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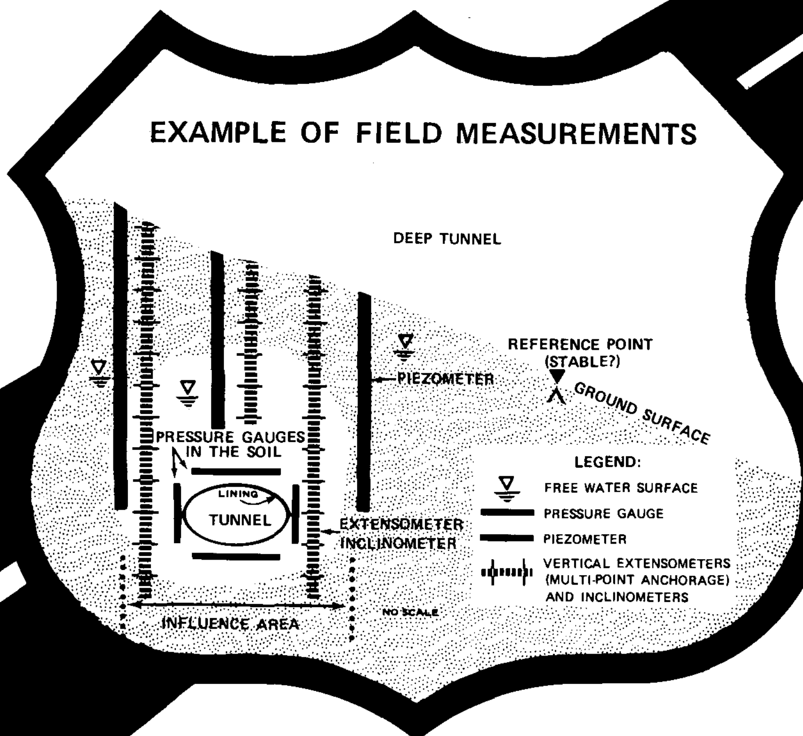
REPRESENTATIVE GROUND PARAMETERS FOR STRUCTURAL ANALYSIS OF TUNNELS

Vol. 4. Case Studies
January 1982
Final Report



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Offices of Research & Development
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FOREWORD

This report contains the results of a research effort to evaluate the influence of ground parameters, based on actual case histories, on the performance of tunnels. Some 160 tunnels, both in rock and soil situations, were studied. The study lists nine elements of importance for the design and construction of tunnels in soft ground and 10 elements for tunnels in rock.

A basic factor of concern for a tunnel either in rock or soft ground is the ground stability which influences the stand-up time of an excavation. Site investigation methods that predict stand-up time of excavation therefore become very important.

This report should serve the needs of geotechnical, structural, and civil engineers who are planning, considering, or designing an underground structure.

Copies of the report are being distributed by FHWA transmittal memorandum. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

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16. Abstract

This report is the fourth in a series reporting the results of the above titled study. This document, Volume 4, is principally concerned with summarization of published case studies on tunnel design and construction with a view toward highlighting lessons learned and benefits received from a rigorous subsurface investigation for tunnel design and construction.

Volume 3 is principally concerned with summarization of currently used analytical design procedures, ascertaining the geotechnical parameters required as input into these design procedures, and assessing the significance of these geotechnical parameters. Volume 2 is principally concerned with in situ site investigation techniques, but also considers the preliminary phases of a site investigation program, classification and correlation systems applicable to underground design and construction, and large-scale field testing procedures. Volume 1 is concerned with the thought processes and considerations related to the planning and implementation of site investigation programs for tunnel design and construction.

It is intended that these documents will provide guidance to those engineers responsible for the planning and implementation of site investigation programs related to tunnel design and construction. This report also will be of value to owners, contractors, and others in the underground construction community.

Volume 1 FHWA/RD-80/012; Volume 2 FHWA/RD-80/013; Volume 3 FHWA/RD-80/014.

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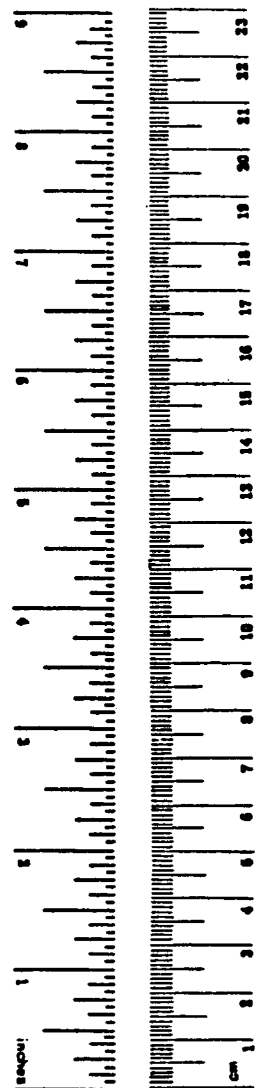
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

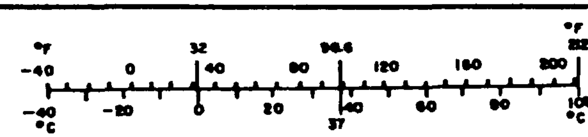
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2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

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



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	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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1.0 INTRODUCTION

1.1 BACKGROUND

Efficient, safe, and economical tunnel design and construction requires a thorough understanding of how the ground will perform during tunneling as well as afterwards. As part of a coordinated research program to reduce underground construction costs by advancing the state-of-the-art of tunneling, the Federal Highway Administration (FHWA) funded this research project entitled "Representative Ground Parameters for Structural Analysis of Tunnels." This study is principally concerned with site investigation and how it influences philosophies and procedures of site characterization as related to tunnel design and construction.

In order to accomplish the objectives of this research effort, the technical report has been divided into four volumes. Volume 1, entitled "Rational Approach to Site Investigation" (Peck, et al, 1980), is principally concerned with the thought processes and considerations related to the planning and implementation of site investigation programs for tunnel design and construction. Topics include: (1) Geotechnical problems peculiar to tunneling, (2) Setting for specific tunneling problems, (3) Approaches to exploration for identifying problems; and (4) Specific procedures for site investigations and their exploration.

Volume 2, entitled "In Situ Testing Techniques" (Hampton, et al, 1980), evaluates in situ site investigation techniques which are applicable to obtaining geotechnical parameters for design and construction of tunnels. In addition, classification and correlation systems applicable to underground design and construction, and large-scale field testing procedures, are discussed.

Volume 3, entitled "Tunnel Design and Construction" (Hampton, et al, 1980), studies the use and significance of geotechnical parameters in the design and construction of tunnels. It discusses tunnel design methods commonly used, the geotechnical parameters required as input to these design methods, and the impact of these required parameters on tunnel design. Also considered are geotechnical parameters which can be obtained a priori, and would be of value in tunnel construction.

Volume 4 (this document) summarizes representative case histories on tunnel design and construction. It attempts to highlight lessons to be learned from the past as well as benefits which have, or may have, accrued from site investigation for tunnel design and construction.

1.2 SCOPE

Due to extensive construction of tunnels throughout the world in recent years, coupled with better documentation of tunneling performance for the same period, progress has been made in advancing the state-of-the-art of tunneling technology. More than 160 representative tunnel cases were studied during preparation of this volume of the report--about 60 of them are discussed individually.

In order to study the influential parameters affecting tunneling performance, the basic tunneling problems in soft ground and rock emanating from published tunnel cases are summarized and analyzed in Chapters 2 and 4, respectively. On this basis, cases of effective construction procedures in both soft ground and rock are selected and studied in Chapters 3 and 5. The importance of site investigation as it relates to tunneling performance also is explored in Chapters 2 through 5. The conclusions and recommendations therefrom are discussed in Chapter 6.

In addition, Appendix A summarizes subsurface site investigations used in connection with certain tunnels which have been constructed. This information is presented without formal evaluation, and for informational purposes only.

2.0 SOME BASIC TUNNELING PROBLEMS IN SOFT GROUND

2.1 INTRODUCTION

In 1969, Dr. Ralph B. Peck presented a state-of-the-art report on tunneling through soft ground. In that report (Peck, 1969), the feasibility of tunneling based on soil types was described; the ground settlement associated with tunneling through various soil types using specific construction procedures was discussed; and, finally, a method of flexible liner design based on previous liner performance was presented. Peck's report summarized the practice of tunnel design and construction in soft ground through 1969.

Since that time, many tunnels have been mined through a variety of soft ground, and additional field performance data have been gathered and analyzed. Therefore, a review of the conclusions and recommendations contained in Peck's report, in the light of recent case histories, is necessary at this time in an attempt to improve tunnel design and construction procedures in soft ground tunneling.

The organization of this chapter basically follows the framework of Peck's report--the newly collected and analyzed field information in the form of case histories is included in each corresponding section.

2.2 STABILITY PROBLEMS AND STAND-UP TIME

Stability of the tunnel face is an important consideration for tunnels in soft ground. It influences the selection of construction techniques, e.g., using a digger shield, dewatering, compressed air, pre-grouting, etc. It is also one of the important factors that determines the uniformity and magnitude of the deformation and loading of the lining. Furthermore, stability is a safety problem for the crew at the tunnel face, and for the structures and traffic on the ground surface.

2.2.1 Stability Analysis of Tunnel Openings

In order to rationalize the behavior of the tunnel face, Deere, et al (1969), categorized soft ground into four types, e.g., coherent frictionless media, coherent frictional media, noncoherent frictional media, and mixed-layered media. Coherent frictionless media consists mainly of plastic clays. Coherent frictional media includes silty or sandy, nonplastic clays, cohesive sands, tills, marls, and loess. Clean sand, clean gravel, and crushed rock are categorized as noncoherent frictional media. In large diameter tunnels, the face may consist of more than one kind of the above-mentioned geologic materials which are called mixed-layered media. The stability of the tunnel face in each of these media will be discussed subsequently.

2.2.1.1 Coherent Frictionless and Coherent Frictional Media

Based on the stability analysis of various slip failure surfaces for a tunnel face, Broms and Bennermark (1967) proposed the use of $p_z/c \leq 6$ as a conservative criterion for predicting the stability of a tunnel face under normal circumstances. Where p_z is the total overburden pressure at the tunnel axis, and c is the average value of the shear strength to a distance from the opening to about one diameter above.

Peck (1969) summarized 10 representative case histories (Table 1) which indicate that tunneling may be carried out without unusual difficulties in plastic clay if the overload factor, $(p_z - p_a)/c$, does not exceed about 5, where p_a is the air pressure above atmospheric. Based on this equation, benefits from the use of compressed air are evident.

From the case histories cited in Table 2, it is observed that the time of exposure is another important factor in reducing the face stability of silty clays. In the Edsådalen Case, failure first occurred after 1.5 hours of exposure. In one segment of the Tyholt Case, failure occurred when face supports were removed after a two-month delay, though the value of the overload factor was less than 5. This failure may have been due to dissipation of negative porewater pressure with time, resulting in a reduction in strength. This phenomenon is particularly important in a clay with silt seams since the strength of silt is highly dependent on the porewater pressure. Thus, the critical value of the overload factor may vary with length of time of exposure. When the material is very brittle or fissured, as in some hard clays or clay-shales, local overstressing also may lead to instability for overload factors less than 5 (Antwerp Case). Additionally, the effect of groundwater may play an important role in cohesive granular media, such as transition materials, loess, marl, etc. Section 2.2.2 presents a discussion on this.

2.2.1.2 Noncoherent Frictional and Mixed Media

Clean sand, clean gravel, and completely crushed rock generally behave as continuous, noncoherent materials. Depending on the amount of binder in the material, the groundwater conditions and rate of advance, silt and clayey or silty sand may behave as coherent or noncoherent materials. Stability analysis by overload factor is not applicable for this type of media.

One of the most important factors for stability of a face in a noncoherent material is the groundwater conditions. Below the groundwater level, stability of the face in an essentially noncoherent material depends on whether the slight cohesion which may be available is able to withstand the seepage forces from the water flowing into the tunnel. For tunnels driven in noncoherent material, a wide variety of soil behavior can be experienced at the tunnel opening, i.e., from firm to flowing ground. In many cases, it is nearly impossible to estimate the available cohesion, and it is necessary to provide a means to aid in the support of the face such as grouting, compressed air, freezing, etc. Dewatering, where feasible, is one of the effective ways to bring the groundwater table down, to eliminate the seepage forces, and to improve the face stability.

For the tunnel face above groundwater table, a material without coherence will not stand unsupported, but will ravel until a stable slope is formed at the face with a slope angle equal to the friction angle of the material in a loose state. If some amount of binder is present, or the sand is sufficiently moist to exhibit an apparent cohesion, a limited height of the tunnel face may be stable. However, when a face with apparent cohesion is exposed for a certain length of time, the strength will deteriorate and the material will ravel or run. Often, either drifting, forepoling, breasting, or a hooded shield, etc., is employed for excavating in this kind of material; but even with these tools, the risk of local instability is still great.

Table 1. Case Histories on Stability of Tunnels in Saturated Plastic Clays (Peck, 1969)

No.	Case	Reference	Soil	Depth z to Tunnel Axis, ft	Tunnel Diameter 2R, ft	Depth / diameter z/2R	Av. Undrained Shear Strength, s_u, ksf	Overburden Pressure, p_z, at Axis, ksf	Air Pressure, p_a, ksf	$\frac{p_z - p_a}{s_u}$
1	London, Ashford	Tattersall, et al, 1955	London clay, fissured, plastic	90	9.3	9.7	21.0	11.0	0	0.5
2	London, Post Office	Ward and Thomas, 1965	do.	55	7.7	7.1	7.2	7.0	0	1.0
3	London, Victoria	Ward and Thomas, 1965	do.	85	14.0	6.1	7.8	10.8	0	1.4
4	Ottawa, Sewer	Eden and Bozozuk, 1968	Leda clay, sensitive	60	10.0	6.0	3.7	6.2	0.6	1.5
5	Antwerp, Gas Storage	deBeer and Buttiens, 1966	Boom clay, fissured, plastic	253	17.7	14.3	7.8	31.5	0	4.1
6	Detroit, Water	Housel, 1942	Plastic glacial clay	68	15.0	4.5	0.8	8.0	3.9	5.1
7	Toronto, Subway	Pers. comm.	Plastic glacial clay	43	17.0	2.5	0.7	5.5	1.4	5.7
8	Chicago, Subway	Terzaghi, 1943	Plastic glacial clay	36	20.0	1.8	0.44	4.3	1.7	5.9
9	Koto, Tokyo, Subway	Shiraishi, pers. comm.	Normally loaded sensitive clay	74	23.0	3.2	0.76	5.6	1.2	7.4
10	Osaka, Municipal Railway	Shiraishi, pers. comm.	Normally loaded sensitive clay	51	23.0	2.2	0.60	5.0	1.0	6.6

Remarks:

1. Stable. Shield driven.

2,3. Face stable. Walls stood for a length of time; occasional problems with overbreak associated with fissures, stratification, etc.

4. Driven with mechanical shield; wall exposed before liner placement. No problems.

5. Hand mined. Fissured clay formed 45^{o} talus slope at face. Wall and roof unstable except for short spans.

6. Hand mined. Concrete placed daily directly against clay. Some squeeze.

7. Stable. Only 4 ft clay cover, dense sand above.

8. Hand mined, horseshoe-shaped; stable with moderate squeeze at air pressure of 12 psi; excessive squeeze on drop of air pressure to 7 psi; ratio $(p_z-p_a)/s_u = 7.4$.

9. Shield-driven, face supported during shove. Difficult to keep shield from diving; deviations from grade as much as 14 in.

10. Shield-driven; face closed except for 2.3% opening; took in up to 80% of theoretical volume of tunnel. Inward squeeze of clay could not be controlled at air pressure of 0.8 ksf. Downward deviations from grade as much as 1 ft.

NOTES:
psi = pounds per square inch
ksf = kilopounds (kips) per square foot
1 ft = 0.3 m
1 ksf = 48 kN/m^2
1 psi = 6.9 kN/m^2

Table 2. Case Histories on Vertical Openings in Frictionless Clay (Broms and Bennermark, 1967; Deere, et al, 1969)

Case	Depth to Tunnel Axis z, ft	Tunnel Diameter B, ft	Average Shear Strength c, ksf	Overburden at Axis p_z, ksf	Air Pressure p_i, ksf	$\frac{p_z - p_i}{c}$	Remarks
Edsådalen	28	6.6	0.305	2.97	0	9.8	Failure after 1 1/2 hour. Hole in sheetwall. Varved, silty clay, sensitive.
Brannkyrka	8.5	3.9	0.210	0.90	0	4.3	Stable. Jacked pipe.
Gothenburg	10-11	4.0	0.510	1.25	0	2.5	Stable. Jacked pipe.
Gothenburg	14-15	4.0	0.330	1.70	0	5.2	Stable. Jacked pipe.
Mårten	14	2.6	0.270	1.50	0	5.6	Stable. Jacked pipe.
Ringon	18-22	4.5	0.420	2.34	0	5.6	Stable. Jacked pipe.
Spanga	11.5-15	4	0.275	1.60	0	5.8	Jacked pipe. 4.25' intake tube. Variable soil strength.
Spanga	10	4	0.185	1.06	0	5.7	
Bromma	30	7.7	0.340	3.18	0	9.4	Jacked pipe. 13-20' clay plug in pipe to prevent inflow.
Tyholt, St. 199	56	26	0.720	7.80	3.30	6.3	Face propped, sectional excavation, clay with silt layers, sensitive.
Tyholt, St. 194+5	72	26	0.610	6.20	2.50	6.1	Squeeze-in.
Chicago	40	25	0.600	4.40	1.87	4.2	Partly closed shield. Opening stable.

NOTES: 1 ft = 0.3 m
1 ksf = 48 kN/m^2

The stability of a tunnel face consisting of mixed media will usually be determined by the properties of each individual layer. For instance, if instability is indicated for a soft clay layer of substantial thickness or for a clean sand stratum, special precautions must be taken.

2.2.2 Catastrophic Ground Loss in Soft Ground Tunnels

Catastrophic ground loss and resulting surface settlement are characterized as singular, large, sudden, and unrestrained ground movements. They frequently cause major damage to surface facilities, and may completely halt tunneling progress. Catastrophic ground loss commonly occurs at the tunnel face. On occasion, it may also happen through the lining some distance behind the face. Frequently, it is too late to stop such movement by the time it is first observed. The only sure way to stop such movements is to prevent them from occurring.

Heuer (1976) identified ground conditions particularly susceptible to catastrophic loss as: (a) cohesionless soil below the groundwater table, (b) weak, highly overstressed clay with an overload factor greater than 5, (c) high vertical stress or high water pressure in cohesive granular soil, (d) excessive disturbance in sensitive soil, and (e) seepage and erosion at interface between different materials. He further presented six case histories to illustrate catastrophic ground loss behavior.

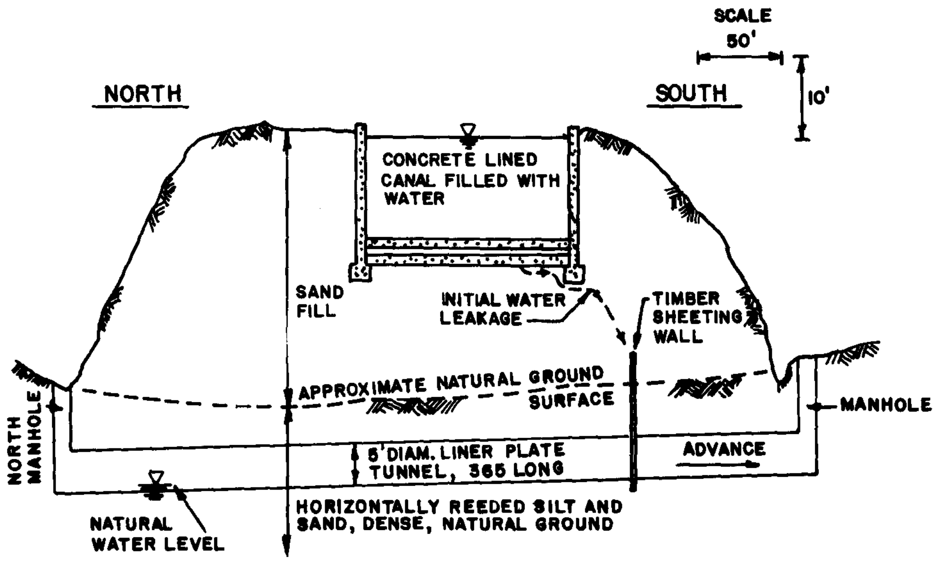
Case 2A - Cohesionless Soil Close to a Water Source

As illustrated in Figure 1, the tunnel was shield-driven in natural silt underlying sand fill which carried a canal structure over a stream valley. Several weeks after completion of the tunnel, water leakage through the crown was observed coming from joints in the liner plate south of the canal at the location of the cut wooden sheeting wall (see Figure 1). This was the first time water leakage from the tunnel crown was observed in the tunnel. Within one hour, the 5-ft tunnel was flowing full of water, welling up out of the 11-ft-deep north manhole shaft. The water flow increased rapidly, washed out the north manhole shaft support, and began eroding the silt and sand fill over the tunnel. This erosion proceeded southward, undercutting and collapsing the sand fill, and eventually undermining the canal floor. About three hours after the first seepage was observed, a 100-ft-long section of canal floor collapsed, sending a 4-ft wall of water rushing down the stream valley and damaging a number of houses downstream.

This disaster was induced by the tunneling activity. Thus, when tunneling under a large water source, the geologic strata, soil types, field permeability, etc., should be investigated very carefully and proper pre-construction soil modifications, or special construction techniques, should be implemented.

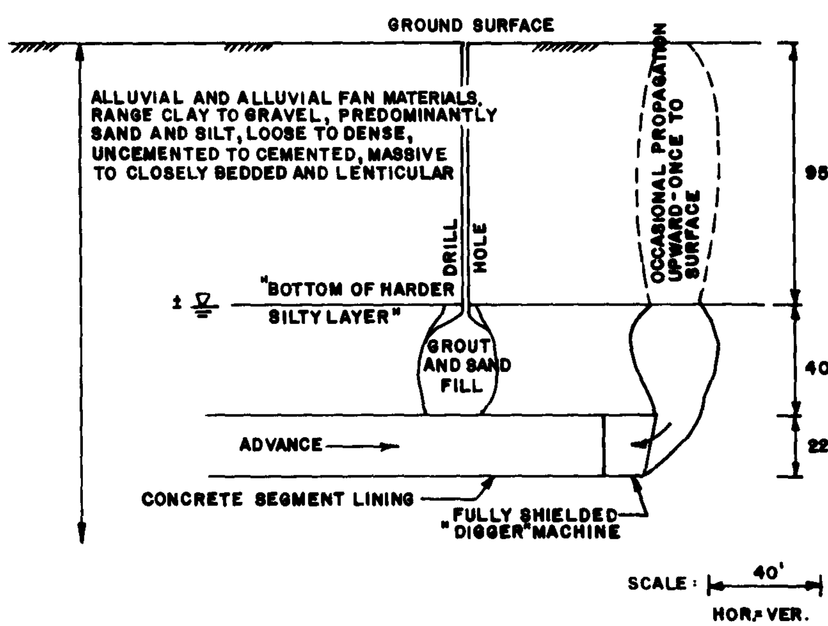
Case 2B - Cohesionless Soil Under Groundwater Table

In one section of a tunnel length bounded by faults which acted as natural dams, the natural water table was about 40 ft above the tunnel crown (Figure 2). Pumping tests prior to construction indicated an average permeability sometimes as high as 10^{-2} to 10^{-3} cm/sec for these materials, with considerable variability depending upon the gradation of various lenses and beds. An attempt was made to advance the tunnel through this zone without prior dewatering, but difficulty was experienced with flowing ground and caving of the face. On several occasions, complete face collapse occurred, with inflows of



NOTE: 1 ft. = 0.3m

Figure 1. Cohesionless Soil Close to a Water Source
(Heuer, 1976)



NOTE: 1 ft. = 0.3m

Figure 2. Cohesionless Soil Underwater Table
(Heuer, 1976)

several thousand gallons of water and soil in a few seconds. The ground would cave in about 40 ft over and ahead of the machine, to about the level of the natural water table, i.e., about 95 ft below the surface. On one occasion, the cavity reached only 40 ft below the surface, and on another occasion the cavity propagated to the ground surface, forming a 10- to 15-ft-diameter hole at the surface. The cavities were filled with sand and grout from the surface through drill holes. The water problem was eventually controlled by drilling several 3- to 4-in.-diameter holes about 200 ft ahead of the tunnel to predrain the ground.

Case 2C - Overstressed Soft Clay and Cohesive Granular Soil Under Water Pressure

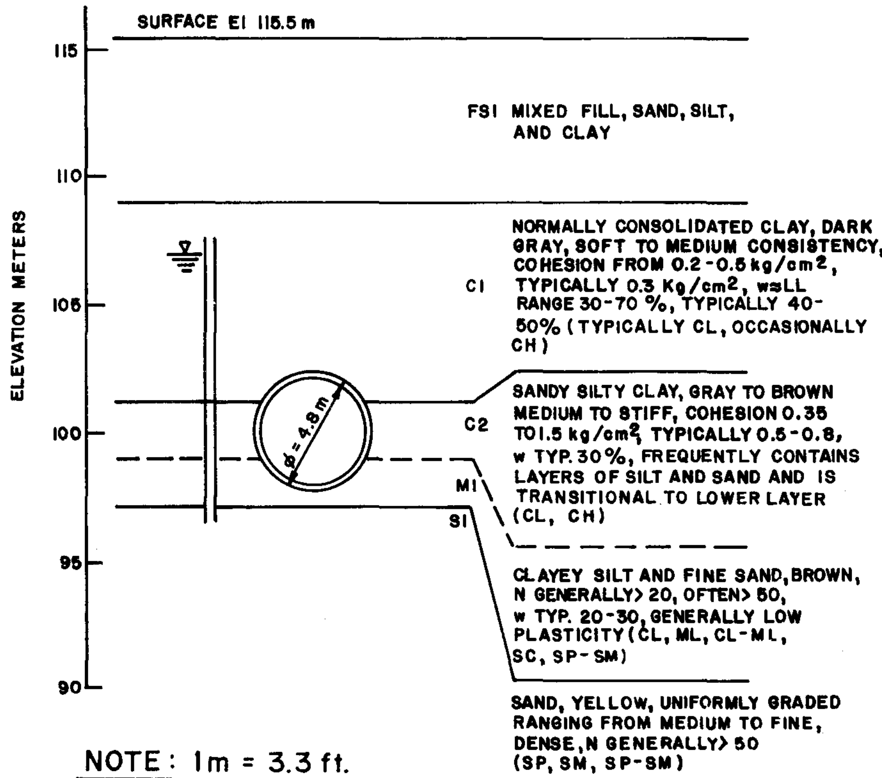
A typical soil profile of a tunnel excavated by a boring machine in free air and supported with ribs and lagging is presented in Figure 3. The overload factor for the Cl clay at the tunnel crown was about 8; the corresponding surface settlement was generally in the range of 25 cm. At one location, the tunnel invert encountered a particularly sandy portion of the Ml layer in communication with groundwater in the underlying Sl sand, at a pressure head about 10 m above the invert. Water began seeping into the tunnel invert and lower walls through the lagging about 20 m (65 ft) behind the boring machine, and began washing silt and fine sand particles into the tunnel. This erosion of soil materials undermined the rib and lagging support allowing the tunnel lining to settle 90 cm (36 in.) and the overlying ground surface to settle 50 cm (20 in.) in several days. The rib and lagging system was severely distorted and on the verge of collapse. For temporary stabilization, the heading was buckheaded and flooded with water to stop the water inflow and soil erosion, and to prevent complete tunnel collapse. The tunnel heading was recovered and excavation successfully completed with minor settlement using compressed air at approximately 1 kg/cm^2 (14.4 psi or 10 m water head), which gave an overload factor in the Cl clay of 4.6 and a balance water pressure in the underlying sand. This case history demonstrates the effectiveness of the compressed air technique, and supports the validity of the overload factor criterion (OF $\leq$ 5; Peck, 1969).

Case 2D - Seepage and Erosion at Interface Causing Disturbance of the Overlying Sensitive Soil

The tunnel was being mined into a mixed-face situation (Figure 4). The till was a few feet thick and overlain by a very sensitive-to-quick, silty clay to clayey silt of medium to stiff consistency. The face was breasted, and a small amount of water was seeping into the face along the till and rock interface. Within 16 hours, a collapse occurred and formed a hole about 55 ft in diameter and 40 ft deep at the surface. A matrix of thoroughly remolded clay containing blocks of undisturbed clay up to one cubic ft in volume flowed into the tunnel for a distance of 300 ft back from the heading. Apparently, water seepage and washing of soil particles at the till and rock contact undermined and loosened the breasting, permitting raveling and caving of till in the face, and resulting in loss of support for the overlying clay. The overload factor for the clay over the crown was in the range of 6 to 7. The overstressed clay began to squeeze into the face. At resulting high strains, the very sensitive-to-quick clay liquefied and flowed into the tunnel such as a viscous fluid.

Case 2E - Overstressed Sensitive Clay

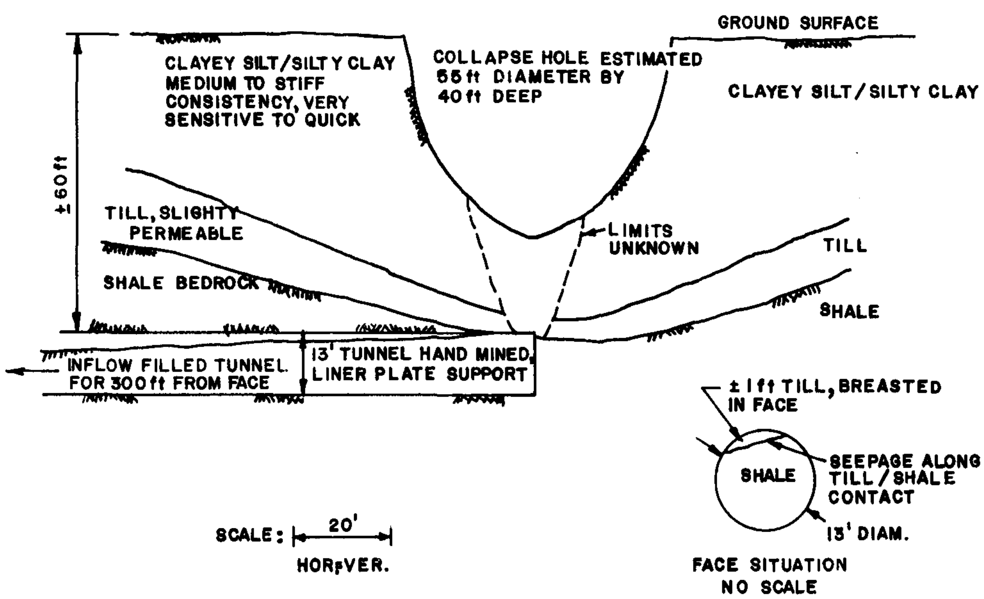
As indicated in Figure 5, the tunnel was being driven with a pressurized slurry face machine in an attempt to stabilize the clay. While at this depth, the free air overload factor for the clay was above 10. Problems were experienced in alignment of bolt holes in



NOTE: 1m = 3.3 ft.

Figure 3. Overstressed Soft Clay and Cohesive Granular Soil under Water Pressure

(Heuer, 1976)



NOTE: 1 ft. = 0.3 m

Figure 4. Seepage and Erosion at Interface Cause Disturbance of the Overlain Sensitive Soil

(Heuer, 1976)

the segmented precast lining, so all bolts were not in place. Excessive deformation and pumping of clay in the invert of the tunnel ahead of the cast-in-place lining were observed just before complete collapse resulted in 3 m (10 ft) of surface subsidence and filled the tunnel with clay. Obviously, the highly overstressed clay around the tunnel was sufficiently disturbed by the tunneling activities to cause it to liquefy, burst through the lining, and flow into the tunnel.

Case 2F - Dewatering Problems

Figure 6A illustrates a case of groundwater in a small depression in sand overlying a stiff fissured clay. Water and sand worked down through clay fissures opened by the tunneling process, and broke into the face causing collapse of the thin clay cover above the tunnel and led to a sinkhole to the ground surface. In Figure 6B(1), the face stability problem is induced by incomplete dewatering of a permeable layer over an impermeable material. A similar problem of perched water retained on an impermeable lens is shown in Figure 6B(2). Numerous variations of this situation can be envisioned.

In summary, based on review of the above six case histories, it was found that most of the disasters could have been prevented if the soil type, soil strata, and groundwater condition had been identified, and if special precautions were taken in advance. Ground behavior in tunneling is not simply an inherent behavior of the ground material within its given physical properties. Rather, ground tunneling behavior also depends on the tunnel depth below the ground surface, the location of the groundwater table, the size of the tunnel cross-section, and the construction procedures used. For example, a medium-to-stiff clay of high sensitivity may give no trouble when tunneled at a shallow depth where not highly overstressed. The same clay, when excavated with the same machine at a greater depth where there is an overload factor greater than 5, may strain and squeeze ahead of the face to the extent that it completely liquefies.

In a cohesive granular soil such as cemented sand, loess, marl, etc., above the water table and at a shallow depth where not highly stressed, these materials are among the most favorable for tunneling. At greater depth where vertical stresses in the ground are more than about one times the ground's unconfined compressive strength, the ground will ravel and require much greater care in excavation to prevent cohesive-running behavior and catastrophic ground loss. Due to size effects, the same transformation from firm to raveling or cohesive running behavior may occur at shallow depth if the tunnel cross-section changes from small to large. Below the natural water table, even at shallow depth in a small tunnel, seepage pressure may exceed the ground strength, causing flowing ground behavior with catastrophic ground loss. Positive groundwater control such as dewatering or compressed air would be required for satisfactory tunneling in such cases.

Thus, in order to prevent the possibility of catastrophic ground loss, care must be taken in 1) identifying soil strata and soil type, 2) evaluating the effects of in-situ stress state, groundwater condition, and tunnel size, and 3) selecting ground modification techniques and construction methods.

2.2.3 Stand-up Time

A fundamental feature of most tunneling methods is first to excavate an opening of some size (this size may be the full face or part of it), then leave this opening standing for a short time until the necessary support is placed. Almost every type of ground will stand unsupported for a short period of time over and in front of an opening of the same size.

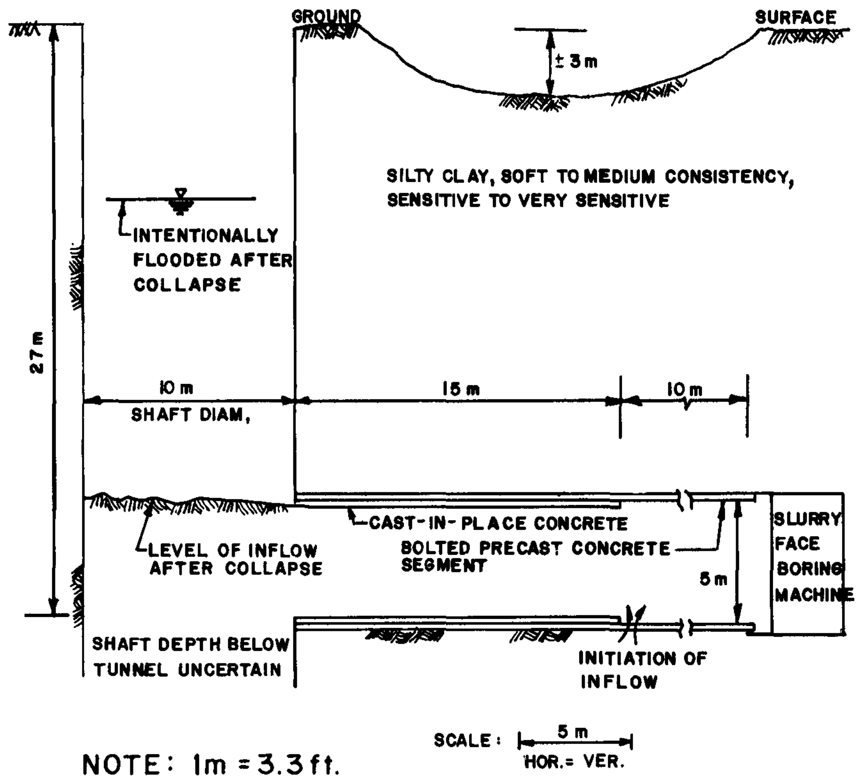


Figure 5. Overstress Sensitive Clay (Heuer, 1976)

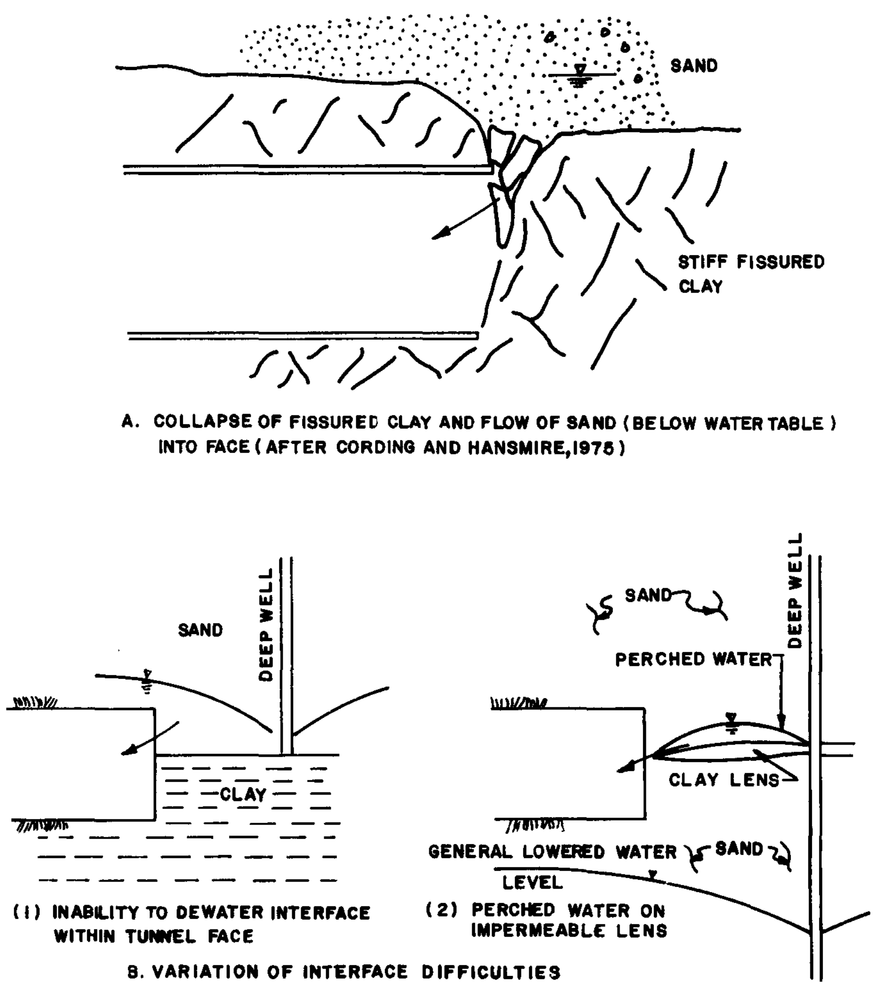


Figure 6. Dewatering Problems (Heuer, 1976)

Successful tunneling requires matching the work methods to the stand-up time of the ground. Tunneling problems, progress, and costs are very sensitive to the stand-up time and stability of the excavated face. Short stand-up time is the distinctive character of soft ground tunneling. Most special methods and equipment developed for soft ground tunneling are directly related to this problem.

Table 3 presents a basic classification of soft ground tunneling behavior based on the stand-up time of the ground. The classification was first proposed by Terzaghi (1950), and further modified and extended by Heuer (1974) to some other soil types and groundwater conditions. On the same basis, Deere, et al (1969), quantified and correlated the stand-up time of the soft ground tunneling behavior and the Unified Soil Classification System with consideration of groundwater conditions. This correlation is presented in Figure 7.

One of the most important factors determining stand-up time is the cohesion of ground related to the stresses in the ground around the tunnel. The circumferential stress which tends to develop at the tunnel wall is approximately twice the in-situ stress, i.e., twice the overburden pressure (Heuer, 1974). When the circumferential stress is the same magnitude as the ground's unconfined compressive strength, the ground may start failure from exposed surfaces and may behave as slow raveling and squeezing ground. If the circumferential stress is much greater (three times or more) than the ground's unconfined compressive strength, the ground fails almost immediately upon exposure, i.e., the stand-up time is short, and the ground will squeeze rapidly, run, or flow. If the circumferential stress is less than the ground strength, the ground may be firm and stand unsupported for a period of time depending on the soil type, groundwater condition, and tunnel size. However, the rate of ground squeezing depends on the overload factor as described in the earlier sections.

Myer, et al (1977), investigated the effect of tunnel size, advance rate, and depth of cover on the stand-up time of tunnels in squeezing ground. The stand-up time is defined as the time elapsed before instability develops, i.e., increasing deformations and deformation rates rather than a catastrophic collapse of the tunnel. Based on a series of 12 physical model tests on a sand-wax material, test results show a 25% increase in stand-up time can be attained by halving the size of the opening or by increasing the advance rate by a factor of four. Decreasing the depth of cover or increasing material strength by 10% also will increase the stand-up time by 25%.

In the same research, some of the most important factors that influence the stand-up time in squeezing ground are listed below:

1. Strength deformation characteristics of the ground including time-dependent strength-deformation characteristics.

2. In-situ stress conditions.

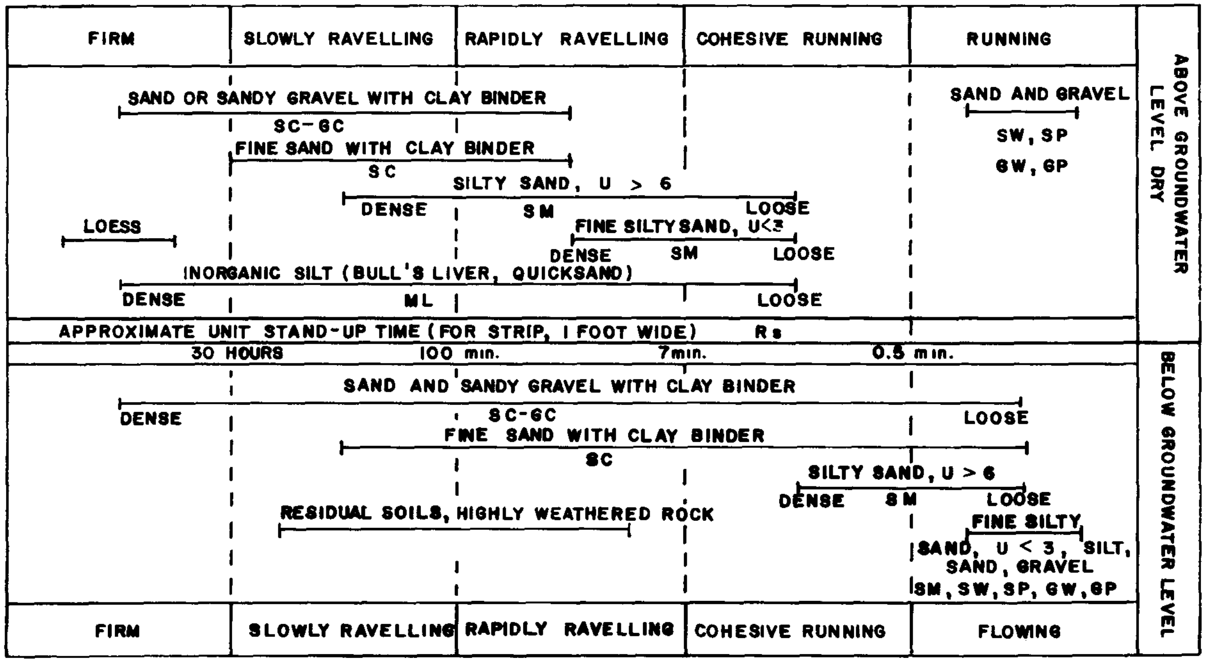
3. Groundwater regime.

4. Size and shape of the opening.

5. Method of excavation.

Table 3. Modified Tunnelman's Ground Classification (Terzaghi, 1950; Heuer, 1974)

Classification		Behavior	Typical Soil Types
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.
Raveling	Slow Raveling ------------ Fast Raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to overstress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling clays depending upon degree of overstress.
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss or continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff-to-hard clay under high cover may move in combination of raveling at execution surface and squeezing at depth behind surface.
Running	Cohesive Running ------------ Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose ($\pm 30^\circ - 35^\circ$). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running.
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.



NOTES:

1. AIR LOSS (IN TUNNELLING UNDER COMPRESSED AIR) AND WATER INFLOW IS GOVERNED BY THE PERMEABILITY, LARGELY FUNCTION OF D_{10}.
2. BEHAVIOR BELOW GROUNDWATER TABLE UNDER SUITABLE AIR PRESSURE IS APPROXIMATELY THE SAME AS ABOVE GROUNDWATER LEVEL.
3. LOOSE IS HERE DEFINED BY $N < 10$ (STANDARD PENETRATION TEST), DENSE BY $N > 30$.
4. DESCRIPTIVE TERMS OF MATERIALS ACCORDING TO THE UNIFIED SOIL CLASSIFICATION.
5. BEHAVIOR MAY BE SOMEWHAT BETTER THAN SHOWN ABOVE GROUNDWATER LEVEL, IF MATERIAL IS MOIST AND FINE OR SILTY.

Figure 7. Behavioristic Classification of Various Soils (Deere, et al, 1969)

6. Rate of advance.

7. Method of support and/or reinforcement and lining.

Furthermore, based on case history review, two recurring solutions to the stand-up time problem become evident: (a) Adjustment of excavation procedure, for instance, increasing excavation rate and compressed air pressure, etc.; (b) reduction of the size of the excavation. Some of the cited case histories related to squeezing soft ground are outlined briefly below.

Case 2G - Tyholt Railroad Tunnel, Norway--Increase Air Pressure to Extend the Stand-up Time

This case is the same project as listed in Table 2, but for different sections. The tunnel was excavated at a depth of 65 ft in clay with zones of quick, low strength clay (Myer, et al, 1977; Hartmark, 1964). The strength of the clay varied from 0.43 ksf to 2.04 ksf. A full-face shield was used with provision for use of a bulkhead in front. Compressed air was utilized also where the clay strength was low. In sections of the tunnel where the clay was of higher strength, and overload factors were less than 6, no stand-up time problems were experienced. However, at one point in a zone of low strength material, the air pressure dropped from 13.1 psi to 4.7 psi, and the face moved about 1 ft into the tunnel. A slide into the tunnel occurred where the tunnel entered into the weakest strength clay. An increased air pressure of 26.8 psi was necessary to maintain the face stability.

Case 2H - Wilson Tunnel, Hawaii--Using Multiple Drift to Increase the Stand-up Time

The Wilson Tunnel is 33 ft in diameter and was built under 50 ft to 100 ft of cover (Myer, et al, 1977; Peck, 1981a). It was hand-mined full face with an electric power shovel. The ground was a residual silty clay derived from lava flows and had brittle stress-strain characteristics. At points of excessive overbreak and poor support, raveling became excessive; eventually, the ground became almost fluid filling the tunnel and causing a set of three sinkholes at the surface. After this incident, the face was attacked by the multiple drift method after allowing time for drainage, and the stand-up time was increased so that overbreak and raveling problems were eliminated.

Case 2I - Antwerp Gas Storage Galleries, Belgium--Using Successive Pilot Bores to Eliminate the Short Stand-up Time Problem

This case is the same project cited in Table 1, Number 5. As mentioned in Section 2.2.1.1, a 45° talus slope formed at the face and at one point left 18 ft of unsupported crown in danger of collapse in the full-face gallery excavation (Myer, et al, 1977; deBeer and Butteins, 1966). The excavation method was then changed to precede the main face by a pilot bore of 5 ft to 10 ft in length. Only enough material was excavated at one time in the pilot bore to allow erection of a set of steel ribs, leaving a core of material in the middle of the bore. The pilot bore was then enlarged set by set to full diameter. Another cycle of the same was then followed. It was noted that the clay not only deformed into the tunnel at the face, but also at the crown. It deformed toward the face and into the tunnel, tending to pull in the top of the support.

Case 2J - Schewaikheim Railway Tunnel, Germany--Utilization of Multiple Drifts in Very Short Stand-up Time Ground

This tunnel is 30 ft in diameter with about 1000 ft in length under 65 ft of overburden (Myer, et al, 1977; Rabcewicz, 1969). The tunnel was mixed face; the lower part was in limestone with clay lenses; the upper part was in weak clay. The multiple drift method was utilized for excavation because of the short stand-up time of the ground. Instrumentation placed ahead of the face detected movement of the clay three diameters ahead of the face.

In summary, based on the discussion in this section, stand-up time is one of the important influential factors in soft ground tunneling. The parameters affecting the stand-up time and its quantitative determination are also described. A few case histories involving a possible solution for the problem of short stand-up time in squeezing ground tunnels are cited also. A few recently developed ground modification techniques and construction methods for improving stand-up time characteristics in a soft ground tunnel will be discussed in Chapter 3.

2.3 SETTLEMENT PROBLEMS AND DAMAGE TO SURROUNDINGS

The construction of every soft ground tunnel is associated with a change in the state of stress in the ground and with corresponding strain and displacements. Damage due to soil movements around soft ground tunnels is one of the most critical problems in tunneling in urban areas. Many of the design and construction decisions on a soft ground tunnel project must be directed toward preventing excessive damage to structures or utilities near the tunnel.

2.3.1 Settlement Around Tunnels

Ground movements can be separated into two categories. In the first category is the sudden, large loss of ground that may occur locally due to raveling, flowing, or running of ground that progresses above the tunnel crown, forming a slump or deep settlement trough at the ground surface. Such losses cannot be accurately predicted as to their location along the tunnel alignment or their magnitude. The possibility of the occurrence of the catastrophic loss can be considered by comparing proposed construction methods with the anticipated range of ground conditions, as discussed in Section 2.2.2.

In the second category is the ground movement that can be expected under normal conditions where large, localized losses do not occur. In this category, the magnitude of the movement is more amenable to quantitative estimate. The nature of the settlements and those associated with workmanship are largely dependent upon the type of ground, groundwater conditions, and geometry and depth of the tunnel.

Peck (1969) discussed the loss of ground and settlement on the basis of four principal groupings of soils. They are: (a) Granular soils with no cohesion, but maybe with capillarity; (b) cohesive granular soils; (c) nonswelling stiff to hard clays; and (d) stiff to soft saturated clays. Since the tunneling behavior of these soil groups has been discussed in Volume 3: "Design and Construction of Tunnels" (Hampton, et al, 1980) of this series of reports, only the prediction of settlement associated with the tunneling will be discussed in the following paragraphs.

In cohesionless granular materials, if the dewatering is completely done, and if there are no impervious lenses for trapping perched groundwater, the loss of ground in a dense material can be exceptionally small. On the other hand, the settlement may increase considerably as a result of erosion or migration due to seepage into the heading at localized zones, and the settlements may reach catastrophic proportions if runs develop on account of insufficient groundwater control or inadequate precautions against raveling. The likelihood of loss of ground is greatly increased if the sand is loose or contains loose zones in which positive porewater pressures may be developed. Further, if the compressed air is used for groundwater control, and if the permeability of the soil permits air to escape from the face, or particularly from the crown of the tunnel, the escaping air may dry the soils completely, whereupon the soil may become truly cohesionless and runny.

Cohesive granular materials include a number of types, ranging from clayey sands to cohesive silts. Residual soils possessing a cohesive bond including many saprolites, loess, and certain calcareous clays with a stable cluster including marls, often fall into this category. All these materials have several characteristics in common. They exhibit nearly linear stress-strain curves until the bond strength is approached, the initial tangent modulus of unconfined specimens is relatively high, and failure often occurs on a pre-existing surface of weakness. If these materials are excavated with proper support, the accompanied loss of ground or settlement can be very modest or negligible. On the other hand, if raveling or piping is allowed to develop, the consequences may be catastrophic. Most materials in this category are sensitive to adverse seepage pressures. Hence, positive control of groundwater must be provided. The settlements due to raveling in these kinds of materials may be delayed for many years; therefore, adequate back packing must be done during tunneling and a perishable material should not be used.

In general, nonswelling, stiff-to-hard clays have the more desirable properties than the other groups unless they possess a well-developed secondary structure, e.g., fissures. These clays are unlikely to ravel or to be adversely influenced by seepage toward the opening. The loss of ground is a function of strength of the clay, and the size and depth of the tunnel. If other factors are equal, the settlement directly above the tunnel is approximately proportional to the tunnel diameter. The small settlements associated with good construction techniques in these materials can be anticipated if the overload factor is less than 4.

Soft-to-stiff saturated clays are characterized by values of undrained shear strength ranging from 0.2 to 2.0 ksf at depths of cover up to as much as 100 ft. For practical purposes, these materials may be regarded as impervious and their sensitivity may range from low to very high. Settlements above and adjacent to tunnels in these materials may be dramatically larger than those above tunnels in stiffer and more brittle cohesive granular soils. However, the settlements and movements in the tunnel are not likely to develop with catastrophic speed such that the heading might be buried. The compressed air technique is an effective means of reducing loss of ground because it reduces the changes in stress due to excavation. The tailpiece clearance between the excavated tunnel and shield is one of the main contributors to excessive lost ground. Development of a means to eliminate or reduce the movements associated with this clearance is a promising field for investigation. Furthermore, the long-term settlement due to consolidation of the clay around the tunnel also should be considered. These settlements may spread much more widely than those due to tunneling operations.

Based on available data (over twenty case histories), Peck (1969) demonstrated that a cross-section through the settlement trough over a single tunnel usually can be represented by the error function, or normal probability curve. Such expedients are needed for judging the necessity of underpinning or shoring the adjacent structures, or of relocating important utilities. The pertinent properties of the error function and its relationship to the dimensions of the tunnel are shown in Figure 8. The maximum ordinate of the curve is the maximum settlement, δ_{max}, which is empirically estimated, based on tunnel construction methods and workmanship. Values of i were correlated from 17 tunnels above which reasonable, reliable, and sufficient settlement data are available. These data are assembled in Table 4. A dimensionless plot of the pertinent information for the 17 tunnel cases is presented in Figure 9. The numbers in this figure are the identification of each individual case. From this figure, a tentative separation of the results according to the soil type can be observed. Appreciably greater values of i/R appear to be associated with tunnels in plastic clay than in the several varieties of granular materials. A significant exception is tunneling in sand below groundwater table, where control of lost ground is especially difficult. As expected, the greater the depth of the tunnel, the greater the spread of the settlement trough.

Along the same line of investigation, Cording, et al (1976), and MacPherson, et al (1978), summarize 19 recent tunnel cases (Table 5). They first divided the settlements and volume losses into four stages: Those developed (a) ahead of the face; (b) over the shield (if a shield is used); (c) during erection of the lining (at the tail of the shield); and (d) with time and further advance of the heading, as the lining deflects. Based on the measured subsurface settlement data (Table 6), they related the ground movement to observed construction and soil conditions. Through careful field instrumentation programs and detailed analysis of available and related data, they reached the following conclusions:

1. The volume of ground lost into a tunnel can be estimated using deep settlement points located above the tunnel crown. The most unpredictable conditions develop in the tunnel face, particularly when groundwater is present.

2. Large losses of ground can occur over the shield due to excessive pitch or yaw, in addition to the volumes lost by overcutters on the shield.

3. The loss at the tail can be reduced by filling the voids with grout or by expanding the lining before the ground collapses into the void. Immediate expansion or grouting is required in granular soils to prevent ground loss.

4. Downward deflection of the tunnel crown can also contribute to ground loss. Lateral deflection of the lining and lateral compression of the soil and voids outside the springline are the cause of most of this loss.

Based on the same tunnel cases, Cording, et al (1976), and MacPherson, et al (1978), further studied the relationship between volume of the settlement trough, volume of ground lost into the tunnel, and the shape of the settlement trough (Table 7). They deduced the following findings:

1. Vertical compression of the soil outside the springline due to stress increase around the tunnel will contribute to the volume of the surface settlement trough. The compression at this location cannot be measured with a settlement point above the crown but can be measured with a settlement point located outside the tunnel springline at an elevation at or below the crown elevation.

2. Long-term settlements in soft clay may be due largely to consolidation of the soils outside the tunnel springline. The disturbed zone immediately around the shield will tend to undergo the largest consolidation. For clays, the volume of

Table 4. Maximum Settlement and Typical Width of Settlement Trough Above Tunnels (Peck, 1969)

No.	Case	Reference	Radius of Tunnel, R (or R'), ft	Depth of Tunnel Axis, z, ft	z/2R (or z/2R')	i, ft	i/R (or i/R')	δ_{max}, ft	Settlement Volume, ft^3/ft	Settlement Volume, %	Remark
1	Toronto Subway	Pers. files	8.75	34-44	2.0-2.5	6.4	0.73	0.28	4.5	1.9	First tunnel
			8.75	34-44	2.0-2.5	8.0	0.92	0.46	9.2	3.8	Second tunnel
	Dense sand above groundwater level										
2	do.	Matich & Carling (unpubl.)	8.75	35	2.0	24	2.75	0.04	2.4	1.0	First tunnel
3			20	35	0.88	25	1.25	0.1	6.3	1.3	Total settlements
	Below groundwater level, crown in sand, invert in till										
4	San Francisco (BART)	Pers. files	8.75	36	2.1	18	1.0	0.03	1.35	0.56	Settlements from first and second tunnels independent and equal
	Cemented dense sand, above groundwater level										
5	G.N.R.R. Seattle	Hussey, et al, 1915	19.5	125	3.2	20	1.0	0.7	35	2.6	
	Hard clayey glacial till (horseshoe)										
6	Toronto Subway	Matich & Carling (Unpubl.)	8.75	43	2.4	20	2.3	0.03	1.5	0.62	First tunnel
7			20	43	1.1	20	1.0	0.12	6	1.25	Total settlements
	Medium glacial clay										
8	Ottawa Sewer	Eden & Bozozuk, 1968	5	60	6	22	4.4	0.023	1.25	1.6	
	Medium Leda clay										
9	Chicago Subway S-6	Terzaghi, 1943	10	39	2.0	16	1.6	0.075	3	0.75	First tunnel
	Soft glacial clay										
10	Chicago Subway S-3	Terzaghi, 1942	52	40	0.77	22	.85	—	—	—	Total settlement over two tunnels
	Soft glacial clay										
11	San Paolo	Terzaghi, 1950	4.5	100	11	19	4.2	0.67	32	50	Many construction difficulties
	Stiff clay										
12	San Francisco (BART)	Pers. files	9	59	3.3	24	2.7	0.18	11	4.3	
	Medium clay										
13	Sulphur Extraction	Deere, 1961	175	1400	4.0	320	1.8	—	—	—	
	Rock										
14	Mine	Wardell, 1959	770	2620	1.7	510	0.66	—	—	—	
	Rock										
15	Mine	Wardell, 1959	61	370	3.1	39	0.65	—	—	—	
	Rock										
16	Mine	Pierson, 1965	620	1000	0.8	420	0.68	—	—	—	
	Rock										
17	Mine	Berry & Sales, 1961	275	1970	3.6	475	1.7	—	—	—	
	Rock										

NOTES:

R' is one-half the width of a horseshoe tunnel, or R + d/2, where d is the spacing of twin tunnels, center to center.

Volume of settlement trough is calculated by $V_s = 2.5 \cdot \delta_{max}\ i$.

1 ft = 0.3 m = 300 mm

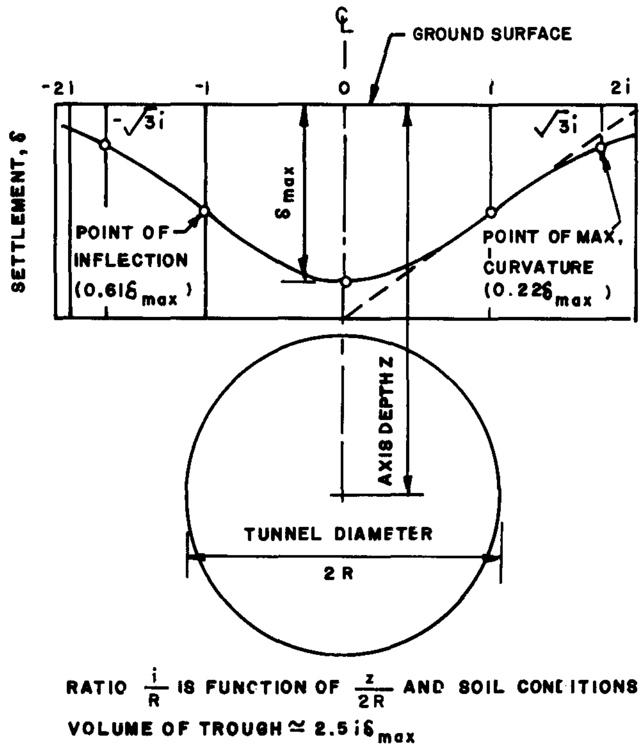


Figure 8. Properties of Error Function or Normal Probability Curves as used to Represent Settlement Trough above Tunnel (Peck, 1969)

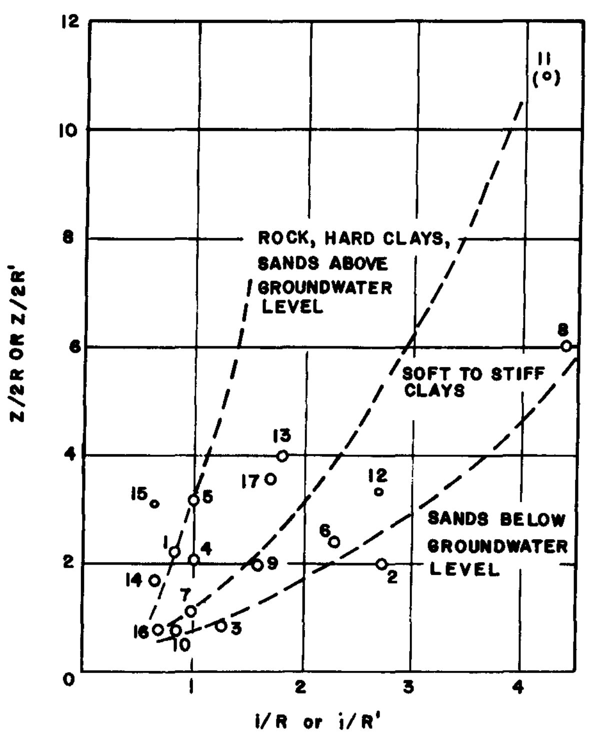


Figure 9. Relation Between Dimensionless Width of Settlement Trough and Dimensionless Depth of Tunnel for Various Tunnels in Different Materials (Peck, 1969)

Table 5. Soil and Construction Conditions (Cording, et al, 1976; MacPherson, et al, 1978)

Case	Soil Type	Construction Method and Initial Lining
1. Washington, D.C. Metro, Section A-2, C line, B line, A line. (Hansmire, 1975)	Medium-dense silty sand and gravel, interbedded with sandy, silty clays (S_u = 75 kN/m^2, $\gamma z/S_u$ = 4).	Shield, bucket digger. Steel ribs and timber lagging, expanded during and after shove. Partial dewatering with wells spaced 60 m on center.
2. Washington, D.C., Treasury Yard (Hansmire, 1975)	Medium-dense silty sand and gravel, interbedded with sandy, silty clays (S_u = 75 kN/m^2, $\gamma z/S_u$ = 3).	
3a. Washington, D.C. Metro, Section F2a, L'Enfant - Pentagon (L) Route, twin, single-track tunnels, 2R = 18 ft (5.4 m), z/2R = 3.7 to 4.1	Dense sand and gravel, very dense clayey sand, overlain by silty sand and gravel interbedded with sandy, silty clays.	Articulated shield with digger arm. Steel segments erected within tailskin and grouted prior to shove. Partial dewatering with deep wells spaced 160 ft (50 m) on center.
3b. Washington, D.C. Metro, Section F2a, Branch (F) Route, twin, single-track tunnels, 2R = 18 ft (5.4 m), z/2R = 2.0	Medium-dense silty sand and gravel interbedded with sandy, silty clays.	Identical to L Route (Case 3a).
4. Frankfurt Shield, Fahrgasse*	Sand, some limestone and clay marl lenses.	Shield, bolted concrete segments.
5. Frankfurt Shield, Domplatz*	Frankfurt clay marl, some limestone and sand lenses, S_u = 130-550 kN/m^2, $\gamma z/S_u$ = 0.6 - 2.5.	Shield, bolted concrete segments.
6. Frankfurt Shield, Dominikanergasse*	Sand, some limestone and clay marl lenses.	Shield, bolted concrete segments.
7. Frankfurt, no shield, Baulos 17*	Frankfurt clay marl, some limestone and sand lenses, S_u = 130 - 550 kN/m^2, $\gamma z/S_u$ = 0.6 - 2.5.	No shield; heading and bench. Shotcrete and light steel ribs, soil anchors.
8. Frankfurt, no shield, Baulos 18a, Tunnel 13*	Frankfurt clay marl, some limestone and sand lenses, S_u = 130 - 550 kN/m^2, $\gamma z/S_u$ = 0.6 - 2.5.	No shield; heading and bench. Shotcrete and light steel ribs, soil anchors.
9. Tyneside (Attewell, et al, 1975)	Tyne laminated clay, S_u = 73 kN/m^2, $\gamma z/S_u$ = 2.05.	Shield, with tailskin, 5 segment concrete lining, cement grouted after every third ring.
10. London Transport (Attewell and Farmer, 1974)	London clay, S_u = 270 kN/m^2, $\gamma z/S_u$ = 2.2.	Shield, 7 segment cast iron lining erected in tail, cement grouted after every shove.

Table 5. Soil and Construction Conditions (Cording, et al, 1976; MacPherson, et al, 1978)
(Continued)

Case	Soil Type	Construction Method and Initial Lining
11. Heathrow Cargo Tunnel (Muirwood and Gibb, 1971; Smyth-Osbourne, 1971)	Upper portion of London clay, 3.6 m clay cover under wet gravel S_u = 72 to 275 kN/m^2, $\gamma z/S_u$ = 1 to 4.	Shield, hand mined. No tail, expanded concrete segments behind shield.
12. Boa Vista (Costa, et al, 1974)	Sand and clay lenses.	Shield, compressed air.
13. Brussels Metro (Vinnel and Herman, 1969)	Uniform cohensionless sand in upper half of tunnel, clayey sand in lower half.	Shield, hand mined, liner segments installed in tail.
14. Mexico City, Siphon II Manuel Gonzales (Tinajero and Vieitez, 1971)	Plastic lake clay, S_u = 40 kN/m^2, $\gamma z/S_u$ = 5.	Shield, oscillating cutters. Steel lining grouted 8 m behind lining. Cutters support 1/3 of face. Dewatering prior to tunneling.
15. Lower Market St. BART, San Francisco, Kuesel (1972)	Soft, plastic clay; S_u = 75 kN/m^2.	Shield, rotating cutter wheel. Compressed air, segmented liners, grouted.
16. Frankfurt, no shield.* Baulos 25	Frankfurt clay marl, some limestone and sand lenses, S_u = 130 - 550 kN/m^2, $\gamma z/S_u$ = 0.6 - 2.5.	No shield; heading and bench. Shotcrete and light steel ribs, soil anchors.
17. Washington, D.C. Metro, Section D-9, 2nd tunnel	Hard, fractured and slickensided Cretaceous clay, c = 300 kN/m^2; $\gamma z/S_u$ = 1.5 overlain by Pleistocene sand and gravel above tunnel crown.	Shield, backhoe digger, steel ribs and timber lagging expanded after shove.
18. Rockford, Ill., ESLLIRP, Contract 1A, sewer tunnel, 2R = 9.3 ft (2.8 m), z/2R ≈ 3.4 to 4.5	Medium-dense sands with some gravel.	Rotating-wheel tunnel mole. Steel ribs with timber lagging expanded after shove. Dewatered with deep wells spaced 200 ft (60 m) average on center.
19. Washington, D.C. Metro, Section G1, second tunnel, twin, single-track tunnels, 2R = 21 ft (6.4 m), z/2R = 2.1	Transition from sandy, silty clays and medium-dense, silty sand and gravel to hard, fissured clay.	Articulated shield with digger arm (same as Case 3). Steel ribs and timber lagging expanded after shove. Dewatered with deep wells on irregular spacing.

NOTES: * (Chambosse, 1972; Sauer and Lama, 1973; Breth and Chambosse, 1972)

1 m = 3.3 ft

1 kN/m^2 = 0.15 psi

Table 6. Lost Ground Around Single Tunnels (Cording, et al, 1976; MacPherson, et al, 1978)

Case	Diameter 2R, m	Depth to Axis z, m	$\frac{z}{2R}$	Lost Ground: Before Face	Lost Ground: Over Shield	Lost Ground: At Tail	Lost Ground: Lining Deflection and Time Dependent Movements	Total	Comments
1a. Washington, D.C. Metro, Section A-2, 1st tunnel, C line. (Hansmire, 1975) (Hansmire and Cording, 1972)	6.4	14.6	2.3	δ =12 mm V_L=0.06 $\frac{m^3}{m}$ $\%V_L$=0.2%	250 mm 1.82 $\frac{m^3}{m}$ 5.6%	41 mm 0.29 $\frac{m^3}{m}$ 0.9%	42 mm 0.30 $\frac{m^3}{m}$ 0.9%	345 mm 2.46 $\frac{m^3}{m}$ 7.6%	Settlement point 0.45 m above crown. Expanded lining.
1b. Washington, D.C. Metro, Section A-2, 2nd tunnel, C line (Hansmire, 1975)	6.4	14.6	2.3	δ =14 mm V_L =0.06 $\frac{m^3}{m}$ $\%V_L$ =0.2%	58 mm 0.43 $\frac{m^3}{m}$ 1.4%	24 mm 0.18 $\frac{m^3}{m}$ 0.6%	28 mm 0.21 $\frac{m^3}{m}$ 0.7%	124 mm 0.92 $\frac{m^3}{m}$ 2.9%	Settlement point 0.45 m above crown.
3a. Washington, D.C. Metro, Station F2a, L Route, First and Second Tunnels at cross-sections	5.5	20-22	3.7-4.1	δ =3-8 mm V_L =0.02-0.07 $\frac{m^3}{m}$ $\%V_L$ =0.1-0.2%	5-30 mm 0.05-0.05 $\frac{m^3}{m}$ 0.2-1.2%	3-5 mm 0.02-0.05 $\frac{m^3}{m}$ 0.1-1.2%	3-10 mm 0.02-0.10 $\frac{m^3}{m}$ 0.1-0.4%	18-53 mm 0.17-0.47 $\frac{m^3}{m}$ 0.6-2.0%	Grouted lining. Construction procedure nearly identical to F Route. Tunnels 20 to 23 in. apart cc.
L Route--range of values along both tunnels		16-23	3.0-4.1	--------	--------	--------	--------	18-104 mm 0.17-0.38 $\frac{m^3}{m}$ 0.6-3.4%	
3b. Washington, D.C. Metro, Station F2a; F Route, first tunnel	5.5	11.0	2.0	δ =5 mm V_L =0.04 $\frac{m^3}{m}$ $\%V_L$ =0.1%	51 mm 0.40 $\frac{m^3}{m}$ 1.7%	18 mm 0.14 $\frac{m^3}{m}$ 4.6%	10 mm 0.07 $\frac{m^3}{m}$ 0.3%	81 mm 0.65 $\frac{m^3}{m}$ 2.7 %	Settlement point 1.2 m above tunnel crown. Grouted lining.

Table 6. Lost Ground Around Single Tunnels (Cording, et al, 1976; MacPherson, et al, 1978) (Continued)

Case	Diameter 2R, m	Depth to Axis z, m	$\frac{z}{2R}$	Lost Ground: Before Face	Lost Ground: Over Shield	Lost Ground: At Tail	Lost Ground: Lining Deflection and Time Dependent Movements	Total	Comments
F Route, second tunnel at test section				$\delta = 10$ mm $V_l = 0.08\ \frac{m^3}{m}$ $\%V_L = 0.4\%$	43 mm $0.14\ \frac{m^3}{m}$ 1.5%	13 mm $0.09\ \frac{m^3}{m}$ 0.4%	13 mm $0.10\ \frac{m^3}{m}$ 0.4%	79 mm $0.62\ \frac{m^3}{m}$ 2.7%	Tunnel 8.5 m apart cc.
F Route—range of values along both tunnels		10-14.3	1.9-2.6	--------	--------	--------	--------	2.8-130 mm $0.24\text{-}1.11\ \frac{m^3}{m}$ 1.0-4.7%	
4. Frankfurt, Shield (Fahrgasse) (Chambosse, 1972; Sauer and Lama, 1973; Breth and Chambosse, 1972)	6.5	12.4	1.9	$\delta = 8$ mm $V_L = 0.08\ \frac{m^3}{m}$ $\%V_L = 0.2\%$	37 mm $0.37\ \frac{m^3}{m}$ 1.0%	25 mm $0.25\ \frac{m^3}{m}$ 0.6%	*	70 mm $0.69\ \frac{m^3}{m}$ 1.8%	Settlement point 1.7 m above crown. * Final readings not included.
10. London Transport (Attewell and Farmer, 1974)	4.1	29.3	7.1	$\delta = 8$ mm $V_L = 0.07\ \frac{m^3}{m}$ $\%V_L = 0.5\%$	4 mm $0.04\ \frac{m^3}{m}$ 0.3%	2 mm $0.02\ \frac{m^3}{m}$ 0.15%	4 mm $0.04\ \frac{m^3}{m}$ 0.3%	18 mm $0.17\ \frac{m^3}{m}$ 1.3%	
11. Heathrow Cargo Tunnel (Muirwood and Gibb, 1971; Smyth and Osbourne, 1971)	10.9	13.3	1.2	$\delta = 10$ mm $V_L = 0.13\ \frac{m^3}{m}$ $\%V_L = 0.14\%$	-5+8=3 mm $-0.07+0.12=0.05\ \frac{m^3}{m}$ 0.05%	1 mm $0.01\ \frac{m^3}{m}$ 0.01%	0 0 0	12 mm $19\ \frac{m^3}{m}$ 0.2%	
12. Boa Vista, Sao Paulo (Costa, et al, 1974)	5.5	11.8	2.1	$\delta = 2$ mm $V_L = 0.03\ \frac{m^3}{m}$ $\%V_L = 0.1\%$	18 mm $0.2\ \frac{m^3}{m}$ 0.8%	20 mm $0.3\ \frac{m^3}{m}$ 1.3%	30 mm $0.4\ \frac{m^3}{m}$ 1.7%	70 mm $0.9\ \frac{m^3}{m}$ 3.8%	Settlement cross-section. 4 m above crown.

Table 6. Lost Ground Around Single Tunnels (Cording, et al, 1976; MacPherson, et al, 1978) (Continued)

Case	Diameter 2R, m	Depth to Axis z, m	$\frac{z}{2R}$	Lost Ground: Before Face	Lost Ground: Over Shield	Lost Ground: At Tail	Lost Ground: Lining Deflection and Time Dependent Movements	Total	Comments
14. Mexico City, Siphon II Manuel Gonzales (Tinajero and Vieitez, 1971)	2.9	11.7	4.0	δ=40 mm V_L=0.2 $\frac{m^3}{m}$ $\%V_L$=3%	30 to 80 mm est. 0.1 to 0.4 $\frac{m^3}{m}$ 2% to 6%		100 to 50 mm est. 0.5 to 0.2 $\frac{m^3}{m}$ 7% to 3%	170 mm 0.8 $\frac{m^3}{m}$ 12%	Settlement point 1.2 m above crown. Total at 28 days.
16. Frankfurt, no shield Baulos 25 (Chambosse, 1972; Sauer and Lama, 1973)	6.5	14.6	2.2	δ=33 mm V_L=0.33 $\frac{m^3}{m}$ $\%V_L$=0.8%	(No Shield)	12 mm 0.12 $\frac{m^3}{m}$ 0.3%	13 mm 0.13 $\frac{m^3}{m}$ 0.3%	56 mm 0.55 $\frac{m^3}{m}$ 1.5%	Settlement point 1.7 m above crown .
17. Washington, D.C. Metro, Section D-9, 2nd tunnel	6.4	16.1	2.5	δ=26 mm V_L=0.21 $\frac{m^3}{m}$ $\%V_L$=0.7%	51 mm 0.41 $\frac{m^3}{m}$ 1.3%	13 mm 0.11 $\frac{m^3}{m}$ 0.3%	13 mm 0.11 $\frac{m^3}{m}$ 0.3%	103 mm 0.83 $\frac{m^3}{m}$ 2.6%	Settlement point 1 m above crown.
18. Rockford sewer tunnel	2.8	9.8-12.8	3.4-4.5	δ=3-10 mm V_L=0.02-0.05 $\frac{m^3}{m}$ $\%V_L$=0.3-0.7%	51-109 mm 0.26-0.51 $\frac{m^3}{m}$ 4.1-8.0%	28-66 mm 0.15-0.32 $\frac{m^3}{m}$ 2.3-5.0%	3 mm 0.01 $\frac{m^3}{m}$ 0.2%	91-185 mm 0.46-0.87 $\frac{m^3}{m}$ 7.3-13.7%	Wheel excavator type mole, expanded lining. Settlement points 1-1.2 m above tunnel crown.
19. Washington, D.C. Metro, Section G1, 2nd tunnel	6.3	13.4	2.1	δ=8-10 mm V_L=0.07-0.09 $\frac{m^3}{m}$ $\%V_L$=0.2-0.3%	23-18 mm 0.22-0.16 $\frac{m^3}{m}$ 0.7-0.5%	10-8 mm 0.10-0.07 $\frac{m^3}{m}$ 0.3-0.2%	15-8 mm 0.14-0.08 $\frac{m^3}{m}$ 0.5-0.3%	56-43 mm 0.54-0.41 $\frac{m^3}{m}$ 1.7-1.3%	Expanded lining. Settlement point 1.5 m in above tunnel crown.

NOTES:

δ = Vertical settlement of deep settlement point

V_L = Volume lost into tunnel

$\%V_L = \frac{\text{Volume lost into tunnel}}{\text{tunnel volume}}$

1 mm = 0.04 in.

1 m = 3.3 ft

Table 7. Volumes and Displacements for Single Tunnels (Cording, et al, 1976; MacPherson, et al, 1978)

Case	Tunnel dia. 2R, m	Tunnel depth (to axis) z, m	z/2R	Vertical displace. δ_{max}, mm surface	deep	Volume of Movement, m^3/m: Surface V_S	Tunnel V_L	Compres. at side V_C	Expansion over tunnel V_E	Trough widths, m: i/r	W=2.5 i	Slope of Surface Settlement Trough: Cross-section avg.	max	Longitudinal section
1a. Washington, D.C. Metro, Section A-2, 1st tunnel C line	6.4	14.6	2.3	152	345	1.7	2.5 (7.8%)	0.3	1.0	1.4 (β = 28°)	11(14)	1:75	1:50	1:50
B line	6.4	14.6	2.3	139		1.5				1.3	11(14)	1:75	1:50	1:60
A line	6.4	14.6	2.3	76		1.0				1.7	14	1:180	1:140	1:200
(Hansmire, 1975)						(3 to 5%)				(β = 36°)				
2. Washington, D.C., Treasury Yard (Hansmire, 1975)	6.4	11.6	1.8	280	350	1.4 (4.3%)	2.5 to 3 (8%)	-	1.0-1.5	0.6 (β = 9°)	5	1:18	1:13	
3a. Washington, D.C. Metro, Section F2a, L Route, first tunnel at cross-section	5.5	20-21.9	3.7-4.1	3-10	18-53	0.20-0.37 (0.8-1.6%)	0.17-0.47 (0.6-2.0%)	-	-	3.1-3.6 (β = 39°- 45°)	21-25	1:2500	-	1:1800-1:2500
Range of values along L Route, first tunnel		16-23	3.0-4.1	3-28	18-104	0.07-0.38 (0.3-2.3%)	0.17-0.75 (0.6-3.4%)	-	-	-	-	-	-	-

Table 7. Volumes and Displacements for Single Tunnels (Cording, et al, 1976; MacPherson, et al, 1978) (Continued)

Case	Tunnel dia. 2R, m	Tunnel depth (to axis) z, m	z/2R	Vertical displace. δ_{max}, mm surface	Vertical displace. δ_{max}, mm deep	Volume of Movement, m^3/m Surface V_S	Tunnel V_L	Compres. at side V_C	Expansion over tunnel V_E	Trough widths, m i/r	W=2.5 i	Slope of Surface Settlement Trough Cross-section avg.	max	Longitudinal section
3b. Washington, D.C. Metro, Section F2a, F Route, first tunnel at cross-section	5.5	11.0	2.0	46	81	0.41 (1.7%)	0.65 (2.7%)	-	-	1.3	9 (β = 30°)	1:200	1:180	1:380 - 1:1000
Range of values along F Route, first tunnel		10.4-14.3	1.9-2.6	3-46	28-130	0.08-0.46 (0.4-2.0%)	0.24-1.12 (1.0-4.7%)	-	-	-	-	-	-	-
4. Frankfurt Shield, Fahrgasse (T-9) *	6.5	12.4	1.9	70	130	0.86 (2.6%)	1.1 (3.3%)	0.08	0.3	1.5	12 (β = 35°)	1:174	1:60	1:300
5. Frankfurt Shield, Domplatz *	6.5	15	2.3	23	52	0.39 (1.2%)	0.46 (1.4%)	0.10	0.17?	2.1	17 (β = 42°)	1:700	1:500	
6. Frankfurt Shield, Dominikanergasse *	6.5	10.3	1.6	140		1.36 (4.1%)				1.2	10 (β = 33°)	1:70	1:30	
7. Frankfurt, no Shield, Baulos 17 *	6.5	13.3	2.1	13	17	0.23 (0.7%)	0.17 (0.5%)	0.03	0?	2.2	18 (β = 48°)			
8. Frankfurt, no Shield, Baulos 18a * Tunnel 13	6.5	16	2.5	10	18	0.18 (0.5%)	0.15 (0.4%)	?	0?	2.2	18 (β = 43°)	1:1800		
9. Tyneside (Attewell, et al, 1975)	2	7.5	3.8	6	12	0.06 (1.9%)	0.06 (1.9%)	0.1?	0?	4	10 (β = 50°)	1:1600		1:1200
10. London Transport (Attewell and Farmer, 1974)	4.1	29.3	7.1	6	17	0.19 (1.4%)	0.17 (1.2%)		0?	6.1	32 (β = 46°)	1:5000		1:5000

Table 7. Volumes and Displacements for Single Tunnels (Cording, et al, 1976; MacPherson, et al, 1978)
(Continued)

Case	Tunnel dia. 2R, m	Tunnel depth (to axis) z, m	z/2R	Vertical displace. δ_{max}, mm surface	deep	Volume of Movement, m^3/m: Surface V_S	Tunnel V_L	Compres. at side V_C	Expansion over tunnel V_E	Trough widths, m: i/r	W=2.5 i	Slope of Surface Settlement Trough: Cross-section avg.	max	Longitudinal section
11. Heathrow Cargo Tunnel (Muirwood and Gibb, 1971; Smythnsbourne, 1971)	10.9	13.3	1.2	12	14	0.19 (0.2%)	0.19 (0.2%)		0?	1.2 (β = 38°)	16	1:1300		
12. Boa Vista, Sao Paulo (Costa, et al, 1974)	5.5	11.8	2.2	70	74	1.2 (5%)	0.9 (4%)			2.5 (β = 50°)	17	1:240		
13. Brussels Metro (Vinnel and Herman, 1969)	10	16	1.6	150		2.0 (2.5%)				1.1 (β = 26°)	13			
14. Mexico City, Siphon II Manuel Gonzales (Tinajero and Vieitez, 1971)	2.9	11.7	4.0	105 (28 days)	170	2.1 (38%)	0.8 (12%)	1.3?	0?	5.4 (β = 58°)	20	1:190		
15. Lower Market St. BART, San Francisco (Duesel, 1972)	5.5	19	3.4	36		0.64 (2.7%)				2.5 (β = 37°)	17	1:500		1:300-1:500
18. Rockford sewer tunnel, at cross-sections	2.8	9.8-13.7	3.4-4.8	15-46	91-185	0.09-0.40 (1.5-6.3%)	0.46-0.87 (7.3-13.7%)	-	-	1-3 (β = 11°-36°)	3-9	1:130-1:470		

NOTES: *Chambosse, 1972;
Sauer and Lama, 1973;
Brath and Chambosse, 1972

1 mm = 0.04 in.
1 m = 3.3 ft

the surface settlement trough can be estimated as the sum of the volume lost into the tunnel, volume of compression outside the tunnel springline, and the consolidated volume immediately around the tunnel.

3. For single tunnels in dense sand, volume of expansion above the tunnel crown may reduce the settlement volume at the surface.

4. For most of the tunnel cases studied, the normal probability curve for representing the shape of the settlement trough is still valid. However, as shown in Figure 10, in some special tunnel cases, the relationship between soil type and normalized width of the settlement trough may need some modification. Introduction of the angle of draw, β, may help the interpretation of the observed data and prediction of the amount of settlement in future tunnel projects, as shown in Figure 11. It is discerned from Figure 10, for tunnel depths in the range of $z/2R \leq 4$, that the limits given by Peck (1969) (Figure 9) for rock, hard clay, and sand above the water table correspond to $\beta = 11^{\circ}$ to 33°. In the same depth range, the limits for soft clay correspond to $\beta = 33^{\circ}$ to 50°. Values of β were greater than 50° in soft Mexico City clays where large volume decreases developed due to consolidation of clay outside the tunnel springline.

5. When maximum settlements are large, the shape of the settlement trough may no longer correspond to a normal probability curve. As the displacements continue, further displacements tend to concentrate in the center of the trough. At the edge of the trough, very little additional settlement develops, and the trough width tends to reach a limit once a certain settlement is reached. Based on the tunnel cases summarized, further displacements were concentrated in the center of the trough once δ_{max}/z exceeded 0.5%.

6. For twin tunnels, settlements at the ground surface will be larger than the sum of the settlement volumes for two single tunnels. In clays, the additional settlement due to interference results from compression of the pillar between two tunnels, and from deflection of the lining of the first tunnel. In sands, in addition to the interference effects previously described, volume decreases will develop in the previously expanded zone over the first tunnel. The volume of the settlement trough for twin tunnels can be conservatively estimated by ignoring the volume increase over the tunnels. The slope of the trough outside the tunnel centerline may not change appreciably from the slope for a single tunnel. However, if the tunnels are deep and closely spaced, settlement troughs may overlap strongly enough to increase the slope at the edge of the trough.

7. Estimates of settlements can form the basis for decisions about the need for and extent of underpinning required to protect buildings and utilities. The procedures to estimate potential amounts of ground loss and related surface settlement are as follows:

   a. Estimate expected ground losses, V_L (face, shield, tail, etc.).

   b. Estimate settlement volume, V_s, using V_s vs. V_L correlations or simply assume $V_s=V_L$.

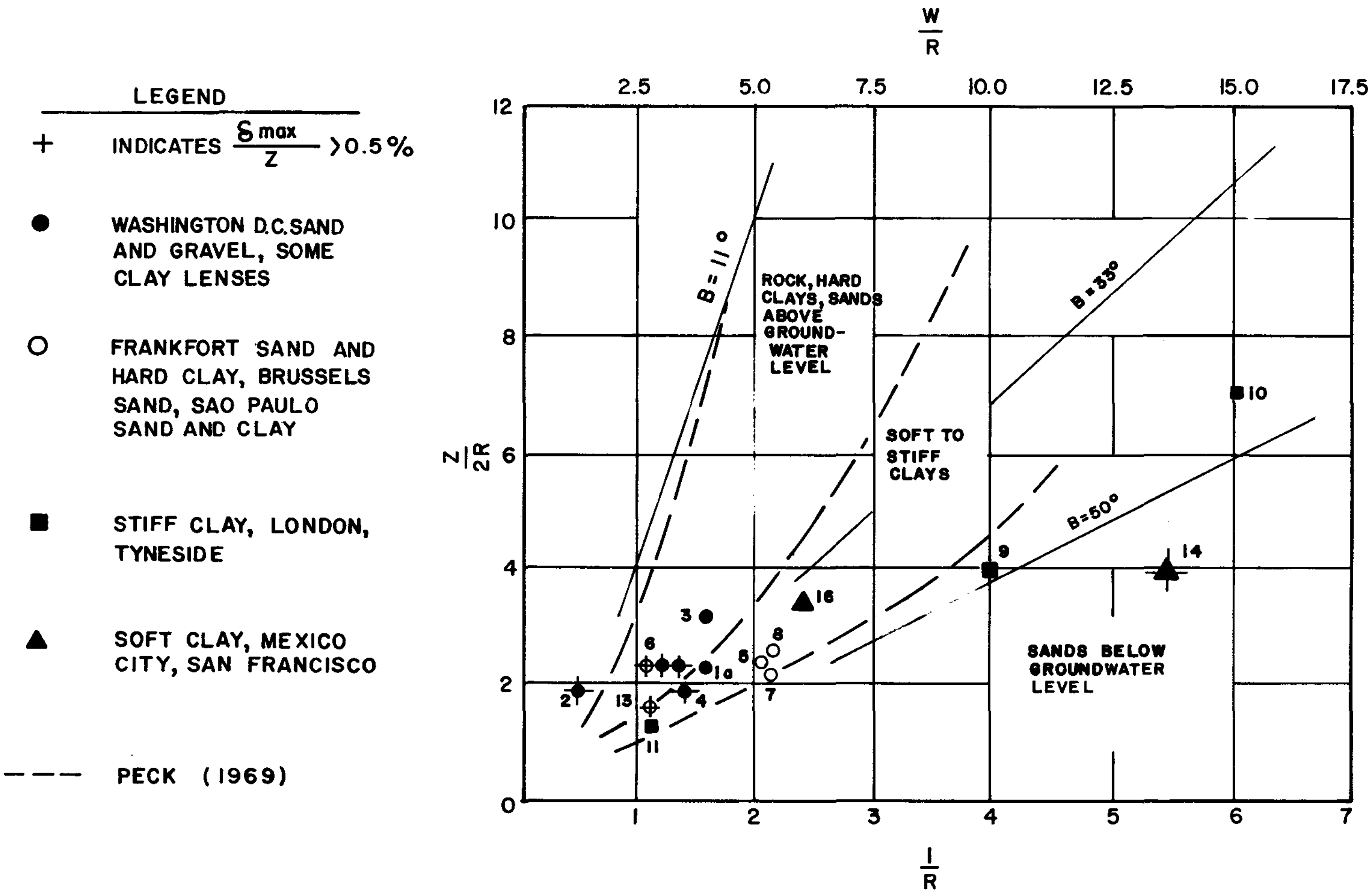


Figure 10. Modified Width of Settlement Trough
(Cording, et al, 1976; MacPherson, et al, 1978)

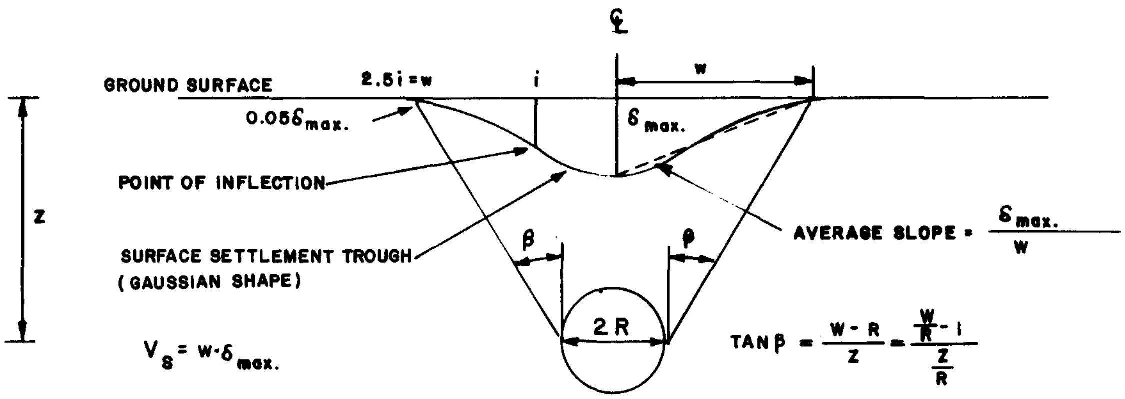


Figure 11. Ground Surface Settlement Trough Geometry
(Cording, et al, 1976)

c. Estimate settlement trough width, w, using Figure 10.

d. Calculate surface settlement over tunnel centerline using the estimated V_s and w ($\delta_{max} = V_s/w$).

This procedure provided an estimate of the magnitude and extent of the average surface settlement likely to occur for a single tunnel driven with good workmanship and technique. These estimates exclude the possibility of the so-called catastrophic movements.

8. In view of the direct relationship between surface settlement volume and ground loss, it is important to control and limit ground losses during tunneling in order to limit settlements. Desirable construction practices to reduce ground loss include:

   a. Adequate control and support of the tunnel face by advance dewatering, use of compressed air, advance grouting of the soils, full-face breasting or breasting shelves, or other means.

   b. Limiting the projection on the exterior of the shield to reduce over-excavation.

   c. Providing a maneuverable shield with a small length-diameter ratio to reduce plowing of the shield.

   d. Using a thin tailskin on the shield, and promptly and completely grouting the tail void while the lining is within the tailskin or as the lining emerges from the tailskin, or promptly expanding the lining into full contact with the soil as the lining emerges from the tail.

2.3.2 Damage to Adjacent Structures

A substantial portion of the cost of soft ground tunnels in urban environments is devoted to the protection and repair of adjacent structures and utilities. In some instances, the locations of tunnel routes are selected to avoid large or sensitive structures. However, if (1) the range of the typical ground movements for various combinations of ground conditions and construction procedures can be estimated, and (2) the response of nearby structures and utilities to the different range of ground movements can be evaluated, an appropriate tunneling system that is compatible with surrounding structures as well as economical can be developed.

Building damage may be divided into three general categories (Boscardin, et al, 1978), as follows:

1. Architectural damage - Damage affecting the appearance of structures. It is usually related to cracks or separations in panel walls, floors, and finishes. Cracks referred to there are in the order of 1/32 in. wide.

2. Functional damage - Damage affecting the use of the structure. It is usually related to jammed doors and windows, cracking and falling plaster, tilting of walls and floors, etc. This kind of damage requires nonstructural repair to return the building to its full service capacity.

3. Structural damage - Damage affecting the stability of the structure. It is usually related to cracks or distortions in primary support elements such as beams, columns, and load bearing walls. The above classifications are very broad and considerable overlap among the categories often occurs depending on the function of the structure. For instance, architectural damage to a museum may also be considered functional damage. In contrast, the limit for functional damage of a warehouse may coincide with the limit for structural damage. Thus, the function and unique characteristics of a specific building must be considered in a discussion of damage with respect to that particular structure.

Two parameters commonly used for developing a correlation between damage and differential settlement are the angular distortion and the deflection ratio. Angular distortion is defined as the differential settlement between two points divided by the distance separating the points, minus the possible rigid body tilt. In this way, the value represents the deformed shape of the structure. The deflection ratio is defined as the maximum displacement relative to a straight line between two points divided by a distance separating the points.

Skempton and MacDonald (1956) investigated the limits of tolerable building distortions based on field observation of structures damaged as a consequence of settlements. They noted that angular distortions exceeding 1/150 were associated with structural damage, while angular distortions of about 1/300 were related to cracking in panel walls and load bearing walls. Grant, et al (1974), studied 95 buildings and also agreed with these limits.

Meyerhof (1956) regarded framed panels and load-bearing brick walls separately, and suggested utilization limiting angular distortions of 1/250 for open frames, 1/500 for solid frames, and 1/1000 for load-bearing walls or continuous brick cladding. Ploshin and Tokar (1957) noted a critical tensile strain of 0.5×10^{-3} as the limit for observable cracking of masonry and concrete walls.

Burland and Worth (1975) noted that, for load-bearing walls, damage occurs at lower distortions than for frame structures, and that convex settlement is a more severe condition than sag or concave settlement. The distortion limit is 1/1000 for a structure spanning the edge of the trough which would be subject to convex settlements.

Breth and Chambosse (1975) observed the settlement of a three-story and five-story structure during driving of tunnels in Frankfurt, Germany. The slope of the building foundations depended on their position within the trough, but were typically of the same order as, or less than, the average slope of the trough (Figure 12). They also observed a maximum longitudinal settlement slope of 1/800 for driving a tunnel beneath a three-story reinforced concrete frame structure. Final settlement was 80 mm. No significant damage was observed, although two 0.3 mm cracks were found on a basement wall.

Schmidt (1974) reported settlements of three buildings directly above the Toronto Subway--Section E-1 tunnels were significantly less than free field settlements; average floor and wall settlements ranged between 0.5 and 1.7 in. One of the buildings, the Glenhill School (three-story brick), was protected by grouting a 5-ft-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 and only minor plaster cracking appeared in the school building.

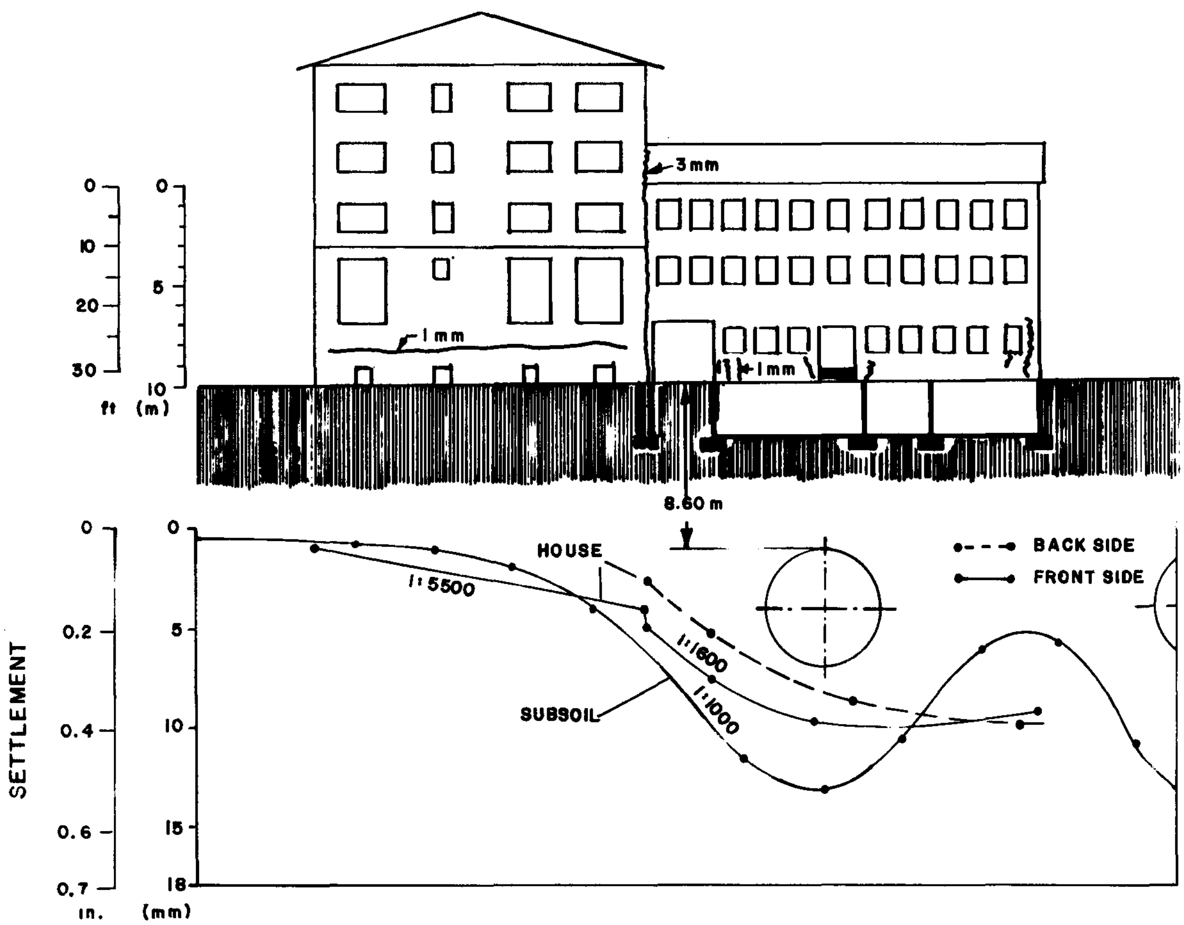


Figure 12. Settlement of Structure Over Frankfort Tunnel
(Berth and Chambosse, 1975)

Schmidt (1974) and Chambosse (1972) reported that tunnels passed directly under a number of buildings, and no underpinning was provided. They found that, in general, the building withstood settlement well. The maximum settlements were somewhat smaller than the adjacent ground surface settlements, and the buildings tended to settle without bending, i.e., the settlement profiles were nearly straight lines.

Boscardin, et al (1978), reported three case histories on building damage due to tunneling. They are briefly summarized as follows:

Case 2K

A 50- to 80-year-old four-story brick bearing wall structure was 22 ft by 60 ft in plan and was about 22 ft from the nearest tunnel centerline. A pair of tunnels 20.8 ft in diameter, 37 ft apart, and 48 ft deep, were excavated through medium dense sands and gravels with occasional clayey and silty lenses. The tunnels were shield-driven with steel ribs and wood lagging as initial support. The steel sets were at 4-ft intervals corresponding to the length of shove used.

The foundation for the structure was provided by rubble strip footings under the walls. The bearing walls were 16-in.-thick brick masonry, while the front facade wall was a 12-in.-thick brick wall clad with a veneer of architectural stonework. Prior to tunneling, the building was abandoned and unused.

Damage to this structure was extensive and resulted in its being declared structurally unsound. Surface settlements over the centerline of the nearer tunnel were in excess of 10 in. The maximum settlement recorded was 2.8 in. at the front building line. Angular distortions of 17×10^{-3} (1/60) and 8.3×10^{-3} (1/120) were calculated along the transversal direction to the tunnel axis. Angular distortions in excess of 5×10^{3} (1/200) were calculated along the longitudinal direction of the tunnel axis.

Both bending cracks and diagonal cracks were readily visible in an exposed bearing wall (Figure 13). The diagonal cracking occurred near the front of the structure with a distance from the excavation equal to the height of the building (H). The bending cracks occurred approximately at a distance, H, from the front of the building and near the top of the bearing wall. At the fourth floor, it was noted that the facade wall had pulled away 1 in. or more from both the ceiling and the floor. From the exterior, the facade wall cladding appeared to be on the verge of buckling and separating from its support. During the tunnel excavation through the site, two pieces of the architectural stone cornice fell from the facade. In the basement, a 2-in.-wide vertical crack was observed in one bearing wall near the front facade wall.

Much of the problems associated with the combination of excavation and structure were due to the occurrence of large, localized runs at the face of the tunnel excavation. The run tended to create erratic variations in the ground settlement which in turn caused severe distortions of the structure. It is apparent that the angular distortions of the structure (1/60 to 1/200) are in the range where significant structural damage is to be expected and sufficient to warrant condemning the structure.

Case 2L

A three-story masonry structure, 270 ft by 205 ft in plan, was located approximately 30 ft from the nearest tunnel centerline. Each tunnel was about 21 ft in

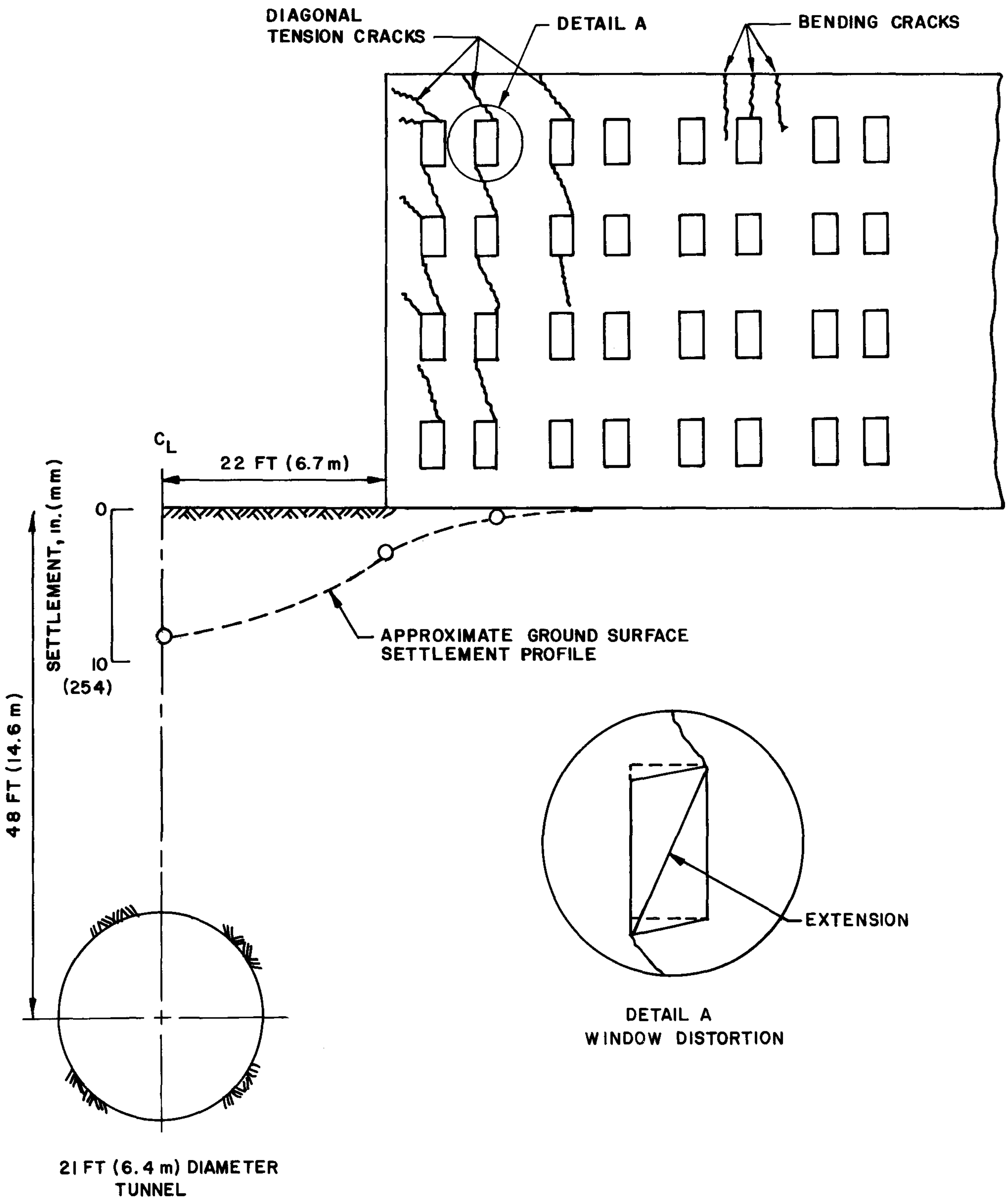


Figure 13. Cracking in the Side of Building, Case "A"
(Boscardin, et al, 1978)

diameter, 37 ft apart, and 62 ft deep. Excavation of the tunnels was accomplished with a shield and support provided by steel ribs and wood lagging. The steel sets were 4 ft apart. The soil profile at this location consisted primarily of dense silty sands and gravels.

The long dimension of the building paralleled the tunnel, and had a basement and three stories above ground. Concrete blocks with marble facing formed the 4-ft-thick exterior walls. Spread footings provided the foundation support for walls. No underpinning was employed; however, the ground mass around the tunnels was stabilized through grouting operations.

The building sustained extensive damage, cracking appeared on all floor levels in both walls and ceilings and required major repairs. Large continuous cracks appeared in the crowns of the corridor barrel vaults due to horizontal extension on all three floors. Cracks in excess of 1/4 in. appeared, and shoring to provide additional support for the barrel vaults was required. The construction files indicated that problems with dewatering were encountered and many runs occurred in the sandy soil. This resulted in large local ground losses and erratic settlement patterns.

Street settlements in excess of 6 in. and building settlements ranging up to 1 in. developed. An angular distortion of 1.1×10^{-3} (1/910) was calculated for the north end of the structure. At the south end of the structure, where most of the damage was observed, angular distortions of 3.6×10^{-3} (1/280) and 7.2×10^{-3} (1/140) were calculated from building settlement data. In addition to building damage related to angular distortion, the structure appeared to have suffered distress from horizontal extension.

Case 2M

A pair of two-story brick bearing wall structures with basements was located adjacent to two 21-ft-diameter tunnels, as shown in Figure 14. The soil profile indicates that the test section is in a transition from dense sands and gravels in the river flood plain deposits to hard, clayey Cretaceous soils. Observations made at the tunnel heading during excavation beneath the test section indicated that the heading material was a hard red clay with occasional weathered and sandy zones near the tunnel crown.

The average slope of the settlement trough beneath Building I was 1/230, with maximum settlement of 1.6 in. at the center of the trough and 1.4 in. at the nearest corner of the building. The structures settled and strained laterally in compliance with ground movements, and did not appear to restrain the ground movements to any significant extent.

Some of the distortions during development of the settlement trough were larger than the final distortion recorded. This is observed in Building I, where the final lateral extension, 1/12500, was less than the lateral extension of 1/3300 during settlement trough development. Reversals of curvature are often induced in buildings as the settlement trough develops, and can cause greater distortions than the final distortions. The longitudinal settlement wave may experience similar reversals of curvature and horizontal movement as those of the transversal settlement trough.

The final modes of deformation of the structures were directly akin to the position of the structures related to the settlement trough. For Building I, located primarily within the concaved bowl-shaped portion of the settlement trough, the predominant mode

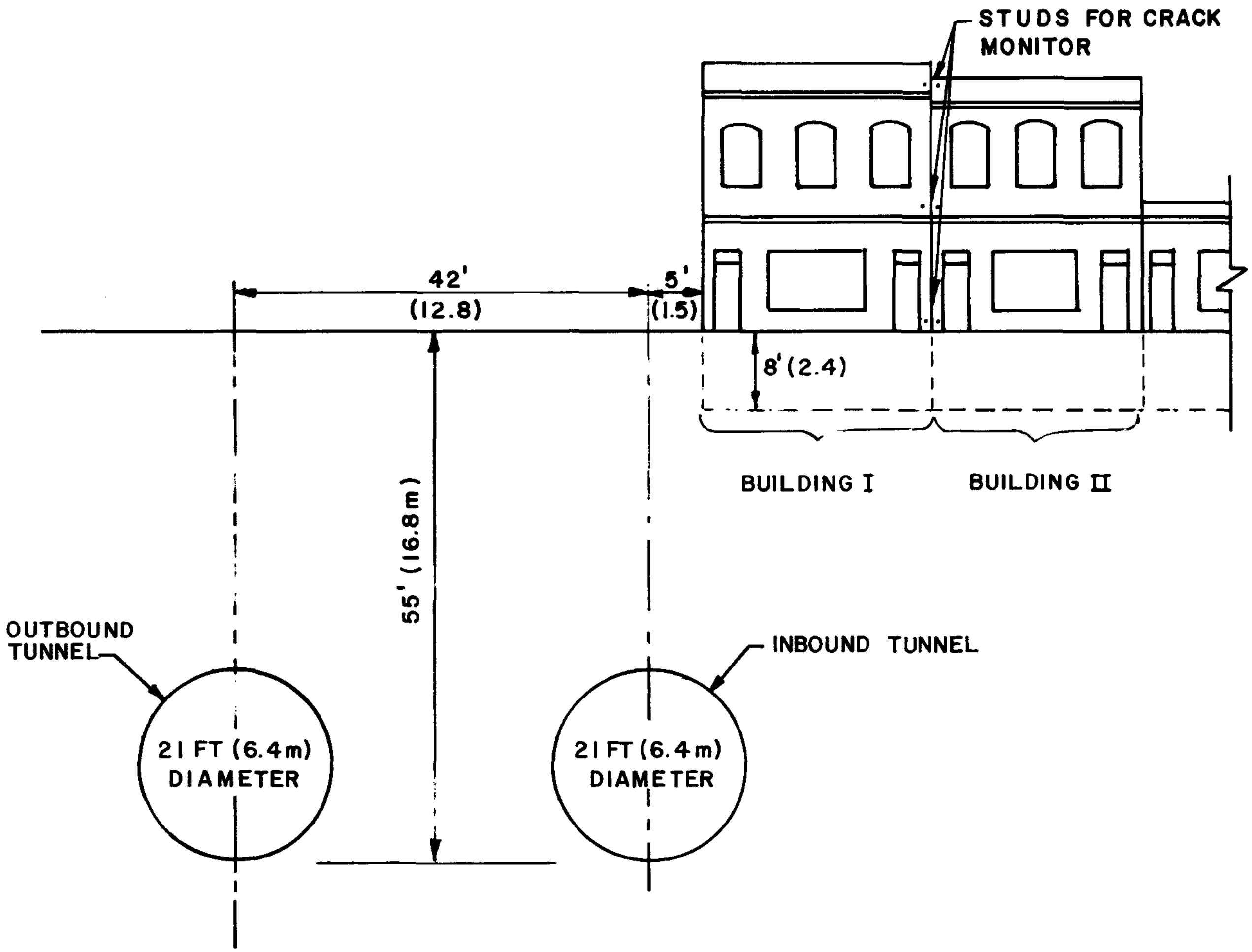


Figure 14. Profile Showing Relative Positions of Buildings and Tunnels
(Boscardin, et al, 1978)

of deformation was due to angular distortion. The building width was equal to approximately 1/3 the half width of the settlement trough so that there was a significant rigid body rotation (1/520) of the building. This resulted in an angular distortion of 1/410, which was less than the average slope of the settlement trough beneath the building (1/230).

For Building II, located on the convex portion of the settlement trough, lateral extension was significant in causing building deformation (angular distortion equal to 1/2000 and lateral extension equal to 1/3100). The convex bending produced larger lateral extensions (1/1300) in the upper floor. Most of the lateral extension was concentrated in one crack parallel to and immediately adjacent to the bearing wall nearest the center of the excavation. Larger lateral strains developed in the upper floor because the joists and facade walls between the bearing walls provided very little resistance to bending.

Cracking and damage in Building I was minor. The cracking and crack widening that did occur, approximately 1/32 in. to 1/16 in., were not significant owing to the poor initial condition of the structure. Cracking and damage in Building II were caused primarily by the lateral extension and convex bending. The crack widened to about 1/8 in. in the basement and 1/4 in. at the second floor. Nearly all of the lateral extension strain across the building was concentrated in this one crack.

Based on the above review, the soil properties and construction workmanship, the location, orientation, and size of the structure, the stiffness of the foundation and superstructure relative to the ground as well as the magnitude of the ground movements, are the main parameters related to structural damage. The angular distortion and lateral strain are the two direct sources that cause damage to the building.

2.4 CONSTRUCTION PROBLEMS AND PERFORMANCE OF FLEXIBLE LINERS

In the course of mining a tunnel, lots of construction problems can occur. Although some of the expected tunneling problems have been discussed in previous sections, some unexpected tunneling events may completely stop the advance and increase the cost. These particular tunneling events may need special treatment based on each individual situation. Some of these particular experiences will be reviewed in this section.

In recent years, there has been a general trend toward using the one-step flexible liner; for instance, fabricated steel liner, cast iron liner, and pre-cast concrete segmental liner. The design of these types of liners has been discussed in Volume 3 (Hampton, et al, 1980) of this series of reports. However, one of the basic assumptions for the design of these types of liners is a diameter distortion of 0.5% based on past performance of various flexible liners. Some additional flexible liner performance data are available at the present time, and also will be included in this section.

2.4.1 Tunnel Construction Problems

Schmidt, et al (1976), reported five case histories related to unexpected construction problems. They are summarized as follows:

Case 2N - Interceptor Sewer, Staten Island--Boulder Problem

An interceptor tunnel, 7000 ft long and about 10 ft in diameter, was mined along Richmond Terrace in Staten Island, New York. A major portion of the tunnel runs through a glacial till containing numerous boulders, many larger than 2 ft in longest dimension. A mole employing an articulated hoe excavator and a conveyor belt advanced the tunnel, with a steel segmented liner being erected inside the tail of the shield.

Though a mole, in theory, can advance a tunnel several tens of feet per shift, the actual production rate for this project was frequently only 2 to 4 ft per shift. Since a 2-ft boulder was the maximum size that could be handled by the conveyor, larger boulders had to be split by hydraulic means. Boulders encountered along the periphery of the shield had to 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 cost approximately $500,000, and justification for the use of such expensive machinery was based on the potentially high production rate. With production rates between 5 to 15 ft per day (typical for the bouldery area), the return on investment was questionable. However, an adjacent, similar tunnel contract employed a similar shield and excavator, but with a wider conveyor belt for muck removal. Because there was less need for boulder splitting, tunneling progress was significantly less influenced by the presence of boulders.

Another incident happened in this tunnel contract: the breaking of a 16-in. sewer located about 9 ft above the tunnel and 5 ft 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 a large inflow of sewage and temporary face instability. The sewer was exposed and repaired the next day. The cost of the incident can be estimated at somewhere above $10,000 including the repair of the sewer, but fortunately, it caused no injury or surface traffic accidents.

Case 2O - Detroit Tunnel--Compressed Air Leaking Problem

During construction of a tunnel in Detroit, compressed air inadvertently found its way into an old permeable brick sewer, causing it to back up explosively. A residential house 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.

Case 2P - South Charles Relief Sewer, Boston, at Charles River--Compressed Air Leaking Problem

The tunnel was shield-driven (1958-1960) with air pressure varying from 6 to 12 psi. Its outside diameter was approximately 11 ft, 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 4 psi to reduce air loss while remedial measures were taken to plug the leak. It caused only limited damage and a modest delay in construction.

Case 2Q - Washington Metro, Contract C4--Disposal Gas Problem

In the area of the Watergate Apartment project, an unusual and unexpected tunneling problem was encountered during construction of the Twin Metro Tunnels and two shafts. 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 of fluids from the gas works carried tar-like substances into the ground where they settled out predominately near the soil-rock interface.

This tar-like material was first uncovered during excavation for the shafts. It gave off noisome fumes that were, on occasion, ignited by the action of the excavation tools, and was generally 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 the Coast Guard skimming equipment. Fortunately, the quantity of noisome, flammable, and potentially explosive fumes was small, and no serious accidents occurred.

Case 2R - San Francisco BART, Lower Market Street, Contract B0031--Pile Obstruction Problem

Two 18-ft outside diameter, segmented steel-lined tunnels passed through soft clay beneath the Ferry Building whose foundation 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. A total of 896 piles were encountered, including one steel H-pile and one 12 in. x 12 in. concrete pile. Although the locations of piles beneath the Ferry Building presumably were known of in the plan, at least at the pile cap elevation, the pile locations actually observed in the tunnel face had little resemblance to the pile plan (Figure 15).

If the shield was driven up against a pile, there would be a considerable risk of displacing it horizontally, thus 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 in., or 10 in. longer than the standard shove. Timbers were severed with a hydraulically operated chain saw, cutting the pile above the top of the shield but leaving a stub of about 2.5 ft 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 6 in. of a pile before the pile was cut free of the soil by hand and sawed.

Some distortions of liner rings resulted due to 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 plated between the ribs. Some difficulties also arose in connection with the caulking of segment joints to secure watertightness.

2.4.2 Performance of Flexible Liners

Based on the discussions of soft ground tunnel design procedures in Volume 3 (Hampton, et al, 1980) of this series of reports, the semi-empirical design procedure for

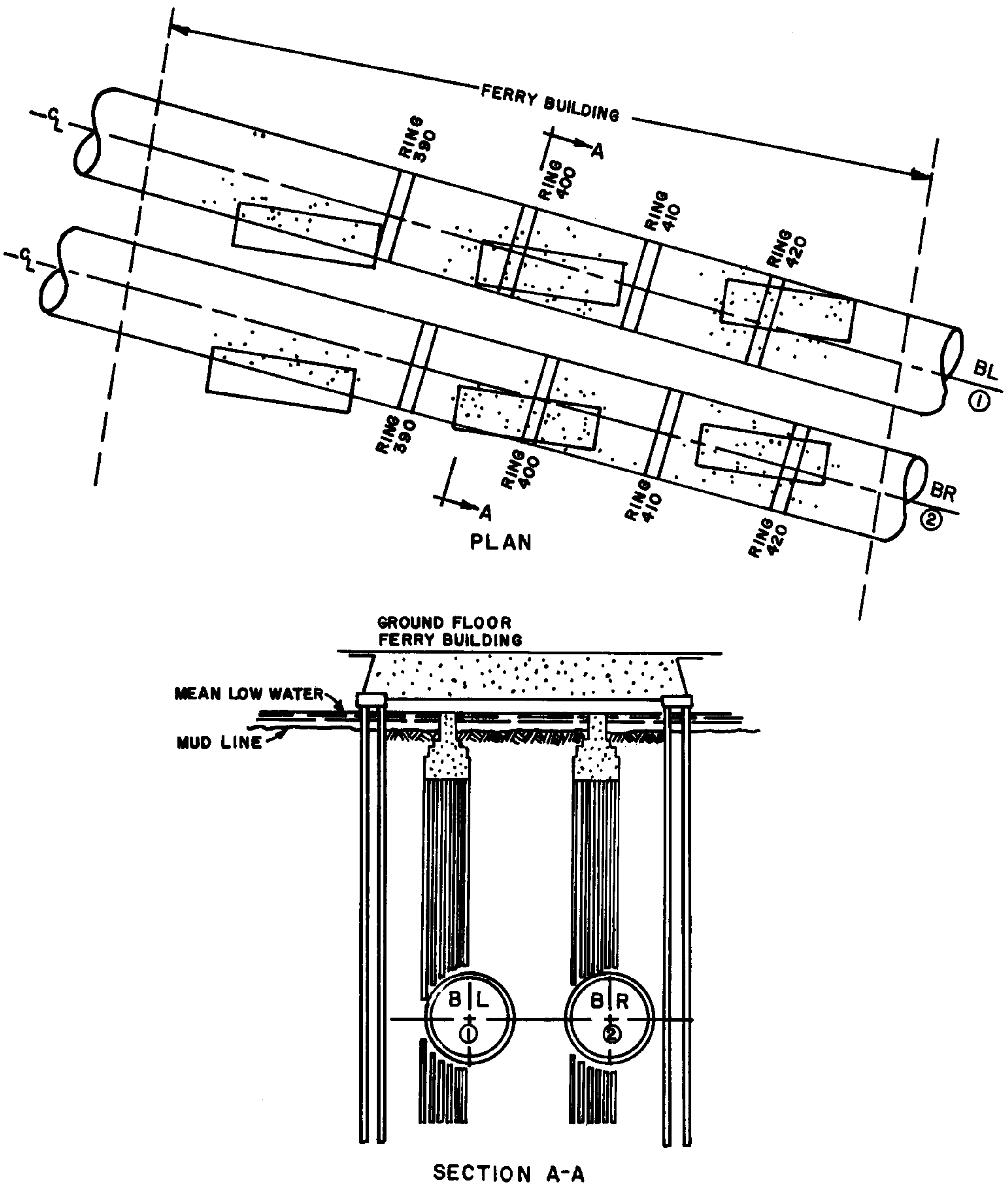


Figure 15. Actual Pile Locations at the Tunnel Face and the Presuming Pile Locations in Construction Plan

(Kuesel, 1972)

flexible liners proposed by Deere, et al (1969), and Peck (1969) consists of four separate steps, as follows:

1. Provide adequately for ring loads to be expected.

2. Provide for the anticipated distortions due to bending.

3. Give appropriate consideration to the possibility of buckling.

4. Make allowance for any significant external conditions.

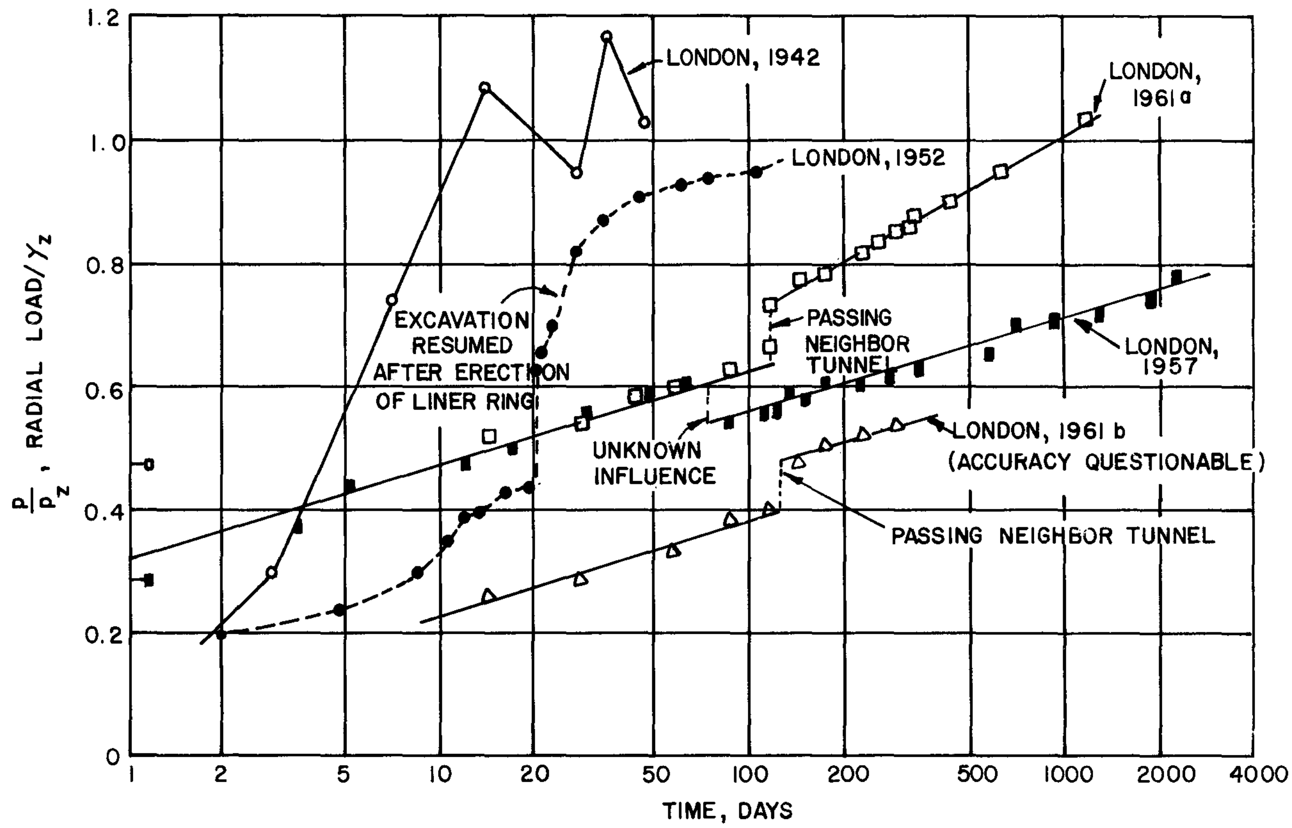
For point 3, design of the liner for buckling rigidity against the external water pressure is based on the theoretically developed formula ($P_{cr} = 3EI/R^3$). Thus, no further discussion is needed. For point 4, the significant external conditions may include jacking forces from the shield against the lining, the interaction between parallel tunnels, the influences of existing foundation loads and future adjacent excavations, etc. The design of the flexible liner for these external conditions is highly dependent upon the actual set-up of each individual case. Generally, it appears reasonable to increase the ring loads and the diameter distortions for the flexible tunnel liner design.

2.4.2.1 Ring Loads

Peck (1969) summarized ring load (see item 1., Section 2.4.2) information for various tunnels in soft ground, as presented in Figure 16. According to this figure, the ring load for a single tunnel in clay after a long period (for example, 100 years) is not likely to exceed p_z, where p_z is the total vertical overburden pressure at the tunnel springline elevation. Loads (p) have been determined from direct pressure measurements or calculated from measured related ring strains. For swelling and overconsolidated clay, the ring loads may be higher than p_z , while for nonplastic soils the final ring load for a single tunnel could be considerably smaller than p_z. General data concerning the construction of some of the tunnels in Figure 16 are included in Table 8.

Evans and Hampton (1974) reported measurement of the soil load on a horseshoe-shaped tunnel liner located in the mixed face condition, and the tunnel was hand mined. According to their report, the field measured liner loads are approximately 43% of the assumed total vertical overburden pressure at the tunnel springline elevation after 3 weeks. Since the overburden soils are predominately compact sand and gravel, the arching effect occurred over the tunnel and overburden loads were redistributed to the bedrock on each side of the tunnel. The liner loads may be increased with time. One contributing factor to these increases could be hydrostatic pressure build-up after the groundwater table is allowed to return to its original level. However, these low, temporary loads on the liner are an asset in the design of temporary or short-term loaded structures.

Beloff, et al (1979), reported the performance of a 10-ft-diameter steel tunnel liner in soft ground. Through the monitoring program of six instrumentation stations and for six months after liner erection, they found that the apparent liner loads are much less than those predicted for all stations (as shown in Figure 17). However, the relative magnitudes of the liner loads are still dependent on the soil types and construction conditions. They further found that the maximum combined stresses (bending and thrust) occurred near the center of the segments, with only minor stresses occurring at the longitudinal joints, indicating only partial movement restraint along the joints. In general, the hoop stresses were approximately 5% of the combined maximum stresses.



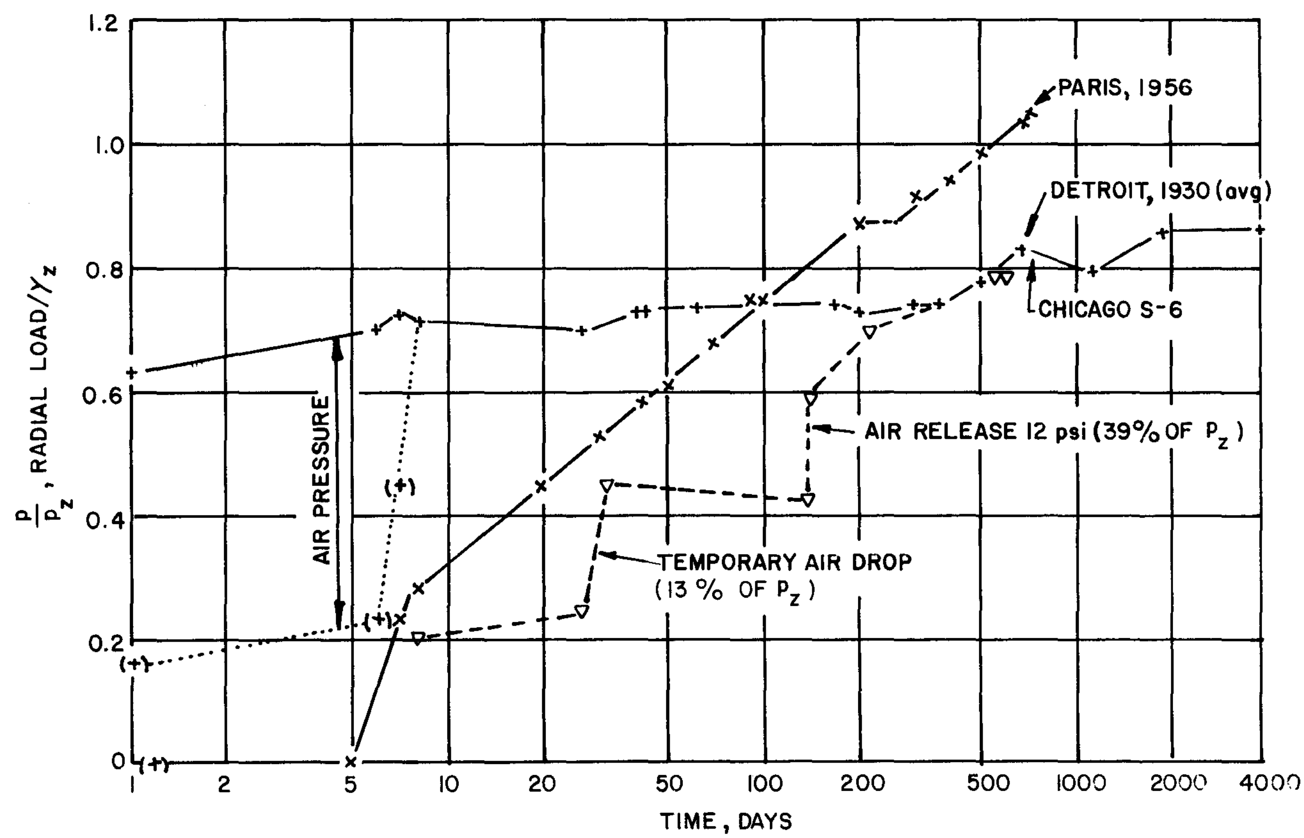


Figure 16. Time-Dependent Radial Load on Flexible Tunnel Liners in Soft Ground (Peck, 1969)

Table 8. Thrust and Distortion of Flexible Tunnel Liners in Soft Ground (Peck, 1969)

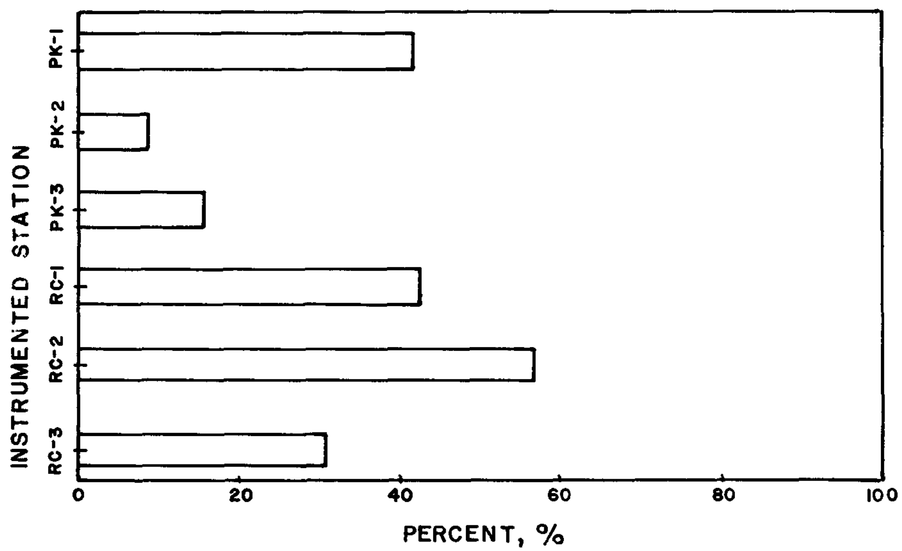
Case	Reference	Lining: Type	Lining: Radius R ft	Lining: Thickness t, in.	Lining: Rigidity ksi	Soil: Type	Soil: Strength s_u ksf	Total Overburden P_z, ksf
London, 1942	Skempton, 1943	Bolted iron segments	6.4	1.33	small	London clay	5.75	13.8
			Crown thrust after 46 days: p = 14 ksf, stable Moment: insignificant					
London, 1952	Cooling & Ward, 1953	do.	12.7	2.3	-.003	London clay	7-22	12.5
			46 days: Distortion $\Delta R/R$ = .072 to .168%, av. .124%; Moments small; Thrust load p = 5.6-11.2 ksf					
London, 1957	Ward & Thomas, 1965	do.	3.8			London clay	7.2-9.6	6.95
			72 months: Distortion $\Delta R/R$ = .152 to .185% Thrust load p = 5.3 ksf					
London, 1961 (Victoria) (1)	Ward & Thomas, 1965	"Flexible" concrete segments	6.6	9	0.66	London clay	7.8-8.5$^+$	10.8
			4 months: Distortion $\Delta R/R$ = .15%; Thrust load p< 6 ksf; Moment < 12 k ft/ft					
London, 1961 (Victoria) (2)	do.	Flexible iron segments	6.4		-0	London clay	7.8-8.5$^+$	10.0
			4 months: Distortion $\Delta R/R$ = .16%; Thrust load p = 3.5 ksf; Moments = 0 at joints					
London, 1952 (Ashford)	Tattersall, et al, 1955	Don-Segs	4.4	12	4.8	London clay	22	11
			14 months: Distortion $\Delta R/R$ = .024 (hor.) to .057 %(ver.); Thrust load 5.2 ksf					
Charleston, 1968	Gould, 1968	Unlined horseshoe	3.5			Silty marl	4.5$^+$	10
			Relative horizontal diameter change $\Delta R/R$ = .25%					
Ottawa, 1961	Eden & Bozozuk, 1968	Corr. steel liner plates (primary)	5.0			Leda clay	3.7	6.1
			1 month: Distortion $\Delta R/R$ = .08 to .17 %					
Toronto, 1964 (1)	Matich & Carling, 1968	Bolted iron segments	8.5			Silty clay and clayey till	-0.7	4.4
			Distortion under air pressure (1.44 ksf) $\Delta R/R$ = .06%					

Table 8. Thrust and Distortion of Flexible Tunnel Liners in Soft Ground (Peck, 1969)

(Continued)

Case	Reference	Lining: Type	Lining: Radius R ft	Lining: Thickness t, in.	Lining: Rigidity ksi	Soil: Type	Soil: Strength s_u ksf	Total Overburden P_z, ksf
Toronto, 1964 (2)	Matich & Carling, 1968	Bolted iron segments	8.5			Sand and clayey till	dense -0.7	5.5
			Distortions erratic but < .1% (?)					
Norway, 1949 (Tyholt)	Hartmark, 1964 & 1968	Concrete, segments & cast-in-place	10.8	26	2.7	Sensitive clay	0.7-1.2	7.8
			17 years: Distortion ΔR/R = 0.24 to 0.65%; initial distortions not measured					
New York, 1906 (Penn. RR)	Jacobs, 1910	Bolted iron segments	11.5			Hudson R. silt	?	5.5 max
			First 2 weeks: Distortion ΔR/R = -.44 to -.54%; after several months: nearly back to circular					
New York, 1936 (Lincoln)	Rapp & Baker, 1936	do.	15.5			Hudson R. silt	?	6.5 max
			First 9 days: Distortion ΔR/R = -.4 to -.67% Moment 51 ft k/ft. After 175 days: R/R = -.09 to -.13%; Moment 25 ft k/ft; Thrust load 5.6 ksf					
Boston, 1960 (Callahan)	C.E. Jan, 1961 Richardson	Bolted steel segments	15.4	5/8" web w. stiffeners		Boston blue clay	v. soft	
			First week: Distortion ΔR/R = -.55%					
Garrison, 1951	Burke, 1957; Lane, 1957		17.5 - 18.0			Ft. Union shale		13.2 to 21.6
4A		Ribs & lagging.	18 months: Distortion ΔR/R = .35%					
4B$_2$		Yielding ribs.	18 months: Distortion ΔR/R = .42% (vert.> hor.)					
4C		Ribs & lagging.	18 months: Distortion ΔR/R = .43%					
4D		Slotted concrete.	18 months: Distortion ΔR/R = .27% (3' thick)					
Other tunnel sections		Ribs.	2 to 4 months: Distortion ΔR/R = .07 to .19%					
			Vertical loads (single tunnels) ranged to 15% of overburden pressure.					
Chicago, S6, 1940	Terzaghi, 1943	Ribs & liner plates, horseshoe shape	10.0			Chicago clay	0.7	4.9
			Ultimate distortion ΔR/R = .25%					
Chicago, S3, 1940	Terzaghi, 1942	Bolted steel segments, circular	12.5	3/8" web w. stiffeners	.019	Chicago clay	0.6	4.4
			First few days, distortion ΔR/R = -.33 to -.50%, then reversal of trend In second tube: Δ R/R = .05%					
Oakland, 1968 (BART)	-	Bolted steel segments	100			Clay		
			Distortion ΔR/R = -1.5% max (?) during construction (in shield, no tie rods) - imperceptible after grouting					

NOTES:
(1) Radius is exterior radius for steel and iron linings, average radius for concrete linings.
(2) Thickness is average or equivalent thickness.
(3) Rigidity is computed by EI/R^3, ksi.
(4) Distortion is positive when lining squats.
(5) "Thrust load" is average radial stress on lining corresponding to measured thrust, or is radial stress measured directly.
(6) 1 in. = 25 mm
1 ft = 0.3 m
1 ksf = 48 kN/m^2
1 ksi = 6900 kN/m^2



MAJOR SOIL TYPE	MAXIMUM SURFACE SETTLEMENT (IN.)	REMARKS
DENSE SAND	>12.0	DUE TO BROKEN WATER MAIN IN THE AREA
WEATHERED SERPENTINE	SURFACE SETTLEMENT NOT MEASURABLE	———
DENSE SAND	<1.0	———
ORGANIC SILT	4.0	———
ORGANIC SILT	6.0	SHEET PILING DRIVEN OUTSIDE TO BLOCK WATER FLOW
DENSE SAND	< 1.0	———

NOTE: 1in. = 25mm

Figure 17. Maximum Average Vertical Apparent Load as a Percentage of Total Overburden and the Soil Types and Construction Conditions for each Test Section

(Beloff, et al, 1979)

In the writers' opinion, as for Beloff's case, the hoop stresses of the liner entirely in organic silt (RC-1 and RC-2) would eventually reach approximately 100% of total overburden, while some arching effects would still exist in these single tunnel sections in dense sand and weathered serpentine (PK-1, PK-3, RC-3, and PK-2), as shown in Figure 17. However, since the hoop stresses are only about 5% of the combined maximum stresses, the arching effect, in this case, may not significantly influence the design for the dimensions of the flexible tunnel liner.

2.4.2.2 Diameter Distortions

Based on the flexible liner performance of 18 case histories (Table 8), and discussion of the soil-liner interactions, Peck (1969) proposed a liner design method for resisting movement based on the assigned amount of diameter distortions. He found that even in soft or plastic soils, distortions of more than a few tenths of a percent of the diameter of a flexible liner are effectively prevented by the strength mobilized in the surrounding ground. Moreover, in such soils the rate of distortion decreases with time, as shown in Figure 18. Inasmuch as the distortion appears to increase roughly linearly with the logarithm of time, curves such as those in Figure 18 can be used to judge the maximum distortion to be anticipated during the lifetime of the tunnel. Although almost all the tunnels listed in Table 8 were constructed in plastic soils, the magnitudes of the diameter distortions and the rate of distortion with time would, in all likelihood, be smaller in dense or slightly cohesive sands. Thus, if a 1.0% diameter distortion is assigned, the designed liner, in general, would be on the safe side to resist induced movement for a single tunnel in soft ground.

As a continuation of previous efforts, Kuesel (1972) reported additional records of field performance of diameter distortions under a variety of soil types and construction conditions. The design criteria for the Bay Area Rapid Transit (BART) flexible tunnel liner (rings) included the requirement to resist a uniform radial pressure corresponding to the full overburden weight at the springline plus a 0.5 in. (0.25%) diameter distortion at normal working stress, or 1.0 in. (0.5%) diameter distortion at yield stresses. Because of the effects of the working of the joints, slipping of the bolts in their slightly oversized holes, and progressive development of yield hinges at the springlines and crown, a rational analysis was made which indicated that the actual distortion capacity of the ring at working stress is about 2.5 in. (1.25%) of the diameter, with 5.0-in. (2.5%) distortion at significant yield, and an ultimate capacity of 15 in. (7.5%) before collapse. These estimates are consistent with observations of distortions in steel ribs in rock tunnels. Although the actual distortions of the BART tunnel rings (Figure 19) have generally exceeded those stipulated as criteria for the elastic ring analysis, the distortions were generally within the rational working stress capacity of the rings. There has been no evidence of structural distress, cracking of protective coatings, or leaking developing after caulking of the joints, except the special situation encountered under the Ferry Building (Section 2.4.1, Case 2R). All of the tunnel liners tended to squat slightly until they were grouted. The rings produced under the BART design criteria have performed satisfactorily and at an economical cost.

Schmidt, et al (1976), summarized the field performance of the crown deflections and squat of tunnel rings in the Toronto Subway, Section E-1. Two tunnels, 26 ft apart, were constructed individually in dense silty sand by hand mining in a 17.5-ft-diameter shield. Each tunnel was lined with a 2-ft cast iron ring. The groundwater table was below the tunnel invert; the diameter distortions were monitored after the ring was shoved out of the tail of the shield. The observed data are presented in Figure 20. The average

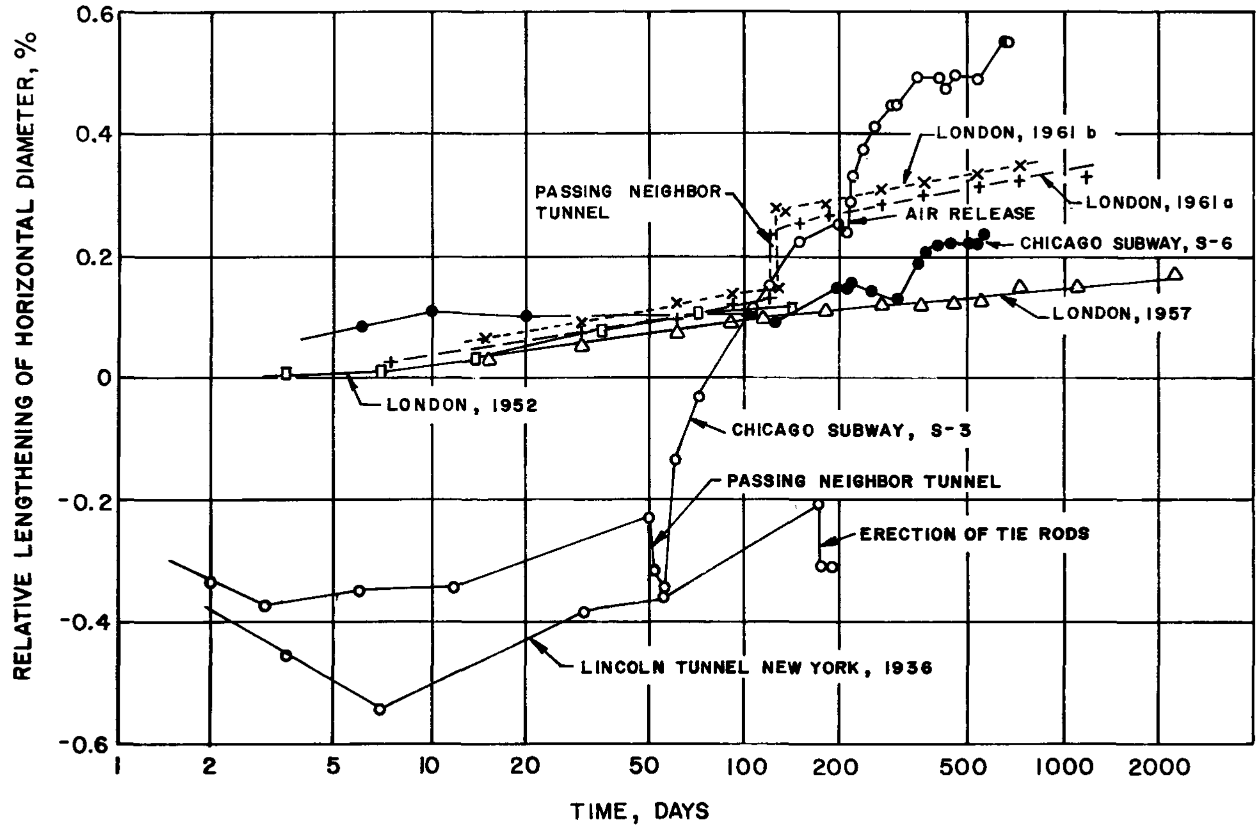
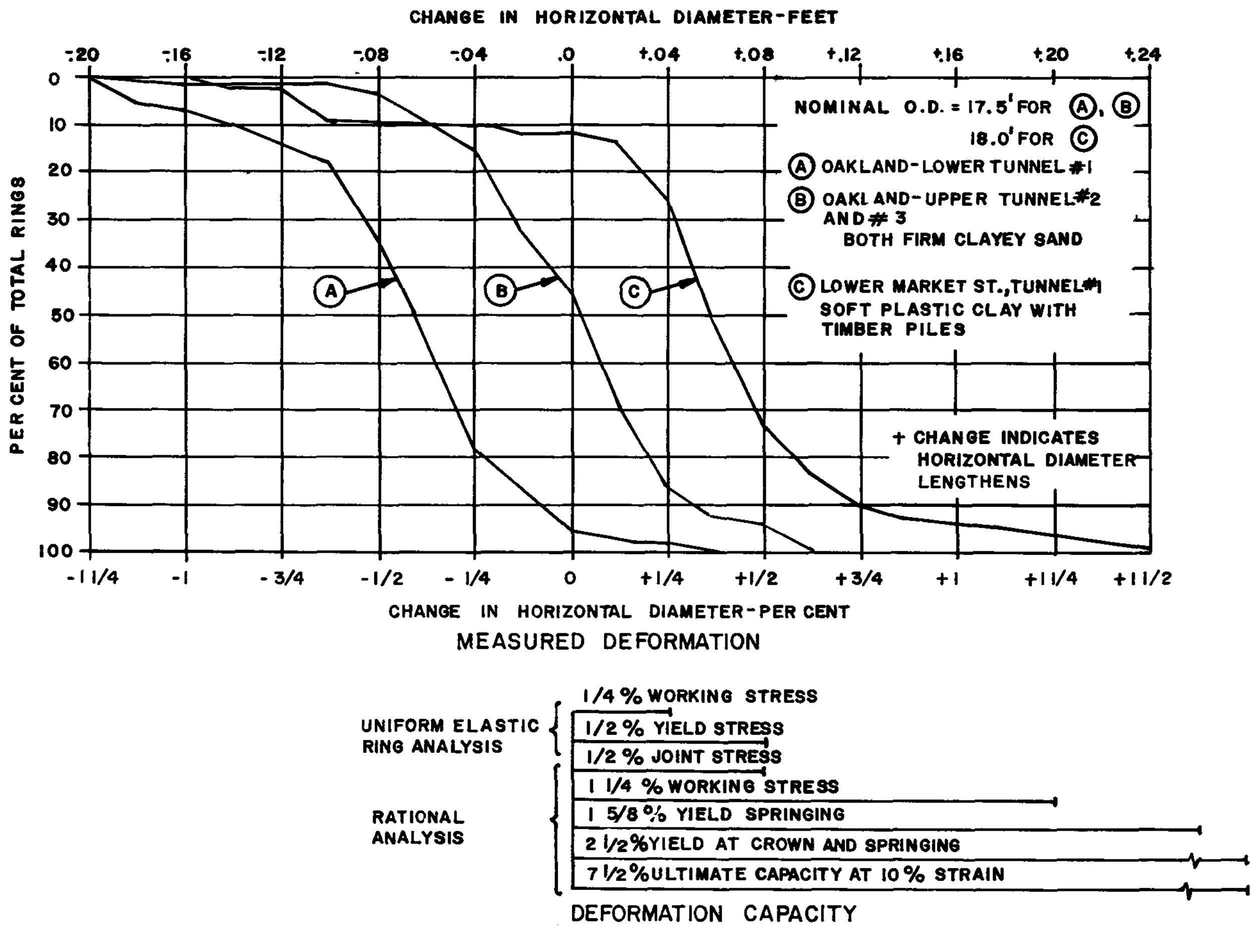
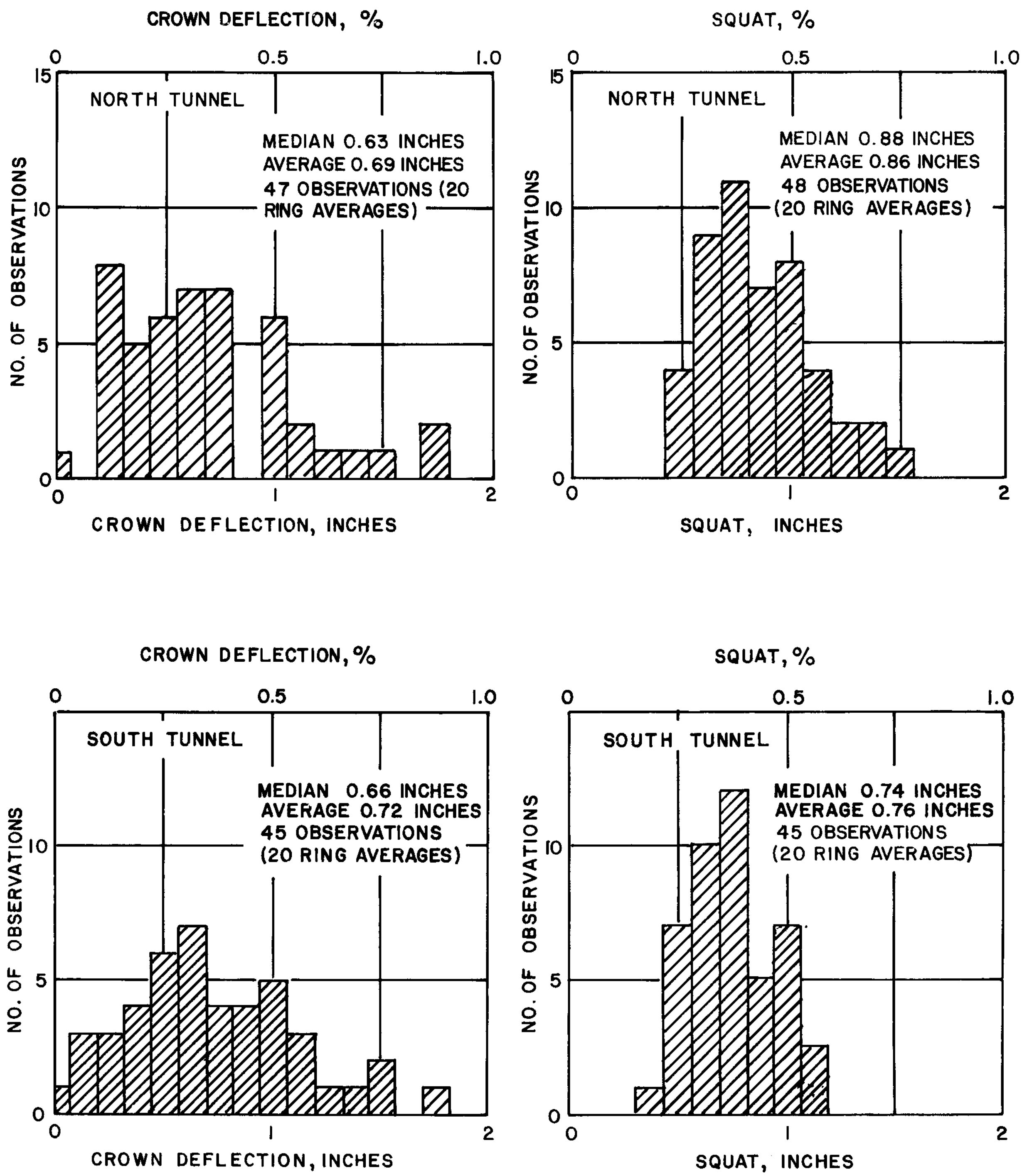


Figure 18. Percentage of Horizontal Diameter Variation for Various Tunnels in Soft Ground as a Function of Time (Peck, 1969)



NOTE: 1ft. = 0.3m

Figure 19. Flexible Tunnel Ring Distortions in BART System (Kuesel, 1972)



NOTE: 1 in. = 25 mm

Figure 20. Flexible Tunnel Ring Distortions in Soft Ground; Toronto E-1 (Schmidt, et al, 1976)

amount of distortion was about 0.4% of the tunnel diameter, with a maximum of 0.7%. The squat of the north tunnel (which was driven first) is slightly greater than that of the south tunnel, possibly because of the influence of the south tunnel construction on the north tunnel. Crown deflections are a little more irregular than the squat deformation, probably because of the irregularities in the way the tunnel rings settle toward the bottom of the tail void space.

2.5 SUMMARY AND CONCLUSION

Stability and stand-up time are the main considerations in connection with the feasibility of tunneling. They not only dictate the soil modification and tunnel construction techniques to be used, but also influence the amount of soil deformations surrounding tunnels.

In dealing with the stability problem, categorizing the soft ground according to the type of soil is, in general, the first step. For coherent media, the overload factor, $(p_z - p_a)/c$, is the governing factor for determining the tunnel face stability. If the overload factor does not exceed about 5, tunneling may be carried out without unusual difficulties in clayey materials. For noncoherent media, the stability analysis by overload factor is not applicable. Adequate dewatering is one of the effective techniques to eliminate seepage forces and to induce apparent cohesion (capillary tension) in this kind of material. Often drifting, forepoling, breasting, or a hooded shield are also employed for excavation in noncohesive media. Or, to provide a means for aiding in the support of the face, grouting, compressed air, or freezing may be used.

Based on the review of six case histories of catastrophic ground loss in tunneling, it is apparent that most disasters can be prevented if the soil type, strata configuration, in-situ stress state, and groundwater condition can be identified, and if the correct precautions in construction procedures are taken in advance. These procedures usually involve groundwater control or ground stabilization.

Ground stability is the basic factor governing the safety of the tunnel. The criteria for selecting the tunnel excavation procedure, ground support, and materials handling procedures are determined by the stand-up time of the ground. The stand-up time is basically dependent on the type of soil, groundwater conditions, and the size of the opening (as shown in Table 3 and Figure 7). The stand-up time also may be influenced by the in-situ state of stress, method of excavation and support, and rate of tunnel advance. Based on the case histories reviewed, the stand-up time can be improved (increased) by increasing the excavation rate, increasing compressed air pressure, or reducing the size of the excavation.

Soil movement around soft ground tunnels is one of the most critical problems related to tunneling in urban areas. To evaluate the ground loss and the amount of settlement, ground is categorized into four groups according to soil type. They are: (a) Granular soils without cohesion, (b) cohesive granular soils, (c) nonswelling stiff-to-hard clays, and (d) stiff-to-soft saturated clays. Through study and analysis of an additional 19 case histories, Cording, et al (1975) and MacPherson, et al (1978), indicated that the normal probability curve for representing the shape of the settlement trough is valid for most of the cases studied (as presented in Figure 10). They introduce the angle of draw (Figure 22), which may help the interpretation of the observed data. For tunnel depth in the range of $Z / 2R \leq 4$, the limits for rock, hard clay, and sand above the water table correspond to $\beta = 11^{\circ}$ to 33°. In the same depth range, the limits for soft clay correspond to $\beta = 33^{\circ}$ to 50°. Values of β were greater than 50° in soft Mexico City clays where large volume decreases developed due to consolidation of clay outside the tunnel springline. Cording, et al (1975) and MacPherson, et al (1978), also divided the settlements and volume losses into four stages: (a) Ahead of the face; (b) over the shield; (c) during erection of the lining; and (d) with time. Based on the measured subsurface settlement data, they related the ground movement to observed construction and soil conditions. Further, an estimation procedure for the settlements was developed. From this basis, the decisions about the need for and extent of underpinning required to protect buildings and utilities can be made. Also, the desirable construction practices to control and limit ground losses were proposed.

With regard to building damage, the soil properties and construction workmanship; the location, orientation, and size of the structure; the stiffness of the foundation and superstructure relative to the ground; as well as the magnitude of ground movements are the main parameters related to structural damage. Angular distortion and lateral strain are the two direct causes of building damage.

In accordance with the case histories reviewed, some of the unexpected tunnel construction problems are boulder problems, compressed air leaking problems, and man-made obstruction problems. An adequately performed site investigation program and a properly planned construction procedure may reduce their potential detrimental effects.

Based on the additional liner performance data reviewed, no modification of the semi-empirical design procedure for flexible tunnel liners (summarized in Volume 3, [Hampton, et al, 1980] of this series of reports) is needed at the present time. Although some arching effects have been observed in the ring loads of liners in predominantly granular soils, additional field-measured ring load data in this kind of soil are required before any modification can be considered. It should be noted also that fluctuations in groundwater table or ground motion associated with seismic activity of pile driving can radically affect the loading condition by modifying the ground arch.

Based on the case histories reviewed, performance of the soil is mainly a function of soil type, groundwater condition, size of opening, and the construction procedure. The latter two factors can be adjusted, accordingly, if the former factors can be identified. Therefore, an adequate site investigation is a primary step toward reducing the tunneling cost.

3.0 SOME EFFECTIVE CONSTRUCTION PROCEDURES FOR PROBLEMED TUNNELS IN SOFT GROUND

3.1 INTRODUCTION

As reviewed in Chapter 2, the tunnel face stability, surface and subsurface settlement, and excessive groundwater during construction are the main problems related to tunneling in soft ground. In recent years, quite a few new ground modification techniques have been utilized to overcome these problems. For example, chemical grouting and ground freezing can effectively improve the stability of a tunnel face; compaction grouting can reduce the settlement due to tunneling; predrainage and newly developed tunneling machines, e.g., slurry face and earth pressure balance machines, can sufficiently eliminate the construction problems during tunneling. However, most of these techniques are relatively expensive in comparison to conventional tunneling methods, and they are usually only cost-effective in certain combinations of soil types, substrata configurations, groundwater conditions, and construction constraints.

In this chapter, some representative case histories of effective ground modification techniques are reviewed. In each individual case, the subsurface conditions and construction constraints are summarized, and the tunnel performance after treatment is outlined. Finally, the advantages and limitations of each technique are discussed.

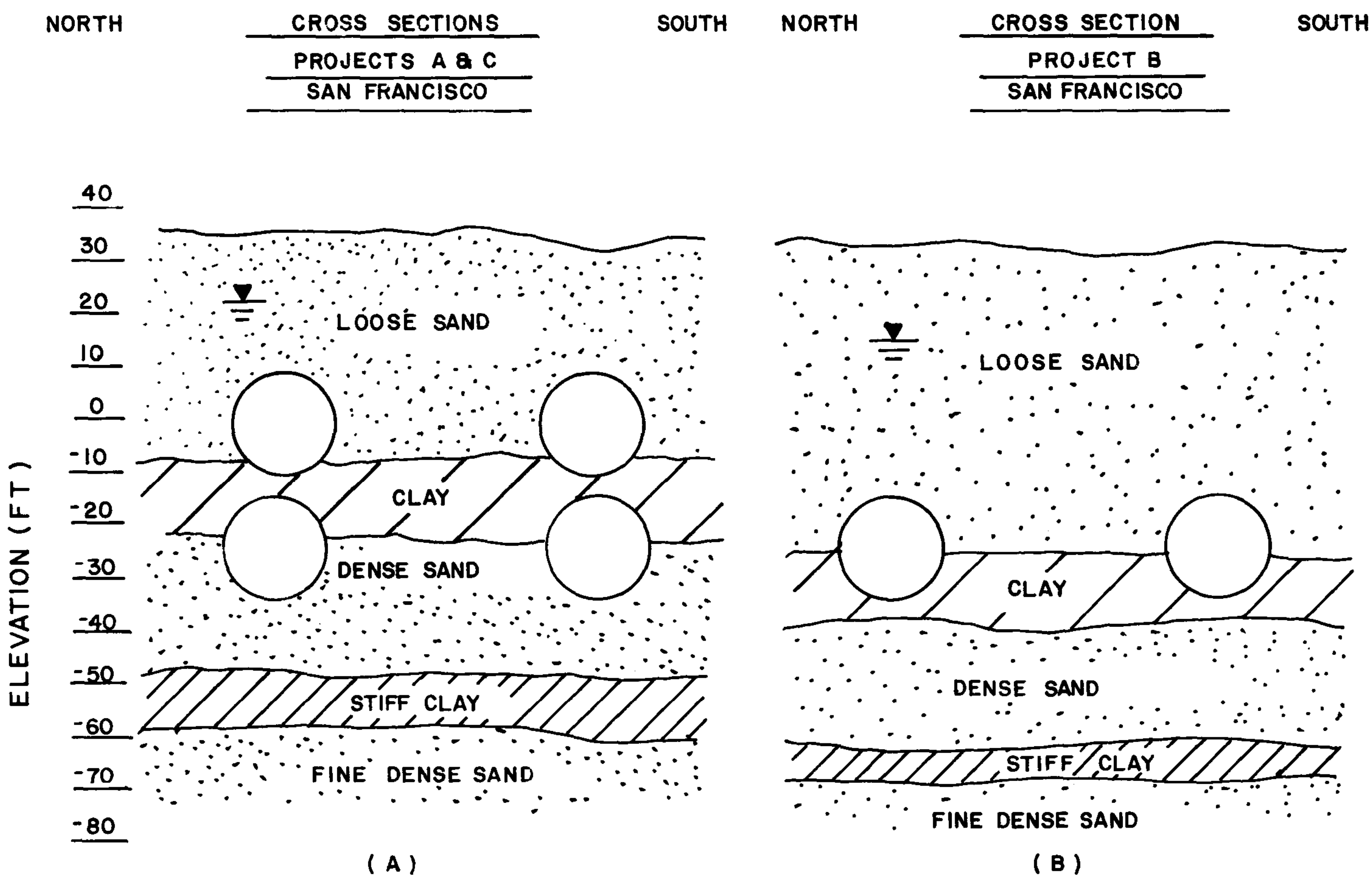
3.2 PREDRAINAGE

In tunneling through water-bearing formations, the expense of controlling the water can have a major impact on overall construction cost. In this section, the predrainage stabilization techniques, i.e., lowering of the groundwater table in advance of tunnel excavation, are discussed. Both free air and compressed air tunneling cases are described.

The dewatering systems must be installed well in advance since construction of wells and pumping take time. If the tunnel heading reaches a critical area which has not been dewatered, costly construction-related problems may result. Thus, a thorough site investigation, including an analysis of groundwater hydrology, is essential to avoid such problems.

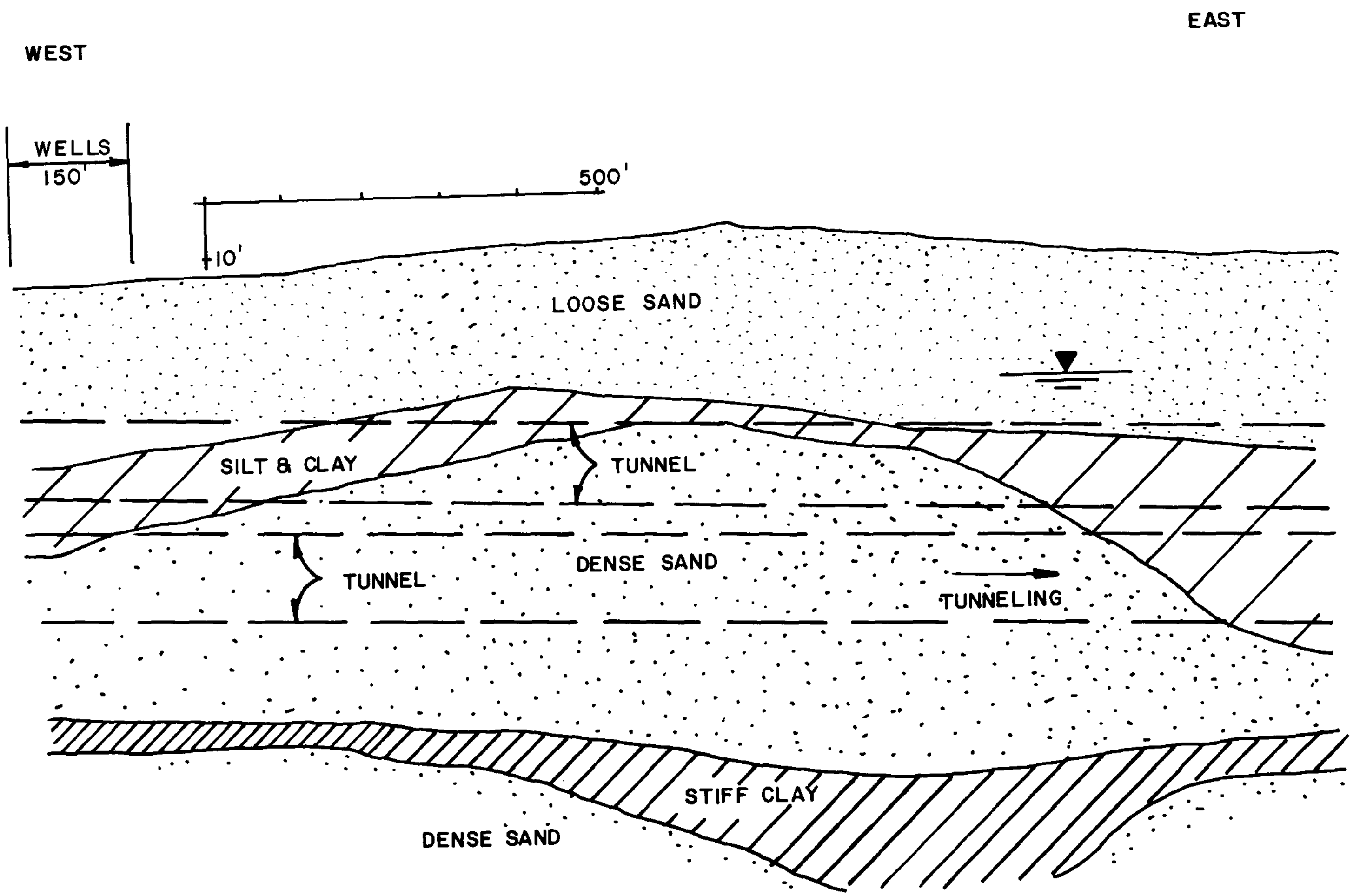
Except in simple aquifer situations, the cost of complete dewatering is quite high. Most of the aquifers to be dewatered for tunneling are systems of strata, channels, and pockets with variable interconnections. The geologic mechanism by which most soils are laid down creates stratification so that vertical permeability is generally much less than the horizontal. In the usual case, sufficient predrainage equipment is installed to control major water flows, and a manageable volume of water is anticipated in the tunnel itself.

There are a number of aquifer characteristics that potentially can have an impact on the dewatering problem which must be carefully evaluated in the course of geological investigation. These include: Type of aquifer; thickness of the aquifer; storage of water in the aquifer; and recharge to the aquifer. On the other hand, the volume of groundwater that can be left safely and economically at the tunnel face depends on the soil



NOTE: 1 ft. = 0.3 m

Figure 21. Soil Profile for BART System, Project A, B and C (Powers, 1972)



NOTE: 1ft. = 0.3 m

Figure 22. Soil Profile Along Tunnel Line, BART System, Project A (Powers, 1972)

characteristics. A dense well-graded sandy deposit with particle sizes ranging from cobble to 20 percent silt has relatively low permeability, and a low rate of seepage results. Also, the well-graded soil has inherently stable characteristics, i.e., as the fines wash out, the coarse sands and gravels form a filter which retards further movement of fines. A loose, uniform, aeolian deposit, such as dune sand which is relatively high in permeability and groundwater flow, can cause continuous movement of soil and dangerous face instability.

Whenever there are indications of a significant groundwater problem, a pumping test (the most reliable means of evaluating aquifer characteristics) should be considered. However, the pump tests should not be designed and performed until the bulk of the geologic investigation and the test boring information are available. The results, including the interpretation of the pumping test, should be made a part of the geotechnical report furnished to bidders.

The basic tools for the predrainage technique are wellpoint systems, deep wells, and ejector systems. The wellpoint system is commonly used for open cut excavations. However, because of its suction lift limitation, it is restricted to tunnels with very shallow cover (less than 20 ft). Deep wells are the most widely used tool for tunnel predrainage. However, the unit cost per well tends to be high since each has its individual pump, power supply, and discharge connection. Ejector systems have been successful in dewatering some very difficult situations, particularly in silts and fine sands that have low flows and high seepage pressures. In these soils, little or no water can be accepted safely at the tunnel face. Individual ejectors have a much lower unit cost than deep wells and close spacing is more economical. Ejectors are capable of producing a vacuum in the surrounding soil, and can more effectively dewater the fine-grained soils. Ejector systems have some limitations, too. Power costs of the system are relatively high. If the groundwater is hard, or contains iron, maintenance of the system may be difficult. Thus, a careful review of the water quality in the aquifer is needed before selection of the ejector method.

Powers (1972) presented a few representative tunnel predrainage case histories to illustrate the use of deep wells and ejector systems. These cases emphasize the dramatic effectiveness of the predrainage system adjustments made in accordance with the configuration of soil layers, the characteristics of the soil, and the proximity of recharge sources. Summaries of these cases are presented in the following paragraphs.

Case 3A - BART System, Project A

Four tunnels were to be driven about 1600 ft between two stations (Figures 21A and 22). Conventional shields were planned. The stations at either end were being predrained, and the pumping proceeded for many months before tunneling began. It was decided to place five deep wells approximately 50 ft on center at the west end to dewater.

As shown in Figure 22, the lower tunnels were driven through a dense fine sand. This sand layer was stratified in clean silty lenses interconnected since there was very little recharge to the dense sand layer beneath the clay, and during the months of pumping on the stations and five deep wells, the groundwater level in the lower sand layer continued a slow but steady decline. Both lower tunnels were mined through with a minimum of difficulty. Because the groundwater level was higher than desirable at the mid-length of the project, the groundwater level was lowered below the tunnel crown by installing and pumping from several additional deep wells in this area.

The soil condition for the upper tunnels was quite different (Figure 22). The crown of the tunnels was in a loose, cohesionless, uniform dune sand. The groundwater in this sand layer was perched on top of the clay layer, and could not drain to the lower sand layer. Also, there was a source of recharge on the north side of the west end of the project. When the north upper tunnel pushed off, water appeared in the face. The flow was moderate (less than 10 ft of water head), but sufficient to cause great difficulty with stability of the loose dune sand above the clay. Some loss of ground was experienced. All four tunnels in this project were driven in free air.

Case 3B - BART System, Project B

Two tunnels were to be driven with a wheel type mole through a loose, cohesionless, uniform dune sand. The cross-section and simplified profile are presented in Figures 21B and 23, respectively. Compressed air was planned, with deep wells to lower the air pressure to below 14 psi. The wells were spaced on fairly wide centers and would eventually prove adequate when the air was turned on. However, the tunneling machine required a considerable distance behind it for locks before air could be employed. Though near the tunnel driving shaft, which was lined with soldier piles and wood lagging, a portion of the perched water above the clay layer had drained. Before the machine had moved far enough to accommodate the locks, the perched water in the loose sand layer caused substantial loss of ground. Additional deep wells were drilled on 5- to 10-ft centers on both sides of the tunnel alignment for a sufficient distance to set air locks without significant loss of ground.

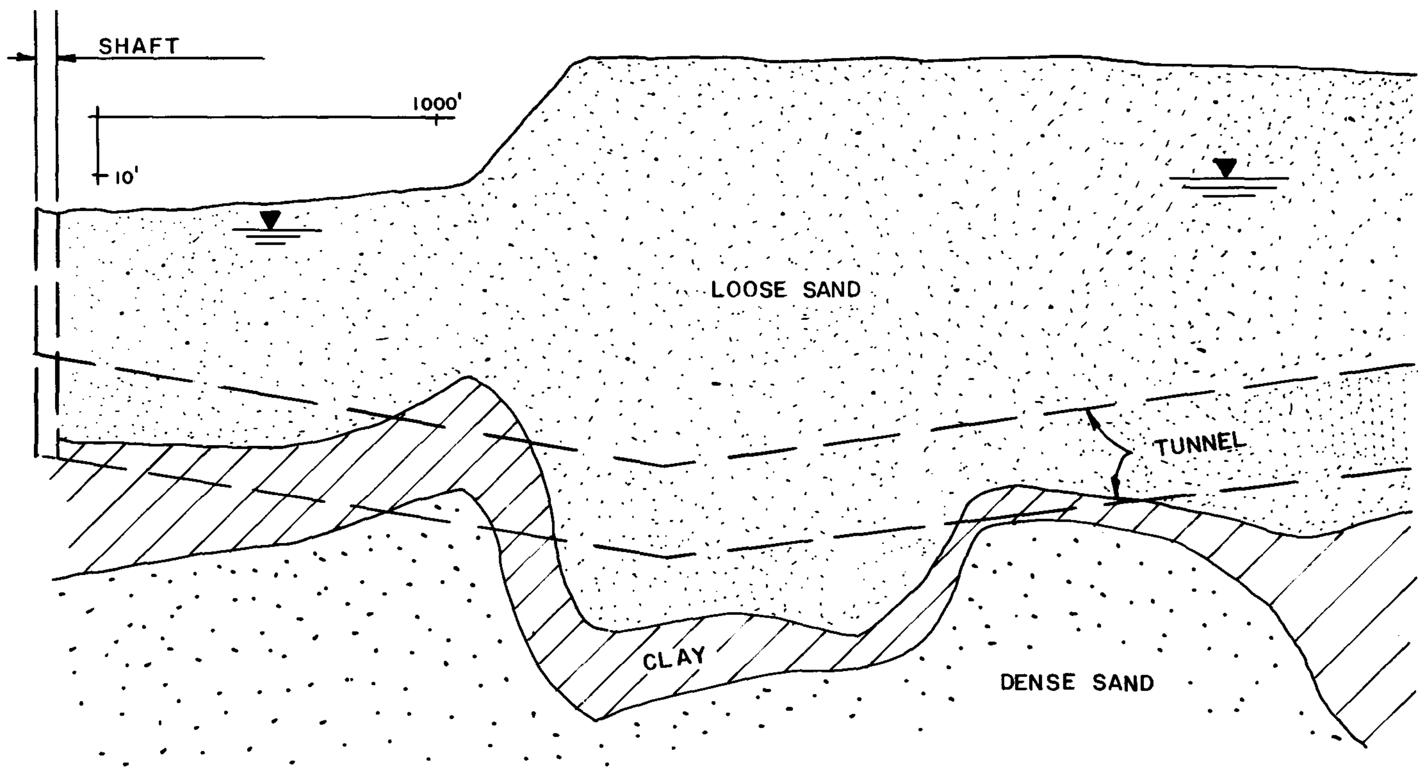
Case 3C - BART System, Project C

Four tunnels were to be driven 1900 ft with a wheel type mole between two stations. The cross-section and simplified profile are shown in Figure 21B and Figure 24, respectively. Compressed air was specified since a peat deposit existed at the west end of the project. Lowering of the groundwater table was prohibited in this area due to possible settlement. The west side station was constructed inside a slurry pile, tremie concrete wall, with interior dewatering and exterior artificial recharge operation to keep the original groundwater table outside the wall.

Tunneling was planned from a shaft at the east end. A distance of 150 ft out was required for air locks and associated equipment. Two deep wells and 26 ejectors on 12.5-ft centers was the system designed to predrain the east end shaft 150 ft out. For the next 1100 ft, deep wells were placed on 100-ft centers to reduce the required air pressure below 14 psi. The closely spaced ejectors successfully dewatered the first 150 ft of tunnel length, and the deep wells were effective in reducing the air pressure for the next 1100 ft of tunnel length.

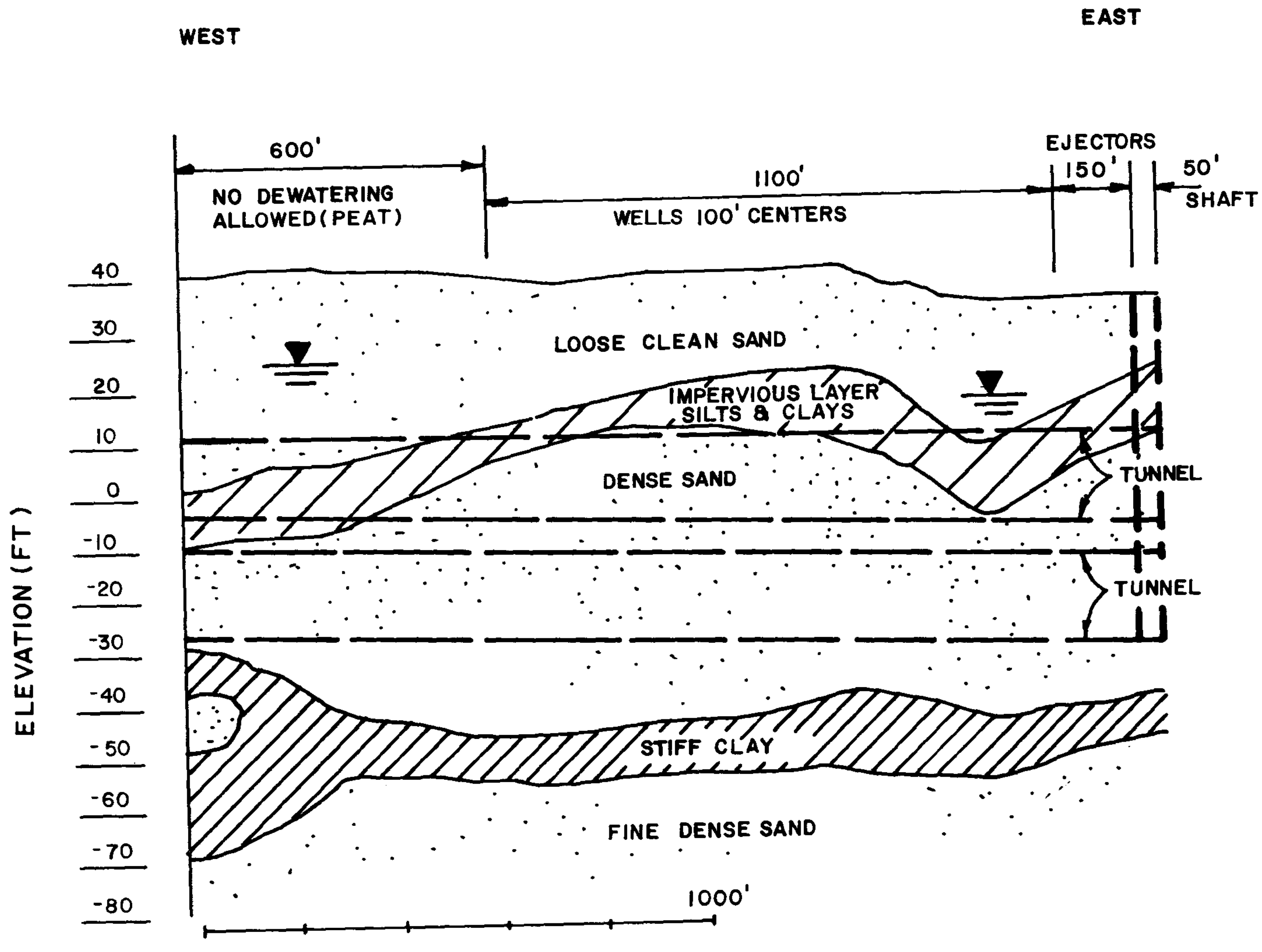
Case 3D - Sewer Tunnel, New York City

The simplified profile of this mixed face tunnel is shown in Figure 25. Material above the rock was sand, gravel, and boulders of varying permeability. The rock was Manhattan schist. Because of the shallow cover, air at sufficient pressure to support the face caused blows to the surface. Because of suspected compressible soils in the area, the owner had been reluctant to authorize dewatering. Settlement might endanger existing structures, including an elevated highway, a railroad, and industrial and commercial buildings.



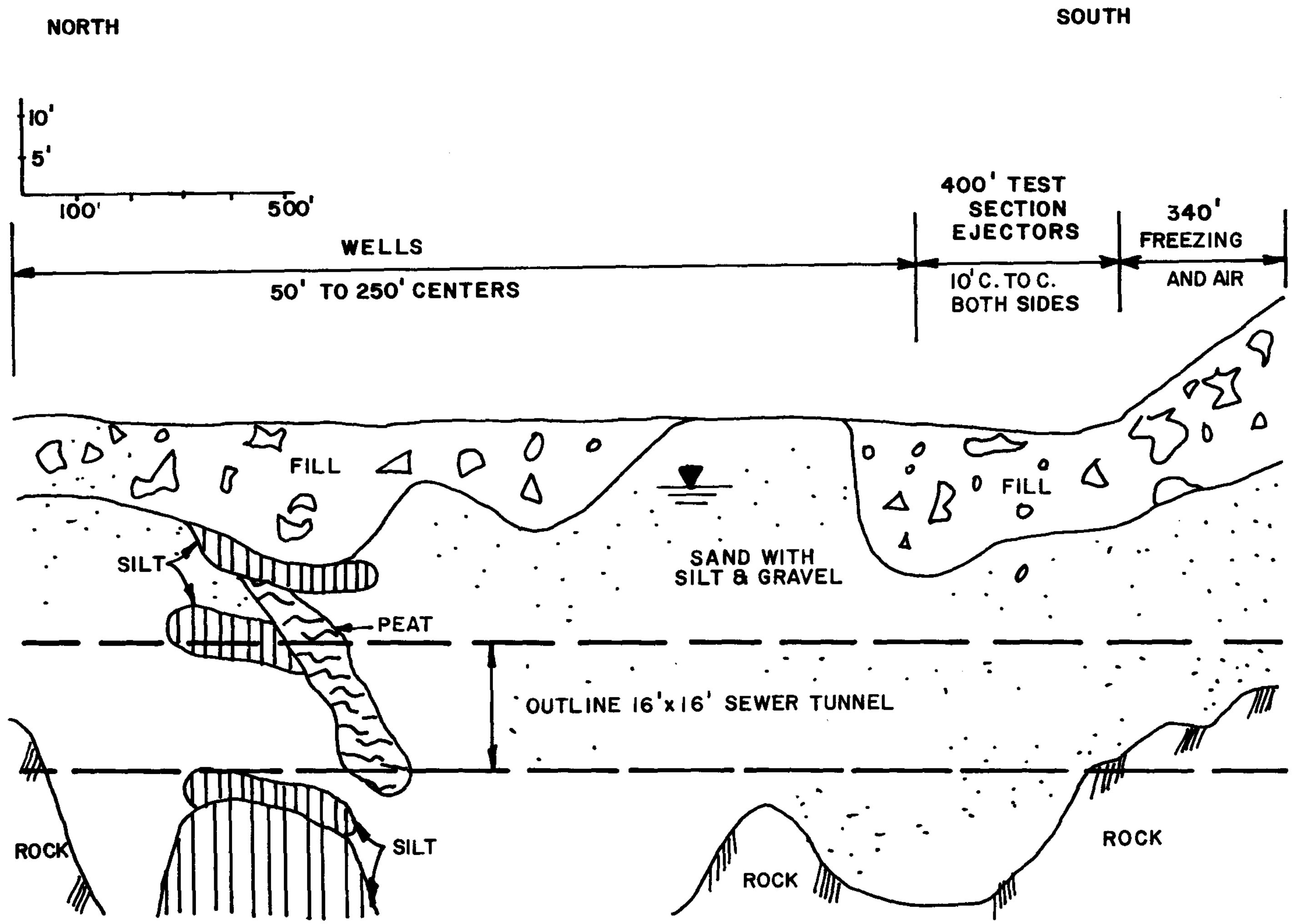
NOTE: 1ft = 0.3m

Figure 23. Soil Profile Along Tunnel Line, BART System, Project B (Powers, 1972)



NOTE: 1 ft = 0.3 m

Figure 24. Soil Profile Along Tunnel Line, BART System, Project C (Powers, 1972)



NOTE: 1ft = 0.3m

Figure 25. Soil Profile for Sewer Tunnel, New York City (Powers, 1972)

A freezing operation was begun, and about 340 ft of tunnel were driven in the south heading with the aid of both freezing and compressed air. The work proceeded slowly and at considerable expense. In view of the difficulty experienced with this method, it was decided to predrain a 400-ft test section to determine the feasibility of the dewatering method and to observe any effect on surrounding structures.

Designed was a system of ejectors on 10-ft centers on both sides of the tunnel. This system succeeded in lowering the groundwater table. The tunnel was driven under low air pressure at an advance rate of up to 20 ft per day. No significant settlement was observed. An additional section was approved for dewatering. When the rock dipped below the tunnel invert, the predrainage tool was switched to deep wells beginning on 50-ft centers. Progress continued to be good. The well spacing was gradually increased as soil conditions improved, and was as far as 250 ft apart where the tunnel was mined through.

Case 3E - Subway Tunnel, Osaka, Japan

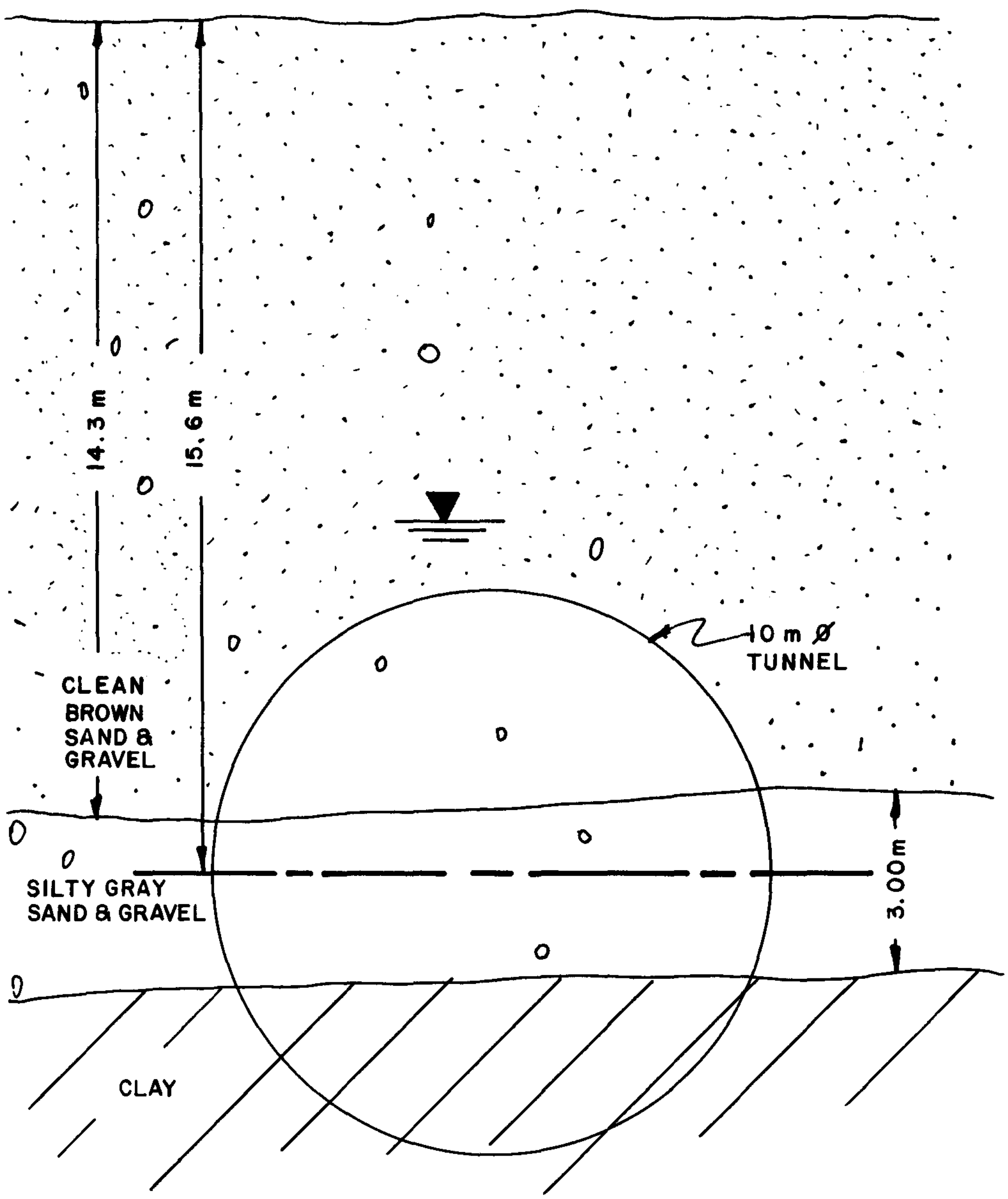
A 10-meter (32.8 ft) diameter, two-track subway tunnel was being driven by a wheel type mole. As shown in Figure 26, the bottom one-third of the tunnel was in firm clay. Above the clay were approximately 3 meters of sand and gravel with about 20 percent silt. Above this sand and gravel layer was a clean sand and gravel layer containing less than 5 percent fines.

A system of widely spaced deep wells was provided for dewatering the initial 70 meters for the installation of air locks. Due to difficult soil conditions, the wells were only partially effective. The saturated silty sand, with close recharge (groundwater from the clean sand layer above), was extremely unstable. Severe loss of ground occurred and tunneling was stopped. A system of ejectors on 3-meter (9.84 ft) centers on both sides of the tunnel was designed. This system pumped 200 gpm, and was successful in reducing the groundwater level to a few centimeters above the silty sand layer. Inflow at the tunnel face decreased from 20 gpm to 2 gpm. Although the silty sand layer remained saturated, with the source of recharge eliminated the tunnel face became quite stable. Subsequent tunneling under compressed air proceeded with a minimum of difficulty.

3.3 CHEMICAL GROUTING

Preinjection of chemical grouts into permeable soils around the tunnel perimeter is an effective technique for stabilizing soils for tunnel construction. In this technique, the grout penetrates the soil in a fluid form but hardens, or gels up, in about one hour due to a chemical reaction. As a result, the soil is strengthened and made less pervious, thus improving the stand-up time, limiting groundwater inflow, and controlling movements of the heading during tunneling.

The ideal condition for use of chemical stabilization is in medium-to-coarse sand with uniform gradation. Under such conditions, grout can be injected freely so as to form a continuous zone of stabilized soil surrounding the axis of a future tunnel opening. The opening of a tunnel in the stabilized zone creates stresses in the material around the tunnel. If the stabilized zone is stiff and strong relative to the surrounding untreated soil, the stress changes created by the tunnel excavation would be concentrated in the stabilized zone. In other words, a structurally competent stabilized soil zone around a tunnel opening would act as a compression ring and prevent stress changes from being felt in the soil beyond it.



NOTE: 1m = 3.3 ft

Figure 26. Soil Profile for Subway Tunnel, Osaka, Japan (Powers, 1972)

In many cases, the soils which can be treated by a grouting technique (gravels, sands, silty sands) are sandwiched by layers of ungroutable soils such as silts and clays. Under such conditions, a continuous stabilized ring of soil around the tunnel cannot be formed. Thus, the effectiveness of the grouting treatment is dependent on a number of factors, e.g., the strength of the ungroutable layers, their location, and thickness. If the ungroutable layer is stiff or hard, it has a positive influence on the performance of the stabilized zone. On the other hand, if it is soft or weak, it works to the detriment of tunnel performance.

Koenzen (1975) and Kasali (1978) reported that chemically stabilized soils will creep under constant load and yield time-dependent deformations. This observation is very important since, if unrestrained, creep in the stabilized zone around a tunnel will lead to excessive ground movements. Thus, the faster a liner system is installed to restrain the ground, the smaller the movements will be. In order to minimize the ground movements in the aforementioned situations, both chemical grouting and good construction procedures should be employed.

Although the chemical stabilization technique is expensive, in general it is used to control the ground movements where critical structures are located above soft ground tunnels. Historical and high rise buildings, shallow underground pipelines and tunnels, highway bridges and underpasses, railroad structures and tracks are examples of typical critical structures. The cost of chemical stabilization may be less than conventional structural underpinning, and other stabilization techniques may be impossible to apply.

Clough (1977) described the practice of chemical stabilization in England and Europe. Table 9 summarizes the significant information from those case histories. The primary type of chemical grout used in these countries employed sodium silicate as the base grout. There has been a wide variety of reactants (catalysts) mixed with the silicate to generate the gelling process. Typical products for this purpose are ethyl acetate, triacitine, sodium succinates, or formamide. Sodium silicate generally was used in relatively high percentages in the grout mix, ranging from 50 to 75 percent. Since the grout mix is more viscous at higher sodium silicate percentages, the 70 percent type solution can be used only in coarse sandy soils. The strength of the grouted soil and the cost of the grout mix are directly proportional to silicate content.

Where groundwater is a problem, the grouted zone must completely seal off the pervious soils around the tunnel. In cases where pervious soils surround the tunnels and the groundwater table is high, the grouted zone must completely encircle the tunnels. In nonhomogeneous soil profiles, the grouted zones in some European cases often varied in depth and along the tunnel route. The thickness of the grouting zone around the tunnel was selected to ensure water tightness and to minimize surface settlements above the tunnel. The average thickness appears to be 8 to 10 ft, according to European cases.

In a majority of the cases cited in Table 9, the strength of the grouted soils ranged from 70 to 420 psi. These strengths were probably from unconfined compression tests conducted at a relatively high loading rate. The actual field strength of the grouted soil may be much lower since the strength would be around 200 psi in a rapid test for the usual groutable soil injected with relatively high silicate concentration (60 percent). Thus, the strength values in Table 9 can be reached only under optimum grouting conditions.

Table 9. Case History Data, Grouted Tunnels (Clough, et al, 1977)

Location	Ref.	Size of Tunnel or Opening	Purpose of Treatment	Description of Grouted Area	Estimated Maximum Strength of Grouted Soil (Type of Treatment)
Paris	Janin, et al, (1970)	10 m diam.	Make sands impervious. Minimize movements. Reduce pressure of compressed air.	Surrounds tunnel-thickness, 4 m top bottom.	966 kN/m^2 (Combined treatment, clay cement, silica gel and resin grout).
Paris	Janin, et al, (1970)	27 m span	Same as 1. 3 m bottom.	Surrounds opening-thickness, 8 m top 3 m bottom.	(not given)
Hamburg	Haffen and Janin (1972)	2 tunnels 7 m diam. 5 m apart	Minimize movements.	Rectangular zone around upper half of tunnels, 7 m thick 10 m long.	1450 kN/m^2 (Silica gel grout).
Munich	Haffen and Janin (1972)	2 tunnels 7 m diam. 5 m apart	Same as 1.	Rectangular zone around upper half of tunnels, 2 m thick.	1970 kN/m^2 (Clay cement and Silica gel grout).
Frankfort	Hannef and Janin (1972)	7 m diam.	Minimize movements of overlying rail-road tracks 10 m long.	Rectangular zone around upper half of tunnels, 3 m thick 10 m long.	2890 kN/m^2 (Silica gel grout).
Vienna	Haffen and Janin (1972)	7 m diam.	Same as 1.	Rectangular zone around upper half of tunnels, 7 m thick.	1970 kN/m^2 (Silica gel grout).
Milan	Haffen and Janin (1972)	2 tunnels, 7 m diam. 10 m apart.	Minimize movements of overlying under-ground canal and expressway.	Rectangular zone around upper portion of tunnels, 7 m thick.	(not given)
London	Dunton, et al (1966)	4 m diam.	Same as 1.	Not described.	(not given)
London	Gartung and Kany (1975)	3 m diam.	Make sands impervious.	Arched zone around sandy bottom of tunnel.	480 kN/m^2 (Resin grout).
Seafield Colliery, England	Gartung and Kany (1975)	6 m wide	Stabilize running sandstone.	Arched zone around top of opening.	(not given) (AM-9 grout).
London, 1975	Pers. comm.	6.4 m diam.	Same as 1.	Arched zone around top half of tunnel, 2.5 m thick.	(not given) (Silica gel grout).
Paris, 1975	Pers. comm.	7 m diam.	Same as 1.	Circular zone surrounds tunnel, 2-5 m thick.	(not given) (Clay cement, silica gel grout).
Nurem-burg	Pers. comm.	5.6 m diam.	Minimize movements.	Trapezoidal arches 2.5 to 5 m thick.	1030 kN/m^3 (Silica gel grout).

NOTES: 1 m = 3.3 ft
1 kN/m^2 = 0.15 psi

The performance of grouted tunnels in Europe and England generally indicated very satisfactory behavior. However, some problem areas have been identified (Clough, et al, 1978) such as:

1. Settlements caused by careless drilling procedures for large numbers of grout holes.

2. Lack of grout penetration into silty zones or very dense cohesionless soil zones.

3. Creep of grouted zones under extended loading.

Generally, these problems have led to no more than minor difficulties.

In the last ten years, chemical grouting technology has been used for tunnel construction in the United States. Most of the applications have involved work for subway tunnel construction. The experience gained from these works is, in many instances, unique and can serve as a guide for the use of grouting technology in future soft ground tunnel projects.

Clough, et al (1979), documented five case histories whose pertinent data are summarized in Tables 10 and 11. Tunnel sizes and depths below the ground surface, as well as soil conditions, were similar in all cases. The tunnels were about 20 ft in diameter, and the tunnel crown was 15 to 30 ft below the ground surface. The soils basically consisted of alternating sand, clay, and silty sand strata. However, soil stratification details were different at each site and significantly affected the effectiveness of the chemical stabilization treatment. Also, the stratification often was not constant, with depths and thicknesses of layers varying along the tunnel route. Chemical injections could be made only in the sandy layers.

In all cases, the chemical stabilization techniques were utilized to control the settlement where critical structures were located above the tunnels. The stabilization treatments were similar in most of the applications and involved injecting sodium silicate solutions into the ground from the surface through a plastic pipe installed in a drill hole. The grouting solutions primarily were made up of 50 to 60 percent liquid sodium silicate, 4 to 10 percent organic reactant (catalyst), and the remainder water. A detailed description of each case has been reported by Clough, et al (1978).

Principal findings in these case histories can be summarized as follows:

1. No flowing or running ground occurred in any instance at the tunnel face where chemical stabilization was used. Running ground did occur in a number of cases in untreated but groutable soils near the grouted zone. This means chemical stabilization can effectively reduce the ground loss at the tunnel face.

2. Where the soils in the crown area of the tunnels basically were ungroutable (more than 20 percent clay and silts), and only the interbedded sand layers were treated, the surface settlements were at or below the lower limits of the settlements in ungrouted areas (Table 11).

Table 10. Case Histories Covered by Study (Clough, et al, 1979)

	Number of Project and Identification				
	I Treasury Yard Section A.2	II New Jersey Avenue Trunk Sewer Section F.1b	III Seventh Street Bridge Section F.2a	IV East Capitol Street Sewer Crossing Section G.2	V Conrail Crossing Section G.1
Date Completed	1972	1974	1976	1977	1977
Tunnel Description	Two tunnels, 6.5 m diam., 9 m below surface.	Two tunnels, 6.5 m diam., 10 m below surface.	Four tunnels, 5.3 m diam., 5-7 m below surface.	Two tunnels, 6.5 m diam., 10 m below surface.	Two tunnels, 6.5 m diam., 10 m below surface.
General Soil Conditions	Mixed sand and clay strata.	Mixed sand and clay strata.	Mixed sand and clay strata.	Mixed sand and clay strata.	Mixed sand and clay strata.
Stabilization	30 m of soil along axis of second tunnel line treated. Sodium silicate grout solution used.	Zone above tunnels along axis of overlying New Jersey Ave. Trunk Sewer treated. Sewer runs diagonally to tunnels. Sodium silicate grout solution used.	Sandy soils above upper two tunnels stabilized along tunnel axis for 70 m under 7th St. Bridge and I-95 Freeway. Sodium silicate grout solution used.	Two schemes: a. Zone above tunnels along axis of overlying box culvert treated. b. Face of tunnel treated in a number of locations. Sodium silicate solution used primarily.	Sandy soils around and above tunnels treated along tunnel axis for 80 m. Sodium silicate grout solution used.
Reasons for Treatment	Serious movement during first tunnel passage.	Prevention of damaging movements to sewer above tunnels.	Prevention of damaging movements to 7th St. Bridge and I-95 Freeway.	a. Prevention of damaging movement to box culvert. b. Control of face of tunnel in running ground.	Prevention of damaging movements to mainline railroad tracks above tunnels.
Available Information	Surface settlements.	Surface settlements.	Surface settlements.	Surface settlements & limited subsurface movement.	Subsurface and surface movements.

NOTE: 1 m = 3.3 ft

Table 11. Settlement Data for all Case Histories (Clough, et al, 1979)

Case History		Surface Settlements (mm)				Comments
		Range		Average		
		Ungrouted	Grouted	Ungrouted	Grouted	
I		50-150	25-50	75	35	Clay strata in crown area.
II		50-75	25-48	55	35	Clay and silt strata in tunnel cross-section. Grouting in sand is located above the tunnel.
III	A	20-130	10-25	40	15	Branch Route tunnels - tunnel crown in grouted sands.
III	B	Data n/a	10-40	Data n/a	20	Pentagon Route tunnel - tunnel crown in clay strata. Grouted sands above tunnel.
IV		50-220	30-55	140	40	Crown of tunnel in clay primarily in clay; face grouting only in most areas.
V		110-120	110-120	115	115	Crown of tunnel in clay.

NOTE: 1 mm = 0.04 in.

3. In the fifth case, settlements reached 4 inches; backpacking or tail grouting was not called for behind the primary liner plate, and this settlement took several weeks to accumulate in the grouted area. It appeared that the tunnel support system was not uniformly in contact with the soil and could not prevent creep of the grouted soils. Thus, the use of chemical grouting does not eliminate the need for good tunnel construction practices.

4. In case IIIA, the soil above the tunnel springline should be uniformly grouted since surface settlements were only 1 in. or less.

5. Where the soils in the crown area of the tunnel were ungroutable and the groundwater table was below tunnel invert, there was no significant difference in ground tunneling behavior whether large or small grouted zones were used in the groutable soil layers.

6. If a chemical grouting operation is employed for all groutable soils around the tunnel alignment, the predrainage operation can be eliminated in this section since the grouted soils usually have a low coefficient of permeability similar to clayey silt. However, the grouted zone may act as a dam to interfere with the effectiveness of the predrainage system for the chemically untreated tunnel sections.

7. The main shortcoming of chemical stabilization is its ineffectiveness for stabilizing ungroutable soils.

Based on the review of cases related to chemical stabilization for soft ground tunnels, it can be concluded that the effectiveness of chemical treatment is dependent largely on the soil configurations and soil types around tunnels. Thus, a sound subsurface investigation is a priority for the success of chemical grouting operations.

3.4 GROUND FREEZING

Although ground freezing has been used in the past to stabilize the tunnel face during construction, it was invariably used as a last resort measure where other conventional measures could not be applied. The more recent freezing applications, however, have become increasingly cost competitive with conventional stabilization techniques, and sometimes are included as part of the design concept.

The most commonly used freezing method is the Poetsch process. It consists of a brine coolant, typically calcium chloride and water. Freezepipes are placed in boreholes in the ground. The freezepipe consists of an outer pipe which is closed at the bottom end, and an inner pipe through which a refrigerant is pumped. The coolant returns through the annular space between the inner and outer tubes. As the chilled brine passes through the annular space, heat is extracted from the ground, causing the brine to rise in temperature. The brine is then returned back to the refrigeration plant. Typical brine temperatures are about $-30^{o}C$. The freezepipes are arranged in a linear array so that an impervious structural barrier will eventually be formed. There are basically three stages of freezing. In the first stage, the frozen soil forms in a radial direction from each freezepipe until closure of the wall occurs. In the second stage, the average temperature of the soil lowers and the frozen wall increases in thickness until the design thickness is reached. In the third stage, maintenance freezing is used to keep the frozen wall at the design thickness during the tunneling operation.

In the past five to ten years, there have been an increasing number of tunnel projects in Europe and Japan constructed with the aid of ground freezing techniques. Jones and Brown (1979) presented several representative ground freezing projects. A brief summary of these projects follows.

Case 3F - Aarburg, Switzerland

The 810-meter-long Born Tunnel, which passes through a large mountain near Aarburg, Switzerland, is part of a double-track rapid transit system. The slopes in the end portions of the tunnel were critical sections due to the potential for slope instability along the sides of the mountain. The freezing scheme was selected over the other alternatives, since freezing offered the shortest construction period and other construction methods did not provide adequate protection against slope instability.

Figure 27 illustrates the tunneling scheme that was used. The freezepipes were placed from the ground surface on spacings varying from 3 to 7 ft, depending on the location of the pipes relative to the tunnel. The frozen soil was limited in extent by the use of freezing pots. The resulting frozen soil arch varied in thickness from 7 to 10 ft which was sufficient for the required stand-up time of five to six weeks until the concrete internal liner could be completed.

The tunnel was excavated in two stages, starting with the upper roof portion. Gunite was applied to the frozen soil immediately after excavation to prevent thawing. Field measurements of the gunite lining of the tunnel roof showed deflections of approximately 0.2 in.

Case 3G - Tokyo, Japan

Four parallel tunnels, Lines No. 10 and 11, and two utility tunnels were constructed by ground freezing beneath the Nihonbashi River. Tunnel construction was complicated by the presence of an old concrete bridge across the river and piers supporting an expressway on either side of the tunnel (Figure 28).

Insulation elements were placed on the river bottom to prevent thawing of the frozen ground by the flowing river water. The insulation boards were L-shaped, 16 ft in height, half the river width in length, 10 ft in width, and 1.3 ft in thickness. The L-shaped boards formed a U-shaped configuration on the river bottom. The voids between the boards and the bridge abutments were filled with concrete, and the voids between the river bottom and the boards were grouted with cement mortar. Insulating material was packed on top of the boards.

The freezepipes were placed horizontally, at about 2.5-ft centers, from vertical shafts installed on either side of the river. The freezepipes were 80 ft long with a 7-ft overlap at the center. A design freezing temperature of $-5^{\circ}C$ was used for the frozen soil. To eliminate any risks during construction, the entire tunnel cross-section was frozen. To prevent damage due to frost expansion, heating water pipes were placed between the frozen soil and the expressway piers. H-beam cross-bracing was used to support the excavation for the tunnels.

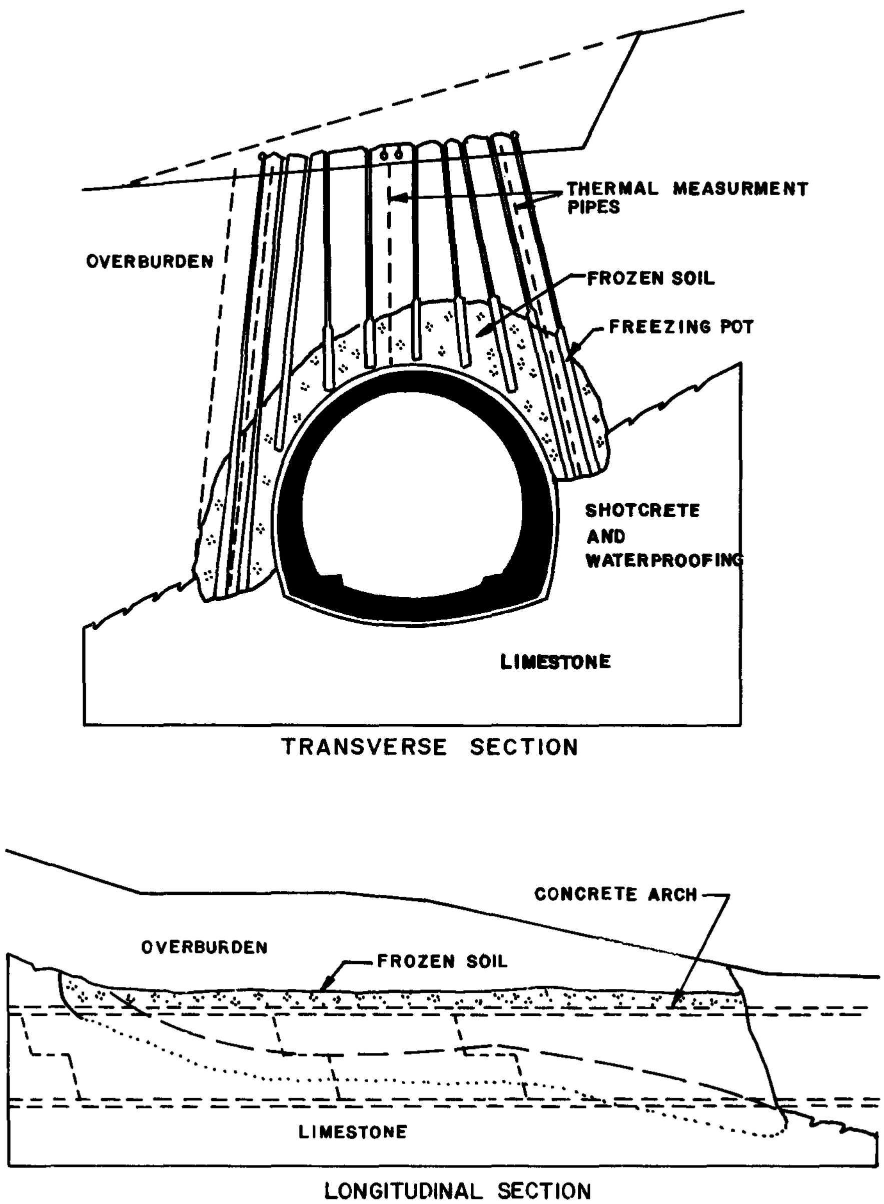
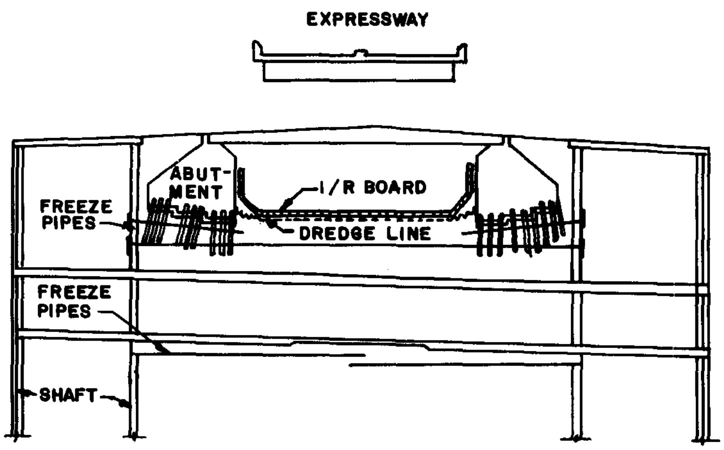
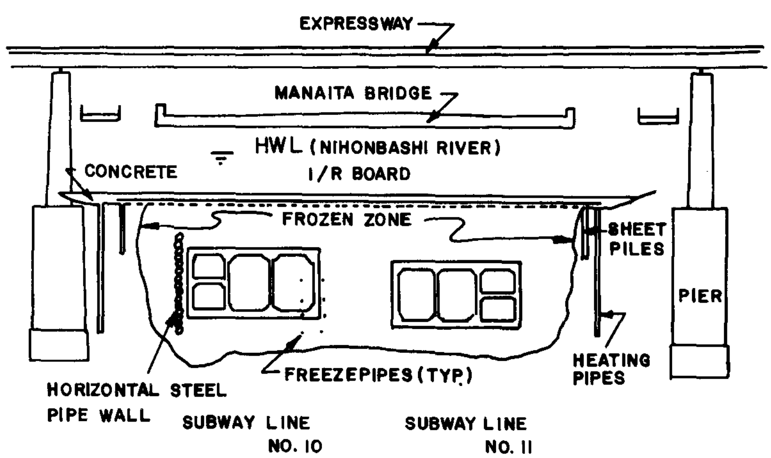


Figure 27. Ground Freezing Scheme for the Born Tunnel (Jones and Brown, 1979)



LONGITUDINAL SECTION



TRANSVERSE SECTION

Figure 28. Tokyo Transit Tunnel Constructed by Ground Freezing (Jones and Brown, 1979)

Case 3H - Helsinki, Finland

A section of the Helsinki Metro construction occurred in an 80-ft-deep tectonic trench located between competent rock. The trench was filled with glacial deposits of silty sand and rock fragments varying from pebble size to boulders. The groundwater table was located approximately 3 ft beneath the ground surface. The section could not be constructed by the cut-and-cover method because of surface disruption to heavily-traveled streets and potential consolidation settlements due to dewatering. To facilitate construction of this 20-ft-diameter tunnel, ground freezing was employed as shown in Figure 29. A total of 28 freezepipes was placed around the periphery of the tunnel, four of which were placed in the central portion to essentially freeze the entire space. As shown in Figure 29, the borehole for freezepipes was drilled from drilling chambers. The tunnel was excavated by drilling and blasting in short sections, then lined with cast iron liner plates.

Case 3I - Frankfurt, Germany

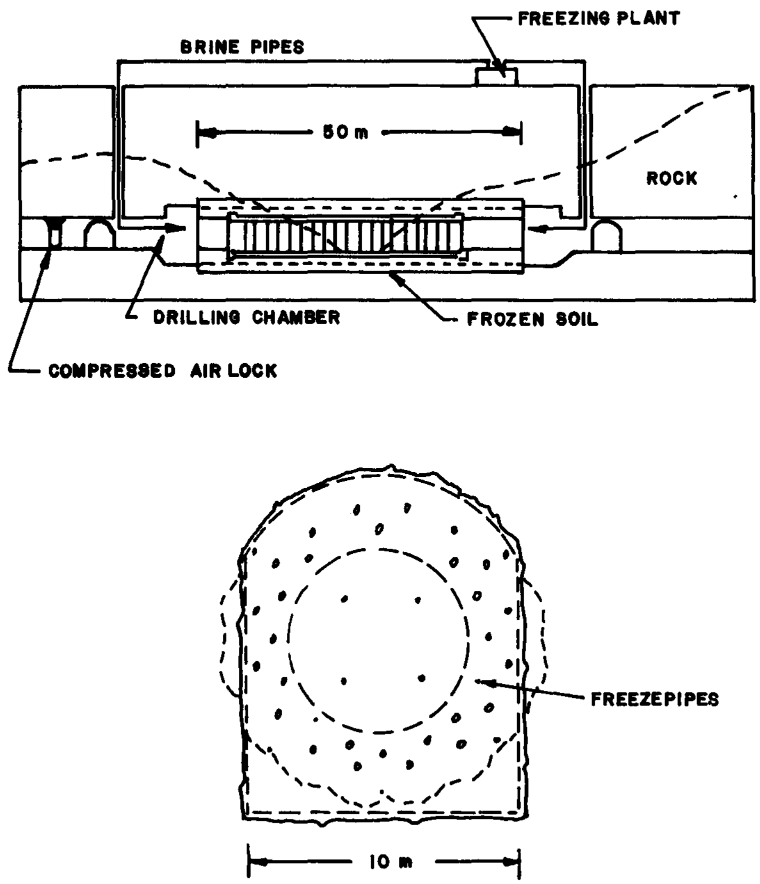
One of the difficult sections of the Frankfurt Subway System involved the construction of two 630-ft-long single tunnels beneath the Main River. Due to the presence of stiff plastic Frankfurt clay, the New Austrian Tunneling Method (NATM) was employed. The groundwater table, which was not connected to the river, was lowered by the use of deep wells adjacent to the tunnels. Ground freezing was used in conjunction with the NATM for two reasons: (1) Additional protection was desired to prevent damage to the existing bridge, and (2) Due to the shallow thickness of cover between the river and the tunnel, the possibility of ground running or flowing existed because of the presence of soil fissures and the old World War II bomb craters. Due to the length of the tunnel, the construction sequence, shown in Figure 30, had to be used. The freezing pipes were placed in 98-ft sections. In the last 28 ft of each excavated section, the upper portion of the tunnel was overexcavated, allowing room for the drilling equipment and additional freezepipe placement.

Case 3J - Washington Tunnel

As a part of the Anacostia River Force Main Project, a 110-ft-long, 12.5-ft-diameter tunnel was required to pass beneath four sets of heavily-traveled railroad tracks (CONRAIL) in Washington, D.C. An important aspect of this project was that the tunnel had only 9 ft of cover beneath the track.

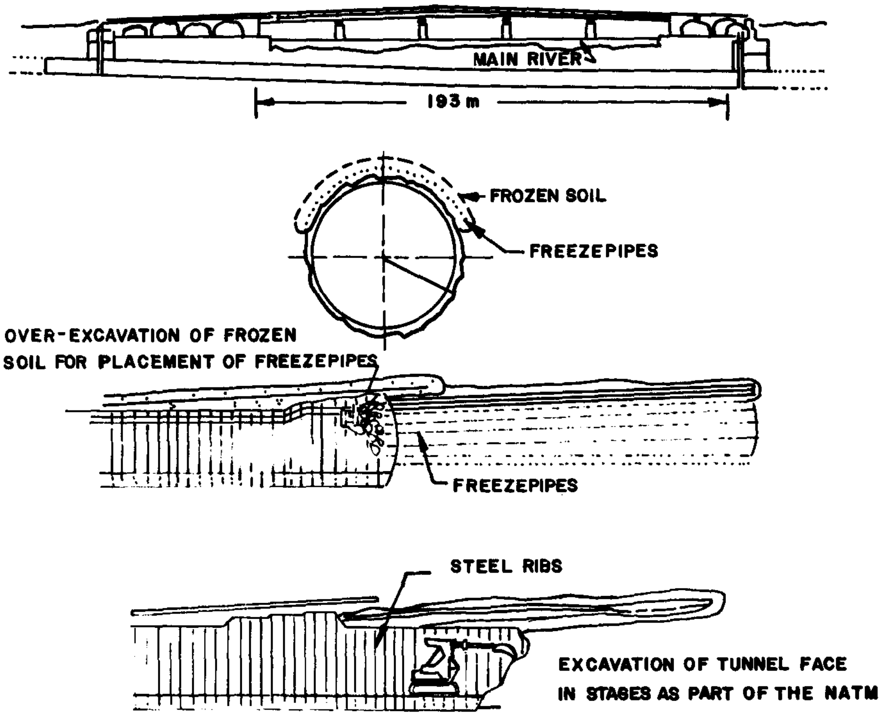
The subsurface conditions consisted of clayey sand, sand and gravel with varying amounts of clay and silt to an average depth of 25 ft. Below this depth, a thick deposit of very soft organic silt was present. The groundwater level was located approximately 5 ft below track level. Figure 31A presents a longitudinal section illustrating the subsurface conditions and heave measurement locations. Due to the presence of a high groundwater table and sand and gravel deposits overlying the soft silt deposit, it was determined that ground freezing would serve both purposes of providing an impermeable cut-off and temporary structural support for the tunneling operations. A steel liner was used for the permanent lining system.

Horizontally placed freezepipes, spaced approximately 3 ft apart around the entire periphery of the tunnel, were used to form the frozen structure. The design calculations indicated that a frozen zone approximately 3 ft thick would be sufficient for structural support. However, an average thickness of 5 ft was used because of the owner's safety precautions.



NOTE: 1 m = 3.3 ft

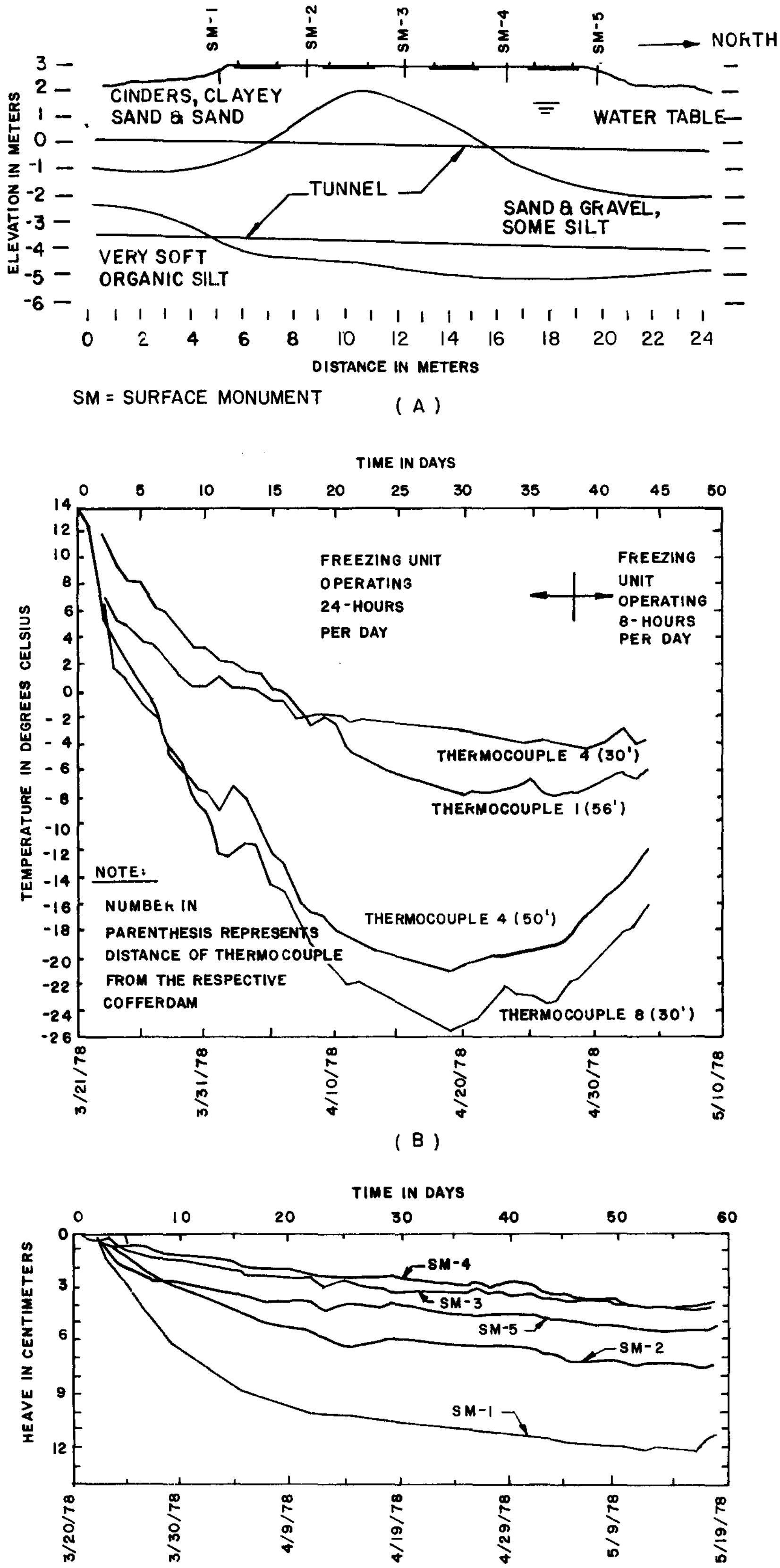
Figure 29. Ground Freezing System for the Helsinki Tunnel (Jones and Brown, 1979)



NOTE: 1m = 3.3 ft

Figure 30. Freezing System Combined with NATM for Construction of the Main River Tunnel

(Jones and Brown, 1979)



NOTES: 1cm = 0.4 in. (C)

1m = 3.3ft

Figure 31. Subsurface Conditions and Tunneling Performance for the Washington Tunnel (Jones and Brown, 1979)

Twelve thermocouples were used to record temperatures within the frozen zone. Figure 31B is a time history of ground temperatures at selected locations throughout the period of freezing. The readings indicated that an average temperature of less than -10°C (the design temperature) was attained in the planned frozen zone in three weeks. Mining operations were initiated one month after the freezing operation started. Following a one-week waiting period, the entire tunnel face was frozen--not just the 5-ft-thick annual ring as planned.

Heave measurements were obtained throughout the course of the freezing operation. Figure 31C is a record of the heave measurements. As expected, the major portion of the heave occurred during the phase change of porewater to ice in the soft silt materials. Also, the greatest amount of heave occurred at the south end of the tunnel, due to a greater thickness of soft silt being frozen at that end.

Based on the review of ground freezing cases, it can be concluded that the freezing technique is useful as a temporary support for soft ground tunneling, particularly in some difficult situations where other techniques cannot be utilized. Although the application of this technique has no restriction on the variation of soil types, the ground disturbance (heave) is directly related to the total thickness of the soft cohesive layers. Besides, the groundwater flow rates have a direct impact on the cost of the freezing operation. Thus, a thorough subsurface investigation is important in guaranteeing the success of this technique. Finally, since there were only a few tunneling projects involving freezing in the U.S., construction experience using this technique is very limited and associated performance data are rare. Therefore, more instrumentation programs should be initiated on tunnels advanced with the aid of freezing.

3.5 COMPACTION GROUTING

Compaction grouting was originally used for rehabilitation (jacking-up) of settled structures (Brown and Warner, 1973) similar to the mud jacking technique. Compaction grouting also can be utilized to stabilize vibrating foundations. In the last five years, compaction grouting methods have been used in association with the soft ground tunneling operations to reduce the near-surface settlement, and to protect the structures and utilities above the tunnels.

According to a cost effective analysis of various phases in future tunnel construction, some of the techniques are projected to save many millions of dollars. Table 12 gives examples of such cost savings. From this table, it is gleaned that compaction grouting has significant saving potential in future tunnel projects. Other significant techniques such as chemical grouting are described in the early part of this chapter. Subsurface exploration and precast concrete liners are discussed in Volume 2 and Volume 3, respectively, of this series of reports. Rock bolts and shotcrete uses are summarized in Chapter 5 of this volume. Slurry wall techniques are used mainly for cut-and-cover tunnels. Thus, they are not included in this series of reports. However, slurry face machine tunneling, the earth pressure balance shield method, and shield tail grouting are discussed in the next section.

Compaction grouting consists of the injection of a highly viscous non-penetrating grout into the soil mass for the purpose of developing sufficient pressure to fill the existing voids and to cause consolidation or densification of the surrounding soils. Compaction grouting is capable of modifying the strength of soft and loose soils. In the

Table 12. Examples of UMTA Program Applications and Cost Savings (U.S. Department of Transportation, 1980)

Major Accomplishments	Demonstration Cities	Actual Savings (in millions)	Projected Savings Over Next Decade (in millions)
Compaction Grouting Methods	Baltimore, MD	$10	$75
Chemical Grouting Methods	Washington	1	50
Subway Environmental Simulation Handbook	Washington, Chicago, Atlanta, Baltimore, New York	10	20
Rock Bolts and Shotcrete Ground Support Methods	Atlanta, New York, Washington	98	100
Subsurface Exploration Methods	Boston		50
Precast Concrete Liners	Baltimore, MD		200
Slurry Walls	Boston		150
Muck Utilization	Washington		25

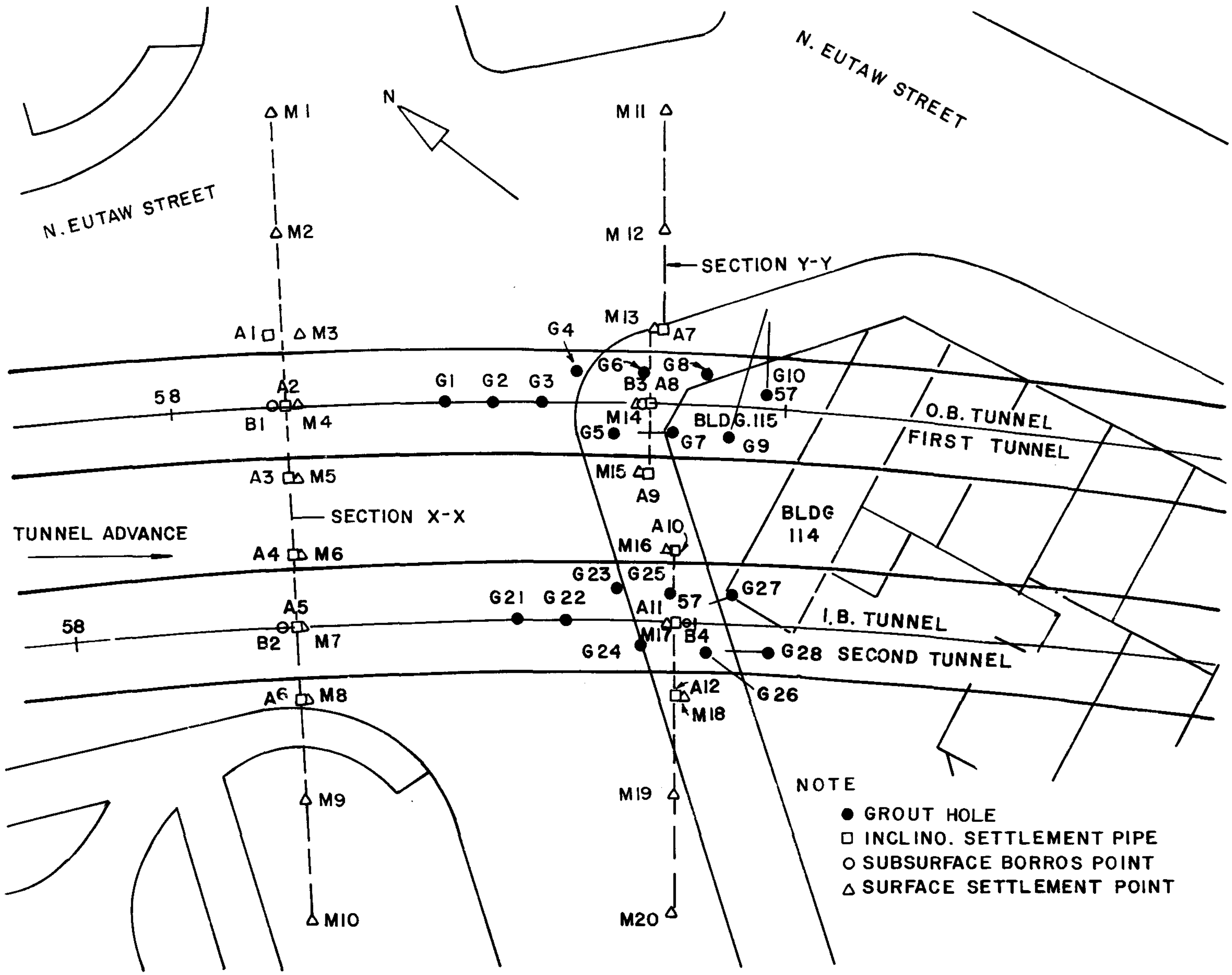
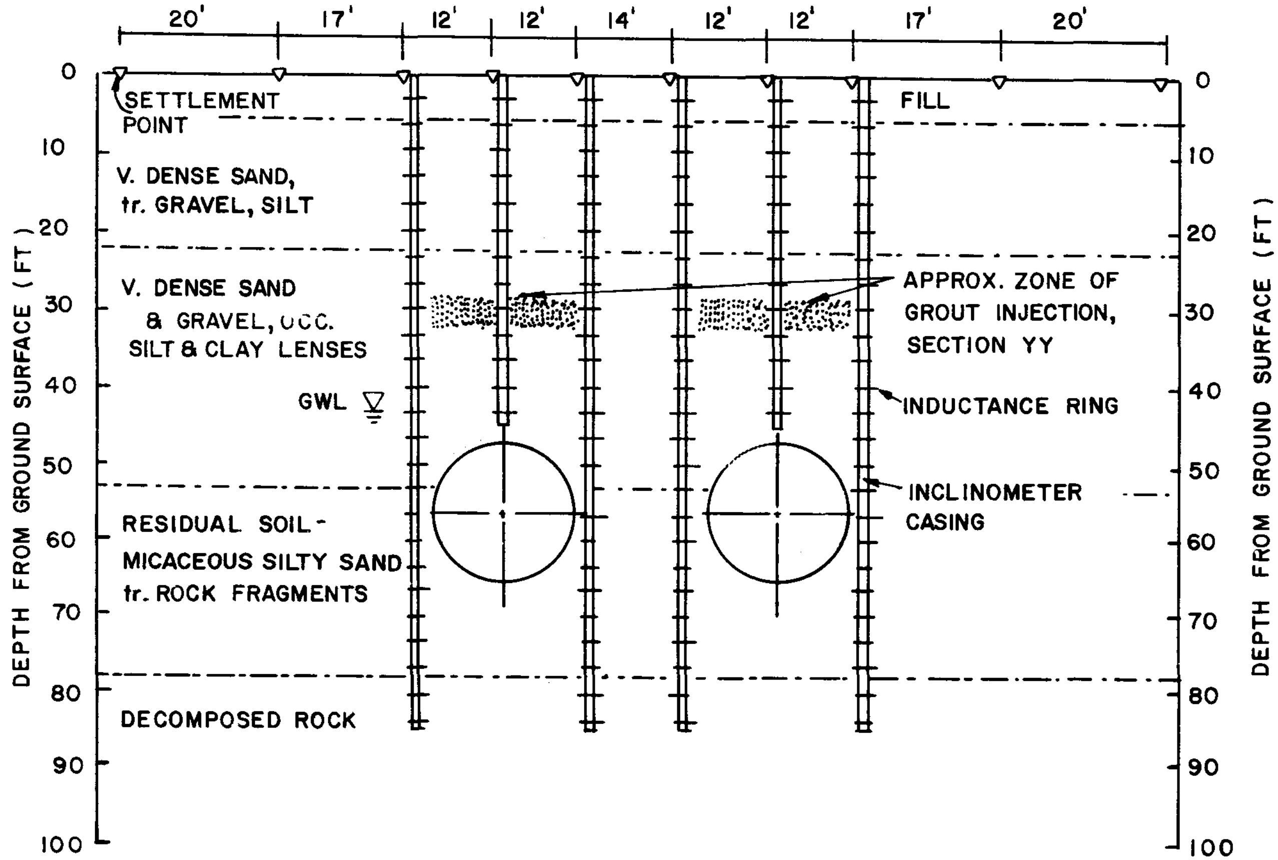


Figure 32. Plan of Test Section, Bolton Hill Tunnels, Baltimore, Maryland (Cording and MacPherson, 1979)



NOTE: 1 ft = 0.3 m

Figure 33. Profile of Test Section, Bolton Hill Tunnels, Baltimore, Maryland (Cording and MacPherson, 1979)

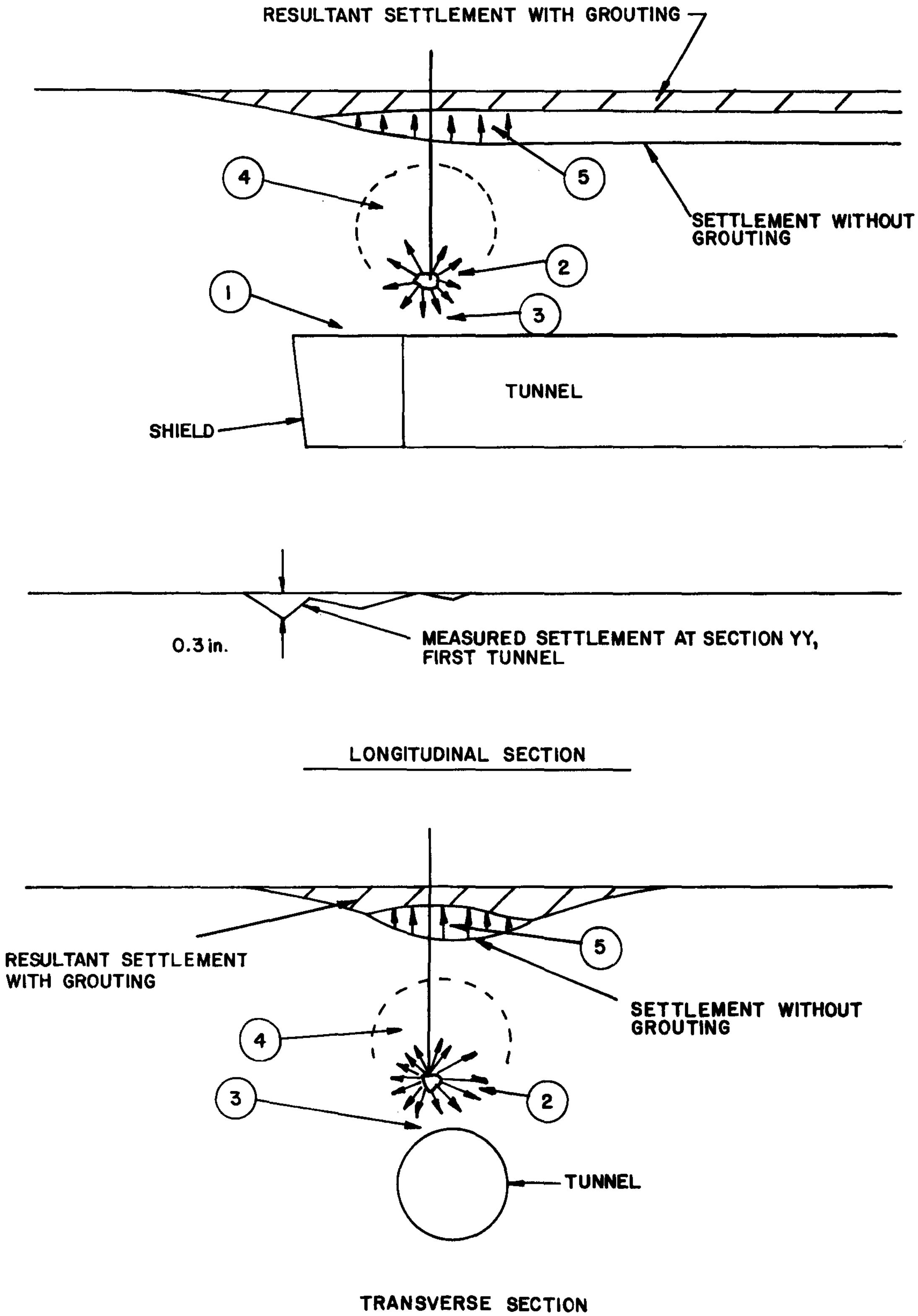


Figure 34. Effect of Compaction Grout Injection on Surface Settlement (Cording and MacPherson, 1979)

case of slow draining soft soils, the design of the compaction grouting program must establish a controlled grouting procedure in order to prevent soil failure and to allow the soil to consolidate. Highly sensitive soils probably should not be subjected to compaction grouting.

Important parameters in designing a compaction grouting program are the spacing and size of the grout bulbs, the rate of injection, soil type, and depth of treatment. The design should result in the best combination of these parameters to cause the maximum amount of consolidation and densification for the given soil and area to be treated. The extent of the area treated is dependent on the extent of loosened or disturbed soil within the influence of building foundations, utilities, etc. The amount of grout required can be approximated based on the estimated extent of ground loss experienced, or the volumetric change required to produce a sufficient decrease in void ratio and the desired increase in strength. However, the actual volume of grout injected is frequently determined by the ground heave or injection pressure.

In the following sections, two significant compaction grouting case histories related to soft ground tunneling are presented.

Case 3K - Bolton Hill Tunnels, Baltimore, Maryland

The design engineers of the Bolton Hill tunneling project specified use of the compaction grouting technique to minimize the settlements of major structures along the tunnel route. These tunnels were typically located approximately 35 to 50 ft below ground surface, the grout pipes were placed for a particular structure in an area 10 ft above the crown of the tunnel where immediate ground losses were most likely to occur (Baker, 1978). As the shield passed beneath a specified structure, compaction grouting was performed and the majority of the movement involved in the settlement trough was prevented. The settlements of the compacted protected building were kept under 1/8 in. during the tunneling operation, while settlement up to 3/4 in. occurred outside of the compaction grouting area.

Cording and MacPherson (1979) evaluated the effectiveness of the compaction grouting operation based on field performance data of a test section in this project. The plan and profile of the test section are presented in Figures 32 and 33, respectively. Some significant findings are summarized as follows:

1. The compaction grouting was able to replace the voids lost around the tunnel and to compress and heave the soils in a cone above the grout bulb. This effect extended to the ground surface and reduced movements whose source was over the shield ahead of the grout bulb location. Figure 34 illustrates the condition.

2. The grouting pressure pushed the soil above the tunnel downward, compacting the loosened soils over the tail and replacing the lost volume above the tail void.

3. Grout pressures were high enough to cause shearing deformations in the soil above the grout bulb, resulting in heave of the soil to a distance of 20 to 30 ft above the crown. Densification of soil immediately above the grout bulb took place, as well as some local densification horizontally. The zone influenced by the grout bulb above the bulb was conical, extending at an angle of 30 to 40 degrees from vertical. Densification decreased with distance from the bulb.

Case 3L - Sewer Rehabilitation, Pennsylvania Avenue, Washington, D.C.

During construction of the WMATA Section D-6 tunnel, extensive ground losses occurred. An approximate 900-ft length of a 5-ft-high horseshoe-shaped combined sewer was affected by this ground loss during tunneling. A cross-section of the rapid rail tunnel and combined sewer is illustrated in Figure 35. The distress caused a series of transverse cracks and widening of previous cracks in the sewer structure (Gularte, 1979).

A compaction grouting program was outlined to replace lost ground and densify the loosened soil. A series of grout pipes were placed on approximately 7- to 10-ft centers along the entire line of the brick sewer. Due to traffic constraints, grouting holes were inclined and positioned approximately midway between the outbound tunnel and brick sewer just above a stiff clay layer.

A zero slump soil-cement grout was pumped into the pipe until a slight movement (1/16 in.) of the brick sewer occurred. A continuous line of fluid level devices was used to monitor the sewer movements. Heave ranging from 1/16 in. to 1/3 in. was experienced in the sewer, and after several days approximately 30 to 40 percent of this heave subsided. In addition, some surface heave was detected by gaps in the curbing and pavement closing. After several weeks, the structural integrity of the sewer was restored by an epoxy adhesive grouting program.

In summary, the compaction grouting technique is effective in protecting near-surface structures from excessive settlement during tunneling in sandy soils. However, this technique may not be effective in very soft soil (peat) or highly sensitive clay because high injection pressures ranging from 150 to 500 psi are involved. A thorough site investigation is needed before employing this technique.

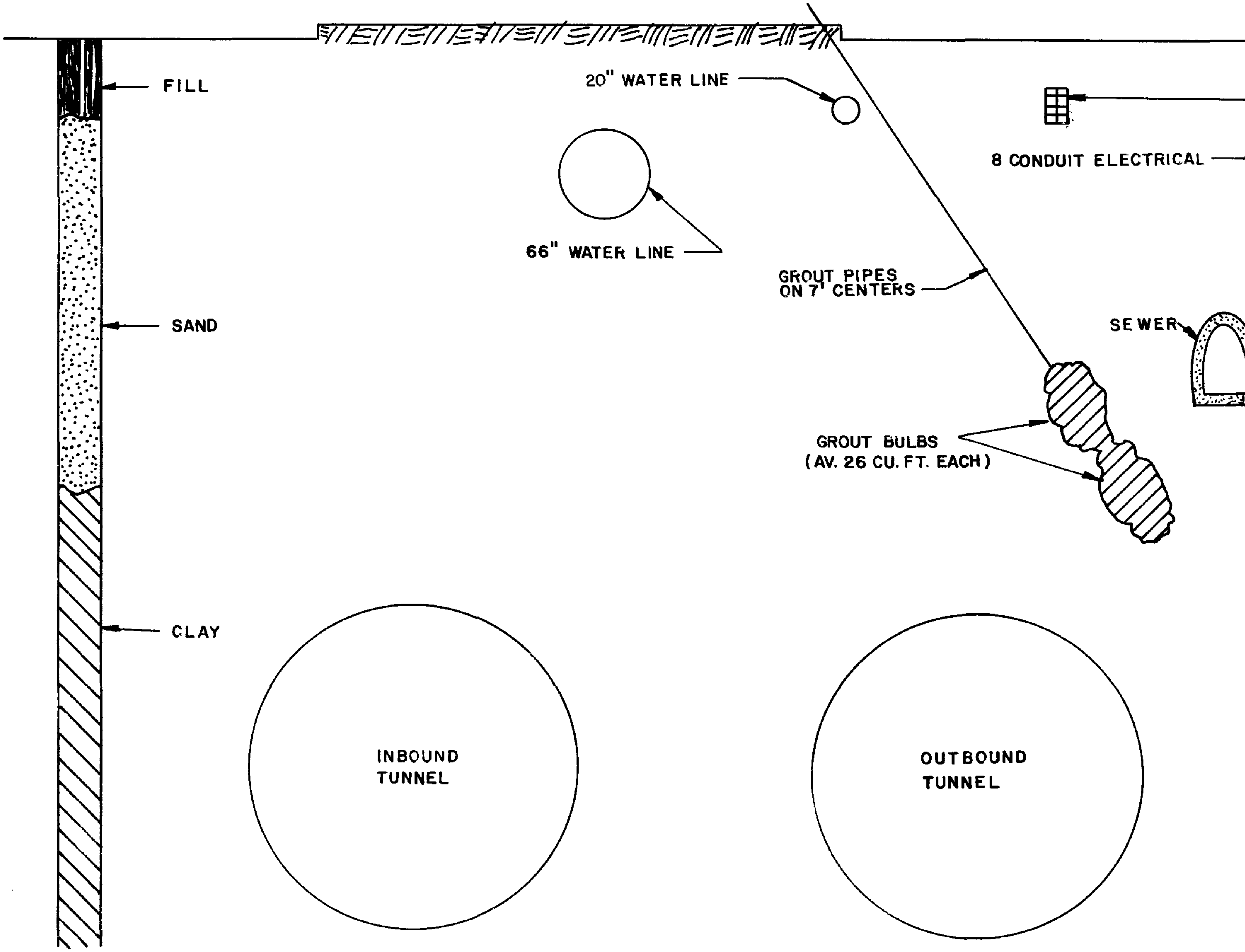
Existing cases of tunneling with the compaction grouting technique are very limited. More tunnel performance data are needed in developing the compaction grouting technology.

3.6 NEWLY DEVELOPED EQUIPMENT FOR TUNNELING IN SOFT GROUND

In the previous sections of this chapter, techniques to modify the ground adjacent to tunneling zones have been discussed. These techniques usually are executed on the ground surface or from a shaft. Thus, holes need to be drilled which not only increases the total tunneling cost but disturbs the ground intended to be stabilized. The effectiveness of the aforementioned techniques is time dependent, in most cases, and may present problems in scheduling tunnel excavation operations. The slurry face tunneling machine and earth pressure balance tunneling method may offer a cost-effective solution in some situations.

3.6.1 Slurry Face Tunneling Machine

Development of the slurry face tunneling machine was inspired by the need for a tunneling machine to excavate in nonself-supporting granular soils below the groundwater table. The concept finds its origin in the method of ground support employed in slurry trench walls.



NOTE: 1 in. = 0.025 m

Figure 35. Soil Profile of Sewer Rehabilitation, Pennsylvania Avenue, Washington, D.C. (Gularte, 1979)

The Bentonite Tunneling System in England and the Hydroshield in Germany have used bentonite as the face support medium. The Japanese have constructed over 450 km of slurry-faced shield tunnels (Peck 1981b). The various Japanese systems have only occasionally used bentonite and only when the ground has had a relatively high permeability. In general, the Japanese tend to use the fine material from within the tunnel face to form the clay slurry with additives such as Carboxy Methyl Cellulose to prevent particles from settling out of the suspension, and also to introduce viscosity into the slurry.

The slurry face tunneling process has the ability to significantly reduce the ground movements, since a change of the state of equilibrium in the ground is minimized. Also, the bentonite slurry imparts a temporary increase in cohesion to the ground in the vicinity of the excavated face. The ground movements resulting from use of the Bentonite Tunneling Machine have been carefully monitored in London and at Warrington (Biggart, 1979). The measurements confirmed the ability of the system to limit ground settlements in cohesionless soils to values similar to those produced by conventional tunneling methods in London Clay. The volume of the settlement trough has been kept within the range of 0.5 to 1.4 percent of the total excavated tunnel cross-section.

The slurry face tunneling machine has some disadvantages. The slurry machine basically is more complicated than the open face tunnel boring machine. In the case of the Hydroshield, it contains three basic elements of control: the air system, slurry system, and hydraulic system. When it is necessary to have access to the face for maintenance, repair, or removing obstacles, air pressure must be increased in the front chamber displacing slurry until the cutter and the tunnel face are exposed under compressed air rather than submerged in mud. The crew may then enter the face region through an airlock. The Bentonite Tunneling System also can be used as an open face machine, but when it is required to convert it to use as a bentonite machine the conversion may require several working shifts.

There are three case histories in Germany (Jacob and Meldner, 1979) and England (Biggart, 1979) on the performance of tunneling with a slurry face machine. They are briefly summarized below:

Case 3M - Wilhelmsburg Sewer Tunnel

The Hydroshield was first applied in Hamburg, Germany in 1974. The tunnel was driven under the harbor region through coarse and fine sands with up to a 60-ft hydrostatic head of water. Outer diameter was 15 ft, and the length was 2.9 miles. The area contains commercial properties including a railroad, piers, and a workshop.

Ground conditions encountered were similar to those anticipated, except the amount of gravel and stone was higher than expected. Hydraulic mucking of gravel via 4 in. through 6 in. pipes on long distances was the most important problem. The mucking difficulties accounted for more than 50 percent of all down time. Improved cobble traps and pressure-controlled switches were installed to overcome this problem.

An incident occurred while crossing under the Ross-Channel at an overburden of 10.5 ft, involving an unidentified bomb crater from World War II. This crater allowed a large quantity of slurry to escape, leading to an instant drop in sustaining overpressure. The result was a complete cave-in of the face with a total filling of the front chamber. However, the crew behind the bulkhead was not harmed.

Although the surrounding circumstances caused numerous standstills, the tunnel was completed within the schedule and without additional costs. This was mainly due to the result of a good advance rate achieved on the latter part of the tunnel length.

Case 3N - Antwerp Subway Tunnel, Section 113

In 1977, 2.4 miles of single track tunnel were to be built in the downtown area of Antwerp, Belgium. A major constraint for this project was that compressed air could not be used for the purpose of groundwater control. It was this particular requirement, in addition to other local conditions and the limitation of ground movement, that led to application of the Hydroshield. A 21-ft-diameter Hydroshield was specially designed to meet these requirements.

The soil in Antwerp is characterized as a uniform fine sand with some interlayers of clay. However, the presence of more clay on the drive than was anticipated caused some problems in the mucking system and at the separating plant. The hydrocyclones, designed to remove particles larger than 40 microns, have difficulty separating clay, thus a denser slurry results. Eventually, to overcome pumping difficulties, it became necessary to discard around 15 percent of each day's bentonite supply. The larger amounts of water present in the disposal material meant higher transportation costs, but this turned out to be cheaper than installing settlement tanks or centrifuges to draw the water off.

Aside from the aforementioned problems, the field performance of the Hydroshield was excellent. At about 30 percent project completion (April 1979), the maximum surface settlement was 5/16 in.

Case 3O - Warrington, England

A 10-ft outside diameter sewer tunnel, 4600 ft in length, was to be constructed. The first 800 ft were excavated through a full face of Bunter sandstone. The next 300 ft were expected to be in a mixed face of sandstone and sand, while the final 3500 ft were expected to be in a full face of sand. During driving, a layer of hard granite and dolerite boulders were found to be lying on the sand and sandstone interface while the sandstone rock never dropped below the tunnel bottom throughout the length of the tunnel. A bentonite machine was used in this 3500-ft mixed face of sand with sandstone in the lower half of the face and 12-in. hard boulders lying at the interface. This tunnel has been completed and has resulted in the development of a process to deal with the mixed and variable conditions described.

Based on a review of the above three cases, it is indicated that the slurry face tunneling machine is useful in difficult ground conditions and when environmental requirements are high. For instance, a) quicksand or other extremely adverse conditions demand particular safety precautions; b) granular soil below the water table is too porous and would cause unacceptable air losses; and c) water pressure requires high air pressure in the occupied work area which would reduce working time per shift and result in additional labor costs. There are many areas, however, where further development is required. Work must be concentrated on widening the range of ground conditions for which the slurry face tunneling machine is suitable.

3.6.2 Earth Pressure Balanced Shield Method

An earth pressure balanced shield similar to the slurry face tunneling machine is one of the newly developed pieces of tunneling equipment which is particularly useful in an unstable, water-bearing layer with high permeability. Although this geological situation can be modified by using chemical grouting or by lowering the groundwater table, extra expenses and time would be required.

The basic structure of the water pressure type earth pressure balanced shield tunneling machine is presented in Figure 36. From this figure, soil excavated by the rotation of the disk and thrust of the shield machine is taken into the cutter frame through slits in the cutter disk. The excavated soil in the cutter frame is compressed as it is transported backward by rotation of the screw. Depending on the rotational speed of the cutter frame and screw, an internal earth pressure can be built up to counter the earth pressure at the cutter face. The internal soil pressure can be controlled by regulating the opening of the hydraulic gate, shield advance rate, and other elements, so that appropriate amounts of muck best suited to particular ground conditions can be reached. To prevent groundwater from being discharged with excavated soil, a hydraulic gate is provided with a mucking adjuster into which bentonite-free water is supplied to generate a water pressure.

If groundwater is not a problem, for instance, when soil media consists largely of silt and clay with low permeability, the muck adjuster is not needed. The excavated soil passing through the hydraulic gate can be loaded directly on belt conveyors or trucks for disposal. This method is called the earth pressure type shield in contrast to the water pressure type shield.

Table 13 illustrates five tunneling projects mined with the water pressure type earth pressure balanced shield method (Matsushita, 1979). The diameters of the excavated tunnels ranged from 6.5 ft to 17 ft, and soil conditions at the sites were silt, sand, and gravel layers with a high groundwater table. It is a typical alluvium formation.

In the case of Project 1, the site was in a highly urbanized district and special attention was required to prevent damage to underground conduits, roads, railway tracks, houses, and other buildings along the route. Thus, the predrainage stabilization technique was ruled out. Also, from the standpoint of environmental protection, the chemical grouting operations were restricted only in the starting and ending points of the shaft. Because of these reasons, the water pressure type shield tunneling method was used. Surface and subsurface soil movements, and the variation of porewater pressures were monitored during tunneling as governing factors for the shield advance rate. The whole project was completed without causing a drop in groundwater level or damage to overlying structures. The normal advance rate reached 150 ft per week.

In addition, there has been another project in which the earth pressure type method was employed in alluvium layers (Matsushita, 1979). The earth pressure type was adopted in view of a shallow depth of overburden (6 ft) and low hydrostatic head (3 ft). Other site conditions were similar to those of the five projects using the water pressure type shield tunneling method (as listed in Table 13).

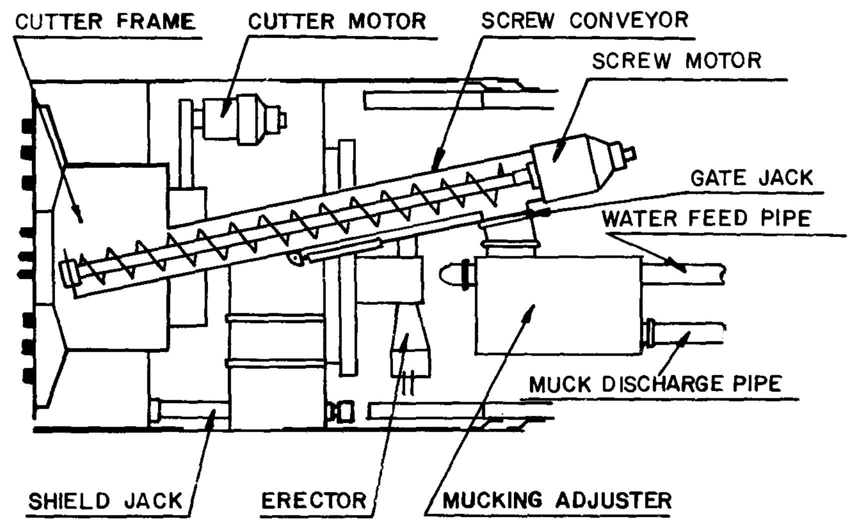


Figure 36. Basic Structure of Earth Pressure Balanced, Shield Tunneling Machine (Matsushita, 1979)

Table 13. Outline of the Tunneling Projects Using Water Pressure Type Earth Pressure Balance Shield Method (Matsushita, 1979)

PROJECT NUMBER	1	2	3	4	5
LOCATION OF PROJECT	TOKYO	TOKYO	OSAKA	KANAGAWA	TOKYO
PURPOSE FOR USE	SEWERAGE	ELECTRIC POWER	ELECTRIC POWER	SEWERAGE	WATER-WORKS
CONSTRUCTION PERIOD	AUG. 76 to MAR. 79	APR. 77 to NOV. 79	APR. 78 to MAR. 79	JUL. 78 to MAR. 80	MAR. 78 to APR. 80
TUNNEL LENGTH (m)	1,634	150	374	678	1,045
MACHINE DIAMETER (mm)	5,240	3,490	1,976	4,940	3,740
MINIMUM RADIUS OF TUNNEL LINE (m)	60	130	150		170
OVERBURDEN (m)	10∿12	10∿12	7∿8	7	16∿25
SOIL CLASSIFICATION	Fine Sand	Silt Gravel	Gravel	Gravel	Fine Sand
N-VALUE	5∿15	15∿50	50	50	50
WATER CONTENT (%)	22∿33	74	13	21∿38	25∿32
SILT & CLAY CONTENT (%)	5∿15				7∿32
UNIFORMITY COEFFICIENT	2∿6	50	10	2∿10	3
COEFFICIENT OF PERMEABILITY (cm/sec)	10^{-3}	10^{-3}	10^{-2}	10^{-2}	10^{-3}
UNDERGROUND WATER LEVEL (m)	G.L.-1.0	G.L.-5.0	G.L.-2.0	G.L.-3.0	G.L.-3.0

NOTES: 1 mm = 0.04 in.
1 m = 3.3 ft

3.7 SUMMARY AND CONCLUSION

Based on the study and review of soft ground tunneling cases related to effective construction procedures and newly developed tunneling equipment, some of the significant findings can be summarized as follows:

1. The selection of cost-effective tunneling procedures is dependent heavily on soil strata configurations, groundwater conditions, soil type, and construction constraints. The success of a selected tunneling procedure is closely related to the accuracy of predicted subsurface conditions. Thus, a thorough subsurface investigation is the basic requirement to guarantee economical tunneling. For example, if a predrainage or chemical grouting modification technique is applied adequately as required by subsurface conditions, the tunnel face will be in a stable state and will enhance the tunneling advance rate. On the other hand, a thorough subsurface investigation also can help to eliminate unnecessary ground modification operations and to reduce the total construction cost. For example, in a very dense, well-graded sandy soil with adequate predrainage and good tunnel mining procedures, limited ground movement (1/2 in.) may be tolerable for some of the overlying structures. Thus, a compaction grouting operation for reducing this settlement would not be necessary.

2. Predrainage is one of the common techniques utilized to reduce the hydrostatic pressure (due to seepage) of water-bearing granular soils at the tunnel face during tunneling. To effectively stabilize the tunnel face, the granular soil configurations, seepage rate, and hydrological conditions (i.e., source of recharge) must be determined before design of the dewatering scheme. Thus, an adequate hydrological survey is a prerequisite step to ensure the success of a predrainage program. There also are some restrictions in the application of this technique. For example, since lowering of the groundwater table will increase the effective stress in the soil layers, it also may induce consolidation of clayey layers. In some urban area situations, the consolidation-related settlements would not be tolerable for the overlying structures. Thus, other ground modification techniques would be required.

3. Although the operation of chemical grouting is expensive, this technique is very effective in stabilizing the uniform coarse-to-medium sands, especially if the full tunnel face is in this type of material. For sandy soils stabilized by this method, not only is the temporary cohesion improved but also the temporary permeability is reduced. The sandy soil is basically transformed to silty or clayey soil behavior; thus, predrainage generally can be eliminated. However, the chemical grouting cannot improve anything significantly in a nongroutable soil layer, so therefore, an appropriate site investigation program must be carried out to determine the soil layer configuration and soil type in each layer. Soils containing less than 15 percent fines (number 200 sieve size) basically can be categorized as groutable soil. For chemically grouted soil with an adequate tunnel construction procedure, the surface settlement can be reduced to a minimum (less than 0.5 in. for a single tunnel).

4. Similar to the chemical grouting technique, the ground freezing technique can effectively eliminate ground runs at the tunnel face. Although there is no restriction on the type of soils to be frozen, the surface heave is directly related to the natural water content of the soft cohesive soil, and the rate of freezing is influenced by the flow rate of the groundwater in the ground. Thus, the groundwater hydrology and subsurface strata conditions are the governing factors on the effectiveness of this method. Usually, employment of the ground freezing technique is the last resort when other techniques are not applicable. This is due mainly to the limited construction experience and high cost. However, as the ground freezing technology develops with time, this method could become a more competitive alternative in the future.

5. Compaction grouting is one of the most economical methods in reducing the surface and subsurface movements and in protecting the overlying structures during tunneling. As the case histories show, this technique has been successful in a number of locations in Washington, D.C. and Baltimore, Maryland. However, due to the radial nonuniform and limited displacing nature of grouting, the effectiveness of this technique with respect to non-connecting voids, soft or sensitive soils, and long-term settlements needs further investigation through analysis of tunnel performance data for a variety of soil types.

6. The newly developed tunneling machines are basically an attempt to combine all ground stabilization techniques in one tunneling operation under existing subsurface conditions and for given construction constraints (for instance, limited settlements). They also are developed to suit a variety of ground conditions with convertible excavation heads. The basic philosophy behind these developments probably was initiated from shield tunneling with compressed air. The Hydroshield, or Slurry Face Tunneling Machine, basically requires fixing the cutting surface as close to the cutter head as possible through a pressure balance slurry to reduce possible tunnel face ground loss. The Earth Pressure Balance Shield Method essentially involves bringing the equilibrium inside the shield, and having the cutting surface totally in contact with the cutter frame. According to case histories in Germany, England, and Japan, these machines are effective, although their structure is relatively complicated. The Earth Pressure Balance Shield Method has been used successfully in a soil strata containing a humus layer. This method is potentially useful when tunneling through very soft plastic clay in substituting for the usual compressed air method.

4.0 SOME BASIC TUNNEL PROBLEMS IN ROCK

4.1 INTRODUCTION

In the geologic past, tectonic forces induced massive failures in rock mass. These failures exist today, extending deep below the ground surface as faults or shear zones. Since their formation, weathering and hydrothermal activities have often resulted in further disintegration of the materials in the fault zones. Advanced chemical decomposition of some of the minerals in shales, schists, and igneous rocks produced clay. Accordingly, fault zones may contain both sheared crushed rock and relatively soft, plastic clay gouge. Although the squeezing and crushed rock zones may run only a short distance along the tunnel alignment, they may create major problems, especially when they are encountered unexpectedly.

All rock masses have mechanical defects. They consist of fractures and joints, the latter can be open or closed. Closed joints may be nearly invisible. Some rock masses have additional sources of weakness such as bedding or cleavage planes. The majority of a tunnel alignment may be constituted of rocks which are likely to exhibit loosening and falling behaviors during tunneling.

Water problems in rock tunneling generally are associated with shear and fault zones, altered rock contacts, etc. If excessive water conditions are predicted, they may not cause a serious tunneling problem. However, if an unexpected high volume and/or high pressure water flow occurs in the tunnel, this flow of water will carry with it some fine materials, and the problem can become especially critical.

Combustible and harmful gases such as methane, hydrogen sulfide, and carbon dioxide repeatedly have been encountered in regions of volcanic activity and in coal or anhydrite bearing shale formations. To prevent tunnel hazards and explosions, special precautions must be taken to monitor for gas at the tunnel face in these formations.

This chapter studies problems associated with the prediction of rock conditions at the tunnel face. Tunneling problems related to squeezing and loosening rocks, water problems, and other problems associated with tunneling are described and discussed. Each of these problems is based on actual case studies of rock tunnel construction.

4.2 PREDICTION OF ROCK CONDITIONS ALONG THE TUNNEL ROUTE

Dowding (1976) made a quantitative comparison of predicted and encountered shear zone size, location, and rock type for tunnels in the Colorado Mineral Belt. This Belt is composed of igneous formations with associated metamorphic rocks and faulting. It represents one of the most complicated provinces for geological predictions.

Figure 37 presents the geographical locations of tunnels in this study. Figure 38 shows the geological features related to these tunnels. The continental divide is the principal geographic feature associated with the tunnel construction. Three steps were involved in the investigation: First, geological predictions were gathered from agency

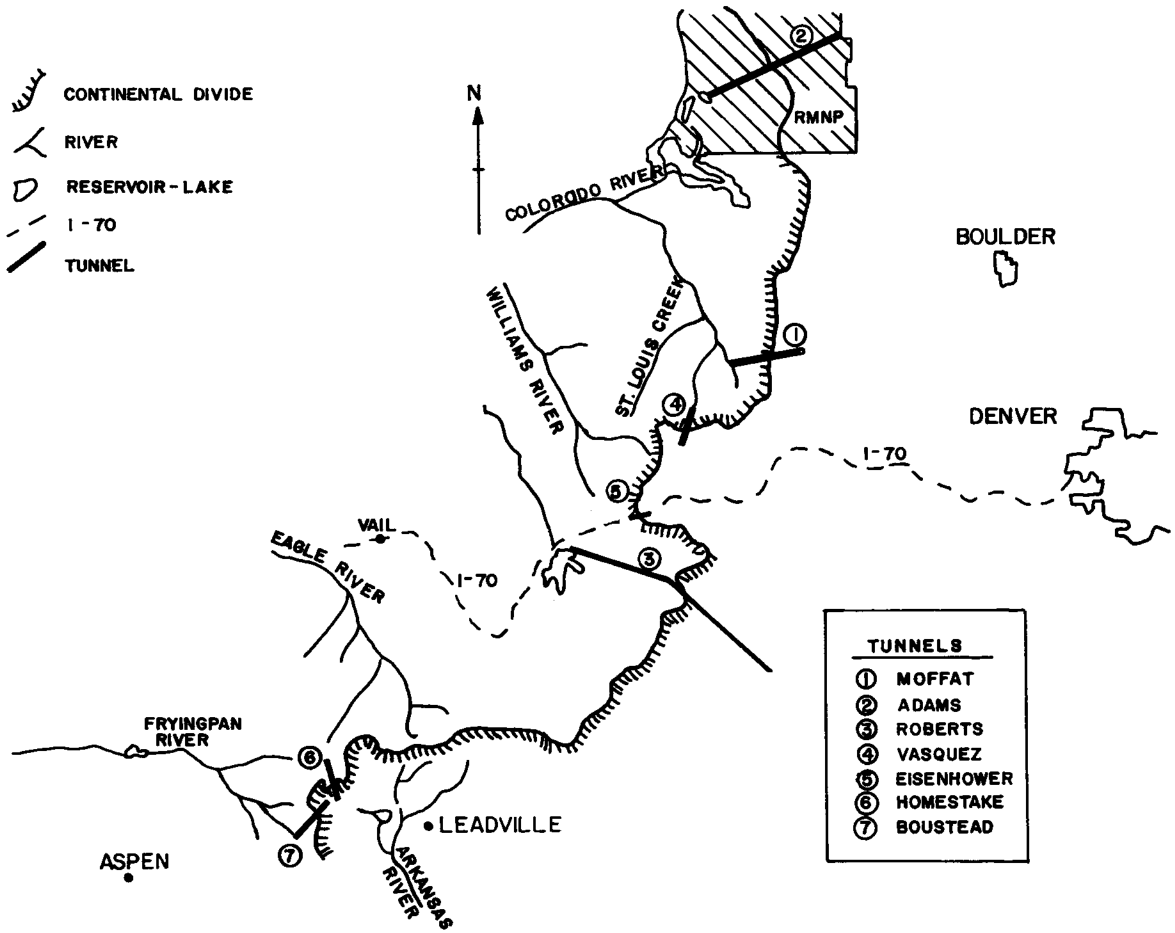


Figure 37. Geographical Locations of Seven Colorado Tunnels (Dowding, 1976)

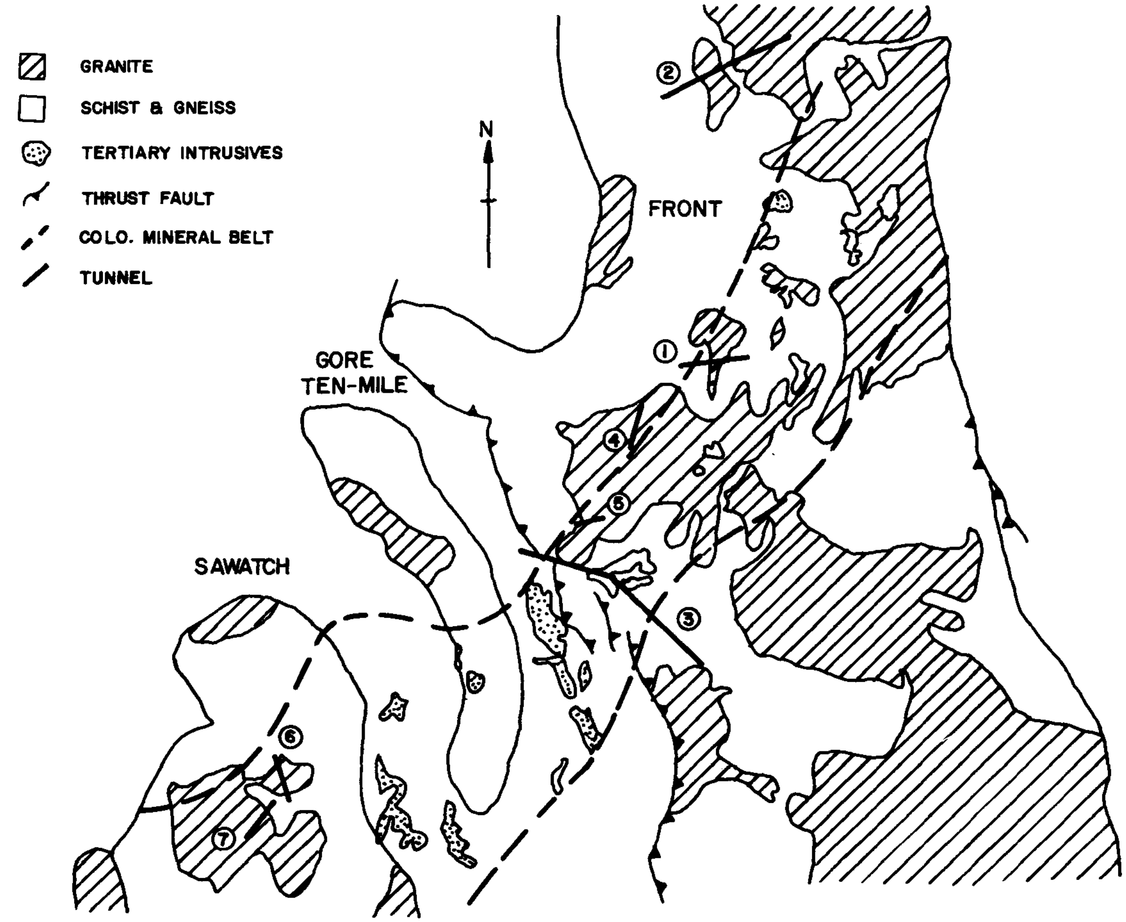


Figure 38. Geological Features Related to Seven Colorado Tunnels (Dowding, 1976)

files; second, observation of the pilot bores of the actual tunnels was made; and finally, the computed location and orientation of the shear zones and rock types were compared to those encountered.

Predicted and encountered data for all tunnels were divided into homogeneous intervals of variable length. Four criteria, used to categorize each interval, were: rock type, shear zone width, strike and dip of the shear zone, and depth of overburden. Shear zones less than 10 feet in width were not considered. There were 100 shear zones predicted to intersect the tunnel alignments. However, 267 shear zones wider than 10 feet were encountered. There were 301 homogeneous intervals predicted to cover the 58 miles of tunnel routes, but 1105 actually were encountered for the same route lengths.

Based on the investigations of seven Colorado tunnels (Dowding 1976), it was found that:

1. For the prediction of a shear zone location at a depth of 1250 ft, the zone would occur with 95 percent certainty within 750 ft of both sides of the vertical projection of the shear zone centerline on the ground surface. These zones would occur within 400 ft of both sides of same with 68 percent certainty.

2. The widths of the shear zones encountered were much smaller than those predicted, e.g., the total width was within 4 percent of the total predicted length.

3. If only very basic and inclusive categories of rock types are considered, the predicted total length of a particular rock was within 25 percent of actually encountered conditions.

4. From a detailed study of the correlation between encountered geological variables of the Boustead Tunnel and the placement of temporary support and excavation cost, it was found that the order of importance decreased as follows: (1) Shear zone width; (2) cosine of the shear zone dip; (3) cosine of the strike of the major joint set; and (4) joint spacing. From a construction point of view, the rock load is related more to the structural and joint features of the rock section than to rock type.

These geologic predictions actually were based on one fundamental assumption--the continuous and planar character of the geological features between the ground surface and the tunnel grade--although this condition usually does not occur in nature beyond a certain distance (say, 200 feet). Thus, it is unrealistic to assume that the conditions encountered will be locationally the same as predicted when exploration is limited to surface mapping and portal boring. For example, in the Moffat Tunnel (1928), two of the sections were located in gneiss. One section located in intact gneiss was unsupported. Other sections located in crushed and decomposed gneiss required heavy timbering (Proctor and White, 1968).

Vertical subsurface borings along the tunnel route, and sometimes pilot bores along the tunnel alignment, are presently used for subsurface investigation. Information from the aforementioned sources usually is sufficient for designing and estimating the initial support systems as well as evaluating and selecting the excavation methods (Desai, et al, 1976; Bock, 1976).

4.3 TUNNELING IN SQUEEZING AND SWELLING ROCK CONDITIONS

When a fault zone is encountered in a hard-rock tunnel, based on experience, it is difficult to predict the potential for squeezing and the amount of supports required. If exceptionally severe squeezing is encountered, heavy side pressure develops, and ground loads cannot be sustained by conventional temporary support systems unless the ground is permitted to deform into the tunnel. If the ground is permitted to squeeze into the tunnel, remining and realignment of the steel sets are necessary but may result in renewed squeezing. Therefore, study of the in-situ behaviors for fault gouges, and excavation and supporting techniques for the squeezing rock tunnels is important in advancing the state-of-the-art for rock tunneling.

In this section, the physical nature and tunneling methods of some case histories for tunneling in squeezing and swelling rock conditions are reviewed.

Case 4A - Mono Crater Tunnel

The 12-ft horseshoe shaped Mono Crater Tunnel of the Los Angeles Aqueduct (1939) encountered flowing and squeezing ground in several places at the contacts between metamorphic and granitic rocks (Proctor and White, 1968). The contact zones were characterized by much faulting and weathering with the feldspathic minerals in the rocks highly kaolinized (Semple, et al, 1973). Of the 59,800 ft of this tunnel, 8400 ft were driven by forepoling through morainal deposits, crushed rock, sand and gravel, mud, silt, fault gouge, and clay--a soft-ground condition at an average depth of 1000 ft. Very heavy forepoling and 6-in. ribs at spacings as small as 9 in., with invert struts, were sometimes required to advance the tunnel.

Problems were complicated even more by a 600-ft head of water above the tunnel. Because of past volcanic activities, the water was charged with gas and that presented a very difficult problem of providing adequate ventilation. In many places, holes were drilled deep into the rock to intercept the water which was bled to pipe drains by closed piping, thereby preventing it from liberating its dissolved gases.

Another problem experienced in this tunnel involved the service period of the timber supports initially installed. This time period was so long that the wood lagging rotted out and caused a collapse. When remined, temporary steel supports were installed before the final liner.

Case 4B - Tecolote Tunnel

During construction of the Tecolote Tunnel (1950-1955) in the Santa Ynez Mountains of Southern California, extreme conditions were encountered, including high water, inflows in temperatures, and explosive methane gas (Crocker, 1955). The tunnel had to be mined through the Santa Ynez fault which has several thousands feet of overthrust displacement. The excavation was in extensively folded and faulted sedimentary units at depths up to 3,000 ft. The peak water inflow was 13,000 gpm with a maximum temperature of 118^{o}F and up to 2,000 ft of head. The water came in mostly through closely jointed shale, and was difficult to handle partly because of the small diameter of the tunnel. Because the tunnel penetrated the upturned layers of adjacent petroleum-bearing formations, much gas entered the tunnel and resulted in two bad explosions. A special deep exhaust well was drilled from the surface to provide the needed special ventilation. The tunnel had to be rebid to finish it because the first contractor went broke (Waggoner, 1981).

The 9-ft horseshoe tunnel was plagued by sheared ground that exhibited delayed squeezing. Initially supported by 4-in. H Beams at 5-ft centers, the supports failed in the course of the year following placement or were displaced into the tunnel. A continuous program of realigning and installing additional supports was carried out. After as many as three phases of adjustment over several months, the support comprised 6-in. sets at 18-in. centers with invert struts. The zone of sheared rock was at a depth of about 800 ft.

Case 4C - Carley V. Porter Tunnel

The Carley V. Porter Tunnel (1966-1969) was built in Tehachapi Mountains as part of the California Aqueduct System (Arnold, et al, 1972). It is 4.8 miles long with a finished diameter of 20 ft. The rock was of various types including crushed igneous, metamorphic, and Pliocene lakebed deposits. The lakebed deposits consisted of faulted and sheared claystones and siltstones with plastic clay and marl mixed in. Fault zones measuring several hundred feet in width and an average of 140 ft in depth were encountered. The 24-ft outside diameter tunnel was excavated by drill and blast methods in conjunction with a shield. Heavy 4-ft-wide steel segments were installed and immediately filled and covered with gunite for additional strength (Varello, 1970). Breastboarding was often necessary, and the ground tended to squeeze. Completion of construction was considerably delayed by caving and squeezing ground in fault zones containing decomposed granite and gouge with large amounts of water. On one occasion, the shield jammed because of heavy ground loads. Within 24 hours the shield skin, which was 1.25 in. thick, and the internal support members cracked and deformed. Eleven struts (W10 x 100 and W12 x 190) were welded horizontally into the shield. Eventually, the squeezing became so great that these struts buckled, and the entire shield was destroyed a week later. Remining in this fault zone required 10-in. sets at 2-ft centers.

During mining of the lake deposits some grade was lost, and remining was required. During remining, excessive squeeze began and led to collapse of the tunnel. Remining of the collapsed zone met with stand-up time problems. At first, the full face method was attempted with steel sets placed on 2-ft centers. The initial 38 ft were remined in this manner. The ground squeeze into the tunnel happened at rates which varied from a couple of inches per day to a couple of feet per day. Then the squeeze accelerated, and in one 48-hour period the ground advanced 15 to 20 ft into the tunnel, sweeping away the breastboards at the face. The excavation method was changed to top heading and wallplate drift, but ground pressure collapsed the drifts. Finally, a multiple drift system was successful in mining through the unstable zone.

Case 4D - Berkeley Hills Tunnel

Two 21-ft horseshoe tunnels were driven through various sedimentary and volcanic units in the Berkeley Hills (1965-1968) between Oakland and Orinda for the Bay Area Rapid Transit (BART) project. The tunnels were driven full-face using a jumbo. The tunnel crossed the active Hayward fault zone which is creeping continuously with movement of 0.5 to 1.0 in. per year. This creates high residual stresses not often encountered. A pilot bore had been driven in the crown along one tunnel line. The geology consisted of complex sections of conglomerates, siltstones, sandstones, shale, and volcanics. Numerous seams and faults containing wet, badly-sheared soft shale, serpentine and gouge crossed the tunnel line. The ground tended to ravel and the tunnel caved in on several occasions. Support consisted of steel sets on 2- to 4-ft centers. Four-foot rounds were mined. Squeezing ground caused stability problems in both supported and unsupported sections of the heading, and also became a problem near a portal of one tunnel under 40 ft of cover. Steel sets were being augmented by crownbars and a breastboard. At one point the breastboard fell out and the face caved in. Excavation continued, using two breastboards and crownbars or spiling. Only a small portion of the face could be open at any one time. The face advanced 36 ft in one week as compared to 30 ft per day in sections of the tunnel in better ground. The ground movement continued to accelerate, and the tunnel was declared unsafe when cracks from the caving ground reached the surface. Extra support was added to stabilize the ground behind the face before excavation continued. Myer, et al (1977), noted that more prompt installation of crownbars, breastboards and steel support would have averted the problem.

Away from the portal area, in the fault zones, Ayers (1969) indicated that the ground was not difficult to excavate, and it initially applied relatively light loads to the steel sets. However, two months after excavation the timbers had crushed and the ground squeezed 8 to 10 in. into the tunnel between the supports. Sets were distorted and re-alignment and installation of invert struts close to the heading were required on several occasions. The rate of squeezing accelerated each time the ground was cut back for support realignment. In addition, the tunnel settled as much as 18 in. in the fault areas. Tunnel depth was about 500 ft in the fault zone.

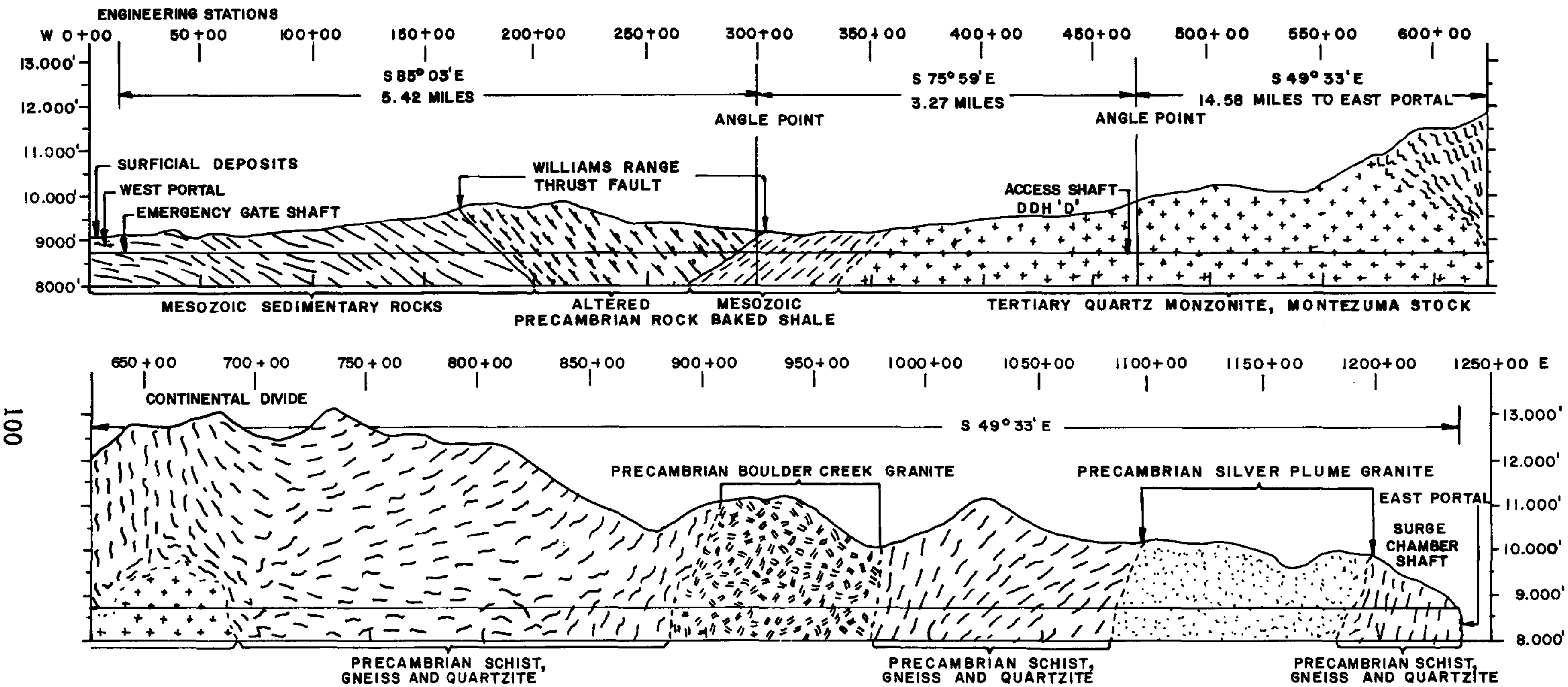
Case 4E - Adams Tunnel

The Adams Tunnel of the U.S. Bureau of Reclamation—Colorado Big Thompson Project (1940) passed through the Continental Divide. In this 12-ft horseshoe tunnel, a major fault zone was encountered where the saturated rock had been altered to a plastic clayey mass. The ground squeezed into the excavation from sides, arch, and bottom, displacing and bending steel, and producing an invert heave of about 12 in. (Proctor and White, 1968). The squeezing material was cut back to relieve the steel supports on several occasions. Even with invert struts and gunite shot between the ribs, failure of the 6-in. steel sets on 1-ft centers occurred. Excessive side pressures caused failure of a few invert struts. These were replaced with heavier sections and the whole series, both new and old, was reinforced with knee braces. Yet, only 230 ft away, no support was required.

Case 4F - Harold D. Roberts Tunnel

The Harold D. Roberts Tunnel (1956-1960) passes beneath the Continental Divide in Colorado. During excavation of the 12-ft horseshoe tunnel a variety of sedimentary, metamorphic, and igneous rock units were encountered. Although much of the tunnel alignment required minimal support, the tunneling methods, advance rates, and support requirements changed radically when a major fault zone was encountered at a contact between metamorphic and sedimentary rocks (Wahlstrom, 1962). The water-saturated fault zone contained highly sheared, pulverized gneiss, with abundant gouge exhibiting montmorillonite alteration. Squeezing and swelling ground deformed, thus dislocating the heavy steel supports and required extensive installation of heavy 8-in. intermediate sets with invert struts after the rock was first encountered in the tunnel heading. The tunnel depth at the thrust fault zone was about 1000 ft; however, high swelling pressures confined to gouge in narrow faults proved to be of minor concern.

In other sections of the Roberts Tunnel, closely fractured water-bearing granite and gneiss, and closely sheared shale running ground were encountered. Breastboards were installed to contain this highly incompetent running ground around Station 300+00 (Figure 39).



NOTES: 1 ft = 0.3 m
1 mile = 1.6 kilometer

Figure 39. Generalized Geological Sections of the Harold D. Roberts Tunnel, Colorado (Wahlstrom, et al, 1968)

A groundwater problem also occurred in this tunnel. At one section, high water flow at approximately 1300 gpm was encountered in the fractured, brittle, quartzitic sandstone tunnel face. A bulkhead of concrete-filled sacks was installed, and then grout was injected to seal off the heavy water flow (Wahlstrom, 1973).

Case 4G - Henderson Haulage Tunnel

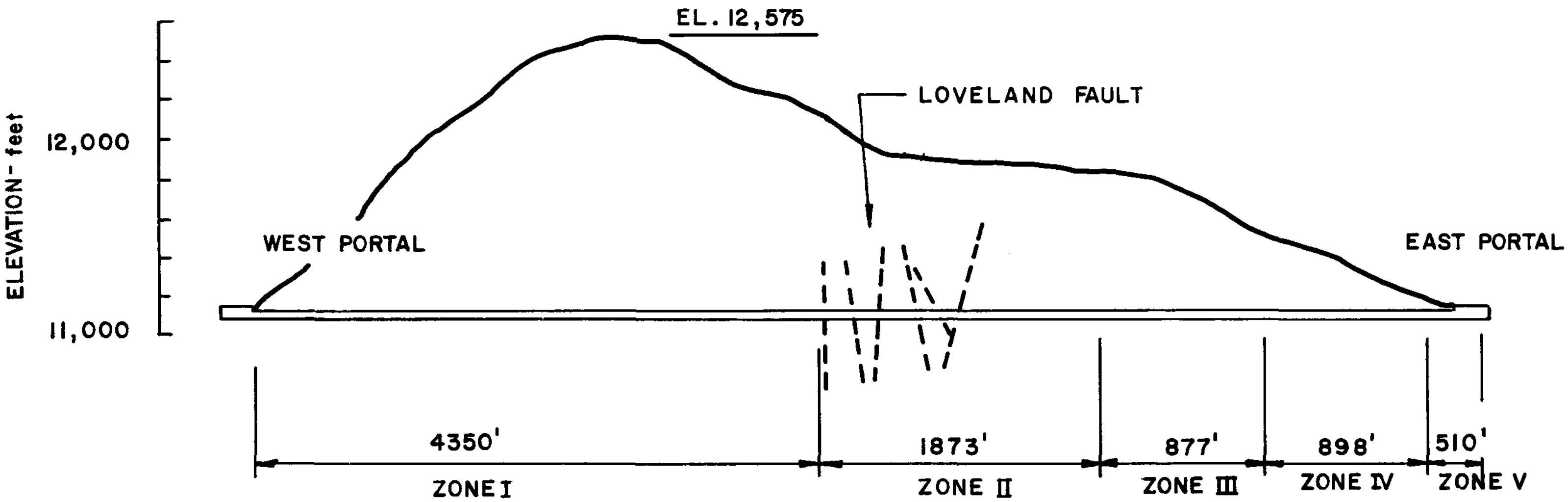
The Henderson Haulage Tunnel is a 10-mile-long mine haulage tunnel running beneath the Continental Divide in Colorado. The 15-ft by 17-ft tunnel is beneath as much as 1200 ft of cover. The rock consists of gneiss and granite. The tunnel was driven full face and little support was needed except in fault zones. In one 62-ft-long zone the tunnel encountered gouge consisting of a mixture of dark clay and coarse to fine sand. The gouge began to squeeze into the tunnel, so a bulkhead was erected at the face. However, the squeeze pushed the bulkhead into the tunnel at about 1 in. per hour, and finally accelerated to a run over the bulkhead, filling the tunnel. The heading was advanced through the zone using a top heading and bench with crownbars. Some index tests on the gouge material indicated significant swell potential for the gouge, yet in-situ behavior was predominantly that of squeezing ground. The high in-situ stresses produced large time dependent deformations which overshadowed the swelling behavior of the gouge (Brekke and Howard, 1969; Myer, et al, 1977).

Case 4H - Dwight D. Eisenhower (Straight Creek) Tunnel

Construction of the Straight Creek Tunnel, North Bore (1968-1973), a highway tunnel crossing the Continental Divide of the Rocky Mountains west of Denver, Colorado, was preceded by an 11-ft horseshoe pilot tunnel (1963-1964). The main rock type was granite, and the major structural feature was the Loveland fault zone. The 8900-ft-long, 48-ft by 50-ft tunnel had up to 1450 ft of overburden. The tunnel line was divided into zones for reference (Figure 40), based on assessed ground conditions. Despite unprecedented geological reconnaissance and the benefit of ground load and displacement measurement data, and laboratory index testing data from this exploratory tunnel, construction of this vehicular tunnel proved extremely troublesome. Squeezing movements in the pilot tunnel of about 12 in. at the side and 24 in. in the invert occurred in fault gouge having the consistency of a stiff clay (Hopper, et al, 1972). Several stages of retimbering and realignment were required.

Excavation of the main tunnel proceeded simultaneously from the east and west portals. An attempt was made to drive Zone II full-face using a shield. The shield was erected at the end of Zone I approximately 125 ft from the Loveland fault. It had advanced 70 ft when it developed mechanical problems in its support rollers. By the time the roller support was changed to a sled system, ground pressures had frozen the shield into place. Therefore, the original plan to excavate full-face in the fault zone using 14-in. steel sets was revised. Instead, thirteen contiguous 8-ft drifts were driven around the perimeter of the tunnel and later filled with concrete (ENR, 1971).

In the top heading of Zone III, stability problems also developed. The ground in this zone was very blocky and seamy-to-highly decomposed. Stand-up time problems at the face, and distortion of sets due to ground loads which increased with time, were experienced. These problems led to adoption of the multiple drift method for excavation of Zone II. The crown drift was driven first, and the worst material encountered was gouge consisting of stiff clay with blocks of decomposed weak rock. Squeezing rock was a



NOTE: 1 ft = 0.3 m

Figure 40. Simplified Profile of Eisenhower Tunnel (Hopper, et al, 1972)

problem even in this 8-ft by 9-ft drift. It was advanced at a rate of 6 to 7 ft per day. Displacements of 0.5 in. were noted within 1 to 1.5 hours after excavation. The timber blocking of the crown drift temporary supports was crushed by 6 in. of ground squeezing one week after excavation. As the 8-in. steel sets also showed signs of distress, the ground was cut back by about 12 in., and retimbering performed. Knee braces were welded between the ribs and invert struts. Some sections were retimbered several times indicating ground movements on the order of 18 inches. The lower sidewall drifts experienced some stand-up time and squeeze problems, but successive drifts in general had fewer stability problems.

The south tunnel, Edwin C. Johnson Bore, was constructed between 1975 to 1979. The two tunnels were 100 ft apart at the portals and about 250 ft apart near the center, major shear zone. With experience acquired during construction of the pilot tunnel (1963-1964) and north tunnel, the south tunnel was designed similarly to the north tunnel with some additional refinements. Gay (1980) summarized the main refinements:

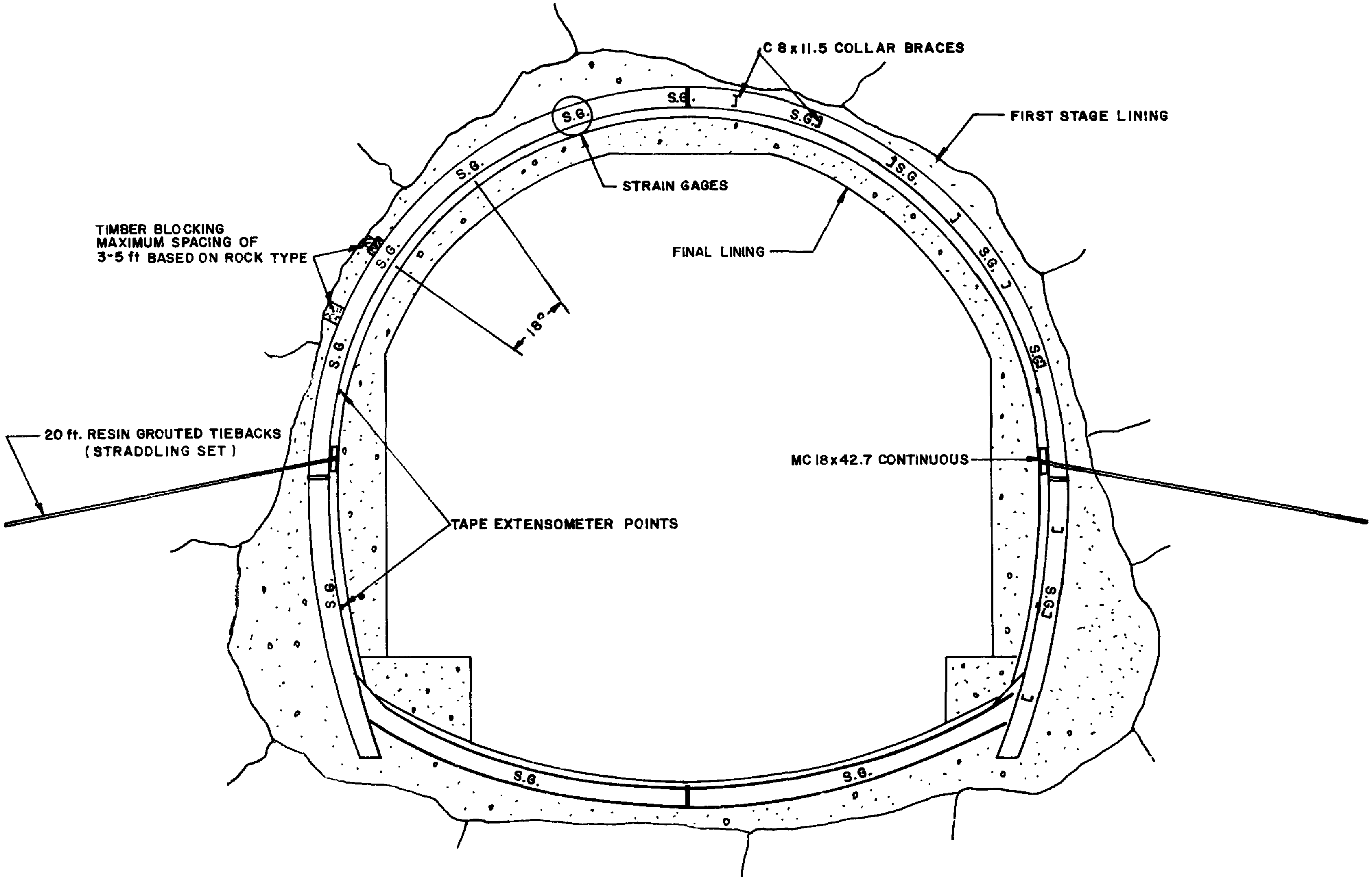
1. Rock reinforcements were placed around the arch and ahead of the excavation.
2. The length of unsupported ground was determined by rock type, and tunnel support was designated by specific location.
3. Foundation drifts were used throughout the eastern portion of the tunnel.
4. The time and/or distance lags were specified behind the face for concrete liner pouring.
5. A radial drainage system was designed.

For the western half of the mountain, a modified horseshoe with a No. 11 rebar tie-back support system was designed, utilizing invert struts in the poor rock area (as shown in Figure 41). In the center 500-ft major fault zone, nine small drifts that backfilled with concrete and invert struts were specified (see Figure 42). Since much of the rock in this fault zone had decomposed into clays, and some large inclusions within this mass might cause large concentrated loads on the structure, this support system was designed to support a 50 ksf pressure. The eastern half of the tunnel is intersected by a series of shear zones under locally high loads. The design through this area included two foundation drifts, a crown drift, arch sets, and invert struts (Figure 43).

During construction of the south tunnel, there was no change in design, only some remedial measures at specific locations. In an area of the west top heading near the multi-drift structure, excessive movements were observed. Additional 20-ft ties were installed through the first stage concrete to stabilize the structure. The convergence at springline was 2.5 inches.

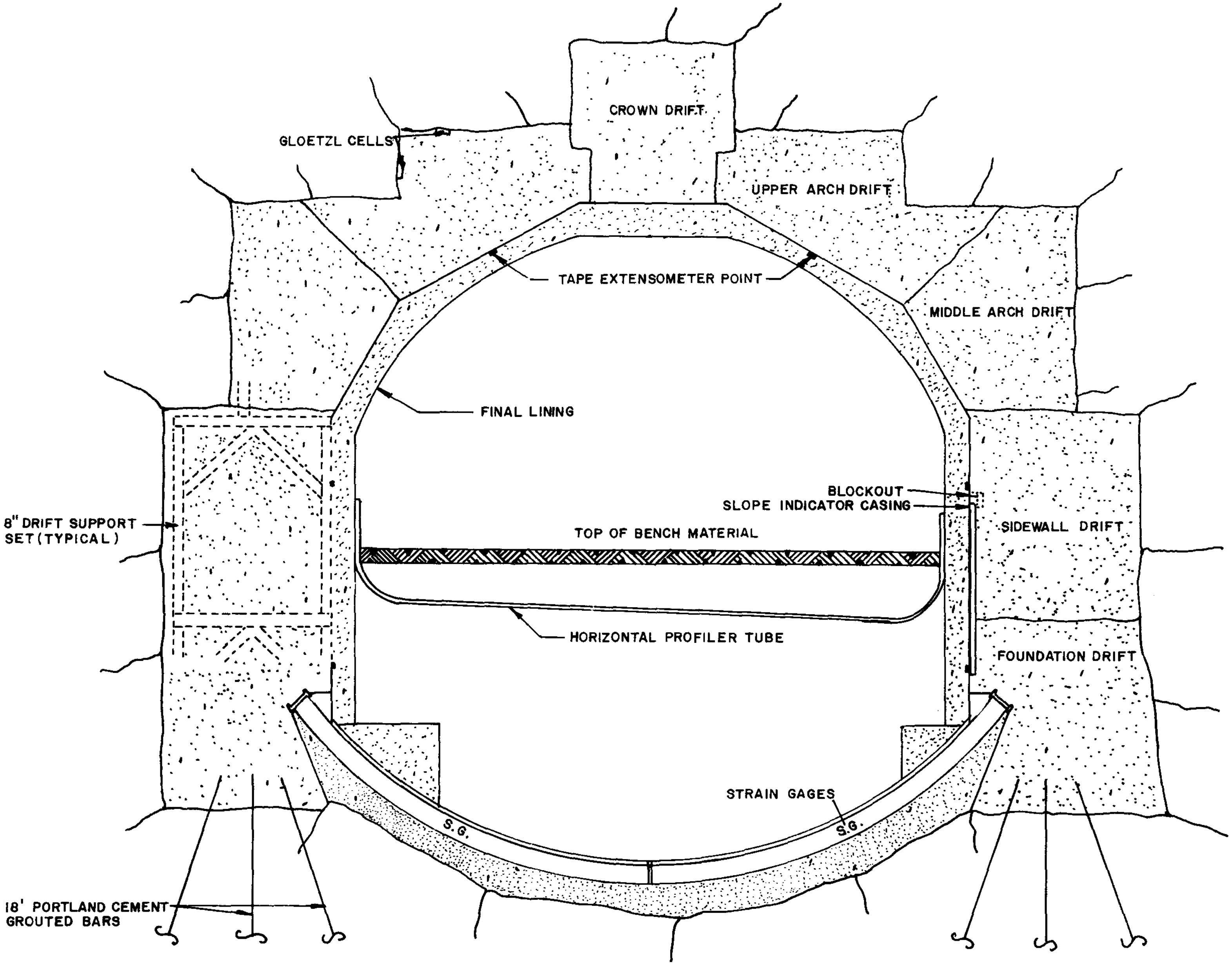
In the multiple drift section during excavation of the side wall drifts, up to 9 in. of settlement were observed in the crown drift. Some remedial measures were utilized, and these settlements did not interfere with the final lining.

At one section of the east top heading, approximately 40 ft of decomposed metasediments ran out from under the crown drift. This produced about 6 in. of



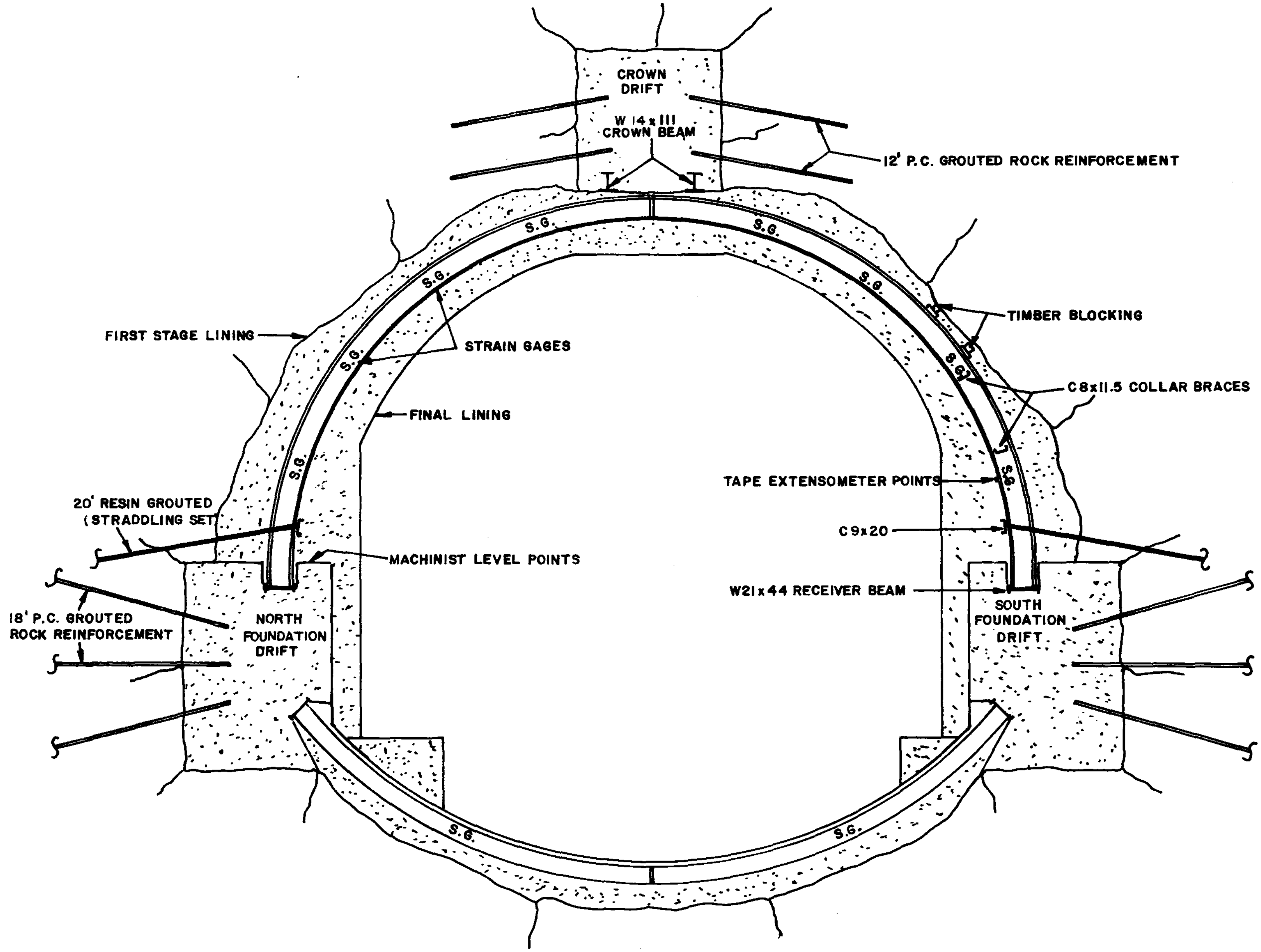
NOTE : 1 ft = 0.3 m

Figure 41. Typical Support System for West Side, South Bore, Eisenhower Tunnel (Gay, 1980)



NOTES: 1 in. = 25 mm
1 ft = 0.3 m

Figure 42. Typical Support System for Multi-Drift, South Bore, Eisenhower Tunnel (Gay, 1980)



NOTE: 1 ft = 0.3m

Figure 43. Typical Support System for East Side, South Bore, Eisenhower Tunnel (Gay, 1980)

settlement before the crown drift could be supported. The contractor placed a pattern of 30-ft, resin-grouted rock bolts in the face to stabilize it. The progress through this area was about 4 ft per day (Gay, 1980).

Due to comprehensive observation during construction of the north tunnel, and an adequate instrumentation program utilized during excavation of the south tunnel, better control of the effect of ground conditions was provided and thus resulted in minimal cost overruns.

Case 4I - Alpine Highway Tunnel

Rabcewicz (1975) reported on a 4-mile-long, and 33-ft-diameter highway tunnel in the Alps. Overburden was up to 3150 ft and the rock was mainly of weak phyllites. Excavation proceeded by a method of top heading followed by two benches. The average advance rate was 7 ft per day. Stability problems were persistent and severe including, at one point, a large face collapse which injured two men. The major problem was one of large deformations occurring quickly which, when unchecked, led to collapse. The stand-up time was short. At first, shotcrete reinforced with rock bolts and wire mesh was placed immediately after excavation to avert instability. However, continued deformation cracked the shotcrete. Stabilization was finally achieved by using reinforced shotcrete with gaps running along the tunnel axis plus steel ribs with joints, both measures allowing for compressive displacement (Myer, et al, 1977).

The nine case histories reviewed above report many spectacular and very costly experiences with squeezing and swelling rocks. Except for the problems created by inflow of large volumes of water, ground that squeezes or swells should not be considered as a major tunneling hazard. The squeezing and swelling rates in rock masses are largely dependent on local geologic conditions. It is generally safe to predict that, given sufficient time, natural processes will tend to bring the material into a condition of approximate stabilization, and movement will slow down and eventually stop. In many cases, heavy pressures on tunnel supports initially installed to contain the material gradually diminish in intensity over short to long periods of time. Therefore, the appropriate method for dealing with squeezing and swelling rock in underground openings would be to allow the material to move under controlled conditions into the opening with the logical expectation that the movement will diminish and eventually stop. In many situations, failure of tunnel linings can be avoided either by delaying placement of the lining until squeezing or swelling has become negligible or by leaving openings in the lining that will allow pressure relief until stabilization is finally attained.

For predominantly swelling rock, if the tunnel opening can be kept dry and the rock surface is properly sealed, the creation of swelling pressure may effectively be prevented. For example, the 20-ft Glendora Tunnel (1965-1968) intersected the Sierra Madre fault zone in the San Gabriel Mountains in California (Proctor, et al, 1970). The significant problem in this tunnel was swelling pressure derived from bentonite-rich layers in the shales being excavated. No problems were developed because of low cover and absence of water in the tunnel opening.

4.4 TUNNELING IN LOOSENING AND CRUSHED ROCK CONDITIONS

Most of the rock materials away from fracture and shear zones can be categorized as loosening rock. Thus, the rock tunneling procedures including support systems are basically designed for loosening rock.

Currently, tunneling projects such as the Tunnel and Reservoir Program of the Metropolitan Sanitary District of Greater Chicago, and the Transit System in Atlanta being built by the Metropolitan Atlanta Rapid Transit Authority, basically encountered no major construction problems involving tunneling through loosening rocks (Rose, 1979; Dalton, 1979). Loosening rocks have a more predictable engineering behavior than squeezing and swelling rocks.

In this section, some problems of loosening rock, such as falling rock and rock overbreak cases, are reviewed. Tunneling problems associated with cohesionless crushed rock in the sheared zone are also discussed.

Case 4J - Oak Dam Tunnels

The Oak Dam Tunnels were excavated in Pierre shale, a clayey soil with an unconfined compressive strength of about 70 to 200 psi (Underwood, 1965). The shale could be cut easily with air spades. The ground tended more toward blocky and seamy than pure squeezing. The cover over the 29-ft-diameter tunnels was up to 73 ft. A mole was used for full face excavation. Support consisted of ring beams on 4-ft centers. In one tunnel, the advance rate was 68 ft per day until a fault was intersected and fallout stopped the machine. Progress decreased drastically to 91 ft in 30 days, and an average overbreak of 20 ft was experienced over this distance. In another fault zone, 15 ft of unsupported ground collapsed over one weekend. In another tunnel the magnitude of fallouts was so frequent that only 13 ft were mined in 31 days. Overbreak averaged 20 to 30 ft. Finally, a very large fallout buried the machine and the remainder of the tunnel was hand-mined using side drifts, top heading and bench methods. Myer, et al (1977), pointed out that the major excavation problem was the lack of appreciation for the short stand-up time of the material. At first, when a fault was encountered and fallout blocked the cutter head of the mole, the practice was to withdraw the machine and take another run at it. This took too much time and large cave-ins ahead of the machine resulted. When this practice was reversed and the machine was not backed off in bad ground, progress improved.

Case 4K - WMATA Section C-4 Tunnel

The Section C-4 Tunnel of the Washington Metropolitan Area Transit Authority (WMATA) was excavated under the Potomac River. One of the major tunneling problems of this project was handling the granite gneiss which had a tendency to overbreak during blasting. In order to overcome this problem, reinforced crownbars were used. These bars, in effect, created an umbrella over the crown of the tunnel, and hence, eliminated most of the overbreaks. Consequently, a more uniform configuration was formed, the need for support lagging was reduced, and the tunnel opening was safer. At the crossover structure, it was necessary to use wallplates and steel supports due to the close tolerances required with one tunnel running directly under another. Steel ribs and shotcrete were used in certain portions of the subway station as both temporary and permanent support systems (Shea, 1976).

Case 4L - WMATA Section K-1 Tunnels

The Section K-1 Tunnels of the Washington Metropolitan Area Transit Authority (WMATA) are of a 20.7-ft outside diameter and an approximate 8600-ft length of single-track, parallel tunnels (Figure 44). The K-1 tunnels are located in three general types of

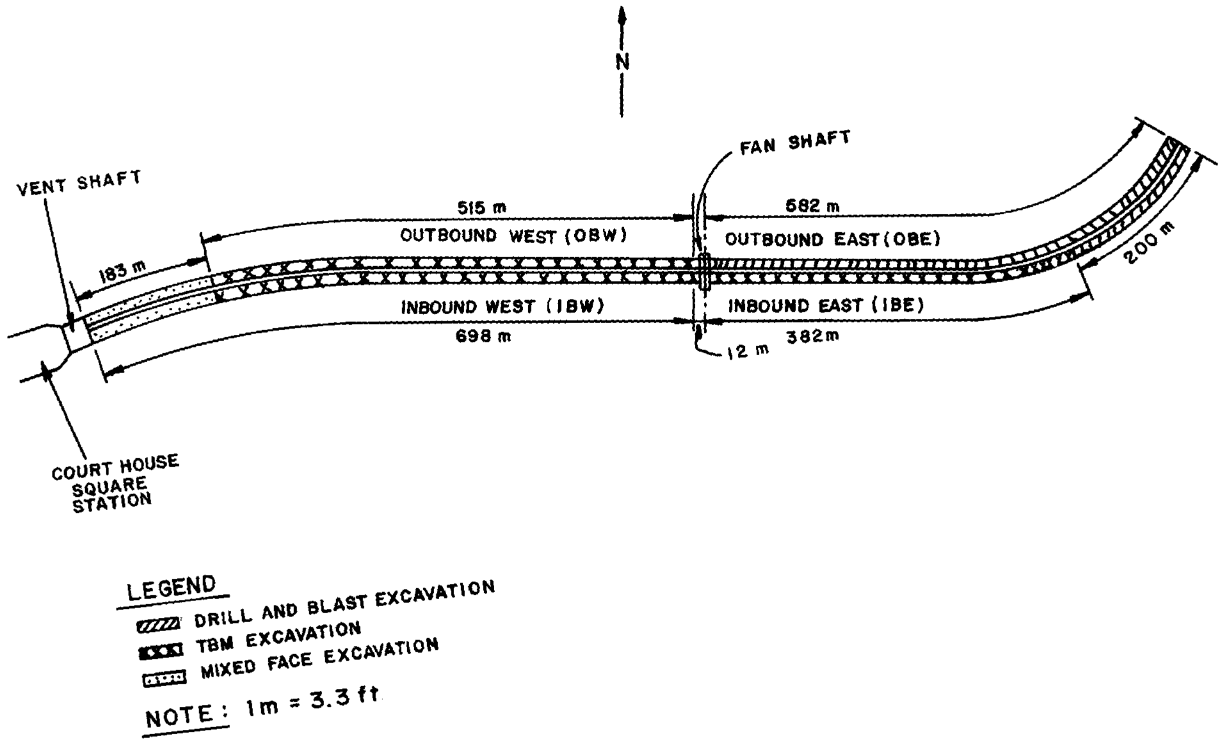


Figure 44. Plan of WMATA Section K-1 Tunnel (Garbesi, 1979)

ground: (1) Fractured and metamorphosed rock, i.e., quartz-hornblende-biotite gneiss, (2) the transition zone between weathered and competent rock, and (3) the weathered rock or residual soil zone. Approximately 4420 ft of tunnel were mined through competent rock, 2680 ft through a combination of bedrock and weathered rock, and 1500 ft through in-situ weathered material. The weathered material can be described as stiff to hard micaceous fine sandy silt, or very compacted silty sand to medium-grained sand with the texture of the parent rocks. In addition, the groundwater table was about 70 ft above the tunnel. Thus, hand-mining methods were required through the weathered material zone.

It can be seen from Figure 44 that the majority of the Inbound West, Inbound East, and Outbound West tunnels were TBM mined. The TBM was rebuilt to cope better with the caving, blocky ground encountered. The support systems utilized in these segments are shown in Figure 45. Due to pressures for an accelerated completion of the project, the remainder of Inbound East and the full length of Outbound East were excavated by the conventional full face drill and blast method.

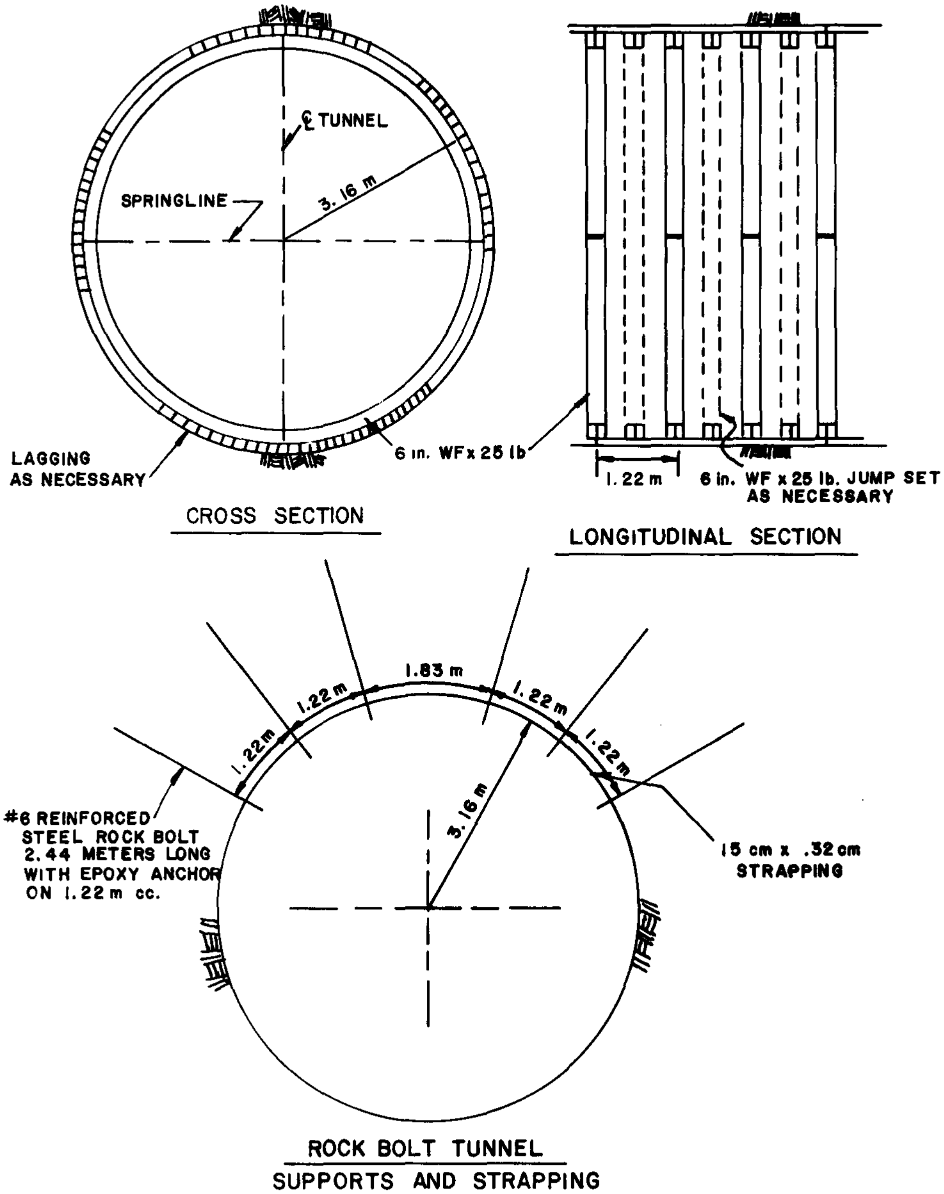
Due to stress cracks and movements of the ground in the pillar just above the tunnel springline and in the quarter arches, as well as the constant caving of the tunnel arch, the final 600 ft of both Inbound West and Outbound West were mined by the top-heading and bench method. Because of the softness of the material, and the tendency for rock blocks to drop out of the arch, a part-face Roadheader excavator was used for mining to avoid shock from blasting.

The 600 ft of Inbound West were mined before Outbound West. Extreme difficulties occurred occasionally while advancing the top heading due to the short stand-up time of the tunnel arch and face. In these cases, extensive breastboards and crownbars were required since the No. 11 rebar spiling was not effective when working in soft ground. The initial support system of this 600-ft inbound segment and the first 100 ft of the outbound segment are shown in Figure 46.

During the mining of the first 100 ft of the Outbound West segment, failure of the pillar between tunnels developed, requiring extreme remedial measures to arrest the failure. The top-heading mining of the outbound segment was suspended. The remedial operation for the Inbound West segment included basically: (1) Shotcreting the ground under the wallplates in the areas of failure; (2) installing preloaded invert struts at locations exhibiting maximum movement; (3) installing 1-in.-diameter pre-tensioned tie rods to strengthen the pillar; and (4) pouring the permanent concrete liner and filling the space between liner and ground with contact grout.

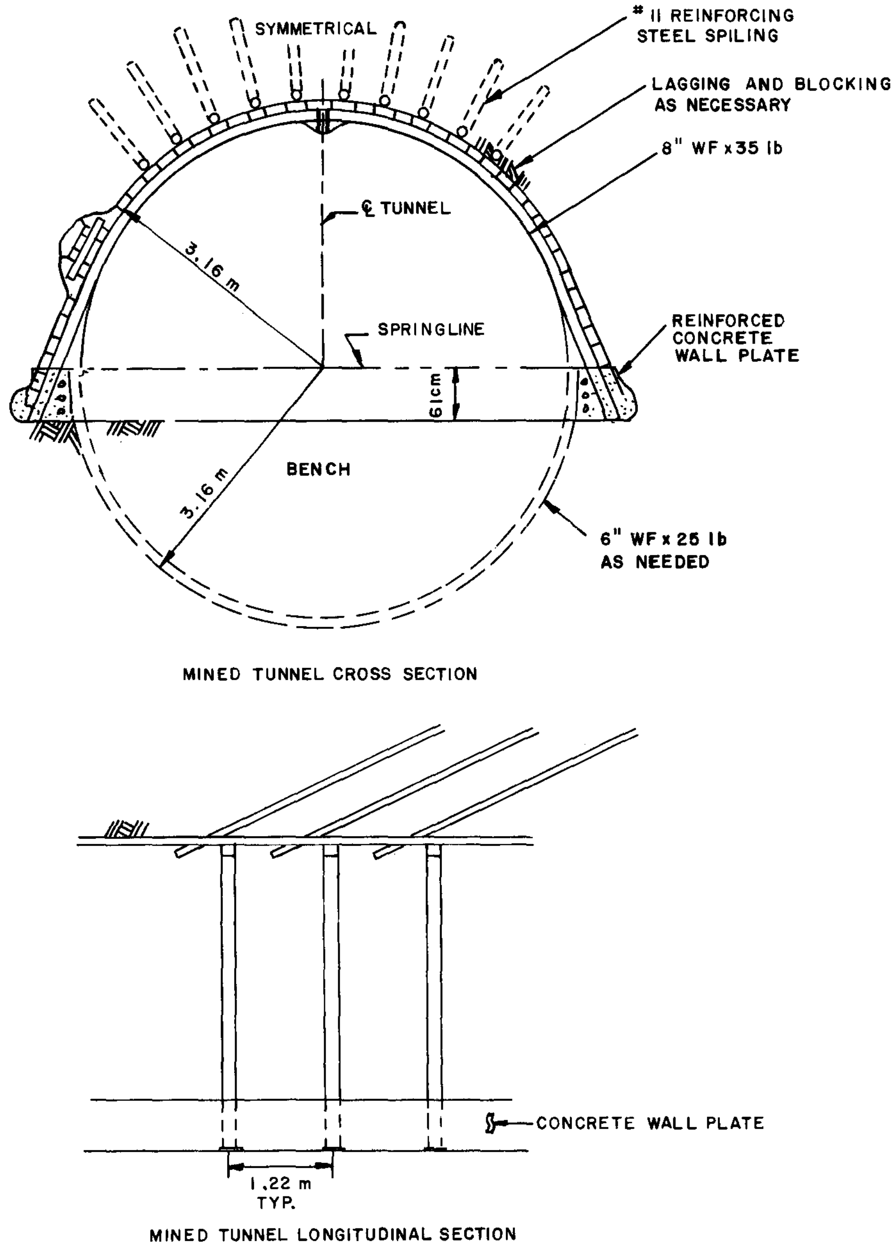
In the meantime, a gradual failure of the lagging and blocking, and some buckling of the steel support also were observed in the adjacent 100 ft of the outbound segment. Shotcrete was applied and no additional movement was detected in this support system. The support system for the final 500 ft of the outbound segment is shown in Figure 47. Due to the almost zero stand-up time of the material, it was necessary to mine and install the wallplate prior to any other mining of the face. It was necessary also to facilitate the placement of shotcrete at any time in the mining cycle.

Additionally, it should be noted that the material strength estimated by the Standard Penetration Test (75 to 100 blows per ft) based on homogeneous ground does not represent the actual strength of the weathered and discontinuous rock mass. This may be one of the major causes of the above-mentioned tunneling problem.



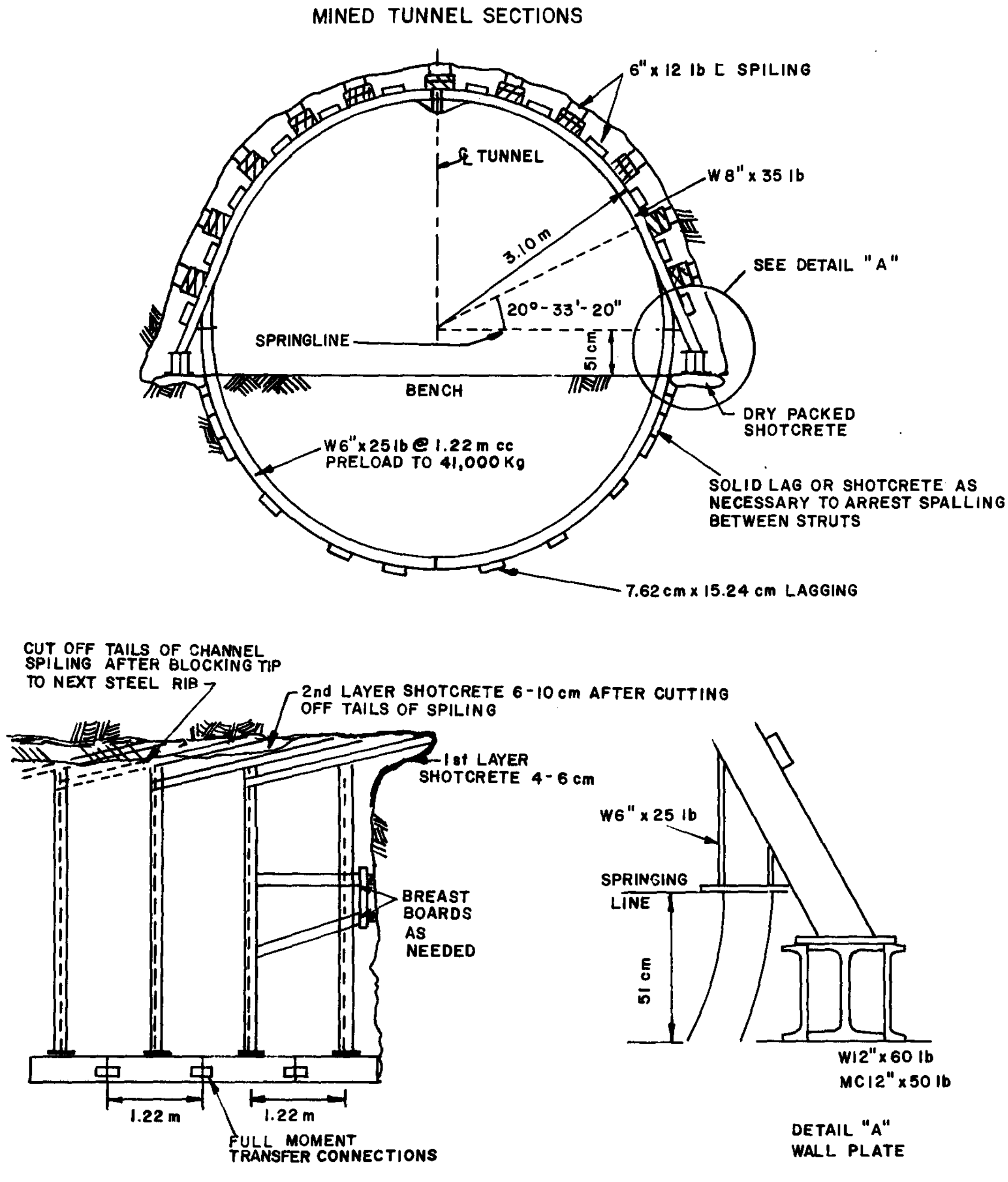
NOTES: 1 in. = 2.5 cm
1 ft = 0.3 m
1 lb = 0.45 kg

Figure 45. Support System Utilized in Tunnel Segments Mined With Tunnel Boring Machine, K-1 Tunnel
(Garbesi, 1979)



NOTES: 1 in. = 2.5 cm
1 ft = 0.3 m
1 lb = 0.45 kg

Figure 46. Top Heading and Bench Support System Utilized in Inbound West Mixed Face Segment, K-1 Tunnel (Garbesi, 1979)



NOTES: THE FIRST 100 ft, TOP HEADING SUPPORT SYSTEM WAS THE SAME AS INBOUND WEST MIXED FACE SEGMENT.
1 in. = 2.5 cm
1 ft = 0.3 m
1 lb = 0.45 kg

Figure 47. Top Heading and Bench Support System Utilized in Outbound West Mixed Face Segment, K-1 Tunnel
(Garbesi, 1979)

Case 4M - Fairmont Hill Tunnel

The Fairmont Hill Tunnel was driven through a faulted zone of the Franciscan formation as a part of the Bay Area Rapid Transit (BART) project. The tunnel is 20 ft in diameter and under a maximum cover of 140 ft. Excavation proceeded with a tunnel boring machine (TBM), and the support system included "I" steel ribs on 2- to 4-ft centers. The material in the fault zone was a decomposed serpentine characterized as a saturated, swirly, sheared fibrous mass of green-to-black earthy material (Myer, et al, 1977). The stand-up time of the tunnel face in this material ranged from one to six hours. The cutter head of the tunnel boring machine persistently clogged in this material, thus, the advance rate was reduced. As the stand-up time was exceeded, more and more material collapsed into the tunnel. A 10-ft by 12-ft cavity in the crown and a 6-ft cavity in the tunnel wall were formed. Cribbing was used to support the crown cavity, while cribbing and shotcrete were required to stabilize the tunnel wall cavity.

Case 4N - Kensico and Tygart Tunnels

Proctor and White (1968) cited two examples to illustrate that the heading and bench method is used where rock conditions will not permit full-face operations, i.e., the stand-up time is too short. The Kensico Tunnel was built as a part of the Delaware Aqueduct. The 24-ft diameter, full-face, hand-dug tunnel headed in a fault zone. Fairly sound rock was encountered in the roof, but the bottom was in fairly dry crushed rock. Side pressures developed and the bottom proved too soft to sustain the ribs. The tunnel was enlarged to use full circle ribs for support and make room for a 1-in. thick interliner. Two weeks later, the full heading encountered jointed and partly decayed-to-crushed and decayed gneiss in faults which necessitated tunneling by the top heading and bench method.

On the Tygart River Dam project, a 22-ft wide and 31-ft high tunnel was driven. Due to bad roof conditions caused by cemented gravel in some portions, and a thin stratum of sandstone separated by immature shale in other locations, the shale provided little resistance to slippage between layers; thus, top heading and bench methods were used. The top heading was holed through before the bench was taken out. Rib, wallplate, and post-type supports were used. On the same project, a few hundred yards away, a tunnel was driven full-face without difficulty because the crown was located in a thick sandstone layer which eliminated the stand-up time problem.

Based on this review of tunneling cases in loosening and crushed rock, evidence indicates that if appropriate excavation and ground support operations can be completed within the range of the stand-up time for a tunnel heading, there should not be a time-dependent stability problem. Thus, stand-up time is the main factor for tunnel construction in these types of rocks. For crushed rock, the existence of a groundwater head also can influence stability and stand-up time because the crushed rock can change to flowing or running ground after saturation. When running ground is encountered, it always presents a problem that must be coped with immediately. Basically, there are two construction procedures to increase the stand-up time in loosening and crushed rock (Myer, et al, 1977). They are: (1) Changing excavation procedures, and (2) reducing excavation size.

4.5 OTHER PROBLEMS ASSOCIATED WITH ROCK TUNNELING

When heavy, wet, running ground is encountered, there is a temptation to stop the tunneling operation to install perforated pipes allowing the material to drain, or to seal the tunnel face and stabilize it by grouting. Usually these procedures have little success because of time limitations imposed on these operations. Nevertheless, many battles are won in tunnel sections of running ground because of natural drainage not because of the aforementioned procedures. The gravity removal of fine-grained materials can gradually decrease the running tendency in residual fragmental rock material. Also, adjustment in the adjacent rock tends to fill openings left by running ground and may improve stability. In general, if the constant condition permits, installation of totally confined support should be delayed as long as possible where there may be short-term and long-term dislocations of materials into a tunnel opening.

There is no known method for estimating exact locations and volumes of underground water flows that might be encountered in rock tunnels below the groundwater table. A heavy inflow of water into tunnels sometimes is associated with fissures in brittle, competent rocks and, in many tunnels, is encountered in the least expected places. Karstic limestone is another setting for possible large inflows. Depending on the cross-section area of the tunnel and the degree of probability that groundwater might be encountered, feeler holes should be drilled ahead of the tunnel heading.

Gas outbursts occur during tunneling when gas under pressure is liberated in substantial quantity. The event is often accompanied by a rock failure which in many instances is violent. However, gas outbursts are often experienced in coal mining, and frequently in salt, potash, and other evaporite mineral deposits. If gas occurs in a separation, or parting, overlying or underlying an opening, a load is imposed on the roof or floor member which may cause a roof or floor failure. To eliminate this hazard, regularly spaced bleeder holes should be drilled in the roof or floor to relieve the pressure.

A variety of gases of natural origin have been encountered in tunnels, e.g., carbon dioxide, methane, sulfur dioxide, hydrogen sulfide, hydrogen, and radon. There are many devices available for monitoring tunnel air for undesirable constituents. Effective procedures to deal with the flammable or toxic gas problem in a tunnel are: (1) Monitoring the concentrations of harmful gases at all times during tunnel operation and either removing or diluting the concentration of harmful gases in order that the environment not be toxic to the workers; (2) maintaining a high air velocity in the tunnel; and (3) in the event of any harmful gas reaching the intolerable level, the operation should be stopped and all personnel should retreat from the tunnel until the air in the tunnel cleans up.

In this section, some typical water, gas, and other problems which occurred during rock tunnel construction are reviewed.

Case 40 - Navajo Indian Irrigation Project

During the excavation of Navajo Tunnel No. 2 (1965) in New Mexico, a high underground water flow was one of the main problems. Up to 7200 gpm at a pressure of 395 psi was encountered at a part soft material and part rock mixed-face location. This caused a major problem with the support system. In another Navajo Tunnel, there was a major cave-in when the squeezing shale came into the tunnel opening during an attempt to strengthen the support system. Although moving shale was not a constant problem, there were areas where the shale was 5 to 6 ft above the tunnel springline. Four-inch I-beam arch supports with protective lagging were used in these areas (Wahlstrom, 1973; Shea, 1976).

Case 4P - Tehachapi Tunnel No. 3

The Tehachapi Tunnel No. 3 is located in the Tehachapi Mountains and is part of the California Aqueduct System (Peters, 1972). The tunnel is 5400 ft long with a 24-ft finished diameter. The rock mass contains granite, gneiss and quartz diorite with numerous shears and faults with gouge material. The tunnel was excavated by the full-face method until weak material was encountered in fault zones. A top heading and bench system was then used. When the top heading entered the hanging wall in a fault zone, high water flow and very unstable, blocky material along with gouge material were encountered. A 4-ft by 5-ft pilot bore was begun, but the ground was still unstable. Finally, the top heading was advanced with the aid of crownbars and spiling (Myer, et al, 1977).

Case 4Q - Tonner Tunnel

Tonner Tunnel (1972) is located in Southern California. The rock mass 300 ft from the heading was classified as gassy. This tunnel was the first to be built after the Sylmar Tunnel disaster in California, so consequently, the safety requirements were stringent. Everything in the gassy area of the Tonner Tunnel had to meet the Bureau of Mines' "Permissible Classification." The major safety items required in the specification were:

1. Gasoline-powered equipment was not permitted.

2. Gas testings were required in the tunnel at all times when mining was in progress.

3. Installation of automatic gas-monitoring equipment which would give an alarm and shut down the electrical equipment at ten percent of the lower explosive limit of methane.

4. A minimum air velocity of 100 fpm was required in the tunnel.

The tunnel was mined with a 50-ton Calweld boring machine, and no major problems from flammable or toxic gases were experienced (Shea, 1976).

Case 4R - Addison to Wilmette Tunnel

The Addison to Wilmette Tunnel is the most northerly portion of the Metropolitan Sanitary District of Greater Chicago's Tunnel and Reservoir Plan (TARP). This project includes 24,020 ft of 22-ft-diameter and 27,750 ft of 30-ft-diameter rock tunnels. The tunnel is about 200 to 250 ft from ground surface in a dolomite limestone formation. Most of the running tunnels were mined by tunnel boring machines and were not lined with concrete (Mixon and Kennedy, 1979).

It was noted that during the initial excavation period, the penetration rate of the 22-ft-diameter machine was quite low when compared with a machine of comparable size, power, and thrust. Through the contractor's investigation it was revealed that the loading-per-disc, although closely spaced, was too low to effectively penetrate the Chicago area limestone. The multiple disc cutters were converted to single disc cutters and the cutter spacing was stretched from 3 in. to 3.5 in. The entire conversion was done in 3 weeks inside the tunnel. After the conversion on this 22-ft-diameter machine, the penetration rate increased from 5 to 10 ft per hour for the remaining 18,000 ft of tunnel. Although the total available thrust could not be utilized because of torque limitations, the performance increase was satisfactory enough to the contractor.

For the 30-ft-diameter machine, at approximately 3000 ft in from the main shaft, a full face of clay was encountered for a distance of 30 ft. About two months were spent to pass through this area. The actual mining required about half of this time, the other half was used to investigate the approach to this problem, to negotiate with the owners, and to procure necessary supplies. It was decided to hand-mine the top heading by using steel supports placed outside the tunnel line. After the crown was secured, full circular steel supports were placed under the original crown supports as the machine advanced. This machine encountered a few other clay areas, but they occurred only in the crown of the tunnel which was secured with an umbrella of half-circle steel sets supported by wallplates pinned at the springline of the tunnel.

4.6 SUMMARY AND CONCLUSION

Most of the typical rock tunneling problems and the appropriate excavation methods and support systems to combat these problems are reviewed in this chapter. For the purpose of a clear discussion, rock tunneling problems are categorized in three groups. They are: time-dependent problems such as squeezing rock; initial stand-up time problems such as loosening rock; and tunnel construction problems such as safety. The main findings in this chapter are summarized below:

1. It is unrealistic to assume that the encountered conditions will be the same as predicted when exploration is limited to surface mapping and portal boring. Vertical subsurface borings along the tunnel routes and sometimes pilot bores along the tunnel alignments should be performed. The information from this program should be sufficient for designing and estimating the initial support systems as well as evaluating and selecting excavation methods.

2. For squeezing and swelling rocks, the inward ground movement rates are largely dependent on local geologic conditions. It is difficult to predict either the total movements or magnitude of the rock load. Thus, the appropriate method for dealing with squeezing and swelling rock in tunnel openings would be to allow the material to move, under controlled conditions, into the opening with the expectation that the movement will diminish and eventually stop.

3. For predominantly swelling rock, if the tunnel opening can be kept sufficiently dry and the rock surface is properly sealed, the creation of swelling pressure may be prevented effectively.

4. For loosening rock, the stand-up time is the main factor of concern. If an appropriate excavation and adequate support operation can be completed within the range of stand-up time for a tunnel head, there should be no time-dependent problem.

5. For crushed rock, the existence of a groundwater head can have a strong influence on its stability and stand-up time because crushed rock will change to flowing ground after saturation. Therefore, positive control of groundwater head is needed in this type of rock.

6. If geologic studies indicate the presence or near presence of known or probable active faults, one should expect strong active residual stresses in the rock. In areas where enormous past loading has occurred, such as thousands of feet of glacial ice, one should expect residual stresses that may result in strong sudden releases, e.g., rock bursts.

7. There is no known method for estimating exact locations and volumes of underground water flows that might be encountered in rock tunnels. If groundwater might be encountered, feeler holes should be drilled a substantial distance beyond the tunnel heading.

8. A variety of gases of natural origin have been encountered. However, if adequate precaution is taken, the gas problem can be eliminated effectively during tunneling.

5.0 SOME EFFECTIVE CONSTRUCTION PROCEDURES FOR PROBLEM TUNNELS IN ROCK

5.1 INTRODUCTION

As reviewed in Chapter 4, the basic tunneling problems in rock are time-dependent related to initial stand-up time and methods of tunnel construction. In order to conquer these difficulties, advanced tunnel machines and innovative tunnel construction methods have been developed in recent years. However, those effective machines and techniques are sometimes very sensitive to the tunneling environment. They usually are cost-effective only in certain combinations of geological conditions, construction constraints, and availability of the special working crews.

In this chapter, some typical case histories of effective tunneling techniques are reviewed. In each individual case, rock conditions and some important construction procedures are described. Finally, the advantages and limitations of each technique are discussed.

5.2 MECHANICAL TUNNELING

The tunnel boring machine (TBM) can be an effective piece of tunneling equipment in ideal rock conditions. Such optimum conditions include good rock material characteristics (medium strength, high uniformity, moderate hardness, and low abrasivity), good rock mass characteristics (few joints, absence of faults and shears, and unweathered), and good geologic environmental characteristics (low hydrostatic pressure, moderate in-situ stress, and absence of gas). Such idealized rock conditions can eliminate support problems, fallout at face and grippers, frequent cutter changes, as well as water and gas problems. The advance rates normally can reach 150 ft per day (Deere, 1981). Medium strength means the strength should sufficiently resist the gripper pressure and also prevent stress slabbing, fallout, and squeeze at the tunneling depth. The desirable in-situ stress is when the level of stress is sufficiently high to hold the rock mass together tightly but not so high to cause spalling or squeezing.

The idealized rock conditions described above are seldom encountered in nature. Adverse geological conditions associated with the operation of tunnel boring machines have been discussed in detail in Chapter 4. In this section, the requirements of subsurface explorations and ameliorating measures to reduce the impact of unfavorable rock conditions for tunnel boring machines will be discussed.

Subsurface exploration for machine tunneling should be more extensive than that usually done for conventional tunneling. This is because a large initial investment is required for machine tunneling, and if the rock conditions are not amenable to machine boring there will be further delay and cost in changing over to conventional tunneling. As indicated by Deere (1981), average boring spacing of 1500 ft to 3000 ft would be adequate for long and deep pressure tunnels on a hydroelectric project. For short, shallow tunnels (such as for subways and utilities), average boring spacing should be about 150 ft to 300 ft where the overburden depth and rock weathering are irregular. Leonard (1981) proposed that adequate borehole spacing be 2.75 times the distance between the top of the rock and the tunnel invert.

The rock cores should be logged carefully to include the description of joint and fracture spacing, grain structure, and grain size. Sufficient core samples of each rock type to be tunneled should be tested for unconfined compressive strength and total hardness. (Total hardness is the combination of an abrasion wheel index and the Schmidt hammer hardness.) Then, the correlations between total hardness with penetration rates, cutter types, and costs can be made (Tarkoy, 1979). The compressive strengths have value for comparing rocks of the same type. However, for different rock types, such as limestone with a granite, misleading conclusions could be reached on the basis of compressive strengths only.

The tunnel boring machine is a specialized tool and not applicable to all jobs. Leonard (1981) listed job conditions that limit the economic use of the tunnel boring machine:

1. The machine should provide a thrust per cutter of at least twice the compressive strength of the rock. For granite rocks, a thrust up to three times the rock strength may be needed.

2. The intact rock strength should be at least three times the vertical pressure by the overburden. Otherwise, the machine may become stuck in the rock formation.

3. If the RQD over a significant distance (around 50%) of the project is less than 50, even if no major fault zone exists, the tunnel boring machine may become support-bound. When a large percentage of time is spent waiting for the mined rock surface to be supported, the tunnel boring machine cannot perform well.

4. Delivery of a new machine usually takes 12 to 15 months.

5. It may not be economical to use a tunnel boring machine for a tunnel less than two miles long.

Even when the geological formations are well defined and the aforementioned job conditions are fulfilled, it may still be impossible to predict how the formations will react when penetrated by a tunnel. When the reactions are adverse, there are three alternatives available: (a) Fight through the formation with no change of the tunnel boring machine, (b) modify the machine, or (c) park or pull the machine and mine tunnel conventionally (Shea, 1981). Item (c), the conventional tunneling method in difficult rock formations, has been discussed in Chapter 4. In this section, only those case histories categorized in items (1) and (2) are summarized in the following paragraphs.

Case 5A - Tunnel Boring Machine in Fault Gouge Rock Formation

The fault gouge zone shown in plan view in Figure 48 was encountered by a 16.5-ft tunnel boring machine with a long roof shield. The gouge zone was 25 ft wide, vertical, and ran at an acute angle to the tunnel alignment. It contained completely weathered, weak, wet earthy material with occasional blocks of mudstone or sandstone. The rocks on either side of the zone were competent and fresh, and the sandstone had a quartz content of 90 to 100 percent and a compressive strength in excess of 36 ksi.

The machine drove into the gouge without major difficulty. As the heading advanced to chainage 15 m, the main rear anchor pads and support platform came into the gouge zone and a partial collapse of the tunnel roof and walls induced the following difficulties:

1. The anchor pads could not grip the very weak tunnel walls and slipped continuously. This was partially remedied by timber packing, but penetration rates were limited to 0.5 to 1.5 ft/hour.

2. Loose material 5- to 10-ft above the long roof shield was removed from the support platform while the roof was being supported by rockbolts and mesh.

3. The face mucking chutes and conveyor belt had to be cleared of blockages, and hand mucking was carried out on both sides of the machine.

As the driving continued, the extra thrust required to cut a full face of the sandstone accelerated slippage of the gripper pads, particularly on the left-hand wall. This was built up with cast-in-place concrete. Slow driving continued to the support platform past the gouge zone. It was found that the machine was out of alignment, horizontally and vertically, by approximately 1.5 ft.

The delay could have been minimized if the machine had allowed access for roof support and for the walls to be built up shortly behind the cutter head. This case history also illustrates the importance of the width of the fault zone. Subsurface investigation should provide such data, particularly where the fault zone may be gouge filled (McFeat-Smith and Tarkoy, 1980).

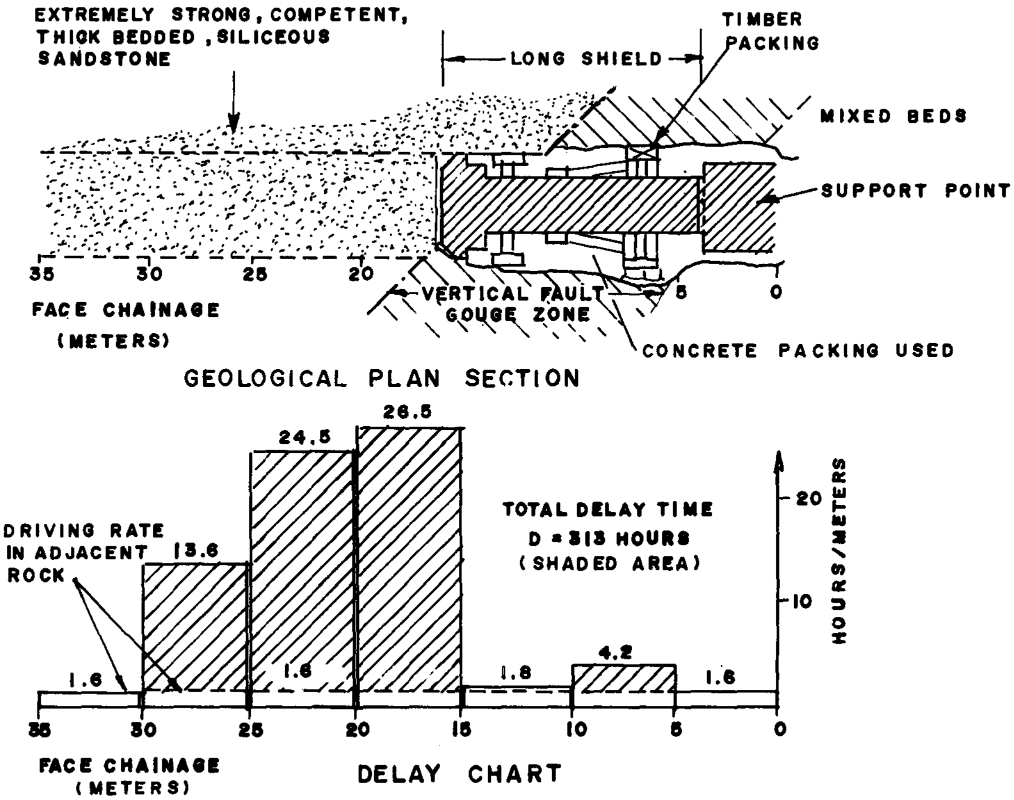
Case 5B - Tunnel Boring Machine in Intensely Shattered Rock Formation

As shown in Figure 49, an intensely shattered zone was encountered between two minor faults. The sandstone on both sides of this zone was thickly bedded, fresh, moderately strong with wide and tight joints. In the fault zone, the same sandstone was reduced to a highly weathered, very weak, close to very closely jointed (generally less than 6 in.) wet, clayey, sandy and blocky material. The overlying mudstone was in a similar shattered state. When excavated, these materials tended to fall under their own weight.

In this tunnel, the machine employed had a short roof shield and steel arches, and a Bernold sheet could be placed immediately behind this. With this method, overbreak was limited to 1.5 to 2.0 ft. The delay chart shows only 76 hours were spent in this 75-ft long shattered section and the operation can be regarded as a success (McFeat-Smith and Tarkoy, 1980).

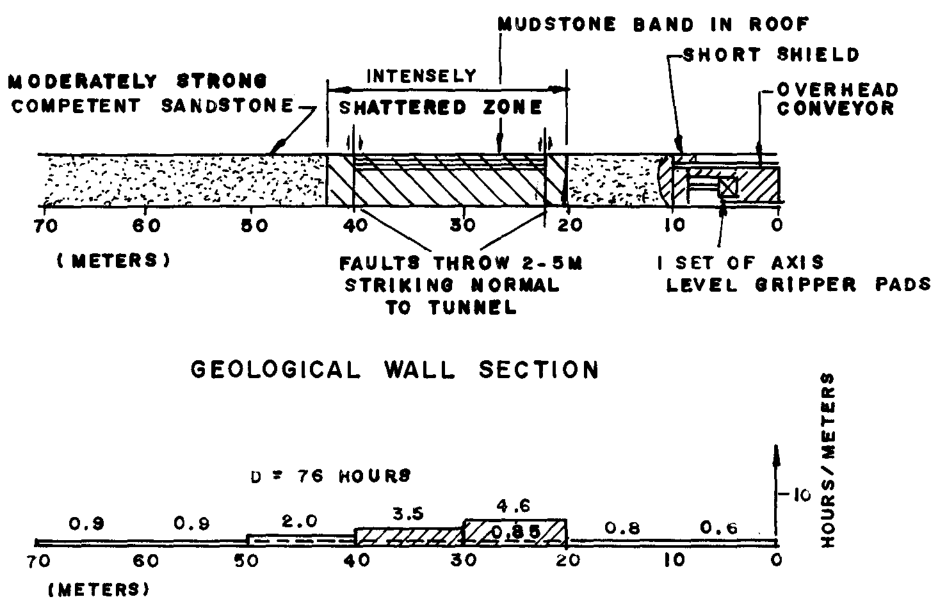
Case 5C - Tunnel Boring Machine in Continuous Minor Faulting Rock Formation

The geological wall section in Figure 50 shows a 300-ft section of tunnel intersected by six minor (6- to 13-ft dislocation) faults and three shattered zones. The strata encountered here consisted of a thickly bedded, slightly weathered, strong, widely and tightly jointed sandstone overlying a 3- to 7-ft-thick, very weak, earthy material. This material is underlain by mudstone. Near faults the earthy material was in a slumping condition, while the mudstone was generally shattered. Water inflow in this region was about 185 gpm, and was concentrated mainly at the faults and shattered zones.



NOTE: 1m = 3.3 ft

Figure 48. TBM in Fault Gouge Rock Formation (McFeat - Smith and Tarkoy, 1980)



NOTE: 1m = 3.3 ft

Figure 49. TBM in Intensely Shattered Rock Formation (Mc Feat - Smith and Tarkoy, 1980)

At chainage 16 m, the driving head encountered high water inflows with unstable roof conditions. As driving advanced to 19 m, earthy material fell from the roof causing partial collapse at the face and walls together with high water inflows generating the following effects:

1. Steel arches and sheet packing were used as the main form of support.

2. Timber packing was required for the gripper pads.

3. Hand mucking was carried out at the sides of the machine.

4. The continuous high water inflows created unpleasant working conditions which resulted in labor problems.

When the heading reached 28 m, the machine came in contact with earthy material and mudstone beds. The machine sank about 1 ft below the grade. Following a two-week holiday and a one-day jacking attempt, the machine had sunk an additional 1 ft and was 2 ft below grade. At this stage, it was realized that a more substantial approach was required. The machine was pulled back from the face, and the floor was cleared of debris and concreted. Four 6- to 10-ft-long track rails were set in the concrete to give a sharp rise in the face. Driving progressed with the machine tilted upwards and the grade was raised by 1 ft at 30 m and finally back on target around chainage 50 m.

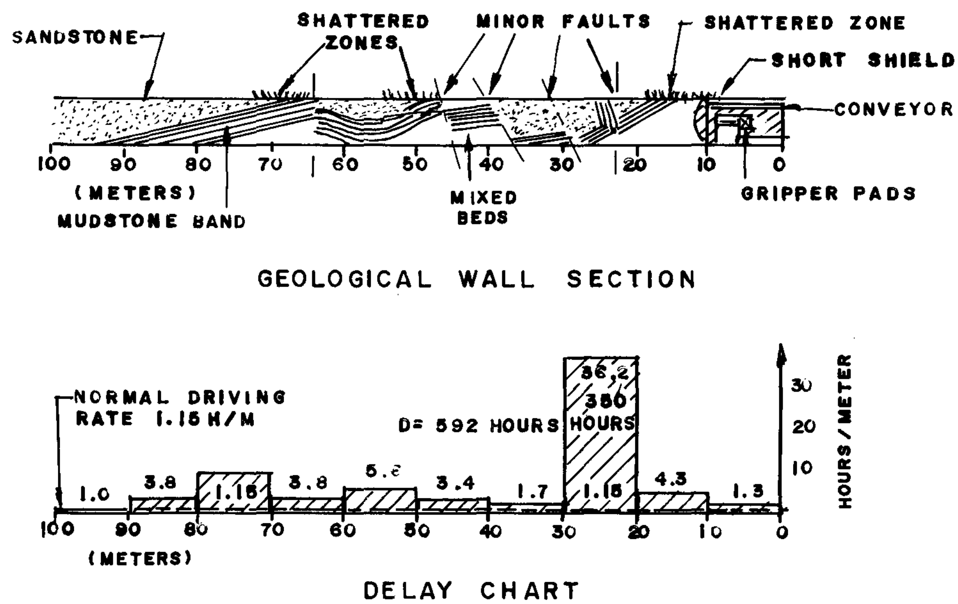
Further driving in this faulting section was more successful. The principal operations required immediate bracing and local support. Overbreak in the walls and roof varied from 0 to 6 ft; however, the experience gained by the contractor allowed him to minimize the steering problems (McFeat-Smith and Tarkoy, 1980).

Case 5D - Tunnel Boring Machines in Open Joints Rock Formation

As shown in Figure 51, open joints were encountered in the sandstone roof. The sandstone was massive, slightly weathered, strong, and contained an occasional 1.5 in. of weathered shaley bands. Underlying the sandstone was a thinly bedded mixed bed. The jointing in this zone consisted mainly of vertical perpendicular joint sets intersecting the tunnel alignment at about 45 degrees. The joint spacing averaged 1.5 ft, and in the sandstone one major set was open 0.5 to 2.5 in. and clay-filled, while the other major set was open 0.1 in. and partially clay-filled.

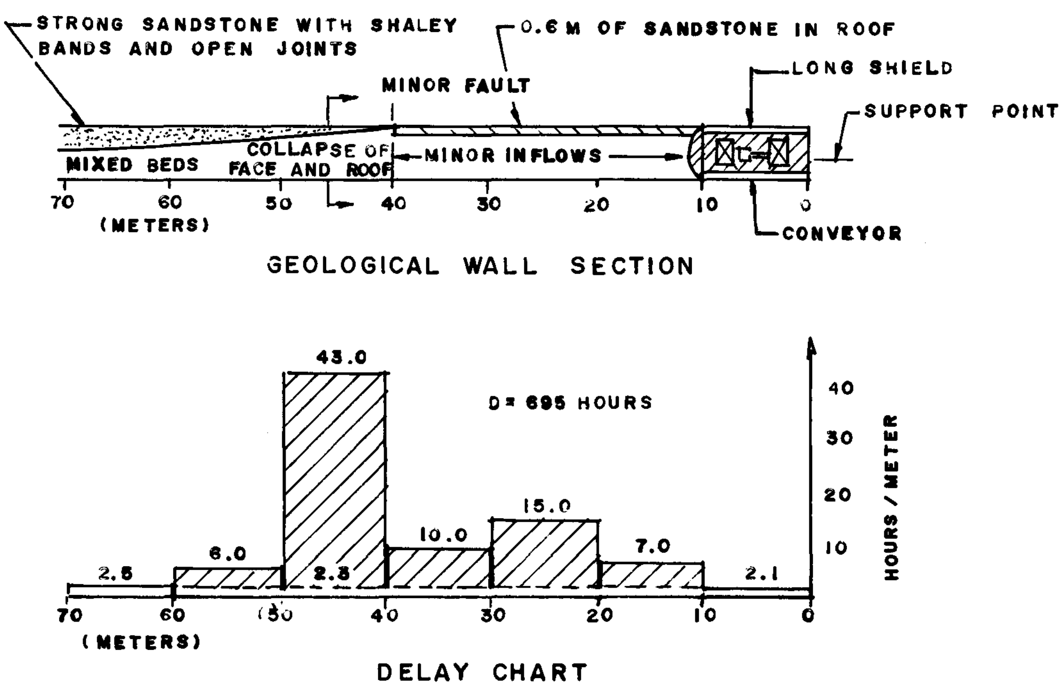
The machine initially encountered joint block overbreak, particularly on the shoulder and also some locally on the roof. This was caused by a lack of cohesion in the open joints and weathered shaley bands. At 15 m, a large sandstone block fell from the face, jamming the cutter hand and mucking system. The machine had to be pulled back about 13 ft, and the blocks blasted and cleared by hand.

From 20 m to 39 m, minor water inflows were encountered and overbreak up to 7 ft occurred locally on the roof and shoulders. At 39 m, a minor fault intersecting the tunnel's normal to moderate water inflows (approximately 100 gpm) was encountered. When the machine cutter head reached 45 m, the fault induced continuous overbreak in the sandstone roof and shoulders along the length of the shield, and a complete fallout at the face which jammed the cutter head. A time span of 340 hours was spent to form a 5-ft path above the roof shield for access to the heading and to hand muck sandstone blocks and support the walls.



NOTE: 1m = 3.3 ft

Figure 50. TBM in Continuous Minor Faulting Rock Formation (McFeat-Smith and Tarkoy, 1980)



NOTE: 1m = 3.3 ft

Figure 51. TBM in Open Joints Rock Formation (McFeat-Smith and Tarkoy, 1980)

From 45 m to 60 m, the water inflow decreased and driving improved gradually. Special facilities were made to allow rock-bolting through slots in the roof shield at about 6 ft behind the head. In spite of the crowded conditions, this operation helped to minimize the overbreak.

In this case, the combination of minor geological conditions, i.e., open joints, weathered shale, water inflow, and faulting may create difficult driving conditions (McFeat-Smith and Tarkoy, 1980).

Case 5E - Buckskin Tunnel

The Buckskin Tunnel project is part of the Central Arizona Plan that will bring Colorado River water to and beyond the Phoenix area. The project consisted of constructing a length of 35,900 ft of 22-ft finished diameter tunnel. The tunnel boring machine, with segmented concrete liners, was used for mining (Shea, 1981).

Prebid geological information indicated that the tunnel would be driven through andesite, agglomerate, and volcanic tuff. The andesite had compressive strengths in excess of 40 ksi. Other tests showed that the rock was brittle and there were numerous fractures in the rock formation. A shielded tunnel boring machine was built on the assumption that the fractures in the formation would help the rock cutting.

The tunnel was driven some 1400 ft without major problems and then came to an abrupt halt. Those fractures that were assumed to help cut the rock were so open and vertical that they caused rock falls and created a huge overbreak. Core borings from the ground surface were conducted, and it was found that the blocky rock was not in a localized area. Cement grout was tried to stabilize the ground. Check holes indicated that the grout was not consolidating the rock.

Finally, considerable modification of the cutter head (false front on the head) was made and it was successful. The tunnel was completed approximately two years later (1980).

Based on a review of the above five case histories and discussions of this section, some of the main points pertaining to the performance of tunnel boring machines in rock can be summarized as follows:

1. In competent rock considerable advantages can be gained by use of the tunnel boring machine in terms of increased progress rates, reduced labor, overbreak, and support costs.

2. Even in the most stable geological regions, occasional fracture zones may occur.

3. The locations of these zones along the tunnel alignment must be identified and their impacts on the performance of a tunnel boring machine assessed. The machine performance rate depends largely on the boreability (penetration rate and tool wear) and utilization (geological formation, tunnel support system, and muck system).

4. Progress can be improved through a more flexible tunneling system: (a) short roof shields with facilities for the installation of a range of temporary support types immediately behind the heading, (b) easy access to the gripper pads for hand mucking and bracing in broken ground, (c) easy access routes through the cutter head to minimize work delays at the tunnel face, and (d) provision of probing facilities if substantial lengths of soft ground may be encountered.

5. More extensive site investigation should be performed for machine tunneling than for conventional tunneling since the design and performance of a tunnel boring machine is heavily dependent on the rock strength, rock hardness, joint spacing, joint width, etc.

5.3 ROCK PREREINFORCEMENTS

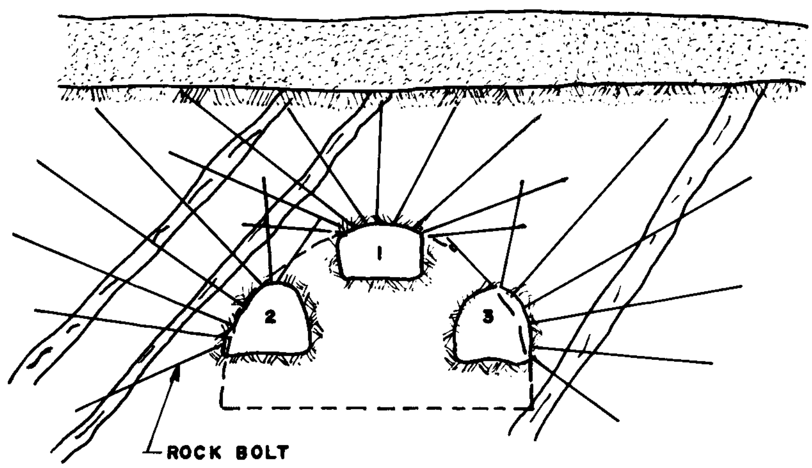
Rock masses are quite strong if progressive failure along the discontinuities of low strength are prevented. It is the purpose of the prereinforcement to prevent this failure, thereby allowing the rock to support itself with its inherent strength. Prereinforcement generally involves placing in predrilled holes, untensioned steel members such as reinforcement bars ahead of the tunnel excavation. The prereinforcement can also be fully grouted or pretensioned when capsules are used. The purpose is to improve the rock mass in terms of stand-up time by preventing loosening and to contribute to the permanent stabilization of the opening by restricting deformations.

The use of rock bolting for permanent support was advanced by the construction of the Snowy Mountain Scheme in Australia between 1952 and 1962. A 60-ft span underground powerhouse had been supported initially by steel ribs. For final support, concrete arches were supplemented which cracked as the walls yielded inward when the excavation was deepened. When field measurements showed rock bolts could be an effective control for the yield in both arch and walls, bolts successfully replaced both the steel ribs and the concrete arches in latter stages of the work. In this project, a systematic pattern of prestressed and grouted rock bolts were used.

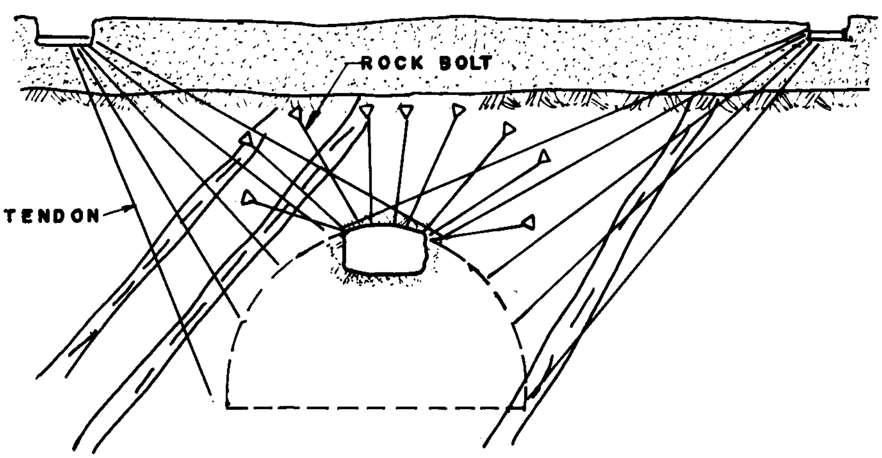
If bolts alone are used for final support of the large shallow rock chamber, it is necessary to install rock bolts of significantly greater length than commonly used in deep chambers. Bolt lengths in deep chambers in good quality rock are typically on the order of one-third the width of the chamber. To support all wedges in a shallow chamber, the bolts will have to be longer and tied back laterally. As shown in Figure 52, tendons could be installed either from side drift or ground surface prior to opening the full width of the arch (Cording and Deere, 1972).

Rock bolts are sometimes difficult to utilize in badly broken or weak rock formations. The intake tunnel of Eklutna Dam near Anchorage, Alaska was constructed in mostly sound argillite during 1952 to 1965. A 1-in.-diameter expansion shell type of rock bolt was used during the early stages of excavation. Due to numerous seams, thin bedding planes, and poorly cemented joints, rock bolts proved to be ineffective and were discontinued (Department of the Army, 1975). It is questionable whether expansion shells were the proper type of anchor for the ground at this location.

For the south bore of the Eisenhower Memorial Tunnel, summarized in Chapter 4, spiling reinforcement was tested and extensively used. A total of six test stations, each consisting of one to five instrumented reinforcement bars (a total of 18 installed) and a



(A) INSTALLED FROM DRIFTS



(B) INSTALLED FROM GROUND SURFACE, PRIOR TO EXCAVATION

Figure 52. Rock Bolt Support of Shallow Chambers (Cording and Deere, 1972)

strain gauged steel set, were established in a rock formation ranging from slightly blocky rock to moderately squeezing rock. Through this field investigation, Korbin and Brekke (1978) have the following major findings:

1. Deformation induced considerably more tension than bending in the spiling reinforcement. Compared to the traditional forepoling, the mechanism of spiling is more effective in providing the immediate and permanent stabilization of a tunnel opening for most of the rock formations.

2. Formation of a stable reinforced arch is basically a self equilibrating process. The required thickness of the arch is strongly related to rock type and construction method, and less to the opening size, shape, and depth. The range of thickness, depending on rock type, was from 3 ft to 8 ft. Arch capacity related to induced forces within the reinforcement is highly dependent on the opening size, shale, and initial state of stress. Increased capacity can be attained through an increase in the size or number of reinforcement bars. In order to reach an effective design of the prereinforcement system in a given tunnel environment, either instrumented reinforcement or extensometers should be employed to monitor the behavior of the reinforced arch and to check the design.

3. The long-term measured rock loads were less than 25 percent of the anticipated rock loads (from Terzaghi's rock load table). Thus, the rock mass reinforcement system was the primary factor in the permanent stabilization of the tunnel opening, whereas the internal support system performed in a secondary role for the control of rock loosening.

In summary, based on review of the above case histories, the rock bolting system is effective in improving the stand-up time during tunneling, and in providing temporary and permanent support for tunnels in rock. However, the prereinforcing may not be effective in badly broken rock formations. Besides, due to the inherent complexity of the nature of rock formation, instrumented reinforcement or extensometers should be used to reach an economical design of the rock reinforcement system in a given tunnel environment.

5.4 SHOTCRETE LININGS

Recent experience around the world has shown that shotcrete can save large amounts of money in tunnel construction. In many cases, shotcrete can provide a finished tunnel lining that is superior to any other design. However, like rock bolts, the application of shotcrete has not reached the stage where it is possible to effectively design a shotcrete lining for structural support. Note that great care should be exercised in water conveyance tunnels if shotcrete is to be used as a final lining. It is basically unsuitable if swelling clay minerals are present or if continued squeezing is anticipated.

Shotcrete behavior observed in transportation tunnel construction in the United States has been primarily in loosening rock formations. In such conditions, rock loads develop from the self-weight of individual rock blocks that tend to loosen from the wall and roof of the opening. The objective of the shotcrete in loosening rock formations is to provide support early enough to minimize loosening that would cause instability in the tunnel heading or apply excessive load to the final tunnel lining. In contrast to these conditions, the function of the shotcrete in squeezing and swelling rock formations is to seal the exposed surface against excessive moisture and to allow controlled movements so that high lining stresses do not develop. The use of this type of shotcrete and other support elements in squeezing and highly stressed rock formations is often associated with the New Austrian Tunneling Method (NATM). This technique will be discussed in Section 5.5.

The present methods for selecting thin shotcrete tunnel linings in loosening rock conditions are largely based on empirical rules formulated from experience. In order to improve the methods of selecting shotcrete linings, laboratory testings, field observations of shotcrete behavior, descriptions of failure modes, and the influential rock conditions are summarized in the following paragraphs.

Large-scale model tests to evaluate the structural behavior of thin shotcrete linings were carried out (Fernandez, Mahar, and Parker, 1977) to study, (a) the geometrical configuration of the simulated rock surface, (b) the end condition of the shotcrete layer, (c) the thickness of the shotcrete, and (d) the bond strength between the shotcrete and the simulated rock surface. Basically four types of failure modes were observed: (a) Adhesive failure developed in poorly bonded shotcrete layers with unsupported ends; (b) shear failure developed in the thinner layers as the adhesive strength increased; (c) moment-thrust failure developed in supported end layers with a smooth arch configuration of the simulated rock surface; and (d) bending failure developed in end-supported flat layers and in end-supported arched layers over the protruding simulated rock blocks with low bond. On this basis, the performance of shotcrete tunnel lining in loosening rock formation can be quantitatively estimated.

Field studies were carried out in Washington Metro tunnels in blocky and seamy foliated gneisses and schists in which the boundaries of rock wedges were formed by continuous planar joints and shears. Observations were performed during the construction of 30-ft-wide and 22-ft-high double track tunnels (Mahar, Gay, and Cording, 1972). These tunnels are either supported with shotcrete and rock bolts or with shotcrete and steel ribs. The initial support consisted of approximately 2 in. of shotcrete sprayed over the crown and arch within one to two hours after blasting. Concurrently, muck was removed from the tunnel heading. When both operations were completed, the drill jumbo was moved up to the heading to install either the rock bolts or steel ribs. Rotation, outward displacements of a block, and slippage along the shotcrete rock interface were observed where the shotcrete was placed over irregular surfaces bonded by clean, smooth, planar, and often slickensided joint surfaces. Tunnel sections instrumented with strain gauges embedded in shotcrete have shown that tensile stress developed in the shotcrete placed over the smooth surfaces of protruding blocks. Thus, in these tunnels shotcrete was used primarily in combination with other support in (a) more than 20-ft diameter tunnels, (b) zones where blocks were bounded by smooth to slick joint surfaces; overbreak was prominent and block sizes were typically 4 ft or more in width, or (c) vertical side walls greater than 10 ft in height that were backed by steeply dipping joints (Fernandez, et al, 1979).

A shotcrete test program was carried out at a test site in the Peachtree Center Station in Atlanta, Georgia. The rock is a foliated granite gneiss with widely spaced joints and no sheared or planar joints in the test area. Foliation joints are tight, wavy, and very irregular. The rock surface in the cavern is dry and irregular, and does not need supports. Included in the test program were four 2-ft by 2-ft steel plates which were embedded in the shotcrete and pulled to evaluate the capacity of the shotcrete placed in situ, and to obtain a direct comparison with the laboratory test data. The results from all of the tests showed that the natural irregularities in a dry and clean rock surface can increase the adhesive strength several times beyond the values measured in the laboratory where shotcrete was placed over a concrete surface. The increase is larger for layers in arch configuration because compressive stresses tend to develop at the irregularities and the shotcrete must fail in shear (Fernandez, et al, 1979).

Based on the above review, the influential rock conditions affecting shotcrete loads and behavior are:

1. Geometry of the critical rock wedges requiring support.

2. Potential configuration of the tunnel surface.

3. Adhesion characteristics of the rock joints.

Although the guidelines for shotcrete use can be established, exact requirements are difficult to accomplish prior to construction. Minor differences in rock and construction conditions can result in support requirements. Thus, field observations in the early stages of the project are useful in developing support requirements. Even with the flexibility in shotcrete usage, the designer should properly assess the feasibility of the shotcrete lining and determine other types of support systems such as rock bolts and steel ribs which may be required to provide adequate support. Thus, characteristics of the rock must be carefully analyzed prior to construction.

5.5 NEW AUSTRIAN TUNNELING METHOD

The New Austrian Tunneling Method (NATM) was first named by Professor Rabcewicz, because the original idea and subsequent development of this method came from Austrian engineers during the last 30 years. The requirement for quick sealing of the squeezing and swelling rock surface, previously experienced, could be fulfilled only with the development of shotcrete. Following that was the finding of a possible reciprocal relationship between the required lining resistance and deformations in some rock formations by Fenner in 1938. In 1944 Rabcewicz found that the time-dependent behavior of the rock mass was fundamental for predicting the behavior of the tunnel support system. Development of the shear failure theory for tunnels under high overburden, the necessity for semirigid linings, the semiempirical design approach using in-situ measurements as an integral part of the technique, and the incorporation of rock and soil in the support system were the main subsequent steps in the development of the New Austrian Tunneling Method (Gosler, 1981).

The NATM is a tunneling concept of limited deformations where a new state of equilibrium after excavation is reached by controlled pressure release. In order to achieve this goal, a number of basic features have to be taken into account: (a) Consideration of the geomechanical ground behavior; (b) the most suitable shape of excavation; (c) avoidance of unfavorable stresses and deformations by means of suitable support works installed in the proper sequence; (d) optimization of the support resistance as a function of allowable deformations, and (e) control by deformations. The deformations of the rock should be controlled during excavation in such a way that they remain small in order to avoid a decrease of rock strength, yet large enough to activate the rock to form the load-bearing ring to reduce the external support requirement.

Some of the main advantages of the NATM are the adaptability to varying rock conditions, the flexibility of different shapes for Station or Switch sections, and better economy. Since the NATM is basically a hand-mined operation, it required a well-trained field staff and experienced working crews. Also, there are no definite design procedures for this technique. A combined effort among owner, designer, and contractor is needed throughout the duration of the project.

Following are case histories which illustrate application of the NATM under varying rock conditions:

Case 5F - Perjen Tunnel

In the 9450-ft-long Perjen Tunnel are contained 1950 ft of thick banded dolomites and limestone of the northern calcareous Alps and 7380 ft of quartz phyllonite and gneiss phyllonite of the Landeck quartz phyllite. In the area of the northern calcareous Alps, a large zone of fractured material was encountered extending more than 80 ft. This material was a fractured product of dolomite in a soft and plastic state. Partially open cracks were found and no rock pressure was observed. The fault zone was encountered at the interface between the northern calcareous Alps and the Landeck quartz phyllite. The tectonic processes have put the quartz phyllonite under stress affecting a length of about 600 ft due to the acute angle at which the schistosity planes crossed.

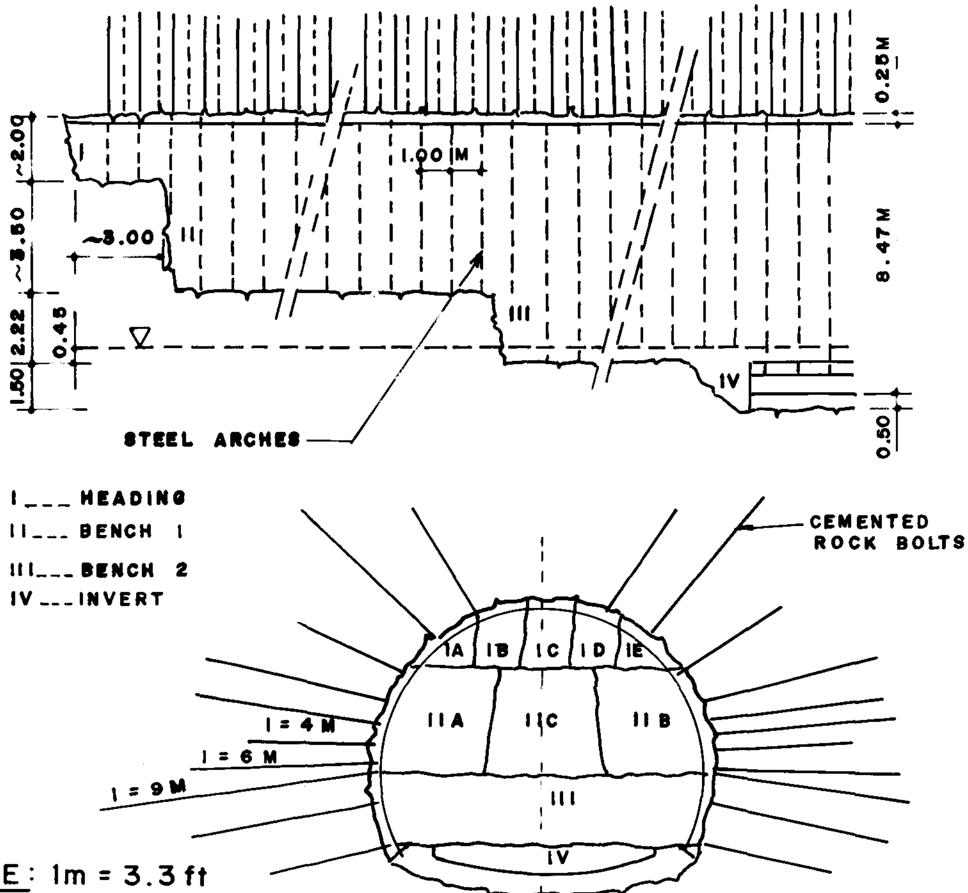
While driving in the fractured zone, only limited areas could be excavated without the spalling of materials from the face and sidewalls. As shown in Figure 53, a 6-ft-high heading was opened in sections 3 ft long and 6 ft wide. After each section was opened, steel rods were rammed ahead of the face, and steel wire mesh was installed behind the rods. A first layer of 6-in. shotcrete was applied at the sidewalls. At the same time, the face was sealed with 2 in. of shotcrete. After all parts of the heading were opened, the tunnel arches were installed and the shotcrete was increased to 10 inches. Bench one, following the heading at about 9 ft, was also excavated in sections with a maximum length of 3 ft. Bench two and the invert were excavated after the tunnel was cut through.

Driving in the fault zone was performed with a 10-ft-high heading with bench one following at 10 ft. Since large deformations of 0.4 in. per day were observed, 13-ft anchors in the beginning, and later 30-ft anchors, were installed between steel arches (Figure 54). The movement became slower, but remained at 0.08 in. per day. For this reason, it was agreed after six weeks to excavate bench two. In order to contain the movement during excavation, 30-ft rock bolts were installed from bench one downwards into the sidewall before removing bench two.

Case 5G - Pfander Tunnel

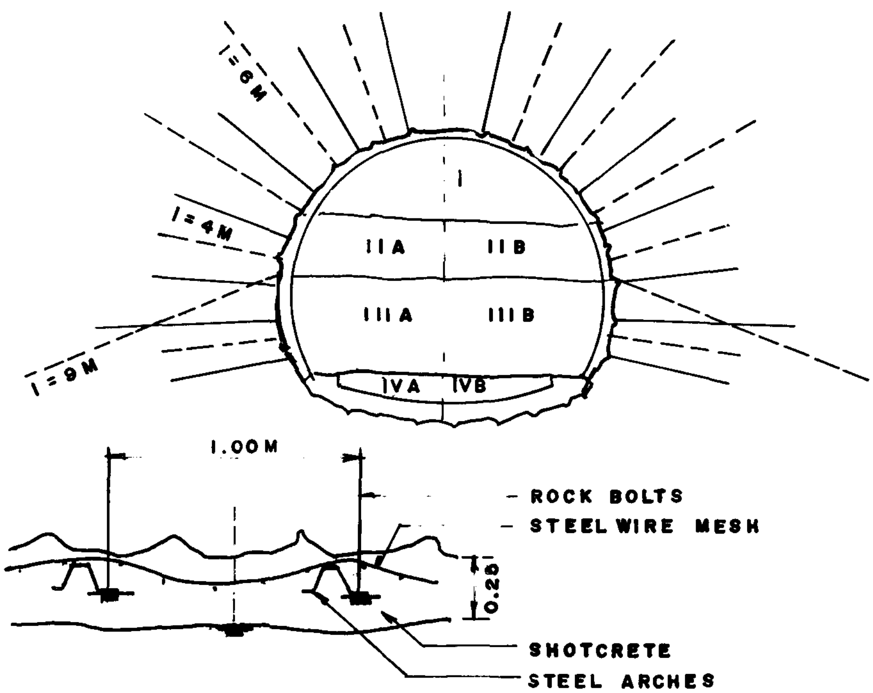
The Pfander Tunnel is a 6.7-km-long, 82 to 94 m^2 cross-section transportation tunnel, which is located near the Austrian-Swiss border. Ground encountered along the tunnel alignment is a dipping sedimentary sequence of conglomerate, sandstone, mudstone, and marl. Due to the presence of the clay mineral montmorillonite, swelling occurred and the New Austrian Tunneling Method (NATM) was used to prevent weathering of mudstone and marl, as well as to control ground deformations.

In order to immediately seal the rock mass, a 4- to 5-in.-thick coat of shotcrete was immediately applied to the tunnel invert. Subsequently, a reinforced invert arch was



NOTE: 1m = 3.3 ft

Figure 53. Construction Stages in the Fractured Zone of Perjen Tunnel (John, 1981)



NOTE: 1m = 3.3 ft

Figure 54. Support Measures in the Fault Zone of Perjen Tunnel (John, 1981)

installed, and then the shotcreting for the primary support above the invert arch was completed in order to create a closed arch effect. Extensive ground squeeze control measurements were performed in the course of these works, such as extensometers, pressure cells, and leveling. The results of these measurements were used to determine whether, and to what extent, rock bolting with prestressed permanent rock bolts would be necessary. Excavation of the horseshoe-shaped opening was by drill and blast in full section.

Site investigation for the Pfander Tunnel included a 3.6 m diameter pilot tunnel driven by two full-face tunnel boring machines. A pilot tunnel was considered advantageous for reducing geologic uncertainties, estimation of support measures, ventilation shaft construction and ventilation of the main excavation.

5.6 SUMMARY AND CONCLUSION

The four representative advanced rock tunneling techniques studied (mechanical tunneling, rock prereinforcements, shotcrete linings, and New Austrian Tunneling Method) basically can be categorized into two extremes. At one end, emphasis is placed on the development of a highly mechanized tunnel boring machine, or probably a tunnel boring machine with a convertible cutting head for weak rock (RQD less than 50) with thick cover. At the other end, emphasis is placed on the development of highly skilled working crews. They can combat any rock formation encountered by available and effective hand-mined techniques. However, it is not economical and sometimes impossible to have these two extreme options available at the same time for one particular tunnel project. Thus, a preconstruction subsurface investigation is a requirement for selection between these two extremes.

The main findings of each of the effective tunnel construction techniques reviewed in this chapter are summarized as follows:

1. Use of a tunnel boring machine in competent rock can increase the advance rate and reduce labor, overbreak, and support costs. However, the success of a tunnel boring machine depends largely on the rock conditions. Thus, extensive site investigation should be performed for machine tunneling to determine the applicability of the TBM.

2. The rock bolt system is effective in improving the stand-up time during tunneling, and in providing temporary and permanent support for tunnels in rock. However, this system is not effective in badly broken rock formations.

3. The shotcrete lining is useful in loosening rock formations. However, its effectiveness may be influenced by the geometry of the critical rock wedges, configuration of the tunnel surface, and adhesion of the rock joints.

4. One of the main advantages of the NATM is the adaptability to varying ground conditions, the flexibility of changing tunnel cross-section, and better economy. However, it requires a well-trained field staff and experienced working crews.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the studies reported herein, the following conclusions and recommendations can be advanced:

6.1 TUNNELING IN SOFT GROUND

1. Stability and stand-up time are the main factors in connection with the feasibility of tunneling in soft ground. They not only dictate the soil modifications and tunnel construction techniques to be used, but also influence the amount of soil deformation surrounding tunnels.

2. The stand-up time at a soft ground tunnel face can be improved (increased) by dewatering, increasing the excavation rate, increasing compressed air pressure, or reducing the size of the excavation.

3. Unexpected tunnel construction problems such as boulder problems, compressed air leak problems, and man-made obstruction problems in soft ground tunneling can be reduced by an adequate site investigation program and properly planned construction procedures.

4. The tunneling performance in a soil mass is mainly a function of soil type, groundwater condition, size of opening, and the construction procedure. The latter two factors can be adjusted accordingly if the former factors can be identified. Thus, an adequate site investigation is a primary step to reduce the tunneling cost.

5. The selection of cost-effective tunneling procedures is heavily dependent on the soil strata configurations, groundwater conditions, soil type, and construction constraints. Success of the selected tunneling procedure is closely related to the accuracy of the predicted subsurface conditions. Thus, a thorough subsurface investigation is the basic requirement to guarantee economical tunneling.

6. The chemical grouting technique is very effective in stabilizing uniform coarse-to-medium sands, especially if the full tunnel face is in this type of material. However, the chemical grout cannot improve in a non-groutable soil layer.

7. The ground freezing technique can effectively eliminate ground runs at the tunnel face. However, the surface heave is directly related to the natural water content of soft silty soils, and the rate of freezing is influenced by the flow rate of the groundwater in the ground.

8. Compaction grouting is one of the most economical methods in reducing the surface and subsurface movements and in protecting the overlying structures during soft ground tunneling. However, long-term settlements after grouting require further investigation.

9. The newly developed tunneling machines for soft ground are basically attempting to combine all ground stabilization techniques into one operation. These techniques have the potential usefulness to tunnel through very soft or complicated ground conditions.

6.2 TUNNELING IN ROCK

1. For squeezing and swelling rocks, the inward ground movement rates are largely dependent on local geologic conditions. It is more difficult to predict either the total movements or the magnitude of the rock load. Thus, the appropriate method for dealing with squeezing and swelling rock in tunnel openings would be to allow the material to move, under controlled conditions, into the opening with the expectation that the movement will diminish and eventually stop.

2. For predominately swelling rock, if the tunnel opening can be kept sufficiently dry and the rock surface is properly sealed, the creation of swelling pressure may be prevented effectively.

3. For loosening rock, the stand-up time is the main factor of concern. If an appropriate excavation and adequate support operation can be completed within the range of stand-up time for a tunnel head, there should be no time-dependent problem.

4. For crushed rock, the existence of a groundwater head can have a strong influence on its stability and stand-up time because crushed rock will change to flowing ground after saturation. Therefore, positive control of groundwater head is needed in this type of rock.

5. If geological studies indicate the presence or near presence of known or probable active faults, one should expect strong active residual stresses in the rock. In an area where enormous past loading has occurred, such as thousands of feet of glacial ice, one should expect residual stresses that result in strong sudden releases such as rock bursts.

6. There is no known method for estimating exact locations and volumes of underground water flows that might be encountered in rock tunnels. If groundwater might be encountered, feeler holes should be drilled a substantial distance beyond the tunnel heading.

7. A variety of gases of natural origin have been encountered. However, if adequate precaution is taken, the gas problem can be eliminated effectively during tunneling.

8. Use of a tunnel boring machine in competent rock can increase the advanced rate and can reduce labor, overbreak, and support costs as compared to conventional drill and blast techniques. However, the success of a tunnel boring machine depends largely on the rock conditions. Thus, more extensive site investigation should be performed for machine tunneling than conventional tunneling.

9. The rock bolt system is effective in improving the stand-up time during tunneling and in providing temporary and permanent support for tunnels in rock. However, this system is not effective in badly broken rock formation. The shotcrete is useful in loosening rock formations. However, its effectiveness may be influenced by the geometry of the critical rock wedges, configuration of the tunnel surface, and adhesion of the rock joints.

10. One of the main advantages of the NATM is the adaptability to varying ground conditions, the flexibility of changing tunnel cross-sections, and better economy. However, it requires a well-trained field staff and experienced working crews.

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APPENDIX A

Table A-1. Summary of Site Investigation for Selected Tunnels

Owner	Tunnel Project	Tunnel Dimensions	Tunnel Length	No. of Borings	No. of Borings to Tunnel Elevation	Linear ft of Borings	Depth Range of Holes, ft	Costs: Bid	Costs: Actual	Period of Consideration	Remarks
Water & Power Resource Services	Telecote, California	7 ft, I.D. shape varies from horseshoe to modified circular	6.4 miles	None	N/A	N/A	N/A	4,764,853	12,301,237	1950-56	Visible surface geology only.
	Clear Creek, California	17.5 ft, Circular 385 ft, of 15 ft, 8 in. @ outset	10.8 miles	20	20	3,625.1	28.5-406.2	14,772,409	45,106,801	1957-62	
	Eklutna, Alaska	9 ft, I.D.	4.46 miles (23,550 ft)	12	10	≈1,850	80-300	17,348,865	18,248,399	1951-54	
	Helena Valley, Montana	7 ft, Horseshoe	2.65 miles (13,908 ft)	12	12	2,481.4	64-476.8	2,595,088		1957-58	
	Bacon 1, Washington	23 ft 3 in., Horseshoe	10,045 ft	7	2	1,073.04	32.1-591.57	3,494,700		1946-50	Pilot tunnel driven.
	Bacon 2, Washington	28 ft 6 in., I.D. Horseshoe	9,950 ft	9	9	1,611.7	150.8-283.4				
	Fremont 1 & 2, Wyoming	18 ft, Circular	3 miles	24	24	2,900	45-300	18,742,140 (entire project)	Unknown	1957-61	
	Water Hollow, Utah	13 ft, O.D. 10 ft 4 in., I.D.	21,566 ft	25	—	1,601.8	20-180				
	Pachecho Inlet, Chapman	13 ft, I.D.; 7 ft I.D. Horseshoe	1.8 miles	36	—	4,492.7	25.6-322.0	6,933,888		1964-61 1965-67	Borings in portal areas only.
	South Fork	8 ft, I.D.	16,244 ft	22	—	1,233.1	40.9-86.0			1965-67	Most borings in east portal area. South Fork and Chapman are adjacent tunnels.

Table A-1. Summary of Site Investigation for Selected Tunnels
(Continued)

Owner	Tunnel Project	Tunnel Dimensions	Tunnel Length	No. of Borings	No. of Borings to Tunnel Elevation	Linear ft of Borings	Depth Range of Holes, ft	Costs		Period of Consider-ation	Remarks
								Bid	Actual		
	2. Power Structures—Downstream Tunnels & Liners	1. 24 ft, I.D. 2. 24 ft, I.D. 3. 24 ft, I.D. 4. 24 ft, I.D. 5. 24 ft, I.D. 6. 24 ft, I.D. 7. 24 ft, I.D.	1,877 ft ≈ 2,000 ft 2,030 ft 2,063 ft 2,300 ft 2,400 ft 2,500 ft	104	95	20,776.0	51.9-388.0				
	3. Power Structures—Intake Shafts and Upstream Tunnels	1. 24 ft, I.D. 2. 24 ft, I.D. 3. 24 ft, I.D. 4. 24 ft, I.D. 5. 24 ft, I.D. 6. 24 ft, I.D. 7. 24 ft, I.D.	≈ 980 ft ≈ 1,060 ft ≈ 1,180 ft ≈ 1,125 ft ≈ 1,122 ft ≈ 1,152 ft ≈ 1,290 ft	81	68	16,388.77	13.6-387.0				
Corps of Engineers, Seattle District	Flathead, Montana	Height 25 ft 1 in., I.D. Width 18 ft, I.D. Horseshoe	7 miles (36,955 ft)	31	17	4,207.3	30.1-498.8			1966-69	
Corps of Engineers, Sacramento District	New Melones	Height 29 ft 2 in. I.D., Width 20 ft 8 in I.D., Horseshoe	3,900 ft	21	21	5,633.5	77.1-429.4	26,421,384	27,243,000		

NOTES: 1 in. = 2.5 cm
1 ft = 0.3 m
1 mile = 1.6 kilometers

*U.S. GOVERNMENT PRINTING OFFICE:1982 361-428/1536 1-3

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, th Planning and Research (I National Cooperative Hig (NCHRP) managed by the Board. The FCP is a carefu ects that uses research and obtain timely solutions to engineering problems.*

The diagonal double stripe represents a highway and the FCP category that the stripe is used for category I light blue for category 3, I for category 5, green for c orange stripe identifies cat

FCP Category

1. Improved Highway for Safety

Safety R&D addresses the responsibilities of Highway Safety Act an appropriate design sta signing, and physical formulation of improv

2. Reduction of Tr Improved Operatio

Traffic R&D is conc operational efficiency advancing technology existing as well as new the demand-capacity management techniqu preferential treatment rerouting of traffic.

3. Environmental Co Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

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0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.