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SUBSURFACE EXPLORATION METHODS FOR SOFT GROUND RAPID TRANSIT TUNNELS
Volume II: Appendixes A-F

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APRIL 1976
FINAL REPORT

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16. Abstract

The objectives of the Urban Mass Transportation Administration (UMTA) Tunneling Program are to lower subway construction costs and reduce construction hazards and damage to the environment. Some measure of each of these objectives for bored tunnels and deep excavations can be achieved through a more detailed knowledge of the subsurface and of how changes in soil types or characteristics will affect construction.

This study assesses subsurface exploration methods with respect to their ability to provide adequate data for the construction of rapid transit, soft-ground bored and cut-and-cover tunnels.

Geophysical and other exploration tools not now widely used in urban underground construction are investigated, their potential is discussed, and performance specifications and ideas for future development are presented. The effect of geotechnical variations on construction costs is modeled, and the effect of the prior knowledge of variation, including preliminary designs, specifications, cost estimates, and development plans, are formulated.

Volume one contains Sections 1-6 and all references. Volume two contains Appendixes A-F.

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PREFACE

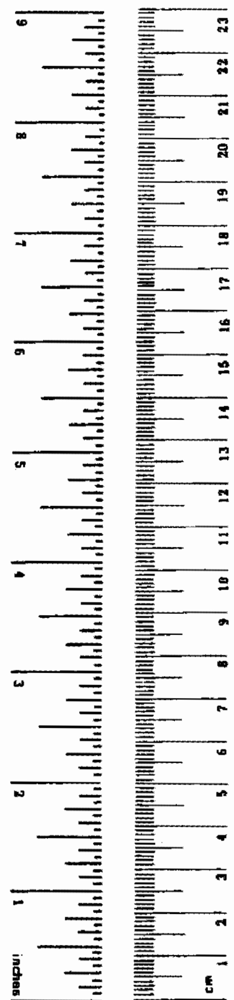
The study investigation of subsurface exploration methods for soft ground rapid transit tunnels, described in this two-volume report, was sponsored by the Rail Technology Division of the Urban Mass Transportation Administration, Office of Research and Development. The effort was conducted under contract with the Transportation Systems Center, contract DOT-TSC-654, for the Urban Rail Supporting Technology Program.

George Kovatch and Andrew Sluz were contract technical monitors for TSC. Birger Schmidt of Parsons, Brinckerhoff, Quade & Douglas, Inc. was Project Manager responsible for overall coordination and the principal writer of the sections 1 through 4. Bruno Matarazzi, Economist with Parsons, Brinckerhoff, Quade & Douglas, developed the economical analyses in Appendix A, assisted by Robert D. Budd of Mason and Hanger. The inventories and the detailed development of new methodologies, sections 5 and 6, were largely in the hands of C. John Dunnicliff, Chief Engineer, and Stephen A. Alsup, Geophysicist, both of Soil and Rock Instrumentation, Inc., with consultation services provided by Seismograph Services Corporation, Inc.

METRIC CONVERSION FACTORS

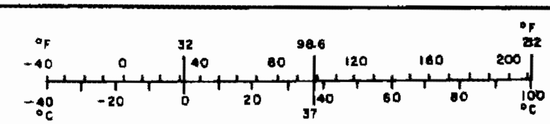
Approximate Conversions to Metric Measures

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



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	°F



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APPENDIX A - ECONOMICS OF SOFT GROUND TUNNELING

A.1 GENERAL CONCEPTS

A.1.1 Introduction

This section analyzes and measures the effect of variations of geotechnical parameters on the cost of soft ground tunneling. Variations in the value or nature of geotechnical parameters are translated into cost differentials to detect the most critical cost components. The ultimate objective is to ascertain whether a more precise knowledge of geotechnical variables may result in cost savings for the typical urban rapid transit tunneling project. To produce a qualitative as well as quantitative answer to this question a value analysis has been developed along the following line of logic:

1. Devise an equation or a system of equations correlating cost components and geotechnical parameters.
2. Quantify this system of equations in dollar terms by conventional cost estimating techniques.
3. Perform a sensitivity analysis of the parameter-cost correlation to find the most critical variables.
4. Measure or estimate the potential economic benefits that could be generated by a more precise identification and valuation of geotechnical parameters achieved by way of a refinement of the traditional subsurface exploration technology and methodology.

A.1.2 Tunnel Cost Components

The block scheme in figure A1 presents the traditional cost classification for tunnel projects. A typical percentage breakdown is also reported for the two major construction classes, Pneumatic and Nonpneumatic (compressed air and free air components). It eliminates the a priori cost factors that have minor influence, thus focusing the discussion on the most parameter-sensitive cost components.

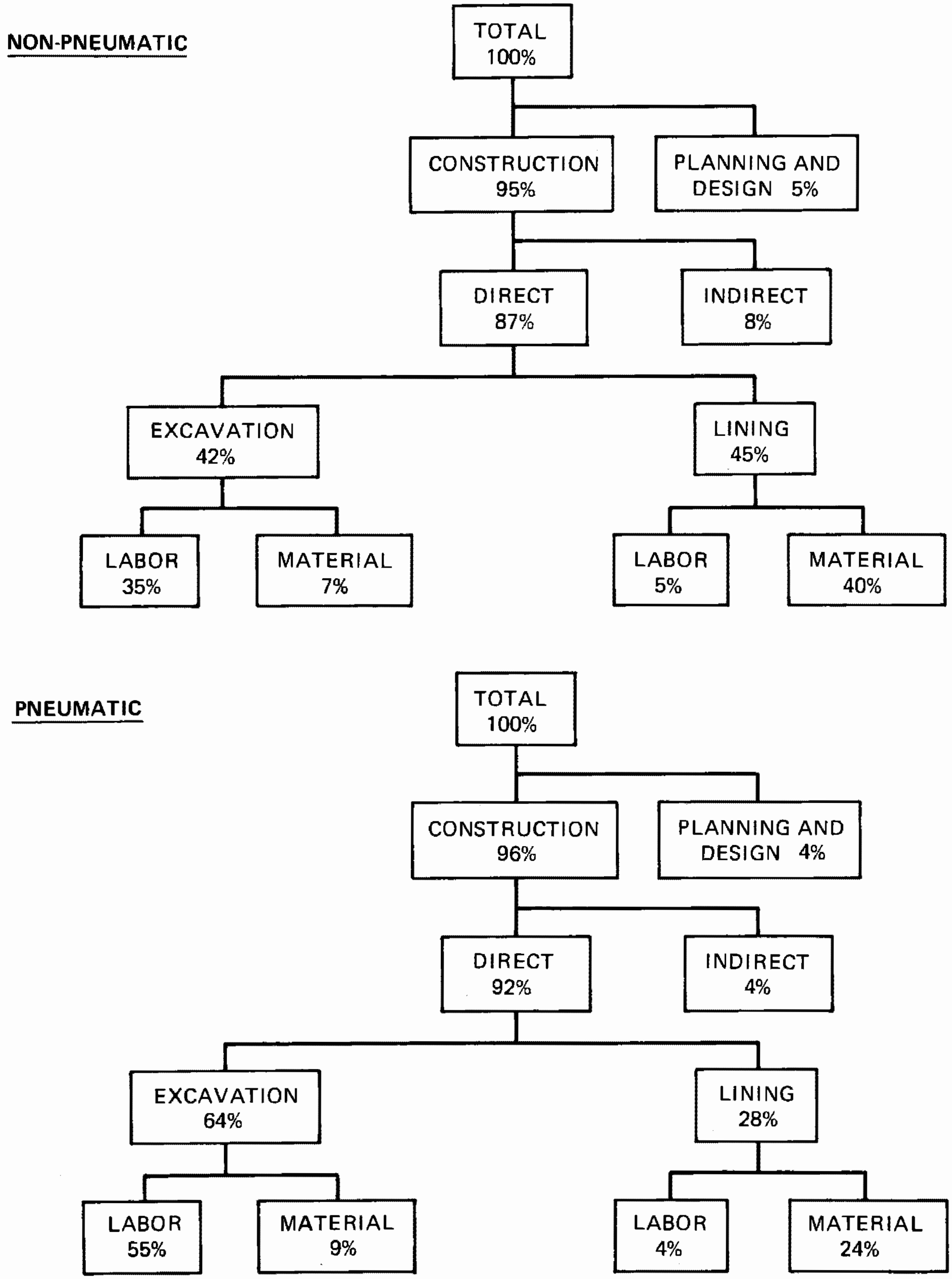


Figure A1. Typical Tunnel Cost Components.

There is no doubt that geotechnical variables affect planning and design costs; a complex geological stratification, heterogeneous subsurface soil conditions, and the presence of a water table will demand different exploration and engineering procedures. Yet planning and design costs for the typical urban rapid transit tunnel absorb only some 5% of the total project cost. A parameter variation generating a 20% differential in engineering cost will add or subtract a mere 1% to the total tunnel cost; nonetheless, the same parameter variation may generate a construction cost differential in the order of 100%. It is therefore assumed that for the purpose of this analysis, planning, and design costs can be disregarded.

Direct and indirect construction costs, broadly identifiable as excavation and lining cost, typically account for 95% of the total tunnel cost. In principle, lining costs, both labor and materials, should be parameter-dependent. Actually, traditional engineering and construction practices render the bulk of the lining cost for the typical tunnel practically insensitive to parameter variations, except for extreme cases. Accordingly, in developing a cost-parameter relationship, such cost elements will be treated as a constant or nearly constant entry.

Excavation cost is the crucial determinant of the total tunneling cost. A cursory examination of statistical data confirms this observation: for tunnels of similar geometry, unit excavation costs may vary by a factor of five or more, depending upon different soil conditions. A simple alteration of the soil conditions resulting in a 20 percent change in the rate of heading advance-and proportional increase in excavation labor cost - may generate differentials in the order of 10 to 15 percent of the total tunnel cost. Actual construction experience shows that variations of this magnitude in labor excavation cost are often exceeded.

Consequently, the following analyses will be focused on construction cost, with special emphasis on the correlations between parameter values and excavation labor costs.

A.1.3 Tunnel Cost Evaluation Criteria

Currently available literature on cost analysis of tunneling work is as a rule of a qualitative nature. Where an attempt has been made to present actual dollar figures, these are usually given in the form of a set of statistically averaged data, grouped on the basis of such characteristics as tunnel diameter and layout, soil conditions excavation techniques, etc. In other instances where a set of real cost data is directly reported it frequently consists of bid prices for specific tunnel projects.

For the purpose of calculating cost differentials for various geotechnical conditions, such cost series are not satisfactory. The use of statistically averaged costs precludes a sound analysis of the cost-parameter relation, because even the cost of tunnels of identical geometry and excavated in similar soil conditions may, for instance, vary substantially with wage rates, union rules and safety regulations. These three factors are highly variable in time and location. Thus, cost averaging of data collected over a certain, even narrow time and locus spread produces nearly meaningless results.

On the other hand, since the scope of this study is to produce conclusions valid for a typical urban tunnel and for an extensive range of parameter values, cost data retrieved from one or a few specific real cases would limit the significance and latitude of this investigation.

These obstacles have been circumvented by developing original cost estimates for a model tunnel, and for a wide set of geological, hydrological and soil conditions to be encountered in tunneling operations in an urban environment. Historical cost data will, however, be presented as documentary evidence to support the conclusions of this analysis.

The tunnel geometry and the geotechnical environment for such a typical tunnel have been defined with the following parameters:

1. Twin tunnel, 2 x 3000 ft., segmented steel lining

2. External diamter, 20 ft.

3. Depth of crown, 60 ft.

4. Maximum grade, 3%

5. Mechanical excavation, single heading

6. Various soil conditions and obstacles, above and below water table, in free air

7. Idem, with pressure range 0 to 44 psi in compressed air.

Original cost calculations have been made for this project. The results have been satisfactorily verified for accuracy and consistency with field engineers and construction men. These cost estimates may be expected to contain the same degree of accuracy as the usual tunnel bid price estimates. It is recalled at this point that the primary objective of this study is not to estimate with strict rigor total tunneling costs, but rather to calculate differential tunneling costs for various values of geotechnical variables. No consideration has been given to expenditures for underpinning of structures, access ramps and shafts, pavements, and other complementary work not directly related to the tunnel boring operation proper. Such additional cost may be very substantial, often more than 30% of the total tunnel project. The variable incidence of supplementary works is another explanation for frequency discrepancies in unit cost of tunneling projects, apparently carried out under identical conditions.

The cost estimates have been based on wage rates projected to the middle of 1975. Tunnel construction operations typically span over a period of several years, and contract documents, in most cases, preclude any wage differential adjustment above the original unit bid prices. It is, therefore, common practice among tunnel cost estimators to enter in their computation the expected value of wage rates at the time of midpoint construction, to account for inevitable wage escalation over the construction period.

On the other hand, current costs have been entered for material, equipment and supply. Such cost items are relatively stable, at least under normal economic conditions. (From 1965 to 1973 average hourly earnings in the contract construction sector rose 74.6%, while prices for intermediate industrial commodities increased only 43.4% according to Department of Commerce data). Tunnel bidders-contractors often cover themselves with contractual agreements with their suppliers, to get certain quantities of material at fixed prices, even over an extended period of time.

Inevitably, estimates of construction cost are subject to almost immediate obsolescence. In fact, even "true" cost data (i.e., bid prices), published in research reports of this nature are already obsolete before reaching the interested readership. The cost estimates here presented take 1975 as the base wage year, and thus possess a somewhat limited time validity. However, although the cost-parameter equations A.2.1 and A.2.2 do not directly allow readjustmants for changes in the labor wage base, it is possible to interpolate unit tunneling cost over a wide range of labor rates.

A.2 MODELING GEOTECHNICAL PARAMETERS AND TUNNELING COST

A.2.1 Introduction

The analytical approach to quantify cost-parameter relationships is based on the cost computations presented in paragraph A.2.2. The framework of this cost-parameter equation is based on the assumption that the total tunnel cost can be computed as the summation of one or more of the following costs:

1. Indirect cost
2. Construction cost in Free Air for various parameter values
3. Construction cost in Compressed Air for various pressure ranges
4. Cost of encountering an extraordinary obstacle.

The following primitive construction cost model does not intend to approximate closely the multiple function of a true cost model. This system of equations should simply be considered as a mathematical tool for quick calculation of tunneling cost and tunneling cost differentials generated by various geotechnical variables.

Ideally, the cost equations will have the form:

$$C_{total} = C_{free\ air} + C_{compressed\ air} + C_{indirect}$$
$$+ C_{extraordinary\ obstacles} \qquad (A.2.1)$$

It is thus possible to develop the total cost equations as a summation of the above four component elements. Where not otherwise stated, cost calculations are made per lineal foot of the tunnel.

Indirect Cost - Indirect costs, are defined as general overhead construction charges not directly identifiable with production operations, and represent, when computed on a daily basis, a fixed dollar amount, practically parameter insensitive. By definition:

$$\text{Indirect Unit Cost} = \frac{\text{Indirect Daily Cost}}{\text{Rate of Advance}} \qquad (A.2.2)$$

Since the daily rate of advance is a function of geotechnical variables; indirect unit cost, therefore, is a quantifiable function of soil parameters, and may be entered in the equation.

$$C_{direct\ labor} = \frac{15,550}{A} \quad (\$/ft) \qquad (A.2.3)$$

In a perfectly uniform soil, above water level and in absence of natural or man-made obstacles, the variable A, rate of advance, has been estimated to assume the following values (in ft/day):

36.6 in firm and medium clay
34.1 in soft clay
35.4 in silt
36.6 in sand, cohesive
34.1 in sand, non cohesive
28.0 in sand and gravel
26.9 in glacial till

By compounding the equations A.2.2 and A.2.3, it is possible to represent Unit Direct Labor and Indirect costs, as a function of the soil nature:

$$C = \frac{15,550}{A} + \frac{6,000}{A} = \frac{21,550}{A} \quad (\$/ft) \qquad (A.2.4)$$

By entering the mean value of the (nearly) fixed costs (material supply and miscellaneous fixed labor costs), the total unit cost in Free Air can be expressed as:

$$C = \frac{21,550}{A} + 1185 \quad (\$/ft) \tag{A.2.5}$$

Dewatering operations have been computed to add \$130 per linear foot (W) to the equation A.2.5. Likewise, the cost of boulders can be accounted for by introducing a boulder factor β. Since the effect of boulder presence is to slow down the daily advance rate, it will suffice to give β empirical values - suggested by construction experience - in order to enter in the equation A.2.5 the proper rate of advance: i.e., $\beta = 0.8$ or $\beta = 0.7$ for boulders resulting in a decrease of the advance rate of 20% and 30% respectively. The whole series of unit costs tabulated in table 3.3 can thus be represented by:

$$C = \frac{21,550}{A.\beta} + 1185 + W \quad (\$/ft) \tag{A.2.6}$$

Construction Cost in Compressed Air - Following the same logic outlined in discussing costs in free air, unit construction cost in compressed air can be represented by:

$$C = \frac{P + 6,000}{A.\beta} + 1830 \quad (\$/ft) \tag{A.2.7}$$

where P and A assume the following values, in function of the working pressure range.

Working Pressure (PSI)	P	A
0-14	25,650	23.4
14-22	26,580	23.4
22-32	36,980	22.0
32-38	38,350	21.0
38-44	39,420	20.0

The variable P represents the direct cost (\$/24 hour) of the excavation crew. Under different air pressure ranges. The other symbols have the same meanings as in A.2.6.

It is worth observing that, unlike the case of free air, in compressed air tunneling the advance rate is practically independent of the soil nature, barring obstructions, the working pressure value range is the critical factor.

Cost of Encountering an Extraordinary Obstacle. Lump sum costs for a series of obstacles of an occasional nature are listed below: In principle, this cost category should contain a list of events ad infinitum. Calculations have here been limited to a selection of the most likely obstacles to be encountered. Such cost estimates are given as typical only, actual costs may range from a fraction to 100% of the figures here shown depending upon a precise definition of the obstacle and other variables. The rarity of such obstacles in the typical rapid transit urban conditions and their relative insignificance in the total tunneling cost renders meaningless a deeper dissection of this cost item.

E_1 =	live sewer	\$120,000
E_2 =	dead sewer	\$ 30,000
E_3 =	cable bank	\$ 50,000
E_4 =	gas main	\$ 15,000
E_5 =	concrete pile	\$ 3,000

Such estimates may be entered in the Equation 1. as:

Where $E_1 \ldots E_n$ are the lump shown above, and the constant $K_1 \ldots K_n$ can be given values 1 or 0 respectively to indicate the presence or absence of the relative cost items.

The unit cost equation A.2.1 can be mathematized as:

Unit construction cost in Free Air

$$C = \frac{21,550}{A.\beta} + 1185 + W + \Sigma \kappa_i E_i \ (\$/ft) \qquad (A.2.8)$$

Unit construction cost in Compressed Air

$$C = \frac{P + 6,000}{A.\beta} + 1830 + \Sigma \kappa_i E_i \ (\$/ft) \qquad (A.2.9)$$

And for the total construction cost:

$$C = \sum_{1}^{6,000} \left(\frac{21,550}{A.\beta} + 1185 + W + \Sigma\, \kappa_i E_i\right)$$

$$C = \sum_{1}^{6,000} \left(\frac{P + 6,000}{A.\beta} + 1830 + \Sigma\, \kappa_i E_i\right)$$

Note the necessity of the sum (Σ) operation to take into account the fact that different sections of the 2 x 3000 tunnel under consideration may be given different values of the A, P, W, β factors.

A.2.2 Tunneling Cost Computations

The purpose of these cost estimates is to compute with ground tunneling cost as a function of geotechnical variables. These are original cost estimates expressly developed for this research project.

A fictitious tunnel has been selected, whose geometry is representative of a typical rapid transit tunnel. A series of construction costs have been calculated for several sets of geotechnical variables. In addition, a set of cost-parameter equations has been developed allowing the calculation of tunnel cost for any generic parameter values and combinations. The material included covers the numerical computations for estimating direct and indirect tunnel construction cost, and is directly reported in a manual form, a usual procedure in developing construction cost estimates. These calculations contain very fine numerical details. It is assumed that the reader interested in analyzing such estimates will be familiar enough with estimating tunnel construction work, to be able to interpret the various steps of the analysis without further commentary.

It is fair to mention that, while direct construction cost has been computed with the most exacting accuracy, somewhat less meticulous criteria have been used in estimating indirect costs such as management, clerical and other ancillary functions and equipment not directly related to construction operations. Daily overheads have in fact been interpolated from actual estimates for a branch of the Washington Metro. This does not affect the accuracy and validity of the calculations, because daily overhead components and cost factors are independent of geotechnical variables.

1. The tunnel has been defined with the following characteristics:

   a. Twin tunnel 2 x 3000 ft.

   b. Steel lining, external diameter 20 ft.

   c. Depth at crown 60 ft.

   d. Maximum grade 3%

   e. Mechanical excavation, single heading

   f. Various soft soil conditions, various obstacles and water conditions (free air)

   g. Idem, with compressed air ranging from 1 to 44 psi pressure. (The higher pressures would, in fact, not be needed in a 60 feet deep tunnel.)

   h. Labor rates, union rules and safety regulations, in effect in Washington, D.C. as of May, 1973.

2. Detailed calculations have been carried out for the following conditions:

   a. Tunneling in Free Air:

      1. firm/medium clay
      2. soft clay
      3. silt
      4. sand, cohesive
      5. sand, non-cohesive
      6. sand and gravel
      7. galcial till

      Costs have been calculated above and below water table and in the presence of boulders of various sizes and frequencies.

b. Tunneling in Compressed Air

Various combinations of soils, obstacles and boulders, for the following compressed air working ranges:

1. 0-14 psi
2. 14-22 psi
3. 22-32 psi
4. 32-38 psi
5. 38-44 psi

A.2.3 Cost Data Tunneling in Free Air

1. General. 20 Ft. OD of Shield - 19'6" OD C.I. or Steel Lining

Excavated area for shield

$10^2 \times \pi$ = 314 SF	or 11.6 CY/LF

Area & Vol. of lining

$9.75^2 \times \pi$ = 298.50 SF	or 11.05 CY/LF
Volume of Grout	0.575 CY/LF
Allow 20% Extra 0.525 x 1.2 =	0.689 CY/LF

Excavated Volume per 2.5' Shove = 2.5 x 11.6 CY = 29.00 CY/Shove

Grout Required Per Shove = 2.5 x 0.63 CY = 1.58 CY/Shove

Wheel Excavator in Firm & Medium Clay Excavates 2"/Min.

Time to Excavate 2.5 Ft	= 30" ÷ 2"/Min	15 Min.
Lining Erection 8 Segments @ 4 Min.		32 Min.
1 Key at 4 Min.		4 Min.
Delay for Train change		5 Min.
		56 Min.

Total Minutes/24 Hour Day		1,440
Lost Time - Deck & Track	90 Min.	
3 Lunch Periods @ 30 Min.	90 Min.	
Repair & Service Machine (4 hr.)	240 Min.	
		420 Min.
Max. Productive Time		1,020 Min.

Maximum Progress $\frac{1{,}020 \text{ Min.}}{56 \text{ Min.}}$ = 18.21 Cycles of 2.5 Ft. = 45.53 LF/Day

Average Progress 90% of 45.53 - Say 41 Ft./Day
(After break in period at start up.)

Allow 15 days break in period @ 17 LF/day average.

Total length driven 3,000 LF	=	3,000 LF	
Break in 15 days @17 LF/Day	=	255 LF	= 15 Days
Production Driving		2,745 LF @ 41'/Day	= 67 Days
		Total Driving Period	82 Days

Average Advance $\frac{3{,}000 \text{ LF}}{82 \text{ Days}}$ = 36.59 LF/Day

Each Shove = 23.56 CY Solid x 1.35 Swell = 31.81 CY Loose

$\frac{31.81 \text{ CY}}{4 \text{ CY/Car}}$ = 7.95 Cars/Shove Say 2 trips of 4 cars.

Maximum grade 3% against loads.

4 CY/Car x 3,000/LB/CY	=	12,000
4 CY Car Tare		6,000
		18,000 or 9 Tons/Car

Max. Train 5 Cars @ 9 Tons	=	45 Tons
Locomotive 15 Ton	=	15 Tons
Train Weight	=	60 Tons

Rolling Friction, Level	=	20#/Ton
Grade Effect 3 x 20#/Ton	=	60#/Ton
Allow 30 Lb. Ton Acceleration	=	30#/Ton
Total	=	110#/Ton

60 Tons x 110/Ton = 6,600 Lb. Tractive Effort

6,600# T.E. x 4 = 26,400 Lb. Min. Locomotive Weight

Recommend 15 Ton Loco.
One Train Change per Round

Shaft Hoisting Rate 4 CY Car @ 4 Min.

Average Cycle	56 Min.
Hoist 8 Cars @ 4 Min.	32 Min.
For Other Supply	24 Min.

If average cycle is exceeded second crane to handle supplies will pay for itself.

Grout

There should be no difficulty in placing grout behind liner by grouting each shift.

Average advance 36.6 feet x 0.63 = 23.06 CY/3 Shift Day or say 7.68 CY/Shift

Using 6 grout men. Each man must place 2 CY/Shift

7.68 CY Grout = 7.68 x 27 = 208 Sacks Pea Gravel/Shift
Using 2 men on - 104 Sacks per Man/Shift.

Allowing 6 hours working time on grout = 17 Sacks/Hr.

Mechanized placing hopper cars will simplify.

Cement Grout - 1/2 Pea Stone

No problem for 2 men to mix and place.

Use

1 Foreman
2 Miners on Grout Hoses
4 Men on Materials
1 Man on Grout Cocks
8 Men Total

Erecting Iron (Also extend deck and rail.)

1 Foreman
2 Men Handle Materials
2 Men Plates to Erector
2 Men Set and First Bolt Plates
2 Men Completing Bolt Up
2 Men Back Bolt
11 Men Iron

Haulage

Avg. 3 Locomotive Oper.
3 Brake
6

1 Shield Driver
1 Wheel Oper.
1 Mech.
2 Electricians
5

Permanent Materials

Cost of 19"-6' Ductile Iron 30" Segmented

Primary Lining Delivered	\$576.68
Bolts	\$ 30.69
Caulking Lead	\$ 17.85
Bonds	\$ .70
	\$625.92
Grout Materials 32.00	\$ 32.00
Say	\$658.00

Job Materials

Labor x 5% For Hand Tools Say	\$ 15.00 LF
Cutter Bits $\frac{60 \text{ Bits @ } 25.00}{200 \text{ LF}}$ =	\$ 8.00 LF
Grout Supplies 8.00 x $\frac{19.5}{17.5}$ =	\$ 9.00 LF
Electrical Supplies	\$ 8.00 LF
	\$ 40.00 LF

Pumping (Driving in Firm and Medium Clay)
Pumping tests taken in advance of design and bidding indicate that no dewatering before driving is required.

Pumping required will be limited to pumping at face, and at construction shafts.

Driving Period	82 Days
TBM Erection & Dismantle	30 Days
	112 Days Pumping

112 Days x 3 Shifts x 2 Men/Shift Avg. = 672 Pump Shifts

		Unit on 3,000 LF
672 x 69.44	= $46,664 Labor	$ 16.00
672 x 8.37	= $ 5,624 Fringe	2.00
19% Labor	= $ 8,866 Tax & Comp.	3.00
	Total Pumping	$ 21.00

2.. Firm or Medium Clay - Costing Impact. These classifications can be combined, because, with the use of a tunnel boring machine, the rate of advance would be the same in both cases.

a. Little or no difference in bit life would exist.
b. Power requirements might vary, but not significantly.
c. Repair parts would not increase.
d. The other cost items are not effected.

Progress - 36.6 LF/Day Average.

3. Soft Clay - Costing Impact. By definition, moist, plastic, not requiring dewatering by external pumping. Material of this type will require breasting or a closed face wheel type tunnel boring machine.

Soft sticky clay may cause some problem in cleaning the conveyors and in changing cutter teeth.

The following costs will change.

#1 - Labor Slower Progress

#2 - Fringe Slower Progress

#3 - Tax & Insurance Slower Progress

#8 - Equipment Charges - Longer Rental Period

#9 - Tunnel Facilities - Increased Cost of Closed Face Machine (All other costs remain the same.)

Tunnel Progress

Cycle in Firm Clay	56 Min.
Add for Conveyor Clean up	5 Min.
	61 Min.

$$\frac{\text{1,020 Min. Day Working Time}}{\text{61 Min./Cycle}} = \text{16.72 Cycles @ 2.5 Ft.} = \text{41.8 Ft. Day}$$

Use 90% Eff. Factor = 37.62 Ft.

Driving 255 LF @ 17 Ft./Day	=	15 Days
Driving 2,745 LF @ 37.62 Ft./Day	=	73 Days
Total Driving		88 Days

$$\frac{\text{3,000 LF}}{88} = \text{34.1 LF/Day}$$

Equipment Rentals $1,616.00 Day - 34.1 Ft./Day = $47.39
Say $47.00

Tunnel Facilities

Open Face Boring Machine	900,000	
Add For Face Enclosure	60,000	
Total Cost	960,000	
Salvage	200,000	
Job Charge	$760,000	- 6,000 LF = 127.00 LF

Extra Cost of Closed Face Machine $10.00 LF

4. Silty Clay & Silt Above Water Table - Costing Impact. By definition, both of these materials lie above the existing water table. There is no predictable difference between these materials. Costs lie between firm clay and soft clay. A straight average will be used between these limits. Average advance 35.35 LF/Day.

5. Silty Clay and Silt Below Water Table - Costing Impact. Under certain conditions sand seams or the natural consistency of silty clay and silts which lie below the water table permit control of ground water with eductors installed from the surface. This is a rare occurrence but the additional for pumping has been shown under this heading.

   Costs for silty clay and silt below the water table which will require driving in compressed air will be computed in another section.

6. <u>Sand (Cohesive) Above Water Table - Costing Impact</u>. In this type of material average progress would be the same as for firm-medium clays, that is 36.6 Ft/day.

No difference in costs except on job materials and repair parts.

a. <u>Materials</u>.

Increase Cutter Bit Cost on Job	8.00 LF
Job Materials Firm Clay	45.00 LF
Job Materials in Sand Use	53.00 LF

b. <u>Repair Parts</u>.

Repair Parts Increase 10%	
Repair Parts in Clay	39.00
Add for Sand	4.00
	$43.00

7. <u>Sand (Cohesive) Below Water Table Subject to Dewatering - Costing Impact</u>. Progress will be same as cohesive sand 36.6 Ft./Day Avg.

All costs except pumping will be same as for cohesive sand above water table.

Install Eductor System 3,000 LF	=	$112,500
Pumping 11 Months @ $21,000/Mo.	=	231,000
Total Pumping		$343,500
Cost per LF on 3,000 LF	=	$115.00
Add Tunnel & Shaft Pumping	=	21.00
Total Pumping	=	$136.00

8. <u>Sand - Non-Cohesive Above Water Table - Costing Impact</u>. Same progress as for soft clay or 34' LF/Day Average.

a. Close face tunnel boring machine head should be provided.

b. Use Tunnel Facilities 170.00 LF for closed face.

c. Use Job Materials same as for cohesive sand or $53.00/LF.

d. Use Repair Parts same as for cohesive sand or $43.00/LF.

e. Use Equipment Rental same as for cohesive sand or $49.00 LF.

9. Non-Cohesive Sand Below Water Table Susceptible to Dewatering - Costing Impact. All costs, except Pumping, will be same as for non-cohesive sand above water table.

   a. Average Progress 34.1 Ft./Day.

   b. Pumping costs can vary widely. This will be displayed on our Detailed Unit Cost set up.

10. Sand & Gravel Above Water Table - Costing Impact.

   a. Progress (Average).
   Wheel Excavator will excavate 1-1/2"min. in sand and gravel.

Time to excavate 2.5 Ft. = 30" ÷ 1.5"/Min.	=	20 Min.
Erecting Ring	=	36 Min.
Delay for train change	=	5 Min.
Total cycle per ring	=	61 Min.
Add for occasional boulder 10%		6 Min.
Average cycle		67 Min.

24 Hr. Day		1,440 Min.
Lost time		
Deck & Track Extension	90 Min.	
3 Lunch Periods @ 30 Min.	90 Min.	
Repair & Maint. TBM	300 Min.	
		480 Min.
Total Available Productive Time		960 Min.

$$\frac{960 \text{ Min.}}{67 \text{ Min./Cycle}} = 14.33 \text{ Cycles/Day of 2.5 Ft.} = 35.83 \text{ LF/Day}$$

Average Progress 90% of 35.82' = 32.35 LF/Day
(After break-in period and start-up)

Drive during start up 255 LF @ 12' Day	=	22 Days
Drive 2745 LF @ 32.24 LF/Day	=	85 Days
		107 Days

Drive $\frac{3{,}000 \text{ LF}}{107 \text{ Days}}$ = 28.0 LF/Day Average Advance

Note: In most sand and gravel strata occasional bounders can be expected.

b. Costing Extensions.

Labor as developed from Progress

Fringes as developed from Progress

Tax & Insurance as developed from Progress

Permanent Materials Constant

Job Materials - Double Cutter Bit Costs

5% x 435 Labor	= Say	22.00
Cutter Bits	=	16.00
Grout Supplies	=	9.00
Electrical Supplies	=	8.00
Total	=	55.00
F, L & P as developed from Progess	=	22.00
Extra power on wheel 12.52 @ 16%	=	2.00
Total Job Materials		24.00

Repair Parts as developed from Progress

Equipment as developed from Progress

Tunnel Facilities Constant

Muck Disposal Constant

Pumping 21.00 x $\frac{36.7}{28}$ = 28.00

11. Sand & Gravel Below Water Table - Costing Impact

a. Dewater by Pumping From Surface.

Cost of installation of 3,000' header and wells	$112,500	
Pumping 9 Mo. @ 21,000/mo.	189,000	
Total Cost for 3,000 Ft.	$301,500	
or		101.00 LF
Cost of Pumping in Tunnel		28.00 LF
		129.00 LF

b. All other costs same as for sand and gravel above water table.

12. Glacial Till Above Water Table - Costing Impact. This material when dry and mechanically mined would permit the same advance as cohesive sand. However, frequent boulders are to be expected. Frequent boulders suggest the use of a mechanical hoe type excavator mounted in the shield. The hoe type excavator will excavate at same rate as a wheel type excavator.

On a contract recently observed, where such a machine was in use, 33% of total time was spent on boulders. Advance was thus only 67% of that which could be realized in boulder free ground.

Thus progress estimated for cohesive sand would extrapolate to till as 36.6 LF/Day x 0.67 = 24.5 LF/Day.

Use: 24 LF/Day.

Cost of Hoe Type Excavator same as wheel type.

Use: $160.00/LF

a. Job Materials. While hoe teeth cost less than bits for wheel type excavator, extra cost for tools for cracking boulders will offset this.

1. Use same Job Material Cost as for wheel.
2. F&L - reduce cost in sand by 10%.
3. Repair Parts - Equal cost for sand.
4. Power costs reduce x 10%.

b. Pumping.

Driving 3,000 LF tunnel at 24 LF/Day	=	125 Days
Add for shield set up and dismantle	=	30 Days
Total Working Days	=	155 Days
155 working days ÷ 21 working days	=	7.38 Month
Add for prepump	=	1.62 Month
		9 Months

Exterior Pumping		
Install 1,500 LF Header & Wells	=	$ 56,250
Testing 1,500 LF	=	28,125
Pumping 9 Mo. at $21,000/mo.	=	189,000
Total for 3,000 LF Exterior pumping	=	$273,375 or $91.00/LF
Pumping in Tunnel		
21.00 LF x $\frac{36.6}{24}$	=	32.00 LF
Total Pumping	=	123.00 LF

A.2.4 Cost Data Tunneling in Compressed Air

Shield 20".

Grade 3% Max.

Length 3,000 LF.

Use Tunnel boring machine.

Shaft - large enough to put in boring machine in free air.

Ground - subject to dewatering for first 250 LF.

Ground - requires compressed air for 2,750 LF.

Maximum Air Pressure 14 psi.

Duration of shift 6 hours.

Estimates based on 0 to 14 psi air pressure.

Materials, if worked in compressed air, little or no difference in cost for:

1. Firm Clay
2. Medium Clay
3. Clayey Silt
4. Silt
5. Sand Cohesive
6. Sand Noncohesive

All of these will be considered as equivalent materials and worked with closed face tunnel boring machine.

Sand and gravel with occasional boulders will be costed for such a condition.

Glacial till and boulders will require hoe type excavator and mechanical face breasting and will be costed for this condition.

Progress in Compressed Air 0 to 14 psi.

Using closed face tunnel boring machine rate of advance will be 1.5"/Min.

Cycle for 30" ring or 23.6 Solid CY.

Boring Machine 30" @ 1.5"/Min	=	20 Min.
Dry and Clean Tail of Shield with blow pipe	=	10 Min.
Erecting Ring	=	36 Min.
Train Changes	=	5 Min.
Total Cycle Possible	=	71 Min.

Working Time		
Day		1,440 Min./Day
Lost Time Track & Deck Extension	90	
Lunch Periods 3 @ 30 Min.	90	
Repair & Service TBM in Comp. Air	300	
Average Lost Time Breasting	36	
Lock Delays Men & Materials	120	636 Min./Day
Total Available Working Time		804 Min./Day

$\frac{804 \text{ Min./Day Work Time}}{71 \text{ Min./Cycle}}$ = 11.32 Cycles @ 2.5' = 28.3 Ft./Day.

Allowing 90% Overall Eff. = 25.48 Ft./Day Average

Drive in Free Air 255 LF @ 13.4 Ft./Day During Start	=	19 Days
Drive in Comp. Air 2,745 LF @ 25.48 Ft./Day	=	108 Days
Total Driving Period		127 Days

Average Advance $\frac{3,000 \text{ LF}}{127}$ = 23.62 Ft./Day Average

A.3 GEOTECHNICAL PARAMETERS AND COST COMPONENTS

A.3.1 Introduction

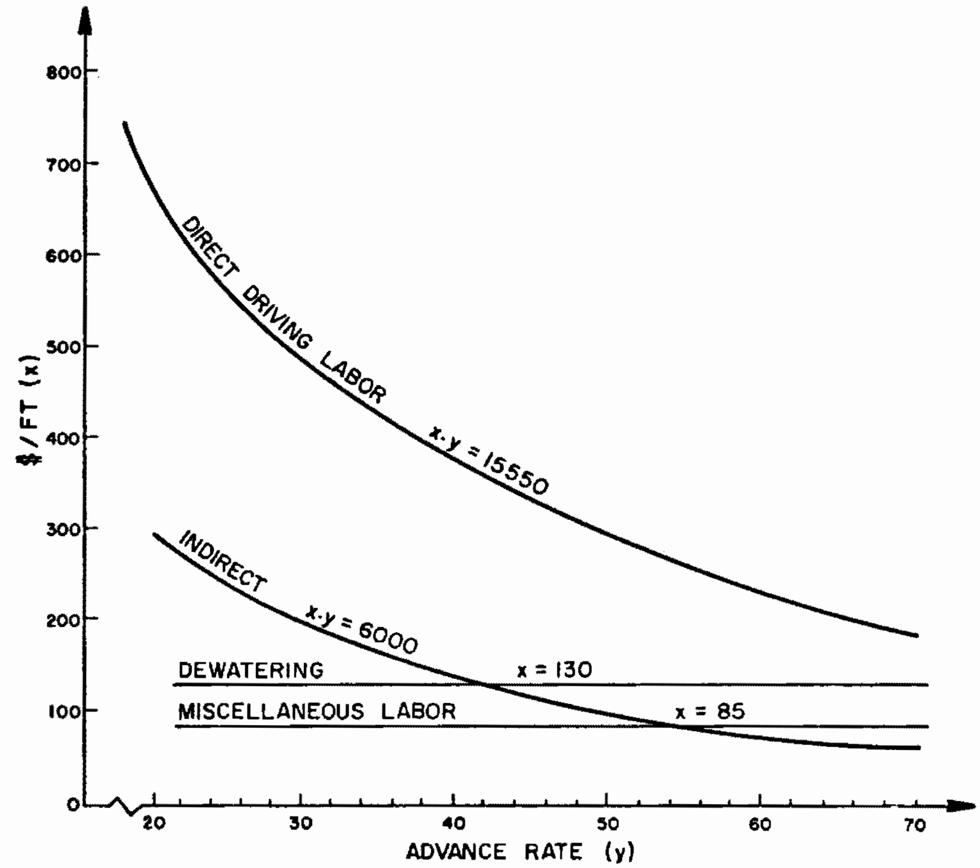
The computations in this appendix permit us to dissect a series of unit tunnel costs into their major components. Such a cost breakdown is reported in figures A2 and A3 for free air and compressed air respectively. Table A1 contains typical tunneling costs for A, B, P, and W as used here and defined in equations A.2.1 through A.2.9 in paragraph A.2. The purpose of such a tabulation is to analyze the relative incidence of various cost components as a function of geological, hydrological and soil variables for the tunnel of the typical geometrical characteristics considered.

In principle, it might appear appropriate to correlate costs with such numerically identifiable parameters as permeability, specific weight, granularity, friction angle, cohesion, etc. Due to the complex interaction of these parameters, however, it is practically impossible to define soil conditions with numerical parameters only. Thus, the necessity arises for a conventional nomenclature to take into account peculiar physical characteristics not exactly measurable.

An analysis of the various unit cost data condensed in figures A2 and A3 immediately demonstrates a few general conclusions. For the sake of clarity the cases of free air and compressed air will be considered separately. All the dollar figures here reported, except where otherwise indicated, represent costs per linear foot of single tunnel. For simplicity, all costs are reduced to round figures.

A.3.2 Cost of Material, Equipment and Supply ($1000 to $1100 in Free Air; $1200 to $1400 in Compressed Air)

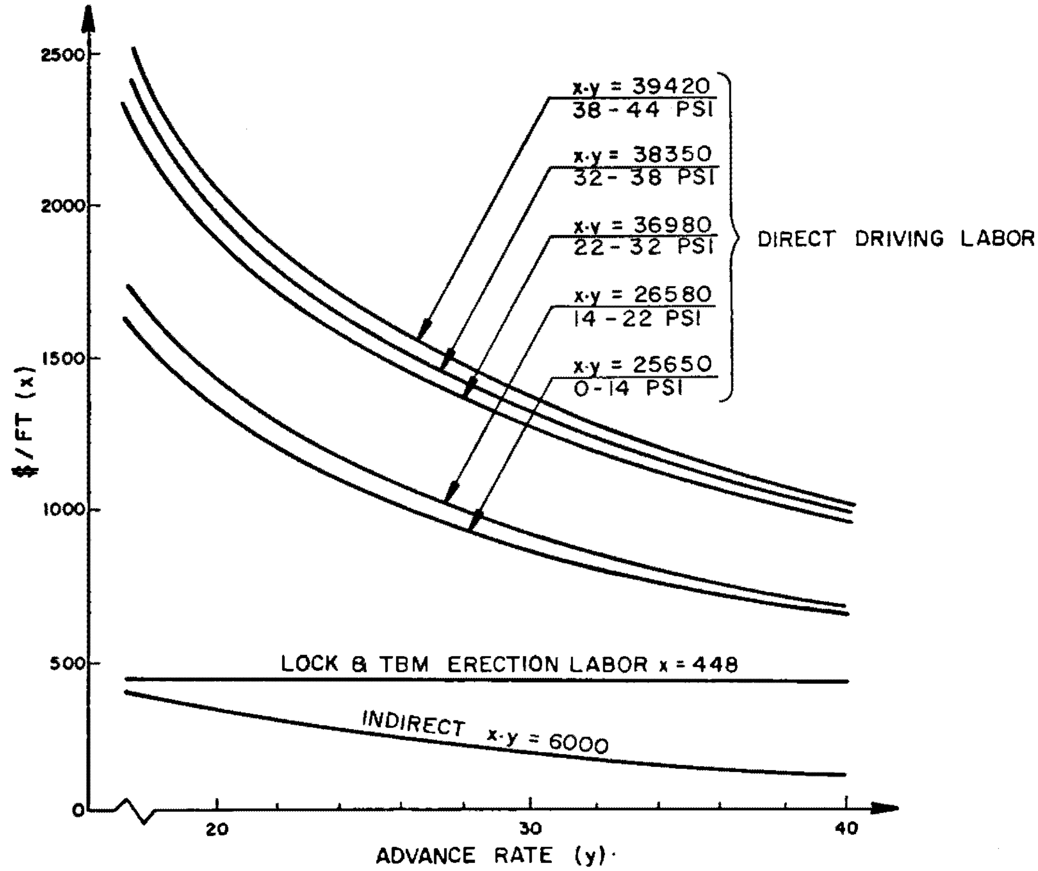
This cost component includes all non-labor expenditures. In free air, as well as in compressed air, such cost may be considered nearly constant (±10%), a breakdown by cost subcomponent explains why.



UNIT COST BREAKDOWN ($/FT)

	Firm/Medium Clay	Soft Clay	Silt	Sand (cohesive)	Sand (non-cohesive)	Sand and Gravel	Glacial Till
Advance Rate (ft/day)	36.6	34.1	35.4	36.6	34.1	28.0	26.0
Driving Labor	425	457	442	425	457	555	598
Other Labor	85	85	85	85	85	85	85
Total Direct Labor	510	542	527	510	542	640	683
Material, Equipment and Supplies	1038	1046	1049	1050	1060	1090	1130
Total Direct	1548	1588	1576	1560	1602	1730	1813
Indirect	164	176	169	164	176	214	231
Total	1712	1764	1745	1724	1778	1944	2044

Figure A2. Unit Construction Cost, 2 x 3000 Ft. Tunnel, Diameter 20 Ft., Steel Lining (Free Air).



Pressure Range (psi)	0-14	14-22	22-32	32-38	38-44
Advance Rate (ft/day)	23.4	23.4	22.0	21.0	20.0
Driving Labor	1096	1136	1681	1821	1971
Lock and TBM Labor	448	448	448	448	448
Total Direct Labor	1544	1584	2129	2269	2419
Material, Equipment and Supplies	1235	1249	1315	1350	1350
Total Direct	2779	2833	3444	3619	3769
Indirect Driving Labor	256	256	272	285	300
Indirect Lock and TBM Labor	32	32	32	32	32
Total Indirect Labor	288	288	304	317	332
Total	3067	3121	3748	3936	4101

Figure A3. Unit Construction Cost, 2 x 3000 Ft. Tunnel Diameter 20 Ft., Steel Lining (Compressed Air).

TABLE A1. TYPICAL TUNNELING COST

(2 x 3000 ft., 20 ft., Diameter Steel Lining)
- based on 1975 Labor rates in Washington D.C. -

FREE AIR

Soil Condition	Dewatering** Extra Cost	Boulders (β=0.8) Extra Cost	Total Cost* ($)
Firm & Medium Clay	---	---	10,270,000
Soft clay	---	---	10,585,000
Silt	730,000	---	10,470,000
Sand (Cohesive)	780,000	---	10,345,000
Sand - (Non-Cohesive)	780,000	---	10,670,000
Sand & Gravel	1,200,000	1,250,000	11,665,000
Glacial Till	1,200,000	1,400,000	12,265,000

*Without dewatering and boulders

**Dewatering costs are highly variable; typically they range from $130 to $200 per linear foot of tunnel.

$$C = \frac{15,550 + 6,000}{A\beta} + 1,185 + W \text{ (Equation A.2.1)}$$

COMPRESSED AIR

Working Pressure (PSI)	Boulders (β=0.8) Extra Cost	Total Cost* ($)
0 - 14	2,700,000	18,400,000
14 - 22	2,760,000	18,670,000
22 - 32	2,930,000	22,700,000
32 - 38	3,140,000	23,675,000
38 - 44	3,170,000	24,845,000

*Without boulders.

$$C = \frac{P + 6,000}{A\beta} + 1,830 \text{ (Equation A.2.2)}$$

1. <u>Fixed materials</u> (primarily lining materials, $660 in free air, $660 in compressed air). Under current engineering practices, design of a tunnel lining and thus its cost, is often independent, except for a few cost irrelevant details, of geotechnical variables.

2. <u>Equipment</u> (Construction equipment; tunnel boring machine and any accessory plant equipment and tools, $250-280 in free air, $400-450 in compressed air). Although different soil variables require machinery of different features, cost differentials for machines of the same basic design but dissimilar details are in the order of $200,000-$400,000. As substantial as this amount may appear, this is $30-$60 per tunnel foot, a small fraction of the total unit cost. Compressed air tunneling requires additional equipment, compressed air plant, locks, bulkheads, etc. This accounts for the $200 difference in equipment cost between free air and compressed air.

3. <u>Supply</u> (construction materials, repair parts, oil and lube, energy cost; $100-200 in free air, $200-$300 in compressed air). This is the sole non-labor cost component that depends on geotechnical conditions, and, within certain limitations, is directly related to the daily rate of advance. Longer operation time per unit length of tunnel requires larger consumption of supplies, as shown above. This difference between free air and compressed air for this cost component is due to the larger energy requirements and generally more mechanically complicated equipment required in compressed air operations.

A substantial quota of non-labor cost in the order of 80 to 90% may thus be assumed by and large insensitive to geotechnical conditions. In this cost category, differences between compressed air and free air, originate mainly from the cost of supply.

Only 10-20% of non-labor costs appears to be contingent on geotechnical parameters. Whatever the value of such parameters and the rate of advance attained there is a $900-$1,000 basic cost floor that cannot be lowered. This floor represents lining materials and equipment amortization costs, which are relatively independent of geotechnical conditions.

A.3.3 Cost of Labor

A cursory examination of figures A2 and A3, attests to the dramatic dependence of labor cost upon geotechnical conditions. The daily rate of advance is the critical factor linking labor cost and soil characteristics. Labor cost components will be commented on in accordance with the breakdown shown in table A2, for free air and compressed air.

1. Fixed labor cost ($100 in free air, $450 in compressed air.) This is the fraction of total labor cost largely independent of soil conditions, in free air or compressed air operations. Such cost nevertheless increases dramatically from free air to compressed air. This item represents cost of manpower for equipment erection and dismantling (tunnel boring machine, compressed air, ventilation, and power plants). The large difference between free air and compressed air is due to the more complex equipment set up in compressed air.

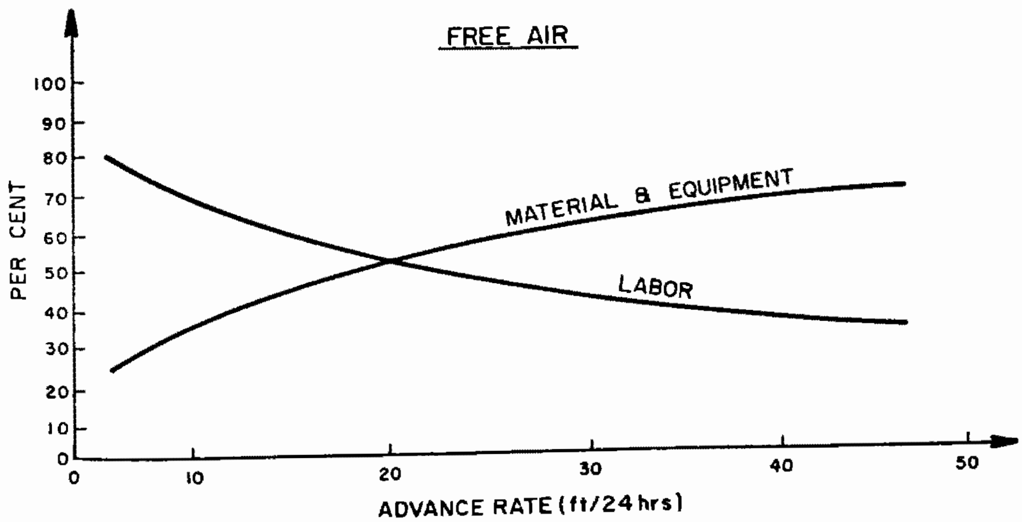
2. Variable labor cost ($300 to $2000 in free air, $1,500 to $4,000 in compressed air). This cost includes direct labor costs strictly attributable to driving operations, plus indirect labor costs (in fact a small fraction of ancillary equipment has been included in indirect labor to simplify calculations). This is the item that produces very large cost differentials even in construction of tunnels of similar geometry. Table A2 and figure A4 show variable labor cost as a function of the rate of advance, with the warning that while rates of advance below 10 ft/day are a common occurrence, rates of advance above 40 ft/day in compressed air and 80 ft/day in free air are seldom achieved today. By and large, for a given tunnel geometry, in free air the set-up of the excavation crew, and thus its cost per day, is independent of geotechnical parameters;

TABLE A2. LABOR AND NON-LABOR COST

(as a function of Hypothetical Rates of Advance
2 x 3000 ft., diameter -20 ft. Tunnel)

ROA	LABOR (fixed & variable)	NON-LABOR	TOTAL
	COST ($/ft)		
	FREE AIR TUNNELING		
10	2,230	1,250	3,480
20	1,160	1,150	2,310
30	800	1,080	1,880
40	620	1,020	1,640
60	430	1,020	1,450
80	350	1,010	1,360
100	300	1,000	1,300
120	260	990	1,250
140	240	980	1,220
160	220	970	1,190
180	200	960	1,160
200	180	950	1,140
250	170	940	1,110
300	150	930	1,080
400	140	920	1,060

ROA	LABOR (fixed & variable)	NON-LABOR	TOTAL
	COST ($/ft)		
	COMPRESSED AIR TUNNELING (22-32 psi)		
10	4,140	1,420	5,560
15	2,910	1,350	4,260
20	2,290	1,330	3,620
30	1,670	1,320	2,990
40	1,370	1,310	2,680
50	1,190	1,300	2,490
60	1,060	1,290	2,350
70	970	1,270	2,240
80	910	1,250	2,160
90	860	1,230	2,090
100	820	1,220	2,040



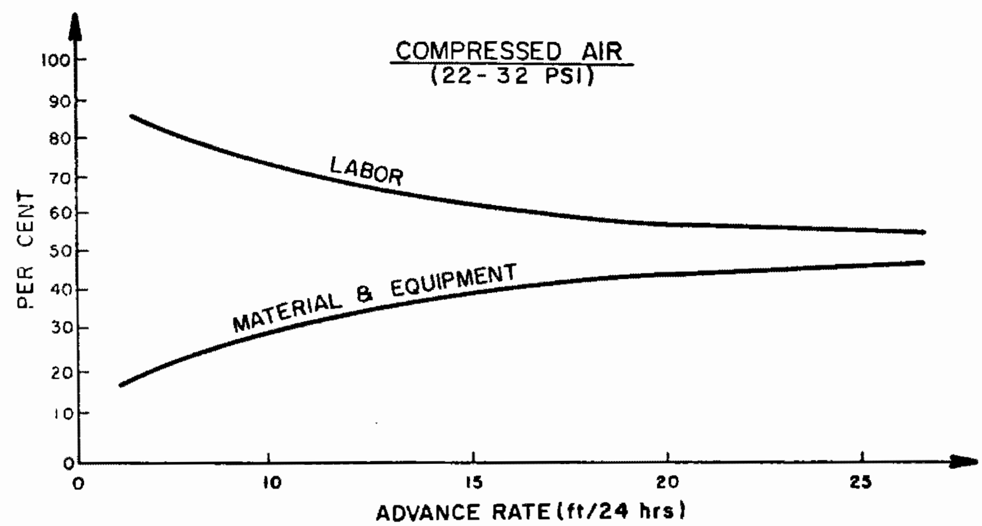


Figure A4. Labor and Non-Labor Costs (as a function of the Rate of Advance, 2 x 3000 Ft. Tunnel, Diameter 20 Ft., Steel Lining).

hence labor cost per foot of tunnel is a direct function of the daily production rate. This is clearly indicated in figure A2, where the constant K of the various $X \cdot Y=K$ equations represents the fixed daily crew cost, Y the rate of advance and X the resulting unit labor cost. Since the rate of advance is directly related to geotechnical variables there is clearly a relation between labor cost and soil variables. An identical argument is valid for the case of compressed air tunneling. In the absence of obstacles, the critical variable is the working pressure, which depends upon such geohydrological factors as water table location, permeability, cohesion, porosity, etc.

Within each of the two excavation techniques, labor costs may vary by a factor of four yet free air and compressed air operations generate vastly different labor costs, a difference of a factor of 10 or more. Strangely enough such dramatic cost differentials for free air and compressed air tunneling operations are more the result of intagible elements, than real physical factors.

In fact, the reasons for labor cost escalation in compressed air, in order of priority, are:

a. Increased labor insurance rates: from 19% of payroll cost in free air to 46% in compressed air.

b. Man-hours paid for, but not actually spent in productive work. OSHA regulations, and, on a more restrictive basis, union rules limits the maximum daily hours a worker can spend under compressed air (see table A3). At 22 psi OSHA prescribes maximum working time of 4 hours, the balance to a full 8 hour shift being spent in the decompression room. Thus, while in free air a 24-hour day may be covered by 3 crews, in compressed air 4 to 6 shifts may be needed. Sharp escalation in payroll outlays ensues.

TABLE A3. COMPRESSED AIR SAFETY WORK RULES

Work Pressure and Working Period for three locations

Washginton D.C.		New York		California	
(psi)	(hours)	(psi)	(hours)	(psi)	(hours)
1-14	6	1-16	4	1-14	6
14-22	6	16-26	4	14-18	6
22-32	4.5	26-31	3	18-22	6
32-38	4	31-36	2	22-26	4
38-44	4	36-41	1.5	26-32	4
		41-46	1	32-38	3
				38-44	2

c. Unlike free air, in compressed air there is no hourly wage differentials among various labor skills. Hourly wages, however, are progressively escalated, and productive time per shift shortened with rising compressed air values.

d. Inherently compressed air operations result in lower production rates, because of time lost for more frequent shift changes. As a matter of curiosity it is worth noting that labor productivity in compressed air is decisively higher than in free air: higher oxygen content per unit of inhaled air volume, accelerates motions and reflexes of the compressed air crew. This compensates, partially, for time lost during more frequent shift changes.

As a result, in Washington D.C. the cost of one hour of production work in compressed air at 38-44 psi is about 2.5 higher than in free air (under New York union rules and insurance rates one hour of productive work at 36-41 psi would cost 7 times more than in free air).

A.3.4 Conclusions

Equations A.2.1 and A.2.2 permit us to summarize the points discussed in the previous paragraphs into total tunneling cost differentials for various sets of geotechnical conditions. Ideally, i.e., in homogeneous geological, hydrological and soil conditions, the boundaries of tunneling cost can be calculated as follows for our 2 x 3000 feet twin tunnels:

Total cost in free air, firm-medium clay:

$$\left(\frac{21,500}{A.\beta} + 1185 + W + \Sigma \ \kappa_i \ E_i\right) \cdot 6000 = \$10,270,000$$

for $A = 36.6$ ft/day, $\beta = 1$, $W = 0$, $\kappa_i = 0$

Total cost in Compressed Air, 38 to 44 psi.

$$\left(\frac{R + 6000}{A.\beta} + 1830 + \kappa_i \ E_i\right) \cdot 6000 = \$24,845,000$$

for $A = 20$, $\beta = 1$, $\kappa_i = 0$

However, in the presence of more realistic geotechnical variables, i.e., mixed soil conditions, interfaces, boulders, strata of various severity, dewatering or even compressed air operations; the average rate of advance may drop to 10 ft/day or even less; compressed air tunneling cost for the same twin-3000' tunnels thus escalates in the extreme to about \$40,000,000. Below this extreme lies a series of intermediate costs for various geotechnical parameter combinations, as tabulated in table A1.

In summary, in free air tunneling, total costs are relatively insensitive to parameter changes except for major delays or significant dewatering costs. For the average advance rate achievable in typical free air tunnel driving, variable labor and fixed material costs are about of the same order of magnitude. Total cost variances for various soil conditions are usually in the order of 20-30%. In contrast, in the case of compressed air tunneling where labor costs are greater, different air pressure ranges may generate total cost differentials in the order of 200%.

This analysis has clarified how geotechnical conditions affect tunneling cost components: excavation labor cost is the crucial parameter dependent cost variable. Any mix of geotechnical parameters results in a certain average advance rate and relative total construction cost. However, advance rate and cost are not directly related, rather the relation varies with the value of the advance rate itself. For values of advance rate generating material/labor cost ratios above a certain value, advance rate variances produce little total cost deviation.

Apparently a ceiling exists in the economic benefits of increasing the advance rate to its maximum theoretical value. Based on the angular velocity of its rotating head, the maximum theoretical production of a typical tunnel excavation for soft ground boring is in the order of 300 ft/24 hour. With current labor/material cost ratios, as indicated in table A2, for hypothetical values of advance rate above 100-150 ft/day gains in the advance rate produce rapidly decreasing total cost reductions; and could even result in cost increase if the high advance rates would require the use of costlier equipment and materials.

In reality, since historically wages have been rising and are expected to rise at a faster pace than cost of materials, the ratio of labor to materials cost will tend to move toward a higher value. Thus today it is justified to aim for values of the advance rate above the apparent range of present economic incentives, in order to balance future labor rate escalation.

In actual tunneling in an urban environment, various combinations of geotechnical conditions may be encountered along the tunnel alignment. This could result in the need for machinery and equipment of various types to be amortized over a shorter tunnel length. The unit cost equations have been developed by amortizing the equipment purchase price (less resale value) along 6000 ft. of tunnel. For accounting rigor, thus, in case of tunnel requiring multiple sets of equipment (i.e., tunnel excavated 50% in free air and 50% is compressed air), some equipment and fixed labor cost as locks and locks erection and dismantling should be amortized on a fraction of the total tunnel footage. In such a case, tunneling costs are actually higher than an analysis of the unit data here presented would suggest. Costs series for mixed geotechnical variables and heterogeneous excavation techniques can be calculated by interpolating the cost analyses shown in the appendix.

As geological, hydrological and soil characteristics cannot be defined with a unique numerical value, there is also no single geotechnical variable which can be cited as the sole determinant of the cost of tunneling. This cost analysis appears to indicate that geotechnical macroparameters (boulders, groundwater, interfaces, manmade obstacles) should be rated higher as tunneling cost determinants, than geotechnical microparameters (granularity, cohesion, permeability, density...), although it may be argued that, to a certain extent, the cost significance of the former is conditioned by the value of the latter.

Figure A3 documents this point. In ideal conditions (that is in absence of extraordinary factors) cost is not significantly different whether tunneling in medium clay or sand, where the microparameters are of a complete different order of magnitude. On the other hand, the presence of water, boulders and possible compressed air operations, may introduce vastly different cost elements. It has been anticipated that tunnel geometry and geotechnical factors being equal, tunneling cost is dependent upon local union rules, labor and insurance rates, etc. To illustrate the point it is enough to recall that compressed air costs shown in tables A1 and A2 refer to the Washington, D.C. area, where, for instance, for compressed air pressures of 32 psi 4 hours of production work are permitted. Were the same tunnel to be built in New York City, where union rules allow maximum working time of 2 hours for the same pressure value, total labor costs would nearly double. That means that the same 2 x 3000 ft. tunnel priced at $29,000,000 in Washington, D.C. would in New York City cost $45,000,000, a 55% difference even disregarding labor wage differentials between the two areas. The priority rating of geotechnical factors with respect to tunneling cost is in fact conditioned to the above "institutional" conditions.

A.4 OPTIMIZING COSTS THROUGH BETTER GEOTECHNICAL PREDICTION

A.4.1 Introduction

In previous sections, the relationship between cost components and certain geotechnical parameters have been analyzed and quantified. The objective of this chapter is to evaluate the cost savings that may be achieved by eliminating mistakes and ambiguities in the prediction and interpretation of geotechnical parameters in soft ground tunneling. The problem can be broadly defined as follows: a tunnel is planned, designed, constructed and operated assuming geological, hydrological and soil condition "A"; what is the extra cost incurred when unpredicted condition "B" is actually encountered?

It is clear that savings cannot be estimated simply by deducting the tunnel cost in the condition "A", from tunnel cost in the condition "B". Although costs for "A" and "B" can be calculated with the cost equations developed previously, this does not include details such as the cost of lost time for retooling, equipment salvage value, and other related cost, which are critical for estimating realistic savings. It would certainly be possible to develop a model that includes such details, but this would exceed the scope of this study. It is more practical to compute savings by a case by case dissection of the most significant operations necessary to carry out a tunneling project. In fact, an error in the identification of geotechnical factors may be reflected into cost penalties along the whole chain of activities of a tunnel project, from planning and design to construction and maintenance operations. Accordingly, an analysis of economic benefits of proper identification of geotechnical variables on specific cost items is presented here.

A.4.2 Planning Tunnel Alignment

Table A1 shows typical sets of tunneling costs as functions of soil variables compressed air working pressure, and the presence of water and boulders. If the tunnel alignment could be selected strictly on the basis of cost considerations, a reliable prediction of geotechnical factors will enable the owner to select the "minimum cost" tunnel alignment. A minor or major horizontal or vertical relocation of the tunnel axis, may realign the tunnel in a boulder free area. An unstable water bearing lens may be avoided allowing tunnel boring with a lower air pressure, or even in free air.

The potential cost savings are in the order of millions of dollars and can be directly visualized by differentiation among the cost series reported in tables A1 and A2. For instance, by selecting a tunnel route with less severe boulder factor (i.e., from $\beta=0.6$ to $\beta=0.9$), savings of $1,970,000 (20% of total project cost) in labor construction cost can be achieved. For the typical case, dewatering operations add about 10% of the total tunnel costs. By avoiding water bearing strata or aligning the tunnel along soil strata with favorable permeability/porosity/cohesion parameters, dewatering cost can be reduced, if not eliminated. More dramatic cost benefits of the order at 50-100% of the final total cost can be realized, when the tunnel route selection implies alignment alternatives requiring compressed air operations.

A.4.3 Designing Lining Material

Under current engineering practice, the design of tunnel support structures is based on essentially empirical stability equations and hypothetical loads. As a result, as pointed out in the 1971 Annual Report of the Federal Excavation Technology Program, "most of the tunnel support systems in the United States are now over-designed".

Overdimensioning is fully warranted however, because current geotechnical exploration practices do not provide designers with sufficiently reliable and complete data on soil behavior. Thus it is necessary to work with high safety factors to control phenomena not fully known or understood. The implementation of exploration practices to identify geotechnical variables should introduce more refined design criteria allowing adequate safety factors. Refinements of geotechnical data alone, however, is not sufficient to achieve better lining design economy. New design practices must be developed, to better utilize improved geotechnical data.

The order of magnitude of the savings involved may be evaluated with the following two examples:

1. A reduction of 1/10 of one inch in the thickness of steel lining segments for a dual 3000 foot tunnel, saves 700 tons of steel, worth about $300,000 in material cost alone. Lighter lining segments will also result in some modest cost savings along the whole chain of construction operations, i.e., lower transportation cost, reduced manpower, equipment and energy requirement for handling and erection.

2. On the same twin 3000' tunnel, decreasing the number of bolts on the steel lining perimeter by one bolt per linear foot of tunnel, will result in the saving of about 100 hours of construction time worth $100,000 to $200,000.

The same reasoning of course can be extended to all the material components of the tunnel structure. In principle, every design detail, concrete lining work, grouting, water tightening details, just to mention the most visible ones, is a candidate for design refinement and consequent cost reduction. The direct cost of additional design engineering work to reach this objective is minor, when compared with the amount of the cost savings in question.

A.4.4 Construction: Time/Cost Overruns

Construction costs by nature are conditioned by external physical factors, either actual or anticipated and thus hold the highest potential for cost reduction by proper identification of geotechnical parameters.

Analysis of the cost data tabulated in figures A2 and A3 shows that the construction cost component most sensitive to geotechnical variables is direct driving labor cost, and in a lesser measure indirect overhead cost.

Construction material and supply cost per foot can be considered, under current design practices, nearly constant. Similarily daily driving labor cost as well as overheads can be considered invariable. Construction cost differentials per tunnel foot, are almost solely conditioned by the daily production rate; thus the element time is the critical factor determining cost differentials in a given tunnel project.

Unanticipated geotechnical conditions, or generally any unexpected obstacle (manmade or natural), will result in time delays. Construction cost overruns due to inadequate soil exploration can thus be measured as the cost of the time lost by the construction crew in surmounting the unanticipated condition.

For the purpose of calculating the cost benefits of predicting certain geotechnical parameters, only the "net lost time" is here considered, rather than the "total working time" required for advancing the tunnel through the obstacle. Prior knowledge of the obstacle does not eliminate the obstacle itself; it serves only to neutralize the "surprise" factor. In turn the cost of the "surprise" factor is represented by the crew time lost to identify the sudden obstacle ahead, evaluate possible alternatives, plan a course of operation and provide the proper tools to surmount it.

Table A4 shows daily fixed operating cost for labor crew and indirect overheads for the typical soft ground tunneling operation. Such expenditures are basically payroll outlays, which, for practical reasons, once built up to full operation level, possess an intrinsic inertia, independent of the production activity on the job side. Short term work stoppages, slowdowns and even sudden interruptions lasting a few days, while reducing the average tunnel footage bored per pay, have no significant mitigating effect on the level of payroll expenditures.

Depending upon different excavation methods, such fixed payroll costs range from $21,550 to $45,420, that is from about $1,000 to $2,000 per hour, on a 24 hour basis. These data allow

TABLE A4. TYPICAL TUNNELING COST:

(2 x 3000 ft., diameter 20 ft., steel lining)

FIXED RUNNING COST FOR 3 SHIFT, 24 HOUR OPERATION ($/Day)*

(Air Pressure, psi)	Free Air	0-14	14-22	22-32	32-38	38-44
Labor	15,550	25,650	26,580	36,980	38,350	39,420
Indirect	6,000	6,000	6,000	6,000	6,000	6,000
Total	21,550	31,650	32,580	42,980	44,350	45,420

*Also measures the extra cost incurred per day of lost time.

us to calculate the extra labor cost for unanticipated conditions arising during construction; simply by multiplying the crew "lost time" by its daily cost as reported in table A4. While daily crew costs and indirect overheads are exactly computable, the evaluation of the "lost time" may vary within a wide band, depending upon the nature of the obstacle, the skill level of the construction crew and the quality of the supervisory personnel: that is, the human element here plays an important role.

A.4.5 Construction Equipment

Proper selection of the type of excavation equipment, ground control and dewatering methods, and grouting equipment (to consider some of the most visible factors) is the crucial determinant for the final construction cost.

An accurate identification of the geotechnical variables along the tunnel is essential for selecting the optimum equipment and excavation techniques to minimize construction costs. Equipment and excavation methods and their ranges (cohesion, permeability, presence of boulders and water, mixed interfaces, etc.) When a discrepancy arises between the actual and predicted conditions, without exception, extra costs are incurred, varying from an insignificant percentage to 100% of the initial cost estimates.

Even such apparently irrelevant occurrences as boulders of a size larger than predicted may result in skyrocketing labor cost.

A 20-foot tunneling machine costs from $500,000 to $1,000,000 (after salvage value), depending upon different design features and accessory equipment. When amortized on a twin-3000 foot tunnel, this comes to $80 to $160 per foot, or 2 to 3% of the total unit cost. This compares with labor cost normally in the order of $800 to $2000 per foot. Although some savings are in principle possible on the purchase price of the machine, the crucial implication in the machine selection is that the construction crew production rate is conditioned by the proper matching of the equipment features with the actual geotechnical conditions. There have been cases such as the one where a small a design detail as the inadequate width of the mucker's conveyor belt for that particular geotechnical setting, which have severely impeded the whole excavation process. The penalty for boring with equipment not exactly suited is thus reflected in a lengthening of the construction schedule: with labor cost running at $20,000 to $45,000 per day, it is plainly clear how mistakes in the selection of the equipment can result in significant extra costs. As an example, an inadequate machine imposing a time penalty of 15% on the theoretical optimum rate of advance for the 2 x 3000 tunnel, means a cost penalty of about $600,000 in free air operation, and $2,000,000 in compressed air operations.

Time delays due to unanticipated geotechnical factors bear a further cost penalty. Rental of equipment is somewhat more expensive than equipment ownership. Nevertheless, contractors sometimes prefer rental to outright purchase to reduce their capital exposure. Rental cost for a large crane and operator may be in the order of $300 per day. This gives an idea of the cost penalty when a large train of rented equipment has to be idled or the renting period extended because of unexpected conditions.

In actual construction practices, once an unanticipated condition is encountered (i.e., a water bearing strata, or a bouldery area), since the severity and extent of the situation ahead are unknown, no sound decision can be made whether it would be more economical to suspend the operation for tool modification or outright retooling, or rather continue operations with a lower productivity machine.

As illogical as it may seem, given the low cost of exploratory work, to protect themselves against "probable" unpredicted conditions, contractors often make provision on the job site for alternative equipment (i.e., a large compressed air plant) should the suspected but unanticipated condition actually occur. In any case, the outcome is substantial extra cost, (the cost of a compressed air plant and accessory equipment is in the order of $500,000). Lacking exact geotechnical information leaves decision making on expenditures of the order of millions of dollars to chance or guesswork.

A.4.6 Indirect Costs

A careful review of the cost calculations shown in this appendix has uncovered an interesting cost factor, namely labor related insurance costs (Workmen's Comepnsation et al).

Insurance costs on direct labor may amount to as much as 46% of the total payroll base in compressed air tunneling, and 19% in free air tunneling. As an example, here below are tabulated some typical labor insurance costs. In addition, figure A5 relates insurance cost to labor cost and total costs.

($ x 1,000)	Free Air	0-14	14-22	22-32	32-38	38-44
Total Tunnel Cost	10,300	18,400	18,700	22,700	23,700	24,800
Labor Ins. Cost	580	2,900	3,000	4,000	4,400	4,600
Percentage	6%	16%	16%	18%	19%	19%

These insurance costs are based on an ideal "surprise free" tunnel. Any geotechnical difficulty or obstacle will increase the proportion of labor cost and consequently the proportion of labor insurance cost.

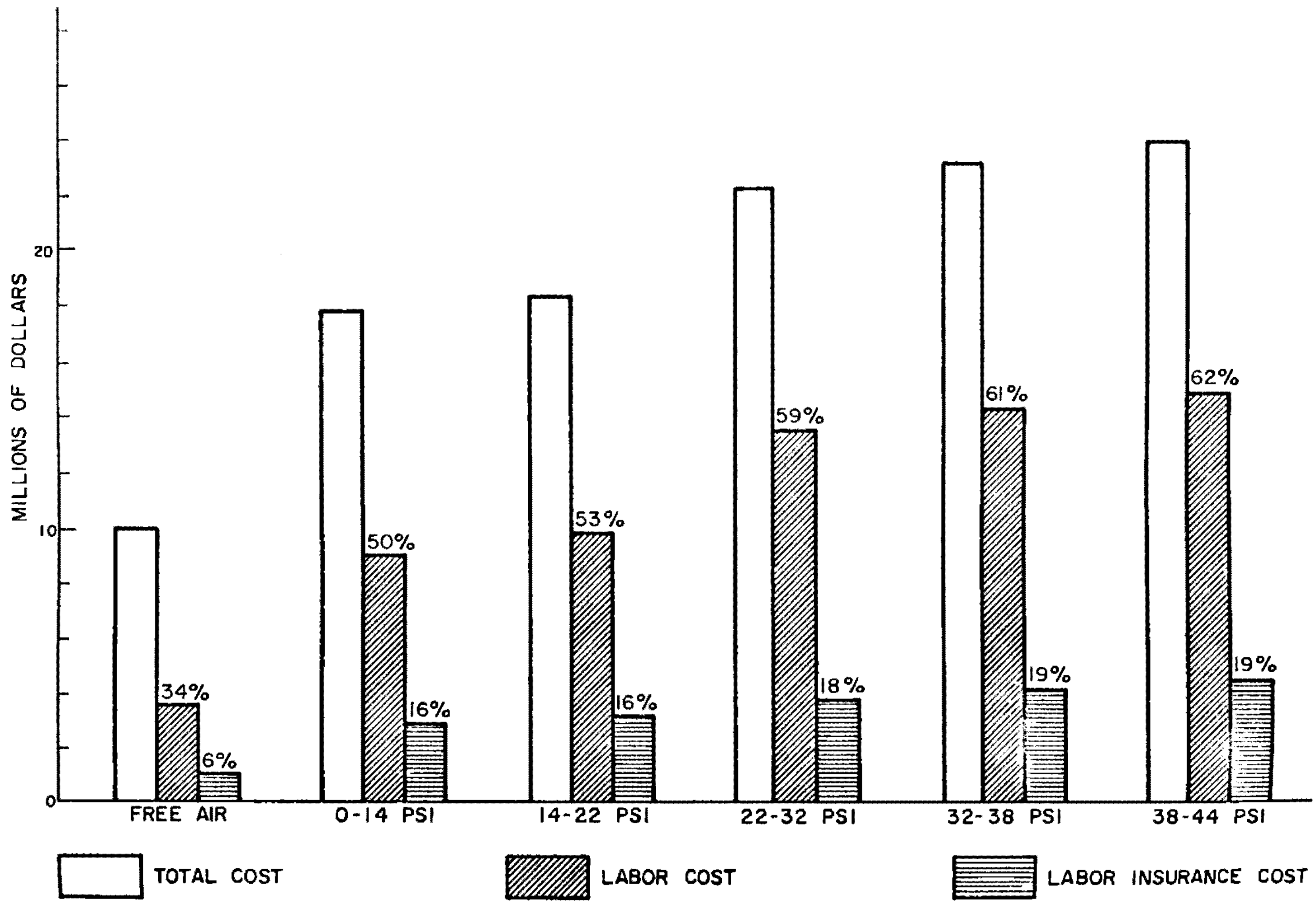


Figure A5. Typical Tunneling Cost, 2 x 3000 Ft., 20 Ft. Diameter, Steel Lining (Free Air - Compressed Air) Labor & Labor Insurance Costs vs Total Costs.

The present labor insurance rates are, or should be justified by a high frequency of labor accidents. There can be no doubt that the rate of occurrence of accidents in soft ground tunneling is somehow dependent upon the degree of knowledge - or ignorance - of geotechnical and man-made factors prior to or during construction operations. The prediction of such factors, will reduce the frequency of labor accidents, and consequently the insurance rates. Because of its complexity and great cost significance, this topic has been more extensively investigated in paragraph A8.

In addition to direct outlays in the form of insurance premiums, job site accidents, especially when they occur at the tunnel face, inevitably result in extra cost for time lost. Disruption of the excavation routine (from one hour to many days) is especially severe when critical geotechnical factors are involved (i.e., tunnel face collapse, or blow-out in compressed air operations, gas explosion).

A.4.7 Environment

In principle, more accurate geotechnical data should permit more reliable predictions of ground movements, maximum settlements, and stability of the tunnel-environment system. Empirical or semi-empirical soil mechanics equations exist, supplemented by direct measurements, allowing designers and engineers to estimate the probable behavior of the adjacent soil.

This affords the possibility of minimizing cost items related to the impact of driving a tunnel in an underground urban environment. Yet preventive soil exploration for this purpose is usually scanty.

The consequence is overdimensioning, or worse underdimensioning of support foundations, increased hazards to people and structures, and longer disruption of human activities. All this, in the final analysis, is translated into financial waste.

Because of the peculiar and infinite modes with which the tunnel structure reacts with the environment, a comprehensive analysis of the benefits of precise geotechnical information is not feasible. However, an examination of a few aspects of this problem may help in giving an idea of the order of magnitude of the cost involved.

1. Cost of construction delay. The cost of an unexpected interruption, or slow down of construction operation due to sewer collapses, excessive settlements endangering overhead manmade structures, etc., has already been evaluated at $22,000 to $45,000 per day of delay or slow down.

2. Cost of supporting structures (underpinning, etc.). Incomplete knowledge of the tunnel-environment system due to scarce geotechnical data compels design of supporting structures with greater safety factors at extra cost. At worst, untimely or inadequate design results in permanent damages or collapse of the overhead structure. In any case extra costs in the order of millions of dollars are a tangible possibility.

3. Property/liability insurance cost. Insufficient knowledge of the soil behavior and consequent higher frequency of partial or total damages to manmade structures and even to the tunnel itself, determines the property/liability insurance rate. Current rates for comprehsnsive property/liability insurance coverage are in the order of 1% of the total tunnel cost. Better knowledge of the interaction of the tunnel-environment system should reduce the damage/injury frequency and thus pull down the property/liability insurance rate. Sometimes, especially in the cases of large tunnel projects, coverage is provided with self insurance. This does not change the problem since damages have to be compensated in any case.

4. Cost of disrupting human activities. Damages to surrounding structures, or road traffic interruptions because of excessive settlements or caveins result in lost productive manhours chargeable to the environmental

economic activities. A 2 minute car/man slowdown in a partially obstructed 10,000 car/day highway, for 20 days results in about 7,000 lost man hours, the cost of which can be conservatively estimated to $35,000. Considering an inevitable multiplier to account for human intersections along the economic chain, the cost of the disruption may rise to the order of hundreds of thousands of dollars.

A.4.8 Conclusions

The familiar question decision makers of various disciplines want answered is: What is the value, of a precise geotechnical prediction in terms of cost reduction in tunneling work?

A lengthy series of case histories could be researched documenting that almost without exception, an exact identification of the geotechnical variables, would have resulted in cost savings, varying from a minimal fraction to 50% or more of the total cost. A large number of cases could be collected to estimate how much resources have been wasted because of inadequate geological information. Familiar tools of statistical and probabilistic analysis would then permit to be interpolation of a numerical answer valid for the typical tunnel project.

Yet, it is doubtful that such a mathematical approach could ensue in a more accurate estimate than one arrived at simply by speculative reasoning on the data presented in this chapter. In fact, an answer to the initial question makes sense only if viewed in the wider context of a national scale. A national perspective is presented in paragraph A.12.

Ultimately the economics of cost-reduction in tunnel construction work can be summarized in a few crucial points. For a typical 20 foot diameter tunnel job, the payroll for a 3 shift work day runs to about 200 men. Including social charges, premium time pay and other mandatory charges, the outlay comes to $20,000/45,000 per day, independent from the production rate. Rigidly enforced union work rules bar controlling manpower costs by trimming the labor force on job site. On the other hand, traditionally real wages of tunnel workers have been rising faster than the average in the construction sector (in Washington. D.C., in the period May 1969-May 1973 the hourly earning base for compressed air tunnel workers (14-22 psi) rose 57 percent, while national hourly earnings for construction workers went up 35 percent.)

The sole solution for checking the labor cost is to balance and offset the payroll by increasing productivity. Construction time is here the crucial variable: daily payrolls of $20,000 to $45,000 puts the worth of one hour time (lost or saved) at $1000 to $2000. A time overrun of 5 percent on a typical 24 to 30 month contract means a cost overrun of $500,000 to $1,500,000. This is the reason why in tunnel contracting the final bill is often decided in a court room.

In addition to visible construction costs, lengthier construction time results in other hidden but nonetheless real and tangible costs. A tunnel financed by a $40,000,000 public bond issue may carry interest charges of:

$$\frac{30,000,000 \times 0.07}{12} = \$175,000 \text{ per month,}$$

or $8,000 per work day. Delay in the scheduled tunnel completion carries this cost penalty in extra charges. Cost of disruptive human activities may be still higher. Exact knowledge of geotechnical parameters in the planning stage removes the ultimate source of unproductive time in tunnel construction work.

There is a consensus among people in tunneling business about the cost penalty of inadequate geotechnical information and the concrete cost benefits of improving prediction methodology. It will suffice to cite a few quotes to document the universal concern on this matter:

A.S.C.E. Tunneling Conference, N.Y.C. February 1974:

"Tunneling in Chicago is cheap because conditions are much more predictable"

"There must be a way to determine uncertainties"

"More geological information and expert interpretation will result in lower costs"

"All major NYC Tunnels ended with a law suit. Better geological information would have spared most of these law suits"

"Uncertainty of geological conditions makes tunnel bidding similar to betting horses"

Committee for Rapid Excavation, Panel Report 1968 - N.R.C.:

"The importance of adequate geological prediction cannot be overestimated. Extreme and often unpredicted heterogeneity has caused delays running into many months, cost of many hundreds of thousands of dollars more than anticipated, and even loss of life. These excessive costs and delays could possibly have been sharply reduced and in some instances eliminated if the geological condition of the excavation were known before the work begun. With such information a more favorable site or route could possibly have been chosen.

A.5 COST PENALTY FOR INACCURATE GEOTECHNICAL INFORMATION: CASE HISTORIES

To document the arguments and conclusions of this study, a number of case histories were researched. The cases here presented are not necessarily the most significant. In fact, selection has been made on the basis of accessibility, reliability and actuality. Prices, bid or paid for, are generally a matter of public record. True contractor cost data, on the other hand, carry a strict confidentiality label. Pending or potential pecuniary claims, especially for such sensitive items labeled as "extra cost for unexpected conditions" render contractors unwilling to disclose their cost for publication.

In the cases here presented, facts and occurrences have been investigated and recorded. The extra costs incurred however have to be considered as "no-commitment" figures estimated by the contractor and the author of this report.

1. Staten Island Sewer Tunnel (1973-1974): A 7,000 ft. long, 9 ft. diameter, steel lined sewer tunnel is being bored in predominantly glacial till strata with areas of heavy boulders, and ground water of variable severity. The following occurrences were observed:

   a. The nature of the soil and the presence of boulders and ground water were anticipated. On the basis of available geotechnical information, mechanized boring was considered the most economical solution. A full scale wooden model of the tunnel section and the prospective tunnel boring machine was constructed, to decide upon the machine design details (a 9ft. diameter tunnel has a theoretical section of 64 sq. ft. only). An open face TBM was finally selected equipped with a hoe type hydraulic tool for boulder handling. With such a machine, (about $500,000 in cost), the contractor anticipated an average rate of advance of 40 ft/24 hours; he estimated his running cost (direct and indirect labor, and job consumption supply) at $10 per minute, that is $600/hour or $14,400 for a 24 hour work day.

   During construction it became apparent that the frequency of the boulders was more severe than anticipated. During 39 out of the first 43 working days, boulders of variable severity were encountered. Moreover, the size of the average boulder was larger than had been expected. Round and oblong shaped boulders 8-10 cu. ft. in volume were common. While the hoe tool has proved satisfactorily effective in loosening and dislodging even the larger boulders in the heading front, the width of the mucker's conveyor belt - an integral part of the machine, was too small to carry the larger boulders from the tunnel face to the muck train. It was necessary to send a crew of 2-3 people with manually operated pneumatic hammers to the tunnel face ahead of the machine face to break the boulders to a size compatible with the conveyor belt capacity. From the initially envisioned 40 ft/day the average rate of advance dropped to about 15 ft/day. In dollar terms, this means that the contractor's labor cost per tunnel foot rose from an anticipated $\frac{14,440}{40}$ = 360 \$/ft to $\frac{14,440}{15}$ = 960 \$/ft, a 270% increase for this particular length of the tunnel.

If the size and severity of the boulders had been exactly anticipated, a machine of different features, or even a different excavation technique could have been selected. The contractor stated that under the actual geotechnical conditions, there was actually no economic justification for the type of machine now in operation: had he known in advance the true extent of the boulder problem, he might have opted for a manual or semimanual excavation method, which under this condition could have resulted in lower unit cost.

b. Extensive dewatering operations with conventional techniques were carried on during construction. This has so far been satisfactory for controlling ground water. However, because of the uncertainty of the severity of the ground water problem, contract documents specified a large multistage stand-by compressed air plant and accessory equipment (including a medical air lock) just in case uncontrollable water problems would demand working under compressed air. The cost of such additional stand-by equipment can be estimated at \$500,000. A priori identification of the ground water condition and its governing factors, would have saved this expenditure.

c. A 16-inch live sewer, running a few feet above the tunnel crown, in a position not exactly known previously, collapsed over the tunnel heading. This resulted in lost time of about one 8 hour shift, equivalent to about \$5,000 in running cost. The breakage of a gas main brought about the same penalty. If the location of the sewer and gas main had been exactly known, preventive actions could have been taken to avoid lost time and extra cost during construction.

2. South Branch Interceptor Sewer, East Side Lower Manhattan, N.Y. (1963-1964): Total 18,000 ft., 12 ft. driven diameter: 8,000 ft. of tunnel; 10,000 cut and cover excavation at bid.

   Work was started in cut and cover in a section parallel to the lower east side of the island. An unpredicted situation of continuous finger piers, broken piles, and old foundations was encountered, such as to make it impossible to dewater the bottom even with the heaviest pumping plant except during 2 hours at low tide. After 2000 feet of cut and cover excavation, with rate of advance about 1/4 of what was initially estimated, the decision was made to proceed by tunneling in compressed air, as the sole solution for controlling water.

   The contract amount was \$22.4 million (1962 \$). In current 1974 dollars the work would have been quoted at about \$50 million. With better geotechnical information, the severity of the water condition could have been anticipated and a more favorable alignment could have been selected. It is estimated that 1/3 of the cost could have been saved.

3. Governor's Island Ventilation Shaft: Brooklyn-Battery Park Tunnel, N.Y. (1946-1948).

   After the twin tunnel was bored, a ventilation shaft (100' x 50') was bored to 175 feet below water level, using compressed air to 40 psi; 1200 cylinder piles were driven over an artificial sand island (200' x 200'), to protect the caisson shaft. The following occurrences took place:

   a. The cylinder piles encountered an unexpected area of broken rock slabs, probably material loosed by the previous tunnel boring operation. It was necessary to anchor the cylinders at a deeper level.

b. A corner (30' x 20') of the caisson shaft encountered an unpredicted uneven, heterogeneous soil layout. It was necessary to underpin the shaft with a support wall erected at about -175 feet.

Had better information been available, a more favorable location for the shaft could have been selected, or the cylinders on the caisson could have been properly dimensioned for surmounting the obstacle. The entire work was completed at a cost of \$3.8 million (1946 \$). Occurrences a and b resulted in extra costs respectively of \$360,000 and \$115,000, a total of \$475,000 or 12.5% of the total contract amount. It has been estimated that the same project would cost \$35,000,000 in 1974 dollars (the cost of a compressed air man-day was \$13 in 1946 and about \$150 in 1974). The extra cost in question thus amounts to \$4.4 million in 1974 dollars.

4. Unidentified New York City Sewer Tunnel (1957-1959): 12' diameter, 5,000 ft. compressed air; 1,000 ft., cut and cover, at bid.

   An unexpected perched water layer was encountered (in addition to bottom water) while sinking an 80 ft. shaft: 16 weeks were spent for the job. A second shaft, where identical hydro-geological conditions were exactly anticipated was more expeditiously executed: working time in this case was reduced to 6 weeks. Considering a daily payroll of 50 men, this is translated into a cost saving of about \$500,000 (1974 \$) in direct and indirect labor cost and job materials. On the 1,000' section originally provided for cut and cover excavation, it was found impossible to control water by dewatering operation only. It was necessary to switch to compressed air tunneling. In addition, the unpredicted existence of an old creek bed, resulted in severe settlements in buildings 3 blocks ahead of the tunnel face. Extra cost of about \$1,000,000 (1974 \$) would have been avoided with proper geo-hydrological information.

A.6 COSTS AND PRACTICES IN GEOTECHNICAL EXPLORATION

In investigating the geotechnical characteristics for a planned tunnel route, owners and contractors still largely avail themselves of the traditional borehole methodology, complemented by laboratory testing. Observation of a series of real cases in typical urban soft ground tunneling works, indicates that it is customary practice to space the borehole testing stations at about 300-foot intervals. Direct cost of borehole testing can be fairly averaged to $600 per hole.

For the typical, twin, 3000 ft. tunnel on which the tunnel cost analysis for this report is based, provision should be made for 10 test holes for a total direct cost of $6,000. An additional $1,500 per hole is required for laboratory testing, geotechnical analysis and ancillary engineering work. A pumping test may add $40,000. Total soil exploration cost can thus be estimated at $61,000, or $10 per lineal foot of tunnel (these are prices for contracted work: direct costs would be about 50% lower). Under current practices only 0.3 to 0.4% of the total tunnel cost is allocated for the specific purpose of identifying geotechnical conditions. In view of the fact than an exact knowledge of the soil parameters has here been proven critical in minimizing tunneling cost, with potential savings of a significant percentage of the total tunnel cost, it can be argued that current efforts and expenditures for exploration activities are at best deficient, if not totally inadequate.

Any attempt to suggest an "optimum rate of return" level for exploration expenditures is of course strongly contingent on subjective judgment. Yet on the basis of the concepts and findings outlined in paragraphs A.4 and A.12 more precise geotechnical data for the 2 x 3000 ft. tunnel under examination, are potentially worth 100 ÷ 200 $/ft. (5 to 8% of the total cost). Consequently, disbursements of 20 to 30 $/ft. (0.8 to 1.2% of the total cost) to gather better geotechnical data are amply warranted.

A.7 PROMOTING EXPLORATION PRACTICE

Although the development of new exploratory tools would certainly enhance the efficiency of geotechnical exploration, the comprehensive survey of instruments and instrumentation techniques included as a part of this study indicates that adequate exploration technology is available for the identification of most cost sensitive geotechnical variables. Rather than deficient technology, the problem appears to be a widespread skeptical attitude about the real economic benefits of more intensive exploration practices, as well as a general unfamiliarity with the available tools and methodologies. The point is that, were the currently available exploratory tools put to more intensive use on a nationwide scale, substantial savings could be realized even within the present technological limits.

It is here suggested that the appropriate mechanism for transferring existing technology into field practices is a program of "Education and Information Dissemination" (EID) to use government budgetary terminology).

Current information programs in the field of exploration technology are primarily based on random conferences sponsored by government agencies, research and professional organizations, and various distribution channels of technical literature. It is evident that current information dissemination methods fail to reach both the decision makers and the people at drawing boards and job sites. Exploration technology, and the understanding of its potential benefits remain in the domain of a few specialists, and fails to reach the level of people who make cost sensitive decisions.

For the years 1971-1973, the U.S. Excavation Technology R&D Fund budgeted outlays of $79 million. Of this amount only 0.5% or $130,000 per year, was allocated for EID. It is reasonable to assume that only an infinitesimal sum went for dissemination of exploration technology. Discounting some fractional funding by private institutions, here may lay the explanation for the lack of penetration of the current information dissemination programs.

A permanent or semi-permanent advisory body, government or industry sponsored, can be envisioned to "educate and disseminate information" in the field of exploration technology in tunnel work. National coverage could be provided by a four-man group positioned in key geographical points. (This embryonic plan is offered here solely to point out the nature of the problem and a possible solution. A more detailed analysis would exceed the scope of this report).

The magnitude of the "Education and Information" problem has been here summarily outlined in the following points a and b.

a. People involved in cost sensitive decision making:

Consulting	Construction	Authorities	Research/Education
planners	contractors	government	research institutes
designers	labor unions	state	educational bodies
engineers	insurance underwriters	municipal	
estimators	equipment mfrs	transit	
geologists			

b. Overall tunneling activity in the U.S.:

It has been estimated (North American Tunneling Conference, 1972, Proceedings) that in any given year, 100-150 tunnel projects are under construction or in an advanced planning stage in the country. This includes highway, subway, sewer and water tunnel work in soft ground as well as in rock, but excludes mining tunnels. Expected underground excavation work, averaged over the next decade comes to about $2 billions per year. (See paragraph A.11.)

A rough estimate of the information dissemination program yearly cost is attempted here below:

Personal	Quantity	Unit Annual Cost	Total Annual Cost
Professional staff	4	$50,000	$200,000
Clerical help	2	12,000	24,000
Office space	2	10,000	20,000
Travel	4	12,000	48,000

Personal	Quantity	Unit Annual Cost	Total Annual Cost
Equipment	1.s.	$10,000	$ 10,000
Conferences	25	2,000	50,000
Publications	10,000	0.5	5,000
Contingencies (10%)	1.s.	40,000	40,000
TOTAL ($ per year)			$397,000

Ideally after the first 3 to 5 years of an intense penetration effort, the program activities and its costs, could be reduced. Total outlays over a 10-year period would thus be in the order of $3,000,000 (1974 $).

Compared with the above cited past budgerary allocation of the Excavation Technology R&D Fund, this is clearly a substantial sum. On the other hand, if this amount is measured against the projected national expenditures for non-mining underground excavation work in the period 1975-1984 ($6 billion to $25 billion depending upon the sector considered, see paragraphs A.11 and A.12).

$$\frac{\text{Cost of Information Program}}{\text{Underground Excavation Work}} = \frac{3\text{x}10^6}{(6 \text{ to } 25) \text{ x } 10^9}$$

$$= 0.0005 \text{ to } 0.0001,$$

the cost of the program turns out to be in the order of $\frac{1 \text{ to } 5}{10,000}$ of the latter.

In paragraph A.12 it is anticipated that better exploration practices could result in nationwide tunnel cost reduction estimated in the order of percentage integers. If this conclusion is accepted as valid, then there appears to be ample justification for capital allocation 1/10,000 of national excavation expenditures in an effort to achieve such cost savings.

A.8 SAFETY ECONOMICS IN SOFT GROUND TUNNELING

The cost of labor accidents in tunnel works - at least the part of such cost which can be anticipated - is represented by the insurance premium the contractor has to pay to provide mandatory coverage, under Workmen's Compensation (WC) statutes. Of course, contractors enter this item into their bid prices. Labor accidents thus become a component of the total tunnel cost. In this paragraph we analyze the mechanism through which frequency and severity of labor accidents are translated into construction costs. The role of geotechnical exploration on the safety aspect of SGT will be assessed. Literature on this topic is totally nonexistent: in order to remain within the scope of this research, this analysis has been limited to a "fact finding" brief overview. Certainly, because of cost significance, the topic would merit deeper investigation.

1. Workmen's compensation coverage. Private insurance carriers, as well as state sponsored funds sell tunnel contractors WC insurance policies against on-the-job accidents and occupational diseases. There are 30 states and Washington, D.C., where only private insurers operate. Private companies and state funds may compete in 12 states (New York and California belong to this group). Finally, 7 states mandate purchase of WC insurance through an exclusive state WC fund. Data shown in Bulletin No. 312, Wage and Labor Administration, DOL, 1969, indicate that, traditionally, private companies insure 80% of the total WC business (all industry sectors); state funds cover the balance 20% - very probably the same breakdown applies to WC for tunnel work.

2. Rate making. Practically all private WC insurance is sold by so called "bureau insurers." Bureau insurers are voluntary members of rating bureaus; they use as basic rates those developed by the bureau they belong to. A rating bureau computes the basic WC premium rates for every work classification on the basis of profit and loss reports from its members. It merits pointing out that rating bureaus do not collect or handle accident statistics, but only profit and loss data furnished by the member companies. For the purpose of rate making, private insurers classify all tunnels as "pneumatic" and "non pneumatic." Therefore, there is no rate difference for

WC coverage in soft ground or in rock tunneling. Premium rates are quoted as a percentage of the basic payroll cost for the covered workers. Premium rates may vary substantially from state to state because of different levels of mandatory benefits and different state regulatory criteria - (i.e., WC insurance for free air tunneling is rated 10% and 18% of the payroll cost, in New York and California respectively; for compressed air in both states the rate is about 45%). In addition different tunnel contractors must be charged lower or higher rates depending upon their specific risk rating. In the case of large tunneling projects, special - and sometimes peculiar - arrangements are negotiated: WMATA tunnel work has been rated under a "National Defense Project Rating Plan" which implies a substantial discount versus the standard rates (it was here assumed that the Washington subway tunnels may be used as bomb shelters.)

3. Premiums/benefits. The above cited DOL Bulletin reports WC premium/loss statistics. The following data have been abstracted from this source:

1958-1967 Cumulative WC Data ($ Millions)

	Premium Income	%	Losses Paid	%
Private Insurers	18,205	80.1	9,815	62.8
State Funds	4,275	19.9	3,785	24.2
Self Insurers	----	----	2,037	13.0

It appears that private insurers incurred losses totaling 54% of the premium income, while state fund losses were 89% of premium income. No comprehensive nationwide WC premium/loss statistic exist in the tunnel construction sector. However, partial data covering WC tunnel work in 36 states for the period 1969-1971 were made available by the National Committee for Compensation Insurance (largest national rating bureau with 300 member companies). These data, although very fragmentary, indicate a premium/loss ratio of 54%, identical to the ratio above reported for all industries. This seems to indicate that WC data for tunnel work compare with the all industry averages.

The same DOL publication again indicates that for the period 1962-1966 state WC funds returned in benefits $0.92 out of every "net premium" dollar. Private insurers disbursed only $0.70 per dollar at "new premium." This means that one dollar of WC benefits cost $1.09 if insured with a state fund, and $1.43 if covered by a private insurer. Private insurers are inherently more expensive than state funds because of commissions to sales agents (state funds do not maintain a sales force); higher administrative expenses ("private" salaries are higher than "public" salaries); state and local taxes (state funds are tax exempted).

No data exist indicating the amount of WC premium collected in tunnel work only. A rough estimate - a very rough estimate indeed-can be attempted with the elements developed in paragraphs A4 and A12: soft ground excavation in the transportation sector has been estimated in the order of $500 million per year; 60% of this amount should represent labor cost, which includes 10% of WC insurance cost. Nationwide WC insurance coverage in this sector should thus cost $30 million per year, or $300 million over the next decade.

4. Accident statistics. Nationwide tunnel accident statistics are very scarce and fragmentary. Rate making bureaus do not collect such data. OSHA regulations require contractors to keep detailed records of labor accidents, but there is no obligation to report. Tunnel accident data recorded by contractors, labor unions and private insurers are practically inaccessible because of the financial implication underlaying such figures. The latest nationwide survey of labor accidents in tunnel work (Reports 318, BLS, U.S. Dept. of Labor 1967) shows 1961 data (see table A5 and figure A6). Accident frequency and injury-severity rates in tunnel work are here compared with other

TABLE A5. WORK-INJURY RATES IN THE HEAVY CONSTRUCTION INDUSTRY, BY KIND OF CONSTRUCTION AND TYPE OF OPERATION, 1961

Kind of construction and type of operation	Number of establishments	Number of employees	Employee-hours worked (in thousands)	Frequency rates of-- All disabling injuries	Deaths	Permanent disabilities	Temporary-total disabilities	Severity: Average number of days lost or charged per-- Disabling injury	Temporary-total disability	Severity rate
Total	2,346	90,367	186,919	27.3	0.5	1.5	25.3	177	25	4,829
KIND OF CONSTRUCTION										
Bridges: Total	223	10,262	19,445	28.3	.5	1.4	26.4	166	32	4,687
Substructure and superstructure	129	6,852	12,697	23.5	.6	1.2	21.7	206	33	4,830
Substructure only	51	2,580	5,207	31.7	.4	1.7	29.6	114	33	3,613
Superstructure only	43	830	1,541	56.0	.6	1.9	53.5	128	27	7,141
Dams	85	6,839	14,442	20.7	.8	2.3	17.6	340	34	7,041
Docks and piers	78	2,569	5,417	30.3	.9	1.5	27.9	287	32	8,684
Dredging	114	3,359	7,308	30.0	.6	1.5	27.9	148	17	4,433
Heavy foundations	33	6,197	14,120	25.4	.3	.6	24.5	103	32	2,607
Industrial plants and equipment	56	1,866	3,503	14.6	.3	.3	14.0	170	21	2,472
Land clearing	87	1,934	4,144	22.6	.3	.5	21.8	88	12	1,995
Pile driving	78	1,106	2,119	39.4	--	1.9	37.5	56	33	2,193
Pipe lines, gas lines, gas mains	282	9,401	19,378	41.2	.3	2.3	38.6	86	18	3,548
Power lines	193	6,101	12,307	28.8	1.2	1.3	26.3	342	23	9,845
Railroads	51	740	1,482	36.1	--	2.2	33.9	44	34	1,597
Sewers and water mains	823	18,927	37,342	31.4	.4	2.0	29.0	150	22	4,700
Tunnels	22	1,255	2,830	56.8	1.4	7.9	47.5	270	47	15,326
Other	144	9,902	21,821	11.8	.3	.7	10.8	205	21	2,427
Unclassified	76	9,909	21,260	--	--	--	--	--	--	--
TYPE OF OPERATION										
New construction only	1,356	51,857	108,825	27.3	.6	1.7	25.0	209	24	5,712
Repair work only	61	791	1,529	25.2	--	2.0	23.2	47	31	1,179
Both new construction and repair work	838	31,684	64,069	27.5	.3	1.3	25.9	128	27	3,509
Unclassified	90	6,035	12,496	--	--	--	--	--	--	--

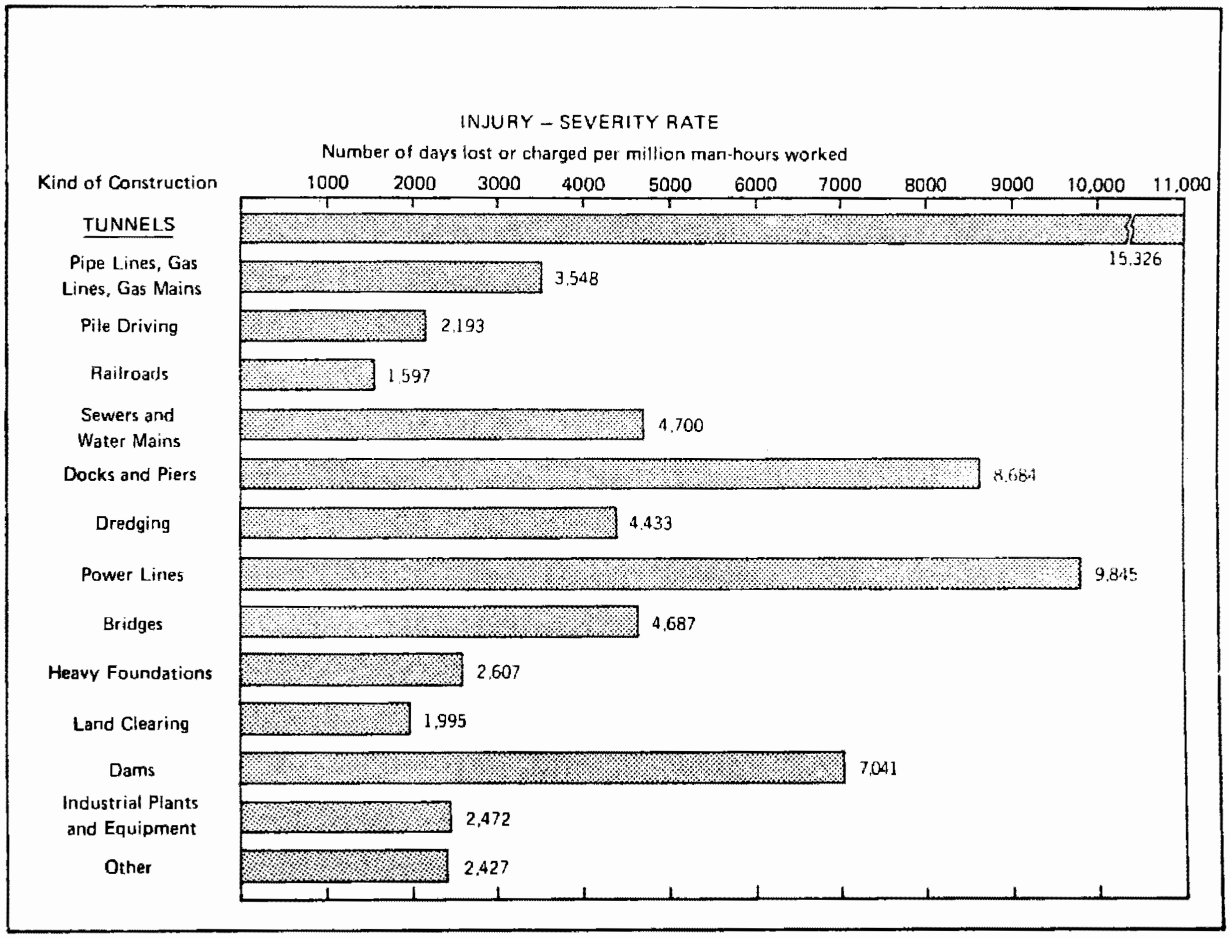
Source: BLS Report N.318, 1967

Note: Because of rounding, sums of individual items may not equal totals. Dashes indicate that no data were reported.

SEVERITY RATE: 'adjusted' lost days per 1,000,000 man-hours worked.

FREQUENCY RATE: number of injuries per 1,000,000 man-hours worked.

Tunnel construction sample based on 3,000,000 man-hours. Lost man-hours as % of total man-hours worked: $\frac{15,326 \times 8}{1,000,000} = 12\%$



U.S. DEPARTMENT OF LABOR , Bureau of Labor Statistics

BLS REPORT NO. 318

Figure A6. Work Injuries in the Heavy Construction Industry, by Kind of Construction, 1961.

construction sectors. The most significant data are that for every 100 man-hours worked, 12-13 hours of lost time occur because of accident. This is about 4 times the average (tunnels excluded) for the entire heavy construction industry.

It is worth mentioning however, that the U.S. Dept. of Interior reported an injury-severity rate for Bureau of Reclamation tunnel work (1967) of less than one half the rate above indicated. It reflects certainly the result of well known stricter safety practices in government work.

There is no source for national tunnel accident statistics classified by accident type (i.e., explosions, caveins, collapse). The California State Division of Labor Statistics has collected accident data for 17 major tunnel projects occurred in the state in 1967. It appears that a major cause of work injuries is rock falls and accidents related to handling of explosives. This suggests that basically such accidents occurred in rock tunnels, and thus are irrelevant to the scope of this study.

5. Conclusions. To ascertain whether better knowledge of geotechnical factors could affect the frequency and severity of accidents in SGT work, it is necessary first to identify the major class of accidents and secondly analyze whether some class of accidents can be influenced by better geotechnical variables. Although it is common knowledge that some type of severe accidents (gas explosions, caveins, collapse) are directly related to geological factors, the lack of statistics of this type does not allow us to theorize and quantify the effect of geotechnical information on the overall accident frequency. Certainly the correlation exists and has been long recognized: the Safety Engineering Department of the Liberty Mutual Insurance Co., the largest U.S. WC insurance carrier, about 10 years ago developed an instrument to predict and monitor soil/rock movements in excavation work.

A few rate making agencies and insurance companies have been consulted on this topic. Without exceptions, all of them have agreed that an improvement in the national tunnel accident record will result in lower insurance rates. Time lapse between cause and effect has been estimated in the order of five years.

It is interesting to note that since there is no rate differential between soft ground and rock tunneling, an improvement in the accident record in SGT will be not fully translated into lower insurance rates unless rock tunneling injury-severity rates also decrease. On the other hand, rock tunneling is more hazardous than soft ground tunneling (rock falls, explosions and collapses, 3 major causes of accidents, are typical of rock tunnel work). It seems thus that WC insurance rates for lower risk soft ground work is financing the higher risk of rock tunneling.

In addition to mandatory WC costs, another aspect of the safety economics in SGT is the effect of large compensation sum sought via law suits. In the Port Huron (Michigan) tunnel accident where 9 people died and 23 were injured, WC claims totaling about $1 million have been paid so far. However, suits for about $100 million have been filed for not insured compensations.

To prevent is cheaper than compensate: if for no other reason, the expenses to administer a compensation system (30% of premiums collected) are avoided. The basis for an efficient accident prevention program is the systematic collection of nationwide accident statistics by type of accident. The infrastructures for this project exist (OSHA). At this point the eventual and probably concrete - role of geotechnical exploration be a preventive tool for a certain class of accidents can be assessed.

In the course of this research, the following agencies were contacted:

> National Council on Compensation Insurance; National Safety Council; New York and California Workmen's Compensation Boards; U.S. Department of Labor (OSHA, BLS); tunnel contractors; tunnel workers unions; private insurance carriers.

A.9 THE NEED FOR A COMPREHENSIVE COST MODEL

A.9.1 General

Whatever its aesthetic appeal or technical originality, the validity of a design feature or construction method in a tunneling project has to be judged and measured in terms of economic benefits. Even the concept of environmental impact is ultimately assessed in dollar terms. Because of the complexity and multiplicity of the factors determining the final tunnel cost, in principle it appears that the use of a computerized cost model is fully justified; it would provide a decision making tool of the most general use, for designers, engineers, contractors and owners, to rationalize tunnel design criteria, to optimize construction methods and ultimately to reduce overall tunneling costs. Inherently the practical utility of such a cost model requires the availability of exploration technologies and methodologies for a precise and reliable identification of geotechnical conditions.

The benefits produced by the use of a comprehensive cost model are summarily outlined here below:

A.9.2 Planning and Design

Every planning and design detail is inevitably reflected in the final project cost. Yet in practice, many technical decisions are made without rigorous consideration of the cost involved. Design and cost analysis functions seem to be carried out as unrelated activities. A cost model, by rendering designers cost-conscious, promises to bridge the gap. It will provide a fundamental and easy to use decision instrument for designers and engineers to quickly evaluate and compare, on the drawing board, the cost effect of alternative design choices, and ultimately to select optimum cost options in tunnel geometry, lining material and a large number of final design details.

A.9.3 Tendering and Bidding Preparation

A convincingly dependable cost model will drastically curtail time and manpower requirements in tender and bid documents preparation. It will reduce, if not eliminate, the manual work of calculating a lengthy series of unit cost items and quantity estimates.

A.9.4 Bidding Evaluation

A cost model will permit the fair evaluation of heterogeneous bid offers for a given project. In most cases even significant cost differences in lump sum bid prices are easily seen but not readily understood, because the true project cost may depend on such details as term of payments timing, interest rates, different estimate in take off quantities, labor cost readjustment formulas (if any) etc. A cost model will allow the leveling of the various bid offers to a common denominator, uncovering the true lowest bid. With current contractor selection practices, however, the selection is made on a preestablished set of quantities and payment procedures, so that a cost model may not be an enforceable tool.

A.9.5 Construction Planning

A cost model would be an excellent tool for selecting the manpower-machinery mix to maximize overall construction productivity. For a tunnel of a given geometry and for given geotechnical parameters, the optimum degree of mechanization may differ in time and place according to local union rules, labor rates, safety and hazard prevention regulation. Today equipment features and, generally, capital allocation for construction equipment are decided upon on the basis of empirical experience and contractor's "feel." A cost model will permit the rigorous cost calculation of various alternatives and help in selecting the solution of minimum cost.

A.9.6 Construction Cost Control

By means of a cost model supplemented by CPM or PERT reticular networks, it would be possible to monitor costs along the construction period of the project. Here too, the continuous analysis of the cost effect of inevitable construction variances not provided in the planning stage, will permit prompt decision making to identify minimum cost solution. Moreover it will be possible to forecast the tunnel cost at completion as a function of expected trends in labor and material cost.

A.9.7 Model Development Probability

Is such a wide ranging, multipurpose cost model actually feasible? While in principle there appears to be no conceptual obstacles, the development of a cost model incorporating the features above outlined, presents arduous practical impediments, demanding interdisciplinary cooperation and multiple skills. Existing cost models for tunnel work have little practical value, since in most cases, their objective seems simply to represent, in some detail, construction costs. As important a component as this may be, construction cost has to be considered a "derivate" cost component only. In fact the most promising cost optimization area lies at the drawing board, in the planning and design stage of the project.

Great analytical obstacles will be encountered in attempting to correlate into multiple relationships, geotechnical variables (both micro- and macro-), with design details of the tunnel structure and construction techniques. Such variables as labor rates, insurance rates, safety regulations, details of construction methods, unit material costs, maintenance costs, all need to be made an integral part of such a model. Finally, if the objective is cost optimization, any cost model neglecting to link construction and design details with geotechnical parameters will have doubtful value, and will be relegated into the realm of futile mathematical exercise with no utilitarian use.

A.10 PRODUCTIVITY IN SOFT GROUND TUNNELING: A VALUE ENGINEERING APPROACH

In line with the original scope of this report, Value Analysis work has been here confined to cost elements ascribable to the degree of accuracy in predicting geotechnical variables.

While pursuing the intended objectives, inevitably a few thoughs of a wider scope were born, which merit reporting. After all, the ultimate objective of research work of this nature is to reduce the cost of tunnel projects in the nation. In recent years, basic changes in the U.S. economic outlook, alerted government policy makers to the necessity of maximizing productivity (broadly defined as the ratio of input resources to output products) at all levels of economic activities. Underground excavation work - in economic jargon an "intermediate product" - contributes about $2 billion to the current GNP: concern for productivity in this sector is fully justified.

Tunnel "cost," expressed in dollar figures, is a conventional numerary to value the physical resources employed in creating the tunnel "product" (material, equipment, manpower, energy, capital). Optimizing tunneling costs, thus means minimizing the use of the above resources for a final product which has to satisfy certain prefixed standards of functionality, aesthetics and environment. Thus the objective of minimizing cost in SGT is unequivocally identified with the objective of maximizing productivity.

The discipline of Industrial Economics has long made available the theoretical principles and practical tools for cost optimization in manufacturing operations. There is no obstacle in attacking soft ground tunneling costs with similar methodology, if the target is to minimize costs for a given quality/functionality standard. The bulk of institutionally sponsored research seems directed toward analyses of technical nature and exploring for technological breakthroughs. In the long run, there is no doubt that advancement in the status of the art will contribute to reduce tunnel cost.

However within the limits of the available technology, the most immediate, direct instrument for controlling cost probably is a systematic, strict Value Engineering approach to the whole tunneling process, from planning and design details, downstream to construction and maintenance operations.

Unfortunately, Value Engineering work in SGT is, at best, fragmentary. Cost analyses which could be equated to some rudimental form of Value Engineering, are mostly concerned with technical details, and in comparing alternative solutions too often ignore crucial cost factors such as union rules, wage levels and trends, and safety regulations. All too often, design choices are made on the basis of the best "technical" or "political" solution, cost being considered a numerary seldom seriously questioned in the planning/design state of SGT projects.

Engineering solutions for a given functionality standard are manifold. Sometimes a few seemingly innocent words, carelessly entered in critical documents (i.e., specs prescribing a concrete gravel mixture of quality not available in the project area) may result in a significant cost burden. Still more costly (as it has been here proven) can be tender documents containing incomplete or imprecise geotechnical information. Both these cases could and should be avoided, on the basis of Value Analysis considerations. In fact, the very scarcity of Value Analysis literature dealing with SGT has hindered designers and engineers, not legendarily cost conscious anyway.

It is not difficult to envision a Value Engineering Manual for SGT discussing minimum cost decision criteria in planning, design tendering/bidding and construction operations, as a function of the whole band of factors determining cost. By the very nature of the objective, that is minimizing cost for given quality/functionality standards, such a manual will not contain directives but rather discuss criteria for "technical" decision making: tunnel geometry and geotechnical factors being equal, regional differences and time instability of such heterogenerous factors as labor rates, work rules, safety regulations, insurance costs and interest rates, just to mention a few, will suggest different minimum cost solutions for the same problem in different areas and/or times.

To conclude, independently from and parallel to research work of strictly technical nature, a comprehensive Value Engineering study of the whole soft ground tunneling process appears to be the tool without which cost/productivity optimization will remain more an empirical art, with doubtful impact on a national scale, than a rigorous and fruitful discipline.

A.11 CONCLUSIONS

A.11.1 General

This appendix has dissected the economics of soft ground tunneling in regard to the cost components affected by geological, hydrological and soil conditions. Most of the quantitative analyses here outlined are based on estimated data. Although a conservative approach has been taken in estimating data, the possibility exists of some error on the optimistic side. Nonetheless two irrefutable conclusions can be drawn with absolute confidence:

1. Tunneling cost is dependent upon geotechnical variables, introducing cost ranges from 1 to 5 or more (see paragraph A.2).
2. Tunneling cost is influenced by the degree of precision with which geotechnical variables are predicted (see paragraph A.4).

Consequently, adequate and accurate knowledge of geotechnical parameters in the planning and design stages is a necessary condition, although not per se sufficient, for tunneling cost optimization.

A.11.2 Geotechnical Information

Correct geotechnical information may produce cost benefits in every step at a tunnel project:

Planning: Select the most geohydrologically favorable alignment. (Minimize cost of construction.)

Design: Work with more reliable safety factors and stability formulas. (Economize on cost of material.)

Construction: Select excavation technique/equipment for optimum rate of advance; eliminate time delay for unexpected conditions; reduce the frequency of certain types of accidents; minimize negative tunnel impact on the environment. (Realize cost savings in manpower, equipment, insurance premiums, environmental impact.)

On a nationwide scale such economic benefits have been estimated to be in the order of 5-8% of the value of underground excavation work: on this basis for the period 1975-1984, cumulative cost savings of up to approximately $2 billion (1974 $) could be expected, depending upon the construction sector considered.

A.11.3 Man Hour Rates

Finally, from 1969 to 1973 man hour rates for tunnel workers in Washington D.C., New York and California averaged increases of about 50%. This exceeds the increment of any other major inflationary indicator. It is safe to assume that gains in the average rate of advance in soft ground tunnel work (if any) have been lower than the rate of increase of unit labor costs. Whatever the reason for such a trend, this means that during the past five years the real cost of tunneling has been drifting upward, in spite of intense R&D effort to reverse the trend.

A.11.4 Reduce Tunneling Cost

The key to control and reduce tunneling cost lies in a comprehensive effort to increase productivity, as broadly defined in paragraph A.8. Dismissing for the present time technological breakthroughs, which seem too far away in the future, methods and means exist which can be put to work today to achieve the objective:

1. A rigorous Value Engineering approach to the whole tunneling process.
2. A computerized cost optimization model (a practical one only).

3. Collection of adequate and accurate geotechnical information and proper use of same.
4. Safety Engineering, and more reasonable labor insurance arrangements.
5. Moderation in wage demand and union work rules (elimination of feature bidding and often unreasonable demands).

APPENDIX B - CASE HISTORIES TUNNELS IN SOIL

B.1 CASE HISTORY NO. 1: TORONTO SUBWAY, SECTION E 1

Two tunnels, 26 feet apart, were constructed individually in silty, dense sand, by hand mining in a 17.5-foot diameter shield; each tunnel was lined with two-foot cast-iron rings. The water table was below the tunnel invert. The tunnel construction was carefully monitored, and the following types of information were recorded:

Surface settlements along tunnel centerlines.

Surface settlements along lines at right angle to centerline (generally six points on each line).

Building settlements, floors and walls.

Crown deflection and lengthening of horizontal axis after ring was shoved out of tail of shield.

Grout consumption for tail void filling, by bag count.

Runs recorded at face, approximate cubic yard estimates, with commentary and description.

Records of soil characteristics encountered, percent silt, and moisture content.

Centerline settlements were significant, varying from 0.8 inch to a maximum of 4.0 inches over the north tunnel (which was driven first), and from 0.7 to 8.4 inches over the south tunnel. Figure B1 shows the general distributions of centerline settlements for the two tunnels. Settlements over the south tunnel are clearly distributed more irregularly and are about twice those over the north tunnel.

Typical cross section profiles, showing the development of two relatively independent settlement troughs, are given, on figure B2. It is apparent that the general shape of the troughs is independent of the magnitude of settlements. An analysis of the settlement troughs shows that the trough width, defined as 2i (the width between points of inflection) is 13.5 to 15.5 feet, a rather narrow trough. The analysis also indicates settlement volumes from the north tunnel of 1.0 percent average, 2.5 percent

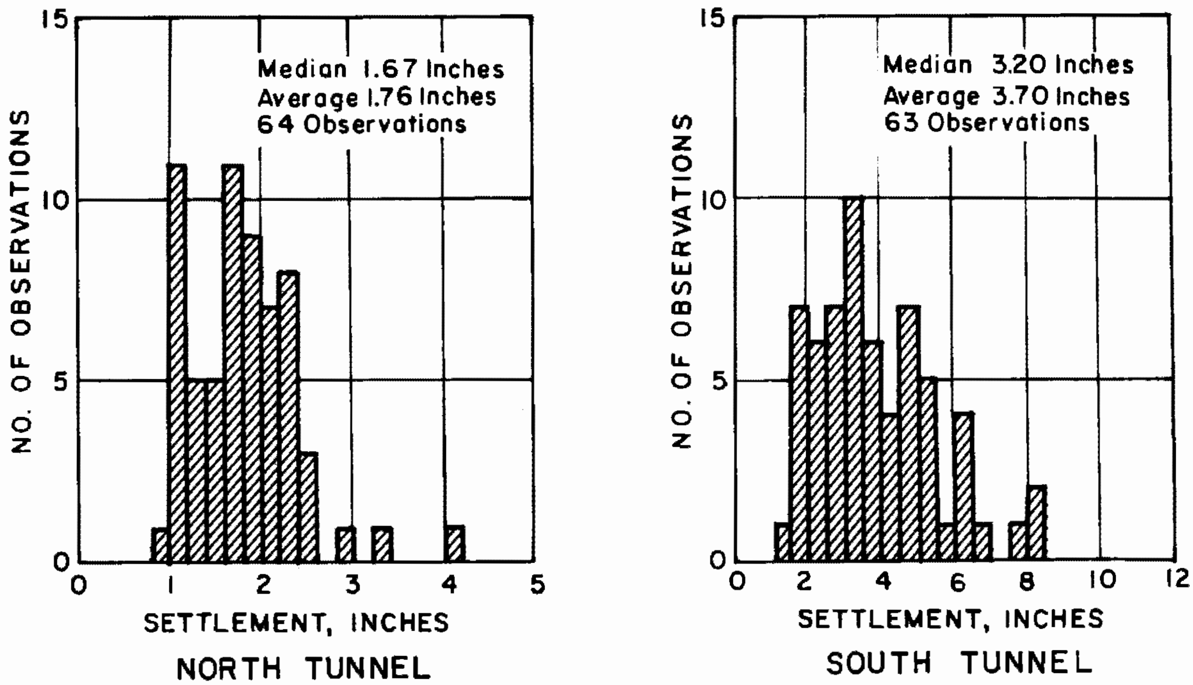


Figure B1. Centerline Settlements, Toronto E1.

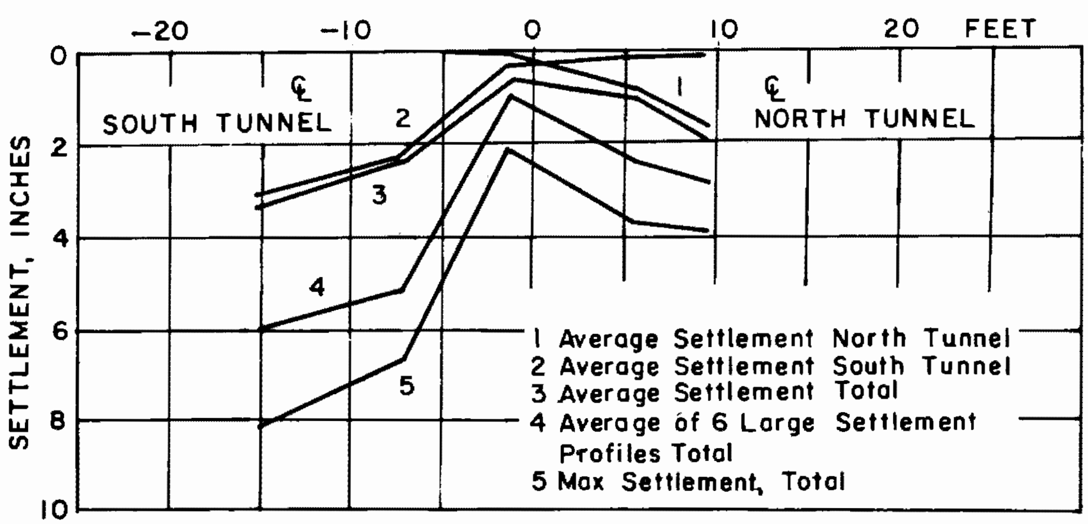


Figure B2. Typical Settlement Trough Profiles, Toronto E1.

maximum; and for the south tunnel, 2.3 percent average, 5.3 percent maximum, measured as percentages of the tunnel volume.

For a stretch of 800 feet of the north tunnel, recorded face runs amount to 30 cubic yards, or about 0.4 percent of the excavated volume; grout takes were of the order of 85 to 90 percent of the theoretical tail void volume, indicating an additional ground loss of 0.5 to 0.7 percent. This yields a total ground loss of 0.9 to 1.1 percent, where the average settlement volume was about 1.0 percent.

For the south tunnel, recorded face runs on the same stretch amounted to 83 cubic yards, or 1.1 percent of the excavated volume; the grout take was about 75 percent of the theoretical tail void volume, indicating an additional ground loss of about 1.2 percent. Here the total ground loss was about 2.3 percent, where the average settlement volume was 2.3 percent. On the average, then, the ground loss due to face runs was smaller than the tail void loss, though locally it could be much greater.

This case history illustrates the influence of ground disturbance on the settlements. The construction of the north tunnel had imposed such deformations on the soil that the subsequent driving of the south tunnel resulted in settlements more than twice those of the north tunnel. This is characteristic of tunneling in soil whose small cohesion can be destroyed by minor disturbance.

Based on a comparison between recorded soil characteristics and ground losses it has been concluded tentatively that a silt content of five percent provided sufficient cohesion for the lower half of the face to remain stable, but a silt content of 10 percent was required for the upper half of the face to remain stable. A much higher silt percentage was required to prevent sand from falling on top of the rings in the tail void.

Summaries of crown deflections and squat (or increase in the horizontal diameter) of tunnel rings are shown on figure B3. The average amounts to about 0.4 percent of the tunnel diameter, with a maximum of 0.7 percent. The squat of the north tunnel is slightly greater than that of the south tunnel, possibly because of the

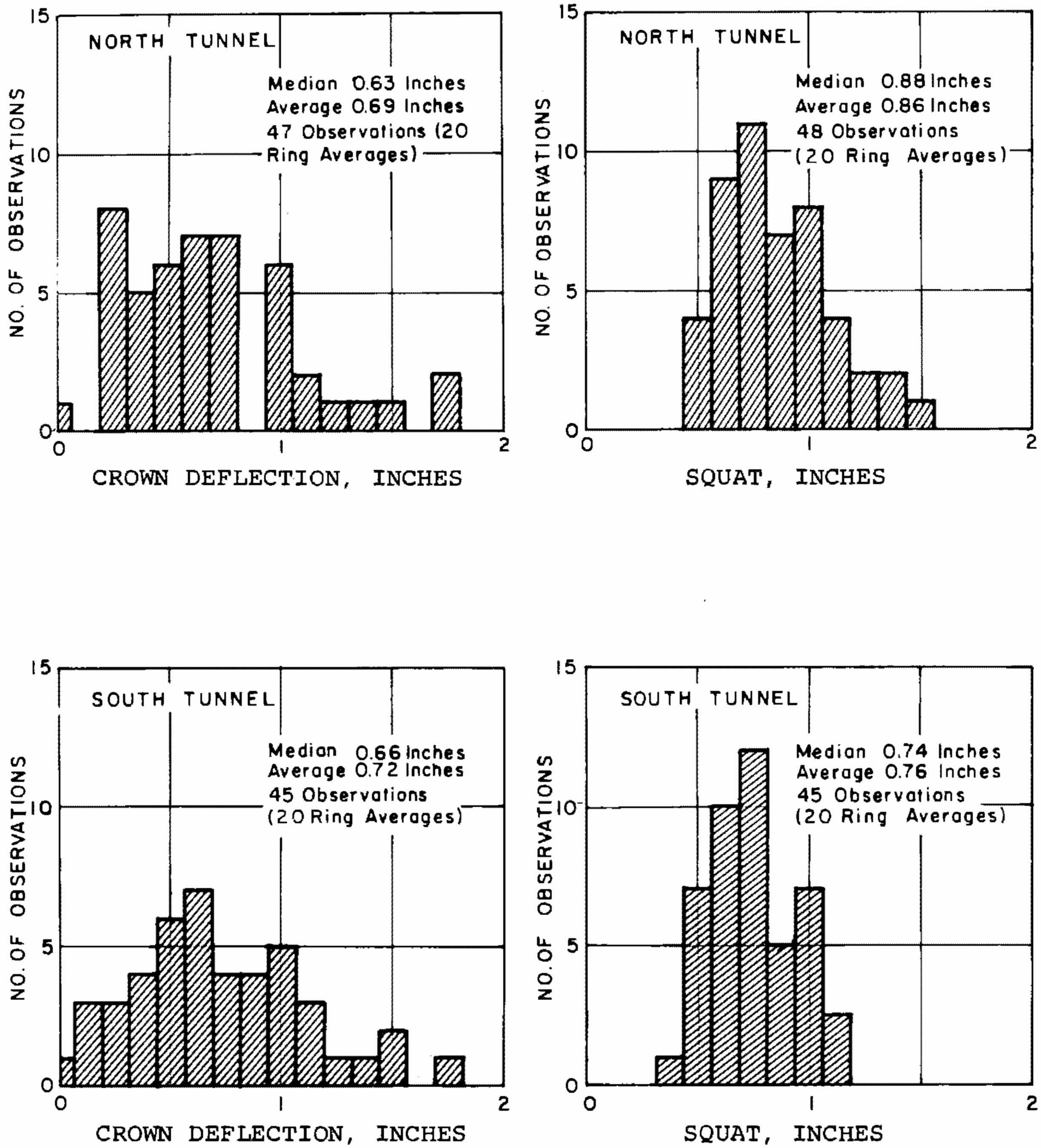


Figure B3. Lining Distortions, Toronto E1.

influence of the south tunnel construction on the north tunnel. The squat is independent of soil conditions and ground losses. Crown deflections are rather more irregular than the squat reformation, probably because of the irregularities in the way the tunnel rings settle toward the bottom of the tail void space.

Settlements of three buildings directly above the tunnels were significantly less than free field settlements; average floor and wall settlements ranged between 0.5 and 1.7 inches. One of the buildings, a three-story brick building, was protected by grouting a five-foot-deep soil volume beneath footings with chemical grout (chrome-lignin); after the tunnel shield passed, cement grouting was applied to fill any voids. The protection program was successful; only minor plastic cracking appeared in the school building.

B.2 CASE HISTORY NO. 2: TORONTO SUBWAY, SECTION B 4

Two tunnels were driven 21 feet apart center to center, 34-40 feet deep to the centerline, by hand mining in 17.5-foot-diameter shields. The lining consisted of two-foot cast iron rings grouted with neat cement grout. The tunnels, about 1800 feet long, were driven with invert and sides in glacial till, but with the crown in water-bearing sand or silt, or in silty clay. Compressed air (five to 11 psi) was used except for the first few hundred feet.

Two typical soil columns are shown in figure B4. The groundwater is high, and the soils above the ground generally granular and dense. The silty or varved clay shown at or below crown elevation has liquid limits of 24 to 40 percent, plastic limits of 15 to 20 percent, and natural moisture contents or 20 to 37 percent; its undrained shear strength is about 700 psf. The glacial till just beneath is less plastic (LL = 23 percent, PL = 15 percent, MC = 15 percent), but has only slightly higher strength. The glacial till below the invert is very hard (shear strength 6000 to 7000 psf). Boulders in the till reached a maximum size of four feet but in general did not present excavation problems.

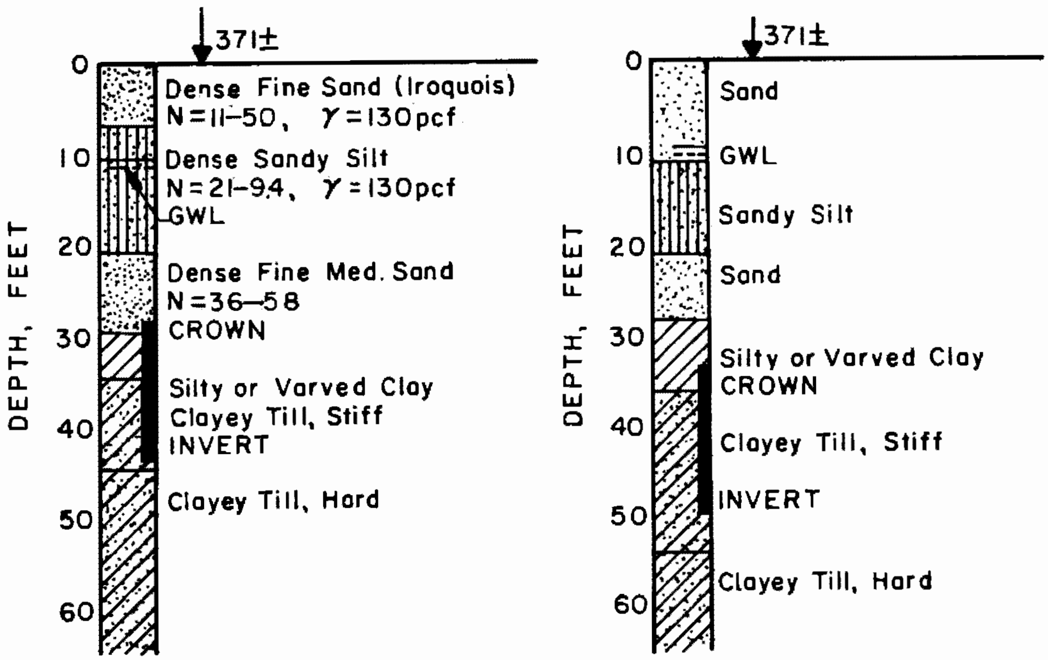


Figure B4. Typical Soil Conditions, Toronto B4.

Tunnel construction was monitored by settlement measurements taken at about 100 locations, including steel points driven into open ground, building walls and isolated footings. The ground-water level was measured in a small number of piezometers, and tunnel progress and soil conditions were recorded. In addition, ring distortions were measured, and an attempt was made to determine face movements through probes driven into the face.

The settlements where sand prevailed in the crown were quite variable, but tended to follow a reasonable pattern when ranges and averages of a number of readings were viewed. Settlements over the first driven tunnel, the south tunnel, are shown in figure B5. The trough width is about 2i = 68 feet, and the average trough volume is about 2.8 cubic feet per foot (or about 1.2 percent of the excavated tunnel volume) while the maximum trough volume is about 1.8 percent of the tunnel volume. Figure B6 shows a variety of total settlement data after the passing of both tunnels, in areas with and without air pressure, on buildings

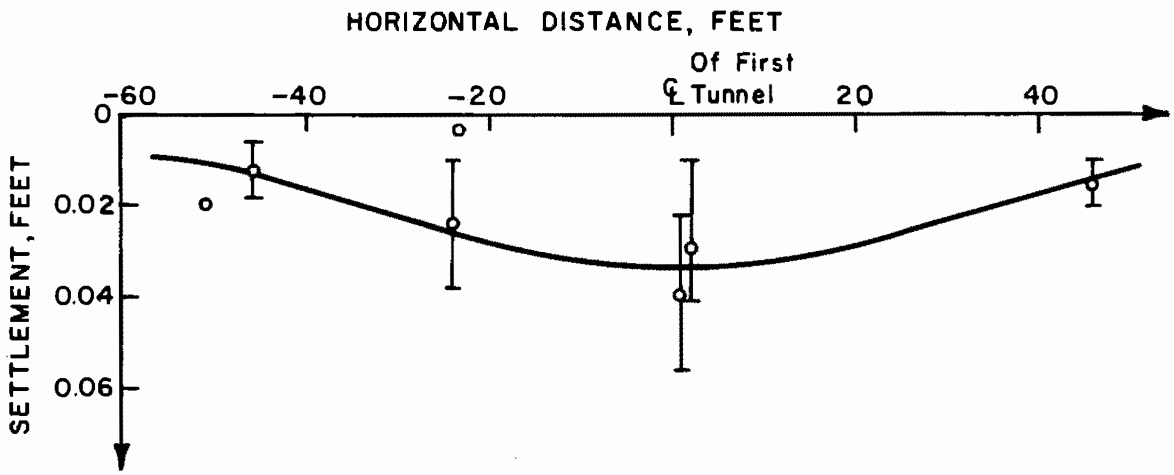


Figure B5. Settlements Over First Tunnel, In Sand.

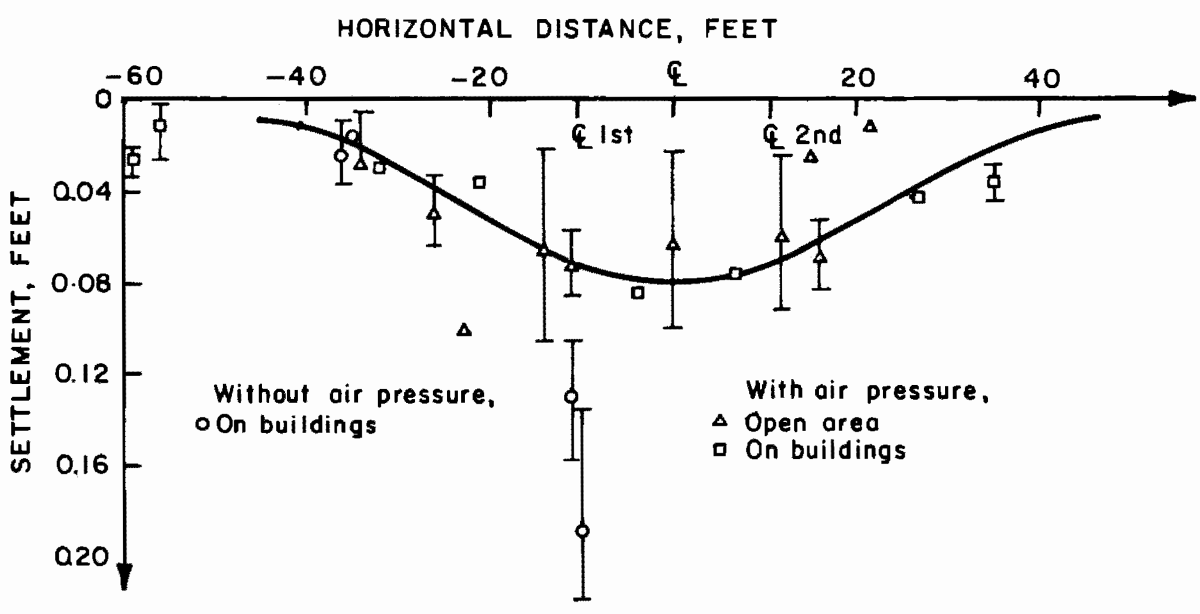


Figure B6. Settlements Above Both Tunnels, In Sand.

and in open areas. The total settlement pattern is nearly symmetrical about a line halfway between the two tunnels, as if one tunnel of a width equal to the distance between outer springlines had been driven. The average trough width is 2i = 44 feet, the average trough volume is 4.4 cubic feet per foot (0.9 percent of the excavated volume), while the maximum volume is about 3.0 percent.

These settlement troughs are unusually wide, though the magnitudes of settlement are small to moderate. A plausible explanation for the unexpected width of the trough may lie in the stratification of the soil. Ground loss due to soil-water flow would extend in horizontal directions rather than veritically because of the cohesion of soils above; secondly, this cohesive soil would tend to distribute settlements over a wider area. It would seem in this case that the disturbance from the first tunnel did not significantly increase settlements above the second one.

Consider settlements in the area with silty or varved clay in the crown. Figure B7 shows settlements due to the driving of the south tunnel and the total settlements after both tunnels were finished. The first trough has a width of 2i = 32 feet; the second, total trough 2i = 38 feet. The trough volumes are, respectively, 0.6 and 1.2 percent of the excavated volumes. Even though the settlement volume caused by the second tunnel is significantly greater than that from the first tunnel, the final trough is nearly symmetrical about the mid-point. A possible explanation may involve latent displacements, generated by the first tunnel and made actual by the driving of the second tunnel.

A plot of settlement versus distance from the shield is very useful for diagnostic purposes. Figure B8 shows that most of the settlement occurs over the tail void rather than the face. This is verified by the fact that no movement toward the tunnel could be measured on several probes driven into the stiff glacial till face.

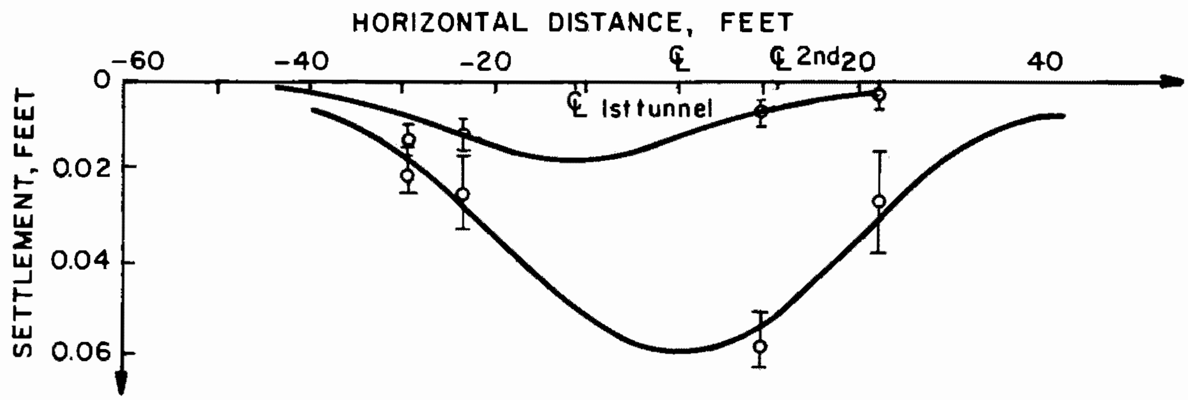


Figure B7. Settlements Above Tunnels In Clay.

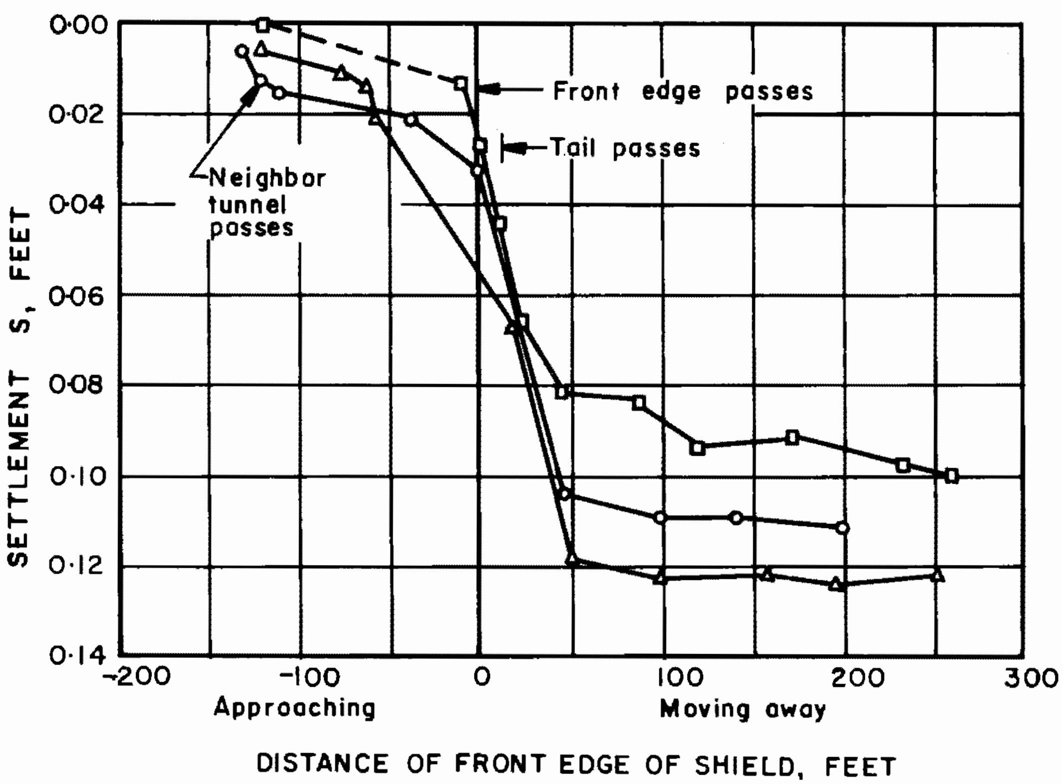


Figure B8. Centerline Settlements As Function
Of Shield Advance, Toronto B4.

Squat due to the passing second tunnel amounted to a maximum 3/4 inch, of which 1/4 inch occurred at once, the rest over 10 days. Such distortions, however, were only found in the first stretch, where no air pressure was used and where significant stability problems and grouting difficulties had occurred. Elsewhere, the distortion of the south tunnel was of the order of 3/16 inch, that of the north tunnel about 1/8 inch. An additional 1/4-inch distortion occurred when the air pressure was normalized, accompanied also by a very minor additional settlement (0.02 feet). The order of magnitude of the total relative distortions (squat/diameter) was of the order of 0.18 to 0.48 percent.

A number of structures were located directly above the tunnel, including: a two-story steel frame concrete block building; a two-story brick building; a two-story steel frame brick building; a one-story brick building with basement; a one-story steel frame building with corrugated steel sidings; seven horizontal cylindrical steel tanks 10 x 25 feet on brick saddles; and operating railroad tracks. No underpinning or other protection was provided and virtually no damages were incurred. Only a one-story brick office building with basement showed minor distress under settlements of 0.15 feet, differential settlements of 0.05 feet.

B.3 CASE HISTORY NO. 3: INTERCEPTOR SEWER, STATEN ISLAND

An interceptor tunnel, 7,000 feet long and about 10 feet in diameter, is being mined by Richmond Constructors along Richmond Terrace in Staten Island, New York. A major portion of the tunnel runs through a glacial till with numerous boulders, many larger than two feet in longest dimension. A Robbins mole employing an articulated hoe excavator and a conveyor belt advances the tunnel, with the steel segmented liner being erected inside the tail of the shield.

Though the mole in theory can advance the tunnel many tens of feet per shift, the actual production rate is frequently only two to four feet per shift. Two-foot boulders are the maximum that can be handled by the conveyor, larger boulders must be split by hydraulic means. Boulders encountered along the periphery of the shield must be worked out and properly positioned for splitting, the work performed in part manually. Such a procedure occasionally leaves large voids outside the tunnel that are difficult to backfill.

The mole costs approximately $500,000; the justification for the use of such a relatively expensive piece of machinery lies in the potentially high production rate. However, with production rates between five and 15 feet a day (typical for the bouldery area) the return on the investment is questionable, and the use of the mole can be justified only if the bouldery area constitutes a relatively short portion of the total tunnel length.

On December 28, 1973, the Staten Island "Advance" (a newspaper) reported the breaking of a 16-inch sewer, located about nine feet above the tunnel and five feet below the ground surface. Presumably, the breakage occurred because of ground movements generated when a large boulder was removed in the crown, leaving a void above the crown of the face. Tunneling had to be halted because of large inflows of sewage. The sewer break with its associated water inflow and temporary face instability occurred about 3 a.m.; the sewer was exposed and repaired during the following day. The incident required the temporary rerouting of surface traffic and most of a day's disruption of tunnel work but fortunately caused no injury or surface traffic accidents. With tunnel construction costs at about $600 per hour, the cost of the incident can be estimated at somewhat above $10,000. This includes the repair of the sewer, but does not account for inconveniences associated with surface traffic detours.

An adjacent, similar tunnel contract employs a rather similar shield and excavator, but with a wider conveyor belt for muck removed. Because there is less need for boulder splitting, tunneling progress has been significantly less influenced by the boulders.

B.4 CASE HISTORY NO. 4: DETROIT TUNNEL

During construction of a tunnel in Detroit, compressed air inadvertently found its way to an old permeable brick sewer, causing it to back up explosively. A fine home was virtually filled with sludge and eventually had to be purchased by the contractor. Thus, old sewers may be significant obstacles to certain types of tunneling, even though they may not be in direct interference.

B.5 CASE HISTORY NO. 5: SAN FRANCISCO BART, LOWER MARKET STREET, CONTRACT B0031

Two 18-foot o.d. segmented steel-lined tunnels passed through soft clay beneath the Ferry Building, whose foundations had been picked up by underpinning. The old timber piles had been left in place. Timber piles were also known to exist beneath abandoned wharves and cable car railways, but their exact locations were not known. Some 600 piles were expected in this area, and provisions were made in the specifications for anticipated pile problems, including a bid item for each pile cut.

The bid price for cutting timber pile was $750 per pile; the real and possibly greater cost to the contractor, however, is not known. The highest bid price for cutting piles from any contractor was $3,800 per pile. A total of 896 piles were in fact, encountered, including one steel H-pile and one 12 x 12-inch concrete pile.

Though the locations of piles beneath the Ferry Building were presumably known in plan, at least at the pile cap elevation, the pile locations actually observed in the tunnels bore little resemblance to the pile plan (see figure B9).

If the shield were driven up against a pile there would be a considerable risk of displacing it horizontally, creating an opening for a serious air loss. Therefore, each shove was preceded by a probe, using an air-operated wood auger every six inches around the hood perimeter. The probe reached outward at least 40 inches, or 10 inches longer than the standard shove. Timbers were severed with a hydraulically operated chain saw,

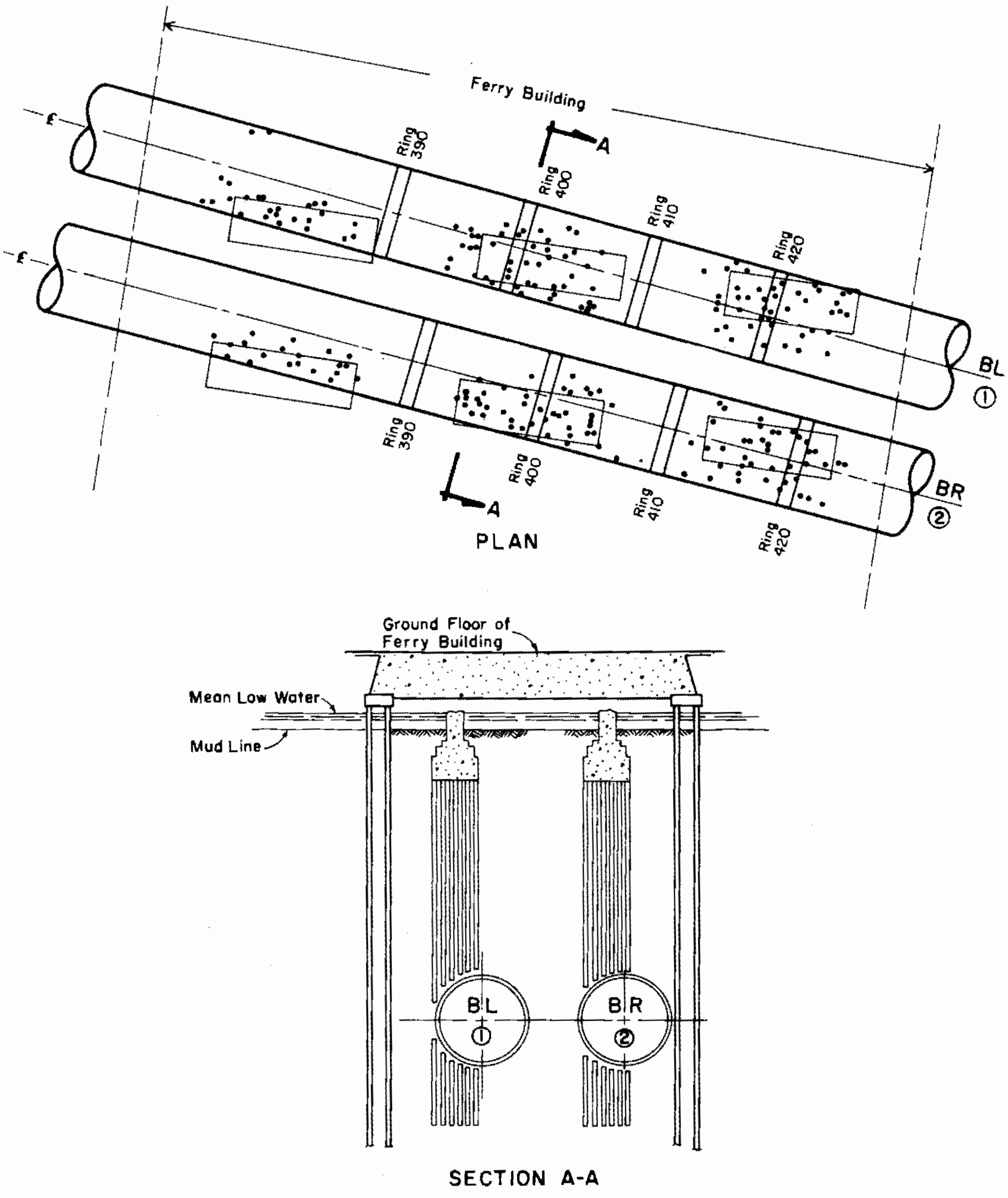


Figure B9. Tunneling Conditions at Ferry Building.

cutting the pile above the top of the shield but leaving a stub of about 2.5 feet above the shield bottom. The bottom part of the pile was then pushed over by the shove. In general, the shield was shoved to within six inches of a pile before the pile was cut free of the soil by hand and sawed.

Some distortions of liner rings resulted from residual loads from the piles and from disturbances around the piles, and point loads on the liner occasionally caused visible dimples, which were reinforced by welding stiffener plates between the ribs. Some difficulties arose in connection with the caulking of segment joints to secure watertightness.

Probing ahead and cautious shield shoving no doubt slowed construction work, but it is doubtful that an accurate prior knowledge of pile locations would have reduced costs significantly. A more efficient method of probing ahead of the shield, on the other hand, might have reduced certain risks but probably not costs, since such probing would have subtracted from productive driving time. To be really useful, prior location of piles would have to be done with an accuracy of six inches to a foot, so that probing would not be required. (Kuesel, 1972; Whiteman, 1969).

B.6 CASE HISTORY NO. 6: SOUTH CHARLES RELIEF SEWER, BOSTON, AT CHARLES RIVER

This tunnel was shield driven (1958-60) with air pressure varying from 6 to 12 psi; its outside diameter was 11.33 feet, and it was lined with steel liner plates reinforced to resist shove jack pressure. The tunnel shield passed close to a batter pile supporting a bridge abutment. The ground disturbance and relative movements of soil, pile, and shield created a chimney to the surface along the batter pile, permitting the escape of compressed air and the loss of material. The blow lasted about 15 minutes; the air pressure was allowed to drop from 10 psi to four psi to reduce air loss while remedial measures were taken to plug the leak. Though this incident received some attention from the news media, it caused only limited damage and a modest delay in construction.

B.7 CASE HISTORY NO. 7: WASHINGTON METRO, CONTRACT C4

In the area of the Watergate Apartment Project, an unusual and unexpected tunneling problem was found during the construction of the twin Metro tunnels and two shafts (C4-1 and C4-2). Unknown to the general soils consultant and the section designer, this locality was for many years the site of a Washington Gas Light Company installation. It would appear that long-term seepage or disposal of fluids from the gas works carried tar-like substances into the ground, where they settled out, predominantly near the soil-rock interface.

During excavation for the shafts, this tar-like material was first uncovered. It gave off noisome fumes that were on occasion ignited by the action of the excavation tools, and was in general unpleasantly sticky and messy. Construction drainage water pumped into the Potomac River from the tunnel was heavily polluted and formed an oily scum on the river, a problem partly managed by Coast Guard skimming equipment.

Fortunately, the quantity of noisome, flammable, and potentially explosive fumes was small, and no serious accidents occurred.

This is one instance where a gas detector, employed during geotechnical preconstruction investigations, might have disclosed a problem that could have been more serious than it turned out. On the other hand, it may well be argued that since Washington is not known for significant natural gas occurrences, it would not be reasonable to employ such gas detectors.

B.8 CASE HISTORY NO. 8: CORROSION PROTECTION OF TUNNELS IN NEW YORK, SAN FRANCISCO AND WASHINGTON

The 63rd Street Tunnel in New York is a trench type tunnel. The river portions consist of prefabricated tube sections made of concrete and with a steel skin plate, and completely covered with soil backfill. The prefabricated steel shell sections are electrically isolated from the adjoining rock tunnel sections by special insulated joints. The steel sections are protected with a sacrificial anode cathodic protection system, designed on the basis of special stray current measurements made before and after installation of the tube sections.

Segmented steel or iron lined tunnels for the San Francisco BART system and for the Washington Metro are similarly prepared for cathodic protection. In all three instances, tests will be made after completion of construction and after train operations have started in the tunnels to determine if cathodic protection is needed, and if so, to what extent. The details of the cathodic protection scheme will then be worked out and implemented.

B.9 CASE HISTORY NO. 9: TORONTO SUBWAY, UNIVERSITY AVENUE LINE

To eliminate noise problems caused by driving soldier piles along this cut-and-cover subway, the 12-inch soldier piles were installed in prebored holes, 20 inches in diameter, to rock at depths of 15 to 40 feet. In the northern part, where the overburden soils were relatively clean silty clays and clays, predrilling proceeded at an average rate of 40 feet per rig hour. On the other hand, in the southern part of the line, removal of near-surface concrete and brickwork debris from old structures and fill, and a multitude of shale and limestone lenses in the lower overburden soils, reduced the average drilling rate to six feet per hour for a block length of structure. This had a significant effect on the overall cost and rate of production. It is not clear whether a better solution to the problems would have been found if better data regarding these obstructions had been available, but it is certain that a more equitable bid price, a lower contingency, and a more satisfied owner and contractor would have resulted.

APPENDIX C - OVERVIEW OF GEOPHYSICAL EXPLORATION METHODS

The following tabulation of geophysical techniques represents a summary of current state-of-the-art capabilities and applications as described in the technical literature. Since it is intended to represent the techniques in terms of soft-ground considerations, certain common-use applications are de-emphasized in preference to utility in the soft ground environment. Statements of accuracy are intended to represent overall capability: specific ground conditions may result in considerably improved accuracy, or accuracy may be degraded beyond the values listed.

Available geophysical methods are separated into two groups in the tabulation, with the first group including techniques conducted at the ground surface, and the second group those usually performed in or between boreholes.

TABLE C1. OVERVIEW OF GEOPHYSICAL METHODS EMPLOYED FROM THE GROUND SURFACE

Method	Effective Depth Range	Brief Description of Technique	Applications	Parameter Measured	Measurement Accuracy	Parameters Inferred	Accuracy of Inference	Comments
Seismic Refraction	0-200 ft. + (typical)	Seismic impulse introduced at or near ground surface, impulse transit time to a linear array of geophones measured, pattern of transit times interpreted to determine subsurface velocity units, unit thicknesses, and attitudes	Mapping of subsurface soil/water table/bedrock velocities, depths, and thicknesses. Materials classification	Transit times of elastic waves	±1 millisecond	Apparent horizontal velocity (V) interface depths (D)	±5% x V ±10-15% x D	Survey depths approximately one-third of maximum source-geophone distance, resolution of layers limited by seismic wave length/velocity, and density contrasts. Calibration by observation in boreholes improves accuracies significantly
Seismic Reflection	200 ft. +	Seismic impulse introduced at or near ground surface, impulse transit times from surface to subsurface reflector to surface recorded and measured at surface geophone positions, pattern of transit times interpreted to determine subsurface velocity units, unit thicknesses and attitudes	Mapping of subsurface soil/water table/bedrock velocities, depths, and thicknesses	Transit times of elastic waves	±1 millisecond	Apparent vertical velocity (V) interface depths (D) attitudes of interfaces	±5% x V ±5% x D ±10°	Shallow surveys subject to direct/refracted signal interference. Resolution of layers limited by seismic wave length/velocity and density contrasts. Calibration by direct measurement in borehole improves accuracies significantly
"Vibroseis" seismic survey (primarily reflection)	200 ft. +	Seismic sweep frequency signal introduced at ground surface, transit times of correlated waveform to linear array of geophones measured, pattern of transit times interpreted to determine subsurface velocity units, unit thicknesses and attitudes	Mapping of subsurface soil/water/bedrock velocities, d depths, thickness, and attitudes	Transit times of elastic waves	±2-3 millisecond	Apparent vertical velocity (V) interface depths (D) attitudes of interfaces	±10% x V ±5-10% D ±10°	See notes for refraction and reflection surveys
Seismic Holography (primarily reflection)	no limits	Seismic impulse introduced at or near the surface, transit times and amplitudes of primary signal detected and measured at a grid of geophone positions, pattern of transit times and amplitudes are interpreted to determine subsurface velocity units, unit thicknesses, attitudes, and discontinuity in subsurface units	Mapping of subsurface velocity units, discontinuity of the units in particular	Transit times, reflected elastic wave amplitudes	±2-3 millisecond	see seismic notes above	see seismic notes above	Experimental, resolution is a function of discontinuity size relation to seismic wave lengths and amplitude. Large antenna composed of many geophones is required
Sonic	0-5 ft.± (max.)	Acoustic (sound) energy introduced into ground using a loudspeaker or other device as a source, transit time and signal amplitude detected either by geophone or a microphone, transit times and amplitudes interpreted to determine anomalous travel path conditions	Location of shallow discontinuities (with depth or lateral position)	Transit times Amplitudes Phase	see notes	see notes	see notes	Experimental only

TABLE C1. OVERVIEW OF GEOPHYSICAL METHODS EMPLOYED FROM THE GROUND SURFACE - Continued

Method	Effective Depth Range	Brief Description of Technique	Applications	Parameter Measured	Measurement Accuracy	Parameters Inferred	Accuracy of Inference	Comments
Resistivity/ Conductivity	no limits	Constant or slowly varying DC introduced into ground by two powered electrodes, pattern of voltages is converted to apparent resistivity pattern, resistivity pattern interpreted to determine subsurface apparent resistivity units and unit thicknesses. A wide variety of electrode positioning is used	Mapping subsurface of soil/water table/bedrock resistivity units. Rapid areal mapping of strong subsurface resistivity contrasts, grounding potential for high voltage operations	Induced voltage (V)	±1% x V (nominal)	Apparent resistance (ohms)	±2-5%	Theoretical penetration depth ranges from 0.1 to 0.4 times power-receiver electrode separation (depends upon configuration of electrode placements).
Electro-magnetic Induction	no limits	Varying magnetic field is induced by varying current in a surface coil or pair of long lines, magnetic field induces subsurface currents, currents induce a second subsurface magnetic field which induces current in a surface detector coil or pair of lines. Pattern of amplitude and phase of secondary field is interpreted to determine subsurface conductivity units and unit depth/thickness	Mapping subsurface of soil/water table/bedrock conductivity units: widely used for mineral reconnaissance	Induced voltage (V)	±1% x V (max.)	Apparent conductivity (mhos)	±1%	Penetration depth approximately 0.7 times source-receiver separation
Electro-magnetic Subsurface Profiling	0-50 ft. (max.)	Electromagnetic energy pulses introduced into the subsurface in a narrow beam, reflected energy detected very near source, pattern and strength of signal from reflectors in subsurface interpreted to determine depth to reflecting horizons or objects	Mapping of subsurface reflectors, location of anomalous subsurface discontinuities	Reflected EM pulse (transit time, amplitudes)	see notes	interfaces 2 ft. separation	variable	Amplitude of reflected pulse is recorded in shades of grey inferring interface dielectric contrasts. Wet clay layers tend to disperse signal and limit penetration depth. Interface depths a function of pulse velocities, pulse velocities variable
Electro-magnetic Pulse Sounding	no limits	Electromagnetic energy pulses introduced into the subsurface by long wire (s) reflected pulse is detected by another long wire (s), pattern, strength and shape of signals from subsurface reflectors interpreted to determine depth to reflecting horizons or objects	Mapping of subsurface reflectors, location of anomalous subsurface discontinuities	Reflected EM pulse (transit time, amplitude, phase)	see notes	see notes	see notes	Experimental, see above
Electro-magnetic Subsurface Wave Analysis	no limits	Electromagnetic field strength pattern of low frequency (~300KHz) radiowaves is measured at multiple frequencies, pattern of field strength/frequency/phasing is interpreted to determine conductivity and conductivity units in the subsurface	Mineral exploration, subsurface soil/groundwater/bedrock mapping	EM pulse amplitude/ phase	not known		not known	Experimental: has been used for some surveys, results not widely reported, theoretical basis published. Probably lacks resolution for soft ground details

TABLE C1. OVERVIEW OF GEOPHYSICAL METHODS EMPLOYED FROM THE GROUND SURFACE - Continued

Method	Effective Depth Range	Brief Description of Technique	Applications	Parameter Measured	Measurement Accuracy	Parameters Inferred	Accuracy of Inference	Comments
Magnetic	no limits	Natural magnetic field strength is measured in terms of total field or horizontal and vertical components. Strength of field and rates of change of field in traverses are interpreted to infer presence, amounts, and depths to units containing magnetic materials	Mapping of subsurface presence, amount, and depth of units containing magnetic materials	Field strength in gammas	±1 gamma	same	same	Field strength decreases with square of distance from sensor, magnetically susceptible materials in subsurface strata required.
Gravimetric	no limits	Relative strength of earth's gravity field measured at a series of points on a traverse line or at points on a grid, gravity measurements corrected for topography, elevation, and other factors, corrected measurements and rates of change from point to point interpreted to inter-subsurface density units, unit thickness, and lateral unit limits	Mapping of subsurface density units and subsurface unit discontinuities	changes in gravitational field in milligals (gal-gravitational acceleration at earth's surface	$\pm 1 \times 10^{-4}$ milligals	same	same	Field decreases as square of distance from sensor, considerable correction effort required for mass of bodies in vicinity (autos, buildings, topography, etc.) Continuous profiling system under development for land surveys.
Thermometric	0-2 ft.± (max.)	Natural thermal radiation from ground surface measured at points or on continuous traverse anomalous changes in absolute temperature interpreted to determine presence and location of materials with differing radiative properties	Location of different soil contacts, shallow buried objects, or anomalous radiative materials	Temperature °F or °C absolute relative	±1-2°C ±.1-.2°C	same same	same same	Interference from different soil colors, density, microtopography, vegetation, etc., may be greater than anomaly sought by survey.
Radioactivity	0-2 ft.±	Natural radiation from radiogenic materials measured with Geiger-Mueller counter or scintillometer, continuously or at points in a traverse or on a grid, measurements interpreted to determine presence, location, and amount of radioactive materials	Presence, location, and amount of radioactive material in near-surface deposits	Radiation flux (particles/cm²/sec.		see note	see note	Accuracy is affected by relative amounts, distance of observation, etc., and sensor saturation time constants.
Nuclear	0-1 ft.±	Surface bombarded with high energy neutrons or gamma rays, rate of particle return with distance and time from emitting source is measured, rate interpreted to determine material density, moisture	Subsurface material bulk density, moisture content	Neutron or gamma ray flux		Bulk density Moisture content	±1% ±2%	Typically requires calibration for particular soil types to achieve accuracy indicated

TABLE C2. OVERVIEW OF GEOPHYSICAL METHODS EMPLOYED IN BOREHOLES

Logging Method	Brief Description	Applications	Parameter Measured	Measurement Accuracy	Borehole Condition	Logging Rates	Comments
Sonic/Acoustic	Pulsed transmitter in borehole tool emits sound waves that propagate through borehole fluids and sidewall strata, transmitted wave arrivals detected by transducer on tool, transit times of elastic waves interpreted from continuous log of borehole response.	Continuous subsurface seismic wave velocity profile, velocity contrast locations, inferred engineering parameters from wave velocities, fracture zone identification and location.	Transit time of elastic waves, relative wave amplitudes, depth from cable length and tool measurement.	± 0.1 milliseconds, ~1 ft. depth	fluid-filled cased/uncased	30-100 ft./min.	Eccentered tool for dry boreholes under evaluation, hole-to-hole surveys (~50 ft. separation) experimental. High Void Ratio materials (VR > 0.3) limit usefulness in soft ground
Seismic	Seismic waves initiated by impulsive source (explosives common). transit times to surface detectors (uphole survey) or detectors in other boreholes (crosshole velocity surveys) analyzed to obtain compressional and shear wave velocities	Semi-continuous subsurface velocity profile, inferred engineering parameters from seismic wave velocities	Transit time of elastic waves, depth from cable length	± 1 millisecond	fluid-filled/dry, cased/uncased, uncased common	~1000 ft./day	Has relatively wide use in engineering community; analytical difficulties in identification of direct shear wave because of interference by direct or converted compressional waves, and in identifying actual wave paths
Spontaneous Potential (SP)	Single electrode lowered into borehole, electrochemical and electrokinetic voltage potential measured against surface ground potential	Continuous subsurface profile of electrical potential in borehole inferring stratigraphy, stratigraphic changes, porosity, permeability, fluid conductivity, bulk density	Spontaneous Potential (voltage, V), depth from cable length	± 0.1% V ± 2.0 ft. depth	fluid-filled, uncased	~ 50 ft./min.	Provides clear indication of water level in borehole, often used to correlate with other logs or correct other log readings. very simple to obtain, useful for borehole reconnaisance May be obtained from other tools which include at least one electrode.
Normal Resistivity	2 or more electrodes lowered into borehole, one powered and other(s) as sensing electrodes with typical: spacing 16", 64", and 15 ft+ (short, long, and lateral, respectively). Logging consists of a continuous recording of voltage variations at sensing electrodes caused by resistance changes in sidewall strata	Continuous subsurface profile of electrical potential in borehole inferring stratigraphy, stratigraphic changes, porosity, permeability, fluid conductivity, bulk density	Potential (voltage) at each sensor electrode, depth from cable lengths	± 0.1% V ± 2 ft.depth ± 1 ft.depth at sharp contrasts of resistance	fluid-filled, uncased	~100 ft/min.	Theoretical average sidewall penetration 0.25 - 0.50 times electrode spacing (influenced by actual electrical resistivities and stratigraphic sequence), corrections for borehole conditions, strata thicknesses, fluid resistivities, and borehole fluid migration into sidewall are required.

TABLE C2. OVERVIEW OF GEOPHYSICAL METHODS EMPLOYED IN BOREHOLES Continued

Logging Method	Brief Description	Applications	Parameter Measured	Measurement Accuracy	Borehole Condition	Logging Rates	Comments
Focussed Resistivity	Similar to normal resistivity above, except additional "bucking" electrodes are used to force the current flow into a thin disc around the tool, and electrode spacing is fixed for each particular tool	Continuous subsurface profile of electrical potential in borehole inferring stratigraphy, stratigraphic changes, porosity, permeability, fluid conductivity, bulk density	Potential voltage at sensor electrodes, depth from cable length + tool measurement	± 0.1% V ± 1 ft depth ± 2 in. absolute	fluid-filled, uncased	~100 ft/min.	As above, but borehole and near-borehole influences are reduced, resolution for thin beds a function of the width of the current "disc"
Micro Resistivity	Same as above, except electrodes are held against borehole wall, electrode spacings 1 inch ±	Continuous subsurface profile of electrical potential inferring, on a very small scale, the stratigraphy, stratigraphic changes, etc.	Potential voltage at sensor- electrodes, depth from cable length + tool measurement	± 0.1% V ± 1 inch depth or less on a relative scale, ± 1 ft. absolute	fluid-filled, uncased	~100 ft./min.	As above, noting that very shallow penetration into sidewall (~ 1 inch) is typical, used primarily to detail bedding or for strata dip measurements across borehole.
Induction/ Focussed Induction	Transmitting coil in borehole tool radiates electromagnetic field into strata around borehole, field induces electrical currents in strata which induce a secondary field, secondary field is detected by receiving antennae (coil) in the tool.	Continuous subsurface profile of secondary field strength inferring stratigraphy, stratigraphic changes, porosity, permeability	Induced electromagnetic field strength (V) depths from cable length + tool measurement	0.1% V (nominal) ± 1.0 ft. depth	fluid-filled or dry, uncased or non-conductive casing.	~100 ft./min	Induction logging, particularly focussed induction logging, is considered an effective replacement for most of the resistivity logs with the added benefit of less stringent borehole environment requirements. Most effective in moderate to high void ratio materials.
Electromagnetic Nuclear Response	Transmitting coil in borehole tool radiates electromagnetic field into strata around borehole, mobile ions in strata oriented to ambient earth's magnetic field rotate to alignment with resulting new field, and precess back to earth's field when the applied field is released. Secondary field strength generated by precessing ions is measured.	Semi-continuous subsurface profile of secondary field strength inferring ground water mobility (permeability of strata)	Secondary field strength depth from cable length + tool measurement	Nominal	Fluid-filled (or dry) uncased or non-conductive casing	10 ft/min (estimated-experimental)	Provides only method to profile in-situ water mobility and inferred strata permeability vertical detail. Reduces ambiguity in other log response to bound hydrogen ions (clays in particular)
Gravimetric	Borehole traversed by tool with "vibrating string", then locked to borehole wall at selected intervals. Changes in frequency of the vibrations caused by changed gravitational attraction of borehole strata interpreted in terms of density changes	Semi-continuous subsurface profile of bulk density, also inferring stratigraphy, stratigraphic changes	Frequency changes in vibrations with depth, depth from cable length	± 2% freq. ± 0.004 milligals ± 2% density ± 1 ft. depth	any	~ 10 ft./min.	Other gravimetric measurement techniques also used (commonly less accurate), slow logging rate, data interpretation technique based on potential field theory with ordinary limits of ambiguity.

TABLE C2. OVERVIEW OF GEOPHYSICAL METHODS EMPLOYED IN BOREHOLES - Continued

Logging Method	Brief Description	Applications	Parameter Measured	Measurement Accuracy	Borehole Condition	Logging Rates	Comments
Thermometric	Borehole traversed by thermister or other temperature sensor, absolute or relative temperature recorded, temperature gradient measurement common. Based upon change of resistance as a function of temperature for senser materials.	Location of water table, inflow-outflow zones, casing anomalies, grouting levels outside casing.	Change of resistance with temperature, depth of change (cable length)	$\pm$ 1.0°C absolute $\pm$ 0.1°C relative $\pm$ 1 ft. depth	any	~100 ft./min.	Provides corrections needed for other logs if temperature variations are extreme or absolute temperatures high (> 150°C).
Visual/Imagery	Borehole traversed by camera or ultrasonic transmitting/receiver, tool, direct photographs or high resolution images from reflected acoustic waves obtained from logging run.	Semi-continuous or continuous "picture" of borehole wall conditions to identify stratigraphy, stratigraphic changes, and physical appearance to infer stability, fractures shear zones, gross grain size distribution, permeable zones, etc.	Change in visual appearance, or change in acoustic reflectance, depths from cable length	$\pm$ 1/32 in object resolution, otherwise direct location at image elevation ($\pm$1ft. depth)	any, dry or clear fluids most common	~100 ft./min.	Most uncased boreholes require flushing with clear water to use photography or television tools in saturated zones, or mud conditioning for ultrasonic tools.
Natural Gamma	Borehole traversed by Scintillometer or Geiger Counter, recording natural gamma radiation from strata, may also be used for radioactive tracer detection	Continuous subsurface profile of natural gamma radiation intensity, inferring stratigraphy (particularly clays), stratigraphic changes, and permeable zones (tracers)	Gamma radiation flux intensity, depth from cable length and tool measurements	$\pm$ 1.0 ft. depth	any	~20 ft./min.	Typically recorded with most nuclear logs as a fringe benefit, advanced analysis of spectrometric peaks adds radiogenic element proportions, makes significant contribution to estimates of clay fraction in strata, logging rates determined by sensor response time constant.
Gamma/Gamma	Borehole traversed by tool including radioactive isotope source of gamma rays and gamma ray detector (Scintillometer or Geiger Counter) response/effect of strata on gamma rays recorded continuously. Primary response/effect related to electron density of strata (electron density is directly related to bulk density).	Continuous log of gamma ray response/effect with depth, inferring stratigraphy, stratigraphic changes, porosity (with other logs), and bulk density of materials penetrated by borehole.	Flux rate of gamma rays from strata, depth from cable length and tool dimensions.	$\pm$ 2-3% density typical	any	Max. recommended for most surveys: ~20 ft./min. (depends upon sensor response time	Mudcake, borehole roughness, chemical composition tool size/borehole dimension corrections required. Improved accuracy if eccentered compensated tool is developed. May also be used in "spectrometric" mode of operation (energies less than 200,000 electron volts) to identify elements or relative amount of elements in the strata.

TABLE C2. OVERVIEW OF GEOPHYSICAL METHODS EMPLOYED IN BOREHOLES - Continued

Logging Method	Brief Description	Applications	Parameter Measured	Measurement Accuracy	Borehole Condition	Logging Rates	Comments
Neutron/Gamma	Borehole traversed by tool with steadily emitting neutron source and gamma ray detector, response of strata to neutron flux recorded continuously, primary response to hydrogen content in strata.	Continuous subsurface profile of strata response to neutron flux, inferring water content, porosity, stratigraphy, stratigraphic changes, percent saturation.	Induced gamma ray flux intensity, depth from cable length and tool measurements.	± 1.0 ft. depth porosity (void ratio) ± 10%	any	~ 20 ft./min.	May also be used in "spectrometric" mode of operation by recording particular energy bands as a means to identify presence and relative amounts of specific elements in strata, logging rates limited by sensor response time constant. Corrections usually required for borehole condition.
Neutron/Neutron	Borehole traversed by tool with steadily emitting neutron source and neutron detector, response/effect of strata on emitted neutron flux recorded continuously, primary response/effect related to total hydrogen in the strata (thermal neutron, 1.0-0.01 electron volts).	Continuous subsurface profile of strata response/effect on neutron flux with depth, inferring water content, porosity, stratigraphy, stratigraphic changes, percent saturation	Neutron flux intensity, depth from cable length and tool measurement	± 1.0 ft. depth porosity (void ratio) ± 10%	any	20 ft./min.	May also be strongly influenced by chlorine content of water (inferring salinity), logging rates limited by sensor response time constant. Corrections usually required for borehole conditions.
Neutron/ Epithermal Neutron	Borehole traversed by tool with steadily emitting neutron source and epithermal neutron detector (epithermal neutrons 1.0 to 10,000 electron volts), response/effect of strata on neutron flux continuously recorded, primary response to hydrogen content of strata.	Continuous subsurface profile of strata response/effect on neutron flux, inferring water content, porsity, stratigraphy, stratigraphic changes, percent saturation.	Epithermal neutron flux intensity, depth from cable length and tool measurement	± 1.0 ft. depth porosity (void ratio) ± 5%	any	20 ft./min.	Less effected by strata chemistry than neutron/neutron or neutron/gamma logs, logging rate limited by sensor response time constant, corrections usually required for borehole condition.
Pulsed Neutron Sources	Borehole traversed by tool with controlled neutron generator and either neutron or gamma ray detector, logs as described above may be obtained, or "lifetime" logs recorded by measuring flux decay rates after a pulse, unique use in identification of water mineralization.	Continuous subsurface profile of strata response/effect on neutron flux, inferring water content and degree of mineralization (salinity in particular), porosity stratigraphy, stratigraphic changes, percent saturation.	Gamma ray or neutron flux intensity, decay rates, depth from cable length and tool measurements	± 1.0 ft. depth, ± 5% degree of mineralization ± 5% porosity (void ratio)	any	See notes	Logging type selected by detector choice, essentially all neutron logs available from the basic tool with appropriate detector, borehole influence less for this method than for the others, logging rates dependant upon sensor time constants (rates not well established at present stage of development).

APPENDIX D - SPECIFICATIONS - DIRECT PERMEABILITY MEASUREMENTS

The specifications here presented are technical specifications judged sufficient and adequate for contractors to perform the required work. The specifications include items recommended for development in section 6. They do not include any general provisions or separate cost items that the Government may desire to include in a final procurement specification.

Appendix D is presented in three sections:

D1.1 CONTRACT OBJECTIVES

The objective is to design, fabricate, field test and prepare detailed specifications for the usage of two direct measurement permeability devices. This contract will require the following tasks:

a. Design and fabricate the necessary hardware for a borehole permeability probe and a perforated casing permeability test.

b. Conduct detailed field testing of both devices.

c. Conduct laboratory model testing and analytical evaluations necessary to study the imposed flow patterns and develop solutions for calculating permeability values.

d. Develop production drawings of the hardware.

e. Prepare a manual specifying standardized installation procedures, testing techniques, and data analysis.

f. Prepare guidelines for appropriate usage of the tests on tunnel projects, including estimated user costs.

Preliminary conceptual design and performance criteria are specified below.

D1.2 DESCRIPTION OF BOREHOLE PERMEABILITY PROBE

A conceptual sketch of the proposed hardware is shown in figure D1-1. In essence, the device consists of a porous tipped probe coupled with a packer assembly. The probe would connect to standard drill rods and could be driven and/or jetted into the soil at the bottom of a standard cased test boring. By injecting water into the probe, a falling and/or constant head infiltration test could be performed. The packer allows for sealing off the casing against upward flow during infiltration testing. The hardware and associated test must conform to the following minimum criteria:

a. The hardware and test technique must be applicable to soils having permeability ranging from 10^{-1} cm/sec to 10^{-5} cm/sec and to depths of 160 feet.

b. The system must be compatible with existing boring techniques and equipment.

c. The test must be performed quickly at minimal cost.

d. The packer assembly must allow for sealing off any flow up the casing.

e. The device must minimize clogging of soil by wash water and/or soil fines.

f. The probe must have the necessary strength to withstand driving forces.

g. The porous tip must exhibit negligible or correctable head loss but in no case greater than 10% of the total head loss.

h. The porous tip must be readily interchangeable in the field to suit the specified variety of soils.

i. Provisions for field replacement of the packer must be incorporated.

j. All peripheral equipment for conducting infiltration tests, both constant head and falling head shall be neatly packaged in a single portable unit. The unit shall contain a recording flow meter of appropriate quality; a stopwatch; a means of measuring the piezometric head in the drill rods during falling head test; a means of connection to a separate water storage tank; and a means of pressurizing the water injected into the probe.

k. Peripheral equipment required for activating and deactivating the packer must be provided in a neatly packaged single portable unit.

D1.3 DESCRIPTION OF PERFORATED CASING PERMEABILITY TEST

A preliminary conceptual sketch of the required hardware is shown on figure D1-2. In essence, the device consists of a section of special perforated casing which can be sealed at the bottom. This casing is to be incorporated into the casing string of a standard cased test boring and driven down as the hole is advanced. The hardware and associated test must conform to the following minimum criteria:

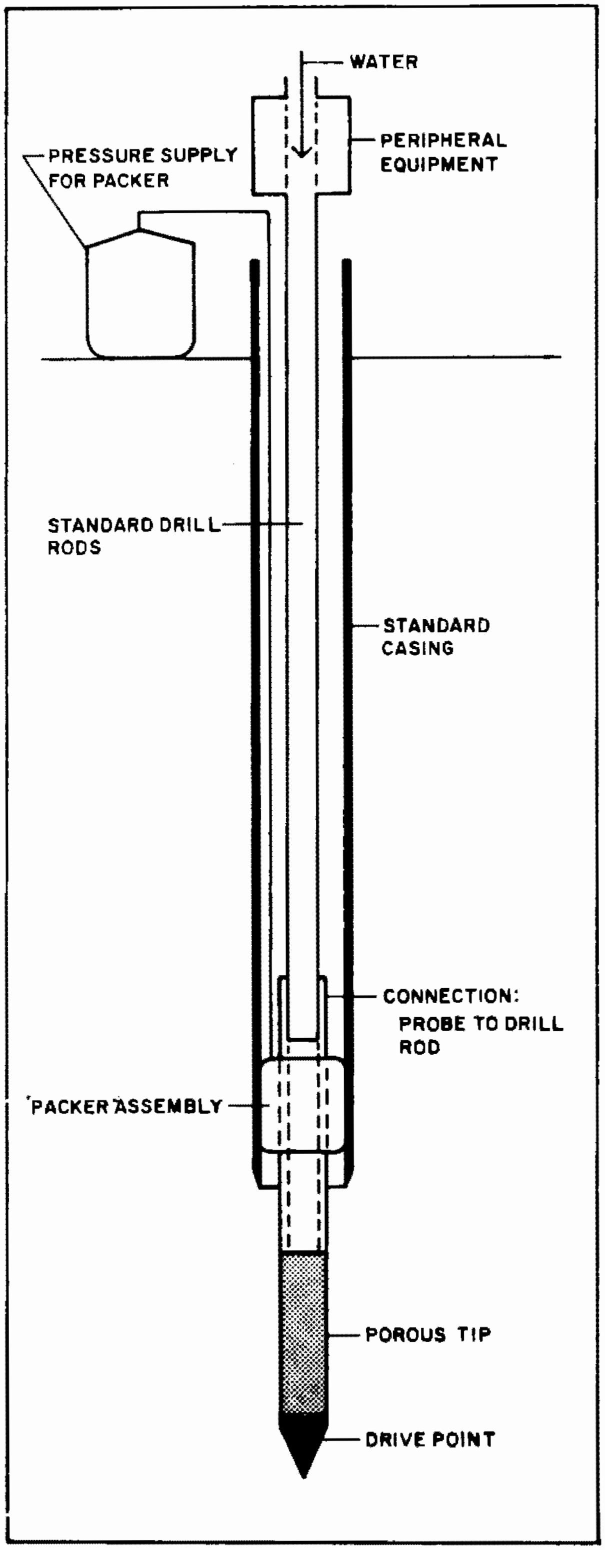


Figure D1-1. Schematic of Borehole Permeability Probe.

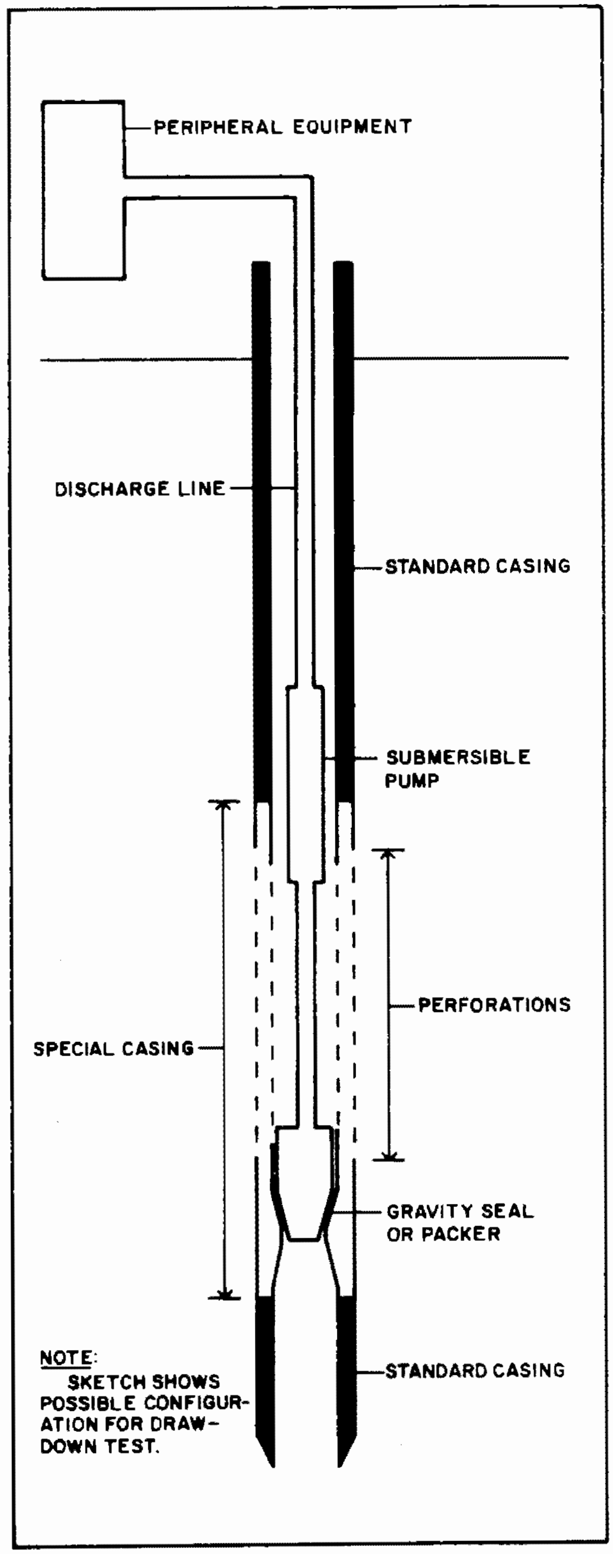


Figure D1-2. Schematic of Perforated Casing Permeability Test.

a. The hardware and test technique must be applicable to soils having permeability ranging from 10^{-1} cm/sec to 10^{-5} cm/sec and to depths of 160 feet.

b. The special casing must mate with currently used casing and become part of a standard cased borehole.

c. The test must be performed quickly at minimal cost.

d. The casing must be designed to allow for a seal at the bottom; this need not be a packer type seal, but could be a simple gravity seal as shown on figure D1-2.

e. The ability to conduct both infiltration tests and drawdown tests must be provided.

f. The casing must have the necessary strength to withstand driving forces.

g. Consideration should be given to various perforation sizes and shapes intended to minimize head loss through the casing and minimize smearing and disturbance of soil adjacent to the perforated casing.

h. Any permanent constrictions required on the internal side of the casing must be of such a nature as to avoid interference with the passage of the currently used standard downhole tools or the borehole permeability probe.

i. Peripheral equipment for infiltration tests shall be neatly packaged in a single portable unit. Consideration should be given to developing accessories or attachments to the borehole permeability probe item such that one piece of peripheral equipment will satisfy the needs of both tests.

j. Peripheral equipment for drawdown tests must be developed and neatly packaged in a conveniently portable unit.

D1.4 DELINEATION OF TASKS (See Table D1)

D1.4.1 TASK A: Design of Hardware

This work will include at least the following:

1. Review, describe, evaluate, and synthesize available data on hardware and test boring technology pertinent to the specified devices.
2. Prepare detailed drawings of the device and peripheral equipment.
3. Document the considerations given to existing methods and show analytical rationale, based upon currently existing techniques, for both devices.
4. Outline proposed permeability testing procedures.
5. Detail proposed field program for testing the prototype devices.
6. Submit to the Contracting Officer an interim report of findings and recommendations summarizing results of Task A.

D1.4.2 TASK B: Hardware Fabrication and Field Testing

Subject to review and acceptance of the work completed under Task A, the contractor shall proceed with the fabrication and field testing program. This work will include at least the following:

1. Fabrication of prototype hardware for both specific devices.
2. Select site or sites for field testing of the two systems. The sites should be of known geohydraulic conditions with representative soil types ranging in permeability from 10^{-1} cm/sec to 10^{-5} cm/sec. Priority consideration should be given to areas where large scale pumping tests were performed, areas where documented construction of tunnels or excavations requiring extensive dewatering have been completed, or other areas where test results can be correlated with geohydrologic information.

TABLE D1. PROPOSED BUDGET FOR HARDWARE DEVELOPMENT AND TESTING OF A BOREHOLE PERMEABILITY PROBE AND A PERFORATED CASING PERMEABILITY TEST

ITEM	ALLOTTED TIME	ALLOTTED MAN POWER	COST*
Task A	6 months	6 man months	$24,000.
Task B	9 months	4 man months	$16,000.
Task C	6 months	4 man months	$16,000.
Tasd D	3 months	3 man months	$12,000.
Outside Services and Materials			
Test Boring Rig 40 days @ $375/day			$15,000.
Fabrication of Prototype			$ 6,000.
Laboratory and Computer Costs			$ 6,000.
		Total Cost	$95,000.
Total Allotted Time: 18 months			

*Based on $4000. per man month.

3. Develop and finalize techniques for installation, for testing, and for operation of peripheral equipment.
4. Investigate various schemes of minimizing smear, limiting siltation, and creating the necessary seals.
5. With respect to the perforated casing test, investigate the need for flush joint casing and relative positions of the perforated casing within the casing string.
6. Evaluate the redundancy and quality of test results.
7. Make necessary hardware and/or procedural refinements to optimize test results.
8. Document and summarize results of the hardware development and field testing.

D1.4.3 TASK C: Development of Solutions for Determining Permeability From Test Results

This task shall be performed concurrently with Task B and include at least the following:

1. Develop detailed analytical solutions for computation of soil permeability from the data obtained with the two devices.

2. Accurately determine flow patterns created by the tests for various soil, groundwater, and imposed gradient conditions. Consideration should be given to laboratory model studies, computer and electrical analogy techniques, and possible instrumentation of field tests.

3. Assess limiting boundary and/or soil conditions and document all assumptions.

4. Relate solutions to results of field tests performed under Task B.

5. Present finalized solutions in the form of readily usable charts and tables.

D1.4.4 TASK D: Preparation of a User Manual

Prepare detailed production drawings of the necessary hardware. Prepare a manual specifying standardized installation procedures, testing techniques and data analysis, and guidelines for the appropriate implementation of the tests on tunnel projects, including estimated user costs.

D2.1 CONTRACT OBJECTIVES

The objective of this effort is to develop procedures, requirements, and analytical techniques for large scale pumping tests and full scale dewatering field tests as related to ground water problems in the design and construction phases of rapid transit tunneling in soft ground. This contract will require the following tasks:

a. Assessment of present methods of large scale pumping tests and currently used tunnel dewatering techniques.

b. Development of a modified full scale pumping test to better serve the needs of the tunneling industry.

c. Determination of the need, justification and requirements for performing full scale field testing of the anticipated dewatering systems prior to tunnel construction.

d. Preparation of a manual specifying standardized installation procedures, testing techniques, data analysis and guidelines for the appropriate implementation of the tests on tunnel projects, including estimated user costs.

D2.2 LARGE SCALE PUMPING TEST

Research and development of a modified large scale pumping test shall give consideration to the following minimum criteria:

a. Standardization of equipment and procedures.

b. Better assessment of rates of drawdown and recharge.

c. Better assessment of the cone of influence and effects of overlapping cones of influence.

d. Assessment of permeability at various levels within the well in conjunction with the overall permeability.

e. Assessment of changes in required pumping rate as the aquifer is dewatered.

f. Utilization shall be made of existing hardware and techniques to the fullest extent possible.

g. Other criteria deemed appropriate.

D2.2.1 Suggested Areas for Modification

1. Changes in the number and placement of observation wells.

2. Instrumentation of the well to measure flow at various levels within the well.

3. Placement of a cluster of two or more closely spaced small diameter wells to better assess the influence of overlapping cones of influence.

4. Changes in pumping procedures to better assess drawdown and recharge periods. This might include alternating intervals of pumping with intervals of recharge.

5. Improved non-equilibrium test procedures to limit adverse effects of drawdown.

6. Other modifications deemed appropriate.

D2.3 FULL SCALE FIELD TEST OF DEWATERING SYSTEM

Research and development of a full scale field test of dewatering systems shall give consideration to the following minimum criteria:

a. Tests shall be performed only in very critical areas and/or in areas where dewatering predictions are likely to be unrealistic.

b. Tests shall give information which will enable the accurate prediction of the performance of the anticipated dewatering system or systems.

c. Tests shall be performed only in cases where complete dewatering ahead of the tunnel face is anticipated during construction.

d. The results of the tests shall be presented in a form convenient for use by contractors.

D2.4 DELINEATION OF TASKS (See Table D2)

D2.4.1 TASK A: Review of Current Technology

Review, describe, evaluate, and synthesize current technology on large scale pumping tests and tunnel dewatering as related to the specific criteria of this study. This work will include at least a literature research, documentation of case histories, and consultation with leading engineers and contractors.

D2.4.2 TASK B: Development of a Modified Large Scale Pumping Test

This work will include at least the following:

1. Specify standardized procedures for the tests.
2. Detail the necessary hardware requirements.
3. Present detailed justification for the proposed test including the reasons for specific hardware and procedures.
4. Detail analytical solutions for the proposed test including limiting boundary and/or soil conditions and assumptions. Results should be presented in the form of readily usable charts and tables.
5. Develop user cost data.
6. Prepare a manual specifying standardized installation procedures, testing techniques, and data analysis.
7. Develop guidelines based on subsurface conditions, tunnel design and past experience in geographic area, for appropriate usage of the pumping test on tunnel projects. Prime consideration should be given to user costs, requirements of design engineers, reduced bid price of tunnel construction due to contingencies, and increased safety.

TABLE D2. PROPOSED BUDGET FOR RESEARCH AND DEVELOPMENT OF LARGE SCALE PUMPING TEST AND FULL SCALE DEWATERING FIELD TEST

ITEM	ALLOTTED TIME	ALLOTTED MAN POWER	COST*
Task A	6 months	4 man months	$16,000.
Task B	6 months	4 man months	$16,000.
Task C	6 months	5 man months	$20,000.
Outside Services			
Computer Costs			$ 6,000.
			$58,000.

Total Allotted Time: 12 months

*Based on $4000. per man month.

D2.4.3 TASK C: Development of Guidelines for a Full Scale Field Test of Dewatering Systems

This work will include at least the following:

1. Develop criteria and guidelines for conducting these tests.
2. Detail necessary hardware requirements.
3. Detail solutions for using the results of these tests to analyze the performance of dewatering systems during actual tunnel construction. Results should be presented in the form of readily usable tables and charts.
4. Prepare a questionnaire, solicit responses from leading tunnel dewatering contractors, and analyze results. The questionnaire shall assess the practicability of the proposed full scale field tests in lowering the contractor's bid price, lowering actual construction costs, and enhancing safety.
5. Develop estimated user cost data.

6. Prepare a manual specifying the design, installation, testing techniques, and data analysis of the tests.

7. Develop guidelines based on subsurface conditions, tunnel design, past experience in geographic area, safety, and economy for the appropriate implementation of full scale field tests of dewatering systems on tunnel projects. Prime consideration should be given to user costs, reduced contingencies, and increased safety.

D3.1 CONTRACT OBJECTIVES

The objective of this contract is twofold:

a. Develop improved theoretical methodology for analyzing geohydraulic data to assess the impact on tunnel construction and to predict dewatering requirements.

b. Create a data bank for collection and analysis of information on groundwater related design and construction efforts on completed tunnel projects.

D3.2 CONTRACT REQUIREMENTS

The contract will require the following minimum tasks:

a. Review and evaluate present computer technology relating to groundwater hydrology.

b. Develop computer technology to analyze geohydrologic data, to assess the impact of groundwater on tunnel construction and to evaluate various dewatering schemes.

c. Develop the technology for collecting, analyzing, and presenting actual field performance data relating to the improvement of predictions dealing with groundwater related tunnel design and construction.

d. Develop finalized computer software, procedures for data collection, and specific detailed recommendations for continual updating information, data retrieval, and data presentation.

D3.3 IMPROVED THEORETICAL METHODOLOGY

The objective of this effort is to develop computer techniques to analyze geohydrologic data, to assess the impact of groundwater on tunnel construction and to predict dewatering requirements. The computer techniques should be capable of treating the following conditions and analyzing the following problems:

a. Soil stratigraphy, ground water conditions and soil permeabilities as inferred from the subsurface explorations.

b. Various dewatering methods including well points, deep wells and ejector systems with various spacing, sizing, depth, and pumping rates.

c. Dewatering schemes involving limited regional drawdown through the use of recharge wells.

d. Predictions of rate of drawdown, changes in required pumping rates as water is lowered, and steady state conditions.

e. Prediction of shape and extent of drawdown.

f. Effects of rainfall infiltration, bodies of water, possible "leaky" utilities, and influence of surrounding structures.

g. Rate of recharge after dewatering is terminated.

D3.4 DATA BANK

The objective of this effort is to create a data bank, using computer techniques, for the collection and analysis of information on ground water conditions relating to the design and construction of completed tunnel projects. The intent is to validate and update the use of direct measurement permeability data and the use of sophisticated groundwater analyses. Data collected and analyzed shall include at least the following:

a. Subsurface information obtained prior to and during construction.

b. Details of the design related to groundwater and permeability.

c. Documentation of the construction progress of the tunnel pertinent to groundwater and permeability.

d. Other pertinent data which may have affected the construction such as rainfall records, river levels, tidal records, etc.

e. General comments on the overall dewatering effort.

f. Estimated and actual costs of dewatering including "changed conditions" claims sought by the contractor.

The resulting data will be utilized both by engineers and contractors to optimize on design and construction and to gain acceptance of new technology.

D3.5 DELINEATION OF TASKS

D3.5.1 TASK A: Review of Current Technology

Review, describe, evaluate, and synthesize available information on computer techniques and automatic data processing programs (ADP) related to geohydrologic studies. This work shall include at least the following:

1. Review literature, review available case histories, contact leading universities, and consult with leading experts in the field.
2. Assess the major ADP programs including the language used and the rationale upon which any computations are made.
3. Evaluate the effectiveness and feasibility of using the ADP programs as an aid to tunnel construction in an urban environment.
4. Submit a statement concerning the relevance of existing computer techniques, ADP programs and/or any portion thereof.

D3.5.2 TASK B: Recommendation for Improved Theoretical Methodology

Prepare specific recommendations for the development of computer technology to analyze geohydrologic data, to assess the impact of groundwater on tunnel construction and to assess various dewatering schemes. The findings and recommendations of Task B shall be presented to the Contracting Officer in an interim report.

D3.5.3 TASK C: Recommendation for Data Bank

Prepare specific recommendations for the development of technology for collection, analysis and presentation of actual field performance data relating to the improvement of predictions of groundwater related tunnel design and construction efforts. The findings and recommendations of Task C shall be presented to the Contracting Officer in an interim report.

D3.5.4 TASK D: Development of Software and Implementation Program

Subject to the review and acceptance of Task B and Task C, the recommendations resulting from these tasks shall be developed into the necessary software, guidelines, and specific recommendations to allow immediate implementation by the U.S. Department of Transportation on tunnel projects. This work shall include at least the following:

1. Test and debug all required software.
2. Relate accuracy, reliability and practicability of software to known design and construction conditions on several completed tunnel projects.
3. Develop details of the implementation of the data bank including recommended report forms, data retrieval and presentation procedures.
4. Detail a program for periodically updating the theoretical methodology and direct permeability measuring techniques, as new technology emerges.
5. Develop cost estimates including initial implementation, administrative and updating costs. Justify these costs.
6. Prepare necessary manuals.

TABLE D3. PROPOSED BUDGET FOR RESEARCH AND DEVELOPMENT OF IMPROVED THEORETICAL METHODOLOGY AND DATA BANK FOR GROUNDWATER RELATED DESIGN AND CONSTRUCTION OF SOFT GROUND TUNNELS

ITEM	ALLOTTED TIME	ALLOTTED MAN POWER	COST*
Task A	6 months	5 man months	$20,000.
Task B	9 months	7 man months	$28,000.
Task C	9 months	4 man months	$16,000.
Task D	9 months	10 man months	$40,000.
Outside Services			
Computer Costs			$40,000.
Consultants			$10,000.
			$154,000.

Total Allotted Time: 24 months

*Based on $4000 per man months.

APPENDIX E - TECHNICAL SPECIFICATIONS FOR BOREHOLE LOGGING TOOLS FOR SOFT GROUND LOGGING TO 200 FEET DEPTH

The specifications here presented are technical specifications judged sufficient and adequate for contractors to perform the required hardware development and prototype manufacture and testing work. The specifications include all of those hardware items recommended for development in Section 6. They do not include any general provisions or separate cost items that the Government may desire to include in a final procurement specification. The specifications are basically performance specifications, and the preliminary design effort has concentrated on a definition of the required specific performance. It is recommended that the general provisions provide for review and options for rejection of subcontractor's conceptual and detailed designs prior to prototype manufacture. It is noted that these hardware specifications do not include any developments or modifications of theories of interpretation, or any computer programs for interpretation and display. While all the hardware here included may profitably be developed by one single subcontractor, the software development need not necessarily be performed by the same contractor, though that may be desirable, depending on the capabilities of the subcontractor.

E.1 SYSTEMS REQUIREMENTS

E.1.1 General

Instrumentation for the recommended systems shall be designed to operate in the hostile environments common to off-the-road and urban explorations, and all elements shall be shock and vibration insensitive in terms of operating capability. The surface equipment, in particular, shall be capable of resisting any interfering effects of dust, temperature, and moisture, both in transit to an exploration site and during the explorations.

Borehole instrumentation connectors and cables shall also be designed to withstand any effects common to water, drilling mud or corrosive fluids that might commonly be encountered in soft ground boreholes, and shall be sealed to prevent moisture from entering or causing electrical leakage. All tools shall be designed to operate and provide measurements in either saturated or dry boreholes. Tools that require decentralization shall have a surface-controlled variable decentralizing pressure.

All connectors, cables, slip-rings, etc. shall be adequate to transfer data from the borehole tools and to interface appropriately with the recording systems. Tools, circuits, cables, connectors, recorders, and readouts from all tools shall form an electronically compatible system for the purpose of obtaining, recording, and displaying data obtained from a logging run.

A demonstration of operability of each particular logging method developed is required. Demonstration of calibration procedures and accuracy of calibration is also required. All standards of environmental acceptance (particularly for the nuclear tools) shall be met.

All recordings shall be taken and presented with controlled depth measurements which will ensure that resulting plots show logging response versus depth with an error no greater than $\pm$ 1 inch.

E.1.2 Fabrication

All fabrication shall be of the highest quality according to the standards of the electronics industry.

All equipment shall be fabricated so that component parts are easily accessible to permit rapid maintenance and repair.

E.1.3 Safety

Circuit boards and associated hardware shall be fabricated of flame-retardant material.

The system shall include adequate interlocks and safety devices to ensure the safety of personnel and to prevent damage to equipment.

The system shall be designed so that power failure during operation will not result in damage to the system. Parts of the system utilizing radioactive materials shall be designed and packaged within AEC requirements and industry standards.

E.1.4 Downhole Package

All downhole packages shall be streamlined to minimize friction and prevent damage during downhole operations.

The connectors required at the top of each package shall be interchangeable between packages unless otherwise specified.

A protective cap which mates to the connector shall be furnished for each package. The cap shall include a 1-1/2" I.D. ring or eyebolt at the top which will be used for handling the package. This device shall be capable of supporting the package in any position.

E.1.5 Dimensions

The maximum dimensions of any single or combined downhole package of the system shall be no greater in diameter than 2.25 inches and no longer than 12 feet (excluding cable head). The cable head itself shall have minimum cable/head parting strength of at least 10,000 pounds. Multiple conductor cable is to be used; standard 7-conductor 5/16 inch (or 3-conductor 3/16 inch) is adequate for the system. Cable stretch of more than 1 inch for a logging depth of 200 feet is unacceptable.

E.1.6 Drawworks

Drawworks for the system shall have a minimum drum capacity of 500 feet of standard 5/16 inch cable, as described above, and a minimum drawing power of 10,000 pounds. The range of drawing rates shall be continuously adjustable from 2 to 100 feet/minute, and a fail-safe braking system to take effect if the drive motor fails is required. A separate rate and total footage meter, which measures on the cable is also required.

E.1.7 Miscellaneous

The Contractor shall furnish all equipment, unless specified otherwise, necessary to provide an operable system, including but not limited to: borehole packages, surface equipment, interconnecting cable, internal power supplies, cooling equipment, mating connectors for all external connectors, recorders, and calibration equipment.

All terminals, plugs, connectors, circuit wiring shall be labeled on the equipment and referenced in the fabrication drawings.

E.1.8 Drawings and Manuals

The Contractor shall furnish two complete sets of fabrication drawings and three complete maintenance and operation manuals (including a recommended spare parts list with supply source) for each type-system. These drawings and manuals shall be furnished to the Contracting Officer at the time the system is delivered for acceptance testing.

If needed revisions to drawings or manuals are revealed during acceptance testing, revised drawings (two copies) and/or manual addenda (three copies) in printed form shall be furnished by the Contractor within 30 days at no additional cost to the Government.

Final payment for each type-system shall not be made until all required drawings, manuals (including revisions) and spare parts lists have been received and accepted by the Government.

E.1.9 Test Requirements

In-Plant Tests. The Contractor shall conduct in-plant tests to demonstrate to the Government representative that each system conforms to the design features and operational characteristics specified herein. Four copies of all test data shall be furnished to the Contracting Officer when each system is delivered.

The Contracting Officer shall be notified at least 15 days in advance of in-plant tests, and all in-plant tests shall be witnessed by a DOT representative. Corrective action shall be the Contractor's responsibility. No separate price will be paid for in-plant testing or corrective actions.

E.2 SYSTEM IDENTITY AND INTENDED USE

The systems required and their intended use are as follows:

a. Small Diameter Pulsed Neutron Logging System to obtain a continuous profile of borehole materials response to 14 Mev neutrons in neutron-epithermal neutron and neutron-gamma "lifetime" mode of operation, and to provide semi-continuous data for neutron activation analysis for density, porosity, and soil strata measurements.

b. Small Diameter Electromagnetic Nuclear Response Logging System to obtain a semi-continuous profile of mobile ion response to an intermittently applied electromagnetic field for the purpose of estimating saturated strata permeability and fluid mobility.

c. Small Diameter Focused Induction Logging System to obtain a continuous profile of bulk strata electrical conductivity to identify stratigraphic changes, and contribute to strata identification, fluid content estimates, and for correlation purposes.

d. Small Diameter Microlog Dipmeter/Caliper System to obtain a continuous profile of borehole diameter and borehole sidewall electrical resistivities for the purpose of identifying and measuring changes in borehole and strata dip across borehole, and correlation purposes.

e. Small Diameter Eccentered Neutron-Epithermal Neutron Logging System to obtain continuous profile of epithermal neutron flux in subsurface strata after exposure to a high energy isotopic neutron source for estimation of percent saturation, porosity in saturated strata, and correlation purposes.

f. Small Diameter Compensated Gamma-Gamma Logging System to obtain a continuous profile of natural radioactive gamma emissions and gamma emissions resulting from exposing the strata to an isotopic gamma source for the purpose of estimating stratigraphic constituents, bulk density, and for borehole to borehole correlation.

E.3 DEFINITIONS

System

The term "system" used here is defined as one complete set of components and all integral parts of the design required to perform the borehole logging functions described in section II. Parts of each separate logging system identified, if compatible (such as cables, connectors, recorders, power supplies, etc.), may be used as common parts of a total logging system within the definition.

Downhole Package

The term "downhole package" is defined as the common borehole "probe" or "sonde" part of the system that includes the electronic and physical functions performed in the subsurface while logging.

Signal Conditioning

The term "signal conditioning" is defined here as the electronic treatment of signals received from the downhole package to convert the signals to voltages acceptable to both analog and digital recorders.

E.4 SMALL DIAMETER LOGGING SYSTEMS

E.4.1 Small Diameter Pulsed Neutron Logging System

The pulsed neutron logging system shall consist of a downhole source-sensor instrument, a supporting cable with signal/power transmission conductors, and appropriate power, signal treating and recording equipment appropriate for high quality analog and digital magnetic tape recording. The system shall be constructed to include the following (not necessarily simultaneously).

1. A pulsed neutron generator of the particle-accelerator type emitting 14 Mev neutrons at a rate between 500 and 1000 per second with a generator duty cycle of 1 percent.
2. A gated scintillometer detector, electronics, and timing suitable for neutron collision-capture "lifetime" logging.
3. A gated thermal-epithermal detector suitable for "lifetime" logging.
4. A scintillometer/recording system to record data for neutron activation analysis.

The downhole instrument shall contain a surface controlled decentering device, and the instrument system shall be capable of operating as a continuous logger in all modes but number 4. above.

Surface equipment shall consist of a system to supply power to the downhole instruments, a multi-channel time analysis to record flux at the various time intervals and gates, and circuitry to provide records of integrated flux measurements. Magnetic tape recording capability to retain activation data is required.

E.4.2 Small Diameter Electromagnetic Nuclear Response Logging System

The electromagnetic nuclear response system shall consist of a downhole instrument package, a supporting cable and signal/power conductors, and appropriate surface power, signal conditioning, and recording equipment for high quality analog and digital magnetic tape recording. The signal detecting system shall include an adjustable signal center and bandwidth control with typical center frequency of 2000 Hertz and bandwidth of 50 Hertz. Adequate timing circuits for signal frequency measurements of 1 part in 10^5 are required. Surface equipment shall include a portable total field magnetometer in addition to appropriate signal conditioning, instrument controls, power controls, and recording/display circuitry.

At a free fluid index level of 15%, the system must provide accurate free fluid index values with standard deviation not exceeding ±3% and precision of measurements at all ranges not to exceed ±5%.

E.4.3 Small Diameter Focused Induction Logging System

Equipment and instrumentation for the focused induction system shall include a downhole instrument package, a supporting cable and signal/power conductors, and appropriate surface power, signal conditioning, and recording equipment for high quality analog and digital recordings. The downhole package shall include a set of electromagnetic coils for inducing and detecting the induced signals: minimum main coil spacing is 18 inches (60 cm), and focusing ("bucking") coils for maximum vertical resolution of strata resistance changes are required.

The system shall have response capability ranging from 1 millimho to 1000 millimho/meter with error no greater than 2 millimhos/meter throughout this range.

In addition to appropriate power, signal conditioning, and recording capabilities, the inducing signal generator shall have a minimum variable frequency range between 100 and 10,000 Hertz. The borehole instrument package shall include controllable centralizing skids expandable from borehole diameters of 2.25 inches to 7.0 inches.

E.4.4 Small Diameter Microlog Dipmeter/Caliper Logging System

The small diameter dipmeter/caliper system shall consist of a downhole instrument package, supporting cable and power/signal conductors, and appropriate surface power, signal conditioning, and recording equipment for high quality analog and digital recordings.

The borehole package shall include a minimum of four (4) electrodes with capability to expand to 12 inches borehole diameter (either continuously or with extensions), an inclinometer, and a magnetic compass. At least four (4) independent measurements of resistivity, borehole diameter, borehole inclination, and tool orientation with respect to the earth's magnetic field shall be recorded continuously. Resistivity capabilities shall range from 0 to 1000 ohm-meters^2/meter with a strata dip accuracy of $\pm 1^{\circ}$ from horizontal and inclination of the borehole within $\pm 0.25^{\circ}$ from vertical.

E.4.5 Small Diameter Neutron-Epithermal Neutron Logging System

The neutron-epithermal neutron system shall include a downhole instrument package, supporting cables and control/signal conductors, and surface controls, signal conditioning and recording instrumentation for high quality analog and digital recordings.

The borehole packages shall include an isotopic neutron source of 3 to 6 Mev neutrons and a Cadmium-shielded Helium-3 detector to prevent detection of thermal neutrons. Source-detector spacing of approximately 16 inches (40 centimeters) is required. The borehole package shall operate decentralized during logging with a variable decentralizing force ranging from 0 to 50 pounds.

Accuracy of the system response resolution shall be the equivalent of 0.25 American Petroleum Institute (API) porosity unit; accuracy of measurements within 1.0 API porosity unit (with corrections).

E.4.6 Small Diameter Compensated Gamma-Gamma Logging System

The compensated gamma-gamma system shall include a borehole instrument package, supporting cable and control/signal conductors, and surface controls, signal conditioning, and recording capability for high quality analog and digital recordings. The downhole instrument package shall include an isotopic source of gamma rays and at least two scintillometer-type detectors to detect gamma rays back scattered from the soil strata. The purpose of multiple detectors is to compensate the gamma counts for borehole conditions. The package shall be able to operate both as a natural gamma detector, and as a decentralized compensated logging tool. Decentralization must be surface controlled and variable in pressure from 0 to 50 pounds.

Surface equipment shall include appropriate controls and signal conditioning for logging all detector outputs and for at least one cross plot pair of the detector outputs (after conditioning) or standoff trace.

Counting rates of all detector systems shall be accurate within ±3%, bulk density calculated from the working rates must be accurate within ±4%.

E.4.7 Recording Systems

Analog recording systems for the small diameter logging system shall have the minimum capability of displaying the output from each detector plus one additional crossplot trace as a function of depth beneath the ground surface. The recorder shall have a positive affiliation with the cable footage and rate measurement device to insure that direct response/depth correlations are maintained. All analog recording channels shall have a linear response to inputs (no distortion), and be adjustable over a range to accept and accurately record the output of all signal conditioning units in the logging system. Either dry-write or wet pen recorders are acceptable.

The digital recording system shall have the minimum capability of recording data from each detector output plus one crossplot channel, one depth channel, log type identifiers, and sufficient channels for tape recording rate and slew corrections. Maximum logging rates of 100 feet/minute are anticipated; adequate capability to record the maximum number of detectors with a sampling rate of 100 samples/second shall be provided. Recording format and data identifiers shall be readable by standard digital computer terminals. Dynamic range of the digital system shall be adequate to record data two times greater than indicated by the system response requirements.

E.5 IN-PLANT TEST REQUIREMENTS

The following operational tests and demonstrations shall be performed by the Contractor and witnessed by a Government representative:

a. One 8-hour circuitry test shall be conducted while each small diameter system is operating continuously with the downhole package immersed in water. The test shall demonstrate that variations in atmospheric pressure electrical power, and temperature variations from 0^{o} to 150^{o}F, will not influence operation of the system.

b. The output from each downhole package shall be demonstrated to faithfully reproduce at least two significant changes in the parameter measured within the range of parameters specified. Extremes of each specified range shall be included in this test.

c. The capability to open, close, and vary pressures of the decentralizing skids from a remote position must be demonstrated.

d. The capability of the analog and digital recording systems to produce the same analog recordings must be demonstrated.

e. The capability of the system to record changes in the parameter measured within ± 1 inch of the change (both relative and absolute) must be demonstrated.

APPENDIX F - REPORT OF INVENTIONS

This report contains a comprehensive review of subsurface investigation methodologies for sites of approximately 0 to 200 feet in depth. Although no innovations or discoveries were made, several ideas for innovations were presented and performance specifications and rough illustrations were developed.

The borehole permeability probe and the perforated casing permeability test both described in this report, could, if developed, provide significant improvements in the accuracy of *in situ* permeability testing. The guidelines presented for developing standardized field pumping tests would lead to criteria which would definitely improve the state of permeability predictions based on large and full scale pumping tests. The recommendations for repackaging of existing borehole logging tools suggests a new and innovative use for tools generally used in much deeper holes in harder materials.

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