THE ATLANTA RESEARCH CHAMBER

APPLIED RESEARCH FOR TUNNELS

BLASTING TECHNIQUES
CONVENTIONAL SHOTCRETE
STEEL-FIBER-REINFORCED SHOTCRETE

MONOGRAPHS

ON THE STATE-OF-THE-ART OF TUNNELING

JUNE, 1979

INTERIM REPORT

Prepared for

U.S. Department of Transportation

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16. Abstract

This Interim Report discusses research performed for the Atlanta Research Chamber from October 1977 - June 1979. It also includes monographs on the state-of-the-art of tunneling.

This Interim Report is subject to review and change. Interested readers should consult the Final Report issued Sept. 1979.

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ACKNOWLEDGEMENTS

PREFACE

This 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.

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 owners, contractors, labor, 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 is 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.

This June 1979 Interim Report is subject to review and change. The Final Report which will follow should be consulted by interested readers.

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	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	
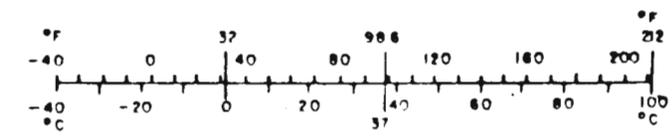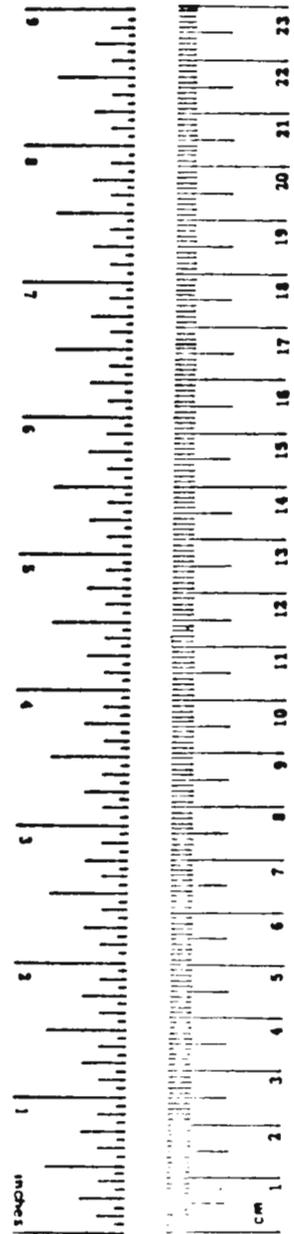
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CHAPTER I.

INTRODUCTION

The desirability of utilizing underground space is apparent to all, and the need for rapid transit sytem 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. 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", Engineering News-Record, December 14, 1979, page 9.

parallel to, the twin Running Tunnels. 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 responsible 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 STRAAM Engineers 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 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 volume may serve to promote the common goal, which is to construct underground space economically and safely.

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

Map labels:

NORTH AV.
CIVIC CENTER
PEACHTREE CENTER
ASHBY ST.
VINE CITY
OMNI
GEORGIA STATE
FIVE POINTS
GARNETT ST.
KING MEMORIAL
WEST END
DORAVILLE
CHAMBLEE
BROOKHAVEN
LENOX
LINDBERGH CENTER
BROOKWOOD
N. DRUID HILLS RD.
ARTS CENTER
N. DECATUR RD.
NORTHSIDE DR.
MIDTOWN
DECATUR
KENSINGTON DR.
INDIAN CREEK
PERRY HOMES
INMAN PARK REYNOLDSTOWN
AVONDALE
BANKHEAD HY.
WEST LAKE AV.
EAST LAKE DR.
FAIRBURN RD.
EDGEWOOD-CANDLER PARK
HIGHTOWER RD.
OAKLAND CITY
LAKEWOOD
EAST POINT
HAPEVILLE
COLLEGE PARK
AIRPORT

PROGRESS AT A GLANCE
(JUNE 1979)

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

I-4   FIGURE I-1

FAIRLIE ST.

WILLIAMS ST.

SPRING ST.

CARNEGIE WAY

PEACHTREE TOWER
APARTMENTS

DAVISONS

WESTERN
INTL. HOTEL

PEACHTREE
CENTER

MERCHANDISE
MART

COASTAL
STATES
BLDG.

GEORGIA
POWER

PEACHTREE STREET

LOEWS
GRAND
THEATER

PILOT TUNNEL

THE ATLANTA
RESEARCH CHAMBER

PEACHTREE
CENTER

HYATT REGENCY

CITY

PEACH

HOUSTON ST.

BANKS PL.

ELLIS ST.

International Blvd

HARRIS ST.

BAKER ST.

IVY ST.

IVY ST.

FIGURE I-2

PEACHTREE CENTER STATION

N

Peachtree Center
Station

Peachtree Center Cavern

12' x 12' Pilot Tunnel

Ellis Street Entrance

Access Shaft

Corridor

x
y
z

Ground Surface

Study Section Using 2-D FEM

NR Track Tunnel

Carnegie Way Entrance

Entrance

Connecting Tunnel

8' x 8' Pilot Tunnel

Atlanta Reseach Chamber

NL Track Tunnel

FIGURE I-3   OBLIQUE VIEW OF THE PEACHTREE CENTER STATION, SOUTHERN HALF

ELEVATION

1080
1070
1060
1050
1040
1030
1020
1010
1000
990
980
970
960
950
940

EXISTING PAVEMENT

ORIGINAL
GROUND SURFACE

TOP OF ROCK
WATER TABLE

Pilot Tunnel

CONCOURSE LEVEL

RUNNING TUNNEL

MARTA T/R

PLATFORM

RUNNING TUNNEL

H/V=2/1

PEACHTREE CENTER STATION

FIGURE I-4

CHAPTER II.

SUMMARY AND OVERVIEW

A. SUMMARY

1. Geology

The geology of the Atlanta Research Chamber was excep-
tionally well understood, because the Pilot Tunnel excavated ear-
lier 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 Founda-
tion 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 appear-
ance. It was technically a gneiss and the foliation was primar-
ily 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 de-
tails.

2. Design

The design of the Atlanta Research Chamber was deliber-
ately 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. Be-
cause the existing Pilot Tunnel was not parallel to the twin Run-
ning Tunnels, the Research Chamber does not parallel the Pilot
Tunnel.

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. Instrumentation

A complex instrumentation program was developed and installed. The instrumentation data indicated that although rock movements were small, they were different than predicted by the 2-D FEM design study. Dr. Weir-Jones discusses the instrumentation program in detail in Chapter V.

4. Excavation

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. 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 the anisotropy existing in the rock mass, in that near-horizontal planes in the gneiss existed, 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 discussed 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 STRAAM Engineers 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 shot-

crete 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 may be 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-Delegado of the University of Illinois, who were the team members in charge of the work. Also see the monograph by Jan Blanck.

B. OVERVIEW

The Atlanta Research Chamber pioneered several "firsts" in North American rapid transit tunnel practice, as well as providing 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.

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.

1.

21+00

20+32   20+47.5   20+63.5

1035'

1025'

1015'

1005'

ATLANTA RESEARCH CHAMBER

PILOT TUNNEL

995'

985'

CROWN OF NORTH LINE

975'

RUNNING
TUNNEL

965'

INVERT OF NORTH LINE

955'

945'

BOREHOLE EXTENSOMETER LOCATIONS.

2.

ATLANTA RESEARCH CHAMBER

PILOT TUNNEL

RUNNING
TUNNEL

INCLINOMETER AND SEISMIC PROFILER HOLES.
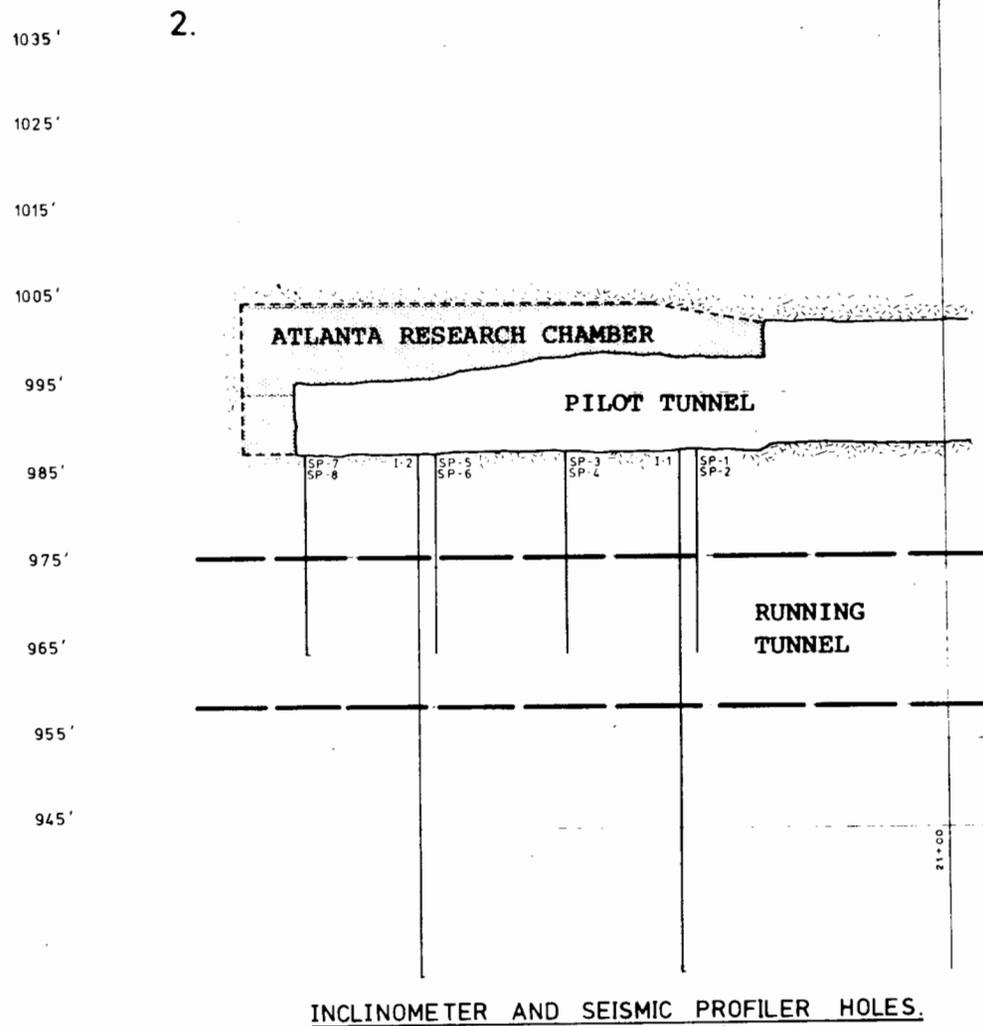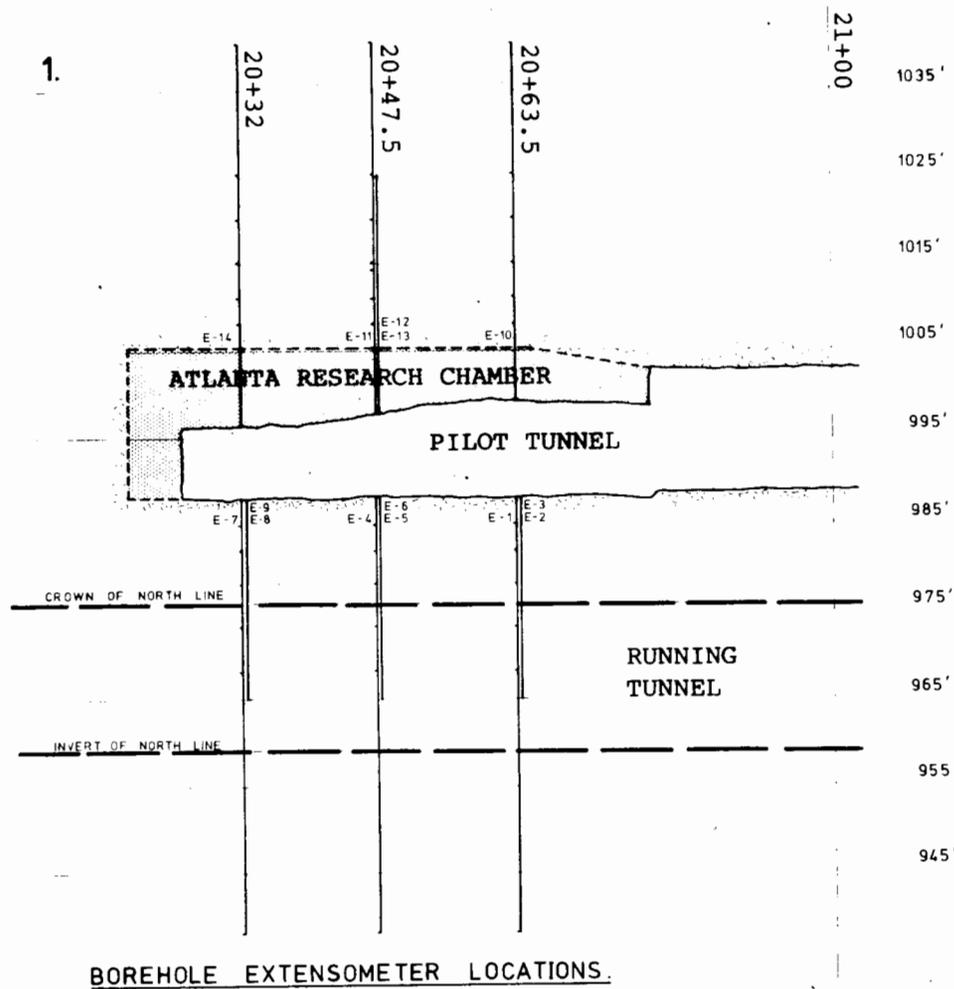
INSTRUMENTATION PROFILE

FIGURE II-1
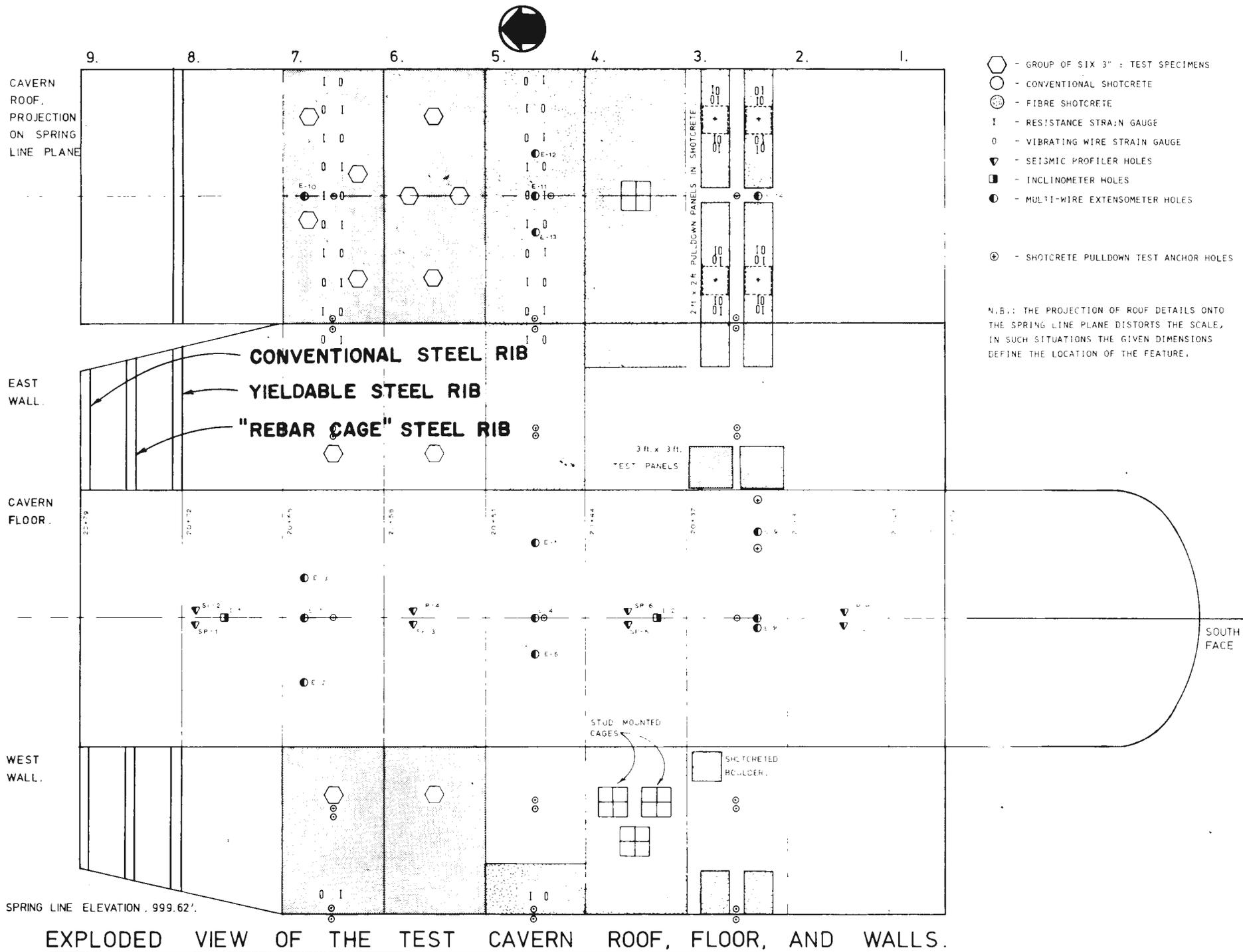
SECTION AT 20+32

SECTION AT 20+47.5

SECTION AT 20+63.5

INSTRUMENTATION - SECTION

FIGURE II-2

EXPLODED VIEW OF THE TEST CAVERN ROOF, FLOOR, AND WALLS.

FIGURE II-3

STATION 20+72
LOOKING SOUTH
SCALE: ¾"=1'-0
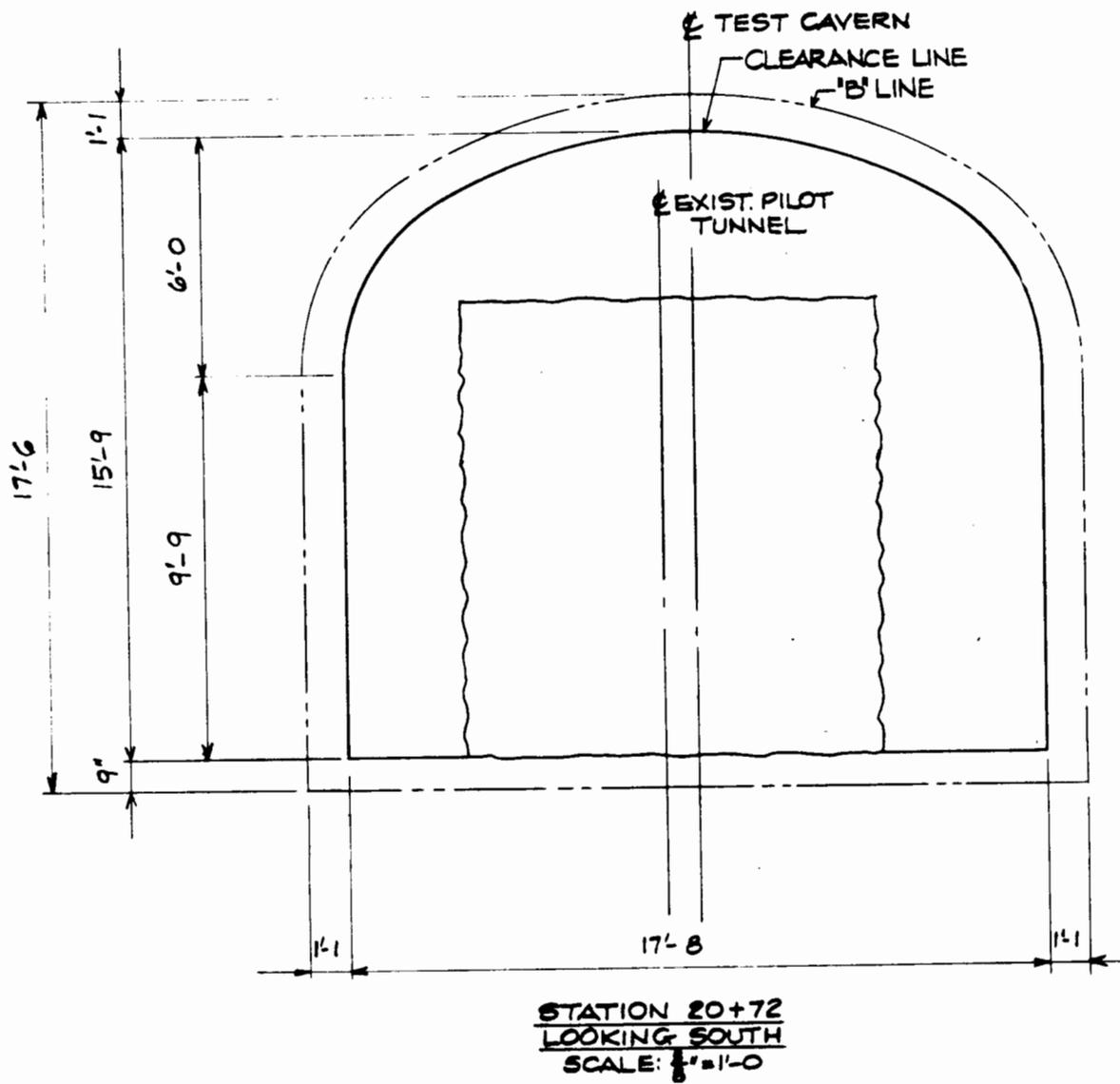
The Atlanta Research Chamber

FIGURE II-4.
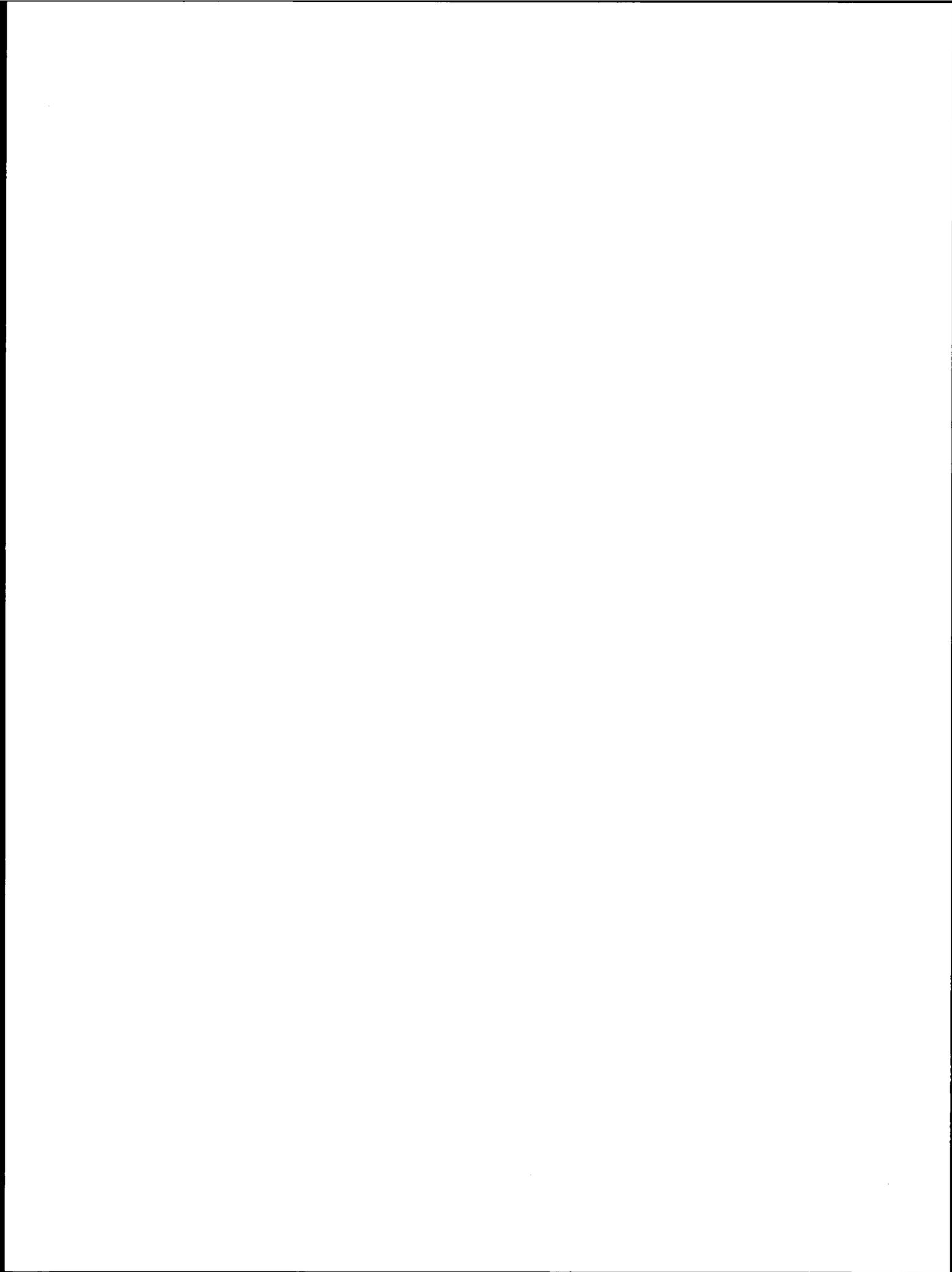
CHAPTER III.

GEOLOGY

A. ATLANTA REGIONAL GEOLOGY

Atlanta is located within the Piedmont geologic province, which extends from Alabama to New York. 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 milion 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. The most recent and most intense of these periods occurred ap-
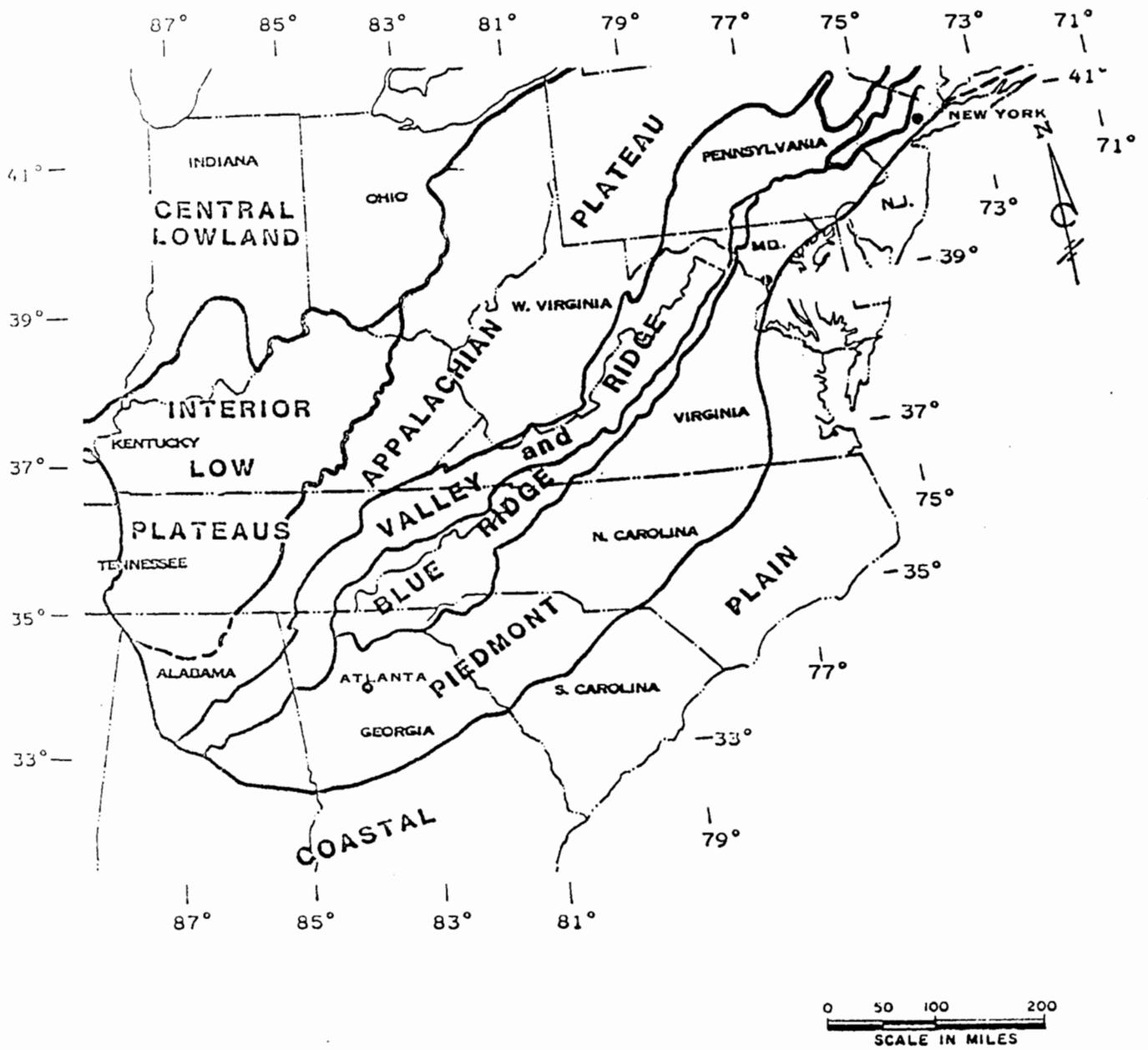
Figure III-1 GENERALIZED GEOLOGY OF THE EASTERN UNITED STATES

proximately 250 million years ago, late in the Paleozoic era.[1], [2], [3], [4]

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.[5], [6]

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

[1] Crickmay, Geoffrey W., 1952, "Geology of the Crystalline Rocks of Georgia": Georgia Geological Survey, Bulletin No. 58.

[2] Fullger, 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.

[3] 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.

[4] Massachusetts Institute of Technology, 1958, "Age Study of Some Crystalline Rocks of the Georgia Piedmont": U.S. AEC Dept. NYD-3938, pp. 58-60.

[5] Fullager, P. D., op. cit.

[6] 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.

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.

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, 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 depth to hard rock is much less than typical elsewhere in the Atlanta area, and the rock recovered is normally fresh and sound. The top of the rock is 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.

B. ATLANTA RESEARCH CHAMBER GEOLOGY

The Atlanta Research Chamber geology has been summarized on a sidewall map and a geologic cross-section. The Plan and Sidewall Map (Figure 2) were developed from field observations. The Geologic Cross-Section (Figure 3) was developed from examination of cores from extensometer and overcoring holes which were drilled during Pilot Tunnel construction.[7] 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.

1. 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.[8] 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.

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

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

ag+bg

q

ag+bg   q

bg+q

q

20,20   20,40   20,60   20,80

q

bg+q

ag+bg   q

bg+ag

q

ag+bg

→ N
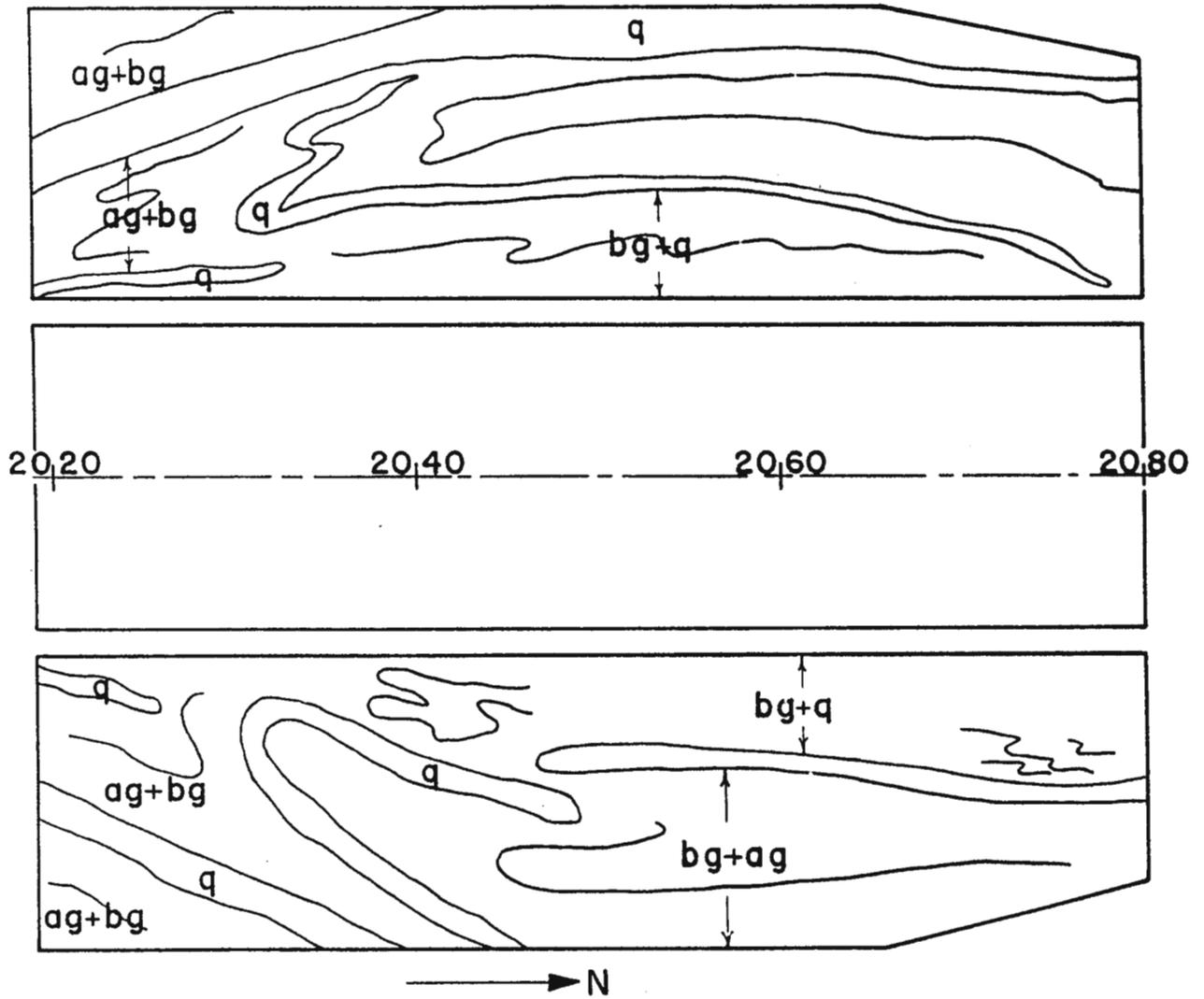
Figure III-2 — PLAN AND SIDEWALL MAPS OF THE RESEARCH CHAMBER

Figure III-3 GEOLOGIC CROSS-SECTION AT STATION NR 20 + 50

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 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° to the south. There is considerable local variation in foliation orientation, as the rock is recumbently folded on an axis of approximately N15°E horizontal (see Figure 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[9], [10], the physical properties of the rock in the area of the Atlanta Research Chamber have been presented. These proper-

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[9]   Law Engineering Testing Company, 1976, op. cit.

[10]   Law Engineering Testing Company, 1977, op. cit.

ties were determined from both laboratory and in-situ testing, and include determinations of in-situ stresses, elastic moduli, unconfined compressive strengths, rock hardness values, and direct shear strengths of joints in rock.

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

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

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

CHAPTER IV.

DESIGN

A. INTRODUCTION

        During the last 15 years, a substantial body of know-
ledge has developed relating to the behavior of underground open-
ings in rock. The pertinent studies have largely followed two
divergent paths. The first relates to the increasing sophistica-
tion in analytical methods (in particular, finite element me-
thods) 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
these data. Both methods have their advantages, as well as
shortcomings, in particular situations. In this chapter an at-
tempt 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 studies are presented to illustrate the use of this me-
thod to analyze the response of the rock mass at the Atlanta Re-
search Chamaber, south of the Peachtree Center Station. In this
area, a Pilot Tunnel was constructed; it was enlarged to become
the Atlanta Research Chamber and then the twin Running Tunnels
were constructed beneath the Research Chamber. The Research
Chamber was designed to have the same dimensions as the twin Run-
ning Tunnels. The finite element studies concentrate on the in-
teraction of the Research Chamber and twin Running Tunnel open-
ings. These design studies using 2-D FEM were conducted during
June 1977, before in-situ testing was complete and before the
openings were excavated.

B. ANALYSIS/DESIGN PHILOSOPHY

        Before discussing the two dimensional Finite Element Me-
thod (2-D FEM), it is important to review the intent and meaning
of these 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 leads to the first major aspect of design - a sound understanding of the geological environment at the site. 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 the in-situ stress state.

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

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

The fourth point is construction and time. 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. For example, with a moderate size and depth opening of simple geometry, sound, massive rock units, no

major discontinuities or water problems, in-situ stresses near isotropic and good excavation techniques, 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.

C. 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 iιr which finite element methods have been used to analyze opening behavior, after the opening was completed, and they all show reasonably good correlations with the measured quantities. Predictions are rather rare, but this is precisely what must be done in the design of a real 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:
1. represent geometric and boundary conditions,
2. incorporate variable geologic strata and initial stress states,
3. simulate excavation operations following nearly any arbitrary sequence, and
4. include nonlinear and stress-dependent stress-deformation characteristics for the rock materials and the rock discontinuities, if warranted.

The minimum criteria for representing items 1, 2 and 3 are discussed in detail by Kulhawy (1974)[1] and an approach for item 4 is given by Kulhawy (1975).[2]

The main keys to establishing a sound model are the representation of the significant geologic units and discontinuities, the initial stress states and the mechanical behavior of the rock materials and discontinuities. The detailed geology and the initial stresses often are not known to a high confidence level 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 usually are not known precisely, it is usually necessary to conduct analyses representing the bounding conditions so that the varying behavior within the expectable range can be evaluated. Published parametric studies may be useful in this context.

D. 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.

1. Geometric and Geologic Generalization

The problem under consideration was the behavior of the twin Running Tunnels in rock and the interaction of these tunnels

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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.
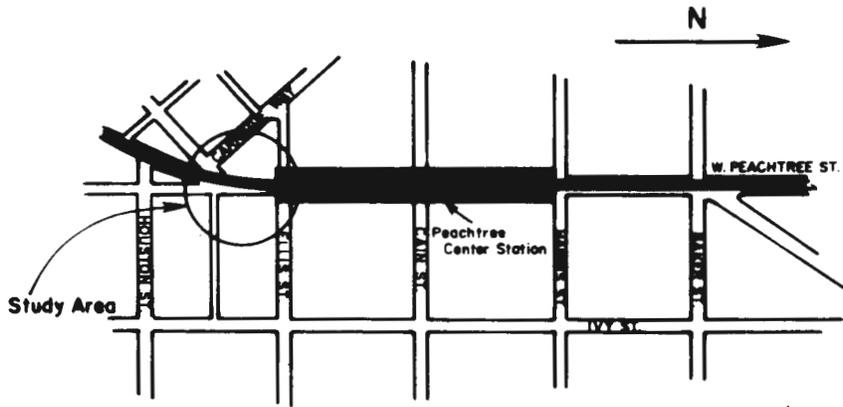
Fig. IV-1 Study Area, Peachtree Center Station, MARTA
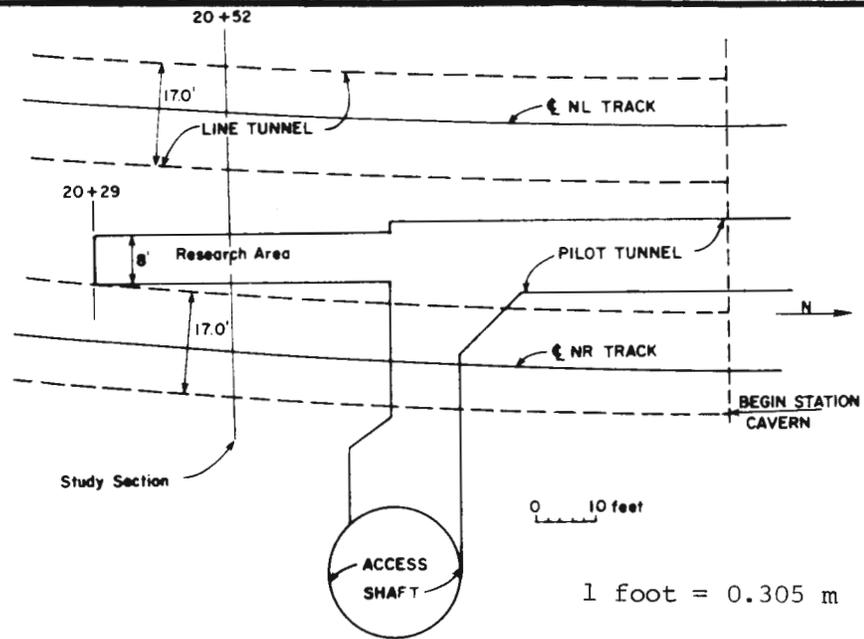
Fig. IV-2   Plan View of Tunnels and Research Area

1 foot = 0.305 m
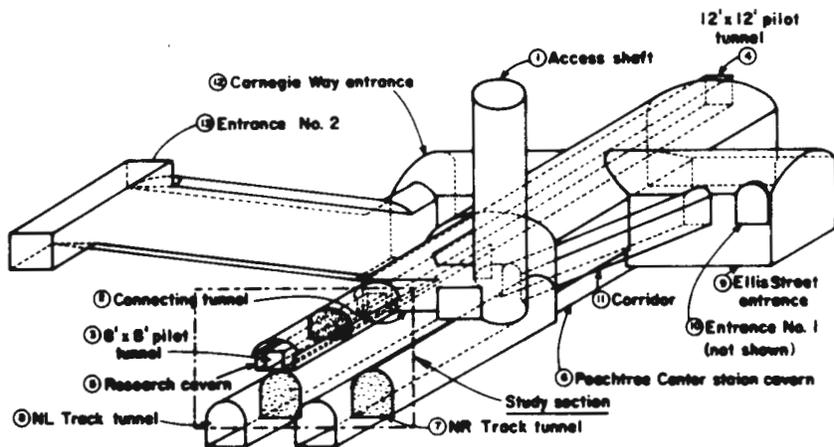
Fig. IV-3

South End Oblique

View, Peachtree

Center Station
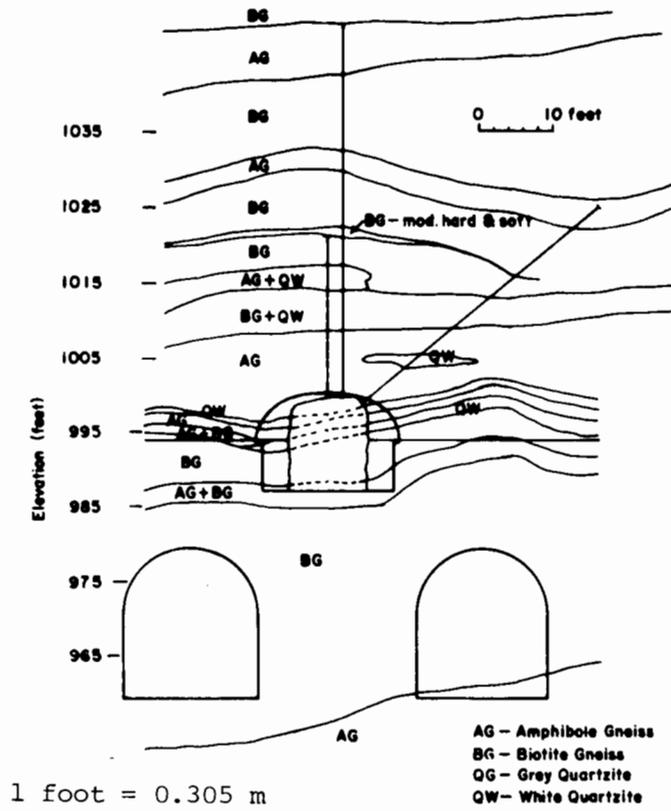
Fig. IV-4 Initial Section at Station 20 + 52
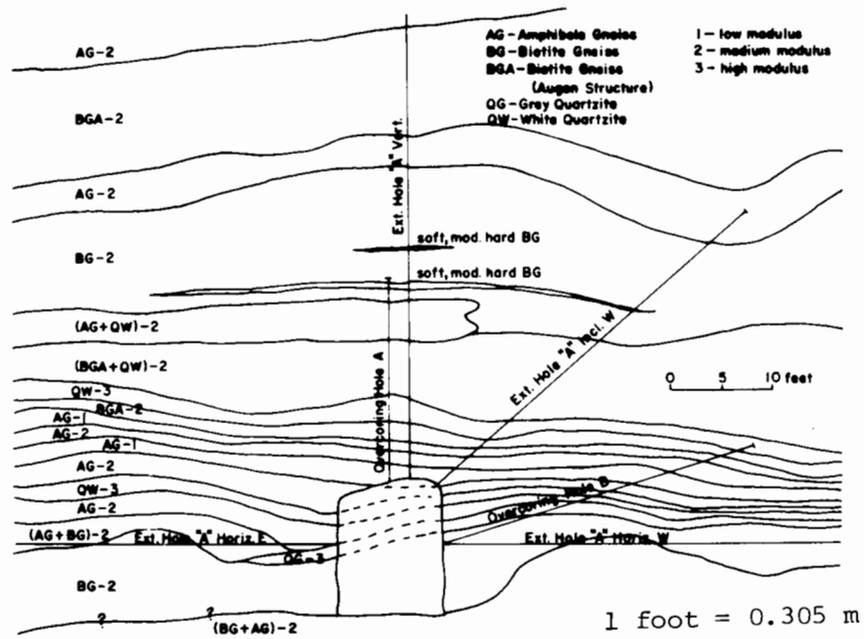
Fig. IV-5 Revised Geologic Section

with the Research Chamber to the south of the Peachtree Center
Station. Figure 1 shows a general location plan of the station
and the tunnels, while Figure 2 shows a more detailed plan view
of the openings at the south end of the Peachtree Center Sta-
tion. Figure 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 stand-
point of idealization, concentrating on the tunnels and Research
Chamber, it can be seen that reasonable two-dimensional represen-
tation can be taken about mid-length along the Research Cham-
ber. This "Study Section" is shown on Figures 2 and 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 struc-
ture 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 geolo-
gic data available at the time were plotted, in a east-west sec-
tion looking south, as shown in Figure 4. Subsequent data ob-
tained in the field resulted in the modification shown in Figure
5. These two figures, plus the correct opening locations shown
in Figure 2, were the primary bases for establishing the geome-
tric and geologic conditions.

2.   Finite Element Mesh Design

The design of the finite element mesh is the first ac-
tual step in the modeling procedure because the physical system
to be analyzed is established at this point. The geologic sec-
tions 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 con-
ducting many analyses in the past, and a reasonable grouping of

Ground Surface (Assumed level, Elevation 1078)

Soil (silty sand)

Weathered Rock

Medium Modulus Rock

Discontinuity

Medium Modulus Rock

Low Modulus Rock

Pilot Tunnel — Research Chamber

Left Tunnel

Right Tunnel

Section for which stresses are presented

Medium Modulus Rock

1 foot = 0.305 m

0   50 feet

Note: Rock materials typically are Amphibole Gneiss and Biotite Gneiss

Fig. IV-6

Generalized Section

for Analysis

Fig. IV-7

Finite Element Mesh

1 foot = 0.305 m

193
150
100
50
0

Feet

0   50   100   150   200

Feet

613 Nodal Points   571 Elements
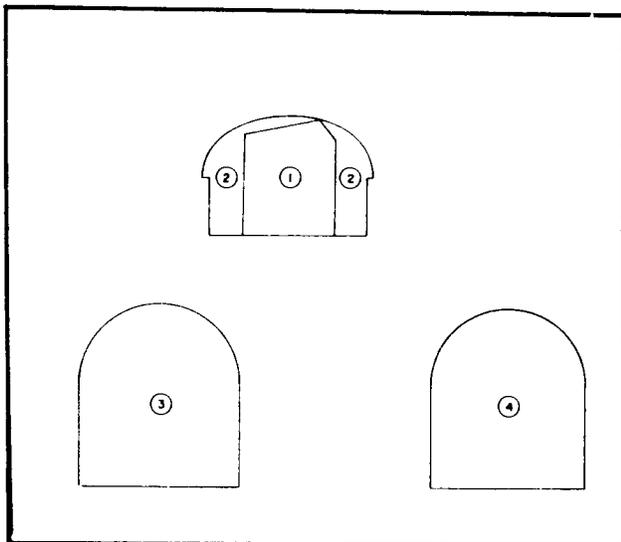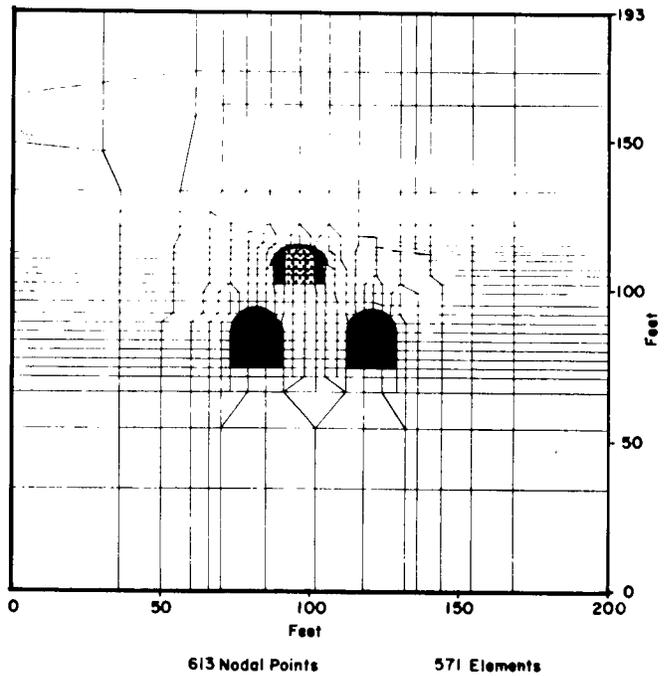
Fig. IV-8

Sequence of Excavation Operations

geologic units with apparently similar mechanical properties (1, 2 or 3 as shown in Figure 5), the generalized geologic section shown in Figure 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 three-dimensional effects 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 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 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).

3. 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 related to the elastic parameters. 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 large 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 were actually present.

4. Excavation Operations

The sequence of excavation operations for this project is given in Figure 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 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 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.

5. Material Properties

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

For the soil overburden, the weathered rock, and the discontinuity above the Research Chamber, representative values for the properties were assumed, based on extensive literature surveys conducted on soil materials (Kulhawy, Duncan and Seed, 1969)[3] 5d 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 2.

The rock properties given in these tables are essentially those for 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 im-

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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.

[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.

	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	1×10^6 psi (144,000 ksf)	0.15×10^6 psi (21,600 ksf)	0.17
Low Modulus Rock	180	3×10^6 psi (432,000 ksf)	0.45×10^6 psi (64,800 ksf)	0.10
Medium Modulus Rock	180	5×10^6 psi (720,000 ksf)	0.75×10^6 psi (108,000 ksf)	0.17
Discontinuity	Normal Stiffness = 31,830 ksf/ft			
	Shear Stiffness = 6,366 ksf/ft			

$$1 \text{ psi} = 6.89 \text{ kN/m}^2$$

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.

Table IV-1   Range of Rock Material Properties

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

Range noted in parentheses.

$$1 \text{ psi} = 6.89 \text{ kN/m}^2$$

Table IV-2   Properties Used in Analyses

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

E.   RESULTS OF ANALYSES

A number of analyses were conducted to evaluate the rock mass response to the 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.

1.   Displacements

The displacements of the openings for the case with high
horizontal stress are shown on Figure 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 ra-
tio 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 be-
tween 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 dif-
ferent excavation steps.

Figure 9 shows that during excavation for the Pilot Tun-
nel and Research Chamber, the wall displacements inward are
larger than those at the crown and invert, as would be ex-
pected.   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 se-
quential movement patterns develop when the initial stresses are
isotropic.   The final displaced openings for the isotropic stress
case are shown on Figure 10.

Extensometer 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 displace-
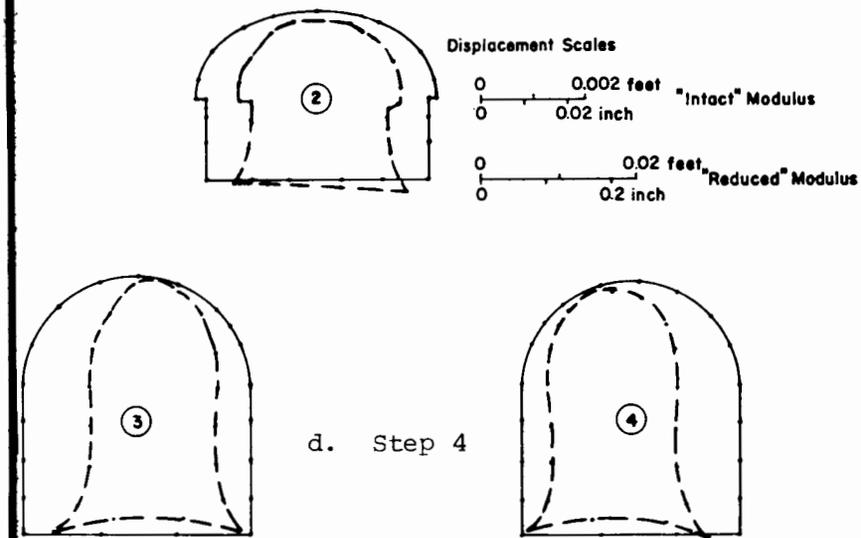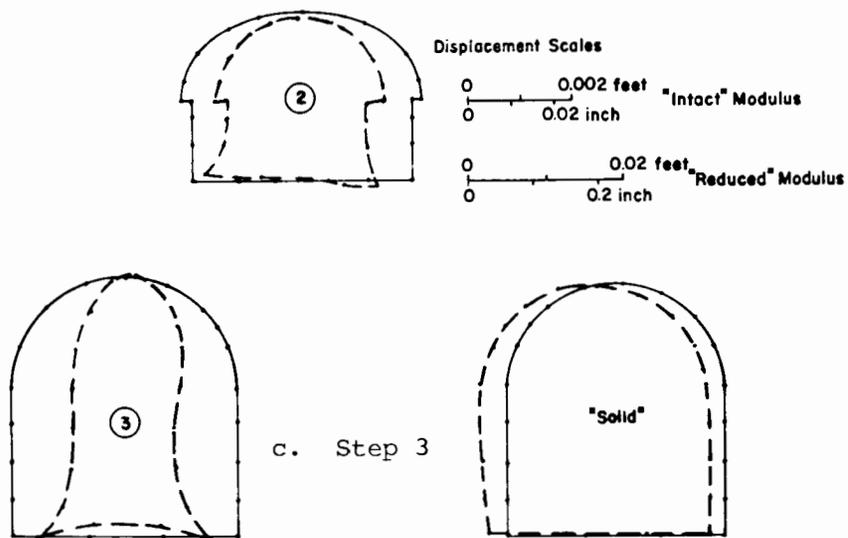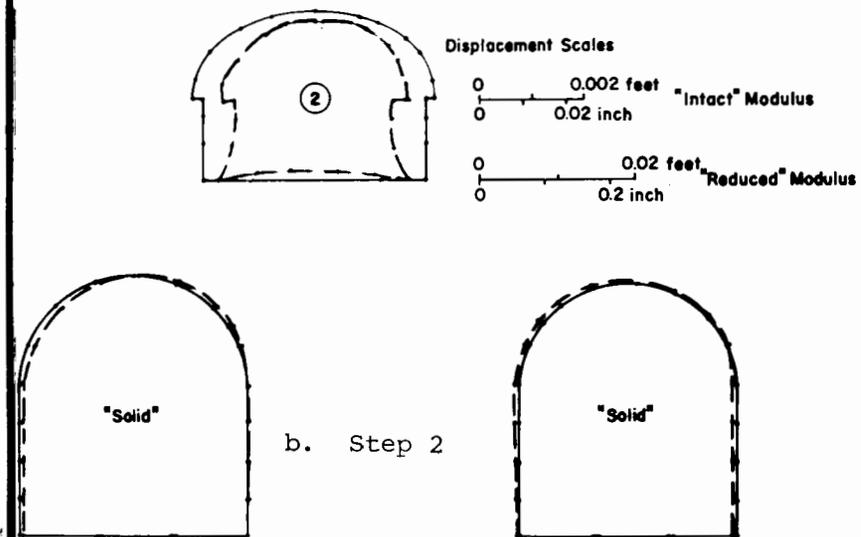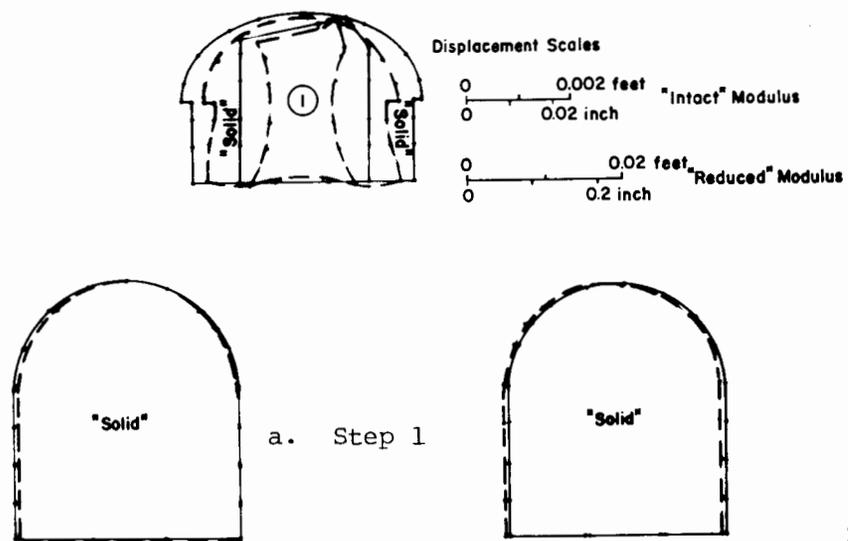
Fig. IV-9   Computed Opening Displacements, K = 2.5   1 foot = 0.305 m, 1 in = 25.4 mm

Displacement Scales

② — "Intact" Modulus
0.002 feet
0.02 inch

— "Reduced" Modulus
0.02 feet
0.2 inch

1 foot = 0.305 m, 1 in = 25.4 mm

③   ④
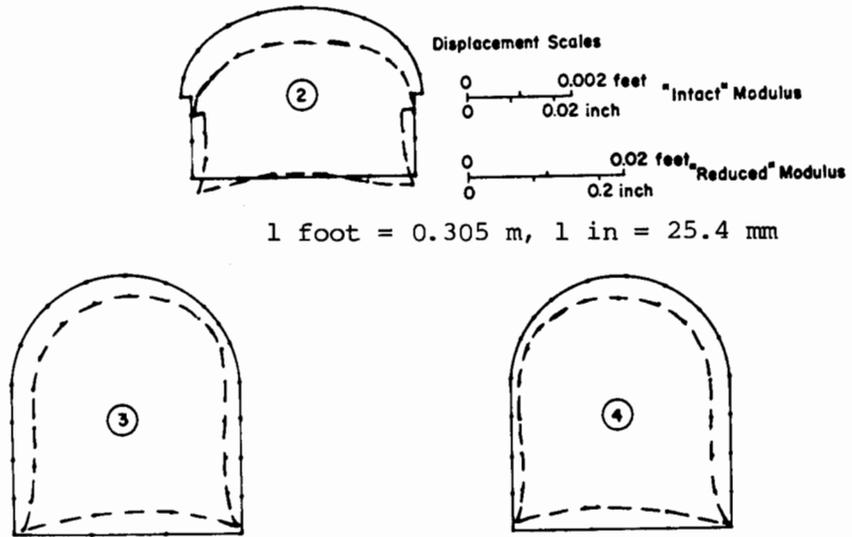
Fig. IV-10 Computed Opening Displacements, K = 1, Step 4

ments 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.

2. Stresses

The maximum (o_1) and minimum (o_3) principal stresses computed for the different analyses are shown on Figures 11 through 14 in contour form. These solutions are correct for both modulus assumptions. These figures also show the openings in correct relative size and location, indicate the excavation step for a particular opening, and show future openings with dashed lines.

The o_1 and o_3 stresses were selected for presentation because the maximum stress concentration can be compared to the rock strength to determine whether there is any potential for compression failure. The minimum 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 potential "fall-out" zone which, in turn, would indicate the height of rock to be supported by the opening support system.

Figure 11 shows the o_1 contours during excavation for the analysis with high horizontal stresses, while Figure 12 shows the final o_3 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 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 around the Research Chamber are lower than those around the twin Running Tunnels.

Figures 13 and 14 show the o_3 contours for two analyses. These figures show significant stress reductions around the

a. Step 1

b. Step 2

c. Step 3
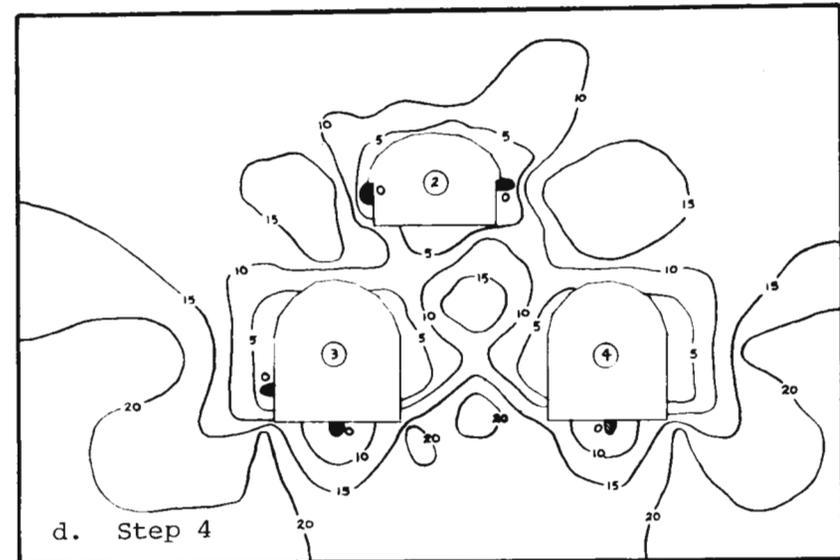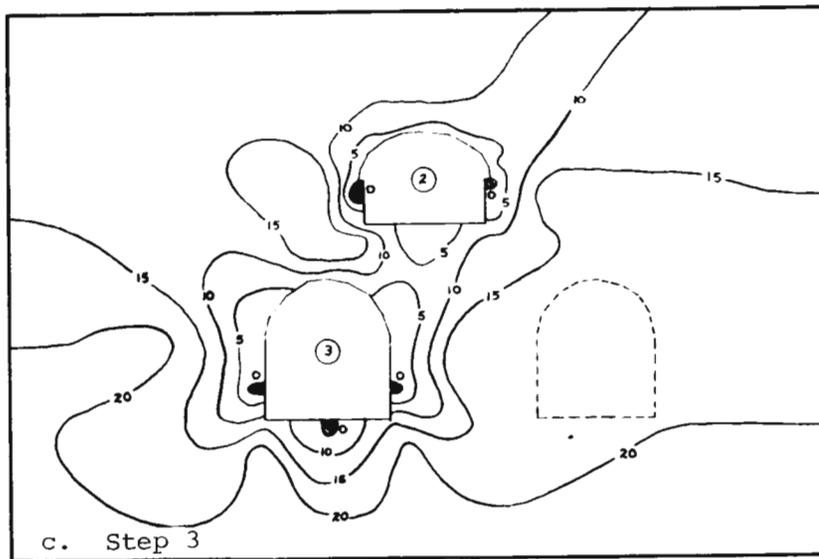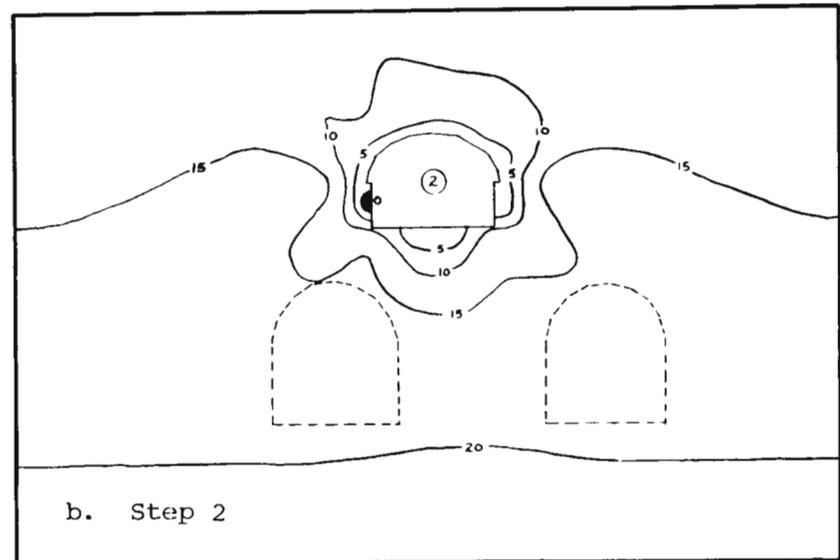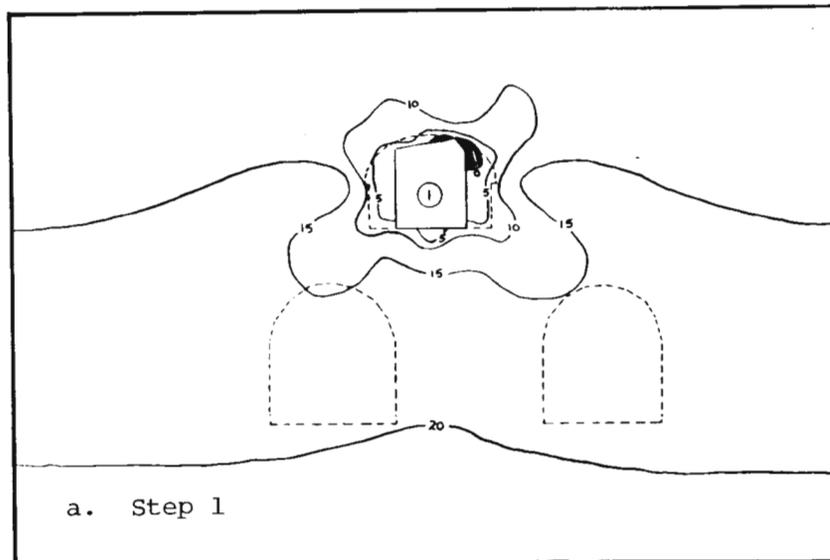
d. Step 4

Fig. IV-11 Computed σ_3 Contours (in ksf), K = 2.5   1 ksf = 47.8 kN/m^2
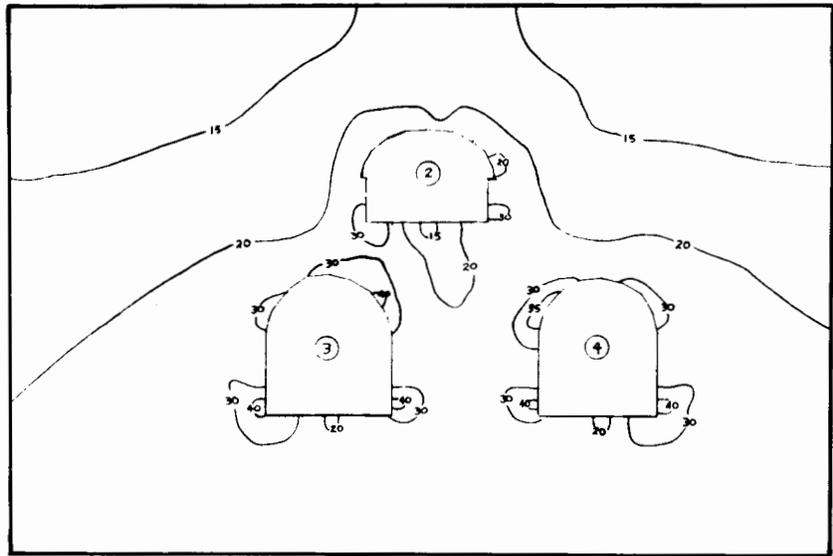
Fig. IV-12 Computed σ_1 Contours (in ksf), K = 1, Step 4
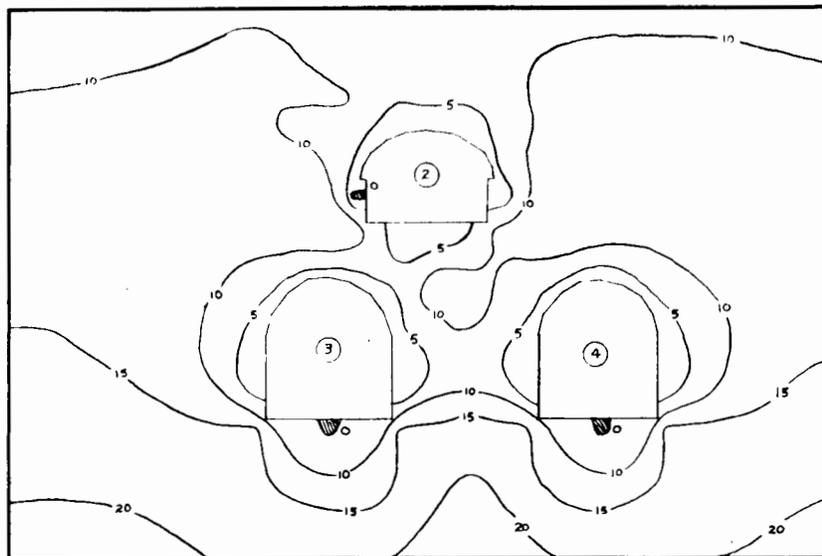
$$1 \text{ ksf} = 47.8 \text{ kN/m}^2$$

Fig. IV-14 Computed σ_3 Contours (in ksf), K = 1, Step 4

$$1 \text{ ksf} = 47.8 \text{ kN/m}^2$$

a. Step 1

b. Step 2

c. Step 3
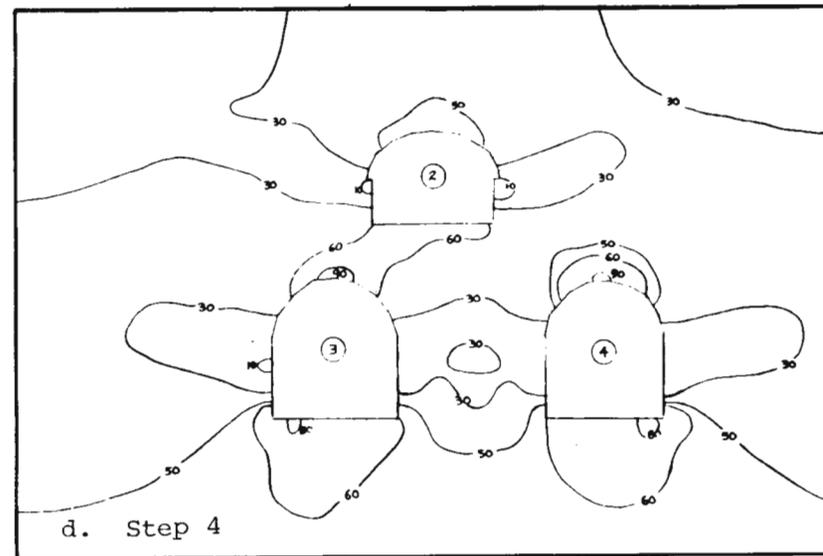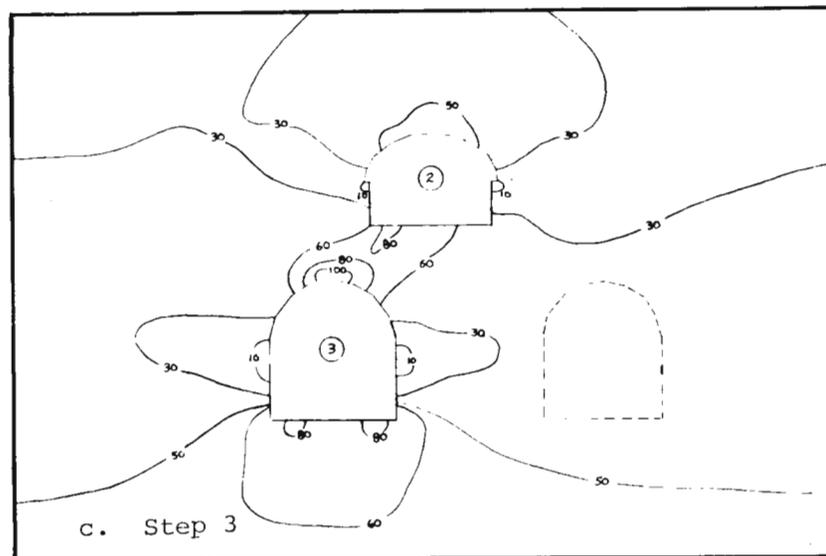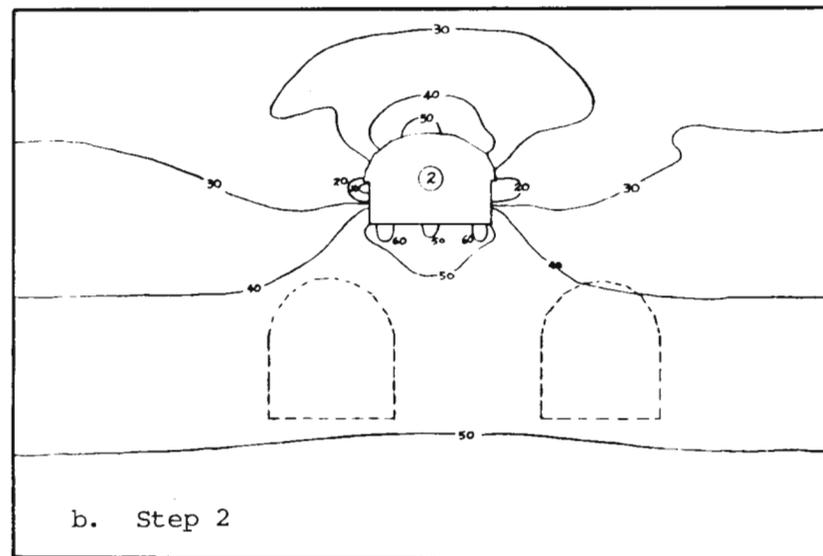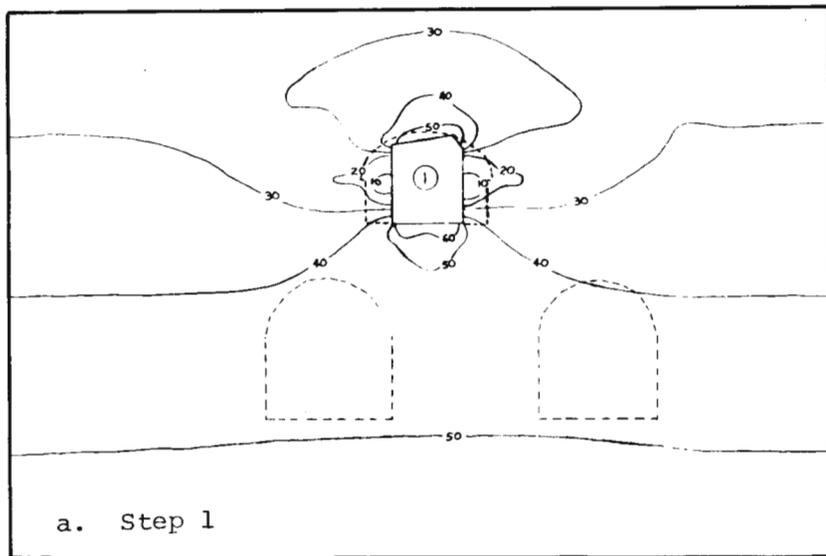
d. Step 4

Fig. IV-13 Computed σ_1 Contours (in ksf), K = 2.5

$1 \text{ ksf} = 47.8 \text{ kN/m}^2$

FIGURE IV-15
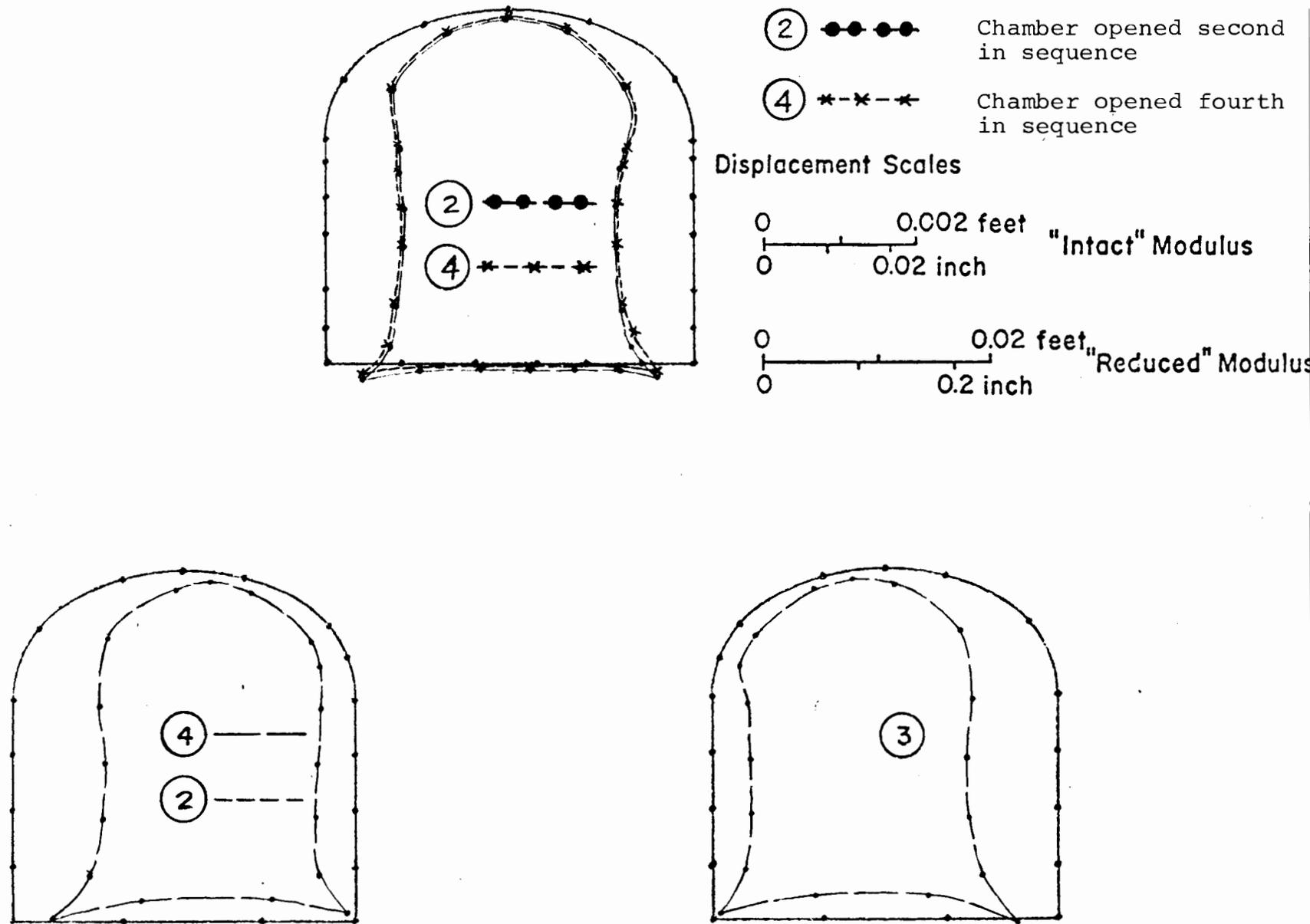
Opening Displacements, Atlanta Research Chamber, K = 2.5, Step 4

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

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 13a 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

Because the actual construction sequence did not resemble the sequence assumed in the designing study and because the small measured rock movement in the floor of the Atlanta Research Chamber was upward rather than downward, a post-construction 2-D FEM analysis was performed by Tudor Engineering Company in June 1979. As shown in Figure IV-14, that 2-D FEM study of a different excavation sequence but using the same mesh and input as before, indicated that rock movements would be only slightly changed.

The reasons for the discrepency between predicted and measured rock movements will be discussed in the Final Report.

G.   SUMMARY AND CONCLUSIONS

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

The analyses showed 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 it only suggested the potential for a minor fall-out and the possible need for support of only a meter or so of rock. No fallouts occurred in the real Research Chamber.

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 developed.

Further discussion will be made in the Final Report.

The Three Dimensional Finite Element Method study by Herbert H. Einstein and his co-workers at MIT, which is a Monograph to be found in this report, did predict a small uplift in the Research Chamber floor. See Monograph for explanation of these studies which is a 3-dimensional effect not seen in a 2-dimensional study.
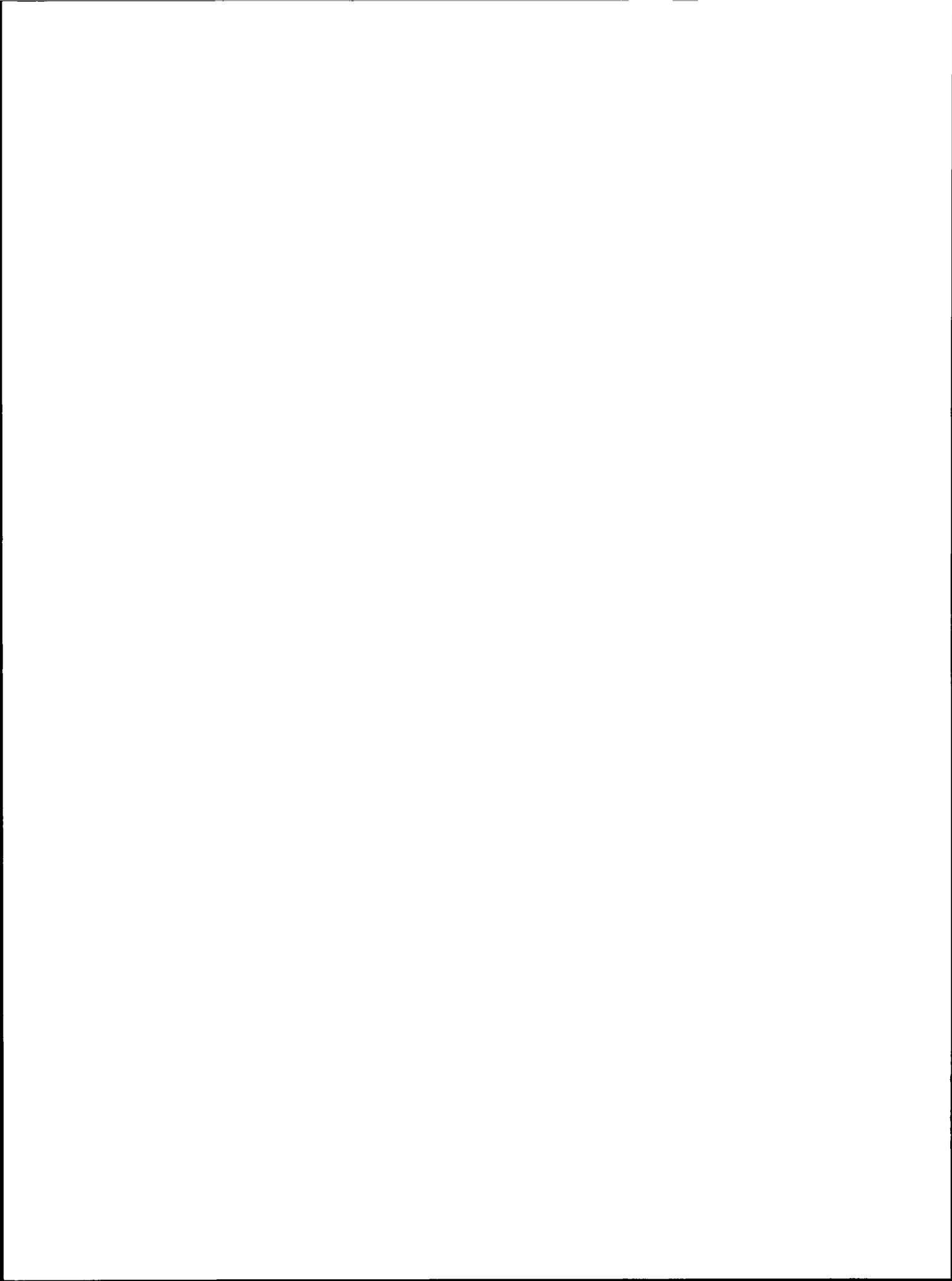
CHAPTER V.

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 within an actual construction environment, without the normal 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 procedures. 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, and this assumption was supported by the initial two-dimensional finite element studies. The only significant deviation from the anticipated elastic reponse would tend to be irrecoverable deformation and fracturing in the immediate vicinity of blasting.

The study of the behavior of the rock mass around the Research Chamber also would provide a valuable opportunity of comparing the actual deformations with those predicted by three-dimensional finite element studies being performed by Einstein et al at MIT. That 3D-FEM study is 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 being studied were:

a. Radial deformations around the Research Chamber at distances of up to approximately three times 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 be 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
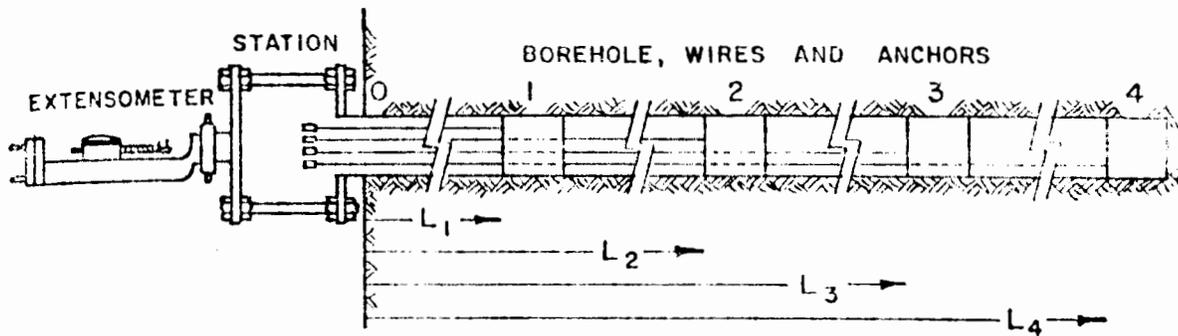
	Movement	Strain	
Wire L_1	ΔL_1	$\epsilon_{0,1}$	$= \dfrac{\Delta L_1}{L_1}$
Wire L_2	ΔL_2	$\epsilon_{1,2}$	$= \dfrac{\Delta L_2 - \Delta L_1}{L_2 - L_1}$
Wire L_3	ΔL_3	$\epsilon_{2,3}$	$= \dfrac{\Delta L_3 - \Delta L_2}{L_3 - L_2}$
Wire L_4	ΔL_4	$\epsilon_{3,4}$	$= \dfrac{\Delta L_4 - \Delta L_3}{L_4 - L_3}$

FIGURE V-1 Diagram Illustrating an Installed Borehole 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 are 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 the wedges 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.

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 at-

tached 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 $\pm$ 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 with 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 N-S and E-W directions, and by com-

paring 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 shown on the drawings 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.

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 + 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 down toward the

FIGURE V-2

EXPLODED VIEW OF THE TEST CAVERN ROOF, FLOOR, AND WALLS.

Legend:
- ⬡ — GROUP OF SIX 3" ⌀ TEST SPECIMENS
- ◯ — CONVENTIONAL SHOTCRETE
- ◯ — FIBRE SHOTCRETE
- I — RESISTANCE STRAIN GAUGE
- O — VIBRATING WIRE STRAIN GAUGE
- ▽ — SEISMIC PROFILER HOLES
- ⬒ — INCLINOMETER HOLES
- ◑ — MULTI-WIRE EXTENSOMETER HOLES
- ○ — ROCK BOLT PULLOUT TEST HOLES
- ● — ROCK BOLT OVERCORING HOLES
- ⊕ — SHOTCRETE PULLDOWN TEST ANCHOR HOLES

N.B.: THE PROJECTION OF ROOF DETAILS ONTO THE SPRING LINE PLANE DISTORTS THE SCALE, IN SUCH SITUATIONS THE GIVEN DIMENSIONS DEFINE THE LOCATION OF THE FEATURE.

CAVERN ROOF, PROJECTION ON SPRING LINE PLANE

EAST WALL.

CAVERN FLOOR.

WEST WALL.

SPRING LINE ELEVATION, 999.62'.

STAGING AT EL. 997.37'.

SOUTH FACE

ROCK BOLT TEST HOLES @ 0'

3 ft × 3 ft. TEST PANELS

STUD MOUNTED CAGES

SHOTCRETED BOULDER.

WEIR-JONES ENGINEERING CONSULTANTS LTD

MARTA TEST CAVERN, INSTRUMENTATION LAYOUT.

Based upon MARTA drawing no. RC-SK-11.

DRAWN BY	DATE 17 Mar. 19??	DRAWING NUMBER
CHECKED BY	SCALE 1 in. E + '	NJEC-M-3.
APPROVED BY	SHEET 3 of 4	

FIGURE V-3

SECTION AT 20+32

SECTION AT 20+47.5

SECTION AT 20+63.5
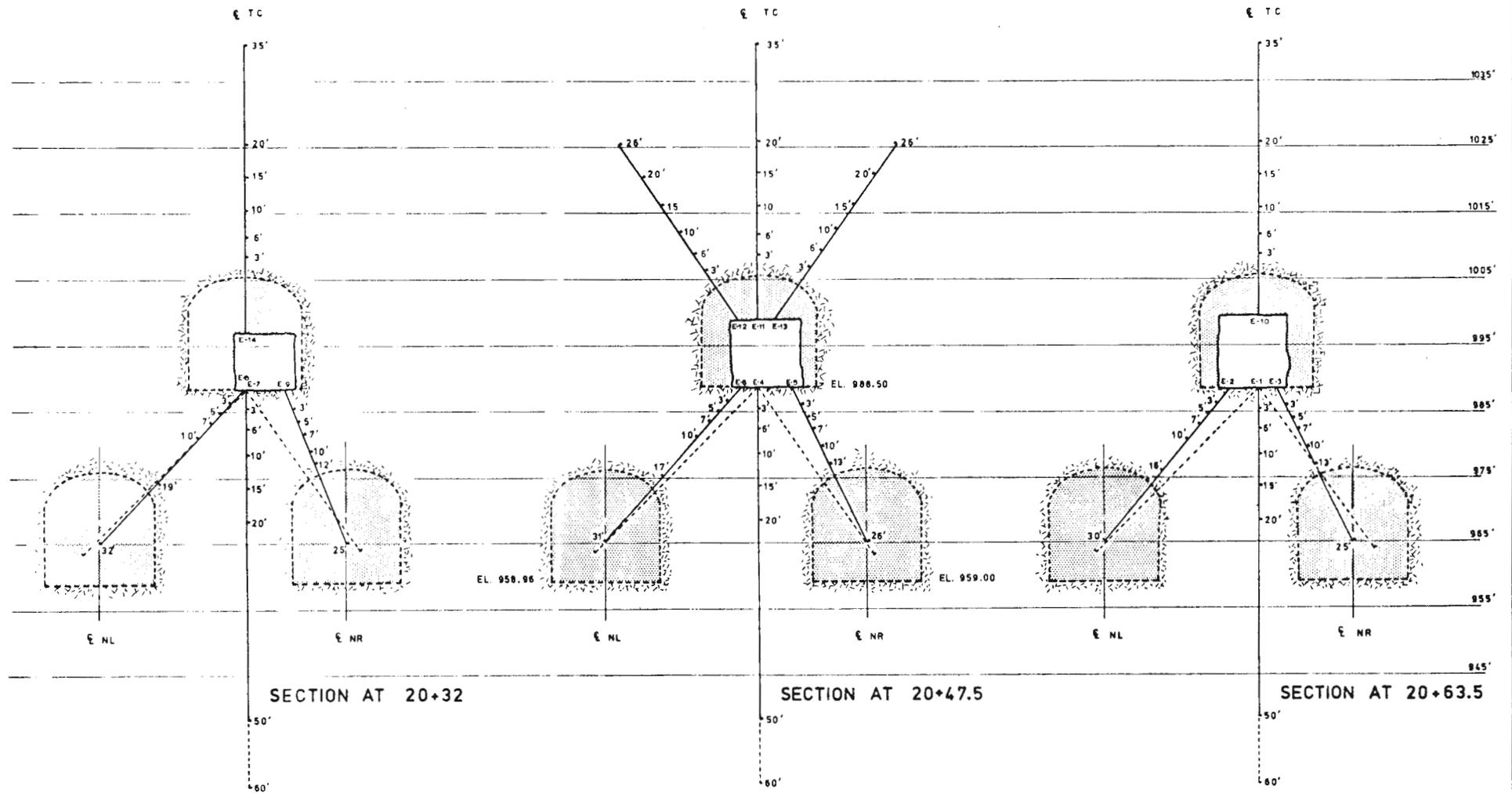
NOTES:

(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 10' LONG, DIPPING @ 55° E
SP-2, SP-4, SP-6, AND SP-8, ARE 9' 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.

FIGURE V-4

1.

2.

1035'
1025'
1015'
1005'
995'
985'
975'
965'
955'
945'

E-14 E-12 E-10
E-11 E-13

CROWN OF NORTH LINE

INVERT OF NORTH LINE

E-9 E-8 E-3
E-7 E-8 E-4 E-5 E-1 E-2

SP-7 I-2 SP-5 SP-3 I-1 SP-1
SP-8 SP-6 SP-4 SP-2
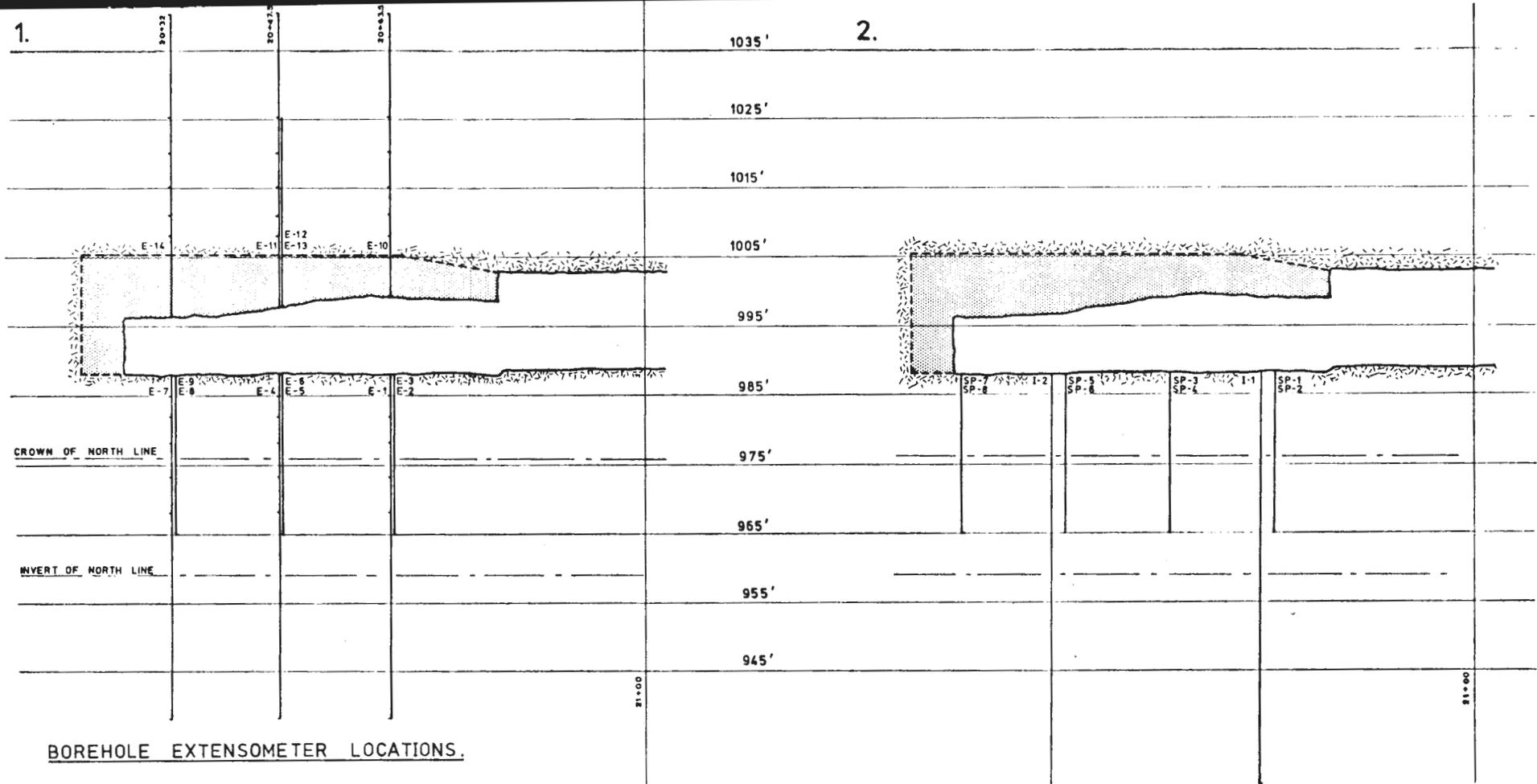
BOREHOLE EXTENSOMETER LOCATIONS.

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" ⌀ 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.		
DRAWN BY	DATE 22 Mar.1978	DRAWING NUMBER
CHECKED BY	SCALE 1 in ≡ 10 ft.	WJEC-M-1.
APPROVED BY	SHEET 1 OF 4	

FIGURE V-5

1.

≈ 26"

21" c/c

≈ 26"

21" c/c

2" φ holes

1/4"
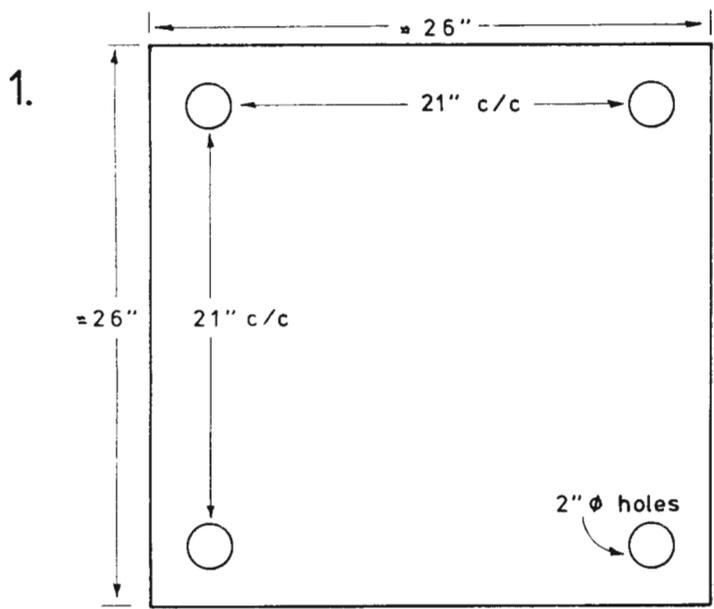
NOTES:

(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.

A

2" φ holes

2.

≈ 6"

14 1/2" c/c

2 1/2" φ

≈ 21"

3 1/4" φ

≈ 12"

hole length varies.

≈ 12"

≈ 24"

≈ 12"

4.

1 1/4" φ

4"

4"

A

≈ 21"

3.

SECTION ON A-A

Final surface of test cavern.

Running Tunnels as these were driven below the completed Research Chamber. Extensive modifications to the proposed excavation schedule were in fact made and the actual sequence did not correspond to that anticipated, which did not permit optimum instrument utilization, as 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 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-4 show the positions of eight seismic profiler 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.

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 profiling 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 both the NL and NR tunnels had been driven past the station before installation of instrumentation commenced. This was a particularly unfortunate situation as 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 the inclinometers I1 and I2 which were not read until after 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 data which was ob-

tained suggests that there may have been a slight rock mass re-
laxation in a northerly direction towards the main Peachtree Cen-
ter Station cavern. The absence of inclinometer data is most un-
fortunate as the system had the capability of monitoring the hor-
izontal 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 sel-
dom been installed in locations which permit this type of analy-
sis 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. Futher damage may have occurred between October 28
and November 9, as the magnitude of the readings decrease 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 summarized the
results obtained from E1 for the period from mid September until
November 9. It is immediately apparent that, apart from the
anomalous downward movement of the anchor 3 feet below the col-
lar, there is a general indication that rebound is ocurring.
This may be due to the stress relief taking place in the longi-
tudinal pillar between the NL and NR Running Tunnels.

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

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'
10' .011'
NO DATA
AVAILABLE
10' .044'   13' .004'
*13'-8'
15' .039'   *NOTE: BOTH RUNNING
TUNNELS HAD ADVANCED
PAST THIS CROSS-SECTION
AT THE TIME OF INSTRUMENT
PLACEMENT.

*22'-0'

20' .063'

NL   NR

50' .000'

FIGURE V-6

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

EXTENSOMETER NUMBER E1

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

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
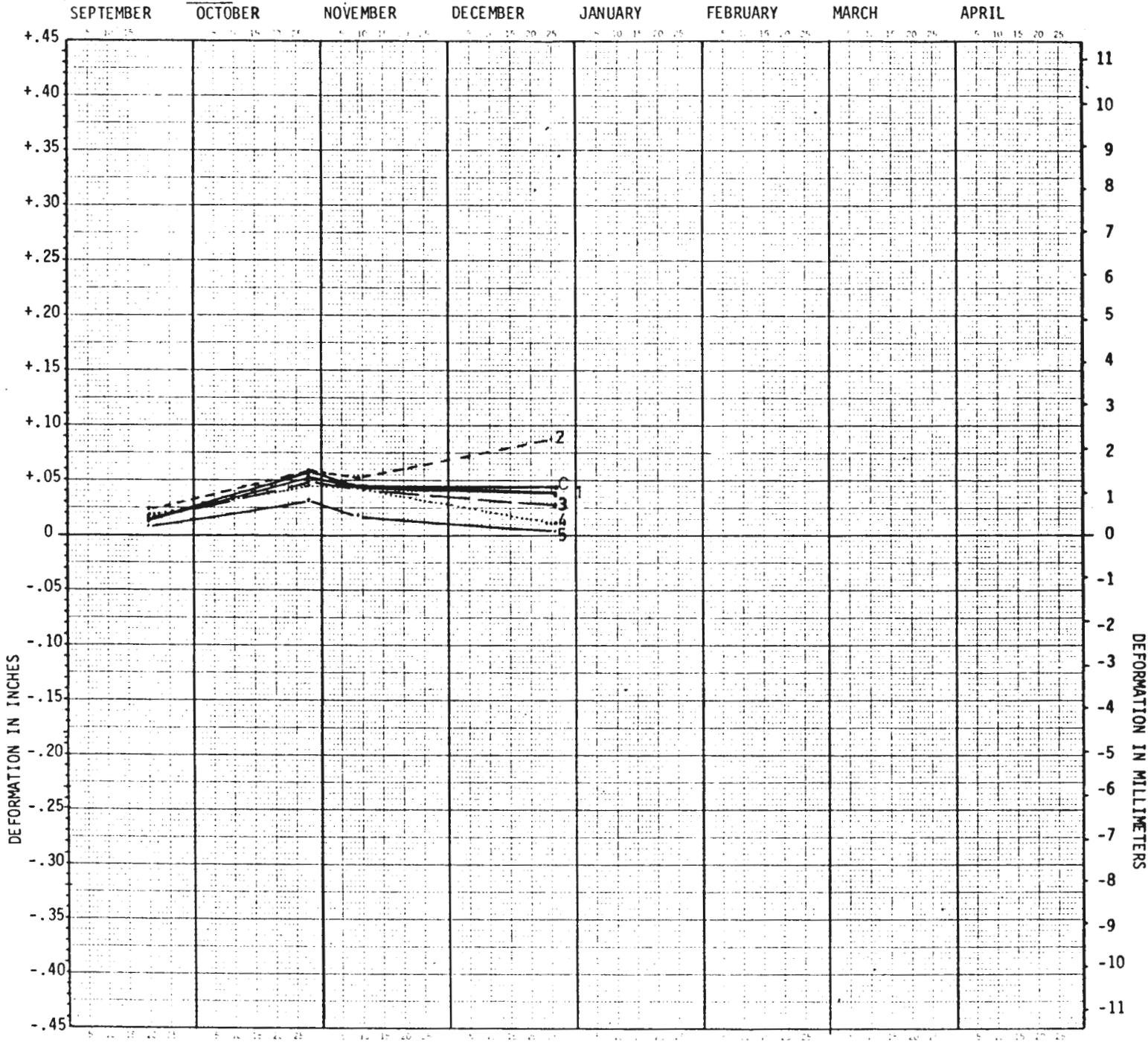
FIGURE V-8

Extensometer E3 yielded data for over 3 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 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 and V-11 show the displacements for the anchors at station 20 + 45.

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 cumultive uplift at the collar of about .07 inches. This movement is in the same order of magnitude as that predicted by Kulhawy using 2-D FEM (see Chapter IV) with the reduced modulus assumptions, but it is in the opposite direction. It is also somewhat more than the predicted elastic rebound caused by the enlargement of the Research Chamber test cavern which had been calculated using rather crude 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 initial predictions made by Kulhawy in Chapter IV based upon his FEM analysis may be due to the fact that the actual excavation sequence bore no resemblance to the one assumed in the original FEM analysis.

The vectorial presentation of the E4 displacement data shown on Figure V-9 covers the period mid-September to Decem-

FIGURE V-9

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

EXTENSOMETER NUMBER E4

WEIR-JONES ENGINEERING CONSULTANTS LTD.
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7
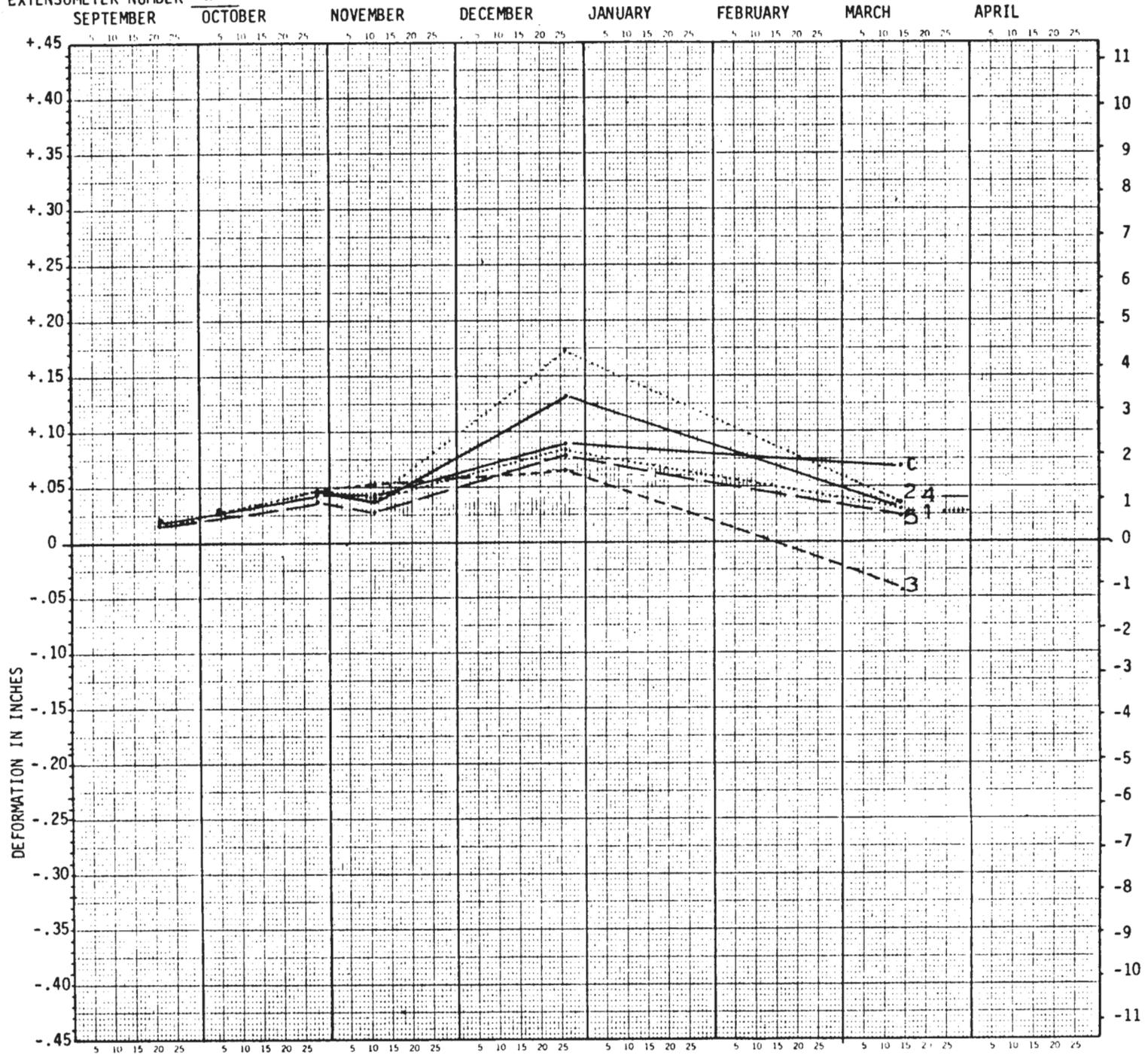
FIGURE V-10

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

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

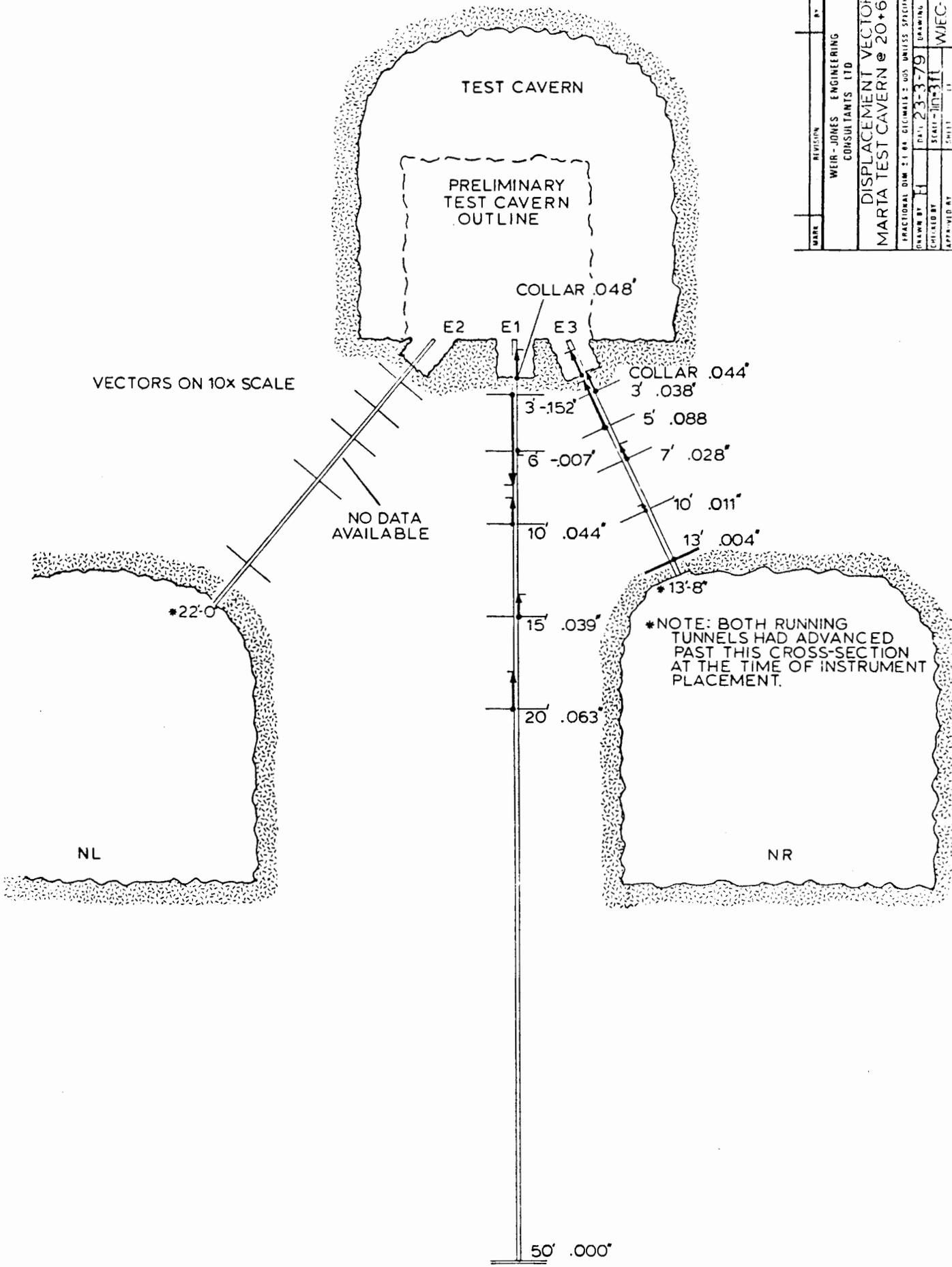EXTENSOMETER NUMBER E5
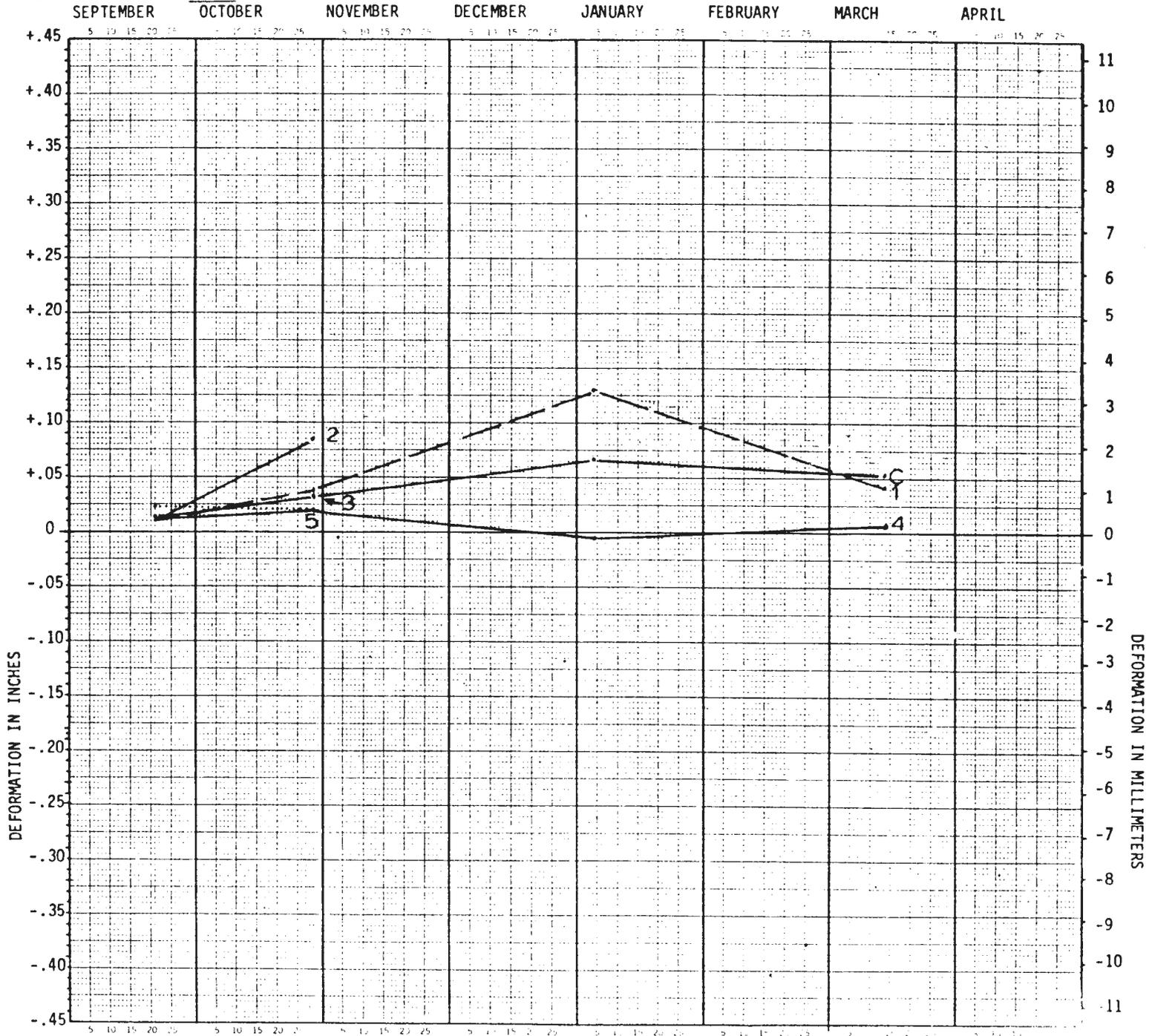
FIGURE V-11

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

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

EXTENSOMETER NUMBER E6
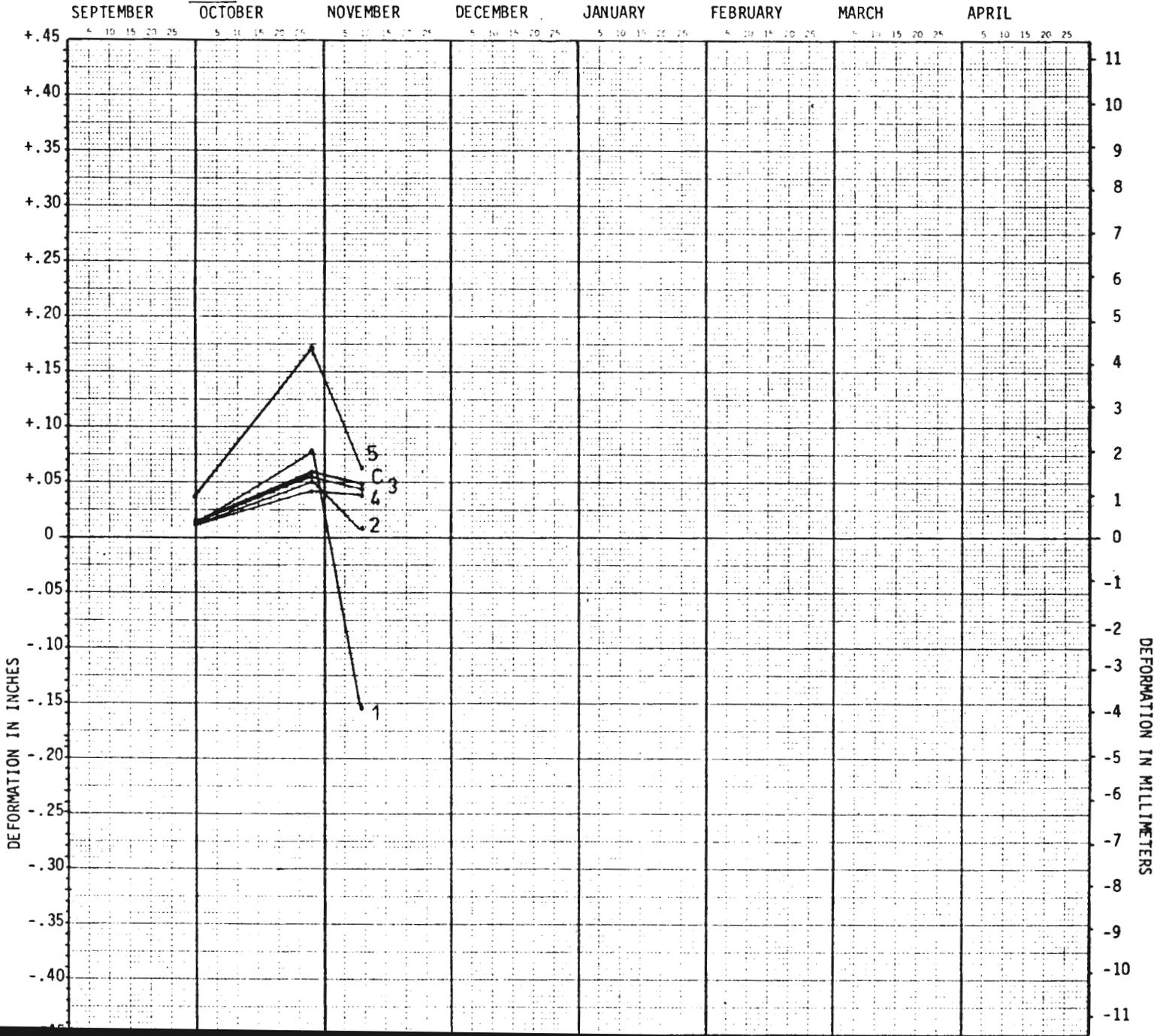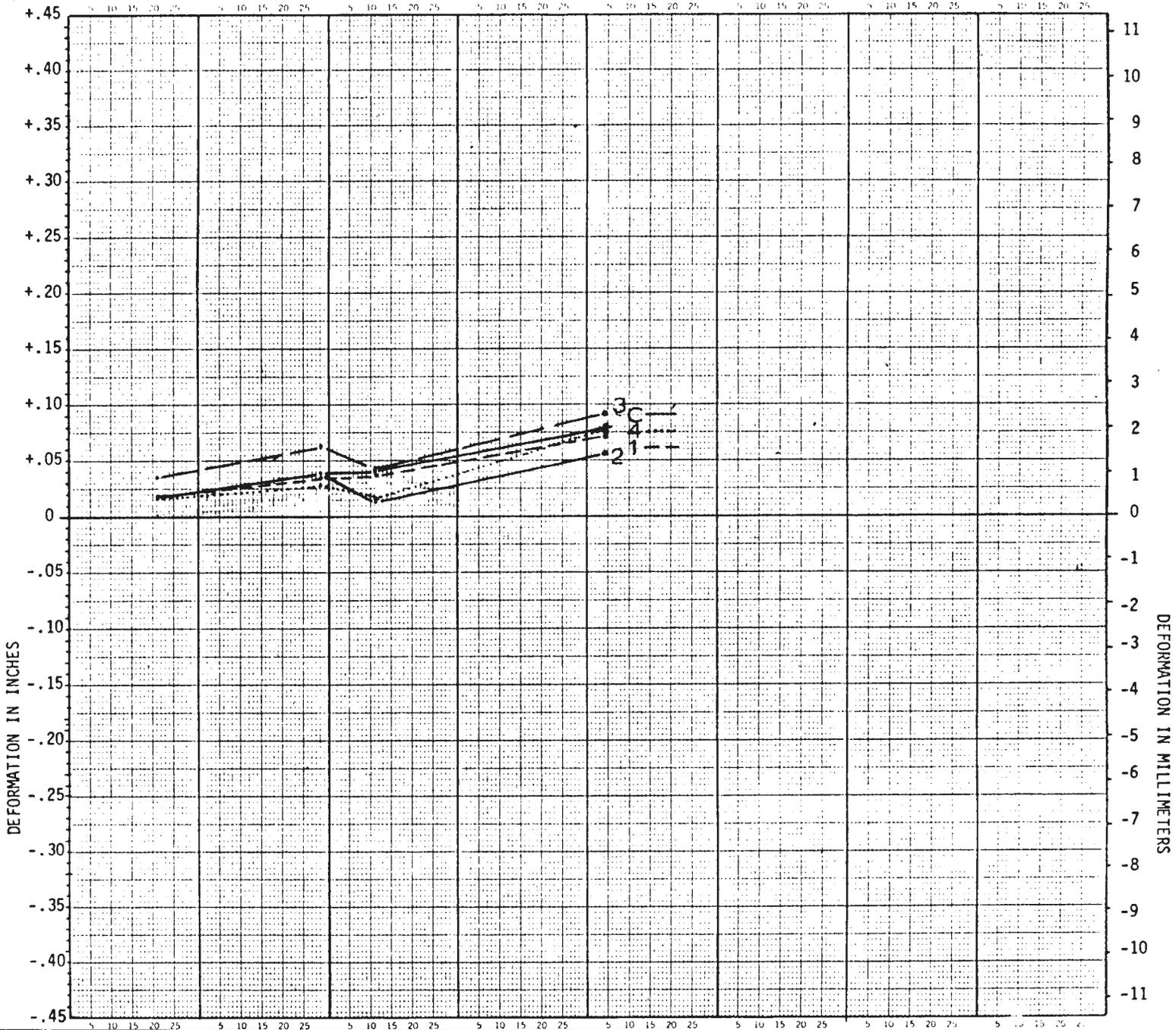
FIGURE V-12

ber 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 vectorial presentation on Figure V-9 only covers the period from mid-September to late October. However, there is apparently uplift at the collar.

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 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.

TEST CAVERN

VECTORS ON 10× SCALE

OUTLINE OF PRELIMINARY
TEST CAVERN

COLLAR .119'

COLLAR .017'

E8   E7   E9

COLLAR .158'

3 .064'

5' .360'

7'-.069'

10' .000'

19' .000'

31'

NL

4' .013"

7" .016"

11' .014"

15' .014"

20' .090'

3' .166'

5' .697'

7' .684'

10'-.307"

12' .000'

26'

NR

470' .000'
47.5' .000'

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
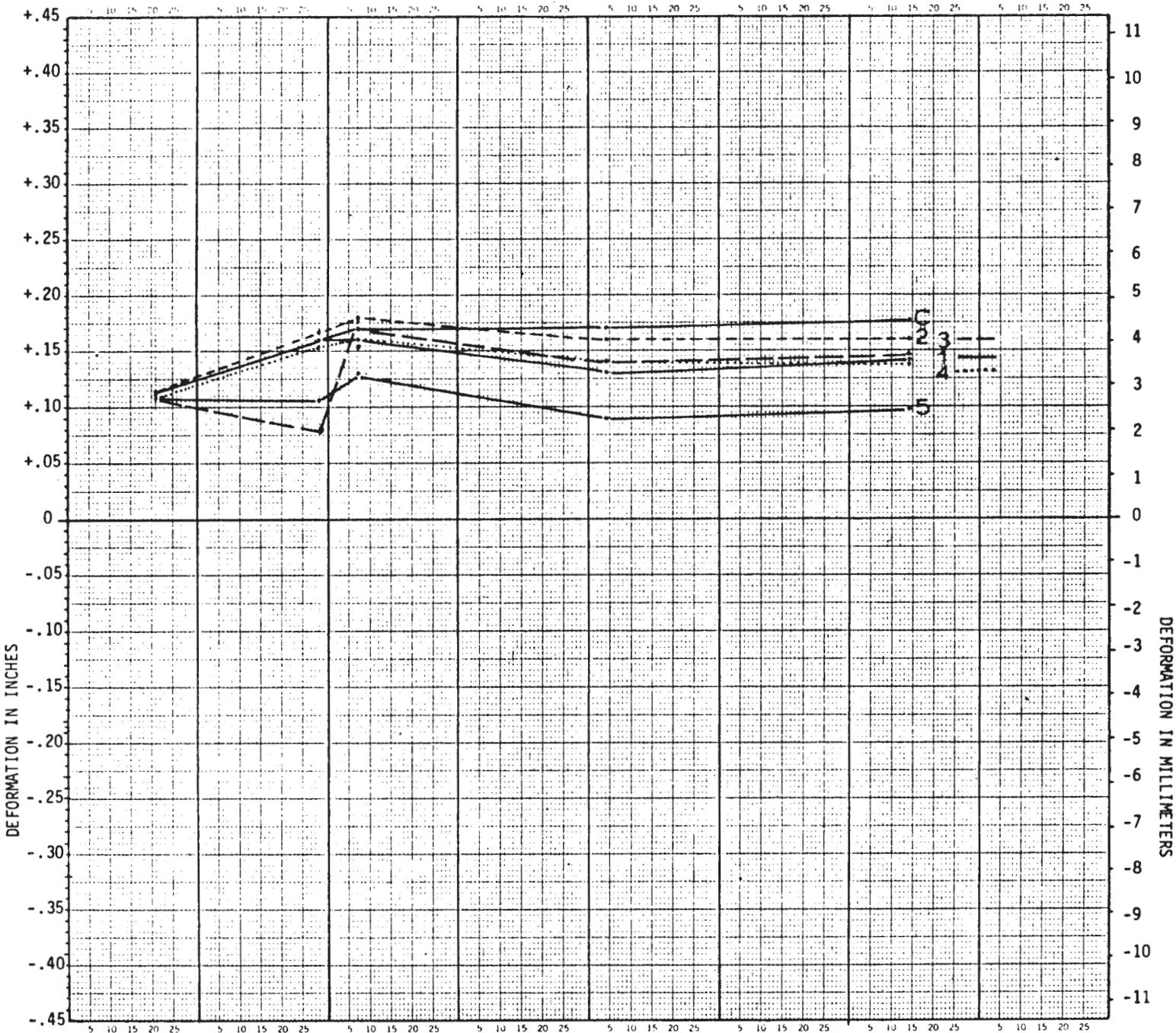
FIGURE V-14

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

EXTENSOMETER NUMBER E 8

WEIR-JONES ENGINEERING CONSULTANTS LTD.
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7
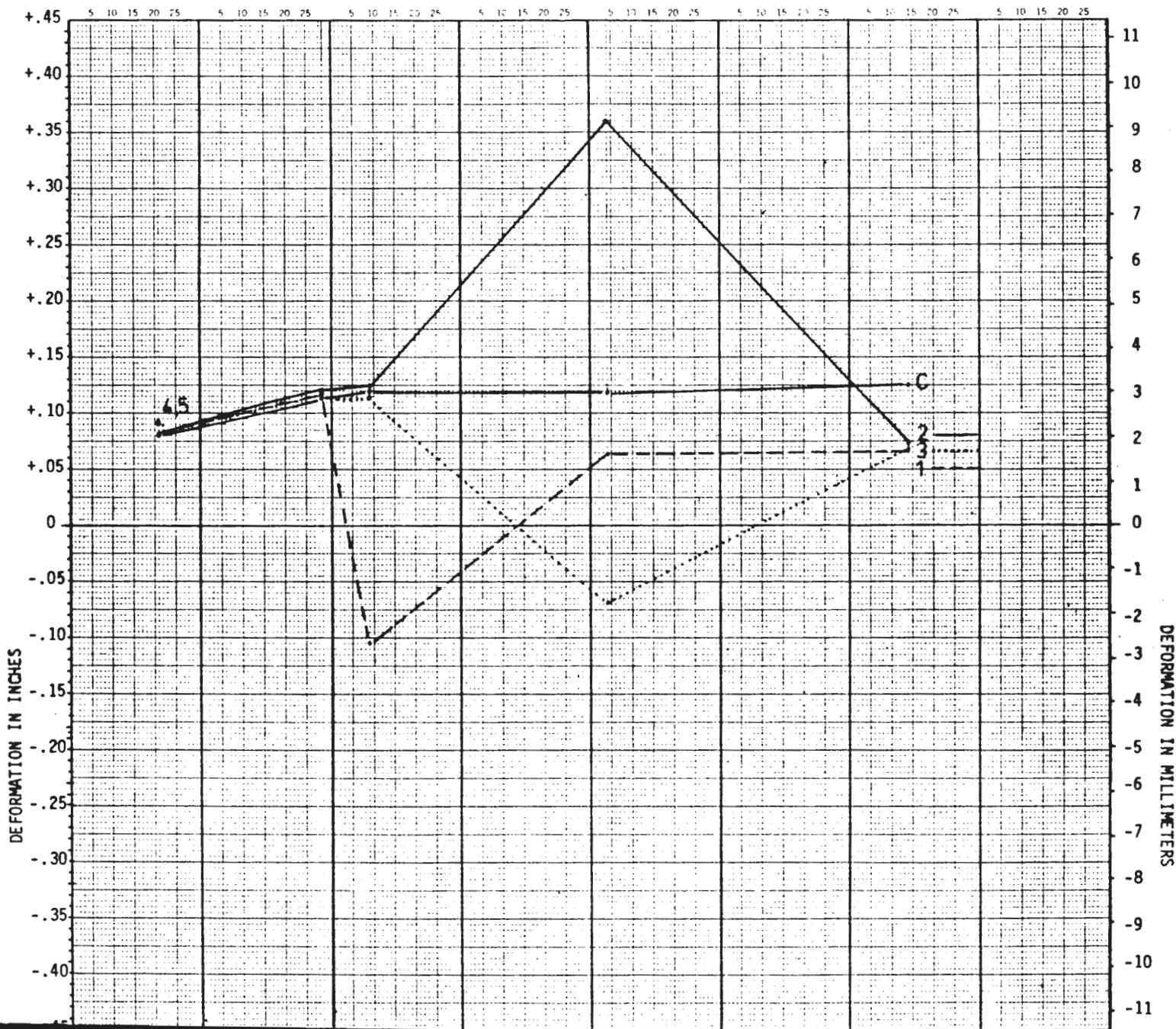
FIGURE V-15

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
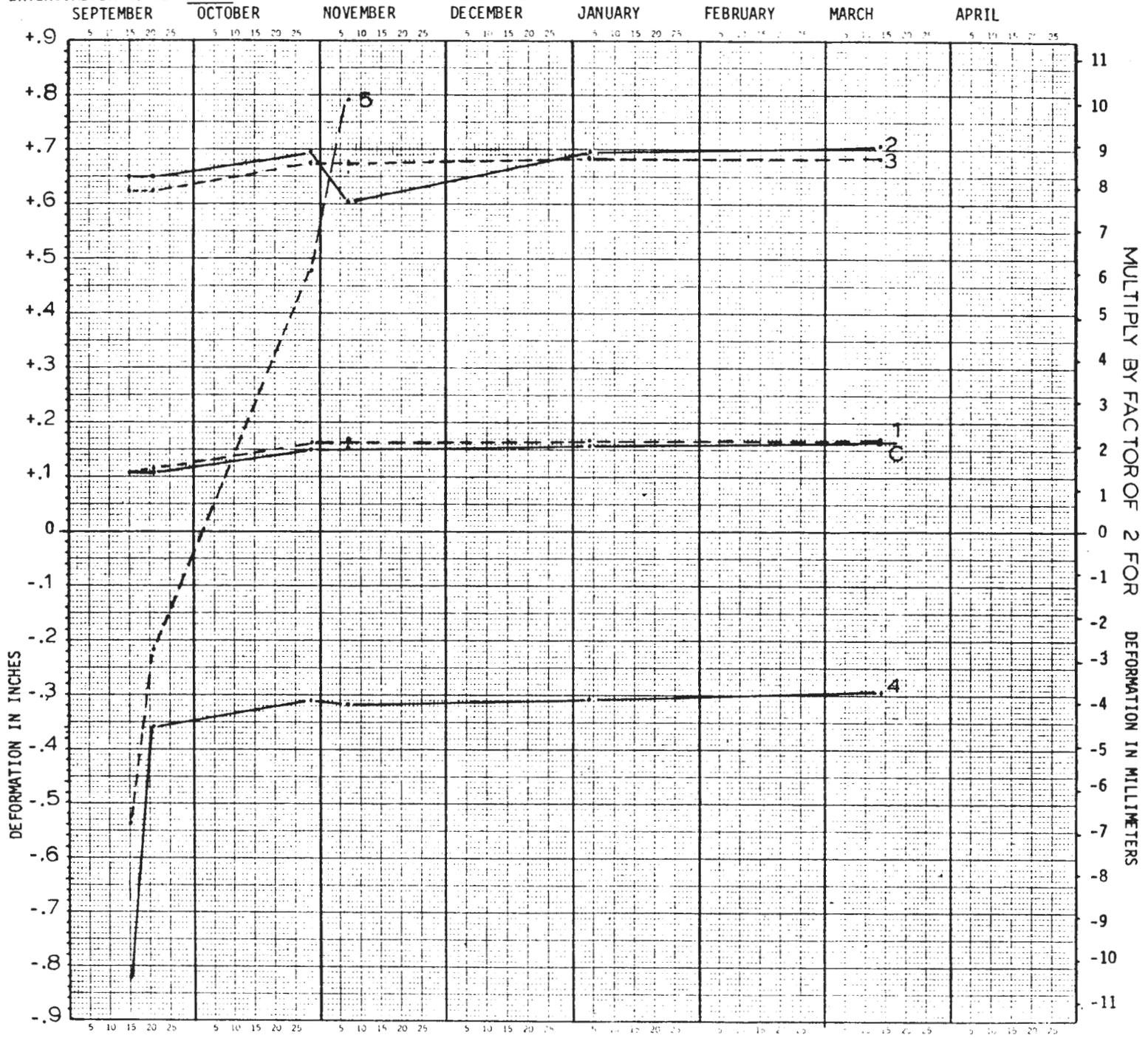
FIGURE V-16

The results obtained from extensometer E8 (Fig. 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. This abnormal data can be eliminated and this would result in a much less curious vectorial presentation than the one currently shown for this borehole on Figure V-13.

Extensometer E9 suffered blast damage soon after installation when the NR Running Tunnel intersected the hole. This can be seen in Figure V-15 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 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 I1 and I2

The inclinometers I1 and I2 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 possi-

GRAPHICAL REPRESENTATION OF MARTA TEST CAVERN EXTENSOMETER DATA

EXTENSOMETER NUMBER E 13

WEIR-JONES ENGINEERING CONSULTANTS LTD.
1310 WEST 6th AVENUE, VANCOUVER, B.C. V6H 1A7
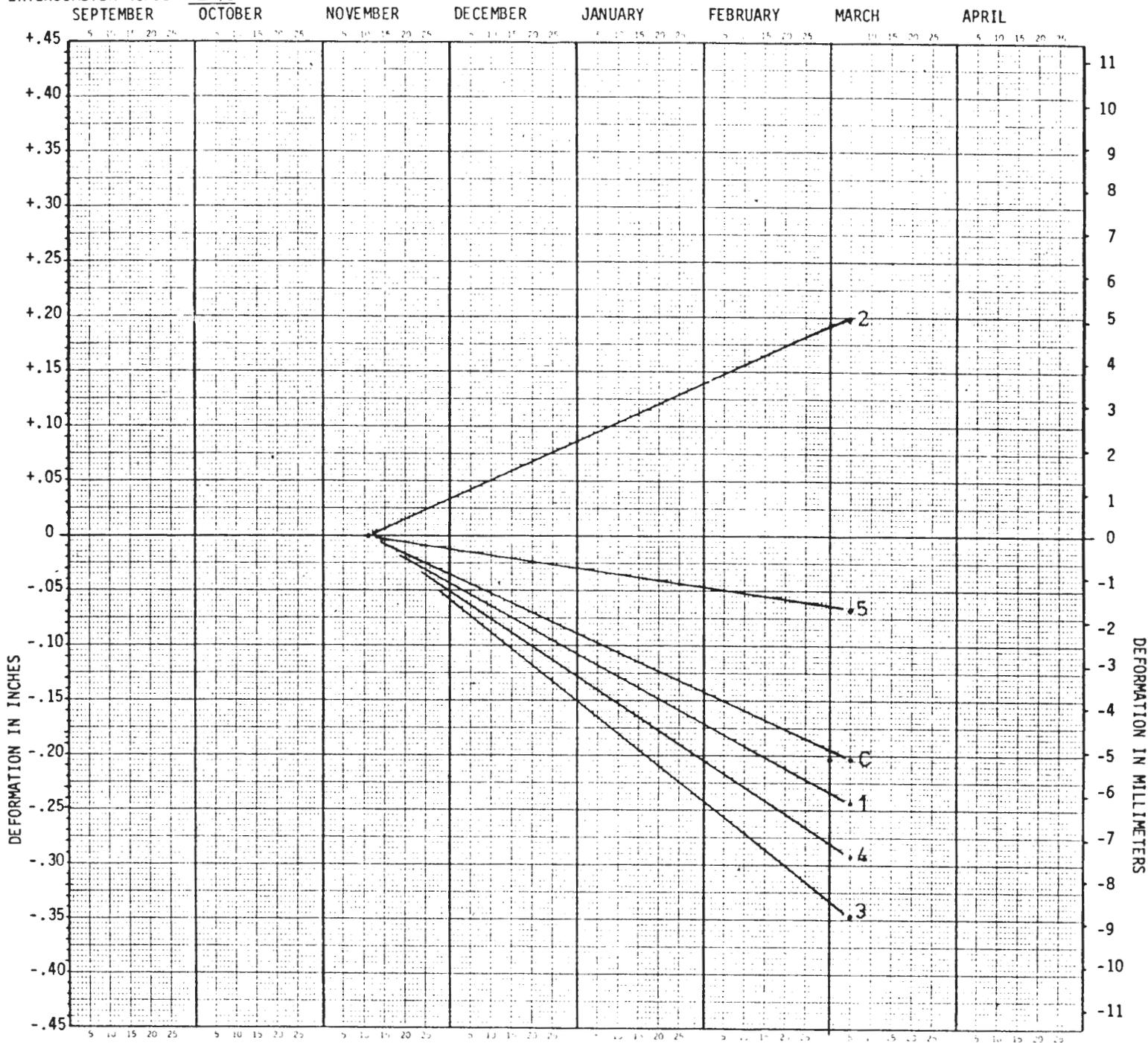
FIGURE V-17

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
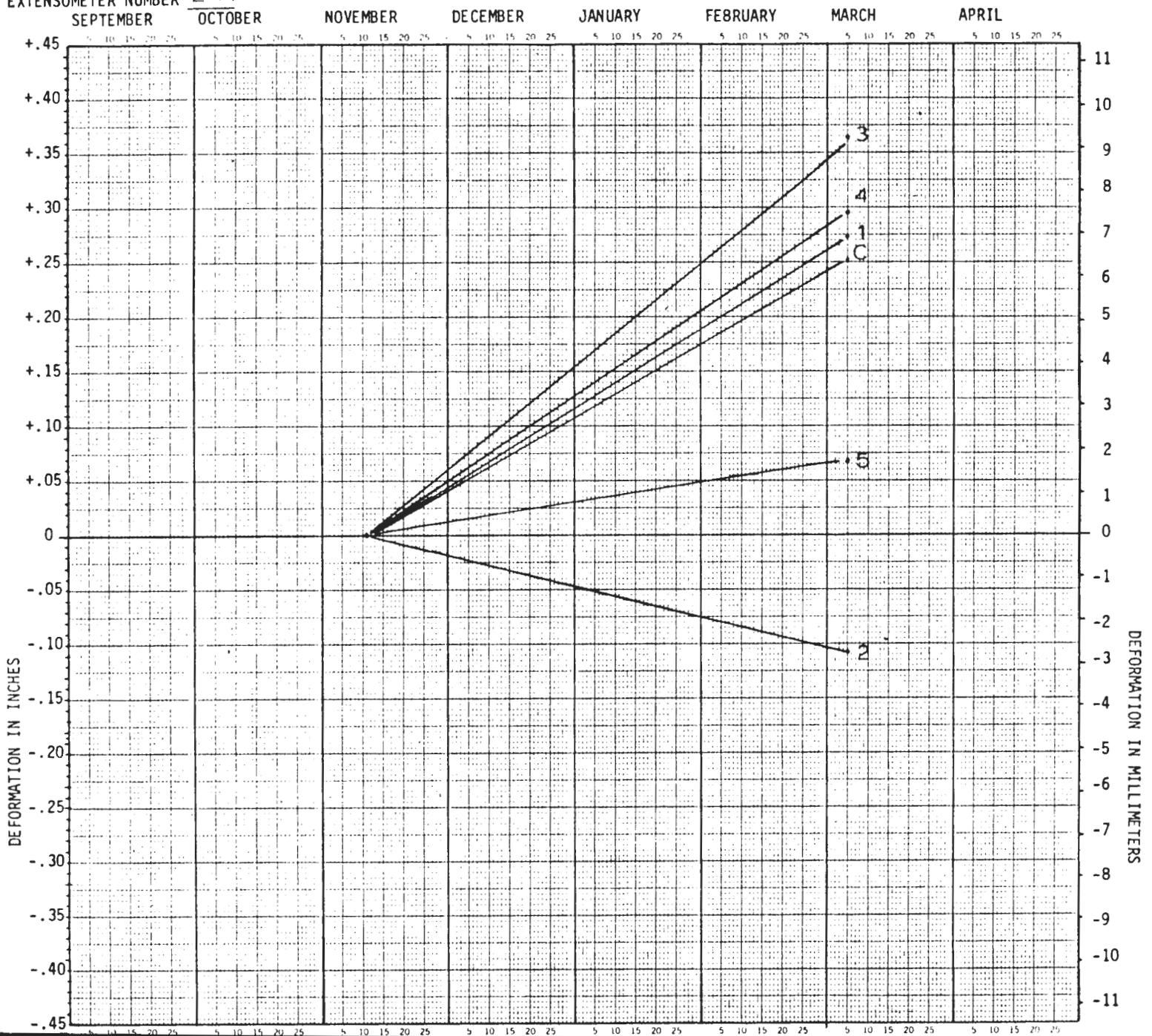
FIGURE V-18

bility 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 the cumulative displacements developed in the north-south and east-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 I1 than at I2 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 I2 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 instruction program, no further lateral deformations will take place.

Due to the 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.

VI. <u>Conclusions</u>

Conclusions will be presented in the Final Report.

```
◆◆◆◆◆◆◆◆◆◆◆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
```
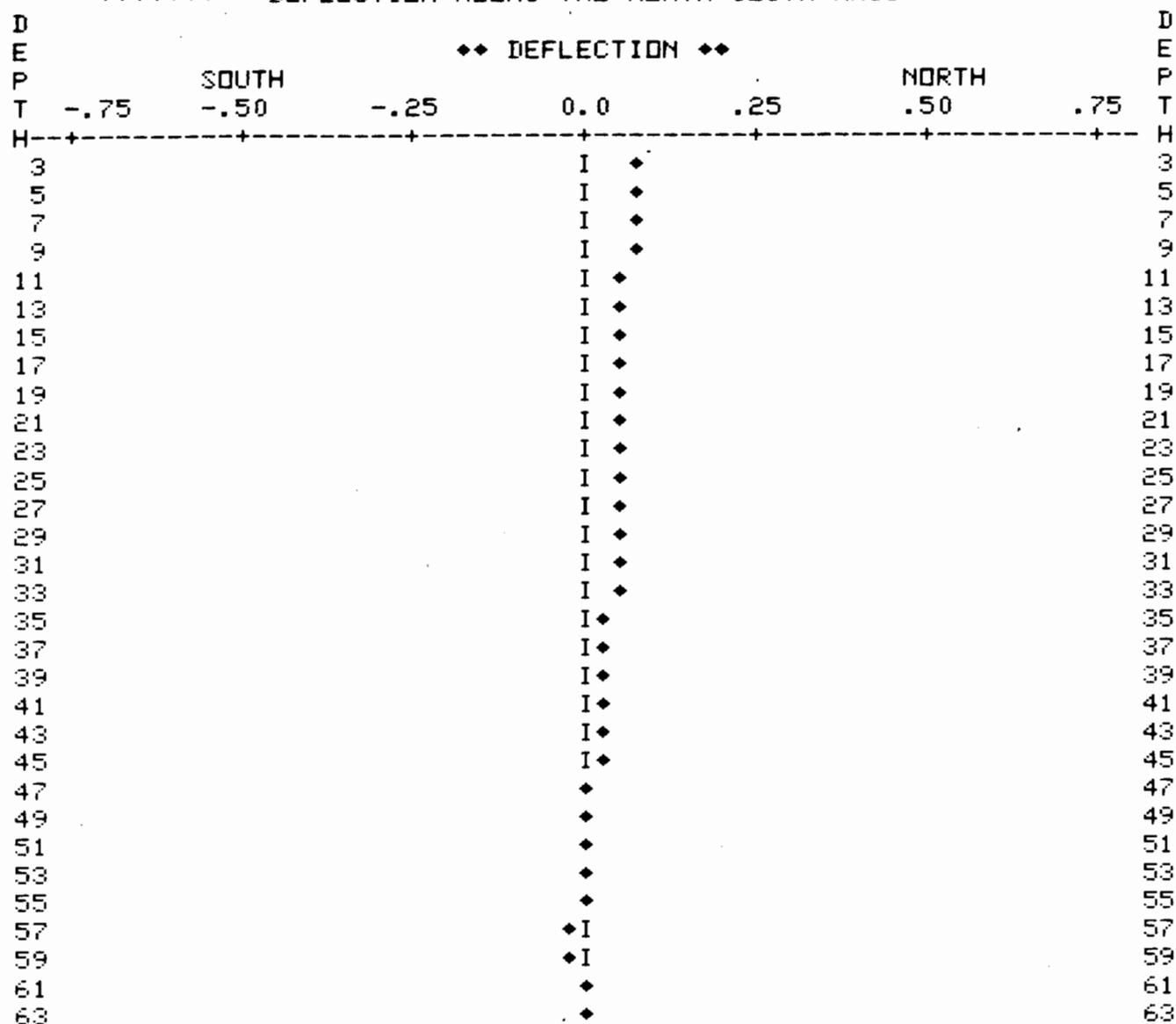
FIGURE V-19

◆◆◆◆◆◆◆◆◆◆◆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

```
◆◆◆◆◆◆◆◆◆◆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

```
◆◆◆◆◆◆◆◆◆◆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
```
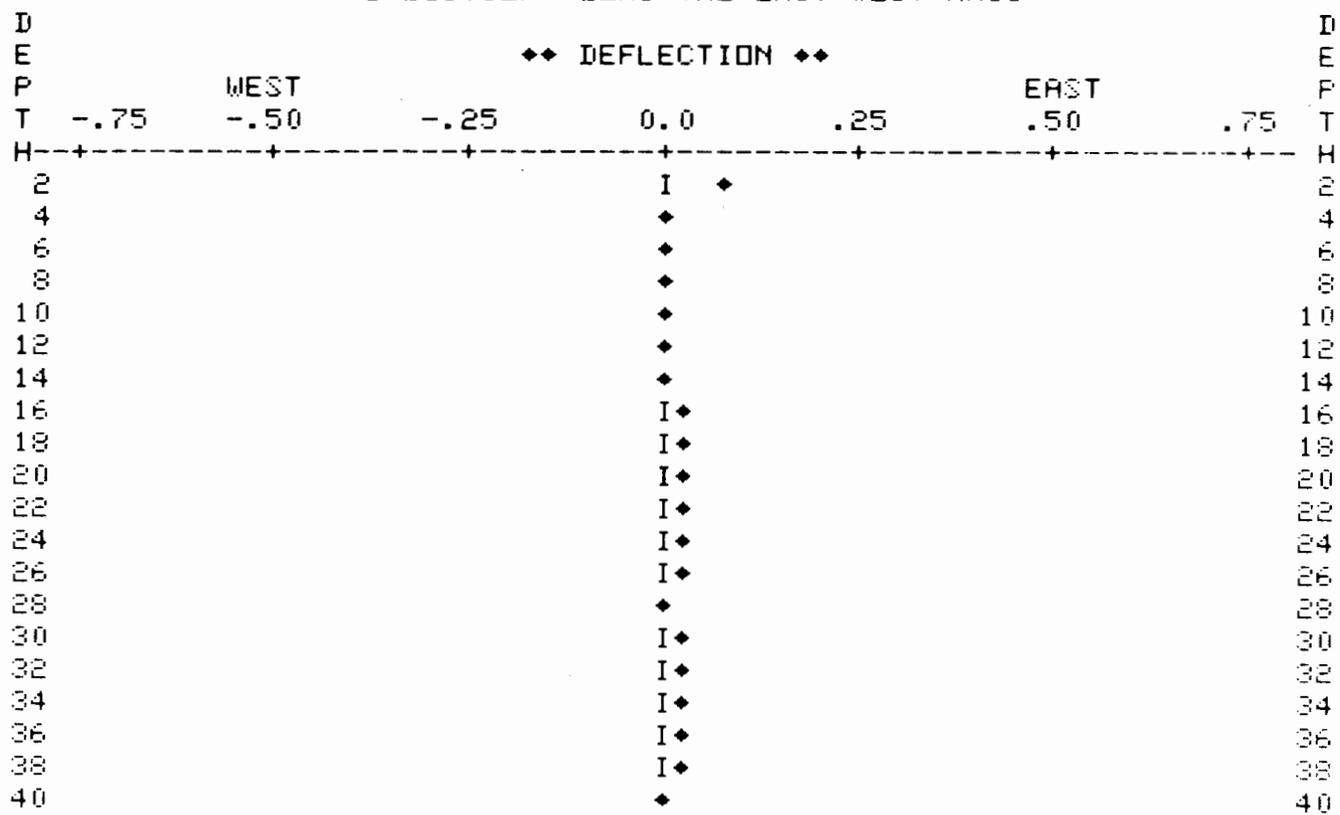
FIGURE V-22

At this time (Interim Report, June 1979) it is not clear if the upward displacement actually observed in the floor of the Research Chamber would have been predicted by the 2-D and 3-D FEM studies, if the actual sequence of excavation which was in fact used had been known and used in th FEM computer studies.

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.

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.

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. 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 blasted at the Contractor's option with no special precautions being taken.

Enlarging the Pilot Tunnel to the 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, de-coupled charges in order to prevent shattering and overbreak of final surfaces.

The Contractor placed 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 drilling than that which was done in the Peachtree Center Station main cavern and Running Tunnels, illustrating 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 de-

sirable to use extremely high-pressure water jets to slot the
perimeter holes, but funds were not available for this extra ef-
fort. 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 deto-
nated ahead of the main part of the round. Small-diameter car-
tridges of explosives are placed in a larger-diameter hole so as
to de-couple the charges from contact with the rock. This re-
duces the pressure against the rock surface and eliminates the
shattering. Figure 1 illustrates the manner in which the
stresses are developed to enhance the desired crack growth. De-
coupled 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 en-
hanced. However, the compressive stresses are reduced by de-
coupling the charges, so that no compression damage is done. At
the same time, rock is much weaker in tension than in compres-
sion, by a factor from 10 to 50 times. Therefore, enough explo-
sive 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 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 develop only one or a few long cracks rather than 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 requires very little preparation, but the results can be quite variable. The latter option was used for the Atlanta Research Chamber. 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 the method is successful under appropriate conditions, when properly applied. The experience at other sites suggests that properly executed scribing can make possible the reduction of explosive quantities to about 1/5 of the normal concentration, and/or moving the drill pattern to much wider spacings between holes. At this site, excellent results could be obtained without fracture-control blasting (see Photo 2). Consequently, experiments with the method were used as demonstration rather than as a need 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 3 and 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 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. This bit consited of an over-size bit ground down to accomodate the original drill hole. It was self-centering the produced a more uniform set of notches. Such a bit is shown in Photo 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 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 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 showing details of the perimeter blasting are presented after the text at the end of this chapter, listed chronologically from Panel 9 through Panel 1.

Panel 9 was shot as a two-stage round using smooth blasting as the perimeter technique. Photo 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) 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 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. Adepate 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 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 centimetrs (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 centimeter (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 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 section 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 is the case with 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-infinte" confinement for pre-splitting, whereas it 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 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 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 break-age. 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 ca-vern and the Running Tunnels. This is thought to be due to (1) greater effort applied to drilling control, and (2) more cau-tious blasting. It is this writer's opinion that the small addi-tional effort for drilling control and a modest reduction in pe-rimeter 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 break-age to vertical surfaces such as the side walls. In some of the areas showing the greatest anisotropy, the influence of the foli-ation gave dramatic results. An example was that of a crack fol-lowing the contorted banding around a tight fold in the rock rather than continuing in a straight line.

Experiments with fracture-control blasting in the Atlan-ta 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 say 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 alrady 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.

TENSION

ENHANCED

COMPRESSION

A

B
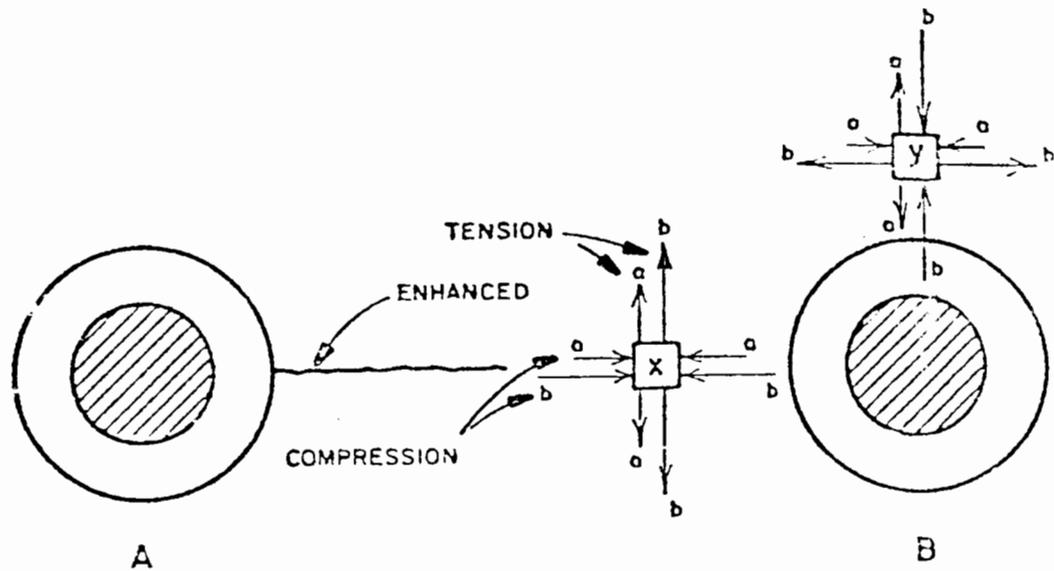
FIGURE VI-1   STRESS DISTRIBUTION FOR PRE-SPLITTING
AND SMOOTH BLASTING.
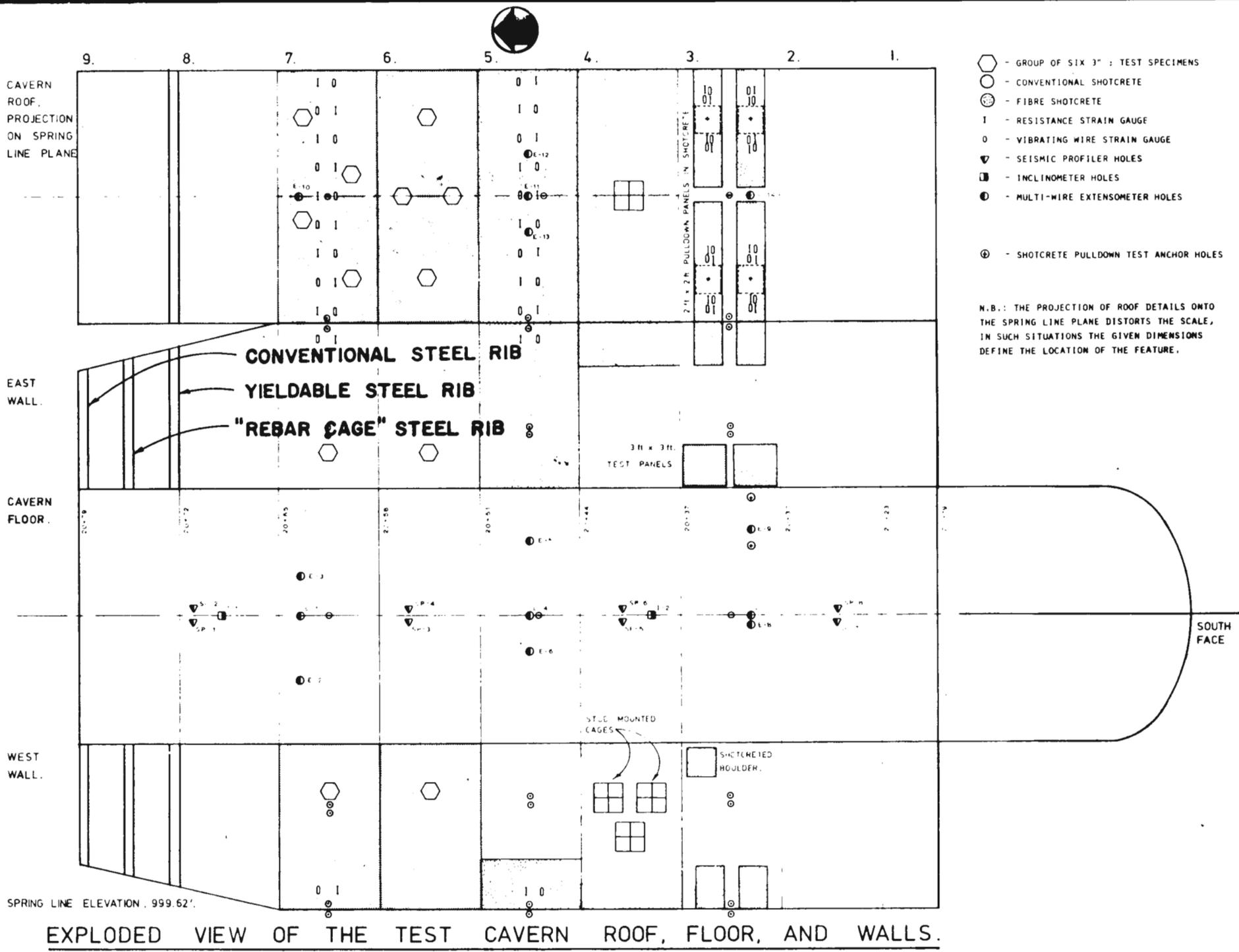
STATION 20+72
LOOKING SOUTH
SCALE: ⅜"=1'-0

The Atlanta Research Chamber

FIGURE VI-2

EXPLODED VIEW OF THE TEST CAVERN ROOF, FLOOR, AND WALLS.

FIGURE VI-3

PANEL 9

"2"   "3"

"4"

"1"   "5"
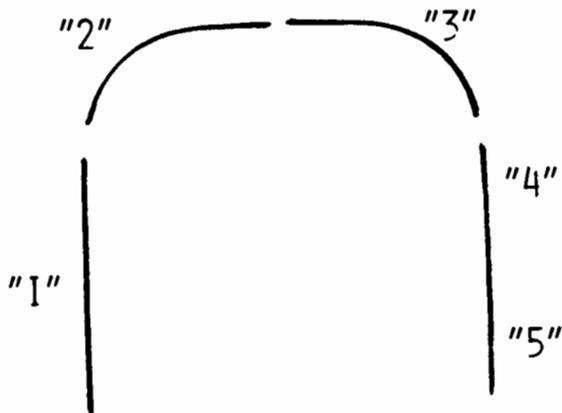
TWO-STAGE ROUND.
SMOOTH BLASTING WITH
    NO SCRIBING.
PERIMETER HOLES:
    SPACING = 18 INCHES.
    BURDEN = 24 INCHES (UP TO
                  32 INCHES IN CROWN)
    DIAMETER = 1 3/4 INCHES.
    DEPTH   = 8 FEET.
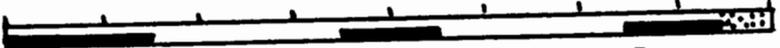
DELAY "1"
3½ FT. T-I PLUS 100 GRAINS/FT. PRIMACORD
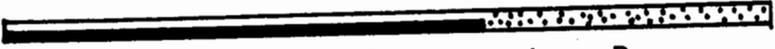
DELAY "2"
5 FT. T-I PLUS 100 GRAINS/FT. PRIMACORD
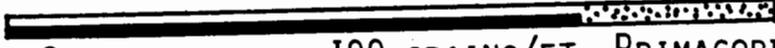
DELAYS "3",
"4", "5".
6 FT. T-I PLUS 100 GRAINS/FT. PRIMACORD

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

FIGURE VI-4

PANEL 8

"1"   "3"

"2"   "4"
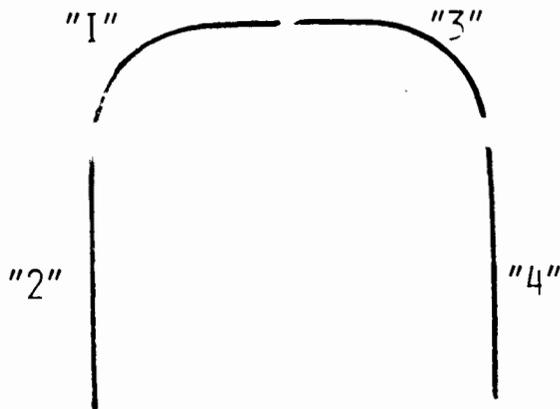
TWO-STAGE ROUND.
SMOOTH BLASTING WITH
    EAST HALF SCRIBED.
PERIMETER HOLES:
    SPACING = 18 INCHES.
    BURDEN = 30 INCHES.
    DIAMETER = 1½ INCHES.
    SCRIBED TO 2 INCHES FOR
        DELAY GROUPS "1" AND "2".
    DEPTH   = 7 FEET.
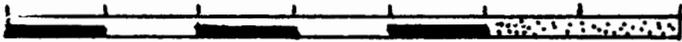
DELAYS "1" AND
      "3"   3 FT. T-I PLUS 50 GRAINS/FT. PRIMACORD
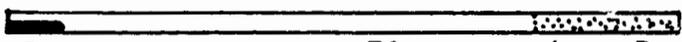
DELAY "2"   6 INCHES T-I PLUS 50 GRAINS/FT. PRIMACORD
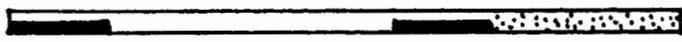
DELAY "4"   2 FT. T-I PLUS 50 GRAINS/FT. PRIMACORD

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-I USED IN BOTTOM.
          BOTTOM BROKE OUT, LEAVING 5 FT. COLLAR.
    "4" - FRACTURED BUT NOT REMOVED.
    CLEAN-UP SHOT WITH 5 FT. T-I IN "2" AND "4".

FIGURE VI-5

PANEL 7

"8"

"9"

"10"
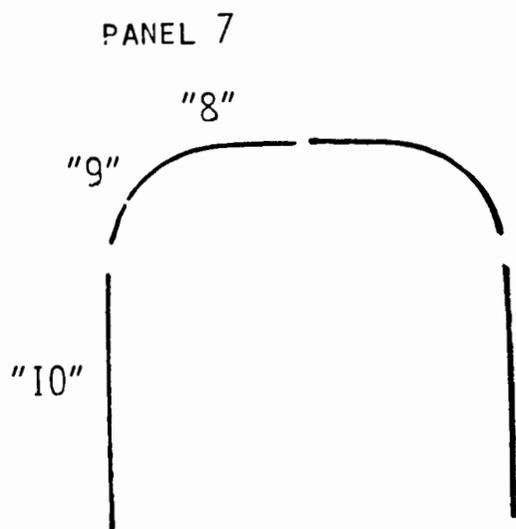
FULL-FACE ROUND.
PERIMETER FIRED ON LAST DELAYS.
EAST HALF SCRIBED.
PERIMETER HOLES:
    SPACING = 24 INCHES.
    BURDEN = 30 INCHES.
    DIAMETER= $1\frac{1}{2}$ INCHES.
    SCRIBED HOLES "8", "9", "10"
    SCRIBING DESCRIBED AS POOR, -
       LESS THAN 1/8 IN. NOTCHES.
    DEPTH = 7 FEET.

DELAYS "8" AND "9"
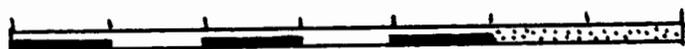
3 FT. T-1 PLUS 100 GRAINS/FT. PRIMACORD

DELAY "10"
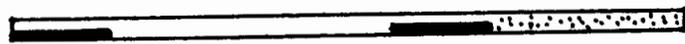
2 FT. T-1 PLUS 100 GRAINS/FT. PRIMACORD

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

"2"

"1"
(B)

"1"
(A)
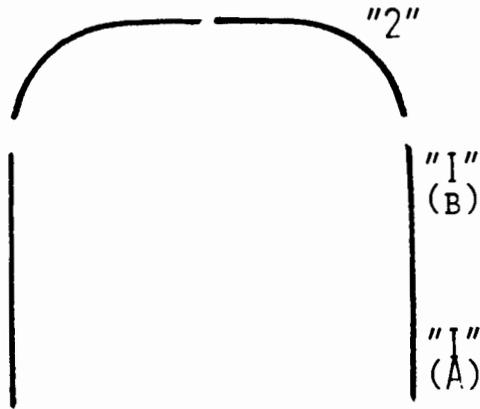
TWO-STAGE ROUND.
SMOOTH BLASTING WITH
     WEST HALF SCRIBED.
PERIMETER HOLES:
     SPACING = 30 IN. FOR "1"
             = 36 IN. FOR "2"
     BURDEN = 36 IN. FOR "1"
             = 18 IN. FOR "2"
     HOLES "1" AND "2" WERE SCRIBED,
        BUT NOTCHES NOT WELL DEFINED.
     DEPTH   = 6 FT.

DELAY "1"(A)

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

DELAY "1"(B)
  AND "2"

4 FT. T-1
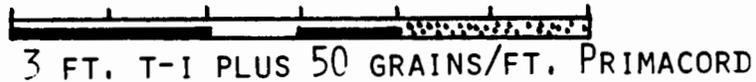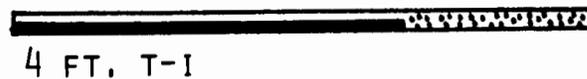
ALL HOLES STEMMED.
NON-ELECTRIC CAP IN EACH HOLE WITH 50 GRAIN
     PRIMACORD TRUNKLINE FOR EACH DELAY GROUP.
RESULTS:
     HOLES "1"(A) DID NOT BREAK WELL IN BOTTOM,
        LEAVING A STUB.
     HOLES "1"(B) AND "2" BROKE TO FINAL LINE.

FIGURE VI-7

PANEL 5A AND 5B
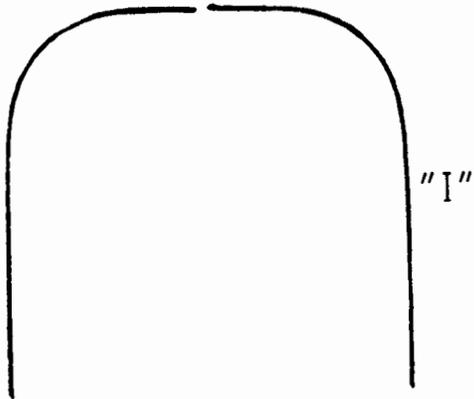
5A - TWO-STAGE ROUND,
        SMOOTH BLASTING,
5B - PERIMETER SHOT AS PRE-SPLIT,
WEST HALF OF BOTH ROUNDS SCRIBED,
PERIMETER HOLES:
    SPACING = 30 INCHES,
    BURDEN = 12 INCHES,
    DEPTH   = 6 FEET,

"I"

DELAY "I"

4 FT, T-I PLUS 50 GRAINS/FT, PRIMACORD
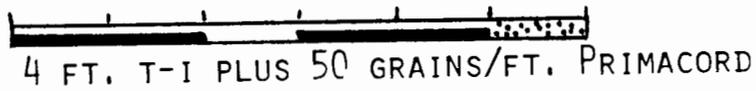
HOLES THOUGHT TO BE STEMMED,

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

"6"   "3"

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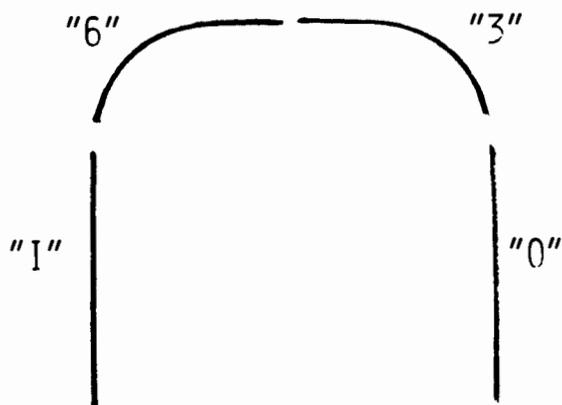
PRE-SPLIT BLAST.
WEST HALF SCRIBED.
PERIMETER HOLES:
    SPACING = 30 INCHES.
    BURDEN = 15 INCHES.
    DEPTH   = 8 FEET.
    HOLES "0" AND "3" SCRIBED.

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

"3"   "1"

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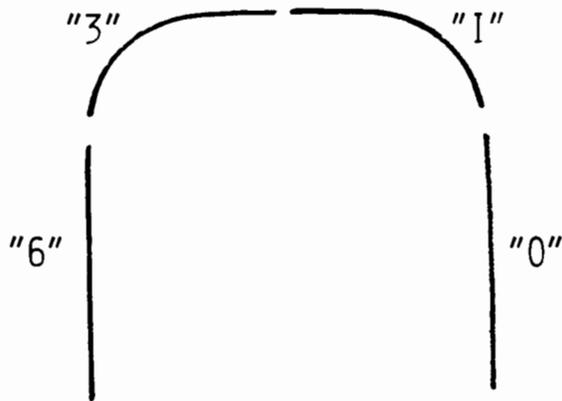
PRE-SPLIT BLAST.
WEST HALF SCRIBED.
PERIMETER HOLES:
    SPACING = 30 INCHES.
    BURDEN = 6 INCHES.
    DEPTH   = 10 FEET.
    HOLES "0" AND "1" WERE SCRIBED,
      BUT THE INSPECTOR'S NOTES
      INDICATE THAT THERE WAS A
      DIFFICULTY WITH THE SCRIBING
      TOOL ROTATING.

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 I

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

PHOTO NO. VI-1

PHOTO NO. VI-2

PHOTO NO. VI-3

PHOTO NO. VI-4

PHOTO NO. VI-5
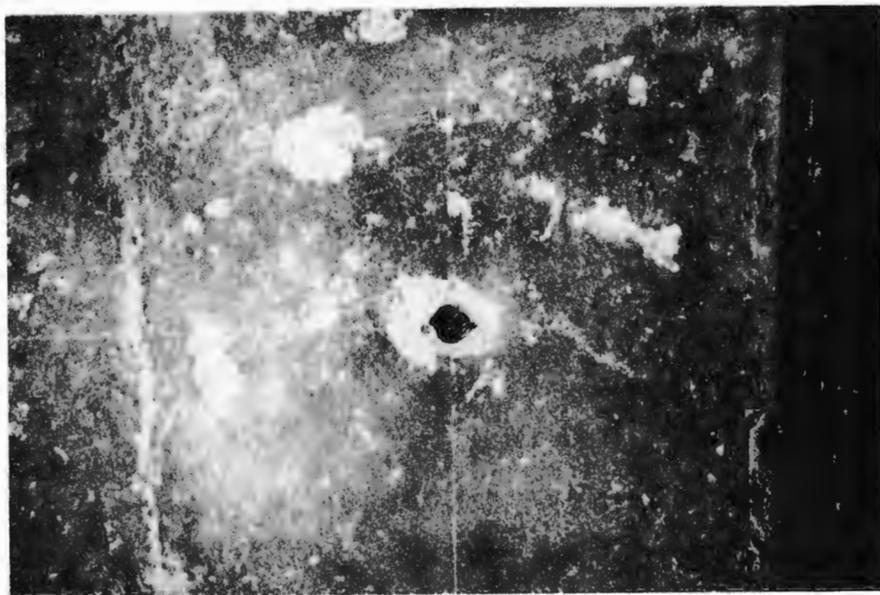
PHOTO NO. VI-6

PHOTO NO. VI-7

PHOTO NO. VI-8

PHOTO NO. VI-9

PHOTO NO. VI-10

CHAPTER VII

CONVENTIONAL SHOTCRETE

(to be prepared)

CHAPTER VIII

STEEL-FIBER-REINFORCED SHOTCRETE

(to be prepared)

MONOGRAPHS - SUMMARY AND OVERVIEW

Don Rose
Supervising Civil Engineer
Tudor Engineering Company
San Francisco, California

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 added, to contribute their ideas on insurance, legal matters, specifications, and so on. The resulting collection of 24 monographs on 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 are three overview papers by owners representives (by Kaiser, Gallagher, and Birrer). Two labor union monographs follow (by Weatherl and Marr). Four contractors (Burtleson, Jensen, Gillis and McCusker) then express their views. A brief explanation of a new labor training program follows (by Scott). Legal matters are covered next (by Max Greenberg), followed by specifications by (Larsen) and 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 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 Cording, Mahar, Fernandez-Delgado and Van Sint.) Steel-fiber-reinforced shotcrete as used in the MARTA Running Tunnels is the subject of a case history (by Blanck). The last monograph is on tunnel photography (by Oliveira and Morrison).

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, specifications 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.

In the overview, it seems to me that monographs by engineers, labor, contractors, legal, specifications 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

Don Rose
Supervising Civil Engineer
Tudor Engineering Company
San Francisco, California

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 the cause of many overall 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 if 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 sepa-
rate contracts later. $\underline{1}/$

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 dif-
ferences (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 com-
monly 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 univer-
sity 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 successful-
ly using the tools provided by this education to manipulate num-
bers and thus engineer safe designs. Of course, the basic numer-
ical data input is highly reliable: careful preconstruction test-
ing of steel, concrete and wood guarantees that the materials
used are uniform in composition throughout the project, and the

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$\underline{1}/$ "Urban Tunnels - An Option for the Transit Crisis" Matt S.
Walton and Richard J. Proctor, <u>ASCE Transportation Journal</u>, Nov.
1976.

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. Being often outdoors and being 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 into number form and subsequently manipulating these numbers. 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

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

a modest exposure to the subject [of tunneling] in their own background." Further, Peck reports, "Several individual suggestions were also received. Among these, perhaps the most persistant was the 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 would be 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 lagging 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

[3] Rock Tunneling with Steel Supports, Proctor and White, Commercial Shearing and Stamping Company, Youngstown, Ohio, 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 lagging 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 <u>not</u> 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 and keep small loose pieces from dropping out of the roof.

These principles are true for any type of ground and any type of tunnel lining. The 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 numbers describing the ground will be different 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 pressure and displacement on a tunnel lining on 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 frequently done by PhD 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 these limitations is the inexact input data describing lumpy, impure and fractured ground. Another significant problem is that computed 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 acount 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 permit a marked reduction in initial ground pressure to occur. Hence, no real tunnel lining will ever take the heavy ground loads a perfectly rigid lining might carry if it were placed so promptly that it attempted to hold 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 as pieces of 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 more than three to four feet
thick lie as a massive dead load on a circular twenty foot dia-
meter 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 point of difference between North American and Euro-
pean designers and builders is this: should tunnels be built so
that the ground is supported quickly and excessive loosening is
never permitted, or not? If the only ground loads to be sup-
ported are those resulting from the first inward displacements
tending to close the new void, then thin linings are quite suf-
ficient 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, the moral is:
since dead loads of loose ground, or rock blocks more than about
one meter thick require thicker linings and heavier supports,
don't permit such a loose zone to develop. Install thin, flex-
ible linings promptly, before any ground actually loosens.

There are several excellent reports and highly theore-
tical PhD thesis papers which have analyzed the behavior of the
ground and the necessary tunnel lining strength to stabilize the
opening. These are all laudable and some are readable, but the
real proof is in the low cost successful tunnels built in other
countries using these principles. Details may vary and theory
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? One reason is lack of good tunnel edu-
cation. Another is because the engineer might get sued and lose

his shirt in court if a new method doesn't work well. If one is 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 PhD 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: the design and construction of 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. Satisfaction is internal and comes from knowing a job was well done using the very best of one's 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 the 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, that the federal government has 80 percent of the money in savings to gain, in promoting innovative engineering and non-luxurious projects. The local owner may gain 20 percent of the total 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 has some risk and 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 tunnel techniques be used and will provide additional funds when occasionally 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, it is my view that the human desire to do a job well will be motivation enough 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.
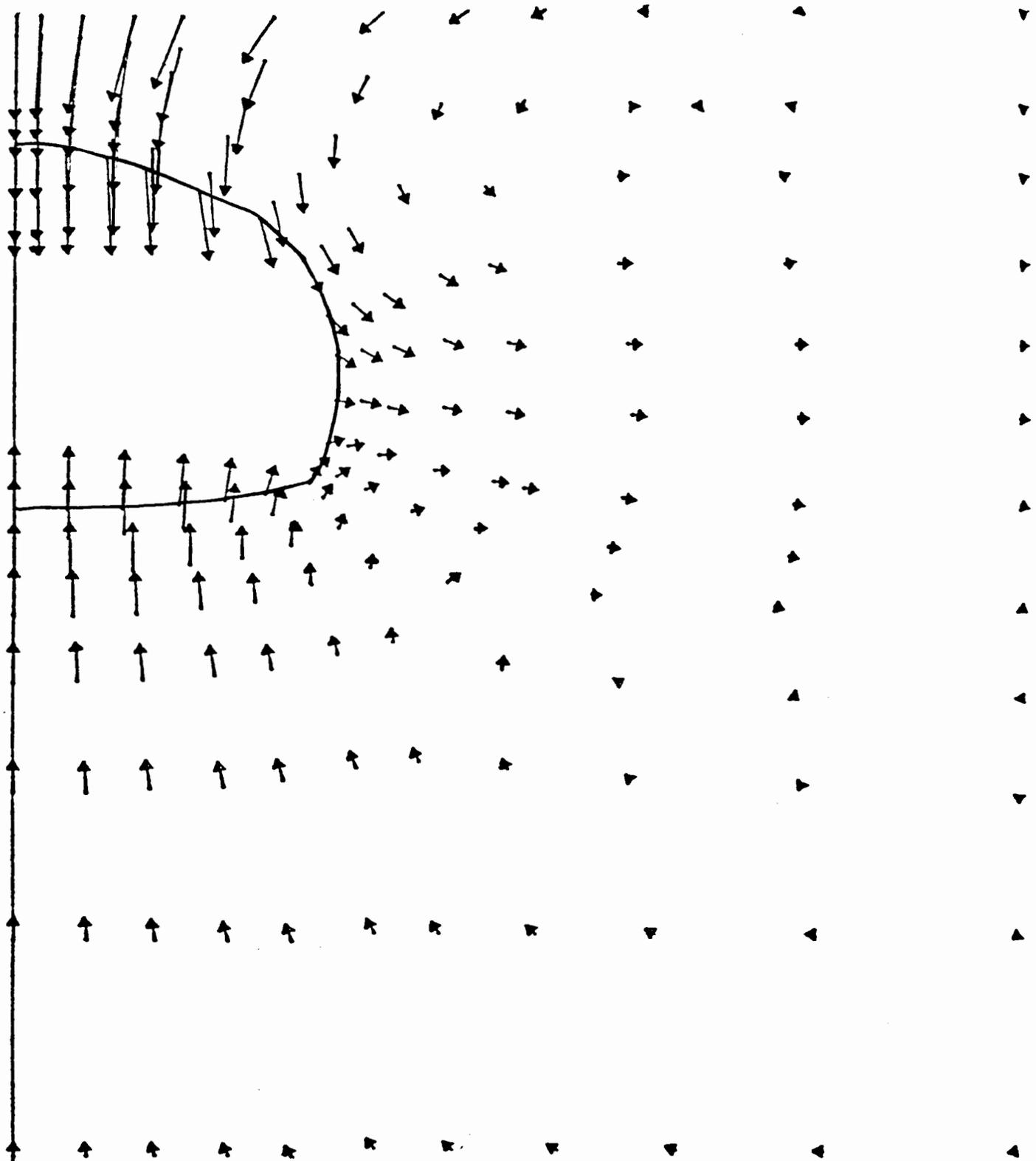
TUNNEL FACE
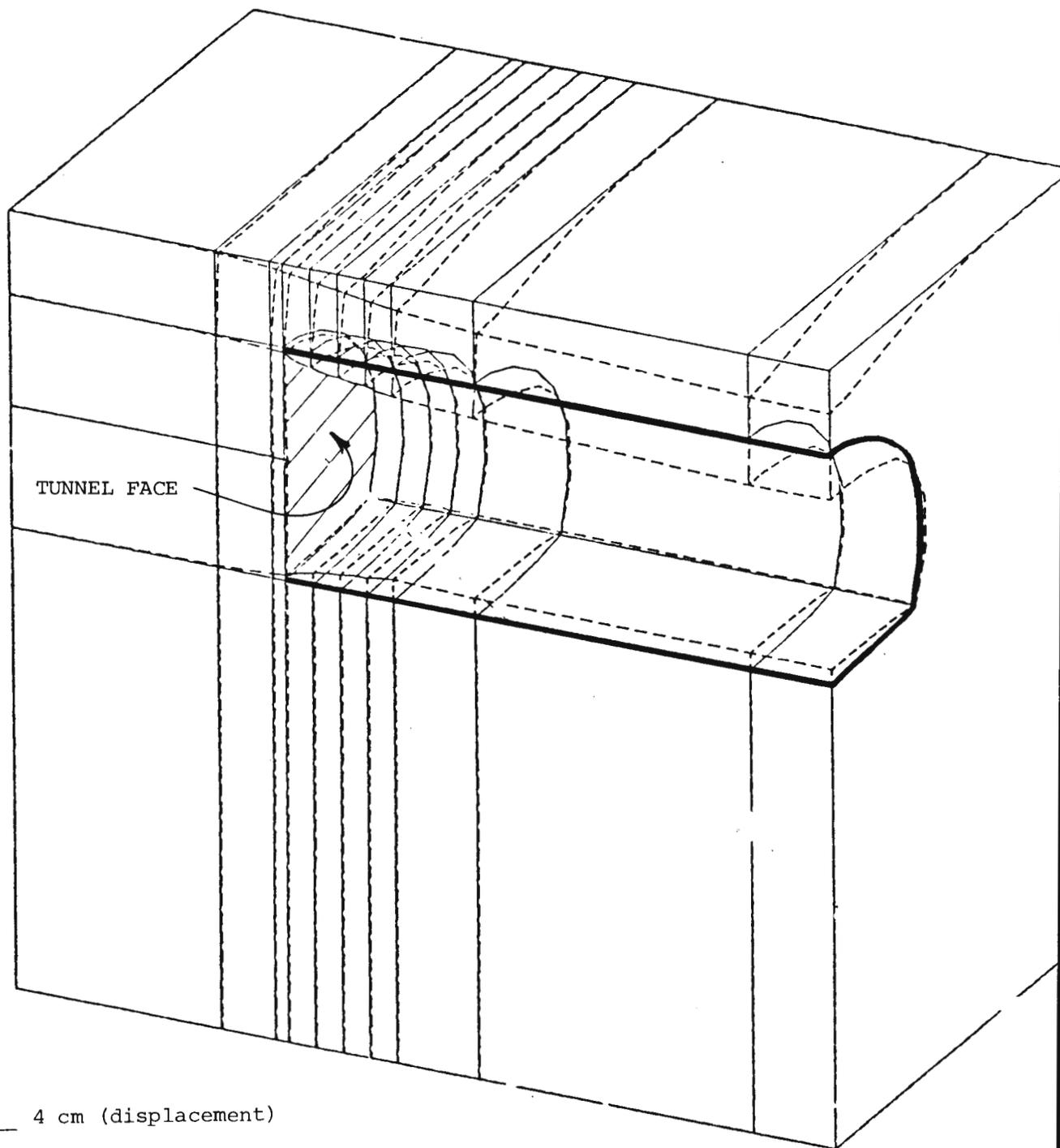
___ 4 cm (displacement)

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.
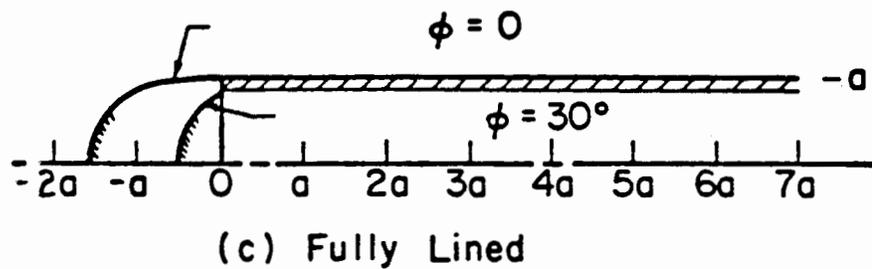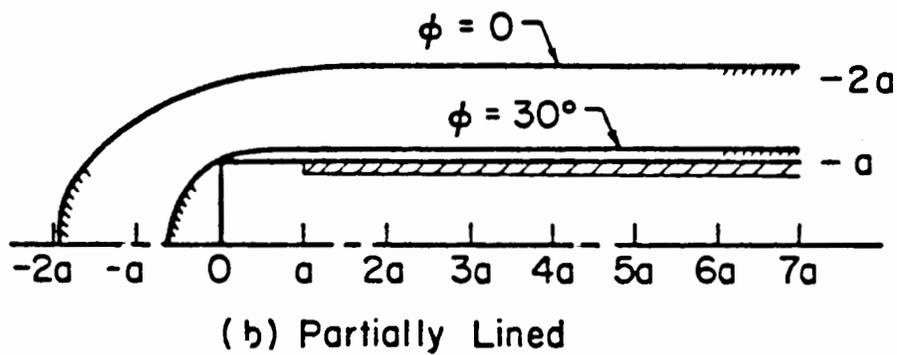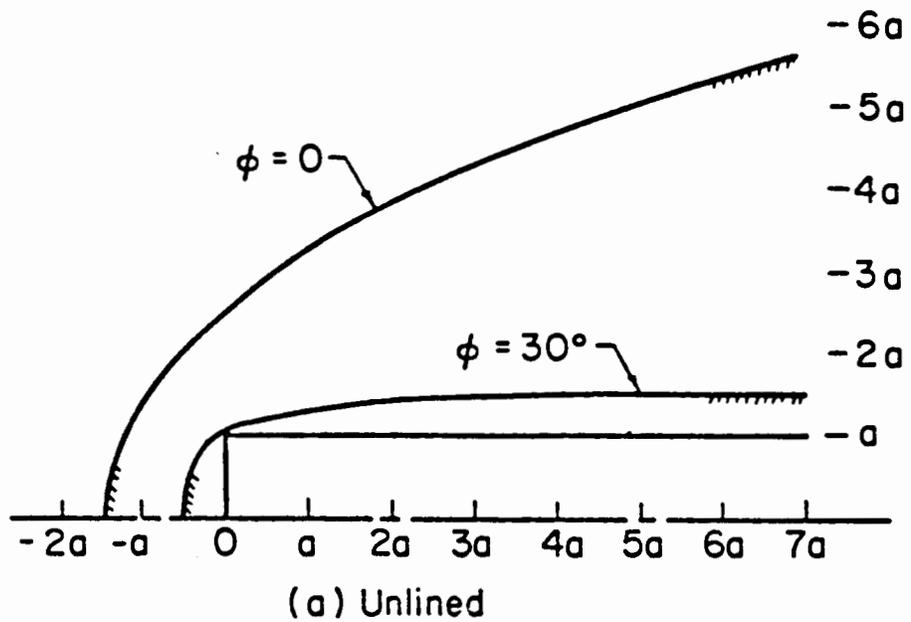
After Gartung, 1978

FIGURE 2

(a)   L < 4D

Zones of influence around tunnel faces

Tunnel II   D

Tunnel I   D

L

(b)   L > 4D

Tunnel II

Tunnel I

L

(c)

Tunnel II

Tunnel I   Existing tunnel
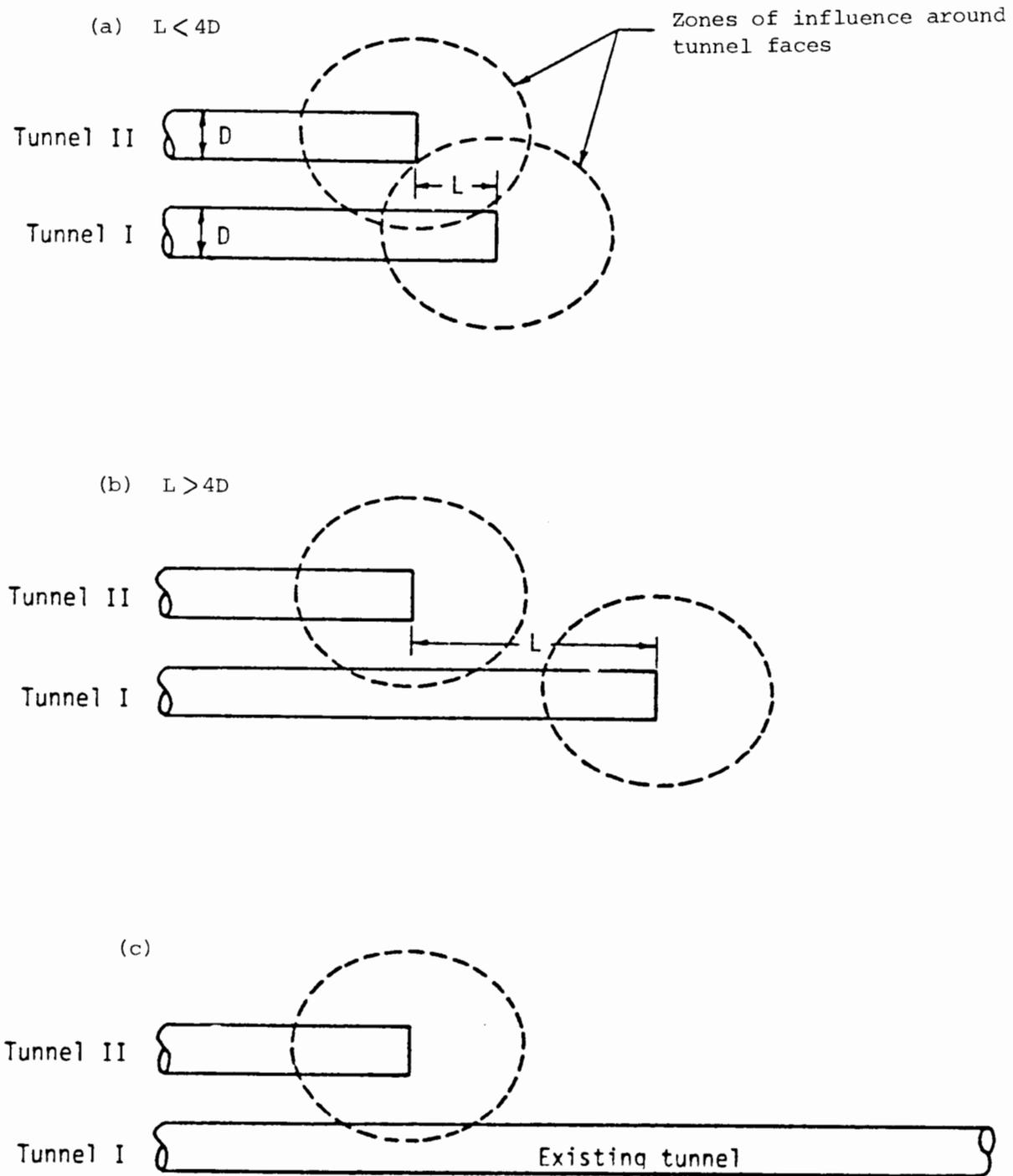
EFFECT OF RELATIVE POSITION OF TWO TUNNEL FACES

ON STRESSES IN ADJACENT TUNNEL

After Ranken et al 1978

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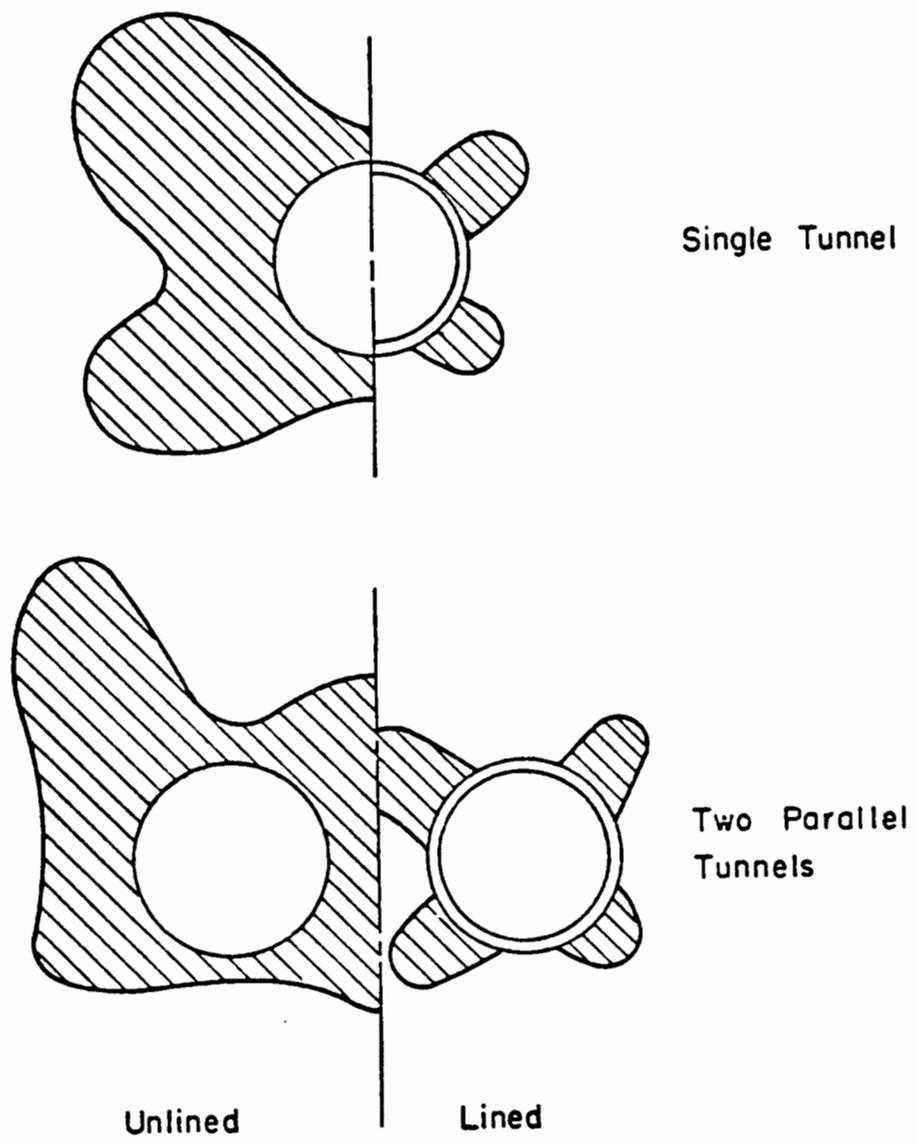for Tunnels"   UMTA-IL-06-0043-78-3

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.
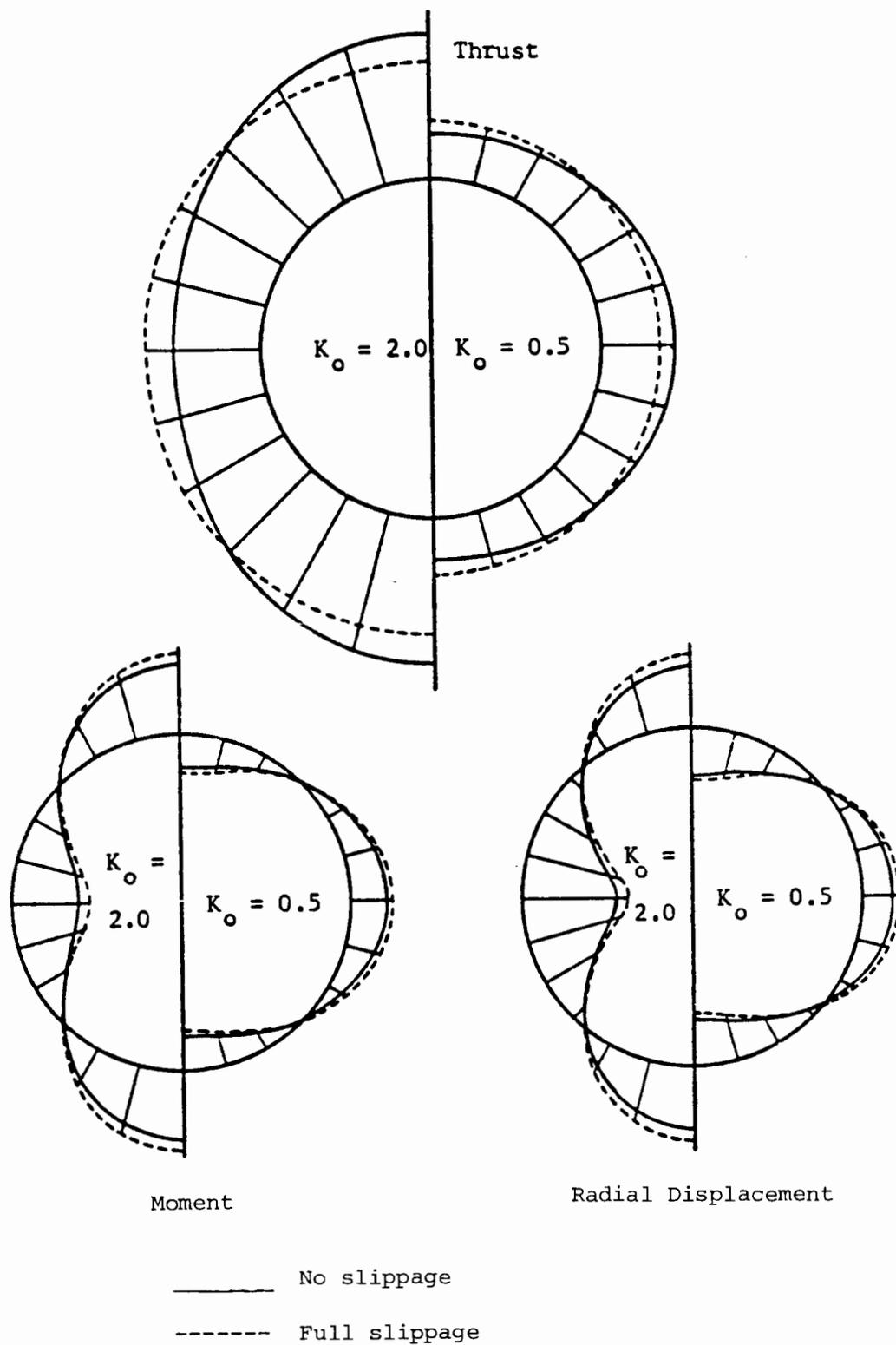
FIGURE 4

Single Tunnel

Two Parallel
Tunnels

Unlined   Lined

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

After Ranken et al 1978

FIGURE 5

Thrust

$K_o = 2.0$ | $K_o = 0.5$

$K_o = 2.0$ | $K_o = 0.5$

Moment

$K_o = 2.0$ | $K_o = 0.5$

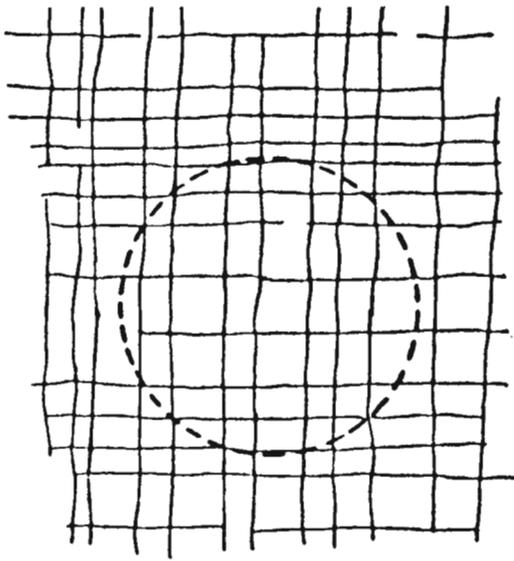Radial Displacement

_____ No slippage

------- Full slippage

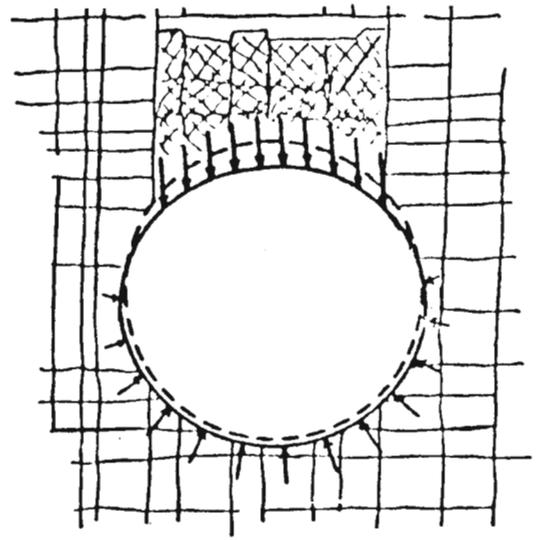DISTRIBUTIONS OF LINER THRUSTS, MOMENTS AND RADIAL DISPLACEMENTS

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.
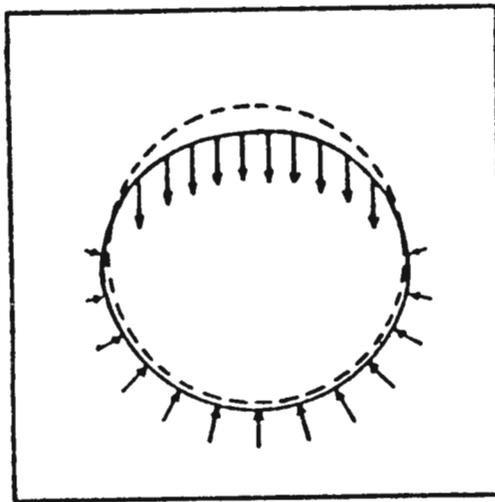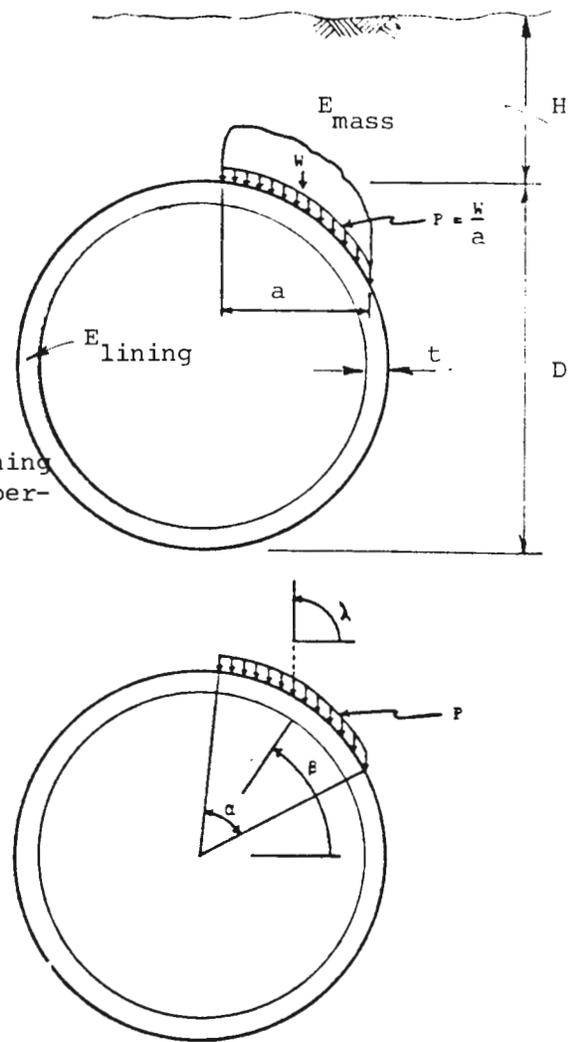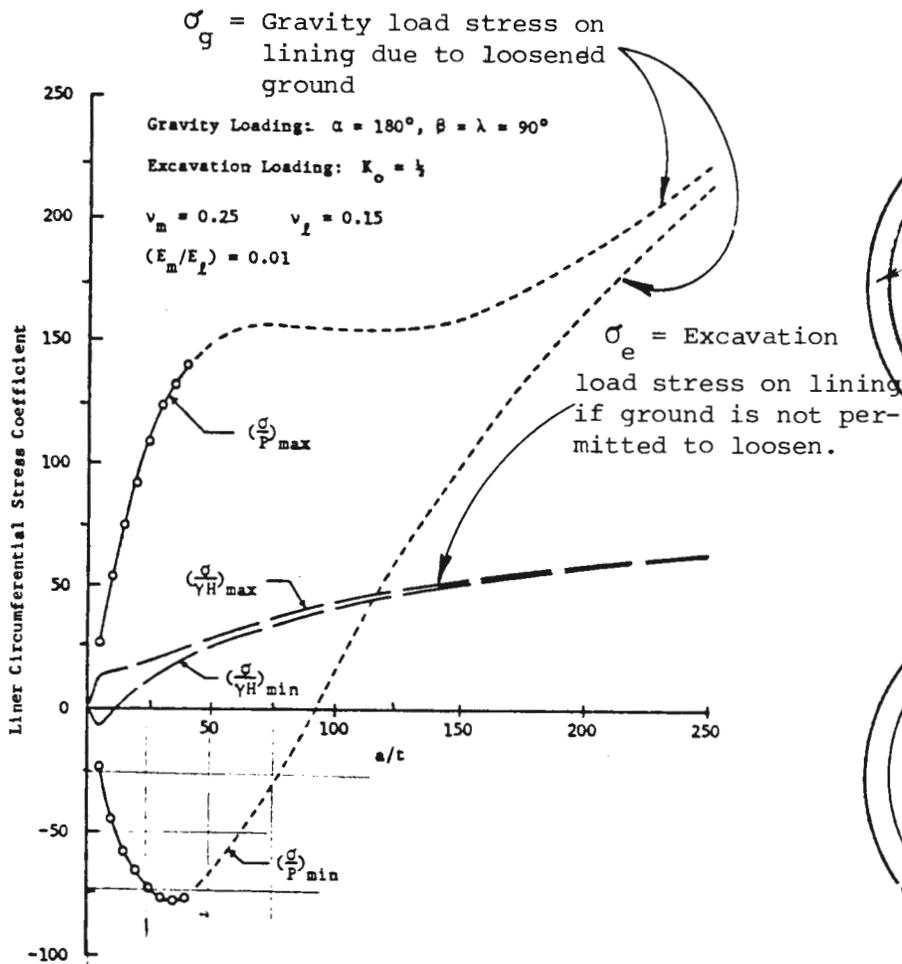
a and b : Problem to be simulated

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

σ_g = Gravity load stress on lining due to loosened ground

Gravity Loading: $\alpha = 180°$, $\beta = \lambda = 90°$

Excavation Loading: $K_o = \frac{1}{2}$

$\nu_m = 0.25$ $\nu_\ell = 0.15$

$(E_m/E_\ell) = 0.01$

σ_e = Excavation load stress on lining if ground is not permitted to loosen.

250

200

150

100

50

0

-50

-100

Liner Circumferential Stress Coefficient

$(\frac{\sigma}{P})_{max}$

$(\frac{\sigma}{\gamma H})_{max}$

$(\frac{\sigma}{\gamma H})_{min}$

$(\frac{\sigma}{P})_{min}$

50   100   150   200   250

a/t

E_{mass}

$P = \frac{w}{a}$

E_{lining}

t

H

D

a

w

λ

β

α

P

γ = Unit weight of ground

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_1 = 3,000,000 psi), the ratio E_m/E_1 = 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 σ_g = (P)(-80) = $\frac{3' \times 10' \times 150\ pcf}{10'}$ (-80) = 36,000 psf = 250 psi.

Caution: These curves are only valid for E_m/E_1 = 0.01 and other curves (not shown) must be used for other values of E_m/E_1. In general, as E_m/E_1 increases (stronger ground) the lining load decreases, and vice versa.

FIGURE 8

ON HARD ROCK AND HARD PROGRESS

T. R. Kuesel
Senior Vice President
Parsons, Brinckerhoff, Quade & Douglas, Inc.
New York, New York

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 seized on the only available text on the matter. Where Terzaghi tried to describe the configuation of a stable arch formed of natural rock, all these "designers" saw in his paper were 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" US 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. The aims became 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, and 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.

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 lin-
ing 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 de-
signers to question the basis for the concept. But now the ex-
cessive 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 con-
cepts, methods, and details that have worked satisfactorily on
past similar projects and under similar conditions. This by it-
self is healthy, but unfortunately the range of acceptable pre-
cedents 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 hydro-
electric power projects is deemed irrelevant.

Third is government practice in procurement of engi-
neering 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 se-
cure what seems to them to be a speculative reduction in con-
struction 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 compartment 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 became dim.

Finally, overriding all these constraints 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. involved 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 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 sup-
port 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 prin-
cipal 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 in-
> sert a relatively thin concrete roof arch
> in addition (to the rock bolts and shot-
> crete, 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 instrumen-
ted 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 contructors 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

Professor T. D. O'Rourke
School of Civil and Environmental Engineering
Cornell University, Ithaca, N.Y.

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, C01, pp. 51-63.

WP-F-139

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 the customary interpretation of contracts and engineering authority. In some instances, therefore, the application of a local practice may not be appropriate for other systems. On a broad basis, however, a comparative survey of construction paractice not only points to different technologies, but points out 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 cost 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 divided by the number of stations included in the cost. This
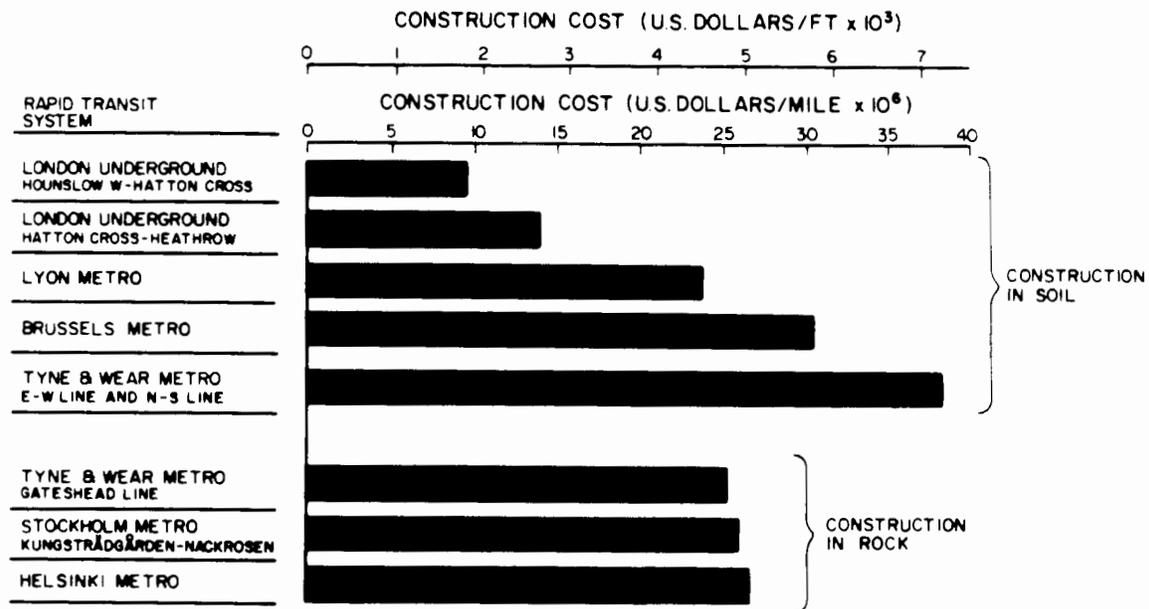
Figure 1. Unit Construction Costs for European Metros

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

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 differ-
ence 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 Under-
ground 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 per-
cent were constructed with bolted iron segments and 40 percent
were constructed with bolted concrete segments. In addition, ap-
proximately 60 percent of the tunnels were driven under com-
pressed 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 con-
struction 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

RAPID TRANSIT SYSTEMS

PERCENTAGE (%) OF CONSTRUCTION COST

LONDON UNDERGROUND
(HOUNSLOW W - HEATHROW)

LYON METRO

TYNE AND WEAR METRO
(N-S AND E-W LINES)

STOCKHOLM METRO
(KUNGSTRÄDGÅRDEN - NACKROSEN)

HELSINKI METRO

LEGEND : ████ — % COST OF STATIONS
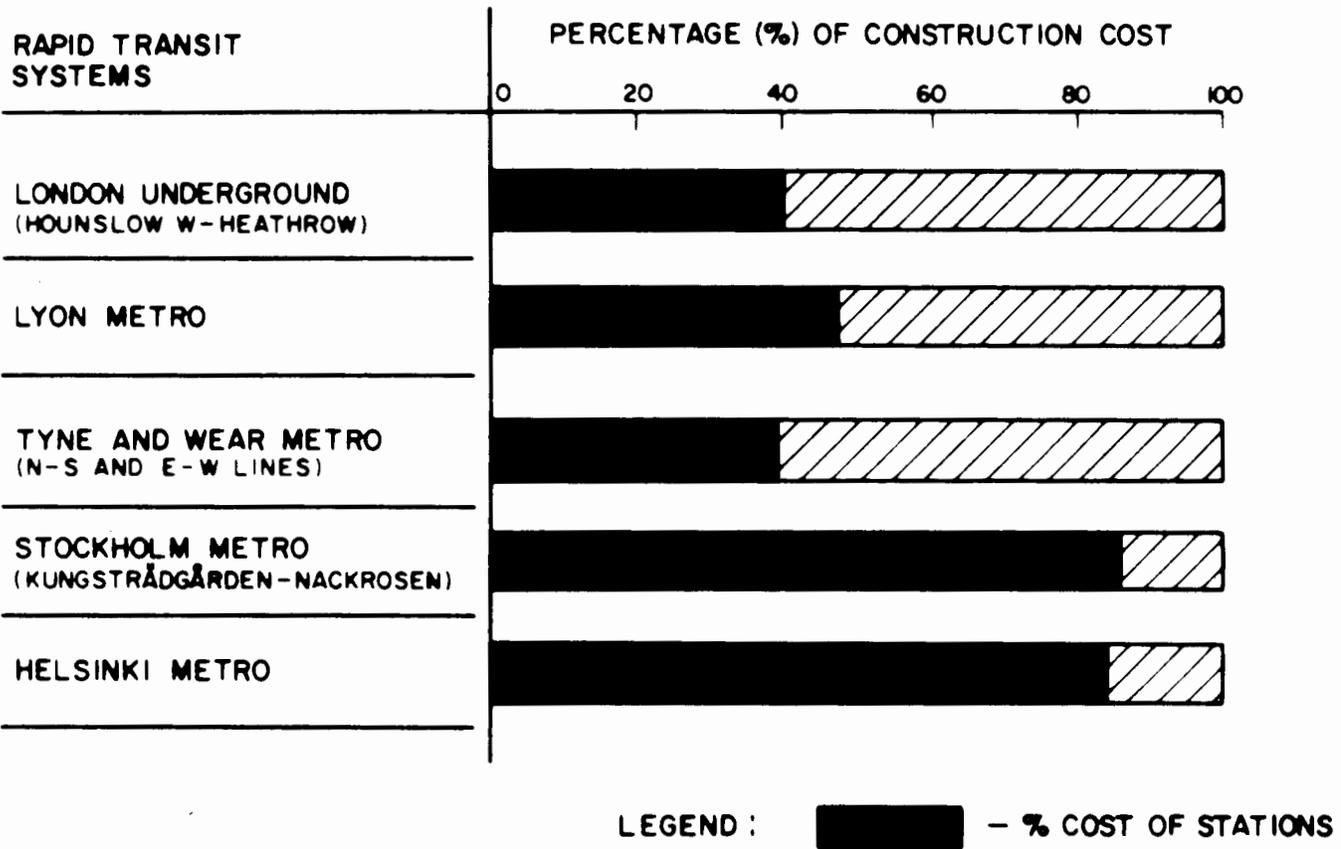
Figure 2. Relative Cost of Stations for European Metros

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 Scandanavian metros, station construction accounts for an extremely large proportion of the construction cost. The main reason for the apparent 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/foot and $590/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 round 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/foot. The rock tunnel for this system was advanced in approximate 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 ft. for the Stockholm Underground are the largest in Western Europe and, on a world-wide bases, 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, direct comparisons among the costs of metro systems will be misleading. For example, the number of stations included in the unit prices of the U.S. metros are

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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

CONSTRUCTION COST (U.S. DOLLARS/MILE x 10^6)
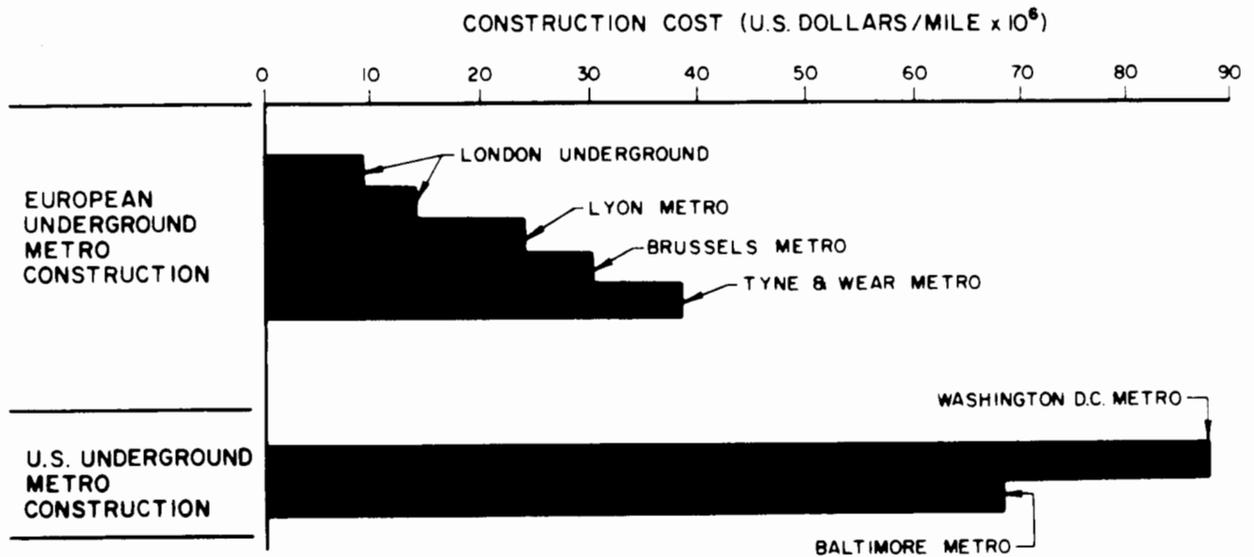
Figure 3.   Unit Construction Costs for United States and European Metros

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 di-

ameter 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 rom 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 the Atlanta and Washington, D.C. Metros are 183 meters

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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.

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 soldier 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 does provide a greater baseline of options with which to approach different tunneling problems.

4. Consolidation of Technical Judgements and Services: The complexity of large-scale urban construction points toward consolidation of services rather than greater

diversity. 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 arrangments 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

RESEARCH AND DEVELOPMENT PROGRAMS - AN OWNER'S VIEWPOINT

Eugene A. Kaiser
Manager of Design Review
Metropolitan Atlanta Rapid Transit Authority
Atlanta, Georgia

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 and it is because of this that future owners of underground public facilities want those facilities to prudently reflect most current design and construction concepts.

Unfortunately, the growth of the scope of experience 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 with innovative approaches quite often being discounted.

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 the 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 is in receipt of a design, which is both safe and well addresses functional requirements. 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, if the engineering firm were to be defi-

client in any aspect of its professional effort, this aspect would most probably be the economy of its design. The engineering firm is highly motivated to produce a design which is inherently safe and can be constructed with safety. It has both a liability and reputation 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 economy, its concern is less direct and further 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 those 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 construction practice in the field. There are exceptions to this, of course, but these instances are comprised of very dramatic cases which are rare. Consequently, 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.   Tests Used To Aid In The Development Of Design Methods
      Tests performed to corroborate, calibrate and/or deter-
      mine 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 procedure 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

    Test involving 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 im-
    ply that final design of support structures could be
    done as construction progresses 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 docu-
    ments would probably be unacceptable. Additionally, in
    the American contractual climate such a practice would
    amount to "construction by change order" with the owner
    being very vulnerable during negotations. 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 ex-
    cess of the savings.

    The contract documents will, generally, be drawn in such
    a manner so as to minimize change orders and make "bid
    unbalancing" of minimum value to the contractor. How-
    ever, MARTA will open mindedly entertain "value engi-
    neering proposals" by the contractor during construc-
    tion. It is felt that in this manner, under American
    contracting practices, the value received will be opti-
    mized 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 conceivable 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 that to the greatest extent possible that required expertise for its construction be readily available. However, in some instances there may be exceptions to this. Those would be cases where a selected expertise, even at a premium price, would be cost effective and 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 affect 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 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 and 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

Richard Gallagher
Manager & Chief Engineer
Rapid Transit Department
Southern California Rapid Transit District
Los Angeles, California

Because of great increases in rapid transit system costs over the past several years and some startling project cost estimate increase 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 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 the congressional appropriations haven't gone near 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 appropriation bill. However, we happen to concur that

such an effort is necessary and overdue. It apears that the rea-
son this hasn't happened sooner is (as has been the case with
several "owners") that 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 of 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 size and automation, a very appreciable percen-
tage can be attributed to a combination of bureaucratic regula-
tions, 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, it seems that our
most likely hopes for effecting economies will result from making
changes in risk-sharing and making method, material and equipment
trade-off analyses during the preliminary engineering process.
After all, it is in preliminary engineering where the key deci-
sions 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 at-
tention are just "things to have done with as quickly as
possible" for the designers and contractors.

The owner's primary concerns, in addition to the pro-
blems involved in getting the job approved by the community, get-
ting it funded and then meeting all state and federal regulations

and getting fast decisions from UMTA, are its system's:

> 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 in addition to getting the job and 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 due to a combination of the owner's grandiose ideas about stations and train consist without full appreciation of the effect on costs. Also, a factor is the, not uncommon, tendency on the part of the designer to think that the bigger the system and the greater degree and more complicated the automation, the better-coupled with his strong reluctance to take risks (which is understandable in this day and age). It seems that it is too much to expect designers to operate on the basis that "small and simple is beautiful" when owners have a hard time with that approach.

From the operations standpoint, however, the many and varied sub-systems are where the trouble lurks. 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, not having 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. 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!

OWNER'S MONOGRAPH

Don Birrer
City Engineer
San Francisco, California

(to be prepared)

SHOTCRETE AND ROCK BOLTS: LABOR'S VIEW

Audrain E. Weatherl
International Representative
Laborer's International Union of North America, AFL-CIO-CLC
Burlingame, California

I have been somewhat disturbed at the reported under-
standing that tunnel contractors and contracting agencies (own-
ers) believe that labor wouldn't or, in most instances didn't
like to, work with rock bolts or shotcrete in tunnels. That sim-
ply is not true. Our concern is with the methods and procedures
of usage and the adequacy of equipment and 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, that 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.

I firmly believe that if rock bolts are going to be the
total support, (the diameter of the bolt depending on the size of
the tunnel, the length of the bolts again depending on the area
to be supported,) that pattern bolting must be used. Not 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 adhered to 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, they are
left out entirely, or located out of place. We demand that the

correct bolt goes 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 put in a regular pattern; rather, they are put where some supervisor said they should go, not taking time to get them where they must be, and a lot of the time simply left out.

We know that most kinds of rock can be bolted for support, _if_ the bolts are long enough and thick enough in the pattern, and someone is there to be sure they all go where they are supposed to go.

I was told in New South Wales, Australia, about twenty years ago, by a Snowy Mountain Authority engineer 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, that 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 because we have become accustomed to it. 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 in a short distance, the smallest only 8' diameter, one 14' diameter and the other 23' diameter. I was called be-

cause the miners, members of the Laborers' International Union, had refused to rock bolt in the 14' or the 23' tunnels, saying that in their opinion it was simply too dangerous. I will have to admit that also was my thought. This is what they call basalt rock, the blockiest and most fractured rock that I have seen anywhere. In the small tunnel (8') they had constructed a shield of heavy steel at each end and then laced it with 2" pipe and took this into the tunnel with a backhoe boom to set it up, for the miners to drill the rock bolt holes from underneath this contraption. The 2" pipe was far enough apart to let large rocks fall on the drillers. The rock was so jagged it was impossible to drill effectively for pattern bolting.

The rock being what it was and the way they had to do it, 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 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' 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 muck-

ing 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 which was in the area 10-20 feet from the face and two more inches on the round, 20-30 feet from the face, making 6" of shotcrete. 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. A tunnel this size and condition worked correctly. 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. What we require is a reasonably safe place to work. We know that both are here to stay, and 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.

LABOR VIEWPOINT

Dale Marr
Operating Engineer

(to be prepared)

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.

TUNNELS

PIONEER
RESERVOIR
SITE

INTERSTATE 5

INTERSTATE 80

BROADWAY

JEDEDIAH SMITH SCHOOL

RIVER OAKS
HOUSING PROJECT

McCLATCHY WAY

EL SIXTO

SOUTHERN PACIFIC

SACRAMENTO CITY LIMITS
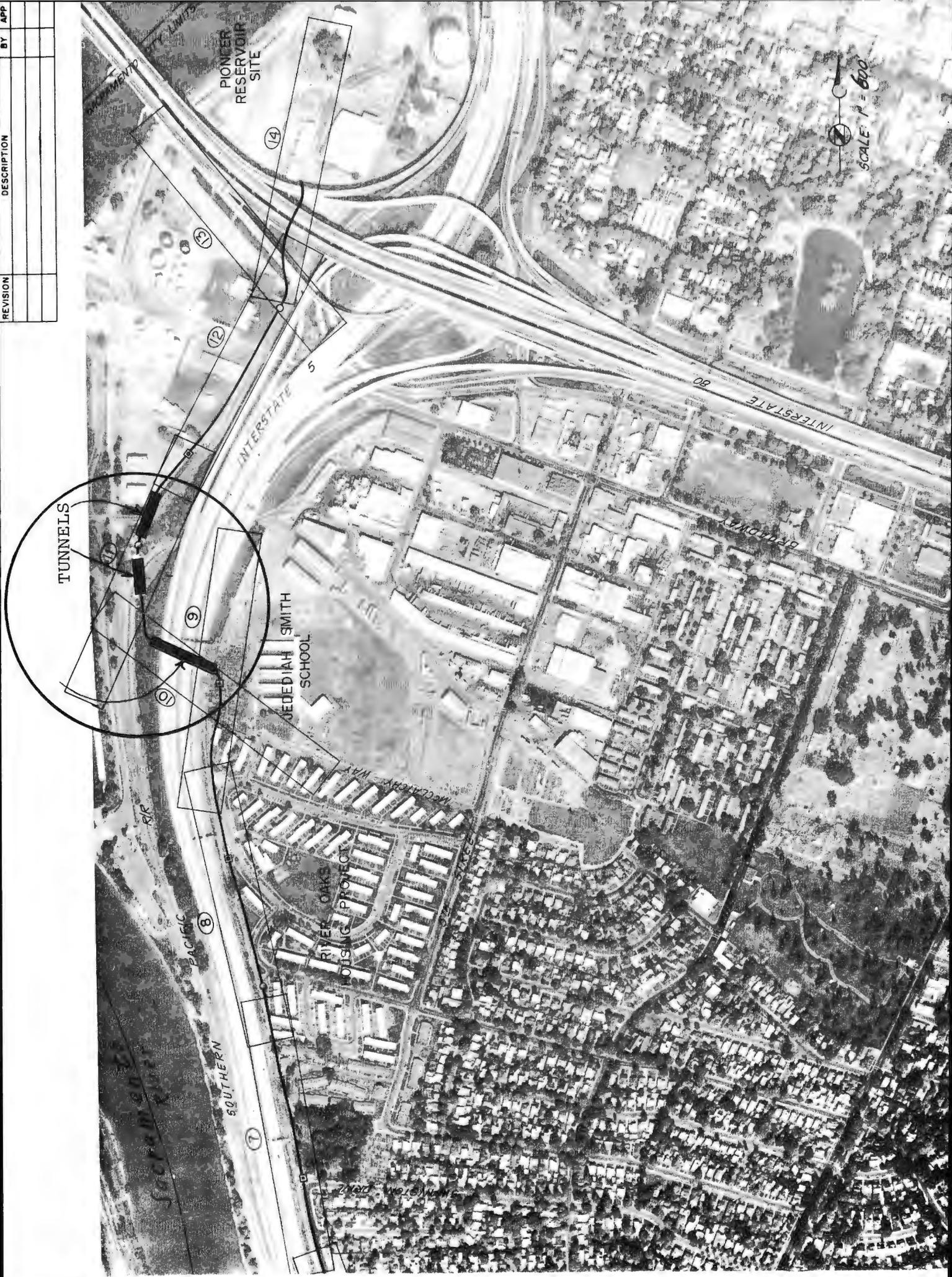
SCALE 1" = 600'

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The groundwater flow was artesian in nature.   Because
of the coarser material underneath, the flow tended to be verti-
cal instead of horizontal.

There had been two sewer construction projects in the
area in the past several years and both ran into groundwater con-
trol difficulties.   The first project just fought the groundwater
and the pipe ended up in washboarded (up and down) condition be-
cause of the heaving invert from artesian groundwater.   The
second project relied on the experience of the first project and
the owner specified a well point groundwater control method.
This method did not work either 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 pro-
jects 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 the ground-
water 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 free-
way.

The groundwater was successfully controlled with deep
wells 25 meters deep to tap into the coarser stratas.   The con-
struction was planned from May to October during the low flow of
the Sacramento River.   The deep wells drew down the recharged
groundwater table and was able to maintain the groundwater one
meter below tunnel invert because the river was not recharging

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the groundwater table.   The pumping was accomplished at just a
few locations but many observation wells were installed to moni-
tor the groundwater levels.   The exact level was maintained by
slowly lowering the pump in the deep well and 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 which is approximately a 12 lane freeway immediately ad-
jacent to Interstate 80 Freeway in Sacramento.

The conventional liner plate style of construction ori-
ginally 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 adja-
cent 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.

Based upon our experience we did not feel that the
ground would stand long enough for us to be able to fill the an-
nular void with grout and we could envision 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 be-
tween the flanges of the ribs (see figure 2).   This is a strong
support system for the shield to push against in order to propell
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 the

3 x 8(R) D.F. Timber
Lagging Solid
Around Perimeter

4 Piece Ribs 4H13 13'-6" O.D.
@ 4'-0" Centers

Expand Ribs
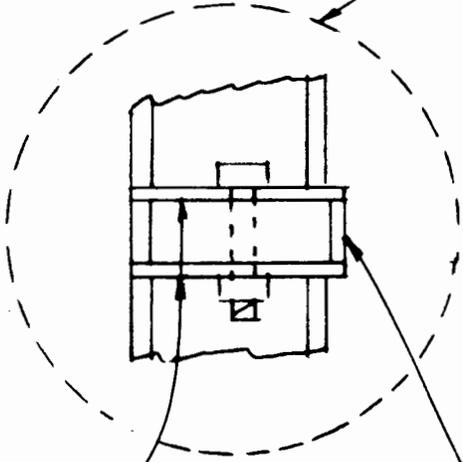
Butt Plates

TYPICAL SECTION
Symmetrical @ ₵

Steel Pipe Spacers, cut to length

FIGURE 2
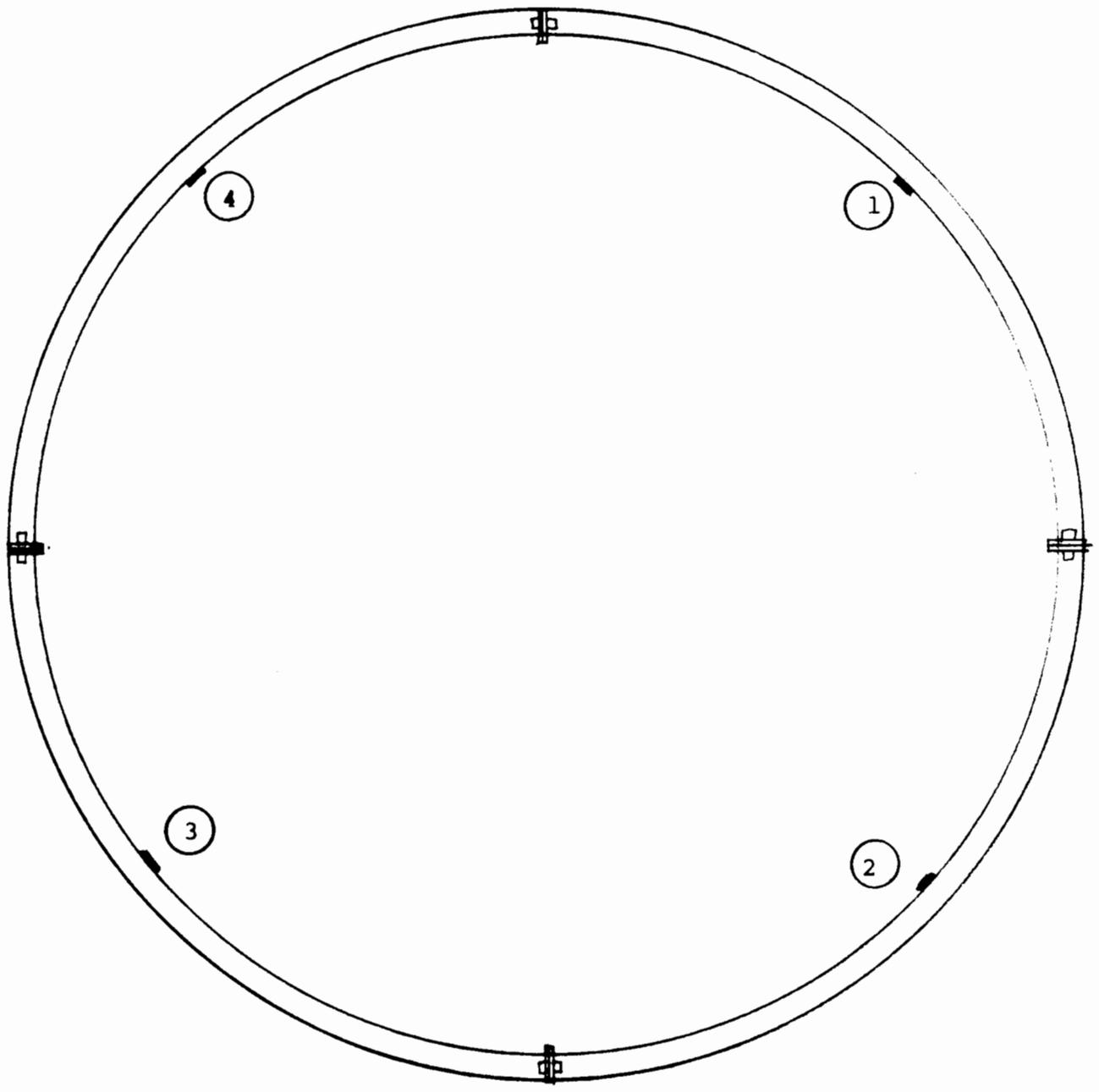
STRAIN GAGE LOCATIONS

FIGURE 3

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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 in-
stallation 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 expan-
ding the rib, then daily for one week, then weekly for one month.

Strain Gage Results

Of the 20 strain gages installed, 19 worked and there
was good correlation from day to day. Of the 19 gages, 18 indi-
cated 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 7 days.

Conclusion

This tunnelling system worked. The settlements
measured on the freeway above were less than 3 centimeters and
the loads experienced on the tunnel supports were a small percen-
tage of the loads normally designed for. The design criteria
used was AASHO "Design Specifications for Tunnel Liner Plates"
which takes into account the ratio of fill to tunnel diameter and
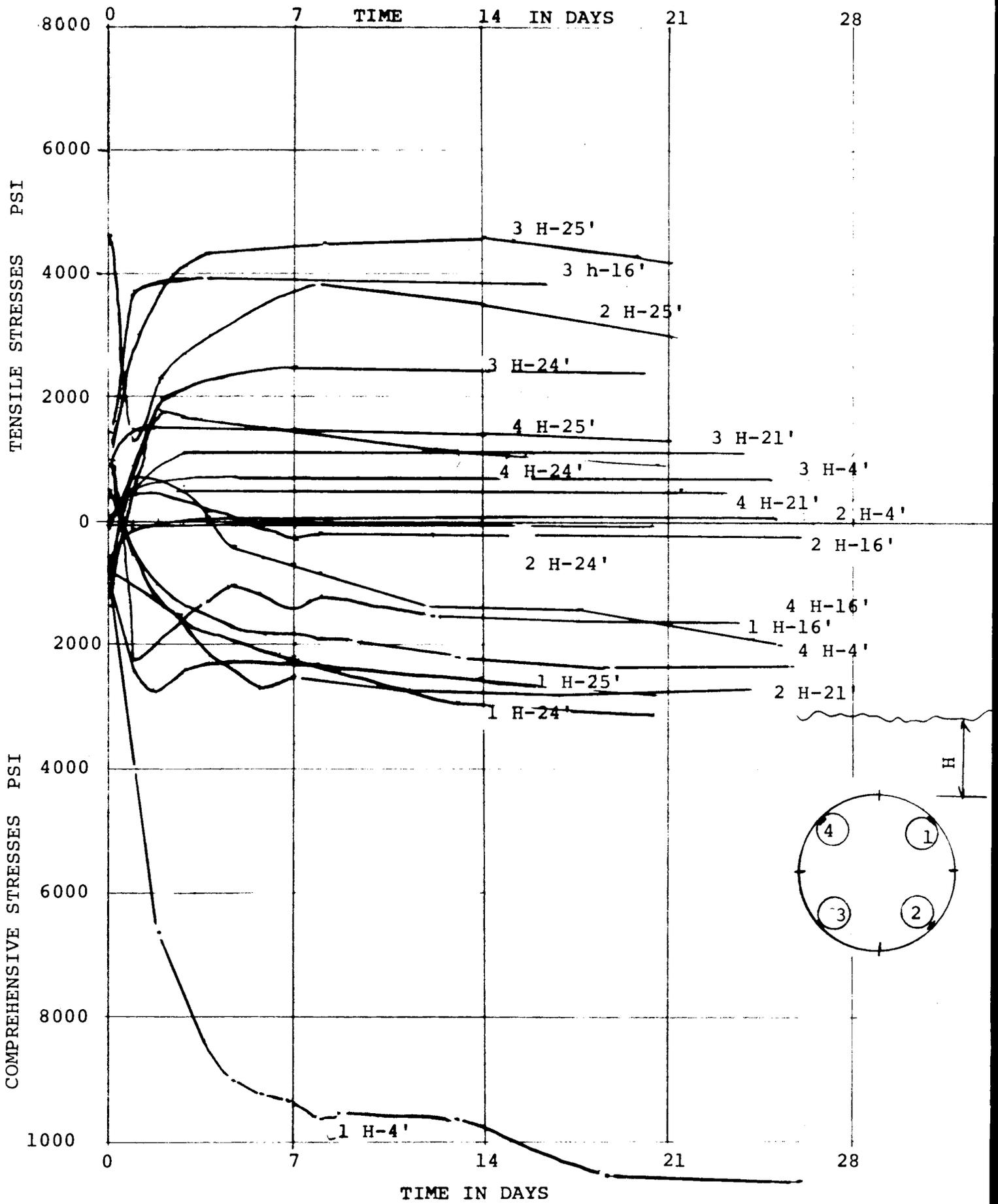
FIGURE 4

PLOT OF STRESSES IN STEEL RIB VS. TIME

Contractor's View: Innovative Contractors Can Mean Light at the
Beginning of the Tunnel - A Case Study
Alf Burtleson
Page 9

type of soil and then using Marston's formula Pd=Cd WD. We anti-
cipated getting loads which would produce steel stresses of up to
24,000 psi but the actual strsses were only about 20 percent of
the design stresses.

The shield method of construction is also safe for the
tunnel workers as they are installing the supports back from the
face under the protection of the shield.

The tunnel method described is for temporary support
and 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.

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 tun-
nel contractor is stuck following the specs in recipe fashion
with no deviation allowed, but 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 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 for aid in strain gage selection.
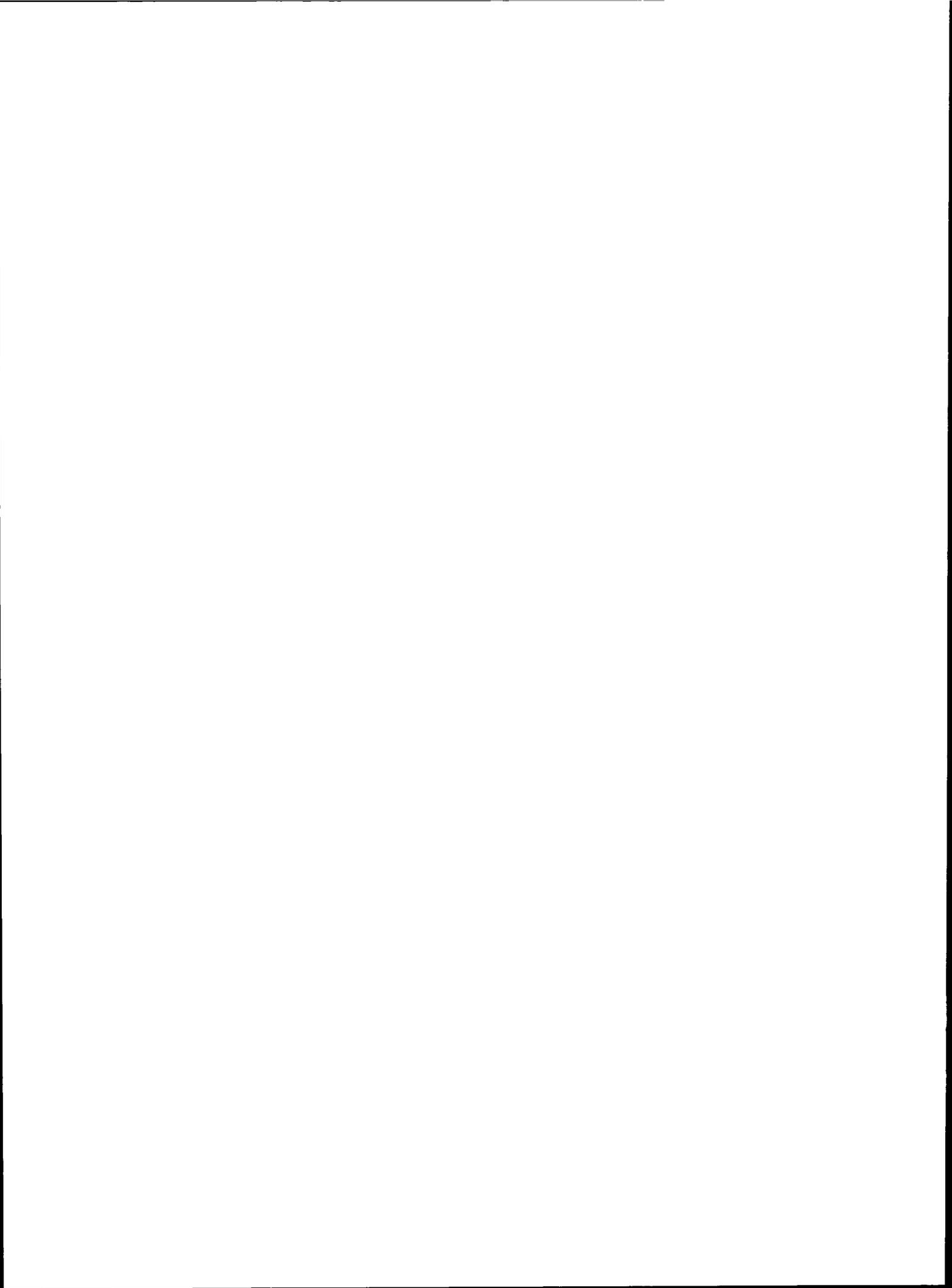
TUNNEL PRACTICE IN THE UNITED STATES:
A CONTRACTOR'S VIEWPOINT

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 which seems to be most prevalent is time; time to prepare the bid and time to construct the works. The length of time necessary to prepare a bid varies as 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 make significant changes in the scope of the work, he would be able to put together a realistic bid. If there are sophisticated or unique items to be supplied or installed which are likely to be difficult to obtain, additional time may be in order or other methods of handling such items in the bid schedule should be provided. 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 which in essence adds time to the bid period.

Site showings for tunnel work often reveals 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 result of this situation has been a decision to not 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, such 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 with a calculated contingency. If the owner accepts the risk and makes provision to compensate the contractor, he will very likely get it at cost.

Items 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

and the owner can then evaluate the bids on a comparitive ba-
sis. "Lump Sum" items require that the contractor absorb over-
runs no matter what the cause. He, therefore, must apply an un-
calculable contingency, otherwise known as a guess on the high
side.

Contractors are accused of not being innovative and al-
so 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 those that fit into the allowable time
frame. The owner must also be prepared to accept some of the re-
sponsibility for deviations from the tried and proven methods.
Schemes like the "New Austrian Tunnel Method" offer some interes-
ting ideas which are improvements over customary practice in the
United States, but not one which a contractor can afford to ex-
periment with on a fixed price bid. All too often in attempting
to utilize a new scheme such as this where admittedly everyone is
in the learning stage, the owner sees fit to inspect and test ev-
ery item to the point of distraction mainly for the purpose of
learning just what to expect. I have seen too many delays resul-
ting from vigorous application of the exact letter of the speci-
fication to want to attempt to use shotcrete as the primary means
of support. The short time schedule usual for tunnel construc-
tion often forces the contractor into adopting the customary
drill/shoot/set-steel approach as a matter of expediency. From
experience we know that a crew can be trained to follow this rou-
tine and soon achieve a predictable rate of progress. The exis-
tence 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 on-time results. 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 the 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 contactor 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 redundent effort had to be financed from the contractors indirect costs.

When unusual ground conditions are anticipated, the solution is to be very explicit in the specifications and be prepared to accept state of the art results, e.g. in the case of the long shell anchors failing in the schist, accept this rock characteristic and work within the parameters present in that situation, perhaps a larger number but smaller bolts could develop the same load capacity.

In the use of gunite and shotcrete, I have experienced the invention of the wheel on a 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 and the inevitable conflict led to many disagreements. This happens to be a human nature thing that no specification writer can avoid. 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 this is no where near the time and materials costs during the trial and error period. Adjustments in design which do not interfere with production are no problem but design 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 which then requires that a heading be supplied with two (or more) specially designed pieces of equipment. Often these units will not pass in the tunnel and must be moved out, or to an enlarged passing area so that 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 designated well in advance, no particular time

will be lost; however, if a decision is to be made on a day-by-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 adhered to. 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 must use the "best guess" approach and have entered into a contract in good faith. Such an open-minded approach 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 and the base estimate should be prepared by knowledgeable and experienced people. It is one thing to prepare an estimate if the estimator is reasonably sure he will not 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

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contractor enthusiasm.   It occurs to me that any one estimate could as easily be incorrect or unrealistic as any other.   If all bids are considerably in excess of the engineers' 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.   Perhaps, a costly re-design and rebid exercise could be avoided.

These few brief remarks are not intended to revise the tunnel bidding/construction system, but only convey some ideas 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.
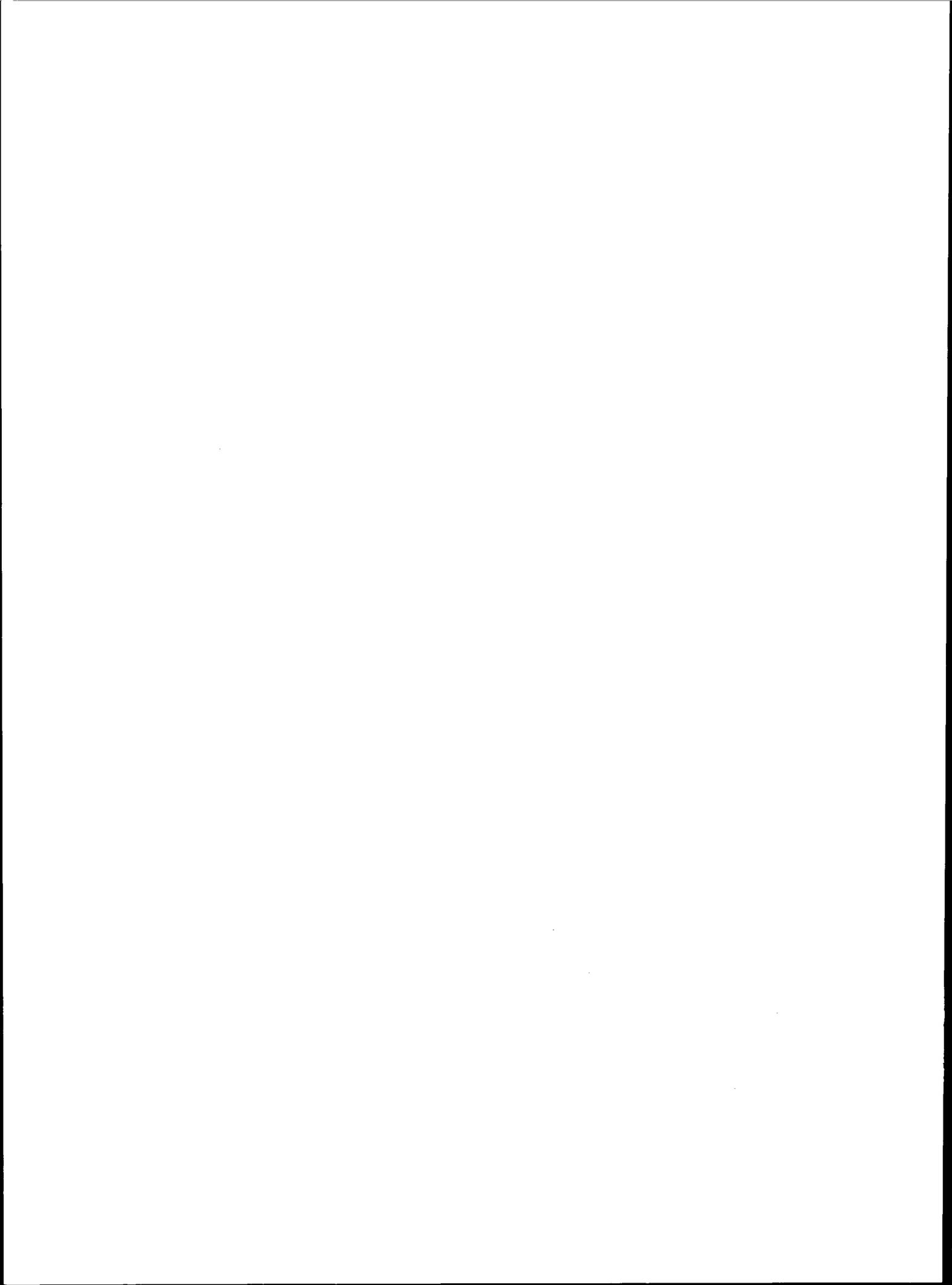
CONTRACTOR'S VIEW

Don Gillis
Horn/Fruin - Colnon, J.V.
Atlanta, Georgia

(to be prepared)
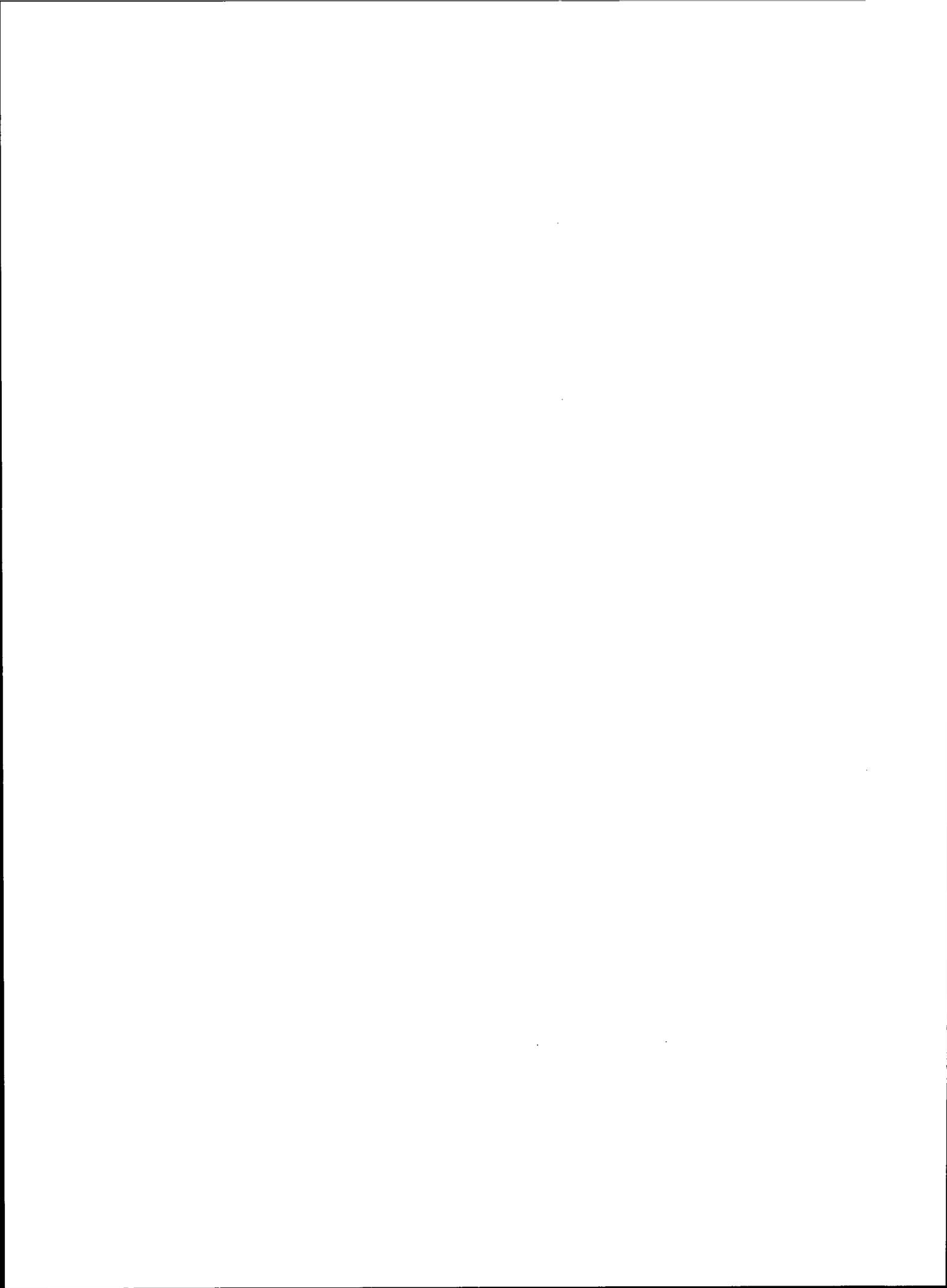
TUNNELING IN THE UNITED STATES:
A CONTRACTOR'S VIEWPOINT

by

Terence G. McCusker, Tunnel Consultant
Wayne, New Jersey

The remarks which follow are based primarily on recent contracting experience in order to reflect as well 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 technically adversary relationship can easily become adversary 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 by the resultant hostility.

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 the owner to take the risk he has unwittingly accepted than taking the responsibility himself for proposing a change and being charged with the consequences if the proposal does not work out well. In summary, the mutual suspicion leads to a considerable reduction in the flexibility of both parties in dealing with job problems and may not permit 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 attitude prevailing in contracting in the United Kingdom where the consulting engineer, although he is the owner's representative, acts in most cases as the arbitrator of disputes between the owner and contractor. Bidding is not open to all, 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 the problems which

arise are engineering problems, 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 in-
vited to bid again if he proves intransigent or if his probity
proves 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 similarly
substantially free of litigation. The contactor 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 from
those who have left a more active life in the contracting indus-
try, there may be an understanding and helpful approach to mutual
problems. However, many inspectors have never worked in contrac-
ting and many of those who have, have left because they were un-
successful. Neither of these backgrounds will dispose the in-
spection staff to be cooperative. It is more importantly the
case that many of the senior staff and especially resident engi-
neers have spent their entire careers as designers or owner's re-
presentatives. 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 prob-
lem becomes a major time-waster. It is a very real grievance the
contractor has 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 power to enter into an equal partnership of authority and decision.

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 between 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 hardened 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 colorable 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 and who received their compensation equally from the owner and the contractor. The arbitrators should be engineers familiar with the law, rather than lawyers familiar with engineering, should have wide experience, mature judgment and be widely respected.

It would be possible to set forth rational timetables for submittal, rebuttal and judgment to the end that jobs would 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 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 specified in detail preserving conservative methods 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!

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 using precast concrete segments - a technology which has almost entirey displaced metallic 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 and still less when combined with inhibition of invention.   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 bidder being 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

There is currently such a lack of consistency in lining design methods as to make it appear sometimes 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. This goes in part to the previous discussion under Designer's Limitations of an industry which cannot produce a code of practice which will ensure that the owner gets a fair shake while giving 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 invited bids from selected contractors, often accompanied by qualifications and then a round of negotiations to secure the best bargain.

The Government acts in much the same way to select 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 borne 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 a further possibility, more difficult to pro-
tect from corruption, perhaps.   This is the acceptance of quali-
fied bids, with post-bid negotiation to resolve the qualifica-
tions.   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 -
lack of individual incentive - insurance instead of design - loss
of benefit to the good performer both currently and in his exper-
ience rating.

All large owners are essentially self-insured anyway,
as are large contracting organizations.   The only worthwhile ar-
gument for wrap-up is that it deals effectively wih the subroga-
tion problem.   But, why should workmen's compensation be inclu-
ded?   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 busi-
ness.

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 pro-
tects him. It is a common fallacy that 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. His inquiry into
the technical or managerial competence of the company is inconse-
quential.

10. Labor Skills

Tunnel labor has typically been recruited from the in-
experienced 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 pro-
grams for tunnel laborers instead of pursuing the haphazard
course we have up to now?

This question is being addressed currently by certain
western locals 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
modules - 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 as to 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 and 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 any more than a drop in the ocean, but as the weight of opinion begins to lean in a particular direction, the greater the chance that it will result in effective action. The hope is clearly that I have influenced the opinion of others as a step towards the goal of problem-free construction.

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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, equip-
ment, 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,
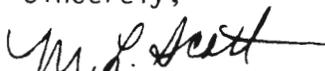
M. L. Scott
State Director

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

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.

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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 area 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.

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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, 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 responsible where a contractor encountered boulders in the area of a reservoir filled in many years before where 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)).

When, in the early days of the City of New York a canal existed in a certain area which now consists of apparently solid land, the designer should have known 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 reasponsible 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 as to

subsurface or latent conditions by reason of the time re-
quired and the cost required of examining into those sources
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 con-
tractor 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 avoided effective-
ly then there is an obligation on someone to formulate a sub-
surface exploration program and to adequately study available
sources of information as to subsurface conditions to form a
basis for proper preparation of plans and specifications in
such substantial detail as can form the basis of an intelli-
gent and knowledgeable bid. Ovbiously the only logical per-
son to make such study for preparation of plans and specifi-
cations is the owner through his engineer. He can take all
the time needed for an adequate study. Years if necessary.
The owner - not the contractor - in estimating his probable
costs should include a substantial percentage to cover the
risks of uncertainties. This should be reflected in the ap-
propriation. 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 done 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 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. The more money spent on engineering at the 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, he should be required to disclose not only what he knows but what he "should have known." By "should have known" is meant that the engineer should be obligated to examine into maps,

records of previous construction historical data such as the
previous existence of canals, streams, wells, etc. previously
backfilled and all similar sources of available informa-
tion. This places on the engineer the burden of making a
diligent effort to ascertain subsurface conditions and im-
poses liability on the owner if the engineer fails to do
so. Stated otherwise, there should be a duty on the part of
the owner, through his engineer, to make a reasonable and
thorough effort to discover the actual subsurface condi-
tions. The knowledge thus obtained should be disclosed in
detail, including a reference to the sources of information
stated by him and the nature of the exploration work done by
him.

The rule requiring the owner to disclose what he
knows or should have known is the practical rule since the
owner alone has the means of access and the adequate and ne-
cessary time to explore, analyze and study available sources
of information as to subsurface conditions. It is suggested,
therefore, that the risk as to subsurface conditions, to the
extent they were ascertainable by reasonable exploration and
study of existing sources of information, be on the owner;
that the owner be obligated to disclose all such information
to prospective bidders. If contractors were actually to make
their own explorations, their bid prices would have to in-
clude this cost, resulting in a duplication of expense to the
owner, without a corresponding benefit to the owner.

It is suggested that, in addition, the owner list
his sources of information; prospective contractors who have
knowledge of conditions or information indicating conditions
differing from that disclosed by the owner have the moral ob-
ligation to disclose that fact. While it may not be feasible
to enforce such obligation on the part of prospective con-

tractors, certainly the differences within his knowledge, when encountered, would not be a basis for the recovery of damages.

It is also suggested that the owner disclose the written reports of experts employed by him, including the conclusions they have drawn from disclosed facts and their 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 an expert opinion and not a representation of conditions. The prospective contractor may accept or reject them; he may form his own conclusions based upon his own expertise or those whom he employs. The owner should not be bound, as a representation, by the opinions 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 the plans and specifications as to be free from errors or omissions, based upon the disclosed information.

By the elimination of disclaimers and disclosure of subsurface information, owners would be encouraged to engage in well conceived and executed underground investigations, sufficient both for design and construction purposes. This will benefit owners by reducing the margin of uncertainty, enabling better design and planning, leading to more economical construction. It is believed that the added time and expense of a thorough site investigation, in the long run, is much less costly than the lost time and extra expenses incurred during construction in overcoming the consequences of inadequate underground information. The better the underground information, the less need there will be to disclaim responsibility for it, 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 by reason of explorations and study, the owner would pay for the cost of performance as required by those true conditions since bidders would base their estimates thereon. It is logical 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 logical, therefore, that the contract contain a "changed conditions" clause (also known as a "differing site and subsurface conditions" clause) and place the burden where it would have been if the true conditions were originally known - on the owner.

What is meant by the term "changed conditions"? This is commonly defined as unknown conditions not normally expected to be encountered in the area involved, i.e., conditions which the disclosed information would not lead one to expect to encounter. Under a contact with a"changed conditions" clause such variance from the disclosed conditions would entitle the contractor to payment of the resulting additional costs.

A differentiation should, however, be made between factual data and opinion data, 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 warrant the accuracy of opinion data, where equally competent professionals may draw different interpretations. Opinion data 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.

Where the suggested "changed conditions" clause is employed it 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 is included 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 the existence of more contractors willing and financially able to engage in such work, and (2) by the elimination of 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, the ultimate result will be a savings to both parties to the contract.

SPECIFICATIONS

Grant Larson
TUDOR ENGINEERING COMPANY
San Francisco, California

(to be prepared)
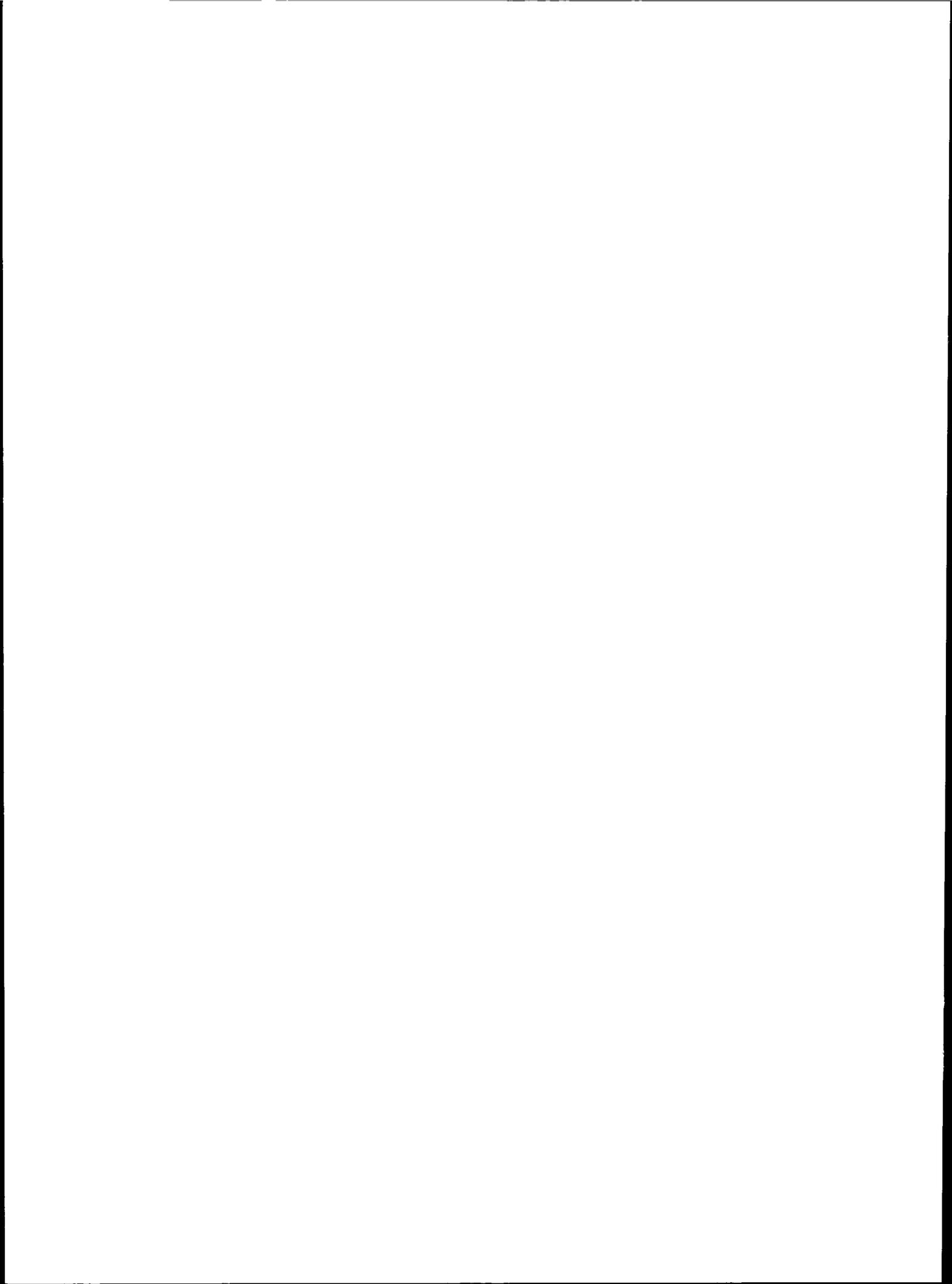
RISK MANAGEMENT & INSURANCE IN TUNNELING

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 possiblity of suffering harm or loss.

Our modern business society 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:

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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.

Where Risk Management produces the best results, is where all related functions have been placed under a qualified Risk Manager reporting to the Chief Executive Officer, and the Board of Directors have approved and published a "Risk Management Policy"; finally, 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 "Assump-

tion 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, and requires the coordinated efforts of all facets within an entity to maintain a continual indentification 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 by the line operation, and usually becomes ineffectual. This is the same situation as 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 mangement 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 or large firms the insurance is pretty much self rated as to experience incurred. The basic fundamental of insurance is the spread of risk over homogenous units using the law of large numbers. Whenever this is out of balance, such as catastrophic losses exceeding the expected or the current "social" change in tort law broadening the policy coverages not anticipated in the rating structure, then large or massive rate increases may be anticipated.

What is liability? "A state of 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" is one of the most onerous forms of transferring risk ever devised. As an example, California passed legislation prohibiting engineers from making Contractors assume the engineers professional liability through "hold harmless" 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 _all_ facts known to owners and engineers, _must be given_ (not "made available") to the Contractor, thus reducing the unknowns to a minimum. Owners must spend more money either researching projects or allow more money on change orders for inadequate research and design. The need to create a

close working non-adversary relationship between owners/engineers
and Contractors/Subcontractors must take place to reduce the cost
of construction.   The risk bearing must be taken by those whose
risk it is - not passing it on to others.   This only results in
the costs coming full circle back to the general public, the ul-
timate consumer.

The problem of liability, today, and its attendant high
costs, passed back to the consumer lies within the very heart of
our changing Tort laws.   The technical achievements of our
society creates a fertile field for the changing social mores to
make someone pay. In 1975, the Ohio Supreme Court refused to al-
low Punitive damages to a plaintiff suing the City of Cleve-
land.   A punitive damage -ward would ultimately be imposed on
taxpayers forcing those seeking services to bear the burden,
(Ranells vs. City of Cleveland 321 NE 2nd [Ohio, 1975]).   This
limiting factor is a step in the right direction towards tort re-
forms.   Another example is the pilot legislation being introduced
and passed by many state legislatures limiting Completed Opera-
tions Products Liability awards in time and amounts.

Because of the broadened "social" change in tort law,
there has become a closer relationship and crossover of these
risks which has created some untenable problems of insurance ca-
pacities throughout the world insurance market thus, causing in-
voluntary self insurance which in turn has caused management to
assume "Pure Risk" as a "Business Risk" speculating with corpo-
rate assets.   This is forcefully shown by the "social" change in
tort law as respects products liability over the past five years.

One of the most acute problems facing the owner-en-
gineer-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 "social"
adaptations of tort law which says "somebody has to pay" and the

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. Owners assuming their risks, engineers assuming their risks and contractors assuming their risks is another prime step.

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 owner and contractors. There have been several expensive claims in recent years from this class of business; which include 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 considering alternatives and trade-offs.

There is no panacea 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.

WRAP-UP INSURANCE

This controversial topic has been discussed almost to infinity. However, in the face of open or tight insurance markets it is the only viable solution to large projects. It is here to stay, and some contractors for many years have used this method of project insurance for their own account when owners had not put in such plans. Most significantly the US 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 a uniform and presumably high level of insurance expertise applied to establishment and execution of a program for management and control of safety and risks for the overall project.

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 one set of standards must be complied with instead of several, as

might be so if numerous insurers were involved. This should eliminate delay in complying with recommendations and provide for regular systemized attention.

3. It places responsibility for handling and payment of claims in 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. Ths 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, which will provide better defense against claims of damage and more uniform and prompt disposition of claims that damage resulted 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 Contractors' Viewpoint

1. Dividends, discounts or experience credits inure to the buyer, while loss of the premium volume which is eliminated by Wrap-Up adversely affects the contractors' position with their own regular insurers, particulartly 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 in-

(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 heading referred to above are:

(1) Safety in the excavation

- 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 thorouhfare
- Warning of damage to utility lines

MODERN INSTRUMENTATION FOR TUNNELS

Ian Weir-Jones, Ph.D., P. Eng.
Weir-Jones Engineering Consultants Ltd.
Vancouver, British Columbia
Canada

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:

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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.

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(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 reliable and capable of providing useful data for the duration of the project.

(2) The instruments and all ancilliary 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 as 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 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:

(1) Monitoring equipment installed from within the excavation

(2) Monitoring equipment installed from outside.

Each group has both advantages and disadvantages which have very significant influences upon the feasibility and desirability of employing particular equipment in a specific location and these can be summarized as follows:

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 producton 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.

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 can 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 referred to above, the instrumentation systems for relatively shallow subway excavation in urban environments typically combine elements of both surface and subsurface installed systems. However, deep subway tunnels tend to place only minimal reliance on surface monitoring systems. The same would apply for tunnels driven in mountainous regions, or underwater, where it would be clearly 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 differ-
ent types are available for monitoring the various significant
parameters associated with the construction of tunnels. The pro-
cedures or categories of equipment which are currently applicable
are:

(1) Convergence measurements within the tunnel to moni-
tor 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 dis-
placements.

(3) Precise levelling equipment, standard surveying
procedures or EDM equipment to monitor the development of subsid-
ence 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.

(8) Strain and load measurements made within the sup-
port 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

TABLE 1

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

FEATURES

- Best installed by production personnel
- Best installed by superviosry 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

(A matrix of dot markers cross-references each SYSTEM row against the FEATURES columns.)

diametral end points. However, even in the most simple applica-
tion, 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 ex-
tensometers) 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 tempera-
tures which prevail underground, the convergence measuring de-
vices 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 mesurements, and a tape or wire which is secured at the
other end of the bay by a single point mounting. With a well de-
signed instrument, the system repeatability and accuracy should
be better than $\pm$ .002 inches ($\pm$ .05mm), and the range should be
about 2 inches (50 mm), before resetting is necessary.

The number of installations and the frequency of taking
mesurements with convergence monitoring equipment will depend es-
sentially upon the conditions in the specific tunnel under inves-
tigation. In competent, self supporting ground, convergence sta-
tions 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

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. Conse-
quently, the improvement of the programs must be consi-
dered as a general benefit to design without direct ef-
fects in the field.

2.   Materials and Materials Applications Tests
     Test involving installation and performance of materials
     and hardware are done with the intention of either cor-
     roborating existing design information or developing de-
     sign information, which has previously not been avail-
     able. 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 docu-
     ments 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 docu-
     ments.

     It is MARTA's intention to utilize the research infor-
mation 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 im-
    ply that final design of support structures could be
    done as construction progresses 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 docu-
    ments would probably be unacceptable. Additionally, in
    the American contractual climate such a practice would
    amount to "construction by change order" with the owner
    being very vulnerable during negotations. 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 ex-
    cess of the savings.

    The contract documents will, generally, be drawn in such
    a manner so as to minimize change orders and make "bid
    unbalancing" of minimum value to the contractor. How-
    ever, MARTA will open mindedly entertain "value engi-
    neering proposals" by the contractor during construc-
    tion. It is felt that in this manner, under American
    contracting practices, the value received will be opti-
    mized 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

diameters away, at which time the reading frequency would de-
crease.

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 in-
stalled from the tunnel as soon as the collar location has been
exposed.   Bore hole extensometers installed from inside the tun-
nel 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, ro-
bust, 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 ex-
tensometers take longer to install, require a specialized instal-
lation crew, and 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 of be-
ing capable of being installed 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 man-
datory 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, and 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 read out 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 in-experienced personnel attempting to install equipment which the manufacturers claim to be a standard production unit, but which subsequently turns out the 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 prior-ities of the contractor, the field supervisor and the design engineer,is primarily one of a lack of comprehension or apprecia-tion. 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 rea-listic and can not impinge too seriously upon the production work. If it does, it runs the risk of being ignored as super-fluous, or tacitly sabotaged. In either case the result is the absence of data which could have been beneficial to all the par-ties involved. Similarly, if the field supervisory personnel can be made to understand that the functions of the intrumentation systems in the tunnel are to aid in maintaining safe working con-ditions, to improve the design, and to provide the owner or de-signer 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 for 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 a rapid decrease in the volume and/or quality of data. The reason for this is often that, in a large orgaization with many responsibilities, the task of data collection is ultimately assigned to quite junior technical personnel who have neither the knowledge nor the 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.

The author wishes to express his thanks to the individuals and organization who supported the Atlanta Research Chamber contract of which this work formed a part. In particular the continuous support of Gilbert L. Butler, Project Manager for the Urban Mass Transportation Administration (UMTA), is gratefully acknowledged.
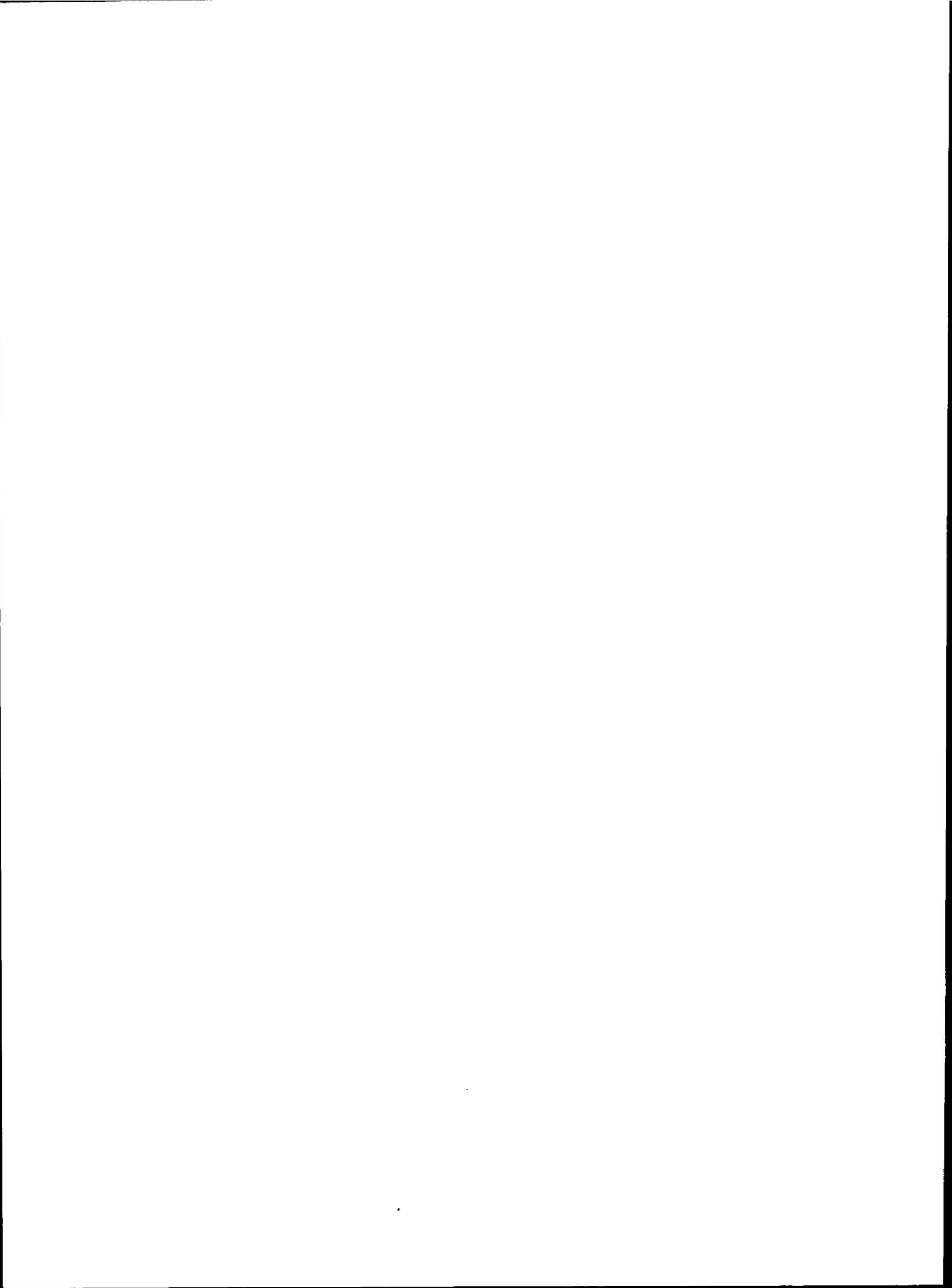
MODERN BLASTING IN AN URBAN SETTING

by

Lewis J. Oriard, President

Lewis L. Oriard, Inc.

Huntington Beach, California

Introduction

The Atlanta Research Chamber was appended to the construction of the Peachtree Center Station 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 con-

tracts 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 penetraion rates much higher than those for ordinary pneumatic drills, but require much more care in the operation and maintenance. These drills must be kept in very clean condition. Recently, the manufacturers of pneuma-

tic 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. Unfortunatley, 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 most "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 nitro-glycerine 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 rela-

tively 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 in 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 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, - that the initiation of the main charge is accomplished with 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 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 per-
imeter is incapable of shattering to the perimeter and
beyond.

For the Atlanta Research Chamber project, a pilot tunnel
had been excavated through the alignment of the research adit.
No controlled blasting methods were used for the pilot tunnel.
It was done entirely at the Contractor's option except for gen-
eral dimensions and alignment.

Enlargement of the pilot tunnel to the full-size re-
search adit 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 tun-
nel 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 ex-
pected. In order to demonstrate such an expectation, the first
round, the last two rounds, and parts of the other rounds were
basted with "standard" controlled blasting of this type.

Fracture Control

In addition to the use of standard controlled blasting
techniques, some effort was put into the evaluation of the poten-
tial 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. The scribing was done
mechanically (although a high-pressure water jet can do an even
better job).

PHOTO 1

PHOTO 2

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. 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.

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.

Unfortunately, the rock at this site proved to be extremely 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 horse-shoe. Micaceous minerals tended to gather in relatively prominent thin layers along the planes of foliation. This produced a very

PHOTO 3

anisotropic rock with hard-and-soft thin bands.

An illustration of the dramatic influence which the foliation had a crack formation can be seen in Photo 3. One might ordinarily expect a crack to extend in a straightline from one drill hole to the next. Here we see that a crack was attempting to propagate in a vertical plane form the drill hole downward. However, instead of forming a straight line, the crack followed the contorted path of a tight fold inthe 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 poduce 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 question 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 this type have crept into the professional literature without proper verification. There is common knowledge of many accounts of eye-witnesses describing the observation of damage occurring, only to discover irrefutable evidence that the "damage" pre-existed such observation.

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 haz-

ard 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.

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 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", not being aware 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 non-electric 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 is usually 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 a cut-off holes if too many delays are spread out over too long a time period.
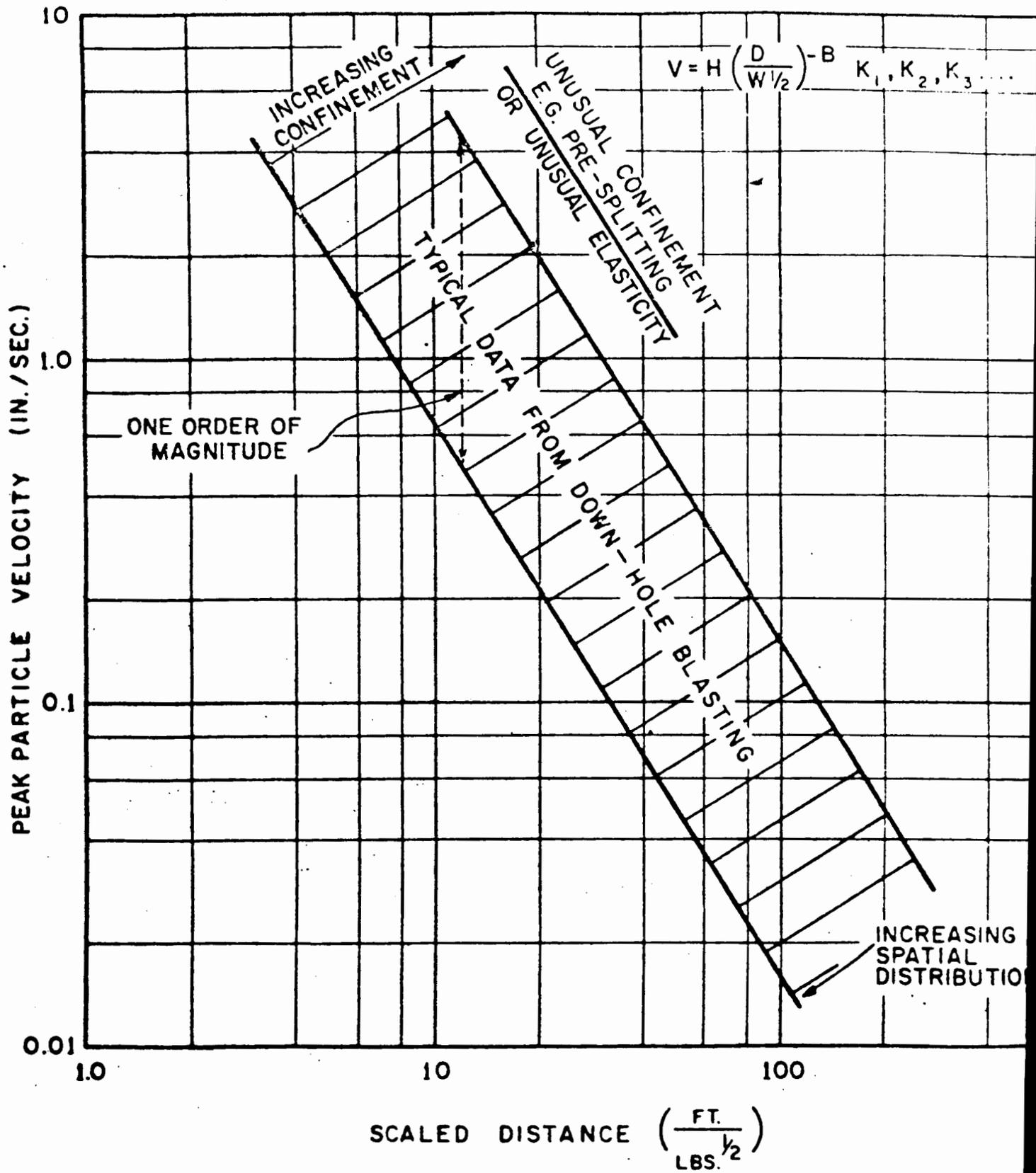
FIGURE I. ORIARD PREDICTION CURVES FOR BLASTING VIBRATIONS.

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 1. 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 ne-
cessarily to the same degree. Even a very small quantity of ex-
plosive 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, whe-
ther 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 em-
placement 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 diffi-
culty 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 over-
pressures continue toward more conservative levels, it will be-
come necessary to take more exotic measures to reduce these ef-
fects, 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 retrictive 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

Amr S. Azzouz
Herbert H. Einstein
Charles W. Schwartz

Department of Civil Engineering
Massachusetts Institute of Technology
Cambridge, Massachusetts

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 anlaysis.

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 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.

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[1] K. J. Bathe, (1967a), "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. Bath, (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.

Excavation and support installation operations can be modelled using a "birth/death" option in which elements are acti- vated 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 anal- yses. 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 sol- ver. This results in maximum system capacity, virtually elimi- nating 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-dimen- sional problems in underground openings in general, and of the Atlanta Research Chamber and Peachtree Center Station in particu- lar, preprocessor and postprocessor computer programs were de- veloped 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.

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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.

Three Dimensional Finite Element Method
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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):

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;

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[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.

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[5]Law Engineering Testing Company (1977), "Report of Geology and Instrumentation--Peachtree Center Station Pilot Tunnel (Construction Unit CN-124)", 3 Volumes.

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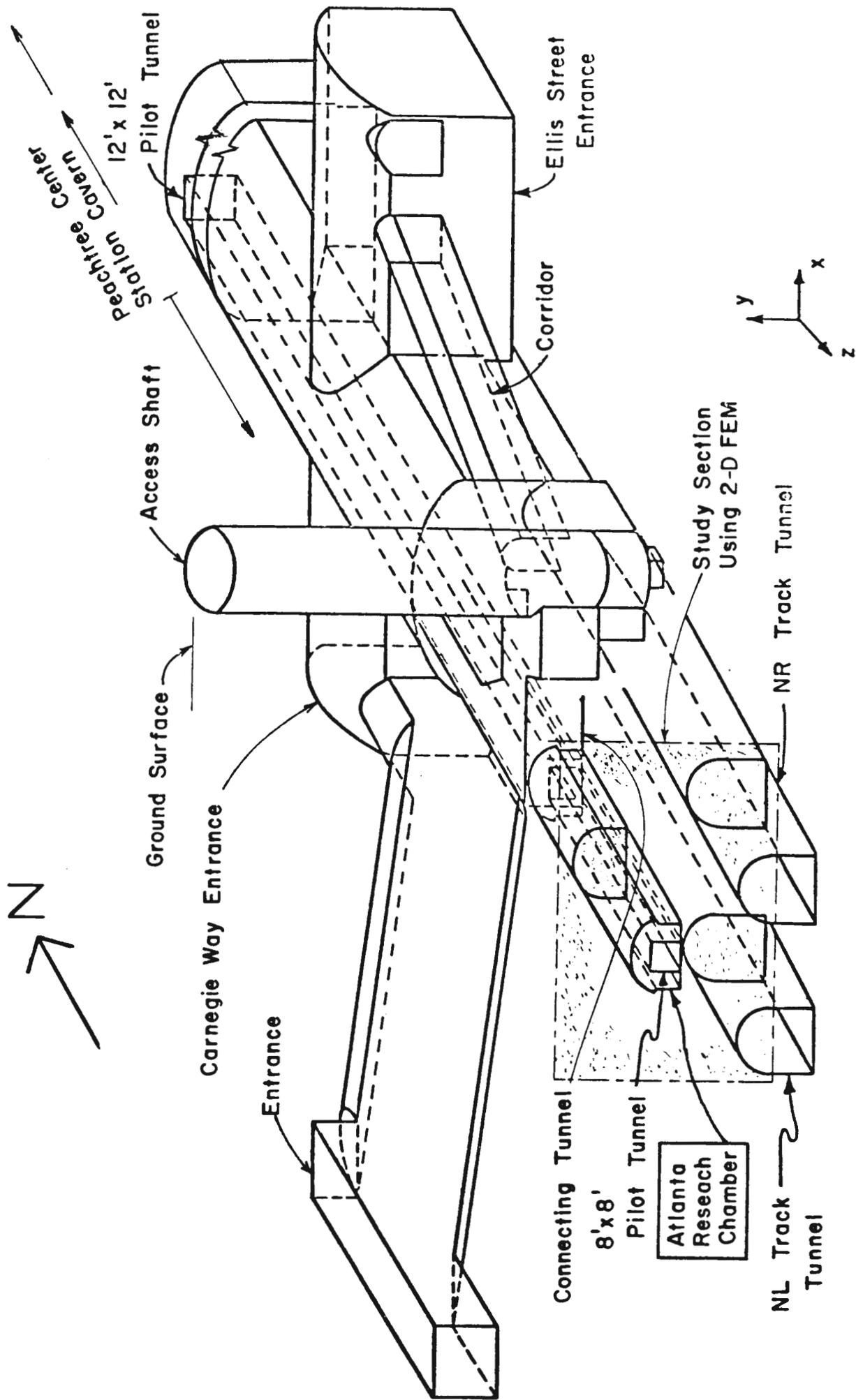
FIGURE I   OBLIQUE VIEW OF THE PEACHTREE CENTER STATION, SOUTHERN HALF

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. 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 $v = 0.17$.

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:

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 construc-
      tion of the running tunnels at this location. Further-
      more, 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 Re-
      search 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 uniform-
      ly.   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 displace-
      ments 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

20+50   21+00   21+50   22+00

Atlanta
Research
Chamber

Peachtree
Center
Station
Cavern

SOUTH

□ Step 2

O Step 4

Scale: 20 Ft.   (Geometry)

          0.005 Ft.   (Displacements)

NORTH
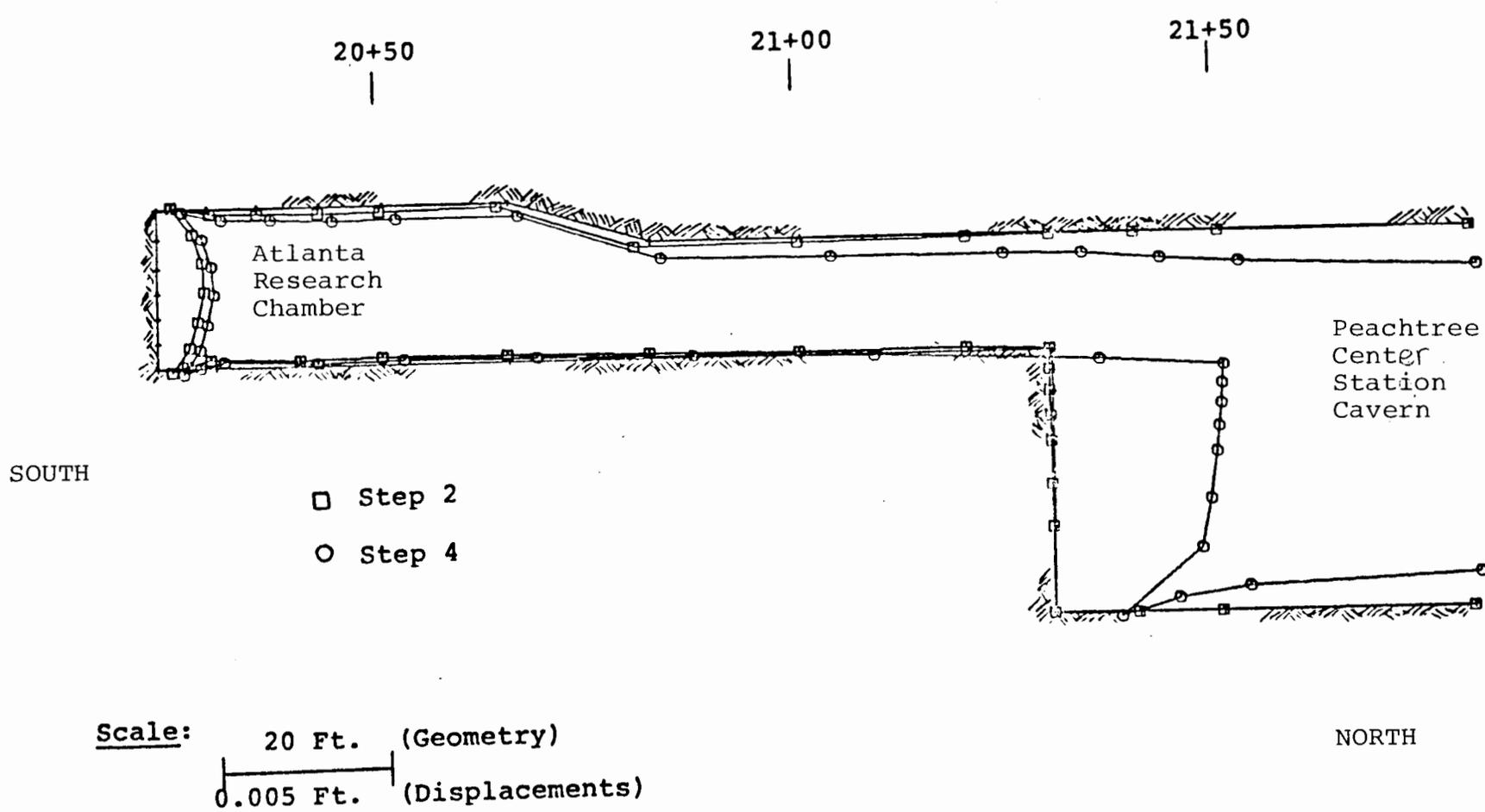
FIGURE 2   DEFORMED GEOMETRY OF OPENINGS

and consisted of:

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

20+35   20+50   20+65

Atlanta Research Chamber

Pilot Tunnel

NORTH
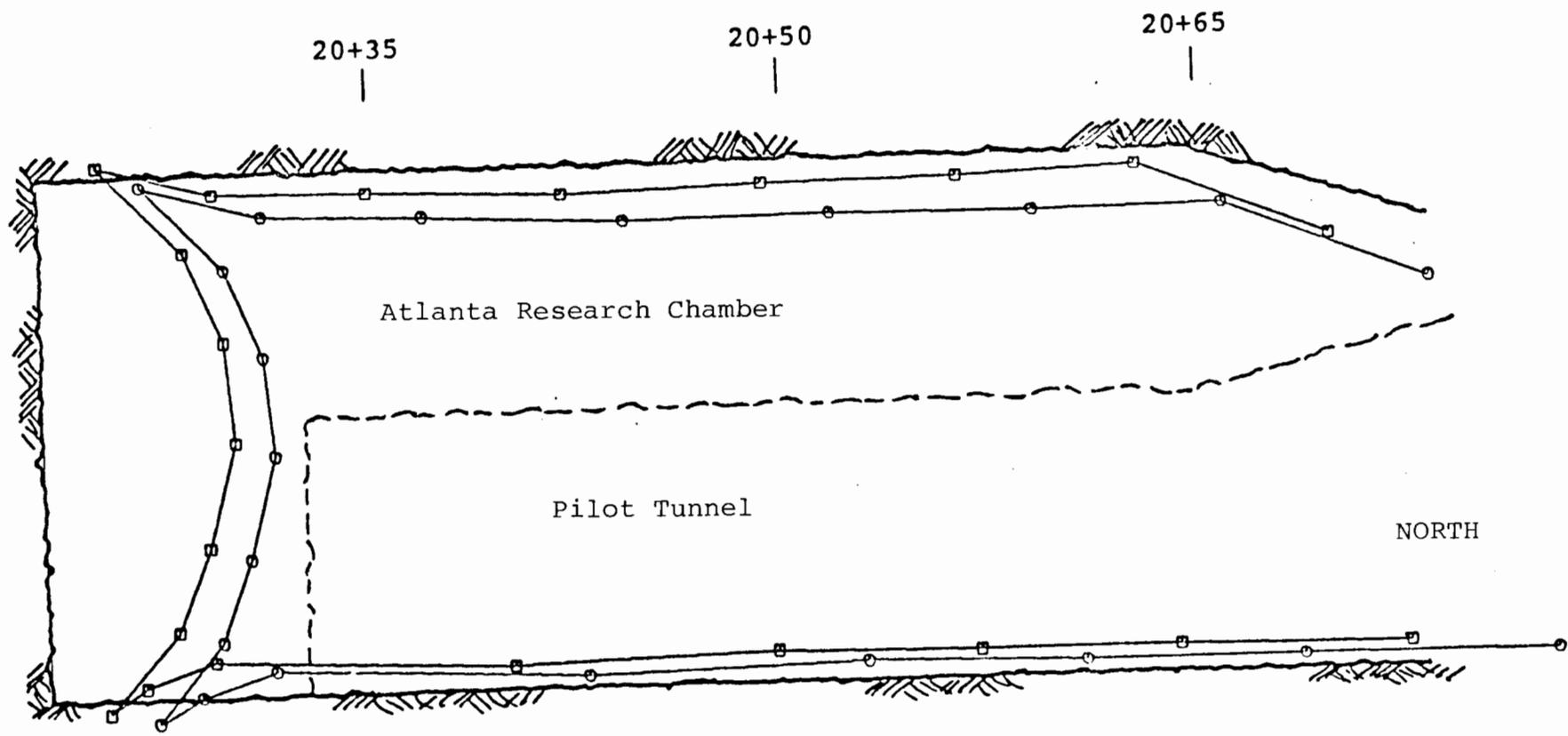
□ Step 2

O Step 4

Scale:
___10 Ft. (Geometry)___

0.0025 Ft. (Displacements)

FIGURE 3   .   DEFORMED GEOMETRY OF OPENINGS

Chamber is a good, although somewhat simplified, example of mo-
delling the performance of a geometrically complex underground
structure as it is excavated. Performance features were predic-
ted that could not have been determined from two dimensional
analyses.[6/] The effects of this ability to predict performance
on the final cost of the structure has to be weighed against the
substantially higher cost of analysis. The designer has thus to
consider carefully if sufficient information on ground and struc-
tural characteristics are available and if the value of the re-
sults obtained with 3-dimensional finite element analysis is
worth the increased cost.

[6/]F. H. Kulhawy, (1977), "Finite Element Analyses of Twin Running
Tunnels and Research Chamber at MARTA Cain Street Station,
Atlanta, Georgia", Report Prepared for Parsons, Brinkerhoff,
Quade & Douglas, Inc./Tudor Engineering Company.
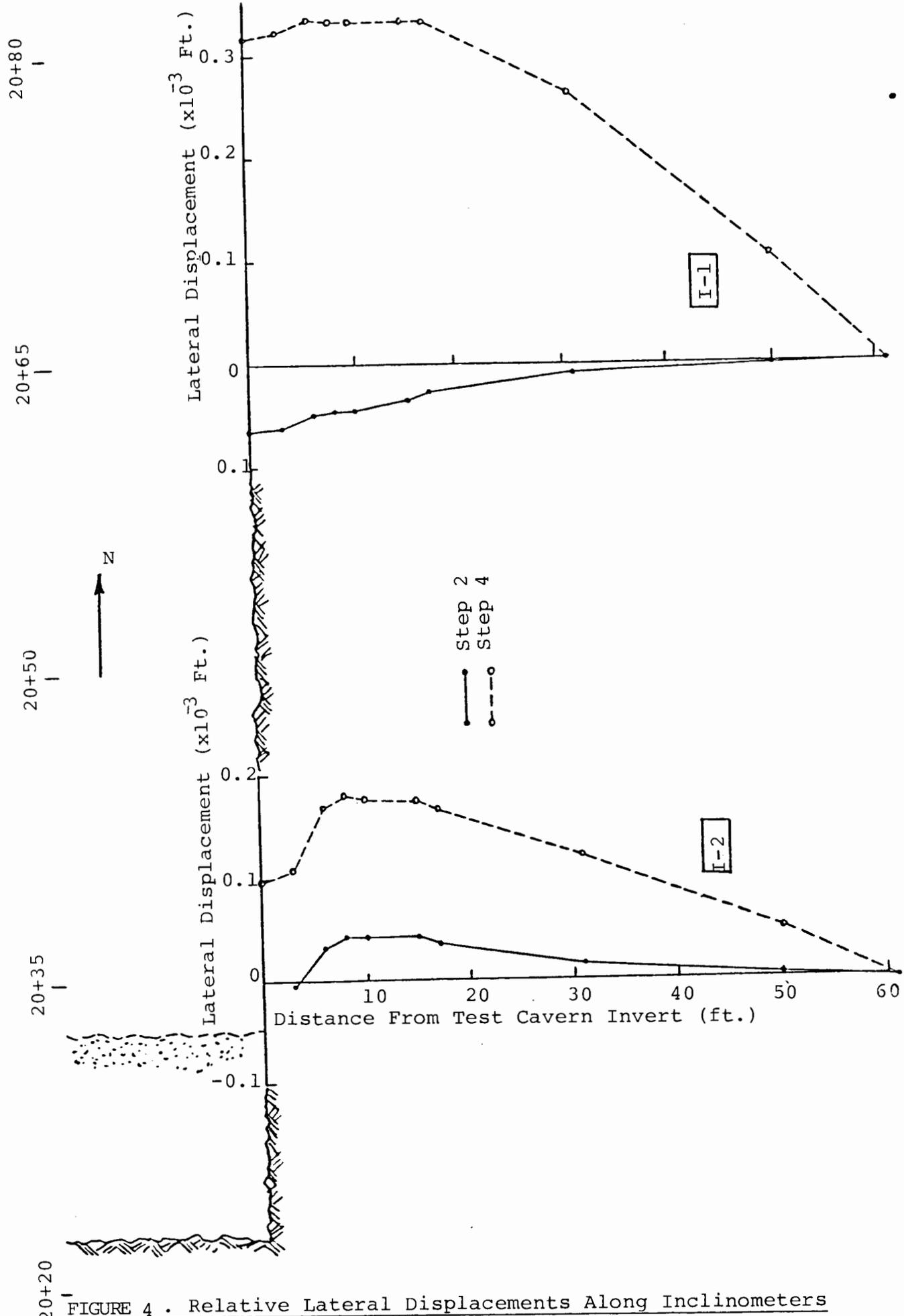
FIGURE 4. Relative Lateral Displacements Along Inclinometers

THREE DIMENSIONAL FINITE ELEMENT METHOD STUDY
OF A SUBWAY TUNNEL AT NUREMBERG

Erwin Gartung, LGA Nuremberg, FR Germany
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 co-operation 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
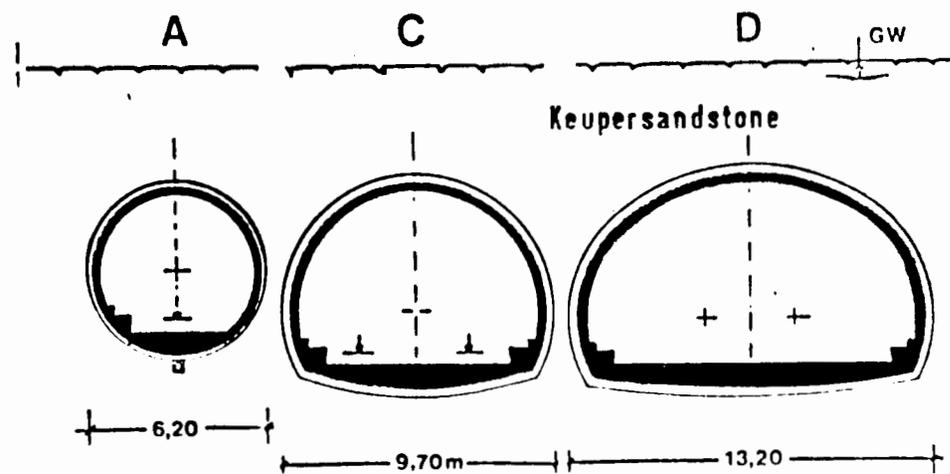
Fig. 1   Cross sections of subway tunnels in soft Keuper sandstone at Nuremberg

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 connec-
tion 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 com-
parative 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 di-
mensional 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 sta-
bility 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
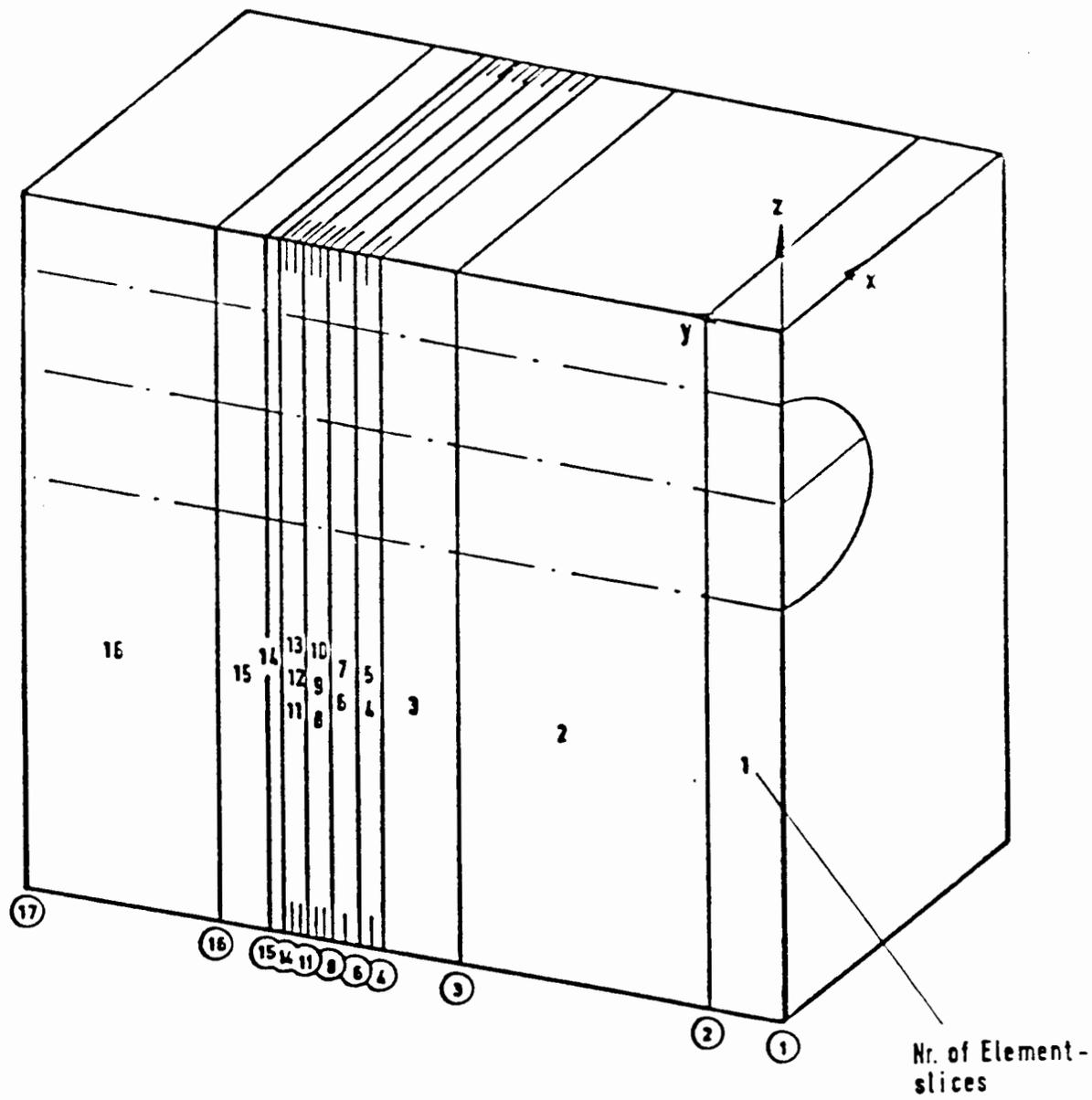
Fig. 2   Sketch of the 3D-FEM mesh
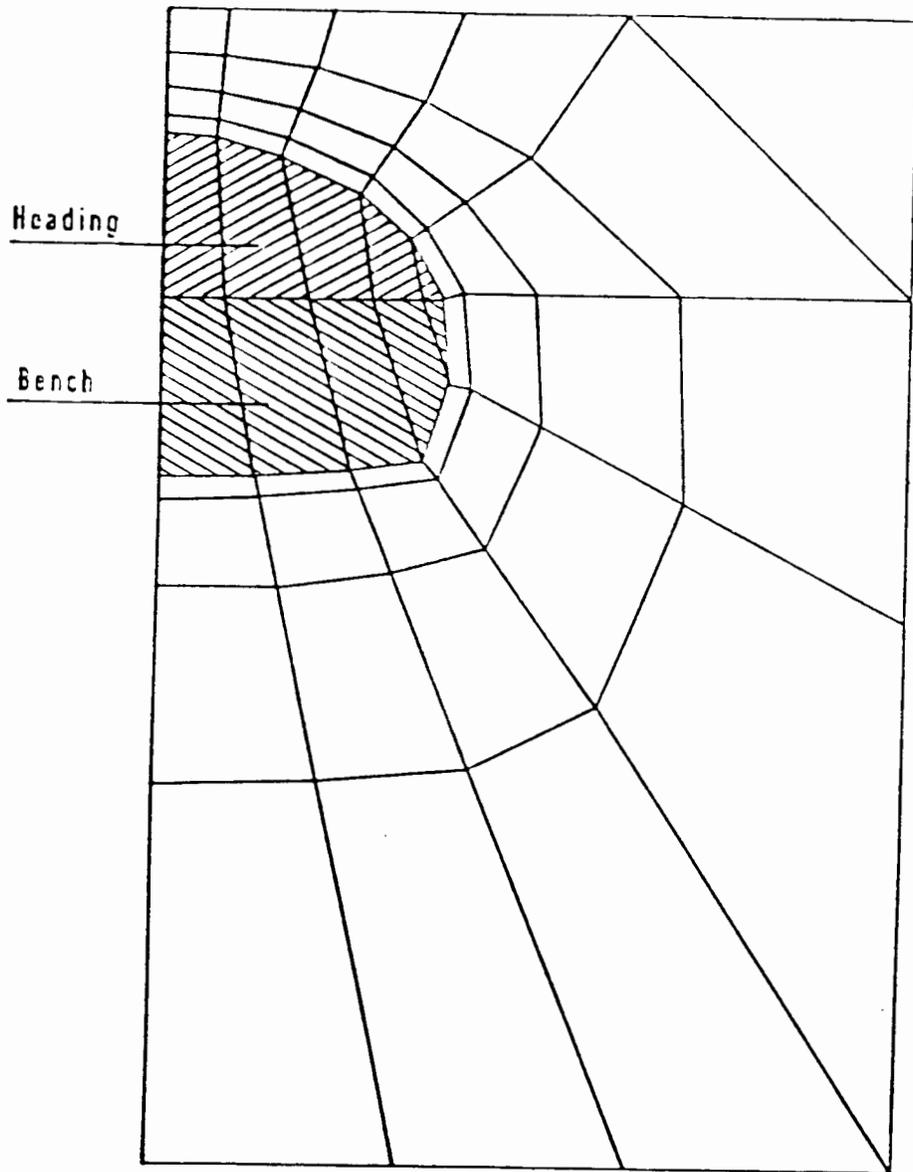
Fig. 3   Plane section through the block of the rock mass
         perpendicular to the tunnel axis

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.
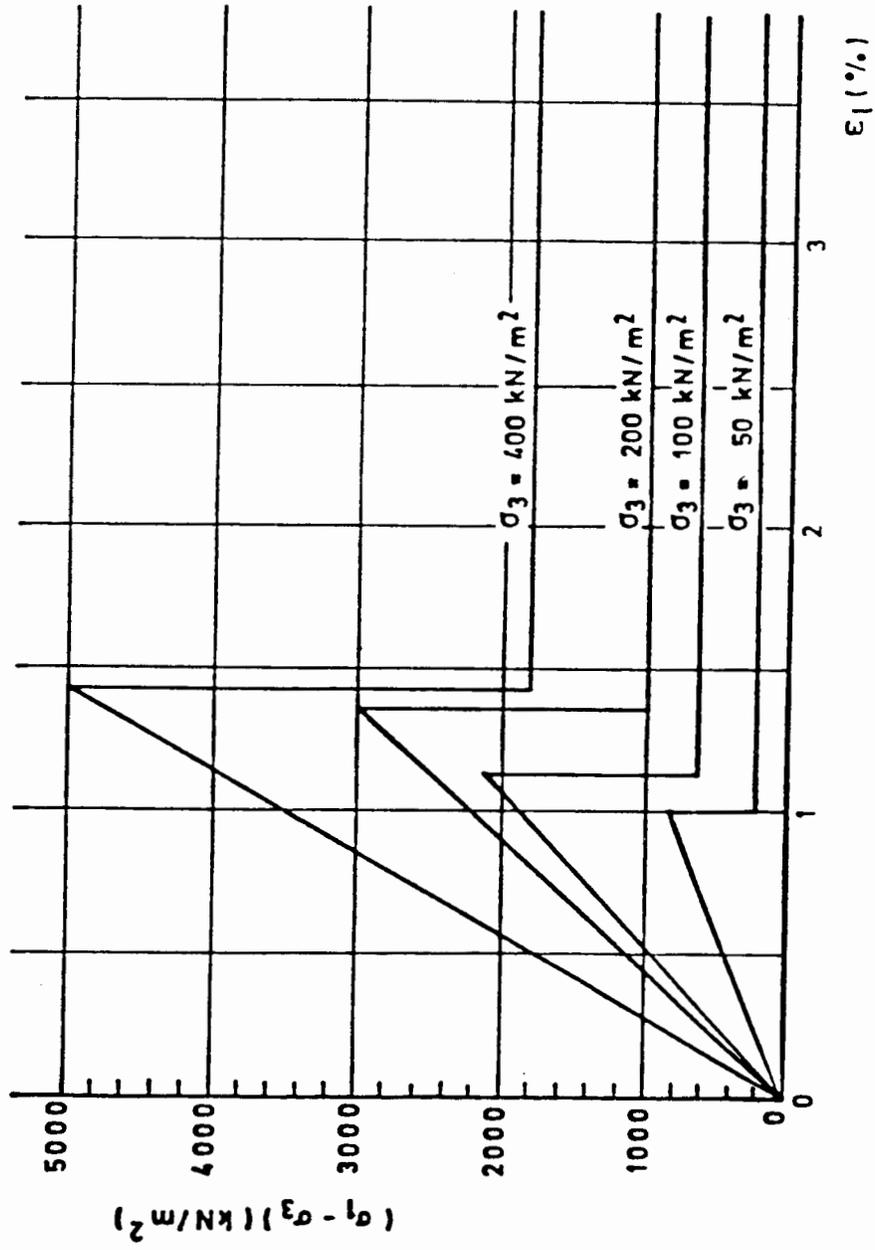
Fig. 4   Simplified constitutive law of Keuper sandstone

Run	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
						E − Modulus · 1000 MN/m² in Elementslice Nr.					
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 correct simulation of the supporting effect of shot-crete, 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.

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.

Front cut
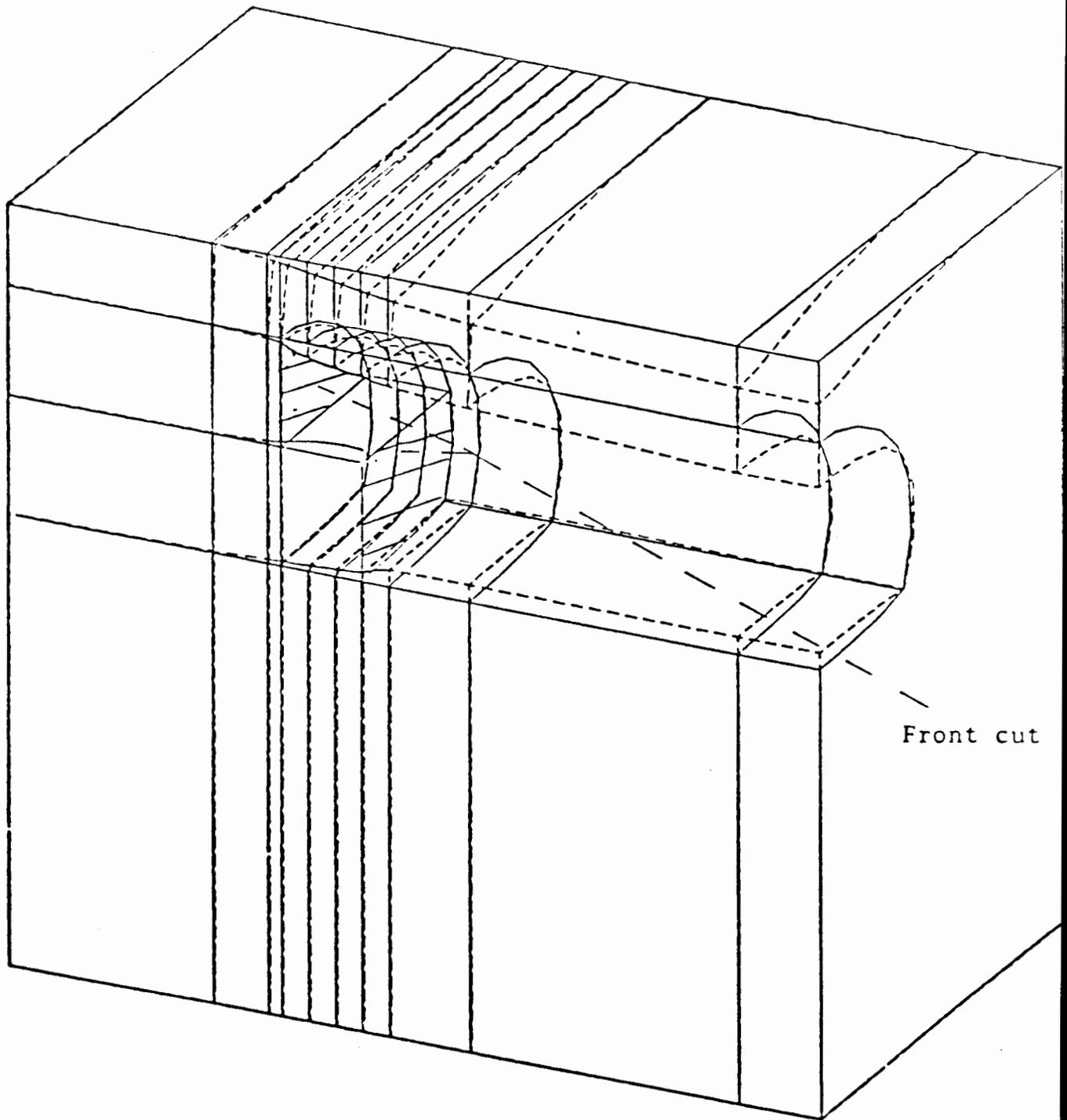
—— 4 cm (displacement)

Fig. 5   Deformed 3D – FEM mesh, computer plot of run 9.
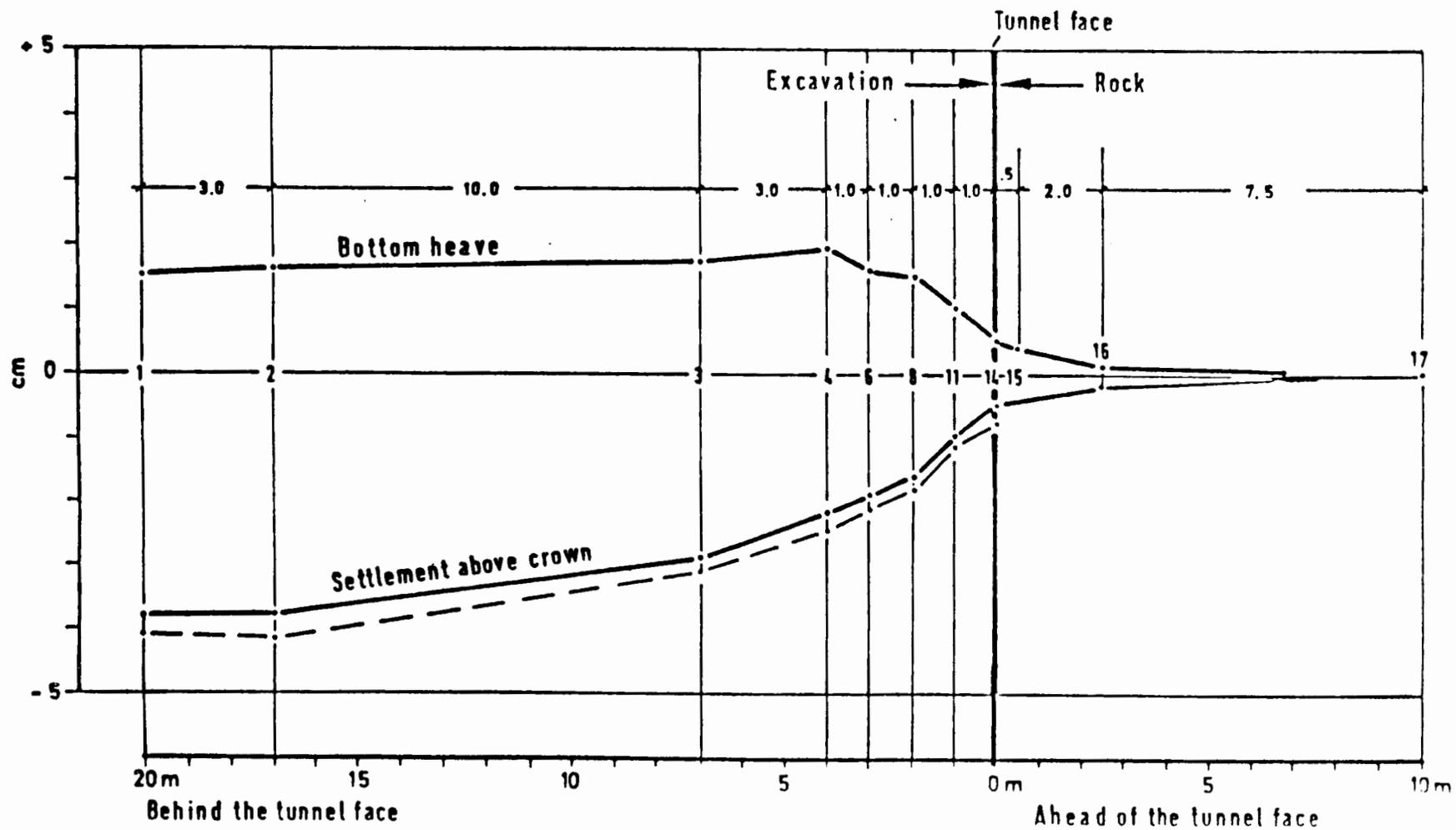
Fig. 6　　Vertical displacements along the tunnel axis
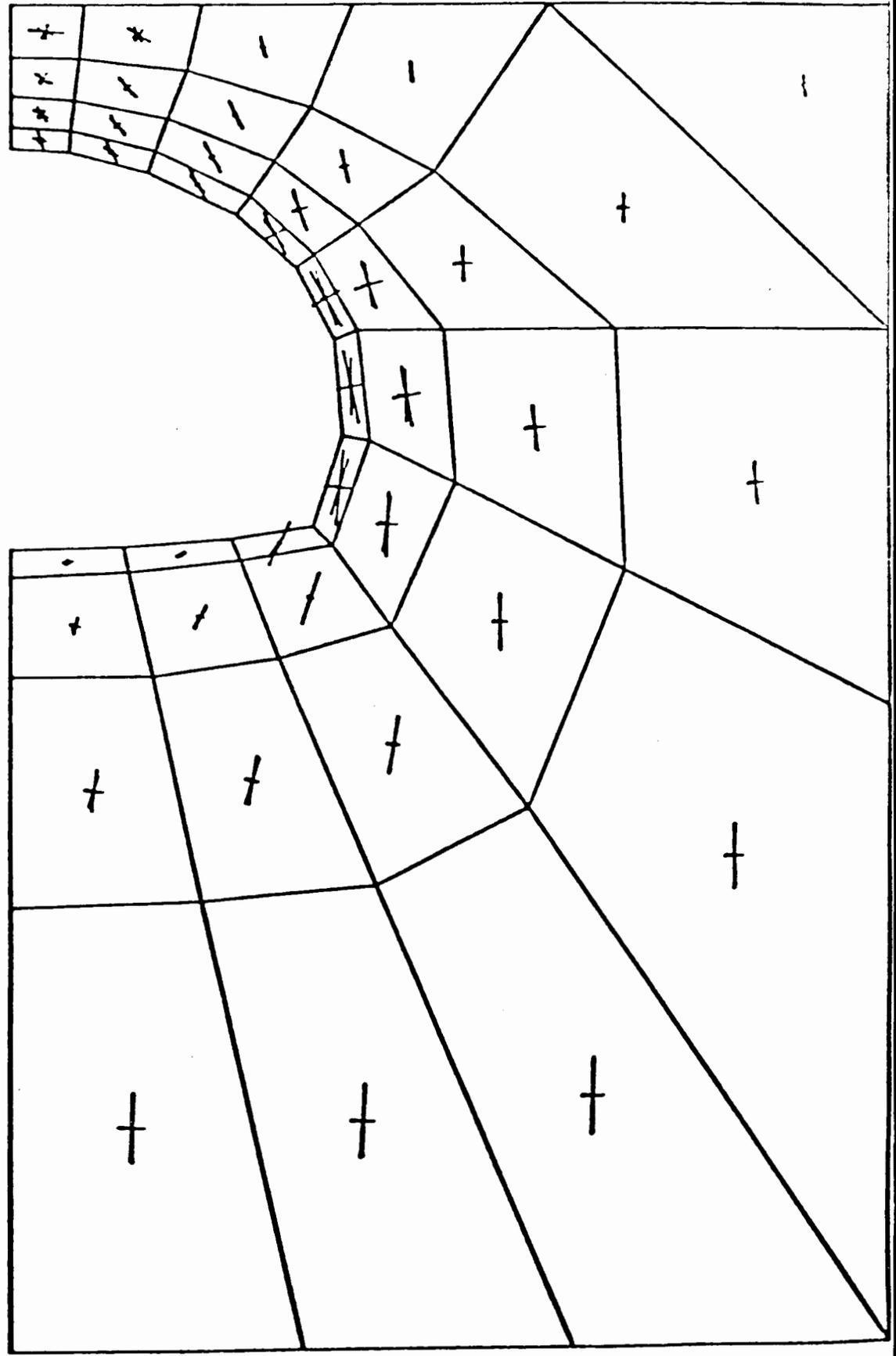
Fig. 7   Principal stresses in a section perpendicular to
         the tunnel axis 5,5 m behind the tunnel face
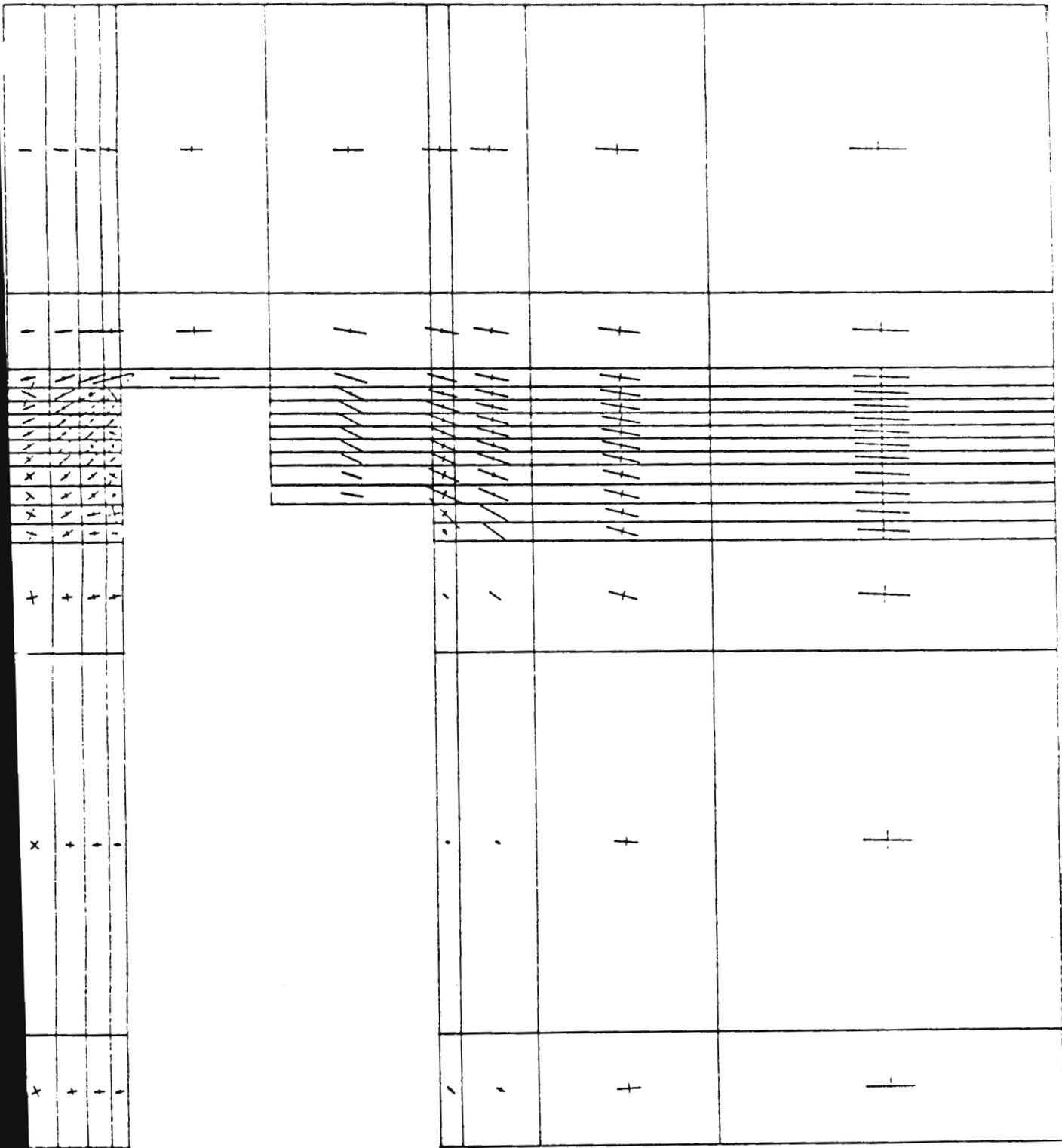   (slice 3) - comparison with slice 9, run 12

Fig. 8   Principal stresses in a section parallel to the tunnel axis, run 9
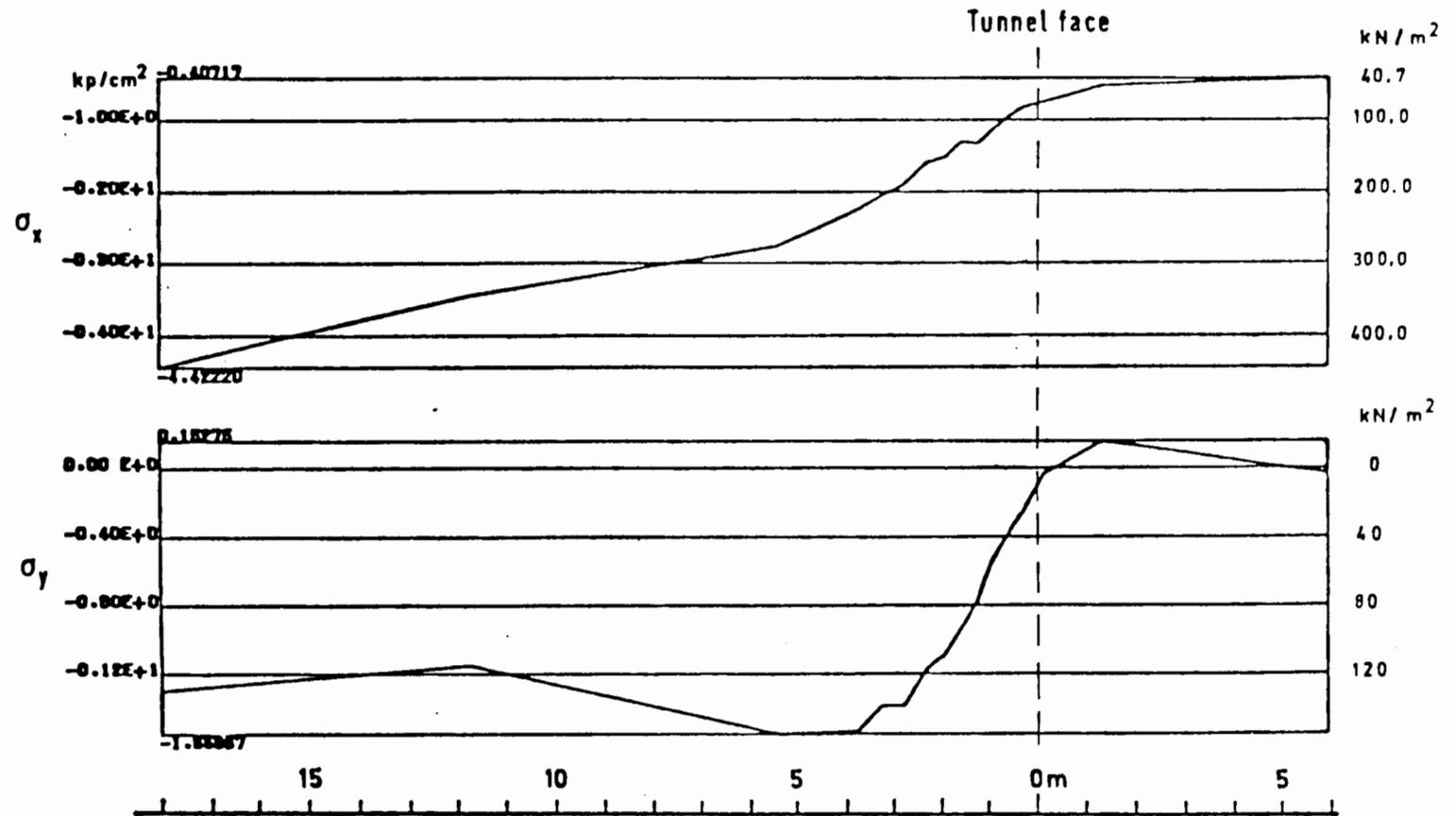
Fig. 9   Stresses in element 1,1 above the crown at the sandstone surface

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 fig. 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
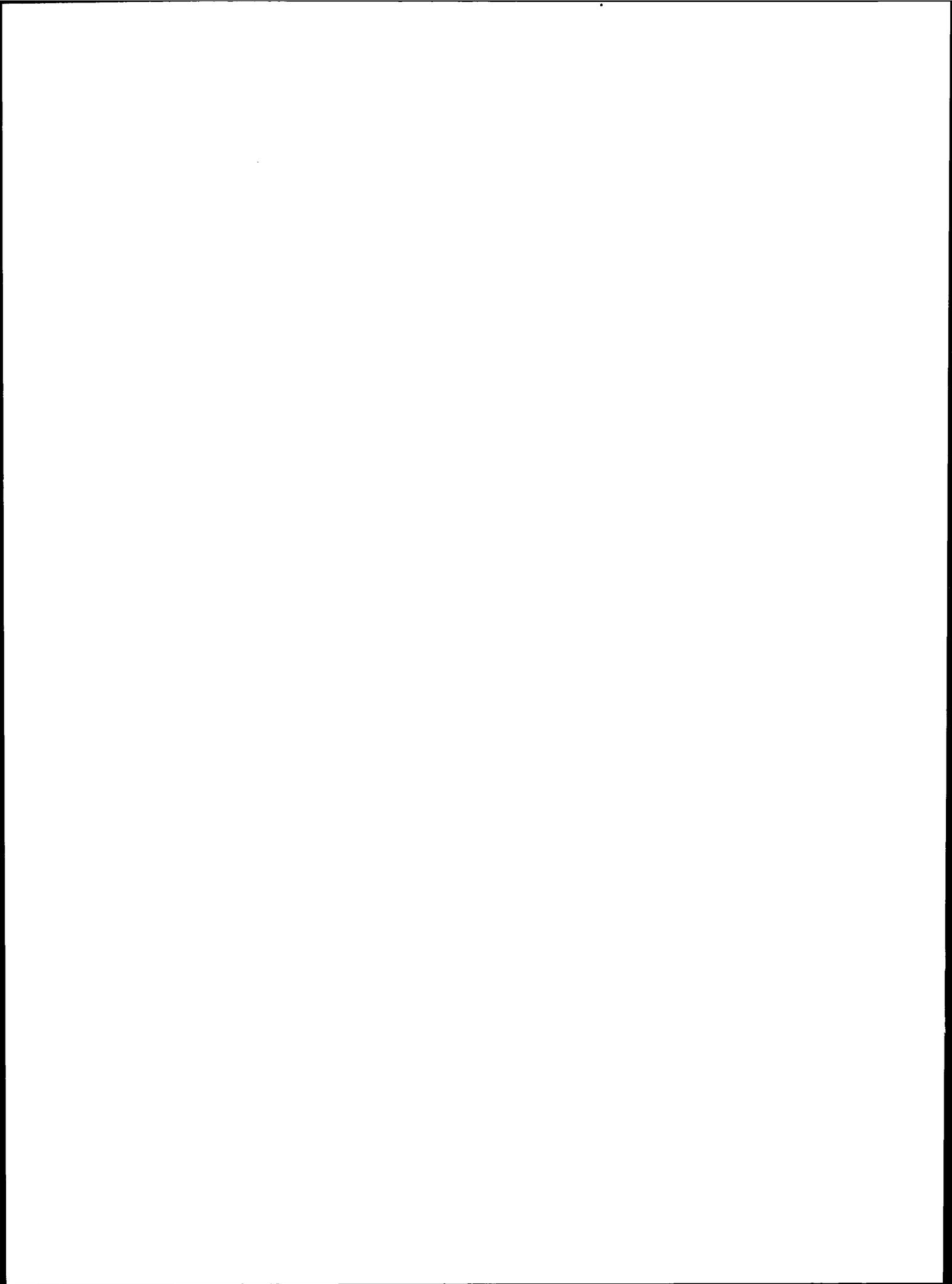
THE NEW AUSTRIAN TUNNELING METHOD

Johann Golser

Geoconsult

Salzburg, Austria

(to be prepared)
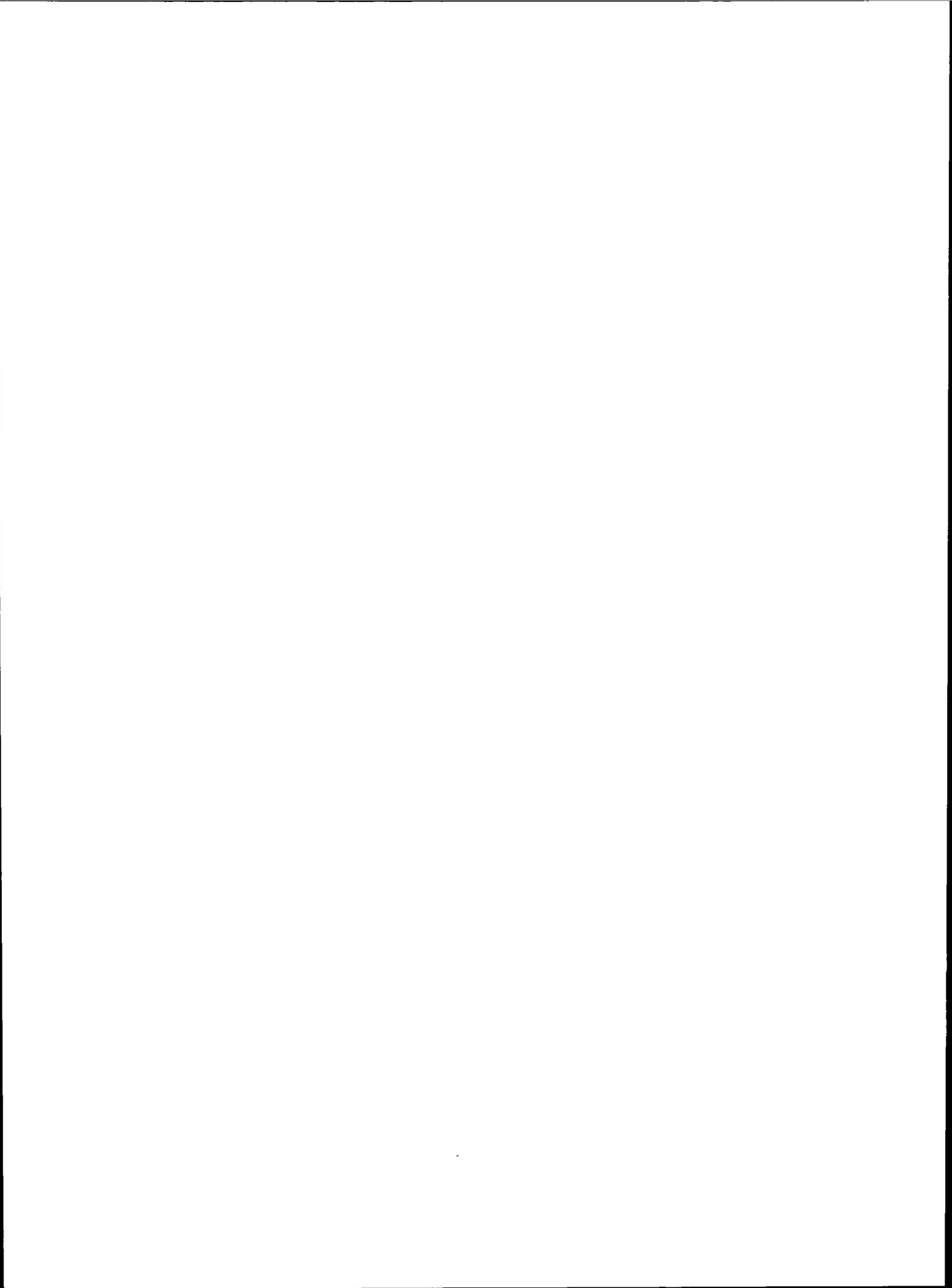
THIN SHOTCRETE LININGS IN LOOSENING ROCK

by

Gabriel Fernandez-Delgado[1], Edward J. Cording[2]

James W. Mahar[3], and Michel L. Van Sint Jan[4]

1. INTRODUCTION

Shotcrete behavior observed in subway tunnel construction in the U.S. has been primarily in blocky or blocky and seamy rock 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. Such conditions can be classed as loosening ground, in contrast to squeezing ground in which high pressures are generated on the tunnel lining with time as the ground creeps under the natural ground stresses. In the philosophy often associated with the New Austrian Tunneling method, emphasis is placed on the use of shotcrete and other support elements in squeezing, highly stressed ground, in which the support system is designed to allow controlled movements so that high lining stresses do not develop. In contrast to these conditions, the concern in loosening ground is to provide support soon enough to minimize loosening that would cause instability in the tunnel heading or apply excessive load to the tunnel lining. Very small movements will relieve high pressures, thus

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[1]Visiting Assistant Professor, Department of Civil Engineering, U. of Illinois, Urbana, IL 61801.

[2]Professor of Civil Engineering, U. of Illinois, Urbana, IL 61801.

[3]Visiting Assistant Professor, Department of Civil Engineering, U. of Illinois, Urbana, IL 61801.

[4]Graduate Research Assistant, Department of Civil Engineering, U. of Illinois, Urbana, IL 61801.

it is not necessary to make special accommodations in the design to allow movements.

The present methods for designing or selecting thin shotcrete tunnel linings in 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 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). Some of the results of this study are described in Section 3.

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 a 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. Section 2 describes the result of the laboratory studies.

Recently, studies were carried out at a test site in the Peachtree Center Station, Atlanta. 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 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. The results of the Atlanta study are summarized in Section 4.

Based on the results of the studies, the rock and support parameters that should be considered in selecting a shotcrete lining are outlined in Section 5.

2. 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-ft x 2-ft 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-ft-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
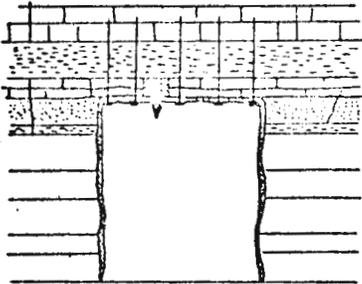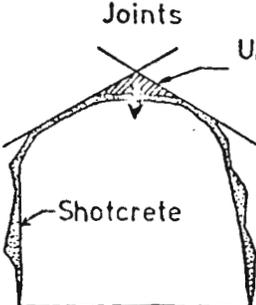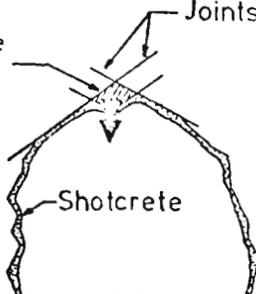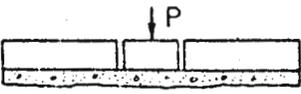
TUNNEL CONFIGURATION	PLANAR ROOF	Arched with smooth surface	Arched with irregular surface
GEOMETRICAL CONFIGURATION & TEST CODE	PLANAR S- TESTS	ST-	STM-
SHOTCRETE STRENGTH	1000 psi - 3500 psi (6.9 MPa) (24.1 MPa)		
ADHESIVE STRENGTH	LOW 10 psi (69 KPa) 20 psi (138 KPa)	HIGH 60 psi (414 KPa) 180 psi (1241 KPa)	
LAYER THICKNESS	1.0 in — 6.0 in (2.5 cm – 15.0 cm)		
LATERAL BOUNDARY CONDITIONS	FREE END		
	SUPPORTED END Rock Bolts		
LAYER REINFORCEMENT	NONE	FIBER	MESH
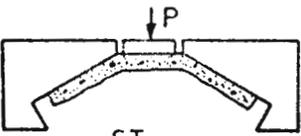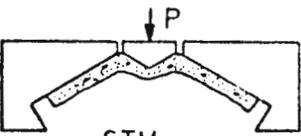
TABLE 1.   PERTINENT VARIABLES AND TEST MODEL

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
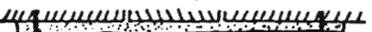
* If enough shotcrete is placed to fill in the recess and develop a smooth arch configuration, the failure mode changes to moment thrust.

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 developed in the laboratory tests, the maximum thickness at which a shear failure would still develop was found to be 2 in. The support capacity when shear failure occurs is proportional to the lining thickness; on the other hand once the adhesive strength is exceeded increasing shotcrete thickness provides no additional capacity. Field evidence from in situ tests performed in a good quality granitic rock (gneiss) in Atlanta, Georgia (Section 4) show 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 developed 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, 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 in.; 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 in.) than the failure displacements for unreinforced planar shotcrete layers.

A 4-in. x 4-in.-square, 1/8-in.-diameter mesh placed close to the outside surface of the layer, with 8-ft 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 5 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

TABLE 3   Capacities of Unreinforced Shotcrete Layers
Tested in the Laboratory

Failure Mode	Capacity		θ			Range of Typical Displacements at Failure (in.)
			$0°$	$15°$	$30°$	
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 / 0.32(3)	0.12 / 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,
         $\theta \geq 30°$.

(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.
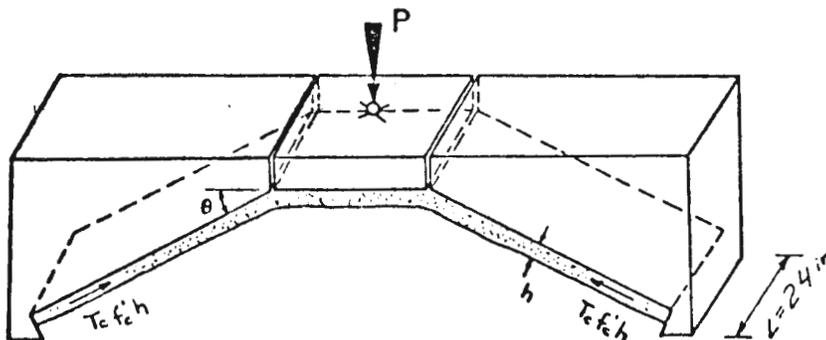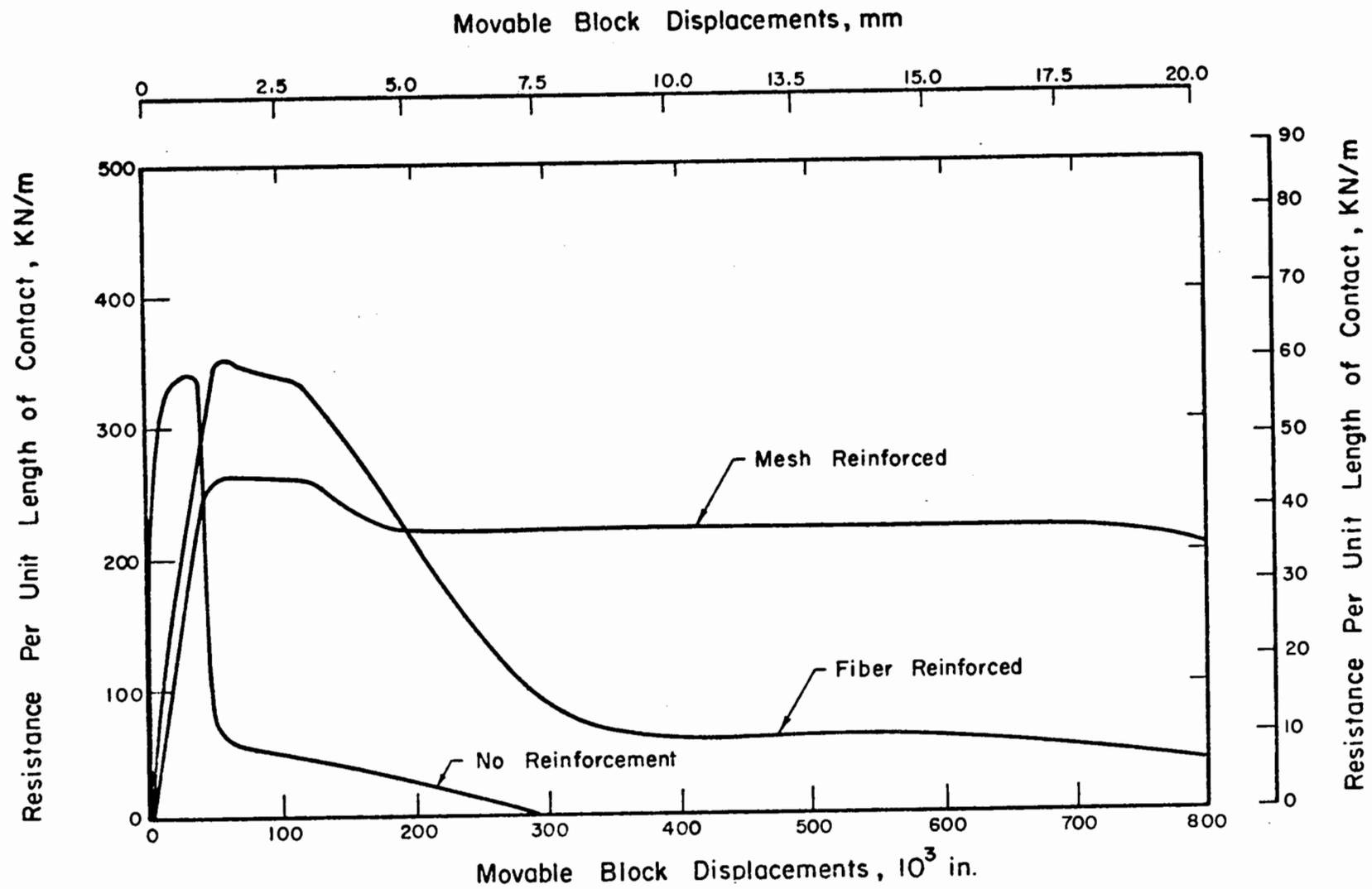

Model Configuration

FIG. 1   TYPICAL LOAD-DISPLACEMENT RELATIONSHIPS FOR LATERALLY-RESTRICTED
LAYERS IN THE PLANAR CONFIGURATION

effective to increase the capacity of the layers by filling up the recess areas until a smooth arch configuration is obtained; this procedure will change the failure mode from bending along the apex into a moment-thrust failure along the inclined portion of the layer.

3. 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-ft-wide and 22-ft 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 folia-tion 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 sometime 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 ft in dimension, which must be supported as the

headings are opened. Typical shotcrete compressive strength at 24 days were

equal to 5000 psi.

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 Fig. 2a. A rock block

approximately 8 ft x 8 ft was bounded by joint surfaces coated with clay

and chlorite. The slickensided joint surfaces not only facilitated 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 in. 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:

$$F.S. = \frac{P}{W(\text{rock wedge})} \tag{8}$$

where $W = 8(ft) \times 8(ft) \times 1(ft) \times 160 \text{ pcf} = 10,240 \text{ lbs}$ and

$P = B_c f'_c h^2 \frac{L}{d}$ (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
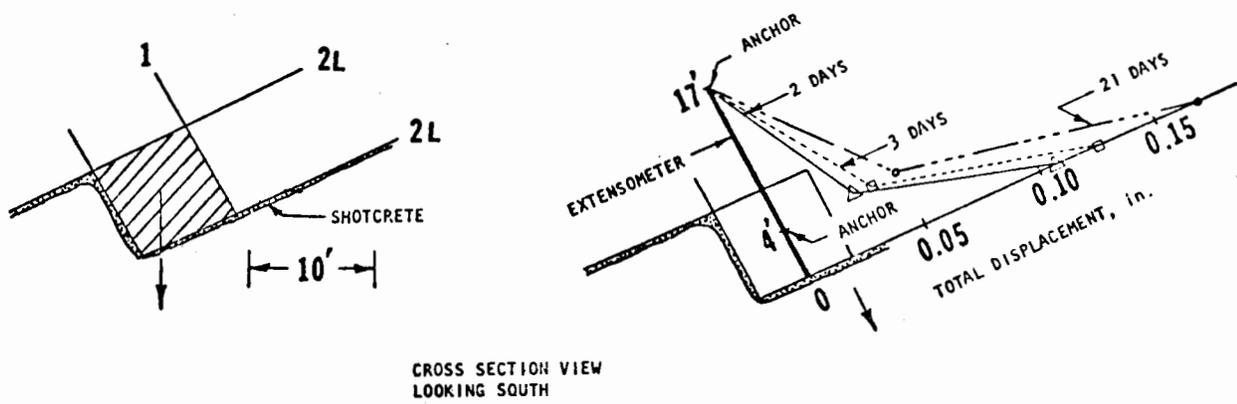
Fig. 2a   Rock Displacement in a Shotcrete-Steel Rib Section
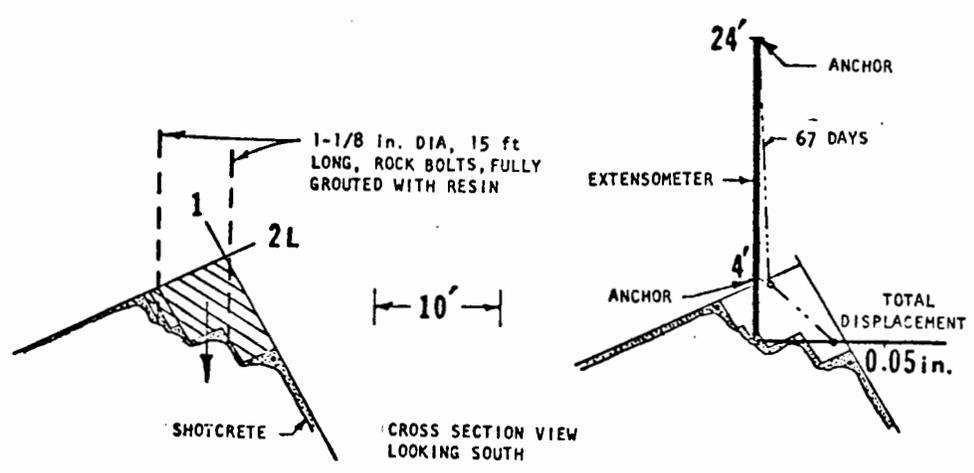   of Tunnel (Mahar, Gau and Cording, 1972).

Fig. 2b   Rock Displacement in a Shotcrete-Rock Bolt Section
           of Tunnel (Mahar, Gau and Cording, 1972).

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 in., 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,

$$P = 0.25 \times 2000 \times 6.25 \times \frac{1}{4} = 780 \text{ lbs}$$

The resulting factor of safety can then be expressed as:

$$FS = \frac{2560}{780} = .33$$

assuming that only 25 % of the wedge weight was acting on the liner at this time.

The low factor of safety is consistent with the observed behavior; crackings 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-ft long rock bolts fully grouted with resin and placed on 4-ft centers. Rock bolts were installed within 2 ft of the tunnel face. In this case, continuous joints

combined to form an 8-ft x 10-ft wedge of rock hanging in the tunnel arch, as indicated in Figure 2b. Both joints were coated with up to 1/4 in. 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 ft in size, separated by several tight joint planes.

Rock movement of 0.030 in. was recorded between the crown and the 4-ft anchor when the heading was advanced 5 ft 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 in. movement was recorded behind the No. 2L plane, (between the 4-ft and 24-ft 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 surface. (Cording, 1974).

Field observations at sections of the Washington D. C. tunnels instrumented with strain gages embedded in the shotcrete, have shown that tensile
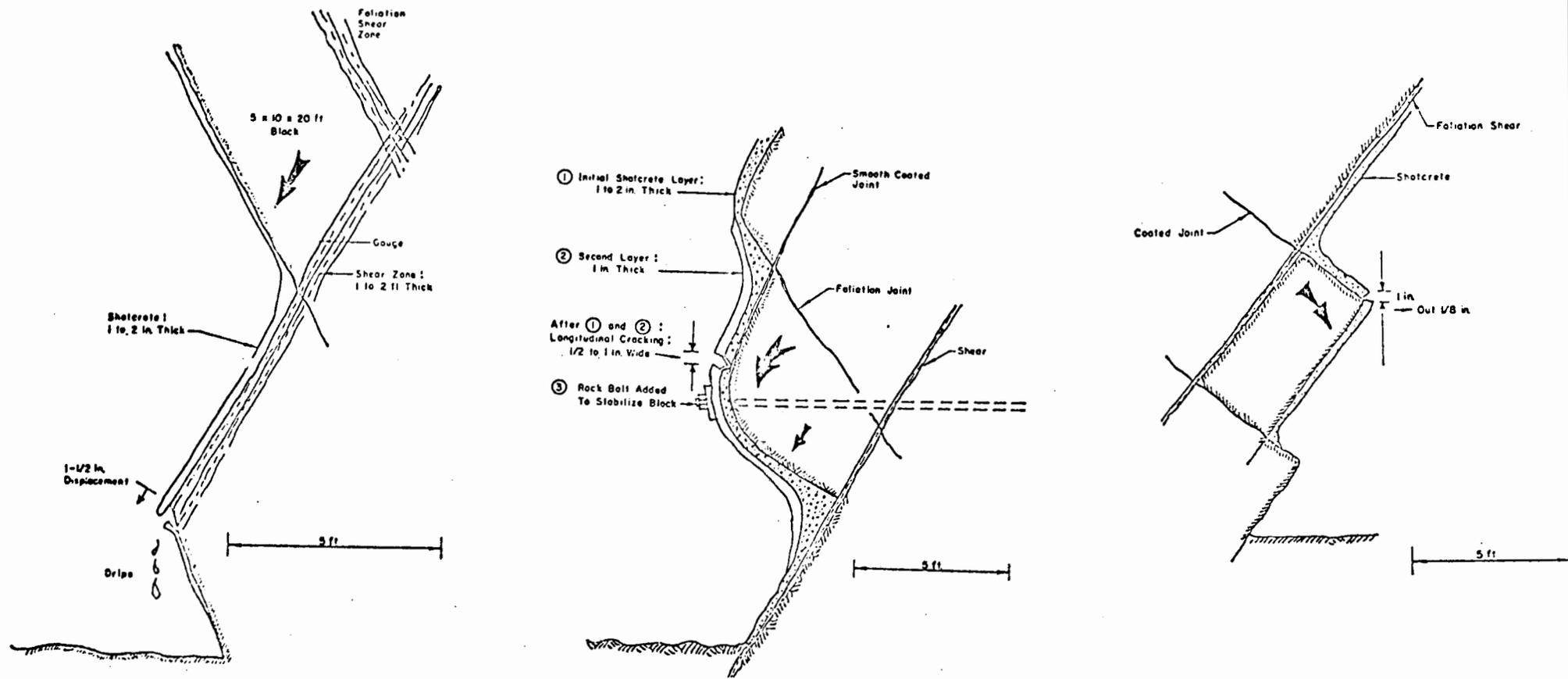
Fig. 3   Field Observations of Different Displacement Modes
(Cording, 1974).

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 ft 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 ft or more in width, or c) on vertical side walls greater than 10 ft in height that were backed by steeply dipping joints.

4. IN-SITU TESTS, PEACHTREE CENTER STATION, MARTA, ATLANTA

A shotcrete testing program has been conducted in a test section in the Peachtree Center Station. The rock is a foliated granitic gneiss, with foliation dipping approximately 15° to 25° to the SE and striking north-north east, 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-ft x 2-ft steel plates embedded in the shotcrete were pulled to evaluate the capacity of the shotcrete placed in situ and obtain a direct comparison with the laboratory tests (Fig. 4). The plates were placed in contact with the rock and covered with a layer of

TEST N° 1

TEST N° 2

~25°   ~30°

TEST N° 3

TEST N° 4

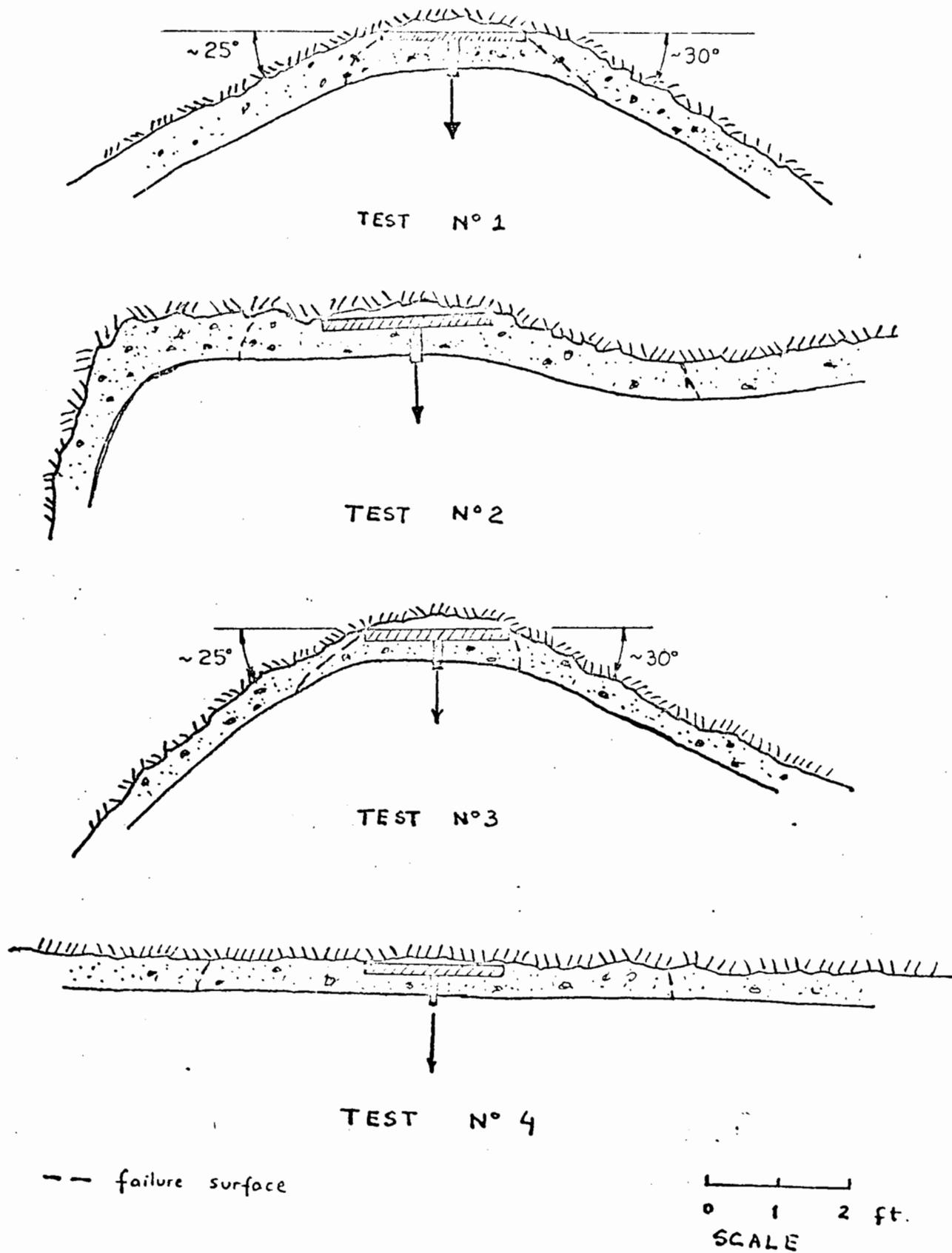- - failure surface

0   1   2 ft.
SCALE

Fig. 4   Test Configurations and Failure Surfaces.   MARTA, Atlanta.

shotcrete slightly wider than 2 ft and extending some 8 ft 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 fiber shotcrete. In each case the two plates were placed so as to test one planar and one arch configuration, Fig. 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 in. and 8 in. thick shotcrete layers with arched configurations failed in shear; shear strength 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 in. 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 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 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 in. to 2 in. before failure, whereas the flat shotcrete layer without fiber reinforcement failed brittlely, with little warning.

5. ESTIMATING SHOTCRETE REQUIREMENTS

5.1 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 Fig. 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.

TABLE 4

Summary of Shotcrete Capacity Tests. Atlanta Research Chamber.

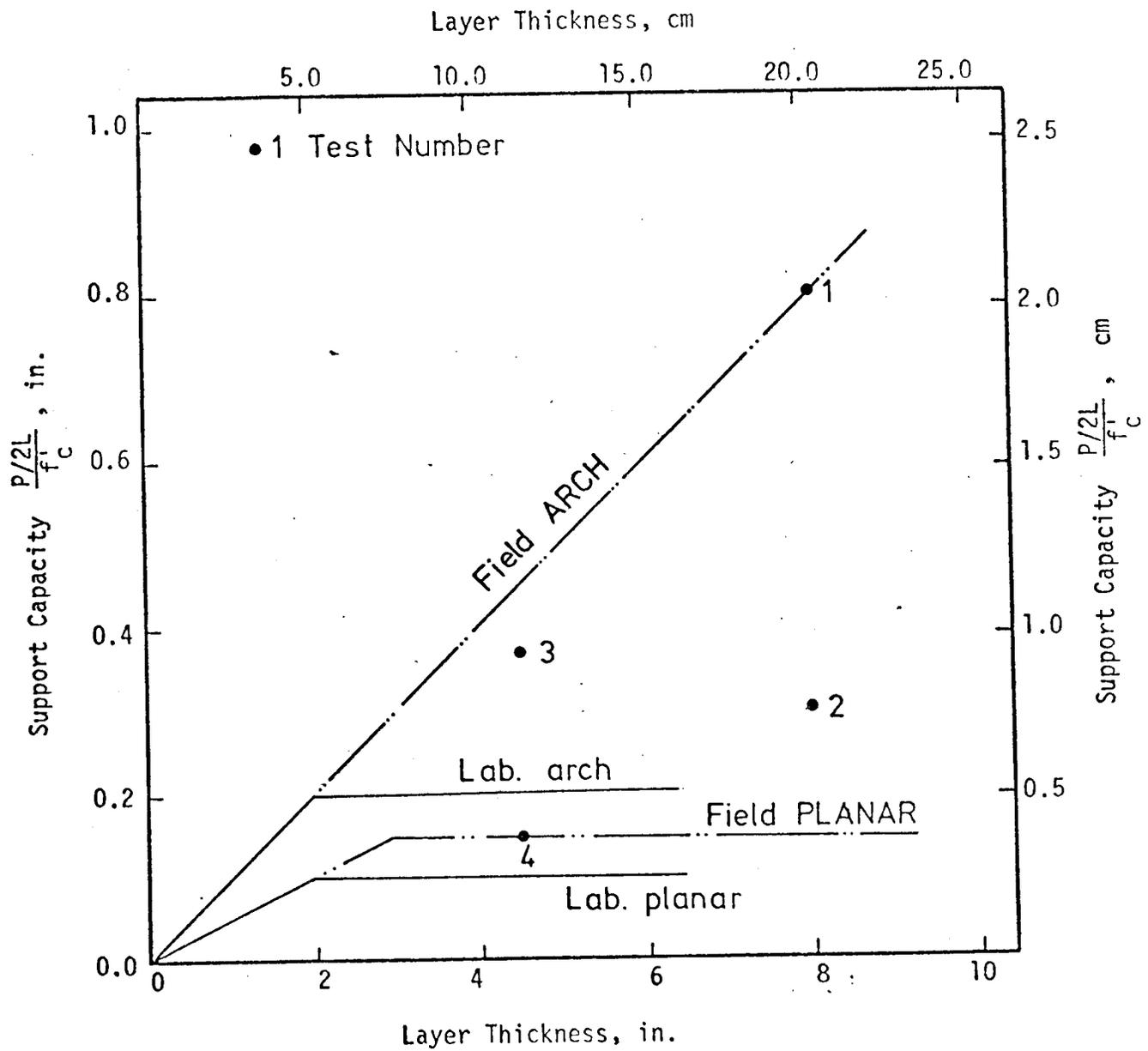| Test No.	Configuration	Shotcrete Age	Reinforcement	Failure Mode	f'_c psi	Thickness in.	Capacity lb	Shear Strength f_d	Adhesive Strength, a_o
1	Arch	10 hs.	None	Shear	1400	8	52,000	0.10 f'_c	-
2	Flat	24 hs.	None	Adhesion	3500	8	50,000	-	0.15 f'_c
3	Arch	7 hs.	Fiber (3%)	Shear	400	4.5	7,000	0.08 f'_c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

Layer Thickness, cm

Fig. 5 Maximum Capacity of High Adhesion Shotcrete Layers with Free End. Field Results from Atlanta compared with Laboratory Studies.
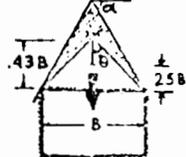
(α) DIP ANGLE	(θ) HALF ANGLE	(nB) HEIGHT of EQUIVALENT ROCK LOAD	MINIMUM CONDITION FOR FAILURE	
0° - 30°	90° - 60°	(0 - .15)B	Both planes wavy, offset	
30° - 45°	60° - 45°	(.15 - .25)B	One plane wavy or offset; One plane smooth to slightly wavy	
45° - 60°	45° - 30°	(.25 - .43)B	One plane sheared, continuous and planar; One plane slightly wavy	
60° - 75°	30° - 15°	(.43 - 1.0)B	Both planes sheared, continuous and planar	
75° - 90°	15° - 0°	> 1.0 B	Low lateral stresses in arch; Surfaces planar, smooth, possibly open; or progressive failure aided by separation along low angle joints	

Fig. 6   Conditions for Wedge Formation in Tunnel Crown Based on Observations in New York City and Washington, D.C. (Cording and Mahar, 1974).

5.2 BASIC DESIGN PROCEDURE

Figure 7 illustrates how the field and lab tests 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, θ, 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 ($\theta = 30°$) smooth-arched layers tested in the laboratory, Table 3. Minimum thickness of shotcrete required to provide a factor of safety 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, smaller wedges 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, ω, less than 0.5R ($\theta < 15°$), the planar geometry case described in previous section, Table 3, would be the limiting, and conservative, case.
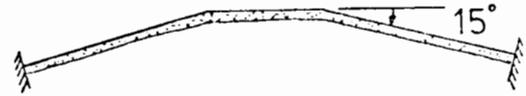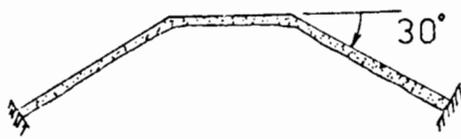
Equivalent Shotcrete Geometry

w=r, θ=30°, T_C = 0.4
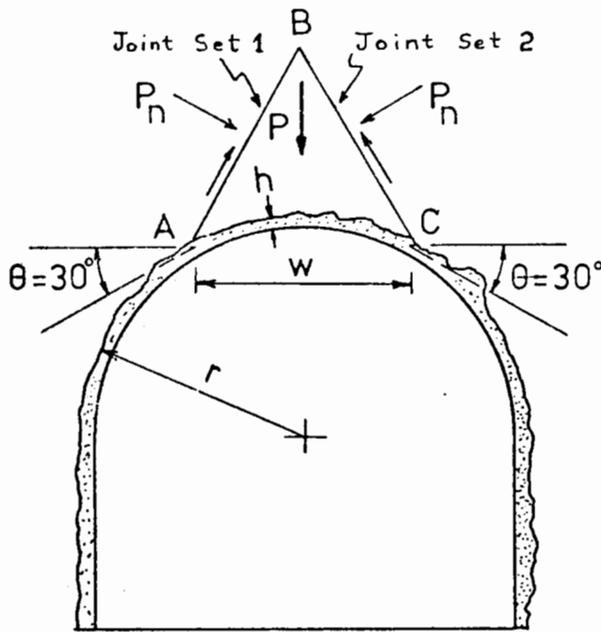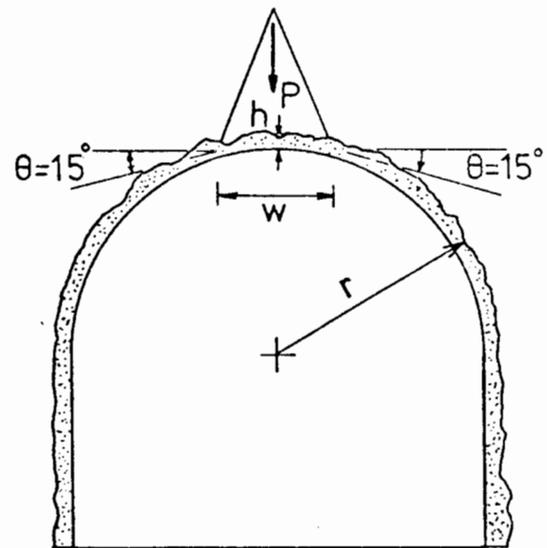
w=1/2r, θ=15°, T_C = 0.3

$$P = 2 T_c [\sin \theta] [h] [f'_c \cdot L]$$

where h is the thickness of the continuous portion
of the arch.

Fig. 7   Cross Section.   Triangular Moving Wedge in Smooth
Surface Tunnel

5.3 POOR BOND

If shotcrete-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 in 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, the bond may be so poor that the shotcrete tends to loosen and separate from the roof under its own weight. The shotcrete can be held in place if it is reinforced and 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 mesh reinforced (4-in. x 4-in. x 1/8 in.) shotcrete layer between rock bolts can be analyzed as a slab in bending with an average moment coefficient, $M/(f_c' b h^2)$ 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 the thrust coefficient increases with the increase in abutment angle (Refer to Fig. 7). Critical wedge sizes can be selected on the basis of considerations given in Fig. 6. Continuous, regular arches with good bond can be analyzed in a similar manner.

5.4 GOOD BOND

Thin shotcrete linings 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 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 ft wide and 2 ft deep.

Reinforcement can be beneficial for limiting crack widths and providing the post-crack resistance that permits a more ductile failure. 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 Peachtree Center Station clearly show that the 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.

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

TABLE 5   Two-dimensional Wedge Sizes for Adhesion Failure of Well-bonded Shotcrete Layer, 2 inches or More Thick

Adhesion values and the wedge sizes would be increased above those shown in Table 5 for bond conditions like those existing in the Peachtree Center Station. The Peachtree Center Station tests show that significant increases in capacity can be obtained by placing shotcrete layers in thicknesses of 4 to 8 in. The adhesive strength was great enough to develop the shear strength of the layer at thicknesses of 4 to 8 in.

6. CONCLUSIONS

The results of the studies on shotcrete behavior in loosening ground have provided quantitative estimates of shotcrete-rock bond strength and the coefficients for shear and thrust capacity for various lining configurations. The effectiveness of shotcrete for support in a tunnel depends strongly on the adhesive strength between shotcrete and rock. The exploratory program should be designed to provide information on the rock properties affecting shotcrete-rock adhesion, as well as on the properties that determine the size of the rock wedges requiring support. Sheared, planar surfaces are particularly critical.

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, adding to the support and observing the rock-shotcrete behavior until it is determined that an adequate support has been achieved.

This same characteristic of the shotcrete has made it difficult to predict exact requirements for its 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, and to determine the need for reinforcement and 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 depend on other factors, such as the maintenance requirements and permanence of the lining, and the potential exposure of personnel or the public in the cavern.

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STEEL-FIBER-REINFORCED SHOTCRETE
IN MARTA RUNNING TUNNELS

Jan Blanck
Shotcrete Contractor
Lanham, Maryland

(to be prepared)

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. The tunnel environment may be hostile to the equipment, however, so some thought must be given to protection of equipment. Due to high moisture and/or dust levels it would be ideal if the camera could be completely sealed as in equipment used for underwater work. Since this is generally not the case the next best thing is to be aware of the problem and to attempt to neutralize the adverse effects 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. This

will not only 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 principals 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 is better than a single, for example ground glass and split screen. 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 alternate 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 alternate 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 are 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 with the result of lowering 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. This 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 if 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 staionary 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. Automtic 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 expo-sure. 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 cumber-some; 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 condi-tions, the intended uses of the photos, and the preferences of the individuals involved. If high intensity lighting is avail-able, low speed films may be used; otherwise high speeds are re-commended. 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 protray 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:

Film	Exposure Time (seconds)		
	1	10	100
Kodacolor II	+1/2 Stop	+1/2 Stops cc 10c	+2 1/2 Stops & cc 10c & 10G
Kodacolor 400	+1/2 Stop	+1 Stop	+2 Stops
Ektachrome 200	+1/2 Stop & cc 10R	Not Recommended	Not Recommended
(Tungsten)			
Ektachrome 160	+1/2 Stop & cc 10R	+1 & cc 15R	Not 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 diferences 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° K
Flood Light (incandescent)   3200° - 3400° K
Day Light   500° K
Electronic Flash   6000° K
Skylight   10,000° K

If it is obvious that most of your work is going to 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 day-light 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

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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 reqired, 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 to include 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 exactly 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 of this 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 accomplished at the time of shooting by simply giving a litle extra thought to the set up and framing of the original. 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 is desired on one particular feature it may be advisable to take action to enhance it. This may be acomplished by 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 out-line, someone may be placed in the section to be shown. This person faces away from the camera holding a flash and either man-ually 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 true 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 sta-tion. This photographic outline can then be compared to a tem-plate of the design tunnel outline at the same scale, to reveal the presence of "tights" or other unacceptable as-built condi-tions.

We recommend that time be taken to shoot several test rolls trying as many different exposures and techniques as pos-sible. 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 pro-cessing procedures.

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Two publications which may be helpful are "Kodak Professional Photoguide"[1] and "Cave Photography"[2].

We wish to express our appreciation to Mssrs. 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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