Report No. UMTA-IL-06-0041-78-1

"TUNNELING FOR URBAN TRANSPORTATION: A REVIEW OF EUROPEAN CONSTRUCTION PRACTICE"

T. D. O'Rourke

Assistant Professor
School of Civil and Environmental Engineering
Cornell University

Formerly: Visiting Assistant Professor
University of Illinois at Urbana-Champaign

AUGUST 1978

FINAL REPORT

Prepared for

U.S. Department of Transportation

Urban Mass Transportation Administration
Washington D.C. 20590

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
UMTA-IL-06-0041-78-1		

4. Title and Subtitle	5. Report Date
Tunneling for Urban Transportation: A Review of European Construction Practice	August 1978
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
O'Rourke, T. D.	UILU-ENG-78-2024
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
Department of Civil Engineering University of Illinois at Urbana-Champaign Urbana, Illinois 61801	11. Contract or Grant No. DOT-UT-80018
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Urban Mass Transportation Administration Department of Transportation Washington, D. C. 20590	August 1976 - August 1978
	14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

Several underground construction methods are examined with reference to recent European applications. An evaluation of the inherent strengths and weaknesses associated with each method is made. The methods under review include grouting in soil, grouting in rock, ground freezing, cast-insitu walls, prefabricated walls, secant piles, slurry shield tunneling, and the New Austrian Tunneling Method as applied to soft ground conditions.

The economics of underground construction are examined. The tunneling costs associated with six European metro systems are summarized. Where appropriate, the construction costs are itemized and the cost structures are viewed in the light of the ground conditions and construction methods used.

The operation and organization of several European metro authorities are discussed. Tunneling practice in the United Kingdom is studied and used as a focal point for examining such issues as the apportionment of risk under contract and the resolution of contract disagreements.

Comparisons are made between urban tunneling costs for rapid transit in the U.S. and Europe. Recommendations for improving tunneling practice are offered.

17. Key Words	18. Distribution Statement
Tunneling, Transportation, Excavation, Contract Practice, Economics, Grouting, Ground Freezing, Ground Deformations, Concrete Diaphragm Walls, Slurry Shield Tunneling	Document is available to the public through the National Technical Information Service, Springfield, Va. 22151

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	210	

PREFACE

This report was performed under Contract No. DOT-UT-80018, sponsored by the Urban Mass Transportation Administration, with the Department of Civil Engineering of University of Illinois at Urbana-Champaign. The work for the contract was undertaken from August 1976 to August 1978.

This report is part of a research and development program sponsored by the U. S. Department of Transportation (DOT) in conjunction with the U. K. Transport and Road Research Laboratory (TRRL). Special thanks are extended to Messrs. Russel K. McFarland, Loyd Money, and Gil Butler, all of the DOT, for their help and encouragement. The generosity and constant support given by the TRRL must be emphasized. In particular, a note of gratitude and recognition is offered to Mr. Myles O'Reilly and Dr. L. J. Griffin. Without their assistance this work would not have been possible.

Special recognition must be extended to Dr. John Hudson of the U. K. Department of the Environment for his constant aid and enthusiasm. Dr. Hudson helped gather much of the information herein, set up important meetings, and reviewed the manuscript at several stages of development.

Others who have contributed by way of discussion and correspondence are listed below according to the firm or organization with which they are associated:

<u>London Transport</u>

Mr. David G. Jobling

<u>Tyne and Wear Metro Authority</u>

Mr. Peter Layfield

Lyon Metro Authority (S.E.M.A.L.Y.)

Mr. C. Bouyat

Brussels Metro Authority (S.T.I.B.)

Mr. C. Dochy

Mr. M. Woitchik

Stockholms Gatukontor

Mr. H. Wilborg

Mr. A. Rinkert

Mr. K. H. Anmark

Helsinki Metrotoimisto

Mr. Esko Leppänen

Paris Metro Authority (R.A.T.P.)

Mr. Henri David

Bochum Metro Authority

Mr. Peter Schultze

Mr. Hűsserman

Tampella

Dr. Kalle Hakalehto

Dames and Moore

Mr. Leif Erikson

Norwegian Road Research Laboratory

Dr. Kaare Flaate

Mr. Arne Grønhaug

Ingeniørene Bonde and Company

Mr. Tormod Jøsang

<u>Norwegian Geotechnical Institute</u>

Dr. Kjell Karlsrud

<u>Construction Industry Research and Information Association</u>

Mr. John Leeney

<u>Mott, Hay and Anderson</u>

Dr. Lionel M. Lake

Mr. Peter Garbett

Mr. Ian Hill

Mr. D. L. Chambers

<u>Sir William Halcrow and Partners</u>

Mr. Cedric Mellor

<u>Edmund Nuttall Ltd.</u>

Mr. A. R. Biggart

<u>Wayss and Freytag</u>

Mr. Volker Meldner

<u>Hagconsult</u>

Mr. Carl-Olof Mordfelt

Mr. S. Magnus Bergman

Dr. Ulf E. Lindblom

Mr. Sven Hultsjö

<u>Foraky Ltd.</u>

Mr. John S. Harris

<u>Soletanche Enterprise</u>

Mr. G. Y. Fennoux

Mr. Gabriel R. Jorge

Mr. Jacques E. Levallois

Mr. Jean Mouxaux

Mr. Maurice Guilland

Tarmac Construction Ltd.

Mr. B. J. Woodman

Soil Mechanics Ltd.

Mr. K. Moller

Mr. Mann

Cementation Ground Engineering, Ltd.

Mr. P. A. Jessop

Ove Arup and Partners

Dr. David J. Henkel

Mr. A. Stevens

Mr. Alan Steele

Lilley-Waddington, Ltd.

Mr. D. Neal

Mr. Stone

Building Research Station

Dr. Hugh St. John

French Railway Authority (S.N.C.F.)

Mr. Jacques Fournier

Ruhr Universität, Bochum

Dr. Herbert Kutter

Dr. Fritz Rommel

University of Newcastle-Upon Tyne

Dr. Ian Farmer

Thanks are offered to Dr. R. B. Peck, Mr. A. P. Chase, Mr. H. H. MacPherson, Mr. Loren Lorig, and Mr. John Wolosick who reviewed the manuscript.

The acknowledgements would not be complete without a note of gratitude offered for the assistance provided by University of Illinois staff. In particular, to Ron Winburn for drafting the figures; to Mary Ann Speck, Laura Battle, and Kathlyn Marr for typing the manuscript; and to Don Jensen for printing and assembling the final copies of the text.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

1.1 Background

Within recent years over 20 different metro systems in western Europe, alone, have either been initiated or expanded as part of an effort to develop urban rapid transportation. A substantial portion of the work has been performed underground, often in central business districts where heavy congestion has been a problem in staging the construction and where the proximity of large, and often sensitive, structures has placed special emphasis on stability and the control of ground movements. Ground conditions have varied widely among the metros under construction and, if taken collectively, provide an extensive cross-section of tunneling environments.

To cope with the various ground conditions and design requirements, special construction techniques have been introduced and refined. Some of these methods include grouting in soil and rock, ground freezing, concrete diaphragm walls, shotcrete support, and slurry shield tunneling. In some instances, the methods have been used successfully with clear savings to the cost of projects. Other applications have shown the limitations of certain techniques, particularly in light of special ground conditions that may offer problems to an otherwise useful method.

Work on the metros has been approached in a variety of ways that reflect special attitudes toward organization. In addition, they emphasize managerial and technical skills that are consistent with the experience and cultural reinforcements among various societies. Correspondingly, a survey of underground construction practice among different countries not only points to different technologies, but points out different ways of implementing the technologies at hand.

This report deals with technical, financial, and managerial aspects of urban underground construction in Western Europe. The technical strengths and weaknesses among various construction methods, costs of tunneling, metro organization, and contract practices are studied and discussed.

1.2 Purpose

It must be emphasized that urban tunneling is examined in this report on a selective basis with examples of practice being cited for specific projects that often were concentrated in one or two countries. Nevertheless, the report has been organized to be reasonably comprehensive so that tunneling practices in many different countries are covered.

When a choice of priorities is made, it must be done with the understanding that no single list can do justice to all the interests at hand. Three objectives, in particular, have been selected for this report:

1. To review specific underground technologies, for which there is an acknowledged expertise in Europe. Several construction methods are summarized with reference to recent European applications and an evaluation of the inherent strengths and weaknesses associated with each method. The methods under review include grouting in soil, grouting in rock, ground freezing, cast-insitu walls, prefabricated walls, secant piles, the New Austrian Tunneling Method as applied to soft ground conditions, and slurry shield tunneling.

2. To study the economics of underground construction. For this purpose, the underground construction costs associated with six European metro systems are summarized. Where appropriate, the construction costs are itemized and the cost structures are viewed in light of the ground conditions and construction methods used.

3. To examine the various forms of organization for underground work in Europe. In pursuit of this goal, the operation and organizational structure of several metro authorities are summarized. Various forms of contract practice are studied. In particular, emphasis is placed on contract practice for underground construction in the United Kingdom.

1.3 Scope

The report is divided into seven chapters, of which the first summarizes the incentives for and intended goals of the report. The second through fourth chapters deal with various technologies. These include, in order of ascending chapter number: ground treatment by grouting and freezing, concrete diaphragm walls, and tunneling with the New Austrian Tunneling Method and slurry shield machines. To avoid redundancy with previous work, the introduction to each of these chapters refers to major reports or papers dealing with the methods discussed therein. The chapters emphasize aspects of the underground methods and associated case histories either not covered or not examined with the same detail in the reference works. The fifth chapter examines the underground construction costs associated with six European metro systems. It is intended that this chapter be used with Appendix A, which summarizes the passenger capacities, dimensions, soil conditions, and construction methods associated with each system. The sixth chapter deals with the structure of various European metro authorities and discusses tunneling practice in the United Kingdom. The seventh chapter compares underground construction costs in the U.S. with those in Europe and offers recommendations for improving tunneling practice.

2 GROUND TREATMENT BY GROUTING AND FREEZING

2.1 Introduction

Traditionally, drainage wells and compressed air have been used to improve ground conditions when tunneling in pervious materials by eliminating or reducing the hydraulic gradients imposed by the tunnel. Both methods, however, are subject to certain limitations. For example, wells may be ineffective when draining silty material or pumping in a soil profile with a substantial portion of interbedded clay. Compressed air, on the other hand, is expensive and may impose medical restrictions on the work.

Grouting and freezing are methods of changing the permeability and strength properties of the ground. They may be applied as either an alternative or supplement to the use of drainage wells and compressed air. In this section, grouting and freezing procedures are examined in light of European tunneling practice. In particular, grouting is examined for both soil and rock constructions.

Several excellent studies have been published on the subjects of grouting and freezing. A state-of-the-art review for grouting practice in soils has been made by Tallard and Caron [84] and Herndon and Lenahan [36]. Grouting in rock has received extensive treatment by the U.S. Department of Army [18]. Freezing techniques have been summarized by Lenzini and Bruss [54], Shuster [77], and Sanger [76].

2.2 Grouting in Soil

2.2.1 Review of the Method

In simple terms, grouting is the pressure injection of a liquid to fill the voids in soil. The liquid hardens over a period of time such that the properties of the soil are changed in either or both of two ways: 1) the permeability is diminished; 2) the strength is increased.

There are fundamental considerations common to all grouting applications for underground construction. These can be summarized under several headings, each of which deals with an important aspect of the grouting operation either as it relates to the soils at hand or to the constraints imposed by the environment and construction set-up. These considerations form a check-list for trouble-shooting problems likely to be associated with grouting projects. They include:

1. Likelihood that grout penetrates the soil needing treatment
2. Change in the properties of the treated soil
3. Effect of grouting on the environment
4. Effects of grouting on the construction program
5. Cost

The above headings are examined and discussed with reference to European practice as follows:

<u>Likelihood that grout penetrates the soil needing treatment</u>

This is the most important consideration related to any grouting project. Efficient grout penetration requires that the grout selected is suitable for the soil needing treatment. This selection should be directed by a thorough knowledge of the subsurface conditions as well as a system for monitoring the quantities of grout injected into various strata.

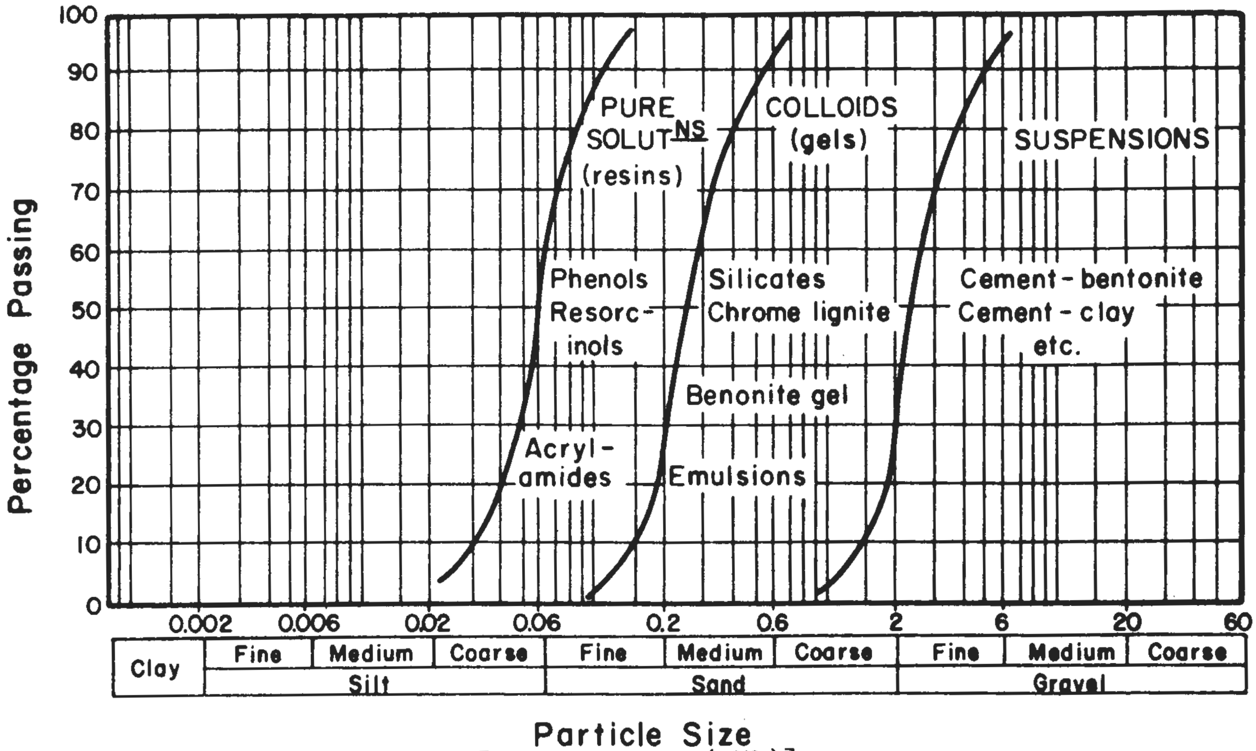
As a standard practice, grout is chosen according to the grain size distribution of the soil. Several commercial firms and research agencies have assembled tables or charts showing the penetration capabilities of various grouts in relation to grain size characteristics [3, 36, 62]. One such chart is illustrated in Fig. 2.1.

In Europe the development of grout materials has favored silicate-base grouts. Ethyl acetate has been used frequently as a reagent with sodium silicate although, recently, additional reagents of a proprietary nature have been introduced by several grouting specialists. Silicate-based grouts are commonly used for treatment of medium and fine sands having a coefficient of permeability, k, as low as 10^{-3} cm/sec. For treatment of fine and silty sands ($k \geq 10^{-5}$ cm/sec.), grouts with viscosities approaching that of water are used. These include acrylamides (AM-9), resorcinal-formaldehyde, and lignin and polyphenolic resins. Grouts of this type have been used to treat some very fine-grained soils as shown in Figure 2.2.[1]

Matching the various grouts with the soils at hand requires a detailed knowledge of the subsurface conditions. The extent of the site investigation will vary according to the scale of the project and to the special characteristics of the job. As a minimum, most grouting specialists require grain size distribution curves for the soils in question as well as borehole information extending well above and below the strata needing treatment. An accurate knowledge of the water levels is usually required.

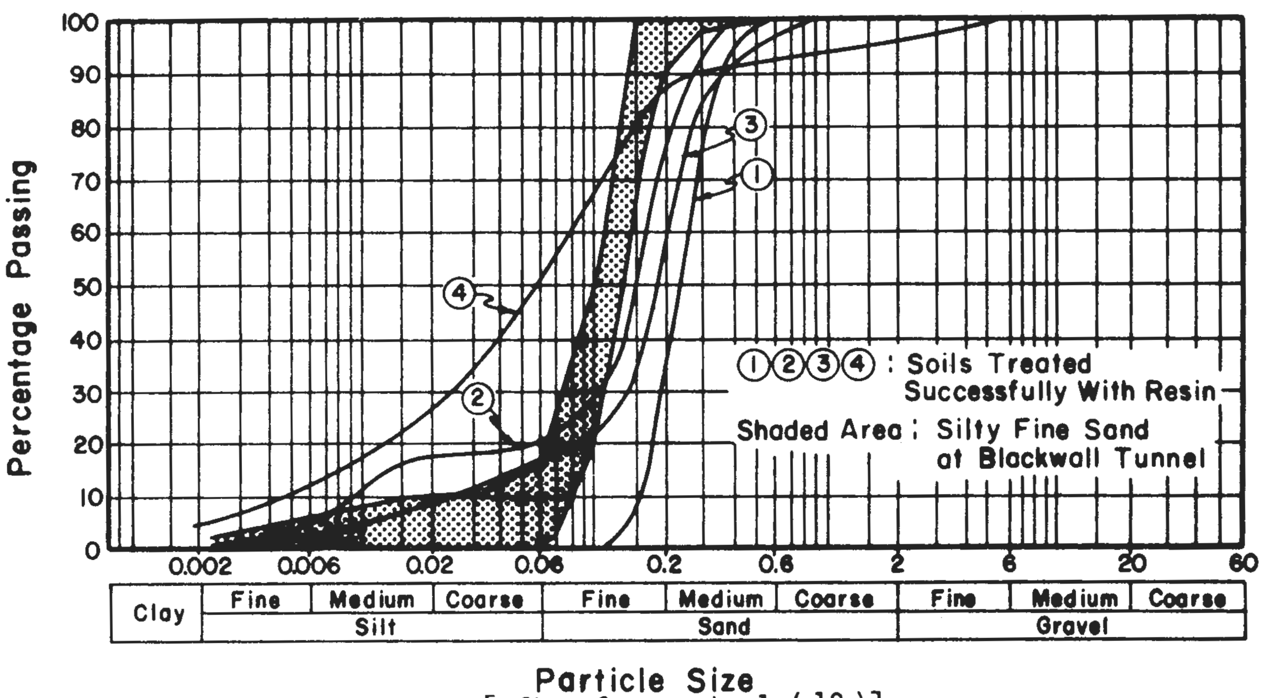
---

[1] The percentages of silt for several of the soils indicated are relatively high and exceed the limits commonly associated with the gravity drainage of water . The soil containing over 50% silt was treated by using injection rates of 38 liters/hr. (10 gal/hr.). This is exceptionally slow as production rates are generally set at approximately 400 liters/hr. (106 gal/hr.).



[after Moller (62)]

Figure 2.1 Soils and Related Grout Types



[after Caron et al (12)]

Figure 2.2 Soils Treated with Resin Grouts

Permeability is often measured insitu as an aid in both choosing the grout type and estimating rates of injection. The driven lance method of grouting [61] lends itself to this type of measurement in that, after the grout pipe has been driven to its desired elevation and retracted slightly, water can be pumped under pressure while inflow rates are measured. The inflow rate of the grout can be estimated by scaling the water inflow in accordance with the ratio of water to grout viscosity.

In some instances special drilling equipment is used, as is the case with a rig developed by Soletanche Enterprise that records the instantaneous penetration rate of the bit, thrust on the drill rods, torque applied to the rods, and pressure of the drilling fluid. The four parameters are logged continuously on a strip recorder. Interpretation of the substrata is made on the basis of the combined variations in parameters. This drilling method was used for grouting projects associated with construction of the Vienna Metro. In France it has been incorporated in various urban tunneling projects as part of the specifications issued by the RATP and SNCF[2].

For large projects a test zone may be injected with a variety of grouts according to the anticipated subsurface conditions. A pit is then excavated into the grouted soil for visual examination and correlation of observed conditions with pumping records. This kind of full-scale test is preferred on large or difficult jobs because it provides for a detailed and reliable evaluation of the grouting design.

Investigation of the soil and matching of grouts with various strata are measures taken <u>before</u> injection. Most grouting specialists, however, would not be satisfied without the use of regulatory measures <u>during</u>

---

[2]The Paris Metro Authority and French Rail Authority, respectively.

injection. These include monitoring various aspects of the system, such as the grouting pressure, the cumulative grout take, the rate of grout inflow, and surface heave.

Estimates of grout take are made on the basis of the inferred percentage of voids for a given volume of soil. Pumping records are kept for specific elevations so that the amount of grout injection can be evaluated against the estimated volume of voids.

One of the most extensively used methods of injection, especially for depth in excess of 10 m (33 ft.), is the tube-a-manchette technique [39]. Basically, this involves grouting through sleeved openings at various elevations along the grouting tube. The technique allows for selective matching of grout types with specific depths and permits multiple regrouting of all levels.

The grouting pressure may be monitored at each sleeved opening. If the grouting pressure is recorded as a function of time, there are certain pressure/time responses that can be "read" to judge the effectiveness of the operation. Figure 2.3 shows a circular strip recording of the pressure/time relationship at two elevations along a tube-a-manchette set-up. Typically, there is an initial peak in the pressure that is associated with bursting the rubber sleeve and solid grout surrounding the perforated portion of the grouting tube. The pressure immediately falls off, whereupon it increases gradually as voids in the soil are filled. At times there may be a second peaking of the pressure followed by a rapid decline to a constant, low level of pressure. This response is diagnostic of hydraulic fracturing or "claquage." The grouting pressure, having exceeded the insitu soil stresses, opens a fissure in the ground through which grout

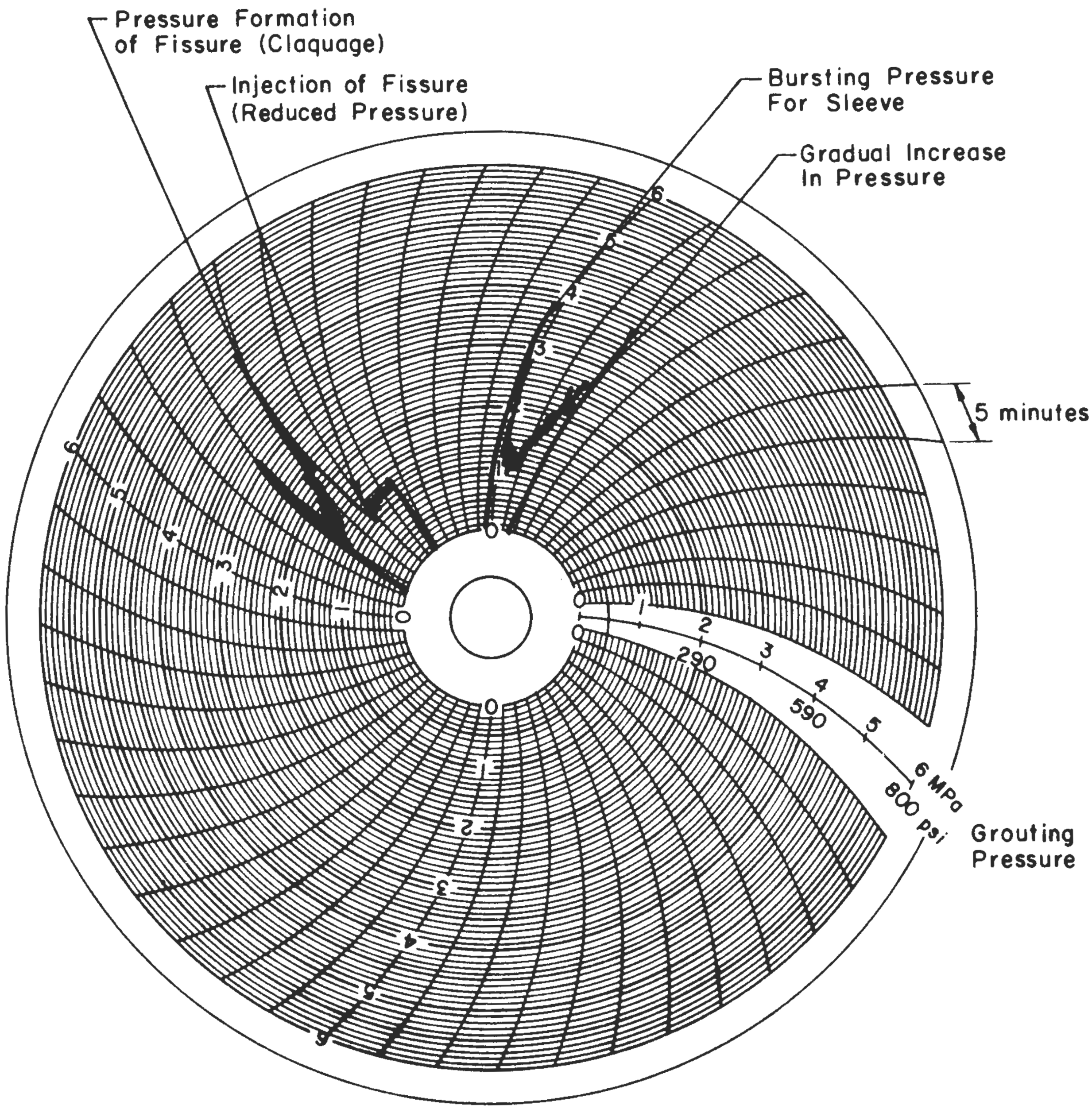


Figure 2.3 Strip Recording of Pressure Injection of Grout

flows at reduced pressure. Injection of this kind is by virtue of displacement, not infiltration. As such, there is a corresponding increase in the volume of the soil under treatment.

Hydraulic fracturing, or claquage, can accumulate to cause surface heave. Therefore, for most purposes, it is regarded as an undesirable response. Once claquage has occurred, injection will be stopped and moved to another elevation. The grouting engineer may return to the elevation of the observed claquage and regrout with either the same grout type or a grout with diminished viscosity. Regrouting the same elevation may occur as many as four or five times. In fact, many grouting specialists believe that multiple regrouting, often with grouts of diminished viscosity, is the key to a successful job.

In some cases the ground may be intentionally fractured to form an interconnecting pattern of grouted fissures. The resulting network of grouted fissures is intended to reinforce the ground. This method of grouting, referred to as claquage grouting, is generally not performed beneath surface structures and is accompanied by careful heave measurements.

The choice of grouting pressure is often made on the basis of experience. There are no well-defined rules. In Britain, it is not uncommon to use pressures of 350 kN/m^2 (50 psi) for depths of 5 to 10 m (16 to 33 ft.). French grouting specialists often use grouting pressures that exceed the overburden pressure by a factor of three or four. For example, grouting of the Cusien Sands in Paris is generally carried out at pressures of 790 to 1480 kN/m^2 (114 to 213 psi) for depths in the order of 10 to 20 m (33 to 66 ft.). It should be noted that grouting pressures are measured at the top of the grout tube. Hence, the applied pressure is generally less than the measured pressure owing to friction along the tube and pressure losses at the point of injection.

Interpretation of the pressure response combined with a record of cumulative grout take provides feedback to the grouting specialist and allows him to judge if the soil voids have been properly infiltrated. In some instances, surface heave is used as an indirect indicator of grout penetration. For example, French specialists frequently anticipate a small amount of heave [in the order of 5 to 15 mm (0.2 to 0.6 in.)] which, on the basis of their experience, is regarded as characteristic of an effective grouting operation.

<u>Change in the properties of the treated soil</u>

Once the soil has been infiltrated, the grout must solidify, or set, in order to form an impervious barrier. In some instances, the grout may deteriorate before setting as a consequence of chemical attack. For example, the hardening of urea-formaldehyde grouts can be retarded in a calcareous, or basic, soil. Conversely, the effectiveness of silicate grouts with bicarbonate reagents can be impaired by an acidic environment. Grout selection must be made with regard for the ground water chemistry.

Silicate grouts are subject to a particular type of decomposition known as synaeresis [11]. This phenomenon is a progressive deterioration of the silicate gel and is related to the size of the soil voids. For grain diameters less than 2 mm, synaeresis is generally not a problem. As a preventative measure, soil deposits, especially those with gravel and coarse sand, are initially grouted with a bentonite cement to fill the larger voids.

The improvement in strength, which can be derived from grouting, may be difficult to evaluate. Herndon and Lenahon [36] have pointed out that a lack of standard testing procedures has led to a wide variety of strength parameters quoted in the literature. In general, laboratory tests [11] show an increase in compressive strength with diminishing grain size for sands treated with silicate grouts.

Long-term stability, or permanence, may be a fundamental consideration when grouting to increase the load-carrying capacity of soil. Clough [13] reports that difficulties with time-loss of strength has been experienced by German engineers and that they have specified tests to establish both the short term and long term capabilities of grouted soil. Tallard and Caron [85] have summarized extensive data concerning the time-dependent strength properties of grouted soil.

Effect of Grouting on the Environment

Grouting will affect the immediate surroundings and, in some cases, may influence areas at a considerable distance from the zone of injection. The impact of grouting on the environment is considered under four general headings as follows:

Toxicity. Many grouts are, to some extent, toxic. Grouts of this type would include those with lignin, acrylamide, and formaldehyde bases. For example, acrylamide is neurotoxic by skin contact, swallowing, or inhalation. Sodium dichromate, which is used with lignin base grouts, may cause ulcerous sores upon contact and in addition may introduce into the ground water or utility lines chromium ions that are lethal for bacteria in sewage treatment plants. Although, silicate grouts are generally not poisonous, they may be toxic depending on the reagents used [85].

Grout migration. As with any fluid under pressure, grout will infiltrate soil along the path of least resistance. Frequently, these paths occur at soil-structure interfaces such as tunnel walls, buried pipelines, and building foundations. Grouting in the vicinity of buildings, especially old, masonry structures, will occasionally lead to grout seepage into basements. At least one instance during grouting for construction of the London Underground can be cited where basement infiltration of toxic grout was harmful to the building occupants [43].

Cement or bentonite-cement grouts are often used in advance of chemical grouts to seal the interface between soil and brickwork. The suspension grouts, being relatively viscous, are less likely than chemical grouts to infiltrate the structure at hand. However, basement walls may contain fractures or separations through which the relatively thick grouts can enter.

Surface Disturbance. Grouting requires a variety of equipment which includes drilling or driving rigs, mixing and proportioning equipment, pumps, injection pipes, and monitoring devices. The stationing and accumulated use of this equipment can have notable repercussions on the area of grouting, especially in urban environments where traffic flow and building access are important. Figure 2.4 shows tube-a-manchette grouting along the line of intended tunneling in a residential area. Local ponding from grout and drilling mud overflow are evident as well as the considerable surface space taken up by the operation. Although the surface disturbance caused by grouting will be temporary, its short-term effects must be considered.

Heave. During grouting, volume expansion can result from hydraulic fracturing, or claquage. In turn, the volume expansion causes surface heave, which can damage buildings and pipelines if the curvature imposed by the heave exceeds the limits of tolerable structural distortion. Surface heave

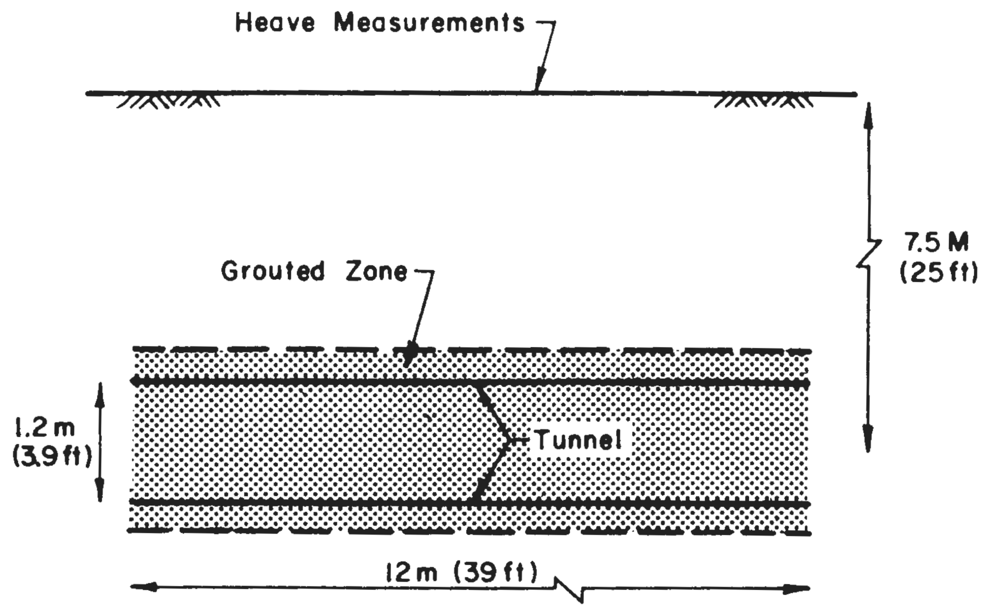
Figure 2.4 Surface Grouting in Suburban Neighborhood

depends on the grout type with respect to soil grain size, rate of grout inflow, previous injections, and depth of injection.

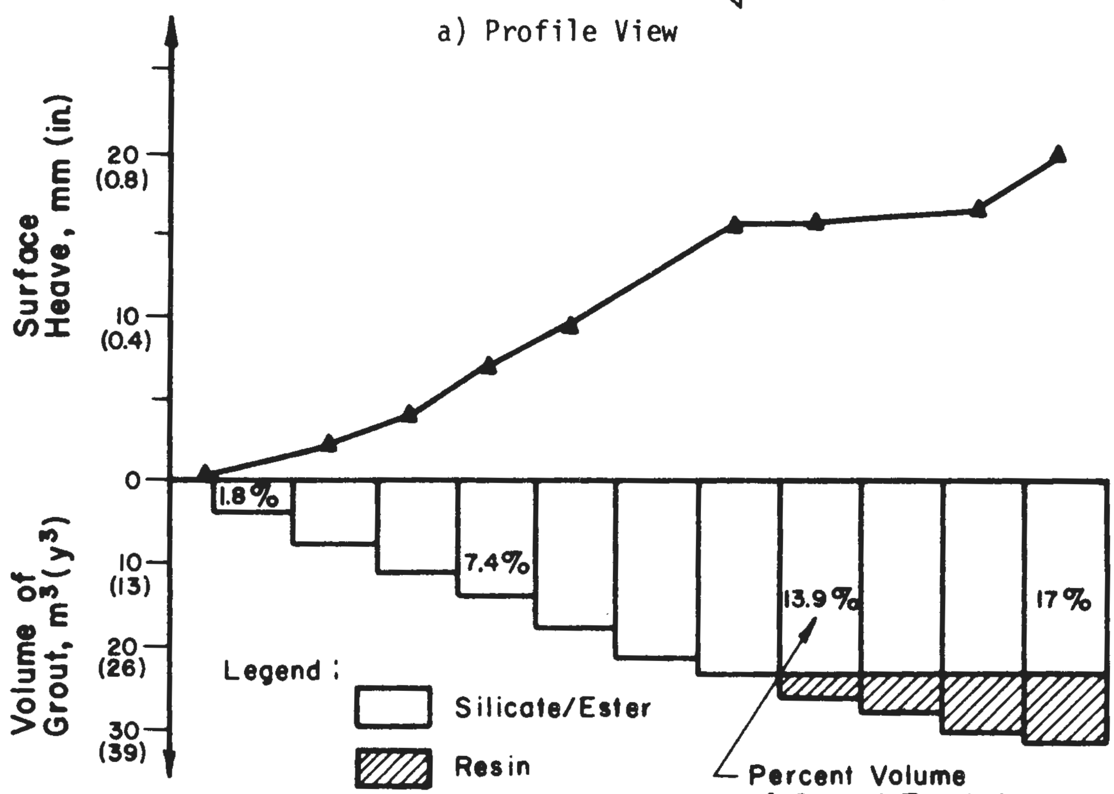
Figure 2.5 illustrates the relationship between grout injection and surface heave during the construction of a 1.2 m-diameter (4 ft.) sewer tunnel. The tunnel intercepted a stratum of silty fine sand that, because of its water-bearing nature, tended to run when exposed during excavation. The unstable soil was treated from the surface with driven lances, using both silicate-ester and resin grouts. As the soil in question was near the lower limit of feasible grouting, leveling stations were established on the ground surface above the tunnel centerline to monitor the heave that was expected in response to grouting. The figure shows the typical surface heave as a function of the volume of injected grout, which is also expressed as a theoretical percentage of the treated ground. When a surface heave of 15 mm (0.6 in.) had developed, the silicate-ester grout was discontinued and a resin grout introduced. An additional 2.5 m^3 (3.3 yd^3) of grout were injected with no apparent increase in volume until a sudden 4 mm (0.16 in.) rise caused the injections to be stopped. The rate of grout injection was between 300 to 400 liters/hr. (79 to 106 gal/hr.).

Rates of injection are chosen according to 1) the soil's capacity to absorb the grout without excess fissuring and consequent surface heave, and 2) the time and economic constraints of the job. With regard to the latter consideration, slow injection rates may be incompatable to meet deadlines without additional personnel and equipment.

Often the rates are established by experience. For example, grouting for the Hong Kong Metro has required the treatment of decomposed



a) Profile View



b) Surface Heave vs. Volume of Grout

Figure 2.5 Volume of Injection and Surface Heave for Tunnel Grouting granite. A grout made up of approximately 30% silicate and 70% water by weight was used at depths of 10 to 25 m (33 to 82 ft). It was found that injection rates of 300-400 liters/hr. (73 to 106 gal/hr.) caused surface heave in excess of 3 cm (1.2 in.), whereas injection rates of 100-200 liters/hr. (26 to 52 gal/hr.) reduced the heave by 50 to 70%, which was considered acceptable [65].

Small and uniformly distributed heave may have beneficial results. Kerisel and Lupiac [48] discuss heave caused by grouting during construction of the Auber Station where upward movement compensated for settlement caused by excavating the station.

Effect of Grouting on the Construction Program

Grouting is frequently one of the first jobs to be performed on a given project. As such, a timely completion of the work is emphasized, as other stages of the construction cannot begin until the grouting is complete. Occasionally, the interception of difficult soils can call for sensitive judgement. For example, if soil is encountered that has a grain size finer than planned for, a slow rate of injection may be necessary to minimize surface heave. However, slow injection rates may delay the project. A decision regarding the grouting procedure must therefore be made by judging the potential difficulties from surface damage as opposed to construction delay.

Surface grouting is not always feasible in an urban setting where it is often necessary to maintain the street right-of-way and minimize inconvenience to pedestrians. At such times, the grouting may be performed from special adits as was the case during construction of the Auber Station [33] or from pilot drifts in advance of the main tunnel. In these cases, grouting requires careful planning and staging in order to coordinate the soil treatment with other phases of construction.

Cost

Costs per unit volume of grout in the United Kingdom and France are summarized in Tables 2.1 and 2.2, respectively. Each cost is expressed in the form of a ratio of cost for a particular grout type to the cost of cement-bentonite grout. The costs per unit volume are for the grouting material only; they do not include expenses for drilling, tube installation, or injection.

The cost of the grouting material varies through a wide range. Even for silicate grouts, costs can more than double depending on the ratio of silicate to water in the mix. Costs for low viscosity grouts (acrylamides, resorcinols, other resins) are relatively expensive, which may be prohibitive if required on a large scale. Relative costs, in part, encourage staged grouting wherein the least expensive grout types are injected first so that the less viscous and more expensive grouts that follow occupy successively less volume.

Pumping costs in France are included in Table 2.2. The pumping costs per unit volume of cement-bentonite grout generally exceed the material costs per unit volume by three to four times. Pumping costs per unit volume of silicate grout runs at between 75 to 100% of the unit volume, material cost.

2.2.2 Recent Applications

Grouting has been integrated into the basic design and construction schemes of several major subway projects in Europe. The result has been a wide and diversified application of the method with some jobs of

Table 2.1
Relative Costs Per Unit Volume of Grout
in the United Kingdom
(Courtesy of Soil Mechanics, Ltd.)

Category	Grout Type	Cost Ratio
Suspensions	Cement-bentonite (3.75 : 1)	1
	Deflocculated bentonite	1.8
Solutions (silicates)	Silicate aluminates	3.3
	Silicate esters	4 to 7
	Two-shot application of silicate and calcium chloride (Joosten process)	6
Solutions (resins)	Polyphenols	6 to 18
	Acrylamide (AM-9) } Resorcinals	11 to 27

Note: Based on 1976 comparative process for materials delivered to site in the United Kingdom.

Table 2.2
Relative Costs Per Unit Volume
of Grout in France

(After Jorge [44] and Cambefort [11])

Category	Grout Type	Cost Ratio	Pumping Costs* [per m^3 (1.31 yd^3)]
Suspensions	Cement-bentonite	1	$50-60
	Deflocculated bentonite	0.8 to 1	
Solutions	Silicate-based gels:		
	dilute	2 to 4	$60-70
	concentrated	6	
	Organic Resins	10 to 500	

*Costs are estimated on basis of 5 French francs: 1 U.S. dollar such large scale that both the cost and execution of the construction project have depended heavily on the grouting procedure. This section summarizes three grouting projects in France and Britain and makes reference to two general patterns of injection that have been particulary useful in underground construction.

Vienna Urban Subway, U_1 Line

An example of grouting on a large scale is the construction of the U_1 line of the Vienna urban subway system. The grouting for this project has been discussed briefly by Haffen and Janin [33].

Figure 2.6 shows a longitudinal and transverse profile of the soils and grouting scheme for the Karlsplatz Station on the U_1 line. Outlines of the subway construction and existing structures are also provided. The soil profile in Vienna consists of approximately 2 to 4 m (6.6 to 13 ft) of fill underlain by 16 to 18 m (52 to 59 ft) of sandy alluvia, all of which overlies a stratum of Tertiary clay.

Previous to large-scale grouting, a test zone [6 m x 6 m (20 ft. x 20 ft.) in plan] was injected in the alluvial soils. A shaft was excavated in the test zone where visual examination and strength tests were performed.

Grouting was performed primarily with the tube-a-manchette method. The grouting tubes were separated by an average distance of 1.2 m (4 ft.). Silicate and cement-bentonite grouts were injected at pressures between 0.8 and 1.2 MPa (114 and 170 psi).

Grout quantities equivalent to 35 to 40% of the ground volume were injected into the soil immediately outside the projected tunnel boundaries. Inside the line of tunnel construction the grout acceptance was limited to between 15 and 20% of the soil volume to facilitate the forthcoming excavation.

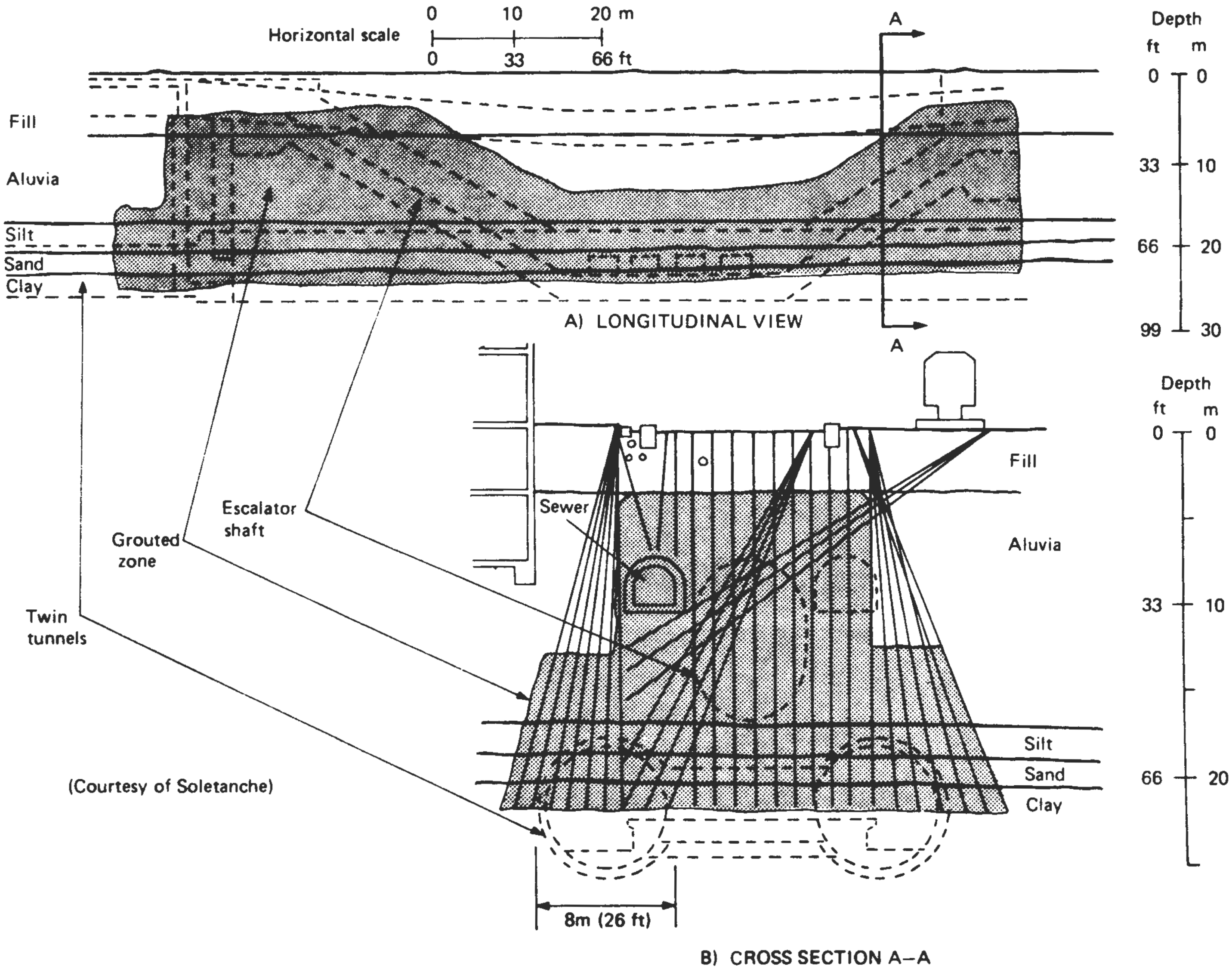


Figure 2.6 Grouting Scheme for Karlplatz Station on the Vienna Metro

Grouting was performed concurrently with monitoring both the surface structures and roadways, where optical levelling points were installed to check for heave. In addition, marker points were referenced to anchor rods that extended beneath the grouted zone. Special sensors were affixed to the foundations of several structures. These sensors were tied into a central alarm system to provide immediate warning of undesirable heave.

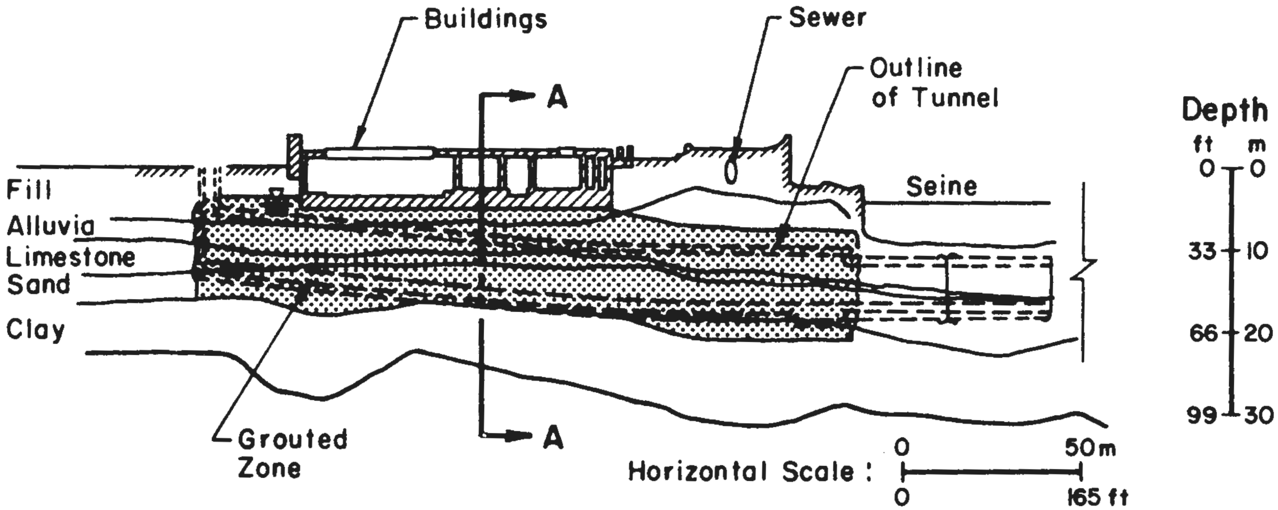
Approximately 43,000 m^3 (56,000 yd^3) of soil were treated, which required some 18,000 m (59,000 ft.) of drilling holes. The average strength of the treated ground was estimated at 2 MPa (289 psi). Tunneling was performed through the grouted soil using the New Austrian Tunneling Method (NATM)[3]. Essentially, this consisted of bench and heading excavation with shotcrete applied to the exposed soil as a temporary support.

<u>Paris Metro, Junction of Lines 13 and 14</u>.

Large scale grouting projects have been undertaken during construction work for the Paris Metro. An example of particular interest is the grouting and tunneling performed for the junction of Lines 13 and 14. This portion of the system is composed of 129 m (423 ft.) of tunnel beneath the Seine River with two subsurface approach sections on opposite sides of the river. Bougard [9] has discussed the use of prefabricated box structures to effect the subaqueous crossing. The general grouting scheme has been described [4].

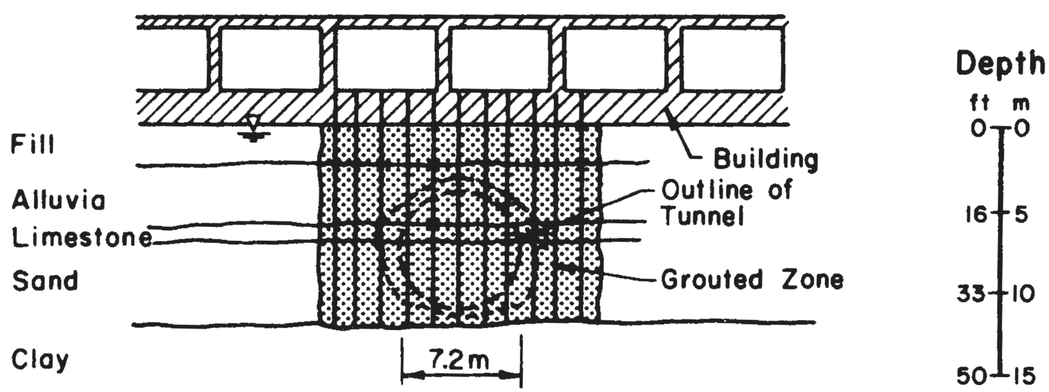
Figure 2.7 shows a longitudinal and transverse profile of the soils and grouting scheme on the Left Bank of the Seine at this site. The soil profile is variable. In general, it is composed of 4 m (13 ft.) of fill, 4 to 7 m (13 to 23 ft.) of alluvial soils, a 1 m (3.3 ft.)- thick stratum of limestone, 5 m (16 ft.) of medium to fine and silty fine sands,

[3] NATM is discussed in Chapter 4.



a) Longitudinal View

(courtesy of RATP)



b) Cross-Section A-A

Figure 2.7 Grouting Scheme for Junction of Lines 13 and 14 on Paris Metro all of which are underlain by a stratum of clay. The alluvial soils contained lenses of silt, silty fine sands, and marl.

Grouting was performed primarily from the surface using the tube-a-manchette method, although shafts were sunk to grout zones that were otherwise impossible to reach. The grouting tubes were separated by an average distance of 1.5 m (5 ft.). The grout types were selected according to the characteristics of each stratum. In the alluvial deposits, for example, silicate equivalent to 37% of the ground volume was injected as opposed to 5% for cement-bentonite grouts. In the limestone stratum, rock fissures were injected using silicates and cement-bentonite in quantities equivalent to 5% and 3% of the ground volume, respectively. Claquage grouting was performed in the marl and silt pockets.

For the entire project, approximately 66,400 m^3 (86,800 yd^3) of ground were treated. The minimum compressive strength of the grouted soils was estimated as 1 MPa (144 psi).

During the work, some difficulty was encountered in the alluvial soils. It was found that slow pumping rates, in the order of 100 to 150 liters/hr. (27 to 40 gal/hr.), could be used to grout these materials without excessive heave. However, the slow rates of injection were incompatible with the time constraints of the project. After conferring with engineers from the Paris Metro Authority, it was decided to grout at rates in the order of 400 to 500 liters/hr. (106 to 135 gal/hr.). Heave of approximately 30 to 40 mm (1.2 to 1.6 in.) occurred, which damaged an overlying building.

Passenger Subways at Heathrow Airport

In Britain, the scale of grouting projects are determined by the application of grout on a specialist basis to compliment specific tunneling schemes and as a remedial measure in locally unstable ground. An excellent example of a grouting project in Britain is the ground treatment during construction of the passenger subways at the Heathrow Central Station of the London Underground.

Figure 2.8 shows a plan and profile view of the grouting scheme for a portion of the passenger subway project. The soil profile at Heathrow consists of 6 to 8 m (20 to 26 ft) of pervious sand and gravel (1 cm/sec $\geq$ k $\geq$ 10^{-2} cm/sec) overlying stiff, London clay. The water level is approximately 4 m (13 ft) below the ground surface. Grouting was performed with driven lances, using the Joosten two-shot method [36]. A visual inspection of this site has been reported by Clough [16]. Grout concentrations were conspicuous throughout gravel lenses. The overall strength of the treated ground was estimated at approximately 3.5 MPa (500 psi).

The subway structures were excavated and constructed in stages near public facilities and airport buildings. Secant piles were used to form the walls of the permanent structures. As the secant piles had to be installed a short distance from existing structures, temporary support was required in the areas adjoining ducts, pipelines, and building foundations.

The grouting worked well in combination with the secant pile walls. Areas adjacent to and underlying buried ducts and pipelines were rendered impervious and stable during excavation. Grouting was used to form temporary cut-offs, which were subsequently excavated as the construction was extended longitudinally. In addition, grouting was used to underpin the footings of buildings adjacent to excavation.

Grouting Patterns

There are certain patterns of grouting that have been used on a recurrent basis. These patterns have been extremely useful within the context of European tunneling and, as such, are worthy of comment in the form of specific examples.

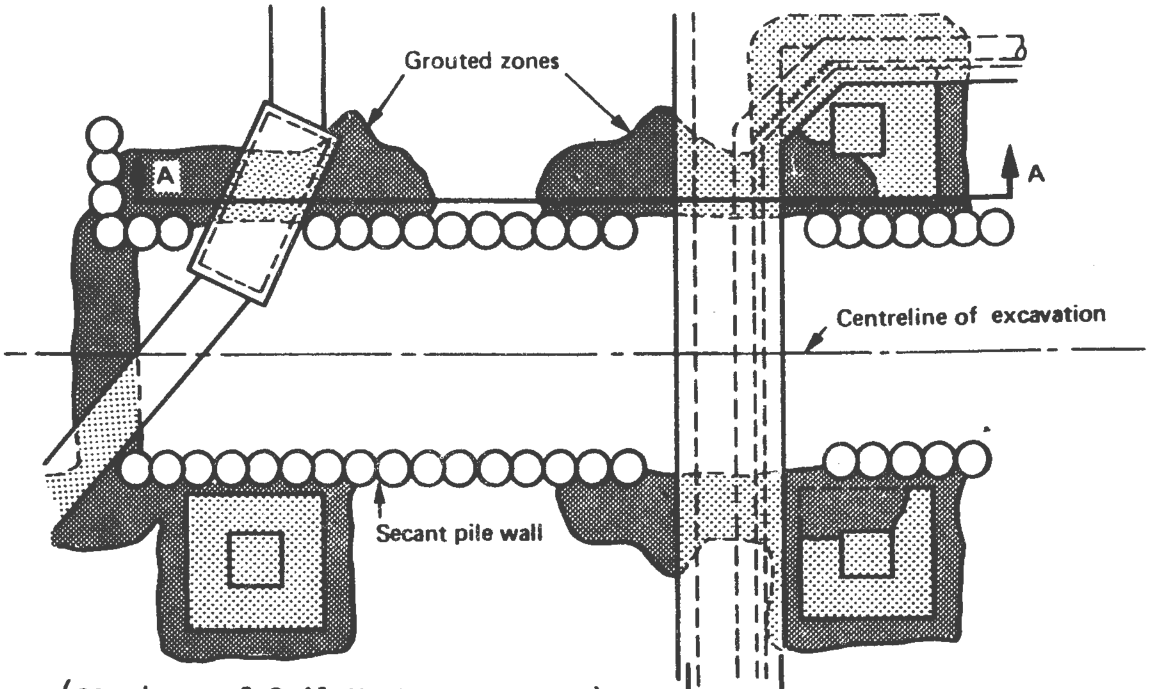
One such pattern is the formation of an impervious zone at the base of cut-and-cover excavations. This is illustrated in Figure 2.9, which shows a profile view of the application as used during construction of the Lyon Metro. The grout, which was injected into pervious sand and gravel (1.0 cm/sec. $\leq k \leq 10^{-1}$ cm/sec.), was made up in a volumetric proportion of 11.4 : 1.9 : 1.0 (water to cement to bentonite). The depth of the grouted zone was designed to resist the hydrostatic uplift forces and was determined by the following equation [27]:

$$\gamma_w \, (h_w + h_t + h_i) < \gamma h_t \; + [(1-n)\gamma_d \; + \alpha n \gamma_G] h_i$$

where h_w, h_t, h_i are defined according to the dimensions idicated in the figure, n is the porosity of the soil, α is a function of the injection pressure (less than 1) γ_w is the density of water, γ_G is the density of the grout, and γ and γ_d are the saturated and dry density of the soil, respectively. A factor of safety of 1.25 was applied in deteriming the depth of grout zone, h_i. The depth of the untreated zone, h_t, was specified as being at least 0.5 m (1.6 ft.) to act as a buffer against the upward migration of grout.

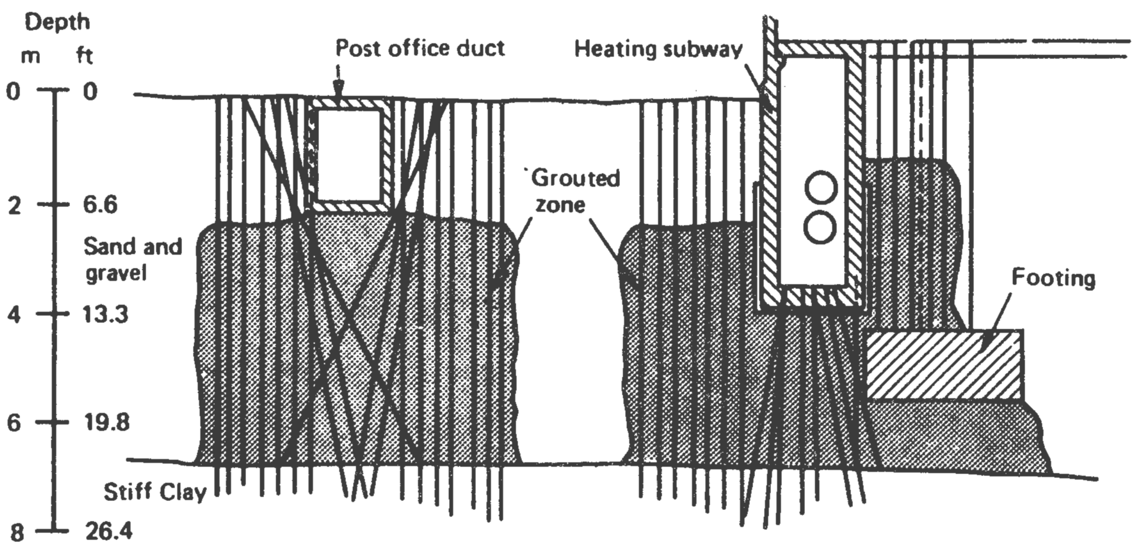
Another pattern that is used extensively is the formation of an impervious and strengthened zone over the crown and shoulders of the tunnel. This pattern is particularly useful where the depth of stable cover is variable such as in stiff to hard clay when tunneling near an overlying stratum of sand beneath the water table.

An illustration of the pattern is found in Figure 2.10, where the grouting scheme associated with tunneling at Heathrow Central Station



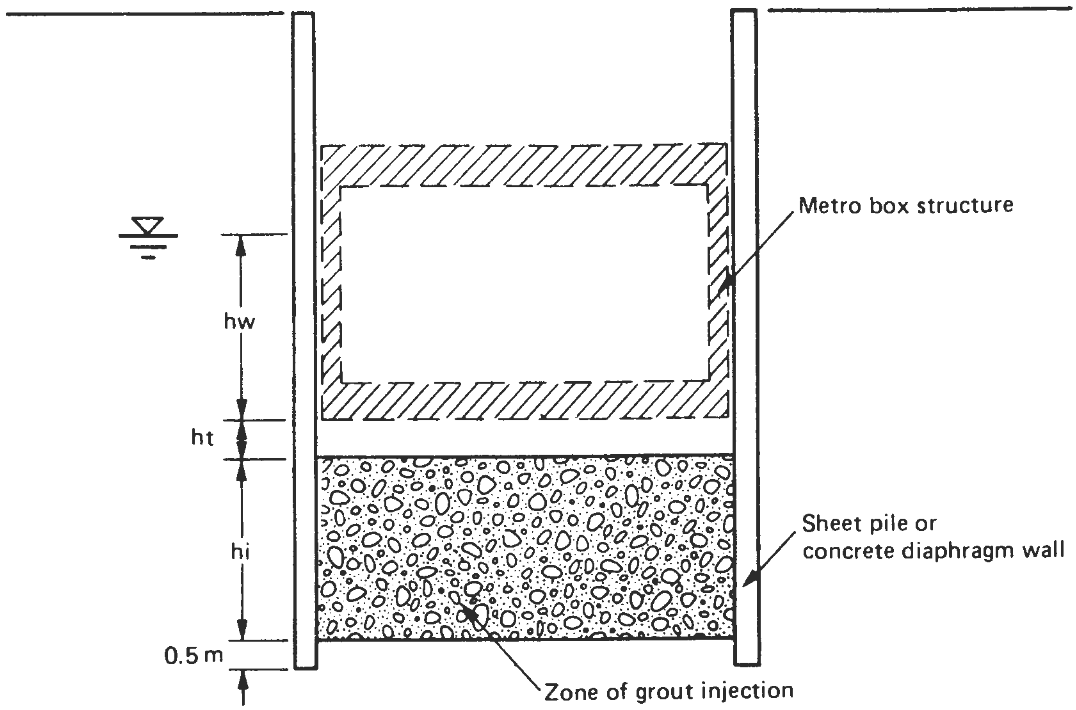
(courtesy of Soil Mechanics, Ltd.)

A) PLAN VIEW



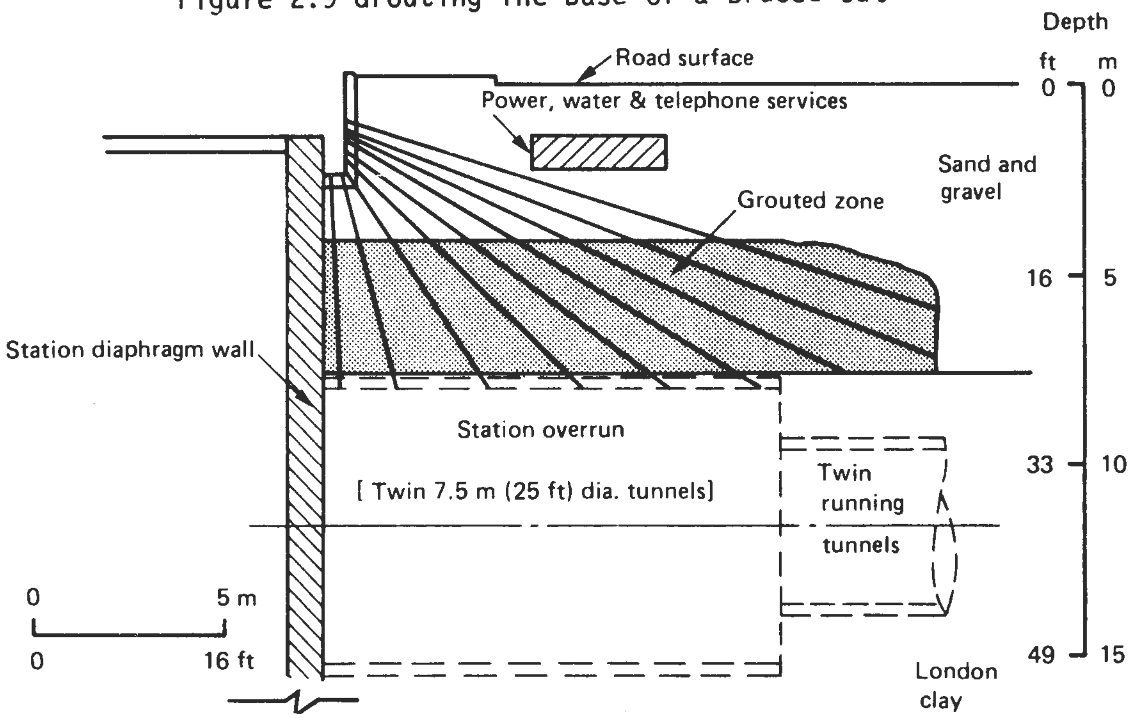
B) CROSS-SECTION A–A

Figure 2.8 Grouting Scheme for Passenger Subways at Heathrow Airport



[after Ferrand et al (27)]

Figure 2.9 Grouting The Base of a Braced Cut



[after Moller (61)]

Figure 2.10 Grouting above the Crown in a Mixed Soil Profile is shown. The crown of the 7.5 m-diameter (24.6 ft.) tubes, which form the approach passages to the station, were very near an upper sand and gravel stratum. A protective "canopy," or cover, was injected over the tubes with a series of tube-a-manchettes, using bentonite-cement and silicate grouts. The grout holes were installed from a staging area at the station cofferdam. This pattern has been used on a great number of projects and is frequently referenced in the literature [33, 36).

2.2.3 Discussion

Grouting has been used in a variety of tunneling projects, some of which have required extensive ground treatment on a scale substantially affecting both the cost and scheduling of the project. Quality control and monitoring systems have been developed to compliment the size and range of grouting applications and, in fact, have become an integral part of the grouting. It is not uncommon for records to be taken of depth, pumping pressure, rate of grout inflow, and total grout take for each injection point. In addition, measurements of surface heave and observation pits are frequently used.

With regard to the design and organization of a grouting job, there are two points worthy of comment:

1) Effective grouting relies heavily on individual judgement. The grouting specialist interprets the pumping records and field measurements according to his familiarity with the equipment and previous experience. His judgement is central for the selection of grout type and the multiple regrouting of various elevations.

2) The grouting process should be flexible to adapt to the variable nature of the soils at hand. Consequently, rigid specifications may be counterproductive. The decision to use grouting and the resulting organization of work should be made so that the grouting procedures can be changed to fit the ground conditions as they are encountered.

Grouting for underpinning purposes has been confined mostly to gravels and medium to coarse sands. The emphasis in practice on coarse sands and gravels is somewhat inconsistent with laboratory tests that show improved strength with diminishing grain size. This inconsistency seems to reflect the field difficulties and cost of impregnating finer grained materials. Some grouting specialists, in fact, are reluctant to treat fine or silty sands for underpinning [63].

A consistent procedure for both testing and sampling grouted soils needs to be developed as a prelude to rational design for soil strengthening. It should be recognized that sampling by coring is frequently destructive for coarse soil (gravel, coarse sands) as forces transmitted through the larger grains will tend to break up the grout matrix. Furthermore, the long-term properties associated with different combinations of soil and grout deserve study. As the grout take and, thus, the ground strength will be variable according to variable soil profiles, case studies emphasizing the ground conditions are extremely useful for judging the practical limitations of strengthening natural soils.

Finally, the relationship between grouting and surface heave should be recognized, especially if pumping pressures in excess of the overburden are used. There has been little published information on the subject. A better understanding of this behavior could be obtained from case studies, particularly in regard to the magnitude of the injection pressure, grout type, soil profile, and depth of injection.

2.3 Grouting In Rock

2.3.1 Review of the Method

Broadly speaking, rock grouting for tunneling is performed for two reasons:

1. To stabilize pervious ground that represents a potentially unstable condition when intercepted by tunneling. This would include zones of weathered rock or sediments in buried valleys, both of which may change to flowing ground where located below the water table.

2. To retard water inflow under conditions where stability is not compromised. This would include protection against flooding or the reduction of water flow to promote a reasonable working environment. In addition, grouting may be applied against small scale infiltration that might draw down the water table in local drainage basins or damage electrical and mechanical equipment during operation of the permanent structure.

In the former case, grouting is closely associated with site investigation. Unstable zones should be located before interception by the tunnel, and a variety of investigation techniques are available for identifying potentially treacherous ground. These would include increasing the number of vertical boreholes, probing in advance of tunneling, or geophysical methods. As this subject is beyond the scope of the report, grouting for this application is not discussed. Instead, this section concentrates on rock grouting as applied to ground water control under stable conditions.

Rock grouting is most frequently performed with cement or a bentonite-cement mixture. This type of grout is low in cost and develops a relatively high strength. Penetration, however, is limited by the particle size of the cement with respect to the thickness of the discontinuity needing treatment. As a general rule, the ratio of the crack width to particle size should be at least 3 [55]. This limits the thickness of fissure that can be treated with most commercially available cements to approximately 0.2 mm (.008 in.).

Openings too small for treatment with cement, can be treated with solution grouts. These would include grouts with silicate, lignin, or resin bases. An estimation of the relative penetration capabilities among several kinds of grout can be gained from Fig 2.11, which shows a plot of the penetration rates of several different grouts as a function of gap width for various sized openings. The plot was developed from tests in which grout was injected between two closely spaced, glass plates [0.003 to 0.1 mm (.0012 to .004 in.) separation] at a pressure of .05 MPa (7.2 psi) and a temperature of 5°C (41°F). The tests were supplemented with additional measurements of grout penetration rates through sand columns at a pressure of .01 to .02 MPa (1.4 to 2.9 psi) and temperature of 5° and 20°C (41 and 68°F). The results of the sand column tests were converted to fictive gap widths by dividing the pore volume by one half the total surface area of the particles.

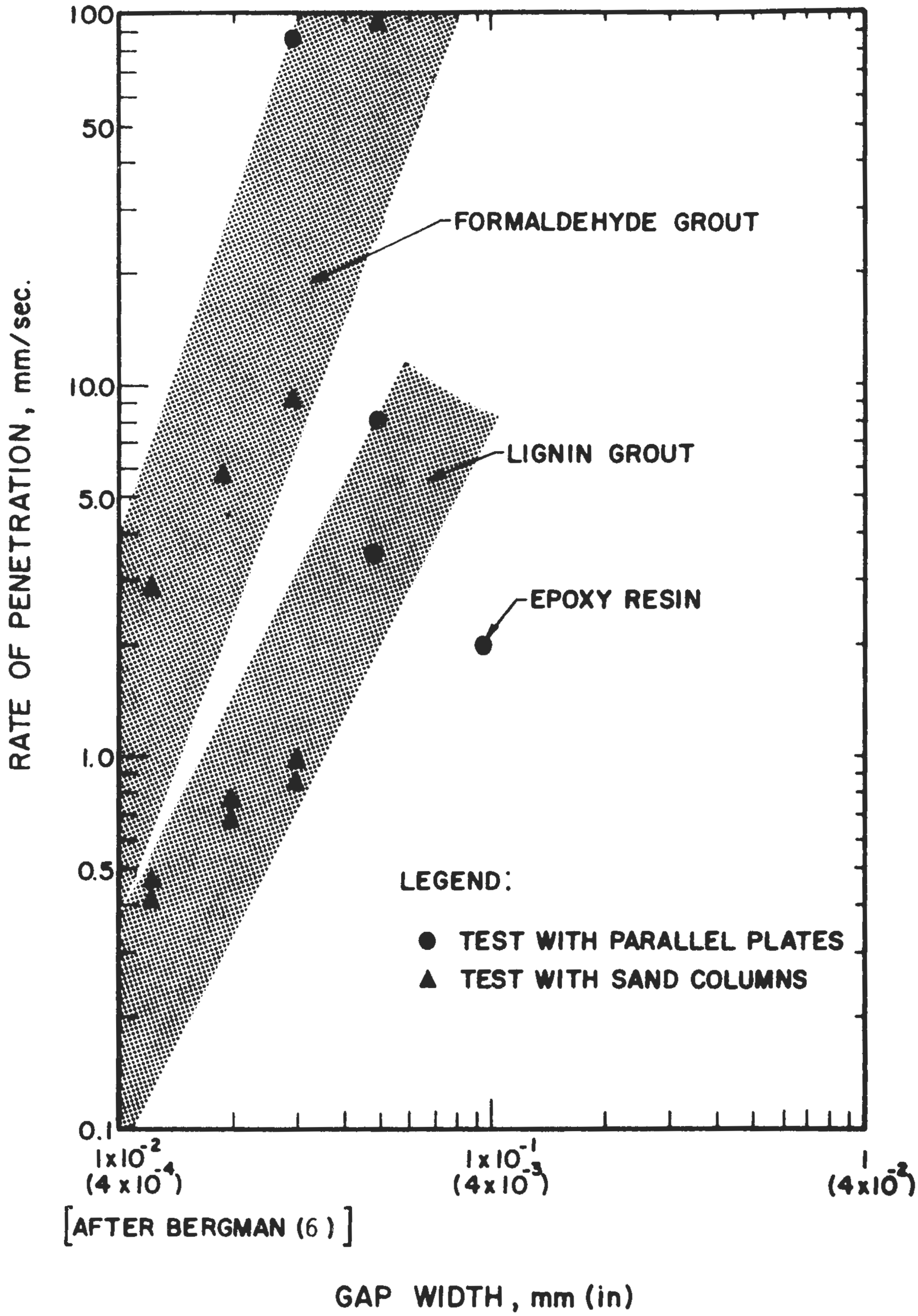


Figure 2.11 Grout Penetration Rates as a Function of Fissure Width

Grouting for control of infiltration is performed with grout holes that are generally between 35 to 75 mm (1.5 and 3.0 in.) in diameter. The length of the holes is determined by the dimensions of the the grouted zone. For transportation tunnels the length is often between 4 and 6 m (13 and 19.7 ft.) and will rarely exceed 13 m (42.6 ft.).

Grouting is frequently preceded by water testing the holes to estimate permeability. A measurement is made of water loss per meter length of hole over a unit time under a constant pumping pressure. On the basis of local experience, a threshold value of water loss may be chosen below which grouting is not required. Water loss per hole can be used to set the sequence of grouting, as the most pervious holes will often be treated first. In some cases, the effectiveness of grouting may be monitored with Lugeon testing.

Frequently, the water to cement ratio will be reduced in stages as grouting is performed for each hole. For example, grouting may start with a water-to-cement ratio of 8:1 or 6:1 (by weight) and be diminished in the order of 4:1, 2:1, 1:1 and 0.5:1. Thin grouts are used in the initial stages to carry cement particles into narrow fissures. As the particles begin to accumulate, the grout is thickened to supplement the initial take with greater quantities of cement. Pumping is often carried out at pressures equal to or slightly larger than the overburden pressure. For tunneling in Scandanavia, grout pressures are reported to have been 2.5 to 3 times the overburden [57].

When tight control on water inflow is required, grouting in advance of the tunnel is used. This is commonly referred to as pre-grouting.

This procedure has been adopted as a standard technique for urban tunneling in Sweden and Finland. In these countries the water levels in buried valleys can be drawn down by subsurface excavation with consequent settlement of sensitive clay or damage to timber piles.

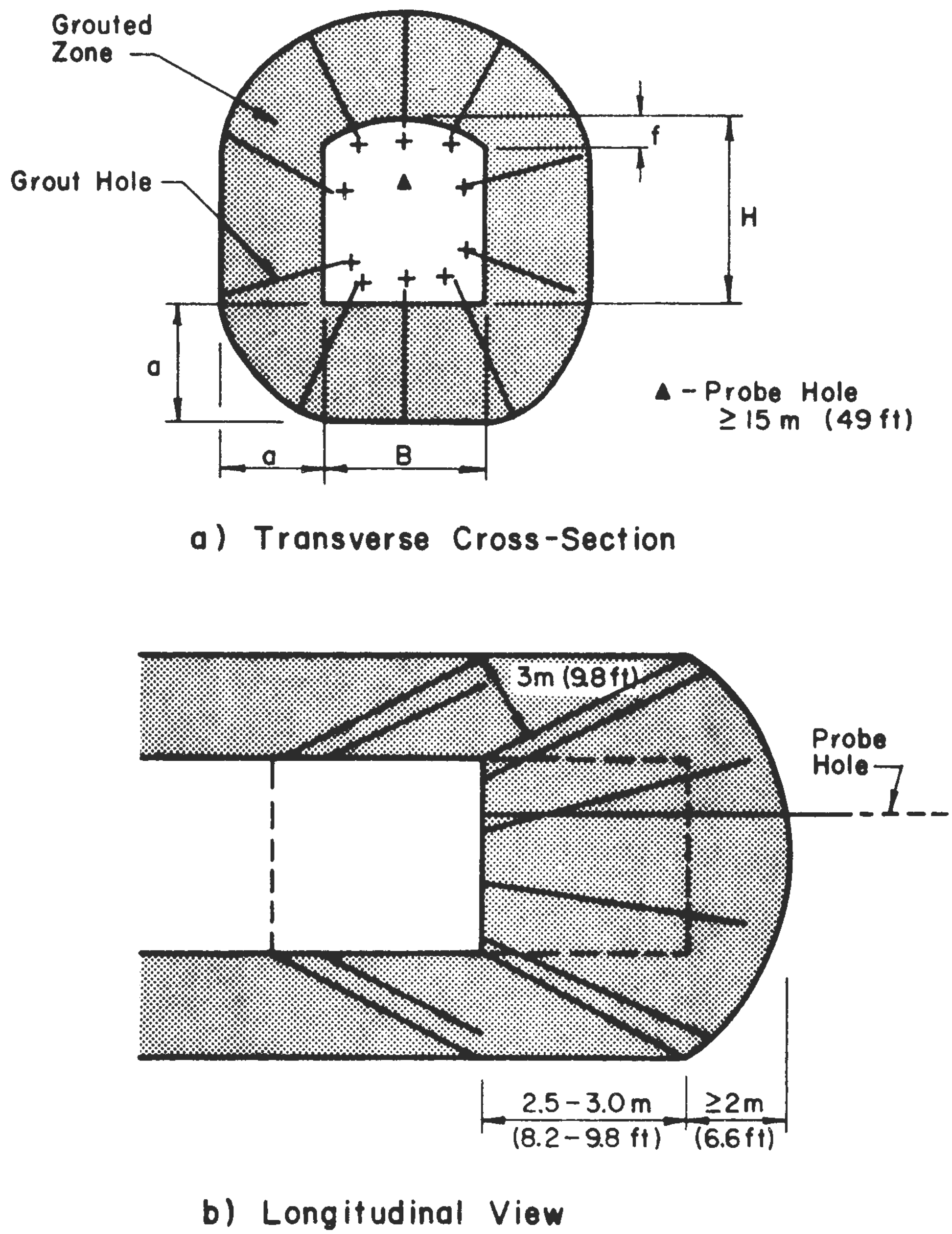
Figure 2.12 shows the typical pattern of pregrouting that is performed for rapid transportation tunnels in Stockholm. A probe hole is directed 15 m (49 ft.) in advance of the tunnel face and water tests performed to estimate grouting requirements. Grouting is performed from a series of holes that are splayed foreward around the periphery of the tunnel. The holes are advanced a minimum horizontal distance of 2 m (6.6 ft.) beyond the forthcoming drill-and-blast round.

Pregrouting was performed on a consistent basis during tunneling for the Helsinki Metro. Grout holes, on 3 m (9.8 ft.) spacings around the crown and shoulders of the tunnel, were splayed foreward from the face to cover a horizontal distance of at least 2.5 m (8.2 ft.) in advance of the forthcoming drill-and-blast round [generally 2.5 m (8.2 ft.) long]. Water testing was performed to determine the need for grouting. If water loss exceeded 0.3 liters/min. per 98KN/m^2 (14.4 psi) per meter of hole under a maximum pressure of approximately 1.5 MPa (215 psi), pregrouting was performed. The grout was made up in a 4 to 1 proportion of water to Portland cement.

Pregrouting is usually supplemented by the local grouting of conspicuously high inflows after a portion of the tunnel has been driven. This is referred to as post-grouting. Extensive post-grouting is usually undersirable before it interferes with tunneling operations, such as mucking, that require rapid access to the face.

Type	H	B	f	a
Station Tunnel	6.75	9.20	≥1.40	3.50
Double Track Tunnel	6.00	8.10	≥1.60	3.50
Single Track Tunnel	5.60	4.30	≥1.00	3.00

Note: All Dimensions In Meters



[After Rosell et al (76)]

Figure 2.12 Rock Grouting Pattern for Tunnels on the Stockholm Underground

2.3.2 Recent Applications

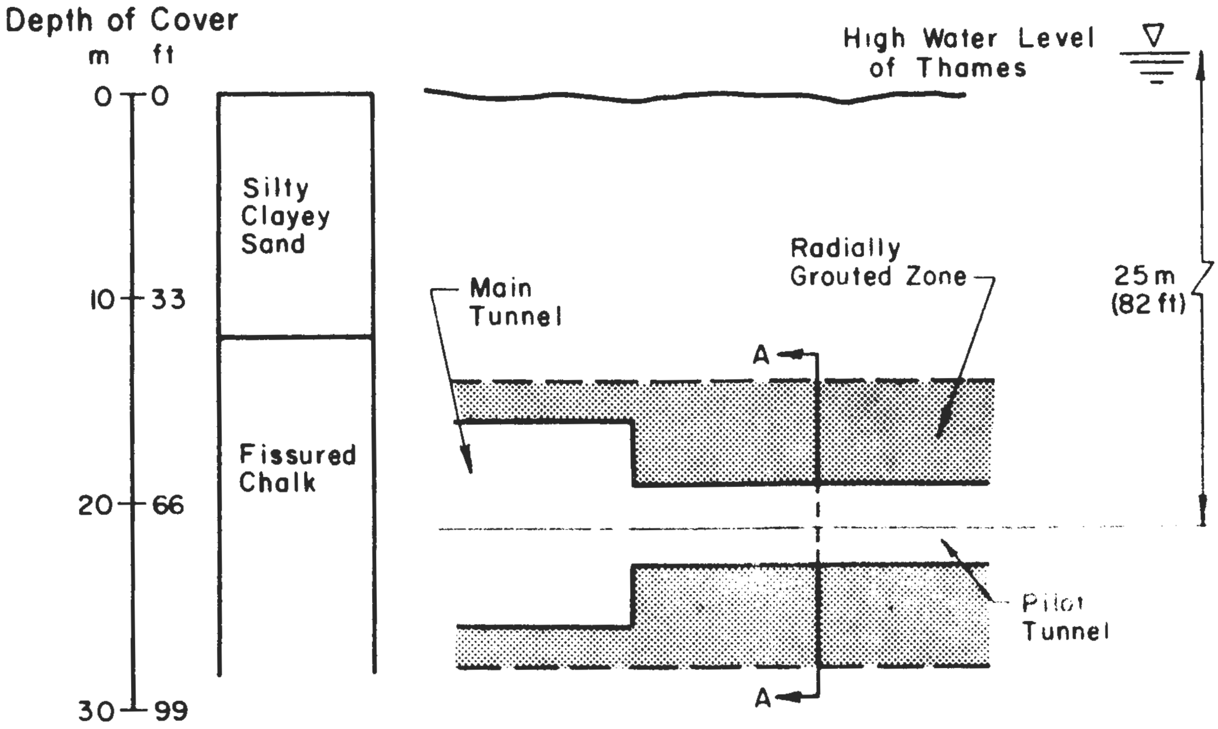
In this section rock grouting is described for three tunneling projects under separate headings as follows:

The Second Dartford Tunnel.

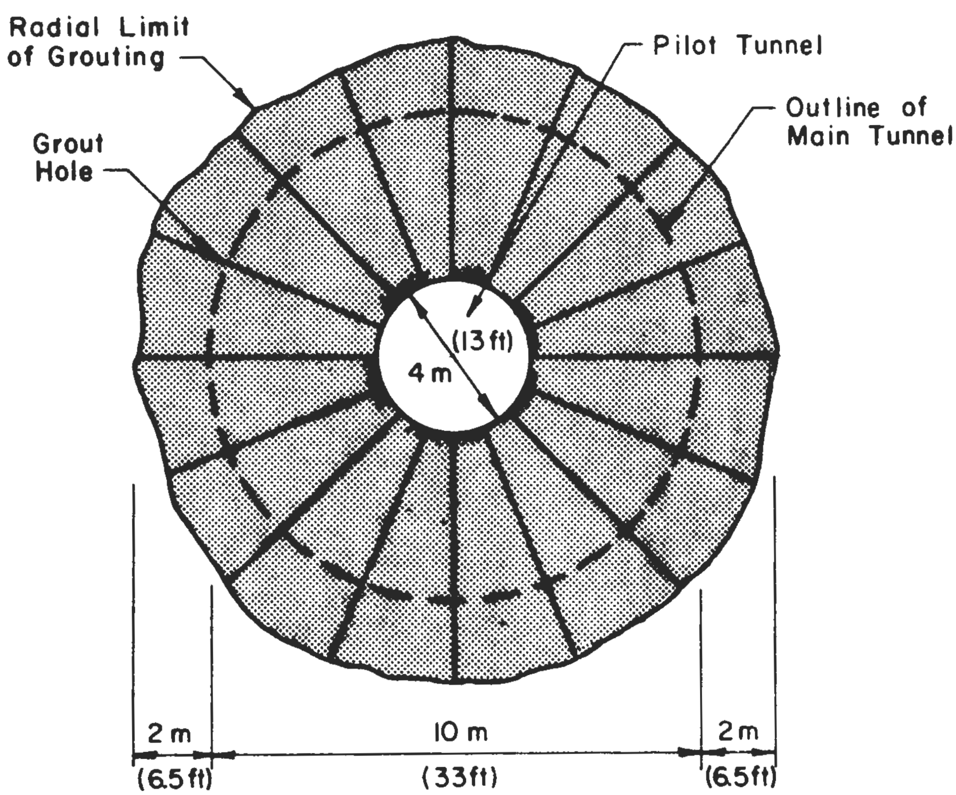
A second subaqueous crossing of the Thames was made at Dartford [approximately 32 km (20 mi.) east of London] to expand the volume of traffic carried by the first Dartford Tunnel as part of the A282 Motorway. The bulk of the tunneling was preceded by a 4 m (13 ft.) - diameter pilot bore, from which rock grouting was performed in advance of the main excavation. The project has been discussed briefly by Clough [13] who has, in particular, reported on the grouting scheme associated with driving the pilot tunnel.

A simplified profile of the soil and rock conditions beneath the central channel of the Thames is shown on the longitudinal view of the tunnel in Fig. 2.13a. The high water level of the Thames is 25 m (82 ft.) above the centerline of the tunnel. The ground profile along this section is composed of approximately 12 m (39 ft.) of silty, clayey sand underlain by fissured chalk. The chalk is blocky, being cut more or less continuously by fissures on approximately 200 mm (51 in.) spacings. Although many of the fissures were tight, clay-filled and open fissures were observed.

A series of preliminary grouting tests were performed at two sections along the pilot tunnel, one in relatively tight and the other in relatively loose ground with high water inflow. On the basis of the tests,



a) Longitudinal View



b) Cross-Section A-A

Figure 2.13 Rock Grouting Pattern for Second Dartford Tunnel a neat Portland cement was chosen as the grouting medium. A bentonite-cement mixture (4% bentonite by weight) was tested, but was judged less suitable than the cement as it was overly thick for penetration of thin fissures in the chalk.

The tunnel was shield driven, using rotating cutters to break up the chalk. A lining, composed of segments with a combined section of steel plate and precast concrete, was erected as the primary support for the tunnel.

As shown in Fig. 2.13b, a radial pattern of 16 holes were installed from the pilot bore to grout the rock 2 m (6.6 ft.) beyond the periphery of the main tunnel. Each hole was 50 mm (2 in.) in diameter and approximately 5 m (16.4 ft.) long. Greater lengths were required at some locations where the pilot bore had drifted off the center line of the main tunnel. The grout was injected in decreasing proportions of water to cement until maximum grouting pressures, between 560 and 690 kN/m^2 (80 and 100 psi), were attained.

Each hole was water tested both before and after grouting as a check on the effectiveness of the treatment. A threshold value of 5 Lugeons[4] was specified as indicating a suitablely grouted hole. Grouting was performed at 6 m (19.7 ft.) intervals along the tunnel. On the basis of the water tests and observed infiltration, additional holes were placed so that the longitudinal spacing was mostly between 3 and 1.5 m (9.8 and 4.9 ft.)

---

[4]One Lugeon unit corresponds to a water test where one liter/min. of water per meter length of hole is absorbed by the rock under a pumping pressure of 1 MPa (144 psi).

Previous to the drive, it was believed that an operational air pressure of 110 kN/m^2 (16 psi) in combination with the grouted rock would be sufficient to restrain water inflow to reasonable levels of control and work productivity. During the drive, complaints from the contractor and tunnelers resulted in an increased air pressure of 190 to 208 kN/m^2 (27 to 30 psi). The higher pressures, in turn, lead to tighter medical restrictions and diminished working time per shift. Without grouting, it was estimated that air pressure in excess of 300 kN/m^2 (43 psi) would have been required to suitably restrict water inflow.

Sewer Tunnels in Gothenburg.

A good example of grouting to control small scale seepage is the rock treatment for sewer tunnel construction in Gothenburg, Sweden. The project has been reported in detail by Lysen [57], and only its salient features are summarized here.

During the early stages of construction, infiltration of ground water into the tunnels caused lowering of the water level in overlying sediments which, in turn, promoted large surface settlements. Remedial measures, which included shotcreting and grouting after tunneling, did not restore the original water level. To cut down on this problem, subsequent tunneling was performed in combination with a treatment scheme that called for 1) pre-grouting the rock in advance of tunneling, the extent of which was determined by the local geology, 2) post-grouting, according to observed seepage, after the tunnel had been driven.

Over 70% of the tunnel line was driven through granite, with the remaining portions being excavated mostly in gneiss and norite. The granite was generally sound and unweathered. The gneiss and norite were mostly intact with occasional zones of marked schistocity. Several faults were intercepted by the tunneling.

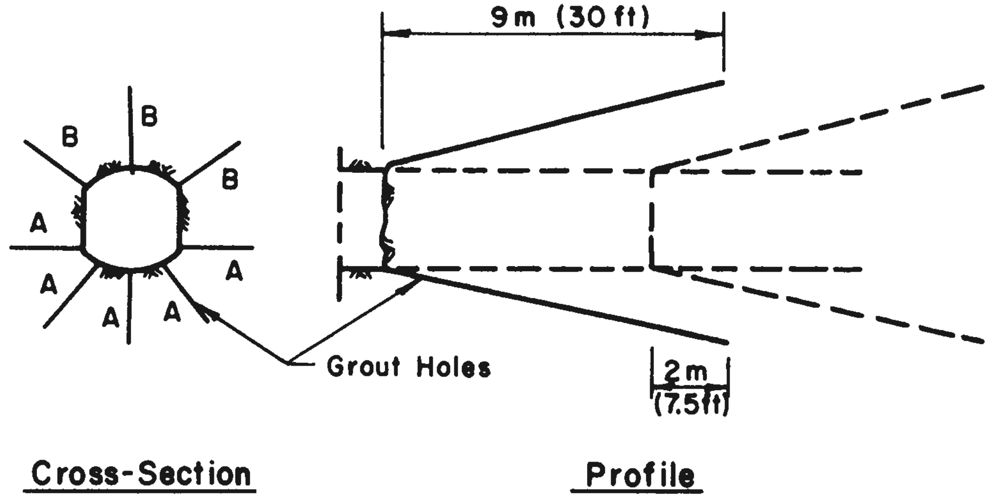
The tunnels were driven at an average depth of 35 m (115 ft.). Approximately 5 km (3.1 mi.) and 3 km (1.9 mi.) of tunnels were constructed with cross-sectional areas of 9-12 m^2 (97 to 129 $ft.^2$) and 5-8 m^2 (54 to 86 $ft.^2$), respectively.

The pattern of pre-grouting is shown in Fig. 2.14 for tunnels of each cross-sectional area. The grouting was performed in advance of the heading so that, at least, 2 m (6 ft) of grouted rock was ahead of the face at all times. For the 5-8 m^2 tunnels, grouting was performed a horizontal distance of 9 m (30 ft.) ahead of the face and the tunnel was drilled and blasted in 3 - 2.3 m (7.5 ft.) rounds before the next grout injection. For the 9-12 m^2 tunnels, grouting was performed a horizontal distance of 12 m (39 ft.) ahead of the face, and the tunnel was drilled and blasted in 4 - 2.4 m (7.9 ft.) rounds before the next grout injection. All sections of tunnel were grouted near the invert (bottom grouting), whereas full front grouting was performed only where the rock quality was judged to be sufficiently poor to require more comprehensive treatment. In all, full front grouting was performed over 13% of the tunnel line. At any section, the grouting inspector was authorized to change the number, siting, and angle of the holes as required by local ground conditions.

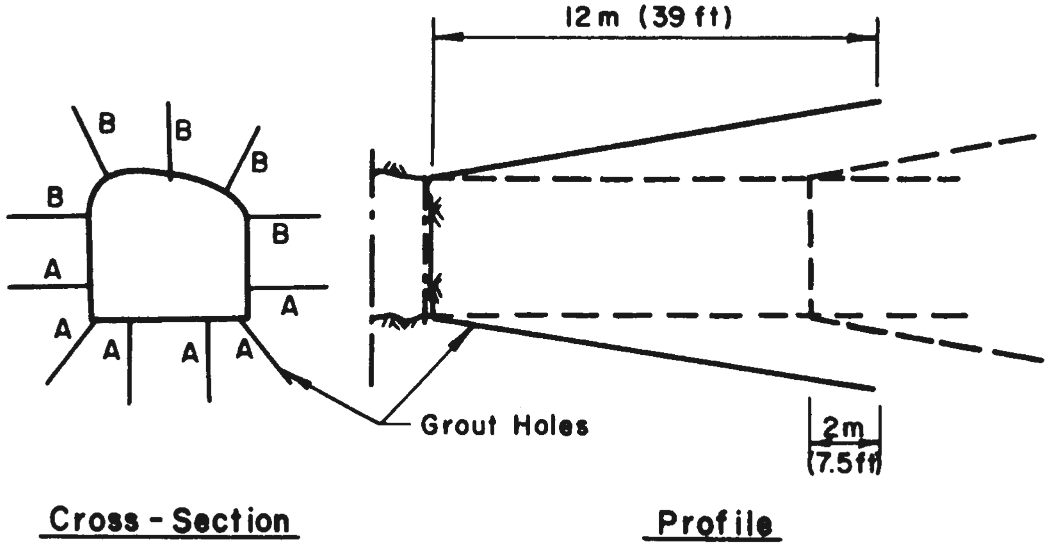
The grout holes were pressure-tested with water to determine the order of grouting. Grouting was started in the lowest lying holes with the largest water losses. Limhamn's[5] rapid setting cement (specific

---

[5]Equivalent to Portland Type III cement



a) Tunnel Area : 5-8 m^2 (54-86 ft^2)



b) Tunnel Area : 9-17 m^2 (97-129 ft^2)

Note : A = Bottom Grouting
B = Full Front Grouting

[After Lysen (57)]

Figure 2.14 Rock Grouting Pattern for Tunnels in Gothenberg, Sweden surface = 450m^2/kg) was used as the grouting medium. For each hole, grouting was initiated at a water-to-cement ratio of 3 and successively reduced in the order 2, 1, 0.8, and finally 0.5. Grouting was stopped for each hole at a maximum pressure of 3 MPa (433 psi).

Post-grouting was directed toward locations of relatively high water inflow. It was performed along a given section of the tunnel after the face had been advanced well beyond the area of treatment. Multiple post groutings of some areas were carried out. Upon completion of the project, the average water inflow was 7.8 l/hr. (2 gal/hr.) per 10m (32.8 ft.) length of tunnel.

A comprehensive system of ground water level and settlement measurements was started prior to tunneling. These indicated that tunneling had a negligible effect on water levels and surface subsidence.

An average 80 kg/m and 120 kg/m (57 lbs/ft. and 80 lbs/ft.) of cement were required to pre-grout the 5-8 m^2 and 9-12 m^2 tunnels, respectively. Similarly, an average 20 kg/m and 50 kg/m (13 lbs/ft and 34 lbs/ft.) were used to post-grout the 5-8 m^2 and 9-12 m^2 tunnels, respectively. The quantity of grout used was closely related to the rock quality. Grout consumption in fault zones was as much as 5 times greater than in sections of relatively intact rock. The costs of bottom grouting and full-front grouting corresponded to 50 and 100%, respectively, of the total blasting costs.

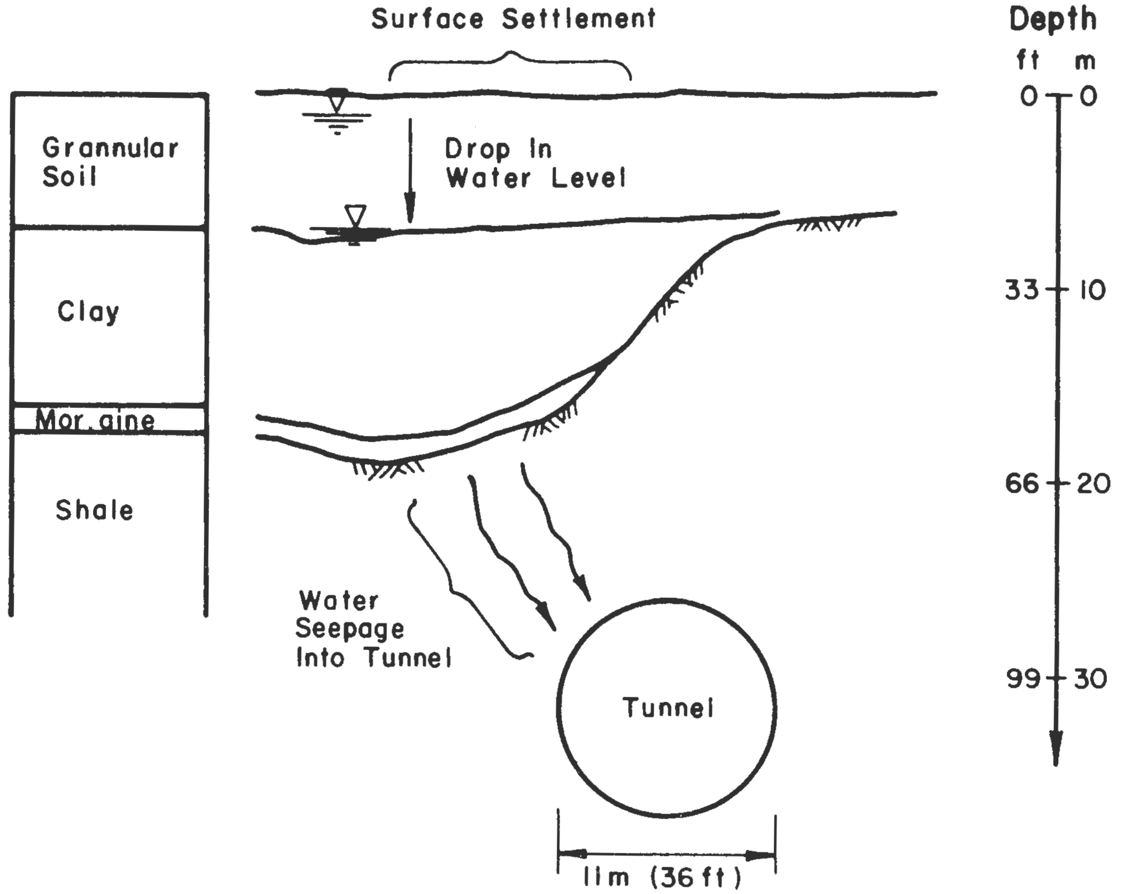
East-West Rail Tunnel in Oslo.

Another example of grouting to control small-scale infiltration is associated with the construction of subsurface railroad tunnels in Oslo. The bedrock geology of Oslo differs significantly from the Precambrian granite and gneiss structures that are predominant in most of Scandanavia. Oslo is located on a graben structure of Cambrio-Silurian rocks. The rocks are mostly sedimentary in origin.

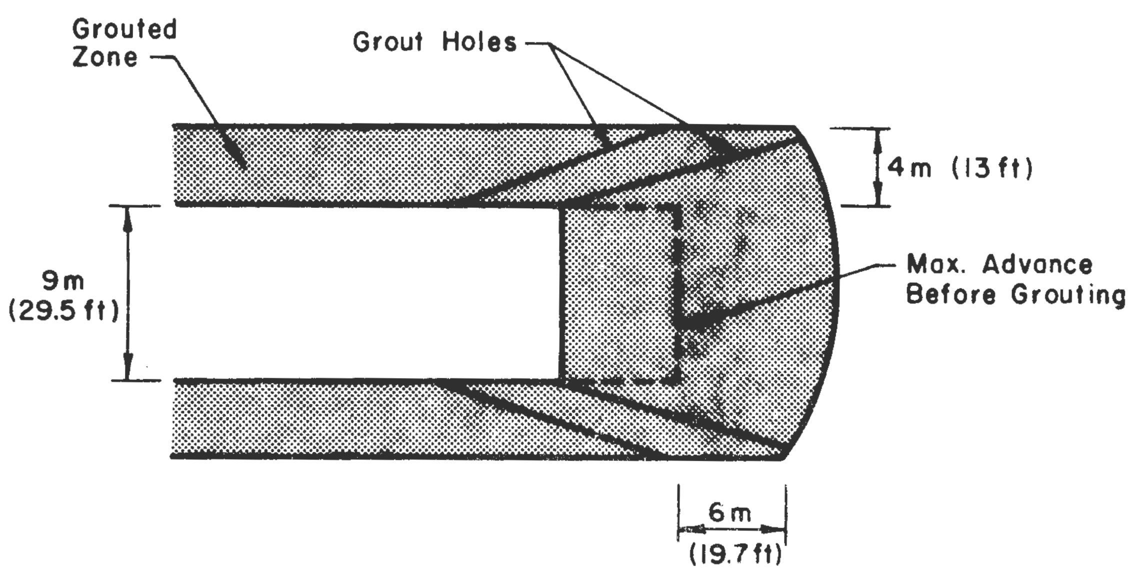
Tunneling in these rocks is occasionally carried out beneath buried valleys containing soft, compressible clays. The very sensitive to quick nature of the clays makes them susceptible to large settlement. When a tunnel is opened near a buried valley, it acts as drain into which water will migrate from the overlying deposit. Consequently, tunneling can be expected to cause substantial surface settlements by virtue of lowering the water level.

During construction of the east-west railway link through the center of Oslo, tunnels were driven beneath several buried valleys. Figure 2.15 shows the approximate condition of tunneling for a 220 m (722 ft.) - long section of the line just west of the Frogner Station. The tunnel was driven with drill-and-blast methods through a brown, non-fissle shale, that was dissected by numerous, tight joints. There was about 10 m (32.8 ft.) of rock cover between the tunnel and a buried valley, which contained a 10 m (32.8 ft.) - thick deposit of soft, sensitive clay. The clay was underlain by a 1 m (3.3 ft.) - thick layer of sandy moraine.

Pregrouting was performed for this section of the tunnel. As shown in Fig. 2.15, the grouted zone was placed a radial distance of 4 m (13 ft.) beyond the tunnel periphery and a minimum horizontal distance of 6 m (20 ft.) in front of the face. Each pregrouting stage was performed through an average 54 holes [approximate spacing = 1.5 m (4.9 ft.)] around the boundary of the tunnel. Approximately 7300 kg



a) Transverse Cross-Section



b) Longitudinal View

Figure 2.15 Drainage and Rock Grouting Pattern for Rail Tunnel in Oslo

(8 tons) of cement were injected during each pregrouting stage under a maximum pressure of 2 MPa (289 psi). Initially, water testing was performed but was discontinued because of excess leakage from neighboring holes.

Despite pregrouting, the water level in the overlying soil was drawn down several meters with consequent settlement exceeding 150 mm (5.9 in.) in places. Several remedial measures were adopted to reduce infiltration and restore the original water level. These included post grouting at locations of conspicuous inflow, grouting at high pressure behind the permanent lining, and recharging the water level with wells that were directed from either the ground surface or the tunnel.

2.3.3 Discussion

Grouting is intended to fill voids in decomposed or weathered rock and to plug rock fractures that vary in thickness, orientation, and the amount and nature of infilling materials. As such, the procedures used in rock grouting are highly dependent on the local geology, as well as the available equipment, and experience of the personnel involved. Grouting costs, especially for control of small scale infiltration, can be equal to or greater than drilling and blasting costs. Frequently, grouting jobs are performed under constraints that are mutually competitive such as the need for rapid treatment as opposed to careful, methodical injections.

Combined post and pre-grouting have proved useful for restricting water inflow as is evidenced by the low infiltration rates [1.2 to 10 liters/hr. (0.31 to 2.6 gallons/hr) per 10 m length of tunnel] that have been attained during tunneling for the Stockholm Metro [76]. However, the success of grouting may be highly dependent on the rock type and character of the local geology. The grouting program used in the granite and gneiss underlying Gothenburg, Sweden was successful in preventing surface subsidence, whereas combined pre and post- grouting through highly, tightly fissured shale in Oslo did not prevent a lowering of the water level and consequent surface settlement. At Dartford, grouting in highly fissured chalk cut down on water inflow, but was less effective than anticipated in eliminating the use of high air pressures.

Morfeldt [64], in reviewing the procedures for rock grouting, has emphasized the importance of flushing water through rock discontinuities to wash away infilled material previous to grout injection. In addition, he has pointed out ground water chemistries, such as water containing pyrite, that may retard cement setting and contribute to the eventual break-up of injected fissures.

When a continuous lining is used as the tunnel support, it is necessary to grout the annular void between the lining and exposed rock surface. As a rule, grouting of this kind is performed as soon as possible after the rock has been excavated. Correspondingly, the exposed rock is buttressed against the lining before time-dependent strains and construction-related vibrations can lead to loosening of the rock mass. This procedure may be especially important when tunneling through grouted rock. If grouting behind the lining lags the excavation by a substantial margin, loosening of the rock will promote further inflow of water and will work against the beneficial effects of the original grouting.

2.4 Ground Freezing

2.4.1 Review of the Method

Ground freezing is the lowering of the ground temperature to freeze interstitial water in soil or rock. Correspondingly, ground behavior is improved owing to a decrease in permeability and an increase in mechanical strength.

Freezing can be applied to any range of grain size. Generally, only soils beneath the water table are frozen, although freezing will improve the stand-up time and strength capabilities of partially saturated soils. Freezing is generally not applied when ground water flow is in excess of 1.2 m/day (4 ft/day). Large concentrated flows can be restrained where extremely cold temperatures are directed to a specific location. For example, liquid nitrogen has been used to control gushing water from sheet piles driven out of interlock.

The most widely used freezing system consists of a compressor operating with ammonia or freon to cool calcium chloride brine, which circulates through freeze tubes in the ground. Systems of this kind can deliver cold brine at -20°C (-4°F). Figure 2.16a shows a schematic of this system. Where required, compound compression of the ammonia or freon will cool the brine to -35°C (-31°F).

Extremely cold temperatures can be achieved with liquid nitrogen. The actual cooling is caused primarily by a phase change of the nitrogen from liquid to gas. As indicated in Figure 2.16b, this process does not recirculate the nitrogen, which is bled off to the atmosphere. Correspondingly, consumption costs are high. On average, it takes about 1000 liters

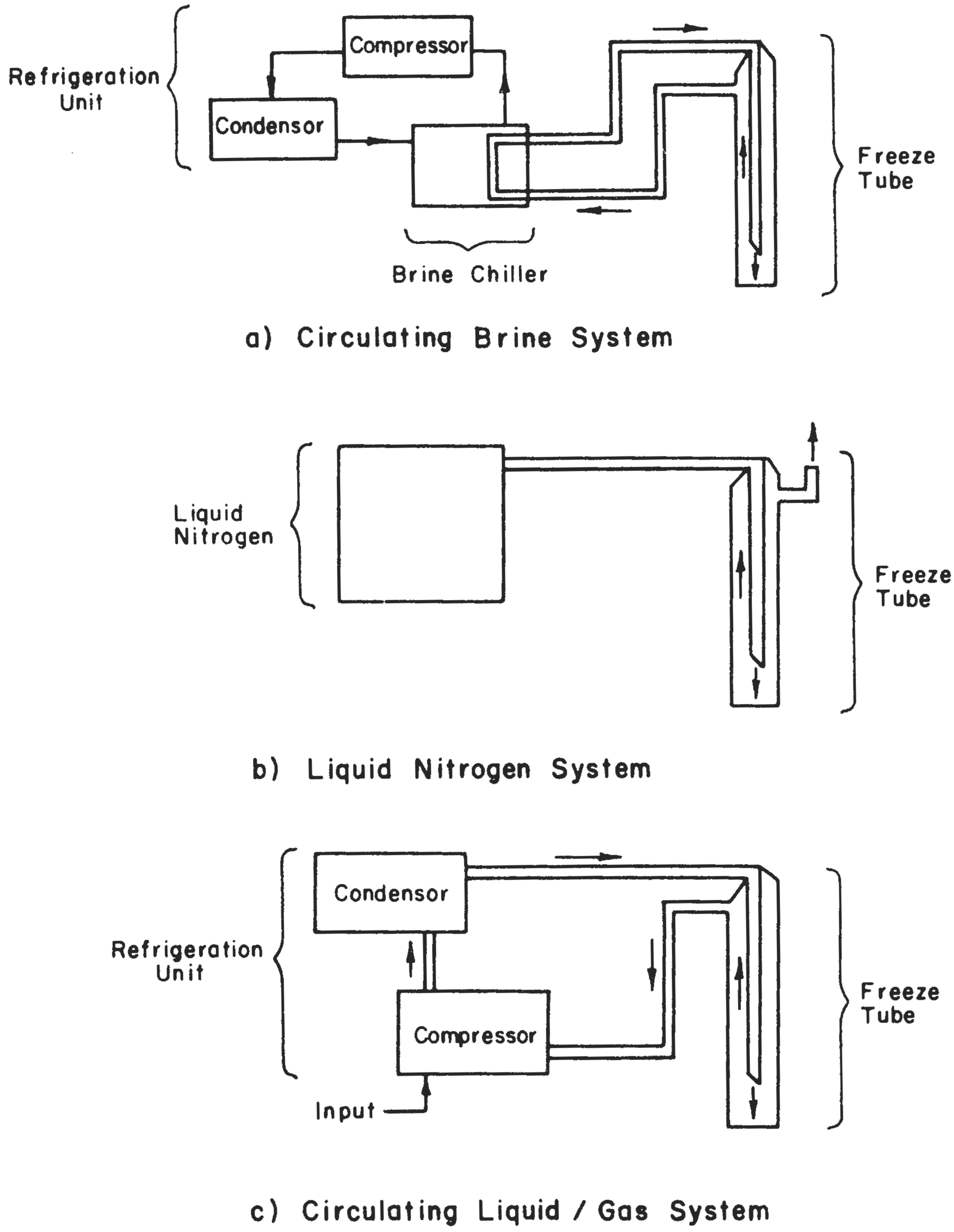


Figure 2.16 Ground Freezing Systems

(260 gallons) of liquid nitrogen to freeze 1 m^3 (1.31 yd^3) of soil. The consumption rate needed to maintain the frozen ground is approximately one third the rate needed to freeze it.

A third, less commonly used, system is shown in Figure 2.16c. The system introduces a liquid that evaporates within the freeze tubes. The resulting gas is condensed and, in turn, recirculated as a liquid. This process has been used during construction of the Helsinki Metro, where freon was used as the cooling medium.

Systems that use recirculation require compressors, condensors, and, if no external power is available, generating units. The noise associated with the equipment may be a problem, especially since the systems operate 24 hours a day. In contrast, liquid nitrogen is introduced directly from high pressure tanks and, since recirculation is not used, there is little noise from the operating machinery.

The decision to use a brine or liquid nitrogen system is based on considerations of time and expense. Liquid nitrogen is useful for rapid development of frozen ground and, thus, is attractive for emergency situations where time is essential in achieving stability. On the other hand, costs for the brine system are generally lower, particularly when the job will develop over an extended period of time or substantial zones of frozen ground are required. The two systems may be used in tandem with liquid nitrogen to establish a quick freeze and brine to maintain the frozen ground.

In almost all cases, temperature measurements of the frozen ground are performed. The measurements are accomplished with special bore-holes in which thermal transducers are placed. Readings of temperature provide a direct feedback as to the efficiency of the treatment. They are an essential part of the method.

Frequently, temperature measurements are supplemented by observation of a pressure relief hole. The hole is generally drilled near the center of the prospective tunnel or shaft. When closure of the ice occurs, there is a sudden surge of water from the hole. This observation combined with measurements that confirm sub-freezing temperatures are an excellent indication that freezing is continuous around the intended zone of construction.

A layer of frost often accumulates on the surface of the frozen ground. In a tunnel, this can lead to difficulties because melting will leave a void between the soil and tunnel lining. In addition, ice formation in the ground will resist compression and infiltration if grouting behind the lining is performed during the freeze. For example, during constuction of an escalator shaft for the London Underground, freezing was used to stabilize silts and silty fine sands. The shaft was lined with cast iron segments. After the freezing was stopped, the lining shifted in response to ice melting which, in turn, caused fractures in the water-proof caulking. Even when concrete is poured insitu, voids may be left by the frost melt, which are a source of lost ground and eventual settlement.

This problem can be minimized by careful grouting behind the lining. A two-stage program has been devised [34] wherein the first grouting is performed a short time after the lining is placed. This stage is intended to fill voids left after the "initial" thaw. For a concrete-lined tunnel this would occur after the concrete has set (approximately 5 to 7 days after pouring) and would compensate for melting during hydration of the concrete. The second grouting is perfomed after the freezing has been stopped and thermal sensors have indicated a return to temperatures above freezing.

To diminish the influence of cold temperature on the setting and hardening of concrete, either cement with a high specific surface or special additives may be used in the mix to promote a more exothermic reaction. In some cases, the thickness of the lining may be designed to be 10 to 15% greater than the dimension normally required for support.

Freezing in silty soil may lead to the formation of ice lenses as water is drawn to the frozen zone by capillary forces. The volume expansion caused by ice lensing can damage overlying structures and buried utilities. After thawing, the ground can settle significantly as thawing may transform the soil into a supersaturated medium of reduced shear strength and compressibility.

2.4.2 Recent Applications

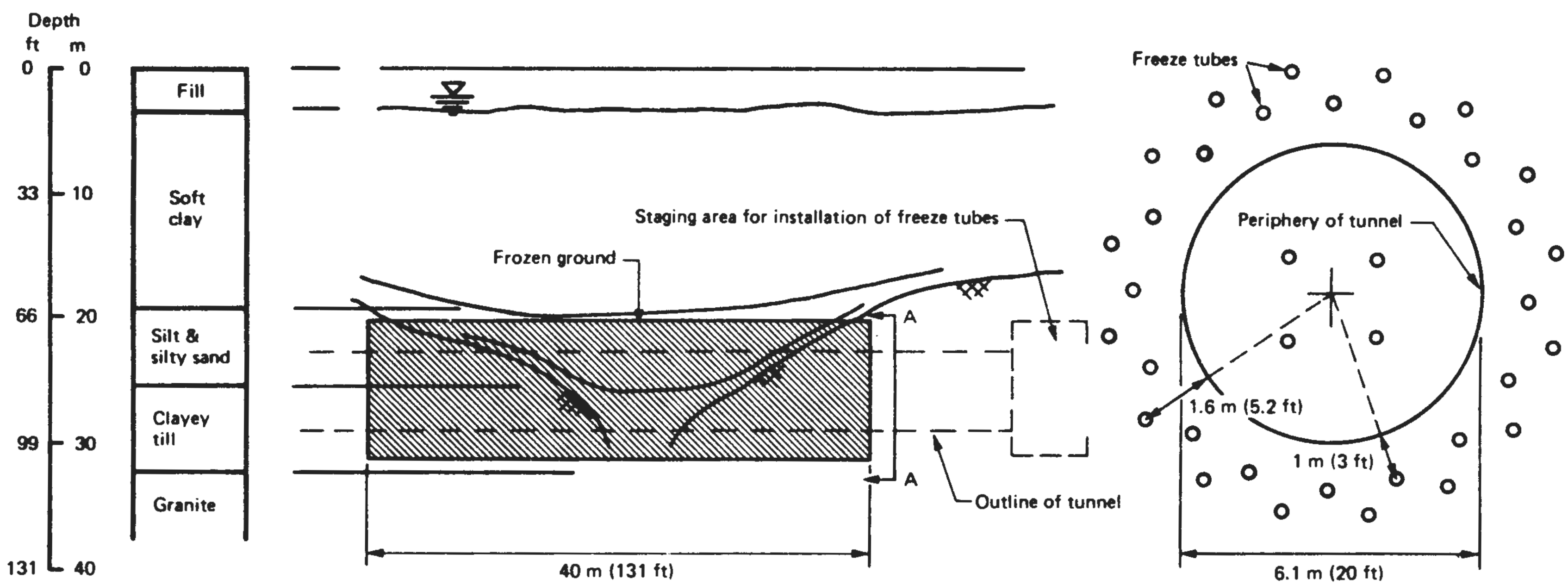
Case histories of ground freezing have been reported, although the literature generally lacks information of a specific nature concerning details of the technique and equipment. Because freezing is practiced by relatively few companies, information of this sort is often omitted from published case histories in the interest of maintaining a competitive standing. The two case histories summarized in this section, which refer to applications of the method for tunneling in Finland and France, describe elements of the process that are not usually found in the literature.

Helsinki Metro, Finland.

During construction of the Helsinki Metro, freezing was used to stabilize the ground at a fault zone, which intersects the line of the running tunnels. The fault, known as the Kluuvi Cleft, is deeply weathered and filled with sediments of till, silty sand, and soft, lightly overconsolidated clay.

Figure 2.17 shows a longitudinal view of the tunnel to scale with the soil profile. Freezing was performed over a 40 m (131 ft.) length of each twin tunnel by installing an array of freeze pipes as shown in cross-section A-A. Drilling for the installation of the pipes was performed from two staging areas, one of which is shown in the figure. The staging areas were separated by approximately 80 m (232 ft.) and drilling was coordinated so that there was an overlap of 3 m (10 ft.) at the center of the frozen zone. The bore-holes were set to within a maximum separation of 1.8 m (6 ft.), which was checked with a special survey device. To prevent ground loss during both the drilling and installation of the freeze pipes, the 114 mm-diameter (4.5 in.) casings as well as the drill bits were left in place when the drill rods were withdrawn. An air lock was set up so that compressed air could be used as a back-up system in the event of difficulties.

The freezing system, which used freon as the cooling medium, was similar to the type shown in Figure 2.16c. The pattern of the freeze tubes was intended to secure a 2.5 m-thick (8.2 ft.) annulus of frozen



A) LONGITUDINAL VIEW

B) CROSS-SECTION A–A

Figure 2.17 Ground Freezing Scheme for Tunnels on Helsinki Metro ground. Temperatures were monitored with thermistors[6] that were placed on 3 m (10 ft.) spacings along 5 boreholes around the tunnel. The ground was cooled to temperatures of between -7 and -12°C (20 and 10°F). Approximately 6 weeks were required to create a suitable frozen zone.

Excavation was performed by drill-and-blast techniques. Each round was approximately 1.2 m (4 ft) long and was pulled in 3 individual stages: a circumferential blast followed by a central blast with subsequent removal of the intervening portion. A cast iron,segmental lining was installed using 12 segments per ring, and caulked with lead.

The cost of the tunneling operation, including freezing, was estimated at $4 million.[7] This was approximately one-third the cost of building a station for the Helsinki Metro.

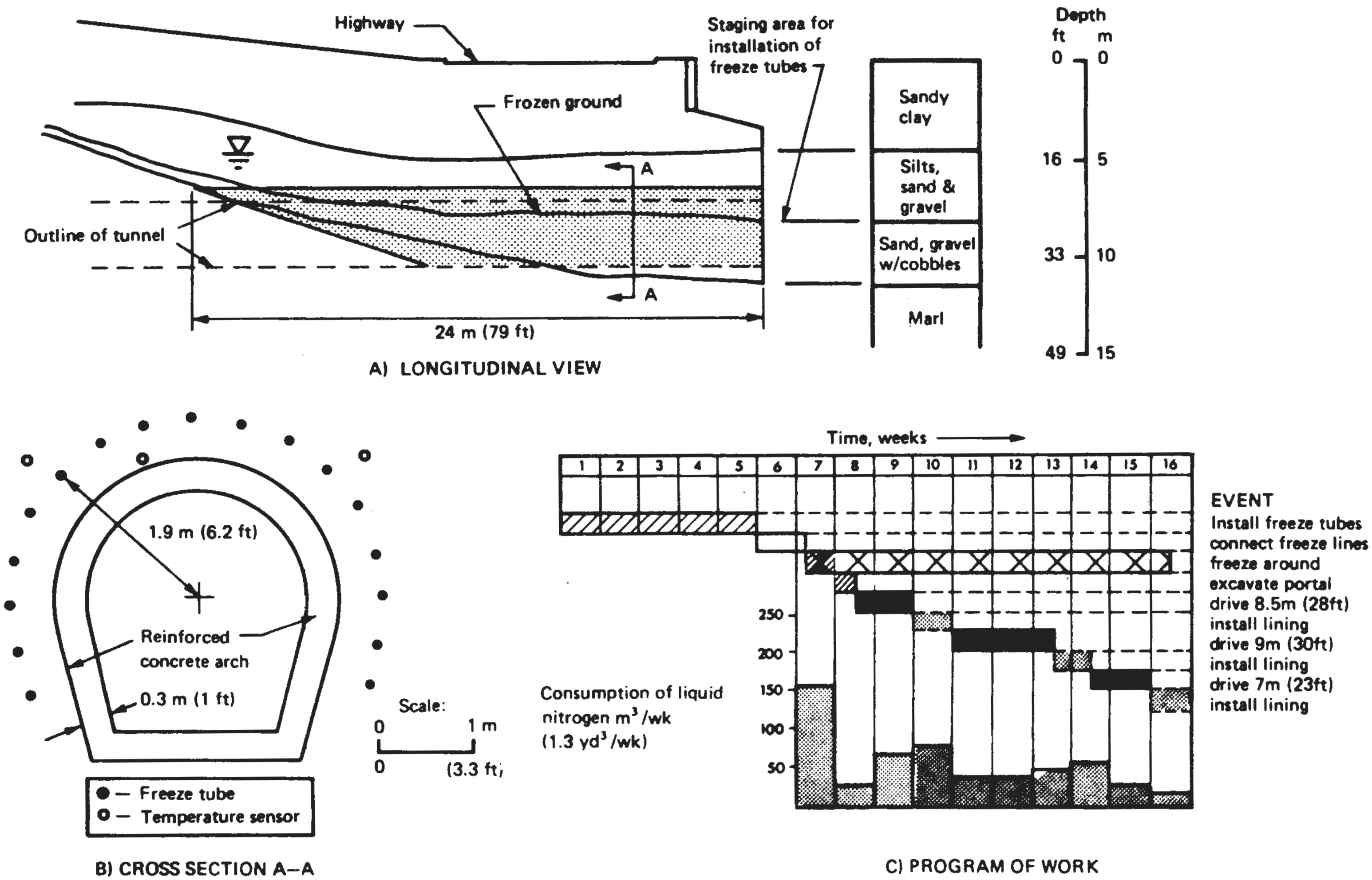
Water tunnel near Chambery, France. A second example of ground freezing is the construction of a water tunnel near Chambery, France. A detailed description of the job has been provided by Chapeau [14] and only its salient features are summarized here.

Figure 2.18a shows a longitudinal view of the tunnel and the soil profile through which it was driven. Before entering a stratum of marl and boulder clay, the tunnel had to be driven under a two-lane highway through silty alluvial deposits.

The freezing was performed over a length of 24 m (79 ft.) using liquid nitrogen as the cooling medium. A transverse profile of the tunnel is shown in Figure 2.18b in which the pattern of freeze tubes is illustrated. The holes for the freeze tubes were installed from an adjacent braced ex-

[6]Device using inverse proportionality between temperature and resistance.

[7]Based on a currency rate of 4 Finnish marks: 1 U.S. dollar



[after Chapeau (14)]

Figure 2.18 Ground Freezing Scheme for Tunnel near Chambery, France cavation. They were drilled with a tricone bit, using reverse circulation, and lined with 114 mm (4.5 in.)-diameter casings. The drilling was carefully performed and surveyed so that the maximum deviation of an individual hole was 60 mm (2.4 in.). Each freeze tube was composed of an interior 19 mm (0.75 in.)-diameter casing through which nitrogen was bled off to the atmosphere. The freeze tubes were set in the ground with a clay-cement grout. Two boreholes were made in which temperature sensors were placed at approximate 4 m (13 ft.) spacings.

Six tanks of 26 m^3 (34 yd^3) of liquid nitrogen were used to freeze approximately 130 m^3 (170 yd^3) of soil surrounding the tunnel. This corresponds to a consumption of 1200 liters (312 gallons) of liquid nitrogen per 1 m^3 (1.3 yd^3) of frozen ground. After approximately 10 hours, temperature at the sensor locations were measured between -1° and -10°C (30 and 14°F). Figure 2.18c shows the consumption of liquid nitrogen in relation to various stages of the excavation and time.

The tunnel lining was installed as a minimum 350 mm (14 in.) thickness of reinforced concrete to assure the setting of an effective 300 mm (12 in.)-thick arch. As a further measure against the cold, a special additive was used to supplement the exothermic reaction of the concrete.

During freezing a maximum heave of 26 mm (1 in.) was measured at the ground surface. After thawing an average centerline settlement of 100 mm (4 in.) was observed.

2.4.3 Discussion

Ground freezing has, in particular, two advantages. It can be applied to soils with a variety of grain size, which include fine sand and silt. In this respect, large variations of permeability associated with stratified or lenticular deposits will have little influence on the effectiveness of freezing. Furthermore, frozen ground is self-supporting and, under the majority of tunneling conditions, requires no additional reinforcement.

Flowing conditions may significantly increase the time necessary to secure a satisfactory freeze. Lee (52) describes the freezing used to stabilize soil surrounding an escalator tunnel, where a combined system of brine and liquid nitrogen was used in silty sands. Measurements showed that ground water across the 10 m-wide (33 ft.) frozen section was subject to a differential water head of 0.23 m (0.75 ft.). The time required to achieve a suitable freeze was over 3 months.

Ground freezing is sensitive to concentrations of heat that might occur if freeze tubes are damaged or if ground water flow is locally high. Either of these conditions can cause "windows" in the frozen ground where water seepage will contribute to instability. For these reasons freezing set-ups should be carefully monitored for temperature. This provides direct feedback on the condition of the soil and is essential for judging the effectiveness of the scheme.

The costs related to freezing are cumulative. They increase with the duration of the project as the expense of running the equipment increases, or, for the case of liquid nitrogen, as the nitrogen losses accumulate. Consequently, ground freezing is often priced at a higher level than other methods of ground stabilization, which tends to limit the technique to emergency cases or as a last resort method.

With respect to cost, it should be acknowledged that, for jobs of short duration, freezing can be competitive with other methods of ground treatment. Experience gained during tunneling for the London Underground has shown that, when the maintenance of frost by brine circulation is limited to 5 or 6 weeks, the cost of freezing and grouting are similar [approximately $\$160/m^3$ ($\$122/yd^3$)] for equivalent tunneling conditions.

There are three aspects of the freezing technique that deserve careful consideration when tunneling:

Frozen Ground Relative to the Tunnel. In some instances, only the ground in the upper portion of the tunnel needs to be stabilized, and freeze tubes can be installed from the ground surface to accomplish this. Other conditions, however, will require treatment of the ground surrounding the entire tunnel. An annulus of frozen soil is difficult, if not impossible, to achieve by freeze tubes installed from the surface. As is evidenced by the previous case histories, the ground under these circumstances is frozen with a horizontal array of tubes. Installation of the tubes requires a staging area and, correspondingly, affects the actual tunneling scheme. Tunneling must be coordinated with the freezing operation to provide for placement of the freeze tubes in addition to lowering the ground temperature and excavating the frozen material.

Thawing of the Ground. Ground freezing may cause a layer of frost to form between the tunnel lining and adjacent soil. In addition, the frozen soils resist compression and infiltration when grouting behind the lining. When thawing occurs, voids can form throughout the zone bordering the tunnel which, in turn, can cause lining deformation and loss of ground. Careful, two-stage grouting at different times during the thaw has been used to remedy this problem.

Soil Heave and Settlement. Freezing, particularly in silty soil, will promote the formation of ice lenses and consequent surface heave. It has been suggested that rapid freezing will cut down on ground expansion [78], but, as freezing for the water tunnel near Chambery has shown, significant heave can occur even when liquid nitrogen is used. Thawing may be accompanied by settlement that exceeds the amount of heave because the shear strength and compressibility of the soil can be altered by freezing.

3. CONCRETE DIAPHRAGM WALLS

3.1 Introduction

A concrete diaphragm wall is a continuous concrete wall built in sections from the ground surface. In general, the wall is formed by excavating a series of adjoining panels or circular shafts into which reinforcing steel and concrete are placed. The details of construction will vary according to the specific type of diaphragm wall.

This report concentrates on three types: 1) cast-insitu walls, 2) prefabricated walls, and 3) secant pile walls.

Concrete diaphragm walls have been used extensively on rapid transportation systems in Europe where they have replaced more traditional support elements such as soldier pile-lagging or steel sheet piles. Because they are stiff, continuous structures, they can be used in lieu of both dewatering the soil and underpinning adjacent buildings. In addition, the walls can be incorporated into the permanent underground structure.

Several works have been published that include an examination of concrete diaphragm walls. A state-of-the-art review on lateral support systems and underpinning by Goldberg et al (32) contains a detailed summary of the method. Other notable studies on the subject include works by Xanthokos (96), D'Appolonia (20), and Sliwinski and Fleming (79).

3.2 Cast-Insitu Walls

3.2.1 Review of the Method

A cast-insitu wall is formed by establishing a line of contiguous, concrete panels. Each panel is made by excavating a slurry stabilized trench into which a cage of reinforcing steel is placed. Concrete is subsequently tremied into the trench, working from the bottom upward so that the slurry is displaced by the rising movement of concrete.

In plan dimensions, the panels generally range from 0.6 to 1.0 m (2 to 3.3 ft.) in width and from 4 to 7 m (13 to 23 ft.) in length. In Britain, panel lengths are most frequently between 5 and 6 m (16.4 and 20 ft.). Panel lengths may be determined by the proximity of buildings. For example, the construction of cast-insitu walls for the Brussels Metro has tended to follow the pattern summarized in Table 3.1.

A common form of construction is to build the wall in a staggered sequence of primary and secondary panels as shown in Fig. 3.1A. The junction between primary and secondary panels is formed by casting the primary units with two stop-end casings that are removed after the concrete has set. The secondary panel can then be excavated and concreted to form a half-circle joint with the primary panel. Sections of the London Underground and Lyon Metro were constructed in this manner. An alternate method consists of building the wall in a continuous sequence of panels. As shown in Fig. 3.1B., the leading edge of the most recently excavated panel is provided with a stop-end casing. The casing is removed to form a joint with the next panel in the sequence. This method was used for cast-insitu walls on the Brussels Metro.

Generally, the slurry is between 4 and 6% bentonite by weight, but there are no hard and fast rules concerning the slurry composition. For example, trenches in soft to medium Norwegian

TABLE 3.1

Lengths of Cast Insitu Panels

Constructed for the Brussels Metro

Panel Length, m	(ft.)	Distance from Buildings, m (ft.)
9	(29.5)	5 (16.4)
5	(16.4)	between 2 (6.6) and 5 (16.4)
2.4	(7.9)	2 (6.6)

NOTE: Panels: 0.8 to 1.0 m wide (2.6 to 3.3 ft.) and between 10 and 20 m (33 and 66 ft.) deep. Soil profile is described in Appendix A.

clay have been stabilized on a production basis by using water only (21). For very pervious deposits, fine sand may be added to the bentonite slurry to assist in plugging the voids of the surrounding soil. In this regard, the slurry supervisor is critically important. His decision concerning the slurry mix will be founded on previous experience and, often, on intuitive and textural judgements regarding the best slurry for a given ground condition.

The slurry at the bottom of the trench tends to become dense and viscous as debris settles during excavation. A dense, basal slurry can resist displacement when concrete is tremied into the panel. Perhaps one of the most difficult situations in this regard occurs when panels are "toed into" calcereous materials such as chalk or limestone. A thick, basal slurry is best removed prior to placement of the reinforcing cage. This can be accomplished by dredging through an air lift or submerged pump directly from the bottom of the trench as fresh slurry is simultaneously introduced at the top of the panel.

Opinions differ as to the best type of excavator for establishing the panel trench. Trenching may be performed with either a wire-suspended clamshell or Kelly-bar excavator. At the Stundenterlunden section of the Oslo underground railway and along sections of the Brussels Metro, excavation with a wire-suspended grab resulted in panels that were placed within a vertical offset of between 1:100 and 1:200. In contrast to a wire-suspended grab, the Kelly-bar system has the apparent advantage of greater stiffness, especially at shallow depths. As the depth of the trench increases the stiffness of the Kelly-bar is reduced

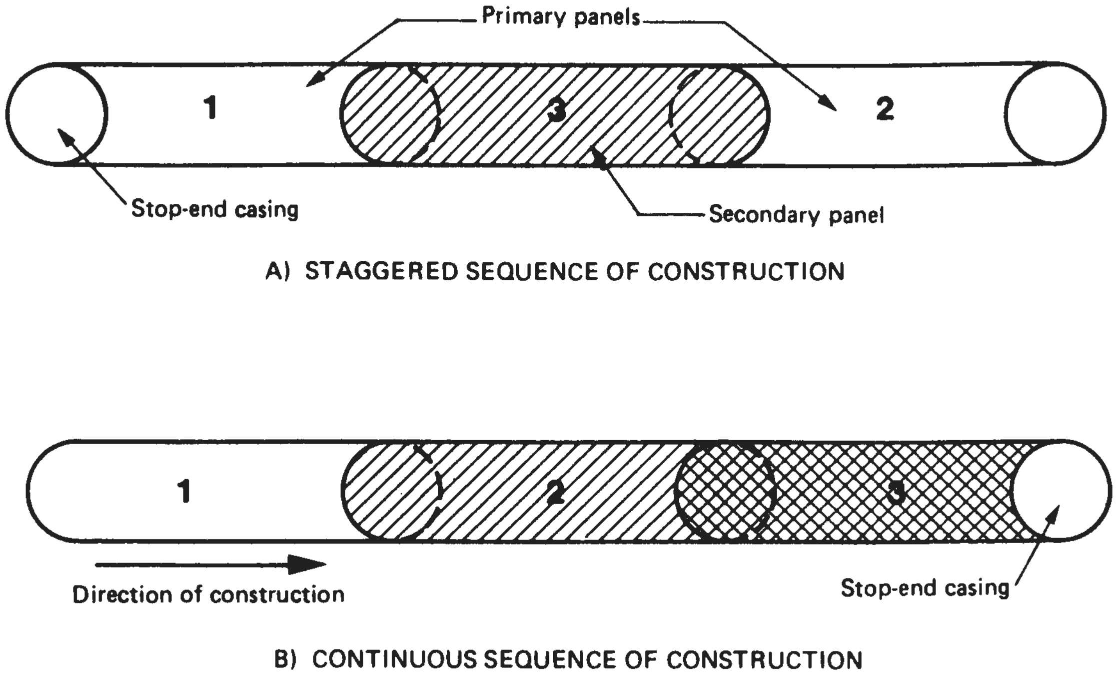
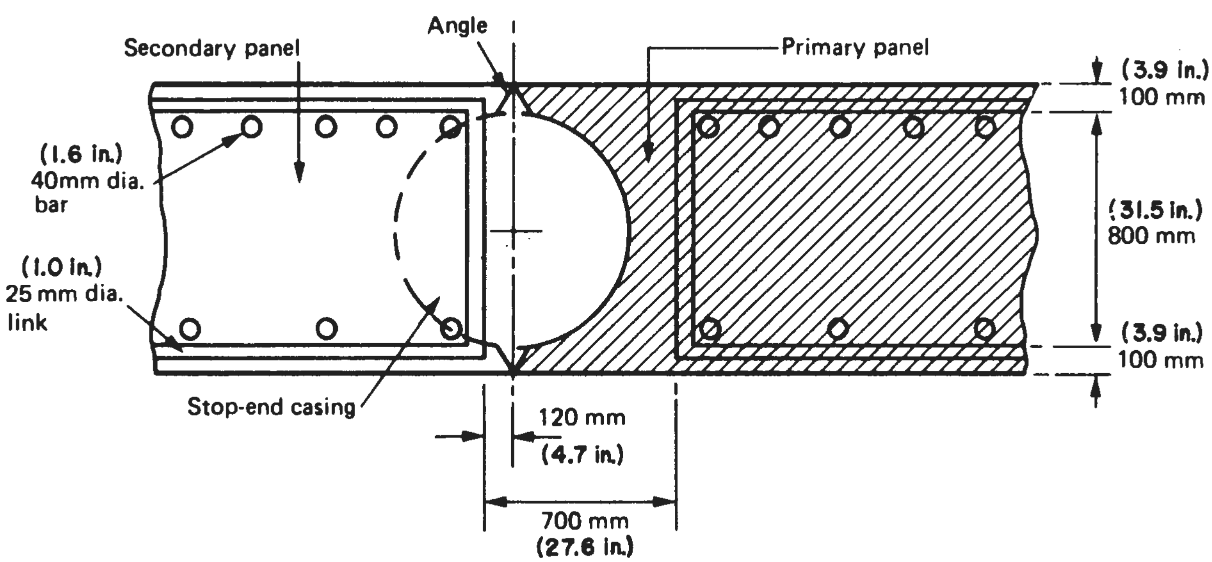


Figure 3.1 Sequence of Panel Construction



[after Jobling and Lyons (41)]

Figure 3.2 Cast-Insitu Wall at Heathrow Airport so that for deep trenches there is little practical difference between the two systems. However, shallow excavation with a stiff system contributes to an initially straight trench that acts as a guide in directing deeper penetration of the excavator.

The amount of reinforcing steel used in the walls will vary according to design requirements. Fig. 3.2 shows a plan view of a typical section of the cast-insitu walls for the Heathrow Central Station of the London Underground. The slurry walls are the permanent walls of the structure. They are approximately 20 m (66 ft.) deep in a soil profile consisting of 8 m (26 ft.) of sand and gravel underlain by stiff to hard London clay with the water level at 5 m (16.4 ft.) below ground surface. The amount of reinforcing steel for the structure is in the proportion of 90 kg of steel per m^2 area of wall (18.0 lbs/$ft.^2$). In certain instances the amount of steel may be considerably higher, as in the approach section for the second Dartford Tunnel where as much as 140 kg/m^2 (28.6 lbs/$ft.^2$) of steel is used. In this case a special crane was necessary to place the reinforcing cages, some of which weighed 35,000 kg (38.5 tons). Panels constructed for sections along the Brussels Metro contained between 80 and 180 kg/m^2 (16.4 and 36.8 lbs/$ft.^2$) of reinforcing steel.

Increasing the amount of reinforcing steel may cause problems in the construction of the panels because, as the number of steel bars increases, the ability to displace the slurry with tremied concrete diminishes. This can lead to inclusions of bentonite slurry within the concrete. Some contractors (94) consider 100 kg/m^2 (20 lbs/$ft.^2$) as an upper bound for the efficient construction of panels between 0.8 and 1.0 m wide (2.6 and 3.3 ft.). Engineers for the Brussels Metro favor an increase in wall thickness to 1.5 m (4.9 ft.) with a corresponding decrease in reinforcing steel (23). This improves the economy of the structure because the cost of increased concrete is offset by savings in the steel.

A flush contact between adjoining panels is difficult to achieve on a consistent basis and, consequently, leakage at some of the joints should be expected if no special precautions are taken and the walls are to retain water-bearing soils. Occasionally, a portion of the half-circle contact area will break off during excavation leaving a void which is vulnerable to slurry inclusion. At other times, concrete may migrate around the stop-end casings where it hardens and subsequently interfers with excavation in the adjoining panel. Both these conditions are illustrated in Fig. 3.3. Most often, the joints showing bentonite inclusions and minor leakage can be raked and treated with a sealant as they are exposed during excavation. In rare instances the leakage can be severe and may necessitate treatment by grouting along the exterior face of the joint.

3.2.2 Recent Applications

This section of the report concentrates on projects where field measurements have shown several important characteristics of wall behavior at two stages: 1) during excavation of the panel trench, and 2) after the walls are in place and the excavation is deepened to subgrade.

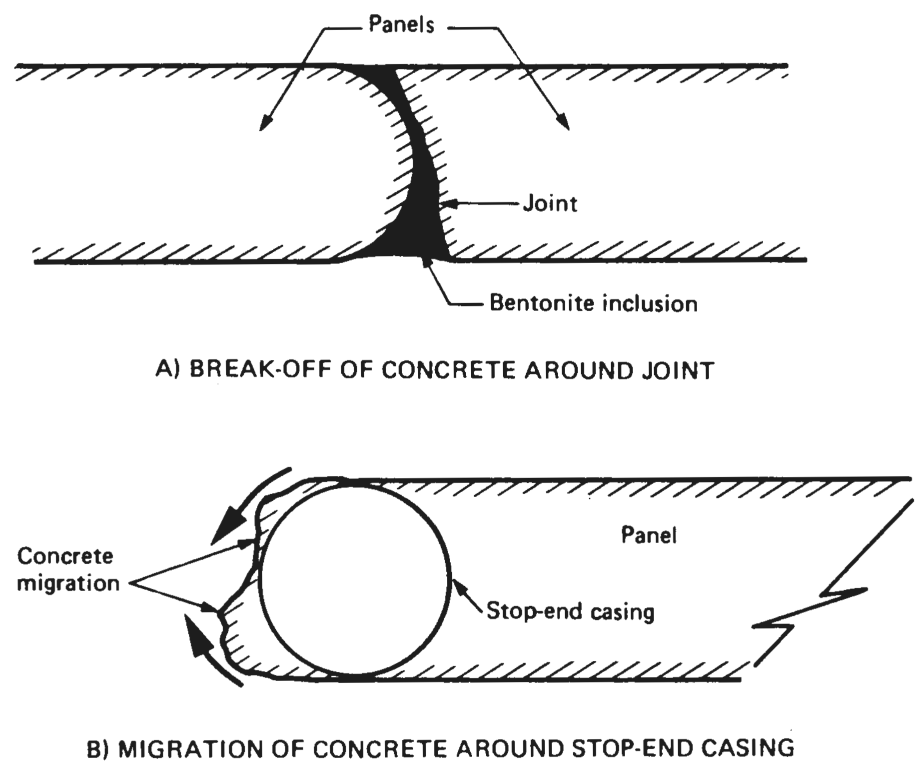


Figure 3.3 Defects in Panel Construction

Excavation of the Panel Trench

Figure 3.4 summarizes the lateral displacements measured in the ground adjoining three different test panels in various soil profiles. The plan dimensions and depths of the panels, as well as the plan location of the inclinometer tubes with which the measurements were taken, are indicated in the figure.

Figure 3.4A shows the lateral displacements caused by excavating a slurry-filled trench in London clay. A maximum movement of 16 mm (0.63 in.) was measured just below the sand/clay interface, which is typically a densely fissured zone. The lateral displacements for panel excavation in soft to medium clay, shown in Fig. 3.4B, have been derived, in part, from convergence measurements by assuming that the displacement at one side of the trench was equivalent to half the trench convergence at a particular elevation. The measurements reflect the influence of time and the specific gravity of the slurry, which was decreased from 1.24 to 1.10 to 1.00 at 12, 19, and 30 days after excavation. In contrast to the movements in stiff and soft clay, the lateral movements caused by panel construction in decomposed granite (Fig. 3.4C) were relatively large. These movements were measured for a test panel in Hong Kong. The test panel took 10 days to excavate. An additional 10 days elapsed before measurements were made corresponding to the lowered slurry level. The long period of time during which the panel was open and the lowering of the water level contributed to the large movements.

The maximum settlements caused by excavating the panels were approximately 6 mm (0.25 in.) in all cases and were confined within a

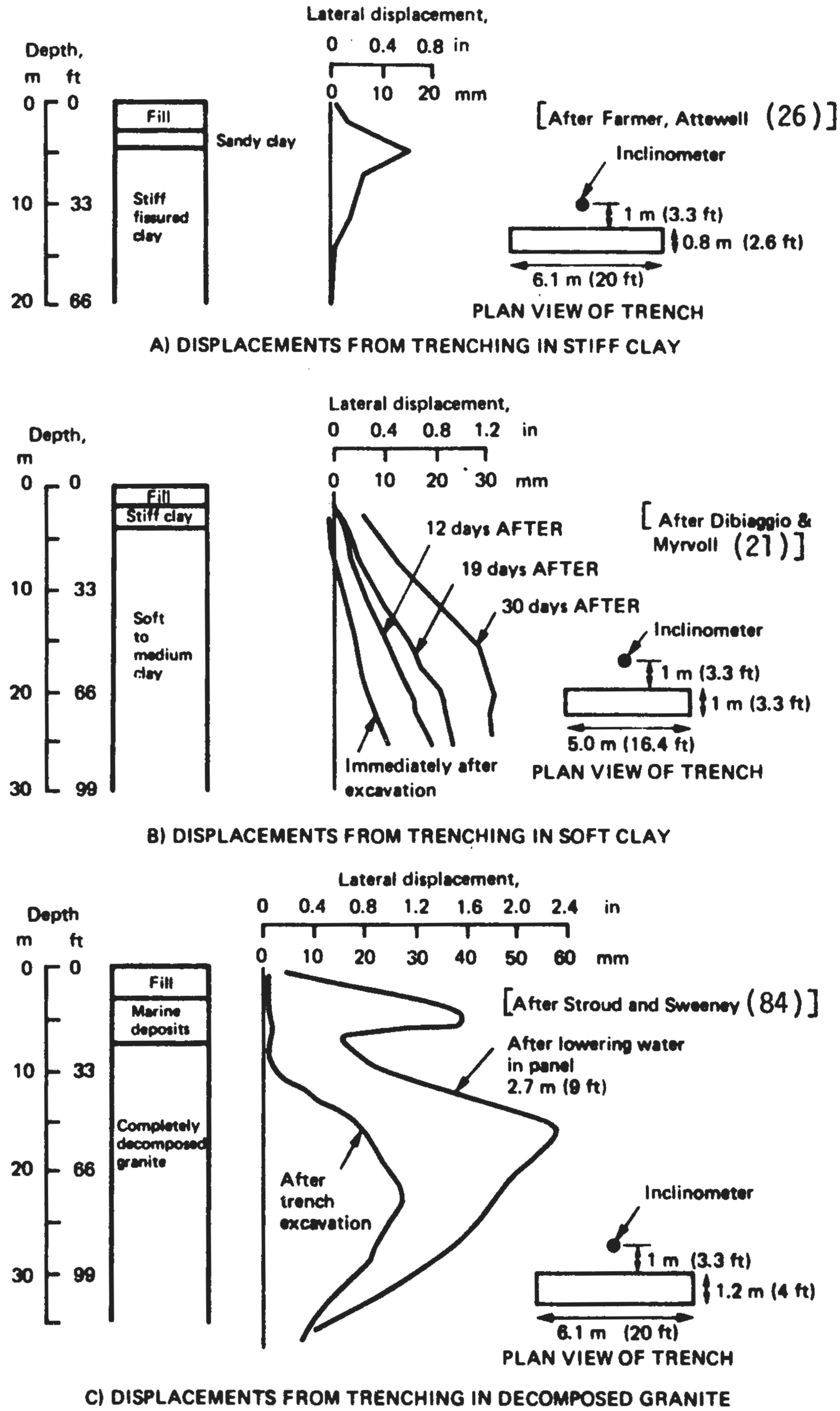


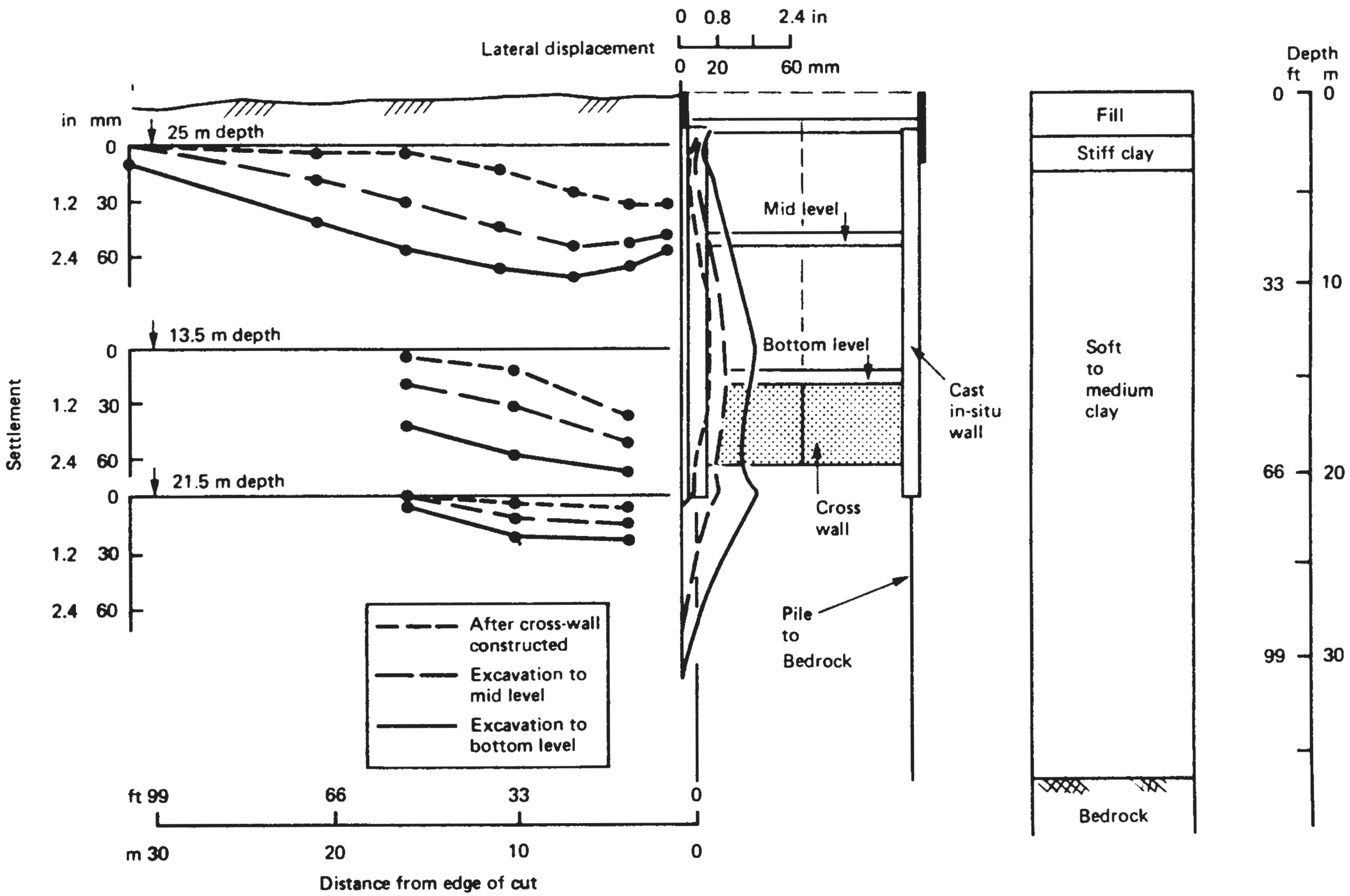
Figure 3.4 Summary of Lateral Movements Caused by Panel Excavation

distance of 5 to 6 m (16.4 to 19.7 ft.) from the sides of the panels. The volume of settlement in each instance was substantially less than the volume of lateral displacement. It should be noted, however, that the relatively small length of a panel promotes arching in the soil. This prevents the full influence of deep volume loss from being transmitted to the ground surface. The arching effects can be partially or entirely lost upon further extension of the wall. In fact, the excavation of additional panels in the decomposed granite led to a cumulative increase in settlement. The volume of settlement following the construction of several adjoining panels was roughly equal to the volume of lateral displacement adjacent to an individual panel.

In certain instances, large movements may develop during panel construction as a result of spalling or partial collapse along the side of the trench. Measurements of trench width summarized by Karlsrud (46) show spalling in a basal stratum of sand and gravel during panel excavation into bedrock, even though bentonite slurry was used. During construction of the Heathrow Central Station, local sloughing occurred in the upper portion of some panels, mainly in a zone of gravel and coarse to medium sand. In this case, the sloughing may have been aggravated by the concurrent driving of sheet piles near the area of panel excavation. Trenches in stiff, fissured clay, if open for a significant period of time, will lead to progressive softening and swelling of the clay. During construction of the Barbican Centre, a trench in London clay collapsed after it had been left open with bentonite slurry for approximately three weeks. Additional observations at Barbican (82) indicate that local collapse depends, in part, on the shape of the trench. Special panels, which were used at corners and other junctions of the foundation, showed considerable spalling at the high angle intersections of T and L-shaped panels.

Excavation with Walls in Place

A comprehensive series of measurements have been summarized by Karlsrud (45, 46) for the 15 m-deep (49 ft.) Studenterlunden excavation in Oslo. The special construction techniques employed for this cut have been discussed by Eide et al (25). The soil profile is predominently made up of normally consolidated, marine clays, ranging in shear strength from 18 to 36 kN/m^2 (370-740 psf). The walls of the cut were composed of 1 m-thick (3.3 ft.) cast-insitu panels. Prior to the main excavation, 1 m-thick (3.3 ft.) transverse walls were constructed at 4.5 m (14.8 ft.) intervals along the length of the open cut. These special cross-walls were established at the bottom of the cut and were between 6 and 12 m (19.7 and 39.4 ft.) deep. Bottom heave was restrained by shearing resistance developed along both the longitudinal panels and the crosswalls. To support the structure against settlement, H-piles were driven to bedrock through casings that were cast in the longitudinal panels. The settlements at various depths behind the edge of excavation and the lateral displacements of a cast-insitu wall are shown in Fig. 3.5 with respect to various stages in the construction sequence. The movements are small, especially considering the soft nature of the clay. These displacements compare favorably with movements measured at another cast-insitu wall excavation in soft to medium Norwegian clay, described by Di Biaggio and Roti (22).



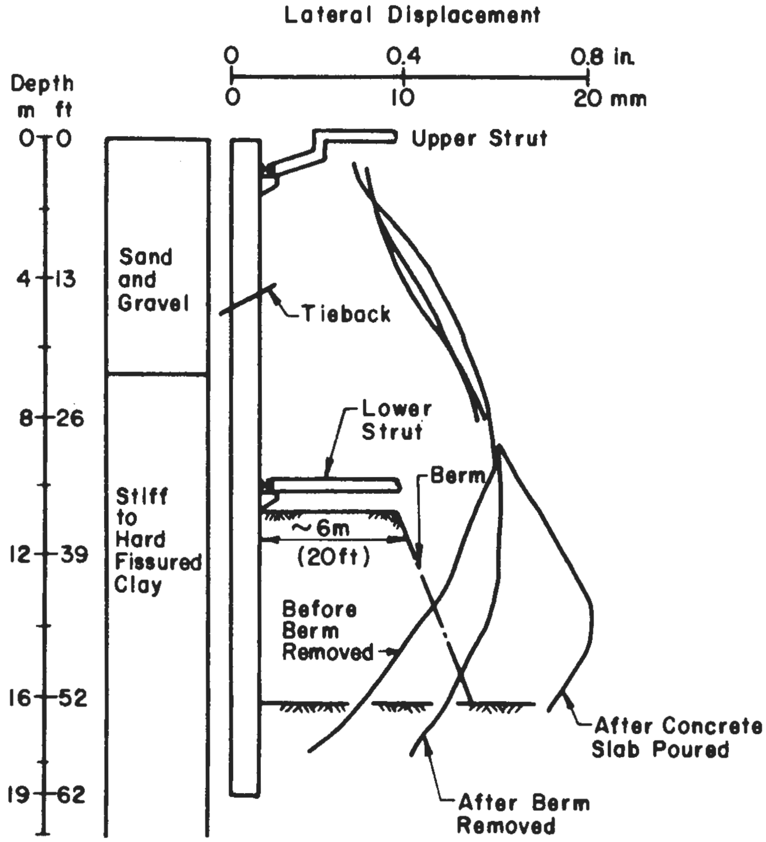
[after Karlsrud (46)]

Figure 3.5 Soil Displacements Associated with the Studenterlunden Excavation, Oslo

At times, substantial movement can result even where experience has shown that cast-insitu walls are well suited for the soil at hand. Of special interest in this regard are a set of lateral wall movements described by St. John (83). Fig. 3.6 shows the lateral displacement profiles both prior and subsequent to berm removal at the bottom of a 16 m-deep (52.5 ft.) excavation in London clay. The cast-insitu walls at this project were 0.6 m-thick (2 ft.) and were extended 3 m (9.8 ft.) below the bottom of excavation. After berm removal, loads in the lowest brace more than doubled and continued to increase with time. A lateral displacement of as much as 20 mm (0.8 in.) occurred in the bottom portion of the wall, primarily as a rotation about the lowest brace. When over-consolidated clay is exposed in this manner, progressive swelling and softening occurs as negative pore pressures in the material are dissipated. By pouring the bottom slab, both drainage and the slaking influence of the atmosphere are reduced so that the clay is more likely to retain its inherent strength. Correspondingly, berms contribute to stability by virtue of their insulating effect on the toe of the cut.

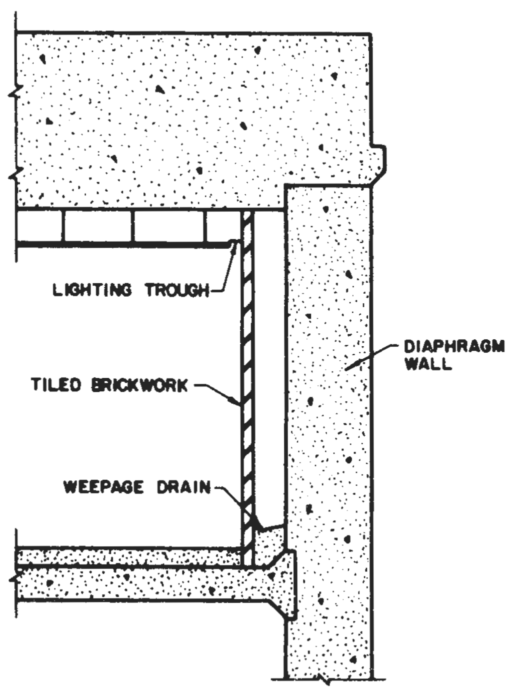
In London clay, excavations supported by cast-insitu walls have been accompanied by relatively small displacements. Settlements for several excavations reported by St. John (83) were in the order of 0.2 to 0.4% of the excavation depths, even though some cuts were as deep as 18 m (59 ft.).

Settlements adjacent to cast-insitu walls for the Brussels Metro have been very small. For example, the settlements along the building fronts adjacent to slurry wall excavation on the Rue Sainte



[After St. John (83)]

Figure 3.6 Lateral Wall Movement for a Braced cut in London clay



(Courtesy of London Transport)

Figure 3.7 Cross-Section of Diaphram Wall at Heathrow Central Station

Marie were between 2 and 5 mm (0.1 to 0.2 in.). The buildings were approximately 10 m (33 ft.) from the edge of the cut. The excavation was 15 m (49 ft.) deep in a soil profile composed mostly of sands and gravels with a water level 3 m (9.8 ft.) below the ground surface. Three levels of struts were used at this site. On average, the buildings adjacent to slurry wall excavation in Brussels have settled 5 to 10 mm (0.2 to 0.4 in.), even though many were located between 2 and 5 m (6.6 to 16.4 ft.) from the edge of excavation.

3.2.3 Discussion

Field measurements indicate that ground movements associated with cast-insitu walls in a wide variety of soils are likely to be small if care is exercised during trenching and adequate bracing is provided for the excavation. Even in soft to medium Norwegian clay, excavations in excess of 15 m (49 ft.) have been performed such that adjacent settlements were less than 0.5% of the total excavation depth. The small ground movements associated with careful construction recommend the use of cast-insitu walls in lieu of underpinning adjacent structures.

Cast-insitu walls need not be restricted to use as temporary support. In fact, considerable savings may be realized by incorporating the walls into the permanent structure as has been done for the London Underground and Brussels Metro. Water, which occasionally seeps from the joints between adjoining panels,can be controlled with a small drainage space intervening a false wall and the cast-insitu structure. Such a design was used for Heathrow Central Station of the London Underground, a cross-section of which is shown in Fig. 3.7.

The construction of cast-insitu walls requires skill and familiarity with the equipment. Perhaps the most important person associated with wall installation is the platform foreman, who co-ordinates the various aspects of work at a given rig. In France, the platform foreman is generally an experienced construction manager. His judgements are crucial with respect to verticality of the wall and continuity of adjoining panels.

The construction of cast-insitu walls is likely to result in some voids and separations between adjoining panels. In the great majority of cases these require only minor treatment, such as cleaning the internal surface of the joint and refilling with cement or other sealants. Local discontinuities in the wall are usually inpregnated with bentonite, which will generally block leakage until the void is treated. However, when deep walls are constructed to retain a large hydrostatic head, there is a possibility that high water pressures can lead to erosion and local boiling conditions before the void is treated. Difficulties of this kind, although rare, are not unknown. It should be recognized that, even when treated, joints may show local dis-coloring from moisture and small areas of gradual water accumulation.

Excavation of slurry stabilized trenches involves the batching, cleaning, and disposal of bentonite slurry. During the excavation process, slurry may overlap the sides of the trench or drip from the excavation equipment. Surface accumulations of bentonite slurry and spillage onto roads can have an adverse impact on the surroundings.

3.3 Prefabricated Concrete Walls

3.3.1 Review of the Method

Prefabricated walls are composed of reinforced concrete panels that are cast previous to their installation in the ground. Each precast panel in placed within a slurry stabilized trench, the excavation of which is performed in the same way as for cast-insitu walls. Most walls are built as a line of continuous panels that interlock by means of tongue-and-groove or other types of joint. Panel thickness will vary according to structural requirements, being commonly between 400 and 500 mm (1.3 and 1.6 ft.). Panel lengths are generally between 2.5 and 4 m (8.2 and 13 ft.). The weight of an individual panel often is controlled by the load capacity of the crane on site. As city ordinances may place restrictions on the size of the crane, panel dimensions may need to be coordinated with local construction guidelines. Usually, panel depth is limited to a maximum 15 m (49 ft).

Figure 3.8 shows a three-dimensional view of two adjacent panels. After an individual panel has been lowered into place, it is suspended in the slurry from H-beams that are supported on the guide walls of the trench. The slurry is composed, in part, of cement so that the bentonite-cement mixture acts as a grout. Once hardened, the grout becomes an integral part of the structure because it seals the separations between adjoining panels. For this purpose, Soletanche uses a bentonite-cement grout whose setting time is regulated by special additives. Figure 3.9 shows an idealized plot of the grout strength as a function of time. Ideally, the grout is intended to remain fluid during excavation and panel placement,

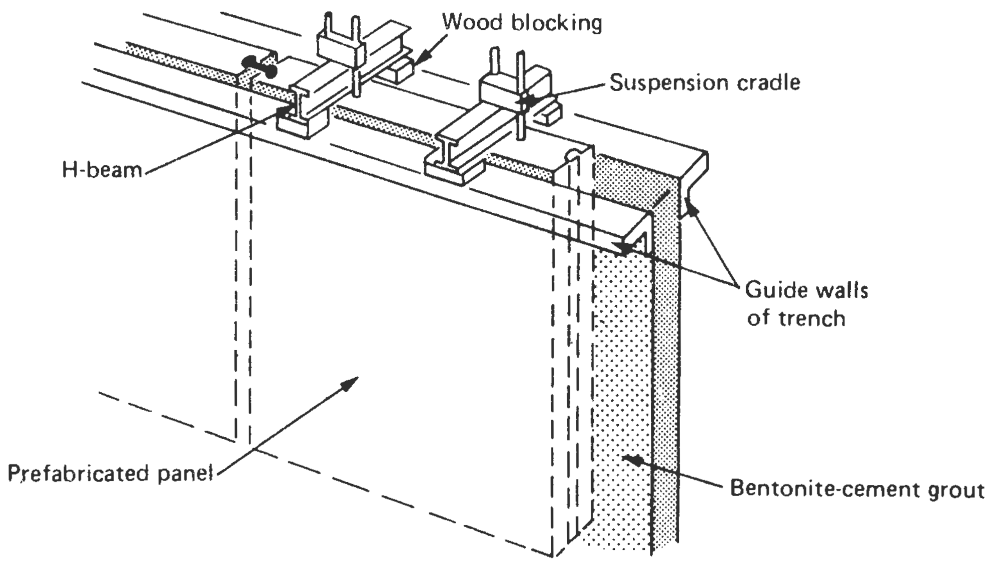


Figure 3.8 Three Dimensional View of a Prefabricated Concrete Wall

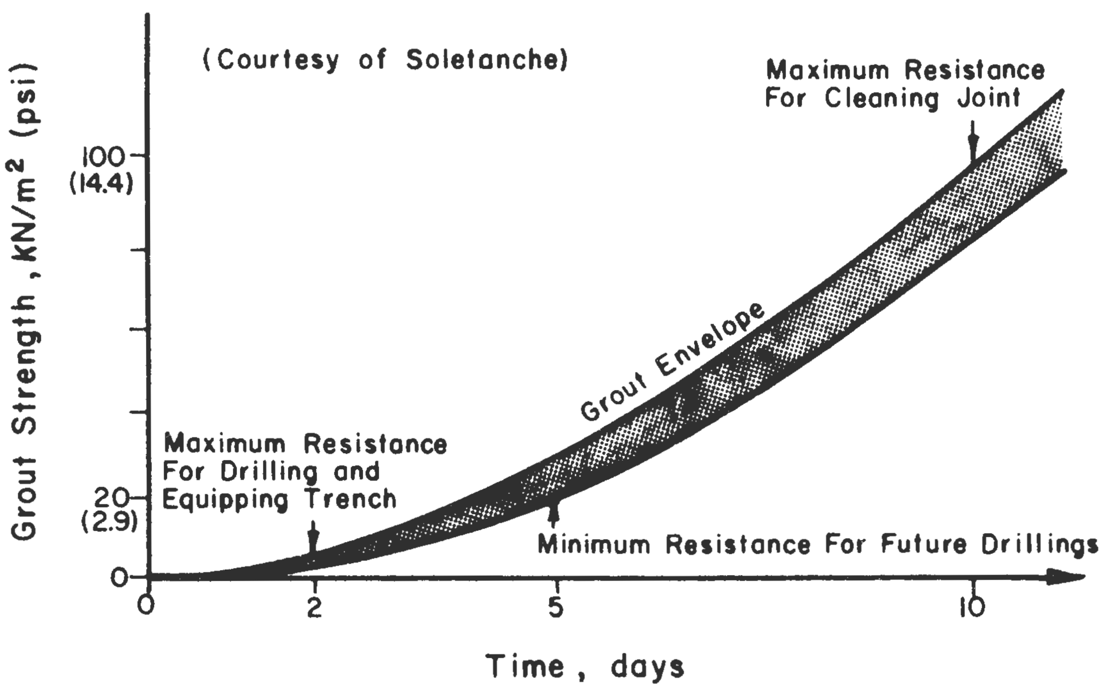


Figure 3.9 Grout Strength as a Function of Time

but attain sufficient strength in several days so that an adjacent trench can be excavated without grout loss from the previously established trench. As the setting and hardening characteristics of the grout may be affected by ground water chemistry, close supervision is required to insure that it does not become contaminated during trenching and panel placement. An alternate method of setting panels makes uses of a conventional bentonite slurry to stablize the trench during excavation. A bentonite-cement grout is introduced at the trench bottom while the precast panel is lowered into place (32). In this way, the panel displaces the bentonite slurry so that only the bentonite-cement mix remains.

In practice, the bentonite-cement grout is subject to shrinkage cracks. Consequently, some local dampness and seepage of water may occur at the joints of the final structure. In these cases, the joints are usually treated by raking the internal faces of the joints and filling them with a combination of epoxy sealant and cement.

The inside face of each panel is glazed with a special compound before placement to facilitate grout removal when the excavation for the final structure is carried to subgrade. In this way the internal face of the wall can be scaled with relative ease to render an architecturally finished surface.

To improve the water-tightness of the walls, the joints between precast panels may be fitted with flexible waterstops. Figure 3.10 shows a plan view of both a tongue-and-groove and waterstop joint. In the field the panels are prepared for waterstops by inserting an inflatable hose in a groove in the joint adjoining the next trench to be excavated. This is illustrated in Fig. 3.11. Before placing the adjoining panel, the hose

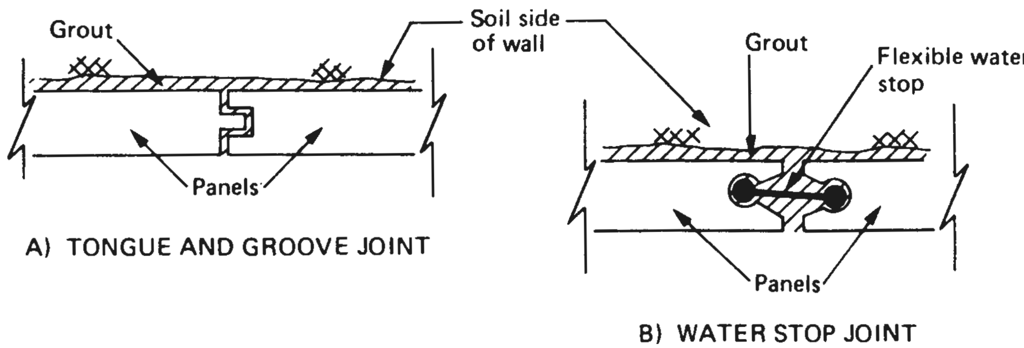


Figure 3.10 Joints of Prefabricated Walls

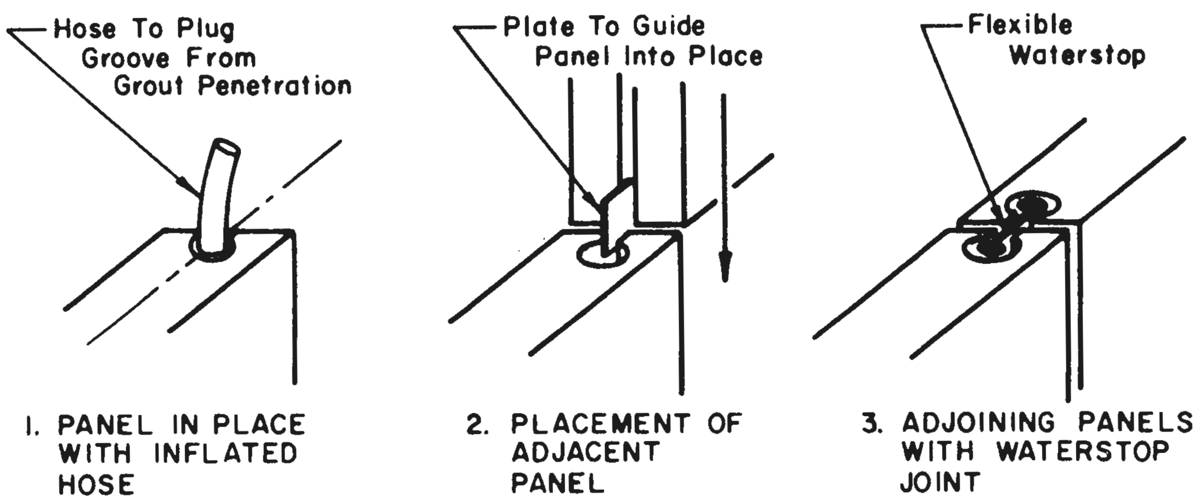


Figure 3.11 Sequence for Placing Precast Panel

is removed. The adjoining panel is lowered into place with the help of a blade-shaped plate that is attached to the bottom of the unit. The plate slides into the groove of the previously set panel, thus guiding the waterstop into position.

Lowering the panels requires care and skillful manipulation so that adjoining panels are placed within relatively small separations. Figure 3.12 shows the tolerances that were required for panel placement during construction of the Invalides-Orsay Connection for the French Railway in Paris. The tolerances, which were determined by the maximum allowable distortion of the waterstop, were set at a maximum 73 and 46 mm (2.9 and 1.8 in.) for transverse and horizontal separations, respectively.

Because the trenching proceeds in a continuous line, excavation must be stopped after placing an individual panel to allow for suitable hardening of the bentonite-cement grout. To decrease the time for construction, two or more sections of the job may be worked concurrently. As the grout is setting up in one section, trenching and panel placement can proceed in another. If two sections of the same wall are worked toward each other, the cumulative effects of lateral and vertical positioning may require a panel with special dimensions to close the final gap.

3.3.2 Recent Applications

In this section, the use of prefabricated concrete walls is described briefly with regard to two rapid transit projects under the following headings:

<u>Invalides-Orsay Connection</u>

The Invalides-Orsay Connection of the French Rail System is an important link in the development of regional transportation servicing

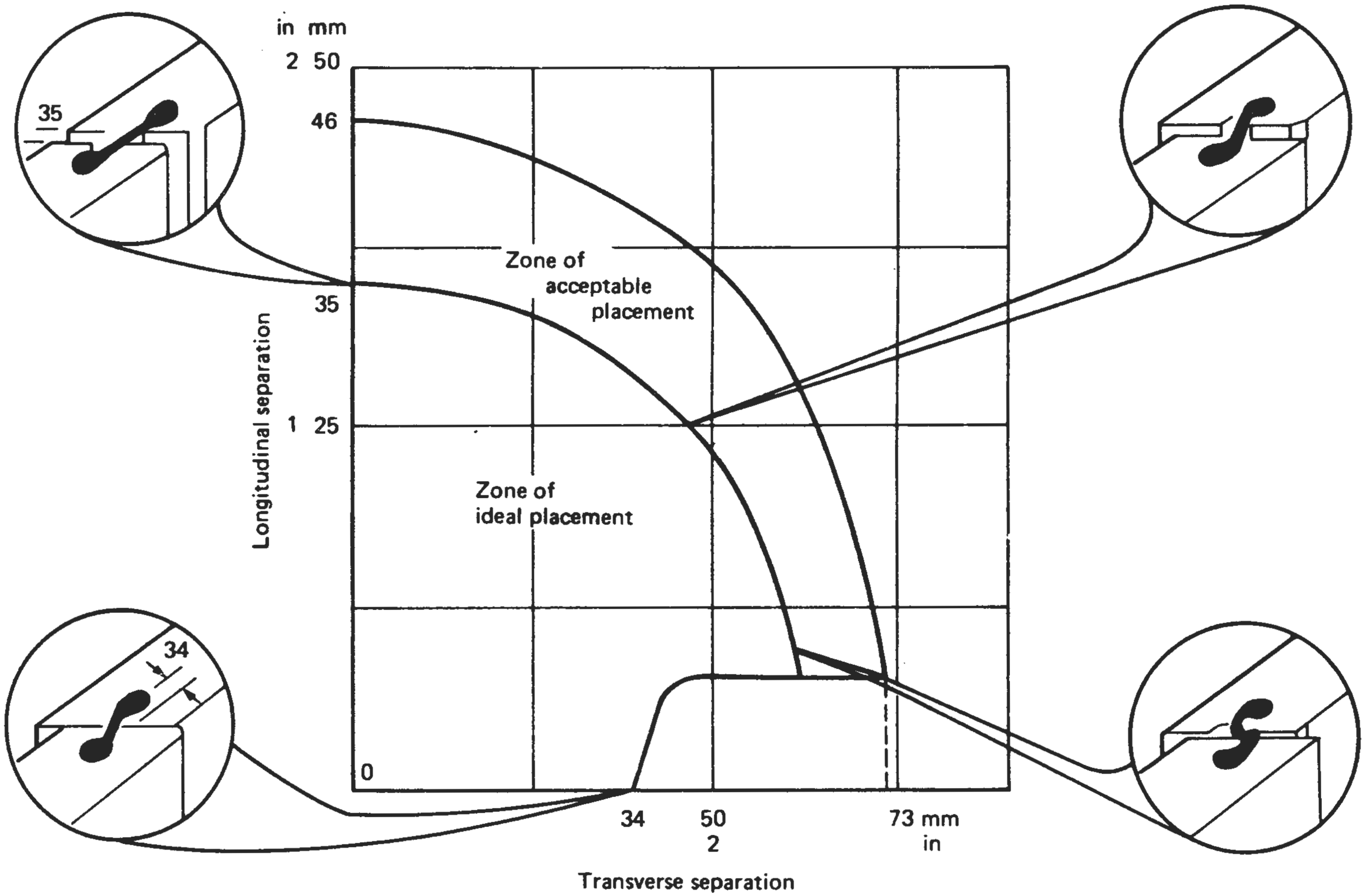


Figure 3.12 Tolerances for Separations between Precast Panels with Waterstop Joints

Paris and surrounding cities. The general layout and construction of the project have been described by Bertran (7) and Fournier (31).

The precast panels used on the project were 450 mm (1.5 ft.)-thick, 2.4 m (7.9 ft.) wide, with an average depth of approximately 13 m (43 ft.). The maximum weight of an individual panel was 30,000 kg (33 tons). Panels were delivered to site in the evening to cut down on trucking during the peak traffic hours. The trenches for the panels were excavated through a soil profile of approximately 5 m (16.4 ft.) of fill and 6 m (19.7 ft.) of sandy alluvium. A 1.0 m (3.3 ft.)-deep socket in limestone was provided at the base of each panel. The water level was approximately 2 m (6.6 ft.) below the ground surface.

The joints of the wall were sealed against water infiltration by using a flexible waterstop (See Fig. 3.12). Extreme care was taken during installation so that the offset between adjoining panels was within the tolerance set for the waterstop. The joints falling within the zones indicated in the figure were judged to be watertight and were not treated. Those falling outside the zones needed some treatment, generally in the form of raking the internal face of the joint, installing a vertical drain, and resealing with an epoxy filling. Joints that were particularly defective were treated by grout injection at the external face of the waterstop. Only 3% of the joints on the project required treatment.

Lyon Metro

Prefabricated walls were used on several sections of the Lyon Metro, notably along very narrow streets such as the Cours Vitton and Victor Hugo Street. As buried utilities were rerouted in 1 m-wide (3.3 ft) zones adjoining the metro box structure, it was imperative that the walls of the structure were kept minimally thick so that the subway line, parallel utilities, and access to buildings could be accommodated within the width of the streets. This was possible using precast walls because the quality of concrete and reinforcement associated with the units allowed for a wall thickness of approximately 300 mm (1.0 ft).

The soil profile in Lyon is discussed elsewhere in this report (See Appendix A). Essentially, it is made up of 2 to 5 m (6.6 to 16.4 ft) of granular fill, 10 to 20 m (33 to 66 ft) of very pervious sands and gravels, and an underlying deposit of molasse.[8] The water level was at a depth of approximately 5 m (16.4 ft).

The prefabricated panels varied in thickness, depending on their use at sections of running tunnel or near stations. Thickness ranged from 250 mm (0.8 ft) to 450 mm (1.5 ft). The walls near the Part-Dieu Station were 12 m (39.4 ft) deep. Tongue-and-groove joints were used. Adjoining panels were generally placed within 20 and 70 mm (0.8 and 2.8 in.) of each other, although a separation as high as 130 mm (5.1 in.) was reported. Where leakage was noted, the interior face of the joint was raked and treated with a polyurethane sealant.

The cost per square meter of wall was approximately 45% higher for the precast units than for cast-insitu walls. Even so, Ferrand and Boller (28) mention that precast walls were preferred to cast-insitu units because they could be installed more easily and because they minimized the cleaning down work necessary to prepare the wall surface.

---

[8] molasse refers to an angular, coarse to fine silty sand that was deposited as the result of rapid erosion from nearby mountainous areas.

3.3.3 Discussion

Prefabricated concrete walls share many characteristics with cast-insitu walls. Both are stiff, continuous units that can be used as the permanent walls of the underground structure. As the installation of precast panels requires the excavation of a slurry stabilized trench, the method is subject to the same environmental hazards as is the cast-insitu technique. The precast panels, however, can be built to higher strength. The concrete can be vibrated into place during fabrication, and there is no concern over the influence of bentonite slurry on the bonding between the reinforcing steel and concrete. Consequently, the panels can be built in relatively thin sections, which may be crucial for construction along narrow streets, as was the case for the Lyon Metro. Furthermore, the panels can be molded into various shapes so that recesses for mechanical or electrical appurtenances can be provided. Similar features on cast-insitu walls are difficult to construct because special forms must be attached to the reinforcing cage and subsequently removed from the wall during excavation. Tolerances associated with setting the reinforcing cage in the slurry trench and the possibility of dislodging the forms place limitations on the variety of wall cavities that can be developed. Precast panels can be provided with starter bars for an improved connection between the walls and base slab of the structure. Finally, the interior surfaces of the precast panels are generally of a high standard. After the bentonite-cement grout has been removed, the wall surface tends to have the appearance of quality cast concrete.

Prefabricated concrete walls cost more than cast-insitu units. On the basis of cost per square meter of wall surface, they are generally 20 to 30 % more expensive than their cast-insitu counterparts. A cost differential as high as 45 % has been reported for the Lyon Metro.

Although speed of construction has been quoted as an advantage of the method, this claim must be viewed in light of the special characteristics of each project. Transportation of the panels, careful installation to minimize the separations between adjoining panels, and setting time for the bentonite-cement grout will all contribute to the time required for wall construction. Bentonite-cement grout helps to establish an impervious barrier, but does not guarantee the absence of all seepage. Some joints are likely to require treatment by raking the internal face of the joint and applying a sealant. A small portion may require special grouting at the external face of the wall.

3.4 Secant Pile Walls

3.4.1 Review of the Method

A secant pile wall is formed by establishing a line of bored, insitu-cast piles that intersect each other to develop a continuous wall. Figure 3.13 shows a plan view of a typical secant pile construction. Female piles are bored such that the distance separating them is smaller than the diameter of an individual member. Male piles then are placed between them. For the female piles it is common to specify a compressive strength of 5 to 10 MPa (720 to 1440 psi) less than that for the male piles. In addition, male piles are generally bored at a time closely following installation of the female

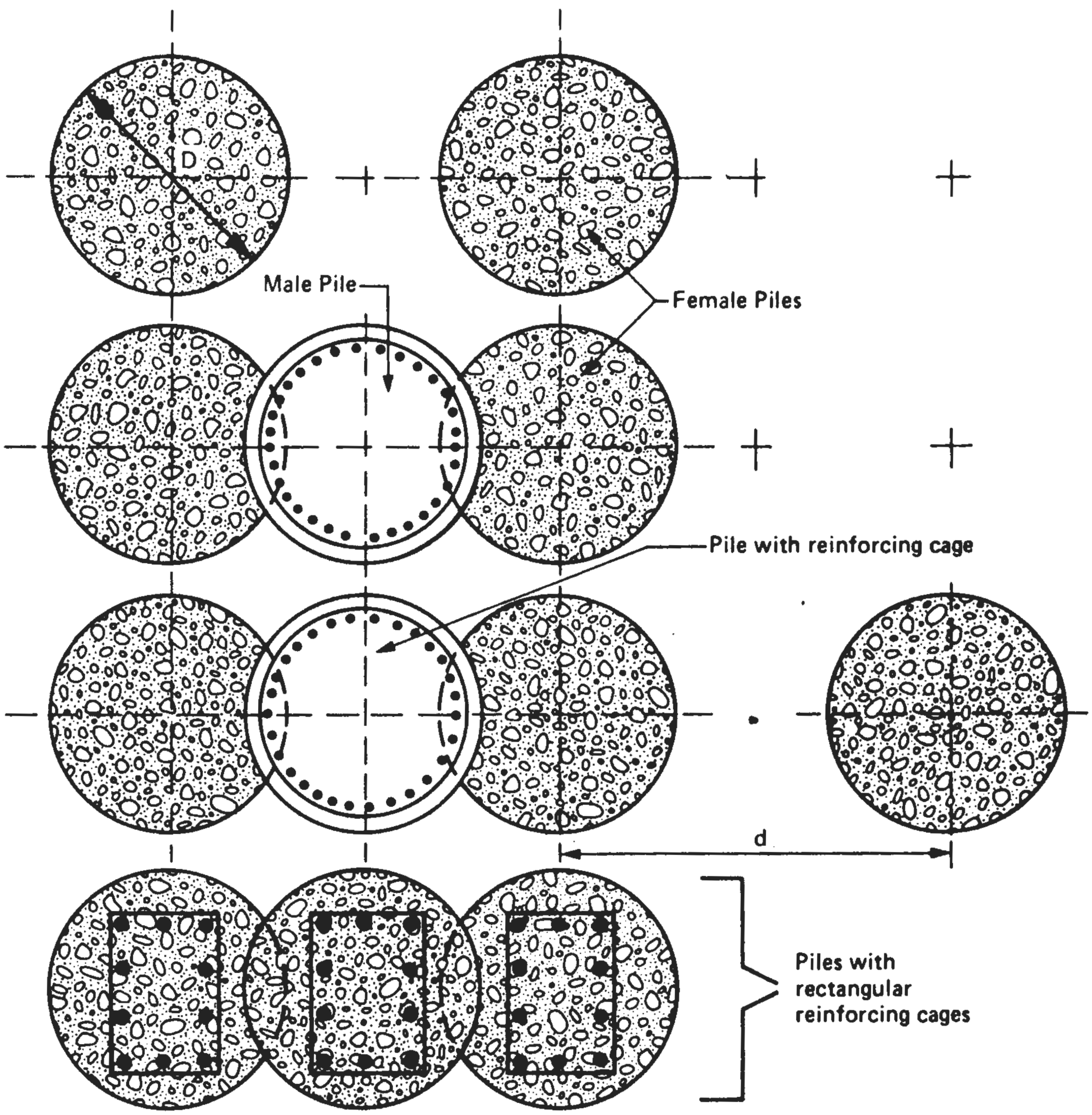


Figure 3.13 Plan View of Secant Pile Wall Construction

units when the concrete previously placed has set, but has attained only a portion of its ultimate strength. Retarding agents or low heat cement may be used in the concrete to slow down the hardening process. In the majority of secant pile walls only the male piles contain reinforcing steel, however a special rectangular cage can be used in the female piles so that all wall elements are reinforced. Where heavy loading or high shear resistance is required, steel H-beams can be placed within individual piles.

Secant piles often are constructed with the aid of a Benoto rig, an illustration and photo of which are shown in Figs. 3.14 and 3.15, respectively. The rig drives a special tube into the ground as soil is simultaneously excavated from inside by a mechanical grab. Hydraulic jacks act through a collar attached to the tube to rotate it back and forth as it is pushed into the ground. The tube is equipped with a serrated bottom that acts as a cutting edge under the semi-rotational movement. Additional sections of casing are coupled to the sections previously driven to make up the required depth of pile. The reinforcement cage, if required, is then placed and concrete is skipped or tremied into the pile as the sections of casing are withdrawn. A semi-rotational movement is imposed on the casing as it is withdrawn. In Fig. 3.16, which shows a portion of the secant pile wall in use for the Tyne and Wear Metro at Monument Station, the herring-bone pattern on the pile surfaces has been formed by the cutting edge of the bottom tube as it was pulled from the recently tremied piles. The semi-rotational movement helps to compact the concrete but, more importantly, prevents the concrete from adhering to the sides of the casing and, thus,

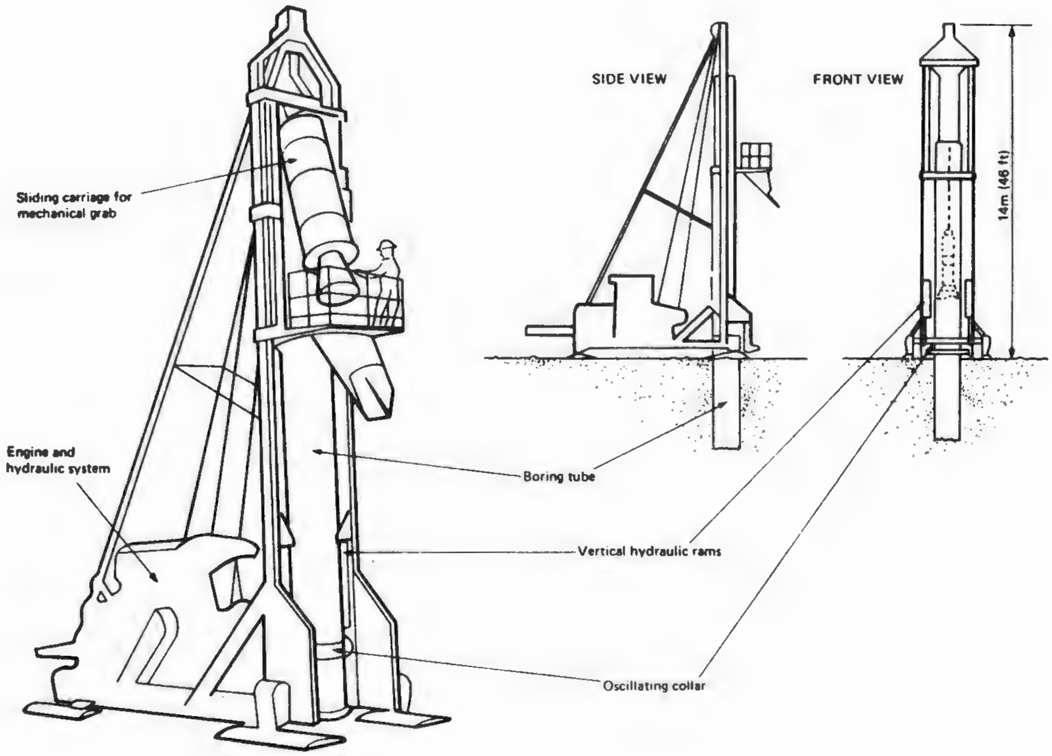


Figure 3.14 Three-Dimensional and Profile View of the Benoto Rig

Figure 3.15 Benoto Rig

Figure 3.16 Secant Pile Wall for Monument Station on the Tyne and Wear Metro.

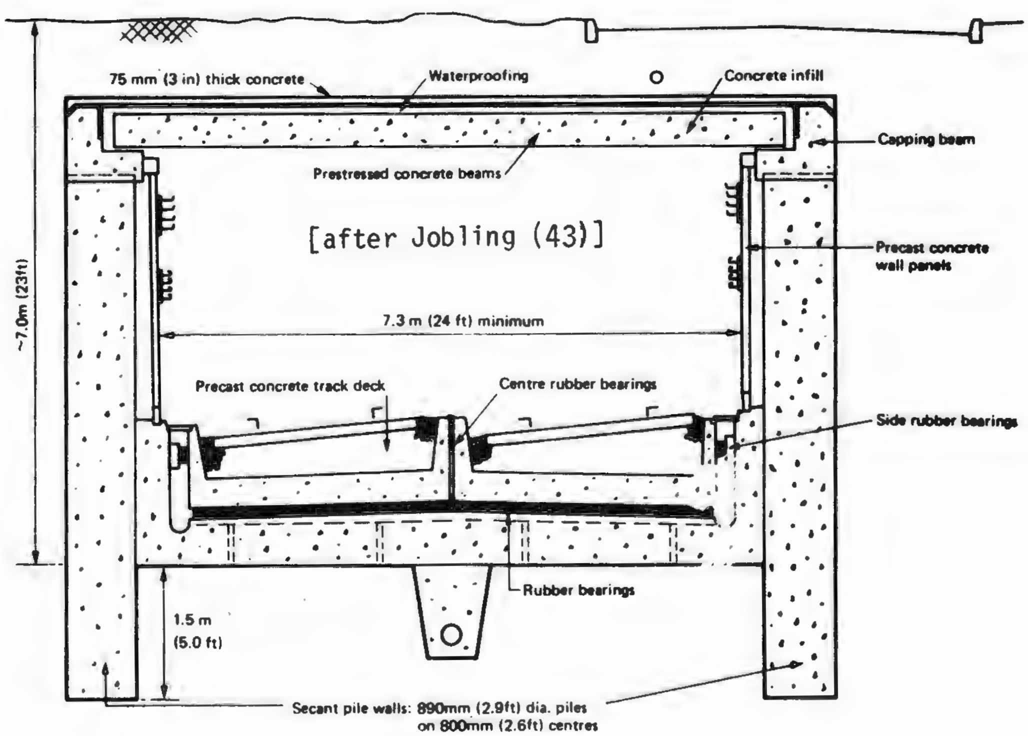


Figure 3.17 Cross-Section of Metro Structure with Secant Pile Walls

forming a void in the completed pile. Among the several projects under review for this study, no instance of voids within an individual pile was reported.

Installation of the piles is preceded by placing a guide wall along the intended pile line. The guide walls are frequently composed of 6 to 10 m - long (20 to 33 ft.) timbers, 305 mm (12 in.) square, that are layed and leveled on concrete beddings and backed with concrete to face the pile line. The location of each pile is determined by taping from the guide walls. At the start of driving, the bottom tube of the Benoto casing is positioned by plumbing the tube on center with a mark midway between the guide walls. Verticality is checked by spirit leveling at the top of the casing. Position and verticality are checked again when the casing is driven approximately 6 m (20 ft.).

Secant pile walls depend heavily on the relative location and vertical alignment of the members. If adjacent piles fail to intersect, a "window", or unsupported section of soil, is formed. For granular soils beneath the water table, windows may result in the erosion or heaving of soil into the excavation.

3.4.2 Recent Applications

Secant pile walls have been used for rapid transit construction in Britain, Belgium and Germany. Their use on several projects is summarized under the following headings.

London Underground Extension to Heathrow Airport

The extension of the London Underground to Heathrow Airport entailed the construction of approximately 1.83 km (1.14 mi.) of running tunnel and two stations, using secant pile walls as part of the cut-and-cover method of construction. For both the stations and running tunnel, the secant piles are used as permanent structural support. The depth of secant piles is between 8 and 19 m (26 and 62 ft.). The verticality of the piles was specified and controlled to within the limits of 1:200, or about 50 mm (2 in.) out of plumb over a 10 m (33 ft.) depth. The 880 mm-diameter (2.9 ft.) piles were set on 800 (2.6 ft.) centers.

A description of the overall construction and planning considerations on the project is given by Jobling and Lyons [41]. The soil profile consists of approximately 0.2 m (0.7 ft.) of topsoil overlying a 4.5 (14.8 ft.) m-thick stratum of coarse sand and gravel atop very stiff to hard London clay. The water level is approximately 2 m (6.6 ft.) below ground surface. Figure 3.17 shows a profile view of the completed box structure for the running tunnel. No windows, or voids, between adjacent piles were observed during construction. There were damp areas at some of the joints as well as local accumulations of moisture. The damp areas were treated by raking and caulking the joints with a waterproof cement.

During peak construction, five rigs were working simultaneously. Under good conditions five piles of roughly 8 to 10 m (26 to 33 ft.) depth could be installed by one rig during a working day of 11 hrs. to form approximately 180 m^2 (1937 $ft.^2$) of wall per week.

<u>Passenger Subways at Heathrow Central Station</u>

A system of passenger subways was constructed at Heathrow Central Station to provide causeways among the three flight terminals at the airport. On site, the soil profile is composed of approximately 8 m (26 ft.) of sand and gravel underlain by stiff to hard London clay. The water level is located 5 m (16.4 ft.) below the ground surface.

The depths of excavation were approximately 5 to 7 m (16.4 to 23 ft.), and the corresponding depths of secant piles were between 8 and 10 m (26 to 33 ft.). The 880 mm-diameter (2.9 ft.) piles were placed on 800 mm (2.6 ft.) centers. Plumbness of the piles was specified and controlled to 1:200. No windows between piles were observed during construction. Occasionally, delay in placement required that male piles be installed through the hardened concrete of adjacent piles. Driving under these conditions was completed within the desired accuracy of verticality. Secant piling was well suited for the congested areas of the airport. The relatively compact size of the rig allowed for pile installation at crowded work sites. Piling, roofing, and reinstatement could be performed in stages to avoid closing roads and interfering with airport business. In one area of work, the headroom available for installation of the piles was severely limited. A special rig was fabricated for this section by mounting elements of the Benoto equipment on the base of a standard hydraulic excavator. The rig was able to operate within a minimum headroom of 3.5 m (11.5 ft.).

Tyne and Wear Metro

Secant pile walls have been used on two portions of the Tyne and Wear Metro System at the Monument and St. James Stations. At Monument Station secant piles form the walls of a 20 m-deep (66 ft.) cut-and-cover excavation. The soil profile consists principally of stiff to hard boulder clay. Laminated clay with some soft clay and silty sand lenses are encountered to depths of approximately 8 m (26 ft.). Although the water level is below the excavation bottom for most of the construction area, lenses of silt and sand containing water were occasionally intercepted and had to be cut off by forcing the pile casing ahead of the excavation. The 1080 mm-diameter (3.5 ft.) piles were placed on an average 925 mm (3.0 ft.) spacing at depths of 20 to 30 m (66 to 98 ft.). Because of the large depths, all piles were reinforced. Rectangular reinforcing cages were placed in the female piles. The reinforcement was designed to provide access for both a tremie pipe and submersible pump.

Boulders were intercepted during the pile driving operation. They generally were composed of sandstone, and most were smaller in diameter than the casing. No serious problems arose as the boulders were broken up, mainly under the falling action of the mechanical grab.

The secant piles will be provided with an architectural finish to form the walls of the permanent structure. The verticality of the piles was specified as 1:200.

At St. James Station, 1180 mm-diameter (3.9 ft.) piles were installed on 975 to 1050 mm (3.2 to 3.45 ft.) centers. The soil profile is composed of 2.5 to 12 m (8.2 to 39 ft.) of boulder clay overlying Coal Measures strata of sandstone, shale, and coal. The piles were installed up to 26 m (85 ft.) into the Coal Measures rock. Due to the heavy loading at this section, all piles were reinforced with 914 by 305 mm (36 by 12 in.), steel wide-flange beams.

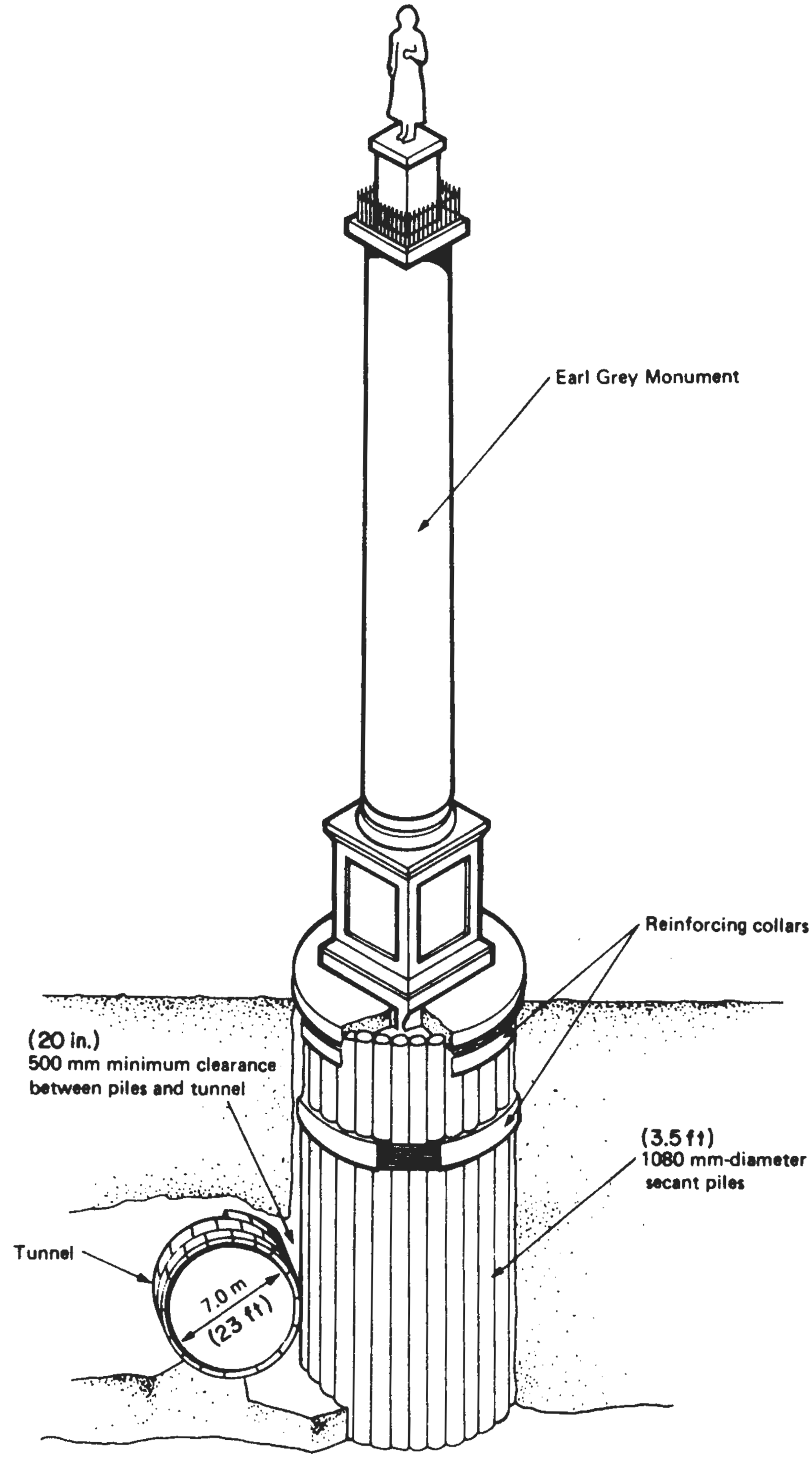
Secant piles were also used for underpinning purposes at Monument Station where the Grey Monument, weighing 940,000 kg (1034 tons), was supported against tunneling movements by a circular enclosure of 36 secant piles. As shown in Fig. 3.18, lateral restraint is provided by 3 prestressed, reinforcing collars.

Brussels Metro

Secant pile walls have been used along two sections of Line No. 1 of the Brussels Metro. At the Monnaie Station secant pile walls were established through a soil profile of 4 to 5 m (13 to 16 ft.) of fill, underlain by 5 m (16 ft.) of clayey alluvium and silt that, in turn, was underlain by sand. The station floor lies 13.5 m (44 ft.) below the ground surface. No information was given concerning the specific details of pile installation at this site. During excavation, windows were encountered in the secant pile wall, and soil was washed into the cut. The resulting ground loss caused damage to nearby buildings.

3.4.3 Discussion

Secant piles, when properly installed, form a rigid, continuous wall. The stiffness of a secant pile wall is equivalent to that of other concrete diaphragm walls, or approximately an order of magnitude larger than that of a soldier pile - lagging or sheet pile wall. Furthermore, the piles can be reinforced to support structural loads and used as the permanent walls of the system as was done for the London Underground and Tyne and Wear Metro.



(Courtesy of Lilly-Waddington, Ltd.)

Figure 3.18 Secant Pile Underpinning for the Grey Monument, Newcastle-Upon-Tyne

Secant piling requires a relatively small space and, thus, is useful in congested areas. With the Benoto rig, isolated segments of work can be performed and the construction areas reinstated with comparative ease. If there is no room on site for fabrication, reinforcing cages for individual piles can be easily transported. Headroom for the Benoto rig is approximately 14 m (46 ft.), however, modification or alternate equipment can be used for areas of restricted height. As was mentioned, a special rig with a minimum headroom of 3.5 m (11.5 ft.) was used for pile installation at Heathrow Airport. It should be noted that, because of their small size, the output per Benoto rig is limited to about half the capability of diaphragm walling equipment.

Benoto rigs can bore through obstructions such as dense rubble, abandoned pipelines, and foundations. The rigs are also capable of breaking up boulders or being used for socketing piles into bedrock. Penetration of obstructions, however, will increase the time for installation.

If the verticality and alignment of the piles is not maintained within strict tolerances, there may be serious problems with water tightness. As "windows" in the secant pile wall generally cannot be located until excavation has uncovered them, there is little advance warning or time for remedial measures when a piping or heaving condition occurs. In this regard, experience with secant piling in Brussels points out the difficulties that can develop with construction below the water table.

Secant piling is a specialty technique and there are relatively few firms that are experienced with the method. The competitive environment of underground construction places great emphasis on familiarity with the equipment and experience in different types of ground.

The depth of pile installation is limited by resistance to sinking the Benoto casing. Consequently, the presence of boulders or the frictional resistance generated along the casing at large depths may prevent or seriously retard progress. There are few guidelines for appraising the extent of this difficulty. Moderately sized, sandstone boulders were successfully penetrated during construction of the Tyne and Wear Metro. However, recent experience with building the Hong Kong Metro has pointed out some limitations. At Hong Kong secant pile walls were designed and installed for depths up to 36 m (118 ft) in a soil profile consisting principally of decomposed granite. Penetrating large segments of sound rock required special drilling equipment and caused delays. Any delay in driving time was compounded by later difficulty in boring the male piles. Low heat cement was used to slow down the hardening of concrete and helped minimize difficulties associated with placing the male piles. Experience on this project suggests that 25 to 30 m (82 to 98 ft) is likely to be the maximum economic depth for installing a secant pile wall under similar conditions.

4. SYSTEMS OF EXCAVATION AND SUPPORT

4.1 Introduction

This chapter examines two systems of excavation and support. They are slurry shield tunneling and the New Austrian Tunneling Method (NATM) as applied to soft ground conditions. On the one hand, slurry shield tunneling has been developed to cope with weak soil and potentially running ground where stand-up time of the material is small or negligible. The use of NATM, however, implies that the ground is self-supporting for, at least, as long as required to apply shotcrete support. In addition, NATM is based firmly on an observational approach, whereas slurry shield tunneling obstructs direct observation of the tunnel face.

Slurry shield design concepts and details of construction have been summarized by Bartlett et al (5), Walsh and Bartlett (91), and Jacob (40). In addition, there has been considerable development and practical experience with slurry shields in Japan, which are not covered in this report. The principles of NATM, as applied to soft ground tunneling have been discussed by Müller and Spaun (66, 67) and Laabmayr (50).

4.2 Slurry Shield Tunneling

4.2.1 Review of the Method

Slurry shield tunneling uses a pressurized, bentonite slurry at the face of the tunnel to provide support for the soil during excavation. The method requires a special tunneling machine, which has two essential features:

1) A sealed chamber at the tunnel face, called the plenum chamber, in which the bentonite slurry is maintained under pressure.

2) A circulating system for the bentonite slurry. This system, which acts as the mucking system, forms a closed loop. Slurry is pumped continuously to the tunnel face and removed to the ground surface where it is treated and subsequently recycled.

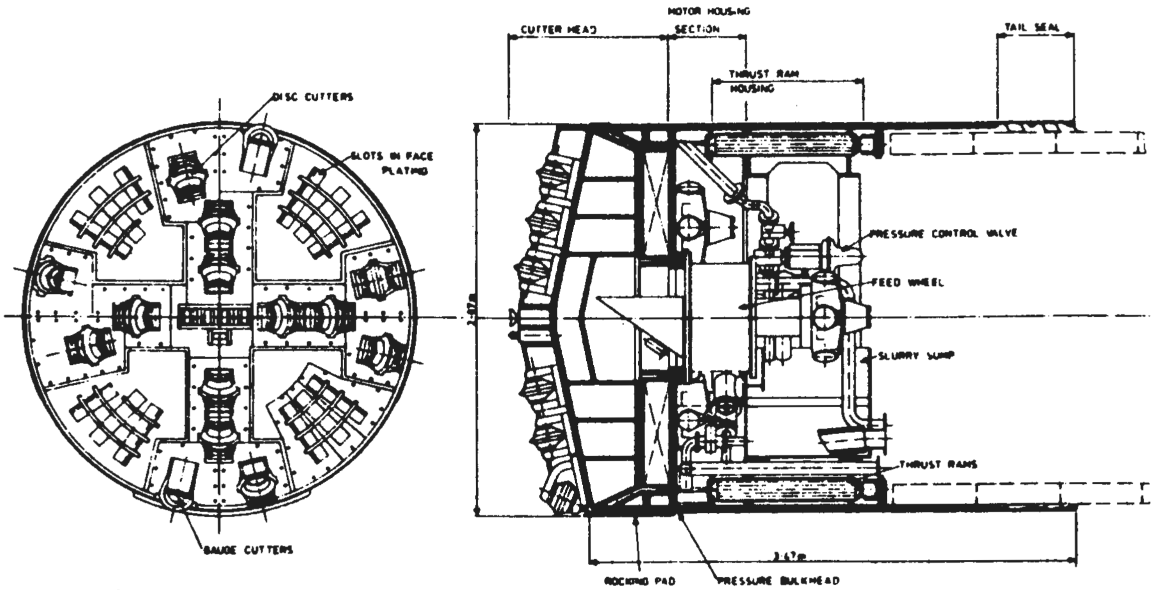
There are two types of slurry shield machine used in Europe. One has been developed under license to Edmund Nuttall, Civil Engineering Contractors in Britain and the other has been designed and marketed by Wayss and Freytag in the Federal Republic of Germany.

Figure 4.1 shows a transverse and longitudinal profile view of the British slurry shield. The system is based on the tunneling machine, manufactured by Robert L. Priestly Ltd., which has a cruciform arrangement of cutting tools. The machine shown in the figure was used in ground containing boulders. Consequently, the face is plated to help break up the rock. The machine can be fitted with both disc and pick cutters, although picks are standardly used for tunneling through a full face of soil.

Slurry is delivered to the plenum chamber near the invert by means of a constant volume pump. There are two outlets for the slurry:

1) at the center of the head through a hopper and feedwheel assembly and

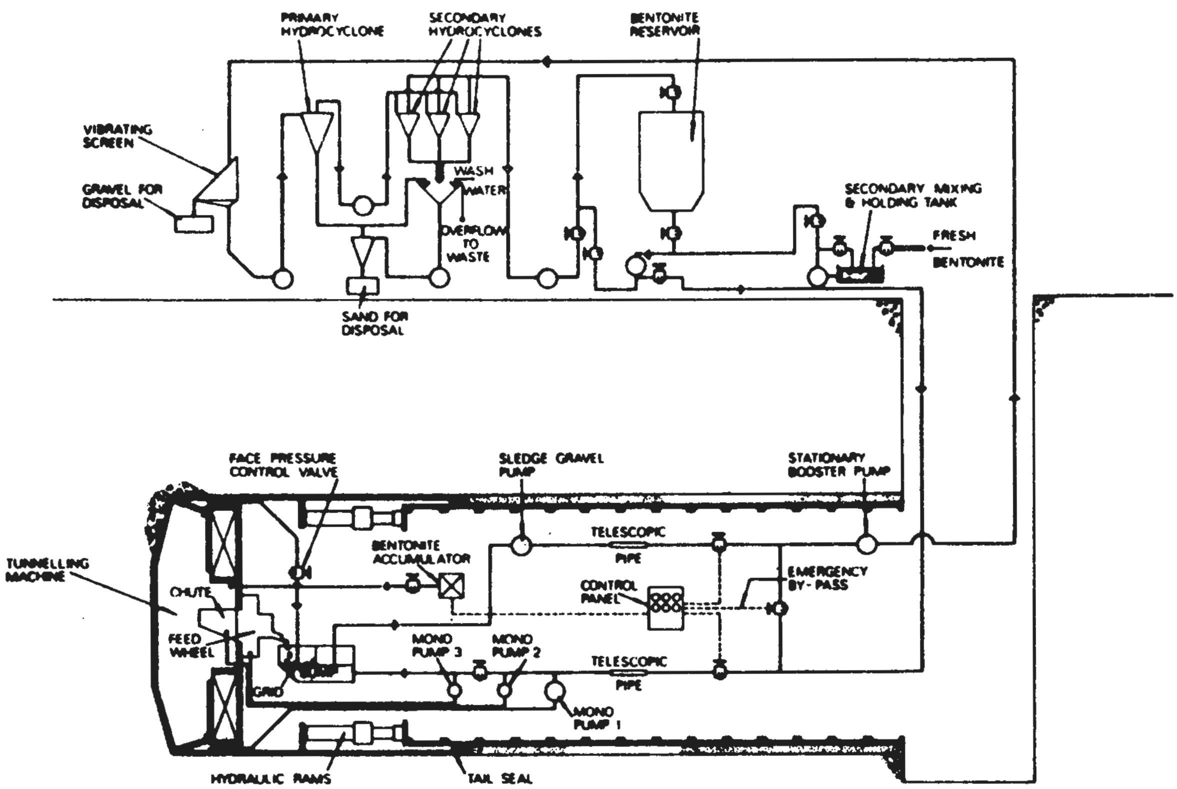
2) near the crown through an outlet pipe that connects with a pressure control valve. A pressure cell is attached to the bulkhead to monitor the pressure in the plenum chamber. Fluctuations in pressure generate a



a) Cutting Face b) Profile View

[after Walsh and Biggart (91)]

Figure 4.1 British Slurry Shield Tunneling Machine



[after Walsh and Biggart (91)]

Figure 4.2 Schematic Diagram of Slurry Circlation System signal for setting the pressure control valve. Consequently, pressure variations at the tunnel face are compensated by pressure adjustments in the slurry system. In addition, an air bleed is provided at the crown of the plenum chamber as a safeguard against air accumulation.

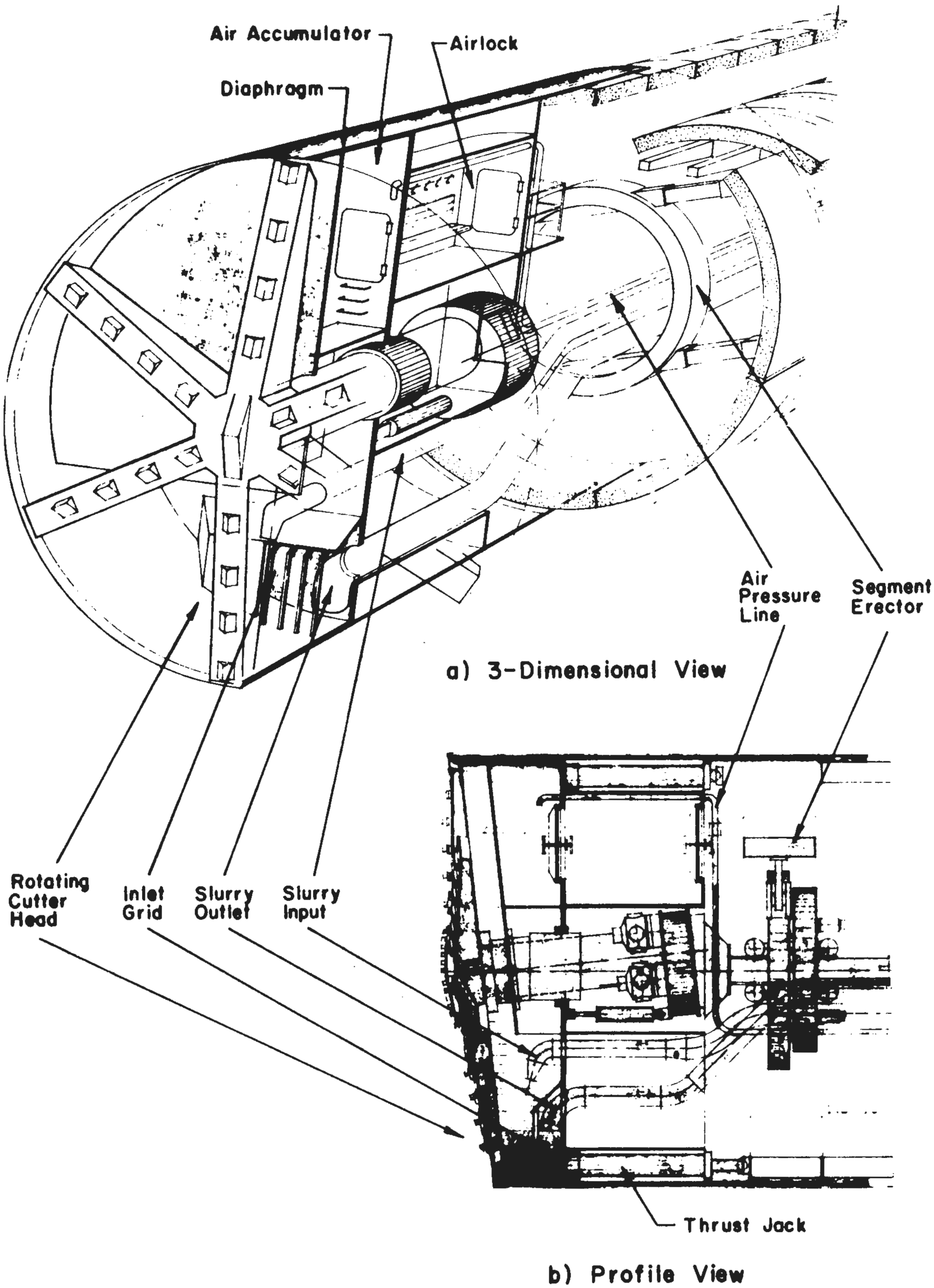
The feedwheel design is unique to the British machine and is composed essentially of a cylinder, with heilcal blades, that channels slurry from the tunnel face by acting much like an Archimedes screw. In addition, some slurry is delivered directly into the feedwheel compartments to act against the gulping of air. The feedwheel is capable of passing boulders up to 300 mm (1 ft) in diameter. The debris from the feedwheel is washed across a 100 mm (4 in.) grid to remove large particles before the slurry is collected at the sump and pumped out of the tunnel.

Figure 4.2 shows a schematic diagram of the slurry circulation system. Generally, the slurry is maintained at an approximate 6% proportion of bentonite. As the percentage of fines in the slurry increases, the fluid viscosity and tendency for silting work against efficient circulation. Consequently, not only must the larger particles of debris be removed from the slurry, but also a portion of the fines. On the ground surface, the slurry is passed through a series of filtering devices. To decrease its velocity, the slurry initially is directed into a boiling box from which it passes onto vibrating screens that remove all particles to 3 mm. The slurry then is channeled through two hydrocyclone units. The primary and secondary hydrocyclones filter the material down to 0.06 and 0.015 mm, respectively. Owing to the very fine grain size, the secondary hydrocyclones are self-cleaning to avoid clogging. Fresh bentonite slurry is added to the system as required from a special reservoir.

An important element in the slurry shield system is the tail seal between the tail skin of the shield and erected tunnel lining. The tail seal prevents slurry from seeping into the tunnel at the rear of the shield and, thus, performs two vital functions: 1) it contributes to a suitable working environment by stemming leakage, and 2) it provides a coupling against slurry inflow to minimize pressure losses in the plenum chamber. In practice, the tail seal is subject to severe wear by tearing and crushing against the tunnel lining as well as encasement by grout. The tail seal currently in use was radically redesigned from its original form during field tests to incorporate two pneumatic seals and new material. The details of the tail seal are of a proprietary nature.

Tunneling with the slurry shield machine is generally performed with an air lock installed in the tunnel as a back-up pressure system. This contingency allows for compressed air in the event of an emergency and also permits access to the face for observation or repairs.

Figure 4.3 shows a three-dimensional and profile view of the German slurry shield machine, which is referred to commerically as the Wayss and Freytag Hydroshield. The unique characteristic of the German machine is its pressure control system. Pressure for the slurry at the tunnel face is applied across a slurry-air interface in the air accumulator. A diaphragm separates the machine face from the air accumulator where slurry is maintained at a free liquid level above the shield axis. Correspondingly,



(Courtesy of Wayss and Freytag)

Figure 4.3 Three-Dimensional and Profile View of Wayss and Freytag Hydroshield compressed air in the air accumulator acts as a cushion to maintain a constant slurry pressure. Slight fluctuations in the volume of slurry have no appreciable effect on the slurry pressure because the air cushion will expand at constant pressure to compensate for volume loss. Consequently, the hydraulic pressure at the tunnel face is virtually independent of fluctuations in slurry quantity. In addition, the diaphram wall is constructed below the shaft of the cutter head to prevent air from migrating toward the face even when the liquid level is low.

The slurry input and outlet pipes are located near the invert. They control the flow of slurry to and from the plenum chamber. Bulky material is prevented from entering the outlet line by means of an inlet grid. If boulders or other large debris are intercepted, they accumulate at the invert level of the plenum chamber. When a quantity of material has collected that offers significant resistance to machine advance, the plenum chamber can be drained of slurry while maintaining a constant air pressure. Tunnelers then can enter the plenum chamber through the air lock to break up and remove the debris.

The cutting head of the machine is composed of a cutting wheel with six arms that support pick cutters. The shaft of the cutting head is inclined slightly towards the shield axis so that there are no large undercuts at the tunnel face even though the cutting head is conically shaped.

The slurry circulation and filtering equipment are similar to those on the British system. The tail seal consists of a pneumatic and plunger seal in tandem. The details of the seal, like those of its British counterpart, are of a proprietary nature.

4.2.2 Recent Applications

The use of the slurry shield method is described briefly with specific reference to applications of the British and German machines under the following headings:

New Cross and Warrington

The British slurry shield was originally field tested at New Cross during tunneling for a small portion of the London Underground. Observations, performance monitoring, and geotechnical measurements for this project are summarized elsewhere (8). The design of the machine was modified on the basis of its use at New Cross, particularly with respect to the feedwheel and tail seal. Subsequently, a new machine was manufactured and used for sewer tunneling at Warrington in north-west England. The first portion of this job has been reported on (91).

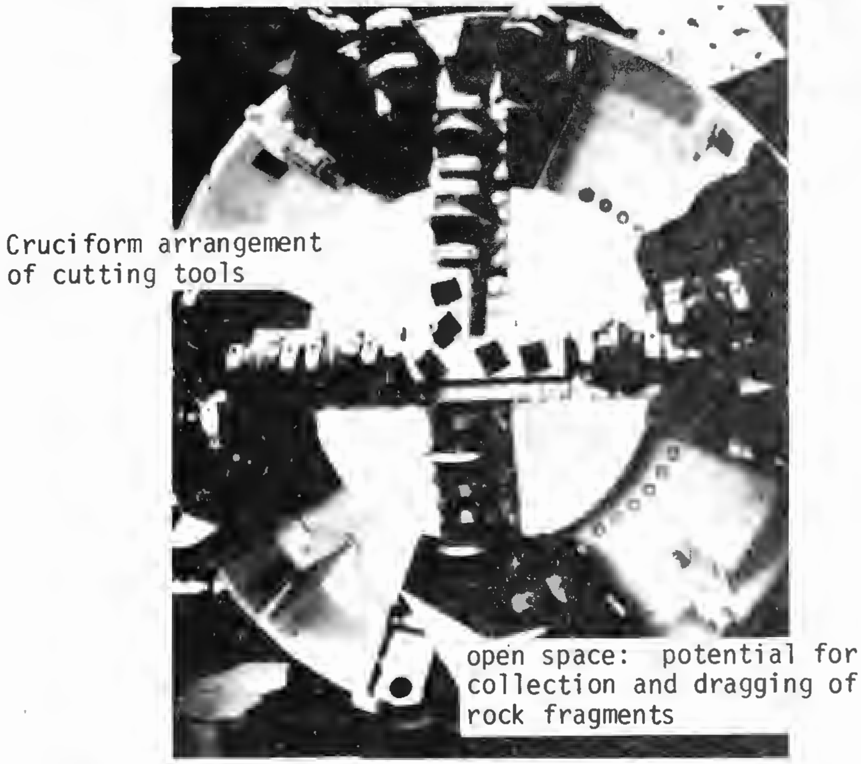
The tunnel at Warrington, which is 2.45 m (8 ft.) in diameter and 1370 m (4500 ft.) in length, was lined with bolted precast segments. The tunnel depth varied from approximately 5 to 7 m (16.4 to 23 ft.). Roughly 25 % of the tunnel length was driven in a conventional manner without pressurized slurry through a full face of sandstone and a mixed face of sandstone and sand. In the mixed face sections, the sand was treated with silicate grout.

Boulders were intercepted near bedrock and caused great difficulty with excavating and steering the machine. To facilitate the breaking up of boulders, the cutting head was fully plated (see Fig. 4.1a) leaving slotted areas [100 mm (3.9 in.)-wide slots] for muck intake. Figure 4.4 shows pictures of the cutting head before and after plating.

The remainder of the tunnel was advanced under a pressurized slurry with the plated head and disc cutters. A substantial portion of the remaining tunnel was driven in dense to medium sand with a water level, as indicated from the site investigation borings, at approximately the springline level of the tunnel. Centerline settlements along this section were less than 25 mm (1 in.). Unexpectedly, the tunnel intercepted very loose to loose cohesionless sands (standard penetration resistance between 2 and 9) that were part of the infill for an abandoned canal. In this material large losses of slurry occurred at the tunnel face and local boils of slurry were observed at the ground surface adjacent to tunneling. Even though the soil in advance of tunneling was treated with grout, substantial centerline settlements occurred. Brick-bearing wall dwellings, which flanked the line of the tunnel along this section, were damaged (69).

Hamburg Sewer Tunnel

The German slurry shield was originally used to drive 4.6 km (2.9 mi.) of 3.7 m (15 ft.)-diameter, sewer tunnel in Hamburg, West Germany. This job has been reported on elsewhere (40). Briefly, the tunnel was advanced at a depth of approximately 20 m (66 ft.) with the water level at 16 m (52.5 ft.) above the tunnel invert. An effective tail seal had not been developed at that time for the high water pressures on site [approximately 160 KN/m^2 (23.5 psi)]. Consequently, the tunnel was driven under compressed air even though the shield was used with pressurized, bentonite slurry. Loss of slurry was reported during tunneling through a back-filled



a) Slurry Shield Face before Installation at Site

b) Portion of the Face, Plated to Excavate Boulders

Figure 4.4 Alteration of the Slurry Shield Face for Tunneling in Boulders bomb crater. No specific measurements of surface settlement were given, although ground movements were reported to have been low.

4.2.3 Discussion

Often, the use of compressed air for tunneling is unattractive either because there is potential for excessive air loss or because there are health hazards and medical restrictions associated with the work. The slurry shield method is an alternative to compressed air tunneling. The capability of maintaining a constant, hydraulic pressure on the face, which is sealed from the rest of the tunnel, allows the work to be performed under atmospheric conditions. Furthermore, by substituting liquid for air, the likelihood of excessive fluid loss is diminished.

The attractive aspects of slurry shield tunneling are coupled, at the same time, with its most limiting characteristic. Tunneling is not so much a science that visual inspection of the face and textural judgements about ground conditions are not important, in many instances, for deciding on the best methods of excavation and support. The addition of a pressure bulkhead to the tunneling shield not only impedes access to the face but also changes the nature of the information on conditions at the face. With slurry shield tunneling, only indirect observations of ground conditions are available. These occur as pressure readings from the plenum chamber, shove resistance of the cutting head, and settlement measurements at the ground surface. In addition, the machine may be equipped with slurry monitoring devices to check the volume of outbound flow against the input flow and theoretical volume of excavation. These methods, however, are not likely to detect small volume losses that, nevertheless, can lead to significant settlement. The decision to use slurry shield tunneling implies that the loss of direct access to the face is more than offset by supporting the face with hydraulic pressure.

The slurry shield method is sensitive to variations in geology. This restriction places great emphasis on the quality of site investigation. For example, if intercepted with the German slurry shield, boulders or timber pile fragments would accumulate against the inlet grid until the shove resistance of the cutting head was high enough to warrant entering the plenum chamber under compressed air and removing the debris. Tunneling progress would depend closely on the density of obstructions, which often is difficult to predict from vertical borehole records.

Blow-outs of bentonite slurry are possible. This is illustrated by the tunneling at Warrington where slurry losses were associated with the interception of very loose to loose sands. When the pressure in the plenum chamber exceeds the confining pressure in the soil, slurry loss by hydraulic fracturing will occur. The highest pressures during slurry shield operation are likely to develop as the machine is thrust forward. Correspondingly this part of the tunneling cycle may require special attention from the machine operator.

4.3 The New Austrian Tunneling Method for Soft Ground Tunneling

4.3.1 Review of the Method

The new Austrian Tunneling Method (NATM) is based, in part, on the application of shotcrete as a temporary support for soil exposed during tunneling. In addition, the method coordinates observations of the ground conditions and construction performance to make continuous changes in the excavation and support procedures. Consequently, the method is developed around a firm reciprocity of observation and construction change. As applied to soft ground, NATM is a composite of four basic principles:

1. The use of shotcrete to support the ground. Generally, the shotcrete is applied in two stages. When the ground is first exposed, a 20 to 30 mm (0.8 to 1.2 in.)-thick layer of shotcrete is sprayed to provide immediate support. Around the crown and sides of the tunnel, this layer subsequently is enhanced with wire mesh, steel arches, and additional shotcrete to make up the primary tunnel lining.

2. Careful observation of the ground behavior. Observations may include optical leveling of surface markers, deep settlement measurements with extensometers, and convergence measurements of the lining.

3. Observant field personnel. This particular aspect of NATM has been heavily emphasized (68). Tunnellers, as well as engineers, are encouraged to pay attention to the nature of the exposed ground, remaining alert for raveling conditions and water inflow.

4. A flexible use of various support members and excavation increments. Depending on the ground conditions, increments of longitudinal excavation will be varied between 0.5 and 1.0 m (1.6 to 3.3 ft.). Likewise, excavation of the face can be performed with one or more benches. In addition, ground support will include shotcrete with variable quantities of steel arches and posts, wire mesh, and possibly, earth anchors or fully-grouted bolts.

Figure 4.5 shows a profile view of tunnelling using the principles of NATM. Excavation of the top heading is followed by shotcreting the exposed ground surface. This initial application of shotcrete includes spraying the tunnel face with a 20 to 30 mm (0.8 to 1.2 in.)-thick layer which is subsequently excavated as the tunnel is advanced. At the sides and crown of the tunnel, wire mesh is applied and a steel arch is erected. The arch is subsequently covered with shotcrete so that the total thickness of the primary lining is roughly 200 to 250 mm (7.9 to 9.8 in.). The bottom portion of the bench is removed after which the sides of the tunnel are shotcreted. Steel posts are usually installed at the sides. Most often, they are vertical, but may be inclined when close spacing and rapid excavation are required. Finally the invert is excavated and an invert slab is concreted to close the lining. Considerable emphasis is placed on closing the invert as rapidly as possible so that a continuous tube structure is provided to support the ground. A secondary concrete lining, generally 0.3 to 0.4 m (1 to 1.3 ft.) thick, is poured after the shotcrete and steel arch lining have been placed.

4.3.2 Recent Applications

The general procedures related to NATM with specific reference to metro constructions have been reported in the literature (50, 66, 67). In addition, measurements of ground movements caused by this type of tunneling have been summarized (66). As applied to urban tunneling, NATM has been confined principally to underground work in the Federal Republic of Germany (West Germany). However, NATM principles also have been adapted for metro construction in Marsielles, as described by Vincent (90). In this

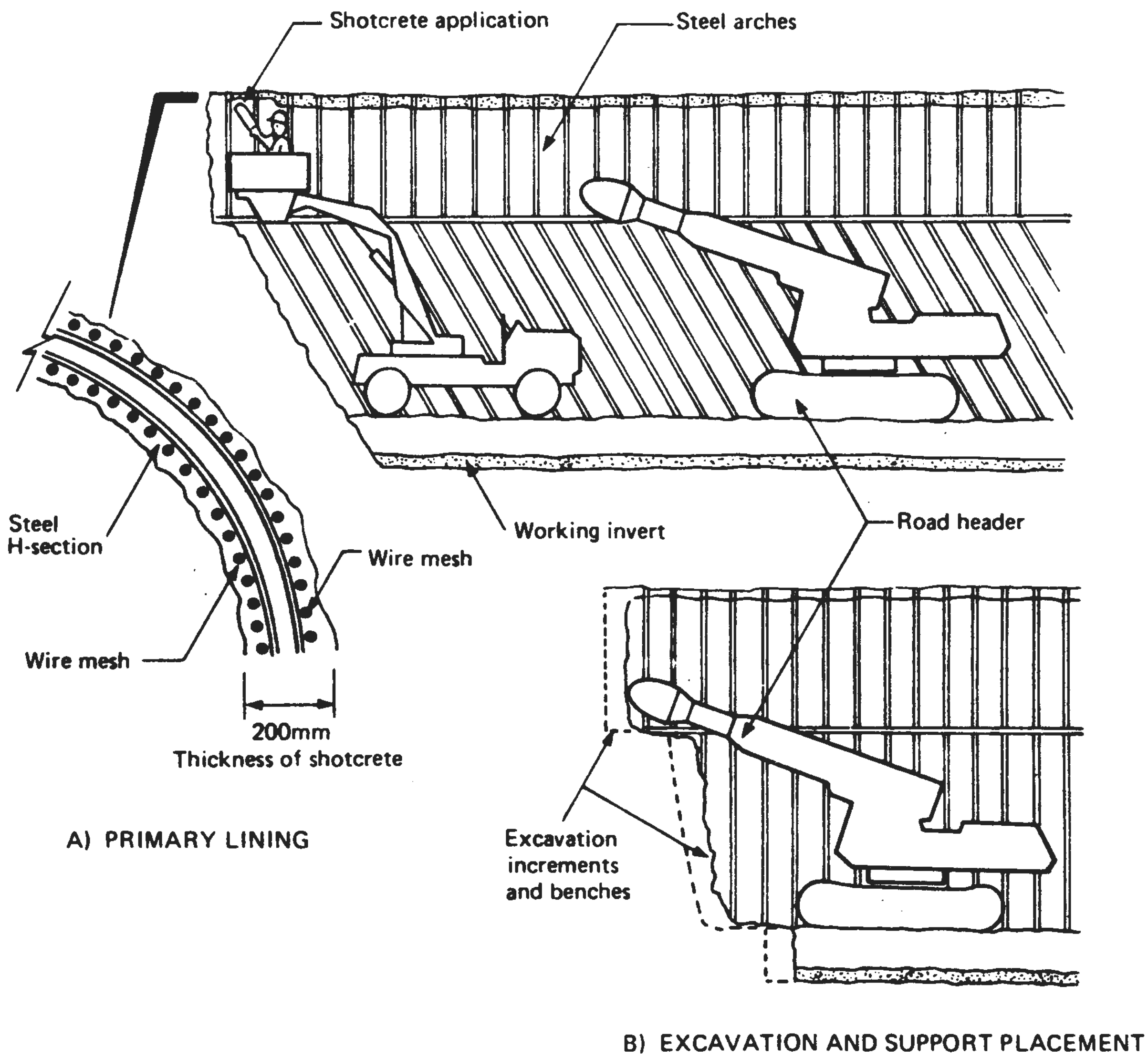


Figure 4.5 Excavation and Support with NATM

section, a brief summary of NATM application to metro construction in Bochum, West Germany is provided. The rapid transit work in Bochum forms part of a regional transportation project, covering several cities in the Ruhr Valley.

A profile and crossectional view of tunnel construction under the Castoper Strasse in Bochum is shown in Fig. 4.6. The soil profile consists of approximately 2 m (6.5 ft.) of fill underlain by a fluvial deposit of 6 to 8 m (20 to 26 ft.) of dense sand and interbedded stiff clay, all of which rests on a basal stratum of a glauconitic sandstone, known as greensand. The water level was generally below the invert. The sand possessed some cohesion, possibly as a result of clay binder or a slight calcitic cementation. The greensand had an undistrubed compressive strength of roughly 10 Pa (1440 psi). As the material was hygroscopic, however, it deteriorated when exposed until it attained the consistency of a weak, friable sandstone. Occasionally, there was seepage from the insitu fissures in the greensand. If water was intercepted, drillings would be performed in advance of the face to drain the water.

The upper 4 m (13 ft.) or so of the heading were in sand and clay. For purposes of stability two benches were established in this material. The uppermost portion of the excavation was semicircular with a maximum height of 1.8 m (6 ft.). Steel arches (120 mm-deep (5 in.) H sections) were placed in this region and steel posts were erected at the sides of the tunnel. The primary lining was 180 to 200 mm (7 to 7.9 in.) thick with double steel mesh. Twenty to 30 mm (0.8 to 1.2 in.) of shotcrete were applied to the exposed face to stabilize the soil while support was installed. Figure 4.7

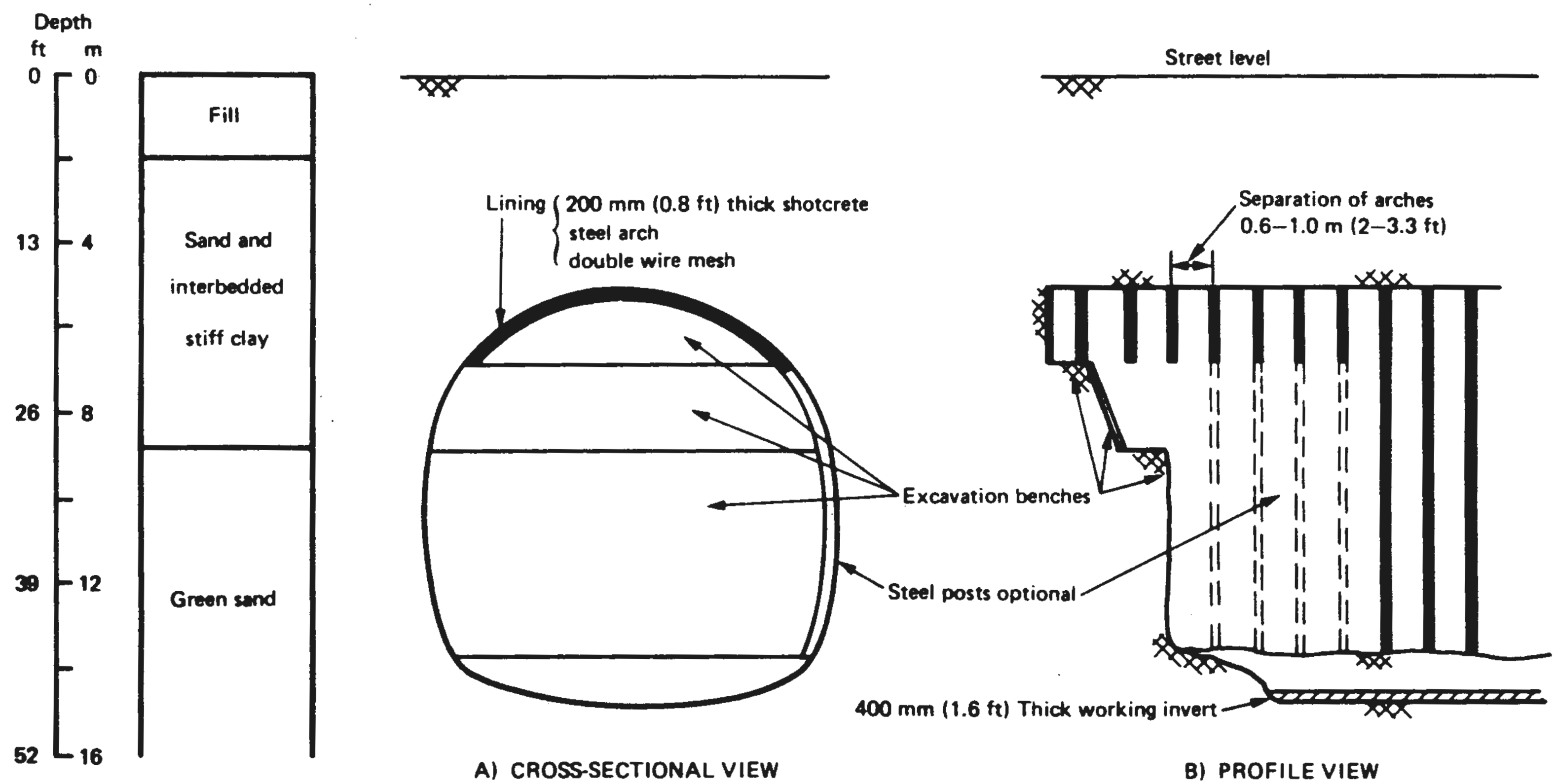


Figure 4.6 Transverse and Longitudinal Profiles of Tunneling for the Bochum Metro

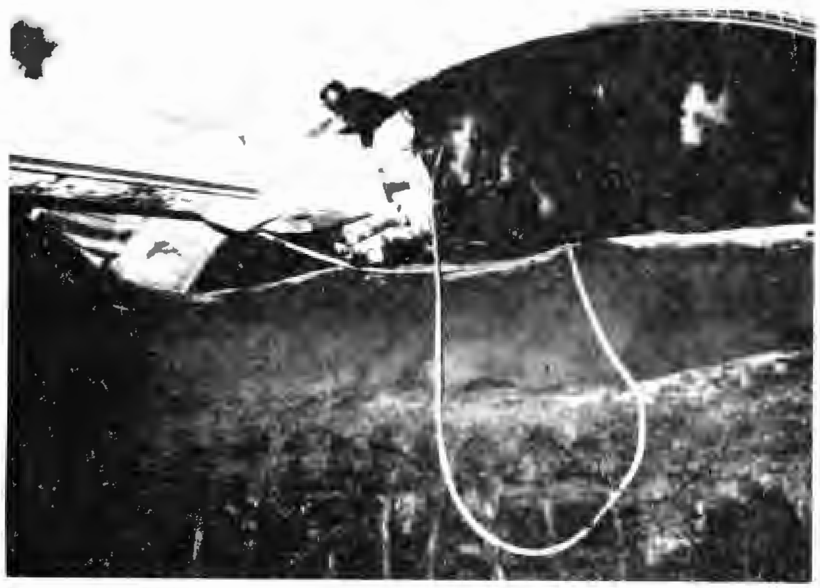


Figure 4.7 Upper Portion of the Heading during Tunneling for the Bochum Metro

Figure 4.8 Close-up of Crown and Side Wall during Tunneling for the Bochum Metro

shows the upper portion of excavation during the placement of wire mesh. Figure 4.8 shows a close-up view of the bench and sidewall area of the heading. At the bottom portion of the heading, vertical grooves from the point of a roadheader-type excavator can be seen in the greensand. After the base of the tunnel was excavated, a 400 mm-thick (15.8 in.) layer of concrete was placed as a working invert slab. It took approximately 30 hours between opening the upper heading and closing the invert. The secondary lining and final invert were 300 mm (12 in.) and 1.2 m (4 ft.) thick, respectively.

Transverse cross-sections of optical survey points were established on the road at 10 m (33 ft.) centers along the length of the tunnel. Settlement measurements in the vicinity of tunneling were taken twice daily. Although there were only 5 m (16.4 ft.) of cover between the crown and road surface, the maximum surface settlements were between 15 and 20 mm (0.6 and 0.8 in.).

At the beginning of tunneling, steel arches were spaced on 0.6 to 0.8 m (2.0 to 2.6 ft.) centers. As the tunnelers gained in experience and measurements showed a satisfactory level of settlement above the tunnel, the spacing was increased to 1 m (3.3 ft.). The steel posts were regarded as optional and were used only in the initial portion of the drive. With the continuation of favourable ground performance, they were discontinued.

Advance rates under the Castoper Strasse, where the cross-sectional area of excavation was approximately 80 m^2 (861 $ft.^2$), were between 1.5 and 1.75 m/day (5 and 6 ft./day). This compares with average advance rates of 3 m/day (10 ft./day) for single track, running tunnels near the Planetarium Station [cross-sectional area of approximately 30 m^2 (323 $ft.^2$)].

4.3.3 Discussion

NATM has been described (51) as the combination of effective observation with a support system that can be strengthened in stages. The use of field measurements and observations to make adjustments in the excavation and support technique is, in essence, the observational approach to civil construction that has been advocated and used for many years (73). What is perhaps unique about NATM is the way the observational approach is incorporated as a part of the contractual practice. Payment for tunneling usually is made on the basis of the excavation and support techniques used for the ground encountered. That is tantamount to a cost plus contract. Final judgement pertaining to the nature of the ground behavior and required support is made by the resident engineer. Because the chief engineer for the Bochum Metro is employed by the municipality, he is the direct representative of the owner. Consequently, observations are not only a feedback mechanism for controling ground movements and stability, but also are an expedient for executing the contract.

Owing to the flexibility associated with shotcrete support, a variety of cross sections can be handled within a single job. This is especially useful for station construction where converging sections and complex geometries caused by station junctions with portions of the tube system can be accommodated with relative ease.

Where ground conditions permit, large, double-track sections can be opened. These have ranged up to approximately 11 m (36 ft) in diameter. The working space provided leaves adequate room for excavation, support, and mucking operations. In addition, excavation with road headers in mixed face conditions of soil and low-strength rock allows for progress where shields would have difficulty.

Most soft ground materials in which NATM has been applied have had some inherent strength and capability of self-support. For example, tunneling in Frankfurt and Nuremburg has been performed principally in stiff to hard clay and clay-cement sandstone, respectively (66). In Marsielles, NATM principles have been used for tunneling in marl with a compressive strength ranging from 2 to 5 Pa (289 to 722 psi). In Bochum some of the more difficult cross-sections involved an upper heading of slightly cohesive sands and interbedded clay seams. Here, the water level was below the elevation of the sand and clay.

Water-bearing or cohesionless sands and gravels can lead to rapid raveling or flowing ground conditions for which the application of shotcrete will be inadequate. This represents a potentially hazardous situation and a major limitation of the method. In addition, the difficulties associated with cohesionless or waterbearing, granular soil places great emphasis on site investigation so that ground behavior will be adequately characterized before excavation.

At Bochum, the average advance rates for tunneling ranged between 1.5 and 3.0 m/day (4 and 10 ft./day). Among several different metros, the maximum progress rates have been quoted as varying between 3 and 4.5 m/day (10 and 15 ft./day), depending on the cross-sectional area of the tunnel drive (66). This includes only excavation and placement of the primary lining, which then must be followed by a secondary concrete lining. It has been pointed out that these advance rates tend to be low when compared with progress rates for shield driven tunnels in similar ground (53, 72). Furthermore, shield excavation can often be performed concurrently with the erection of the permanent tunnel lining.

5. CONSTRUCTION COSTS

5.1 Introduction

The comparison of underground construction costs among different rapid transit systems is not a straight forward procedure. The factors that influence cost are highly variable and include routing of lines, passenger capacity, siting of stations, soil or rock type, ground conditions, water levels, proximity to buildings, density of existing underground services, architectural design, and institutional constraints to name a few. As Girnau has pointed out (31), costs within the same city can vary from line to line and even from contract to contract. Furthermore, the cost of an underground project will stem directly from the objectives fixed for its eventual operation. In many instances, these objectives are coordinated either within the regional organization of transportation facilities or a pattern of urban renewal. This latter aspect has been emphasized by Lupiac (56) in his discussion of expenses related to underground constructions in Paris.

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

In this section the construction costs associated with six European rapid transit systems are examined. The transit systems include the London Underground extension to Heathrow Airport, the Tyne and Wear Metro System, the Lyon Metro, the Brussels Metro, Line 3 of the Stockholm Underground and the Helsinki Metro. All the constructions were performed at approximately the same time with major portions of work undertaken between 1972 and 1977. None of the lines contain exceptionally large or complex units, such as would characterize the Gare de Lyon and Chatelet-Les Halles structures of the Paris Metro (54, 75). The unit costs, therefore, are not influenced by anomalously large projects. The ground conditions and complexity of construction however, do vary widely among the lines discussed. Each underground system is described in Appendix A with specific reference to passenger capacity, dimensions, structural materials, geology and soil profile, construction methods, and breakdown of costs.

For this report construction cost is defined as the expense associated with the excavation and subsequent placement of the structure in the ground. As such, it includes the diversion of public facilities, which is a prelude to, at least, a portion of most underground construction in urban areas. Work of this nature would be related to rerouting and resurfacing roads as well as diverting and reinstating buried pipelines and cables. Construction cost also includes ground water control, excavation, temporary support, building the final structure, and site supervision associated with the work. It does not include underpinning adjacent buildings, electrification, signaling, track, ventilation equipment (although ventilation shafts are included), rolling stock, or land acquisition.

The costs summarized herein reflect price levels for 1975-1976, which have been established as the base comparison years for this report. The single exception is the cost associated with the London Underground extension to Heathrow Airport, which is a composite of 1975 and 1977 prices (see Appendix A). Where cost information from local transportation authorities was referenced to a year previous to the base years, this cost was adjsted to a 1976 level by using inflation indices provided by either the British Department of Environment or the Organization for Economic Cooperation and Development. Adjustments of this type were made for the Lyon Metro and the North-South line of the Tyne and Wear Metro. All national currencies with which the costs were originally quoted have been converted to U. S. dollars as per the exchange rates quoted in Appendix A. These exchange rates reflect the rates prevalent during the base comparison years.

5.2 Summary of Construction Costs

Figure 5.1 shows a bar graph of unit construction costs associated with several European Metro systems. The costs related to construction in soil and rock have been separated. All unit costs are expressed in terms of U. S. dollars per double track mile or ft. The costs have been determined by dividing the total construction cost for a given distance of line by the length of line under study. Hence, the unit costs include the expenses of building both stations and running tunnel. All costs have been abstracted from portions of rapid transit systems that have been constructed almost entirely underground. Sections of surface line, which may be included in the unit costs, are so small relative to the subsurface work that they have very little influence on the prices quoted.

Table 5.1 summarizes basic information regarding the sections of rapid transportation systems referred to in Fig. 5.1. The table contains information on the average depth, dimensions of both running tunnel and stations, distance of line per station, and approximate geology. The distance

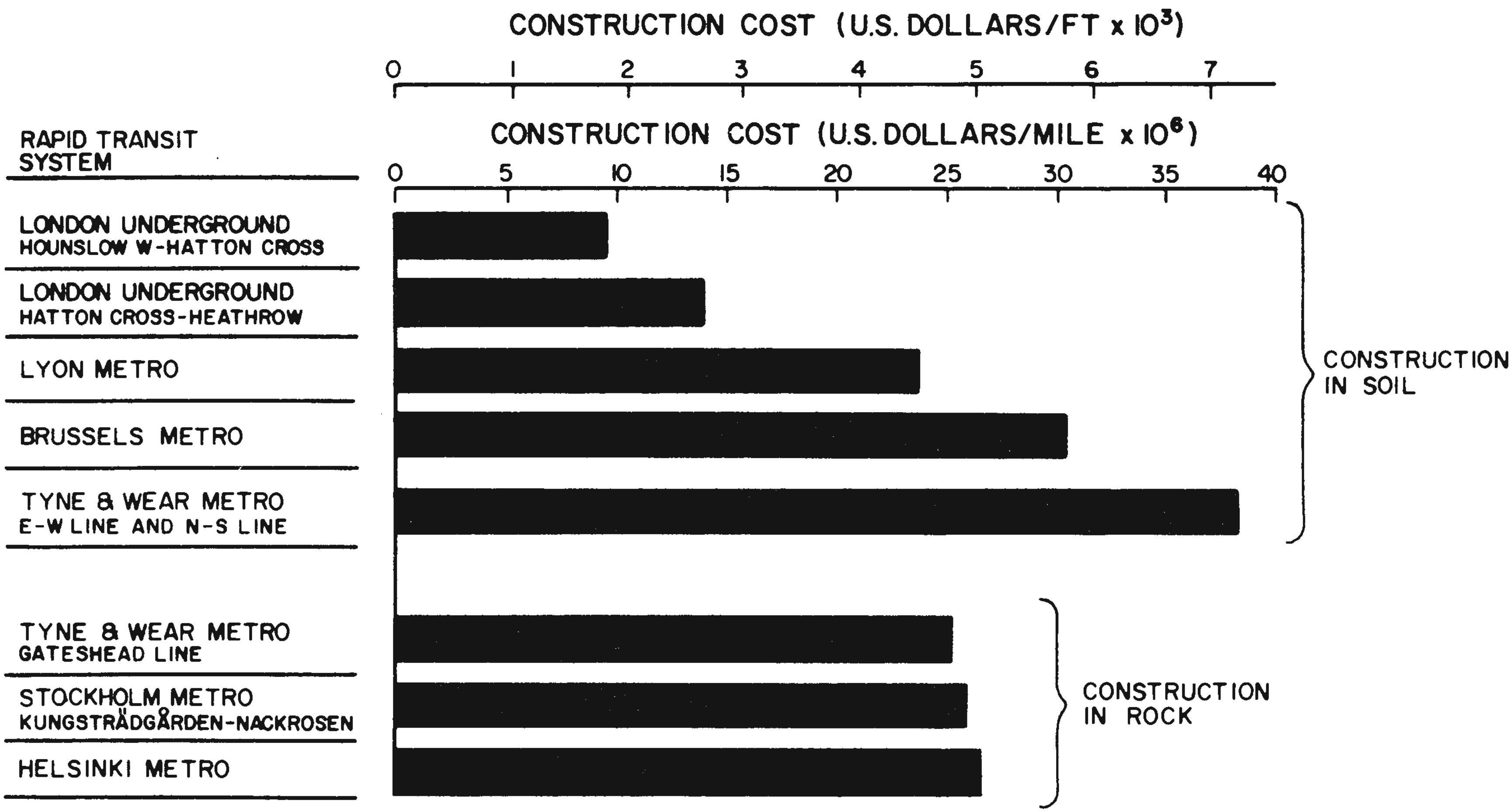


Figure 5.1 Underground Construction Costs for Several European Metros

RAPID TRANSIT SYSTEM	AVRG. DEPTH (FT)	TUNNEL CROSS SECTION (FT)	STATION DIMENSIONS (FT)	DIST OF LINE PER STATION (Mi)	APPROX. GEOLOGY
LONDON UNDERGROUND HOUNSLOW W-HATTON CROSS	23	13 x 24	PLAN: 400 x 79	1.05	SAND BELOW WATER LEVEL; HARD CLAY
LONDON UNDERGROUND HATTON CROSS-HEATHROW	56	12.5 I.D.		1.30	
NEWCASTLE METRO N-S LINE AND E-W LINE	59 AND 46	15.6 I.D.	23 I.D. PLATFORM LENGTH = 312	0.42	BOULDER CLAY
LYON METRO	25	13 x 24.6	PLATFORM LENGTH = 232	0.47	SAND BELOW WATER LEVEL
BRUSSELS METRO	25–60	15 x 24.6	PLATFORM LENGTH = 312	0.48	SAND BELOW WATER LEVEL
NEWCASTLE METRO GATESHEAD LINE	45	TWIN D-SHAPE 19 x 18	PLAN: 328 x 82	0.54	LIMESTONE, SANDSTONE
STOCKHOLM METRO KUNGSTRÅDGÅRDEN-NACKROSEN	100	D-SHAPE 15 x 26.5	PLATFORM LENGTH = 590	0.61	GRANITE, GNEISS
HELSINKI METRO	89	TWIN D-SHAPE 17 x 18	PLAN: 443 x 59	0.50	GRANITE, GNEISS

Table 5.1 Summary of Information for Several European Metros

of line per station refers to the total lnegth of metro line for which each cost applies divided by the number of stations included in the cost. This provides an index of station density and allows the unit costs to be judged in light of the relative number of stations covered by the costs. It must be emphasized that the table is intended primarily as a convenience so that rough comparisons among the systems can be made. Information of a more detailed nature, especially with regard to geology and construction method, is provided in Appendix A.

The unit costs of construction in soil vary through a wide range. For example, there is a substantial difference in cost between twin tunnel sections of the London Underground and the Tyne and Wear Metro. The main reason for the large difference is that the number of stations included in the unit cost for the Tyne and Wear Metro is nearly three times larger than the corresponding number for the London Underground. As the stations are major sources of expense, the unit construction costs of the metros are very sensitive to station density. With the exception of the London Underground, the distance of line per station for the reported constructions in soil and rock fall within very narrow bounds.

An additional reason for the difference in cost between the twin tunnel sections of the London Underground and the Tyne and Wear Metro is the difference in tunnel diameter and type of lining used for the systems. The tunnels for the London Underground are 3.8 m (12.5 ft) internal diameter and were constructed with expanded concrete rings. The tunnels for the Tyne and Wear Metro are 4.8 m (15.6 ft) internal diameter of which approximately 60 % were constructed with bolted iron segments and 40 % were constructed with bolted concrete segments. In addition, approximately 60 % of the tunnels were driven under compressed air for the Tyne and Wear system.

5.3 Components of Construction Cost

Figure 5.2 provides a bar graph showing the percentage of the total construction cost that was taken up by station construction for several metro lines. For the London Underground extension to Heathrow Airport, 40 % of the construction cost can be attributed to building the stations. This is consistent with previous cost experience during tunneling for the Victoria Line of the London Underground (29). The percentage also compares well with a similar analysis of both the Lyon and the Tyne and Wear Metros.

On the Scandanavian metros, station construction accounts for an extremely large proportion of the construction cost. The main reason for the apparent high price of station construction is the relatively low cost of building the running tunnels. Rock excavation, rock bolt support, and grouting varied in expense between \$730-\$880/ft and \$590/ft for double track tunnel route on the Stockholm and Helsinki systems, respectively. Although there are notable exceptions, transit tunnels in Stockholm and Helsinki were often advanced with drill and blast rounds of 2.5 m (8.2 ft) for single track tunnels, approximately 25 m^2 (269 ft^2) in cross-sectional area, and permanently supported by a 30 mm (1.2 in.)-thick shotcrete arch in the crown with rock bolts as required. In contrast, construction cost of the double track running tunnel on the Gateshead Line of the Tyne and Wear Metro was approximately \$3100/ft. The tunnel was advanced in

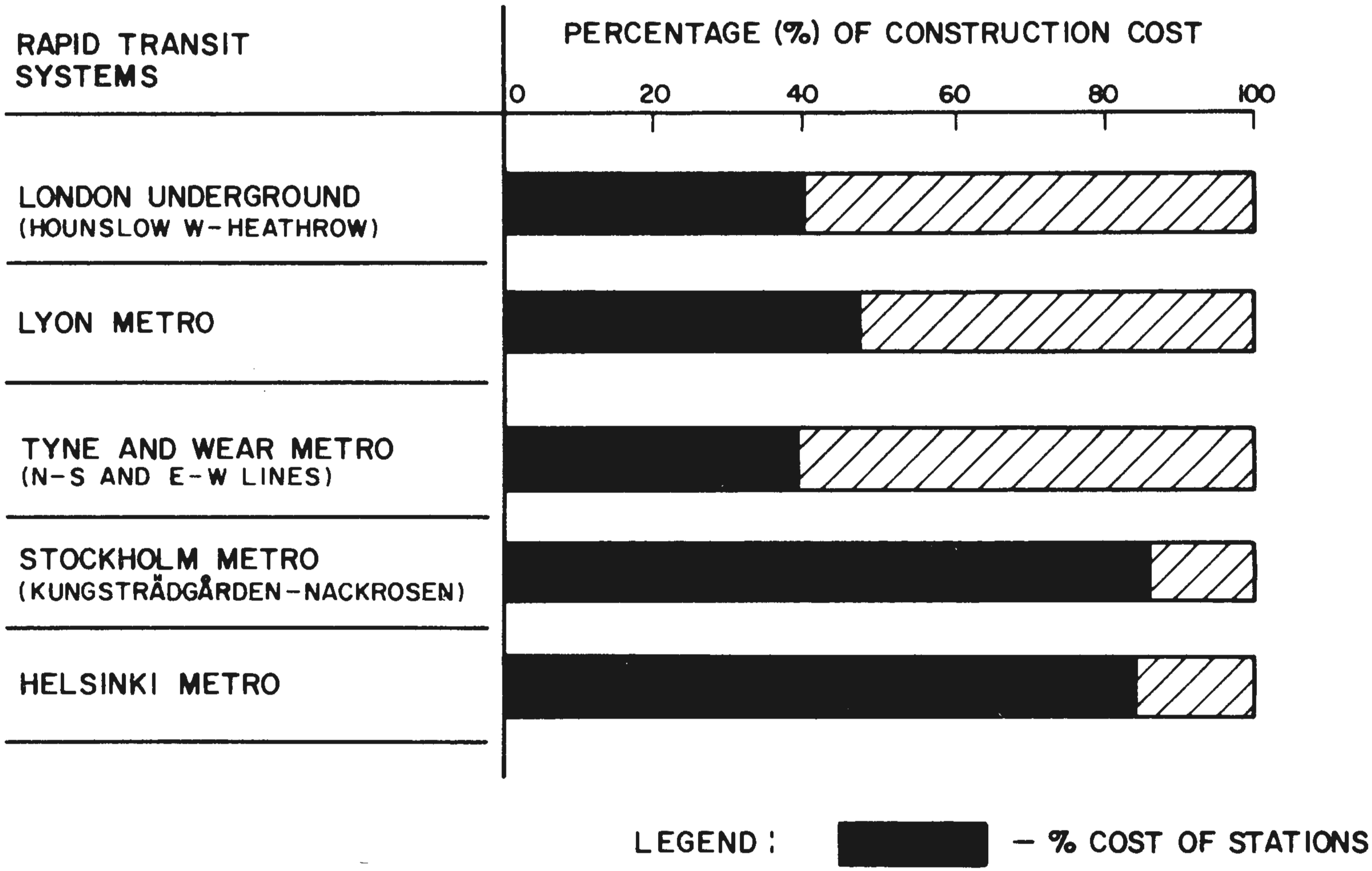


Figure 5.2 Costs of Station Construction

approximate 1 m (3.3 ft) lengths with a heavy roadheader-type excavator [total weight approximately 54,500 kg (60 tons)]. Steel arches were erected for temporary support and concrete was poured for the final tunnel lining.

Cut-and-cover methods will often involve extensive diversion or underpinning of buried services as well as road diversion and resurfacing. For the Lyon Metro, roughly 30 % of the construction cost was related to work falling under this category. For the London Underground extension to Heathrow Airport and the Brussels Metro, approximately 15 % of the construction cost was used for similar purposes.

Figure 5.3 shows a bar graph of the construction cost as a percentage of the total system cost for both the London Underground Extension to Heathrow Airport and the Lyon Metro. The total system cost is the full expense of establishing an operational system, including construction, operating equipment, land acquisition and financing. Construction costs account for approximately 60 % of the total system cost for both metros.

5.4 Cost of Labor

A detailed study of labor costs is well beyond the scope of this report. It was felt, however, that some indication of relative labor costs would be useful when judging the constituent expenses related to metro construction in different countries.

Table 5.2 shows the approximate salary range for tunnelers in five European countries. The wage information was obtained through informal discussions with contractors, engineers, and owners in the

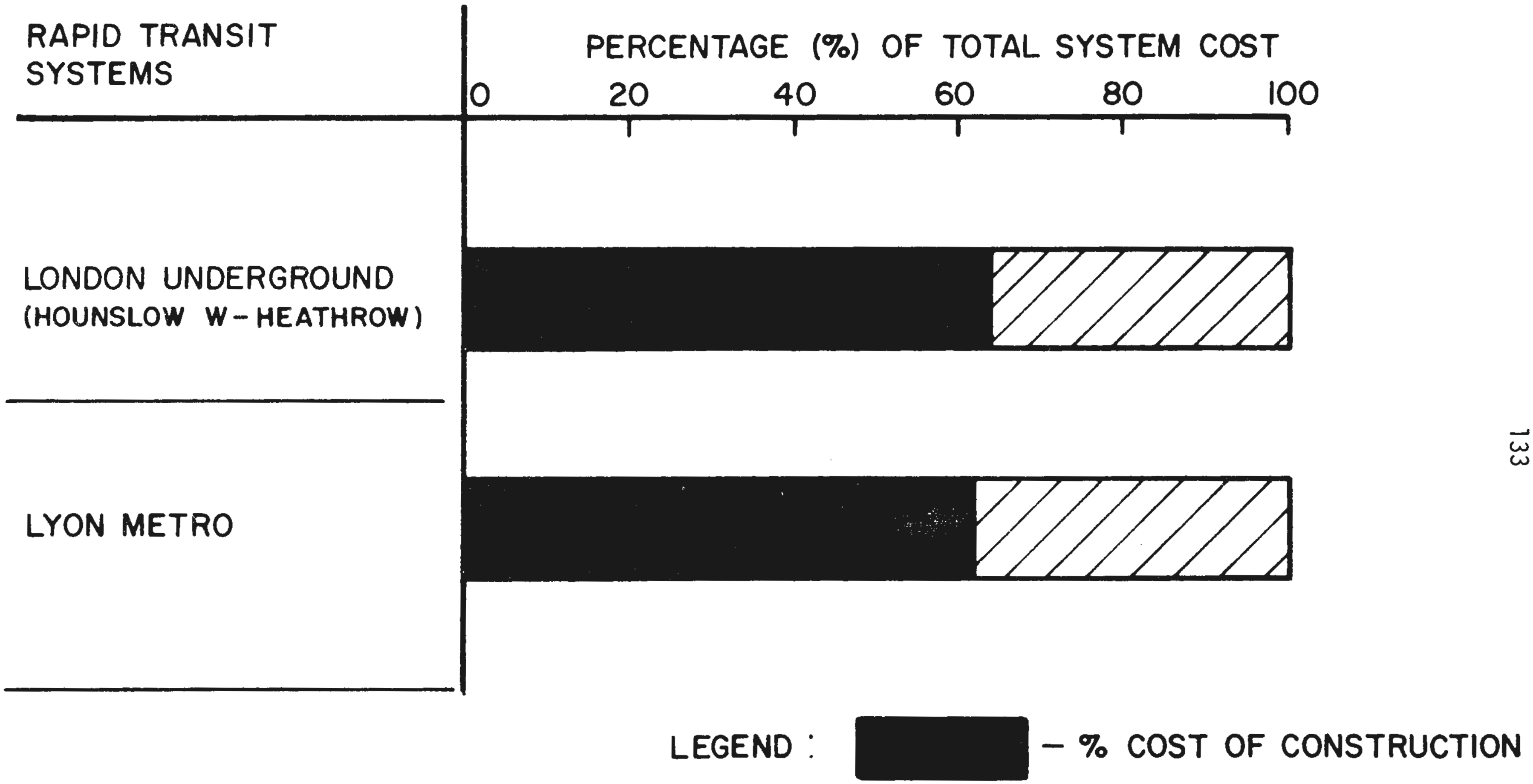


Figure 5.3 Construction Costs Relative to Total System Costs

various countries listed. The wages do not represent official labor statistics. All wages listed are for tunnelers and do not reflect the cost of other personnel such as engineers, electricians, or teamsters. In Britain, for example, tunnelers' wages are generally higher than those of other tradesman by a substantial margin. It is useful to note that bonuses for the tunnelers, as a production incentive, are frequently included within the contract package. Such practice is commonplace in Britain, Belgium, and Norway.

TABLE 5.2 TUNNELERS' WAGES IN VARIOUS EUROPEAN COUNTRIES

Country	*Wage (U. S. Dollars/wk)
**Great Britain	350-475
France	275-350
Belgium	450-550
Austria	225-275
Norway	500-600

* Wages represent 1977 price levels.

** Wages based on 50 hr week.

Note: U. S. tunnelers' wages for the same time period were $400-450/wk.

6. CONTRACT PRACTICES

6.1 Introduction

This chapter deals with European contract practices for underground construction. In particular, attention is directed to three European metro authorities so that a capsule view of how contract practices are structured can be developed for each agency. In addition, tunneling practice in the United Kingdom is studied and is used as a focal point for examining such issues as the apportionment of risk under contract and the resolution of contract disagreements.

6.2 Organization of Work Among Metro Authorities

This section summarizes, in brief, the organization and contract practice associated with three European metro authorities. The treatment in this section is not intended to be a detailed examination, but rather to provide an overview of work organization. A representative cross-section of agencies is chosen that includes the London Transport, the Regie Autonome des Transports Parisiens (Paris Metro Authority), and the Stockholms Gatukontor (directing agency for the Stockholm Underground).

The London Transport

The London Transport maintains and operates the rapid transportation system for the metropolitan area of London. Specifically, the New Work Department of the London Transport is responsible for new construction and for alterations and renewals of existing lines. The department is composed of about 100 engineers of degree standard. Engineering consulting firms are taken on to work with overload and peak demands, such as was required for construction of the Victoria Line (29) and Heathrow Airport extension (41).

The London Transport uses an admeasurement, or bill of quantities, contract. This form of contract provides for the measurement of completed work coupled with payments according to originally tendered or subsequently negotiated rates. Prospective contractors for civil construction are chosen from a list compiled on the basis of company reputation and previous work experience with London Transport. Of the prequalified contractors, London Transport is not obligated to select the lowest bidder. Instead, the choice of contractor is made according to both cost and technical merits. Alternative construction schemes may be submitted and, indeed, are often encouraged.

If a specialty service, such as grouting, is required, the specialty contractor is chosen from a group nominated by the London Transport. The London Transport reviews the specialty schemes and authorizes work on the basis of their review. Consequently, the London Transport assumes responsibility for schemes, barring negligence on the part of the specialty contractor.

Legal consultation is solicited for land purchases, Parliamentary leases, and reaching agreements with other operating authorities. If required, legal input is used to draft special contract conditions. The scale of legal involvement generally is limited. For example, during construction of the London Underground extension to Heathrow Airport, lawyers were not involved in drawing up or administering the construction contracts. The only time lawyers were involved at the construction stage was with third party claims, such as alleged property damage.

Regie Autonome des Transports Parisiens

The Regie Autonome des Transports Parisiens (RATP), maintains and operates the rapid transportation system for Paris. Engineering for new works is performed by the RATP, which designs and inspects any addition to the metro system.

The general form of contract for civil construction is the admeasurement, or unit price, contract. Work for the RATP frequently is bid by joint ventures, which are pre-selected by the RATP for participation during tender. The RATP is not required to choose the lowest bidder. Instead, the final choice of contractor is made on the basis of cost and technical merit. Alternative designs are usually encouraged at the bidding stage. Often, the alternative schemes are suggested in the contract documents.

Contractors, specializing in grouting, are generally a part of the joint ventures that bid the jobs. Grouting work is let on the basis of cost per m^3 of treated ground.

The Stockholms Gatukontor

The Stockholms Gatukontor is the engineering and supervising agency responsible for transportation works in the city of Stockholm. The work performed by the Gatukontor involves both the engineering and construction of underground transportation facilities.

Independent contractors will be called in for civil construction to cover overload or peak demands. Contracts may be awarded for only a portion of the tunneling work on a given project. For example, the Stockholms Gatukontor may perform the blasting and support on a particular section of tunnel whereas the rock pre-grouting and post grouting may be let to an independent contractor. Grouting work is undertaken according to quality specifications. In this way, rock treatment is performed until the measured inflow is less than a specified minimum quantity.

Independent contractors are not, as a rule, prequalified for participation at the tendering stage. The Stockholms Gatukontor selects the contractor by evaluating both the cost and technical aspects of the proposed work. The Gatukontor is not obligated to choose the lowest bidder.

In its invitation to tender, the Stockholms Gatukontor usually asks for 1) a lump sum, 2) a priced list of quantities, and 3) a list of unit prices. The lump sum is the total of all works mentioned in the list of quantities. Deductions or supplementary works are settled according to the unit price list. The offered unit price is generally valid for changes up to 25 % of the volume specified. Special negotiations are required for larger alterations.

As can be seen in the preceding examples, work on European rapid transportation systems tends to be highly centralized with the metro authority being the owner and engineer of the system. In some cases, such as the Stockholms Gatukontor, the metro authority actually constructs the system, using independent contractors where appropriate to cover peak demand. This particular practice is not uncommon for Scandanavia. For example, the Norwegian

Road Research Laboratory employs a permanent work force for the construction of roads and tunnels that it designs. The authority of the metro agencies is further consolidated by prequalifying bidders and by not being obligated to choose a contractor on the basis of lowest bid.

Private engineering firms do not generally participate in the overall design and coordination of construction work for continental European metros. Consulting engineers, however, figure pre-eminently in metro construction in the United Kingdom. Here, the engineering design and resident inspection of several adjacent contracts will be performed by the same firm. A close contact between the metro authority and consulting engineer is maintained. Often, a liaison engineer, representing the metro authority, will be stationed on site with the consultant's resident engineering staff.

6.3 Contract Practice for Tunneling in the United Kingdom

There are, in the main, three features that are fundamental to the structure of contract relationships in the United Kingdom. They include 1) the use of standard conditions of contract, 2) the role of the engineer during construction, and 3) the procedure for settling contract disputes. Each is discussed under a separate heading as follows:

Standard Conditions of Contract

The majority of civil construction in the United Kingdom is performed in accordance with a standard form of contract. The core document for contracting purposes, entitled "Conditions of Contract and Forms of Tender, Agreement and Bond for Use in Connection with Works of Civil Engineering Construction" [ICE Form (38)], is published by the Institution of Civil Engineers (ICE).

Since its inception in 1945, the document has been revised periodically. A discussion of the ICE form in context with U.S. contracts practice has been made by Durkee (24). A critical, legal examination of the document has been made by Abrahamson (1).

The form of contract put forth by the document is an admeasurement form of contract. As such, a bill of quantities is encorporated in the contract as a schedule that fixes rates of payment. Unless otherwise specified, the bill of quantities and measurements thereof are assumed to be in compliance with the procedure set forth by the ICE in a core document, "Civil Engineering Standard Method of Measurement" [CESMM (37)]. This, in effect, codifies the procedure for drawing up the bill of quantities and of making subsequent measurements. In general, the rules for measurement and valuation of tunneling go further than those for other classes of work in limiting the risk assumed by the contractor.

A provision for special conditions is made in the ICE Form so that the contract can be structured according to the particular demands of the job. In addition, provisions for price escalation are available in an addendum to the ICE Form.

The ICE Form benefits from constant use as its contents become known and better understood. Furthermore, periodic revisions help clarify articles shown to be ambiguous through practice. Abrahamson (1) has pointed out that the use of the ICE Form is so widespread that, in practice it acts much as a private code of legislation. The ICE Form is generally accepted by individual employers and contractors. It is subject to adjustments for special cases rather than agreement on terms that are bargained for in detail between employer and contractor.

The Role of the Engineer

The British engineer is called on to act in a dual capacity during construction. In some instances, he is expected to represent only the owner, while in other circumstances he must decide impartially on the rights of the owner and contractor under the terms of contract. The engineer's independence when making judgements on contract disputes is taken seriously, being institutionalized within the ICE Form (38), and receiving continuous attention and review by the tunneling industry at large (15).

Settlement of Disputes

In British practice serious contract disagreements are resolved by arbitration. Engineers with experience in contractual matters are chosen as arbitrators. Their decisions are binding and final except where points of law are involved. The procedure for arbitration can be arranged by the parties under contract and, thus, the practice carries a potential advantage in that the procedure can be tailor-made to fit the particular work and disposition of the job participants.

From the above background, the form of contract relationship that emerges is one in which legal involvements are restrained as the engineers' powers and responsibilities, correspondingly, are expanded. The ICE Form is structured to be serviceable without recourse to lawyers, although legal assistance may be necessary when drafting special clauses or presenting a case for arbitration. The role of the British engineers as the impartial representatives of owner/contractor rights and the fact that arbitration is performed by engineers places them in a protected and authoritative position with respect to making decisions. If insurance premiums are regarded as a rough measure of prospective litigation, then it is noteworthy that professional indemnity insurance for engineers if five times greater in the U.S. than in Britain.

In recent years there has developed in British practice an interest in and professional dialogue concerning tunneling risk. Various forms of contract have been studied with emphasis on recognizing the risk elements inherent in a given contract and on judging the suitability of different forms of contract for projects with different subsurface conditions (2, 15, 74).

Perhaps the most significant recommendation for improving British contract practice has been the introduction of ground reference conditions as a basis for both tender estimates and payments during tunneling (15). This would, in effect, expand the bill of quantities by setting definitions and bounds on various types of ground and associated rates of payment before construction. In this way, there is a clear allocation of risk between the contractor and owner. The problem, here, is choosing a definition of the ground that reasonably reflects the support requirement and rate of advance. The recommended approach is not without precedent in Britian. The contract agreement for the 32 km (20 mi.) of tunneling at the Kielder Water Scheme (10) provided for payment according to a rock classification system based on lithology, degree of fracturing, joint characteristics, and ground water. The concept of ground reference conditions has generally been accepted with favor by participants in the British tunneling industry with the recognition that exceptionally complex ground behavior may be best treated by target or cost-reimbursable contracts (74) whereas relatively straightforward projects are best tendered with the admeasurement contract as it currently stands.

7 SUMMARY AND RECOMMENDATIONS

7.1 Construction Methods

This report has reviewed several underground construction methods with respect to general principles, recent applications, and technical strengths and weaknesses. Various features of this review are summarized according to construction method under the following headings:

Grouting in Soil

In Europe, grouting in soil to decrease permeability and increase strength is used widely. Grouting projects are frequently a fundamental part of the basic design and construction plan. It is not uncommon for soil volumes exceeding 50,000 m^3 (65,000 yd^3) to be treated in preparation for building a given segment of tunnel.

Quality control and monitoring systems have been developed to complement the size and range of grouting applications. In addition to vertical boreholes, site characterization often makes use of special drilling equipment, insitu permeability tests, and test pits excavated into grouted soils. For large-scale projects records frequently are taken of depth, pumping pressure, rate of grout inflow, and total grout take for each injection point. Tube-a-manchette grouting is commonly used, which allows multiple regroutings of a particular elevation, often by grouts with sucessively reduced viscosities. Grout pressures may exceed 2 to 3 times the overburden pressures.

Grouting may be accompanied by surface heave, especially if pumping pressures in excess of the overburdenstress are used. Surface heave is caused by pressure fissuring of the soil, (referred to as claquage). Pressure fissuring is related to the grout viscosity, soil grain size, and rate of injection. Because excessive fissuring is undesirable in urban environments, injection rates and associated grouting pressures often are chosen to minimize claquage. Currently, there is need for more comprehensive field data on surface heave related to injection pressure, grout type, soil profile, and depth of injection. Urban grouting should be accompanied by monitoring overlying foundations and other surface structures for heave.

The long term effects of grouting, particularly with respect to the strength of grouted soil, deserve further study. This is especially important when grouting for underpinning purposes. It should also be recognized that many low-viscosity grouts, such as those using polyphenols and tannins, may be highly dispersive for adjacent clay layers. The effects of polyphenols and tannins on the consolidation behavior of clays has been documented elsewhere (81).

Grouting in Rock

In Scandanavia, rock tunneling has been accompanied by infiltration that has drawn down the water level in local drainage basins, thereby damaging timber foundation piles or causing consolidation of sensitive clay. Consequently, a systematic grouting procedure has often been integrated into the construction cycle when tunneling through rock under urban areas. The details of various grouting procedures used in Sweden, Finland, and Norway are presented in the text.

Many grouting programs have been effective in reducing water inflow, particularly in competent granites and gneisses. Infiltration rates as low as 1.2 to 10 liters/hr. (0.31 to 2.6 gal/hr.) per 10 m (33 ft) length of tunnel have been reported for work on the Stockholm Metro.

The success of grouting may be highly dependent on rock type and character of the local geology. For example, during construction of the Frogner Station in Oslo, grouting through fissured shale did not prevent lowering of the water level and consequent surface settlement. During construction of the second Dartford Tunnel in Britain, grouting in fissured chalk cut down on water inflow, but was less effective than anticipated in eliminating the use of high air pressures.

When rock grouting to control water inflow is used in combination with the erection of a segmental tunnel lining, grouting the annular void between the lining and exposed rock surface may be especially important. If grouting behind the lining lags the excavation by a substantial margin, loosening of the rock will promote further inflow of water and work against the beneficial effects of the original grouting. Consequently, grouting behind the lining should follow the excavation as quickly as possible within the construction cycle.

Ground Freezing

In Europe, ground freezing has been used during the construction of several metro systems. Although the relatively high price of freezing tends to limit the technique to emergency cases or as a last resort method, freezing occasionally has been financially competitive with other methods of ground treatment. Work on the London Underground has shown that, when the maintenance of frost by brine circulation is limited to 5 or 6 weeks, the costs of freezing and grouting are similar for equivalent tunneling conditions.

Ground freezing may cause a layer of frost to form between the tunnel lining and adjacent soil. When thawing occurs, voids can form throughout the zone bordering the tunnel which, in turn, can cause lining deformation and loss of ground. Careful, two-stage grouting at different times during the thaw has been used to remedy this problem.

Freezing, particularly in silty soil, can lead to the formation of ice lenses and consequent surface heave. In these cases, thawing is often accompanied by settlement that exceeds the amount of heave.

Concrete Diaphragm Walls

Field measurements associated with excavations using concrete diaphragm walls in a variety of different soils have shown maximum settlements of 0.2 to 0.5% of the maximum excavation height. The small ground movements associated with careful construction have led to the use of diaphragm walls in lieu of underpinning adjacent structures.

The construction of concrete diaphragm walls is likely to result in some voids and separations between adjoining panels. In the great majority of cases these require only minor treatment, such as raking the interval surface of the joint and refilling with cement or other sealants. When treated, joints may show local discoloring from moisture and small areas of gradual water accumulation.

Rapid transportation systems, such as the London Underground and Brussels Metro, have used concrete diaphragm elements as the walls of the permanent structure. Where architectural paneling is required, a drainage space intervening the architectural facade and concrete wall can be used to control seepage.

Difficulties with secant pile construction have been experienced on the Brussels Metro. In contrast, secant pile walls constructed for the London Underground and Tyne and Wear Metro have performed successfully and have shown significant benefits for urban work. Because the installation equipment for the piles needs only a relatively small space, isolated segments of construction can be opened up and reinstated with comparative ease. The piling rigs can bore through obstructions such as dense rubble, abandoned pipelines, and foundations. In addition, they are capable of breaking up boulders and socketing piles into bedrock.

Information on the structural design and construction details associated with 1) cast-insitu, 2) precast, and 3) secant pile walls are included in the text.

Slurry Shield Tunneling

European technology in slurry shield tunneling has achieved a firm foundation in experience. Field use of both British and German machines has led to improvements in pressure control, mucking, and filtering devices. Considerable effort has been directed to the design of a rugged tail seal between the tail skin of the shield and erected tunnel lining.

Slurry shields are attractive because they can isolate and stabilize conditions at the face by means of a sealed pressure chamber. This process will also impede access to the face. Usually, only indirect observations of ground conditions are possible. These include pressure readings from the plenum chamber, shove resistance of the cutting head, and settlement measurements at the ground surface. These limitations place great emphasis on the quality of the site investigation program, especially since the slurry shields often are sensitive to variations in geology. Blow-outs of bentonite slurry are possible, especially for shallow tunneling in loose sediments.

The New Austrian Tunneling Method for Soft Ground Tunneling

The New Austrian Tunneling Method (NATM) is developed around a firm reciprocity of observation and construction change. The method as applied to soft ground involves the use of shotcrete as temporary support in combination with careful observations of the ground behavior that are coordinated with changes in the amount of support and increments of excavation. NATM principles have been used extensively for urban tunneling in the Federal Republic of Germany and for the Marseilles Metro.

Owing to its flexibility of application shotcrete can be used to support a variety of cross-sections within a single job. In addition, roadheader excavators can be easily incorporated into the construction cycle, thereby allowing progress in mixed face conditions of soil and low-strength rock where shields might have difficulty.

Water-bearing or cohesionless sands and gravels represent a potentially hazardous situation and a major limitation of the method. In addition, advance rates associated with NATM tend to be low when compared with progress rates for shield driven tunnels in similar ground.

7.2 The Nature of Specialty Techniques

The tunneling methods reviewed in this report share a common characteristic. They depend heavily for their success on experience and personal judgments from field supervisors. For example, the grouting specialist will interpret pumping records and field measurements according to his familiarity with the equipment and previous experience. His judgment is central for the selection of grout type and the multiple regrouting of various elevations. In a similar fashion, an important person associated with cast-insitu wall construction is the platform foreman. He coordinates various aspects of the work at a given rig, oversees the panel alignment, and makes textural judgments regarding the slurry.

When specialty methods of construction are used, the contractor and field inspector should be selected on the basis of work record and experience with the technique. In addition, the selection of job participants should also carry an explicit recognition of their experience such that changes in method can follow immediately from field judgments. Rigid specifications may be counterproductive. The decision to use specialty methods and the resulting organization of work should be made so that the procedures can be modified to fit the ground conditions as they are encountered.

Perhaps in no industry outside underground construction does experience contribute so heavily to the success of a project. Reluctance to experiment or use new methods developed elsewhere carries a significant penalty in the loss of experience to key personnel. Thus, inexperienced supervisory staff tends to amplify initial conservatism when specialty methods are considered at a later date.

7.3 Organization of European Metros and Contract Practices

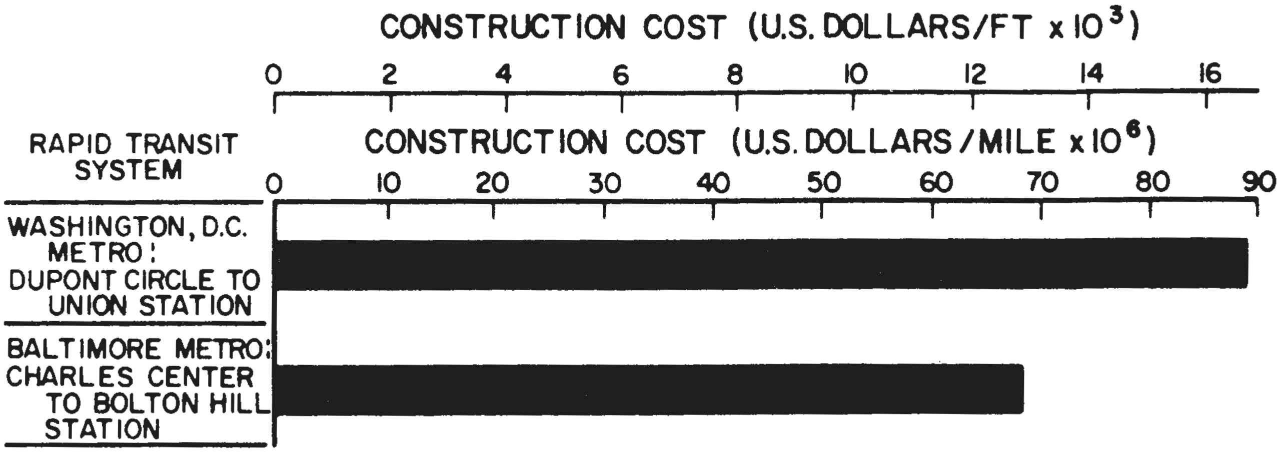
Work on European rapid transportation systems tends to be highly centralized with the metro authority being the owner and engineer of the system. European metro authorities generally have an in-house staff of qualified engineers with capabilities in design and construction management for additions to their systems. In some cases, such as the Stockholms Gatukontor, the metro authority actually constructs the system, using independent contractors where appropriate. Often, the control exercised by metro agencies is further consolidated by prequalifying bidders and by not being obligated to choose a contractor on the basis of lowest bid. In the United Kingdom, where private consulting firms figure prominently in metro construction, a close contact between the metro authority and consulting engineer is maintained. Frequently, a liason engineer, representing the metro authority, will be stationed on site with the consultant's resident engineering staff.

Contract practices generally are organized to limit participation by the legal profession. In the United Kingdom, for example, the majority of underground construction is performed in accordance with a standard form of contract. The core document for contracting purposes is published by the British Institution of Civil Engineers. In practice, the core document acts as a private code of legislation that is automatically accepted by most employers and contractors. In addition, serious contract disagreements are decided by arbitration. Engineers with experience in contractual matters are chosen as arbitrators. Their decisions are binding and final except where points of law are involved.

7.4 Underground Construction Costs

The compilation of unit construction costs among several European metros provides an opportunity to compare construction costs in Europe with those in the U.S. Two U.S. metros, the Washington, D.C. and Baltimore Metros, have been chosen to represent U.S. rapid transit construction.

Figure 7.1a shows a bar graph of unit construction costs associated with the Washington, D.C. and Baltimore Metros. The costs are derived from the initial construction stages of each system. The costs for the Washington, D.C. Metro are based on contract awards made between 1969 and 1971 (92, 93) which have been adjusted to 1975 prices by an inflation factor. Additional cost adjustments have been made for claims approved as of January, 1977 to obtain a total construction cost for each project included in the unit price. The costs for the Baltimore Metro are based on contract awards made during



a) Bar Chart of Construction Cost

RAPID TRANSIT SYSTEM	AVRG. DEPTH (FT)	TUNNEL CROSS SECTION (FT)	STATION DIMENSIONS (FT)	DISTANCE OF LINE PER STATION (MI)	APPROX. GEOLOGY
WASHINGTON, D.C. METRO: DUPONT CIRCLE TO UNION STATION	55-60	12 x 30	PLATFORM LENGTH=600	0.51	DENSE SAND BELOW WATER LEVEL
BALTIMORE METRO: CHARLES CENTER TO BOLTON HILL STATION	60-70	I.D. 17.9	PLATFORM LENGTH=450	0.65	DENSE SAND BELOW WATER LEVEL

b) Summary of Information

Figure 7.1 Underground Construction Costs and Information Summary for Two U. S. Metros

1977 (59). It should be recognized that the Baltimore prices do not reflect potential savings from value engineering proposals or cost overruns. A detailed breakdown of the project costs contributing to the unit expense of each system is included in Appendix B. The unit costs are for underground constructions in the central business districts of both cities.

Construction cost is defined in the same way as in Chapter 5 with the exception that the expense of underpinning adjacent structures also is covered in the unit price. All unit costs have been determined by dividing the total construction cost for a given distance of line by the length of the line under study. Hence, the unit costs represent cost per double track mile or foot and include the expense of building both stations and running tunnel. All costs apply to underground portions of the rapid transit systems.

Basic information pertaining to the sections of the metros represented in the bar graph is summarized in Fig. 7.1b. The information is intended primarily as a convenience so that general comparisons can be made between the systems.

Figure 7.2 compares the underground construction costs for U.S. and European metros in the form of a bar chart. All unit costs are for urban underground construction in soil. On the basis of the information summarized, U.S. costs are substantially in excess of European costs.

In some cases, direct comparisons between the costs of metro systems may be misleading. For example, the number of stations included in the unit prices of the U.S. metros are approximately twice the number for the London Underground. Furthermore, the ground conditions under which the London tunnels were driven cannot be equated with the waterbearing sands and

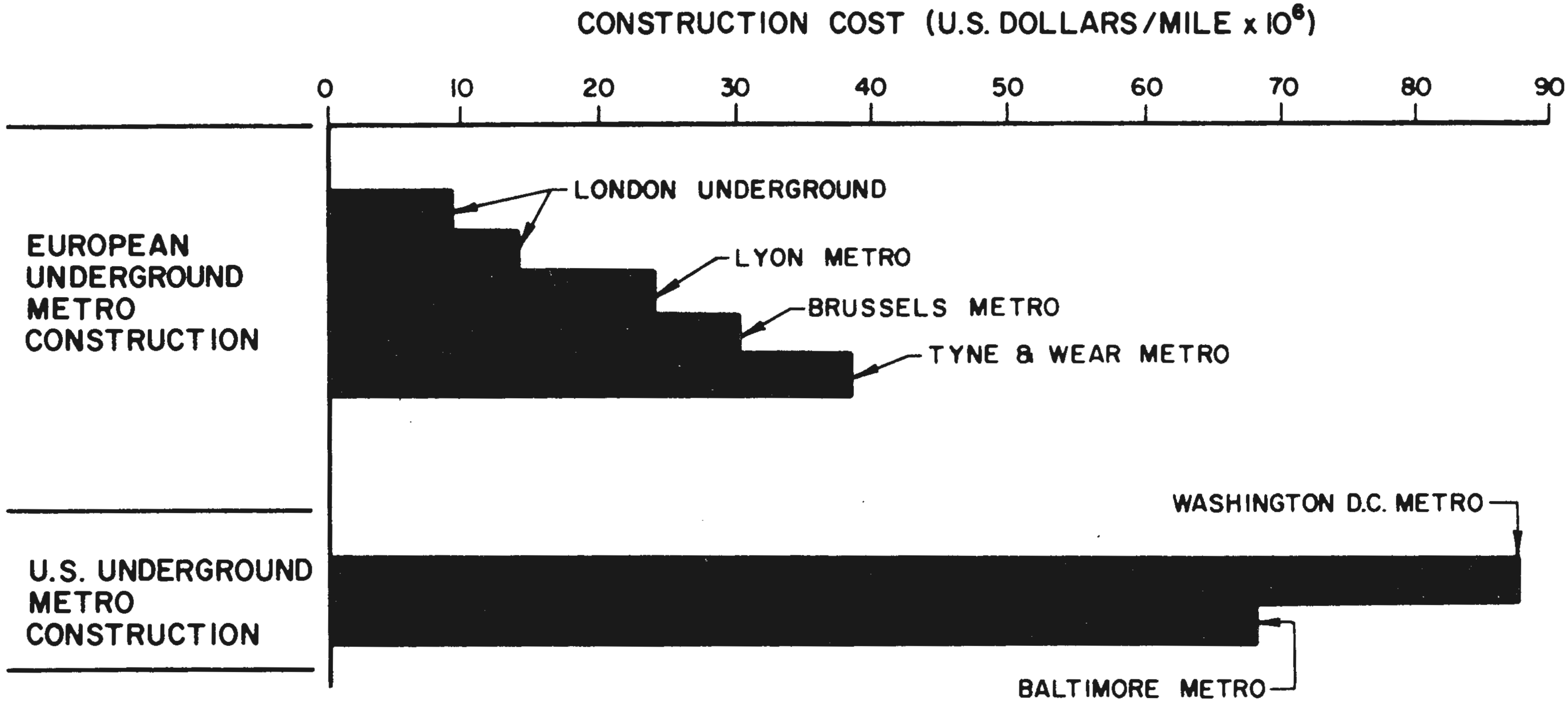


Figure 7.2 Comparison of U. S, and European Underground Construction Costs

silts in which the Washington, D.C. and Baltimore Metros were constructed. Crew sizes used to drive single track tunnels for the London Underground were less than half that for single track tunnels in Washington, D.C. and Baltimore.

On an overall basis, however, the unit costs for the European metros cover a variety of ground conditions, some of which are similar to those prevalent for the U.S. metros. Approximately 60% of the soil tunnels for the Tyne and Wear Metro were driven under compressed air. Cut-and-cover construction for the Brussels Metro was typically extended to depths of 18 m (60 ft) through sands and interbedded clay below the water table. Furthermore, the number of stations per mile on several of the European metros (see Table 5.1) exceeds the corresponding number on the U.S. metros. In terms of labor costs, tunnelers' wages in Europe are comparable to those in the U.S. (see Table 5.2).

Costs reflect a variety of factors, many of which are grounded in institutional constraints or contract practices that affect the scale of construction, design criteria, work organization, and legal involvement. Aided by comparisons among construction practices in various European countries and in the U.S., recommendations for improving urban tunneling are offered in the following section.

7.5 Recommendations

Recommendations concerning technical judgments, metro management, and working relationships are numbered and discussed under the following headings:

1. <u>Reassessment of the Scale of Metro Structures</u>

In Europe, metro stations contribute between 40 and 85% of the construction cost for an underground line. Because they occupy a disproportionately large segment of the cost structure, increases in station size will have a substantial impact on the cost of a metro system.

In most cases, platform length will be proportional to station size. Correspondingly, platform length can be used for comparative purposes to judge station size. The platform lengths for the Washington, D.C. and Baltimore Metros are 183 and 137 m (600 and 450 ft), respectively. The platform lengths for the Tyne and Wear, Lyon, and Brussels Metros are 100 m (328 ft) or less. It is unlikely that these size variations can be attributed solely to passenger capacity, as the European cities included in the comparison have metropolitan area and suburban populations equal to or slightly smaller than those of the U.S. cities.

Variations in scale also extend to tunnel diameter. For example, the internal diameter of single track tunnels for the London Underground, the Tyne and Wear and Bochum Metros are 3.8, 4.75, and 6.0 m (12.5, 15.6, and 19.5 ft), respectively. In the U.S., the diameter of a single track, rapid transit tunnel is approximately 5.5 m (18 ft). Variations in diameter are prevalent despite the fact that construction economies are closely linked with size. In a summary analysis of 60 different contracts the Construction Industry Research and Information Association [CIRIA (16)] has shown that the average cost of a tunnel doubles as the diameter increases from 3.8 to 5.5 m (12.5 to 18 ft) for construction in Britain. In addition, Girnau (31) has shown that unit costs may increase by over 50% in response to similar increases in tunnel diameter for construction in the Federal Republic of Germany.

Occasionally, architectural concepts may call for large openings to create a monumental effect. Under these circumstances, expansion in scale might be limited to a small number of stations so that the bulk of the line is constructed with smaller, less costly stations while the system can still benefit from the "showcase" structures that are accessible from all parts of the system. Furthermore, it should be recognized that lighting and color often can be used in lieu of increased size to create similar effects.

A comparative analysis of tunneling practice points to the need for greater appreciation of how size variation will affect system cost.

2. Elimination of Redundant Support

Cut-and-cover tunnels are frequently supported on a temporary basis with soldier pile-and-lagging or sheet pile walls. In many instances, concrete diaphragm walls are used for temporary support primarily to take advantage of their inherent stiffness and the associated relaxation of dewatering requirements. When the permanent underground structure is built within temporary bracing, the cost of the project must absorb two systems of support. Recent design practice in Europe, as is evidenced by the Brussels Metro and London Underground, have incorporated concrete diaphragm walls as part of the permanent structure. Pertinent examples have been discussed (see Chapter 3) and suggest consideration for similar use elsewhere.

3. Evaluation of Material and Performance Specifications

Material and performance specifications probably vary more widely than any other aspects of tunnel construction. Occasionally, materials used with great success in one system are prohibited in another. Segmental, concrete liners are an example of a support item that were applied for many years in European practice, but only recently have been used in U.S. tunnels.

An area of special interest centers on the requirements regarding leakage. Until recently, concrete diaphragm walls for station construction on the Paris Metro were used solely as temporary support owing, in part, to concern about leakage at panel interfaces. In contrast, station design on the London Underground has recognized that minor seepage is likely and provides for a false wall and drainage culvert at the station interior. In a similar manner, water proofing for running tunnels is approached from different perspectives. For example, the water-proofing requirements in the Federal Republic of Germany have been likened to the demand that the tunnel should be so dry that dust will not cling to it (31). Consequently, casting tolerances of as low as 0.5 mm (.02 in.) may be required for concrete segmental lining elements. These relatively stringent specifications are reflected in increased cost. The unit expense of tunnels with segmental concrete linings in Germany have been priced at levels twice as high as segmental concrete tunnels of the same diameter in similar ground under British practice (16, 31). In the United Kingdom, the tunnel seepage rates specified by various railway authorities may differ by a factor of 20 (17).

In view of the economic effects, it seems reasonable to question the degree of stringency incorporated into waterproofing specifications. Considerations of water-tightness must include the type of ground, methods of measurement, and long term maintenance, but should not lose sight of economic impact.

4. <u>Adoption of Specialty Methods for Underground Construction</u>

In Europe, certain specialty methods of construction are widely used. In particular, the development of and experience gained with soil grouting and diaphragm wall construction recommend these methods for consideration elsewhere. Although the use of a specialty technique does not guarantee savings in cost, familiarity with and willingness to use the methods does provide a greater baseline of options with which to approach different tunneling problems. The successful application of grouting or diaphragm wall construction requires, in all cases, a contractor thoroughly familiar with and experienced in the method.

5. Consolidation of Technical Services

Work on European rapid transportation systems tends to be highly centralized. Not only are metro authorities both the owners and engineers of the systems, but some are contractors for major works. In Britain, where private consulting engineers figure prominently in new work, the design and administration of several contracts often will be let as a block to a single firm. In all cases, design and construction management are performed by the same engineers. In fact, the ICE Form is formulated explicitly on the assumption that the designers will administer the work.

The complexity of large-scale urban construction points toward consolidation of services rather than greater diversity. Often, conservatism is amplified by separating responsibilities. For example, when section design and construction management are performed by different agencies, the design engineer must deal with uncertainties in site inspection and construction quality as part of a future event over which he has no direct control. His assumptions, correspondingly, are likely to reflect these uncertainties in an effort to cover a wide range of contingencies and protect himself from future litigation.

6. Selective Award of Contract

European metro authorities award contracts on a selective basis. Their control of contract award may be structured in either or both of two ways: 1) contractors are pre-qualified for tender, and 2) contract award is not obligated to the lowest bidder.

Tight control of selection, if administered with fairness and strict compliance with technical judgments, carries certain benefits. The most important of these include the screening of bids that are purposefully or naively low. This, in turn, helps safeguard contractors against being penalized because they allowed for legitimate eventualities that competitors had overlooked.

Alternatively, selective practices carry certain disadvantages. Selective tendering can be used to punish a contractor for previous disagreement even though he had made a justifiable claim. In addition, there are problems with regard to new entrants into the industry.

It seems reasonable, given the size and complexity of underground projects, especially with regard to uncertainties about subsurface conditions, that relatively strict controls of contract award should be exercised. It is not necessary that control take the form of prequalifying bidders, although this measure may be appropriate under certain circumstances. Stipulations imposed on public agencies to award contracts on the basis of lowest tender should be reevaluated. Practices of this sort are open to cost overruns because claims and time loss are likely substitutes for initial savings based on a job erroneously underbid.

7. Improved Control of Disputes

There has been a good deal of concern expressed about contract disputes and the best ways of limiting both their origins and potential size (2, 15, 20, 47, 60, 71, 89). Frequent recommendations have been made for a clear assignment of risk and liability among the parties to the construction contract. Owners, especially of large public systems, should recognize that a fair and expeditious interpretation of changed ground or quantity variations will offset claims and recurrent high bidding. At the level of the individual contract, attention should be directed to dealing with contingencies inside the framework of the contract document. A bill of quantities, for example, may be flexible or rigid according to the provisions for which it is drafted. If potential difficulties are foreseen on the basis of the engineer's judgment, then bill items can be specified under which extra work or delay are to be measured and paid for if they materialize.

In general, education on legal principles as applied to underground construction seems to be lacking as a prelude to litigation. It would be useful, therefore, to consider a more active and more widely coordinated participation on the part of the legal profession in summarizing and explaining principles of law as applied to underground work. This could take the form of seminars and workshops directed, in particular, at practicing engineers, public works administrators, and underground contractors. Subjects for discussion could include the nature of claims likely to arise in given practical circumstances, clauses regarding variation in rates and changed conditions, professional liability, and court rulings with commentary on how these rulings make up general principles of Common Law.

Because disputes associated with tunneling often require expert evaluation of geotechnical and structural problems, decisions can be prolonged and, indeed, obstructed by formal litigation. Correspondingly, the adoption of alternative methods of settlement, particularly arbitration, should be seriously considered where appropriate. Along these lines, the U.S. National Committee on Tunneling Technology has recommended and outlined procedures for binding and non-binding arbitration (88).

8. Preservation of Technical Authority

An overview of underground construction practice discloses one clearly perceptible pattern: the authority of the engineer diminishes as the legal involvement in the system increases. Tunneling carries an inherent uncertainty about the ground conditions that, by nature, provides a basis for disagreement. When this potential for disagreement is amplified by vigorous legal activity, it is not surprising for the engineer to act with increased conservatism and to distribute responsibility among various review and regulatory agencies.

Occasionally, the institutional framework of the system protects the engineer. For example, in British practice, the engineer's role as an independent arbiter of owner/contractor rights and the fact that contract disputes are decided by arbiters, who are engineers, places the engineering profession in a clearly protected position with respect to the decisions that its members make.

In some cases, it may be appropriate for the owner to share the risk of introducing new construction methods or making cost-saving modifications in design. Federal government agencies and municipal authorities are often in an advantageous position to share the financial risk of an engineering judgement, particularly if the judgement results in diminished expense.

Underground construction requires judgements of a technical nature for which there are no adequate substitutes in legal consultation. The preservation and extension of technical authority should be a basic concern of the tunneling industry and a continuous source of critical reexamination.

REFERENCES

1. Abrahamson, M. W. (1975) "Engineering Law and the ICE Contracts", Applied Science Publishers, 3rd Edition.

2. Abrahamson, M. W. (1973) "Contractural Risks in Tunnelling: How They Should be Shared", Tunnels and Tunnelling, November, pp. 587-593.

3. American Cyanamid Company (1965) "AM-9 Chemical Grout".

4. Association Francaise des Travaux en Souterrain (1975) "Recommandations Concernant les Travaux d'Injection Pour les Ouvrages Souterrains", (Recommendations for the Use of Grouting in Underground Constructions) Tunnels et Ouvrages Souterrains, July.

5. Bartlett, J. V., A. R. Biggart, and R. L. Triggs (1974) "The Bentonite Tunnelling Machine", Proc. Instr. of Civil Engrs. Part 1, pp. 605-624.

6. Bergman, G. A., K. Lindman, L. Lundström, P. Soderman, and S. Ullerud (1975) "Tunneltätning mot Små Läckage", (Grouting of Tunnels to Prevent Small Scale Infiltration), Report R25: 1975, Swedish Council for Building Research.

7. Bertran, P. (1977) "La Liaison Invalides-Orsay de la SNCF: Presentation Generale et Environment", (The Invalides-Orsay Connection of the SNCF: General Presentation and Surroundings), Tunnels et Ouvrages Souterrains, April, pp. 71-72.

8. Boden, J. B. and C. McCaul (1974) "Measurement of Ground Movements during a Bentonite Tunneling Experiment", TRRL Report 653, Crowthorne, Berkshire.

9. Bougard, J. F. (1975) "Extending the Paris Metro", Tunnels and Tunnelling, May, pp. 43-48.

10. Burston, V. T. and D. J. Coates (1975) "Water Resources in Northumbria with Particular Reference to the Kielder Water Scheme", Journ. Instn. of Water Engr. Scientists, Vol. 29, pp. 226-251.

11. Cambefort, H. (1977) "The Principles and Applications of Grouting", Q. Journ. Engr. Geol., Vol. 10, pp. 57-95.

12. Caron, C., J. P. Delisle, and W. H. Godden (1963) "Resin Grouting, with Special Reference to the Treatment of the Silty Fine Sand of the Woolich and Reading Beds at the New Blackwall Tunnel", Grouts and Drilling Muds in Engineering Practice, London, Butterworths, pp. 142-145.

13. Clough, G. W. (1976) "A Report on the Practice of Chemical Stabilization around Soft Ground Tunnels in England and Europe", Dept. of Civil Engr., Stanford Univ.

14. Chapeau, C. (1977) "Application du Procede de Congelation a L'Azote Liquide", (The Application of Freezing with Liquid Nitrogen), Tunnels et Ouvrages Souterrains, July, pp. 143-150.

15. Construction Industry Research and Information Association (1977) "Tunnelling-Improved Contract Practices", Report 79.

16. Construction Industry Research and Information Association (1977) "Standardization of Tunnel Cross-Sections", Report 66.

17. Construction Industry Research and Information Association (1978) "Waterproofing Tunnels", draft copy, in preparation.

18. Corps of Engineers (1973) Department of the Army Manual No. EM 1110-2-3504, "Chemical Grouting", Office of the Chief of Engrs., Washington, D. C.

19. D'Appolonia, D. J., E. D'Appolonia, and E. Namy (1974) "Proc. of Workshop on Cut and Cover Tunneling: Precast and Cast-in-Place Diaphragm Walls Using Slurry Trench Techniques", Report No. FHWA-RD-74-57, Federal Highway Administration.

20. Dellaire, G. (1976) "Desperate Need to Slash Construction Costs of New Subways", Civil Engineering, Vol. 46, No. 12, pp. 37-45.

21. DiBiagio, E. and F. Myrvoll (1972) "Full Scale Field Tests of a Subway Trench Excavation in Soft Clay", 5th European Conf. on Soil Mech. and Fnd. Engr., Madrid, Vol. 1, pp. 461-472.

22. DiBiagio, E. and J. A. Roti (1972) "Earth Pressure Measurements on a Braced Slurry-Trench Wall in Soft Clay", 5th European Conf. on Soil Mech. and Fnd. Engr., Madrid, Vol. 1, pp. 473-483.

23. Dochy, C. (Brussels Metro Authority), Personal Communication.

24. Durkee, J. L. (1977) "Needed: U. S. Standard Conditions for Contracting", Jour. of the Constr. Div., ASCE, June, pp. 191-200.

25. Eide, O. G. and T. Josang (1972) "Special Application of Cast-in-Place Walls for Tunnels in Soft Clay in Oslo", 5th European Conf. on Soil Mech. and Fnd. Engr., Madrid, Vol. 1, pp. 485-498.

26. Farmer, I. W. and P. B. Attewell (1973) "Ground Movements Caused by a Bentonite-Supported Excavation in London Clay", Geotechnique, Dec., pp. 576-581.

27. Ferrand, J., A. Boller, C. Bouyant, and R. Fages (1976) "Le Metro de Lyon et la Mechanique des Sols", (The Lyon Metro and Soil Mechanics), Technica No. 390, March-April, pp. 13-23.

28. Ferrand, J. and A. Boller (1975) "Le Metro Lyonnais: les Infrastructures de Genie Civil", (The Lyon Metro: The Civil Engineering Infrastructures), Technica, No. 385, May-June, pp. 7-26.

29. Folenfant, H. G. (1969) "The Victoria Line: The Project", Instn. Civil Engineers, Supplementary Vol., Paper 7270S.

30. Fournier, J. (1977) "Realisation des Travaux de la Liaison Invalides-Orsay" (The Construction of the Invalides-Orsay Connection), Tunnels et Ouvrages Souterrains, April, pp. 73-75.

31. Girnau, G. (1978) "Lining and Waterproofing Techniques in Germany", Tunnels and Tunnelling, April, pp. 36-45.

32. Goldberg, D. T., W. E. Jaworski, and M. D. Gordon (1976) "Lateral Support Systems and Underpinning", Report No. FHWA-RD-75-130, Federal Highway Administration.

33. Haffen, M. and Janin, J. (1972) "Grouting Cohesionless Water-Bearing Soils in Cities", RETC Conf., Chicago, pp. 1539-1568.

34. Harris, J. (Foracky Ltd.), Personal Communication.

35. Helsinki Metrotoimisto (1976) "Facts about the Helsinki Rapid Transit", pamphlet.

36. Herndon, T. and T. Lenahan (1976) "Grouting in Soils", Report No. FHWA-RD-76-26 Federal Highway Administration.

37. Institution of Civil Engineers (1976) "Civil Engineering Standard Method of Measurement".

38. Institution of Civil Engineers (1973) "Conditions of Contract and Forms of Tender, Agreement, and Bond for Use in Connection with Works of Civil Engineering Construction", 5th Edition.

39. Ischy, E. and R. Glossop (1962) "An Introduction to Alluvial Grouting", Proc. Instn. Civil Engrs., March, pp. 449-474.

40. Jacob, E. (1976) "The Bentonite Shield: Technology and Initial Application in Germany", Tunnelling '76, Paper 19, London, March.

41. Jobling, D. G. and A. C. Lyons (1976) "Extension of the Piccadilly Line from Hounslow West to Heathrow Central", Proc. Instn. Civil Engrs., Part 1, May, pp. 191-218.

42. Jobling, D. G. (1975) "Diaphragm Walls and Secant Piles in Subway Construction," Proc. Underground Construction Problems, Techniques, and Solutions, Chicago, pp. 321-348.

43. Jobling, D. G. (Director of New Works Department, London Transport), Personal Communication.

44. Jorge, G. (Soletanche Enterprise), Personal Communication.

45. Karlsrud, K. (1976) "Instrumentering av slissevegger ved fellestunnelen gjennorn Studenterlunden" (Instrumentation of a Two-Storey Tunnel Through Studenterlunden Using the Diaphragm Wall Technique), NGI Geotekinhkdagen 76, artikkel 17.

46. Karlsrud, K. (1975) "Prakiske erfaringer med utforelse av slissevegger i Oslo-leire", (Practical Experience from the Excavation of Slurry Trenches in Oslo Clay), NGI publ. No. 110, pp. 39-47.

47. Kellog, J. C. (1976) "Contract Problems in Major Underground Construction Projects", Underground Space, Vol. 1, pp. 101-109.

48. Kerisel, M. and M. Lupiac (1975) "Contraintes Exercees par le Sol sur la Station Auber a Paris", (Soil Stresses Acting on the Auber Underground Railway Station under Paris), 8th Intl. Conf. on Soil Mech. and Found. Engr., Moscow, Vol. 2, pp. 121-126.

49. Kuesal, T. R. (1977) Rebuttal to "Cut Subway Costs", article, Civil Engineering, Vol. 47, No. 4, p. 54.

50. Laabmayr, F. (1976) "Soft Ground Tunnel for the Munich Metro," Proc. Conf. on Shotcrete for Ground Support, Easton, Md., pp. 352-372.

51. Lane, K. S. (1977) "Instrumented Tunnel Tests: A Key to Progress and Cost Saving", Underground Space, Vol. 1, pp. 247-259.

52. Lee, I. (1969) "The Victoria Line; Escalator Tunnel at Tottenham Hale Station", Proc. of Institution of Civil Engrs., Paper 7270-5.

53. Leeney, J. (1976) "The European Tour", Tunnels and Tunnelling, May, pp. 64-65.

54. LeGac, J. (1977) "La Construction de la Gare Souterraine de Paris-Lyon", (The Construction of the Paris-Lyon Underground Station), Tunnels et Ouvrages Souterraines, April, pp. 70-71.

55. Lenzini, P. A. and B. Bruss (1975) "Ground Stabilization: Review of Grouting and Freezing Techniques for Underground Openings", Report No. FRA-ORD&D-75-95, Federal Railroad Administration.

56. Lupiac, L. (1978) "The Paris Conference Organized by AFTES", Tunnels and Tunnelling, letter, Jan./Feb.

57. Lysen, L. (1975) "Tatning av Bergtunnlar" (Sealing of Rock Tunnels), Report to Swedish Council for Building Research, Research Notice P55.

58. Manser, A. W. (1968) "London Underground", Proc. Conf. on Performance of Electrical Railways, IEE Conf. Publ. 50.

59. Maryland Department of Transportation Mass Transit Administration, communication through the Director of Systems and Construction.

60. Mayo, R. S., J. E. Barrett, and R. J. Jenny (1976) "Tunneling: The State of the Industry, Report No. DOT-TSC-OST-76-29, Office of Secretary of Transportation.

61. Moller, K. (1976) "Groundwater Control for Underground Construction", Conf. on Underground Services, London, January.

62. Moller, K. (1972) "Grouting Now", Consulting Engineer, August.

63. Moller, K. (Soil Mechanics, Ltd.) Personal Communication.

64. Mordfeldt, C. (1977) "Ar Injektering i Berg en Svartkonst?", (Is Grouting in Rock a "Black Art"?), Paper presented at Bergmekanikdagen, Stockholm.

65. Mouxaux, T. (Soletanche Enterprise), Personal Communication.

66. Müller, L. and G. Spaun (1977) "Tunnel Construction in Soft Ground", Rock Store, Vol. 3, Stockholm, pp. 3-8.

67. Müller, L. and G. Spaun (1977) "Soft Ground Tunneling Under Buildings in Germany", 9th Int'l Conf. on Soil Mech. and Found. Engr., Tokyo, Vol. 1, pp. 663-667.

68. Müller, L. (1976) "NATM at Bochum", Tunnels and Tunnelling, letter, Sept.

69. New Civil Engineer (1977) "Bentonite Mole on Shaky Ground", 1 December, pp. 16-17.

70. Organization for Economic Cooperation and Development (1976) "Economic Surveys by the OECD: France".

71. O'Rourke, T. D. (1978) "U. S. and U. K. Tunnelling: Developments, Comparisons, and Trends", Tunnels and Tunnelling, January, pp. 56-60.

72. O'Reilly, M. P. (1977) "NATM Settlement and Economics", Tunnels and Tunnelling, letter, Jan.

73. Peck, R. B. (1969) "Advantages and Limitations of the Observational Method in Applied Soil Mechanics", Geotechnique, 19, pp. 171-187.

74. Perry, J. G. and P. A. Thompson (1975) "Target and Cost-Reimbursible Contracts; A Study of Their Use and Implications", Construction Industry Research and Information Association, Report 56.

75. Pilon, B. (1977) "La Realisation de L'Infrastructure de L'Operation de Renovation du Quartier de Halles" (The Organization of the Infrastructure, Operation, Renovation, and Dimensions at Halles), Tunnels et Ouvrages Souterrains, April, pp. 60-63.

76. Rosell, B., C. Ehrman, I. Bäckström, and B. Hillbom (1975) "The Stockholm Underground '75", Stockholm Municipal Services Department and the Greater Stockholm Passenger Transport Company

77. Sanger, F. J. (1968) "Ground Freezing in Construction", Journ. of Soil Mech. and Fnd. Div., ASCE, SMI, pp. 131-158.

78. Shuster, S. A. (1972) "Controlled Freezing for Temporary Ground Support," Proc. RETC Conf., Chicago, Vol. 2, pp. 863-895.

79. Sliwinski, Z. and W. G. Fleming (1975) "Practical Considerations Affecting the Construction of Diaphram Walls", Diaphram Walls and Anchorages, ICE, London, pp. 1-10.

80. Societe d'Etudes de Metropolitan de l'Agglomeration Lyonnaise (1974) "Le Metro Lyonnaise", (The Lyon Metro), pamphlet.

81. Solderblom (1966) "Chemical Aspects of Quick Clay Formation", Q. Journ. Engr. Geology, Vol. 1, pp. 415-431.

82. Stevens, A. (Ove Arup and Partners, London), Personal Communication.

83. St. John, H. (1975) "Field and Theoretical Studies of the Behavior around Deep Excavations in London Clay", Ph.D. Thesis, Cambridge University.

84. Stroud, M. A. and D. J. Sweeney (1977) discussion, "Diaphram Walls and Anchorages", I. C. E. Appendix, pp. 142-148.

85. Tallard, G. R. and C. Caron (1977) "Chemical Grouts for Soils", Report No. FHWA-RD-77-50, Federal Highway Administration.

86. U. S. Department of Labor, Bureau of Labor Statistics (1976) "BLS Handbook on Methods for Surveys and Studies", Bulletin 1910, U. S. Gov. Printing Office, Washington, D. C. 20402.

87. U. S. Department of Labor, Bureau of Labor Statistics (1977) "Handbook of Labor Statistics 1977", Bulletin 1966, U. S. Gov. Printing Office, Washington, D. C. 20402.

88. U. S. National Committee on Tunneling Technology (1977) "Recommended Procedures for Settlement of Underground Construction Disputes", National Academy of Sciences, Washington, D. C.

89. U. S. National Committee on Tunneling Technology (1974) "Better Contracting for Underground Construction", Washington, D. C., National Academy of Sciences, PB 236 973.

90. Vincent, M. (1977) "La Realisation du Metro de Marseille: Comparison de la Nouvelle Methode Autrichienne a la Method Traditionnelle", (The Construction of the Marseilles Metro: Comparison of the New Austrian Method with the Traditional Methods), Tunnels et Ouvrages Souterrains, April, pp. 91-93.

91. Walsh, T. and A. R. Biggart (1976) "The Bentonite Tunnelling Machine at Warrington", Tunnelling '76, Paper 35, London, March.

92. Washington Metropolitan Area Transit Authority, Office of Program Control, Construction Contracts Obligations Forecast Reports, Sept. 10, 1976.

93. Washington Metropolitan Area Transit Authority, Office of Construction Bimonthly Report, Dec. 1976- Jan. 1977 and earlier issues.

94. Woodman, B. J. (Tarmac Construction, Ltd.), Personal Communication.

95. Woitchik, M., "The Brussels Metro: Methods of Execution of Civil Engineering Works", Special Research Depart., S. T. I. B., Brussels, 2nd Edition.

96. Xanthakos, P. P. (1974) "Underground Construction in Fluid Trenches", University of Illinois at Chicago Circle.

Appendix A
INFORMATION SUMMARY FOR SIX EUROPEAN METROS

Introduction

This appendix summarizes information on 6 European metro systems. In particular, the information includes data on passenger capacity, dimensions and structural materials, ground conditions, construction methods, and costs.

The passenger capacity of a rapid transportation system will depend on the number of passengers carried in a single car, the number of cars per train, and the headway, or time interval between trains. Estimates concerning the number of passenger spaces in a single car may be based on a crush loading that, in turn, may be modified for the distribution of passengers among various cars and for the tendency to avoid over-crowding. Furthermore, headways will vary depending on peak and off-peak operation. Many systems are capable of operating on a headway of 90 to 120 seconds, but only for a portion of an hour.

Passenger capacity is computed as the sum of standing passengers and available seats on a typical train multiplied by the number of trains operating in one direction in one hour. In this report the passenger capacity is recorded in two ways. To provide reasonably comparable data, the number of standing passengers is estimated by assuming one standing passenger per 0.37 m^2 (4 ft^2) of available standing area, which is consistent with observed patterns (58). This is used to compute the standard capacity. Peak, or crush, capacities are also quoted. The peak capacity generally is estimated on the basis of one standing passenger per 0.14 $in.^2$ to 0.19 $in.^2$ (1.5 to 2.0 ft^2), although the Lyon Metro uses a slightly higher ratio of one standing passenger per 0.25 m^2 (2.7 ft^2). A four minute headway is assumed, although many of the systems summarized have the capability of operating at headways less than two minutes.

I LONDON UNDERGROUND EXTENSION TO HEATHROW AIRPORT

I.1 General Description

The extension of the London Underground to Heathrow Airport is composed of 5.5 km (3.4 mi) of subsurface rail. As shown in Fig. I.1a, the line is located beneath several busy roads and operational airfields. There are three underground stations. The trains and passenger capacity of the line are summarized in Table I-1. The dimensions and structural materials for the metro are summarized in Table I-2.

I.2 Geology and Soil Profile

The soil profile along the line is relatively constant, being composed of approximately 0.5 m (1.6 ft) of fill underlain by roughly 5 to 8 m (16 to 26 ft) of terrace sands and gravel, all of which rest on stiff to hard London clay. At Heathrow Airport the London clay is approximately 46 m (151 ft) thick. The water table is located at approximately 2 m (6.5 ft) and 5 m (16.5 ft) below ground surface at the eastern and western ends of the line, respectively. Figure I.1b shows a profile view of the metro from Hounslow West to Heathrow Central Station.

I.3 Construction Methods

The construction of the line is discussed in detail by Jobling and Lyons (41) and only the more salient aspects of the construction are described here. Approximately 2.6 km (1.60 mi) and 0.9 km (0.55 mi) of the line were built with cut-and-cover and open trench methods, respectively. Support for the excavation was provided with either steel sheet or secant

TABLE I-1 TRAINS AND PASSENGER CAPACITY FOR HEATHROW EXTENSION OF THE LONDON UNDERGROUND

No. of cars per train:	8
Loading:	900 people/train (Standard)
Passenger capacity: (for train interval of 4 minutes)	13500 people/hr (Standard)
	20000 people/hr (Peak)

TABLE I-2 DIMENSIONS AND STRUCTURAL MATERIALS FOR HEATHROW EXTENSION OF THE LONDON UNDERGROUND

Running Tunnels:
double track box structure,
internal dimensions = 7.33 m (24 ft) wide
approx. 4.0 m (13.0 ft) high
walls,
secant pile walls = 890 mm (2.9 ft) dia.
reinforced concrete walls = 451 mm (1.5 ft) thick

twin tube tunnels,
internal diameter = 3.81 m (12.5 ft)
lining = precast segmental concrete

Heathrow Central Station
plan dimensions = 131.5 m (431 ft) long
22 m (72 ft) wide
concrete box structure
cast-insitu walls = 1 m (3.3 ft) thick

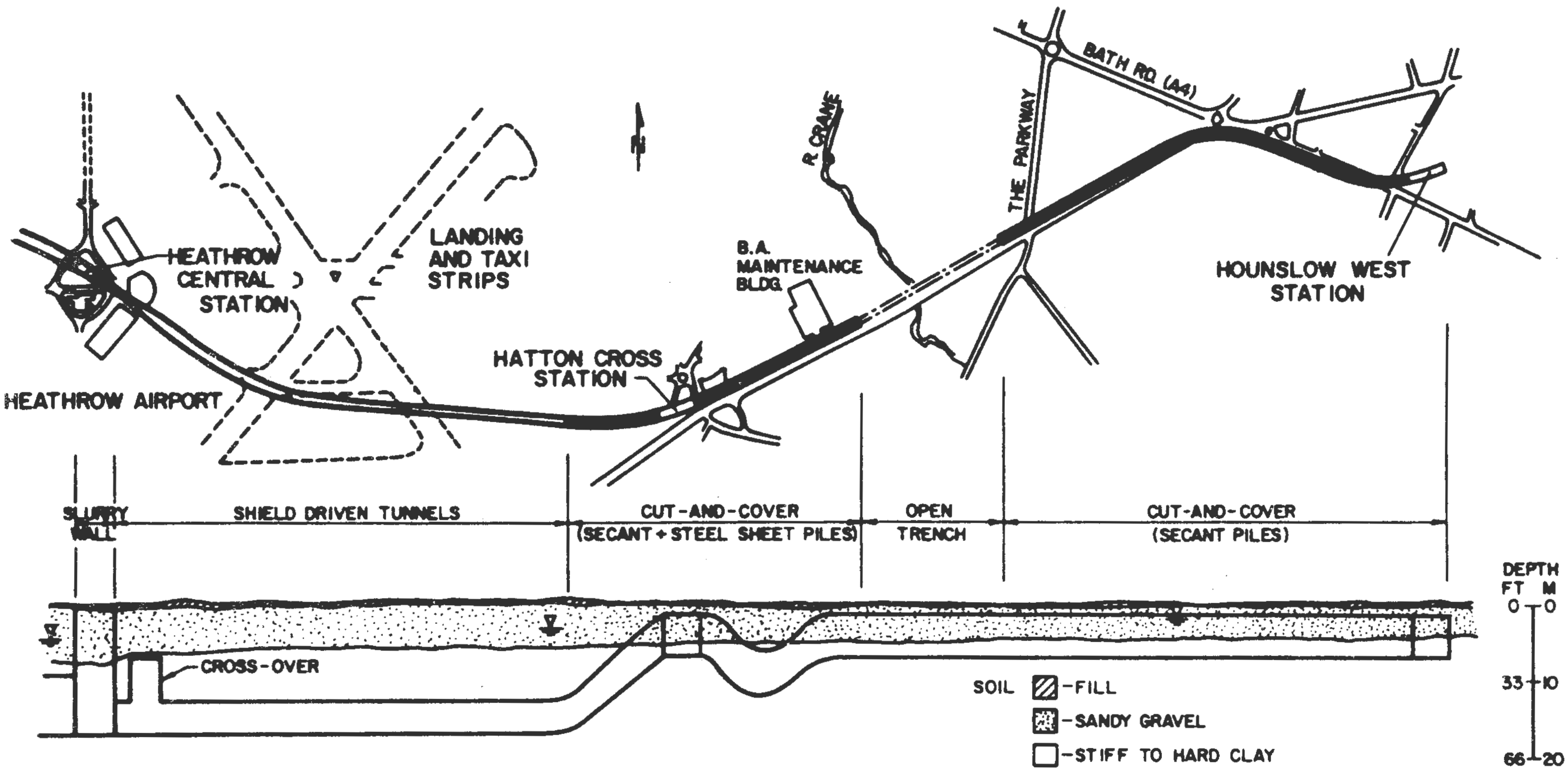


Figure I-1 Plan and Profile Views of Heathrow Extension to the London Underground piles. Difficulties with driving the sheet piles in London clay were experienced. Secant piles were used along most of the excavation. The 890 mm (2.9 ft)-diameter piles were placed on 800 mm (2.6 ft) centers and used as the permanent walls of the structure. In addition, secant piles were used as the permanent structural walls at Hounslow West and Hatton Cross Stations.

Twin, 3.81 m (12.5 ft)-diameter tunnels were driven from Heathrow Central to a point west of Hatton Cross Station. The tunnels were driven with shields and roadheader-type excavators. Expanded, concrete segments were used as the tunnel lining. A substantial portion of the tunnel was lined with segments of pressed concrete. Segments, formed with a hydraulic press, eliminated the need for the large number of moulds that are normally required during production of precast elements. A 9.5 m (31 ft)-diameter cross-over for the system was driven under compressed air. This section of the tunnel was 54 m (177 ft) long and located just east of Heathrow Central Station.

Heathrow Central Station was built as a concrete box structure with ticket hall, mezzanine, and platform levels. One-meter-thick, cast-insitu walls were used as the temporary support for the excavation and as the permanent walls of the structure. They are 20 m (65.6 ft) deep.

I.4 Costs

The construction costs associated with various aspects of the underground extension to Heathrow Airport are summarized in Table I-3. The civil engineering costs reflect mid-1975 prices, whereas the architectural costs reflect mid-1977 prices. Conversion of pounds sterling to U.S. dollars has been made using the exchange rate of 1 pound:1.9 U.S. dollars.

TABLE I-3 COSTS FOR THE LONDON UNDERGROUND EXTENSION TO HEATHROW AIRPORT

Item	($x\ 10^6$ U.S. dollars) *Civil Engineering Costs	**Architectural Costs
Heathrow Central Station	5.50	4.35
Shield Driven Tunnels	8.15	
***Hatton Cross Station	0.85	2.45
****Hounslow West Station	0.89	0.48
Cut and Cover Tunnels and Approach Works	13.55	0.20

* Costs include diversion of public facilities, excavation, support, box and basic tunnel structure.

** Costs include architectural fittings, platforms, ticket halls, pumphouse and ventilation shafts.

*** Costs include electrical sub-station and bus facilities.

**** Hounslow West Station uses a previously constructed ticket hall.

Note: Information on costs by courtesy of London Transport.

II TYNE AND WEAR METRO SYSTEM

II.1 General Description

The Tyne and Wear Metro System will provide rapid rail transportation for the Newcastle area of north-east England. Overall, the transit system will use 40.8 km (25.3 mi.) of existing British Rail track coupled with 14.8 km (9.2 mi.) of new lines. Four and eight tenths km (2.2 mi) of underground lines will form the nucleus of the system within the central business district of Newcastle and Gateshead. The general lay-out of the underground works is shown in Fig. II-1. There are seven underground stations, of which Monument Station provides the cross-over for the North-South and East-West arteries. The trains used and passenger capacity of the system are summarized in Table II-1. The dimensions and structural materials for the underground lines are summarized in Table II-2.

II.2 Geology and Soil Profile

The geology of the Newcastle area consists mainly of glacial sediments overlying Coal Measures strata. Most of the soft ground tunnels were advanced through boulder clay, which is a hard, clayey till with embedded cobbles and boulders. Overlying the boulder clay is laminated clay, which generally is stiff in consistency, but may contain pockets of soft to medium material. Figure II-2 provides a profile view of the soil deposits along the tunnel route from Central to Jesmond Station. Two water levels are evident: one perched in the laminated clay at approximately 6 m (20 ft) below ground surface, and the other located in boulder clay at a depth of roughly 22 m (72 ft) below ground

TABLE II-1 TRAINS AND PASSENGAR CAPACITY FOR THE TYNE AND WEAR METRO

No. of cars per train:	2 or 3
Loading:	135 people/car (Standard)
*Passenger capacity: (for train interval of 4 minutes)	4050 people/hr (Standard) (2-car trains)
	6000 people/hr (Peak)

* Initially, the system will accommodate 2 cars per train, but will have the capacity for 3 car trains if required in the future.

TABLE II-2 DIMENSIONS AND STRUCTURAL MATERIALS FOR THE TYNE AND WEAR METRO

Running Tunnels in Soil:
internal diameter = 4.75 m (15.6 ft)
lining: Precast segmental concrete or segments of gray cast iron

Running Tunnels in Rock:
Arch-Shaped, height = 5.8 m (19 ft)
width = 5.4 m (17.7 ft)
radius = 2.4 m (7.9 ft)

temporary lining = steel arches
permanent lining = in-situ cast concrete

Stations:
internal diameter: 7.0 m (23 ft)
lining: segments of spheroidal graphitic iron
platform length: 95 m (312 ft)
platform width: 3 m (9.8 ft)

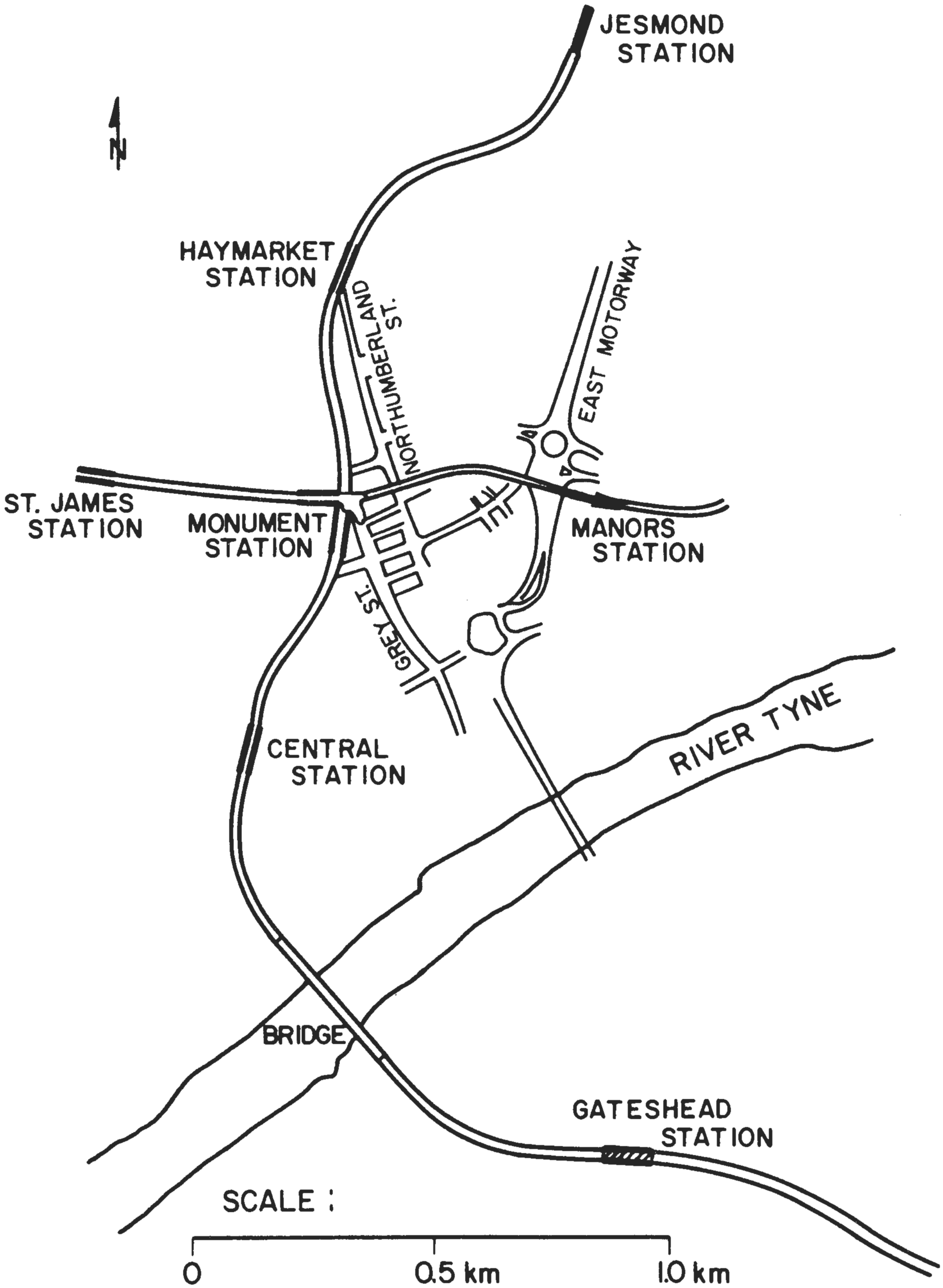


Figure II-1 Plan View of Tyne and Wear Metro

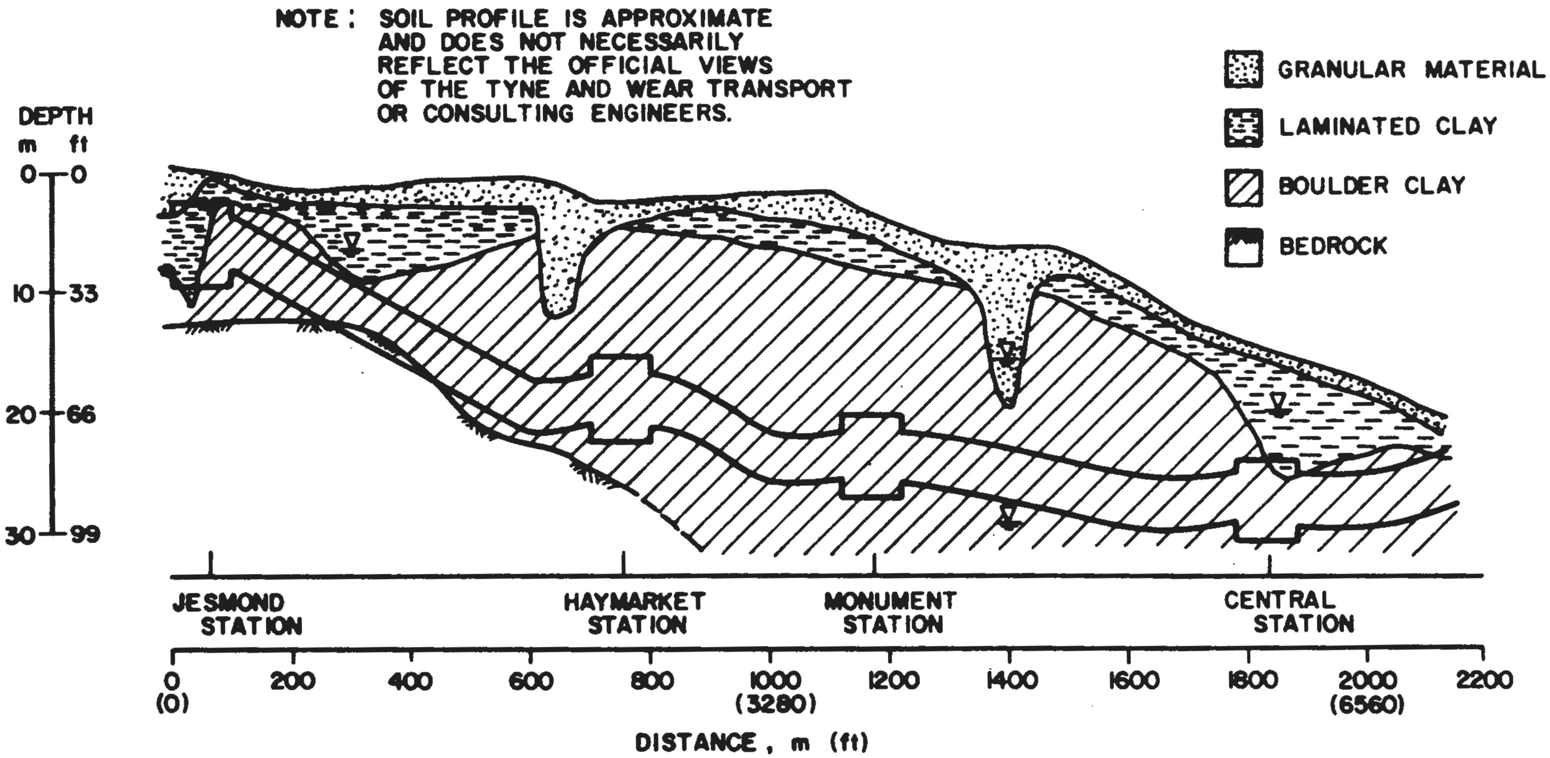


Figure II-2 Profile View of the Tyne and Wear Metro, N-S Line surface. Infilled valleys, which may contain very soft to soft clay, dissect the glacial deposits. Coal Measures strata, which were intercepted by tunnels south of the Tyne, are composed of sandstone, limestone, and shale with extensive coal seams that were mined prior to metro construction.

II.3 Construction Methods

The running tunnels were shield driven in either free or compressed air at approximately 80 kN/m^2 (12 psi). Compressed air was used beneath heavily built-up areas of Newcastle where silty or sandy lenses were suspect.

Several stations (Haymarket, Central, and the north-south portion of Monument Station) were constructed as twin 7.0-m-diameter (23 ft) tubes by hand excavating at the beginning of each station a chamber, roughly 6.6 m (21.6 ft) long, in which a shield was constructed. The remainder of the station-length was driven with the shield and lined with segments of graphitic iron. Box structures for two stations (Monument and St. James Stations) were constructed with secant piles, which provided temporary bracing as well as permanent support.

Rock tunneling was performed with a boomheader excavator (manufactured by Paurat GMBH). The temporary lining was composed of steel H-sections on approximate 1-m longitudinal spacings. Extensive ground treatment with cement/fly-ash grout was required to fill mined-out portions of the coal seams.

II.4 Costs

Underground construction in the central business district of Newcastle and Gateshead was performed under three separate projects.

The construction costs associated with these projects are summarized in Table II-3. All costs have been adjusted to June 1976 prices using indices provided by the British Department of Environment. Conversion of pounds sterling to U.S. dollars has been made by using the exchange rate of 1 pound:1.9 U.S. dollars.

TABLE II-3 COSTS FOR THE TYNE AND WEAR METRO

Project	Avg. Depth	Distance of Tunnel	Number of Stations	Cost
North-South Line	18 m (60 ft)	1.8 km* (1.13 mi)	2 1/2	$44.0 million
East-West Line	14 m (46 ft)	0.8 km* (0.53 mi)	1 1/2	$20.4 million
Gateshead	16 m (52 ft)	0.9 km (0.54 mi)	1	$13.7 million

* distance for twin tunnels

Note: Costs for North-South and East-West Lines include running tunnels station enlargements, escalator shafts, concrete/steel lining, architectural finishes, and secant pile box structures. Treatment of mine workings is not included in the Gateshead project costs.

Information on costs by courtesy of the Tyne and Wear Passenger Transport Executive

III THE LYON METRO

III.1 General Description

The "first line" of the Lyon Metro system is composed of nearly 11.4 km (7.1 mi) of rail, which is distributed throughout the central business district of the city as shown in Fig. III-1a. Approximately 11.0 km (6.85 mi) of the line was constructed underground (excluding bridge and surface sections). There are fifteen underground stations. The trains and design capacity of the system are summarized in Table III-1. The dimensions and structural materials for the underground lines are summarized in Table III-2.

III.2 Geology and Soil Profile

The near surface deposits at Lyon are mostly terrace sands and gravels derived from the Rhone River. The pervious nature of these materials ($1 \geq k \geq 10^{-1}$ cm/sec) in combination with a high water table led to the construction of the underground system at depths as shallow as possible to minimize problems with ground water control. Figure III-1b shows a profile view of the system from Perrache Station to Ateliers.

The soil profile, in general, is composed of 2 to 5 m (6.5 to 16.4 ft) of fill underlain by terrace deposits of sand and gravel to a depth of approximately 15 m (49 ft) where molasse is encountered. The molasse is substantially less pervious ($10^{-2} \geq k \geq 10^{-3}$ cm/sec) than the terrace deposits. The water table is located approximately 5 m (16.4 ft) below the ground surface.

TABLE III-1 TRAINS AND PASSENGER CAPACITY FOR THE LYON METRO

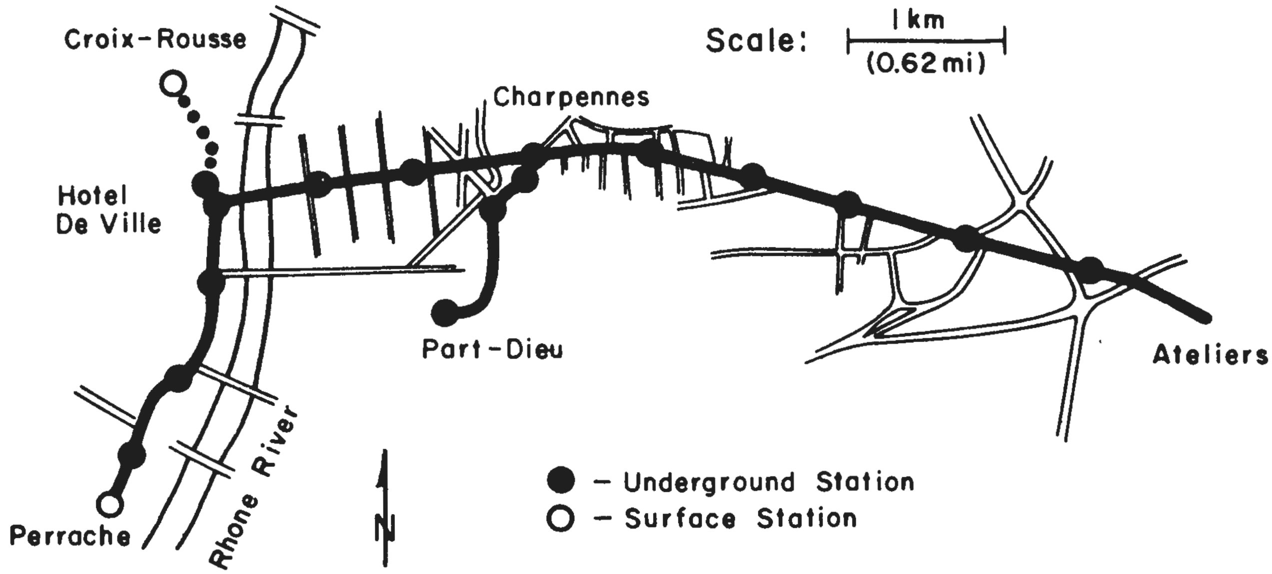
No. of cars per train	3	
Loading:	306 people/train	(Standard)
Passenger capacity:	4600 people/hr	(Standard)
(for train interval of 4 minutes)	5600 people/hr	(Peak)

TABLE III-2 DIMENSIONS AND STRUCTURAL MATERIALS FOR THE LYON METRO

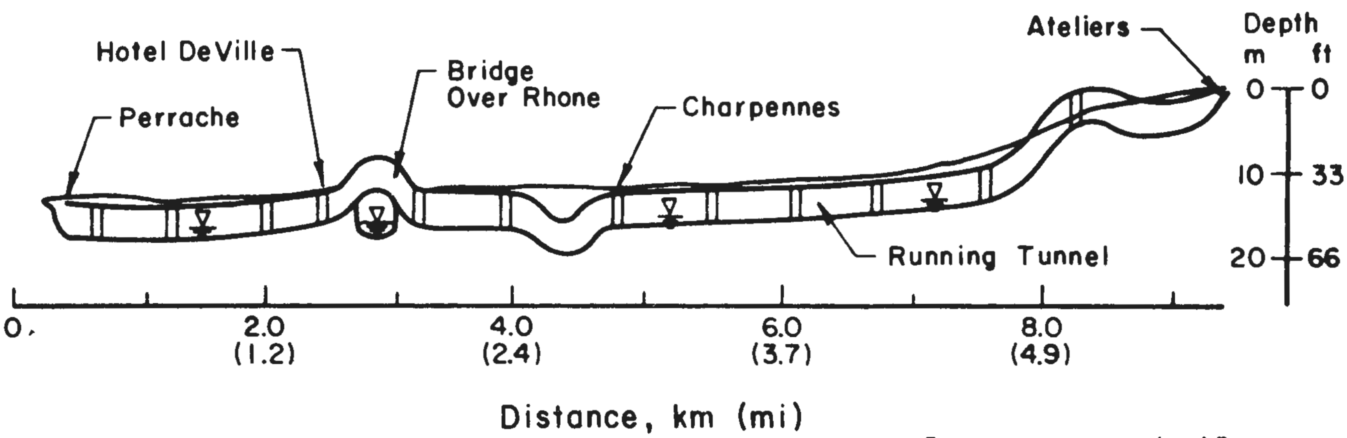
Running Tunnels:
double track box section,
internal dimensions = 7.50 m (24.6 ft) wide
3.98 m (13 ft) high
reinforced concrete walls = 400 to 500 mm
(1.30 to 1.65 ft) thick

Stations:
reinforced concrete box structure
platform length = 95 m (312 ft)
platform width = 3 m (9.8 ft)

At Part-Dieu, plan dimensions = 104 x 32 m (341 x 105 ft)



a) Plan View



[After SEMALY (80)]

b) Longitudinal Profile

Figure III-1 Plan and Profile Views of the Lyon Metro

III.3 Construction Methods

A detailed description of the geotechnical problems and construction techniques has been provided by Ferrand and Boller (28) and Ferrand et al (27). The construction methods were devised for work below the water table and along congested, city streets. Although the metro line was sited as shallow as possible, the lower part of the box structure was generally below the water level.

Most of the line was built by using either steel sheet piles or diaphragm walls as temporary support. The excavation was carried to subgrade either under immersed conditions, in which case the base slab was tremied in place, or under dry conditions brought about by grout injection of the soil below invert. Along the deeper sections, diaphragm walls were embedded in the molasse to form a cut-off against water seepage.

Two types of diaphragm wall were used. In some of the relatively wide streets, cast-insitu walls were formed. Each 0.6-m (2 ft)-thick wall, in combination with the concrete wall of the box structure, yielded a total thickness of 1.10 m (3.6 ft). In the relatively narrow streets, prefabricated walls were placed. Each wall was 0.25 mm (0.8 ft) thick and,in combination with the wall of the box structure, yielded a total thickness of 0.6 m (2 ft).

III.4 Costs

The total cost of the Lyon Metro is summarized under various categories in Table III-3. All costs have been adjusted from 1972 levels to 1976 levels by using an inflation index of 1.69*. Conversion of French francs to U.S. dollars has been made using the exchange rate of 5 French francs:1 U.S. dollar.

* Inflation index determined by compounding the average annual price increase for consumer goods, fuel and power, and semi-processed industrial products from 1972 through 1976. Statistics on price increases are available from Organization for Economic Cooperation and Development (70).

TABLE III-3 COSTS FOR THE LYON METRO

Item	Cost in U.S. Dollars x 10^6
Land acquisition	17.6
Diversion of public facilities and surface constructions	49.9
Civil engineering (including additional operations at Part Dieu and Perrache)	107.1
Site supervision and direction of the work	9.5
Rolling stock	30.7
Equipment	48.6
Lyon Metro Authority	12.4
Insurance and finance	1.0
Total	276.8

Note: information on costs by courtesy of the Societies d'Etudes du Metropolitain de l'Agglomeration Lyonnaise (SEMALY).

IV THE BRUSSELS METRO

IV.1 General Description

The Brussels Metro will ultimately be composed of 5 separate lines servicing the central business district and suburban areas of the city. Currently, work is in progress or complete on 4 lines, of which Line 1 is the most extensive. Figure IV-1 shows a simplified plan view of the city on which the 2.0 km-long (1.2 mi) section of Line 1 from De Broukerie to Maelbeek Station has been emphasized. The trains used and design capacity of the system are summarized in Table IV-1. The dimensions and structural materials for the underground lines are summarized in Table IV-2.

IV.2 Geology and Soils Profile

The soils underlying Brussels are highly variable in character. The depth of fill throughout the city ranges from approximately 0.5 m (1.6 ft) to 10 m (33 ft). Beneath the fill are alluvial deposits made up of clay, peat, sand, and gravel layers. Chalky sands and discontinuous banks of soft sandstone are found below the alluvium. These, in turn, are underlain by interbedded sands and clays, all of which rest on a green, glauconitic sand. The water table varies in depth from 4 to 20 m (13 to 66 ft) below the ground surface.

A profile view of Line 1 from De Broukerie to Maelbeek stations is illustrated in Figure IV-2. The various methods used for construction of this portion of the line are indicated in the figure.

TABLE IV-1 TRAINS AND PASSENGER CAPACITY FOR THE BRUSSELS METRO

No. of cars per train:	4	
Loading:	105 people/car	(Standard)
Passenger capacity: (for train interval of 4 minutes)	6,300 people/hr	(Standard)
	12,600 people/hr	(Peak)

TABLE IV-2 DIMENSIONS AND STRUCTURAL MATERIALS FOR THE BRUSSELS METRO

Running Tunnels
double track box structure,
internal dimensions = 7.5 m (24.6 ft) wide
4.65 m (15 ft) high

Walls,
cast-insitu walls = 0.8 to 1.0 m (2.6 to 3.3 ft) thick

Stations
plan dimensions: 14 m (46 ft) wide
platform = 95 m (312 ft) long

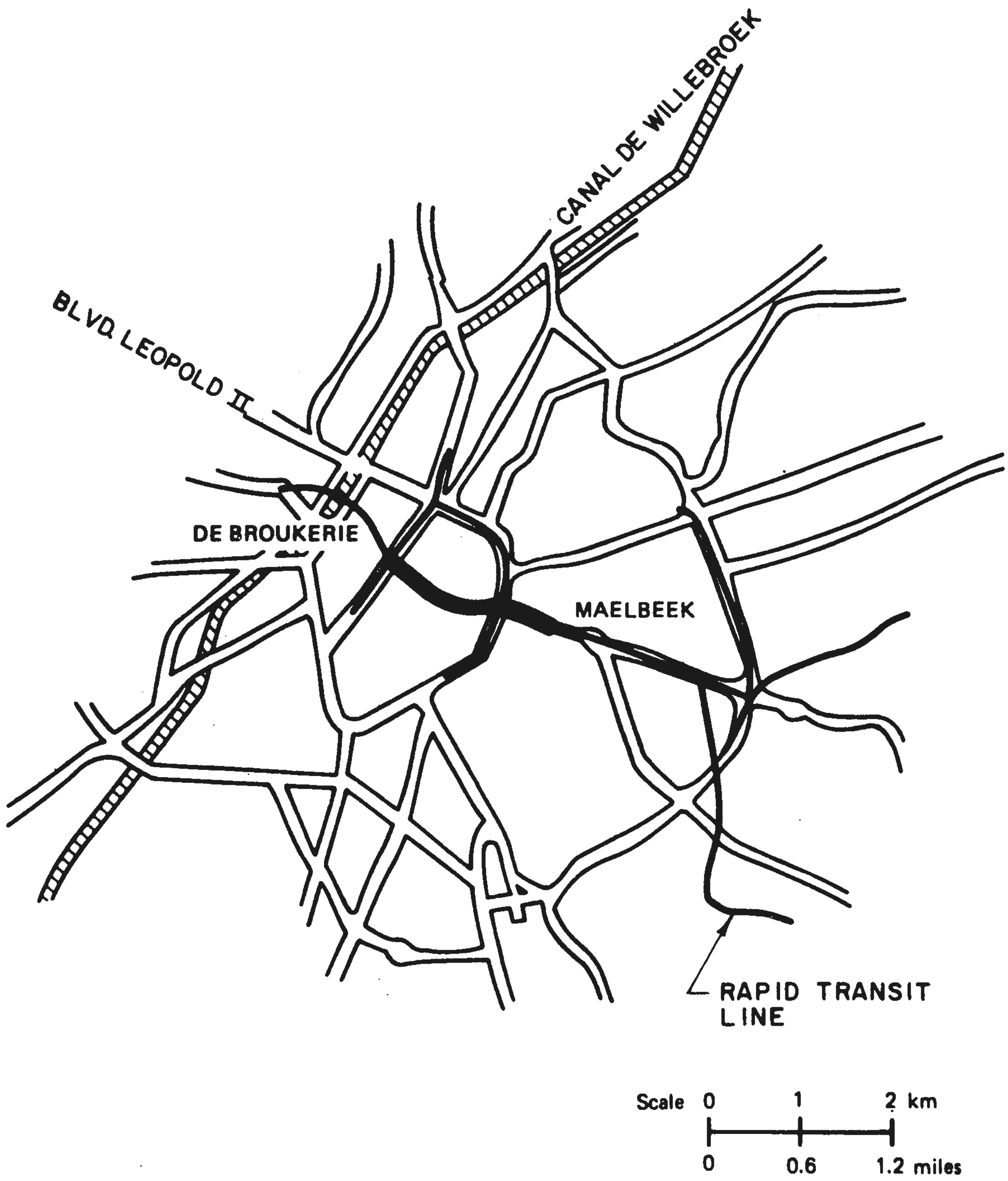


Figure IV - 1
Plan View of the Brussels Metro

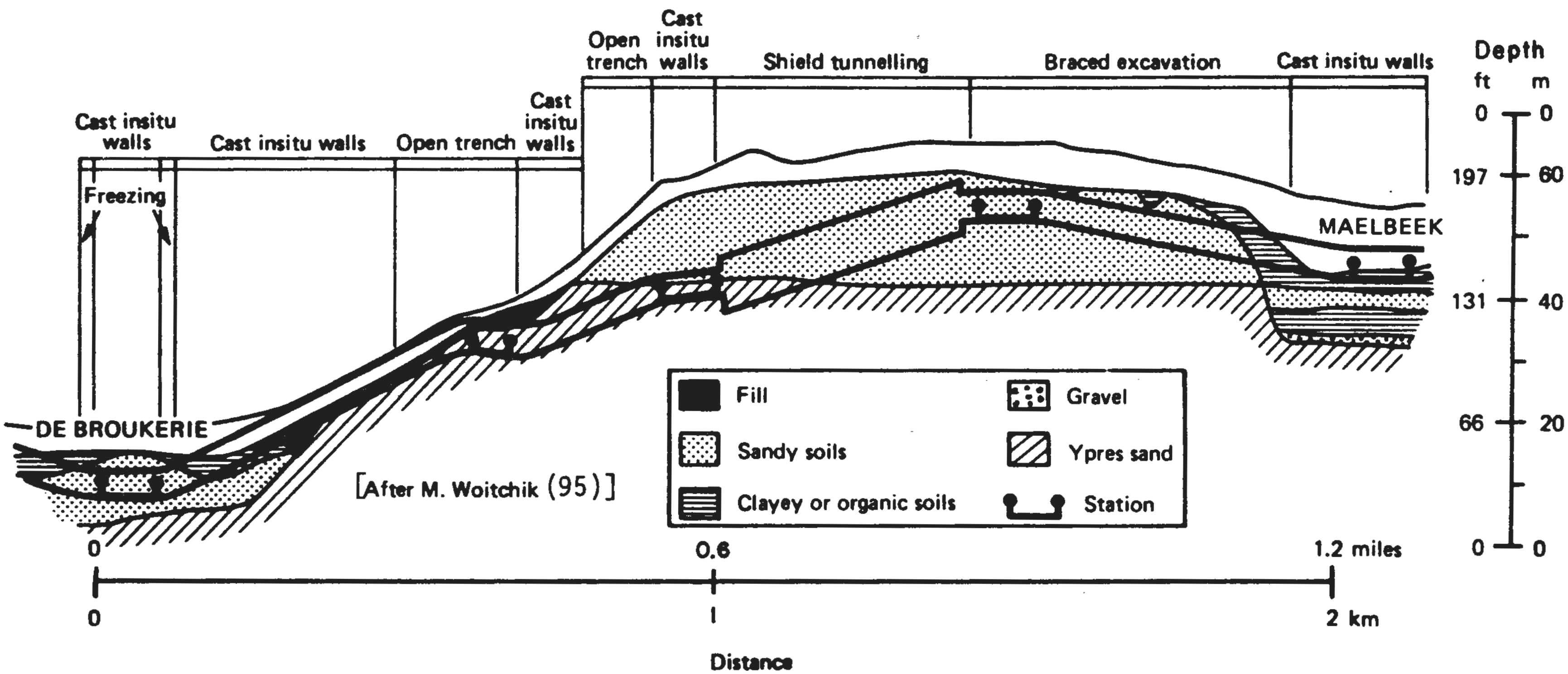


Figure IV-2 Profile View of Line 1, Brussels Metro

IV.3 Construction Methods

A comprehensive review of the construction methods used for the Brussels Metro has been provided by Woitchik (95). Most of the stations and running tunnels were established by means of braced cut methods that used sheet piles, hand-excavated or cast-insitu walls to support the sides of the excavations. Cast-insitu walls, ranging in thickness from 0.8 to 1.0 m (2.6 to 3.3 ft), have been particularly useful for working within the constraints imposed by the soil conditions and congested, city streets. The walls were used for both temporary and permanent support in the running tunnels. A shield driven tunnel [9.9 m (32.5 ft) diameter] was constructed for a section of Line 1 beneath Brussels Park, but large surface movements caused by the tunneling dissuaded engineers from using the method at other sections.

IV.4 Costs

The average costs per km (0.62 mi) associated with the construction and operating equipment of the system are summarized in Table IV-3. All costs reflect August 1976 prices. Conversion of Belgian francs to U.S. dollars has been made using the exchange rate of 0.03 U.S. dollar to 1.0 Belgian franc.

TABLE IV-3 COSTS FOR THE BRUSSELS METRO

Item	Average Cost per km (0.62 mi) in U.S. Dollars x 10^6
Surface Works	3.05
Construction of Stations	5.17
Construction of Running Tunnels	10.7
Additional Works: Electrification, Signaling, Track, etc.	7.5

Note: Costs reflect an average 123 m (403 ft) of station per 1 km (0.62 mi) of line. Information on costs by courtesy of the Brussels Metro Authority (STIB).

V THE STOCKHOLM UNDERGROUND, LINE 3

V.1 General Description

There are three lines which make up the Stockholm Underground, of which Line 3 is the most recently planned and constructed. Line 3 services the central business district of the city by way of the T-Centralen and Kungsträdgården Stations. The line bifurcates at Vastra Skogen Station and extends into the north-west suburban areas where it terminates at Hjulsta and Akalla Stations. A simplified plan view of the system is illustrated in Fig. IV-1 where the section of line from Kungsträdgården to Näckrosen Station is emphasized. The trains used and passenger capacity of the line are summarized in Table V-1. The dimensions and structural materials for the underground sections are summarized in Table V-2.

V.2 Geology and Soil Profile

The geology of Stockholm consists of glacial and post-glacial deposits overlying Precambrian bedrock. Figure V-2 shows a profile view of Line 3 from Kungsträdgården to Näckrosen Station. The soil cover is highly variable in depth. Two separate ground water levels are often observed: a perched water level in granular soils overlying clay and a water level in the granular soils underlying the clay.

The rock is principally gneiss, granite, or amphibolite. The gneiss and granite are competent and have good stability characteristics. The amphibolite occurs as bands or lenses in the gneiss and, often, is highly schistose. Faults, shears, and zones of fractured rock are encountered throughout the area.

TABLE V-1 TRAINS AND PASSENGER CAPACITY FOR LINE 3, STOCKHOLM UNDERGROUND

No. of cars per train:	8 or 10
Loading:	112 people/car (Standard)
*Passenger capacity: (for train interval of 4 minutes)	12,500 people/hr (Standard) 18,700 people/hr (Peak)

* Capacity for 8-car trains.

TABLE V-2 DIMENSIONS AND STRUCTURAL MATERIALS FOR LINE 3, STOCKHOLM UNDERGROUND

Running Tunnels
- single track: 4.3 m (14 ft) wide
  approx. 5 to 6 m (16.4 to 19.7 ft) high
- double track: 8.1 m (26.5 ft) wide
  approx. 5 to 6 m (16.4 to 19.7 ft) high
- lining: minimum 30 mm (1.2 in.)-thick shotcrete in roof with rock bolts as required
- reinforced concrete box in sections through soil or low rock cover

Stations
- length = 180 m (590 ft)
- central platform width = 8 to 10 m (26 to 33 ft) wide

Lining: reinforced shotcrete, 75 mm (3 in.) thick with rock bolts as required

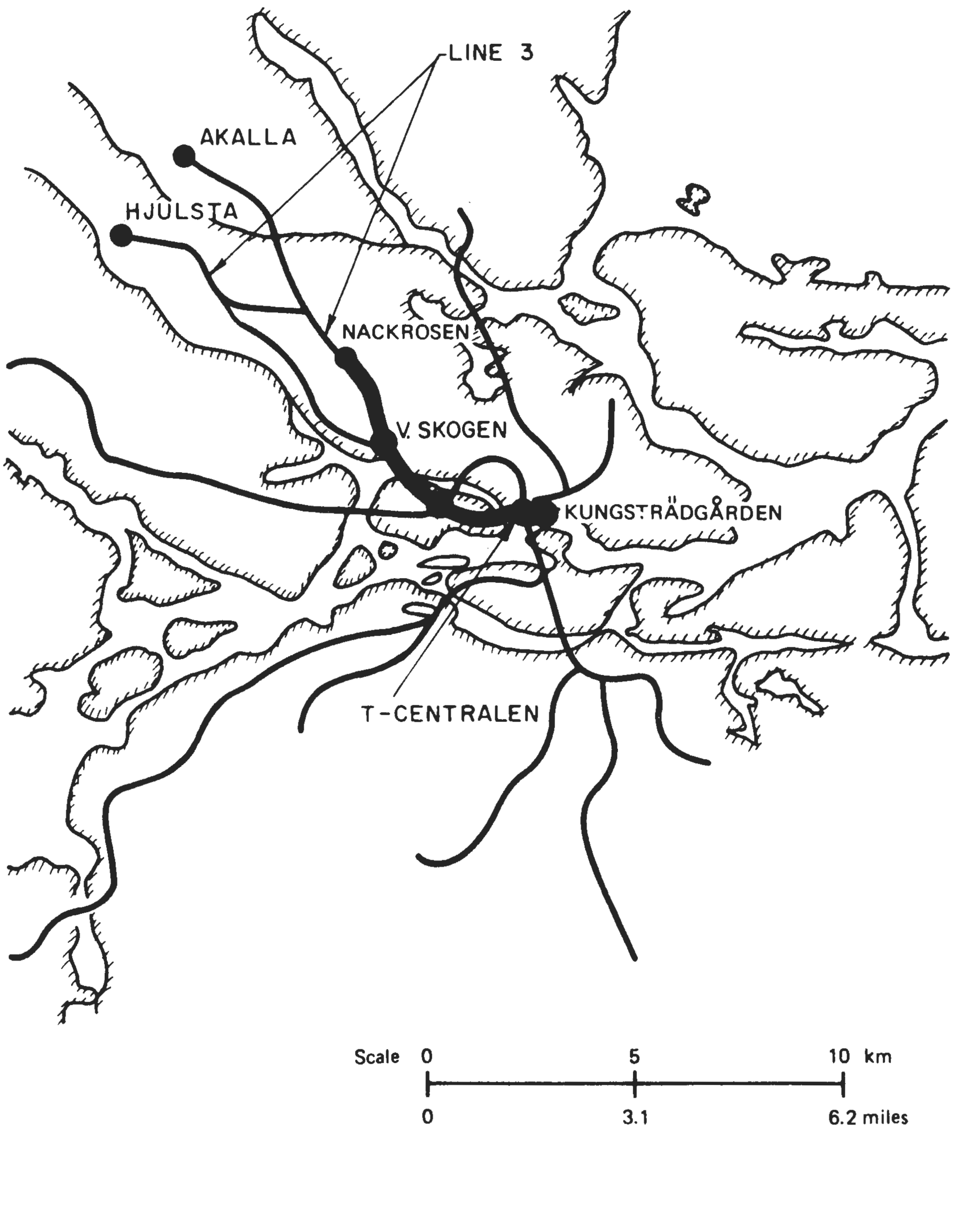


Figure V-1 Plan View of the Stockholm Underground

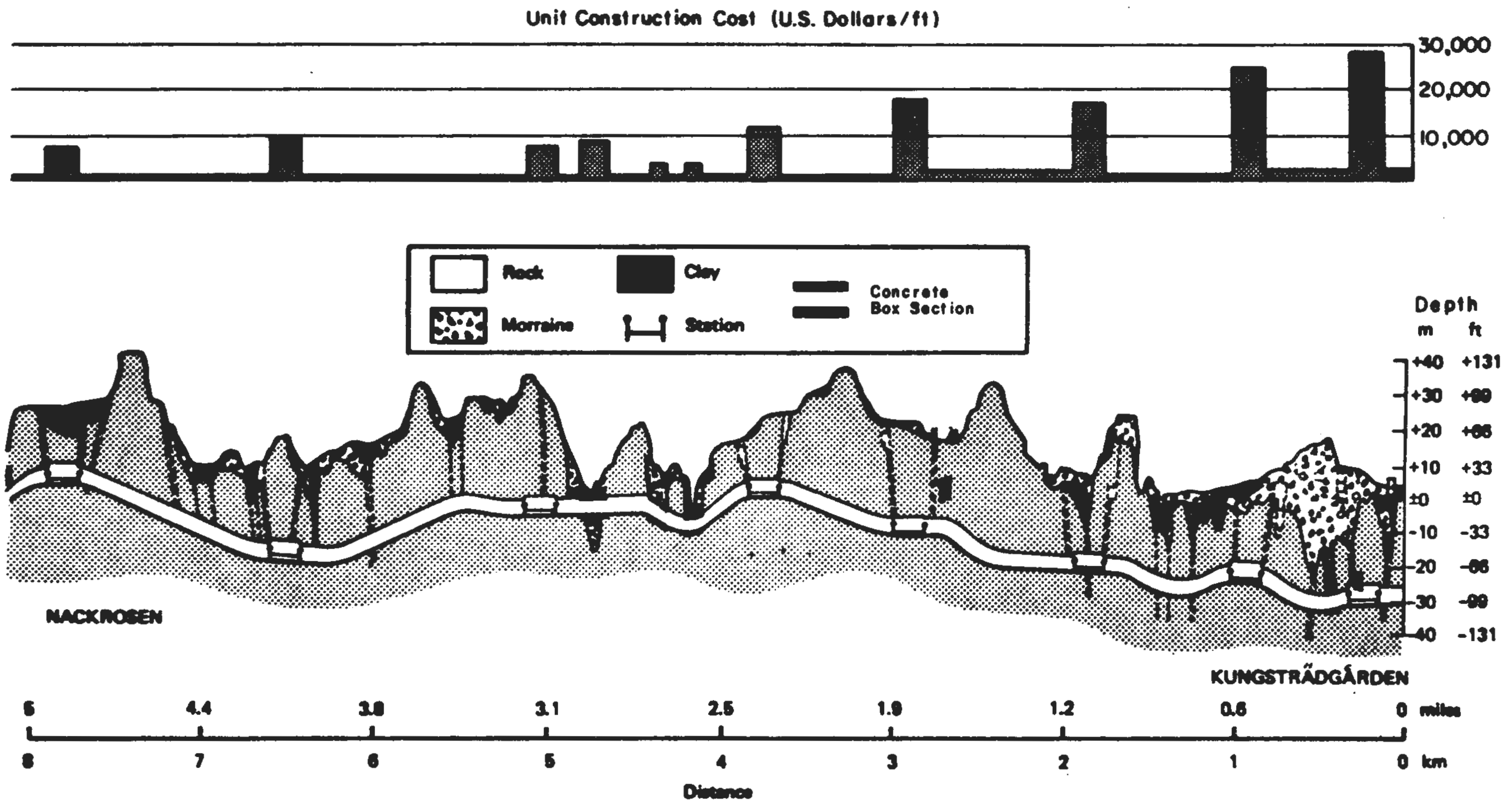


Figure V-2 Profile View of Line 3, Stockholm Underground

The glacial formations are moraines, clay, and eskers. The silt sized fractions of both the moraine and esker deposits are frost sensitive. Post glacial deposits are mostly marine or fluvial sediments occupying in-filled valleys or local depressions. The thickness of clay varies, being as thick as 25 m (82 ft) in some areas. The clay is often soft and highly compressible.

V.3 Construction Methods

A comprehensive treatment of various geotechnical problems and construction techniques has been made by Rosell et al (76). Excavation and support during construction was determined by rock quality, the presence of weathered rock or sediments, and the amount of rock cover overlying the tunnel.

Tunnels in rock were advanced with drill-and-blast methods. For single track tunnels in competent rock, the drill-and-blast rounds were approximately 3.0 m (9.9 ft) in length. Tunnel support was provided by a minimum 30 mm (1.2 in.)-thick, application of shotcrete in the crown, supplemented with rock bolts as required. Reinforced concrete box sections were placed with braced cut methods in sections through buried valleys. Stations in competent rock were supported with a minimum 75 mm (3 in.)-thick lining a reinforced shotcrete. In zones of low rock cover, such as the eastern exit of Kungsträdgården Station, the tunnel was mined in stages and its crown supported by 500 mm (20 in.)-thick, reinforced concrete arches. Pregrouting was performed for sections of the line passing through fractured rock or close to buried valleys.

so that difficulties with ground water lowering would be minimized or avoided.

V.4 Costs

Unit costs of construction for various sections of tunnel are summarized in Table V-3. In addition, costs per running foot of tunnel are plotted with respect to the tunnel profile from Kungsträdgården to Näckrosen Station in Fig. V-2. All costs reflect January, 1975 prices and have been converted to U.S. dollars using the exchange rate of 0.24 U.S. dollar to 1.0 Swedish krona.

TABLE V-3 COSTS FOR LINE 3, STOCKHOLM UNDERGROUND
[After Rosell, et al (76)]

Item	Construction Cost per ft in U.S. Dollars
double track running tunnel in rock	700 to 840
double track, concrete running tunnel	2100 to 3500
stations	2800 to 26600

VI THE HELSINKI METRO

VI.1 General Description

The first phase of the Helsinki Metro is composed of 4 km (2.5 mi) and 7.4 km (4.6 mi) of tunnel and surface line, respectively. Figure VI-1a shows a plan view of the tunnel line which includes 5 underground stations. The invert of the tunnels is at an average depth of 25 m (82 ft) below the ground surface. The trains used and design capacity of the system are summarized in Table VI-1. The dimensions and structural materials for the system are summarized in Table VI-2.

VI.2 Geology and Soil Profile

The geological profile of Helsinki consists of Quaternary soil deposits overlying Precambrian bedrock. Although the thickness of the soil layers varies between 0 and 60 m (197 ft), most of the soil is relatively shallow. Soft, sensitive clay occurs within buried valleys and accounts for 35 % of the soil cover.

The rock at Helsinki is primarily granite and migmatitic gneiss. At some locations, shear zones and prominent foliation are evident. The rock, in general, is massive and competent. Residual, horizontal stresses of approximately 13.9 MPa (2000 psi) have been measured at a depth of 15 to 20 m (49 to 66 ft) below ground surface.

A profile view of the tunnels from Sörnäinen to Kamppi Station is shown in Fig. VI.1b. The water table varies. At the Kluuvi Cleft, the water level is located 2 m (6.5 ft) below the ground surface.

TABLE VI-1 TRAINS AND PASSENGAR CAPACITY FOR THE HELSINKI METRO

No. of cars per train:	2 minimum 6 maximum	
Maximum loading:	880 people/train	(Standard)
*Passenger capacity: (for train interval of 4 minutes)	13,200 people/hr	(Standard)
	25,300 people/hr	(Peak)

*for 6-car train

TABLE VI-2 DIMENSIONS AND STRUCTURAL MATERIALS FOR THE HELSINKI METRO

Running Tunnels:		
twin tunnels	=	Arch-shaped 5.2 m (17 ft) high 5.5 m (18 ft) wide
stations	=	135 m (443 ft) long 18 m (59 ft) wide approx. 10 m (33 ft) high
lining:		150 to 300mm (6 to 12 in.)-thick shotcrete with wire mesh reinforcement

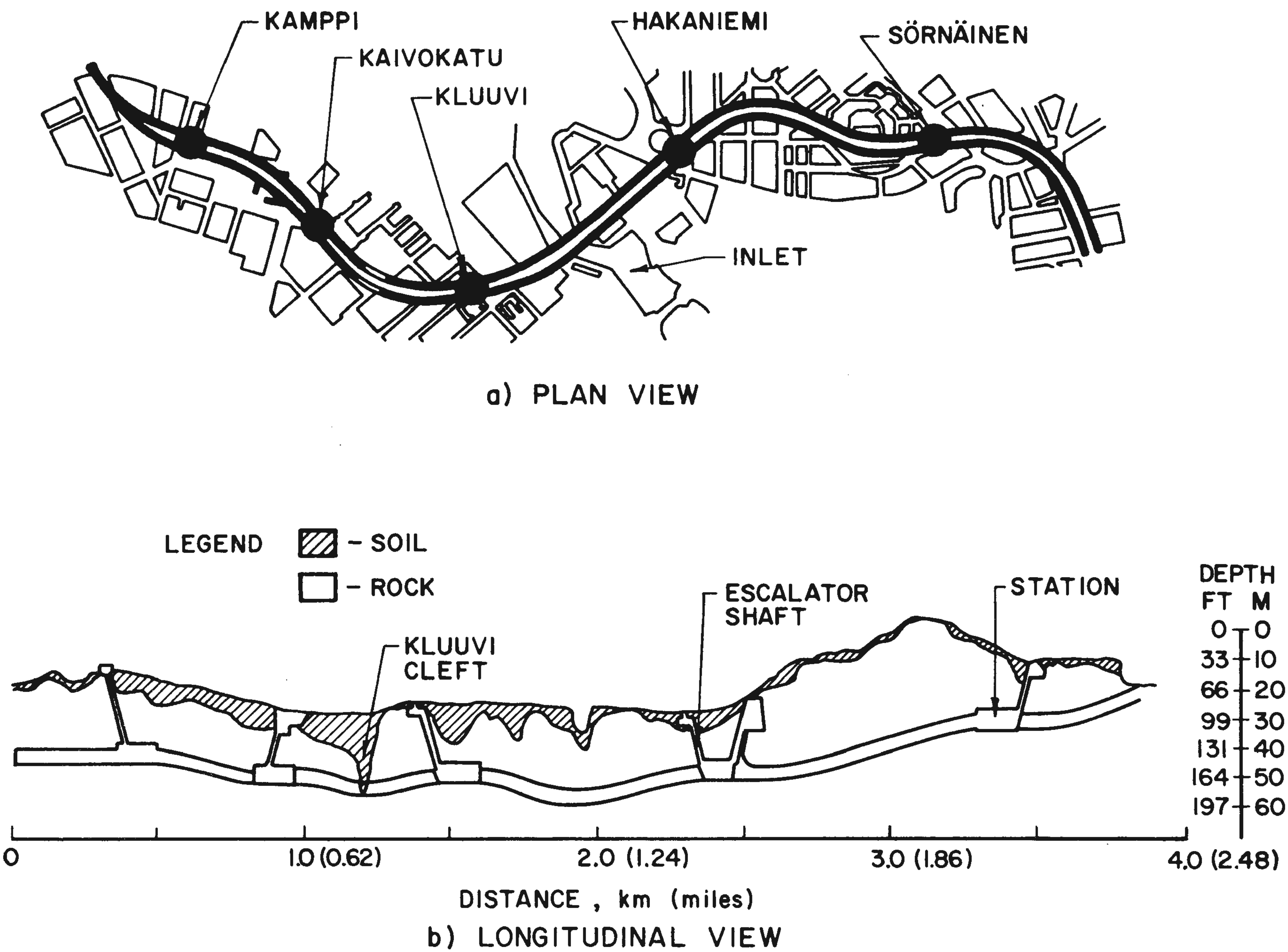


Figure VI-1 Plan and Profile Views of the Helsinki Metro

VI.3 Construction Methods

Twin tunnels in rock were advanced with drill-and-blast methods, by blasting rounds 2.5 to 3.5 m (8.2 to 11.5 ft) long. Support for the running tunnels was provided, where needed, with fully grouted rock bolts. In areas of low rock cover, pregrouting was performed to cut down on water infiltration.

Station excavation was performed by drilling and blasting in stages. At the Kamppi Station, rock pillars [approximately 6.5 x 6.9 m (21 x 22.5 ft) in plan] were left in place along the centerline of the station. Reinforced shotcrete of 150 and 200 mm (6 and 7.8 in.) thickness was applied to the roof and pillars, respectively. Up to 300 mm (12 in.) of shotcrete was placed at the pillar/roof contact areas. Rock bolts were used to strengthen the pillars and were placed in lengths of approximately 4 m (13 ft) across the roof.

VI.4 Costs

The costs for several items of underground construction are summarized in Table VI-3. All costs are quoted and adjusted to U.S. dollars according to 1976 prices.

TABLE VI-3 COSTS FOR HELSINKI METRO

Item	Cost in U.S. Dollars
*Total cost of 5 tunnel stations	68 million
Min. cost per tunnel station	7 million
Max. cost per tunnel station	18 million
Unit cost for twin tunnels (includes blasting and reinforcing)	$590/ft

* Approximately 20 % of the total station cost is related to escalators and the installation of ventilation, heating, waterpiping and electrical equipment.

Note: Information on costs by courtesy of Helsinkin Metrotoimisto

Appendix B

COST INFORMATION FOR U.S. METROS

B.1 Introduction

This appendix summarizes underground construction costs for two U.S. metro systems. The systems are the Washington, D.C. and Baltimore Metros. The costs pertain to construction primarily in soft ground conditions in the central business districts of both cities.

B.2 Underground Construction Costs for the Washington, D.C. Metro

Several projects have been selected to provide cost data for underground construction on the Washington, D.C. Metro. These projects are identified by contract number in Table B.1. The projects are part of the initial construction stages of the system. The original contract costs date from the time period 1969-1971. To provide an equitable comparison with other U.S. and European tunneling projects that were started at a later date, the original contract costs have been multiplied by an inflation factor. The inflation factor of 1.58 has been computed for the years 1970 to 1975 by averaging the increase in union wage rates for building trades and the rise in the Wholesale Price Index (86) for each category of construction machinery and equipment, concrete products, and steel mill products. All data on wage and price levels have been obtained from the U.S. Bureau of Labor Statistics (87). Additional cost adjustments have been made for claims approved as of January, 1977 to obtain a total construction cost for each project.

The projects included in Table B.1 were constructed by cut-and-cover methods with the exception of Contract 1A0021 where twin, 5.5 m (18 ft) internal diameter tunnels were driven. On average, the excavations for the

Table B.1 UNDERGROUND CONSTRUCTION COSTS FOR THE WASHINGTON, D.C. METRO

Contract	Original Contract Cost ($\times 10^6$ Dollars)	*Original Cost Adjusted for Inflation ($\times 10^6$ Dollars)	**Additional Cost ($\times 10^6$ Dollars)	Total Cost ($\times 10^6$ Dollars)	Distance of Double Track Line ($\times 10^3$ FT)
1A0011	38.1	60.2	9.6	69.8	2.67
1A0021	12.4	19.6	6.0	25.6	2.48
1A0031	23.2	36.7	4.2	40.9	1.84
1B0011	33.7	53.2	3.9	57.1	3.49
1B0021	12.3	19.4	0.5	19.9	2.40
1B0032	6.4	10.1	0.6	10.7	0.55

*Cost adjusted for inflation for period 1970-1975. Inflation adjustment based on averaging the rise in union wage rates for building trades and the increase in the Wholesale Price Index for each category of construction machinery and equipment, concrete products, and steel mill products. Cost data provided by U.S. Bureau of Labor Statistics (87).

**Additional cost for claims approved as of January, 1977.

projects were carried to depths of 15 to 18 m (50 to 60 ft) through terrace deposits of sands and interbedded, stiff clay that were located below the water table. The 6 projects listed include the construction of 5 stations, each with platform lengths of 183 m (600 ft). The internal dimensions of the box structure for the running tunnels are approximately 3.7 m (12 ft) in height and 9.2 m (30 ft) in width.

B.3 Underground Construction Costs for the Baltimore Metro

As indicated in Table B.2, 5 projects have been selected to provide cost data for underground construction on the Baltimore Metro. The projects are part of the initial construction stages of the system. The contract costs date from 1977.

Of the projects listed, Contracts NW-02-05 and NW-03-02 are twin tunnel [5.5 m (18 ft) internal diameter] sections of the system and the remaining contracts are cut-and-cover constructions. The invert depth for the projects is approximately 18 m (60 ft). The soils are mostly well consolidated sands with some silt and clay that are located below the water table. The 5 projects include the construction of 3 stations, each with platform lengths of 137 m (450 ft).

Table B.2 UNDERGROUND CONSTRUCTION COSTS FOR THE BALTIMORE METRO

Contract	*Original Contract Cost ($x10^6$ Dollars)	Distance of Double Track Line ($x10^3$ FT)
NW-01-06	32.3	1.18
NW-02-05	17.5	1.58
NW-02-06	27.9	0.79
NW-03-02	41.6	5.71
NW-03-03	13.4	1.04

*Contract cost as of 1977

☆U.S. GOVERNMENT PRINTING OFFICE: 1979 624-722/1125 1-3