U.S. Department of Transportation
Urban Mass Transportation Administration

# The Atlanta Research Chamber

Office of Technology
Development and Deployment
Washington, D.C. 20590

# Applied Research For Tunnels

Blasting Techniques
Conventional Shotcrete
Steel-Fiber-Reinforced Shotcrete
Monographs
On The State-Of-The-Art Of Tunneling

Report No.
UMTA-GA-06-0007-81-1

MARCH 1981

FINAL REPORT

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16. Abstract


This Report discusses research performed for the Atlanta Research Chamber from October, 1977 through February, 1981. It also includes monographs on the state of the art of tunnelling.

The Atlanta Research Chamber was excavated some ten feet above, and approximately parallel to the twin Running Tunnels on the Metropolitan Atlanta Rapid Transit Authority (MARTA) Peachtree Center Station, Contract CN-120, below downtown Atlanta, Georgia. The Research Chamber was approximately 18 meters (60 feet) long and 5 to 6 meters (18 to 20 feet) in diameter. Applied research in controlled blasting, instrumentation, design using 2-D and 3-D Finite Element Method (FEM) analyses, and conventional and steel-fiber-reinforced shotcrete was accomplished and is reported in this document.

In addition to chapters on the work done in the Research Chamber, this Report contains monographs on the current state of the art of tunnelling. Owners, contractors, engineers, labor union representatives, insurance and legal personnel, and foreign and domestic experts have contributed their views.


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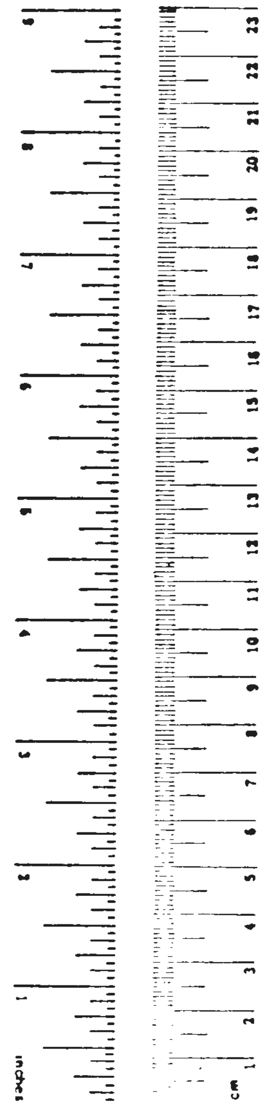
# METRIC CONVERSION FACTORS

## Approximate Conversions to Metric Measures

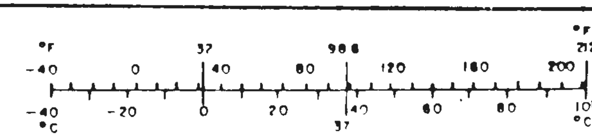
| Symbol | When You Know | Multiply by | To Find | Symbol |
|---|---|---|---|---|
| | **LENGTH** | | | |
| in | inches | *2.5 | centimeters | cm |
| ft | feet | 30 | centimeters | cm |
| yd | yards | 0.9 | meters | m |
| mi | miles | 1.6 | kilometers | km |
| | **AREA** | | | |
| $in^2$ | square inches | 6.5 | square centimeters | $cm^2$ |
| $ft^2$ | square feet | 0.09 | square meters | $m^2$ |
| $yd^2$ | square yards | 0.8 | square meters | $m^2$ |
| $mi^2$ | square miles | 2.6 | square kilometers | $km^2$ |
| | acres | 0.4 | hectares | ha |
| | **MASS (weight)** | | | |
| oz | ounces | 28 | grams | g |
| lb | pounds | 0.45 | kilograms | kg |
| | short tons (2000 lb) | 0.9 | tonnes | t |
| | **VOLUME** | | | |
| tsp | teaspoons | 5 | milliliters | ml |
| Tbsp | tablespoons | 15 | milliliters | ml |
| fl oz | fluid ounces | 30 | milliliters | ml |
| c | cups | 0.24 | liters | l |
| pt | pints | 0.47 | liters | l |
| qt | quarts | 0.95 | liters | l |
| gal | gallons | 3.8 | liters | l |
| $ft^3$ | cubic feet | 0.03 | cubic meters | $m^3$ |
| $yd^3$ | cubic yards | 0.76 | cubic meters | $m^3$ |
| | **TEMPERATURE (exact)** | | | |
| °F | Fahrenheit temperature | 5/9 (after subtracting 32) | Celsius temperature | °C |

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.




## Approximate Conversions from Metric Measures

| Symbol | When You Know | Multiply by | To Find |
|---|---|---|---|
| | **LENGTH** | | |
| mm | millimeters | 0.04 | inches |
| cm | centimeters | 0.4 | inches |
| m | meters | 3.3 | feet |
| m | meters | 1.1 | yards |
| km | kilometers | 0.6 | miles |
| | **AREA** | | |
| $cm^2$ | square centimeters | 0.16 | square inches |
| $m^2$ | square meters | 1.2 | square yards |
| $km^2$ | square kilometers | 0.4 | square miles |
| ha | hectares (10,000 $m^2$) | 2.5 | acres |
| | **MASS (weight)** | | |
| g | grams | 0.035 | ounces |
| kg | kilograms | 2.2 | pounds |
| t | tonnes (1000 kg) | 1.1 | short tons |
| | **VOLUME** | | |
| ml | milliliters | 0.03 | fluid ounces |
| l | liters | 2.1 | pints |
| l | liters | 1.06 | quarts |
| l | liters | 0.26 | gallons |
| $m^3$ | cubic meters | 35 | cubic feet |
| $m^3$ | cubic meters | 1.3 | cubic yards |
| | **TEMPERATURE (exact)** | | |
| °C | Celsius temperature | 9/5 (then add 32) | Fahrenheit temperature |




METRIC CONVERSION FACTORS

# ACKNOWLEDGEMENTS

The Atlanta Research Chamber was conceived by William C. Shepherd, Sr., of Parsons, Brinckerhoff/Tudor (PB/T); Mr. Shepherd is presently a private consultant. James L. Lammie is Project Director for PB/T, the general engineering consultants to the Metropolitan Atlanta Rapid Transit Authority (MARTA); he supported the concept and helped arrange the incorporation of the Research Chamber into the Peachtree Center Station CN-120 contract of the MARTA rapid transit system. Alan F. Kiepper, General Manager, and William D. Alexander, Assistant General Manager of MARTA, were instrumental in obtaining funds from the Urban Mass Transportation Administration (UMTA) for the Research Chamber. Gilbert L. Butler of UMTA, the sponsor's Project Manager, has been unwavering in his support. Without the vision, hard work and practical help these men provided, the Atlanta Research Chamber could not have been built, studied and documented.

PREFACE

This Final Report describes the construction of the Atlanta Research Chamber, and the research conducted in it. In addition, twenty-four monographs on the state-of-the-art of modern tunnel practice are included in the report.

The Atlanta Research Chamber was conceived as a team effort by William C. Shepherd, Sr., who organized eighteen individuals from twelve engineering firms in the United States, Canada and Austria to combine their special expertise to study various aspects of tunnel support systems in hard rock. Later, after Bill Shepherd's resignation to enter private consulting practice, Don Rose became Principal Investigator. The team was then expanded to include prominent university professors, and additional items were added to the research program. Finally, a number of team members were asked to write monographs on modern tunnel practice, summarizing their ideas on the subjects of their individual expertise. To balance these predominantly technical monographs, new team members were recruited to write monographs representing the views of owners, contractors, and labor, and presenting legal, insurance, overseas practice and additional technical ideas.

The first meeting of the initial team members took place in Atlanta in October 1977. Subsequent discussions were held using conference telephone calls to allow all key team members to participate in planning the work. This technique worked surprisingly well. The conference calls seemed as efficient as large face-to-face meetings, and were certainly more economical.

The CN-120 contract to build the Peachtree Center subway station was awarded by MARTA to the joint venture of Horn Construction Company, Inc. and Fruin-Colnon Corporation in January 1978. The Atlanta Research Chamber is a part of this CN-120 subway station contract. The Research Chamber was excavated in October - November 1978 and field research inside the chamber took place in late 1978 and early 1979.

The Atlanta Research Chamber was sponsored by the Urban Mass Transportation Administration (UMTA), U.S. Department of Transportation, through Research and Development Grant No. GA-06-0007. Mr. Gilbert L. Butler of UMTA is the sponsor's Project Manager.

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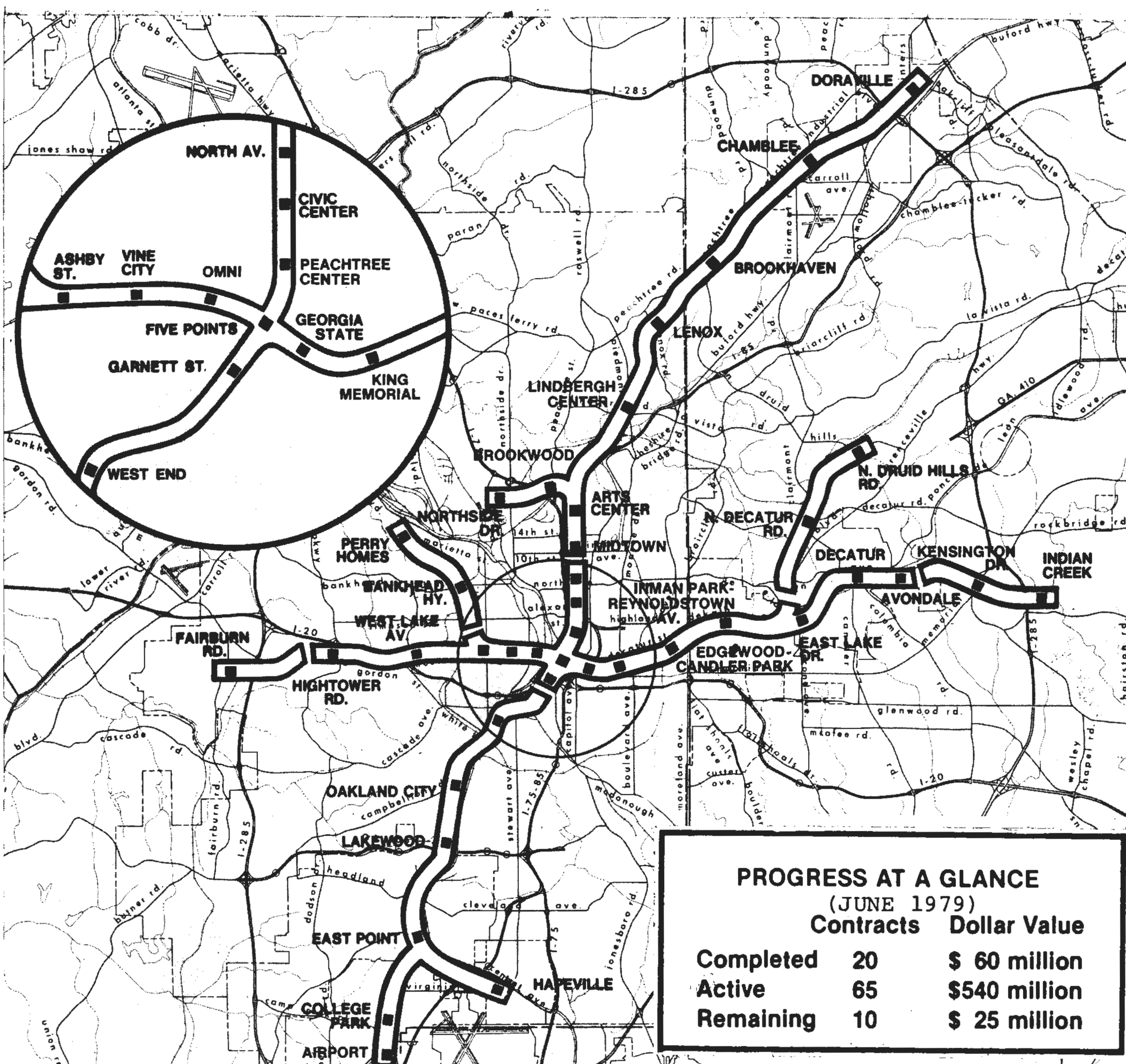
# THE ATLANTA RESEARCH CHAMBER REPORT

## CHAPTERS I THROUGH VIII

THE NEW MARTA TRANSIT SYSTEM • 53 miles of rail lines 8 miles of busways • 41 passenger stations • Comfortable high-speed trains • Electronic train control system • 750-volt traction power system • Modern fare collection system • Radio television communications systems • Fast interchange with 1500-mile bus network




**PROGRESS AT A GLANCE**
(JUNE 1979)

| | Contracts | Dollar Value |
|---|---|---|
| Completed | 20 | \$ 60 million |
| Active | 65 | \$540 million |
| Remaining | 10 | \$ 25 million |

FIGURE I-1

# CHAPTER I.

# INTRODUCTION

The desirability of utilizing underground space is apparent to all, and the need for rapid transit system tunnels in our crowded cities is obvious. Yet in recent years, rapid transit costs have increased in the United States to such a degree that federal funding may be withheld if economies are not made.[1/] The Urban Mass Transportation Administration (UMTA) funded the Atlanta Research Chamber as part of UMTA's continuing attempt to discover ways to reduce tunnel and rapid transit costs.

The Atlanta Research Chamber is located directly below Peachtree Street in downtown Atlanta, Georgia (see Figures I-1 and I-2). It is included in the CN-120 contract for the Peachtree Center Station, a part of the Metropolitan Atlanta Rapid Transit Authority (MARTA) multi-billion dollar "Phase A" 53-mile transit project, which is 80 percent funded by UMTA with 20 percent provided by the people of Fulton and DeKalb counties. The Peachtree Center Station is located in a topographic "high" of excellent granitic gneiss, although the entry and exit twin Running Tunnels extend away from this high ground into less desirable rock and finally into soft ground conditions north and south of the station. The Atlanta Research Chamber, therefore, is located in some of the best rock in the region.

The Research Chamber is located immediately south of the main Peachtree Center Station cavern, and is in part an enlargement of the pre-existing Pilot Tunnel excavated to provide bidders on the CN-120 contract a view of the rock conditions. The Research Chamber lies some 3 meters above, and is approximately

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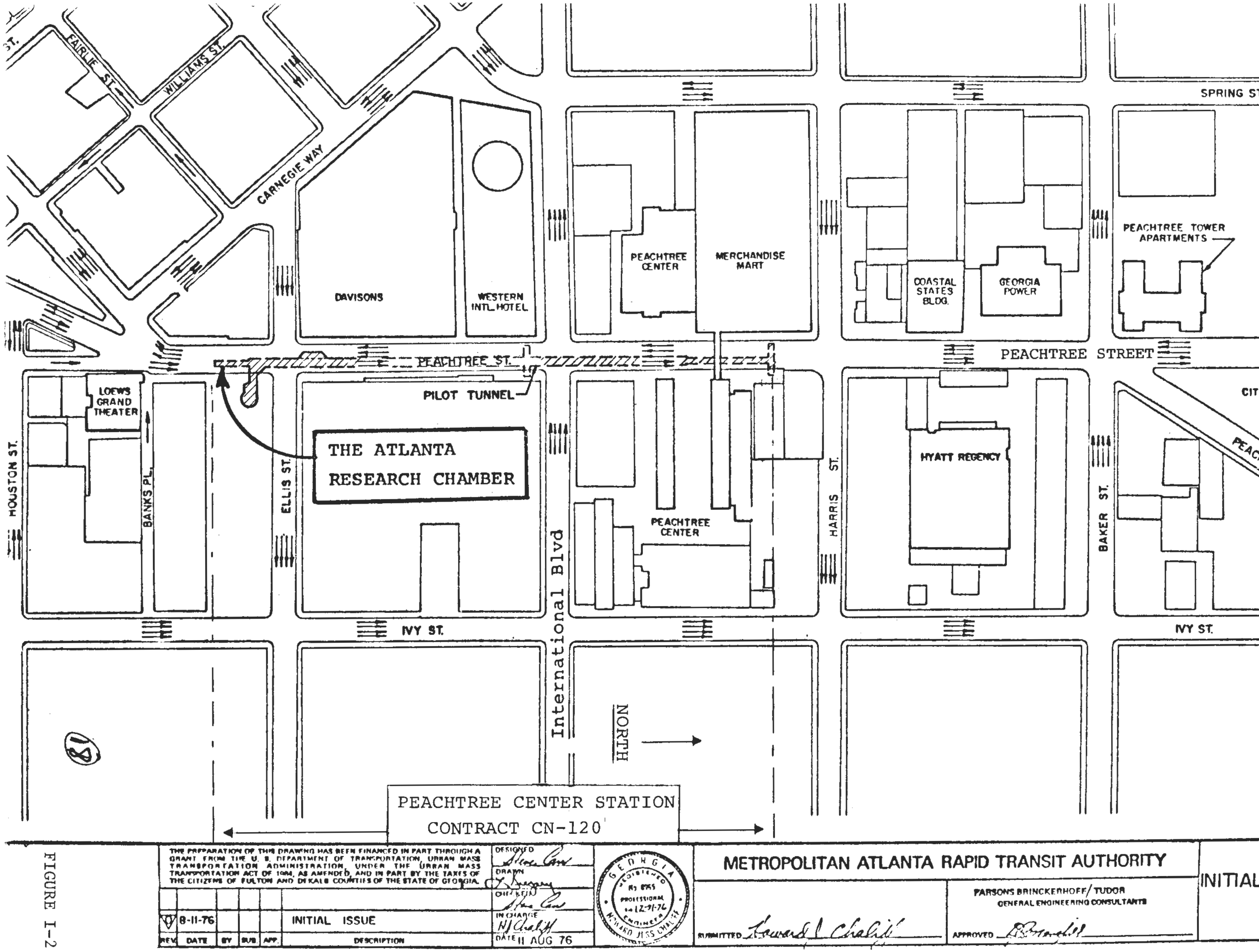
1/ "Soaring Costs Could Nip New Rail Transit Plans", <u>Engineering News-Record</u>, December 14, 1979, page 9.


FAIRLIE ST.
WILLIAMS ST.
CARNEGIE WAY
SPRING ST.
DAVISONS
WESTERN INT'L HOTEL
PEACHTREE CENTER
MERCHANDISE MART
COASTAL STATES BLDG.
GEORGIA POWER
PEACHTREE TOWER APARTMENTS
PEACHTREE ST.
PEACHTREE STREET
PILOT TUNNEL
LOEWS GRAND THEATER
THE ATLANTA RESEARCH CHAMBER
HOUSTON ST.
BANKS PL.
ELLIS ST.
International Blvd
PEACHTREE CENTER
HARRIS ST.
HYATT REGENCY
BAKER ST.
CITY
PEACHT
IVY ST.
IVY ST.
NORTH
18
PEACHTREE CENTER STATION
CONTRACT CN-120
THE PREPARATION OF THIS DRAWING HAS BEEN FINANCED IN PART THROUGH A GRANT FROM THE U. S. DEPARTMENT OF TRANSPORTATION, URBAN MASS TRANSPORTATION ADMINISTRATION, UNDER THE URBAN MASS TRANSPORTATION ACT OF 1964, AS AMENDED, AND IN PART BY THE TAXES OF THE CITIZENS OF FULTON AND DEKALB COUNTIES OF THE STATE OF GEORGIA.
8-11-76
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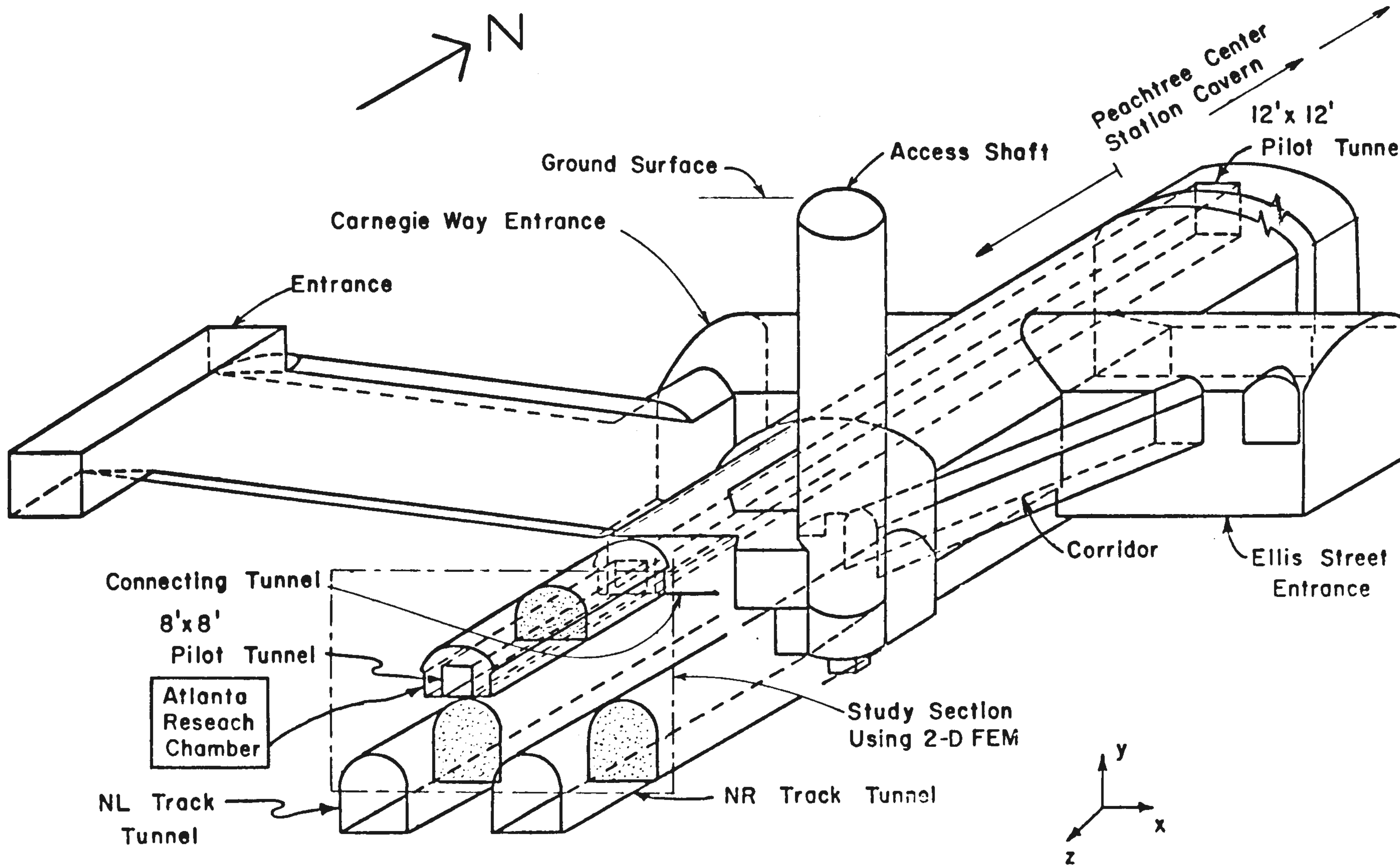
GEORGIA REGISTERED PROFESSIONAL ENGINEER
METROPOLITAN ATLANTA RAPID TRANSIT AUTHORITY
PARSONS BRINCKERHOFF/TUDOR
GENERAL ENGINEERING CONSULTANTS
SUBMITTED
APPROVED
INITIAL

FIGURE I-2

parallel to, the twin Running Tunnels (see Figures I-3 and I-4). The Research Chamber is 18 meters long and has the same 5.5 meter diameter horseshoe-shape design dimensions as the twin Running Tunnels.

The research project is described in the following pages. As described earlier in the Preface, work in the Atlanta Research Chamber was a team effort. The initial draft of each chapter in this report was written by the team member in charge of the particular work; the final text was prepared by the Principal Investigator in order to ensure a certain uniformity in style. The introduction and a brief overview found in Chapters I and II, respectively, were written by Don Rose of Tudor Engineering Company of San Francisco, who is the Principal Investigator for the project. The geology of the Research Chamber and in-situ stress conditions are described in Chapter III by Harold Whitney and Ken Akins of Law Engineering Testing Company of Atlanta. Design of the Research Chamber and an evaluation of rock movements expected as the twin Running Tunnels were excavated immediately below the Research Chamber, using two dimensional Finite Element Method (2-D FEM) techniques, is described in Chapter IV by Dr. Fred Kulhawy of Cornell University. Geotechnical instrumentation of the Research Chamber was extensive and is discussed in Chapter V by Dr. Iain Weir-Jones of Vancouver, British Columbia, Canada. Excavation by blasting in part utilized a "scribing tool" to notch perimeter drill holes, in an attempt to initiate crack propagation at the notches and so control the direction of cracking at the Research Chamber perimeter. This is discussed in Chapter VI by Lew Oriard of Huntington Beach, California. Extensive laboratory and field work on conventional shotcrete preceded the placement and testing of this material in the Research Chamber. This is described in Chapter VII by Robin Mason and Loren Lorig of A. A. Mathews of Rockville, Maryland. Steel-fiber-reinforced shotcrete was also subjected to careful pre-construction study before placing this material in the Research Chamber as discussed in Chapter VIII by Professors




OBLIQUE VIEW OF THE PEACHTREE CENTER STATION, SOUTHERN HALF

20

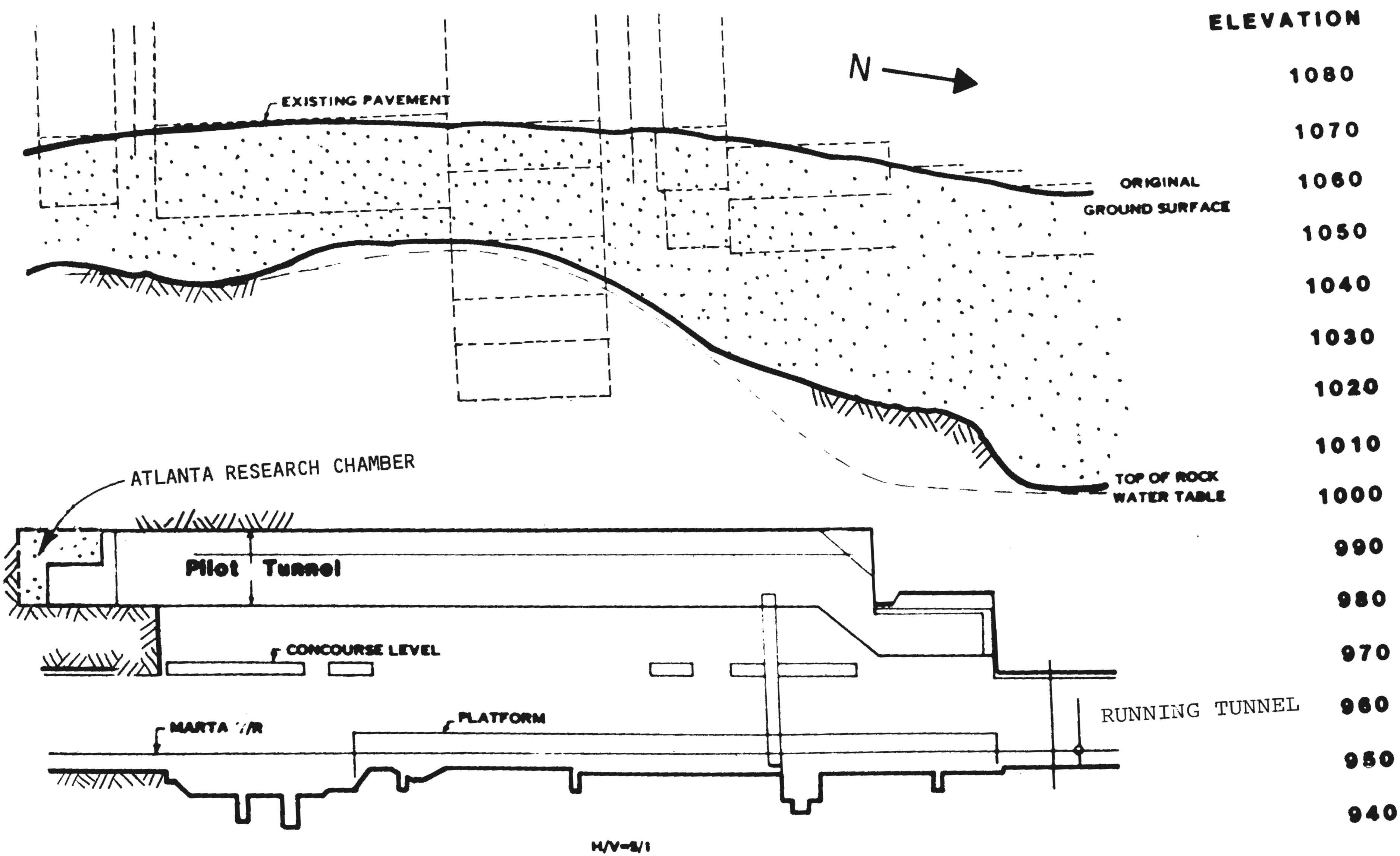
FIGURE I-3

Ed Cording, Jim Mahar and Gabriel Fernandez-Delgado of the University of Illinois.

All of the items studied were of practical use in rapid transit tunneling. The steel-fiber-reinforced shotcrete was subsequently used in a sixty meter section in one of MARTA's twin Running Tunnels, in place of the conventional shotcrete for tunnel support and final lining used elsewhere on the CN-120 contract.

Following the chapters listed above, which discuss the technical work done in the Atlanta Research Chamber, are a series of monographs by a number of outstanding experts. These monographs are designed to tap the huge reservoir of experience and expertise possessed by the team members. Tunnel practice overseas; the state-of-the-art of shotcrete design; three dimensional Finite Element Method (3-D FEM) computer studies of the Atlanta Research Chamber and of the Nuremburg subway tunnels; and similar technical monograph topics have been balanced by monographs from owners, contractors, labor, legal, insurance experts and others, which discuss the larger view. All have practical application, and, by being gathered together in one report, may serve to promote the common goal, which is to construct underground space economically and safely.

The views expressed in the twenty-four monographs are those of the separate authors, and the texts of their monographs are presented here substantially unchanged. As noted earlier, however, the text of Chapters I to VIII concerning the Atlanta research *per se* was first drafted by the team members in charge and subsequently edited by Don Rose of Tudor Engineering Company, the Principal Investigator. Various cross-references to the several research chapters and certain overall conclusions were written by the Principal Investigator, and the final text is his responsibility.




PEACHTREE CENTER STATION

FIGURE I-4

# CHAPTER II.

# SUMMARY AND OVERVIEW

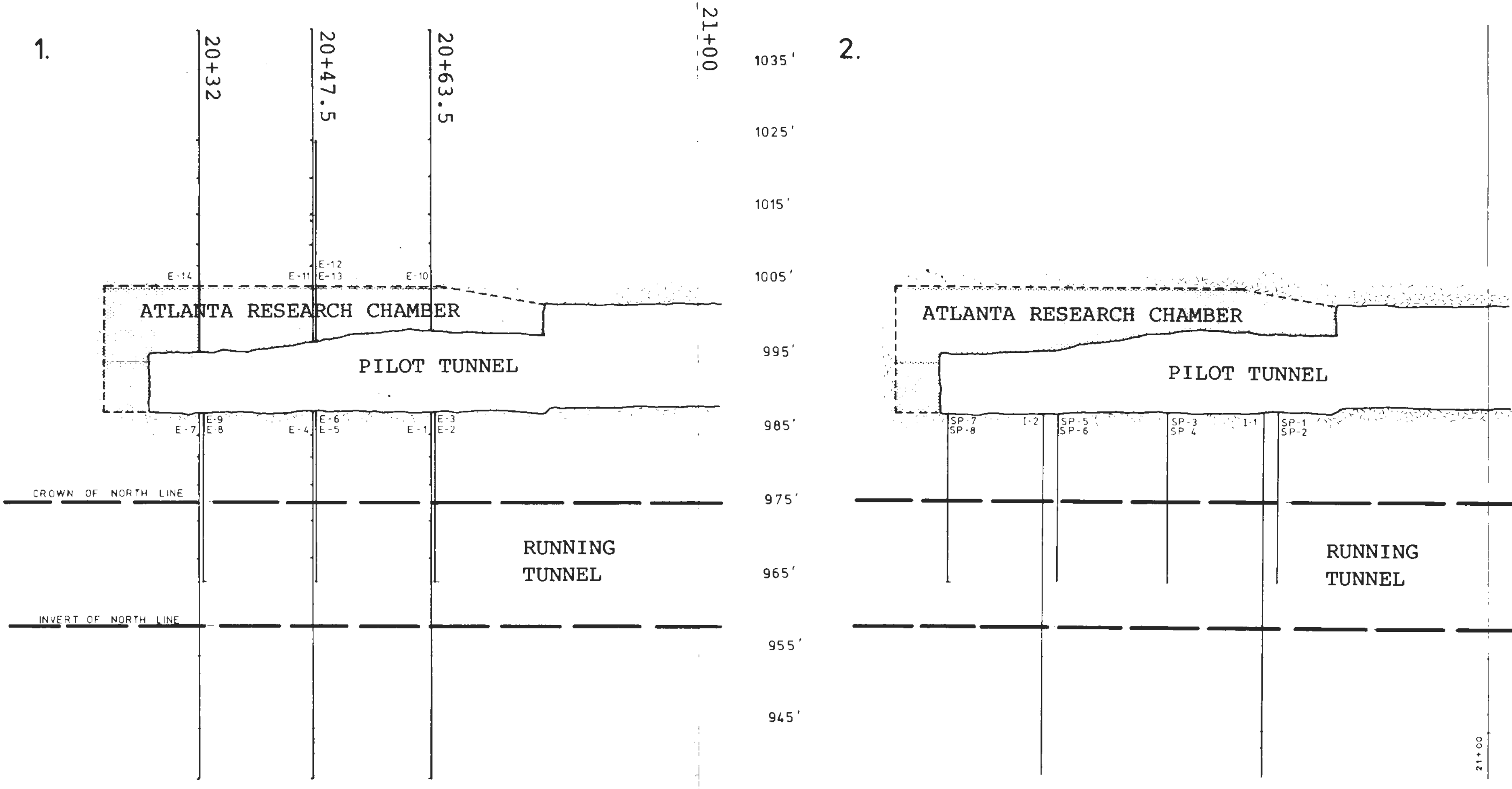
## A. SUMMARY

### 1. Geology

The geology of the Atlanta Research Chamber was exceptionally well understood, because the Pilot Tunnel excavated earlier for the main Peachtree Center Station cavern revealed the rock in the Research Chamber area, and instrumentation had been installed and in-situ stress measurements had been made as part of the Pilot Tunnel work. The remarkably complete description of the rock was made possible because the same lab personnel that had physically tested the rock were also available to help map and differentiate the rock types exposed in the Pilot Tunnel walls in the Research Chamber area. Mr. Frank Shuri of Foundation Sciences Inc. and Messrs. Robert White and Chris Potter, of Law Engineering Testing Company provided excellent geologic and geotechnical data. The rock was "granitic" in overall appearance. It was technically a gneiss and the foliation was primarily near-horizontal, which tended to control blast surfaces. Rock quality was excellent, and in the Atlanta Research Chamber itself no major joints existed. See Chapter III by Harold Whitney and Ken Akins of Law Engineering Testing Company for details.

### 2. Design

The design of the Atlanta Research Chamber was deliberately made to have the same dimensions and configuration as the twin Running Tunnels which were located some three meters below the Research Chamber. The Research Chamber was an enlargement of the existing 3.6 by 4.2 meter (10 feet by 12 feet) Pilot Tunnel which ran down the crown of the Peachtree Center Station. Because the existing Pilot Tunnel was not parallel to the twin Running Tunnels, the Research Chamber does not parallel the Pilot Tunnel.

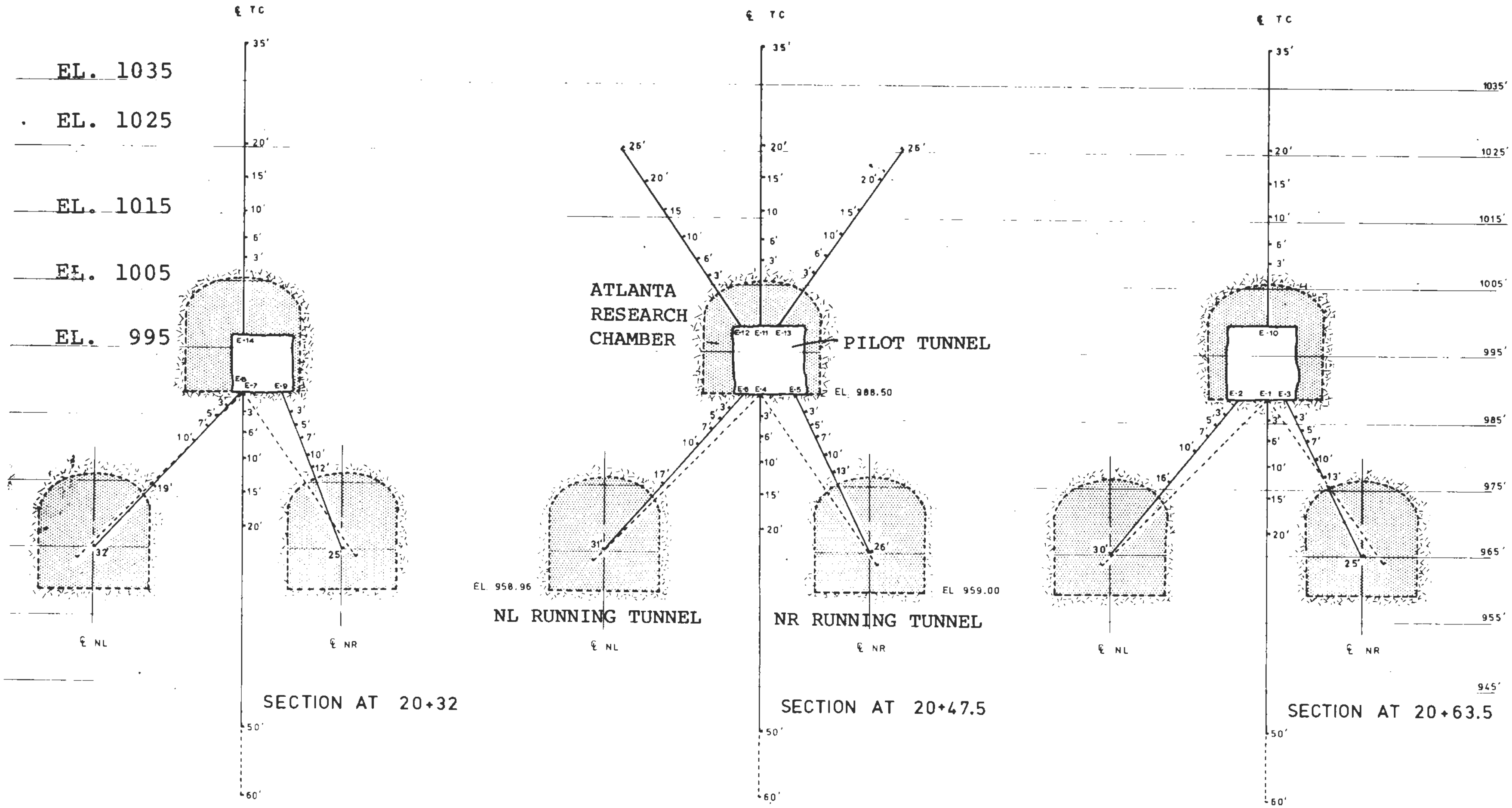



BOREHOLE EXTENSOMETER LOCATIONS.

INCLINOMETER AND SEISMIC PROFILER HOLES.

INSTRUMENTATION PROFILE

FIGURE II-




INSTRUMENTATION - SECTION

FIGURE II-2

A two-dimensional Finite Element Method (2-D FEM) pre-construction analysis was performed to analyze the effect of these multiple tunnel and chamber openings on one another. First, in this analysis, the Pilot Tunnel stresses and deflections were studied. Second, the Pilot Tunnel was enlarged to the Research Chamber dimensions. Third, the first Running Tunnel was excavated. Fourth, the second Running Tunnel was excavated. At each step, the influence of the new excavation on the previous condition was shown. Because in the actual construction sequence the Running Tunnels were excavated before the Research Chamber, a post-construction analysis was also made. See Chapter IV by Dr. Fred Kulhawy of Cornell University for details.

3. <u>Instrumentation</u>

A complex instrumentation program was developed and installed (see Figures II-1 and II-2). The instrumentation data indicated that although rock movements were small, they were different than predicted by the 2-D FEM design study. Difficulty in getting good cooperation from field personnel caused unexpected problems; see Chapter V by Dr. Weir-Jones.

4. <u>Excavation</u>

The 5.5 meter (18 foot) diameter horseshoe Research Chamber was excavated by conventional drill-and-blast techniques, for its full 18 meter (60 foot) length. For convenience, the Research Chamber was divided into nine "panels" where different aspects of the applied research were performed (see Figures II-3 and II-4). Panel 9 was excavated first, and the excavation progressed southward to end at Panel 1. Normally, one panel was excavated in one round. The first eight panels were each 2.1 meters (7 feet) long. The last panel which was Panel 1, was 1.2 meters (4 feet) long. Variations in blasting technique were made to determine the best blasting results for this rock. An innovation was the use of a "scribing tool" to attempt to control the perimeter fracture. It was found that anisotropy existing in the rock mass, in that near-horizontal planes in the gneiss existed,

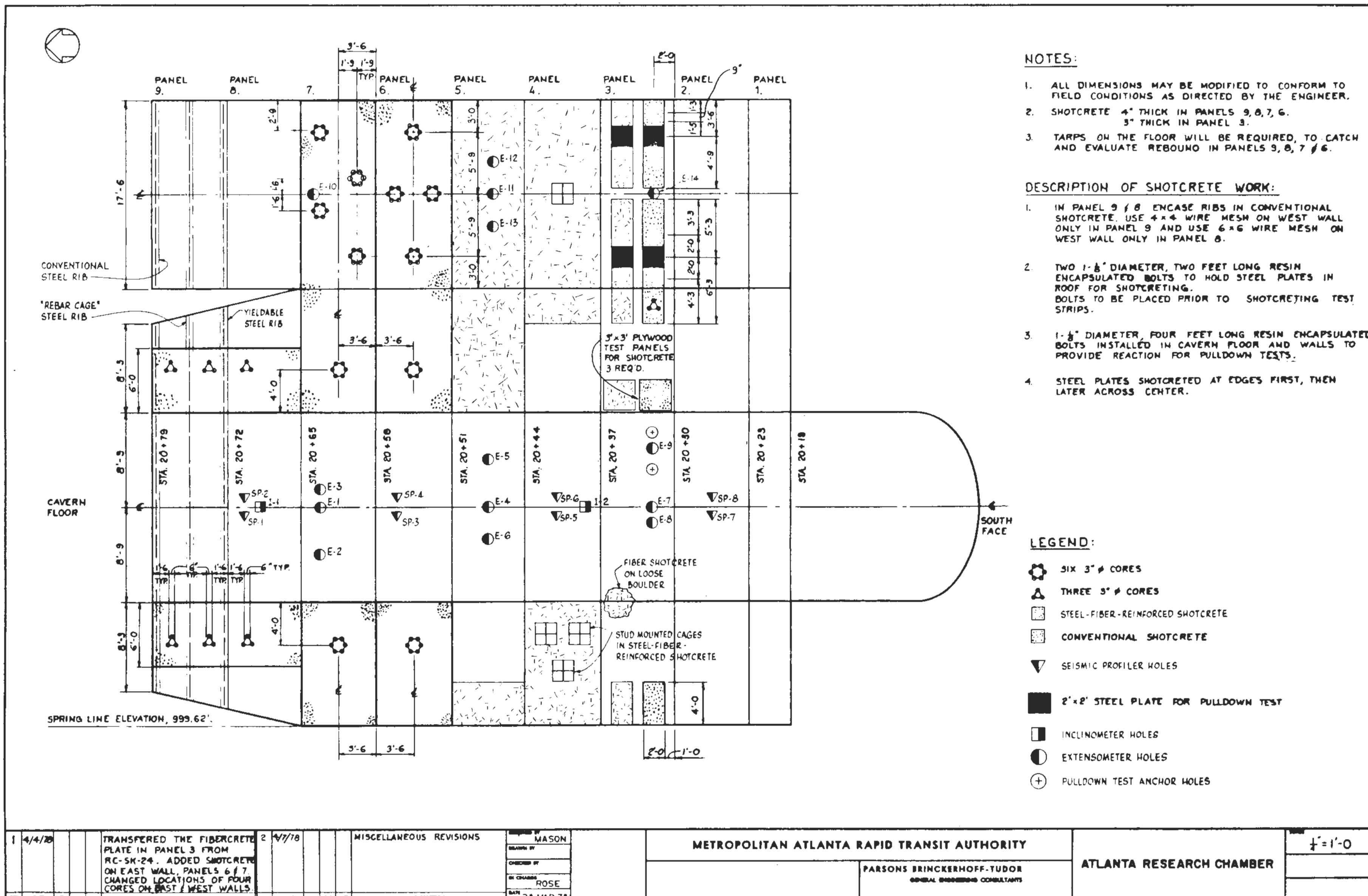
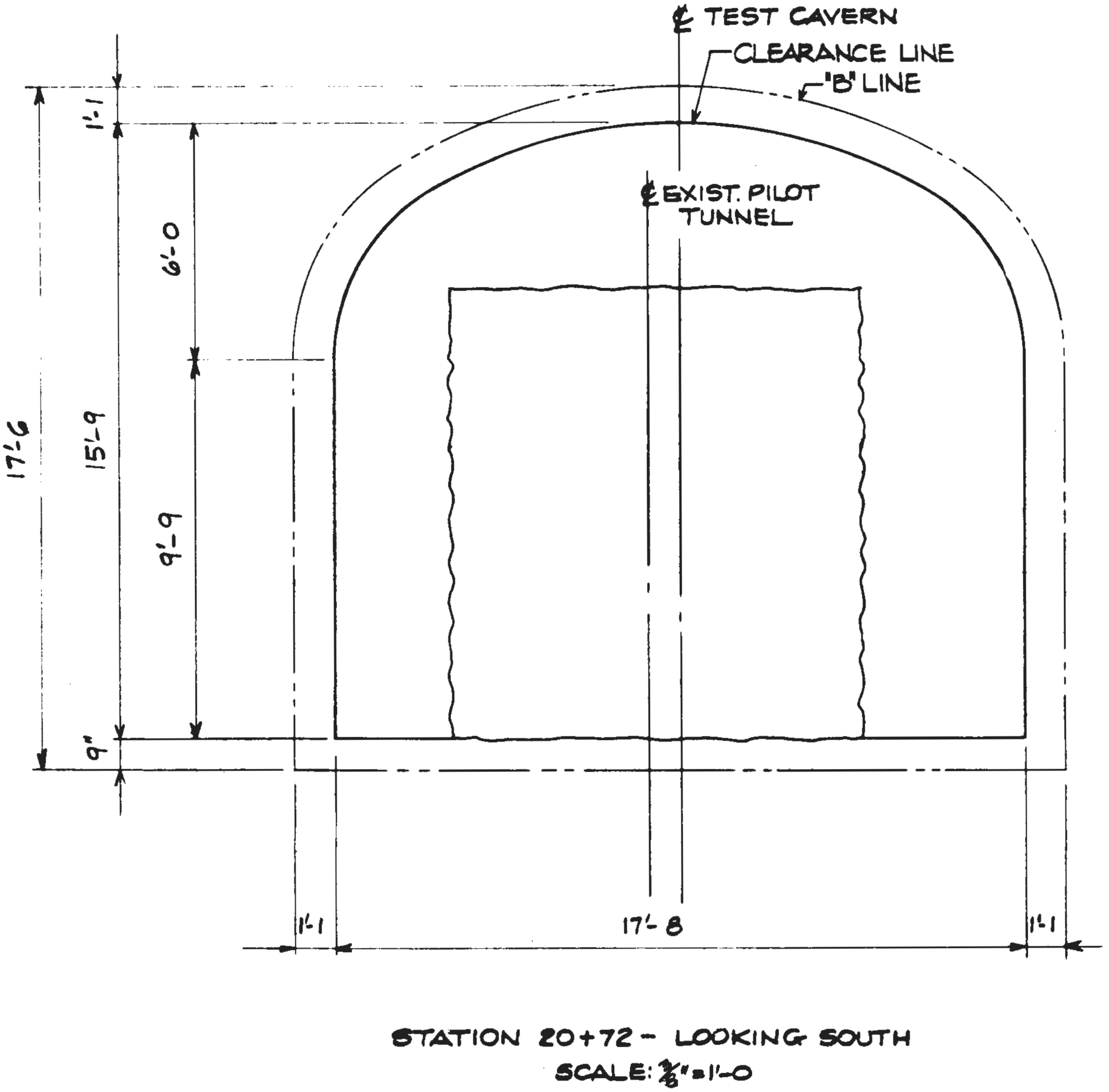



FIGURE II-3 Research Chamber




The Atlanta Research Chamber

FIGURE II-4

virtually controlled the rock breakage. A flat horizontal roof tended to form due to the breakage along near-horizontal foliation, even when the scribing tool was used. The scribing tool technique has been used with very marked success in the Coldspring Granite Company commercial granite quarries in Minnesota, and in a few construction projects elsewhere, and is considered a very promising technique. Lew Oriard of Lewis L. Oriard, Inc., discusses excavation of the Research Chamber in Chapter VI and discusses the scribing tool further in his monograph.

5. Conventional Shotcrete

Research into laboratory tests of conventional shotcrete was made, and it was found that some specified tests could not be duplicated from laboratory to laboratory, and may not be valid tests. Field work with inorganic Sigunit and organic Dry Shot accelerators revealed that if properly added to the mix, both produced excellent conventional shotcrete. Shotcrete placed on the walls and roof of the Research Cavern was subjected to a number of tests. It seemed that shotcrete on the walls of the cavern was more dense and strong than that placed on the roof, which is believed to be a typical condition that should be accounted for in modern shotcrete design. Robin Mason and Loren Lorig of A. A. Mathews discuss conventional shotcrete in Chapter VII.

6. Steel-Fiber-Reinforced Shotcrete

The conventional shotcrete discussed in Chapter VII was used as a base material and U.S. Steel fibers 2.5 centimeters long were added to produce a ductile and tough steel-fiber-reinforced shotcrete. About 70 $kg/m^3$ (116 lb/cy) of fibers were added. This material was tested in the field and laboratory and proved to be an outstanding practical success. The twin Running Tunnels in the CN-120 contract for the Peachtree Center Station were designed using conventional shotcrete with wire mesh for the permanent final lining. After the successful demonstration of steel-fiber-reinforced shotcrete in the Atlanta Research Chamber, a Change Order was issued to use the steel-fiber-reinforced shotcrete instead of conventional shotcrete with wire mesh for sixty meters (200 feet) in one of the twin Running Tunnels. The Contractor expressed a keen interest in the use of steel-fiber-reinforced shotcrete. Besides its superior material properties, it appears somewhat cheaper to install because the time-consuming placement of a wire mesh is not required. See Chapter VIII for a full discussion by Professors Ed Cording, Jim Mahar and Gabriel Fernandez-Delgado of the University of Illinois, who were the team members in charge of the work, in the Research Chamber. Also see the monographs by Tom Buchanan and Gene Root for details of work in the Running Tunnels.

## B. OVERVIEW

The Atlanta Research Chamber pioneered several "firsts" in North American rapid transit tunnel practice, as well as providing new data on several items previously used elsewhere.

Dr. Fred Kulhawy's study is believed to be one of the few using 2-D FEM on a real project to design a complex series of tunnel excavations. Both 2-D and 3-D FEM predictions were compared to the rock movements actually measured in the field.

The scribing tool for controlled perimeter blasting had never been used in tunnel work before, although UMTA-sponsored research in the Boston Red Line on this technique was also performed a few months later, after the work in Atlanta was completed. The Atlanta raw data was made available to the Boston Red Line researchers. Scribing of a drill hole could be accomplished easily in less than two minutes per hole. Proper orientation of a scribing tool around the tunnel perimeter did not require special tools, but could be closely set by a competent driller rotating the drill boom as required. Scribing bits tended to wear, and sometimes tended to spiral down the drill hole along weak planes in the rock. However, it was clear that the scribing technique has value in tunnel and heavy construction work, especially in non-foliated rock. Mr. Joe Peters, of the Coldspring Granite Company in Cold Spring, Minnesota, was of invaluable help to the research program. He has used scribing commercially in uniform granites to cause cracks to propagate in a straight line from scribed drill holes five meters apart, using very light charges.

Our laboratory research in conventional shotcrete indicated that some tests called for in modern specifications cannot be duplicated from laboratory to laboratory and hence may not be useful tests. High strength conventional shotcrete can be obtained using ordinary Sigunit or the newer organic accelerator Dry Shot. Strength is lower in the roof shotcrete than in wall shotcrete. These results are of immediate use in tunnel design.

The use of steel-fiber-reinforced shotcrete in 60 meters of one of the twin Running Tunnels, for the final lining of the subway, is another "first" in North America. The material is superior to conventional shotcrete in several respects and once contractors have confidence in it, may be significantly cheaper. The fibers must be fed into the shotcrete system using a "spreader" to ensure even distribution of fibers and prevention of clumping. The commercially available spreader used in the Atlanta Research Chamber was a Hansen Fiber-Meter, Model 200, which successfully proved the steel-fiber-shotcrete can be rapidly placed without delays.

The Atlanta Research Chamber was primarily a practical effort by practical researchers, attempting to develop useful tools for tunnel designers and builders. New applied research in scribing tool blasting techniques and in steel-fiber-reinforced shotcrete was performed for the first time in North American tunnels. Our team efforts in conventional shotcrete and in 2-D FEM design provided new insights. Geology and instrumentation data were unusually complete and made rational analysis possible. The applied research work in the Atlanta Research Chamber was a team success.

# CHAPTER III.

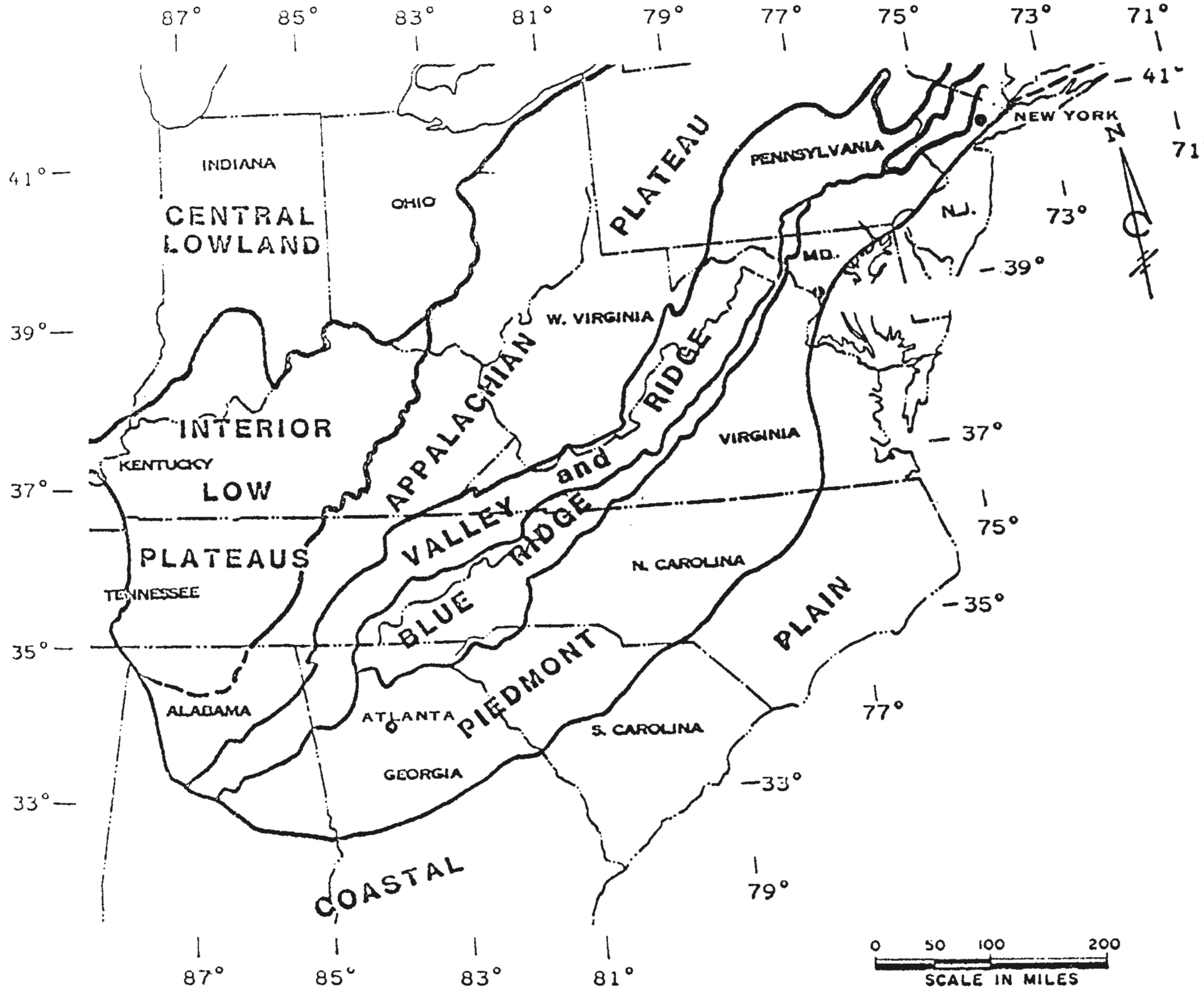
# GEOLOGY

## I. ATLANTA REGIONAL GEOLOGY

Atlanta is located within the Piedmont geologic province, which extends from Alabama to New York (see Figure III-1). Washington, D.C., and New York City are in the same geologic province. The geology of the southern Piedmont geologic province is well known and has been reported in many sources. The following summary of Regional Geology, based on these sources and our experience in the area, gives an introduction to the geologic history, structure, and primary rock types of the region surrounding Atlanta. Further discussions may be found in the sources cited and in the Supplemental Bibliography included in this chapter. Specific details of the Atlanta Research Chamber Geology are discussed in the second part of this chapter.

The Piedmont Province is generally described as a gently rolling plain of moderate relief that is bounded on the northwest by the Brevard Belt and on the southeast by an angular unconformity which is exposed along the western edge of the Cretaceous sediments of the Coastal Plain. The physiographic expression of this unconformity is commonly referred to as the "Fall Line." The Piedmont metamorphic rocks appear to directly overlie a sequence of older, basement gneisses of the Precambrian Era (approximately 1,100 million years old).

The Piedmont metamorphic rocks are derived from sediments deposited 400 to 600 million years ago during the late Precambrian to early Paleozoic era. These sediments were subjected to several periods of intense heat and pressure resulting in widespread deformation, metamorphism, and granitic intrusions.




GENERALIZED GEOLOGY OF THE EASTERN UNITED STATES

FIGURE III-1

The most recent and most intense of these periods occurred approximately 250 million years ago, late in the Paleozoic era.[1/]

Granitic intrusions occurred in the Southern Piedmont between 600 and 250 million years ago. These granite bodies were intruded prior to or during the last major deformation which occurred 250 million years ago. Mafic dikes are the youngest intrusions in the Piedmont rocks, having been intruded between 200 and 150 million years ago in the Mesozoic era.[2/]

Recumbent folding is the predominant structural style in this area of the Piedmont. Orientation of fold axes vary, but they primarily plunge gently to the northeast. The regional foliation in the Atlanta area strikes northeast and dips gently to the southeast. Local variations in attitude are caused by folding (both open and recumbent) on a variety of scales.

Widespread development of joints (naturally occurring planar fractures in rock) occurred after the last major deformation, as a result of volume changes and directed stresses in the rock mass. Volume changes were caused by cooling of rock following episodes of intense heat and pressure and by temperature changes related to magmatic activity. Regional directed stresses resulted from large-scale continental movements. Local directed stresses may have been set up by movement of magma or from local relief of regional stresses.

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1/ Crickmay, Geoffrey W., 1952, "Geology of the Crystalline Rocks of Georgia", Georgia Geological Survey, Bulletin No. 58; Fullager, P. D., 1971, "Age and Origin of Plutonic Intrusions in the Piedmont of the Southeastern Appalachians", Geological Society of America Bulletin, Vol. 82, pp. 1845-1862; Hurst, V. J., 1970, "The Piedmont in Georgia", In Studies of Appalachian Geology, Fisher, et. al., editors, John Wiley and Sons, Chapter 26, pp. 383-396; and Massachusetts Institute of Technology, 1958, "Age Study of Some Crystalline Rocks of the Georgia Piedmont", U.S. AEC Dept. NYD-3938, pp. 58-60.

2/ Fullager, P. D., op. cit., and Smith, James W., Wamples, J. M., and Green, Martha A., 1968, "Isotopic Dating and Metamorphic Isograds of the Crystalline Rocks of Georgia", Georgia Geological Survey, Bulletin No. 80.

Hydrous solutions moving in these joints have deposited mineral fillings. In the Atlanta area, common joint fillings include calcite, zeolite, chlorite, and quartz. Similar joint fillings are widespread through the Piedmont.

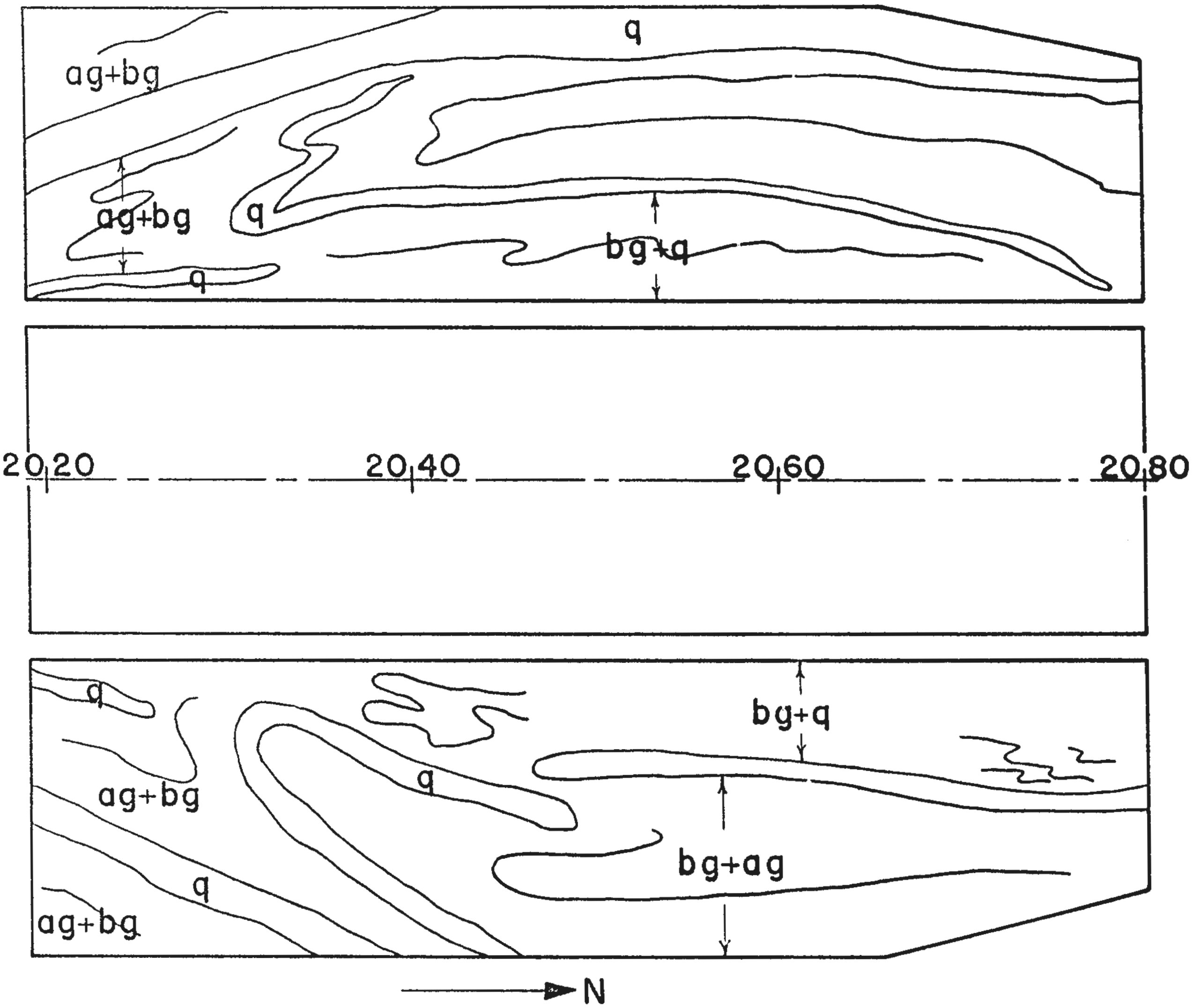
In the southern Piedmont, the salient factors governing the degree of weathering at any given location are: mineralogy, ground water movement, and structure (primary joint development and frequency).

The weathering in the Atlanta area has resulted in an overburden mantle typically 30 to 80 feet deep. The weathered zone consists of red or yellow, sandy to silty surface clays which have lost the relict rock structure, grading into micaceous sandy silts and silty sands, which generally retain the banding and configuration of the original rock from which they were derived. These soils in turn become gradually less altered, grading into partially weathered rock, until competent sound rock is encountered with depth.

Subsurface investigations performed for many construction projects along the crest and flanks of Peachtree Ridge (the location of the Atlanta Research Chamber) generally indicate that the bore to hard rock is much shorter than is typical elsewhere in the Atlanta area, and the rock recovered is normally fresh and sound. The top of the rock shows an irregular surface. However, the top of rock generally drops off very steeply along the flanks of Peachtree Ridge, so that the depth to relatively sound rock increases rapidly off the flanks. Rock types encountered in these investigations are typical for the Atlanta area, with biotite gneisses predominant. However, there are also seams of amphibole gneisses, quartzites, pegmatites and mica schists.

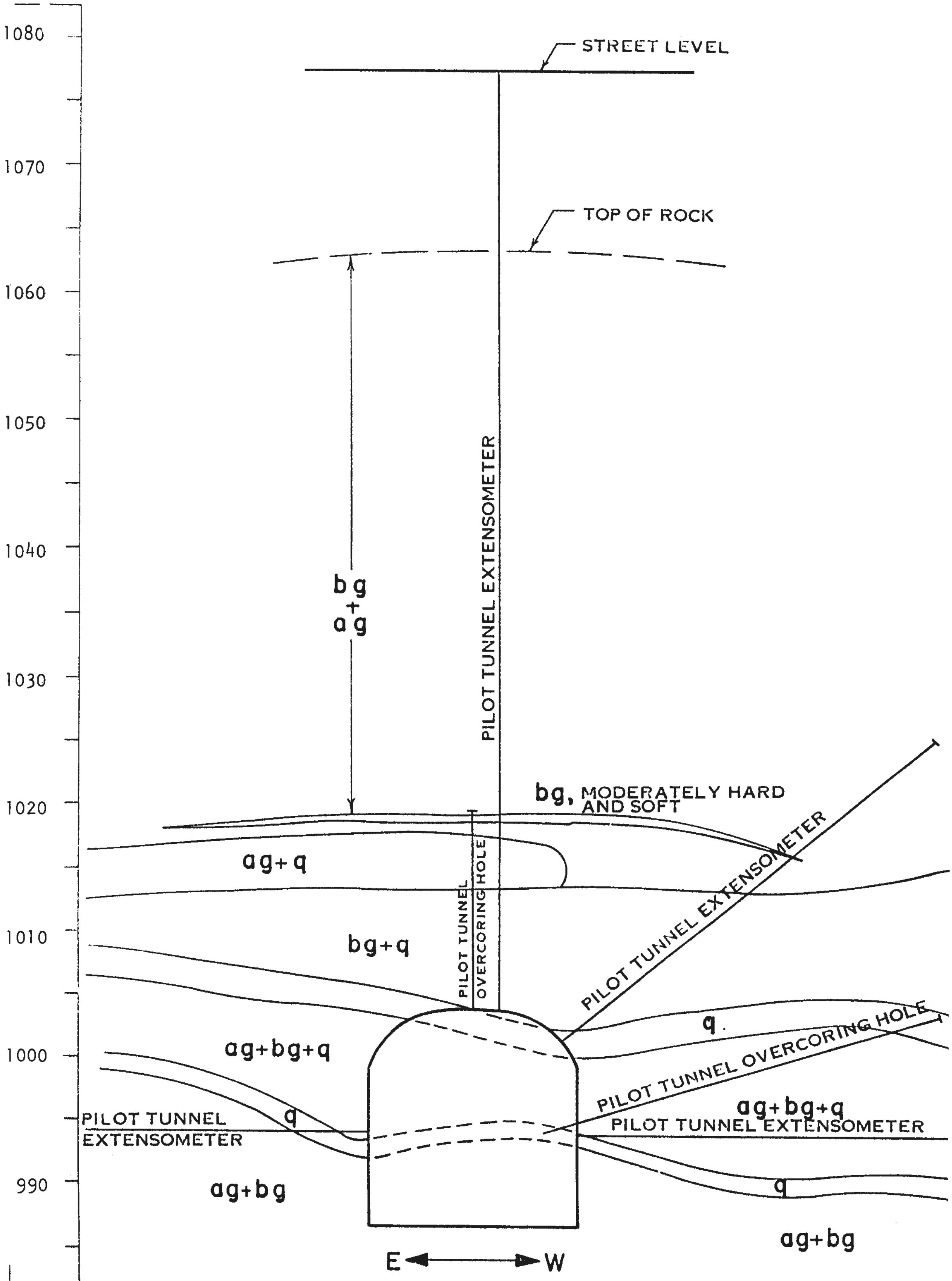
## II. ATLANTA RESEARCH CHAMBER GEOLOGY

The Atlanta Research Chamber geology has been summarized on a sidewall map and a geologic cross-section (see Figures III-2 and III-3). The Plan and Sidewall Map (Figure III-2) were developed from field observations. The Geologic Cross-Section




PLAN AND SIDEWALL MAPS OF THE RESEARCH CHAMBER

FIGURE III-2




GEOLOGIC CROSS-SECTION AT STATION NR 20 + 50

FIGURE III-3

(Figure III-3) was developed from examination of cores from extensometer and overcoring holes which were drilled during Pilot Tunnel construction.[3/] In general, the rock in the Research Chamber is strongly foliated biotite gneiss, quartzite and amphibole gneiss of excellent quality with no continuous joints or open partings, and no natural occurrences of water.

## A. Lithology

The rock types shown on geologic maps and cross-sections are based on visual observation. Petrographic analyses of some representative samples from the Atlanta Research Chamber area are included in a previous Law Engineering Testing Company report.[4/] The following rock types are included on the geologic maps and cross-sections.

Biotite gneiss (bg): Commonly dark gray, strongly foliated biotite-quartz-feldspar rock. It often has a quartz-feldspar augen or flaser fabric. Biotite gneiss is the most abundant rock type encountered in the main cavern area, and is probably derived from shales and siltstones.

Amphibole gneiss (ag): Gray-green to dark gray, primarily composed of amphibole and feldspar, and often containing high percentages of biotite. Very amphibole-rich layers are massive, not strongly foliated, and typically very contorted.

All of these varieties of amphibole gneiss are frequently found interlayered with biotite gneiss.

Quartzite (q): Very hard, quartz-rich, mica bearing rock, occurring in layers up to four feet thick. White quartzite

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3/ Law Engineering Testing Company, 1977, "Report of Geology and Instrumentation - Peachtree Center Station Pilot Tunnel, North Line, Metropolitan Atlanta Rapid Transit Authority".

4/ Law Engineering Testing Company, 1976, "Report of Subsurface Investigation, Final Design, DN-11/Tunneling Alternative, North Line, Metropolitan Atlanta Rapid Transit Authority".

is muscovite-bearing. Gray quartzite is biotite-bearing, and in places it appears to grade into quartz-rich biotite gneiss.

A plus sign (+) on the sidewall maps and section between two rock types indicates that the two rock types are interlayered.

a. Rock Structure

The foliation in the north end of the Atlanta Research Chamber is approximately horizontal, while the foliation in the south end of the Research Chamber dips approximately 20$^{\circ}$ to the south. There is considerable local variation in foliation orientation, as the rock is recumbently folded on an axis of approximately N15$^{\circ}$E horizontal (see Figure III-2). The folding can be easily seen by following light-colored quartzite layers, which serve as excellent "marker units".

There are no continuous joints in the Research Chamber. Some very minor discontinuous joints were noted in the sidewalls, but these have an insignificant effect on construction or engineering properties of the rock.

The crown has a tendency to break to foliation planes, particularly along the bottom of quartzite layers. Further discussions of rock structure effects on the research studies may be found in other chapters.

b. Rock Properties

In previous reports by Law Engineering Testing Company[5/], the physical properties of the rock in the area of the Atlanta Research Chamber have been presented. These properties were determined from both laboratory and in-situ testing, which included determinations of in-situ stresses, elastic moduli, unconfined compressive strengths, rock hardness values, and direct shear strengths of joints in rock.

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5/ Law Engineering Testing Company, 1976, op. cit., and 1977, op. cit.

These data and other data indicate that unconfined compressive strengths vary between $34.5 \times 10^3$ and $207 \times 10^3$ kN/m$^2$ (5 and 30 ksi) and average $103.5 \times 10^3$ kN/m$^2$ (15 ksi). Rock hardness values vary between 40 and 160, averaging 90 (total hardness). Laboratory tangent modulus values are between $20.7 \times 10^6$ and $99.4 \times 10^6$ kN/m$^2$ ($3.0 \times 10^6$ and $14.4 \times 10^6$ psi) and average $47 \times 10^6$ kN/m$^2$ ($5.9 \times 10^6$ psi). In-situ modulus values measured by flat jack tests and overcoring techniques range between $3.4 \times 10^6$ and $69 \times 10^6$ kN/m$^2$ ($0.5 \times 10^6$ and $10 \times 10^6$ psi) with the bulk of the values falling between $6.9 \times 10^6$ and $27.6 \times 10^6$ kN/m$^2$ ($1 \times 10^6$ and $4 \times 10^6$ psi). Laboratory modulus values are higher than field values by approximately a factor of 2 for overcoring tests and 1.5 for flat jack tests.

Interpretations of overcoring data from the Atlanta Research Chamber area suggest an in-situ stress field with the principal stress of approximately 1,000 psi in the north-south direction, parallel to the Pilot Tunnel. The minor and intermediate principle stresses of 100 to 200 psi are in the east-west and vertical directions, perpendicular to the Pilot Tunnel. This surprisingly high in-situ horizontal north-south compressive stress is along the strike of the regional folding throughout the Piedmont. The measured horizontal in-situ stress is several times the overburden stress in each instance, affects the displacements and stresses in underground openings, and is apparently due to tectonic forces which are at right angles to the forces which caused the Piedmont regional folding, which they postdate. These somewhat surprising results show the value of making direct measurements of in-situ stresses instead of simply assuming some apparently reasonable values.

1. Bentley, Robert D. and Neathery, Thornton L., 1970, "Geology of the Brevard Fault Zone and Related Rocks of the Inner Piedmont of Alabama", Alabama Geological Society, Eighth Annual Field Trip Guide Book, pp. 1-90.

2. Callahan, J. T. and Steward, J. W., 1959, "Geologic and Hydrologic Properties of Weathered Crystalline Rocks in the Piedmont of Georgia", Geological Society of America Bulletin, Vol. 70, No. 12, p. 1758.

3. Carter, R. W. and Herrick, S. M., 1951, "Water Resources of the Atlanta Metropolitan Area", U.S. Geological Survey, Circular 148.

4. Cording, E. J., Hendron, A. J., Hansmire, W. H., Mahar, J. W., MacPherson, H. H., Jones, R. A., O'Rourke, T. D., 1975, "Methods for Geotechnical Observations and Instrumentation in Tunneling", The National Science Foundation, Research Grant GI 33644X, Vol. 1, pp. 255-272.

5. Deere, Don U., 1968, "Geologic Considerations", In Rock Mechanics in Engineering Practice, ed. by Stagg and Zienkiewicz, John Wiley and Sons, N.Y. pp. 1-20.

6. Grant, W. H., 1968, "Weathering, Streams and Structures in the Central Piedmont of Georgia", Georgia Academy of Science Bulletin, Vol. 26, No. 2, p. 68.

7. Hatcher, R. D., Jr., 1972, "Developmental Model for the Southern Appalachians", Geological Society of America Bulletin, Vol. 83, pp. 2735-2760.

8. Herrick, S. M., 1950, "Groundwater in the Crystalline Rocks of Georgia", Georgia Academy of Science Bulletin, Vol. 8, No. 1, pp. 12-13.

9. Higgins, Michael W., 1966, "The Geology of the Brevard Lineament Near Atlanta, Georgia", Georgia Geological Survey, Bulletin No. 77.

10. King, Phillip B., 1950, "Tectonic Framework of Southeastern United States", Bulletin of the American Association of Petroleum Geologists, Vol. 14, No. 4.

11. LeGrand, H. E., 1967, "Groundwater of the Piedmont and Blue Ridge Provinces in the Southeastern States", U.S. Geological Survey, Circular 538.

# CHAPTER IV.

# DESIGN

## I. INTRODUCTION

During the last fifteen years, there has been a substantial growth in knowledge concerning the behavior of underground openings in rock. A corresponding body of literature has been produced. The pertinent studies have largely followed two divergent paths. The first relates to the increasing sophistication in analytical methods (in particular, Finite Element Methods) for predicting opening behavior, while the second relates to detailed instrumentation of various types of openings and the interpretation of opening behavior based upon the results of the data produced. Both methods have their advantages, as well as shortcomings, in particular situations. In this chapter an attempt is made to put the Finite Element Method (FEM) in a proper perspective by emphasizing its relative utility in an analysis/ design mode.

Following these sections, the detailed results of finite element modeling studies are presented to illustrate the use of this method to analyze the response of the rock mass at the Atlanta Research Chamber, south of the Peachtree Center Station. In this area, a Pilot Tunnel was constructed; it was anticipated that this Pilot Tunnel would be enlarged to become the Atlanta Research Chamber, and that then the twin Running Tunnels would be constructed beneath the Research Chamber. The Research Chamber was designed to have the same dimensions as the twin Running Tunnels. The finite element studies concentrate on the anticipated interaction of the Research Chamber and twin Running Tunnel openings. These design studies using 2-D FEM were conducted during June 1977, before in-situ testing was complete and before the openings were excavated. As discussed later, the actual sequence of construction was not that which was anticipated, which affected the study.

## II. ANALYSIS/DESIGN PHILOSOPHY

Before discussing the two dimensional Finite Element Method (2-D FEM), it is important to review the intent and meaning of such modeling methods in the context of analysis/design philosophy. There is a current tendency to over-analyze, over-measure and over-study a given project when what is really needed is sound, basic engineering with careful construction control and good excavation techniques.

Minor problems will always occur with underground openings because it will always be the minor geologic details which control the localized behavior, presuming no poor excavation practices. This reminds us that the first major aspect of design is a sound understanding of the site's geological environment. This term is used broadly to include the lithologic units and their spatial relationships, the significant discontinuities at the site, the mechanical behavior of the lithologic units, and the discontinuities, water, and in-situ stress states.

The second major aspect of design is consideration of geometric factors. In other words, the question is how an opening of a given size and shape will alter the geologic environment, specifically how the stress concentrations and relaxations may lead to compressive or tensile failures, and how the kinematics of significant discontinuity-bounded rock blocks may cause movement, potential fall-in or excessive overbreak.

The third major aspect of design is support and rock-support interaction. Of major consideration here is the intent behind the support system. Is the intent to minimize rock or wall working in order to hold discontinuity-bound rock blocks; to prevent rock surface deterioration and minimize squeezing or swelling; or to provide psychological comfort for the designer and the owner? These questions must be answered honestly to properly evaluate the support system.

The fourth design consideration is the effect of construction itself and of time upon the project. Will construction operations and time, in themselves, significantly alter the above three points and, if so, how?

The above four points are fundamental to a rational analysis/design philosophy and should determine the approach adopted for a given job. Thus, for a tunnel of moderate size and depth, with an opening of simple geometry, constructed in sound, massive rock units with no major discontinuities or water problems; when in-situ stresses are near isotropic, and where good excavation techniques have been employed then no major problems should be anticipated. Only simple computations from existing solutions would be warranted, and observations would be limited to a few check points. However, if any one of these factors varies significantly from the example given, detailed analytical and/or observational studies may be warranted.

## III. FINITE ELEMENT MODELING

Finite element methods have been developed to such a level of sophistication that, in principle, they can be applied to almost any type of underground opening problem. The literature contains many examples in which finite element methods have been used to analyze opening behavior after the opening was completed, and these examples all show reasonably good correlations with the measured quantities. Predictions are, however, rather rare, and this is precisely the area in which such models can be of most service in the design of an actual project. Good predictions can be made, if a sound rational model is developed.

A considerable amount of care must be exercised in establishing a sound analytical model, because it must be able to:

A. represent geometric and boundary conditions,
B. incorporate variable geologic strata and initial stress states,

C. simulate excavation operations following nearly any arbitrary sequence, and

D. include nonlinear and stress-dependent stress-deformation characteristics for the rock materials and the rock discontinuities, if warranted.

The minimum criteria for representing items A, B and C are discussed in detail by Kulhawy (1974)[1/], and an approach for item 4 is given by Kulhawy (1975).[2/]

The three main keys to establishing a sound model are the representation of the significant geologic units and their basic discontinuities; the initial stress states; and the mechanical behavior of the rock materials and discontinuities. The detailed geology and initial stresses often are not sufficiently known to generate a high level of confidence during the initial design phase, but expectable best and worst cases or bounds should be estimated during preliminary exploration. The mechanical properties of the materials can be determined or at least be bounded by laboratory and/or field testing. Since the above factors are usually not known precisely, though, it is often necessary to conduct analyses using models which represent the bounding conditions so that varying behavior within the expectable range can be evaluated. Published parametric studies may be useful in this context.

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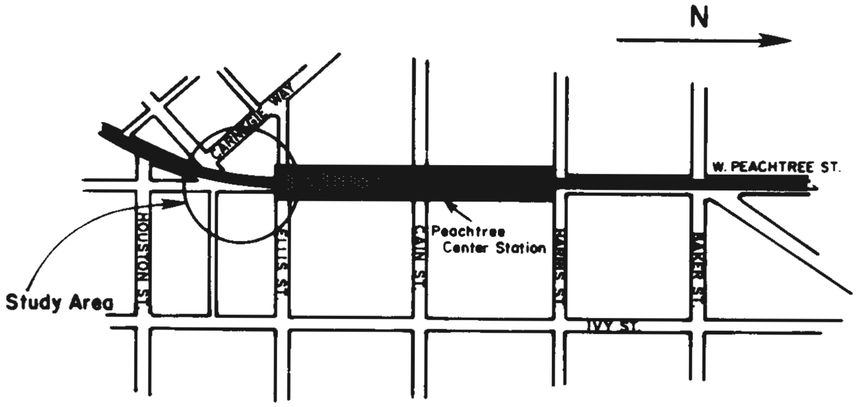
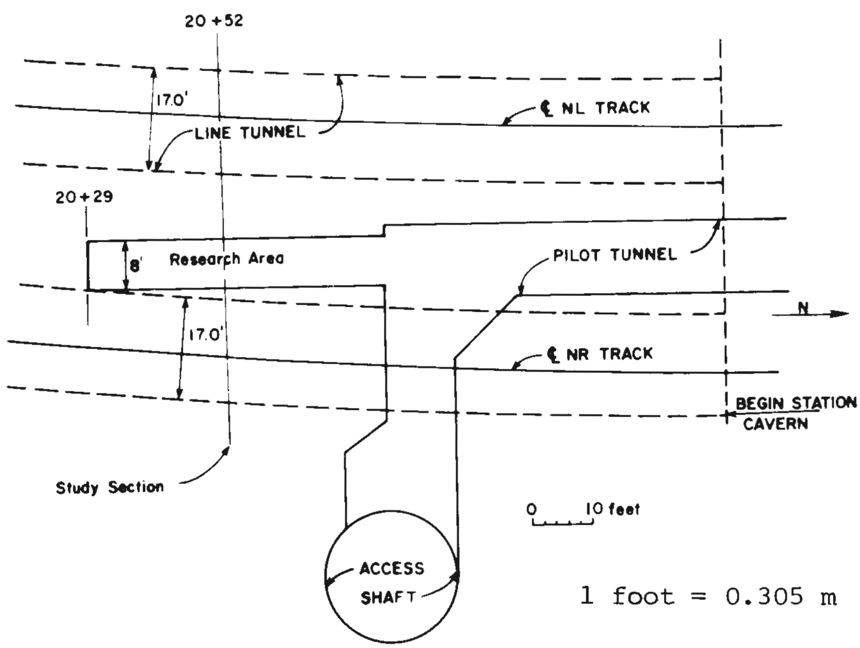
1/ Kulhawy, F. H., "Finite Element Modeling Criteria for Underground Openings in Rock", International Journal of Rock Mechanics and Mining Sciences, Vol. 11, No. 12, Dec. 1974., pp. 465-472.

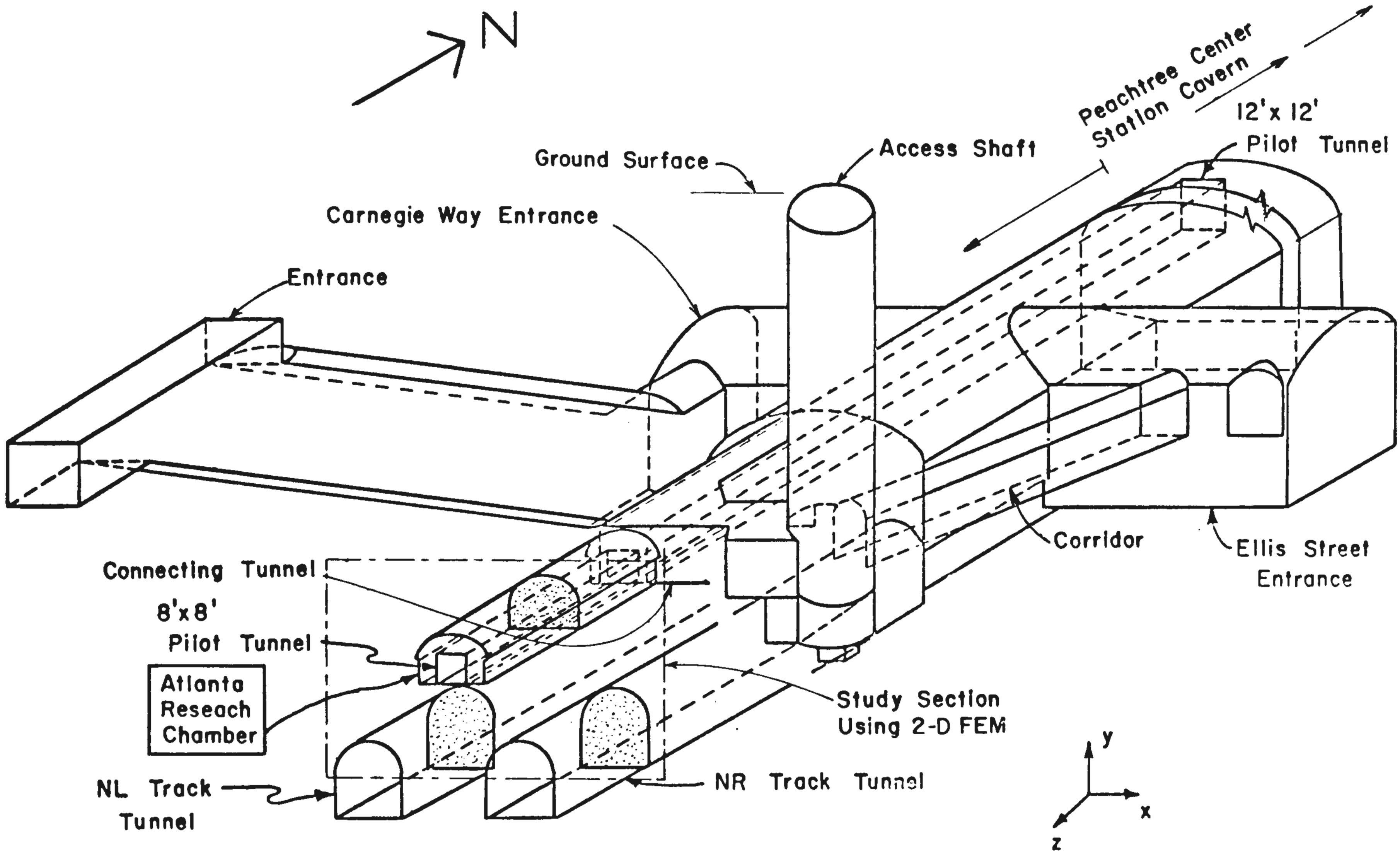
2/ Kulhawy, F. H., "Stress Deformation Properties of Rock and Rock Discontinuities", Engineering Geology, Vol. 9, No. 4, Dec. 1975, pp. 327-350.




Study Area, Peachtree Center Station, MARTA FIGURE IV-1




Plan View of Tunnels and Research Area FIGURE IV-2




OBLIQUE VIEW OF THE PEACHTREE CENTER STATION, SOUTHERN HALF

FIGURE IV-3

## IV. FEM MODEL FOR THE ATLANTA RESEARCH CHAMBER

As described above, the results of a finite element analysis are only as good as the model chosen to represent the problem. The following sub-sections describe the model used in the design study of the Atlanta Research Chamber.

### A. Geometric and Geologic Generalization

The problem under consideration was the behavior of the twin Running Tunnels in rock and the interaction of these tunnels with the Research Chamber to the south of the Peachtree Center Station. Figure IV-1 shows a general location plan of the station and the tunnels, while Figure IV-2 shows a more detailed plan view of the openings at the south end of the Peachtree Center Station. Figure IV-3 shows an oblique view of all of the openings in the Peachtree Center Station southern half. It can be seen that the opening geometry is extremely complex. However, from a standpoint of idealization, concentrating on the tunnels and Research Chamber, it can be seen that reasonable two-dimensional representation can be taken about mid-length along the Research Chamber. This "Study Section" is shown on Figures IV-2 and IV-3. There are a number of reasons this section was selected: (1) a plane strain approximation is reasonable for this section, (2) there is a reasonable concentration of test boring data in this vicinity, (3) detailed geologic mapping of the Pilot Tunnel had already been conducted so that reasonable inferences of geologic structure could be made at the approximate mid-length of the tunnel, and (4) instrumentation existed and additional instrumentation was planned in this area to monitor opening behavior.

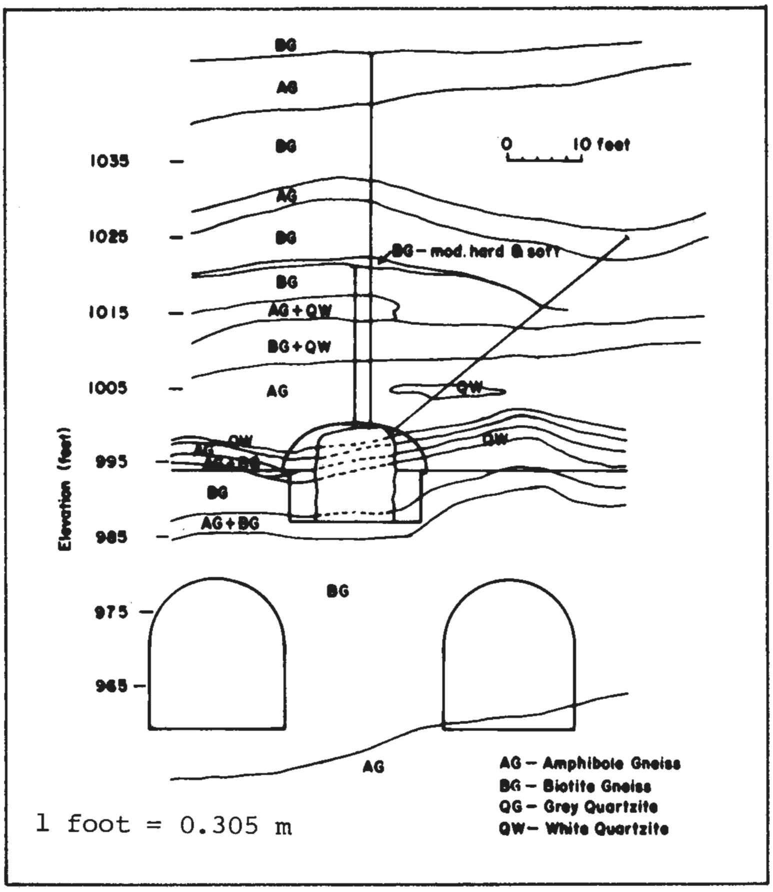
Upon establishing this section for analysis, the geologic data available at the time were plotted, in a east-west section looking south, as shown in Figure IV-4. Subsequent data obtained in the field resulted in the modification shown in Figure 5. These two figures, plus the correct opening locations shown in Figure IV-2, were the primary bases for establishing the geometric and geologic conditions.

B. Finite Element Mesh Design

The design of the finite element mesh is the first actual step in the modeling procedure because the physical system to be analyzed is established at this point. The geologic sections shown previously would be difficult and costly to model in great detail because many of the strata are 30 centimeters (one foot) or less in thickness, and the mesh would require literally thousands of elements. Based upon the experience gained in conducting many analyses in the past, and a reasonable grouping of geologic units with apparently similar mechanical properties (1, 2 or 3 as shown in Figure IV-5), the generalized geologic section shown in Figure IV-6 was chosen for analysis purposes. As can be seen, the overlying soil is taken into account. The rock mass is grouped into three units, each with distinctive mechanical properties, and the only apparent major discontinuity present in this section (a horizontal joint a few meters above the roof) is incorporated into the study. It is believed that this section is an appropriate generalization of the actual geologic conditions. It should be noted that geometric changes in the third dimension normal to this section apparently would be minimal because the geologic mapping in the Pilot Tunnel has shown only relatively small dips in the strata normal to the section.

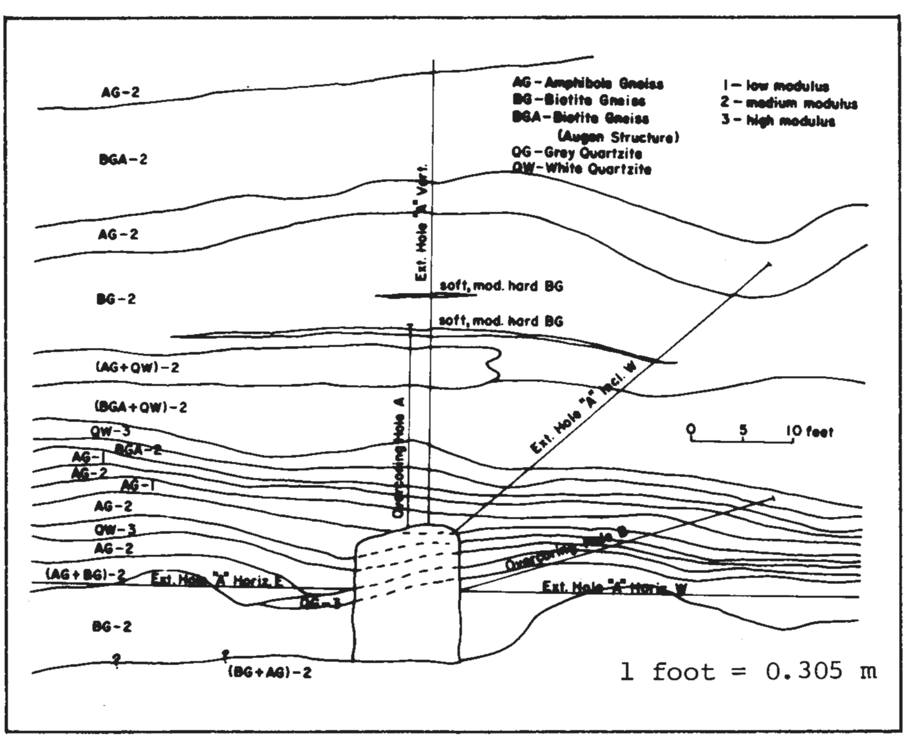
The boundary locations shown in Figure IV-6 were established using the criteria given by Kulhawy (1974). These criteria provide for an adequately flexible system in which the boundaries do not significantly affect the primary areas of interest around the openings.

The final mesh developed is shown in Figure IV-7 and includes 571 elements and 613 nodal points. This mesh evolved by accommodating the generalized geology, the opening geometries and the minimum element criteria given by Kulhawy (1974).



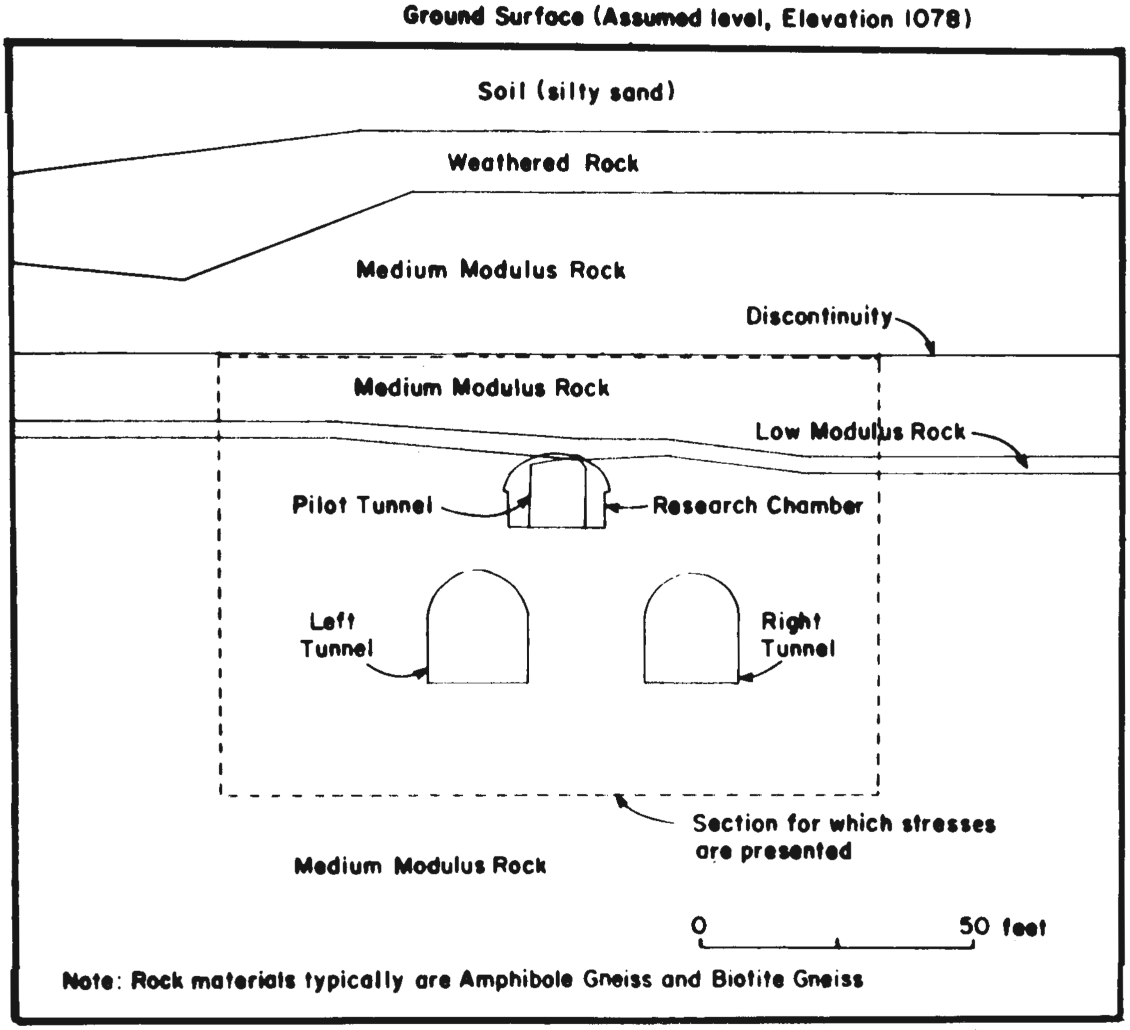

Initial Section at Station 20 + 52

FIGURE IV-4



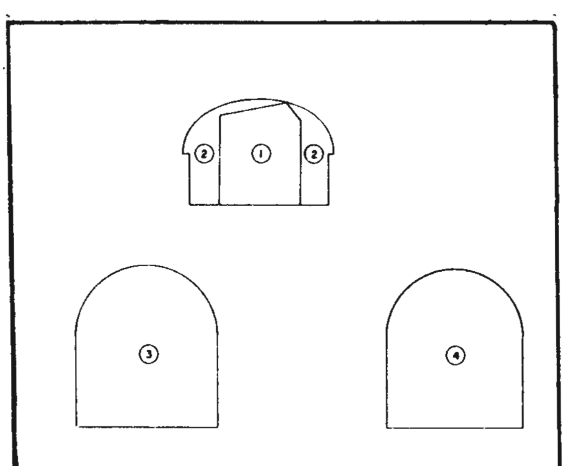

Revised Geologic Section

FIGURE IV-5



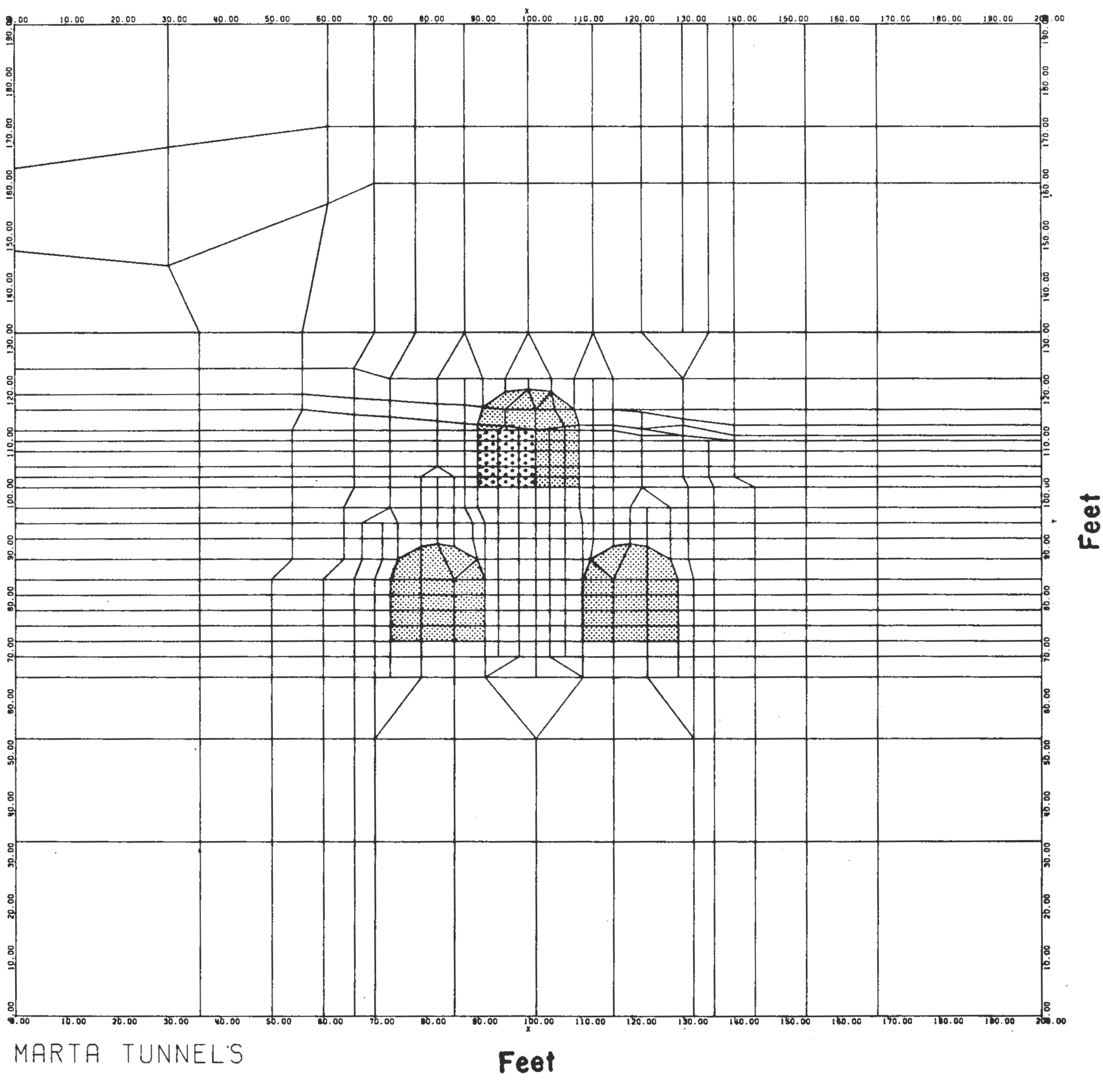

Generalized Section for Analysis

FIGURE IV-6




Sequence of Excavation Operations
Assumed in Initial Study

FIGURE IV-8




**613 Nodal Points** **571 Elements**

2-D FINITE ELEMENT MESH

FIGURE IV-7

## C. In-Situ Stress Conditions

To model any excavation operations in real rock masses, proper account must be made of the in-situ stress state. Commonly used elasticity solutions are inappropriate for this class of problems because direct gravity application to an elastic system will yield initial lateral stresses which are a direct function of Poisson's ratio and therefore can only vary from 0 to 1. Real rock masses usually exist at states of stress different from those directly predicted by use only of elasticity solutions. Accordingly, the initial stresses were input prior to excavation, and it was assumed that the rock mass was at rest and in equilibrium under the input stress state.

In-situ stress measurements were made in the Pilot Tunnel by Foundation Sciences, Inc., using overcoring and flatjack methods. The results obtained showed that one principal stress was vertical, while the other two were horizontal - one normal to and one parallel with the study section. The vertical stress was approximately equal to the overburden stress, the horizontal stress parallel with the study section was about one to two times the vertical stress, and the horizontal stress normal to the study section was perhaps as much as seven times the vertical stress. Based on these results, it was assumed that the vertical stress increased uniformly with depth at an average rate of about 25.4 $kN/m^2$ per meter (162 psf per foot); this gave an initial vertical stress of approximately 758 $kN/m^2$ (110 psi) at mid-elevation between the Research Chamber and the Running Tunnels. Since the horizontal stresses were variable, two solutions were conducted - the first with the horizontal and vertical stresses equal and the second with the horizontal stresses 2.5 times greater than the vertical stresses. With these values, probable field behavior would be bounded, unless unusual stresses beyond those measured were actually present.

## D. Assumed Excavation Operations

The assumed sequence of excavation operations for this project is given in Figure IV-8, which shows four essentially full-face operations - first the Pilot Tunnel, secondly the Research Chamber, thirdly the left Running Tunnel and lastly the right Running Tunnel. (The actual order of construction did not follow this assumed sequence.)

The excavation operations were modeled by evaluating the stress state along a proposed excavation surface, computing equivalent nodal point forces for these stresses, and then applying these nodal point forces in the opposite direction to yield a stress-free boundary. Specifics of this technique are given by Kulhawy (1974).

The excavation boundaries actually used in this study are shown on the mesh in Figure IV-7. During construction in the field, some modifications were made later to the geometry of the openings. These small changes did not significantly alter the results described herein.

E. Material Properties

As described earlier, there are several primary categories of rock materials in the study area. Grouping of the rock materials into three categories based on stress-strain behavior was based upon laboratory test results obtained by Foundation Sciences, Inc. and rock mechanics testing in the Pilot Tunnel. These groups and their respective properties are shown in Table IV-1.

For the properties of the soil overburden, the weathered rock, and the discontinuity above the Research Chamber, representative values were assumed, based on extensive literature surveys conducted on soil materials (Kulhawy, Duncan and Seed, 1969),[3/] on rock materials (Kulhawy, Duncan and Seed, 1969) and on rock materials and rock discontinuities (Kulhawy, 1975). The final values used in the analyses are given in Table IV-2.

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3/ Kulhawy, F. H., Duncan, J. M. and Seed, H. B., "Finite Element Analyses of Stresses and Movements in Embankments During Construction", Contract Report S-69-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Nov. 1969.

| | Unit Weight (pcf) | Intact Modulus | Reduced Modulus | Poisson's Ratio |
|---|---|---|---|---|
| Soil Overburden | 105 | 5,880 psi (847 ksf) | 5,880 psi (847 ksf) | 0.40 |
| Weathered Rock | 162 | $1 \times 10^6$ psi (144,000 ksf) | $0.15 \times 10^6$ psi (21,600 ksf) | 0.17 |
| Low Modulus Rock | 180 | $3 \times 10^6$ psi (432,000 ksf) | $0.45 \times 10^6$ psi (64,800 ksf) | 0.10 |
| Medium Modulus Rock | 180 | $5 \times 10^6$ psi (720,000 ksf) | $0.75 \times 10^6$ psi (108,000 ksf) | 0.17 |

Discontinuity — Normal Stiffness = 31,830 ksf/ft
Shear Stiffness = 6,366 ksf/ft

1 psi = 6.89 kN/$m^2$

Note: Modulus for weathered rock taken as $\frac{1}{5}$ times medium modulus.
High modulus rock not modeled because it only exists in three very thin layers.

Range of Rock Material Properties

TABLE IV-1

| | Unconfined Tangent Modulus (psi) | Poisson's Ratio | Uniaxial Compressive Strength (psi) | Tensile Strength (psi) // foliation | Tensile Strength (psi) ⊥ foliation |
|---|---|---|---|---|---|
| Low Modulus Rock (Amphibole Gneiss) | $3 \times 10^6$ (1-4) | 0.10 (0.06-0.15) | $8 \times 10^3$ (6-10) | 75-250 | $1.4 \times 10^3$ (1-2) |
| Medium Modulus Rock (Amphibole Gneiss, Biotite Gneiss) | $5 \times 10^6$ (4-8) | 0.17 (0.15-0.20) | $12 \times 10^3$ (8-16) | 75-250 | $1.4 \times 10^3$ (1-2) |
| High Modulus Rock (Grey and White Quartzites) | $10 \times 10^6$ (8-12) | 0.27 (0.25-0.30) | $20 \times 10^3$ (15-25) | ≈1000 | $1.8 \times 10^3$ (1.2-2.4) |

Range noted in parentheses.

1 psi = 6.89 kN/$m^2$

Properties Used in Analyses

TABLE IV-2

The rock properties given in these tables are essentially those obtained from laboratory studies on the rock materials, as opposed to properties of the in-situ rock mass. Deere et al. (1967)[4/] demonstrated that the in-situ rock mass modulus is less than the intact (laboratory) rock material modulus and that the ratio of these moduli is roughly correlated with the Rock Quality Designation (RQD). Kulhawy, 1978[5/] has recently developed an improved model which shows that the modulus ratio is related to the RQD through the ratio of the intact core modulus to the normal stiffness of the discontinuities. Using these models and considering a lower range of RQD of about 80 percent in the general study area, along with the properties given in Table IV-2, it can be shown that the in-situ rock mass modulus could be about 15 percent of the intact (laboratory) rock mass modulus. Accordingly, analyses were also conducted using the rock mass modulus reduced to 15 percent of the values given in Table IV-2. It is believed that these two sets of parameters would bound the actual field conditions. Although both the strength and Poisson's ratio values would also be affected by discontinuities, neither of these values were changed because: (1) no strength problems were anticipated, and (2) the effects of changes in Poisson's ratio are small.

It should also be noted that linear elastic behavior was assumed for the rock and soil materials in all of the analyses conducted. This assumption is considered to be reasonable because the extensive core testing by Foundation Sciencies, Inc. in 1976 showed the rock to behave in nearly an elastic manner. In addition,the small amount of soil overburden being modeled in the

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4/ Deere, D. H., Hendron, A. J., Jr., Patton, F. D. and Cording, E. J., "Design of Surface and Near-Surface Construction in Rock", Failure and Breakage of Rock (Proceedings, 8th Symposium on Rock Mechanics), AIME, 1967, pp. 237-302.

5/ Kulhawy, F. H., "Geomechanical Model for Rock Foundation Settlement", Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT2, Feb. 1978, pp. 211-227.

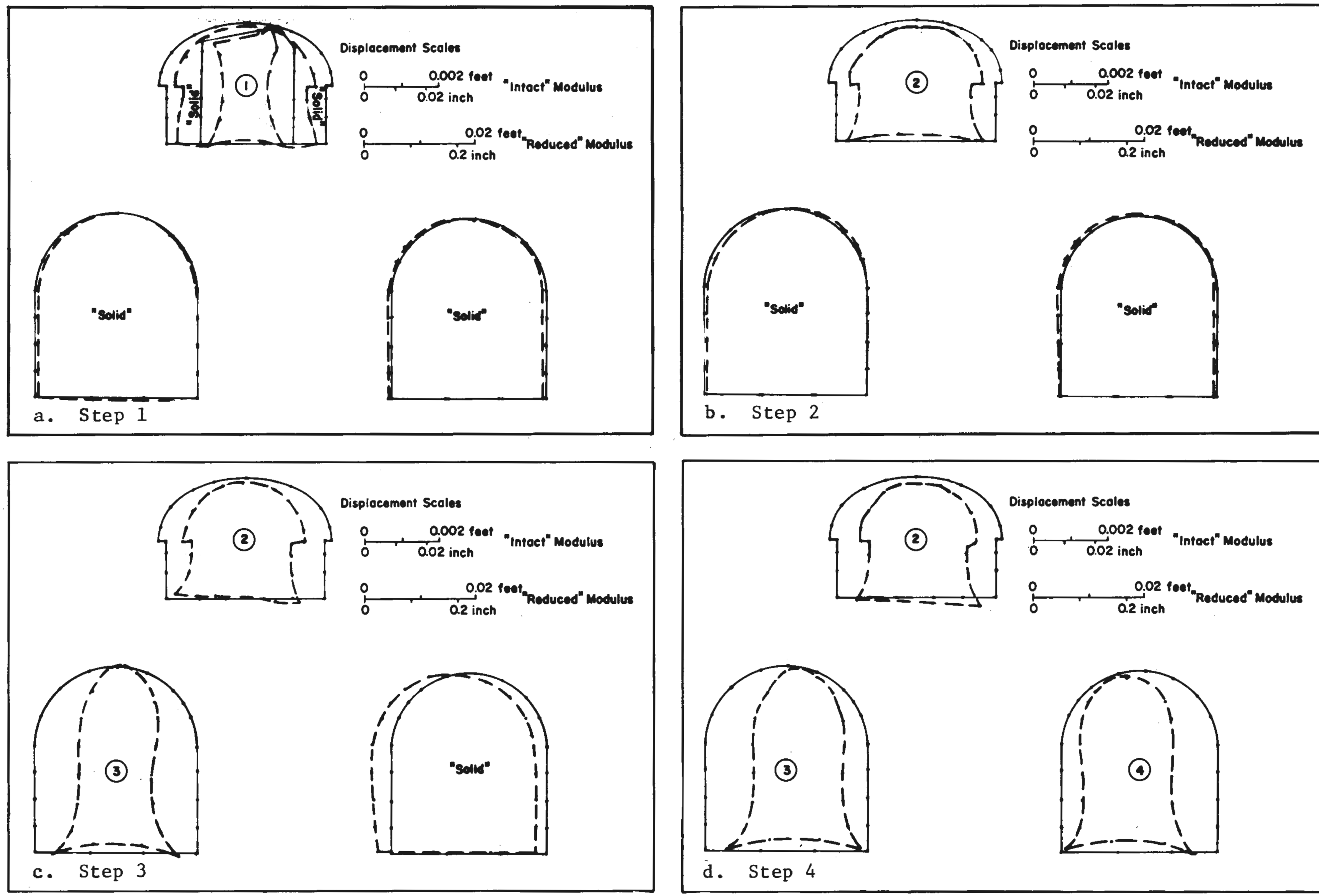



FIGURE IV-9

Computed Opening Displacements, K = 2.5

1 foot = 0.305 m, 1 in = 25.4 mm

analyses would not significantly alter the computed behavior of the openings if it were treated as either a linear or nonlinear material.

## V. RESULTS OF ANALYSES

A number of analyses were conducted to evaluate the rock mass response to the assumed sequential excavation of the openings. In each of these analyses, the initial vertical stresses increased linearly with depth at an average rate of 25.4 $kN/m^2$ per meter (162 psf per foot). The initial horizontal stresses were selected to be equal to the vertical stresses or 2.5 times the vertical stresses. Rock moduli were selected to represent the intact rock or they were reduced to establish a lower bound to the in-situ rock mass moduli. These ranges of stresses and moduli would bound the actual field behavior.

### A. Displacements

The displacements of the openings for the case with high horizontal stress are shown on Figure IV-9. In this figure, the correct relative locations and sizes of the openings are given for each excavation step. When an opening is excavated, it is referred to by number in the correct excavation sequence. The notation "solid" is used to indicate a proposed opening which has not yet been excavated. On this figure two displacement scales are given, corresponding to the "intact" modulus analyses and the "reduced" modulus analyses. These two sets of results are, for all practical purposes, directly related to each other by the ratio of the intact to the reduced moduli. The dots shown on the openings refer to the precise nodal point locations at which the displacements are computed. True vector movements are found between corresponding nodal points in the undeformed (solid line) and deformed (dashed line) geometries. The displacements shown are the cumulative displacements. Incremental values can be found by taking the differences in the displacements between different excavation steps.

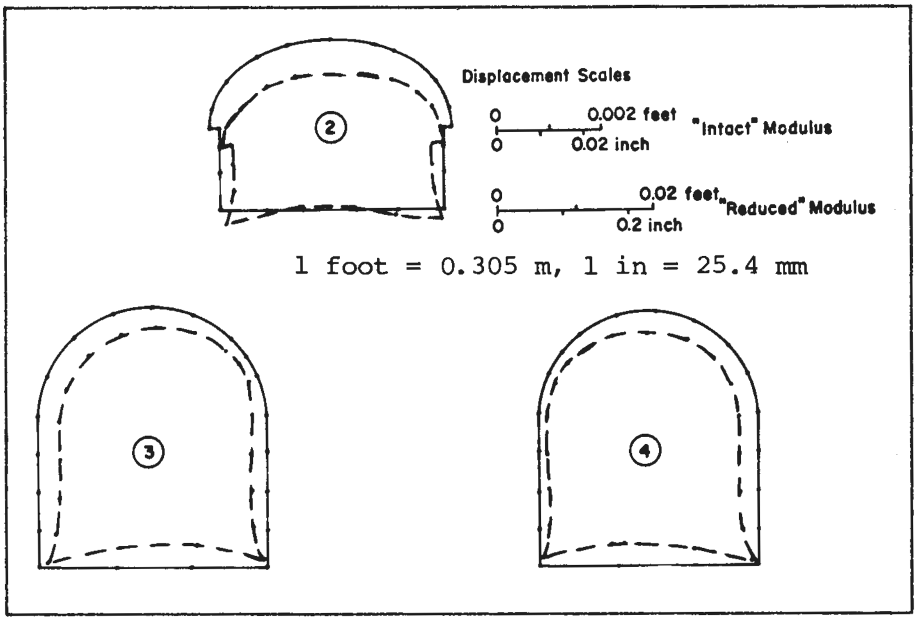
Figure IV-9 shows that during the assumed sequence of excavation for the Pilot Tunnel and Research Chamber, the wall displacements inward are larger than those at the crown and invert, as would be expected. The displacements in the Running Tunnel areas are small. As each tunnel is excavated, significant displacements occur in the Research Chamber and the other tunnel. Similar sequential movement patterns develop when the initial stresses are isotropic. The final displaced openings for the isotropic stress case are shown on Figure IV-10.

Extensometers were placed in the C120 Pilot Tunnel prior to any work done in the Atlanta Research Chamber. Results which became available after the analyses were conducted showed that the inward wall displacements of the Pilot Tunnel were on the order of 0.8 to 1.0 mm. Crown measurements were not reliable because of blast damage. Values computed from the finite element model show inward wall displacements of 0.05 mm or 0.5 mm for isotropic initial stresses, and 0.2 mm or 1.5 mm with the horizontal stress 2.5 times the vertical. These values imply that the lower modulus and higher lateral stress assumptions are more representative of the field conditions.

B. Stresses

The maximum ($\sigma_1$) and minimum ($\sigma_3$) principal stresses computed for the different analyses are shown on Figures IV-11 through IV-14 in contour form. These stress solutions are correct for any modulus assumptions, of course. These figures also show the openings in correct relative size and location, indicate the assumed excavation step for a particular opening, and show future openings with dashed lines.

The $\sigma_1$ and $\sigma_3$ stresses were selected for presentation because the maximum stress ($\sigma_1$) concentration can be compared to the rock strength to determine whether there is any potential for compression failure. The minimum ($\sigma_3$) stress concentration can be used to determine the maximum theoretical tensile zone in the rock. This tensile zone would represent the maximum size of the




1 foot = 0.305 m, 1 in = 25.4 mm

Computed Opening Displacements, K = 1, Step 4

FIGURE IV-10

potential "fall-out" zone which, in turn, would indicate the height of rock to be supported by the opening support system.

Figure IV-13 shows the $\sigma_1$ contours during excavation for the analysis with high horizontal stresses, while Figure IV-12 shows the final $\sigma_1$ contours in the isotropic case. These figures show higher crown and invert stresses and lower wall stresses for the higher initial stress state. The largest stress noted is a bit less than 4.8 $MN/m^2$ (about 700 psi or 100 ksf). Comparing this stress with the rock compressive strengths given in Table IV-1 indicates little chance of compressive rock failure, as long as the actual rock mass does not exhibit any significant jointing which would alter the stress pattern and reduce the in-situ strength. It should also be noted that the stresses predicted around the Research Chamber were lower than those around the twin Running Tunnels.

Figures IV-11 and IV-14 show the $\sigma_3$ contours for two analyses. These figures show significant stress reductions around the openings with the lowest crown stresses for the isotropic case. Tension zones are noted in all cases, but the only one considered to be significant is shown in Figure IV-11a in the crown of the Pilot Tunnel. A zone such as this indicates that there may be instability problems in the crown, if there are discontinuities present which would allow fall-out to occur. This zone also indicates that a meter or so of rock would be the potential loading on supports.

## F. POST-CONSTRUCTION ANALYSIS

In actual construction, the geometry of the Research Chamber and the Running Tunnels was slightly different from that assumed in the pre-design studies. A post-construction 2-D FEM analysis was performed by Tudor Engineering Company in June, 1979, incorporating almost the same mesh, but making the necessary slight changes in geometry. The actual construction sequence did not follow the sequence assumed in the original design study. As shown in Figure IV-15, Tudor's 2-D FEM study of the different excavation sequence indicated that <u>cumulative</u> final

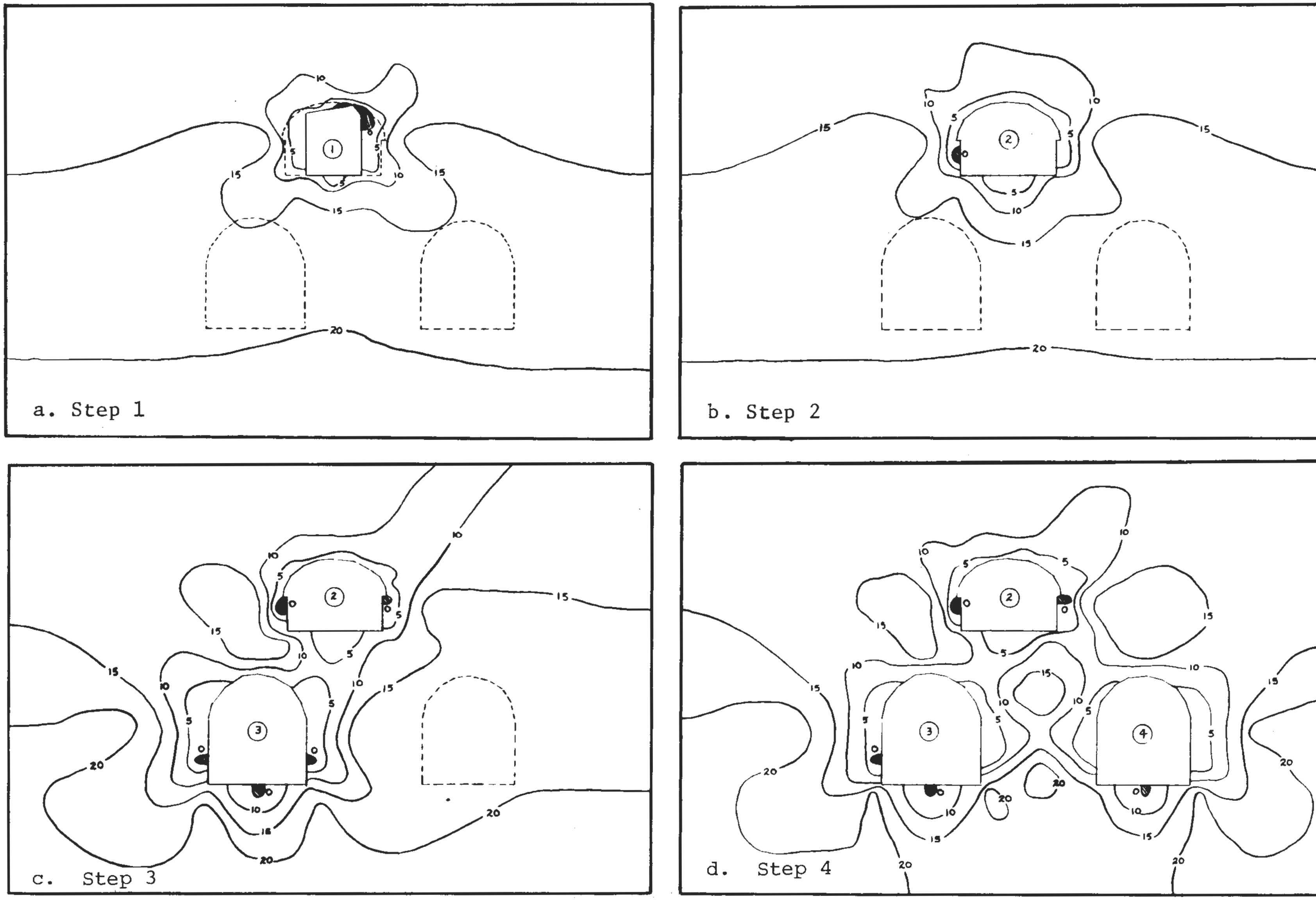
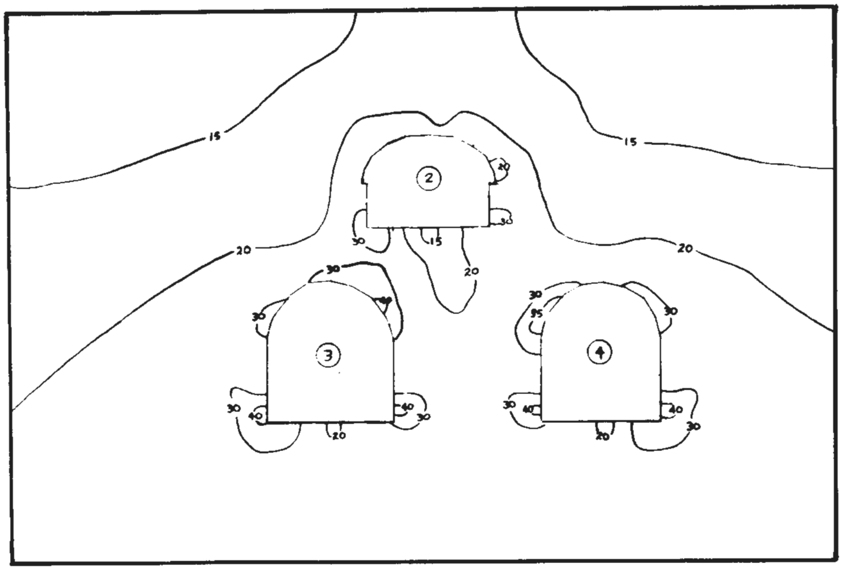



FIGURE IV-11

Computed $\sigma_3$ Contours (in ksf), K = 2.5

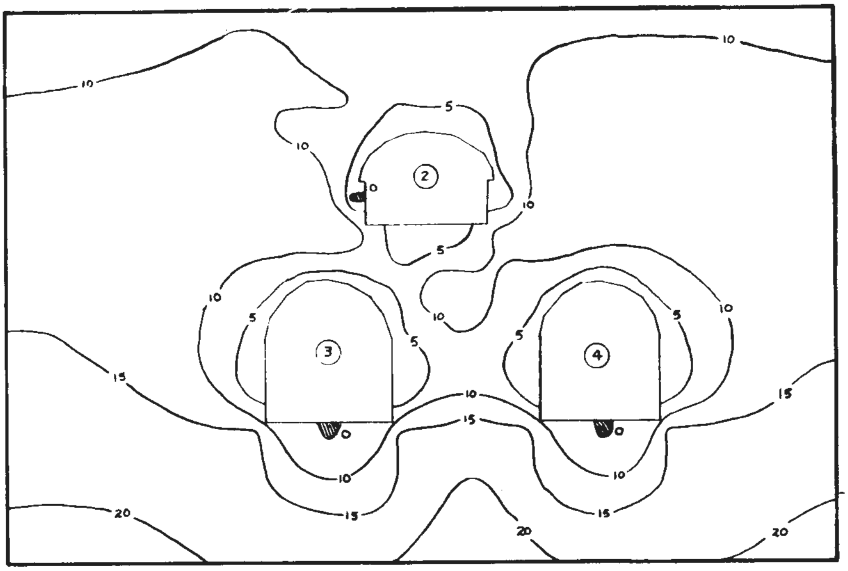
1 ksf = 47.8 kN/m$^2$




1 ksf = 47.8 kN/m$^2$

Computed $\sigma_1$ Contours (in ksf), K = 1, Step 4

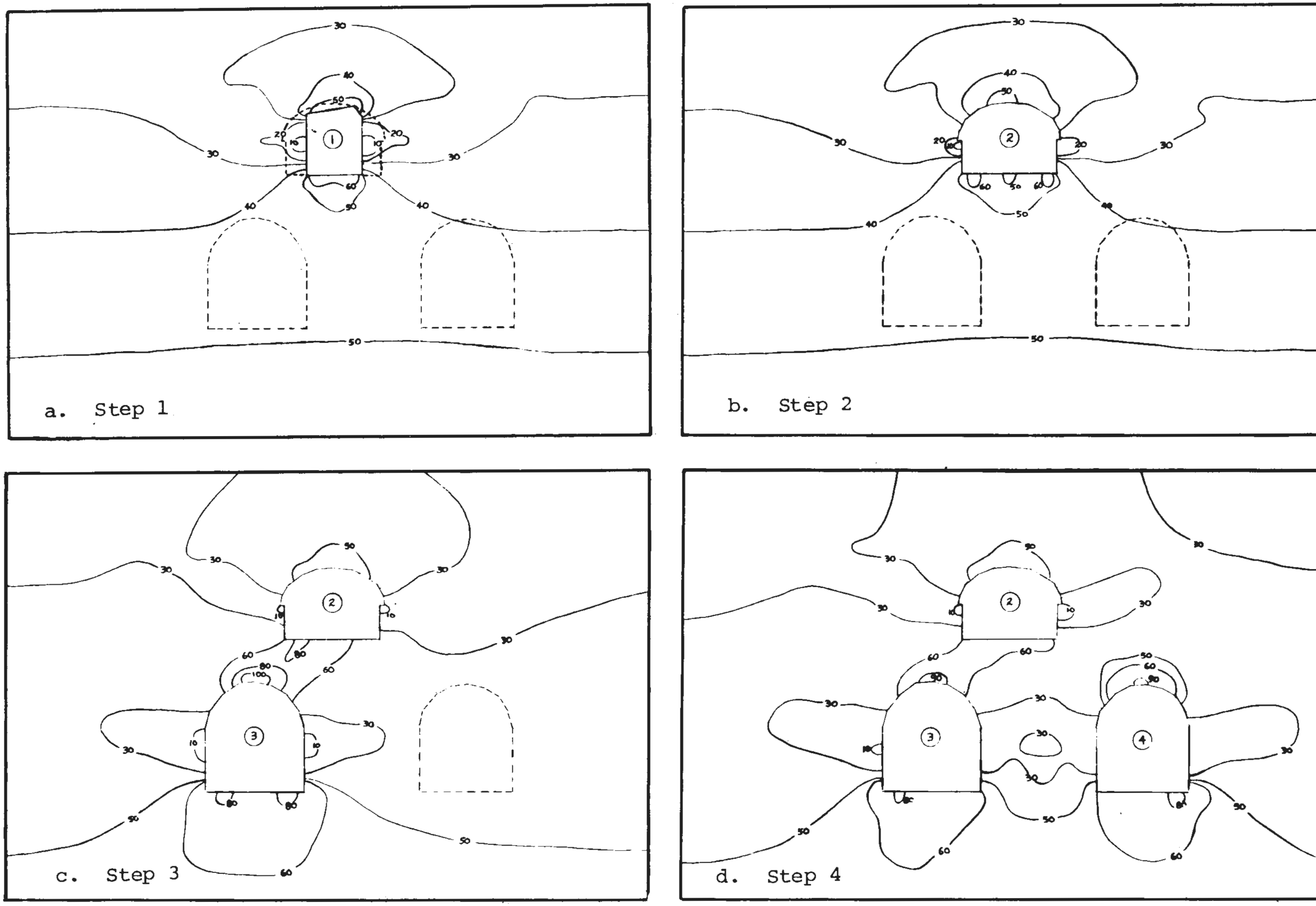
FIGURE IV-12




1 ksf = 47.8 kN/m$^2$

Computed $\sigma_3$ Contours (in ksf), K = 1, Step 4

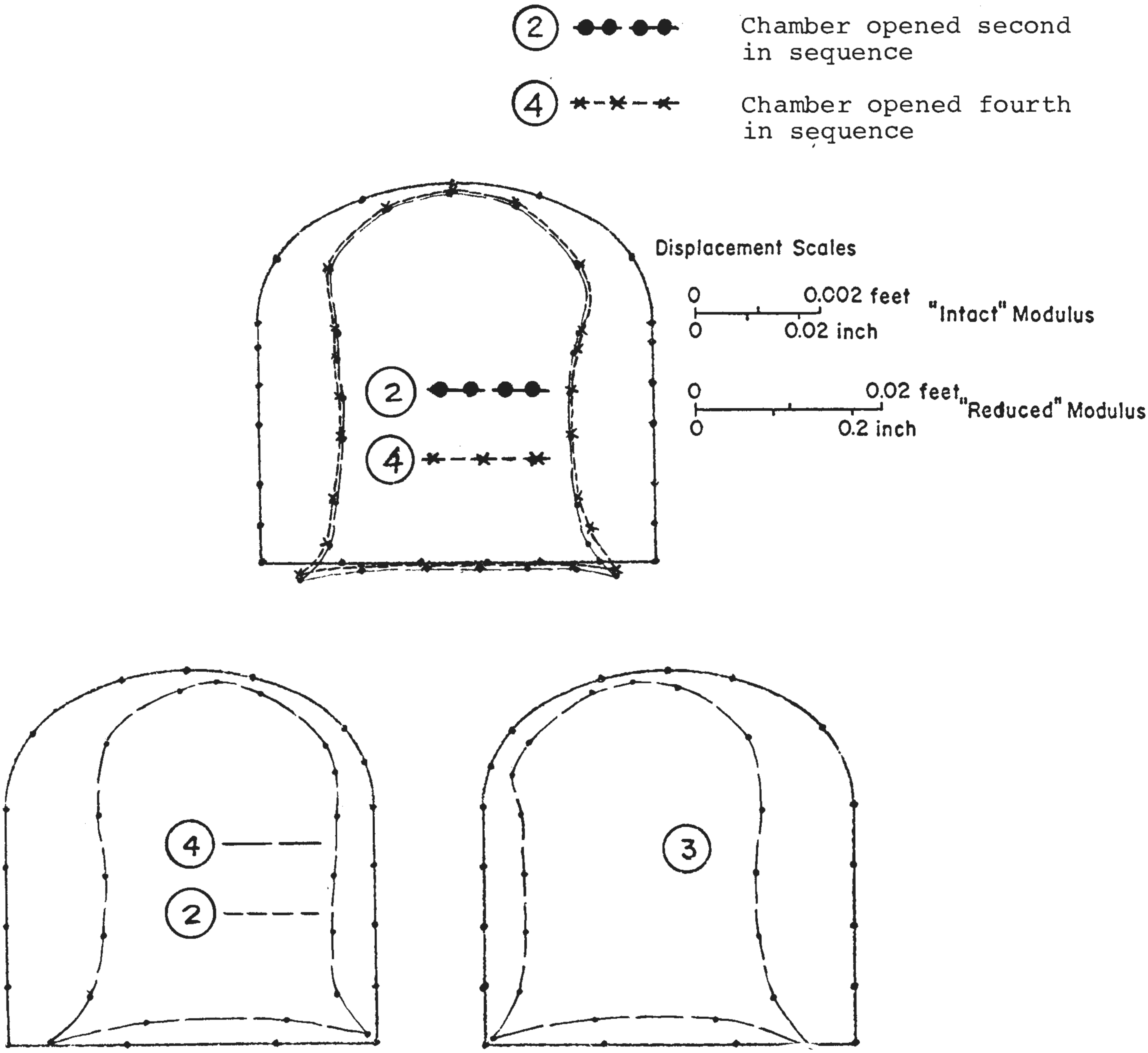
FIGURE IV-14




Computed $\sigma_1$ Contours (in ksf), K = 2.5

1 ksf = 47.8 kN/m$^2$

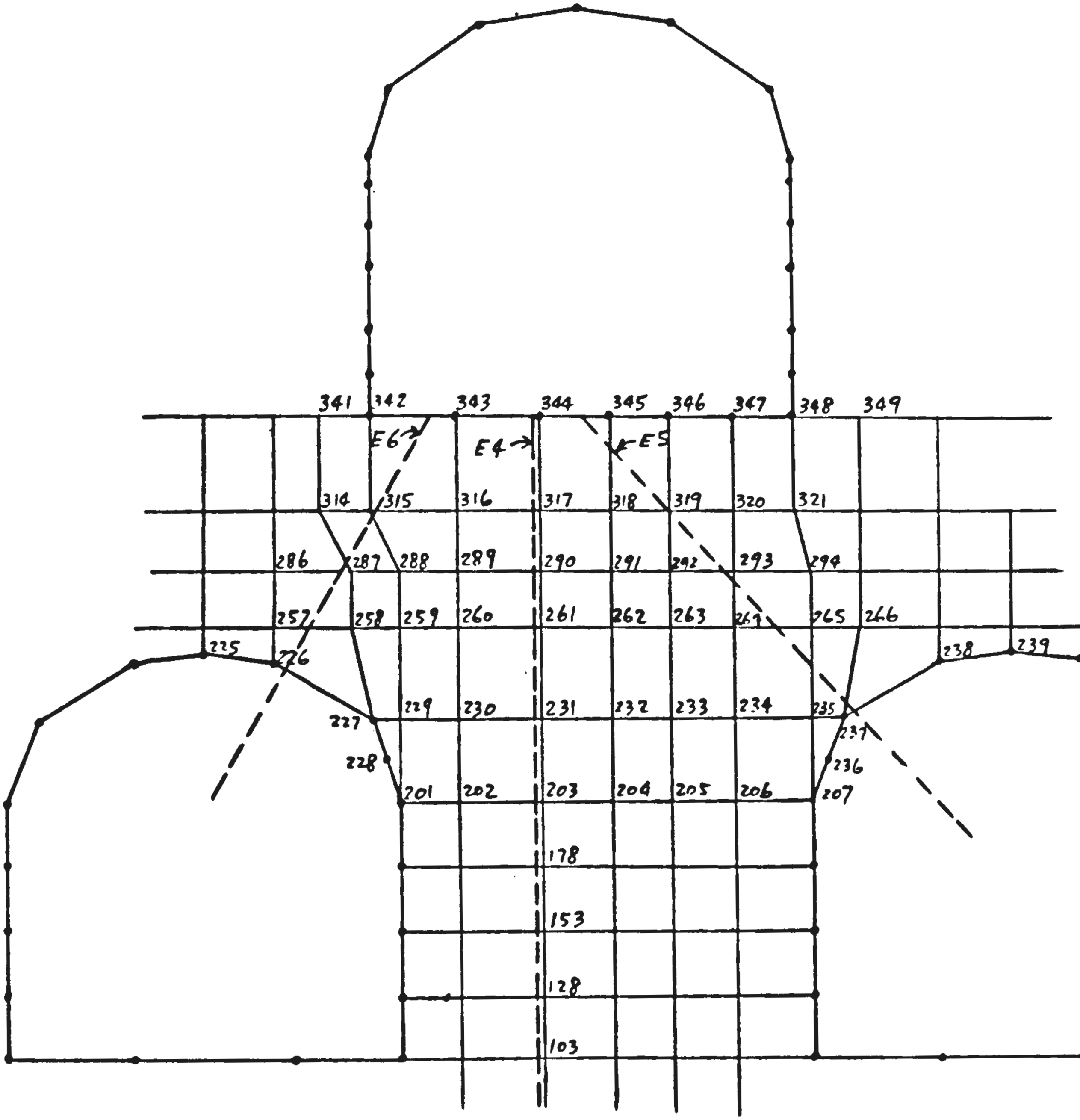
FIGURE IV-13




Opening Displacements, Atlanta Research Chamber, K = 2.5, Step 4.

Predicted displacements in the Atlanta Research Chamber do not vary significantly with changes in excavation sequence.

FIGURE IV-15




Finite Element Nodal Points and Extensometers

FIGURE IV-16

rock displacement would be only slightly changed from the predesign predictions. This is to be expected in a linear elastic study.

The small measured movements in the floor of the Atlanta Research Chamber were upward rather than downward as predicted in pre-design studies. At the suggestion of Dr. Kulhawy, Tudor's computer output was restudied in the light of the actual field installation of extensometers relative to the revised construction sequence (Figure IV-16). It was found that, due to the Contractor's changes in the sequence of construction, the extensometers were not in place in the field in time to measure displacements for Steps 1, 2 or 3. Figure IV-17 shows that the Tudor computer-predicted floor displacement from Step 3 to Step 4 only (and not the cumulative displacement of all steps one through four) is, in fact, upwards, as actually occurred. However, the upward floor movement measured exceeded the predicted amount.

The discrepancy between predicted and measured rock movements seems to have been due to the high horizontal stresses in the third (north to south) dimension, not accounted for in this 2-D FEM analysis. The 3-D FEM analysis of the Atlanta Research Chamber and the Peachtree Center Station Cavern did predict a small uplift in the Research Chamber floor, as discussed in the Monograph by Professor Einstein of MIT and his co-workers.

## G. SUMMARY AND CONCLUSIONS

The studies presented herein illustrate a logical predictive approach for the behavior of underground openings in rock.

The 2-D FEM analyses did accurately predict that the openings would behave well. The maximum stresses predicted were substantially less than the rock strengths and the computed tension zones were minor. The only computed tension zone of any significance was in the crown of the Pilot Tunnel, but only the potential for a minor fall-out and the possible need for

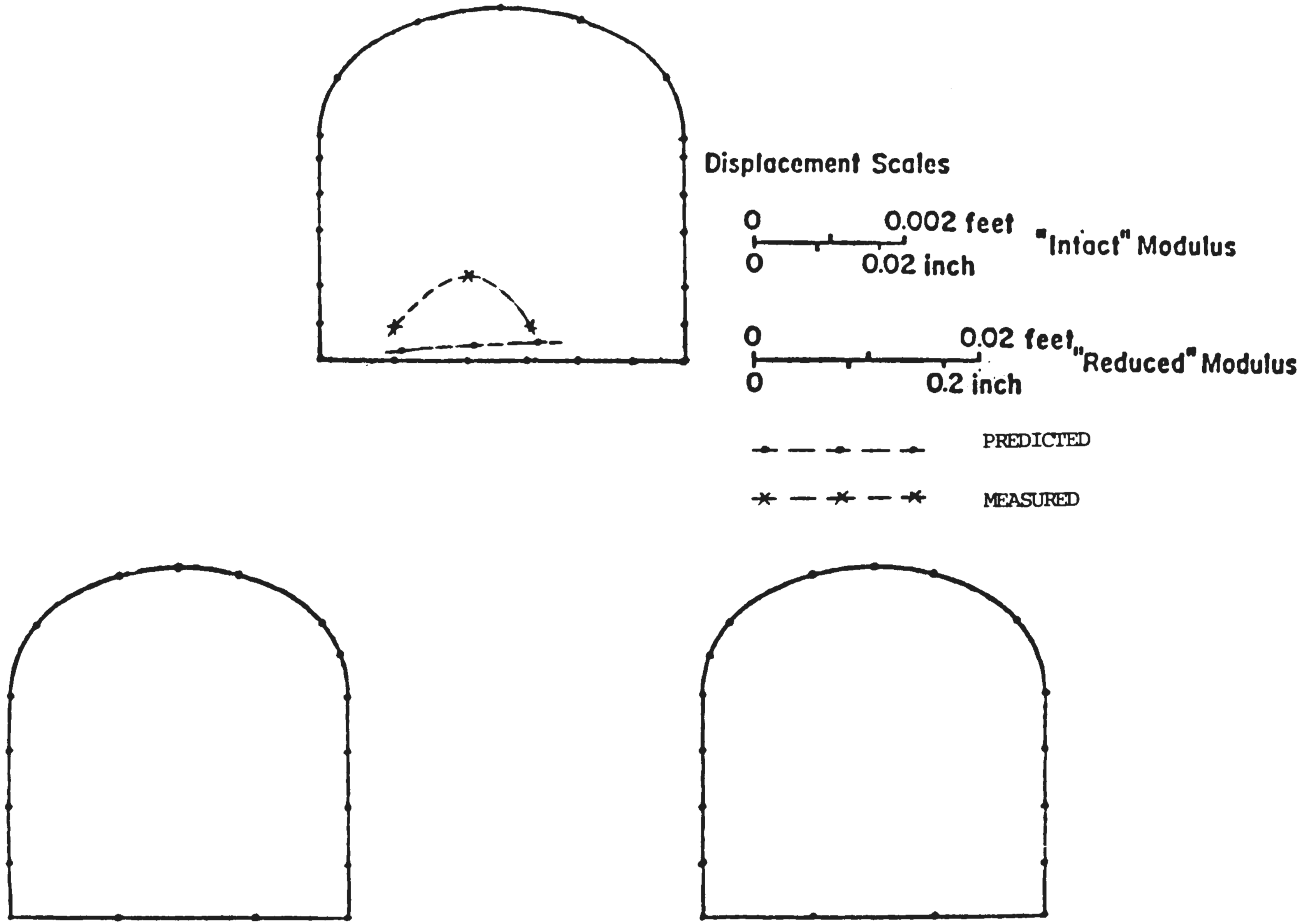



FIGURE IV-17

Opening Displacements, MARTA Chamber/Tunnel Analysis, K = 2.5, Step 3 to Step 4

POST-CONSTRUCTION TWO-DIMENSIONAL "FEM" ANALYSIS support of only a meter or so of rock was suggested. No fallouts occurred in the real Research Chamber, or in the Twin Running Tunnels below.

The computed displacements were small and indicate maximum total movements on the order of 2.5 mm. The displacements computed for the Pilot Tunnel agree well with extensometer measurements and indicate that the solutions with the reduced modulus values and the higher initial horizontal stresses are most consistent with the field behavior.

The openings have been completed through the Atlanta Research Chamber study area and no instability problems have developed.

In summary, the 2-D FEM study of the four openings (Pilot Tunnel, Research Chamber, and Twin Running Tunnels) confirmed that the design was reasonable and safe at all stages of excavation. The cost of the 2-D FEM study was modest (less than $10,000 - only 20%-25% of the cost of the 3D-FEM study by Einstein et al). The 2-D FEM study was made in about six weeks. Input data was unusually complete; few projects will have field and lab data from a Pilot Tunnel, in-situ stress measurements, and extensometers to use in FEM studies. Nevertheless, the design predictions, while of the correct order of magnitude, were not entirely confirmed in actual construction in every detail. This shows once again that analysis alone, even under the best conditions, cannot exactly predict real rock behavior, and that engineering judgment is the most important factor in underground design.

# CHAPTER V.

# INSTRUMENTATION

## A. INTRODUCTION

The Atlanta Research Chamber provided a unique opportunity to study both rock mass deformations and the behavior of full scale shotcrete support systems immediately adjacent to an actual construction environment, without the normally attendant problems of the research work causing interference with production operations. This opportunity existed because of the willingness of UMTA to finance the program and also because of the cooperation of MARTA in providing a convenient site.

## B. OBJECTIVES

The primary objective of the rock mass deformation monitoring program was to provide information about the actual "in situ" behavior of the rock mass which could be compared with the predicted deformations obtained from the Finite Element Method (FEM) design studies. In this way the validity of the FEM procedures could be assessed. It was not the intention to use this research program as a means of evaluating the performance of newly developed instrumentation. For this reason, all the equipment used has a well established performance history extending over several years.

At the time of the initial examination of the site, it was recognized that the rock mass deformations caused by the enlargement of the Research Chamber, and by the subsequent development of the underlying NL (left) and NR (right) Running Tunnels, would be very small as the initial in-situ stresses within the rock mass were low. It was further expected that the deformation induced by these stresses would be elastic and of small magnitude. The only significant deviation from the anticipated elastic response would be any irrecoverable deformation and fracturing in the immediate vicinity of blasting.

The study of the behavior of the rock mass around the Research Chamber also was seen as a valuable opportunity for comparing the actual deformations with those predicted by the pre-design 2-D FEM study (Chapter IV) and by the 3-D FEM study performed by Einstein et al. at MIT (included as a monograph in this report).

The rock mass behavior monitoring program was, therefore, designed to provide information about the small deformations developing within the rock mass around the Research Chamber as the various phases of excavation progressed. The deformation parameters studied were:

a. Radial deformations around the Research Chamber at distances of up to approximately three times the Chamber diameter during enlargement of the Research Chamber and during the development of the Running Tunnels.

b. Diametral deformation measurements inside the Research Chamber during enlargement and during the development of the Running Tunnels.

c. Longitudinal deformation measurements made inside the Research Chamber during development of the underlying tunnels and during the excavation of the adjacent Peachtree Center Station cavern.

d. Lateral displacement measurements within the rock mass made on horizontal planes below the Research Chamber.

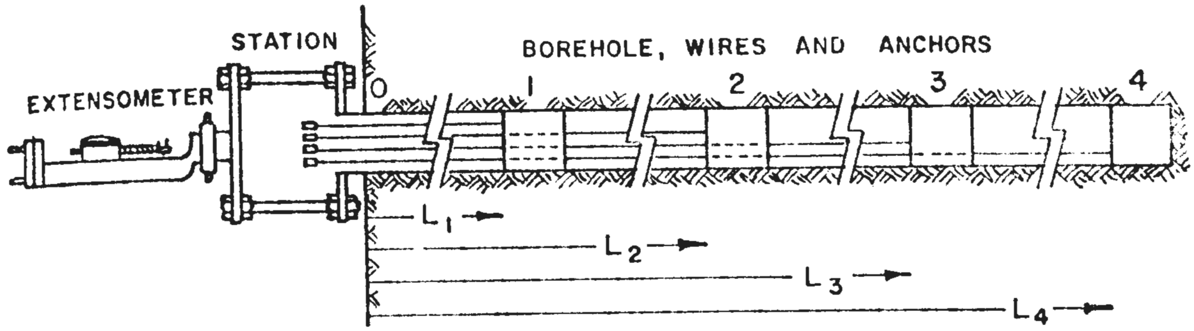
Parameter (a) was studied by means of multiple point borehole extensometers located in holes drilled from the original excavation. Convergence extensometers were to have been used to collect the data for (b) and (c), while borehole inclinometers were used to obtain the type (d) information.

## C. EQUIPMENT

### 1. Borehole Extensometer

The borehole extensometer equipment used to obtain radial deformation information consisted of six-position assemblies employing hydraulically set anchors, stainless steel measuring wires, a fixed collar station which comprised the extensometer mounting point, and a detachable constant tension extensometer. Units of this specific design have been used for several years in many mining and construction operations and, given reasonable installation and operating conditions, they provide the most cost effective method of obtaining precise rock deformation measurements. Equally precise extensometers which employ small diameter rods instead of tensioned wires are available, but these units generally have a somewhat higher cost and are slower to install.

A schematic view of a typical installation is shown in Figure V-1. In the majority of cases in the Research Chamber a total of six anchor points were used in each hole. The anchors consist in part of two steel wedges each with a taper on one edge. The width of these wedges is preselected to match the diameter of the hole into which the extensometer is being placed. The tapered faces of the wedged units fit into milled slots of a cylindrical unit with a flanged end section. The measuring wires are attached to the appropriate anchors by means of tapered locking pins driven into one of the alignment holes in the anchor body. The wires are fed back to the collar of the hole past the bodies of the anchors using the six passage holes provided for the wires in the flange, and the cross section of the anchor bodies is sufficiently small as not to interfere with previously installed wires. The cylindrical anchor unit is equipped with a diametrical hole for a shear pin and the installing tool is equipped with a slotted end cylinder to mate with the anchor beyond the shear pin hole. Holes in the cylindrical walls of the installing tool permit the anchor to be locked in the mounting tool with the shear pin. In use, when hydraulic pressure is applied to the installing tool, the wedges are moved past the cylindrical section of the anchoring unit until they come into contact with the walls of the borehole. An increasing hydraulic pressure is applied to the hydraulic ram until the shear pin breaks and leaves the anchor locked in the hole.




| | Movement | Strain |
|---|---|---|
| Wire $L_1$ | $\Delta L_1$ | $\epsilon_{0,1} = \dfrac{\Delta L_1}{L_1}$ |
| Wire $L_2$ | $\Delta L_2$ | $\epsilon_{1,2} = \dfrac{\Delta L_2 - \Delta L_1}{L_2 - L_1}$ |
| Wire $L_3$ | $\Delta L_3$ | $\epsilon_{2,3} = \dfrac{\Delta L_3 - \Delta L_2}{L_3 - L_2}$ |
| Wire $L_4$ | $\Delta L_4$ | $\epsilon_{3,4} = \dfrac{\Delta L_4 - \Delta L_3}{L_4 - L_3}$ |

Diagram Illustrating an Installed Borehole Extensometer.

FIGURE V-1

The actual measuring instrument used to read the test cavern borehole extensometers is a Potts Mk II Constant Tension Extensometer. This is a portable mechanical device which is attached to the collar station at the time of taking readings by means of a capstan nut which is screwed onto a threaded boss in the center of the collar station. Predetermined tensions are applied to the measuring wires and the extensometer reading is taken by means of a drum micrometer and a linear scale. Under reasonable conditions this type of instrument will repeat to better than ± 0.001 in. (.025 mm).

2. Convergence Measuring System

The convergence monitoring system used in the Research Chamber consists of a number of mounting stations located at various points on the Chamber walls, several high yield strength stainless steel reference tapes, and a Potts Mk IA Reversed Constant Tension Extensometer. The extensometer is of the same basic design as the one used for the borehole measurements. Measurements are always made at the specified tensions and, after temperature corrections have been applied, the measuring element can be assumed to be of a constant length.

3. Borehole Inclinometers

Borehole inclinometers are employed relatively infrequently in underground monitoring programs, but in the case of the Atlanta Research Chamber, the use of an inclinometer in two boreholes extending some 15 meters (50 ft.) below the floor of the Chamber offered the possibility of monitoring deformations occurring in horizontal planes below the Chamber.

A borehole inclinometer is a movable probe which is traversed up and down a borehole line within a plastic or aluminum casing which contains four internal longitudinal grooves. The inclinometer probe consists of a wheel mounted unit about 0.6 meters (2 ft.) long which contains a pair of attitude sensing accelerometers. The entire assembly is moved up and down the borehole by means of a combination readout and hoisting cable. The probe is typically moved in 0.6 meter increments, and after each move the attitude in two mutually perpendicular directions is monitored. In this way it is possible to determine the profiles of the borehole, typically in north to south and east to west directions, and by comparing the profile at appropriate time intervals, the lateral displacements occurring at any elevation can be obtained.

The typical specified system repeatability for equipment of this type in a vertical installation is about $\pm$ .010 ft. per 100 ft. of casing or approximately $\pm$ 20 seconds of arc. Taking into account practical problems, the overall accuracy should be about $\pm$ .02 ft. per 100 ft.

4. Other Equipment

Resistance strain gages, vibrating wire strain gages, and brittle coatings of a special lacquer were planned for use in measuring shotcrete behavior. Unfortunately, time and budget constraints caused these to be omitted from the actual field program. Similarly, the rock bolt testing program initially planned was omitted from the actual field program.

## The Field Program

The details of the instrumentation program designed for the Atlanta Research Chamber are shown in Figures V-2 through V-5. As discussed below, not all of the planned work was actually done.

The borehole extensometers were located on transverse sections at stations 20 + 32, 20 + 47.5 and 20 + 63.5, corresponding to the midpoint of the Research Chamber and the two quarter points. This configuration was adopted in order to minimize the potential boundary effects caused by the end of the Research Chamber at station 20 + 17 and the change in section at station 20 + 79. The vertical coverage of the extensometers was approximately 30 meters (100 ft.), extending from about elevation 940 ft. to elevation 1041 ft. In addition to the vertical extensometers, four inclined extensometers were located at station 20

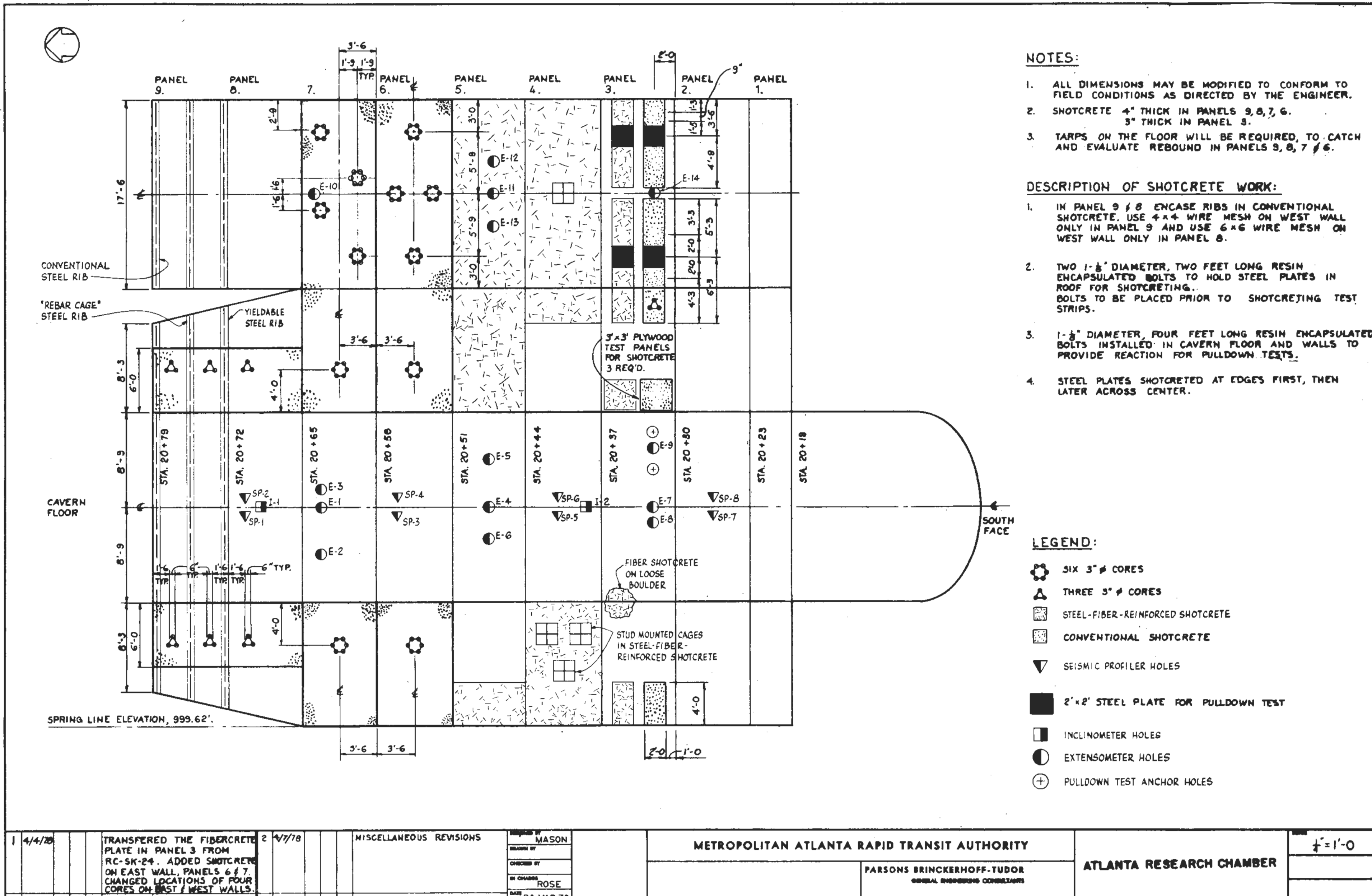
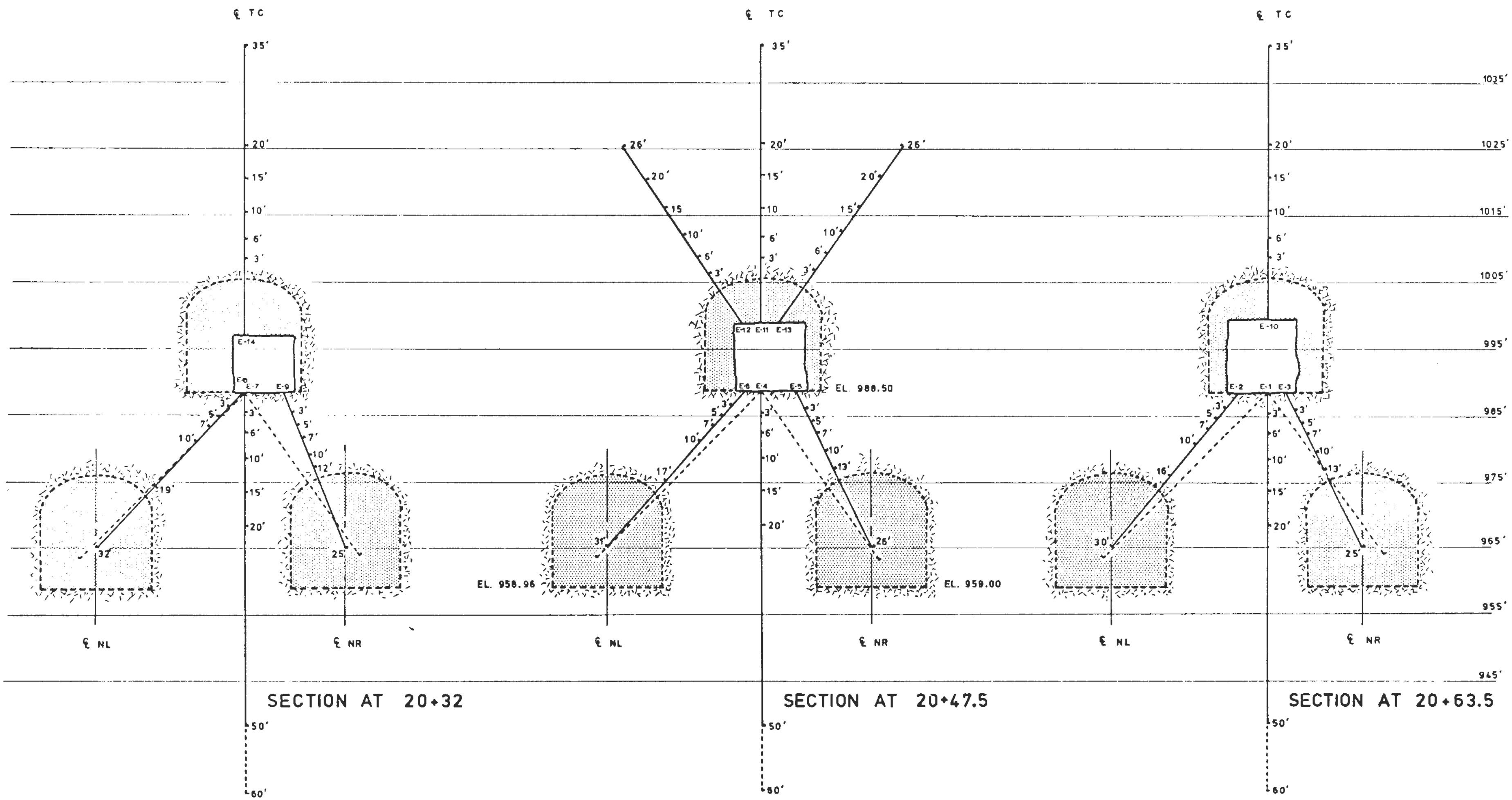



FIGURE V-2 Research Chamber Instrumentation Layout

FIGURE V-3




NOTES:

(1) INCLINOMETER AND SEISMIC PROFILE HOLES ARE SHOWN BY BROKEN LINES AND ARE PROJECTED OFF SECTION. FOR TRUE SECTIONS SEE DRAWING NO. WJEC-M-1.

(2) HOLE INCLINATIONS AND DEPTHS:
SP-1, SP-3, SP-5, AND SP-7, ARE 30' LONG, DIPPING @ 55° E
SP-2, SP-4, SP-6, AND SP-8, ARE 35' LONG, DIPPING @ 45° W
E-2, DIPS @ 50° W, 31' LONG. E-3, DIPS @ 65° E, 26' LONG
E-5, DIPS @ 65° E, 27' LONG. E-6, DIPS @ 50° W, 32' LONG
E-12, UP @ 55° W, 27' LONG. E-13, UP @ 55° E, 27' LONG
E-8, DIPS @ 45° W, 33' LONG. E-9, DIPS @ 70° E, 26' LONG

(3) FOR HOLE DIAMETERS AND DRILLING INSTRUCTIONS REFER TO NOTES ON DRAWING NOS. WJEC-M-1 AND WJEC- M-4

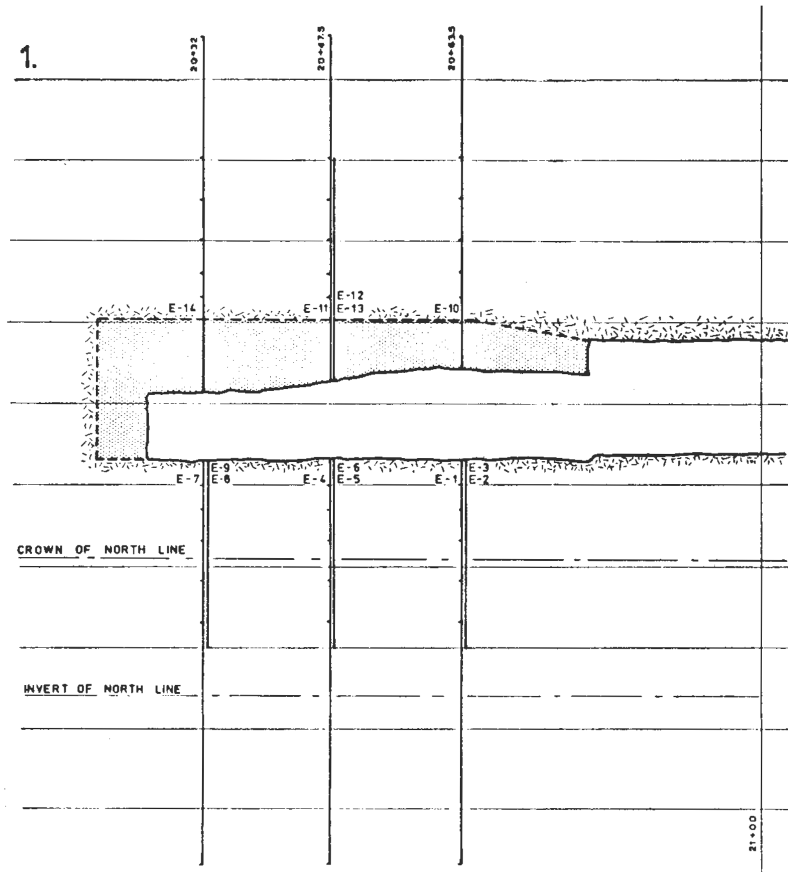
(4) ALL EXTENSOMETER ANCHOR LOCATIONS ARE IN FEET MEASURED FROM THE ROCK SURFACE

(5) FOR DETAILS OF PREPARATION OF COLLAR STATIONS SEE DRAWING NO. WJEC-M-4

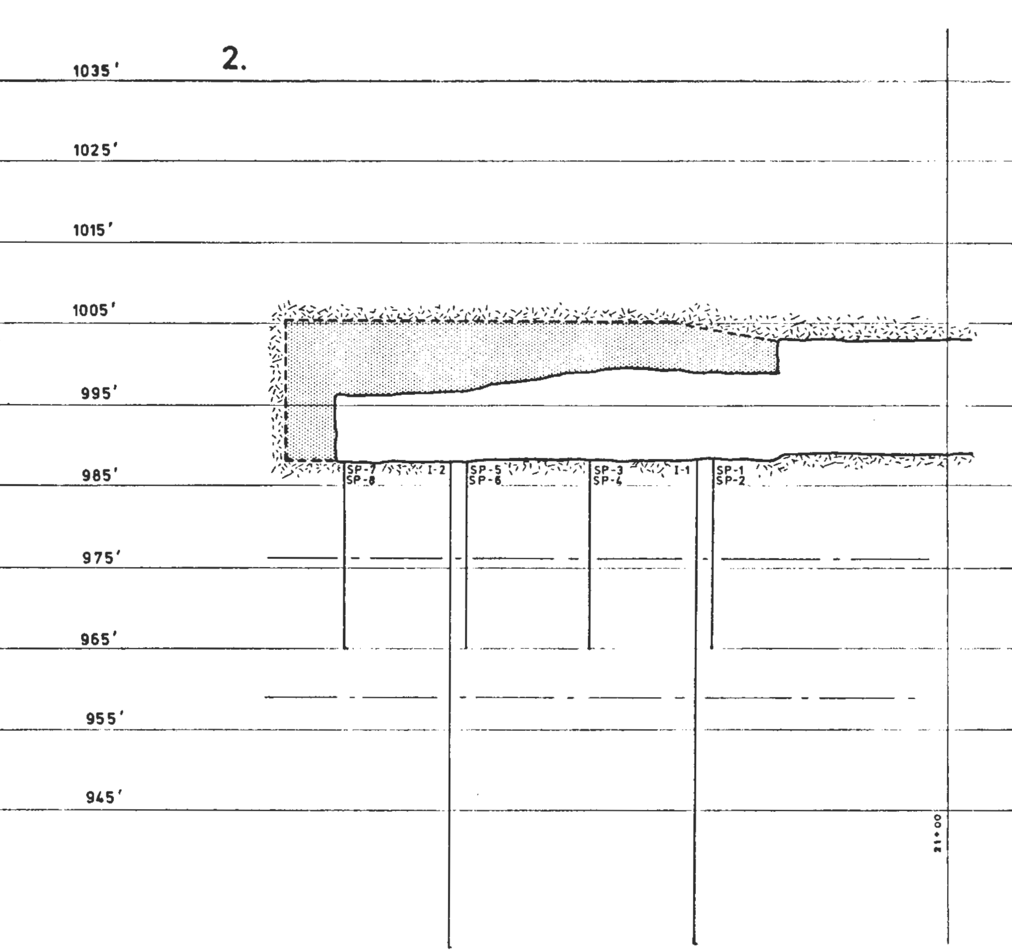
(6) PROTECTIVE EQUIPMENT MUST BE REPLACED AT ALL INSTRUMENTATION STATIONS PRIOR TO EACH BLAST

| | | |
|---|---|---|
| WEIR-JONES ENGINEERING CONSULTANTS LTD. | | |
| TRANSVERSE SECTIONS, MARTA TEST CAVERN. | | |
| Based upon MARTA drawing no. SE 002.R1. | | |
| DRAWN BY: | DATE: 27 Mar. 1978 | DRAWING NUMBER: |
| CHECKED BY: | SCALE: 1 in. ≡ 10 ft. | WJEC-M-2. |
| APPROVED BY: | SHEET 2 OF 4 | |

FIGURE V-4




BOREHOLE EXTENSOMETER LOCATIONS.




INCLINOMETER AND SEISMIC PROFILER HOLES.

NOTES:

1. SP-1 TO SP-8 ARE 2 1/2" DIAMETER PERCUSSION DRILL HOLES
   HOLES SP-1, SP-3, SP-5 AND SP-7 ARE 30' LONG, DIPPING 55° TO THE EAST
   HOLES SP-2, SP-4, SP-6 AND SP-8 ARE 35' LONG, DIPPING 45° TO THE WEST
   ALL SP HOLES ARE COLLARED ON THE TEST CAVERN CENTRE-LINE AND ARE TO BE COMPLETED PRIOR TO THE COMMENCEMENT OF ANY EXCAVATION.
2. I-1 AND I-2 ARE 3 1/4" DIAMETER PERCUSSION DRILL HOLES 60' DEEP. THEY ARE COLLARED ON TEST CAVERN CENTRE-LINE AND DRILLED VERTICALLY DOWN. THEY CAN BE DRILLED AFTER THE TEST CAVERN EXCAVATION AND LINING HAS BEEN COMPLETED.
3. E-1 TO E-14 ARE 2 1/2" DIAMETER PERCUSSION DRILL HOLES FOR MULTI WIRE EXTENSOMETERS, THE LENGTHS AND INCLINATIONS OF THESE HOLES ARE SHOWN ON THE TRANSVERSE SECTIONS ON DRAWING NUMBER WJEC-M-2. ALL EXTENSOMETER HOLES ARE TO BE DRILLED PRIOR TO EXCAVATION OF THE TEST CAVERN.
4. HOLES E-2, E-3, E-5, E-6, E-8, AND E-9 ARE TO BE FILLED WITH WEAK GROUT OR RESIN AFTER INSTRUMENT INSTALLATION BUT BEFORE EXCAVATION.
5. THE INCLINED EXTENSOMETER HOLES E-2, E-3, E-5, E-6, E-8 AND E-9 ARE COLLARED 1 1/2' AWAY FROM THE EXISTING DRIFT WALLS AS SHOWN ON DRAWING NUMBER WJEC-M-2.
6. EXTENSOMETER HOLES E-1 TO E-9 ARE TO BE COLLARED IN A 1' X 1' X 1' DEEP OPENING FORMED BY LINE DRILLING AND BREAKING OUT. THE FIRST 24" OF THE HOLE SHALL BE DRILLED WITH A 3 1/4" ø BIT. SEE DETAIL ON WJEC-M-4
7. EXTENSOMETER HOLES E-10 TO E-14 SHALL BE BROKEN OUT AND COLLARED AS IN NOTE 6 AS SOON AS THE BACK IS EXPOSED. SEE DETAIL ON WJEC-M-4
8. FOR TRANSVERSE SECTIONS, DETAILS OF ROOF AND WALLS, AND GENERAL PLAN SEE DRAWINGS WJEC-M-2/3/4.
9. ANCHOR SPACINGS IN INCLINED EXTENSOMETER HOLES E-2, E-3, E-5, E-6, E-8 AND E-9 SHOWN ON TRANSVERSE SECTIONS ON DRAWING NUMBER WJEC-M-2.

WEIR-JONES ENGINEERING CONSULTANTS LTD.

LONGITUDINAL SECTIONS, MARTA TEST CAVERN.

Based upon MARTA drawing no. SE 002. R1.

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| DRAWN BY: | DATE: 22 Mar. 1978 | DRAWING NUMBER |
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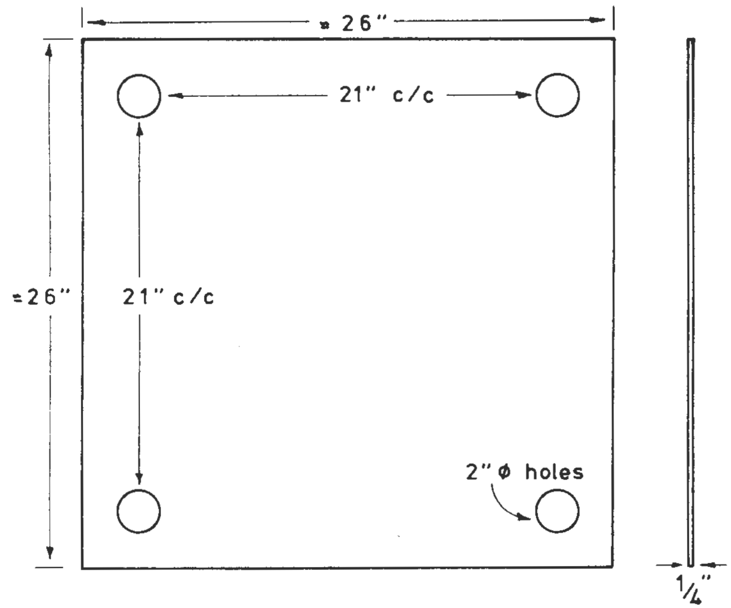
| APPROVED BY: | SHEET 1 OF 4 | |

FIGURE V-5




NOTES:

(1) THE EXTENSOMETER STATION COVER PLATES SHALL BE CUT FROM 1/4" MILD STEEL PLATE. THE PLATES SHALL HAVE FOUR 2" DIAMETER HOLES CUT ADJACENT TO THE CORNERS TO FACILITATE MOUNTING AND SUBSEQUENT HANDLING.

(2) THE MOUNTING HARDWARE FOR EACH PLATE WILL CONSIST OF ONE 9" X 1" UNC REDI-ROD STUD AND TWO LOCK NUTS, THREE 4" X 3/4" UNC BOLTS, AND FOUR MILD STEEL WASHER PLATES.

(3) IN USE THE COVER PLATE WILL BE ROTATED ABOUT THE STUD AFTER THE BOLTS AND WASHER PLATES HAVE BEEN REMOVED.

(4) THE REDI-ROD STUDS AND THREADED SLEEVES FOR THE BOLTS WILL BE FIXED IN THE SHORT PERCUSSION DRILL HOLES BY MEANS OF SUITABLE BONDING AGENTS SUCH AS CYANAMID ROC-LOC, CIMENT FONDU, DEVCON F, OR EQUIVALENT.

(5) THE COVER PLATES WILL BE RETURNED TO THEIR CLOSED POSITIONS AFTER EACH SET OF READINGS HAVE BEEN TAKEN. THIS PROCEDURE WILL BE FOLLOWED UNTIL ALL EXCAVATION AND LINING WORK IN THE TEST CAVERN HAS BEEN COMPLETED.

(6) MATERIALS REQUIRED:

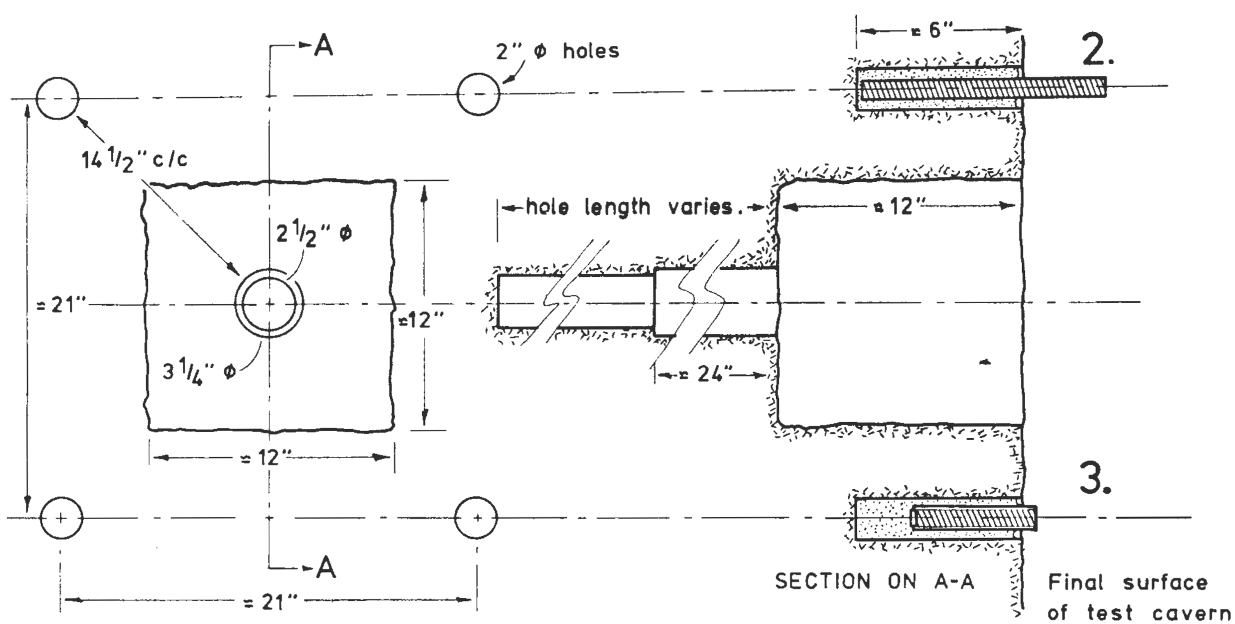
ITEM 1; EXTENSOMETER STATION COVER PLATES.
16 EA 26" X 26" X 1/4" MILD STEEL WITH 4 X 2" DIAMETER MOUNTING HOLES.
APPROXIMATE WEIGHT OF PLATE 50 LBS.

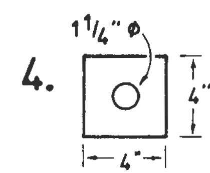
ITEM 2; REDI-ROD STUDS.
16 EA 9" X 1" UNC STUDS C/W 32 LOCKING NUTS.

ITEM 3; THREADED SLEEVES AND BOLTS.
50 EA 6" X 3/4"UNC INTERNALLY THREADED SLEEVES C/W END PLUGS AND 50 EA STAINLESS STEEL 4" X 3/4" UNC BOLTS.

ITEM 4; WASHER PLATES.
70 EA 4" X 4" X 1/4" MILD STEEL PLATES WITH A CENTRAL 1 1/4"DIAMETER HOLE

(7) N.B. THE 6" DEEP X 2" DIAMETER HOLES SHOWN AT 2 AND 3 ARE OFF SECTION.




| MARK | REVISION | BY | DATE |
|---|---|---|---|

WEIR-JONES ENGINEERING CONSULTANTS LTD.

EXTENSOMETER STATION DETAILS
MARTA TEST CAVERN.

FRACTIONAL DIM ± 1 64, DECIMALS ± 005, UNLESS SPECIFIED

| | | |
|---|---|---|
| DRAWN BY | DATE 29 Mar. 1978 | DRAWING NUMBER |
| CHECKED BY | SCALE 1 in. = 4 ins. | WJEC-M-4 |
| APPROVED BY | SHEET 4 OF 4 | |

+ 47.5. Two were inclined down towards the Running Tunnels, and two were inclined upwards.

The function of these extensometers was to monitor the development of the small magnitude radial deformations which were initially expected to develop inward towards the Research Chamber as the enlargement took place, and then deform downward toward the Running Tunnels as the latter were being driven below the completed Research Chamber. Extensive modifications to the proposed excavation schedule were in fact made; the actual sequence did not correspond to that anticipated, which did not permit optimum instrument utilization. This will be discussed later.

The instrumentation layout adopted was much more extensive than would normally be employed for an excavation of this type in ground which was known to be competent and subjected to stresses of a small magnitude. There were two reasons for the adoption of this massively redundant system. The first was the necessity of producing enough high resolution field data to enable the investigators to assess the reliability of the deformation predictions produced by the 2-D and 3-D FEM analyses. The second reason was purely practical. It was assumed that there would be an instrument mortality rate of 30% to 50% during the enlargement of the Research Chamber and during the driving of the Running Tunnels. This rate is fairly high, but taking into account the congested conditions in the Research Chamber and the fact that all six of the downward inclined extensometers were to penetrate the Running Tunnels, it was not thought to be unreasonable. It was assumed that, even if a mortality rate of 50% was encountered, there would be enough data generated by the remaining units to provide an adequate basis for comparison with the 2-D and 3-D FEM studies. Precautions were taken to protect the extensometer and inclinometer collar stations from blast damage and from destruction during the mucking cycle. The design of the shielding is shown in Figure V-5.

Two 18 meter (60 ft.) deep 8.25 cm (3-1/4 in.) diameter percussion drill holes at stations 20 + 39 and 20 + 69 were lined with plastic inclinometer casing in order to measure the horizontal deformations occurring below the invert of the Research Chamber between the two Running Tunnels. An inclinometer was the only practical way of measuring the horizontal deformations developing around the lower tunnels. It was obviously not possible to install extensometers in this location; the inclinometers offered the best solution. In this location the inclinometer would be capable of monitoring the horizontal deformations occurring towards the Running Tunnels, and also those which had developed towards the Peachtree Center Station cavern located to the north. Even though these deformations were expected to be of small magnitude and due essentially to the redistribution of residual and induced stresses, the inherent accuracy of the inclinometer system would permit their reliable measurement.

No attempt was made to use borehole extensometers to measure any of the rock mass deformations which may have occurred between the northern end of the Research Chamber and the Peachtree Center Station excavation. Thus the potentially available rock mass deformation data was confined to the zone around the Research Chamber bounded by stations 20 + 32 and 20 + 69.

Figures V-2 and V-3 show the positions of eight seismic profile holes which were drilled from the original Pilot Tunnel adit using 6.4 cm (2-1/2 inch) diameter percussion equipment. The original instrumentation and testing programme called for these eight holes to be used for obtaining P and S wave velocity profiles in a radial direction from the Pilot Tunnel drift before, during and after the development of the entire excavation sequence. Similar investigations on a number of other projects have indicated that the velocity profiles not only provide an efficient method of computing the mass modulus of the surrounding rock, but they also provide quite precise information about the extent of the relaxed or microfractured zone which is set up around the excavation as a result of blasting. In turn this latter information can be used to select the optimum type and length of anchorage for the rock bolts being used for temporary or permanent support.

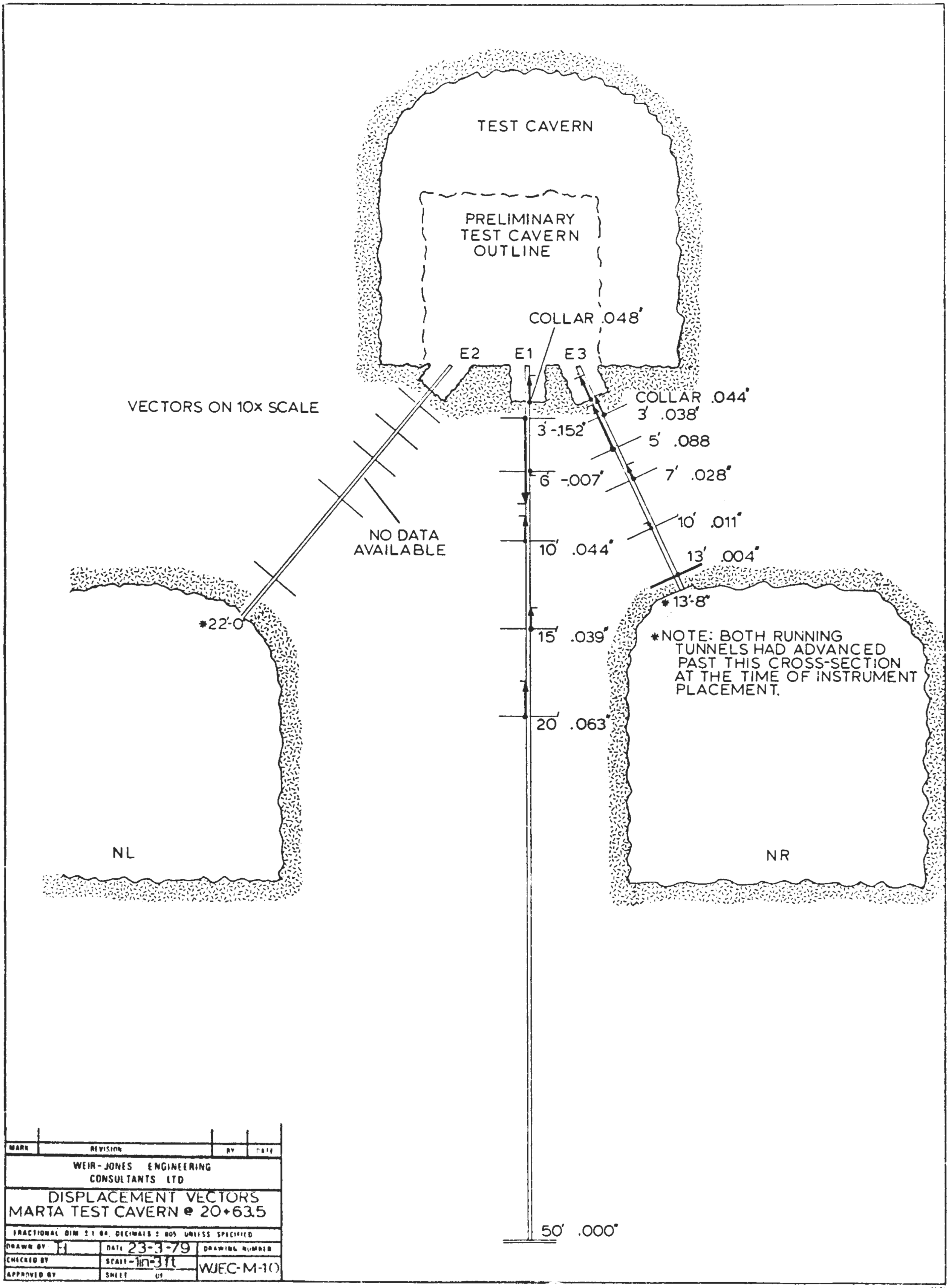

TEST CAVERN
PRELIMINARY TEST CAVERN OUTLINE
COLLAR .048'
E2
E1
E3
VECTORS ON 10x SCALE
COLLAR .044'
3' .038'
3' -.152'
5' .088
6' -.007'
7' .028'
NO DATA AVAILABLE
10' .011'
10' .044'
13' .004'
*13'-8"
*22'-0"
15' .039'
*NOTE: BOTH RUNNING TUNNELS HAD ADVANCED PAST THIS CROSS-SECTION AT THE TIME OF INSTRUMENT PLACEMENT.
20' .063'
NL
NR
MARK
REVISION
BY
DATE
WEIR-JONES ENGINEERING CONSULTANTS LTD
DISPLACEMENT VECTORS
MARTA TEST CAVERN @ 20+63.5
FRACTIONAL DIM ± 1 64, DECIMALS ± .005 UNLESS SPECIFIED
DRAWN BY H
DATE 23-3-79
DRAWING NUMBER
CHECKED BY
SCALE-1in-3ft
APPROVED BY
SHEET 01
WJEC-M-10
50' .000'

FIGURE V-6

Due to modifications in the excavation sequence which was actually followed, and to the contractor's problems in preparing the boreholes in accordance with the specifications, it proved to be impossible to carry out the seismic profile tests.

### The Results

Figures V-6 to V-22 summarize all the results obtained from the borehole extensometers and inclinometers in the test cavern for the period from mid-September 1978 until mid-March 1979. In essence, the available data confirmed the initial assumption of the investigators that the elastic relaxations which would develop towards any of the openings, due to stress relief or redistribution, would be very small. Furthermore, there is no clear indication of the existence of inelastic displacements around either the Research Chamber or the Running Tunnels. The deformations which have been unambiguously measured would appear to lie within the range of values anticipated for the "elastic" recovery of a competent and continuous rock mass.

Unfortunately, the value of the extensometer data was greatly reduced due to the fact that the underlying Running Tunnels were advanced before the Research Chamber was excavated, which was the reverse of the sequence assumed in the design stage. Both Running Tunnels were well-advanced by the time the lower extensometers E1 to E9 were installed. In fact, at station 20 + 63.5 (Figure V-6) both the NL and NR tunnels had been driven past the station before installation of instrumentation commenced. This was particularly unfortunate, because it eliminated the possibility of measuring the elastic relief effect which should have been seen developing towards the free surface.

A similar problem was encountered with inclinometers I1 and I2, which were not read until after excavation of the NL and NR tunnels had progressed some distance to the south below and beyond the Research Chamber. For this reason, it was not possible to measure the cyclical lateral relaxations which were expected to develop as first one and then the second Running Tunnel approached and passed the inclinometer locations. The

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

**WEIR-JONES ENGINEERING CONSULTANTS LTD.**
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EXTENSOMETER NUMBER E1

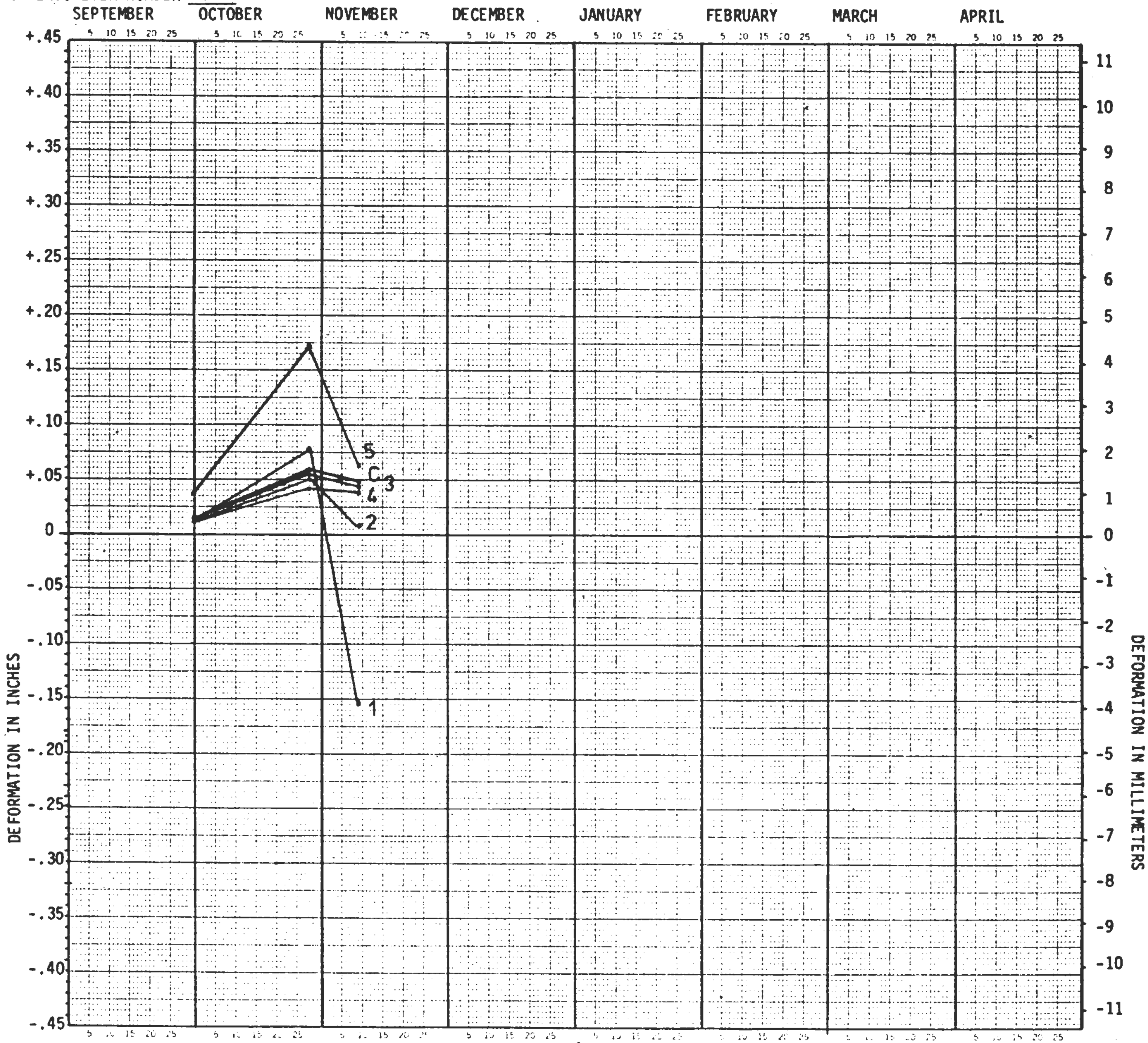



FIGURE V-7

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

EXTENSOMETER NUMBER E 3

**WEIR-JONES ENGINEERING CONSULTANTS LTD.**
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7

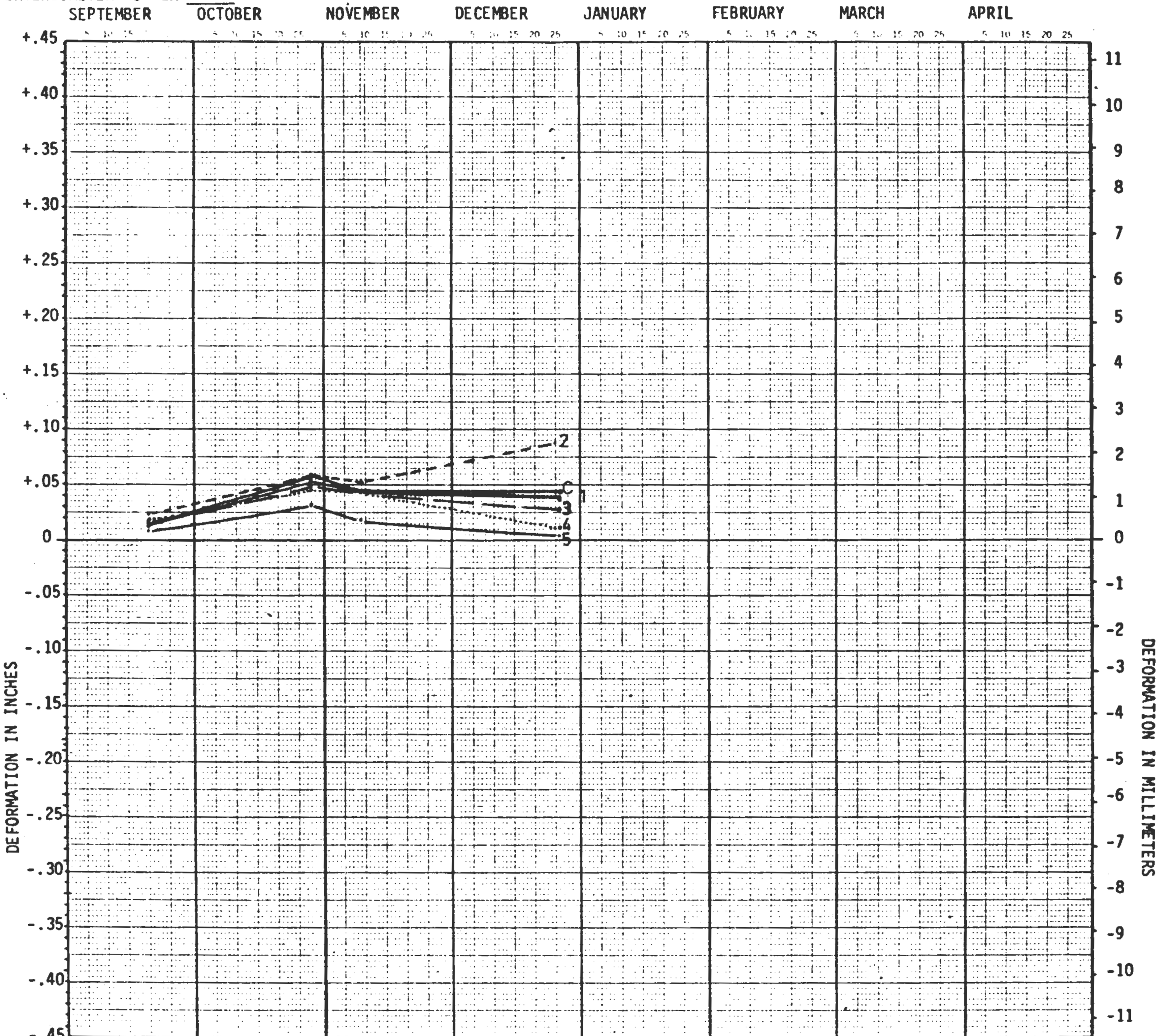



FIGURE V-8

data which was obtained suggest that there may have been a slight rock mass relaxation in a northerly direction towards the main Peachtree Center Station cavern. The absence of inclinometer data is most unfortunate, because the system had the capability of monitoring the horizontal elastic deformations occurring at the elevation of the Running Tunnels with enough precision to allow the computation of the horizontal residual stress vectors. Inclinometers have seldom been installed in locations which permit this type of analysis, and this particular situation would appear to have been unique.

Extensometers E1, E2 and E3

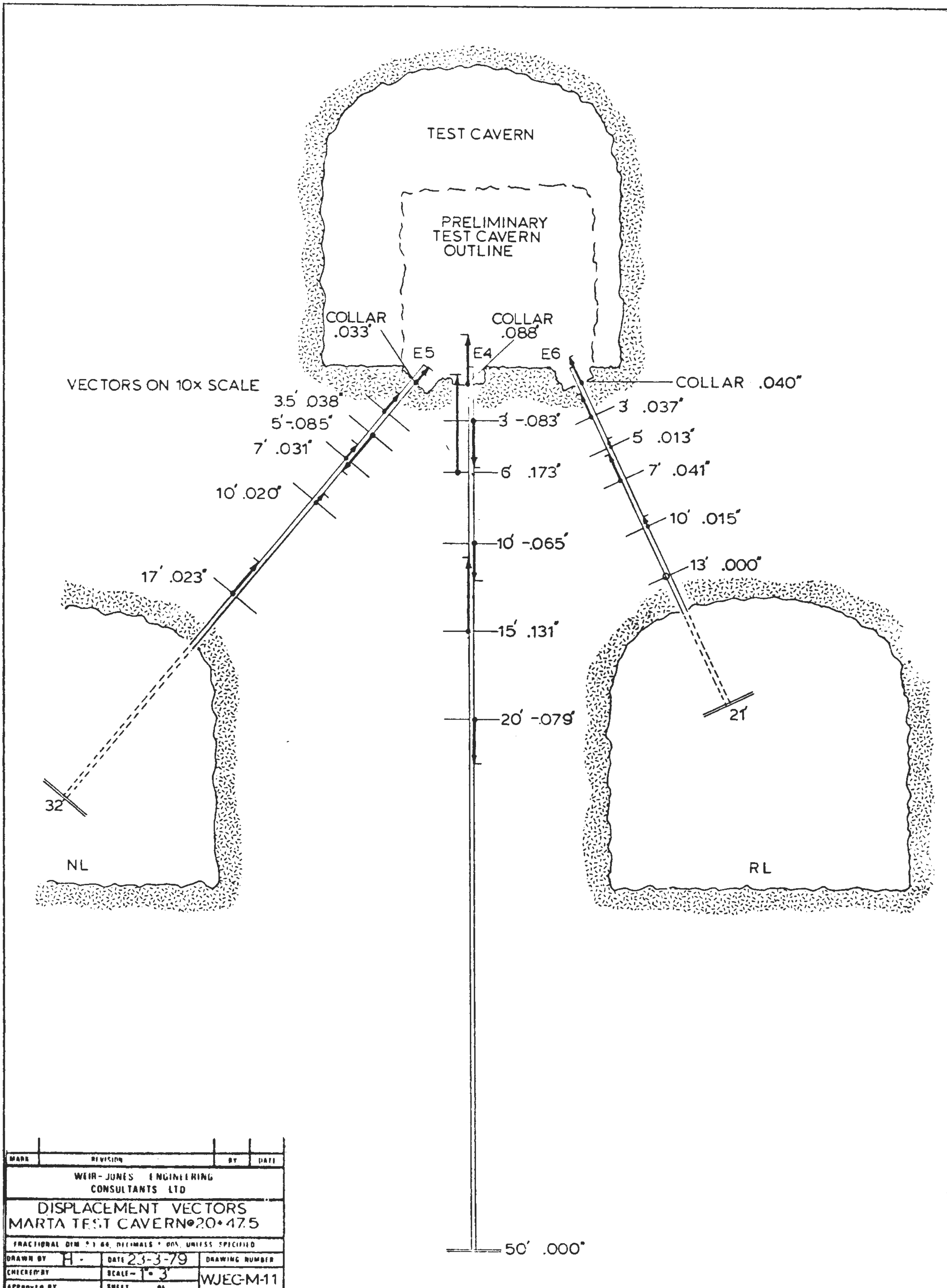
The down extensometers at station 20 + 63.5 (Figures V-6, V-7 and V-8) show the displacements of the various anchors in these holes.

In the case of unit E1, there is evidence to suggest that the collar station was initially disturbed between the end of September and late October 1978, or that the readings are questionable. Further damage may have occurred between October 28 and November 9, as the magnitude of the readings decreases to a level which is in general agreement with the results obtained from adjacent units. Some time after November 9, the instrument was damaged by construction activity in the Research Chamber.

The vectorial presentation on Figure V-6 summarizes the results obtained from E1 for the period from mid-September until November 9. Apart from the anomalous downward movement of the anchor 3 feet below the collar, there are immediately apparent general indications that rebound is occurring.

Extensometer E2 was destroyed by construction activity soon after installation, and no results were obtained.

Extensometer E3 yielded data for over three months until it was destroyed by construction work. The results shown on Figure V-6 and summarized as vectors for the entire period in Figure V-8, once more indicate an elastic rebound developing towards the Research Chamber. The result from anchor #2, 5 feet



TEST CAVERN
PRELIMINARY TEST CAVERN OUTLINE
COLLAR .033"
COLLAR .088"
E5
E4
E6
COLLAR .040"
VECTORS ON 10x SCALE
3.5' .038"
5' -.085"
7' .031"
10' .020"
17' .023"
32
3' -.083"
6' .173"
10' -.065"
15' .131"
20' -.079"
50' .000"
3' .037"
5' .013"
7' .041"
10' .015"
13' .000"
21'
NL
RL
MARK
REVISION
BY
DATE
WEIR-JONES ENGINEERING CONSULTANTS LTD
DISPLACEMENT VECTORS
MARTA TEST CAVERN@20+47.5
DRAWN BY H
DATE 23-3-79
DRAWING NUMBER
CHECKED BY
SCALE 1"=3'
WJEC-M-11
APPROVED BY
SHEET OF


FIGURE V-9

WEIR-JONES ENGINEERING CONSULTANTS LTD.
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

EXTENSOMETER NUMBER E4

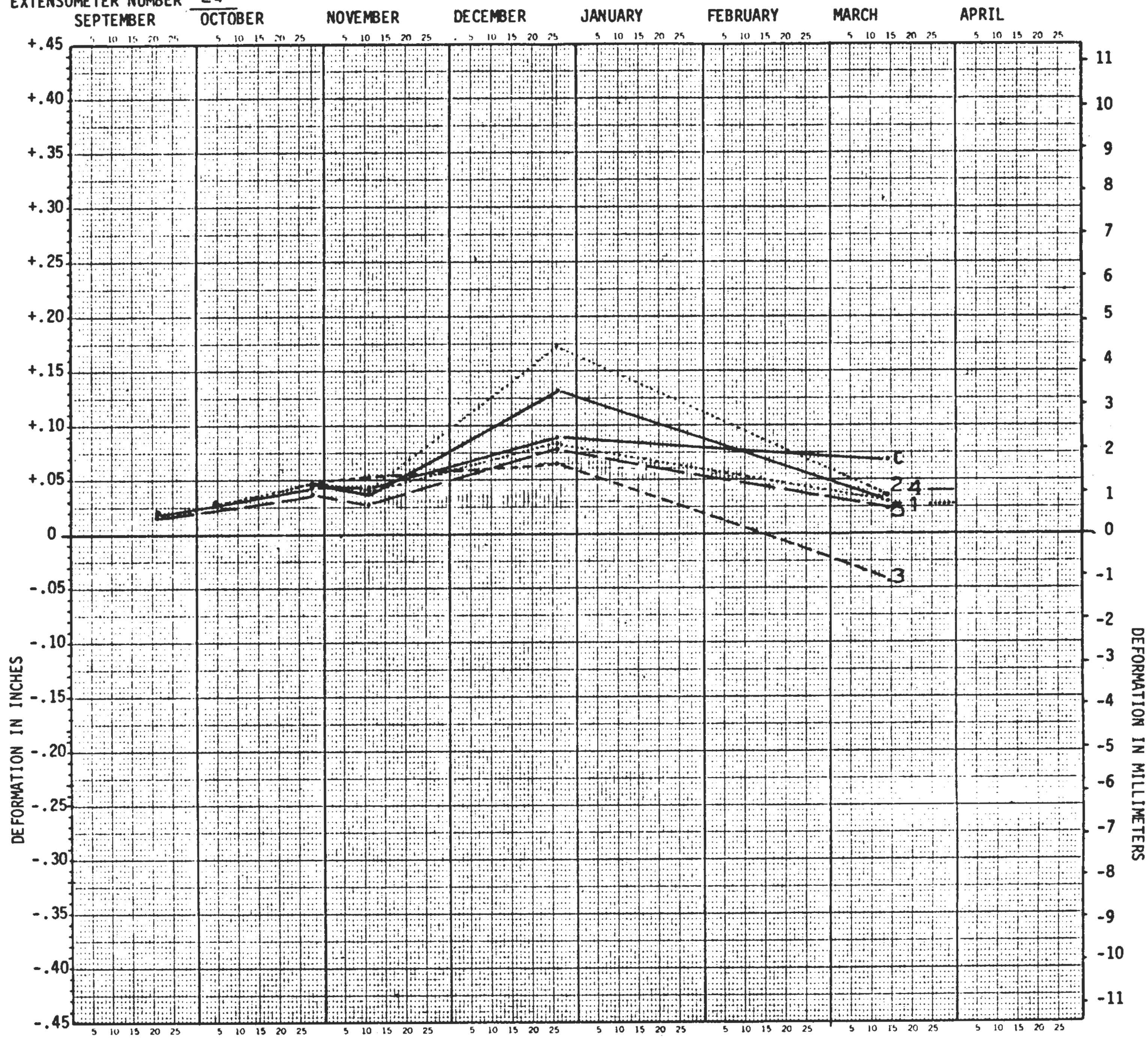



FIGURE V-10

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

EXTENSOMETER NUMBER E5

WEIR-JONES ENGINEERING CONSULTANTS LTD.
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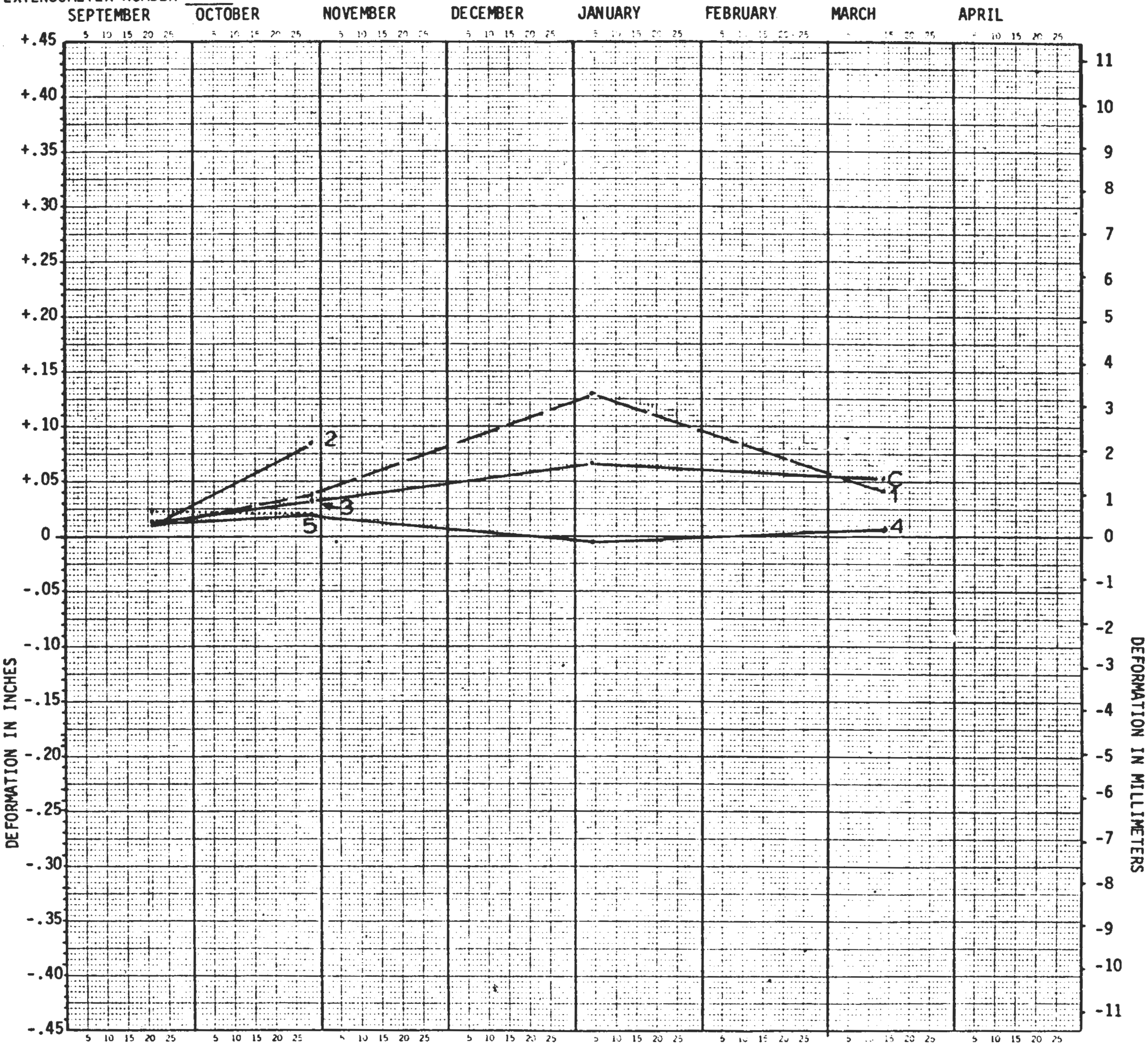


FIGURE V-11

WEIR-JONES ENGINEERING CONSULTANTS LTD.
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GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

EXTENSOMETER NUMBER E6

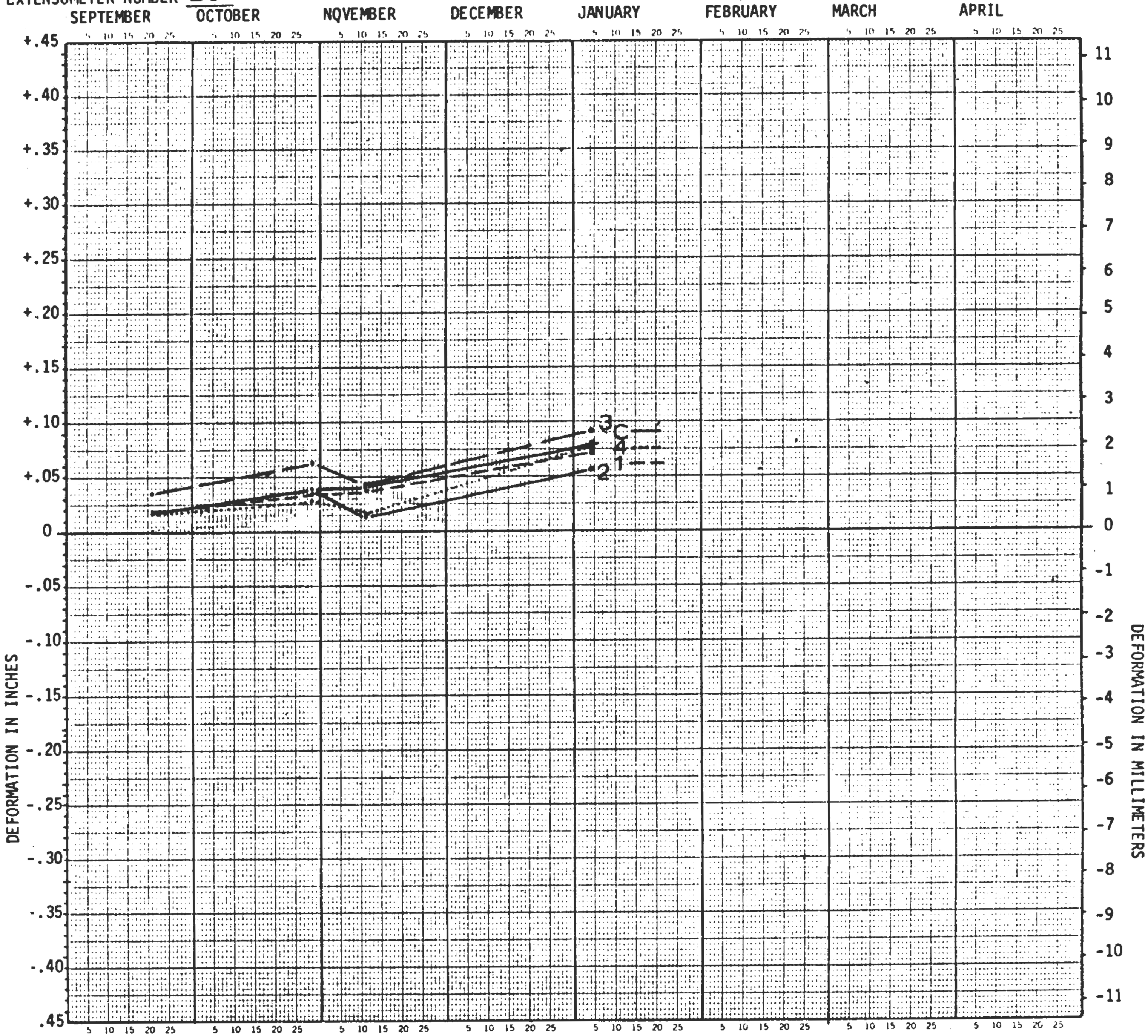



FIGURE V-12

below the collar, may be anomalous. The other vectors show a progressive increase in magnitude from the bottom of the hole to the top.

Extensometers E4, E5 and E6

Figures V-9, V-10, V-11 and V-12 show the displacements for the anchors at station 20 + 47.5.

Extensometer E4 is performing quite well. A minor data aberration can be observed in the readings taken on December 26, 1978. However, a number of valid explanations could be suggested for this. The results obtained in mid-March 1979 restore the trend developed in October and November. Furthermore, they indicate that there has been a cumulative uplift at the collar of about .07 inches. It is somewhat more than the predicted elastic rebound caused by the enlargement of the Research Chamber test cavern which had been calculated using estimates of the modulus and stress concentration factors existing around the initial opening. The disparity between the measurements obtained from E4 (and also E1 and E7), and the FEM model predictions made by Kulhawy in Chapter IV possibly may be explained by the very high in-situ stress in the third (north to south) dimension, which is not accounted for in 2-D FEM.

The vectorial presentation of the E4 displacement data shown on Figure V-9 covers the period mid-September to December 26, 1978. The collar uplift can be seen, but some of the other displacement vectors are questionable due to the final set of anomalous data.

Some of the anchors in extensometer E5 were damaged by construction work. However, the results in Figure V-11 indicate that the remaining anchors are showing little movement over an elapsed period from mid-September 1978 to mid-March 1979.

The results obtained from extensometer E6, Figure V-12 are similar to those of E5. The displacement magnitudes are small, within the elastic range, and there is evidence of a general uplift throughout the entire period from mid-September 1978 to early January 1979. The vectorial presentation of data on Figure V-9 shows a progressive increase in displacement magnitude from the bottom of the hole to the top.

Extensometer E7, E8 and E9

At station 20 + 32, Figures V-13 to V-16 show the displacements of the various anchors.

The results obtained from extensometer E7, Figure V-14, suggest that an anomalous offset was introduced between the first and second set of readings in mid-September. From September 21 until March 14 the data appear to be stable and to indicate that elastic uplifts have occurred which are of a reasonable magnitude. These progressive movements are shown in Figure V-13, which shows the E7 displacement vectors for the period terminating on November 7.

The results obtained from extensometer E8 (Figure V-15) show evidence of anomalous readings being taken on certain anchors on November 9 and on January 4. These anomalous readings are combined with an offset error, which apparently developed between the time of the first readings and those taken on September 21. The abnormal data can be eliminated; a much less curious vectorial presentation than the one currently shown for this borehole on Figure V-13 would then result.

Extensometer E9 suffered blast damage soon after installation when the NR Running Tunnel intersected the hole. This can be seen in Figure V-16 as the large offsets. Since the damage occurred, little deformation has been measured. The initial damage also accounts for anomalous vectors shown on Figure V-13.

Extensometers E10, E11, E12, E13, and E14

No data is available from extensometers E10, E11, and E12. The field personnel were not provided with a means of access to these holes which were collared in the center of the Research Chamber roof. Hence the only measurements made were those

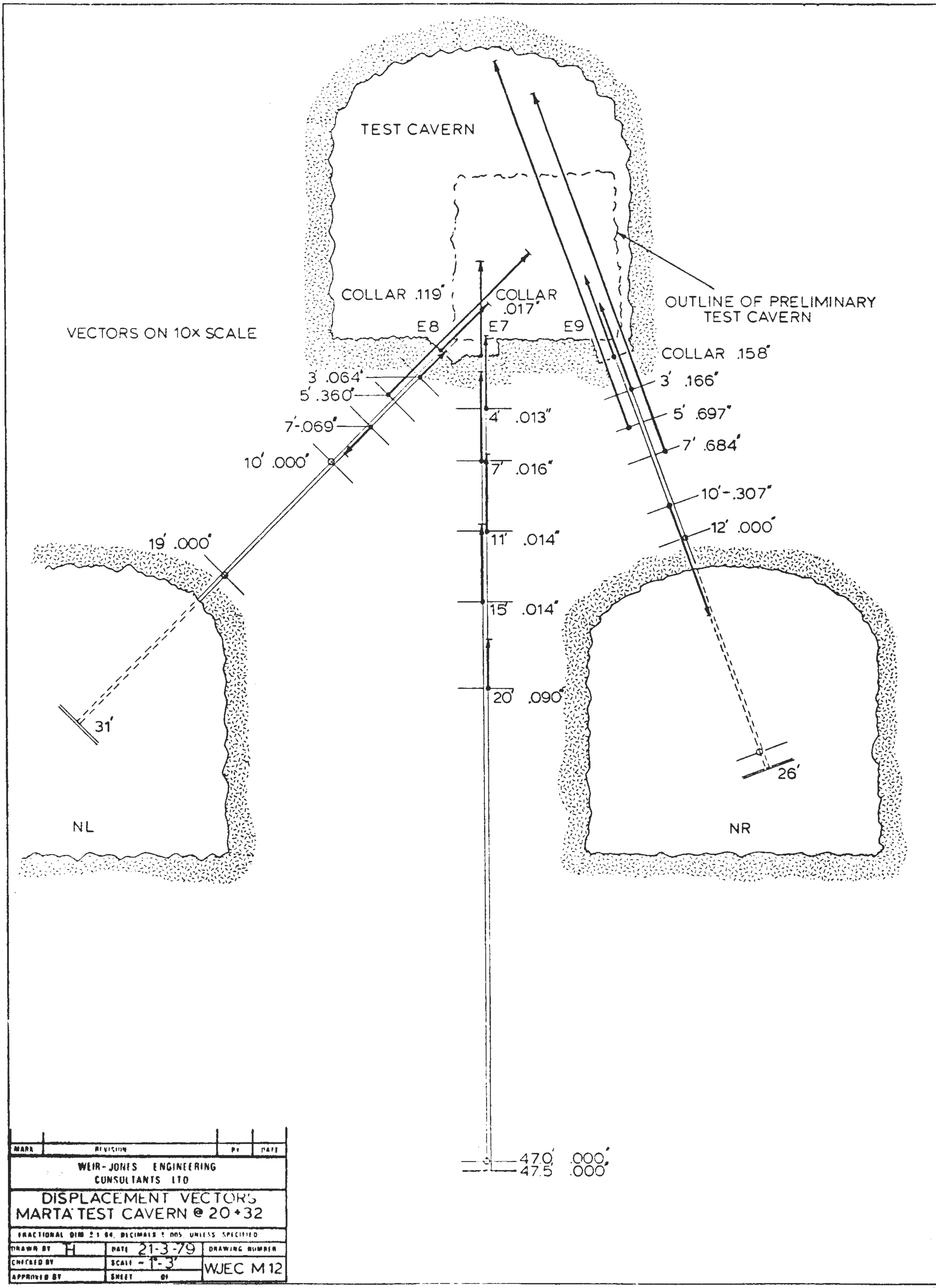

TEST CAVERN
COLLAR .119"
COLLAR .017"
OUTLINE OF PRELIMINARY TEST CAVERN
VECTORS ON 10x SCALE
E8
E7
E9
COLLAR .158"
3' .064"
5' .360"
7'-.069"
10' .000"
19' .000"
31'
4' .013"
7' .016"
11' .014"
15' .014"
20' .090"
3' .166"
5' .697"
7' .684"
10'-.307"
12' .000"
26'
NL
NR
47.0' .000"
47.5 .000"
MARK
REVISION
BY
DATE
WEIR-JONES ENGINEERING CONSULTANTS LTD
DISPLACEMENT VECTORS
MARTA TEST CAVERN @ 20+32
DRAWN BY H
DATE 21-3-79
DRAWING NUMBER
CHECKED BY
SCALE - 1"-3'
WJEC M 12
APPROVED BY
SHEET

FIGURE V-13

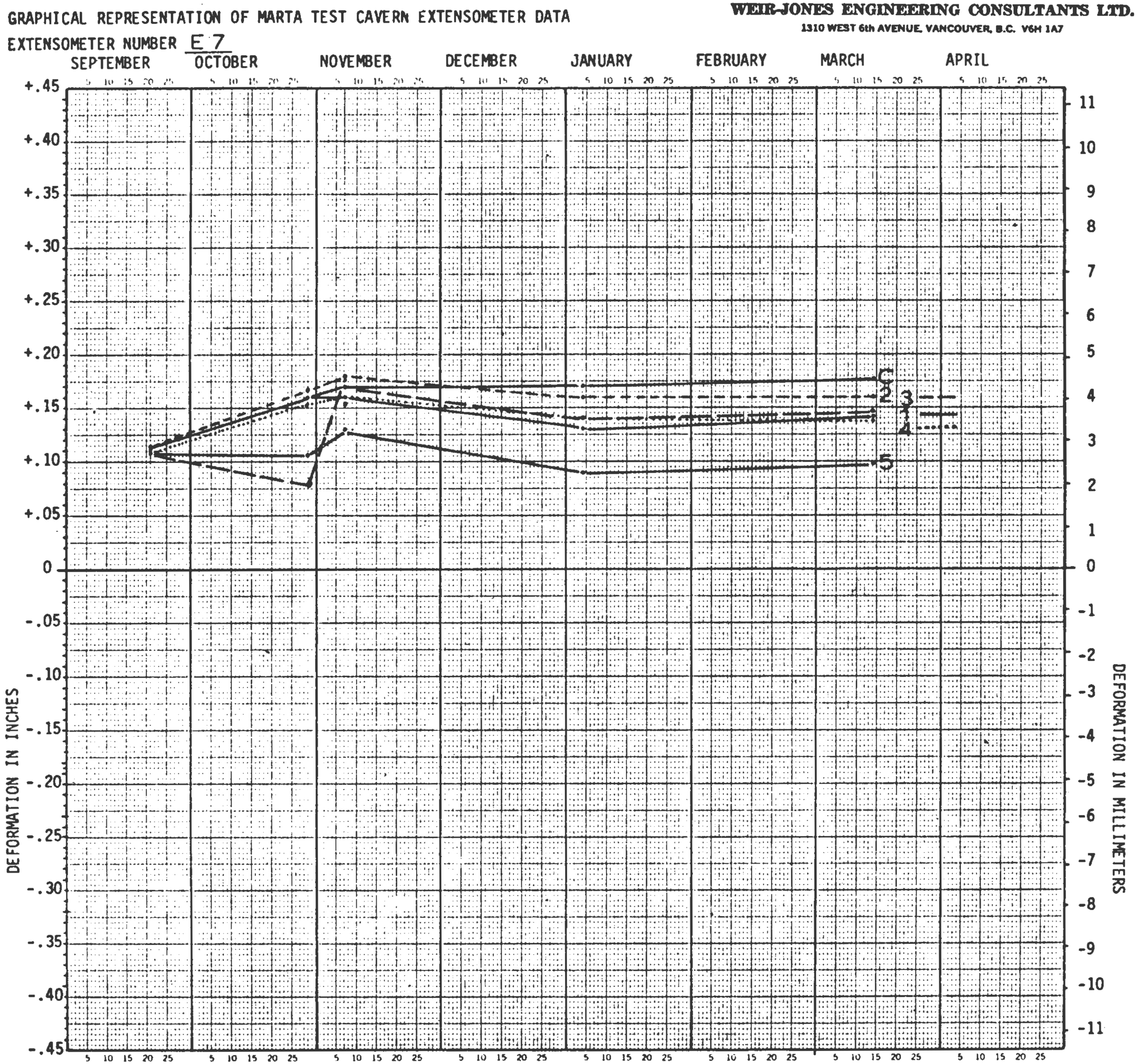

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA
EXTENSOMETER NUMBER E 7
WEIR-JONES ENGINEERING CONSULTANTS LTD.
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
DEFORMATION IN INCHES
DEFORMATION IN MILLIMETERS
1
2
3
4
5

FIGURE V-14

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

**WEIR-JONES ENGINEERING CONSULTANTS LTD.**
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7

EXTENSOMETER NUMBER E 8

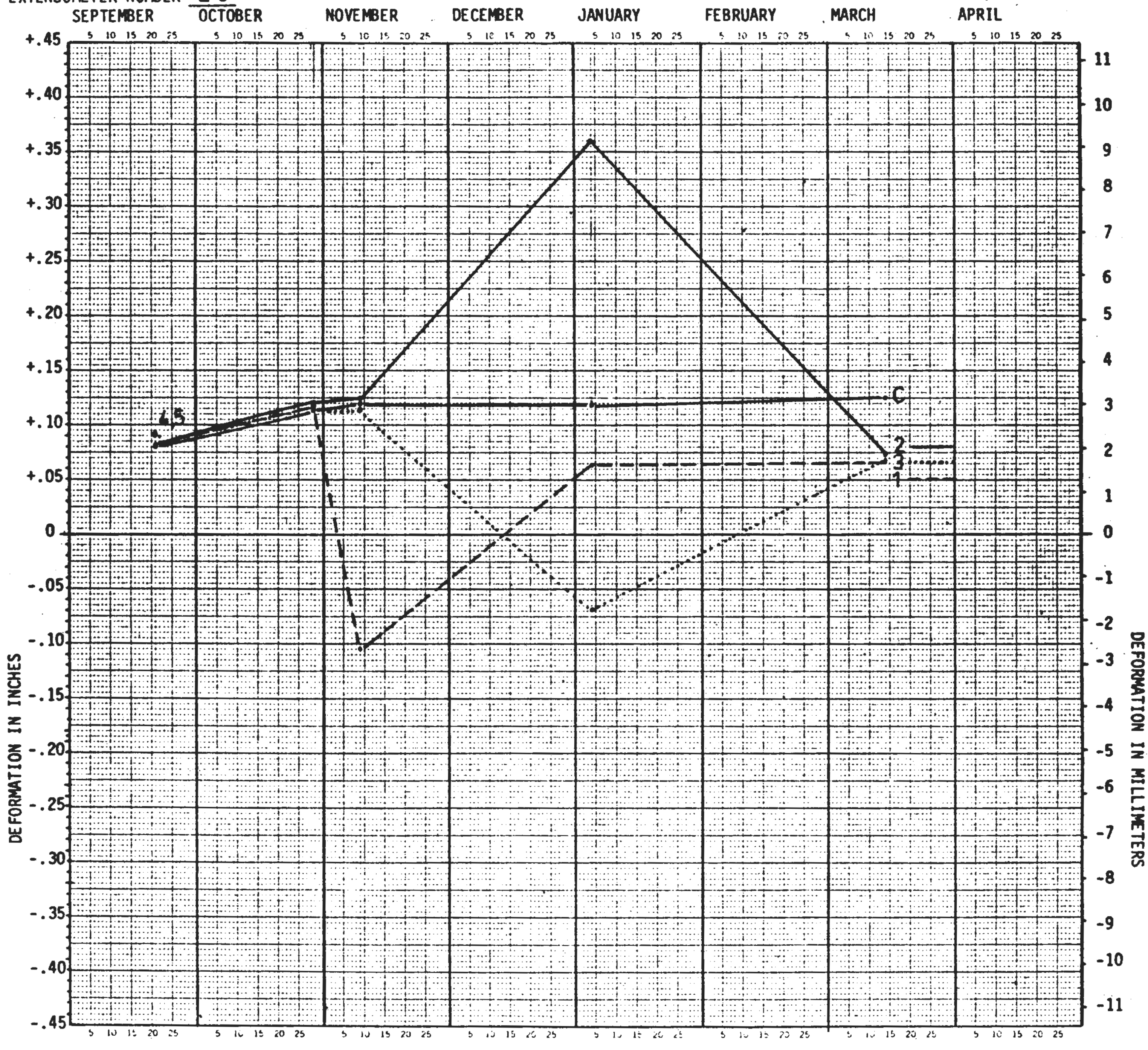
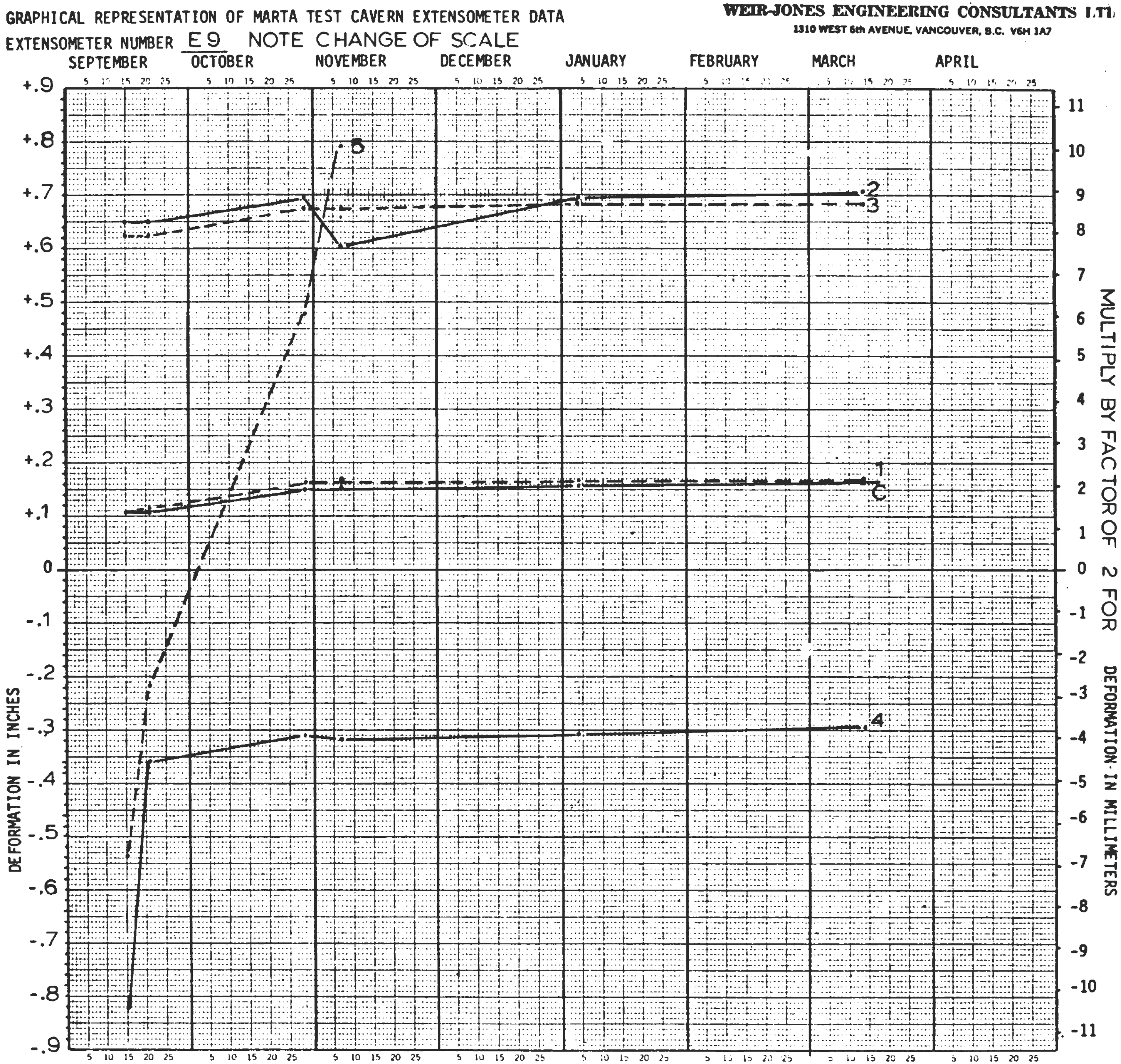



FIGURE V-15

FIGURE V-16



GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA
EXTENSOMETER NUMBER E9 NOTE CHANGE OF SCALE
WEIR-JONES ENGINEERING CONSULTANTS LTD
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
5 10 15 20 25
+.9
+.8
+.7
+.6
+.5
+.4
+.3
+.2
+.1
0
-.1
-.2
-.3
-.4
-.5
-.6
-.7
-.8
-.9
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
5
2
3
1
C
4
DEFORMATION IN INCHES
MULTIPLY BY FACTOR OF 2 FOR DEFORMATION IN MILLIMETERS

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

WEIR-JONES ENGINEERING CONSULTANTS LTD.
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7

EXTENSOMETER NUMBER E 13

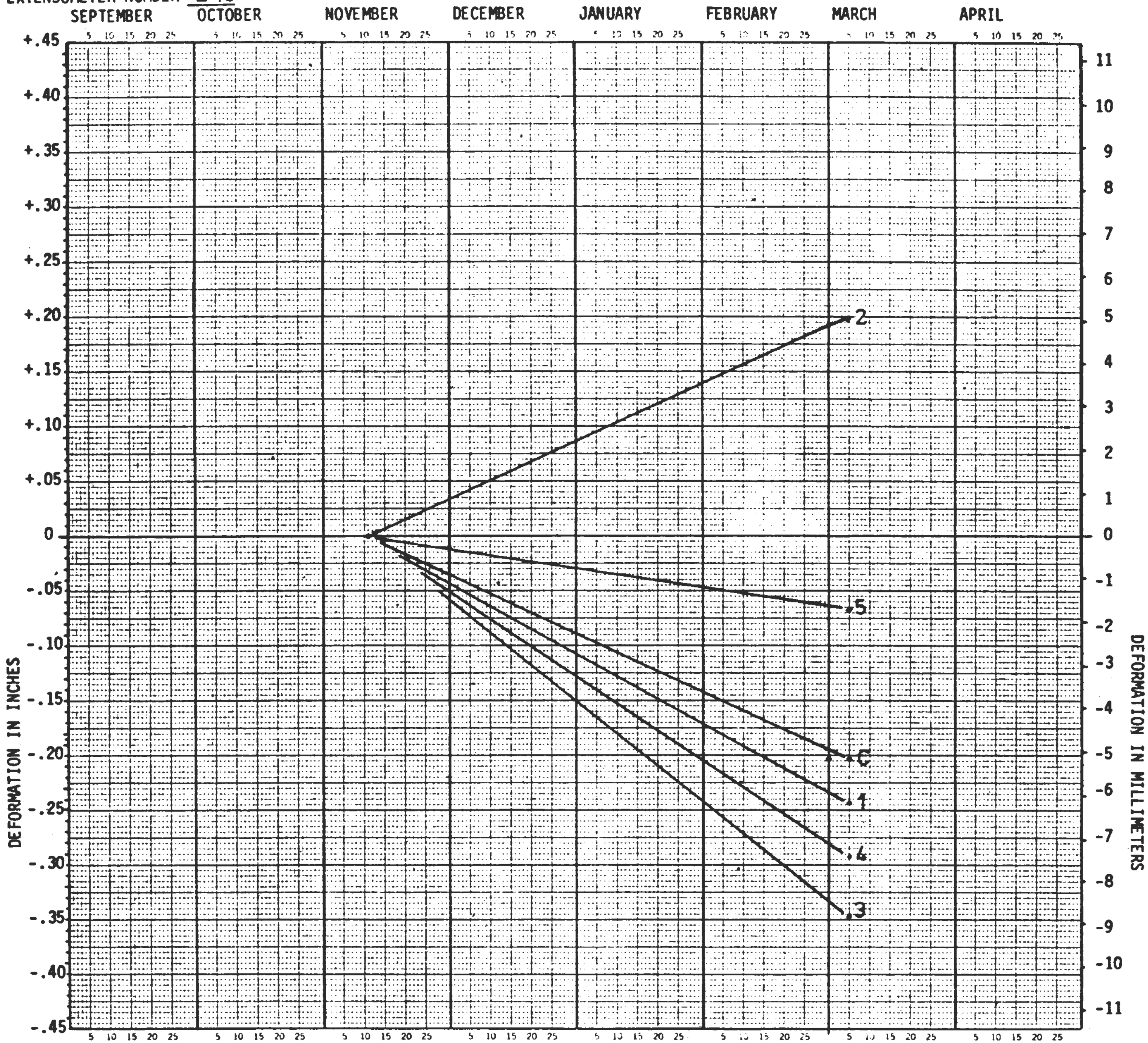



FIGURE V-17

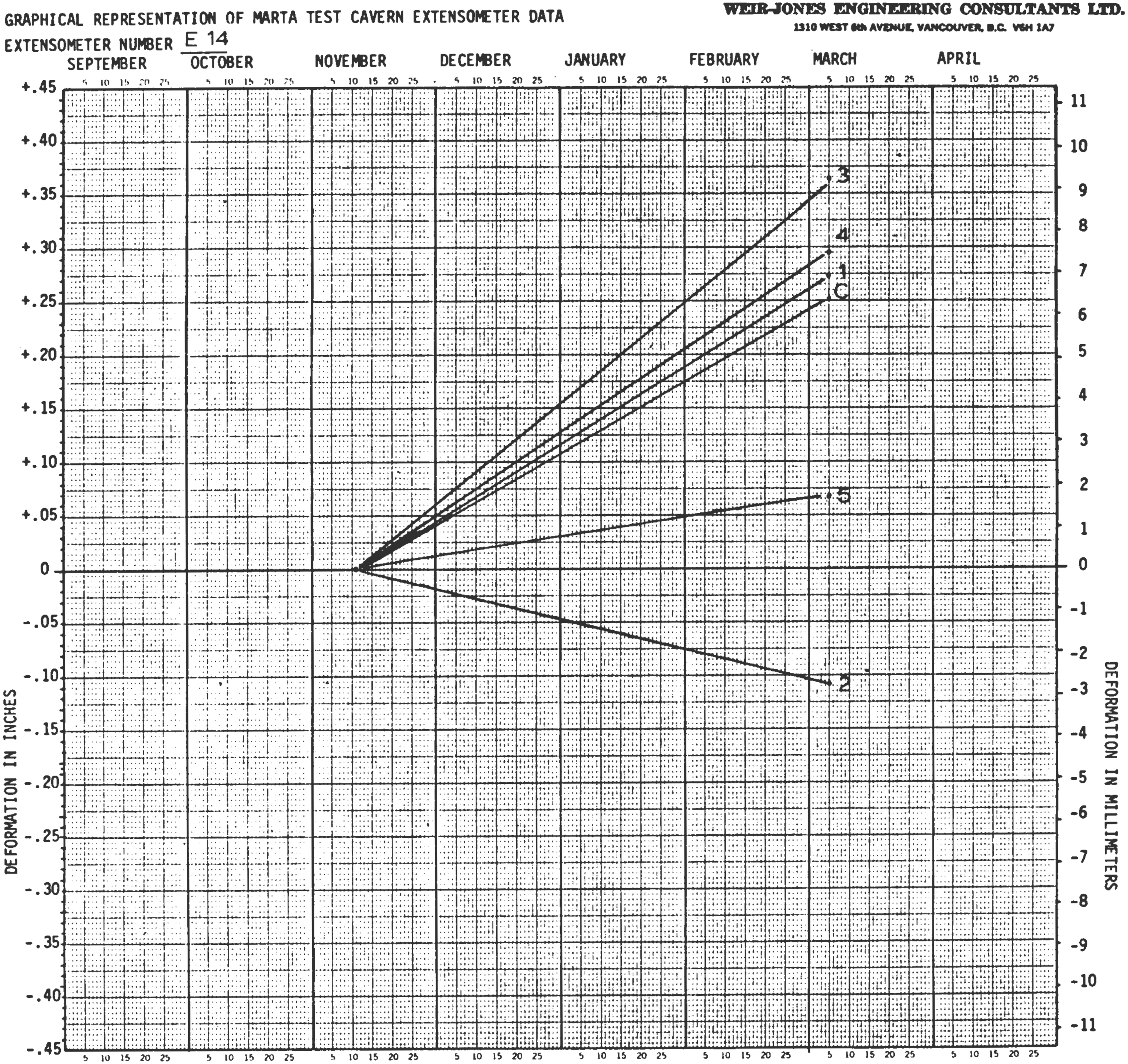

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA
EXTENSOMETER NUMBER E 14
WEIR-JONES ENGINEERING CONSULTANTS LTD.
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
APRIL
DEFORMATION IN INCHES
DEFORMATION IN MILLIMETERS
3
4
1
C
5
2

FIGURE V-18

taken by the installation crew immediately after they had completed their work. Inadequate data is available from the up extensometers E13 and E14 to support comments, as shown on Figure V-17 and V-18.

Inclinometers I-1 and I-2

The inclinometers I-1 and I-2 were installed prior to the enlargement of the Research Chamber but at the time that the NL and NR Running Tunnels were being advanced. For a number of reasons, approximately six weeks elapsed between the collection of the base data at the end of September and the next set of readings on November 7. Due to the absence of data during this period, it is not possible to draw any conclusions about the possibility of the occurrence of cyclical lateral displacements during the advancement of the NL and NR Running Tunnels.

The limited data available from the inclinometers is presented in Figures V-19 to V-22, which show that cumulative displacements developed in the north to south and east to west directions for both holes during the period from late September until early November. The data is presented as a series of computer tabulations which, although they do not show the true displacement profiles, are sufficiently precise to indicate the location of inflection points, etc.

More movement has occurred at location I-1 than at I-2, with the easterly displacement component being about twice that of the northerly one. This shows that, although there is some rock mass relaxation developing towards the Peachtree Center Station excavation to the north of the Research Chamber, the bulk of the movement is towards the NR Running Tunnel. The data from I-2 displays the same trend although the displacement magnitudes are significantly less.

The inclinometers remain intact, but no measurements have been made for several months. It is anticipated that, at this point in the instrumentation program, no further lateral deformations will take place.

```
N-120   TUNNEL     11/7/78      I-1

     ◆◆◆◆◆◆◆◆◆◆◆DEFLECTION ALONG THE NORTH/SOUTH AXIS◆◆◆◆◆◆◆◆◆◆◆
D                                                                            D
E                         ◆◆ DEFLECTION ◆◆                                   E
P          SOUTH                                        NORTH                P
T  -.75    -.50      -.25       0.0       .25       .50       .75            T
H--+---------+---------+---------+---------+---------+---------+---------+-- H
 3                               I  ◆                                        3
 5                               I  ◆                                        5
 7                               I  ◆                                        7
 9                               I  ◆                                        9
11                               I ◆                                        11
13                               I ◆                                        13
15                               I ◆                                        15
17                               I ◆                                        17
19                               I ◆                                        19
21                               I ◆                                        21
23                               I ◆                                        23
25                               I ◆                                        25
27                               I ◆                                        27
29                               I ◆                                        29
31                               I ◆                                        31
33                               I ◆                                        33
35                               I◆                                         35
37                               I◆                                         37
39                               I◆                                         39
41                               I◆                                         41
43                               I◆                                         43
45                               I◆                                         45
47                               ◆                                          47
49                               ◆                                          49
51                               ◆                                          51
53                               ◆                                          53
55                               ◆                                          55
57                              ◆I                                          57
59                              ◆I                                          59
61                               ◆                                          61
63                               ◆                                          63
```

FIGURE V-19

N-120   TUNNEL      11/7/78      I-1

```
       ◆◆◆◆◆◆◆◆◆◆DEFLECTION ALONG THE EAST/WEST AXIS◆◆◆◆◆◆◆◆◆◆
D                                                                  D
E                         ◆◆ DEFLECTION ◆◆                         E
P          WEST                                     EAST           P
T  -.75    -.50      -.25       0.0       .25       .50       .75  T
H--+---------+---------+---------+---------+---------+---------+-- H
 3                               I       ◆                        3
 5                               I       ◆                        5
 7                               I      ◆                         7
 9                               I      ◆                         9
11                               I     ◆                         11
13                               I     ◆                         13
15                               I     ◆                         15
17                               I     ◆                         17
19                               I    ◆                          19
21                               I    ◆                          21
23                               I    ◆                          23
25                               I   ◆                           25
27                               I   ◆                           27
29                               I   ◆                           29
31                               I  ◆                            31
33                               I  ◆                            33
35                               I  ◆                            35
37                               I  ◆                            37
39                               I  ◆                            39
41                               I  ◆                            41
43                               I  ◆                            43
45                               I  ◆                            45
47                               I  ◆                            47
49                               I  ◆                            49
51                               I ◆                             51
53                               I ◆                             53
55                               I ◆                             55
57                               I ◆                             57
59                               I ◆                             59
61                               I◆                              61
63                               ◆                               63
```

FIGURE V-20

N-120     TUNNEL     11/7/78     I-2

```
      **********DEFLECTION ALONG THE NORTH/SOUTH AXIS**********
D                                                                  D
E                          ** DEFLECTION **                        E
P             SOUTH                                NORTH           P
T   -.75      -.50        -.25        0.0       .25       .50       .75  T
H--+---------+---------+---------+---------+---------+---------+--  H
 2                              *I                                  2
 4                               I*                                 4
 6                               I*                                 6
 8                               I*                                 8
10                               *                                 10
12                               *                                 12
14                               *                                 14
16                               *                                 16
18                               *                                 18
20                               *                                 20
22                               *                                 22
24                               *                                 24
26                               *                                 26
28                               *                                 28
30                               *                                 30
32                               I*                                32
34                               I*                                34
36                               *                                 36
38                               *                                 38
40                               *                                 40
```

FIGURE V-21

N-120     TUNNEL     11/7/78     I-2

```
       ◆◆◆◆◆◆◆◆◆◆DEFLECTION ALONG THE EAST/WEST AXIS◆◆◆◆◆◆◆◆◆◆
D                                                                  D
E                         ◆◆ DEFLECTION ◆◆                         E
P         WEST                                     EAST            P
T -.75    -.50      -.25        0.0       .25      .50        .75  T
H--+---------+---------+---------+---------+---------+---------+-- H
 2                               I  ◆                              2
 4                               ◆                                 4
 6                               ◆                                 6
 8                               ◆                                 8
10                               ◆                                10
12                               ◆                                12
14                               ◆                                14
16                               I◆                               16
18                               I◆                               18
20                               I◆                               20
22                               I◆                               22
24                               I◆                               24
26                               I◆                               26
28                               ◆                                28
30                               I◆                               30
32                               I◆                               32
34                               I◆                               34
36                               I◆                               36
38                               I◆                               38
40                               ◆                                40
```

FIGURE V-22

Due to a number of factors not directly related to the instrumentation work, no data is available from any of the convergence stations which were installed as the Research Chamber was enlarged.

## Conclusions

The magnitude of the measured rock movements confirmed the 2-D FEM analysis which was made prior to design and construction of the Atlanta Research Chamber (Chapter IV). This analysis used as the upper bound K=2.5 for the in-situ horizontal east to west stresses and used an overall rock modulus reduced to about 15 percent of the laboratory modulus, even though the rock mass was of excellent quality, following the recommendations by Kulhawy.

However, the 2-D FEM could not take into account the highest in-situ stress, which was in the horizontal north-south direction (i.e., the "third dimension"). This north-south stress was almost seven times the vertical stress. Consequently, the magnitude of the upward displacements actually observed in the floor of the Research Chamber was not predicted by a 2-D FEM cross-section.

The unexpected difficulty in getting consistently good cooperation from field personnel caused problems and reduced the information that could have been made available. On the other hand, the instrumentation program was an overall success and much valuable data was obtained. There were many instances of exceptional and helpful effort by PB/T field inspectors and by Contractor's personnel. On balance, the research results seem as good as can be expected considering that the Research Chamber work was only a small part of the very large and fast-track CN 120 Peachtree Center Station contract.

# CHAPTER VI.

# BLASTING AND EXCAVATION

## A. INTRODUCTION

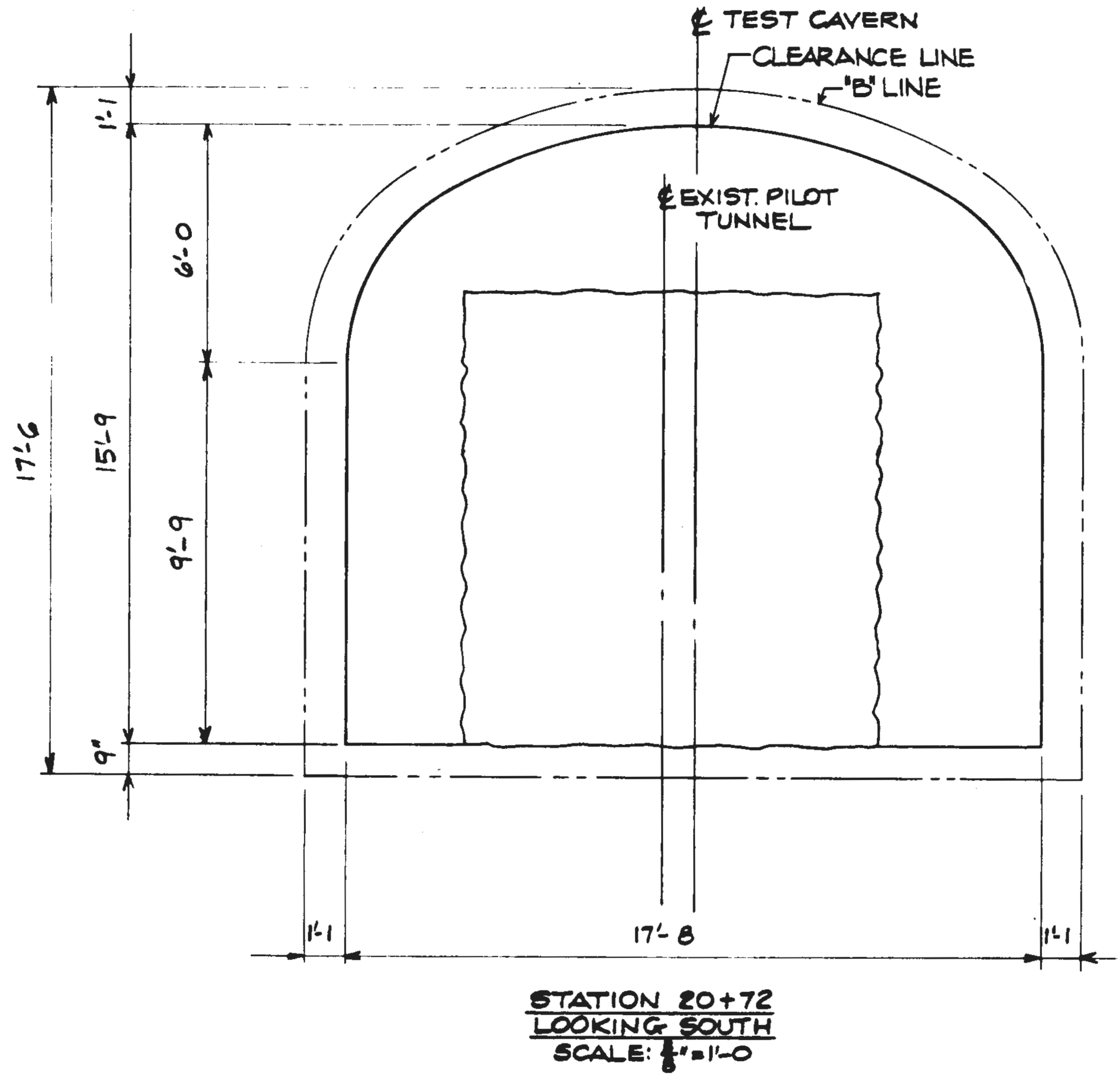
In 1977, a Pilot Tunnel was driven through the top center of the future main cavern site for the Peachtree Center Station. The same Pilot Tunnel was extended through the site of the future Atlanta Research Chamber, extending south of the subway station cavern. The opening of the Pilot Tunnel was roughly 3 meters square. The Pilot Tunnel alignment was several degrees off the alignment of the Atlanta Research Chamber (see Figure VI-1).

The Pilot Tunnel was driven with conventional drilling and blasting techniques. No special controls or procedures were involved. No controlled blasting techniques were used. Consequently, the outlines of the Pilot Tunnel excavation were rough and irregular. However, the rock at this site is of good quality, so the effects of the uncontrolled blasting were limited to the zones immediately adjacent to the perimeter surfaces of the tunnel itself.

The drilling, blasting and excavation of the Atlanta Research Chamber took place during the six-week period beginning October 11, 1978. Since the work was appended to the main contract for the construction of the Peachtree Center Station, it was done by the same contractor, Horn/Fruin-Colnon Company, Joint Venture. Equipment used for the work was that which was already present at the site for use on the main contract.

## B. EQUIPMENT AND PRODUCTS

Most of the drilling in the Atlanta Research Chamber was done with a single-boom Gardner-Denver track-mounted drill, shown in Photo VI-1, supplemented in part with a jack-leg drill. The last two rounds were drilled with an Atlas-Copco hydraulic drill jumbo.




The Atlanta Research Chamber

FIGURE VI-1

Tunnel muck was removed with a 6 cubic meter (8 cubic yard) Wagner mucker and hauled to the main access shaft where it was hoisted to the surface.

Explosives products used for the work consisted of semi-gelatins, Tovex T-1 water gel, Primacord, electric delay caps, NONEL Primadet non-electric delay caps, and stemming and miscellaneous accessories.

Some of the perimeter holes were scribed or notched mechanically with tools specially developed for that purpose.

## C. PROCEDURES

The general procedure was that of slashing into the existing Pilot Tunnel which had been driven during an earlier stage of work. Slashing burdens varied each round because the alignments of the Pilot Tunnel and the Atlanta Research Chamber were not precisely parallel. Relative to the final Chamber, the Pilot Tunnel began at the far right side for the first round, angled across the Chamber, and terminated at the far left side. Although somewhat variable in dimension, the Pilot Tunnel was roughly 3 meters by 3 meters, and was enlarged to the final Atlanta Research Chamber horseshoe shape approximately 5.5 meters wide by 5.8 meters high, 18 meters long.

Most of the perimeter surfaces of the Research Chamber were blasted with smooth blasting (cushion blasting) techniques. This method produced good results, leaving sound, undamaged final rock surfaces.

The walls of the Chamber were divided into nine "panels" for various demonstration purposes after excavation (see Figure VI-2). Ten rounds were drilled and detonated to excavate this length of adit. For the first round, two-stage blasting was done using conventional slashing and smooth blasting techniques. For the following seven rounds, some portion of the perimeter holes was scribed mechanically for experimental fracture-control blasting. Both smooth blasting and pre-splitting techniques were tried in conjunction with the scribing. The last two rounds were

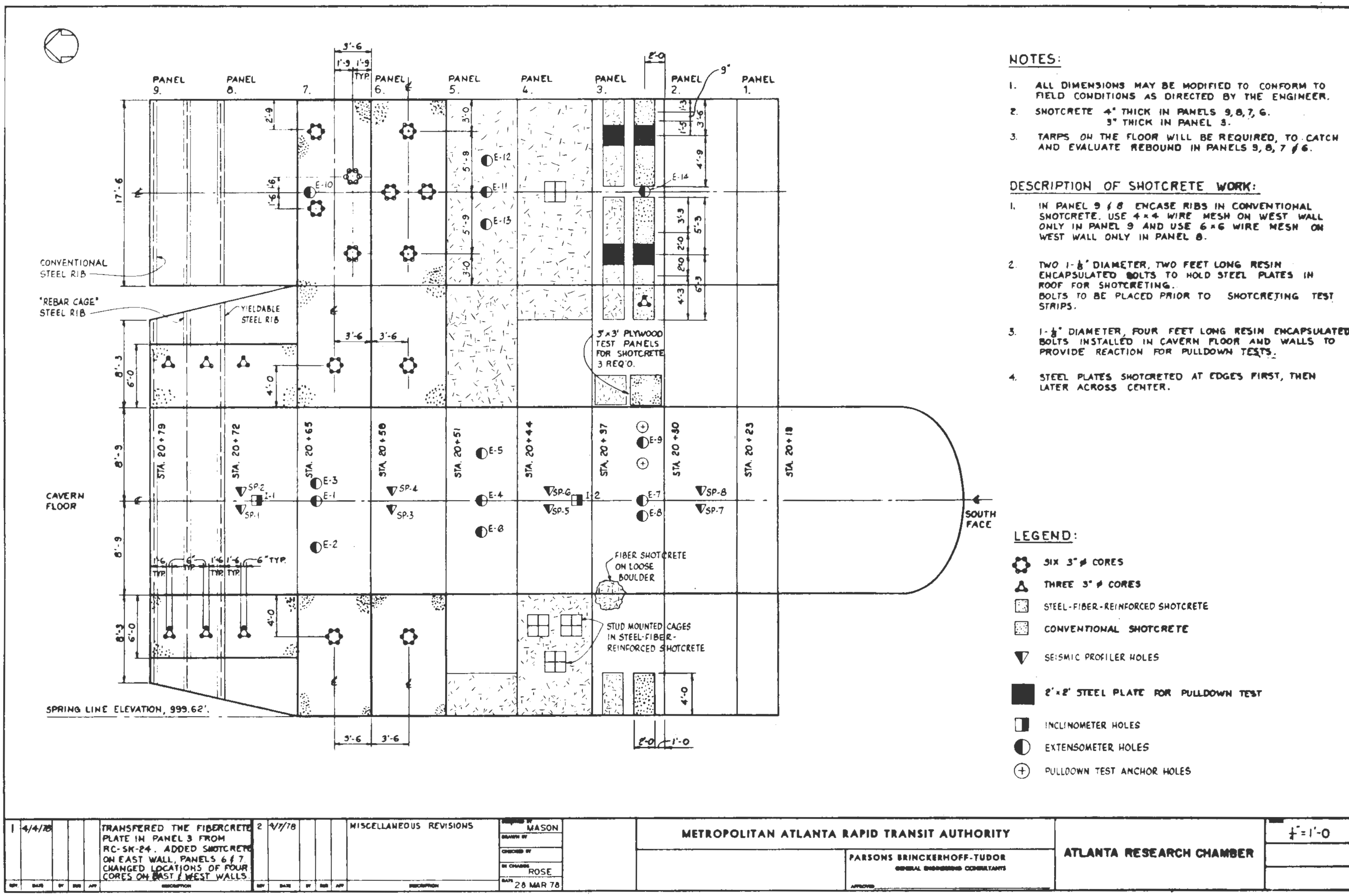



FIGURE VI-2 EXPLODED VIEW OF THE TEST CAVERN ROOF, FLOOR AND WALLS blasted at the Contractor's option with no special precautions being taken.

Enlarging the Pilot Tunnel to a full-size Research Chamber by drilling and blasting did not pose any unique difficulties because the rock was relatively sound. However, this type of task poses two essential requirements for acceptable results. First, drilling must be done accurately, because final perimeter surfaces are outlined precisely as they have been drilled. Final surfaces cannot possibly be more accurate than the perimeter drilling, no matter how cautious the blasting. Only worse results can occur if the blasting is not carefully done. Second, cautious perimeter blasting methods must be used. Simply stated, such controls require the use of small-diameter decoupled charges in order to prevent shattering and overbreak of final surfaces.

The Contractor placed Mr. Frank Jones, one of his more experienced men, in charge of the drilling and blasting. Instructions were given that all of the work would be done on the day shift under the supervision of this one man (for consistency in the results), and that extra effort was to be expended in the drilling. This effort had the effect of producing recognizably greater precision in the Chamber drilling than in that done in the Peachtree Center Station main cavern and Running Tunnels; these results illustrate the importance of the human element in controlling drilling and blasting procedures.

Agreement was obtained to expend a limited effort to evaluate fracture-control blasting in parts of the rounds. This method involves the scribing or notching of drill holes to enhance crack propagation in the desired plane, and to minimize this propagation in other directions. It would have been desirable to use extremely high-pressure water jets to slot the perimeter holes, but funds were not available for this extra effort. Instead, limited work was done with mechanical scribing.

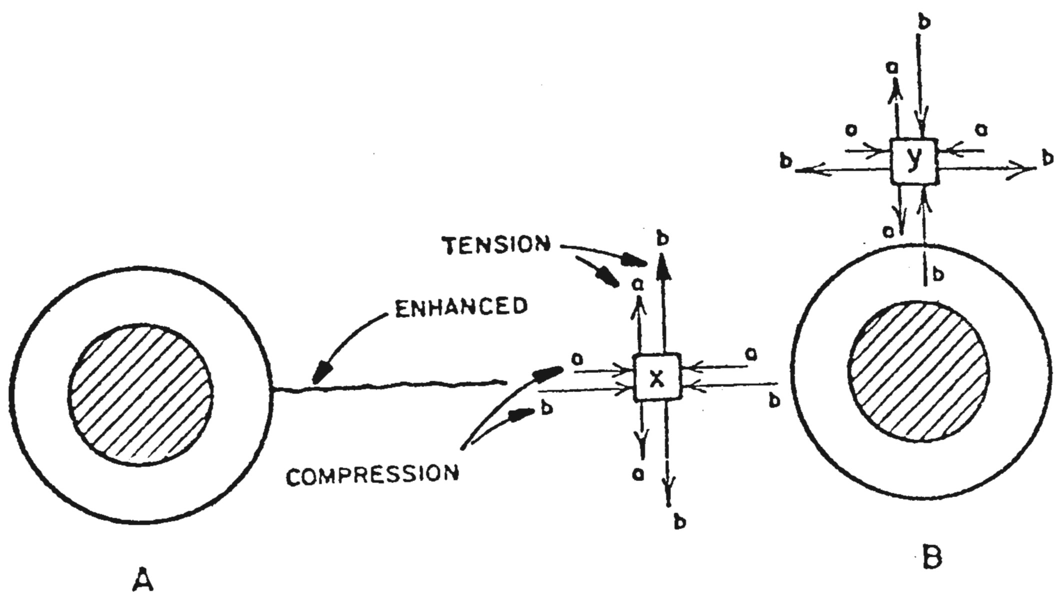
D. CONTROLLED PERIMETER BLASTING METHODS

If an explosive charge is in direct contact with the walls of the drill hole, it is said to be fully coupled to the rock. When such a charge is detonated, the rock is subjected to a very-high-pressure shock wave which pulverizes the rock for a short distance. In addition, radial cracks are extended for some distance into the rock wall.

In order to control these unwanted effects, cautious perimeter blasting methods are employed. In the method called pre-splitting or pre-shearing, the perimeter charges are detonated ahead of the main part of the round. Small-diameter cartridges of explosives are placed in a larger-diameter hole so as to de-couple the charges from contact with the rock. This reduces the pressure against the rock surface and eliminates the shattering. Figure VI-3 illustrates the manner in which the stresses are developed to enhance the desired crack growth. Decoupled charges are detonated simultaneously in adjacent bore holes. At the location of rock particle "y", both compressive and tensile stresses from the two charges oppose one another and no cracks develop. However, at location "x" in a line between the two holes, both compressive and tensile stresses are enhanced. However, the compressive stresses are reduced by decoupling the charges, so that no compression damage is done. At the same time, rock is much weaker in tension than in compression, by a factor from 10 to 50 times. Therefore, enough explosive can be used to develop a tensile crack in the web between the holes, while keeping the charges small enough not to cause compression damage.

If the perimeter charges are detonated after the remainder of the round, rather than before, the technique is called smooth blasting or cushion blasting.

When using the above-described techniques, it is common practice to use an explosive concentration of approximately 2.0-2.4 kg/m$^2$ (0.08 to 0.10 pounds of explosive for each square foot) of perimeter surface. Cartridges weighing 1.8 kg/meter (0.25




STRESS DISTRIBUTION FOR PRE-SPLITTING
AND SMOOTH BLASTING.

FIGURE VI-3

PHOTO NO. VI-1

PHOTO NO.VI-2

pounds per lineal foot) are often placed in holes that are spaced from 45 centimeters to 76 centimeters (18 inches to 30 inches) apart, depending on the rock characteristics. A small additional charge is often placed in the bottom of the hole, and it is customary practice either to place several feet of sand stemming in the collar zone of the hole, or to leave it open (in some underground work).

In theory, there is a certain fracture strength to a given material, and a fracture will propagate when stresses exceed that strength. In practice, there is a noticeable difference in fracture strengths in any given rock according to the sample size and its confinement. In addition to the strength of the rock material, the size and shape offer additional resistance of a structural nature that might be called the "beam" strength, offering still more resistance if the material is heavily confined. For example, the laboratory rock sample will break more readily than the in-situ block confined in a semi-infinite mass.

The presence of a flaw enhances crack propagation, just as the scribing of a pane of glass enhances its breakage along the scribed line. Any existing stress concentrator or weakness in the rock will enhance crack propagation, as will any pre-existing in-situ stresses. Less energy is required to start a fracture along an existing flaw, and less energy is required to extend an existing crack than to develop a new one. Therefore, there is a natural tendency to extend only one or a few long cracks, rather than develop a larger number of small equal cracks.

It is possible to make use of these principles in fracture-control blasting by scribing or notching opposite walls of a drill hole in the plane of the desired crack. One desirable way of doing this is by using high-pressure water jets to cut narrow slots which resemble open joints or cracks. Another option is to cut vee-shaped notches with a mechanical tool driven into the hole after it has been drilled. The latter option, which was used for the Atlanta Research Chamber, requires very little preparation, but requires precise execution for the best results. The mechanical tools used on this project did not produce optimum depth and shape of notches, because they were not carefully matched against the bits used for drilling the original holes. Hence, the results were less than optimum. However, work simultaneously conducted at two other sites indicated that when properly applied the method is successful under appropriate conditions. The experience at other sites suggests that properly executed scribing permits moving the drill pattern to much wider spacings between holes, and can reduce the quantity of necessary explosives to about 1/5 of the normal concentration. At this site, excellent results were obtained without fracture-control blasting (see Photo VI-2). Consequently, experiments with the notching method were used as demonstration rather than to solve a particular problem. It was concluded that closer control over the work would have been required to produce optimum results.

## E. MECHANICAL SCRIBING TOOLS

Photos VI-3 and VI-4 show the mechanical scribing tool first used by the Contractor. It consists of an integral bit on 4-sided drill steel to which has been added a jig or template to prevent the steel from rotating as the tool is driven into the drill hole. The template rides along the drill boom as the steel goes down the hole. The bit has been re-shaped to provide for easy penetration and withdrawal from the drill hole.

It can be seen in Photo VI-4 that the tool has no means of keeping the bit centered in the hole. This would not seem to have any significance in a homogenous rock type, but turned out to be unsatisfactory at this site beause of the anisotropy of the rock. The rock was foliated and consisted of alternating bands of softer and harder material. Consequently, the tool would often show a tendency to scribe only the softer side of the hole.

This writer suggested changing the bit to a type that he was using for similar work at another site. An over-size bit was ground down to accomodate the original drill hole. It was self-centering, and thus produced a more uniform set of notches. Such

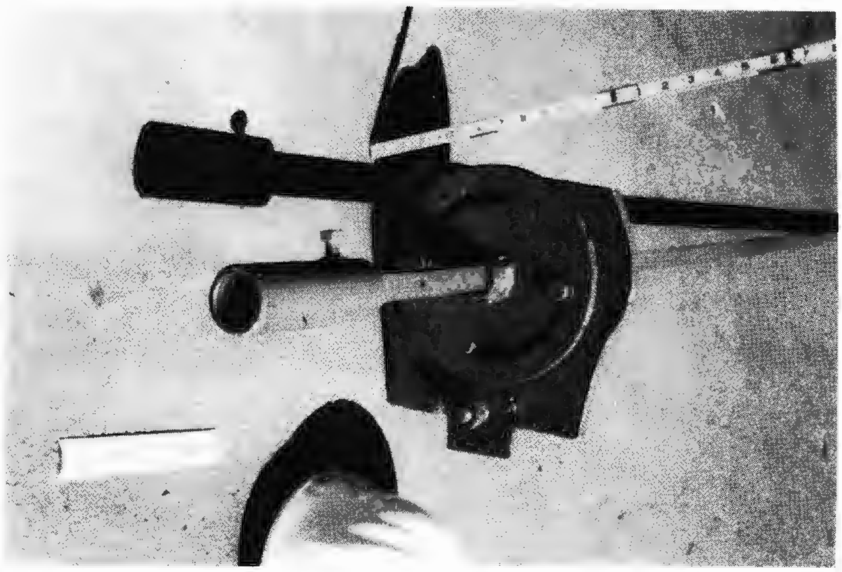


PHOTO NO. VI-3

PHOTO NO. VI-4

PHOTO NO. VI-5

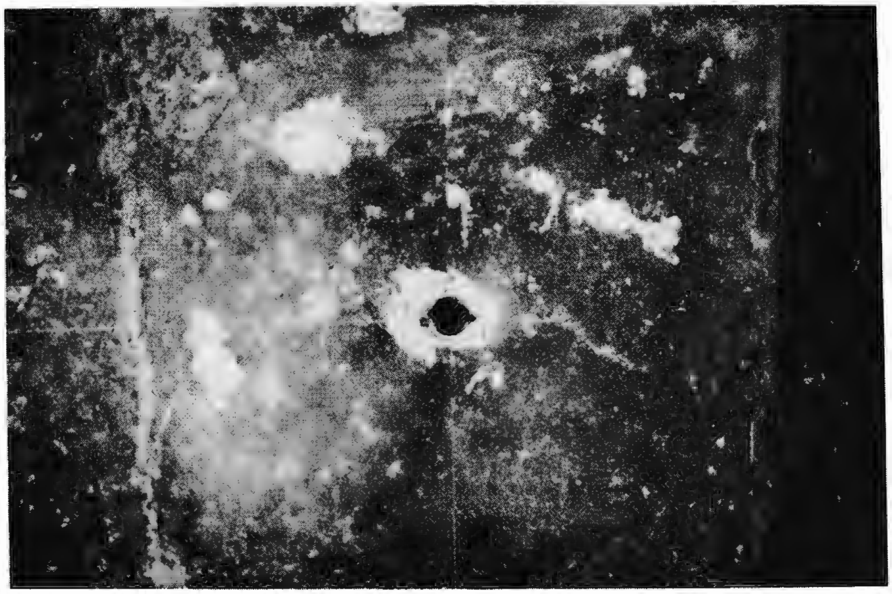


PHOTO NO. VI-6

PHOTO NO. VI-7

PHOTO NO. VI-8

a bit is shown in Photo VI-5. It had been recommended that the bit be of such size that notches would be cut to a depth of 0.6 centimeters (1/4 inch) on each wall of the drill hole. Although this writer did not see the tool in use, it was reported that notches cut by it were cut only to a depth of about 0.3 centimeters (1/8 inch), often less. That depth is considered too shallow to produce the optimum results. Photo VI-6 illustrates the appearance of a typical notched hole (the photo shows a hole notched in concrete, not at this site).

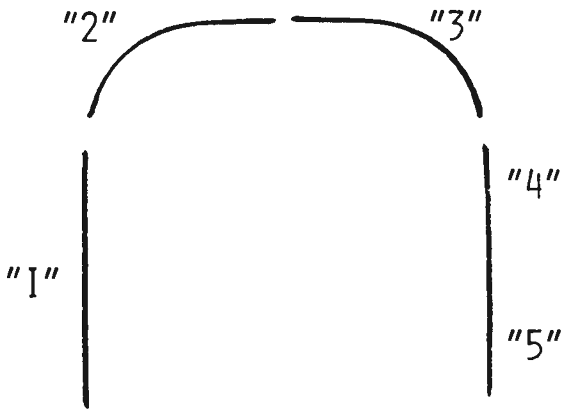
## E. BLASTING TESTS AND RESULTS

Sketches and notes chronologically showing details of the perimeter blasting from Panel 9 through Panel 1 are presented on Figure VI-4 through VI-11 .

Panel 9 was shot as a two-stage round using smooth blasting as the perimeter technique. Photo VI-7 shows the Pilot Tunnel into the Research Chamber, with the rock face marked for drilling Panel 9. The Contractor inadvertently drilled the perimeter holes 4.4 centimeters (1-3/4 inches) in diameter, which was too large to use the scribing tool effectively. Therefore, no scribing was done for this panel. However, it provided results that were useful for comparison. Photo VI-2 shows the left wall of Panel 9. Holes were 2.4 meters deep. It was sufficient to place three pieces of Tovex T-1 at 1.8 kg/m (0.25 pounds per foot), for a total length of 1 meter of T-1, spaced along the hole. Adequate breakage occurred. On the other hand, where 1.5 meters of T-1 and 1.8 meters of T-1 had been used, we can see acceptable results, also. The drill casts are almost perfectly preserved throughout. This shows that there is a fair amount of leeway when blasting in sound, durable rock. These holes were slightly larger than those in subsequent rounds, and showed the best exposure of drill casts. This demonstrates the benefit of de-coupling the charges, i.e., there was less shattering of the final wall surface. Holes were spaced at 46 centimeters (18 inches) for this shot.

PANEL 9

TWO-STAGE ROUND.
SMOOTH BLASTING WITH
NO SCRIBING.
PERIMETER HOLES;
SPACING = 18 INCHES.
BURDEN = 24 INCHES (UP TO
32 INCHES IN CROWN)
DIAMETER = 1 3/4 INCHES.
DEPTH = 8 FEET.




DELAY "1"

3½ FT. T-1 PLUS 100 GRAINS/FT. PRIMACORD

DELAY "2"

5 FT. T-1 PLUS 100 GRAINS/FT. PRIMACORD

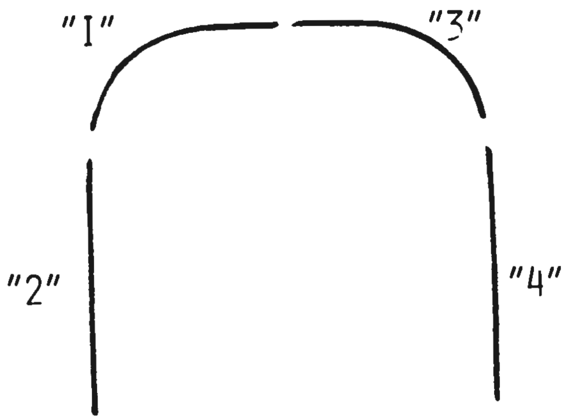
DELAYS "3", "4", "5".

6 FT. T-1 PLUS 100 GRAINS/FT. PRIMACORD

ALL HOLES STEMMED.
PRIMACORD TRUNKLINE USED FOR EACH DELAY GROUP
TO DEVELOP SIMULTANEOUS DETONATION
RESULTS - EXCELLENT.

FIGURE VI-4

PANEL 8




TWO-STAGE ROUND.
SMOOTH BLASTING WITH
EAST HALF SCRIBED.
PERIMETER HOLES:
- SPACING = 18 INCHES.
- BURDEN = 30 INCHES.
- DIAMETER = 1½ INCHES.
- SCRIBED TO 2 INCHES FOR DELAY GROUPS "1" AND "2".
- DEPTH = 7 FEET.

DELAYS "1" AND "3"

3 FT. T-1 PLUS 50 GRAINS/FT. PRIMACORD

DELAY "2"

6 INCHES T-1 PLUS 50 GRAINS/FT. PRIMACORD

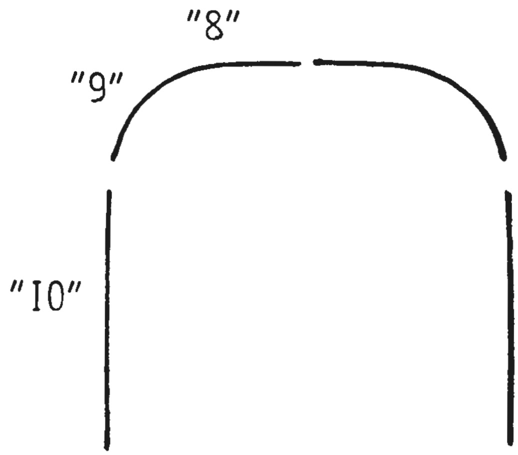
DELAY "4"

2 FT. T-1 PLUS 50 GRAINS/FT. PRIMACORD

ALL HOLES STEMMED.
PRIMACORD TRUNKLINE USED FOR EACH DELAY GROUP TO DEVELOP SIMULTANEOUS DETONATION
RESULTS:
- "1" AND "3" - GOOD.
- "2" - LOADING INSTRUCTIONS WERE NOT FOLLOWED. ONLY 6 INCHES T-1 USED IN BOTTOM. BOTTOM BROKE OUT, LEAVING 5 FT. COLLAR.
- "4" - FRACTURED BUT NOT REMOVED.
- CLEAN-UP SHOT WITH 5 FT. T-1 IN "2" AND "4".

FIGURE VI-5

PANEL 7




FULL-FACE ROUND.
PERIMETER FIRED ON LAST DELAYS.
EAST HALF SCRIBED.
PERIMETER HOLES:
- SPACING = 24 INCHES.
- BURDEN = 30 INCHES.
- DIAMETER= 1½ INCHES.
- SCRIBED HOLES "8", "9", "10"
- SCRIBING DESCRIBED AS POOR, - LESS THAN 1/8 IN. NOTCHES.
- DEPTH = 7 FEET.

DELAYS "8" AND "9"

3 FT. T-1 PLUS 100 GRAINS/FT. PRIMACORD

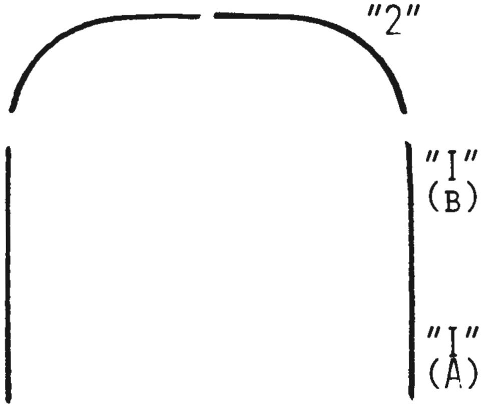
DELAY "10"

2 FT. T-1 PLUS 100 GRAINS/FT. PRIMACORD

HOLES THOUGHT TO BE STEMMED - NOT VERIFIED IN FIELD LOGS.
PRIMACORD TRUNKLINES USED.
RESULTS:
- DESPITE POOR QUALITY OF SCRIBING, THE PERIMETER BROKE, ALTHOUGH SURFACE WAS ROUGH.
- MISFIRES OCCURRED IN OTHER PORTIONS OF THE ROUND. CLEAN-UP REQUIRED.

FIGURE VI-6

PANEL 6




TWO-STAGE ROUND.

SMOOTH BLASTING WITH
WEST HALF SCRIBED.

PERIMETER HOLES:

SPACING = 30 IN. FOR "1"
= 36 IN. FOR "2"

BURDEN = 36 IN. FOR "1"
= 18 IN. FOR "2"

HOLES "1" AND "2" WERE SCRIBED, BUT NOTCHES NOT WELL DEFINED.

DEPTH = 6 FT.

DELAY "1"(A)

3 FT. T-1 PLUS 50 GRAINS/FT. PRIMACORD

DELAY "1"(B) AND "2"

4 FT. T-1

ALL HOLES STEMMED.

NON-ELECTRIC CAP IN EACH HOLE WITH 50 GRAIN PRIMACORD TRUNKLINE FOR EACH DELAY GROUP.

RESULTS:

HOLES "1"(A) DID NOT BREAK WELL IN BOTTOM, LEAVING A STUB.

HOLES "1"(B) AND "2" BROKE TO FINAL LINE.

FIGURE VI-7

PANEL 5A AND 5B

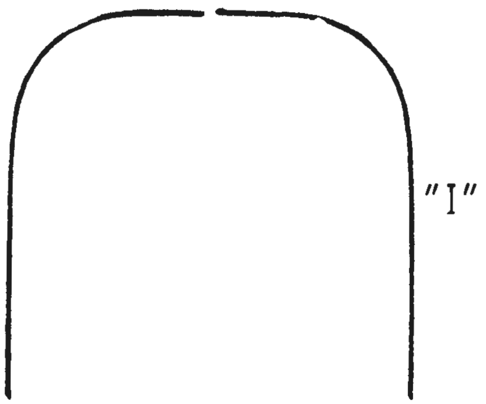
5A - TWO-STAGE ROUND.
SMOOTH BLASTING.

5B - PERIMETER SHOT AS PRE-SPLIT.

WEST HALF OF BOTH ROUNDS SCRIBED.

PERIMETER HOLES:

SPACING = 30 INCHES.
BURDEN = 12 INCHES.
DEPTH = 6 FEET.




DELAY "I"

4 FT. T-I PLUS 50 GRAINS/FT. PRIMACORD

HOLES THOUGHT TO BE STEMMED.

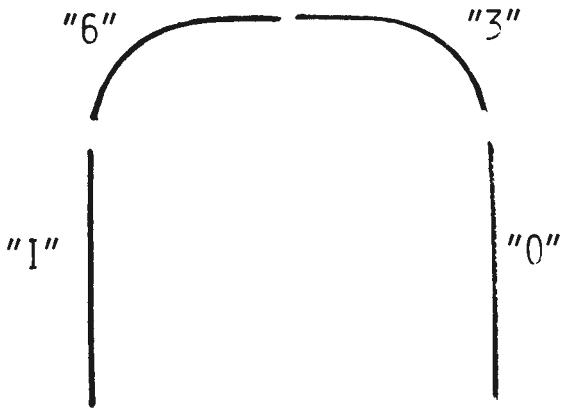
RESULTS:

5A: AS SMOOTH BLAST, CHARGES WERE SUFFICIENT TO BREAK.

5B: AS PRE-SPLIT, THE NEXT ROW (AT 12 IN.) WAS TOO FAR FROM PERIMETER TO REMOVE ALL ROCK. THE LOWER RIGHT QUADRANT BROKE WELL, BUT ROCK WAS LEFT IN THE UPPER RIGHT AND HAD TO BE REMOVED WITH A TRIM SHOT.

FIGURE VI-8

PANEL 4




PRE-SPLIT BLAST.
WEST HALF SCRIBED.
PERIMETER HOLES:
SPACING = 30 INCHES.
BURDEN = 15 INCHES.
DEPTH = 8 FEET.
HOLES "0" AND "3" SCRIBED.

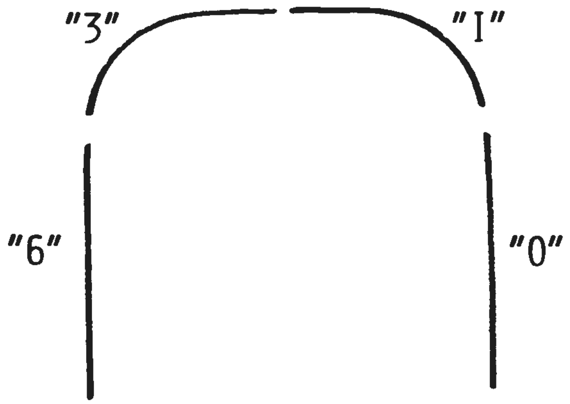
DELAY "0" - 150 GRAINS/FT. PRIMACORD, IN SCRIBED HOLES.
DELAY "3" - 300 GRAINS/FT. PRIMACORD, IN SCRIBED HOLES.
DELAY "6" - 450 GRAINS/FT. PRIMACORD, IN UNSCRIBED HOLES.
DELAY "I" - FOUR ONE-FT. SECTIONS OF TOVEX T-I WITH ALTERNATE ONE-FT. SPACERS, IN UNSCRIBED HOLES.

CHARGES IN SCRIBED HOLES WERE CENTERED WITH WIRE HOLDERS, AND HOLES WERE STEMMED WITH DRY-PACK OF SAND AND CEMENT.

RESULTS:
WEST SIDE DID NOT FRACTURE.
HOLES "6" FRACTURED BUT HELD TO THE PERIMETER WALL.
HOLES "I" GAVE A CLEAN BREAK.

FIGURE VI-9

PANEL 3




PRE-SPLIT BLAST.
WEST HALF SCRIBED.
PERIMETER HOLES:
- SPACING = 30 INCHES.
- BURDEN = 6 INCHES.
- DEPTH = 10 FEET.
- HOLES "0" AND "1" WERE SCRIBED, BUT THE INSPECTOR'S NOTES INDICATE THAT THERE WAS A DIFFICULTY WITH THE SCRIBING TOOL ROTATING.

THIS PANEL WAS SHOT IN THE SAME MANNER AS PANEL 4, EXCEPT THAT THE BURDEN WAS REDUCED TO 6 INCHES.

DELAY "0" - 150 GRAINS/FT. PRIMACORD IN SCRIBED HOLES.
DELAY "1" - 300 GRAINS/FT. PRIMACORD IN SCRIBED HOLES.
DELAY "3" - 450 GRAINS/FT. PRIMACORD IN UNSCRIBED HOLES.
DELAY "6" - FIVE ONE-FT. SECTIONS OF TOVEX T-1, WITH ALTERNATE ONE-FT. SPACERS, IN UNSCRIBED HOLES.

RESULTS:

THE HOLES LOADED ONLY WITH PRIMACORD DID NOT SHOW A FRACTURE. THESE HOLES HAD TO BE RE-SHOT, AND PRODUCED A RELATIVELY ROUGH FINAL SURFACE.

THE HOLES LOADED WITH T-1 BROKE WELL.

FIGURE VI-10

## PANEL 2

NO SCRIBING OF HOLES.

SHOT AS SMOOTH BLAST.

SPACING AND BURDEN = 30 INCHES.

DEPTH OF ROUND = 10 FEET.

SIMULTANEOUS DETONATION OF PERIMETER HOLES.

PERIMETER HOLES LOADED WITH A 7-FT. COLUMN OF TOVEX T-I.

RESULT:

THE PERIMETER BROKE WELL, BUT FEW DRILL CASTS ARE SEEN.

## PANEL 1

NO SCRIBING OF HOLES.

SHOT AS SMOOTH BLAST.

SPACING AND BURDEN = 30 INCHES.

DEPTH OF ROUND WAS VARIABLE TO ACCOMMODATE THE TERMINATION OF THE ADIT.

SIMULTANEOUS DETONATION OF PERIMETER HOLES.

PERIMETER HOLES LOADED WITH A 7-FT. COLUMN OF TOVEX T-I.

RESULT:

A ROUGH SURFACE WITH FEW DRILL CASTS.

FIGURE VI-11

Panel 8 was shot as a two-stage round using smooth blasting for the perimeter. Holes were 2.1 meters (7 feet) deep, spaced at 46 centimeters (18 inches). Three 30-centimeter (1 foot) sections of explosive at 1.8 kg/m (0.25 lbs. per foot) were successful in breaking the upper ribs and the crown section, whether the holes were scribed or not.

Panel 7 was shot as a full-face round with perimeter holes firing on the last delay groups. Holes were 2.1 meters deep, spaced at 60 centimeters (24 inches). The inspector's field notes indicate that the scribing was poor, giving notches less than 0.3 centimeters (1/8 inch) deep. Charges consisted of three 30-centimeter (1 foot) sections of T-1 in part of the holes, and two 30 centimeter sections in part, plus 100 grains of Primacord per 30 centimeters (foot). Despite the poor scribing, the perimeter broke, although the surface was rough. It seems doubtful that the two 30-centimeter charges of T-1 would have been sufficient to break the rock without the help of the scribing.

Panel 6 was shot as a two-stage round, using smooth blasting for the perimeter. The west half was scribed, although the notches were not well-defined (according to the inspector's field notes). The holes were 1.8 meters deep, spaced at 76 centimeters (30 inches) for some, 91 centimeters (36 inches) for others. The charges were 0.9 meters (3 feet) of T-1 for part of the holes, 1.2 meters (4 feet) for the other. The holes containing 1.2 meters of T-1 broke to the final perimeter line. Those containing only 0.9 meters left a stub at the bottom of each hole. Thus, acceptable breakage occurred with a loading concentration of 2.2 kg (1 lb.) for 1.65 square meters (18 square feet) of perimeter surface, or 1.33 $kg/m^2$ (0.055 lbs. per square foot). This is less than normal loading and suggests that there was indeed a benefit even to poorly developed notches. It suggests, further, that the scribing permits placing the holes farther apart. It is doubtful that there would have been suitable breakage between holes in the vertical plane without some assistance from the scribing. Fractures in this plane were forced to develop perpendicular to the strong foliation in the rock. This is more difficult than breakage along the horizontal planes of foliation.

Panel 5 was shot in two rounds. Round 5A was a two-stage round using smooth blasting for the perimeter. Round 5B made use of pre-splitting for the perimeter. Holes were 1.8 meters (6 feet) deep. Spacing was 76 centimeters (30 inches). Two 61-centimeter (2 foot) sections of T-1 were used, plus 50 grains of Primacord per 30 centimeters (per foot) giving a concentration of 1.68 $kg/m^2$ (0.07 lbs. per square foot) of surface area. As a smooth blast, the charges were sufficient to break the rock. As a pre-split shot, a perimeter fracture developed, but the remainder of the blast did not succeed in removing rock out to the perimeter line afterwards.

Panel 4 was shot as a pre-split round. The west half of the perimeter was scribed. The perimeter holes were divided into four groups, each with different loading, using Primacord for the charges in three of the groups and T-1 in the fourth. The Primacord charges in the scribed holes were centered with the use of wire holders, as seen in Photo VI-8. The holes were stemmed with a dry pack of sand and cement. Centering the charges, and using dry pack, should have produced the optimum confinement of the explosive gasses and produced the best results for the given depth of notches. However, charges up to 300 grains per 30 centimeters (per foot) of Primacord did not generate a fracture in the scribed holes. Unscribed holes containing four 30-centimeter sections of T-1 with alternating 30-centimeter (1 foot) spacers gave suitable results. Thus, it appears that the pre-splitting technique does not permit as great a reduction of charge as does smooth blasting, when holes are scribed (as explained previously). No doubt this is due to the confinement of the surrounding rock. There is a "semi-infinite" confinement for pre-splitting, which is greatly reduced for smooth blasting.

Panel 3 was essentially a repeat of Panel 4. Similar results were obtained.

Panel 2 and Panel 1 were shot at the Contractor's option, using solid columns of T-1 in the perimeter holes. The results suggest that such a loading was too heavy, since few drill casts are visible in this area.

Photo VI-9 shows about 7.6 meters (25 feet) of the right (west) wall of the adit. In the left foreground are holes which had been scribed. In the right background are unscribed holes. Most of the casts can be seen for both. Both gave acceptable results. The light-colored bands in the photo are rock structure unrelated to blasting.

Photo VI-10 is a close view of a tight fold in the rock, as seen by the light and dark banding. It is possible to see in this photo how strongly the foliation influences the rock breakage. There is an over-all fracture trend in the vertical plane, from the drill hole downward. However, a close examination shows that the fracture is not linear, but is contorted along the lines of the rock fold. The fracture tends to follow a single band of foliation along a curved path around the fold.

## F. CONCLUSIONS

Results of blasting for the demonstration adit appeared to be superior to those for the Peachtree Center Station main cavern and the Running Tunnels. This is thought to be due to (1) greater effort applied to drilling control, and (2) more cautious blasting. It is this writer's opinion that the small additional effort for drilling control and a modest reduction in perimeter charges are measures that are easily taken and measures that reap considerable rewards.

Although the rock at this site is relatively sound and durable, it has foliation planes that are generally rather flat-lying. This permitted easier breakage across horizontal lines, such as the crown of the adit, and made more difficult the breakage to vertical surfaces such as the side walls. In some of the areas showing the greatest anisotropy, the influence of the foliation gave dramatic results. An example was that of a crack fol-

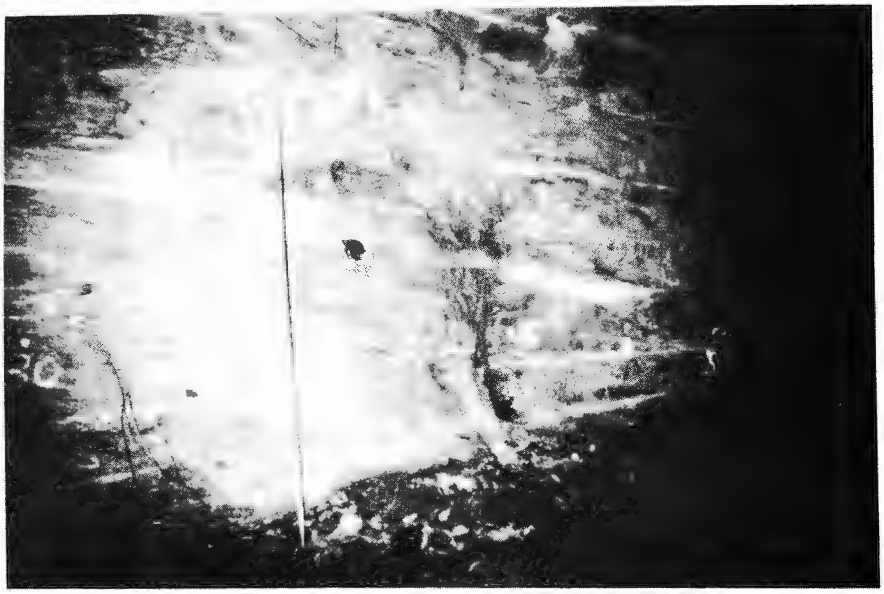


PHOTO NO. VI-9

PHOTO NO. VI-10

lowing the contorted banding around a tight fold in the rock rather than continuing in a straight line, (Photo 10).

Experiments with fracture-control blasting in the Atlanta Research Chamber were not executed with sufficient precision to give this method an adequate evaluation. More information is available from work at other sites. The limited work done here demonstrates that scribing of drill holes permits a reduction of explosives and greater spacing, but the work was not done with sufficient precision to quantify these factors. Work at other sites suggests that explosive charge quantities can be reduced to about 1/5 the normal load for smooth blasting, and/or holes spaced about twice as far apart, or both. Of course, this depends on how carefully the method is applied, and the characteristics of the rock. At the Atlanta Research Chamber site, the method would not appear to have a great deal of need for its application. One reason is that the rock is relatively sound and durable, enabling good results to be achieved with only a small amount of modification of normal procedures (which would be faster and less expensive than fracture-control methods). The second reason is that the anisotropic rock would not lend itself to great spacings between holes in a vertical plane, thus reducing the potential effectiveness of fracture-control methods.

Field observations showed that holes 2.4 meters (8 feet) deep could be scribed in about two minutes each, including positioning the drill boom. For a tunnel with about 32 perimeter holes, each 2.4 meters (8 feet) deep, spaced at 46 centimeters (18 inches), drilling might take 4 hours. If the perimeter hole spacing could be increased slightly to 60 centimeters (24 inches) by using the scribing tool, then only 24 holes would be required, taking about 3 hours to drill, plus about 48 minutes to scribe the 24 holes. Drilling time and related expenses thus would be reduced, and, as noted earlier, lighter exposives loading would be used. The method clearly has promise in suitable rock conditions.

It is this writer's opinion that fracture-control blasting is already shown to be an effective tool for delicate work in critical surroundings. With additional research and development, it can prove to be more cost effective on routine work than is presently the case.

CHAPTER VII

CONVENTIONAL SHOTCRETE

## 7.1 INTRODUCTION

Conventional shotcrete was placed in the Atlanta Research Chamber in Panels 5 and 7, as shown on Figures VII-1 and Figure II-3 (Chapter II). While not all of the planned shotcrete testing was actually accomplished, a great deal of information was in fact collected. The main purpose of placing the conventional shotcrete in the Atlanta Research Chamber was to field test new additives, and to systematically review the shotcrete specifications and procedures required in the MARTA Peachtree Center Station CN120 contract. It became apparent that the CN120 shotcrete specifications, as well as some other modern shotcrete specifications such as those presently in use in the construction of the Washington, D.C. Metro (WMATA), have evolved haphazardly from conventional concrete specifications and often include tests of inadequate reproducibility. Hence, after a review of the work done in the Atlanta Research Chamber on conventional shotcrete, it was decided that a number of recommended changes in shotcrete specifications should be made.

### 7.1.1 Shotcrete for Underground Construction

Shotcrete is a mixture of sand, gravel, cement and water which is projected with compressed air against a receiving surface. A proportion of the mixture (rebound), does not adhere to the surface, falls away and is wasted. Shotcrete hardens in place, thereby eliminating the formwork required for conventional cast-in-place concrete.

Shotcrete, even more than concrete, is a variable construction material. A number of factors which influence shotcrete quality, as indicated by its compressive strength, are the following:

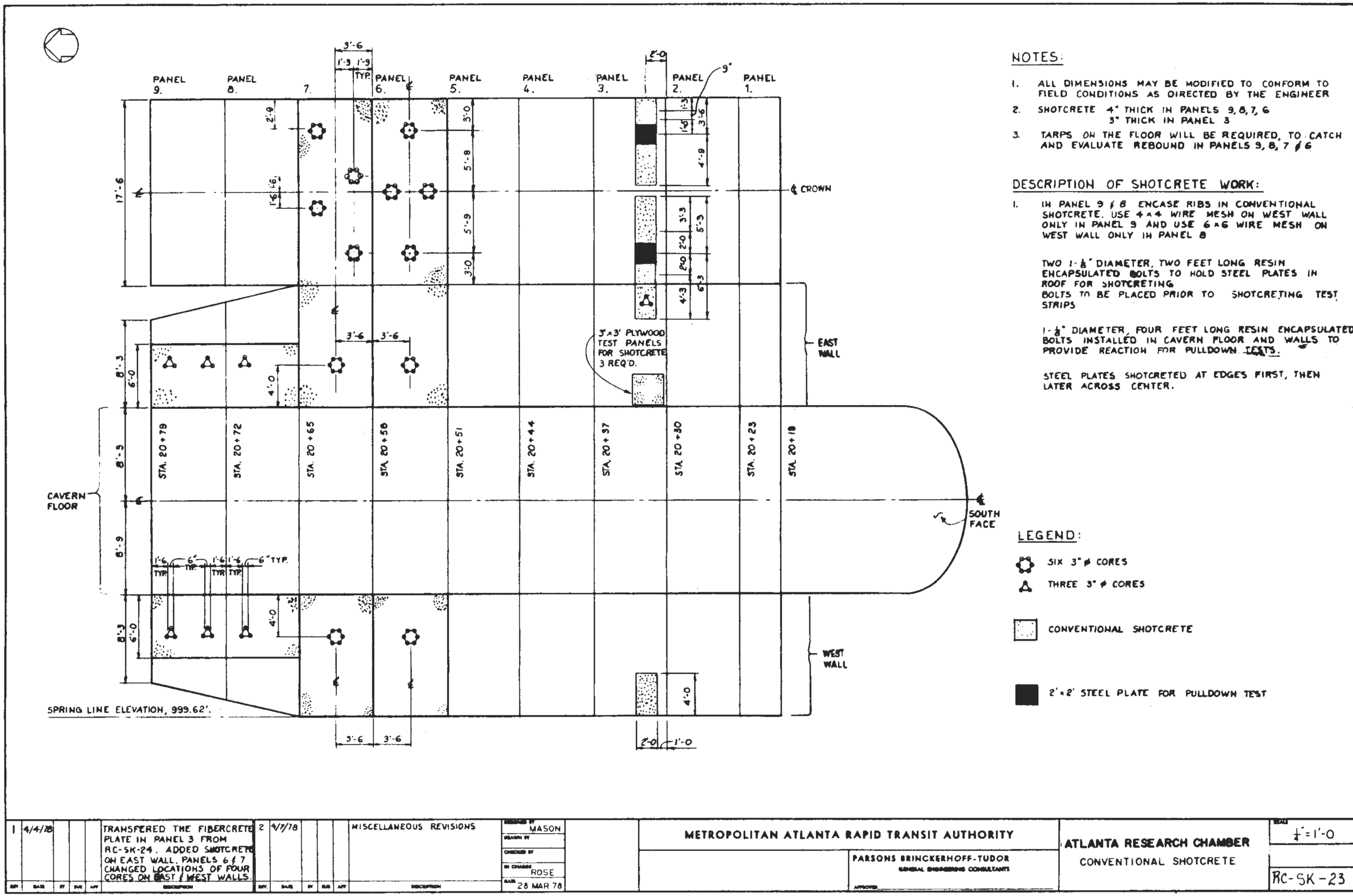



FIGURE VII-1 Placement of Conventional Shotcrete

1. Shotcrete is essentially a low slump concrete which has been consolidated upon impact. However, in order to remain in place without sloughing, shotcrete must set and harden rapidly. The required rate of hardening depends on the orientation of the surface on which it is placed, the thickness placed, the water content (slump) of the shotcrete, and presence of water flows on the placement surface.
2. In order to produce the necessary rapid set and hardening, accelerating-set admixtures (accelerators) are frequently used in shotcrete for overhead placement, in the presence of running water, at moderate to low temperatures, or a combination of all of these conditions, such as for typical underground work. While accelerators produce the required short-term benefits, in the long-term, conventional inorganic accelerators produce a strength loss, usually between 20 and 30 percent at 28 days, compared with unaccelerated shotcrete. Recently-developed organic accelerators are claimed to produce required rapid set and early hardening, while not affecting 28-day strengths.
3. The 28-day strength loss due to use of inorganic accelerators depends on several factors, such as the admixture dosage and admixture-cement compatibility. The required admixture dosage, in turn, depends on the reactivity of the cement due to its temperature and age; on accelerator reactivity; on the extent of pre-hydration which occurs when cement is added to moist aggregates; on the thoroughness of mixing; and on field conditions.
4. When spraying shotcrete directly overhead, higher accelerator dosages are normally used than are used when spraying vertical surfaces. Higher accelerator content decreases the setting time and prevents loss of adhesion and sloughing which might occur if less accelerator were used. For spraying vertical surfaces, accelerator

dosages are minimized, since the risk of sloughing is lower, and accelerator material costs are important.

5. The quality of shotcrete is affected by the quality of materials and by each operation in preparation and placement. Variation in shotcrete quality can result from variation in the following: moisture present in the aggregates; mixing of cement and aggregates; blending of dry mix and accelerator; water quantity; mixing of water and dry mix at the nozzle; material velocity and subsequent impact compaction; and angle of application to the rock. For overhead spraying, the nozzleman suffers considerable discomfort from rebound, which tends to adversely affect quality of workmanship. In addition, the material velocity is frequently decreased by lowering air pressure or by increasing nozzle distance from the placement surface, to relieve discomfort from rebound.

### 7.1.2 Shotcrete Quality Control

Due to its manner of placement and the common use of accelerators, the problems of sampling shotcrete are unique and separate from those related to conventional cast-in-place concrete. Shotcrete is most effectively sampled by removing cores from hardened test panels or from in-situ tunnel lining. Earlier sampling methods, such as spraying shotcrete into cylinder molds or wire mesh baskets, are no longer considered to give satisfactory results since the self-sorting action of shotcrete placement, producing in-place shotcrete plus rebound, is not represented.

Sampling of shotcrete by cutting cubes from test panels or from large blocks of shotcrete cut from the in-situ tunnel lining is convenient and representative, but results are difficult to correlate with other concrete or shotcrete sampling methods.

Shotcrete quality control testing frequently yields unsatisfactory results due to a lack of understanding about the product itself and testing procedures. A summary of potential problem areas is given below:

1. The basic concrete components, aggregates and cement, may not have the potential to produce a high quality shotcrete.
2. Cement and accelerating-set admixtures (accelerators), used to produce rapid hardening, may not produce desired initial set and rapid hardening under field conditions, and if they do, an unacceptable strength loss in the shotcrete may result.
3. Sampling shotcrete is a problem much different from casting standard test cylinders for concrete, and many of the problems of shotcrete quality are based upon poor sampling technique or misinterpretation of test results.
4. Due to several factors, shotcrete placed overhead, for example in an underground opening, frequently exhibits lower strength than that placed on vertical surfaces like tunnel sidewalls, or test panels. This factor is commonly ignored in sampling.
5. Equipment and personnel may not be suitable for high quality shotcrete placement underground.

### 7.1.3 Shotcrete Quality Control Specifications

Quality control specifications were primitive for the first civil application of accelerated-set shotcrete in North America, at the Canadian National Railways Burnaby-Vancouver Tunnel in British Columbia. However, quality control was maintained by testing three-inch cubes from test panels during preconstruction testing, and from shotcrete slabs removed from the tunnel arch during construction. Since the cube strengths were substantially above specified compressive strength of 4,000 psi, and far above measured in-situ shotcrete stresses, shotcrete quality was deemed satisfactory and the initial support lining was approved to serve as the final lining as well.[1]

Difficulties in achieving specified strengths at the New Melones tunnels in California and Nanticoke Cooling Water Tunnel, Ontario indicated that producing high quality shotcrete was not a simple matter. For the Washington Metropolian Area Transit Authority (WMATA), detailed quality control specifications were prepared and first used in 1969 on Sections A-4a, and subsequently on A-4b, and C-5. Preconstruction testing was specified and three-inch diameter cores were tested for quality control during construction. Statistical requirements for data interpretation, from ASTM designation C94, were introduced.

WMATA Standard Specifications, published in 1973, offered the alternative of testing three-inch cores or cubes for preconstruction testing. In addition, the concept of coefficient of variation, and its effect upon required overdesign factors was introduced. Due to the statistical variation, core strengths exceeding the design strength, f'c, were required for preconstruction testing.

The U.S. Bureau of Reclamation followed a course parallel to that of WMATA, deriving their own quality control specifications from experience gained in a number of tunnels.[2] In addition, the Bureau performed laboratory and field studies on shotcrete[3], to enlarge its knowledge of shotcrete properties. It is standard Bureau quality control practice to utilize either

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1/ E. E. Mason, "The Function of Shotcrete in Support and Lining of the Vancouver Railway Tunnel;" in Rapid Excavation, Problems and Progress, edited by Donald M. Yardley, The American Institute of Mining Metalurgical and Petroleum Engineers, Inc., 1970.

2/ U.S. Bureau of Reclamation, Engineering and Research Center, "Use of shotcrete for tunnel lining"; Contract Report S-76-4, State-of-the-Art Review on Shotcrete, Published by U.S. Army Engineer Waterways Experiment Station, 1976.

3/ T. Rutenbeck, "Shotcrete Strength Testing - Comparing Results of Various Specimens" in Shotcrete for Ground Support, ACI Publication SP-54, Engineering Foundation Conference, 1976.

4-inch cubes or 2-1/8-inch cores from test panels for preconstruction testing, and 2-1/8-inch cores from in-place shotcrete for testing during construction.

The American Concrete Institute (ACI) also recommends preconstruction testing in their latest standard specification, ACI 506.2-77, by testing 3-inch diameter cores or 3-inch cubes. Testing during construction may be of cores taken from in-place shotcrete, or from test panels, as determined necessary by the Engineer. In ACI 506.2-77, recommended preconstruction testing includes submittal of cube or core samples from one test panel per mix. Average core strengths for preconstruction testing must equal f'c. ACI 506.2-77 specifications apply to all forms of shotcreting, but contain the disclaimer that they may not be applicable to shotcrete used for underground structural support.

The MARTA Peachtree Center Station CN120 contract, in which the Atlanta Research Cavern is included, contains a number of innovations in shotcrete specifications which were also included in the revised WMATA Standard Specifications for Section A-11b, in 1977. The procedure outlined for shotcrete preconstruction testing includes a test for cement and aggregate quality, utilizing standard concrete cylinders, and a statistically meaningful series of core testing from test panels. Other requirements, such as closely specified compatibility setting times, were introduced in the CN120 specifications. The required core strength for samples taken during construction is less than the design strength f'c, in accordance with ACI Standard 318-71, updated for shotcrete in ACI 506.2-77. The CN120 specifications for shotcrete are included in the Appendix.

### 7.1.4 Purpose of Conventional Shotcrete Testing in the Atlanta Research Chamber

From the foregoing section on quality control specifications, it may be apparent that there are important differences in quality control procedures among different agencies. The result is that at the present time test results, or quality standards, are not comparable from project to project. One of the purposes of the present research effort is to test the practicality and effectiveness of existing specifications and quality control procedures and to recommend changes which would improve their effectiveness.

For the Atlanta Research Chamber, a full preconstruction test program was undertaken, and shotcrete placed in the Research Chamber walls and arch under realistic underground construction conditions. Test cores taken from the Research Chamber walls and arch duplicate those removed in normal construction conditions.

For high quality shotcrete, organic accelerators have been claimed to eliminate the 28-day strength loss incurred by conventional inorganic accelerators. To test this fairly new product under controlled field conditions, an organic accelerator was used on a portion of the shotcrete placed in the Research Chamber. In addition, this organic accelerator was included throughout the preconstruction testing sequence.

## 7.2 BACKGROUND INFORMATION

### 7.2.1 Testing of Shotcrete

Due to its manner of placement and the common use of an accelerator, shotcrete presents problems in sampling that are unique and separate from those related to cast concrete. Generally, structural requirements demand an in-situ shotcrete strength equivalent to that of concrete whose standard test cylinders meet requirements for specified design strengths, f'c, such as 5,000 psi. The actual test values of shotcrete samples would be quite different from the compressive strengths of complying standard concrete cylinders. This difference is outlined below:

1. The design strength, f'c, is used as a basis of design, by which some fraction of f'c is utilized to carry an assumed loading condition.

2. For cast-in-place concrete, the in-situ concrete is said to meet or exceed the design strength if samples separately cast and cured in a standardized manner comply with given requirements. For preconstruction testing, the average of standard cylinder compressive tests must generally exceed the specified strength by an amount depending on the number of samples taken and the variability of concrete production. The variability of the concrete production, or coefficient of variation, depends on uniformity of materials, precision of batching, quality of mixing and placement. The larger the sample size or number of cylinders tested, and the lower the coefficient of variation or greater uniformity of production, the nearer the average strength of samples can be to f'c and still comply with requirements.
3. Standard cast cylinders are 6 inches diameter and 12 inches high made to specified ASTM requirements. If cylinders of less than 12 inches height are tested, these vary in strength from the standard sized cylinders in accordance with the length to diameter ratio (L/D).
4. Shotcrete is most effectively and conveniently sampled by either removing drilled cores or sawn cubes from test panels or insitu. However, drilled <u>concrete</u> cores have been shown to exhibit lower strengths than cast cylinders because of internal damage incurred during coring, and because of sample size differences. Sawn cubes exhibit different strengths from cores and cast cylinders because of their different configuration.
5. If shotcrete cores are used for sampling, these are generally of three-inch diameter for minus three-quarter inch maximum aggregate size. To obtain cores with a L/D ratio of two requires a core longer than six inches, in order to permit sawing the ends. Shotcrete of such thickness is infrequently used, so commonly shorter cores are removed and tested.

The result of the foregoing may be illustrated by the present formula for shotcrete sampling in WMATA A-11b. For a specified in-situ strength, f'c, of 5,000 psi, preconstruction testing of cores must achieve an average compressive strength of 5,540 psi at 28-day age. This test average includes correction factors for comparing cores to cast cylinders, for L/D ratio of cores, for sample size, and for an assumed coefficient of variation.

The average compressive strength is calculated as follows:

1. Required design in-situ strength is 5,000 psi.
2. According to ACI 506.2-77, required core strength for acceptance is 0.85 of design strength 5,000 psi, or 4,250 psi for cores with L/D = 2. Use 4,250 psi as f'c in equation to follow.
3. To determine the overdesign factor required, the following formula from the proposed revision to ACI 214-65 Title No. 73-22 is utilized.

$$f_{cr} = \frac{f'c}{(1 - tV)}$$

where

$f_{cr}$ = average required strength

$f'c$ = design strength specified

$t$ = a constant depending upon the proportion of tests that may fall below f'c (Table 4.1 ACI 214-65)

$V$ = forecast value of the coefficient of variation expressed as a fraction

For concrete designed by working stress methods, ACI 214-65 recommends six tests be averaged to provide assurance that only two percent of the samples will fall below f'c if the mix is designed for $f_{cr}$, the average required strength. To comply with the above, six tests or panels, comprised of three cores each, should be made and tested, to comply with an assumed $f_{cr}$. To estimate the required $f_{cr}$, the above formula may be applied. Utilizing an assumed coefficient of variation of 17 percent, "t" is equal to 0.92 (Table 4, ACI 214-65), and f'c equal to 4,250 psi (0.85 times 5,000 psi), the average required strength is 5,040 psi, for cores corrected for L/D. It should be noted that higher strengths would be required for ultimate strength design.

4. If it is assumed that ASTM C42, Article 5.17 applies to shotcrete cores, a correction of 0.91 is used for correction of L/D. Applying this correction produces an average required core strength of 5,540 psi for 3-inch diameter, 3-inch long cores.

Testing of shotcrete during construction for present WMATA projects is in accordance with ACI 506.2-77, which requires three, 3-inch diameter cores for each sample of the structure in question. The average strength of these cores, when corrected for L/D, must equal or exceed 85 percent of required strength f'c (5,000 psi), with no single core less than 75%. By adjusting these values for L/D equal to one, a required average strength of 3-inch long 3-inch diameter cores is 4,670 psi with 4,120 psi minimum strength.

CN120 specifications take a different approach by requiring a "specified strength" of 4000 psi for shotcrete cores, three inches in diameter and three inches long, with L/D corrections applied. By the logic applied above, this would result in an in-situ strength at 28 days of 4000 ÷ .85 = 4706 psi, corresponding to standard cast cylinders.

CN120 testing of shotcrete during construction specifies that 85 percent of the "specified strength" be achieved by three-by-three-inch cores removed from the lining. Apparently a strength of 4000 x .85 = 3400 psi is intended. This would, however, represent an in-situ strength of 4000 psi corresponding to standard cast cylinders, not to 4706 psi as calculated above.

### 7.2.2 Necessity for Shotcrete Quality Control

Shotcrete in tunnels may be used either for initial support and final lining or as initial support only, with a final lining placed inside of the shotcrete layer. In the former case, some rational structural design method is frequently used, unless the shotcrete has merely a protective rather than structural role. If a structural design is utilized, the designer must use some portion of the ultimate shotcrete strength, f'c, as an allowable stress. He therefore must be assured that field shotcrete strengths will not fall below assumed ultimate stress, within the limits of statistical probability. For permanent structures in public use, such as subway stations, this assurance is essential.

On the other hand, the U.S. contractual climate favors the use of shotcrete for initial support only, to be installed at the contractor's option, and at his discretion. If such is the case, it may be counterproductive to the owner's interests to insist upon complying with stringent preconstruction testing and quality control specifications. A prudent contractor might decide to avoid such inconvenience and utilize more expensive steel rib supports, to neither party's advantage. In any case, the contractor is responsible for the safety of his work, so the quality of shotcrete used for initial support may not be of critical concern to the owner.

In other cases, shotcrete may be used as a protective and semi-structural final lining, for which a rational structural calculation is difficult to justify. In shotcrete-lined water, utility and hydroelectric diversion tunnels, for example, high compressive strengths may be unimportant. In such cases, overall quality of workmanship may be of greater importance, to minimize construction defects where local failure could occur. Quality control specifications should therefore reflect the need for product uniformity.

The differing needs of shotcrete applications explains the widely varying specifications but should not deter their rational conformance. For semi-structural linings, for example, product uniformity is required, and can be monitored by statistical sampling similar to that required for a structural design.

## 7.3 PRECONSTRUCTION TESTING FOR THE ATLANTA RESEARCH CHAMBER

### 7.3.1 Intent

The purpose of the preconstruction testing was twofold: firstly, to ensure that shotcrete made with different additives placed in the Atlanta Research Chamber was of specified quality; and secondly, to field test the logic, assumed overdesign factors, and restrictions contained in the CN120 project specifications. As previously described, some of the requirements contained in that document were also included in WMATA A-11b specifications.

Following analysis of the preconstruction testing results, and of results from testing during construction of the Research Chamber, it was hoped that improved, verified and simplified specifications could be suggested.

Preconstruction testing was performed in three consecutive stages: laboratory testing, trial mix cylinder testing, and field testing of equipment, materials and personnel by spraying test panels, both in vertical and horizontal overhead orientations.

### 7.3.2 Cement - Accelerator Compatibility Tests

Cement-admixture set time compatibility tests were performed as described in "Shotcrete Practice in Underground Construction, Section 0.1.[4/] Two different accelerators, the Sika Dryshot organic product, and Sika Sigunit inorganic accelerator were tested. Numbers included in brackets following the name

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4/ Mahar et al, "Shotcrete Practice in Underground Construction," USDOT Report, FRA-OR&D 75-90, 1976.

TABLE VII-1

TIME OF SET DETERMINATION
ATLANTA RESEARCH CHAMBER

Signal Mountain Type I Cement
Sikaset Dryshot (7005) Accelerator

| Water: Cement Ratio | Dosage Percent | Initial Set Min:Sec | Final Set Min:Sec | Testing |
|---|---|---|---|---|
| 0.40 | 3 | 1:20 | >10:00 | LETCO - 5/1/78 |
| 0.40 | 6 | 1:30 | >15:00 | LETCO - 5/1/78 |
| 0.35 | 3 | 0:50 | >10:00 | LETCO - 5/1/78 |
| 0.35 | 6 | 1:05 | 9:00 | LETCO - 5/1/78 |
| 0.35 | 1 | 3:45 | >30:00 | LETCO - 6/2/78 |
| 0.35 | 2 | 4:15 | 17:30 | LETCO - 6/2/78 |
| 0.35 | 3 | 2:30 | 12:00 | LETCO - 6/2/78 |
| 0.35 | 4 | 3:00 | 10:15 | LETCO - 6/2/78 |
| 0.33 | 6 | 1:05 | 9:00 | LETCO - 5/1/78 |
| 0.30 | 3 | 0:30 | 8:00 | LETCO - 5/1/78 |
| 0.30 | 6 | 0:35 | 6:00 | LETCO - 5/1/78 |
| 0.30 | 1 | 2:35 | 12:30 | LETCO - 6/1/78 |
| 0.30 | 2 | 3:40 | 12:30 | LETCO - 6/1/78 |
| 0.30 | 3 | 3:15 | 10:00 | LETCO - 6/1/78 |
| 0.30 | 4 | 4:45 | 9:00 | LETCO - 6/1/78 |
| 0.30 | 3 | 1:10 | 2:00 | SIKA - 03/1/78 |
| 0.35 | 3 | 1:00 | 3:10 | SIKA - 4/26/78 |
| 0.32 | 1 | 1:50 | 38:00 | BLANCK - 5/12/78 |
| 0.32 | 2 | 0:50 | 13:00 | BLANCK - 5/12/78 |
| 0.32 | 3 | 0:40 | 7:00 | BLANCK - 5/12/78 |
| 0.32 | 4 | 0:35 | 4:30 | BLANCK - 5/12/78 |

Table VII-2
TIME OF SET DETERMINATIONS
ATLANTA RESEARCH CHAMBER

Signal Mountain Type I Cement
Sigunit (8010) Accelerator

| Water: Cement Ratio | Dosage Percent | Initial Set Min:Sec | Final Set Min:Sec | Testing |
|---|---|---|---|---|
| 0.40 | 3 | 1:45 | 150:00 | LETCO - 5/1/78 |
| 0.40 | 6 | 1:30 | 115:00 | LETCO - 5/1/78 |
| 0.40 | 6 | 2:30 | 120:00 | LETCO - 5/1/78 |
| 0.40 | 3 | 3:45 | 30:00 | LETCO - 6/1/78 |
| 0:40 | 4 | 4:30 | 30:00 | LETCO - 6/1/78 |
| 0.35 | 3 | 1:25 | 36:00 | LETCO - 5/1/78 |
| 0.35 | 3 | 1:30 | 9:00 | LETCO - 5/1/78 |
| 0.35 | 3 | 1:20 | 12:00 | LETCO - 5/1/78 |
| 0.35 | 6 | 1:40 | 6:00 | LETCO - 5/1/78 |
| 0.35 | 6 | 2:00 | 8:00 | LETCO - 5/1/78 |
| 0.35 | 2 | 2:30 | 30:00 | LETCO - 6/1/78 |
| 0.35 | 3 | 2:45 | 30:00 | LETCO - 6/1/78 |
| 0.35 | 4 | 4:05 | 30:00 | LETCO - 6/1/78 |
| 0.35 | 5 | 3:45 | 28:00 | LETCO - 6/1/78 |
| 0.35 | 6 | 3:30 | 16:45 | LETCO - 6/1/78 |
| 0.30 | 3 | 0:15 | - | LETCO - 5/1/78 |
| 0.35 | 2 | 1:05 | 2:35 | SIKA - 3/1/78 |
| 0.35 | 3 | 1:05 | 2:10 | SIKA - 3/1/78 |
| 0.35 | 3 | 1:00 | 3:10 | SIKA - 4/26/78 |
| 0.40 | 2 | 4:30 | 60:00 | BLANCK - 5/12/78 |
| 0.40 | 3 | 1:00 | 15:00 | BLANCK - 5/12/78 |
| 0.40 | 4 | 0:50 | 4:30 | BLANCK - 5/12/78 |

Table VII-3
TIME OF SET DETERMINATIONS
ATLANTA RESEARCH CHAMBER

Medusa Type I Cement
Sikaset Dryshot (7005) Accelerator

| Water: Cement Ratio | Dosage Percent | Initial Set Min:Sec | Final Set Min:Sec | Testing |
|---|---|---|---|---|
| 0.35 | 3 | 2:00 | 710:00 | LETCO - 05/1/78 |
| 0.30 | 3 | 1:20 | 5:30 | LETCO - 05/1/78 |
| 0.30 | 1 | 8:30 | 730:00 | LETCO - 3/27/78 |
| 0.30 | 2 | 2:15 | 38:00 | LETCO - 3/27/78 |
| 0.30 | 3 | 2:30 | 28:00 | LETCO - 3/27/78 |
| 0.30 | 4 | 2:30 | 21:00 | LETCO - 3/27/78 |
| 0.30 | 3 | 1:05 | 1:05 | SIKA - 03/1/78 |
| 0.35 | 3 | 2:45 | 4:20 | SIKA - 4/26/78 |
| 0.30 | 1 | 3:30 | 10:30 | BLANCK - 5/12/78 |
| 0.30 | 2 | 0:50 | 8:00 | BLANCK - 5/12/78 |
| 0.30 | 3 | 0:40 | 3:10 | BLANCK - 5/12/78 |
| 0.30 | 4 | 0:35 | 2:10 | BLANCK - 5/12/78 |

Table VII-4
TIME OF SET DETERMINATIONS
ATLANTA RESEARCH CHAMBER

Medusa Type I Cement
Sigunit (8010) Accelerator

| Water: Cement Ratio | Dosage Percent | Initial Set Min:Sec | Final Set Min:Sec | Testing |
|---|---|---|---|---|
| 0.40 | 6 | 2:00 | >8:00 | LETCO - 5/1/78 |
| 0.35 | 3 | 1:45 | >8:00 | LETCO - 5/1/78 |
| 0.35 | 6 | 1:30 | >8:00 | LETCO - 5/1/78 |
| 0.30 | 3 | 1:30 | >8:00 | LETCO - 5/1/78 |
| 0.35 | 2 | 2:15 | 4:15 | SIKA - 03/1/78 |
| 0.35 | 3 | 2:15 | 4:00 | SIKA - 03/1/78 |
| 0.35 | 3 | 3:10 | 5:40 | SIKA - 4/26/78 |
| 0.38 | 2 | 3:15 | >60:00 | BLANCK - 5/12/78 |
| 0.38 | 3 | 1:30 | 55:00 | BLANCK - 5/12/78 |
| 0.38 | 4 | 1:10 | 16:00 | BLANCK - 5/12/78 |

# GRADING CHART FOR SHOTCRETE MIXES

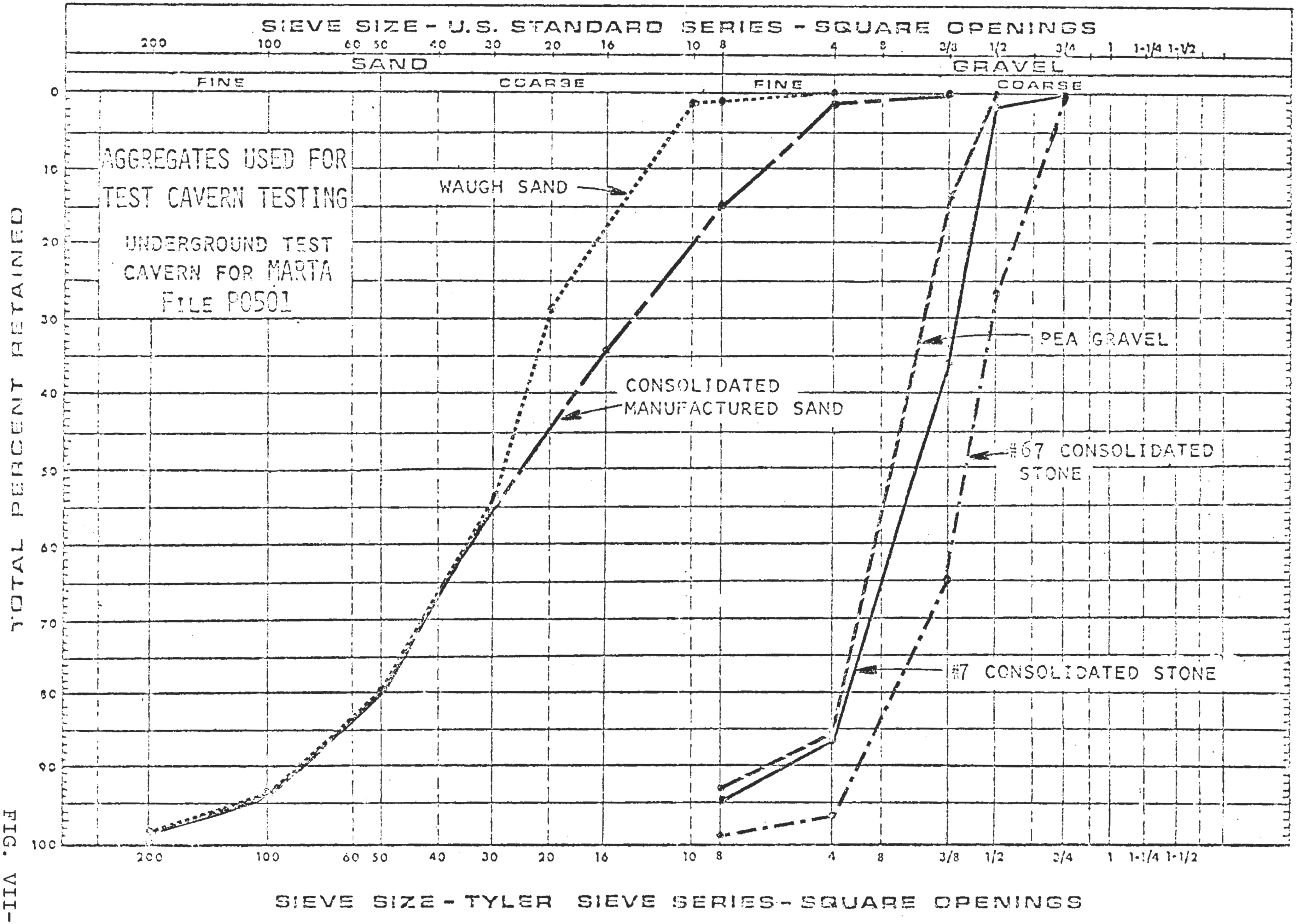


FIG. VII-2

refer to the batch designation. Tests were performed with two cements, Medusa I, and Signal Mountain I. Setting times required by CN120 specifications were sought. CN120 clause 1.01.C.2 requires:

| | |
|---|---|
| Time of set and initial setting | 90 second minimum<br>5 minutes maximum |
| Time for final setting | 12 minutes minimum<br>20 minutes maximum |

Current WMATA compatibility test requirements have been successfully utilized for several projects. Test results were also reviewed in accordance with them as follows:

| | |
|---|---|
| Time of initial setting | 3 minutes maximum |
| Time for final setting | 12 minutes maximum |

The time of setting is determined in accordance with ASTM C266, modified to adapt the standard to shotcrete requirements. (The major difference between this specification and that referenced in CN120 is that the latter requires the use of a special mixing bowl apparatus to facilitate testing.)

The objective of this test is to eliminate cement and accelerator combinations which may not produce required initial and final sets. A summary of test results is given in Tables VII-1 to VII-4. Testing was performed by three parties: Law Engineering Testing Company (LETCO), Blanck-Alvarez Co., Inc. (Blanck) and Sika Chemical Corporation (Sika).

### 7.3.3 Aggregate Gradation

Since the CN120 specifications require that shotcrete aggregates have a gradation as specified therein, available aggregate sources were tested, and a number of different mixes developed. For purposes of the Research Chamber, aggregate grad-

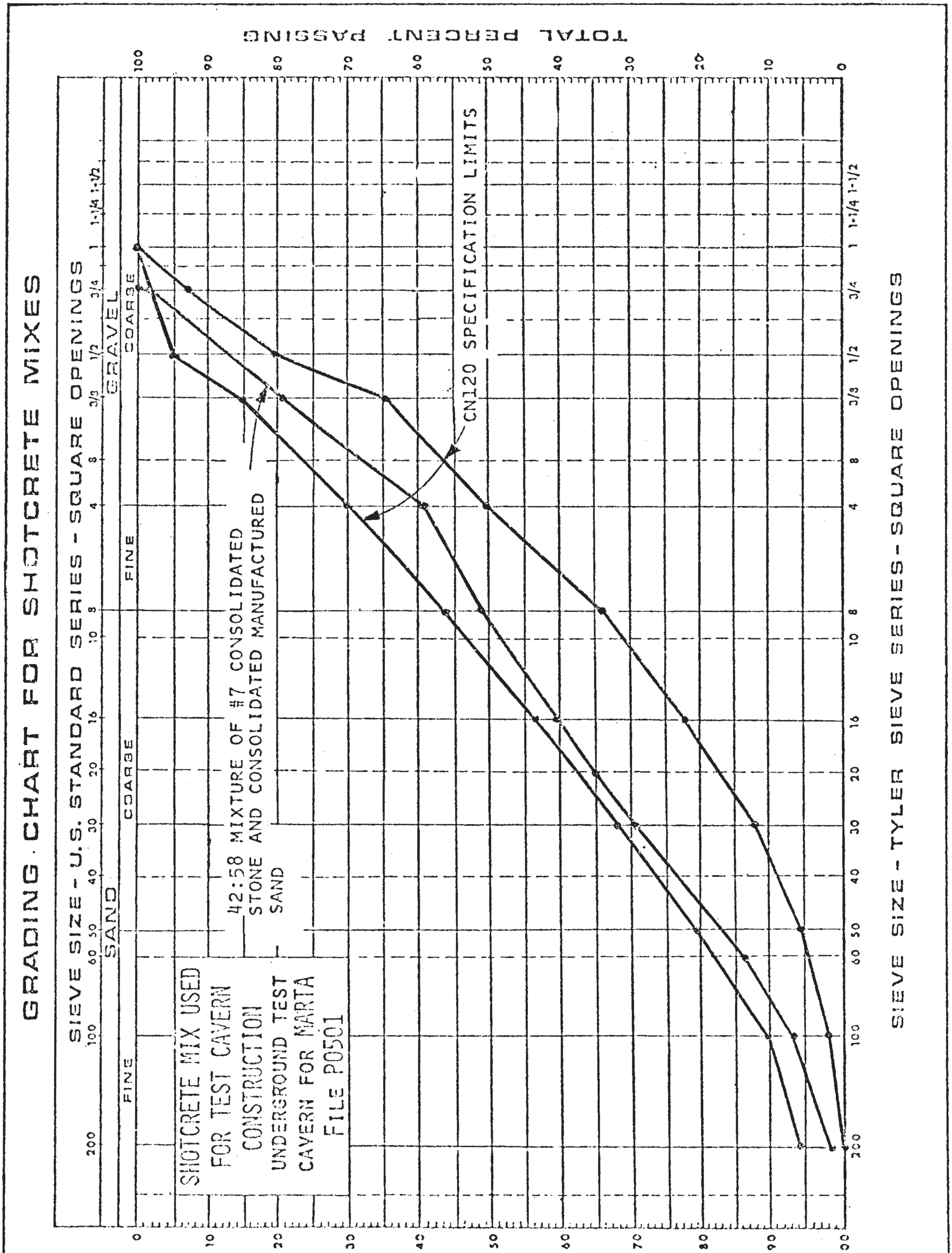
GRADING CHART FOR SHOTCRETE MIXES
SIEVE SIZE - U.S. STANDARD SERIES - SQUARE OPENINGS
200 100 60 50 40 30 20 16 10 8 4 8 3/8 1/2 3/4 1 1-1/4 1-1/2
SAND
FINE
COARSE
FINE
GRAVEL
COARSE
TOTAL PERCENT PASSING
100 90 80 70 60 50 40 30 20 10 0
TOTAL PERCENT RETAINED
0 10 20 30 40 50 60 70 80 90 100
CN120 SPECIFICATION LIMITS
42:58 MIXTURE OF #7 CONSOLIDATED STONE AND CONSOLIDATED MANUFACTURED SAND
SHOTCRETE MIX USED FOR TEST CAVERN CONSTRUCTION UNDERGROUND TEST CAVERN FOR MARTA FILE P0501
SIEVE SIZE - TYLER SIEVE SERIES - SQUARE OPENINGS

FIG. VII-3

# GRADING CHART FOR SHOTCRETE MIXES

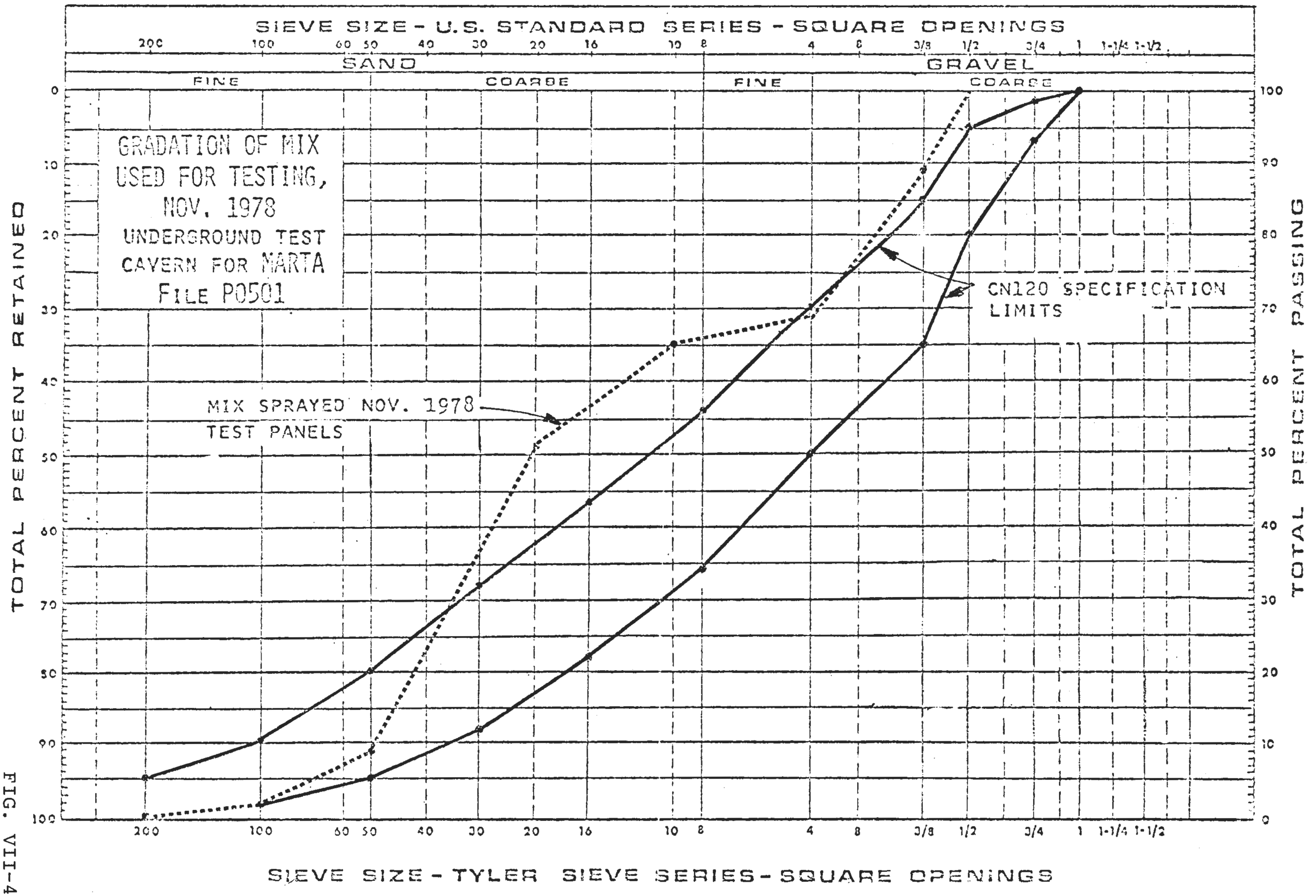


FIG. VII-4

ation was unspecified, in line with the logic developed for WMATA A-11b specifications. These specifications based acceptance on performance, without limiting the Contractor in aggregate gradation, but only quality as verified by the concrete cylinder test.

Three coarse aggregates were tested: Consolidated crushed #67 stone, Consolidated crushed #7 stone, and natural pea gravel. Two fine aggregates were also tested: Consolidated manufactured sand and Waugh natural sand. Gradations of the five aggregates are shown in Figure VII-2.

A mix of 50:50 Consolidated #67 stone and manufactured sand was found to comply with CN120 requirements, and concrete cylinder tests (described further below) verified the aggregate quality. However, the Contractor requested permission to use pea gravel or Consolidated No. 7 stone instead of No. 67, to facilitate shotcrete placement. Since shotcrete containing steel fiber was also to be placed in the Research Chamber, and the presence of smaller coarse aggregate apparently reduces pumping difficulties with fiber, the No. 7 stone was selected.

A preconstruction program of spraying test panels was undertaken in June 1978, using 42:58 mixes of No. 7 stone or pea gravel, combined with Consolidated manufactured sand (Figure VII-3). A further test panel program was performed in November 1978, and Waugh sand was unknowingly substituted for Consolidated manufactured sand in a ratio of 42 parts Consolidated #7 stone to 58 of sand. Results of screen analysis of sand and stone are shown in Figure VII-4.

### 7.3.4 Standard Concrete Cylinders

Standard concrete cylinders were prepared using the proposed aggregate sources and gradation, with enriched eight and nine sack cement content, but without accelerators. Cements tested were those proposed for shotcrete, and water/cement ratios were made as close to 0.4 as possible. When made, cured and tested in accordance with ASTM C31 and C39, the average of three cylinders of the same mix should meet or exceed the required strength by at least 40 percent. The use of high cement content attempts to duplicate shotcrete applied in place, which is naturally cement-enriched due to the rebound of mostly aggregate particles.

The objective of this test is to eliminate trial mixes of materials which do not comfortably exceed the requirements for shotcrete strength. The 40 percent overdesign factor is an empirical one, designed to compensate for the common decrease of 28-day shotcrete strength when using inorganic accelerators. If a low slump, high-cement concrete mix achieves only 5,000 psi, a similar accelerated shotcrete mix (with inorganic accelerator) would probably suffer a 20 to 30 percent apparent strength loss, or give results of 4,000 to 3,500 psi, respectively, at 28 days, provided an identical test procedure was used.

Results of concrete cylinder tests for various aggregates are given in Table VII-5.

Table VII-5

STANDARD CONCRETE CYLINDER COMPRESSIVE STRENGTHS

ATLANTA RESEARCH CHAMBER

| Ratio | Stone | Sand | Sacks Cement | 28 day Strength (psi) |
|---|---|---|---|---|
| 50:50 | #67 | manufactured | 7.4 Signal Mtn. I | 6210 |
| 50:50 | #7 | manufactured | 7.8 Signal Mtn. I | 6660 |
| 50:50 | pea gravel | manufactured | 7.8 Signal Mtn. I | 6070 |
| 50:50 | #67 | manufactured | 9.0 Medusa I | 8430 |
| 50:50 | #67 | manufactured | 9.0 Signal Mtn. I | 8780 |

### 7.3.5 Shotcrete Test Panels

Shotcrete test panels were prepared using ingredients which passed the above tests. The panels were eighteen by eighteen by four inches deep, mounted horizontally for overhead testing and vertically for wall samples. Ambient and material tem-

TABLE VII-6

COMPRESSIVE STRENGTH RESULTS OF SHOTCRETE PANELS, MIX 1
(#7 STONE, MANUFACTURED SAND, SIGUNIT ACCELERATOR)

| Panel Designation | Orientation | Average Compressive Strength (psi) 8 - 12 hr | 3 day | 28 day |
|---|---|---|---|---|
| CS 3V A | vertical | 2085 (6) | 3100 (0) | 4360 (6) |
| CS 3H A | horizontal | 1800 (2) | 2980 (4) | 2900 (2) |

TABLE VII-7
COMPRESSIVE STRENGTH RESULTS OF SHOTCRETE PANELS, MIX II
(PEA GRAVEL, MANUFACTURED SAND, SIGUNIT ACCELERATOR)

| Panel Designation | Orientation | Average Compressive Strength (psi) 8 - 12 hr | 3 day | 28 day |
|---|---|---|---|---|
| S1VB | vertical | 1700 (6) | 4685 (6) | 5470 (6) |
| S1HA | horizontal | 2050 (5) | 2750 (4) | 3520 (5) |

TABLE VII-8

COMPRESSIVE STRENGTH RESULTS OF SHOTCRETE PANELS, MIX III
(PEA GRAVEL, MANUFACTURED SAND, DRY SHOT ACCELERATOR)

| Panel Designation | Orientation | Average Compressive Strength (psi) 8 - 12 hr | 3 day | 28 day |
|---|---|---|---|---|
| D1VB | vertical | 1565 (4) | 5415 (6) | 6240 (6) |
| D2VB | vertical | 2215 (6) | 5370 (6) | 6820 (6) |
| D2HA | horizontal | --- | 2520 (3) | 5390 (3) |

TABLE VII-9

COMPRESSIVE STRENGTH RESULTS OF SHOTCRETE PANELS, MIX IV
(#7 STONE, MANUFACTURED SAND, DRYSHOT ACCELERATOR)

| Panel Designation | Orientation | Average Compressive Strength (psi) 8 - 12 hr | 3 day | 28 day |
|---|---|---|---|---|
| CD3VA | vertical | 3165 (6) | 5410 (6) | 6400 (6) |
| CD3HA | horizontal | 3600 (3) | 6000 (3) | 7420 (3) |

TABLE VII-10

COMPRESSIVE STRENGTH RESULTS OF SHOTCRETE PANELS, MIX V
(#7 STONE, WAUGH SAND, SIGUNIT ACCELERATOR)

| Panel Designation | Orientation | Average Compressive Strength (psi) 8 - 12 hr | 3 day | 28 day |
|---|---|---|---|---|
| S161W | vertical | 1445 (3) | 6330 (3) | 7400 (3) |
| S162W | vertical | 1630 (3) | 2850 (3) | 3520 (3) |
| S163W | vertical | 1210 (3) | 2810 (3) | 3730 (3) |
| S161OH | horizontal | 1465 (3) | 2400 (3) | 3605 (3) |
| S162OH | horizontal | 1730 (3) | 2530 (3) | 2920 (3) |
| S163OH | horizontal | 1610 (3) | --- | --- |

TABLE VII-11

COMPRESSIVE STRENGTH RESULTS OF SHOTCRETE PANELS, MIX VI
(#7 STONE, WAUGH SAND, DRYSHOT ACCELERATOR-POWDER)

| Panel Designation | Orientation | Average Compressive Strength (psi) 8 - 12 hr | 3 day | 28 day |
|---|---|---|---|---|
| DS1W | vertical | --- | 3730(3) | 4440 (3) |
| DS2W | vertical | --- | 5680 (3) | 7595 (3) |
| DS3W | vertical | --- | 5430 (3) | 7340 (3) |

TABLE VII-12

COMPRESSIVE STRENGTH RESULTS OF SHOTCRETE PANELS, MIX VII
(#7 STONE, WAUGH SAND, DRYSHOT ACCELERATOR-LIQUID)

| Panel Designation | Orientation | Average Compressive Strength (psi) 8 - 12 hr | 3 day | 28 day |
|---|---|---|---|---|
| SDS1W | vertical | --- | 3820 (3) | 6990 (3) |
| SDS2W | vertical | --- | 3330 (3) | 6090 (3) |
| SDS3W | vertical | --- | 3900 (3) | 7230 (3) |
| SDS20H | horizontal | --- | 5240 (3) | 8030 (3) |
| SDS30H | horizontal | --- | 4720 (3) | 8030 (3) |
| SDS40H | horizontal | --- | 4980 (3) | 7620 (3) |

peratures duplicated as closely as possible those to be anticipated underground.

Test panels were sprayed on two separate occasions, in June and November 1978. The purpose of the November test program was to resolve difficulties of spraying overhead test panels of shotcrete utilizing Dryshot organic accelerator, so as to permit its use in the Research Chamber. During the November test period Dryshot was tested dissolved in water, instead of being added in powder form to the dry mix. Dissolving the Dryshot in the water was successful and was also adopted for the Research Chamber construction.

Cores were removed from the test panels and tested at 8 to 11 hours, 3 and 28 day ages. Test results are given in Tables VII-6 through VII-12. During the June 1978 test, cores with an L/D of greater than one were tested and corrections were made to correspond to an L/D of two. To comply with CN120 specifications and June testing, the November test results have been adjusted to conform with an L/D of two.

## 7.4 TESTING DURING CONSTRUCTION OF THE RESEARCH CHAMBER

On March 22-24, 1979 shotcrete was placed in the Research Chamber, and two test panels were also sprayed. In addition, aggregate gradation and standard cylinder tests were performed.

### 7.4.1 Research Chamber Construction Plan

Testing of conventional shotcrete was performed in Panels 7 and 5 of the Research Chamber. Panel 7 was sprayed with a thickness of 4 inches of shotcrete containing Sigunit inorganic accelerator, and Panel 5 with a similar thickness but using Dryshot organic accelerator; both accelerators are manufactured by Sika Chemical Corporation (see above). Composition of materials was approximately as shown in Table VII-13.

TABLE VII-13

COMPOSITION OF RESEARCH CHAMBER SHOTCRETE

| | |
|---|---|
| Signal Mountain Type I Cement | 660 lb. |
| Consolidated Crushed #7 Stone | 1,300 lb. |
| Consolidated Manufactured Sand | 1,790 lb. |

The gradation of combined aggregates is shown on Figure VII- and compared with CN120 Specifications. Accelerator percentages of cement weight ranged from 1.5 percent on sidewalls and test panel, to 3 percent in the tunnel arch for Sigunit; and 4 percent on sidewalls and test panel, to 6 percent in the arch for Dryshot. Sigunit was added in powder form to the volumetric batcher mixing auger, and Dryshot powder was mixed with water and fed by gravity to the shotcrete nozzle. The shotcrete pump and volumetric batcher were located at the ground surface adjacent to the access shaft, and shotcrete was pumped through 200 feet of hose 80 feet vertically down the shaft to the Research Chamber. Shotcrete was placed in the Research Chamber from portable scaffolding. A 10-foot long nozzle was used for the Sigunit mix panel and a 3-foot nozzle for the Dryshot mix. Ground surface weather conditions varied from warm and clear to cool and raining, the latter condition causing some minor equipment operation difficulties. Ground surface temperatures varied from approximately 45 to 70 degrees Fahrenheit. Air and water were supplied from the site facilities.

Since the shelf life of the Sigunit 8010 and Dryshot 7005 used for preconstruction testing was exceeded, more recent batches, Sigunit 8123 and Dryshot 8011, were used for construction of the Research Chamber. Unlike Sigunit 8010, the 8123 batch was not specially formulated to match Signal Mountain Type I cement.

Two groups of three cores each were removed from the sidewall, and two groups from the Chamber arch, for each panel. A total of 24 cores from Panels 5 and 7 were cut to three inch lengths and tested in compression at 28-day age. The purpose of these tests was to simulate normal testing during construction, with duplicate groups taken to improve sampling population. The three cores per group were derived from the ACI 506.2-77 requirement described in Section 7.2.

To provide early-age strengths for the two Sigunit and Dryshot mixes, one 18-inch by 18-inch test panel was sprayed for each mix. The test panels were sprayed in vertical position for convenience, and 8-hour, 3- and 28-day cores were removed and tested. The test panels were made while spraying the Research Chamber sidewalls, with no change in procedure.

### 7.4.2 Test Results

To confirm the quality of the basic ingredients, three standard concrete cylinders were cast with the following proportions:

TABLE VII-14

COMPOSITION OF RESEARCH CHAMBER CONCRETE CYLINDERS

| | Wt. (lbs.) |
|---|---|
| Signal Mountain Type I Cement | 25.0 |
| Consolidated Crushed #7 Stone | 34.4 |
| Consolidated Manufactured Sand | 47.5 |
| Water (added) | 10.5 |

Computed water to cement ratio with an assumed 4 percent sand moisture was 0.45. The mix contained approximately nine bags of cement per cubic yard, to duplicate an in-place shotcrete enriched by rebound of mainly coarse aggregate.

Standard concrete cylinders achieved an average 28-day compressive strength of 6120 psi or 153 percent of the required 4,000 psi.

The two vertical test panels achieved results as described below:

TABLE VII-15

VERTICAL TEST PANEL RESULTS

| Panel Designation | Accelerator | Average Compressive Strength (psi) | | | Unit Wt. pcf |
|---|---|---|---|---|---|
| | | 9-11 hrs | 3 days | 28 days | |
| SIG | Sigunit | 1550(4) | 5210(3) | 6030(4) | 143.0(4) |
| DS | Dryshot | 880(4) | 6320(3) | 8090(3) | 142.6(2) |

Average compressive strengths are for three-inch diameter cores three inches long, corrected for L/D = 1. Numbers of samples for test are shown in brackets.

Shotcrete cores taken from the Research Chamber Panels 5 and 7 are described in Table VII-16 below. Due to operational difficulties, the number of tests and ages of the cores did not meet planning requirements. However, the results obtained indicate the shotcrete quality of the two panels.

These samples had an L/D of 0.96 to 1.02, and the strengths given are uncorrected.

Cores were removed immediately prior to testing, so that early-age curing duplicated normal underground construction conditions; that is, no curing was performed.

During placement, some difficulty was experienced in applying the overhead shotcrete with 3 percent Dryshot accelerator, in that the layer sometimes sloughed and fell. With a dosage of 6 percent, no difficulties were experienced, although the final thickness was deficient in some areas.

TABLE VII-16

SHOTCRETE CORES FROM RESEARCH CHAMBER

| Panel | Location | Accelerator | Age, Days | Strength, psi | Avg. Strength psi | Unit Wt. pcf |
|---|---|---|---|---|---|---|
| 5 | West Sidewall | Dryshot | 28 | 8244 | | 143.5 |
| 5 | West Sidewall | Dryshot | 32 | 7535 | | 142.6 |
| 5 | West Sidewall | Dryshot | 32 | 6595 | 7460 | 142.5 |
| 5 | West Sidewall | Dryshot | 32 | 6954 | | 140.6 |
| 5 | West Sidewall | Dryshot | 32 | 7599 | | 143.4 |
| 5 | West Sidewall | Dryshot | 32 | 7386 | 7480 | 142.8 |
| 7 | East Sidewall | Sigunit | 29 | 4660 | | 139.0 |
| 7 | East Sidewall | Sigunit | 29 | 5001 | | 140.9 |
| 7 | East Sidewall | Sigunit | 29 | 5520 | 5060 | 142.8 |
| 7 | West Sidewall | Sigunit | 29 | 8029 | | 141.9 |
| 7 | West Sidewall | Sigunit | 29 | 4874 | | 139.4 |
| 7 | West Sidewall | Sigunit | 29 | 5592 | | 140.6 |
| 7 | West Sidewall | Sigunit | 29 | 6595 | 6270 | 140.8 |
| 7 | Arch | Sigunit | 32 | 5018 | | 140.0 |
| 7 | Arch | Sigunit | 32 | 5377 | | 142.1 |
| 7 | Arch | Sigunit | 32 | 4302 | 4900 | 141.2 |

## 7.5 DISCUSSION OF FINDINGS

### 7.5.1 Cement-Accelerator Compatibility Tests

Compatibility test results were compared with the CN120 specification requiring initial set within 1.5 to 5 minutes and final set 12 to 20 minutes. In addition, the current WMATA requirement of initial set of 3 minutes maximum and final set within 12 minutes was applied, for comparison purposes.

Observations from the testing program are outlined below:

1. Tests with Dryshot 7005 accelerator and Signal Mountain Type I cement were able to comply with CN120 specifications for minimum and maximum initial and final setting times, albeit by varying the dosage rate. CN120 specifications do not indicate if varying the rate is permissible. The accelerator-cement combination also complied with the comparison requirement of 3 minutes maximum initial and 12 minutes final setting times (WMATA).
2. Tests with Sigunit 8010 and Signal Mountain Type I cement also complied with both requirements, although results from two testing periods had to be combined to give required results.
3. Reproducibility of test results is poor. LETCO performed tests on several occasions, trying to achieve results or even reproduce results reported by Sika and Blanck. Despite their duplicating conditions to match those of other testers, they were unable to achieve similar results. (See Tables VII-1 to VII-4).
4. Given the reported poor reproducibility of compatibility testing, the philosophy of requiring field dosages to comply with those derived from laboratory testing seems to be unfounded.
5. The CN120 setting time specification appears to have a number of disadvantages.

a. It is difficult to make the accelerator cement fit between the specified limits, even by varying accelerator dosage rate. Poor reproducibility of results aggravates this difficulty.

b. The setting time limits in CN120 and all previous specifications are arbitrary, and have not been demonstrated to correspond to field conditions, where cement, accelerator and water are subjected to instantaneous mixing. Therefore, it seems unwise to impose further restrictions on specified setting times. In addition, the CN120 specifications permit maximum setting times greater than previous specifications by others, i.e., 5 and 20 minutes in place of 3 and 12 minutes. While there is no known basis for accepting the latter specification, it at least has the advantage of having been utilized a number of times without apparent difficulty. Without substantiating evidence it may be undesirable to specify the greater setting times.

6. For compatibility testing the water/cement ratio (W/C) appears to be a critical factor. Existing specifications do not fix the required W/C.

a. CN120 specifications refer to procedures described in DOT Report FRA-OR&D 75-90 (Mahar et al, 1976)4/, which specifies using W/C of 0.43 or duplicating the W/C of in-place shotcrete.

b. Another DOT report performed by the same researchers, FRA-OR&D 76-06, (Parker et al, 1975)5/, reports an average measured W/C of shotcrete to be 0.31, utilizing conventional inorganic accelerators.

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5/ Parker et al, "Field-Oriented Investigation of Conventional and Experimental Shotcrete for Tunnels", USDOT Report FRA-OR&D-76-06, 1975.

c. An original proponent of the compatibility test for pre-construction testing, Blanck, specifies a W/C of 0.40 in his paper "Shotcrete Durability and Strength - A Practical Viewpoint" (Blanck, 1974)[6/].
However, in tests performed for the Atlanta Research Chamber he utilized a W/C of 0.30 to 0.40.

d. Dryshot organic accelerator acts as a water-reducing agent by improving workability. Therefore, it is reasonable to assume that a lower W/C could be utilized to produce a mix of equivalent workability. The proper W/C of shotcrete using Dryshot accelerator is unknown.

7. Overhead panels of shotcrete containing both Dryshot and Sigunit were successfully placed during preconstruction testing, albeit with some difficulties. It therefore may be inferred that initial and final set times for these mixes were adequate for practical purposes.

### 7.5.2 Aggregate Gradations

None of the shotcrete placed for the Atlanta Research Chamber program complied with CN120 specification regarding maximum aggregate size. CN120 requires 93 to 98 percent of the combined aggregates to pass 3/4-inch mesh, or, conversely, 2 to 7 percent must be retained on 3/4-inch mesh. All of the mixes utilized for preconstruction testing and Research Chamber construction contained No. 7 stone or smaller maximum size. The No. 7 stone utilized had no particles retained on a 3/4-inch mesh, i.e., the aggregate actually used in the Research Chamber was finer than that specified for the MARTA Peachtree Center Station and Tunnels in Contract CN120.

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6/ Jan A. Blanck, "Shotcrete Durability and Strength - A Practical Viewpoint", Use of Shotcrete for Underground Structural Support, ASCE ACI Publication SP045 (1974), pp 320-329.

Apart from the maximum particle size requirement, the mix used for the Research Chamber construction (shown on Figure VII-3), complied with the CN120 gradation specification. On the other hand, some good quality shotcrete was placed during the November, 1978 session which, due to inadvertent use of a different fine aggregate, exhibited a gradation largely outside of the specified limits. This gradation is shown in Figure VII-4.

Aggregate gradations for cast-in-place concrete are specified to improve quality and workability of the material. In-place shotcrete, however, does not contain the same aggregate gradation as the mixed ingredients due to the sorting action of application and rebound. For this reason, specifying gradation of aggregates may reduce rebound quantity, but may not necessarily affect in-place quality. If the Owner does not pay for shotcrete rebound, as is commonly the case, there seems little advantage to specifying aggregate gradation, particularly as this is frequently changed in the field.

Results from pre-construction testing and Atlanta Research Chamber construction indicate that shotcrete quality, as demonstrated by compressive strength, was excellent, although not entirely consistent. The aggregate quality, as demonstrated by the standard cylinder tests, was excellent, but the aggregate gradation was poor compared to CN120 specifications.

It may be premature to judge that it is unnecessary to specify aggregate gradation limits to produce quality shotcrete, but some credence is lent to the performance specification approach, which does not mention gradation.

### 7.5.3 Concrete Cylinder Testing

The CN120 specification requires that standard concrete cylinders with cement-enriched mix must achieve 140 percent of the required shotcrete compressive strength. Since no conversion of equivalent cylinder values to cores is suggested, it is assumed that 140 percent of 4000 psi, or 5600 psi is the intended requirement. All of the cement and aggregate mixes tested exceeded 5600 psi.

The WMATA specification requires cylinder strengths of 7000 psi, for a required in-situ strength of 5000 psi. The equivalent requirement for CN120 may be calculated as follows:

$$\text{Required } f'c = 4000 \div 0.85$$
$$= 4710 \text{ psi}$$

where 0.85 is the ACI 506.2-77 requirement for cores. For the Research Chamber, the required cylinder strength would be

$$(7000 \div 5000) \times 4710 = 6590 \text{ psi,}$$

which would be in accordance with WMATA criteria.

While the mix used for the Research Chamber construction did not achieve 6590 psi in cylinders, previous mixes using different gradation and maximum particle size exceeded 8000 psi. Some of the shotcrete core samples also exceeded 8000 psi, which indicates that the Research Chamber construction mix cylinders may not have been representative. However, the tests do indicate excellent quality of materials.

The CN120 specifications require an overdesign factor of 140 percent of the specified core strength. However, due to the difference in sampling cores and cast cylinders, the overdesign factor is less than 140 percent, 4000 psi cores with L/D = 2 are equivalent to in-situ or cylinder strength of:

$$4000 \div 0.85 = 4710 \text{ psi.}$$

Therefore the actual overdesign factor is:

$$5600 \div 4710 = 119 \text{ percent.}$$

A factor of 119 percent is insufficient to compensate for the anticipated strength loss due to the use of inorganic accelerators. The test program results do not clarify this situation.

### 7.5.4 Preconstruction Test Panels

CN120 specifications require that the average strength of all cores taken from a test panel shall test at least 110 percent of the strength specified for each age. The origin of this 110 percent is not stated, but since six cores are required for three ages (8-hour, 3- and 28-days), presumably two cores for each age are intended. The specification goes on to say that test reports shall contain unit strength at failure "with and without correction for L/D ratio and statistical quantity adjustment". The statistical quantity adjustment presumably refers to the 110 percent requirement, as it is not referenced elsewhere.

The A-11b WMATA specification requires a factor of overdesign of approximately 110 percent (5540 psi for a required f'c of 5000 psi) but this factor is related to comparison of cores to cylinders, L/D ratio, and statistical quantity adjustment or overdesign, as described in Section 7.2.1. The correction, however, applies to three test cores per sample. If two cores per sample are used, as inferred for CN120, the overdesign should be greater, as the reliability of sampling is lower.

Observations regarding preconstruction test panels are discussed below:

1. All of the mixes tested were placed successfully on vertical and overhead panels. Mixes using Dryshot accelerator were more difficult to place on overhead panels due to their greater tendancy to slough off compared to mixes using Sigunit. Mixing Dryshot powder with the nozzle water instead of adding powder to the dry mix reduced this problem.
2. All of the mixes utilized were successfully cored and tested in compression at an age of from 8 to 12 hours. Those mixes successfully tested achieved compressive strengths greatly exceeding the required strength. How-

ever, during the November 1978 testing programs, plywood panels using Dryshot were not successfully cored at this age. The two major differences between June 1978 and November 1978 plywood panel tests of Dryshot and crushed stone mix are that Waugh sand was used in November instead of manufactured Consolidated sand, and the Dryshot accelerator was of increased age. The Dryshot was manufactured in 1977, and although properly stored, may not have had adequate reactivity to produce required early strength. The different sand probably did not affect the early shotcrete strength.

3. Of all the mixes tested, Mix IV containing Dryshot, crushed stone and manufactured sand was the only one to achieve specified strengths. However, due to the difficulty in placing overhead panels, cores with L/D less than one were tested; this brought the results into question. The specified number of panels and cores was not tested.

4. All mixes containing Sigunit produced high early strength shotcrete, but 28-day strengths were below specified quality. Since these mixes were identical in all other ways to mixes using Dryshot, it appears that the lower 28-day strengths are due to the accelerator. This strength loss is in the range of 32 to 61 percent for vertical and overhead panels containing crushed stone and manufactured sand, and 16 to 35 percent for panels containing pea gravel and manufactured sand. The higher strength losses for each mix are for overhead placement. Strength losses of greater than 30 percent indicate cement-accelerator compatibility problems.

5. For mixes containing Sigunit, vertical panels exhibited higher strengths than did horizontal overhead panels. This appears to be due primarily to the increased strength loss of higher accelerator dosage. Mixes containing Dryshot did not exhibit any marked difference whether placed on vertical or overhead panels, and in fact the same dosages are reported.

6. There was no important difference in shotcrete strength between panels using crushed stone or natural pea gravel.
7. None of the test panels failed to comply with the 3-day strength requirement. In our experience, this is generally the case. Therefore, it may be appropriate to examine the usefulness of specifying 3-day tests; perhaps these could be eliminated. WMATA A-11b specifications require testing only at 3- and 28-day ages, not at an earlier age. From the Atlanta Research Chamber test program, it could be concluded that 8- or 12-hour tests might be of greater importance than 3-day tests.
8. June 1978 plywood panel test cores were trimmed to various lengths and corrections made in accordance with ASTM C42. November 1978 tests were made as required by CN120 specifications, that is, three-inch diameter cores three inches long, corrected for L/D. By contrast, the intent of WMATA A-11b specifications is to eliminate the need for making L/D corrections and to simplify adjustment of the required compressive strength of three-inch long cores.

### 7.5.5 Testing During Construction

Two plywood test panels were shotcreted during underground shotcrete construction of the Atlanta Research Chamber, as described below:

1. The test panel containing Sigunit exceeded CN120 preconstruction testing strength requirements for all three ages. However, the panel was sprayed in the vertical position and thus does not represent the most critical (overhead) circumstance. Strengths at 28 days were similar to those of sidewall in-situ cores.

2. The test panel utilizing Dryshot was difficult to core at 9 to 11 hour ages due to slower hardening than Sigunit samples. Considerable wash-out of material at the cored surface was noted. Strength at 28 days appeared similar to sidewall in-situ cores.
3. Test panels for both accelerators exhibited generally higher strengths than those previously tested in June and November 1978. For November tests, a different sand was utilized, but no reason for the improvements over June samples was identified. Equipment operation conditions appear to be identical or slightly superior during June testing, as shorter hoses were utilized.

A total of 16 cores was removed from the Research Chamber and tested at 28 to 32 days age. All of the cores exceeded the strengths specified by CN120. However, only 3 of the cores were taken from the cavern arch, where strengths might be lower than in the sidewalls. Results and their possible interpretation are described below:

1. Compressive strengths of samples taken from the arch of Panel 7 (Sigunit) averaged 14 percent lower than a combined average of all sidewall samples.
2. Compressive strengths of sidewall samples of Panel 7 (Sigunit) were 24 percent lower than those removed from Panel 5 (Dryshot).
3. The Sigunit panel test results were substantially better than were indicated by preconstruction testing. No definitive reason for this improvement, particularly during June preconstruction testing when an identical mix was used, was noted. One possible improvement was the use of a 10-foot long nozzle for the Atlanta Research Chamber placement of the Sigunit panel. However, high quality shotcrete was placed with a short nozzle using Dryshot accelerator, immediately after the application of the Sigunit shotcrete. Another

possibility is the difference in the Sigunit itself, a standard product being used for construction, rather than the special formulation tested earlier in preconstruction testing.

4. Problems indicated in preconstruction testing, such as the difficulty of overhead placement of Dryshot shotcrete, followed through during construction. Use of a higher than normal accelerator content minimized this problem for placement in the Research Chamber.
5. Reference to Table VII-16 demonstrates the fairly large variation between samples within each grouping of 3 or 4 cores. For the west sidewall of Panel 7, the uncorrected compressive strength varied from 4870 to 8030 psi. This variation indicates the poor reproducibility of testing for even immediately adjacent core samples. Moreover, it indicates that a single core sample as specified in CN120 may not represent a given area of completed work.
6. The behavior of the Dryshot shotcrete seemed somewhat contradictory. It had a very fast compatibility test setting time (and appearance), while the early age strength at 8 to 12 hours was lower than exhibited by Sigunit samples. In addition, some problems were encountered of overhead layers of Dryshot shotcrete falling off at an age of several minutes.

## 7.6 RECOMMENDED SPECIFICATION CHANGES

One of the purposes of this test program was to analyze existing shotcrete specifications to seek improvements in quality control requirements. One of the most current set of specifications, the MARTA CN120 document was chosen because of its direct applicability to the Atlanta Research Chamber. Therefore, recommendations for improvement are based upon those specifications, with some reference to the WMATA A-11b document. Recommendations are discussed below:

1. Cement-Accelerator Compatibility Tests. The referenced test procedure from DOT Report FRA-OR&D 75-90 appears to be an improvement over the previously specified ASTM Standard C266, as it facilitates performance of the test. However, a water/cement ratio should be specified to avoid the inherent setting time variation. Tests performed for this study indicate that a specified water/cement ratio of 0.35 would be appropriate.
2. Results from this test program indicate that the specified setting times should be three and twelve minutes maximum, respectively, for initial and final set, rather than the more complicated ranges specified in CN120.
3. The standard cylinder test specification should be changed to 165 percent of 4000 psi, to give the required overdesign factor.
4. Aggregate gradation specifications should be deleted (as they have been for WMATA A-11b).
5. The specified strengths and required test results should be rationalized. Assuming that an in-situ strength (f'c) of 4000 psi is required, application of the logic described in 7.2.1 would produce the following criteria:
    a. Shotcrete test panels should be prepared, using ingredients which have passed the compatibility and test cylinder criteria, and various accelerator dosages as required. Test panels should be 18" x 18" x 4" deep and should be mounted vertically for wall samples and overhead for horizontal samples. Three-inch diameter cores should be removed after seven hours, cut to three-inch lengths and tested in compression at 8 hours, 3 days, and 28 days. Cores should be cured and tested as per ASTM C31 and C42 but not soaked for 48 hours, and no L/D corrections should be made in reporting results. For approval of a mix, nine cores each from six test panels should be submitted. Three of the panels should be

shot overhead. Three samples from each panel for each required age should be averaged, and these averages combined to give an overall average. For compliance, the overall averages should be 885 psi in 8 hours, 3300 psi in 3 days, and 4430 psi in 28 days (in the case of an in-situ f'c of 4000 psi).

b. For testing during construction, three cores should be taken from the area sprayed, for every 250 square feet of shotcrete placed. All three cores in each set should come from the same area (sidewall, quarter arch, and crown), and the average of each set of three cores tabulated separately. Cores should be cut to three-inch lengths and should not include embedded wire mesh or severe laminations between layers. Cores should be taken no more than three days before testing, and stored in a standard moist room as indicated by ASTM C31 until tested. Average, uncorrected core strengths should be 3740 psi at 28 days for three cores from each location. The 28-day strength given is equivalent to 3400 psi on cores whose L/D equals two. Minimum core strength should be 3300 psi. All tests should be performed parallel to the direction of shooting.

6. For testing during construction, three cores should be removed from the same area to represent the test described in ACI 503.2-77, and tested for compliance. The average of the three samples must exceed 85 percent of the required in-situ strength, f'c, and none of the three should be less than seventy-five percent. Arch as well as sidewall sampling areas must be tested.

## APPENDICES

A. CN120 Specifications for Shotcrete.

B. Blanck-Alvarez Shotcrete Equipment.

## SECTION 03YT - SHOTCRETE

### PART 1 - GENERAL

1.01 DESCRIPTION.

A. The work specified in this Section consists of the application of shotcrete to the specified thickness at the locations indicated, and furnishing of materials, equipment, tools and labor necessary to perform the preparation, application and the clean-up pertaining thereto. Shotcrete may also be applied locally at the Contractor's option to facilitate operations under this Contract.

B. At the option of the Contractor, shotcrete may be applied by either the dry-mix process or wet-mix process provided all requirements here are met.

C. Definitions.

1. Shotcrete, for the purpose of this work, is defined as a Portland Cement Concrete, containing aggregate up to one inch in size, with an approved accelerator, if required, applied from a spray nozzle by means of compressed air. Shotcrete shall attain quick set and high early strengths as specified herein.

2. The dry-mix process, means thoroughly mixing the solid materials, feeding these materials into a special mechanical feeder or gun, carrying the materials by compressed air to a special nozzle, introducing the water and intimately mixing it with the other ingredients at the nozzle. The mixture is then jetted from the nozzle at high velocity onto the surface to receive the shotcrete.

3. The wet-mix process, means thoroughly mixing all the ingredients except the accelerator but including the mixing water, introducing the mixture into the delivery equipment and delivering it by positive displacement or compressed air to the nozzle. The mixed shotcrete shall be air-jetted from the nozzle at high velocity onto the surface in the same manner as for the dry-mix process. The accelerator for the wet-mix process shall be added to the shotcrete mixture in such a way that the quantity can be properly regulated and the material uniformly dispersed throughout the shotcrete when it is applied.

1.02 QUALITY ASSURANCE.

A. Nozzlemen shall have had previous satisfactory experience in the application of coarse aggregate shotcrete on at least two projects of comparable nature, or shall work under the immediate supervision of a foreman or instructor with at least five years of such experience. Each crew shall demonstrate, to the satisfaction of the Engineer, acceptable proficiency in the application of shotcrete of Field Trial quality to vertical and overhead test panels before beginning production work.

B. Allowable Tolerance. Thickness of individual layers and tolerances shall be as indicated.

C. Mix Design Criteria.

1. The shotcrete mix shall be developed by laboratory tests and field trials as indicated herein at least 30 days prior to the actual application of shotcrete to any surface forming a permanent part of the work under this Contract. Laboratory trial mixes shall be made with exactly the same ingredients proposed for use in the work. Certification that ingredients comply with the specifications shall accompany the mix design. The proportions of shotcrete mix shall be equivalent to those of a concrete mix having between 6.5 and eight bags per cubic yard. The proportion of accelerator shall not exceed

two percent unless required by placement conditions; in no case shall it exceed six percent of the cement weight.

2. Compressive Strength. The mix design shall be such as to develop strength progressively as follows:

| | |
|---|---|
| Time of set and initial setting | 90 seconds minimum, 5 minutes maximum |
| Time for final setting | 12 minutes minimum, 20 minutes maximum |
| Compressive strength in 8 hours | 800 psi minimum |
| Compressive strength in 72 hours | 2000 psi minimum |
| Compressive strength in 28 days | 4000 psi minimum |

Strengths stated above are for test specimens having a length to diameter ratio (L/D) of two.

3. Accelerating Admixtures. Accelerating admixture shall be used as required to meet the time schedule specified for development of the specified progressive strengths.

4. Laboratory Tests.

   a. Prior to making laboratory tests, a detailed plan shall be submitted showing the methods and materials to be used in such tests. The Engineer reserves the right to witness the tests at any time.

   b. Cement-admixture set time compatibility tests should be performed as described in Shotcrete Practice in Underground Construction - Mahar et al, DOT Report FRA-OR & D 75-90, Section D.1, using herein specified setting times.

   c. A trial mix shall be made of each proposed cement and aggregate mix, without accelerator and with sufficient water for a slump from one to, not more than two inches. However, to compensate for cement enrichment due to rebound, the cement content in the laboratory test cylinders may exceed that proposed for the shotcrete by not more than 20 percent.

   d. Standard six inch by 12 inch cylinders shall be cast from each mix and moist cured by standard ASTM procedures for 28 days. The cylinders shall then be tested by standard ASTM procedures. Mixes not testing at least 140 percent of the specified 28 day strength shall be rejected.

   e. The compressive strength shall be determined in accordance with ASTM C 109.

5. Implementation.

   a. The Engineer will inform the Contractor, in writing, of his acceptance of mixes which meet the requirements. No shotcrete mix shall be used in field trials that has not been accepted by the Engineer.

   b. The exact proportions of ingredients determined on the basis of trial mixes shall be used in the actual application of shotcrete and shall not be varied without the written approval of the Engineer.

D. Job Mock-up and Field Trials.

1. After completion of the laboratory tests and their acceptance, field trials shall be made using approved mixes acceptable to the Engineer to demonstrate capability of equipment, workmanship, and materials under field conditions at least 30 days prior to actual application of shotcrete in permanent work. The mixes selected for field trials shall be confined to those approved by the Engineer following the laboratory testing.

2. The field application of each mix selected for field trial shall be made on horizontal and vertical test panels to simulate construction conditions. Test panels shall be made on wood forms and shall measure not less than 20 inch by 20 inch by four inches. Test panels shall be cured in accordance with ASTM C31, except that test panels shall not be immersed.

3. Six, three inch diameter cores shall be taken from each overhead horizontal and each vertical test panel. Their ends shall be trimmed to provide cylinders three inches high. Except for cores for the eight-hour strength test, the cores shall be taken two days after shooting, moist cured, but not immersed, and shall be tested according to ASTM C42. The average strength of all cores from a test panel shall test at least 110 percent of the strength specified for each age strength. Mixes failing to meet this requirement shall be rejected.

4. All phases of field trial work shall be performed in the presence of a representative of the Engineer. Upon completion, submit at least 36 core specimens of each mix proposed for use in the work together with all relevant data which demonstrates conformance to the Specifications in all respects. The specimens will be tested by the Engineer at various stages of curing ages to verify conformance with these specifications.

5. All test reports shall contain the unit strength at failure with and without correction for L/D ratio and statistical quantity adjustment.

1.03 SUBMITTALS. The following shall be submitted in accordance with the SHOP DRAWINGS, PRODUCT DATA, AND SAMPLES section prior to proceeding with preliminary testing:

A. Specification for and description of proposed equipment for mixing and application of shotcrete;

B. Proposed proportions of shotcrete ingredients;

C. Proposed method of application of shotcrete;

D. Certification of specified materials to the referenced standards;

E. Samples of shotcrete ingredients;

F. Evidence of applicators' qualifications.

1.04 JOB CONDITIONS

A. Permanent drainage shall be installed where directed and in manner indicated. Such drainage will include pipes through the shotcrete.

B. Safety Measures. In applying shotcrete containing toxic admixtures, the nozzlemen and helpers shall wear appropriate hoods supplied with filtered air free of toxic or objectionable material. Gloves and necessary protective clothing also shall be worn to protect against dermatitis.

C. In addition to the lighting and ventilation required by OSHA for tunneling operations, areas to receive shotcrete shall be lighted by additional flood lights and additional exhaust ducts shall be installed and connected to the ventilation system.

1.05 MEASUREMENT.

A. Shotcrete will be measured by the square foot on the A line for the nominal thickness installed except that in subway line tunnels, shotcrete will not be measured for payment.

B. Welded wire fabric will be measured in accordance with the CONCRETE REINFORCEMENT section.

C. Seepage Collector drains will be measured by the linear foot installed including both drain and lateral piping.

1.06 PAYMENT

A. Shotcrete, except in subway line tunnels and where used in the Contractor's safety operations, will be paid for at the Contract unit prices per square foot for SHOTCRETE CAVERN.

B. The costs shall include all labor, equipment and materials, required for development of trial mixes and their testing, test specimens, measuring pins, testing, and curing. No separate measurement will be made of shotcrete used to fill overbreak, nor for shotcrete wasted or rejected for any purpose, nor to prefill irregularities in surfaces to be shotcreted.

C. Welded wire fabric will be paid for in accordance with the CONCRETE REINFORCEMENT section.

D. Shotcrete lining for subway line tunnels will be paid for as part of the Contract unit price per linear foot for LINE TUNNEL TYPE C.

E. Seepage collector drains, including connections, will be paid for at the Contract unit price per linear foot for SEEPAGE COLLECTOR DRAINS.

F. Shotcrete used for the Contractor's safety, shall be furnished and placed at no additional expense to the Authority.

PART 2 - PRODUCTS

2.01 CEMENT. Cement shall conform to ASTM C 150, Type I. Type III cement may be used, if accepted by the Engineer, at no additional expense to the Authority.

2.02 AGGREGATE. Fine and coarse aggregate shall conform to the requirements of the PORTLAND CEMENT CONCRETE section except as hereinafter specified. The gradation of the combined coarse and fine aggregate mixture shall conform to the following limits:

| U.S. Standard Sieve Size | Percent Passing Gradation |
|---|---|
| 1 inch | 100 |
| 3/4 inch | 93- 98 |
| 1/2 inch | 80- 95 |
| 3/8 inch | 65- 85 |
| No. 4 | 50- 70 |
| No. 8 | 34- 56 |
| No. 16 | 22- 43 |
| No. 30 | 12- 32 |
| No. 50 | 5- 20 |
| No. 100 | 2- 10 |
| No. 200 | 0- 5 |

All aggregates shall be uniformly well graded and shall not exhibit extremes of variation. The maximum size of the aggregate may be varied subject to acceptance of the Engineer.

2.03 ADMIXTURES

A. Accelerating Admixture for use in the dry or wet mix process shall not contain chlorides or materials corrosive to steel and shall not entail other detrimental effects such as cracking and spalling. The use of any particular brand or type of admixture shall be subject to approval of the Engineer.

B. Water reducing and accelerating additives for use in the Wet Mix Process shall conform to ASTM C 494 Type E.

2.04 WATER. Water for shotcrete shall conform to the requirements of the PORTLAND CEMENT CONCRETE section.

2.05 WELDED WIRE FABRIC. Welded wire fabric shall conform to the requirements of the CONCRETE REINFORCEMENT section.

## PART 3 - EXECUTION

3.01 PROPORTIONING AND MIXING.

A. Proportioning of aggregate and cement shall be accomplished on a weight or volumetric basis by a suitable batching plant. The batching plant and proportioning devices shall conform to the applicable provisions of the PORTLAND CEMENT CONCRETE section.

B. The moisture content of the combined aggregate at the time of mixing with cement shall be in the range of three percent to six percent of the oven-dry weight of the aggregate for aggregate used in the dry-mix process.

C. Cement and aggregates shall be brought to the shotcreting site separately and mixed at the site.

D. The accelerating additive shall be added immediately prior to final mixing, or if in liquid form and for the dry-mix process, shall be accurately proportioned into the water supply by metering at the application nozzle. Dry additive whether powder or finely ground from a solid at the mixer shall be accurately proportioned by mechanical means and shall be thoroughly mixed with the other ingredients. All additives shall be added by mechanical means. The dry process shall have the powder additives proportioned by mechanical means at the mixer; liquid, proportioned by metering at the nozzle.

3.02 PLACING EQUIPMENT.

A. Dry Mix Process. Placing equipment shall consist of a spray nozzle providing for ejection of materials and water in an intimate mixture, separate hoses to deliver dry materials and water to the nozzle, a suitable machine to introduce the dry materials to the delivery hose under air pressure, and air and water supply system. The water supply system shall consist of a local reservoir and a positive displacement pump capable of supplying water through a regulating valve, easily and accurately controllable by the nozzleman, in sufficient amount and at pressure recommended by the manufacturer of the delivery machine. The entire system shall be so arranged that the nozzleman may use air and water in any combination to prepare surfaces on which shotcrete will be applied.

B. Wet Mix Process. Placing equipment for wet-mix process shall be capable of handling and applying shotcrete containing the specified maximum size aggregate and accelerating and hardening admixture.

C. Both Processes.

1. The air supply system shall be capable of supplying the delivery machine and hose with air at the pressures and volumes recommended by the manufacturer of the machine. No air supply system shall be used that delivers air contaminated by oil or that is incapable of maintaining constant pressure.

2. The delivery machine shall be capable of introducing materials to the delivery hose at a uniform rate, with ejection from the nozzle at velocities that will afford adherence of material to the treated surface with a minimum rebound and maximum adherence and density.

3. A separate air hose and blow pipe shall be available to remove dust and rebound during shotcrete application.

4. The equipment shall be maintained in clean and proper operating condition satisfactory to the Engineer.

3.03 SHOTCRETE APPLICATION - GENERAL.

A. Surfaces, which are to receive shotcrete whether new or previously shotcreted, shall be cleaned of all loose material, mud and other foreign matter which is not automatically removed by the shotcreting operation and shall be washed with a combination water and high velocity air jet. The surface shall be moist at the time shotcrete is applied. The nozzle shall be held at a predetermined distance and position so that the stream of flowing material shall impinge as nearly as possible at right angles to the surface to be covered. Shotcrete of the approved mix design shall be applied in a circular fashion to build up the required thickness of layer. Equipment shall be provided to allow application of shotcrete to surfaces at an approximate range of 3 1/2 feet to five feet from the nozzle. The surface of each shotcrete layer shall be uniform and free of sags, drips, or runs.

B. Prior to application of shotcrete, approved measuring pins shall be furnished and installed on the surfaces to be treated for the purpose of indicating the thickness of shotcrete layers.

C. Pins shall be noncorrosive and so designed as not to cause infiltration of water through the shotcrete. Pins shall be installed on five foot centers in longitudinal and transverse directions, and at other locations as directed by the Engineer.

D. Rock reinforcement shall be installed prior to applying the first full layer of shotcrete in accordance with the ROCK REINFORCEMENT Section except between Station NR 28+00 and NR 29+10 and in other, localized areas where, in the opinion of the Contractor and the Engineer an immediate application is required for safety. Unless indicated otherwise, all laitance, loose material and rebound shall be removed and the surface layer sounded with a hammer for voids, rebound or aggregate pockets, and unbonded areas. Defective areas shall be removed and replaced. Shotcrete shall be built up in individual layers not more than four inches thick.

3.04 SHOTCRETE APPLICATION. Between approximately Station NR 28+40 and Station NR 29+10 the roof of each drift shall be shotcreted individually at the heading.

3.05 FIELD QUALITY CONTROL.

A. Inspection. The Engineer will inspect each shotcrete layer visually and by sounding with a hammer. "Drummy" sounding shotcrete shall be considered as defective shotcrete.

B. Test Cores.

1. The Engineer shall be provided with one core, each three inches in diameter taken from each 250 square feet of arch shotcreted.

2. If any cores taken fail to show adequate bond with the rock or show obvious defects, two additional cores shall be taken within approximately five feet of the unsatisfactory core. If either of these fail to show adequate bond with the rock or show obvious defects, an area of shotcrete surrounding the unsatisfactory cores of the size

determined by the Engineer shall be removed and replaced with new shotcrete.

3. Additional specimens may be required at any time by the Engineer. Should additional specimens show acceptable strength, the Contractor will be reimbursed for the cost of obtaining such additional cores. Should these specimens fail, the cost of additional specimens shall be at no additional expense to the Authority.

4. All core holes shall be carefully plugged with shotcrete.

C. Testing. The Shotcrete cores shall be tested in accordance with ASTM C42. Cores with obvious defects shall not be tested; additional cores shall be taken as required. If the average strength of each set of these cores is less than 85 percent of the specified 28 day strength (97.5 percent when adjusted for a length: diameter ratio of 1), remedial work shall be performed, including application of additional thickness of shotcrete or removal and replacement of the defective shotcrete. Such remedial work shall be performed at no additional expense to the Authority.

D. If the shotcreting system selected by the Contractor fails to provide satisfactory in place shotcrete in accordance with these specifications, as determined by the Engineer, the Contractor shall be required to change to another system of either of the two processes.

3.06 PLACING OF WIRE FABRIC. Welded wire fabric shall be placed as indicated. Adjacent lengths of fabric shall be lapped not less than eight inches and the laps wired together.

3.07 DEFECTIVE SHOTCRETE. Shotcrete which lacks uniformity, exhibits segregation, honeycombing, lamination, shows cracking, lacks water-tightness, or is "drummy" shall be regarded as defective shotcrete. The Engineer reserves the right to order removal of defective shotcrete and its replacement with acceptable shotcrete without additional cost to the Authority. Any remedial measure ordered by the Engineer to correct defective shotcrete shall be at the expense of the Contractor.

END OF SECTION

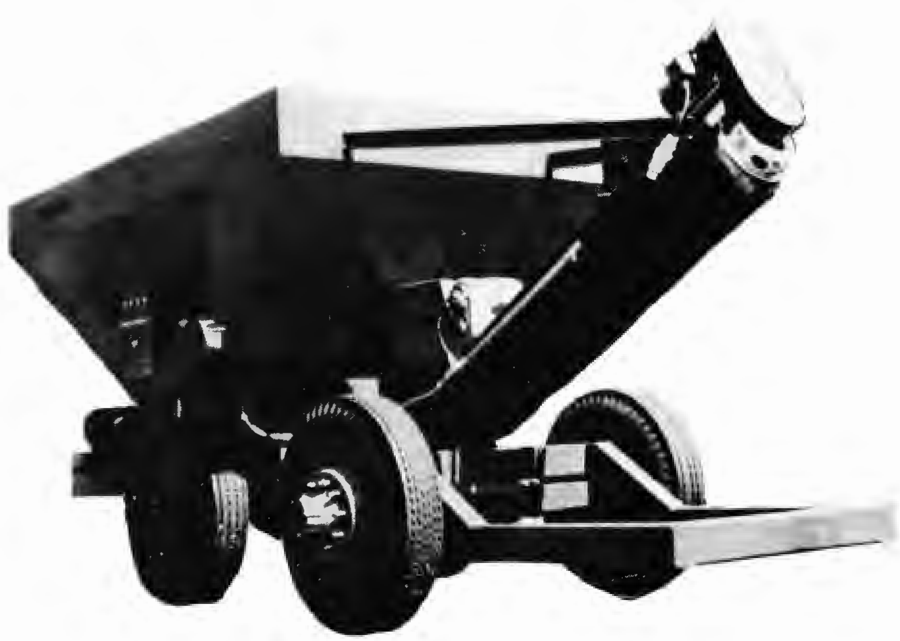
BLANCK - ALVAREZ

SHOTCRETE EQUIPMENT

4 C.Y. VOLUMETRIC BATCHER-MIXER

SPECIFICATIONS:

| | | |
|---|---|---|
| BIN CAPACITIES | SAND | 100 CU. FT. |
| | ROCK | 72 CU. FT. |
| | CEMENT | 35 CU. FT. |
| | ADDITIVE | 1 CU. FT. |
| DIMENSIONS | LENGTH | 10 FT. 8 IN. |
| | WIDTH | 6 FT. 2 IN. |
| | HEIGHT | 5 FT. 0 IN. |
| OVERALL DIMENSIONS (TRAILER MOUNTED) | LENGTH | 19 FT. 6 IN. |
| | WIDTH | 7 FT. 0 IN. |
| | HEIGHT | 7 FT. 6 IN. |
| | WEIGHT APP. | 4,500 LBS. |
| DELIVERY CAPACITY | VARIABLE UP TO 18 CU. YD. PER HR. | |



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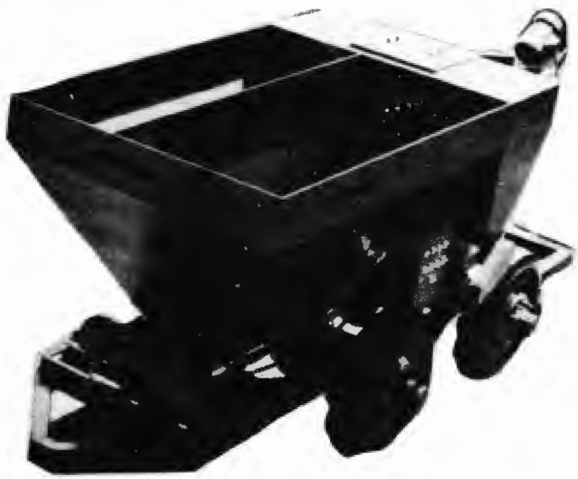


TECHNICAL:

AUGERS

| | |
|---|---|
| AGGREGATES | 2 EA. 8 1/2" x 6' 0"<br>3/8" THICK FLIGHTING |
| MIX AUGER | 8 1/2" x 8' 0"<br>3/8" THICK FLIGHTING |
| CEMENT | CENTER FEED 6" x 6' 0"<br>1/4" THICK FLIGHTING |

DRIVES

| | |
|---|---|
| AGGREGATES | WINSMITH/RELIANCE TEFC 5 H.P.<br>RIGHT ANGLE GEAR REDUCER |
| MIX AUGER | FALK/RELIANCE TEFC 5 H.P.<br>SCREW CONVEYOR DRIVE |
| CEMENT | DAYTON TEFC 2 H.P. ADJUSTABLE<br>SPEED GEARMOTOR |

*NOTE: ALL MOTORS 230/460 V, 3 PHASE, UNLESS OTHERWISE SPECIFIED.

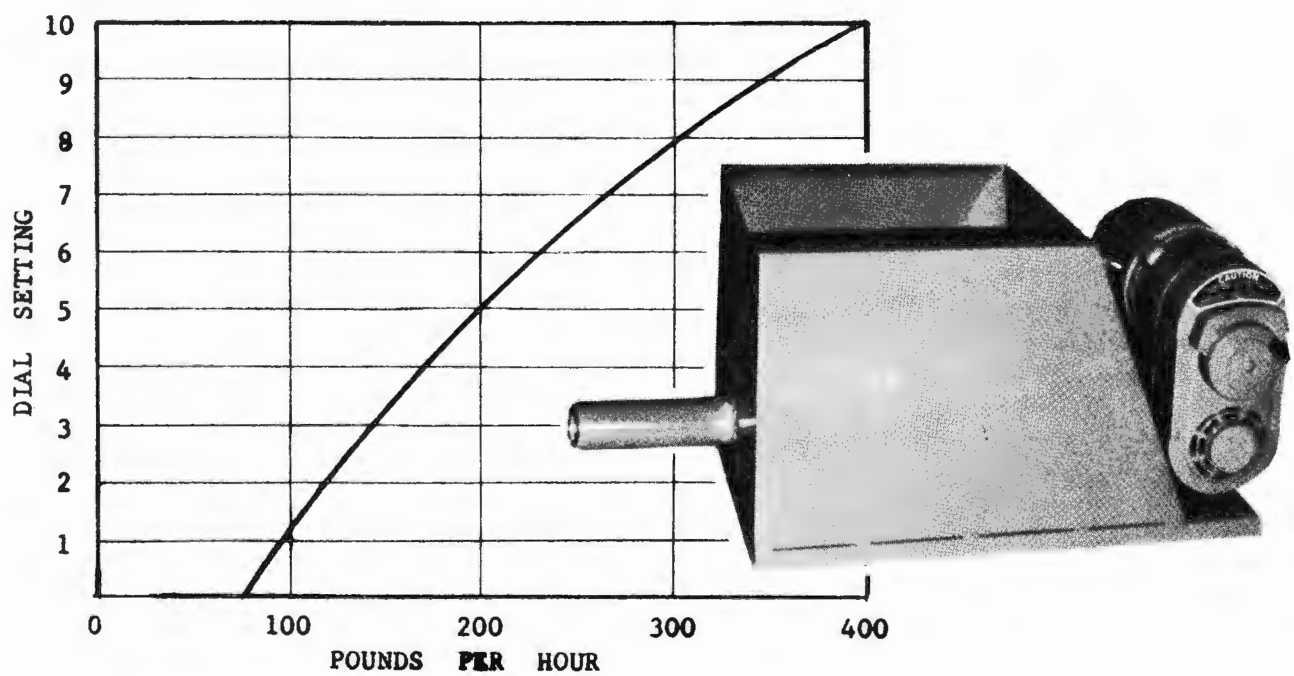
BLANCK - ALVAREZ

SHOTCRETE EQUIPMENT

POWDER ADDITIVE FEEDER MODEL 771

SPECIFICATIONS:

| | | |
|---|---|---|
| HOPPER | VOLUME | 1 CUBIC FOOT |
| | DIMENSIONS | LENGTH 30 IN.<br>WIDTH 15 IN.<br>HEIGHT 15 IN.<br>WEIGHT 85 LBS. |
| DRIVE | 1/2 H.P. ELECTRIC CONSTANT TORQUE, TOTALLY ENCLOSED ADJUSTABLE SPEED GEARMOTOR | 115/230 V OR<br>200/230/460 V |
| CAPACITY | 80 - 400 LBS. PER HR.<br>20 - 110 RPM | |

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# CHAPTER VIII

# STEEL-FIBER-REINFORCED SHOTCRETE

## 8.1 INTRODUCTION AND SUMMARY

This chapter summarizes the results obtained from field shotcrete tests carried out at the Atlanta Research Chamber. Although the structural behavior of conventional shotcrete is also reported in this chapter, emphasis is given to results obtained with steel-fiber-reinforced shotcrete.

Preconstruction testing was carried out in two stages during June 1978 and November 1978. The primary emphasis of the preconstruction program was on determining the compressive strength variation with time through an age of 28 days, and then comparing the strength variations of various mixes.

Testing in the Research Chamber was carried out in March 1979 with the primary objective of determining the structural behavior of conventional and fiber-reinforced shotcrete placed in situ under conditions closely representing actual construction conditions.

In summary, it was found that for young shotcrete (8-12 hours), the compressive strength of steel-fiber-reinforced shotcrete was very similar to that of corresponding conventional shotcrete. At 8 to 12 hours, samples of steel-fiber-reinforced shotcrete with a new organic accelerator (Dryshot) could not be obtained; but sampling was achieved when conventional (Sigunit) inorganic accelerator was used. Variations in water content and admixture dosage produced significant variations in compressive strength.

For intermediate-age shotcrete (3 days), the compressive strength of steel-fiber-reinforced shotcrete was lower than for the corresponding conventional shotcrete.

At 28 days, Dryshot mixes in general had higher strength than Sigunit mixes. The compressive strength of steel-fiber-reinforced shotcrete was generally lower than for corresponding conventional shotcrete.

Compressive strengths for wall samples were higher than for overhead samples. Addition of fibers added ductility to the shotcrete, as shown by field pulldown tests. Adhesion of all shotcrete to the rock was greater than adhesion to laboratory surfaces; conventional shotcrete had a slightly greater adhesive strength than steel-fiber-reinforced shotcrete. Rebound was measured to be 22%, for steel-fiber-reinforced shotcrete placed in the Atlanta Research Chamber. (See the Monograph by Tom Buchanan which discusses the successful use of this steel-fiber-reinforced shotcrete for final lining of one of the twin Running Tunnels on the MARTA CN120 contract.)

## 8.2 DESCRIPTION OF TESTING PROGRAM

### 8.2.1 General Objectives of the Test Program

The data collected in the field tests were intended to provide information regarding the mix design, shooting procedure and structural behavior of both conventional and fiber shotcrete under conditions closely representing actual field environment. The following aspects were studied:

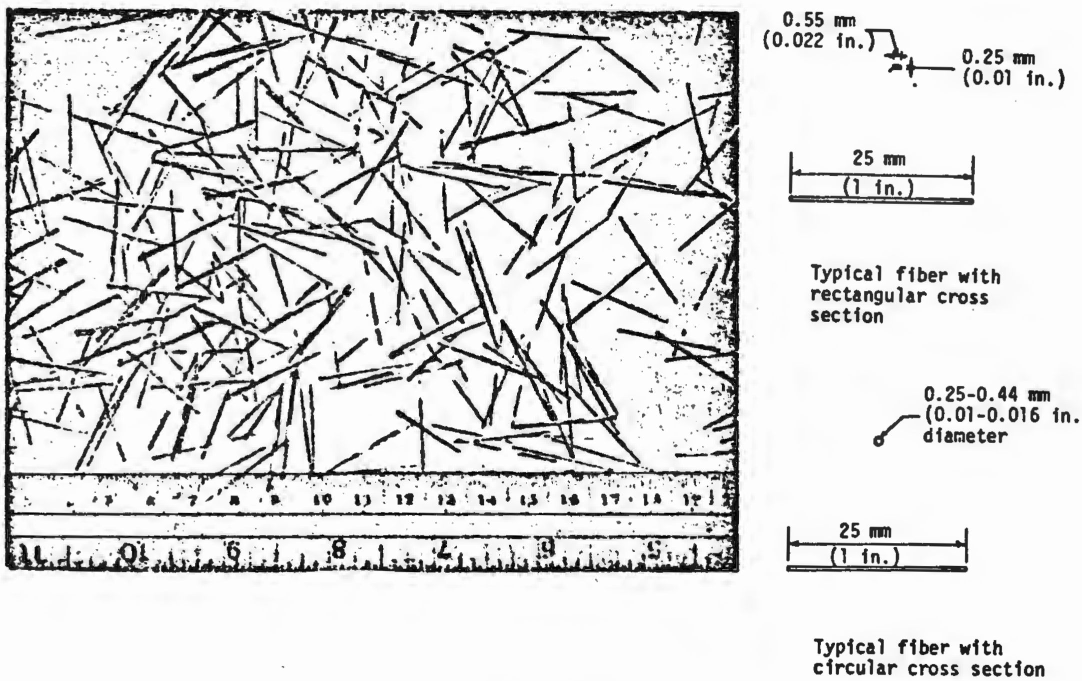
a. Whether the basic components (aggregates and cement) had the potential to produce a high quality shotcrete or not.

b. The cement-accelerator compatibility to produce the desired initial set under field conditions without damaging the long-term strength of the shotcrete material.

c. The batching and mixing of steel fibers with the other shotcrete aggregates, as well as their effect in the strength of the shotcrete materials.

Figure VIII-1: ICOMA Volumetric Batcher-Mixer Unit
with Fiber Feeder.

Figure VIII-3: Coring of Shotcrete Samples.



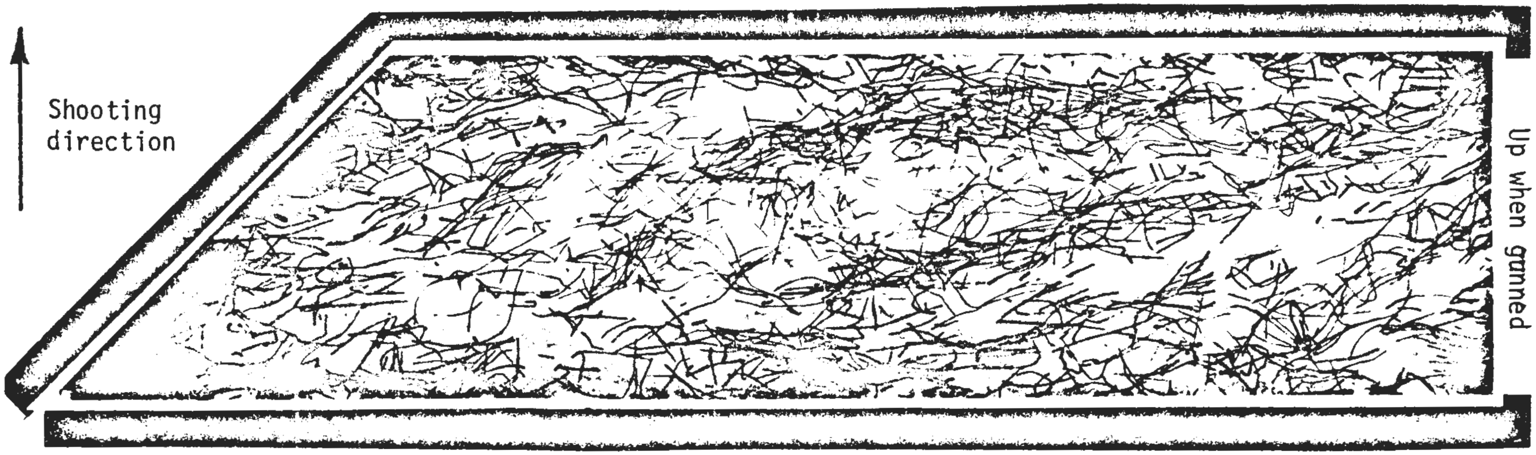

Steel Fibers before Shooting

Steel Fibers after Shooting

FIGURE VIII-2a TYPICAL FIBERS USED IN SHOTCRETE WORK


TYPICAL X-RAY PHOTOGRAPH OF STEEL-FIBER-REINFORCED SHOTCRETE

Figure VIII-2b




TYPICAL X-RAY PHOTOGRAPH OF STEEL-FIBER-REINFORCED SHOTCRETE

from: Parker, Harvey W., et al., Field-Oriented Investigation of Conventional and Experimental Shotcrete for Tunnels, DOT Report No. FRA-OR&D-76-06 (August, 1975).

d. The rebound rate ratio of fiber shotcrete and the characteristics of the rebound material.

e. The feasibility of the coring and testing techniques proposed for the measurement of the adhesion strength between the shotcrete and natural rock sufaces, as well as the values of this adhesion strength.

f. The structural behavior of the in-situ shotcrete liners with geometrical configurations similar to those tested earlier in the large scale model tests at the University of Illinois.1/

8.2.2 Test Sites

The first series of shotcrete preconstruction tests were performed during June 27-29, 1978 at the ground surface in a spot located approximately 100 ft. east of the south access shaft of the Peachtree Center Station, Atlanta. During November 16-18, 1978 a second series of preconstruction tests were performed at the bottom of the Luckie Street shaft. Finally, during March 22-24, 1979 shotcrete testing was performed in the Atlanta Research Chamber.

8.2.3 Shotcrete Equipment

The shotcrete equipment used in the tests was a 4 cubic yard, self-contained ICOMA unit shown in Figure VIII-1. In this self-contained unit the batching, mixing and gunning equipment are at one location. The batching, mixing, and conveyance of materials to the machine are all done automatically by means of a set of controls on the platform. Batching is done volumetrically with chains or belts located beneath the storage hoppers. Materials were batched almost instantaneously and thus there was little chance for prehydration or for waste of large volumes of batched materials. The batching equipment was checked periodically to insure that the materials were being supplied in the newly specified proportions.

1/ Fernandez, G., J. W. Mahar, and H. W. Parker, "Structural Behavior of Thin Shotcrete Liners Obtained from Large Scale Tests," Shotcrete for Ground Support, ASCE and ACI SP-54 (1976), pp. 399-442.

Mixing of the shotcrete materials was done with an auger that conveys the materials from the batching system to the shotcrete machine. The mixing time required for proper blending of the small volume of materials in the auger is very short, generally less than one minute. The powder accelerators were added and mixed with the materials at a small storage hopper located on the top of the auger. Liquid accelerators were gravity fed to the nozzle. The prepared materials were fed directly into the hopper of the shotcrete machine (see Figure VIII-19).

8.2.4 Materials

a. Cement and Accelerator

The cement used in these tests was SIGNAL MOUNTAIN CEMENT TYPE I. Two accelerating set admixtures were used: Sika Sigunit and Sikaset Dryshot.

b. Aggregates

The aggregate material was stockpiled in the construction site in a nearby area and was conveyed to the shooting unit by a front-loader vehicle. Two different types of coarse aggregates were used in these tests. A natural, round, "pea" gravel with a maximum diameter of 1/2 inch and a crushed stone aggregate (No. 7 instead of the initially planned coarser size No. 67) with a maximum size of 3/8 inch. The fine aggregate, sand, basically fell between the No. 4 and No. 200 sieves. The grain size distribution of these aggregates is shown in Chapter VII. A more detailed description of the aggregates used and their proportioning is given in Chapter VII.

c. Fibers

The fibers used in this shotcrete work were circular in cross section, 0.025 cm (.01 inch) in diameter, and have an approximate length of 2.54 cm (1.0 inch). These fibers (shown in Figure VIII-2) are made by U.S. Steel Company and are designated as U.S. fibers.

d. Mix Design

The standard mix in the tests corresponded to a 7 bag (658 lbs. per cubic yard) mix. The batch proportions, per cubic yard, are given in Table VIII-1.

Table VIII-1

STANDARD MIXES USED IN TESTS

| | Standard Conventional Shotcrete Batch Weights (lbs) | Standard Steel-fiber-reinforced Shotcrete Batch Weights (lbs) |
|---|---|---|
| Cement | 660 | 660 |
| Fine Aggregate | 1790 | 1790 |
| Coarse Aggregate | 1300 | 1300 |
| Fibers | -- | 115 |
| Total | 3750 | 3865 |

As indicated in Table VIII-1, the fine and coarse aggregate proportions in both the conventional and the steel fiber shotcretes in the tests were equal to 60 and 40 percent respectively. The weights given in Table VIII-1 include the moisture in the aggregate material which ranged between 5 to 8% in the sand and 1.5 to 2% in the gravel.

## 8.3 PRELIMINARY TESTING

### 8.3.1 Standard Cylinder Testing

Standard compression cylinders were cast in June 1978 using the sand and crushed-stone aggregate initially proposed with eight to nine sacks cement content. This high cement content attempts to duplicate the in-place cement-enriched shotcrete due to the higher aggregate rebound rates. The mix was designed to have a water/cement ratio of 0.45. A series of cylinders were cast using different dosages of the proposed accelerators.

Compressive strength results obtained from these standard cylinders indicated that the proposed aggregate (No. 67) and cement mix meet the requirements of the CN 120 specifications.

On November 22, 1978, three standard compression cylinders were cast using the cement, sand and aggregate used in the shotcrete preconstruction testing. During casting, additional water (beyond that corresponding to the W/C ratio of 0.45) was added to cylinders #2 and #3 to produce a 2-inch slump. Test results did not satisfy the 140% of the specified 28-day strength required by the CN 120 specifications.

Three standard cylinders were cast and tested in March 1979 with the materials to be used in the construction testing. These test results met the required 28-day strength (see Chapter VII).

8.3.2 <u>Cement Accelerator Compatibility Tests</u>

The basic cement-accelerator combination to be used was selected based on results obtained from the cement-accelerator compatibility tests (Gillmore Needle tests, see Chapter VII) which yielded initial and final set times between the recommended (Blanck, 1974)[2/] 3 to 13 minutes range. The accelerator dosage was varied for some tests in order to fill overhead panels and reduce lamination. (See Chapter VII, Section 7.3.2 for details.)

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2/ Blanck, J. A. (1974), "Shotcrete Durability and Strength - A Practical Viewpoint," Use of Shotcrete for Underground Structural Support, ASCE and ACI SP-45, pp. 320-329.

## 8.4 SHOOTING PROGRAM

The test program consisted of three separate series of wood panel shooting listed in Table VIII-2.

Table VIII-2

DATES OF SHOOTING

| Series | | |
|---|---|---|
| 1 | Day 1 | June 27, 1978 |
| | Day 2 | June 28, 1978 |
| | Day 3 | June 29, 1978 |
| 2 | Day 4 | November 16, 1978 |
| | Day 5 | November 18, 1978 |
| 3 | Day 6 | March 22, 1979 |
| | Day 7 | March 23, 1979 |
| | Day 8 | March 24, 1979 |

For ease of reference, they will be referred to in this report as shooting day 1 to 8. Specific goals were established for each day. Mix characteristics and corresponding panels shot in each day are given in Table VIII-3.

Table VIII-3

MIX CHARACTERISTICS AND CORRESPONDING WOOD PANELS SHOT

| MIX CHARACTERISTICS | | | | | NUMBER OF PANELS | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | Vertical | | Overhead | | |
| Day | Coarse Aggregate | Admixture | Dosage % | Fiber % | No. | Code | No. | Code | Total |
| 1 | natural | SIGUNIT | 2 | - | 2 | S-1-V | 2 | S-1-H | 4 |
| 1 | natural | DRYSHOT | 2 | - | 2 | D-1-V | 0 | D-I-H | 2 |
| 2 | natural | DRYSHOT | 3 | - | 2 | D-2-V | 1 | D-2-H | 3 |
| 2 | natural | SIGUNIT | 2 | 3 | 2 | S-2-F-V | 1 | S-2-F-H | 3 |
| 3 | crushed | SIGUNIT | 2 | - | 2 | CS-3-V | 2 | CS-3-H | 4 |
| 3 | crushed | DRYSHOT | 1.3 | - | 2 | CD-3-V | 1 | CD-3-H | 3 |
| 4 | crushed | SIGUNIT | 2.5 | - | 3 | CS-4-V | 3 | S-4-H | 6 |
| 4 | crushed | DRYSHOT | 2.5 | - | 3 | CD-4-V | 0 | | 3 |
| 4 | crushed | SIGUNIT | 2.5 | 3 | 2 | CS-4-F-V | 1 | CS-4-F-H | 3 |
| 5 | crushed | DRYSHOT* | 3.0 | - | 3 | CD-5-V | 4 | CD-5-H | 7 |
| 5 | crushed | DRYSHOT* | 3.0 | 3 | 2 | CD-5-F-V | 0 | | 2 |
| 6 | crushed | SIGUNIT | 1.5 | - | 1 | CS-6-V | 0 | | 1 |
| 7 | crushed | DRYSHOT* | 4.0 | - | 1 | CD-7-V | 0 | | 1 |
| 8 | crushed | SIGUNIT | 1.5 | 3 | 3 | CS-8-F-V | 0 | | 3 |

* Accelerator premixed with water.

Explanation of the panel mix designation code is given in Table VIII-4.

Table VIII-4

EXPLANATION OF PANEL-MIX DESIGNATION

V = Panel shooting position (vertical = V; horizontal = H)

F = Fiber mix (in conventional mixes the letter F is dropped)

2 = Day of shooting

S = Admixture type (S for Sigunit; D for Dryshot)

C = Type of coarse aggregate*

* C indicates crushed stone mixes; the letter C was dropped for mixes in which natural gravel was used.

## 8.5 COMPRESSIVE STRENGTH OF FIBER SHOTCRETE FROM WOOD PANELS

The primary emphasis of this portion of the testing program was on the determination of the compressive strength-variation with time through an age of 28 days and the comparison of these strength variations between different steel-fiber-reinforced mixes. The earliest compressive strength was obtained at an age of 9 hours.

### 8.5.1 Test Procedures

Where possible, the applicable portions of the ASTM test methods (C39, C49 and C116) were followed. The methods were modified only as necessary to accommodate the type of specimen obtained in these programs. The specimens were three-inch diameter cylinders, ranging from 2.5 to 4.0 inches long. They were cored from the panels with the coring bit shown in Figure VIII-3. The cylinder axis was perpendicular to the surface of the panel, so that the back of the panel provided a smooth side at the bottom of the core. The other rough end of the core was trimmed off but no more than necessary to provide a smooth side parallel to the back. Before testing, the dimensions and weight of the specimens were determined. Each specimen was capped with a thin layer of sulphur-based capping compound that hardened to a compressive strength in excess of the strength of the shotcrete. An effort was made to center and align the specimen in the testing machine. Finally, the load was applied at a rate within the range specified by ASTM C39, until the load applied exceeded the maximum resistance of the specimen.

The maximum compressive stress (compressive strength) was computed by dividing the maximum load by the average cross sectional area of the specimen. Shotcrete compressive strengths reported herein are not corrected for L/D.

8.5.2 Compressive Strength Testing

The testing schedule followed in this program is shown in Table VIII-5; see the Appendices for details.

In the subsequent sections, the results of the early tests (9 to 11 hours) will be evaluated first. Next the 3-day results will be treated and finally the 28-day results will be discussed.

Table VIII-5

COMPRESSIVE STRENGTH TESTING PROGRAM
STEEL-FIBER-REINFORCED SHOTCRETE

Quantity of Core Samples Tested At:

| Mix-Code | 8-12 hours | 3 days | 28 days | Total |
|---|---|---|---|---|
| S-2-F-V | 6 | 6 | 6 | 18 |
| S-2-F-H | 3 | 3 | 3 | 9 |
| CS-4-F-V | 6 | 6 | 6 | 18 |
| CS-4-F-H | 0 | 3 | 3 | 6 |
| CD-5-F-V | 0 | 6 | 6 | 12 |
| CS-8-F-V | 3 | 3 | 3 | 9 |

### 8.5.3 Evaluation of Compressive Strength Results of Young Shotcrete

Since the variation in strength from sample to sample within each panel was very small, the average compressive strength measured in each panel, given in Table VIII-6, can be used to evaluate and compare the different steel-fiber-reinforced mixes shot. In general, as indicated in Table VIII-6, all mixes exhibited a high 8 to 12 hour compressive strength, ranging from 1150 to 2090 psi. As expected, the compressive strength of the fibrous shotcrete mixes was very similar to the compressive strength exhibited by corresponding conventional mixes.

Table VIII-6

AVERAGE COMPRESSIVE STRENGTH OF FIBER-REINFORCED SHOTCRETE AT 8 to 12 HOURS

| Panel Mix | Age Hours | No. of Specimens | Average Compressive Strength f'c (psi) | Standard Deviation (psi) |
|---|---|---|---|---|
| S-2-F-V | 8.6 | 6 | 1760 | 270 |
| S-2-F-H | 8.4 | 2 | 2090 | 110 |
| CS-4-F-V | 10 | 6 | 2030 | 620 |
| CS-8-F-V | 12 | 3 | 1150 | 110 |

Young samples, 8 to 12 hours, of Dryshot fiber-reinforced mixes could not be obtained in this testing program. This indicates that, at early ages, steel-fiber-reinforced mixes with Sigunit admixtures would provide a better and sounder in-situ shotcrete material.

Reductions in the Sigunit admixture dosage from 2% on the second day to 1-1/2% on the eighth day, resulted in a substantial, approximately 33%, reduction in the average compressive strength of the fiber mix; see Table VIII-3 and Table VIII-6. Variations in the amount of water added at the nozzle resulted in considerable variations in the average compressive strength of the fiber mixes shot on the fourth day as indicated by the standard deviation given in Table VIII-6. Careful examination of the shotcrete samples before and after testing indicated slight to no lamination in the shotcrete filling the panels. However, in some cases, lamination, usually located 3/4 to 1-1/4 inches from the back of the panel, was so pronounced in the overhead panels that samples broke at this lamination plane during the coring process.

8.5.4 Evaluation of Intermediate, 3 Days, Shotcrete Strength

Similar to the early compressive strength values, intermediate, (3 day) compressive strength also exhibited very small variation among samples of the same panel. Therefore, average compressive strengths, shown in Table VIII-7, could be used to evaluate the strength/time relationship for each mix as well as to compare strength/time relationships between mixes. As indicated in Table VIII-7, the intermediate (3 day) compressive strength of the shotcrete varied between 2890 and 6130 psi for all the mixes.

As previously observed in the early strength samples, a higher lamination potential was observed in the samples taken from overhead panels. Slight to moderate lamination was present in most of the samples obtained from overhead panels. Therefore for most mixes, samples taken from vertical panels exhibited, regardless of the admixture type or dosage, higher compressive strengths than those measured in corresponding samples taken from the horizontal (overhead) panels. Higher average compressive strengths measured in overhead panels marked with an asterisk in Table VIII-7 correspond to cases where samples broke along lamination during the coring process and only the intact portion of the shotcrete core was tested.

Variations in the amount of water added to the nozzle produced considerable variations in the average compressive strength between panels of the same mix as indicated by the high standard deviations of mix CD-5-F-V in Table VIII-7.

Table VIII-7

AVERAGE COMPRESSIVE STRENGTH OF FIBER-REINFORCED SHOTCRETE AT 3 DAYS

| Mix | Age Days | No. of Specimens | Average Compressive Strength (psi) | Standard Deviation (psi) |
|---|---|---|---|---|
| S-2-F-V | 3 | 6 | 2890 | 293 |
| S-2-F-H | 3 | 3 | 3670* | 204 |
| CS-4-F-V | 3 | 6 | 6130 | 199 |
| CS-4-F-H | 3 | 3 | 3160 | 148 |
| CD-5-F-V | 3 | 6 | 5330 | 1390 |
| CS-8-F-V | 3 | 3 | 4600 | 122 |

* Sample broke along lamination during coring.

In general, intermediate compressive strengths of fibrous shotcrete were lower than the strengths measured in corresponding conventional mixes. However, their absolute value was still high enough to satisfy the 60 percent f'c required at 3 days.

8.5.5 Evaluation of 28 Days Shotcrete Strength

As in the previous cases, there was a very small variation in the compressive strength among the samples of a given panel. Therefore, average compressive strengths, given in Table VIII-8, were used to evaluate the strength/time relationship for each mix as well as to compare this strength/time relationship between mixes.

The 28-day compressive strength varied between 3600 and 8660 psi for all mixes. The highest strength was measured in samples obtained from mixes containing 2.7 percent Dryshot. All samples from vertical panels exhibited larger strength than the corresponding samples from horizontal (overhead) panels; only short horizontal samples separated through pronounced lamination yielded compressive strengths higher than those obtained from corresponding vertical samples. Dryshot mixes in general exhibited 28 days compressive strengths higher than those exhibited by Sigunit mixes, regardless of the type of coarse aggregate and percentage of additive used.

Again, variations in the amount of water added at the nozzle (in this case affecting the admixture percentage, since Dryshot was premixed with water) were reflected in considerable variations in the compressive strength of mix CD-5-F-V as indicated by the high standard deviation in Table VIII-8.

As for the intermediate (3 days) compressive strength, the 28 days compressive strengths of fibrous shotcrete were generally lower than the strengths measured in the corresponding conventional mixes. However, the compressive strength of all fibrous mixes, except during the first trial in June 1978 when an appropriate fiber mixer was not available, satisfy the required 4000 psi f'c at 28 days.

Table VIII-8

AVERAGE COMPRESSIVE STRENGTH OF FIBER-REINFORCED SHOTCRETE AT 28 DAYS

| Mix | No. of Specimens | Average Compressive Strength (psi) | Standard Deviation (psi) |
|---|---|---|---|
| S-2-F-V | 6 | 3910 | 350 |
| S-2-F-H | 3 | 3600 | 470 |
| CS-4-F-V | 6 | 7100 | 780 |
| CS-4-F-H | 3 | 4250 | 510 |
| CD-5-F-V | 6 | 8660 | 1800 |
| CS-8-F-V | 3 | 6480 | 390 |

8.5.6 Conclusions

Compressive strengths of steel-fiber-reinforced shotcrete with Sigunit and Dryshot admixtures are plotted with respect to time in Figures VIII-4 and VIII-5.

As indicated in Figure VIII-4, fibrous shotcrete with normal 1.5 percent to 3 percent Sigunit dosages appeared to set quickly enough and gained considerable strength in the first 8-10 hours. The earliest compressive tests conducted on fibrous shotcrete with about 3 percent Sigunit resulted in an average f'c of 1500 psi at an age of 8 hours. Lower compressive strengths values about 1050 psi, were measured in slightly older samples (12 hours) with a reduced 1.5 percent Sigunit dosage. Except for the compressive strength of the earlier samples shot in the first preconstruction test of June 1978 when an adequate fiber feeder was not available (hatched area in Figure VIII-4), fiber-reinforced Sigunit mixes show a gain of strength with time that satisfies standard specifications.

Considerable variation in the strength of the shotcrete material was obtained at all times. There are many reasons why shotcrete under routine construction conditions of placement is a variable material. Some of the reasons for the variability are: 1) improper and inadequate blending of the accelerator before it reaches the wall, 2) the pulsatory nature of the material coming through the line, 3) the variations in the dry mix itself, 4) the variations that occur at the nozzle and at the wall during impact, and 5) the many variable factors associated with the nozzleman, although in this testing program the same excellent nozzleman, Mr. Warren Alvarez, performed all shooting.

Further differences in the laminar buildup of the shotcrete at different locations of the panel and the normal variability of water, cement and aggregates contents are also responsible for some of the differences in strength within panels of the same mix. Slight differences in nozzle distance and angle or

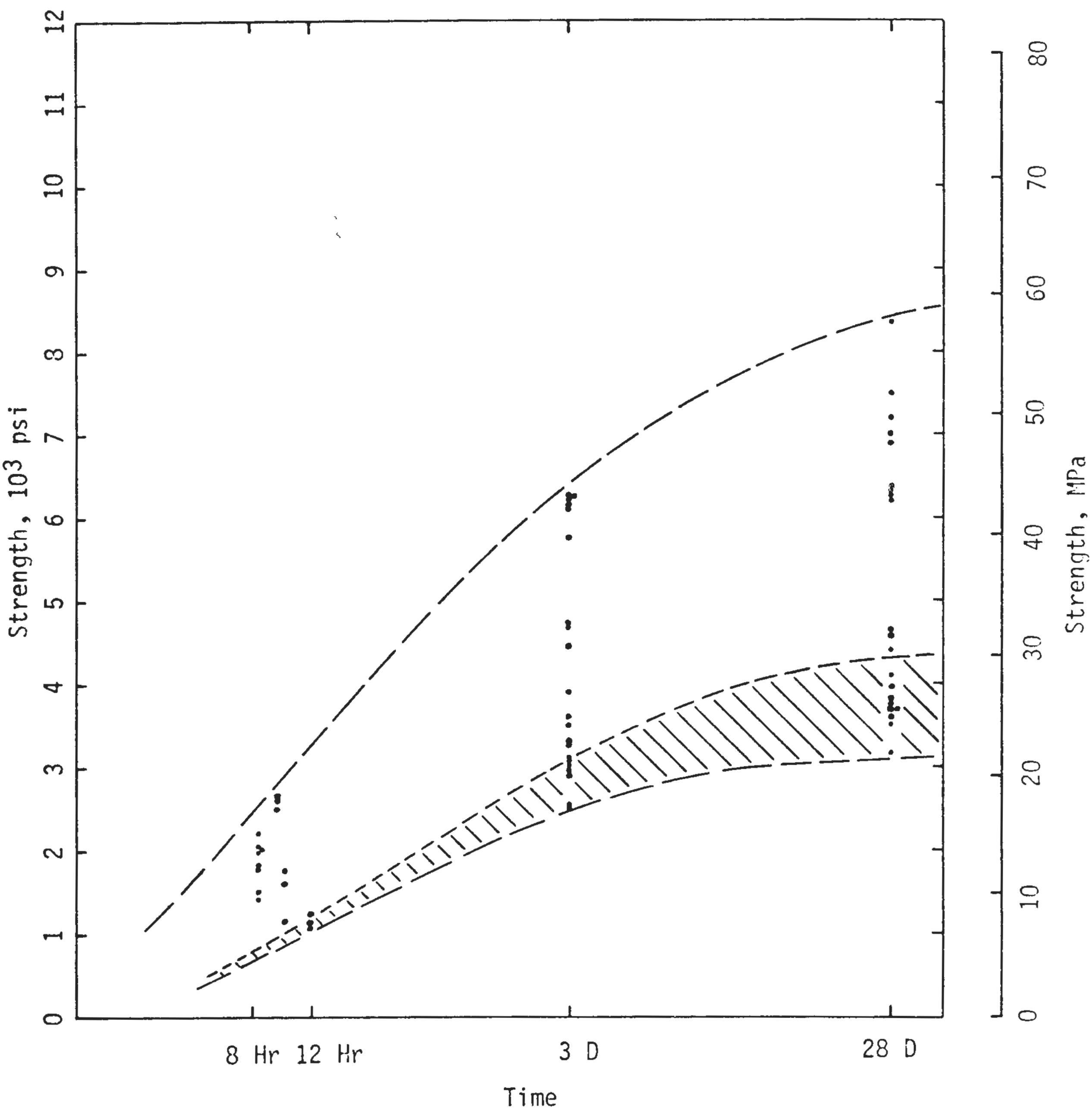


Figure VIII-4: Compressive Strength vs. Time for Sigunit Mixes with Fiber.

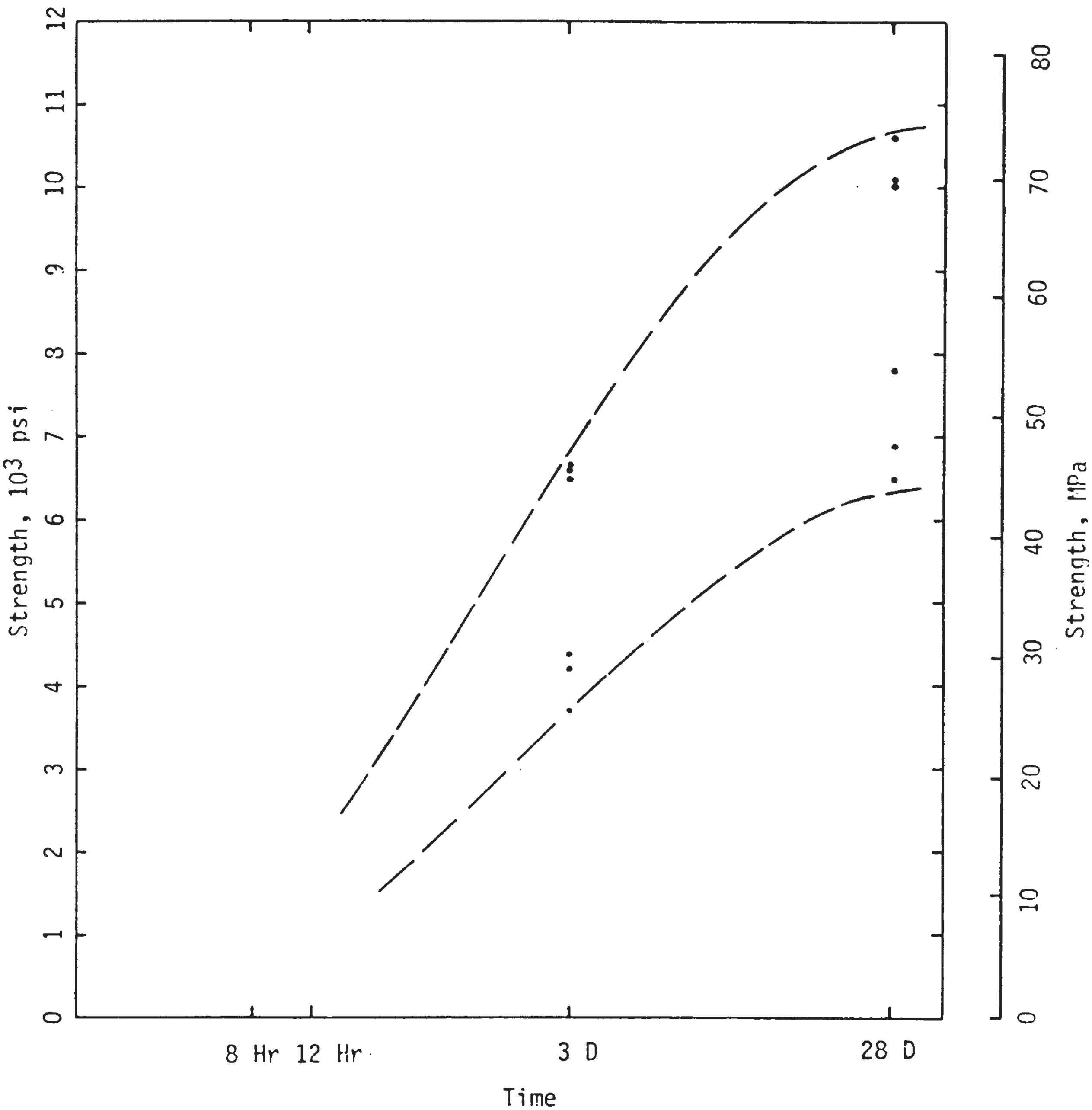


Figure VIII-5: Compressive Strength vs. Time for Dryshot Mixes with Fiber.

variations in the material delivery rate are also responsible for some of the observed scatter.

Despite all the scatter observed in this testing program the coefficient of variation in the average compressive strength of the mixes was low; it varied from 5 to 15% for most mixes except for those where substantial variations in the water added at the nozzle took place. A coefficient of variation of 10% is considered good for results of field tests of 6" by 12" cylinders of ordinary concrete at 28 days (Troxell, et al., 1968)3/. Thus, the panels shot in this program compare favorably as far as the variation of strength tests is concerned. Compressive strength values, at different times, obtained from fiber reinforced Dryshot mixes are shown in Figure VIII-5. As indicated in this figure, compressive strength values were not measured at early ages because samples were impossible to obtain. After 1 or 2 days of curing, Dryshot samples exhibited a considerable gain in strength; Dryshot mixes exhibited compressive strength values at 3 and 28 days considerably higher than those of corresponding Sigunit mixes. Scatter in the compressive strength values obtained from Dryshot samples is similar to that observed where Sigunit admixture was used.

Results indicate that in those situations where shotcrete for temporary support requires the highest strength attainable as early as possible, consistent with long-term strength requirements, the Sigunit mixes would be preferable to those where Dryshot is used.

## 8.6 FLEXURAL STRENGTH

The compressive strength program was complimented with flexural strength determination on beams sawed from fiber-shotcrete panels.

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3/ Troxell, G.E., H.E. Davis, J.W. Kelly (1968), Composition and Properties of Concrete, Second Edition, McGraw-Hill, pp. 513.

8.6.1 Details of Testing

The applicable portions of ASTM C78-64, Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading), were followed as closely as possible.

a. Preparation of Specimens

The specimens usually were 3 by 3 inches (7.6 x 7.6 cm) in cross section ranging from 14 to 19 inches (35.6 to 48.3 cm) long. They were sawed from test panels, and, in all cases, the orientation of the specimen was preserved so that bending was either in or out of the plane of the panel. The outer surface of all specimens older than 7 days was trimmed. Capping compounds or special cushions were used to provide full uniform contact between the loading points and rough surfaces. The dimensions of the specimen and its weight were recorded before testing.

b. Test Procedure

All specimens were tested for flexural strength using third-point-loading as illustrated schematically in Fig. VIII-6. To simulate field conditions, either the front or the back of the beam was on the tension side.

c. Calculation of Strength Parameters

The maximum flexural stress, or the modulus of rupture was calculated with the formula:

$$\sigma_f = \frac{P\ell}{bd^2}$$

where:

$\sigma_f$ = flexural stress (modulus of rupture)
P = maximum applied load
$\ell$ = span length
b = average width of specimen at the failure section
d = average depth of specimen at the failure section

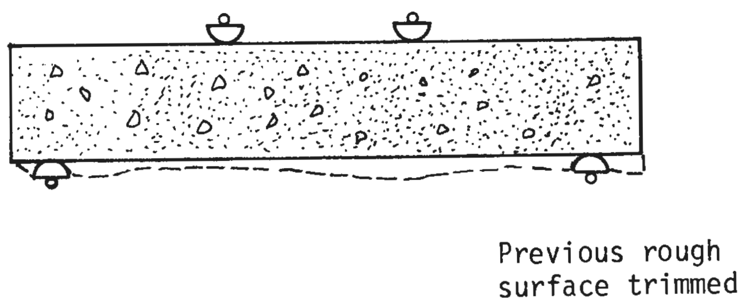



Figure VIII-6: FLEXURAL TESTING SET UP

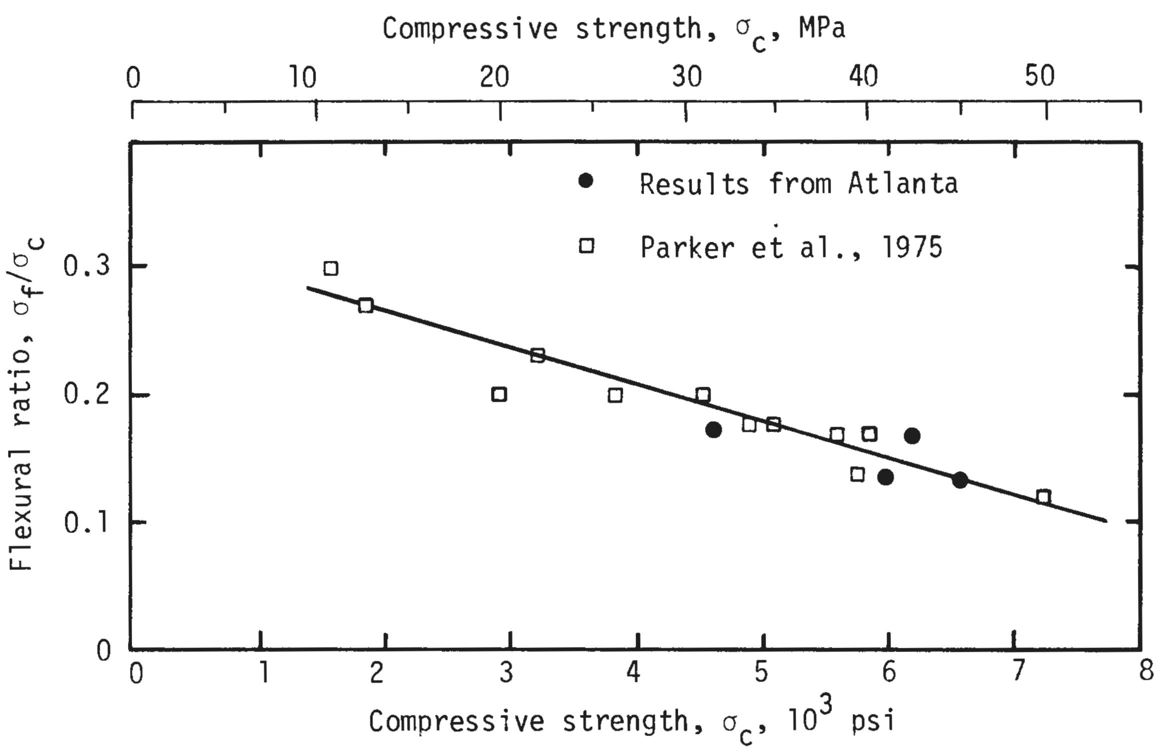



Figure VIII-8: AVERAGE FLEXURAL RATIO VERSUS AVERAGE COMPRESSIVE STRENGTH

8.6.2 Results of Flexural Tests

Results of flexural tests carried out in this program are given in Table VIII-9. As indicated in this table, the 28-day flexural strength of the mix with a 2.5% Sigunit dosage is about 23% higher than that of the mix with a 1.5% Sigunit dosage. These results are consistent with the compressive strengths developed by these two mixes.

There is almost no increase of flexural strength between 3 days and 28 days in the mix with 1.5% Sigunit. This has been observed in other tests (Parker, et al., 1975)[4/], and reflects the fact that the largest increase in flexural strength takes place very rapidly in the first days after shooting.

8.6.3 Relationship Between Flexural and Compressive Strength

Flexural strength is plotted against compressive strength in Fig. VIII-7 to illustrate the general nature of the relationship between these strengths. Also shown in Figure VIII-7 is a line corresponding to a flexural ratio $\sigma_f/\sigma_c = 0.19$, which has usually been found reasonably helpful (Parker et al., 1975) in averaging data obtained from previous field tests.

A plot of the flexural ratio, $\sigma_f$, versus the compressive strength $\sigma_c$ for the mixes tested is shown in Fig. VIII-8. As indicated in this figure, at approximately one month the $\sigma_f/\sigma_c$ ratio for the mixes tested was 0.13 and 0.17. The ratio for ordinary concrete with similar compressive strength ranges between 0.11 and 0.14 (Troxell et al., 1968). The difference may be explained by the higher cement content of the in-situ shotcrete.

Similar field tests carried out previously (Parker, et al, 1975) indicate a tendency for the flexural ratio to decrease

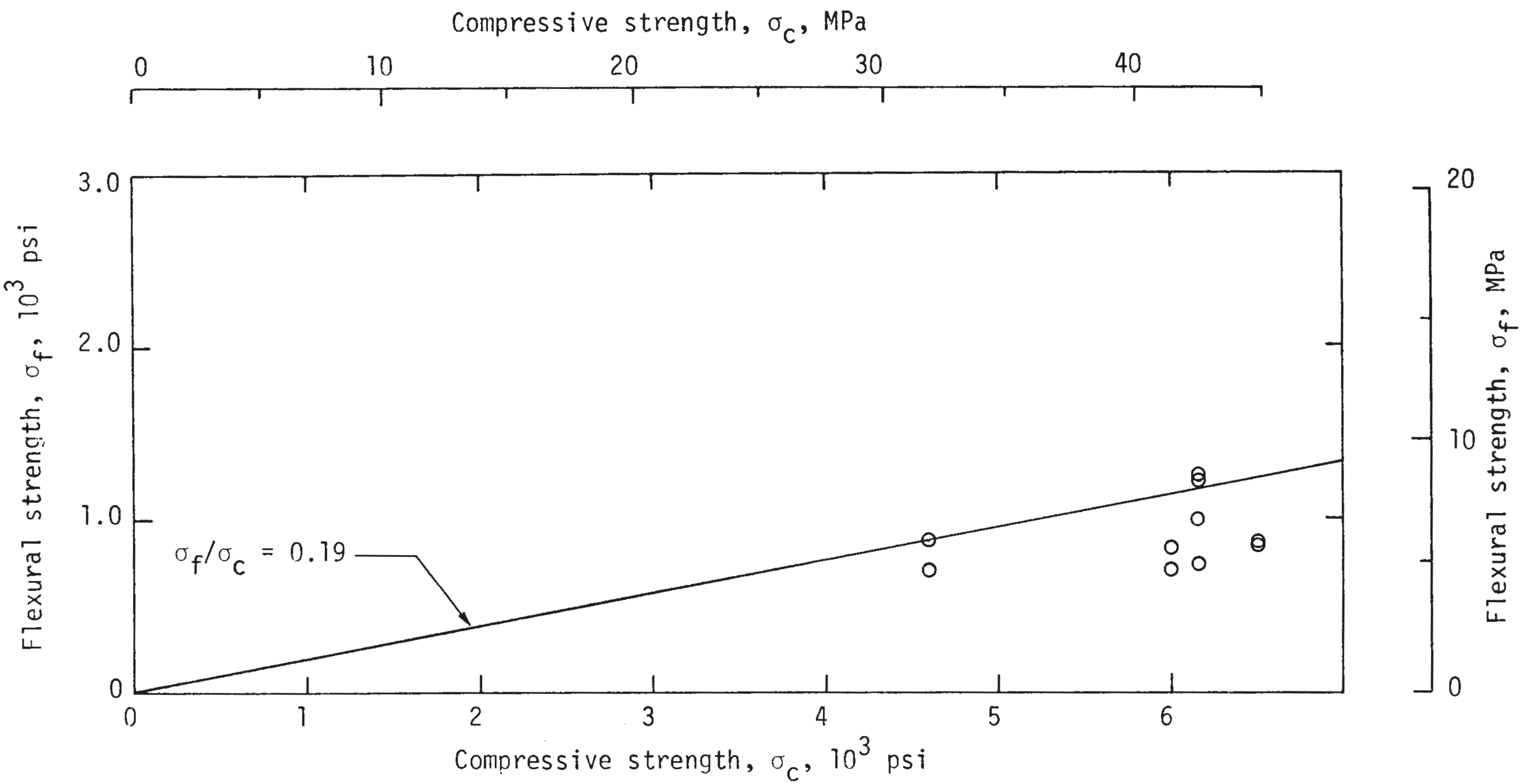
4/ Parker, H. W., G. Fernandez, and L. J. Lorig, "Field Oriented Investigation of Conventional and Experimental Shotcrete for Tunnels," DOT Report FRA OR&D 76-06, August 1975.

TABLE VIII-9
FLEXURAL STRENGTH OF FIBER REINFORCED SHOTCRETE

| | Mix Designation and Age at Testing (days) | | | |
|---|---|---|---|---|
| | CS-4-F-H | CS-8-F-V | | |
| | 28-day | 3-day | 10-day | 28-day |
| Flexural Strength $\sigma_f$ (psi) | 1005 | | | |
| | 742 | 705 | 714 | 861 |
| | 1241 | 881 | 839 | 871 |
| | 1257 | | | |
| Average Compressive Strength, $f'_c$ (psi) | 6150 | 4600 | 6000* | 6480 |
| Flexural Ratio = $\sigma_f/f'_c$ | 0.16 | | | |
| | 0.12 | 0.15 | 0.12 | 0.13 |
| | 0.20 | 0.19 | 0.14 | 0.13 |
| | 0.20 | | | |

* Value interpolated from compressive strength tests at 3 days and 28 days.

Figure VIII-7




FLEXURAL STRENGTH VERSUS COMPRESSIVE STRENGTH TEN DAYS
TO 28 DAYS OLD SHOTCRETE

with time, Figure VIII-8. Results from the few flexural test carried out in this program (circular points in Figure VIII-8), seem to confirm this tendency. It should be pointed out, as shown in Figure VIII-8, that flexural ratios of young shotcrete (8 to 12 hours old) could be as much as twice as high as older cured shotcrete.

8.6.4 Effect of Steel Fiber on Flexural Strength

Although no flexural tests were carried out for mixes without fiber, previous tests performed by the University of Illinois have shown that fiber shotcrete has a peak flexural strength comparable to that of conventional shotcrete as shown in Figure VIII-9. However, conventional shotcrete has a brittle behavior in bending, whereas fiber shotcrete develops a substantial ductility. This particular property of fiber-reinforced shotcrete makes it very advantageous in the support of temporary loads, especially in the case of loosening ground.

## 8.7 STRUCTURAL BEHAVIOR OF SHOTCRETE

Four tests were carried out to evaluate the capacity of the shotcrete placed in-situ. For each test, a 2-foot x 2-foot x 2-inch thick steel plate was placed in contact with the rock and covered with a layer of shotcrete slightly wider than 2 feet and extending some 8 feet away from each side of the plate. After the shotcrete had set, the plates were pulled at their center with a hydraulic jack and the load measured with a load cell. Figure VIII-10 shows the test set-up.

Two plates were covered with conventional shotcrete and two with fiber shotcrete. The geometrical configuration of the tested layers was made as similar as possible to some of the simple geometrical configurations tested in the large-scale tests of thin shotcrete liners performed at the University of Illinois (Fernandez, et al., 1976). Figure VIII-11 shows the geometrical configuration of the tested layers. They were selected so that

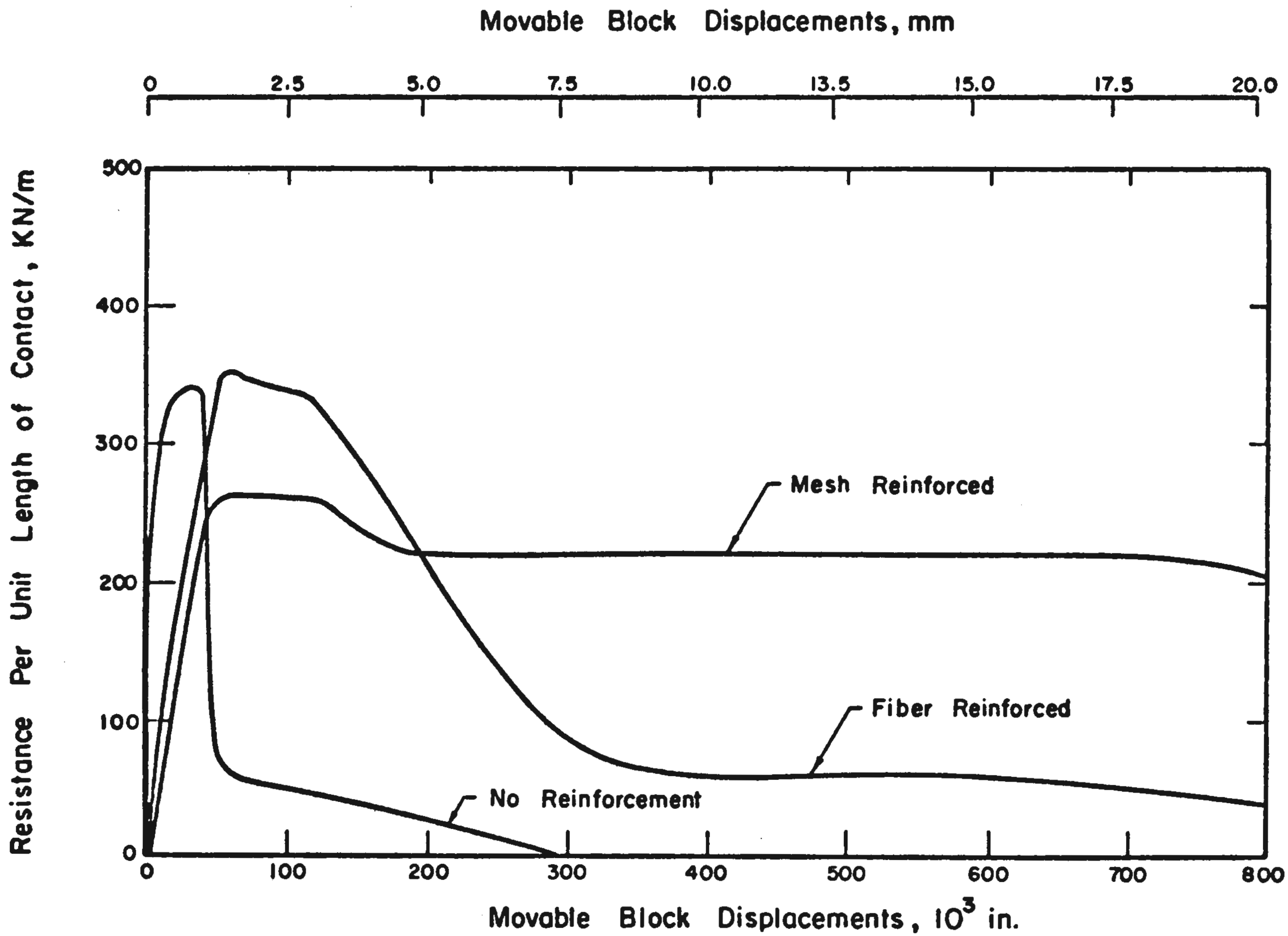
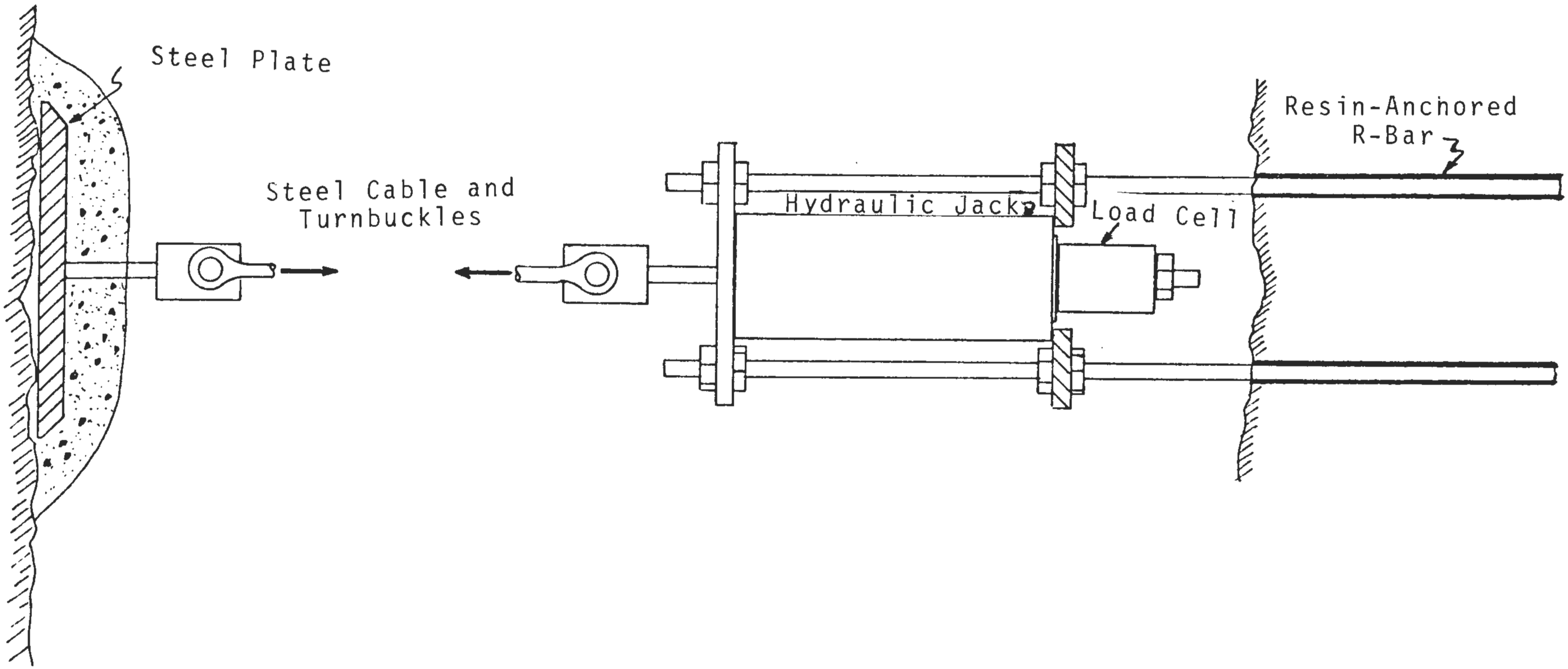



Figure VIII-9

Typical Load-Displacement Relationships for Laterally Restricted Layers in the Planar Configuration.

Figure VIII-10




Schematic Representation of Plate Tests Set Up.

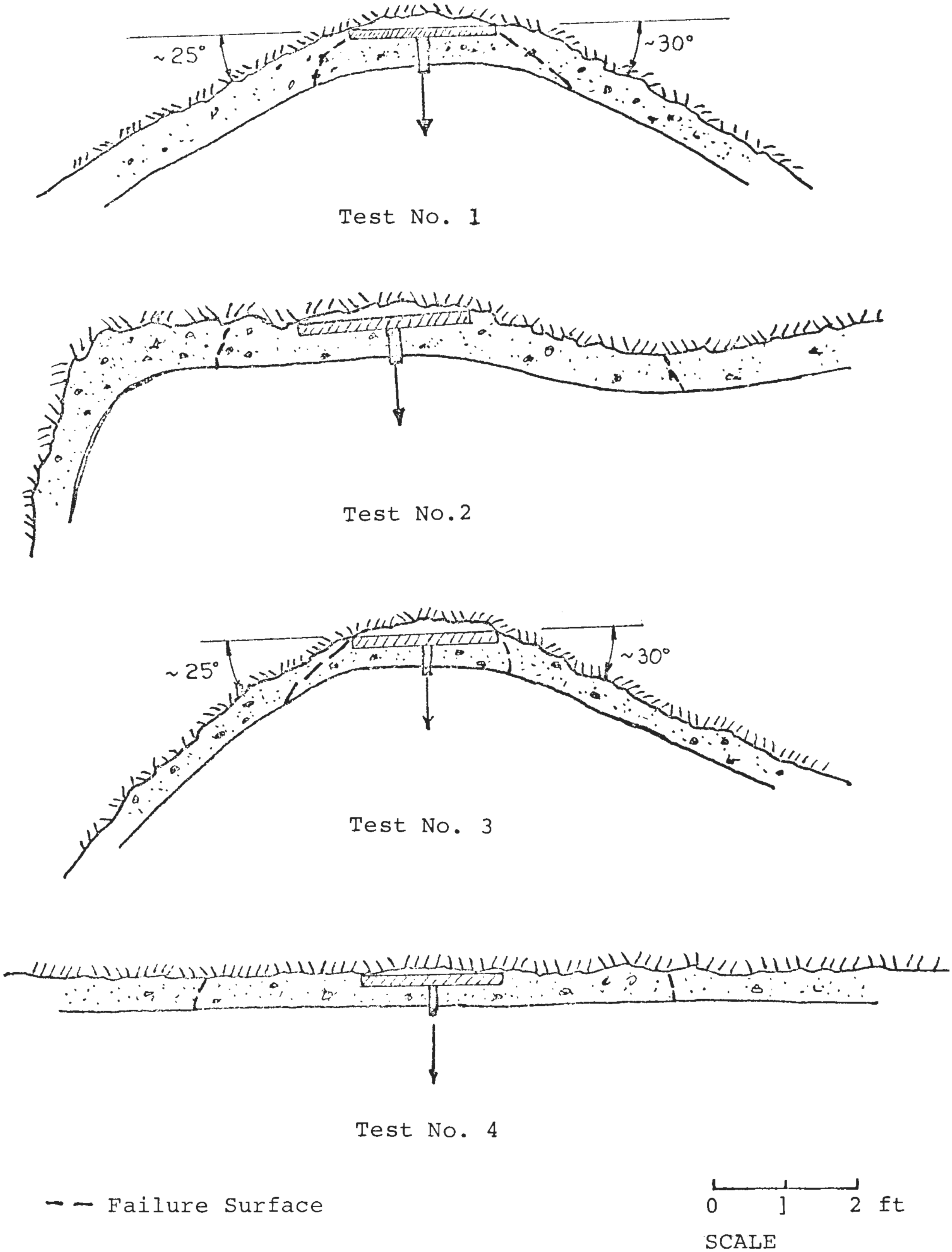



FIGURE VIII-11 : Test Configurations and Failure Surfaces. MARTA, Atlanta.

for each shotcrete mix tested (conventional and fiber shotcrete) a flat and an arch configuration were obtained. The results are summarized in Table VIII-10.

Shotcrete layers in flat configuration failed by adhesion and developed higher adhesive strengths between the shotcrete and the rock, $a_o$, than those observed in the laboratory ($a_o$ varied from 0.07 $f_c$ for Test No. 4, to 0.15 $f_c$ for Test No. 2, which exhibited slight curvature; values of $a_o$ = 0.05 $f_c$ had been obtained in the planar laboratory tests). Both the 4-inch and 8-inch thick shotcrete layers with arched configurations failed in shear; shear strength values, $f_d$, equal to 0.1 $f_c$, consistent with the laboratory test results, were exhibited by the arch-shaped surfaces. However, the results indicate that the adhesive strength required for shear failure to develop was considerably higher than that measured in the laboratory. Test No. 1 on the 8-inch layer indicates that the adhesive strength for the good quality, rough-surfaced gneiss in the arch configuration may be as great as 0.4 $f_c$ (540 psi), as compared with 0.1 $f_c$ for the laboratory tests on concrete surfaces.

The results from all tests suggest that natural irregularities in a dry and clean rock surface can increase the adhesive strength several times beyond the values measured in the laboratory where shotcrete was applied over a concrete surface. The increase is larger for layers in arch configuration because compressive stresses tend to develop at the irregularities; thus, failure occurs not strictly in adhesion at the shotcrete rock interface, but in shear through and around irregularities in the rock and through the irregularities in the shotcrete. Figure VIII-12 summarizes the capacities of these layers and compares these test results with the laboratory results.

The addition of fiber reinforcement increased the ductility of the layers, but it did not increase their capacity. Visual observations during the tests demonstrated that the flat shot-

TABLE VIII-10

Summary of Shotcrete Capacity Tests. Atlanta Research Chamber.

| Test No. | Configuration | Shotcrete Age | Reinforcement | Failure Mode | Capacity | $f'_c$ psi | Thickness in. | Capacity lb | Shear Strength $f_d$ | Adhesive Strength $a_o$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Arch | 10 hr. | None | Shear | $P = f_d \cdot H \cdot 2L$ | 1400 | 8 | 52,000 | $0.10\ f'_c$ | - |
| 2 | Flat | 24 hr. | None | Adhesion | $P = 2\ a_o \cdot 2L$ | 3500 | 8 | 50,000 | - | $0.15\ f'_c$ |
| 3 | Arch | 7 hr. | Fiber (3%) | Shear | $P = f_d \cdot h \cdot 2L$ | 400 | 4.5 | 7,000 | $0.08\ f'_c$ | - |
| 4 | Flat | 11 hr. | Fiber (3%) | Adhesion | $P = 2\ a_o \cdot 2L$ | 900 | 4.5 | 6,000 | - | $0.07\ f'_c$ |

$f'_c$ = compressive strength of the shotcrete, measured in prismatic, 3 in. x 3 in. x 6 in. samples.

$f_d$ = shear strength developed along the shotcrete layer.

$a_o$ = adhesive strength developed between the shotcrete layer and the rock.

L = width of the shotcrete layer (24 in.)

H = thickness of the shotcrete layer.

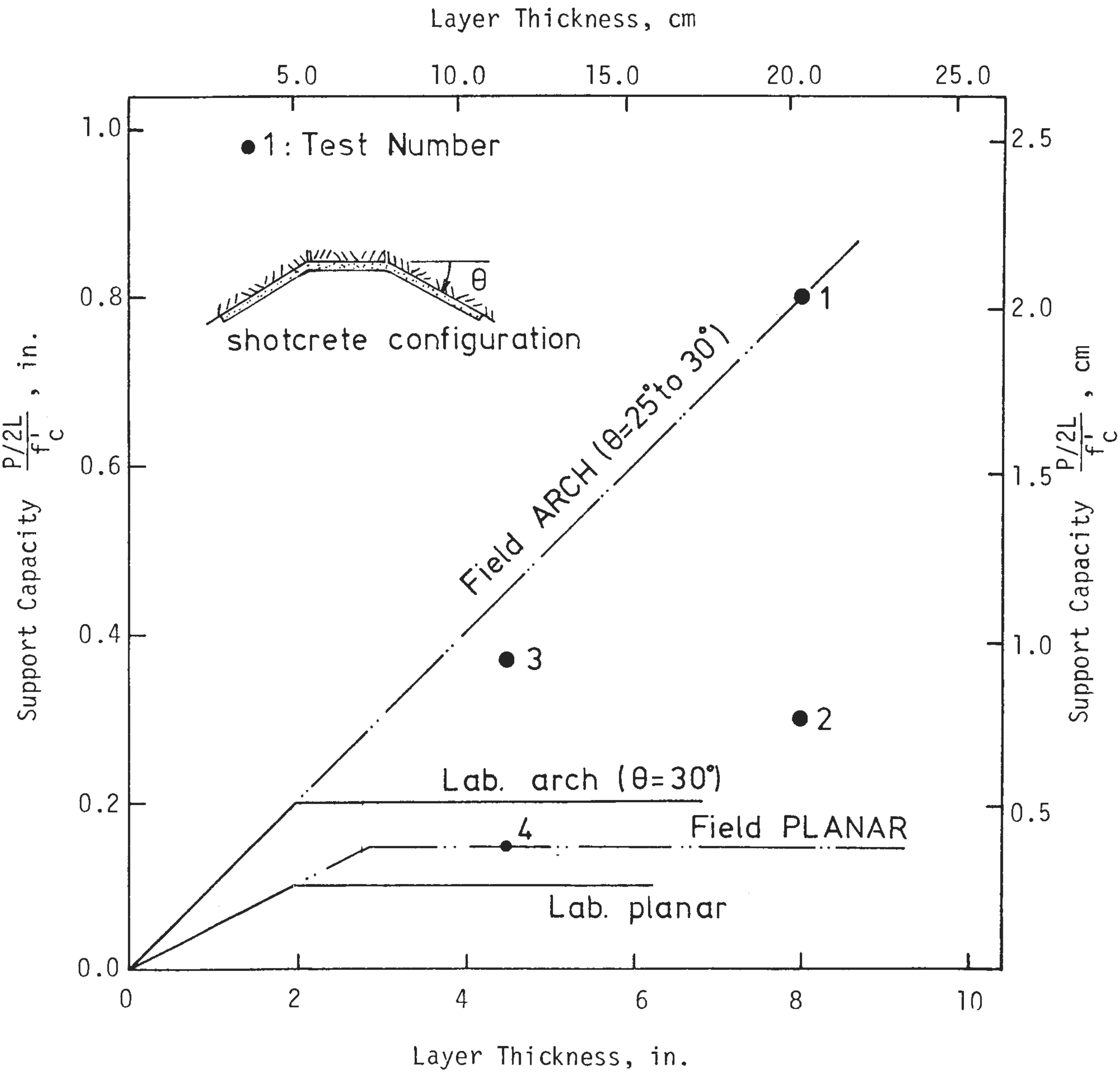
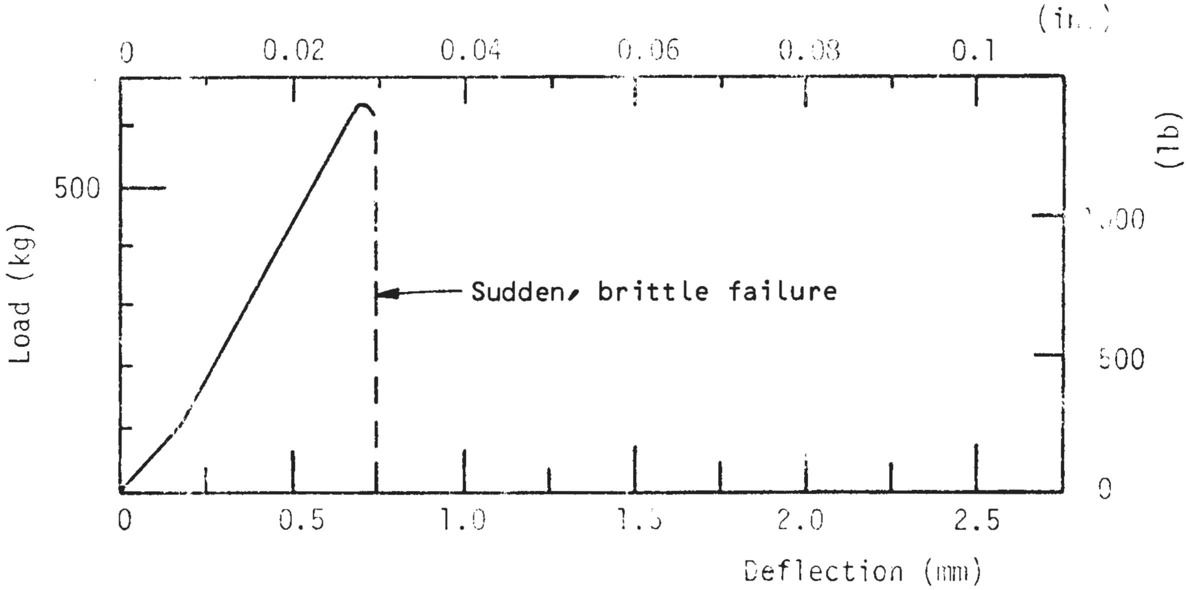
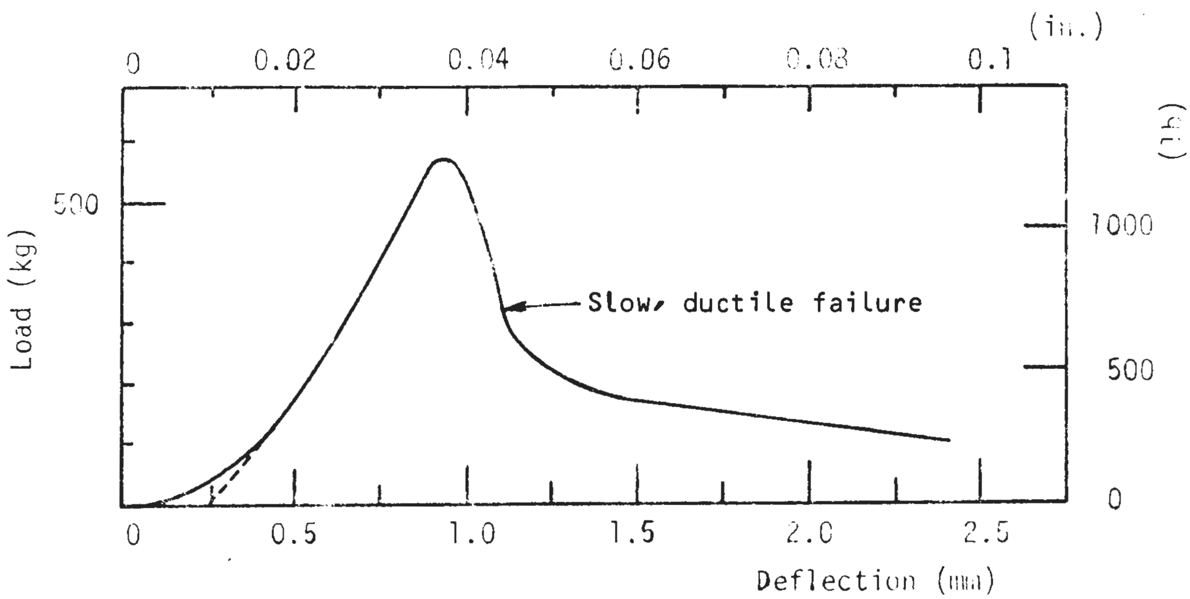



**Figure VIII-12:** Maximum Carrying Capacity of High Adhesion Shotcrete Layers with Free End. Field Results from Atlanta.




a. Conventional shotcrete




b. Steel-fiber-reinforced shotcrete

Figure VIII-13: Typical Load-Deflection Data of Conventional and Steel-Fiber-Reinforced Shotcrete tested in Flexure.

From University of Illinois Data

crete layer with fiber reinforcement developed a series of visible cracks and moved 1 inch to 2 inches before failure, whereas the flat shotcrete layer without fiber reinforcement was brittle and failed with little warning (see Figure VIII-13).

## 8.8 REBOUND TEST

### 8.8.1 Test Procedure

The rebound test consisted of shooting fiber shotcrete from mix CS-8-F against the rock surface for a period of 10 minutes. A 6 foot wide and 10 foot high strip of rock in panel number 3 on the east side of the Atlanta Research Chamber was covered with an average thickness of 3 inches of steel-fiber-reinforced shotcrete.

Before shooting began, a clean tarpaulin was assembled and placed on the ground in front of the test panel. Care was taken that the tarp was large enough to make possible the recovery of essentially all the rebound. At all times during shooting the nozzle was kept perpendicular to the rock wall at a distance of between 3 feet to 5 feet from it.

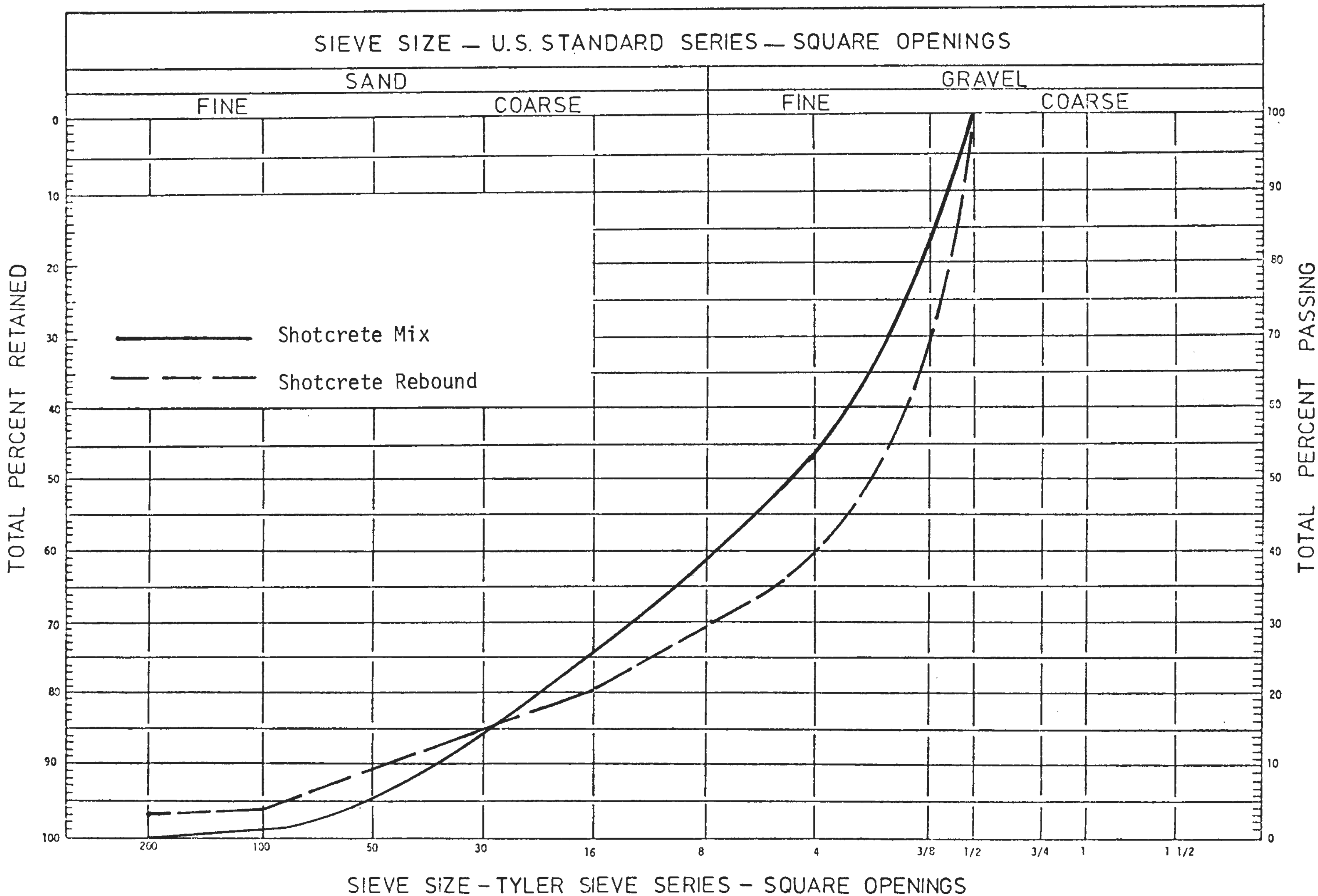
Immediately following the completion of shooting, the rebound on each of the small, square tarps was weighed and samples of the rebound material were collected in bottles.

### 8.8.2 Results of Rebound Test

During the rebound test, a total 2530 pounds of dry mix with fiber were shot in 10 minutes. The rebound obtained on the tarpulins weighed 553 pounds. Thus the average rebound was (553/2530) x 100 = 22% and the material delivery rate was 253 lb/min.

Most of the steel fibers in the rebound material were as straight after shooting as before. A measurement of the weight of fibers in the mix before shooting, and in the rebound

Figure VIII-14




GRADATION CURVES OF SHOTCRETE MIX AND REBOUND. REBOUND TEST, ATLANTA.

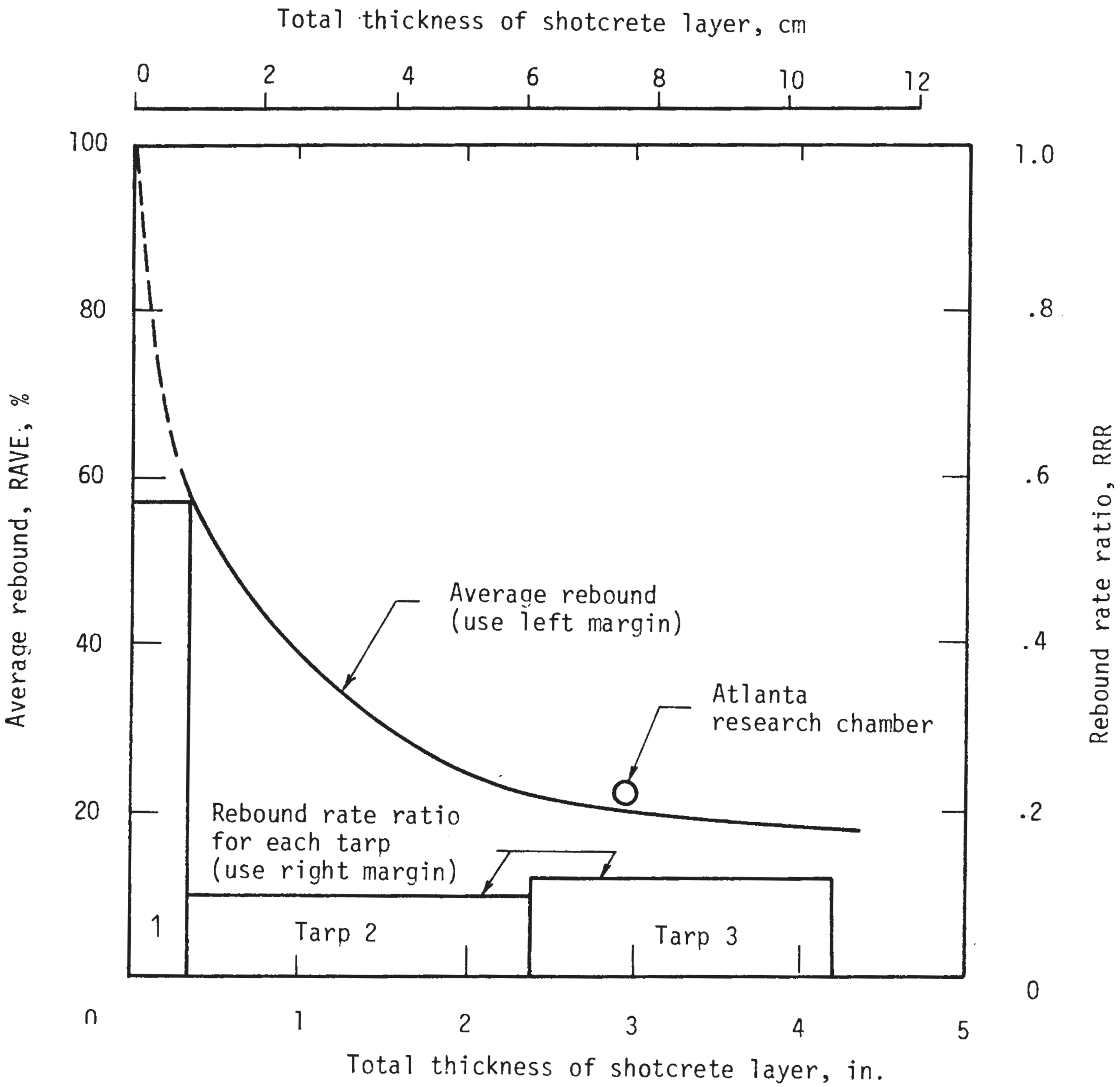



**Figure VIII-15:** Change in measured rebound values with thickness (Parker, et al., 1975) material, indicated that the fiber content was 3.3% (by weight of dry mix) before shooting and 4.6% in the rebound material.

The gradation curves of the shotcrete mix and the rebound material are shown in Figure VIII-14. A comparison of these curves indicates that the rebound has more gravel than the mix (70% as compared to 60% for the mix).

A sample of the dry mix and a sample of the rebound were obtained to determine their water and cement contents. Although no reliable quantitative data could be obtained from the small amount sampled, the results show that the cement content of the rebound was considerably less as compared to the dry mix, and the water content in the rebound material was substantially larger than that of the mix.

Results from a comprehensive series of field rebound tests on 27 different mixes shot under very different conditions (Parker, et al., 1975) have shown that the major variable controlling the average rebound is the total thickness of the shotcrete layer placed in that lift. The relationship between average rebound, rebound rate, and thickness, based on data from those tests, is illustrated in Figure VIII-15. The rate of rebound, shown by the shaded bar graph, drops as soon as an initial critical thickness is established (Phase 1) and then (in Phase 2), is more or less constant with thickness. However, average rebound (the total weight rebounded divided by total weight shot) reduces slowly and at a rate that depends on the magnitude of the initial losses. For the test conditions, it was not until a thickness of about 4 inches (10 cm) had been shot that the change in average rebound curve was not dominated by the high losses during Phase 1. The result from the rebound test carried out in this program (circular point in Figure VIII-15) agrees with the deviation observed in the field test carried out by Parker, et al.

## 8.9 PULL OUT TESTS

These tests were designed to measure the adhesion strength between the shotcrete and the rock surface.

In this test a 3-inch diameter cylinder was isolated from the remaining shotcrete by drilling through it and into the rock with a 3-inch diameter coring bit. A hollow hydraulic jack which reacted on a steel frame attached to the rock was used to pull the shotcrete cylinder from a 3/8-inch stud which had previously been grouted in a hole at the center of the cylinder.

### 8.9.1 Test Procedure

Before shooting, a light frame which supported six 3/4 inch diameter and 1 inch long bolts was attached to the rock surface. The bolts were covered with tape to prevent shotcrete from adhering to them.

After shooting to the desired thickness, a 1/2-inch diameter hole was drilled at the center of the place where a shotcrete cylinder was going to be pulled out. The hole was drilled to a depth about 1 inch away from the rock surface using a hand held concrete driller. A 3/8-inch diameter threaded steel stud was placed in the hole and grouted with Hydrocal.

In order to drill the shotcrete cylinder, the drilling equipment was fastened to the 3/4-inch diameter bolts. The alignment of the drilling bit was kept with a 3-inch diameter centralizing steel cylinder which could rotate freely on the center stud.

When drilling was completed, a steel frame was fastened to the 3/4-inch diameter bolts as shown in Figure VIII-16, and the hollow hydraulic jack was secured to this frame. Through the center hole of the jack, an extension stud and bolt were attached to the central stud, and a load cell was placed at the outer end

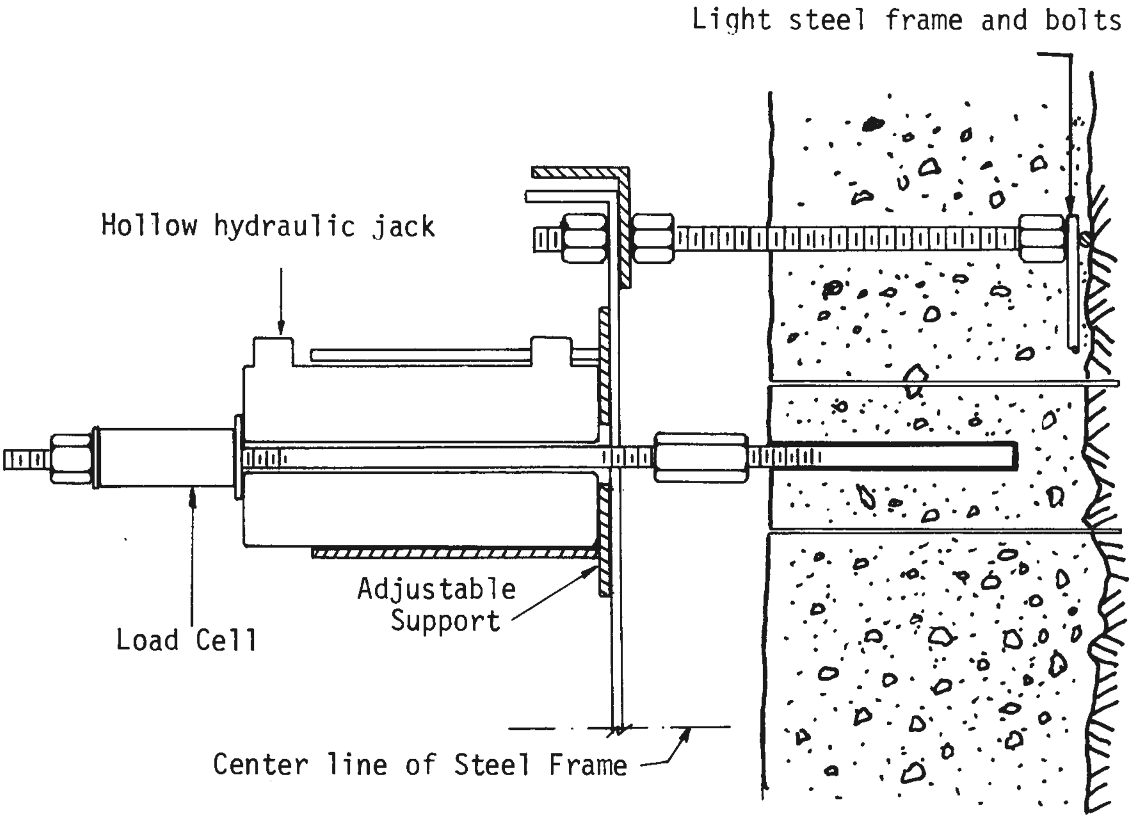



Figure VIII - 16: Set up for pull out tests of this assembly. Figure VIII-17 shows the test set-up. The load was increased slowly up to failure. Figure VIII-18 shows a cylinder of shotcrete after failure in a pull-out test in the laboratory.

8.9.2 Test Results

Only a few pull-out tests at 28 days were successfully carried out in the field. No reliable test data was collected at earlier ages (i.e., 7 hours, 3 days and 7 days) because the shotcrete cylinder tended to break off the rock wall due to vibrations of the drilling equipment during the coring procedure.

The results of these few tests are shown in Table VIII-11. As indicated by this Table, the adhesive strength of the steel-fiber-reinforced shotcrete was 0.02 $f_c$ and that of the conventional (non-fibrous) shotcrete ranged from 0.03 $f_c$ to 0.05 $f_c$.

The same range of adhesive strengths was measured in a similar testing program carried out in the laboratory where the shotcrete was shot against a concrete wall.

8.9.3 Conclusion

These test results tend to indicate that fiber shotcrete has a slightly lower adhesive strength than conventional shotcrete.

The field tests have been instrumental in demonstrating the usefulness of the adhesion pull test now being developed by the University of Illinois. Careful field observations of rock surface properties coupled with adhesive pull tests and structural tests of the shotcrete should provide basic data to improve our understanding of shotcrete behavior under various ground conditions.

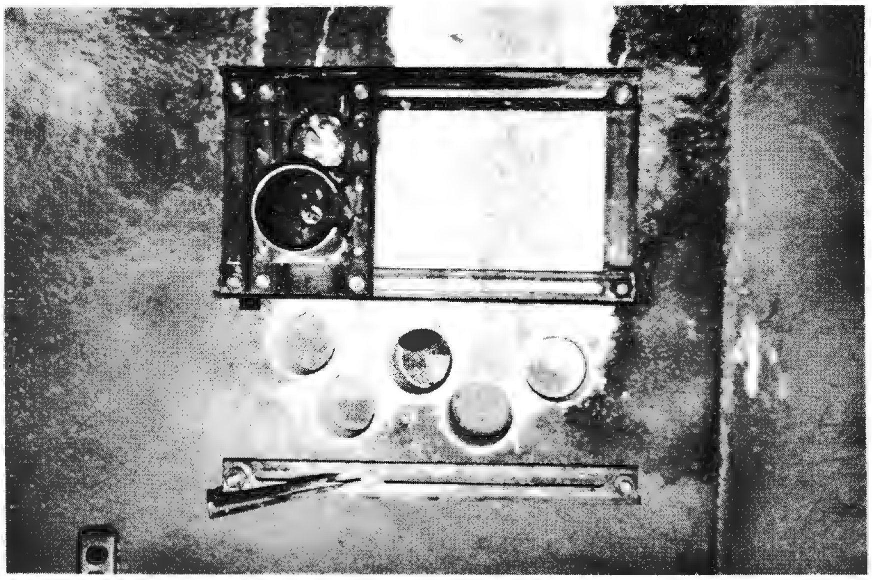


Figure VIII-17


Figure VIII-18: Pull-Out Cylinder of Shotcrete.

TABLE VIII-11

PULL OUT TEST AT 28 DAYS

| Mix | Test No. | $a_o$ (psi) Adhesion | $f'_c$ (psi) | $a_o/f'_c$ |
|---|---|---|---|---|
| D-7 | 1 | 220 | 7400 | .03 |
| | 2 | 375 | 7400 | .05 |
| S-8-F | 1 | 129 | 5900 | .02 |
| | 2 | 130 | 5900 | .02 |

HANSEN FIBER-METER

MODEL 200
(Patent Applied For)

SPECIFICATIONS


| | | |
|---|---|---|
| Height | | 41 in. |
| Width | | 34.5 in. |
| Length | | 62 in. |
| Hopper: | | |
| Height above base | | 38.5 in. |
| Width | | 13 in. |
| Length | | 26 in. |
| Fiber Drum: | | |
| Diameter | | 24 in. |
| Length | | 23-31 in. |
| Capacity | | 6-8 cu. ft. |
| Fiber Discharge Height | | 1 in. |
| Power | 0.5 hp | 115v. |
| Weight | | 371 lb. |

This machine meters steel and stainless steel fibers for concrete, gunite and refractory mixes. Fibers are dumped into the hopper and they are discharged completely separated at a uniform, predetermined rate (about plus or minus 1%).

The r.p.m. of the drum is constant. The rate is adjusted by turning only 4 nuts to expose more or less of the grate through which the fibers fall. Metering rate may vary from 0 to about 90 pounds per minute depending upon the kind of fibers.

Fibers can be metered into conveying augers carrying a uniform volume of material. Only about 3 feet of auger length is necessary to mix the fibers into either a dry mix or a concrete mix. Premixing fibers with sand may be done for gunite. When using a drum mixer, the fibers can be spouted into the revolving drum. A timer can be used to deliver the correct amount of fibers.

When it is not possible or desirable to mount the meter so that fibers flow into the mix a blower can be used for conveying the fibers from the meter.

Because fibers will not flow out of a bin it is suggested that a belt conveyor be used to load the meter if once-a-day loading is desired. The length and speed should be selected for the job. Packages of fibers can be dumped on the belt and it can move automatically or by manual control.

FIGURE VIII-19

TABLE VIII-A-1
CUBE COMPRESSIVE STRENGTH TESTS
ATLANTA RESEARCH CHAMBER
ATLANTA, GEORGIA

| ADMIXTURE | DOSAGE | COMPRESSIVE STRENGTH, IN PSI, AT INDICATED AGE | | | | |
|---|---|---|---|---|---|---|
| | | 8 hours | 24 hours | 3 days | 7 days | 28 days |
| Dryshot | 1% | 20 | 90 | -- | -- | 5750 |
| | | 30 | -- | -- | -- | 6120 |
| | | 30 | -- | -- | -- | 6200 |
| | | (30) | (90) | -- | -- | (6020) |
| Dryshot | 3% | 160 | 220 | -- | 4710 | 6550 |
| | | 160 | 220 | -- | 4650 | 6790 |
| | | 160 | 220 | -- | 4560 | 6780 |
| | | (160) | (220) | -- | (4640) | (6710) |
| Dryshot | 6% | 630 | 2280 | -- | 5900 | 7590 |
| | | 620 | 2290 | -- | 6050 | 7640 |
| | | 590 | 2320 | -- | 6050 | 7640 |
| | | (610) | (2300) | -- | (5940) | (7540) |
| None-Control Mix | | -- | -- | 3160 | 4190 | 7390 |
| | | -- | -- | 3160 | 4360 | 7200 |
| | | -- | -- | 3050 | 4100 | 7260 |
| | | -- | -- | (3120) | (4220) | (7280) |

NOTES: All tests run with water-cement ratio of 0.45. The resulting mixture was "dry" and handling difficulty probably caused the scatter in strengths. Medusa cement, Type I, coarse aggregate No. 67 and Waugh sand were used for all mixes.

TABLE VIII-A-1 (CONTINUED)
CUBE COMPRESSIVE STRENGTH TESTS
ATLANTA RESEARCH CHAMBER
ATLANTA, GEORGIA

| ADMIXTURE | DOSAGE | COMPRESSIVE STRENGTH, IN PSI, AT INDICATED AGE | | | | |
|---|---|---|---|---|---|---|
| | | 8 hours | 24 hours | 3 days | 7 days | 28 days |
| Sigunit | 1% | 370 | 2110 | -- | 4080 | 5490 |
| | | 350 | 2100 | -- | 4170 | 5410 |
| | | 350 | 2130 | -- | 4100 | 5550 |
| | | (360) | (2110) | -- | (4120) | (5480) |
| Sigunit | 3% | 360 | 2090 | -- | 2870 | 4150 |
| | | 350 | 2110 | -- | 2830 | 4150 |
| | | 370 | 2080 | -- | 2970 | 4000 |
| | | (360) | (2090) | -- | (2890) | (4100) |
| Sigunit | 6% | 1080 | 2250 | -- | 3860 | 4530 |
| | | 1110 | 2300 | -- | 4000 | 4390 |
| | | 1080 | 2350 | -- | 3750 | 4610 |
| | | (1090) | (2300) | -- | (3870) | (4510) |
| None-Control Mix | -- | -- | 3160 | 4190 | 7390 | |
| | | -- | -- | 3160 | 4360 | 7200 |
| | | -- | -- | 3050 | 4100 | 7260 |
| | | -- | -- | (3120) | (4220) | (7280) |

NOTES: All tests run with water-cement ratio of 0.45. The resulting mixture was "dry" and handling difficulty probably caused the scatter in strengths. Medusa cement, Type I, coarse aggregate No. 67 and Waugh sand were used for all mixes.

APPENDIX VIII-B

## SETTING TIME OF CEMENT AND ACCELERATOR

The compatibility between the cement and the accelerator is often tested by evaluating the initial and final set times for different percentages of accelerator by means of the Gillmore Needle Test. This test is basically a surface-penetrating resistance test that is used to determine arbitrary initial and final set times for a cement mortar. The ASTM test procedure (C - 266) is usually modified by reducing mixing times to a few seconds to account for the fast setting times. Generally, the compatibility between cement and accelerator may be considered acceptable if the initial set is less than 3 minutes and the final set is less than 12 minutes (Blanck, 1974).

TABLE VIII-B-1
TIME OF SET DETERMINATIONS
ATLANTA RESEARCH CHAMBER
ATLANTA, GEORGIA

| CEMENT | TYPE | ADMIXTURE | DOSAGE | WATER:CEMENT RATIO | INITIAL SET | FINAL SET |
|---|---|---|---|---|---|---|
| Signal Mountain | I | Sigunit(8010) | 3% | 0.40 | 1m 45s | 2h 30m |
| Signal Mountain | I | Sigunit(8010) | 3% | 0.35 | 1m 25s | 36m |
| Signal Mountain | I | Sigunit(8010) | 3% | 0.35 | 1m 30s | 9m |
| Signal Mountain | I | Sigunit(8010) | 3% | 0.35 | 1m 20s | >12m |
| Signal Mountain | I | Sigunit(8010) | 3% | 0.30 | 15s | -- |
| Signal Mountain | I | Sigunit(8010) | 6% | 0.40 | 1m 30s | 1h 55m |
| Signal Mountain | I | Sigunit(8010) | 6% | 0.40 | 2m 30s | 2h |
| Signal Mountain | I | Sigunit(8010) | 6% | 0.35 | 1m 40s | 6m |
| Signal Mountain | I | Sigunit(8010) | 6% | 0.35 | 2m | 8m |
| Signal Mountain | I | Dryshot(7005) | 3% | 0.40 | 1m 20s | >10m |
| Signal Mountain | I | Dryshot(7005) | 3% | 0.35 | 50s | >10m |
| Signal Mountain | I | Dryshot(7005) | 3% | 0.30 | 30s | 8m |
| Signal Mountain | I | Dryshot(7005) | 6% | 0.40 | 1m 30s | >15m |
| Signal Mountain | I | Dryshot(7005) | 6% | 0.35 | 1m 5s | 9m |
| Signal Mountain | I | Dryshot(7005) | 6% | 0.33 | 1m 5s | 9m |
| Signal Mountain | I | Dryshot(7005) | 6% | 0.30 | 35s | 6m |
| Medusa | I | Dryshot(7005) | 3% | 0.35 | 1m 45s | >8m |
| Medusa | I | Dryshot(7005) | 3% | 0.30 | 1m 30s | >8m |
| Medusa | I | Sigunit(8010) | 6% | 0.40 | 2m | >8m |
| Medusa | I | Sigunit(8010) | 6% | 0.35 | 1m 30s | >8m |
| Medusa | I | Sigunit(8010) | 3% | 0.35 | 2m | >10m |
| Medusa | I | Sigunit(8010) | 3% | 0.30 | 1m 20s | 5m 30s |

TABLE VIII-B-1 (CONTINUED)
TIME OF SET DETERMINATIONS
ATLANTA RESEARCH CHAMBER
ATLANTA, GEORGIA

| CEMENT | TYPE | ADMIXTURE | DOSAGE | WATER:CEMENT RATIO | INITIAL SET | FINAL SET |
|---|---|---|---|---|---|---|
| Medusa | I | Sigunit | 1% | 0.48 | 37m | 120m |
| Medusa | I | Sigunit | 1% | 0.48 | >20m | -- |
| Medusa | I | Sigunit | 1% | 0.43 | 13m | >60m |
| Medusa | I | Sigunit | 1% | 0.40 | 13m | >60m |
| Medusa | I | Sigunit | 3% | 0.40 | 4m30s | 60m |
| Medusa | I | Sigunit | 6% | 0.40 | 6m | 22m |
| Medusa | I | Sigunit | 6% | 0.40 | 9m45s | >75m |
| Medusa | I | Dryshot | 1% | 0.40 | 11m45s | 27m |
| Medusa | I | Dryshot | 3% | 0.40 | 7m | 14m30s |
| Medusa | I | Dryshot | 3% | 0.40 | 4m30s | 14m30s |
| Medusa | I | Dryshot | 6% | 0.40 | 4m45s | 9m15s |
| Medusa | I | Dryshot | 6% | 0.40 | 4m | 9m30s |
| Signal Mtn. | I | Sigunit | 3% | 0.40 | 2m30s | >60m |
| Signal Mtn. | I | Sigunit | 6% | 0.40 | 5m15s | >60m |
| Gifford Hill | III | Sigunit | 1% | 0.4 | 5m15s | >30m |
| Gifford Hill | III | Sigunit | 3% | 0.43 | 3m | >60m |
| Gifford Hill | III | Sigunit | 3% | 0.40 | 90s | 45m |
| Gifford Hill | III | Sigunit | 6% | 0.43 | 75s | 15m |

NOTE: THESE DETERMINATIONS WERE MADE FOR INITIAL SAMPLES OF ADMIXTURES FROM THE SIKA CHEMICAL CORPORATION AND CEMENT SAMPLES OBTAINED IN JANUARY AND FEBRUARY, 1978

## TABLE VIII-C-1

### COMPRESSIVE STRENGTH OF FIBER REINFORCED SHOTCRETE AT 8 TO 10 HOURS

| Sample Number | Age Hours | Unit Weight (p.c.f.) | L/D | Failure Stress (psi) |
|---|---|---|---|---|
| S-2-F-H-1 | 8.4 | 139.8 | .73 | 2170 |
| S-2-F-H-2 | 8.5 | 143.1 | .82 | 2010 |
| S-2-F-V-1 | 8.5 | 148.8 | .91 | 1980 |
| S-2-F-V-2 | 8.5 | 150.7 | 1.10 | 2080 |
| S-2-F-V-3 | 8.5 | 144.7 | 1.14 | 1820 |
| S-2-F-V-4 | 8.6 | 148.1 | 1.32 | 1790 |
| S-2-F-V-5 | 8.6 | 143.0 | 1.14 | 1400 |
| S-2-F-V-6 | 8.7 | 145.5 | 1.14 | 1490 |
| S-4-F-V-1 | 10.1 | 135.7 | 1.03 | 1140 |
| S-4-F-V-2 | 10.0 | 140.3 | .97 | 1600 |
| S-4-F-V-3 | 10.0 | 136.9 | 1.03 | 1750 |
| S-4-F-V-4 | 9.6 | 146.7 | 1.00 | 2580 |
| S-4-F-V-5 | 9.7 | 143.7 | 1.03 | 2610 |
| S-4-F-V-6 | 9.7 | 145.6 | 1.03 | 2480 |
| S-8-F-V-1 | 12 | - | 1.06 | 1061 |
| S-8-F-V-2 | 12 | - | .99 | 1102 |
| S-8-F-V-3 | 12 | - | 1.03 | 1283 |

## TABLE VIII-C-2

## COMPRESSIVE STRENGTH OF FIBER REINFORCED SHOTCRETE AT 3 DAYS

| Sample Number | Unit Weight (p.c.f.) | L/D | Failure Stress (psi) |
|---|---|---|---|
| S-2-F-H-1 | - | 0.73 | 3600 |
| S-2-F-H-2 | - | 0.79 | 3510 |
| S-2-F-H-3 | - | 0.62 | 3900 |
| S-2-F-V-1 | - | 1.15 | 3010 |
| S-2-F-V-2 | - | 1.28 | 2570 |
| S-2-F-V-3 | - | 1.25 | 2950 |
| S-2-F-V-4 | - | 1.17 | 2550 |
| S-2-F-V-S | - | 1.20 | 2920 |
| S-2-F-V-6 | - | 1.04 | 3330 |
| S-4-F-H-1 | 143.8 | 1.04 | 3110 |
| S-4-F-H-2 | 145.3 | 1.04 | 3325 |
| S-4-F-H-3 | 134.6 | 1.04 | 3040 |
| S-4-F-V-1 | 146.6 | 1.03 | 6155 |
| S-4-F-V-2 | 144.0 | 0.98 | 6225 |
| S-4-F-V-3 | 146.6 | 1.03 | 6155 |
| S-4-F-V-4 | 147.2 | 1.07 | 5730 |
| S-4-F-V-5 | 146.5 | 1.00 | 6225 |
| S-4-F-V-6 | 148.5 | 1.00 | 6225 |
| D-5-F-V-1 | 144.4 | 1.03 | 3680 |
| D-5-F-V-2 | 146.2 | 0.97 | 4385 |
| S-5-F-V-3 | 146.9 | 1.00 | 4175 |
| D-5-F-V-4 | 148.6 | 1.00 | 6505 |
| D-5-F-V-5 | 148.1 | 1.02 | 6580 |
| D-5-F-V-6 | 147.5 | 1.00 | 6650 |
| S-8-F-V-1 | - | 0.99 | 4668 |
| S-8-F-V-2 | - | 0.99 | 4668 |
| S-8-F-V-3 | - | 0.99 | 4456 |

## TABLE VIII-C-3

### COMPRESSIVE STRENGTH OF FIBER REINFORCED SHOTCRETE AT 28 DAYS

| Sample Number | Unit Weight (p.c.f.) | L/D | Failure Stress (psi) |
|---|---|---|---|
| S-2-F-H-1 | 144.9 | 1.33 | 3170 |
| S-2-F-H-2 | 142.2 | 1.21 | 3540 |
| S-2-F-H-3 | 144.7 | 1.19 | 4100 |
| S-2-F-V-1 | 148.7 | 1.09 | 3700 |
| S-2-F-V-2 | 151.9 | 1.24 | 3640 |
| S-2-F-V-3 | 142.8 | 0.97 | 3960 |
| S-2-F-V-4 | 150.3 | 1.07 | 4590 |
| S-2-F-V-5 | 148.9 | 1.09 | 3840 |
| S-2-F-V-6 | 155.8 | 0.90 | 3750 |
| S-4-F-H-1 | 142.9 | 1.03 | 4385 |
| S-4-F-H-2 | 145.1 | 1.02 | 3680 |
| S-4-F-H-3 | 143.9 | 1.00 | 4670 |
| S-4-F-V-1 | 149.7 | 1.01 | 6220 |
| S-4-F-V-2 | 148.9 | 1.05 | 7215 |
| S-4-F-V-3 | 145.0 | 1.02 | 7000 |
| S-4-F-V-4 | 150.1 | 1.01 | 7500 |
| S-4-F-V-5 | 149.9 | 1.00 | 8345 |
| S-4-F-V-6 | 148.7 | 1.01 | 6365 |
| D-5-F-V-1 | 146.1 | 1.02 | 7780 |
| D-5-F-V-2 | 142.9 | 1.05 | 6930 |
| D-S-F-V-3 | 143.1 | 1.03 | 6505 |
| D-S-F-V-4 | 149.4 | 1.01 | 10115 |
| D-5-F-V-5 | 148.2 | 1.00 | 10610 |
| D-5-F-V-6 | 148.6 | 1.01 | 10045 |
| S-8-F-V-1 | 143.8 | 0.99 | 6249 |
| S-8-F-V-2 | 143.1 | 1.00 | 6258 |
| S-8-F-V-3 | 143.7 | 1.00 | 6928 |

MONOGRAPHS

NBI-H-139-C1

# MONOGRAPHS - SUMMARY AND OVERVIEW

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## Summary

During the planning by the Team Members for the technical studies to be performed in the Atlanta Research Chamber, it became quite apparent that the tunneling expertise available among the Team Members was unusual. It was agreed with UMTA that key Team Members would contribute short papers, or monographs, on matters of their choice on the general subject of tunneling, especially rapid transit tunneling in hard rock using modern support methods.

Subsequently, to balance these predominently technical monographs, additional experts were requested to contribute their ideas on insurance, legal matters, specifications, and so on. The resulting collection of monographs on many aspects of tunneling is found on the following pages.

Overview papers are first. The first three monographs (by Rose, Keusel and O'Rourke) are overview papers by engineers. Next in order of presentation are overview papers by owners representives (by Kaiser and Gallagher). A labor union monograph and equipment dealer monograph follows (by Weatherl and Phillpott). Four contractors (Burtleson, Jensen, McCusker and Carleton) then express their views, and a brief explanation of a new labor training program is presented (by Scott). Legal matters are covered next (by Max Greenberg), followed by insurance (by Novell).

A series of generally technical monographs follows the overview papers. Modern instrumentation for tunnels is discussed (by Weir-Jones), followed by a monograph on modern blasting (by Oriard). Three-dimensional finite element method (FEM) of computer analysis is the subject of the next two monographs (by Azzouz, Einstein and Schwartz; and by Gartung, Bauernfeind and Bianchini). The NATM and the state-of-the-art of conventional shotcrete design is then discussed (by Golser; and by Fernandez-Delgado, Cording, Mahar, and Van Sint Jan.) Steel-fiber-reinforced shotcrete as used in the MARTA Running Tunnels is the subject of a case history (by Buchanan) and a letter from the Contractor who did the work (Gene Root). The last monograph (by Oliveira and Morrison) is on tunnel photography.

## Overview

The engineers generally deplored the backward state of North American design practices; labor and contractors indicated a willingness to be innovative in a prudent fashion, and deplored unfair and restrictive actions by owners and engineers. The owners, who must live with the completed product, were not so attracted to innovation. Legal and insurance authors seemed in favor of new and modern contracting techniques designed to eliminate, or at least minimize, the confrontations between owners, engineers and contractors. The technical monographs indicated progress in the state-of-the-art of tunneling.

The technical monographs on NATM and the European methods of shotcreting (by Golser) and on conventional shotcrete design in the USA (by Fernandez-Delgado, et al., of the University of Illinois) brought to light an interesting difference in approach. The Fernandez-Delgado approach based in part on experience in the Washington D.C. Metro is concerned with large blocks of rock which could "punch through" a thin shotcrete lining. The European NATM approach, based in part on experience in the Alps, is designed for rock masses which may squeeze in, tending to close the tunnel, but which do not fail in large discrete blocks. It is obviously true that both methods are appropriate under certain circumstances.

In overview, it seems to me that monographs by engineers, labor, contractors, legal, and insurance experts all expressed optimism in improving North American tunnel practices. Owners remain to be convinced that new ideas are better ideas.

# AN ENGINEER'S OVERVIEW ON TUNNELING

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## Introduction

Harvey O. Banks, head of California's Department of Water Resources during the days of the huge multi-billion dollar California Water Plan which dammed Northern California rivers and conveyed the water 500 miles south to arid Southern California, used to say that "a project must be technically feasible, economically feasible, and politically feasible." Tunneling in the United States, especially for rapid transit systems, is technically and politically feasible, but is approaching economic infeasibility. There are a number of reasons for this.

Tunnel costs themselves are not yet always so high that rapid transit systems are not feasible to build. Large luxurious subway stations located along the tunnel lines are, in general, the cause of many cost overruns, because the stations represent about half of the construction cost of the subway portion of rapid transit systems. Smaller stations at longer intervals could save millions of dollars. Smaller tunnels, required for smaller subway cars, are used in Europe and could be used here. A cost reduction could be made where the rock is good by excavating several miles of tunnel, using moles (tunnel boring machines) in one large contract, followed by subway station enlargement on separate contracts later. 1/

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1/ "Urban Tunnels - An Option for the Transit Crisis" Matt S. Walton and Richard J. Proctor, ASCE Transportation Journal, Nov. 1976.

There is no doubt, however, that tunnels themselves are expensive. Yet tunnel costs in Europe for the same size tunnel in the same type of ground, normalized to account for wage differences (up or down), are about 50 percent lower in cost than here in the United States. The reason for this is that although the theories explaining behavior are woefully weak on both sides of the Atlantic, the Europeans are more realistic than we are, and consequently less reluctant to use techniques which work even though difficult to quantify theoretically.

## Tunnel Education

Civil and structural engineers in this country are commonly educated for four years or more in statics, dynamics, and strength of materials, advancing through structural theory from moment distribution to the use of the Finite Element Method (FEM) for a lucky few graduate students. More recently, Geological Engineering or Engineering Geology has been added to some university curricula. All of this education provides tools for logical manipulation of basic data in the form of numbers, vectors and what not. Gigantic bridges and skyscrapers are built successfully using the tools provided by this education to manipulate numbers and thus engineer safe designs. Of course, the basic numerical data input is highly reliable: careful preconstruction testing of steel, concrete and wood guarantees that the materials used are uniform in composition throughout the project, and the material expected behavior is well-documented by reproducible tests.

In tunnels, however, there are few or no numbers for basic data input that are reliable and reproducible along the tunnel. Mother Nature created the earth in a very messy and outright leaky laboratory, and wind and rain and flood finished off the deposition process, in many cases. The resulting soils and rocks subsequently have been squeezed, pulled and heaved about for millions of years. Although virtually all sound virgin rocks far exceed the 3,000 to 5,000 psi compressive strength of manmade concrete, real rocks are now so universally broken and fractured (i.e., "jointed") that many tunnels even in good rock require some kind of support to hold them open.

If the world were logical, geologists, engineering geologists and/or geological engineers might be in charge of tunnel designs. Because they often work outdoors and are intimately familiar with Mother Nature's erratic deposits, they should be the people who provide both numerical input data and suitable theoretical formulas for tunnel behavior. Alas, by some quirk of personality, few geologists have been interested in quantifying basic data, translating it into functional equations, and subsequently manipulating these equations. They describe the rock or soil (often using their very own peculiar jargon), but they usually leave the computations and designs to others who are often structural or at least civil engineers.

How well-qualified are these engineers in tunnel design? Ralph Peck reported the results of a survey of universities in the United States and Canada 2/ that offered mining engineering courses. Replies from university faculty members indicated that "only about twenty percent of the teachers have had even a modest exposure to the subject [of tunneling] in their own

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2/ "Preliminary Results of Tunneling Education Survey" Ralph Peck, Tunneling Technology Newsletter, No. 9, March 1975.

background." Further, Peck reports, "Several individual suggestions were also received. Among these, perhaps the most persistant was that there be development of suitable reading material, including possibly a textbook." Thus, even among the faculty members of mining engineering universities, tunnel education is weak. The state of theoretical knowledge among more ordinary working tunnel engineers in charge of designs in the real world, typically men say 30-50 years old with a bachelor's degree in engineering, is no doubt correspondingly lower. In any case, we all would agree that a good textbook in tunnel engineering is desirable.

The closest thing to a textbook readily available today to North American tunnel engineers, written lucidly by an acknowledged expert, is of course Rock Tunneling with Steel Supports [3/] published more than 30 years ago by a steel company, with opening chapters on theory by the emminent Karl Terzaghi. Terzaghi visited a large number of real tunnels and noted how unsupported rock had tended to fall out of the roof, until a stable arch shape was formed in the roof. Then, in the "textbook" he explained in easy-to-understand language how to estimate the dead weight of this rock load which might conceivably loosen over a period of time and fall on to the tunnel lining. The steel company then explained clearly how to install steel supports of the proper size to support this dead load. Because the steel sets must be ordered and bent to shape far ahead of time, they usually do not "fit" in the excavation snugly, and it is customary to use timber blocking to transmit the rock load to the steel ribs. Since both the timber and the steel may weaken with time, a concrete lining of empirical, generous thickness, is conventionally

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3/ R. V. Proctor and T. L. White, Rock Tunneling with Steel Supports, Youngstown, Ohio, Commercial Shearing and Stamping Company, 1968 (Revised Edition), first published in 1946.

poured as well in this typical North American approach. Measurements indicate, however, that in most cases the rock never did loosen and exert a dead load onto the steel ribs, and that the ribs typically took perhaps 20 percent of the design load. Actually, of course, the timber blocking took some of the rock load first, crushing a few wood fibers. The concrete lining takes almost no load at all, being poured long after all rock movement has ceased.

It seems clear that a new textbook, describing among other things the more economical (European) practices, written by an impeccably prominent authority, is badly needed to give the ordinary working tunnel engineer an authoritative reference to quote when he advocates improved methods. What are these new, innovative "improved methods" that modern tunnel engineers should be using to design cheaper tunnels in North America? It is easier to say what they should not be: they should not include a primary support system designed to carry a full dead weight ground load together with an entire duplicate "secondary" lining as though the primary lining didn't ever exist.

## Tunnel Theory

In any new tunnel excavated in any ground material, Mother Nature tries to close the new void, rapidly at first and then slowly. Eventually Mother Nature accepts the new tunnel as a fact: the inward movement stops. If Man foolishly attempts to prevent the initial part of this inward movement, he finds enormous inward pressures exist which can crush his puny tunnel lining systems. If, however, a modest initial inward movement is permitted to occur, pressures are then considerably reduced and a normal tunnel lining system can halt further movement. Any tunnel lining which is placed very late after the opening is made is probably just leaning against a perfectly stable unmoving ground and does nothing except perhaps provide long-term safety by keeping small loose pieces from dropping out of the roof.

These principles are true for any type of ground and any type of tunnel lining. Sufficiently exact numerical input data for exact computations on a real job are available for tunnel lining materials: shotcrete, concrete, steel ribs, liner plates, and so on. However, the sufficiently exact numerical input data to describe the ground Mother Nature has so capriciously created, is not normally available. Even if it is, one can be sure that all computations describing the ground will change a few meters further along the tunnel. Therefore it is usually found to be an impossible task to attempt a literal and exact computation of real ground pressures and displacements on a tunnel lining during a real job. It is, however, extremely worthwhile to make a number of parametric studies on paper which are theoretically correct, so as to give our tunnel designers a feel for reasonable ranges of behavior.

In the past ten years this has been done frequently by Ph.D. students writing thesis papers using the two dimensional and three dimensional Finite Element Method (2D & 3D FEM) computer techniques. The FEM is so vastly superior to previously available tools that many engineers go overboard and try to use it literally and exactly without realizing its limitations. Foremost among the limitations is the inexactitude of input data describing lumpy, impure and fractured ground. Another significant problem is that computing the FEM ground stress distribution depends upon the original in-situ stress, which is in fact difficult to determine in the field. Also, the usual FEM studies do not account for the bulking and volume changes that sometimes occur when tunnels penetrate real sands in soft ground tunneling. The figures shown at the end of this monograph are taken from various parametric 2-D and 3-D FEM studies. Some of these dramatically indicate how the ideal ground behaves. Other figures make it theoretically possible to compute the tunnel lining required to economically hold an ideal ground stable.

Broadly speaking, such parametric studies indicate that ALL tunnel linings (even concrete) are "flexible" relative to the magnitude of the initial ground loads, and hence ALL tunnel linings yield enough to permit small displacements and, hence, the occurance of a marked reduction in initial ground pressure. Hence, no real tunnel lining will ever take the heavy ground loads of a perfectly rigid lining, even if it were placed so promptly that it held back all of the very first ground displacements. The parametric studies confirm what the Europeans have proved in practice: even bad ground can be supported by remarkably thin layers of shotcrete or thin liner plates, promptly placed close to the face. Heavy steel supports are usually not required.

For some lining systems, such as shotcrete or precast concrete liner plates, the initial support lining is made of non-corrosive material which can stay in place and act as the final lining as well. The well-known New Austrian Tunnel Method (NATM) makes use of careful measurement of inward displacement as tunneling proceeds: the thickness of the shotcrete lining in place is adjusted to be only just sufficient to stabilize the observed inward displacements and no thicker. Thus the exact necessary lining thickness, though perhaps difficult to compute even using FEM, has been placed in the ground. There is no unnecessary, overly conservative thick lining used and paid for.

However, we must not misunderstand the situation regarding North American tunnel theory. It has its place. There is no doubt that Karl Terzaghi's arithmetic was correct. If, in fact, the ground over a tunnel were to actually loosen and fall in pieces as dead weight on the tunnel lining, as Terzaghi assumed, then the computations which indicate that thick linings are required to hold this load are indeed correct. Should completely loose, incoherent blocks of rock, to a depth more than three to four feet thick, lie as a massive dead load on a circular twenty-foot diameter tunnel with four-inch thick shotcrete lining, for instance, FEM idealized computations indicate the shotcrete will be stressed to its limit in tension and could begin to crack. Clearly, the difference in thinking between North American and European designers and builders is over the question whether or not tunnels be built so that the ground is supported quickly and excessive loosening is not permitted. If the only ground loads to be supported are those resulting from the first inward displacements, which tend to close the new void, then thin linings are quite sufficient, and the rock never loosens further. If, however, local blocks of rock are given an opportunity to weaken and move down along joints and finally exert a dead load on the lining, the ground is in no way helping to support itself. The entire dead weight must be held up by a lining. Clearly, since dead loads of loose ground or rock blocks more than about one meter thick require thicker linings and heavier supports, such a loose zone should not be permitted to develop. Thin, flexible linings should be installed promptly, before any ground actually loosens.

There are several excellent reports and highly theoretical Ph.D. thesis papers which have analyzed the behavior of the ground and computed the necessary tunnel lining strength to stabilize the opening. These are all laudable, and some are even readable; but the real demonstration of their principles is provided by the low-cost successful tunnels built in other countries using these principles. Details may vary and tunnel theory itself may be incomplete, but North American practice is clearly over-conservative and is still overly influenced by our single, out-of-date text book.

## Why Don't Engineers Use Modern Tunnel Designs in North America?

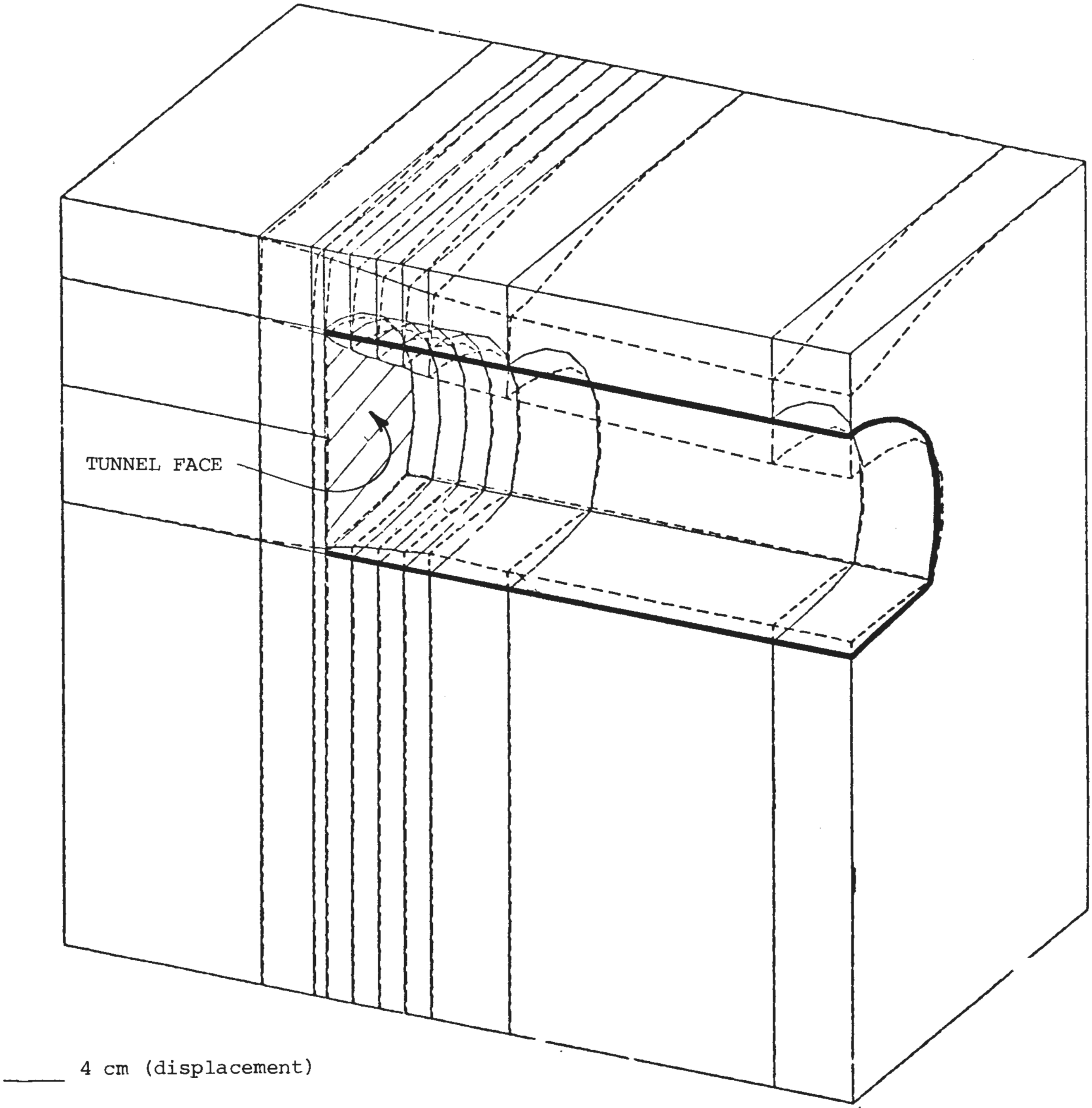
Why not indeed? We have discussed the significance of the lack of good tunnel education to this question. Another factor responsible for the typical engineer's conservatism is that he fears he might get sued and lose his shirt in court if a new method doesn't work well. To be downright frank about it, the owners who would benefit most are not providing sufficient motivation to the engineer to influence him to learn to use new techniques very rapidly. After all, to ask an engineer with a bachelor's degree (our average tunnel designer) to absorb a Ph.D. thesis based on a 3-D FEM study analyzing a case history in Austria, and then risk a lawsuit from an owner or the public if the new method is not executed perfectly the first time, is not realistic.

Opposed to this pessimistic and cynical view, of course, is the natural tendency of all engineers to pursue their life's career goal: to design and construct esthetically admirable, successful, economical and safe works for the use of the public. The engineer is an anonymous personality; almost never are the names of individual engineers signed and engraved on plaques on highways or bridges for the public to admire. The engineer's satisfaction is internal and comes from knowing that a job was well done using his utmost skill. A very large number of North American engineers and contractors are not happy at all to know that their European counterparts are using modern techniques we are not free to use here. North American engineers have plenty of brains and talent; but they need to work in an environment conducive to innovation.

## Closure

Upon reflection, it appears to me that cost cutting must originate with policies determined by owners. Owners must recognize that because they are first on the scene, they initially possess all of the risk for any costly construction unknowns and for overconservative and/or luxurious project concepts. Later, owners may attempt to pass some of the risk for costly construction unknowns to the contractor, who is no fool, and who will simply find a way sooner or later to return these costs back to the owner by change order, claims, or in the courts. In some cases, owners have recently taken to passing some of their risk to the engineer, and have taken engineers to court if things have not worked out well on innovative systems. None of this is healthy, and an owner should accept the fact that these risks are his. It would seem to me that, since 80 percent of rapid transit funding is typically from the federal government, the federal government has 80 percent of the money to gain in savings by promoting innovative engineering and non-luxurious projects. The local owner may gain 20 percent of the total savings in transit construction savings. The engineer and contractor would gain nothing except the personal satisfaction of being associated with a technically innovative and non-luxurious project - which is an intangible gain valued in different amounts by different people.

What risks are run with innovative tunnel design? European practice indicates that the costs of such risks are acceptable. Europeans in fact prefer their innovative designs to North American conservative designs. But any innovation entails some risk and has negative aspects, in the form of potential litigation or other loss, and neither American contractors nor engineers have any desire to altruistically accept these risks of potential loss. It would seem that the buck must be passed back to where it belongs - 20 cents of it to local owners and 80 cents of it to the federal government. If these two owners will simply insist that the most modern tunneling techniques be used and will provide occasional additional funds when necessary to implement the introduction of the new methods; and will, simultaneously, hold engineers and contractors harmless from unwarranted or capricious or third-party lawsuit; then, it is my view that the human desire to do a job well will be sufficient motivation to ensure innovative design and construction. I think that in such a case total costs will be promptly lowered to levels comparable to those accomplished by the Europeans. Should local owners and the federal government actually decide upon a policy to share some of the resulting savings with engineers and contractors, I expect North American costs would be rapidly reduced below costs found anywhere else in the world.



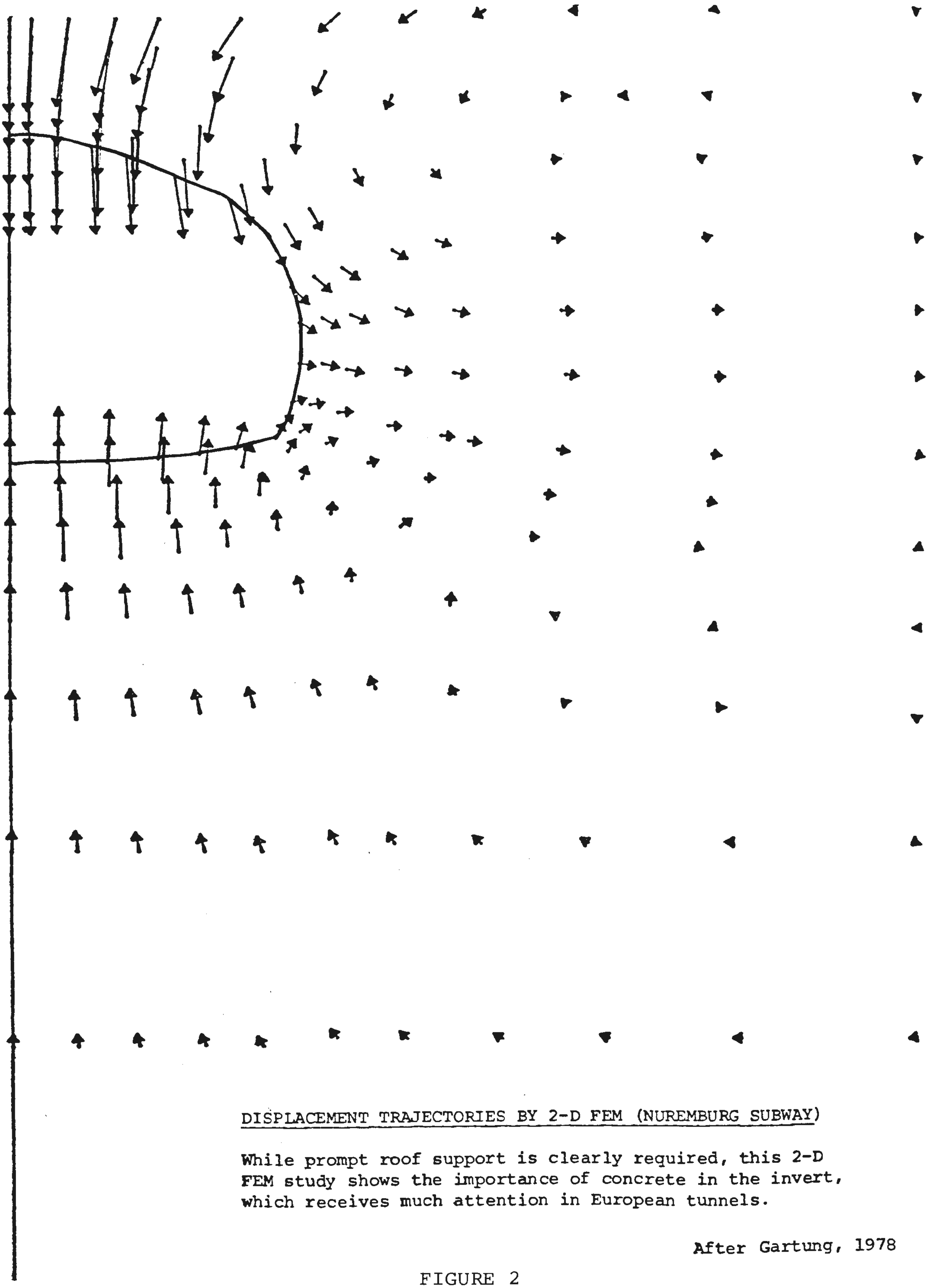

DISPLACEMENT BY 3-D FEM (NUREMBURG SUBWAY) STUDY

The 3-D FEM program includes step-by-step study of top heading and bench excavation. Visco-plastic ground properties are used as input data.

After Gartung, 1979

FIGURE 1

Atlanta Research Chamber Monograph

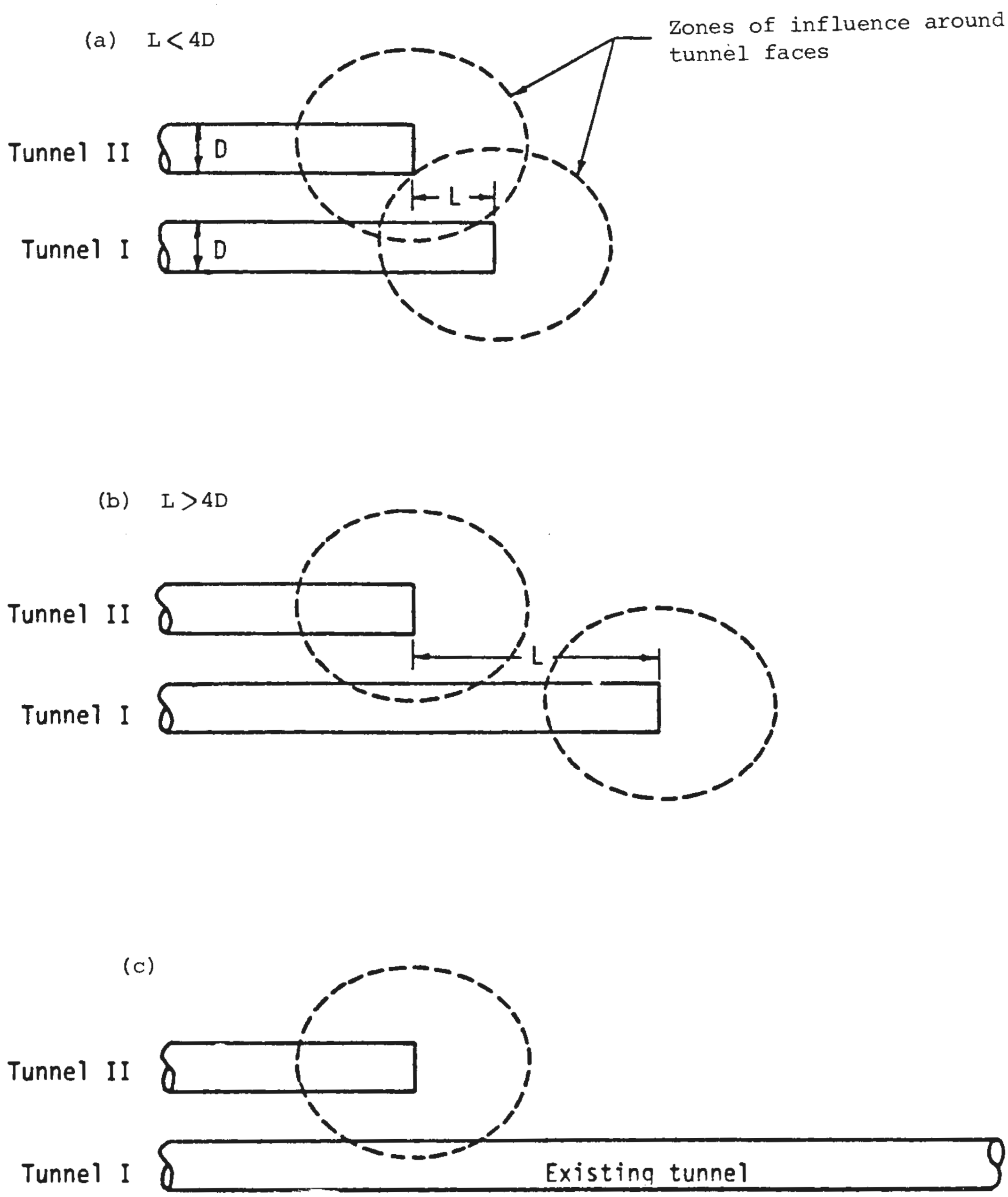


DISPLACEMENT TRAJECTORIES BY 2-D FEM (NUREMBURG SUBWAY)

While prompt roof support is clearly required, this 2-D FEM study shows the importance of concrete in the invert, which receives much attention in European tunnels.

After Gartung, 1978

FIGURE 2



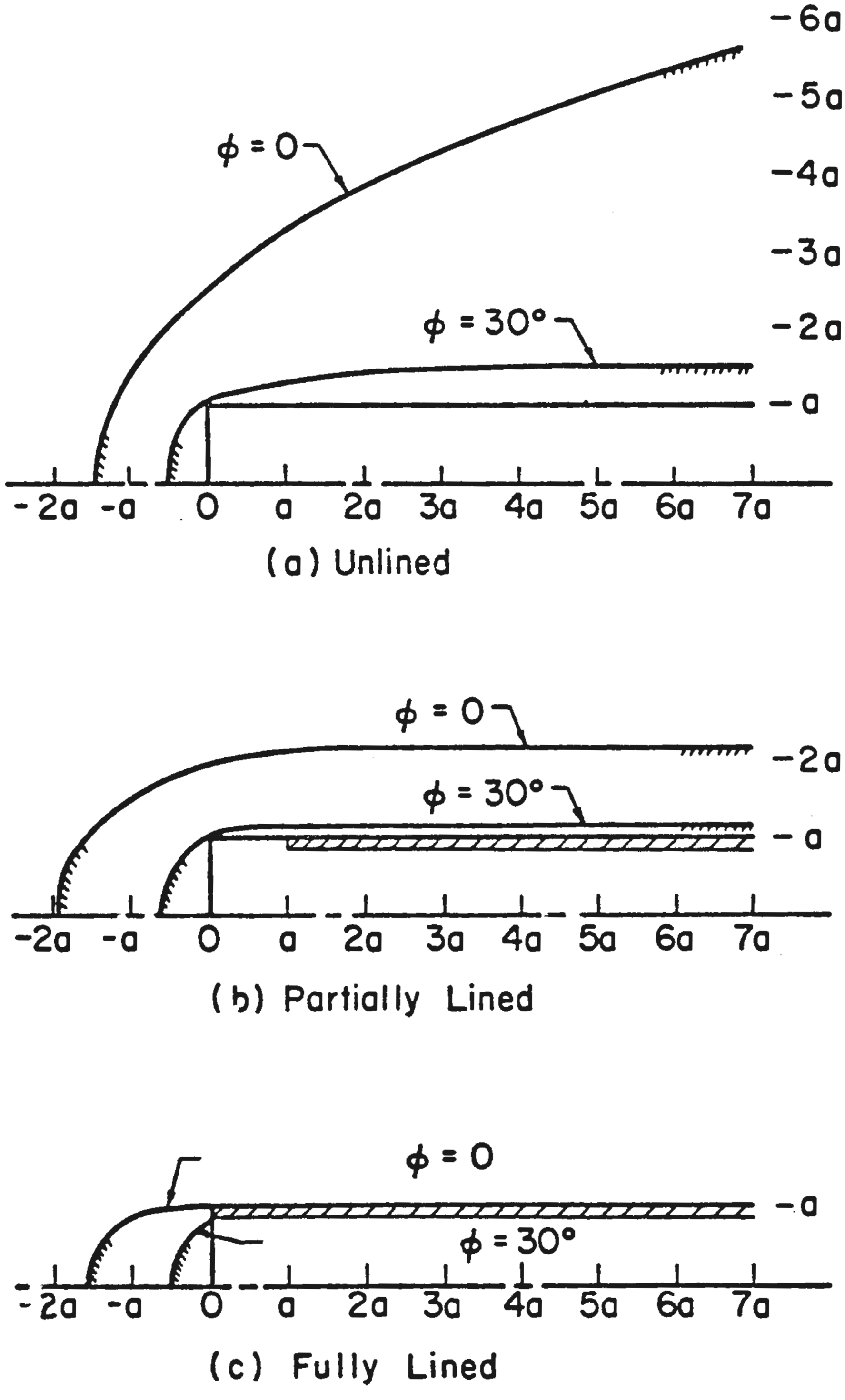

EFFECT OF RELATIVE POSITION OF TWO TUNNEL FACES

ON STRESSES IN ADJACENT TUNNEL

After Ranken et al 1978

"Analysis of Ground-Liner Interaction for Tunnels UMTA-IL-06-0043-78-3"

FIGURE 3

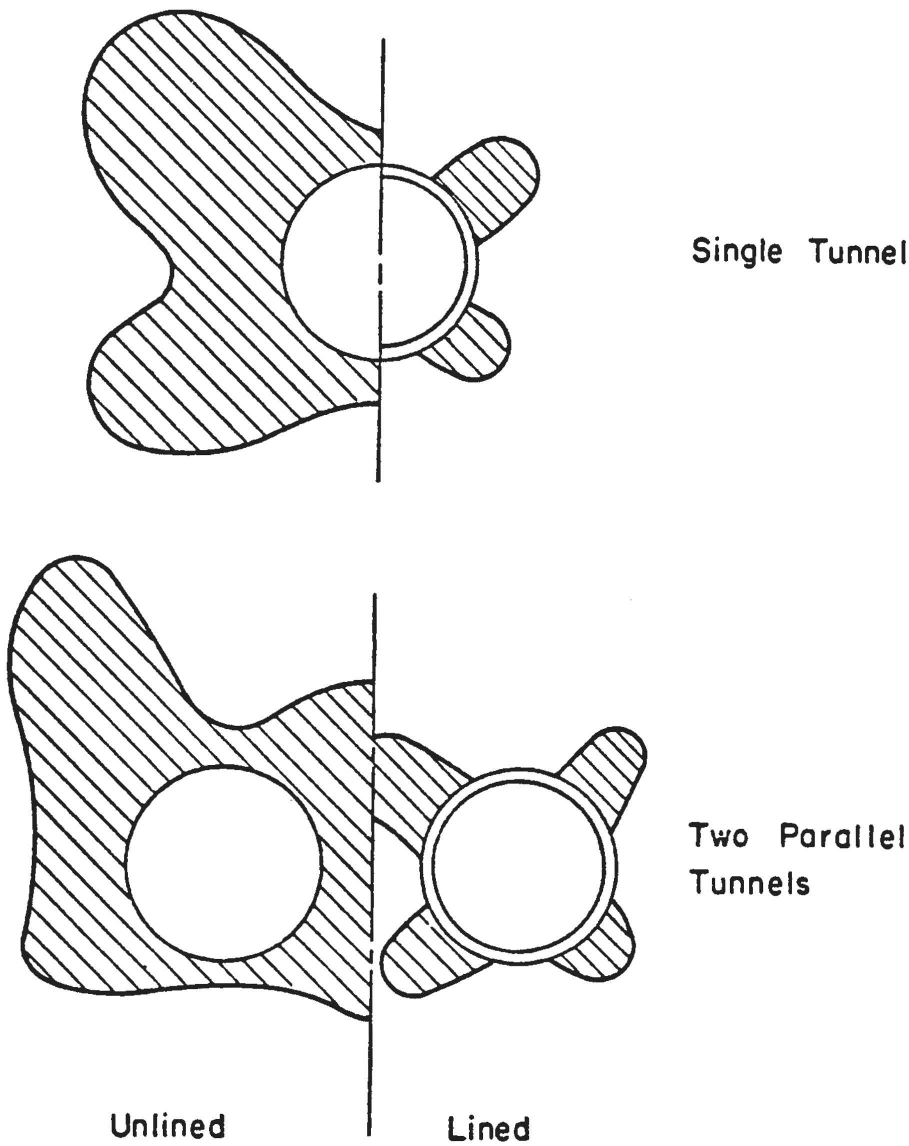



PLASTIC YIELD ZONE AROUND ADVANCING TUNNEL BY 3-D FEM

This axisymmetric-ring FEM study is in effect a 3-D study. Note how prompt lining of the tunnel reduces the extent of the plastic yield zone.

After Ranken et al 1978

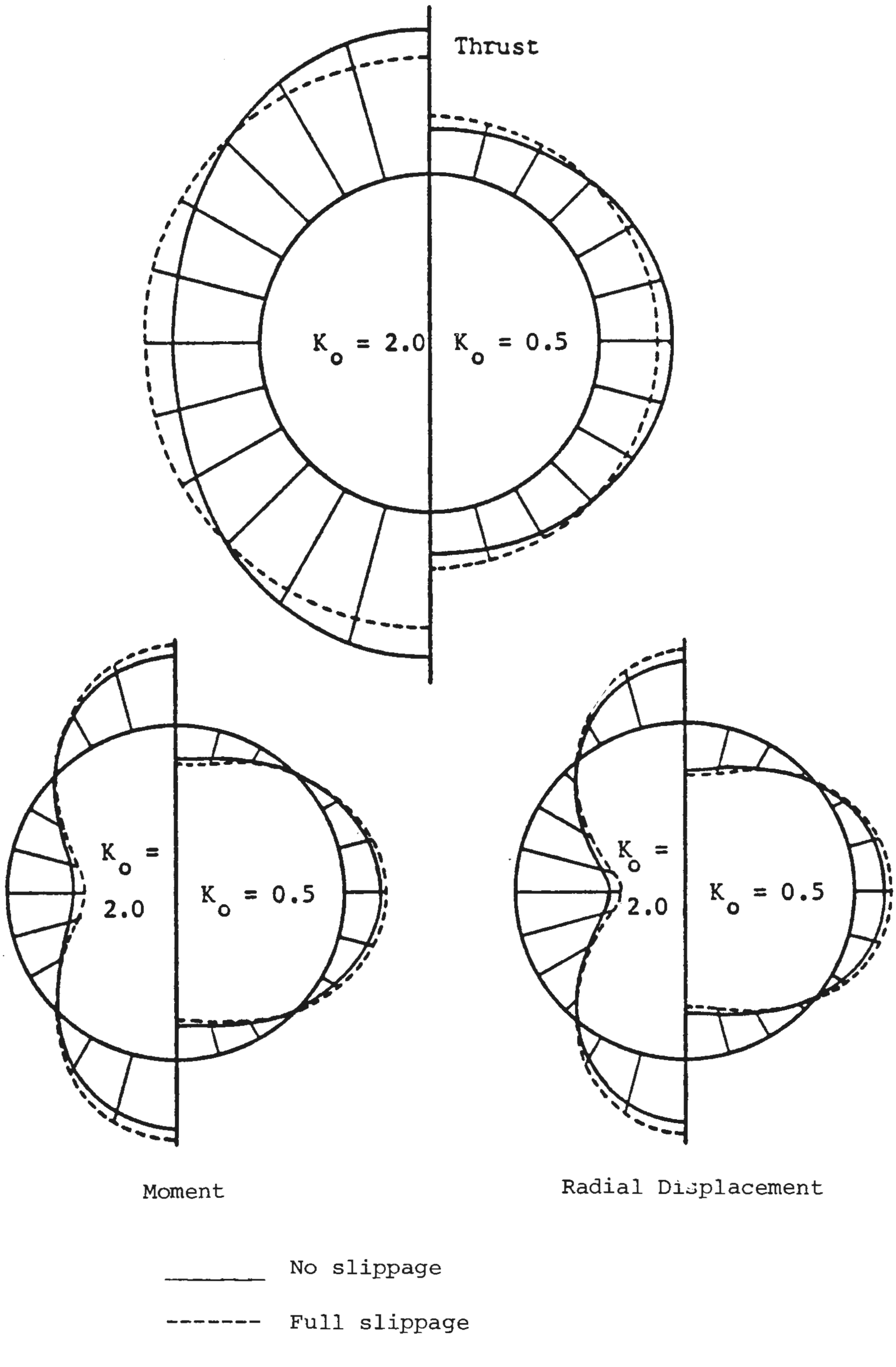
FIGURE 4




TYPICAL PLASTIC ZONES AROUND TUNNELS FROM 2-D FINITE ELEMENT ANALYSES

After Ranken et al 1978

FIGURE 5

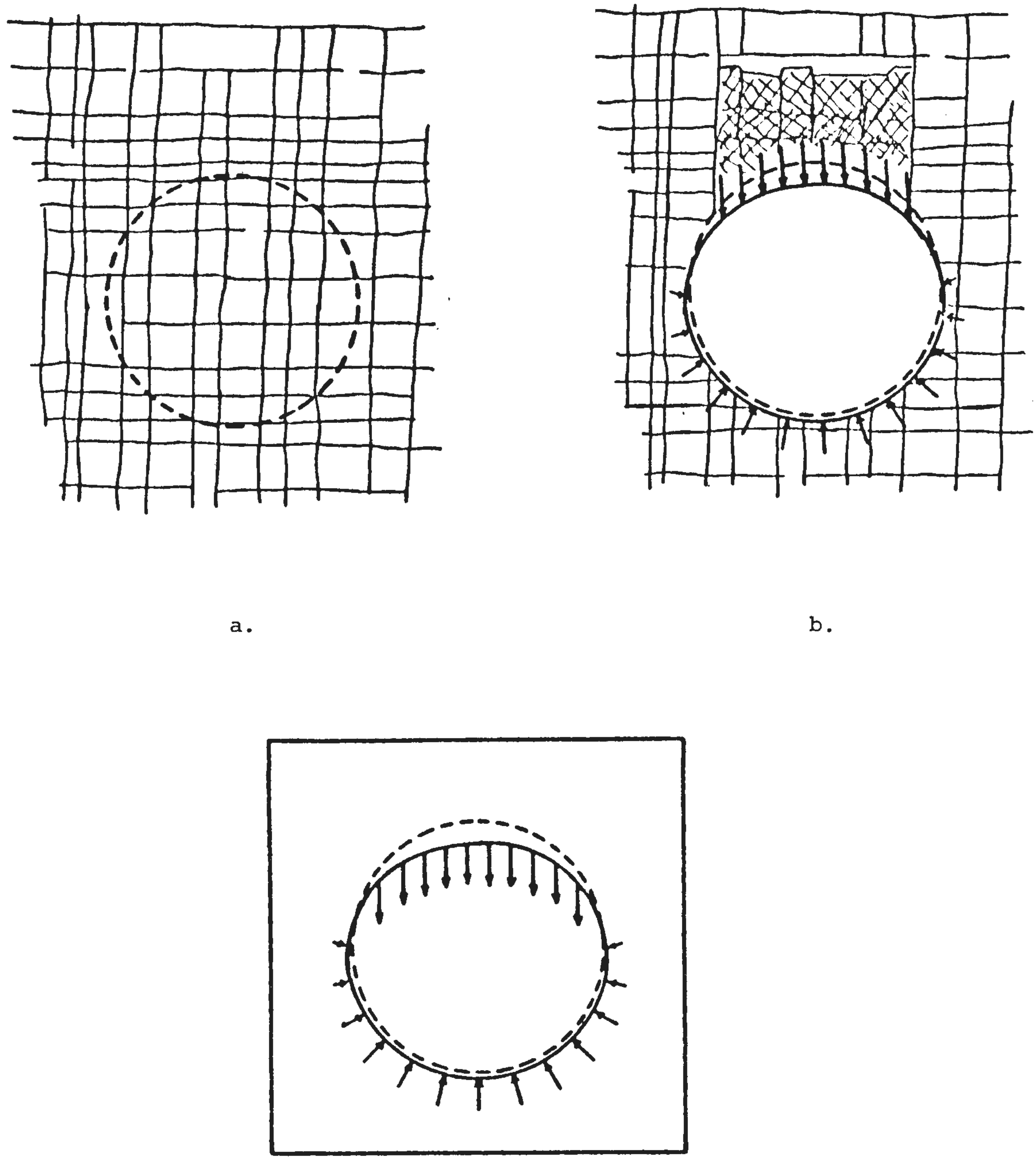



DISTRIBUTIONS OF LINER THRUSTS, MOMENTS AND RADIAL DISPLACEMENTS

2-D stress contours are strongly dependent upon Ko, the ratio of horizontal: vertical in-situ stress. Without accurate data on the real in-situ stress ratio, analysis of specific real jobs may be significantly in error.

After Ranken et al 1978

FIGURE 6



a.

b.

c.

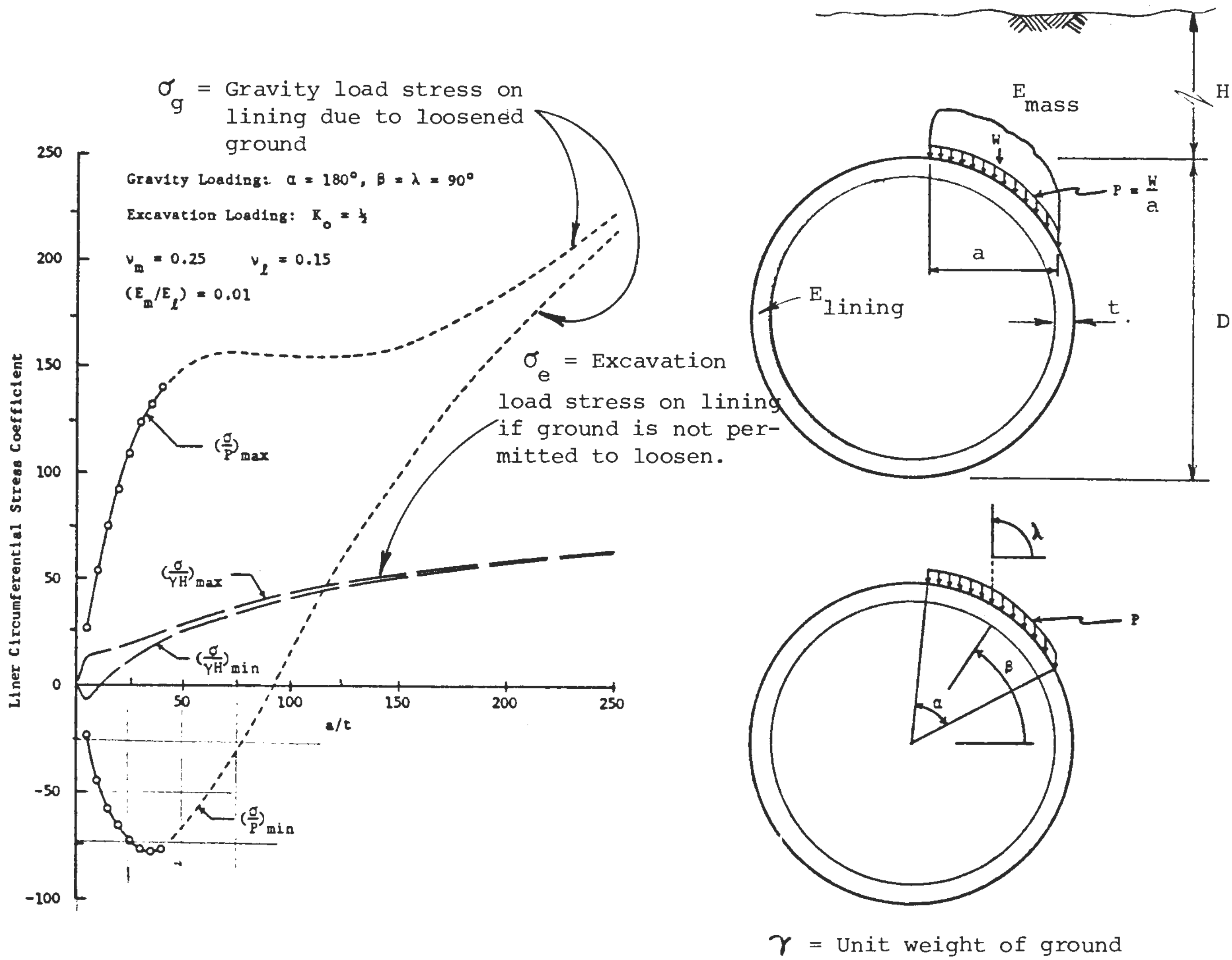
a and b : Problem to be simulated

c : Problem as simulated

IDEALIZATION OF THE LOCALIZED GRAVITY LOADING GROUND-LINER INTERACTION PROBLEM

"Gravity Loading" is the term used here for a loosened ground load acting as a dead weight on the lining. This is the typical North American method, based on Terzazhi and "Rock Tunneling with Steel Supports." After Ranken et al 1978

FIGURE 7




LOADS ON LINING BY 2-D FEM

These curves permit a comparison between "Excavation Loading" occurring if the ground is not permitted to loosen; and "Gravity Loading" if the ground is loose and lies dead weight on the tunnel lining.

Example: For clayey soil ($E_m$ = 30,000 psi) and for concrete lining ($E_l$ = 3,000,000 psi), the ratio $E_m/E_l$ = 0.01 and the curve above shows the stress in a circular lining. For a 4" shotcrete lining, in a 20' diameter tunnel, a/t = 30. For depth H = 100', from the curve it can be seen that excavation loads are about $\sigma_e = \gamma H20$ = (150 pcf)(100 ft.)(20) = 300,000 psf = 2083 psi, well within the compressive strength of shotcrete. However a gravity load, due to loose blocks 3' thick, exerts a tensile stress of $\sigma_g$ = (P)(-80) = $\frac{3' \times 10' \times 150 \text{ pcf}}{10'}$ (-80) = 36,000 psf = 250 psi.

Caution: These curves are only valid for $E_m/E_l$ = 0.01 and other curves (not shown) must be used for other values of $E_m/E_l$. In general, as $E_m/E_l$ increases (stronger ground) the lining load decreases, and vice versa.

After Ranken et al 1978

FIGURE 8

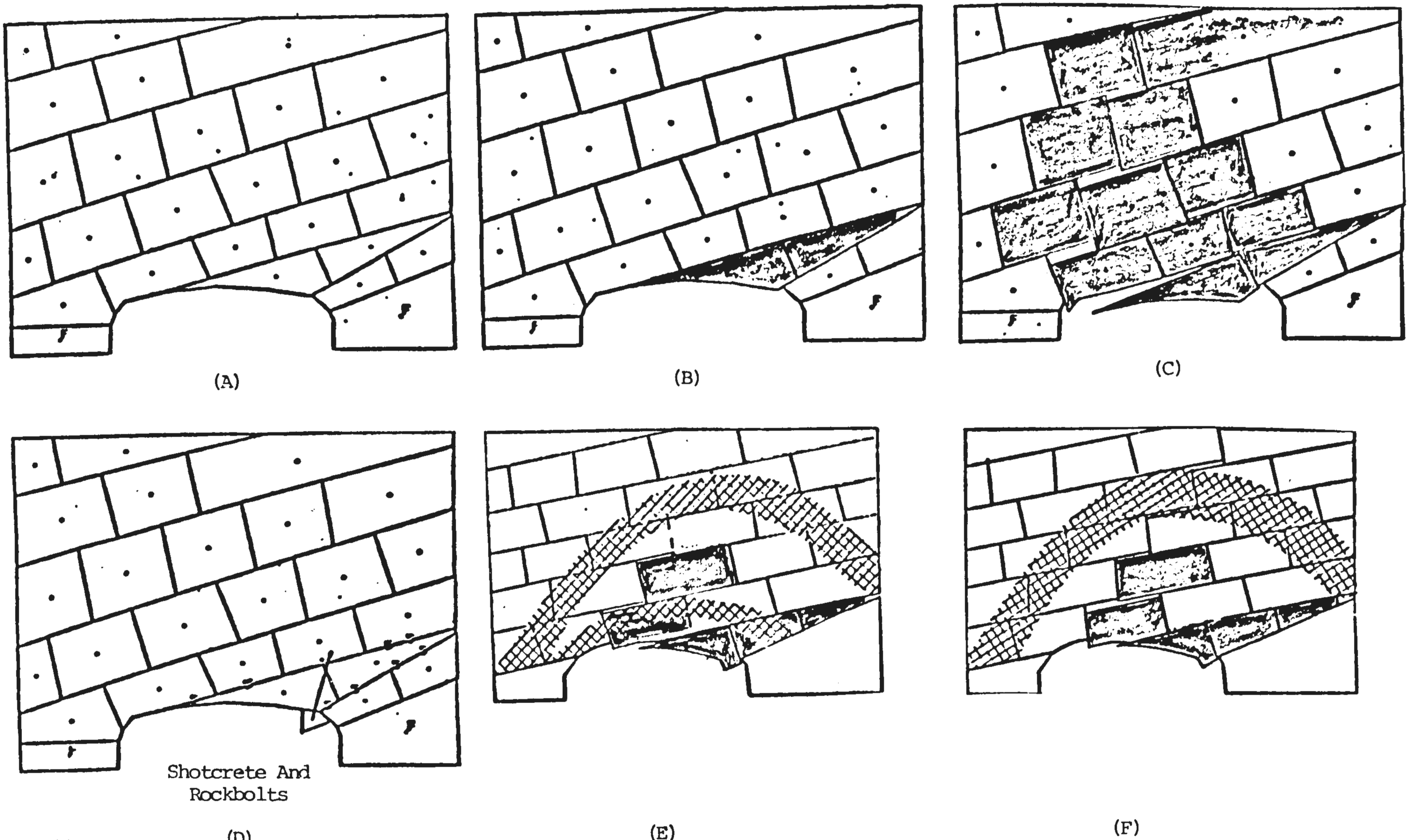



FIGURE 9

PROBABLE ACTION OF SHOTCRETE AND ROCKBOLTS

The unravelling shown in (A)-(B)-(C) can be prevented by a thin layer of shotcrete plus occasional rockbolts. In (E) the block pressures by good fortune form two ground arches which stabilize the tunnel; in (F) large roof fallouts may occur. After Voegle.

# ON HARD ROCK AND HARD PROGRESS

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The concept of rock, in its natural, in-place condition, as a structural material is both very old and very new. Petra and Abu Simbel are wondrous examples of ancient architecture. They are also examples of the intuitive structural sense of their builders, who needed neither computers nor building codes to tell them that sound rock, properly shaped, would hold itself up for thousands of years.

Closer to our own time, many early canal and railroad tunnels were unlined, except for the portal zones, and some have survived in this condition for over 100 years. The Wawona Tunnel at the entrance to Yosemite Valley in California is an example of a highway tunnel that has stood for 40 years with a large portion unlined. In all these unlined tunnels, the only structural material used was natural, in-place rock.

Until comparatively recently, the "design" of rock tunnels was intuitive, based on precedent and experience rather than rational analysis. The first serious attempt to rationalize rock tunnel design was Terzaghi's classification of types of tunneling ground, for which he established in the 1940's proportions of a "loosened rock zone" which might be expected to develop above the tunnel crown. This classification was based largely on experience derived in construction of relatively narrow tunnels in the

Alps, for single-track railroads and water power projects. (Terzaghi's qualitative descriptions of rock behavior are among the best ever written, and even today offer some of the best guidance available to the designers of rock tunnels.)

Unfortunately, Terzaghi's monograph was perverted into a cookbook by a generation of building designers who were assigned to design tunnels, and who seized on the only available text on the matter as their manual. Where Terzaghi tried to describe the configuation of a stable arch formed of natural rock, these "designers" saw in his paper only formulas for calculating the weight of loose rock below the stable arch, which would have to be carried on structural supports. Rock came to be regarded as a load rather than as a structural material. Regardless of geological conditions, rock quality, or rock structure, the "standard" U.S. design for rock tunnels became:

a. Provide a "temporary" lining composed of steel arch ribs, proportioned to carry the weight of a height of rock equal to some arbitrary fraction of the tunnel width.

b. Provide a "permanent" lining of concrete, with a thickness of one inch per foot of tunnel width.

c. Make the contractor responsible for all ground conditions, and let the lawyers handle the claims.

The best that can be said for these tunnel "designs" is that very few of them collapsed.

As the size of underground openings increased, the old rules-of-thumb (based on narrow tunnel experience) became unwieldy and obviously expensive. The designers of large hydroelectric power projects needed better criteria and better concepts, particularly for the large underground powerhouse caverns that started to proliferate in the 1950's. This generated the concept of "rock reinforcement", in which the old intuitive sense was rationalized. New aims were to prevent the loosening of the natural arch, to repair defects in the rock structure, and to use the rock as a structural material, rather than to accept the loosening as inevitable, and to regard the loosened rock as so much liquid dead weight.

Some of the earliest applications of the new concept of rock reinforcement in the U.S. were to the hardened underground defense centers constructed in the 1950's, designed to resist the effects of nuclear weapons. The development resulted primarily, however, from the growth of hydroelectric power projects, in the U.S., Canada, and Australia, where the use of rock bolts and shotcrete as rock reinforcement for both temporary and permanent conditions became virtually a new "standard" by the 1970's.

While not devoid of theoretical support, the design of American rock reinforcement developed largely on an empirical basis, drawing heavily on the experience gained in the construction of instrumented test chambers. As a sufficient body of construction records accumulated, it became possible to document and correlate the degree of reinforcement by bolts and shotcrete that had been demonstrated to give satisfactory performance in a wide range of tunnel and cavern sizes and geological conditions. The American theoretical development has been strongly influenced by rationalization of this performance record.

Simultaneously, a parallel development of rock reinforcement concepts has occurred in Europe, stimulated both by the proliferation of hydroelectric power caverns and by the growth of larger diameter highway tunnels in the Alps and transit construction in Scandinavia. As might be expected from European traditions, their development has had a much stronger and more detailed mathematical component, and the elaboration of theoretical behavior of rock structures has reached astonishing proportions. Nonetheless, the marriage of mathematical theory with construction practice has been successful, and has produced an optimized system (for the geological conditions encountered) in which the loosening and yielding of the rock structure is encouraged and controlled, so as to require the minimum amount of internal support. This development has been widely publicized as the "New Austrian Tunneling Method" (NATM). It depends heavily on interactive instrumentation -- field measurements of rock deformation and movement, which form the basis for field modifications of construction procedures and of the timing and extent of support installation.

It should be noted that the mathematical development of NATM has been principally applied to shallow subway construction, and that its application to deep lying tunnels is still largely empirical. It should also be noted that British, Swiss, and Scandinavian tunnelers have developed similar "yielding ground" systems, but have been less successful in publicizing trade names for them.

Superficially, the American system of rock reinforcement aims at preventing rock movements to retain the strength of the intact rock, while the European system encourages controlled movement as a method of mobilizing the maximum strength of the rock and minimizing the supplemental support required. In fact, the two concepts are not in conflict, but represent opposite ends of a spectrum. American work has largely been constructed in hard, elastic rocks where maximum natural strength is best developed by minimizing rock loosening, while the European work has had to encompass plastic rock conditions and squeezing ground, where the prevention of all movement would require excessive support, and the beneficial effects of controlled movement may be substantial.

The foregoing brief summary gives a general view of the present state of technical knowledge and experience. In the field of hydroelectric power, advanced rock reinforcement techniques are being applied as a matter of course. Unfortunately, the same cannot be said for transportation tunnels and caverns (such as subway stations). The reasons for this technological lag are of some interest.

First is habit. It is still the practice in some jurisdictions to design a "temporary" lining of steel sets to carry the full weight of overburden (regardless of whether it is rock, soil, or water), and to add a separate "permanent" concrete lining designed to carry the same load "because the steel may rust away." When this concept was originally applied to single-track running tunnels, it yielded nominal 6-inch ribs, which made the tunnel excavation crews comfortable, and a lining as thin as could be practically poured. (This was really not a bad design for the rock conditions encountered.) The fact that applying the same concept to a 50-foot wide station cavern yields 24-inch ribs and four foot thick walls has for some reason not caused the designers to question the basis for the concept. But now the excessive cost involved is significant.

Second is precedent. It is generally recognized that successful underground construction depends more on experience than mathematical analysis, and that the variables of geology (and geohydrology) make the performance of underground structures less predictable than that of buildings or bridges. In these circumstances there is a strong prudent tendency to seek concepts, methods, and details that have worked satisfactorily on past similar projects and under similar conditions. This by itself is healthy, but unfortunately the range of acceptable precedents tends to be narrow. In the agencies that fund the design and construction of transportation tunnels, "precedent" tends to be confined to transportation projects, if not projects of that particular agency, and what has become common practice in hydroelectric power projects is deemed irrelevant.

The third reason for technological lag is government practice in the procurement of engineering services. It is obviously cheaper to design tunnels by rule of thumb than by rational analysis. All agencies currently procuring design services have an absorbing interest in reducing the cost of such services as an end in itself. Very few agencies are willing to pay extra for sophisticated design in order to secure what seems to them to be a speculative reduction in construction cost. The basic problem is that the cost of design services is interminably auditable as a negotiated contract and so is highly sensitive, while the cost of construction obtained by competitive bid is not audited. The additional cost of design can therefore be "proved", while the saving in construction cost (or the opportunity for saving that is foregone if the design effort is curtailed) is a matter of opinion. It takes a strong agency director to justify spending hard design dollars by his opinion that he is going to save construction dollars later on.

Fourth is the splintering of the design/construction team. There is a disturbing tendency in some government and private agencies to compartmentalize their engineering forces. An agency may engage different firms to serve as General Engineering Consultant, General Soils Consultant, Section Designer, and Construction Manager. The scopes of services of these separate consultants are carefully drawn to ensure that there is no overlap. Unfortunately, this usually means there is also no communication. Rock reinforcement concepts require close coordination of site investigation, preparation of plans and specifications, field instrumentation, and construction contract administration. When this coordination is hampered by artificial contracting strictures, the opportunities for securing the benefits and cost savings of modern rock construction practice become dim.

Finally, overriding all these constraints in intensity is the fear of liability. In these days when no turn remains unstoned, the incentives for innovation are minimal and the penalties for transgression severe. The threat of censure and litigation oppresses every departure from demonstrable precedent, and the criterion for judging every progressive idea is not whether it will work in the tunnel but whether it can be defended in court. This is a sorry climate in which to attempt to improve the state of the art.

Nonetheless, a few attempts have been made and are worthy of brief note. When the first mined rock cavern stations for the Washington Metro were designed, they represented a pioneering venture for transit tunnels, and the design was cautiously conservative. Instrumentation and observation of the construction behavior of these first stations provided data from which criteria for reduced design loadings were developed for subsequent stations. While rock reinforcement (bolts and shotcrete, supplemented with some steel ribs) has been adopted for some WMATA running tunnels, the stations still require primary steel ribs, supplemented with either poured concrete or massive shotcrete. By European or hydropower standards, even WMATA's improved designs would be regarded as heavy structural support. (It should be noted that the geology of Washington, D.C. contains extensive shear zones which pass diagonally through many of the station sites).

Atlanta's MARTA transit system includes a mined hard rock station beneath Peachtree Center in the city's central business district. Preliminary investigations indicated unusually sound rock, and a pilot tunnel disclosed no serious rock defects and only six significant rock joints in a length of 600 feet, which encompassed the bulk of the station cavern. On the basis of this evidence of unusual rock quality (it was dubbed "nice gneiss") it was decided to eliminate the steel ribs entirely, and to utilize untensioned grouted rock dowels and rock bolts for construction support. Primary reliance was placed on using the inplace rock as a structural arch, with the dowels and bolts serving to knit the rock structure together across joints, and to preclude loosening of slabs between joints. From a structural viewpoint, a permanent thin shotcrete skin was considered adequate for additional permanent support, and was initially recommended. However, uncertainty regarding the long term durability of thin shotcrete in public areas (how can you prove it in court?) eventually resulted in a decision to include a relatively thin 41 centimeter (16-inch) minimum poured concrete arch lining over all public areas. The lining is designed primarily to support the pressure of the contact grout between the lining and the rock. The majority of the station walls will be left as exposed rock, which is used as an architectural material in the station finish design.

It is of interest to note that Dr. Rabcewicz, the principal developer and exponent of the NATM in Europe, encountered similar hesitancy in the adoption of shotcrete for permanent roof support. In his article in Water Power (August, 1969, page 300), in discussing the Kops underground powerhouse project in Austria, he states:

> ...although the owner preferred to insert a relatively thin concrete roof arch in addition (to the rock bolts and shotcrete, subsequent stress measurements showed) that the share of the rock load taken by the concrete arch was close to zero.

In conjunction with the MARTA Peachtree Center Station construction, UMTA sponsored the Atlanta Research Chamber in which a portion of the original Pilot Tunnel has been instrumented and enlarged, to provide data on the behavior of the Research Chamber while the twin Running Tunnels are excavated beneath it. The project is also providing demonstrations and field tests of installations of several forms of shotcrete construction. It is intended that the results of this test program will provide a basis for improved design criteria for future projects.

Finally, for the Pittsburg light rail transit project, on which final design is just starting (Spring 1979), UMTA is funding a full-scale demonstration of NATM. For the Mt. Lebanon Tunnel, comprising twin single-track bores each about 3,000 feet long, a complete separate NATM design will be prepared for bidding as an alternate to the "standard" design. The cost of the additional design and construction inspection attributable to the NATM concept is borne by the UMTA R&D grant, with any savings in bid construction cost accruing to the project owner (the Port Authority of Allegheny County), and indirectly to UMTA through reduction in their Capital Construction grant to the project.

Over the past 10 years, since the start of major underground construction on the Washington Metro, there has been great movement at the forward edge of American design practice for transportation tunnels. Unfortunately the bulk of the glacier lags behind its leading edge. The ability of tunnel constructors to chew through hard rock has greatly outpaced the ability of tunnel designers to beat their way through the institutional and emotional barriers to improved rock tunnel designs. Hard rock excavation is a challenge that constructors have largely surmounted; hard rock design progress remains a challenge to the design profession.

# A COMPARATIVE VIEW OF RAPID TRANSIT TUNNELING

by

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## Introduction

The past decade has been a time of immense activity in rapid transit tunneling. In the United States, rapid transit systems have been developed for San Francisco, Washington, D.C., Atlanta, Baltimore, and Buffalo as well as major extensions undertaken for the New York City and Boston subways. In western Europe over twenty different metro systems either have been initiated or expanded as part of an effort to develop urban rapid transportation. A substantial part of the construction has been performed underground, often in central business districts where heavy congestion has led to special constraints on the work and where the proximity of large, sensitive structures has required care regarding stability and ground movements.

The recent construction activity has been accompanied by substantial escalations in cost. As a consequence, funding for transportation tunneling has been subject to increasingly severe scrutiny by federal and local governments. In some instances, pointed criticism has been directed against current construction procedures [1].

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[1] J. W. Fondahl, B. C. Paulson and H. W. Parker (1979), "Reducing Costs in Urban Transportation Construction", Journal of Constr. Div., ASCE, Vol. 105, No. C01, pp. 51-63.

Construction costs have varied widely both within and among the countries engaged in major urban tunneling. Although cost differences are likely to reflect different ground conditions, variations in expense also are linked closely with differences in planning, work organization, and applied construction technology. Despite the great number of recent tunneling projects, there has been very little explicit information on construction expense.

Construction practices in different countries often reflect special attitudes and local customs in interpretation of contracts and engineering authority. What is useful practice, therefore, in some locales may not be appropriate for other systems. More broadly speaking, however, a comparative survey of construction practice not only uncovers different technologies, but reveals different ways of implementing the technologies at hand. A comparative view of rapid transit tunneling can show alternative approaches to planning and construction and suggest improvements for some systems.

This paper examines underground construction costs for several European rapid transit systems. Much of the data has been condensed from a larger work by the writer [2/], and reference to this should be made for further information. Comparisons are drawn between tunneling costs in the United States and Europe. Recommendations for improving tunneling practice are made.

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[2/] T. D. O'Rourke, (1978) "Tunneling for Urban Transportation: A Review of European Construction Practice", Report No. UMTA IL-06-0041-78-1, prepared for the U. S. Department of Transportation, Urban Mass Transportation Administration, Washington, D. C. 20590.

## Underground Construction Costs

As Girnau has pointed out [3/], costs within the same city can vary from line to line and even from contract to contract. Furthermore, the cost of an underground project will stem directly from the objectives fixed for its eventual operation. In many instances, these objectives are coordinated either within the regional organization of transportation facilities or a pattern of urban renewal. This latter aspect has been emphasized by Lupiac [4/] in his discussion of expenses related to underground constructions in Paris.

A summary of tunneling expenses among different cities must therefore be judged in light of the wide variation in factors affecting the cost. Comparisons among the different expenses must be made with an appreciation for the size, overall organization of the systems, and especially for the geotechnical constraints imposed by the ground conditions.

For this paper construction cost is defined as the expense of the excavation and subsequent placement of the structure in the ground. It includes the costs of ground water control, excavation, temporary support, building the final structure, and site supervision associated with the work. It also includes the costs of diversion and reinstatement of roads, pipelines, and cables. The construction cost does not include the expenses of electrification, signaling, track, ventilation (although ventilation shafts are included), rolling stock or land acquisition. For the European transit systems summarized in this paper, it does not include the cost of underpinning adjacent structures.

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[3/] G. Girnau, (1978) "Lining and Waterproofing Techniques in Germany", Tunnels and Tunneling, Vol. 10, April, pp. 36-45.

[4/] L. Lupiac, (1978) "The Paris Conference Organized by AFTES", Tunnels and Tunneling, Vol. 10, January/February, pp. 27-28.

All costs reflect price levels for 1975-1976, which are chosen as the base comparison years for this paper. Where the available cost information was referenced to a year previous to the base comparison years, this cost was adjusted to a 1976 level by using inflation indices provided by either the British Department of Environment or the Organization for Economic Cooperation and Development. Adjustments of this type were made for the Lyon Metro and the North-South Line of the Tyne and Wear Metro. All national currencies with which the costs were originally quoted have been converted to U.S. dollars as per the exchange rates prevalent during the base comparison years.

## Summary of Construction Costs

Figure 1 shows a bar graph of unit construction costs associated with six European Metro systems. The costs related to construction in soil and rock have been separated. All unit costs are expressed in terms of U.S. dollars per double track mile or foot. The costs have been determined by dividing the total construction cost for a given distance of line by the length of line under study. Hence, the unit costs include the expenses of building both stations and running tunnel. All costs have been abstracted from portions of rapid transit systems that have been constructed almost entirely underground. Sections of surface line, which may be included in the unit costs, are so small relative to the subsurface work that they have very little influence on the prices quoted.

Table 1 summarizes basic information regarding the sections of rapid tranportation systems referred to in Fig. 1. The table contains information on the average depth, dimensions of both running tunnel and stations, distance of line per station, and approximate geology. The distance of line per station refers to the total length of metro line for which each cost applied is divided by the number of stations included in the cost. This

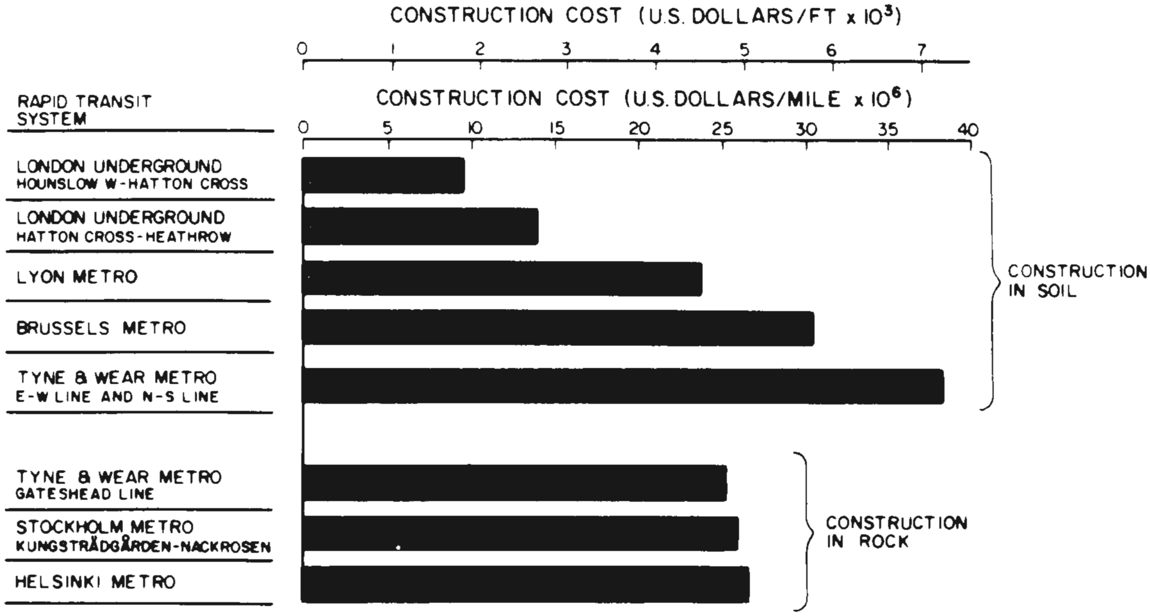


Figure 1. Unit Construction Costs for European Metros

| RAPID TRANSIT SYSTEM | AVRG. DEPTH (FT) | TUNNEL CROSS SECTION (FT) | STATION DIMENSIONS (FT) | DIST OF LINE PER STATION (Mi) | APPROX GEOLOGY |
|---|---|---|---|---|---|
| LONDON UNDERGROUND HOUNSLOW W-HATTON CROSS | 23 | 13 x 24 | PLAN: 430 x 79 | 1.05 | SAND BELOW WATER LEVEL; HARD CLAY |
| LONDON UNDERGROUND HATTON CROSS-HEATHROW | 56 | 12.5 I.D. | | 1.30 | |
| TYNE & WEAR METRO N-S LINE AND E-W LINE | 59 AND 46 | 15.6 I.D. | 23 I.D. PLATFORM LENGTH = 312 | 0.42 | BOULDER CLAY |
| LYON METRO | 25 | 13 x 24.6 | PLATFORM LENGTH = 232 - 341 | 0.47 | SAND BELOW WATER LEVEL |
| BRUSSELS METRO | 25-60 | 15 x 24.6 | PLATFORM LENGTH = 312 | 0.48 | SAND BELOW WATER LEVEL |
| | | | | | |
| TYNE & WEAR METRO GATESHEAD LINE | 45 | TWIN D-SHAPE 19 x 18 | PLAN: 328 x 82 | 0.54 | LIMESTONE, SANDSTONE |
| STOCKHOLM METRO KUNGSTRÄDGÅRDEN-NACKROSEN | 100 | D-SHAPE 15 x 26.5 | PLATFORM LENGTH = 590 | 0.61 | GRANITE, GNEISS |
| HELSINKI METRO | 89 | TWIN D-SHAPE 17 x 18 | PLAN: 443 x 59 | 0.50 | GRANITE, GNEISS |

Table 1. Summary of Information for European Metros

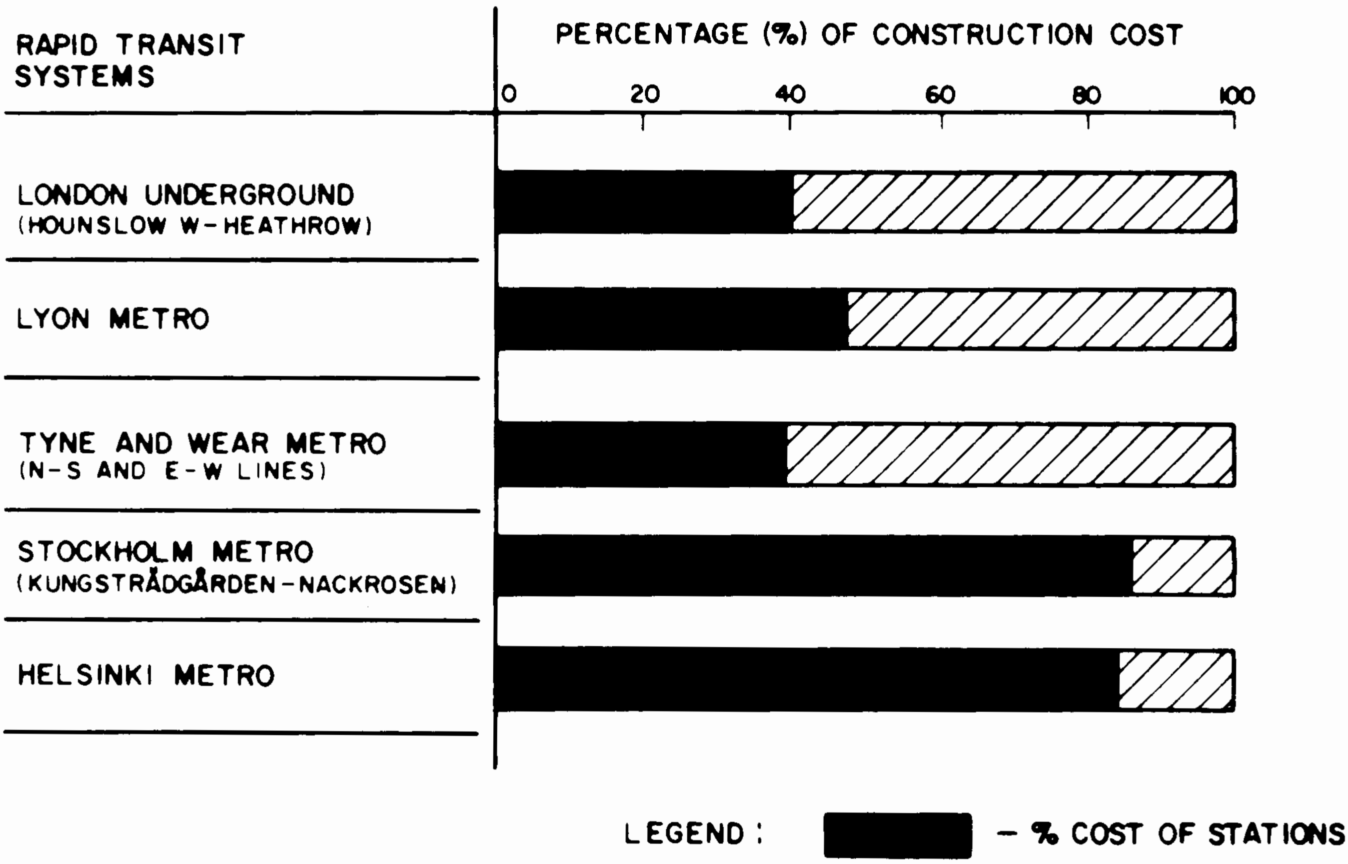


Figure 2. Relative Cost of Stations for European Metros

provides an index of station density and allows the unit costs to be judged in light of the relative number of stations covered by the costs.

The unit costs of construction in soil vary through a wide range. For example, there is a substantial difference in cost between twin tunnel sections of the London Underground and the Tyne and Wear Metro. The main reason for the large difference is that the number of stations included in the unit cost for the Tyne and Wear Metro is nearly three times larger than the corresponding number for the London Underground. As the stations are major sources of expense, the unit construction costs of the metros increase in proportion to increases in station density. With the exception of the London Underground, the distance of line per station for the reported construction in soil and rock fall within very narrow bounds.

An additional reason for the difference in cost between the twin tunnel sections of the London Underground and the Tyne and Wear Metro is the difference in tunnel diameter and type of lining used for the systems. The tunnels for the London Underground are 3.8 meters internal diameter and were constructed with expanded concrete rings. The tunnels for the Tyne and Wear Metro are 4.8 meters internal diameter, of which approximately 60 percent were constructed with bolted iron segments and 40 percent were constructed with bolted concrete segments. In addition, approximately 60 percent of the tunnels were driven under compressed air for the Tyne and Wear system.

## Components of Construction Cost

Figure 2 provides a bar graph showing the percentage of the total construction cost that was taken up by station construction for several metro lines. For the London Underground extension to Heathrow Airport, 40 percent of the construction cost can be attributed to building the stations. This is consistent with previous cost experience during tunneling for the Victoria Line of the London Underground. The percentage also compares well with a similar analysis of both the Lyon and the Tyne and Wear Metros.

On the Scandinavian metros, station construction accounts for an extremely large proportion of the construction cost. The main reason for the apparently high price of station construction is the relatively low cost of building the running tunnels. Rock excavation, rock bolt support, and grouting varied in expense between $730-880 per foot and $590 per foot for double-track tunnel route on the Stockholm and Helsinki systems, respectively. Although there are notable exceptions, transit tunnels in Stockholm and Helsinki were often advanced with drill and blast rounds of 2.5 meters for single track tunnels, approximately 25 square meters in cross-sectional area, and permanently supported by an 8 centimeter thick shotcrete arch in the crown with rock bolts as required. In contrast, the construction cost of double-track running tunnel in rock on the Tyne and Wear Metro was priced at approximately $3100 per foot. The rock tunnel for this system was advanced in approximately 1-meter lengths with a heavy road-header-type excavator. Steel arches were erected for temporary support and concrete was poured for the final tunnel lining.

It is interesting to note that the station dimensions in Scandanavia are relatively large. Station platform lengths of 590 feet for the Stockholm Underground are the largest in Western Europe and, in comparison world-wide, are only smaller than those for the San Francisco, Washington, D.C., and Atlanta Metros.

## Comparison of U.S. and European Costs

Two U.S. metros, in Washington, D.C. and Baltimore, were chosen to represent U.S. rapid transit construction. The costs, quoted for each system are derived from their initial construction stages. Unit costs are defined and computed exactly as they are for the European systems with the exception that the cost of underpinning adjacent structures also is covered in the unit price. Costs for the Washington, D.C. Metro are based on contract awards made between 1969 and 1971, which have been adjusted for inflation [5/] to 1975 price levels. Additional cost adjustments have been made for claims approved as of January, 1977. The costs for the Baltimore Metro are based on contract awards made during 1977. A detailed breakdown of the project costs contributing to the unit expense of each system has been performed elsewhere [6/]. Table 2 summarizes information for each metro on the average depth, dimensions of both running tunnel and stations, distance of line per station, and approximate geology. The sections of the U.S. metros examined in this paper have station densities, expressed as distance of line per station, that are consistent with the bulk of the European metros that have been analyzed.

Figure 3 compares the underground construction costs for U.S. and European metros in the form of a bar chart. All unit costs are for urban underground construction in soil. On the basis of the information summarized, U.S. costs are substantially in excess of European costs.

In some cases, it would be misleading to make direct comparisons among the costs of metro systems. For example, the number of stations included in the unit prices of the U.S. metros

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[5/] U. S. Department of Labor, Bureau of Labor Statistics (1977), "Handbook of Labor Statistics, 1977", Bulletin 1966, U. S. Government Printing Office, Washington, D. C. 20402.

[6/] T. D. O'Rourke, (1978) "Tunneling for Urban Transportation: A Review of European Construction Practice", Report No. UMTA IL-06-0041-78-1, prepared for the U. S. Department of Transportation, Urban Mass Transportation Administration, Washington, D. C. 20590.

| RAPID TRANSIT SYSTEM | AVRG. DEPTH (FT) | TUNNEL CROSS SECTION (FT) | STATION DIMENSIONS (FT) | DISTANCE OF LINE PER STATION (MI) | APPROX. GEOLOGY |
|---|---|---|---|---|---|
| WASHINGTON, D.C. METRO: DUPONT CIRCLE TO UNION STATION | 55-60 | 12 x 30 | PLATFORM LENGTH=600 | 0.51 | DENSE SAND BELOW WATER LEVEL |
| BALTIMORE METRO: CHARLES CENTER TO BOLTON HILL STATION | 60-70 | I.D. 17.9 | PLATFORM LENGTH=450 | 0.65 | DENSE SAND BELOW WATER LEVEL |

Table 2. Summary of Information for United States Metros

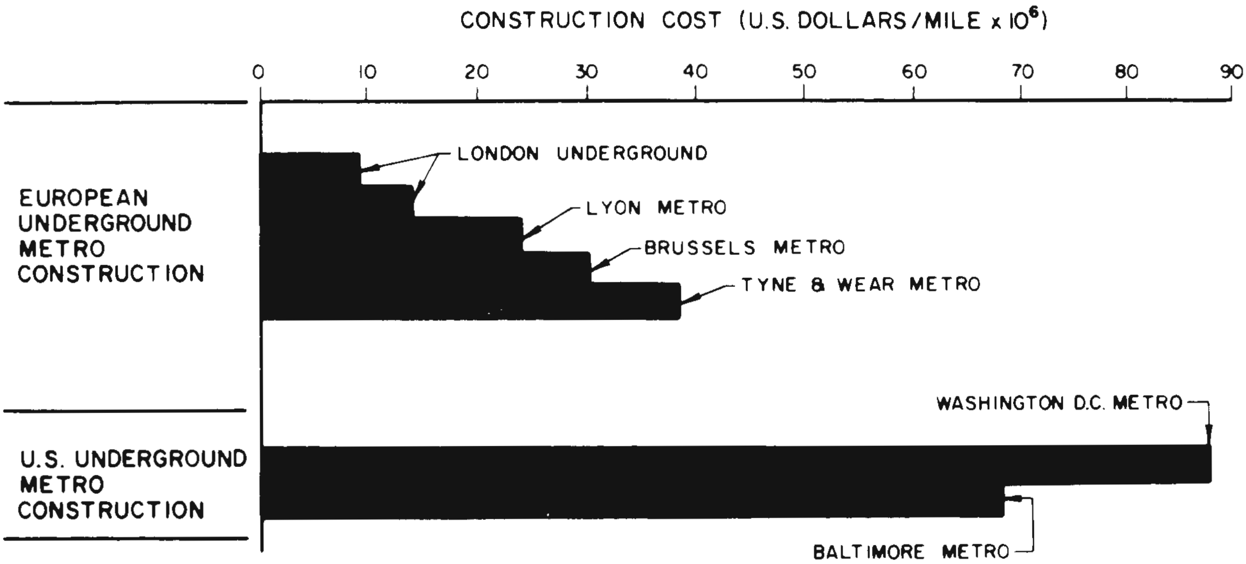


Figure 3. Unit Construction Costs for United States and European Metros

is approximately twice the number for the London Underground. Furthermore, the ground conditions under which the London tunnels were driven cannot be equated with the waterbearing sands and silts in which the Washington, D.C. and Baltimore Metros were constructed.

On an overall basis, the unit costs for the European metros cover a variety of ground conditions, some of which are similar to those prevalent for the U.S. metros. Approximately 60 percent of the soil tunnels on the Tyne and Wear Metro, for example, were driven under conditions requiring compressed air. Cut-and-cover construction for the Brussels Metro was typically extended to depths of 18 meters through sands and interbedded clay below the water table. Furthermore, the number of stations per mile on several of the European metros exceeds the corresponding number on the U.S. metros.

## Cost of Labor

A detailed study of labor costs is beyond the scope of this paper. It is recognized, however, that some indication of relative labor costs would be useful when judging the constituent expenses related to metro construction in different countries.

Tunnelers' wages in Europe during the base comparison years were similar to those in the U.S. In some countries, such as Norway and Belgium, tunnelers' wages were as high as $500 to $600 per 40-hr. week, which was considerably higher than U.S. wages for similar working conditions. Furthermore, bonuses for European tunnelers, as a production incentive, frequently are included within the contract package. Such practice is commonplace in Britain, Belgium and Norway.

Crew sizes are smaller in Europe. For example, single track tunnels on the London Underground were driven with crews approximately half as large as those used for U.S. rapid transit tunnels. However, it must be recognized that the internal diameter of tunnels on the London Underground is only 3.8 meters as opposed to 5.5 meters for tunnels on U.S. systems.

Recommendations

An overview of existing tunneling practice shows several areas where a clearer understanding of alternative measures could lead to immediate improvements in some systems. Five areas, in particular, are emphasized in this paper. They include: 1) the size of metro structures, 2) elimination of redundant support, 3) specialty methods of construction, 4) consolidation of technical judgments and services, and 5) reduction of risk. Recommendations are made for each of these areas under the following headings:

1. Size of Metro Structures: Metro stations are a key item contributing to the cost of a given system. In Europe metro stations account for between 40 and 85 percent of the construction cost of an underground line. In the United States it is not uncommon for individual stations to cost in excess of $30 million. Estimates by O'Neil at al [7] have shown that station expense tends to increase linearly by as much as 25 percent as the station length is increased from 122 to 183 meters for underground work in the United States.

   On a comparative basis, metro stations in the United States are large. For example, the station platforms on

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[7] R. S. O'Neil, J. S. Worrell, P. Hopkinson, and R. H. Henderson (1977), "Study of Subway Station Design and Construction", Report No. UMTA MA-06-0025-77-6, prepared for U. S. Department of Transportation, Urban Mass Transportation Administration, Washington, D. C. 20590.

the Atlanta and Washington, D.C. Metros are 183 meters long. In contrast, station platforms on the London Underground and Paris Metro are approximately 131 and 105 meters long, respectively. If the comparison is extended to areas of similar but smaller population, platform lengths of 137 meters for the Baltimore Metro are considerably larger than the 95 meters long platforms used on the Brussels and Tyne and Wear Metros.

Platform length affects the hourly passenger capacity of a system, which also is strongly influenced by the headway, or time interval between trains. Even when allowances are made for the shorter headways on some European metros, the line-haul capacities of recent U.S. metros are large with respect to those of European metros servicing areas of similar population density. There is a crucial link between initial system planning and large station size. Judgments made at the planning stage regarding peak demand, service level, and station architecture will have substantial ramifications in cost.

Occasionally, architectural concepts may call for large openings to create a monumental effect. Under these circumstances, expansion in scale might be limited to a small number of stations so that the bulk of the line is constructed with smaller, less costly stations, while the system can still benefit from the "showcase" structures that are available from all parts of the system. Furthermore it should be recognized that lighting and color often can be used in lieu of increased size to create similar effects.

A comparative analysis of tunneling practice points to the need at the planning stage for a greater appreciation of how size variation will affect system cost.

2. Elimination of Redundant Support: Cut-and-cover tunnels are frequently supported on a temporary basis with more sturdy pile-and-lagging or sheet pile walls. In many instances, concrete diaphragm walls are used for temporary support primarily to take advantage of their inherent stiffness and the associated relaxation of dewatering requirements. When the permanent underground structure is built within temporary bracing, the cost of the project must absorb two systems of support. Recent design practice in Europe, as is evidenced by the Brussels Metro, London Underground and Tyne and Wear Metro, have used concrete diaphragm walls as the permanent walls of the structure. The savings in expense associated with this practice recommends it for other projects.

3. Specialty Methods of Construction: In Europe, certain specialty methods of construction are widely used. In particular, the development of and experience gained with soil grouting and diaphragm wall construction recommend these methods for consideration elsewhere. Although the use of a specialty technique does not guarantee savings in cost, familiarity with and willingness to use the methods do provide a greater baseline of options with which to approach different tunneling problems.

4. Consolidation of Technical Judgments and Services: The complexity of large-scale urban construction points toward consolidation of services rather than greater di-

versity. Often, conservatism is amplified by separating responsibilities. For example, when section design and construction management are performed by different agencies, the design engineer must deal with uncertainties in site inspection and construction quality over which he has no direct control. His assumptions, correspondingly, are likely to reflect these uncertainties in an effort to cover a wide range of contingencies and protect himself from future litigation.

Underground construction requires judgments of a technical nature for which there are no adequate substitutes in managerial or legal consultation. The work for urban underground projects must be organized so that engineering decisions can be made swiftly and with authority.

5. Reduction of Risk: Underground construction carries an inherent uncertainty about the ground conditions that, by nature, generates a higher level of risk for tunneling than for other civil engineering enterprises. Furthermore, the risks are amplified in urban settings owing to the potential for damage or disruption to buildings and public facilities. Engineers who make judgments under these circumstances are vulnerable not only for their own errors and omissions, but are vulnerable for the mistakes made by others. Under conditions of vigorous legal activity, engineers may be named as third parties in law suits initiated by either the owner or the contractor. It is not surprising, therefore, that engineering judgments are often deeply conservative and that responsibilities for decisions are distributed through various review and regulatory channels.

Reducing conservatism and encouraging new construction techniques may require changes in the traditional procedures for assigning risk and awarding contracts.

European metro authorities award contracts on a selective basis by either prequalifying bidders or by letting contracts without being obligated to the lowest bidder. It seems reasonable, given the immense emphasis on experience for specialty construction methods, that relatively strict controls on contract award should be exercised when techniques such as concrete diaphragm wall construction or soil grouting are among the options for a given underground project. Furthermore, greater flexibility in negotiating contract arrangements for changed ground conditions or quantity variations would help to offset claims and recurrent high bidding.

In some instances, it may be appropriate for the owner to share the risk of introducing new construction methods or making cost-saving modifications in design. Federal government agencies and municipal authorities are often in an advantageous position to share the financial risk of an engineering judgment, particularly if the judgment results in diminished expense.

Acknowledgements

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# RESEARCH AND DEVELOPMENT PROGRAMS - AN OWNER'S VIEWPOINT

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The Atlanta Research Chamber is representative of research and development projects in general. They can be of great value in the development of new design methods and/or construction techniques. However, if the practical value of a test is to be optimized, it must be oriented with consideration for the needs and wants of affected parties.

The concerns of the owner of an underground public facility are, in order:

1. Is it safe and will it continue to be so?
2. Does it adequately perform the function for which it was intended?
3. Was it purchased for a reasonable price?

All of these concerns are directly related to the design and/or construction processes themselves, and, because of this, future owners of underground public facilities want those facilities to reflect prudently most current design and construction concepts.

Unfortunately, the growth of the scope of innovation in underground design and construction has been much slower than has been the increase of knowledge in some other areas. This can both reasonably and naturally be attributed to the following:

1. A structural failure could be singularly catastrophic. Consequently, more conservative designs are favored, quite often at the expense of innovative approaches.

2. A collapse during construction could have tragic results. This acknowledged fact presses constructors into utilizing "tried and true" methods.

3. Natural conditions and materials can be measured, calibrated and assessed, but the resultant findings cannot be applied to either design or construction without a considerable allowance for variation.

An examination of the owner's primary concerns in view of the historic considerations for the design and construction of underground facilities indicates concurrence in most areas. It is only in the area of economy that the owner's concerns are not echoed by traditional considerations. However, it is in the realms of design and construction economy that the owner is most vulnerable.

A qualified engineering firm will address the needs and wants of an owner-client with consideration for good design practice. It will do this with concern for both structural requirements and economy. However, the bases for its engineering decisions will be its own past design experience or the past design experience of other engineering firms in which it has confidence. Generally, the owner-client is comfortable with this procedure, and he almost always receives a design which meets requirements and is both safe and functional. With his two major concerns well covered, the owner-client assumes that economy has also been addressed in the design, and he is usually not in a position to question the extent to which this has been done. However, were an engineering firm to be deficient in any aspect of its professional effort, this aspect would most probably be the financial feasibility of its design. The engineering firm is highly motivated to produce a design which is inherently safe and which can be constructed with safety. It has both a liability and reputation at stake in these areas. It must address the functional requirements of design expressed by the client, or it will not receive compensation. However, in the area of design economics, its concern is less direct; furthermore, the extent to which it does address this aspect is difficult to discern in view of the following:

1. Because the contract will be awarded to a low bidder, the engineering firm cannot be assured of a level of construction competence. This point can be used to justify designs which are more simple but also more expensive to construct.

2. Because American construction practice is very competitive between client and contractor in terms of claims and legal proceedings, the engineering firm may address this aspect in its design. It might then be argued that the contract documents reflect a design, which is somewhat more expensive to construct but more legally defensible.

3. Because natural conditions are never the same, any criticism of a given structural design will in the end evolve to comparative evaluations of geology. This area can be quite subjective.

The management of a qualified construction firm will wish to do a competent job at a profit. However, to get the job they must be low bidders. Their expertise in a given manner of construction is, of course, their natural "edge" in the bidding process; however, if they are to be competitive, they must also utilize the tactics practiced by their fellow-bidders. One practice involves reviewing the contract documents in advance of bidding to determine bid quantities which appear either too low or too high. The unit prices for the apparently low quantity items are then exaggerated upward while those for the apparently high quantity items are exaggerated downward. Consequently, while the bidder's over-all bid price does not change, he will benefit if his analysis of the bid quantities proves to be correct. How much the actual cost of the job is increased by this ploy is moot. However, it does have the effect of highlighting some construction costs in an unreal manner with the net result of obscuring the cost effectiveness viability of given design or construction practices.

As can be seen from the preceding, there are too many affecting factors to permit ready evaluation of the cost effectiveness of a design or of construction practice in the field. Instances of simple evaluations are as dramatic as they are rare. It is generally necessary that a new concept in design or a newly introduced construction practice or manner of hardware be comparatively evaluated under conditions of considerable isolation. The Atlanta Research Chamber south of Peachtree Center Station does provide this situation.

The results of the test at the Research Chamber will be applicable in different ways and under different circumstances. A summary follows:

1. <u>Tests Used to Aid in the Development of Design Methods</u>
   Tests performed to corroborate, calibrate and/or determine needs for modification to the rock Finite Element Method (FEM) programs are of value in the development of this design tool. However, while the FEM design proce-

dure may end up reducing construction costs, it will not of itself be a part of any given contract documents nor will its use be restricted to any one contract. Consequently, the improvement of the programs must be considered as a general benefit to design without direct effects in the field.

2. Materials and Materials Applications Tests
Tests on installation and performance of materials and hardware are done with the intention of either corroborating existing design information or developing design information, which has previously not been available. The results of this testing will have industry-wide implications and also will be directly reflected in appropriate contract documents.

3. Construction Methods Testings
Tests of possible methods of construction will furnish information for future construction where these methods might be applicable. However, in most instances the methods of construction would not be cited in the documents specifically. Rather, the performance that the method is capable of achieving would be specified, and information concerning the method would be available to the contractor, but not as a part of the contract documents.

It is MARTA's intention to utilize the research information gained from the Atlanta Research Chamber to the greatest extent possible in the continuing development of MARTA's Atlanta Rapid Transit Project. Necessarily, this utilization will be done with full consideration for practicalities. Some of these are:

1. Contracting Practices

While it is conceivable that some test results might imply that final design of support structures could be done during the course of construction in order to obtain an optimum design (as is done in the New Austrian Tunnel Method, or NATM), this will not be done. Such a practice would be contrary to many government criteria currently used as guidelines and, consequently, the contract documents would probably be unacceptable. Additionally, in the American contractual climate such a practice would amount to "construction by change order" rendering the owner very vulnerable during negotiations. In view of this, while such a procedure might in effect produce the least expensive support system, it is very doubtful that any of the savings would accrue to the owner. More likely, the contractor, due to his superior negotiating position, would increase his profits to an amount in excess of the savings.

The contract documents will, generally, be drawn up in such a manner as to minimize change orders and make "bid unbalancing" of minimum value to the contractor. However, MARTA will open-mindedly entertain "value engineering proposals" by the contractor during construction. It is felt that in this manner, under American contracting practices, the value received will be optimized in view of the amount spent.

2. Construction Capability

To require that a specific construction procedure be utilized where there is very limited expertise available in that field would, generally, be undesirable. Under such conditions, it is very possible that either the party offering such a service would price himself too highly or a non-qualified party would attempt the procedure with a bad result.

It is MARTA's intention to make readily available to the greatest extent possible required expertise for construction. However, in some instances there may be cases where a selected expertise, even at a premium price, would be cost effective; they would have to be evaluated on an individual basis.

More generally, it is MARTA's intention to offer minimal restriction to the contractor's own innovation. Not only will this restraint on our part tend to reduce the number of change orders, but it should have a positive effect on productivity as well.

3. Construction Liability

One way or another, MARTA will feel liability for its own actions, those of its retained engineers and those of its construction contractors. It is its intention to minimize this over-all liability. Consequently, when MARTA's construction activities are going to impact an outside party, the manner of execution for those activities would be viewed with a concern for liability exposure. Because of this, it is conceivable that a new or rarely used procedure would be rejected and a more conventional and expensive procedure used in its place.

In conclusion, it is very difficult to assess the cost effectiveness of innovations in design or construction in the field. This is because design and contractual climates are complex; they have aspects which tend to obscure the relative performance of a new design concept or construction technique. Consequently, if an innovation in design or construction is to be properly evaluated, it must be done under conditions of considerable isolation. However, should the results of testing confirm the process' viability, it cannot be readily introduced into the design or construction field. Many practical considerations require that the approach to this implementation be a conservative one. However, this same conservatism tends to assure an adopted procedure's success.

# IMPLEMENTATION OF RAPID TRANSIT SYSTEMS - AN OWNER'S VIEW

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Because of great increases in rapid transit system costs over the past several years, and some startling project cost estimate increases which have hit UMTA midway in current projects, UMTA has notified us that we must be prepared to construct a project in Los Angeles within the cost estimate we come up with at the conclusion of our preliminary engineering phase. They will only allow escalation.

Los Angeles, which is the first city to develop and publish jointly with UMTA a combined Draft Alternatives Analysis/EIS/EIR, is also going to be the first to operate under this new policy.

Obviously, therefore, it will be necessary for us to go to much greater depth in preliminary engineering than is normal in subsurface exploration, in the making of trade-off analyses between those alternative materials, equipment and methods for all civil work, as well as for all of the various subsystems that make up a rapid transit system.

It appears that UMTA, which has thus far gone along with the high costs incurred by other U.S. transit properties in comparison with many of their foreign counterparts (so much so that congressional funding appropriations have not gone nearly as far as originally planned), intends to "turn over a new leaf" on the Los Angeles project - which we are now told will have to wait for the next major appropriations bill. However, we happen to concur that such an effort is necessary and overdue. It apears that the reason this did not happen sooner is that (as has been the case with several "owners"), the UMTA hasn't had the personnel needed to concentrate on the problem.

So, we intend to explore all means that appear to have reasonable potential for enabling us to get a quality job done at less cost. That may even mean it will be necessary for the owner to take a few risks! In England, the owner takes all the risks. In the U.S., the owner tries to give all the risks to the designer and contractor. In France, I understand, the risks are negotiated.

While some of the increased costs may be due to over-design, excessive project size and the cost of automation, a very appreciable percentage can be attributed to a combination of bureaucratic regulations, environmental requirements and delays, hesitancy on the part of all parties to take risks (in these days of outlandish judicial decisions and jury awards), and higher labor, material and equipment costs.

Among the above-mentioned factors, our most likely hopes for economizing appear to lie in the area of risk-sharing, and in making methodological material and equipment trade-off analyses during the preliminary engineering process. After all, it is in preliminary engineering that key decisions are made that have the greatest impact on project costs.

Unlike the designers and contractors, the owners have to live with the finished product. This obviously results in different points of view about a project. Many of the facets of a project to which owners must pay particular and continuing attention are, for the designers and contractors, merely just "things to have done with as quickly as possible".

The owner's primary concerns are: addressing the problems involved in getting the job funded and approved by the community; meeting all state and federal regulations; and getting fast decisions from UMTA; and, in addition, considering the following for the project:

Design and operational criteria
Community disruption
Environmental Impacts
Quick completion
Extent of risks
Safety
Reliability
Maintainability
Durability.

The designer's and contractor's primary concerns after getting the job, in addition to getting rapid decisions from the owner, are:

Doing the job within prescribed time limit
Being compensated for changed conditions
Doing a quality job which adds to the firm's reputation
Minimizing risks
Making money on the job ("the bottom line").

The civil works are usually sound and solid, but have been too expensive because of a combination of the owner's grandiose ideas about stations and trains with a lack of full appreciation of costs. Also, there is tension between the not uncommon tendency on the part of the designer to think that the bigger the size of the transit system and the greater its degree and complexity of automation, the better; and his strong reluctance to take risks (which is understandable in this day and age). It is too much to expect designers to operate on the basis of "small and simple is beautiful" when owners do not.

From the operations standpoint, however, the trouble lurks in the many and varied sub-systems. And, it seems to work out that the greater the amount of automation, the greater the trouble. Designers just cannot be experts in all the technology involved in all of the sub-systems. And, of course, because they do not have the responsibility for operating and maintaining the system, they just don't have the same degree of appreciation for the importance of these aspects of a rapid transit project as for construction. This means that owners will do well to call in specialists for advice in many areas.

Preliminary engineering is the phase during which most key policy and design decisions are made which have the greatest effect on project costs. To date this fact does not seem to have had proper recognition. Los Angeles will be the first to approach the matter on this basis. Sure, preliminary engineering will cost more and take more time, but final design will then cost less and take less time!

# SHOTCRETE AND ROCK BOLTS: LABOR'S VIEW

by
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I have been somewhat disturbed at reports that tunnel contractors and contracting agencies (owners) believe that labor will not or, in most instances does not like to, work with rock bolts or shotcrete in tunnels. That simply is not true. Our concern is with the methods and procedures of usage and the adequacy of equipment for safety in application.

We believe that, generally, if rock bolts are going to be the total support, and a lot of bolts are going in, a rock bolt jumbo should be on hand to drill the holes and to put the bolts in. A normal drill jumbo for drilling and blasting the face is not normally very good for rock bolting. The main reason for this is that if the rock is blocky or badly fractured, we like to be back some distance from where the holes are being drilled, in order to be safe from any rock that might fall.

Rock bolts can support a tunnel; the diameter of the bolts selected depends on the size of the tunnel, and the length of the bolts depends on the area to be supported. I firmly believe that, if rock bolts are going to be the total support, pattern bolting must be used -- rather than just a bolt here or there, where some shifter or walker or someone else says there should be one. Once the pattern is laid out or agreed upon, it must be strictly followed, unless or until the rock or the situation changes. What bothers me most in rock bolting is that if for some reason it's hard to drill for a bolt, it is left out entirely, or located out of place. We demand that the correct bolt be put in the pattern at the right spot. Without a bonafide rock bolt jumbo, it is sometimes impossible to put the bolt exactly where it belongs.

Our experience in tunnels where we have used rock bolts is that they have not been placed in a regular pattern. Instead, they are put where some supervisor said they should go, who has not taken time to place them in the best locations. Often they are simply left out.

We know that most kinds of rock can be bolted for support, if the bolts are long enough and thick enough, and someone is there to be sure they all go where they are supposed to go on.

I was told about twenty years ago, by a Snowy Mountain Authority engineer in New South Wales, Australia, that a bucket of marbles could be rock bolted. I don't know if I can go that far, but I do know that rock bolts can be used effectively in most situations.

I do think that in certain types of rock, or ground as we would say, it is important to use wire mesh in conjunction with rock bolts to keep the fine rocks from falling, especially in a large tunnel where the small stuff would be falling a great distance.

I have been in the tunnel and mining business all my working life. The only reason we prefer timbered steel sets is that we have become accustomed to them. We are not afraid of rock bolts and don't mind at all using them if we have the right equipment and if pattern procedures are strictly followed. We don't like to attempt to put them in from a drill-and-shoot jumbo - it's unhandy to say the least, in addition to the other reasons given here.

I recently visited a job in Richland, Washington of three tunnels within short range of each other, the smallest only 8 feet diameter, one 14 feet diameter and the other 23 feet diameter. I was called because the miners, members of the Laborers' International Union, had refused to rock bolt in both the 14-foot and the 23-foot tunnels, saying that in their opinion it was simply too dangerous. I will have to admit that I agreed. This was what they call basalt rock, and it was the blockiest and most fractured rock that I have seen anywhere. In the small tunnel (8') a shield had been constructed of heavy steel at each end; it was then laced with 2-inch pipe and taken into the tunnel with a backhoe boom to set it up: the miners were supposed to drill the rock bolt holes from underneath this contraption. The 2-inch pipe was far enough apart to let large rocks fall on the drillers. Furthermore, the rock was so jagged it was impossible to drill effectively for pattern bolting.

Given the nature of the rock and the way labor had to do the job, the men were correct. They have since shut down the job to renegotiate with the contractor for other support. It could perhaps have been rock bolted if they had had the correct equipment, even under this rock.

## Shotcrete

About the only thing we don't like about shotcrete is the dust that it creates. If put on at the correct time and in sufficient quantity, we know it's all right, and we don't mind it at all, except for the dust that goes with it.

I believe it's most important in ground that will air slack to put it on as soon as possible after blasting, at least a couple of inches thick, to keep the air away before it starts to slack. I observed a railroad tunnel under construction by Northern Construction Company about thirteen years back where they were going up an old river bed -- just small rocks, gravel and sand. They were shooting a 10-foot round; and, immediately after blasting, (using as little powder as possible, shooting very lightly because it was easy to displace), they put two inches of shotcrete from the drill jumbo using a Gantry jumbo. They were mucking with a Conway mucker at the same time. After finishing the two inches on the most recently blasted round, they put two more inches on the previous round (making four inches on that one, which was in the area 10-20 feet from the face) and two more inches on the earlier round, 20-30 feet from the face (totalling six inches of shotcrete there). No failures occurred from the portal to the face. They were in something over 3000 feet when I visited the job. It was a beautiful job. They were totally set up for shotcreting. It worked correctly for a tunnel this size and condition. The job was just outside Vancouver, British Columbia. This job sold me on shotcreting completely, <u>if applied at the right time and in sufficient quantity</u>! It's most important, in my opinion, to apply the shotcrete from some kind of conveyance that straddles the muckpile, so one can be mucking at the same time. It's easy to do this from a Gantry jumbo, if you are on rails, but difficult when the job is on rubber and it takes all the room for mucking.

We aren't afraid of rock bolts or shotcrete. We know that both are here to stay. What we require is a reasonably safe place to work. Under safe conditions, we are prepared to work with rock bolts and/or shotcrete. I will sign off by saying that we are most at ease when rock bolts and shotcrete are used together. This takes care of both the large and small rocks that may come down.

## TUNNEL DRILLING AND VENTILATION EQUIPMENT

by
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### DRILLING EQUIPMENT

Not many years ago, the problem of selecting drilling equipment was comparatively simple. The drills were all operated by compressed air, and the machines were selected to suit:

1. Character of the material to be drilled;
2. Cross-section dimensions of the tunnel.

Knowing these two factors, heavy drifters to lighter ones mounted on jumbos, or jacklegs, or spaders were decided upon. On certain jobs special machines for soft ground, augering drills and electric machines were sometimes used. The majority of jobs particularly required the use of rock drills. Today, we have tunnel boring machines (TBM), back-hoe design mining units and, now, the latest in equipment - hydraulic drills.

When the ground conditions are known; and if the material is uniform for the total length; and if the tunnel is to be straight and very long; then, boring machines are highly favored, provided the rock is not too hard.

The other choice (assuming the ground isn't spader ground), is between the conventional compressed air-driven drills and the recently developed hydraulic drills. Both of these designs can be used for the same duty. A good rule of thumb for comparison is that the hydraulic drill will cost twice as much as the comparable pneumatic drill and will, in most cases, drill twice as fast. The hydraulic drill is quieter and larger. It is also a very sophisticated product which requires a clean operation and emphasis on preventive maintenance.

Obviously the performance of the hydraulic drill depends on the operators' experience and knowledge of hydraulics. At this writing, on isolated jobs, it seems logical to stay with the air-operated drill. This may change as time goes on and the miners become familiar with hydraulics.

The use of either the hydraulic drill or the pneumatic drill (both classified as conventional methods) affords great flexibility in the case of changing ground conditions, whereas the boring machine is without insurance in case of ground formation changes; and, of course, with the boring machine all your eggs are in one basket, so trouble anywhere means the machine is totally down.

Every tunnel project is its own problem. The engineers of the qualified tunnel contractor finalize a job which in every case has the following basic problems relative to the equipment:

1. Initial cost and interest in investment
2. Mobilization cost and time involved
3. Salvage value (can it be used again)
4. Quality, back-up service and parts availability.

Several specific projects in California are using both hydraulic and pneumatic drills. Each seems to have its place, particularly when the project is one having various size tunnels and diverse job conditions.

With improvements continously being made in both the hydraulic and pneumatic drills, it is difficult to predict which will be more popular. We do feel, however, that the dependability and the flexibility of conventional drilling equipment will always rate high in the mind of the experienced contractor.

## VENTILATION EQUIPMENT

Since the days of the huge, noisy, reversible blower installed at the portal, great progress has been made not only to improve the air conditions underground, but to reduce the power costs and noise level.

This has been done by using in-line blade-type blowers and a straight exhaust system. This system starts with a suitable blower (fan) at the portals to ventilate, for example, 2000 lineal feet of tunnel. Similar sized blowers are installed to take care of each additional 2000 feet.

The overall system should have a rock trap and a nose screen at the heading end of the pipe. Reversible starters are sometimes specified for certain emergency situations. Explosion-proof motors and starters for use under gaseous conditions are required.

Another money-saving innovation has been the fabrication of the fan line on jobsite. This saves the expensive transportation cost of prefabricated pipe, which for pipe alone costs more per foot than the pipe rolled on the job.

It has also been interesting to learn by test that pipe rolled on the job with a groove every three inches actually has a lower coefficient of friction than smooth pipe; at the same time, grooving imparts to lightweight pipe the stiffness of material 2 to 4 gauges higher.

Pipe of this design used for transporting wood chips (via air) in lumber mills has a very long life; the vortex effect of the spiraling of the air-wood chip mixture on the inside of the pipe reduces the abrasive action of the chips.

A perfect ventilation system provides good air to the personnel underground without excessive air velocity in the tunnel. This is readily obtained by the exhaust system.

Some unreasonable rules have been set up which cost the contractor more, and of course, are ultimately costly to everyone. Using the top BHP rating (according to one rule) of the diesel units underground brings the total required volume of air to the point of uncomfortable velocity in the tunnel. Hard hats will have to be designed with chin straps!

This rule is wrong because the average BHP used is seldom over 50% of the top rating; also the units are not in the tunnel 100% of the time.

All in all the straight exhaust system with in-the-line blowers is so satisfactory that many times you hear the comment "air in the tunnel seems cleaner than the air outside".

Editor's Note

This article is based on George Philpott, Sr.'s fifty years experience with drilling and ventilation equipment. Geographically, his experience has been in the Western Hemisphere, Pacific and Far East areas, Europe and Africa.

# CONTRACTOR'S VIEW: INNOVATIVE CONTRACTORS CAN MEAN LIGHT AT THE BEGINNING OF THE TUNNEL - A CASE STUDY

Alf Burtleson, President
Alf Burtleson Construction Co.
Tunnel Contractor
Sebastopol, California

## Introduction

Owners, engineers and labor should allow innovations by contractors because if it is the contractor's idea, he will make it work by hook or crook or go broke trying.

A case study is the soft ground tunnels on the County of Sacramento, California wastewater project on the Pioneer Interceptor. The tunnels were constructed by our firm, Alf Burtleson Construction Co.

The tunnels were 4.4 meters in diameter and were specified at the Interstate 5 Freeway, the Southern Pacific Railroad crossing and at Front Street (see Figure 1). This diameter tunnel is just slightly smaller than most rapid transit tunnels in Europe.

## Geology, Groundwater and Previous Construction in Area

All of the tunnels were in one general area and were separated by short lengths of open cut. This area is immediately adjacent to the Sacramento River and is located geologically in the same stream channel deposit stratas. The strata at the tunnel elevation was generally silty/sand with sand and gravel underneath.

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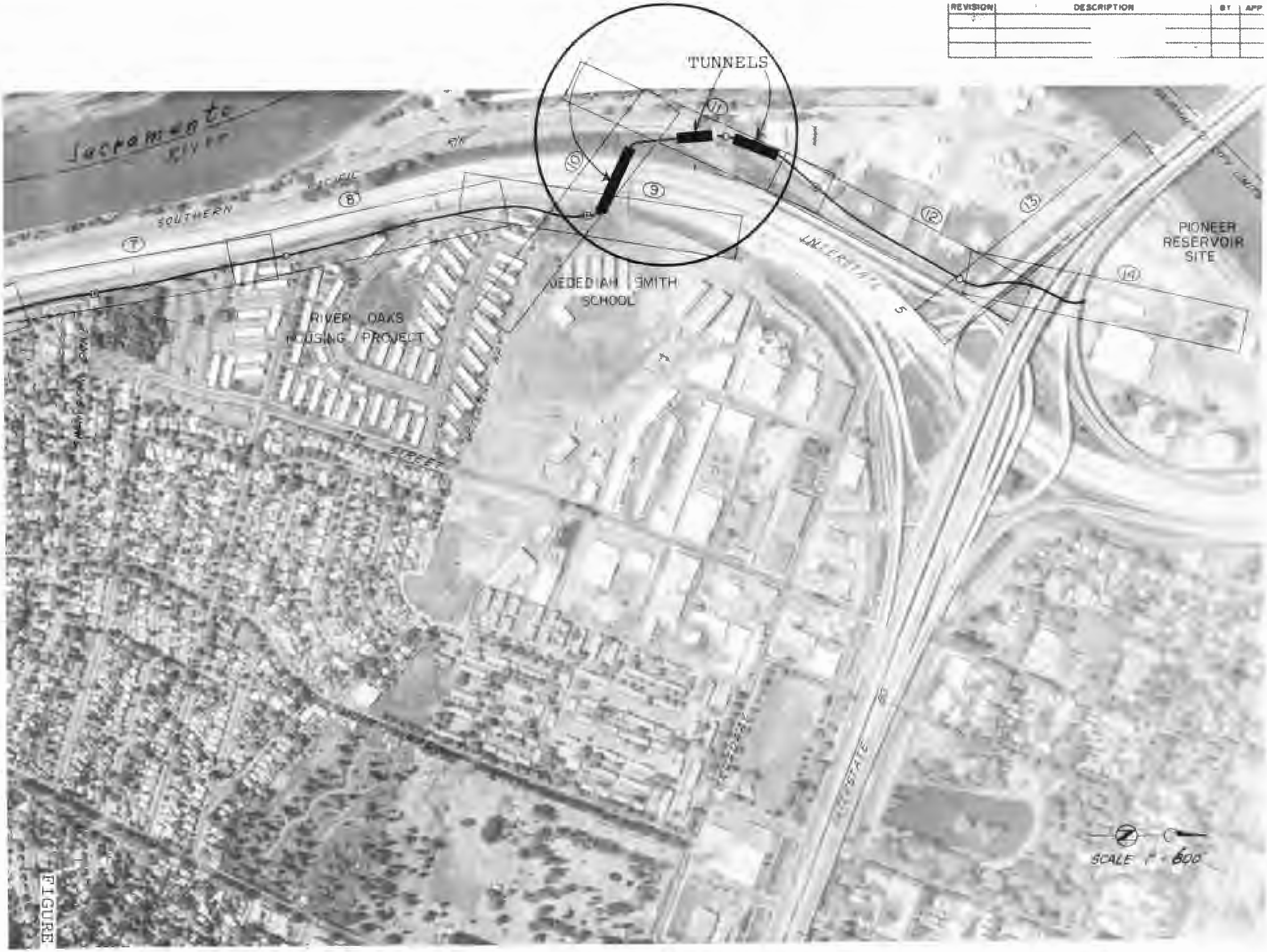



FIGURE 1

SACRAMENTO CALIFORNIA

PHOTO DATE: 10-21-76

The groundwater flow was artesian in nature. Because of the coarser material underneath, the flow tended to be vertical instead of horizontal.

There had been two sewer construction projects in the area in the past several years and both ran into groundwater control difficulties. The first project just fought the groundwater and the pipe ended up in washboarded (up and down) condition because of the heaving invert from artesian groundwater. The second project relied on the experience of the first project and the owner specified a wellpoint groundwater control method. This method did not work, perhaps because, as previously stated, the groundwater flow tended to be vertical instead of horizontal; and again the pipe ended up washboarded. (I believe that these projects are still in litigation.)

## Bidding Conditions

When our case study project was bid in November of 1977 there had been a two-year drought in California, so that the groundwater level was just below the invert of the tunnel. The winter that followed, however, was one of the wettest ever, causing the Sacramento River to reach flood stage (which, of course, recharged the groundwater table).

The original specifications called for a conventional liner plate-style of tunnel construction and required that the groundwater be maintained below the tunnel invert but not more than one meter below, to prevent settlement of the adjacent freeway.

The groundwater was successfully controlled with deep wells 25 meters deep tapping into the coarser stratas. The construction was planned from May to October during the low flow of the Sacramento River. The deep wells drew down the recharged groundwater table and were able to maintain the groundwater one meter below tunnel invert because the river was not recharging

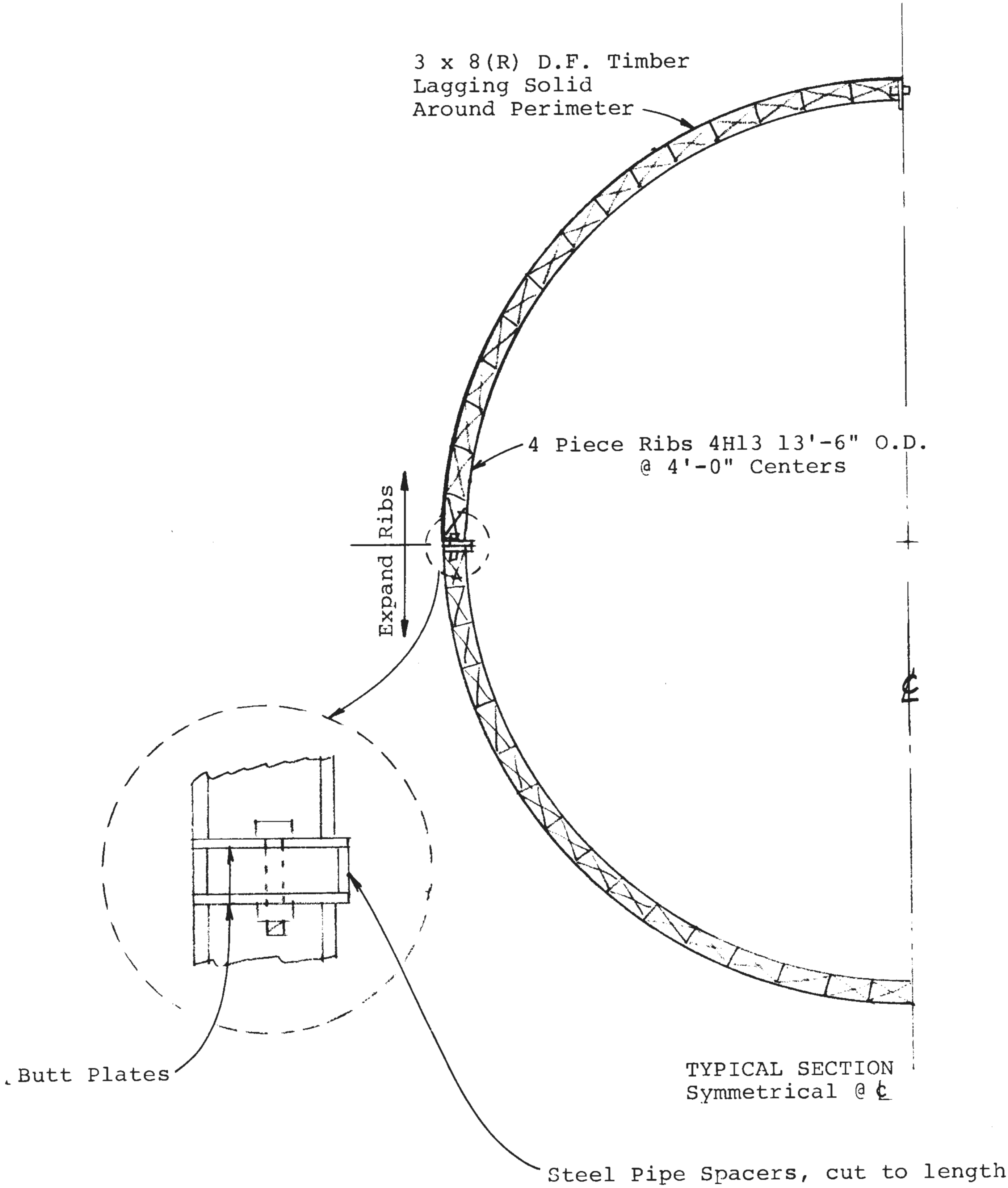

3 x 8(R) D.F. Timber
Lagging Solid
Around Perimeter
4 Piece Ribs 4H13 13'-6" O.D.
@ 4'-0" Centers
Expand Ribs
Butt Plates
TYPICAL SECTION
Symmetrical @ ℄
Steel Pipe Spacers, cut to length

FIGURE 2

the groundwater table. The pumping was accomplished at just a few locations, but many observation wells were installed to monitor the groundwater levels. The exact level was maintained by slowly lowering the pump in the deep wells; when the desired groundwater level was obtained, the pump was then throttled down to maintain that groundwater level.

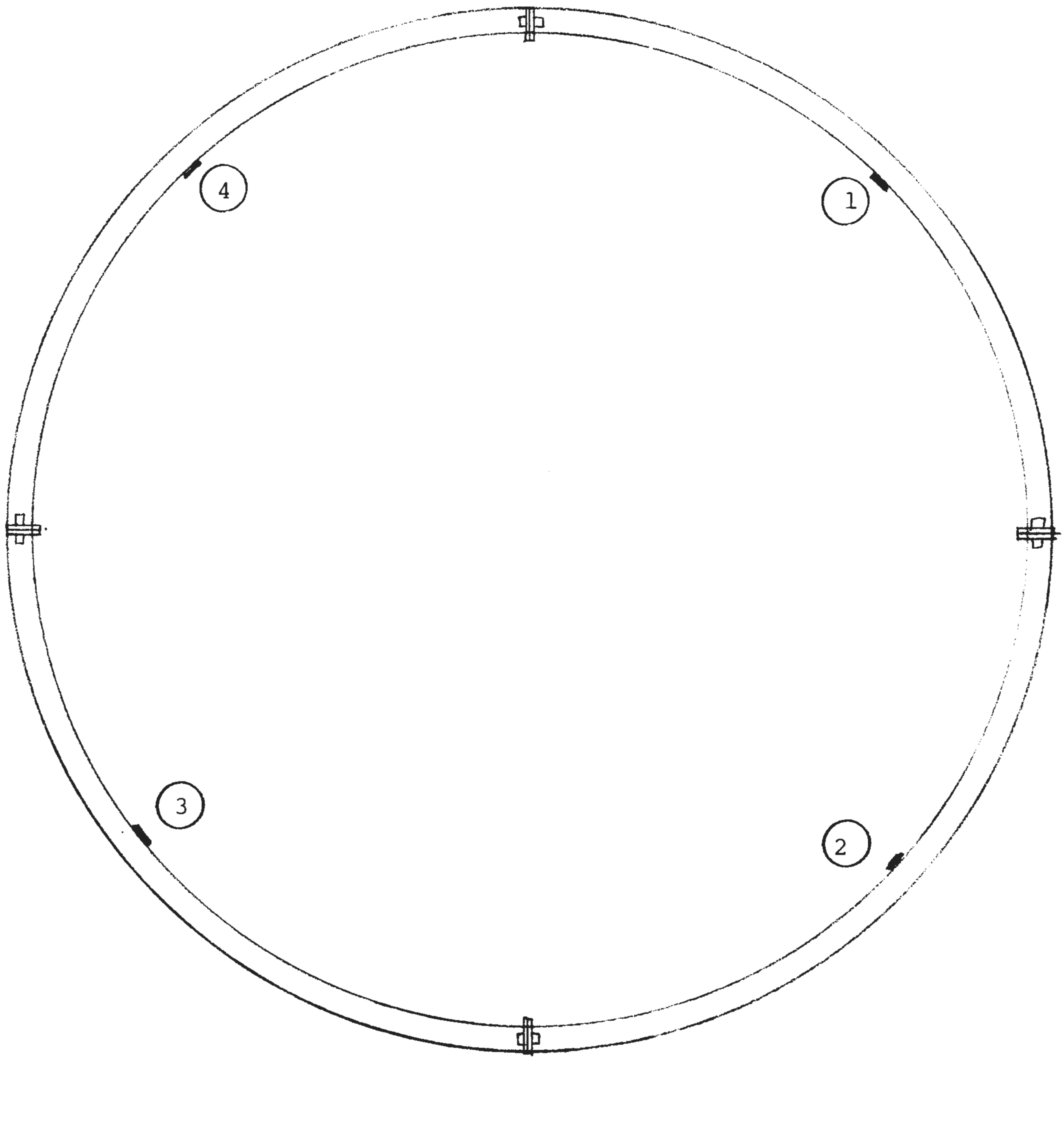
## The Tunnel Problem

The critical area for tunnelling was under Interstate 5 Freeway, a 12-lane freeway immediately adjacent to Interstate 80 Freeway in Sacramento.

The conventional liner plate style of construction originally specified, requires that the ground around the tunnel stand long enough so that sand-cement grout can be pumped into the annular space between the earth and the tunnel liner plates. It takes 6 to 8 hours from the time you decide to grout to the time when the grout is strong enough to support the adjacent ground (about 75 psi; i.e., 220 $kg/cm^2$). If you cannot fill this void before the ground collapses onto the tunnel liner plates then settlements will occur on the freeway above.

It was our opinion, based upon our experience, that the ground would not stand long enough for us to be able to fill the annular void with grout; we envisioned potential settlements on the freeway above.

We proposed a shield-driven tunnel (original specs did not require a shield) with the temporary support being 4H13 steel ribs at 1.3 meter centers with solid timber lagging installed between the flanges of the ribs (see Figure 2). This is a strong support system for the shield to push against in order to propel itself forward. As the rib came out of the tail of the shield, it was expanded with a specially built rib expander and pipe spacer blocks which were cut to fit and installed at the springline. This system provides almost an instantaneous support of




STRAIN GAGE LOCATIONS

FIGURE 3

the ground and is not subject to normal "stand-up time of the soil" as with the conventional grouted tunnel construction.

Approvals were secured from both the owner agency (County of Sacramento) and the owner of the freeway (California Department of Transportation) but were conditioned on strain gage instrumentation of the tunnel supports.

## Instrumentation

The instrumentation was accomplished by welding Slope Indicator Company vibrating wire strain gages onto the 4H13 steel ribs. This strain gage was selected because of its ease of installation and economics.

Five ribs were instrumented at midspan of each piece (see Figure 3). A four piece rib made up one circle, so a total of 20 strain gages were installed. The 5 ribs were chosen at different heights of cover (H) from one meter to 8.5 meters. The strain gages were read initially on the ribs, again after expanding the rib, then daily for one week, then weekly for one month.

## Strain Gage Results

Nineteen of the twenty strain gages installed worked, and there was good correlation from day to day. Of the nineteen gages, eighteen indicated stress in the steel rib of 14,630 $kg/cm^2$ (5,000 psi). See Figure 4 for a graphical plot of the findings. The loads and stresses reached equilibrium after about seven days.

## Conclusion

This tunnelling system worked. The settlements measured on the freeway above were less than three centimeters, and the loads experienced on the tunnel supports were a small percentage of the normal design loads. The design criteria used were AASHO "Design Specifications for Tunnel Liner Plates" which takes into

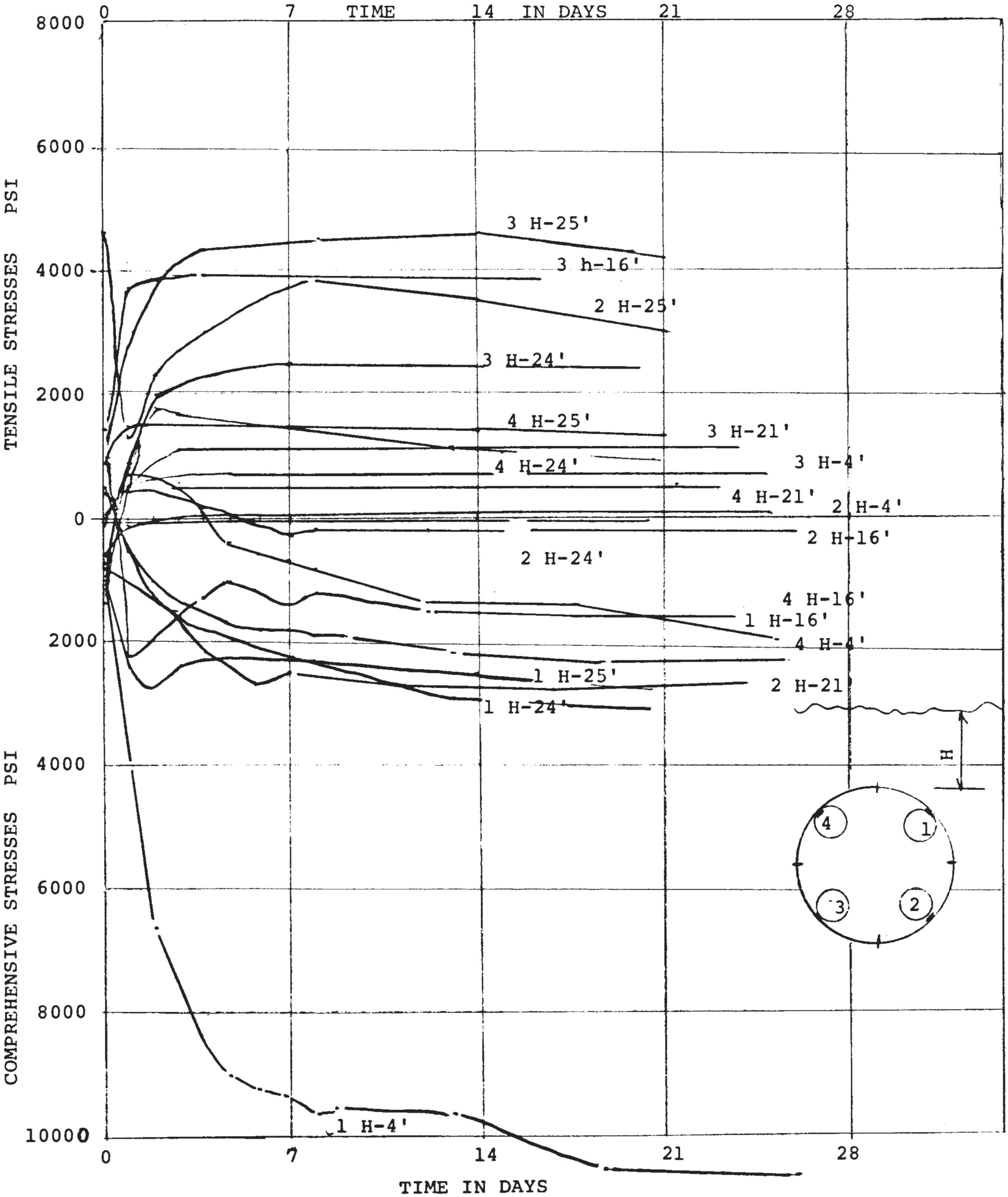



FIGURE 4

PLOT OF STRESSES IN STEEL RIB VS. TIME

account the ratio of fill to tunnel diameter and type of soil. Then Marston's formula Pd=Cd WD was used. We anticipated getting loads which would produce steel stresses of up to 24,000 psi but the actual stresses were only about 20 percent of the design stresses.

The shield method of construction also provides safety for the tunnel workers, because they install the supports back from the face under the protection of the shield.

The tunnel method described is for temporary support. Upon completion, the permanent reinforced concrete pipe was installed through the temporary support, and the remaining annular space between the supports and the reinforced concrete pipe was grouted with a pea gravel concrete grout.

## Conclusions

Innovative contractors can only be innovative if the other parties to the construction contract are receptive to new and better methods. On too many U.S. tunnel construction projects, the tunnel contractor is stuck following the specs in recipe fashion. Although no deviations are allowed, he is also blamed if the job does not turn out properly.

On this contract we received cooperation from the owner agencies previously mentioned, and also from the general contractor, Homer J. Olsen, Inc. and the dewatering subcontractor, Subgrade Construction, in order for our company to be successful with the expanded rib method.

Special thanks to Professor G. Wayne Clough of Stanford University are due for his aid in strain gauge selection.

TUNNEL PRACTICE IN THE UNITED STATES:
A CONTRACTOR'S VIEWPOINT

By J. R. Jensen, Geologist
Guy F. Atkinson Company
South San Francisco, California

There are many problem areas in the contracting business, but the one factor which most frequently seems critical is time; time to prepare the bid and time to construct the works. The length of time necessary to prepare a bid usually varies inversely to the completeness of the design. If the contractor is expected to design a feature, obviously with certain constraints, he expends many man-hours that could be better spent preparing the estimate. It is difficult to make an unqualified statement as to how much time is enough, but in general, if he has the documents in hand for six weeks and does not have to contend with addenda which represent significant changes in the scope of the work, he ought to able to put together a realistic bid.

If within the bid schedule itself, sophisticated or unique items are to be supplied or installed which are likely to be difficult to obtain, it is realistic to schedule additional time for them; other methods of handling such items in the bid schedule should be provided if time is of the essence. It is not unusual to issue preliminary information documents, and have a site showing well in advance of the issuance of the formal bid documents, thus in effect adding time to the bid period.

Site showings for tunnel work often reveal very little about underground conditions. The owner should have on display samples and examples of what he knows of the underground conditions and a knowledgeable representative available to explain salient features. It has been suggested that contractors might or should do exploratory drilling for tunnel bidding. If there is insufficient information to prepare an intelligent bid, one has to question what was used for design purposes and feasibility estimates. It is difficult for me to believe that a contractor could, in a very short period of time, acquire any meaningful information that would influence a bid. It is extremely unlikely that he could locate any problem areas with one or two drill holes even if there was time to make that much of an exploratory effort.

Another difficulty that contractors have to contend with is the time and timing of the viewing period; at the risk of being called soft I would like to point out that winter time is not the best for a job showing in spite of the fact that such timing usually is convenient for an early spring award. I have attempted to evaluate job conditions under adverse winter weather conditions and have rarely been satisfied with the results. The most satisfactory response to this situation has been a decision not to bid the work; submitting a successful bid under these circumstances has usually resulted in a sticky changed conditions claim.

It is the desire of all concerned to utilize a predictable approach; however, idealized conditions are not possible in this particular business. Therefore, the owner should accept the fact that he is venturing into a hazardous undertaking, and be prepared to accept the risk. Problems are inevitable when one party attempts to pass off the risk to the other party in the contractural game. If provisions are not made in the bid schedule to pay for the risk, the contractor is forced to evaluate the risk and include the cost in his bid as a calculated contingency. If the owner accepts the risk and makes provision to compensate the contractor, he will very likely get it at cost.

Cost for an item of work that cannot be adequately evaluated can be included in the schedule as a "Provisional Sum" which allows the contractor to price the remainder of the work realistically; the owner can then evaluate the bids on a comparitive basis. "Lump Sum" items require that the contractor absorb overruns no matter what the cause. He, therefore, must apply an uncalculable contingency, otherwise known as a guess, on the high side.

Contractors are accused of not being innovative and of being unwilling to try new approaches. I often wonder if the accusers are the same ones who write into the bid documents that the contractor shall assume all responsibility for the safety and function of the project. The only way a contractor can accept these conditions is to make use of the systems and schemes with which he is familiar and which fit into the allowable time frame. The owner must also be prepared to accept some of the responsibility for deviations from the tried and proven methods. A scheme like the "New Austrian Tunnel Method" offers some interesting ideas which are improvements over customary practice in the United States, but is not something which a contractor can afford to experiment with on a fixed price bid. All too often in attempting to utilize a new scheme such as this one, where admittedly everyone is in the learning stage, the owner sees fit to inspect and test every item over and over mainly for the purpose of learning just what to expect. I have seen too many delays resulting from vigorous application of the exact letter of the specification to want to attempt to use shotcrete as the primary means of support. The short time schedule usual for tunnel construction often forces the contractor into adopting the customary drill/shoot/set-steel routine as a matter of expediency. From experience we know that a crew can be trained to follow this routine and soon achieve a predictable rate of progress. The existence of the canopy of visible steel imparts a sense of security, needed or not, and the crew functions at an acceptable efficiency level. The use of another means of support, be it rock bolts or shotcrete, would in all probability provide the same amount of security at a lesser cost, but with the risk of work stoppages due to real or imagined unsafe conditions. The problem is to create a safe working environment for the tunnel crew using a system they feel comfortable with while employing a scheme the contractor can trust to produce results punctually. The contractor has reduced his financial risk by the use of a proven scheme and will continue to do so until the owner shows a willingness to share that risk.

Designers have gone to considerable effort to discourage the use of rock bolts. Some few years ago when expansion shell mechanical anchors were the state of the art, I was at a job where rock bolts were considered a desirable option if the contractor could demonstate that the anchor would hold during a pull-out test that would develop a stress considerably in excess of the working strength of the bolts specified. The rock was a highly metamorphosed gneiss and mica schist. Needless to say, anchors placed parallel to the planes of schistosity failed to meet the criteria, and the contractor was then required to install additional bolts at his expense. All suggestions that bolts placed parallel to the schistosity should not be required to achieve anchorage in excess to the working load of the bolt, or that smaller bolts placed in a closer pattern would achieve the necessary load-carrying requirements were rejected. In order to get on with the job and avoid unnecessary loss of time and expense for materials (non-reimbursed), the contractor chose to use the steel rib support option.

In a more recent incident involving epoxy resin anchor, an extensive test program was necessary to demonstrate the suitability of this product.

Data used in the proposal was obtained from a report on extensive testing done by another branch of the same agency. This redundant effort had to be financed from the contractor's indirect costs.

When unusual ground conditions are anticipated, the solution is to be very explicit in the specifications and be prepared to accept results, e.g. in the case of the long shell anchors failing in the schist, the contractor should accept this rock characteristic and work within the parameters present in that situation. Perhaps a larger number of smaller bolts could have maintained the same load capacity.

In the use of gunite and shotcrete, I have experienced the invention of the wheel on few occasions. The proportions necessary to achieve a proper mix are well established, and it seems redundant to have to repeat the process each time it is used. I have seen times when the process was done for the edification of the inspector, who then became the expert over a craftsman with years of experience; the inevitable conflict led to many disagreements. This happens to be a human problem that no specification writer can keep from happening. During early stages of the work, often more time is spent adjusting mix and making cylinders than is spent placing protective shotcrete in a productive manner. The unit price for placement does not approach the time and materials costs during the trial and error period. Adjustments in design which do not interfere with production are no problem, but redesign from "square one" is a different bid item.

A contract that requires mixed methods of support such as steel ribs for some sections and rock bolts for others can increase the cost of support, especially in large tunnels. The jumbo (a moveable scaffold) designed for steel placing will not necessarily be suitable for rock bolting; the latter may require that a heading be supplied with two (or more) specially designed pieces of equipment. Often these units will not pass through the tunnel and must be moved out, or to an enlarged passing area; they are not necessarily readily available for routine changes of support. If the sections of tunnel which will require a given type of support can be pinpointed and designated well in advance, no particular time will be lost. If, however decisions must be made on a day-to-day or round-by-round basis, the method becomes very unattractive, and a contractor prefers to use only one type of support. It is not easy to train miners to place rock bolts unless a constant pattern is maintained. Pattern bolting of course maximizes the carrying ability of rock bolts and is the only reliable way to use them.

It has been said that the competitive bid practice is a lottery in which the loser gets the job. So be it. The low bid-award practice does encourage the contractor to pursue claims in order to cover costs for unforeseen circumstances. A claim has the connotation of being something a contractor is trying to get that he has not earned and would in effect amount to surplus profit. Frivolous claims are easily spotted and as such should summarily be rejected by an owner. Claims that are justified should be thought of as adjustments reflecting the increased cost of doing work where, even after a thorough exploration, we had to use the "best guess" approach after having entered into a contract in good faith. An open-minded attitude towards claims removes some of the stigma from the terminology and puts the concept in a more favorable light.

If bids are to be rejected because they are some arbitrary percent over an estimate, these conditions should be clearly spelled out prior to asking for bids; the base estimate should be prepared by knowledgeable and experienced people. It is one thing to prepare an estimate if one is reasonably sure he will not himself be required to do the job at his price, but quite another if one considers that he will be expected to produce a finished project for his price.

In the December 14, 1978, issue of Engineering News Record, the Urban Mass Transit Administration suggested that bids 7 percent above estimates could be cause for rejection. If this type of unilateral action is adopted, it will certainly reduce contractor enthusiasm. It seems to me that any estimate could as easily be incorrect or unrealistic as any other. If all bids are considerably in excess of the engineer's estimate, there is reason to suspect that the specifications and drawings do not properly convey the owner's intentions. Rather than reject all bids and attempt to get a lower rebid, some means of negotiation with all respondents should be provided. A costly redesign and rebid exercise could, in this manner, perhaps be avoided.

These few brief remarks are not intended to revise the tunnel bidding/construction system, but only to convey some ideas about where improvements could make the business easier to manage and more equitable for both parties. They are my own ideas and do not necessarily reflect the opinions or philosophy of my employer or any contracting organization.

# TUNNELING IN THE UNITED STATES: A CONTRACTOR'S VIEWPOINT

by

Terence G. McCusker, Tunnel Consultant
San Francisco, California

The remarks which follow are based primarily on recent contracting experience in order to reflect as closely as possible a U.S. contractor's opinion of the tunnel construction business. The views expressed are those of the author and do not necessarily represent those of my recent employer or any other tunnel contractors.

It would be otiose to recite everything which is done right and causes no problems. Suffice it to say that, in general, the contracting system works, and sometimes works well. The purpose here, however, is to examine those areas in which it appears deficient and, where possible, to suggest practical solutions.

## 1. The Adversary Relationship

The dominant feature of the contractor/owner relationship is the polarization inherent in the concept that the duty of the owner's representative is to enforce the contract while the contractor's duty is to comply with it. Given this as the basis of the interaction between the parties, it is clear that the differences between contractor and owner can easily make them adversaries in a destructive manner, each party seeking an advantage over the other. Once the contractor perceives that the owner's representative regards him as a knave, and the owner's representative that the contractor regards him as a bloodsucker, much creative energy is diverted in resultant hostilities.

Both parties will tend to adopt positions which they regard as prudent in terms of the potential for future claims and change orders rather than engaging in a mutually supportive and rational examination of the facts. In particular instances, the contractor may perceive that the work can be accomplished more expeditiously or with a greater assurance of success if the specified method is changed. Too often, a proposal for such a change is rejected by the owner's representative because he feels that the contractor is seeking an unfair advantage. The fact that the advantage does not exist and can therefore not be identified, only serves to increase the suspicion of the owner's representative that he is being set up. On the other hand, if the contractor perceives a flaw in the design or specifications, he may avoid drawing attention to it, preferring that the owner endure the risk he has unwittingly accepted rather than take responsibility himself for proposing a remedy, and thus hazard the consequences if the proposal does not work out well. In summary, mutual suspicion leads to a considerable reduction in the flexibility of both parties in dealing with job problems and may prevent full use of the knowledge and experience of either party.

The report entitled "Better Contracting for Underground Construction," prepared by the U.S. National Committee for Tunneling Technology, draws attention to the very different role for the consulting engineer prevalent in contracting in the United Kingdom where, although he is the owner's representative, he acts in most cases as the arbitrator of disputes between the owner and contractor. Bidding is not open to all, and the low bidder is not necesarily successful. Qualified bids are acceptable, and virtually all disputes are resolved during the course of construction. Most important, perhaps, is the concept that when problems arise which are engineering problems, they are to be settled by engineers. In any case, the parties regard each other as gentlemen and settle their disputes with that in mind. The contractor is the more likely to accept the procedure, in that he is unlikely to be invited to bid again if he proves intransigent or if his probity proves to be suspect.

It is worth noting in this regard that for development work in the mining industry in the U.S., the same type of bidding procedure is followed as in the U.K., and is likewise substantially free of litigation. The contractor is also highly motivated to perform to the maximum of his ability in order to retain the favor of the owner.

## 2. Quality of Supervision

To the extent that the inspection staff is made up of those who have left a more active life in the contracting industry, they may exhibit understanding and a helpful approach to mutual problems. However, many inspectors have never worked in contracting; of those who have, many have left because they were unsuccessful. Neither of these backgrounds will dispose the inspection staff to be cooperative. More importantly, it is the case that many of the senior staff and especially resident engineers have spent their entire careers as designers or owners' representatives. Many of them lack any fundamental understanding of the work they are supervising and are, therefore, bound to 'go by the book', being unable to evaluate job problems in any but the most superficial way.

It is very unusual for the resident engineer or his staff to have the power to exert any but negative authority. No decisions of consequence can be made on the site and every problem becomes a major time-waster. The contractor has a very real grievance in that he is compelled to have a representative on the job with full authority to make all decisions regarding the conduct of the work, while the owner's representative has virtually no authority and decision-making power; hence, the contractor is held responsible for most decisions made in the field.

It is a commonly accepted principle of good management that financial authority be delegated to the lowest level of management capable of exercising it responsibly. Why, then, is it felt to be beneficial to the owner to dilute, divide and withhold this authority from those charged with the responsibility of managing his interest? It is certainly not possible for fair and timely decisions to be made under the system now prevailing.

It would appear that many, if not all, of the problems would be mitigated if the industry promoted a climate in which engineers in particular could exchange jobs freely in the areas of contracting, technical supervision and design, and if more decision-making authority were delegated to competent staff on the job who are in the best position to understand and evaluate the problems.

## 3. Timely Settlement of Claims

Contractors have become resigned to the fact that they are unlikely to recover any more than half of their costs when they are forced to make a claim for additional compensation. Not only that, but the recovery is likely to take five years and to be paid in depreciated dollars. Why is this the case? The fact is that if there is any tolerable excuse for not making payment - and often enough when there is no excuse at all - it is not in the owner's interest to make a settlement. It is not even in the interest of his representatives to make a recommendation for settlement. No one is going to receive approval and recognition of work from the owner by spending his money, especially when the need for the additional expenditure arose from a failure of his designers to foresee the costs at the time the contract was being prepared. How many designers are anxious to accept blame?

There is no question that the current system operates unfairly in this regard. It would seem that justice would be more likely if disputes were settled by an independent arbitrator or panel of arbitrators appointed at the outset of the project, receiving their compensation equally from the owner and the contractor. The arbitrators should be engineers familiar with the law, rather than lawyers familiar with engineering; they should have wide experience and mature judgment, and be widely respected.

It should be possible to set forth rational timetables for submittal, rebuttal and judgment, so that jobs could be completed and claims settled at substantially the same time. This would free both the contractor's and owner's engineers from time-wasting, energy-sapping and fruitless activity long after the design event. It would also serve to make contractors and owners less wary of each other and more prone to solve problems than to create them.

## 4. Restrictive Specifications

It is very often the case that contractors are required to accept all responsibility for the work under construction, while being restricted by the specifications to the use of methods or equipment detailed--often conservative methods and equipment known to be applicable to the type of project being constructed. Engineers claim that this is necessary to protect the public safety or to secure adequate performance from marginally competent contractors. It would seem more prudent to exclude the marginally competent contractor altogether!

The result of this approach is to inhibit innovation. It is distressingly apparent that tunnel construction costs in this country are significantly higher than they are elsewhere in the world, and that most innovative techniques are developed elsewhere and imported to this country only with great reluctance. It is notable that an 'experimental' length of subway tunnel is currently under construction in Baltimore using precast concrete segments - a procedure which has almost entirely displaced metal segmented tunnel liners in Europe, and which first was used almost 40 years ago.

We cannot afford the luxury of 'not invented here yet' conservatism; still less can we afford this sentiment when combined with inhibitions about invention. The limitation of bidders to those qualified both technically and financially, together with performance specifications would result in reduction of time spent in preparing loophole-free specifications and minutely detailed drawings while opening up the field to truly competitive bidding based on design and method, as well as on price. It is interesting to note that in West Germany, where alternative designs are routinely accepted from bidders, the owner's design for a subway system was so seldom followed that bids were finally put out giving little more than the alignment, profile and clearance information; the accepted bidder is held responsible for submitting all the necessary construction detail with his bid.

## 5. Designer's Limitations

It is not only site staff who are sometimes lacking in experience and qualifications for directing construction activities. Too often, an owner who is not routinely involved with tunnel construction will select a designer with little or no regard for his experience and knowledge of tunnel design and construction. The designer may well be generally competent and experienced in stuctural design, but there is no way in which he can reflect advances in tunnel technology, or even choose the best and most economic design for the owner, or specify and supervise proper workmanship.

The matter may be of small moment when short tunnels of simple design are involved remote from public use facilities. However, it would be helpful if all designers were more aware of the complexity of, and potential hazards associated with, tunneling, and avoided unaided preparation of contracts in areas where they lack competence.

6. Tunnel Design Code

The current lack of consistency in lining design methods sometimes gives the impression that every bid is for a different type of construction. To the extent that this is so, it means that bids are made with caution because experience is lacking to make a confident estimate of productivity. Caution on the part of bidders naturally leads to high bids.

It is also manifest that if an 8-inch thick precast concrete liner with grout in the tail void will support ground loadings satisfactorily, it should not be necessary to provide 8-inch ribs and lagging with 18 inches of doubly reinforced concrete lining in another part of the same tunnel. This point has reference in part to the previous discussion (Section 5, Designer's Limitations), of the need in this industry for a code of practice which will simultaneously ensure that the owner gets a fair shake and give protection to the designer.

Naturally, such a code should not be so restrictive as to prevent progress in unforeseen directions. But let there be some common sense applied in this area.

7. Open Bidding

Private owners have long recognized that their best interests were not served by open bidding. In fact, the usual practice is invite bids from selected contractors, often accompanied by qualifications, and then to follow with a round of negotiations to secure the best bargain.

The Government acts in much the same way in selecting designers. Why cannot an effective system be devised with appropriate safeguards to permit similar procedures on bids for public works? First of all, the risk born by bidders as well as by owners that an incompetent or unskilled contractor will misjudge the nature of the work and bid low in ignorance would be avoided. Secondly, the task of design and specification writing for all comers would be eliminated with consequent economies. There are other advantages and few disadvantages in terms of cost and reliability. The necessity would be the development of thorough and objective criteria for selection.

There is another alternative, perhaps more difficult to protect from corruption. This is the acceptance of qualified bids, with post-bid negotiation to resolve the qualifications. This does work elsewhere, but it effectively precludes publication of bid details. There would, therefore, necessarily be either a higher degree of trust or provision for review by an appointee of the bidders.

8. Wrap-up Insurance

Wrap-up insurance is a concept beloved by no one except the insurance companies. The arguments against it are many, from lack of individual incentive - insurance security can be at the expense of innovative design excellence - to loss of benefit to the good performer both currently and in his experience rating.

All large owners are essentially self-insured anyway, as are large contracting organizations. The only worthwhile argument for wrap-up is that it deals effectively wih the subrogation problem. But, why should workmen's compensation be included? In fact, by buying such a large policy, the owner is, in fact, buying his insurance in an uncompetitive market. And given the lack of competition, the insurance carrier can minimize his administrative costs by offering only token resistance to claims totaling less than the premium amount.

This appears to be an unsatisfactory way to do business.

9. Bonds

The cost of bonds for the average tunnel project amount to about one-half of one percent of the contract price. This is a small proportion but may amount to a large actual sum. The owner should consider carefully how much the bond actually protects him. It is a common fallacy that the bonding requirement will serve in lieu of prequalification. It does no such thing. The only question of substance the bonding company is interested in is the financial capacity of the contractor. The owner's inquiry into the technical or managerial competence of the contractor is of little moment to the bonding company.

10. Labor Skills

Tunnel labor has typically been recruited from the inexperienced and untrained including a high proportion unable to find other work which they would prefer to do. Yet, the skills required in tunnel construction are many and varied. Virtually all tasks can be performed better than they are and with greater safety. Should we not then encourage and support training programs for tunnel laborers instead of pursuing the haphazard course we have up to now?

This question is being addressed currently by certain western local industries involved in mine development work. The program they are looking at has been developed to train hard rock miners, but it is obviously and easily adaptable to tunneling, given a commitment by the industry. It is obviously easier to develop training programs when a man expects to spend a good deal if not all of his working life with a single employer. However, it should not be beyond the wit of man to solve the problem.

The basic approach taken by the National Apprenticeship Program for Miners is to split the work into self-contained jobs - for instance, installing resin-anchored rock bolts; erecting, blocking and lagging steel ribs; and so on. A training manual is prepared and instruction is given on the best way to perform each task. When a standard of proficiency has been attained, that skill is checked off on the employee's permanent record. In this way, labor can be selected or trained for specific job profiles; the acknowledgement of skill is itself a powerful incentive to the miner to do his job well. Information about this program can be obtained from:

Mr. M. Lee Scott, State Director
Department of Labor - B.A.T.
Room 314, Post Office Building
350 South Main Street
Salt Lake City, Utah 84101
(Telephone - 801-524-5700).

## Conclusion

This essay is a personal effort to set forth some of the problems which appear to need exposure, discussion, and resolution. It would be vain to assume that this contribution is decisive; but, as collective opinion begins to lean in a particular direction, the chance becomes greater that it will result in effective action. Clearly my hope is to have influenced the opinion of others as a step towards the goal of problem-free construction.

# LABOR TRAINING PROGRAM

M. Lee Scott
State Director
U.S. Department of Labor
Salt Lake City, Utah

Editor's Note:

The following letter is from M.L. Scott of the U.S. Department of Labor, whose Apprentice Training Program for underground work has attracted widespread attention.

NBI-G-139-C

**U.S. DEPARTMENT OF LABOR**
**BUREAU OF APPRENTICESHIP AND TRAINING**
Room 314 Post Office Building, 350 South Main, Salt Lake City, Utah 84101

May 11, 1979

Don C. Rose
Tudor Engineering Company
149 New Montgomery Street
San Francisco, California 94105

Dear Mr. Rose:

Once again it was a pleasure to talk to you regarding your very important and interesting work.

As we have discussed the Bureau does not have a clear policy regarding the broad use of the modular training concept which I developed for use in the mining and other energy related occupations.

However, in very brief form the following hopefully will provide you with an overview of what it entails. The modular concept is a very simple, practical, versatile and systematic way of training workers. It eliminates assumption on the part of the trainor and readily adapts to technological change in methods, tools, equipment, materials, etc, and easily fits into productions schedules. Modules are identified after a job analysis has been made and embraces the following characteristics: (1) must be self contained and can stand by itself; (2) must be small and have a single purpose objective; (3) must be easily identified and when combined with one or more module fit a specific job profile; (4) performance objectives can easily be established and employees easily measured; (5) is transferable to other occupations within the same or other related industries.

I trust this letter meets your initial needs, should a further in-depth paper be required, I would like to request that you contact the Administrator, Bureau of Apprenticeship & Training, Robert McConnon, Patrick Henry Building, Room 5000, 601 "D" Street, Washington, D.C. 20213.

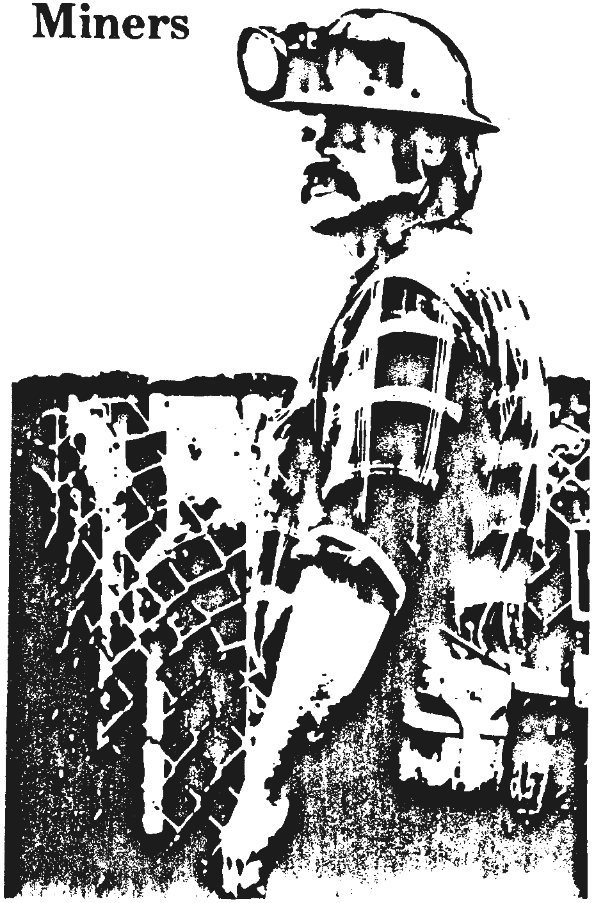
Sincerely,

M. L. Scott

M. L. Scott
State Director

The

# National Apprenticeship Program for Miners



## MODULAR APPRENTICESHIP PROGRAM FOR MINERS

1. Training in all of the mine production units of skill and knowledge.

2. Instructs by on-the-job training.

3. Successful completion of program results in government certification as a **Journeyman Miner.**

4. Training system based on the **modular** concept.

   Modules are small, self-contained pieces of work which can stand by themselves.

   Modules, when combined with one or more other modules, fit a specific job profile.

   Modules allow for the establishment of performance objectives against which the employee's performance can easily be measured.

   Modules are transferable either in whole or in part.

5. Four basic modular training programs will be developed in 1978.

   Underground Hard Rock Miner

   Underground Coal Miner

   Surface Hard Rock Miner

   Surface Coal Miner

6. The programs will be introduced nationwide in 1979.

7. The program has the active support of four international unions.

   U. S. W. A.

   I. U. O. E.

   O. C. A. W.

   U. M. W. A.

8. The program is funded by the U.S. Department of Labor, under a contract with Kennecott Copper Corporation—Metal Mining Division.

9. For more information, contact—

   **National Apprenticeship Program for Miners**
   **Suite 3003**
   **307 West 200 South**
   **Salt Lake City, Utah 84101**
   **Phone: (801) 364-1787**

   **or**

   **M. Lee Scott, State Director**
   **Department of Labor— B.A.T.**
   **Room 314, Post Office Building**
   **350 South Main Street**
   **Salt Lake City, Utah 84101**
   **Phone: (801) 524-5700**

# THE KEY TO PRODUCTION AND SAFETY IS JOB TRAINING

by

Richard C. Carleton
Project Manager
Gates & Fox Co., Inc.
Loomis, California

In May, 1977, when the underground work was begun at the Bath County Pumped Storage Project in Western Virginia, Gates and Fox Company brought in a team of experienced superintendents and walkers to start it off. At first they worked one-on-one with local hires, only a few of whom had ever been underground before. The schedule limited the work area to a few small drain tunnels, and under the close control of the supervisors the men were quickly trained to the level where they could operate safely and efficiently in the underground environment. In this way the crews were gradually expanded to about 100 men. The diversion tunnel was successfully completed in the early spring of 1978 using these job-trained men and a high percentage of supervisors. Then the extent of the problem of finding enough men to do the expanding job became evident.

During the late winter and early spring of 1978, teams of recruiters consisting of personnel men and underground superintendents scoured the country from Maine to New Mexico, wherever miners were known to be available, for additional recruits. A few were hired, but not enough to fill out the crews to the approximately 600 men needed to work in the 13 headings that were scheduled to be worked at one time. Additionally, several pre-employment training programs were begun. They lasted a week and were designed to teach the men the importance of underground safety and to familiarize them with terms and objectives of the tunneling profession.

At that time a survey was made. Of the 180 hourly men on the job, only 15% had ever been underground before and most of them for less than a year. Obviously something had to be done to either find more trained men or to train inexperienced men. Once the decision had been made to follow the latter course a unique program quickly developed.

Jim Sevy, the Gates and Fox Company Safety Supervisor, became the Safety and Training Supervisor and was given the responsibility for developing a complete training program. With the help of Jim Buck of the owner's Public Relations Department and Bill Crump of the Agent's Training Department, a manual was assembled and a video tape was put together showing the equipment the men would be using, emphasizing the importance of safety and encouraging production. Small-scale mockups of a tunnel face were built with a typical drill hole pattern to help demonstrate and give classroom practice in loading the wiring in a shot. A tunnel that was not on the critical path was selected as a training tunnel, and a classroom trailer was set up at the portal.

Each operating heading was then asked to pick a couple of new men who had no previous experience but had shown an interest in learning. Ten of these men were picked and together with a walker chosen for his patience and even temper, an experienced shifter, and a lead miner, they began the first 10-day training period.

On the first day the video tape was shown, the underground equipment and objectives were discussed and the proper use of safety equipment was demonstrated. The men assembled on the drill jumbo, and the shifter and lead miner demonstrated the use of drills and other equipment and emphasized the importance of proper drill alignment, care of the equipment and concern for the safety of their fellow workmen. Whenever it was felt necessary, the drills were stopped and an impromptu lecture took place. This is the particular advantage of a training crew over a normal production crew. The high level of sound underground generally precludes discussion of objectives and methods with new men. When the first round was drilled out, the class went to the training trailer for a demonstration, on the mockup, of proper wiring techniques, along with lessons in the safe care and handling of explosives. The round was then loaded and shot. Next, the operation and care of the mucking equipment was covered and, in succession, the shotcrete and rockbolting operations were demonstrated and practiced.

After one week of stop-and-start training the crew was turned loose to develop their production techniques and to work themselves into a production team. Surprisingly, by the end of the second week these "new hands" were getting two 10-foot rounds a shift in a 14' x 16' tunnel.

On graduation day, the walker, the shifter and the lead miner met and agreed on the individual grades for members of the class. Grades of A+ to D were assigned, along with verbal comments on of the men's special attributes and attitudes; then the class was graduated and distributed to the working headings. Despite some early misgivings among some of the old-timer superintendents, it wasn't long before they were asking for copies of the final grades of subsequent classes and requesting the trainees by name.

Following graduation of the first class, two new 10-man classes were set up on day and swing shift. Most of the men came directly from the Agent's personnel department, often with no work experience. Occasionally they would take one look at the underground environment and walk back to the dry house; a few went through the two weeks, but were judged unacceptable for underground work. These usually went to the bull gang for outside support work.

Every two weeks for the next eight months, twenty raw recruits were turned into underground production hands. The percentage of those who had never been underground prior to working at the Bath County Project remained the same (85%). At the end of that time, they were part of a team that in one 5-day week in January, 1979, excavated 1631 linear feet of tunnel ranging in diameter from 10 feet to 34 feet.

Of equal importance for the production records, the safety record on the project was outstanding. One million three hundred thousand cubic yards of rock were excavated without a fatality and with severity and frequency rates not only considerably less than the mining average, but actually less than the national average for general construction.

It should be emphasized that these men were not considered to be "miners" at the end of their class time, but they were ready to carry their own weight as specialists in the underground operation. Wherever possible they were placed in positions where they showed particular promise and felt comfortable in activities ranging from drilling, to mucking machine operation, to shotcrete nozzling. Since the completion of the excavation operation, many of the men have moved on to other projects, carrying with them the skills learned at Bath

County. Others were given new training at the project and became part of the concrete lining crews.

The safety and production benefits that accrue to a project through specialized training of the men far outweigh the costs of such training. There is no substitute for the learning experience a man receives as part of an actual production crew, but we believe he learns faster, better and more safely if he has had the head start of a training crew experience.

Editor's Note:

This outstanding pumped storage project completed approximately 12 miles of hard rock tunnels and shafts of several different diameters, by conventional drill-and-blast methods as well as TBMs, at world record progress rates and at a remarkable low cost of approximately $1,000 per lined foot. The rock was a gently dipping, regularly bedded sedimentary sequence free of heavy joints or faults or water problems. The Contractor was working on a Target Estimate contract with incentive, and quickly won the maximum bonus possible but continued thereafter at the same outstanding progress, to the Owner's benefit. Gates & Fox Co., Inc. management and personnel are to be congratulated on this fine performance.

## COURSE OUTLINE/SCHEDULE

Lesson #1-----------Basic Orientation

Introductions

Objectives and Methods

Field Inspection of Tunnel and Equipment:

- Mainline Jumbo
- Rockbolt Jumbo
- Shotcrete Jumbo
- Mucking Machine
- Jackleg
- Stoper
- Drill Bit
- Drill Steel

Other Tunnel Terms:

- Heading
- Face
- Crown
- Arch
- Rib
- Mucking

Hand out sheet on tunnel terminology.

Tunnel Safety:

1. Hand on drills
2. Proper clothing
3. Where to walk
4. Watch face when walking on jumbo
5. Danger of air hose
6. Stay under wings of jumbo.
7. Do not stand with back to face of heading
8. Do not stand directly under rock which you are scaling.
9. Do not stand directly under stoper or jackleg when collaring hole overhead.
10. Do not smoke or allow open flame or electric lights around blasting powder or other explosives.
11. Keep all blasting leads, lead wires, and leg (cap) wires shunted until ready to wire in.

Lesson #2-----------Drill Cycle

- Move in Jumbo
- Hook up Jumbo
  - Blow Bull Hose
  - Drilling----hand Signals — Machine damage potential and safety
    - Location
    - Depth
    - Spacing
    - Burn Holes
    - Theory of Blasting
  - Jumbo Servicing

Lesson #3-----------Load and Wire

- Powder Cap and Safety
  - Powder Types
  - Caps and Wire
  - How to Make Primers
  - Proper Handling
- Theory of Blasting
- Application in Heading of the above

Dummy Caps, Shooting Box, Lead Wire, etc.

Lesson #4-----------Smoke and Misfires

- Smoke Time
  - Required waiting time----
  - fifteen (15) minutes in tunnel

- Walker-Shifter responsible for:
  - Shot Round
  - Checking face after blast, first
  - Procedure on misfires:
    - Shunt
    - Check
    - Rewire
    - Reshoot or wash out

Handouts

# POWDER TRUCK DRIVERS INSTRUCTIONS

1. All powder truck drivers shall be licensed drivers.

2. The driver shall be physically fit. He shall not be under the influence of intoxicating liquors, narcotics or other dangerous drugs.

3. No person shall smoke, or carry matches nor any other flame producing device, nor shall firearms, or loaded cartridges be carried in or near a motor vehicle or conveyance transporting explosives.

4. The driver shall see that the explosives signs and the fire extinguishers are kept in place and in good shape.

5. The driver shall not leave a loaded truck unattended at any time. He shall keep the magazines locked at all times, except when loading or unloading explosives.

6. The driver shall keep an accurate record of all explosives used during his shift.

7. The driver should, if possible, check the Jumbos for loose powder and caps at the end of the loading operations, before leaving out of the heading.

8. The driver will take all empty cap and powder boxes to the burning pit for burning. At no time will the truck be parked closer than 50 feet to the burning pit, and to be on the safe side, we recommend a distance of 75 or 100 feet from the pit.

9. Powder trucks going and coming to the burning pit will use the haul road and at no time will the powder trucks be allowed on Route 600 or off the job site, as they are not licensed or inspected to go off the job site.

10. The driver will receive two copies of this directive, he will read one copy, sign his name at the bottom and return it to this office, he will keep the other.

11. The driver will keep the attached portion of the OSHA standards dealing with explosives.

Jim Sevy

Jim Sevy
Safety & Training Superintendent

JS/mbd

Attachment

____________________

JULY 14, 1978

TO: DICK CARLETON
CLAY PAULSON
KEN BURDICK
JIM SEVY

FROM: LEE KENNARD
DEAN DYE
RAYMOND PHEBUS

SUBJECT: MINE TRAINING SCHOOL

RICHARD W. A+

This man is number one in this class. He is very good drilling, loading and wiring in. He is also very good on the mucker, if any one ever had an excellent attitude about his work, this man does. He has been with in the school four weeks, he has never been absent or late. He has the ability and will lead the crew.

TOMMY C A

This man is very good at drilling, loading and wiring in. He can run the mucker, but is a little slow at mucking. he will improve with more experience. He has a good attitude about working.

HENRY H A

This is going to be a good miner with more experience. He is good at drilling, loading and wiring in. He is second best in class on the mucker, which is pretty good. He has a good attitude about working.

JAMES A A

This man is good at drilling, loading and wiring in. He is fair on the mucker, with more experience he will be a good mucker operator. He has a good attitude about his work.

CARL R B+

This man is good at drilling,loading, and wiring in. He can run the mucker but needs a lot more experience. He has a good attitude, but has a tendency to get over anxious about doing things.

PETER A C

We don't know what to say about this man, he has the know how and the ability to make a good hand, if someone could get out of him. He can drill,load and wire in, he can run the mucker. He needs to improve his attitude about work.

LEE KENNARD
Lee Kennard
DEAN DYE
Dean Dye
RAYMOND PHEBUS
Ray Phebus

## LEGAL PROBLEMS IN TUNNELING

by

Max E. Greenberg

Senior Partner, Max E. Greenberg,
Trayman, Cantor, Reiss & Blasky,
New York City and Washington, D.C.

### INTRODUCTION

Tunneling contracts often are one-sided documents strongly favoring the owner over the contractor. This is not surprising since the contracts are normally written by owners' representatives and presented to contractors on a "take-it-or-leave-it" basis. However, court decisions, while far from being uniform, have ignored or construed out of existence literal terms of these contracts often enough to quell any over-optimistic owners' sanguine belief that, notwithstanding the contract language, they would be protected if serious problems were to arise.

Underground construction is fraught with risks and many tunneling contracts seek to assign all these risks to the contractor. This paper will consider what gives rise to current contracting practices, whether they are really in the owner's best interests and what are available alternatives.

### Adversary Relationships

Everyone agrees that it would be wonderful if we could have a team approach in the design and completion of underground construction projects; the team should comprise the owner, engineer and contractor. The trouble is there are conflicting interests among these participants.

The owner wants to know - and in most instances must know - what the project will cost him. Private owners may have only limited funds available or the business desirability of the project may hinge on its costs. Public owners are limited by appropriations or bond issues and must - except in cases of emergencies - know within modest limitations what the cost will be since they may not contract for amounts beyond the funds available.

It is in the interests of the owner, therefore, to attempt to procure a contract for a definite fixed price, or for cost plus a fixed or award fee, or a management contract, all with a guaranteed cost maximum. Any cost reimbursement type contract with a maximum cost but without a guarantee is impractical. If the cost exceeded the maximum, additional funds would have to be provided or a useless uncompleted structure could result.

Often for very practical political considerations, owners prefer to understate project costs when appropriations or bond financing are sought. Once the project is well under way, funds necessary for completion are more easily obtained. High risk factors are either hidden from bidders or masked over by broad, non-specific exculpatory clauses.

The moment a fixed or guaranteed price is sought, a conflict of interest is created. It means an attempt is being made to pass the buck to the contractor for all uncertainties that may arise in the course of the construction. If the contractor wants to remain financially competent to stay in business, he must include in his guaranteed price a percentage to cover these possible contingencies; he must comply with "Murphy's

Law" and assume that the worst that could happen will happen. It is in the contractor's interest to enlarge the contingency percentage as much as possible while it is the owner's interest to keep it down.

As for the engineer, he usually agrees to prepare plans and specifications to produce a structure at a cost not to exceed a specified maximum. If bids for fixed prices substantially exceed that cost, or if cost reimbursement types substantially increase that cost (not counting changes not due to his errors or omissions), he may be in trouble with the owner. He may develop conflicting interests with the owner or the contractor or both.

The engineer naturally is inclined to avoid antagonizing the owner who employs him. However, contracts themselves often place the engineer in the role of arbitrator of disputes between the owner and contractor. A contractor will not expect a purely objective determination uninfluenced by the relationship of the engineer with the owner.

Thus, a basic conflict of interest exists between the parties to a construction project. Nowhere is this conflict more dramatically demonstrated than in the area of responsibility for unanticipated subsurface conditions. It is on this topic that the remainder of this paper concentrates.

## Unanticipated Subsurface Conditions - Attempts at Shifting the Risk Factor

Disclaimers, exculpatory clauses and express assumptions by the contractor of obligations and risks are the normal means invoked in attempts to assure the owner of a definite project cost. Some examples are: "The borings are furnished for such information as the owner has but are not guaranteed and are not to be deemed part of the contract documents"; "Contractor agrees that the owner has made no representations as to subsurface or latent conditions and contractor assumes all risks relating thereto"; "Bidders shall visit the site and shall make their own exploratory examination as to subsurface conditions; Owner makes no representation with reference thereto".

Both sides know this latter provision is ridiculous.

How could the bidder, within the forty-five or sixty days usually allowed for study of plans and specifications actually do sufficient study by exploration or other means to ascertain subsurface conditions? Nor would it be economically feasible for a bidder to do so on every contract for which he bids. He is a successful bidder on only a small percentage of contracts bid for. On the other hand, the owner has the time necessary to make the needed studies.

There are several unfortunate consequences flowing from the inclusion of this type of clause in a contract.

(a) The owner, feeling the contractor is assuming all risks, may be induced to try to save time and the expense of a detailed study and analysis of all available sources of information as to subsurface conditions. He may skimp on exploratory work under the impression he is protected from the cost of unexpected conditions not disclosed by the contract documents.

(b) A responsible contractor will include a substantial contingency item in his bid to cover unexpected costs. The contractor may not attempt to cover all possible contingencies so as not to price himself out of the market. However, he will provide for as many as possible. If in fact such unexpected costs are not encountered, then the owner has been subjected to an increased cost beyond what was reasonable.

Therefore, it is obvious that the goal of the contractor is to overcome the burden of uncertainty by passing its cost along to the owner by way of this contingency item, thus thwarting the owner's intention of casting the burden of uncertainty on the contractor.

(c) On the other hand if the contingency item is not adequate, the contractor will in all probability seek a means to recoup his losses - by claims in litigation, if necessary.

The important point is that, regardless of the form of contract, the owner pays. The more risks imposed on the contractor, the greater his contingency percentage. And, if ultimate performance demonstrates this "hedge" to have been inadequate, an attempt by way of litigation will be made by the contractor to recover the loss.

Contracts containing these exculpatory clauses are breeders of one of the most time-consuming and costly types of litigation; litigation in which both sides lose; where, in addition to the cost of attorneys, accountants and experts, a great amount of the valuable time of the employees of both sides must be expended in searching out facts and records, in interviews and consultations, preparation for and attending trial, etc. Tunneling litigation usualy involves huge sums and the cost of each side may easily run to millions of dollars. The victor simply may lose less than the vanquished.

An added incentive for dropping the use of these clauses is their ineffectiveness. The customary protective clauses do not shield the owner to the extent he believes. In reality, they are exercises in futility and should be abandoned.

The law in the United States imposes a burden on the owner which defies his protective clauses. He must disclose any information he has which will affect the contractor's cost. His failure to do so is treated as a misrepresentation entitling the contractor to recover damages. The knowledge of the owner's engineer is deemed the owner's knowledge.

Even if the owner has no actual knowledge, he is still liable for failure to disclose what he "should have known." He "should have known" what an adequate examination of existing sources of information would have disclosed. Some examples are old maps, records as to experience on previous projects in the area, and pamphlets and treatises concerning the area.

Here a few examples of how far our courts have gone in imposing liability upon owners for failure to disclose what they "should have known":

An owner was held responsible where a contractor encountered boulders in the area of a reservoir filled in many years before. The only source of information was newspaper items in the public library which disclosed the fact that boulders were in the fill (Frank Nordone Contracting Co., Inc. v. City, 269 A.D. 1035, 59 N.Y.S.2d 256 (1st Dept. 1945)).

In the early days of the City of New York a canal existed in a certain area which now consists of apparently solid land. A project designer was deemed responsible for knowledge that the canal had existed, and that it must have been backfilled, and that there were in all probability obstructions of different types, such as sunken vessels, in the area involved (Cauldwell Wingate v. The State, 276 N.Y. 365 (1938)).

As for the impossible requirement that the bidder be responsible for obtaining his own exploratory information, the United States Court of Claims has stated:

> Therefore we hold that because of the inability of the plaintiff to bore, analyze the borings and to compute, prepare and submit its bid within the short time allowed, plaintiff is not bound by the cavetory and exculpatory provisions of the contract and specifications and conversely these provisions do not relieve the defendant of liability for changed conditions as the broad language thereof would seem to indicate. (Fehlhaber v. U.S., 138 Ct. Cl. 571 [1957]).

Therefore, it is apparent that the owner may be suffering a delusion if he feels that he is fully protected by exculpatory clauses; that the existence of such exculpatory clauses tends to make the owner less insistent upon a proper examination of existing sources of information concerning subsurface or latent conditions by reason of the time and the cost required for examining those sources and analyzing the same for the purpose of preparing the plans and specifications; that the owner is being subjected, in most instances, to a contingency cost which may never develop; and that the owner is, in fact, generating a situation fruitful for the development of expensive litigation.

Thus, it is not only in the interest of the contractor to avoid these disclaimers, exculpatory clauses and assumptions of risks, but it is in the interest of the owner as well. If these facts are realized the parties may be in a position to finally work out a contractural relationship with the desired team flavor.

## A Proposed Solution

If exculpatory clauses are to be effectively avoided, a subsurface exploration program must be formulated to adequately study available sources of information, concerning subsurface conditions as a basis for proper preparation of plans and specifications in such substantial detail as can result in an intelligent and knowledgeable bid. Obviously the only logical person to make such study for preparation of plans and specifications is the owner through his engineer. He can take all the time needed for an adequate study -- years, if necessary. The owner, and not the contractor, should include a substantial percentage to cover the risks of uncertainties in estimating his probable costs. This should be reflected in the appropriation. It is better for the owner to do it initially instead of paying for it as a hidden cost in the contract price with increased costs of litigation if the contractor finds his contingency item inadequate. If done properly, contractors' bids will be well below the appropriation. Funds will then be readily available to satisfy claims for those risks actually realized.

There are two basic considerations to be kept in view in determining the contractual arrangement: (1) the owner pays; (2) the contractor would be inclined to exert greater effort to accomplish savings in performance if he gained thereby.

As for incentive to the contractor to keep costs down, it may be given in two ways: (1) by way of bonus for earlier completion, or (2) by way of a portion of savings in agreed estimated costs of fixed fee, reimbursable or management forms of contract.

It may be advisable to follow the European system of breaking the job down into as many unit price items as possible, thereby further reducing contractor's risk and room for argument. Owners often object to this on the ground that it requires more time, effort and engineering analysis. Engineering costs are higher and the owner is forced to assume a greater responsibility. So be it. Engineering expenditures at a project's inception will save the owner money in the long run by reducing contingencies, claims and litigation costs.

The owner's engineer, therefore, should be obligated to study available sources of information and to do adequate exploration work for the purpose of ascertaining subsurface conditions. What he learns or knows should then be disclosed to the prospective contractors. In addition, the engineer will frequently be required to disclose not only what he knows but what he "should have known." That is, the engineer should be obligated to examine maps; records of previous construction; historical data, such as the prior existence of canals, streams, wells, etc. previously backfilled; and all similar sources of available information. This places on the engineer the burden of making a diligent effort to ascertain subsurface conditions and imposes liability on the owner if the engineer fails to do so. Stated differently, the owner has a duty, through his engineer, to make a reasonable and thorough effort to discover actual subsurface conditions. The knowledge obtained should be disclosed in writing and in detail; the engineer should include references to his sources of information and a summary of the exploration work done by him.

The rule requiring the owner to disclose what he knows or should have known is practical, because the owner alone has the means of access and adequate time to explore, analyze and study available sources of information concerning subsurface conditions. It is suggested, therefore, that risks caused by subsurface conditions, to the extent they are ascertainable by reasonable exploration and study of existing sources of information, be assumed by the owner; and that the owner be obligated to disclose all information concerning such risks to prospective bidders. If contractors were actually to make their own explorations, their bid prices would have to include this cost, duplicating expense to the owner, without rendering him a corresponding benefit.

It is suggested that, in addition, the owner list his sources of information; prospective contractors who have differing information about conditions or knowledge contradictory to that disclosed by the owner have the moral obligation to disclose that fact. While it may not be possible to enforce such an obligation upon prospective contractors, certainly they could be reassured that problems they could have foreseen, if encountered, would not be made a basis for the recovery of damages.

It is also suggested that the owner make available the written reports of experts employed by him, including the conclusions they have drawn from disclosed facts and the reasoning forming the basis of their conclusions. Those opinions which are significantly relied upon by the designer should also be identified; they should be clearly identified as expert opinion and not presented as a report of conditions. The prospective contractor may accept or reject them; he may form his own conclusions based upon his own expertise or that of his employees. The owner should not be bound by the opinions or representations of his experts.

Since the plans and specifications are prepared by designers employed by the owner, he should be responsible for their competence. The owner should be held to warrant that the plans and specifications are free from errors or omissions, based upon the disclosed information.

Owners should be encouraged to engage in well-conceived and thoroughly executed underground investigations, by the elimination of disclaimers, and by the disclosure of information sufficient for both subsurface design and construction. This will benefit owners by reducing the margin of uncertainty, enabling better design and planning, and leading to more economical construction. In the long run, the added time and expense of a thorough site investigation is much less costly than the time lost and extra expenses incurred during construction in overcoming the consequences of inadequate preliminary underground information. The better the underground information, the less need there will be to disclaim responsibility for unpleasant surprises, and the less frequently the owner will be faced with claims.

If it were possible, prior to submission of bids, to ascertain the true subsurface conditions through explorations and study, the owner would pay for the cost of worker performance in response to those conditions, since bidders would base their estimates thereon. It is reasonable that the owner pay for the performance imposed by the true conditions. Why should the risk of variance from the expected be imposed on the contractor? In fact, as discussed previously, under existing contract practices the contractor attempts to impose that cost on the owner by including a contingency factor in his estimate.

It is reasonable, therefore, that the contract contain a "changed conditions" clause (also known as a "differing site and subsurface conditions" clause), which places the burden where it would have been if the true conditions had originally been known - on the owner.

What is meant by the term "changed conditions"? This is commonly used to designate unexpected unknown conditions, i.e., conditions which available information would not lead one to anticipate encountering in a given area. Under a contract with a "changed conditions" clause, such variance from the disclosed conditions would entitle the contractor to payment for the resulting additional costs.

A distinction should, however, be made between factual data and opinion as to the binding effect of each upon the owner. The owner should warrant the accuracy of all factual data. Factual data should be binding on the owner for purposes of a "changed conditions" determination. The owner should not, however, be held accountable for the accuracy of opinion in situations where equally competent professionals draw different conclusions and have different interpretations of the same data. Opinion should not be binding on the owner for purposes of a "changed conditions" determination; it should merely be considered as one factor along with all other evidence.

The suggested "changed conditions" clause would be in conflict with the usual disclaimers of responsibility for subsurface information, encouraging contractors to eliminate from their bids contingency sums to cover the uncertainty created by the disclaimers. This is, therefore, another reason for eliminating such disclaimers.

If a "changed conditions" clause were to be included in a contract, the contractor would not feel required to include in his bid any contingency item for unanticipated subsurface conditions. The owner does not have to pay a windfall price when only normal conditions are encountered, and the contractor suffers no disaster when unanticipated conditions arise. The owner only pays as if the true conditions were originally known. Both parties further benefit by the creation of an informal procedure, generally through the engineer, for resolving problems by negotiation rather than litigation. However, from the contactor's standpoint, this benefit is colored by the previously mentioned alliance between the owner and his engineer.

Clearly, the long-term advantage of the inclusion of a "changed conditions" clause in construction contracts is in lowered construction costs, (1) by promoting emergence of more contractors willing and financially able to engage in such work, and (2) by eliminating underground risk contingency costs from bids. The owner, by reason of the increased competition, pays less for the completed project and receives actual money value for what he contracted to have constructed.

## Conclusion

By removing this common source of friction between owners and contractors, the modifications of contract provisions discussed would result in a more harmonious relationship between the parties; it would tend to create an atmosphere of joint enterprise rather than one of conflicting opponents. Logically speaking, the ultimate result will be a savings to both parties to the contract.

# RISK MANAGEMENT & INSURANCE IN TUNNELING

by

Earl K. Novell, Vice President
Molton, Allen & Williams Insurance Corporation
Birmingham, Alabama

Risk Management is a term used and misused in today's business world. The American Heritage Dictionary defines:

RISK: as "the possibility of suffering harm or loss; danger. A factor, element or course involving uncertain danger," and

MANAGEMENT: as "The act, manner or practice of managing, handling or controlling something".

"Risk Management" is therefore the practice of managing, handling or controlling the possibility of suffering harm or loss.

Modern business has developed Risk into two basic categories:

1. Pure Risk: That measurable risk which is primarily the subject of insurance and self insurance where it is possible through the analysis of Risk to utilize one or all of the elements of "Risk Management";
2. Business Risk: That risk which is speculative in nature. This type of Risk is knowingly taken as a gamble of the entity's capital and assets with or without an analysis of the speculative nature of the business risk involved.

The early use of "Risk Management" emerged 15-20 years ago as a developing discipline by those involved with purchasing insurance for their companies. This group, together with the insurance and academic community, proceeded to formalize the Risk Management function:

Risk Management

1. Identification of Risk;
2. Analysis, Measurement and Evaluation of Risk;
3. Elimination of Risk (Loss Control);
4. Reduction and Control of Risk (Loss Prevention);
5. Assumption of Risk (Self Insurance - Deductibles);
6. Transfer of Risk (Insurance, Hold harmless, etc.).

Over the past few years, the Loss Control (Safety) discipline has been included within the Risk Management function, by more and more companies, reporting to the Chief Executive Officer. In a recent study conducted by the University of Arizona, 20 percent of 1084 large corporate entities reported that Loss Control (Safety) functions were an integral part of Risk Management. The Insurance and all claims activities were a necessary part of Risk Management experience.

Risk Management produces the best results where all related functions have been placed under a qualified Risk Manager reporting to the Chief Executive Officer; where the Board of Directors have approved and published a "Risk Management Policy"; and finally, where there is enforcement of that policy by the officers. The more diverse and decentralized an organization, the more difficult an optimum risk management program becomes. But the keys are always

1. Responsibility
2. Authority
3. Two-way communications
4. Management attitude
5. Accountability

As you will note, the insurance part of risk management is under the "Transfer of Risk" category, and after the "Assumption of Risk", of which self-insurance is a major part. However, it is most important to note that in order to reach a point where one knows what insurance to purchase a thorough, in depth study must have been conducted to fulfill the other major risk management steps.

The most important area of Risk Management is the identification of Risk of Loss; it requires the coordination of all facets within an entity to maintain a continual identification process for evaluation and determination of how to handle the risk. The loss control (safety) function becomes an integral part of this process. It is inconsistent to place the loss control function in an area other than Risk Management. Historically, when loss control is placed within the purview of line operations, loss control becomes subordinated to the line operation, and usually becomes ineffectual. This is similar in effect to placing the audit and accounting function within the same department.

When we consider how difficult it is to make a profit in today's business climate, the need for effective loss control management becomes absolute. However, top management, in many cases, is not committed to the prevention of accidental losses as a means of improving overall performance. Accidents are too often viewed by management as unavoidable and isolated incidents, rather than as a clear warning that something has gone wrong with the management system. In addition, businessmen are as susceptible as anyone to the myth that adequate protection can be bought just like any other commodity or service. In the insurance field, we still encounter the attitude that, "We're insured, so why worry about losses? Let the insurance company pay."

Significantly, OSHA and other government safety programs have generally failed to make an appreciable dent in industry's accident-loss record because their approach is essentially punitive. Safety is too important to be left entirely to the safety professionals. The success of any loss control program will depend largely upon the degree to which management takes positive action.

## Insurance and Risk Management

Insurance is the last line of defense to protect assets of an entity. It is not a substitute for sound business practice. Insurance is not for maintenance purposes. It is to protect against fortuitous losses. On large construction projects, and for large firms, the insurance is pretty much self-rated as to experience incurred. The fundamental principle of insurance is the spread of risk over homogenous units using the law of large numbers. Whenever the burden of risk is out of balance, as for example when catastrophic losses exceed the expected or the current "social" change, and, in tort law, the policy coverages not anticipated in the rating structure are broadened, then large or massive rate increases may be anticipated.

What is liability? "A state of being liable, i.e., obligated by law or equity; exposed or subject to some unusual adverse contingency or action." Insurance is one way of handling liability, but not the only way.

The "hold harmless" clause is one of the most onerous forms of transferring risk ever [illegible] example, California passed legislation prohibiting engineers from making contractors assume the engineer's professional liability through "hold harmless" clause or other means. The use of this means of transferring risk should be eliminated. All known risks should be spelled out in a definitive manner.

Full disclosure of <u>all</u> facts known to owners and engineers, <u>must be given</u> (not "made available") to the Contractor, thus reducing the unknowns to a minimum. Owners must spend more money either researching projects or allowing more money on change orders for inadequate research and design. The need to create a close <u>non-adversary</u> working relationship between owners/engineers and contractors/subcontractors must be met to reduce the cost of construction. Risk must be born by those whose risk it is - not passed on to others. This only results in the costs coming full circle back to the general public, the ultimate consumer.

The problem of liability, today, and its attendant high costs passed back to the consumer lies at the very heart of the changes in Tort laws. The technical achievements of our society have created a fertile field for the evolving social conviction that someone need always pay. In 1975, the Ohio Supreme Court refused to allow <u>Punitive</u> damages to a plaintiff suing the City of Cleveland. A punitive damage award would ultimately have been imposed on taxpayers forcing those seeking services to bear the burden (<u>Ranells vs. City of Cleveland</u> 321 NE 2nd [Ohio, 1975]). This limiting factor was a step in the right direction towards tort reforms. Another example is the pilot legislation being introduced and passed by many state legislatures limiting

Completed Operations Products Liability awards in time and amounts.

Because of the broadened interpretations of "social" change in tort law, a closer relationship and crossover of these risks has developed which has created some intolerable problems in insurance capacities throughout the world insurance market, ultimately causing involuntary self-insurance; this, in turn, has caused management to assume "Pure Risk" as a "Business Risk" speculating with corporate assets. This is forcefully shown by the "social" change interpretation in tort law respecting products liability over the past five years.

One of the most acute problems facing the owner-engineer-contractor triad is the need for mutual trust. Until these three groups assume their own risks - both PURE RISK and BUSINESS RISK - there is only a likelihood of mitigating the costs of all risks. Working against this goal is the unreformed "social" adaptation of tort law which says "somebody has to pay," and the practice of juries awarding monies beyond the wildest expectations of business anticipations.

Where do we start? Insurance is not the answer. Tort reform towards limited liability is one area of reform. The assumption of their respective risks by owners, engineers, and contractors is another great step toward mutual trust.

Tunneling creates several specific insurance problems for contractors, particularly in the areas of Workers' Compensation, General Liability and Builder's Risk Insurance. Many insurance companies hesitate to write tunneling insurance because of the high hazards involved and the losses incurred on some projects over the past 15 years.

Workers' Compensation: This coverage becomes difficult to secure and expensive whenever applied to tunneling. The rates per $100 of payroll can be 50 percent or more of actual payroll when compressed air tunneling is involved. Recently there have been instances where contractors on large tunneling projects have self-insured Workers' Compensation because of lack of an insurance market.

General Liability: This coverage is to protect the insured for legal liability arising out of injury, death and property damage. Tunneling can produce some unusual claims both in urban and rural areas including collapse, changing water courses, undermining existing structures, etc.

Builder's All Risk: This coverage responds to damage to the tunnel and structure during construction, and should include both owner and contractors. There have been several expensive claims in recent years from this class of business, which covers collapse, flooding, fire and explosion. Generally, underwriters are restrictive with this class of business.

It is well to remember that insurance theory is "the spread of risk of homogenous units." Thus all contractors/owners pay for the losses of the few. Whenever this appears to the owners/contractors to be unreasonable, then self-insurance and excess insurance programs should be considered for all lines of coverage. This is, however, an individual, job-by-job review and choice between alternatives and trade-offs.

There is no panacea for the need for "insurance," nor will there be in the future. Good management and good loss control programs are the answer to viable Risk Management and insurance needs.

## WRAP-UP INSURANCE

The controversial topic of Wrap-up insurance has been discussed almost exhaustively. However, in the face of open or tight insurance markets it remains the only viable solution for large projects. It is here to stay, and some contractors for many years have used this method of project insurance for their own accounts when owners have not made provisions in their plans. Most significantly, the U.S. D.O.T. UMTA "Report NO. UMTA-MA-06-0025-77-13, Insurance for Urban Transportation Insurance" has researched the subject and reported favorably for Wrap-up. This is recommended reading and can be secured through the National Technical Information Service, Springfield, Virginia, 22161.

Advantages of a Wrap-up Program:

1. It provides uniform and presumably high level insurance expertise for the establishment and execution of an overall project program for management and control of safety and financial risks.
2. It promotes a coordinated safety program, in which the insurance group, the owner, the engineers, and the construction contractors can and must cooperate if the program is to be most effective. One insurer handling the safety engineering program means that only one set of standards must be complied with instead of several, as might be the case if numerous insurers were involved. This should eliminate delay in complying with recommendations, and provide for systematic attention to safety risks.
3. It places responsibility for handling and payment of claims upon a single insurance group, minimizing coverage disputes between insurance carriers, and third-party complaints of uneven treatment.

4. It provides for prompt resolution or settlement of third-party claims by minimizing disputes between insurance carriers. This advantage has important public relations value to the owner and contractors.
5. It makes for a more effectively structured and controlled survey of existing preconstruction conditions, thus providing defense against claims of damage and uniform and prompt disposition of claims charging damage resulting from construction.
6. In a tight insurance market, it assures availability of adequate coverage and thereby permits participation of potential bidders who might not otherwise be able to obtain adequate coverage individually. With all of the insurance - high limits, broad and uniform coverages - provided by one insurer, the buyer does not have to worry about adequacy of coverage.

Disadvantages From Contractor's Viewpoint

1. Dividends, discounts or experience credits insure to the buyer; on the other hand, loss of the premium volume, eliminated by Wrap-Up, adversely affects the contractors' position with their own regular insurers, particularly when there is enough volume for premium discount or experience rating.
2. The sometimes difficult question of where General Liability insurance leaves off and Automobile Liability insurance takes up becomes an important point of issue, because few, if any, Wrap-Up programs include Automobile coverage.
3. When employees of contractors interchangeably work on the Wrap-up project and others, disputes concerning the

particular policy applicable on any one project becomes an additional burden on the subcontractors and can result in an award delay for injured employees.

4. The principal argument of insurance producers is that their normal business relationship with the contractor is disrupted, resulting in loss of income.

Summary

Effective low-cost "insurance" in tunneling will require the application of risk management principles through strong controls, built-in loss prevention, self-insurance retentions and excess insurances. Those who act upon the principle that "we're insured, let the insurance company pay" will not long remain in business, be they owner, engineer or contractor. The owner-engineeer-contractor triad must work as a team with full and total disclosure of all facts involved in the project, particularly in tunneling where there are so many unknowns. Loss prevention through professional work methods and programs is the cheapest form of "insurance". Most Workers' Compensation losses cost $6 for every $1 paid as medical or compensation benefits. Remember the accident IMPACT COST is never covered by insurance, and is always more than the insurance claim payment. However, the cost in human pain and suffering has no true "price". This must be accounted for by the triad.

# MODERN INSTRUMENTATION FOR TUNNELS

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## INTRODUCTION

Qualitative instrumentation and monitoring systems have probably been used in tunnel construction since Roman times. Simple tell-tales, frequently consisting of no more than a pair of wooden wedges, have been used to monitor the rate of opening of a fracture by miners, and the load on timber supports has long been qualitatively assessed by the amount of crushing of the pillow blocks. The builders of the multi-level underground canal systems used for coal transportation in the West Midlands of England in the 18th century were just as concerned with the maintenance of a precise grade as are the constructors of a modern subway and their monitoring systems reflected that concern. These early investigations, were, however, concerned primarily with function, and to a lesser extent with safety. The current objectives of monitoring systems in tunnel construction are somewhat wider.

## THE FUNCTION OF THE MONITORING AND INSTRUMENTATION SYSTEM

A modern instrumentation and monitoring system designed for a tunnel should fulfill the following functions:

(1) It must provide current information about the potential safety of the excavation for the protection of the personnel working there.

(2) It must provide information about factors which may influence the security of surrounding structures so that the safety of the general public is not compromised.

(3) It should provide information for the designers which will help them to assess the validity of the design parameters, and hopefully enable them to be refined.

(4) It should provide a means of assessing the contractors' performance and his adherence to specifications.

(5) It should provide information which can be used by the designer or owner as a means of defense against spurious claims by third parties.

Some of the specific requirements of a monitoring system which can be summarized under the five headings referred to above are:

(1) <u>Safety in the excavation</u>

- Warning of rock or soil collapse
- Warning of support or lining failure
- Warning of inrush, or possible pressure loss in a compressed air tunnel

(2) Safety around the excavation

- Warning of foundation or structural damage in adjacent buildings
- Warning of sudden subsidence in a thoroughfare
- Warning of damage to utility lines

(3) Refinement of Design

- Efficiency of the excavation technique
- Performance of the temporary support system, if appropriate
- Performance of the permanent support system
- Validity of initial assumptions about soil or rock behaviour

(4) Contractors' Performance

- Extent of rock or soil disturbance
- Effectiveness of the installation of the primary support
- Optimization of the time of placement of the permanent support
- Effectiveness of dewatering programs
- Maintenance of alignment and grade

(5) Legal Protection

- Behaviour of the rock or soil mass surrounding the excavation and its interaction with adjacent structures or utilities
- Vibrational effects on adjacent structures

When the instrumentation and monitoring system installed on a tunneling project fulfills the majority of these functions, it makes an extremely positive contribution to the successful completion of the project and represents a highly cost effective investment.

## THE DESIGN REQUIREMENTS OF AN INSTRUMENTATION SYSTEM

In order to be considered a useful factor in the completion of the project by all the parties involved, the instrumentation system should possess the following features:

(1) The equipment must be reliable and capable of providing useful data for the duration of the project.

(2) The instruments and all ancillary equipment must be sufficiently robust to survive in a construction environment.

(3) The instrumentation must be capable of being properly installed with the minimum delay to the contractor.

(4) The instrumentation must be capable of being read with minimal interference to the construction work.

(5) The system must provide redundant data.

(6) The equipment must be field serviceable.

(7) The system cost must be reasonable.

The system cost is of less significance than the preceding factors, as the capital cost of an instrumentation system is generally insignificant in relation to the total project cost. Attempts to reduce the cost of an instrumentation system of a given size will often increase the installation time and decrease reliability and effectiveness.

THE SELECTION OF INSTRUMENTATION AND MONITORING SYSTEMS

It is not possible to define a set of universal specifications for instrumentation systems used in tunnels. Conditions vary greatly, and the system which would be necessary for a shallow tunnel in a given soil or rock type in an urban environment might be totally inappropriate for use in a similar rock or soil at greater depths or under water.

Many selection parameters can be established for instrumentation and monitoring equipment and these are of use to the owner or design engineer who wishes to achieve a specific objective. These could include systems, for example, whose selection is based on whether the equipment is permanently installed, or whether it is movable; or whether or not the equipment is located in bore holes. However, because of the practical considerations relating to the problems of working underground in what are, typically, very congested conditions, the two main groups of instrumentation systems may be considered to be:

(I) Monitoring equipment installed from within the excavation;

(II) Monitoring equipment installed from outside.

Each group has both advantages and disadvantages which significantly influence the feasibility and desirability of employing particular equipment in a specific location, and these can be summarized as follows.

I. Installation From Within The Excavation:

Advantages

(1) Close to work place; a rapid response to changing conditions can be made.
(2) Can be installed as conditions change.
(3) Can usually be installed on a demand basis without the need for extensive additional work outside the excavation.
(4) Installation is possible in built-up areas without the risk of interfering with activities or structures on the surface.
(5) Installations can be accurately located to examine specific geologic or structural features.

Disadvantages

(1) The installation of equipment close to the working face invariably causes some disruption to the production cycle.
(2) The excavation must exist before the equipment can be installed; therefore, some disturbance must already have taken place.
(3) The equipment is located in a hostile environment; thus, the risk of damage is high.
(4) The collection of data may interfere with production.
(5) The data collection procedure may be error prone because of the typically adverse environmental conditions.
(6) The installation work requires the willing cooperation and synchronization of several groups who may have differing objectives.

II. Installation From Outside The Excavation:

Advantages

(1) The instruments can be installed before any disturbances due to excavation develop. Thus a complete history of events at the site can become available.
(2) The installation work causes minimal or zero interference with the excavation work.
(3) The collection of data is typically made on the surface above the tunnel and thus causes no interference to the work below.

(4) The data is collected in an environment which is significantly more benign than the one existing in the tunnel. The data thus tends to be of better quality.

(5) Observations can readily be made on specific surface features which may be subject to damage.

Disadvantages

(1) The transfer of critical data for purposes of safety or design modification is less rapid.

(2) If the excavation is not close to the surface, the cost of placing the instrumentation close to the tunnel may be excessive in relation to the amount of data being collected.

(3) If the tunnel is not close to the surface, it is very difficult to ensure that the instrumentation is precisely where it is required.

(4) If the tunnel lies beneath a street, there is a high probability that the installation work will interfere with traffic flow.

(5) In the case of (4), the collection of data will also interfere with traffic.

(6) Generally speaking, it is only possible to study large scale effects rather than specific features.

(7) It is not possible to examine directly the behavior of linings and support systems.

Due to the specific advantages and disadvantages listed above, the instrumentation systems for relatively shallow subway excavation in urban environments typically combine elements of both surface and subsurface installed systems. However, engineers tend to rely only minimally on surface monitoring systems for deep subway tunnels. The same is true for tunnels driven in mountainous regions, or underwater, where it would clearly be impracticable to use monitoring equipment installed from the surface.

In general, it is desirable to do as much of the monitoring as is practicable outside the excavation, and thus to minimize the amount of interference to the contractors' operations. Whenever possible, the work undertaken inside the tunnel should require as little installation work as feasible, i.e., the collection of convergence data and the use of simple rock bolt extensometers. Obviously, certain types of monitoring can only be done from inside the tunnel, i.e., studies of support or lining behaviour, but it must be remembered that the performance of internal features such as rock bolts or other support systems, can often be inferred from deformation data collected from outside the tunnel.

PROCEDURES AND EQUIPMENT AVAILABLE FOR
MONITORING TUNNEL PERFORMANCE

A wide range of techniques and equipment of many different types are available for monitoring the various significant parameters associated with the construction of tunnels. The procedures or categories of equipment which are currently applicable are:

(1) Convergence measurements within the tunnel to monitor the relaxation of the soil, rock, support system, and lining.

(2) Borehole extensometers installed from either inside the tunnel or from the surface to monitor rock and soil mass displacements.

(3) Precise levelling equipment, standard surveying procedures or EDM equipment to monitor the development of subsidence troughs above shallow tunnels and measure deformations in adjacent structures.

(4) Surface strain and tilt measurements used to define the extent of surface disturbance and to assess the likelihood of damage to critical surface structures or utilities.

(5) Borehole inclinometers installed from the surface to monitor the development of subsurface lateral displacements caused by the horizontal relaxation of the soil or rock mass into the tunnel.

(6) Piezometers installed from the surface to monitor the effect on the local groundwater regime of the tunnelling operation and the effectiveness of any watering programmes which may be called for.

(7) Dynamometers installed on rock bolts to monitor the effectiveness of the installation and anchoring procedures and also to provide information about the total loads being carried by the support system.

TABLE 1

| FEATURES \ SYSTEM | Convergence monitoring | Extensometer, surface, multi | Extensometer, subsurface, multi | Extensometer, subsurface, R.B. | Precise levelling on surface | Precise surveying on surface | EDM equipment on surface | Surface strain monitoring | Surface tilt monitoring | Inclinometer | Electric piezometer | Pneumatic piezometer | Standpipe piezometer | Rock bolt L.C. electrical | Rock bolt L.C. mechanical | Strain in lining | Pressure in lining | Blast & vibration monitoring |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Best installed by production personnel | | | | ● | | | | | | | | | ● | | ● | | | |
| Best installed by supervisory staff/inspectors | ● | | | | | | | | ● | | | | | ● | | ● | ● | ● |
| Best installed by specialists | | ● | ● | | ● | ● | ● | ● | | ● | ● | ● | | | | | | |
| Read by supervisory staff/inspectors | ● | ● | | ● | | | | ● | ● | | ● | ● | ● | ● | ● | | | |
| Read by specialists | ● | ● | ● | | ● | ● | ● | ● | | ● | | | | | | ● | ● | ● |
| Installation causes significant delay to tunnel | | | ● | | | | | | | | | | | | | ● | ● | |
| Installation causes no significant delay to tunnel | ● | ● | | ● | ● | ● | ● | ● | ● | ● | ● | ● | ● | ● | ● | | | ● |
| Reading causes significant delay to tunnel | | | ● | | | | | | | | | | | | | | | |
| Reading causes no significant delay to tunnel | ● | ● | | ● | ● | ● | ● | ● | ● | ● | ● | ● | ● | ● | ● | | | ● |
| Remotely readable | | ● | ● | | | ● | ● | ● | | | ● | ● | | ● | | ● | ● | ● |
| Permanently installed equipment is robust | ● | ● | | ● | ● | ● | ● | ● | ● | ● | | | ● | | ● | ● | ● | |
| Permanently installed equipment is markedly fragile | | | ● | | | | | | | | | ● | | | | | | ● |
| Equipment field servicible | ● | ● | ● | ● | | | | ● | | | | | ● | | | | | |
| Equipment can be fabricated at site | | | | ● | | | | | | | | | ● | | | | | |
| Equipment very reliable | ● | | | ● | ● | ● | | ● | ● | | | | | | ● | | | |
| Results unambiguous within system limitation | ● | ● | ● | ● | ● | ● | ● | ● | | ● | | | | | | ● | | ● |
| Total cost/installation low $250 | ● | | | ● | ● | | ● | ● | ● | | | | ● | | ● | ● | | |
| med $250-1000 | | | | | | ● | | | | | ● | ● | | ● | | | ● | |
| high $1000 | | ● | ● | | | | | | | ● | | | | | | | | ● |

(8) Strain and load measurements made within the support or lining systems to monitor the development of deformation or stress within shotcrete or reinforced concrete linings to indicate the time rate and distribution of load around the tunnel.

(9) Blast and vibrational measurements monitor the effect of blasting or other excavation operation on adjacent structures for the purpose of insurance or litigation protection.

Table 1 is a presentation of the applicability of the various monitoring and instrumentation systems under a range of situation and tunneling conditions. It should be noted that the table refers to general conditions and there are obviously specific situations where a procedure or technique can be employed under what are apparently unfavorable conditions. For example, if other underground openings exist in the vicinity of a proposed deep tunnel, geodetic leveling can be employed to monitor the development of the subsurface subsidence trough even though there may be little or no subsidence discernible at the surface.

A few of the more important characteristics of some of the nine techniques referred to above are discussed below.

## 1. Convergence Measurements

Convergence monitoring in tunnel construction provides a simple and effective method of assessing the time dependent stability of the opening. The monitoring procedure involves measuring the change in length of a number of diameters or chords of the tunnel at a particular cross-section. The simplest procedures is to measure on the horizontal and vertical diameters and to subsequently plot the cumulative deformation versus time.

Greater resolution can be obtained by making measurements on diameters spaced 60°, or 45°, apart or by combining diametral measurements with measurements taken along chords terminating at the diametral end points. However, even in the most simple application, the technique provides an unambiguous indication of how the support system and the surrounding rock or soil mass is behaving.

A number of convergence monitoring devices (internal extensometers) are commercially available. Practically speaking, the most effective are those which use flexible tapes or wires as the gage length, rather than rigid or telescoping rods, as the latter devices are cumbersome and prone to damage. It should also be noted that, even with the relatively constant temperatures which prevail underground, the convergence measuring devices function as precise mechanical thermometers. Therefore, it is essential to apply temperature corrections when making measurements. For the best results, the system selected should incorporate a measuring instrument which is rigidly clamped when making measurements, and a tape or wire which is secured at the other end of the bay by a single point mounting. With a well designed instrument, the system repeatability and accuracy should be better than ± .002 inches (± .05mm), and the range should be about 2 inches (50 mm), before resetting is necessary.

The number of installations and the frequency of taking measurements with convergence monitoring equipment will depend essentially upon the conditions in the specific tunnel under investigation. In competent, self supporting ground, convergence stations may only be installed every 10 - 20 rounds, or whenever the ground conditions change, and in this situation readings may only be made every 1 - 2 weeks, or at longer intervals if there is no sign of movement. However, in the case of a tunnel in unstable ground where the convergence data is being used to define the type of support system, it is quite conceivable that convergence stations could be installed every 3 - 5 rounds, and measurements taken daily, or even every shift, until the face is more than 5 diameters away, at which time the reading frequency would decrease.

2. Bore Hole Extensometers

Bore hole extensometers are extensively used to monitor the behaviour of the rock or soil mass around a tunnel. The units can either be installed from the surface in the area into which the tunnel will ultimately be advanced, or they can be installed from the tunnel as soon as the collar location has been exposed. Bore hole extensometers installed from inside the tunnel are generally fairly short, less than 3 tunnel diameters in length, and they may be either single or multiple position units. The former have the advantage that they are cheap, robust, readily fabricated at the job site using slightly modified rock bolts, and can be installed by any person familiar with the installation of rock bolts. The principal disadvantage of this simple unit is that only one measurement can be made in a hole. Therefore, in order to obtain deformation data at several depths, a cluster of rock bolt extensometers must be installed.

In general, it can be assumed that multiple position extensometers take longer to install and require a specialized installation crew, and that their more complex collar station is much more susceptible to damage. They do, however, offer the advantage of being able to provide several measurements in one hole, and are capable of installation at depths considerably greater than those possible with single position rock bolt extensometers.

When extensometers are to be installed from the surface, it is not practicable to use single position units. In these situations the costs of the installation are such that it is mandatory to obtain the maximum amount of information from each bore hole. Thus multiple position units are invariably selected. Multiple extensometers can be either manually or remotely read units; the latter offer significant advantages if the collar of the instrumented hole is, for example, in the middle of a busy street. In such a situation the electrical readout cable can be routed to a more convenient location. Manually read units are less costly and are just as accurate as the extensometers incorporating an electrical readout. They also tend to be rather more reliable.

Irrespective of the type of readout selected for the extensometer, if the unit is well designed and correctly installed, the repeatability of the unit should be about $\pm$ .002 inches, ($\pm$ .05 mm), and the overall accuracy should be better than $\pm$ .005 inches ($\pm$ .13 mm). An instrument meeting these specifications will be adequate for the vast majority of tunnel instrumentation projects.

A considerable amount of information relating to the characteristics of all the equipment referred to above can be found in the literature. With the exception of the convergence monitoring equipment and the bore hole extensometers, the various devices are very widely employed in many other engineering disciplines and a detailed discussion of their performance characteristics is not necessary.

## CONCLUSIONS

The equipment and techniques currently available for monitoring the behaviour of tunnels are adequate for the majority of projects. Providing that reasonable care is taken at the time of installation, and assuming that the appropriate protective measures are taken while construction is continuing, valid information can be obtained and used to fulfill the objectives referred to earlier in this text. However, all too frequently a considerable investment in terms of time and money is made by the owner or the designer and the net result is either the failure of many of the units due to a multiplicity of reasons, or the acquisition of a mass of data which is frequently not put to any useful purpose.

The problem of instrument failure is unfortunately not an uncommon one. Setting aside the problems encountered by inexperienced personnel attempting to install equipment which the manufacturers claim to be a standard production unit, but which subsequently turns out to be a one-of-a-kind prototype, many of the instrument failures which plague this work are due simply to gross carelessness and the breakdown in communications between the engineer, the field supervisor, and the contractors' site personnel.

This problem, which concerns the attitudes and priorities of the contractor, the field supervisor and the design engineer, is primarily one of a lack of mutual understanding, comprehension or appreciation. The solution is primarily one of further education. The designer must realize that whatever is required to be installed in the field in the midst of the production cycle must be planned realistically and cannot impinge too seriously upon the production work. If it does, it runs the risk of being ignored as superfluous, or of being tacitly sabotaged. In either case the result is the absence of data which could have been beneficial to all the parties involved. Similarly, if the field supervisory personnel can be made to understand that the functions of the instrumentation systems in the tunnel are to aid in maintaining safe working conditions, to improve the design, and to provide the owner or designer with some factual basis upon which to assess the validity of claims, then it should be possible for the engineering personnel to obtain the data they require without having to engage in a continuous war of attrition. Furthermore, if the contractor can be convinced, possibly by contractural modifications, that the instrumentation and monitoring program has not been specifically designed to delay and irritate him and to diminish his profit, or to cater to some engineer's desire to justify his existence, then it may be possible to ensure that this type of work is sucessfully completed with the minimum of delay and inconvenience, and the maximum benefit to all concerned.

The second major problem of obtaining useful information from the types of monitoring systems discussed above is one related specifically to the means of data collection.

Experience on numerous projects has shown that the most efficient method of gathering information of this type is to have one group responsible for the design, installation and acquisition of the data. In this manner there is a vested interest in ensuring that the entire system works satisfactorily and that the results obtained are of good quality. All too frequently a functioning system is handed over to a second party and there is then a rapid decrease in the volume and/or quality of data. The reason for this is often that, in a large organization with many responsibilities, the task of data collection is ultimately assigned to quite junior technical personnel who have neither the rank, knowledge nor desire to ensure that the data gathering procedure is prosecuted in the most efficient manner possible. The conditions under which measurements have to be made on construction projects are usually far from ideal, and experience has shown that the most effective way of obtaining data of consistent quality is to make the engineer or technologist who installed the equipment responsible for gathering the data. This has the effect of greatly reducing the number of data losses which are attributed to equipment malfunctions or faulty installation procedures.

It is the considered opinion of the author that, if a more logical and understanding approach is taken by all the parties who are of necessity involved in the collection of this type of data, then significant benefits will accrue to the owners, contractors and engineers associated with tunnel construction.

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# MODERN BLASTING IN AN URBAN SETTING

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## Introduction

The Atlanta Research Chamber was appended to the construction of the Peachtree Center Station on the CN120 contract of the rapid transit system in Atlanta, Georgia. The function of the Atlanta Research Chamber is to provide demonstrations of several aspects of modern tunneling practices and to promote the advance of such techniques in the United States. In keeping with the general theme of these demonstrations, this monograph will offer comments on modern underground blasting techniques. Included will be comments relating to a new technique for the control of fracture propagation. The technique involves the use of "scribing" or "notching" of drill holes to provide stress concentrations that enhance crack propagation in the plane of the notches in preference to other directions. This paper also offers a discussion of the control of ground vibrations and airblast overpressures as related to this project.

## Blasting Specifications

Although it would be highly desirable to make an exact science of blasting, this does not seem possible. It would be fair to call modern blasting a "technical art", whereas in past times there has been decidedly more art than technology.

The nature of construction contracting in the United States does little to promote advancements in blasting skills, although the nature of construction practice does encourage good safety practices, and penalizes carelessness that would lead to personal injury. Unfortunately, the practice of awarding contracts to low bidders provides a financial incentive (sometimes an economic necessity to the low bidder) to place production rates above any other consideration or goal that the Owner might have regarding the outcome of the work.

It would seem self-evident that a good specification would warn all bidders of the degree of caution that the Owner would wish to have exercised during blasting and excavation. As a minimum for underground chambers, specifications should indicate that special controlled blasting techniques will be required for all perimeter surfaces. It is even more definitive if certain limits are placed specifically on hole dimensions, hole spacings and explosive charge concentrations. For example, a specification might read that hole diameters will be restricted to the range of 1 3/4 inches to 2 1/2 inches, that spacings will not exceed 24 inches (less, if desired) and that charge concentrations will exceed neither 1/4 pound per foot nor 0.08 pound per square foot of perimeter surface area (less for holes spaced more closely). If any unique methods would be required, the specifications should indicate that fact. For example, the specifications for the Peachtree Center Station advised bidders that the arch haunches would require drilling at 6-inch centers and that only detonating cord or equivalent charge concentrations would be allowed for blasting those holes. If those requirements were not explicitly stated, a contractor would not be able to anticipate the Owner's wishes.

Specifications are often assembled from different sources by staff personnel with no special expertise in the subjects involved. It is this writer's recommendation that all specifications be reviewed by persons with expertise in the various subjects, and reviewed over-all by persons with sufficiently broad experience to recognize the possibility of field problems associated with the specifications. This is a prime example of the old adage that an ounce of prevention is worth a pound of cure.

Part of the state-of-the-art of construction blasting should include writing of good specifications which meet a mutually acceptable standard for both the Owner and the Contractor. There should be no surprises for either.

## Drilling

The first input to any blast is the drilling. In cases of controlled blasting for structural excavations, the drilling is extremely important and can scarcely be over-emphasized. No matter how skilled the blaster may be, his efforts cannot possibly be better than the drilling. The very best he can do is to break cleanly between drilled holes at the perimeter of the excavation, wherever those holes were drilled. He can only do worse than that, - not better. Experienced observers will have noted a definite "learning curve" for the drilling precision at the beginning of every underground excavation. When diligent supervision and effort are applied, the improvement is rapid and the learning cycle is very short. Unfortunately, some learning cycles extend beyond the length of the project!

There have been many advances in drilling equipment in recent years. Some of the most dramatic advances apply to large-hole drilling above ground. Many such operations now use blast-holes in the range of 10-15 inches in diameter, and even larger holes are possible. Some of these holes are now advanced as rapidly as small-diameter holes were done only a few years ago. For underground work, one of the most notable changes in drilling has been the increasing use of hydraulically operated drills. These drills have been able to show penetration rates much higher than those for ordinary pneumatic drills, but require much more care in operation and maintenance. These drills must be kept in very clean condition. Recently, the manufacturers of pneumatic drills have been meeting this challenge and have been developing pneumatic drills that deliver much faster hammer blows, faster rotations and higher air pressures. With fewer maintenance problems, these drills are competitive for many projects.

Some drills are fully articulated and semi-automatic in the sense that they can maintain positions that are perfectly parallel to the original setting, when properly operated. Unfortunately, the operator must still exercise his skill in aligning even that first perimeter hole, and for most equipment, all perimeter holes. These holes cannot be parallel to the alignment of the opening, but must "look out", that is, be slightly inclined outward. There is a need for 6 inches to 12 inches of over-excavation at the end of each blast round to provide room for the next positioning of the drill boom. Thus, even the most skillful work produces a "sawtooth" profile rather than a continuously plane surface.

## Explosives Products

Changes and advances in explosives products have kept pace with other technologies. The most widely known changes have taken place in the development of less sensitive, safer blasting agents in preference to the traditional explosives with a nitroglycerine base. The cheapest, hence most popular blasting agent is AFNO, a mixture of prilled ammonium nitrate and fuel oil (about 6% fuel oil). This product is a free-flowing, loose mixture which is normally packaged in bulk form. It can be poured into vertical holes or placed pneumatically in small horizontal holes underground.

With the addition of water, sensitizers and gelling agents, this product can be made into a slurry or "water gel". A wide variety of slurries, water gels and emulsions is now available in all sizes and sensitivities. These products are relatively insensitive to shock and fire, yet are capable of doing the work of the traditional explosives.

Also within the last few years, more choices have become available for detonators, principally among the non-electric types. In past times, non-electric delay blasting required the use of delay connectors inserted into detonating cord lines at the ground surface. Later came non-electric caps made with the delay element inside the cap, initiated with low-energy detonating cord. More recently has come a new type of initiating cord which consists of a thin film of explosive dust on the inside walls of a small plastic tube. The force of the detonation of this film is so weak that it does not even rupture the tube. It can be held in a person's hand without incurring injury during the detonation. This type of non-electric detonator has the same chief advantages of electric blasting; the initiation of the main charge is accomplished by delaying initiation in-the-hole rather than at the ground surface, and that the stemming is not blown from the hole by the initiating device. If electrical hazards happen to be present, this device offers additional safety.

Optimum Results in an Underground Chamber

The final results of blasting in an underground chamber are determined mainly by the techniques used to develop the final perimeter surfaces. Over-blasting in the inner parts of the excavation can often be overcome by taking extra care at the perimeter. For example, a shattered pilot tunnel does not necessarily pre-determine that the final chamber wil be badly formed or unstable, although such overblasting is undesirable and can be damaging if the pilot tunnel is located near final surfaces. The final walls are preserved in best fashion by:

1. careful drilling of perimeter holes;
2. light loading of perimeter holes with explosive charges that are not concentrated nor fully coupled to the walls of the drill holes;
3. making certain that blasting in the row next to the perimeter is incapable of shattering to the perimeter and beyond.

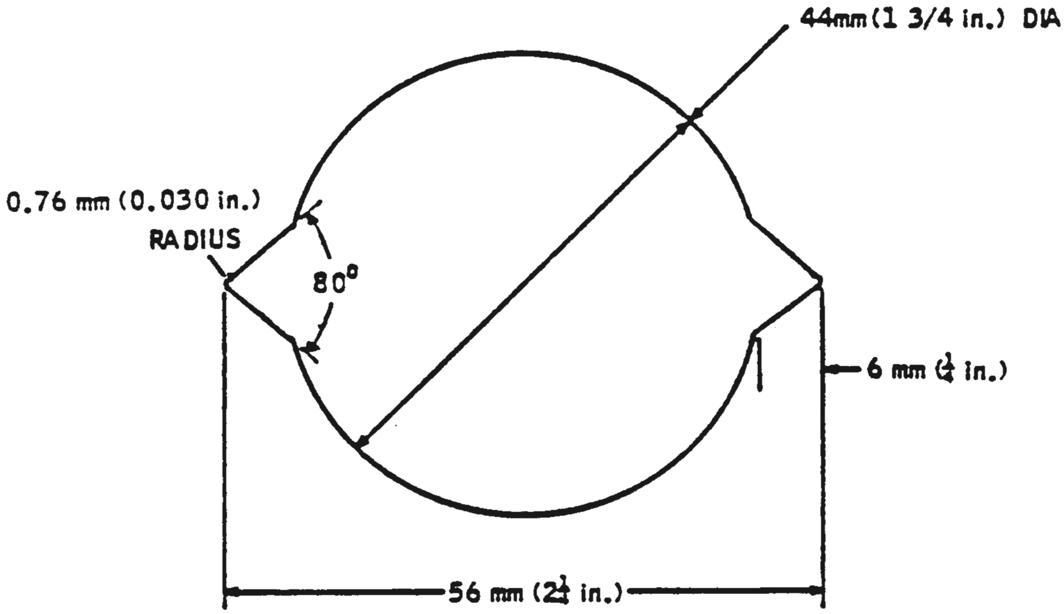
For the Atlanta Research Chamber Project, a pilot tunnel had been excavated earlier through the alignment of the Research Chamber. No controlled blasting methods were used for the pilot tunnel. It was done entirely at the Contractor's option except for general dimensions and alignment.

Enlargement of the pilot tunnel to the full-size Research Chamber would be a relatively simple blasting effort in sound rock such as that found at this site. The main attention would be given to items (1) and (2) above. The pre-existing pilot tunnel could be enlarged easily by "slashing", that is blasting of light charges to cast rock into the pilot tunnel. With careful drilling and the use of light charges, good results could be expected. In order to demonstrate such an expectation, the first round, the last two rounds, and parts of the other rounds were blasted with "standard" controlled blasting of this type, (see Chapter VI for details of blasting in the Research Chamber).

## Fracture Control

In addition to the use of standard controlled blasting techniques, some effort was put into the evaluation of the potential use of a newly developing technique for fracture control. Perimeter holes were scribed or notched on opposite sides in the plane of the desired perimeter surface (see Figure 1). The scribing was done mechanically (although a high-pressure water jet can do an even better job).

It is a well-known principle of fracture mechanics that less energy is required to extend an existing crack than to form a new one. Further, the first crack to form will be that where some flaw or "stress concentrator" exists. Such a stress concentrator can be introduced in the desired plane by cutting small notches into the walls of the drill holes. Experience to date suggests that the notches should be of the order of 3/16 to 1/4 inch in depth in the form of a relatively sharp vee, when done mechanically. If a high-pressure water jet is used, a narrow slot can be cut in the form of a true crack or rock joint.




THE DIMENSIONS SHOWN HERE ARE THOSE RECOMMENDED BY SPERRY ET AL, "CONTROLLED BLASTING EXPERIMENTS AT PORTER SQUARE PILOT TUNNEL" (RETC PROC., ATLANTA 1979). THE WORK IN THE ATLANTA RESEARCH CHAMBER PRE-DATED THE WORK IN THE PORTER SQUARE PILOT TUNNEL BY A FEW MONTHS.

SUGGESTED DIMENSIONS FOR NOTCHES TO CONTROL CRACK INITIATION.

FIGURE 1

PHOTO 1

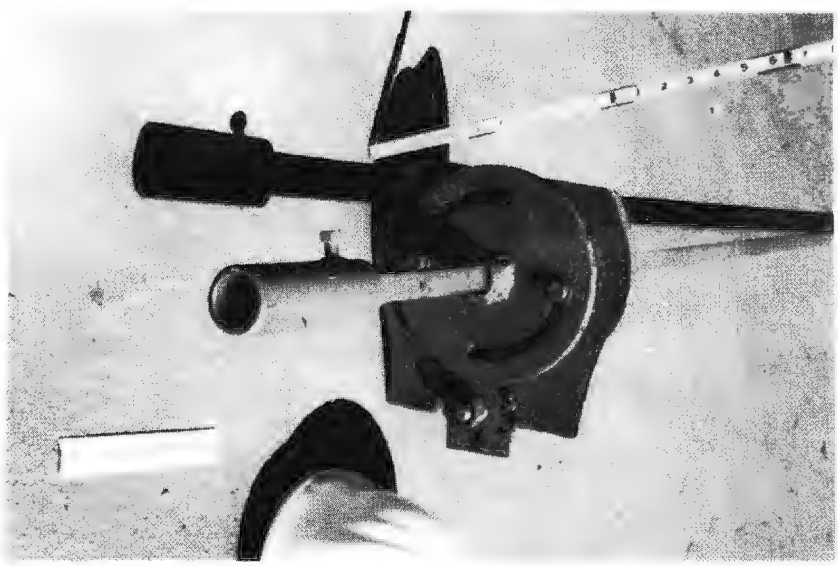


PHOTO 2

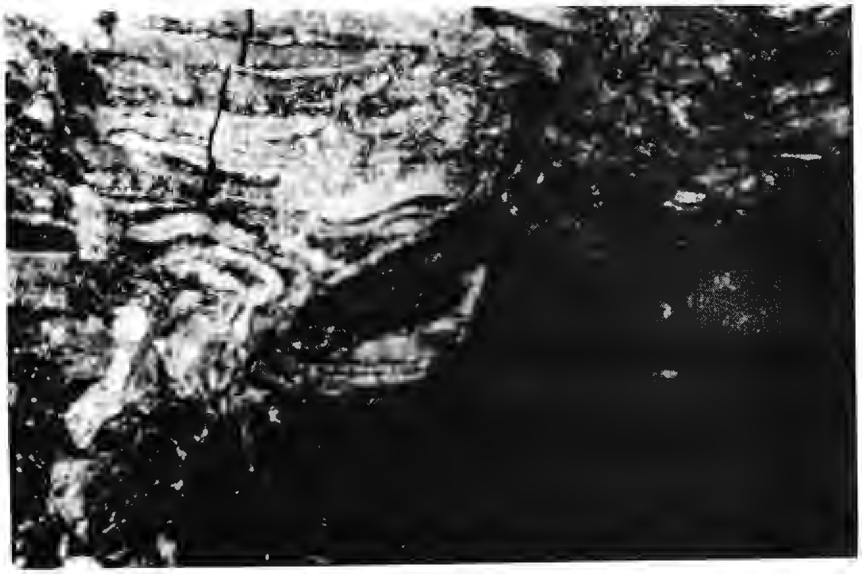
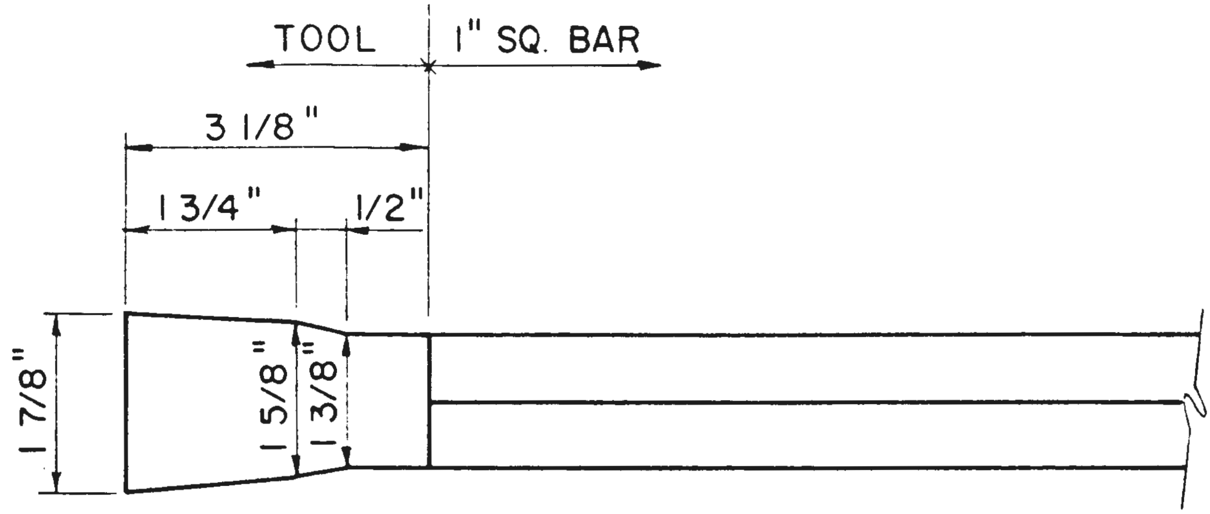


PHOTO 3

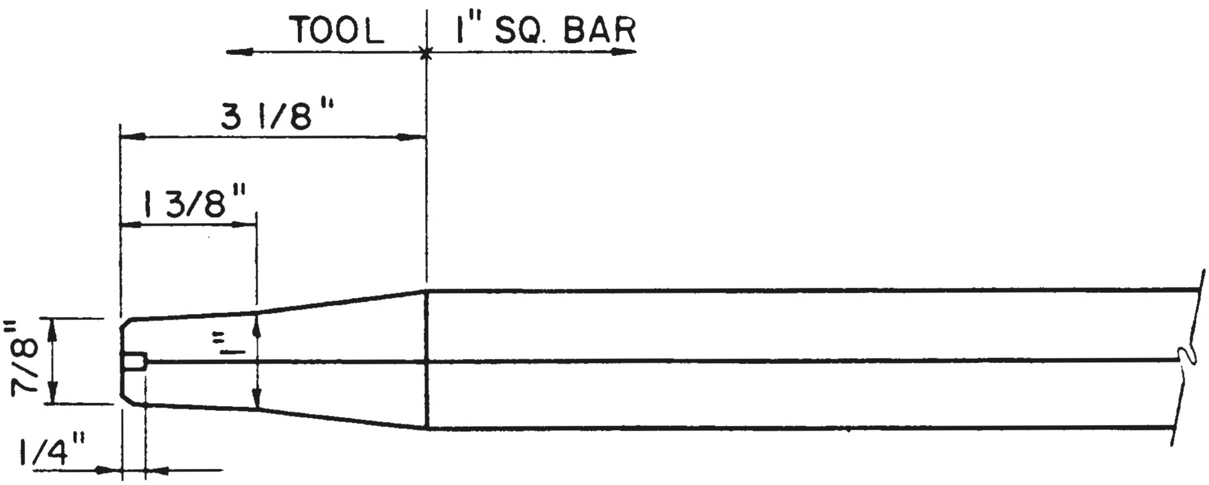
Photos 1 and 2 show a tool for accomplishing the scribing mechanically. Photo 1 is an over-size bit ground on two sides so that the bit collar and two points will enter the previously drilled hole and remain centered in the hole (see also Figure 2). The other two points extend beyond the perimeter of the drilled hole and cut notches 1/4 inch deep into the opposite walls. The bit was attached to a four-sided section of drill steel as seen in Photo 2. The photo also shows a jig or template that prevents the drill steel from rotating as the tool moves down the hole. The template rides along the drill boom as the steel enters the hole.

Unfortunately, the rock at the Atlanta Research Chamber site proved to be anisotropic and prevented the making of a full assessment of the technique. However, the writer was evaluating the same technique simultaneously on two other projects and can provide additional comment. The rock at the Atlanta site is a strongly foliated gneiss with relatively flat-lying planes of foliation. Thus, there was a natural tendency for the rock to break along foliation planes, and to resist breaking perpendicular to those planes. The underground opening therefore tended to approach the shape of a rectangle rather than a precisely-formed horseshoe. Micaceous minerals tended to gather in relatively prominent thin layers along the planes of foliation. This produced a very anisotropic rock with hard-and-soft thin bands.

An illustration of the dramatic influence which the foliation had on a crack formation can be seen in Photo 3. One might ordinarily expect a crack to extend in a straight line from one drill hole to the next. Here we see that a crack was attempting to propagate in a vertical plane from the drill hole downward. However, instead of forming a straight line, the crack




## PLAN




## ELEVATION

SCRIBING TOOL BIT.

FIGURE 2

followed the contorted path of a tight fold in the rock, extending to the lefthand border of the photo.

Over-all, the rock proved to be sound and durable, so that it was not easily shattered or broken beyond the desired perimeter, except for the upper corners of the arch. This material, then, proved to be attractive to the Contractor, allowing leeway in his operations, but not providing a highly illustrative research site for evaluating new blasting techniques.

A low-energy technique such as this would logically tend to show more dramatic results when used as a smooth-blasting technique, where rock has been removed previously to the extent of leaving only a small burden in front of the perimeter holes. For such a case, relatively little energy is required to dislodge a perimeter slab, once crack propagation has separated the slab from the final wall surface. For pre-splitting, there is additional confinement in the form of an extended (semi-infinite) burden. The charge must therefore be large enough to overcome this additional "beam strength" as well as the fracture strength of the rock material. The results are, therefore, not as dramatic for pre-splitting as for smooth blasting.

For small-burden blasting, such as smooth blasting, the hole scribing or notching technique offers the possibility of reducing the charge concentration to about 1/4 or 1/5 that of the normal condition, and a simultaneous increase in hole spacing to about twice that of the normal condition. These figures are based more on the results produced in concrete than those from the anisotropic rock at the research adit in Atlanta.

For optimum results, it is important that the notch or slot in the drill hole be sharp, well-formed, straight and in the plane of the desired breakage. It is equally important that the right kind of stemming be provided in order to contain the explosive gases long enough to allow the cracks to propagate. The quick, high-velocity impact of typical underground blasting without stemming will not produce acceptable results.

## Control of Ground Vibrations and Airblast Overpressure

The first step in the control of vibrations and airblast is that of determining the appropriate levels of intensity for the site conditions. The decision is not automatically made by determining a damage threshold. The public response must be considered, a question that is far more complex than that of structural responses. Are you willing to accept many complaints and damage claims from the public? Are you willing to accept the cost of minimizing these?

It is generally accepted that the best indicator of the potential for damage to structures from blasting vibrations is peak particle velocity. Standards have become rather widely accepted, using poor plaster in a residential structure for the threshold of damage. A peak particle velocity of 2.0 inches per second has become widely accepted as being safe for poor plaster, with the probability of damage increasing as the particle velocity increases beyond that threshold. It is often stated that minor plaster damage may be expected at about 4.0 inches per second, and major damage may be expected at about 8.0 inches per second. (The decimals do not indicate precision.) Several states and agencies have established regulations limiting particle velocities to 2.0 inches per second at public and private buildings. Limitations for concrete and engineered structures have sometimes been more liberal, often in the range of 4.0 to 8.0 inches per second.

It is this writer's contention that no single descriptor of particle motion is sufficiently accurate to cover the wide range of vibration characteristics that we must deal with in blasting, and that the time history (frequency and duration) needs often to be considered carefully. On the one end of the scale, there are those who claim to have observed instances of damage at particle velocities lower than the "standard" 2.0 ips. At the other end of the scale, this writer has observed many cases where old residences were not damaged by vibrations with particle velocities in the range of 10 to 20 ips. The tendency toward damage would increase with low-frequency vibrations impinging on a tall, weak structure with a low natural frequency, and decrease if the same structure were subjected to very high frequencies.

Interestingly, there is a need also to demand serious review of the observational procedures that lead to the conclusion that "damage" has occurred. This issue is often overlooked under the false assumption that any person can identify "damage", especially if it happens to be the "eye-witness" type. Those who understand this problem can only wonder how many cases of false identification have crept into the professional literature without proper verification. It is common knowledge that many accounts by "eye-witnesses" describing their observation of damage occurring are contradicted by irrefutable evidence that the "damage" existed before such "observation" took place.

A review of response spectra theories will demonstrate the importance of the time history of vibration as it relates to the natural frequency of a given structure. Such response analyses are commonly made for large critical structures when it is anticipated that the structures must survive significant vibrations, e.g., the response of a dam to a design earthquake.

Up to the present, it has been widely accepted that blasting vibrations cover a relatively narrow range of vibration frequencies and that it is acceptable to use a single value of particle velocity to identify the damage potential for a given type of structure, e.g., a residence. It is this writer's contention that such a view is too simple, and can cause problems at either end of the frequency spectrum. For the case of low-frequency vibrations impinging in a structure with a low natural frequency, the criterion may not be sufficiently conservative. For the case of high-frequency vibrations, the criteria may be so conservative as to increase costs prohibitively, or rule out the feasibility of conducting blasting operations that are quite safe in reality.

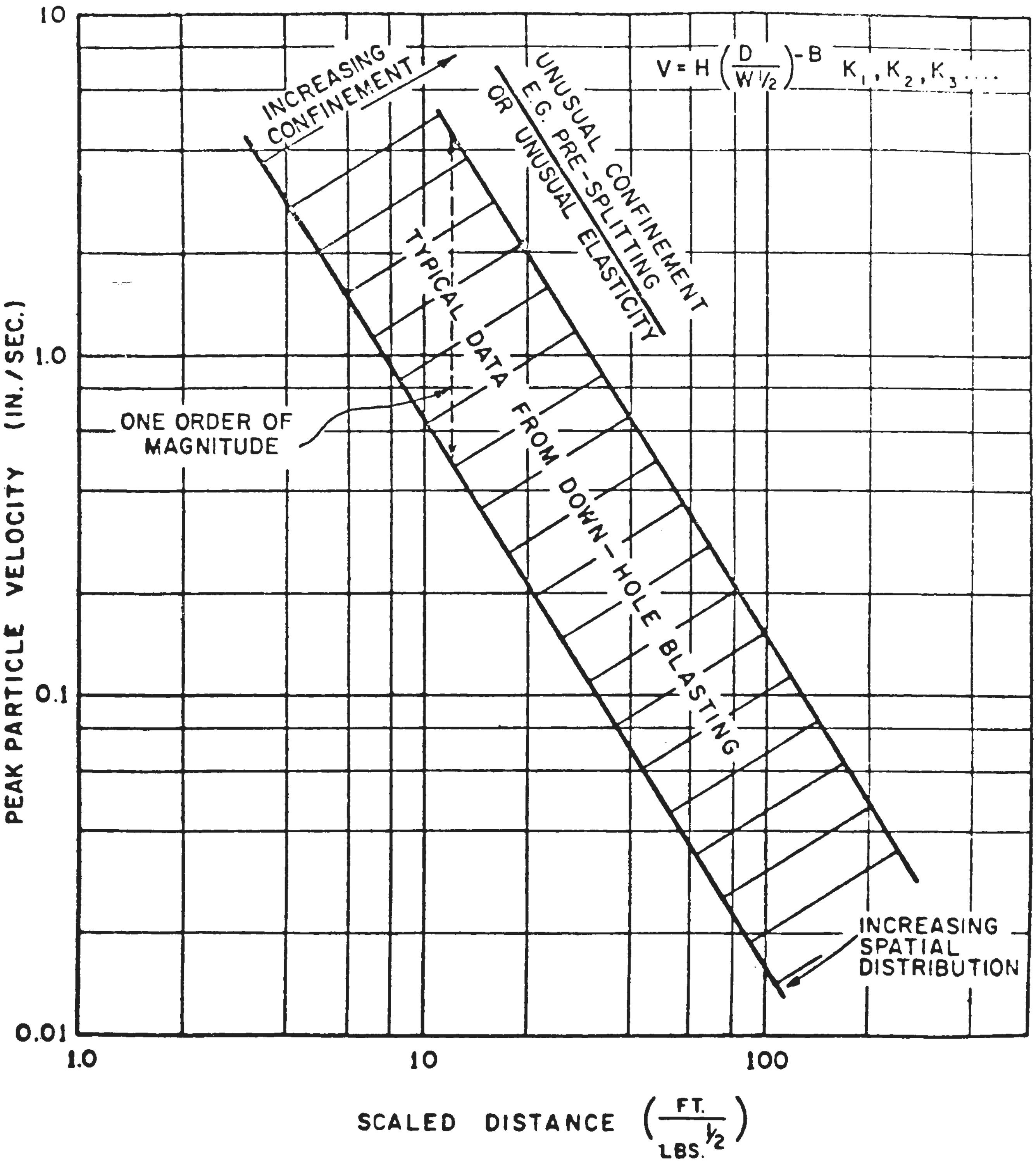
The "safe level" of 2.0 ips has become so deeply ingrained, and applied so widely to all manner of structures and materials in the past, that it seems like heresy even to consider a particle velocity of one to two orders of magnitude higher. Yet, this writer has had occasion to measure particle velocities of several hundred inches per second in engineered structures that were very important structures, - critical to the industrial operations of the country involved. An important aspect, of course, is that the vibrations occurred at high frequencies with relatively lower strains than would be expected for low-frequency vibrations. As one of several instances, the walls of a large powerhouse ($200 million), supplying roughly half of its country's power, were subjected to particle velocities of the order of 275 ips at a high frequency, without incurring damage.

It does not solve the problem for a project involving blasting operations in a highly populated area that a certain particle velocity will not cause damage to residences or commercial buildings. A simple statement regarding the potential hazard of vibrations to structures does not begin to describe or satisfy the over-all problem of blasting in a populated area. Paradoxically, the more serious part of the problem is not structural, although it is thought to be by the community at large. The chief difficulty is the sensitivity of people to sounds and vibrations and their lack of knowledge of the normal static (non-vibratory) physical forces which are involved in their daily lives and which act on the structures they occupy. Consequently, we must devote at least as much time to the study of people as we do to the study of structures.

It has been observed that the average person can easily feel a motion which is of the order of 1/100 to 1/1000 of that needed to cause damage to his home, and will consider it "severe" at about 1/5 to 1/10 of that level. In actual practice, all rules for predicting motion response fall apart when sound effects accompany the motion and the motion is of short duration. In such instances, the average person forms a judgment based largely on his psycho-acoustical responses and is usually unaware of the important distinction between the characteristics of the motion alone and the sound effects that might accompany that motion.

Certain regulatory groups and agencies have recognized the problem associated with the response of the public to blasting operations, and have limited the ground vibrations to lower levels than would be needed to prevent damage to buildings. For example, the proposed Surface Mining Act for the United States contains a limit to a particle velocity of 1.0 ips, the value chosen for the work in downtown Atlanta on the CN120 contract.

Although it is true that there will be more complaints and more damage claims associated with higher vibration levels, one can only reduce, NOT eliminate complaints and claims by




ORIARD PREDICTION CURVES FOR BLASTING VIBRATIONS.

FIGURE 3

reducing the vibrations to a level generally regarded as "acceptable". This is due to the complexity of the human response. Some complaints and claims will come from those who are firmly convinced that they were severely shaken because they heard the sound effects of vibrating panels, doors or windows, even if the motion alone was below the threshold of human perception. Others may not even have been in the area when the blasting occurred, but upon learning that blasting had taken place in their absence, made careful inspections and discovered "damage", unaware that the "damage" was a condition that existed prior to the beginning of the blasting operations.

Thus, it is this writer's opinion that it is unrealistic to conclude that we can eliminate blast damage complaints and claims merely by reducing allowable vibration limits to values that are thought to be acceptable by motion criteria. More contact with the public is needed. This could be, for example, in the form of pre-blast building inspections and dissemination of information prior to the start of blasting operations.

The simplest way to limit ground vibrations generated by blasting is to limit the quantity of explosive detonated at any given instant of time. This quantity is usually called (inexactly) the charge weight per delay, under the assumption that all detonators of the same nominal delay interval will detonate simultaneously.

Recent advances in product development have greatly increased the number of delay intervals available to explosives users. This change has occurred for both electric and nonelectric delays. Judicious use of either system now allows the blaster a nearly unlimited choice of delays, although some expertise is required in the planning and execution of the more complicated delay systems. Thus, there usually is not a severe financial penalty associated with blast designs limited to only a few holes per delay. Sometimes, there are technical limitations that introduce safety hazards, such as cut-off holes if too many delays are spread out over too long a time period.

If potential bidders are advised of the vibration limits that will be imposed on a project, it is usually unnecessary to provide any additional background data to assist them in determining the manner in which charges must be limited to keep below the specified vibration level. At the present state-of-the-practice, nearly all blasting contractors are well informed of procedures for predicting the quantities of explosives that would generate a certain vibration level at a given distance. This writer has published such predicted curves for general use of the industry, as seen in Figure 3. Similar information is available through publications of the Bureau of Mines. Explosives suppliers and independent consultants routinely offer such guidance.

Of course, it is essential that vibrations be monitored when blasting is taking place in a highly populated area. Two purposes are served by such monitoring. One is to ensure that vibrations remain at non-damaging levels, and/or below the criteria established for the work. The second is to be able to prove what vibrations occurred in the event that damage claims are made.

Instrumentation advances have made easier the task of monitoring routine blasting operations, and have made possible the task of measuring "exotic" vibrations, not possible in times past. For the routine monitoring, there is an expression in the industry that "smart" instruments now exist for this work. Instruments are now capable of monitoring all vibrations on a continuing basis, triggering a recorder when the vibrations exceed a certain pre-determined threshold, then turning off after a pre-determined time interval. Recording can be done either on magnetic tape or photographic strip charts. The same instruments can provide a digital read-out of peak particle velocity for instant reading, so that the information can be passed immediately to the blasting supervisor. Some of these instruments are equipped to monitor airblast overpressures simultaneously with the ground motion.

More than a few years ago, it was difficult to measure vibrations at extremely high frequencies and extremely high velocities or accelerations. Now, equipment is readily available for these tasks, though expensive and not routinely used. Recently, for example, this writer was able to measure accelerations to 50,000 times gravity and to 30,000 Hz (cycles per second) with miniature accelerometers recently developed. Velocity gages are limited presently to about 2000 Hz. In all cases, data can be placed easily on magnetic tape (if the budget is available) for computerized analysis and output, such as response spectra.

A similar history of criteria development has taken place for airblast overpressures. Early research led to the conclusion that an overpressure of the order of 1.0 psi would almost certainly cause some window breakage (the first sign of damage), and that it would be acceptable to limit values to the range of 0.1 to 0.5 psi. In recent times, it has been reported that minor plaster damage has been associated with relatively low airblast overpressures. For such reasons, as well as the desire to reduce public complaints, there have been recent proposals to require a reduction to limits of the order of 0.001 to 0.01 psi.

The increase in the conservatism is even greater than suggested above, for the reason that better instruments for detecting airblast overpressures now indicate that earlier instruments were not detecting certain portions of the very-low-frequency energy. Thus, we are simultaneously measuring more energy and reducing the limits.

Some of the measures that serve to control ground vibrations have a similar function in the control of airblast overpressures. For example, the use of delay detonators, limiting the quantity of explosives detonating at any given instant of time, serves the purpose of limiting both effects, but not necessarily to the same degree. Even a very small quantity of explosive can generate a very high overpressure if it is exposed at the ground surface. On the other hand, a very large quantity of explosive buried deeply will generate only small overpressures. Thus, the "depth of burial" is an important consideration, whether the burial is actually some distance below the surface, or is merely an indication that the charge is confined and well stemmed.

For the type of blasting which utilizes vertical holes, the use of stemming is routine and easily accomplished. Coarse sand or fine gravel is easily poured into a hole after the emplacement of the explosive charge. The length of this column of stemming (hence the "depth of burial") is a critical factor in determining the overpressure generated by the blast.

For horizontal holes in underground blasting, it is not customary practice to use stemming, partly because of the difficulty in placing it effectively in the holes, and also because of the time consumed in attempting to do so. Normally, such blasts are sufficiently far removed from the public that there has been no need to change this practice. However, with the increase in underground work in urban settings, this problem is becoming a matter of more concern. If specified limits for airblast overpressures continue toward more conservative levels, it will become necessary to take more exotic measures to reduce these effects, at an increase in time and cost of the blasting.

## Summary

Blasting can no longer be treated by afterthought as an informal appendage to construction plans for a major project. Considering the high cost of construction work, each aspect of the work must be well planned and well executed. In addition, there are safety aspects of blasting that must be well planned and well controlled.

A considerable amount of research has taken place in the last several decades which have raised the technical level of the art of blasting to a much higher plane. It is being scrutinized carefully by researchers in several disciplines of engineering and the earth sciences. New methods, knowledge and data have evolved.

Under the demand of a new generation of well-informed researchers and users of explosives, the manufacturers have developed many new products which make the use of explosives more effective, safer, and allow better control of blast effects.

Rapid advances in instrumentation now permit almost automatic monitoring of routine blasting, and allow the study of certain aspects of blasting that could not be readily measured only a few years ago.

Although there is a trend toward more limitations and more restrictive limitations, recent research and experience also show that some existing limitations are unnecessarily restrictive for certain conditions. Such circumstances can make tremendous

increases in the costs of construction, or entirely prohibit the work from being done.

There is no ready solution to the problem of the sensitivity of the public and the adverse reaction to the motions and sound effects that accompany blasting. There is a trend toward more and more restrictive limitations, but it is this writer's opinion that such an effort will not solve this particular problem. There is no suitable substitute for advanced preparation, dissemination of information, good public relations, and pre-blast inspections.

# THREE DIMENSIONAL FINITE ELEMENT METHOD ANALYSIS FOR DESIGN OF UNDERGROUND STRUCTURES

by

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## INTRODUCTION

Three-dimensional Finite Element Method (3-D FEM) analyses have broad applications in the design of underground structures. These methods can explicitly consider the ground-structure interaction around underground openings and can be used to produce design parameters for a large range of loading conditions, geometries and material properties.

In research currently underway at the Massachusetts Institute of Technology (M.I.T.), 3-D FEM techniques have been employed extensively in an attempt to develop improved design procedures for tunnel supports. In this research, the FEM program ADINA has been used to improve the understanding of ground-structure interaction behavior around tunnels and to develop correction factors for simpler and faster closed-form solutions that are more suitable for practical design use. Furthermore, ADINA was used to model the performance of the instrumented Atlanta Research Chamber and the Peachtree Center Station. This monograph will first discuss the general advantages and limitations of the 3-D Finite Element Method, and then focus on the particular application at the Atlanta Research Chamber.

## Advantages and Limitations of Three-Dimensional Finite Element Methods

Many of the problems encountered in underground construction are three-dimensional in nature. Although simplification to a two-dimensional problem is often possible, there are circumstances in which three-dimensional analyses will have distinct advantages. Some examples are:

1. Problems involving complicated geometries and/or boundary conditions. Such conditions could arise from the geometry of the structure itself, or from that of the surrounding geologic formations.
2. Modelling as accurately as possible such complicated excavation procedures as "heading and bench" operations, "multiple drifting,"...etc.
3. Analyzing the ground-structure interaction near the face of an advancing tunnel.

On the other hand, such analyses have the following limitations:

1. The three-dimensional Finite Element Method representation of an actual problem can be very time-consuming and expensive. Specifically, the discretization process (drawing the mesh) required as a first step in any FEM analysis is especially tedious in three dimensions. Furthermore, the effort required to interpret the results of the analysis is directly proportional to the size of the problem investigated.

2. The cost of a three-dimensional analysis far exceeds the corresponding cost of a two-dimensional analysis.
3. In normal tunneling situations, the movements occurring near the face of an advancing tunnel will only affect the initial support design. Hence, if an advanced design-construction method with performance monitoring and in-site adaptation of design is used, three-dimensional FEM for design is often not warranted.
4. As is the case with any sophisticated analytical method, the applicability of three-dimensional FEM is restricted by the usually limited information on subsurface conditions.

Hence, in a given project, it is always advisable to compare the time and cost commitments with the value of the results likely to be obtained from such a sophisticated method of analysis.

## Computer Program ADINA

ADINA (Automatic Dynamic Incremental Nonlinear Analysis) is a general purpose, linear and nonlinear, static and dynamic three-dimensional Finite Element Method analysis program. It was developed by Professor K.J. Bathe of the M.I.T. Mechanical Engineering Department as a further development of the NONSAP and SAP IV programs[1/] [2/]. The following brief description summarizes

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[1/] K. J. Bathe, (1976a), "ADINA - A Finite Element Program for Automatic Dynamic Incremental Nonlinear Analysis," Report 82448-1, Acoustics and Vibration Laboratory, Mechanical Engineering Department, M.I.T.

[2/] K. J. Bathe, (1976b), "Static and Dynamic Geometric and Material Non-linear Analysis Using ADINA," Report 82448-2, Acoustics and Vibration Laboratory, Mechanical Engineering Department, M.I.T.

the features of the program which are particularly useful in the analysis of underground structures.

ELEMENT TYPES:

1. Three-dimensional truss elements
2. Two-dimensional plane stress or plane strain elements
3. Two-dimensional axisymmetric shell or solid elements
4. Three-dimensional solid elements
5. Three-dimensional thick shell elements
6. Three-dimensional beam elements

MATERIAL BEHAVIOR MODELS:

1. Isotropic linear elastic
2. Orthotropic linear elastic
3. Curve description model (includes tension cracking)
4. Concrete model (includes tension cracking)
5. Elastic-plastic materials, Von Mises or Drucker-Prager yield criteria
6. Thermo-elastic-plastic-creep, Von Mises yield criteria

In addition to material nonlinearity, the effects of large displacements and strains (geometric nonlinearity) can also be included.

Excavation and support installation operations can be modelled using a "birth/death" option in which elements are activated or deactivated during the calculations. Simulation of the incremental advance of the face of a tunnel is thus possible by deactivating each "round" of elements sequentially.

Variable-number-of-nodes isoparametric finite elements[3/] are available for both two and three-dimensional continuum analyses. These elements are efficient and accurate and allow much flexibility in mesh layout and boundary geometry. The variable-number-of-nodes option permits effective modelling from coarse to fine element meshes.

In ADINA, all system matrices and vectors are stored in a compact form and processed using an out-of-core equation solver. This results in maximum system capacity, virtually eliminating any size constraints (i.e., the maximum number of nodel points and elements) for very large problems.

Because of the size and complexity of most three-dimensional problems in underground openings in general, and of the Atlanta Research Chamber and Peachtree Center Station in particular, preprocessor and postprocessor computer programs were developed to aid in the preparation of the input data and in the interpretation of the results. The preprocessor consists of a simple mesh generator and a graphics display for data checking, while the postprocessor programs make it possible to manipulate output data files from ADINA and to plot stress and displacement information.

## ANALYSIS OF THE ATLANTA RESEARCH CHAMBER

The purpose of this study was to examine, by three-dimensional FEM analysis, the ground behavior in the Peachtree Center Station area in Atlanta due to (see Fig. 1):

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[3/] K. J. Bathe, and E. L. Wilson, (1976), Numerical Methods in Finite Element Analysis, Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

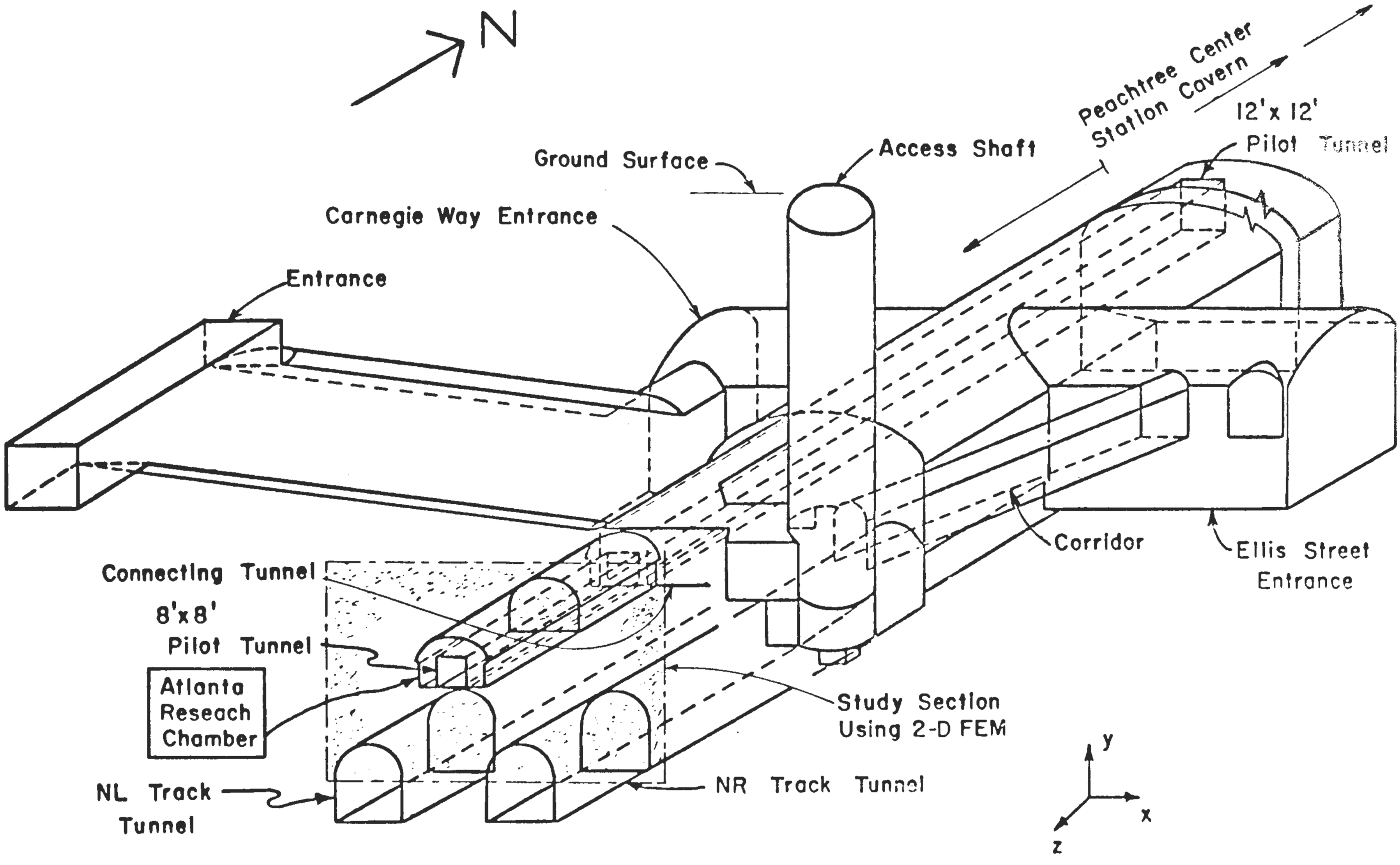



FIGURE I OBLIQUE VIEW OF THE PEACHTREE CENTER STATION, SOUTHERN HALF

1. excavation of the pilot tunnel in an initially stressed rock mass;
2. enlargement of this pilot tunnel to form the test cavern (Atlanta Research Chamber);
3. excavating the main Peachtree Center Station cavern;

and to:

4. predict extensometer and inclinometer movements and compare them with actual measurements.

A brief summary of the analysis performed and the results obtained are given below. A more detailed description is given by Einstein et. al.[4/].

## Analysis Performed

A detailed subsurface investigation program (Law Engineering Testing Company)[5/] indicated that:

1. The intact rock behaves elastically up to failure.
2. The rock mass is of excellent quality with RQD values greater than 90 percent;
3. There are four sets of discontinuities, but in the area around the Atlanta Research Chamber they are all tight and/or widely spaced; and

---

[4/] H. H. Einstein, et. al. (1978). "Improved Design for Tunnel Supports", Interim Report prepared for U.S. Department of Transportation, Volume II, Contract No. DOT-TSC-1489.

[5/] Law Engineering Testing Company (1977), "Report of Geology and Instrumentation--Peachtree Center Station Pilot Tunnel (Construction Unit CN-124)", 3 Volumes.

4. The rock in the Research Chamber area is reasonably homogeneous and without any apparent major open discontinuities. Although there are several rock subtypes, they all have similar properties (no systematic variation was detected).

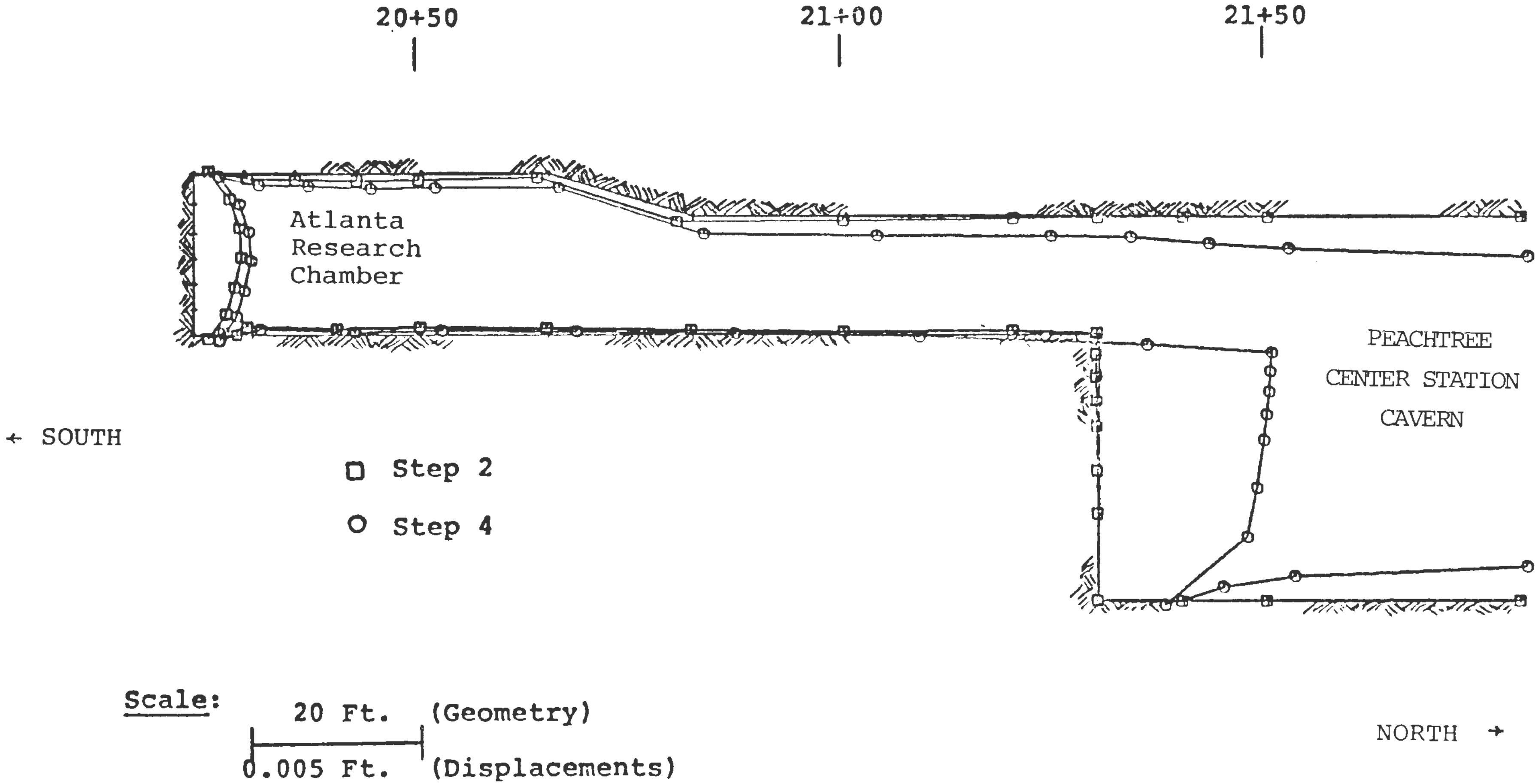
Based on this information, the investigation was limited to static isotropic linear elastic analyses. The entire mass is assumed to be homogeneous with elastic properties $E = 4.6 \times 10^6$ psi and $v = 0.17$. (See also Table IV-1 and IV-2, Chapter IV).

The anticipated sequence of excavation operations were simulated in three steps. The first step involved the excavation of the pilot tunnel. Enlargement of the southern end of the pilot tunnel to form the Atlanta Research Chamber was simulated in step two. Step three, the excavation of the main Peachtree Center Station cavern top heading, was performed simultaneously with step four, excavation of the lower heading. The excavation of the running tunnels was not simulated in this analysis.

The finite element mesh consisted of 16 transverse cross sections and a total of 1093 three-dimensional isoparametric elements. The number of nodes per element varied between 8 to 20, yielding a total of 2915 nodal points.

## Results

Although the assumed material behavior and geometry of the ground was relatively simple, and although in general the stresses and deformations caused by the excavation sequence were small, the 3-D finite element analysis produced several interesting results, of which some would be impossible to obtain using simpler analytical procedures:

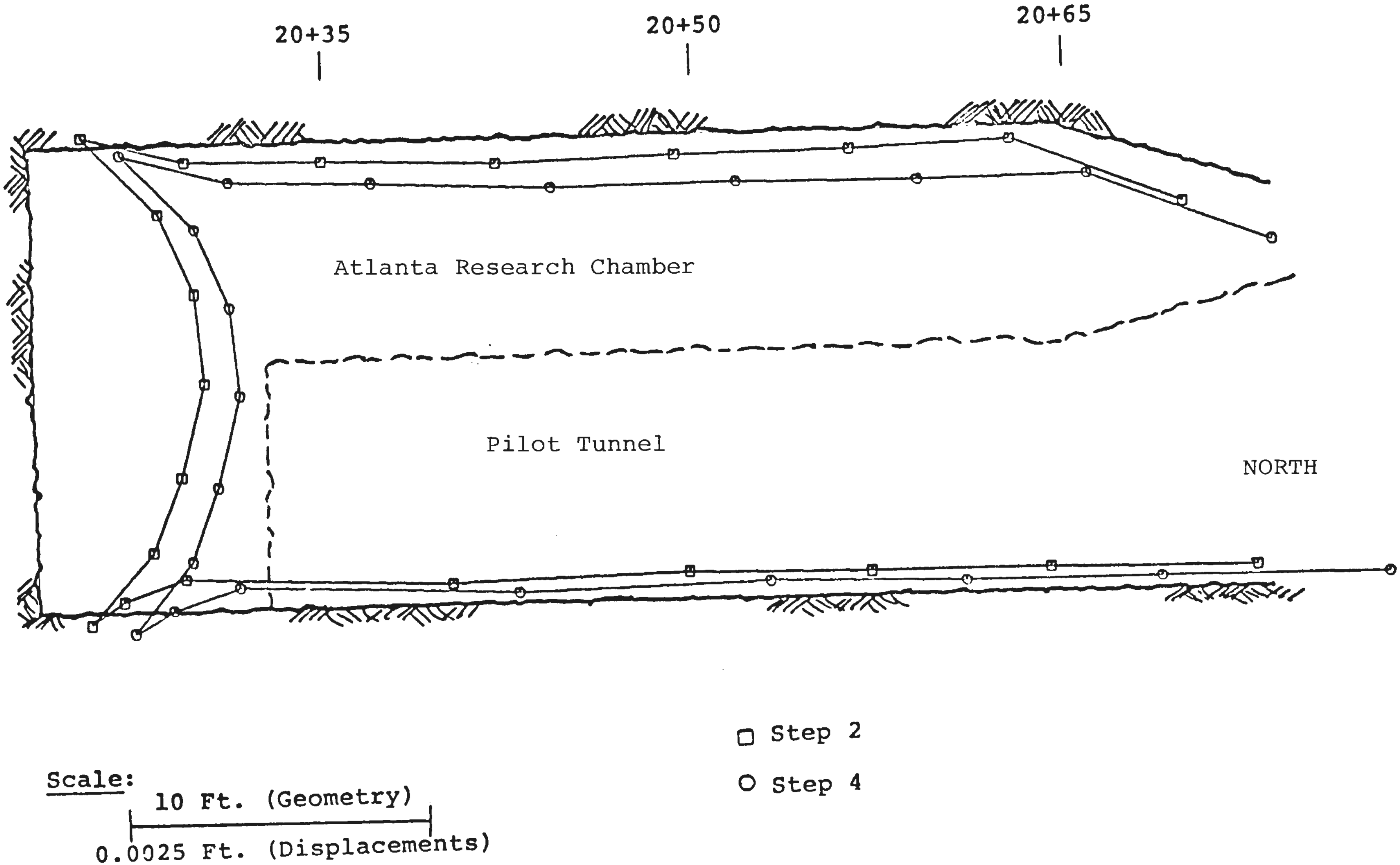



DEFORMED GEOMETRY OF OPENINGS

FIGURE 2

1. A biaxial tensile stress zone forms at the south wall of the main Peachtree Center Station cavern that may cause problems, such as excessive overbreak, during construction of the running tunnels at this location. Furthermore, radial tensile stresses form at the flat parts of the main station cavern's crown, sidewall, and invert.
2. At the transverse cross-sections through the Atlanta Research Chamber, the major and minor principal stresses are approximately tangential and radial with a maximum tangential stress concentration of about 2.
3. The excavation of the main Peachtree Center Station cavern induces relatively large longitudinal movements in the rock surrounding the Research Chamber (Fig. 2). However, the Research Chamber does not tilt measurably in the longitudinal plane as a consequence of the main station cavern excavation. Instead, it settles uniformly. Furthermore, since the inclinometers at the Peachtree Center Station area are approximately located at the y-z plane of symmetry (Fig. 1), a plane strain analysis would predict zero inclinometer movements in all directions (in the x-y plane due to symmetry and in the y-z plane due to the definition of a plane strain analysis). Figure 3 shows these inclinometer displacements as predicted from the 3-D FEM analysis.

The results of this (so far) limited application of 3-D finite element analysis have now to be compared with its cost. The total computer cost amounted to about $11,000 and consisted of:




DEFORMED GEOMETRY OF OPENINGS, 3-D FEM STUDY

FIGURE 3

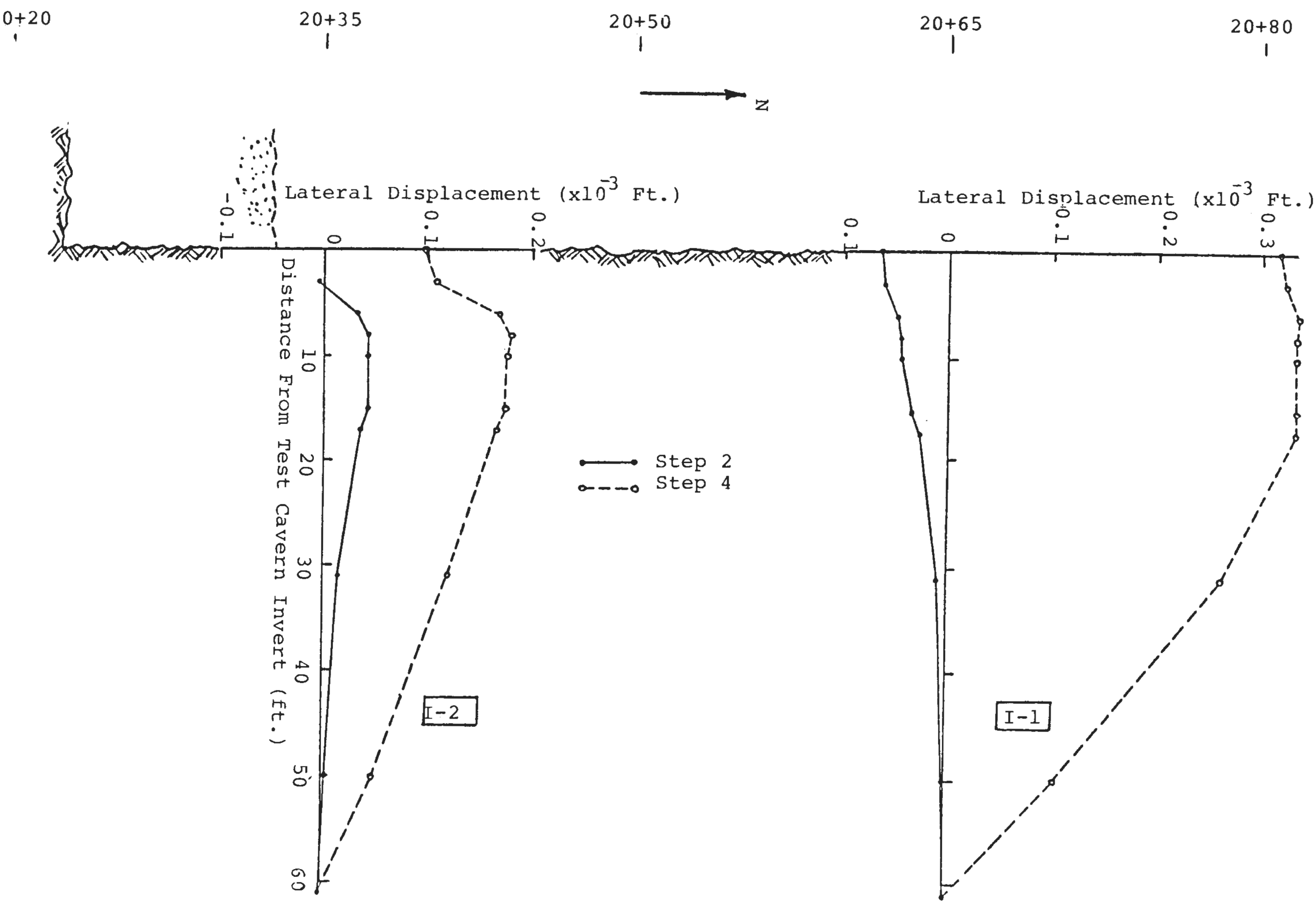



FIGURE 4

Relative Lateral Displacements Along Inclinometers

| | Item | Percent of Total Cost |
|---|---|---|
| 1. | Program modification and implementation | 30% |
| 2. | Input preparation, which included mesh generation and debugging runs | 20% |
| 3. | Actual analysis (3 steps) | 33% |
| 4. | Interpretation and analysis of results | 17% |

A total of four man months by highly qualified engineering personnel was spent on the analysis, broken down in approximately the same way as the computer costs. Since 50% of the cost and time was devoted to problem preparation (items 1 and 2), any further analysis can be conducted at a lower incremental cost. It should also be noted that a substantial part of item 1 consisted of general (non-project specific) modification to and implementation of the ADINA program. These expenses will not be incurred in future applications.

CONCLUSIONS

The application of the 3-D Finite Element Method program ADINA to the analysis of the behavior of the Atlanta Research Chamber is a good, although somewhat simplified, example of modelling the performance of a geometrically complex underground structure as it is excavated. Performance features were predicted that could not have been determined from two dimensional analyses.

The effects of this ability to predict performance in 3-D have to be weighed against the substantially higher cost of analysis. The designer has thus to consider carefully if sufficient information on ground and structural characteristics are available and if the value of the results obtained with 3-dimensional finite element analysis is worth the increased cost. In the case of the Atlanta Research Chamber, the 2-D FEM analysis and report cost about $10,000 as compared to the 3-D FEM study which, under comparable circumstances, might have cost $40,000.

In particular, the 3-D FEM predicted a small upward movement, shown in Figure 3, found to actually take place in the floor of the Atlanta Research Chamber (see Chapter III, IV and V of this report). The 2-D FEM studies could not account for the very large horizontal north-south in-situ stresses, which were in the "third dimension" for the 2-D FEM study.

# THREE DIMENSIONAL FINITE ELEMENT METHOD STUDY OF A SUBWAY TUNNEL AT NUREMBERG

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Paul Bauernfeind, City of Nuremberg, FR Germany
Jean-Claude Bianchini, ESI, Paris, France

## INTRODUCTION

The city of Nuremberg, located in Bavaria in the Federal Republic of Germany, started building a subway in 1967. The construction operations began in a southern suburb, where no complications were encountered in the design and execution of the job. As the first subway line approached the downtown area of the city, the civil engineering challenges increased rapidly. Tunnels had to be excavated next to, or below precious historical buildings, and the work of the responsible engineers was closely watched by the public, who were very anxious about any damage to the existing beautiful old houses and churches.

The job has been handled successfully to date in cooperation with the owner-designer, the geotechnical consultant and the contractors, who jointly went through a process of systematic learning and gathering of experience, and who developed a modern tunneling technique specifically tailored to the natural properties of the encountered ground conditions and the requirements of the city of Nuremberg.

The application of scientific methods has been an important part of this development. Research into the geotechnical properties of the predominantly encountered soft Keuper

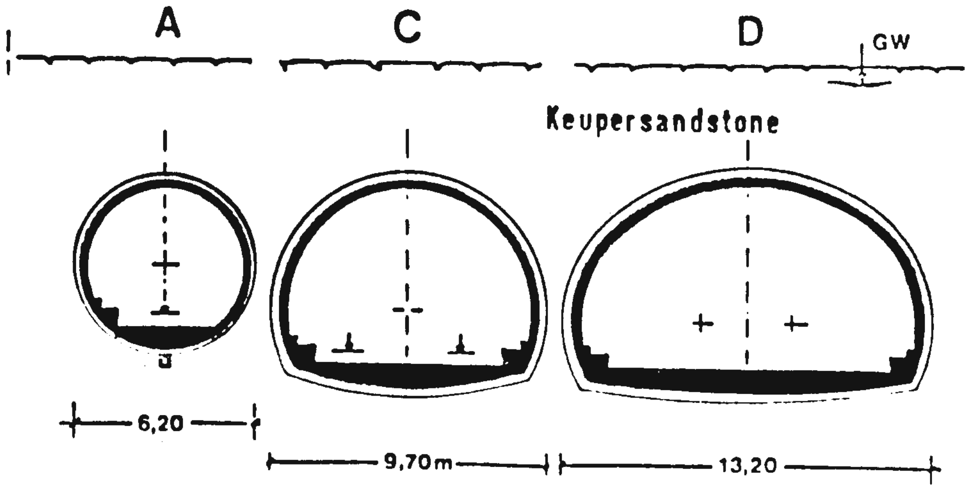



Figure 1 Cross sections of subway tunnels in soft Keuper sandstone at Nuremberg.

sandstone, studies of the geomechanical performance of grout-reinforced sand sections and finite element analyses of various tunnel problems for the prediction of displacements, for the assessment of stability, or as a basis of structural design rules for supporting members, have been carried out. The analytical results have been checked by field measurements in many cases.

The three dimensional finite element analysis which is briefly sketched in the present paper has to be seen in connection with this background.

The engineering problem was to make an extrapolation of the experience gained with tunnel cross sections of standard widths of about 6 m and large sections of 9 to 10 m width, to an extremely wide tunnel section with more than 13 m width under very shallow cover of low strength sandstone (fig. 1). A comparative plane strain finite element study of tunnels with 6, 9 and 13 m width had indicated that the deformations, stresses and failure probability increase progressively with the increasing width of excavation. So, the practical experience gained with sections up to 10 m width, which indicated that the three dimensional state of stress and strains at the tunnel face contain a sufficient degree of safety during construction, appeared to be questionable in the case of the 13 m wide section. This result called for a closer scientific look at the problem of the stability of the tunnel face, employing a fully three dimensional finite element stress/strain analysis.

## Assumptions for the analysis

Geomechanical analyses are carried out with simplifying models which are able to simulate the real behavior of the ground more or less accurately. The more sophisticated the model, the

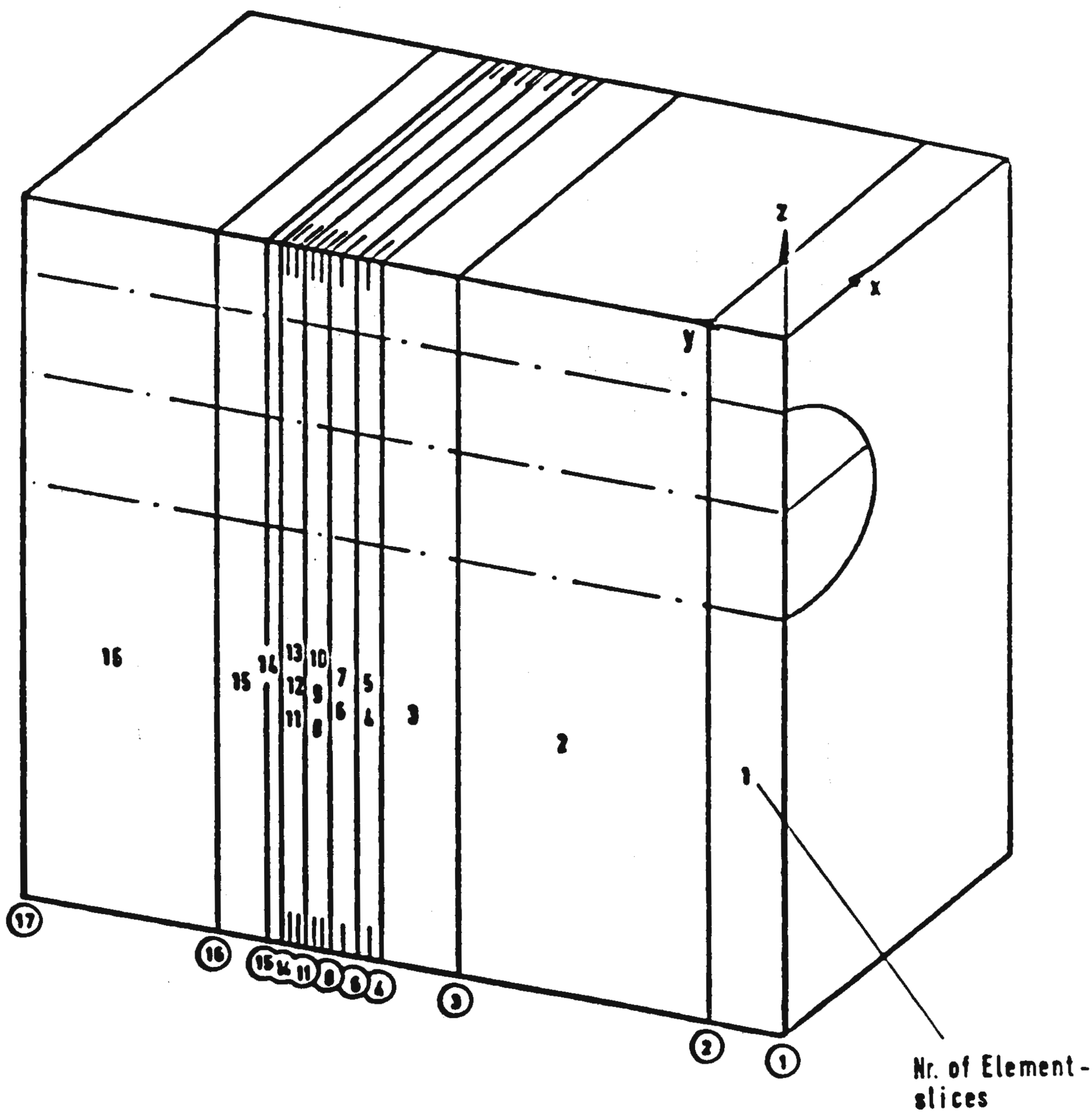



Figure 2  Sketch of the 3D-FEM mesh

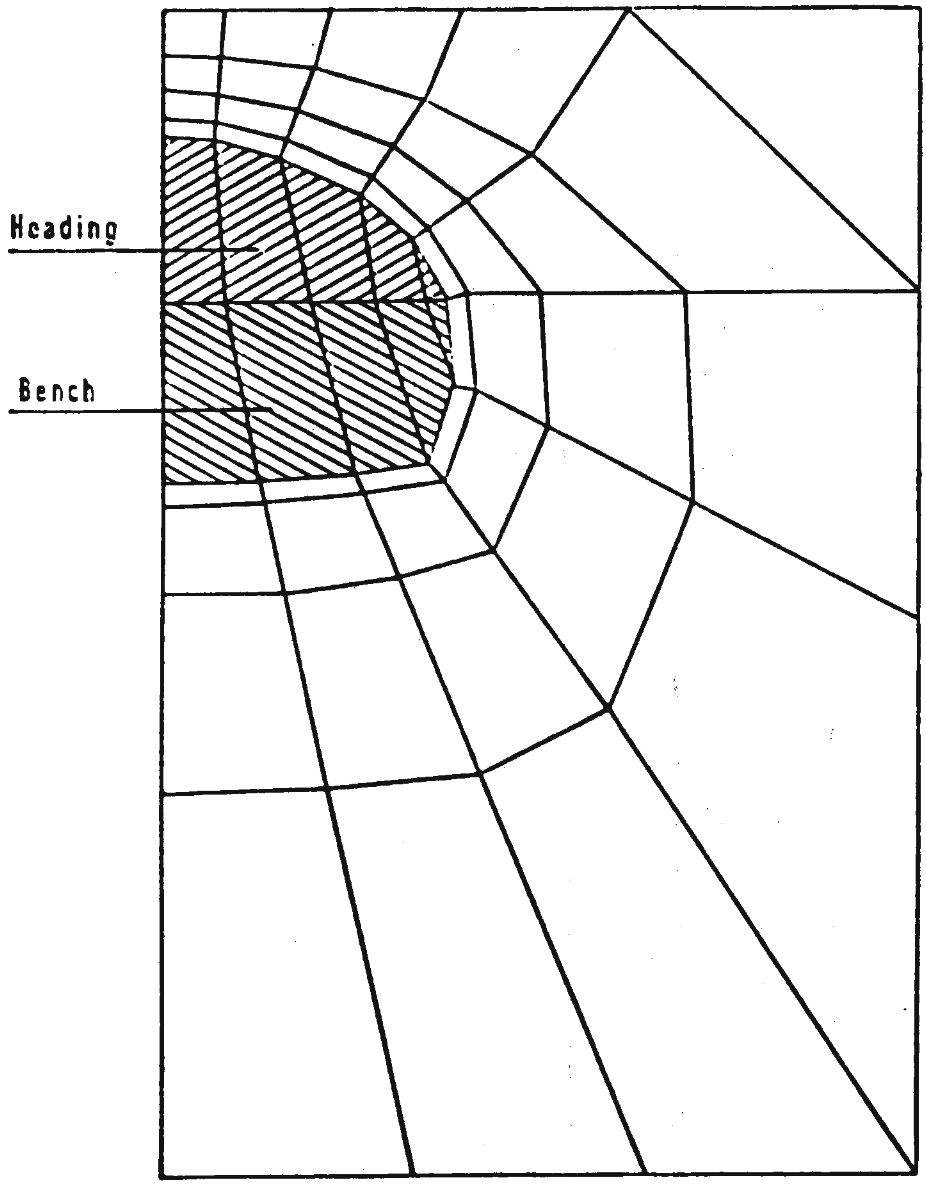



Figure 3 Plane section through the block of the rock mass perpendicular to the tunnel axis

closer an approximation to reality can be obtained, provided that appropriate parameters are used in the analysis.

In the Nuremberg study the geometrical and the loading conditions were symmetrical. Therefore a block of the rock mass, schematically shown on Fig. 2 was used, containing half the tunnel cross section. It was subdivided into 16 slices, each of which consisted of 51 three dimensional elements shown on Fig. 3. The three dimensional element mesh is fine in the central zone of the block of the rock mass where the tunnel face will eventually arrive. It is rather coarse at some distance from the center. So the front cut reaches the zone of interest relatively quickly and without the consumption of too much computer time.

Since there are no significant faults or discontinuities in the sandstone, and experience has shown that local variations of the rock properties do not considerably influence the overall structural behavior, the rock mass could be modelled by a homogeneous, isotropic continuum. The stress/strain relationship was expressed by a nonlinear elastoplastic constitutive law (Fig. 4).

The numerical simulation of the tunnel construction starts with the establishment of the initial state of stress in the ground. Previous analyses, field observations and measurements of displacements indicated that the initial maximum principal stress is oriented vertically and is equal to the weight of overburden or structural loads plus the body weight of the rock mass. The initial horizontal stress component is computed by application of the at-rest coefficient $K_o = 0.3$ in x- and y-directions.

No ground water influences are considered in the three dimensional analysis.

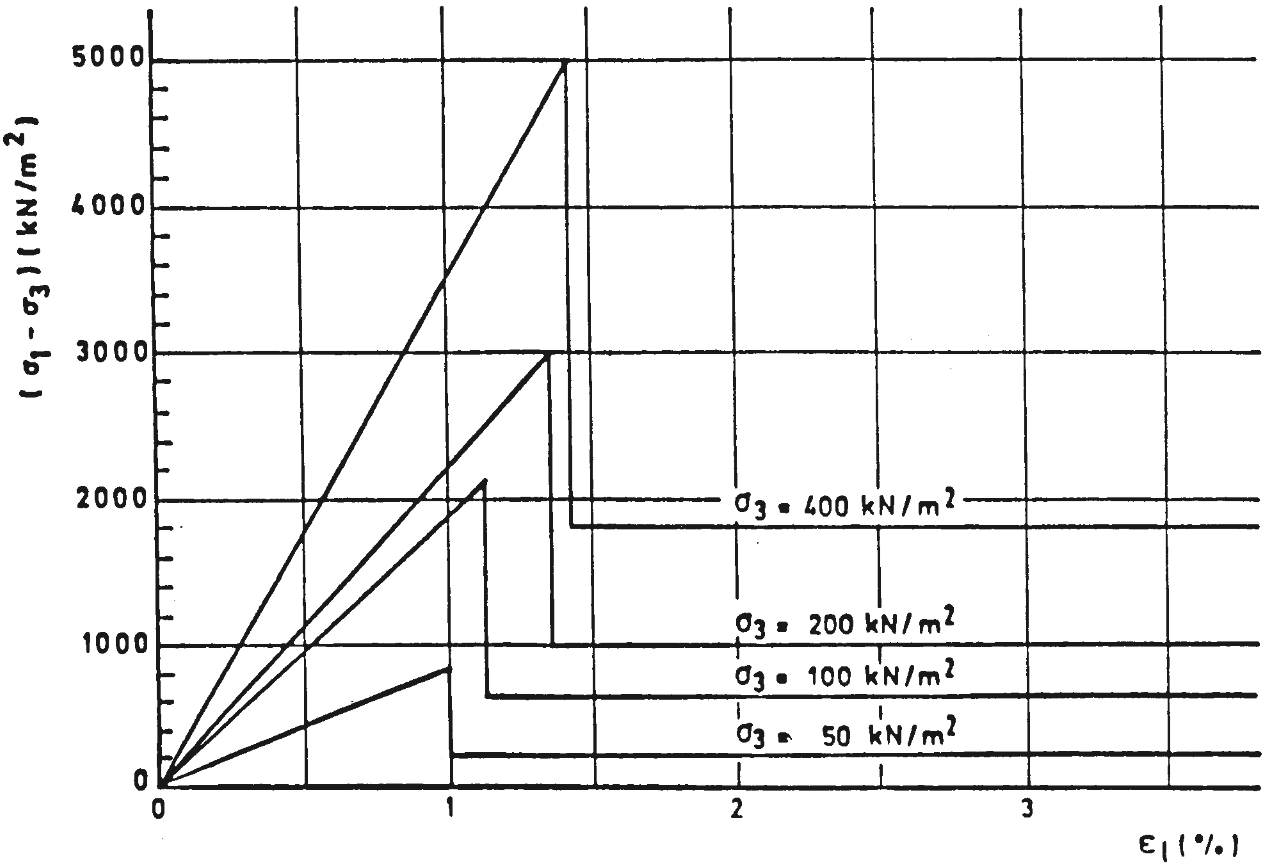



Figure 4 Simplified constitutive law of Keuper sandstone

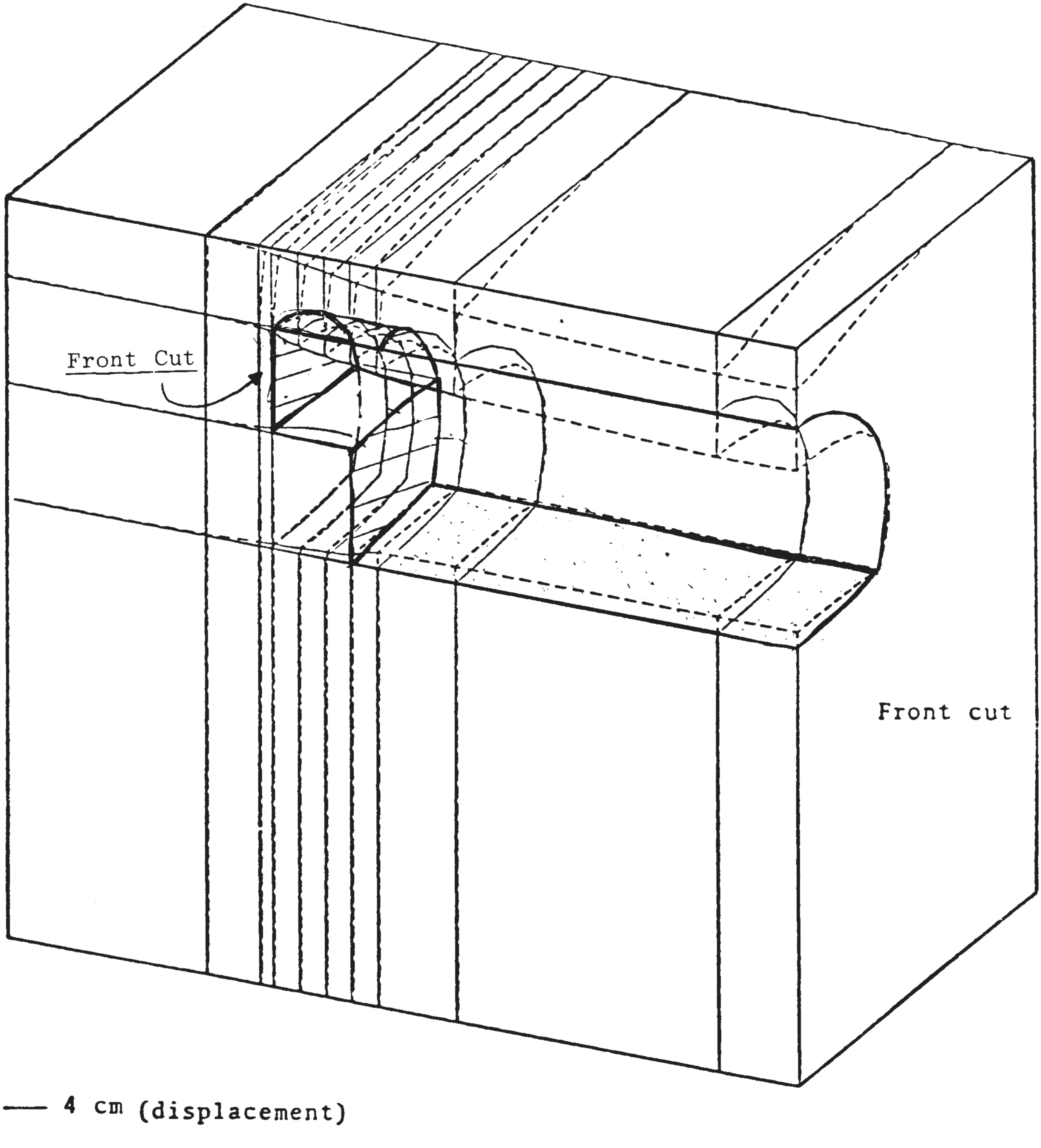



FIGURE 5 Deformed 3D - FEM mesh, computer plot of run 9.

The correct simulation of the supporting effect of shotcrete, anchors and steel ribs is very important in this three dimensional finite element study. At Nuremberg, very light steel ribs are used for practical reasons. They permit fastening of reinforcement meshes and provide support to the soft shotcrete during its application and within the first hours. However, their structural stiffness is negligible compared with that of the aged shotcrete shell. Therefore the steel ribs are neglected in the analysis.

Tensioned, short rock anchors provide confinement to the rock surface in the tunnel. This effect is taken into account by the application of an equivalent internal pressure in the analysis.

The shotcrete is modelled by shell elements with linearly elastic properties which depend on the age of the shotcrete. Under the assumption of an average progress of excavation of 1 m per day, the elastic moduli have to be updated for each element slice in each run according to Table 1.

## Execution of the computations

The computations were carried out by ESI in Paris with the computer program PAM-NL on a CDC-7600 computer. Slight modifications of the program were necessary to suit the requirements of the particular case. A number of test runs preceded the main study and small service programs were written for data pre- and post-processing.

| Run | E - Modulus · 1000 MN/m² in Elementslice Nr. | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 Kal St | 2 Kal | 3 Kal | 4/5 Kal | 4/5 St | 6/7 Kal | 8/10 Kal | 11/13 Kal | 6/7 St | 8/10 St | 11/13 St |
| 1 - 1 | - | - | - | - | - | - | - | - | - | - | - |
| 2 - 1 | - | - | - | - | - | - | - | - | - | - | - |
| 3 - 1 | 9(3) | - | - | - | - | - | - | - | - | - | - |
| 4 - 1 | 26(23) | 17(10) | - | - | - | - | - | - | - | - | - |
| 5 - 1 | 30(29) | 21(16) | 9(3) | - | - | - | - | - | - | - | - |
| 6 - 1 | 30(30) | 22(17) | 10,5(4) | 3(1) | - | - | - | - | - | - | - |
| 7 - 1 | 30(>28) | 22,5(18) | 12(5) | 6(2) | 3(1) | - | - | - | - | - | - |
| 8 - 1 | 30(>28) | 23(19) | 13,5(6) | 9(3) | 6(2) | 3(1) | - | - | - | - | - |
| 9 - 1 | 30(>28) | 24(20) | 15(7) | 10,5(4) | 9(3) | 6(2) | 3(1) | - | - | - | - |
| 10 - 1 | 30(>28) | 25(21) | 15,5(8) | 12(5) | 10,5(4) | 9(3) | 6(2) | 3(1) | - | - | - |
| 11 - 1 | 30(>28) | 25,5(22) | 16,5(9) | 13,5(6) | 12(5) | 10,5(4) | 9(3) | 6(2) | 3(1) | - | - |
| 12 - 1 | 30(>28) | 27(24) | 17(10) | 15(7) | 13,5(6) | 12(5) | 10,5(4) | 9(3) | 6(2) | 3(1) | - |

Table 1

E-Modulus of shotcrete for 3D-FE Analysis

Kal = Heading
St = Bench

(Figures in parentheses refer to the age of the shotcrete in days)

The tunnel excavation was simulated according the following sequence:

| | |
|---|---|
| Run 1: | Establishment of initial conditions |
| Run 2: | Full face excavation, slice 1 |
| Run 3: | Full face excavation, slice 2; rock anchors and shotcrete in slice 1 |
| Run 4: | Full face excavation, slice 3; rock anchors and shotcrete in slices 1 and 2 |
| Run 5: | Heading excavation, slices 4, 5; rock anchors and shotcrete in slices 1 to 3 |
| Run 6: | Bench excavation, slices 4, 5; rock anchors and shotcrete in slices 1 to 3 and in the heading of slices 4, 5 |
| Run 7: | Heading excavation, slices 6, 7; rock anchors and shotcrete in slices 1 to 5 |
| Run 8: | Heading excavation, slices 8, 9, 10; rock anchors and shotcrete in slices 1 to 5 and in the heading to slice 7 |
| Run 9: | Heading excavation, slices 11, 12, 13; rock anchors and shotcrete in slices 1 to 5 and in the heading to slice 10 |
| Run 10: | Bench excavation, slices 6, 7; rock anchors and shotcrete in the heading to slice 13, in the bench to slice 5 |
| Run 11: | Bench excavation, slices 8, 9, 10; rock anchors and shotcrete in the heading to slice 13, in the bench to slice 7 |
| Run 12: | Bench excavation, slices 11, 12, 13; rock anchors and shotcrete in the heading to slice 13, in the bench to slice 10. |

In order to check the influence of a delay in the application of the shotcrete, additional runs were carried out, without the shotcrete shell between the element slices 3 and 14. This study simulated conditions where the shotcrete provides support from 4 m behind the tunnel face only, but where the tensioned anchors are placed in time. The stability was then reduced further by omitting the anchors in this zone as well, leaving the rock entirely unsupported 4 m behind the tunnel face. Computations with a variation of the boundary conditions complemented the study.

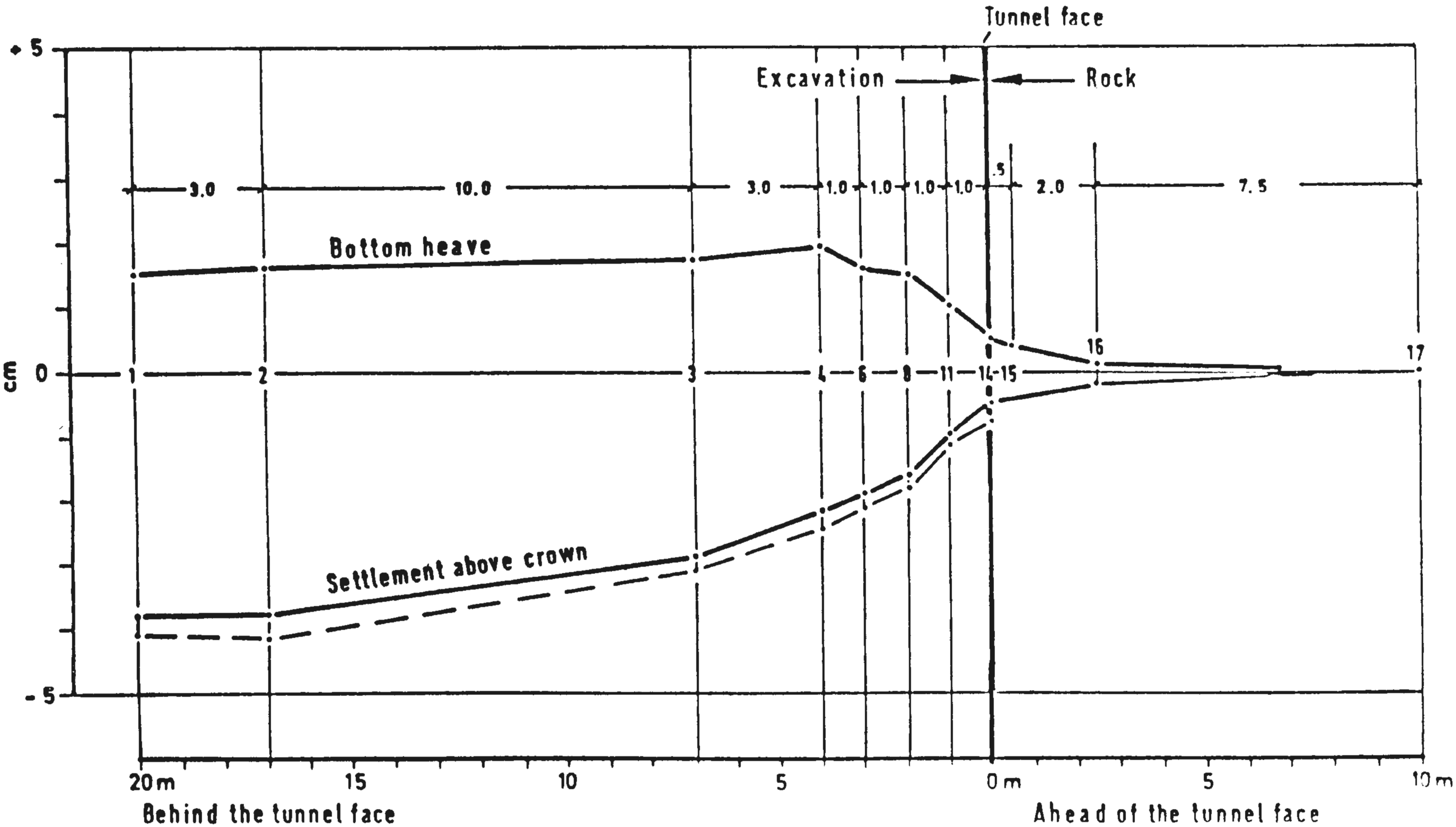
## Results

The computations yielded a tremendous amount of data. More than 3,000 components of displacements were printed in the listings of each run. Thousands of stress components in rock elements and reactional forces and moments in shell elements were given in the output. Therefore a careful choice had to be made as to what information was of significance under engineering aspects, and these data had to be postprocessed and presented in an understandable form.

A very good impression of the three dimensional character of the problem can be obtained from isometric computer plots of displacements of selected nodal points and mesh-lines. An example is shown on Fig. 5 for computer run 9, a stage of construction where the heading advances 3 m ahead of the bench. This type of displacement plot was obtained for all runs.

The next step of exploitation of the results was the study of particular displacements such as settlements above the crown, heave of the invert, lateral displacements of the sides or movements in longitudinal direction of the tunnel above the crown. An example of such a plot is given on Fig 6.

FIGURE 6




Vertical displacements along the tunnel axis

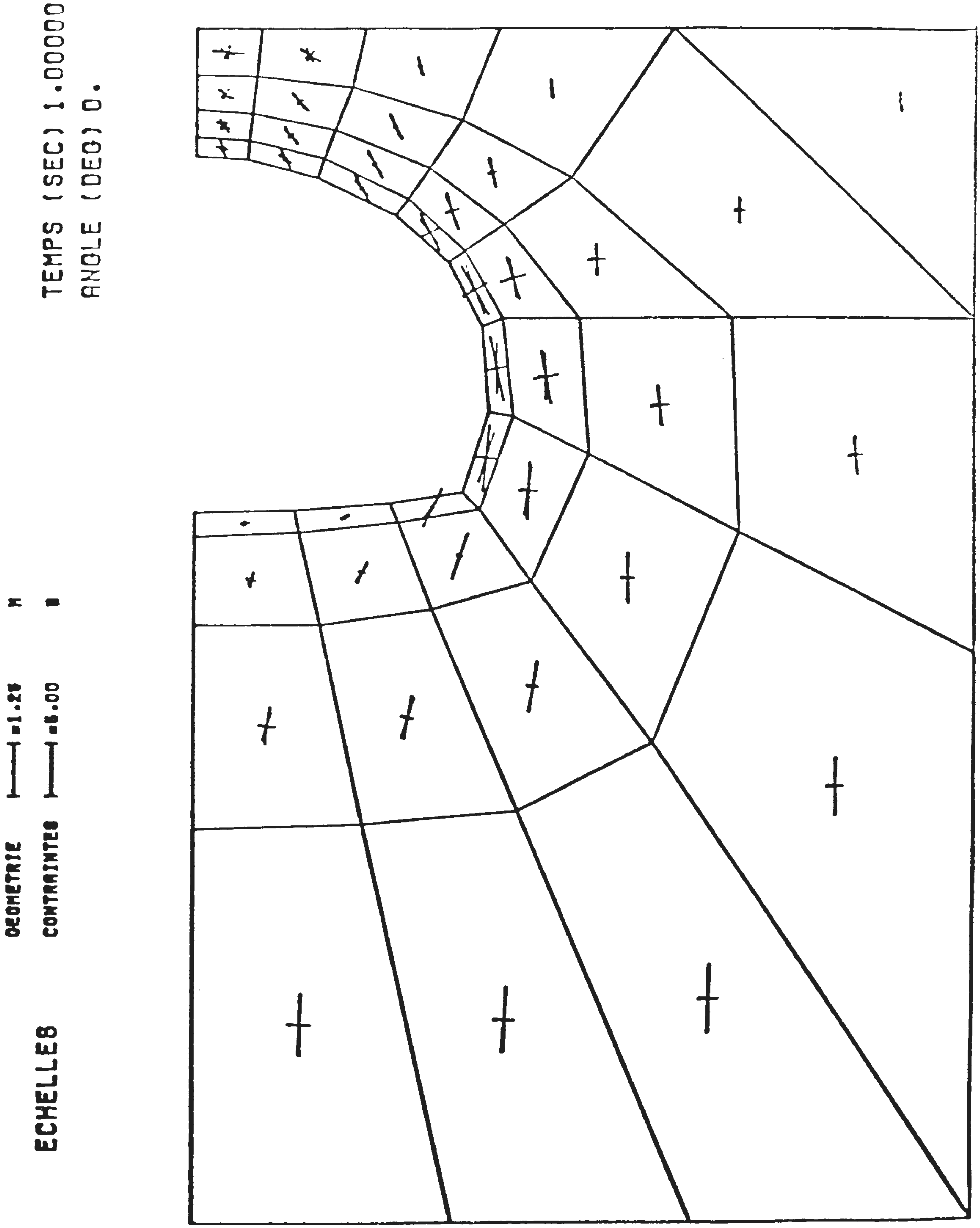



Figure 7 Principal stresses in a section perpendicular to the tunnel axis 5,5 m behind the tunnel face (slice 3) - comparison with slice 9, run 12

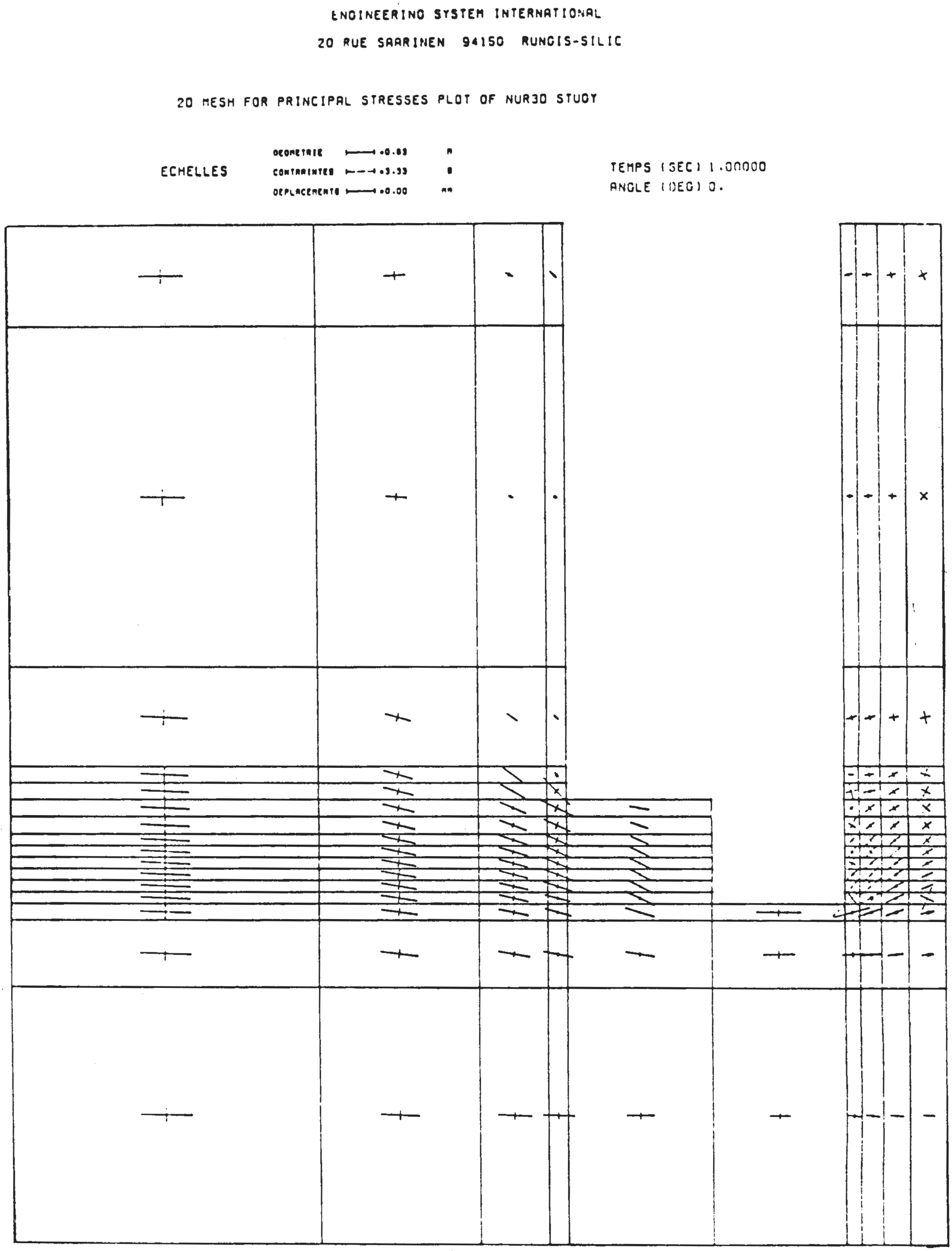
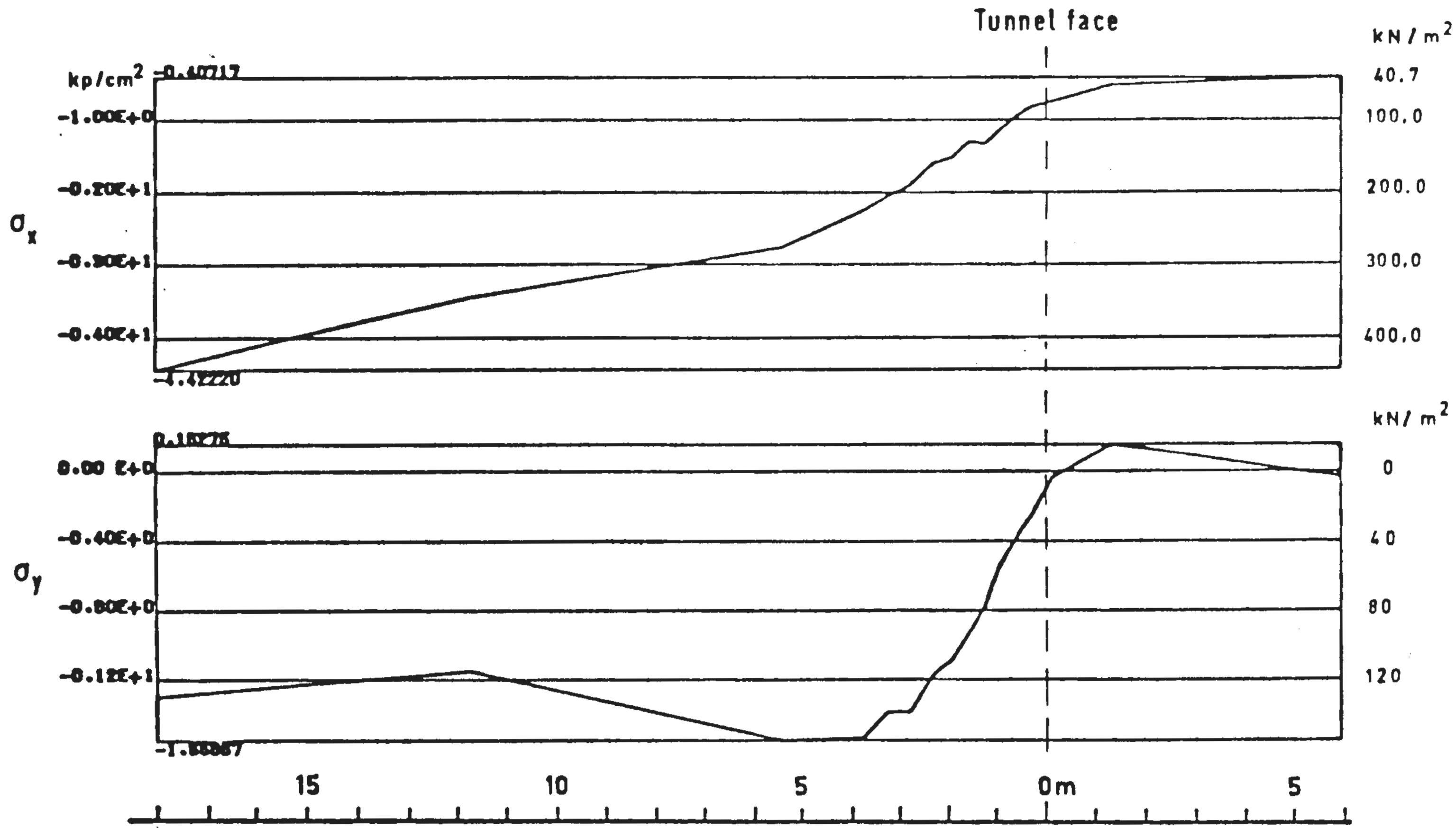



Figure 8 Principal stresses in a section parallel to the tunnel axis, run 9.

Figure 9




Stresses in element 1,1 above the crown at the sandstone surface

A presentation of the state of stress in the ground was obtained from computer plots of principal stresses in vertical planes perpendicular and parallel to the tunnel axis as shown on Figures 7 and 8 for example. The plots indicate stress concentrations, rotations of principal stress directions and unloaded zones.

The variation of stresses in elements of particular interest with distance from the tunnel face was plotted by the computer in the form shown on Figure 9 for an element at the rock surface above the ground, for example.

In the same way, normal forces and bending moments were plotted for selected shell elements of the shotcrete shell.

It is impossible to present more than a few examples of plots in this monograph. This paper can only indicate the importance of postprocessing of data obtained from three dimensional finite element analyses of tunnel problems. It is necessary to select the interesting features and to use an appropriate format of presentation, in order to be able to draw meaningful conclusions from all the data which the computer can provide.

## Conclusions

The results of the three dimensional finite element analysis can be summarized as follows. Pronounced changes from the initial state of stress appear 2 m ahead of the tunnel face. The stress conditions in the front cut zone are relatively stable. The probability of a shear failure of the rock mass is greatest about 1.5, to 2.5 m behind the tunnel face. A three dimensional dome or arching effect in the rock mass can clearly be determined from about 2 m ahead of the tunnel face to 6 m behind the tunnel face. There is a maximum of lateral compressive stresses at the rock surface above the crown about 3 m behind the tunnel face.

The shotcrete shell is highly stressed about 3 to 5 m behind the tunnel face. This result emphasizes the importance of early application and early strength of the shotcrete.

The comparative studies with delayed shotcreting and anchoring showed no great differences in overall deformations but a considerable reduction in safety against local failure in comparison with the design case, assuming early shotcreting and anchoring. This result emphasizes the importance of early support of the underground opening.

Even though the three dimensional dome or arching effect is most pronounced in a narrow zone next to the tunnel face, there is an influence of the third dimension in the structural system from about 10 m ahead to about 12 m behind the tunnel face. Between 6 and 12 m behind the tunnel face, a gradual transition from fully three dimensional to plane strain conditions can be observed.

A careful interpretation of all the results obtained from three dimensional and supplementary plane strain analyses leads to the conclusion that the excavation of the wide tunnel section can be carried out as designed. A monitoring program has been set up according to the information gained from the analyses.

If we try to make a general comment on fully three dimensional finite element analyses for tunnel problems at the end of our present study, we might say that it is a big step forward in our understanding of the stresses and strains in the ground around the opening. But because of the very considerable efforts in work, time and money required for the three dimensional study, we doubt that it might some day become a standard procedure in the design of underground openings. It will remain limited to special problems, whereas 2-D plane strain finite element analyses might well become a standard procedure in the near future.

For special problems, the available numerical techniques offer almost unlimited possibilities to the design engineer. However, the quality of the results of such numerical studies depends entirely on the reliability of the input. So a great deal of preparatory studies, experience and judgment is needed for successful and meaningful three dimensional finite element studies of practical engineering problems in tunneling.

ACKNOWLEDGEMENTS

This paper was prepared as one of the monographs on the state-of-the-art of tunneling which are collected in the Atlanta Research Chamber report to UMTA (UMTA-GA-06-0007-79-1). Don Rose of Tudor Engineering Company of San Francisco is the Principal Investigator for the Atlanta Research Chamber. Gilbert L. Butler is the Project Manager for UMTA, for the Atlanta Research Chamber.

# THE NEW AUSTRIAN TUNNELLING METHOD (N.A.T.M.)

by

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## Introduction

The New Austrian Tunnelling Method (NATM) was so called by Professor L. V. Rabcewicz, because the basic idea and the consequent development of this method came from Austrian engineers during the last 30 years.

Findings, intuitions and experiences in mining from the beginning of this century gave birth 20 years back to a new tunnelling concept.

The demand for a quick sealing of the rock surface, already expressed in 1904, was satisfied only with the development of shotcrete. Following that was the finding of the reciprocal relationship of required lining resistance and deformations by Fenner (1938), and the finding of Rabcewicz (1944) that the time dependant behavior of the rock mass was fundamental for predicting the behavior of the tunnel structure. The development of the shear failure theory for tunnels under high overburden, the demand for so-called semi-rigid linings, the semi-empirical design approach using in-situ measurements as an integral part of the method, and the incorporation of rock and soil in the carrying system were the main subsequent steps in the development of the NATM.

Systematic experiments were made using the NATM, first with the construction of small sections for hydroelectric schemes in the Alps, then with power stations, road and railroad tunnels, and finally with tunnels in soil under low cover in urban areas.

The method is widely applied in Europe, and several successful applications can be cited in Japan, India, Pakistan, Iran and Turkey, as well as in South America and elsewhere where conventional systems failed or were too expensive. There is still a long way to go until the mechanical principles governing the behavior of rock and soil are also fully incorporated in tunnel design concepts, and until an obscurantist conservatism and inertia are overcome.

## Basic Concepts

The NATM is not actually a "construction method", but a tunnelling concept based on principles derived from rock and soil data and on systematically observed experiences.

The principles of the NATM were published fifteen years ago and amplified in later papers by Rabcewicz, but there still exist differing views about the meaning of the term.

Use of more direct nomenclature, such as the "Shotcreting Method" or the "Observational Method" does not solve the problem, since these terms describe only parts of the NATM. The NATM is based on the following concept:

The ground (rock and soil) which surrounds the excavation will be activated to a load bearing ring, enabling the ground to become an important support member in itself.

Using this method, a number of basic features have to be taken into consideration:

- Consideration of the geomechanical ground behavior
- Most suitable statical shape of profile

- Avoidance of unfavourable stresses and deformations by means of suitable support works, installed in the proper sequence.
- Optimization of the support resistance as a function of allowable deformations
- Control by measurements.

A tunnel is a composite structure consisting of rock and supports or strengthening elements (e.g. shotcrete, anchors, steel ribs, etc.). In excavating a tunnel, the prevailing primary state of equilibrium is transformed to a new state of stability or (secondary) equilibrium. The aim of the NATM is to influence these processes in a way which is both economical and technically safe.

The deformations of the rock or solid should therefore be controlled during excavation such that they:

- remain small in order to avoid a decrease of rock strength and
- at the same time are large enough to activate the rock to form the load-bearing ring necessary to reduce the required support resistance.

To reach this goal, a series of subordinate aims must be considered:

- Creating or conserving triaxial stress conditions compatible with the rock strength, and avoidance of detrimental loosening.
- Establishment and consideration of geotechnical parameters (by laboratory tests or in-situ tests), their variation and time dependancy.

- Choice of profile shapes with due regard for the mechanics and condition (especially the primary stress condition) of the rock; strength parameters; and joint systems in the rock.
- Thinness and flexibility of lining within required limits. Avoidance of heavy overexcavations to ensure thinness of lining.
- Direct contact between rock and support elements for load transfer.
- Adjustment of construction procedures to changing rock conditions; standup time; and stability of the face by choosing the proper excavation sequence and length of rounds, with regard to practical and financial aspects.
- Careful excavation without disturbing the rock.
- Installation of support elements without delay and in the correct sequence.
- Ring closure time and distance from the face, depending on the theoretical rock behaviour and as a function of the lining resistance.
- Continuous control of rock and support elements by means of measurements. These constitute an integral part of the method for controlling the safety of the tunnel from preliminary design to final design of the support elements during construction, and they optimize constructional procedures, permitting a correct interpretation for geomechanical documentation.
- If an inner concrete ring is foreseen in the design project, this should also be a thin one.

These demands are in clear conflict with what we see day by day in actual tunnel construction all over the world.

We find designs in poor ground, showing horseshoe shapes with sharp corners and thick, heavily reinforced linings. We observe tunnel driving methods where the existence of smooth blasting methods seem to be unknown, and where the self-supporting capacity of rock or soil is systematically being disturbed by inadequate support systems.

The erroneous belief that heavy reinforcement via thick linings is safer than linings creating a properly engineered tunnel is widespread and definitely leads to wrong-headed, poor designs.

Our aim should be to make rock and soil our partner, to understand the time-dependant behavior of these materials, and to avoid fighting against rock and rock pressures with inadequate means.

The art of tunnelling lies in controlling deformations so that a new state of equilibrium is reached with a minimum of support resistance and, simultaneously, in avoiding detrimental loosening in order to maintain the strength parameters of the surrounding mass.

According to the concept of limited deformations, the NATM could also be characterized as a method where "a new secondary state of equilibrium after excavation is reached by controlled pressure release".

The time-dependent stress rearrangement process can be influenced by the sequence of excavation and support installation procedures, and is to be monitored by in-situ measurements. Such measurements include deformation measurements and stress measurements, using convergence measuring devices, extensometers, levelling, pressure cells, strain measurements, etc.

In Europe, hundreds of miles of water tunnels and more than 200 miles of road, railroad and subway tunnels have been driven using the NATM.

To understand the successful and fast development of the method, it should be mentioned that owners and contractors have - while taking care of their own legitimate interests - generally endeavored to reach their common goal in a combined effort. It is most likely that this obvious teamwork among all involved persons as partners springs from a particular conception of contract conditions. European contractors and the consultants working for them are much more involved in the design process than are their counterparts in the United States, since they are not only permitted but even invited to develop and submit alternative designs. The effect of alternative bidding is, that contractors have an incentive to develop improved construction methods and ideas using their own extensive practical experience. Most of the subway contracts in Germany built according to the principles of the NATM were alternative proposals submitted by contractors.

Where is the incentive for our engineers who are supposed to develop new, better and more economical designs? Professional ethics is obviously not sufficient. Compensation for design work as a percentage of construction costs may cause inner conflicts; while compensation for hours spent in the production of huge amounts of partially useless drawings by an extensive staff is not efficient. Neither stimulate the development of progressive new designs. I see the only solution in competitive alternative bidding and in a more efficient use of value engineering.

## Case Histories

Adoption of these two solutions shall be shown at work with two examples, one from the subway in Munich and one from Bochum. Both cases were alternative solutions prepared by the Contractor.

### 1. Munich

In the case of Munich, the design project of 1974 foresaw a conventional steel support in a horseshoe shaped tunnel, with a heavily reinforced 60 cm thick final inner lining. The bottom slab for the single track tubes would have been 70 cm thick.

We prepared an alternative design, which was approximately 30 percent cheaper than the conventional design. The longitudinal section (Figure 1) shows the geological situation.

The Quaternary soil was cohesionless gravel with maximum size aggregate of approximately 2 inches.

The tubes had to pass at a distance of 1.5 m beneath a concrete canal. The clearance underneath a road and a school was 3m, and only 6m from a church tower foundation (94 m high).

The alternative solution is shown in Figure 2. The top heading has been excavated with a temporary invert and forepoling in the Quaternary soil.

The injection (rock) bolts were omitted after placement in the first measuring section because the stresses measured proved to be much less than the computed values. Also the length of rounds in the bench excavation could be increased after the favorable measuring results. A typical development of settlements is shown in Figure 4.

The surface settlements were maximum 12 mm, an outstanding result for cohesionless soil conditions (Figure 5).

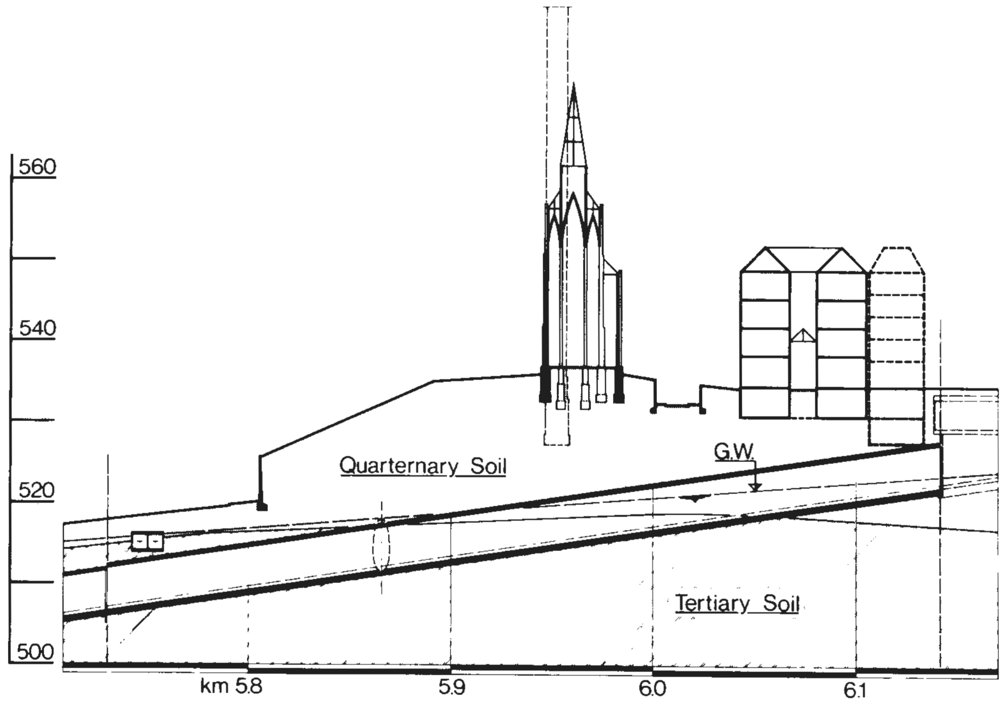


FIGURE 1. Munich—Geological Longitudinal Section, 8.1.

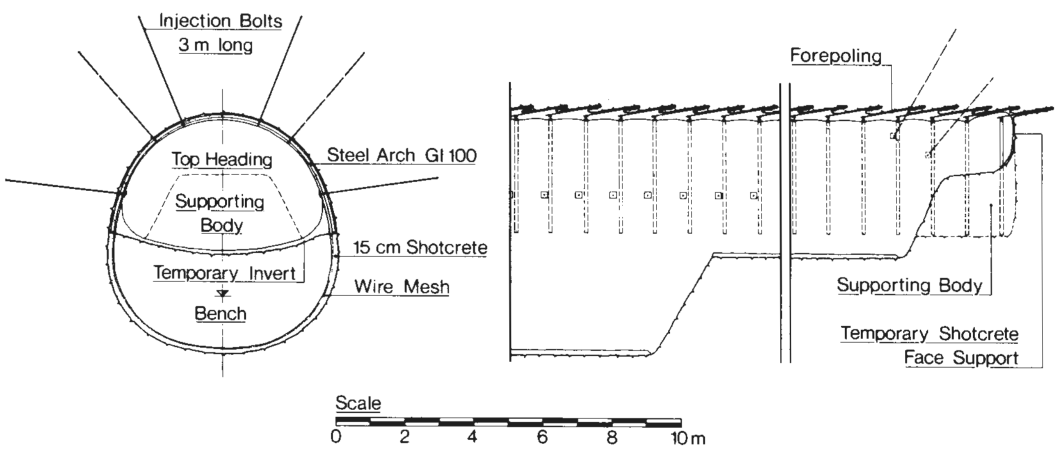


FIGURE 2. Munich—Alternative Solution
( Cross-Sectional View, Section 8.1. )

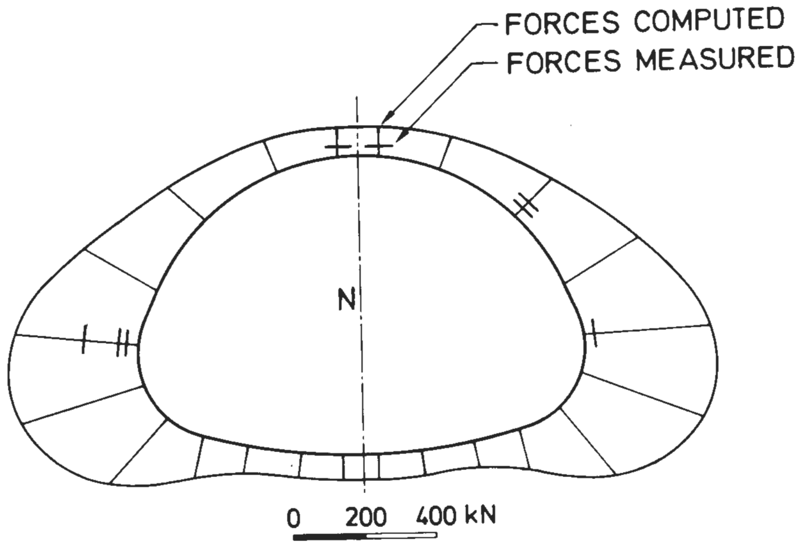


FIGURE 3. Munich—Computed and Measured Normal Forces in the Top Heading, Section 8.1.

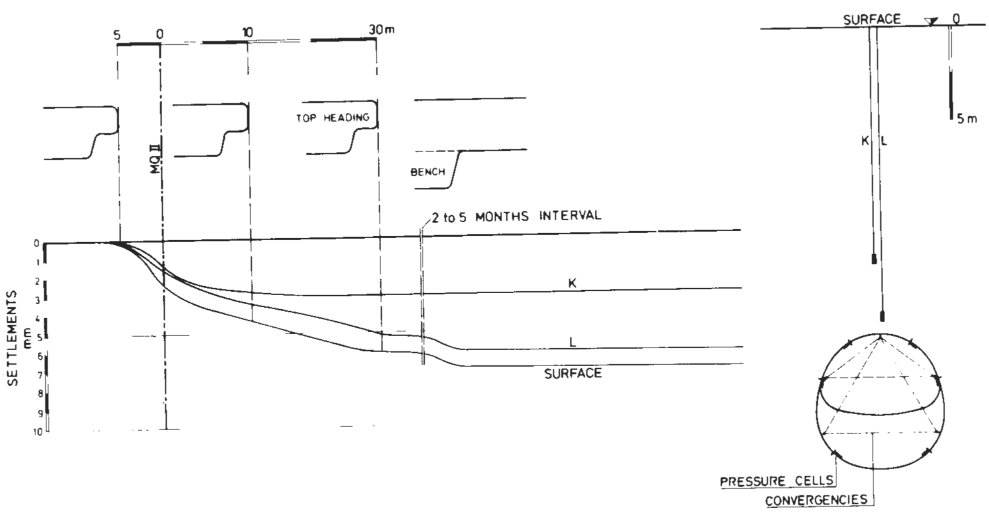


FIGURE 4. Munich—Typical Settlement Development and Extensometer Readings, 8.1.

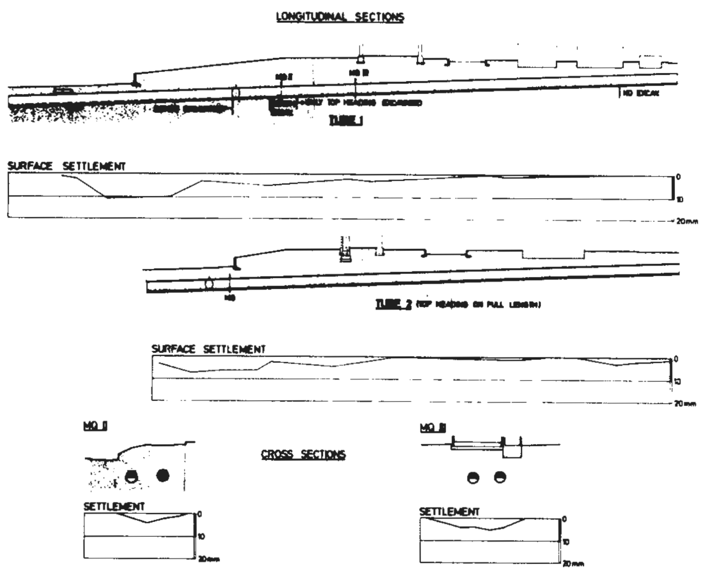


FIGURE 5. Munich—Surface Settlements; Cohesionless Soil, Section 8.1.

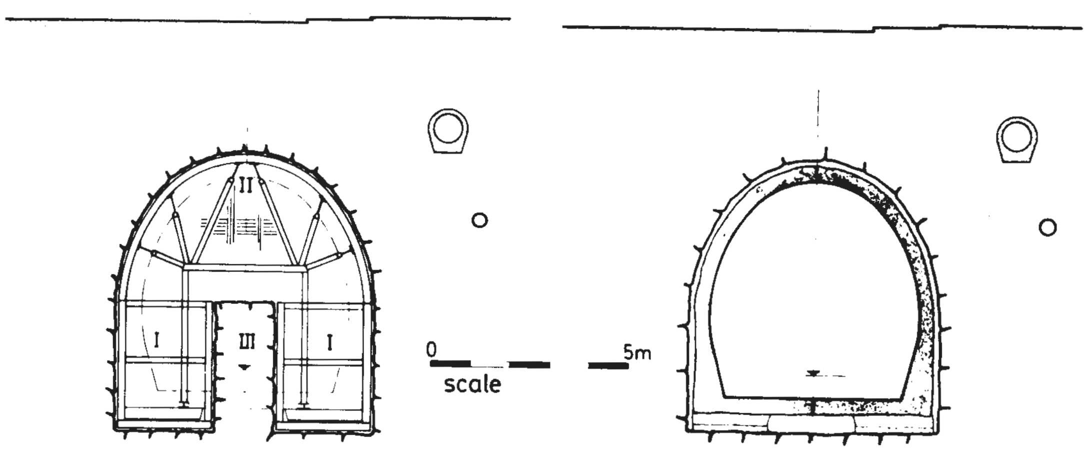


FIGURE 6. Bochum Contract B-1 Tender Solution ( Left: Silt and Fill; Right: Marl )

## 2. Bochum

In Bochum, the Contract B-1 foresaw a solution according to Figure 6 where the left section was for the silt and fill and the right section for marl. Figure 7 shows the plan and Figure 8 the longitudinal section with the geology.

The NATM solution for Section I is shown in Figure 9; for Section II, in Figure 10, and Section IV, in Figure 11.

The alternative solution was 25 percent cheaper than the design solution. The costs for the civil works were on average U.S. $16,800 per lined meter of subway.

The costs in Germany for single tubes and double track subway tubes with the NATM vary between $150 and $250 per cubic meter excavated depending on soil conditions.

In cases where chemical grouting, freezing, etc. are used to underpass extremely sensitive structures, costs increase beyond the above values and may even be tripled.

Tunnel costs (civil works) in non-urban areas such as the highway tunnels in the Alps vary between approximately $90 per cubic meter excavated and approximately $200 for very heavy rock with high rock pressure as for instance in the Arlberg Tunnel West.

The range of applicability of the principles of the NATM is very wide. The main advantages of the method are the good adaptability to frequently changing rock and soil conditions, the flexibility to varying shapes for Station and Switch sections, and the better economy.

The Method also has, however, one disadvantage in its demand for well-trained site staff and experienced working crews. This difficulty can be overcome by good training, and by keeping good teams together. Engineers and owners should abandon overly conventional design concepts, which ignore actual rock/soil conditions, and which are not consistant with structural design principles.

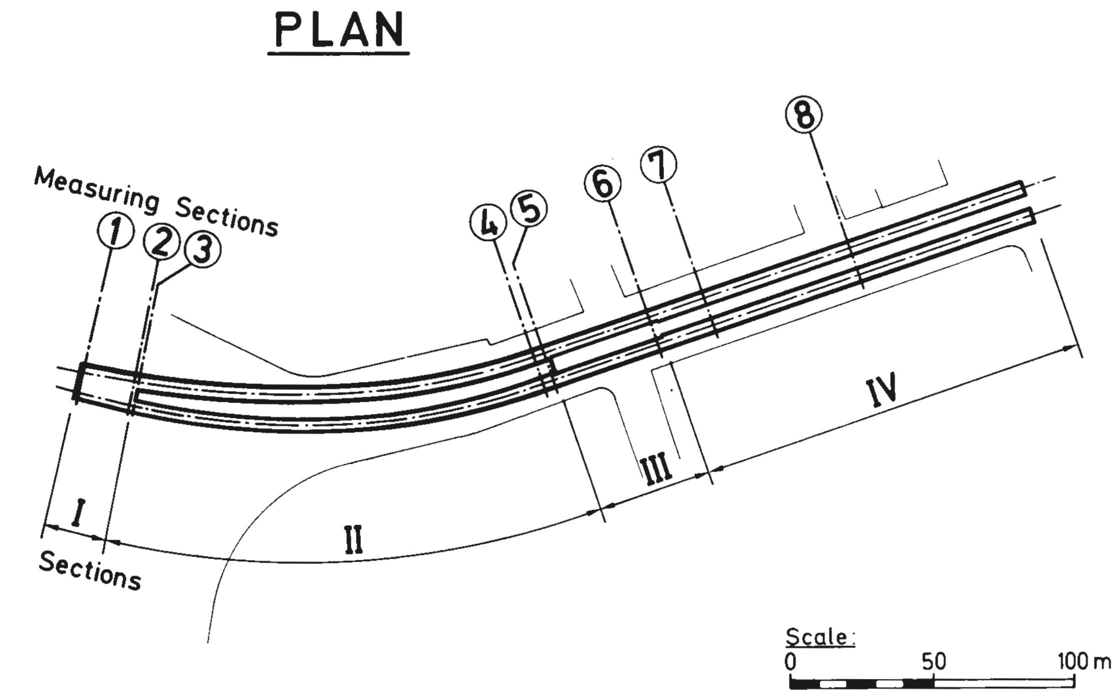


FIGURE 7. Bochum Plan B-1, Section Measurements.

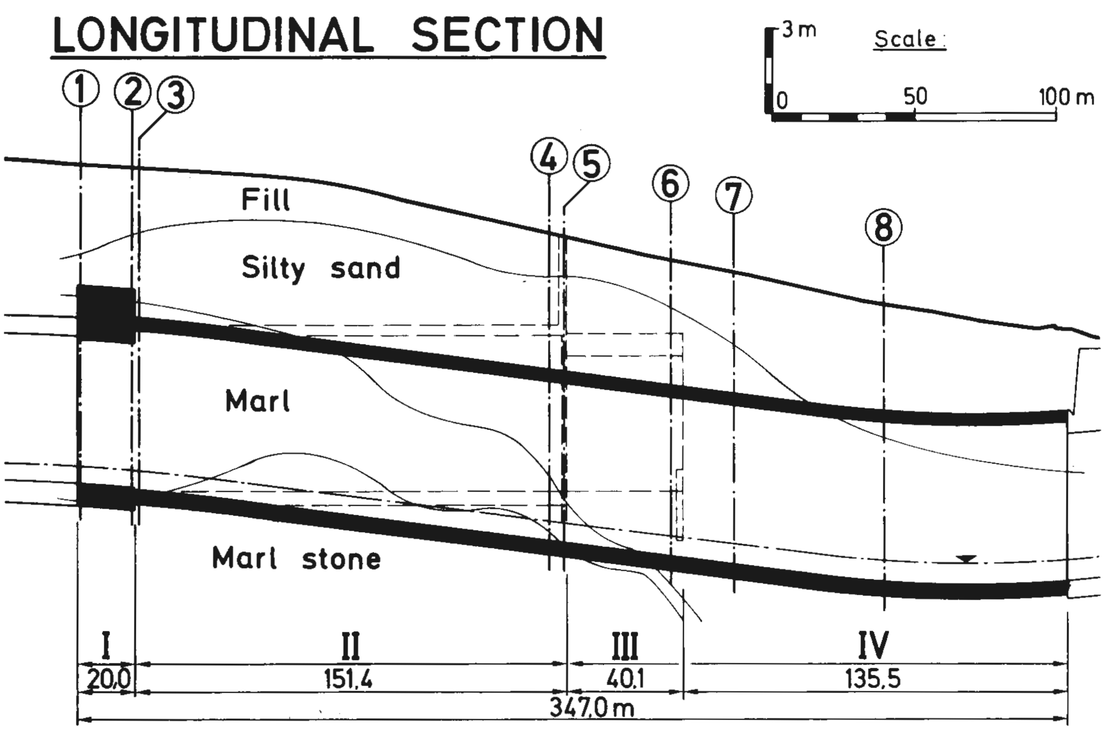


FIGURE 8. Bochum—Longitudinal Section with Geology.

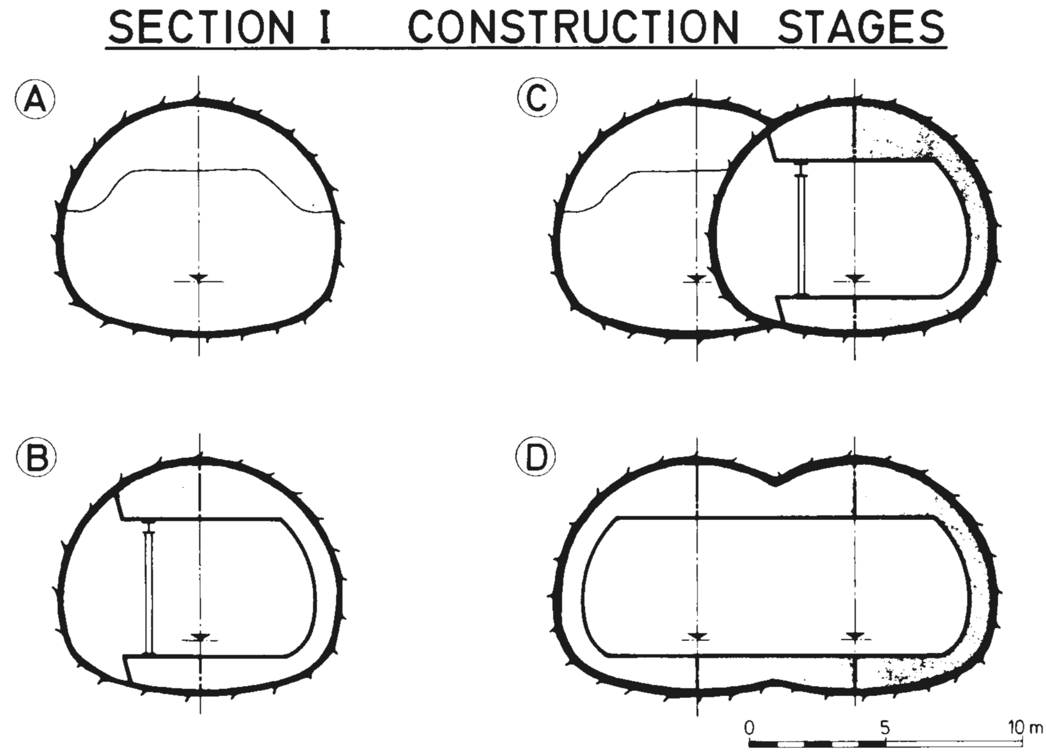


FIGURE 9. Bochum Contract Solution for Section I, Three Tubes.

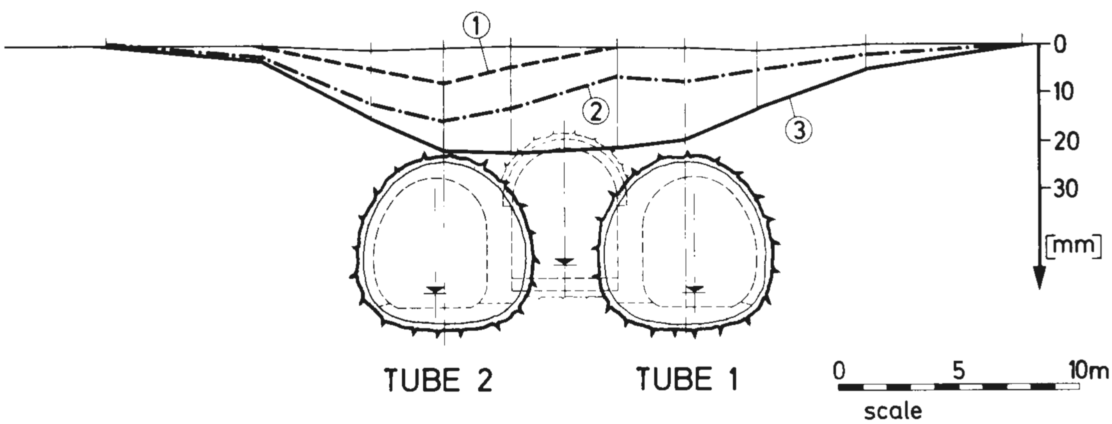


FIGURE 10. Bochum Contract B-1 Solution for Section II, Three Tubes.

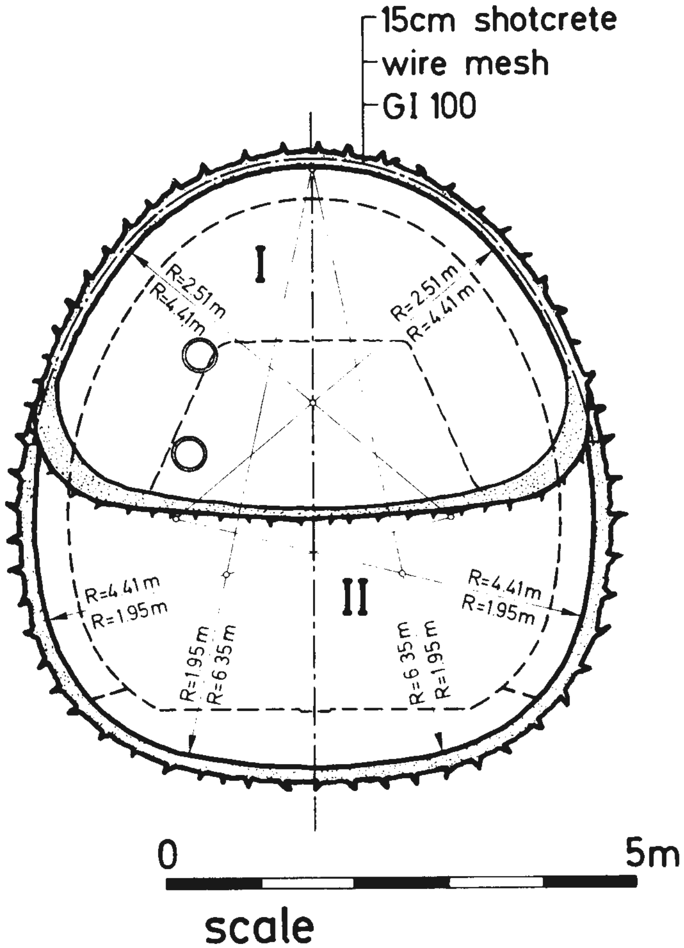


FIGURE 11. Bochum Contract B-1 Solution for Section IV, Single Tubes.

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# THIN SHOTCRETE LININGS IN LOOSENING ROCK

by

Gabriel Fernandez-Delgado[1], Edward J. Cording[2]
James W. Mahar[3], and Michel L. Van Sint Jan[4]

## INTRODUCTION

Shotcrete behavior observed in subway tunnel construction in the U.S. has been primarily in blocky or blocky and seamy rock. One condition, in which rock loads develop from the self weight of individual rock blocks that tend to loosen from the wall and roof of the opening, can be classed as loosening ground. In contrast, squeezing ground refers to a condition in which high pressures are generated on the tunnel lining with time as the ground creeps under the influence of natural ground stresses. In the philosophy often associated with the New Austrian Tunneling Method (NATM), emphasis is placed on the use of shotcrete and other support elements under squeezing conditions and highly stressed ground, where the support system is designed to allow controlled movements so that high lining stresses do not develop. In contrast to these conditions, the concern with loosening ground is to provide support soon enough to minimize loosening that would cause instability in the tunnel heading or would apply excessive load to the tunnel lining. Very small movements will relieve high pressures; thus, it is not necessary to make special accommodations in the design to allow movements in blocky loosening rock.

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The present methods for designing or selecting thin shotcrete tunnel linings for use under loosening rock conditions are largely based on empirical rules formulated from experience. Reliance on experience is appropriate, if the rules are related to the significant rock and shotcrete parameters influencing lining requirements. Careful field observations of shotcrete behavior, and descriptions of failure modes and the significant rock conditions affecting behavior are needed in order to improve the methods for selecting shotcrete linings.

Shotcrete has proven its capabilities in many applications, but there is a continuing need to determine both its capabilities and limitations for various rock conditions so that more realistic evaluations can be made prior to using it on a Project.

The studies outlined in this paper have been directed toward evaluation of the requirements for using shotcrete in tunnels in jointed rock having blocks or wedges that require support. Emphasis was placed on observing the effect of specific rock geometries and rock surface characteristics on the magnitude of the rock loading, and ascertaining the bond characteristics between shotcrete and rock. To accomplish this goal, both field and large-scale laboratory model studies have been conducted over the past 8 years. The field studies were first carried out in the Washington Metro tunnels, in blocky and seamy foliated gneisses and schists in which the boundaries of rock wedges were formed by continuous, planar joints and shears. Shotcrete-rock adhesion was often poor, particularly along sheared surfaces and foliation planes. Observations were made of rock fallout and overbreak, and the relation of rock displacement to cracks and strains in the shotcrete lining. Where rock surfaces were planar and smooth, slippage along the shotcrete-rock interface was observed, with tensile fractures developing on the portion of the shotcrete membrane covering the protruding rock block (Mahar, Gau, Cording, 1972).

Other investigators have noted similar conditions in loosening ground where the bond conditions were poor. Cecil (1970), in a survey of Swedish and Norwegian tunnels, observed adhesion failure on clay-coated bedding plane surface. Brekke (1972) noted adhesion failures in which rotation and displacement of isolated blocks tended to puncture and break out of the shotcrete lining.

Subsequent to the Washington Metro studies, a series of large scale model tests were conducted at the University of Illinois to further evaluate the factors influencing shotcrete support in loosening (or blocky) ground. Shotcrete was sprayed on a loading frame simulating various rock geometries. Rock geometries and surface roughnesses were controlled in order to evaluate failure modes under both good and poor shotcrete-rock bond conditions. Tensile failures similar to those observed in the Washington Metro tunnels developed where the bond was poor.

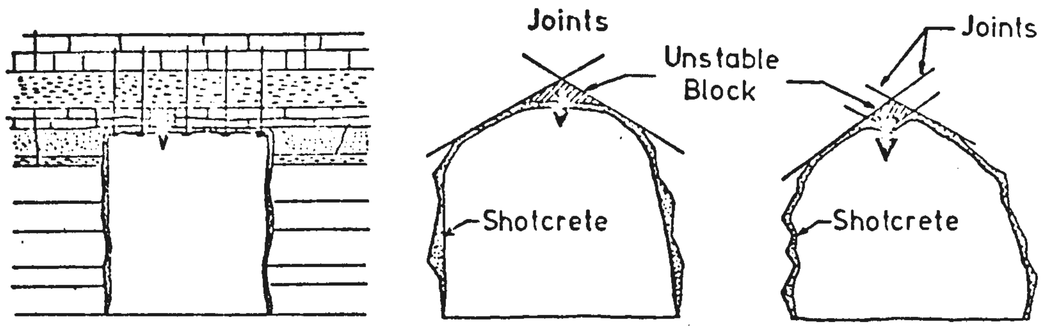
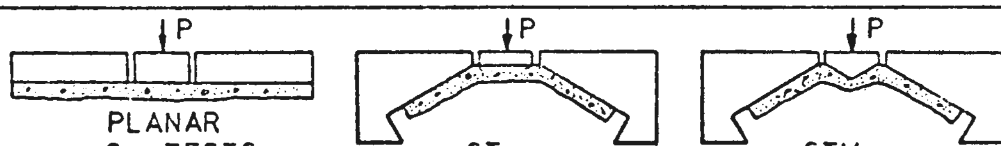
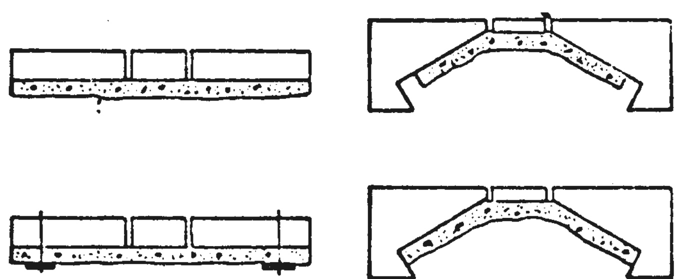
Recently, studies were carried out at a test site in the Atlanta Research Chamber, Atlanta, Georgia. The rock was a high quality gneiss with widely spaced joints, with no sheared, planar joints in the test area. Thus, the test area was excavated and was stable with no support. The test program at the Atlanta Research Chamber was an extension of the model studies, rather than a field investigation of the support requirements at the test site. Loads were artificially imposed on a shotcrete lining sprayed on the rock surface, in order to investigate actual rock-shotcrete adhesion characteristics. Exposed rock surfaces were fresh and rough, and the bond between the shotcrete and rock exceeded the "good bond" that was present in the model tests.

LABORATORY MODEL TESTS

Large-scale tests to evaluate the structural behavior of thin shotcrete liners were carried out at the University of Illinois Laboratory (Fernandez, Mahar and Parker, 1977). Rock block loading was simulated with a 2-foot x 2-foot movable plate loaded by a ram against a shotcrete layer sprayed over the surface of the plate and the adjacent abutment. The model was made two-dimensional, by placing the shotcrete in a 2-foot-wide strip across the plate. Failures occurred on two opposite sides of the plate; the other sides of the plate were free. The main variables studied in these tests are summarized in Table 1.

Table 2 summarizes the principal failure modes observed in the laboratory tests. The main variables controlling the mode of failure are: (1) geometrical configuration of the concrete wall (the simulated rock surface), (2) end condition of the shotcrete layer, (3) thickness of the shotcrete, and (4) bond strength between the shotcrete and the simulated rock surface.

Adhesive failure developed in poorly bonded shotcrete layers with free end, irrespective of the geometry of the simulated rock surface or the shotcrete thickness. As the bond strength increases, the thinner layers develop a shear failure and the thicker layers, where shear strength exceeds adhesive strength, show adhesive failure. For the adhesive strengths

| | | | |
|---|---|---|---|
| TUNNEL CONFIGURATION |   PLANAR ROOF | Arched with smooth surface | Arched with irregular surface |
| GEOMETRICAL CONFIGURATION & TEST CODE |   PLANAR S- TESTS | ST- | STM- |
| SHOTCRETE STRENGTH | 1000 psi - 3500 psi<br>(6.9 MPa) (24.1 MPa) | | |
| ADHESIVE STRENGTH | LOW<br>10 psi (69 KPa)<br>20 psi (138 KPa) | HIGH<br>60 psi (414 KPa)<br>180 psi (1241KPa) | |
| LAYER THICKNESS | 1.0 in — 6.0 in<br>(2.5cm - 15.0cm) | | |
| LATERAL BOUNDARY CONDITIONS | FREE END<br>SUPPORTED END |   | |
| LAYER REINFORCEMENT | NONE | FIBER | MESH |

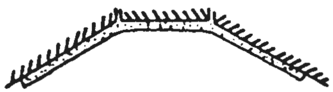
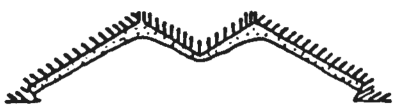
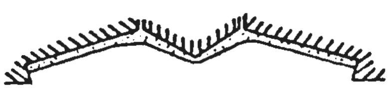
PERTINENT VARIABLES AND TEST MODEL

THIN SHOTCRETE LININGS IN LOOSENING ROCK

TABLE 1

TABLE 2

Failure Modes of Shotcrete Layers Observed in the Laboratory

| | | POOR BOND | | GOOD BOND | |
|---|---|---|---|---|---|
| | | h < 2" | h > 2" | h < 2" | h > 2" |
|  | Planar Free End | Adhesion | Adhesion | Shear | Adhesion |
|  | Planar Supports @ 8' | Bending | Bending | Shear | Bending |
|  | Arched Smooth Surface Free End | Adhesion | Adhesion | Shear | Adhesion |
|  | Arch Smooth Surface Supported End | Moment Thrust | Moment Thrust | Shear | Moment Thrust |
|  | 30° Arch Irregular Surface Free End | Adhesion | Adhesion | Shear | Adhesion |
|  | 30° Arch Irregular Surface Supported End | Bending (apex)* | Bending (apex)* | Shear | Moment Thrust |
|  | 15° Arch Irregular Surface Supported End | Bending (apex)* | Bending (apex)* | Shear | Moment Thrust |

* If enough shotcrete is placed to fill in the recess and develop a smooth arch configuration, the failure mode changes to moment thrust.

developed in the laboratory tests, the maximum thickness at which a shear failure would still develop was found to be 2 inches. The support capacity when shear failure occurs is proportional to the lining thickness; on the other hand, once the adhesive stength is exceeded, increasing shotcrete thickness provides no additional capacity. Field evidence from in-situ tests performed in a good quality granitic rock (gneiss) in the Atlanta Research Chamber showed that the adhesive strength developed between rough rock surfaces and the shotcrete can be several times larger than the values observed in the laboratory so that shear failures can develop in thicker layers.

Moment-thrust failure developed in supported end layers with a smooth arch configuration. Prior to developing the full thrust capacity, the shotcrete separates from the model rock surface. The thrust coefficient, $t_c$, a measure of the capacity at failure, was found to vary between 0.30 and 0.60 depending on the inclination of the arch and the slenderness ratio of the layer, h/l.

Finally, bending failure was observed in end-supported flat layers and in end-supported arched layers over protruding rock blocks with low bond. Bending failure developed after separation of the shotcrete from the rock surface. Relationships established from the large-scale test results for determining the capacities of thin shotcrete layers are shown in Table 3. As indicated in this table, the mode of failure determines the magnitude of the support capacity as well as the typical displacements at failure. Typical values of the adhesive strength, $a_o$, as well as the shotcrete shear strength, $f_d$, developed for the different geometrical configurations tested are also given in Table 3. Typical thrust, $T_c$, and bending,

## Capacities of Unreinforced Shotcrete Layers Tested in the Laboratory

| Failure Mode | Capacity | | θ 0° | θ 15° | θ 30° | Range of Typical Displacements at Failure (in.) |
|---|---|---|---|---|---|---|
| Shear | $P = f_d \cdot h \cdot 2L$ | $f_a$ | 0.05 | 0.05 $f'_c$ | 0.10 $f'_c$ | .004 - .07 |
| Adhesion | $P = 2a_o \cdot 2L$ | $a_o$ (1) | 0.05 | 0.05 $f'_c$ | 0.10 $f'_c$ | .02 - .05 |
| Moment thrust | $P = 2T_c \cdot \sin\theta \cdot f'_c \cdot h \cdot L$ | $T_c$ | - | 0.30 | 0.45 to 0.60 | .15 - .25 |
| Bending | $P = Bc\ f'_c\ h^2\ L/d$ (2) | $B_c$ | 0.12<br>0.32 (3) | 0.12<br>0.32 (3) | 0.25 | .02 - .20 |

(1) $a_o$ values are for good bond

(2) d = distance between rock bolts or any other support element when planar conditions are present.
d = half of the moving block width in the case of an arch configuration, $\theta \geq 30^\circ$.

(3) Using a 4 x 4 x 1/8 in. mesh as reinforcement.

P = total load

$f_d$ = shear strength developed along the shotcrete layer

2L = total length of contact between the movable and fixed blocks (48 in. in the tests)

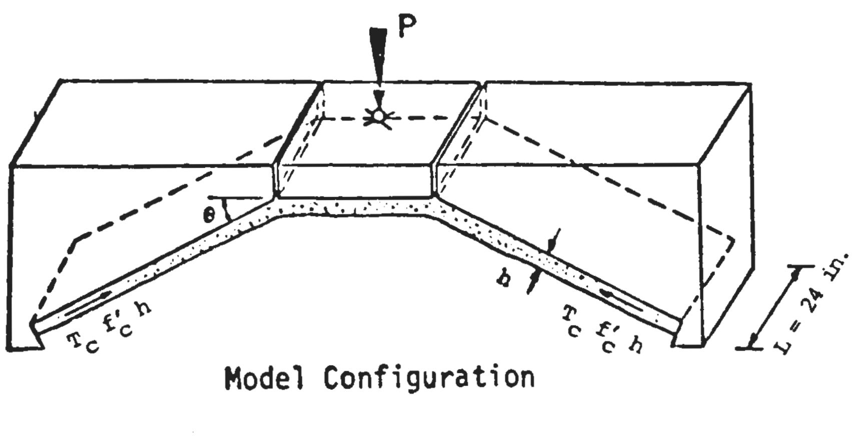
h = thickness of the layer

$a_o$ = adhesive strength developed between the shotcrete layer and the rock.

$T_c$ = dimensionless thrust coefficient, given by the ratio of the axial layer load at failure to the maximum compressive strength times the cross-sectional area of the layer

$f'_c$ = compressive strength of the shotcrete, measured in prismatic, 3 in. x 3 in. x 6 in. samples

$B_c$ = a dimensionless bending coefficient.




Model Configuration

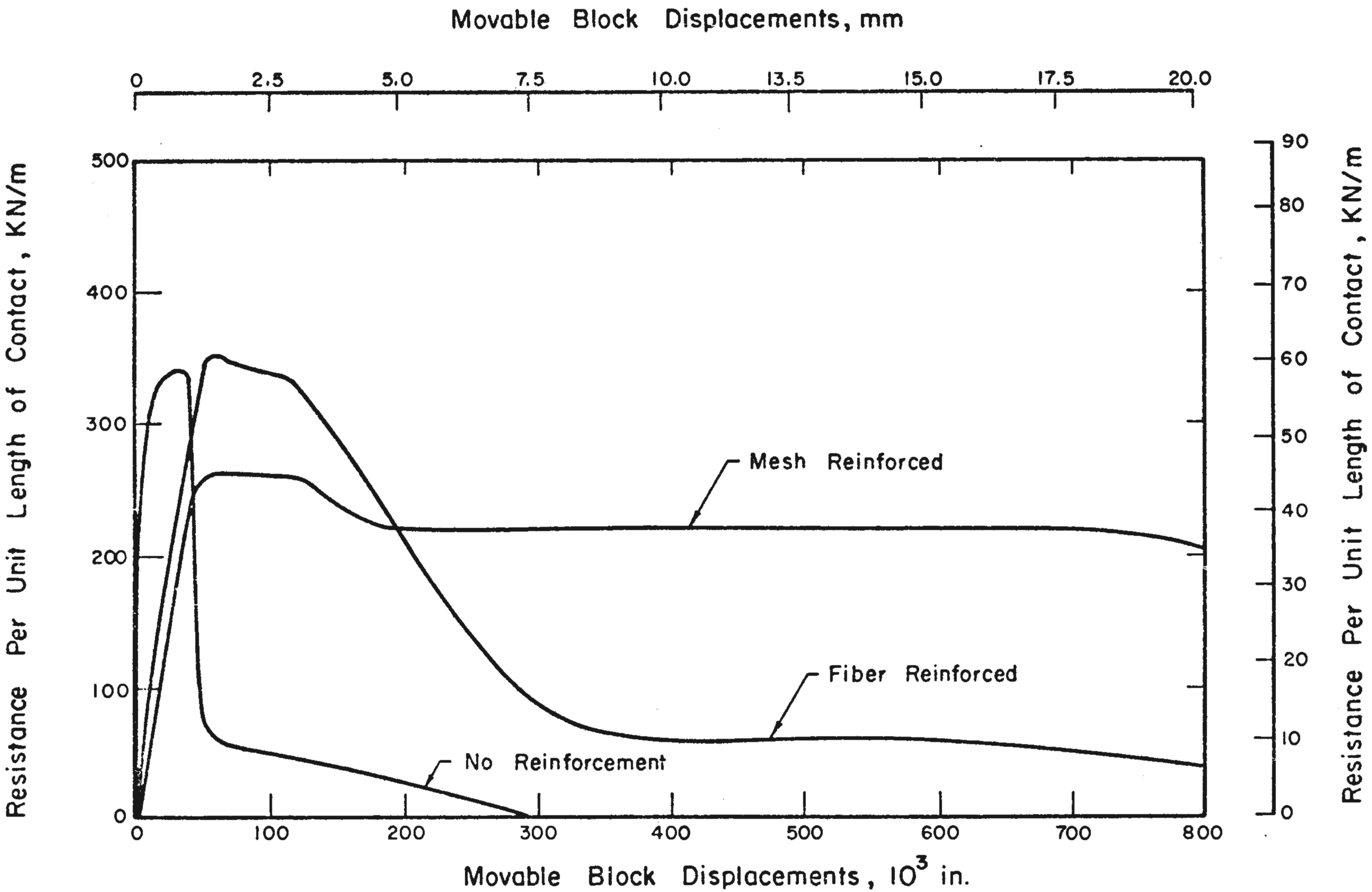
TABLE 3

$B_c$, coefficients measured in the end-supported layers tested are given in Table 3 also.

The effect of different types of reinforcement in the planar shotcrete layers tested can be appreciated in Figure 1. As indicated in this figure, the presence of steel fiber reinforcing (3 percent by weight of 1 x 0.01 x 0.022 inches; 2.54 x 0.025 x 0.055 cm fibers) did not affect the value of the residual capacity. However, the presence of the fiber significantly increased the "ductility" of the layer which exhibited an undiminished residual capacity at movable block displacements 3 times greater (.90 inches) than the failure displacements for unreinforced planar shotcrete layers.

A 4-inch x 4-inch square, 1/8-inch diameter mesh placed close to the outside surface of the layer, with 8-foot span supports, resulted in a five-fold increase in the residual capacity of the layer. Moreover, this residual capacity was maintained through movable block displacements five times larger than those at which complete collapse of unreinforced shotcrete layers took place.

In the case of planar end-supported layers, the beneficial effect of reinforcement cannot be so effectively achieved by any other means. However, in other cases, such as irregular, poorly bonded shotcrete layers, it may be effective to increase the capacity of the layers by filling up the recess areas until a smooth arch configuration is obtained. It should be noted that this procedure will change the failure mode from bending along the apex into a moment-thrust failure along the inclined portion of the layer.




TYPICAL LOAD-DISPLACEMENT RELATIONSHIPS FOR LATERALLY-RESTRICTED LAYERS IN THE PLANAR CONFIGURATION

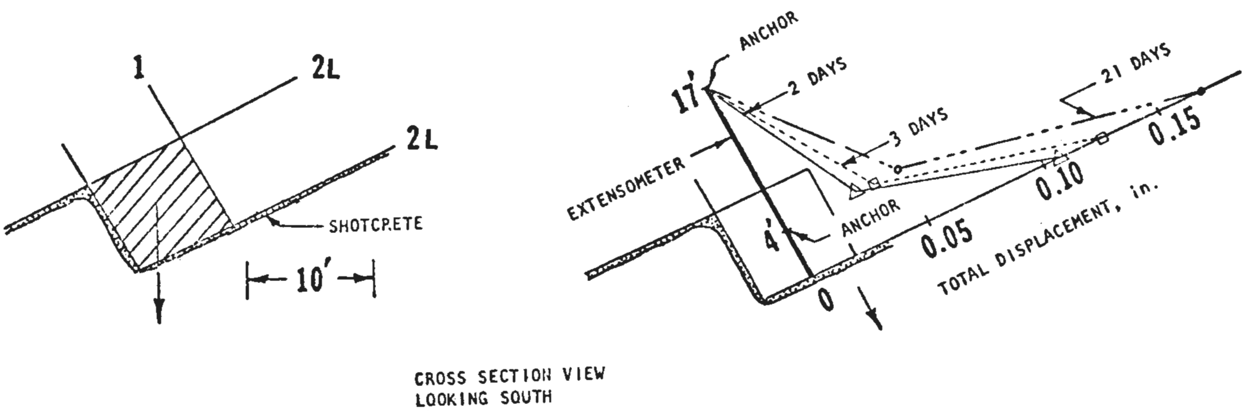
FIGURE 1

## FIELD OBSERVATIONS, WASHINGTON, D. C. METRO

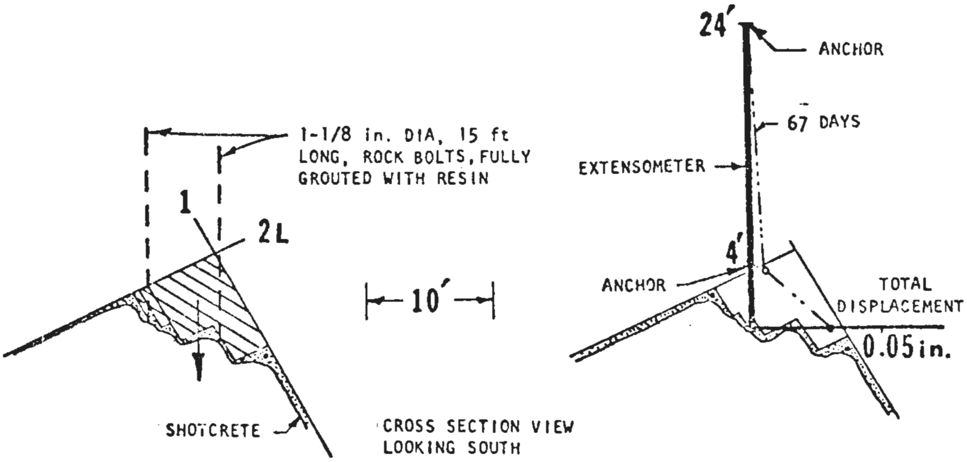
Documented field cases observed in the underground excavations carried out for the Metro System in Washington, D.C. are presented here in order to illustrate the performance of shotcrete linings in tunnels excavated in blocky to blocky and seamy rock conditions, where bond strength is often poor.

Observations were performed during construction of 30-foot wide and 22-foot high double track tunnels (Mahar, Gau, and Cording, 1972). These tunnels are either supported with shotcrete and rock bolts or with shotcrete and steel ribs. The initial support consisted of approximately two inches of shotcrete sprayed over the crown and arch within one to two hours after blasting. Concurrently, muck was removed from the tunnel heading. When these operations were complete, the drill jumbo was moved up to the heading in order to install either the rock bolts or steel ribs.

The rock is predominantly foliated schists and gneisses, with foliation dipping approximately 50° to 70° to the west and striking north-south, almost parallel to the long axis of most of the rock stations. The major geologic feature affecting support in this rock are (1) the shear zones parallel to the steeply dipping foliation and (2) sets of joints and shears which are planar, slick, and sometimes coated with a clay gouge. The joint sets and shears are present in sufficient orientations to produce blocky rock, with blocks typically 2 to 6 feet in dimension, which must be supported as the headings are opened. Typical shotcrete compressive strength at 28 days was equal to 5000 psi.




Rock Displacement in a Shotcrete-Steel Rib Section of Tunnel (Mahar, Gau and Cording, 1972). FIGURE 2a




Rock Displacement in a Shotcrete-Rock Bolt Section of Tunnel (Mahar, Gau and Cording, 1972). FIGURE 2b

Case 1

An example of rock wedge movement in a section where steel ribs were used as part of the permanent support is shown in Figure 2a. A rock block approximately 8 feet x 8 feet was bounded by joint surfaces coated with clay and chlorite. The slickensided joint surfaces not only facilitiated block movement along the joints but also presented a poor adhesive surface allowing the block to slip along its contact with the shotcrete. Maximum block movements were 0.17 inches. The rock block eventually settled onto the steel supports.

A comparison between the maximum weight likely to develop per unit length of liner (equal to the weight of the rock wedge), and the maximum support capacity that could be developed per unit length of liner can be expressed as:

$$F.S. = \frac{P}{W(\text{rock wedge})}$$

where $W = 8(\text{ft}) \times 8(\text{ft}) \times 1(\text{ft}) \times 160 \text{ pcf} = 10{,}240 \text{ lbs}$

$$P = B_c \, f'_c \, h^2 \, \frac{L}{d} \qquad \text{(Table 3)}$$

Large scale tests results indicate an average value of $B_c$ = 0.25 for this configuration. It is assumed that a portion of the weight of the rock wedge began to act on the shotcrete liner a few hours after the shotcrete was applied. It is known that measured rock wedge movements after two days were equal to 0.10 inches, which is almost equal to the failure displacements measured in the large scale tests. It is estimated that the compressive strength of the shotcrete material, $f_c$, at the time of loading would have been equal to or less than 2000 psi;

therefore,

$$P = 0.25 \times 2000 \times 6.25 \times \frac{1}{4} = 780 \text{ lbs.}$$

Assuming that only 25% of the wedge weight was acting on the liner at this time, the resulting factor of safety can then be expressed as

$$F.S. = \frac{P}{W} = \frac{780}{2560} = 0.33$$

The low factor of safety is consistent with the observed behavior; cracking of the shotcrete liner took place and additional support elements, steel ribs in this case, were required to provide adequate roof support.

In other cases, as illustrated in the following example, even though the load was larger than the capacity of the shotcrete liner, the combined action of the shotcrete and timely installation of additional support, such as rock bolts, provided adequate roof support without cracking the shotcrete liner.

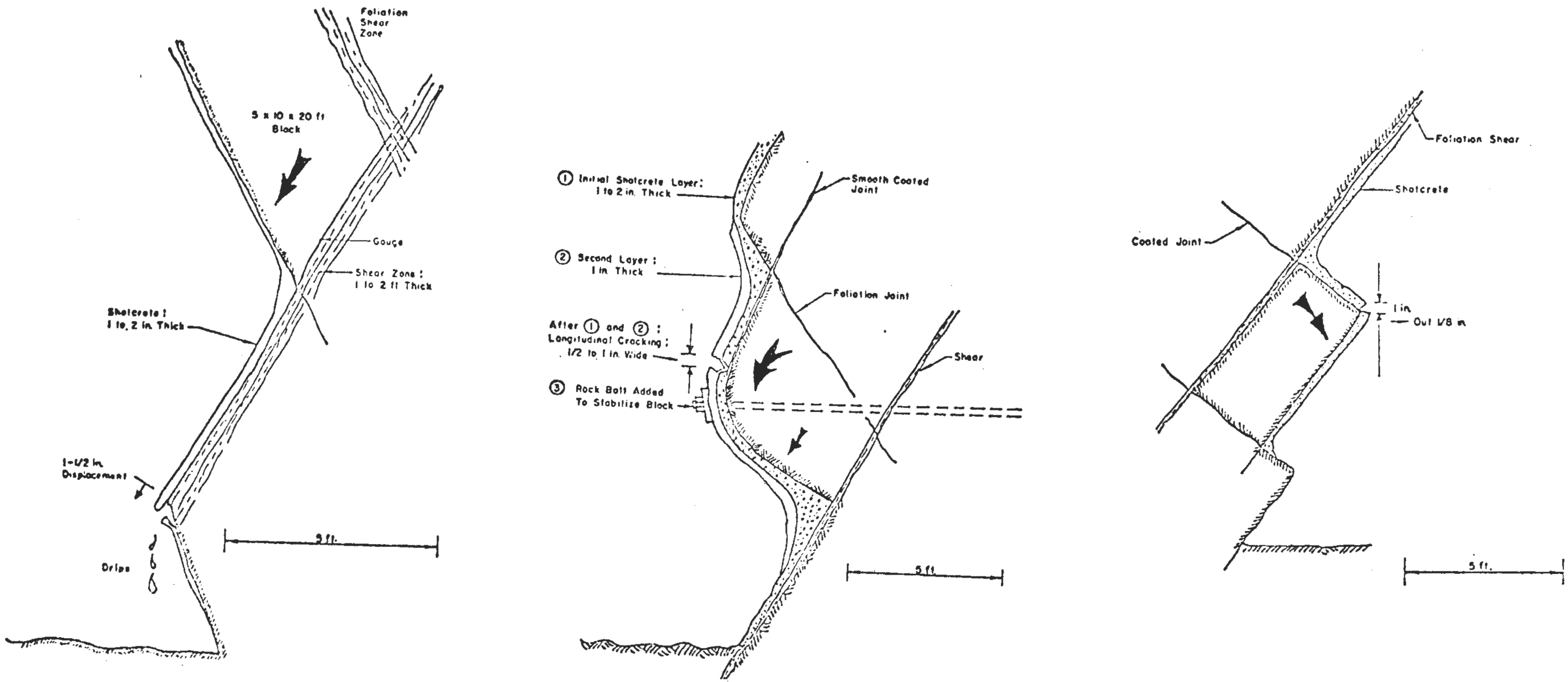
## Case II

The sliding rock wedge treated in this example was observed in the section of the tunnel supported with shotcrete and with 15-foot long rock bolts fully grouted with resin and placed on 4-foot centers. Rock bolts were installed within 2 feet of the tunnel face. In this case, continuous joints combined to form an 8-foot x 10-foot wedge of rock hanging in the tunnel arch, as indicated in Figure 2b. Both joints were coated with up to 1/4 inch of soft clay gouge, exhibited minor slickensides, and contained water. On previous rounds, overbreak had been occurring along both of these joint surfaces. Bolts were angled forward at the heading through the block prior to advancing the tunnel. These bolts prevented further overbreak along the joint planes. The entire wedge consisted of several blocks averaging 1.5 to 2.0 feet in size, separated by several tight joint planes.

Rock movement of 0.030 inch was recorded between the crown and the 40 foot anchor when the heading was advanced 5 feet beyond the extensometer on Day 18. Most of the movement is believed to have involved separation along the continuous joint surfaces which bounded the wedge. Less than 0.002 inch movement was recorded behind the No. 2L plane (between the 4-foot and 24-foot anchors). Measured rock wedge movements indicate that only a small portion of the wedge load was transferred into the shotcrete liner and most of the load was taken by the rock bolt. Case II provides a good example of a successful combination of shotcrete and rock bolts.

Field observations of different displacement modes of rock blocks covered with shotcrete are illustrated in Figure 3. Rotation, outward displacements of a block, and slippage along the shotcrete-rock interface were observed for the tunnels in Washington D.C., where the shotcrete was placed over irregular surfaces bounded by clean smooth planar and often slick joint surfaces (Cording, 1974).

Field observations at sections of the Washington D.C. tunnels instrumented with strain gages embedded in the shotcrete,




Field Observations of Different Displacement Modes
(Cording, 1974).

FIGURE 3

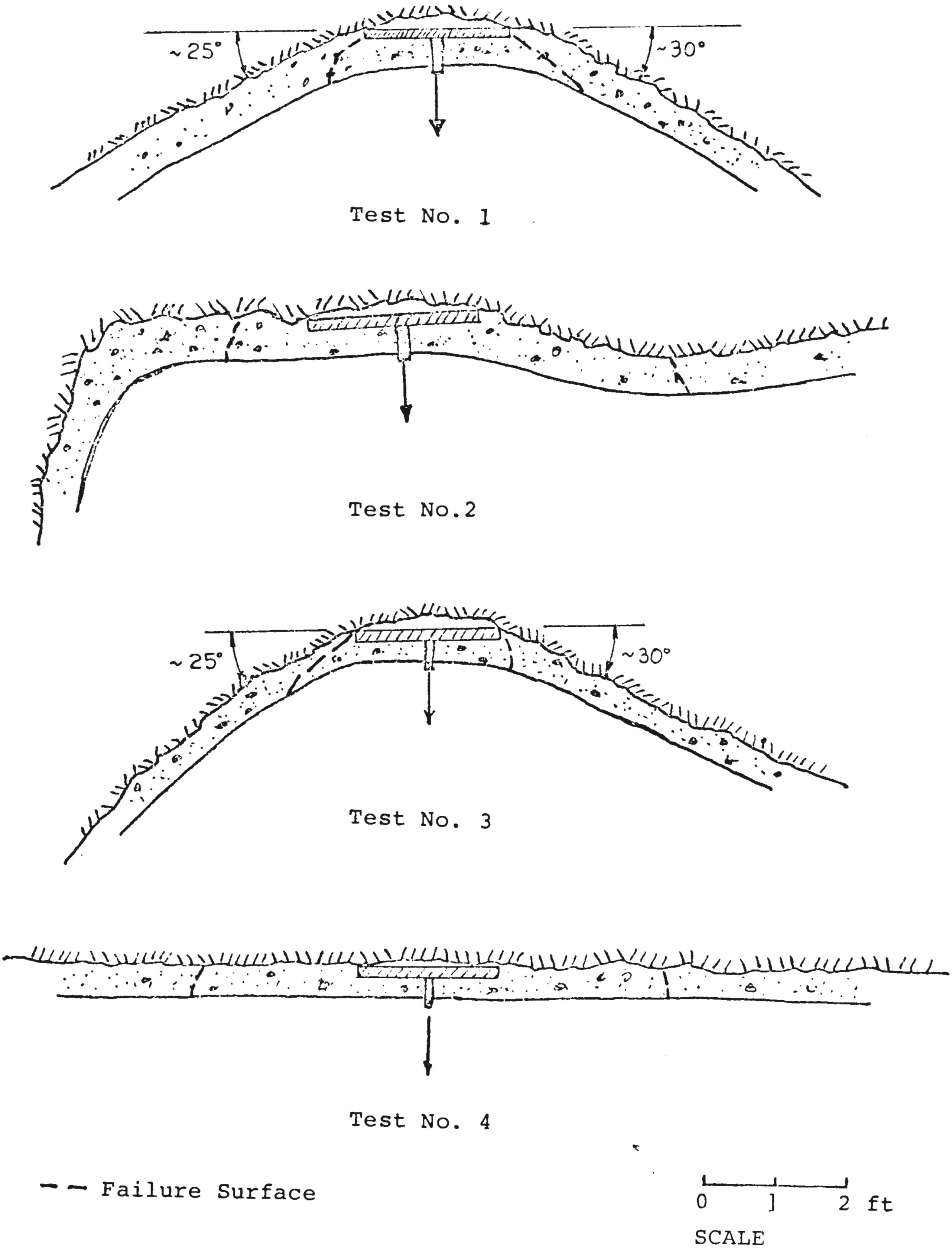
have shown that tensile stresses are often developed in the shotcrete placed over the smooth surfaces of protruding blocks (Jones and Mahar, 1974). These results have also been modelled using the Finite Element Method (Jones, 1976), and provide further confirmation of the importance of using tensile reinforcement when shotcrete is placed on tunnels with irregular and smooth or slick surfaces.

In the blocky and seamy schistose rock of the Washington D.C. tunnels, shotcrete was primarily used in combination with other supports and was not adequate as sole support (a) in the larger tunnels (20 feet or more in diameter); or (b) in zones where blocks were bounded by smooth to slick joint surfaces, overbreak was prominent, and block sizes were typically 4 feet or more in width; or (c) on vertical side walls greater than 10 feet in height that were backed by steeply dipping joints.

## IN-SITU TESTS, ATLANTA RESEARCH CHAMBER, MARTA

A shotcrete testing program has been conducted in a test section in the Atlanta Research Chamber. The rock is a foliated granitic gneiss, with foliation dipping approximately 15° to 25° to the SE and striking north-northeast, almost parallel to the axis of the cavern. Foliation joints are tight, wavy and very irregular. The two other major joint sets have almost vertical dip; one strikes north-south and the second strikes east-west. The rock surface in the cavern is dry and irregular.

During the testing program, four 2-foot x 2-foot steel plates embedded in the shotcrete were pulled to evaluate the capacity of the shotcrete placed in-situ and to obtain a direct comparison with the laboratory tests (Figure 4). The plates




Test Configurations and Failure Surfaces. MARTA, Atlanta.

FIGURE 4

Summary of Shotcrete Capacity Tests. Atlanta Research Chamber.

| Test No. | Configuration | Shotcrete Age | Reinforcement | Failure Mode | Capacity | $f'_c$ psi | Thickness in. | Capacity lb. | Shear Strength $f_d$ | Adhesive Strength $a_o$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Arch | 10 hr. | None | Shear | $P = f_d \cdot H \cdot 2L$ | 1400 | 8 | 52,000 | 0.10 $f'_c$ | - |
| 2 | Flat | 24 hr. | None | Adhesion | $P = 2\ a_o \cdot 2L$ | 3500 | 8 | 50,000 | - | 0.15 $f'_c$ |
| 3 | Arch | 7 hr. | Fiber (3%) | Shear | $P = f_d \cdot h \cdot 2L$ | 400 | 4.5 | 7,000 | 0.08 $f'_c$ | - |
| 4 | Flat | 11 hr. | Fiber (3%) | Adhesion | $P = 2\ a_o \cdot 2L$ | 900 | 4.5 | 6,000 | - | 0.07 $f'_c$ |

$f'_c$ = compressive strength of the shotcrete, measured in prismatic, 3 in. x 3 in. x 6 in. samples.

$f_d$ = shear strength developed along the shotcrete layer.

$a_o$ = adhesive strength developed between the shotcrete layer and the rock.

L = width of the shotcrete layer (24 in.)

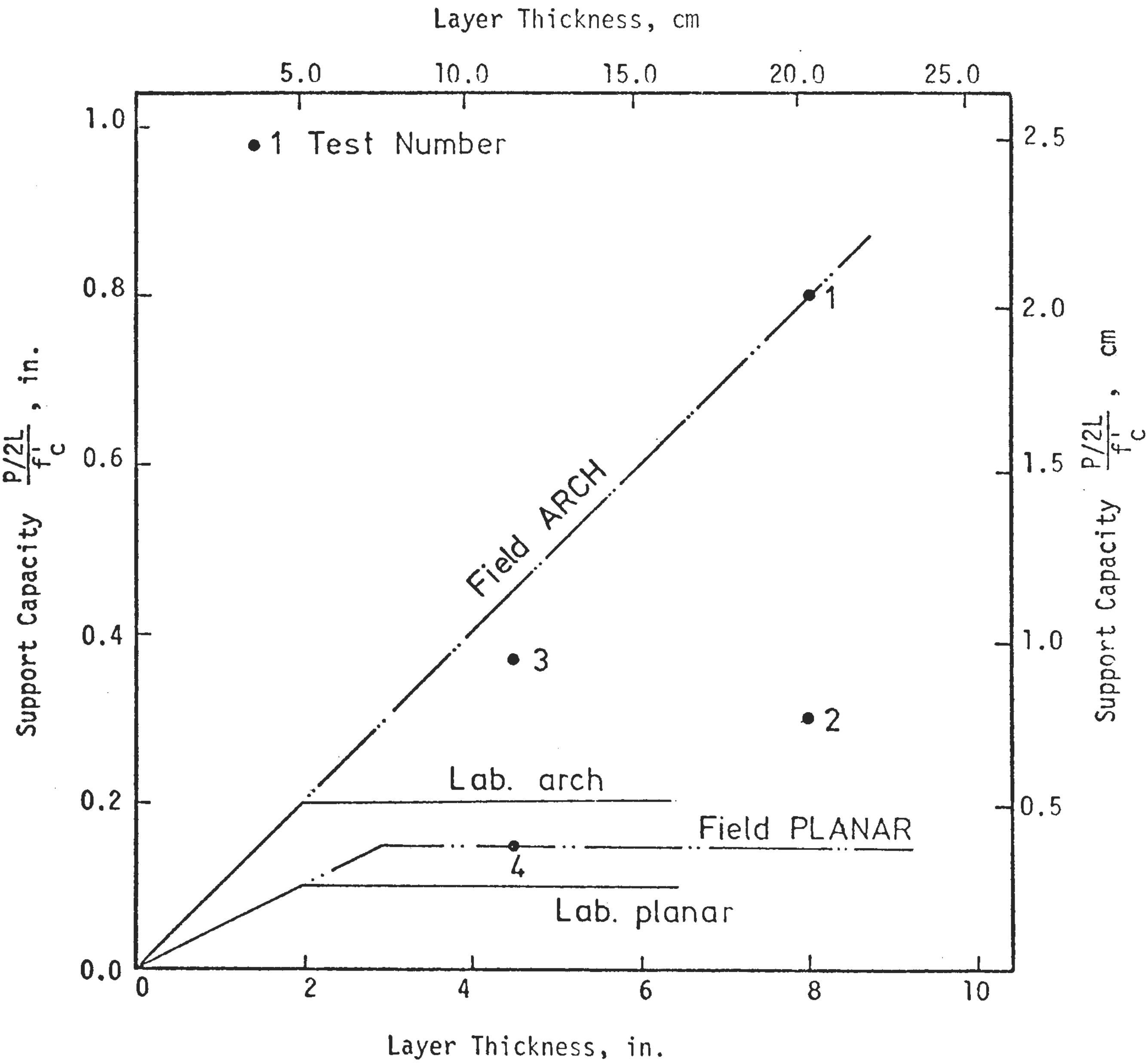
H = thickness of the shotcrete layer.

TABLE 4

were placed in contact with the rock and covered with a layer of shotcrete slightly wider than 2 feet and extending some 8 feet away from each plate. After the shotcrete had set, the plates were pulled with a hydraulic jack and the load measured with a load cell. Two plates were covered with conventional shotcrete and two with steel-fiber-reinforced shotcrete. In each case, the two plates were placed so as to test one planar and one arch configuration (Figure 4). The results are summarized in Table 4.

Shotcrete layers in flat configuration failed by adhesion and developed adhesive strengths higher than those observed in the laboratory ($a_o$ varied from 0.07 $f'_c$ for Test No. 4 to 0.15 $f'_c$ for Test No. 2, which exhibited a slight curvature; values of $a_o$ = 0.05 $f'_c$ were obtained in the planar lab tests). Both the 4 inch and 8 inch thick shotcrete layers with arched configurations failed in shear; shear stength values, $f_d$, equal to 0.1 $f'_c$, consistent with the lab test results, were exhibited by the arch shaped surfaces. However, the results indicate that the adhesive strength required for the shear failure to develop was considerably higher than that measured in the lab. Test No. 1 on the 8 inch layer indicated that the adhesive strength for the good quality, rough-surfaced gneiss in the arch configuration may be as great as 0.4 $f'_c$ (540 psi), as compared with 0.1 $f'_c$ for the lab tests on concrete surfaces.

The results from all tests suggest that natural irregularities in a dry and clean rock surface can increase the adhesive strength several times beyond the values measured in the laboratory where shotcrete was applied over a concrete surface. The increase is larger for layers in arch configuration because compressive stresses tend to develop at the irregularities and




Maximum Capacity of High Adhesion Shotcrete Layers with Free End. Field Results from Atlanta compared with Laboratory Studies.

FIGURE 5

the shotcrete must fail in shear. Figure 5 summarizes the capacities of these layers and compares them with the laboratory results.

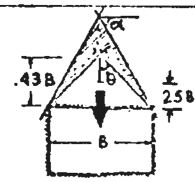
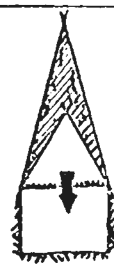
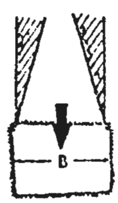
The addition of fiber reinforcement increased the ductility of the layers although it did not increase their capacity. Visual observations during the tests demonstrated that the flat shotcrete layer with fiber reinforcement developed a series of visible cracks and moved 1 inch to 2 inches before failure, whereas the flat shotcrete layer without fiber reinforcement failed brittlely, with little warning.

## ESTIMATING SHOTCRETE REQUIREMENTS

### Rock Conditions Affecting Shotcrete Loads and Behavior

Prior to specifying shotcrete, the following rock conditions should be evaluated at a potential tunnel or cavern site.

1. Determine the geometry of the critical rock wedges requiring support. Irregular, discontinuous and tight joint surfaces will cause only local fallout within a few feet of the opening (Figure 6). Such joint surfaces are most likely to cause overbreak when they form less than a 30° angle to the tunnel surface. Sheared planar surfaces can form deeper wedges. Therefore, one of the major goals of the exploration program should be to evaluate the frequency, occurrence, and the characteristics of the potential shear planes and shear zones at the site. The size of the potential rock wedges in the cavern or tunnel can be estimated by considering relationships such as those outlined in Figure 6.

| (α) DIP ANGLE | (θ) HALF ANGLE | (nB) HEIGHT of EQUIVALENT ROCK LOAD | MINIMUM CONDITION FOR FAILURE | |
|---|---|---|---|---|
| 0° - 30° | 90°- 60° | (0 - .15)B | Both planes wavy, offset |  |
| 30° - 45° | 60°- 45° | (.15 - .25)B | One plane wavy or offset; One plane smooth to slightly wavy |  |
| 45 °- 60° | 45°- 30° | (.25 - .43)B | One plane sheared, continuous and planar. One plane slightly wavy |  |
| 60° - 75° | 30°- 15° | (.43 - 1.0)B | Both planes sheared, continuous and planar |  |
| 75° - 90° | 15° - 0° | > 1.0B | Low lateral stresses in arch; Surfaces planar, smooth, possibly open; or progressive failure aided by separation along low angle joints |  |

Conditions for Wedge Formation in Tunnel Crown Based on Observations in New York City and Washington, D.C. (Cording and Mahar, 1974).

FIGURE 6

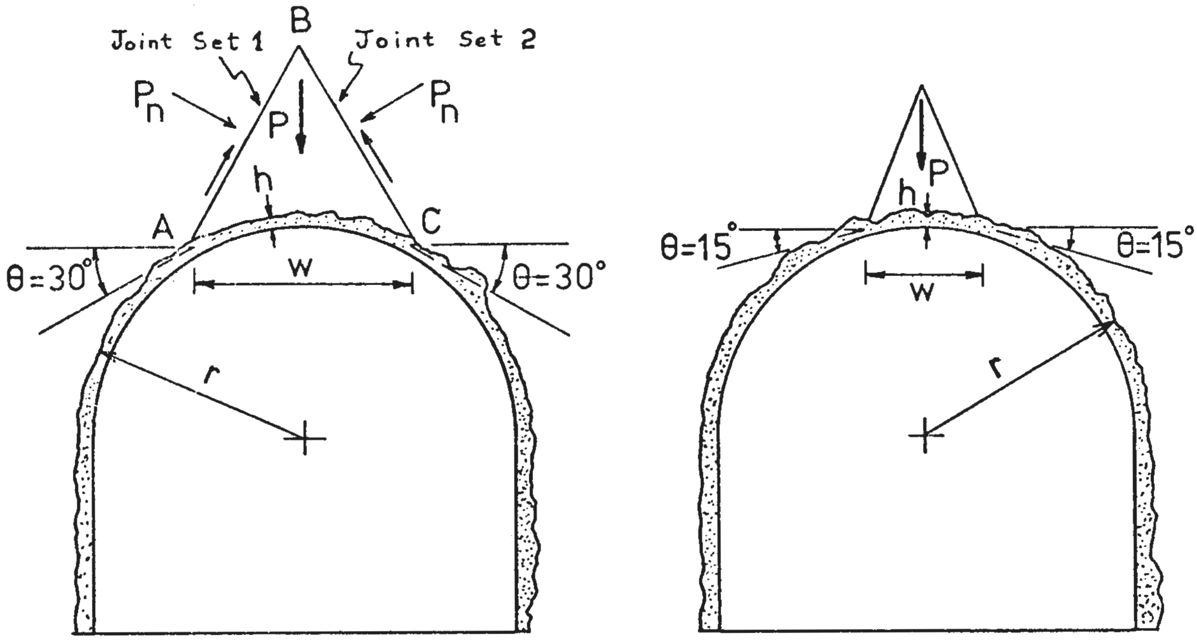
2. Estimate the potential configuration of the tunnel surface. Both the rock properties and excavation procedures should be considered in determining whether the tunnel surface will be planar, smooth arches, or irregular and overbroken.
3. Determine the adhesion characteristics of the rock joints. Micaceous foliation planes, shale bedding planes, slickensided and polished shear surfaces, and altered and clay-filled joints are examples of natural rock fractures having low adhesion. Such features will significantly reduce the capacity of shotcrete applied to planar or overbroken and blocky rock surfaces.

## BASIC DESIGN PROCEDURE

Figure 7 illustrates how the field and lab test results can be applied to design liners for underground openings. Rock wedge sizes are determined by the geometrical characteristics of the discontinuities present in the surrounding rock mass. Joint Sets 1 and 2 in Figure 7 are assumed to have shear strengths and irregularities on their surface that are not sufficient to keep wedge ABC from displacing into the tunnel. In this case, the displacement reduces the normal force, $P_N$, along the joints, increasing the load to be supported by the liner. A limiting load condition, assuming firm rock surrounding the tunnel, will be that in which the total weight of the moving wedge, W, is acting on the support liner.

For an abutment angle, $\theta$ , of 30°, the wedge width, w, would be equal to the tunnel radius, r, and the geometrical configuration of the wedge-support system approaches that of the steep


Equivalent Shotcrete Geometry




$w = r$, $\theta = 30°$, $T_c = 0.4$ $\qquad$ $w = 1/2\,r$, $\theta = 15°$, $T_c = 0.3$

$$P = 2\,T_c\,[\sin\theta]\,[h]\,[f'_c \cdot L]$$

where h is the thickness of the continuous portion of the arch.

Cross Section. Triangular Moving Wedge in Smooth Surface Tunnel.

FIGURE 7

($\Theta$ = 30°) smooth-arched layers tested in the laboratory, Table 3. Minimum thickness of shotcrete required to provide a safety factor of one, assuming the layer to have an adequate end support, can be calculated using a thrust coefficient of 0.45. Preliminary results indicate that, in most cases, if a uniform lining has a thrust capacity sufficient to support a large wedge, a smaller wedge of similar shape will also be adequately supported. As the wedge size and weight decreases, the thrust coefficient also decreases and the thrust acts at a flatter angle. However, since the decrease in maximum support capacity is less than the decrease in wedge weight, the smaller wedge has a higher factor of safety than the larger wedge. For a wedge with a width, $\omega$, less than 0.5R ($\Theta$ < 15°), the planar geometry case described in a previous section, Table 3, would be the limiting, and conservative, case.

## POOR BOND

If the shotcrete to rock bond is poor, then, in most cases, a thin shotcrete lining is not adequate for sole support of rock blocks. However, the shotcrete may serve as a supplemental support between the major elements of the support system, such as rock bolts. Where bond is poor, shotcrete failures are in tension and the use of reinforcement to develop bending and tensile capacity also becomes important.

In poor bond cases, the shotcrete can provide only limited support of rock blocks in the tunnel heading, and it is often necessary to place other support elements, such as rock bolts, close to the face in order to provide adequate protection. Blocky and seamy rock, with large blocks and slickensided surfaces, would fall into this category.

In bedded deposits such as shales, a thin shotcrete layer may be placed on a horizontal bedding plane parting in the roof to limit slaking and deterioration of the rock surface. However, if the bond is poor, the shotcrete may tend to loosen and separate from the roof under its own weight. The shotcrete can be held in place if it is reinforced with wire mesh which is tied into the rock bolt support system. In this case, it would be necessary to place the reinforcement with the first applications of shotcrete.

The capacity of a shotcrete layer reinforced by a 4-inch x 4-inch x 1/8 inch wire mesh between rock bolts can be analyzed as a slab in bending with an average moment coefficient, $M/(f_c bh^2)$ equal to 0.08.

Another alternative where shotcrete bond is poor is to increase the thickness of the shotcrete so that irregularities are filled in and the shotcrete acts as a continuous arch. The lining can then be analyzed as an arch, using appropriate thrust coefficients, as illustrated in Figure 7. In such analyses, the larger wedge widths are usually critical, because their weight increases faster than does the thrust coefficient, which grows larger with the increase in abutment angle (Refer to Figure 7). Critical wedge sizes can be selected on the basis of considerations given in Figure 6. Continuous, regular arches with good bond can be analyzed in a similar manner.

GOOD BOND

Thin shotcrete lining applied to irregular rock surfaces can provide substantial capacity when the rock-shotcrete bond is good. The size of two-dimensional (long) wedges that can be supported with a minimum two-inch thick layer of shotcrete is

7-day (3000 psi) shotcrete: Maximum Wedge Weight = 9000 lb/ft
for $a_o = .05\ f'_c$

| | Wedge Shape | Maximum Wedge Width, w ft | Maximum Cavern radius if $w \leq R/2$ ft |
|---|---|---|---|
| Deep Wedge |  15° | 7.6 | 15 |
| Inter-mediate Wedge |  45° | 15 | 29 |
| Shallow Wedge |  60° | 19 | 39 |

7 hr (1000 psi) shotcrete: Wedge Weight = 3000 lb/ft
for $a_o = .05\ f'_c$

| | Wedge Shape | Maximum Wedge Width, w ft | Maximum Cavern radius if $w \leq R/2$ ft |
|---|---|---|---|
| Deep Wedge |  15° | 2.5 | 5 |
| Inter-mediate Wedge |  45° | 5 | 10 |
| Shallow Wedge |  60° | 6.5 | 13 |

Two-dimensional Wedge Sizes for Adhesion Failure of
Well-bonded Shotcrete Layer, 2 inches or More Thick

TABLE 5

illustrated in Table 5, assuming that the rock surface in the vicinity of the wedge is planar or protruding, rather than arched. The values given in Table 5 are not design values. Load factors and factors of safety are not addressed.

The block size that can be supported at the tunnel heading would be determined on the basis of the early strengths of the shotcrete. At 2 hours, with $f_c$ = 400 psi, the maximum wedge weight for adhesive strength equal to 0.05 $f_c$ would be 1200 lb/ft, equivalent to a slab 4 feet wide and 2 feet deep.

Wire mesh reinforcement can be beneficial for limiting crack widths and providing the post-crack resistance that permits a more ductile failure. Steel-fiber-reinforced shotcrete would be particularly useful in the tunnel heading, because it would provide ductility that cannot otherwise be provided, due to the difficulty in installing mesh immediately in the heading. The results of the laboratory model tests and the field tests in the Atlanta Research Chamber clearly show that the steel-fiber-reinforced shotcrete allowed substantial deformation and visible cracking to occur prior to collapse, rather than the more brittle failures that are typical of the unreinforced shotcrete.

Guidelines for shotcrete use can be established, but exact requirements are difficult to establish prior to construction. Small differences in rock and construction conditions can result in changes in support requirements. For this reason, simple field observations are useful early in the project for developing the support requirements. One of the advantages of shotcrete is the flexibility of the operation, and the ability to place it incrementally, observing the rock-shotcrete behavior as the support is increased until it can be determined that an adequate support has been achieved.

It is difficult to predict exact requirements for shotcrete use prior to construction. Ideally, there should be enough flexibility in the contract to permit additional shotcrete to be placed, if required. For example, in poor ground, more shotcrete can be placed close to the tunnel heading to develop stability, while the shotcrete may be delayed and kept out of the tunnel cycle if ground conditions improve.

Even with flexibility in the usage of shotcrete on a project, it is important that, prior to preparing the specifications, the designer properly assess the potential behavior of the support system to determine if shotcrete is a feasible system; determine the need for reinforcement; and estimate the timing and amount of additional support elements required to provide adequate support. The characteristics of the rock must therefore be carefully analyzed prior to construction.

The final shotcrete requirements for a cavern also depend on other factors, such as the maintenance requirements and permanence of the lining, and the potential exposure of job personnel or the public in the cavern.

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# USE OF STEEL-FIBER-REINFORCED SHOTCRETE IN ATLANTA SUBWAY ROCK TUNNEL

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## PURPOSE OF STUDY

Steel-fiber shotcrete, with its desirable properties of ductility and resistence to sudden brittle failure, is expected to be an improvement over conventional shotcrete when appropriately used to support tunnels cut through rock. In the course of researching steel-fiber shotcrete under actual production conditions, this material was placed on a 200-linear foot (61-linear meter) section of one of the twin running tunnels of the Metropolitan Atlanta Rapid Transit Authority (MARTA) system. This full-scale placement of steel-fiber-reinforced shotcrete on a production basis was used to compare field behavior, laboratory properties, and costs of placement with the properties of conventional shotcrete in the adjacent twin tunnel. The documented use of steel-fiber-reinforced shotcrete is a significant step forward in American tunnel practice.

## CONTRACTUAL ARRANGEMENT

The steel-fiber shotcrete was applied as a part of the MARTA contract for Peachtree Center Station and Subway Lines, Project Number CN120. The design was done by Parsons, Brinckerhoff, Quade & Douglas, Inc./Tudor Engineering Company (PB/T), a joint venture. The construction was done by a joint venture of Horn Construction Co., Inc. and Fruin Colnon Corp. (H-F/C). The Urban Mass Transit Administration (UMTA) of the United States Department of Transportation granted funds totalling $430,000 for various research endeavors, including engineering studies and field inspection, relating to constructing subway tunnels in rock; the steel-fiber-reinforced shotcrete was part of this study. The Atlanta Research Chamber (Ref. 1) constituted the bulk of the UMTA research expenditures. The application of steel fiber shotcrete to a running tunnel was not in the original contract. A change order for a net cost amount of $1,662 for time and materials added 200 feet (61 meters) of steel fiber shotcrete and deleted 200 feet of conventional shotcrete with wire mesh.

## TUNNEL CONDITIONS

Steel fiber shotcrete was placed in the NR tunnel beginning 315 feet (96 meters) and ending 115 feet (35 meters) south of the southern limit of the Peachtree Center Station cavern (Ref. 2). This section of running tunnel was selected on the basis of consistency of rock, both as to hardness and as to the absence of major significant faults, shear zones, or other discontinuities. This tunnel section had been mined in October, 1978.

The street surface elevation varies from 1069 feet (326 meters) at the south end to 1076 feet (328 meters) at the north end of the steel-fiber-reinforced shotcrete section. The elevation of refusal to soil boring and sampling procedures varied from 1053 feet (321 meters) at the south end to 1059 feet (323 meters) at the north end. The invert elevation varied from 960 feet (293 meters) at the south end to 958 feet (292 meters) at the north end. The crown was 18 feet (5.5 meters) above the invert. Hard rock cover above the crown of the tunnel varied from 65 to 75 feet (20 to 23 meters). The original water table elevation varied from 1047 feet (319 meters) at the south end to 1051 feet (320 meters) at the north end. The bearing of the tunnel section varies from N15°E at the south end to N6°E at the north end.

The predominant rock type is muscovite-biotite gneiss with lesser amounts of biotite gneiss and amphibole gneiss. Orientation for the four distinct joint sets are N55°W, 30°SW; N01°E, 85°SE; N55°E, 15°SE (follows foliation); and N10°E, 50°NW. The joints were generally healed except for joint set no. 3, which was open. Below a level 15 feet (4.5 meters) above the crown, the average value for jointing intensity is 0.16 joints per foot (0.52 joints per meter). At the tunnel elevation interval, the recoveries in NX and NQ cores were 100 percent; rock quality designation (RQD) index was usually 100 percent with a minimum of 90 percent.

The tunnel had been driven from north to south by drilling and blasting. The only support prior to shotcrete was occasional epoxy rock dowels as needed for rock blocks adjacent to unfavorable joints. Due to joint set N01°E, 85°SE, there were some slabby breaks along the wall. The crown was slabby, being controlled by the foliation and subparallel open joint set no. 3. The rock surface was blocky and angular, having broken mainly along joints and foliations. No water was seeping from the crown, and only traces were in the walls. Significant amounts of water had accumulated in the invert, primarily due to drainage from upgrade.

The clearance width of the tunnel is 16'6" (5.0 meters). Each wall is 9 feet (2.7 meters) high, and the arch length is about 21 feet (6.4 meters). Drawings indicated nominal thickness of four inches (0.1 meter) for shotcrete on the walls and crown of the tunnel. This four-inch thickness of shotcrete would average about 13 square feet (1.2 $m^2$) in cross-sectional area, per running foot of tunnel.

## MIX DESIGN

The mix design of steel fiber shotcrete consisted of the following proportioning of solid ingredients per cubic yard (cubic meter):

| | | |
|---|---|---|
| Portland Cement | 660 pounds | (392 kilograms) |
| Fine Aggregate | 1790 pounds | (1062 kilograms) |
| Coarse Aggregate | 1300 pounds | (771 kilograms) |
| Steel Fibers | 120 pounds | (71 kilograms) |
| Accelerator | 9.9 to 14.7 pounds | (5.9 to 8.7 kilograms) |

The dry-mix process was specified. The ingredients used by the Contractor included Williams Bros. Type I cement, Sigunit powder accelerator by Sika®, and United States Steel Fibercon® carbon steel fibers having dimensions of .010 " x .022" x 1" (.25 x .56 x 25 mm).

Gradation of coarse aggregate was according to ASTM C33, Size Number 7, as follows:

| Sieve | Percent Passing |
|---|---|
| 19.0 mm (3/4 in.) | 100 |
| 12.5 mm (1/2 in.) | 90 to 100 |
| 9.5 mm (3/8 in.) | 40 to 70 |
| 4.75 mm (No. 4) | 0 to 15 |
| 2.36 mm (No. 8) | 0 to 5 |

Gradation of fine aggregate was according to ASTM C33, as follows:

| Sieve | Percent Passing |
|---|---|
| 9.5 mm (3/8 in.) | 100 |
| 4.75 mm (No. 4) | 95 to 100 |
| 2.36 mm (No. 8) | 80 to 100 |
| 1.18 mm (No. 16) | 50 to 85 |
| 600 µm (No. 30) | 25 to 60 |
| 300 µm (No. 50) | 10 to 30 |
| 150 µm (No. 100) | 2 to 10 |

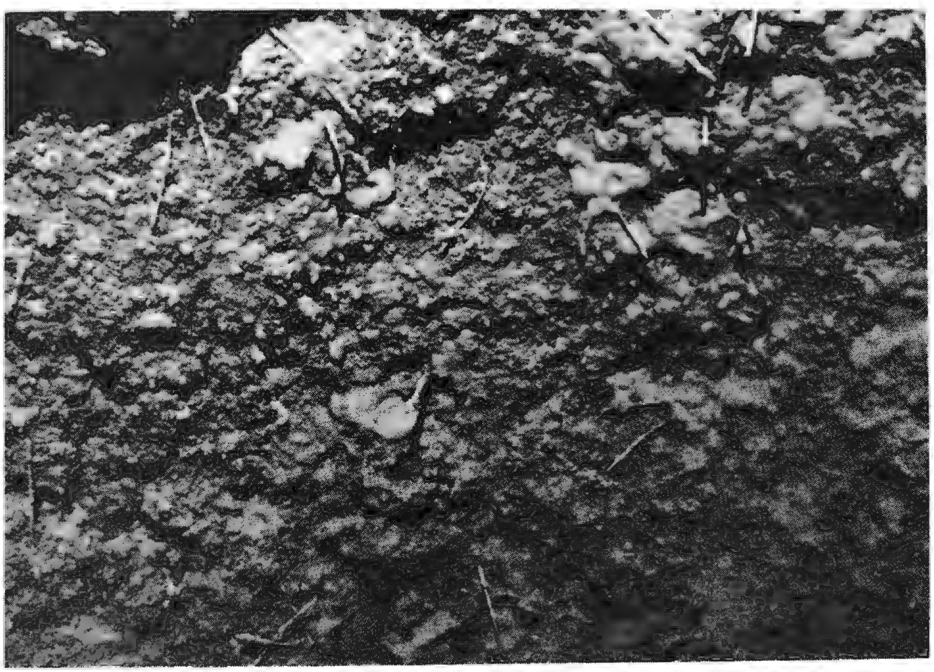
All aggregates were manufactured at a local quarry.

## EQUIPMENT

Bagged cement was hauled by an eight cubic yard Wagner ST-8. Aggregate was hauled by a 2-1/2 cubic yard (1.9 cubic meter) crawler-mounted front-end loader, Case Model 1450. The platform for the nozzleman was mounted on a modified D-6 Caterpillar crawler.

The dry mix machine was a five-cubic yard (3.8 cubic meter) Blanck-Alvarez unit with an air-driven ICOMA IGM-75 pump mounted on a trailer (see Photo page 4). The dry mix machine was not specifically designed for fiber shotcrete, so a Hansen Fiber Meter was added (see Photo page 3 and Figure 1). It consisted of a dispensing drum to drop fibers onto the belt feeding into the pump. The drum had constant rpm, but the quantity of fibers was regulated by a baseplate within the drum, which is extended or retracted by four all-thread bolts, controlling

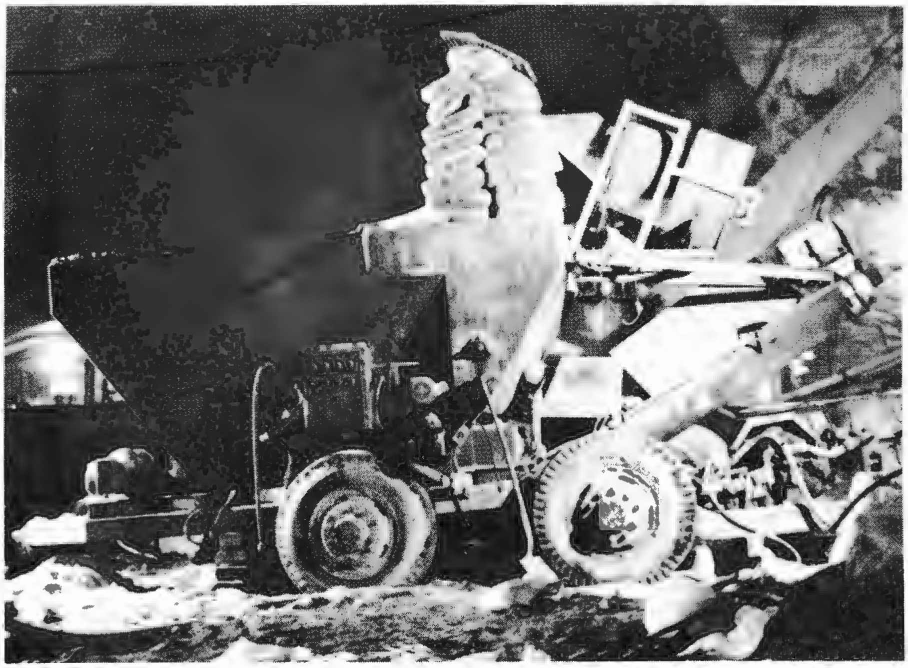
Wire Mesh and Shotcrete

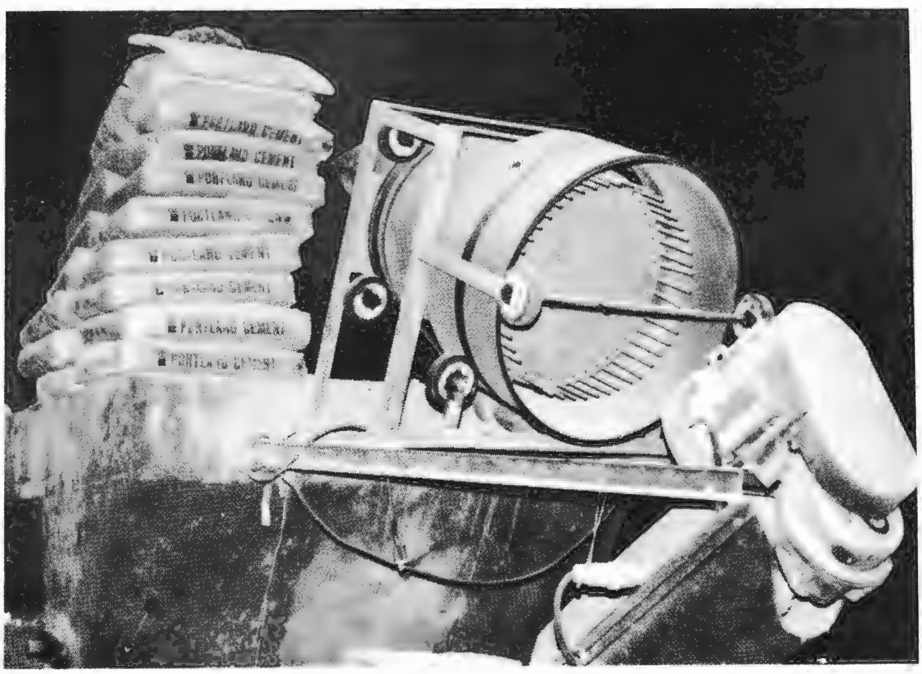


Steel Fiber - Shotcrete "Stick-out"

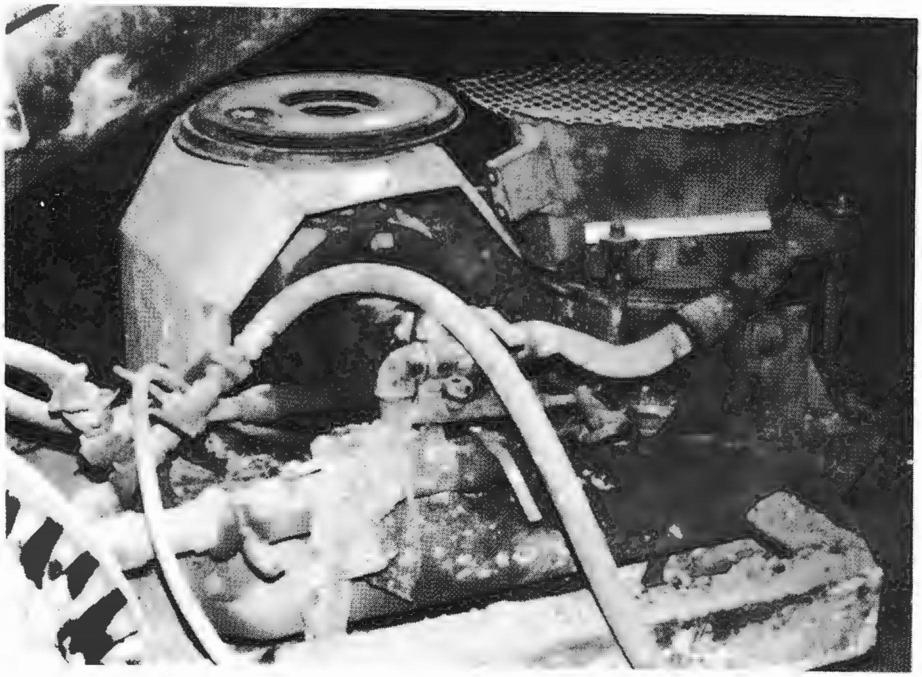
Shotcrete Jumbo

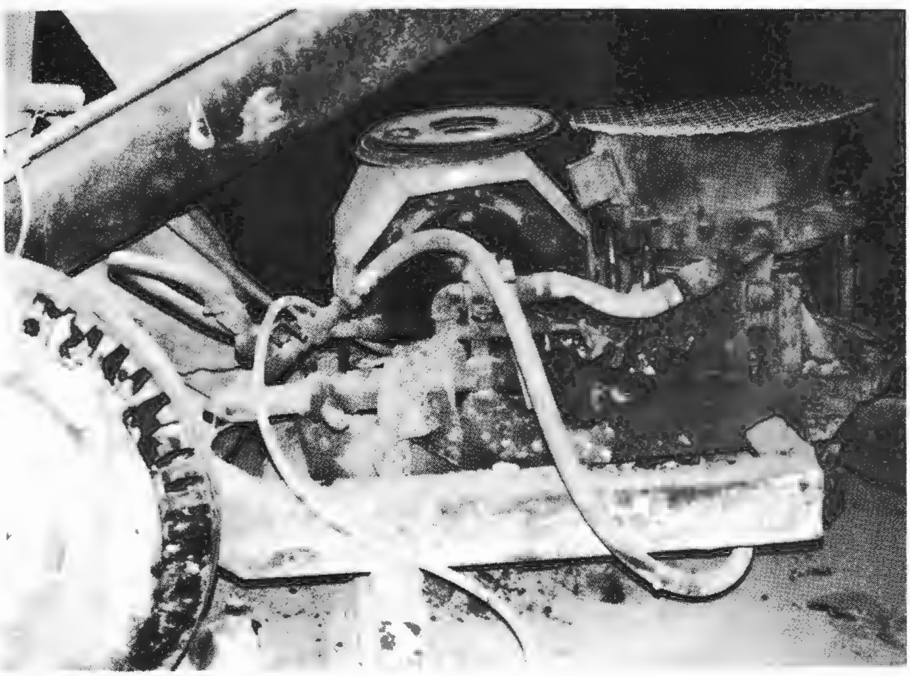
Shotcrete Nozzle






Blanck-Alvarez ICOMA Pumping Unit
with Close-up of Hansen Fiber Meter.





Grout Pump and Hopper

HANSEN FIBER-METER

MODEL 200
(Patent Applied For)

SPECIFICATIONS


| | | |
|---|---|---|
| Height | | 41 in. |
| Width | | 34.5 in. |
| Length | | 62 in. |
| Hopper: | | |
| Height above base | | 38.5 in. |
| Width | | 13 in. |
| Length | | 26 in. |
| Fiber Drum: | | |
| Diameter | | 24 in. |
| Length | | 23-31 in. |
| Capacity | | 6-8 cu. ft. |
| Fiber Discharge Height | | 1 in. |
| Power | 0.5 hp | 115v. |
| Weight | | 371 lb. |

This machine meters steel and stainless steel fibers for concrete, gunite and refractory mixes. Fibers are dumped into the hopper and they are discharged completely separated at a uniform, predetermined rate (about plus or minus 1%).

The r.p.m. of the drum is constant. The rate is adjusted by turning only 4 nuts to expose more or less of the grate through which the fibers fall. Metering rate may vary from 0 to about 90 pounds per minute depending upon the kind of fibers.

Fibers can be metered into conveying augers carrying a uniform volume of material. Only about 3 feet of auger length is necessary to mix the fibers into either a dry mix or a concrete mix. Premixing fibers with sand may be done for gunite. When using a drum mixer, the fibers can be spouted into the revolving drum. A timer can be used to deliver the correct amount of fibers.

When it is not possible or desirable to mount the meter so that fibers flow into the mix a blower can be used for conveying the fibers from the meter.

Because fibers will not flow out of a bin it is suggested that a belt conveyor be used to load the meter if once-a-day loading is desired. The length and speed should be selected for the job. Packages of fibers can be dumped on the belt and it can move automatically or by manual control.

FIGURE 1

the size of the opening between the end of the drum and the baseplate. Rods extending outward from the end of the drum sift the fibers to prevent them from falling in clumps.

The mix hose was 100 feet (30 meters) long. The nozzle had an inside diameter of two inches (50 mm). There was no air gage, but pressure was assumed to be between 40 and 80 psi (276 and 552 kPa). The velocity of discharge from the spray gun is unknown, but the contractor was directed to decrease discharge velocity for steel-fiber-reinforced shotcrete from the velocity used with conventional shotcrete, to compensate for its relatively greater rate of fallout.

To the south, about four hundred linear feet (120 meters) of conventional shotcrete on wire mesh was applied in approaching the tunnel section, requiring steel-fiber-reinforced shotcrete. The nozzle length was varied with the conventional shotcrete. The nozzle was about ten inches (254 mm) long originally, but the mix appeared to be agitated insufficiently. When a 20-inch (508-mm) nozzle was used, the mix appeared to be starting to set up, according to the Contractor's shift engineer. The nozzle was shortened a little at a time and ended up 15-3/4 inch (400 mm) long. This length of nozzle then was held constant for the steel-fiber-reinforced shotcrete.

## SHOTCRETING OPERATIONS

The steel fiber shotcrete was applied in November 1979 from south to north. Shotcrete was applied in two shifts. A typical crew consisted of one operator at the pump end of the mix machine, one part-time operator on the front-end loader bringing in sand and stone, one man on top of the machine dispensing cement and fibers, and one nozzleman.

To begin operations, the nozzleman shot water at the rock surface, cleaning and prewetting it, so that the rock had a clean appearance before the dry mix was fed into the nozzle. He held the nozzle with one hand near the muzzle end while he regulated the water valve with his other hand. He applied shotcrete on the walls up to eight feet (2.4 meters) high while standing on the invert. He shotcreted the arch while standing on the platform. The nozzleman generally applied two lifts on the walls and three on the arch, building up the thickness slowly enough so that the shotcrete would not become overloaded and fall off before it could begin to set up.

The day shift nozzleman shot the crown overhead by holding the end of the nozzle about three feet from the rock surface and methodically making oscillations at the rate of about 45 cycles per minute, with each oscillation two to three feet (0.6 to 0.9 meter) wide. The nozzleman on the swing shift exhibited a less methodical coverage.

In the early stages of shotcrete application, there were frequent cloggings of the hose, severely limiting production. When the hose clogged, the crew would feel along the hose and massage it until the stoppage was cleared. Apparently, the clogging resulted from excessive moisture in the sand, which caused arching bulking. Later, when precautions were taken to supply drier sand, the clogging problem was largely alleviated, until there were only about two occurrences of clogging per shift. Partial and temporary clogging within the hose can be detrimental to uniformity of shotcrete, causing cement-poor sand seams and weak shear zones indicated by low-strength core breaks with failure along sand seams.

## PROTECTIVE ACTIONS

More dust was generated at the mix machine than at the nozzle. The batch men and the nozzleman wore breathing masks. The nozzleman wore gloves; usually the batch men wore gloves also. The nozzleman used goggles only while shooting up on the arch, not while shooting level at the walls. The batch men did not wear goggles. The nozzleman did not like to be close to the target, but usually stayed about four feet (1.2 meter) away from the wall being shot.

Anytime the shotcreting was shut down for moving, meal break or shift change, the batching pump was disassembled and cleaned to prevent accumulation of hardened cement.

## PRODUCTION RATES

Mixing and shooting enough shotcrete to require 150 bags of cement was considered a good shift's work. The production of one shift shot about a length along both walls of about 50 feet (15 meters), with a volume of 21.4 cubic yards (16.4 cubic meters). The next shift covered the corresponding 50 feet (15 meters ) of arch. On one especially productive shift 20,000 pounds (9,072 kg) of cement was used, which supplied about 30.3 cubic yards (23.2 cubic meters) of shotcrete.

The rebound percentage was never actually measured, but was estimated to be one third, for purposes of calculating mix quantities. Rebound of steel-fiber-reinforced shotcrete on the Atlanta Research chamber was measured at 22 percent. Rebound accumulated so high along the walls that it had to be removed so that the lower wall down to invert could be shotcreted.

About 120 pounds of fibers was used per linear foot, or about 3.2 percent of total mix weight. Overbreak and rock roughness required shotcrete thickness to average about six inches instead of the nominal four inches, causing the original supply of steel fibers to be depleted in 150 feet of tunnel instead of the required 200 feet. More fibers had to be ordered.

## FINAL APPEARANCE

Fiber stickout is less than one might expect from the fiber content. In some places, one could feel with one's hand the fibers sticking out every three to six inches, although in other places they were much more frequent. Fiber stickout varies by factor of several times. In some places the fibers lie flat. Some variation in quantity of stickout might be due to variation in fiber content.

As water content is increased, the rebound decreases; but at some point the proportion of water to mix is reached at which sag, slough, and fall-off become a problem. The optimum water content from the nozzleman's point of view (least rebound) is considerably higher than the optimum water content for the designer (highest strength). Most shotcrete applied was wetter than the mix in which the water/mix ratio results in highest strength. The final appearance of the shotcrete indicates the water content at the time of application.

When the applied shotcrete mix is too dry, the resulting surface is rough, with a sandy or grainy texture, and has a clumpy appearance, with numerous small voids, next to which the fibers are visible. Evidently the mix is not plastic enough to close the voids. When the mix is dry, the rebound of large aggregates is high, but most of the paste adheres to the wall.

When the water content corresponds to the highest strength, the surface appears neat and smooth and, except for protruding coarse aggregate, has a dull luster and takes on the subdued shape of the rock surface. Few fibers stick out, and no voids are apparent.

When the mix is too wet, the final surface is uneven, with mounds averaging about 2 inches across. Many fibers are visible around and between the mounds. The general appearance tends to be somewhat sloppy, and the surface begins to have a shiny luster as the water content increases. Some evidences of slough, sag, and creep become apparent, indicating the highly plastic state of the shotcrete during application.

When the mix is much too wet, there are some craters left by fallout of large aggregates, and some holes left by slough or fallout. Fibers generally do not stick out, but are visible parallel to the surface and are covered with cement. The surface appears glazed, and shows grout drips and chevron creep patterns where, in an area a few feet (one meter) across, the center sags more than the edges. The walls somewhat resemble those of a limestone cavern.

## CONTRACTOR'S EXPRESSED VIEWS

The fiber shotcrete superintendent liked fiber shotcrete better than wire mesh shotcrete. With wire mesh the overbreaks (considerable in gneiss) have to be filled in thickly up to the mesh, since it cannot be bent back into the overbreaks. Therefore, for irregular rock surfaces, more shotcrete is required where wire mesh is used than where fiber shotcrete is applied. The one additional ingredient of fibers does not slow down operations much. Fiber shotcrete causes more wear on the pump and hose than conventional shotcrete. Fiber shotcrete itself is definitely less expensive than conventional shotcrete with steel mesh when labor cost of anchoring wire mesh is included.

One Contractor's engineer said that fiber shotcrete went on more easily and adhered better with less rebound than regular shotcrete with wire mesh. On the other hand, another Contractor's man said the conventional shotcrete adheres to the mesh better than the fiber shotcrete adheres to the rock without mesh. This is true, however, only if the nozzleman takes aim at the mesh holes rather than shooting at random; this extra attention minimizes rebound off the wire and covers the mesh from behind so that voids are not formed behind the mesh wires.

However, wire mesh evidently is not the contractor's preferred reinforcement. For him, chain link fence constitutes more convenient reinforcement for conventional shotcrete than the regular four-inch steel mesh. In the first place, it is flexible and can be pulled back into the overbreaks. Secondly, the closer wire spacing greatly reduces rebound losses, almost paying for itself. Thirdly, stiff wire mesh vibrates shotcrete off in the vicinity of impact. The chain link fence vibrates less, causing less fall-off.

## RESULTS OF CORING AND TESTING

The coring machine was powered with compressed air. The drill was anchored to the rock face by rock dowels, and the bit was advanced by a jack on the drill assembly.

Many times during the coring operations the cores broke off through the rock along foliation planes rather than along the shotcrete-to-rock contact surface, indicating effective bonding to the rock. Cores revealed that where the rock surface was rough and angular, some voids had formed in the shotcrete in sheltered corners. In the cores no layering is evident except for the distinctions between the lifts. Occasionally, there

## TABLE 1 - CORE STRENGTHS OF STEEL-FIBER-REINFORCED SHOTCRETE

| Specimen Letter | Station NR | 7-day Strength Each | 7-day Strength Avg. | 21-day Strength Each | 21-day Strength Avg. | 28-day Strength Each | 28-day Strength Avg. | 56-day Strength Each | 56-day Strength Avg. | All Ages Avg. |
|---|---|---|---|---|---|---|---|---|---|---|
| A | 19+50 | 8110 | | 4892 | | 3605 | | 3797 | | |
| | | | 7080 | | 4281 | | 4174 | | 4602 | 5034 |
| B | | 6051 | | 3670 | | 4742 | | 5407 | | |
| C | 19+80 | 4699 | | 3670 | | 4184 | | 5728 | | |
| D | | 7145 | 6373 | 3733 | 3605 | 4699 | 4785 | 4505 | 4977 | 4935 |
| E | | 7275 | | 3412 | | 5471 | | 4699 | | |
| F | 20+00 | 8175 | | 6051 | | 4506 | | 5728 | | |
| G | | 6566 | 7339 | 4956 | 5386 | 4570 | 4549 | 5182 | 5364 | 5659 |
| H | | 7275 | | 5150 | | 4570 | | 5182 | | |
| I | 20+15 | 6244 | | 5600 | | 6508 | | 5793 | | |
| | | | 6630 | | 5439 | | 5153 | | 5956 | 5729 |
| J | | 7016 | | 5278 | | 3798 | | 5600 | | |
| Avg. | | 6856 | | 4641 | | 4665 | | 5162 | | |

NOTES:

1. All strength values are in pounds per square inch (6.9 kPa).
2. All values have been corrected by a factor of 0.91 to make them correlate with an L/D ratio of 2.0.
3. The specimen letters do not correlate between ages. Only the station groupings can be compared as to location.
4. All cores were taken from either the crown or the quarter arch. No further identification of location was made.

BREAK DATES:

| | |
|---|---|
| 7-day | 12-05-79 |
| 21-day | 12-19-79 |
| 28-day | 12-26-79 |
| 56-day | 01-23-80 |

TABLE 2, PART 1

CORE STRENGTHS

OF CONVENTIONAL SHOTCRETE

| Specimen Number | Station | Age | Strength in psi | Date Tested | Avg. Strength |
|---|---|---|---|---|---|
| A-1 | NR 14+70 | 28 | 3282 | 10-26-79 | |
| A-2 | NR 14+70 | 28 | 3605 | 10-26-79 | |
| A-3 | NR 14+70 | 28 | 3990 | 10-26-79 | |
| A-4 | NR 14+70 | 28 | 2897 | 10-26-79 | |
| A-5 | NR 14+70 | 28 | 3540 | 10-26-79 | |
| A-6 | NR 14+70 | 28 | 4055 | 10-26-79 | |
| A-7 | NR 14+70 | 28 | 4441 | 10-26-79 | |
| A-8 | NR 14+70 | 28 | 3089 | 10-26-79 | |
| A-9 | NR 14+70 | 28 | 3797 | 10-26-79 | 3633 |
| B-1 | NR 14+60 | 28 | 3927 | 10-30-79 | |
| B-2 | NR 14+60 | 28 | 3927 | 10-30-79 | |
| B-3 | NR 14+60 | 28 | 3797 | 10-30-79 | |
| B-4 | NR 14+60 | 28 | 5472 | 10-30-79 | |
| B-5 | NR 14+60 | 28 | 3540 | 10-30-79 | |
| B-6 | NR 14+60 | 28 | 3089 | 10-30-79 | |
| B-7 | NR 14+60 | 28 | 4505 | 10-30-79 | |
| B-8 | NR 14+60 | 28 | 3155 | 10-30-79 | |
| B-9 | NR 14+60 | 28 | 4120 | 10-30-79 | |
| B-10 | NR 14+60 | 28 | 3089 | 10-30-79 | |
| B-11 | NR 14+60 | 28 | 2961 | 10-30-79 | 3780 |

TABLE 2, PART 2

CORE STRENGTHS

OF CONVENTIONAL SHOTCRETE

| Specimen Number | Station | Age | Strength in psi | Date Tested | Avg. Strength |
|---|---|---|---|---|---|
| C-1 | NR 14+80 | 29 | 2768 | 11-29-79 | |
| C-2 | NR 14+90 | 29 | 3476 | 11-29-79 | |
| C-3 | NR 15+00 | 29 | 3347 | 11-29-79 | |
| C-4 | NR 15+10 | 29 | N/A | 11-29-79 | |
| C-5 | NR 15+20 | 29 | 3862 | 11-29-79 | |
| C-6 | NR 15+30 | 29 | 4249 | 11-29-79 | |
| C-7 | NR 15+40 | 28 | 4312 | 11-29-79 | |
| C-8 | NR 15+50 | 28 | 3797 | 11-29-79 | 3614 psi (1-16) |
| C-9 | NR 15+60 | 28 | 4635 | 11-29-79 | |
| C-10 | NR 15+70 | 28 | 2189 | 11-29-79 | |
| C-11 | NR 15+80 | 28 | N/A | 11-29-79 | |
| C-12 | NR 15+90 | 28 | 3283 | 11-29-79 | |
| C-13 | NR 16+00 | 28 | 4184 | 11-29-79 | |
| C-14 | NR 16+10 | 28 | N/A | 11-29-79 | |
| C-15 | NR 16+20 | 28 | 2574 | 11-29-79 | |
| C-16 | NR 16+30 | 28 | 4312 | 11-29-79 | 3614 |
| | | | | | |
| C-4 | NR 15+10 | 42 | 5214 | 12-12-79 | |
| C-11 | NR 15+80 | 41 | 3991 | 12-12-79 | |
| C-14 | NR 16+10 | 41 | 3797 | 12-12-79 | 4334 |

TABLE 2, PART 3

CORE STRENGTHS

OF CONVENTIONAL SHOTCRETE

| Specimen Number | Station | Age | Strength in psi | Date Tested | Avg. Strength |
|---|---|---|---|---|---|
| C-17 | NR 16+40 | 28 | 4249 | 12-13-79 | |
| C-18 | NR 16+50 | 28 | 6566 | 12-13-79 | |
| C-19 | NR 16+60 | 28 | 5407 | 12-13-79 | |
| C-20 | NR 16+70 | 28 | 6308 | 12-15-79 | |
| C-21 | NR 16+80 | 28 | 5471 | 12-15-79 | |
| C-22 | NR 16+90 | 28 | 6309 | 12-20-79 | |
| C-23 | NR 17+00 | 28 | 6244 | 12-20-79 | 5793 |
| C-24 | NR 17+10 | 28 | N/A | 12-24-79 | |
| C-25 | NR 17+20 | 28 | 4635 | 12-24-79 | |
| C-26 | NR 17+30 | 28 | 4377 | 12-25-79 | |
| C-27 | NR 17+40 | 28 | N/A | 12-25-79 | |
| C-28 | NR 17+50 | 28 | 4184 | 12-26-79 | |
| C-29 | NR 17+60 | 28 | 4982 | 12-26-79 | |
| C-30 | NR 17+70 | 28 | 4248 | 12-26-79 | |
| C-31 | NR 17+80 | 28 | N/A | 12-27-79 | 4467 |

NOTE 1: N/A denotes unusable samples.

NOTE 2: All cores were taken from either the crown or the quarter arch.

are voids at contact between lifts, although the specific amount of time that lapsed between lifts is unknown.

The Contractor cut cores three inches in diameter and three inches long from four different areas in the crown and quarter arch. The PB/T laboratory broke the steel fiber shotcrete cores at ages of 7, 21, 28 and 56 days, as shown in Table 1, and broke the conventional shotcrete cores at ages of 28 and 42 days, as shown in Table 2. The required compressive strength in 28 days was 4000 psi (27,580 kPa) minimum for test specimens having a length to diameter ration of two.

The only comparable data between strengths of fiber shotcrete and strengths of conventional shotcrete are the 28-day breaks. The average 28-day strengths for conventional shotcrete ranged from 3633 psi (25,050 kPa) specimens A-1 through A-9 to 5793 psi for specimens C-17 through C-23. These values are comparable with the average 28-day strengths for steel fiber shotcrete of 4665 psi (32,170 kPa) for specimens A through J.

There is no definite explanation for why the seven-day strengths exceeded all later strengths in all four zones. However, there is a general trend of increasing strength with order of application (station), probably indicating that as the crews gained experience the strengths increased.

## CONCLUSIONS

The key to making or breaking steel fiber shotcrete is the nozzleman. If he is well trained, he can provide quality work at a profit. The compressive strength of steel fiber shotcrete is comparable to that of conventional shotcrete under actual field conditions. There is a tendency for roof shotcrete to be thinner and weaker than wall shotcrete, because of the nozzleman's lack of ease when working with the materials, because of the addition of accelerators to the mix to hasten drying of the roof work, and, obviously, because of the counter-productive force of gravity. The nozzleman tends to add more water when shooting the crown in order to reduce rebound. Perhaps more protection, such as a grinder's visor, would lessen this temptation to add more water to the mix.

## REFERENCES

1. Rose, Don. "The Atlanta Research Chamber", Proceedings, Rapid Excavation and Tunneling Conference, Vol. 1, AIME, 1979, pp. 751-772.

2. Knesel, Thomas R. and King, Elwyntt. "MARTA's Peachtree Center Station", Proceedings, Rapid Excavation and Tunneling Conference, Vol. 2, AIME, 1979, pp. 1521-1544.

# A CONTRACTOR'S VIEW OF STEEL-FIBER-REINFORCED SHOTCRETE

W.E. (Gene) Root
Project Manager
Horn/Fruin-Colnon Corporation Joint Venture
Atlanta, Georgia

Editor's Note:

The following letter is from Gene Root, Project Manager for Horn/Fruin-Colnon on the MARTA Peachtree Center Station CN-120 contract in Atlanta, Georgia. After steel-fiber-reinforced shotcrete was placed and tested successfully in the Atlanta Research Chamber, the General Consultants (PB/T) made a design change in one of the CN-120 twin line tunnels. The Contractor was directed to use steel-fiber-reinforced shotcrete for 200 lineal feet, instead of the conventional shotcrete with wire mesh. The Owner provided the Hansen Fiber Meter, which retails for about $2,500.00. The Owner also provided the steel fibers, which, in 1978, retailed for about $.265 per pound. The steel fibers were used in lieu of the 8 x 8 - W2.1 x W2.1 welded wire fabric.

Although Gene Root's letter refers to this section of line tunnel as "research", the designers (PB/T) consider the steel-fiber-shotcrete as the permanent final subway tunnel lining, made with a proven construction material.

HORN CONSTRUCTION CO., INC.
AND
FRUIN-COLNON CORPORATION
A JOINT VENTURE
161 SPRING ST., N.W., SUITE 523
ATLANTA, GEORGIA 30303
(404) 577-7550

February 12, 1981

Tudor Engineering Company
149 New Montgomery
San Francisco, Calif. 94105

Attn: Don Rose

Re: MARTA Contract CN-120
Fiber Shotcrete

Dear Don:

On the MARTA Peachtree Center Station, a research project was authorized by the owners and performed by Horn/Fruin-Colnon forces. The details of the research involved the following series of events.

1. The location of the research was in a 650' section of line tunnel that originally was contractually specified to be shotcreted by the conventional method of first placing wire mesh and then applying standard shotcrete.

2. The research section directed to eliminate the wire mesh in a 200 foot section, and to apply shotcrete with steel fibers blended in the mix.

Listed below are the observations that I notice based upon my personal opinion.

1. Time duration to apply fiber shotcrete about 50% less due to the elimination of wire mesh installation.

2. Less rebound due to the absence of wire mesh.

3. Better yield since the specified shotcrete thickness could be used as a guideline and not a thickness that was greater than, in most cases was controlled by the clearance necessary to cover wire mesh. To help clarify the previous statement, one must visualize the near impossibility of installing mesh uniformly against rock in all the cavities that are created by conventional rock blasting methods.

The only difference in applying fibrous shotcrete as far as the equipment that was being used, was the need of a metering device to dispense the steel fibers into the mix prior to entering the distribution chamber of the shotcrete machine. This metering device was furnished by the research team and the cost is unknown to me, but due to the size and the apparent simplicity, I would estimate the cost of it to be very minor.

February 12, 1981

Tudor Engineering Company

Attn: Don Rose

Re: Fibre Shotcrete

The difference in material required is the difference in wire mesh as opposed to the cost of the fibers. The cost of the fibers was borne by the research project and is also an unknown cost to the writer. In conclusion a general evaluation of the steel fiber shotcrete is as follows:

1. Eighteen months after the application it appears structurally as sound as the conventional applied shotcrete adjacent.

2. Concern, by the parties, of having a rough cutting surface due to the steel fibers is not a factor after the passage of time that we have experienced.

3. It is my opinion that the use of fiber shotcrete is a more economical method than conventional and is structually as sound for the intended purpose.

Very truly yours,
HORN/FRUIN-COLNON,J.V.

W.E. Root
Project Manager

WER/jl

# TUNNEL PHOTOGRAPHY

by

John Oliveira
Rodney Morrison
Parsons, Brinckerhoff, Quade & Douglas, Inc./
Tudor Engineering Company
Atlanta, Georgia

## INTRODUCTION

Photography in tunnels may present some unusual, but not insurmountable, problems. This discussion is an attempt to better prepare the individual to deal with some of these problems. It is assumed that the reader has a limited working knowledge of photographic theory and techniques.

### Equipment

Almost any camera with adjustable exposure control may be used successfully underground. The tunnel environment may be hostile to the equipment, however, so some thought must be given to the protection of equipment. Because there are often high moisture and/or dust levels in the tunnel, it would be ideal if the camera could be completely sealed, as is equipment used for underwater work. Since this is generally not done, however, the next best thing is to be aware of the problem and to attempt to neutralize the adverse effects of working underground as much as possible. A thorough cleaning after use to remove dust particles and moisture from all mechanical and electrical parts, as well as any exposed lens elements, is an excellent precaution. The use of a clear filter such as Ultra-Violet on all lenses is good practice. Not only will this protect the lens, but the filter will be easier to clean than the lens element.

There are no real limits to the format size which may be used. However, it is assumed that most people will choose to use the 35mm format. All principles discussed here apply equally to all sizes. If a choice of equipment is available, a camera with a large, bright viewscreen will be easier to focus. Also, any focus system using more than one focus screen type, for example ground glass and split screen, is better than a single. This is largely due to the low light situations commonly encountered in underground work.

Moderate wide angle lenses may be better than normal or telephoto lenses, primarily because these lenses allow work closer to the subject. The advantages of this are:

1. Commonly the tunnel working area is limited, so wide fields of view allow better subject representation.

2. Working close to the subject will reduce the problems caused by fog and dust in the air. This is of particular importance when working with flash.

If any "available light" work is planned, some sort of rigid camera support is essential. The ideal is a sturdy Tripod or Minipod. Some other methods may be useful such as a clamp with a tripod hand attached or, if exposure times are relatively short, a Monopod or Chest Support. Cable Releases are very helpful, particularly if long exposure time is to be used.

Also if "available light" work is planned, a light meter is necessary. Preferably the meter used should be of the combination reflected/incident type. Any type electronic flash may be used. However, the limited output of some of the smaller units may make them unacceptable for tunnel work.

Light

The two most important aspects of light in a photograph are intensity and color balance. Color balance is important only in color photography, but intensity is important with both black and white and color. Both problems apply equally to available light and to flash work. Ambiant light levels may be measured with the standard light meter.

In many underground work areas, such as large caverns and openings, a single light reading may not be sufficient. It is then necessary to take readings in several areas of the scene and pick an average exposure, or make several different exposures and then choose the photo which best suits the purpose at hand.

The incident light meter will be very helpful in scenes which include light sources. The incident light meter will also be of importance at times when low reflective surfaces are being photographed. The procedure with the incident light meter is to take a reading with the meter at the area of the most important aspect and facing the general direction of the light source. An alternative but less effective procedure is to take a reflected light reading from a standard photographic "Gray Card" placed in the "interest center" of the scene.

Regardless of the procedure used to measure ambiant light, it is advisable to bracket exposures, that is, to take several shots at several different exposures both above and below the measured setting. It must be remembered that you will usually not have a lot of opportunity to go back and re-shoot at the original location. An alternative to this is to provide additional light for the scene. This is accomplished in any of several ways.

1. Flood lights may be brought in to illuminate the area. This has the advantage of allowing absolute control of illumination patterns, color balance and exposure. The disadvantages are the substantial time and effort required to handle the equipment and set up each shot. Power requirements can also be a problem.

2. Flash lighting may be used.

   A. Multiple Flash Strobes - Studio Type
   These units allow a complete control of color and light. However, experience is necessary before they can be effectively used.

   B. Multiple Flash - Portable Units
   These weigh light and easy to use. However, the light output of small units is somewhat limited and exposure calculations can be difficult.

3. Use of a Single Strobe - This is probably the simplest method but may have the greatest limitations. The light output of a single unit is limited and puts parameters on the size of the area to be photographed.

When electronic flash units are used, some caution should be used in exposure calculations. Several factors may influence the performance of the flash unit that lower the effective light output. The greater the amount of particulate matter in the air, the less effective the flash output will be. Dust and fog have the effect of both dispersing and blocking some of the light produced. This may cause underexposure due to less than anticipated light levels reaching the subject and/or a "hazy or foggy" picture caused by the flash illuminating the individual airborn particles. The greater the distance from flash to subject, the worse this effect becomes. Haze may be reduced by:

1. Placing the flash closer to the subject than the camera. This requires a long flash line connecting the flash to the camera, or a photocell "slave" on the main flash and a smaller flash unit on the camera used only to trigger the main flash. A second person can also manually fire the flash.

2. Placing the flash unit to one side of the subject rather than on the camera will reduce the "haze" effect by not directly illuminating the particulate matter from the camera's perspective.

3. Using automatic flash units will tend to override some of the exposure problems of the conventional units. These units have a light sensor which measures the light reflected to the unit from the subject area. This sensor controls the duration of the light pulse and cuts the flash off when enough light has reached the sensor to correctly expose the scene. These units must be used with caution when the scene has a large object in the foreground and an underexposed background. If exposure problems come up with this type unit, they may be adjusted by changing the film speed to cause an increase or decrease in light output as needed.

If single flash units are being used in conjunction with "wide-angle" lenses, it may be necessary to place a diffusing lens on the flash to insure even light coverage, as most units are designed to illuminate an area roughly the size of the normal lens coverage. In some situations it may be possible to supplement available light with an artificial source. If flash is used in this situation, the exposure is calculated somewhat differently. The f/stop is controlled by the flash unit in the normal procedure. However, a light meter reading is taken in the area not be illuminated by the flash. The previously determined f/stop is then used with the light reading to determine the needed shutter speed.

Example

The scene to be shot is a large cavern illuminated by floodlights but with one notable dark area. The flash unit is set up at the appropriate spot and it is determined from the exposure scale that the correct exposure is f/8. A light meter reading is taken of the other areas of the scene. The reading indicates a required exposure of f/8 for 12 seconds. The lens is set at f/8 and the shutter is opened, the flash is fired either manually or by electrical connection to the camera, and the shutter remains open for 12 seconds.

Large areas may be illuminated by small flash units using another method. This requires that the camera be set up on a tripod or other stationary support and the shutter locked open, as in the "B" shutter setting on focal-plane shutter cameras. The lens is covered with some opaque material such as a black changing bag. One or more people move into the picture area with flash units and position themselves at a point where they can illuminate a portion of the picture area. The lens is uncovered, the flashes fired, the lens covered, and then each person moves to another area and the process is repeated until the desired area has been covered by flash. Automatic flashes should be used for this process since the flash will give the correct amount of light on the subject each time it is triggered. Manual units may be used if the flash is positioned the same distance from the subject each time.

Ideally each flash should have an identical output, but this is not mandatory. Care is necessary, when attempting this, to guard against overlap of flashes and a resultant over exposure. It is a good practice to always place the flash holders' body between the camera and the flash. This will guard against a light flare from the flash. This procedure is slow and cumbersome; however, it may be the only way in some situations. It is particularly difficult in that careful planning is necessary to insure adequate light coverage and communication between camera and flash operators.

With both this and the preceeding example an additional problem exists if color film is used. It will be necessary to make some sort of color correction of one of the light sources in order to get a good color rendition. This will be discussed in detail later.

## Film

Film choice will be controlled by the prevalent conditions, the intended uses of the photos, and the preferences of the individuals involved. If high intensity lighting is available, low speed films may be used; otherwise, high speeds are recommended. Generally, the higher the film speed, the greater the "grain" of the photo and, as a result, the lower the overall image quality.

The choice of black-and-white or color is simply a matter of need and/or preference. Black-and-white films are available up to very high speeds and in a wide range of contrast qualities. High contrast films are useful in portraying shapes but not necessarily detail. Many black-and-white films have a wide exposure latitude; that is, they will render a subject adequately even if they are over or under exposed by several f/stops. This is of particular importance if the photo areas have high and low light areas. If custom processing or dark room facilities are available, corrections may be made during printing to adequately portray both types of situations.

If both black-and-white and color are required, either color transparencies or color negatives may be made. A black-and-white negative is made from the transparency, or the color negative may be printed onto black and white paper which is sensitive to all colors such as Kodak "Panalure".

The choice between color negative and positive film will be controlled by personal preference and needs. Generally, transparencies are less expensive than prints, with the added advantage that color reproduction is largely not affected by processing. Color printing requires a color reference in order to achieve natural color rendition, and color shifts are not uncommon in automated lab runs. Color negative films generally have a greater exposure latitude than most transparency films.

One problem with some color negative films is their tendency to shift strongly toward the blue end of the spectrum when exposed to electronic flash (for example: Kodacolor and Vericolor).

The film speed should be based on the nature of the light used. Generally, some transparency films may be "Push Processed" to allow the use of a higher film speed. Pushing film one f/stop will not severely affect the results; 2 or 3 stop pushing is possible but should be pursued with caution. It is not advisable to push color negative films as they are less compatible with this process than the transparency films. Not all transparency films can be pushed. Check the manufacturers' literature to be certain.

All films suffer a loss of effective speed when exposed for long periods of time. The phenomenon known as Reciprocity Failure may be corrected by a slight increase in exposure. The film manufacturer will have data sheets giving suggested exposure corrections for this. Any exposure longer than 10 seconds should be suspect, and a slight increase in exposure time or decrease in f/stop will be in order. Reciprocity Failure may also cause a slight shift in color balance.

Transparency films suffer from Reciprocity Failure more severely than negative films. The color shifts which result from this are minor, but may be found to be objectionable. In this case, the use of color compensating (c.c. series) filters may be necessary. The type of filtration needed is supplied by the film manufacturer.

Exposure and Filter compensation for reciprocity of some Kodak film is tabulated on the following page.

| Film | Exposure Time (seconds) | | |
|---|---|---|---|
| | 1 | 10 | 100 |
| Kodacolor II | +1/2 Stop | +1/2 Stops<br>cc 10c | +2 1/2 Stops<br>& cc 10c & 10G |
| Kodacolor 400 | +1/2 Stop | +1 Stop | +2 Stops |
| Ektachrome 200 | +1/2 Stop<br>& cc 10R | Not<br>Recommended | Not<br>Recommended |
| (Tungsten)<br>Ektachrome 160 | +1/2 Stop<br>& cc 10R | +1 & cc 15R | Not<br>Recommended |
| Most Black & White | +1 Stop | +2 Stops | +3 Stops |

(Data from Eastman-Kodak, Rochester, New York)

When a scene is viewed, the brain interprets the colors seen in terms of those it expects. Light from different sources will have some slight differences in actual color. Color films are designed to render the image in terms of the colors that the brain expects.

The balance of color in light is measured in terms of color-temperature expressed as Degrees Kelvin. This is based on the behavior of incandescent light sources which exhibit changes in color of light emitted as their temperature changes. Each color film is designed for use in light of a particular color temperature by chemically adjusting the emulsion layers to reproduce colors as the brain expects them. Below is a general guide to color temperatures:

| | |
|---|---|
| Household Light Bulb | Approximately $2800^{\circ}$ K |
| Flood Light (incandescent) | $3200^{\circ}$ - $3400^{\circ}$ K |
| Daylight | $500^{\circ}$ K |
| Electronic Flash | $6000^{\circ}$ K |
| Skylight | $10,000^{\circ}$ K |

If it is obvious that for most of your work you will be using available light, and the light source is incandescent bulbs, it is advisable to use tungsten light film. If daylight film is shot under only incandescent light, the result will be photos with a yellow-red tint. If tungsten film is shot in daylight or electronic flash, the result will be a strong blue cast. It is not advisable to try to correct for this in the printing process. The best approach is the use of filters during shooting. The following table gives filter corrections for several film-light combinations.

Filtration & Exposure Compensation

| | Film Type | | |
|---|---|---|---|
| Type of Light | Daylight | Tungsten 3200K | Tungsten 3400K |
| Daylight | None | 80A + 3 Stops | 80B |
| Tungsten 3200K | 85B + 2/3 | None | 81A |
| Tungsten 3400K | 85 + 2/3 | 82A + 1/3 | None |
| Fluorescent Daylight | cc 40m + 30y + 1 | 85B + cc 30M + 10y + 1 2/3 | 85 + cc 30M + 10y + 1 2/3 |
| Cool White | cc 30M + 2/3 | cc 50M + 60y + 1 2/3 | cc 50M + 50y + 1 2/3 |
| White | cc 20 c + 30M + 1 | cc 40M + 40y + 1 | cc 40M + 30y + 1 |
| Warm White | cc40c 40M + 1 2/3 | cc 30M + 20y + 1 | cc 30M + 10y + 1 |

(Data from Eastman-Kodak, Rochester, N.Y.)

Light sources may be mixed successfully with a little planning. For instance: if a scene illuminated primarily by tungsten light is to be photographed and additional light is needed, tungsten film may be used in conjunction with an electronic flash if the flash has a filter over the tube to change the color rendition. The filtration will reduce the light output, so unless the flash is an automatic unit the exposure will have to be increased.

Some individuals may find no objection to the color renditions resulting from tungsten light on daylight film. In this case, the use of filtration will be unnecessary. If, however, absolute color control is necessary, additional precautions are necessary. The most involved method would require the use of a color meter to measure color temperature and use of filtration to correct the colors. A more practical approach, if color prints are required, is to include a color reference in the picture. This may be accomplished by having a person in the scene so that the skin tones are clearly visible, or by including a standard photographic neutral gray card in the scene. The best way is to place the gray card in the center of interest or in an area which receives typical lighting for the entire scene. The card should be in a prominent location and close enough to the camera to be clearly visible. An exposure is made of the card, then the scene is photographed without the card but using <u>exactly</u> the same exposure as the first shot. During the printing process the card can be used as a standard to set up exposure and filtration; then the second shot is printed at the corrected values. The gray card has a known color and density, so the photo lab can recreate these. The inclusion of a person is much less precise, but the skin tones may be manipulated to within a generally acceptable range.

This process and a number of other custom operations discussed below are easily within the reach of everyone simply by using a high quality professional color laboratory. It is of paramount importance, however, that good communication exist between the lab and the photographer. Take some examples of typical work to the lab and discuss with the lab personnel what exactly is needed. This way, when the work is submitted, the lab personnel may be aware of what type of alterations are expected and the work will go much more smoothly.

## Custom Operations

A number of custom operations are possible. Color prints can be produced from transparencies by using a positive color paper such as Kodak "Type R" or "Ciebachrome", or the transparency is used to make an internegative. The negative is then used in a standard printing process. Good color rendition is possible from both, and the choice will be controlled primarily by cost considerations. If single prints are needed, the first will suffice; if multiple prints are needed, the second will probably be more cost efficient.

Transparencies may be produced either from color negatives or from color prints. The cost is relatively high for the first transparency, but it has the particular advantage of allowing a color corrected print to be copied onto transparency film. Then multiple duplicates can be made from the copy at a relatively low cost. Transparencies may be slightly altered so that color and density are more acceptable during the copying process.

During the printing process, the composition of any given shot may be altered. This is accomplished by selectively framing and cropping so that unwanted areas are removed from the photo. Although this is not an expensive operation, it is better simply to give a little extra thought to the set-up and framing of the original at the time of shooting. A much more expensive but useful alteration is air brushing. By this process the lab can take some unwanted focal points completely out of the photo. It consists essentially of painting over the unwanted image with dyes so the background becomes consistant. This requires a very skilled operator, thus the price can be high. It does, however, have its uses, and you should be aware of the possibility.

## Techniques

In instances where emphasis on one particular feature is desired it may be advisable to take action to enhance it. This may be accomplished by, for example, painting symbols, outlines, etc., directly on the rock face. If the shot is of a rock feature, it is good practice to include some object as a size reference. A small scale or coin in close shots, or, possibly, people in larger areas will suffice to give a point of reference.

If the intention of a shot is to show the tunnel outline, someone may be placed in the section to be shown. This person faces away from the camera holding a flash; either he manually fires it while the shutter is held open, or the flash is connected to the camera by a long electrical lead. This results in a lighted area in the background with the foreground being dark. Thus a silhouette is formed outlining the tunnel shape.

A technique has been successful where time exposures in a darkened tunnel have been used with a flashlight beam tracing an outline of the walls, roof and floor at a known tunnel station. This photographic outline can then be compared to a

template of the design tunnel outline at the same scale, to reveal the presence of "tights" or other unacceptable as-built conditions.

We recommend that time be taken to shoot several test rolls trying as many different exposures and techniques as possible. This may allow you to find what works best for you. If a particularly important subject must be recorded, making a number of exposures "bracketing" and changing camera and light angles will help insure some acceptable results. Also, remember that some seemingly unusable exposures may be salvaged by custom processing procedures.

Two publications which may be helpful are "Kodak Professional Photoguide"1/ and "Cave Photography". 2/

We wish to express our appreciation to Messrs. Chuck Nelson and David Payne of Meisel Photochrome Corporation, Atlanta, Georgia, for the assistance and consideration extended to us.

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1/ "Kodak Professional Photoguide", Eastman Kodak Company, Rochester, N.Y. 1977.

2/ "Cave Photography", National Speleological Society, Huntsville, Ala.



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