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FUNCTIONAL CLASSIFICATION OF GOUGE MATERIAL FROM
SEAMS AND FAULTS IN RELATION TO STABILITY PROBLEMS
IN UNDERGROUND OPENINGS

CALIFORNIA UNIVERSITY

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FUNCTIONAL CLASSIFICATION OF GOUGE MATERIALS FROM SEAMS AND FAULTS IN RELATION TO STABILITY PROBLEMS IN UNDERGROUND OPENINGS

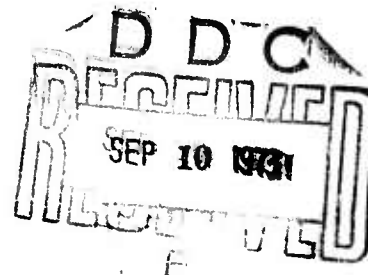
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Tor L. Brekke and Terry R. Howard

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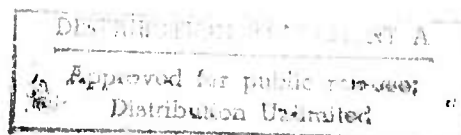


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13. ABSTRACT

The objective of this research effort has been to produce a thorough discussion on the stability problems underground that are associated with faults and seams, and in particular to suggest a functional classification of gouge material. A literature survey was carried out, revealing that while faults and seams constitute a major cause of instability underground, there is only a limited amount of information on behavior characteristics of gouge material. Several construction sites were visited to study the insitu behavior and to sample for a laboratory testing program. The results obtained are reported, and a functional classification system proposed. The fundamental behavior characteristics of different types of gouge is discussed, as is the appropriate construction methods and procedures through seams and faults.

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FROM SEAMS AND FAULTS IN RELATION TO
STABILITY PROBLEMS IN UNDERGROUND OPENINGS

Final Report

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The views and conclusions contained in this document are those of the authors and should not be interpreted as necessarily representing the official policies, either expressed or implied, of the Advanced Research Projects Agency of the U. S. Government.

TECHNICAL REPORT SUMMARY

The objective of this research effort was to discuss underground stability problems associated with geologic discontinuities, to characterize gouge materials, and to suggest a functional classification of such materials.

To accomplish this objective, a thorough literature search was performed, construction sites were visited for studying and sampling gouge materials, engineers and geologists were interviewed, and selected laboratory tests were performed.

The literature study revealed very little data on the description and physical properties of gouge. Case histories of underground construction in both Europe and the United States did, however, point out twelve factors which seem to contribute to underground instability, gouge material being one of the most important.

Samples of gouge materials for laboratory testing were obtained by visiting underground construction projects in progress throughout the United States. The main sampling effort was carried out in the Straight Creek Tunnel (Eisenhower Memorial Tunnel) in central Colorado. An attempt was made to obtain as wide a variety of gouge materials as possible.

The in-situ material behavior was studied in conjunction with the sampling. Observations were made regarding standup time, squeezing behavior, swelling behavior, and drilling and blasting characteristics. Site personnel were interviewed for additional information.

Laboratory tests performed on the gouge samples were selected to provide general characterization of the materials, as well as fundamental properties. The tests performed were Atterberg limits, gradation analysis, free swell, swelling pressures, X-ray diffraction and direct shear tests. These tests provide a means of separating and identifying the different gouge materials as well as indicating the behavior characteristics of the gouge.

Additional strength test results were obtained from the U. S. Bureau of Reclamation. These data seem to indicate that there is nothing unusual in the strength characteristics of gouge materials when compared to the fundamental behavior of soils.

A functional classification system for gouge materials has been developed. Separation of fine grained from coarse grained gouge is based on sample gradation. Gouge samples that have more than 15 percent by weight passing a No. 200 sieve are regarded as either squeezing or swelling type material. Sand-like gouge has less than 15 percent by weight passing the No. 200 sieve. Squeezing ground is separated from swelling ground by having a liquid limit of less than 30. The remaining types, chlorite-talc-graphite-serpentine and calcite-gypsum, are readily identified in hand specimens.

The characteristic behavior of the different gouge types is discussed in detail, as are construction procedures for tunneling through seams and faults.

It is hoped that this report will be helpful in correctly assessing ground conditions involving seams and faults, and in selecting the appropriate construction methods and procedures.

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CONTENTS

TECHNICAL REPORT SUMMARY	ii
ACKNOWLEDGEMENTS	iv
LIST OF FIGURES	vii
LIST OF TABLES	xi
CONVERSION FACTORS	xii
I. INTRODUCTION	1
II. STABILITY PROBLEMS CAUSED BY GEOLOGIC DISCONTINUITIES . .	3
Examples of Slides	3
Factors Influencing the Stability	12
III. CLASSIFICATION OF ROCK MASSES AND DISCONTINUITIES	20
Rock Mass Classifications	20
Classification of Discontinuities	25
Scale	26
Character	31
Strength and Deformability	33
IV. GOUGE - CHARACTERISTICS AND CLASSIFICATION	35
Characteristics of Fault Gouge	35
Faulting	35
Low Grade Metamorphism During Faulting	37
Classification of Gouge	41
V. FIELD INVESTIGATIONS	45
Auburn Damsite, California	45
Nast and Hunter Tunnels	46
Henderson Haulage Tunnel	52
Straight Creek Tunnel	58
General Geology	61
Site Investigations	65

Field Sampling of Gouge	72
Sample Problems	72
VI. LABORATORY INVESTIGATIONS	80
Methods	80
Atterberg Limits	80
Free Swell	83
Grain Size Distribution	83
X-Ray Diffraction	84
Swelling Pressure	85
Strength Testing	88
Penetration Testing	89
Experimental Results	90
Distinction Between Clayey and Sandlike Gouges	92
Chlorite, Talc, Graphite and Serpentine	99
Swelling Clays and Inactive Clays	99
Liquid Limit	100
Plastic Limit	105
Plastic Index and Activity	107
X-Ray Diffraction Results	108
Swelling Pressure Test	110
Swelling Pressure Phenomenon and Testing	118
Strength Testing	128
Direct Shear Testing	129
Penetrometer Testing	129
Test Results from Straight Creek Obtained By the Bureau of Reclamation	132
Tunnel Deformation Measurements from Straight Creek Tunnel	134
Auburn Test Data	138
VII. CONCLUSIONS	140
REFERENCES	148
APPENDIX A - TESTING PROCEDURES	154
APPENDIX B - DESCRIPTION OF FIELD SAMPLES	168
APPENDIX C - RESULTS OF TESTING	174

LIST OF FIGURES

II-1A	Position of Arch-Ribs after Initial Failure Angeles Tunnel	11
II-1B	Position of Arch-Ribs after Complete Failure Angeles Tunnel	11
II-2A	Top Heading Excavated - Castaic Diversion Tunnel	13
II-2B	Top Heading Overbreak - Castaic Division Tunnel	13
II-2C	Removal of Bench Section - Castaic Diversion Tunnel	14
II-2D	Partial Collapse and Block Sliding - Castaic Diversion Tunnel	14
II-2E	Complete Failure - Castaic Diversion Tunnel . .	15
III-1	Definition of "Active Span", ℓ^*	22
III-2	Active Span Versus Stand-up Time.	22
III-3	Classification of Discontinuities According to Scale	28
V-1	Location Map of Nast and Hunter Tunnels	47
V-2	Cross-Section of Nast Tunnel Showing Sample Locations	49
V-3	Location of the Henderson Mine Railroad Haulage Tunnel, and the Straight Creek Tunnel	53
V-4	Generalized Plan and Section of the Henderson Mine Haulage Tunnel	55
V-5	Plan View of Fault Zone, Station 32 + 20, Henderson Mine Haulage Tunnel	57
V-6	Cross Section of the Fault Zone at Station 46 + 03 Henderson Mine Haulage Tunnel	59
V-7	Section of the Straight Creek Tunnel	60
V-8	Example of Variable Nature of Ground - Straight Creek Tunnel	64
V-9	Broken Timbers Caused by High Ground Pressure .	68

V-10	Multiple Drift System, Straight Creek Tunnel, Zone II, Station 82 + 40 to Station 84 + 00 . . .	69
V-11	Multiple Drift System, Straight Creek Tunnel, Zone II, Station 84 + 00 to Station 85 + 60 . . .	70
V-12	Multiple Drift System, Straight Creek Tunnel, Zone II, Station 85 + 60 to Station 100 + 30 . . .	71
V-13	Forepoling Through a Gouge Zone - High Loads Required Extra Bracing of Steel Sets - Straight Creek Tunnel	74
V-14A	Gouge Sample Wrapped in Aluminum Foil - Poly- urethane Foam Liquid Poured Around Sample	77
V-14B	Polyurethane Foam Beginning to Surround the Sample	77
V-15	Diamond Coring of Gouge Sample Surrounded by Polyurethane Foam	79
VI-1	Swelling Pressure Test Apparatus	87
VI-2A	Hydraulic Penetrometer Assembly	91
VI-2B	Hydraulic Penetration Test in Gouge Material	91
VI-3	Gradation Test Summary - Straight Creek Tunnel . .	93
VI-4	Gradation Test Summary - Nast Tunnel	94
VI-5	Gradation Test Summary - Henderson Mine Haulage Tunnel	95
VI-6	Gradation Test Summary - Hunter Tunnel and Auburn Damsite	96
VI-7	Expected Range in Gradation for Running Ground . .	98
VI-8	Clay Water System	103
VI-9	Relationship Between Liquid Limit of the Sample, Liquid Limit of the Clay Fraction, and Percent Clay	104
VI-10	Relationship Between Plastic Limit and Percent Clay	106
VI-11	Relationship Between Plasticity Index and Clay Fraction for Gouge Material	109
VI-12	Relationship Between Activity and Percent Montmorillonite	111

VI-13	Typical Time-Swelling Pressure Relationship	113
VI-14	Swelling Pressure Test Results - Straight Creek Tunnel	114
VI-15	Swelling Pressure Test Results - Nast Tunnel . . .	115
VI-16	Swelling Pressure Test Results - Henderson Mine Haulage Tunnel	116
VI-17	Relationship Between Swelling Pressure and Activity	117
VI-18	Relationship Between Swelling Pressure and Liquid Limit	120
VI-19	Relationship Between Swelling Pressure and Imbibed Water - Sample Exposed to 100% Relative Humidity	122
VI-20	Swelling Pressure Test Sample Preparation - Relationship Between Swelling Pressure and Grinding Time	124
VI-21	Relationship Between Swelling Pressure and Liquid Limit Using the Modified Sample Preparation Procedure	125
VI-22	Relationships Between Swelling Pressure and Imbibed Water Content Using the Modified Sample Preparation Procedure	127
VI-23	Summary of Direct Shear Test Results - Auburn Damsite, Sample No. 1	130
VI-24	Summary of Direct Shear Test Results - Straight Creek Tunnel - Sample No. 6	131
VI-25	Typical Results of In-Situ Penetration Testing Straight Creek Tunnel	133
VI-26	Typical Plot of Tape Measurements - Straight Creek Tunnel	136
VI-27	Compilation of Tape Measurement Data - Straight Creek Tunnel	137
VII-1	Method of Sealing Seams with Reinforced Shotcrete .	146
1A	Soil in Cup of Liquid Limit Device	160
2A	Specimen Before and After Testing in Liquid Limit Device	160

1C-4C	In-Situ Penetration Test Results - Straight Creek Tunnel	179-182
5C-6C	Direct Shear Test Results - Straight Creek Tunnel	183-184
7C-11C	Direct Shear Test Results - Auburn Damsite . . .	185-189
12C-17C	In-Situ Tape Measurement Results - Straight Creek Tunnel	190-195

LIST OF TABLES

II-1	Factors Influencing Actual Behavior	16
III-1	Rockload, H_p , in Feet of Rock	23
III-2	Rock Mass Classification	25
IV-1	Tentative Classification for Fault Gouge Material	42
V-1	Rock Classification, Straight Creek Tunnel	66
VI-1	Atterberg Limits and Indices	81
VI-2	Effect of Angularity and Grading on Peak Friction Angle of Sand	84
VI-3	Atterberg Limits of Clay Minerals	107
VI-4	Clay Mineralogy of Equal Activity Samples	110
VI-5	Factors Influencing the Swelling Pressure of Clayey Gouges	119
VI-6	Swelling Pressure and Tunneling Problems	128
VI-7	Estimated Mineralogical Composition of Rock Samples from Straight Creek Tunnel	135
VI-8	Summary of Mineralogy of Auburn Samples	139
VII-1	Classification of Gouge Material	142
1C	Straight Creek Test Summary	175
2C	Nast-Hunter Test Summary	176
3C	Henderson Haulage Tunnel Test Summary	177
4C	Auburn Damsite Test Summary	178

CONVERSION FACTORS

Units of measurement used in this report can be converted to Standard International (SI) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
inches	2.540	centimeters
feet	0.305	meters
miles	1.700	kilometers
square inches	6.45	square centimeters
square feet	0.093	square meters
acres	4047	square meters
cubic inch	16.4	cubic centimeters
cubic foot	0.028	cubic meters
gallon	3.80	liters
acre-feet	1233	cubic meters
pounds	0.454	kilograms
tons	907.2	kilograms
one pound force	4.45	newtons
one kilogram force	9.81	newtons
pounds per square foot	47.9	newtons per square meter
pounds per square inch	6.9	kilonewtons per square meter

Chapter I

INTRODUCTION

The purpose of the research reported herein has been to produce a thorough discussion of the stability problems underground that are associated with faults and seams, and in particular to suggest a functional classification for gouge material.

It is not uncommon that stability problems associated with faults or seams have been described as "unexpected." This term has unfortunately been used to indicate that no one knew that the fault or seam existed prior to excavation. A careful review of documented cases where serious stability problems have occurred revealed that the existence of these zones most often was known, but that their behavior was incorrectly assessed prior to or during construction. Miscalculations have been made in terms of the type and magnitude of loads that developed, and in particular in terms of the time-dependent behavior.

It is, in this connection, reasonable to note that during the design phase there is often a tendency to over-emphasize the ultimate load that may develop on a support system as the most important design criterion. It is, in fact, very often more important to establish the construction procedure needed to ensure a safe advance through a problem area.

Rock masses are so variable in nature that the chance for ever finding a common set of parameters and a common set of constitutive equations valid for all rock masses is quite remote. Simplified engineering-geological classifications as well as

sophisticated mathematical formulations have in many instances proven to be valuable tools in assessing rock mass behavior. However, they are often given a general validity both in the literature as well as in engineering practice. In many cases, they may be highly inadequate both from the point of view of restrictive assumptions, and from the point of view of the variability of rock masses. Misused in this way, they may be more misleading than helpful, giving a false feeling that adequate design procedures have been followed.

However poor and erratic the knowledge of rock mass behavior is at present compared with the knowledge of most other civil engineering materials and structures, "quantitative" decisions have to be made regarding excavation procedures as well as type and "amount" of reinforcement or support and lining. The choice of correct methods and procedures for driving openings through faults and seams has proven to be among the most difficult decisions in tunneling.

The report is organized in the following way. Chapter II presents, in summary, a review of tunneling problems caused by faults and seams. Chapter III is a review of classification schemes for rocks and rock masses, and for geologic discontinuities. The characteristics of gouge materials are discussed in Chapter IV, followed in Chapter V by a description of the field investigations connected with this study. Chapter VI presents the laboratory testing program used to study gouge samples and the results obtained. Chapter VII gives the conclusions and recommendations.

Chapter II

STABILITY PROBLEMS CAUSED BY GEOLOGIC DISCONTINUITIES

Examples of slides associated with faults and seams in tunnels and other underground openings were sought in the literature in order to determine, as far as possible, the factors that influence the rock mass behavior. Some of these examples are summarized below.

Examples of Slides

Seelmeier (1959) reports on a slide that took place at the face of a 11.4 m^2 tunnel near Hieflauf in Austria. The general rock types encountered were dolomite, slate and limestone. An almost vertical "chimney" with a diameter of approximately one meter formed along the intersection of two faults to a height of 8 meters above the tunnel invert. The gouge material was brecciated limestone in a clay matrix.

Bistritschan (1956) reports on several slides that occurred in the pressure tunnel and diversion tunnel of the Sariyar Hydroelectric Plant in Turkey. The tunnels are situated in Paleozoic sedimentary rocks with some volcanics. Collapses took place in both tunnels due to a fault zone carrying heavily sheared serpentine, partly ground to clayey gouge. At the same place where a 200 m^3 slide occurred while the top heading was driven in the pressure tunnel, a new 1500 m^3 slide occurred during benching.

Brekke and Selmer-Olsen (1965) describe a number of slides in tunnels in Norway associated with the presence of swelling

clay. Five of these case histories are summarized in the following.

One slide developed at Tunnsjødalen powerplant during the excavation of a 35 m^2 tailrace tunnel through a system of thin "fiederspalten" (feather cracks) carrying montmorillonitic gouge. In addition, the feldspars in the wallrock had been altered to montmorillonitic clay. During excavation, the "fiederspalten" seemingly did not represent a problem. In an area with seeping water, however, swelling with ensuing sloughing and sliding started within one week. The recognition of potential problems of this type is often difficult when the gouge and wallrock are dry and appear to have considerable competence. Wahlstrom (1948) mentions several examples attesting to this, as do Rossar (1965) and Morfeldt (1965).

At the Tokke IV powerplant, a montmorillonitic breccia immediately flowed into a 35 m^2 tunnel after blasting; $150\text{--}200 \text{ m}^3$ of this material filled up the face of the tunnel. The tunnel was then driven around the sliding area at a distance of approximately 25 m. When the same zone was encountered again, the montmorillonitic material was considerably more consolidated. This is an example of the variation in consolidation that can be found locally within the same zone.

A 4 km long water tunnel (5 m^2) in Caledonian rocks at Skogn, north of Trondheim, was driven through a zone of montmorillonitic rock that appeared to be competent. However, after one week, sliding occurred, and the tunnel was almost completely filled up with slide masses over a distance of 40 meters.

A slide involving partial collapse of an unreinforced

concrete lining, occurred in the Kvineshei railroad tunnel in 1948, eight years after the tunnel had been finished. A "chimney", with a diameter of 4-6 meters and a height of 34 meters above the railroad tracks, had developed along the intersection of a montmorillonite carrying joint and a brecciated zone of porous calcite and partially altered granitic blocks. It was assessed that the porous calcite, through the years, had dissolved giving space for free swell of the montmorillonitic material. This lubricated the mass in the "chimney". An ensuing earth pressure on the lining, possibly combined with hydrostatic pressure, led to the partial collapse of the lining.

In a headrace tunnel at the Hemsil I powerplant, driven through Caledonian rocks, a dominant system of faults and seams with a direction approximately perpendicular to the tunnel axis was encountered. The thickness of the individual seams and faults varied from under 1 cm to zones up to 8 meters. During the construction period, the presence of montmorillonite in the filling material was realized, and the parts of the tunnel that were considered dangerous were lined with concrete. Smaller and more separate seams were sealed with unreinforced shotcrete. After the tunnel had been in operation for about 8 months, a slide of about 200 m^3 with up to 3 m^3 blocks took place in 1960. A 2-1/2 meter wide section along the tunnel that was inter-woven with thin, chloritic joints between two montmorillonite-carrying seams each a few centimeters thick, had been released along the seams and had slid out and filled the tunnel. The slide went approximately

9 meters up from the tunnel roof, corresponding to the hydrostatic head at this point when the tunnel was filled with water during operation. At several other places in the tunnel, "puncturing" of the shotcrete had also taken place but without causing more serious slides.

Dengerud (1971) presents an example of a major slide in a 65 m^2 water tunnel in Southern Norway. When a major crushed zone was encountered, the gouge that consisted of a chloritic rock with numerous tiny discontinuities with clay coatings immediately ran into the tunnel. The face was bulkheaded, and a realigned tunnel was driven after the zone had been stabilized by freezing.

Detzlhofer (1968) discusses slides in several pressure tunnels. In the 12 m^2 Kauner Valley pressure tunnel in Austria, 5 slides occurred upon de-pressurizing a 1.4 km long test section that was partly reinforced by shotcrete and wire mesh. The crystalline rock mass was intersected by numerous seams and joints, many with clay fillings and slickensides. The sliding volumes for the different slides ranged from 90 m^3 to 1550 m^3 .

Detzlhofer also discusses the failure, found after 7 years of operation, in the Kemano-Kitimat pressure tunnel in British Columbia, originally reported by Cooke et al. (1962). The major failure involved $19,000 \text{ m}^3$ of slide masses. Faults, as well as many minor discontinuities, were filled with easily erodable gouge and partially with soluble calcite. Shotcrete as applied proved inadequate in ensuring long-time stability.

One of the most severe cases of high water inflow associated

with faults occurred in the San Jacinto tunnel near Banning, California (Thompson, 1966). A number of faults were intersected by the tunnel. The hanging-wall side of the faults were heavily jointed and highly permeable. Driving the tunnel towards the hanging walls of the faults did not involve very serious problems since the rock mass was drained over a period of time prior to reaching the fault. However, when the faults were approached from the foot wall-side, no predrainage could take place due to the impervious gouge material in the faults, and sudden inflows of water occurred. The maximum flow from one point reached 16,000 gpm, and from all headings the peak flow was approximately 40,000 gpm. Up to 3,000 cubic yards of crushed material surged in together with water through the ruptured diaphragm represented by fault gouge. Water pressures of 600 psi were measured in a few instances, and pressures ranging from 150 to 300 psi were common.

Trefzger (1966) described the problems associated with gouge material in the Tecolote Tunnel near Santa Barbara, California. The tunnel is 6.4 miles long and 7 feet in diameter and carries water to the coastal area. The problems were associated with squeezing ground which occurred in and near fault zones and in areas of deformed and sheared rock. The Santa Ynez Fault was the major fault encountered requiring replacement of several 6-inch H-beam ribs and struts at 18 inch centers.

Undesirable swelling and squeezing ground was encountered in the Harold D. Roberts Tunnel in Colorado and is described by Wahlstrom et al. (1968). The progress of construction in this 23.3 mile long tunnel and the support requirements were directly related to the intensity of faulting and alteration and to some

extent the joints. The major problems were experienced in driving through several fault zones containing "extensive gouge", accompanied by wall rock alteration and high water pressure. The squeezing continued for several months after exposure, requiring continual maintenance. Of the difficulties Walhstrom said: "In these sections considerable time, money, and delay could have been avoided if the subsequent behavior of the rocks had been anticipated at the time they were exposed in the tunnel heading. In retrospect, if field tests . . . had been made, sufficient support could have been installed as a part of the routine tunneling operation to maintain the tunnel opening safely until the lining was placed."

A. B. Arnold et al. (1972) reported on three case histories of tunnel support failures in the California aqueduct system. The report is summarized here.

The Carley V. Porter tunnel, completed in 1970, is a concrete lined tunnel with a finished diameter of 20 feet. The tunnel was driven through highly fractured to crushed igneous and metamorphic rocks of the Garlock fault zone, deeply weathered to altered and faulted granite, and soft, faulted lakebed deposits of claystones, siltstones and mudstones. A collapse, which occurred over a 42 foot portion of the tunnel, took place in the faulted lakebeds.

The initial excavation through the lakebeds was rather complicated and resulted in a 4 foot vertical misalignment of the tunnel necessitating re-mining to bring the tunnel back to grade. To do this the initial circular steel liner plates were modified to a weak elliptical section. The collapse took place in this

area. It was noted that high squeezing pressures were developing, causing, in some places, deflections of up to 1 foot in the steel liner. This high ground pressure and a rapid rate of squeeze continued throughout the remining period and destroyed the breast-board operation several times.

The ground in this area was described as a "stiff, cohesive plastic clay fault gouge with slickensides." Test results accompanying the report tend to substantiate that description. The results also indicate a fairly high montmorillonite content and associated high swelling pressures.

The authors give the following three reasons as the apparent cause of failure of the support system:

1. The mobilization of the faulted lakebed deposits due to remining operations.
2. The structural weakness of the remined liner plate sets.
3. The presence of a prominent plane of weakness in a N 55° W trending fault.

The Angeles Tunnel is a circular-shaped 30 foot diameter pressure tunnel that was driven by the top heading and bench method. The rocks in the area of support failure consist of soft dark grey thinly bedded siltstones interbedded with sandstone. Three faults with associated shearing had fractured the rock mass, creating a partially to completely crushed material.

In the top heading the tunnel support consisted of arch ribs on 5 foot centers resting on wallplates consisting of two 15I50 steel members welded together with gusset plates and with sack concrete footings.

Failure initiated in the right wallplate in an area where the wallplate was founded in a 3 foot wide fault zone dipping 60 degrees as shown in Figure II-1A. The failure was attributed to sloughing away below the wallplate of the weak faulted material. Squeezing of the ground in this area seemed to accompany the sloughing and probably accelerated the process.

The downward movement of the arch ribs removed support from the crown area. This activated the squeezing faulted ground and complete collapse occurred, with a slide extending approximately 1-1/2 tunnel diameters above the tunnel as shown in Figure II-1B.

The failure in the 27-foot diversion tunnel for the Castaic Dam is an example of failure along clay-coated bedding planes. The failure occurred over a 70-foot-long section and consisted of a slide extending approximately 70 feet above the crown.

The tunnel was excavated through the Castaic formation, composed of interbedded sandstone and shale. The sandstone is fine-grained and has a short standup time due to a weak clay cementation. The shales are low to moderately hard and, when exposed, slake to a lean clay. Thin, soft, plastic clay seams or slickensides resulting from adjustments during folding were often present along bedding planes. Shears and faults that contained soft clay were also present.

The failure occurred over a week's time and the sequence of events is illustrated in Figures II-2 A-E. Figure II-2A shows the initial mining position of the top heading and the

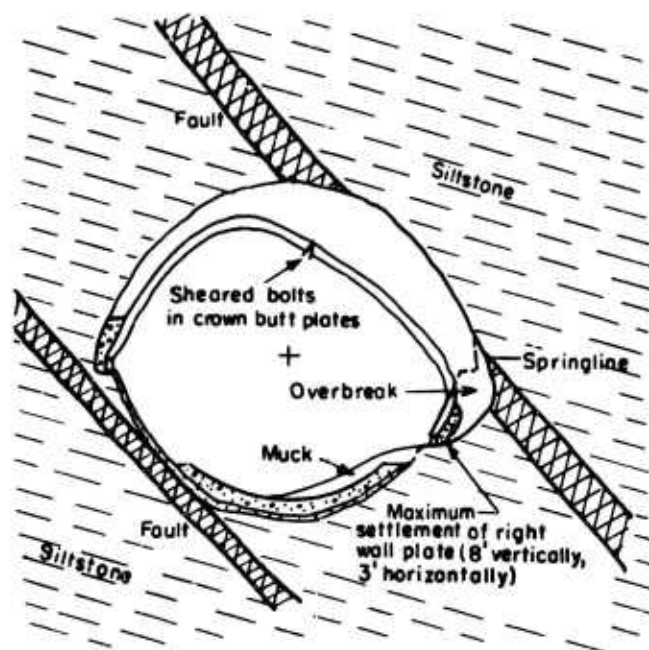


FIGURE II-1A POSITION OF ARCH-RIBS AFTER INITIAL FAILURE - ANGELES TUNNEL. (after Arnold et al., 1972)

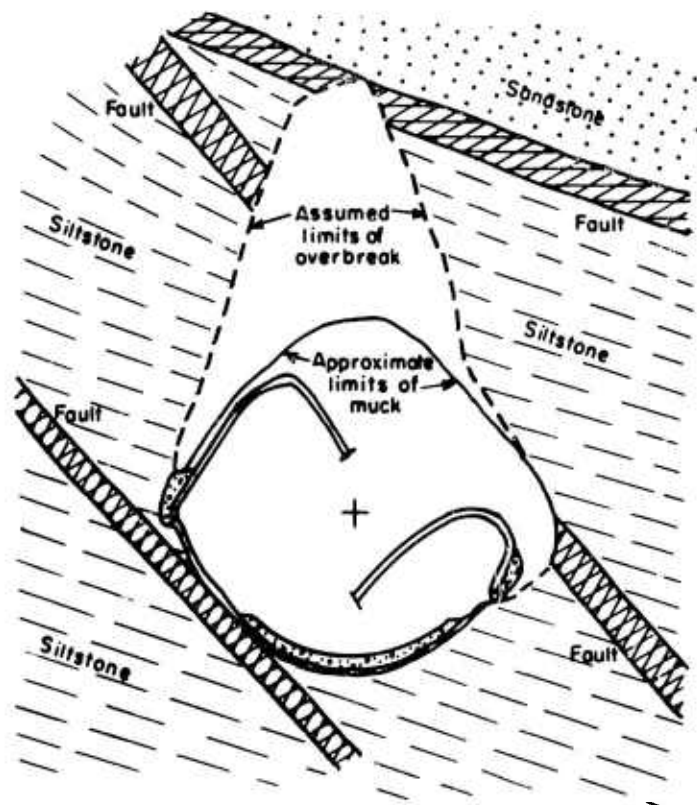


FIGURE II-1B POSITION OF ARCH-RIBS AFTER COMPLETE FAILURE - ANGELES TUNNEL. (after Arnold et al., 1972)

position of the initial overbreak. The rock was a saturated sandstone. This overbreak was not adequately supported and was left unattended over the weekend, causing a portion of the face to collapse and the overbreak to grow larger.

The overbreak continued until it was approximately 25 feet above the crown at which time it was cribbed, blocked with timber and backpacked with pea gravel. As shown in Figure II-2B there was apparently a void area remaining which allowed continued rock deformation.

The bench section was then removed as shown in Figure II-2C. The left wall plate deflected prior to placement of the posts. The left wall was intersected by a 1/4 to 1/2 inch thick clay-filled seam that dipped approximately 45 degrees into the tunnel causing high lateral loads near the left foot block.

The collapse occurred when the final three feet of the invert was removed as shown in Figure II-2D. Evidently the reduced bearing strength allowed the footblocks to fail. Subsequently, a large rock block slid down along the clay seam. Complete failure occurred shortly thereafter as shown in Figure II-2E. The drillhole shown was drilled later in order to assess the extent of the slide.

Factors Influencing the Stability

From the case histories summarized above and from other studies and observations, it is quite clear that the stability problems that faults and seams may cause underground are influenced by a number of factors. All of these interplay in the final behavior of the rock mass. It is therefore somewhat

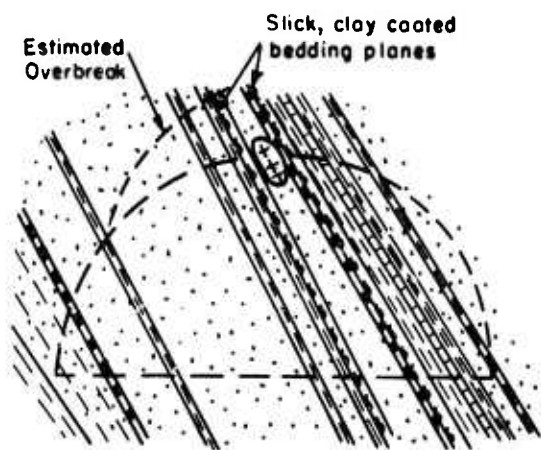


FIGURE II-2A TOP HEADING EXCAVATED - CASTAIC DIVERSION TUNNEL. (after Arnold et al., 1972)

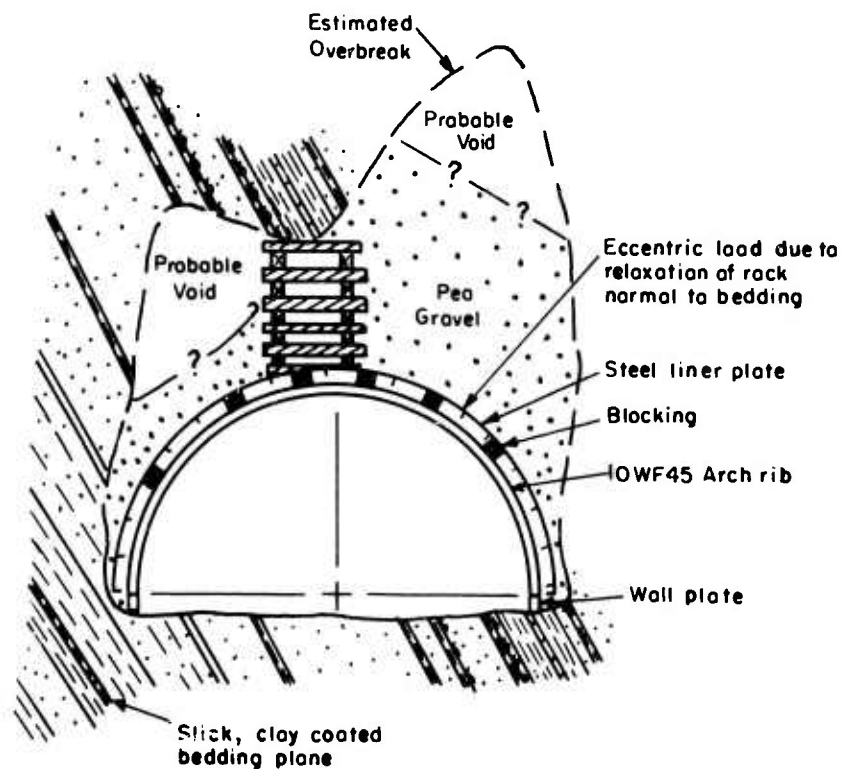


FIGURE II-2B TOP HEADING OVERBREAK - CASTAIC DIVERSION TUNNEL. (after Arnold et al., 1972)

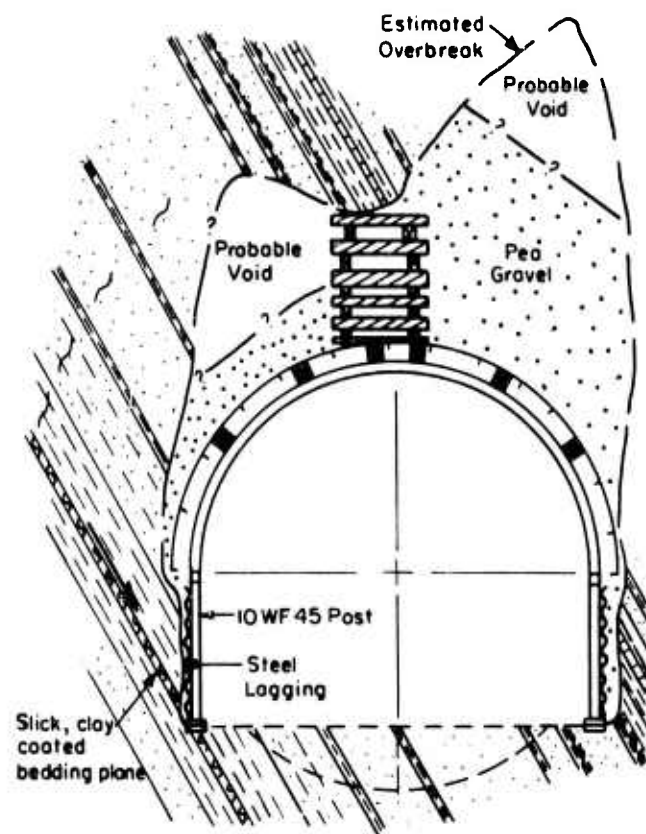


FIGURE II-2C REMOVAL OF BENCH SECTION - CASTAIC DIVERSION TUNNEL. (after Arnold et al., 1972)

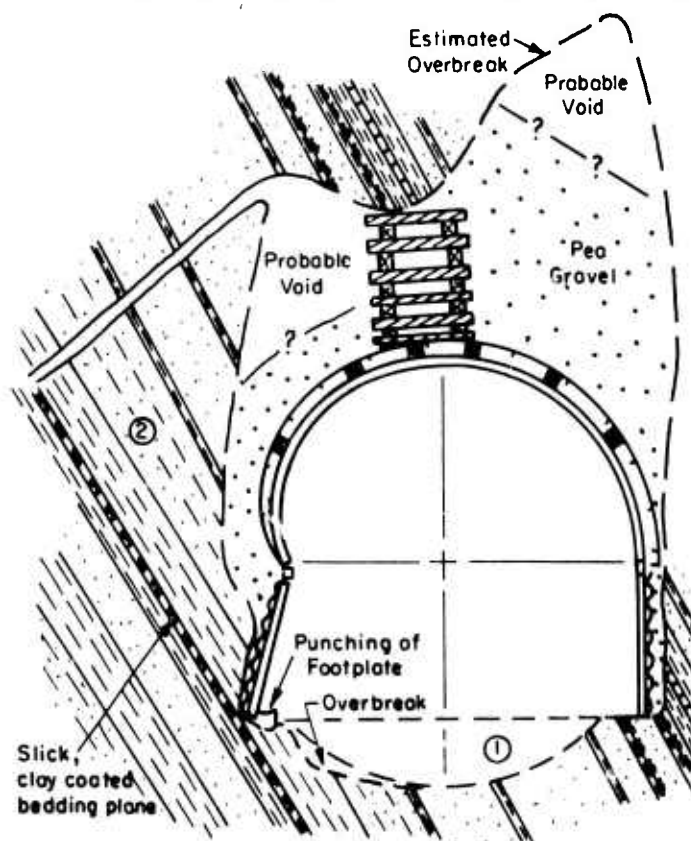


FIGURE II-2D PARTIAL COLLAPSE AND BLOCK SLIDING - CASTAIC DIVERSION TUNNEL. (after Arnold et al., 1972)

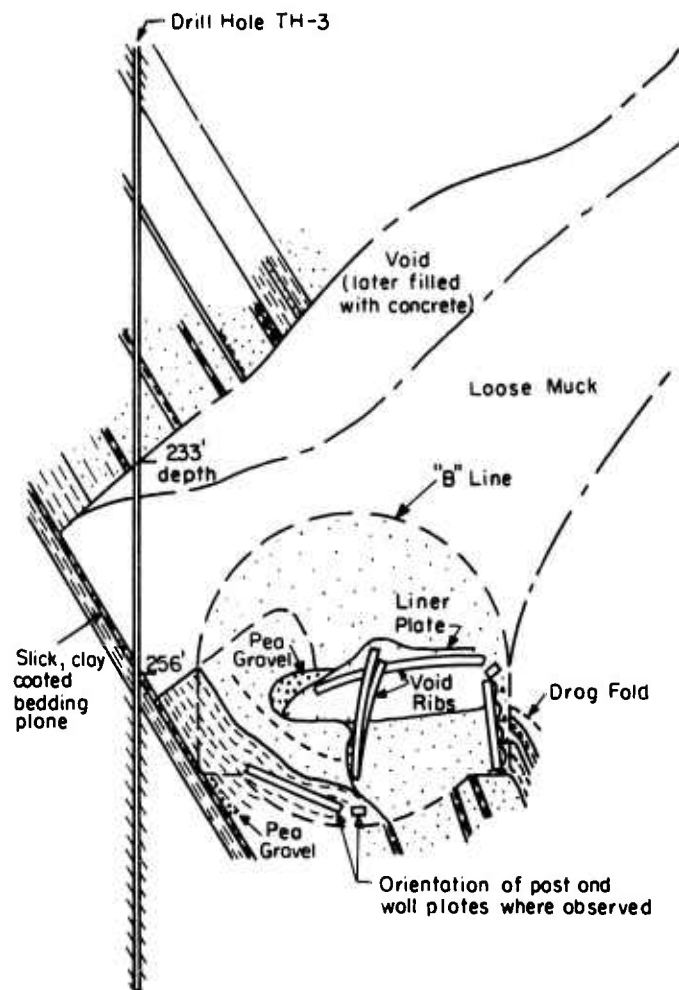


FIGURE II-2E COMPLETE FAILURE - CASTAIC DIVERSION TUNNEL
(after Arnold et al., 1974)

dangerous to isolate the different factors without viewing their influence in an overall context. However, it may be helpful to list in summary some of the observations that can be drawn from documented experience regarding the influence of the different factors. This is done in Table II-1. Several of the factors will be discussed in more detail in later chapters.

Table II-1

FACTORS INFLUENCING ACTUAL BEHAVIOR

The size and shape of the opening.
 The method of excavation.
 The method of support, reinforcement or lining.
 The orientation of the opening in relation to
 the strike and dip of the fault or seam.
 The width of the fault or seam.
 The existence of adjacent seams and faults
 (if any).
 The frequency, orientation and character of
 adjacent joint sets.
 The competence of the wall rock.
 The gouge material.
 Time elapsed after excavation
 The in-situ state-of-stress.
 The water regime.

One group of factors relates to the opening itself, its size and shape, the method of excavation, and the method of support or reinforcement.

The size of the underground opening influences the standup time and the ultimate loads that may develop. While the standup time is strictly a construction problem, the ultimate load that may develop can influence stability during construction as well as during operation.

Terzaghi (1946) found that generally the loads will increase linearly with size of the opening except in swelling ground

where high swelling pressure may develop regardless of the size of the opening. Either through the use of Terzaghi's assessment of rock loads, or more recently through the adoption of the so-called Kastner formula (Kastner, 1962, Labasse, 1949, 1950) the effect of size has been considered in design of the support and lining systems as far as the ultimate loading is concerned.

From a review of the literature, it seems quite clear that the effect of the size on the standup time of gouge material is much more significant. The problems encountered increase drastically as the standup time decreases.

The method of support or reinforcement influences the outcome by how quickly they can and are applied after excavation, and the amount of deformation that they will allow.

The orientation of an opening in relation to the strike and dip of an individual fault or seam can have a considerable influence on the stability of the opening. In general, the problems increase as the strike becomes more parallel to the opening. However, even if the strike is across the opening, low-dipping seams and faults can represent a hazard. If the opening is driven from the hanging-wall side, the discontinuity will first appear at the invert and it is most often possible to prepare for adequate support or reinforcement when driving through the rest of the zone. If, on the other hand, the opening is driven from the foot wall side, the zone will first appear in the crown, with the possibility that the wedge defined by the zone and the tunnel will fall in without warning.

Seemingly, the problems associated with a fault or seam would increase with the width of the discontinuity. However, the width should always be assessed in relation to the attitude of the discontinuity and to the frequency, orientation, and character of adjacent joint sets, the existence of adjacent seams or faults (if any), and the competence of the wall rock type. Several severe slides in tunnels have occurred where each individual seam or fault has been of a small width, but where the interplay between several seams and faults has led to instability.

It was noted in several of the case histories summarized earlier in this chapter that the influence of water in terms of swelling, dissolving, and outwash is not necessarily obvious during construction. Therefore, a long-term effect may easily be under estimated during the construction period, and the measures taken to insure long term stability prove inadequate.

A real hazard exists where large quantities of water in a permeable rock mass are released when an impervious fault gouge is punctured through excavation. In this instance, large quantities of gouge and rock can be washed into the opening when the water inflow occurs.

The state-of-stress in a rock mass will not necessarily be reflected in the density of gouge material. This seems particularly true when studying gouge on a "local" scale. On a larger scale, and in particular for very wide fault zones, it must be expected that the in-situ stresses in the gouge increase with depth and also will reflect tectonic or remanent stresses in the area. Therefore, the loads that may develop through squeezing must be expected to be related to the overall in-situ

stress for wide fault zones.

Finally, the stability of an opening will depend on the character of the gouge material.

Chapter III

CLASSIFICATION OF ROCK MASSES AND DISCONTINUITIES

Rock Mass Classifications

One of the first efforts to classify rock masses in terms of the rock loads that may be mobilized against support in an underground opening was made by Bierbaumer (1913) in Leipzig. Between the World Wars the development in Central Europe was led by Professor Stini; his textbook on tunnel geology (1950) includes a classification of rock masses, and a very detailed and well documented treatise on adverse conditions in tunnelling. More recently, classification systems have been suggested by Rabcewicz (1957) and Lauffer (1958).

Rabcewicz's work is known in the United States through several papers in "Water Power" describing the so-called Austrian tunnelling method (Rabcewicz 1964, 1965, 1969). The basic principle of this method is to install reinforcement or support as fast as possible after excavation in order to secure that the inward movement of the rock mass is kept at the minimum. The method has sometimes been assumed to be synonymous with the use of shotcrete, while it in general includes any measure that can rapidly be applied close to the face.

The major contribution by Lauffer (1958) is his emphasis of the importance of "stand-up time" relating for different classes of rock masses the time an opening can stand unsupported to the "active span." The active span is the width of the tunnel or the distance from support to the face in case this is less than the width of the tunnel as shown in Figure III-1.

Figure III-2 gives the relationship as found by Lauffer, mainly based on experience from the Alps. The main point that should be made in regard to this chart is that an increase in tunnel size leads to a drastic reduction in stand-up time since the allowable size of the face obviously must be related to allowable active span. Thus, while a pilot tunnel may successfully be driven full face through a fault zone it may prove impossible, in terms of stand-up time, to drive a large heading through the same zone, even with the help of spiling and breastboarding. Multiple drift systems, e.g. like those used in the Straight Creek Tunnel in Colorado, may be the only feasible approach in such a case.

Other major contributions from Central Europe to the understanding of rock mass behavior as related to underground openings include the work of Zaruba and Mencl (1961), Kastner (1962) and Müller (1963).

In Scandinavia, where extensive use of underground openings has been made during the last thirty years, classification systems that either fully or in part refer to tunnelling conditions in Sweden have been suggested by Bergmann (1965); Hansagi (1965) and Hagerman (1966). In Norway, Selmer-Olsen (1964) and co-workers have used a more "individual" approach, assessing as detailed as possible the extent of particular problems to be met in each underground opening, and ascribing in each case the remedial measures that seem necessary and sufficient. This approach has proven to be the most rewarding both technically and economically under the generally good tunnelling conditions in Norway.

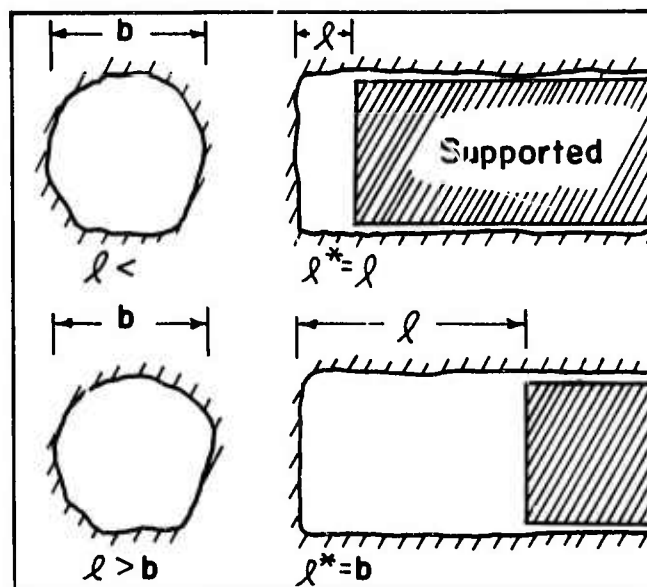


FIGURE III-1 DEFINITION OF "ACTIVE SPAN", l^* . (after Lauffer, 1958)

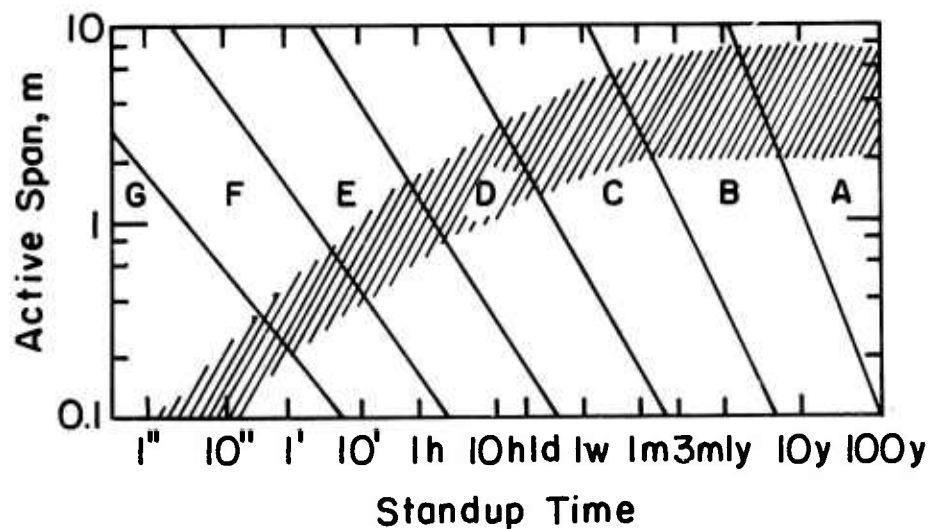


FIGURE III-2 ACTIVE SPAN VERSUS STAND-UP TIME. A - BEST ROCK MASS, G - WORST ROCK MASS. SHADED AREA INDICATES THE PRACTICAL RANGE. (after Lauffer, 1958)

In the United States, the classification system proposed by Terzaghi (1946) has for the past 25 years been completely dominant, either in its original form or with some modification. Terzaghi identified nine different rock conditions and assigned each a rock load factor (H_p , in feet) that when multiplied by the density of the rock mass yields an ultimate design load for steel supports. His rock conditions and the corresponding rock loads are given in Table III-1.

Table III-1

ROCKLOAD, H_p , IN FEET OF ROCK

(after Terzaghi, 1946)

B = width of tunnel, H_t = height of tunnel

	Rock Load H_p in feet
1. Hard and intact	zero
2. Hard stratified or schistose	0 to 0.5 B
3. Massive, moderately jointed	0 to 0.25 B
4. Moderately blocky and seamy	0.25 B to 0.35 $(B+H_t)$
5. Very blocky and seamy	(0.35 to 1.10) $(B+H_t)$
6. Completely crushed but chemically intact	1.10 $(B+H_t)$
7. Squeezing rock, moderate depth	(1.10 to 2.10) $(B+H_t)$
8. Squeezing rock, great depth	(2.10 to 4.50) $(B+H_t)$
9. Swelling rock	Up to 250 ft irrespective of value of $(B+H_t)$

Perhaps the most important element of Terzaghi's work is that he recognized the influence of the size of the opening on the

ultimate load that may develop. The load factors are, strictly speaking, only valid for steel supports or similar "passive" systems. They should not be directly applied to "active" systems, e.g., bolting. It should further be noted that Terzaghi's load assessments for blocky and seamy rock are quite conservative as he takes into consideration high water "loads" when the tunnel is situated below the ground water table. These "loads" are seldom present in practice (Brekke, 1968). The rock conditions that involve faults or seams are "Moderately Blocky" and "Seamy through Swelling Rock".

More recently, a general rock mass classification system suggested by Deere (1966) has found wide-spread use in the planning of underground openings as well as other rock structures in the United States. A Rock Quality Designation (RQD) is assigned to a rock mass based on a modified core recovery including in percent of length drilled only the parts of the core that occur in pieces that are 4 inches long or longer. Obviously weak material (e.g. gouge) is disregarded even if it appears in an intact piece of a core that is 4 inches long or longer. The RQD and the corresponding Rock Quality Description are presented in Table III-2.

Each one of the classification systems available has tried to meet the following requirements:

- (1) being simple and meaningful in terminology.
- (2) being based on parameters that can be measured or assessed rapidly and inexpensively.

Table III-2

ROCK MASS CLASSIFICATION
(After Deere, 1966)

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

(3) being exact enough to yield quantitative data that can readily be applied in engineering design.

Most systems have proven to be of great value in geological engineering when carefully used, considering the conditions that are specific to each individual site. On the other hand, most of them are continuously misused because the premises for and assumptions made in developing the classification systems have not been carefully studied by the users, and because they have been given a validity for "quantification" of rock mass behavior that is far more general than was intended by their authors.

Classification of Discontinuities

Discontinuities are formed through failure in extension/tension, in shear, or in more complex failure through a combination of both. Unless subsequent alteration of the wall rock has taken place, rupture surfaces formed in extension or tension are characteristically rough and clean with little detritus. If such surfaces are pressed back together and a direct shear test performed, the resulting load-

deformation curve would display a peak rising well above the residual strength which would only be reached at large values of displacement (Goodman, Taylor, and Brekke, 1968).

Simple surfaces of shearing are characteristically smooth with considerable detritus. A direct shear test along the surface would not yield as great a contrast between peak and residual strength as in the case of extension fracture. Furthermore, discontinuities formed in shear are in general much more susceptible to alteration than are those formed in extension. Normally, such alteration leads to a lowering of the strength and to other disadvantageous conditions for construction.

There are no distinct and generally accepted rules or nomenclature for classification of discontinuities for engineering purposes. However it seems reasonable to make use of three criteria for classification:

SCALE, based on aperture, persistence and typical spacing.

CHARACTER, based on smoothness and the properties of filling material or coatings if any.

STRENGTH AND DEFORMABILITY, based on measured values obtained in laboratory and/or field testing.

Scale

Selmer-Olsen (1964) makes a distinction between the "gross discontinuity pattern" and the "detailed discontinuity pattern". This is from a functional point of view a very important distinction. The gross pattern, involving major zones of weakness, should be studied and assessed in order to find an optimum placement of the most sensitive parts of a system of underground openings; e.g.

siting the major openings in an underground powerplant away from major discontinuities.

The detailed pattern, involving prevalent sets of minor discontinuities, should be studied and assessed in terms of finding an optimum orientation and, sometimes, shape of the major openings involved. Other factors may also influence this decision, notably the in-situ stress conditions and functional considerations.

In order to present a complete discontinuity classification based on scale it seems reasonable to include microfissures as well as bedding planes and foliation partings although they are not true discontinuities. Likewise, fractures induced by blasting or formed through buckling in highly stressed areas should be included.

A classification based on scale was presented last year (Annual report, 1972, U.S.B.M. - ARPA Contract H0210023) and was published the same year (Brekke and Howard, 1972). This classification has now been revised, principally because it was not compatible with standard geologic terminology. The revised classification, expanding from Selmer-Olsen's basic description is shown in Figure III-3.

Microfissures are "internal" defects or flaws that influence the strength and deformability characteristics of the rock involved.

Bedding planes and foliation partings are separations that define weakness directions in the rock. Since they are often tight and rough, they may not be of major concern unless their orientation alone or in combination with other discontinuities

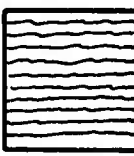
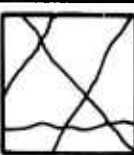
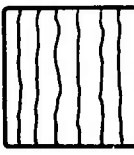
GROUP	TYPICAL DISCONTINUITIES	TYPICAL SCALE
Rock Type Defects	Microfissures	0.2' 
	Bedding Plane Partings Foliation Partings	2' 
Detailed Discontinuity Pattern	Joints Minor Shears Minor Seams	20' 
	Shears Seams	200' 
Gross Discontinuity Pattern	Major Shears or Crushing Zones	2000' 
	Regional Fault Zones	20,000' 
Induced Fractures	Buckling or Spalling Fractures	0.2' - 20' 

FIGURE III-3 CLASSIFICATION OF DISCONTINUITIES ACCORDING TO SCALE.

is unfavorable in relation to the orientation and shape of the underground opening. For instance, close horizontal partings in a tunnel roof may necessitate a rather pointed cross-sectional profile and bolting of the flanks. Bedding plane partings, and in particular foliation partings may be developed to joints or shears. They should then be classified as such.

Joints have a great variety in aperture, persistence, frequency and character. By geologic terminology, they are discontinuities formed in extension or tension, without any perceptible shear movement. The aperture may vary from tight to a fraction of an inch, the persistence from less than a foot to hundreds of feet; exceptionally a mile (tectonic sheet joints).

A typical feature of joints (and also often of shears and seams) is that they most often occur in sets. In each set, the joints have approximately the same strike and dip, and usually the same character at least over a certain area, e.g. a few square miles. The joint system of a rock mass is usually made up of three or four sets. One or two sets can be completely dominating in frequency and/or persistence. A few "wild" joints and seams not belonging to the joint sets are not uncommon, and it also happens that the jointing is mosaic, i.e. there are apparently no distinct sets present in the joint system. Granitic rock masses may have a mosaic joint pattern but will more often have a joint system that yields roughly cubic rock blocks. Regional metamorphic rocks often have a system yielding rhombic rock blocks. Thick lava

beds often have columnar jointing yielding hexagonal, prismatic rock blocks. In folds, sets often develop parallel to the bedding, perpendicular to the bedding, radially along the fold axis, and perpendicular to the bedding and the fold axis.

Sheet joints or lift joints are spalling features resulting from extension failure due to unloading (e.g. erosion of overlying masses) or very high residual stresses due to other causes. They are best developed in massive rock masses, such as the granitic bodies of the Sierra Nevada or Jurassic sandstones of the Colorado Plateau. They tend to be parallel to the surface, and their frequency decrease with depth, in some cases abruptly.

A particular kind of joint often found in regional metamorphic rock masses is the so-called "fiederspalten" or "feather cracks". They are tensile ruptures that often represent the first stage in the development of a fault.

Shears are, as the name infers, formed in shear through faulting. They have a wide variation in scale as indicated in Figure III-3. The term seam indicates an often clay-filled zone with a thickness of a few inches. When occurring as a weak clay zone in a sedimentary sequence, a seam can be considerably thicker. Otherwise, seams may represent altered zones along joints, dikes, beds or foliation. Seam is often used as synonymous with shear, and the detailed discontinuity pattern is referred to as "joints and seams."

Faults are the major rupture zones encountered. Minor faults range in width from several inches to several feet; major faults from several to hundreds, occasionally thousands of feet.

In geology, great attention is given to the relative displacement along fault zones, and the faults are basically classified as normal, reverse, lateral and thrust faults. From an engineering point of view, excluding seismic considerations, this classification may not be of great importance. More important is the realization that most fault zones are the result of numerous ruptures throughout geological time, and that they quite often have other parallel discontinuities that decrease in frequency and thickness when moving away from the central zone.

Character

It is just as important - often more important - to classify discontinuities according to character as it is to note their scale parameters. For the detailed discontinuity pattern, the first distinction can be made between rough versus smooth surfaces. The latter will most often be much more severe from an engineering point of view than a rough discontinuity. Secondly, it is important to distinguish between major types of coating and/or filling materials. It seems relevant to discuss seven groups.

(1) Joints, shears and seams sometimes may be healed through precipitation from solution of such minerals as epidote quartz, or calcite. In this instance, the discontinuity may be "welded" together. Such discontinuities may, however, have broken up again, forming new surfaces. Also, it should be emphasized that such minerals may well be present in a discontinuity without "healing" it.

(2) Clean discontinuities i.e., without fillings or coatings. Many of the "rough" joints or partings have this favorable character. Close to the surface, however, it is imperative not to

confuse clean discontinuities with "empty" discontinuities where filling material has been leached and washed away due to surface weathering.

(3) Calcite fillings may, particularly when they are porous or flaky, dissolve during the lifetime of an underground opening. Their contribution to the strength of the rock mass will then, of course, disappear. This is a long-term stability (and sometimes fluid flow) problem that can easily be overlooked during design/construction. Gypsum fillings may behave the same way.

(4) Coatings or fillings of chlorite, talc, graphite or serpentine make discontinuities very slippery, i.e. low strength, in particular when wet.

(5) Inactive clay material in seams and faults naturally represents a very weak material that may squeeze or be washed out.

(6) Swelling clay may cause serious problems through free swell and consequent loss of strength, or through considerable swelling pressure when confined.

(7) Material that has been altered to a more cohesionless material (sand-like) may run or flow into the tunnel immediately following excavation.

It should be emphasized again that the character of the discontinuities is at least as important as frequency from an engineering point of view. Thus, the frequency of discontinuities per se is not a sufficient basis for evaluating the behavior of a rock mass. A much more complete classification, and at the same time, short and meaningful, would be, for example, " slick,

chloritic, persistent joints spaced 2 ft - 4 ft", or "1 inch seam carrying highly consolidated swelling clay and porous calcite."

Strength and Deformability

The shear strength and deformation characteristics of joints are presently being studied by R. E. Goodman and co-workers at the University of California. Their investigation includes variations in normal stress, roughness, thickness or filling, shear rate, and water pressure. Ohnishi (1973) has obtained laboratory results that indicate the pore pressure in joints can significantly influence the behavior of jointed rock masses.

Through the development of the so-called "joint element" (Goodman, Taylor, and Brekke, 1968) it became feasible, by finite element analysis, to study the behavior of jointed rock masses. Originally, the element behavior was defined as having no strength in tension, a shear strength (cohesion and friction), a normal stiffness, and a shear stiffness. A classification system was suggested on the basis of these parameters. Recently, dilatancy has been included (Dubois and Goodman, 1971).

The joint element is one-dimensional. As such, it is quite adequate for modeling the detailed discontinuity pattern in a "blocky" rock mass. For a stability analysis of a tunnel through a fault zone, however, it seems from a scale point of view more realistic to model the zone as a two-dimensional, preferably three-dimensional continuum. The major problem is to determine the correct constitutive equations for the behavior of the gouge material. The shear test data and the field data presented later

can hopefully contribute to some extent to the solution of this problem.

Chapter IV

GOUGE - CHARACTERISTICS AND CLASSIFICATION

Characteristics of Fault Gouge

The term gouge as used herein is to include all materials within a fault zone or seam regardless of whether the material has been altered to a different mineralogy during or after faulting. The gouge material in a given zone often reflects several stages of genesis throughout geologic time, and is therefore often quite complex in composition and properties.

Using the above definition of gouge there are four means or combinations thereof by which gouge can be formed.

1. Faulting
2. Low grade metamorphism during faulting
3. Hydrothermal alteration
4. Weathering

Faulting

The mechanics of faulting have been thoroughly discussed in the literature (e.g. Price, 1966, Billings, 1959) and will not be presented in detail here. However, some comments will be made to help point to the type and character of gouge materials likely to result from the physical effects of faulting alone.

The causes of shear stresses that result in faulting are many and lead to many possible combinations of relative movement. The type of faulting determines to a large extent the amount of crushing and grinding associated with the faulting. The amount of grinding is a function of the amount of movement

and the magnitude of effective compressive stresses across the fault plane. Under high hydrostatic pressure, the crushing or grinding may be minimal.

The type of wall rock also helps determine the results of shearing along a fault plane. In competent and brittle rocks such as granites and some sandstones, brecciation is likely to occur. In this case, the gouge will consist of relatively large, angular fragments of broken rock surrounded by somewhat finer material. These zones are usually highly permeable and are therefore highly susceptible to further alteration. High water inflow problems are sometimes associated with this type of gouge when encountered in underground construction.

Less competent rocks, such as shales and some of the basic igneous rocks, behave much more plastically under shearing stress and generally mylonitization results. Mylonites are extremely fine-grained crushed rock, densely cemented together. Faults in this type of rock usually contain slickensides and have a greater number of shear surfaces causing a larger zone of failure. Faults in more competent rock tend to be confined to a few failure planes. The amount of displacement also controls the thickness of a fault zone. Higher displacements cause thicker zones.

In general faulting involves a combination of the following: flattening, grinding, abrasion, granulation, pulverization, mylonitization and chemical alteration. It is therefore reasonable to expect that a given fault zone may contain highly permeable brecciated gouge adjacent to highly impermeable clay gouge.

Low Grade Metamorphism During Faulting

Regional metamorphism results from the combined effects of temperature, pressure, and shearing stresses (Moorhouse, 1959). It is possible, then, due to the heat and pressure generated during faulting, to cause a low-intensity metamorphism of the gouge. This means that the minerals present are altered to minerals which are stable under the pressure and temperature associated with faulting.

The product of metamorphism depends not only on the temperature, pressure and shear stresses, but also on the available minerals in the parent rock. For example, in a highly sheared zone, a basalt might become a chlorite schist made up predominantly of chlorite, with some sericite, carbonate, quartz and albite, while sandstones, carbonates and acid igneous rocks can be expected to undergo few changes in mineralogy.

It is then possible with knowledge of the rock type involved, to predict to some extent the mineral assemblage which could be expected to be present in a fault zone. In this light it should be noted that the minerals associated with low grade metamorphism, often are of questionable quality for engineering works. Care must be taken in predicting the mineral composition of a fault zone because the minerals formed during the faulting may later have been subject to alteration with clay gouge as an end product.

Hydrothermal Processes. Hydrothermal solutions are mineral bearing liquids at a high temperature (50°C - 500°C) and generally high pressure. These solutions are believed to originate from deep-seated magmas at a high temperature, and as they move through

the surrounding rock, they alter it either by the deposition of minerals from solution or by replacement of minerals in the country rock.

The solutions are acidic when leaving the magma and will therefore attack most minerals except quartz and alkali feldspar (Bateman, 1950). Carbonates and basic igneous rocks are very susceptible to attack, and this reaction would cause hydrothermal solutions to become more alkaline. Temperature also decreases with distance from the magma, causing a decrease in solubility and precipitation of minerals. In some cases the fractures are completely filled with a sound mineral, such as quartz, and the fractures are said to be "healed". In other cases, the mineral deposited may be one of the sulphides such as pyrite which, upon oxidation, produces sulphuric acid which may attack and weaken the wall rock or a concrete structure. Calcite is also a common mineral and is considered detrimental when porous or flaky. Thus, from an engineering standpoint, the invaded rock may be improved or deteriorated depending upon the mineral deposition.

Hydrothermal mineral deposits are generally accompanied by a band of alteration of the wall rock. The nature and intensity of the alteration depend on the wall rock, the chemical character of the solution and the pressure-temperature relationship. It is this process of alteration of faulted rock that is of important engineering significance because alteration generally leads to the production of clay minerals. In fact, practically all the various clay minerals have been reported in hydrothermal bodies (Forbes, 1949).

In most cases, the alteration products are mixtures of clay minerals. However, a few cases have been reported in which the hydrothermal body is composed substantially of one clay mineral. The problem then is predicting the type of clay that could possibly be present.

Much of the information published on wall rock alteration is in connection with ore bodies as an aid in prospecting (e.g. Bateman, 1950). If hydrothermal alteration is intense (high temperature, long duration) all alteration products will tend to be the same regardless of their parent rocks, except for carbonate and quartzitic rocks. Limestones tend toward silicification, and dolomites tend toward the magnesium-rich clay minerals.

If hydrothermal alteration is slight, the primary minerals determine the end product. The magnesium-rich minerals (hornblende, biotite) change to chlorites. In the presence of alkalis, except potassium, the micas, the ferromagnesium minerals, and plagioclase alter to montmorillonite. The presence of magnesium in solution favors the formation of montmorillonite and potash favors micas. In general, basic igneous rocks alter to montmorillonite. Kaolinite may form from any rock if the alkalies are present or if the conditions are acidic and at a moderate temperature.

Although metasomatism may well be involved in most cases, it has been found that hydrothermal deposits of clay in seams can be formed without any involvement of the

wall rock. This has the consequence that one cannot exclude the possibility of finding clay gouge in a rock that is not normally associated with the formation of clay minerals.

Weathering. The following discussion of weathering will be limited to chemical weathering after the mechanical processes of grinding and pulverizing during faulting have occurred. To a large extent, weathering and hydrothermal alteration give the same products. The main difference is the depth of action of the two processes. Weathering usually occurs near the surface due to the influence of meteoric waters and hydrothermal alteration occurs at depth due to juvenile water. Weathering is therefore not a large factor in the formation of gouge.

Many factors control weathering, and it is difficult, if not impossible, to generalize on the relative importance of the various factors. Weathering is the process of ion exchange whereby rocks and soils which are unstable in their environment are transformed to more stable products. These products are generally quartz, hydrated sesquioxides of iron and aluminum, and the clay minerals. The minerals of engineering importance are the clays, and their production depends primarily on the weathering environment and time (Mitchell, 1970).

Parent rock type is important, but mainly in the initial stages of weathering. Soils containing kaolinite and soils containing montmorillonite can both develop from the same parent rock given different climatic conditions and time periods. Conversely, given enough time, the same clay soil can develop from different rocks.

Kaolinite, one of the most common clay minerals, is formed by conditions which remove the alkalies and therefore can form from any rock type. Kaolinite is the characteristic clay mineral of soils in humid regions and is generally associated with acidic

rocks.

Montmorillonite clays are formed in dry, cool climates under poor oxidizing conditions. Alkaline solutions favor their formation. The presence of potassium favors the formation of illite.

Dissolving may also be considered a weathering process which is particularly important in carbonate rocks, leaving cavities.

As described above gouge material is very seldom uniform, indicating that numerous movements have taken place within the fault zone. Blocks or even plates of intact rock may float in a basic matrix of soft material that again may have bands or seams of hard material such as quartz or calcite. The gouge may be a sand like material which can be expected to exhibit all the physical characteristics of sand under high pressure. Gouge may also be a clay material, highly consolidated and with a high shear strength. This type of gouge may be found adjacent to the same type of clay in a state of underconsolidation and low strength. It should again be emphasized that gouge is normally a very complex, heterogeneous material both in regard to mineralization and physical properties. It is therefore extremely difficult to sample and even more difficult to characterize by testing.

Classification of Gouge

Table IV-1 presents a diagnostic classification of gouge based on an assessment of dominant type of material in the gouge in terms of behavior. It is not necessarily the most abundant

Table IV-1
TENTATIVE CLASSIFICATION FOR FAULT GOUGE MATERIAL

Dominant Material in Gouge	Potential Behavior of Gouge Material	
	At Face	Later
Swelling clay	Free swell, sloughing. Swelling pressure and squeeze on shield.	Swelling pressure and squeeze against support or lining, free swell with down-fall or wash-in if lining inadequate
Inactive clay	Slaking and sloughing caused by squeeze. Heavy squeeze under extreme conditions.	Squeeze on supports or lining where unprotected, slaking and sloughing due to environmental changes.
Chlorite, talc, graphite, serpentine	Ravelling	Heavy loads may develop due to low strength, in particular when wet.
Crushed rock fragments or sand-like gouge	Ravelling or running; standup time may be extremely short.	Loosening loads on lining. Running and ravelling if unconfined.
Porous or flaky calcite, gypsum	Favorable condition	May dissolve, leading to instability of rock mass.

material in the gouge. A more detailed discussion is given in the following chapters.

It should be emphasized that gouge material being so complex is likely to overlap the various categories, e.g. porous or flaky calcite bands in the swelling clay matrix.

The potential behavior is listed in summary for "at face" and "later". It is common to distinguish between loads on the primary

support, reinforcement, or lining system installed to ensure stability during construction, and loads in the final reinforcement or lining usually added to ensure stability for the lifetime of the structure. The time expired between the two stages can, however, vary considerably from casting of the final lining at the face to casting several months after excavation. It is therefore considered more meaningful to use the proposed distinction. "At face" relates to the difficulties involved in driving through the fault zone, "later" to the adequacy of applied methods of support, reinforcement and lining.

The first two divisions deal with swelling clay and inactive clay. The two terms are self explanatory but there is a rather subtle distinction to be made between them. In some clayey gouges a small amount of montmorillonitic clay may dominate the material behavior, and the gouge would therefore be classified as swelling. In other instances the clayey gouge may be rather inactive in spite of a substantial content of montmorillonitic material. It would then be classified as inactive. The distinction is difficult and must be made on examination of the gouge, testing and comparing the results with those described in Chapter VI; and finally in terms of which classification would most adversely affect the underground structure. If a material is classified as swelling clay it may also be expected to exhibit squeezing pressures. On the other hand if the material is classified as inactive clay, it will not be expected to swell.

The third group is intended to include blocky gouge material that is heavily interwoven with slickensided seams and joints filled or coated with chlorite, talc, graphite or serpentine.

These materials characteristically have a very low shear strength, particularly when wet. The potential behavior of this class of gouge is immediate ravelling or a very short standup time. It eventually develops heavy loads and behaves much like squeezing ground.

The fourth major group includes the crushed rock fragments or sand-like gouge materials. This material is typically cohesionless. The major problem with this type of gouge is very short standup time.

The fifth group covers the readily soluble materials of calcite and gypsum. The problem associated with this group is that after a long period of stability, instability may arise due to the dissolving of the minerals mentioned.

Chapter V

FIELD INVESTIGATIONS

Several underground construction projects throughout the United States were selected for site visits to obtain samples of gouge material. An attempt was made to obtain as wide a variety of samples as possible to assure an adequate representation of gouge materials.

The main sampling effort was made in the Straight Creek Tunnel in Colorado because of the size of the project and because extensive information concerning construction methods and problems, and instrumentation and testing results were available. The other tunnels that were visited were exploratory drifts at the Auburn Damsite in California, Nast and Hunter tunnels, the Henderson Mine Haulage Tunnel in Colorado, the Metro System in Washington, D.C., and the Twin Buttes Mine in Arizona.

It was decided to use gouge samples only from tunnels in order to eliminate weathering effects. Therefore, test results on the Twin Buttes Open Pit Mine samples are not included herein.

Auburn Damsite, California

The Auburn Dam will be a thin, double curvature concrete arch dam about 865 feet high with a crest length of about 4000 feet. The site of the dam is in a broad V-shaped canyon of the North Fork of the American River. This region is cut by several northwest striking faults. Carlson and Livingston (1972) and Frei (1972) give a detailed account of the site geology.

Amphibolite is the major metamorphic rock unit. Serpentine

and talcose serpentine occur within the amphibolite as either irregular masses or lenticular bodies parallel to the foliation. These bodies are usually bordered by sheared talc and chlorite schists.

There are also numerous seams (shear zones), some parallel to the foliation but generally cross-cutting it and parallel to the major joint set. The seams range in nature and continuity from a single plane with a thin coat of clay gouge that is continuous for less than 100 feet to wide shear zones with large altered areas, continuous for over 1000 feet.

The major fault is designated F-1 and supplied the samples for this investigation. The fault zone with associated gouge material is up to 10 feet thick with an additional 4 to 5 feet of chlorite schist in the drag zone of the fault. Samples of gouge were taken from an exploratory drift driven through the F-1 fault and near the area of in-situ testing carried out by the Bureau of Reclamation.

Nast and Hunter Tunnels

The Nast and Hunter tunnels are a part of the Fryingpan - Arkansas Project in Central Colorado. These two tunnels will eventually connect with a system of tunnels that will divert water from the Western Slope watershed in the Colorado River Basin to the Arkansas River Basin east of the Rockies.

Nast Tunnel - The Nast tunnel is located as shown on Figure V-1. This tunnel is unique in that it is being driven by a German made mole 9'9" in diameter. The excavation history of this tunnel has recently been presented by Geary (1972).

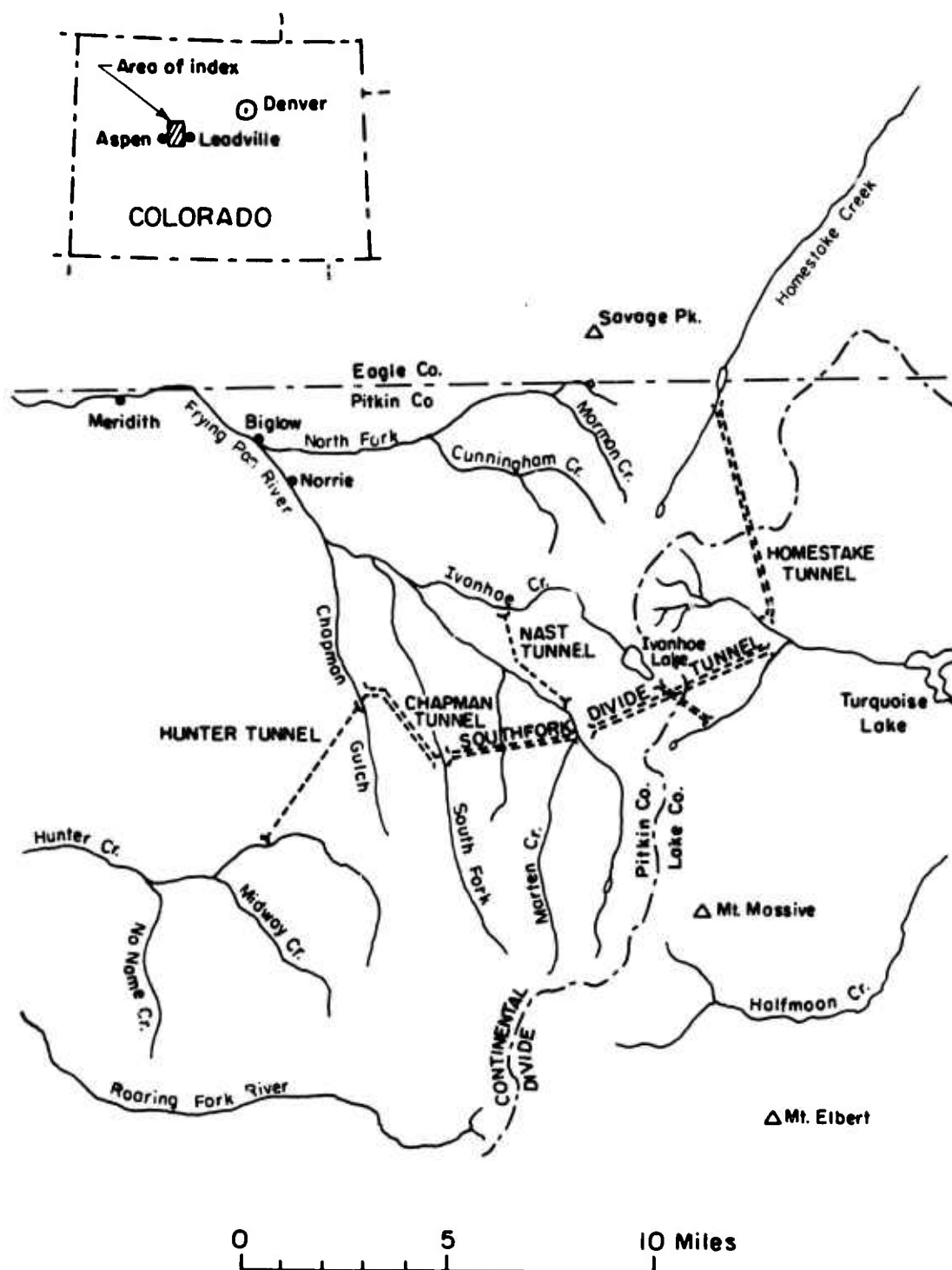


FIGURE V-1 LOCATION MAP OF NAST AND HUNTER TUNNELS

Preliminary geologic exploration for this tunnel was in the form of surface geologic mapping, several borings, and a seismic survey outlined in a pre-construction Engineering Geology report prepared by Bureau of Reclamation personnel. They spent forty man-days doing the detailed and reconnaissance field mapping. Eleven drill holes were drilled along the alignment to a maximum depth of 340 feet. Generally the holes were 40 feet or less in depth.

The stratigraphy of the area consists of a bedrock of folded, layered sequence of granite gneiss, porphyritic granite gneiss, biotite schist and phyllite with minor gneissic granodiorite. A swarm of felsitic dikes cross the area, trending northwest.

The porphyritic granite gneiss is probably the best quality rock. It is composed of a medium grained granitic matrix with a trace to 75% of 1/4 to 1/2 inch pale pink feldspar phenocrysts. The granite to granite gneiss is a fair to good quality rock made up of fine to medium grained biotite, feldspar and quartz. It is faintly to strongly banded and foliation planes range from indistinct to prominent. When strongly foliated the rock breaks readily along schistosity and banding.

The area of gouge sampling is shown in Figure V-2 on a geologic cross section of a portion of the Nast tunnel. Sample locations are marked and a description of each sample area follows.

Station 460 + 50. (Sample N1) One of three 1/2 inch wide seams in a vertical, 18 inch wide, highly altered zone in competent rock. The soft area had swelled or squeezed out about 2 inches into the tunnel in approximately 2 months time. The area had been steel supported.

Station 462 + 60. (Sample N2) This is a zone of many intersecting, high and low dip angle shears within a 200 foot wide zone

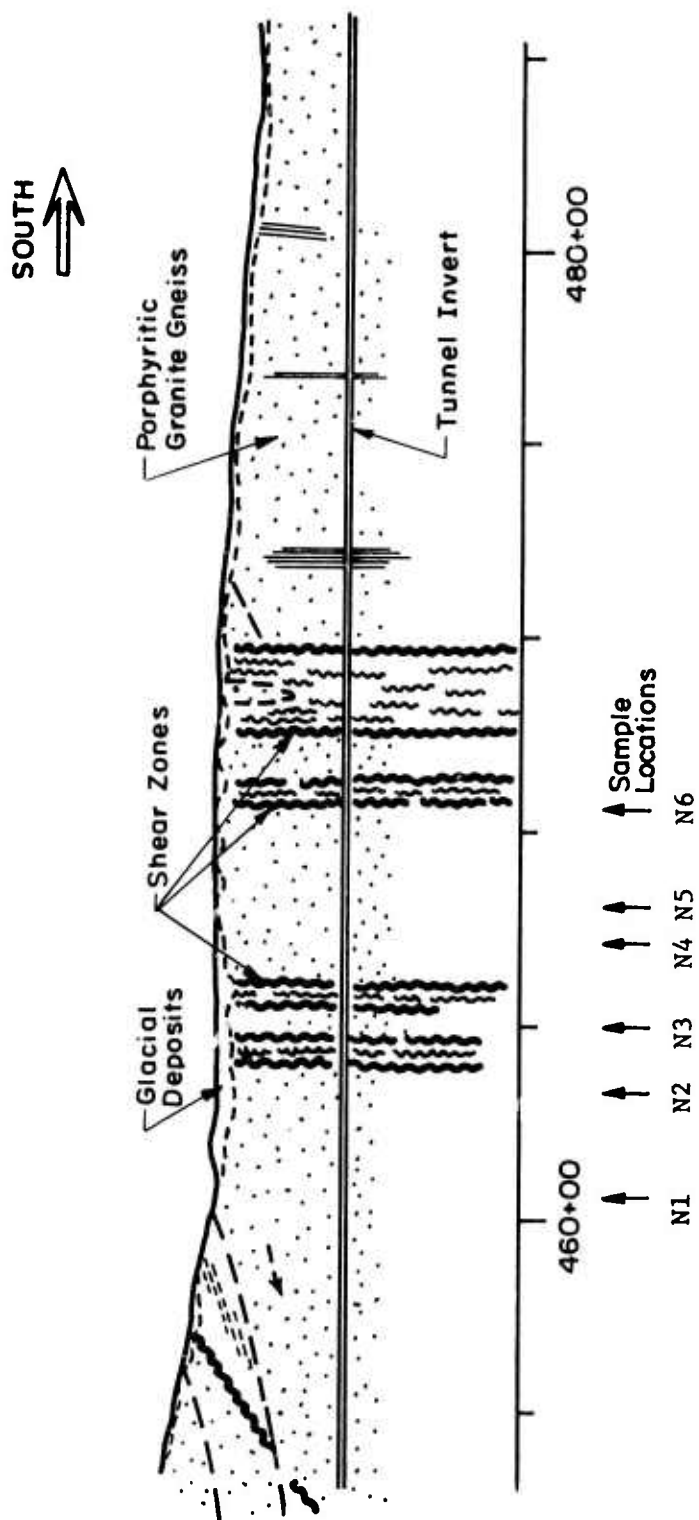


FIGURE V-2 CROSS-SECTION OF NAST TUNNEL SHOWING SAMPLE LOCATIONS

of moderately soft, altered porphyritic granite. The rock had squeezed outward up to 4 inches since excavation three months prior to sampling. This zone was steel supported. The sample was taken from an 18 inch wide zone of highly altered breccia with closely spaced, hairline clay seams.

Station 464 + 00. (Sample N3) This sample was taken from a 3 inch seam in a 75° dipping, 12-18 inch wide shear in a 50 foot wide zone of soft, altered, porphyritic granite. This shear is one of several similar shears spaced about 4 feet apart. The ground was steel supported and had squeezed in about 1 inch since excavation.

Station 465 + 85. (Sample N4) The sample was taken from a small seam within a 2 foot wide highly altered shear zone. The wall rock surrounding this shear zone is a fairly competent granite. The area was rockbolted and had only sloughed slightly since excavation.

Station 466 + 45. (Sample N5) The sample was taken from a 2 inch seam in a 60° dipping, 2 to 3 foot wide highly decomposed zone in competent granite. The zone had swelled in about 2 inches and the adjacent rock had been cracked since excavation. The area was rockbolted.

Station 468 + 46. (Sample N6) The sample was taken from a seam located in a 45° dipping, 2 1/2 foot wide, highly altered shear zone in a competent granite. The altered zone squeezed or swelled in about 2 inches within a few days after excavation, and subsequently remained stable.

As described above the gouge material generally occurred in seams in highly altered fault breccias. The gouge generally appeared as a brown to green sandy clay. The zones were not large but caused problems by squeezing or swelling into the tunnel. The main problem was, however, a very short standup

time, usually about 2 hours. The maximum standup time was 8 hours. This was especially critical because it took the mole several hours to advance enough to allow miners to install the support system.

Hunter Tunnel - The Hunter tunnel, located as shown in Figure V-1, is being driven by conventional means through essentially the same type of rock as the Nast tunnel. The geology has been described by the Bureau of Reclamation geologist, Mr. Jerrold Bartell, for the inspector's final report.

Approximately 39% of the tunnel length is through a granite gneiss to gneissic granite, depending on the degree of foliation or banding. The rock breaks randomly except when altered and then tends to slough along planes parallel to the banding. The rock is composed of about 65% medium-to coarse-grained quartz and feldspar which make up the light colored foliation bands, and about 35% fine-to medium-grained biotite making the dark bands.

The joint spacing within this rock type ranges from closely spaced (4-8 inches) adjacent to sheared zones to areas lacking joints for 30-40 feet of tunnel length. For the most part, the joints are tight, persistent and are spaced at 1-2 feet. They are a high dip-angle joint set striking diagonal to the tunnel, which causes slabbing along the tunnel walls and lower arches.

Gouge samples were taken at Stations 273 + 76 (Sample H1) and 273 + 80 (Sample H2); one each from two of several 3-inch wide seams in a 20 foot wide, near vertical, dry, highly altered zone that strikes nearly normal to the tunnel. The area was supported by strutted, 4 inch wide flange steel on 2 and 3 foot centers. The sets had taken some load as indicated by deformed and broken lagging.

Henderson Haulage Tunnel

The Henderson Mine Haulage Tunnel will extend approximately 10 miles under the Continental Divide to a point near the Urad mine. The position of the tunnel is shown on Figure V-3. The purpose of the tunnel will be to haul ore from the mine to a mill presently being constructed in the Williams Fork Valley about 4 miles below the west portal.

The geology of the tunnel has been reported by Robinson and Stewart (1971). A generalized plan and section of the tunnel is given in Figure V-4. The tunnel has a modified horseshoe cross-section, and is 15 feet high and 17 feet wide.

The geology is that of the Front Range of Colorado, the rocks consisting of folded Precambrian igneous and metasedimentary rocks. The rock mass is extensively faulted and intruded by dikes of Tertiary age. The rock units encountered in the tunnel are the same as encountered in the Straight Creek Tunnel, that is, Precambrian metasedimentary rocks equivalent to the Idaho Springs Formation and granite rocks equivalent to the Silver Plume Granite.

Robinson and Stewart (1971) report that wall rocks adjacent to veins show evidence of hydrothermal alteration in the form of hydrous clay minerals with accompanying silicification and pyritization.

The metasediments are a variety of fine-to coarse-grained biotite gneisses. Foliation is distinct and due to compositional layering ranging in thickness from 1 to 150 mm. The average thickness is 25 mm. The dark layers alternate with light colored layers and are made up of biotite and plagioclase. The light layers are rich in quartz and plagioclase. Granitic materials are interlayered

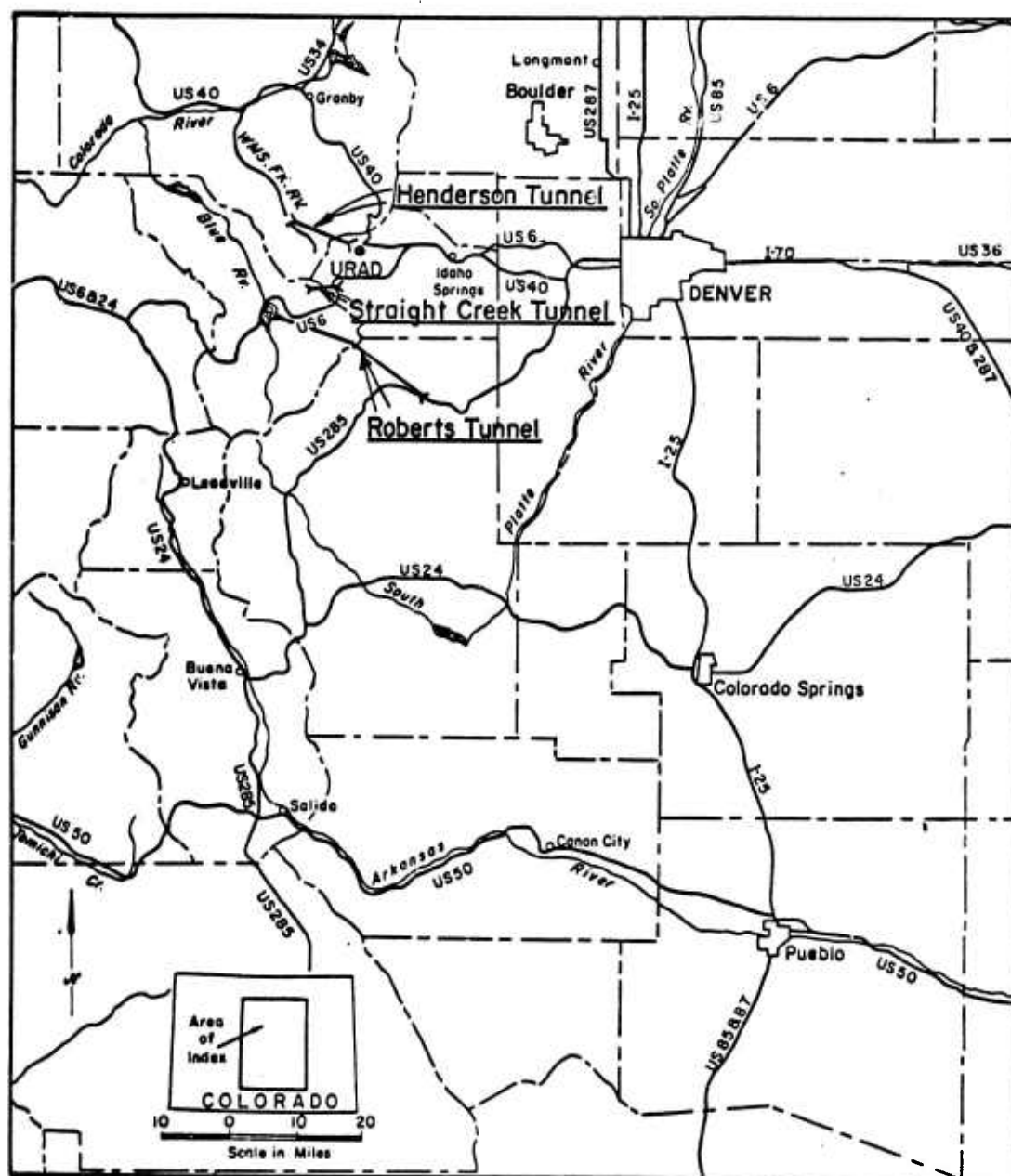


FIGURE V-3 LOCATION OF THE HENDERSON MINE RAILROAD HAULAGE TUNNEL, AND THE STRAIGHT CREEK TUNNEL

with the gneisses.

The granitic rocks are of the same texture and composition as the Silver Plume Granite (Lovering and Goddard, 1950). This is typically a medium grained, porphyritic quartz monzonite. The texture varies from medium to fine-grained equigranular to coarse grained porphyritic. Aplite and pegmatite dikes are common and are younger than the Silver Plume granite. The Tertiary Dikes mapped by Robinson and Stewart are of two types: augite diorite and porphyritic rhyolite dikes.

Preliminary information indicated that the foliation would be approximately parallel to the tunnel in Province I and II (Figure V-4) and at right angles to the tunnel alignment in Province III. However, during excavation of the tunnel, it was found that the foliation intersected the tunnel at almost 45° in Province III. The foliation is the principal direction of jointing in the meta-sedimentary rocks. The principal joint set in the granitic rocks forms an acute angle with the tunnel, and the secondary set runs parallel to the tunnel.

From the surface geology and from the information obtained from three borings drilled 2500 feet off the tunnel line (see Figure V-4) it was expected that the rock conditions would be quite good with very little support required. However, several major fault zones had been encountered with the result that the projected tunnel cost had doubled at the time of sampling. At that time (August, 1971), approximately 1/5 of tunnel had been excavated. Four major gouge zones were studied in this tunnel and each zone is described below.

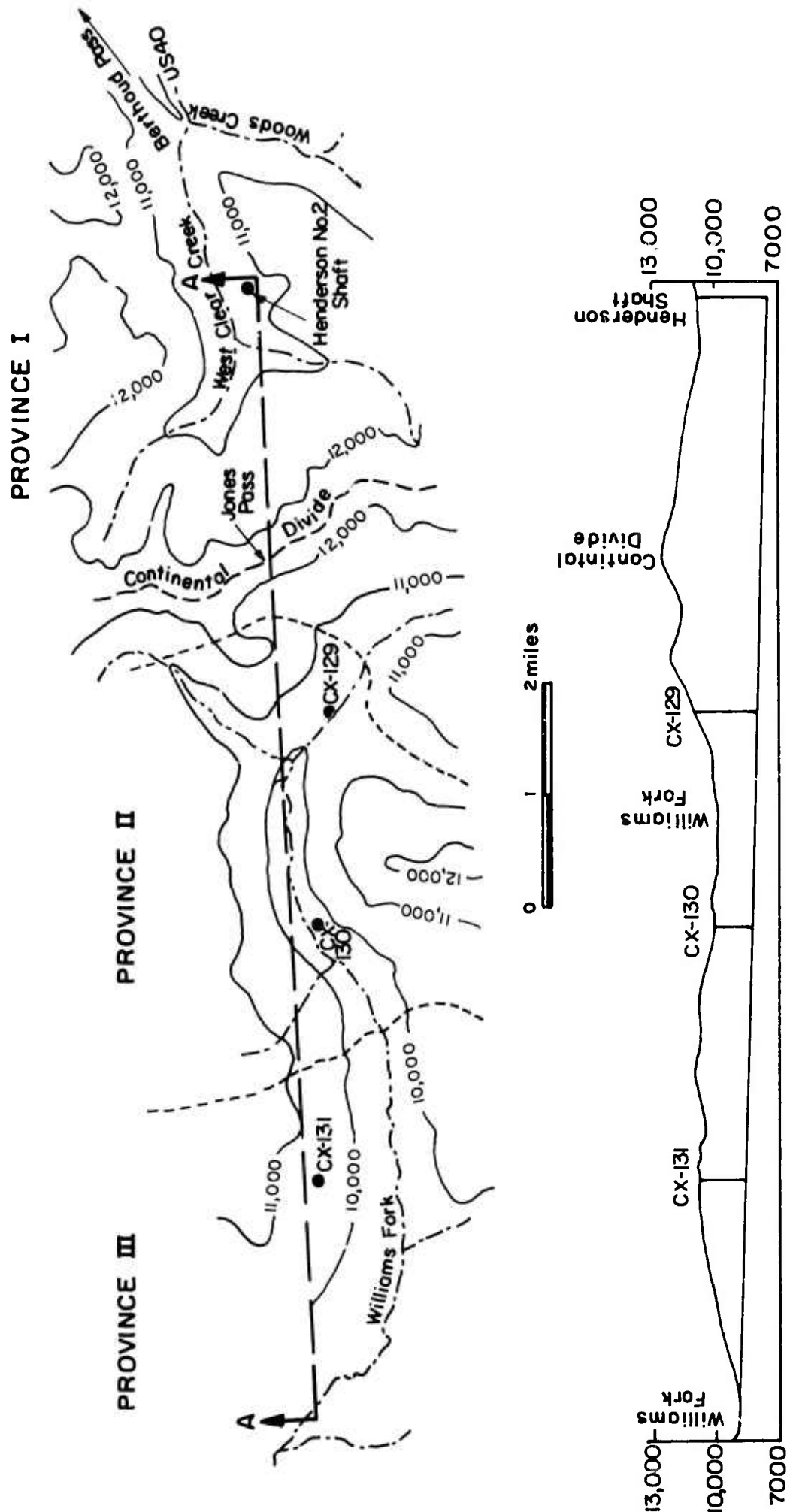


FIGURE V-4 GENERALIZED PLAN AND SECTION OF THE HENDERSON MINE HAULAGE TUNNEL
(after Robinson and Stewart, 1971)

Station 31 + 03 to 34 + 00. This was considered by the tunnel personnel to be the first fault zone encountered that gave major problems. An area between station 31 + 03 and 32 + 62 required steel support with invert struts although the fault zone extended to station 34 + 00. The tunnel alignment and fault attitude are shown in Figure V-5.

Tunnel advancement was halted at about 32 + 00 due to failure of steel sets and partial closure of the tunnel behind the face. High loads came on for several tens of feet behind the face immediately upon encountering the fault zone. It was necessary to remine and replace steel sets. Jump sets were also added to give 6" sets on 2' centers through the zone. Shotcrete 15 to 18 inches thick was then applied. Deformation continued, however, until gusset plates of 4" x 3/4" steel were welded to the straight leg portion of the sets and more shotcrete was applied. Along with this, 20' long grouted #13 rebars were installed in the wall rock and welded to the sets. Six of these were used for each leg in order to stop the inward movement.

Advance information on the extent of the fault zone was obtained by drilling from the face. Core recovery through the gouge zone was zero.

Once deformation behind the face was controlled, tunnel advancement continued with only minor problems through the zone. The ground was classified in the field as squeezing ground and exhibited high lateral or side pressure. Two samples were taken from this zone, one at Station 32 + 85 (Sample HM1) and one at Station 33 + 30 (Sample HM2).

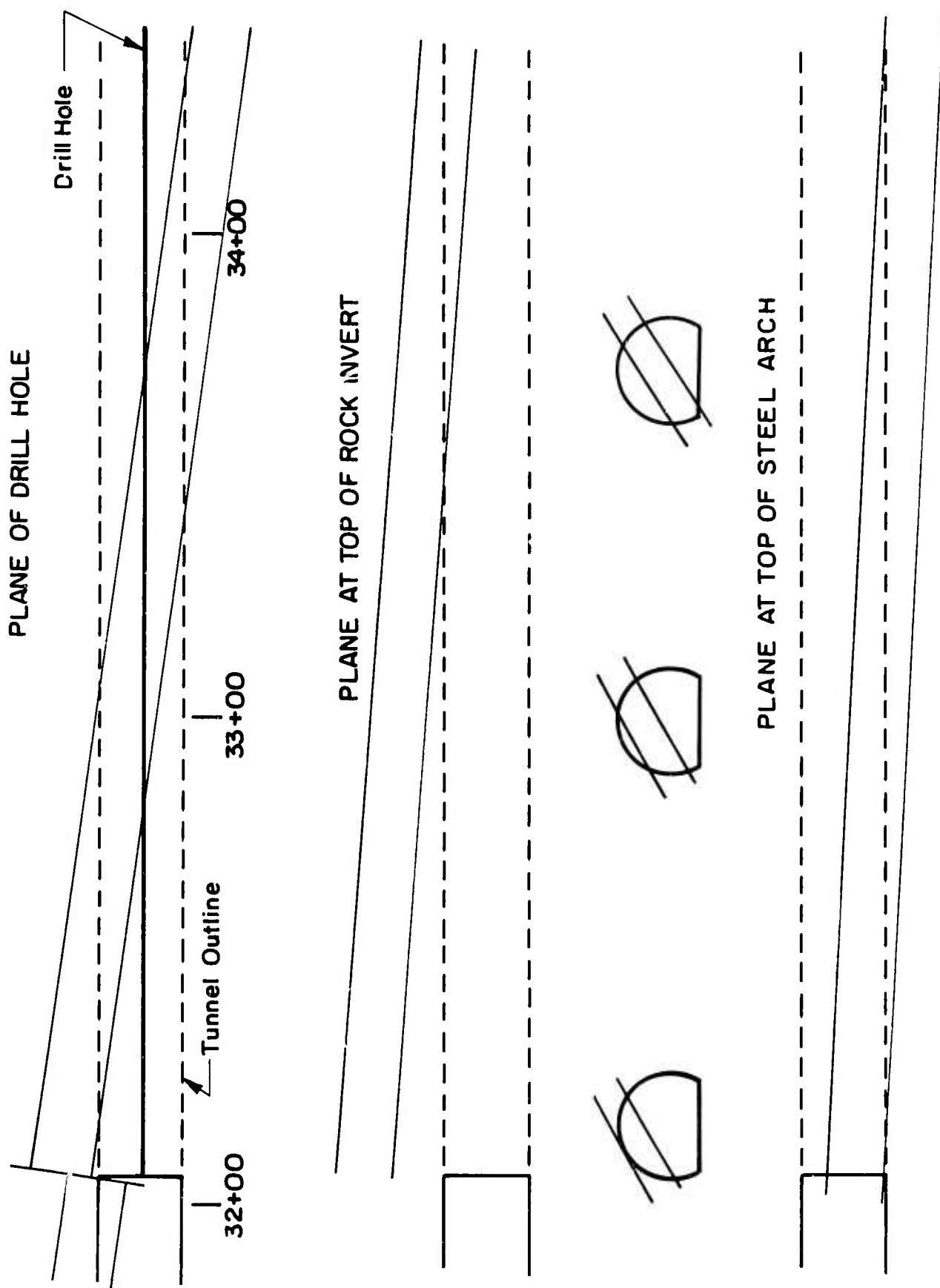


FIGURE V-5 PLAN VIEW OF FAULT ZONE, STATION 32 + 20, HENDERSON MINE HAULAGE TUNNEL (after C. Bellows)

Station 34 + 85. (Sample HM3) This sample represents gouge from a small running ground zone. The fault zone was vertical, about 4 feet wide, and intersected the tunnel line at about 15°. The gouge was brown clayey coarse sand-like material. The zone was crushed gneiss, slightly altered and was a fairly common type. The gouge was wet, running into the tunnel when exposed. The tunnel was advanced by spiling from crown to footing through the zone.

Station 45 + 86 to 46 + 48. (Sample HM4) The loading characteristics of this zone was similar to that at Station 31 + 03. The gouge material was different, being a grey-blue clay near the center of the zone and grading to sandy clay near the borders. Invert struts were installed from Station 44 + 94 through the zone.

The gouge material was first encountered in the back at Station 46 + 03 as shown in Figure V-6. The gouge began to squeeze into the tunnel and a bulkhead was placed against the face. Deformation continued and a movement of the bulkhead of 1" per hour was recorded. Finally, a very large run took place over the top of the bulkhead, filling the tunnel with gouge material. The zone was finally advanced through with a heading and bench, using crown bars. Floor heave also took place through the gouge zone and a solid wooden floor was installed between Stations 45 + 86 and 46 + 48.

Straight Creek Tunnel¹

The major field effort for this study was conducted at the Straight Creek Tunnel in Central Colorado, shown in Figure V-7.

¹The tunnel was upon completion in 1973 renamed the Dwight D. Eisenhower Tunnel. The old name is used here since it is used in all references.

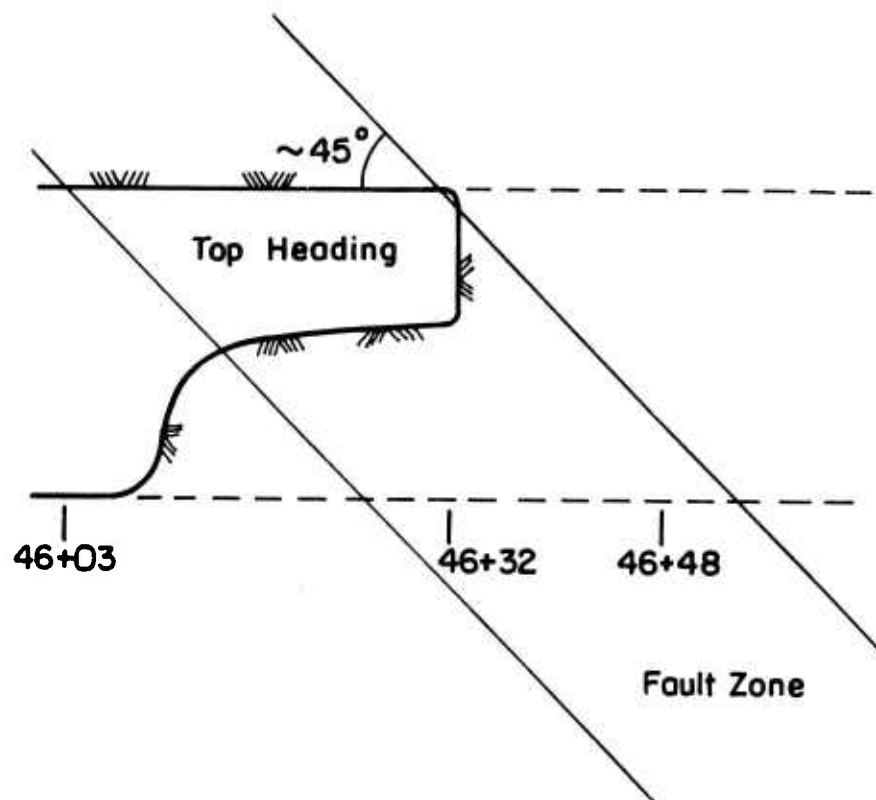


FIGURE V-6 CROSS SECTION OF THE FAULT ZONE AT STATION 46 + 03
HENDERSON MINE HAULAGE TUNNEL.

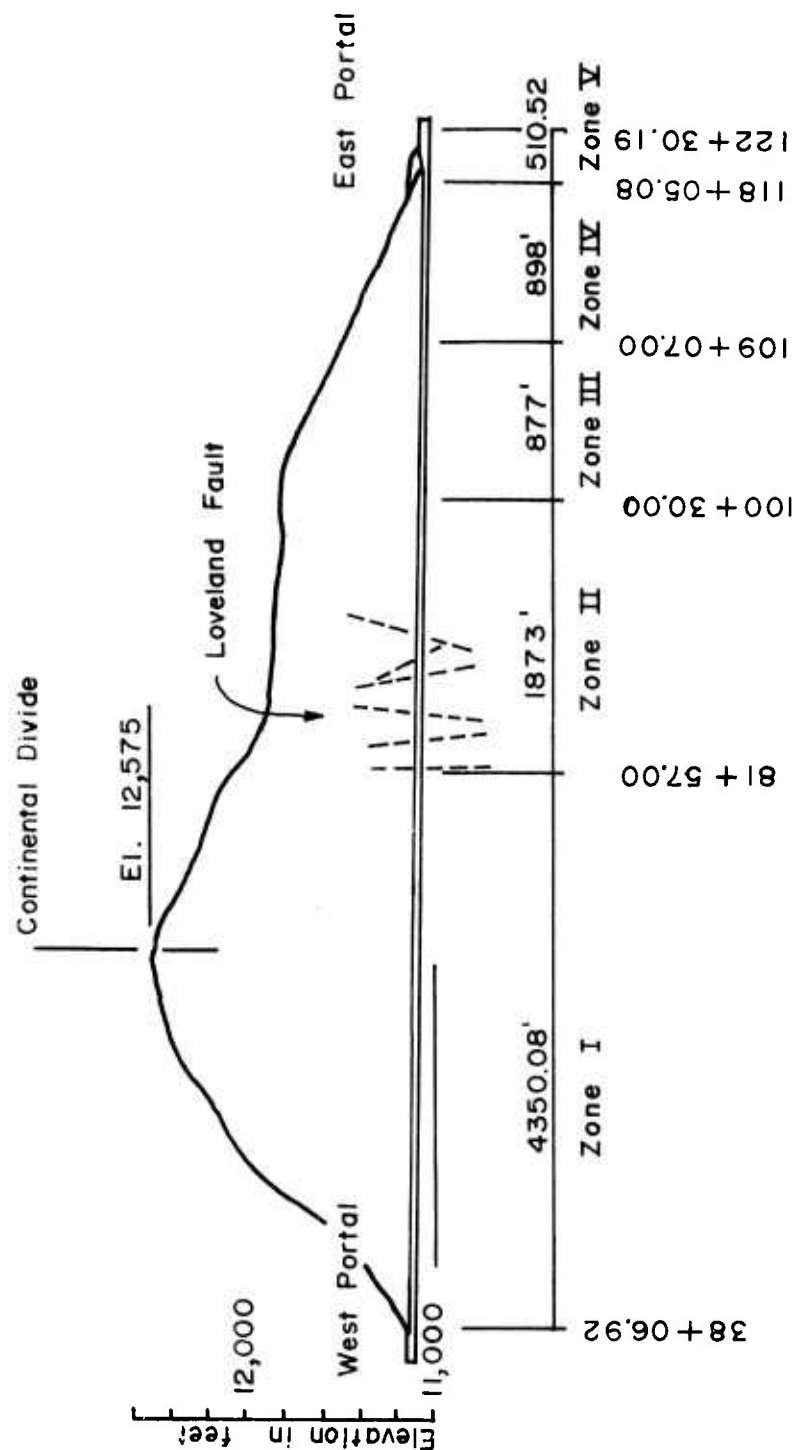


FIGURE V-7 SECTION OF THE STRAIGHT CREEK TUNNEL
(after Hopper et al., 1972)

Sampling and the study of in-situ conditions were accomplished during a 45 day period in the summer of 1971. Access to the tunnel as well as construction records were provided by the Colorado Division of Highways' personnel.

The Straight Creek Tunnel is located about 65 miles west of Denver and is part of the Interstate Route 70 system, shown on Figure V-3. The tunnel passes under the Continental Divide in the Loveland Pass area at an elevation of approximately 11,000 feet. The tunnel is approximately 8,900 feet in length. Tunnel construction began in January, 1968, and the finished tunnel was put into operation in February, 1973.

This project has been much publicized due to its size and to the problems encountered during construction. The geology of the site has been described in several reports by Robinson and Lee (1962, 1963, 1965) Abel (1967) and by Robinson (1971). A thorough description of the construction of the Straight Creek Tunnel is given by Hopper, Lang, and Mathews (1972). Extensive use of these reports has been made in the following.

General Geology

The bedrock in the Straight Creek Tunnel area consists mainly of granite with inclusions of metasedimentary rocks. Augite - diorite dikes cut through the rocks. The most complete description of the regional geology is given by Lovering and Goddard (1950). The main geologic feature of the tunnel site is the Loveland fault zone. This steep zone is about 2 miles wide and consists of numerous north-trending faults and seams separated generally by relatively competent rock.

The metasedimentary rocks include a wide variety of biotite-rich gneisses, schists and some local quartzites. These rocks, as mentioned, generally occur as inclusions in the granite. The metasedimentary rocks are fine to medium-grained and consist predominantly of plagioclase feldspar, quartz and biotite. The rocks show distinct foliation, or layering, as the result of concentration of the biotite. The layers range in width from 0.1 inch to 1 inch. The biotite has often been altered to chlorite, and the feldspar has been slightly to completely altered to clay.

The granitic rocks vary somewhat in composition, but are generally quartz monzonite. The rock is light gray to pinkish gray and fine to medium-grained. It is made up chiefly of light gray plagioclase feldspar, light gray to pink microcline, quartz, and biotite with some muscovite. The plagioclase, microcline and quartz are generally in about equal amounts and the amount of biotite is 1 to 15 percent. The microcline grains generally have subparallel orientation giving the rock its foliation.

Pegmatite associated with the granite is pink and consists of coarse grains of microcline and quartz. The pegmatite occurs as irregularly shaped bodies or as dikes a few inches to several feet in width.

Augite-diorite dikes cut the granitic and metasedimentary rocks and range in thickness from a few feet to about 60 feet. The dikes trend north to northeast. The rock is dark gray and consists of fine-grained augite and plagioclase with small amounts of biotite and hornblende.

The structural features that affected the construction of the tunnel were the faults and shear zones of the Loveland fault zone and the joints of the various rock units. The fault and shear zones showed varying degrees of shearing, crushing and alteration. The intensity of faulting is reflected to some extent by the gouge material. Where faulting had been less intense, the shear planes were from 0.1 to 1 inch apart. As the intensity increased, the distance between shear planes decreased. Where shearing had been most intense and accompanied by alteration, the gouge was clayey. The alteration produced a discontinuous clay gouge that occurred in streaks from a few inches to several hundred feet long and 0.1 inch to several feet wide. The clay gouge generally did not occur near the center of the shear zone but rather near one margin.

The effect of faulting was different on the different rock types. The granite gave sharp, angular, sand-size grains and rock fragments. Some alteration generally gave the sandlike material a clay matrix.

The metasedimentary rocks showed foliation parallel to the direction of shearing regardless of the original direction of the bedding. Seams and slickensides occurred parallel to the foliation. Even under the most intense subsequent alteration, the original foliation structure was retained and could be recognized.

The faults and seams varied drastically within a short distance, both in width and extent. Some zones contained blocks of rock which were relatively unsheared although surrounded by intensely sheared rock. Figure V-8 gives an indication of the variability of the ground.



FIGURE V-8 EXAMPLE OF VARIABLE NATURE OF GROUND - STRAIGHT CREEK TUNNEL.

Joint aperture ranged from less than 0.1 to 0.5 inches. The joints, particularly in the metasedimentary rocks, were typically filled with clay gouge. In a few cases, however, joints without filling were found.

Site Investigations

Although the general geology of the site area was well known, the proposed tunnel was to be 1500 feet below the surface, making it very difficult to predict the conditions to be encountered from the surface. A pilot tunnel was therefore driven 120 to 250 feet south of the proposed tunnel. Extensive mapping and research was carried out in this tunnel (Robinson and Lee, 1965, Abel, 1967). Geophysical investigations, including resistivity and seismic velocity measurements, were made and laboratory testing of samples included triaxial tests, mineralogy, and volumetric change tests of gouge material. Predictions were then made for the main tunnel.

As a result of those preliminary investigations the rock classification given in Table V-1 was prepared. For convenience during construction, the tunnel was also divided into several zones as shown in Figure V-7.

Zone I - This zone included mainly granitic rocks with joint spacing generally greater than 1 foot, although in some areas joint spacing was less than 0.5 feet. Excavation of this zone was carried out by the heading and bench method. In the top heading, the steel ribs were set on double I50 wall plates. The section was stiffened by adding concrete to a height of 4 feet above the wall plate. Rock bolts were then installed through the concrete 10 feet into the rock as tiebacks.

Table V-1
ROCK CLASSIFICATION, STRAIGHT CREEK TUNNEL

Rock Class	Description
1	Massive and intact or slightly blocky, having a joint spacing greater than 1.0 feet with no alteration.
2	Moderately blocky and seamy, having a joint spacing greater than 0.5 feet, with little or no alteration.
3	Very blocky and seamy, having a joint spacing less than 1.0 feet, moderately to highly altered with zones of moderate to intense shearing.
4	Highly sheared and completely crushed, squeezing or moderately swelling ground, having a joint spacing from less than 0.1 feet to 0.5 feet; highly altered.

The bench was removed in 30 foot sections for a distance of 1600 feet with no difficulties. Following this, it was decided to increase the section of bench removal to 60 feet which resulted in a collapse of 60 feet of wall plate. Subsequent investigations revealed a series of joint planes dipping into the tunnel. These joints were coated and filled with talc and clay, allowing movements along the joint planes which caused high lateral loads on the ribs and wall plates.

Apart from this very little difficulty was encountered in Zone I.

Zone II - Zone II, between Station 81 + 57 and 100 + 30, was the worst zone to be encountered and caused the most serious problems

associated with this tunnel. This zone was the heart of the Loveland fault zone. The ground was highly sheared and faulted. In most cases, alteration of the sheared material was complete and clay gouge was common. High ground pressure was present as shown in Figure V-9.

The original excavation plan was to advance in the material full face using an open bottom horseshoe shaped shield supported on concrete footings constructed in foot drifts. However, due to the extremely poor ground conditions predicted in Zone II in particular in terms of stand-up time, it was decided to abandon the shield method of excavation. Instead, it was judged that a multiple drift system would be necessary to advance through this zone. Three different multiple drift schemes were developed. The most adverse conditions existed from Station 82 + 40 to 84 + 00 and required completely surrounding the tunnel outline as shown in Figure V-10. Figures V-11 and V-12 show the schemes used to excavate the remainder of Zone II.

Zones III, IV, V - Excavation through these zones started in Zone V in blocky ground by excavating the top heading. Very little difficulty was encountered during this phase. After the heading had been advanced to near the end of Zone III large loads began to come on the steel sets at a number of locations. The steel sets were being severely deformed and twisted, as were the wall plates. This condition required that before the excavation could be completed the top heading would have to be stabilized.

The method used was to cast in place a reinforced wall plate, to completely encase the steel ribs in concrete, and to fill all



FIGURE V-9 BROKEN TIMBERS CAUSED BY HIGH GROUND PRESSURE.

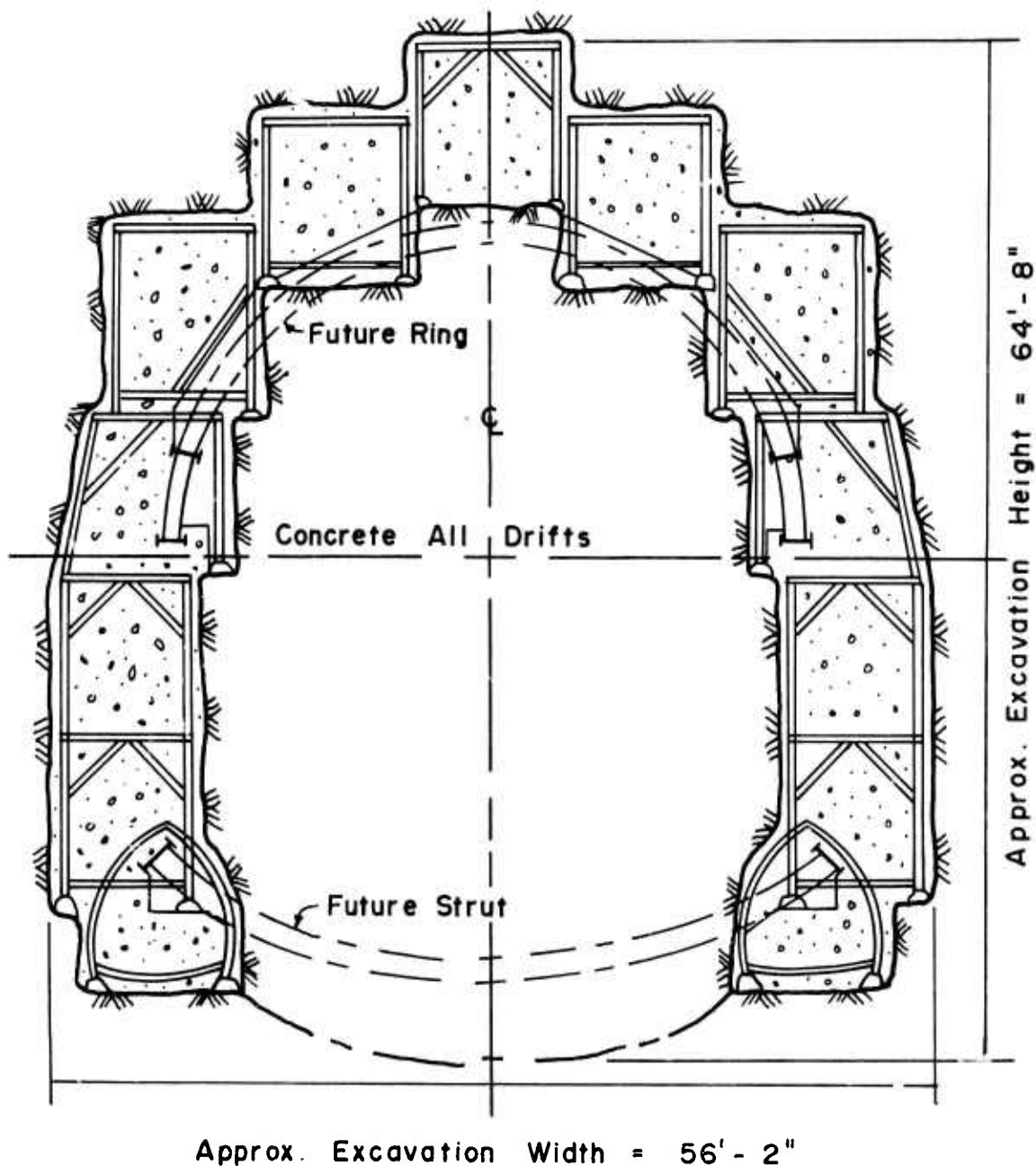


FIGURE V-10 MULTIPLE DRIFT SYSTEM, STRAIGHT CREEK TUNNEL, ZONE II,
STATION 82 + 40 TO STATION 84 + 00 (after Hopper et al.
1972)

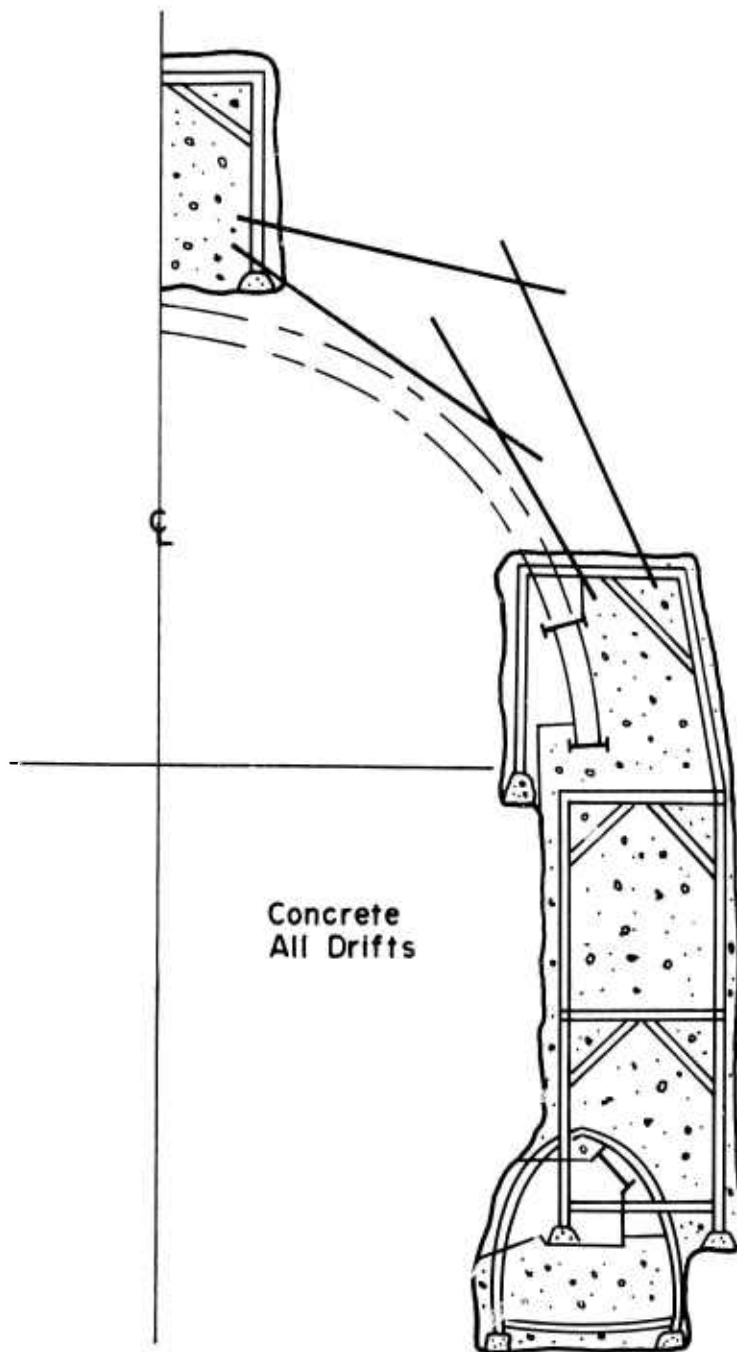


FIGURE V-11 MULTIPLE DRIFT SYSTEM, STRAIGHT CREEK TUNNEL, ZONE II, STATION 84 + 00 TO STATION 85 + 60 (after Hopper et al., 1972).

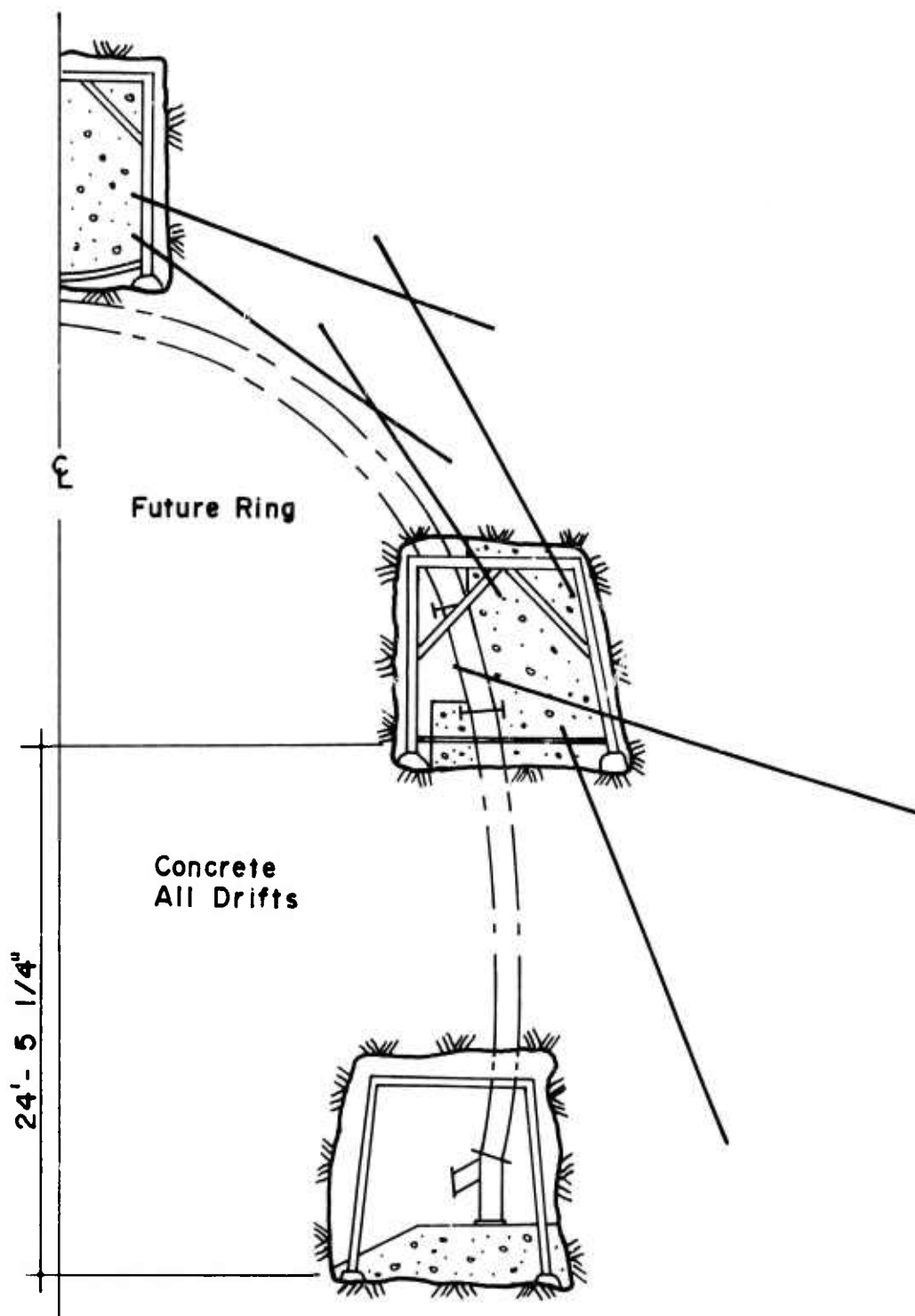


FIGURE V-12 MULTIPLE DRIFT SYSTEM, STRAIGHT CREEK TUNNEL,
ZONE II, STATION 85 + 60 TO STATION 100 + 30
(after Hopper et al., 1972)

voids with grout. To further reinforce the system, grouted 20 foot #8 rebars were installed on a 5 foot x 5 foot pattern. These measures were successful in stabilizing the rock arch in the top heading so that the bench could be removed.

Field Sampling of Gouge

The field work took place during July of 1971. At this time the tunneling operation was concentrated on the north and south wall plate drifts between Stations 83 + 00 and 84 + 00 in Zone II. The samples collected through this interval seem to represent the gouge materials found throughout Zone II, as well as the other zones. Sampling generally took place at the face immediately after blasting to assure "fresh" samples and to study the in-situ conditions at that time.

South Wall Plate Drift Station 83 + 34 (Sample SC1) This sample is an altered mica schist along a small fault trace. The actual fault surface was clay coated and had slickensides. The loads in this area were small; the steel sets were on 3 foot centers and very little lagging was needed.

South Wall Plate Drift, Station 83 + 64 (Sample SC2) This is a sample of highly sheared and altered material from a 3 foot wide shear zone. Moderate loads developed in a few days.

South Wall Plate Drift, Station 83 + 82 (Sample SC3) This is a green clayey sand-like gouge that was wet and appeared to be a running ground type of material if in large quantities. However, the shear zone was only 1.5 feet wide and only a small quantity of gouge flowed into the drift.

South Wall Plate Drift, Station 83 + 95 (Sample SC4) This

sample is from a large area that had several shear zones and was highly altered. The result was a fairly large overbreak between Station 83 + 88 and Station 83 + 98. The gouge is a clayey material with some coarser fragments.

North Wall Plate Drift, Station 83 + 25 (Sample SC5) The area of this sample and the next two samples was a very large zone of sheared and altered or decomposed granite. There were numerous thin shear planes throughout. Each shear plane was marked by a clay band with slickensides through the clay material. The material was dry and broke off in small aggregate pieces. High loads were common, requiring bracing of the steel sets as shown in Figure V-13. Forepoling was used to aid in mining the zone.

Drilling was difficult through this zone. It took around 3 hours to drill a 5 foot hole.

North Wall Plate Drift, Station 83 + 25 (Sample SC6) This is a sample of the clay filling material on the shear planes. The material broke out in large blocks and was dry and hard.

North Wall Plate Drift, Station 83 + 32 (Sample SC7) This sample is of a clayey, fine sand-like gouge

North Wall Plate Drift, Station 83 + 80 (Sample SC9) This sample is from a shear zone that was intersected at Station 83 + 60 and was made up of soft, altered granitic material interfingered with altered schists. The miners reported that the material acted spongy and swelled when in contact with water. The sample is of the granitic material.

North Wall Plate Drift, Station 83 + 40 (Sample SC10) This is a sample of the altered schist from the same area as described above. This material, however, was fairly dry and hard. As it



FIGURE V-13 FOREPOLING THROUGH A GOUGE ZONE - HIGH LOADS
REQUIRED EXTRA BRACING OF STEEL SETS -
STRAIGHT CREEK TUNNEL.

became more moist it did act like a squeezing ground.

North Wall Plate Drift, Station 83 + 27 (Sample SC11) This sample was from a green chloritic schist which had altered to coarse sand-like and clayey gouge but also contained some large unaltered blocks. The material had been exposed for several weeks and had continually sloughed and flowed into the drift. The area was wet and the overbreak at the time of sampling was about 10 feet.

South Foot Drift, Zone III, Station 101 + 00 (Sample SC8) This sample was from a 6 inch wide seam containing clay gouge. There were some small inclusions of altered mica schist. Loads through this area of several seams were moderate.

Completed Full Bore Section, Zone III, Station 106 + 60 (Sample SC12) The sample of a sandy clay gouge was from one of several shear zones through an area of squeezing ground.

Sample Problems

Obtaining relatively undisturbed samples of gouge material proved to be a difficult if not an impossible task. Several different sampling procedures were attempted, each with limited success.

Completely disturbed bag samples were, of course, the easiest to obtain and for the most part met the requirements of the project. That is, samples were needed for laboratory tests to characterize gouge materials; tests which did not require undisturbed samples. However, some strength tests were required and therefore undisturbed samples were needed.

The U. S. Bureau of Reclamation had been contracted by the Colorado Division of Highways to obtain samples for triaxial testing and to perform such tests. Their sampling program was carried out in the pilot tunnel using a special triple barrel diamond core

sampler. Several relatively undisturbed samples of gouge material were obtained, but at high cost.

The same type of drilling and sampling equipment was used for this project. No useable samples were obtained, however owing to:

1. Inexperience of the personnel
2. Short time in which to carry out sample drilling because of the on-going construction.
3. Material conditions which constantly changed from soft to hard.

It is probable that adequate samples of soft gouge materials can be obtained in this manner provided that sufficient time and substantial funds are available.

It might also have been possible to obtain useable samples using "drive" sampling techniques. However, this would have required the use of rock bits due to the variable nature of the materials and some sample disturbance would have been inevitable.

Undisturbed samples were obtained in the Straight Creek tunnel by "block" sampling immediately after excavation whenever possible. These samples were quite close to being undisturbed, although the effect of blasting and the immediate relaxation toward the opening may have led to some disturbance.

The samples were immediately surrounded with a polyurethane foam for protection against disturbance and against the loss of moisture. The samples were wrapped in aluminum foil and polyethylene sheets, placed on polyurethane stock in cardboard boxes and foamed as shown by Figure V-14a and b. The aluminum foil is necessary to protect against adhesion of the foaming material to the sample.



FIGURE V-14a GOUGE SAMPLE WRAPPED IN ALUMINUM FOIL - POLYURETHANE FOAM LIQUID POURED AROUND SAMPLE.



FIGURE V-14b POLYURETHANE FOAM BEGINNING TO SURROUND THE SAMPLE.

Coring of these block samples to obtain specimens for direct shear testing was then attempted in the laboratory by the method shown in Figure V-15. The polyurethane foam surrounding the samples had the added advantage of holding and confining the samples during the coring operation. Several specimens for testing were obtained in this manner but in general the procedure proved difficult either because hard rock fragments were encountered in the soft gouge or washout occurred.

In summary, it may be concluded that obtaining specimens of gouge material for testing which require undisturbed samples is extremely difficult. It would seem that the best conditions under which to sample would be in an exploratory drift such as at the Straight Creek Tunnel. When this is not available samples can only be obtained by drilling from the surface. It is difficult to know when the drill bit is encountering soft gouge material let alone sample the material.

Drive sampling such as used in soil and foundation engineering practice might prove successful. Another alternative could be some form of in-situ testing. Along this line, in-situ penetration tests were attempted in the Straight Creek Tunnel. These tests will be discussed in the next chapter.

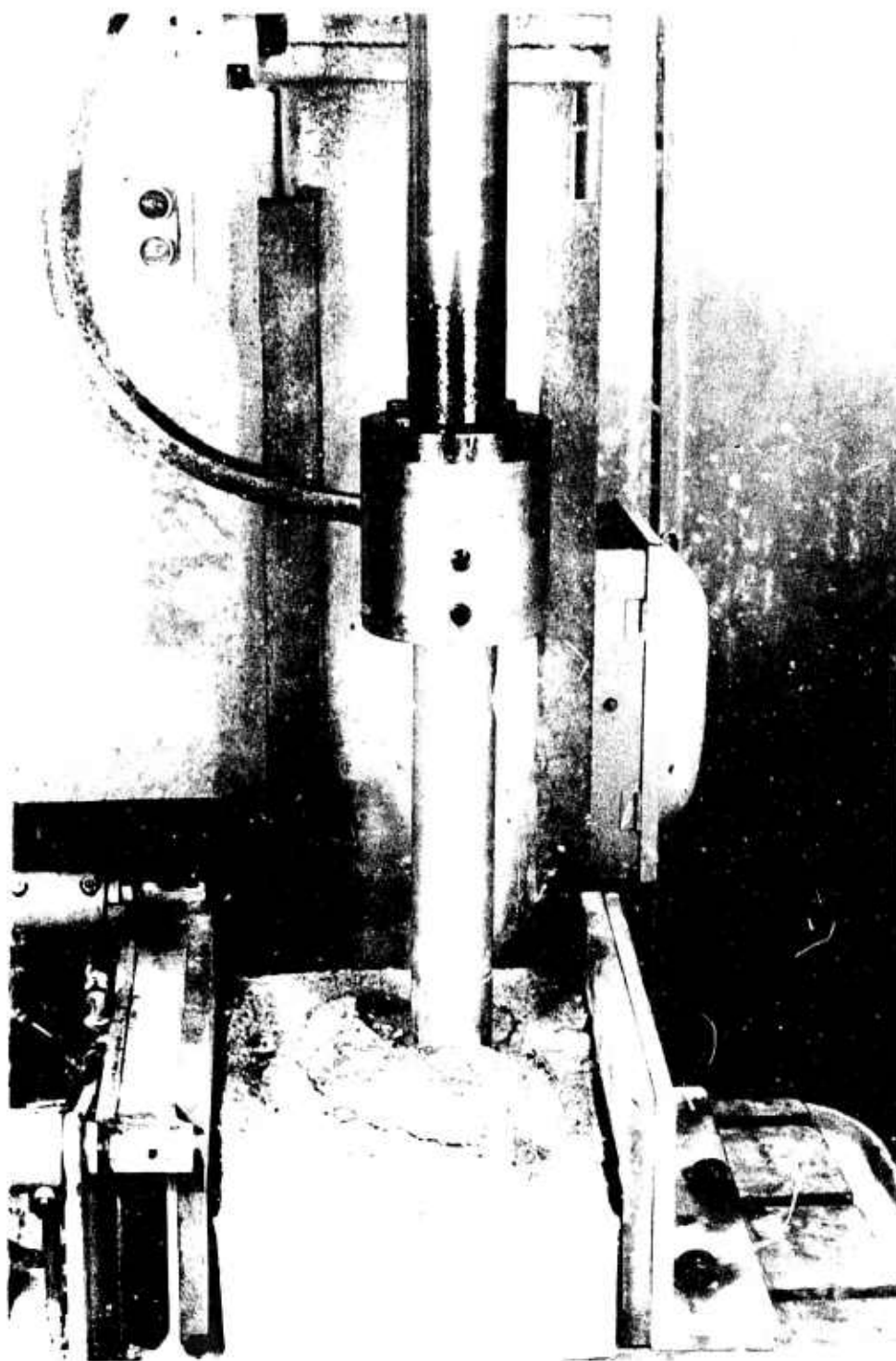


FIGURE V-15 DIAMOND CORING OF GOUGE SAMPLE SURROUNDED BY
POLYURETHANE FOAM.

Chapter VI

LABORATORY INVESTIGATIONS

Methods

The purpose of the testing program was to study the properties of selected gouge materials in order to establish some simple methods and techniques that could yield pertinent information on the potential behavior of the material. A general testing program was selected in order to obtain the basic physical properties and characteristics of as many different gouge materials as possible. It was also thought that some of these test results might correlate with those properties which are known to control behavior such as strength or swelling potential.

The tests selected for this purpose were the Atterberg limits, a free swell test, and grain size distribution. X-ray diffraction was used to establish the clay mineralogy of the samples since clays, when present, generally dominate the behavior of the material.

A swelling pressure test was used to measure the swelling pressures generated by the various types of gouge. The direct shear test was selected to measure the strength of the samples. Finally, a penetration device was constructed for the field, to be used for measuring in-situ properties of gouge materials. The test procedures for these tests are presented in Appendix A.

Atterberg Limits

It has long been recognized that the Atterberg limits of an argillaceous soil correlate quite well with the potential behavior of a soil. In this regard Terzaghi (1926) wrote, "If we know the

three limits of a soil, we are already in a position to compare this soil with others and can at least anticipate what its properties may be. If we know in addition the results of physical tests performed on another soil with fairly identical limits we can say the soil is known." The Atterberg limits have the great advantage that they require a minimum of equipment and time and that the tests to establish the limits can easily be performed in a field laboratory.

It was in 1911 that Atterberg proposed a series of tests for determination of soil properties which are now known as the Atterberg limits. These properties define the water content ranges of the plastic and other states as shown in Table VI-1.

Table VI-1

ATTERBERG LIMITS AND INDICES

Liquid state		
Plastic state	—— Liquid Limit, w_L	—— Plastic Index, $w_L - w_p$
	—— Plastic Limit, w_p	
Semi-solid state		
Solid state	—— Shrinkage Limit, w_s	

The basic definitions for the Atterberg limits are:

The liquid limit (LL) is the water content w_L , at which the two halves of a pad of soil begin to flow together when tested in a standard liquid limit device at 25 blows.

The plastic limit (PL) is the water content, w_p , at which a 1/8 inch diameter thread of soil just begins to crack.

The plastic index (PI) is the numerical difference between the liquid limit and the plastic limit.

Skempton, in several continuing papers (1948, 1950, 1953) established another index to be used to indicate the colloidal activity of a clay. The activity, A , of a clay is given by the ratio of the plastic index to the clay fraction (percent by weight of particles finer than 2 microns) of a soil. Three classes of clay were recognized in terms of the ratio $PI/clay$ fraction, namely, inactive, normal, and active:

inactive clays: $A < 0.75$

normal clays: $0.75 < A < 1.25$

active clays: $A > 1.25$

Typical values for the various minerals are:

Quartz	0.0
Calcite	0.2
Kaolinite	0.2 - 0.4
Illite	0.5 - 0.9
Calcium montmorillonite	1.0 - 2.0
Sodium montmorillonite	4 or more

The use of the various limits and related functions is well established in the literature. Probably the single most important use is the Unified Soil Classification System in which the fine grained soils are classified by using the liquid limit and the plastic index.

Seed, et al. (1962) conducted a number of tests on artificial soils prepared by mixing commercially available clay minerals with different proportions of sand. A well-defined relationship was established between the clay fraction, activity of the soil, and swelling potential (defined as the percentage of swell under a

1-psi surcharge of a sample compacted at optimum moisture content to a maximum dry density in the standard AASHTO compaction test). A simplified method for the prediction of swelling potential based on the plasticity index gave values within +33 percent error.

Relationships have also been established for such properties as the compressibility of soils and the undrained shear strength.

Free Swell

The free swell test was performed by slowly pouring 10 cc of air-dried soil, passing the No. 40 sieve, into a 100 cc graduated cylinder filled with water and noting the swelled volume of the soil after it comes to rest at the bottom. This test has been used by Holtz and Gibbs (1956) to group clays for swelling potential prediction as shown below.

<u>Swellability</u>	<u>Free Swell in % of Dry Volume</u>
High to Very High	> 100
Medium	50 - 100
Low	< 50

Grain Size Distribution

The grain size distribution or gradation of a soil-like material is a fundamental property of a soil and is not only used as a basis for classification but also indicates much about the potential behavior of a soil. This could also be the case for gouge.

It seems reasonable to expect that sand-like gouge materials would possess the same properties and therefore exhibit much the same character as surficial sands. In the same manner clay-like

gouges could possibly resemble clayey soils. The grain size distribution could serve as a basis for distinguishing between clayey gouges and the sand-like gouges.

The grading of a sand also has an effect on its strength. For instance, a well-graded sand typically has both a smaller void ratio and a larger friction angle. This is because a better distribution of particle sizes produces better interlocking. This trend is shown below in Table VI-2.

Table VI-2

EFFECT OF ANGULARITY AND GRADING ON PEAK FRICTION ANGLE OF SAND
(after Sowers and Sowers, 1951)

Shape and Grading	Loose	Dense
Rounded, uniform	30°	37°
Rounded, well graded	34°	40°
Angular, uniform	35°	43°
Angular, well graded	39°	45°

X-Ray Diffraction

The above mentioned methods used to indicate the type of clay present in a sample were developed for soils found at or near the surface, and that normally have been subject to weathering. These methods might not be valid for clays formed at depth.

X-ray diffraction techniques were used to determine the type of clay minerals present in each sample. Only the 2 micron or smaller size material was used and the percentages estimated from this test for the different gouges does not include material over 2 microns. The percentage estimates are only based on peak

intensities of the various sample components, and the accuracy is therefore limited.

Swelling Pressure

Several methods of making swelling pressure predictions for potentially expansive soils have been outlined in the literature. Holtz and Gibbs (1956) made an extensive study of this problem and recommend an empirical method for identifying and predicting the expansive characteristics of compacted clay. Their method measures the expansion of soils from the air-dry to the saturated condition.

The PVC (potential volume change) test was initially devised to determine swelling characteristics of foundation soils. Procedures and instrumentation are described by Lambe (1960), who developed the technique under the auspices of the Federal Housing Administration.

The volumenometer method described by Bruijn (1961) is based on the initial moisture content, moisture content changes governed by the permeability characteristics, and the boundary conditions of the soil body.

Jennings and Knight (1958) used a double oedometer test for estimating the heave of a structure. They based the results on the degree of dessication, the activity of the soil, overburden, and applied load. Seed, et al. (1962) defined as mentioned earlier the swelling potential as the percentage of swell under a 1-psi surcharge of a sample compacted at optimum moisture content to a maximum dry density in the standard AASHTO compaction test.

Quantitative information about the potential swelling pressure for fault gouge is very difficult to obtain either in the field or in the laboratory. It would naturally be desirable to obtain swelling pressure measurements on relatively undisturbed samples of material since this would more closely control the factors affecting the swelling pressures. However, both the very variable character of gouge material and the difficulty in sampling generally preclude this approach.

It is necessary, then, to use a test procedure that will have the most important factors influencing the swelling essentially unaltered from the in-situ conditions. If this is possible, the results obtained from such a test are not the true swelling pressure of the material but the "relative potential swelling pressure" (Brekke, 1965). This simply means that the results of a single test have no meaning when reviewed alone but must be compared to a large number of test results.

The test procedure used for the swelling pressure tests, modified after Brekke (1965), is presented in Appendix A. Briefly, the procedure requires the separation of the -200 sieve size material from the total sample, oven drying, exposure for 24 hours to 100% relative humidity, compaction and finally testing. This procedure assumes that the type of clay mineral(s), the amount of clay mineral(s), the type of dominant cation and ion concentration remain the same.

The apparatus that was used for these tests has been described by Brekke (1965), and is shown in Figure VI-1. It is a modified consolidometer produced by Geonor of Oslo,

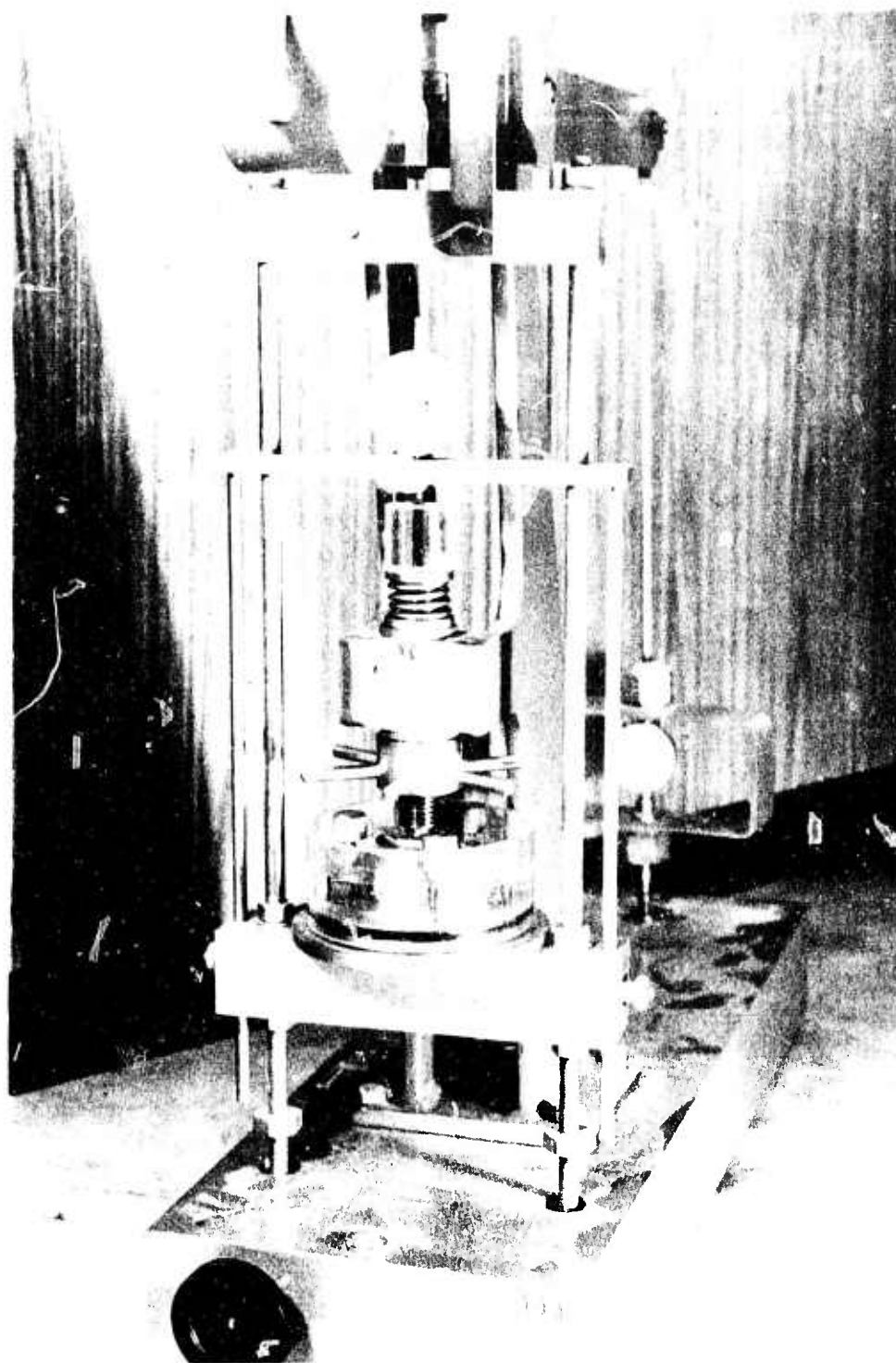


FIGURE VI-1 SWELLING PRESSURE TEST APPARATUS.

Norway. The apparatus is equipped with a device which permits continuous regulation of the load on the sample. After preparation the sample is exposed to water and the swelling pressure is measured by recording the load necessary to maintain a constant sample height. The maximum swelling pressure is generally obtained within 2 hours. The sample is then allowed to expand a certain portion of its volume, generally in increments of 2%, and the procedure is repeated.

Strength Testing

There are two objectives in strength testing in which the deformation and rupture under load are measured. The first is to gain a fundamental understanding of the mechanics of the particular material. The second is to analyze or predict the behavior of the material under field loading conditions. For this study the first objective was considered to be the more important, although achievement of the first objective facilitates achievement of the second.

The direct shear test was selected for determining the shear strength parameters and the shape of the stress-deformation curve for gouge materials because this test most closely approximates the in-situ movement along discontinuities.

The direct shear test employs a cylindrical specimen which is encased in a split box. A normal force is applied to the top of the box after which a shear force drives the top of the box across the bottom, causing the specimen to shear along the plane defined by the split between the box top and bottom. The normal

load is applied by a rigid plate that is free to move as the specimen deforms. The sides of the box are also rigid so that no lateral deformation is permitted. The top and bottom of the specimen are enclosed by a porous material to permit drainage of any pore fluid expelled during the loading.

Penetration Testing

Penetration tests are fairly rapid and simple to perform, and a penetration device was therefore assembled as a possibly valuable tool for assessing the physical properties of gouge.

For a purely cohesive material ($\phi = 0$), the bearing capacity is defined by the equation

$$q = cN_c + P \quad [\text{Meyerhof, 1951}]$$

where q = the ultimate bearing capacity, or penetration resistance

c = the undrained shear strength = $\frac{1}{2}$ the undrained compressive strength

N_c = the bearing capacity factor

P = the overburden pressure

Generally N_c is taken as 9.5, and the overburden pressure, P , is neglected, and the equation becomes

$$c = q/N_c = q/9.5$$

However, most soils are not purely cohesive and the value of N_c may differ considerably from 9.5 and may change with depth of penetration.

In recent studies of the penetration resistance of soils, a method has been developed by Durgunoglu (1972) which determines the ϕ and c strength relationship from penetrometer data. In order to obtain specific values of ϕ and c , however, the method

requires that one of the two be defined by other means.

A field penetration device was as mentioned developed for field testing. The device is hydraulic and consists of a small pump, a pressure gauge, and a piston. Several lengths of threaded pipe are used to position the piston prior to penetration. The penetration device is shown in Figure VI-2a and b.

Experimental Results

The testing program was as stated above, originally designed to attempt to find some simple test or tests which would allow the prediction of the fundamental properties of the various types of gouge material. This meant that if the different types of gouge material outlined in the tentative classification system given in Table IV-2 could be identified by simple tests, then the important properties assumed to be associated with the particular types of gouge could be tested for in more detail.

Of the five gouge types identified in the classification system, only the first four will be dealt with because the fifth, namely porous or flaky calcite and gypsum, can readily be identified without involving laboratory procedures. Chlorite, talc and graphite also fall into this category but these materials characteristically have a wide range in properties and should be studied individually. Therefore, the samples involving these gouge materials have been tested and the results obtained are included in this report.

If it is assumed that gouge materials behave underground in a manner similar to surface materials of the same type, then it would seem reasonable to separate clayey gouges from sand-like gouges on the basis of gradation. Several samples were identified in the field

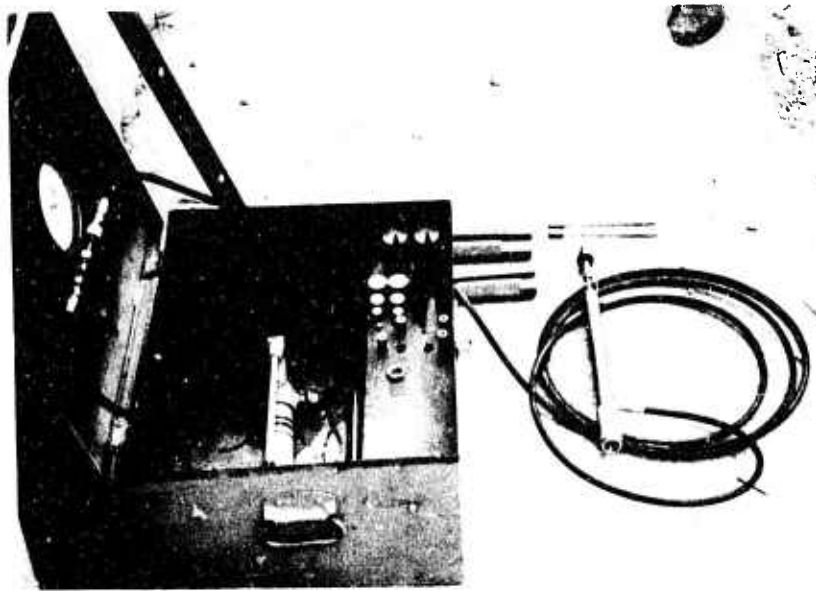


FIGURE VI-2a HYDRALIC PENETROMETER ASSEMBLY.



FIGURE VI-2b HYDRALIC PENETRATION TEST IN GOUGE MATERIAL.

to have the characteristics of running ground and attempts to establish a positive identification of these samples will be discussed. The main difference between the clayey gouges is in their swelling characteristics. These are a function of the type of clay minerals present and it is therefore necessary to identify a test which would reflect the type and amount of clay in the gouge. The Atterberg limits and the activity value would be expected to fill these requirements based on experience with near-surface clays. The swelling pressure test was used to define the potential swellability of the gouge.

A complete listing of the samples taken at the different underground construction sites along with their field description is presented in Appendix B. A complete summary of the test results for both the samples collected from the field and the laboratory prepared specimens is given in Appendix C.

Distinction Between Clayey and Sandlike Gouges

The Unified Soil Classification system separates clayey or fine-grained materials from the sandlike or granular materials on the basis of gradation. If more than 50 percent of the material passes the No. 200 sieve (0.74 mm) it is classified as fine-grained, otherwise it is classified as granular.

The grain size distributions are given in Figures VI-3 to VI-6. All the gouge samples satisfy the granular soil criteria except samples SC6, N6, and HM4. In each case the samples are fairly well graded. That is, each grain size group is represented in the sample in fairly equal amount. The samples fall into the silty sand or clayey sand category except the samples mentioned in the silt or

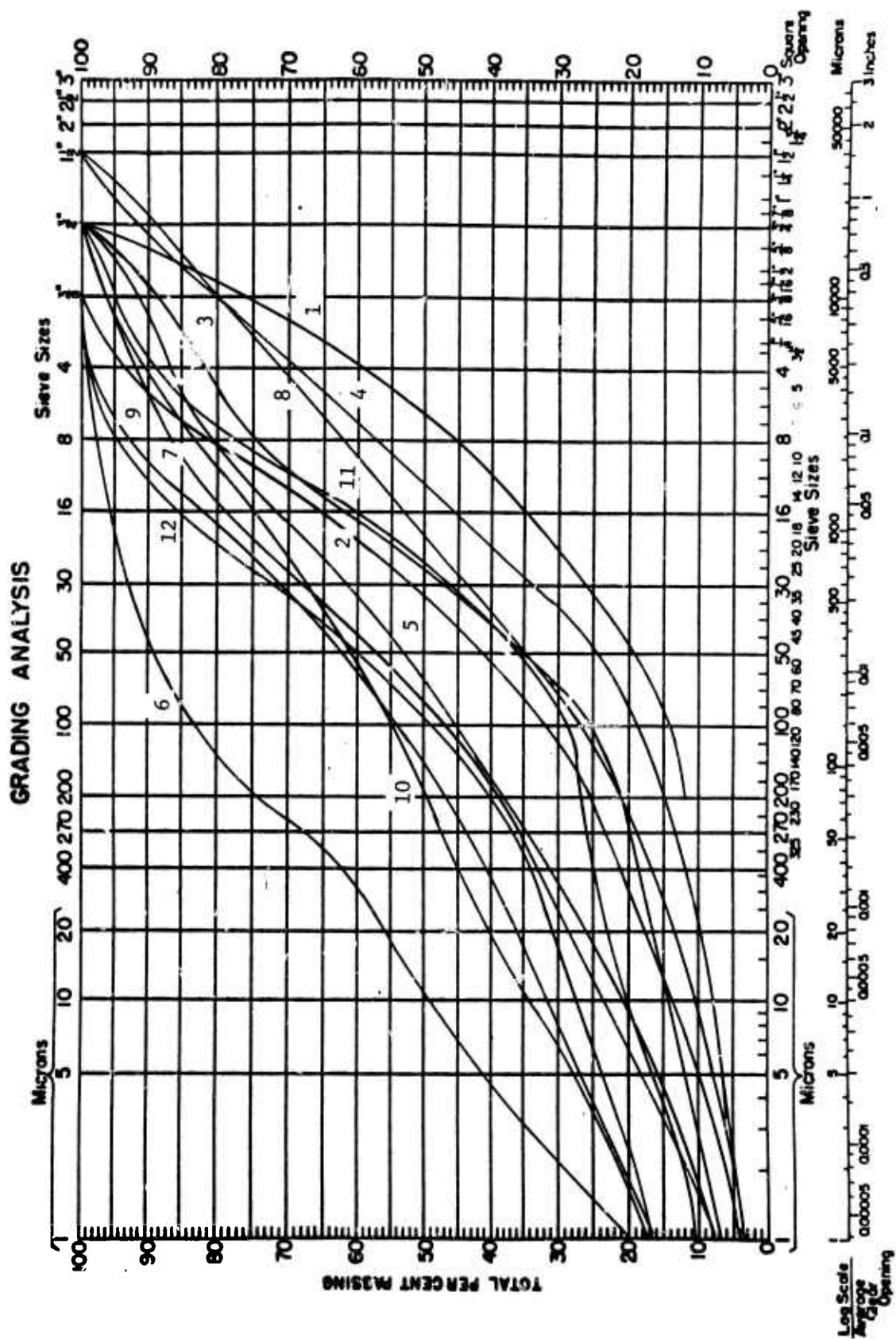


FIGURE VI-3 GRADATION TEST SUMMARY - STRAIGHT CREEK TUNNEL

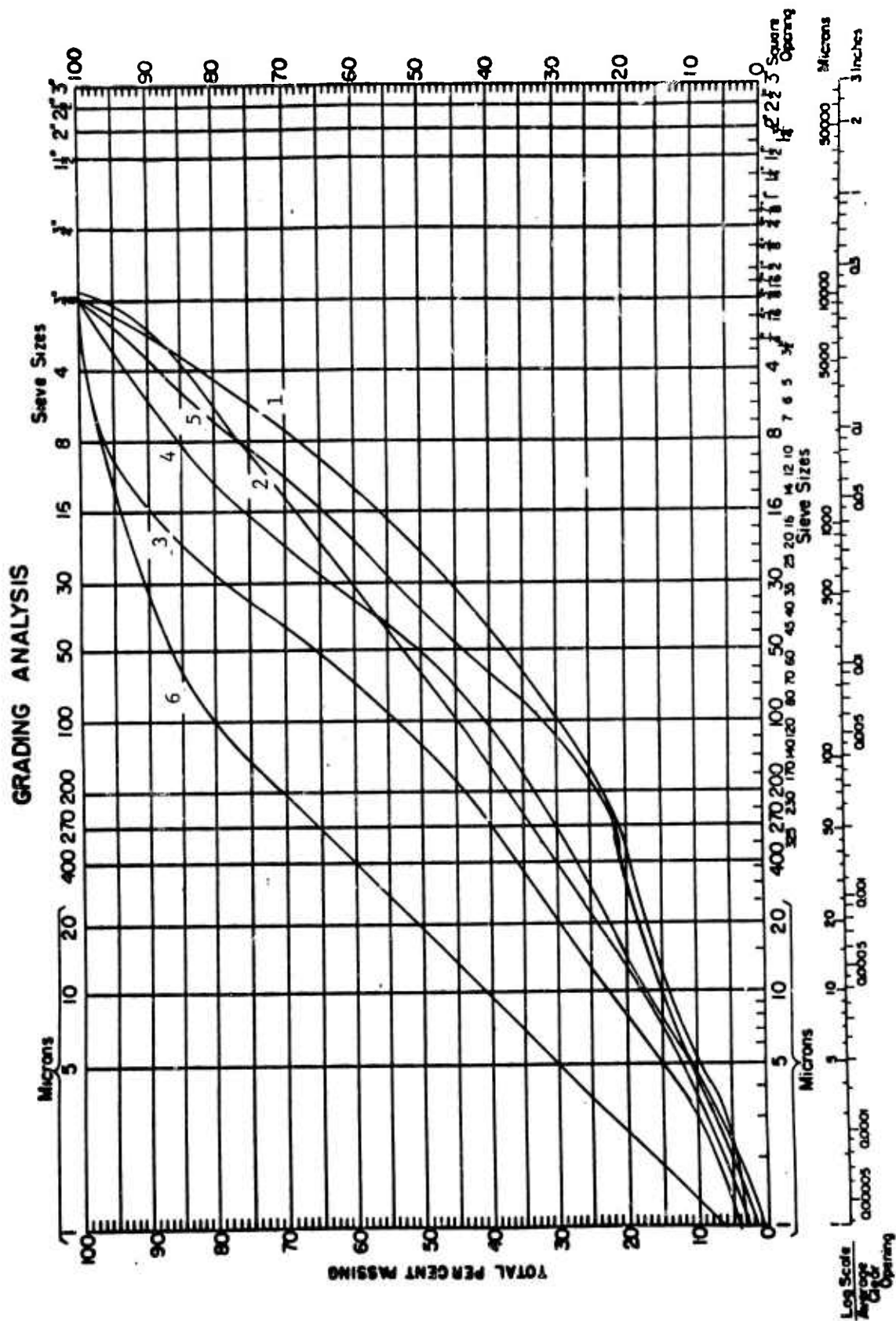
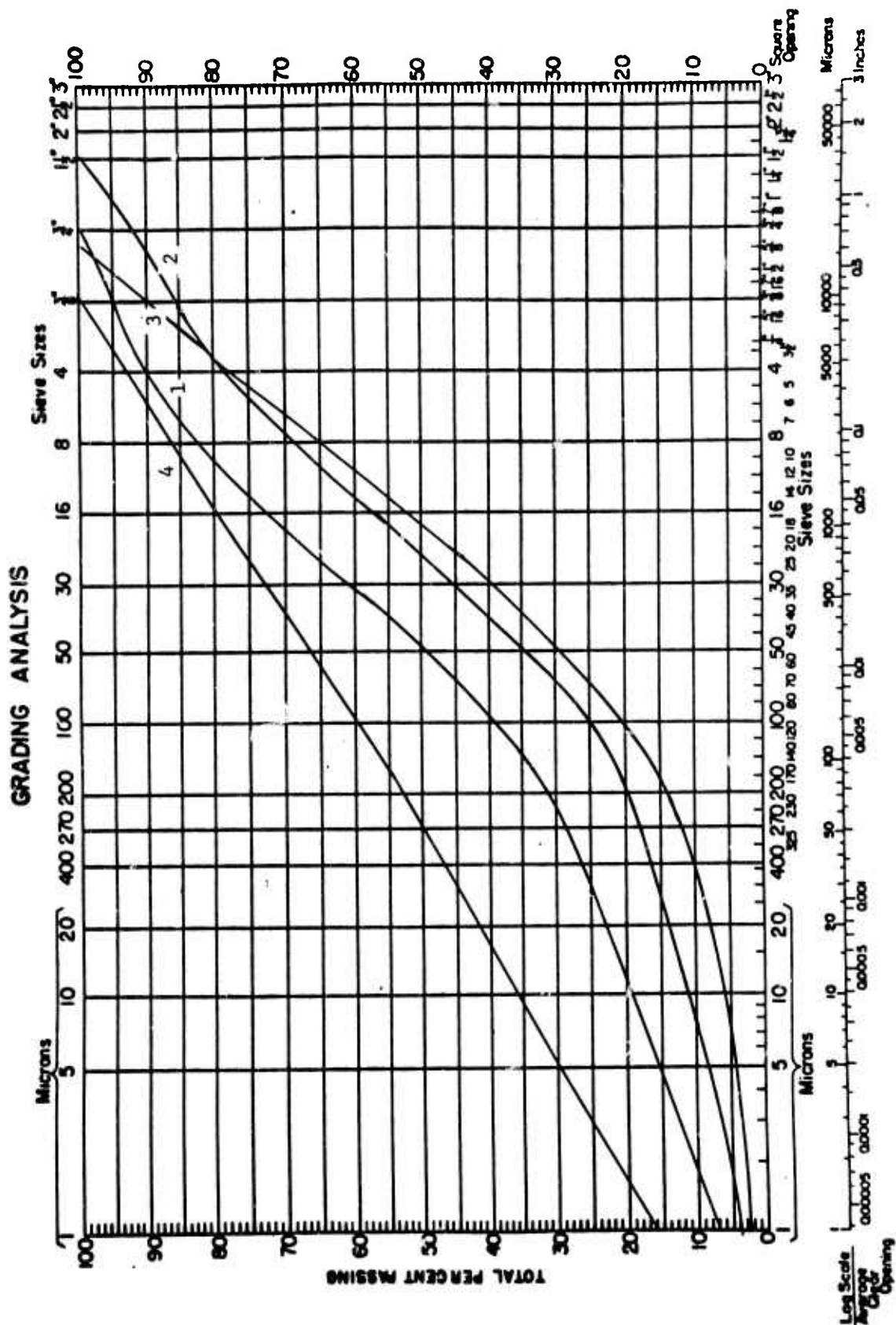
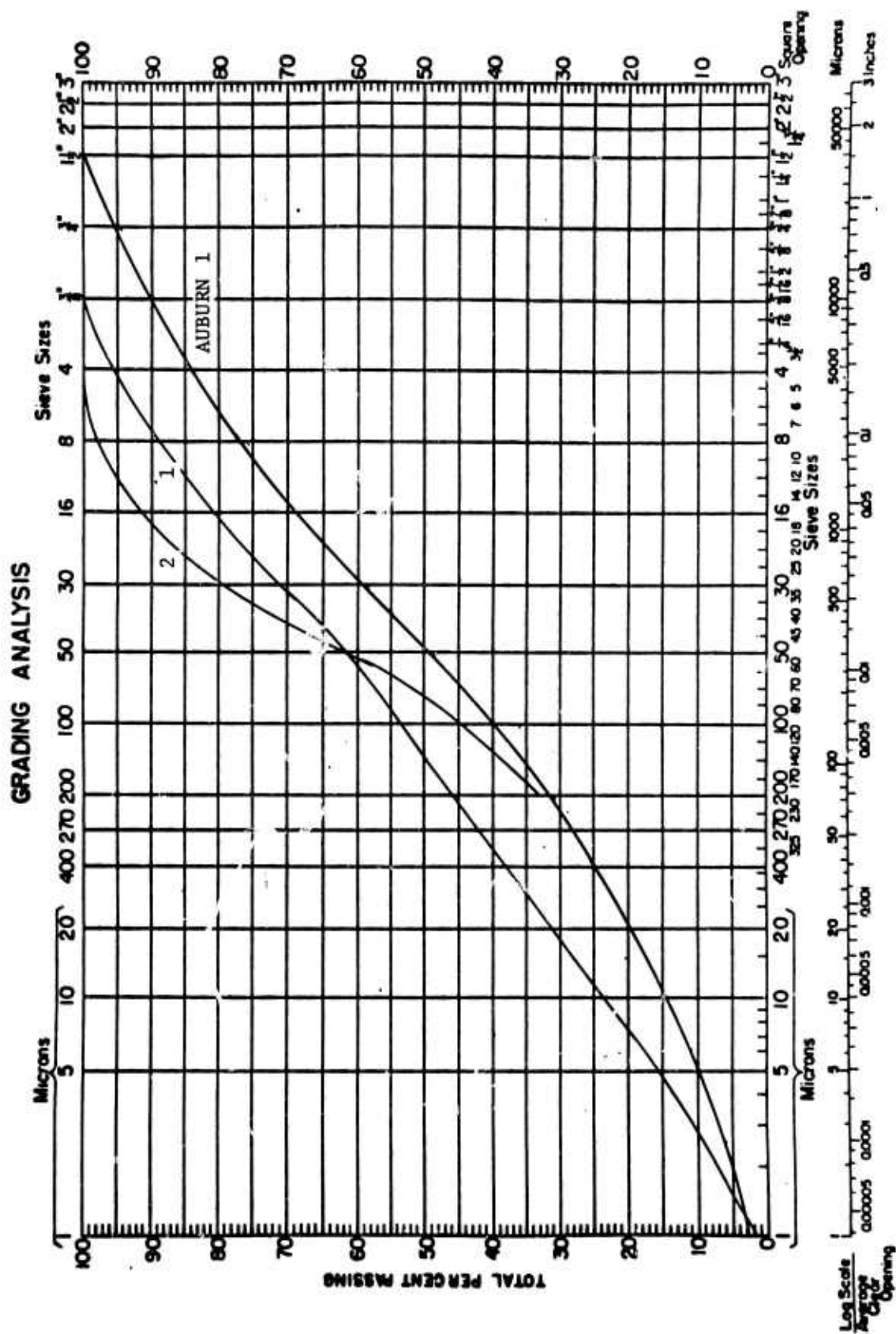


FIGURE VI-4 GRADATION TEST SUMMARY - NAST TUNNEL





clay categories. This is somewhat surprising since most of the samples are clayey in behavior.

In order to act like a running or flowing ground a material should fall into the silty sand area. Unfortunately, there was no extensive example of running ground found in the tunnels visited. However, there were three samples, SC3, SC11 and HM3, that, in the field, exhibited running in small quantities. Each of these materials was wet and came into the tunnels fairly rapidly. These materials are representative of gouge with a very short stand-up time. Note in Figure VI-7 that the three curves are slightly steeper through the sand size portion of the curves than the curves for the other samples given in Figures VI-3 to VI-6. This indicates that a fairly uniform gradation may be characteristic of running ground material.

It seems somewhat dangerous to deduce from the limited data available a firm gradation criterion for running ground. In dry ground, a fine sand or coarse silt without a cohesive fraction (i.e. clay) can run. In wet ground, the running potential is enhanced, and one can expect running (often called flowing when wet) even if the sandy or silty material has a small component of cohesive material. It is not clear what percentage of fines will give sufficient cohesion to prevent runs since density and water pressure also will influence the behavior. It is, however, assessed that a running ground can have at least a 15% fraction passing the No. 200 sieve. This is reflected in the shaded area in Figure VI-7 that indicates the probable grain size criteria for running ground.

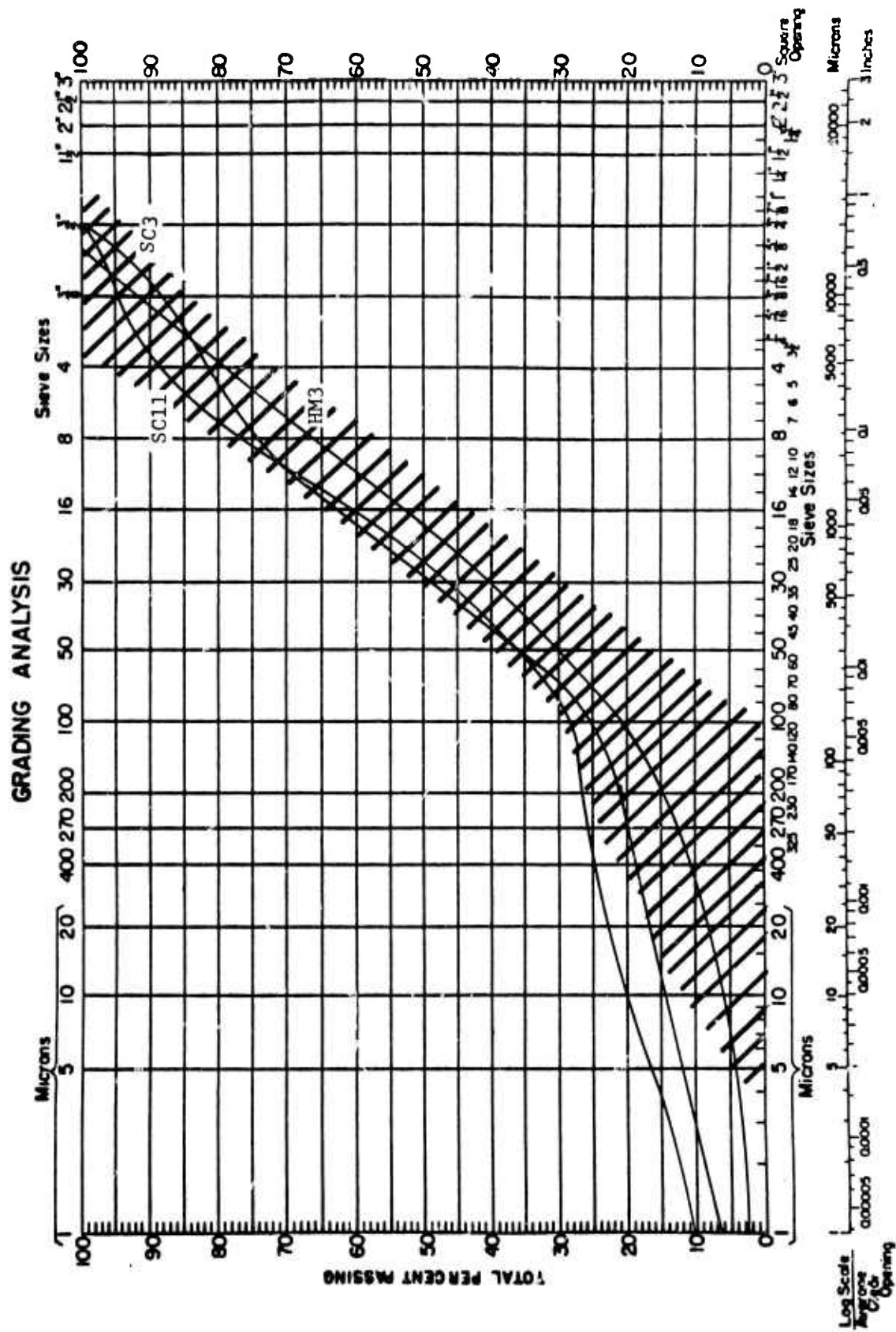


FIGURE VI-7 EXPECTED RANGE IN GRADATION FOR RUNNING GROUND

Chlorite, Talc, Graphite and Serpentine

The minerals chlorite, talc, graphite and serpentine can readily be identified in the field. All are generally formed by alteration, and are all very soft. Chlorite is usually green but may vary from white to brown and black. The hardness is 2 to 2.5. It has a perfect basal cleavage causing it to split into thin sheets. Talc as a mineral has a hardness of 1 to 1.5, varies from white to greenish and has a greasy feel. Graphite is black and has a hardness of 1-2.

Serpentine has a wider range in properties, and is anything from very massive and competent to sheared or fibrous. The hardness ranges from 2.5 to 4 and the color varies from white through all shades of green to black. Serpentine has a greasy feel and has a waxy luster.

The material from the Auburn damsite is in this category as confirmed by X-ray diffraction (approximately 70% talc, 30% chlorite). The test results for this material are given in Appendix B. The only deviation in test results from the other gouge samples is in the Atterberg limits. The liquid limit is substantially lower than the others and there is no plasticity index. The fines were found to be plastic in all the other types of gouge materials tested.

Swelling Clays and Inactive Clays

It is evident from the case histories and from the field observations associated with sampling for this project that the majority of gouge materials fall into one of these two categories.

Terzaghi (1946) described swelling rock as one that advances

into the tunnel by expansion, and indicates that the capacity to swell is limited to the rocks which contain clay minerals such as montmorillonite that have a high swelling capacity. Squeezing rock, on the other hand, is argillaceous rocks that slowly advance into the tunnel without perceptible change in water content.

There is no distinct boundary between these two types of ground conditions in the field. The samples for this study were classified in the field by observing the reaction of the ground to the mining operation. In most cases it was difficult to tell the difference between squeezing and swelling ground. Often, it appeared that a combination of the two was present.

The discussion which follows involves the physical significance of the various tests as well as the interpretation of the results obtained for the clayey gouge samples. The ultimate aim is as mentioned to separate squeezing and swelling ground by some simple test or tests.

Liquid Limit - The physical significance of the Atterberg limits has been thoroughly discussed by Seed, et al. (1964a) and their findings will be used herein. Casagrande (1932) showed that the liquid limit test is analogous to a shear test on the soil and that each blow of the standard device required to close the standard groove could be considered equivalent to approximately 1 gm per sq. cm. of shear strength. Norman (1958) conducted subsequent testing concluding that the shear strength is about 0.8 gm per sq. cm. per blow. Allowing for shear strength resulting from internal friction, it appears that, at the liquid limit, the shear strength of about 20 gm per sq. cm. (25 blows) is due to

net attractive forces between the clay particles. It can therefore be reasoned that since the liquid limit depends on the intensity of the net attractive forces between clay particles and therefore on the surface activity of the clay, the spacing between particles must be such that the net attractive force is reduced to a value which corresponds to a shear strength of 20 gm per sq. cm. The spacing between particles is controlled by the water content.

If it is assumed that the interparticle forces on the nonclay portion of a sample are negligible, and also that this portion has a negligible amount of adsorbed water, then the entire water content may be associated with the clay fraction. Further, it can be assumed that the nonclay particles are completely surrounded by clay with its associated water. Then, for a given type of clay the water content of the clay at the liquid limit w_{cl} , will be fixed and will depend only on the clay. That is, the value of the liquid limit of a soil will depend only on the w_{cl} and the portion of the clay present in the soil.

From this reasoning a graph can be constructed relating the liquid limit of the clay portion, the liquid limit of the sample, and the percent of clay in the sample.

If all the water is associated with the clay fraction, then

$$\begin{aligned} \text{Water content of clay} &= \frac{\text{Weight of water}}{\text{Weight of clay}} = \frac{\frac{w}{100} W_s}{\frac{c}{100} W_s} \\ &\times 100\% = \frac{100w}{c}\% \end{aligned} \quad (1)$$

where c = % clay in sample

w = % water in sample

W_s = dry weight of sample

If clay controls the behavior, a soil will be at its liquid limit, w_l , when the clay is at its liquid limit, w_{cl} and therefore

$$w_{cl} = \frac{100w_l}{c} \% \quad (2)$$

or

$$w_l = \frac{c}{100} w_{cl} \quad (3)$$

This says that for material containing a given type of clay having a unique value of w_{cl} the liquid limit is directly proportional to the proportion of particles in the clay fraction of the material. This is true only when the nonclay particles are separated or when the clay particles and water is greater than the volume of the voids in the nonclay fraction. For a sample containing 100 grams of nonclay particles at its loose void ratio of e_L , the volume of voids is

$$VV = \frac{100}{G_{sg}} \cdot e_L \quad (4)$$

where G_{sg} is the specific gravity of the solids.

The weights and corresponding volume relationship for an unknown, X , quantity of clay is shown in Figure VI-8, where G_{sc} is the specific gravity of the clay solids. The volume of clay and water necessary to just fill the voids, VV , of the nonclay fraction is

$$\frac{100}{G_{sg}} e_L = \frac{X}{G_{sc}} + \frac{X \cdot w_{cl}}{100} \quad (5)$$

and solving

$$X = \frac{100 e_L}{G_{sg} \left(\frac{1}{G_{sc}} + \frac{w_{cl}}{100} \right)} \quad (6)$$

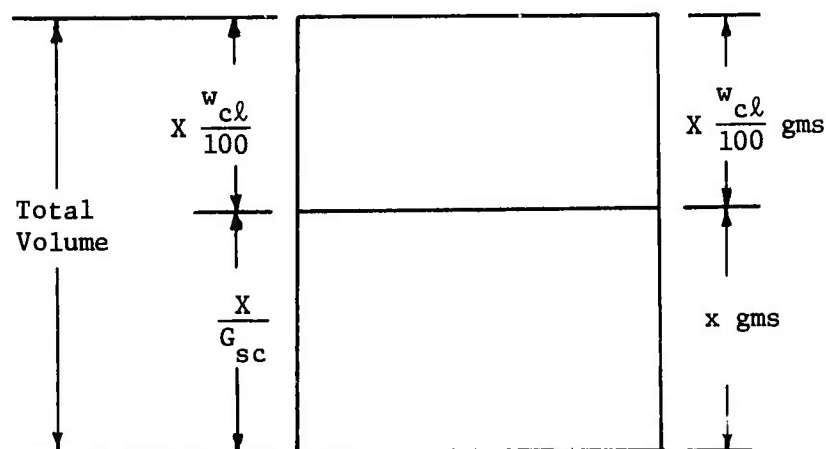


FIGURE VI-8 CLAY WATER SYSTEM

Because the X gms of clay are mixed with 100 gms of nonclay, the percentage of clay, c_{fv} , required to just fill the voids is

$$c_{fv} = \frac{X}{100 + X} \cdot 100 \quad (7)$$

With the aid of equations 3, 6 and 7 a relationship between the liquid limit of a sample w_L , the liquid limit of a given clay, w_{cl} , and the percentage of clay in the sample can be developed. The results are plotted in Figure VI-9.

This plot can be used to determine the type of clay that is present in a given gouge sample. The Atterberg limits were determined by using the material passing the No. 40 sieve. Since Figure VI-9 requires knowledge of the percent clay in the material upon which the liquid limit was obtained, the gradation results must be modified to reflect the percent of 2 micron material (clay size) in the portion of the sample passing the No. 40 sieve. This is a simple calculation and is given by

$$\% \text{ clay in the liquid limit sample} = \frac{\% \text{ Passing } 2\mu}{\% \text{ Passing No. 40}} \quad (8)$$

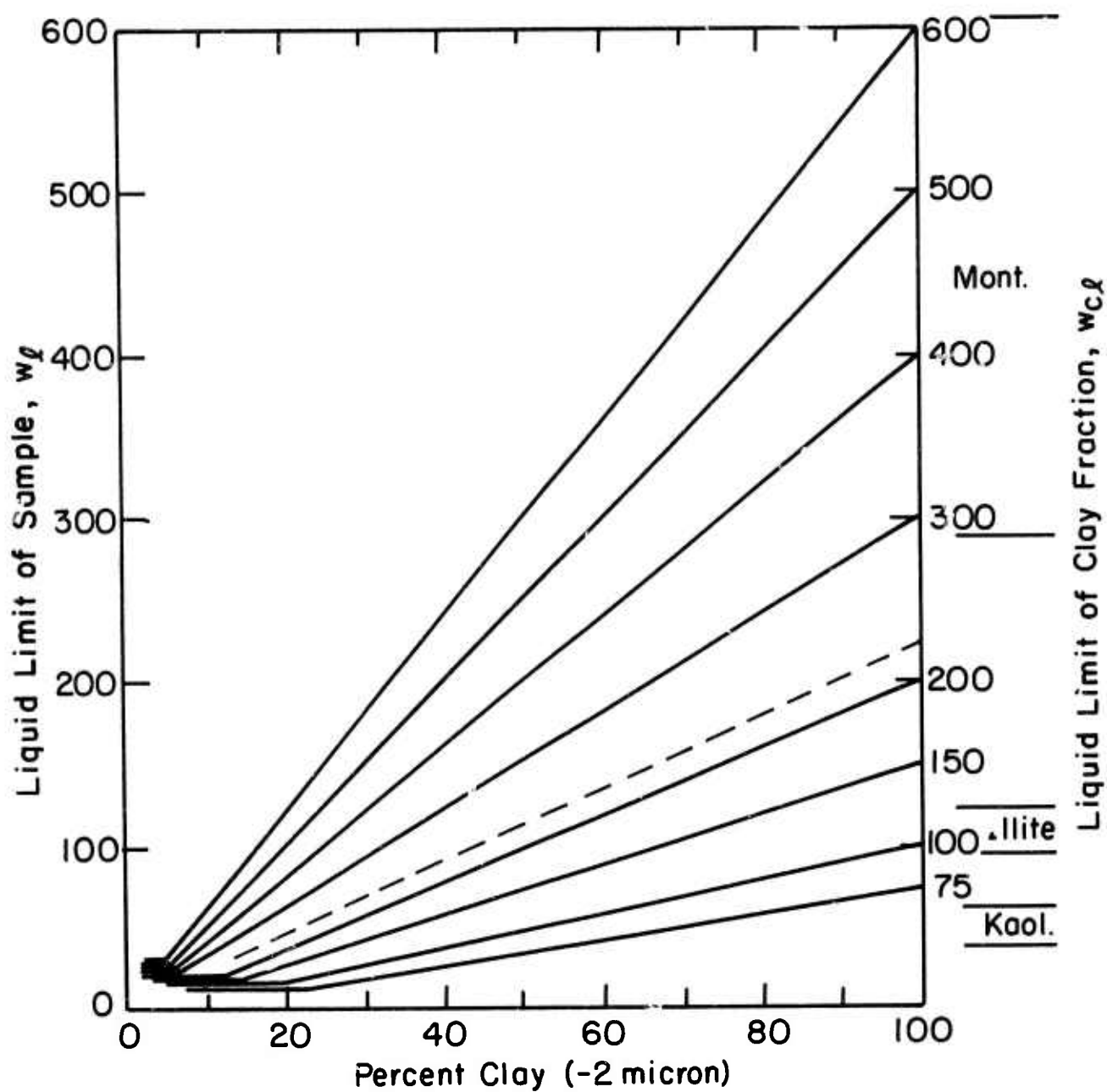


FIGURE VI-9 RELATIONSHIP BETWEEN LIQUID LIMIT OF THE SAMPLE, LIQUID LIMIT OF THE CLAY FRACTION, AND PERCENT CLAY (after Seed, et al., 1964a)

By relating the liquid limit of the clay, w_{cl} , to known values of liquid limit, Figure VI-9 can be used to predict the type of clay minerals in a gouge material. Table VI-3 presents the Atterberg limits of the various types of clay. These values are shown to the right on Figure VI-9.

As an example, Figure VI-3 shows for Sample SC-2 6% passing the 2 micron size and 36% passing the No. 40 sieve. The percent clay in the liquid limit test is then 16.5%. The corresponding liquid limit value, w_l , is 37% and the resulting plot is shown by the dashed line in Figure VI-9.

The dashed line intersects the w_{cl} boundary at 225% between limits of the different clays. This indicates a mixture of clay minerals, a result substantiated by X-ray diffraction data which shows the -2 micron portion to contain 50-60% montmorillonite and 40-50% kaolinite. The remaining data could also be plotted in this manner.

Plastic Limit

The primary physical significance of the plastic limit is that it is a lower bound of the range in water contents within which a soil exhibits plastic behavior. Terzaghi (1926) suggested that for moisture contents equal to or smaller than the plastic limit, the physical properties of the water are no longer identical with those of free or ordinary water.

Using the same reasoning as used for the liquid limit, a relationship between the plastic limit and the clay content may be developed. This leads to the plot shown in Figure VI-10 of plastic limit of the sample vs. percent clay. The lines denote the

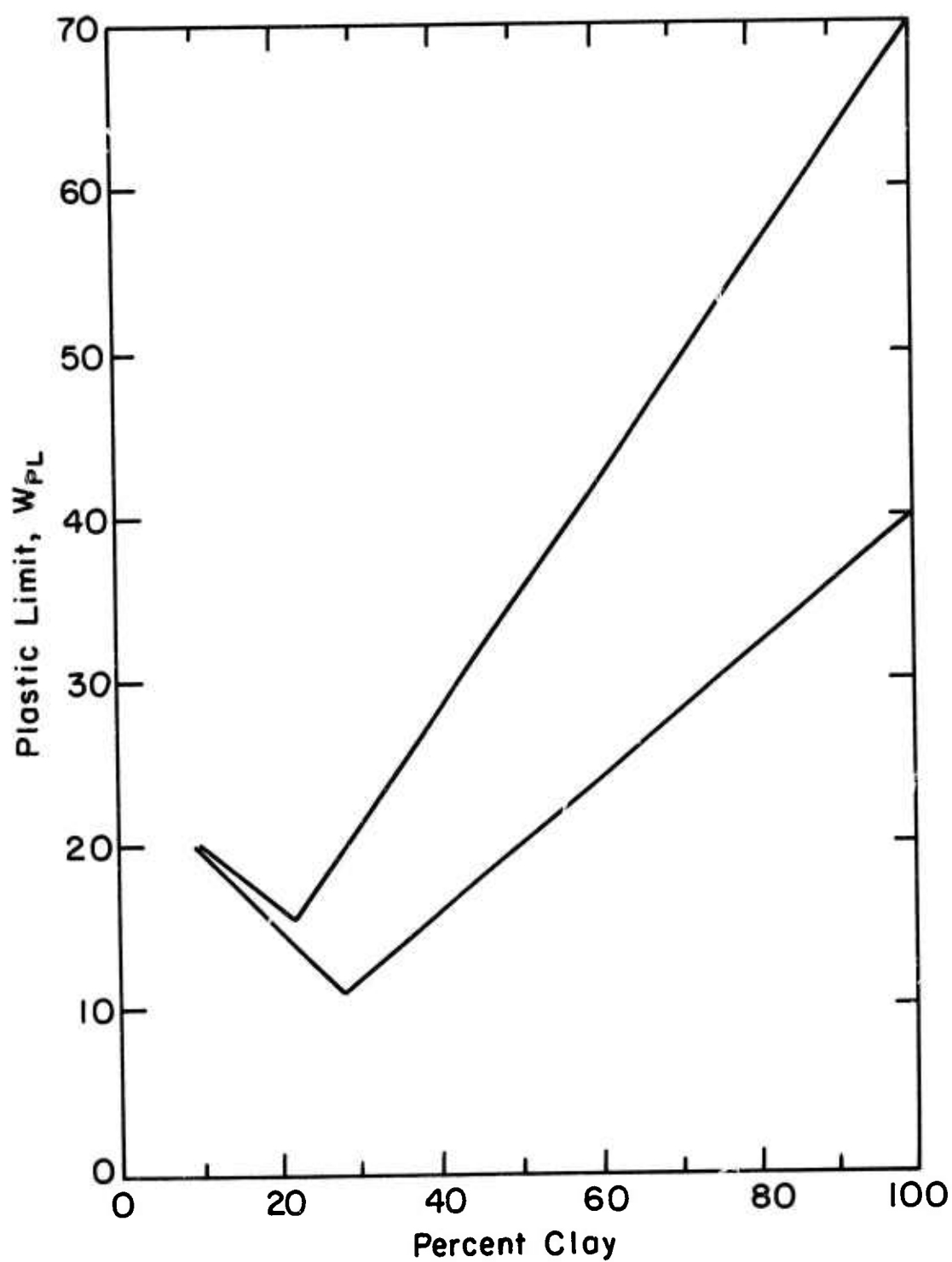


FIGURE VI-10 RELATIONSHIP BETWEEN PLASTIC LIMIT AND PERCENT CLAY (after Seed et al., 1964a)

normal range in plastic limit values of the clay fraction.

Seed et al. (1964a) point out that the plots near the origin are curved and present data to show this. The curved portion reduces the usefulness of this plot.

Plastic Index and Activity

The plastic index ($PI = w_L - w_p$) gives the magnitude of the water content range over which the soil remains plastic. As indicated by the data in Table VI-3, the plastic limit for pure clays generally falls between 40% and 70%, while the

Table VI-3

ATTERBERG LIMITS OF CLAY MINERALS (after Lambe and Whitman 1969)

Mineral	Exchange- able Ion	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Shrinkage Limit (%)
Montmorillonite	Na	710	54	656	9.9
	K	660	98	562	9.3
	Ca	510	81	429	10.5
	Mg	410	60	350	14.7
	Fe	290	75	215	10.3
Illite	Na	120	53	67	15.4
	K	120	60	60	17.4
	Ca	100	45	55	16.8
	Mg	95	46	49	14.7
	Fe	110	49	61	15.3
Kaolinite	Na	53	32	21	26.8
	K	49	29	20	-
	Ca	38	27	11	24.5
	Mg	54	31	23	28.7
	Fe	59	37	22	29.2
Attapulgite	H	270	150	120	7.6

liquid limit generally varies between 80% and 600%. As a result, the plastic index can range between 40% and 500%. The PI may then reflect the type of clay present.

Activity ($A = \frac{PI}{\% \text{ finer than } 2\mu}$) has proven to be a better indicator of clay type than PI, in single clay systems. This is because a great increase in surface area with decreasing particle size, and a resulting change in the amount of attracted water is largely influenced by the amount and type of clay involved in the material. Thus, activity has greater usefulness than simply identifying swelling potential in simple systems.

Figure VI-11 is a plot of the plasticity index against the clay fraction for the gouge samples. Each data point has been labeled with its activity and lines of equal activity have been constructed. It can be observed that the samples tested have a wide variation in terms of activity.

Seed et al.(1946b) pointed out that the activity serves to identify the clay regardless of the amount of granular material present, provided that only one type of clay is present. However, the activity varies widely for different clay combinations. Stated another way, different clay combinations can give the same activity. This is shown by comparing the samples with essentially the same activity as done in Table VI-4.

X-ray Diffraction Results

The gouge samples were each analyzed by X-ray diffraction to make qualitative and semi-quantitative determinations of the clay mineralogy. A complete description of the technique used is presented in Appendix A. Appendix B contains the results of the clay mineral analysis.

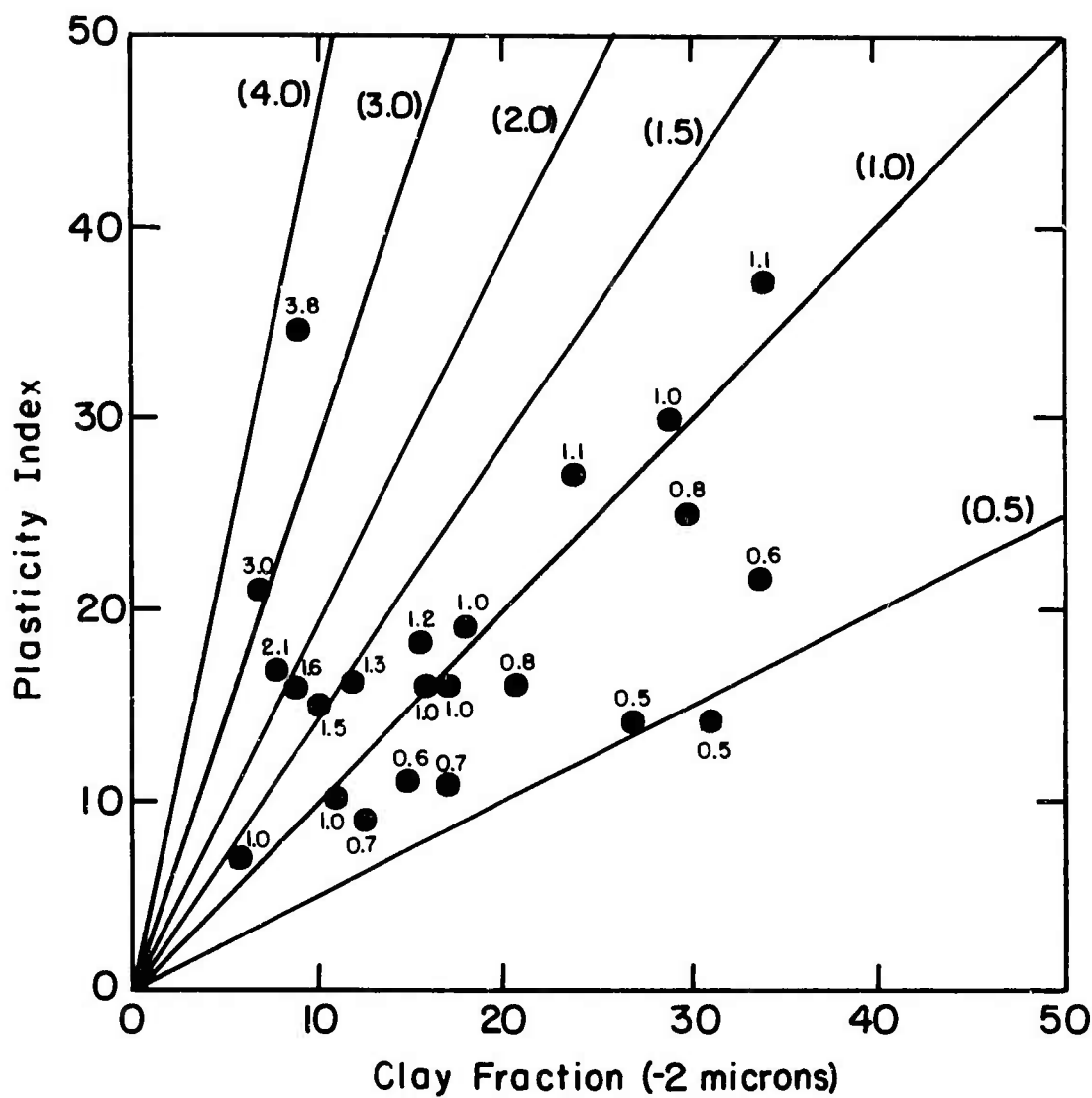


FIGURE VI-11 RELATIONSHIP BETWEEN PLASTICITY INDEX AND CLAY FRACTION FOR GOUGE MATERIAL

Table VI-4

CLAY MINERALOGY OF EQUAL ACTIVITY SAMPLES

Sample	Activity	% Montmorillonite	% Kaolinite	% Illite	% Other
SC2	1.2	56	44	-	-
SC7	1.0	60	40	-	-
SC10	1.1	80	14	6	-
N6	1.0	9	58	9	24
NM1	1.0	65	26	9	-

Following the theme that the activity points to the clay mineralogy, the data in Figure VI-12 was collected. Laboratory prepared samples have a very good correlation, however, the relationship between the activity and the percent montmorillonite in the natural system is shown to give a wide scatter. This result would seem to point out that using the activity as an indicator for the mineralogical composition of natural gouge materials as outlined previously is quite questionable.

Swelling Pressure Test

The objective of this phase of the test program was to separate squeezing ground from swelling ground using the results of laboratory tests. The most reliable indication of swelling ground in the tunnel that can be obtained of laboratory tests is the swelling pressure test.

The swelling pressure test was conducted on the material passing the No. 200 sieve for each sample. Most of the tests were carried out as outlined by Brekke (1965) using 20 gms of material exposed to 100% relative humidity for 24 hours, placed in the machine and statically compacted under a pressure of 200 tons/m² for 24 hours.

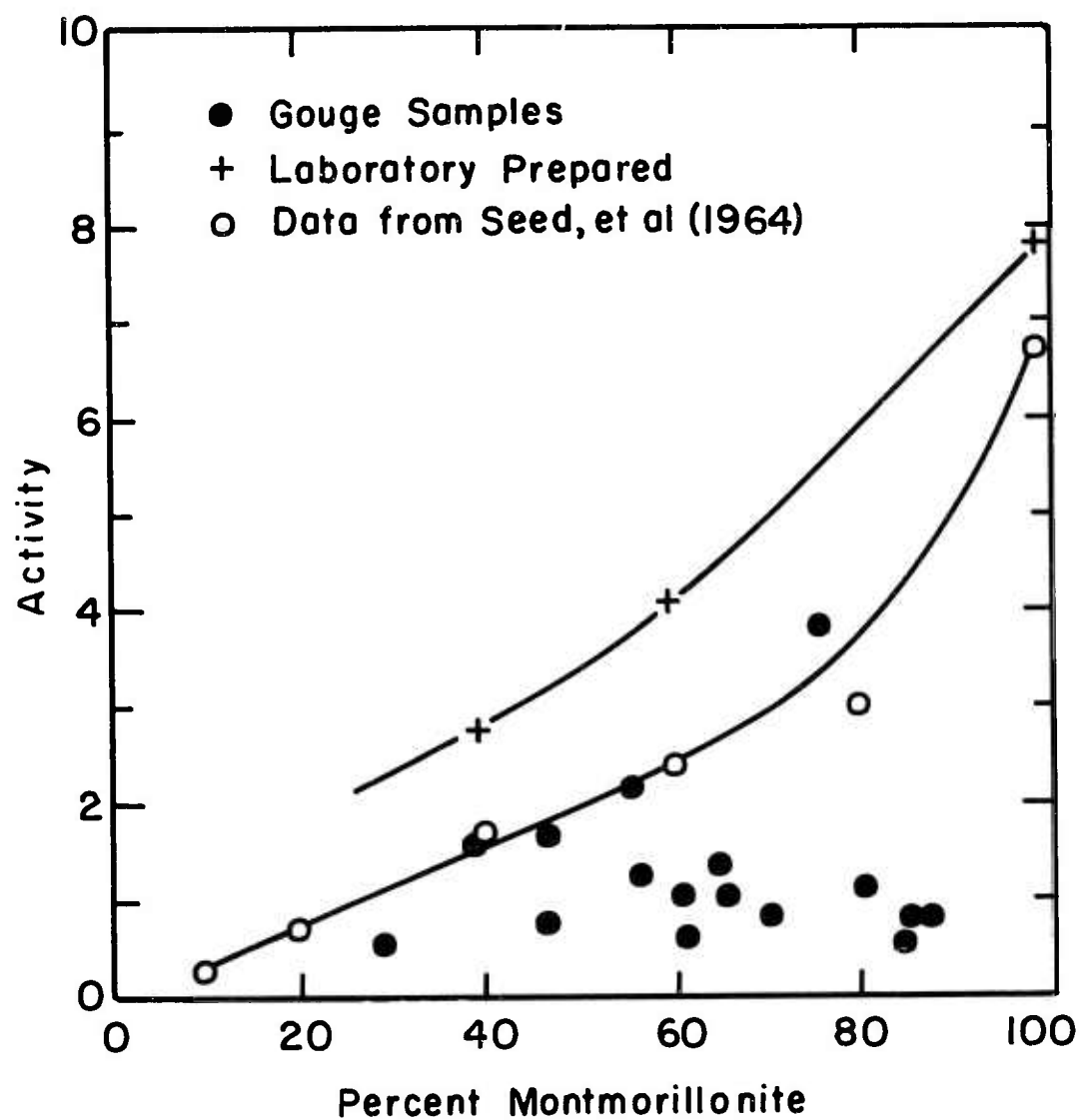


FIGURE VI-12 RELATIONSHIP BETWEEN ACTIVITY AND PERCENT MONTMORILLONITE.

Water is then supplied to the sample and the swelling pressure mobilized at a constant sample height is measured. Similar results are obtained after allowing 2% and 4% volume increase. A typical time-pressure curve is shown in Figure VI-13. The maximum swelling pressure is obtained in a fairly short time period, generally less than 2 hours. The swelling pressure then falls off slightly, probably due to stress relaxation within the sample. As shown in Figure VI-13 the swelling pressure decreases drastically with a very small allowed volume change, in this case 2% and 4%. The swelling test results for the different samples are given in Figures VI-14 - 16.

Quantitative distinctions between swelling and non-swelling ground from laboratory swell test results will be discussed later in this chapter. Because the swelling test apparatus is not widely available and because the apparatus and test procedure are moderately complex for use by inexperienced personnel, a correlation of swelling pressure with the results of a more simple, well-known laboratory test was sought. The Atterberg Limits test was considered a prime candidate because of its simplicity. The Atterberg Limits test yields liquid limit, plastic limit, plasticity index, and together with the grain-size distribution, the activity. Relationships between these quantities were discussed earlier in this chapter. Since the swelling pressure is dependent upon the clay present and the activity is a measure of the type of clay present and because a fairly good correlation has been found between activity and swelling pressure for surface clays, it was logical to try this correlation first, as given in Figure VI-17. However,

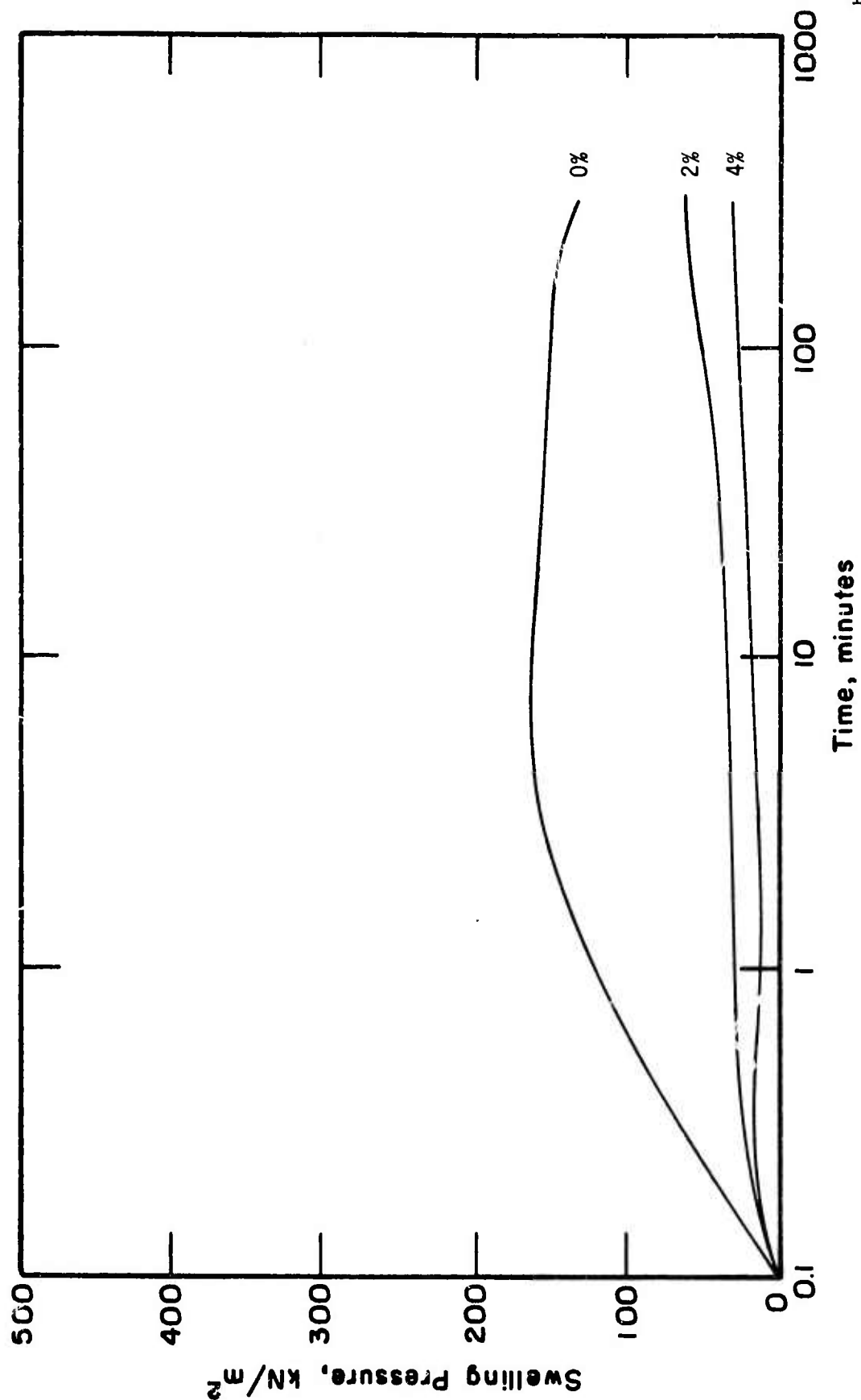


FIGURE VI-13 TYPICAL TIME-SWELLING PRESSURE RELATIONSHIP

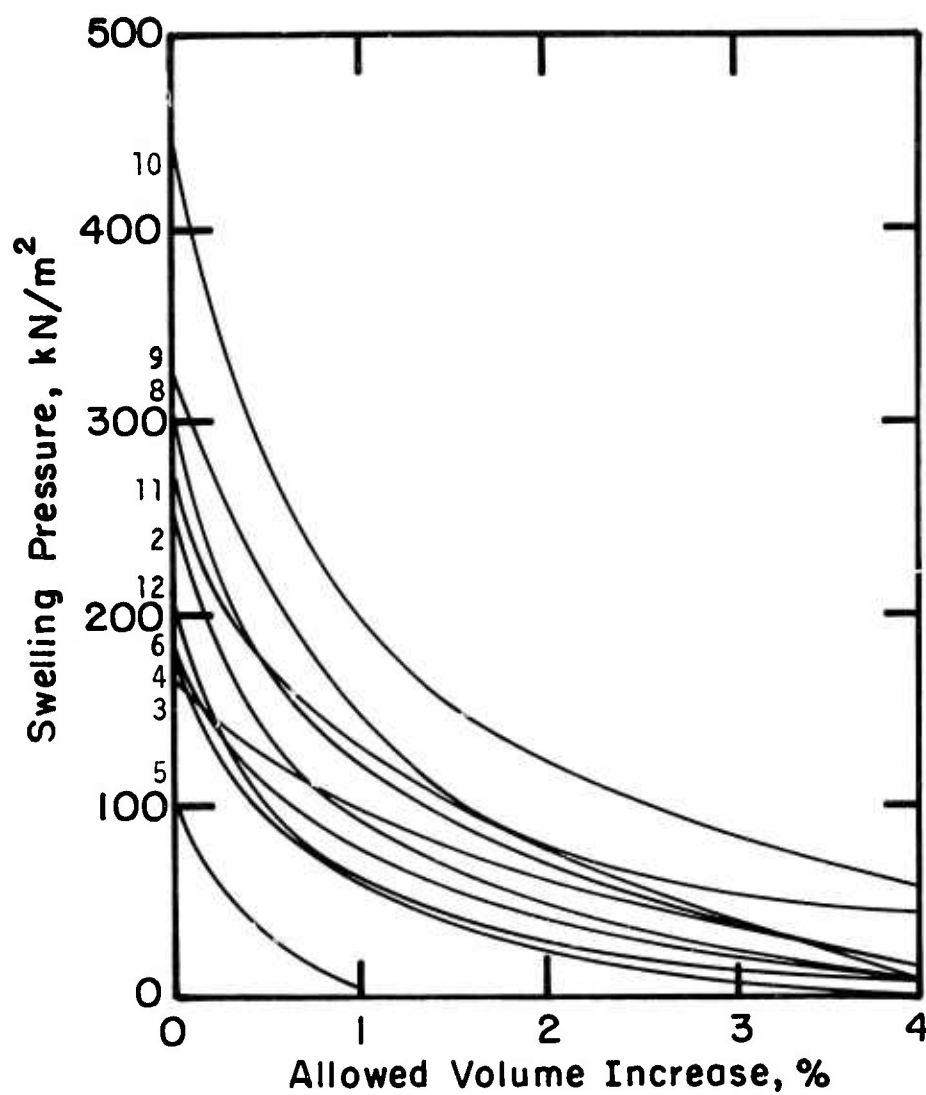


FIGURE VI-14 SWELLING PRESSURE TEST RESULTS - STRAIGHT CREEK TUNNEL.

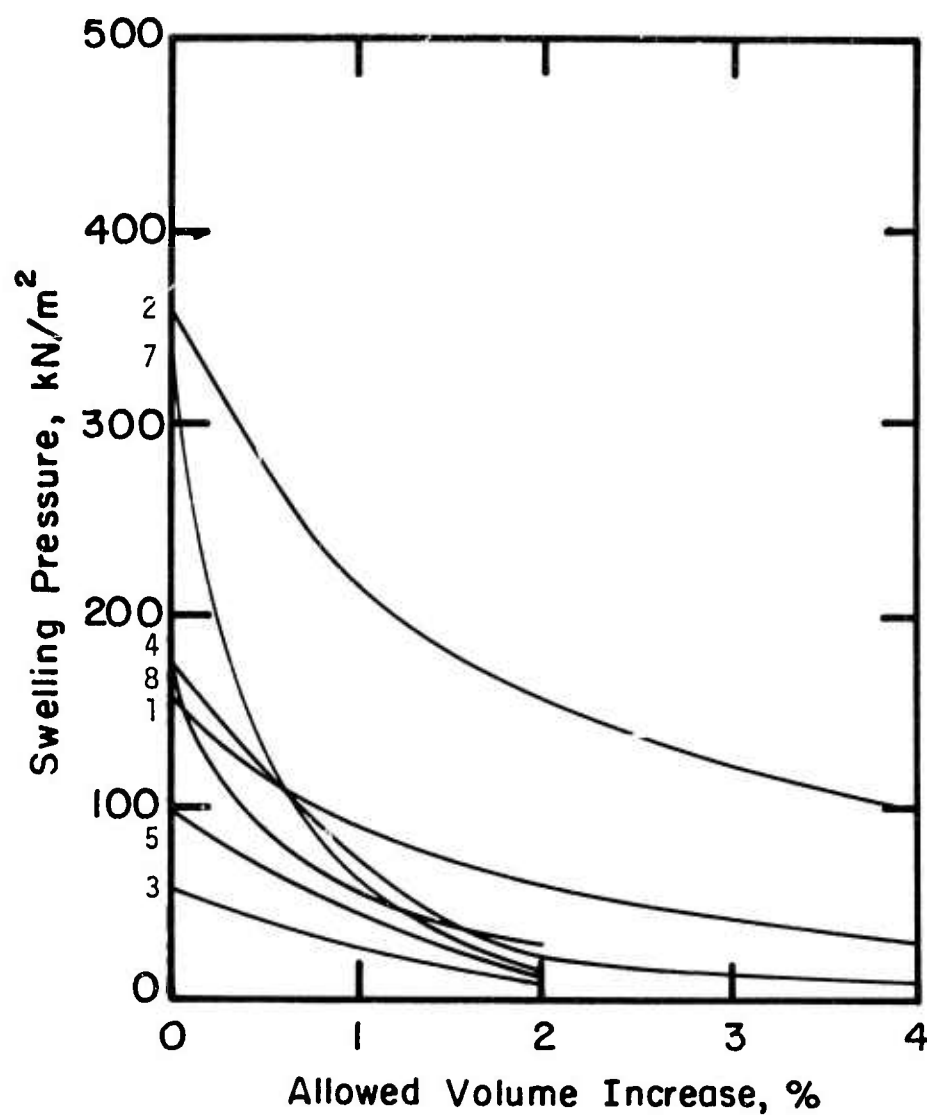


FIGURE VI-15 SWELLING PRESSURE TEST RESULTS - NAST TUNNEL.

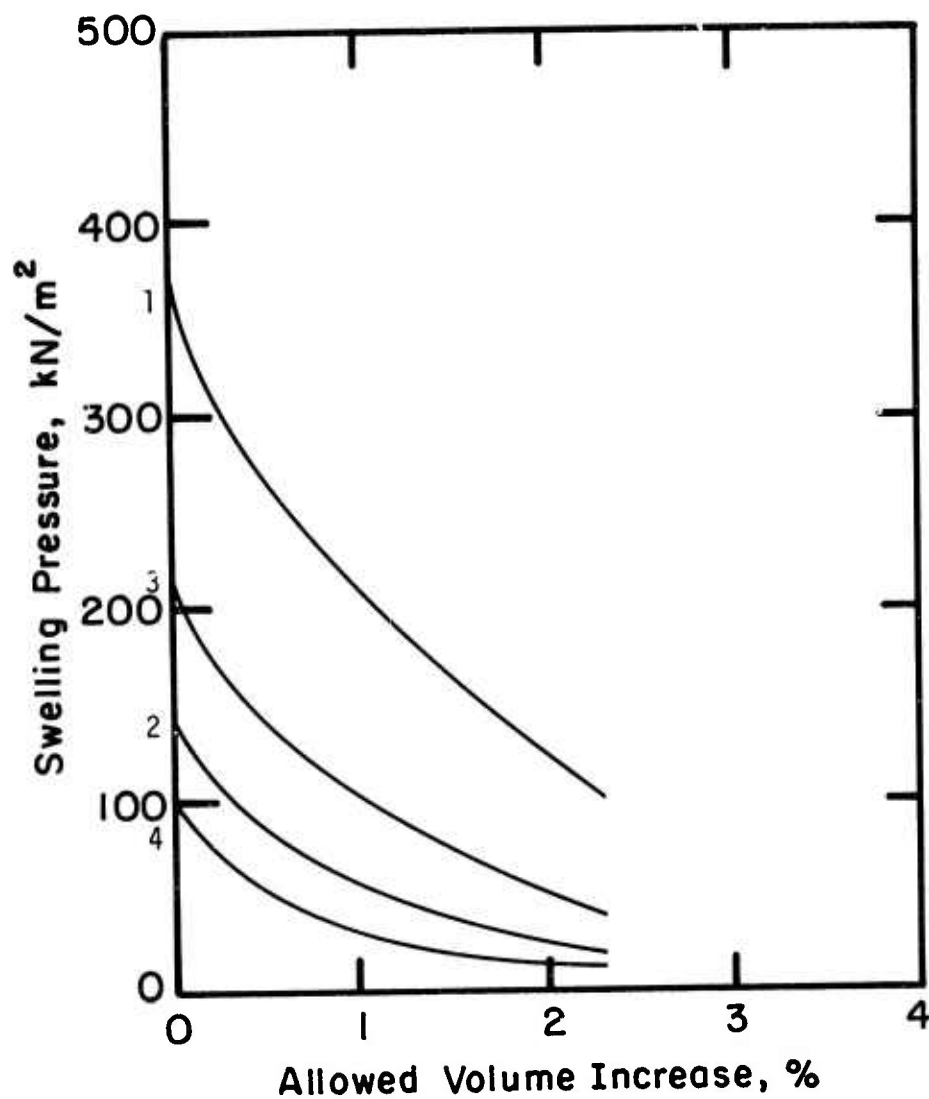


FIGURE VI-16 SWELLING PRESSURE TEST RESULTS - HENDERSON MINE HAULAGE TUNNEL.

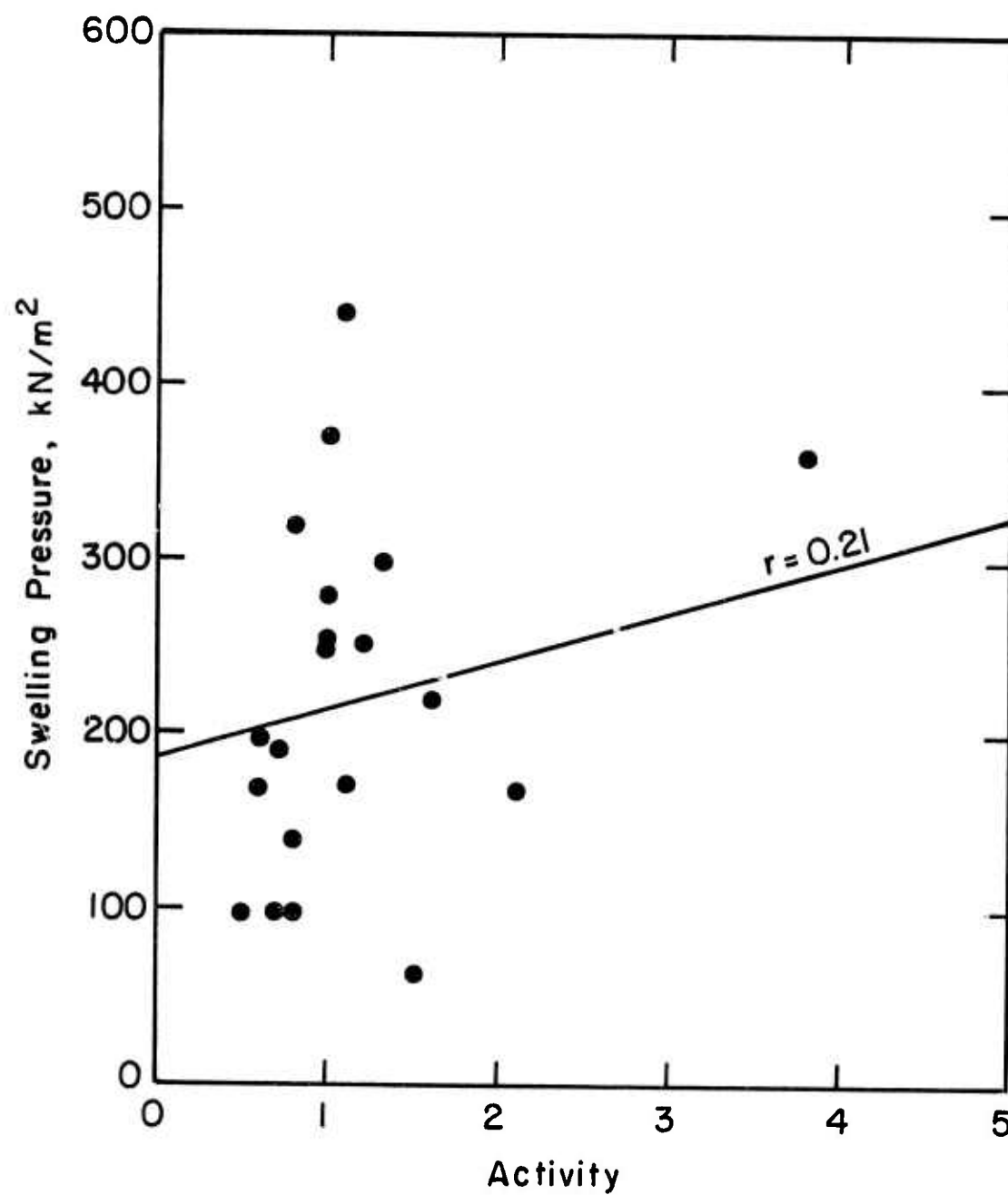


FIGURE VI-17 RELATIONSHIP BETWEEN SWELLING PRESSURE AND ACTIVITY.

the correlation between these two parameters, as given by a least squares linear regression analysis, is poor.

Since the activity did not correlate well with the measured swelling pressure, it was decided that a careful reassessment of the swelling test procedures was warranted.

Swelling Pressure Phenomenon and Testing

A number of authors (Dawson, 1959; de Bruyn, 1956; McDowell, 1959; Ladd and Lambe, 1961; Seed, 1962; Brekke, 1965) have dealt with the problem of identifying expansive soils and predicting the swelling potential of the soils. Many have provided correlations between various index properties and swelling pressure, for the most part with limited success.

In performing tests to measure the swelling pressure of a material it should be realized there are two processes involved which must be accounted for. The extent to which a material can swell or the potential for swelling is a function of the type and amount of clay present. The extent to which a material actually swells or the magnitude of the swelling pressure is determined by environmental factors. These factors are listed in Table VI-5.

The Group 1 factors affect the potential swellability that the gouge may have and depend only on the soil properties. Therefore, the Group 1 factors can be determined by laboratory testing. The factors in Group 2 determine the extent to which the potential swellability will be realized and depend only on environmental conditions or boundary conditions.

This study is concerned primarily with the Group 1 factors in as much as they affect the relative potential swellability of

Table VI-5

FACTORS INFLUENCING THE SWELLING PRESSURE
OF CLAYEY GOUGES

<u>Group 1</u>	<u>Group 2</u>
Type of clay mineral	Structure of material
Amount of clay mineral	Density of material
Particle sizes of clay mineral	Availability of water
	Electrolyte concentration of water
	Possibility of volume change during swelling
	Eventual counterpressure (e.g. lining)

the gouge. The laboratory tests determine only the nature of the gouge particles, not the environmental conditions. Therefore, it is impossible to develop relationships between the laboratory data and the amount of swelling to be expected in an underground structure. It may be possible, however, to construct relationships for the relative potential swellability (Brekke, 1965) and then relate that to field conditions.

To be able to compare the swelling pressure in different gouges it is necessary to compare swelling pressures developed under a standard set of conditions. Initially, tests were conducted by compacting the material under a standard static load. Several correlations of different parameters with the swelling pressures were found to be fair, one of which is the swelling pressure vs. the liquid limit of the sample as shown in Figure VI-18. This should be expected because the liquid limit reflects all of the Group 1 factors, type of clay, amount of clay and clay size. Rokkoengen (1972, personal communication) reported that investigations at the Technical University of Norway, Trondheim,

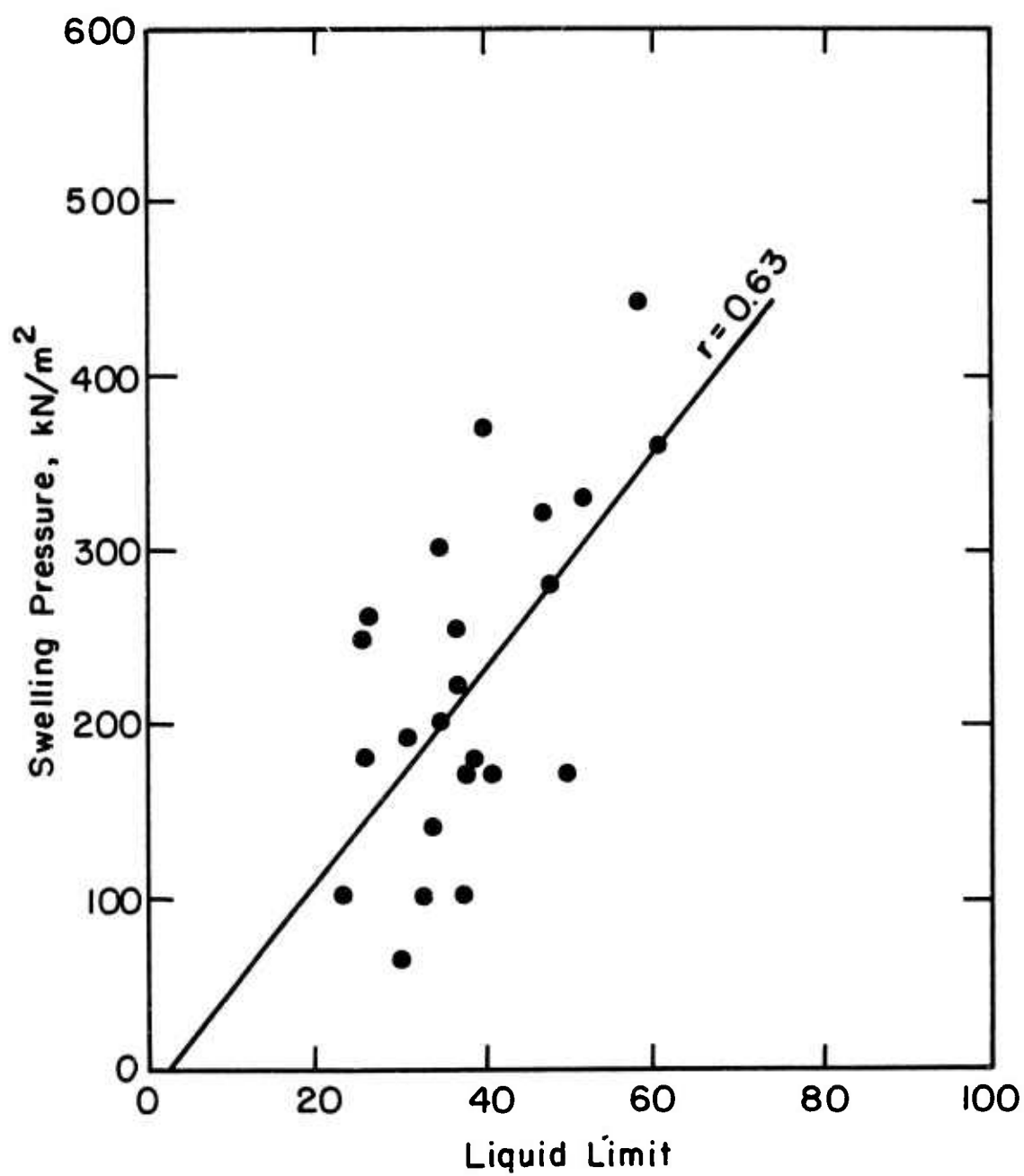


FIGURE VI-18 RELATIONSHIP BETWEEN SWELLING PRESSURE AND LIQUID LIMIT.

had shown a very good correlation between swelling pressure and the liquid limit. Another correlation which is fair is the swelling pressure vs. the amount of water imbibed in the swelling pressure sample when exposed to 100% relative humidity as shown in Figure VI-19. During the course of conducting the initial set of tests it was noted that the static compaction procedure gave widely varying values of void ratio or density. The density is one of the parameters listed in Group 2 in Table VI-5 and undoubtedly affects the swelling pressures measured. This variation was first observed when it was noted that the test results above the line in Figure VI-19 tended to have a lower void ratio and those below the line tend to have a higher void ratio. These data indicated that as void ratio decreased the swelling pressure increased. This dependency was further checked by conducting swelling pressure tests on artificial soils of varying liquid limits with constant initial void ratio. The void ratio chosen as a standard is 0.70. All specimens were compressed by static loading in the swelling pressure apparatus until a density corresponding to this void ratio was attained (small allowance for rebound was made). When the desired void ratio was reached the compressive stress was removed and after rebound had occurred a small seating load was applied. Then the water was added to the specimen and the swelling test was begun as described earlier.

In addition to the modifications being made to the swelling test procedure itself, standardizations were also being considered for the soil preparation procedure concurrently. The swelling pressure is controlled primarily by the clay type, which in turn controls the total surface area of the clay particles for a given

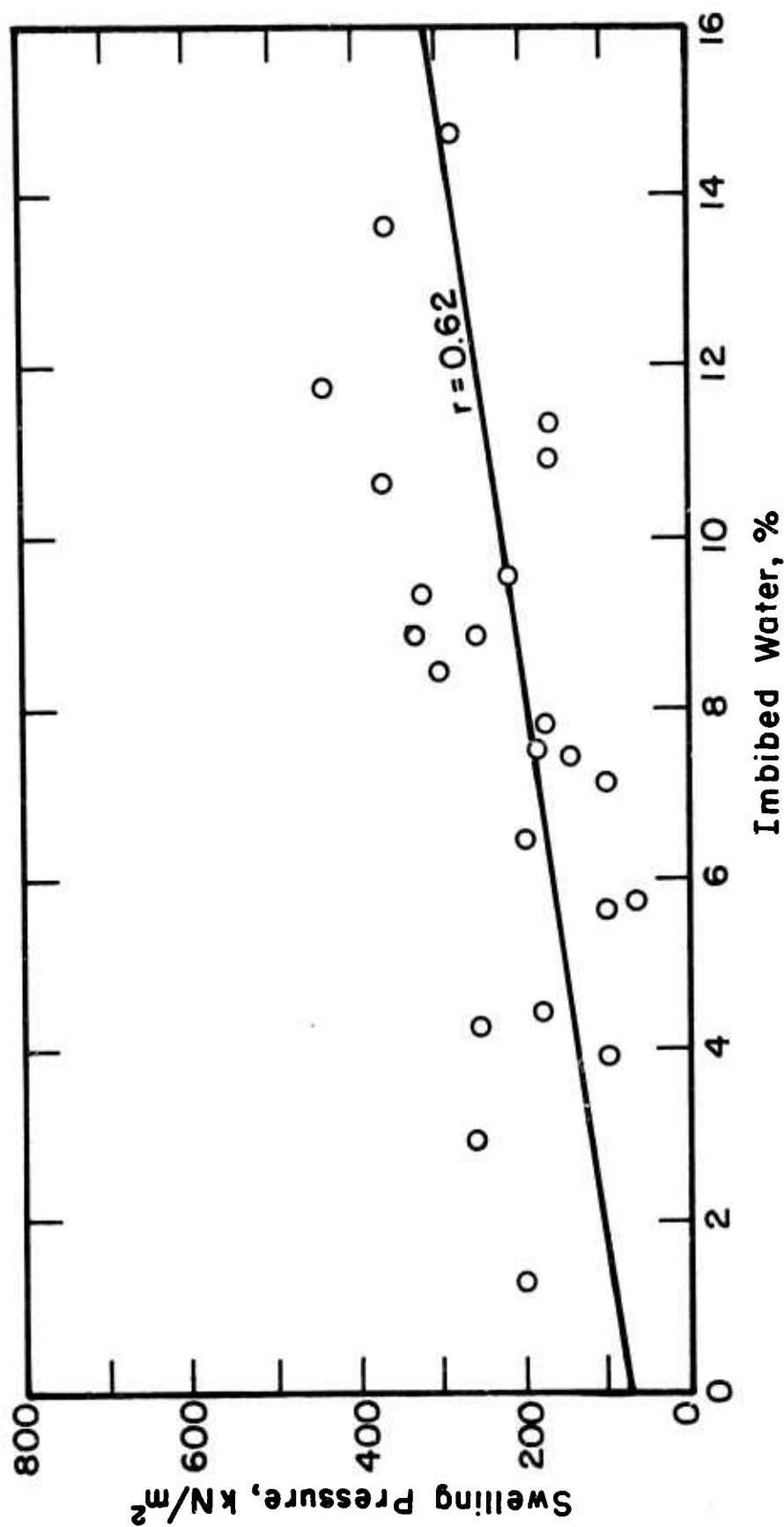


FIGURE VI-19 RELATIONSHIP BETWEEN SWELLING PRESSURE AND IMBIBED WATER - SAMPLE EXPOSED TO 100% RELATIVE HUMIDITY.

specimen size and shape. Unfortunately, when the material has been previously dried, the surface area available for water adsorption is also influenced to some extent by the duration of the grinding period which follows drying and precedes sieving. The data in Figure VI-20 shows the dependence of swelling pressure on grinding time with a mortar and rubber-tipped pestle.

In order to remove the effect of this variable insofar as possible, a standard grinding time of 7 minutes is proposed. However, for the soil shown in Figure VI-20, no significant change occurs between 5 and 7 minutes.

The modified sample preparation procedure and swelling test procedure were used for a series of tests on artificial clays and the results are shown in Figure VI-21. Similar results for an additional series of tests on some of the fault gouges used in this study are shown on the same Figure VI-21.

Although the number of test points is relatively small and a correlation coefficient was not calculated, these data show that the correlation obtained is excellent. It appears from these results that the correlation has been improved by standardizing the grinding time and precompressing all specimens to the same initial densities although it is not possible to separate the contributions of these two modifications.

The correlations obtained in Figure VI-21 are not quantitatively the same. This difference is attributed to differences in the mode of clay formation and the chemical environment for fault gouge and surficial clays. Until additional data becomes available, the upper curve in Figure VI-21 is proposed as the best

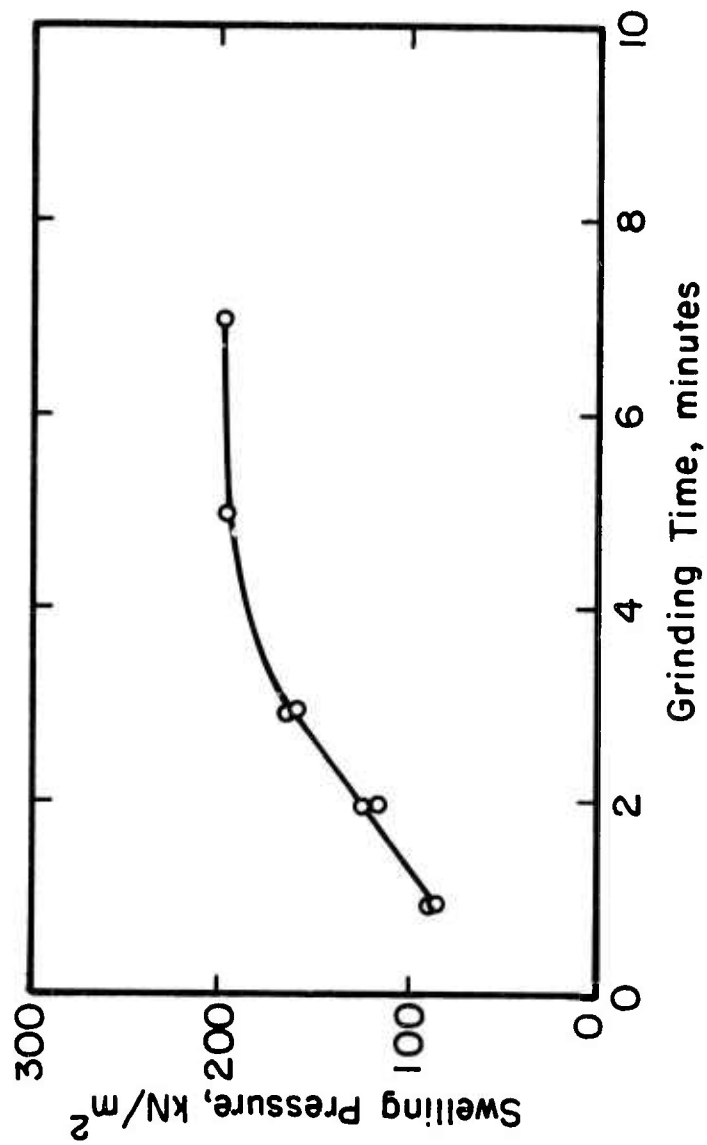


FIGURE VI-20 SWELLING PRESSURE TEST SAMPLE PREPARATION -
RELATIONSHIP BETWEEN SWELLING PRESSURE AND
GRINDING TIME.

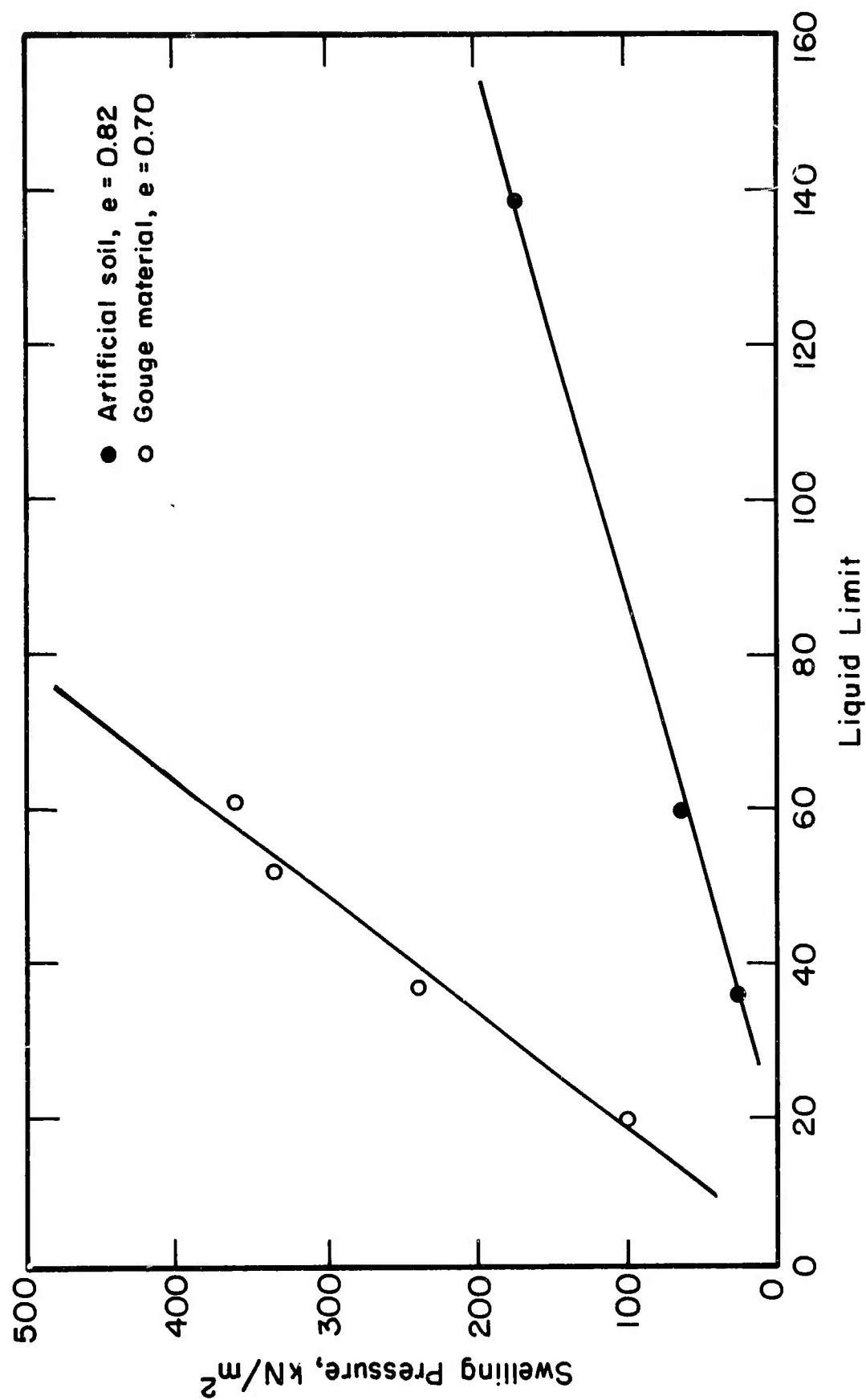


FIGURE VI-21 RELATIONSHIP BETWEEN SWELLING PRESSURE AND LIQUID LIMIT USING THE MODIFIED SAMPLE PREPARATION PROCEDURE.

estimate of the relationship between liquid limit and swelling pressure for gouge materials.

An additional correlation between swelling pressure and percent imbibed water during exposure to 100% relative humidity was pursued. Since the percent imbibed water is normally determined while preparing a specimen for swell testing, this data is readily available.

Figure VI-22 shows the correlations obtained for both artificial clays and fault gouge after modification of the test procedures and again the correlation obtained is excellent. The lower curve in Figure VI-22 is recommended for gouge material until additional data becomes available.

The results of the swell testing program indicate that at least three methods may be used to estimate the potential swellability of a tunnel material:

1. Direct measure of swell pressure by a laboratory swelling pressure test.
2. Estimation of swelling pressure from liquid limit test result - Use Figure VI-21.
3. Estimation of swelling pressure from percent imbibed water under 100% relative humidity - Use Figure VI-22.

Method 1 is of course considered to be the most reliable because it is a direct measurement. A detailed description of the procedure recommended for this test is given in Appendix A.

Either method 2 or 3, depending on which is more convenient, may be used when performance of swelling pressure tests appear unjustified. The procedure for the liquid limit determination

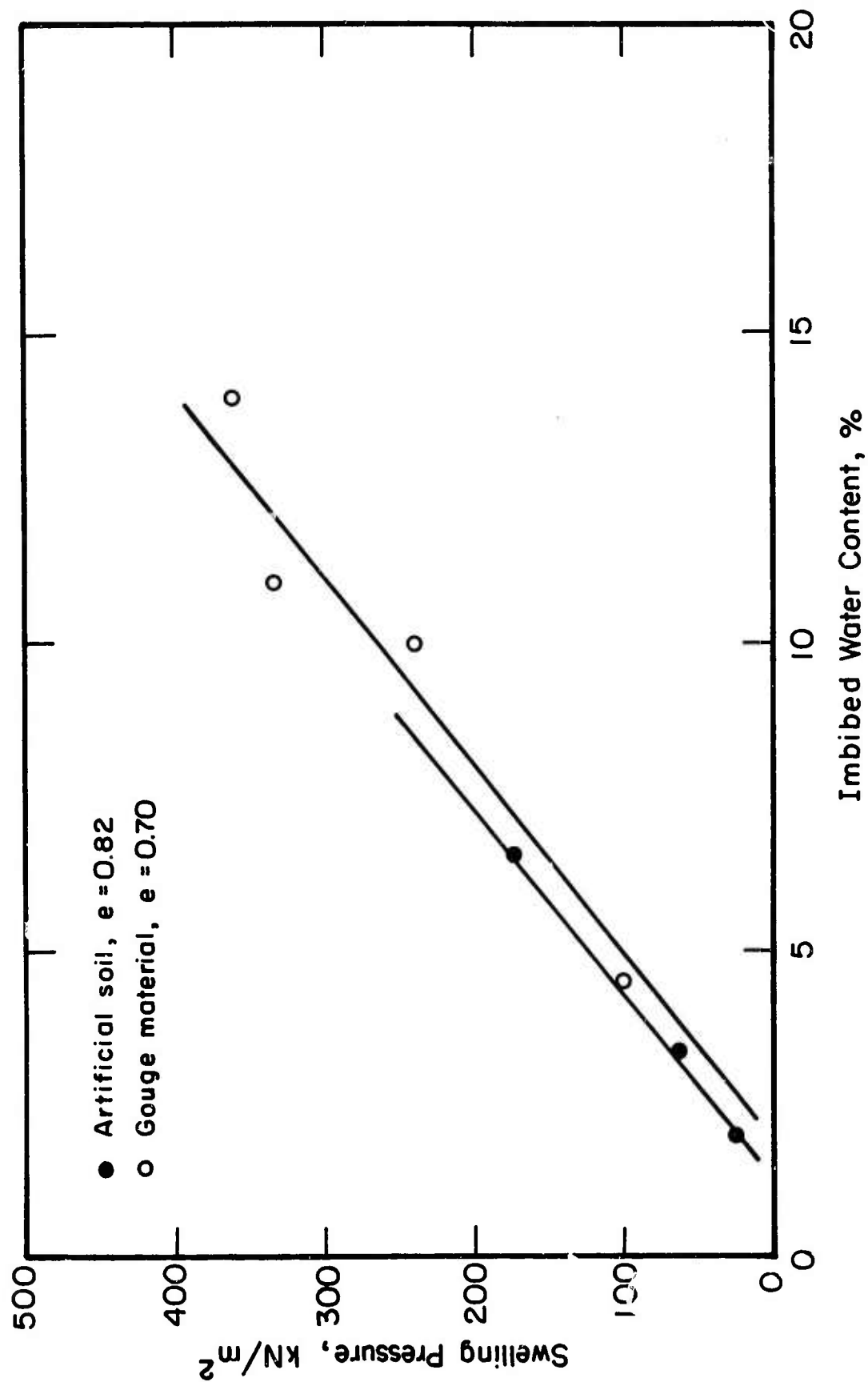


FIGURE VI-22 RELATIONSHIPS BETWEEN SWELLING PRESSURE AND IMBIBED WATER CONTENT USING THE MODIFIED SAMPLE PREPARATION PROCEDURE.

is described by ASTM D 423-66. The procedure for measuring the percent imbibed water is described in Appendix A.

When the swelling pressure has been estimated by one of the three methods above, the guidelines in Table VI-6 may be used to classify the material with respect to its capacity for causing swelling problems.

Table VI-6

SWELLING PRESSURE AND TUNNELING PROBLEMS

Estimated Swelling Pressure*	Conclusions
$< 150 \text{ kN/m}^2$	Significant swelling problems very unlikely
150 kN/m^2 to 250 kN/m^2	Inconclusive
$> 250 \text{ kN/m}^2$	Significant swelling problems quite likely if Group 2 factors in Table VI-5 are favorable for swelling

*From laboratory tests

Strength Testing

Attempts were made to obtain strength data on the various gouge materials. Direct shear testing was performed in the laboratory and penetration testing was attempted in the field; both with limited results. Triaxial tests had been performed in the Bureau of Reclamation Laboratory in Denver. Sampling for these tests took place in the pilot bore using a very sophisticated 4 inch diameter triple barrel diamond core sampler. The cores were caught in stainless steel liners in the sampler. These samples

were taken in conjunction with uniaxial jacking tests.

Direct Shear Testing

The specimens for direct shear testing under this project were obtained by coring large block samples in the laboratory, using a diamond core barrel the same diameter as the direct shear sample container. Of the 15 coring attempts made, only seven specimens were obtained which could be used for testing; five from Auburn and two from Straight Creek. The results obtained are summarized in Figure VI-23 and VI-24. The detailed results are given in Appendix C.

The Auburn samples give values for the angle of internal friction $\phi = 21^\circ$, and $\phi_{\text{residual}} = 19^\circ$. The corresponding values for the Straight Creek samples are $\phi = 7^\circ$, and $\phi_{\text{residual}} = 4^\circ$. These are very low values, and the fact that only two samples were successfully tested adds to the uncertainty in the validity of the results obtained.

Penetrometer Testing

As mentioned before, the possibility for using a penetrometer to assess the in-situ mechanical properties of gouge material was considered a possible field method, and a penetrometer device was designed. However, during field use in the Straight Creek Tunnel the penetrometer proved to be slightly smaller than that needed to obtain good results. The piston rod deflected excessively during penetration. A major problem was the variable nature of the gouge material. Large unaltered rock pieces in the gouge caused the penetrometer to deflect excessively or the penetration force to build up rapidly with very little penetration.

SHEAR TEST SUMMARY

AUBURN DAM SITE
Sample No. F-1

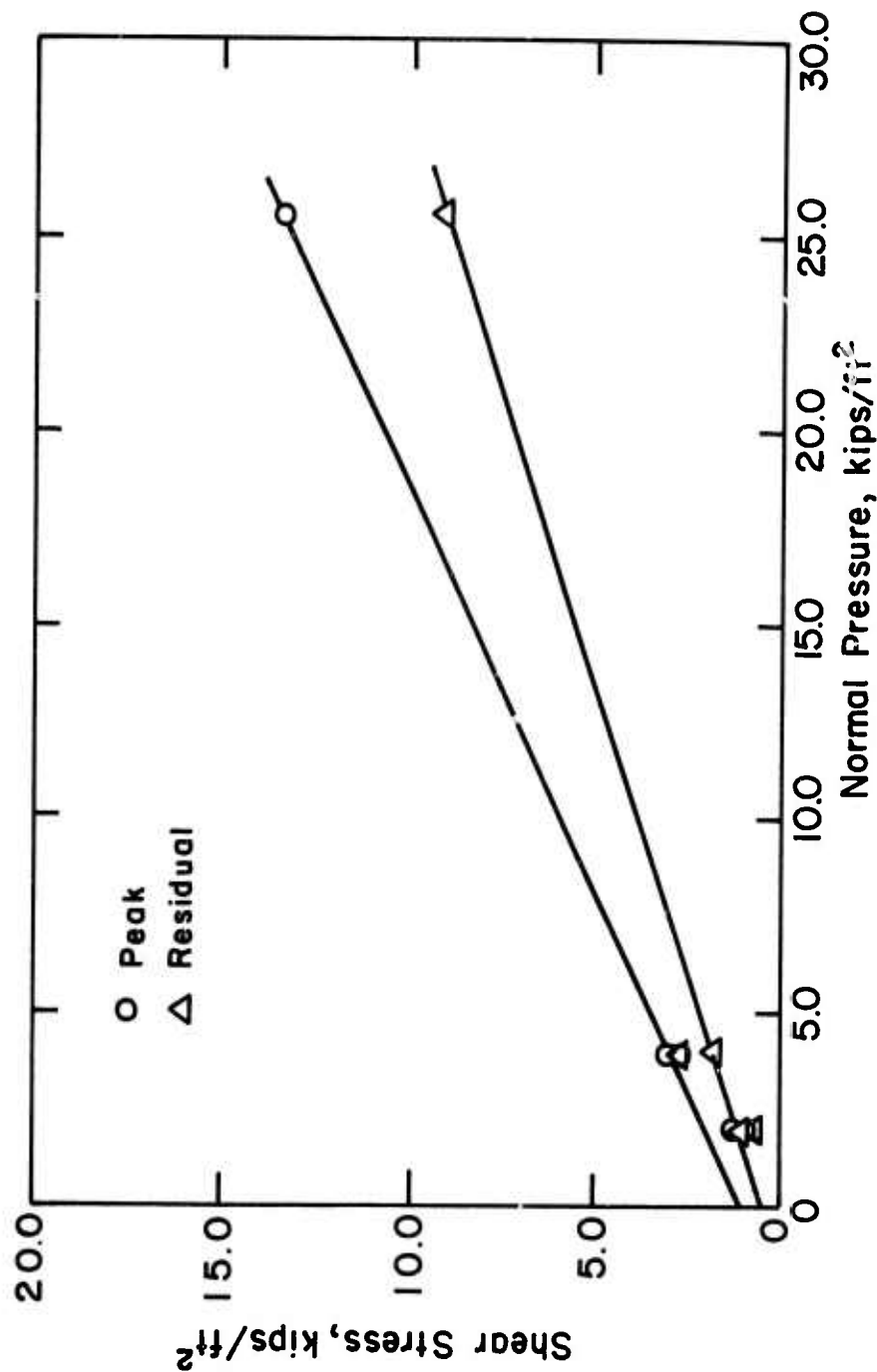


FIGURE VI-23 SUMMARY OF DIRECT SHEAR TEST RESULTS - AUBURN DAM SITE, SAMPLE NO. 1.

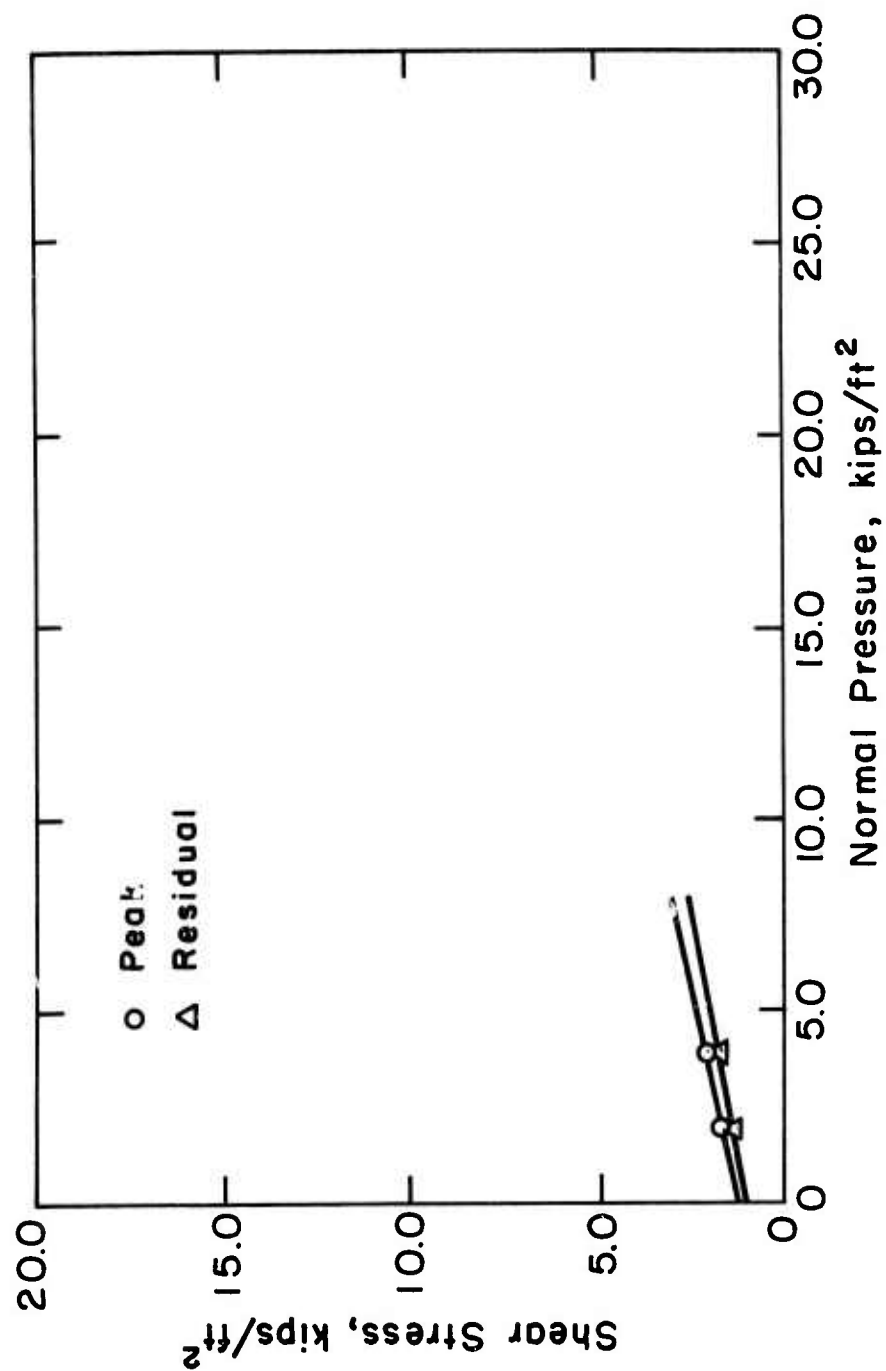


FIGURE VI-24 SUMMARY OF DIRECT SHEAR TEST RESULTS - STRAIGHT CREEK
TUNNEL - SAMPLE NO. 6.

Typical test results are shown in Figure VI-25. The remaining penetration results are included in Appendix C. It has not proved possible to deduce meaningful conclusions from the results obtained.

Test Results from Straight Creek Obtained by the Bureau of Reclamation

During August of 1970 fortytwo 3 inch and 4 inch diameter core samples were taken by the Bureau of Reclamation at Station 883 + 95 in the pilot tunnel. This is an area of a large shear zone and the samples were taken in conjunction with in-situ uniaxial jacking tests. The Bureau reported that due to the difficulties encountered while coring the decomposed granite gouge and due to the heterogeneity of the material, portions of only nineteen of the cores could be used for testing. The remaining cores were either composed of hard, competent granite or the gouge was so thoroughly disturbed by the coring operation that any test results would have been unreliable.

The tests performed on these samples were triaxial consolidated-drained, unconfined compression, and triaxial tests with zero lateral strain (K_0).

The triaxial test results were used to obtain the angle of internal friction and cohesion for the material. Values for the effective friction angle, ϕ' , ranged from 12.6° to 36° . Cohesion values ranged from 0 to 29 kN/m^2 (0.7 kips/ft^2).

The unconfined compression tests were used to determine the modulus of elasticity. These tests yielded values of $E = 11000 \text{ kN/m}^2$ for the stress range of 5.5 kN/m^2 to 8.4 kN/m^2 and $E = 16400$ for stresses in the range of 31.0 kN/m^2 to 250 kN/m^2 .

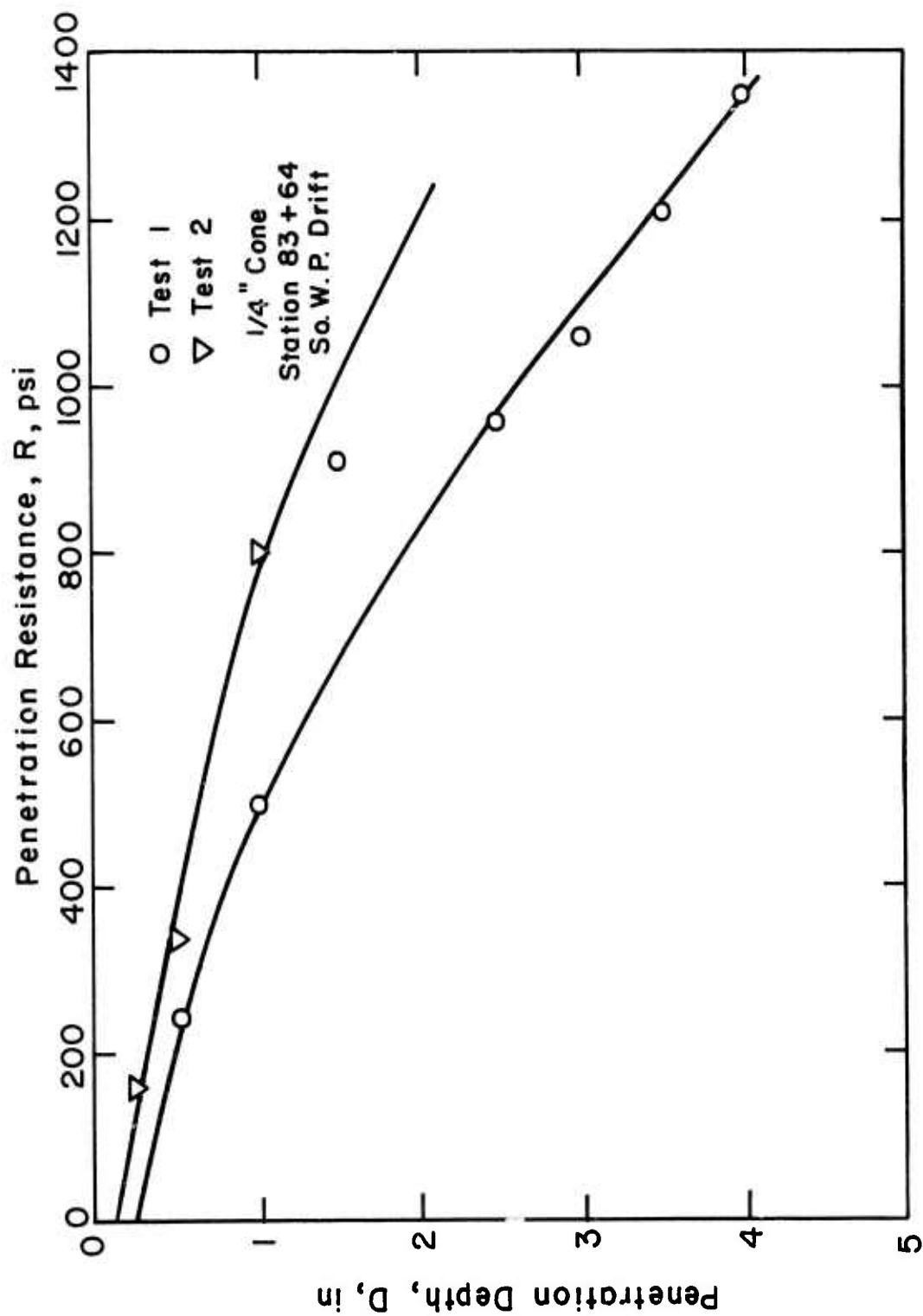


FIGURE VI-25 TYPICAL RESULTS OF IN-SITU PENETRATION TESTING - STRAIGHT CREEK TUNNEL.

The K_0 tests or triaxial tests with zero lateral strain were used to obtain values of Poisson's ratio, μ , the coefficient of earth pressure at rest, K_0 , and the constrained modulus of deformation, E . In the K_0 test the specimens are constrained from deforming in the lateral direction by increasing the lateral pressure in the triaxial chamber as the axial stress is increased.

Ten samples were subjected to mineralogical analysis using X-ray defraction, differential thermal analysis and a stereoscopic microscope. The samples were described as altered, coarse-textured metamorphic rock, probably gneiss. They were coarsely mottled in color varying from dark gray to white. The dark gray portions were highly micaceous, the light-colored portions more quartzose. Shearing was evident and clay minerals were present in varying amounts as a result of alteration of the micas, and the feldspars. The estimated mineralogical composition from three samples is shown in Table VI-7.

Four uniaxial jacking tests were performed to determine the deformability of the in-situ rock and gouge. The modulus of deformation for the rock and the gouge respectively, was determined to be 855600 kN/m^2 and approximately 4140 kN/m^2 . The corresponding elastic modulus were computed to be $52.5 \times 10^6 \text{ kN/m}^2$ and 8300 kN/m^2 .

Tunnel Deformation Measurements from Straight Creek Tunnel

Tunnel deformations during the mining operation were monitored with strain gages and tape measurements by the Colorado Division of Highways personnel. The strain gages were attached to steel sets and the readings were used to monitor movements during construction.

Table VI-7

ESTIMATED MINERALOGICAL COMPOSITION OF ROCK SAMPLES FROM
STRAIGHT CREEK TUNNEL (Bureau of Reclamation Test Data)

Mineral	Sample Number and Percent		
	52N-21	52N-22	52N-23
Quartz	30%	25%	20%
Feldspars	15	5	10
Calcite	2	2	7
Dolomite	15	2	-
Siderite	3	-	-
Illite and Micas	20	20	25
Kaolinite	3	15	10
Montmorillonite	5	20	20
Miscellaneous and unidentified	7	11	8

The tape measurements were made from one side of the tunnel to the other to measure the convergence or change in tunnel cross section. Measurements were taken about every 15 days and the corresponding tunnel advancement was noted. Figure VI-26 is a typical plot of this type of data. The top heading at this station was excavated 11-20-71 and the tape stations were installed on 11-24-71. The initial reading is shown as 0 and the total convergence thereafter is plotted. As can be seen the rate of convergence was fairly uniform until after the first stage concrete was poured. The rate of convergence slowed down after that and continued at almost no change until the bench was taken out sometime later as shown in Figure VI-27. This figure is a compilation of the tape measurement data throughout the tunnel alignment. This plot begins where the plot in Figure VI-26 ends and is also

STRAIGHT CREEK TUNNEL
Station 84+45

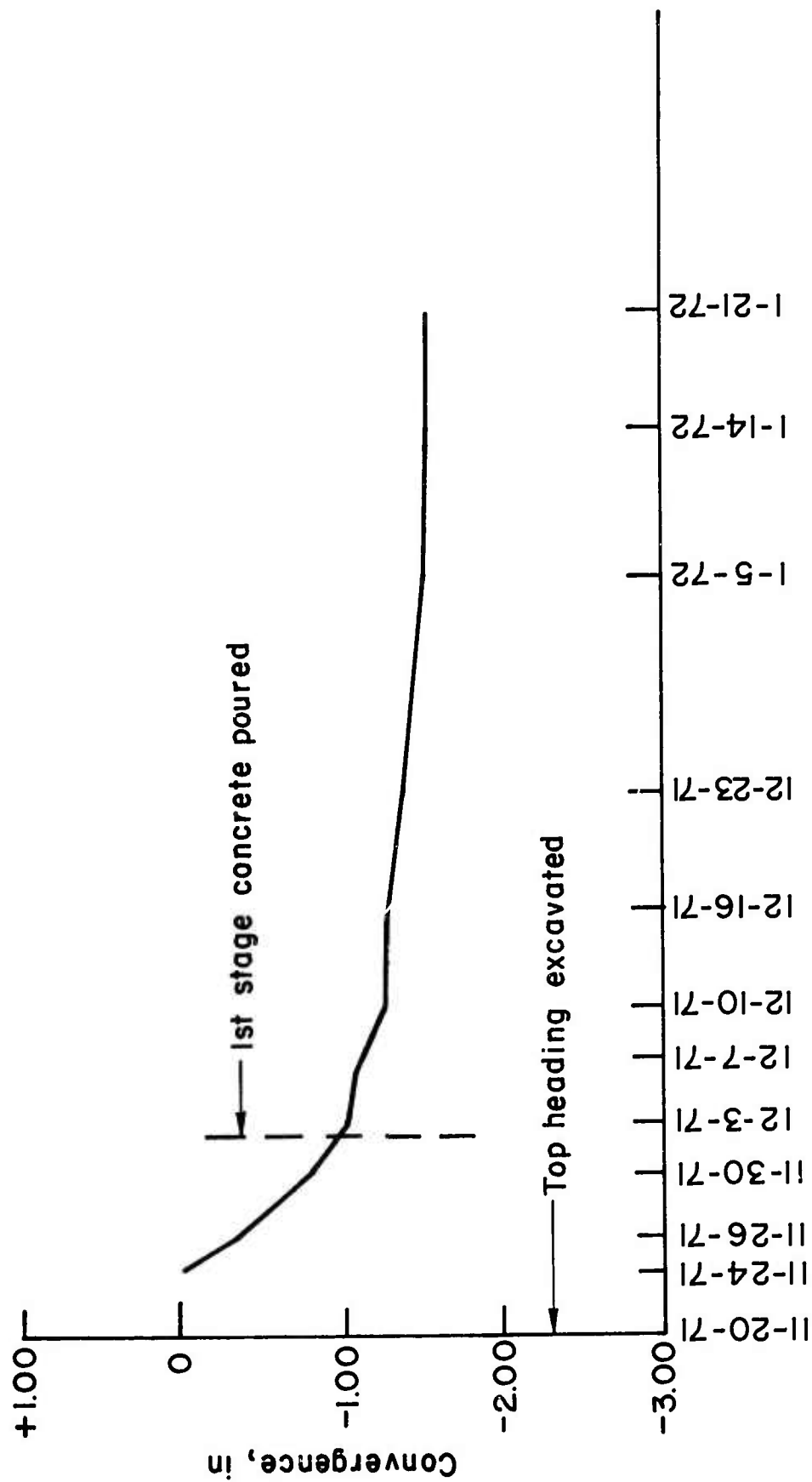


FIGURE VI-26 TYPICAL PLOT OF TAPE MEASUREMENTS - STRAIGHT CREEK TUNNEL.

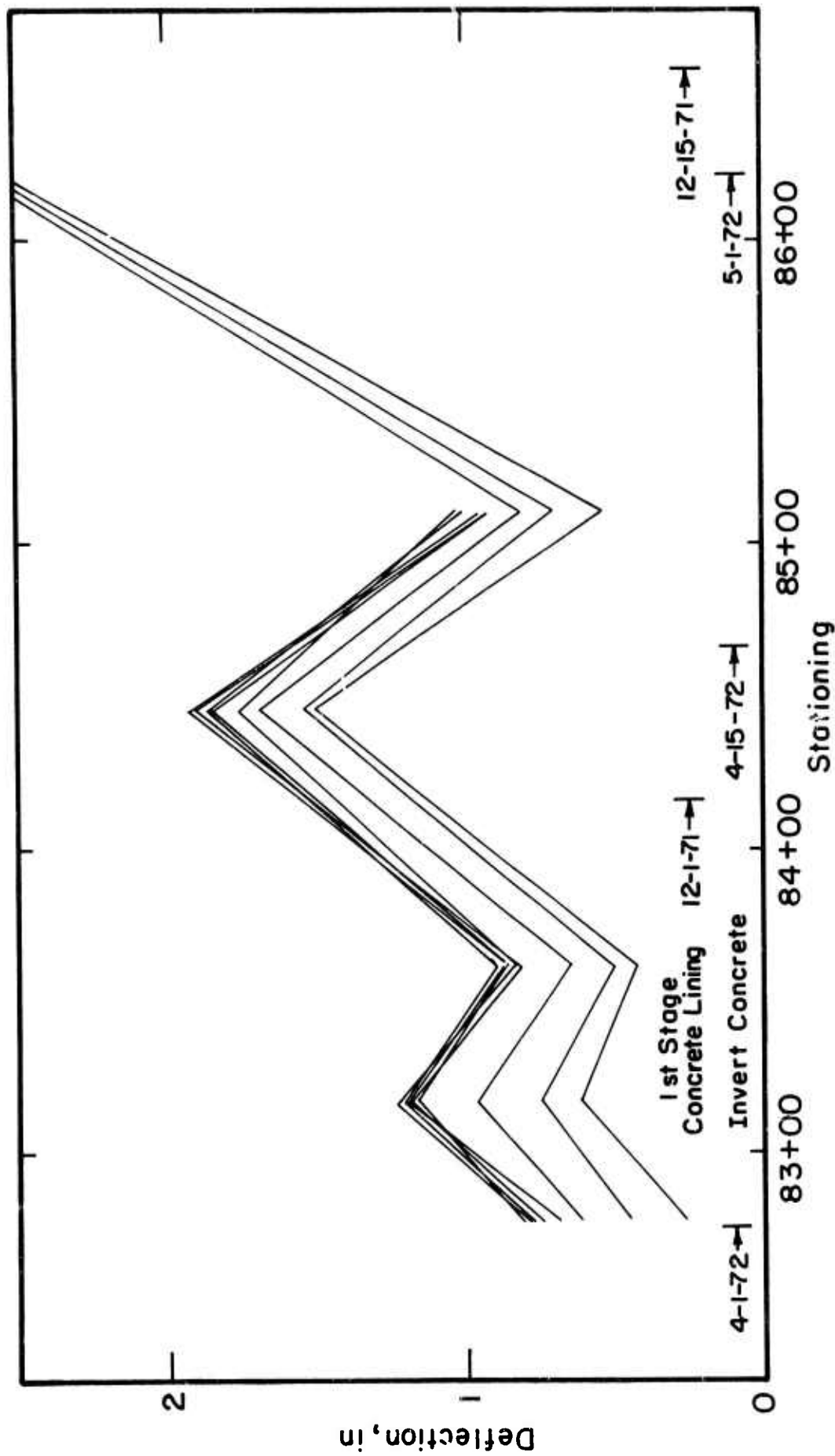


FIGURE VI-27. COMPILATION OF TAPE MEASUREMENT DATA - STRAIGHT CREEK TUNNEL.

correlated to tunnel construction activity. For example the reading at Station 84 + 45 show no movement taking place between 1-12-71 (Figure VI-26) and 3-15-72 (Figure VI-27). The movement which took place between 3-15 and 4-1-72 reflects the bench being excavated at Station 82 + 80. As the bench excavation came to and past Station 84 + 45 the convergence rate accelerated and then decelerated fairly rapidly, probably due to the invert concrete being placed closely behind the bench advance. The remaining tape measurement data is given in Appendix C.

This type of data is difficult to correlate with the properties of the gouge samples taken at specific locations. The tunnel deformations correlate well with the load increase due to the tunneling operation and indicates squeezing ground. This is further substantiated by field observations in which the gouge was noted to be squeezing past the steel sets.

Auburn Test Data

The Bureau of Reclamation provided test results for the Auburn Dam Site exploration program and provided access to the exploration drifts. Visual inspection and the results from the mineralogical study, summarized in Table VI-8 showed that the gouge in the numerous discontinuities was fairly uniform.

The sheared zones were composed chiefly to compacted gouge material with rock fragments ranging up to 80 mm in diameter. The particles shape was angular to subrounded with most particles being generally flat and elongated. Evidence of slickensides was also present.

Table VI-8

SUMMARY OF MINERALOGY OF AUBURN SAMPLES

	151-1	151-2	152-1	152-2	154-1	154-2	154-3
Dolomite	30	25	5	-10	18	12	5
Talc	20	35	25	40	45	55	50
Chlorite	20	15	50	30	18	8	40
Serpentine	30	25	20	20	5	25	Tr.
Calcite	-	-	-	-	5	-	-

Triaxial shear tests gave a range in values for the strength parameters. The angle of internal friction, ϕ , ranged from 8° to 34° with a corresponding range in the cohesion of 50 to 1150 kN/m^2 .

Chapter VII

CONCLUSIONS

It is fully realized that the case histories studied, the sites visited, and the samples tested do not necessarily provide a representation of all ground conditions that are associated with seams and faults. The variability that can be encountered even within one fault zone certainly calls for great caution in making quantitative conclusions that would have general validity.

It is, on the other hand, obviously advantageous if some qualitative guidance can be provided in terms of a systematic approach for assessing the type and magnitude of problems that a given seam or fault may cause. The conclusions that follow are made from this viewpoint.

1. Faults and seams represent a major cause of stability problems encountered in hard-rock tunneling. The literature survey as well as the studies of tunnel sites clearly demonstrate that a number of factors influence actual behavior. These factors, listed in Table II-1, are partially related to the opening itself and the excavation procedures and partially to geologic features. Thus, it is important to recognize that the characteristics and properties of the gouge material involved are only one of the factors that influence actual behavior in-situ.
2. Based on the findings from the literature survey, the field studies, and the laboratory investigations,

it is proposed that gouge material be classified as outlined in Table VII-1. This classification system is diagnostic in the sense that it identifies the type of possible problems that the gouge material may cause. It does not give a quantitative assessment of the "loads" involved or the standup time.

Table VII-1 includes the potential behavior at the face and later, and also indicates how an initial, simple identification can be made. The numerical values given for gradation and Atterberg limits must be viewed as tentative since the data available are limited. Based on the initial identification, additional testing of properties (e.g. swelling pressure, strength, deformability, solubility) can be selected. Measured or estimated values of laboratory swelling pressure may be used in conjunction with the guidelines in Table VI-6, for planning purposes.

The strength data available for the different gouge materials do not indicate any unusual property associated with gouges as compared with soils. Strength testing, as always, should be based on the merits of the situation. It should be remembered that undisturbed samples of gouge material are very difficult to obtain so that, very often, only samples of the "best" gouge will survive sampling to be tested. Test results will then be an upper bound rather than a true value of the strength of the in-situ material.

Table VII-1

CLASSIFICATION OF GOUGE MATERIAL

Dominant Material in Gouge	Potential Behavior of Gouge Material		Initial Identification of Gouge Material	
	At Face	Later	Gradation	Atterberg Limits
Swelling clay	Free swell, sloughing.	Swelling pressure and squeeze against support or lining, free swell with down-fall or wash-in if lining inadequate	>15% -No. 200 Sieve	Liquid Limit LL > 30% or Imbibed Water > 6%
	Swelling pressure and squeeze on shield.			
Inactive clay	Slaking and sloughing caused by squeeze. Heavy squeeze under extreme conditions.	Squeeze on supports or lining where unprotected, slaking and sloughing due to environmental changes.	-No. 200 Sieve	Liquid Limit LL < 30% or Imbibed Water < 6%
Chlorite, talc, graphite, serpentine	Ravelling	Heavy loads may develop due to low strength, in particular when wet.	-	Plastic Index PI < 5%
Crushed rock fragments or sand-like gouge	Ravelling or running; standup time may be extremely short.	Loosening loads on lining. Running and ravelling if unconfined.	<15% -No. 200 Sieve	-
Porous or flaky calcite, gypsum	Favorable condition	May dissolve, leading to instability of rock mass.	-	-

3. Both the literature survey and the field studies clearly demonstrate that gouge material is quite complex and will often overlap the groups listed in Table VII-1, e.g. porous or flaky calcite in a clay matrix.

It must also be recognized that the behavior of a given gouge material is a function of the other factors listed in Table II-J. Thus, a potential swelling gouge may, in a high stress field, exhibit typical squeezing behavior.

This seems to have been the case for certain areas in the Straight Creek Tunnel. As the opening size increased from the small 10-foot diameter drifts to the large 50-foot diameter tunnel the individual action of the geologic discontinuities was not as important as was the composite or overall action of the total rock mass. Although a small montmorillonite seam would swell, the swelling pressure would in this case be masked by the total squeeze. On the other hand, if swelling does occur in montmorillonitic gouges the loss in strength due to the swell can cause additional loosening loads and in some cases lead to failure, as demonstrated by the case history from the Kvineshei Railroad Tunnel in Norway.

The clay gouges in the Nast and the Hunter Tunnels were generally located in small fractures and seams. The rock was competent and the stresses low. Problems were not related so much to high loads, as to a short

standup time. This would perhaps infer a small clay content or low cohesive strength. However, the percentage of fines was high, and the material cohesive. The short standup time was therefore probably related to swelling of the active clays and a subsequent loss in strength of the gouge.

At the Henderson Mine Haulage Tunnel, the in-situ stresses were fairly high, and therefore the gouges acted as a squeezing ground. The ground continually moved into the tunnel and required excessive amounts of support. There was also a problem associated with the attitude of the faults in relation to the tunnel alignment. The tunnel was being driven into the hanging wall of the faults causing an immediate transfer of stresses back down the tunnel. Loads increased on steel sets up to 1000 feet behind the face.

These examples are intended to demonstrate the difficulty of correctly assessing the actual behavior associated with faults and seams.

4. The literature survey revealed that there are examples of slides that have taken place some time after the completion of construction because of inadequate final reinforcement or lining. Most problems, however, have been associated with the excavation process, and have often involved standup time rather than ultimate load.

It was documented earlier that standup time is extremely sensitive to active span, i.e. the size of the

opening. In ground with short standup time, it may prove necessary to use a multiple drift system, as used, for example, at the Straight Creek Tunnel. From such drifts grouted rebars can be installed to reinforce the surrounding ground prior to further excavation. Grouted rebars installed in this way or from the face forward at approximately 30° to the tunnel axis have proven to be very efficient, both in terms of improvement of the standup time, and in terms of reducing the ultimate load by a reinforcing effect.

Traditional spiling or forepoling with crown bars, in conjunction with steel sets, can significantly improve the standup time and reduce overbreak through a cantilever and beam effect. They do not, however, improve the ground itself as do grouted rebars installed as mentioned above.

Temporarily freeing a protective zone around an opening before it is excavated, supported, and lined, is a very good stabilizing method in particularly difficult zones. It has the disadvantage of being quite time-consuming and costly, but may be justified in some cases.

Grouting ahead of the face is quite efficient in running ground provided that the water inflow and pressure encountered are not excessive.

Shotcrete alone has, on several occasions, proven to be inadequate to stabilize zones with faults and

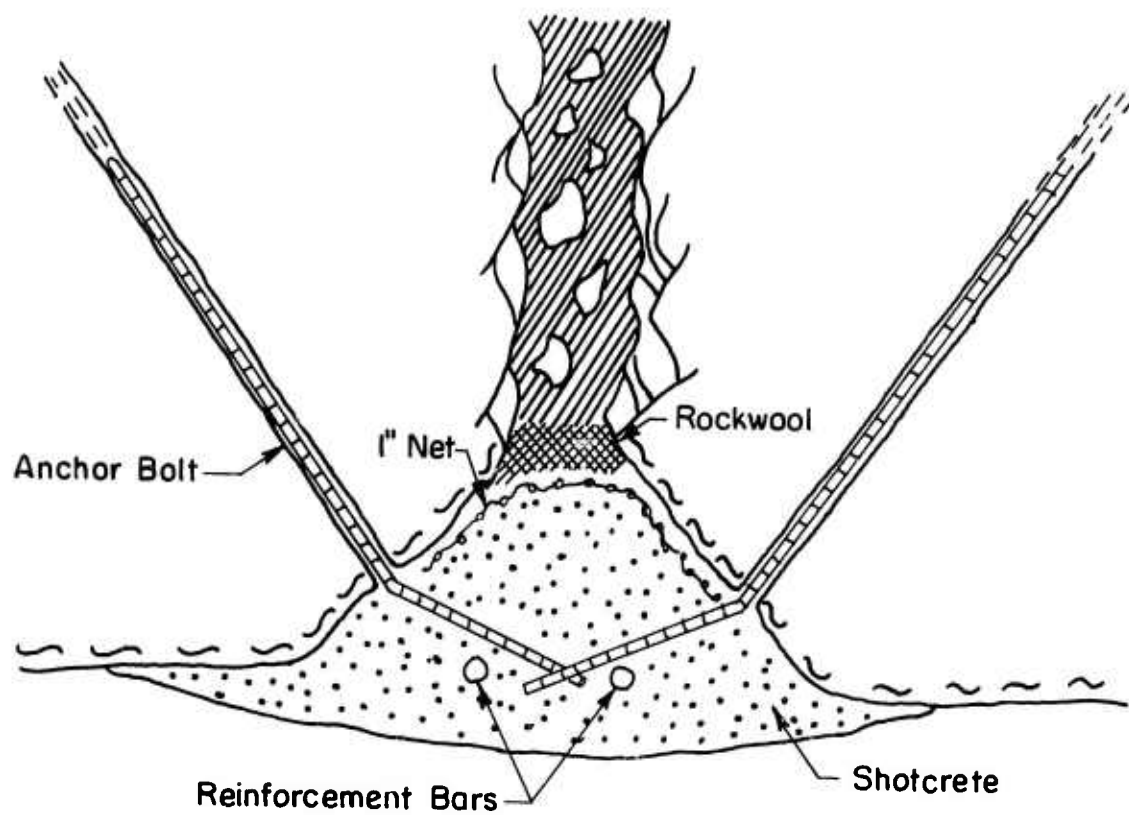


FIGURE VII-1 METHOD OF SEALING SEAMS WITH REINFORCED SHOTCRETE
(after Selmer-Olsen, 1970).

seams. Sealing individual seams as shown in Figure VII-1 by reinforced shotcrete has, on the other hand, proven to be very effective. The conditions required are that the zone be moderate in thickness, that the wall-rock be competent, and that parallel seams do not occur in the immediate vicinity. The rockwool layer (another compressible material may be substituted) in Figure VII-1 is used to allow some swell and thereby reduce the potential swelling pressure.

Shotcrete has also become increasingly used in combination with steel sets. When applied at the face, it acts as continuous lagging, and also gives, depending on the thickness, a substantial contribution to the circumferential thrust capacity of the system. This means that the weight and/or spacing of the steel sets often can be considerably reduced.

The complexity of gouge materials and the influence of a number of other factors on the actual behavior of a fault or seam when encountered underground do not allow for classifications or design-construction procedures that would be of general validity. However, the results and observations presented here can provide guidance in correctly assessing the possible problems that a fault or seam may cause and this guidance may contribute to adequate design and construction procedures.

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APPENDIX A
Testing Procedures

Each field sample was treated in the same manner upon arrival at the laboratory in order to assure compatible test results. First, the moisture content was determined and the entire sample was dried at 140°F. The sample was then broken into its individual parts for seiving by hand, and divided into segments for the various other tests. A description of each test follows.

Swelling Pressure Testing

The swelling pressure test measures the swelling pressure exerted by a specimen when exposed to water. The specimen, by necessity, is prepared in the laboratory and therefore does not represent true field conditions. Individual swelling pressure test data have no value because of this. The data obtained for a given sample are useful only when compared with other values. It is the relative swelling pressure that is of use and it is this value that should be evaluated in terms of known field conditions.

For these reasons, the following swelling pressure test procedure has been developed to provide data which can be compared and which can be correlated with the liquid limits of specimens.

Two factors have been observed to be important in the testing; the grinding time during sample preparation and the void ratio at which the test is conducted.

Grinding time means the time spent with a mortar and rubber-tipped pestle in grinding the No. 40 sieve material prior to screening to obtain minus No. 200 sieve material. Tests indicate that a minimum of 7 minutes grinding time should be used.

A void ratio of 0.70 was selected for the test because this void ratio was obtainable with the gouge samples tested and it was low enough to give high swelling pressures. This helped to accentuate swelling pressure differences for easy comparison.

Swelling Pressure Procedure

Apparatus

Geonov Swelling Apparatus

Sample Preparation

1. Separate approximately 75 grams of the material to be tested by screening over a No. 40 sieve. (Enough -No. 40 material is needed to produce 20 grams of - No. 200)
2. Pulverize the -No. 40 material by grinding 20 to 25 gram portions for 7 minutes each.
3. Pass pulverized material through No. 200 sieve.
4. Weigh out 20 grams of minus No. 200 material and place in a relative humidity of 100% for 24 hours at about 20° C.
5. After 24 hours weigh material and compute the percent of imbibed water.
6. Place material in testing apparatus by distributing evenly.

Test Procedure

1. Consolidate specimen under the static load necessary to obtain a height which corresponds to a void ratio of 0.70 after removal of consolidation load. Allow for expansion of the specimen upon removal of load. Time required may be as much as 6 to 12 hours, depending on the load applied.
2. Remove load and allow specimen to expand to an equilibrium height. The closer this height corresponds to

a void ratio of 0.70 the more consistent the swell data may be expected to be.

3. Apply small seating pressure and add water to chamber.
4. Maintain sample height constant by applying the necessary load with adjusting screw. Record load required and time lapsed.
5. Continue the test until equilibrium swell pressure is reached, which usually requires less than 6 hours.

Atterberg Limit Test Procedure

The Atterberg Limit Tests are a series of tests which determine the liquid, plastic, and shrinkage limits of a fine-grained soil. The shrinkage limits were not determined in the course of this testing program. They are performed on the fractions of a soil finer than the No. 40 sieve. Separation, if necessary, of the coarser portions may be carried out by either wet or dry sieving.

Apparatus

No. 40 sieve

Liquid limit device

Grooving tool

Spatula

Evaporating dish

Aluminum moisture content determination cans

Balance

Large glass plate for plastic limit test

Liquid Limit Determination

1. Adjust the height of fall of the cup of the liquid limit device to 1 cm with the aid of the gauge on the handle of the grooving tool. The height should be measured from the base to the point of contact of the cup. This point, it should be noted, is not the lowest point of the cup when the cup is raised.
2. Place identification number on and obtain empty weight of all moisture content dishes.
3. Obtain an air-dried sample of soil large enough to yield

at least 150 grams of minus No. 40 material. Pulverize this material with a mortar and rubber-tipped pestle to a powder and then separate the sample on a No. 40 sieve. Test only the minus No. 40 fraction. In an evaporating dish, mix approximately 150 gms. of sample with distilled water until the mixture becomes a stiff paste. Thorough mixing is required to obtain a uniform moisture content throughout the sample.

4. Place enough of the paste in the cup of the liquid limit device to reach from the front lip of the cup to a point approximately an equal distance beyond the point of impact (Figure 1A). The top surface of the soil pat should be essentially planar and horizontal when the cup rests on the rubber base. The maximum depth of paste should be about 1.1 to 1.2 cm when the surface is leveled. Save the paste not used to fill the cup.
5. Make a cut down the center of the sample using the grooving tool; as the cut is being made, hold the tool perpendicular to the surface of the cup.
6. Turn the crank at the rate of about two revolutions per second and count the number of blows required to close the groove for a distance of about 1 cm (Figure 2A).
7. Remix paste in cup and repeat steps 4 and 5 until the number of blows required to close the groove is the same (within one blow).
8. Determine the moisture content of essentially all of the paste in the cup.

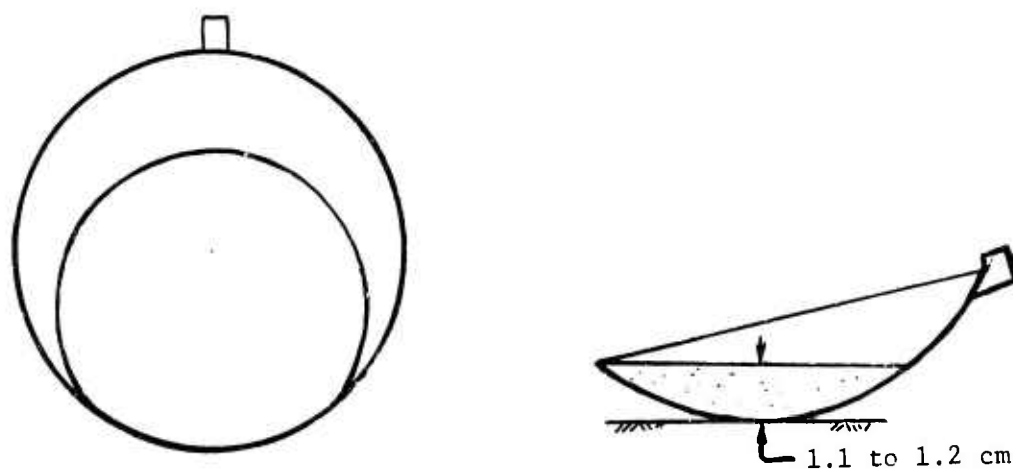
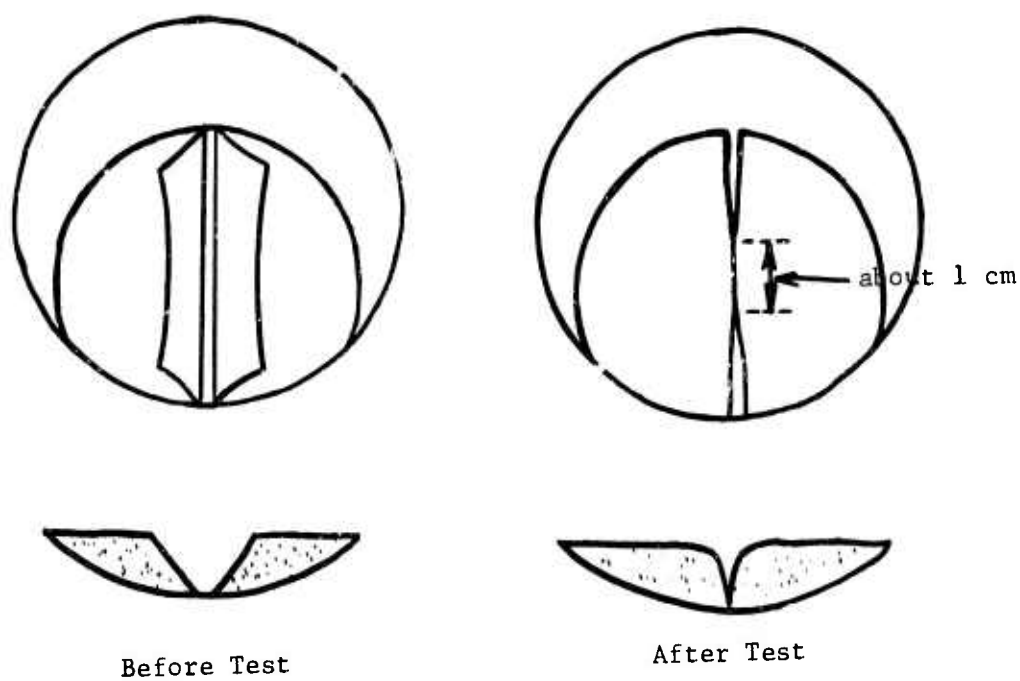


FIGURE 1A Soil in cup of liquid limit device



Before Test

After Test

FIGURE 2A Specimen before and after testing in liquid limit device

9. Return any remaining paste to evaporating dish, add a small amount of water and remix with the unused paste. Repeat steps 3 thru 7 to determine two points on the flow curve between 35 and 25 blows and two points between 25 and 15 blows. Always proceed from the drier to the wetter condition of the soil. The brass liquid limit cup should be moderately cleaned between each determination by wiping or washing.

Plastic Limit

1. In an evaporating dish, mix approximately 50 gms. of sample with distilled water until the mixture becomes pliable enough to be shaped into a ball.
2. Attempt to roll the ball, between the palm of the hand and a heavy glass plate, into a thread $1/8$ in. (3.2 mm) in diameter. If the thread just begins to crumble at this diameter and at a length of about 3 cm, determine its moisture content.
3. If the thread crumbles at a larger size, add water to the entire sample and repeat step 2.
4. Repeat steps 2 and 3 to obtain another determination which can be averaged with the first to give the plastic limit.

Calculations

1. Determine the liquid limit using a semi-logarithmic plot (1 cycle paper is recommended) of data obtained using the liquid limit device.
2. Calculate the plastic limit and plasticity index.

X-ray Diffraction Procedure

Qualitative and semi-quantitative determinations of clay mineralogy were performed by x-ray diffraction analysis. Tests were performed on oriented samples of -2μ material in: 1. the dry state, 2. in a glycol saturated state, 3. after heat treatment to 600°C for 2 hours.

Summary of Mineral Identification Criteria

Illite and Mica by approximately $10\text{\AA}$ reflection in dry state.
 Chlorite by $14\text{\AA}$ reflection in dry and heated state (the peak is sharpened by heating).
 Vermiculite by disappearance of $14\text{\AA}$ region after glycol treatment.
 Montmorillonite by reflections in the $17\text{\AA}$ region after glycol treatment.
 Kaolin by $7.2\text{\AA}$ reflection in dry state and absence of this reflection after heat treatment.
 Quartz by $4.26\text{\AA}$ reflection.
 Feldspars by reflections in $3.19\text{\AA} - 3.25\text{\AA}$ region.
 Calcite by $3.03\text{\AA}$ reflection in dry state.

Quantitative determinations were performed by assuming that equal quantities of minerals would give equal relative intensities on the x-ray diffraction diagram. It should be noted that the quantities reported are for the -2 micron fraction only and that estimations based on this procedure can be in error by up to 50 percent.

Sample Preparation

1. Initially, separate fines by sieving over the No. 200 screen.
2. Prepare a suspension of -2μ material as follows.
 - a. Mix 20 grams (dry weight) of the -200 mesh material with 200 ml distilled water and thoroughly disperse using

mixer. Add 5 ml of prepared Calgon dispersing solution to aid in dispersion of the sample.

- b. Separate -2μ fraction by centrifuging. Use a 50 ml sample of the suspension prepared under a. Make the separation twice. After the first separation decant the supernatant liquid, add more water to the precipitated soil, mix, and centrifuge it again. The supernatant suspensions obtained from the two separations contain only particles finer than 2μ . The two supernatant suspensions should be mixed together. Use the resulting suspension for preparation of x-ray diffraction specimens of -2μ clay as indicated below.
3. Using a pipette, carefully transfer 2 ml of suspension to each of at least 2 glass microscope slides.
4. Allow slides to air dry slowly.

X-Ray Diffraction Analysis

1. The following X-ray patterns were obtained for each specimen.
 - a. -2μ air-dried, oriented.
 - b. -2μ oriented and glycolated. Air-dried samples on glass slides were equilibrated overnight over ethylene glycol at 60°C .
 - c. -2μ oriented and heated for 2 hours at 600°C .

FREE SWELL TEST PROCEDURE

1. Dry -No. 40 material (passing a #40 sieve) is poured, by means of a funnel, into a 10 ml graduated cylinder in an amount sufficient to reach the 10 ml mark. Care is taken during the pouring to prevent settlement of the material because a maximum void-ratio state of the material is required.
2. The 10 ml of -No. 40 material is poured into a 50 ml graduate cylinder containing 40 ml of distilled water.
3. The cylinder is corked, shaken vigorously, and allowed to stand.
4. When the water is sufficiently clear the volume of the settled material and the new water level are recorded.
5. The percent free swell is given by

$$\frac{V_m - 10}{10} \times 10^2$$

where V_m = Volume of settled material

10 = Volume of dry maximum void-ratio material (Step 1.)

The saturated state void-ratio is given by

$$\frac{V_m - V_s}{V_s} = \frac{V_v}{V_s}$$

where V_m = Volume of settled material

V_s = Volume of solids obtained by noting increase above
40 ml of water level (Step 4.)

V_v = Volume of voids

Direct Shear Test Procedure

The direct shear test is used to determine the shear strength parameters and the shape of the stress-deformation curve for soils. However, values of strain cannot be calculated and undrained tests are very difficult to perform.

When the shear box is placed in position, operation of the driving motor causes the bottom half of the box to be moved to one side. The upper half of the box is held stationary by rigid supports. When a shearing load is applied to the sample by moving the lower part of the box, the magnitude of the force is registered on a proving ring dial gage. A dial extensometer fastened to the platform bears on an arm attached to the lower half of the shear box and registers the horizontal displacement. The vertical strain dial indicator bears against the vertical loading yoke and registers the change in thickness of the sample during the progress of the test.

Test Procedure for Dry Specimens

1. Disassemble shear box. Place porous stone in bottom of box and fix upper and lower sample rings together with screws provided.
2. Place a spacer block of known thickness in space provided for specimen. Thickness of spacer block should be approximately equal to the expected thickness of the specimen after consolidation. Reassemble shear box, place on loading platform, and obtain reference reading on vertical dial gage.
3. Without changing vertical dial gage position, disassemble

- box and prepare for specimen placement as in step 1.
4. Fill shear box with soil, reassemble, and place on loading platform.
 5. Check horizontal loading rod to see that it has been returned to starting position. Adjust if necessary.
 6. Select desired normal pressure, close toggle valve, set required conbel gage reading, and open toggle valve to apply normal pressure. Make sure air pressure relief valve is closed.
 7. Zero proving ring dial indicator. Advance circular nut on horizontal loading rod until very small load is applied to proving ring (less than 0.01 kN).
Zero horizontal extension dial indicator.
 8. Remove the three clamping screws from the specimen rings and screw them into the three threaded holes until they contact the lower specimen ring.
 9. Lift the upper specimen ring by advancing the screws one additional full turn; then withdraw the screws two full turns.
 10. Estimate shear strength and choose reading interval so as to obtain about seven readings prior to the estimated maximum shear load.
 11. Set desired deformation rate and turn on motor.
 12. Read proving ring dial gage, horizontal displacement dial gage, and vertical strain dial gage during progress of test.
 13. Reduce data, plot shear stress vs. horizontal displacement,

observe maximum shear stress, and calculate the value of the friction angle ϕ .

APPENDIX B

DESCRIPTION OF FIELD SAMPLES

Straight Creek Tunnel

Sample Number	Field Location		Description
SC 1	So. W.P.D.*	83+34	Altered mica schist with some chlorite. It is dark gray, 3/4" maximum size; individual particles are subangular and soft; the rock breaks down to rock particles, quartz and biotite.
SC 2	So. W.P.D.	83+64	Altered granite - It is light brown, sand-like material with 3/8" maximum size. The larger particles are soft and break easily. Smaller sizes are rock particles and quartz. Feldspar has been altered to clay.
SC 3	So. W.P.D.	83+82	Altered granite - It is a light gray material, 3/4" maximum size with about 15% plastic fines.
SC 4	So. W.P.D.	83+95	Highly altered granitic rock - The material is light gray and gravelly with about 50% sand and 15% plastic fines.
SC 5	No. W.P.D.**	83+25	A large block of altered granitic rock - The material is light gray and soft, made up of 50 to 60% sand, 40 to 50% plastic fines and a trace of gravel. The sand is mainly a light gray quartz and the clay fines contain a trace of biotite.
SC 6	No. W.P.D.	83+25	Large block of hard, dry clay - The sample is dark gray and very fine grained with a maximum size of 1/4" with a small percentage of fine sand.

* So. W.P.D. = South Wall Plate Drift

** No. W.P.D. = North Wall Plate Drift

Straight Creek Tunnel

<u>Sample Number</u>	<u>Field Location</u>	<u>Description</u>
		Some soft altered feldspar is present. The visible material is mainly quartz and biotite.
SC 7	No. W.P.D. 83+32	Altered granitic rock - Light gray fine grained material. Approximately 50% coarse to fine sand which is mainly subangular quartz. The remainder is dark gray plastic fines.
SC 8	So. Ftg. D. 101+00	Altered granitic rock with local inclusions of soft mica schist - The sample contains large particles (1 1/2") of altered granite that breaks down to 40% coarse to fine sand made up of rock particles, quartz and biotite. There is about 20% plastic fines.
SC 9	No. W.P.D. 83+80	Block of altered granitic rock - The sample is light brown and about 50% sand and 50% fines. It has been altered to mainly quartz particles, some rock fragments and plastic fines.
SC 10	No. W.P.D. 83+80	Highly altered granitic rock - Light brown with a maximum size of 3/4" but composed mainly of coarse to fine sand (50%) and plastic fines (50%). The recognizable minerals are quartz and biotite. The remainder is silt and clay.
SC 11	No. W.P.D. 83+27	Altered chloritic schist - The material is green, composed mainly of sand size particle with about 20% fines and a trace of gravel size.
SC 12	Zone III 206+60	Altered granitic rock - The material is light brown and fine grained. It is made up of angular rock fragments, subangular quartz particles

Straight Creek Tunnel

<u>Sample</u> <u>Number</u>	<u>Field</u> <u>Location</u>	<u>Description</u>
--------------------------------	---------------------------------	--------------------

and a trace of biotite.
There is also a trace of
altered, soft, feldspar.

Nast Tunnel

<u>Sample Number</u>	<u>Field Location</u>	<u>Description</u>
N 1	460+50	Crushed granitic rock - The sample is light gray, fine grained slightly altered granitic material. It is composed mainly of angular rock particles, and quartz. It is approximately 70% sand and the rest silt and clay.
N 2	462+60a	Crushed and altered granitic rock - The material is light brown and fine grained made up of angular rock particles (45%), quartz (40%) and biotite (5%). It contains about 20% plastic fines, and 80% sand.
N 3	464+00	Crushed and altered granitic rock - The sample is light gray with a high (40%) percentage of fines. The recognizable material is angular rock particles.
N 4	465+85	Highly altered granitic rock - The sample is gray and composed of gray subangular quartz and plastic fines with a few altered rock pieces. The rock pieces are white and very soft.
N 5	466+45	Crushed and altered granitic rock - The sample is fine grained and made up of 40% quartz particles, 20% rock fragments, 40% plastic fines and a trace of biotite.
N 6	468+46	Highly altered granitic rock - A dark gray very fine grained sample made up of quartz, soft altered granite particles and about 60% fines.
N 7	418+00	Highly altered granitic rock - The sample is composed of about 50% fines, 30% quartz, 15% soft altered rock particles and 5% biotite.
N 8	462+60b	See Sample N 2 for description. This sample was taken in the same area about 3 feet from the location of Sample N 2.

Henderson Mine
Haulage Tunnel

<u>Sample Number</u>	<u>Field Location</u>	<u>Description</u>
HM 1	32+85	Altered granitic rock - The sample is brown and fairly well graded from gravel to fines. The larger particles are altered granite; pieces of quartz surrounded by soft feldspar. This sample also contains some soft fine-grained particles that resemble shale that breaks down to thin plates. The total sample is composed of granitic rock, shale plates, quartz, feldspar and fines.
HM 2	33+30	Altered granitic rock - The sample is light brown, mainly coarse to fine sand with about 20% gravel and 20% plastic fines. The recognizable particles are made up of soft altered angular rock pieces, quartz and some soft feldspar. There is a trace of mica.
HM 3	34+85	Altered granitic rock - The sample is brown, 1/2" maximum size and composed of quartz, altered granite rock particles, feldspar and a trace of mica. The altered granite pieces are quartz surrounded by soft feldspar and mica.
HM 4	46+50	Dark gray clay - This material is a dark gray to blue clay with coarse to fine sand. The sand is composed of granitic particles, feldspar biotite and quartz.

Hunter Tunnel

<u>Sample Number</u>	<u>Field Location</u>	<u>Description</u>
H 1	273+76	Highly altered granitic rock - The sample is very fine grained and composed of about 50% fines and 50% coarse to fine sand. The sand is angular rock pieces and subangular quartz, feldspar and a trace of biotite.
H 2	273+80	This sample is the same as H 1 except wetter and neither as dense nor as altered.

APPENDIX C

Results of Testing

This appendix contains the results of both the laboratory and in-situ testing. Tables 1C through 4C are summaries of all the laboratory test results obtained by this investigation. The figures present the results the individual tests.

Table 1C

STRAIGHT CREEK TEST SUMMARY

Sample	Specific Gravity g/cm ³	Atterberg Limits		Gradation		Activity	Field Moist. %	X-ray Diffraction, -2μ				Free Swell		Swell Pressure	
		LL-PI %	PI %	%-200 Mesh	%-2μ			Mont. %	Kaol. %	Illite %	Other %	% Swell	% Swell	% Water Imbibed	KN/m ²
1	2.68			25	6		6.5	69	24	7					
2	2.63	37-19-18		25	7	2.6	8.5	56	44			30		8.9	255
3	2.65	50-23-27		23	10	2.7								11.4	172
4	2.61	31-20-11		15	5	2.2		46	50	4		23		6.5	193
5	2.66	33-17-16		39	12	1.3		85	15			35		7.2	99
6	2.70	41-19-22		75	30	0.7		61	39					11.0	169
7	2.68	34-18-16		39	10	1.6		60	40						
8	2.64	35-19-16		21	5	3.1	18.1	64	36			33		8.5	298
9	2.68	47-22-25		51	20	1.2	16.4	87	13			80		9.4	322
10	2.67	58-21-37		51	22	1.7	10.8	80	14	6		65		11.8	443
11	2.65	48-18-30		25	12	2.5								14.8	282
12	2.62	35-21-14		40	18	0.8		84	8	8		43		3.3	199

Table 2C

NAST-HUNTER TEST SUMMARY

Sample	Specific Gravity g/cm ³	Atterberg Gradation		Activity	Field Moist. %	X-ray Diffraction, -2μ			Free Swell %	Swell Pressure		
		Limits LL-PL-PI %	%-200 Mesh			Mont. %	Kaol. %	Illite %		Other %	% Water Imbibed	KN/m ²
NAST												
1	2.61	38-21-17	23	3.5	5.0	14.0	55	38	7	35	7.9	169
2	2.80	61-27-34	24	5	6.5	23.5	75	21	4		13.7	362
3	2.67	30-15-15	43	7	2.1	13.3	37	63		10	5.8	64
4	2.72	26-15-11	32	4	2.9	14.5		52	48	7	4.5	179
5	2.64	24-15-9	37	6	1.5	11.2		100		10	4.0	99
6	2.68	32-16-16	72	15	1.0	14.8	9	58	9	24	7.3	684
7	2.65	52-15-36	53	27	1.3		51	22	9	17	8.9	330
8	2.60	39-16-23	14	5	4.6		46	35	6	13	7.6	179
HUNTER												
1	2.69	26-20-6	46	7	0.4	7.3				20	4.3	252
2	2.67	27-17-10	42	4	2.2	19.5				10	3.0	258

Table 3C

HENDERSON MINE HAULAGE TUNNEL TEST RESULTS

Sample	Specific Gravity g/cm ³	Atterberg Gradation			Activity	Field Moist. %	X-ray Diffraction, -2 μ				Free Swell		Swell Pressure KN/m ²
		Limits LL-PL-PI %	%-200 Mesh	%-2 μ			Mont. %	Kaol. %	Illite %	Other %	Swell %	% Water Imbibed	
1	2.64	40-21-19	31	10	1.9		65	26	9		17	10.7	370
2	2.61	34-22-12	20	6	2.0		70	22	8		11	7.5	139
3	2.66	37-21-16	14	3	5.3		46	54			23	9.6	217
4	2.72	38-14-14	54	22	1.1		29	38	33		20	5.7	99

Table 4C

AUBURN DAM SITE TEST SUMMARY

Sample	Specific Gravity g/cm ³	Atterberg Gradation		Activity	Field Moist. %	X-ray Diffraction, -2 μ				Free Swell		Swell Pressure	
		Limits LL-PL-PI %	%-200 Mesh			Mont. %	Kaol. %	Illite %	Other %	Swell %	Water Imbibed KN/m ²		
F-1	2.80	18-18-0	26	4	0	7							240
						Talc 70 Chl 30							

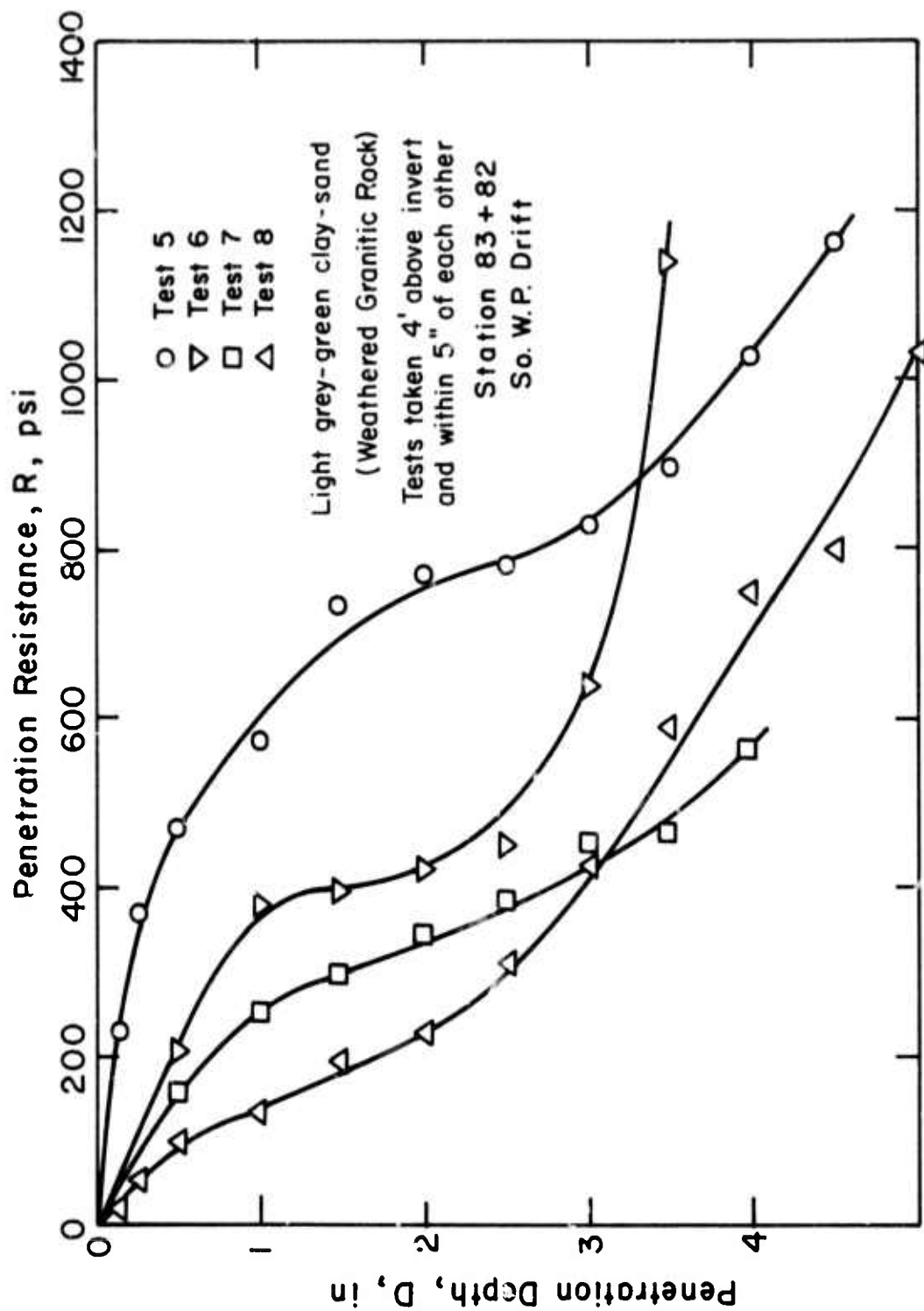


FIGURE 1C IN-SITU PENETRATION TEST RESULTS - STRAIGHT CREEK TUNNEL

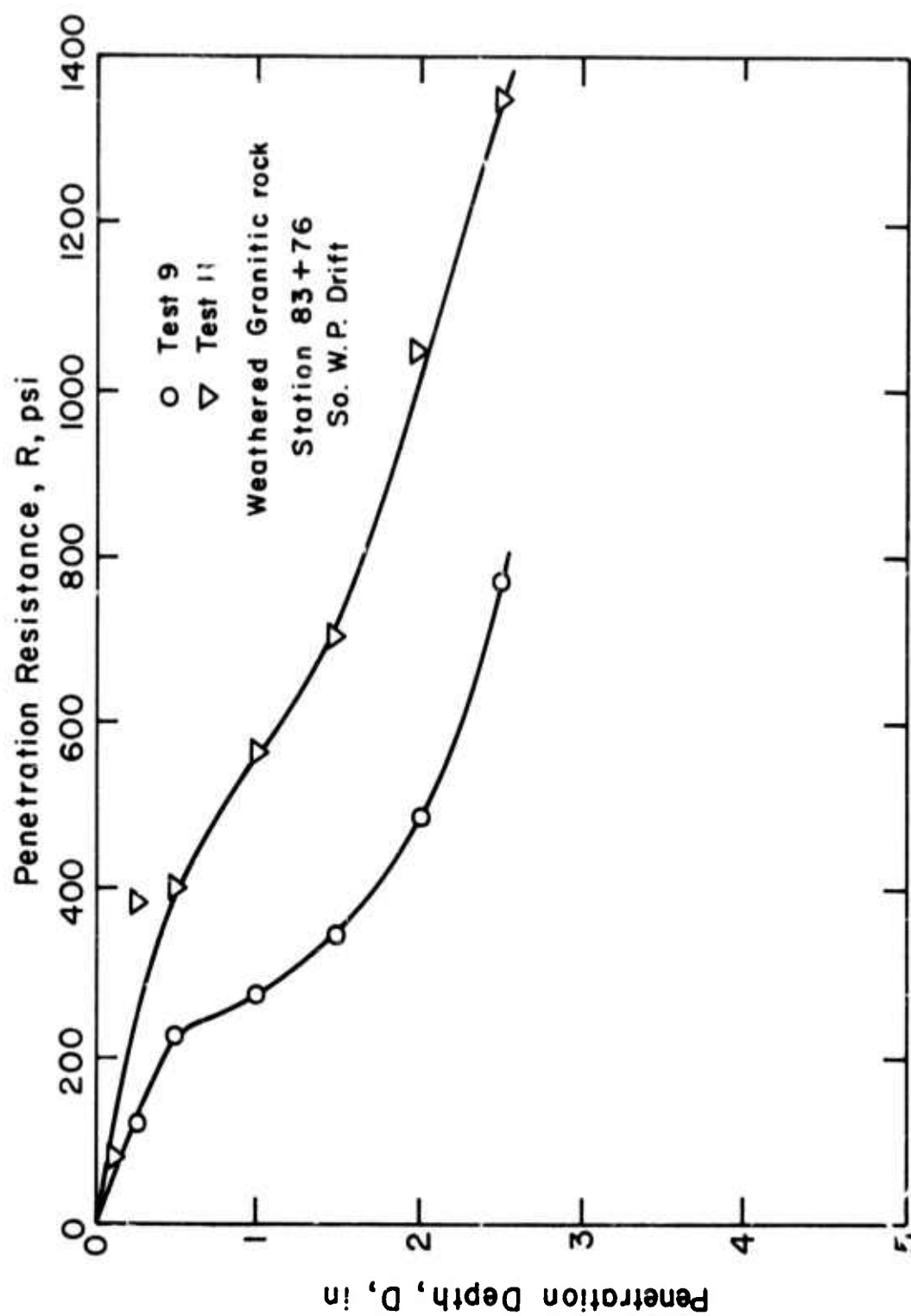


FIGURE 2C IN-SITU PENETRATION TEST RESULTS - STRAIGHT CREEK TUNNEL

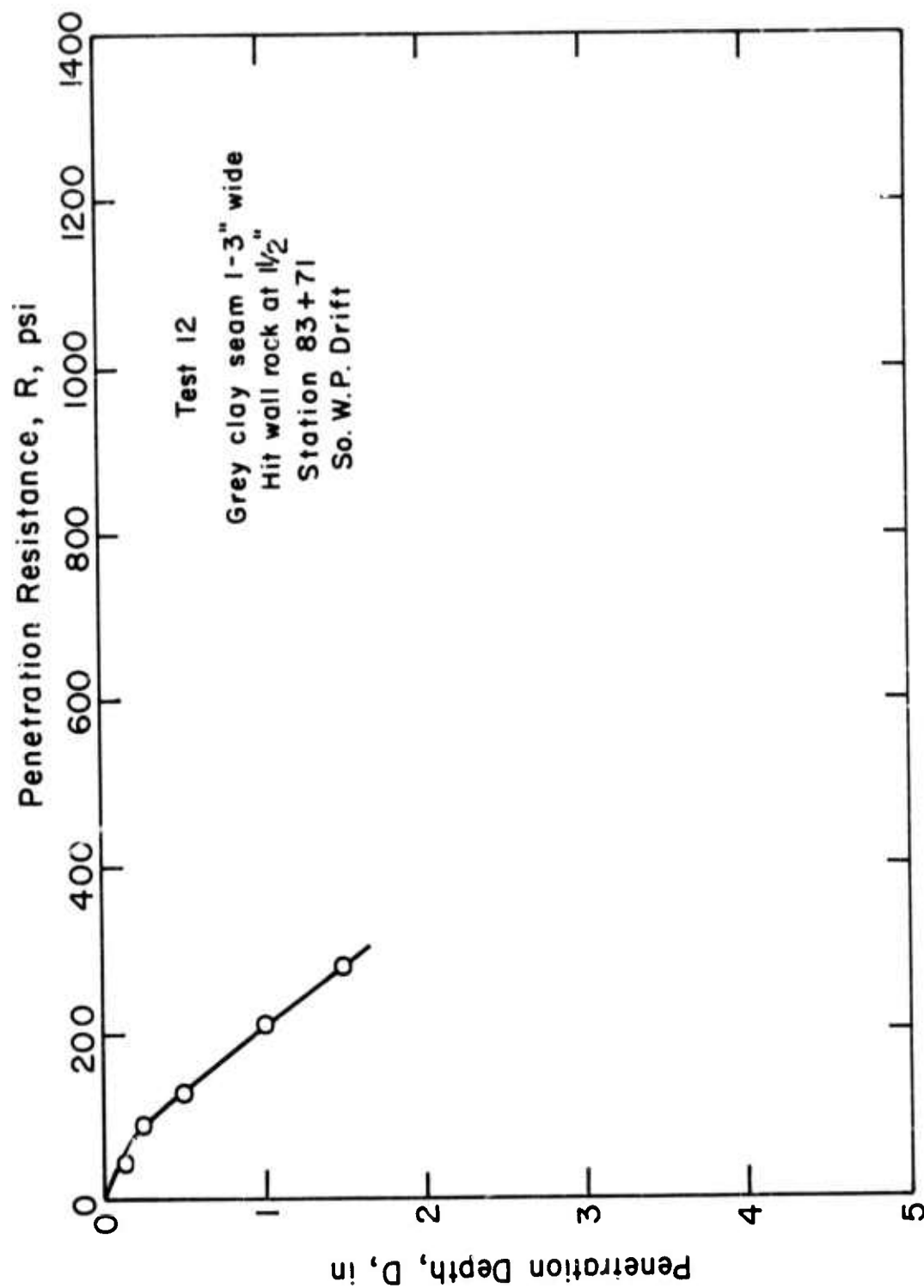


FIGURE 3C IN-SITU PENETRATION TEST RESULTS - STRAIGHT CREEK TUNNEL

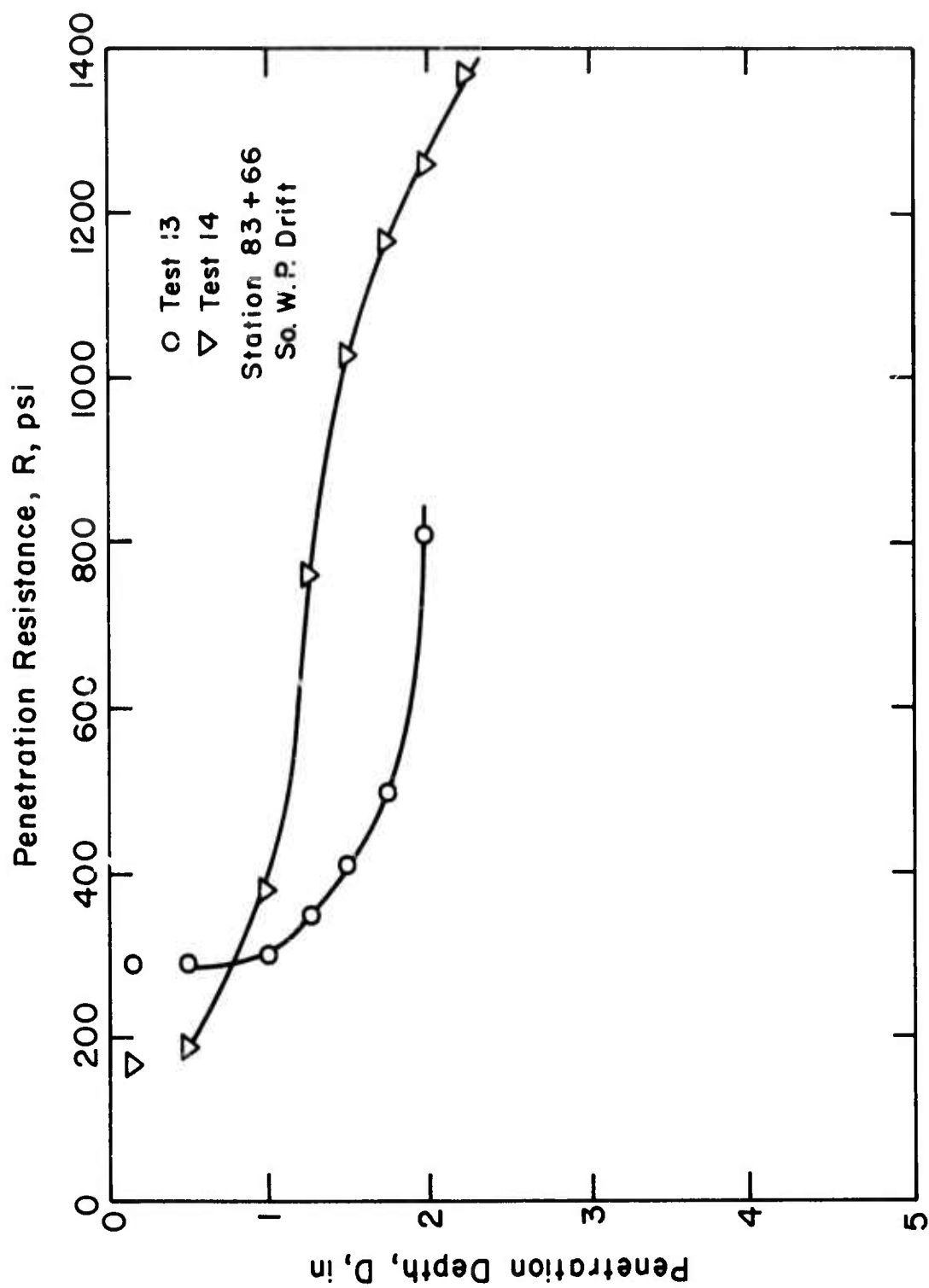


FIGURE 4C IN-SITU PENETRATION TEST RESULTS - STRAIGHT CREEK TUNNEL

DIRECT SHEAR TEST RESULTS

STRAIGHT CREEK TUNNEL
Sample No. 6

Normal Stress - 2 ksf
Moisture Content - 11.7%
Dry Density - 122 pcf
Void Ratio - .38

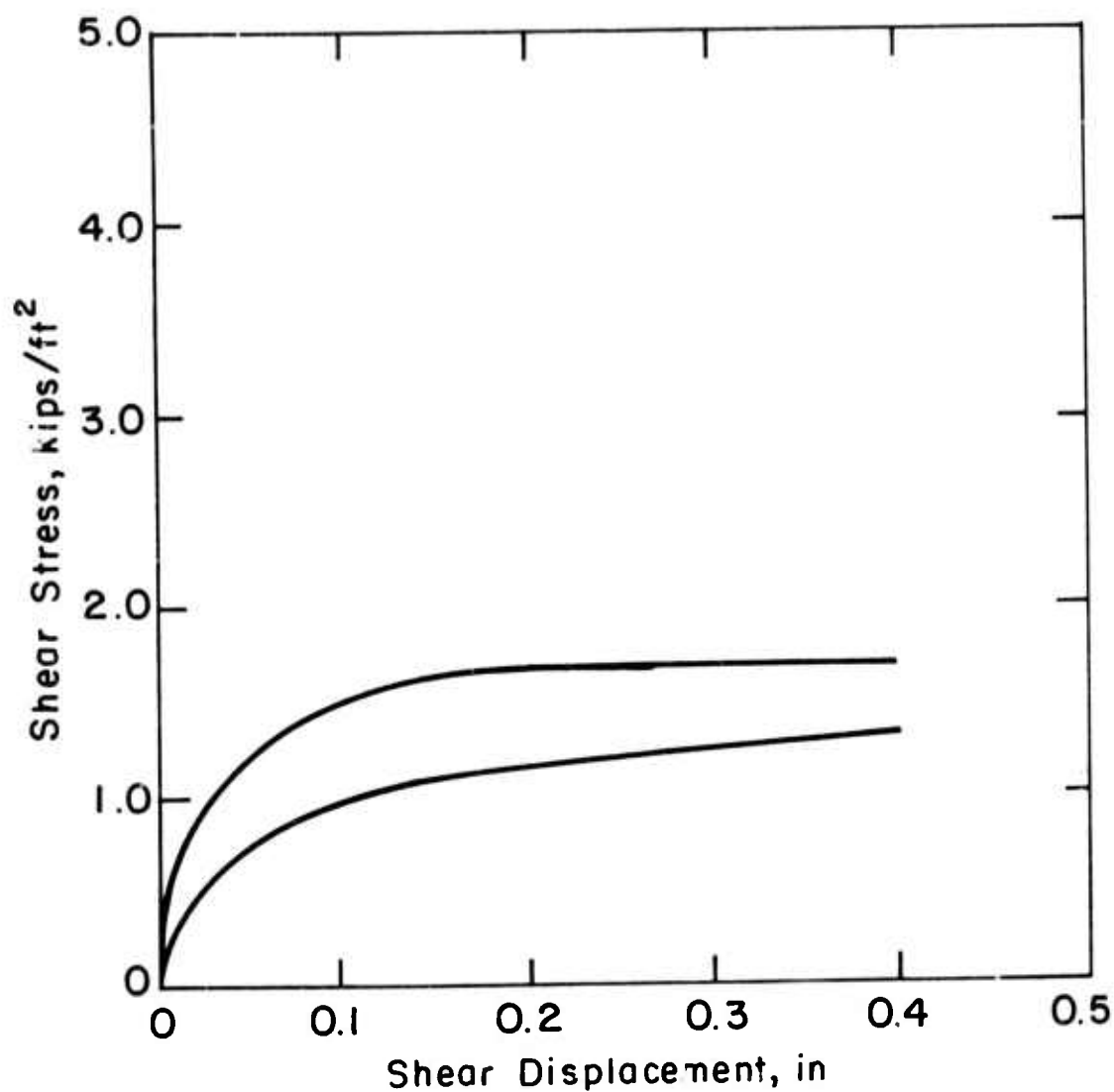


FIGURE 5C DIRECT SHEAR TEST RESULTS - STRAIGHT CREEK TUNNEL

DIRECT SHEAR TEST RESULTS

STRAIGHT CREEK TUNNEL
Sample No. 6

Normal Stress - 4 ksf
Moisture Content - 11.4%
Dry Density - 122 pcf
Void Ratio - .35

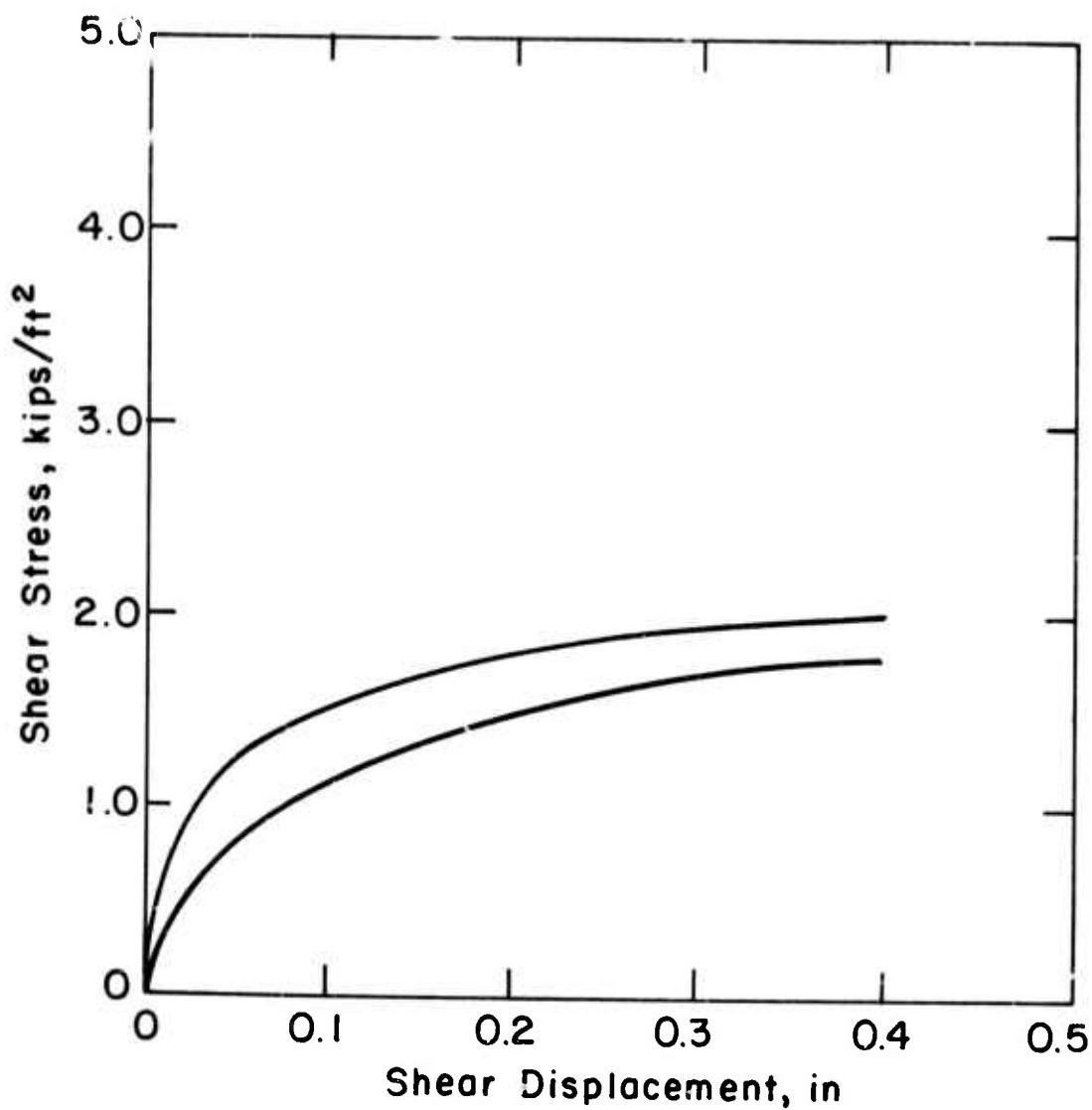


FIGURE 6C DIRECT SHEAR TEST RESULTS - STRAIGHT CREEK TUNNEL

DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #1

Normal Stress - 2 ksf
Moisture Content - 8%
Dry Density - 157 pcf
Void Ratio - .11

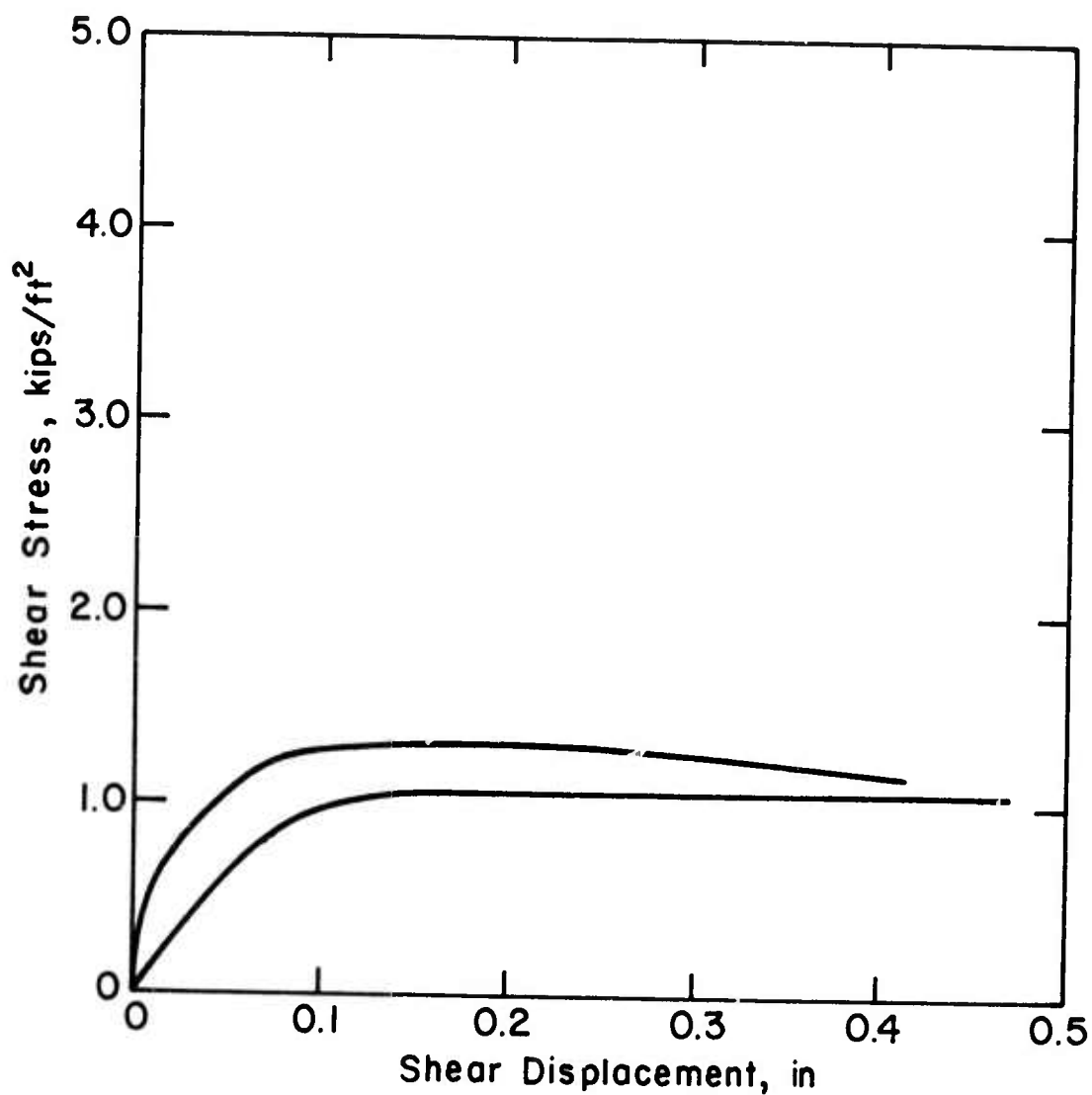


FIGURE 7C DIRECT SHEAR TEST RESULTS - AUBURN DAM SITE

DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #2

Normal Stress - 2 ksf
Moisture Content - 8%
Dry Density - 157 pcf
Void Ratio - .11

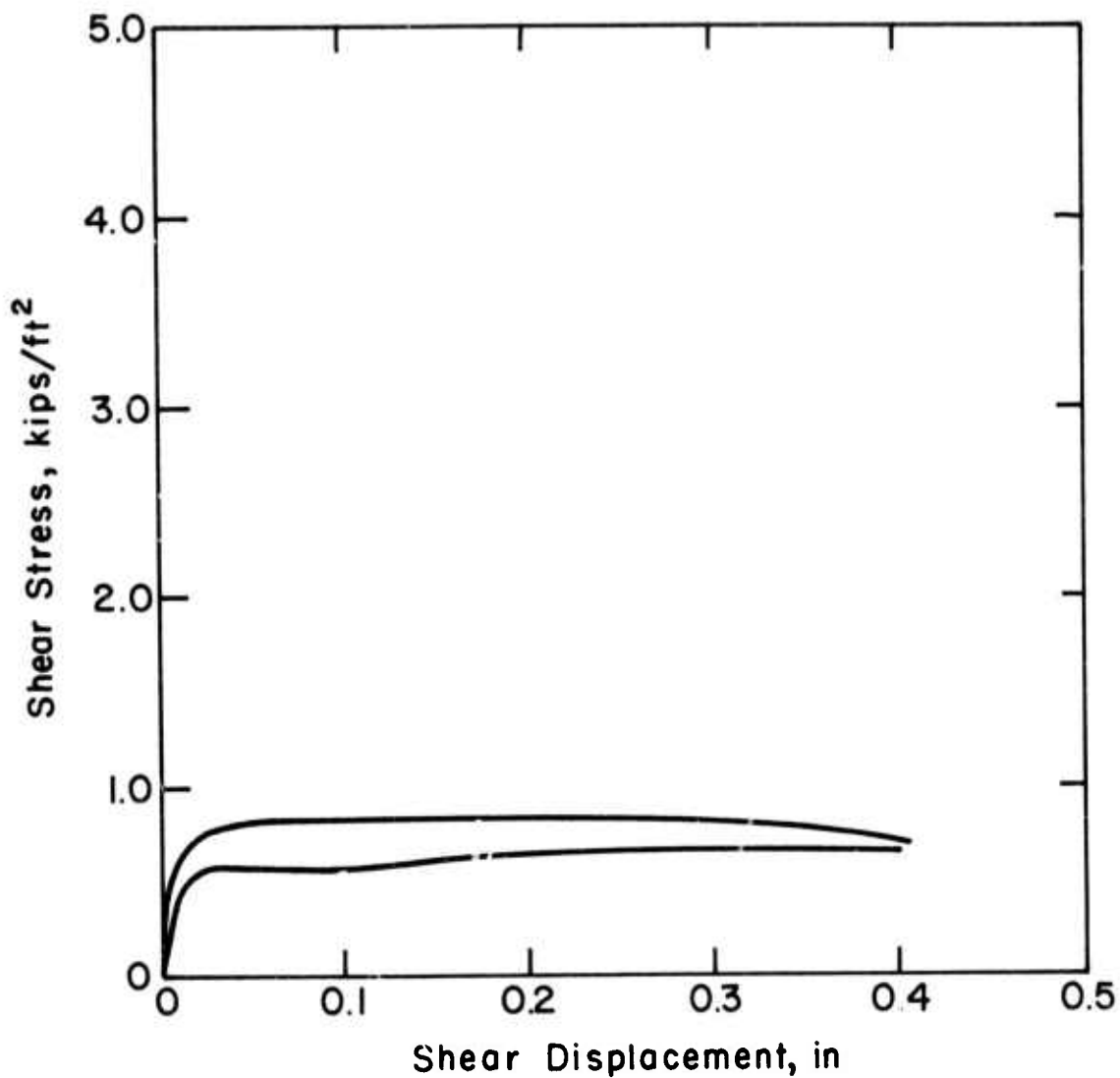


FIGURE 8C DIRECT SHEAR TEST RESULTS - AUBURN DAM SITE

DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #3

Normal Stress - 4 ksf
Moisture Content - 6%
Dry Density - 138 pcf
Void Ratio - .29

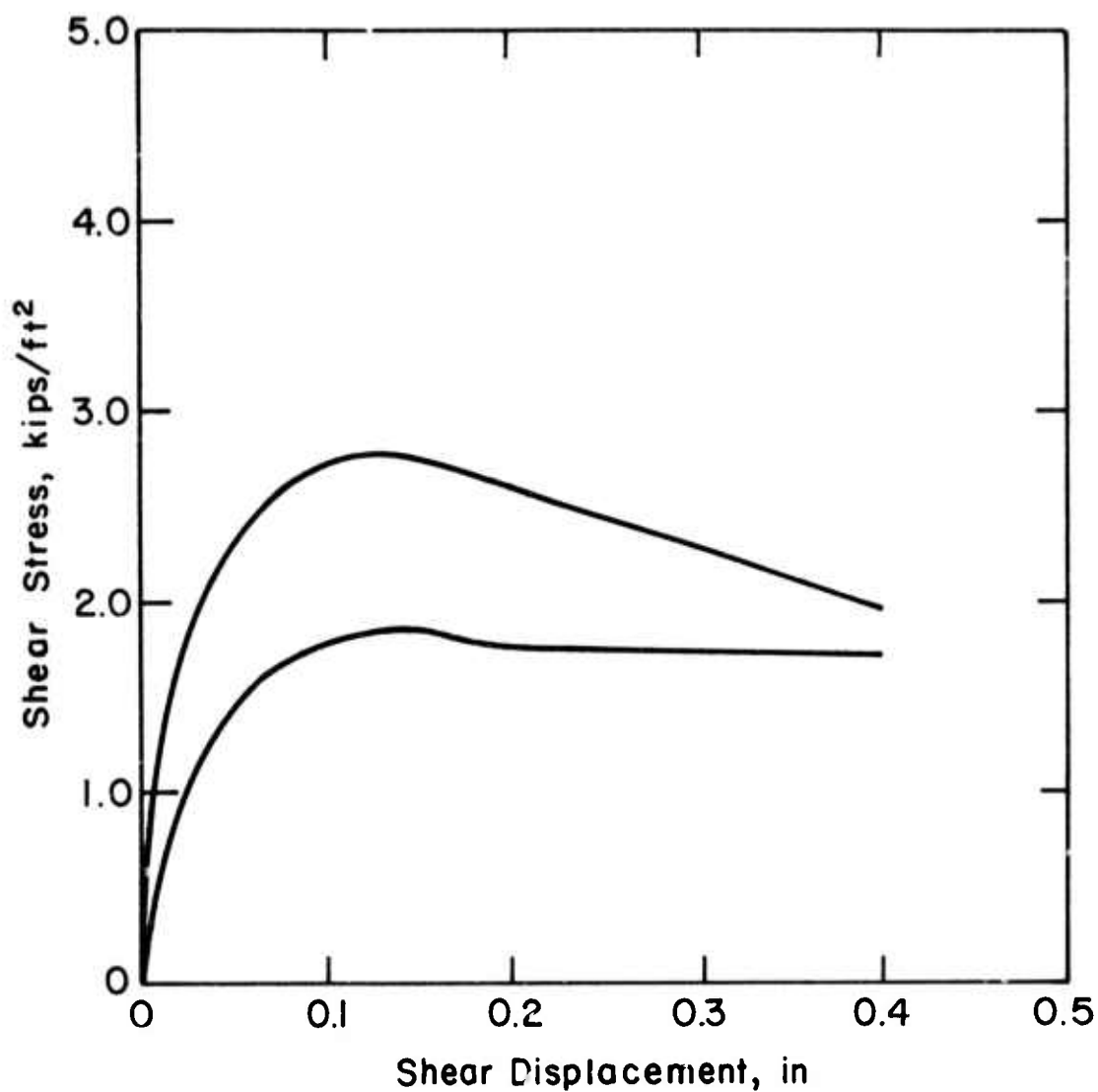


FIGURE 9C DIRECT SHEAR TEST RESULTS - AUBURN DAM SITE

DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #4

Normal Stress - 4 ksf
Moisture Content - 8%
Dry Density - 135 pcf
Void Ratio - .29

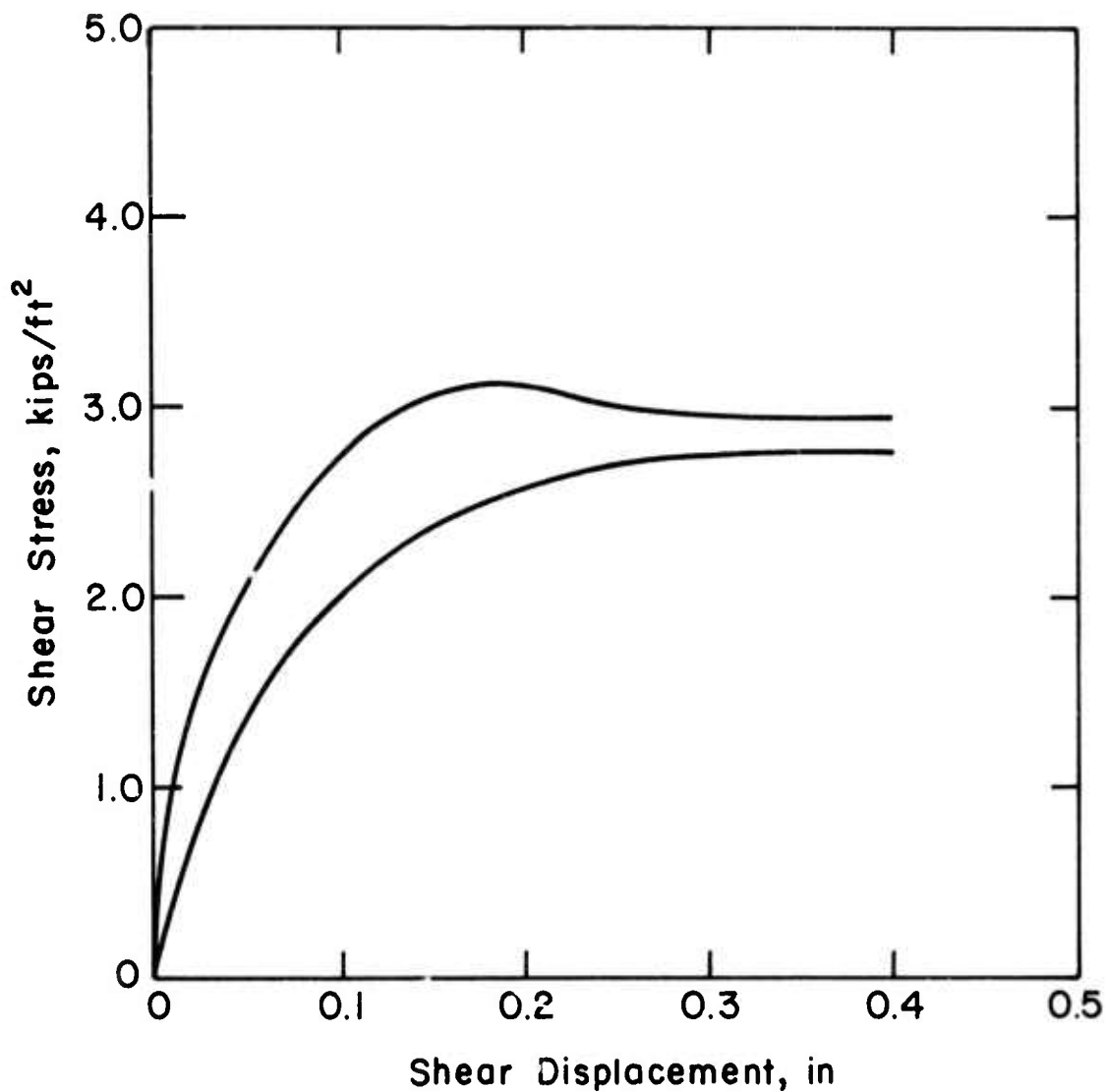


FIGURE 10C DIRECT SHEAR TEST RESULTS - AUBURN DAM SITE

DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #4

Normal Stress - 25 ksf
Moisture Content - 6.5%
Dry Density - 157 pcf
Void Ratio - .12

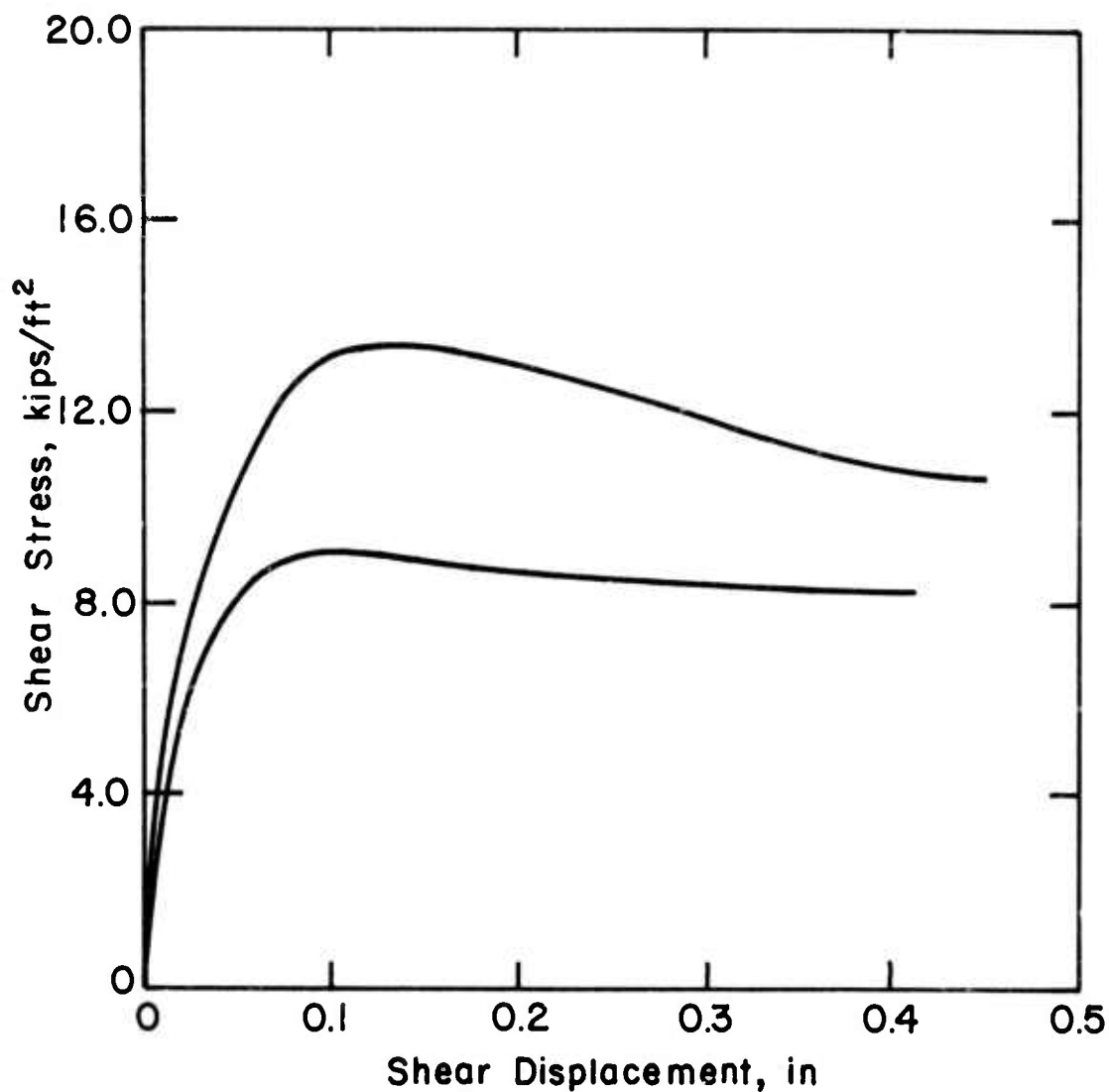


FIGURE 11C DIRECT SHEAR TEST RESULTS - AUBURN DAM SITE

STRAIGHT CREEK TUNNEL
Station 85+08.9

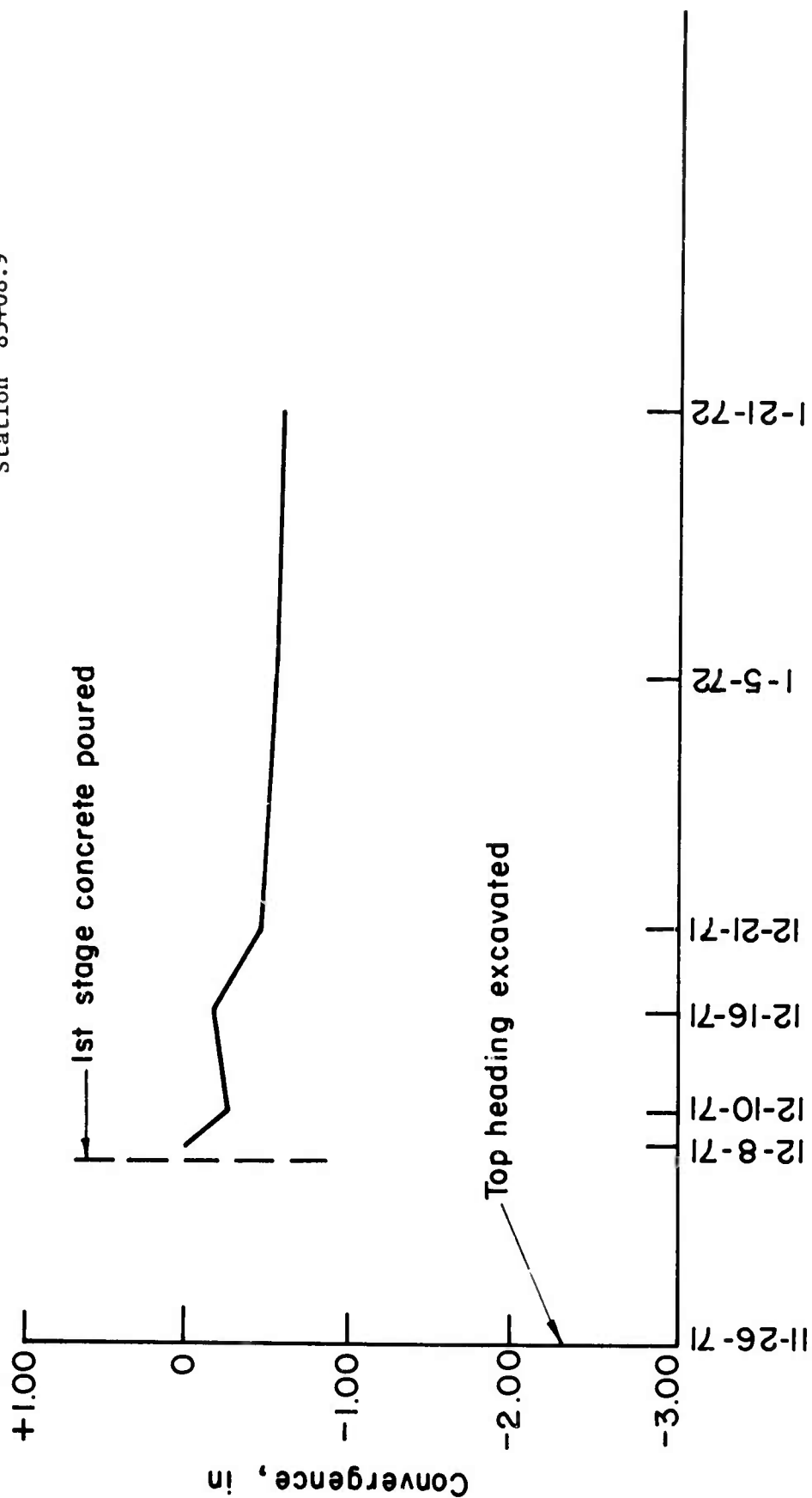


FIGURE 12C IN-SITU TAPE MEASUREMENT RESULTS - STRAIGHT CREEK TUNNEL

STRAIGHT CREEK TUNNEL
Station 85+57

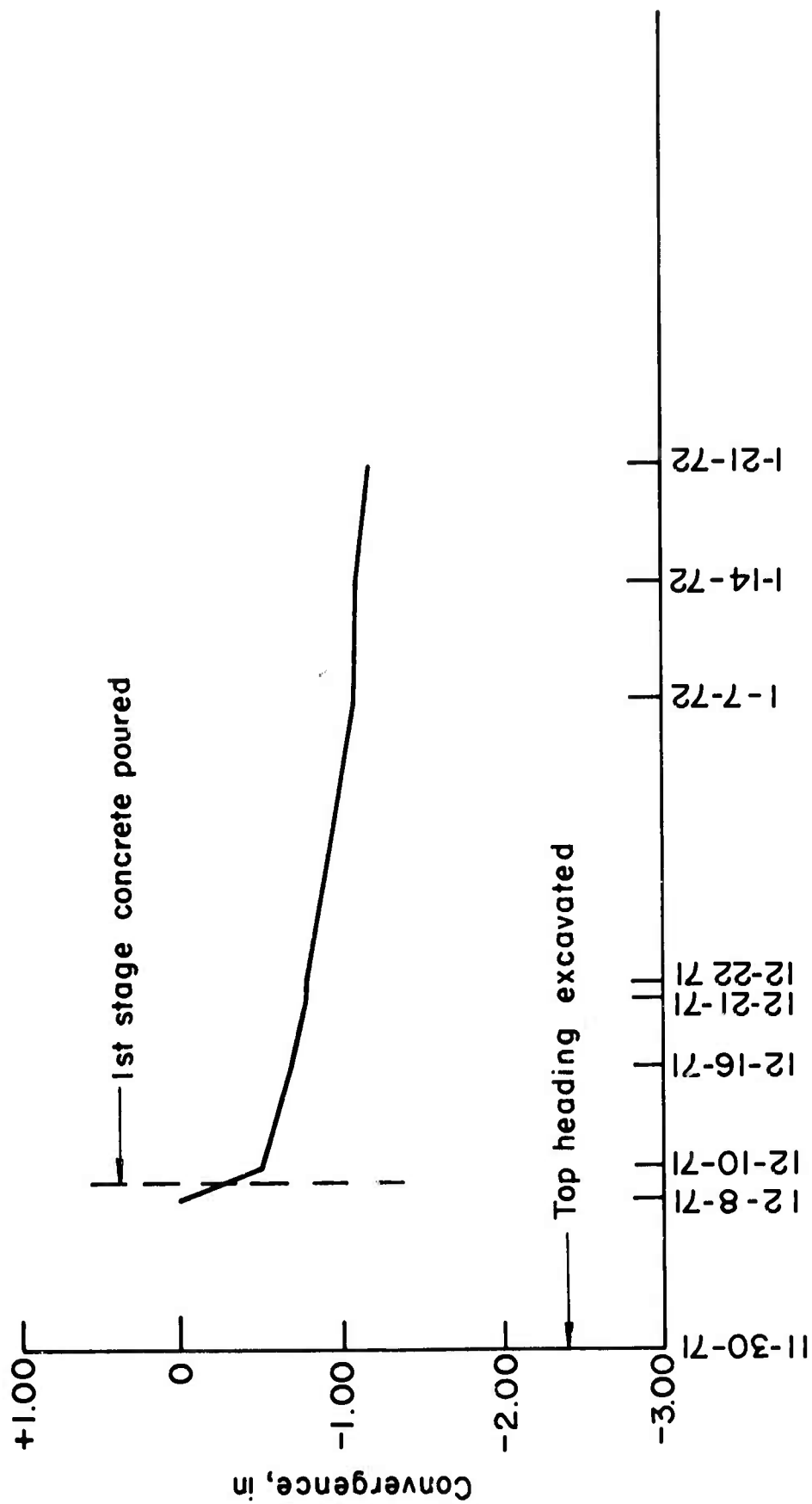


FIGURE 13C IN-SITU TAPE MEASUREMENT RESULTS - STRAIGHT CREEK TUNNEL

STRAIGHT CREEK TUNNEL
Station 86+29

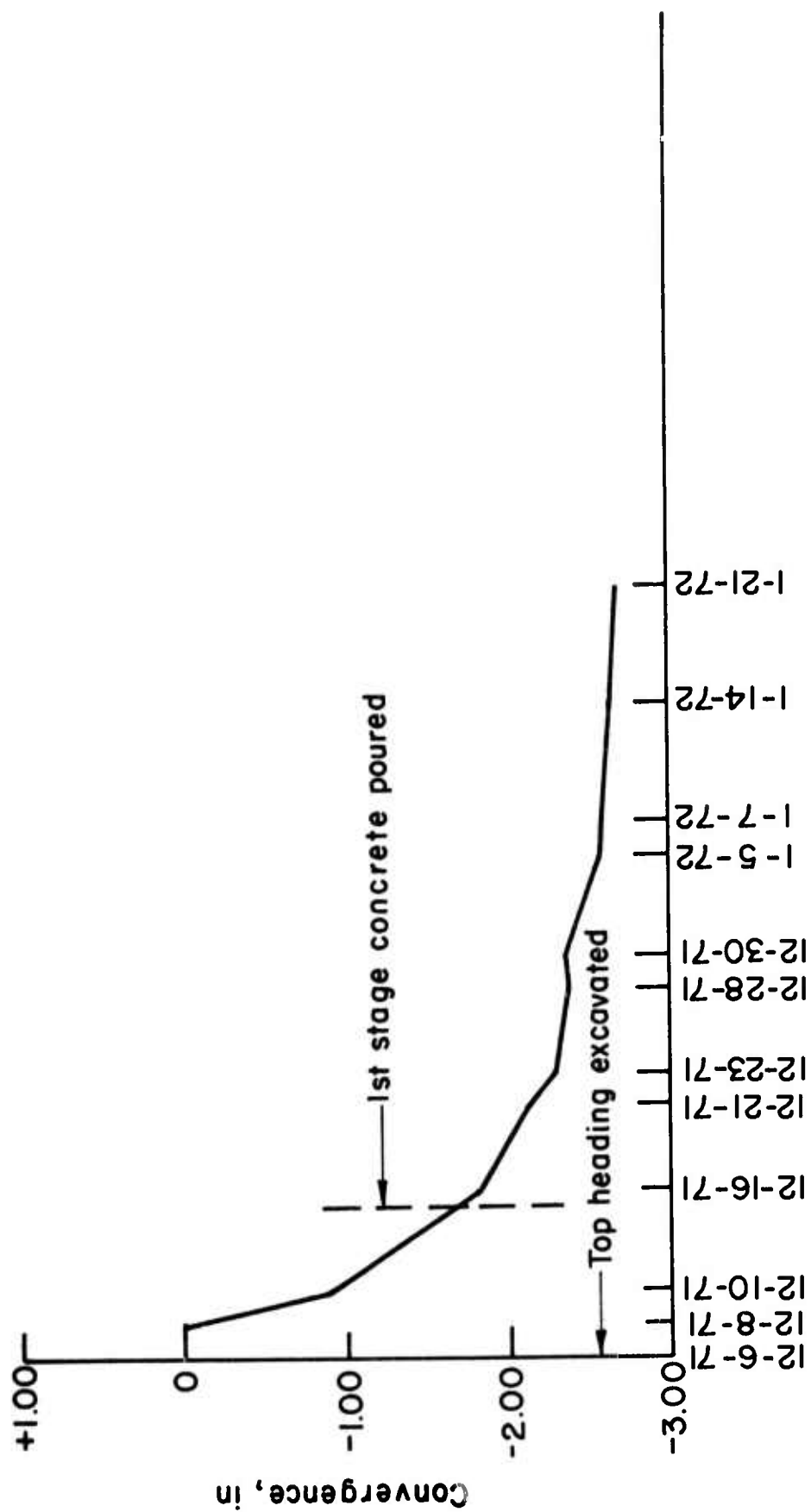


FIGURE 14C IN-SITU TAPE MEASUREMENT RESULTS - STRAIGHT CREEK TUNNEL

STRAIGHT CREEK TUNNEL
Station 87+00

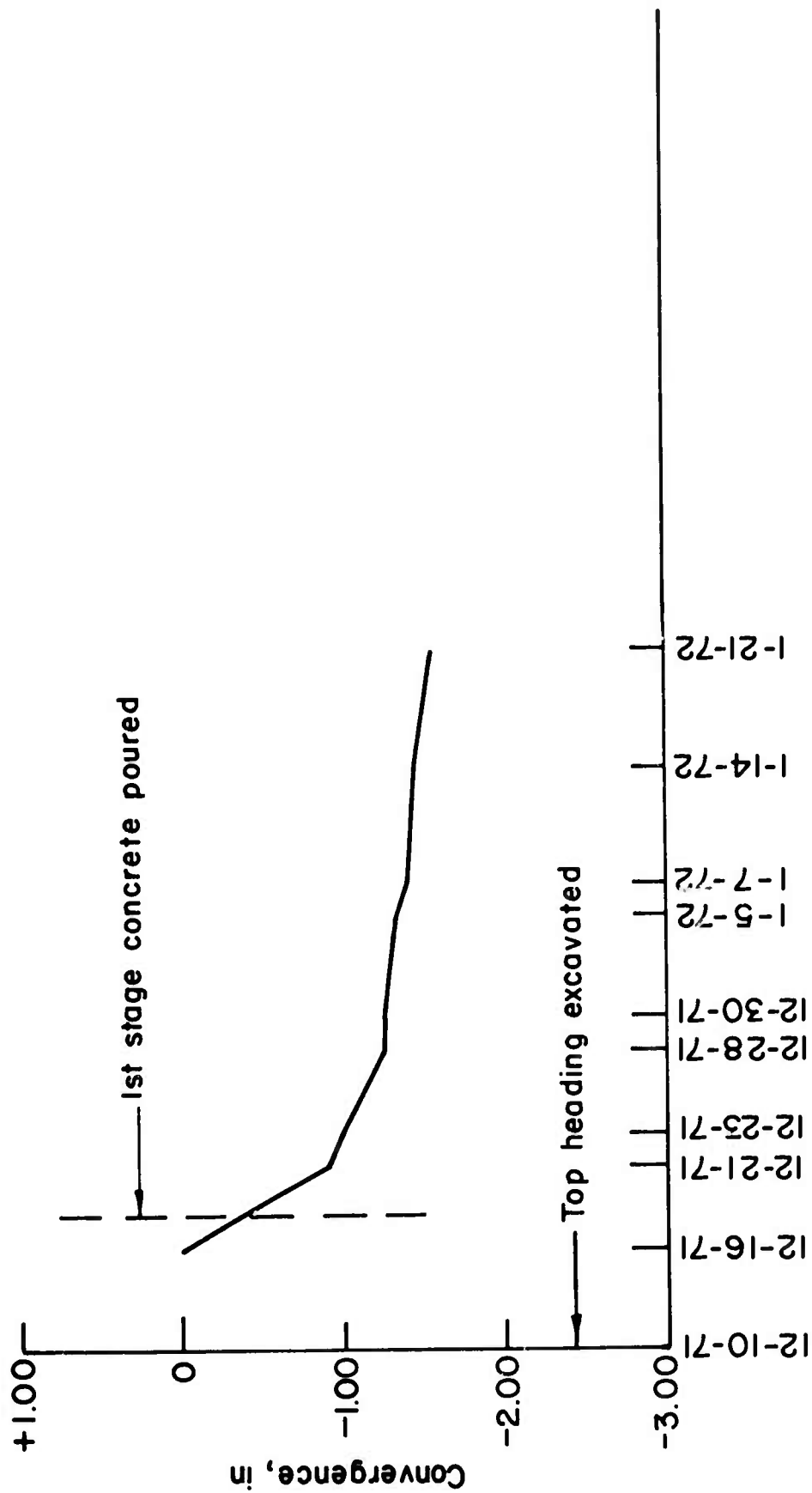


FIGURE 15C IN-SITU TAPE MEASUREMENT RESULTS - STRAIGHT CREEK TUNNEL

STRAIGHT CREEK TUNNEL
Station 87+60

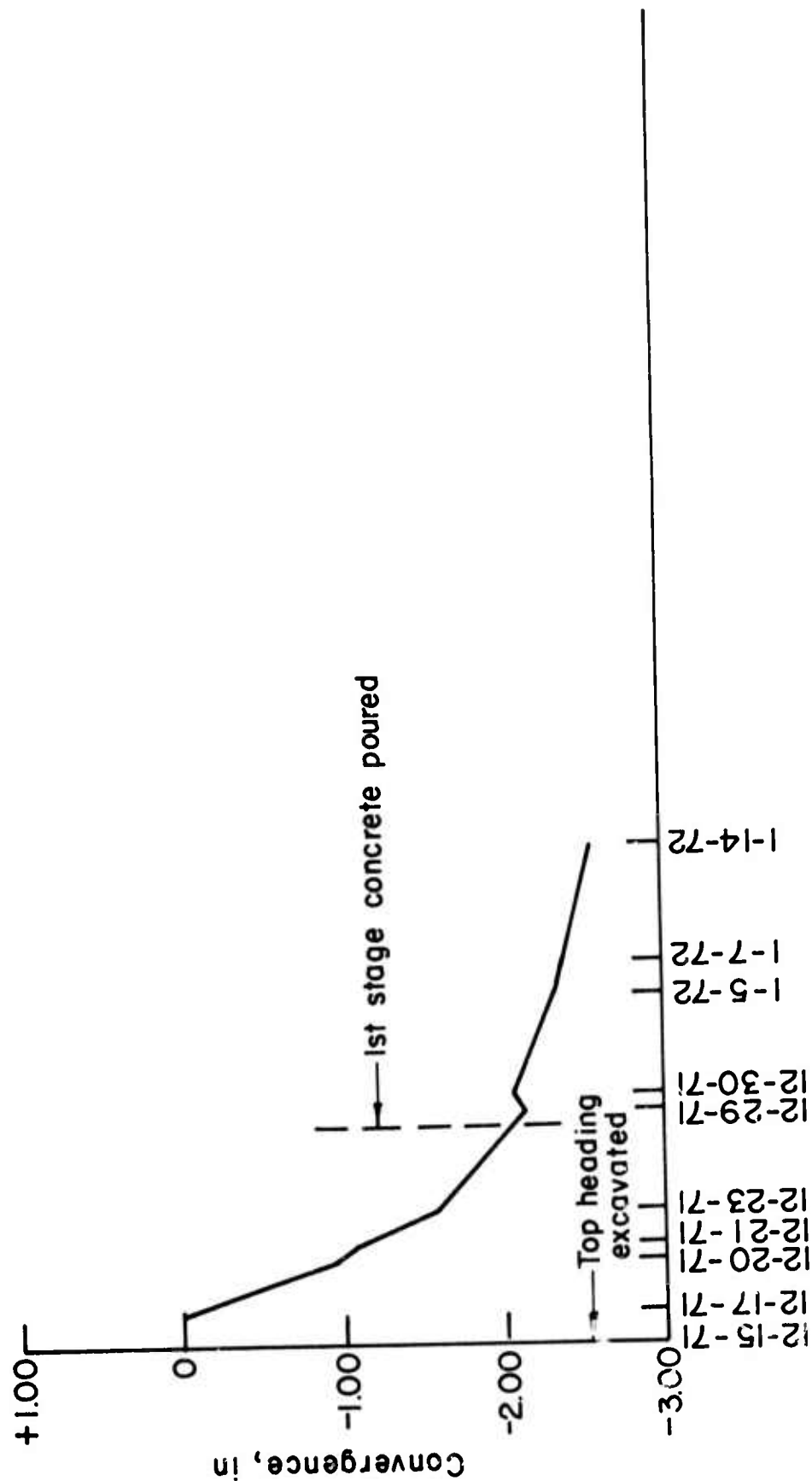


FIGURE 16C IN-SITU TAPE MEASUREMENT RESULTS - STRAIGHT CREEK TUNNEL

STRAIGHT CREEK TUNNEL
Station 88+10

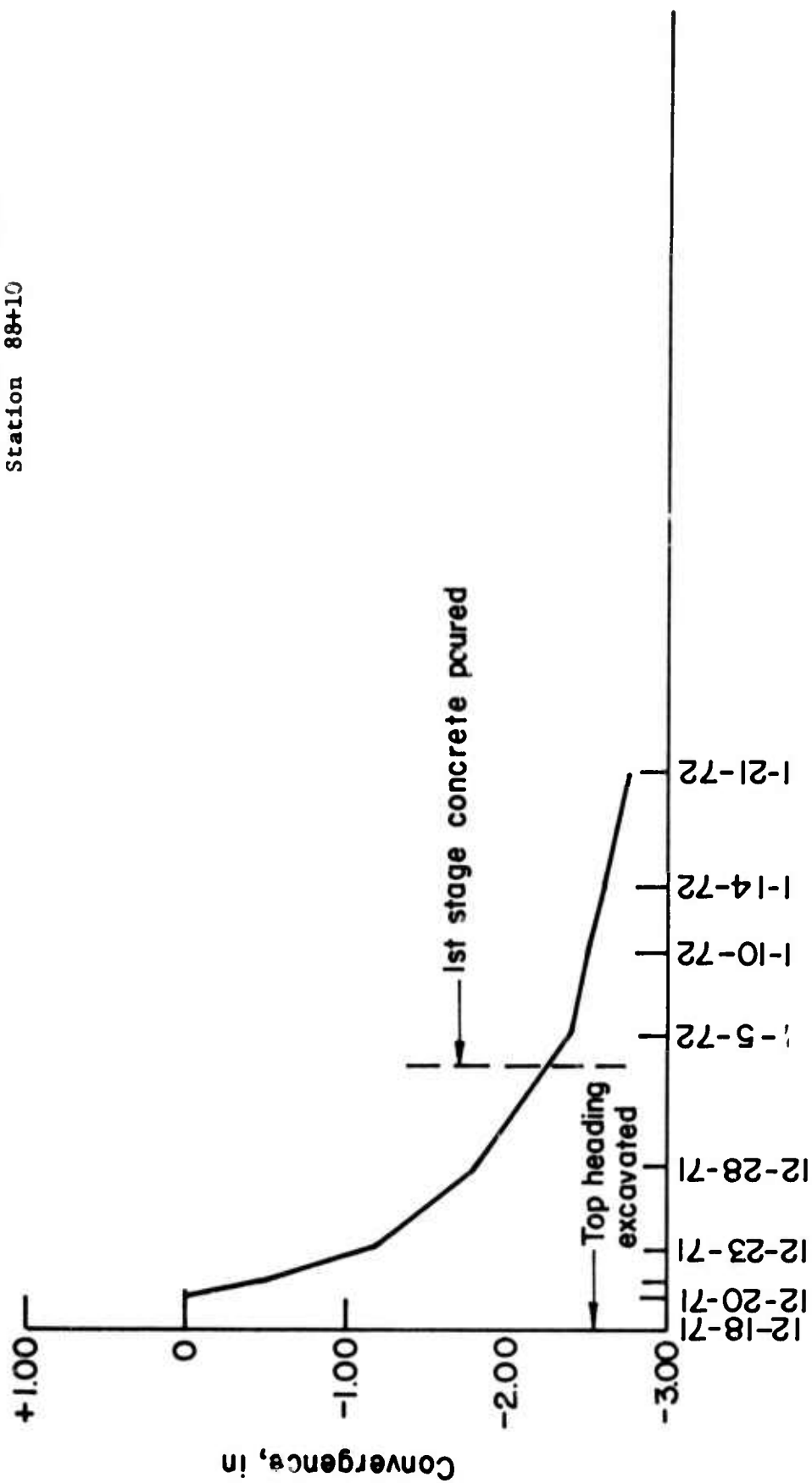


FIGURE 17C IN-SITU TAPE MEASUREMENT RESULTS - STRAIGHT CREEK TUNNEL