proportional to its hardness. Both direct reading and dial reading models are available. Since the diamond hammer point is small (approximately 0.03-inch diameter) it may strike an individual mineral grain. In a coarse-grained granite, rebound from a quartz grain would be higher than from a mica crystal. Therefore, several tests must be made and results are best reported in terms of the average value and the standard deviation.

Moisture content has been found to affect the rebound characteristics of some limestone and sandstone, both of which may exhibit higher rebounds in drier conditions.

Experimentally determined relationships between Scleroscope hardness and compressive strength and abrasive hardness, respectively, have been published in Report of Investigations 4459 by the U.S. Bureau of Mines for numerous samples of the three principal rock types.

The Schmidt impact hammer is a small field instrument developed for determining the compressive strength of concrete. It consists of a spring-loaded piston which is projected against an anvil held in perpendicular contact with a flat rock surface. The piston rebounds from the anvil to a scale height of 100 arbitrary divisions. This value is an indicator of "drillability." Several readings are taken and averaged.

SCRATCH HARDNESS

A widely recognized standard known as Mohs' scale of hardness is used for ranking the relative hardness of individual minerals. The scale is based on the following ten minerals listed in order of decreasing hardness:

Diamond
Corrundum
Topaz
Quartz
Orthoclase
Apatite
Fluorite
Calcite
Gypsum
Talc

Each mineral will scratch those minerals below it on the scale and will in turn be scratched by those above it.

Since rocks are comprised of a number of minerals, the Mohs' hardness scale is more difficult to apply to rocks. The Mohs' hardness of coarse-grained rocks would be best expressed by giving the hardnesses of individual minerals whereas that of fine-grained rocks would be the average resistance to scratching.

CRUSHING HARDNESS

The Protodyakonov test measures the rock's resistance to crushing. In this test a rock sample is hammer broken to produce 5 test specimens of 2-4 centimeters dimensions, each having a volume of 10-20 cubic centimeters. The specimens are individually placed in a tubular drop tester and impacted a standard number of times (from 5 to 15) with a 2.4 killogram weight dropped from a height of 60 centimeters. The broken material from the 5 tests is combined and separated with a 0.5 millimeter wire screen. Fines passing the screen are placed in a volumeter tube and the height of the dust recorded.

The strength coefficient, (f) often referred to as the Protodyakonov hardness number, is determined by the following equation:

$$f \simeq \frac{20n}{L}$$

where n is the number of blows and

L is the height of the powder column in millimeters.

The Protodyakonov hardness number is dependent upon the weight of the sample used, the number of blows, screening time, and degree of compaction of the dust in the volumeter. Standardization of these factors will result in better comparison between samples.

ABRASION RESISTANCE

Abrasion resistance is a relative property usually determined by one of several techniques using accepted standardized equipment. Three commonly used systems are the Dorry abrasive hardness tester, the paddle type machine and the Los Angeles machine.

The Dorry test, developed by the French School of Bridges and Roads for studying characteristics of road-building materials, uses a 0.25 millimeter diameter cylindrical rock specimen which is abraded against a rotating steel disc. The specimen, loaded to 1,250 grams, is ground for 1,000 revolutions using crushed quartz (Number 30 to Number 40 sieve sizes) for an abrasive. The weight of rock abraded during the test is determined and the hardness coefficient calculated from:

$$H_d = \frac{60 - W_a}{3}$$

where H_d is the hardness coefficient and

W_a is the weight of the rock abraded in grams.

The paddle type machine, developed by the Pennsylvania Crusher Division of Bath Iron Works Corp., uses a 400-gram sample of broken rock passing a 0.742 and retained on a 0.371 Tyler Standard screen, which is placed in a 12 3/16-inch I.D. drum rotating at 74 rpm. A steel paddle of standard hardness and composition, 1 inch wide, 3 inches long, and 1/4 inch thick rotates within the drum at 632 rpm in the opposite direction to the drum. Four sample batches are run for 15 minutes each. The paddle is carefully weighed before and after each test with the loss in weight of the paddle in tenths of a milligram being the numerical measure of abrasion determined.

The Los Angeles abrasion testing machine and its application to testing coarse aggregate is described by ASTM. A standard charge of specified weights of rock in different size ranges plus a standard number of steel balls approximately 1-7/8 inch in diameter, weighing between 390 and 445 grams each is placed in the ball mill type cylinder which is rotated at 30 to 33 rpm for 500 revolutions. Abrasiveness, reported as the percentage of wear, is then determined by comparing the original sample weight with the weight of the pulverized material removed which passes a Number 12 sieve.

MICROBIT DRILLING TESTS

Drill bit and tunnel machine manufacturers often test the "drillability" of subsurface core and other rock samples by actually drilling holes in them with miniature bits of similar design and proportion to standard bits. Many of the methods and rating systems are considered proprietary by the organizations involved in this competitive field.

A common problem experienced with all laboratory rock property test methods is that tests conducted on flawless intact specimens often provide results substantially different from the properties exhibited by larger scale, in situ volumes of rock. An example would be a hard, but closely jointed, granite which in a small specimen size would be quite difficult to drill, but which might prove to be drillable by a tunnel boring machine because of the abundant planes of weakness.

WEATHERING RESISTANCE

Rocks encountered at depths to 500 feet (the zone of interest in this study) are likely to have been weathered to varying degrees during the passage of geologic time, especially in the upper 100 feet. Weathering causes decreases in the various types of rock strength. Numerous tests are applied to determine the physical and mechanical properties of the rock in the condition in which it is found. Tunnel support and lining requirements are estimated by the combined evaluation of several properties and conditions.

Of additional concern to tunnelers is accelerated weathering which may occur to some tunnel rock after excavation. Rock which is quite competent at the time of excavation may become quickly or gradually weakened due to various processes of mechanical or chemical weathering during the anticipated effective lifetime of the tunnel. Potential delayed stability problems of this nature should be recognized in advance and provisions made to minimize their occurrence.

Clays, mudstones, shales, carbonates, carbonate-cemented rocks and montmorillonite-rich rocks are those most subject to exposed surface deterioration. Also, all rock types cut by joint or fault discontinuities which are filled with swelling or nonswelling clays, gypsum or carbonate minerals.

Clays, mudstones, and shales are subject to exposed surface deterioration by the slaking action of air and water and by water erosion. Carbonates and carbonate-cemented rocks (such as calcite-cemented sandstones) may suffer outer surface and interior deterioration by solution, especially when acidic groundwater is introduced. Rocks rich in montmorillonite clays and some types of illite clays may undergo rapid weathering by slaking and erosion. All rock types, no mater how strong or inert, may be cut by joint or fault discontinuities filled by swelling or nonswelling gouge, gypsum, or carbonate minerals. These discontinuities may be affected by some combination of slaking, solution, and erosion, thus causing a decrease in stability of the rock mass as a whole.

Samples of suspect rock types and/or of discontinuity filling materials recovered from drill holes should be tested by chemical and physical laboratory means. Applicable tests include:

1. Chemical determination of the carbonate content for certain rocks.
2. Extended leaching tests of carbonate rocks in low concentration acidic water (if such is known to exist in the groundwater which may later become introduced to the tunnel level).
3. Air-water slaking tests on clay-rich rocks and discontinuity filling material.
4. Clay swelling tests (described in the laboratory testing section on swelling of soils).
5. Clay mineral determination by the X-ray diffractometer method.

6. Clay mineral determination by differential thermal analysis.
7. Clay mineral determination by electron microscope examination.

UNIAXIAL COMPRESSIVE STRENGTH

Probably the most commonly measured property in tunnel investigation, uniaxial compressive strength of rocks, was first standardized by ASTM in 1971 under designation D2938-71 which has been modified in version D2938-71a.

Specifications call for loading right-circular, cylindrical specimens to failure in a press. Specimens ideally should have a length to diameter ratio between 2.0 to 1 and 2.5 to 1, and a minimum diameter of NX core size (approximately 2 1/8 inches).

The uniaxial compressive strength for standard-dimensioned specimens is calculated by dividing the maximum load applied by the cross-sectional area.

When only cores having a length to diameter ratio of less than 2 to 1 are available, the following corrective calculation should be used:

$$C_c = \frac{0.889hC_m}{0.778h + 0.222d}$$

where C_c is the computed compressive strength of an equivalent length to diameter ratio of 2 to 1 specimen,

C_m is the measured compressive strength of the specimen tested,

d is the test core diameter, and

h is the test core height.

It is recommended that continuous loading be at a rate which will cause failure after a period of 5 to 15 minutes.

Other important factors to be considered in these tests are perpendicularity and smoothness of the specimen's bearing surfaces, smoothness and circularity of the cylinder sides, moisture content, alignment of the press swivel head, and the effect of friction between bearing plates and specimen.

Uniaxial compressive strength data is used in the design of underground openings and is an important factor considered in the determination of rock drillability.

Several procedures have been tried for determining the unconfined shear strength of rocks, but none are considered to be satisfactory.[111] Figure G-5 shows four of the various testing techniques where samples are loaded to failure.

Unconfined shear strength in the single shear test is computed from:

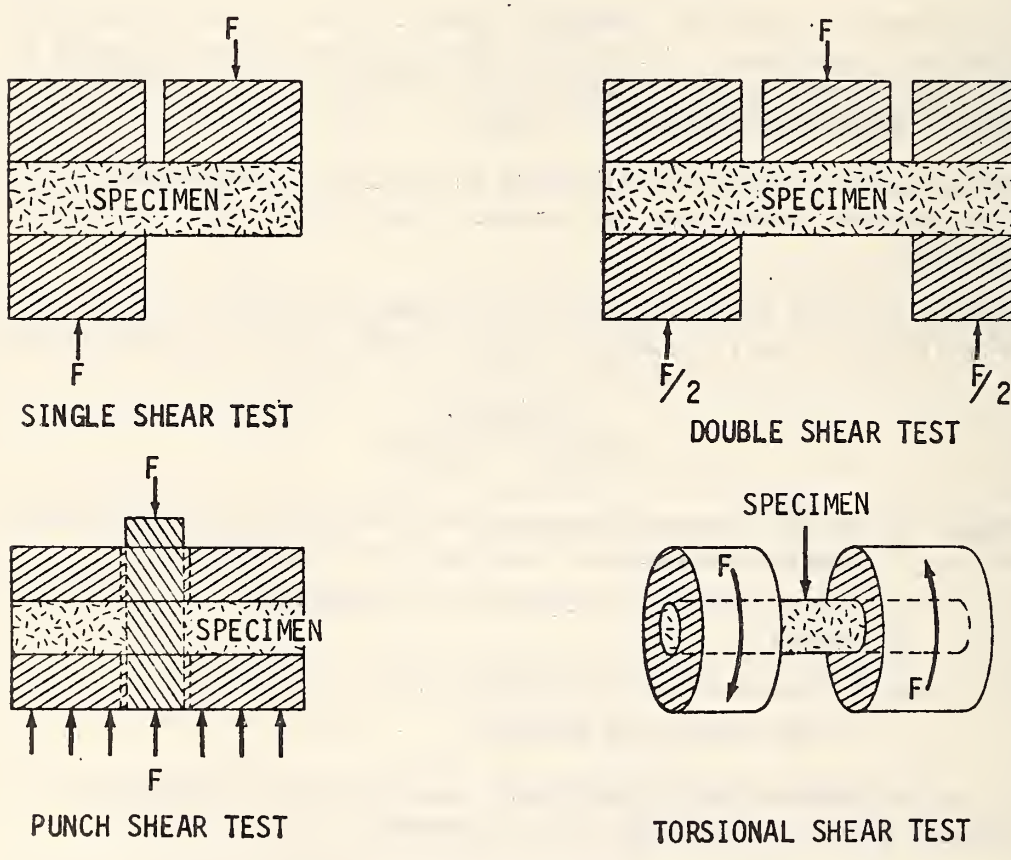

Figure G-5. Unconfined shear strength tests. (Schematic.)

[111] Obert, Leonard and Wilbur I. Duval. Rock Mechanics and the Design of Structures in Rock. John Wiley & Sons, New York. 1967

$$S_o = \frac{F_c}{A}$$

and in the double shear test from:

$$S_o = \frac{F_c}{2A}$$

where S_o is the unconfined shear strength,

F_c is the force required in the direction of plane A to produce failure, and

A is the cross-sectional area of the sample.

In the punch shear test:

$$S_o = \frac{F_c}{2\pi\, ra}$$

where r is the radius of the punch and

a is the thickness of the sample.

In the torsional shear test:

$$S_o = \frac{16M}{\pi d^3}$$

where M is the applied torque at failure and

d is the diameter of the cylinder.

In the single, double, and punch shear tests, major stress buildups occur along the shear, or punch and die, edges, causing failure along fractures starting at these edges and proceeding across the sample in a direction different from that of maximum shear stress. Also, undesired stress is caused by the clamping action of grips. In the torsional shear test, results may be distorted due to the clamping action of the grip and to bending.

In view of the unrealiability of uniaxial shear testing, its applicability to tunnel site investigations is considered very limited.

UNIAXIAL TENSILE STRENGTH

Test methods developed for measuring tensile strength include:

1. Direct Tensile Test - for the standardized version see ASTM designation D2936-71.
2. Brazilian (indirect) Test.
3. Ring Test.
4. Point Load Test.

A related tensile test, the modulus of rupture determination (flexure) is discussed under a separate heading.

DIRECT TENSILE TEST

ASTM designation D2936-71 specifies the use of right circular cylindrical specimens having a length to diameter ratio within the range between 2.0 to 1 and 2.5 to 1, with a diameter no less than NX core size (approximately 2 1/8 inches). The test cylinder is cemented at each end to metal caps. A tensile load is applied and gradually increased at a rate which will cause failure after a period of 5 to 15 minutes.

The tensile strength of the rock is calculated by dividing the maximum load by the cross-sectional area of the specimen.

BRAZILIAN (INDIRECT) TENSILE TEST

In this easily performed method, a short rock cylinder is compressed along the diametral plane to failure (Figure G-6). Rupture usually occurs on the diametral plane passing through the lines of loading. While this test itself is quite simple, the results are less easily evaluated than those of the direct test because the compressive load causes compressive stress and crushing in producing the tensile stress.

In the Brazilian test:

$$\sigma_t = \frac{2F_c}{\pi d L}$$

where σ_t is the tensile stress,
F_c is the maximum load at failure,
d is the diameter of the specimen, and
L is the length of the specimen.

RING TENSILE TEST

A variation of the Brazilian test, this method utilizes a short

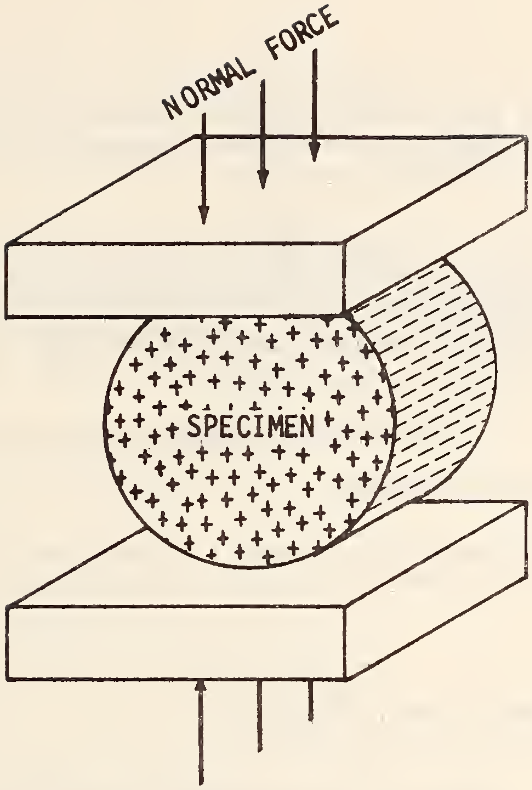

Figure G-6. Brazilian indirect tensile test. (Schematic.)

cylindrical specimen with a cylindrical hole cut from its center. Discussion and calculations for this method may be consulted in Clark and others,[112] Adinall and Hackett,[113] and Ripperger and Davids.[114]

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[112] Clark, George B., John W. Brown, Charles J. Haas, and David A. Summers. Rock Properties Related to Rapid Excavation. Report for U.S. Dept. of Transportation, OHSGT, Contract 3-0143. Rock Mechanics & Explosives Research Center, University of Missouri - Rolla. March, 1969.

[113] Addinal, E. and P. Hackett. "Tensile Failure in Rock-like Materials." Proceedings of 6th Symposium on Rock Mechanics. University of Missouri - Rolla. 1964. pp. 515-538.

[114] Ripperger, E. A. D. and N. Davids. "Critical Stresses in Circular Ring." Transactions American Society of Civil Engineers, Vol. 112. Paper 2308. 1947.

POINT LOAD TEST

Another easily performed tensile strength test is the method where point loads are applied to break cores, prisms, or discs. For this method the maximum tensile stress is:

$$\sigma_t = \frac{KP}{h^2}$$

where σ_t is the maximum tensil stress,

P is the applied load,

h is the distance between loading points, and

K is a constant.

A knowledge of tensile strength is principally of value to designers in conditions where the tunnel roof is comprised of layered rock.

UNIAXIAL FLEXURAL STRENGTH

The flexural strength, or modulus of rupture test, is a determination of the outer fiber tensile strength of a material. This test usually consists of center-point loading cylindrical or prismatic specimens to failure[115]. (See Figure G-7)

For cylindrical specimens, the flexural strength is calculated from:

$$R = \frac{8PL}{\pi d^3}$$

where R is the flexural strength,

P is the applied load at failure,

L is the length between bearing edges of the lower plate, and

d is the diameter of the specimen.

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[115]Obert, Leonard and Wilbur I. Duval. Rock Mechanics and the Design of Structures in Rock. John Wiley & Sons, New York. 1967.

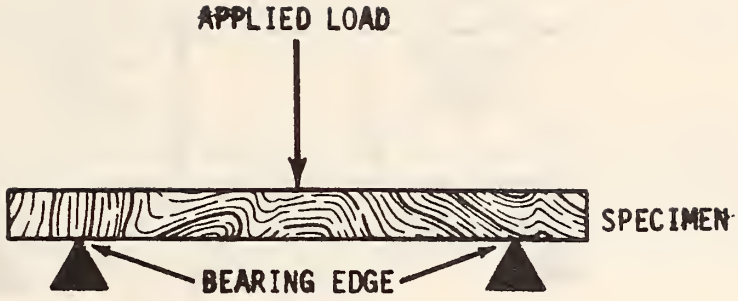

Figure G-7. Flexural Strength - Center Point Loading Method. (Schematic.)

For prismatic specimens, the computation is:

$$R = \frac{3\ PL}{2\ ba^2}$$

where a is the specimen thickness and

b is the specimen width.

A variation of this test, termed the third point loading method and illustrated by Figure G-8, is described by Clark and others.[116]

No standardized test method has yet been adopted for all rock types. Useful information on test procedures may be found in the following ASTM standards developed for specimens of rectangular cross-section:

1. Modulus of Rupture of Natural Building Stone, ASTM Designation C99-52.
2. Flexure Testing of Slate, ASTM Designation C120-52.
3. Flexural Strength of Concrete, ASTM Designation C293-64.
4. Flexural Strength of Concrete, ASTM Designation C78-64.

Flexural strength information is important for tunnel design, especially in bedded roof conditions.

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[116]Clark, George B., John W. Brown, Charles J. Haas, and David A. Summers. Rock Properties Related to Rapid Excavation. Report for U. S. Department of Transportation, Rock Mechanics & Explosives Research Center, Univerity of Missouri. March, 1969

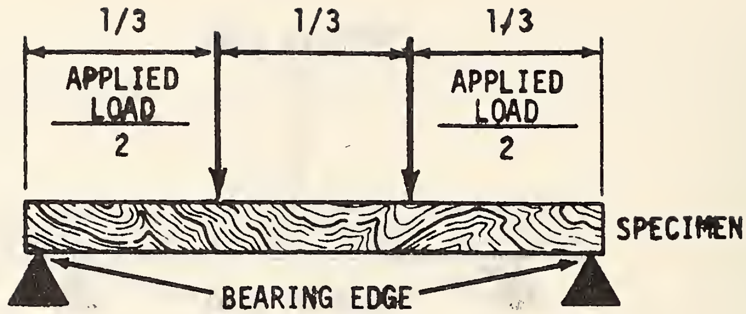

Figure G-8. Flexural strength - third point loading method. (Schematic.)

TRIAXIAL COMPRESSIVE AND SHEAR STRENGTHS

Triaxial compressive and shear strength tests are performed by axially loading right-circular, cylindrical specimens to the point of rupture while simultaneously applying a constant hydraulic-confining pressure to their curved surfaces. The samples are enclosed in flexible, impervious membranes to prevent the entry of hydraulic fluid into the pores. When sleeves are not used, the pore pressures must be taken into account if the specimen is porous.

ASTM designation D2664-67 specifies the standard test method for the triaxial compressive strength of undrained rock core specimens without ore ressure easurements. The standard requires that specimens have length to diameter ratios of between 2.0 to 1 and 2.5 to 1 and a minimum diameter of NX core size (approximately 2 1/8 inches).

The standard triaxial test provides data for determining the shear strength at various lateral pressure, angle of shearing resistance, cohesion intercept, and deformation modulus.

It is recommended that three tests be made at each of three confining pressures. The loading rate is to be such that a constant strain rate is applied. The strain rate used should be the same as that which would cause rupture in a similar specimen under unconfined compression within a period of 2 to 15 minutes. Sample deformation is recorded as required during the test.

The following calculations and graphical plots are required from a triaxial test:

1. Construct a stress difference versus axial strain curve. Stress difference is the maximum axial stress minus the lateral confining pressure.

2. Construct Mohr stress circles using shear stresses as ordinater and normal stresses as abscissar.
3. Draw a "best-fit" Mohr's envelope curve tangent to the circles.

If the envelope is a straight line the angle it makes with the horizontal is the angle of shearing resistance. The intercept of this line at the vertical axis is the cohesion intercept, C. If the envelope is not a straight line, values of the angle of shearing resistance should be determined by constructing a tangent to each Mohr circle and noting the corresponding cohesion intercepts.

Triaxial compressive and shear data are used for calculating the bearing capacity of foundation rock for surface structure and to a lesser extent, in the design of underground openings.

STATIC ELASTIC CONSTANTS

The two most commonly measured static elastic constants are the modulus of elasticity (Young's modulus) and Poisson's ratio. These constants are determined by subjecting a prismatic or cylindrical specimen to an increasing and/or decreasing axial compressive or tensile stress and measuring the corresponding change in strain parallel and normal to the direction of applied stress.

Because stress-strain curves are typically nonlinear, (Figure G-9), both the tangent modulus and secant modulus are obtained from:

$$E_t = \frac{\Delta \sigma_z}{\Delta \epsilon_z}$$

and

$$E_s = \frac{\sigma_z}{\epsilon_z}$$

where E_t is the tangent modulus of elasticity,

E_s is the secant modulus of elasticity,

ϵ_z is the vertical strain,

ϵ_x and ϵ_y are the lateral strains, and

σ_z is the axial compression or tensile stress.

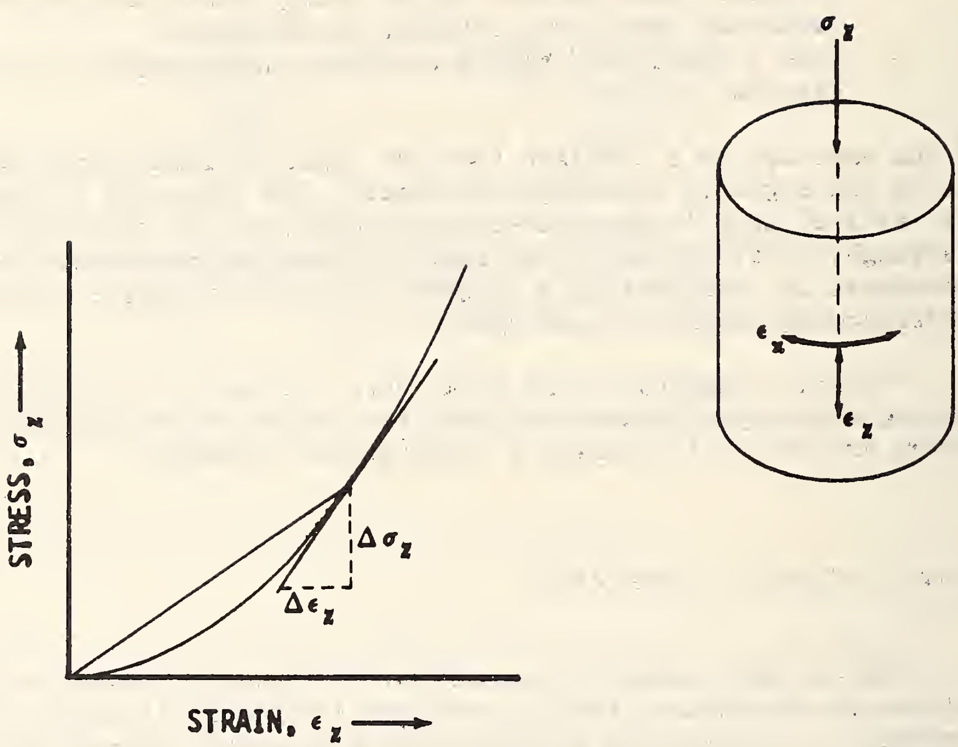

Figure G-9. Tangent and secant modulus of elasticity.

Poisson's ratio is found from:

$$\mu = - \frac{\epsilon_x}{\epsilon_z} = - \frac{\epsilon_y}{\epsilon_z}$$

where μ is Poisson's ratio.

Figure G-10 shows the typical stress-strain relationships for several rock types.[117]

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[117] Stagg, K. G. and O. C. Zienkiewicz, editors. Rock Mechanics in Engineering Practice. John Wiley & Sons, London. 1968.

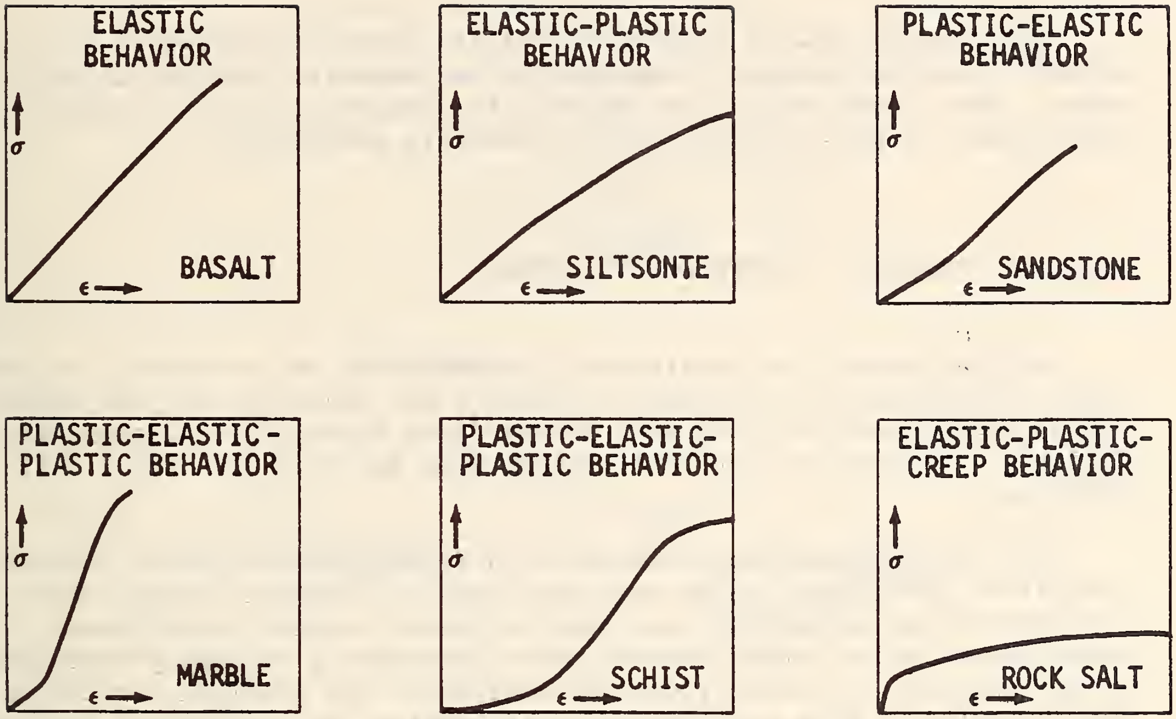

Figure G-10. Typical stress-strain curves for uniaxial compressive rock tests. (After Stagg and Zienkiwicz)

The static modulus of elasticity has also been determined by measuring the deformation of a borehole in a prism and be measuring the load-deflection relationship in a beam or cantilever. The latter method is not considered as reliable as the uniaxial method.[118] Obert developed a method for measuring the triaxial elastic constants of stress-reliefed cores obtained by the overcoring sampling method. Since subsurface in situ rock is under triaxial stress, the triaxial test methods make it possible to determine the elastic constants under a state of stress approximating that which exists in situ.

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[118] Obert, Leonard and Wilbur I. Duval. <u>Rock Mechanics and the Design of Structures in Rock</u>. John Wiley & Sons, New York. 1967.

DYNAMIC ELASTIC CONSTANTS

The dynamic elastic moduli of rock are commonly determined by two methods known as resonant frequency, or bar velocity, and ultrasonic pulse. The dynamic modulus of elasticity, dynamic modulus of rigidity, and dynamic Poisson's ratio are the constants determined.

RESONANT FREQUENCY (BAR VELOCITY) METHOD

In this method the longitudinal, transverse, and torsional frequencies of vibrating rock prisms or cylinders are measured, and the dynamic moduli then calculated. An ASTM standardized test method, Designation C215-60, developed for concrete, can be used for rock with minor modifications.

The testing apparatus consists of a driving circuit which induces controlled vibrations in the test specimens at different frequencies; a support which permits the specimen to vibrate without significant restriction; and a pickup circuit which generates a voltage proportional to the amplitude, velocity, or acceleration of the specimen, amplifies it and indicates it with a voltmeter, milliammeter, or cathode-ray oscilliscope. By varying the placement of the specimen with respect to the driving and pickup units; the transverse, longitudinal, and torsional frequencies are separately determined (Figure G-11). In each case, the specimen is vibrated at varying frequencies and that which results in the maximum reading on the indicator is recorded as the fundamental transverse, longitudinal, or torsional frequency respectively.

The following calculations are made:

Dynamic modulus of elasticity knowing the fundamental transverse frequency,

$$E_{dyn} = CWn^2$$

where E_{dyn} is the dynamic modulus of elasticity in pounds per square inch,

W is the specimen weight in pounds,

n is the fundamental transverse frequency in cycles per second,

C is $0.00416 \dfrac{L^3\ T}{4}$ seconds squared per square inch for a cylinder or

is $0.00245 \dfrac{L^3\ T}{bt^3}$ seconds squared per square inch for a prism,

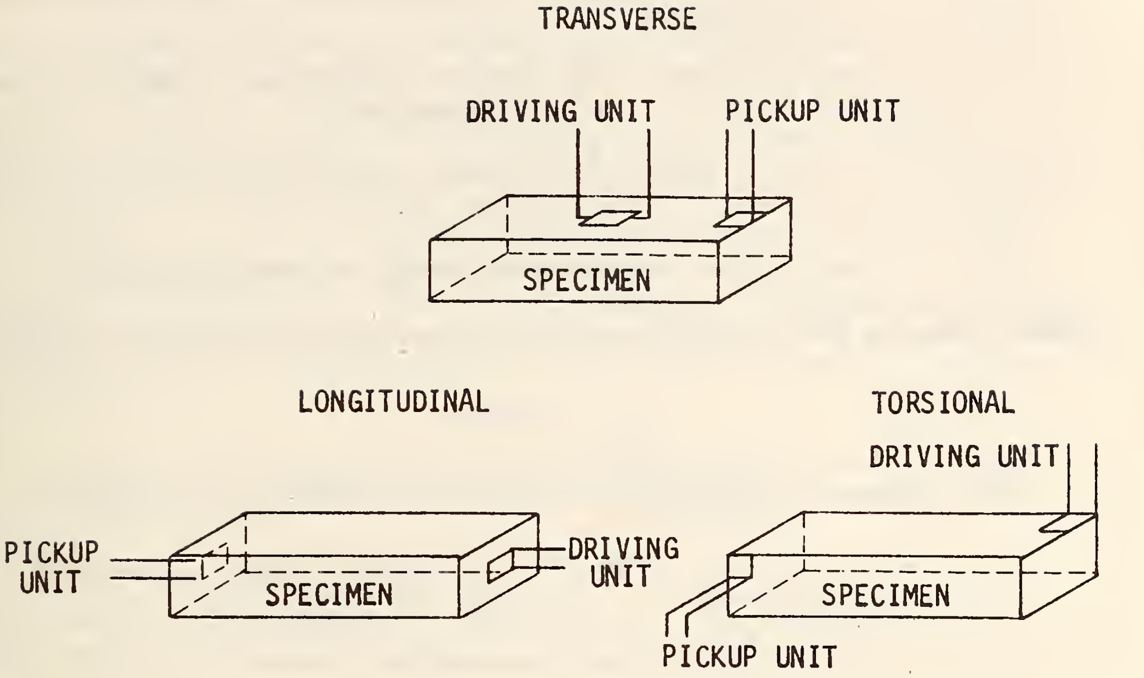

Figure G-11. Typical driving unit and pickup unit placement for resonance vibration tests.

L is the specimen length in inches,

d is the diameter of the cylinder in inches,

t and *b* are the prism cross-sectional dimensions (*t* is the driver direction) in inches, and,

T is a correction factor given in tables appearing in the ASTM standard C215-60.

Dynamic modulus of elasticity knowing the fundamental longitudinal frequency.

$$E_{dyn} = DW\,(n')^2$$

where n' is the fundamental longitudinal frequency in cycles per second

D is $0.01318 \frac{L}{d^2}$ seconds squared per square inch for a crylinder or

is $0.01035 \frac{L}{bt}$ seconds squared per square inch for a prism, and

W, L, d, t, and b are the same as defined earlier.

Dynamic modulus of rigidity knowing the fundamental torsional frequency,

$$G_{dyn} = BW(n'')^2$$

where G_{dyn} is the dynamic modulus of rigidity in pounds per quare inch,

n'' is the fundamental torsional frequency in cycles per second,

B is $\frac{4LR}{gA}$ seconds squared per square inch,

W and L are the same as defined above,

R is a shape factor (1.0 for circular cylinder and 1.183 for square cross-sectioned prism),

g is gravitational acceleration of 386.4 inches per second per second, and

A is the cross-sectional area of the specimen in square inches.

Dynamic Poisson's ratio,

$$\mu_{dyn} = \frac{E_{dyn}}{2G_{dyn}} - 1$$

where μ_{dyn} is the dynamic Poisson's ratio.

This method is useful for detecting changes in the dynamic moduli due to weathering or alteration.

ULTRASONIC PULSE METHOD

In this method the pulse velocities of both compression waves and shear waves through isotropic or slightly anisotropic rock are measured and the ultrasonic elastic constants calculated. ASTM standard test method Designation D2845-69 has been adopted for this procedure.

The pulse generator unit consists of an electronic pulse generator and external power amplifier if needed. The voltage output is in the form of rectangular pulses or a grated sine wave. The transducers consist of a transmitter which converts electrical pulses into mechanical pulses which are then imparted to the test specimen and a receiver which converts mechanical pulses into electrical pulses. Ceramic piezoelectric material such as lead-zirconate-titanate is often used for the transducer elements. A voltage preamplifier may be required if the voltage output of the receiving transducer is low or if the display and timing units are insensitive. The display and timing unit consists of a cathode-ray oscilloscope capable of simultaneously displaying both the direct and transmitted pulses, and an electronic timing device capable of measuring time intervals.

Test cylinders are recommended to have proportions such that the ratio of pulse travel distance to the minimum lateral dimension does not exceed five.

The test procedure requires measurement of the pulse travel distance through the specimen and determination of the specimen's weight, volume, density, apparent porosity, and degree of saturation. Density is calculated from

$$\rho = \frac{W}{gV}$$

where ρ is the density in pounds-square seconds per inches^4,

W is the specimen weight in pounds,

V is the specimen volume in cubic inches, and

g is the gravitational acceleration of 386.4 inches per second per second.

To determine the apparent porosity and degree of saturation, weigh the specimen, oven-dry it at 105° C. to a constant weight, then immerse it in filtered or distilled water under a vacuum for 24 hours, remove it, blot with a damp cloth, and reweigh. The same process is repeated until the increase in weight between treatments does not exceed 0.1 percent of the final weight. Then

$$\text{Apparent porosity} = \frac{(W_t - W_s)}{V \gamma_w}$$

where W_t is the weight of the saturated specimen,

W_s is the weight of oven-dried specimen,

γ_w is the unit weight of water, and

V is the volume of the specimen.

and

$$\text{Degree of saturation} = \frac{(W_n - W_s)}{(W_t - W_s)}$$

where W_n is the weight of the specimen before oven drying, and W_o and W_s are as defined above.

Pulse travel times through the specimen are measured and the propagation velocities of the compression and shear waves are calculated as follows:

Compression pulse propagation velocity,

$$V_p = \frac{L_p}{T_p}$$

Shear pulse propagation velocity,

$$V_s = \frac{L_s}{T_s}$$

where L is the pulse travel distance in inches and
T is the effective pulse travel in seconds.

Then the ultrasonic elastic constants are calculated as follows:

Modulus of elasticity (Young's Modulus),

$$E_{dyn} = \frac{V_s^2 \left(3V_p^2 - 4V_s^2\right)}{V_p^2 - V_s^2}$$

Modulus of rigidity (shear modulus),

$$G_{dyn} = p\ Vs^2$$

Poisson's ratio,

$$\mu_{dyn} = \frac{V_p^2 - 2V_s^2}{2\left(V_p^2 - V_s^2\right)}$$

Lame's Constant,

$$\lambda_{dyn} = \left(V_p^2 - 2V_s^2\right)$$

Bulk Modulus,

$$B_{dyn} = \rho\left(\frac{3V_p^{\,2} - 4V_s^{\,2}}{3}\right)$$

where ρ is the density

The values of ultrasonic elastic constants are often different from those determined by static laboratory methods and in situ methods.

Fairhurst[119] developed a variation of the pulse technique in which measurements are made while the specimen is under a uniaxial load.

Ultrasonic determination of rock properties is useful for preliminary estimation of static properties and evaluation of the effects of uniaxial stress and water saturation on pulse velocity, a knowledge of which is helpful in engineering design.

PERMEABILITY

Permeability of rock is a measure of its ability to transmit fluid under a pressure gradient. Two types of rock permeability must be considered:

1. Primary or original permeability - the permeability existing between undisturbed mineral grains. This is genereally of minor importance but should be determined in a tunnel site investigation.

2. Secondary or induced permeability - due to fractures or dissolved openings. This is of major importance to tunnel site investigations.

PRIMARY PERMEABILITY

Primary rock permeability may be determined by laboratory tests of intact core specimens using either gas or liquid permeameters. Explanation of the use of a gas permeameter is given below.

[119] Fairhurst, C. "Laboratory Measurement of Some Physical Properties of Rock." Proceedings of 4th Symposium on Rock Mechanics, Bull. Mineral Industries Experimental Station, Penn. State University. 1961. pp 105-118.

Standardized procedure for determining gas permeability of core samples are described in American Petroleum Institute publications API RP 27 and API RP 40.

Tests may be conducted on well consolidated, full diameter cores obtained from drill holes or on small cores of 0.75-inch or 1-inch diameter which have been cut from larger cores or a surface sampler. Friable, soft or shaly cores may be mounted in plastic or optical pitch for support. Samples must be dried and cleaned prior to testing. Permeability tests are often run on samples cut in two directions: horizontal (or in the plane of anistrophy) and vertical (or perpendicular to the plane of anistrophy).

Permeability is determined using dry compressed air and an air permeameter that is comprised of a core holder, a pressure regulator, inlet and outlet pressure measuring devices, and a flow rate metering device. The permeameter should be capable of measuring air permeabilities within the range of 0.1 to 500 millidarcys. Dry air permeability is calculated by the formula:

$$k = \frac{2000\, Q_o\, P_o\, L\mu}{A\left(P_i^{\,2} - P_o^{\,2}\right)}$$

where k is the permeability to dry air in millidarcys,

Q_o is the rate of flow of outlet air in cubic centimeters per second,

P_o is the outlet pressure in atmospheres (absolute),

P_i is the inlet pressure in atmospheres (absolute),

μ is the viscosity of the air in centipoises,

L is the length of the sample in centimeters, and

A is the cross-sectional area of the sample perpendicular to the direction of flow in square centimeters.

Calculations may be eliminated by preparing graphs for given pressures. The flow rates are measured and permeabilities then determined by consulting the graph curves.

SECONDARY PERMEABILITY

Most high water flow problems in rock tunnels are caused by secondary permeability, so advance evaluation of this type of problem is one of the most critical parts of a tunnel site investigation. It is also one of the most difficult aspects to assess because of the inhomogeneous distribution and irregularities of water-transmitting discontinuities.

Laboratory test methods are of no value in determining secondary permeability. The various field techniques for evaluating this potential problem are discussed in the ground water hydrology section of In Situ Testing in Appendix F.

CREEP

Creep or time-dependent tests are usually performed by applying a constant uniaxial or triaxial load to a rock specimen and measuring its deformation as a function of time. These tests are more qualitative than quantitative as they mainly determine behavioral characteristics. Many types of time dependent tests have been performed, but no standardization has been adopted.

Common creep testing arrangements are shown in Figure G-12[120].

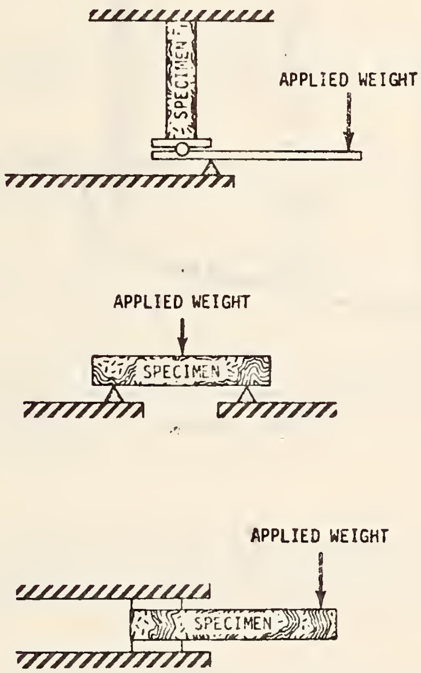

Figure G-12. Creep testing arrangements. (Schematic.)

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[120]Obert, Leonard and Wilbur I. Duval. Rock Mechanics and the Design of Structures in Rock. John Wiley & Sons, New York. 1967

Figure G-13 illustrates an idealized creep curve for rock which includes the transient phase, the steady state phase, and the tertiary phase which culminates in failure.

Very little use of creep tests is made for tunnel site investigations at present.

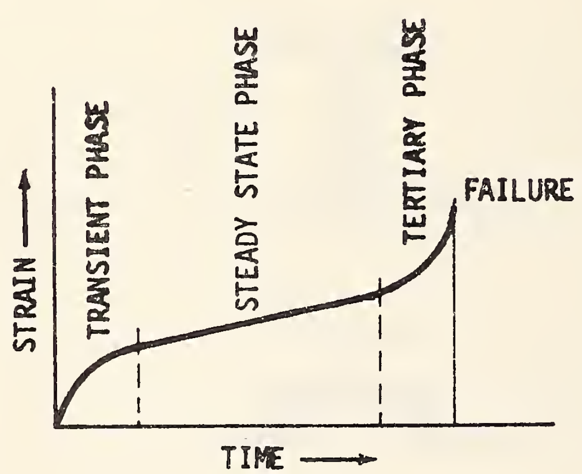

Figure G-13. Idealized creep curve.

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