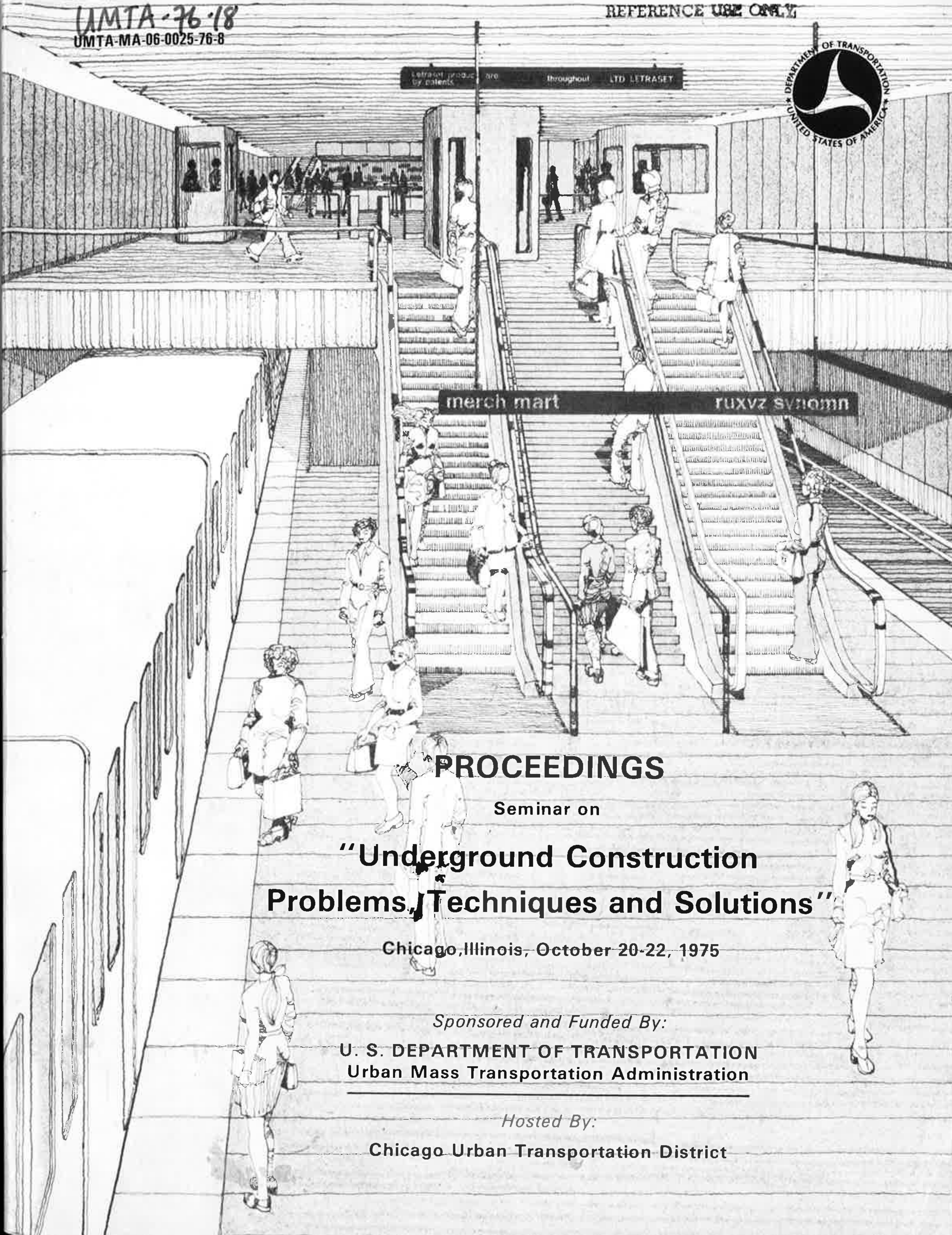

PROCEEDINGS

Seminar on

"Underground Construction Problems, Techniques and Solutions"

Chicago, Illinois, October 20-22, 1975

Sponsored and Funded By:

U. S. DEPARTMENT OF TRANSPORTATION
Urban Mass Transportation Administration

Hosted By:

Chicago Urban Transportation District

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16. Abstract

The seminar on "Construction Problems, Techniques and Solutions" held at the First Chicago Center in Chicago on October 20-22, 1975, was organized to focus on anticipated construction problems of the Chicago Central Area Transit Project to include underground construction techniques, new technology, ground engineering techniques (underpinning, dewatering, grouting), and involved an exchange of experiences among owners, design teams, contractors, and other pertinent agencies.

Because of the continuing requests for seminar materials 19 papers prepared for the seminar follow in their entirety. The authors are identified by their titles and associations as of October 1975. Additionally, a complete summary of the panel discussion held during the last afternoon of the seminar and moderated by Mr. Nelson, CUTD Executive Director, is furnished because of the pertinent views that were expressed therein.

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FOREWORD

The Urban Mass Transportation Administration (UMTA) is one of the participants in the Department of Transportation's Coordinated Tunneling Program. UMTA's assigned responsibilities in the tunneling program are carried out through the Urban Rail Supporting Technology Program Office at the Transportation Systems Center (TSC) Cambridge, Massachusetts, via in-house and contracted efforts. One of TSC's assigned responsibilities is the dissemination of new technology to transit properties, designers, contractors, planners and the tunneling and underground construction industry.

The seminar on "Construction Problems, Techniques and Solutions" held at the First Chicago Center, One First National Plaza, Chicago, Illinois 60670, on October 20-22, 1975, was organized to focus on anticipated construction problems of the Chicago Central Area Transit Project. An outstanding group of speakers and participants was assembled for the seminar to review underground construction techniques (underpinning, dewatering, grouting), and to exchange experiences among owners, design teams, contractors, and other pertinent agencies.

The seminar presentations by representatives of the U. S., France, England, and Japan were enthusiastically received by more than 250 engineers, contractors, and administrators who attended from all parts of the U. S. and Canada. Because the seminar developed into a valuable summary of the state-of-the-art of urban underground construction technology, because of the many comments received and continuing requests for seminar presentation material, this set of proceedings has been prepared.

The papers prepared for the seminar follow in their entirety, and the authors are identified by their titles and associations as of October 1975. Additionally, a complete summary of the panel discussion held during the last afternoon of the seminar and moderated by Harold E. Nelson, CUTD Executive Director, is furnished because of the pertinent views that were expressed therein.

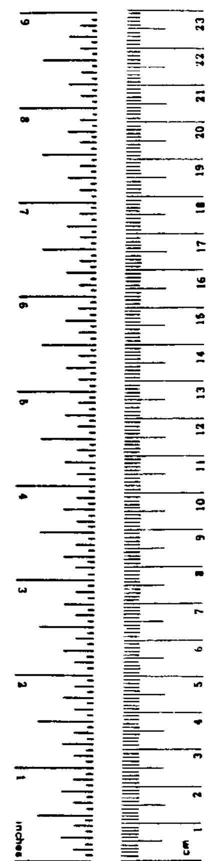
The three-day seminar was funded by UMTA through the Transportation Systems Center as part of its tunneling program. These proceedings were compiled by the Chicago Urban Transportation District with funding assistance from UMTA.

We would like to acknowledge the most valuable assistance and guidance of the many U. S. Department of Transportation employees who participated in the development of the seminar and the many employees of the Chicago Urban Transportation District and its consultants.

METRIC CONVERSION FACTORS

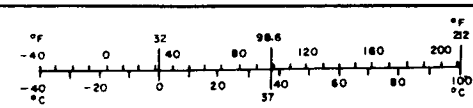
Approximate Conversions to Metric Measures

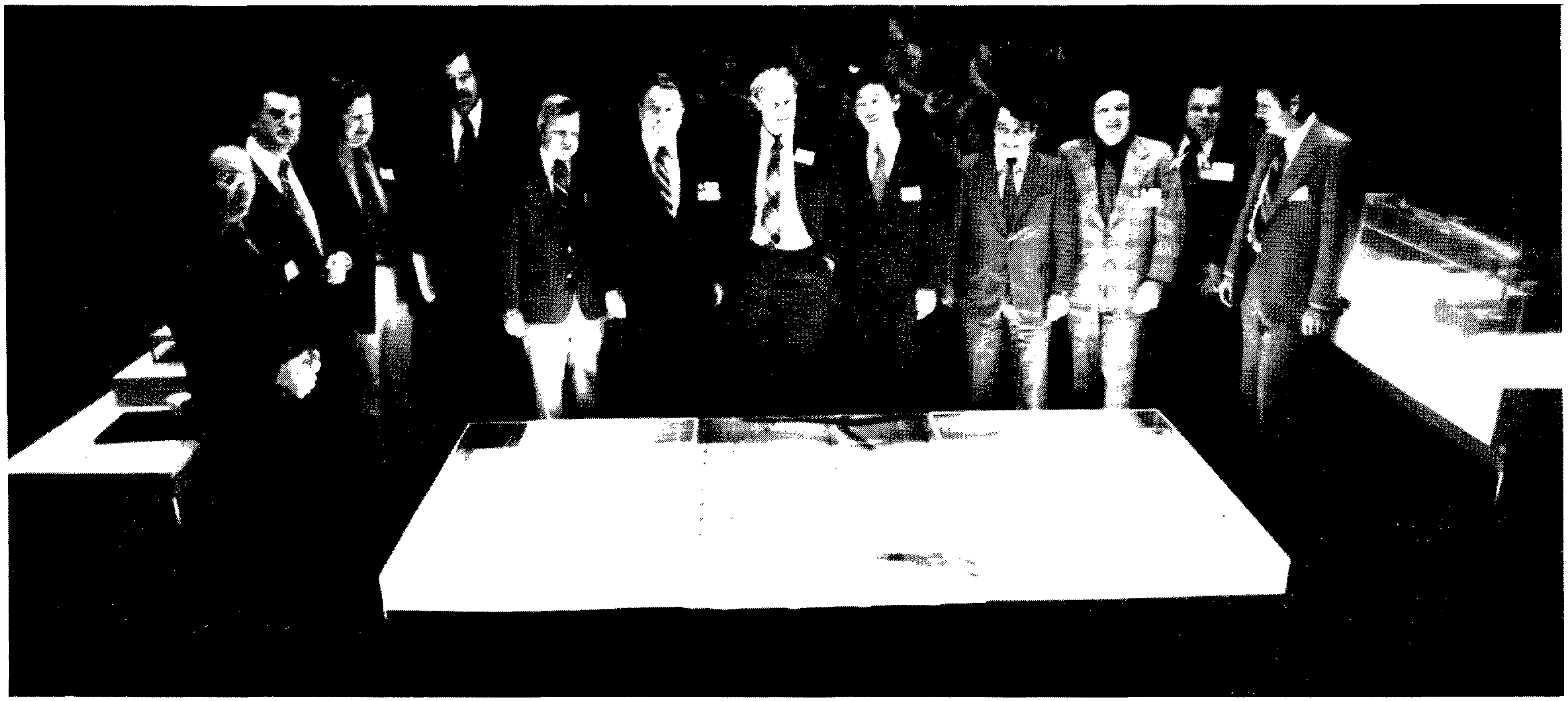
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2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



Approximate Conversions from Metric Measures

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





Group of seminar participants in model display area, Chicago Central Area Transit Project. Left to right: G. Tamaro, I.C.O.S. Corp. of America; A. L. Ressi di Cervia, I.C.O.S. Corp. of America; G. Tallard, Soletanche & Rodio, Inc.; R. K. McFarland, U.S. Department of Transportation; T. N. Harvey, Urban Mass Transportation Administration; H. E. Nelson, Chicago Urban Transportation District; D. J. Jobling, London Transport; T. Kato, Sumitomo Construction; J. F. Bougard, New Works Department, Paris; G. Randich, DeLeuw-Novick, Inc.; W. L. Barnes, Chicago Urban Transportation District, and G. L. Butler, Urban Mass Transportation Administration.

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Chicago Central Area Transit Project

Harold E. Nelson, P.E.
Executive Director
Chicago Urban Transportation District
Chicago, Illinois

PAPER I

THE CHICAGO CENTRAL AREA TRANSIT PROJECT

by

HAROLD E. NELSON

Executive Director
Chicago Urban Transportation District
Chicago, Illinois

Welcome!

The Board of Trustees and the Project Team join me in welcoming you to a discussion of underground transit construction in Chicago. In turn we are confident that you join us in expressing our appreciation to the U.S. Department of Transportation for sponsoring this timely seminar.

As we begin our program, we would like to briefly introduce you to the Chicago Central Area Transit Project. We invite your attention to the attached System Map, Project Background Information and Organization Chart as well as to the Chicago Urban Transportation District's Annual Report which have been included in your seminar kit. You will find further details on the technical aspects of the project in the pre-conference material.

It is also appropriate that before we begin our technical discussions we share with you the philosophy on which the District and the project are based.

We suggest that it become the philosophy of all those who would participate in underground transit construction in urban areas. Although the comments reflect the Chicago situation, they also apply to transit projects in other urban centers.

First of all, we believe in the future of urban America - that large cities are governable - and can be made economically viable and socially acceptable.

Society depends on cities to provide large masses of people with services and facilities they cannot provide for themselves.

All people, irrespective of levels of income, are concerned with certain basic needs:

Food
Shelter
Health
Culture
Recreation

Cities exist to assist people in meeting these basic needs and in seeking social and economic independence of these needs through:

Education
Jobs, and
Mobility; mobility to provide access to home, school, job - as well as service, cultural and recreational facilities.

Mobility, especially in the form of public transportation is basic to the entire relationship - a basic function of society, cities and people.

People are what public transportation is all about - people who are riders as well as people who operate this labor intensive industry. Mass transit is indeed "The People's People Mover."

Meeting that need is not a simple matter. Systems must be designed and built that are responsive, quiet, comfortable, secure and safe - that offer a pleasant interlude in the day's activities - systems that can be sold to new riders as well as the public in general - systems that will help change the basic attitude about public transportation.

Some change in public attitude is already evident, partially the result of higher gasoline prices - a condition that will intensify - and as a result, increase the number of transit dependents among the middle and lower income groups.

Thus, an added dimension emerges, that of ensuring sufficient capacity to meet this demand as it develops, which in turn is a function of the type of system utilized, time, technology and the availability of funds:

> Space limitations alone make it quite clear that the basic public transportation network of most large urban centers of 2 to 4,000,000 population should be underground, rail rapid transit. Such networks in turn will usually require intermediate and secondary supporting systems.

Time has already run out in providing adequate underground facilities in the nation's largest cities and will become critical as energy problems and EPA restrictions change the economics of the automobile alternative.

Technology in rapid transit design and construction is lagging behind demand. Noise abatement, train control, tunneling, contracting procedures, management of design and construction, energy consumption, joint development and marketing require immediate attention. Only a concentrated effort in basic and applied research can fill this void.

Funding is also critical because large cities are unable to finance completely the capital and operating costs of public transportation systems. This will continue until national priorities and policies reflect that public transportation is essential to the survival of urban society.

Thus, the District, the consultants and the contractors on this project will be involved not only in a complex technical undertaking but one of major socio-economic significance, one that can have a profound effect on the city, its citizens, and in turn on the entire region.

Only through the dedicated efforts of a well organized and cooperative partnership made up of the Project Team, design consultants, contractors, and the several private and governmental agencies involved can the ultimate objective be achieved.

The following areas will be of concern to those participating in the project:

Responsiveness: The client on this project is the public and they alone will be the judge of our performance. They already have a billion revenue dollars invested in the CTA system and we must augment that system with a project that will not only satisfy the needs of present riders but also attract new riders. We will be operating in the very heart of the city and we must ensure that interferences and inconveniences are held to a minimum.

Economy: We must insist on economy of design and construction. An old design captain once reminded me that "Gold Bricks Gold Plate" and indeed much more effort is required to design and construct an economical project that at the same time meets all other criteria. We are not only interested in economy of money, materials and manpower, but economy of time, especially as it applies to the cost of loss of time to service. And finally, the system must be constructed with an absolute minimum of interference with the business and social tempo of the inner city as well as provide the basic collection and distribution system for the regional network.

Risk and Responsibility: Much of the success of the project will depend on the willingness to accept responsibility and the ability to identify and distribute risk. Even with sound contracts and strict adherence by the parties involved, serious problems will arise due largely to non-contractual involvements. Good contracts are essential but contracts alone won't necessarily result in a good job. Only through careful planning, complete communications and prudent management on a total project basis can such problems be overcome.

Quality: We want to build the best system possible within our means and we want to take advantage of the latest state of the art - within reason. We will accept new and innovative procedures providing they have been either proven in the field or can be fully tested in time to meet the project schedule. We must insist on a systems approach to control of design and construction. Traditional control through architectural and engineering disciplines must be augmented with the functional system components of responsiveness, economy, security, safety, human factors, constructability, quality, reliability and maintainability as well as interface and configuration management. Fully engineered construction is a concomitant requirement.

Management: We recognize the value of money - that time is money. We also recognize the frustrations faced by those who do business with governmental agencies - delays, changes, late payments, lack of decisions, and unrealistic restraints. We will be dealing with a multiplicity of local, state and federal agencies, countless property owners and some 3,000,000 people of widely diversified interests and concerns. Nevertheless, these normal frustrations can be sorted out - and to avoid the usual crises-oriented approach, we plan to hold a series of management seminars on various aspects of the project including project management, contract administration, joint development and public participation as well as technical seminars on design, logistics, procurement and construction.

Key members of the Project Team, including the District staff, our Supervising Consulting Architects and Engineers and our Board of Senior Consultants are here and look forward to visiting with you. Representatives of the federal, state and local agencies involved in the project are also attending the conference. Most of the participants represent consultants and contractors who might later be involved in the project. The success of the conference will depend on the extent to which all of those present participate in the discussions following each presentation.

As in most projects of this magnitude, many, if not most of us attending this seminar will not be around when the project is finished. Nevertheless, our efforts at the critical beginning of the project will have a profound impact on the ultimate success of the project. Hopefully, those who follow will accept our efforts as a job well done.

Again, we welcome you - we are glad you are here - and we look forward to a good exchange of ideas.

PAPER 2

The Baltimore Area Transit Project

Frank Hoppe, P.E.
Director of Engineering and Construction
Mass Transit Administration
Maryland Department of Transportation

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PAPER 2

THE BALTIMORE REGION RAPID TRANSIT SYSTEM

by

FRANK HOPPE, P.E.

Director of Engineering & Construction
Mass Transit Administration
Baltimore, Maryland

Ladies and gentlemen, I would like to thank the Urban Mass Transportation Administration and the Chicago Urban Transportation District for this opportunity to describe some aspects of the proposed Baltimore System to this distinguished gathering.

The PHASE I System consists of two legs-each approximately 14 miles long. The two lines cross at the center of the Downtown Business District, at a major transfer station called Charles Center, where a vertical transfer of passengers is necessary to transfer from one line to the other. These and all future proposed lines will be grade separated and a vertical transfer will be required. There are to be 21 passenger stations, nine of which are sub-surface and 12 are aerial or at grade. The System is approximately 1/3 underground, 1/3 aerial and 1/3 at grade. There is to be a direct subway station entrance to Baltimore-Washington International Airport Terminal Building.

The System is steel-wheel/steel-rail D.C. traction power (of 750V nominal third rail pick up). There will be about 24 traction power substations converting 13.2 KV A.C. supplied by Baltimore Gas Electric Company to our desired traction voltage. The vehicles are to be 75 ft. long and in married pairs. The platforms will be 450 ft. long to accept the ultimate six-car train. Opening in 1981-1982, planned operations will consist of four-car trains on four minute headway. The ultimate capacity of the system is estimated to be some 29,000 one-way passenger per peak hour with six-car trains on two-minute headways. It is estimated that this peak capacity will not be obtained until well beyond the year 2000. Projections this far in the future, although very popular, are nebulous to say the least. Urban transportation could be at that time by telepathy, or automobile densities may force a mode exclusively by pedestrianism.

The vehicles are to be air-conditioned. The stations will be air-cooled to maintain outside ambient temperature.

Public hearings for Capital Grant Application have been held on 14 miles of the PHASE I System and a Capital Grant has been applied for and received for eight miles. This eight miles is estimated to cost some $600 million for a complete operational system. At present, all eight miles are under final design, and the aggregate percentage of completion is about 40%. One soft-ground tunnel section is out on the street now for bidding, and other tunnel sections are beyond the 65% review stage. Pending a review of transportation priorities by the Maryland Department of Transportation and go ahead from this, our parent organization, we will be in full construction during the spring and summer of 1976.

The MTA is one of five modal Administrations that make up the Maryland Department of Transportation. Other sister agencies are the State Highway Administration, Maryland Port Administration, Motor Vehicles Administration, and the State Aviation Administration. The funds for local participation for all capital and operating programs for all modal Administrations are allocated from a common source--The Consolidated Transportation Fund. Thus all revenue from all Administrations are consolidated into one fund, and quoting Secretary of Transportation Hughes, "None of the Consolidated Transportation Funds is sacrosanct".

The Mass Transit Administration has three major functions, and they are: 1) Public Transportation Development--the Division that aids other areas of Maryland in their Urban Transportation needs, studies and Federal applications; 2) Metropolitan Transportation--the Division charged with operations and maintenance of our existing 900 plus bus fleet, and 3) Rapid Transit Development Division--responsible for design and construction of the rapid rail system.

In areas served by the two proposed rapid rail lines, the present extensive bus system will be integrated and revised to act more as a feeder system than as at present a line-haul system.

After this general overview of our Baltimore programs, let's get to some engineering specifics.

Out of the first eight miles, some five miles will be subsurface along with six passenger stations. The passenger stations are proposed to be constructed by the cut-and-cover method, along with adjacent vent shafts and mid-line vent shafts. The remainign portion of this five miles will be constructed by tunneling methods, thereby minimizing surface disruption. This section of line is approximately 50% rock tunneling and 50% soft ground. In the soft ground area, our boring program indicates a predominance of sedimentary sand and gravel with lenses of silty sands with traces of clay. These strata are sediments of the cretaceous age which have been highly preconsolidated and are generally very dense and hard. The water table in this section is approximately 30 to 35 ft. below the surface which is approximately crown line to 15 feet above crown. Pump tests in the vicinity have indicated that dewatering will have to be carefully designed in order to be effective in these soils.

Below these sedimentary soils are materials residual in nature having been formed through the decomposition and weathering of the parent formations. We have subdivided these residual soils, as recommended by the United States National Committee on tunneling technology into classifications of RS, RZ-1 and RZ-2 in order to assist the bidder in evaluating the characteristics of rock-like material more definitely.

RS Material having been formed from either in-situ decomposition of the parent rock or the reworking of residual soils. This material is basically soil-like and does not exhibit visible remanent rock structure. It may contain rock fragments but they are usually friable and small.

RZ-1: These materials are considered as transitional between the residual soil and the underlying RZ-2, on rock, although they do occur in certain instances immediately below the deposited soils. They have been derived in-situ from the decomposition of the parent formation and consist of soil-like components and partially weathered and/or fresh rock-like components. Visible remnant rock structure is usually apparent in these materials, and they have a cohesive-like strength because of their origination.

RZ-2: This zone is basically comprised of partially weathered or fresh rock components commonly including a soil-like matrix or filler. This material is less weathered than the RZ-1 material. It usually requires rock sampling techniques, but can sometimes be penetrated with soil boring equipment. This material will probably have to be removed with rock excavating techniques.

Rock: The rock is composed of the Baltimore gneiss with some local pegmatite intrusions. The gneiss exhibits numerous highly broken and jointed zones with clay as filling materials. Thin quartz and pegmatite lenses occur in some samples, seldom exceeding 3" in width. Unconfined compression tests performed in this formation indicate values ranging from 400 to 2100 KSF, depending primarily on the minerology of the test specimen.

The majority of the route in this underground section is beneath city streets in built-up areas either commercial or by the typical red brick, marble stone stairs of Baltimore row-house 3 to 4 stories tall. (Structural settlement may be expected along the soft-ground route, but not in the magnitude to create structural damage.)

Any building damage resulting from tunneling operations are, by the newspapers and public, the MTA's fault. Rarely does one see a newspaper article where XYZ construction company's tunnel project created such and such. It is always the Mass Transit tunnel or the State Highway's project, etc.

The Section Designer, at MTA'a direction, has specified building protection on selected structures only. All other structures within a zone of influence will be repaired at MTA's direction. I think we have taken a novel approach to this matter by relieving the construction contractor of the crystal ball gazing as to how many other buildings may be damaged and how much should be put into his bid for this work. Our approach is to set a pre-determined lump sum in the bid documents for this cosmetic repair. Upon the confirmation of actual damage by our pre-construction surveyor the MTA will instruct the contractor to seek competitive sub-contracts to perform the repair work. Small minority firms will be encouraged to participate in this bidding process. The cost of this sub-contract will be deducted from the previous set amount plus 5% for administrative fee to the prime contractor. Some of the advantages we feel of this approach are:

1. Ability to quickly react to the public.
2. Open another door for minority participation.
3. The 5% administrative fee is not so attractive that it will encourage poor tunneling techniques.
4. The removal of a contingency bid item from the bid document.

Naturally, any unexpended funds of this item will revert to the MTA. In the case of emergency, force account action by the General Contractor will be utilized.

In addition to the adjacent buildings as a potential problem, we in Baltimore have been blessed with our subway having to pass under two existing operating and one future railroad tunnel. Coordination of design, effect and agreement with a railroad is to say the least, time consuming. Although effecting a full agreement has been and will continue to be a delicate subject, I am sure it will come to pass. In these thirty minutes of allotted time, this problem need not be dwelled upon any more.

During the six years of construction for this, the largest single public works project in the history of the State of Maryland, the public will be inconvenienced. Neighborhoods will be disrupted by construction equipment and noise; commercial business may suffer diminishing trade; traffic will be detoured; streets will be restricted in flow, but we at the MTA are attempting to coordinate and communicate our construction program with a myriad of agencies, organizations and community groups. The white hat that transit is wearing today will certainly look grayer in Baltimore before MTA is finished with its program.

Thank you for inviting me to be here in Chicago today, and thank you for your attention.

PAPER 3

Subsurface Environment in Chicago

J.O. Osterberg
Walter P. Murphy Professor of
Civil Engineering
Northwestern University
Evanston, Illinois

PAPER 3

SUBSURFACE ENVIRONMENT IN CHICAGO

by

J. O. OSTERBERG

Walter P. Murphy Professor of Civil Engineering
Northwestern University
Evanston, Illinois

INTRODUCTION

"Subsurface environment" is hardly a sufficient descriptive title for this paper. If there were no limit to the length of a title the paper might better be described as "Geological and human history of the central Chicago area as applied to soil problems in deep foundations and tunneling and how it relates to the technical, human, legal and political problems in construction of subways in the central Chicago area in the light of recent developments in construction techniques and methods." Implied in this long-winded title is that it is insufficient to simply describe the subsurface soil conditions, to discuss possible construction problems, and to consider construction methods. In view of past experience it is imperative that human, legal, political and practical problems also be considered because in so many instances it was these considerations that determined the success or failure of an excavation. The history of damage to adjacent structures due to excavation and tunneling in the Chicago area is not a good one, and when it is realized that in most cases of damage there was nothing wrong with the engineering aspects of the shoring and bracing methods used, but rather in the sequence and manner in which the work was done, or, as in many cases, due to a lack of proper measurements, or, as in other cases, due to a lack of proper communication among the parties concerned. Therefore it is just as important to discuss these human and seemingly non-technical problems that contribute to settlements, damage and legal action as it is the soil properties, anticipated technical construction problems, and foundation methods.

GEOLOGICAL HISTORY OF THE CHICAGO AREA

The bedrock under the Chicago area consists almost entirely of limestone from the Silurian Age, deposited some 300 million years ago. The limestone was formed in a shallow large inland sea to depths of up to 500 feet. The upper part of the limestone, known as the Niagaran formation, is a dolomite whose surface is only slightly weathered. It appears that after it was deposited the sea disappeared, and there was a lowering of the water table which formed solution channels and some minor weathering of the rock surface. The Niagaran limestone was covered with a shale deposit of the Devonian Age. The thickness of this deposit is not known since virtually all of it was removed from the Chicago area during the Pleistocene Ice Age. Only traces of the shale exist in low areas and fissures in the underlying limestone. The Pleistocene Ice Age began about a million years ago and was characterized by four major ice advances and retreats. The ice sheets from all four ages crossed over and covered the Chicago area, but each succeeding ice sheet removed virtually all the deposits of the previous sheets. Thus, almost all the soils deposited over the bedrock in the Chicago area consist of deposits laid down as a result of the ice movements in the last of the ice ages, known as the Wisconsin. During the Wisconsin Age many ice sheets advanced and retreated, in some cases removing completely the deposits of one or more previous ice advances. Thus soil deposits from the many advances or substages during the Wisconsin have been identified, but they are not found everywhere, and only some of the deposits from the various Wisconsin substages are found in the Chicago area. The last of the ice retreated only some 10,000 years ago.

Deposits directly from underneath the ice are known as tills and consist of sands, silts, gravel and boulders which were incorporated into the ice as it advanced and then dropped at the bottom as it melted. Many mineral types are represented, since the ice sheets moved over large distances plucking up rocks and soils as they advanced. However, the most common mineral found is illite which comes from the underlying and nearby Devonian shale which was picked up and largely ground to silt and clay sizes by the ice movement. Many small pieces of pebble size are frequently found in the soft clay and silt. As each ice sheet retreated, large fluvial and outwash deposits were dumped in front of the ice. In the Chicago area the gentle eastward dip of the limestone and the presence of an early moraine deposit to the west of Chicago formed a barrier to the drainage of water to the west and south. Since the glacial ice advanced from the north and east and retreated north and east, lakes were formed as the front melted and retreated. Deposits of silt and clay, largely illite from the Devonian shale, were left in these lakes before the ice completely melted and retreated, allowing the lakes to drain. Subsequent advances removed some of the lake deposits, but some remain. Over these lakes or lacustrine deposits later ice advances deposited tills. The last ice sheet to cover the Chicago area blocked the water as it retreated so that a lake known as Lake Chicago was formed at a level which reached 55 feet higher than the present Lake Michigan. The bottom of this lake, only slightly above the level of Lake Michigan, formed the land surface when Chicago was first settled. Because of the poor drainage and frequent flooding, the downtown level was raised over the years beginning in 1855 by filling with sand and debris from buildings, and was finally completed by filling in with the debris from the great Chicago fire of 1871. The present level of downtown Chicago is about 15 feet above the original natural level and some 15 to 20 feet above Chicago City datum which is the average lake level of 579.94 feet above mean sea level.

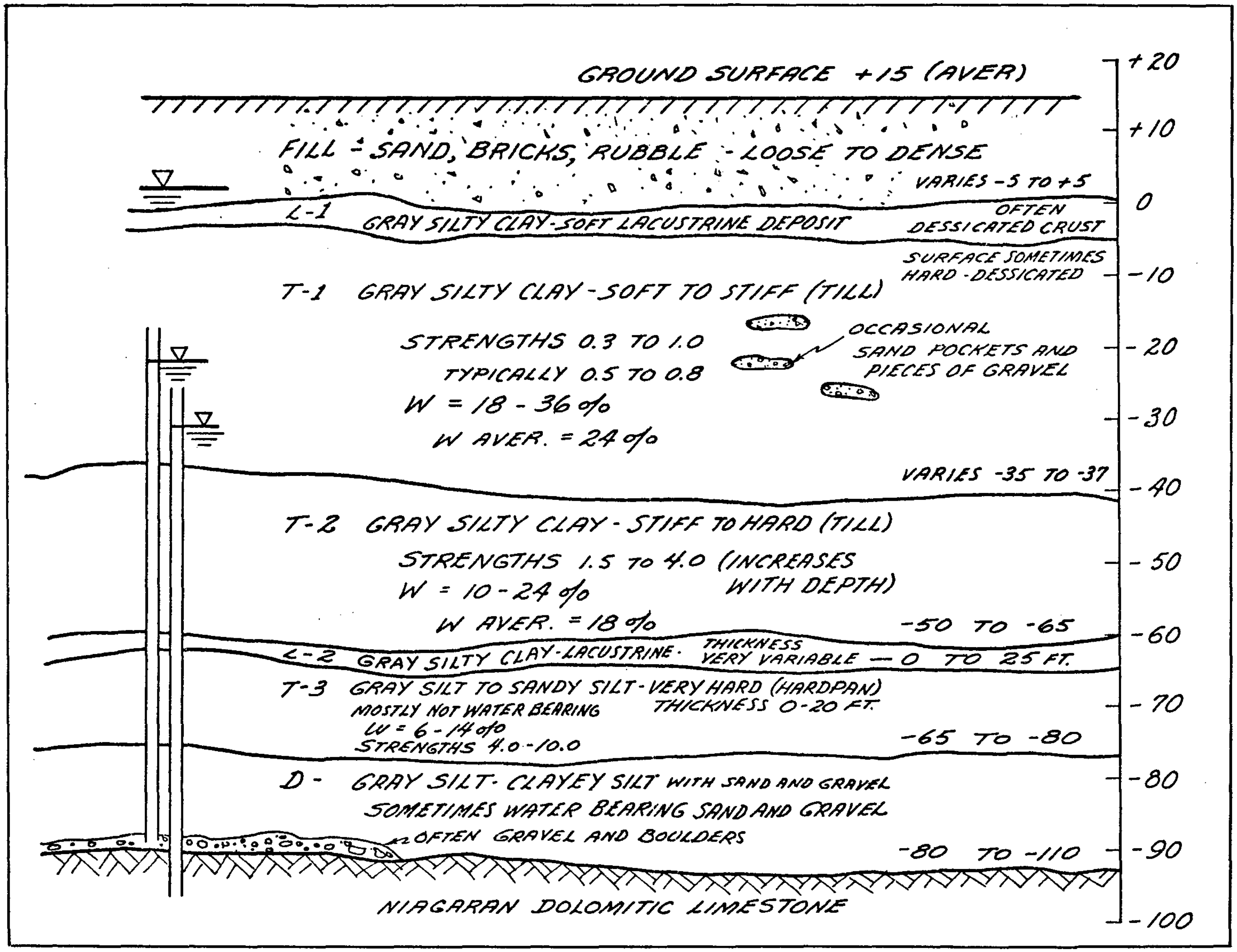
SOIL CONDITIONS IN THE CENTRAL CHICAGO AREA

Figure 1 represents average and typical soil conditions in the central Chicago area. The upper fifteen feet consists of man-made fill containing sand, bricks and rubble from the Chicago fire, old foundation masonry, wood, cinders, glass and concrete. It is extremely variable in character and can be very loose in some areas and dense in others.

Layer L-1 is a silty clay and clayey silt lacustrine deposit of quite variable water content (from 18 to 40%) which is quite thin to non-existent in most areas and up to 15 feet or more in thickness in other areas. It is soft, usually very silty, with an average unconfined strength of about 0.5 tsf.

Layer T-1 is the upper formation which ranges in thickness between 20 and 45 feet. It is a gray silty clay quite uniform in water content and strength and is generally referred to as the soft Chicago clay. It has an average strength of about 0.7 tsf. and a water content of 24%. It consists mostly of illite and frequently has more silt than clay. It contains occasional lenses of water bearing sand which are not interconnected. When encountered in construction, these layers bleed water but soon dry up. In some areas the top of this till is dessicated with higher strength and somewhat lower water contents. Where the dessicated crust exists it is rather thin (up to 15 feet) and brownish gray in color. However the crust is absent in most of the central Chicago area and may have been removed by natural erosion or by excavations or may never have formed in the first place.

Layer T-2 is a till formation very similar in grain size and mineral composition to the till above T-1 and sometimes cannot be distinguished from it. However the water contents are generally lower, not as uniform as in T-1, and average about 18%. It varies in thickness from 12 to 28 feet and the strength increases with depth with an average strength of about 2.5 tsf. and up to 4.0 and sometimes higher near the bottom. Layers and lenses of sand and gravel are more common and extensive than in T-1 and sometimes are very troublesome in construction.



TYPICAL SOIL PROFILE - CHICAGO CENTRAL AREA

FIGURE 1

Layer L-2 is a lacustrine deposit which exists over most of the area and can have thicknesses up to 25 feet. Water contents are generally a little higher than the till above and the strengths somewhat lower (w = 23%, strength = 2.0 tsf.). There is little or no evidence of dessication of the top of this layer and it appears that much of the original deposit has been removed from above the present top of the layer by till sheets.

Layer T-3 is a very dense and hard, mostly uniform, silt with little clay and low water content (6-14%) and high unconfined strength. It is often loosely referred to locally as "hardpan". It is mostly not water bearing and has strengths up to 10 tsf. with an average of about 6.0 tsf. It is almost non-plastic and very quickly and readily softens to a wet mud consistency on exposure to free water. In its natural state it appears quite dry due to its low water content even though it is saturated. It is believed that hundreds if not thousands of feet of ice covered this layer, making it highly over consolidated and of low compressibility. It is on this layer that most hardpan caissons are constructed. When no water is encountered, construction is easy, quick and economical.

Layer D is a glacial drift deposit mostly lacustrine but in some places outwash, which can be laminated silts to silty sands and in some areas coarse sand with gravel and boulders. Usually the bottom part overlying bedrock can be water bearing under artesian pressure. The water bearing, coarse portion may be from a few feet up to more than 20 feet in thickness.

When the water bearing layers are encountered in drilling shafts to rock, they frequently cause loss of ground and considerable difficulty for the contractor.

Bedrock, which is found under Layer D, is the Niagaran dolomitic limestone which is light gray to tan in color and fractured in the upper part. In some areas the rock is quite sound and hard with lack of fractures and in others many feet near the top are badly fractured and filled with soft clay and silt in the fractures. Remnants of reef rock exist in scattered areas. The reef rock is generally very porous. Fractured pervious rock and the very porous reef rock can cause considerable difficulty in excavating to sound rock. Water flow is so large that it is hardly controllable by pumping.

Water conditions at different levels are very important for construction considerations and are often not taken into account. The upper water level in the fill and sand above the clay is at or somewhat above lake level or about 10 to 15 feet below the surface. This water level may be considered as perched because it has no connection with the lower water in the sands and gravels above the rock and the water in the limestone. The 60 feet or so of clay between the upper fill and the lower sand and gravel effectively seals off the lower water. However, when observation wells are installed into the sand and gravel just above the rock, the water level will rise some fifty feet to elevations -20 to -40. Generally wells installed in the upper part of the limestone and sealed off from the sand and gravel above show water levels about the same as in the sand and gravel, but sometimes it is 10 or more feet lower and on rare occasions somewhat higher. The importance of the water levels in deep excavations and tunneling is that the upper water should be effectively sealed off in the excavation and measures taken so that there is no bottom blow-up due to the artesian pressure when excavation and/or tunneling gets close to the previous deposit under water pressure.

An often neglected phase of soil exploration is the information of accurate water levels in the lower sand and gravel and in the rock. Adequate numbers of wells which are properly sealed off from all layers other than the one in which the head is desired are essential for reliable water level information.

EXPERIENCE WITH PAST DEEP FOUNDATION EXCAVATIONS AND TUNNELING

Past experiences with deep excavations in the Chicago central area are not good. While there have been few outright failures of sheeting, bracing and other support systems, the vast majority have been associated with large lateral movements, causing nearby ground settlements, resulting in damage to buildings, streets and utilities. In fact over the past 20 years it has been unusual to find a deep excavation job that did not result in some damage. Damage was caused not only by movements of sheeting or the supports for the bracing. Damage has been caused by losing sand in the installation of wooden lagging in soldier beam and lagging supports, in plastic flow of clay in the process of installing lagging, and in improper excavation which left voids behind lagging into which soft clay could squeeze. Damage has been caused by soft clay squeezing in under excavations. Damage has been caused by excessive pumping of water from excavations. Damage has been caused by excessive pumping of water from excavations which removed large amounts of fines with the water and/or by consolidation settlement in adjacent structures due to lowering of the groundwater table. Damage has been caused by loss of ground in drilling caissons and digging shafts.

In most cases damage could have been avoided or minimized if movements had been detected early and proper remedies made. But in so many of the cases few or no measurements were taken so that small movements were not detected, and it was not until visual damage to buildings and other structures was obvious that there was any realization that movements had occurred. Also in many cases lack of measurements made it difficult if not impossible to track down the exact cause of the damage. After-the-fact analyses of these cases are made difficult because of lack of information concerning the initial positions of the sheeting, braces and the damaged structures. Total and relative movements of the bracing system and the structure are impossible to determine since the initial zero or starting point had not been determined. Thus in so many instances it is impossible to determine the exact cause. What is needed is an effective "early warning" system which will alert us to small movements so that they can be followed and corrected if necessary to avoid damage to adjacent structures.

Large movements resulting in settlement damage to structures are sometimes due to poor workmanship. Lack of proper installation of wood lagging is often an example of poor workmanship. Omission and sometimes removal of seemingly minor structural details on the bracing system by workmen have resulted in damage. Loss of ground in excavating may also be due at least in part to poor workmanship. Lack of proper inspection and suitable means to stop the contractor until faulty workmanship is corrected often result in compounding minor damage to major proportions. The consulting structural engineer is often reluctant to stop a contractor when movements and/or damage are first noticed for fear he may be sued by the contractor for financial loss caused by the work stoppage. To minimize liability of the engineer and the owner in such cases, the specifications and contracts should be written in a manner which gives them the right to stop work. This of course is not easy to do in the contract, at least in such a way that it gives the engineer and owner absolute protection against a claim by the contractor. To some extent, insurance companies are responsible for allowing their insured contractors to continue poor methods or techniques which result in damage to structures. It seems that many insurance companies only pay attention to a job long after damage has occurred and a claim for damages against the contractor has been filed. In one instance in the Chicago Loop area involving a deep excavation for a large building a contractor knowingly pumped large amounts of silt in attempting to dewater caisson excavations long after he realized his pumping of silt caused nearby buildings to settle. In spite of the specifications containing a clause which said "there shall be no loss of silt due to pumping of water" the contractor actually removed several truck loads of silt caught in a water detention box, and neither the engineer nor the owner stopped the contractor for fear of a claim and possible lawsuit against them by the contractor.

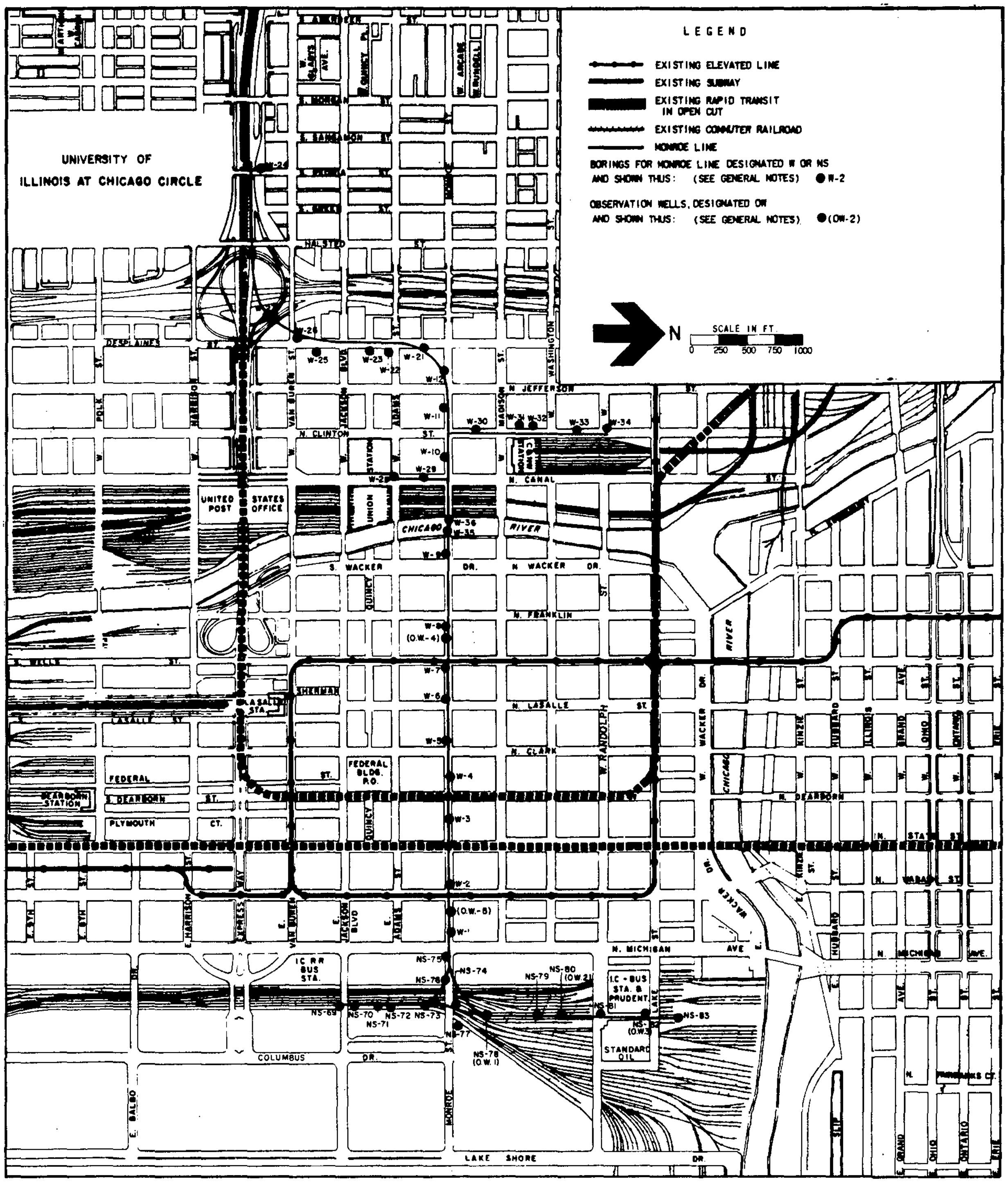
There are quite a number of cases in the Chicago area involving deep excavations in soft clay for which the sheeting and bracing systems were quite adequate in design, but in which the various stages of installation were not thought out correctly or in which the contractor did not provide suitable support during the various stages of cutting and installing walers and struts, so that movements occurred during the installation. Again in many of these cases the engineer either did not dare to stop the contractor and/or the design and specifications themselves were defective in spelling out all the detailed steps in the installation.

The above examples indicate that in many cases damage to structures and subsequent costly claims are many times not due to inadequate engineering or design know how, but to human and legal reasons. It seems that these kinds of problems are more difficult to solve than the engineering aspects.

What can be learned from these experiences which will keep us out of trouble with subway and other construction involving deep excavations? We do not seem to learn much from past experience because the types of occurrences described have been repeated over and over. First we must have an adequate early warning system to alert us in time of possible damaging movements. We must then have means for translating these warnings into action - action which will stop the contractor and force him to take corrective action promptly. We must have adequate inspection to detect faulty workmanship as it occurs and to detect movements, and to translate these inspections into positive action. We must have specifications and contracts written in such a way that the engineer and owner have the right to stop a contractor and force him to take corrective action without the danger of lawsuits. Insurance companies should assume their responsibility and get involved sometime before a claim is about to go to court.

REVIEW OF "SOILS AND FOUNDATION REPORT" - MONROE LINE

A report entitled "Soils and Foundation Report - Chicago Central Area Transit Project - Monroe Line" for the Chicago Urban Transportation District has been prepared by DeLeuw-Novick, Supervising Consulting Engineers, dated December 1974 and revised March 1975. The writer has drawn freely on information contained in this report and in a sense this entire paper is a review of the report. However, it is worth reviewing what the report contains and its proper place in the system design. A detailed subsurface investigation was performed with borings located as shown in Figure 2. Included in the report are the soil boring logs, field and laboratory testing and an investigation of existing foundations of buildings and other structures which adjoin the proposed route. Based on the results of these investigations a detailed description of subsurface conditions in relation to the proposed structures was developed. Soil parameters are given, which can be applied directly to



Boring Location Plan

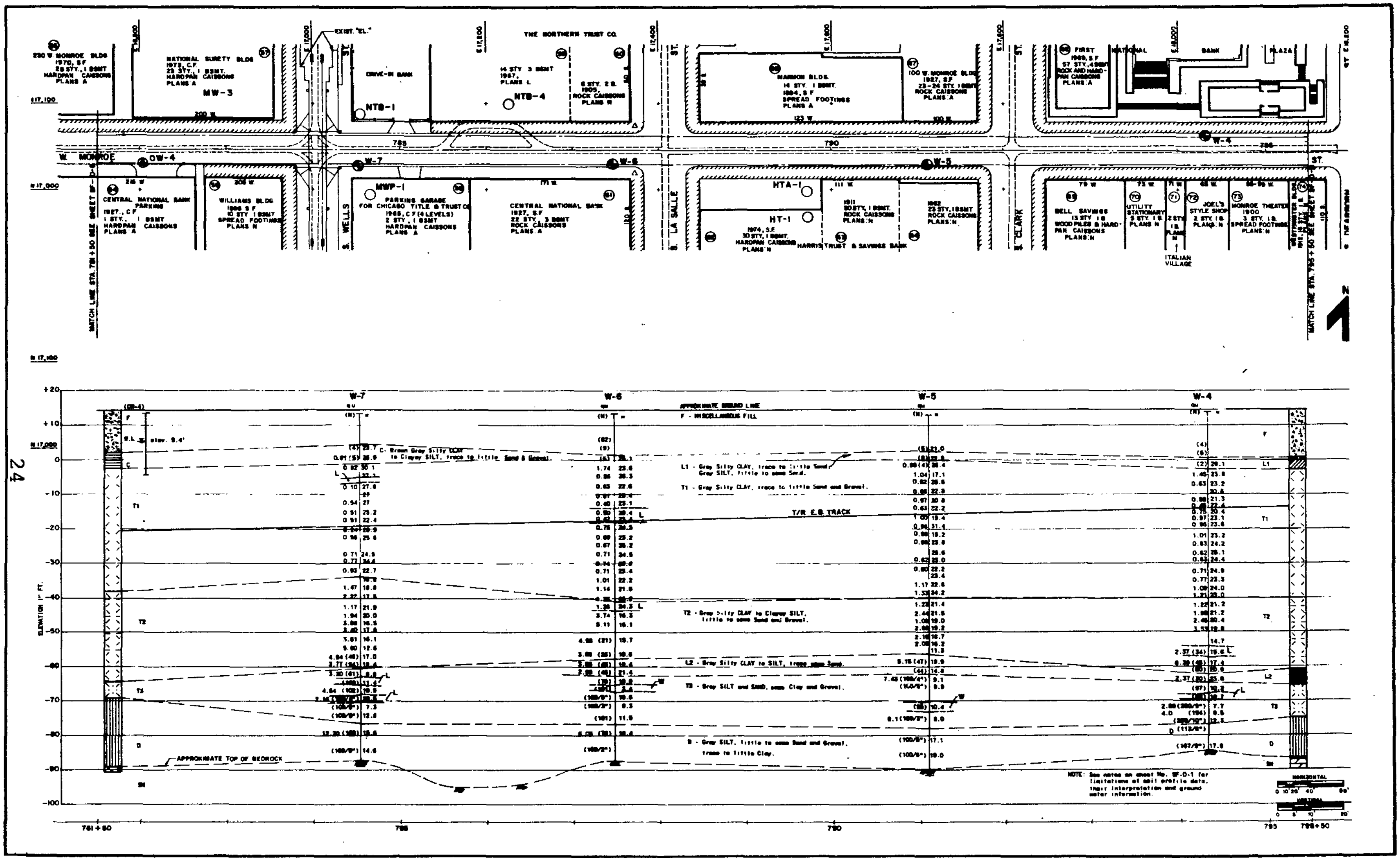
FIGURE 2

design. The Monroe Line included in the report extends from the University of Illinois Station at the western terminus through Stetson Station at the eastern terminus. Borings and proposed methods include: cut and cover subway sections, cut and cover stations, mezzanines, and pedestrian passageways; tunneling; portals; and retaining wall sections. A total of 3,500 lineal feet of soil borings were made and over 700 3-inch diameter thin-wall tube samples were taken. Unconfined compression tests, triaxial tests, consolidation tests were made on many of the samples. Complete boring information in the form of logs for each boring including all test data are presented. Geologic soil profiles are given along the entire route with all the soil test data. In compiling the profiles, certain borings made by the city of Chicago and others were utilized. An example of the soil profile is given in Figure 3, which shows a portion of the profile from Wells Street to Dearborn along Monroe. A plot of all buildings and other structures along the entire route is given in the report. A section in the report entitled "Foundation Studies" takes successive sections along the route starting from the beginning at the University of Illinois Station to the end at Stetson Station in increments varying from 700 to 1100 feet in length and discusses in detail the possible soil problems and planned methods of construction. Where buildings exist along the route specific structures are pointed out with their existing foundations, which it is believed will require special evaluation.

Sufficient soil boring information and soil test data and information on adjacent buildings and structures is given in the report to enable detailed designs to be made and construction methods decided. There will, however, undoubtedly be a need for special borings and soil information in certain critical areas and exploration to determine conditions under certain buildings and structures for which no information could be found. There would undoubtedly be a need for additional borings and soil test information at specific locations as problems develop during construction. Where the clays are softer than usual and the design of slopes in cut areas and walls in cut and cover and tunnel areas are critical it may be necessary to obtain a more detailed and accurate determination of shear strengths by means of in place vane shear tests. Additional information will undoubtedly be needed on water levels at certain critical locations. However, the vast majority of the design for the subway can be based on the soils information given in the report without need for further borings and soil test.

ANTICIPATED SOILS PROBLEMS

There will undoubtedly be many difficult soil problems to solve, not only during the detailed design stage but while construction is actually in progress. In spite of all the borings that have been made already and will be made in the



Geologic Profile: Monroe Street from Wells to Dearborn

FIGURE 3

future, we can also expect unanticipated problems in the form of surprises. Perhaps the biggest single problem is the squeezing of the soft clay. Problems can occur in the cut and cover sections due to wall movements whether they be steel sheeting, soldier beams and lagging, slurry trench walls, or other construction. It will not only be sufficient to design the walls for pressure conservatively, but to insure that the anchorage, struts and rakers have sufficient support so that they do not move. Squeezing can be a problem in the bottom trenches and cuts where the bottom is in the softer clays. To prevent this it may be necessary to extend walls deeper than the bottom of the subway excavation. Squeezing can also be anticipated in the driven tunnel section, both at the open face as the shield is pushed and in the space behind the rings created as the shield advances. Much experience has been gained on the BART system tunnels and the Washington tunnels so that the minimum expected surface settlement due to tunneling operations can be predicted. However, softer zones of clay, water bearing sand lenses may cause settlements in excess of that predicted and normally expected. Improper bracing at the face of the tunnel may also cause excessive squeeze and consequent large surface settlement.

Because of the large number of utilities that have to be encountered in excavation it will be difficult to drive or install sheeting in the upper part without encountering obstacles. Pipes, tunnels and conduits will make it difficult to create a tight face. Water bearing sands may wash through cracks and voids in the walls, causing surface settlements. Where soldier beams and wooden lagging is used, difficulty is often experienced just above the contact between the upper fill which is mainly sand and the top of the soft clay. Water accumulates in this zone and when opened it is sometimes difficult to secure the lagging in place before the sand runs out. Also, if the sections of lagging are not absolutely tight together the fines can wash out through the cracks between the lagging pieces. Where soil boring information indicates that this is a distinct possibility, other methods such as interlocking steel sheeting or possible slurry wall should be considered. However neither of these methods may be feasible if there are water mains, sewers, gas lines, electric service and telephone conduits to be encountered. Excavation using soldier beams and lagging may be the only method in such cases and where there is clearly a danger the water bearing formation should be grouted beforehand.

CONSTRUCTION METHODS WHICH SHOULD BE CONSIDERED

Several papers following this one will discuss various types of tunnel construction. It is therefore not within the scope of this paper to describe the details of each method. However, possible methods of construction applicable to the

Monroe Line for the specific soil conditions along the route are listed with limitations of each. Some of the methods listed and discussed below are not discussed in other papers at this conference.

<u>Box Section</u>

1. Soldier beams and wooden lagging
   - A. Soldier beams
     - (1) driven steel H-sections
     - (2) drilled shafts filled with weak concrete around H-section
     - (3) drilled shafts filled with reinforced concrete
     - (4) open holes
     - (5) slurry-filled holes
   - B. Lateral supports
     - (1) walers and across the cut struts
     - (2) tie-back anchors
     - (3) combination of struts and tie-backs
     - (4) embedment of soldiers below excavated depth
2. Interlocking steel sheeting with walers - lateral supports
   - A. Walers and across the cut struts
   - B. Tie-back anchors
   - C. Combination of struts and tie-backs
   - D. Embedment of sheeting below excavated depth
3. Slurry walls
   - A. Reinforced concrete walls which become part of permanent structure
   - B. Soldier beams placed in slurry trench with poured concrete walls between-across the cut strut bracing
   - C. Slurry walls with sufficient depth which become the vertical walls on cut and cover construction - top of cut and cover poured and backfilled before excavation completed - traffic restored on street above while excavation under roof is completed and bottom concrete poured
4. Concrete secant piles - continuous wall made of intersecting drilled shafts filled with reinforced concrete - walls act as permanent tunnel walls - tie-backs and/or internal struts needed as with slurry trench.
5. Diaphragm walls
6. Soil freezing to form temporary wall for excavation and building concrete box section

<u>Tunnel Sections</u>

1. Shield driven tunnel under compressed air
2. Shield without air followed by steel rings and wooden lagging

3. Shield driven tunnel without air lined with precast concrete or steel sections
4. Sunken prefabricated tunnel sections at river crossing
5. Freezing of ground for support - conventional excavation

The above listed methods for the cut and cover where box sections are planned should all be carefully considered. In general the soldier beam and wooden lagging is the most economical, but because of the possible loss of ground it may not always be the method which would cause the least damage to adjacent structures. In relatively shallow sections of the subway it will undoubtedly be the best method, everything considered. Where the bottom of the cut and cover section is deep and support of nearby buildings is important, the soldier beam and lagging method will probably be inappropriate. Some sections requiring cuts up to 65 feet deep the slurry trench method should be seriously studied and the secant pile method should not be overlooked as a possible alternative. Where cut and cover sections go through very congested areas with a maze of utilities in the way and the required excavation is rather deep, it may be necessary to use a combination of methods. Since a slurry trench or a secant pile method requires removal of obstruction and relocation of utilities, where there is maze of utilities it may be necessary to excavate the upper portion using soldier beam and lagging or steel sheeting or wooden sheeting to below the utilities and other obstructions which cannot be moved or relocated and then use the slurry trench method inside the sheeting and below the utilities. In many instances there will be too many sewers, water mains, power lines and electric utilities crossing perpendicular to the tunnel route which may make it impossible to use either the slurry trench or the secant wall method. In these cases the utilities will have to be supported temporarily while either sheeting or lagging is installed below. More meaningful consideration of construction methods can be given by considering typical sections along the line.

TYPICAL PROFILES ALONG MONROE LINE ROUTE AND ANTICIPATED PROBLEMS

Figure 4 shows a section along Des Plaines Street where the tunnel turns north to Monroe Street. The section is about four city blocks long and the buildings along this section are four to eight story commercial buildings for which no foundation plans were found. The buildings are all old and it can be assumed that at least some of them are on spread footings. At the south end (left side of figure) the cut and cover section is entirely within the soft silty clay where the required depth of excavation is 40 feet. At the north end (right hand side of figure) the bottom of subway is 60 feet below the surface and is just into the stiff to

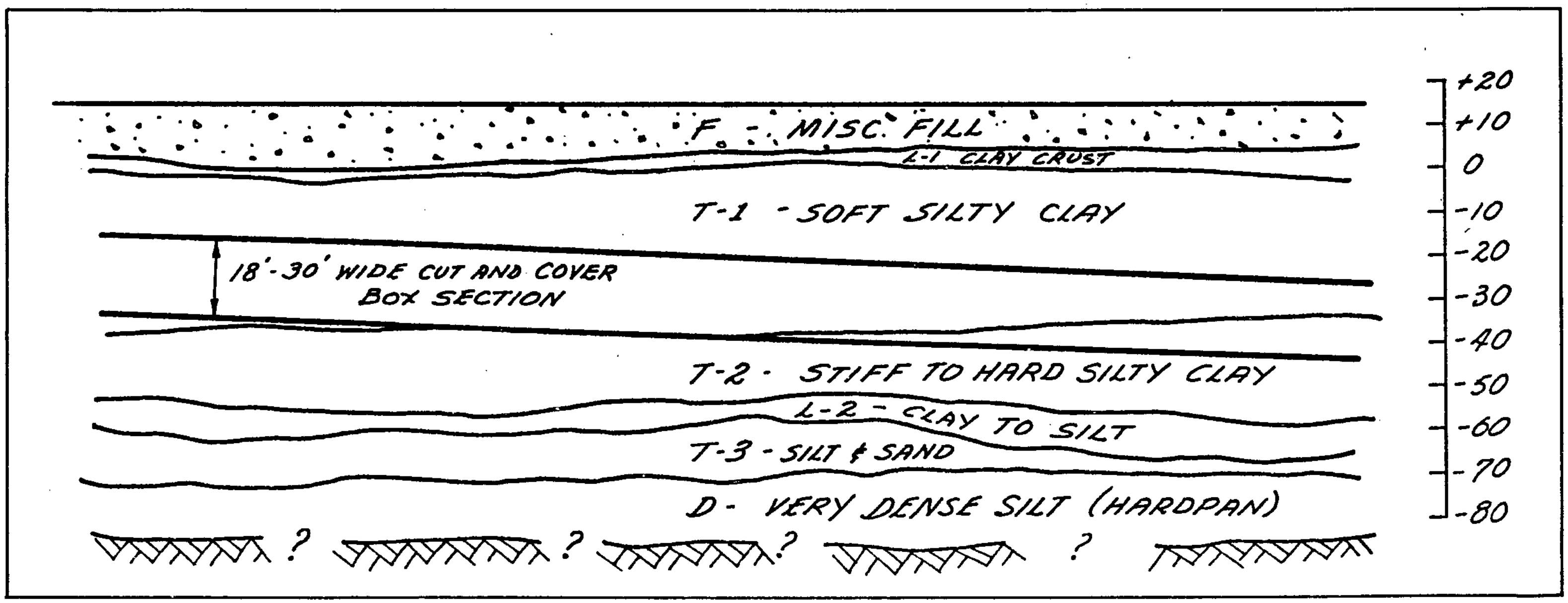


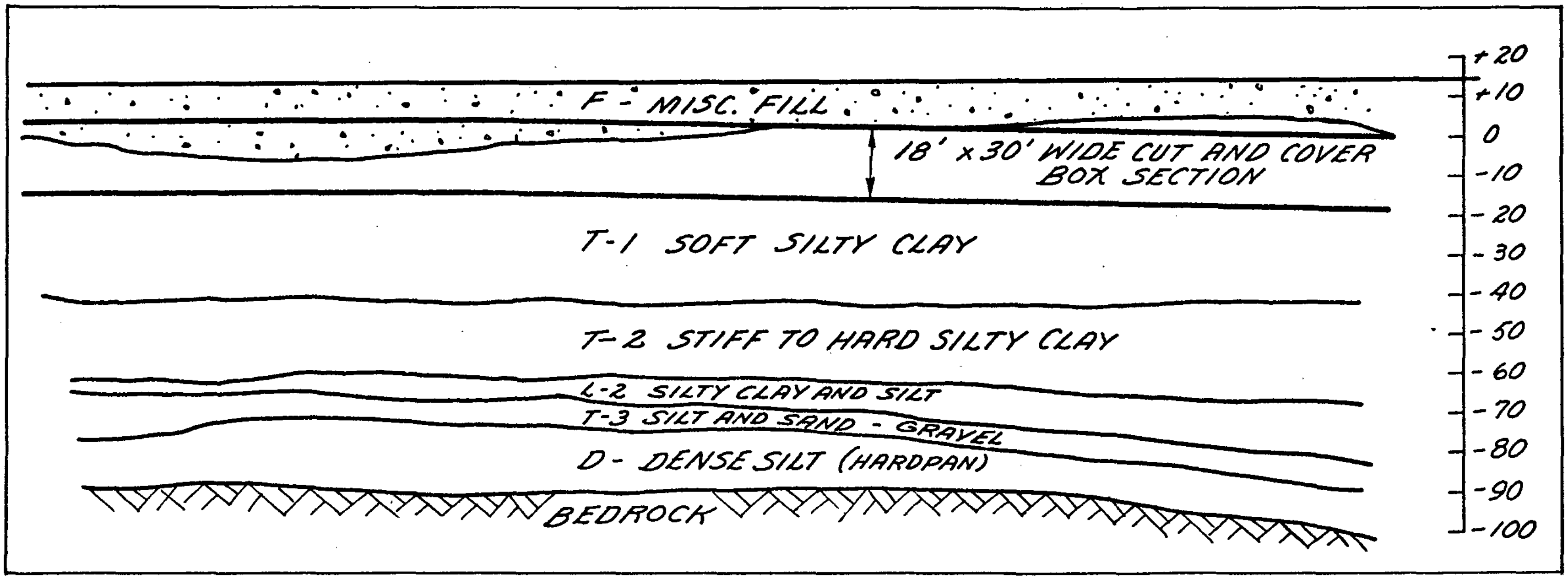
FIGURE 4

SECTION ALONG DES PLAINES STREET
VAN BUREN TO MONROE

hard clay. Since the strength of the soft clay averages about 0.5 to 0.6 tsf. and is as low as 0.3 in some places, lateral flow and bottom heave is a distinct possibility. Use of the slurry trench method or the secant pile method seems to be the best method of minimizing any lateral displacement and bottom heave. The trench would extend to some distance below the subway sufficiently into the stiff to hard clay to prevent bottom heave and to provide passive resistance to inward movement. However, a maze of utilities might make it difficult to install a trench or other continuous wall and it may be necessary to resort to a prebored pile and wood lagging system which could be braced internally at the top and anchored into the hard silty clay below in the lower part. The value of the buildings is such that it would seem hardly worthwhile to underpin them. A further detailed study of the number and location of utilities which might provide an obstacle should be made before a definite recommendation can be advanced. However if at all possible a slurry trench wall appears to offer the best chance of preventing settlement of the adjacent buildings.

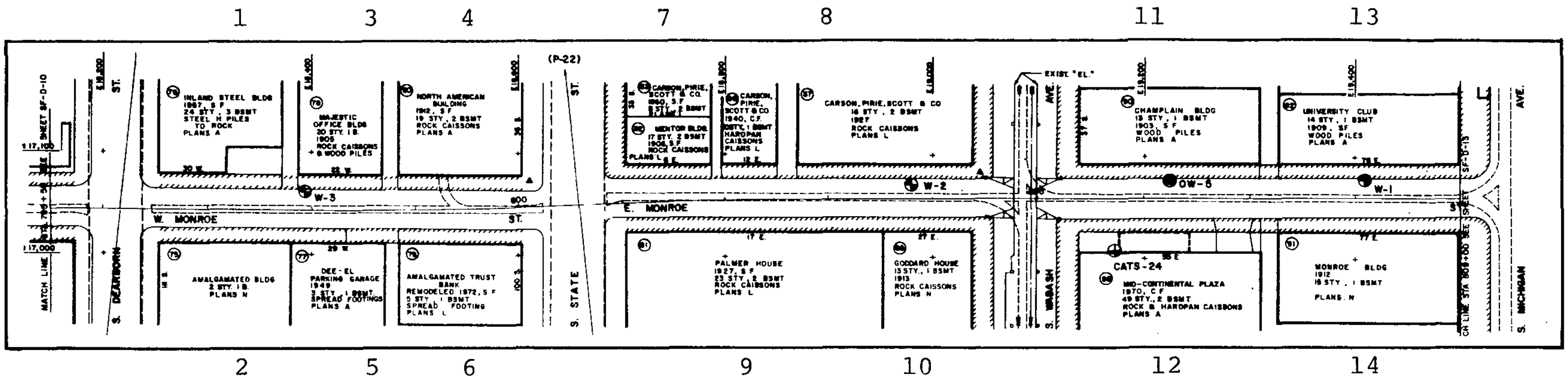
A special problem exists at the north end (right side of figure) where St. Patrick's Church is on the west side of the cut. This structure rests on rubble stone footings. Underpinning of this structure should be considered. In addition to the conventional underpinning, consideration should be given to freezing the ground below the footings and between the cut and the structure. This method has successfully been used to protect structures near deep cuts.

Figure 5 shows a three city block section along Monroe Street between Dearborn and Michigan Avenue. Here the cut and cover box section will be relatively shallow, requiring cuts from 30 to 35 feet. The bottom of the cut is entirely within the soft silty clay and the top of the subway is almost entirely within the upper fill material. A look at Figure 6 indicating the buildings along this section and their foundations shows that several have spread footings as foundations and that two are on timber piles. In addition, the Inland Steel Building on steel piles has the outer-most row of piles in the path of the box section and must be removed to provide space for the box structure. In addition, the elevated structure at Wabash will certainly have to be underpinned with the load from the underpinned columns taken to an area outside of the box structure. Because of the very soft clay, the buildings on spread footings and others on piles, the slurry trench method would certainly afford the best possible protection against lateral movement. However because of the very complex system of utilities this is virtually out of the question. Therefore, the soldier pile method will have to be resorted to in this section. Because of the very soft clay and proximity to buildings, even the conventional prebored pile may cause a lateral squeeze during excavation. It is suggested that perhaps the



MONROE STREET - DEARBORN STREET TO MICHIGAN AVENUE

FIGURE 5

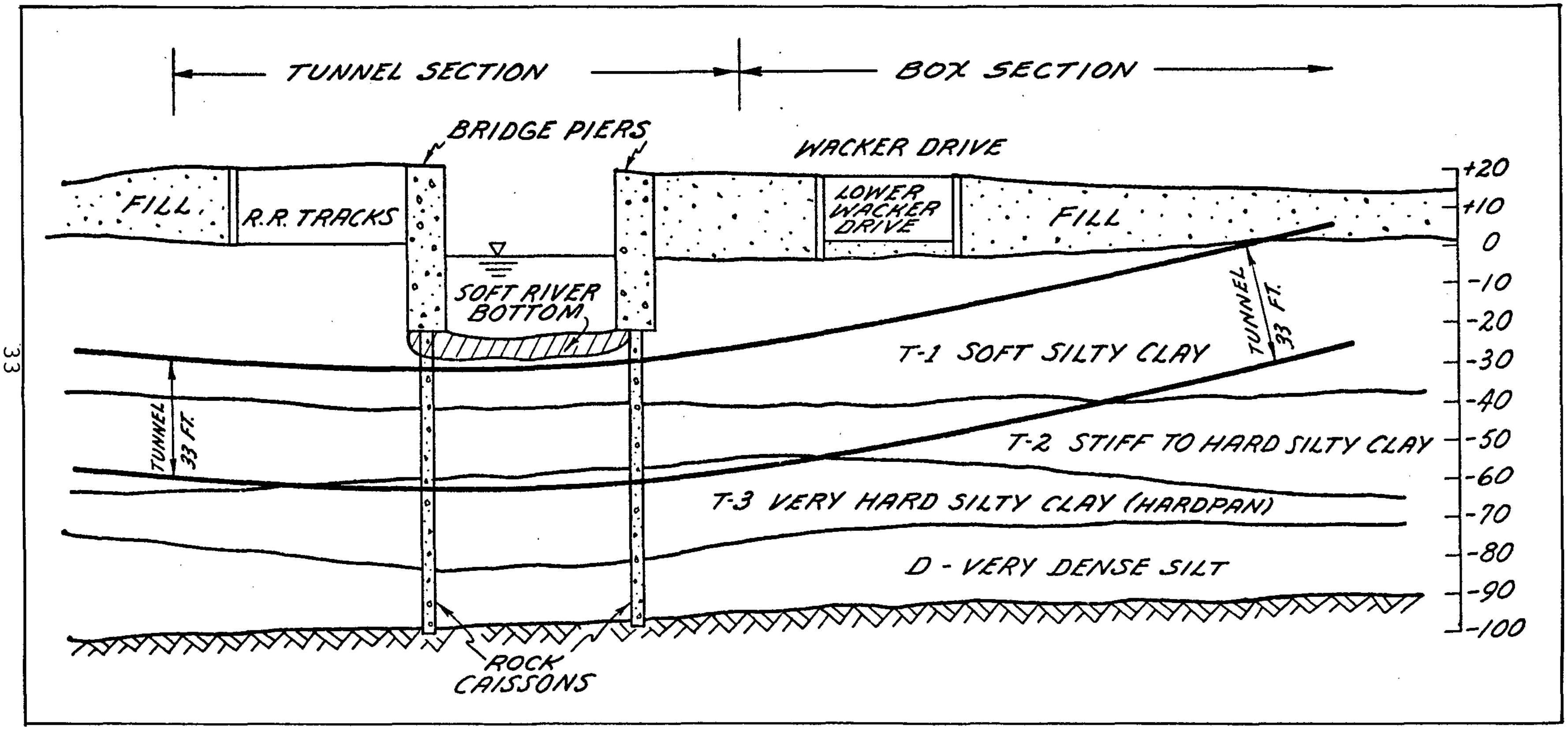


	BUILDING	HEIGHT/TYPE	FOUNDATION
1.	Inland Steel	19 Story Office Building	Steel Piles to Rock
2.	Amalgamated Bldg.	2 Story Shops/Offices	Not known
3.	Majestic Bldg.	20 Story Office	Caissons
4.	N. American Bldg.	19 Story Office	Hardpan & Rock Caissons
5.	DEE-EL Parking	4 Level Parking Structure	Spread Footings
6.	Amalgamated Trust	5 Story Bank Bldg.	Spread Footings
7.	Mentor Building	17 Story Office Bldg.	Rock Caissons
8.	Carson, Pirie, Scott	15 Story Commercial 10 Story Commercial	Rock Caissons Steel Underpinned Piles to Rock
9.	Palmer House	25 Story Hotel & Shops	Rock Caissons
10.	Goddard Building	13 Story Office Bldg.	Rock Caissons
11.	Champlain Building	13 Story Office Bldg.	Wood Piles
12.	Mid-Continental Plaza	49 Story Office Bldg.	Hardpan & Rock Caissons
13.	University Club	14 Story Club Bldg.	Wood Piles
14.	Monroe Building	16 Story Office Bldg.	Rock Caissons

FIGURE 6

prebored piles be constructed using bentonite slurry in the hole to prevent this squeeze. Also because of the soft clay at the bottom of the cut it is recommended that prebored concrete piles reinforced be constructed to the stiff to hard clay some 20 feet below the cut to form a continuous wall wherever possible and to use wooden lagging only in places where it is not possible to make the prebored piles continuous. The prebored piles thus will form an intermittent wall, the spaces between being filled with lagging. In local places where there is danger of squeezing of the soil at the openings between the intermittent walls it may be necessary in some instances to freeze the ground until the lagging or some other support is installed between the breaks in the wall. It may also be possible to avoid underpinning some of the buildings by installing continuous prebored reinforced concrete piles at an angle sloping downwards toward the building, thus forming a wall to support a building which overhangs the box structure. Such construction might well be used at the Inland Steel Building to support it while the outer row of steel piles is removed one by one and the support replaced by the box structure. This type of construction has been used on a number of occasions in Europe.

A very difficult problem exists where the tunnel section along Monroe Street crosses under the Chicago River. This is shown in Figure 7 where a tunnel section 33 feet in diameter is planned under the river and for some distance to the east and west with box sections connecting. Here the top of the tunnel and the soil above it on both sides of the river is in the very soft clay and the top of the tunnel under the river will only have 10 feet of cover under the river. This cover is a soft mixture of sand, silt, organic matter, and debris with very little shear strength. It is suggested in the report that a shield driven tunnel under compressed air be used for this section. In order to avoid a blow-out under the river where the 10 foot cover is soft and weak, it is suggested that this material be removed by dredging and replaced with a layer of plastic-soil-cement mix of gravel, sand and clay to provide an airtight blanket over the tunnel where it passes under the river. Compressed air is both very expensive and can be a dangerous method. An alternate suggestion would be to remove by dredging the soft material over the tunnel and some distance north and replace it with sand and perhaps increasing the cover several feet to the maximum which could be allowed and still provide navigation. This could then be frozen with the frozen wall technique and a wall frozen both north and south of the tunnel to provide an inverted U-shaped frozen mass some five feet thick through which the tunnel can be bored. It may also be necessary to freeze some of the soil for some distance away from the river behind the piers. This is illustrated in Figure 8c. A similar technique was used for an intake structure in Lake Huron in fifty feet of water. Sand was



Monroe Street - Chicago River Crossing

FIGURE 7

dumped on the bottom and frozen as a roof which was then covered with a concrete mat. A circular wall was frozen below this roof some forty feet to rock, leaving a 20 foot diameter circular section unfrozen under the roof which was excavated upward from a tunnel in the rock and concreted to form a twenty foot diameter vertical pipe. Certainly all possible schemes should be studied carefully to avoid the necessity of using compressed air.

Figure 8a shows the tunnel section passing through one of the two bridge piers. The clearance between the outside of the tunnel and the rock caissons supporting the bridge pier is estimated to be 2 inches. Figure 8b shows the tunnel section under the river between the bridge piers without removal of the soft river bottom cover.

MEASUREMENTS DURING CONSTRUCTION

In the past 10 to 20 years many techniques have been developed for measuring movements, pressures and strains in soils which did not exist during the previous subway construction in Chicago. Certainly maximum advantage should be taken with these techniques to adequately assess what is happening during construction. Slope indicators, settlement gauges, pressure meters, strut load measuring devices - all should be used to measure what is actually happening in the field. Techniques of measuring bottom heave while excavation is in progress have been used and should be used here. Though these installations are costly and take considerable personnel to make the measurements, when judiciously and strategically placed, they provide valuable information which could very well prevent failures, movements which cause settlement and damages to adjacent structures. Such a system of measurements provides an early warning system which detects movements, pressures and stresses before they can become dangerous and before they could otherwise be detected. These early warning systems can provide sufficient warning so that work can be stopped temporarily if necessary and corrective measures can be considered, studied and designed and executed before damaging movements and even collapses occur. Such measurement systems have been used on the BART system and the Washington subway as well as many other deep foundations in recent years. These installations should be studied and evaluated with a view to making careful choices for the most efficient and economical installation along this new subway system.

In addition to providing the installation of measuring devices it is necessary to organize qualified and trained personnel to take the measurements and to take them at proper intervals. Also provision in the contract should be made so that the contractors provide access for measurements and that the contractors do not interfere with, damage or destroy the measuring instruments. Provision should be made for properly recording the data and where necessary evaluating

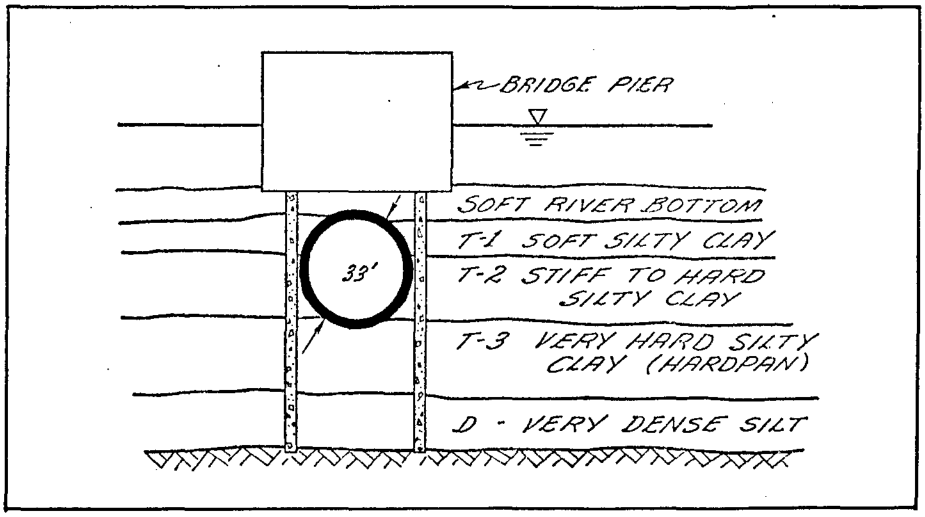


Figure 8a

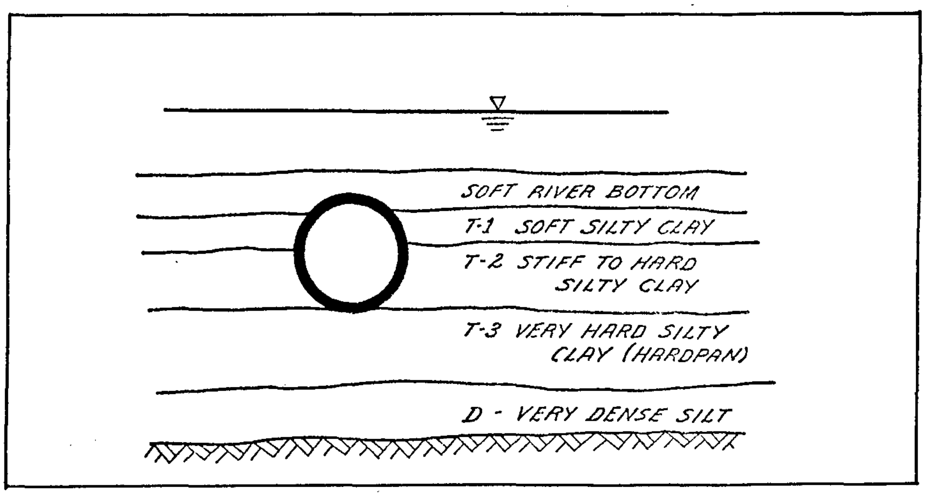


Figure 8b

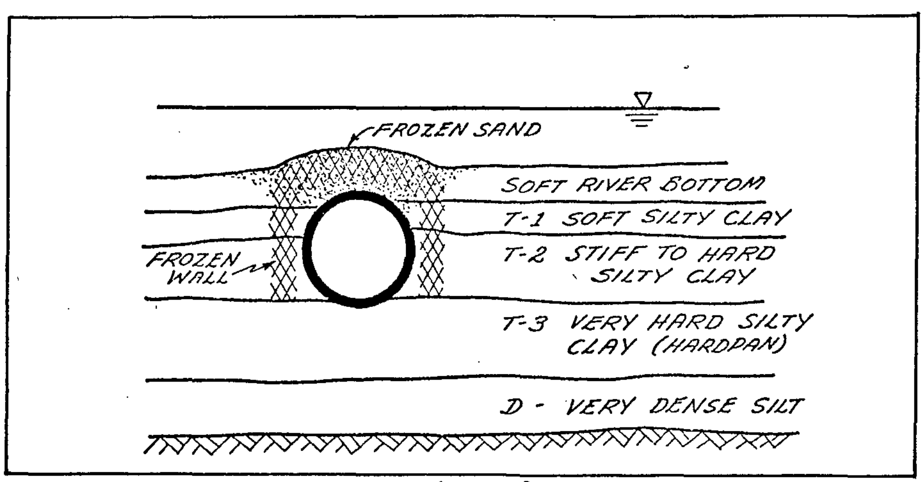


Figure 8c

FIGURE 8

it in the form of computations promptly. It may be necessary to use a computer system in which the raw data can be fed and movements, slopes and pressures, written out by the computer in an immediately useful form.

ORGANIZING FOR ACTION - DELEGATION OF RESPONSIBILITY

One of the weakest links on any construction job is the lack of communication and the inability to take quick action when something occurs. Often considerable data is taken and recorded but not reported to anyone. Often inspectors notice occurrences of importance but fail to report them to their superiors. When items of importance are reported they are frequently shoved aside and not acted upon. Often, after a collapse, accident or occurrence not anticipated, it is not known who said what, making it difficult to pin responsibility. Therefore, on any job such as this considerable study should be given to the organization so that these human failings do not occur. There should be a clear line of responsibility. All matters of importance should be relayed in writing. There should be a system of acknowledgment of receipt of information. There should be proper delegation of responsibility. There should be frequent review preferably by someone not involved in the everyday operations to be sure that the methods agreed upon are enforced. This is one area where we seem to make the same mistakes over and over again without learning from the previous experiences.

It is not simply enough to take measurements, record them, and report them. Some one person should be responsible for interpreting the measurements, deciding on whether they are acceptable or not, and when immediate action is needed. This person should be carefully instructed by the designers and the consultants before starting and should meet with them periodically. In addition, organization should exist so that in the case of an emergency the proper persons involved can be contacted and assembled as quickly as possible so that corrective action can be taken. The technical, financial and legal aspects should be determined quickly and not interfere with the expeditious solution of an urgent problem.

Individual contracts should be set up in such a manner that maximum advantage can be taken of the contractor's ingenuity. Frequently, specific methods are set up for contractors to execute and they are allowed little or no say in how the work is done. Machinery should exist for accepting contractors' alternative methods where they appear to be superior and provision for adjustment in cost should be made. Certainly where a contractor can come up with a better method at a lower cost, he should be given a reward in the form of a bonus for his suggestion and successful execution. In the writing of the plan specification and contract documents, provision should be made for allowing contractors to bid on their own alternate methods. Alternate proposals should be very carefully screened and studied before they are rejected or accepted, not only from a technical and construction viewpoint, but from a possible financial and legal viewpoint. And lastly, while under contract during construction, provision should be made for changes in methods proposed either by the consultants, designers or the contractor, if it is found to be necessary for safety, for better construction, or for economy.

PAPER 4

A History of Underground Construction in Chicago

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PAPER 4

A HISTORY OF UNDERGROUND CONSTRUCTION IN CHICAGO

by

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INTRODUCTION

This paper summarizes the history of underground installations in Chicago. It is being presented at this conference to illustrate the ingenuity which has accompanied the construction of projects in Chicago and to provide background information regarding construction techniques which have proved successful in Chicago's subsurface. It is hoped that the information contained herein will be useful in planning construction for the proposed Chicago Central Area Transit Project, the extent of which has been discussed earlier by Mr. Nelson. Most of the information has been obtained from construction records on file at the Bureau of Engineering, Department of Public Works. Other publications which have been consulted are included in the Bibliography accompanying this paper.

The underground construction industry and Chicago can be said to have grown up together. When Chicago was born, it was the result of a massive earth moving project. During Chicago's infancy, before the Fire, a 2 mile long tunnel under the Lake was constructed. At the same time, the entire downtown area was crisscrossed with shallow open-cuts for sewer and water mains. Also, the River had been traversed with 2 vehicular and pedestrian crossings and 5 water-pipe tunnels.

When Chicago was rebuilt after the Fire, the underground construction industry learned how to solve problems. The techniques learned in Chicago's growth period before World War I provided the basis for all underground construction, as now practiced. It was in this very short era that the art of foundation engineering and installation was perfected to the degree that the age of the skyscraper could follow as soon as architects learned how to utilize the various foundation types.

As Chicago stabilized, between World War I and World War II, the underground construction industry perfected and improved the previously learned techniques. It was in this time period that the huge interceptor sewers of the Sanitary District were built as well as the initial subway system. The "Chicago" method of installing caissons was copied all over the nation. The soil data compiled on Chicago installations at this time provides the basis for the determination that the fundamental theories of soil mechanics relate to actual soil behavior.

As Chicago modernized after World War II, the underground construction industry became highly mechanized. Earth tunneling machines as well as rock tunneling machines, have been used successfully in recent sewer construction. Machine dug caissons have become the mainstay of the foundation business and now support 3 of the 5 tallest buildings in the world in Chicago's business district.

Much is known about how to successfully build on, in, and through Chicago's soils; and conversely, much is known about unsuccessful techniques. Experience indicates that adverse construction conditions will be encountered in any endeavor of the magnitude of the Central Area Transit Project. Some of these conditions are saturated sand under and connected to Lake Michigan waters, squeezing clay, flowing sand and/or silt seams and pockets, methane gas pockets, large boulders, and saturated granular material under great external pressure; however, at one time or another, these conditions have been solved even if the past solutions leave room for improvement.

CHICAGO'S PRE-FIRE ERA (PRIOR TO 1871)

In order for Chicago to exist, it needed to become a commercial center; otherwise, there would be no need for people to move into the swamp. As its eastern boundary was Lake Michigan, trade with eastern states presented no problem; however, there was no water passage from Lake Michigan to the west and the need for such a link was vital. The City's founders rectified this problem by excavating the Illinois and Michigan Canal between the South Branch of the Chicago River and the Illinois River, a distance of some 96 miles. The excavation was accomplished between 1836 and 1848 and involved the removal of both earth and rock. As the rock excavation was not anticipated (which again proves the value of a competent preconstruction subsurface investigation), the Canal's cost was considerably more than anticipated; however, excavating and removing techniques tried on this project were to be perfected on later Chicago projects of a similar nature and made possible the economical and rapid construction of other large open-cuts around the world.

After the start of construction on the I & M Canal, Chicago grew rapidly (from 350 in 1833 to 28,000 in 1848) and the need for a sewage collection system soon became evident. The installation of Chicago's sewer system was started in 1854; and, by 1860, 80% of the collectors which presently serve the downtown area were constructed. During this initial sewer construction program, the area which was served is presently bounded by Division Street (1200N), Ashland Avenue (1600W), 16th Street (1600S) and Lake Michigan on the east. This collection system is located 8 to 20 ft. below the present ground surface and was constructed in a unique manner for an underground installation. The sewer mains consisted of a single brick lining having a circular cross section and a diameter of between 3 ft. and 6 ft. The mains were constructed on the existing ground surface or in shallow excavations (no deeper than 10 ft.) and the ground surface surrounding the mains was raised from 3 ft. to 15 ft. over that which existed prior to construction of the sewer main. This type of installation produced no major technical innovations in underground construction; however, the methods for moving buildings, as well as the art of underpinning the buildings of that time were developed to perfection. As an example, the Briggs House, a hotel located at Randolph and Wells, was raised 4-1/2 ft. by use of jacks and put on new foundations with no interruption in its daily business. Another present day effect of this installation was the creation of the approximately 8 to 10 ft. of non-select, compressible, decayable fill which now underlies Chicago's downtown ground surface and makes shallow utility cuts difficult and conventional shallow foundations even for very light buildings impossible.

While the sewer collector system and grade raising worked in relation to getting sewage and storm water off the streets and out of the buildings, it, of necessity, emptied into the Chicago River which, at that time, flowed naturally into Lake Michigan. In order for Chicago to survive without choking on its own garbage, it soon became apparent that the water pollution problem had to be rectified. To immediately combat this problem, the City made two far reaching decisions: (1) Extend the drinking water source from the near shore of Lake Michigan 2 miles into the Lake; and (2) Reverse the flow of the Chicago River from Lake Michigan to the Illinois River.

In order to extend the drinking water source farther into the Lake, it was necessary to construct a tunnel (approximately 2 miles long) under the Lake to connect the new intake structure (Two Mile Crib) with the new pumping station (present day site of the Chicago Avenue Station, 800N and 100E). This feat was accomplished between 1864 and 1866 and is acclaimed as an international feat of engineering.

The finished tunnel consisted of a double brick lining having a circular shape and an internal diameter of 5 ft. The tunnel has an invert elevation of -69 (C.C.D.) and its entire cross section and length was mined through very tough to hard silty clay. The construction work was performed around the clock and an average of 84 ft. of tunnel per week was constructed.

The majority of the tunneling proceeded from the land towards the crib; however, after the crib was placed in the winter of 1863, another heading towards land was started. The two headings met about 1/2 mile west of the crib and were only 7 inches out of line. This almost perfect line and grade is even more amazing when one notes that the crib was set a "few" inches out of its planned location due to some stormy weather.

Mining in both headings was accomplished by hand using a technique of full face excavation with no temporary supports. The permanent brick lining at the invert arch was kept within 4 to 10 ft. of the face and at the crown arch within 10 to 20 ft. Muck removal was by donkey train and timber supported by-pass tunnels were constructed to assure no muck removal delays. The brick lining for the crown arch was temporarily supported by a circular sheet of boiler iron until it "set-up". At the same time that the crown arch was "setting-up", the void space between it and the mined earth was filled with mortar. Since the "set-up" time took less than 30 minutes, it was relatively easy to establish a uniform construction sequence of mining and mortaring which resulted in a progress differential of $\pm$ 10 ft. per week from the average.

By astute planning or, if you prefer, luck, no major hazards or problems were encountered during excavation. The clay stratum was essentially uniform with occasional "quicksand" seams, soft clay pockets, and boulders. These nonconforming subsoil features were troublesome but caused no serious difficulty once it was realized that the flowing sand seams were not connected to the lake water, that the soft clay pockets only needed temporary supports, and that the boulders could be blasted successfully if that was necessary. The worst safety hazard encountered was an occasional pocket of methane gas; however, the gas was diluted without any mishap. The worst construction problem encountered was in preventing the upper sand underlying the Lake's water level from entering the land shaft. This problem was solved by sinking a 9 ft. diameter cast iron cylinder through the sand and sealing it into stiff clay at about elev. -30 (C.C.D.).

In constructing this tunnel, the land and lake shafts were sunk using the same basic principles as would be used today. The tunneling procedure was a variation of the same hand mining technique used in the initial subway system in 1940. About the only changes which would be made today would be in materials and tools.

In order to reverse the flow of the Chicago River, the I & M Canal was deepened between 1865 and 1871 from the South Branch of the Chicago River to the DesPlaines River (approximately 30 miles). This excavation involved only clay removal and is most noteworthy because it didn't work, making the construction of the present day Sanitary and Ship Canal necessary. Engineering-wise, the deepening was successful; however, no sooner was it completed than land developers increased the flow into the DesPlaines River causing it to back up the I & M Canal into the Chicago River and, in turn, into Lake Michigan during storms.

The rapidly expanding population of Chicago (approximately 200,000 in 1865) made it desirable to establish underground crossings as well as bridges across the Chicago River to increase the accessibility to the downtown business area. Between 1867 and 1869, the first underground vehicular and pedestrian crossing was constructed at Washington Street (100N); and, between 1869 and 1871, at LaSalle Street (200W). These tunnels were identically constructed in dry open-cut by cofferdamming a little over half of the river at a time. Both tunnels were of massive brick construction inverting at about elev. -35 (C.C.D) and crowning at about 1 ft. below the then river bottom elevation of -15 (C.C.D.). Construction was hampered by the soft compressible clays; however, the difficulties were overcome and those tunnels served for over 25 years as major arteries into Chicago's downtown area, as well as providing major escape routes during the Fire.

A series of incidents during the 1860's showed the advisability of installing the major water-pipe distributors in tunnels under the river instead of either hanging them on the bridges or laying them on the river bottom. Accordingly, in 1870, brick tunnels were built crossing the river (both branches at Division Street 1200N), at Chicago Avenue (800N), at Archer Avenue (2000S), and at Adams Street (200S). Construction of these was similar to the Washington and LaSalle Street River Crossings except on a much smaller scale and no serious difficulties were encountered.

As can be seen, in Chicago's infancy, the underground construction principles of deep open-cut excavating, shallow trenching, tunneling, and constructing inside cofferdams had already been formulated.

CHICAGO'S REBUILDING AND GROWTH ERA (1871 TO WORLD WAR I)

After the great Fire of 1871 left Chicago in ruins, a period of subsurface construction ensued rivaling that at any place in any time in history. The nature of the subsoil underlying Chicago made it necessary to pioneer underground construction techniques. The art of present day foundation engineering and construction evolved in Chicago during this period, as piles replaced footings and were in turn replaced by caissons faster than architects could envision buildings to be supported by the new foundation types. Over 70 miles of deep open-cut excavation which resulted in permanently reversing the flow of the Chicago and Calumet Rivers was performed. Also, many miles of medium depth (10 to 30 ft.) trench excavations requiring substantial bracing systems were dug. In addition, over 62 miles of tunneling through the soft compressible

Chicago clay had been accomplished; as well as about 100 miles of tunneling through the hard clay, hardpan, and rock underlying the compressible clays.

Prior to 1871, it was common to found buildings on relatively shallow footings bearing on the sand or desiccated clay overlying the compressible clay. As the size and weight of structures grew, it became evident that the heavier they grew the deeper they sank after they were built. By 1874, it was common practice to found major structures on timber piles which just barely penetrated the soft clays (approximate bearing level elev. -40 (C.C.D.). In the 1880's, sand piles (caisson excavations filled with well tamped sand) which penetrated the commpressible clay were a foundation type much in use. In the early 1900's, the development of concrete to fill the caisson excavation gave rise to the "Chicago caisson". In 1903, the 39th Street Pumping Station was founded on 50' deep hardpan caissons. In 1908, 100 ft. deep caissons, 9 ft. in diameter, were installed bearing on the bedrock for the support of City Hall.

In 1890 and 1891, a typhoid epidemic made it imperative that Chicago's sanitation problem be alleviated. Between 1892 and 1900, the present day Sanitary and Ship Canal was excavated for a length of 28 miles between the South Branch of the Chicago River and the DesPlaines River at Lockport, Illinois. This Canal has a minimum width of 160 ft., a minimum water depth of 24 ft. (4 times the depth of the I & M Canal), and a downgrade away from Chicago at 1 ft. for every 4 miles. Its completion in 1900 assured the continued drainage of the Chicago River away from Lake Michigan. During construction, earth moving machinery, muck conveyances, blasting techniques, heavy crane-type machinery, etc., were developed which made this type of operation possible all over the world. The Panama Canal was built by the technology developed in the "Chicago school of earth moving". Also, our own 16 mile long Cal-Sag Canal to divert the flow of the Calumet River away from Chicago was constructed between 1911 and 1922.

In this time period, approximately 2,000 miles of sewer were constructed, most of which was installed in open-cut between 6 and 30 ft. below ground surface. The sewers were circular and ranged in diameter from 2 ft. to 20 ft. Prior to the early 1900's, the sewers were constructed of a single or double brick lining; and, after the turn of the century, cast-in-place concrete was used. It became impossible to walk more than 1/2 mile in any direction without seeing Wakefield wood sheeting sticking out of the ground or, later on in this period, steel sheeting. Bracing systems varied from being extremely skimpy to being so massive that there didn't appear to be room for the structure.

The success of the first Lake Tunnel was duplicated between 1872 and 1874 when its twin was built paralleling the alignment of the first tunnel. However, as a part of this second tunnel system, a portion under land was constructed in a straight line from the Chicago Avenue Pumping Station at elev. -69 (C.C.D.) to 2200 South Ashland Avenue at elev. -25 (C.C.D.). From Chicago Avenue until west of the South Branch of the Chicago River, no problems were encountered; and then, at Polk Street (800S) the first blow in occurred in a Chicago tunnel. This difficulty was soon overcome by changing the alignment of the tunnel and by-passing the troublesome sand layer, which was probably a buried stream channel. Also, while not a part of the tunnel construction itself, compressed air was first used in Chicago to facilitate sinking the new lake shaft at the Two-Mile Crib.

As an interim sanitation measure, a 12 ft. diameter conduit was constructed between 1874 and 1878 under Fullerton Avenue (2400N) from the North Branch of the Chicago River to Lake Michigan. This conduit was used to pump Lake Michigan water to the North Branch to create a flushing and diluting action in the Chicago River. For 3/4 of a mile to the east, away from the River, the conduit was built in open-cut about 25 ft. below ground surface with an invert elevation of -13 (C.C.D.). This was the first major braced trench excavation in Chicago. The next 1-1/4 miles of conduit was installed at lower elevations by tunneling, and, at Larrabee Street (600W), Chicago's first mixed face (rock invert and clay crown) was encountered. Since it took twice as long to construct 1 mile of this conduit as it did to construct 2 miles of water tunnel under the Lake, the effect of the mixed face heading is obvious; however, this started the learning process in tunnel blasting techniques.

In 1879, the first tunnel to have its full cross section in rock was constructed at 18th Street (1800S) for a water-pipe tunnel crossing the South Branch of the Chicago River. In 1800, the first tunnel to have the majority of its cross section in soft compressible clay was constructed at Cortland Street (1900N) for a water-pipe tunnel crossing the North Branch of the Chicago River.

In 1886, it was decided to extend Chicago's water source farther into the Lake and construction on the Four-Mile Tunnel was started. An 8 ft. diameter tunnel immediately got into difficulty when soft squeezing clay was encountered in the crown and granular material under an external water pressure was encountered in the invert. After trying for a year without success to utilize a shield, the solution to the problem, decided on in 1888, was to divide the 8 ft. diameter tunnel into two 6 ft. diameter tunnels slightly lower in elevation and to pump against the flowing granular material. After 3 years of futility, air pressure (approximately 30 psi) was finally used to keep the water out and the tunnel finished in 1892.

In 1888, the 7 ft. diameter Old Polk Street Tunnel was constructed in soft to tough clays. No air pressure was used until 1891 in the Four-Mile Tunnel; but, while some settlement of overlying streets and structures occurred, tunneling was done successfully by limiting the heading advancement to one set of timbers in much the same manner as the "Box Tunnels" of today are constructed.

In 1889, construction was started on the 6 ft. diameter Lakeview Tunnel; almost immediately, a mixed face was encountered. After about 4 years of indecision, this construction difficulty was solved by lowering the elevation and mining a full face heading of rock. Blasting techniques were improved a great deal on this 2 mile long project and it was finally completed in 1896.

From 1890 to 1898, the 68th Street Tunnel complex was constructed. In general, the 5 ft. to 7 ft. diameter tunnels were advanced through hard clay and hardpan; however, in a 1/3 mile area about halfway between land and crib, a sinkhole in the clay was encountered. Air pressure was finally used to keep the Lake out; but, it proved difficult to keep the air from escaping and a multiple series of safety bulkheads had to be built which also helped slow the project down.

Between 1896 and 1900, the 8 ft. to 10 ft. diameter Northwest Land and Lake Tunnel was constructed. The lake portion had a heading consisting of a hardpan invert and soft squeezing clay crown which was prone to allowing blow ins. The soft crown problem was solved by air pressure and a slow pickaxe and shovel excavating procedure as opposed to a less safe but faster blasting operation. The land portion encountered a soft clay crown and/or a mixed face heading in different areas, both of which required tight timber supporting.

In 1899, construction was started on the first two interceptor sewers. These sewers were to have a 20 ft. diameter in 39th Street and a 16 ft. diameter in Lawrence Avenue; and their purpose was to eliminate that discharge which had, up to this time, been directly into the Lake. These two projects had to penetrate soft compressible clay and water-bearing sand layers and were, without a doubt, the inventive contractor's dream.

It was proposed to excavate the 39th Street Sewer using a shield and between 8 and 10 psi air pressure. After much experimenting and modifying, the shield method of mining was born. The sewer heading which consisted of a soft clay invert and saturated sandy silt crown was successfully mined; however, a problem, which has yet to be solved, occurred. The overlying street settled between 4 and 8 inches due to squeeze around the tail piece.

It was proposed to excavate the Lawrence Avenue Sewer with a machine. After 2 years, the contractor defaulted; however, if he was still alive 60 years later, he could have said, "I told you so." Eventually in 1905, the sewer was completed using the same procedure used in the 39th Street Sewer.

In 1901, construction of the Chicago Freight Tunnel system was started and by 1909, over 62 miles of eliptical 6'x 7'-6" tunnel had been constructed. Almost all of the system inverts at elev. -31 (C.C.D.) except in the vicinity of the eleven River crossings where the invert is about 20 ft. lower. This, in general, resulted in the majority of the tunnel system having an invert in stiff to tough clay and a crown in soft compressible clay; and, at the river crossings, having an invert in hard clay and a crown in tough clay. Prior to 1906, all tunneling was done under approximately 10 psi air pressure with full face heading. No temporary lining was used except local timbering, and the permanent lining was concrete. The concrete lining was formed during construction by a wood lagging-steel rib system. No settlement of overlying streets and/or structures occurred during this period of construction. After 1906, some portions of the system were tunneled without air pressure using a full face heading kept no more than 3 ft. in front of a temporary wood lagging-steel rib lining. This method of construction was not very successful in protecting overlying streets and structures as some large settlements occurred. The only tunneling problems encountered were water bearing sand seams and boulders; and these created no great difficulties. The construction methods utilized to build the freight tunnel system formed the basis for those used in the later construction of the initial subway system in similar soil conditions.

In 1906, the last two water tunnels to be constructed in earth were started in the New Polk Street Tunnel and the Blue Island Avenue Tunnel. Both of these tunnels were lined with concrete packed between temporary steel forms and the outside earth. Both tunnels were constructed under 10 psi of air pressure using open face mining techniques. Also in 1906, construction on the first tunnel, which was from the beginning planned to be full face in rock, was started. The 9 ft. to 14 ft. diameter Southwest Land and Lake Tunnel was constructed by top heading and bench blasting in a full face rock heading located between elev. -100 (C.C.D.) and -150 (C.C.D.). The blasting technique left large overbreaks; however, it was improving each time a tunnel was built. The tunnel was lined with concrete packed between the rock line and temporary steel supports.

In the period from 1908 through 1912, the three existing vehicular tunnels crossing the Chicago River had to be lowered to permit the dredging required by the U.S. Supreme Court's ruling against diversion of Lake Michigan's waters. A discussion of the construction of the original LaSalle Street and Washington Street Tunnels is contained earlier under the Pre-Fire heading and the original Van Buren Street (400S)

Tunnel was constructed in 1893 in a similar manner. Three different solutions were utilized for the same basic problems.

The Van Buren Street Tunnel was reconstructed wholly within its original structure and no new tunneling was performed. First the roof of the tunnel was dropped about 4 ft. by supporting the girders for the new concrete roof on the original brick walls. Next, the original invert brickwork was removed and the existing walls underpinned with 5 ft. wide concrete sections every 25 ft. to the required depth (approximately 4 or 5 ft.). Then, the remainder of the excavation for the new concrete walls and invert was completed and the new tunnel constructed. The worst proplem encountered during excavation occurred when the original cofferdam piling was unearthed; this allowed river water a source of entry into the excavated area under the new roof.

The Washington Street Tunnel was reconstructed by tunneling with air pressure under a new roof which had been constructed in 1906 similar to the procedure used at the Van Buren Tunnel. The reason for air pressure tunneling as opposed to the Van Buren Tunnel method was the necessity to lower the invert 15 ft. instead of 4 or 5 ft. The tunneling was performed by starting small drifts under both original side walls supporting or underpinning the original footings with structural steel arches every 3 ft. These arches were permanent and served to reinforce the underpinning concrete for the new walls which was placed as soon as the large center drift could be excavated. The excavation for both the small drifts and the center portion was done using 2 benches and a 3-stage heading. The construction went smoothly and no difficulty of importance was noted.

The LaSalle Street Tunnel was reconstructed by completely abandoning and destroying the original tunnel. This necessitated rebuilding two tunnels: one for vehicular traffic and one for a water-pipe crossing. The 12 ft. X 16 ft. water-pipe tunnel was excavated first about 20 ft. lower than contemplated for the vehicular tunnel. This tunnel was excavated full face by blasting through hardpan, which necessitated constant shoring of the roof. All would have been well, had the Contractor not gambled and blasted the face when directly under a crossing of the already abandoned Crosstown Water Tunnel. Since, as is the case today, it is impossible to bulkhead tightly an abandoned water tunnel, a considerable amount of water entered the new tunnel delaying the work. The approaches to the River for the vehicular tunnel were constructed by a combination of tunneling and open-cut. The tunneling was started near the River's edge by a center drift which progressed away from the River to a point where the crown became too close to the surface to continue. Then, side drifts about 10 ft. wide were dug perpendicular to the center drift every 30 ft. All the drifts were temporarily supported by timbers and the permanent concrete tunnel lining was placed in 10 ft. sections at each side drift heading. From where tunneling was stopped to where the tunnel grade and street grade meet, the excavation was performed in open-cut using steel sheeting and 12" x 12" timber bracing. The width and deepest depth of cut were each approximately 40 ft. and the actual earth cutting was done by clay knives. The river crossing section of tunnel was constructed by floating in a precast twin tube section which was sunk into a pre-excavated trench. The precast section was originally lowered onto piles at each end of the section and sand pumped underneath to provide firm bearing. Before lowering the precast section, a crib cofferdam was attached to each end of the section to provide a connection for the steel sheeting cofferdam subsequently connecting the precast section with the shore.

The last project started prior to World War I was the 12 ft. to 13 ft. diameter Wilson Avenue Water Tunnel constructed by top heading and bench blasting in full face rock heading located at approximately elev. -100 (C.C.D.). The tunnel is under Wilson Avenue (4600N) from 3 miles out into the Lake to the Mayfair Pumping Station (4800W). This tunnel was constructed between 1913 and 1918 and was unusual in that the rock being mined was used in the concrete mix. Other than that, its construction was just like that for the Southwest Land and Lake Tunnel, except that overbreak due to blasting was reduced.

In this period of Chicago's rebuilding and growth, every technique of underground construction was tried and improvements were made in those which seemed the most promising. In tunneling, the use of air pressure to avoid damaging settlement as well as to prevent water infiltration was introduced, a shield was used relatively successfully to advance a full face heading, the bench and staged heading method of hand mining had been established, blasting techniques were being improved, the art of placing tight temporary timber supports was advanced, and concrete lining was begun. In excavating, hand-dug caissons were established, steel sheeting was introduced, earth moving and digging machinery was replacing hard labor in open-cuts, and the clay knife was invented. It was time for perfecting existing methods.

CHICAGO'S STABILIZING ERA (WORLD WAR I TO WORLD WAR II)

Between the two wars, the growing rate slowed down in Chicago. It was a time to improve or replace worn out or deficient installations, to consolidate and strengthen existing systems; to build up, rather than out; and to perfect construction techniques.

The first new noteworthy construction technique performed after World War I was the installation of belled caissons to support the Franklin-Orleans Bascule Bridge in 1918. Up until this time, all caisson excavations were straight shaft of a diameter large enough to provide the necessary bearing area.

This improvement in an existing construction method allowed a more ecomonical design as now the shaft diameter was no longer dependent on bearing requirements.

In 1919, construction of the 12 ft. diameter Western Avenue Water Tunnel commenced. This tunnel is located between 73rd (7300S) and State Street and 49th (4900S) and Western Avenue (2400W) at or below elev. -100 (C.C.D.) and had a full face rock heading. Mining was performed by the top heading and bench method and muck removal was by hand into mule train. Some of the blasted rock was again transported to a rock crusher and concrete aggregate manufactured. The tunnel was given a concrete lining supported by 30 ft. long metal forms. No problems of significiance were encountered during mining; however, the number of clay seams encountered in the rock necessitated a large amount of temporary timber supporting.

The 10 ft. to 16 ft. diameter Chicago Avenue Land and Lake Tunnel was constructed from 1922 to 1935 and is the last crib intake connection needed to supply Chicago with its drinking water. This tunnel system is located under Chicago Avenue (800N) from 3-1/2 miles out in the Lake to the Springfield Avenue and Central Park Avenue Pumping Stations (3800W), and is located below elev. -100 (C.C.D.). Mining was performed by the standard top heading and bench method in a full rock face with the bench being about 30 ft. long. Blasting was done in about 15 ft. sections with both face and bench being blasted simultaneously. The drilling to hold the blasting powder was done by the first "jumbos" capable of drilling 10 holes per "jumbo". Muck handling at the face was accomplished by the first mucking machine (a primative high lift) and its removal was by steam locomotive train. Muck removal from the tunnel at the shaft was by steam operated elevator, which was another first on this tunnel. The rock needed to manufacture concrete aggregate was transported to a rock crusher and from there to a concrete mixer. During placing of the concrete lining, the concrete was moved to the form work by a conveyor belt and hand packed behind the form which was a 60 ft. long steel plate. The construction of this tunnel could have been performed last week and, except for new and more powerful machinery, little discrepancy in today's methods would be noted.

The majority of the Sanitary District Interceptor Sewer System was built in the 1920's and early 1930's. These sewers, of which the previously described 39th Street and Lawrence Interceptors were the first, had diameters from 4 to 20 ft. and were located at least 30 ft. below ground surface. The purpose of these sewers is to intercept the flow from the high level collector system before it reaches the collector's original river outlet, thereby preventing pollution of the area's rivers. The intercepting systems are on both sides and basically parallel to the Chicago, Calumet and DesPlaines Rivers as well as to the Sanitary and Ship Canal; and they eventually feed into sewage treatment plants. The completion of the intercepting sewers marked the last phase of Chicago's solution to a sanitary problem which threatened its very existence. This solution has been designated one of the seven engineering wonders of the United States by the American Society of Civil Engineers. The construction of the sewers were such that every soil condition in Chicago's subsurface was encountered; and, there was ample opportunity to perfect the tunneling techniques learned before World War I. Air pressure techniques were improved; the shield advancement procedure was established; air spades instead of pickaxes were developed; the bench and staged heading method of hand mining was perfected, as were blasting techniques to eliminate as much over-break as possible; temporary timbering advanced to temporary steel ring and liner plate supports; muck removal machines were made more versatile; concrete was improved so that it was possible to pneumatically place the permanent lining, instead of hand packing it; and grouting, to plug leaks and fill voids, became commonplace. In addition, the necessity of digging deep and wide open-cut excavations resulted in the improvement of steel sheeting shapes and interlocks, in the learning of the art of spacing strut lines, and in the continued perfection of earth digging and removing machines.

Between 1928 and 1930, the massive project of straightening the South Branch of the Chicago River was completed. This project resulted in advancements in dockwall construction which included the use of a sheet piling wall for an anchorage system and in the first experiment with jacking small diameter pipe under live railroad tracks being performed. Also, it gave earth movers a chance to improve their equipment developed earlier on the Sanitary and Ship and Cal-Sag Canal projects.

In 1937 through 1942, the Desplaines Street and Stewart Avenue Water Tunnels were constructed. The Desplaines Street Tunnel has a 13 ft. diameter and runs from Chicago Avenue (800N) to Harrison Street (600S) under Desplaines Street (640W). The Stewart Avenue Tunnel has a 10 ft. diameter and runs from 73rd (7300S) and State Street to 104th (10400S) and Stewart Avenue (400W). Both tunnels were located to have a full rock face and the methods used to mine and line these tunnels were similar to those so successfully used on the Chicago Avenue Land and Lake Tunnels. The improvements gained in the Sanitary District's Interceptor Sewer Program were of course utilized; the most important of which was the pnuematic placement of the concrete lining and the use of collapsible forms for supporting the lining.

In 1939 through 1943, the initial subway system was constructed. This system included the State Street Subway from Sheffield Avenue (1000W) and Willow Street (1700N) to 13th (1300S) and State Street; and the Dearborn Street Subway from Paulina Street (1700W) and Ellen Street (1300N) to Dearborn Street

(50W) and Van Buren Street (400S). The project consisted of one floated and sunk river crossing tube, of approximately one mile of 45 to 60 ft. wide open-cut excavation approximately 50 ft. deep, and of approximately six miles of either double or twin tube tunnel section. All the open-cut excavation required bracing and all the tunneling was done under 10 to 15 psi air pressure. Two of the six miles were excavated using a shield and the remaining four by the now perfected bench and staged heading method of hand mining. The shield was used in the area known as Chicago's Loop because the subsurface exploration program had determined that the invert soil would be too soft to support a steel rib and lagging temporary lining. The construction of this six miles of subway was the culmination of 75 years of tunneling technology within the Chicago area.

The shields had a circular shape with an outside diameter of 25'-1". They had a cylindrical length of 15'-6" and a back overhang (tailpiece) at the crown with a length of 4'-0" and a thickness of 2-1/2 inches. The shield had 6 adjustable openings at the face and was advanced by 24 jacks pushing against the already placed temporary lining. The temporary lining consisted of six segments of steel liner plate per circumferential section bolted together. Each section of liner plates had a cylindrical length of 33 inches and an internal diameter of 24'-9. The excavation prodedure consisted of advancing the shield for a distance of 33 inches, installing the 33 inch liner plate section under the tailpiece, and grouting the 3-1/2 inch annular space behind the previous liner plate section. As the liner plate section was bolted together, it was also bolted to the previous section. The grout was 3/8" pea gravel placed pneumatically. The shield tunnels were all of the double tube section. One 25 ft. diameter tunnel was excavated followed at least 200 ft. behind by another parallel 25 ft. diameter tunnel, which was separated from the first by 3 ft. to 8 ft. of clay. After both tunnels had passed through a station or cross-over area, the center column of clay was removed by hand mining in the same maner as described later herein.

The hand mined tunnels were advanced in a horseshoe shape approximately 21 ft. x 21 ft. The temporary lining consisted of a circumferential section of 12 inch wide liner plates supported at both ends by steel ribs. Each liner plate section had a length of 26 inches between the center lines of the ribs. The ribs were supported at the invert on 2 ft. x 2 ft. wood blocks. The heading was advanced by using three benches and four cutting faces with each bench extending between 3 and 8 ft. behind its respective face. Two methods of excavation were used:

1. The crown ribs were supported on the bench closest to the heading on temporary wooden blocks and tied to the previously placed ribs by bolted stiffeners. In this method, the majority of the invert face was

kept about 30 ft. behind the second face or last temporary crown rib support. This meant that side drifts having a height from the second bench to the invert level were needed along both perimeters in order to install the side ribs needed to extend the temporary lining support to the invert level. These side drifts had cutting faces on the same line as the second face and extended backwards to the fourth or invert face.

2. A "monkey drift" extending from the second face to approximately 4 ft. in front of the first or crown face was excavated at the level of the second bench. An 8 ft. long temporary H-beam was placed on the ground at the second bench level. This beam was used to temporarily support the crown ribs and, as long as the third bench was no longer than 8 ft., was very effective. The side ribs were installed in side drifts by bolting them to the temporary H-beam support and placing them on their wooden footings.

The hand mined tunnels were used to form both double tube sections and twin tube sections. As with the shield tunnels, one hand mined tunnel was excavated and then followed at least 200 ft. behind by another. In the double tube section, the two tunnels were separated by between 3 ft. and 8 ft. of clay; and, in station or cross-over areas, the center column of clay was removed by hand mining in the same four face-three bench method described above. In the twin tube section, the second tunnel used the first tunnel's near side rib to support its crown rib.

Regardless of the method used to advance the heading, excavation was performed using clay knives. After the clay was cut, it was hand dumped into hopper cars and transported back to the shaft by battery operated engines. In all cases, it was necessary to support the lead tunnel by horizontal struts when the following tunnel's heading passed by. Also, obviously, no permanent concrete lining was placed until after both tunnels had been excavated. This project saw the first use of pumpcrete for the permanent lining. Ready-mix concrete was poured and pumped from the ground surface in the formwork. By trial and error, it was learned that this could be done effectively for about 250 ft. in either direction from the pumping shaft.

The open-cut excavations were supported by both steel sheeting and soldier beam-lagging systems. The soldier beam-lagging techniques learned here were the forerunners of the principles used to design and construct the cut and cover system for the Kennedy Rapid Transit Project built in 1967. As in 1967, the biggest problem was to successfully support existing utilities without damaging them or surrounding structures in the process.

The installation of this initial subway system did necessitate extensive underpinning projects. The most complicated involved the Cinestage Theatre at the corner of Dearborn Street and Lake Street. This building had to be underpinned because the subway went under it; and, the underpinning had to be located so as not to interfere with the shield tunneling methods. The underpinning was done successfully with caissons and concrete encased steel grade beams. This kind of system was the one generally used in the initial subway system installation.

The State Street-Chicago River subway crossing was constructed in much the same way as the LaSalle Street Vehicular Tunnel river crossing was in 1910. The tube section was constructed out of steel and concrete in dry dock and floated to the proper spot in the Chicago River and sunk into a pre-excavated trench. The trenching for the crossing was done by a 3 yard clam bucket operating off a derrick. Before the tube section was sunk, the trench bottom was leveled with sand, the concrete landing pads were installed and steel sheeting was driven to form a cutoff wall at the end of each tube section for later use in conjunction with the cofferdams. The sinking of the tube section was controlled by cables holding it up and weights added to and on the outside of the tube section forcing it down. After the tube section was sunk, the steel sheet piling cofferdam was completed from each shore.

The completion of the initial subway system marked the end of Chicago's stabilizing era. While very little completely new underground construction innovations were forthcoming in this era, remarkable progess in improving existing techniques had occurred.

CHICAGO'S MODERNIZING ERA (POST WORLD WAR II)

This era marks a change in the underground construction industry as significant as the change in Chicago's sky line. While the principles which guided construction techniques between the World Wars remained pertinent, their implementation became mechanized after World War II.

In the early 1950's, rotary auger machines were used to excavate caissons and their success has greatly influenced the foundation industry. At the same time, pile driving hammers and extractors have beome many times more efficient than their initial models. These modernized foundation techniques coupled with architectural high-rise innovations have seen Chicago grow considerably taller than before World War II.

In 1947 and 1948, the secondary portion of the initial subway system was completed. This continued the Dearborn Street Subway by turning west up Congress Street to the west bank of the South Branch of the Chicago River. This very short length of subway included a double tube section, involved a tunneled river crossing, and a single tube turnaround tunnel east of the River. Tunneling was done exactly as before during the initial subway installation and the most interesting aspect of the project was the underpinning of the LaSalle Street Railroad Station. The underpinning was done with very little interruption to railroad operations and again involved a caisson-grade beam system.

In the early 1950's, construction of the Auxiliary Outlet Sewer System was started. This System involves the replacement of some small collector sewers with much larger collectors. The intent is to provide storm relief by increasing the carrying capacity, thereby allowing a faster discharge. To date, approximately 200 miles of sewer replacement has been accomplished. These new sewers are generally between 10 ft. and 22 ft. in diameter and, while shallower than the Sanitary District's Interceptors, they are deeper than those they are replacing. During construction of these sewers, several advancements in construction techniques have occurred:

1. In 1960, the first tunneling machine was successfully used. This machine cut a 5 ft. diameter bore through a tough to hard silty clay.

2. In 1966, a tunneling machine was successful in cutting a heading having a soft compressible clay crown and a hard clay invert. This was also the first time that expanding ribs were used in an attempt to cure the tailpiece-annular space problem. The attempt was partially unsuccessful, but it has left room for hope.

3. In 1973, tunneling was accomplished by the jacked pipe method. For small diameter projects, this procedure of always providing support by pressure grouted bentonite has proven very successful.

4. In the 1960's, the trench excavating method using a "sand-box" was perfected. This was a significant advance in providing temporary shoring where squeezing or flowing soil is not a problem.

5. Throughout this era, newer and smaller but yet more powerful earth digging machines have been built. This has had the effect of allowing more and faster machine excavation within a braced trench.

In 1954, the Dearborn Subway System was finally completed when the double tube heading which had crossed under the South Branch of the Chicago River in 1947 was extended into the Eisenhower Expressway median strip. The excavation was done under 10 psi air pressure and was done by hand mining methods similar to those used throughout the initial subway system. This project involved the underpinning of the Post Office Building which was supported on belled caissons bearing at about mid-height of the tube. The underpinning was unique in that the original caissons, which had to be removed during the tube advancement, were re-established bearing on the new subway tubes after being temporarily shored at the ground surface some 60 ft. higher.

Between 1955 and 1960, the last of Chicago's massive water tunnel system was completed. The 16 ft. diameter Wilson Avenue Connecting Tunnel, the 16 ft. diameter 79th Street Tunnel, and the Columbus Avenue Tunnel were all constructed in a full rock face heading. Excavation was accomplished by the full face blasting method with drilling done by "jumbos". Excavation went well except in the 79th Street Tunnel where about 7,000 ft. of weathered rock and clay seams necessitated an abundance of temporary steel rib supports.

In 1967, about 2-1/2 miles of new subway construction occurred when the Dearborn Subway System was extended into the Kennedy Expressway median strip. This installation was performed in a cut and cover operation where a soldier beam-lagging earth retention system served a dual purpose by incorporating deck beams in the bracing system to support a wood plank roadway over the excavation. The procedure used to install the soldier beams was to prebore oversize holes, to insert the soldier beam in the oversize hole, to concrete in the portion of soldier beam below the proposed invert level, and to fill the remainder of the hole with a weak grout. After the soldier piles were in place, the interior area was excavated about 5 feet below the top of the soldier beams and the deck beams placed on top of the soldier piles. The excavation sides between soldier beams were lagged by the "contact sheeting" method and the excavation carried deeper so that the next strut line could be installed. Theoretically by using proper machinery and controlling the excavation and bracing installation operations, the remainder of the opencut could now be made under cover of a temporary roadway. Unfortunately, it did not work out this way; however, it was possible to place the cover before concreting was performed.

Between 1967 and 1972, the initial portion of the Chicago Underflow and Storm Water Storage System was constructed. This system involves large diameter sewers located at about elev. -225 (C.C.D.) which will eventually empty into a very large sewer to be located under the Chicago River and Sanitary and Ship Canal. This large diameter sewer system is to replace the above mentioned waterways as discharge outlets during heavy storms and should be the last link in effectively preventing pollution of Chicago's lakes and rivers.

The Lawrence Avenue Underflow Sewer was the first rock tunnel in the area to be mined by machine. A 13 ft. diameter "mole" was used to mine the entire 4 mile length and about half of this was enlarged by blasting to a 17 ft. internal diameter. Even though many problems developed in the "mole", the desired

Even though many problems developed in the "mole", the desired result of a smooth enough rock-bore to eliminate the need for a concrete lining was accomplished in the 13 ft. diameter section. In order to correct local deficiencies, rock bolts and wire mesh were installed and coated with gunite; however, this was accomplished in such a manner to prevent any disturbance to the hydraulics of the system.

The Crawford Avenue Underflow Sewer was also constructed by "mole" about a year after the Lawrence Avenue Underflow Sewer. A 17 ft. diameter tunnel was bored and improvements in the "mole" made this project highly successful. Local deficiencies were corrected as before.

At the same time the Crawford Avenue Underflow Sewer was started, the LaGrange-Brookfield Underflow Sewer was also started. This 14 ft. diameter tunnel was also bored by "mole" and the mining was again very successful. This project did run into difficulty due to its proximity to an abandoned quarry which had filled with about a 200 ft. head of water. After it became evident that the water inflow problem existed, mining was stopped and the tunnel bulkheaded. A massive pressure grouting project from the ground surface was started in an effort to fill the joints and interstices in the rock to stop the water inflow. The grouting program was successful and mining was continued utilizing a small diameter probe hole in front of the heading as a checking device. Local deficiencies were corrected as before.

At some future time, it is possible that the 30 to 40 year period following World War II may be looked upon as being as significant as the Pre-World War I era is today. The mechanization which has occurred in the construction industry has been proven to work; however, there is considerable room for improvement. Caisson auger methods presently do not support the sides of their earth excavations. Pile driving methods cause considerable damage to surrounding property because of the resulting earth vibrations and/or heave. Earth tunneling machines cause settlement of overlying structures. Rock tunneling machines are more effective than existing muck removal systems. The "sand-box" is not effective in squeezing or flowing soil. It is said that Chicago is a city that works. This is not accomplished by its leaders, including those in its engineering and construction professions, resting on their laurels. Improvements will be made to solve existing as well as new problems; and, if history is a guide, many of those improvements will be made in Chicago.

CONCLUSIONS

Underground construction has played a tremendous role in Chicago's development. In turn, the techniques tried, learned, and perfected in Chicago's subsurface have become the bylaws of the industry.

In the upcoming Central Area Transit Project, it will be necessary for new techniques to be tried in order to solve the problems which will occur. The problems will not all be the same ones as in previous years, but many will be repetitions of those which have not always been successfully solved in the past. It is even more important in today's busy age to prevent damage to overlying and/or adjacent pavements and structures. If the Eisenhower Expressway crossing, which will probably be located in the water-bearing granular hardpan, results in a pavement failure, it could affect trucking all over the nation. If utilities have to be taken out of service in Chicago's Loop area, disaster could befall those affected.

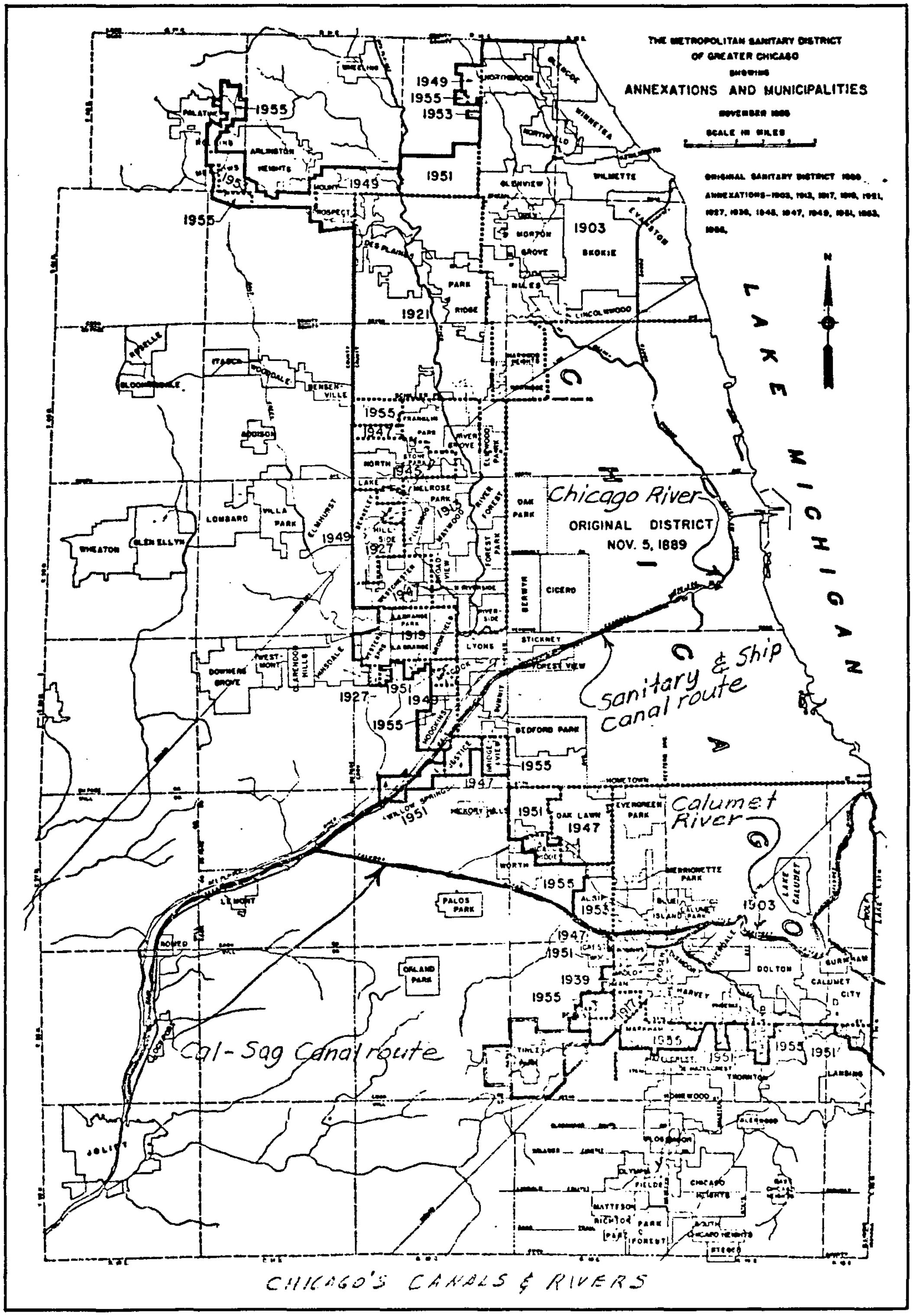
While it is beyond the scope of this paper to solve the upcoming problems, it is felt that the information contained herein can be a useful guide to those charged with that future responsibility.

APPENDIX A

ILLUSTRATIONS

ON

CHICAGO'S WATERWAYS



CHICAGO'S CANALS & RIVERS

APPENDIX B

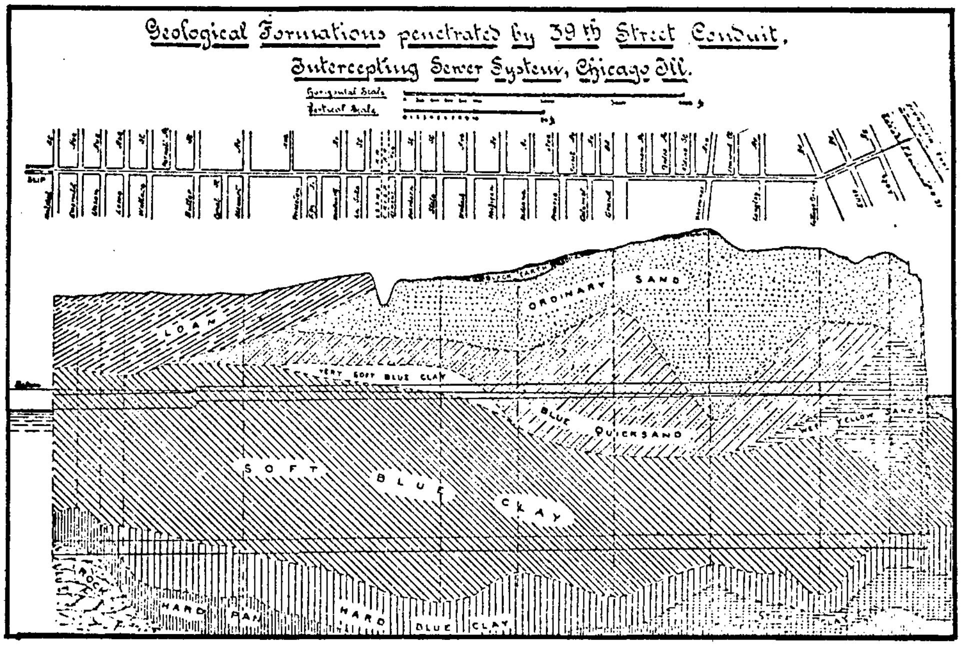
ILLUSTRATIONS

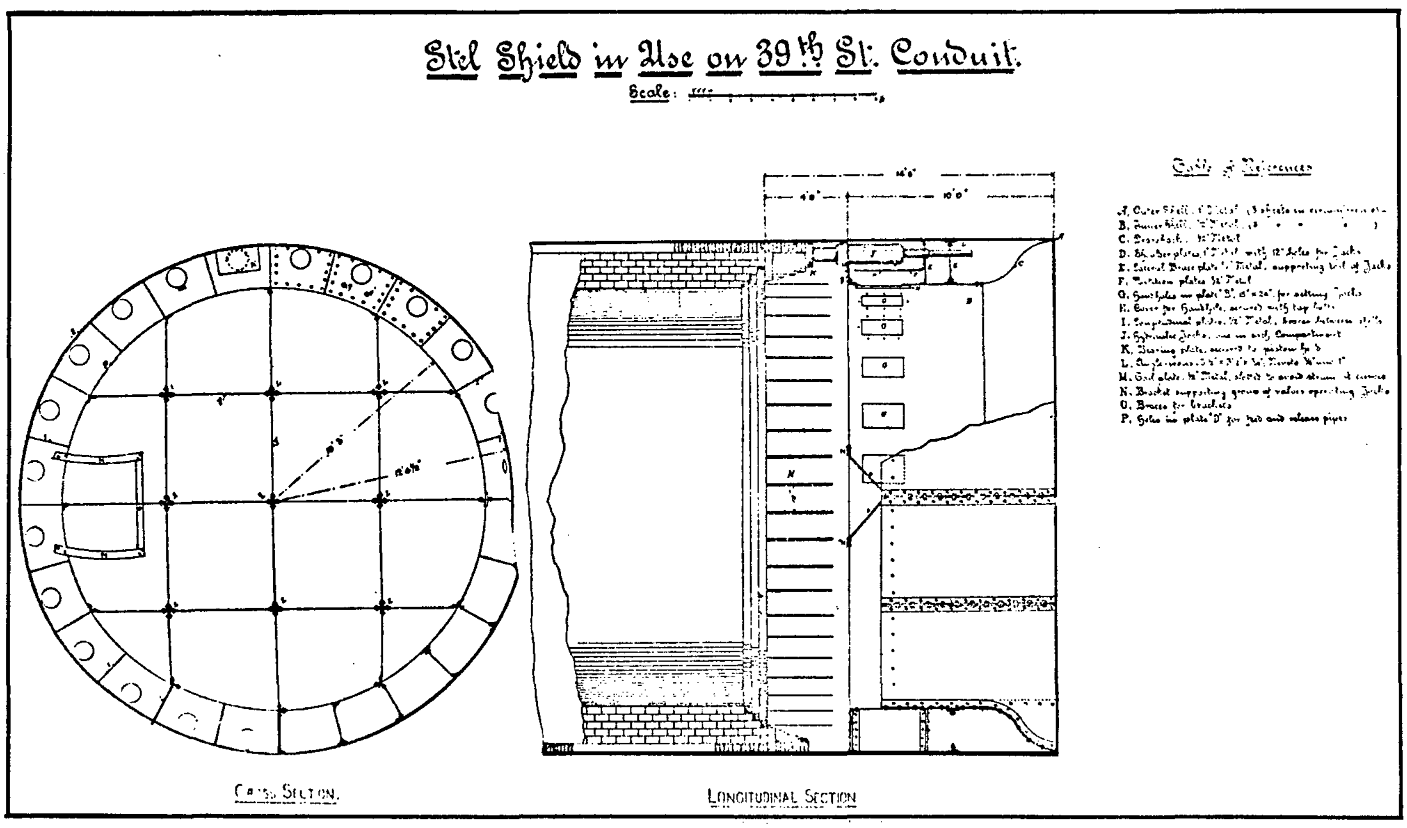
ON

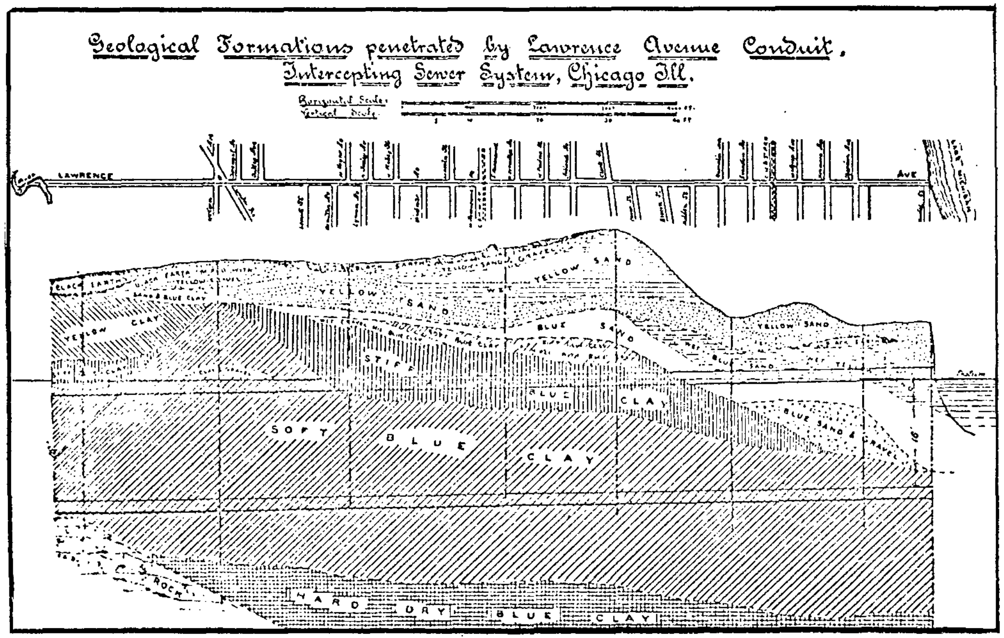
CHICAGO'S INTERCEPTING SEWER SYSTEM

&

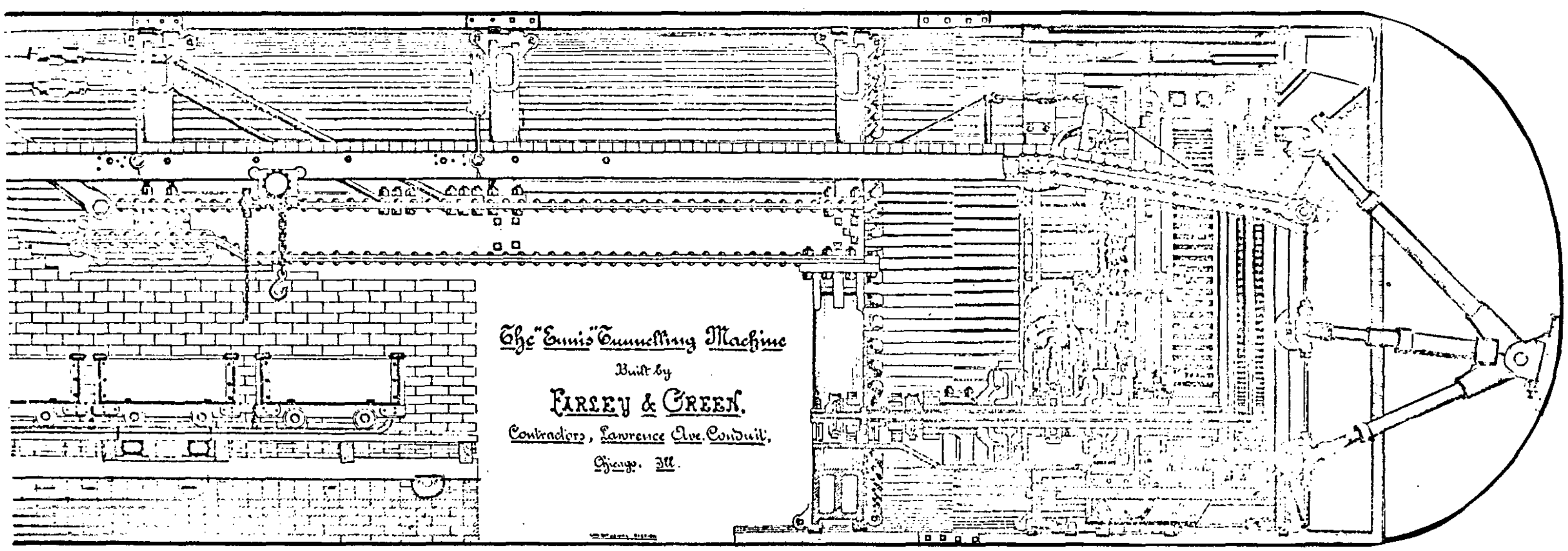
CHICAGO'S AUXILIARY OUTLET SEWER SYSTEM


Geological Formations penetrated by 39th Street Conduit.
Intercepting Sewer System, Chicago Ill.
ORDINARY SAND
LOAM
VERY SOFT BLUE CLAY
BLUE QUICKSAND
YELLOW SAND
SOFT BLUE CLAY
HARD PAN
HARD BLUE CLAY

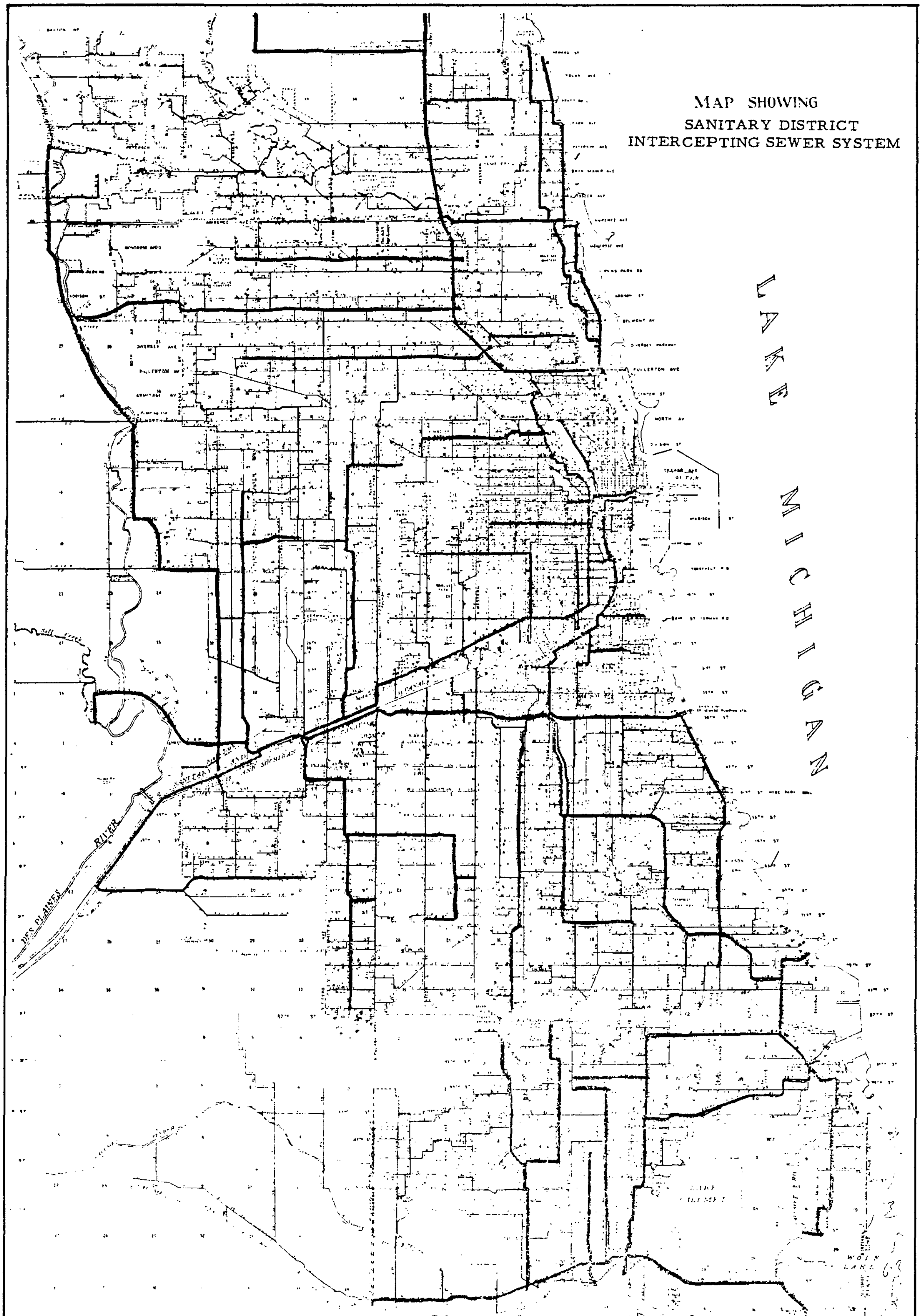

Steel Shield in Use on 39th St. Conduit.
Scale:
Table of References
Cross Section.
Longitudinal Section

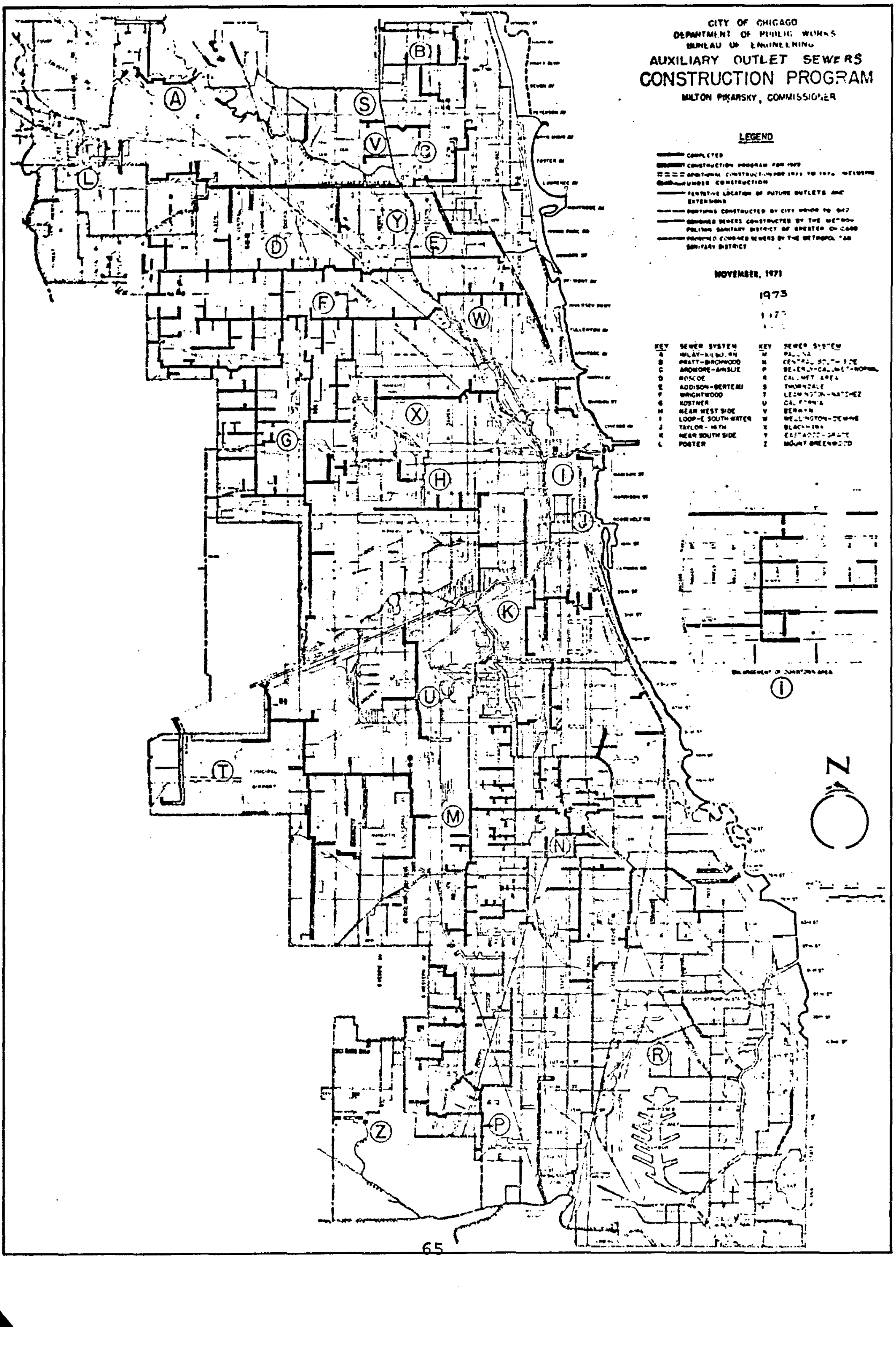


Geological Formations penetrated by Lawrence Avenue Conduit,
Intercepting Sewer System, Chicago Ill.
LAWRENCE
AVE
YELLOW SAND
BLUE SAND
YELLOW CLAY
STIFF
BLUE CLAY
SOFT BLUE CLAY
HARD DRY BLUE CLAY
ROCK



The "Ennis" Tunnelling Machine
Built by
FARLEY & GREEN.
Contractors, Lawrence Ave. Conduit,
Chicago. Ill.


MAP SHOWING
SANITARY DISTRICT
INTERCEPTING SEWER SYSTEM
LAKE MICHIGAN
DES PLAINES
RIVER



CITY OF CHICAGO
DEPARTMENT OF PUBLIC WORKS
AUXILIARY OUTLET SEWERS
CONSTRUCTION PROGRAM
MILTON PIKARSKY, COMMISSIONER
LEGEND
COMPLETED
UNDER CONSTRUCTION
NOVEMBER, 1971
1973
KEY
SEWER SYSTEM
A
B
PRATT-BIRCHWOOD
C
ARDMORE-AINSLIE
D
ROSCOE
E
ADDISON-BERTEAU
F
WRIGHTWOOD
G
KOSTNER
H
NEAR WEST SIDE
I
LOOP-E SOUTH WATER
J
K
NEAR SOUTH SIDE
L
FOSTER
KEY
SEWER SYSTEM
M
N
P
R
CALUMET AREA
S
THORNDALE
T
U
V
W
X
Y
Z
A
B
C
D
E
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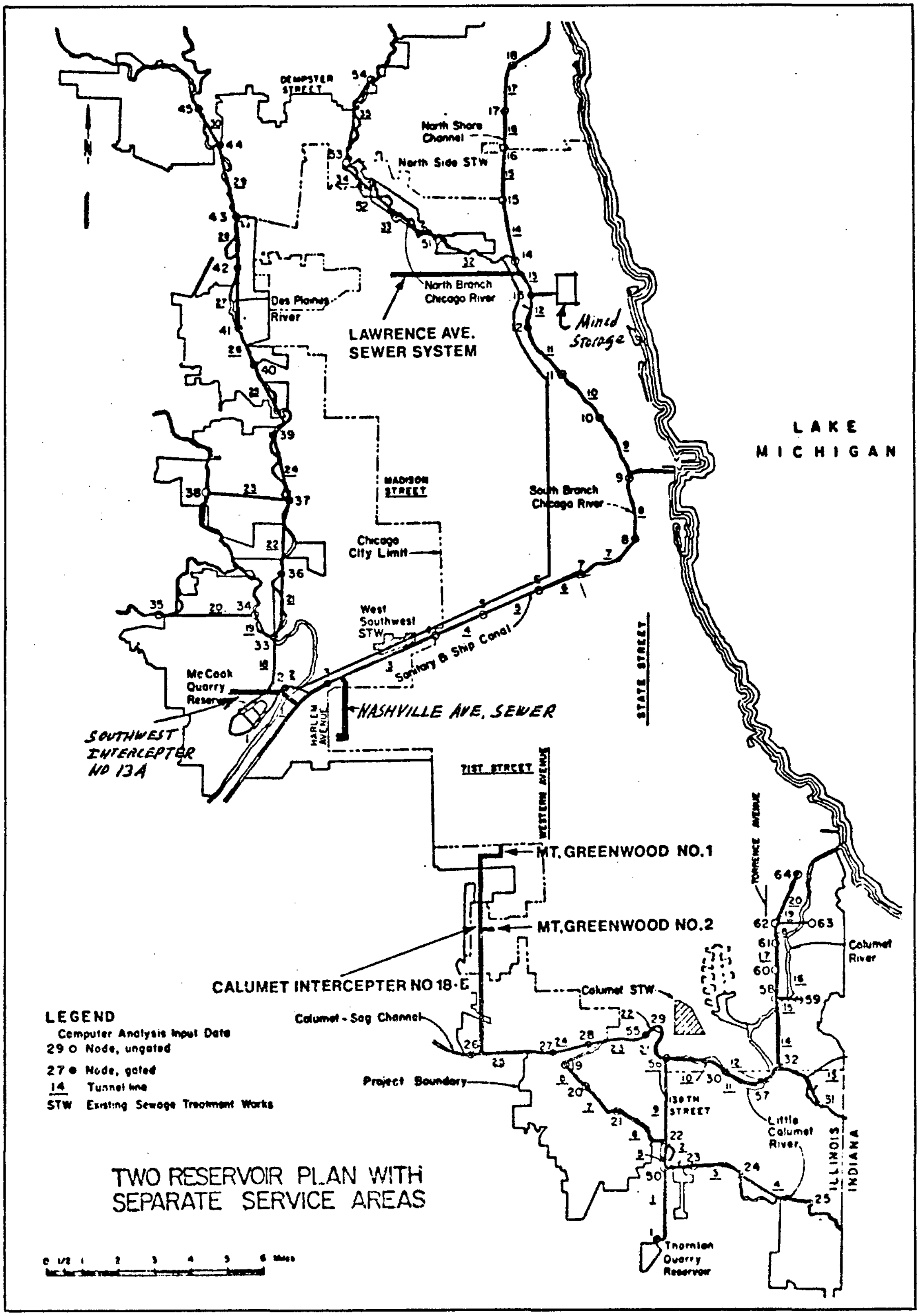
N

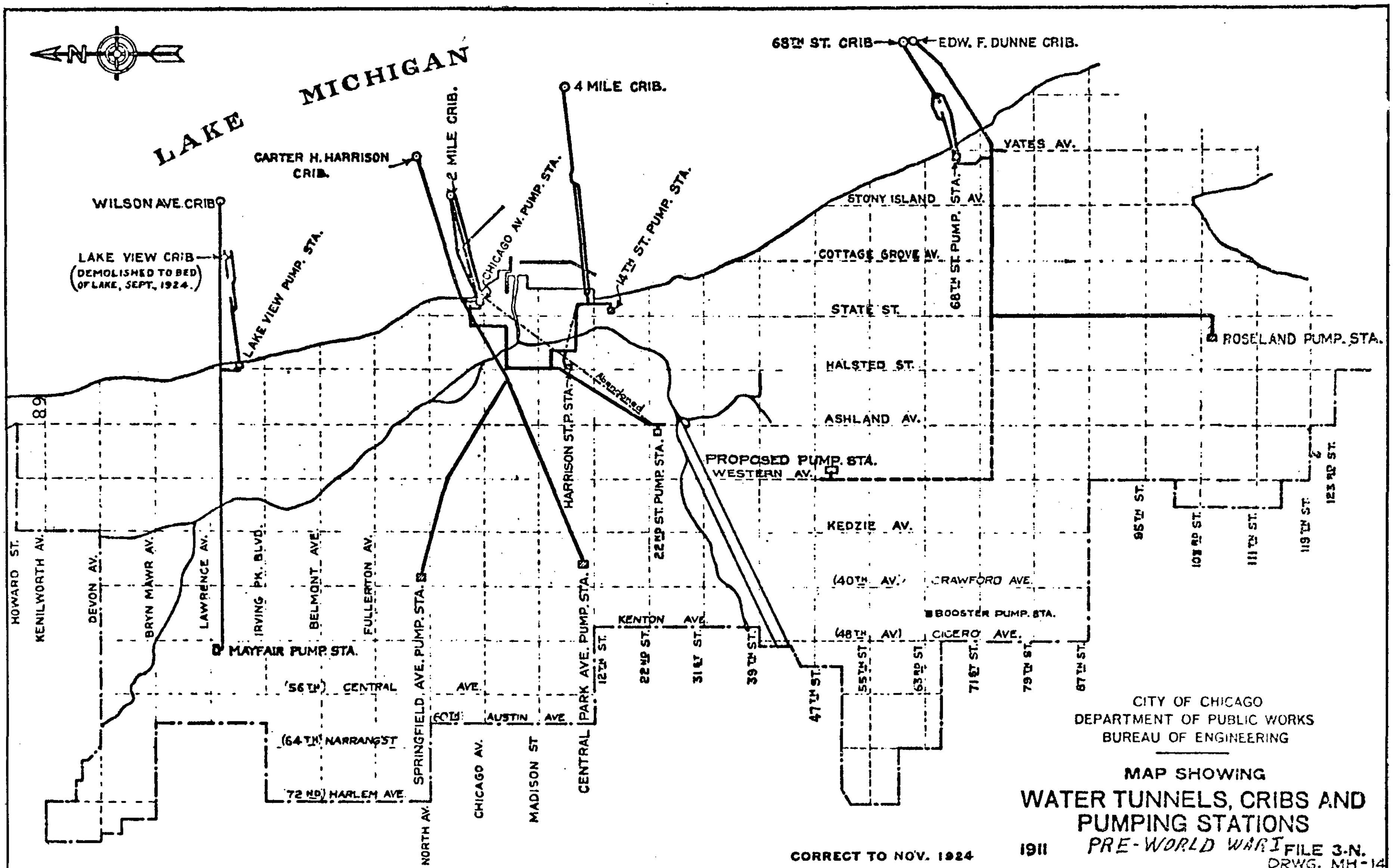

LAKE MICHIGAN
DEMPSTER STREET
North Shore Channel
North Side STW
North Branch Chicago River
Des Plaines River
LAWRENCE AVE. SEWER SYSTEM
Mined Storage
MADISON STREET
South Branch Chicago River
Chicago City Limit
West Southwest STW
Sanitary & Ship Canal
McCook Quarry Reservoir
HARLEM AVENUE
NASHVILLE AVE. SEWER
SOUTHWEST INTERCEPTER NO 13A
STATE STREET
71ST STREET
WESTERN AVENUE
TORRENCE AVENUE
MT. GREENWOOD NO.1
MT. GREENWOOD NO.2
CALUMET INTERCEPTER NO 18-E
Calumet STW
Calumet River
Calumet - Sag Channel
Project Boundary
130TH STREET
Little Calumet River
ILLINOIS
INDIANA
Thornton Quarry Reservoir
LEGEND
Computer Analysis Input Data
29 O Node, ungated
27 ● Node, gated
14 Tunnel line
STW Existing Sewage Treatment Works
TWO RESERVOIR PLAN WITH SEPARATE SERVICE AREAS
0 1/2 1 2 3 4 5 6 Miles

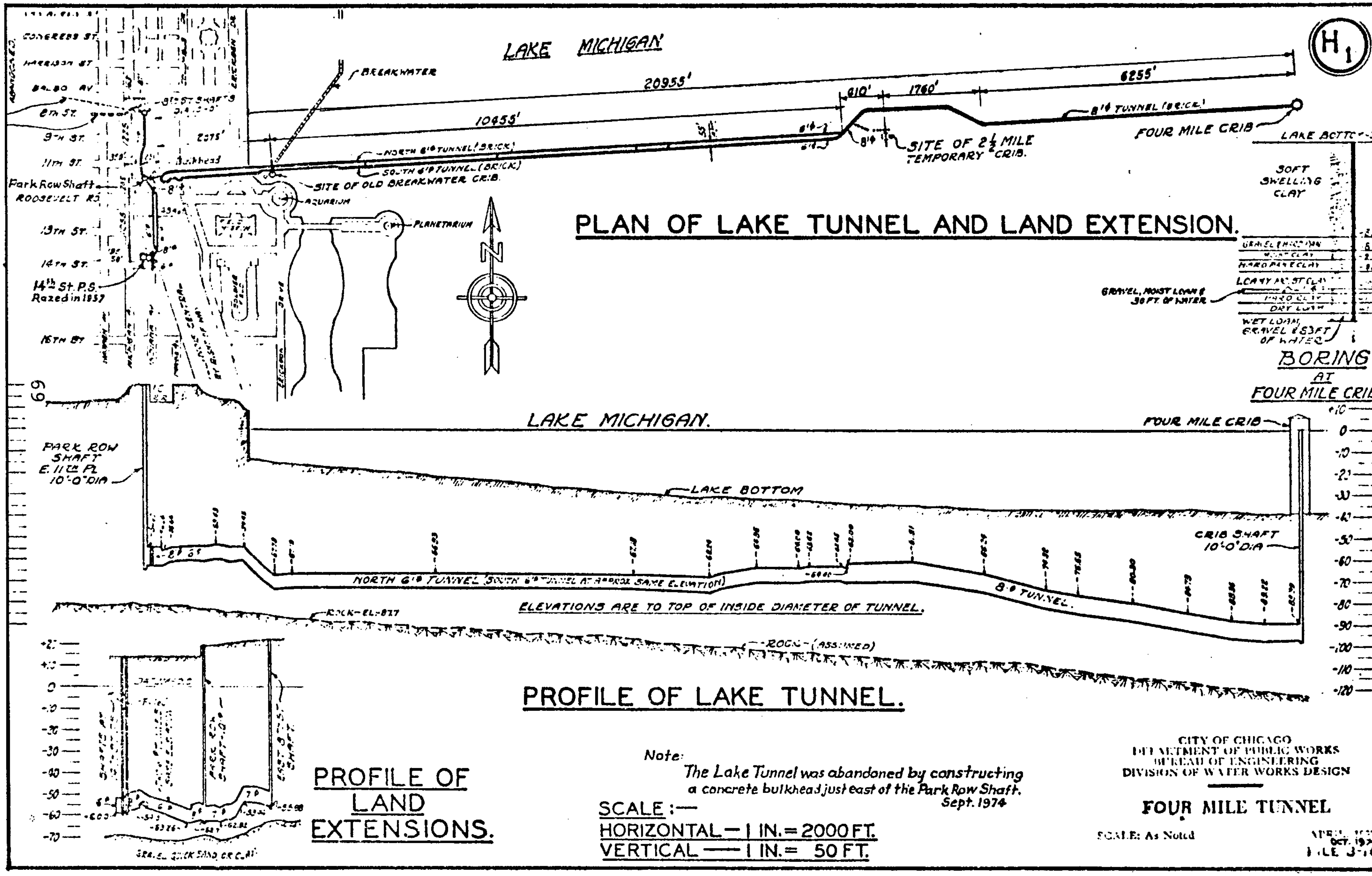
APPENDIX C

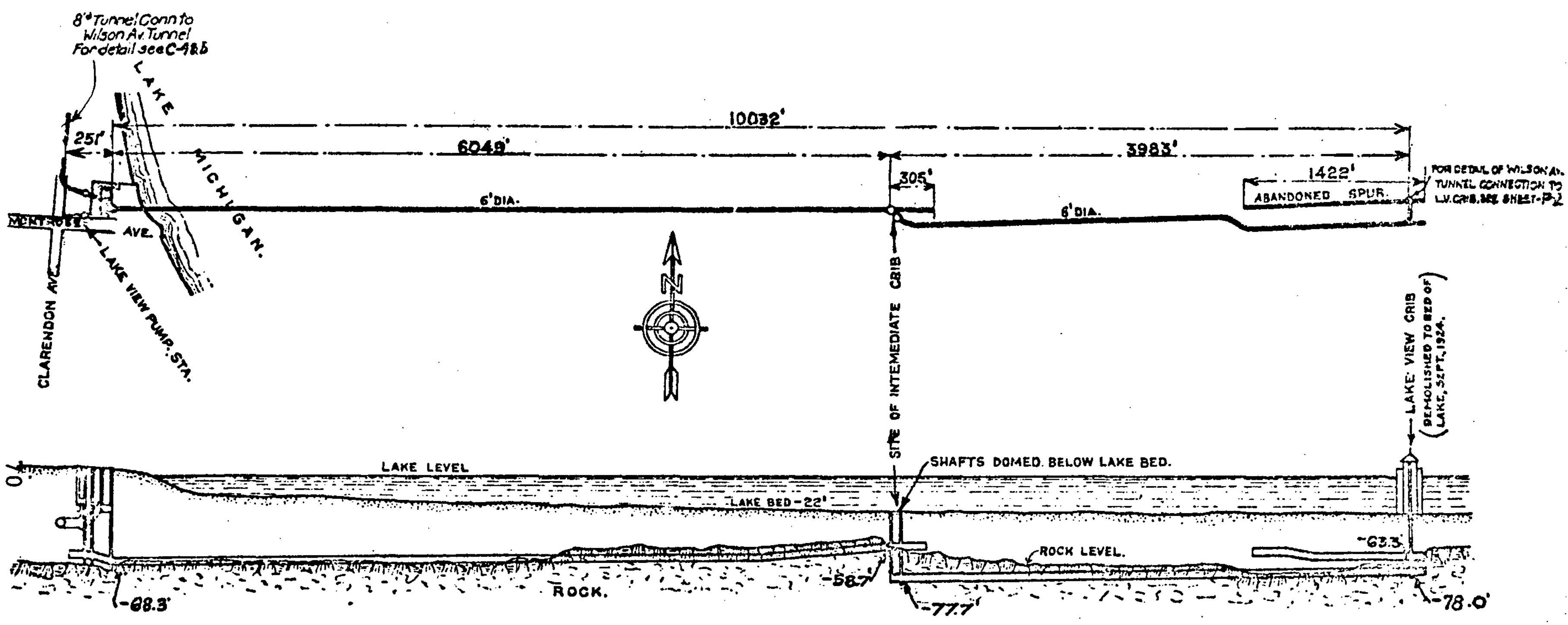
ILLUSTRATIONS

ON

CHICAGO'S WATER TUNNEL SYSTEM


N
LAKE MICHIGAN
WILSON AVE. CRIB
LAKE VIEW CRIB (DEMOLISHED TO BED OF LAKE, SEPT., 1924.)
LAKE VIEW PUMP. STA.
CARTER H. HARRISON CRIB.
2 MILE CRIB.
CHICAGO AV. PUMP. STA.
4 MILE CRIB.
14TH ST. PUMP. STA.
68TH ST. CRIB
EDW. F. DUNNE CRIB.
68TH ST. PUMP. STA.
YATES AV.
STONY ISLAND AV.
COTTAGE GROVE AV.
STATE ST.
HALSTED ST.
ASHLAND AV.
WESTERN AV.
KEDZIE AV.
(40TH AV.) CRAWFORD AVE.
BOOSTER PUMP. STA.
(48TH AV) CICERO AVE.
KENTON AVE.
ROSELAND PUMP. STA.
HARRISON ST. P. STA.
Abandoned
22ND ST. PUMP. STA.
PROPOSED PUMP. STA.
MAYFAIR PUMP. STA.
SPRINGFIELD. AVE. PUMP. STA.
CENTRAL PARK AVE. PUMP. STA.
('56TH) CENTRAL AVE.
60TH AUSTIN AVE.
(64TH) NARRANGST
(72ND) HARLEM AVE.
HOWARD ST.
KENILWORTH AV.
DEVON AV.
BRYN MAWR AV.
LAWRENCE AV.
IRVING PK. BLVD.
BELMONT AVE.
FULLERTON AV.
NORTH AV.
CHICAGO AV.
MADISON ST
12TH ST.
22ND ST.
31ST ST.
39TH ST.
47TH ST.
55TH ST.
63RD ST.
71ST ST.
79TH ST.
87TH ST.
95TH ST.
103RD ST.
111TH ST.
119TH ST.
123RD ST.
CITY OF CHICAGO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING
MAP SHOWING
WATER TUNNELS, CRIBS AND PUMPING STATIONS
1911 PRE-WORLD WAR I
CORRECT TO NOV. 1924
FILE 3-N.
DRWG. MH-14


H1
LAKE MICHIGAN
20955'
10455'
6255'
910'
1760'
BREAKWATER
NORTH 6'0 TUNNEL (BRICK)
SOUTH 6'0 TUNNEL (BRICK)
SITE OF OLD BREAKWATER CRIB
8'0 TUNNEL (BRICK)
SITE OF 2½ MILE TEMPORARY CRIB
FOUR MILE CRIB
LAKE BOTTOM
AQUARIUM
PLANETARIUM
CONGRESS ST.
HARRISON ST.
BALBO AV.
8TH ST.
9TH ST.
11TH ST.
Park Row Shaft
ROOSEVELT RD.
13TH ST.
14TH ST.
14th St. P.S. Razed in 1957
16TH ST.
PLAN OF LAKE TUNNEL AND LAND EXTENSION.
SOFT SWELLING CLAY
GRAVEL, MOIST LOAM & 50 FT. OF WATER
WET LOAM, GRAVEL & 53 FT. OF WATER
BORING AT FOUR MILE CRIB
LAKE MICHIGAN.
PARK ROW SHAFT E. 11TH PL. 10'-0" DIA.
LAKE BOTTOM
CRIB SHAFT 10'-0" DIA
NORTH 6'0 TUNNEL (SOUTH 6'0 TUNNEL AT APPROX. SAME ELEVATION)
8'0 TUNNEL
ELEVATIONS ARE TO TOP OF INSIDE DIAMETER OF TUNNEL.
ROCK-EL-83.7
ROCK-(ASSUMED)
PROFILE OF LAKE TUNNEL.
PROFILE OF LAND EXTENSIONS.
Note:
The Lake Tunnel was abandoned by constructing a concrete bulkhead just east of the Park Row Shaft. Sept. 1974
SCALE:—
HORIZONTAL — 1 IN. = 2000 FT.
VERTICAL — 1 IN. = 50 FT.
CITY OF CHICAGO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING
DIVISION OF WATER WORKS DESIGN
FOUR MILE TUNNEL
SCALE: As Noted



Pumping Station Shaft, Bottom of Invert −63.7 ft.
Shore Shaft " " −68.33 "
Intermediate Crib Shaft " " −58.7 "
Intermediate Outer Crib Shaft, Bot. of Invert −77.7 "
Outer Crib Shaft, Bottom of Invert, Spur. −63.3 "
" " " " " " Main Tun. −78.0"

CORRECT TO NOV., 1924

CITY OF CHICAGO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING

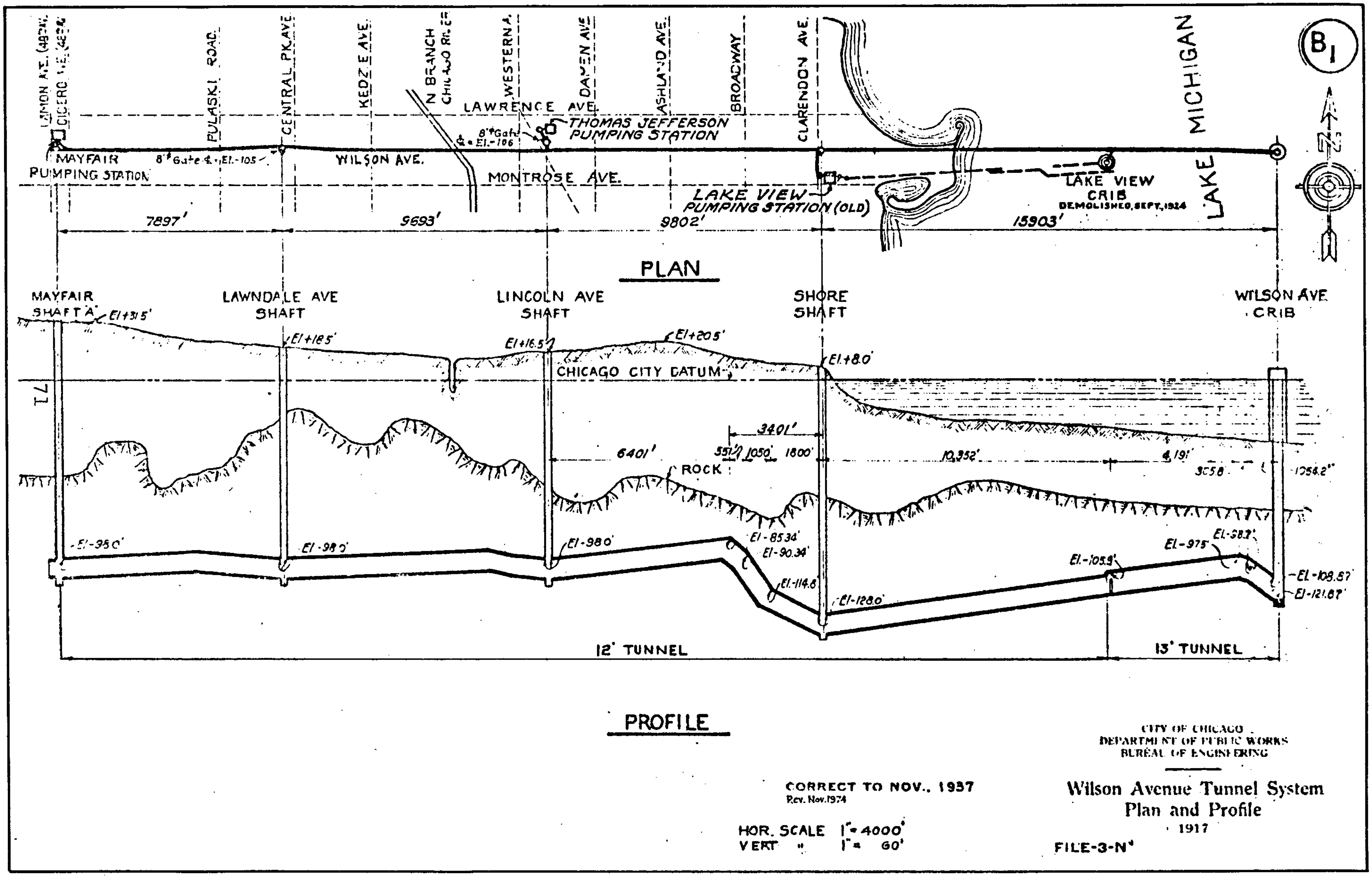
PLAN AND PROFILE OF

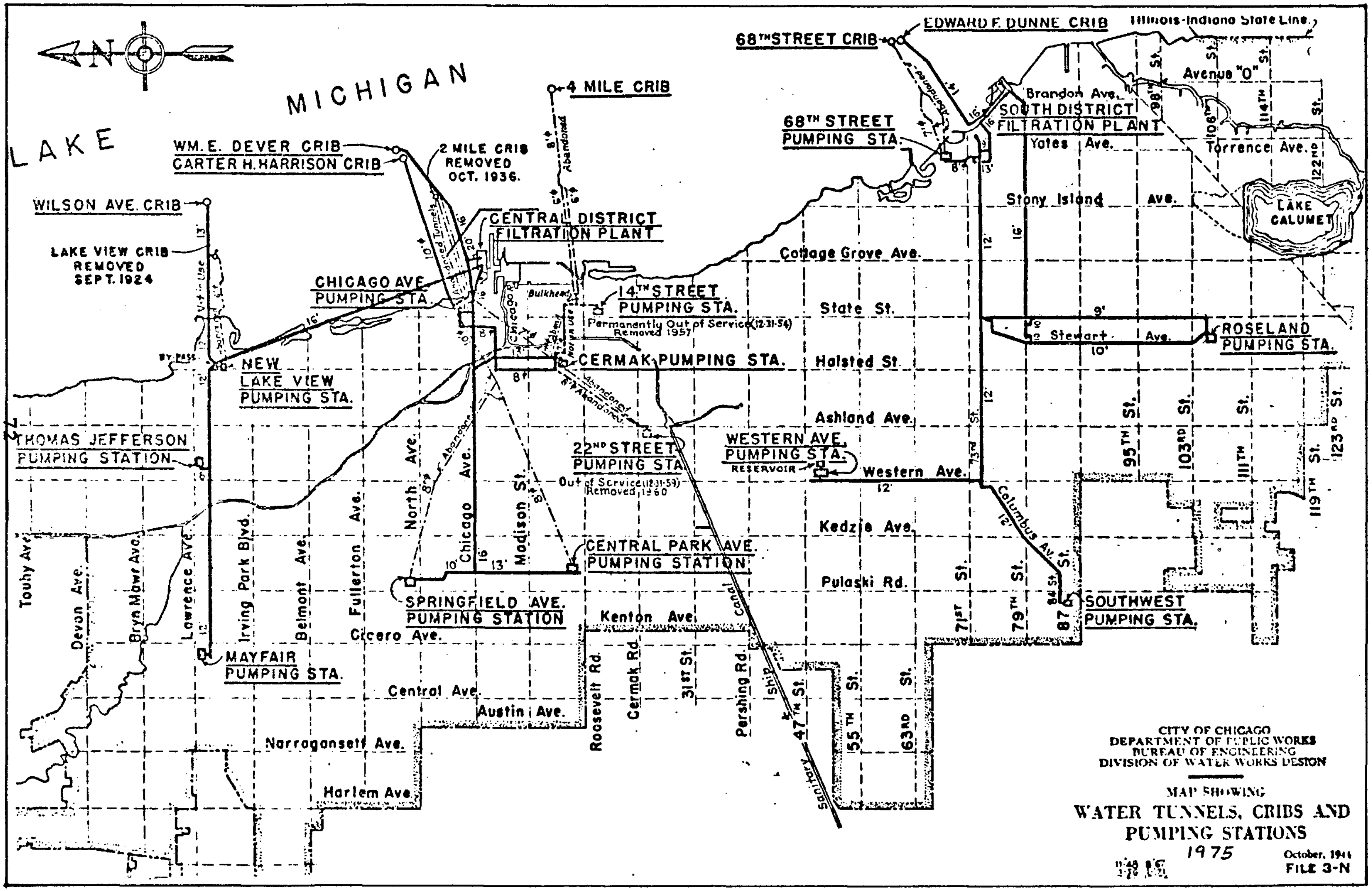
LAKE VIEW TUNNEL

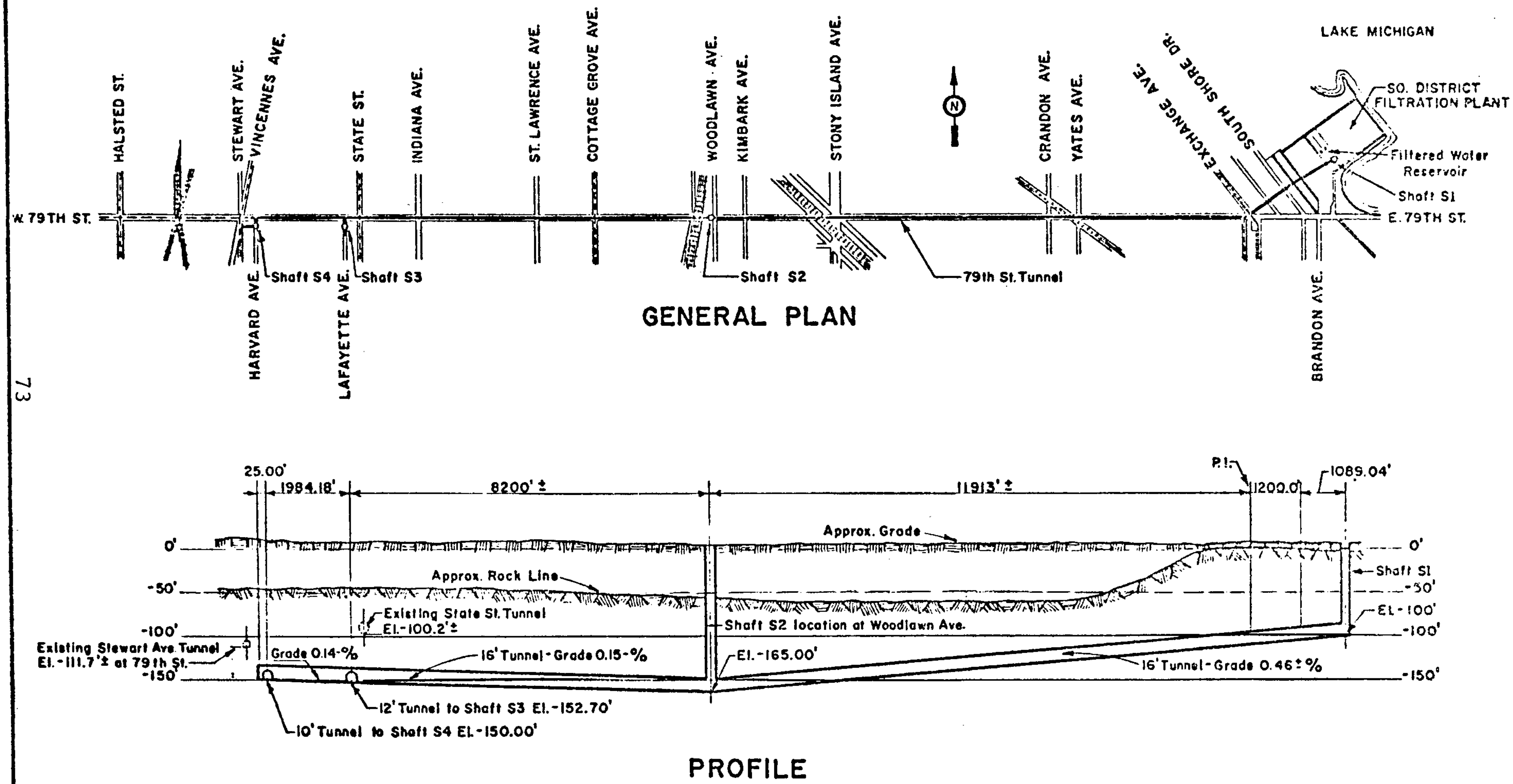
1911 FILE 3-N.

SCALES.

1000 500 0 1000 2000	HORIZONTAL.
100 50 0 100 200	VERTICAL.


B1
N
LAMON AVE. (4824W)
CICERO AVE. (4824W)
PULASKI ROAD
CENTRAL PK. AVE.
KEDZIE AVE.
N BRANCH CHICAGO RIVER
WESTERN A.
DAMEN AVE.
ASHLAND AVE.
BROADWAY
CLARENDON AVE.
LAWRENCE AVE.
THOMAS JEFFERSON PUMPING STATION
8" Gate & El.-106
MAYFAIR PUMPING STATION
8" Gate & El.-105
WILSON AVE.
MONTROSE AVE.
LAKE VIEW PUMPING STATION (OLD)
LAKE VIEW CRIB
DEMOLISHED, SEPT. 1924
LAKE MICHIGAN
7897'
9693'
9802'
15903'
PLAN
MAYFAIR SHAFT A
El +31.5'
LAWNDALE AVE SHAFT
El +18.5'
LINCOLN AVE SHAFT
El +16.5'
El +20.5'
CHICAGO CITY DATUM
SHORE SHAFT
El +8.0'
WILSON AVE CRIB
3401'
6401'
551/2
1050'
1800'
10,352'
4,191'
3058
1056.2"
ROCK
El -98.0'
El -98.0'
El -98.0'
El -85.34'
El -90.34'
El -114.8'
El -128.0'
El -105.9'
El -97.5'
El -98.2'
El -108.87'
El -121.87'
12' TUNNEL
13' TUNNEL
PROFILE
CORRECT TO NOV., 1937
Rev. Nov. 1974
HOR. SCALE 1" = 4000'
VERT " 1" = 60'
CITY OF CHICAGO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING
Wilson Avenue Tunnel System
Plan and Profile
1917
FILE-3-N'


N
LAKE MICHIGAN
WILSON AVE. CRIB
LAKE VIEW CRIB REMOVED SEPT. 1924
WM. E. DEVER CRIB
CARTER H. HARRISON CRIB
2 MILE CRIB REMOVED OCT. 1936.
4 MILE CRIB
68TH STREET CRIB
EDWARD F. DUNNE CRIB
Illinois-Indiana State Line.
CENTRAL DISTRICT FILTRATION PLANT
CHICAGO AVE PUMPING STA.
14TH STREET PUMPING STA.
Permanently Out of Service (12-31-56) Removed 1957
CERMAK PUMPING STA.
NEW LAKE VIEW PUMPING STA.
THOMAS JEFFERSON PUMPING STATION
22ND STREET PUMPING STA.
Out of Service (12-31-59) Removed 1960
WESTERN AVE. PUMPING STA.
RESERVOIR
CENTRAL PARK AVE. PUMPING STATION
SPRINGFIELD AVE. PUMPING STATION
MAYFAIR PUMPING STA.
68TH STREET PUMPING STA.
SOUTH DISTRICT FILTRATION PLANT
ROSELAND PUMPING STA.
SOUTHWEST PUMPING STA.
LAKE CALUMET
Abandoned
Abandoned Tunnels
Bulkhead
Chicago
By-Pass
Brandon Ave.
Avenue "O"
Torrence Ave.
Yates Ave.
Stony Island Ave.
Cottage Grove Ave.
State St.
Stewart Ave.
Halsted St.
Ashland Ave.
Western Ave.
Kedzie Ave.
Pulaski Rd.
Columbus Av.
Kenton Ave.
Cicero Ave.
Central Ave.
Austin Ave.
Narragansett Ave.
Harlem Ave.
Touhy Ave.
Devon Ave.
Bryn Mawr Ave.
Lawrence Ave.
Irving Park Blvd.
Belmont Ave.
Fullerton Ave.
North Ave.
Chicago Ave.
Madison St.
Roosevelt Rd.
Cermak Rd.
31ST St.
Pershing Rd.
47TH St.
55TH St.
63RD St.
71ST St.
73rd St.
79TH St.
84 St.
87
95TH St.
98TH St.
103RD St.
106TH St.
111TH St.
114TH St.
119TH St.
122ND St.
123RD St.
Sanitary Ship Canal
CITY OF CHICAGO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF ENGINEERING
DIVISION OF WATER WORKS DESIGN
MAP SHOWING
WATER TUNNELS, CRIBS AND PUMPING STATIONS
1975
October, 1944
FILE 3-N



79TH STREET TUNNEL
GENERAL PLAN AND PROFILE

APPENDIX D

ILLUSTRATIONS

ON

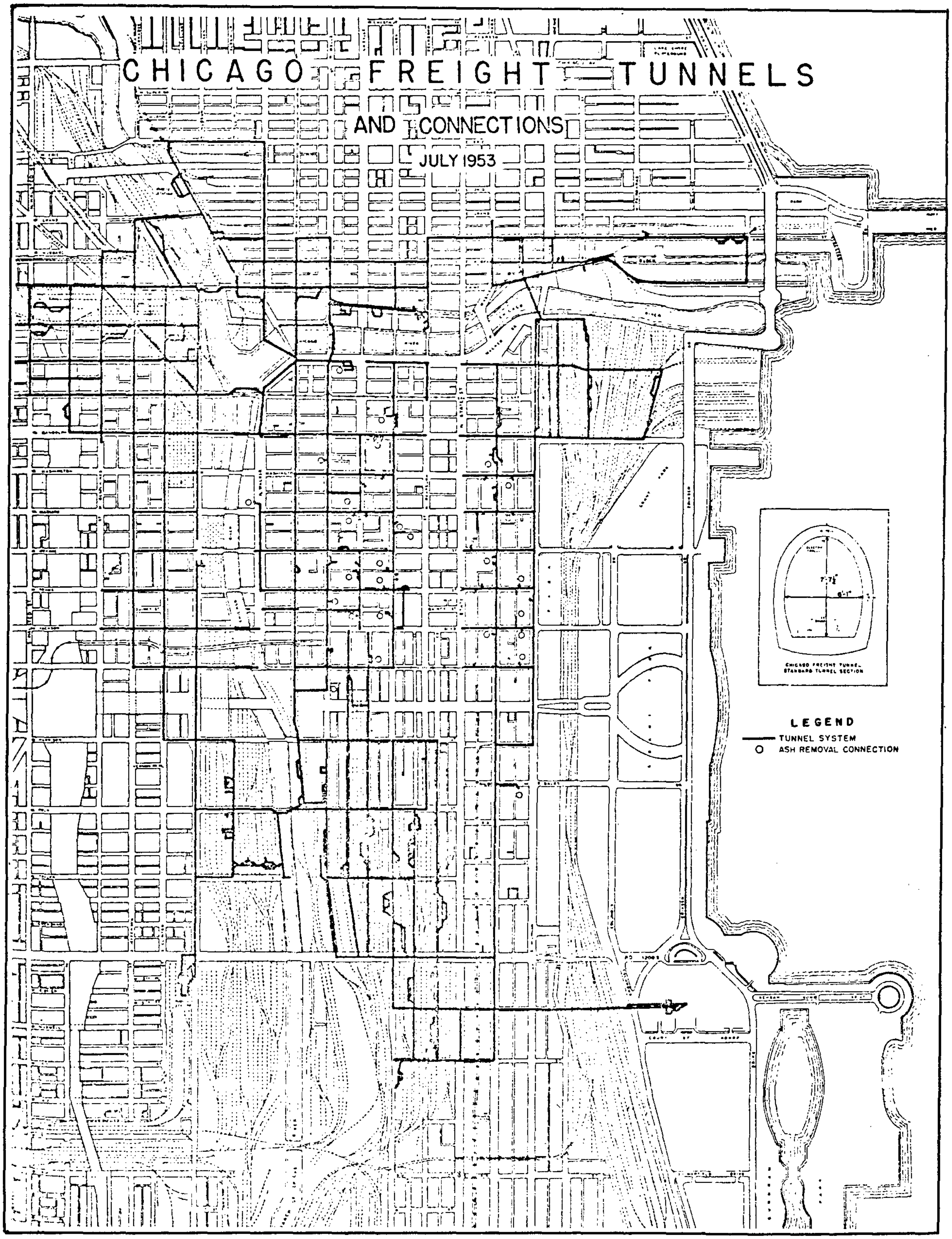
CHICAGO'S FREIGHT TUNNEL SYSTEM

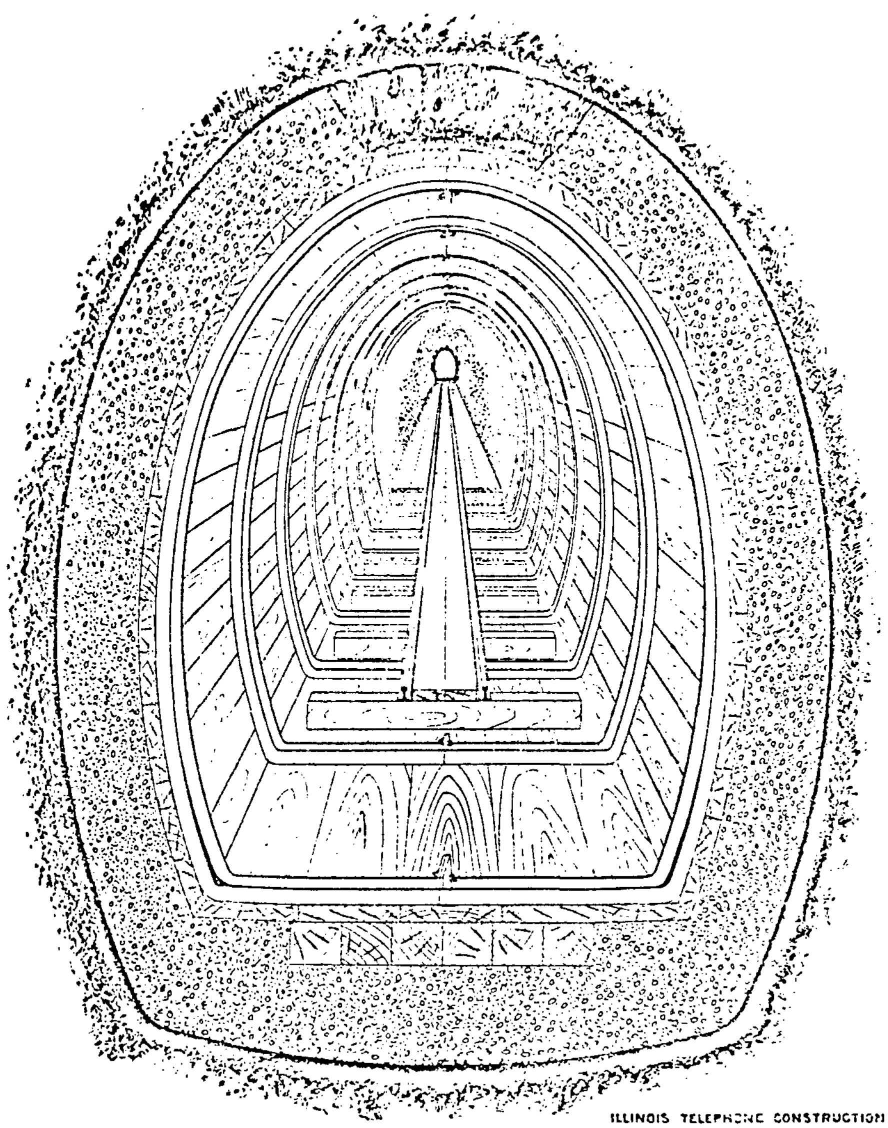
&

CHICAGO'S VEHICULAR RIVER CROSSING TUNNELS

&

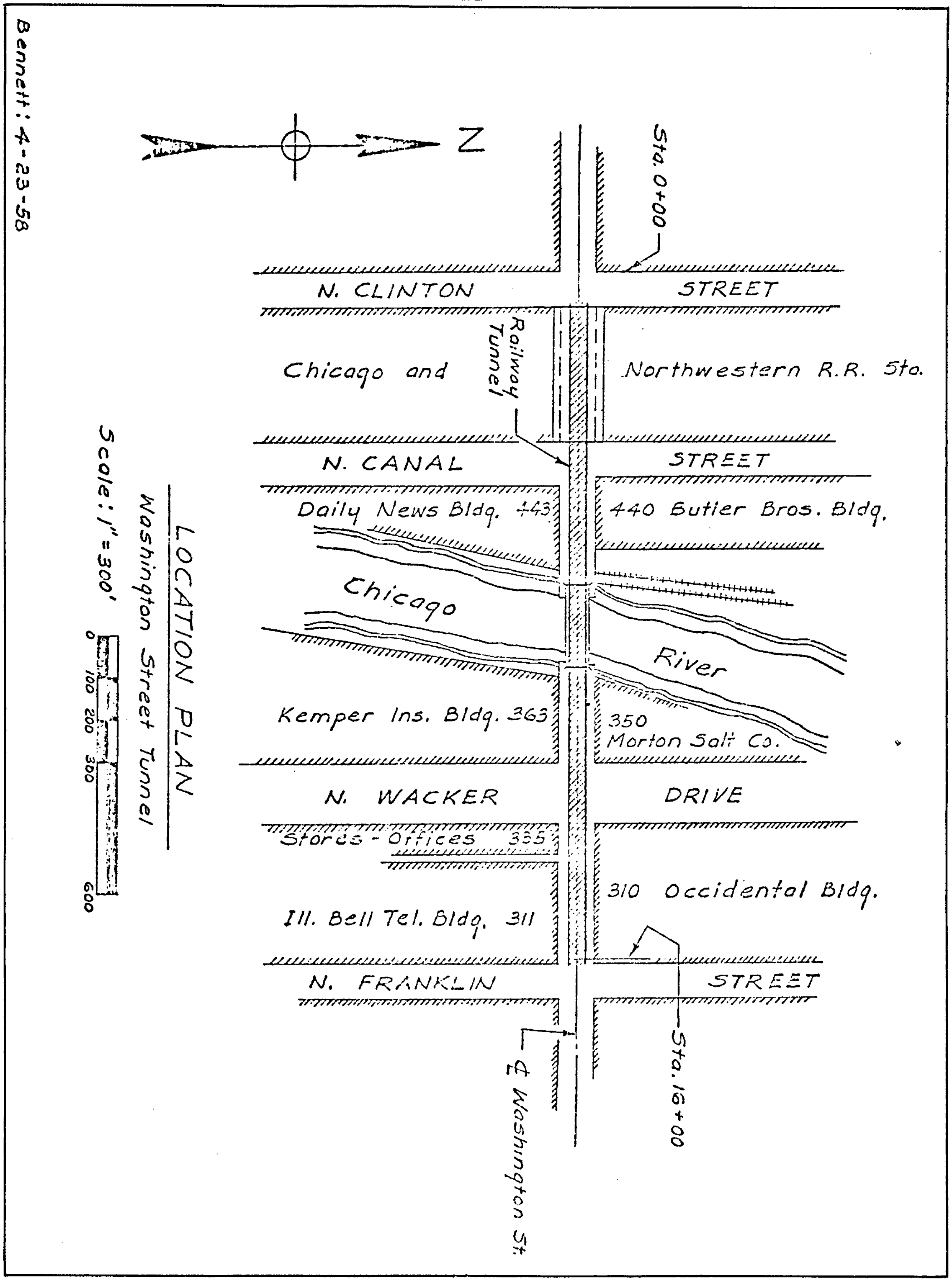
CHICAGO'S SUBWAY SYSTEM


CHICAGO FREIGHT TUNNELS
AND CONNECTIONS
JULY 1953
LEGEND
TUNNEL SYSTEM
ASH REMOVAL CONNECTION



METHOD OF TUNNEL CONSTRUCTION.

First, 14 inches of concrete is placed in bottom. Five-inch by five-inch timbers are used to form recess in order to lay the 56-pound rails. On top of 5-inch by 5-inch timbers 2-inch lagging is placed. Then 5-inch steel channels weighing 10 pounds to the foot are placed 3 feet to centers. Then 2-inch by 6-inch lagging is placed, and concrete rammed between lagging and excavation, 6 inches in height, filling all voids before another section of the 2-inch by 6-inch is placed, and so on until concrete is placed as shown in cut. The channel ribs and lagging are left in place until concrete has had at least six days to set.



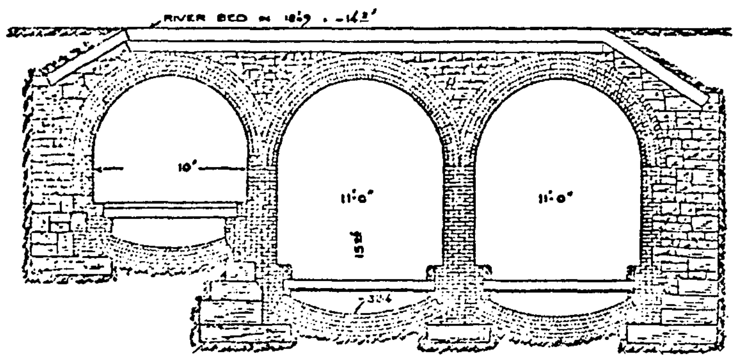
LOCATION PLAN
Washington Street Tunnel
Scale: 1" = 300'

WASHINGTON ST · TUNNEL

CHICAGO · ILLINOIS ·

1867 - 1869

2

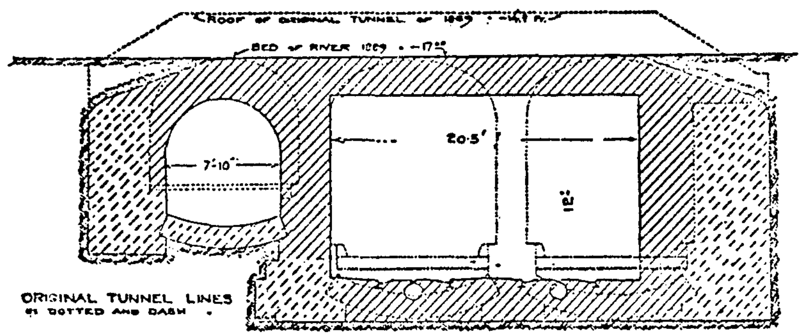


WASHINGTON ST · TUNNEL

CHICAGO . ILLINOIS ·

—1889—

3



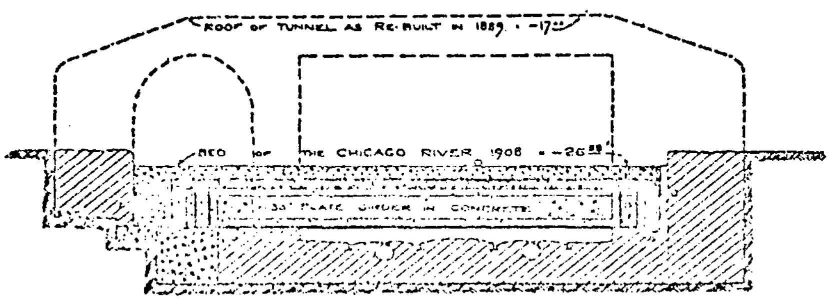
ORIGINAL TUNNEL LINES BY DOTTED AND DASH .

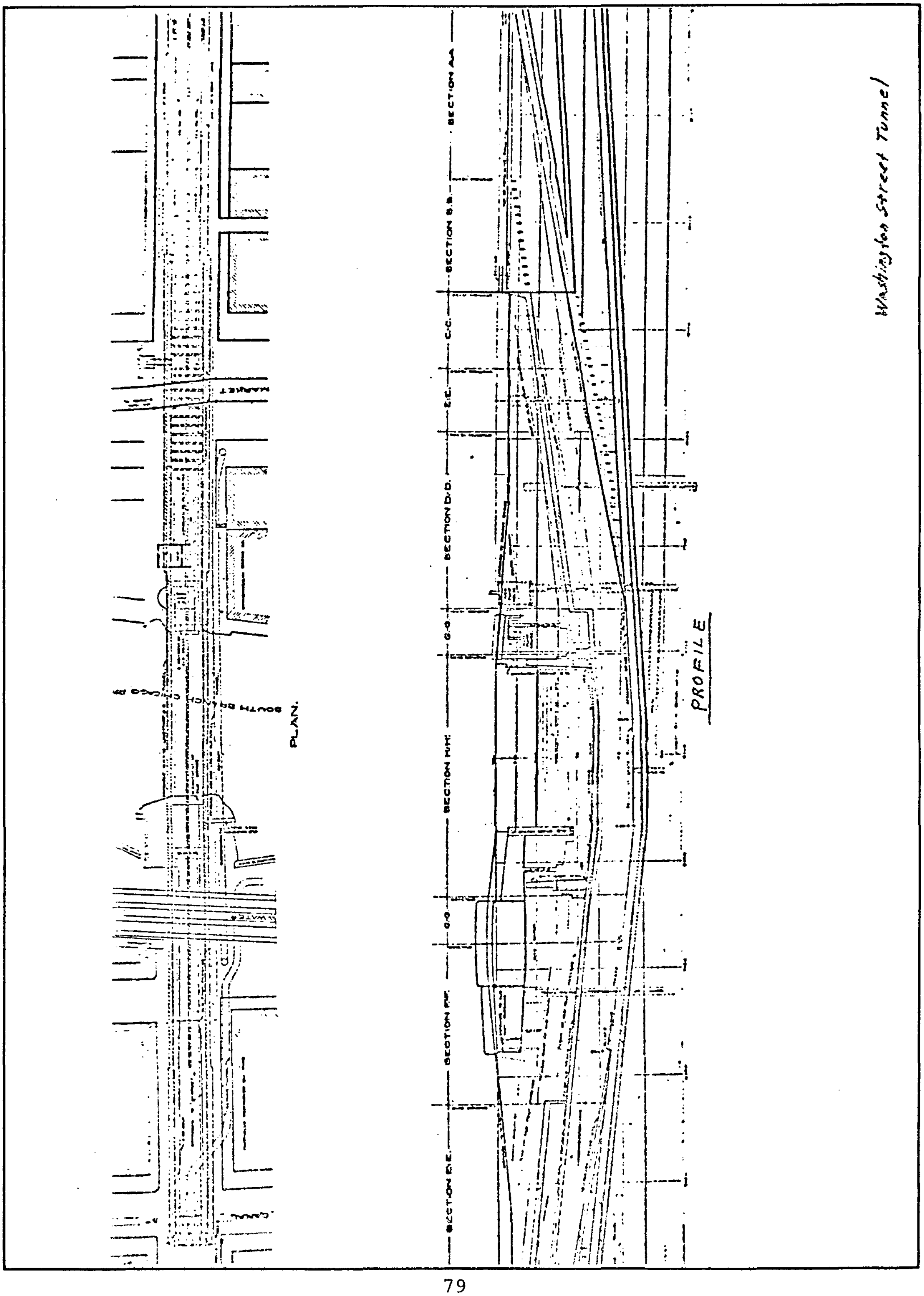
WASHINGTON ST · TUNNEL

CHICAGO · ILLINOIS ·

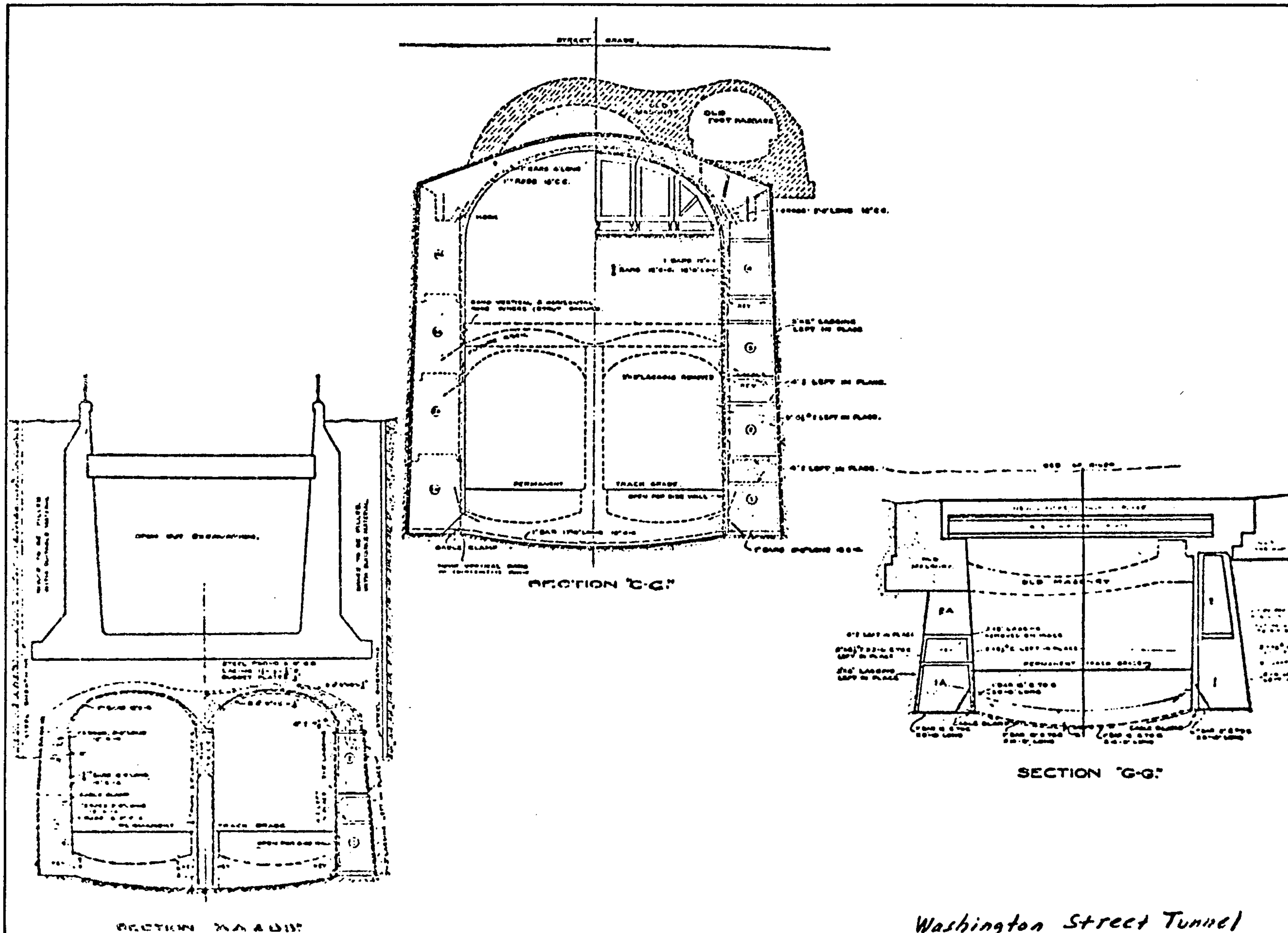
1907-1908

4

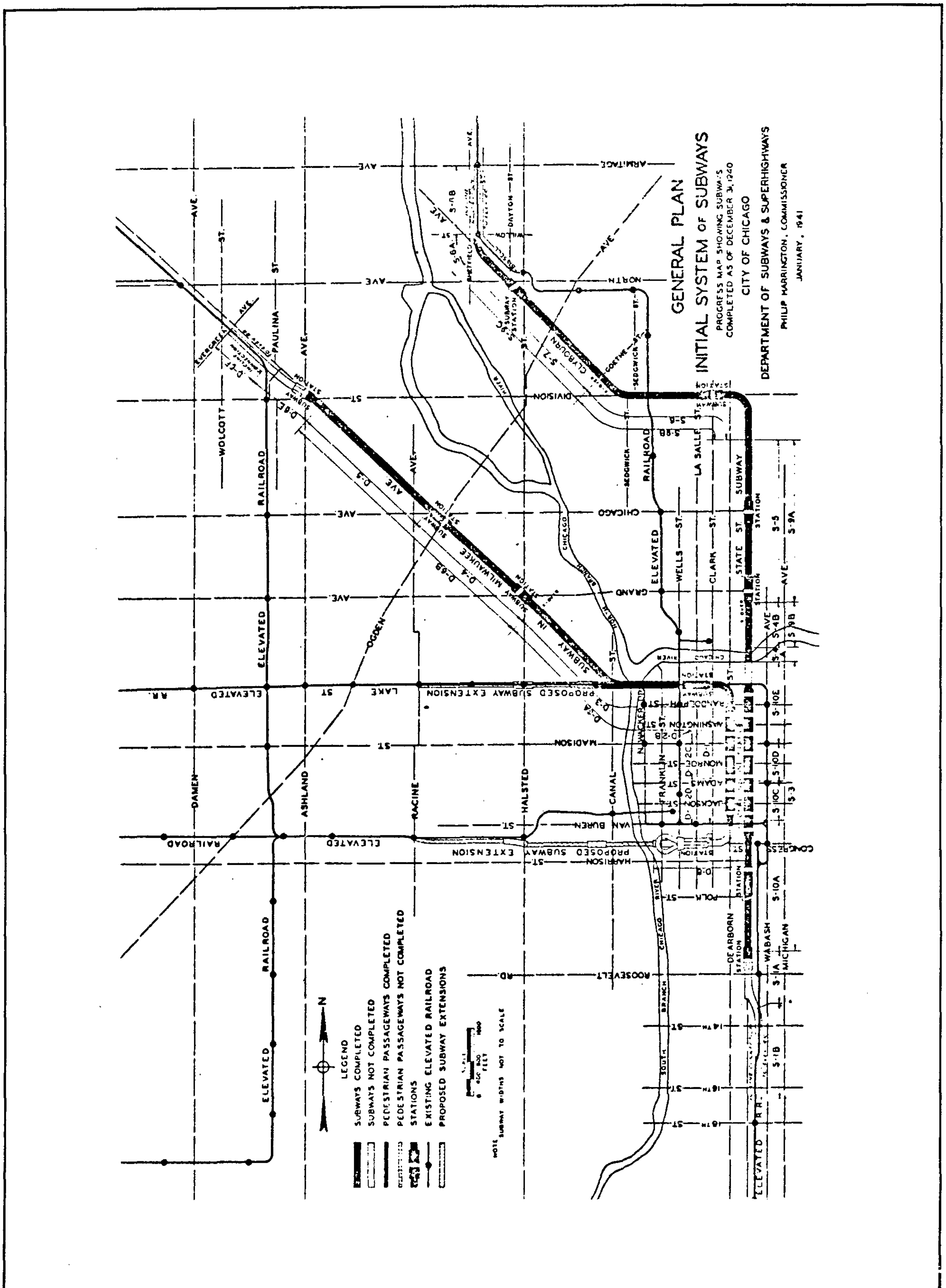


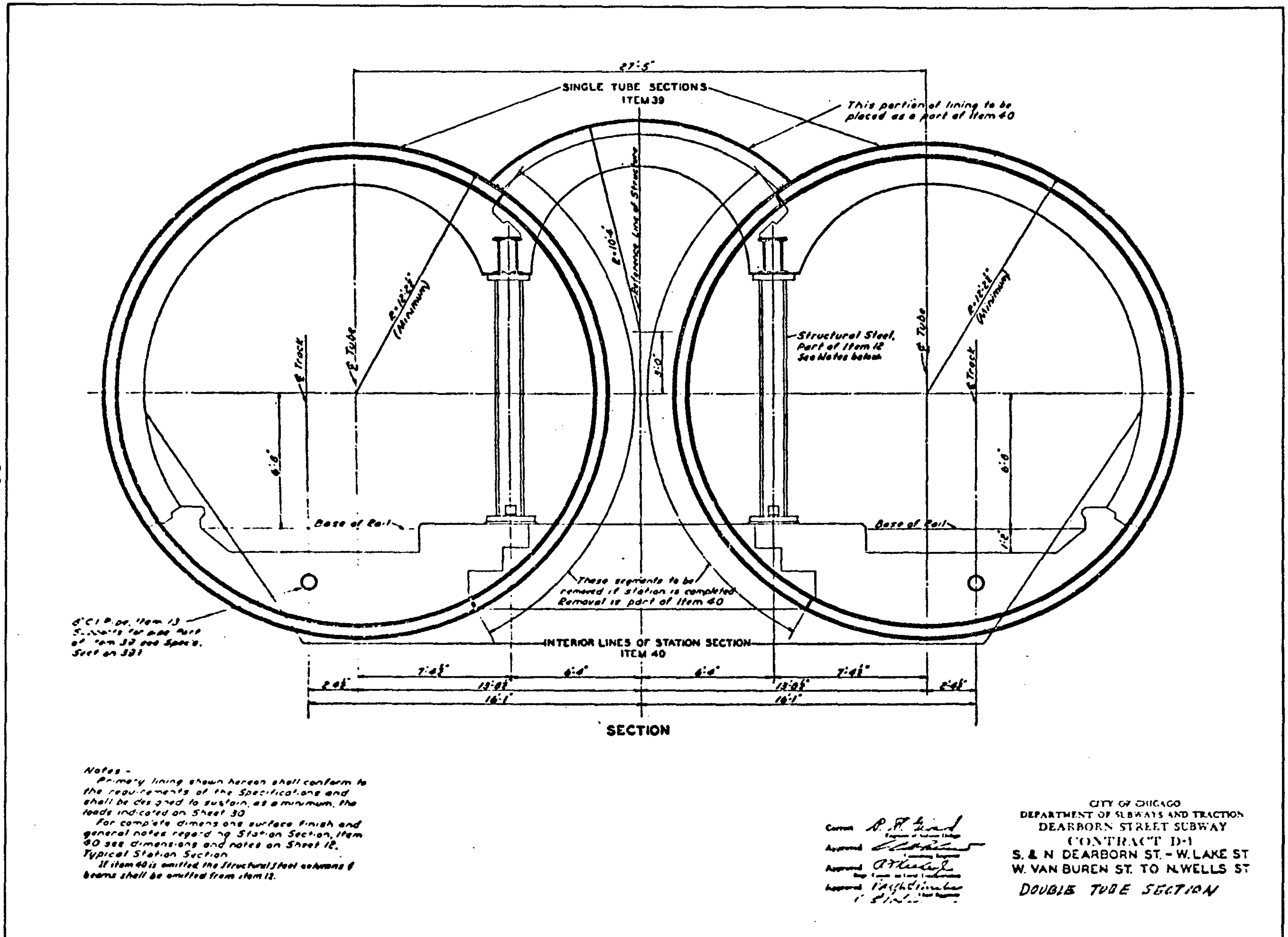


Washington Street Tunnel
PROFILE
PLAN.
SOUTH BRANCH CHICAGO R
MARKET
CANAL
SECTION A-A
SECTION B-B
C-C
E-E
SECTION D-D
G-G
SECTION H-H
O-O
SECTION F-F
SECTION E-E

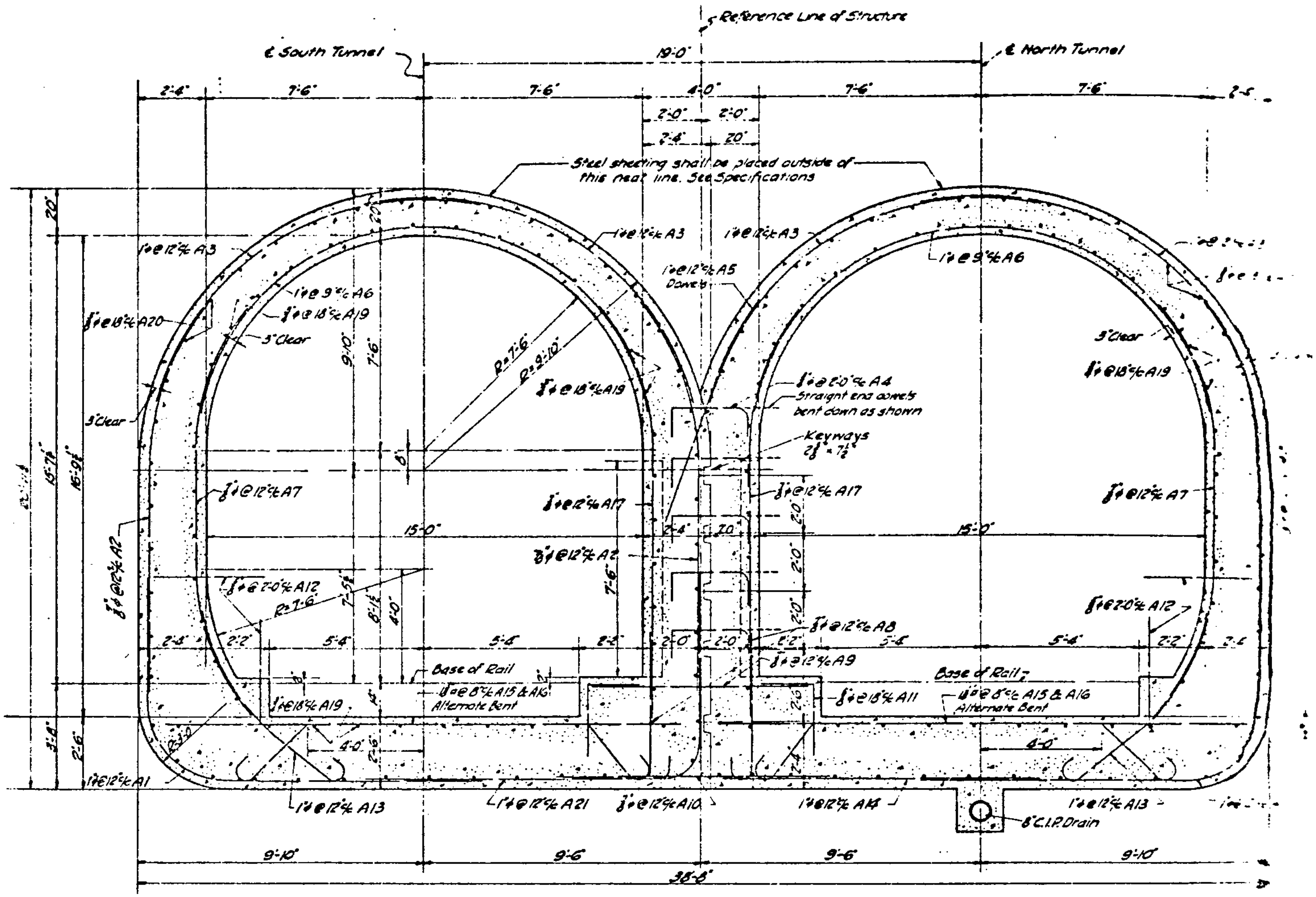


Washington Street Tunnel


GENERAL PLAN
INITIAL SYSTEM OF SUBWAYS
PROGRESS MAP SHOWING SUBWAYS COMPLETED AS OF DECEMBER 31, 1940
CITY OF CHICAGO
DEPARTMENT OF SUBWAYS & SUPERHIGHWAYS
PHILIP HARRINGTON, COMMISSIONER
JANUARY, 1941
LEGEND
SUBWAYS COMPLETED
SUBWAYS NOT COMPLETED
PEDESTRIAN PASSAGEWAYS COMPLETED
PEDESTRIAN PASSAGEWAYS NOT COMPLETED
STATIONS
EXISTING ELEVATED RAILROAD
PROPOSED SUBWAY EXTENSIONS
NOTE SUBWAY WIDTHS NOT TO SCALE



27'-5"
SINGLE TUBE SECTIONS
ITEM 39
This portion of lining to be placed as a part of Item 40
R=10'-4"
Reference Line of Structure
R=12'-2½" (Minimum)
Structural Steel, Part of Item 12 See Notes below
℄ Track
℄ Tube
6'-8"
5'-0"
Base of Rail
1'-2"
These segments to be removed if station is completed Removal is part of Item 40
6" C.I. Pipe, Item 13 Supports for pipe Part of Item 32 see Spec'n. Sect on 321
INTERIOR LINES OF STATION SECTION
ITEM 40
7'-4½"
6'-4"
2'-4½"
13'-8½"
16'-1"
SECTION
Notes -
Primary lining shown hereon shall conform to the requirements of the Specifications and shall be designed to sustain, as a minimum, the loads indicated on Sheet 30
For complete dimensions surface finish and general notes regarding Station Section, Item 40 see dimensions and notes on Sheet 12, Typical Station Section
If item 40 is omitted the Structural Steel columns & beams shall be omitted from item 12.
Correct
Approved
Approved
Approved
CITY OF CHICAGO
DEPARTMENT OF SUBWAYS AND TRACTION
DEARBORN STREET SUBWAY
CONTRACT D-1
S. & N. DEARBORN ST. - W. LAKE ST
W. VAN BUREN ST. TO N. WELLS ST
DOUBLE TUBE SECTION



SECTION
Scale: $\frac{1}{2}$" = 1'-0"

Note:
Reinforcement steel as shown is typical for sections without niches or openings.
The amount of steel may vary, depending upon final design requirements as determined by conditions encountered in construction.
Construction points other than shown on the plans shall be as approved by the Engineer.
Main reinforcement steel shall be placed 3" clear from forms.
See Sheets 16 to 20 for details of electrical conduits and boxes.
See Sheet 11 for drainage details.
Construction of all openings and niches in walls of twin tube section are part of Item 1.

Correct — Engineer of Subway Design
Approved — Consulting Engineer
Approved — Engr. Comm. on Local Transportation
Approved
Approved

CITY OF CHICAGO
DEPARTMENT OF SUBWAYS AND ...
STATE STREET SUBWAY
CONTRACT S-7
N. CLYBOURN AVENUE
GOETHE STREET TO CONCORD PL.
TWIN TUBE SECTION

BIBLIOGRAPHY

Artingstall, William, Chicago River Tunnels - Their History and Method of Reconstruction, Western Society of Engineers Journals, 1911.

Chicago, City of, The Straightening of the Chicago River, 1926.

Chicago, City of, Department of Public Works, Chicago Public Works, A History, 1972.

Chicago, City of, Department of Public Works, Bureau of Engineering, A Century of Progress in Water Works 1833 - 1933, 1933.

Chicago, City of, Metropolitan Sanitary District, Engineering Works, 1928.

Illinois, State of, Division of Waterways, Documentary History of the Illinois and Michigan Canal, 1956.

Peck, Ralph B, "Earth Pressure Measurements in Open Cuts, Chicago Subways", ASCE Transportation, Volume 108, 1943.

Pikarsky, Milton, "Sixty Years of Rock Tunneling in Chicago", ASCE Annual Meeting, 1969.

Public Efficiency, Bureau of, The Water Works System of the City of Chicago, 1917.

Salter, George S., "Tunnel Construction of Chicago", American Water Works Association Journal, Volume 48, 1956.

Subways and Superhighways, Department of, Annual Reports, 1939 - 1945.

Terzaghi, Karl, "Liner - Plate Tunnels on the Chicago Subway", ASCE Transportation, Volume 108, 1943.

Terzaghi, Karl, "Shield Tunnels of the Chicago Subway" Boston Society of Civil Engineers Journal, Volume 38, 1942.

Walker, Ward, The Story of the Metropolitan Sanitary District, 1956.

PAPER 5

Site Conditions Affecting Rapid Transit Construction in the CBD

G.M. Randich, P.E.
Project Director, Supervising Consulting Engineer, CUTD
DeLeuw-Novick, Inc.

PAPER 5

SITE CONDITIONS AFFECTING RAPID TRANSIT CONSTRUCTION IN THE CBD

by

G. M. RANDICH, P.E.

Project Director, Supervising Consulting Engineer, CUTD
DeLeuw-Novick, Inc.

INTRODUCTION

Construction methods and equipment are selected largely on the basis of the design requirements established by the plans and specifications and the physical interrelationship of man-made and natural objects. We refer to these as site conditions and make note that every kind of project encompasses a complete range and variation of site problems. This paper discusses site conditions peculiar to the Chicago Central Area Transit Project; however, the problem is not uncommon to central business districts of principal cities where rapid transit development programs are either now underway or in the planning and contemplation stage.

Site conditions affect the cost of general construction, that is, the cost to put into place an adequate structural envelope to house the transit system's facilities. Our experience expressed in today's dollars has shown that general construction for a guideway in the median of a freeway would cost about $1000 per foot, while cut and cover construction in an urban neighborhood would be about $4000 per foot, compared to $12,000 per foot for construction of a facility within the CBD. Track, power, signals, mechanical systems, and fare collection add about $4000 per foot.

In this day of alternatives analyses it is important to first put into perspective the elements of site which contribute to such a wide variation in capital requirement, and second to illustrate site conditions so that new or proposed construction techniques can be fairly analyzed.

CBD SYSTEM PLAN CHARACTERISTICS

In terms of system planning we all first recognize that rapid transit within the Central Business District provides a rather unique service. Maximum effectiveness in operation is achieved when the system transcends from a line haul facility to either a distributor or collector type system (depending on time of day) within a densely populated and compact CBD such as in Chicago, New York, Philadelphia, and several others. Close station spacing, heavy peak hour patronage, interline transfer, quick accessibility, and adjacent building interface are common characteristics of CBD transit. In fact here in Chicago we subscribe to the continuous platform concept within the heart of the CBD. Station platforms may be 3500 to 4500 feet long, serviced from a number of mezzanines conveniently spaced to interface with bus routes, principal employment centers, and pedestrian activity. Maximum long term system effectiveness and flexibility are thereby achieved. For example, if during operations patronage at a particular station exceeds design forecasts, train stops from opposite directions can be staggered to optimize use of vertical circulation elements and platform space.

Growing CBDs are experiencing considerable high rise commercial construction and planned development. A case in point is Chicago's Sears Tower complex located adjacent to our proposed Franklin Line. Current employment population for this building is 16,000 persons. We expect 35% of these to be serviced by transit. Further, since our project was originally conceptualized over 35 new buildings have been constructed adjacent to or within walking distance of the system. This represents a private investment of approximately $2 billion. Public transit is a vital service feature and the catalyst in attracting investments of such magnitude.

We can therefore see that CBD fixed guideway systems must have the capability to respond to unpredicated changes in land use that might occur 25, 50, or 100 years hence. As such, perceptive planning is required in the broadest sense. Typically, in terms of a constructed facility CBD fixed guideway systems can best be described by these unusual requirements:

1. A grade line as near the surface as practicable to minimize patron time in vertical circulation.

2. An integrated station stop/mezzanine entrance system providing continuous and handy access to the system.

3. Direct transfer capability to all other public transportation modes, including other lines of the same mode.

4. Generous platform widths of continuous length within the most dense areas of the CBD.

5. High peak 15 minute patron demands with convenient underground pedestrian circulation routes.

6. Aesthetic quality, maximizing interface potential with adjacent structures.

Simply stated, underground fixed guideway systems within the CBD consume a fair amount of underground space and public rights of way, and they require large structures of varying cross sections to provide the necessary spatial freedom inherent to CBD transit service goals.

With this in mind, let us focus then on CBD site conditions which impede construction and profoundly impact costs.

THE CITYSCAPE

Most older CBDs were laid out in the middle to late 1800's by government surveys to the classic rectangular grid system based on the chain and rod system of measurements. Block dimensions were established at 5 chains x 10 chains (330 feet x 660 feet) or 10 x 10 chains generally circumscribed by 1 chain (66 feet) wide streets. Here and there a main street might have a width of 1-1/4 or 1-1/2 chains (80 or 100 feet), but these streets have since been fully utilized by earlier tansit routes or heavy surface traffic.

The scarcity of vacant space and the high value of property within the CBD offer no opportunities to widen or expand the public way. Therein lies the first constraint in the development of fixed guideway transit; narrow rights of way adjacent to intensely developed property.

ABOVE-SURFACE ENVELOPE

Secondly let us examine the streets right of way itself, that is, the space bounded by multi-storied buildings on either side of the street, the street surface, and the sky above. This is the space in which construction is to take place and in which construction equipment will be operating. Here we find several kinds of encroachments affecting the usability of space.

We acknowledge that architecture in the CBD has changed appreciably over the years. New high rise structures incorporate plazas and arcades with building setbacks. The trend is to get away from the canyon effect inherited from the development boom of the late 1930's. These new buildings however are few and far apart offering irregular, if any, opportunity for spatial freedom. It is common in older buildings to see permanent ornamental work and building cornices overhanging the sidewalks. One building in our particular project has projections which overhang the property line by 3.4 feet and many portions of the first floor facade extend one or two feet beyond the property line.

Where the transit structure outer walls are to be located in close proximity to the property line the operation of construction equipment might be impaired, particularly if the building has some historical value and should not be defaced. Here a portion of the temporary supporting structure to be ultimately used for excavation support may be installed during an underpinning effort as a means of avoiding such conflict. From a construction standpoint and the risk-damage factor involved it is good practice to plan an underground structure such that heavy equipment need not operate any closer than two feet from the face of a building. Taking into account this clearance and the space required for an excavation support structure, the maximum width of a transit structure would be roughly ten feet less than the right of way within a typical CBD street.

To a lesser extent of impact there are numerous items of street furniture, storefront signs, theater marquees, and such also occupying the above-surface space. These however are easily removed and later replaced, with only an intervening space-storage problem. The storage aspects of this problem may be crucial since the availability of vacant space increases in proportion to the distance from the CBD. Construction materials and equipment also need storage space and the contractor may be confronted with long travel times from storage site to construction site.

BELOW-SURFACE ENVELOPE

At this point my best advice to the transit planner or designer is, "Beware of that which you cannot see," for below the surface every conceivable use of space is made. I will refrain from discussing utilities for the moment, but rather let us review some other uses of underground space. Perhaps most dominant is under-sidewalk vaulted space. On our Project, one bank building has principal safekeeping activities located under the sidewalk, another has locker room facilities which service an adjacent swimming pool. In total, 76% of the sidewalk area within the CBD section of our Monroe Line is devoted to underground sidewalk use. Typically the vaults extend from the first basement level of the building at the property line to the curb line, sometimes consuming 10 to 20 feet of underground public way. At the curb line a gravity type retaining wall usually separates the roadway portion of the street from the sidewalk. Where multi-storied vaulted space is developed the street closure wall is frequently founded on piles or caissons. Basement and subbasement floors extend directly into the vaulted space without property closure walls.

Permission to use the under-sidewalk space is granted by City ordinance or, in some cases, by annual permit to the building owner. The right to revoke is usually preserved; however, there is substantial amount of effort required to implement vacation of these spaces. Before a closure wall can be built at the property line, a space for the relocated facilities must be found elsewhere in the building. Frequently extensive mechanical and electrical renovation is required. The possibility also exists that building owners may opt to negotiate that vaulted space be maintained during construction. In this situation, construction equipment loads on the sidewalk must be carefully controlled.

Various existing substructure elements also encroach upon the below-surface envelope and include ciasson bells, pilecaps, piles, and spread footings. Normally, underpinning operations are required to protect existing buildings and other structures from distress caused by settlement or lateral movement due to changes in foundation support brought about by excavation for the transit structure. Because of space encroachments, underpinning may also be required even though a particular structure foundation extends well below the planned excavation limit. Where plans of existing buildings have been made available, we have found that a complete range of foundation conditions exist. Some are shallow and spread out, while others are on short timber piles, or footings may be on steel piles or caissons to firm bearing in hardpan or rock. We have found that about 60% of the structures are on hardpan caissons, 20% on wood piles, and another 20% on spread footings. Usually the older building structures encroach the most. The Marmon Building built in 1894 has, for example, spread footings extending almost seven feet beyond the property line. More significant, however, within one mile length of the Monroe Line which bisects the heart of the CBD foundation plans are available for one out of every five structures.

In addition to extensive use of under-sidewalk vaulted space, the streets of the Chicago CBD contain a rather elaborate network of freight tunnels. Known as the "Chicago Freight Tunnel System," approximately 60 miles of single bore 6' x 7.5' concrete lined tunnels were constructed at a depth of 40 feet below street level in the period 1898 to 1906. The system is no longer operating, and only about eight miles of it have been abandoned under past subway construction projects (State and Congress Lines). Over 20 ventilating connections have been made to various buildings for the purpose of furnishing cool air; however, it is unknown how many exist today. In addition, several existing buildings have access to the tunnel system by means of adits. The tunnel generally interferes with transit construction, and will be filled, bulkheaded, or removed depending on its location with respect to the transit structure. The feasibility of utilizing the tunnel system as an excavation removal expedient has been studied and found to be uneconomical.

THE UTILITY INFRASTRUCTURE

Public and private utilities own facilities which occupy the street right of way, generally to a depth of 11 feet below the surface. The degree of complexity seriously influences transit system planning and design in terms of both cost and construction techniques. In the CBD 30% to 40% of the total street width is occupied by the various utilities; outlying areas comprise about 10% or less. More importantly in open cut construction severe time constraints can be expected: in fact, within CBD areas eight to ten months is not an unusual period of time to spend in utility preparatory work. That is, the digging of test pits to verify location of individual ducts, pipes and mains, uncovering the utility and temporarily reinforcing it with planking and strapping so that the utility may eventually be suspended from the temporary excavation structure. This preparatory work must be done under controlled traffic conditions mostly by had excavation before the contractor can proceed with the mass excavation for the subway trench.

Compared to outlying urban areas, CBD utilities are not only more extensive but also unique because they are generally high capacity systems all buried within the streets for aesthetic purposes. Further, they have been installed over a long period of time expanding on demand by its customers, with little or no order to the location, best described by visualizing a "spaghetti" maze. As many as 15 different utility ownerships can be identified. Not only do they provide essential water, sewer, electric, gas and telephone services, but also street lights, traffic signals, fire and police alarms, telegraph, and other special purpose utilities such as Chicago Press Association wires and pneumatic tubes. All these must be continuously maintained irrespective of costs. The Bell System uses its largest size cables within the CBD, containing 2700 pairs of wires. Any damage to one or more pairs disconnects one or more customers from service. Service cannot be restored until the wires are either repaired or the service is switched to a spare pair of wires, if available. Utilities are also configured on a grid system, interconnecting with each other at street intersections so that in the event of maintenance shut-down, services can be provided via other routings on the grid. For example, most of the existing large buildings in the Central Business District are served by two water mains, each in a different street, so that water services and fire protection can be maintained. There is also a network of abandoned utility conduits, but unfortunately very poor records concerning their location and future disposition are available.

All utilities will require some form of rearrangement during construction, but there are several options on accomplishment. depending upon utility owner preferences and agreements to be consummated. Rearrangement or replacement may be done by the general construction contractor, or by subcontract to him. Alternatively, the utility company may select several qualified firms to be chosen competitively, or elect to perform the work with its own forces. In any event, the utility companies retain the right to inspect and direct construction for compliance with utility operations, standards, and specifications. Also, they can be expected to occupy the work site to furnish materials and do installations where special skills and testing procedures are required. As such, in addition to the general contractor work force, several utility companies may occupy the site concurrently, demanding access to their facilities for ongoing maintenance, service extensions, and repair.

The most serious and important consideration for the utilities is the necessity of safeguarding continuous services to their customers who are located at the street right of way line. Typically transverse service connections to the buildings occur at frequent intervals throughout a block's length, and transverse distribution and transmission grid networks occur at the street intersections between blocks. Herein another site constraint is imposed, for a continuous trench excavated from the surface would intercept all transverse utilities.

STREET ACTIVITY

Human and motor traffic characteristics within a Central Business District street impose variable constraints, depending on the time of day. Characteristically, during the normal 12 hours of daylight 71% of the total daily traffic uses the CBD streets, and perhaps as high as 98% of the total pedestrian traffic. Only 7% of the traffic is trucks, but typically a single CBD street may accommodate one or several surface bus routes. Over the years narrow streets within CBDs have become quite inadequate to handle growing traffic demands, and the popular solution has been to establish a system of one-way couplets to improve traffic through-put. More recently many cities have installed integrated, computer controlled traffic signal systems to further improve through-put capacity. Average daily traffic in the 7,000 to 15,000 range can be expected, with about 10% of that occurring in the peak hour.

In this situation it is not easy to detour traffic for a long period of time to other streets, for the user must suffer the consequences of adverse travel and further delay on streets where normal average speed is only about eight or nine miles per hour. When a street or a portion of a street is to be closed at least 80% of the total traffic would require redistribution to adjacent couplets, with the remaining 20% hopefully using other modes or remote alternative routes. In turn, a severe congestion problem is inflicted on streets where capacity is already marginal. Nonetheless, a street can never be totally closed to traffic. Daily delivery of goods and services to business and commercial establishments is a function of everyday operation. Moreover, police, fire, and emergency access to these establishments must be maintained.

On an average weekday almost a half million persons come to Chicago's Central Business District, each person having several walking trips in addition to the work-to-home journey. These constitute shopping, recreation, and business-related walking trips. An origin destination survey indicated an average of 5.8 walking trips per person. These occur in three distinct peaks; the morning peak from 8:00 a.m. to 9:00 a.m.; the midday peak from 12:00 noon to 1:00 p.m.; and the afternoon peak form 4:00 p.m. to 6:00 p.m.

In a recent survey it was found that on many CBD streets inadequate sidewalk capacity currently exists, although distribution of these walking trips to the street grid system is widespread. Pedestrian midblock counts within a ten hour period range from a high of 33,000 person in the most dense activity areas to 5000 in the lesser activity areas. Along the CBD section of our Monroe Line, high to low midblock pedestrian volume ranged from 15,500 persons to 7500 persons during a ten hour period. At least 3000 persons desire sidewalk usage at the midday peak hour. It can thus be seen that underground rapid transit construction will not only inhibit pedestrian flow, but also curtail construction contractor's mobility during the daylight hours. It further suggests that special overhead and lateral protection to pedestrians may be required.

CONSTRUCTION EXCAVATION CRITERIA CONSIDERATIONS

In evaluating the feasibility of various construction excavation techniques for cut and cover construction the distinct and unusual nature of site conditions within the CBD must first be brought into focus. It is suggested that the following serve as a guide in this respect.

1. Equipment Clearance and Storage
   Construction equipment at or near buildings should not operate within a clear zone of two feet. Materials and equipment storage should be offsite.

2. Special Site Constraints
   Joint opportunities exist in interfacing the facility with adjacent structures; these include retail/commercial access to the structure, or construction expedients such as including temporary excavation support during an underpinning operation.

3. Utility Interferences
   Utility service connections to adjacent buildings as well as distribution and transmission networks in the cross streets must be kept in operation at all times.

4. Utility Preparatory Work
   Readying the utility infrastructure for temporary support is time comsuming, employing hand excavation methods under heavy street traffic.

Successive operations of traffic maintenance, excavation, plating, or backfilling are required.

5. Traffic and Pedestrian Provisions
Under limited street closures of short duration, at least one lane of roadway must be made available from one end of the construction zone for emergency vehicles and service access. Heavy pedestrian interference can be expected to the extent that both side and overhead protection should be considered.

6. Street Traffic Operational Requirements
The street surface must be totally decked over during construction. Traffic must occupy at least two traffic lanes and sidewalk areas must be designated. Cross street traffic must be maintained at all times; only one-third of the cross street should be closed to traffic at a time.

7. Working Hours
The ideal time to perform massive work on the surface streets in CBDs is during the non-peak periods of 8:00 p.m. to 6:00 a.m. daily and 6:00 p.m. Friday to 6:00 a.m. Monday on weekends. Construction effort during the day should be limited to below deck operations, minimizing interference with surface activity.

The construction of an underground transportation system clearly represents an intrusion to the routine and everyday functions of the Central Business District in terms of the existing conditions of site. New underground excavation procedures need to be employed to minimize construction time, cost, and site degradation and to provide effective transportation so urgently needed within our CBDs. It is hoped that the matters of site discussed herein bring into focus a clearer definition of associated problem areas.

BIBLIOGRAPHY

Consoer, Townsend & Associates, "Utility Tunnel Economic Feasibility Analysis in Conjunction with the Construction of Subways at the Cut and Cover Method," January 1975.

Pedestrian Mall Task Force Subcommittee on Pedestrian Movements, "Chicago Loop Pedestrian Movements Study," December 1973.

Supervising Consulting Engineers: De Leuw-Novick; Supervising Consulting Architects: C. F. Murphy Associates, Inc.; Associate Supervising Consulting Architects: Loebl Schlossman Dart & Hackl, Inc., "Chicago Central Area Transit Project Monroe Line Michigan/Franklin Station Alternative Concept Analysis," May 1974.

PAPER 6

Contractual and Legal Aspects of Underground Construction

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PAPER 6

CONTRACTUAL AND LEGAL ASPECTS OF UNDERGROUND CONSTRUCTION

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INTRODUCTION

This paper was prepared for the seminar on construction problems, techniques and solutions to be held in Chicago on October 20, 21 and 22, 1975. Since the seminar focuses on the construction problems of the Chicago Central Area Transit Project and the Baltimore Region Rapid Transit System, those aspects of the subject which are particularly relevant to these two projects will be emphasized.

In its broadest sense, the subject of contractual and legal aspects of underground construction encompasses almost all facets of a project other than the technical. In its narrowest sense, it would cover only the contractual relationship between the owner and the individual construction contractors. Since the other subjects on the seminar program are primarily technical, this paper will address the subject in its broader sense.

Standing Subcommittee No. 4, "Contracting Practices", of the U. S. National Committee on Tunneling Technology, recently completed a study on this subject. Its report has been published under the title "Better Contracting for Underground Construction," and contains more detail on many of the items discussed herein. Although the study concentrated on the narrower aspect of the subject, it recognizes that many of the problems encountered in the contracting phase of the project are the result of actions taken in the development, planning and design phases.

The following sections of this paper will examine the contractual and legal aspects of underground construction which are commonly encountered in an urban, mass-transit project. For convenience, they are divided into separate classifications, although a considerable degree of overlap will be apparent.

CONTRACTING OBJECTIVES

Obviously, the overall objective of the contracting procedure is to get the project constructed in the alloted time frame and within the budget. To accomplish this, virtually all public bodies in the United States rely upon the competitive bidding system. Since there is little immediate chance of a major change in this basic system for public construction, this paper will concentrate on suggested improvements to the basic system and on recommended actions to be considered during the preconstruction phases of the project.

There can be little question that the American system of contracting has produced a family of builders whose genius is unequalled in the world. When the contractor must, in order to survive financially, pit his wits against his competitors, first to get the job, and then to finish it successfully, the result must be a dynamic and vital industry.

Nevertheless, underground construction remains as much an art as a science. In order to derive maximum advantage from the contractor's skills and ingenuity, the project, from its very inception, must be designed for construction economy and feasibility. The contractor's profit motive must be channeled to save the owner money.

PHYSICAL CONDITIONS

1. Natural Subsurface

One of the greatest hazards in underground construction is the risk of encountering unexpected natural subsurface conditions. A certain amount of subsurface information is, of course, a prerequisite to design. And, it is common practice to make such information available to prospective bidders. In fact, to withhold such information is to risk serious legal consequences.

The prospective bidder, however, is interested in the subsurface in a somewhat different context than is the designer. He is interested in how it will behave during construction rather than its effect on the permanent structure. He is interested in the effect of water on the stability of the ground rather than in its absolute hydrostatic pressure.

During the planning and design phases, the owner or his engineer accumulates a vast amount of subsurface information which should be of great interest to prospective bidders. Usually the owner's geologist is able, from this factual information, to derive interpretations as to the likely behavior of the ground and its effect on various construction methods.

In the past, many jurisdictions have deliberately withheld all such geologic interpretations from prospective bidders, making available only that factual information which is legally required. Their rationalization is that the dissemination of such geologic reports and interpretations introduces an exposure to potential claims in case the interpretations prove to be inaccurate.

Such an attitude is short-sighted and, in the long run, not in the best interests of the owner. The more information which can be given to prospective bidders, be it factual or professional interpretation, the more competitive will be the bids and the less likelihood will there be for costly disputes and litigation later. Even though the professional interpretation might appear to be detrimental to the owner, and withholding it might encourage lower bids, the final cost to the owner of unpleasant surprises is likely to be much greater than it would be were the bidders forewarned.

In a similar category is the use of disclaimers. Here, the owner makes the subsurface information available, but disclaims any responsibility for it and warns prospective bidders to use it at their own peril. This is a particularly vicious policy and frequently operates to the disadvantage of the owner. It often impels the contract administrator to deny otherwise legitimate claims which are later resolved, at much higher cost, in favor of the contractor.

In an effort to encourage bidders to avoid the inclusion of speculative contingencies in their prices, many owners include a changed conditions clause in the contract. The clause found in federal contracts is used most frequently, and it provides a body of judicial interpretation giving information on its administration. The use of such a clause has a subsidiary effect in that it impels the owner to be more careful in defining the subsurface conditions which are indicated in the contract.

Such a clause is recommended for all underground construction contracts. It is further recommended that the physical conditions which are anticipated at the site be defined as accurately as possible. In addition, a specified time should be allowed for an administrative determination as to the validity of any changed conditions claim. This can only promote better working relations among the parties and inure to the overall benefit of the owner.

One particularly onerous problem involving the subsurface is the effect of ground water. On some tunnel projects, the actual volume of water encountered becomes a real problem. But in most urban transit projects, the problem is more likely to involve the effect of the water on the stability of the ground and the methods for controlling it.

The "Better Contracting" report recognizes this and recommends special contractual treatment where the effects of ground water cannot be accurately predicted. However, the details of such "special contractual treatment" are not very well described.

There have been cases where the contractor has unsuccessfully attempted to dewater the ground. Such cases often encourage the owner to prepare a dewatering specification, specifying a maximum spacing of wells, a minimum drawdown of the water table, and other requirements. The propriety of such a specification is questionable, to say the least. It is likely to either require a dewatering system which is more costly than necessary or to specify an impossible result. In either case, the owner is the loser.

Frequently, a specific method is specified (such as the use of compressed air). Where the owner has very strong feelings as to its necessity, this avoids any uncertainty on the part of prospective bidders. However, if that method is not actually necessary, the owner will pay a very high premium for his demand.

Sometimes, the contract will require the contractor to install a specified low air plant, with its use to be optional with the contractor. In effect, this insures the contractor against the delay and mobilization cost should air become necessary. He then has to evaluate only the contingency of its possible operating cost.

Considering the advancing state-of-the-art, with the availability of chemical grouts, pressure head mining machines, various types of dewatering systems, compressed air, and freezing, it might seem preferable to simply specify the final result--drive the tunnel or make the excavation without undue loss of ground or subsidence--and leave the method up to the contractor. Even this has its pitfalls, since an incompetent contractor could attempt the wrong method and cause the owner irretrievable delay. On the other hand, prequalification of bidders, a comprehensive geotechnical report, and well thought out performance requirements would minimize the adverse exposure.

In conclusion, it can be said that the contractual provisions relating to the effect of ground water should be thoroughly examined for each particular job and tailored to suit its conditions.

2. Utilities and Obstructions

Obviously, when an open cut excavation intercepts utilities, they must either be temporarily supported in place, or temporarily or permanently relocated. The planner or designer can usually make such decisions based on the local situation.

Utilities overlying a proposed tunnel are another story. Particularly questionable would be, for example, a large sewer or water conduit lying close to the crown of the proposed excavation. To artificially support the conduit might be very costly. But to tunnel under it without causing damage might require very meticulous workmanship.

Other obstructions, such as abandoned piling, foundations, and other tunnels also require consideration. Their presence may affect the method or sequence of construction, the contract time, and the cost.

All available information respecting the location and condition of subsurface utilities and obstructions should be collected. Decisions as to the timing and responsibility for relocations, temporary supports, and related construction methods must be made. All of the remarks heretofore made concerning disclosure of information, disclaimers, and changed conditions clauses apply also to the underground utilities and obstructions.

3. Existing Structures

Existing structures, principally the foundations of buildings, bridges and the like are frequently affected by underground construction. It is recommended that for critical items, the owner should design the system of temporary or permanent support and provide suitable construction specifications. Such items might include heavy foundations which are well within the zone of influence of the excavation, and structures which are strongly exposed to the possibility of high property damage from the excavation work. In some cases, a prohibition against dewatering the ground under foundations beyond the zone of influence might be appropriate.

The protection of non-critical items, such as light footings and items where the property damage exposure is low, can be left up to the discretion of the contractor. The specifications, however, should be explicit, so that the contractor's responsibility is clearly defined. Design criteria for contractor-designed temporary support systems should be specified and the use of a registered professional engineer for their application should be required.

In any case, it behooves the owner to collect as much relevant information as possible about structures exposed to the work. This would include details concerning foundations, loads of piers and footings, etc.

4. Surface Culture

For any underground project, the planner should consider the permanent easements of right-of-way for the finished project, temporary easements for necessary access during construction, the contractor's space requirements for plant and facilities, storage yards, etc., possible sites for the disposal of surplus excavation, and the effect of the contractor's operation upon the public.

Since the municipality owns the city streets, it is natural to utilize these to the maximum for horizontal alignment. However, the validity of this practice should not be taken for granted. Sometimes, the use of private property for certain reaches of the project will produce significant reductions in construction cost. For example, the number and degree of horizontal curves may be reduced and this will facilitate the work, particularly for machine tunneling.

In any case, all necessary right-of-way for the project should be identified and that for each construction contract should be acquired before a contract is awarded. If, for some reason, this is not expedient, the contract should be as explicit as possible regarding the dates for availability of each reach or segment of right-of-way. It should be remembered that owner-responsible right-of-way delays, when not provided for in the contract, can generate expensive claim settlements.

The same logic applies to temporary easements for access during construction. This is obvious for, say, an easement to construct a cut-and-cover section through private property. On the other hand, owners sometimes make the contractor entirely responsible for obtaining entry permits to perform necessary underpinning. As a general rule, it is considered good contracting practice for the owner to provide all required right-of-way, easements, right-of-entry, permits, and environmental approvals which are required for the work. Only those permits associated with his own equipment and facilities should be left up to the individual contractor.

Often during the planning and design phases of a project, the owner is in a position to make arrangements for needed construction facility and storage space on much better terms than could the contractor after the project is advertised and/or awarded. Generally, the more congested the project area and its immediate environs are, the greater is the need to consider this possibility. The same logic should be applied to the disposal of surplus excavation. Frequently, the prospective bidder will be impelled to include a substantial allowance in his bid price for muck disposal. However, during the time required for planning and design, the owner might find many attractive uses for the material.

The contractor's operations must inevitably have many detrimental effects upon the public. However, by thoughtfully selecting the contract packages, the points of access, and the project schedules, these effects can be minimized. Sometimes a section of completed tunnel can be used for muck removal and concrete delivery rather than city streets. Specified work shafts can sometimes be located where the public will be inconvenienced the least.

In brief, the planner and designer should consider all aspects of the interface between the project and the neighborhood surface culture. They should then, within their parameters for the completed project, tailor the construction packages for optimum economy.

DESIGN CRITERIA

1. Configurations

Usually, many alternative configurations will serve the same function. A large double track tunnel may replace two single track bores. A station may consist of two separate narrow chambers with the necessary connecting passages, mezzanine space, etc., instead of one large chamber. Four single bore tunnels might replace a large four compartment tunnel. A deep alignment might offer significant construction cost savings at the expense of minor additional operating costs.

Obviously, these alternatives are affected by the geologic setting, operating problems, surface culture, existing structures, and public preferences. Advancements in the state-of-the-art of certain construction methods can change the economic comparison almost over night.

In these times of relentless inflation and soaring interest rates, many communities simply cannot afford the luxury of copying the configurations of others, nor of setting arbitrary criteria. Each project should be designed for the existing local conditions and the configuration which meets reasonable functional standards at least construction and/or lifetime cost should be selected.

2. Ground Support

In recent years, there have been significant advances in the variety of support systems available for underground construction. Likewise, there have been improvements in the accepted methods for calculating or predicting the structural behavior of a given system. Nevertheless, there still persists a wide variation in the support systems which individual designers specify for similar geologic conditions.

This can exert a marked effect on the cost of construction. Not only is there a significant difference in the cost of materials for different systems. One system might permit greater overall tunneling progress than another, and thus provide lower total costs.

Because of continual advances in the state-of-the-art and because of the widely varying subsurface conditions from project to project and even from job to job, it can only be said that this subject should be considered independently on each project and decisions based on the local situation.

3. Protection of Existing Structures

The previous section considered existing structures in the context of the physical conditions affecting the work. In addition, the design of the permanent structure itself may influence the protection of existing structures. For example, an open cut construction designed for the slurry wall technique may adequately protect adjacent structures and obviate the need for other measures.

In some cases, construction by tunneling methods, when properly executed, may eliminate the need for underpinning or temporarily supporting overlying structures.

The design criteria for any project should consider these possibilities.

4. Construction Methods

Since the behavior of an underground structure is almost always influenced by the construction method, the designer must be acquainted with the construction procedures likely to be used for his structure. It is common to specify the standard of workmanship required for a given project. But in some cases, this may not be sufficient. Where the performance of a given design may be unusually sensitive to the construction method, it may be desirable to specify the method in detail. If this is not done, the contractor may propose an unacceptable method and a costly change order may be necessary.

Even for those elements of a project where the behavior of the structure is not overly sensitive to the construction method, the designer should have his design concepts reviewed for construction feasibility and economy. Construction costs can frequently be reduced without sacrificing the ultimate quality of the work.

5. Materials

The designer should keep abreast of the latest developments in construction materials. Oftentimes, a material which is unsatisfactory in one environment may be entirely acceptable in another and also offer significant cost savings even though its own cost appears high.

By the same token, fluctuations in prices can affect the relative economy of alternative materials. The relative economy of steel elements compared to concrete varies with changes in materials, prices and wage rates.

PLANNING

The planning of the construction program can exert a tremendous influence on the individual construction contracts. Although most facets of the project must be considered in the planning, the following items are of special importance to the construction contractor.

1. Scheduling

The scheduling of the construction involves the timing for the start and completion of each contract. Obviously it must be coordinated with the anticipated cash flow from the project funding, with the scheduling of the various design efforts, and with reasonable delivery schedules for necessary materials and equipment.

It is also important to schedule the bidding phase of each contract so as to attract the most competition. This involves a consideration of the bidding schedules of other similar projects around the country. Keeping the construction industry well informed as to proposed advertising schedules is a necessity.

The scheduling of adjacent and successive contracts requires special attention. When one contract must interface with another, every effort should be made to avoid the dependence of either upon the progress of the other. System-wide contracts, such as track work or electrical installations extending over several civil contracts pose unusual problems in this respect. It would obviously be imprudent to defer a system-wide award until all related civil contracts are complete. But to make the system-wide award too early involves the risk of substantial claims should one or more of the civil contracts delay or disrupt the work.

2. Construction Packaging

This involves the selection of construction contract limits so as to produce the optimum ultimate economy for the owner. Depending on the contracting climate in the particular municipality, it might be advisable to strive for qualified small contractors in the community, the resulting competition could very well produce lower bid prices than would be produced by large construction packages where the competition is limited.

Of equal or greater importance is the structuring of the construction packages. The interfaces should be such that interference between adjacent contractors is minimized. Where a tunneling machine is to be used, thought should be given to the means for removing the machine at the contract limit.

It should also be kept in mind that short lengths of tunnel cannot be economically constructed by modern mechanized methods. If it is desirable to take full advantage of mechanization, tunnel reaches should be of substantial length, even though this necessitates driving them through the station locations. Obviously then, the station should be designed and scheduled to take maximum advantage of the tunnels.

3. Labor

Although the project planner does not wish to become involved in controversial labor problems, he still should consider the labor situation in his community. He should plan the work so as to not unduly strain the existing or potential labor pool. When the labor demands for a project exceed the local supply, efficiency and safety suffer and construction costs rise.

The planner must also be sensitive to local labor customs and working rules. Designs, schedules, and construction methods should, where possible, be tailored so that the work will not be unduly penalized. In some cases, it might be advisable for the owner to sponsor negotiations with the aim of developing working rules which are compatible with local or federal safety standards. This might be the case if compressed air tunneling is contemplated in a community where existing labor agreements have no provisions for such work.

FINANCIAL CONSIDERATIONS

1. Mobilization Payments

One of the problems facing present day contractors is the high cost of equipment. Since the owner can usually obtain his financing on much better terms than can the contractor, it makes economic sense to finance the contractor's mobilization costs. This not only reduces the contractor's financing cost, it improves his cash flow picture and permits him to quote lower prices. In many cases, it undoubtedly encourages more bidders to quote. The "Better Contracting" study recommends mobilization payments.

2. Escalation

For projects of relatively long duration (more than 1-1/2 or 2 years), uncertainty regarding wage and price escalation constitutes a significant risk. Prudent bidders will allow for this in their bid prices, and the owner will foot the bill even though the escalation was over-estimated. By sharing this risk with the contractor, the owner obtains lower bid prices and probably saves money in the long run.

Whether the contract escalation clause should apply to wage rates alone, or should also apply to certain construction materials and other items, depends mainly on the economic climate at the time of bidding. In any case, a formula should be developed wherein the owner assumes only part of the risk, so that the contractor still has an incentive to resist escalation. The "Better Contracting" study has more information on this subject.

3. Retentions

Like mobilization costs, the practice of retaining a percentage of the contractor's earnings increases his financing cost and raises his bid price. The sometimes-used companion practice of conditioning the return of the retained percentage as an inducement to waive all claims at the end of the job is deplorable and only serves to aggravate the owner-contractor relationship. The "Better Contracting" study recommends the retention of only a sufficient sum to induce timely completion of the work.

4. Wrap-up Insurance

This is a controversial subject, and the "Better Contracting" study does not take a position on either side. Ostensibly, it saves the owner money through reduced premiums made possible by centralized administration of the insurance program and the elimination of broker's fees. Most contractors dispute this contention.

A related question concerns the effect of wrap-up insurance upon the contractor's incentive to avoid property damage and personal injury claims.

Nevertheless, any prospective owner of a large-scale project must consider this subject. Since a performance history is gradually being developed on other projects, it is hoped that a basis for a rational decision can be found.

5. Owner-furnished Materials

It is logical that, when a project requires vast quantities of certain construction materials and equipment, the owner is in a much better bargaining position than the individual contractors, who require much smaller quantities. In additon, this relieves the individual contractor from the escalation risk on such items. Also, for items requiring long lead time for delivery, it permits the construction contract to be awarded at a much later date than would be otherwise necessary, thus avoiding some of the scheduling and delay problems which might otherwise develop.

When considering this question, the owner will, of course, consider these and other advantages. He should also consider the disadvantages. It places more responsibility upon the owner--responsibility for testing, expediting, delivery, and storage. Also, in some cases, it locks the owner into a given design (for example, the purchase of all of the tunnel liners for a project).

It can only be said that all aspects of the question should be examined before making a decision.

CONTRACTUAL CONSIDERATIONS

1. Bidder Prequalification

Most underground construction projects require a level of expertise not possessed by the average contractor. However, under the generally accepted American method of contracting, it is difficult to reject a low bid on the grounds that the bidder is not qualified. This problem can be avoided by prequalifying the prospective bidders, as recommended in the "Better Contracting" study.

2. Bid Pricing

This is a subject which deserves more attention than it is ordinarily given. It makes sense to provide bid items for all units of work which the contractor may be required to perform. However, the contract price is developed from the extension of the various unit bid prices by the estimated quantities in the bid schedule. Unless a great deal of thought is given to these, and to the contractual provisions affecting them, the contractor may be in a position to "unbalance" his unit prices to the owner's disadvantage. The result is usually an unnecessary overrun in the final cost of the contract.

There are many ways to avoid this problem, the least effective being a provision permitting the rejection of obviously unbalanced bids. The problem then is to prove what constitutes an unbalanced bid.

This problem is particularly relevant to the items involving the escavation and support of a tunnel, and especially in the early phases of a project, when the behavior of the ground is not thoroughly known. As more experience in a particular area is gained, contractors become more willing to assume all responsibility for temporary tunnel support and bids for excavation and support on a lineal foot basis can be solicited. While this may appear to be in violation of the "Better Contracting" recommendations, it can be justified on the basis of the reduced risk emanating from earlier experience in the area.

The development of an equitable bid pricing schedule policy for a given project is a complicated subject. To avoid unpleasant surprises in the early phases, it is recommended that the initial proposed bid schedules be subjected to a simulated bidding procedure by qualified consultants. After experience is gained on a given project, the schedule format can then be modified as indicated.

3. Value Engineering

This concept is well described in the "Better Contracting" study and in other literature and needs no further explanation here. It is recommended for all construction contracts.

4. Alternative Bidding

During the "Better Contracting" study, a consensus of opinion could not be obtained on this subject. Consequently, the two opposing points of view are explained in the report.

Actually, the principle of alternative bidding can be applied to any degree desired. For example, a tunnel design may indicate segmental rings of 30-in. width. Prospective bidders might be permitted to quote on alternative widths providing they respond completely to the base schedule and also supply all necessary design details with their alternative quotation.

It is the writer's opinion that the owner can enjoy significant savings through the judicious use of alternative bidding procedures. With the value engineering concept, only the money-saving ideas of the low bidder can be evaluated, and then the owner enjoys one-half or less of the saving. With alternative bidding, the money-saving ideas of all bidders become available for the owner to select the one (if any) which promises the lowest ultimate cost.

Alternative bidding permits an owner to specify a particular construction method which is compatible with his design and his interpretation of subsurface conditions without denying prospective bidders the opportunity to propose improvements and/or to assume additional risk if they so desire. Instead of speculating as to the relative economic merits of two or more concepts, the owner can allow the marketplace to decide.

5. Types of Contracts

The "Better Contracting" study describes several types of contracts and gives criteria to aid in the selection for any given situation. While this data is all entirely valid, it is assumed that, at the present time, public transporation projects will continue to use the firm, unit price, competitively bid type of contract.

CONTRACT ADMINISTRATION

No contract is any better than the parties who make it. A good contract can be frustrated by capricious administration or by an unscrupulous contractor. An enlightened administrator and a charitable contractor can live with the poorest of contracts. However, since most owners are neither capricious nor completly enlightened, and most contractors are neither unscrupulous nor very charitable, it behooves the owner to write the most comprehensive and equitable contract he can and then administer it both fairly and firmly.

The contractor is expected to staff the job with qualified people--people with the experience and know-how to complete the job in accordance with the plans and specifications. By the same token, the owner's staff should be equally qualified. And, since the day-to-day relationships on the job largely involve human contact, both staffs should be versed in the rudiments of "how to win friends and influence people."

The resident engineer should be prepared and should be given the authority to render prompt decisions. But the contractor's manager should not expect these without thorough documentation and justification. Letter writing should be a means for record keeping, not a vehicle for outdoing one another with clever rhetoric.

ADJUDICATION OF DISPUTES

If a contract takes advantage of all of the foregoing advice, the incidence of disputes will be minimized. But even so, differences of opinion will inevitably arise. It is said that a tunnel is never completely designed until it is finished. In other words, in the underground, it is almost certain that some changes will have to be made during the prosecution of the work.

1. Change Negotiations

Change negotiations can be greatly simplified when rules governing their application are provided in the contract. Provision for unit price changes for stipulated substantial variations in quantities should be provided for specified important bid items.

Most disputes over the cost of changes revolve around allowance for indirect costs, payment for equipment ownership, and allowances for financing, profit, and home office overhead. As pointed out in "Better Contracting," some of these items can be specified in the contract and others can be made susceptible to mathematical formulae.

2. Arbitration

When disputes cannot be resolved at the job or project level, recourse to litigation is normally the next step. This is always expensive and time consuming. Frequently, both sides could be considered the loser, regardless of the verdict.

Arbitration has been advocated as a substitute for litigation and it has some advantages. It can usually be initiated sooner and the arbitration panel is usually better qualified in the particular technical field than is the average judge or jury. Even so, it still has obvious deficiencies.

Even though it can normally be initiated faster than litigation, it is frequently delayed until after completion of the contract. The result is that much more time is consumed in the collection and submission of evidence than would otherwise be necessary. This discourages the recruitment of highly qualified arbiters who might find it difficult to devote their time to extended hearings.

Under the guidance of Subcommittee No. 4 of the U. S. National Committee on Tunneling Technology, the American Arbitration Association is currently preparing an Arbitration Program specifically aimed at underground construction. The nationwide facilities of AAA would be made available whenever needed. A slate of highly qualified individuals will be recruited and ready to serve.

It is anticipated that three types of arbitration will be offered:

a. Final and Binding Arbitration
b. Non-binding, or Advisory Arbitration
c. On-going Arbitration, available for the duration of a contract or a project.

It is felt that any municipality will be able to benefit from one or more of these options and that this program will result in substantial savings in the cost of underground construction. The program is scheduled for publication about the first of the year and it is recommended that all interested parties acquaint themselves with its provisions.

CONCLUSION

While the technical aspects of underground construction have received a lot of attention recently, we have been neglecting the contractual and legal aspects. As a result, we see more and more financial disasters, cost overruns and prolonged litigation. Much of this can be avoided if we will devote the kind of energy to these problems which we have to the technical.

PAPER 7

Underpinning for Transportation Tunnels

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PAPER 7

UNDERPINNING FOR TRANSPORTATION TUNNELS

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INTRODUCTION

Underpinning is an ancient building art. In early times it was mostly remedial; i.e., its purpose was to provide adequate support for settling structures.

Being so recognized as an art, for many years design and construction of underpinning work was mainly done as the sole responsibility of specialist foundation contractors. Especially was this true in building construction (as opposed to transportation tunnels) but even on New York City subway work - by far the greatest amount done - the contractor was required to "maintain, protect and if necessary, underpin" the buildings along the route. Arguments sometimes arose over the "if necessary" clause, but in the main, the entire responsibility was on the contractor.

In recent years, that is, since the proliferation of geotechnical consultants, design standards and construction specifications have become much more codified. Much of this has taken the form engineering rationalizations of procedures that contractors have empirically found to work well. This is especially so beginning with the great San Francisco Bay Area Rapid Transit (BART) project. This trend has been continued on subway work now going on (Washington, D.C. and New York) and work just getting underway (Atlanta and Baltimore). "To underpin or not to underpin?" that is the question that has been answered in quite some detail by the New York City Transit Authority. (NYCTA) (15).

NEED FOR UNDERPINNING

"It is impossible to carry out any excavation, including a tunnel excavation, without causing some subsidence and deformation of ground (21)." The purpose of underpinning is to minimize the damage that may be caused by such subsidence and deformation (5).

In this context, "Underpinning is the permanent supporting structure designed to transmit foundation loads to the lower bearing levels necessary to securely maintain the structure being underpinned." (23) To the above definition, the NYCTA has added their following long-standing requirement that the adjacent structure loads be "permanently transmitted to ... an appropriate lower soil level that will prevent foundation related pressures from being transmitted to the final railroad structure." (15)

CATEGORIES OF STRUCTURES FOR WHICH UNDERPINNING MAY BE NECESSARY

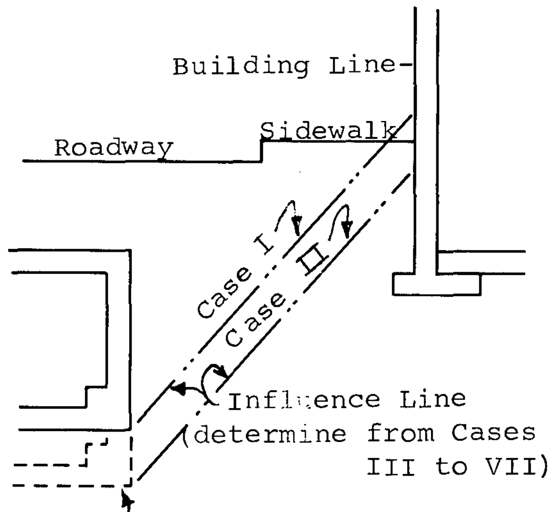
The concept of two categories was formalized by BART and this principal has been followed in most subsequent transit specifications. For example, it is stated:

"**Structure Categories**":

a. Category 1 structures are structures for which underpinning is necessary and has been designed by the Engineer.
b. Category 2 structures are structures which the Contractor has the option to support temporarily, underpin, or both, or not to support or underpin because they are likely to be affected by this operation. The decision rests solely with the Contractor who is entirely responsible for the results." (23)

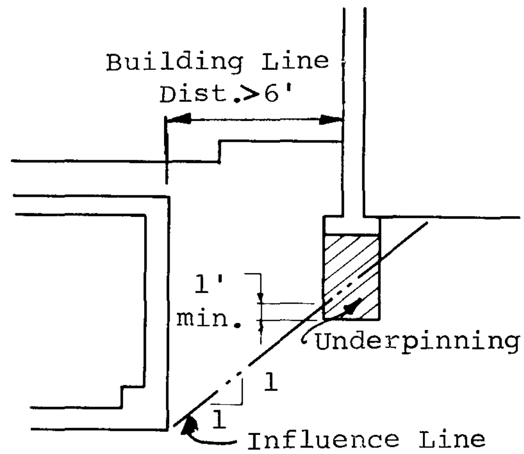
The advantages of such an arrangement are several: First, on important buildings where damage might in absolute terms be high, a conservative plan can be drawn up by the Engineer; second, in the bidding stage all bidders know where they stand so that public project requirements for equal competition are fulfilled. The disadvantages are: First, the owner reaps no advantage in an economical price which the Contractor's (or his underpinning subcontractor's) know-how, experience and ingenuity may offer; second, there is less incentive or possibility of the underpinning being done by an experienced, competent specialist and, consequently, there is more burden placed on the Owner for close inspection and policing of the work. Third, in the event of unforeseen circumstances requiring changes in the Engineer's plans there will be more difficulty in negotiating change orders. By placing buildings in Category 2, the Owner can forge ahead in the ways cited above. Recent practice has been to place small, unimportant buildings in Category 2. To this must be added buildings situated beyond some influence line from the bottom of the adjacent excavation, and also buildings with foundations which are deep relative to influence lines. (15) In the execution of subway contracts in recent years it has generally turned out that no underpinning of Category 2 buildings has been done by the Contractors. This is not to say that

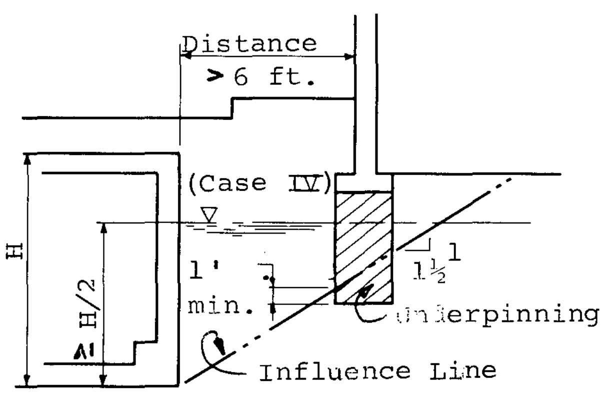
Case I - No Underpinning or Maintenance Required. Influence line does not intersect building line below the ground surface.



Case II - No Underpinning but Maintenance may be required. Influence line intersects building line below ground surface but above the subgrade of the foundation.

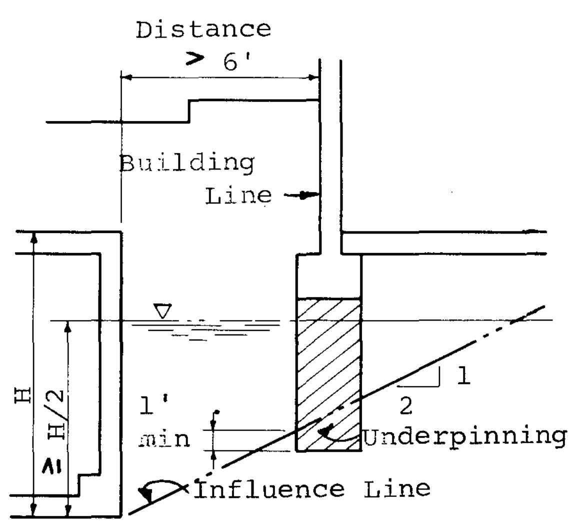
Case III - Average Soil Conditions. No water (or below sub grade of excavation). Building line more that 6'-0" Distant.





Case IV Average Soils Conditions
Water Present

Case IVA Poor Soil Conditions
No Water Present



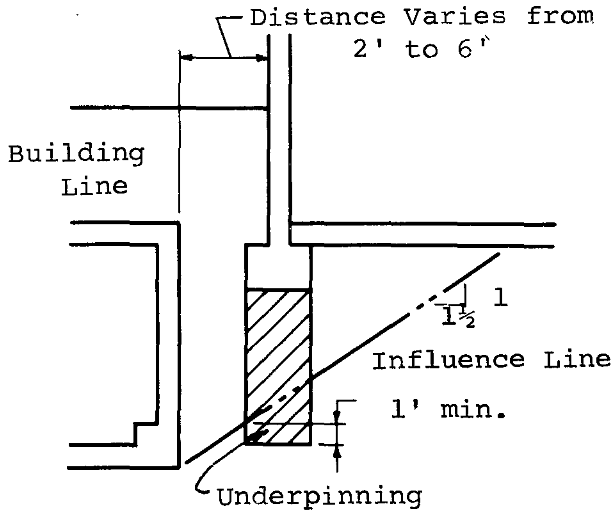
Case V - Poor Soil Conditions
Water Present

A steeper influence line (1:1 in Case IV - 1:1½ Case V) may be used if the ground-water level outside the excavation is either maintained at pre- construction levels by the use of a water-tight earth retaining structure or effectively controlled by a dewatering technique approved by the Engineer.

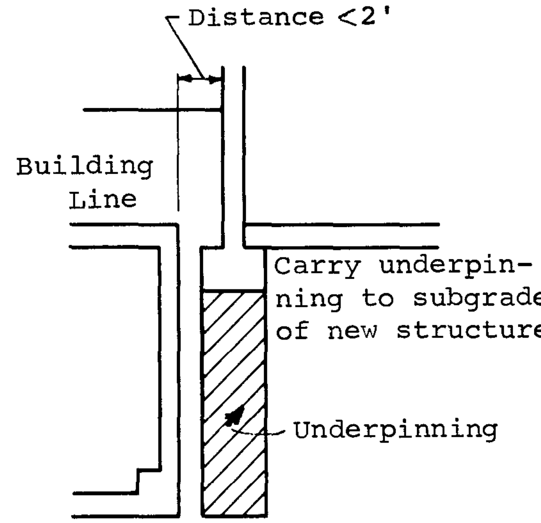
Underpinning Criteria, NYCTA Field Design Standards, Ref.(15)

FIGURE 1

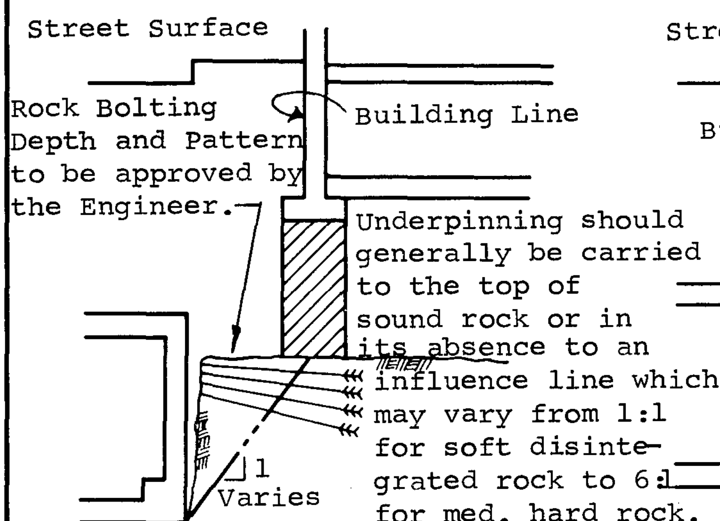
Case VI - Average or Poor Soil Conditions with or without Ground-water .



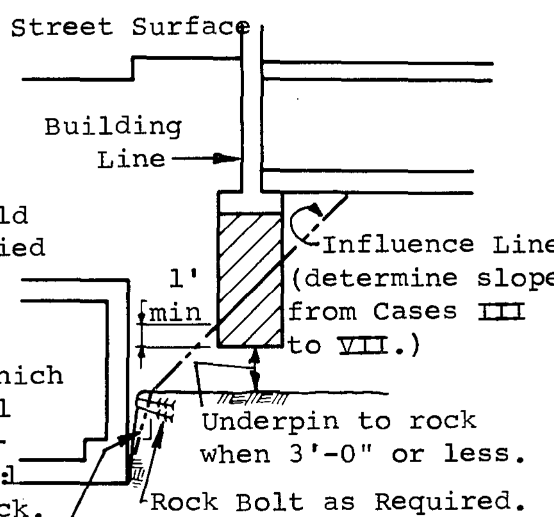
Case VII - Average or Poor Soil Conditions with or without Ground-water.



Case VIII - Top of Rock above Subgrade.



Case IX - Top of Rock Above Subgrade.



Underpinning may be stopped at an elev. above the designated influence line if suitable reinforcement of the rock face adjacent to the underpinning can be provided as approved by the Engineer.

May vary from 1:1 for soft rock to 6:1 for medium hard rock. In hard sound rock the influence line can be considered vertical.

Underpinning Criteria, NYCTA Field Design Standards, Ref. 15

FIGURE 2

damage to buildings has not been done, but generally it has been minor.

To mitigate the disadvantages of placing structures in Category 1, it would be wise for the Owner's form of contract to contain a Value Engineering clause, giving the opportunity for the Owner to save money, time, shed responsibility, and perhaps even have a safer job. As examples of this, two cases may be cited:

Case 1 - Chrystie Street Subway, Lower East Side, Manhattan, N. Y. A new subway was to pass underneath an existing-track subway. The ground at the site was sand with a fairly high water-table. The Owner's suggested plan, shown in the contract documents, was to support the existing structure by means of rivetted plate girders picking up the steel wall and interior columns. These girders inside the old structure would have had to have been installed in the wee, small hours of the night (at great overtime expense) to minimize interruption of train service. However, the underpinning subcontractor proposed a system of pit and Pretest pile underpinning which eliminated the need to enter the existing, operating subway. The Owner accepted the proposed alternate under a note on the contract drawings which permitted him to consider alternates. (7) See also Fig. 6.

Case 2 - Fine Arts Building, Seventh and "G" Streets, N.W., Washington, D. C. Underpinning for the excavation of the METRO nearby was shown on the bidding documents in detail as to both design and procedures. In general, cut stone walls of a monumental building begun in 1836 were to be first carried by structural steel needles and grillages. Next, large pits dug to approximately subgrade were to be installed during which time the building's walls were suspended, as it were, over the pit excavation. Large settlement cracks appeared in the art gallery rooms above forcing them to be closed to the public, all of which was headlined and illustrated in the newspapers of the day. In this case, in spite of a Value Engineering clause, the Owner had not encouraged the submission of alternate underpinning methods which would have avoided the trouble (14). Simple pit underpinning in short sections without the use of needling would have been perfectly adequate for such a problem; to cite one case out of many where this was done, there was the very successful pit underpinning of The White House in 1950 (19).

TYPES OF UNDERPINNING

1. Pits

Pit underpinning is the oldest and simplest type of underpinning. It is more properly called pier underpinning as it is the concrete pier cast within the confines of pit sheeting that forms the underpinning. Pits are generally square or rectangular. Minimum cross-sectional area is 3 ft. by 4 ft. It is determined by the elbow room a man needs in order to dig with a shovel. A large pit might be as large as 10 ft. square. Sheeting of pits is done by means of 2 in. or 3 in. thick planks, often called "lagging" which support each other at the corners of the pits. For drainage and back-filling the inevitable over-digging behind the boards, approximately 1 in. to 1-1/2 in. wide louvers are allowed to remain between the boards. This is very important to keep loss of ground at a minimum and allow the least settlement. Generally, in sandy soil, only 6 or 8 or 10 in. of depth is excavated at one lift; then a set of lagging is installed. In cohesive soils, these lifts may be increased to 5 or 6 ft.

In Chicago, hand-dug foundation piers - until the advent of machine drilling - were circular wells, four ft. or more in diameter, dug 5'-4" at a time; lagging was 2"x6" tongue-and-groove boards placed vertically. These boards were supported against the lateral pressure of the clay by means of small steel channels bent to a circle (26). These Chicago wells are still used in underpinning; since they require practically no headroom, they can be sunk directly beneath footings and walls of building to be underpinned.

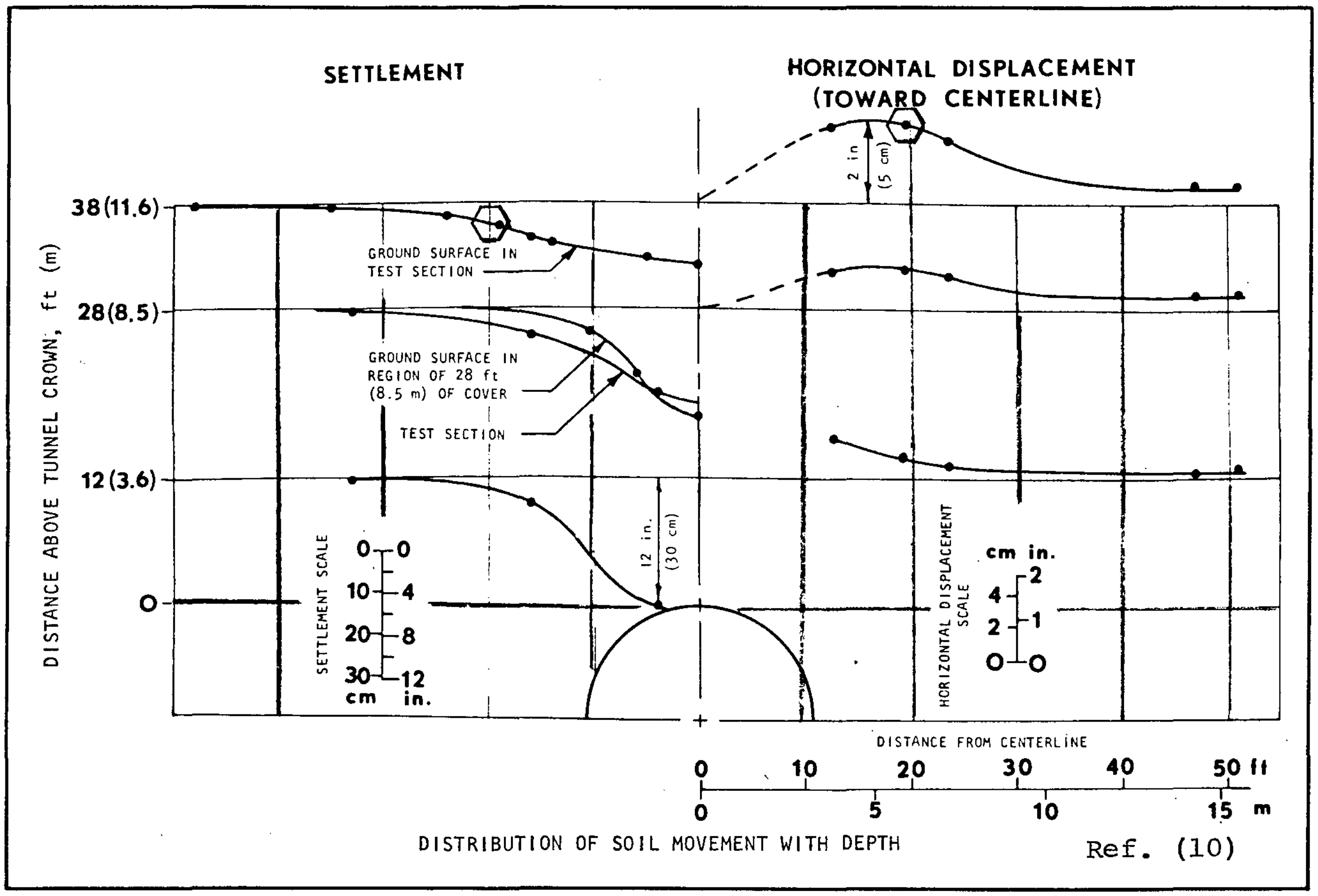
To obtain the required bearing area on the ground at the depth down to which the pits or wells are put, "belling" or enlarging the base of the pit or well may be done, provided the soil is cohesive enough (19,26).

The day after the concrete is poured to within about 3 in. of the bottom of the footing overhead, it is permissible to drypack or ram into the 3 in. space a very slightly moistened, crumbly mixture of one part plain Portland cement and one to two parts of masonry sand. The day after that, digging of the next adjoining underpinning pier may be begun.

Shallow concrete piers or existing footings themselves may be used for "maintenance" underpinning. The "maintenance" underpinning can be an economical method of keeping a building more or less level, in spite of the subsidence generally caused by tunneling in soft ground. This subsidence generally shows up at the ground surface as a "trough" or perhaps several inches over the center line of the tunnel and extending out some distance beyond the vertical projection of the side of the tunnel, gradually tapering off to zero. If a wall of a building should fall within the "trough" zone, it might suffer damage owing to settlement and also lateral movement towards the center line of the tunnel. Field investigations have shown that horizontal movements can equal settlements to one side of the tunnel. For example, in Washington, D.C., a point, shown thus ◯ on the plot of soil movements (Fig. 3) on the surface of the ground 20 ft. from the center-line of a single track subway tunnel with 38' cover, settled

2 in. and moved horizontally toward the centerline of the tunnel 2 in. The tunnel was 21 ft. in diameter, shield driven through drained granular river terrace deposits (10). Full underpinning to the depth of the invert would most likely prove to be quite expensive. However, hydraulic jacks, manned as the heading of the tunnel approaches and goes by, placed on top of the shallow concrete piers or footing and reacting against the wall or column above can be used to keep the building fairly level. This method is useful where settlements are not expected to be too great or to involve lateral movements as well. Even so, some cracking may occur. On the BART project, 35 buildings were equipped with "column pickups" (11). In the event of large settlements or lateral movements tipping of the pier may take place, making it difficult to use the jacks. Pretest piles may also be used for maintenance underpinning.

Currently, average pit underpinning is priced about $400 per cubic yard (of concrete in the pit) but, of course, there can be a wide variations from this figure.



DISTRIBUTION OF SOIL MOVEMENT WITH DEPTH

Ref. (10)

FIGURE 3

2. Pretest Piles

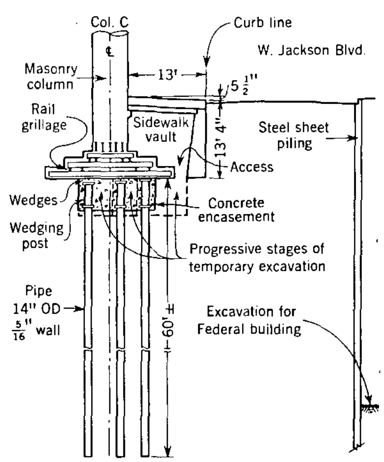
In water-bearing soils, which may make any excavation slow, expensive and dangerous because of loss of ground, open-end jacked pipe piles are usually used. To start them, a small "jacking" pit is dug and sheeted beneath a portion of the footing. The remaining portion of the footing should be sufficiently large to safely carry the loads coming down from above. The footing carries itself and the wall above by natural arching action. This is an important phenomenon which enables, in many cases the underpinning to be done without preliminary support, such as shores, needles, grillages, etc. A 4 ft. length of pipe, say 12 in. to 16 in. in diameter, is then set plumb on the earth at the bottom of the pit and a 40-ton or heavier capacity hydraulic jack is placed on top of the pipe which is capped by a steel plate. The jack reacts against a plate dry-packed to the underside of the footing. It forces the pipe into the ground, some of the soil rising inside the pipe. When the top of the pipe has reached the bottom of the pit, the jack is removed, another 4 ft. length of pipe being connected by an outside sleeve which has a sliding fit over the pipes. No welding is necessary where only vertical loads are concerned. The jack is then set in place on top of the pipe and the jacking-down process is repeated.

If resistance is encountered, jacking is facilitated by excavating the ground from within and even below the tip of the pile. This excavation or "cleaning-out" is done by a variety of tools: dwarf orange peel buckets, post hole diggers and "pancake" augers (all usually hand operated); suction pumps and jets; and when large gravel, cobbles and boulders are present, churn drilling equipment may be used. This churn drilling has become fairly common in pile underpinning on the Washington METRO where gravel and boulder strata are frequently found in the alluvium. Attempts to develop powered augers have not been outstandingly successful; they are awkward to set up and require wide and deep pits for head room and clearance. These excavations themselves can aggravate or be the cause of settlement.

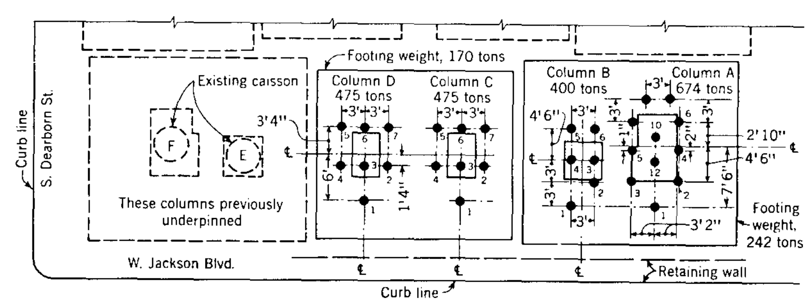
Upon the tip of the piles reaching the desired stratum, two hydraulic jacks are placed on the top of the pile and a load of 150% of design load (design loads commonly range from 40 tons to 120 tons) is applied to the pile. If no settlement occurs, a "wedging beam", i.e., a vertical I-beam, is placed between the two plates and tightly wedged up with steel wedges. This is done while the test load is being applied by the jacks and it is done to prevent "rebound" and subsequent settlement of the pile, the so-called "hysteresis" effect (24,P.633). Since its invention 60 years ago, underpinning piles installed in this manner have been called "Pretest" piles.

Under certain circumstances, even where there is no ground water problem, Pretest piles can be more economical than pit underpinning. For example, to put down a Chicago well beneath a small footing carrying 100 tons, it might be necessary to first install a column pick-up system or "preliminary support", a combination of needle beams, grillage beams, mats and screw jacks before going under the footing to sink the caisson (26). This system can be costly and quite inconveniently take up valuable space in the basement of the building. Instead, two Pretest piles can be installed one at a time without need for preliminary support, thus effecting considerable savings. As an example of this, in the early 1950's when Wacker Drive was being built it was found much cheaper to install Pretest piles under the old four and five story buildings than the conventional Chicago caisson underpinning.

Another striking example was the underpinning at two different times (1940 and 1967) of parts of the Monadnock Building. The earlier underpinning, required by the construction of the Dearborn St. subway, extensively employed grillages, needle beams, mats, screw jacks and Chicago caissons (26). The recent underpinning, made necessary by the excavation for the Federal Office Building across Jackson Boulevard, eliminated all these and substituted instead 60-ft. long Pretest piles of 14" O.D. by 5/16" wall pipe. (3).



Section through underpinned column adjacent to excavation for the Federal building.



A small excavation permitted moving under old footings to install Pile No. 1 and pick up 80 tons of the load using the pretest system of transfer. Then the other piles were installed progressively from carefully sheeted excavations while part of the load was still carried by the spread footing.

Pretest Pile Underpinning of the Monadnock Building, Chicago 1967 (3)

FIGURE 4

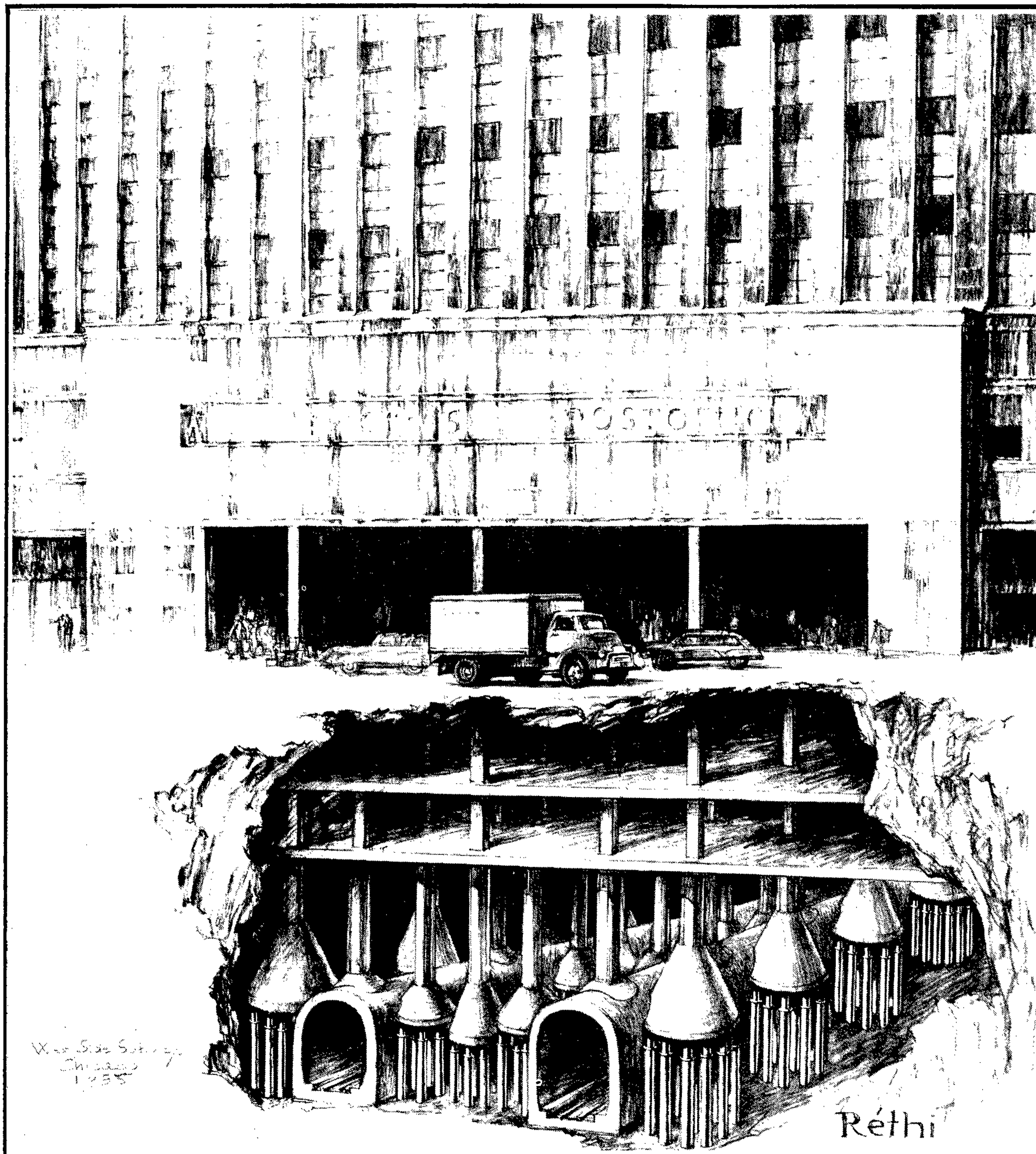
Large loads may also be carried on Pretest piles. For example, in the sandy soil of Long Island 20,000 linear ft of 16 in. by 0.375 in. thick cylinders were jacked down 30 ft to 70 ft (average depth of 51 ft) to support concrete abutments and retaining walls of the Long Island Railroad and to permit shield-driven tunnels for the new Archer Avenue subway to pass underneath these abutments. Design loads on these piles were as much as 70 tons each. Pretest piles in ground unobstructed by gravel or boulders are being currently bid at the price of $200 to $300 per linear foot.

In 1955 a very unusual installation of Pretest pile underpinning was made in connection with the driving of subway tunnels under the main United States Post Office Building just west of the Chicago River. The tunnels were being driven under compressed air through the blue clay when they unexpectedly encountered the edges of large bells of caissons resting on hardpan. According to record plans made available when the job was bid, the caissons had originally been sunk to rock without enlarging the shafts. This was obviously not the case and underpinning of the caissons was called for. Pretest piles were installed under each caisson working also under the compressed air within the tunnel, one at a time by drifting in sideways from the tunnel heading. An obvious lesson to be learned from this is that one should not automatically trust records. Some investigation or verification should be attempted. Especially is this true of old wooden pile foundations where the tips of the piles puport to have penetrated sufficiently below an influence line (See drawing of Fig. 5).

In a current example in Chinatown in New York city cast-in-place concrete piles were driven as foundations for a high-rise housing development. A section of the Second Ave. Subway was then excavated alongside to within 7 ft of subgrade. The uncompleted building started to settle. Excavation was halted and Pretest piles were jacked between the driven piles. In this case the records were accurate, but the design of the driven piling was skimpy.

4. Driven Piles and Augered Caissons

For underpinning walls and columns of buildings driven piles and augered caissons have been used in conjunction with needles and grillages. Sometimes it can be arranged that the driven piles are located outside the building and this is economical. Otherwise, the piles must be driven in short lengths in limited headroom inside the building (8). The same goes for augered caissons. With unlimited headroom, conventional machine drills can be used; inside buildings caissons would be dug by hand.



Underpinning of Post Office, West Side Subway, Chicago. 16 caissons were underpinned. Column loads ranged from 1186 kips to 4376 k. Variation in diameters: Shafts 4 ft. to 7 1/2 ft., bells 12 ft. to 24 ft. There are 6 to 22 Pretest Piles per caisson - pile shells 16 in. x 5/8" in. - tips minimum 3 ft. below subgrade of subway (El. - 63) - bottom of bells El. 48 - design working load each Pretest Pile = 200 kips. All underpinning and tunneling performed under compressed air.

FIGURE 5

An application of this type of underpinning might be the support of the existing elevated columns at Wells and West Monroe Streets, Chicago Central Area Transit Project, Monroe Line. Where such columns fall within the net line of the subway structure, the load may be transferred in the final stage to the subway roof which will be heavied-up to take such loads. (19, pp. 212-218).

5. Ground Freezing

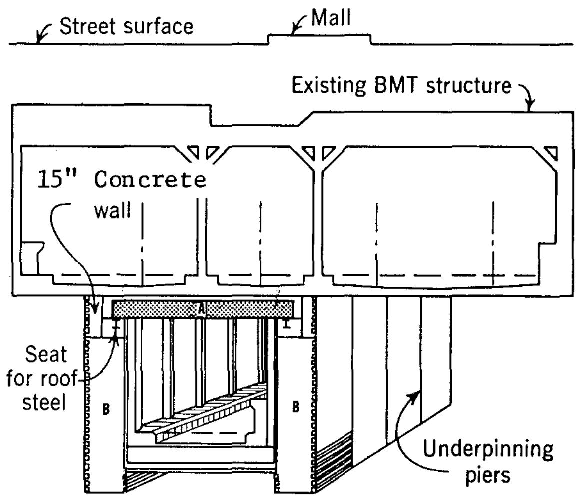
This could be used as a solution to certain knotty underpinning problems for example, where very fine, difficult to drain, water bearing silty sand exists beneath footings to be underpinned. This can make it difficult or almost impossible to install jacking pits. Freezing, to facilitate excavation of jacking pits, would solve this problem as it was done in righting a tilting, 24-story building in Sao Paulo, Brazil in 1941 (5a, 5b). In the construction of a highway underpass in Minneapolis a few years ago a church was underpinned by means of "ground freezing." The soil was sandy, no ground water being present. At first blush, it seemed cheaper than conventional pit underpinning. But this author's post-mortem analysis showed that in the end, owing to the length of time required to maintain the ground in a frozen state (with freezing plant operators receiving double-time rates for overtime on nights and weekends and holidays), the freezing alternate proved to be somewhat more expensive.

6. Chemical Grouting

More frequently used than freezing in the right kind of soil (free draining), sandy chemical grouting has proven to be an acceptable underpinning method. Similar to freezing, it requires closely spaced holes to be drilled because the chemical solutions cannot be made to travel very far. The chemicals are very costly materials to buy and special plant, specialist personnel and close supervision are needed for this type of work.

7. Root Piles

Another type of pile used as underpinning, especially in Europe but hardly at all in the United States, is the so-called "root pile." This is a very small diameter (3 to 5 in. generally max. 8 to 9 in.) pile drilled into the ground by exploratory type rotary drills. After the hole is drilled to the desired depth, a steel bar is inserted into the drilled hole and grouted in place somewhat similar to the installation of earth tiebacks. The load of the footing is picked up by the steel bar which is grouted into the footing. The drills work from a level above the footing instead of beneath as with a jacking pit so that it is usually not necessary to dig beneath the footing. Root piles carry small loads, 10 to 15 tons, and so a great many are required

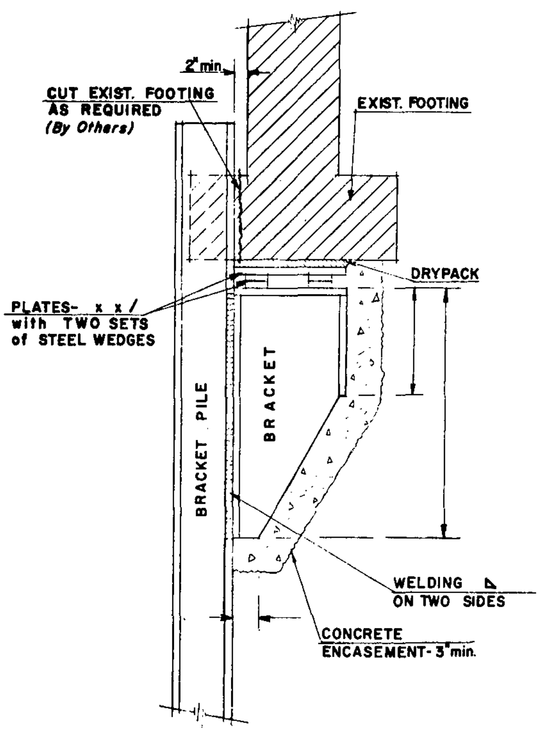


Permanent steel roof beams (A), carried on concrete underpinning walls (B), used for temporary support during construction

Chrystie Street Subway, New York City

FIGURE 6

Bracket Piles



BRACKET DETAIL

N. T. S.

Where loads are not heavy, as say for the walls on one, two, and three story buildings, they can be underpinned - clearances permitting - by means of hammer driven "bracket piles" These are generally H-piles, driven immediately alongside a wall and picking up the wall by means of a welded-on bracket installed in a small pit dug under the wall opposite the location of the pile. Of course, this eccentric loading puts some bending, as well as axial load, into the pile.

FIGURE 7

compared to other types of piles. It is a simple matter to batter them at many different angles (2,16).

8. Compaction Grouting

This is a relatively new grouting technique, and consists of using the injection of a highly viscous soil cement grout to form a grout bulb. Through pipes driven to the necessary depths a grout is extruded into the soil mass. The highly viscous soil-cement remains at the tip of the injection pipe and grows as a bulb, physically displacing the soil particles adjacent to the grout. Controlled pressure exerted by the grout bulk is used to life the soil mass and structure to accomplish a levelling of the structure.

The Foregoing is an outline from the specifications for a section of the Baltimore Subway for which, as of this writing, bids are being asked (1). The section is mainly shield-driven tunnels with a thick cover of stiff soils supporting buildings as high as 6 stories. The compaction grouting is looked upon mainly as a "maintenance" operation. Compaction grouting has been used mainly in California as a method for levelling light structures built on random fills and other poor soils (4,22).

9. Support for Lagging

Because subway structures often extend out to building lines, the underpinning can serve a second purpose, that of supporting lagging. In other words concrete piers, Pretest piles and bracket piles can act as soldier beams (19, pp. 178-9). In the case of the Pretest piles, if the spans between wales are long it may be found desirable to weld the joints in the steel shells; this is most economically done as the general excavation exposes the joints.

REFERENCES

1. Baltimore Region Rapid Transit System, "Bolton Hill Tunnels," Contract No. NW 03-02 Specifications Book, September 1975.

2. Bares, F. A., "Use of Pali Radice (Root Piles) for the Solution of Different Foundation Problems," ASCE/EIC/RTAC Joint Transportation Engineering Meeting, Montreal Preprint MTL-41, July 1974.

3. Bethel, Roger, "Underpinning a Landmark," Civil Engineering-ASCE Vol. 38, No. 8, August 1968, p. 67,68.

4. Brown, Douglas R. and Warner, James, "Compaction Grouting," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 99, No. SM8, Proc. Paper 9908, August 1973, pp. 589-601. Discussion by Brown, R. W. and closure Vol. 100, No. GT 12, December 1974, pp. 1300-1303.

5. Caspe, M. S., "Surface settlement adjacent to braced open cuts," Transactions ASCE 132, p. 373, 1967, Vol. 92, No. SM4, July 1966, pp. 51-59.

6a. A. Dumont Villares, "Underpinning of 26-Story Companhia Paulesta de Seguros Building, Sao Paolo, Brazil," Geotechnique Vol. 6, No. 1, pp. 1-14 (March 1956).

6b. "Sao Paolo Tower is Righted with Jacks," Engineering News-Record, Vol. 131, p. 113 (October 21, 1943).

7. Goldfinger, H., "Permanent steel used as subway shoring," Civil Engineering, March 1960.

8. Gould, Howard R., "How to Drive Piles Where You Shouldn't," Construction Methods & Equipment, August 1959.

9. Greathead, John F., "Meeting the Soil Halfway on William Street," Public Service Record (NYC), Vol. II, No. 9, September 1915, pp. 1-8.

10. Hansmire, Wm. H. and Cording, Edw. J., "Performance of a Soft Ground Tunnel on the Washington Metro, First North American Rapid Excavation and Tunnelling Conference, Chicago, 1972, Vol. 1, Chapter 23, pp. 371-389.

11. Kuesel, Thomas R., "Soft Ground Tunnels for the BART Project," First North American Rapid Excavation and Tunnelling Conference, Proceedings, Vol. 1, Chapter 20, Chicago, 1972, pp. 287-313.

12. "Massive underpinning job paves way for construction of D. C. Subway," Engineering News-Record, Vol. 186, No. 5, February 4, 1971, p. 37.

13. Mellert, Frederick H., "Underpinning the Old Post-Office Building," Public Service Record (N.Y.C.), Vol. IV, No. 6, June 1917, pp. 1-3,7.

14. Mergentime, Charles E., "Underpinning Massive Structures," Civil Engineering, March 1972, pp. 40,41.

15. New York City Transit Authority, "Underpinning, Section UP, Field Design Standards, Engineering Department, March 1, 1974.

16. "New underpinning system solves explosive problem," Engineering News-Record, Vol. 191, No. 4, July 26, 1973, p. 21.

17. Peck, Ralph B., "Deep Excavations and Tunneling in Soft Ground," Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico, 1969, State of the Art Volume, pp. 225-281.

18. Peck, R. B., Hendron, A. J. Jr. and Mohraz, B., "State of the Art of Soft Ground Tunneling," First North American Rapid Excavation and Tunneling Conference, Proceedings Vol. 1, Chapter 19, Chicago, 1972, pp. 259-266.

19. Prentis, E.A. and White, L., "Underpinning -- Its Practice and Applications," 2nd Edition, Columbia University Press, New York, 1950.

20. "Prestressed beams underpin subway tunnel," Engineering News-Record, Vol. 186, No. 15, April 15, 1971.

21. Ward, W. H., "An introduction to tunnelling for the local authority engineer," Underground Services (Brentwood, England), Vol. 3, No. 2, 1975, pp. 36-39.

22. Warner, James and Brown, Douglas R., "Planning and Performing Compaction Grouting," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT6, Proc. Paper 10606, June 1974, pp. 653-666; Discussion by R. W. Brown, Vol. 101, No. GT2, February 1975, pp. 211-12; closure.

23. Washington Metropolitan Area Transit Authority (WMATA), "Design Standards and Specifications," 1973, Section 210.

24. White, Edward E., "Underpinning," Chapter in Foundation Engineering Handbook, edited by Winterkorn, H. F. and Hsai-Yang Fang, Van Nostrand Reinhold, N. Y., 1975, pp. 626-648.

25. White, Robert E., "Caissons and Cofferdams," a Chapter of Foundation Engineering, G. A. Leonards (Editor), McGraw-Hill, New York, 1962.

26. "Underpinning Beats Subway Threat to Famous Chicago Building," Engineering News-Record, Vol. 125, No. 13, September 1940, pp. 64-68.

27. Ware, Kenneth R., "Underpinning Methods and Related Movements," R.E.T.C. Proceedings, Vol. 2, 1974, pp. 1255-1274.

PAPER 8

French Practices in Underground Construction for Transportation

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PAPER 8

FRENCH PRACTICES IN UNDERGROUND CONSTRUCTION FOR TRANSPORTATION

by

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INTRODUCTION

Perhaps the only sensible solution to the problem of traffic congestion in urban sites is the construction of underground transportation systems. This, however, generally encounters numerous technical difficulties and often involves unusually high construction costs. A careful study of the available techniques which may be feasible and suitable for this type of work is of prime importance, and it is the responsibility of the engineer that this study is made with due consideration to all details.

In this respect RATP (REGIE AUTONOME DES TRANSPORTS PARISIENS) has gained valuable experience because of the great number and variety of underground structures which this Authority has built in the past and intends to build in the future. Indeed, an important program has been implemented in a few years in order to create an express railway system and extend the present metro, necessitated by the extensive suburban population expansion. Thus, RATP has been responsible for a variety of projects, i.e., from structures in open cuts to deep tunnels through very loose ground or hard rock.

In this paper I will review the main underground construction techniques as they are currently used in Paris. Essentially, these may be divided into two groups: those that involve open construction or cut-and-cover, and those that involve primarily tunneling work.

Where the existing geology and the number and type of subgrade obstructions present favorable conditions, underground structures are built in open cut and as close to the surface as practicable. The advantages inherent in this case are best utilized if the difference in grade between the platforms and the ground surface can be kept to a minimum. Examples of underground railways built in this manner are found in Paris, Montreal, Milan, Santiago etc. However, when costs and time considerations become governing factors, and when traffic maintenance is essential, it is necessary to adapt techniques allowing rapid and economical construction. Improvements in the Berliner method (soldier piles with lagging) and the slurry trench method allow this goal to be fulfilled.

It is not always possible to carry out the work in open cuts due to a variety of reasons, for example existing facilities, high cost involved in diverting existing utilities, streets which are too narrow, etc. It is then necessary to consider tunneling, and in this case the control of ground movement and surface settlement due to construction becomes part of the organizational problems. This situation is, in fact, more critical where cities have been built near the banks of rivers, seafronts or lakefronts, and their geology is dominated by the presence of sandy soils with water. This requires the use of technical methods specially suited to soft ground, unless the choice is to tunnel at great depth as in London or Moscow. In any case, these techniques should not result in any appreciable ground movement and settlement since this may be detrimental and eventually dangerous to surroundings.

Examples from actual construction work in Paris will help illustrate how this problem has been solved. Whether, the construction is carried out using cut-and-cover or tunneling, it is essential that the associated operations do not produce effects harmful to the neighborhood. In particular it is necessary to intercept the transmission of vibrations due to construction operations.

OPEN CUT METHODS

Construction in open cut is not possible unless the face of the excavation is protected. From the technical point of view, problems related to cut-and-cover work involve ground support methods and bracing requirements. In the past this has been solved with soldier beams and lagging, and more recently with the use of slurry walls, with or without tiebacks. Indeed, both these techniques can provide efficient ground support, and are relatively inexpensive. Nonetheless, the overall cost depends on other factors and mainly on site conditions such as the divertion of underground utilities, and it may well be that for the same site a structure built in open cut to a certain depth within a subsoil environment sometimes has a higher overall cost than if the same structure is built by tunnelling. Thus, each case must be studied carefully and in terms of all pertinent factors before a decision is made about the use of the most favorable technique.

1. Berliner Walling

This method, initially used extensively in Berlin, is known to American practitioners as soldier pile walls. Vertical holes are prebored at intervals from 6 to 12 ft. I beams are then lowered in the holes and set in place, and the space is filled with a lean concrete mix. The excavation is carried out to suitable depths so that the ground is at least temporarily competent to remain stable, and usually from 3 to 9 ft. Lagging is progressively inserted between the soldier piles after the concrete casing is removed. This lagging may consist of wood boards, precast concrete panels, or may be cast-in-place concrete.

In this manner a ground support system is built from the surface down to the base of the future excavation, and the main structure is then constructed within the area thus protected. Almost invariably the soldier piles are set fairly below the bottom of the excavation to provide adequate embedment, but when the depth exceeds a few meters this embedment alone is not sufficient to provide overall stability and cross bracing or tiebacks are commonly used. Quite often single struts provide the lateral bracing and are set against the walls. This is practical and economical if the walls are not too far apart. The presence of any cross bracing with an excavation may affect earth moving operations and may also interfere with the construction of the permanent structure.

On the other hand, tiebacks allow a completely unobstructed excavation area, and in this respect it has been possible to achieve some spectacular results. The ability to support a wall by means of ground anchors has enabled the elimination of the actual length, depth or width of the excavation.

The Berliner wall, an attractive solution in many instances, is nevertheless somewhat restricted. For example, this ground support system is not suited to water bearing ground, or where the ground cannot stay temporarily stable at least until the lagging is installed. Moreover, the direct transfer of vertical loads is not possible.

2. The Slurry Wall Method

Under unfavorable soil conditions, soldier piles with lagging cannot be installed unless they are used in conjunction with other ground engineering techniques, for example grout injections to strengthen, consolidate and stabilize the soil to be excavated. This operation is time consuming and often implies high expenditures. Moreover, injections cannot always be carried out, regardless of ground conditions. Thus, the development of the slurry wall method proved to be an unquestionable improvement. A motion picture will be presented to describe the construction of a wall using this technique. Hence, it is sufficient to mention herein only the fundamentals of the method.

Along the perimeter of a proposed excavation a trench is dug in panels 12 to 15 ft long, 2 to 4 ft wide, and up to 180 ft deep. This trench is simultaneously filled with bentonite slurry to ensure stability. When a panel is excavated, the reinforcing cage is inserted and the wall is cast from the bottom up, the slurry being displaced by the rising concrete. A wall thus built may serve as temporary ground support, i.e., an earth retaining wall, or it may become part of the permanent structure. In the latter case special reinforcement details are incorporated in the wall to facilitate connections with other parts of the structure.

After the first applications of this method in urban sites in the 1960's it again was evident that the lateral bracing of the wall still remained a problem to be solved. The most effective solution became possible in 1964 with the introduction of a process utilizing prestressed tiebacks. This combination eliminated all obstructions within an excavation, and made earth moving a totally mechanized operation. Following these improvements, it became feasible to design larger and deeper underground structures. The best illustration of the use of the two methods, Berliner walling and slurry trench, is given by the construction of the Chatelet-Les-Halles station.

In this project two main regional metro lines, an East-West and North-South, will intersect and an interchange will be provided with four other lines of the metro network. Consequently, this station will become the focal point of the Paris transport system. Interchange will also be provided with the trains of the French Railways (SNCF) after a decision was made to support regional lines aligned with the international structure gauge by taking over certain SNCF services.

The incorporation of all these facilities required a very large structure to accommodate eight tracks, 270 ft wide with platforms 750 ft long, and front and rear station structures. This station will have a top concourse level for passenger traffic. The dimensions of this station would have virtually prevented its underground construction. In view of these considerations RATP decided to select the site of the former central market for this project, where it would be possible to accommodate open construction in conjunction with the renovation of the area.

Prior to the main operations which involved some 2 million cubic yards of excavation preliminary work had to be carried out. This was intended to provide a watertight enclosure to protect the excavation, and also to ensure the integrity and safety of numerous structures and buildings in close proximity. Soldier piles with lagging were installed in sections where enlargement of the excavation was temporarily restricted. On the other hand, the slurry trench method was used in sections with permanent excavation where the wall could be part of the permanent structure. In both cases prestressed tiebacks, 100 + to 200 + capacity, were used to brace the structure.

3. The Precast Diaphragm Wall

In the classical class-in-place slurry wall the excavation of the trench and the placement of reinforcement do not present serious problems. But the placement of concrete is a delicate operation. Any incident occurring after a pour has commenced may be the cause of trouble: breakdown of the batching plant or the delivery tracks, delays in transporting the ready mix concrete, etc.

These are partly the reasons that have led to the development of the precast diaphragm wall. This concept would make it possible to better satisfy the quality standards, comply with control and safety standards, and fully utilize the advantages of precasting, namely: dimensional accuracy and uniformity of panels, better strength and watertightness of the reinforced concrete, and simplification of the operations at the site.

According to this method, the trench is excavated using the same equipment as for the classical slurry wall. The panels, usually prefabricated at a nearby casting yard, are inserted in the trench, set in place and supported vertically until the special cement-bentonite mix has attained sufficient strength. Apparently, the quality of this mix is quite important; the mix must first insure the stability of the trench as does conventional bentonite slurry, and secondly it must set within a given time to provide bearing for the panels and seal them to the ground. Its final strength must be at least equal to the strength of surrounding soil. This development of strength as a time function is regulated by the addition of chemical agents to the cement-bentonite mix.

The watertightness of the wall is very satisfactory because precast concrete is more impervious than concrete cast under slurry and because the cement mix is eventually squeezed between the panels and the ground and provides a continuous screen.

A good example of precast walls is the extension of an existing line to Saint-Denis, north of Paris, built in 1974. This project required a tunnel 1800 ft long under the Rue de la Legion d'Honneur, which is a narrow street (39 ft between buildings), very busy and faced with old buildings and monuments. The tunnel is located close to the old Basilica of Saint-Denis, which has been shattered more than once over the centuries thus precluding any construction method involving ground movement or lowering of the water table. The ground consists of top fill, plastic marls and sand, and is of very poor quality.

After many studies RATP decided to disregard any method requiring pumping or infections. The structure was instead located as close to the surface as possible to keep the excavation to a minimum and also to impart minimum variation to the water level. The classical cut-and-cover method was not considered advantageous in this case because it would have required excessive pumping, and because of the size of the ground support. This arrangement would also disturb traffic for too long. The decision, therefore, was to provide a rectangular tunnel building first the side walls before any excavation, and then the roof slab to restore street traffic. The final scheme consisted of precast wall panels 15 in. wide set in a 23-in. wide trench.

This presented the following advantages: improved rate of construction, since the insertion of the cage and the casting of concrete were replaced by the insertion of the precast panels; equipment reduction at the site such as delivery tracks and the associated problems; minimum disturbance to surroundings because of a fast construction; and minimum effects upon existing buildings since the construction was carried out within a protected excavation and was independent of the ground water conditions.

The work involved the following stages: (a) relocation and divertion of the numerous underground utilities such as water, electricity, telephone, gas, etc.; (b) construction of two lateral sewers within six weeks to replace a central one; (c) closing of the street to traffic and installation of the precast walls in eight weeks; and (d) construction of the roof slab. The portion of the excavation above the roof slab was then backfilled, the street resurfaced, and the traffic restored. The construction was completed under cover safely and without incidents.

UNDERGROUND CONSTRUCTION METHODS

Quite frequently the geology of urban areas is dominated with soft ground. Thus, if no precautions are taken, the excavation required for underground construction creates a decompression within the soil and may lead to settlements detrimental and even dangerous to nearby buildings. Every project involving tunneling must be implemented in view of these considerations. It probably would be too long to review every technique used in Paris to prevent these effects, and this paper will rather discuss some of them briefly.

1. Improvement of Soil Characteristics by Grouting

Very often it is advantageous and economical to supplement the construction with ground strengthening. This can be obtained by grouting. Recent remarkable improvements in the technology of grouting have made it possible to give, for example, a strength of 300 psi to fine sands or to sands and gravels, and provide an excellent watertightness to any soil located below the water table.

A most impressive demonstration of grouting application for soft ground tunnels is the Auber station project located in the center of Paris. This station, which will handle as many as 50,000 passengers per hour in each direction, is about 750 ft long, 66 ft high and 135 ft wide. Both sides are located under existing buildings since the street is only 72 ft wide. For its entire length the station is below an existing metro line in service. Its invert is 120 ft below the surface and 60 ft below the water table.

In order to avoid any dangerous settlement during construction, the excavation was protected by a grouted arch 27ft thick. The grout type was varied in this case to accommodate variations in the geology, namely alluvium, sands and marls. The lower part under the arch was grouted with a 9-ft thick treatment to provide a watertight seal. During construction the observed settlement was always less than 1/3 in.

Of course, these results are exceptional and expensive, and RATP always tries to avoid grouting. Nonetheless, construction sometimes is not possible without grouting for ordinary tunnels as was the case for the Auber station. A further example of grouting is two tunnels under the banks of the Seine river for an under-river crossing built of precast reinforced boxes sunk directly under the river bed. The poor soil conditions necessitated grouting from the surface with cement, bentonite, low viscosity silicates and resins.

Another interesting application is to create in the ground itself an artificial retaining wall by injection of the soil on one side of the tunnel thus preventing decompression from propagating in this area. This has been done many times successfully.

2. Ground Recompression

As mentioned, sometimes despite all the precautions excavation causes decompression and leads to settlement. Since grouting is an expensive and sometimes long process, it must be confined to unusually difficult situations. This is mainly the reason that RATP, although it has largely contributed to the improvement of this technique, has always sought other procedures for restoring the initial state of stress in a soil through which a tunnel is bored. The objective is to prevent, or at least stop, the decompression of the ground by pushing the roof of the tunnel upwards as soon as possible. Such recompression (it might be more appropriate to say control of decompression) was applied to the Auber station immediately after digging the headings. The beams supporting the roof were forced upwards with the aid of hydraulic jacks until a pressure was developed comparable to the weight of soil above the tunnel.

The same technique is used to construct the upper part of a tunnel. Steel ribs are supported by radial hydraulic jacks that force them outwards against the ground. The steel or timber sheeting is supported at one end by the ribs and at the other by the concrete arch which has been already cast. The last method has, however, a serious drawback in that it restricts the access to the working face and thus does not allow the use of boring machines. This problem was recently solved with the utilization of heavy self supporting steel bent beams. These can be pushed upwards by hydraulic jacks and can also be expanded by the action of horizontal jacks placed on top.

RATP has also developed other methods specifically adapted to situations involving large-span structures or when tunneling under limited overburden and beneath existing buildings. These methods combine the temporary support with the final lining by using precast reinforced arch sections, articulated at the connection points and expanded against the ground by flat jacks placed inside the sections. The arch portion of the Auber station is a good example; after constructing the abutments, the arch was advanced in segment lengths of 2.5 ft. Each segment was then placed in position with the aid of a heavy steel rib. The segments were provided with a dip at the upper part, and contained a rubber bag covered with a concrete slab. The annular space between the arch and the ground was filled with shotcrete, whereas the keystone jacks were activated so as to push the arch against the ground. The articulation at the joints of the sections enabled the arch to expand freely and adapt itself to the heterogeneous soil conditions and to center the pressure line. Finally, the bags of the sections were injected and raised the overburden.

The same method has been used repeatedly, and the results are very satisfactory.

3. Special Methods

Two more methods will be briefly mentioned because of their particular interest.

Protection with tubes. Near Place de la Nation the east-west regional line is carried in two single separate tunnels in a rather close alignment. The tunnels are located beneath the foundation of sometimes old buildings along a relative shallow profile, and lie entirely within the so-called "Beauchamp sands" which constitute a specially dangerous ground since they are water bearing formations. These geologically and geotechnically unfavorable conditions required a method that could adapt itself to the danger of differential settlement and the associated disastrous consequences at the surface. Furthermore, this method would have to provide ground support and allow the lining to be set as close to the face as possible.

After consideration and comparison of possible solutions, RATP decided to concentrate on the possibility of creating prior to excavation a protective "Umbrella" consisting of contiguous steel tubes 100 ft long, set above the roof and pushed forward into the sand horizontally. The tubes were 7.5 in. in diameter, they had a thickness of 3/4 in., and were spaced at 20-in. intervals.

During excavation one end of the tubes remains anchored into the face, and a relief arch is thus created which supports the ground above. The excavation is made from access chambers normal to the tunnel so that the bundle of tubes placed from one chamber comes into contact with the bundle placed from the next one. The tubes are set into the ground using a special drilling machine in 4-ft sections screwed to one another, and the first section is provided with a lost drilling crown. Since any deviation in the direction of drilling will cause corresponding errors in the profile of the tunnel, special care must be exercised to control drilling and it has been possible to complete the work with a maximum error of 0.5%.

Under the "umbrella" the tunnel is processed in 3-ft sections, and the roof is immediately protected with gunite. The concrete lining follows at a maximum of 7 ft from the open face. This method is excellent, and has kept the actual settlement within safe limits.

Protection with self propelling shield. The extension of an existing metro line in the south section of Paris crossed a heavily traveled freeway in such a way that the roof of the tunnel was only 7 ft below the road bed. The problem was intensified by the very poor soil conditions, old quarry fill erratically distributed and without any cohesion. Since traffic maintenance was a prime factor, RATP had no choice but to build the roof of the tunnel under the protection of a special shield.

In principle the method consisted of using a very rigid shield made with 20 steel poling plates each 20 ft long, 20 in. wide and 4 in. thick, resting at one end on the last cast arch and on the other on three heavy steel self supporting ring beams. The poling plates were connected together by a guiding rail allowing the plates to be extended independently, and which provided also a way for steering the shield to match the profile of the tunnel. The poles were advanced by means of hydraulic rams resting on the heavy ring beams and in slots provided in the poles.

The interesting feature of this self propelling shield is that in soft ground the ends of the poling plates generate sufficiently into the soil ahead before the excavation to provide an efficient protection. The lining is installed directly under the rear part of the poles, whereas the annular space left behind by the advancing plates is immediately filled by injecting cement grout. When all the poles have been advanced from one position to the next, the ring beam is moved forward and placed under the advanced part of the poles at the face.

TRACK LAYING

The classical methods of installing tracks essentially involve rails placed on wood sleepers laying on ballast. On the Paris metro this has been done for a long time in the past, presumably because of the inherent advantages, i.e., the damping effect of the ballast on noise and vibrations, low cost and the simplicity of installation. Nonetheless, this method of track laying has serious drawbacks and requires an expensive maintenance. Indeed, the track bed is distorted due to the amount of energy dissipated as the trains are moving, and it becomes necessary to do a periodic leveling of the ballast at frequent periods.

This operation is very costly since it requires considerable labor and can only be carried out at night during the few hours when services are haulted. Experience shows that this work is most difficult because of the presence on the track of numerous devices and equipment necessary for and incidental to the operation, such as for example signalling, ATO, etc. On the other hand, vibrations from train movement are more objectionable now than have been in the past.

Thus, in recent years RATP carried out research to improve the methods of track installation in order to reduce maintenance cost and the level of vibrations. After visiting various countries and testing possible solutions, the following method has been adapted. Each sleeper is made of two concrete blocks connected by a spacer bar. The lower (bottom) portion of each block is wrapped with a rubber plate 1/2 in. thick. A rubber pad 0.2 in. thick is inserted between rail and concrete block. Tests carried out to compare the damping effect of the new and the old system showed that the conversion enabled a considerable reduction in the vibration level, and in fact the new installation transmits 100 times less vibratory energy. Nonetheless, this reduction is possible for a rail tread that is in perfect condition. On the other hand, the noise level is slightly higher for the concrete sleepers, which shows the damping effect of the ballast.

CONCLUSIONS

The city of Paris is not as fortunate as some other large cities to have a uniform geology, and this brief paper demonstrates that for each case and project engineers must study carefully the site conditions and select, or sometimes develop, the best suitable technique in order to insure the integrity of surroundings and avoid damage to nearby structures and buildings. This means that a preliminary evaluation of every potential method and its applicability to the conditions at hand is quite important, since misjudgment in the final choice may have a most critical effect and most serious consequences.

But this also means a greater responsibility for the engineers in charge and therefore a more challenging work.

PAPER 9

Dewatering and Grouting as Supplementary Ground Engineering Techniques

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PAPER 9

DEWATERING AND GROUTING AS SUPPLEMENTARY GROUND ENGINEERING TECHNIQUES

by

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INTRODUCTION

There are two basic approaches to the handling of ground water in the areas where deep excavations are contemplated: (a) by pumping with well points or deep wells and/or sumping systems, either before or after ground water enters an excavation; and (b) by intercepting the inflow with the use of impervious barriers and techniques such as freezing, vertical and horizontal grout curtains, slurry walls, compressed air and/or combinations therefrom.

In addition to the basic requirement of keeping ground water out of a proposed excavation, it is of utmost importance to control and minimize the effects on surroundings of the dewatering technique selected at the site. If the control of ground water is not handled properly, instability of the sides and the bottom of the excavation can occur and cause excessive lateral displacement and settlement of adjacent structures.

The following pages present a superficial view on ground water control and the techniques ordinarily used to deal with the associated problems. The examples that illustrate the discussion are taken from construction in urban areas where the consequences of side effects due to dewatering can be critical.

DEWATERING

1. Subsurface Conditions

The design of dewatering systems is often based on the assumption of uniform ground water flow and homogeneous subsurface conditions, which seldom represent the actual conditions at the site. Whereas basic soil characteristics such as grain size and permeability are essential, in-situ pumping experience is most valuable for the selection of a dewatering system. Whether the engineer or the contractor is responsible for the design, the following data and tests are required.

1. Soil exploration to determine the continuity of subsurface conditions and collect soil samples for laboratory testing.
2. Measurement of ground water levels and in-situ permeability by installing observation wells and piezometers.
3. In-situ pump tests monitoring water levels in piezometers and wells.
4. Laboratory analysis of soil samples, and especially grain size distribution for granular soils and consolidation characteristics for plastic soils.

The cost of a detailed exploratory and testing program usually is compensated by a corresponding reduction in the contingency items of the bid and by a reduced probability of damage to adjacent buildings and facilities.

2. Dewatering for Cut-and-Cover Excavations

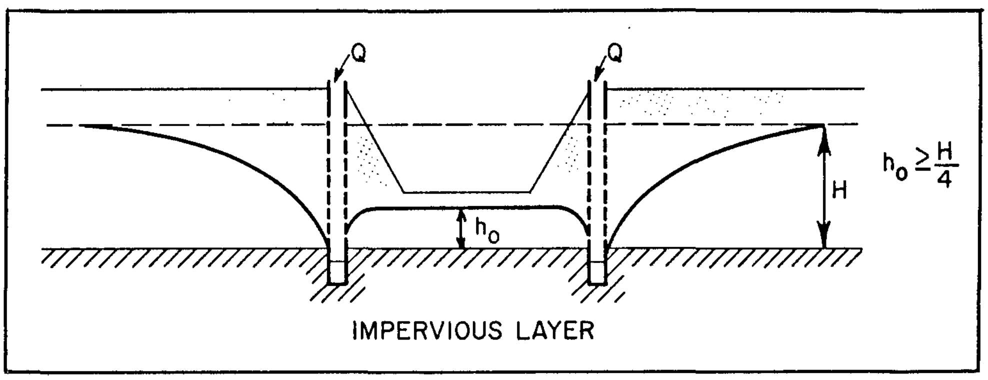
In selecting a suitable dewatering system for a particular set of circumstances, one must be aware of the consequences of pumping too much water, and especially damage to surroundings. A good dewatering system satisfies the basic goal of working in the dry with adequate safety against stability failure while removing only the quantity of water required. In practice subsurface conditions generally are complex, and thus a combination of dewatering techniques is most efficient. The following are examples:

1. To handle situations involving unusually heavy flows, large diameter deep wells at moderate spacing may be combined with sump pumping or well points.
2. Light flows can be handled with small diameter deep wells at close spacing with sumping as required.
3. Both these situations may be handled with a combination of deep wells with somewhat impervious shoring system and sump pumping.

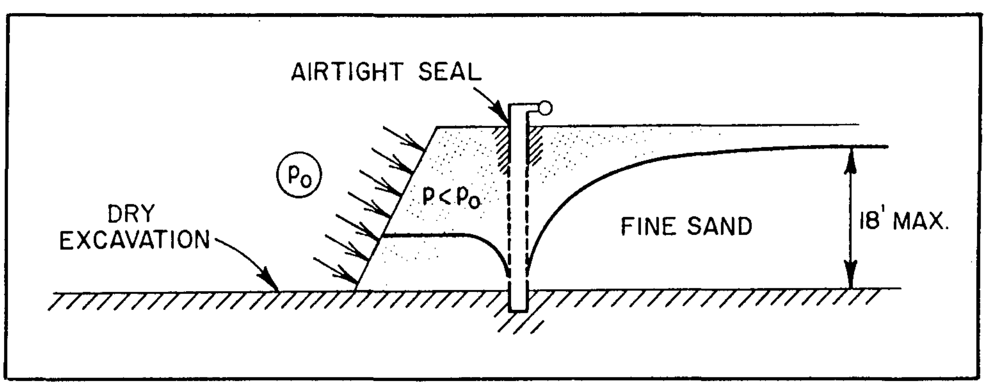
4. Completely impervious cutoff systems such as a slurry wall or a plastic cutoff wall can handle all ground water except trapped and surface water which can be sumped out. Deep wells may be required in this case if the cutoff wall is not keyed into an impervious layer.

A distinction must be made between flow in its quantitative aspect and pressure and exit gradient in relation to bottom stability and safety against boiling or piping. Flow into the excavation area can be tolerated where bottom stability is not a problem. The installation of a filter bed at the base of the excavation can be used to control loss of fines to the sump areas.

Each of the foregoing examples has its limitations. Thus, cast-in-place cutoff walls cannot perform positively unless they are keyed into an impervious layer beneath the bottom of excavation. Likewise, deep wells perform efficiently as long as they are drilled to an impervious formation, although it is likely that more than 75% of the total water head will be reduced thus requiring internal well points or sumping to perform the additional pumping if the bottom of the excavation must penetrate into the impervious layer.

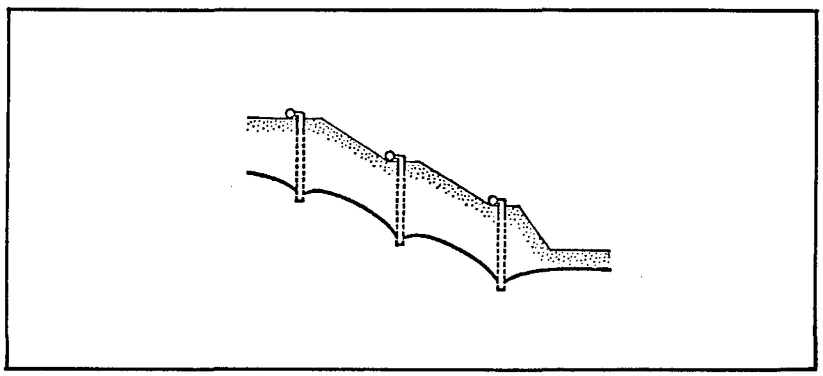


ESTIMATED MAXIMUM EFFICIENCY OF DEEP WELLS **FIGURE 1**

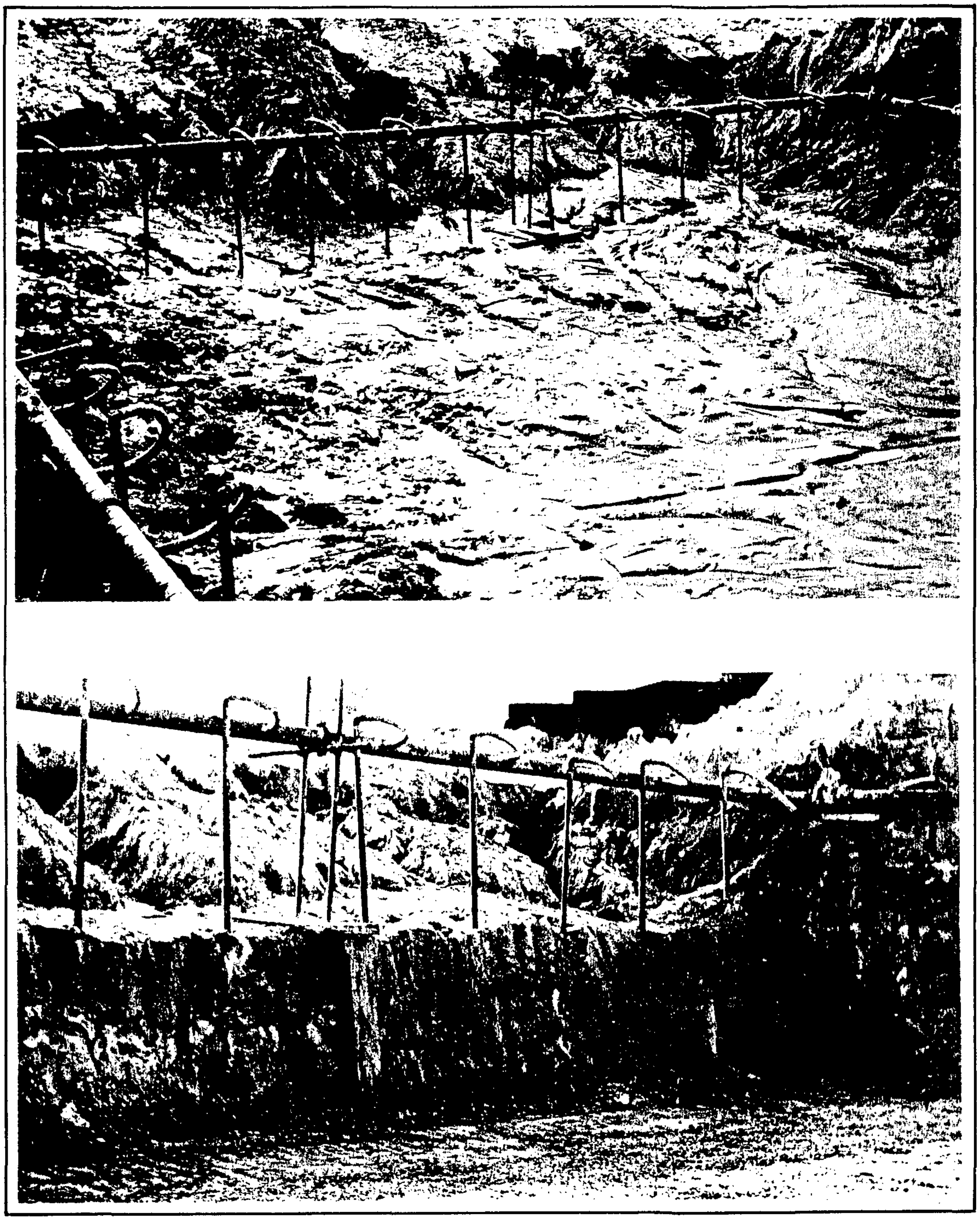


SHALLOW DEWATERING DOWN TO AN IMPERVIOUS LAYER EMPLOYING WELL POINTS **FIGURE 2**

The efficiency of deep wells and well points is shown in Fig. 1 and 2 respectively. Well points are efficient means of dewatering down to an impervious layer when the overlying soil is fine sand. In this case atmospheric pressure helps in maintaining the stability of the slope as shown in Fig. 2. It should be noted that well points are effective if the excavation depth from the initial water table does not exceed about 18 ft, and must be installed in steps as shown in Fig. 3 to dry deeper excavations thus requiring large working areas. For this reason well points are not practical for dewatering deep cut-and-cover excavations. Results of well point dewatering under ideal conditions are shown in Fig. 4.

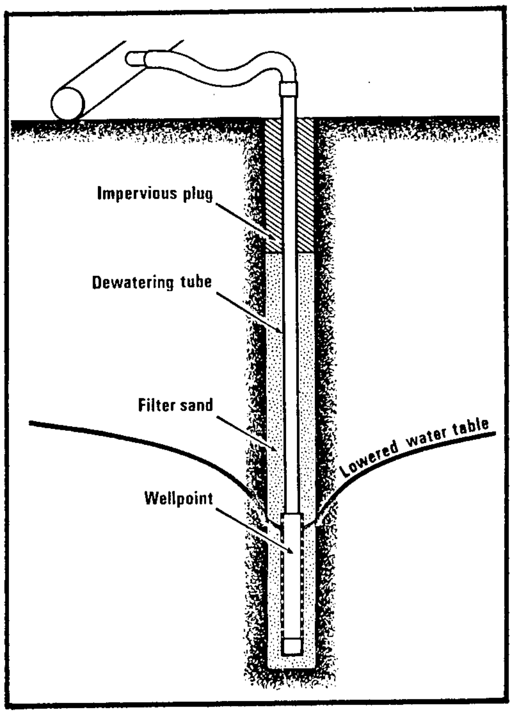


STAGED GROUND WATER LOWERING BY WELL POINTS **FIGURE 3**

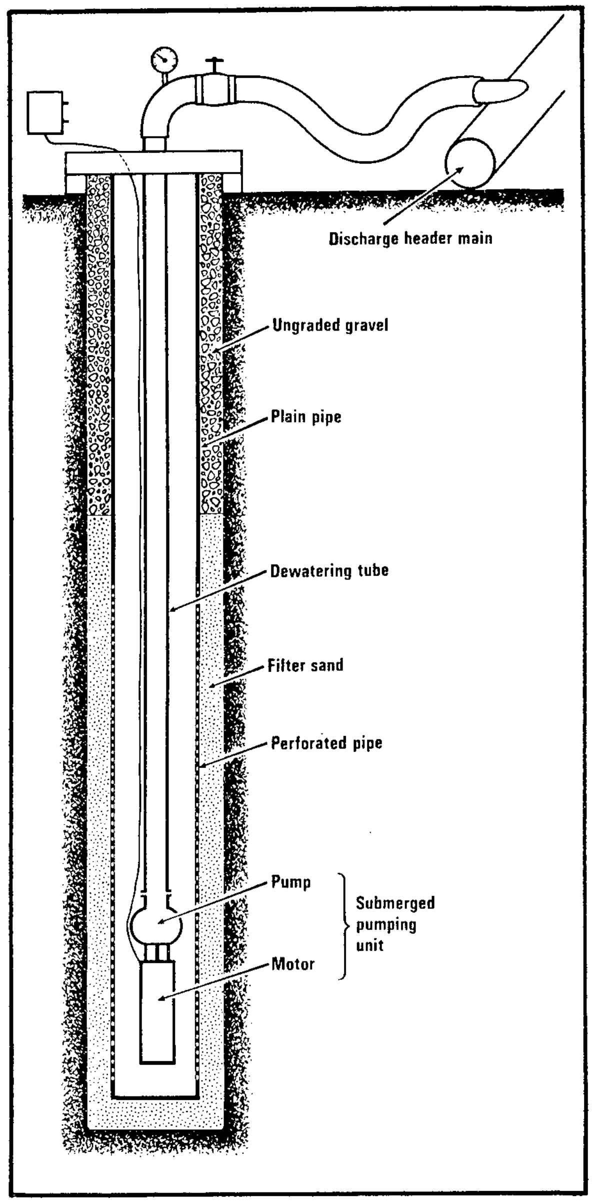


EFFECTIVE WELLPOINT DEWATERING **FIGURE 4**

A schematic design of deep wells and well point installations is shown in Figs. 5 and 6.



TYPICAL WELL POINT DESIGN **FIGURE 5**



TYPICAL DEEP WELL DESIGN **FIGURE 6**

3. Dewatering for Soft Ground Tunneling

Dewatering is in this case required when cohesionless saturated granular materials must be excavated. Construction by tunneling is preferred when alignment and depth prohibit trenching. Due to depth, water is generally found above the tunnel invert. Under the shield method, the dewatering problem at the face of the tunnel requires the same controls no matter whether the excavation is carried out manually or by mechanical means.

A long time practice has been to place deep wells along the alignment of the tunnel in order to lower the water table below the invert until the tunnel is lined and grouted. The deep wells are drilled and operated from the surface. The method causes desaturation of the soil and cannot prevent excessive overbreak in cohesionless formations.

Dewatering from inside the tunnel requires the installation of a drainage system around the base with intakes ahead of the shield and drilled from the rear where the lining is contact grouted. The result of this drainage is to reduce percolation with a subsequent reduction in water pressure ahead of the face of the tunnel. The system can be regulated in order to keep the soil moist and provide some cohesion. Some inconvenience is caused by the slower rate of advance because of interference between drain installation and lining, and also handling any water entering the tunnel through the drains. These drains act as relief wells in a surface excavation.

Alternatively, compressed air is used to solve the dewatering problem in tunnels, under various versions such as (a) English-type shields where the entire tunnel is under pressure; (b) mechanized shields where again the entire tunnel is pressurized; and (c) mechanized shields with compressed air only at the face in front of the machine and a gasket system between the skirt of the shield and the tunnel lining.

The last method (Robbins) allows a higher head of water since only a portion of the man power needs to work under compressed air so that the air pressure can vary between 1.2 atm to 2.8 atm. The volume of air required depends on the method used and the actual leaks occurring to the ground and around seals. If the compressed air technique is handled properly, overbreak can be minimized and no water has to be drawn from the ground. Where precast lining segments are installed by the boring machine, the quoted tunnel rate is satisfactory (42 lin ft/day, 30-ft diameter, Robbins RER Paris).

One major cost factor besides the initial mobilization cost is the requirement to work around the clock with 4 shifts instead of 3. This is compatible with work under compressed air and in line with labor regulations.

4. Associated Effects and Ground Settlement

Any change in the state of stress within a mass of soil due to excavation results in changes in the surrounding soil, and may accelerate normally occurring phenomena. Of these the most pertinent is ground settlement outside an excavation, for instance in the case of cut-and-cover construction or along the alignment of a tunnel.

There are two basic causes of settlement in cut-and-cover: (a) the ground support system may move horizontally inwards causing a corresponding vertical displacement of the adjoining earth including buildings and facilities; and (b) dewatering for the excavation may result in dewatering the entire area adjacent to the excavation. In the later case, the reduction in pore pressure leads to a corresponding change in density and grain arrangement in the previously saturated earth structure. The consequences of long term dewatering are evident in ongoing subway construction.

The extent of ground settlement or consolidation depends on the soil type, the physical layout of different strata, and other factors. Where relatively watertight ground support systems are used, a fairly undisturbed water table outside the excavation can be maintained merely by pumping inside the excavation and discharging the water pumped outside the excavation in recharge wells.

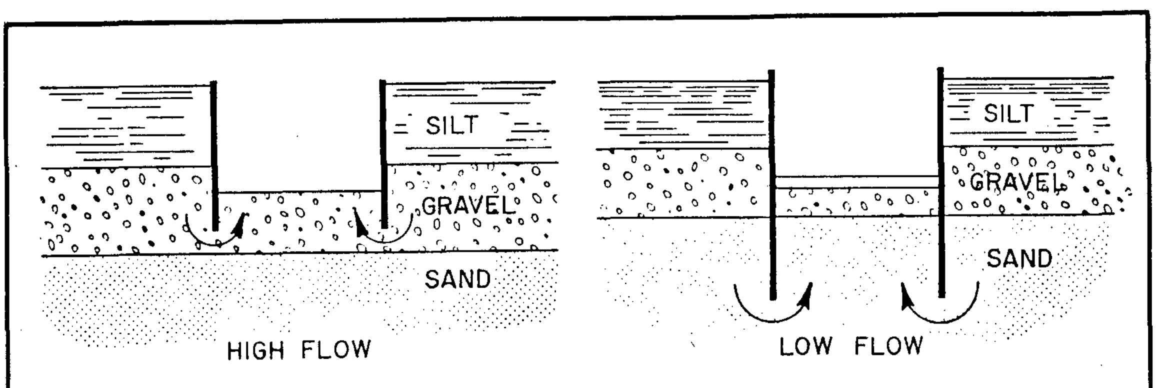
On the other hand, in tunneling there are three main sources of settlement: (a) decompression of natural soil around the bore propagating to the surface; (b) excessive loss of material in overbreak, which contributes to decompression and can only be avoided by immediate contact grounting behind the shield (the latter favors the precast concrete lining segments); and (c) in unusually great depths, dewatering with deep wells can result in critical surface settlement. An example of the last problem is construction along Avenida de los Cien Metros in Mexico City, which involved several feet of settlement; with the use of air shields in the downtown area of the same city, the actual settlement was less than one inch.

5. Design Considerations

Several theories have been proposed to relate the flow of ground water into wells. A suitable theory and a coefficient of permeability taken from in-situ measurements generally provide a satisfactory basis for designing a dewatering system. The results thus obtained should be checked as soon as the system is installed by pumping tests on portions of the installation; it is conceivable that some wells may be deleted or more wells may be required. In the former case the cost of unnecessary wells can still be saved; in the latter case time can be saved and possible damage to the support system and adjacent structures can be prevented.

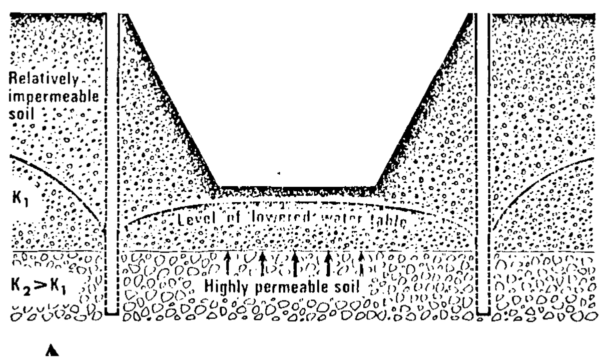
It should be emphasized that most of the available theories about ground water flow, seepage, etc. apply primarily to homogeneous soils and where the permeability is different in the horizontal and vertical direction. In-situ permeability tests generally are interpreted to indicate the horizontal permeability although this remains to be proven. This procedure usually is safe in the sense that soil deposits are always bedded to a certain degree, and horizontal permeability generally is higher than vertical. In some instances it might be too conservative. In any case, it is recommended to estimate the water flow on the basis of two-dimensional flow theories.

Relative permeabilities. In heterogeneous ground with various layers of different materials, it is important to understand the concept of relative permeabilities. For example, water flow will follow a different pattern depending on the permeability ratio between two adjacent layers. A ratio of the order of 10 is sufficient to bring about this change. Thus, with respect to flow analysis a sand layer can be considered impervious in comparison with an overlying gravel layer. The relative permeability phenomenon is schematically depicted in Fig. 7.



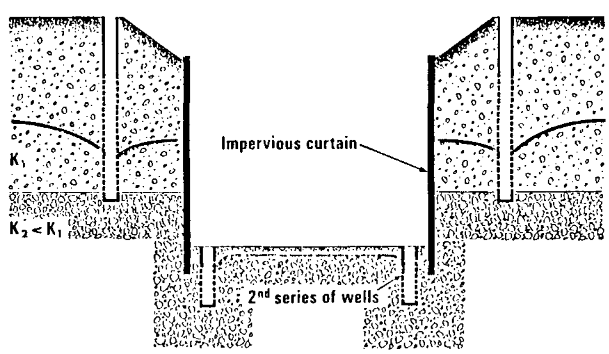
RELATIVE PERMEABILITY PHENOMENON AS IT AFFECTS EMBEDMENT **FIGURE 7**

Fig. 8 and 9 show how to handle ground of relative permeabilities. Where highly pervious soils are overlaid by relatively impervious soils, a deep well system will be effective. However, when the excavation is within less pervious soil a combination of staged dewatering and an impervious curtain may be required as shown in Fig. 9.



DEWATERING UNDER TWO LAYERS OF DIFFERENT PERMEABILITIES
$K_2 < K_1$

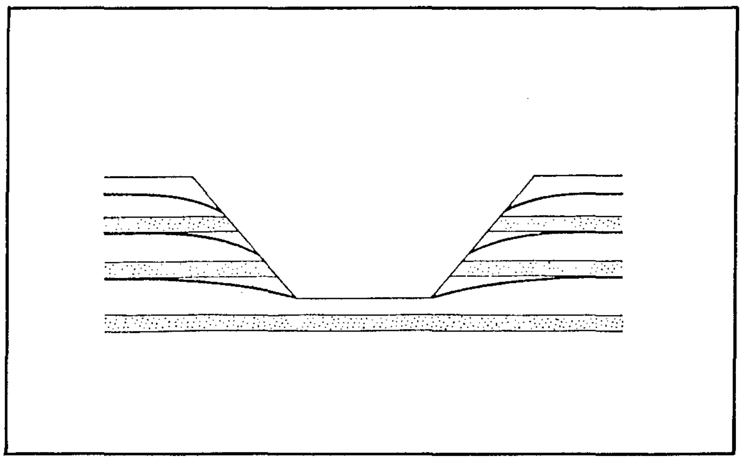
ONE STAGE DEWATERING **FIGURE 8**



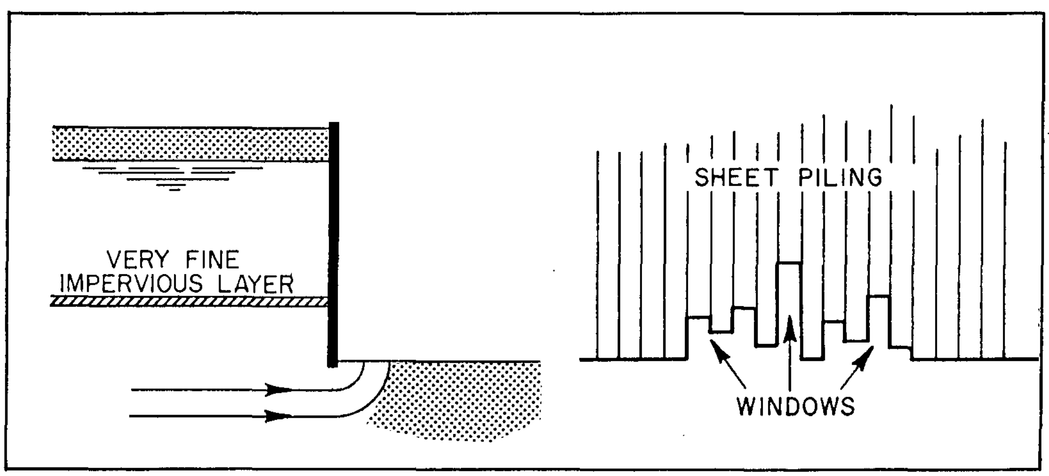
DEWATERING UNDER TWO LAYERS OF DIFFERENT PERMEABILITIES
$K_2 < K_1$

TWO STAGE DEWATERING **FIGURE 9**

Multilevel aquifers require multilevel dewatering (Fig. 10). It is generally not effective to attempt to place the various aquifers into communication. On the other hand, heterogeneous subsurface conditions often present both favorable and unfavorable situations such as those shown in Fig. 11. The engineer must then be prepared to utilize the unexpected good effects and effectively deal with the bad ones.



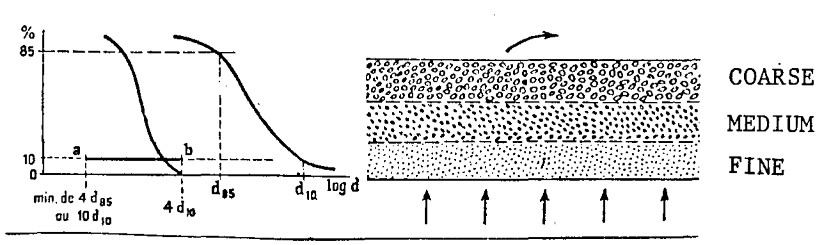
PERCHED AQUIFERS **FIGURE 10**



FAVORABLE UNFAVORABLE **FIGURE 11**

Percolation. This is a force independent of ground water flow, and is only related to the hydraulic gradient. It is this force that causes internal erosion and instability that leads to boiling.

In order to modify the limit conditions of ground water flow that might be critical, wells are sometimes installed at the base of an excavation to provide relief against water pressure. It is not necessary to pump from these wells, and it is only for reasons of economy and safety that pumps are used. For example, in order to avoid piping under an unevenly driven sheet pile curtain because of exceptionally high density, relief wells can be installed with pumps to relieve the percolation forces horizontally instead of vertically.



FILTER DESIGN TO CONTROL PIPING **FIGURE 12**

Another way to counteract the percolation forces and the washing out of fine soil is to install a surface filter bed at the bottom of the excavation (Fig. 12). Based on empirical data, the filter material should have a grain size ten times the size of particles to be retained as given in the grain size curves of Fig. 12.

6. Cost of Dewatering

The costs of dewatering are generally broken down into two categories:

Fixed Costs

Installation
- Drilling
- Installation of filters and well development
- Connections for discharge
- Power setup
- Piezometric control system installation

Costs Related to Time

Maintenance
- Labor
- Parts
- Repair

Energy
- Electricity
- Fuel
- Air (air eductors)

It is impossible to discuss actual costs since situations vary enormously. For example, deep wells can cost between $1,500 and $15,000 each depending on the characteristics of the layers to be dewatered.

7. Conclusion about Dewatering

Compared with other methods, dewatering an excavation often is economical considering the total cost of the project. However, dewatering costs do not usually reflect reconstruction costs for curbs, paving, damage to utility lines, mud jacking or underpinning of buildings on spread footings. These costs usually are included in insurance coverage or loss to the owner.

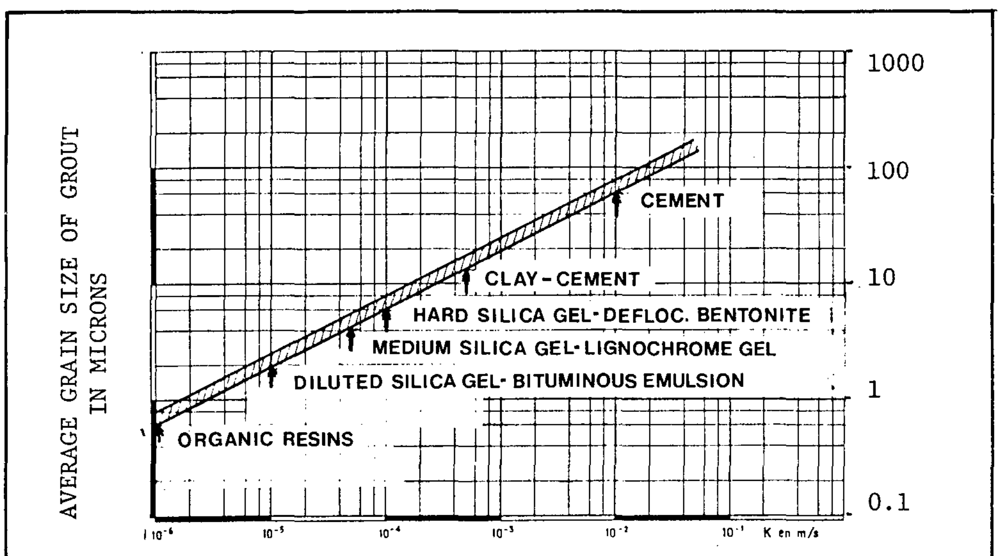
In order to prevent or control the associated effects of dewatering, better approaches are necessary than the present extreme methods. Thus, instead of a radical decision to lower the water table of the entire area by use of intense pumping in deep wells, the water flow may be better controlled by refining the drainage methods and the filter and relief well techniques.

GROUTING

There are two basic grouting applications in soils; the first is to reduce permeability, and the second is to improve the strength of the soil. In granular soils both improvements can be accomplished by impregnating the voids between grains with suitable materials. Plastic soils usually are impervious so that grouting is used to increase the load carrying capacity. This is accomplished by consolidation grouting whereby cement grout is injected into the earth structure to form a skeleton. The soil is consolidated by the hydraulic effect of grouting. This technique has been developed empirically and received extensive use abroad, although it is generally unknown in the United States.

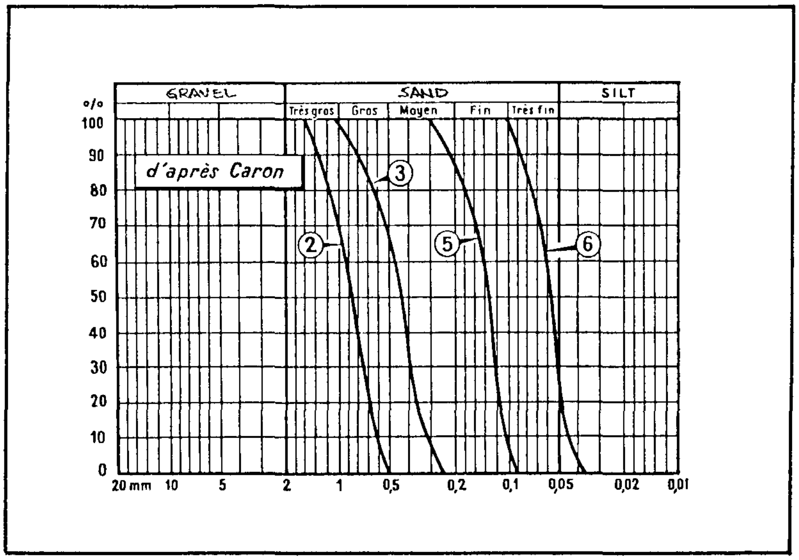
1. Grouting Granular Material

For this application it is quite important to know the in-situ permeability and grain size distribution of the strata to be treated. The selection of a suitable grout material is made on the basis of the pore size to be penetrated, hence the viscosity of the grout varies according to the characteristics of the voids. The diagrams and tables shown in Fig. 13-15 may be used a a rough guide to determine the range of products that can be used to accommodate the most common granular soil types. It is possible to render almost all granular soils impermeable, and by building vertical and/or horizontal grout curtains to prevent water seepage.



LIMITS OF GROUT PENETRATION ACCORDING TO PERMEABILITY ACCORDING TO CAMBEFORT

FIGURE 13



LIMITS OF GROUT PENETRATION ACCORDING TO GRAIN SIZE **FIGURE 14**

GROUT SELECTION

Up to curve (2) use bentonite cement grout
From curves (2) to (3) use clay grout
From curves (3) to (5) use silica gels
From curves (5) to (6) use bituminous emulsion

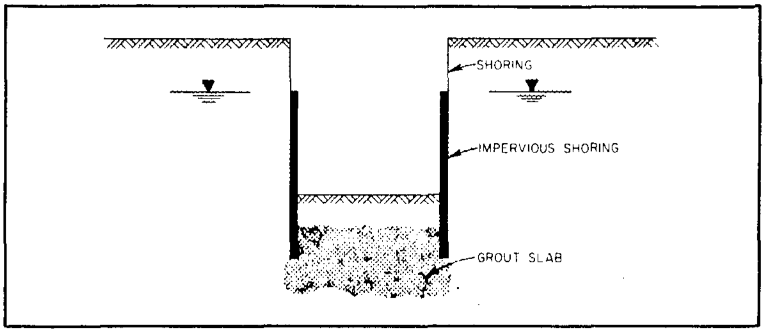
CHARACTERISTICS AND POSSIBILITIES OF MAJOR GROUTS

Type of Grout			Unconfined Strength*	Material Cost per cm^3 (Ratio)*	Field of Application*	Grouting Procedure
Suspensions	Unstable Grouts	Suspension of cement in water (+ sand event.) $\frac{w}{c}$ = 10 to 1	Similar to concrete	4.2	Fissures or cracks in rock or masonry	No limited quantities. Pumping until refusal.
	Stable Grouts (Segregation of a small %)	Cements and activated mortars: Prepakt, Thermocol, Colcreete	Similar to concrete		Filling of large voids	Limited quantities
		Cement + clay (+ sand)	1 to 50 kg/cm^2 1 to 700 psi	1	Wide rock fissures and sands and gravels with k>2.5, 10^{-6} ft/m	
		Special clays	<.0125 psi_2 <1 g/cm	1.1		
Liquid Grouts (Chemicals)	Hard Gels	Sodium Silicate +$CaCl_2$	10-20 kg/cm^2 150-300psi	10.7	k>10^{-6} ft/m	Grouting in 2 phases
		Sodium Silicate +Ethylene Acetate	mortar up to 600 psi (40 kg/cm^2)	11		Grout of one type
		Lignosulforate + bichromate k	5 psi 300 g/cm^2 (mortar 50 to 70 psi)	6.5 to 8	k>10^{-7} ft/m	
	Plastic Gels	Sodium Silicate + reagent	7 psi 50 kg/cm^2	2 to 4	k>5 10^{-7} ft/m	Limited Quantities
		Deflocculated bentonite	1.5 to 3 psi	1.8	k>5 10^{-6} ft/m	
	Organic Resins	AM9	<14 psi or 1 kg/cm^2	50 to 130	k>.5 10^{-8}	
		Resorcinol formaldehyde	15 to 1400 psi	10 to 40		
		Urea formaldehyde (acid grout)	300 to 1400 psi			
		Precondensed polymers (Epoxy)	up to 14,000 psi comp. up to 4300psi traction	150 to 500	Concrete cracks	
	Hydro-Carbon Based Grouts	Bituminous emulsion with silicate or resorcinol	1.5 psi mortar 150 psi	6 12	k>10^{-7} ft/m	
		Hot bitumen	very viscous fluid		abundant water circulation	

* Strength given for pure grout (for very low values taken as twice the rigidity)
* Base 1 for material cost comparison - 300 kg of clay and 200 kg of cement per m^3 (520 lbs. clay, 350 lbs cement per cy)
* Permeabilities shown correspond to granular soil susceptible to impregnation. Excessively fluid viscous grouts should not be introduced into too pervious soils. (inefficient technically and waste of money.)

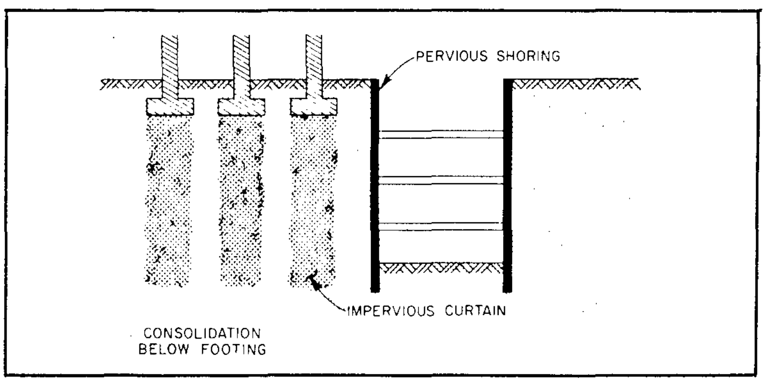
FIGURE 15

Grouting has received extensive application in eliminating seepage through the bottom of excavation, used to provide an impervious horizontal barrier beneath the base where the latteral ground support is fairly watertight as shown in Fig. 16. Arching effects occur in this case in a slab thick enough to overcome uplift pressures, although theoretically dead load weight is not sufficient to insure stability of the bottom of the excavation. Since grouting is carried out before reaching the water bearing layers, all excavation can be done in the dry provided trapped and surface water is pumped out.



GROUT SLAB TO PREVENT UPWARD SEEPAGE AND CONTROL UPLIFT **FIGURE 16**

With respect to vertical impervious curtains, if a slurry cutoff wall can be built the cost will generally be lower. Where the situation at hand involves dewatering as well as underpinning problems, an impermeable consolidated gravity earth wall may be feasible. In view of the wide range of strengths that can be attained using suitable grouting methods and products, considerable increase in ground support can be imparted to superficial foundations. Taking into account the flexibility of access to the mass of soil to be consolidated, grouting can be competitive with conventional underpinning methods. The combined effects of consolidation and watertightness might be desirable for reducing water flow as well as changing earth pressure loading conditions on the lateral support system. (Fig. 17).



UNDERPINNING BY GROUTING **FIGURE 17**

2. Grouting Nongranular Methods

Plastic soils such as clays and cohesive silts are basically impervious, although fissures tend to develop under certain states of consolidation or desiccation. These soils are nongroutable in the sense of impregnation, yet substantial improvement in their characteristics is possible by the use of coarse grouts that first penetrate the fissures or weak planes and create a skeleton of cement-base grout by successive grouting phases. This is a form of compaction grouting, but can provide a homogeneous result using the sleeve pipe grouting method which allows repeated grouting without additional drilling. Clay can thus be consolidated increasing its bearing capacity up to ten times the initial value.

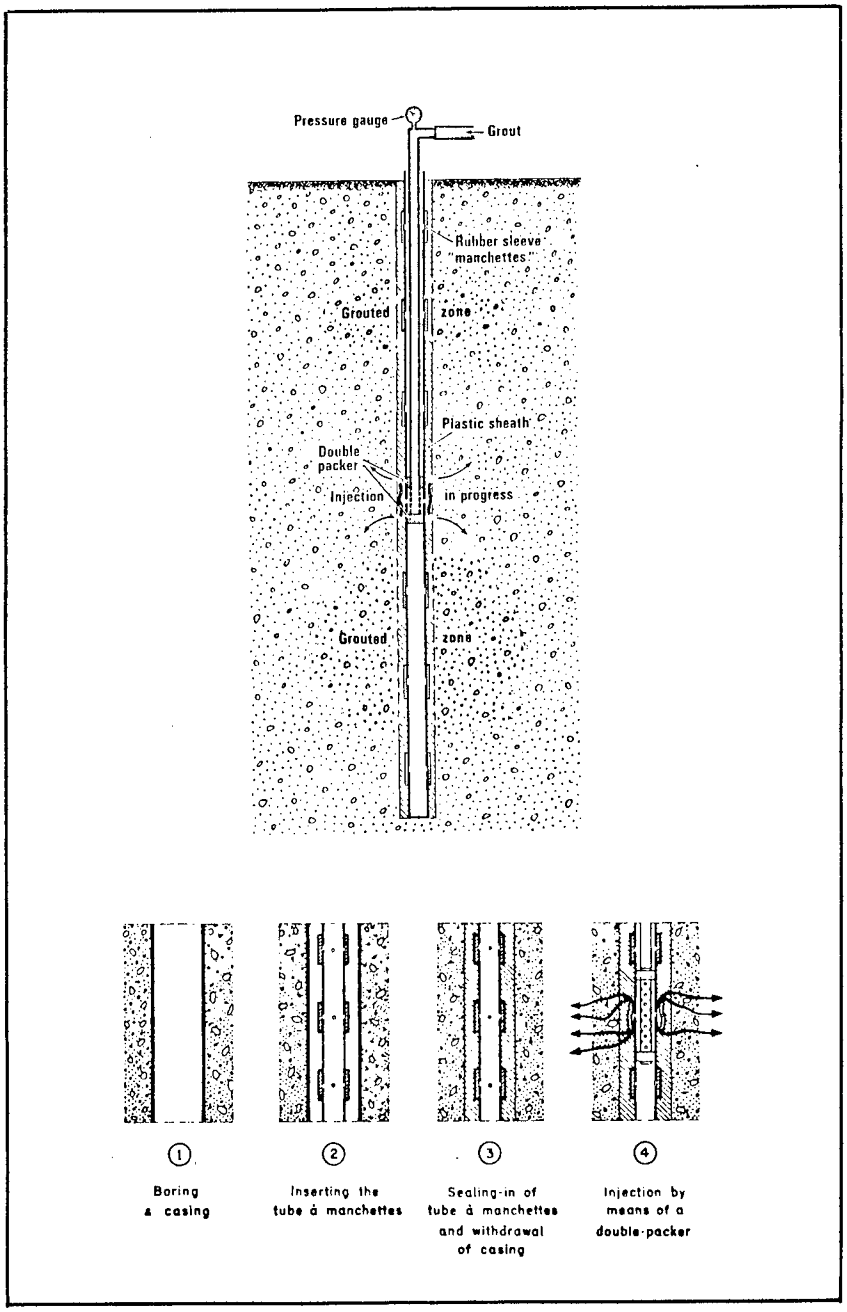
3. Sleeve Pipe Technique

Soil grouting techniques have received extensive application in underground construction in Europe due to detailed study and product development of grouts and the many advantages made possible with the sleeve pipe technique, such as efficient control of the distribution of grout by measurements taken at one-foot intervals of the amount of grout pumped per phase and initial and final pressure reached at each phase. In addition, sleeve pipes can be reused as many times as necessary. Fig. 18 illustrates the use of sleeve pipes for grouting.

4. Applications

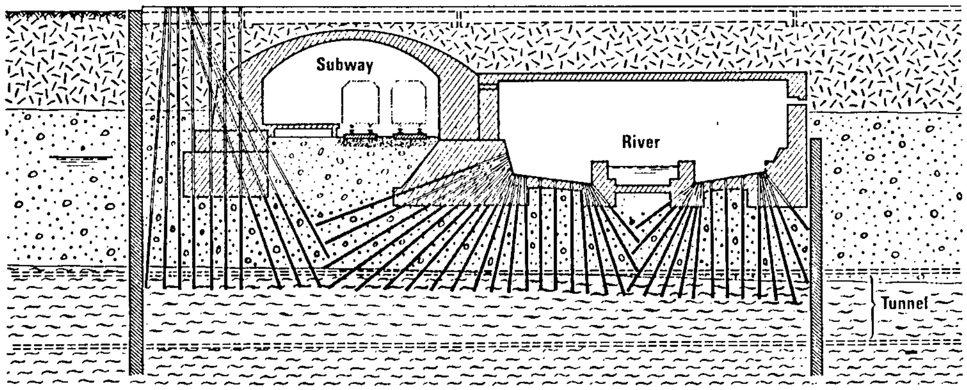
Figs. 19-22 show some of the actual projects where grouting was used under various conditions and for various applications in underground construction.

In particular, grouting provides an excellent tool in tunnel construction. Considering the fact that a shield can accommodate only a circular configuration, grouted soils can be excavated conventionally in a variety of sections. Grouting applications to tunneling generally provide both watertightness and consolidation which results in isolating the bore from outside conditions and minimizing risks of ground surface settlement.



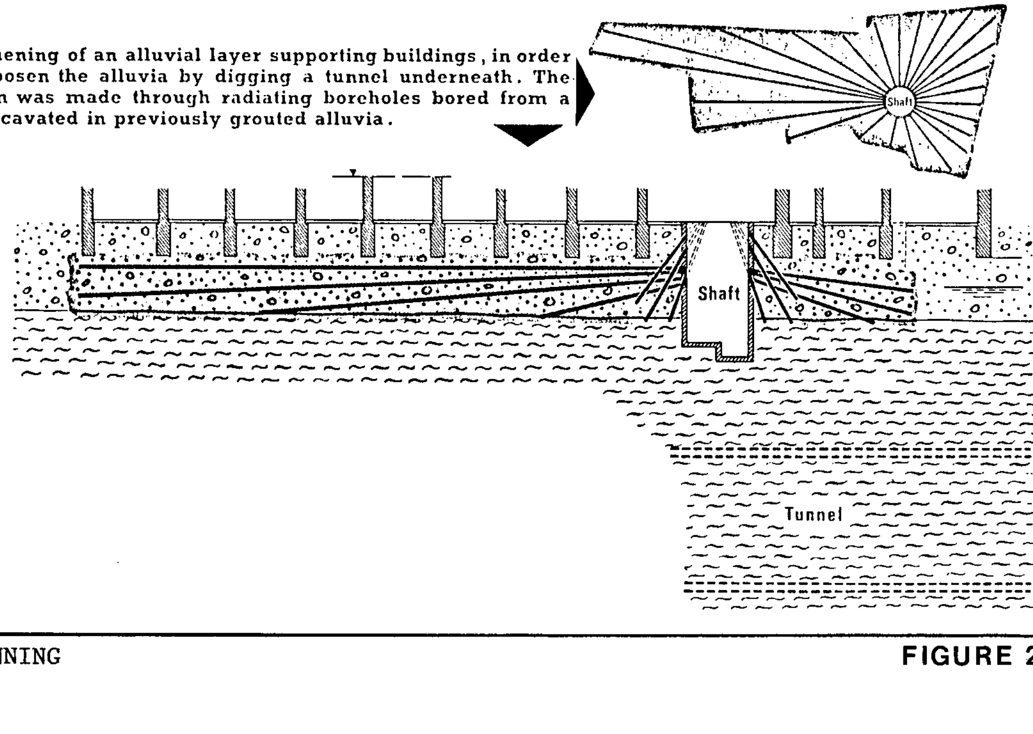
PRINCIPLES OF SLEEVE PIPE GROUTING

FIGURE 18

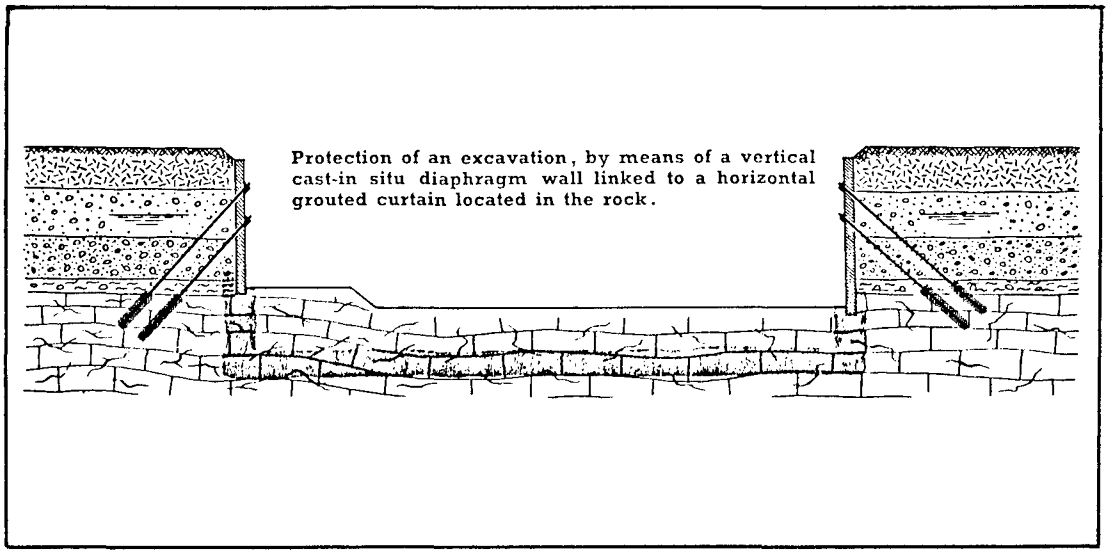


SEALING AND UNDERPINNING **FIGURE 19**

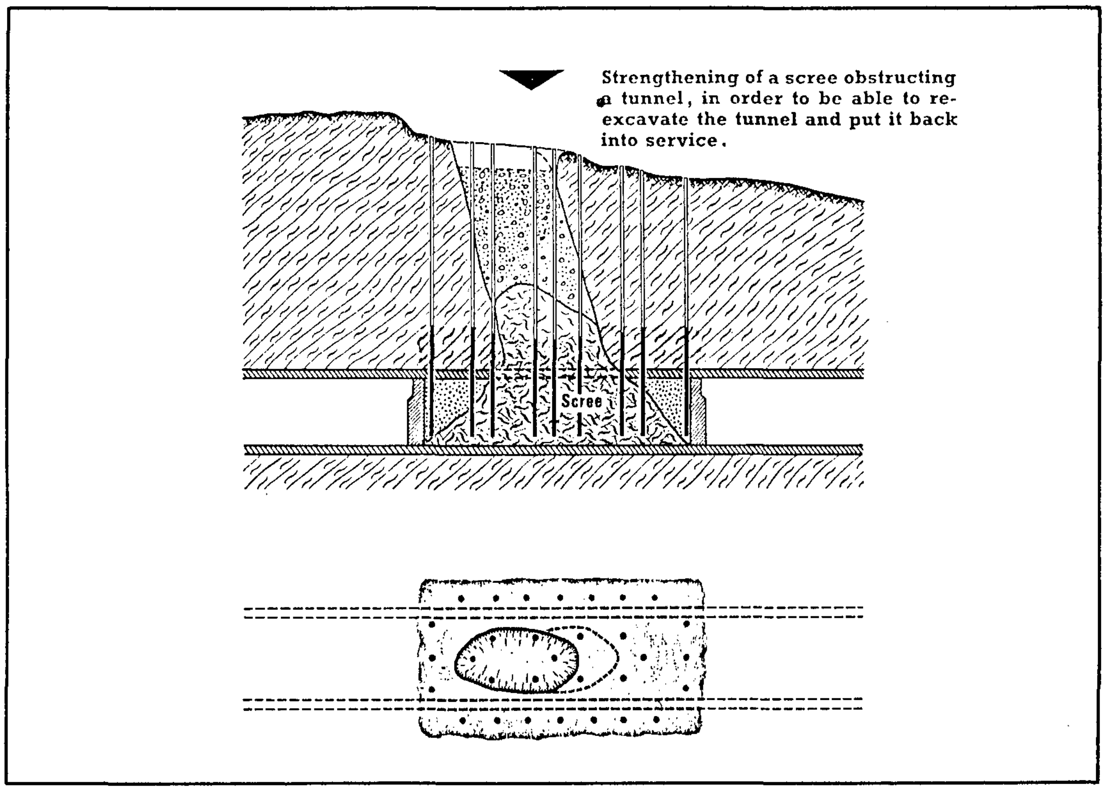
Strengthening of an alluvial layer supporting buildings, in order not to loosen the alluvia by digging a tunnel underneath. The injection was made through radiating boreholes bored from a shaft excavated in previously grouted alluvia.



UNDERPINNING **FIGURE 20**

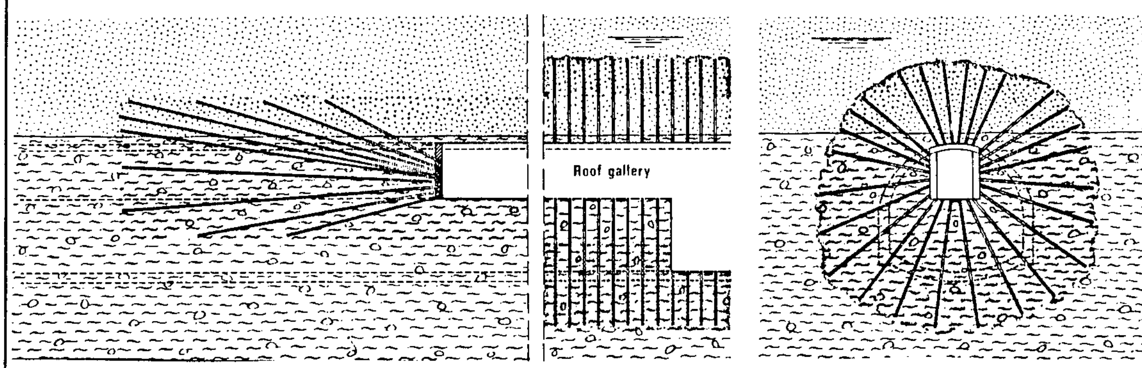


GROUND WATER CONTROL **FIGURE 21**



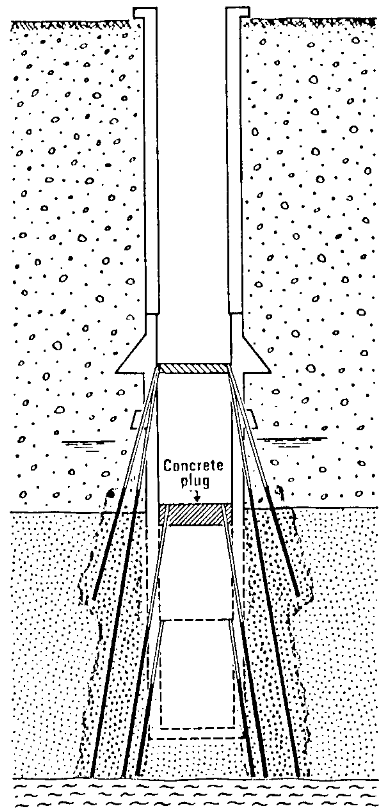
EMERGENCY STABILIZATION OF DEBRIS IN TUNNEL COLLAPSE **FIGURE 22**

It is quite essential to correlate grouting technology with the excavation scheme so that a safe and optimum rate of progress can be achieved. Depending on the conditions at hand, grouting can be carried out from the ground level or from underground galleries not necessarily with the excavation area. Figs. 23-26 show grouting applications in shaft and tunnel construction.



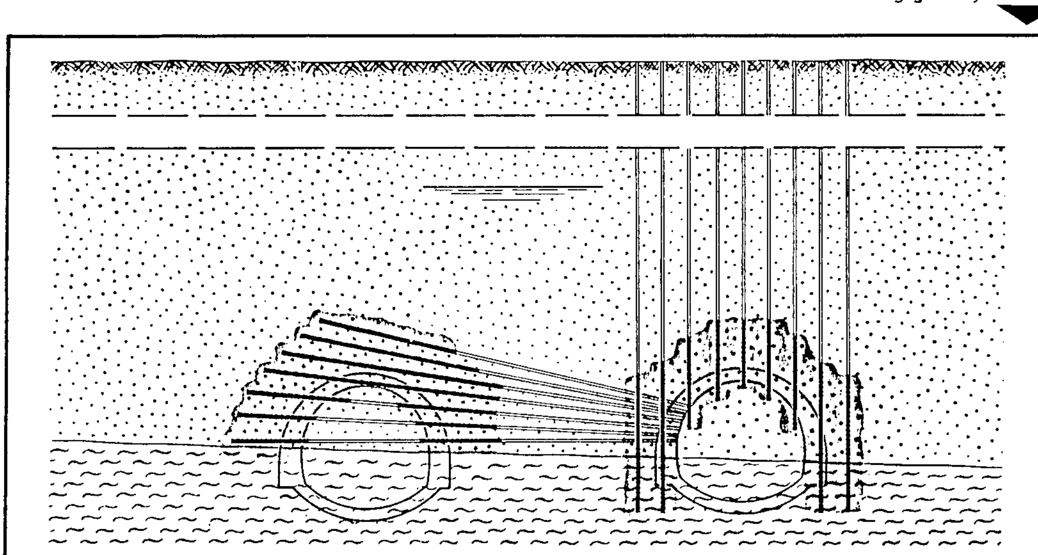
Excavation of a working gallery under shelter of progressive injections which are carried out from groutholes, bored in a fasces pattern from the working face. From this gallery, the ground treatment is completed through radiating boreholes, to make it possible to excavate the whole section of the tunnel.

TREATMENT FROM PILOT TUNNEL IN CONVENTIONAL EXCAVATION **FIGURE 23**



CONSTRUCTION SHAFT **FIGURE 24**

Preliminary strengthening of the ground surrounding the vaults of 2 tunnels, located at great depth in quicksand under a high water head.
To avoid the deep boreholes required for the treatment of the first tunnel, the treatment of the second tunnel was executed from the first tunnel which was used as a working gallery.

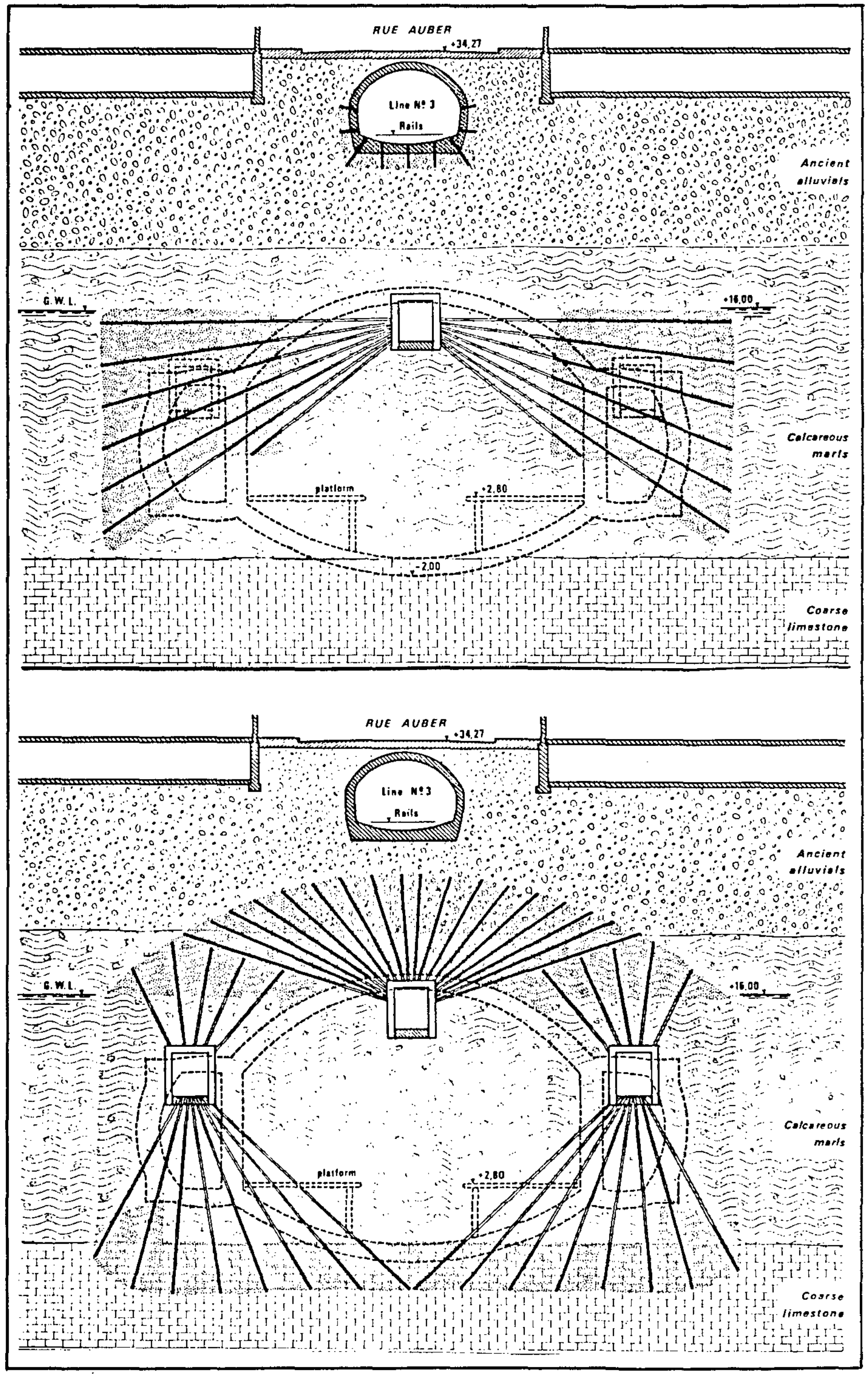


GROUT TREATMENT FOR DOUBLE BORE TUNNEL **FIGURE 25**

The central heading at crown level which was partially above the water level, was excavated by conventional methods.
The injections made from this heading reached the zone of the side walls of the future station. The object of this treatment was:

1—to seal the ground surrounding the lateral working galleries so as to permit excavation in the dry;
2—to seal the side walls of the final structure.

In a second phase the soils surrounding the arch of the station were consolidated over 7 to 8 m by means of radiating boreholes drilled upwards through the roofs of the three working galleries.
Simultaneously, injections made through the boreholes driven downwards from the lateral galleries enabled the sealing of the lower part of the side walls and the raft of the station to be carried out.



GROUT TREATMENT FOR LARGE SINGLE BORE TUNNELS **FIGURE 26**

5. Cost Analysis

It generally is very difficult to provide an estimate of cost in areas where grouting is a mandatory ground engineering technique, hence it is even more difficult to obtain an estimate for grouting applications in new areas. Although considerable technological progress has been made in developing materials for grouting, their cost continues to be a serious factor in the total cost of the application. Every attempt should be made to use the least expensive products available, and nature often helps by confronting the engineer with actual conditions that make certain grout materials suitable for field use although they are not found suitable in laboratory tests.

The cost of a grouted cubic yard of soil can vary from $15 for a cement bentonite grout to as high as $150 for sodium silicate gels which are among the least expensive chemical grouts. Accordingly, the ability to modify grout materials in the field to accommodate variable conditions can often save considerable money.

CONCLUSIONS

The necessity of underground construction in congested urban areas has resulted in the development of new construction techniques to implement sizable projects with minimum inconvenience to the public and minimum effects on private property. This, together with the need for deeper excavations, explains why the idea of coping with the ground water problem through pumping is constantly losing ground and is replaced by more sophisticated techniques which have little, if any, dependence upon the water table and therefore do not affect the ground and structures around the excavation area. Although seemingly more expensive, these techniques prevent side effects associated with conventional construction methods, and also allow a better control of the excavation progress.

Underground construction is still an art that must remain sufficiently flexible to adapt itself to special and changing conditions. Thus imagination is often required to pursue elegant solutions, and experience is a necessary prequalification for those in charge of the work.

Among the most promising new underground construction techniques bound to have extensive application in the United States first ranks the cast-in-situ diaphragm wall (slurry wall), because of the immediate possible uses. Grouting techniques, on the other hand, will remain unfamiliar to most engineers for some time, and exceptional applications will not occur until a more systematic approach is taken to make the technique standard construction practice.

However, I believe that grouting will receive immediate applications to remedy some unique problems caused by the concentration of utilities at crossings where open cuts are a nightmare. Tunneling in a grouted area between two open cuts will become more common in the near future.

Without changing appreciably current contracting practices, substantial economy could be possible by furnishing more detailed subsoil information than is presently customary so that the feasibility of grouting could be studied at the bidding stage. This could involve, for example

> water level readings, accurate and brought up to date,
> grain size curves for the different soil strata encountered,
> accurate permeability tests, and
> results of pump tests for inclusion in all geotechnical reports for projects where dewatering is a problem.

Contingencies which are part of all underground construction estimates could be sharply reduced when projects are designed if maximum geotechnical information is gathered and presented in contract documents. This procedure is more satisfactory than expecting potential contractors to undertake their own exploratory work prior to bidding.

The flexibility and applications of grouting techniques can be enormous, but inefficiency in grouting design can result in unjustifiably high costs.

PAPER 10

Review of Tunneling Technology

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PAPER 10

REVIEW OF TUNNELING TECHNOLOGY

by

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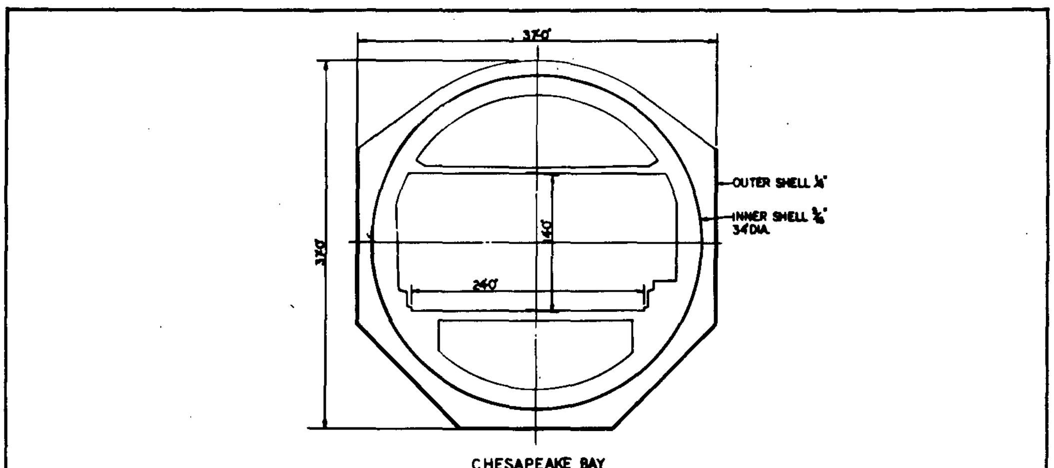
SUNKEN TUBES

The sunken tube is purely an American invention. Probably as early as 1906 a distinguished group of engineers gathered together to work out a method for building a railroad tunnel under the Detroit River. This group put forward the concept of the sunken tube, and decided that it could be feasible to sink and align with sufficient accuracy a preassembled tunnel tube into a preexcavated trench in the river bed. The steel tubes for this project were built in a nearby shipyard, towed to the site and sunk into a specially prepared bed by admitting water into the tube. The first application had its troubles; water was liable to run to one end or the other, and this prevented the tube from being sunk level. Finally the steel tube section reached the bottom where they were firmly set and fastened together. This tunnel was opened in 1910 and is still in service.

One of the early sunken tubes was built and sunk here in Chicago for the Chicago subway directly underneath the State Street Bridge. This is a double-barrel tunnel, 200 ft long. The tube was built at a shipyard in South Chicago.



Cross Section of Sunken Tube for the Chesapeake Tunnel- **FIGURE 1**

Fig. 1 shows a cross-section of the Chesapeake Bay Tunnel. This consists of two twin tubes, each about one mile long. Each tube accommodates a 24-ft roadway with 14-ft clearance. The tubes were sunk in 300-ft long sections. The trenches in the bay were excavated by floating dredges while crushed stone was carefully screeded into place by a drag screed working from tracks on two barges. The tube sections were sunk into this bed and joined together by divers pulling them together using hydraulic jacks. Current practice is to drain water from between the two end bulkheads, so that the entire hydrostatic pressure joins the two ends together. These were later welded for watertightness after the trench was filled over the tubes and made level with the river bottom.

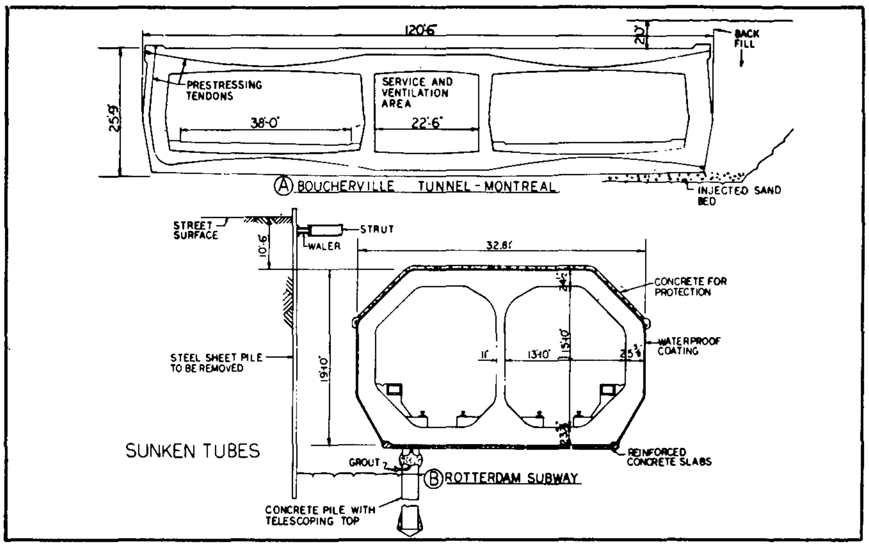


FIGURE 2

Fig. 2 shows two more cross sections for sunken tubes. Part (a) is for the Boucherville Tunnel in Montreal, and part (b) shows the Rotterdam Tunnel built in Holland during the German Occupation. These show the Canadian and European approach to tunneling which is combined with the concept of precasting. The tunnel tubes are made of precast pretensioned concrete sections up to 260 ft long in specially built cofferdams near the permanent site. The cofferdams are subsequently flooded, and the sections are taken out one at a time and towed to the sand bed where they are set. If the preparation of the bed and the setting phase are not completely successful, some settlement may follow resulting in the cracking of the concrete sections and a subsequent leakage. For the Boucherville Tunnel, settlement has made the structure liable to continuous leakage which is corrected by grouting from within the tunnel year after year.

Sunken tubes had a wide application on the BART project for the portion of the tunnel in the San Francisco bay. This section was 3.6 miles long and too deep for conventional mining with a shield or compressed air; indeed the pressures would have been too high for men to work. The 63rd Street Tunnel in New York under the East River consists of four tracks; two for the subway and two for the L.I.R.R. The initial design provided two tracks and proposed to drive the tunnel with a shield, but when the design changed to accommodate four tracks it was found more economical to use sunken tubes.

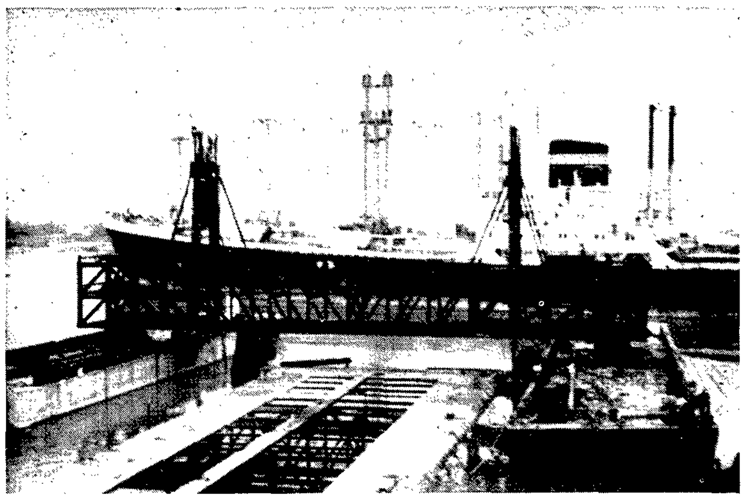


FIGURE 3

Sunken Tubes for the Mobile River Tunnel in Alabama.
View of the tunnel sections and the barge-mounted truss.

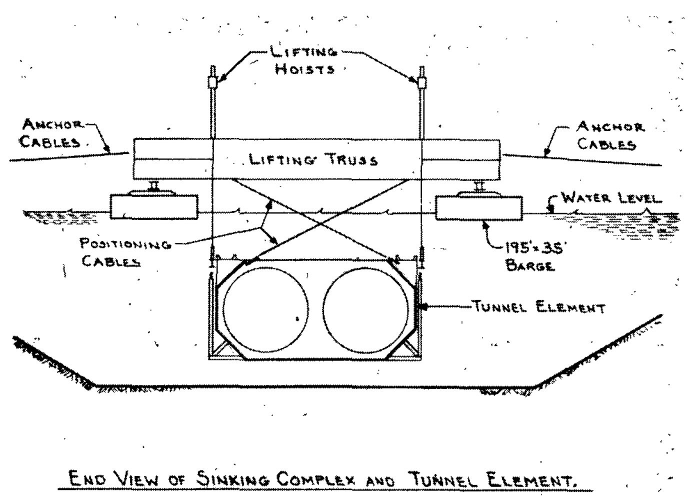


FIGURE 4

Mobile River Tunnel; end view of the sinking complex and tunnel element.

A recent example of sunken tubes for highway tunnels is the Mobile River project in Alabama. A barge-mounted truss (see Fig. 3) combined with a jack system was used to sink and set seven 2000-ton sections of precast twin-tube into a deep trench excavated across the river. Before sinking, the tunnel bed was excavated using first a hydraulic dredge and then a clamshell dredge. A layer of gravel in the trench provided the bed for the sections. Each lifting truss straddled a pair of barges as shown in Fig. 4. The precast sections were towed into place between the barges one at a time and secured to the trusses prior to sinking. Suitable controls of the truss mounted jacks and cables enabled the operators to lower and guide the sections to the specified alignment regardless of placement depth. Horns and pans at each end locked each tube with the preceding and the following sections.

BASIC ECONOMICS OF TUNNELING VERSUS CUT-AND-COVER

Quite often engineers have emphasized the need for good geological investigations without which no intelligent decision on a tunneling project could be made. Thus, the nature of the ground is perhaps the single most important factor in tunneling and has a decisive influence on both the process to be used and the cost involved.

The economics of tunneling versus construction in cut-and-cover is a second area of concern among engineers. In this context a clarification seems appropriate between construction in open cut and cut-and-cover. These terms sometimes are used interchangeably. Open cut involves precisely what it means; a section is excavated and remains open for as long as construction continues. The project is completely in the open, including earth moving and backfill. Cut-and-cover, on the other hand, involves an initial excavation and shoring followed by the installation of a temporary and sometimes a permanent deck. In this manner the street is restored and the surface activities resumed whereas the remaining of the construction, including earth moving, proceeds under cover. An example of cut-and-cover is the recent construction of the Washington subway for which contractors had to work along one side of the street to provide shoring and temporary decking, and then move to the other side so that traffic was alternately switched. In this manner traffic is maintained as best as possible, but despite all efforts experience shows that there always is a certain interference with surface activities.

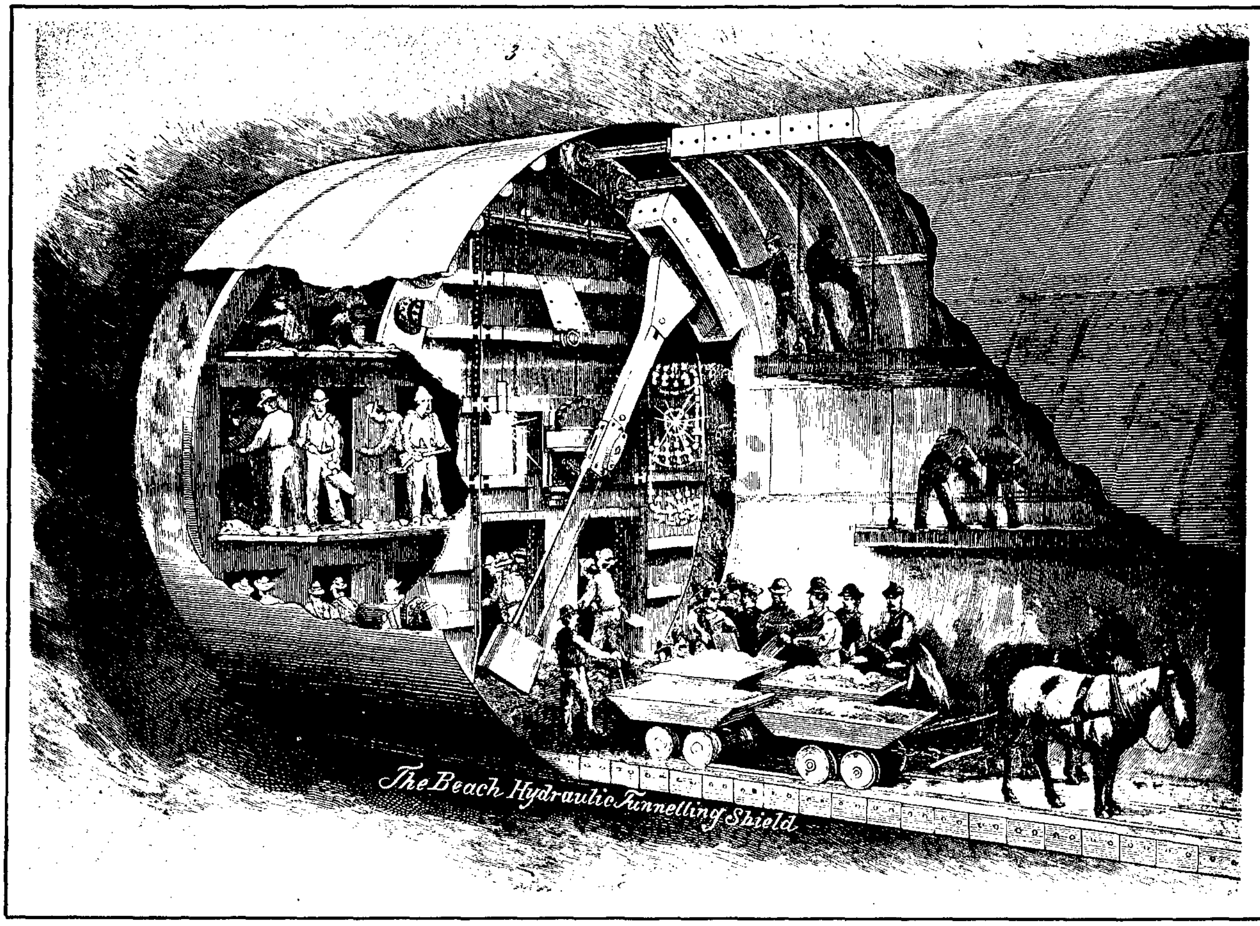
As a rule of thumb and under average site and soil conditions, where the depth of excavation does not exceed 40 ft cut-and-cover is probably the most economical construction method. If the excavation is more than 60 ft deep, tunneling is probably the most economical choice. The type, number and location of underground utilities and structure, and the soil type and strength can change this decision drastically.

In cities with good soil such as Chicago it might be that any excavation more than 50 ft deep should be carried out by tunneling. The doubtful range of 10 to 20 ft between these limits should be evaluated very carefully since it may be either cut-and-cover or tunneling. However, there are certain factors favoring tunneling such as disruption of traffic, partial closing of streets, construction at street level and all the associated problems. On the other hand, a carefully driven shield tunnel should not necessarily cause more settlement than an ordinary cut-and-cover construction. The record shows that some settlement always results regardless of method.

TUNNEL SHIELDS AND COMPRESSED AIR

1. Tunnel Shields

Since the very early stages of tunneling, for example the St. Clair River Tunneling Project for the Grand Trunk Railroad built in 1888, only few major changes have been made in the basic techniques of soft ground tunneling. For those who are not familiar with shield-driven tunnels, Fig. 5 shows probably more than any present day photographs about the entire tunneling process. It is true that the available equipment has steadily improved, but the process remains essentially the same. As a guideline for the initial evaluation of the most appropriate tunneling technique, it should be pointed out that usually mechanical moles are most economical for tunnels over 2 miles (3.2 km) long, whereas shorter tunnels are better suited to the use of tunneling shields.



St. Clair River Tunnel, 1888, for the Grand Trunk Railroad between Port Huron, Michigan, and Sarina, Ontario (6000ft. long, 21 ft. outside diameter). **FIGURE 5**

The utilization of a shield begins with the construction of an access shaft to the tunnel grade; in general, the bottom of the shaft is covered with concrete to prevent base instability and heave. Two walls should line the shaft; a hole is then cut in the inner wall, and the shield is shoved through this hole to the position of the outer wall. The space in front of the shield is filled with sand, and the shield is advanced through the other wall by extracting the piling. A good practice in advancing the shield is to place jacks in 3.14-ft intervals around the shield's circumference, and breasting jacks should also be used to maintain the tunnel face in place.

The use of shields has received wide acceptance not only in the United States and North America but all around the world, and many problems have been worked out from these applications. The principal advance in the use of shields in the last ten years probably has been the installation of a backhoe at the face of the shield for breaking down the face.

Referring to Fig. 5, the entire tunnel driving and installation can be seen as it was carried out. Ahead of the tunnel men hand excavate under the protection of a tunneling shield, and the excavated materials are removed in muck cars. Behind the excavation the erector arm can be seen setting the cast iron lining segments, and on the upper stage miners are ready to bolt them in place. Half-way down other workers are corking the seams. Essentially, this is still the way in which shield driven tunnels are completed at present.

In unfavorable ground such as sand and gravel breast jacks should be used on the shield to keep the face breasted at all times. The operation begins at the top by moving the breast boards ahead and holding them with a jack until the whole system is moved ahead by the length of one shove. Then shoving commences and simultaneously the jacks are allowed to retract slowly in order to keep them on the face at any time. Unfortunately, when a backhoe is used for mucking it is not possible to keep a square face which is necessary for the breast jacks to work and keep the pressure on the face.

2. Compressed Air

The requirements for compressed-air tunneling work are covered in detail in the OSHA (Occupational Safety and Health Act) regulations; only the New York provisions are more stringent than the limitations set forth in these regulations.

It might appear that work under compressed air is a gradually dying industry for several reasons. Firstly, new techniques have been introduced in underground construction (see also other Papers) for ground strengthening and ground water control, and besides the conventional dewatering methods we use today extensively grouting and cutoff impervious walls to deal with the problems of unfavorable ground. Secondly, and perhaps most important, it seems that OSHA has adapted the idea that compressed-air tunneling work is dangerous and harmful to man's health, and it should therefore be eliminated. This speaker was on the Compressed Air Committee when the code was under preparation, and from experience it would appear reasonable to state that it is safe for men to work under one atmosphere, which is approximately the equivalent air pressure of 15 psi. However, the OSHA Committee decided that the allowable air pressure for tunneling work should be limited to 12 psi. Compressed-air work at pressures over 12 psi or with tunnels larger than 14 ft (4.2m) in diameter require a separate man lock and muck lock. The efficient flow of both man and materials is the objective in the layout of these locks. But even at 12 psi these requirements call for a medical lock attendant to remain on the job for five hours afterwards, and according to most interpretations this is necessary even at 2 psi pressure.

FIGURE 6

Air locks for the 2nd Boston Harbor Tunnel.

Air locks. Fig. 6 shows the Boston Harbor Tunnel. It is conceivable that this tunnel might be flooded. This project was under the Bay. The arrangement required the muck lock shown in the lower portion, about 10 ft in diameter, and two locks in the upper compartment of the tunnel. The lock shown to the left was used for the access of working men in and out of the tunnel face. The lock shown to the right was 6 ft in diameter, and was used as emergency lock with a door open on the far side at all times so that in case of emergency such as a sudden flood it would be possible for working men to get out safely.

In the East and especially in the New York area it is necessary to transport workmen and their supplies in a man car, and this is a small vehicle that should carry from ten to twelve men. This vehicle with its load should pass a lock door opening 4 ft x 6 ft.

CONVENTIONAL TUNNEL LINERS

The subject of new standard lining systems, especially precast sections, is discussed rather extensively by other speakers. Hence, this paper will concentrate on more conventional tunnel linings, the standards adapted at present, and on the objection of using a shield-driven tunnel as it relates to the high cost of primary linings.

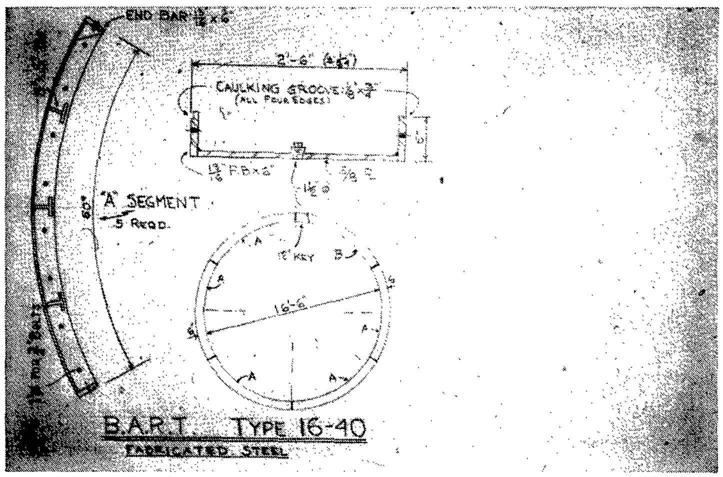


FIGURE 7

Prefabricated steel lining sections for the BART project.

Cast iron linings have been used in the past and successfully withstood corrosion. The recent practice is to use prefabricated steel liners, usually of a 5/8" skin plate. This has proved to be very satisfactory judging from the wide use on the Washington Subway and the BART project. Such a steel liner is shown in Fig. 7 for the BART system. The engineers were in this case concerned about possible corrosion so that the sections were sand blasted in the shop, and then spray coated with a suitable epoxy. After they were installed they were given some outside protection as an added safeguard against corrosion.

Cast iron segments were proposed for the BART system but finally were not adapted. These were designed using a 3/8" skin plate as compared to much thicker plates used for the East River tunnels in New York. The sections were provided in the upper center with a special bar for handling purposes. This bar can be moved from one position to another in any suitable pair of holes, and provides a handle for the erector arm to grab.

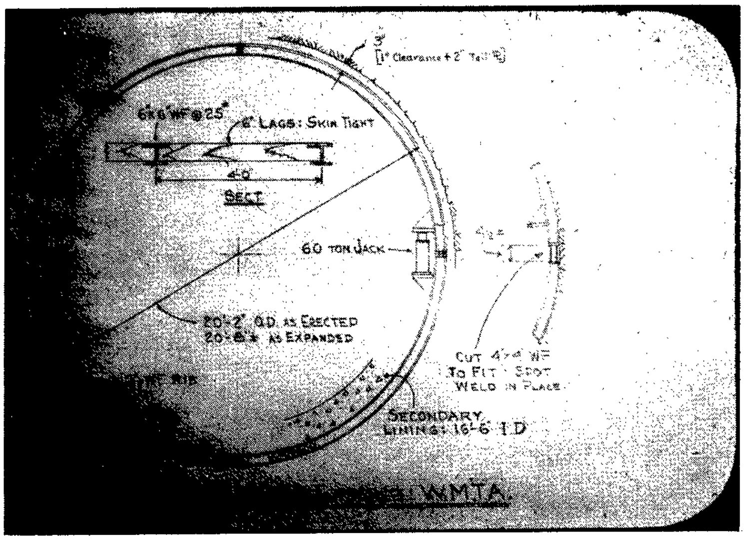
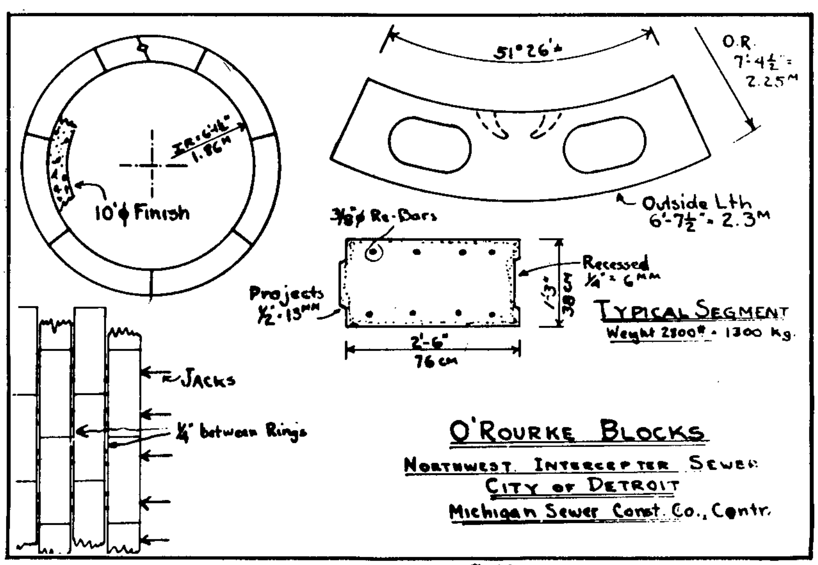


FIGURE 8

Rib-and-wood lagging primary lining system for the Washington, D.C., subway.

Another popular lining system is the one shown in Fig. 8, consisting of ribs and wood lagging. Most of the contractors on the Washington subway have been using this type of lining as a primary lining system. The construction is in this case completed with the installation of a secondary lining, usually a reinforced cast-in-place concrete tube as shown in Fig. 8.



O'Rourke concrete blocks used as tunnel liners.

FIGURE 9

In the past I have mentioned the so-called O'Rourke Blocks which were some time ago in use but were later abandoned. This concept of precast concrete segments is shown in Fig. 9. Whereas it still is a good concept, it has a serious drawback in that the lining has to be followed by a secondary concrete lining because it is not completely watertight. The reason for lack of complete watertightness is that the rings were kept about 1/8" to 1/4" apart so that they would not break upon contact. In water bearing formations it was quite difficult to seal the joint, and this necessitated the secondary lining.

This speaker is presently involved in a study for the U.S. Department of Transportation on contractor's views particularly on tunnel liners. The consensus in this case is that precast concrete liners should receive more emphasis in the United States. These systems have been in wide use in Europe and Japan primarily because of the considerably lower cost involved. Bids were taken for the BART projects on concrete segments that proved to be less expensive, but these lining systems were not used.

Without getting beyond the scope of this discussion, I should mention some of the precast liners used abroad. An example of an early precast segment is very similar to our corrugated steel plates, except that it is much heavier and probably cheaper. Some engineers have expressed concern that unless the installation is successfully controlled, it still is necessary to provide a secondary lining to avoid leakage of flowing water and sewage into the tunnel area.

Probably one of the earliest types of precast segments goes back to about 1936, originally developed in Germany and first sold to contractors in Buenos Aires sometime in 1938 or 1939. During the war this liner could not be exported because of the sea blockage. This speaker was eventually contacted by contractors who used it, and developed a special shield. This shield had face jacks so that at the end of the job the breasting was clear up against the shield or within 6 in. Although this dates back to 1941, it was not until much later that a manufacturer in St. Louis could build the hydraulic jacks and pumps. The shield proper was built in Buenos Aires and was detailed in the metric scale. Attempts to use this idea for the Pennsylvania Railroad for some tunnels under the main line in New Jersey were made but without success. The idea was also presented to the BART project and to the Sewer Division of the city of Washington, D.C. but again without success.

SETTLEMENT DUE TO TUNNELING OPERATIONS

In relation to the construction of the Washington, D.C., subway an investigation was carried out about two years ago for the section that passed in front of the White House, and the first shield that was used to advance the tunnel resulted in a 6-in. settlement. The second shield was modified and shortened, and the resulting settlement was of the order of 2-1/2", which was within realm. From these and other data, the diagram of Fig. 10 has been prepared allowing an estimation of the loss of ground. Knowing the settlement at the center y and the total spread of the settlement x, the volume of ground lost per linear foot of tunnel length is 1/2xy. For an average tunnel depth of 50 ft and tunnel diameter of 20 ft, the loss of ground would be about 15 cu. ft. per linear foot if the settlement at the center is 6 in. Most settlements, however, are from 2 to 3 in. so that the average loss of ground is from 6 to 7 cu ft per foot of length.

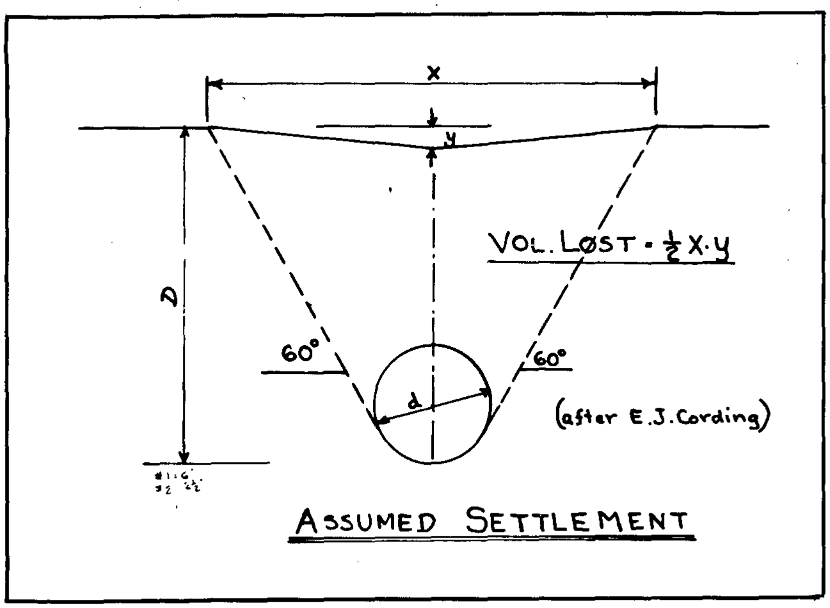


FIGURE 10

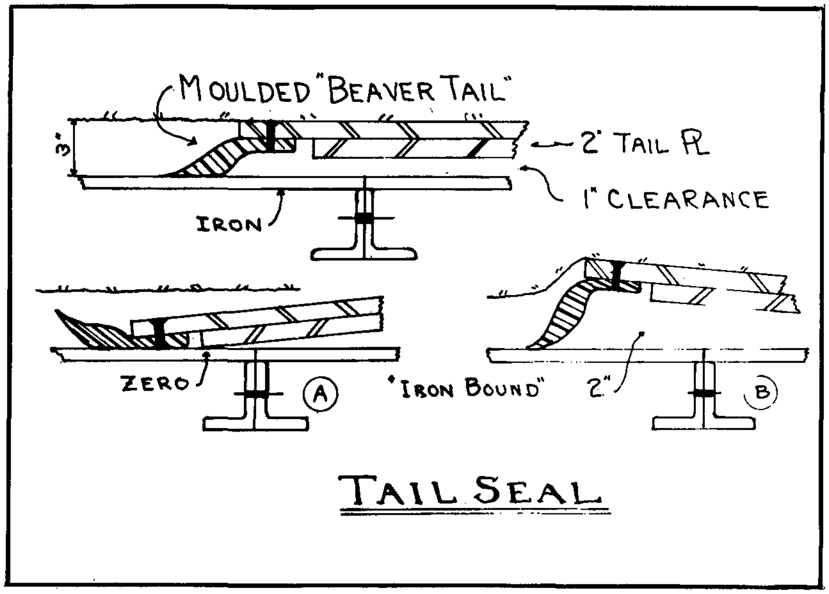
Diagram for estimating loss of ground due to tunneling.

It is important that this settlement is caused mainly by two reasons; one is loss of ground at the tunnel face, and the other is failure to fill the tail voids. The former is erratic and quite difficult to predict; the latter is better predicted and calculated, and involves loss of flow of grout into the tail void, crushing of old sewers above the excavation, and heave or crushing of adjacent old foundations sensitive to the operation.

Since the tail void can be estimated, and since it commonly is filled with pea gravel, it is possible to measure the quantity of pea gravel placed at any time and correct, if necessary, the placement method. As an example, for an 18-ft diameter tunnel with 3 in. tail void, the total volume created behind the shield is 21 cu ft per linear ft of tunnel, and this provides an indication of probable settlement. The remaining settlement should be due to the face operations.

At present the recommended practice is to use pea gravel injections, probably using a 4-cu yd car with two pea shooters, one at each end, noting that this must be designed and built so that it can pass through the lock doors. In connection with the shield method, the pea gravel is injected as soon as the iron has left the tail of the shield; two or three rings later cement grout is injected into the same space to fill any voids left by the pea gravel and cement the pores, and to stop leaks of water in or air out of the tunnel.

The following is a rule of thumb for the use of pea gravel filling around the tunnel lining. The usual practice is to provide for a thickness of the tail void of 0.1 in. per ft (8.2mm per m) of tunnel diameter. Whereas no minimum space is required for the pneumatic injection of pea gravel, plenty of holes should be provided for this operation. Grout material, although perhaps better suited for filling voids, tends to run back into the tunnel and is thus less practical than pea gravel except as supplementary material. A so-called "beaver tail seal" is used in Japan; this is shaped like a beaver tail, and when cement grout is injected it stops it from running back into the shield so that one injection is sufficient. In the United States contractors have been using a similar tail seal shown in Fig. 11. However, when the shield is steered as shown in (a) it produces an iron bound resulting in zero clearance causing the tail seal to stick in the air and be grouted in. On the other hand, the arrangement shown in (b) has twice the clearance, and in some instances the tail seal has been blown inwards by the external pressure, a situation highly undesirable since it is not possible to replace a tail seal, if it moves out of place in this manner.



Tail Seal. **FIGURE 11**

Slurry-faced shields. Fig. 12 shows possible new equipment for tunneling. Both the British and the Japanese manufacturers have developed a slurry-faced shield; the one shown in Fig. 12 was developed by the National Research Development Corporation of the British Government. The photograph shows clearly the cutter face which revolves and peels off the sand mixing it with slurry. The slurry-sand mixture is then pumped to the surface by reverse circulation, where the soil fraction is separated from the slurry in vibrating screens and cyclones and the slurry is returned back to the face.

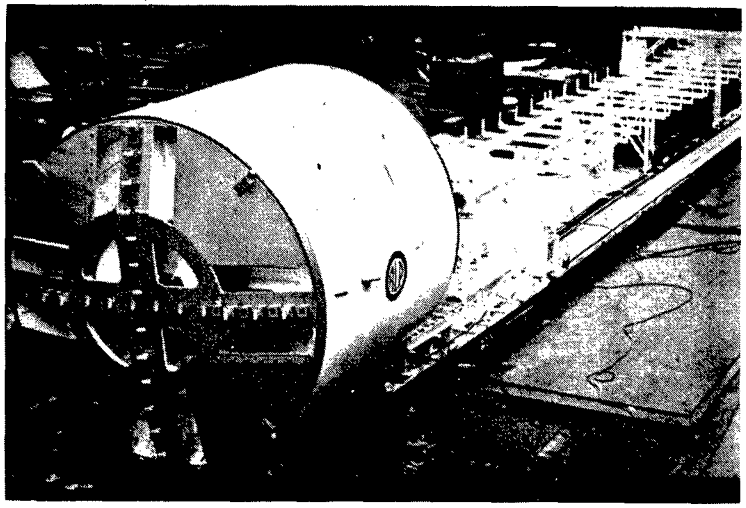


FIGURE 12

A slurry faced shield made in England.

This operation presumably facilitates materials handling and may result in better control of the face excavation, hence in less settlement above the face.

PAPER 11

Soft Ground Tunneling Machines

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PAPER 11

SOFT GROUND TUNNELING MACHINES

by

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HISTORICAL HIGHLIGHTS

Soft ground tunneling by machine seems to be a fairly recent development. Without making an effort to research the subject, I cannot recall reference to the use of tunneling machines in soft ground earlier than about twenty-five years ago. A historical search might, on the other hand, turn up all sorts of machines which must have grown out of the first soft rock machines of the middle of the last century, where they were developed in Great Britain for the Channel Tunnel project.

There appears to be a real gap between the work done in the late Nineteenth Century and the rejuvenation of soft rock tunneling in the early 1950's. My guess is that some machines of the type developed for the Channel Chalk - a beautiful material for tunneling - were probably applied in London clay tunneling, or other soft ground work. They may have failed due to the fact that soft ground is generally not as consistent as the Channel Tunnel Chalk. I am using the phrase soft ground to indicate soils of various types as distinct from soft rock. Soils are usually heterogeneous material in layers and lenses. They are often unstable when water is present and some soils will run when dry. The early rotating head machines designed for an ideal material like Channel Tunnel Chalk would probably be a failure in most soil formations.

One must consider also the effect of the development of the soft ground tunneling shield at about the same time. Shield tunneling was so much more reliable than methods used previously in soft ground that it became universally adopted, and has been practiced with very few changes up to the present.

Therefore it is possible the rotary head machines where never seriously developed for use in soft ground until the late 1950's or early 1960's.

MACHINES FOR SOFT GROUND TUNNELS

In this paper, the term "machines" will be used to refer to equipment which performs the job of excavating the tunnel face as well as providing the necessary support of the ground. The classical shield, a tunneling machine in one sense of the term, is basically a mobile support system providing work areas for face excavation and erection of the tunnel lining.

1. Rotary Head Machines With Wall Grippers

Gripper type machines must be used in very stiff and competent soils. This type of machine has also been used in soft rocks like the Channel Chalk or soft shales. A surprising number of machines of this type were built in the United States and used mainly in the Detroit and Chicago areas where some ideal soils may be found. Some of these machines were built by tunnel contractors in their own work shops. The open spoked wheel cutterhead does not provide face support and there is usually not sufficient torque to turn the head if the face caves. Ring beams and solid wood lagging (ribs and boards) is the type of tunnel support usually used with these machines.

2. Rotary Head Shield Machines

The rotary head shield (called a drum digger in England) is probably the most commonly used type of soft ground tunnel borer. They have been used wherever the soil was generally good, self supporting, dry and stable. They have open spoked cutterheads and are therefore not satisfactory in unstable ground, but their shield offers good protection against a cave-in at the crown or walls.

Segmental lining, either cast iron or precast segments, are assembled behind the shield and expanded directly against the clay. This technique of expanding the lining is also used in the Midwestern United States where ribs and boards are assembled under protection of the shield and expanded after the shield tail has passed.

These rotary head shield machines are simple in construction, inexpensive, and give good, reliable results when used in stable soils.

3. Rotary Head Shields With Closed Face

Face support is the most difficult thing to accomplish with a rotary head machine. Machines have been built in the United States, Germany, Japan, and England which have a cutterhead almost entirely closed, or in some cases completely closeable hydraulically. The objective of this development has been to keep the cutterhead in close proximity with the tunnel face being cut and only take material in through the cutterhead which is shaved off by the cutters.

If the soil is not stable, a face cave-in can occur which, incidentally, can not be detected by the personnel in the tunnel but, in theory at least, this can only be a small cave-in since there is not sufficient room for a large cave-in to occur. The usual practice in that event is to increase thrust and keep going forward as fast as possible. This gives you a chance to get under the caving area with the soil at the face, using it as a rotating face breasting system. The cutterhead drive must be capable of developing very high torque to use this rotary breasting technique. Face cave-in has occured in some cases where a run develops in only one part of the face while the remainder of the face is standing holding the machine back. The machine can then excavate a large cavity before the condition is detected. Some contractors have used continuous conveyor weighing or matched loaded muck cars with advance of the machine to detect an over excavation at the face as quickly as possible.

4. Oscillating Cutterhead Machines

Machines have been built in Germany, Japan and the United States with oscillating cutterheads. The drive is usually a pair of powerful hydraulic cylinders which extend and retract, causing the cutterhead to oscillate about a central bearing. An American manufacturer built machines of this type with several different segments of the cutterhead oscillating independently about separate centers with the top half of the cutterhead inclined forward. The hydraulic cylinder drive provides the torque to enable the rotary breasting technique but the reciprocating or intermittent motion reduces efficiency.

5. Excavator Shields

The soft ground tunneling machines described so far have been of the full face types with rotating or oscillating cutterheads. The excavator type of shield was developed to cope with the variable soil conditions which so often exist in soft ground tunneling. The excavator shield is a classical shield with one or more excavators mounted on the interior in position to dig the face and load the excavated material onto a conveyor or directly into haulage vehicles. Some large diameter machines have mounted several standard backhoes on benches inside the shield, however, the specially designed ripper excavator provides much greater performance and reliability.

One of the principle advantages of mounting an excavator tool inside a shield is that the shield can react large digging forces, much greater than those exerted by a mobile vehicle such as a backhoe. Therefore special excavators have been developed to dig the face selectively and provide the capability to dislodge and remove large boulders or cut through layers of cemented soils and conglomerates as hard as concrete. Single point rippers have been built which exert over 80 tons on the ripper tooth.

Another advantage of the excavator shield is that at least partial face breasting is possible with the excavator in place. Some machines have been built to provide hydraulic actuated breasting doors, which can close the face in a few seconds. Others have incorporated hydraulically extended poling plates as an extensible hood.

This ability to use the excavator selectively to break or remove boulders or hard layers while keeping the remainder of the face closed off with hydraulically operated doors is a significant development in shield tunneling. These machines can tunnel through much harder material than the rotary head types, particularly in boulders, while maintaining face control over the unstable parts of the face.

6. Pressure Bulkhead Shields

The first Pressure Bulkhead Shield to come to the attention of the writer was built by Entreprises Campenon Bernard in 1961 and used in Paris for a section of the high speed Metro subway system. This machine was a shield with a pressure bulkhead to permit compressed air to be applied to the tunnel face and leave the remainder of the tunnel in free air.

Excavation was carried out by means of several hydraulic excavators mounted on the forward side of the pressure bulkhead with the operator standing in free air to the rear looking through bulkhead windows. This may have been the first machine to disclose the concept of permitting the fluid to rise in the face and regulating the fluid pressure with compressed air in an air chamber between a front and rear pressure bulkhead. In this way the machine could be operated with the face completely filled with fluid, partly filled or dry. Much could be hoisted by a clamshell grab from the wet forward portion and deposited on a conveyor in the dry compressed air section. This was the forerunner of today's compressed air bulkhead and slurry face machines.

In 1962 The Robbins Company built the first rotary head pressure bulkhead machine. It was used to bore a 35 foot diameter double track subway tunnel for the Paris Metro. This machine operated in running, silty sands under a maximum of two atmospheres of pressure. The soil developed sufficient cohesion to stand at the face when subjected to compressed air. However, unusual circumstances occured when the machine encountered lignite coal layers which burst into flame and dried and subjected to the extra free oxygen of compressed air. The face had to be flooded and a grout curtain injected to proceed through this area. The need for a machine which could operate with a wet face was recognized at that time.

While this Paris machine was operating the Bade Company in Germany built a pressure bulkhead machine which was successfully used to bore a subway tunnel. This machine used the rotary breasting technique with the use of compressed air restricted to the tunnel face.

Komatsu Company in Tokyo, a Robbins licensee, built a pressure bulkhead machine which was used to bore a pedestrian underpass tunnel. Komatsu developed a rotary muck discharge system which required less space than the double hoppers used at Paris.

These pressure bulkhead machines were successful in operating with air pressure applied to the tunnel face while the machine operators, lining assembly crews and other tunnel personnel worked in free air. Key elements of the designs were the seals between the shield tail and the segmental lining, segment sealing systems and muck discharging systems.

7. Slurry Face Shields

Some soils cannot be properly stabilized by the application of compressed air. If one can maintain these soils in their natural state, while excavating the face without seriously altering the forces acting on the material, a natural stability will be maintained. This can be accomplished by a combination of rotary breasting the face while shaving it off and, at the same time, maintaining the fluid pressure resulting from the ground water present. The use of this system requires the cutterhead to be operating in a fluid medium. The stabilizing effect can be enhanced by the addition of a thixotropic mud such as bentonite to the fluid to penetrate the surface layer of soil at the face and suspend the particles while maintaining the fluid pressure. This admixture reduces the dependence on an effective rotary breasting system.

It has been reported that the first disclosure of slurry face tunneling system was in a German patent which has since expired. The Japanese had tunnels utilizing this technique under construction in 1971.

A slurry face tunnel borer was used to bore a twelve foot diameter main sewer in Houston, Texas by the Gardner Engineering Company in 1968. This machine was probably the first to use this technique in the United States. The shield had a forward pressure through which a central shaft protruded. A ditcher chain was cantilevered from this shaft in a radial position so as to cut the entire face, as it was slowly rotated about the center, while the chain traveled from the center radially outward to the periphery of the bore, then back to the center.

The system was built to take advantage of the possibility to pump out the tunnel muck as city water was pumped into the top portion of the face and out of a port in the lower portion of the bulkhead together with the muck slurry. A 6 inch Moyno pump was used to send the muck out of the tunnel.

It has been reported that quick sands which occured in pockets or lenses created a problem. They could pump out a whole pocket of sand in a hurry without making the required advance. They consequently had some difficulty in maintaining grade in the sand areas.

In 1972 Edmund Nuttall Ltd. drove a short section of test tunnel with a bentonite slurry face machine which was built by Robert L. Priestley Ltd. with the support of the National Research Development Corporation. The NRDC are holders of patent rights to this development which was the brainchild of Mr. T. V. Bartlett, a Director of Mott, Hay and Anderson, consulting civil engineers. The efforts of this impressive team of engineers resulted in a satisfactory trial after the usual series of debugging operations.

This method will be used to drive a 1000 meter long sewer tunnel in Warrington, England.

In the meantime, the German firm Wayss and Freytag were having their first slurry face machine built at the shops of Shaffer and Urbach. This new German development is a sophisticated system utilizing bentonite slurry mixture, compressed air and water. One of the most notable features is the inclusion of the compressed air chamber as a pressure regulator in a way similar to that of the Campenon Bernard shield but in this case, in combination with a rotary cutterhead. the W & F machine, termed the "Hydroschild", is now under construction in a 3.70 M diameter sewer tunnel at Wilhelmsburg.

Another sophisticated slurry face machine recently completed a 3,444 foot, 16.7 foot diameter sewer tunnel in downtown Tokyo. The Tekken Kensetsu Company Ltd. of Tokyo developed a machine which worked in interbedded quicksand, silt and clay at rates up to 907 feet in one month. This rate could have been considerably improved were it not for limited space on the surface for an adequate slurry-muck separation facility. This Tekken slurry face tunneler has set the performance standard which must become the goal of others in this field. Their main developmental breakthroughs were in the key areas of pressure regulation, measurement and regulation of rate of muck removal from the cutterhead and automation of controls.

8. Soft Ground Machines in the Future

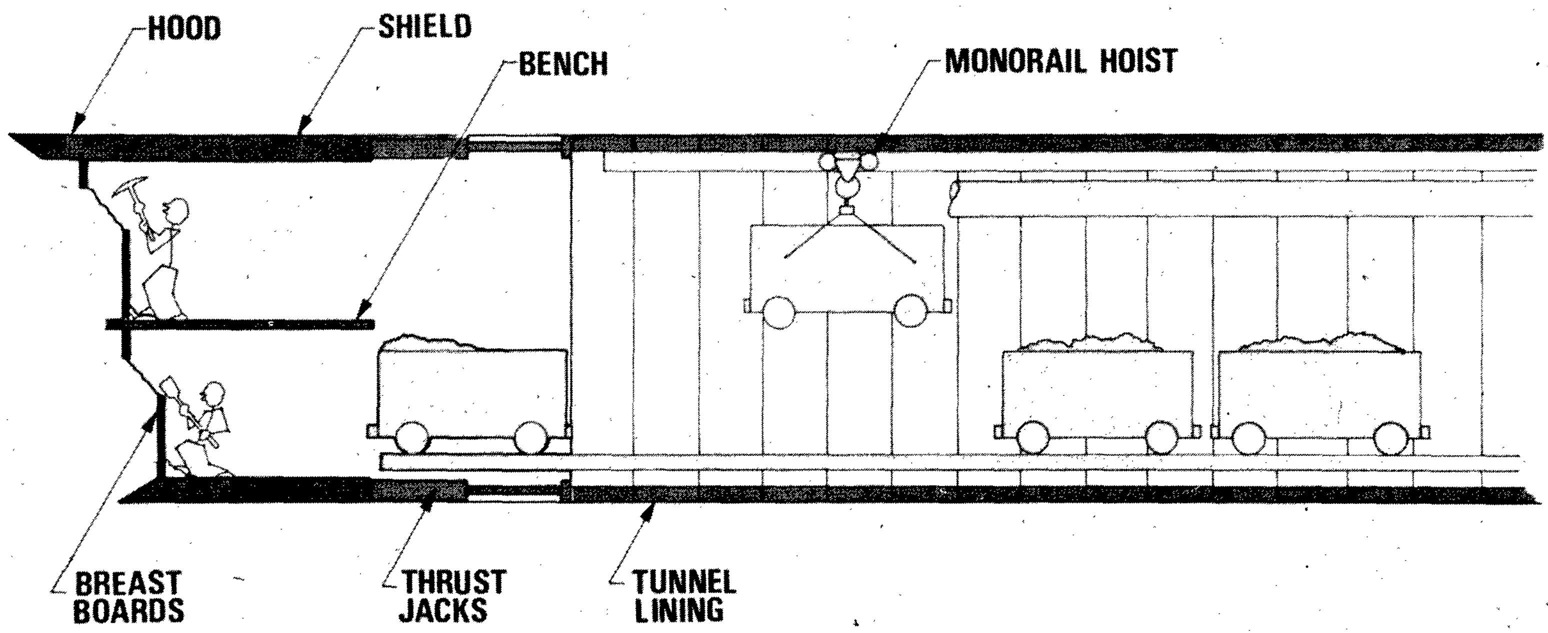
The serious problems of soft ground tunneling are now being attacked by slurry face tunnelers around the world. However, there are still many limitations. What about large granitic boulders as can occur in a glacial till? What if these boulders occur in the pressure of cohesionless sands and gravels below the water table? These conditions have already been noted in some locations in the United States and abroad where new subway systems are being contemplated. Some cities like Hong Kong have been forced to locate tunnels at the interface between running soft ground and a hard granitic bedrock where the tunnel may run into and out of a full face of either material.

There exists clearly a challenge to the ingenuity of future tunnel Builders. A slurry face machine to cope with these conditions must use cutters which can withstand serious shock and cut very hard rock. The single edge disc cutter might fill the bill but how well will it do in very soft material? It may have to use hydrostatic bearings to support the shock loads while remaining free enough from friction to keep turning in the soft material.

Cutter inspection and changing must be reduced to avoid serious delays which will result from men entering the pressurized environment. Can we use telemetry to provide continual indication of overheating or excessive wear to the cutters? This method is being used at this time in a British tunneling machine test in chalk.

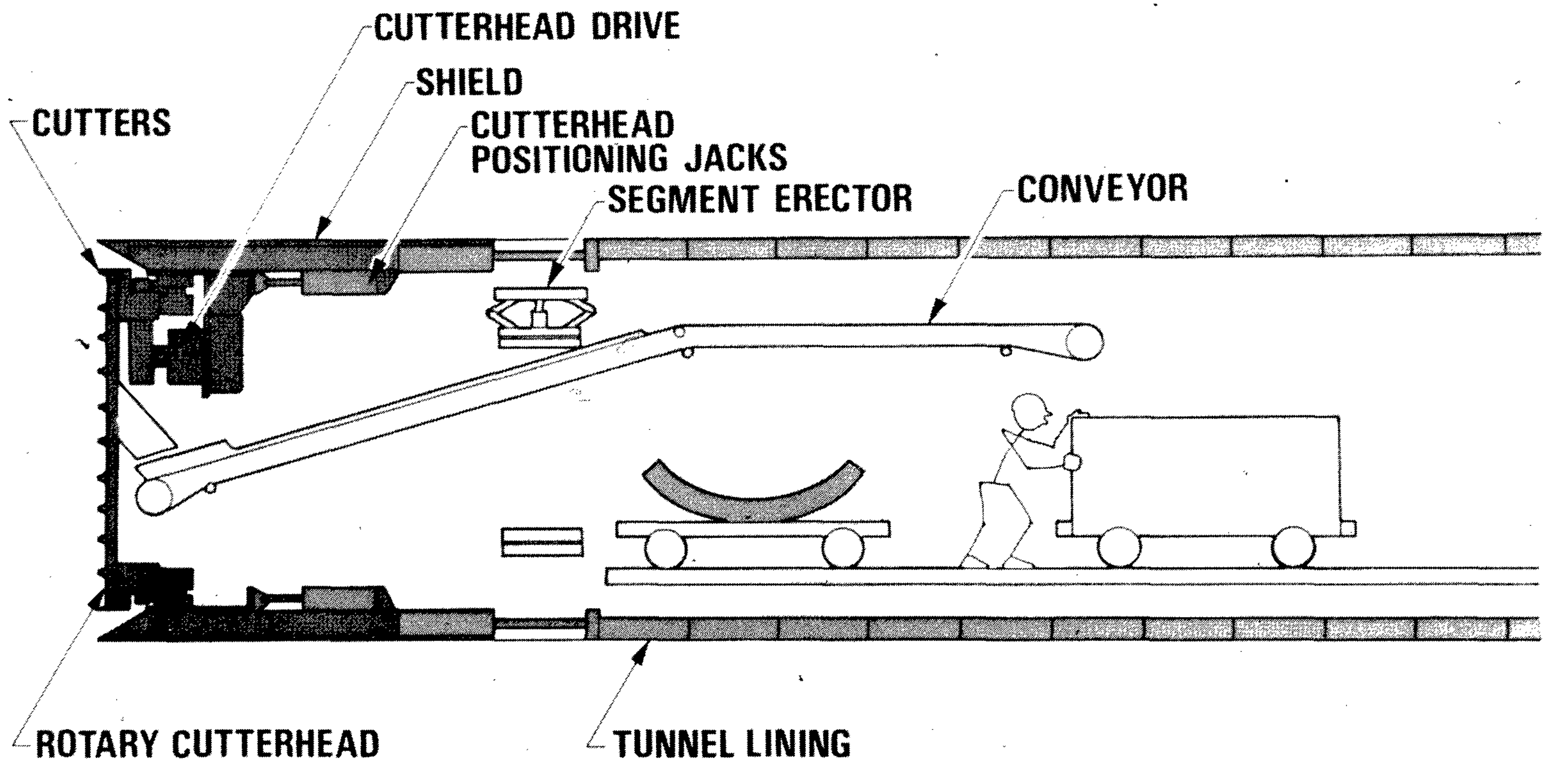
Segmental tunnel support systems have been developed to provide a leak-free seal at the segment joint faces as well as at the tail of the shield. It now appears that these designs must be pushed further to get segments which are cheap to produce and can be assembled in place at the rapid advance rates of tunneling machines.

As we solve these problems we will be working toward the universal soft ground tunneling system which can be used without any ground surface settlement in any ground condition which is encountered. We still have a long way to go.



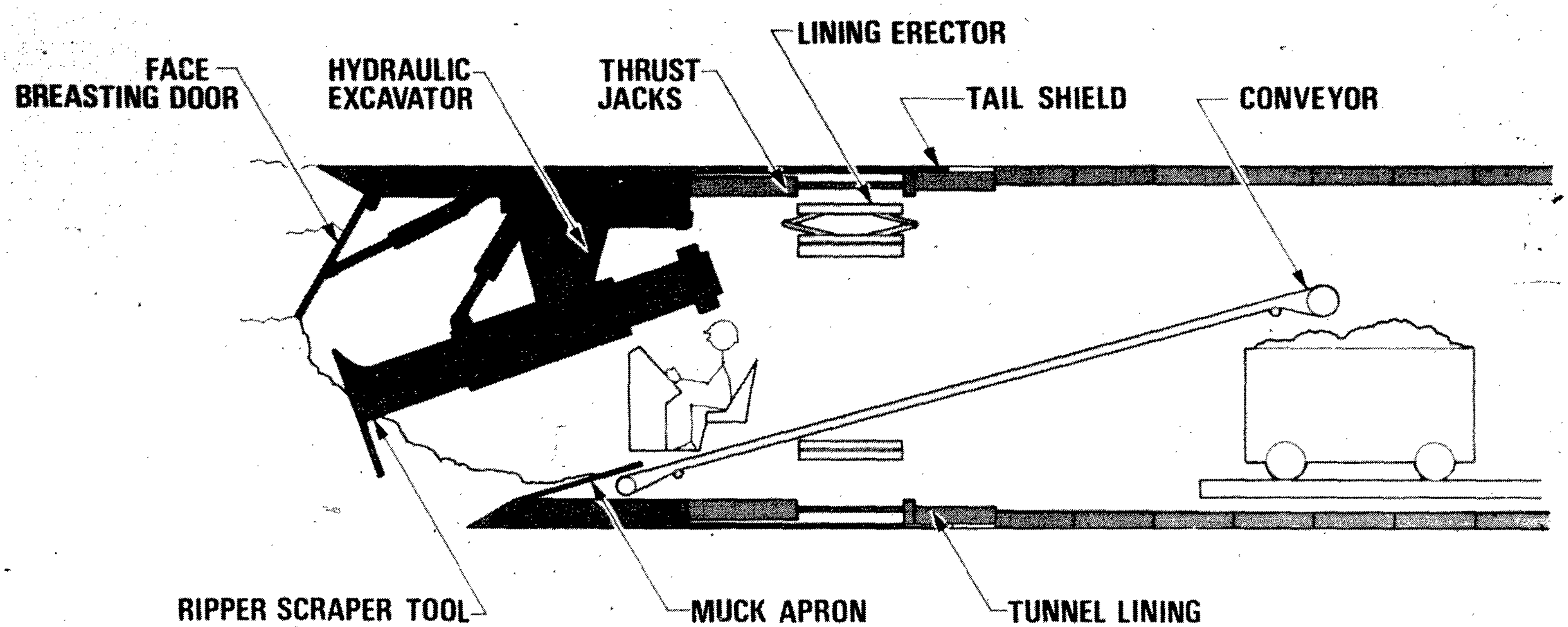
Classical Shield

FIGURE 1



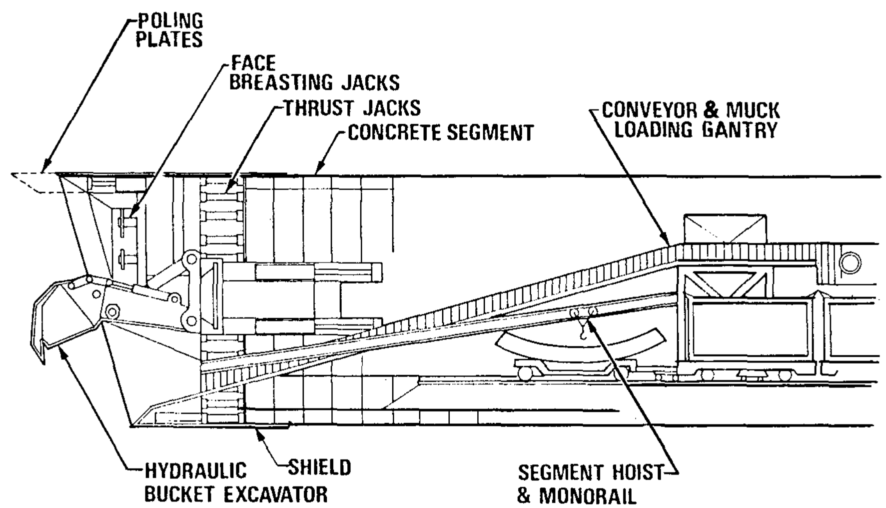
Rotating Head Shield

FIGURE 2



Excavator Shield With Breasting Doors

FIGURE 3



Excavator Shield with Poling Plates

FIGURE 4

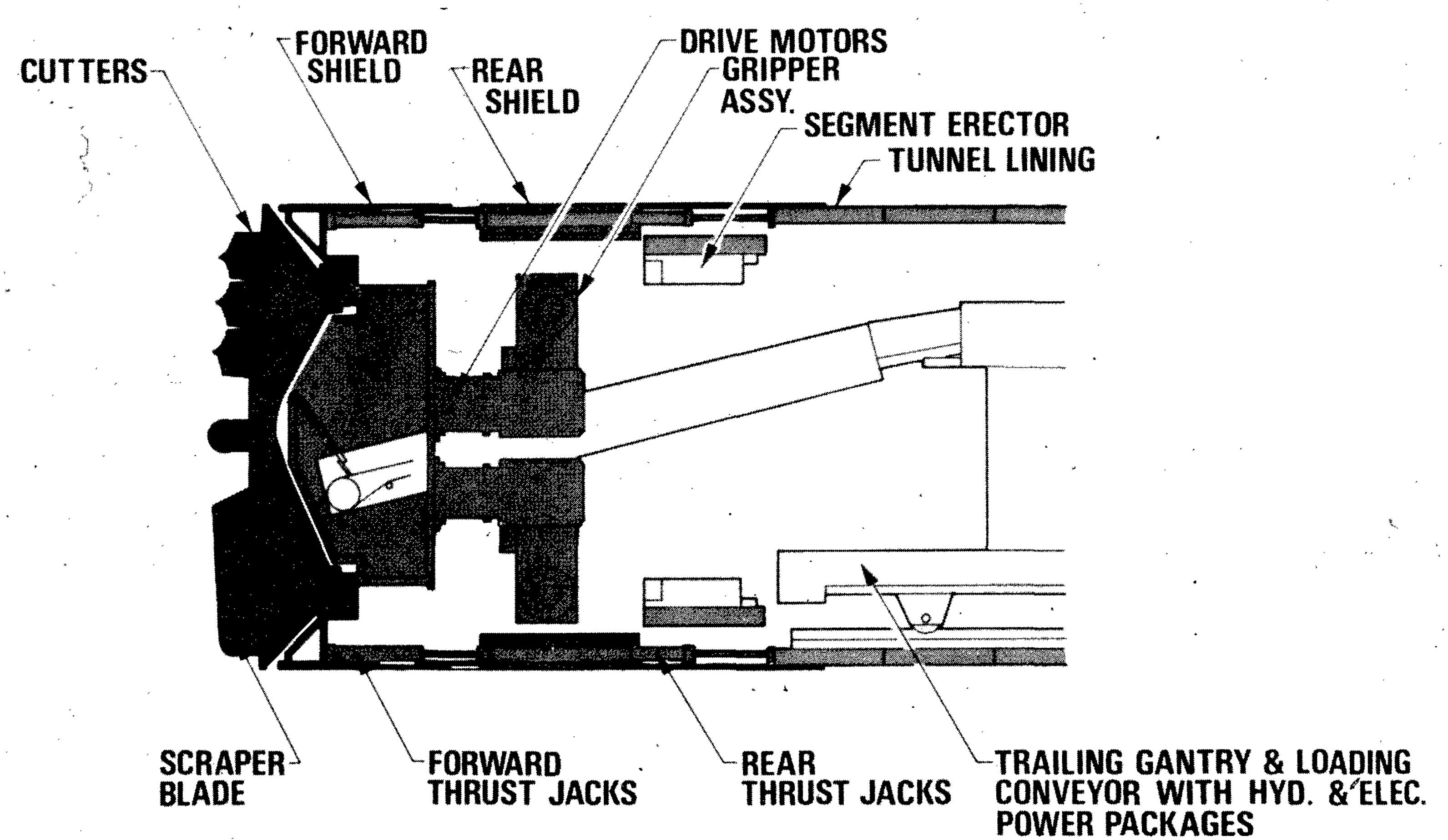
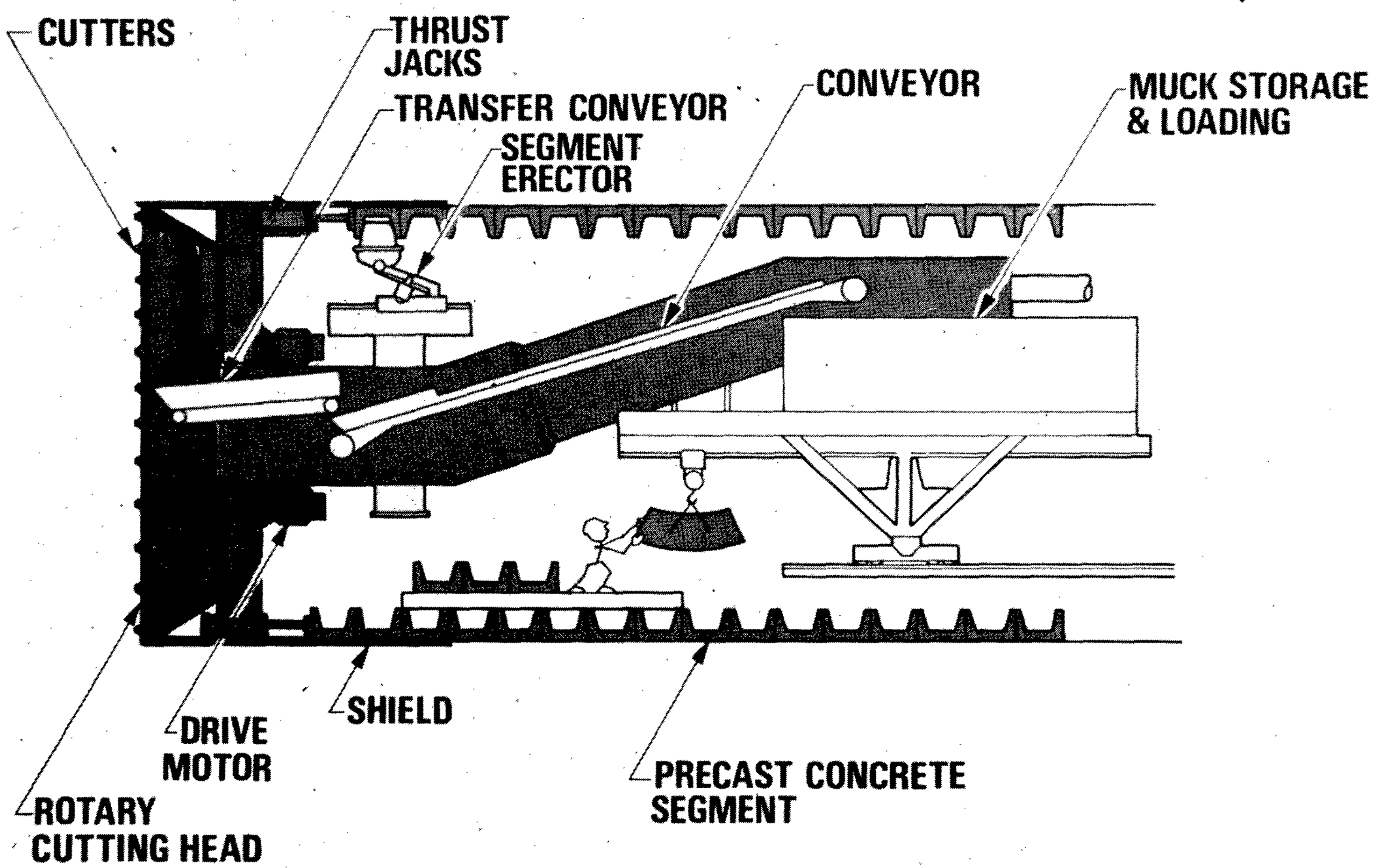


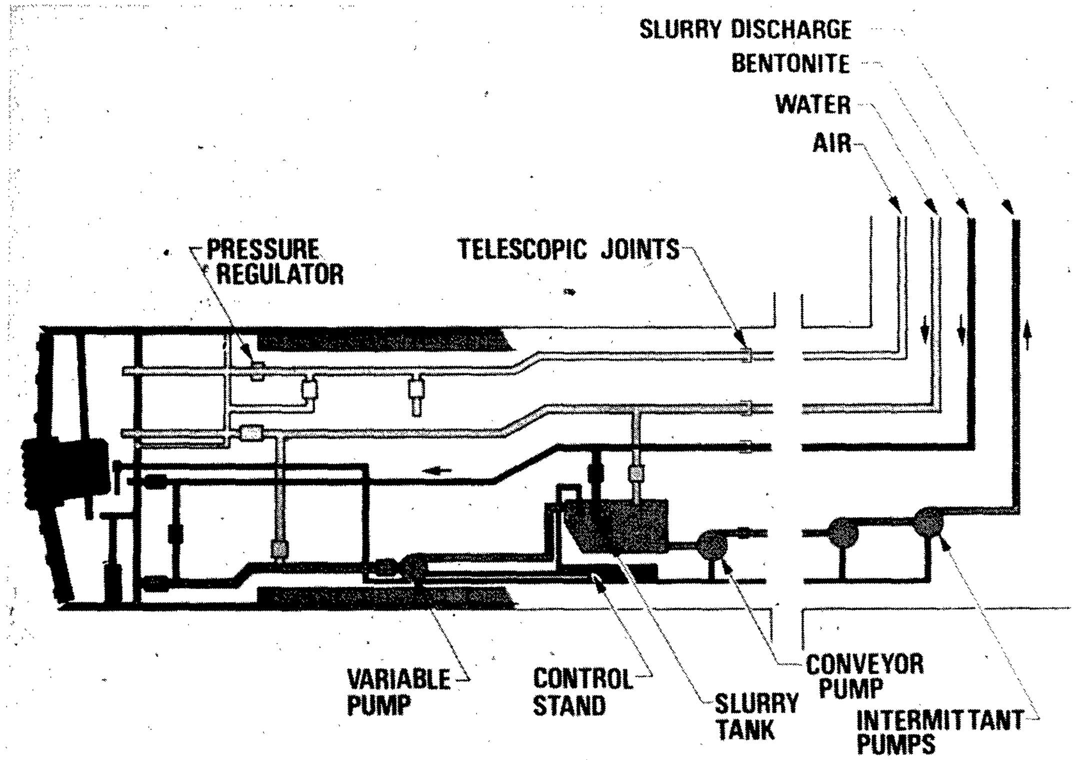
FIGURE 5

Telescoping Double Shield Rotary Head Machine



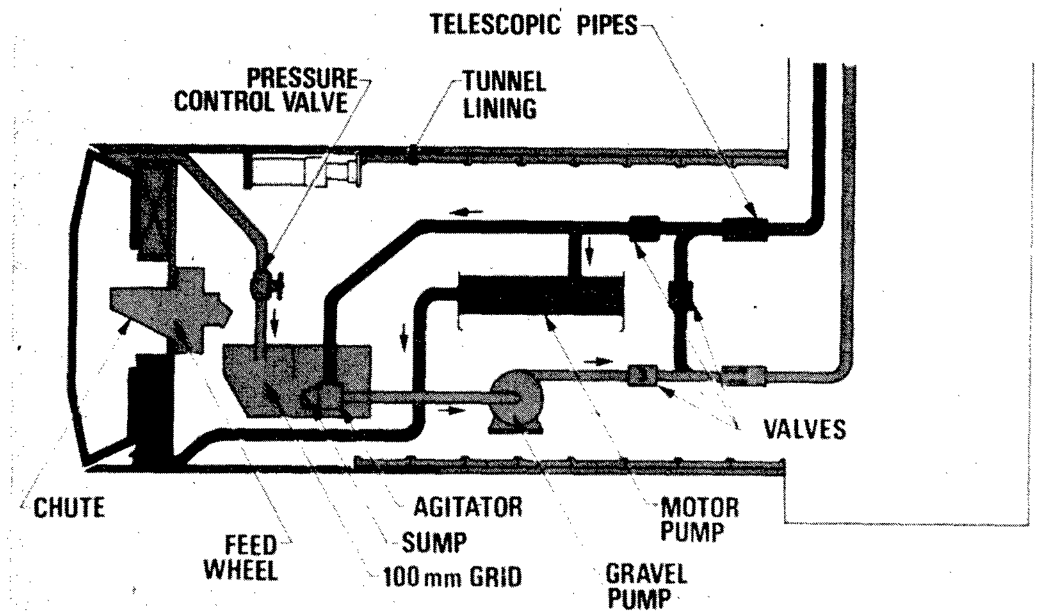
Pressure Bulkhead Shield

FIGURE 6



Hydroschild by Wayss & Freytag

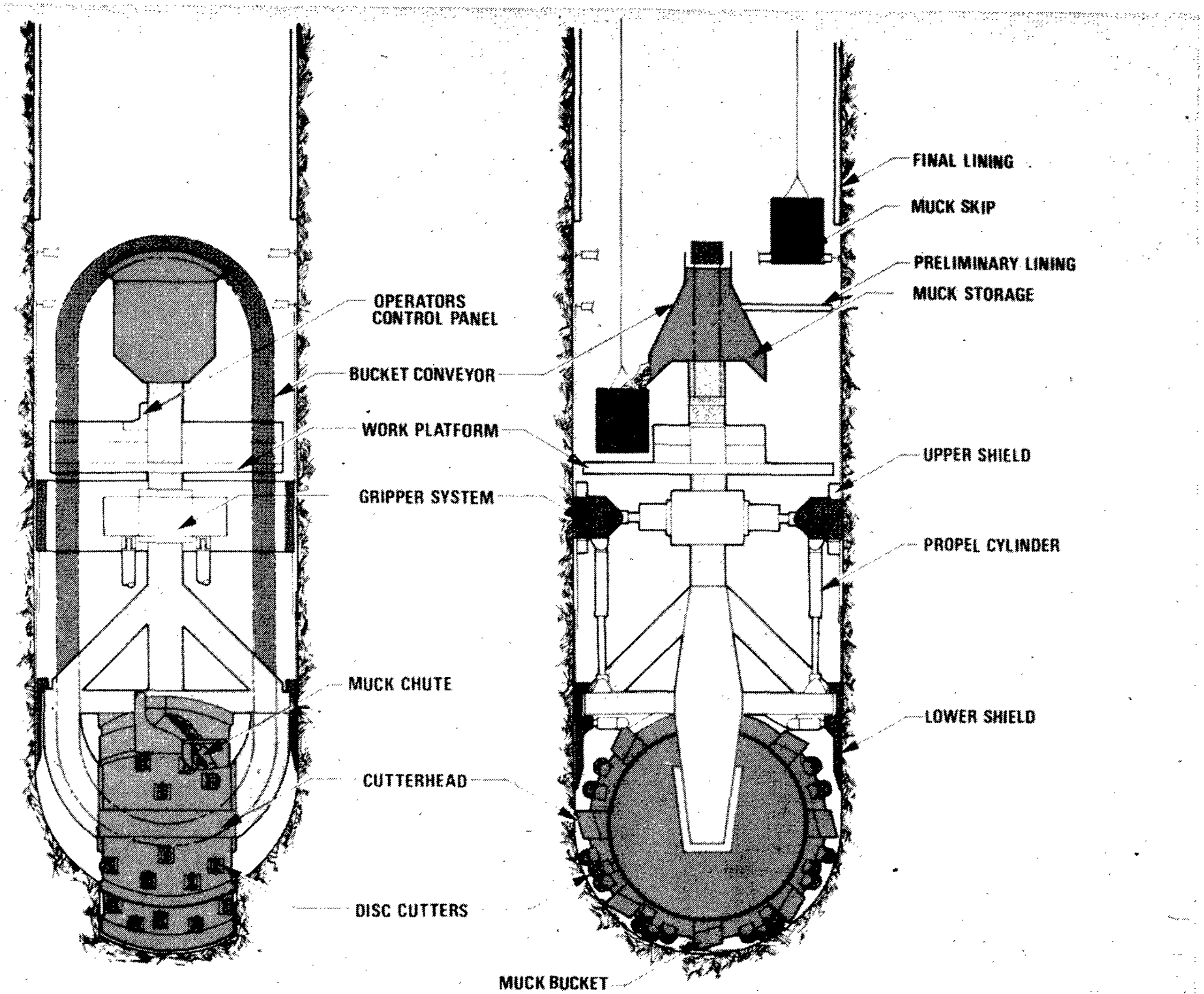
FIGURE 7



Bentonite Shield by Edmund Nuttall Ltd.

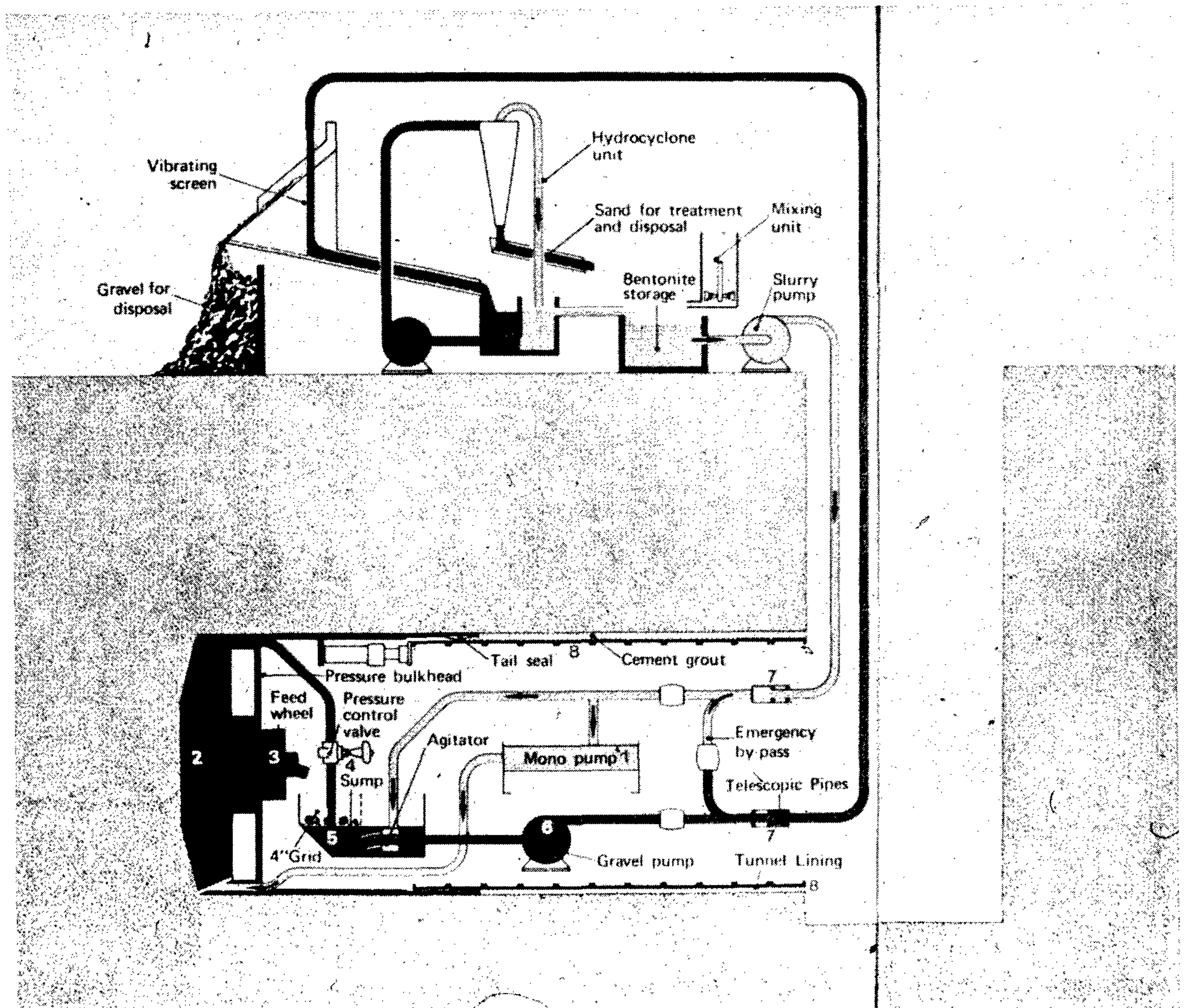
FIGURE 8

Figures 7 & 8 derived in part from International Construction, August 1974, Vol. 13



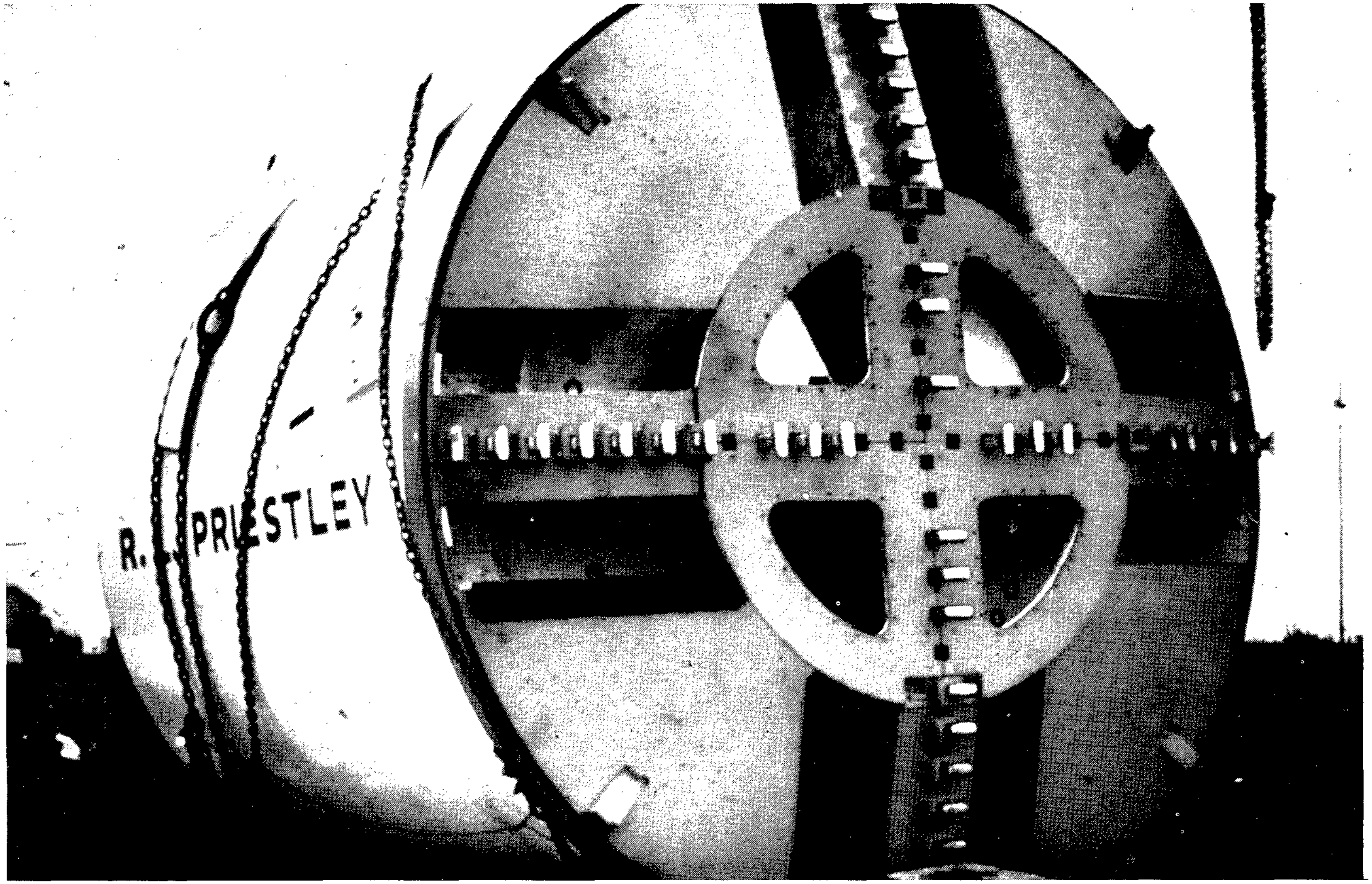
Mechanical Blind Shaft Boring Machine

FIGURE 9



Nuttall - Priestley Bentonite Shield

FIGURE 10



Nuttall - Priestley Bentonite Shield

FIGURE 11

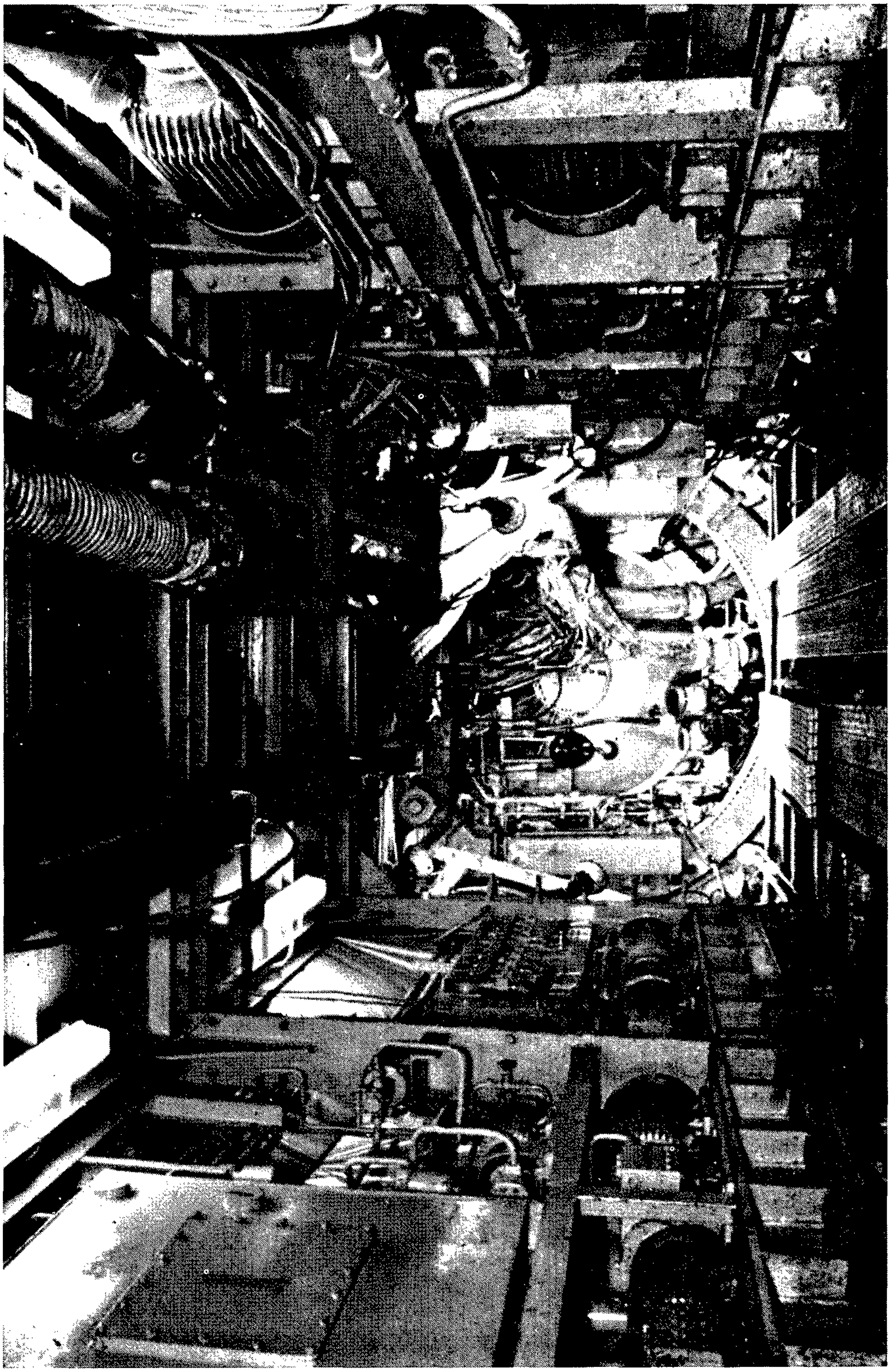


FIGURE 12

Tekken Kensetsu Slurry Face Machine

PAPER 12

Precast Concrete Tunnel Liners

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PAPER 12

PRECAST CONCRETE TUNNEL LINERS

by

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INTRODUCTION

With several transit systems currently in various stages of development in the United States, and with the increasing desire to tunnel a greater proportion of the underground structure, a reassessment of the entire tunneling process is very appropriate at this time.

One of the major components of a tunnel is the lining which, because of the particular geologic and other physical conditions imposed upon transit tunnel construction, generally must consist of prefabricated segmented rings. In the past, these have been composed of metallic segments, fabricated in steel or in cast or nodular iron. Rapidly escalating costs of these materials has led to the substitution of precast concrete for rapid transit tunnel linings by most countries outside the United States. Similar high costs of metallic tunnel linings in this country initiated the DOT/UMTA sponsorship of this Systems Study of Precast Concrete Tunnel Liners.

The principal objective of the study was to review the applicability and economics of precast concrete segmented linings to transit tunnels in the United States and to recommend the suitable lining system, for the conditions in this country.

To provide an overview of the study, certain material has been extracted from the report and attached herewith. The material is presented in the following pages.

SUMMARY AND CONCLUSIONS

This study develops tunnel lining systems in precast transit construction. In the course of the study, existing precast concrete systems designed or constructed in Europe, Japan, and the United States are evaluated. Using these as a point of departure, designs for lining systems applicable for the specific conditions encountered to date in the United States are developed. An economic analysis is performed and recommendations for testing and implementations are made.

The work plan for the study was as follows:

1. Define the basic performance requirements of a tunnel lining for rapid transit.

2. Review existing tunnel lining systems; select those which are appropriate to the objectives of the study; and analyze their special or unique features.

3. Formulate design criteria and develop designs and details of tunnel linings suitable for use in the United States, and perform an economic analysis.

4. Recommend specific testing programs which should be undertaken to verify the developed systems and their components.

5. Discuss and recommend methods of implementing the systems.

Summaries of the individual sections follow.

1. The Fundamental Requirements of a Tunnel Lining

The two sets of service conditions required of a tunnel lining are: (1) the permanent service conditions which include structural integrity, watertightness, durability, and the inside surface of the tunnel; and (2) the temporary service conditions which include the segment size, segment fastenings, erection stresses, transportation, and erection.

The items in each set of conditions are discussed and related to precast concrete lining design.

2. Existing Tunnel Lining Systems

Systems developed in the United States, Japan, Europe, and the Soviet Union are displayed and features described. In particular, the details of water sealant systems and fastening methods are discussed.

3. Systems Development in Study

Using a review of Section 2 as background material, the basic assumptions for development of the lining systems are outlined. The assumptions include both the type and condition of ground which may be encountered and the tunneling construction methods and equipment which may be used.

The three lining systems developed are: (1) Composite, which incorporates a steel perimeter angle; (2) Bolt Pocket, which uses steel face plates anchored in the concrete for transmitting the bolt forces; and (3) Through Bolt, where bolts passing through the segments tie the segments and the rings together continuously.

The proposed water seal system is described. All systems employ high-strength bolts and have identical water sealing systems. Comparative cost estimates are made for the three systems developed, as well as for one concrete system, previously designed, and two of fabricated steel. Costs are itemized for the principal components of the linings and then summarized as lining costs per linear foot of tunnel, if manufactured in three geographic areas. A comparative evaluation of the component itemized costs is made.

4. Engineering Design of the Lining

The several aspects which determine the criteria for the design are discussed. These aspects include the interaction between the lining and the surrounding ground, both firm and soft. Two recommended ground groups for the design are described and the loading criteria for firm ground and soft ground established. The salient aspects of the analysis and design of the systems developed are discussed. Reference is made to the design drawings contained in Appendix A.

5. Concrete

Three types of concrete for tunnel linings are discussed with respect to two desirable quality categories. These are (1) structural, which includes high-strength, low-modulus of elasticity, and low shrinkage; and (2) durability, including resistance to chemical attack and fire.

The three concretes are normal aggregate, lightweight aggregate, and polymer-impregnated. A general description of their compositions is given and a comparison of their properties and qualities made. Lightweight aggregate concrete is recommended for the designs in firm ground.

6. Water Sealing Systems and Tests

Water sealing systems used in segmented linings are described. The two systems in current use are caulking of the lining segment perimeter from within the tunnel, and application of a gasket to the segment faces before its erection in the lining. The materials used in each system are evaluated. A sealant system for the linings developed in the study is proposed and procedures for testing it are suggested.

7. Implementation of a Standard Lining System

Three entities are interested in the functional and economic aspects provided by a standardized precast concrete tunnel lining design. The entities are: implementors and engineering consultants of rapid transit systems; precast concrete manufacturers; and tunneling contractors. Their specific interests are discussed and promotional approaches to each are outlined.

8. Conclusions

The study has determined the fundamental requirements for tunnel linings in rapid transit construction. After a review of precast concrete linings which have been developed or used in this country and abroad, it can be concluded that precast concrete is eminently suitable as a lining for transit tunnels in the United States.

Three applicable bolted lining systems were designed and the cost of their manufacture was estimated. The cost of each system was very sensitive to the amount of structural steel incorporated in the design.

A comparative cost analysis between the linings designed in the study and one existing concrete design, and two in fabricated steel, revealed appreciably lower costs for all the concrete linings against those in fabricated steel. Also, the most economical study design lining, Through Bolt, appeared considerably lower in cost than the existing Caracas design. The analysis indicates that the Through Bolt rings may be manufactured for about one-third of the cost of fabricated steel rings, e.g., for firm ground conditions, see Table 1. Providing that the cost of installing the linings in the two materials would be approximately the same (and this appears likely), the resulting lowered cost of concrete lined tunnels may, in a number of situations, demonstrate favorable tunnel construction versus that in cut-and-cover. The study recommends that consideration be given to adoption of the Through Bolt design as a standard.

The study disclosed that the joint sealing system is a most critical detail in precast concrete tunnel linings. A system was developed during the course of the study which showed considerable promise of performing successfully in bad water situations. It is recommended that the system be laboratory tested.

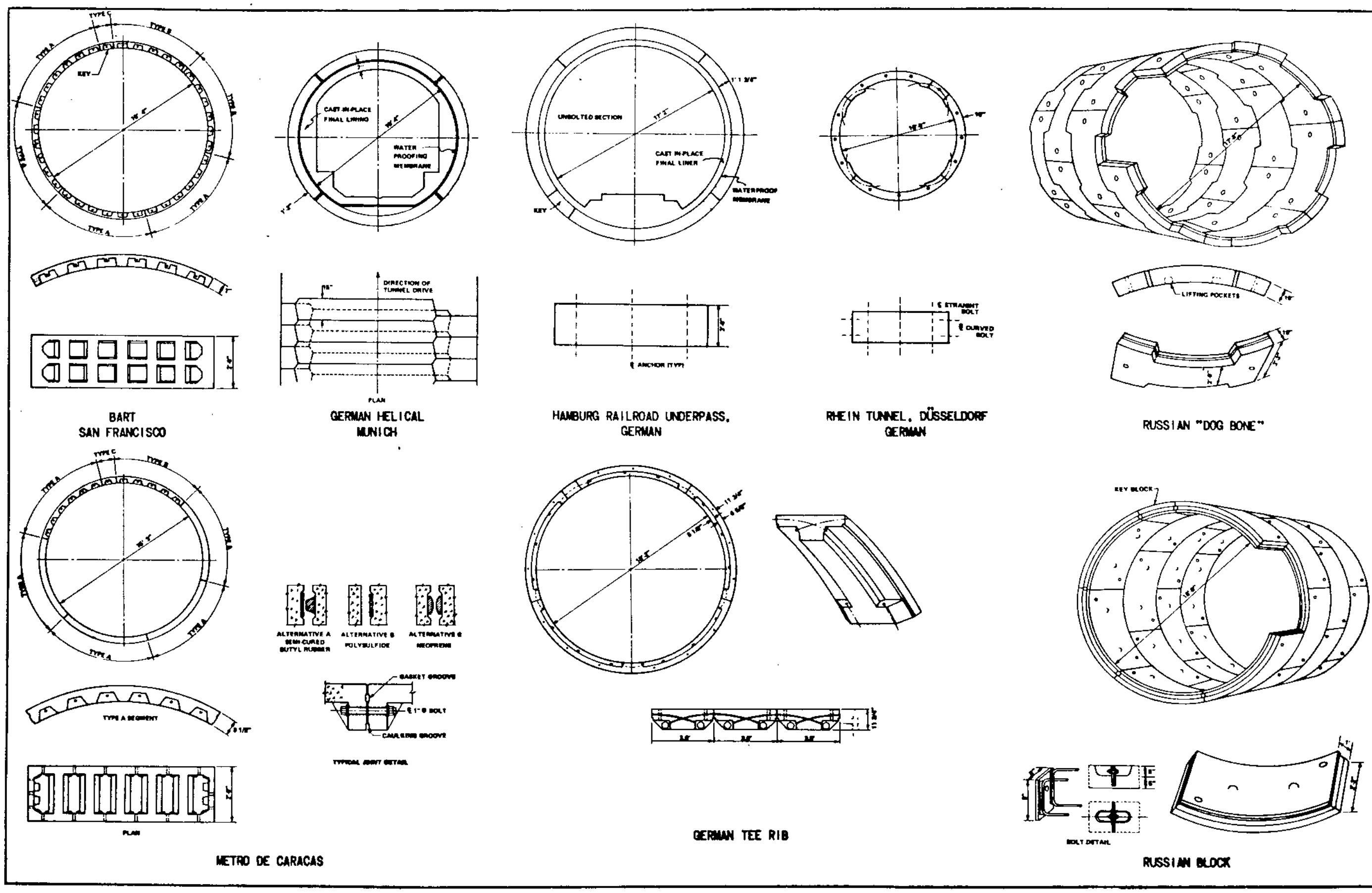
EXISTING TUNNEL LINING SYSTEMS

Introduction

In order to ascertain the present state of development in precast concrete lining systems, a review of all available literature was undertaken. This was supplemented by a visit of the author to England, where a number of pertinent tunnel projects for the London Transport Executive (L.T.E.) were inspected. Discussions were held with the responsible engineers and contractors. A Bechtel representative on special assignment to investigate tunnel construction procedures and systems in Japan provided useful information concerning some of the precast concrete tunnel systems being used in that country.

In the United States there was no tunneling work using precast concrete linings under construction. However, a visit was made to a tunnel project using steel segments in Staten Island, New York, where a sealant system was inspected which would also be applicable to precast concrete segments.

As a result of these investigations, representative systems from several different countries have been displayed in Figures 1, 2, and 3.



Existing Tunnel Lining Systems

FIGURE 1

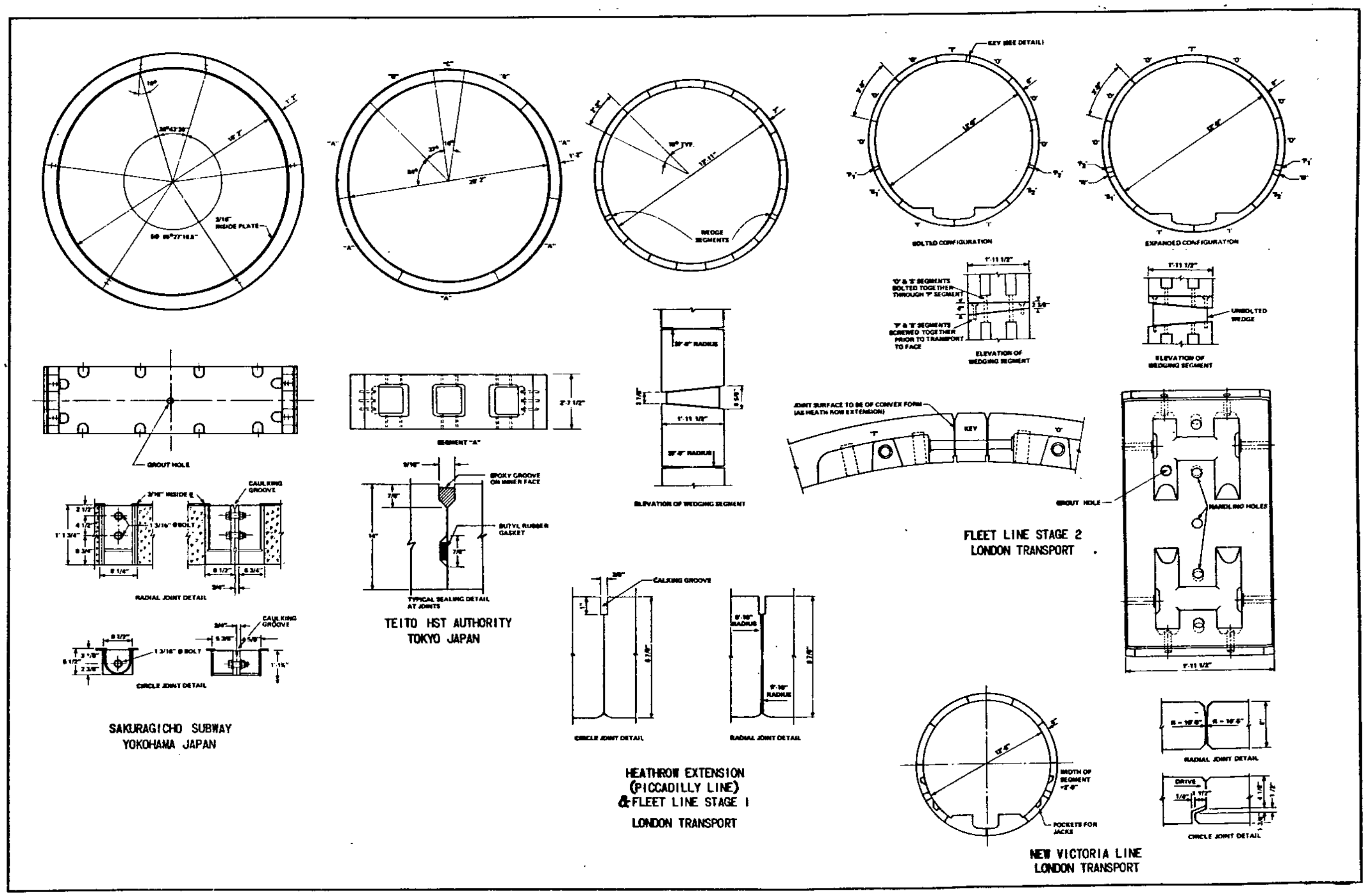
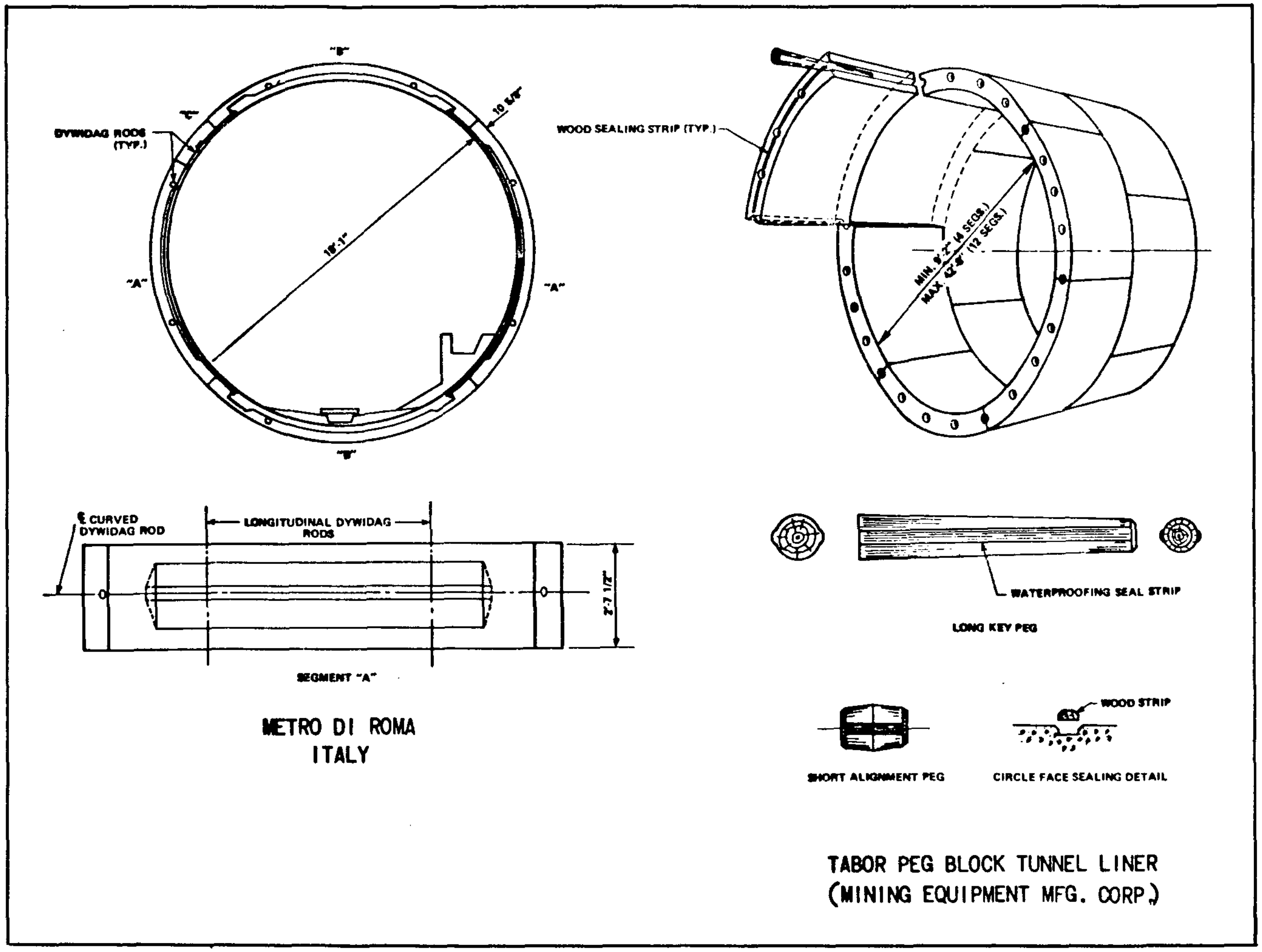


FIGURE 2

Existing Tunnel Lining Systems



Existing Tunnel Lining Systems

FIGURE 3

A data schedule listing the features of interest of additional tunnels lined in precast concrete which have been constructed in Japan and Europe is included in Appendix B of the detailed study.

SYSTEMS DEVELOPED IN STUDY

1. Basic Assumptions

The basic assumptions upon which the lining systems in this study were based are as follows:

a. In the United States, most cities with fairly immediate plans for rapid transit tunneling in soft ground (e.g., Baltimore; Washington, D.C.; and Chicago) have a mix of ground conditions. Accordingly, for the purpose of developing design criteria, two ground groups, or types, were assumed. The first, which would probably represent the majority of the ground encountered, comprises firm soils with a water table 10 ft below the ground surface. The second group comprises soft plastic, or squeezing soils.

b. It was assumed that the tunnels in both groups of ground would be constructed using shields, either mechanized or manual. As the width of a ring and the number of segments are subject to a number of variables, including a contractor's preference, it was decided not to recommend a standard segment size. Instead, a lining system would be developed in which the segment size could be varied without appreciably modifying the design or details.

2. Systems Developed

Predicated upon the above two assumptions, three systems were developed for two ground groups. The systems developed were: (1) Composite, incorporating a steel perimeter angle. Pockets are provided in the concrete for the installation of high-strength bolts which connect the segment through the steel agles. (2) Bolt Pocket, which uses steel face plates anchored in the concrete for transmitting the bolt forces. (3) Through Bolt, where bolts passing through the segments tie the segments and the rings together in a continuous fashion. The two design ground groups were (1) firm soils with high groundwater tables, and (2) soft or plastic soils. In all the designs, a six-segmented ring 3 ft wide was adopted for the design base. Comparative cost estimates for these designs, as well as for a ring width of 4 ft, have been provided.

As discussed in the foregoing sections, the inherent drawbacks of bolting through the flanges of a channel cross section led to the development of segments of constant cross section.

All the systems in firm ground and in plastic ground are similar in concept; the segments are essentially solid in cross section; are bolted together along their four faces; and have identical waterproofing details.

3. Features of the Systems

A discussion of the principal features of the three systems is included below.

Composite System (See Figure 4)

This system comprises a steel angle frame with a reinforced concrete in-filling and shear connections to provide composite action. The 7 x 4 x 5/8-in. angle which frames the four faces of the segment is provided with 1/2 in. -diameter welded studs which serve both as shear connectors and tension anchors. Three-quarter-inch diameter high-strength bolts are used to connect the segments. The bolt pockets are sized to permit the use of a standard direct drive impact wrench.

A 1/4- x 2-in. steel plate welded to the circle faces is machined to 1/8-in. thickness to a specified tolerance (0-1/32-in. is suggested). The purpose of this plate is to locate the high-jacking forces on the transverse center of the segment, thus avoiding the possibility of spalling the edges of the concrete behind the facing angles.

Essentially, the system provides the advantages of a metallic segmented system, in that steel to steel is bolted tightly together. If adopted as a standard, the system would greatly simplify the concreting aspects of the segment manufacture, both in the speed of final production and in maintaining the required tolerances.

Steel Pocket System (See Figure 5)

As in the case of the composite system, the steel bolting faces permit the use of short high-strength bolts to ensure tight concrete interfaces and competent structural joints. The bolt pockets indicated are shorter than those provided in the composite system and would require the use of right angle drive torque wrenches, of which there are several suitable types, either presently on the market or under development.

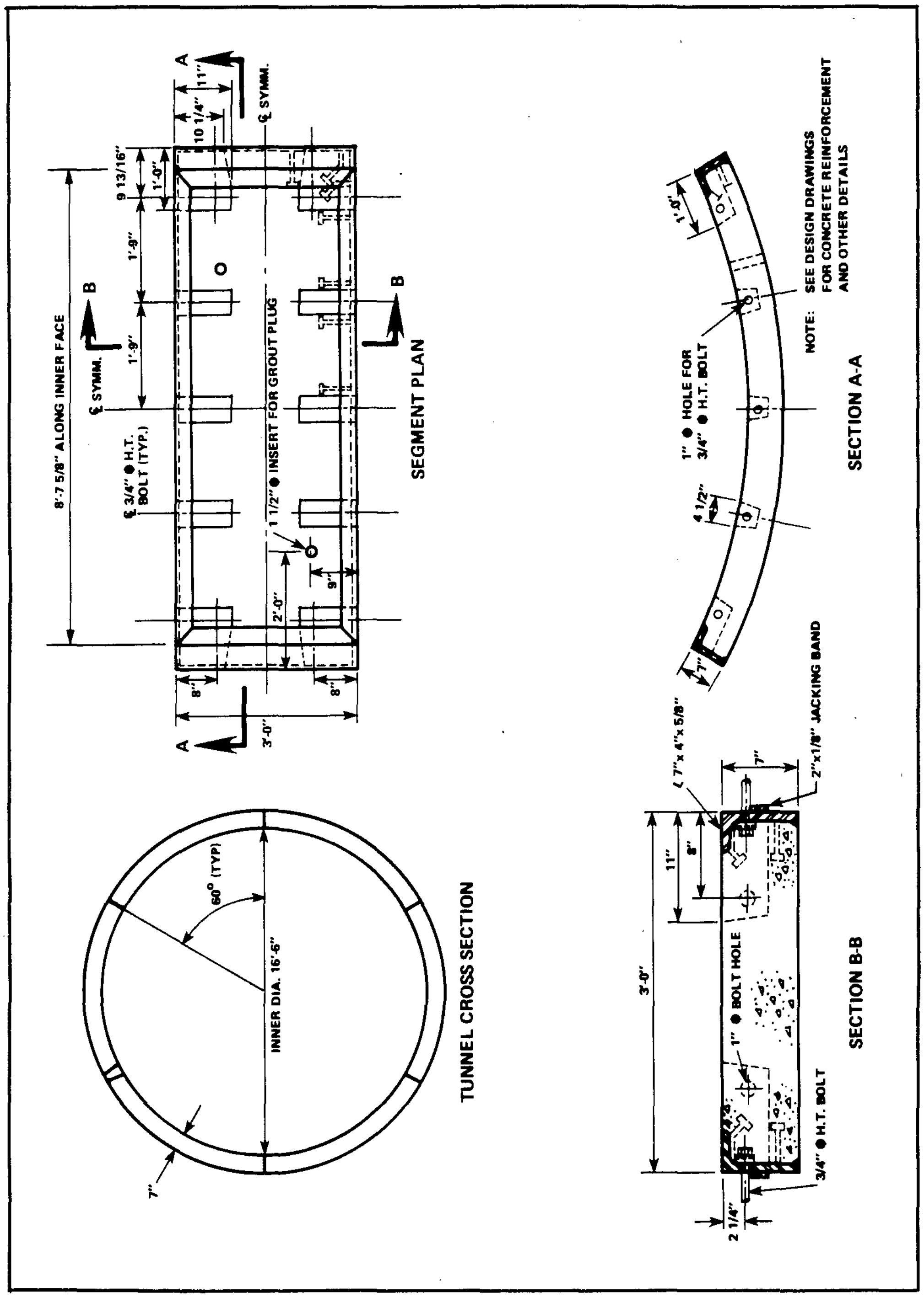
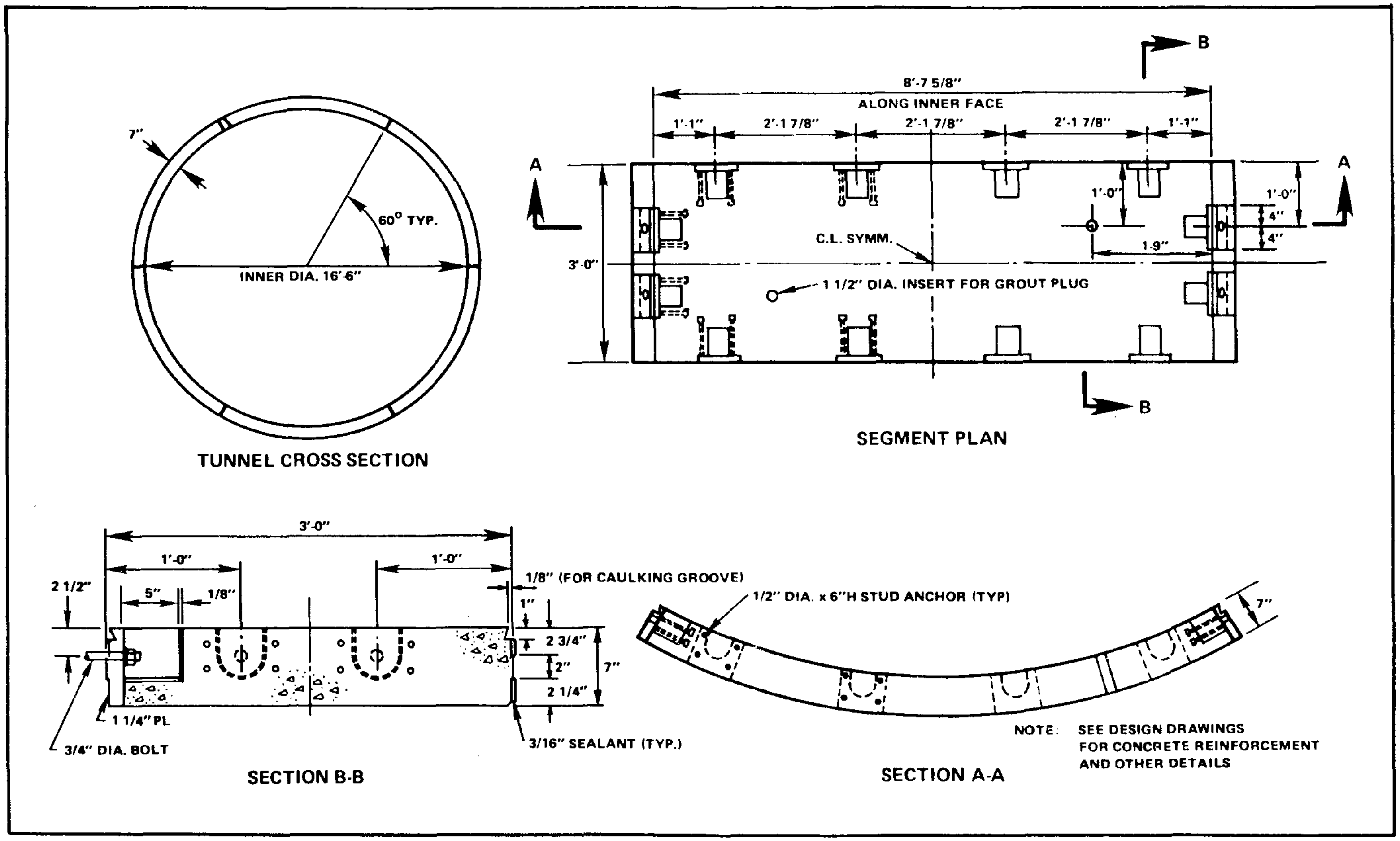


FIGURE 4

Composite System



Steel Pocket System

FIGURE 5

The 1/8-in. x 2-in. raised band on the circle face aids in attaining specified tolerances of the circle faces as well as centering the jacking forces on the segment cross section.

Through Bolt System (See Figure 6)

This concept differs from that of the steel pocket in the fastening details. The segment rings are fastened together by post-tensioned bolts, located centrally in the thickness of the segment and anchored in the segments of the adjacent rings. The radial joints are provided with 3/4-in.-diameter high-strength (H.S.) capping bolts.

The through bolts are indicated in Figure 7 as typical solid rock bolts, but the alternative hammer head bolt would provide a more economical and direct anchorage. For convenience, the bolts would be placed in the segment - not projecting from either face - before transportation to the tunnel heading. The erection procedure would consist of placing the segment in position in the ring, pushing the bolt and anchorage back in the bolt pocket of the previously erected ring, torquing either the bolt or the nut according to the system used, to obtain the specified bolt tension. This sequence is established rock bolting procedure and can be performed with one wrench socket actuated by a direct drive impact wrench. About 9 in. would be required between the segment and the jack shoe for the operation. Tightening of the circle bolts and the radial joint cap bolts may be undertaken concurrently.

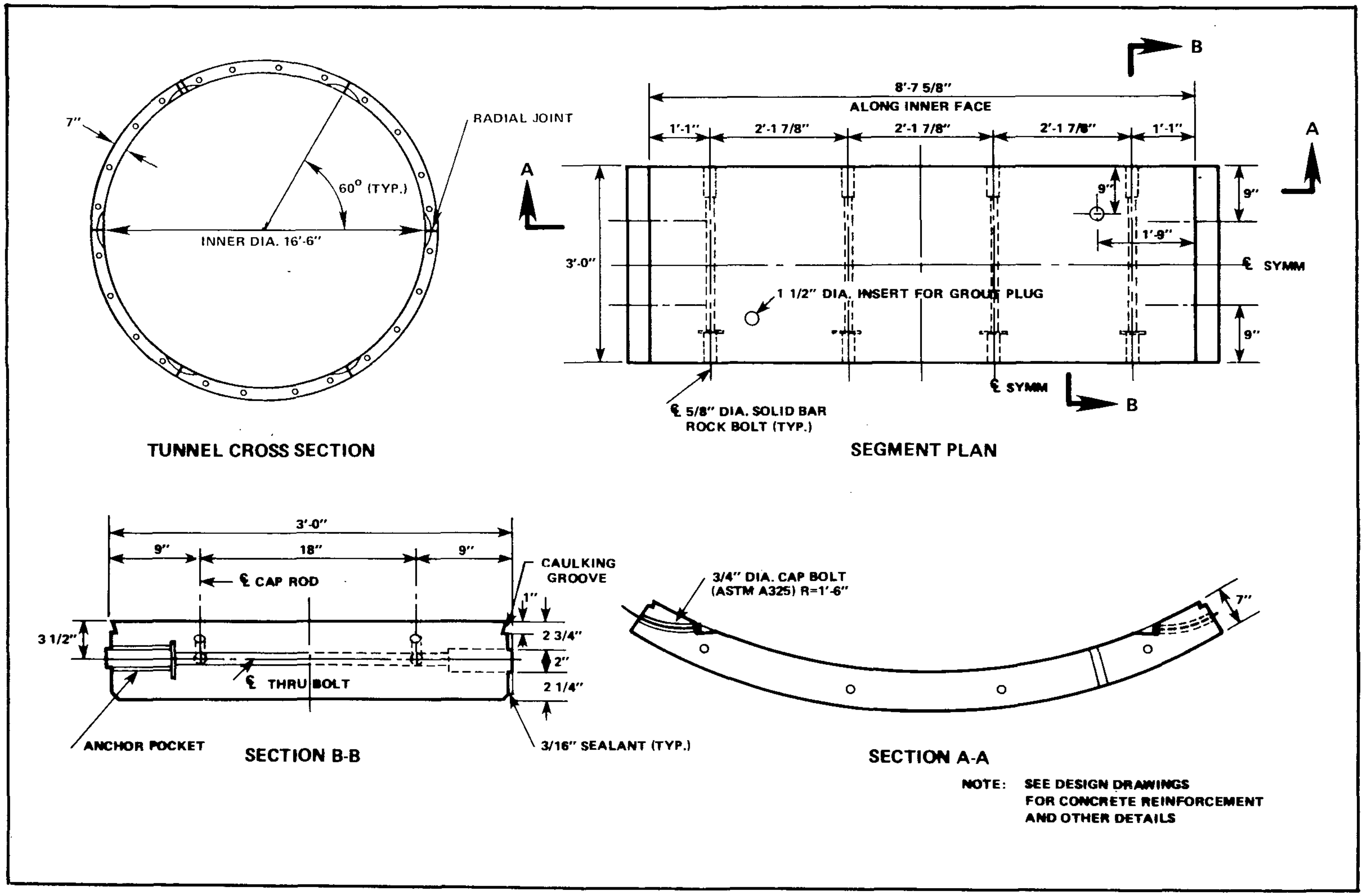
All of the Through Bolts would be grouted for corrosion protection; the 3/4 in.-cap bolts could also be grouted at the engineer's option.

Water Seal System (See Figure 8)

The three lining systems are provided with identical primary gasket seals and secondary caulking seals.

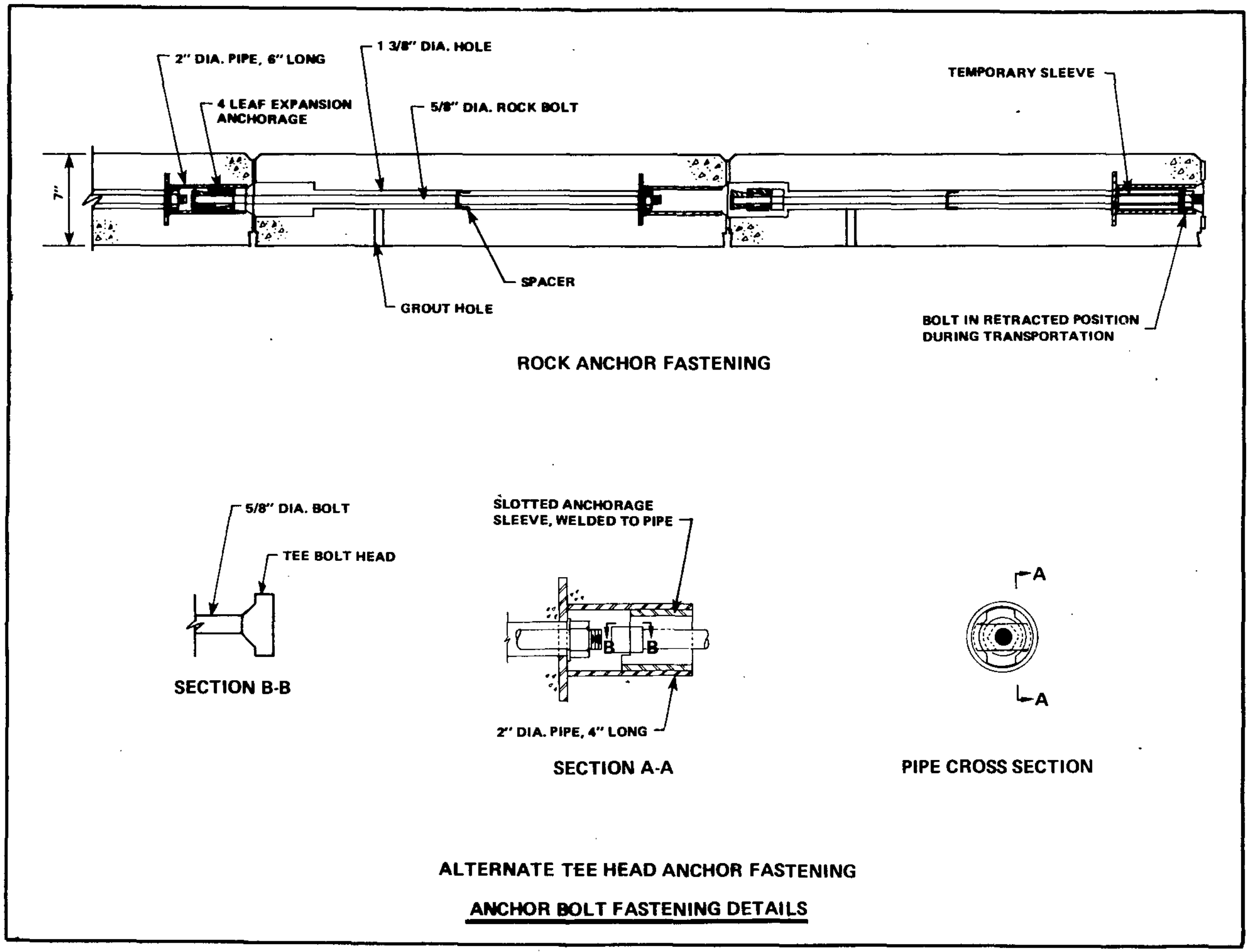
The primary gasket in the circle faces consists of two 3/16-in. -thick bands of butyl rubber applied to the faces on either side of the raised central jacking band. As the segments are placed, the circle bolts and shield shove jacks bring the faces together, concrete-to-concrete, at the jacking bands, compressing sealants to 60 percent of their original thickness, and thereby, effecting a positive water seal.

On the radial faces, the primary gasket material would be applied 1/16-in. thickness across the full face of each



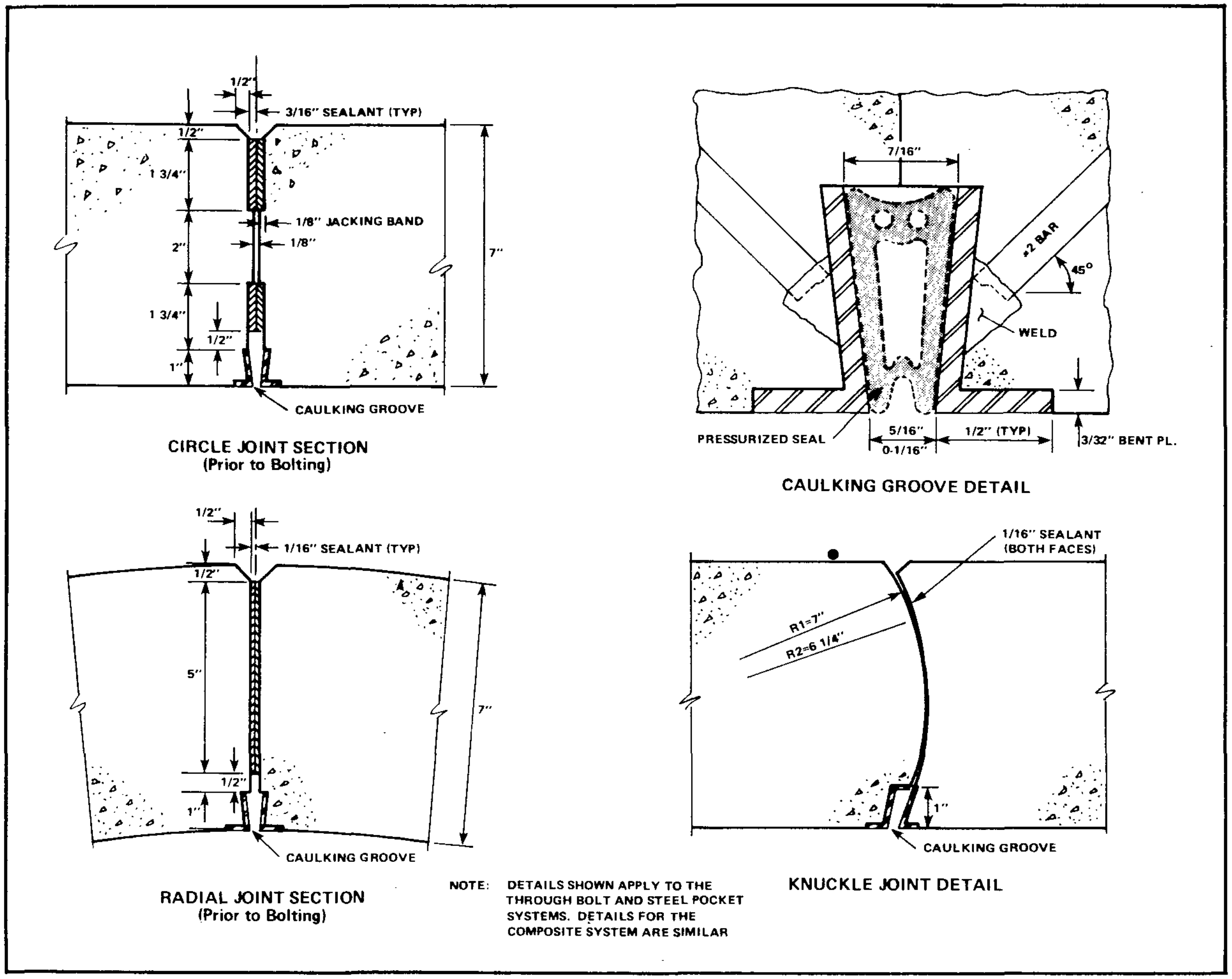
Through Bolt System

FIGURE 6



Typical Anchor Bolts

FIGURE 7



Sealant System and Knuckle Joint Detail

FIGURE 8

segment. It is anticipated that the tightening of the radial bolts and the ground loading will compress these sealants about 50 percent of their combined original thickness. That is, the final sealant thickness of the two bands at the interface would average 1/16 in.

Due to the distortion of the lining, the circumferential bending moments will cause a varying compression across the joint, and a corresponding variation in the thickness of the gasket. It is also anticipated that due to the flow of the sealant under compression, the inherent voids which are present at the intersection of the segment corners and which always are a source of leakage, will be filled satisfactorily.

The secondary caulking seal is provided as a backup system in the event of failure of the primary seals. This seal is in the nature of a caulking groove provided along the four inside edges of the segments.

The caulking seal presently proposed would be in the form of an inflatable tube of "O" ring which, after being inserted in the grooves, would be pressurized by injecting a liquid filler. This would solidify after placement, thereby maintaining permanent expansion of the seals. While both the proposed primary and secondary sealant systems are simple in concept - and a certain amount of experience already has been gained with the primary one - it is deemed advisable to verify by testing the properties of the materials and the principles of their intended application. Such a test program, together with a more detailed discussion on sealant systems, is presented in Section 7 of the detailed study.

Alternate Radial Joints

Given a sealant system which would provide satisfactory watertightness when movement occurs in the segment joints, the fastenings between the segments could be reduced, certainly for the firm ground condition, to the minimum required for that of the erection of the lining. This would permit the elimination of the radial joint bolts altogether and allow articulation of the radial joint, thereby reducing the cost of both segment manufacture and ring erection. A detail for a proposed articulated "knuckle" joint and the related water barrier system is shown in Figure 4-6.

COMPARATIVE COST ESTIMATES

Cost estimates were prepared for the study lining designs, and for comparative purposes, linings for the designs of Caracas and Washington Metropolitan Area Transit Authority (WMATA) were also estimated. The manufacture of 10,000 ft of lining at the rate of fifty rings a week was used as a basis for the estimates.

The costs for the main components of the concrete linings are summarized for firm ground conditions in Table 1 and for soft ground conditions in Table 2.

The total cost in dollars per linear foot of tunnel was first compiled on a San Francisco base and then modified to reflect the differences of labor, materials, and productivity in three other geographic areas. The study lining types were estimated both for 3- and 4-ft wide rings.

The costs indicated for the fabricated steel linings were based on the costs applying to linings bid for delivery to the WMATA Project, 1st Quarter 1975. That for the metallic (cast iron) was the low bid price received by MTA Baltimore in May 1975. The cast iron low bid was slightly lower than the next lowest for fabricated steel. The price for the metallic linings would (probably) be based on costs escalated to the mid-point of the delivery schedule which would be about the third quarter in 1976. Table 3, Inflation Analysis Cost Trends, illustrates these cost differences.

Table 1

ESTIMATED MANUFACTURE COSTS, $/FOOT RUN TUNNEL – FIRM GROUND CONDITIONS

PRICE ITEM	LINING TYPE										
	PRECAST CONCRETE								FABRICATED STEEL		METALIC
	CARACAS		COMPOSITE		BOLT POCKET & CAP RODS		THROUGH BOLT & CAP RODS		KAISER STEEL	COMMERCIAL SHEARING INC.	M.T.A. BALTIMORE
	3'		3'	4'	3'	4'	3'	4'	2'-6	4'	4'
CONCRETE	43		42	42	42	42	42	42			
REINFORCEMENT	135		58	58	68	68	52	52			
MISC. STEEL & BOLTS	26		568	486	313	273	146	121			
LABOR	187		102	92	104	94	100	90			
OVERHEADS PLANT MARK-UP	70		48	45	48	45	48	45			
SUBTOTAL	461		818	723	575	522	388	350			
CONTINGENCIES	89		42	37	55	48	42	40			
GRAND TOTAL SAN FRANCISCO BASE	550		860	760	630	570	430	390			
LOS ANGELES CA.	476		823	724	590	531	395	356			
CHICAGO IL.	569		872	772	643	582	442	402			
BALTIMORE MD.	568		872	772	643	503	443	403	1100	950	1250*

COSTS 1ST QUARTER 1975

*BID RECIEVED MAY, 1975

Table 2

ESTIMATED MANUFACTURE COSTS, $/FOOT RUN TUNNEL – SOFT GROUND CONDITIONS

PRICE ITEM	LINING TYPE									
	PRECAST CONCRETE								FABRICATED STEEL	
	CARACAS		COMPOSITE		BOLT POCKET & CAP RODS		THROUGH BOLT & CAP RODS		KAISER STEEL	COMMERCIAL SHEARING INC.
	3′		3′	4′	3′	4′	3′	4′	2′-6	4′
CONCRETE					49	49	49	49		
REINFORCEMENT					114	114	114	114		
MISC. STEEL & BOLTS					229	190	152	126		
LABOR					115	105	115	105		
OVERHEADS PLANT MARK-UP					48	45	48	45		
SUBTOTAL	NOT ESTIMATED				555	503	478	439		
CONTINGENCIES					55	47	52	41		
GRAND TOTAL SAN FRANCISCO BASE					610	550	530	480		
LOS ANGELES CA.					554	495	474	425		
CHICAGO IL.					626	565	546	495		
BALTIMORE MD.					625	563	545	493	1600*	1400*

COSTS 1ST QUARTER 1975

* PRORATED FROM BART LINING COSTS FOR FIRM GROUND AND SOFT GROUND.

Table 3

INFLATION ANALYSIS COST TRENDS
THROUGH BOLT AND FABRICATED STEEL (4-FT WIDE BASIS)

	Base Cost	Annual Inflated Costs				
	1975	1976	1977	1978	1979	1980
P.C. Through Bolt (Cost per foot)	403	439	476	512	548	584
Fabricated Steel (Cost per foot	1100	1198	1304	1408	1510	1615

1980 Increase Over Base Year		Annual Average (÷5)
P.C. Through Bolt	$45^{\underline{4}}$ %	$9^{\underline{08}}$ %
Fabricated Steel	$46^{\underline{8}}$ %	$9^{\underline{38}}$ %

PAPER 13

Situations in Underground Construction Where Braced Excavations Are Appropriate

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PAPER 13

SITUATIONS IN UNDERGROUND CONSTRUCTION WHERE BRACED EXCAVATIONS ARE APPROPRIATE

by

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St. Louis, Missouri

INTRODUCTION

For purposes of this paper we will define cut-and-cover tunneling as the process of installing a permanent structure below ground by excavating an area of sufficient width constructing the permanent structure at the bottom of the excavation, and then covering the structure with soil. The excavation may be left open during construction or temporary decking may be installed to permit life and traffic to continue at the surface while excavation and construction proceed beneath the deck.

In some cases an open excavation could have sloped sides with no ground support whatsoever. However, most situations have limiting right-of-way requirements necessitating some sort of system of ground support with vertical walls. These are the conditions to be discussed in this paper and, since other systems will be described in detail in other papers we will confine our remarks to internally braced ground-wall support systems, or linear braced cofferdams.

In urban areas the limitations of available space and minimization of disruption to the community during construction dictate some sort of ground support system for cut-and-cover tunnel construction. In the following sections we will discuss some of the various types of ground support systems in use with particular emphasis on sheet pile walls and soldier beam and lagging systems.

GROUND SUPPORT SYSTEMS

1. Sheet Pile Walls

This ground-wall system, which is most suited to soft cohesive soils, consists of interlocking sheet piling driven deep enough to provide a ground-water cutoff and to prevent bottom failure. Since this is a continuous wall, all utilities in the immediate vicinity of the wall must be accommodated by shifting or relocation prior to driving of the sheets.

The low section modulus and flexibility of sheet piling requires support by closely spaced wales. These, together with preloaded interior struts, must be installed as the excavation is progressed down.

2. Secant Caisson Walls

Secant caissons are intersecting circular cast-in-place concrete piles about three feet in diameter forming a supposedly watertight wall.

Holes for alternate piles are drilled first, and concrete having a controlled setting time is placed. While this concrete is still green, holes for the piles in between are drilled thus cutting a notch into the previously cast piles.

Flight augers are generally used for hole excavation in cohesive soils without casing. In sandy soils, holes must be supported by either casing or slurry to prevent cave-ins. In usual practice every other caisson is reinforced with a bar cage.

Secant caissons are suitable for most types of soil; the cutting edge of the casing will go through brick, rubble and small boulders. The system is relatively quiet and clean, but requires excellent maintenance and skilled operators.

Ground loss during installation is minimal and structural rigidity is high. Resulting ground movements will be small, making underpinning of minor adjacent buildings unnecessary.

Extensive relocation of utility connections is required. Not only must utilities be shifted out of the way of the first alternate caissons, but must be shifted back over them when they are cut off to allow intermediate caissons to be placed.

This method is extremely safe and has an impressive record of success in Europe.

3. Pipe Caissons and Grouting

This ground-wall system consists of steel sheet caissons, about four feet in diameter, positioned at about five to eight feet on centers with the soil between them thoroughly grouted; and the whole forming a ground-water cutoff wall with ground support through arching action of the grouted soil.

Caisson installation is similar to that for secant caissons, except that the casing is left in place and the caissons need not be immediately concreted. Indeed, they may be used as vertical grouting galleries.

Grouting may be accomplished from the surface or grout holes can be drilled from inside the caissons at about two feet vertical spacing in the direction of adjoining caissons, and grout injected into the soil. This has been the most successful application for this system.

Close, frequent, and verified grouting normally makes excavation possible without any lagging between caissons. Lagging, of course, is available as a back-up measure should some incompletely stabilized areas be encountered.

The favorable features of secant caissons are present in this system including installation, rigidity and underpinning. The available leeway in caisson positioning should permit clearance of most existing utility connections. Grouting and coring is an intricate time consuming operation, and it may prove to be impractical and expensive.

4. Soldier Piles and Lagging

Soldier piles with lagging is a time tested ground-wall system consisting of steel wide flange beams or steel caissons installed as vertical soldier piles at about five to eight feet centers with lagging supporting the space between. The lagging may be of any material capable of supporting the area between the piles which in turn support the lagging.

Soldier piles are usually installed by conventional methods such as pile-driving equipment or drilled in. Due to environmental objections, soldier piles used in urban areas are generally installed in predrilled holes today.

Ground loss can occur during the process of excavating for and installating lagging, especially in the more granular soils. Surface and groundwater leaking through the lagging will carry fine material and result in ground movements.

In the following sections soldier piles with lagging and sheet pile walls are discussed in some detail.

SOLDIER PILES WITH LAGGING

1. Loss of Ground and Movement Due to Excavation

Installing soldier piles in prebored holes presents additional dangers of ground loss through hole collapse. For instance, a fall-in of the bottom 20-ft of a 40-ft deep 3-ft diameter hole will cause an ultimate four inch settlement in a 20-ft square area.

Hole collapse can be prevented through the use of casing or bentonite slurry as support prior to installing soldier pile and lean concrete fill.

Precast concrete walls and tremie concrete walls are two other methods which are discussed in other papers. Two other systems worthy of mention here are freezing of the ground and use of air pressure to support excavation walls.

In every system the primary purpose of the "ground-wall," or cofferdam is to provide temporary support for the adjacent ground and prevent soil movements which can cause damage to nearby buildings, utilities or subsidence of pavements.

Basically these soil movements may result from either or both of two different causes, loss of ground through the wall or deflection of the wall system itself. A third cause of group failure which must be considered in the design of ground support systems in certain soil conditions is the phenomenon of "bottom heave."

The degree of control of loss of ground through the wall depends on the permeability of the wall system used and the workmanship used in installing the wall. Diaphragm walls are virtually impermeable at one end of the spectrum while soldier pile and lagging walls poorly installed can only be classified as pervious.

Deflections permitting ground movements are too often ignored in the design of excavation support systems. Sheet pile walls can be classified as flexible walls, soldier pile and lagging as semi-rigid and most forms of diaphragm walls as rigid. However, in even the most flexible wall, deflections can be controlled with sufficient support as illustrated in Figure 6. This is no longer a flexible system but it would severely constrict the work on the final structure.

Movements will occur in even the most rigid systems such as secant caisson walls and other forms of diaphragms. No matter how well designed and how well constructed, these soil movements will occur and with damaging results if not controlled. Deflections, and resulting soil movements can be minimized by careful selection of bracing or tie back spacing and installation of such braces promptly before excavation progresses too deep.

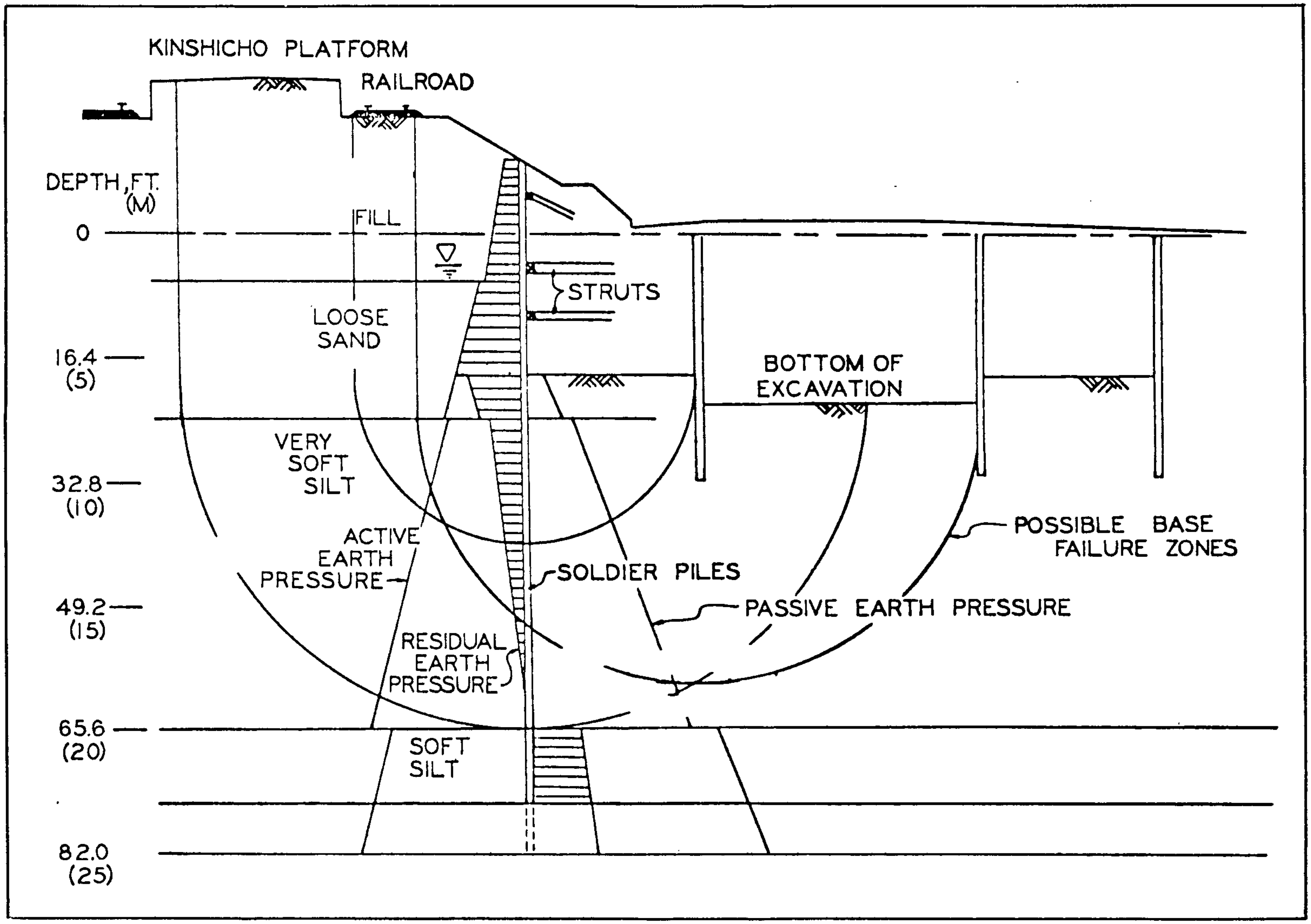
2. Importance of Bracing

In designing the bracing system, the designer must take into consideration the final structure. The wall deflection of the temporary cofferdam will not stop when the excavation has been completed. The wall movements will only stop when the final structure is in place, all the struts have been removed, the excavation has been back-filled and the area has been restored. Therefore, the designer of the bracing system has to take all these into consideration, including how to transfer the loads to the final structure when the brace is removed. Some of the ways of removing the braces are: building the final structure around each brace, transferring the load to the final structure, then cutting the brace and filling the void. Another way of doing it is moving the final structure sufficiently strong so that when the braces are removed the final structure can support the bracing load.

We in North America can generally consider ourselves fortunate because we generally have good soils with very few exceptions, such as San Francisco Bay mud. Considering such areas as Norway, Holland, coastal areas of Japan (Tokyo, for example) where they have soft organic clays or quick clays, considerable ground movements at the surface are caused by bottom heave. Some of the methods used to stabilize the bottom are quite extreme. In Norway, flooding the excavation was tried, then excavating through the water with a grab. In Holland, the tunnels were sunk and piles were driven to keep the tunnel in place or compressed air had to be used.

3. Base Instability

Base instability or bottom heave can be anticipated in soft materials such as soft organic clays. Figure 1 shows the earth pressures on temporary cofferdams and indicates possible failure zones. The weight of the soil outside the excavation moves downward, forcing the soil below the excavation to move laterally; this can be regarded as



LATERAL EARTH-PRESSURE DISTRIBUTION, Kinischicho Station. (By permission of the Sociedad Mexicana de Mecanica de Suelos, S.A.)

FIGURE 1

bottom heave and results in settlement of the adjacent area. This settlement can be minimized by getting the wall system to sufficient depth below the excavation to prevent these lateral movements.

Another type of bottom heave can result in stiff to hard cohesive material This heave is due to unloading, or stress relief of the material, and/or due to the resulting negative pore pressures in the material.

Another bottom heave can result where an impervious layer is below the base of the excavation and above an aquifer. Due to the excavation, a differential hydrostatic head may result. This difference in the hydrostatic head may lift the whole bottom of the excavation or, if a localized area, may result in piping. To stabilize this condition relief wells can be installed.

All of these possibilities of bottom failure must be considered in the design of the tunnel. Conditions causing bottom heave may result in the settlement of the final structure.

Where the bottom of the excavation is unstable or where hydrostatic pressures have not been anticipated nor considered in design, the result may be serious construction delays or possibly failure of the cofferdam.

4. Methods of Installation

Generally we think about lateral soil pressures in design of excavation support systems but we must remember that the wall will also support vertical loads such as utilities, temporary decking, and live loads and soil drag forces behind the wall. Therefore the wall tips must penetrate to sufficient depth to support all these loads. There have been cases when soldier piles were driven to refusal, but started to settle as the excavation progressed downwards. This is caused by decrease of the friction support of the piles and decrease in the effective over burden pressure due to excavation. However, a very simple solution is to increase the end area of the H-pile by backfilling with concrete.

Bracing methods employed must adapt to local subsurface conditions and to the presence of man-made obstacles, natural obstacles, and the final structure. Examples of man-made obstacles are utility lines, sewers, and tunnels. Their location and the procedures used for handling utilities demand special attention. Broken telephone cables or ruptured sewer pipes could cause costly disruptions in a construction project in addition to the resulting community inconvenience.

Figures 2 and 3 show the wall support system used for a portion of Toronto Subway. This basically consists of a soldier pile, in this case two channels welded together, timber lagging and the wedging system. Note the lean concrete used as backfill between the channels.

Lean concrete is frequently used as backfilling around the soldier piles because of the ease of backfill, the certainty that all voids have been filled, and the soldier pile is assured of being in contact with the natural material.

The use of I-or wide-flange beams driven as vertical soldier piles with timber lagging supporting the area between dates from constructing the U-Bahn in Berlin in 1893. The Berlin method, as the technique is often called, is the most common form of excavation support in use today.

Rough-hewn logs about 12 inches in diameter were originally used as lagging and were wedged between the beam flanges. Timber lagging may be either new or used, and while a variety of timber is used fir is a fairly common choice. Precast concrete, steel, and other type lagging have also been tried.

Horizontal members called wales are installed at selected levels to transfer the soldier beam loads to the more widely-spaced internal bracing.

Normally, holes are pre-bored with an auger. Soil conditions and groundwater level determine the installation technique, such as filling the hole with grout, allowing for the displacement caused by the soldier pile as it is lowered into the grouted hole and driven to the required depth. Figure 4 illustrates this procedure.

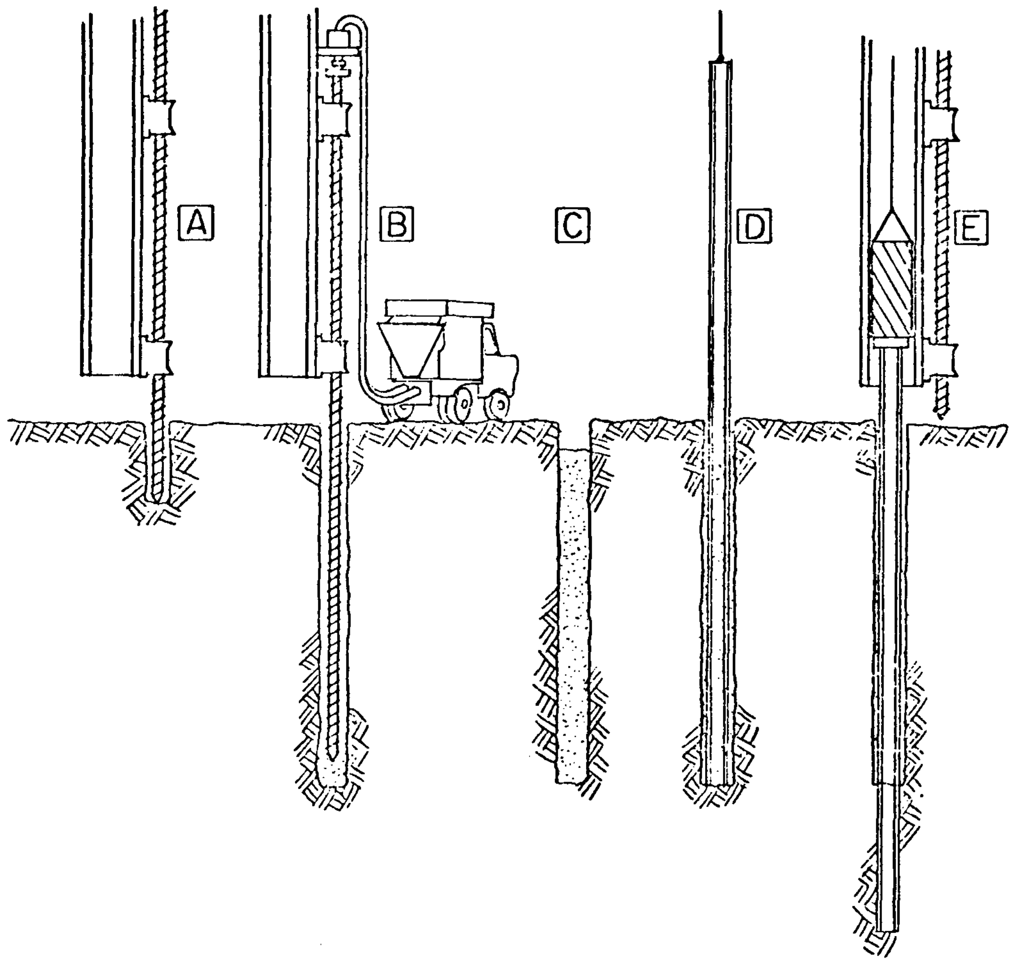
Another technique involves casing the hole and then filling the auger-drilled hole with slurry as the casing is withdrawn. After the soldier beam is set in place the last few feet are driven to achieve proper seating. But if this cannot be done, the bottom of the hole is filled with structural grade concrete. After the soldier beam is positioned, this concrete is used to fill the hole to the bottom of the excavation, and a lean concrete fill may be used to fill the remainder of the cavity above the bottom

FIGURE 2

TORONTO SUBWAY, Yonge Street Construction, looking north along the subway below Yonge Street. The space in the picture is between the roof of the subway and the street. Utilities can be seen hanging from the deck beams in the middle of the picture. The soldier piles in this location are two channels and not the usual I-beams. Lagging has been wedged against the soil. The light area in the upper part of the picture is from an opening in the decking. Two struts have been used in this section, the deck beam at the top, and the strut immediately above the roof of the subway. Foreground--the strut above the roof has been removed.

FIGURE 3

TORONTO SUBWAY SYSTEM, Wall-Support System. A close-up of a soldier pile, timber lagging, and the wedging system. The material between the flanges of the soldier beam is lean concrete.



A HOLE AUGERED TO REQUIRED DEPTH

B GROUT PUMPED THROUGH HOLLOW-STEM AUGER FILLING HOLE AS AUGER IS WITHDRAWN

C GROUT-FILLED HOLE READY FOR H-BEAM. GROUT LEVEL PROVIDES FOR DISPLACEMENT OF H-BEAM

D SOLDIER BEAM LOWERED INTO GROUTED HOLE

E SOLDIER BEAM DRIVEN TO REQUIRED DEPTH

INSTALLATION STEPS FOR GROUT SOLDIER-BEAM **FIGURE 4** of the excavation. This is necessary because the flanges are normally exposed to receive the wood lagging as the excavation progresses.

Thus hole collapse can be prevented by using a casing or bentonite slurry support prior to installation of the soldier pile and lean concrete fill. However, the cost of such precautions may dictate the economy of another system such as pipe caisson soldier piles.

Figure 5 illustrates several methods of installing lagging with wide flange soldier piles.

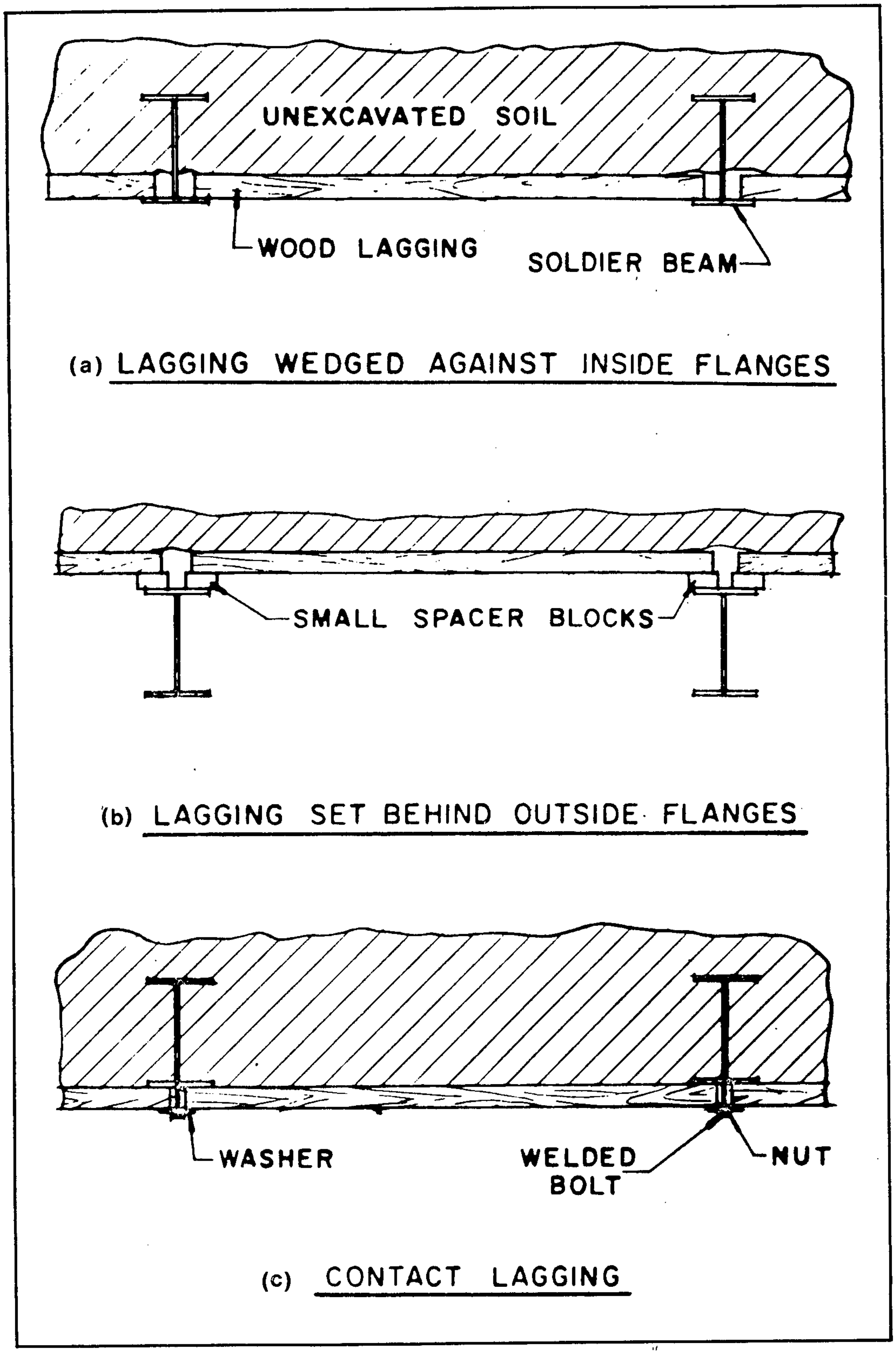
Wood lagging is usually installed against the inside flanges of the soldier piles as shown in (a). In order to avoid utilities, the soldier pile spacing may be varied and the lagging must be cut to fit. This is the biggest drawback with concrete lagging. Lagging may be set behind the outside, or soilside flanges and wedged in place with spacer blocks as shown in (b). This permits incorporating the soldier pile in the permanent structure and can solve the problem of too little space between existing structures and the permanent transit structure. However, it may require special corrosion protection and water seal where the soldier piles intrude into the final structure.

The use of welded studs on the face of the pile for bolting the lagging in place permits a tight fit. It also permits the use of steel concrete lagging. While this lagging method is the most effective, creating fewer void spaces, it is a patented method and may be more expensive than other systems.

Grout or mortar is often installed behind the lagging to obtain a tighter fit and more nearly fill the voids, thus minimizing the settlement. Salt hay is frequently used for this purpose, or one of the filter cloths may be effective.

One distinct advantage of this type of wall system is its permeability. Hydrostatic pressure cannot build up behind the wall. By the same token, water coming through the wall can bring soil fines, resulting in the loss of ground that invariably must be avoided.

Soldier piles may be either left in place or removed when the permanent structure is completed. Removal leaves void spaces which may result in settlement. When left in place they are generally cut off five to eight feet below the surface. Timber lagging is almost always left in place except for the top five to eight feet. This is usually



METHODS OF INSTALLING LAGGING AT SOLDIER PILES **FIGURE 5** well above the permanent water table, leaving much of the timber to eventually rot out.

Soldier piles and lagging are probably the most economical method for supporting excavation walls developed so far, particularly if the piles can be pulled and reused. But there are other methods which more effectively hold the soil in position while excavation proceeds, thus enabling different approaches to underpinning or holding adjacent buildings in place. Savings can frequently be realized if a temporary support wall can be incorporated into the permanent structure.

SHEET-PILE WALLS

Another method used for bracing excavations exploys sheet piles. Sheet piling generally refers to steel sections with interlocking edges. Driven vertically they form a relatively impervious wall and have been used to build cofferdams for many years.

Sheet piling is available in a number of different shapes and steels of various strengths. Concrete and timber sheet piling have also been used.

Sheet piling is driven with either an impact, or a vibration, type hammer. When used for temporary support of an excavation it is generally pulled and reused. It is not suitable for urban areas because of the danger of cutting utilities. Boulders or other obstacles in the ground make driving sheet piles difficult. Noise and vibration from driving operations are objectionable in urban areas.

The physical properties of sheet sections generally limit sheet piling to shallower excavations. However, methods have been developed to overcome this difficulty. In Holland sheet pile walls are driven, and then excavation is done using compressed air to hold the wall in position eliminating most of the internal bracing requirements. A similar procedure has been used in Oslo to counteract bottom heave in clays and eliminate the need for dewatering. In that procedure the sheet piles are driven, excavation then proceeds down to roof elevation and the roof in cast. Excavation continues below the roof under compressed air.

Sheet piles are effective for cutting off groundwater and for supporting relatively shallow excavations where some movement can be tolerated. Other bracing methods are more suitable for difficult soil conditions such as very dense or hard ground, till or boulders. Because of its inherent flexibility sheet piling should not be used if ground movement must be rigidly controlled.

Figure 6 shows the excavation for a subway entrance at Olaf, Norway with cross-lot braced sheet piles. An excavation for cut-and-cover tunnel construction can be braced by struts extending across the excavation from one face to the other.

BRACING STRUTS

Figure 7 illustrates several types of cross-lot struts. Cross-lot struts are usually steel, eithershapes or tubular sections. Timber is unsuitable for wide excavations. This speaker knows of no instance of precast concrete being used, but this would be a possibility, especially if the strut could be incorporated in the final structure.

Precast concrete struts have been used in cofferdam construction sometimes with disastrous results. When steel or timber start to fail the process is usually slow and noticeable, but when precast concrete starts to fail it can happen suddenly.

Struts should generally be preloaded to minimize ground movements. A common basis for design is to preload 50 percent of the design load, but sometimes as little as 25 percent of the design is used. The struts should be preloaded to 50-100 percent of the design loads based on at-rest earth pressures if the excavation is passing near a building and the intention is to use preloaded struts in lieu of underpinning.

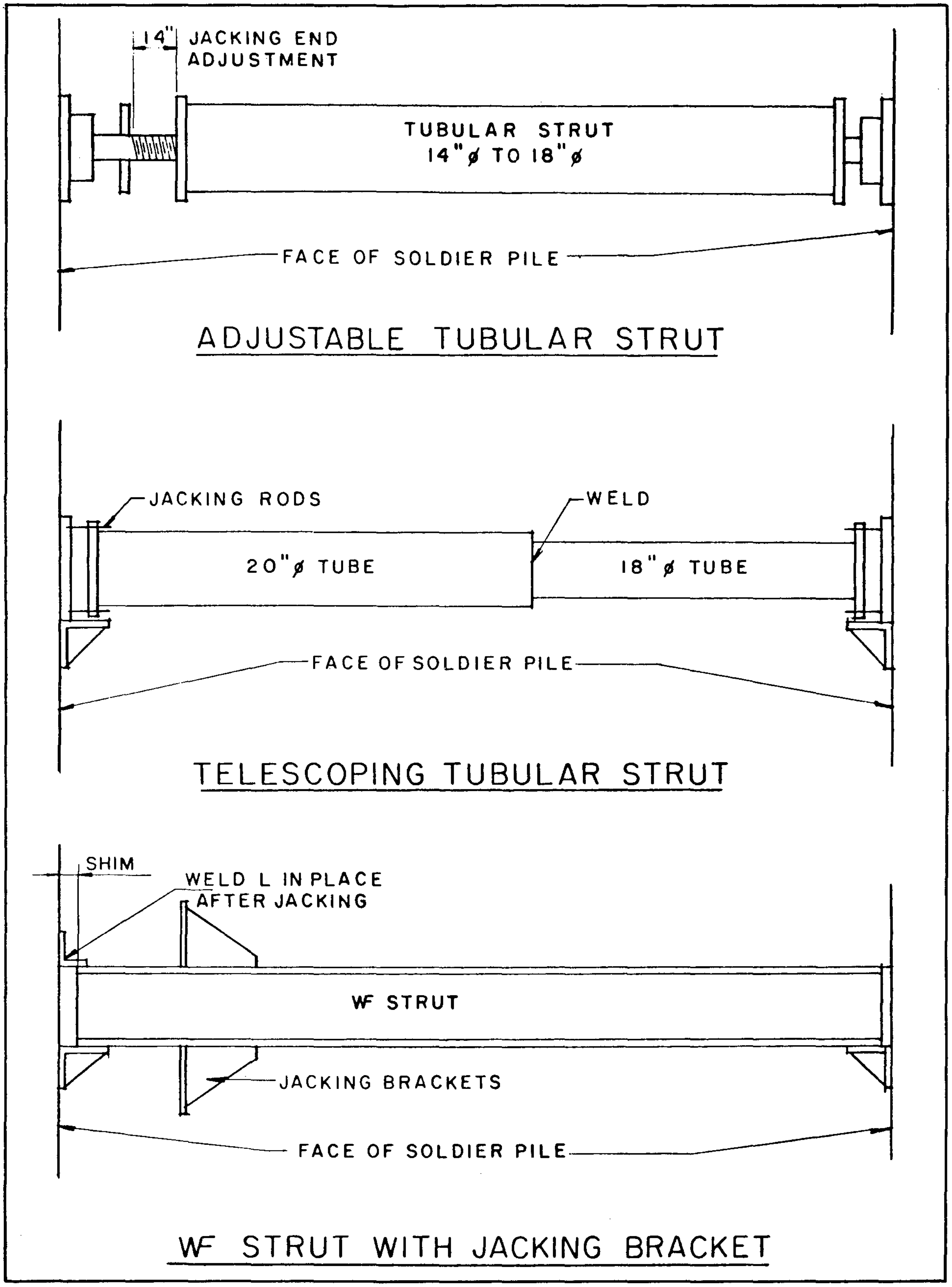
Preloading to 30 or 40 percent of the design load is probably sufficient if the area is such that small ground-movements may be permitted. This would apply in areas such as a wide street or through parks or residential areas with one-and two-story frame houses.

Both wide-flange and tubular struts can be installed with jacking brackets near the end. The strut is preloaded by hydraulic jacks and then shimmed in final position. This eliminates elastic shortening of the strut and the preload force can be accurately evaluated.

Either full- or partial-length wales may be used to increase the strut spacing, thereby keeping the excavation area as open as possible.

FIGURE 6

EXCAVATION FOR SUBWAY ENTRANCE AT OLAF KYRRES PLACE, Oslo, showing cross-lot braced sheet piles.



TYPES OF CROSS-LOT STRUTS

FIGURE 7

Tubular struts may have definite advantages over wide-flange members because the sections are uniform about both axes and thus orientation is not affected.

Prompt placement of the strut and wale as excavation progresses is always very important in arresting the potential initial ground movement in areas where settlement may be a problem.

TEMPORARY DECKING AND UTILITIES

Cut-and-cover construction in urban areas usually requires some form of strut decking to maintain traffic in the area while work continues. Temporary decking consists of beams supported at the excavation side walls by temporary walls such as soldier piles or by the permanent structure. Beams supporting the decking are also used to brace the side walls of the excavation and to provide support for utilities. Decking sections are installed as soon as practicable, to minimize disruption of traffic. The deck and temporary supporting structure are completely removed when tunnel construction is completed.

Permanent decking has been used to a limited degree, but only in Europe where in one instance precast prestressed-concrete decking was used.

Timber is the most frequently used material in the United States, but metal plates are used occasionally for small areas. In a few instances precast concrete was used.

Figures 8 and 9 show decking in use in the construction of the Washington, D.C. Metro. In the second slide note the access opening.

Relocating utilities is one of the most difficult problems in excavation and causes the most delays. This is probably due largely to the operation in the United States of both public and private utilities. The two tend to assign different priorities for relocations, and the involvement of several agencies generally tends to make coordination more difficult.

Hand excavation is generally required for the first few feet because the exact utility locations are not known. Unsuspected utilities may be present also, resulting in a prolonged period of street disruption.

FIGURE 8

WASHINGTON, D. C. METRO, Decking 11th and G Street

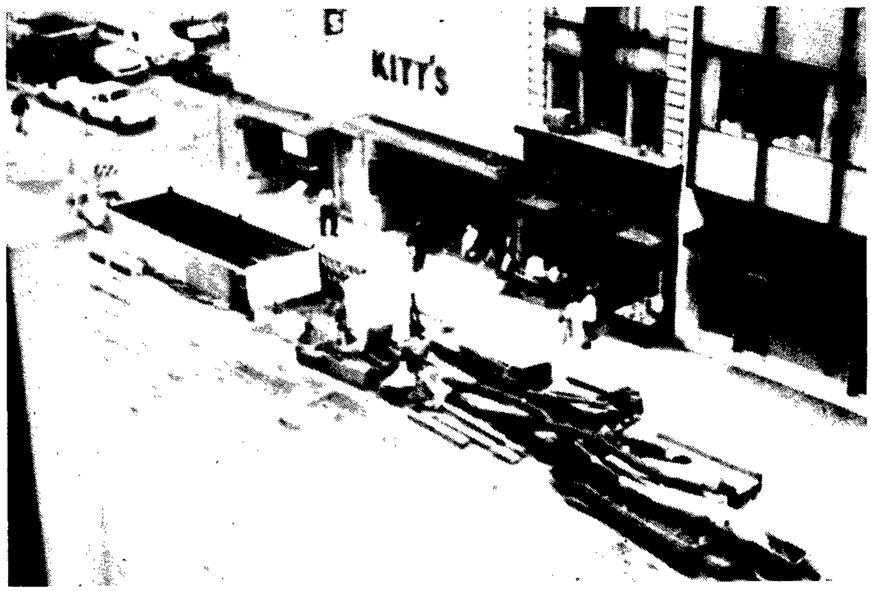


FIGURE 9

WASHINGTON, D. C. METRO, Decking and Access Opening

Figure 10 taken from the BART project shows soldier piles and cap beams with live water lines and two live telephone ducts to be supported.

Figure 11 is a view of work on the Washington, D.C. Metro as utilities are exposed.

Figure 12 is a view of the New York Transport Authority project showing the exposed utilities.

Utility locating normally begins with an inventory of existing utility maps. Conventional field location methods include digging and probing, as well as electromagnetic devices.

Existing storm and sanitary sewers should be relocated outside the limits of the excavation. In the case of sewers, not only the alignment but the proper grade must be maintained. When a sewer cannot be moved outside the excavation the entire sewer can be encased in a cast-in-place reinforced concrete box.

Another way of crossing the excavation is to build selected segments of the tunnel first and then route all utility crossings into this area, making all connections and crossings before constructing the remainder of the tunnel.

Figure 13 shows the steps made in crossing utilities. In Step 1 a segment of the tunnel is built. In Step 2 a new sewer and manholes are constructed picking up laterals or house connections and crossing the tunnel over the completed segment. The 3rd Step is starting construction of remaining segments of the tunnel.

Transverse and longitudinal water lines can be supported within the excavation if necessary, using brackets supported from excavation support walls or temporary street-deck structure.

Figure 14 shows some typical methods of supporting utilities within the excavation.

Water line hangers should be placed as close as possible to the beam supports to minimize flexing, or supports should be made independent of the street deck-supports. This is to avoid rupturing lines from the continual flexing caused by traffic loads on the decking.

Gas lines should be either permanently relocated outside the excavation or rerouted temporarily over the street near

FIGURE 10

SOLDIER PILES AND CAP BEAMS. Live water lines and two live telephone ducts were to be supported. (Courtesy of Fruin-Colnon Contracting Company.)

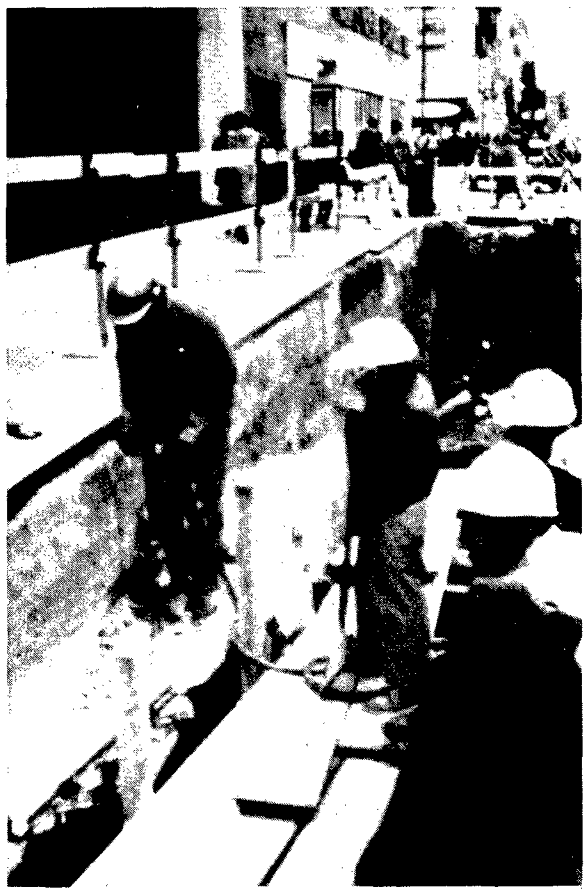


FIGURE 11

WASHINGTON, D. C. METRO, Exposing the Utilities

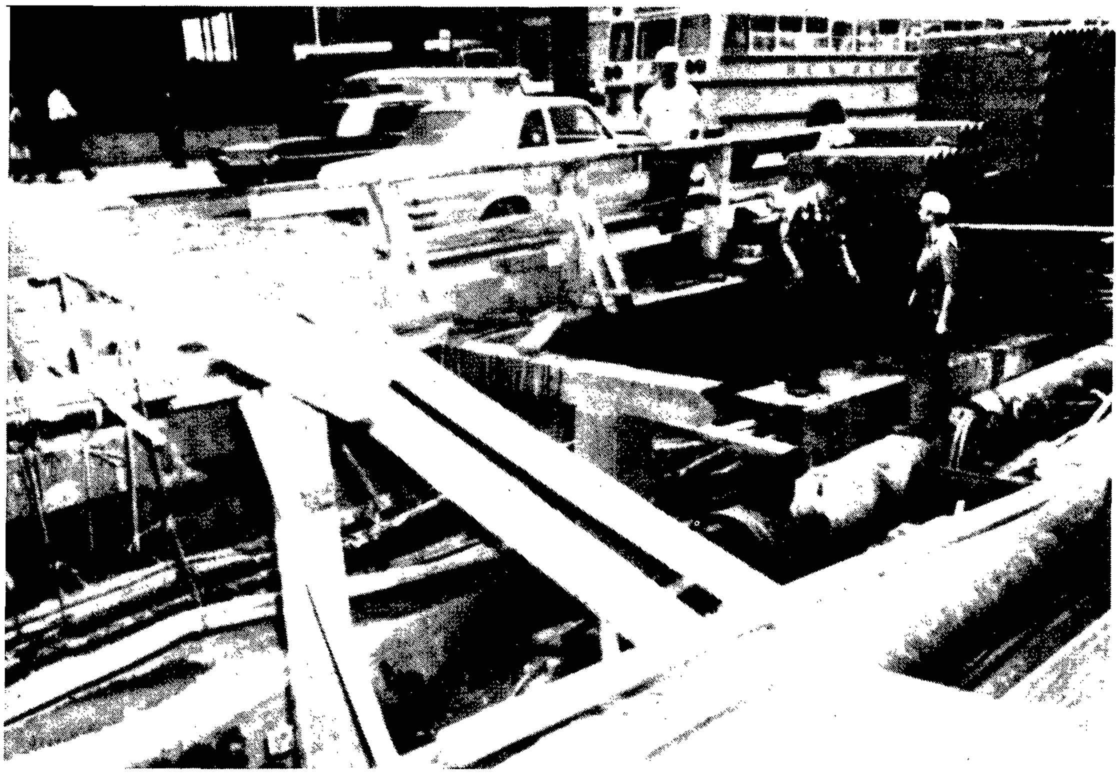
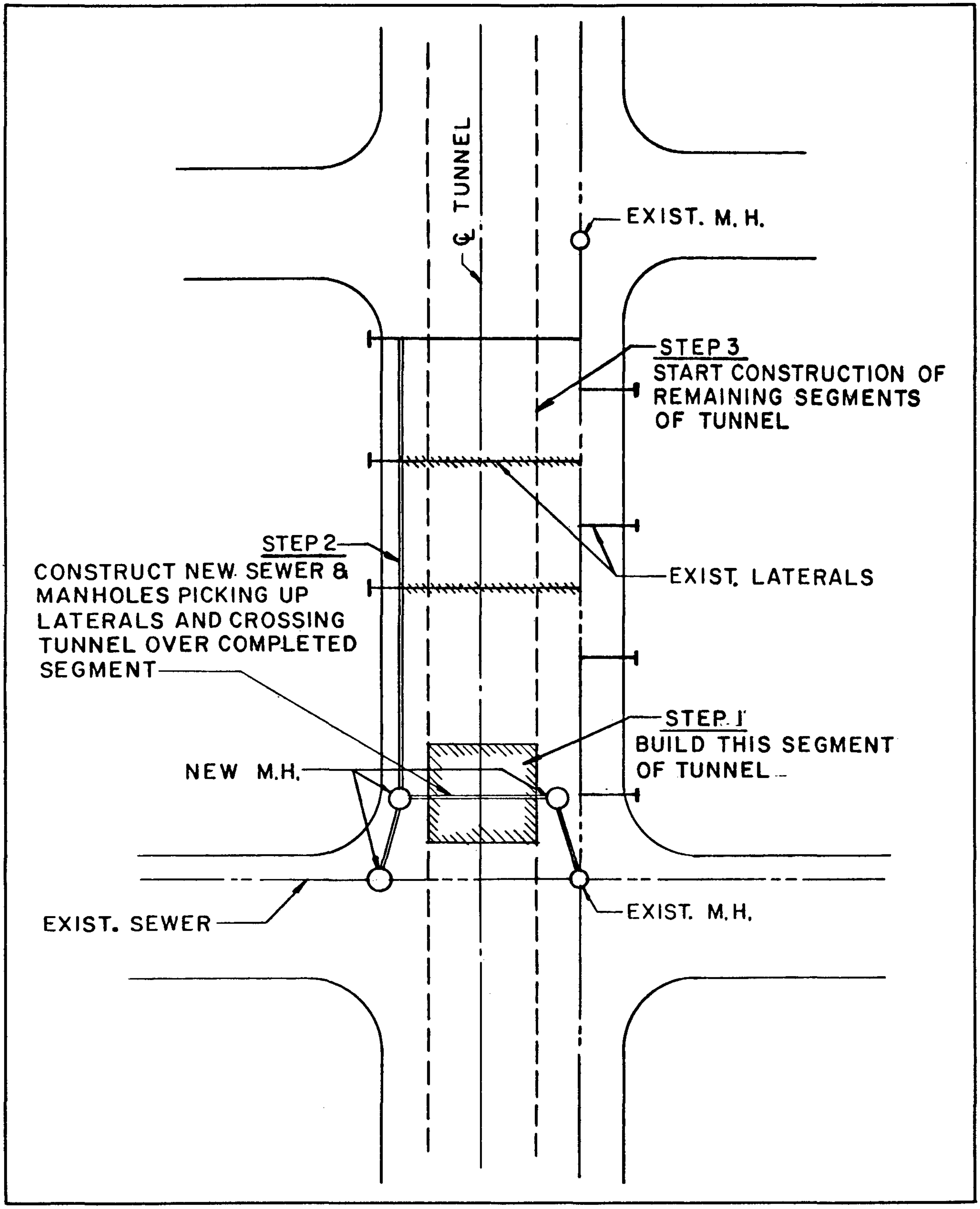


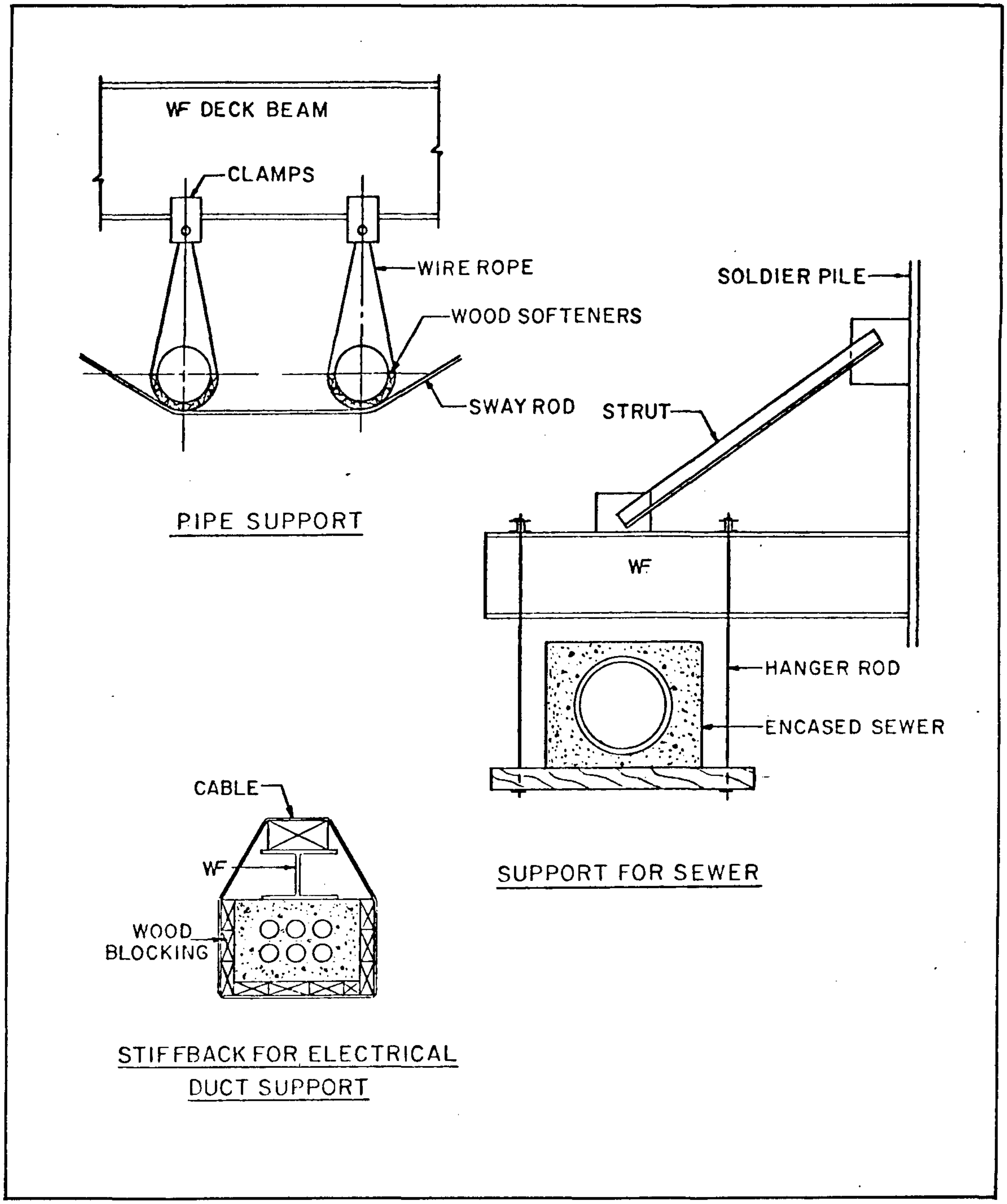
FIGURE 12

NEW YORK CITY TRANSIT AUTHORITY, Exposed Utilities. A maze of utility pipes and cables is close at hand for sidewalk superintendents. (From Engineering News-Record, October 20, 1966, p. 27, by permission.)



UTILITIES CROSSING CONSTRUCTION METHOD

FIGURE 13



TYPICAL UTILITY SUPPORTS

FIGURE 14

the excavation. Supporting a gas line within an excavation should be done under only the most rigidly controlled conditions because gas line rupture can be catastrophic.

Most telephone service lines are in duct banks made up of fiber ducts encased in concrete. Supporting such banks is not too difficult if they are easily accessible. Relocating a bank, however, may require several months to install the new manholes and splice perhaps several thousand pairs of wires.

Electric service is also often placed in duct banks and is handled much the same as telephone lines.

In the United States it is general procedure for the public utilities such as sewer and water lines to be relocated by the general contractor and the private utilities such as gas, electric and telephone to be relocated under separate contracts.

This makes controlling and scheduling operation very difficult and often leads to repeated street disruptions as well as work delays while a particular utility is relocated.

A highly functional method of handling utilities is to construct a utility tunnel or Utilidor encasing all utilities. This is a system used frequently in Europe. The space above a cut-and-cover tunnel should be readily and economically available for such use. The advantages of locating utilities in such a space include:

1. easier accessibility for installation, inspection, and maintenance;

2. the environment is less likely to cause deterioration and corrosion than direct furial in the ground;

3. there is less interference with surface traffic during installation, repairs, and utility-line expansions;

4. greater flexibility in installing new utilities or expanding existing ones to keep pace with growth.

So far in the country, however, there has been substantial resistance from the utility companies and public agencies to the proposal for common use of space.

PAPER 14

An Overview of Construction in Slurry Trenches

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PAPER 14

AN OVERVIEW OF CONSTRUCTION IN SLURRY TRENCHES

by

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CONCEPTS OF TRENCH STABILITY

The stability of slurry-filled trenches usually is considered on the basis of conventional limit theory, assuming simple wedge criteria for the conditions existing at failure. In general, the face of a trench can remain stable provided the hydrostatic force exerted by the slurry balances and exceeds the lateral component of the weight of the active wedge. Quite commonly, the assumption is made in the analysis that an essentially impermeable filter cake is formed at the junction of the soil and the slurry, allowing the latter to exert its full hydrostatic thrust. Where conditons warrant, this assumption should be substantiated by tests and in-situ observations, otherwise it may lead to an overestimation of the actual force acting upon the face. Xanthakos (1975) gives a complete summary of stability analyses for various soils, trench configurations and shapes, and under a variety of site conditons.

The stability problem and the general way in which it is analyzed is largely based on theoretical and empirical relations governed by the shear strength of a soil. On the other hand, soil movement and deformation resulting from such excavation is difficult to assess and handle. Certain predictions can be made on the basis of past experience and observations. These show that ground settlement caused by excavating slurry-filled trenches generally is small and diminishes away from the trench. However, exceptions must be expected in very loose or in very soft soil, or when excavating under very heavy surcharge loads. Whether predictions are in this case based on a theoretical evaluation of the settlement process or rely on field controls is academic, since the practical consequences of the actual settlement will depend on the type, sensitivity and proximity of existing foundations and buildings.

Evidently, the stability of a trench is greatly aided by such factors as panel length and shape, and arching of soil. These can be readily included in the analysis to modify the two-dimensional characteristics of the problem. Yet, it is appropriate to suggest that any soil having a friction angle less than 10° and negligible cohesion should be excavated with great caution.

The record of the few incidents reported in slurry trench excavations shows that slip failure or partial collapse of the face is confined to the upper part of the trench, and is commonly observed just below the guide walls. This may be due to the fact that the strength of a soil usually is improved with depth, so that deep wedge failure is quite uncommon and presumably occurs under extreme conditions.

The most usual causes of incidents are (a) sudden rise in the groundwater table during protracted rainy spells and especially where the excavation is in granular soils; (b) excavation under dynamic pore water; (c) rapid drawdown of slurry caused by sudden escape to open ground; (d) excavation carried out in recently placed hydraulic fills or in very soft marine clays; (e) excavation at the toe of a slope; and (f) excavation in fissured clays. Other possible problems relate directly to slurry contamination leading to partial or complete destruction of the filter cake and the subsequent collapse of the face.

SOIL-SLURRY INTERACTION

1. Formation of Seal

Fig. 1 shows schematically how a colloidal suspension penetrates into a porous granular medium. In (a) the slurry enters into the pores due to some pressure difference, and some colloid particles are deposited in the voids. In (b) the slurry is filtered and separated from the solid fraction which now accumulates in the soil pores and forms the so-called filter cake. The process continues until the cake is covered by a thin protective film along the face as shown in (c), and at this stage the seal practically offers complete resistance to further penetration.

Since the seal and the cake actually are obtained through the filtration of the slurry, they are influenced by the permeability of the soil. It follows, therefore, that a certain degree of permeability is necessary for this process to occur and, in fact, protective films are not formed in soils of very low permeability.

Several distinct mechanisms can lead to the formation of a seal under varying conditions, described in the following statements.

a. Surface filtration occurs when the typical filter cake is formed in the pores of a soil with a relatively low permeability, allowing only a very limited penetration of the soil pores. During the after formation of the cake, diluted slurry or water may continue to percolate through it and towards the soil.

b. Deep filtration occurs when slurry penetrates into coarse sand and gravel clogging the pores and building up a cake-like material such as the one exemplified in Fig. 1. For both surface and deep filtration the water or slurry loss consists of two parts: initial loss, which extends over the period required for the formation of the cake, and subsequent loss which may obey a linear flow law. Slurry loss during and after the filter cake formation can be represented by the curve of Fig. 2.

c. Rheological blocking occurs when slurry flows into porous soil until restrained by its own shear strength. This mechanism takes effect within fairly large pores, and is confirmed when bentonite is detected at considerable distance from the excavation. The process is characterized by a gradual gelation of the slurry in the zone of penetration.

Of these distinct mechanisms, surface filtration is the preferred process because (a) it eliminates slurry loss which can lead to face instability, and (b) it prevents unnecessary changes in the soil characteristics.

2. Thixotropic Gelation

The importance of the sealing process is particularly apparent with regard to this method of excavation. The existence of a low permeability film along the face is confirmed in the field, and is essential to the behavior and control of stabilizing slurries. From the point of view of practical field controls, significant improvements were possible through the use of thixotropic materials in fluid systems. In general terms thixotropic behavior is based on the initial nonequilibrium of interparticle forces due to remoding or disturbance, and the effects of subsequent structure changes within a slurry system when left undisturbed. Thixotropy is of quite general occurrence in all fine-grained materials, and consists of a reversible process of softening caused by disturbance, followed by a time-dependent return to the original state.

Laboratory and field tests confirm that the sol-gel transformation of dilute thixotropic suspensions such as the ones used as slurries is completely reversible. Upon cessation of shaking or stirring the suspension, the material gels. When stirring again commences, the material liquefies. This reversibility may continue indefinitely, and offers important advantages in engineering applications.

A practical aspect of thixotropic slurries is the linkage between colloid particles to form a structure which, at least over some period of time, causes the system to gel in the soil pores. Within the colloid concentration range which is practical and attainable for trenching and concreting operations, gelation leads rapidly to the build-up of the protective film at the interface. Although theoretical thixotropic strength change equations are not available to permit direct correlation of thixotropic effects to design problems, the field behavior of thixotropic fluids can be reasonably predicted within safe limits by instituting a simple testing program before and during construction.

3. Stability against Sloughing and Peel Off

The effect of the filter cake in keeping individual soil particles and grains near the face in their initial position is very important since they are thus compelled to remain in the earth structure. This action is necessary particularly in cohesionless soil where kinetic and rolling friction are the only forces that prevent the outer most particles of the exposed face from collapsing under the action of the tools and due to gravity. If enough soil grains could break away under the force of gravity, the exposed face would reach a state of imminent collapse. Sloughing and peel off are examples of this failure. In order for this type of collapse to be prevented, soil particles along the exposed face must be carried by the shear strength of the cake, the latter acting as plaster.

It is possible to test the ability of a colloidal suspension to prevent peel off, by using simple testing procedures such as the one illustrated in Fig. 3. A container is filled with an undisturbed soil sample, extracted and placed so that the vertical face becomes horizontal inside the container. Before commencing the test, the sample is saturated with water. Subsequently, bentonite suspension of a known gel strength is introduced into the tank, while the water is allowed to discharge through a drainage line. The container is then rotated as shown to reach an inclination sufficiently steep to force peel off of the outermost soil grains. When it reaches a rotation of 90°, it gives an indication of stability against peel off for a vertical face. Sample and volumetric disturbance during the test may give rise to difficulties in the interpretation of results and their relationship to field conditions. Nonetheless, the test is very useful and provides valuable information regarding the time for the formation of the filter cake hence the probable loss of grains at the face until the plastering becomes effective. The test can also disclose the influence of grain size upon the expected extent of penetration.

One of the functions of slurries is to keep in suspension soil particles accidentally left in the trench, and prevent them from settling down to the bottom where they may form a soft, unconsolidated layer of soil. Regarding a single grain as individual sphere of average diameter D, and assuming that the sphere is submerged in a fluid system of shear strength t_f, it is possible to prevent motion of the grain in any direction as long as the total shear force exerted upon its surface exceeds its submerged weight. This leads to the expression

$$D = \frac{3}{2} \frac{\tau_f}{\gamma - \gamma_f} \tag{1}$$

where

D = average particle size
τ_f = shear strength of slurry
γ = density (or weight), grain without voids
γ_f = density of slurry

Eq. (1) is also applicable to the stability of overhanging boundaries or to the undersides of irregular portions of the face, where soil grains tend to collapse due to gravity. The application of this theory to practical situations has, however, limitations. For example, it is neither necessary nor possible to keep in suspension large size soil grains, since this might require excessive gel strength in the slurry. In most excavations it is practicable to keep in suspension medium to coarse sand grains and allow larger particles accidentally mixed with the slurry to settle to the bottom, and then removed by passing an air lift.

FUNDAMENTALS OF SLURRY TECHNOLOGY

The technology of slurries has become increasingly more complex as a result of the many functions and requirements in present applications. The study of colloidal suspensions is now a separate field in slurry technology and involves the investigation of particle association and interaction, the phenomenon of flocculation and gelation, the so-called minimum gel structures, the phenomenon of peptization and deflocculation, the spontaneous swelling of montmorillonite, and applications therefrom to stability controls of slurries. These aspects relate directly to practical problems. An example is the separation of dispersed solids and soil in a slurry (usually sand and clay cuttings) either in special tanks or in vibrating screens and cyclones. Other important examples are the filtration and stability of filter cake. and the pumpability of slurry in reverse circulation hoses where the system is returned to the separation units transporting all excavated materials. These topics are treated rather extensively in the references mentioned at the end of this paper.

1. Properties of Slurries

Slurries possess the following main properties: (a) viscosity, gel strength and thixotrophy; (b) density of specific gravity; and (c) rheological characterics.

The behavior of a material in a state of flow is expressed by the relation between the shear stress τ and the resulting rate of shear D, the latter measured in reciprocal seconds, while τ is expressed in dynes/cm^2, lb/in^2 or lb /100 ft^2. The simplest expression is

$$\tau = \eta D \tag{2}$$

indicating a straight line passing through the origin in the τ versus D diagram. Liquids obeying Eq. (2) are called "Newtonian" fluids. The proportionality η is called the coefficient of viscosity, or simple viscosity. The unit of viscosity in the metric system is grams. cm^{-1} . sec, dyne. sec.cm^{-2}, or "poise". The viscosity of water at 20°C is about 0.01 poise or 1 centipoise (cP).

Slurry systems display more complicated flow characteristics than the simple linear pattern expressed by Eq. (2). Figure 4 shows four typical types of fluid behavior. At each point along these curves an "apparent viscosity" exists, which is equal to τ/D. Alternatively, the flow behaviour is expressed in terms of the "plastic viscosity", which is the cotangent of the angle α . The flow behavior exemplified curve 4 is observed quite commonly in colloidal systems used as slurries, and is called "Bingham plastic flow".

Fig. 5 shows in some detail the flow behavior of a Bingham fluid, i.e., a slurry. Several stresses on the curve have particular significance. For example, the flow curve intersects the shear stress ordinate at a point designated by τ_s. This is the stress called gel strength, and represents the minimum shear stress required to produce flow. Extrapolating the straight line portion of the flow curve on the shear stress axis gives point τ_0 , called "Bingham yield stress", or simply yield stress and yield point. If τ_s and τ_0 coincide, the slurry is in the ideal plastic flow of curve 3 of Fig. 4. However, for most slurries $\tau_0 > \tau_s$. The linear part of the curve follows the relation

$$\tau = \tau_0 + \eta_p D \tag{3}$$

in which η_p denotes the plastic viscosity. Apparently, the plastic viscosity from Fig. 5 is η_p = (By)/(AX), whereas an apparent viscosity is obtained regarding the fluid as Newtonian so that η_a = (OY)/(OX).

As mentioned, the gel strength of slurries generally is caused by the linkage of colloid particles, preferentially in an edge-to-face association. If a shear stress is applied to such a system, no flow occurs until the gel strength is just exceeded at which point the gel structure breaks down although not completely destroyed (Jefferis, 1972). If the slurry is allowed to stand, the gel structure will reform according to the process already called thixotrophy.

The density of a slurry (over and above that of plain water) is due to the presence of solid materials. The solid fraction is provided by the main colloid (usually bentonite), and soil left and mixed with the slurry as byproduct of the excavation. For the usual applications the initial density of a slurry ranges from 1.04 to 1.15 g/cm^3 (65-72 lb/ft^3), but may rise to 1.25 g/cm^3 (about 78 lb/ft^3) toward the end of excavation.

The rheological characteristics relate to the deformation-time behavior, but mainly depend on (a) the viscosity of the suspension; (b) concentration of the suspended matter; (c) size and shape of the suspended particles; and (d) the type of particle interaction.

For measurement of flow and physical properties the reader may consult the references mentioned at the end of the paper.

2. Materials used in Slurries

Bentonite is usually used as the main colloid solid. It is produced from natural clays containing the montmorillonite mineral, and resembles a gray powder. There are different grades of bentonite available, and the better ones attain the proper mixing levels and flow properties at lower concentrations. The selection of a suitable bentonite brand must be based on the conditions to be contended with, but also is governed by cost.

Bentonite is generally suited to all applications except when unusually high salt concentrations are encountered in the excavation. Whereas bentonite swells vigorously and spontaneously in pure water, swelling may be restrained if the mineral is added to salt solutions, and in high salt concentration it does not occur at all.

Thinners and dispersants are used to convert a gelled slurry into a stable suspension. The conversion reduces the apparent viscosity and gel strength. These materials are quite effective and useful in slurry trench applications involving cast-in-place concrete walls, since they can resist contamination by cement and neutralize the effect of sea water. Such a material is FCL (ferrochrome lignosulfonate) which usually is added in ratios from 0.1 to 0.3% by weight.

The conversion from a dispersed to a gelled system is achieved by the use of flocculants and polyelectrolytes. Among these CMC (sodium carboxymethylcellulose) is commercially produced in several varieties and grades. The apparent viscosity and yield stress of a slurry increase quite rapidly with the addition of CMC. Thus, this material is effective where it is necessary to thicken or modify the flow properties including the filtration characteristics.

Alternatively, the protective action of CMC at higher concentrations forms the basis for its use in salt water, salt domes and offshore drilling. The stabilizing effect under these conditions becomes quite important and exceeds the effect of peptizers.

In general, slurries for trench excavations are water base systems. The water should be of good quality, contain a minimum amount of impurities, and have neutral consistency. Whenever possible the use of salt or sea water as the base liquid should be avoided.

Other materials used in slurries are fluid loss control agents (sand, starch, potassium aluminate, and CMC), and lost circulation agents such as fibrous or flaky materials used to seal very porous formations.

3. Functions of Slurries

Slurries must perform the following functions;

a. Form the filter cake and provide sufficient thrust to support the face; also prevent sloughing and peel off.

b. Remain in the trench rather than flowing towards the ground, hence minimize fluid loss in pervious formations.

c. Suspend detritus and thereby prevent sludey unconsolidated layers from accumulating at the bottom of the trench.

d. Prevent sedimentation in the mud circuit, where one is used. Additionally, slurries must fulfill several conditions and be compatible with certain operations. In this respect they must:

e. Ensure free flow of concrete from tremi pipes and allow displacement by fresh concrete; also minimize interference with the development of bond strength.

f. Allow easy pumping and facilitate the handling of materials from the excavation.

g. Allow sedimentation in tanks, and screening or hydrocycloning in order to separate and remove the solid fraction.

h. Facilitate their disposal in public drainage systems or in dump areas.

It is evident from these considerations that the slurry requirements are conflicting. Thus the first four functions require slurries of high viscosity and gel strength, whereas the last four conditions are best fulfilled by light, thin and free flowing suspensions. A reasonable compromise in the range of properties is therefore necessary to deal with practical situations.

For convenience Table 1 gives a summary of the physical and flow properties of slurries, and also indicates the current test method. The usual control limits are summarized in Table 2, based on the functions and conditions described in the preceding paragraphs.

4. Proportioning the Slurry

This involves the following steps.

a. Determine the amount of noncolloid fraction necessary to provide the required density for face stability (weighting agents). A small amount of soil from the excavation may be included in the computations.

b. Select a suitable viscosity by reference to the soil type and porosity.

c. From Table 2 establish the applicable control limits of the flow properties.

d. Determine whether control agents are necessary and economically justified.

e. Proportion the constituent materials. This phase merely consists of a quantitative estimation of bentonite, noncolloid fraction, and control agents. The proportioning may be empirical knowing the properties of the materials selected, or it may have a technical basis for tests and estimations.

The relative economy of slurries depends on the actual cost at the site of the basic materials, and essentially bentonite, chemicals, weighting agents and water. This cost depends on the availability of materials in the immediate locality. Where high quality bentonites are not available, transportation costs may dictate what the main colloid should be and whether chemicals should be used.

DIAPHRAGM WALL TYPES. GENERAL REVIEW

Diaphragm walls and related elements built in slurry-filled trenches or holes are used in the following situations: as underpinning structures, as retaining walls and ground support systems, as seepage cutoffs for ground water control and as load bearing elements under both temporary and permanent conditions. This Section gives a summary of wall types and configurations suited to these functions.

1. Continuous Cast-in-Place Diaphragm Walls

These are probably the most widely used walls. Construction procedures and details are reviewed in other papers in this set; hence only some main points will be mentioned herein.

Perhaps the most important aspects in the entire process are the requirements for assuring a reinforced concrete structure of an acceptable quality and strength, with emphasis placed on the effects of construction conditions and the method of placement of fresh concrete. For instance, defects will be evidenced if improper joint details are used for load transfer or for watertightness. Equally important are the connections with adjoining structural members and their method of execution, since any related problems carry through to the finished structure.

Regarding the concrete technology for cast-in-place walls, the following comments are appropriate:

a. When designing concrete mixes consideration must be given to the actual conditions during placement. Thus, besides strength, workability and flowability are also governing factors influencing the materials and proportions of a mix.

b. Interruptions and improper procedures during concreting may result in a variety of defects, namely: cold joints; zones of segregated or contaminated concrete; trappings of bentonite mud and inclusions of impurities; and even complete cavities.

c. Plug flow of fresh concrete (initial batches always on top) benefits the quality of set concrete by minimizing trappings and inclusions of noncrete materials, and by sweeping the bentonite coating from around the bar for a more efficient bond.

d. Plug flow generally occurs with a close tremie pipe spacing and if the pipe is sufficiently submerged into the fresh concrete. Further, the flow is more vigorous if the tremie pipe is filled to the top. The set concrete is of a better quality if pouring is carried out at a uniform speed.

e. The strength of set concrete may be influenced by (a) the flow motion of fresh concrete; (b) changes in the water/cement ratio; (c) the introduction of bentonite and slime; and (d) the conditions with respect to curing time, moisture, and temperature.

f. At present, lack of an acceptable detailed record makes it difficult to fully explain the bond distribution. Since a mechanism for predicting stresses at failure is not readily available, bond analysis for each particular problem should be based on a conservative approach.

g. A reinforcing cage should give minimum impediment to the flow of concrete. Hence, it should be assembled using larger size bars at greater spacing, but this should not be exaggerated since it also means reduced bond.

h. Construction joints should be compatible with the desired watertightness of the wall, and with the loads and stresses to be transferred at the joint.

i. A wall depends for strength and structural integrity on three main factors: (a) strength of concrete; (b) details and effectivness of construction joints; and (c) details and strength of connections to adjoining structural members.

2. Prefabricated Diaphragm Walls

<u>Grout Systems.</u> These constitute specially made cement-bentonite mixes which usually include suitable agents to regulate and retard premature stiffening. A grout must be introduced under conditions which are compatible with trench excavation, placement of prefabricated sections, and development of seal at the joints.

A single grout consists of water, bentonite, cement and some additives. Initially this system supports the excavation, but later it seals the joints. The twofold function places special requirements: the grout must be a self-hardening material, usually called coulis. The development of mechanical strength of a typical coulis must be a controllable function of time after mixing. The setting time must be controlled within a narrow range; the grout should not stiffen until the panel excavation is completed and the precast panel is positioned in the trench, but it must rapidly gain strength thereafter. The final strength is influenced primarily by the content of cement materials, and usually is in the range of 7 to 15 kg/cm^2 (100 to 200 lb/in^2). A displacement-type grout (as opposed to a single grout) is used in many cases. This consists of two systems. The initial bentonite slurry is intended only to support the face of the trench, and is replaced by a suitable bonding grout just before the precast sections are placed. The properties of the bonding grout are, therefore, less dependent upon the construction process and are rather selected to allow the grout to become part of the finished wall. In general, this material can gain sufficient strength and also penetrate the joints vigurously for watertightness. The use of a displacement-type grout offers three main advantages: (a) it eliminates strict adherence to the work schedule; (b) allows better control of ultimate strength; and (c) reduces the danger of contamination during the process of excavation. On the other hand, the displacement of slurry by grout requires special care.

Construction Examples. A prefabricated wall with identical panels such as the one shown in Fig. 6 can be used where there are stiff or dense layers just below the bottom of the excavation. In this case the wall derives its stability with minimum embedment so that most of its height actually is usable in enclosing a basement area or in protecting an underground installation. In unfavorable soil the embedment may sometimes be from 30 to 50% of the total wall height. For these conditions it is more economical to build the wall with beam-and-slab panels such as the ones shown if Fig. 7.

Such a structure acts like a soldier pile with lagging. The beams are relatively slender, hence they can be made longer without exceeding the maximum practicable weight for handling and lifting. They are sufficiently embedded below the base of excavation, whereas the slab sections are stopped about 3 ft below that level. The installation is carried out as shown in Fig. 8. Primary panels consisting of two beam sections and one slab section are inserted in alternate trenches. The secondary panels are installed between the primary, and include two slab sections and one beam. In this manner the number of units in each panel is three, and this gives an optimum daily schedule for one excavating machine and one crane. The construction of panels follows the sequence shown.

Favorable Considerations. The main advantage of prefabricated walls are the merits inherent in precasting; the general appearance of the exposed wall is satisfactory, and the face is smooth and reasonably clean so that further treatment is not needed. Improved concrete quality control invariably gives a better assurance of specified strength, and allows a better accuracy in placing the reinforcement. Owing to savings on materials prefabricated walls generally are thinner than cast-in-place walls. Furthermore, the final structure is built to finer tolerances, and openings and miscellaneous inserts can be more accurately positioned.

The continuous grout cover on the exterior face acts as a waterproofing membrane, and if necessary this face can be treated with waterproofing compounds after the section is fabricated. If exposed concrete is not desired on the interior face, a special finish or face treatment can be applied to it during prefabrication. A further advantage results from lessening the difficulties of construction, particularly the usual problems in the placement of concrete. On the other hand, the installation of precast walls requires careful preparation and strict adherence to schedule.

The foregoing considerations must be balanced against the particular conditions of a project, for instance the minimum job size necessary to offset certain fixed costs inherent in precasting; bad soil conditions causing the grout to crack at the joints; or the intended functions of the wall with emphasis on its structural continuity.

3. Bored Pile Walls

A serious advantage of this type of ground support is that it can be adapted almost on any site and almost under any conditions; the presence of buildings and utilities or a very difficult soil offers no serious impediment to the construction operations. Furthermore, bored pile walls can be designed and built to be reasonably watertight and resist full lateral stresses. If necessary, they can be capped with a reinforced concrete beam to distribute structural loads. Under appropriate circumstances, bored pile walls may be constructed in suitable ground at a lower cost than any other support system. The installation becomes less dependent upon the site conditions and, therefore, is more economical in many instances.

A bored pile wall may be made contiguous, i.e., with the piles in contact, merely touching one another, or adjoining; interlocking as is the case with the secant pile method; or the pile spacing may exceed the pile diameter where the soil is fairly stable. Several type of bored pile walls are shown in Figs. 9 and 10. The configuration must be worked out to satisfy the requirements of the structure and the soil conditions. The following examples demonstrate the choice of design.

a. In competent soil and where ground water is not a problem, it usually is advantageous and economical to install only one row of piles, choose a larger diameter, and space the piles as shown in Fig. 9. Precut lagging is inserted, or the installation may be combined with a face wall.

b. In caving soil or in soil with water, contiguous bored piles are necessary and built as shown in Fig. 10 (a) and (b). Since the piles are touching one another, the wall commonly is given the name tangent wall.

c. Under extremely unfavorable ground conditions, the wall can be built using the secant method shown in Fig. 10 (c) and (d).

Installation under Slurry Support. Preboring with bentonite slurry can be used to facilitate the installation and extraction of temporary casings in the upper part, or as an independent method in conjunction with rotary drilling and reverse circulation. The use of slurry for the entire depth of the hole can eliminate the need for any casing, since the probability of encountering instability in a slurry-filled hole is extremely remote. However, unless the machine is handled along a sufficiently controlled alignment, a temporary casing can provide a useful function in setting the correct spacing and line. The main considerations that influence the construction are: (a) deviation from the true vertical line; (b) hole enlargement due to some loss of grains at the face; and (c) slurry mud communication between holes and in the same vicinity when the excavation is in very pervious ground. In this case it is advantageous to construct a bored pile wall by the hit-and-miss technique.

Where bored pile walls pass through water bearing formations, difficulties will arise if it is necessary to seal the wall. This is usually done by means of various grouting techniques, but it must be emphasized that grouting does not always provide a seal which is absolutely watertight. Under normal conditions the sealing is adequate in that it allows the general excavation to proceed with minor interference and problems. Current sealing techniques include the presetting of grout tubes in the shaft at the construction stage.

Uses and Limitations. Bored pile walls built with auger-type tools supplemented with casings are best suited to cohesive soils or where the upper crust is a water bearing formation but of a limited depth. Contiguous or tangent walls built in slurry-filled holes have only a minor dependence on the type and conditions of soil, and the most serious impediment to their use probably is the presence of hard rock and similar obstructions.

The method has an inherent flexibility, particularly with regard to existing shallow footings and underground utilities. When the walls are built as individual piles, it usually is possible to chisel through obstructions, and to bore close to and around footings with a minimum risk of damage in a minimum of time.

At times lack of working space, headroom, accessibility, and working time can preclude the use of other ground support systems including diaphragm walls from the standpoint of construction efficiency and in spite of favorable soil conditions. Under these circumstances, bored piles are probably the best alternative. However, like all other techniques, bored piles have limitations and certain disadvantages. Excessive overbreak can occur in unstable ground while the casing is withdrawn, or an oversized hole can result with the use of slurries to protect the excavation. Additionally, seepage and leakage through the joints cannot always be stopped to the degree desired, especially with high water table and deeper than normal excavations.

Secant Piles. Secant piles are shown in Fig. 10 (c). The cutting of the piles marked 2, 4, 6, 8 etc. into the two neighboring piles must be done within a short time after the latter are concreted so that the concrete does not harden excessively. This is a special operation carried out by means of a hydraulically actuated casing fitted with a special cutting hedge. The casing is guided at two points on a heavy boring rig, and is therefore equipment made specially for this type of work. Such a special rig is the Benoto rig, widely used in Europe.

Since all piles in a secant wall share the lateral loads equally, it is advantageous to provide reinforcement in every pile, arranged as shown.

Secant piles are more expensive than any other type of bored pile walls, but are very useful in troublesome soils with ground water. The method of installation produces a structural wall and ensures a reasonable watertightness.

A variation of a secant pile wall developed by this author is shown in Fig. 10 (d). The interlocked holes usually are made with a diameter one-half that of the cutting holes, and are filled with a bentonite-cement type grout that attains a strength in the range of 100-200 lb/in^2. The main advantage of this installation is economy; the material used in the interlocked piles is the initial slurry which gradually hardens to the extent desired by the addition of cement. Drilling of the main holes requires no special equipment, but can be done with conventional auger rigs or rotary drills. For a watertight construction the use of this type of wall is particularly recommended.

4. Composite Walls

An example of composite walls is shown in Fig. 11. In this case diaphragm all panels are combined with steel I beams to produce a relatively stiff foundation wall. These variations often are dictated by the construction requirements, or they may be merely a matter of custom and regional development.

Composite walls are indicated in unstable ground and in difficult soils showing erratic variations in properties. Alternatively, they are suited to certain congested sites where the effect of heavy surcharge or concentrated loads necessitates the use of short panels to improve trench stability. In the United States the steel-and-concrete wall initially was developed regionally on the west coast and tried on the difficult sites of San Francisco.

The construction usually follows the sequence shown in Fig. 11. The primary elements are built in alternate panels using either round or square-end machines to carry out the slot excavation. The steel I beams are assembled with the reinforcing cage, then they are lifted and inserted in the panel which is concreted between the webs of the beams. The wall is completed by filling the secondary panels as shown in (b).

Advantages and Disadvantages. Steel-and-concrete walls have the following advantages: (a) they accommodate the concept of short trenches and thereby reduce the risk of collapse; (b) they transfer the lateral earth stresses to the steel sections, which generally offer a greater moment resisting capacity and their actual ultimate strength is better controlled; (c) they enhance the possibilities of structural bracing; and (d) they are adaptable where space and time restrictions can influence construction.

Some concern is expressed regarding the durability of steel I beams in earth, but the actual record shows that the life of these members exceeds the theoretical expectations. Thus, failure due to corrosion does not appear to represent a potential problem. However, these observations apply to steel sections embedded into undisturbed soils which are deficient in oxygen at levels a few feet below the surface. The effects on steel under disturbed soil conditions may be significant, and at present it is not known whether the method of installation and trenching might constitute a direct disturbance to the soil in so far as these effects are concerned. These doubts apparently form the basis for taking protective measures to extend the durability of steel.

Some waste of materials often is unavoidable with these walls. I beams are most effective if they are used in deeper sections, usually 36-in. sections or deeper. This sometimes means a concrete wall thickness greater than necessary for the average beam spacing and depth. This, in turn, means more excavation and slurry.

5. Slurry Trench Cutoff Walls

These walls are built below grade for the control of ground water. The method generally consists of excavating a continuous trench while keeping it filled with slurry, and then backfilling with selected materials, either flexible or rigid, to produce a cutoff. The excavation is carried out with dragline buckets, backhoes, clamshells, or conventional trenchers. Alternatively, a watertight screen can be built from the surface by injecting clay-cement grout under pressure into a preformed narrow slot.

The objective of the method is to provide a positive control measure provided this can be accomplished at a reasonable cost, and without altering the groundwater level during construction.

Cutoff walls are characterized by the degree of freedom to deform and by the type of backfill materials. Hence, they are grouped according to the following general categories.

a. Selected earth backfill walls; used when a fully impervious system is not required or economically justified, and when flexibility is a desirable feature.

b. Grout curtains or grout wall cutoffs also called solidified walls; used when an impervious system is required, often for a limited duration, and must be obtained with minimum sequence of construction events.

c. Vibrated membrane and injected grout curtains, suitable when horizontal differential ground movements are not anticipated and when the excavation is preferentially shallow.

d. Plastic concrete backfill walls also called flexible concrete walls; adapted for both strength and deformability.

e. Concrete diaphragm walls, either plain or reinforced; used when rigidity is the prime factor for the conditions under which the structure will be in service.

The main difficulty in selecting a suitable type lies in the sufficient determination of ground movement under the expected consolidation, transient flow and gradient, and also in making a theoretical solution sensitive to the probable construction imperfections inherent with each type and project. Many cutoffs bear the evidence of action which as a problem does not relate to the theoretical efficiency of the material, but demonstrates the need of securing a construction compatible with the conditions of service. Among the types mentioned, the following are reviewed briefly.

<u>Cement-Bentonite Grout Wall Cutoffs.</u> Basically these consist of cement-bentonite mixes containing no aggregate other than soil mixed with the slurry during excavation. The cutoffs are sufficiently flexible to accommodate ground movement, hence they offer a practical solution where this condition exists. Furthermore, the choice is economically attractive because the construction involves limited use of materials and is completed with a minimum sequence of operations. A further advantage is that it does not require slurry disposal, which remains a serious problem on urban sites.

The method consists of reusing the bentonite slurry in the excavated trench and transforming it into a construction material by adding cement in certain proportions. When such a system is placed in the ground and allowed to set, it attains sufficient strength and yet remains essentially elastic so that it can deform without cracking. The quoted strength range is between 10 and 30 kg/cm^2 or 150 and 400 lb/in^2, and the modulus of elasticity is from 200 to 500 kg/cm^2 or 2800 or 7000 lb/in^2. Evidently, no set relationship exists between strength and modulus of elasticity, and when variables are introduced such as the bentonite content and incidental soil fraction, it is reasonable to expect broad variations in the elastic behavior. The quoted permeability is in the order of 10^{-8} cm/sec. The material is said to remain impermeable and free of cracks as long as it remains in the ground.

The materials are mixed in the following proportions: bentonite, 2-4%; cement, 15-20%; lignosulphite, 0.1%; and aggregate from the excavation, 5-10%. The remaining fraction is provided by water.

This type of cutoff has had increased use and applications due to the simplicity of construction and the apparent economy. On the other hand, it must be noted that a cement-bentonite-soil mix is less homogeneous than other types of cutoffs. Due to the high water ratio it is quite difficult to predict the long term exchange between the material and the soil. Such interaction suggests that there may be circumstances under which the watertightness may decrease somewhat with time. The long term behavior has not been fully documented, and information as to what extent adverse effects can be prevented is lacking. It is interestine to recall that large surface cracks observed in tests confirm the reversal from swelling to shrinking under the influence of progressive dehydration.

<u>Injected Grout Screens</u>. A thin impervious screen can be built from the surface by making a continuous slot in the ground and then filling the space with clay-cement grout. No slurry is required or used for stability purposes.

The sequence of the operation is shown in Fig. 12. It consists of driving steel H piles with the flanges back-to-back through the ground to be sealed off until their tips reach the underlying impermeable soil. The piles are then extracted one at a time, and the void thus created is filled with grout injected under pressure. In this fashion, a screen is produced consisting of a continuous core of grout occupying the voids left by the extracted piles and overlapped by a cemented zone of soil which is permeated by the injected material.

The thin fillet of earth left between the flanges of adjacent piles while being driven is disrupted and removed not only as the piles are extracted, but also under the pressure of the grout as it is pumped. This is confirmed by visual inspection of exposed finished screens.

In the early stages of this application the installation had depth limitations, but current modifications in the equipment and grouting procedures have allowed screens to be built at a depth in excess of 50 ft. Data from actual grout consumption confirm that the soil is penetrated by several inches for the average application and where the ground is groutable.

6. Load Bearing Elements

Load bearing structures are grouped into circular piles, linear walls, and prismatic elements. Circular piles can vary in diameter from 2.5 to 10 ft. Linear walls are primarily ground supports that must also transfer vertical loads. Less conventional shapes are shown in Fig. 13, and in many situations these may offer construction possibilities beyond the scope of circular piles. Where a diaphragm wall is to be constructed as ground support on a site, the use of prismatic foundations on the same job may be economically and technically justified in view of the availability of plant and equipment for the same type of work.

Base Preparation. Any trench intended for bottom bearing to support the loads must be cleaned out carefully and completely before concrete is placed. It is important to realize that different machines give bottoms of different shapes. In some instances the bottom is nearly flat, but it may be irregular or rounded according to the equipment used (for example a round-end clamshell or a rotary drill). At present there is no evidence to indicate that the shape of the bottom makes appreciable difference in the bearing capacity, at least from the theoretical standpoint, hence there should be no objection to the use of machines which do not cut nearly flat or square bottom. On the other hand, experience appears to demonstrate that a smooth base is better cleaned. Accordingly, it is advantageous to specify a reasonable flat and square bottom, which is achieved by passing a square-end clamshell upon completion of the excavation, or by slightly moving the long drill to level the base.

Most difficulties in obtaining a clean bottom are overcome by passing an air lift, while observing the discharge outlet to obtain a visual contact with the conditions at the bottom. A small amount of dry of plastic cuttings left at the bottom will make only a little difference in bearing capacity. However, if one inch or more of soft mud is allowed to accumulate at the base, it may not consolidate and compress in the short time required for the concrete to harden. In this case these soft materials will constitute a zone of postconstruction settlement.

Transfer of Load. Under the working load condition (limit design), the transfer of load to the surrounding soil generally begins with skin friction and is completed by base bearing when sufficient displacement has occurred vertically. Under the slurry method, failure to displace the bentonite from the interface can adversely affect the skin friction that may be considered in the design. The basic question, therefore, is whether the bentonite mud is or can in reality be replaced or absorbed by the rising concrete, and under what conditions, to what extent, and by what amount skin friction is affected by a bentonite layer left on the contact face between soil and the hardened concrete.

The suspicion that bentonite slurries may adversely affect the development of shaft resistance may have originated from the fact that they are used as lubricant for caisson and tunnel work. Nonetheless, the conditions and the manner in which bentonite is used in diaphragm wall construction are quite different from those situations where a free layer is intentionally introduced and maintained. The adhesion between the bentonite gel and the soil face is actually equal to the shear strength of the protective film, which can vary considerably with the type of concentration of bentonite but generally is low. As the concrete rises, it exerts a sweeping action on the face. The shear force of the advancing fresh mix is at least several times greater than the adhesion of bentonite so that all free bentonite is essentially removed.

Test on load bearing elements have shown a satisfactory load transfer in both sands and clays. As an example, a model pile test in sand revealed that a 5-mm thick film around the pile caused a reduction of load transfer of about 105. Other rests on piles with and without bentonite show no major difference in the load transferring characteristics (Fig. 14).

Despite the rapidly advancing knowledge of the soil-structure interaction, the design of load bearing elements is at best semi-empirical. Although it generally is desirable to provide a foundation which is essentially free of settlement, some downward displacement will take place in spite of the method of excavation, face support, cleanout practice, and concrete placement. This downward displacement generally benefits the transfer of load since it is beneficial in mobilizing the shear and bearing resistance. Because sidewall shear is developed at much smaller vertical displacements, appreciable wall shaft resistance usually is mobilized before any load can be transferred by base bearing.

A detailed analysis of load transfer and recommended design procedures of load bearing elements are included in the references mentioned at the end of this paper.

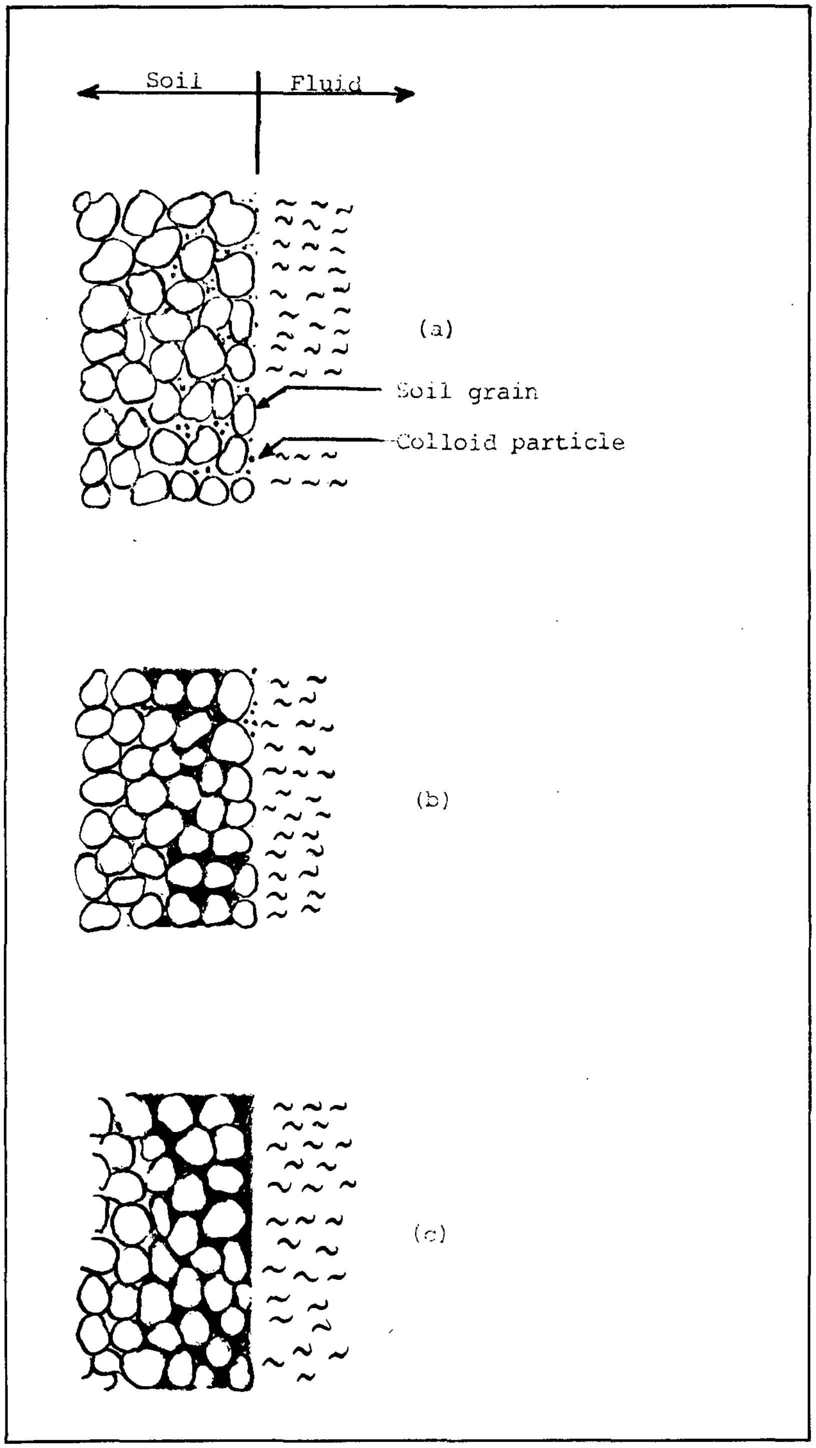
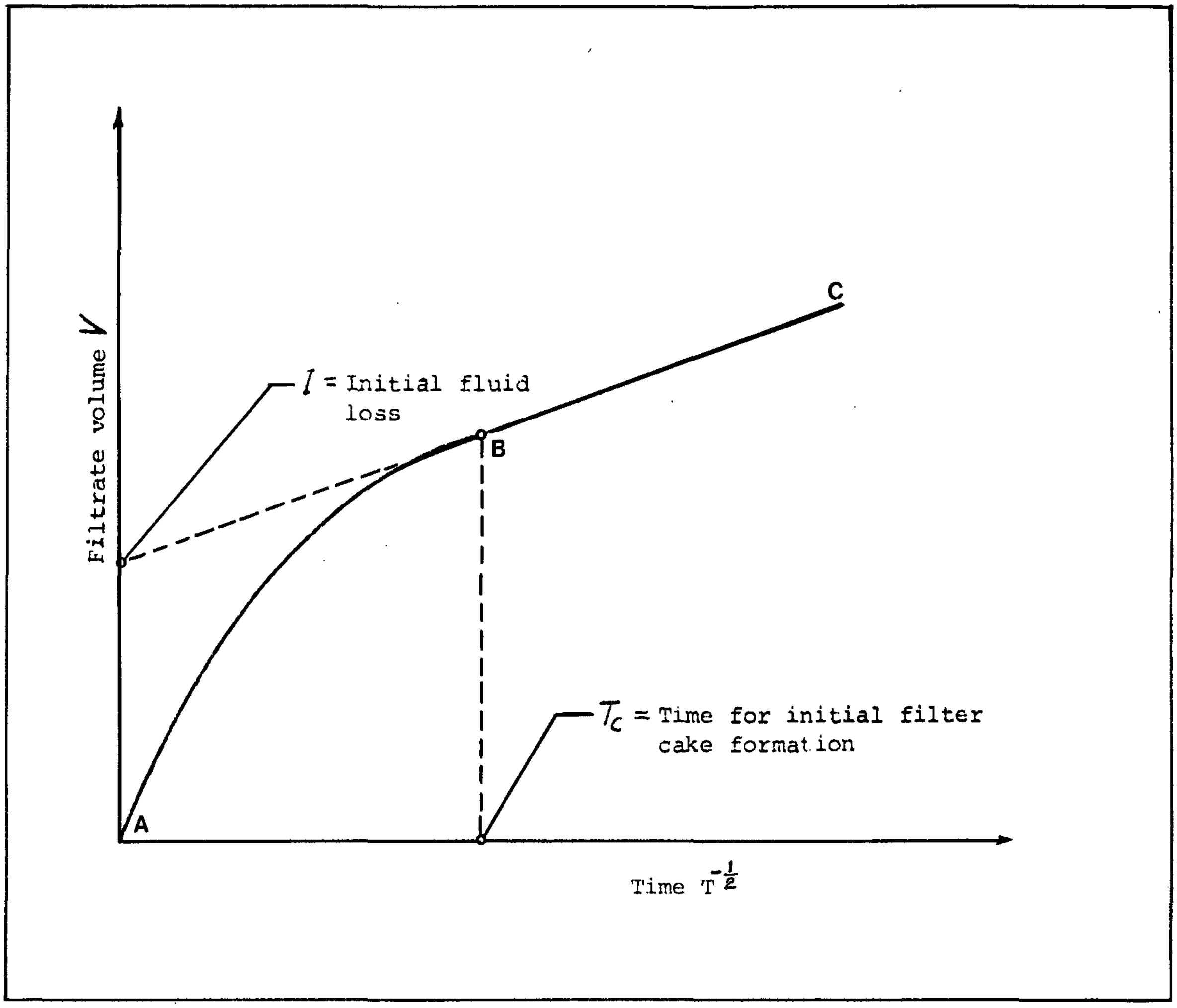


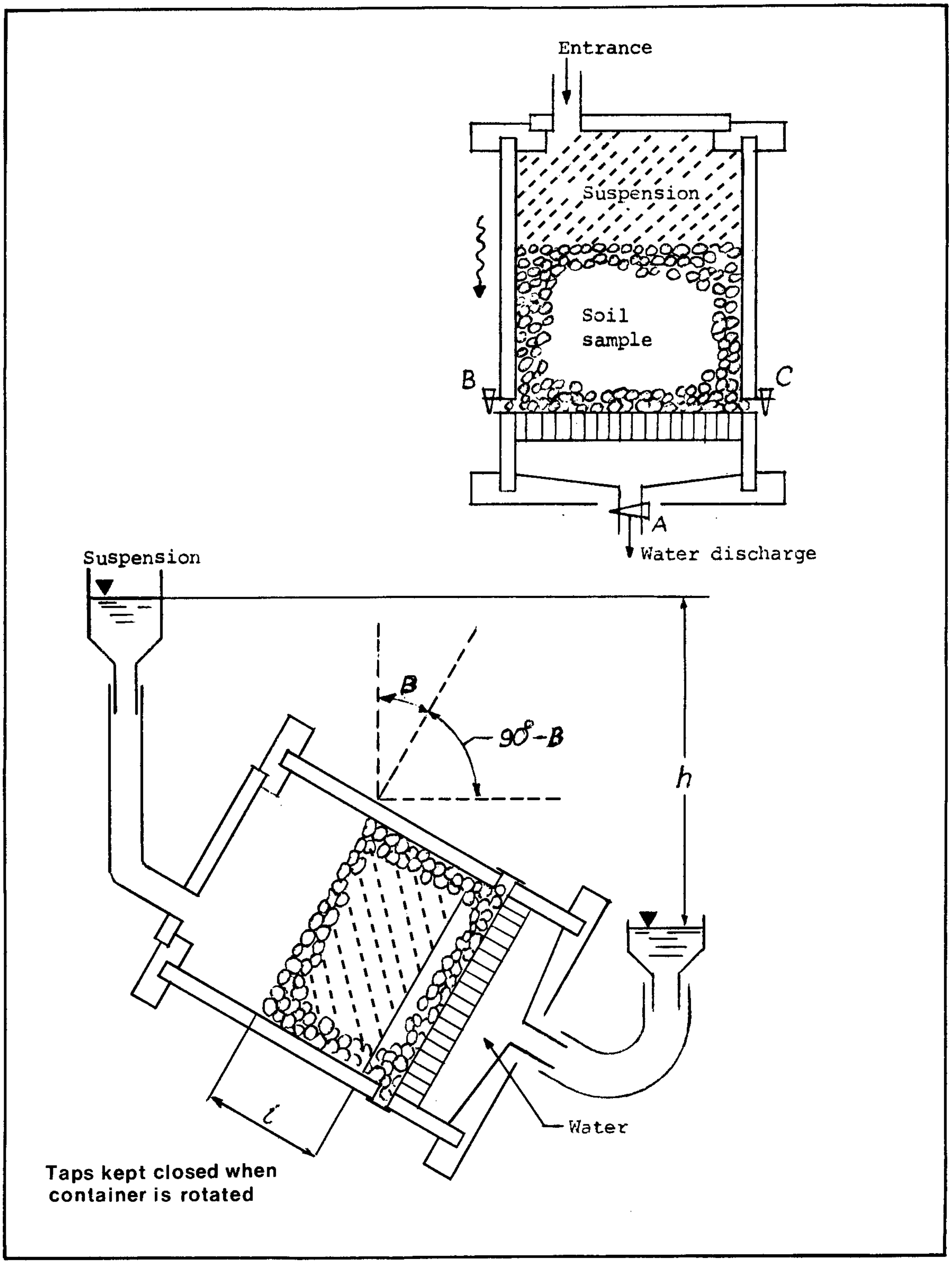
FIGURE 1

Formation of filter cake. (a) Disposition of colloid fraction in the soil voids; (b) filtration of slurry; (c) formation of impermeable film along the face.



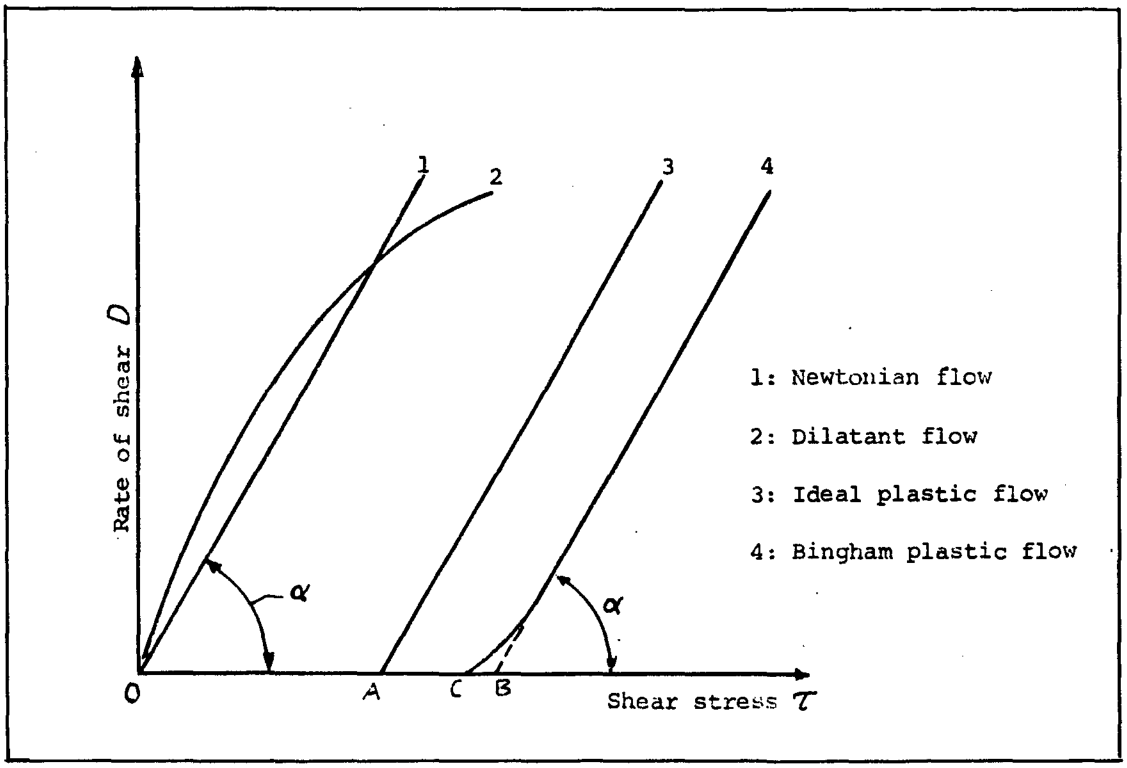
Fluid loss during, and after filter cake formation (from Hutchinson et al, 1974).

FIGURE 2



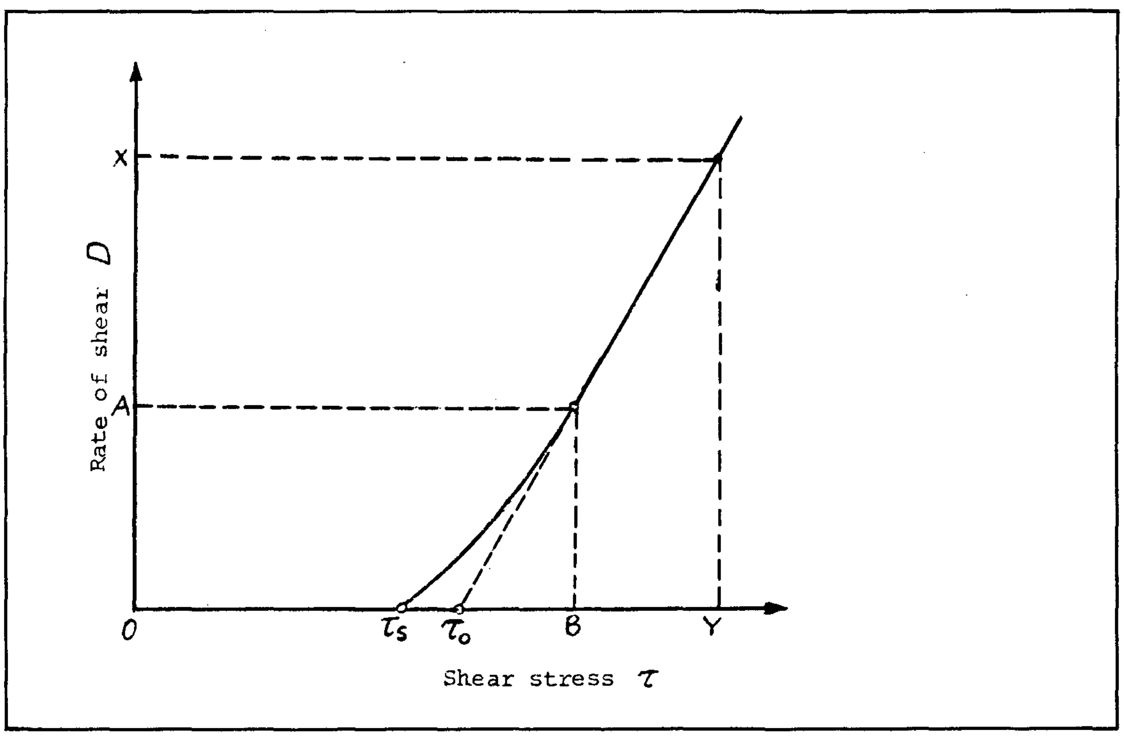
Peel off test to investigate stability of individual grains at various angles of inclination.

FIGURE 3



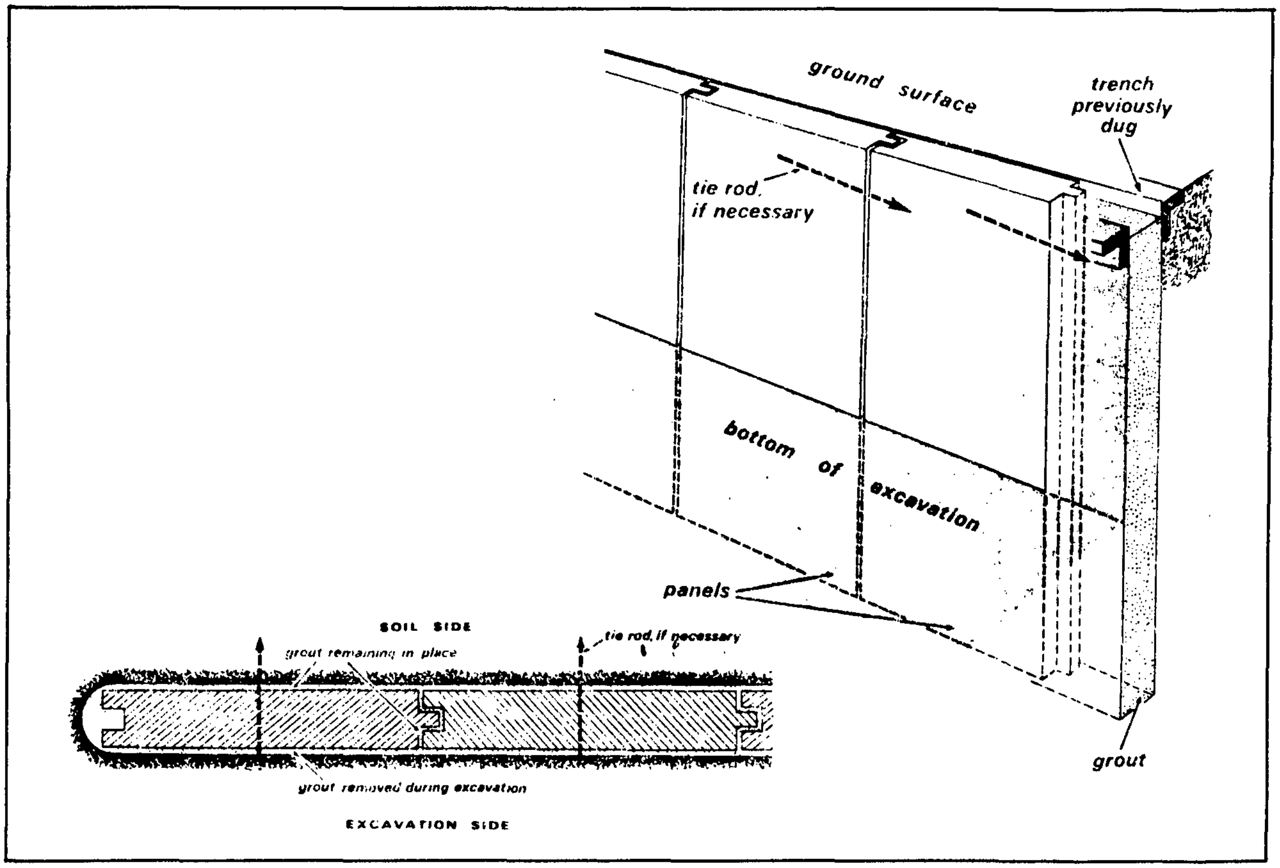
Relations between shear stress and rate of shear.

FIGURE 4

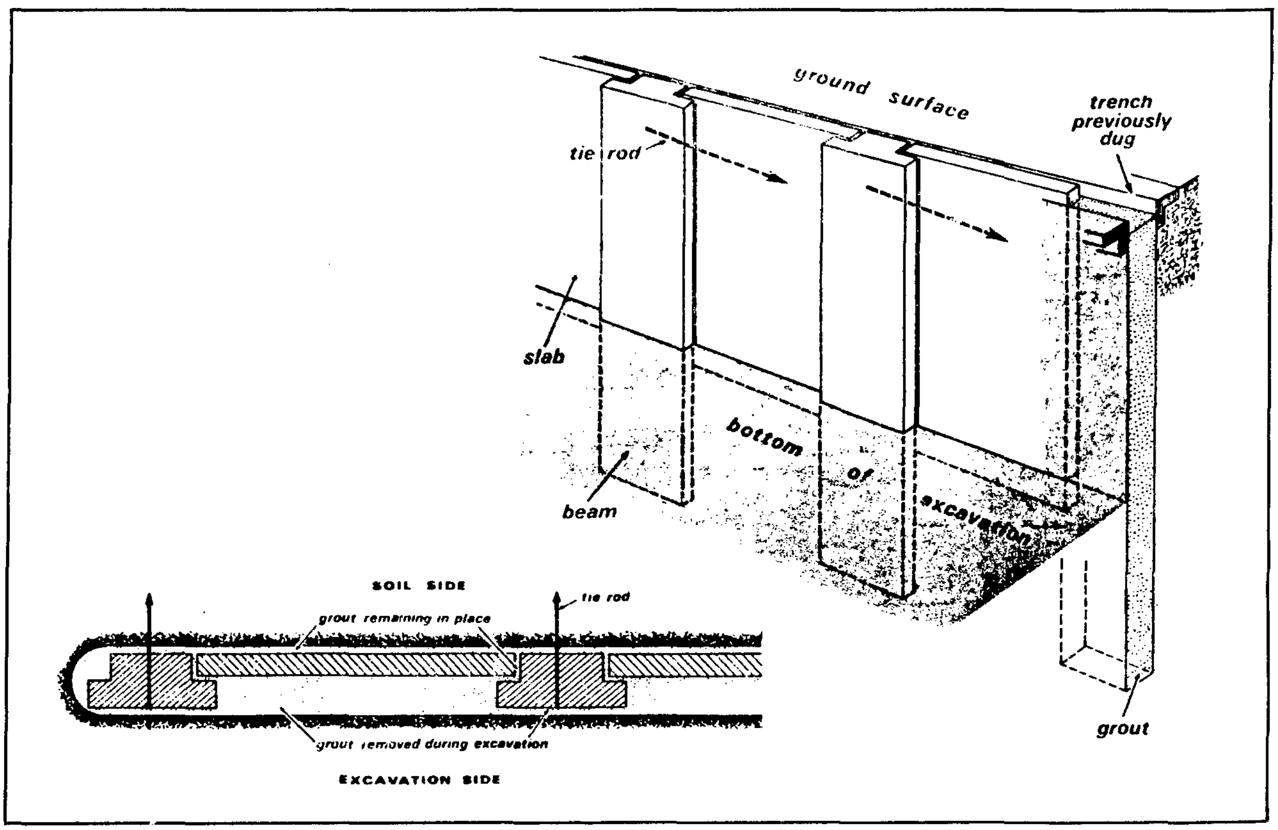


Flow curve for a Bingham fluid.

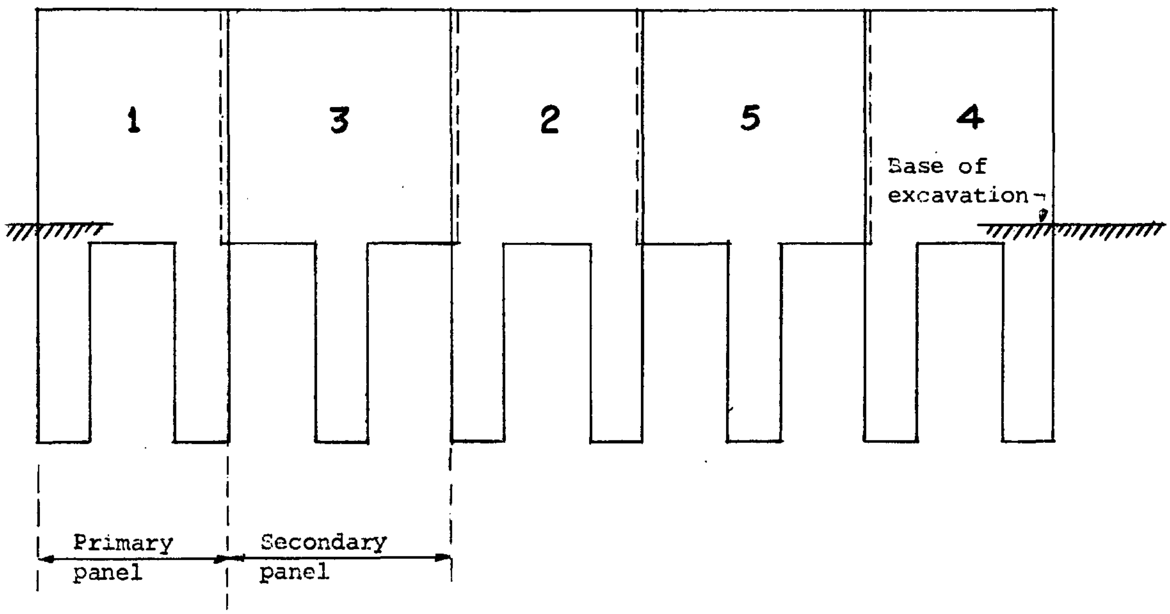
FIGURE 5



Prefabricated wall with identical panels. **FIGURE 6**



Prefabricated wall with beam-and-slab panels. **FIGURE 7**



One primary panel = two beam sections and one slab section

One secondary panel = two slab sections and one beam section

Installation of a prefabricated wall consisting of beam and slab sections.

FIGURE 8

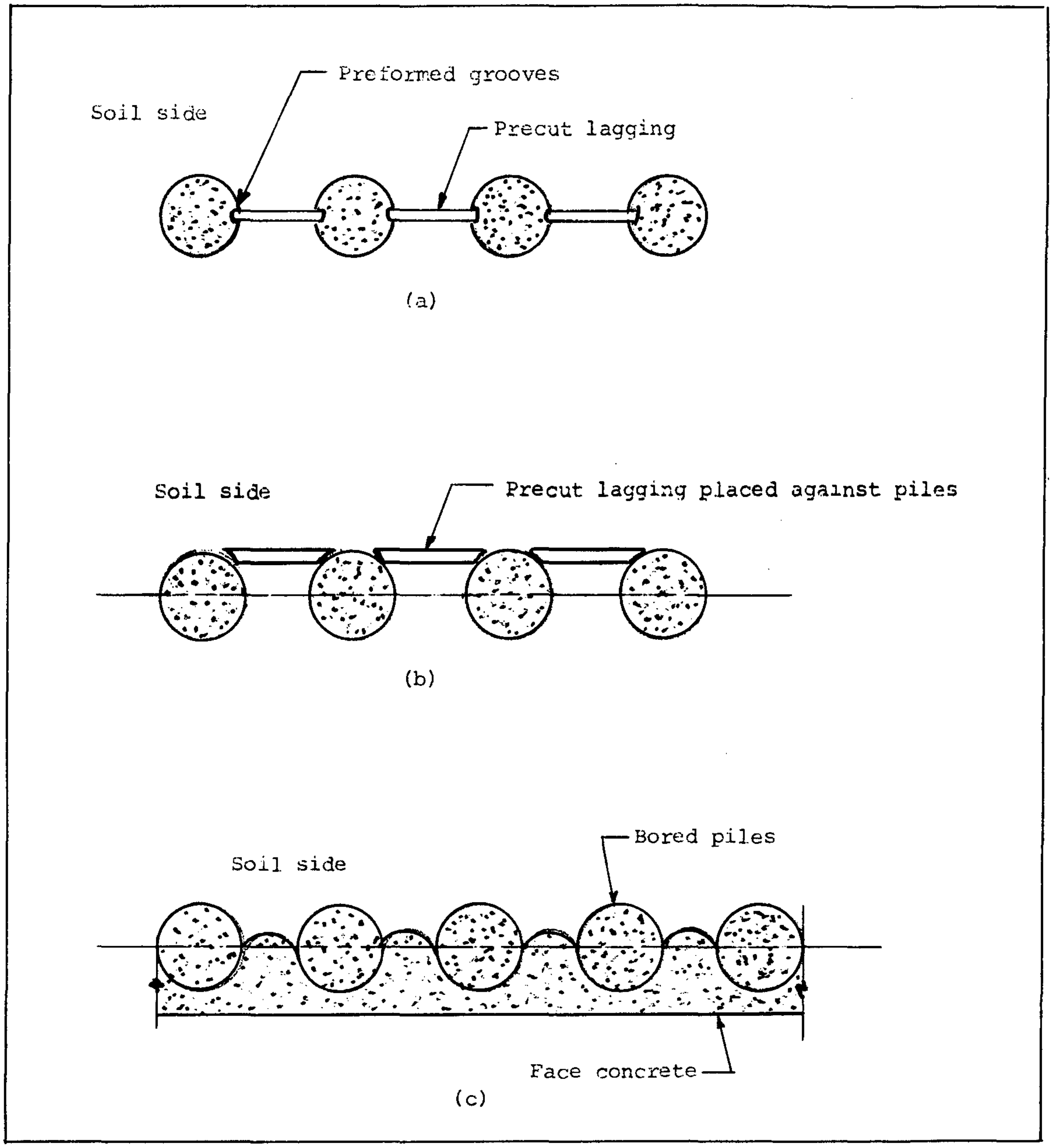
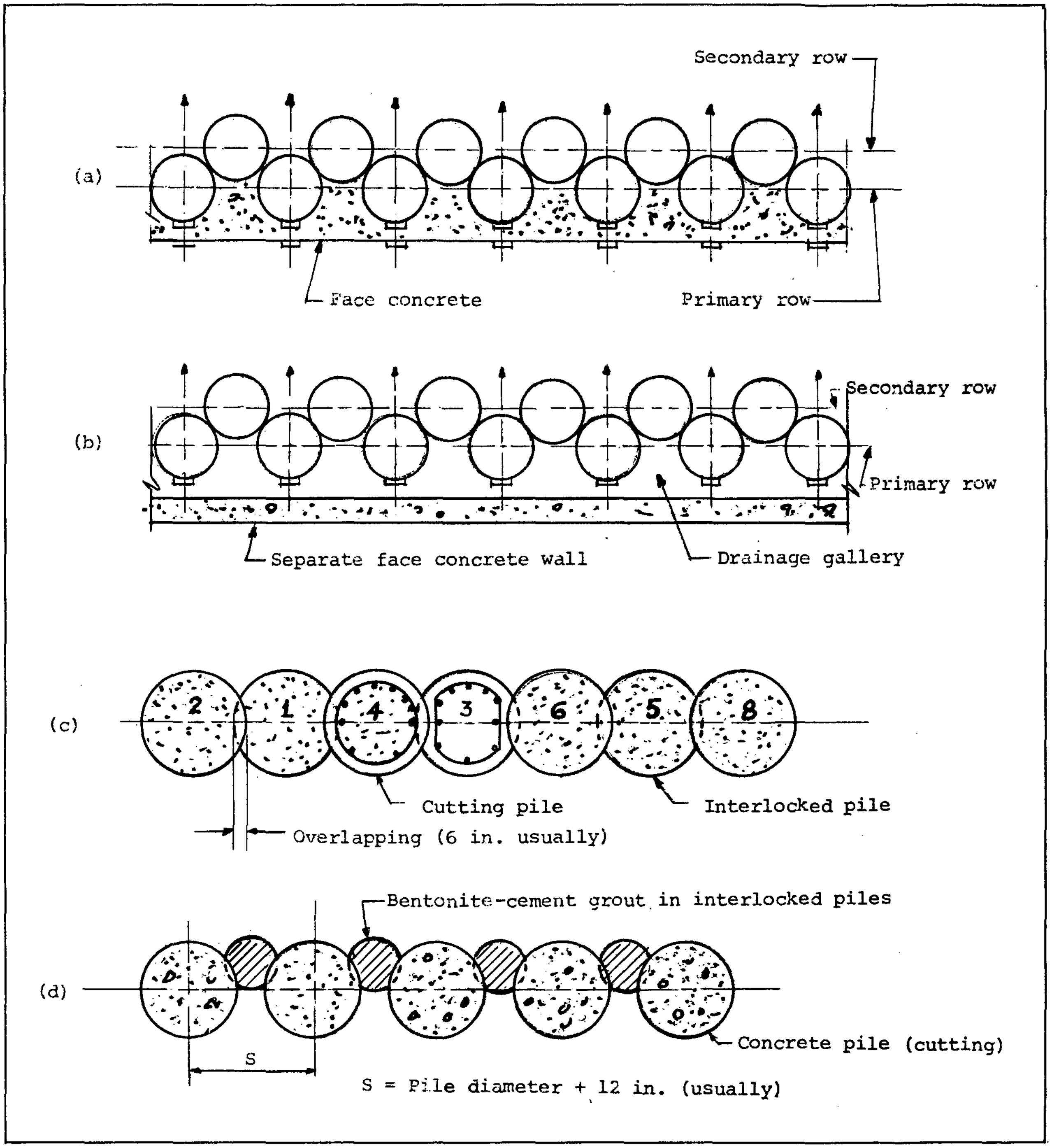


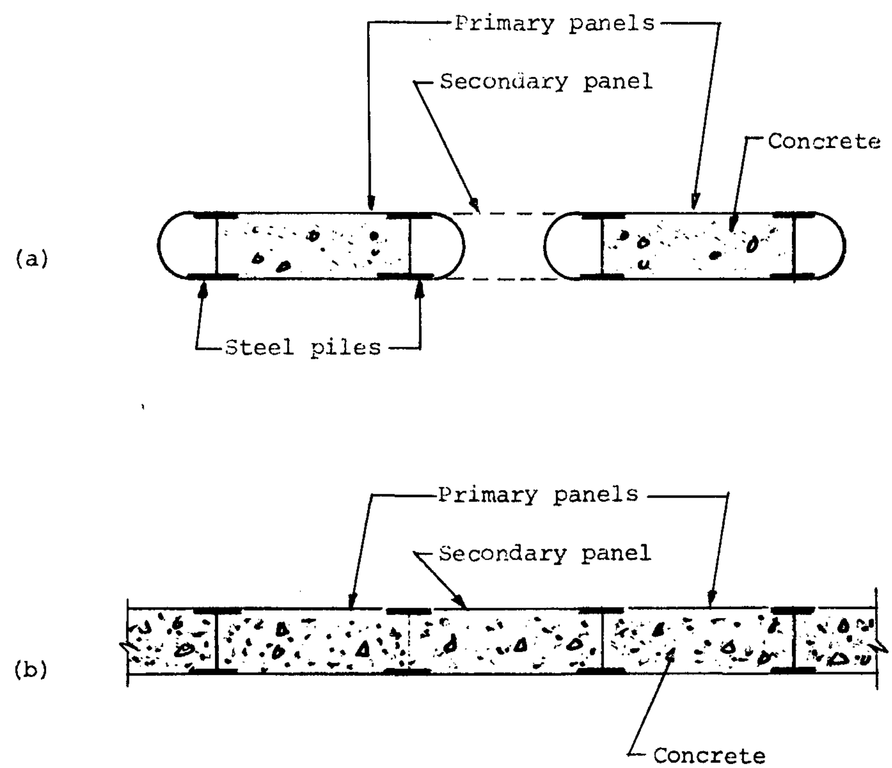
FIGURE 9

Various types of bored pile walls. (a) Wall with lagging inserted in performed grooves (made by positioning metal forms or foamed plastic strips); (b) wall with precut lagging placed against piles as the excavation is carried down; (c) bored piles with a separate concrete face wall.



Contiguous bored pile walls. (a) Tangent wall with a concrete face cast against the piles; (b) tangent wall with a separate concrete facing for seepage and drainage control; (c) secant wall formed with equal diameter piles; (d) secant piles of different materials.

FIGURE 10



Typical composite wall; (a) outline of excavated panels; (b) finished wall.

FIGURE 11

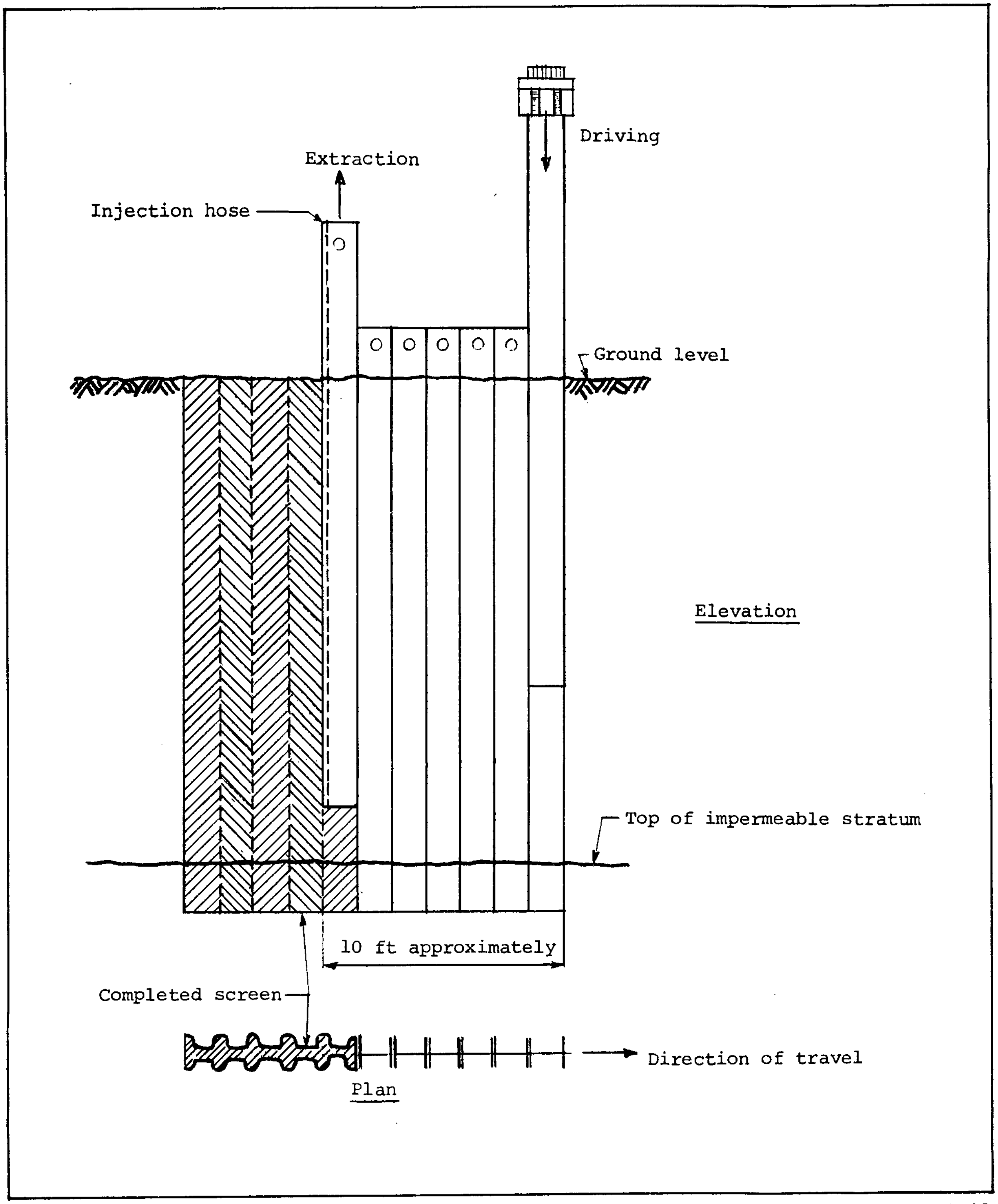
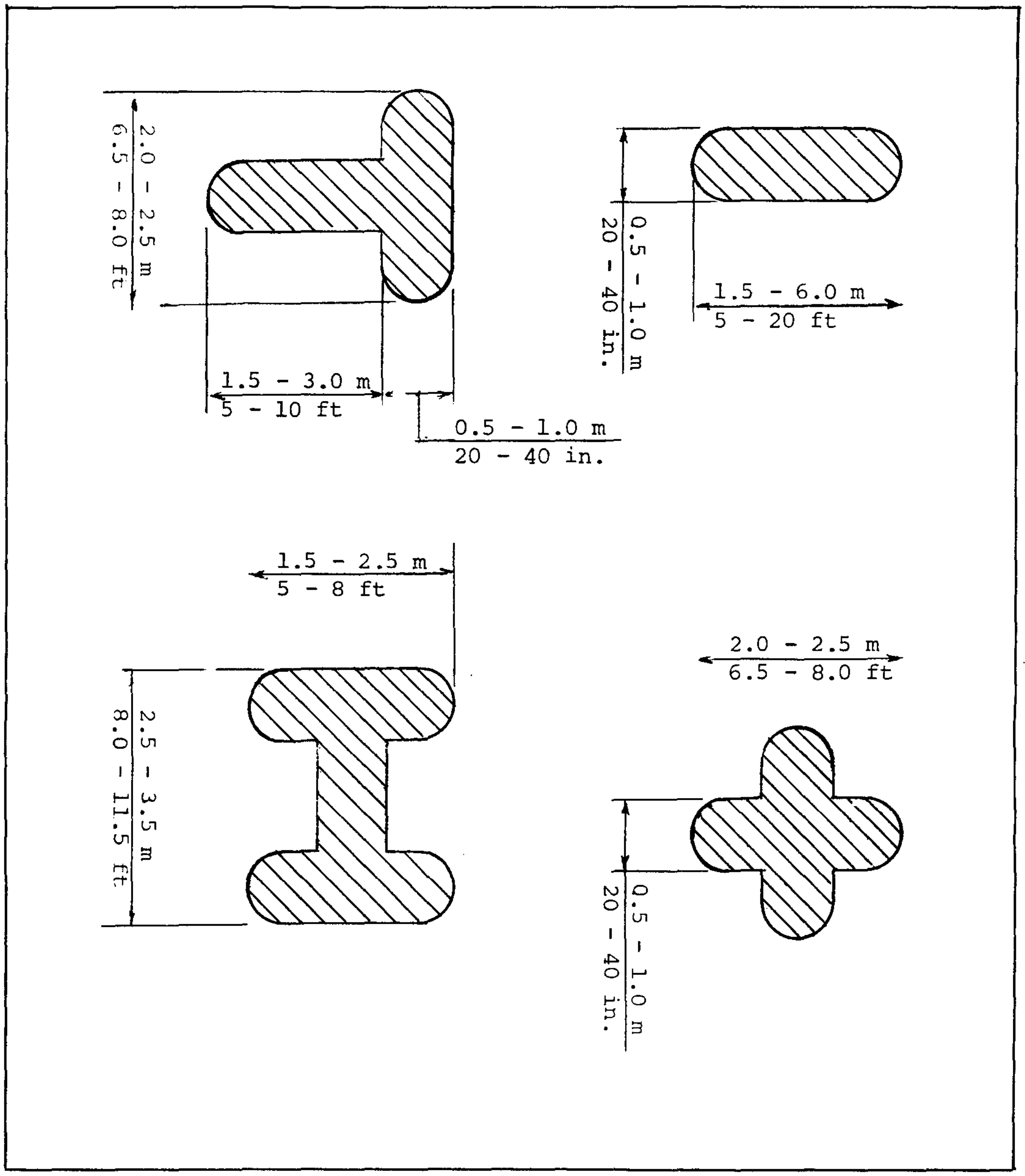
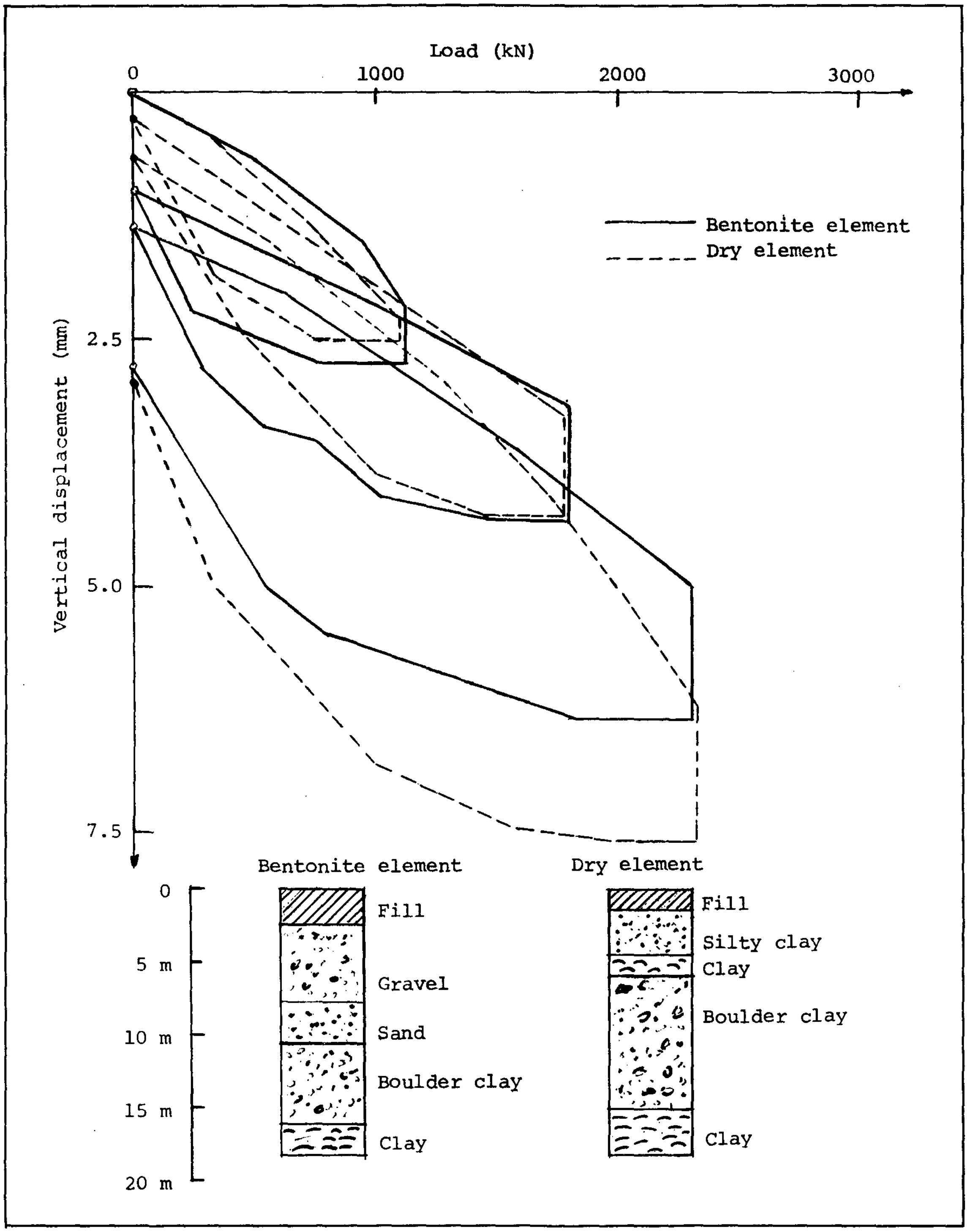


FIGURE 12

Method of installation of an injected grout screen.



Cross Sections of typical load bearing elements. **FIGURE 13**



Load test comparison of two piles 600 mm diameter on same site, one under bentonite and the other dry (from Sliwinski and Fleming, 1974).

FIGURE 14

TABLE I

COMMON SLURRY PROPERTIES

Property	Definition of measurement	Current test method
Concentration	lb of bentonite/100 lb of water kg of bentonite/100 kg of water lb of bentonite/ft^3 of water	- - -
Density	Mass of given volume of slurry	Mud balance
Plastic viscosity Apparent viscosity Yield stress	For a slurry behaving as a Bingham body, the flow law is $\tau = \tau_0 + \eta_p D$, where τ = shear stress τ_0 = yield stress η_p = plastic viscosity D = rate of shear apparent viscosity = τ / D	Fann V-G viscometer
Marsh cone viscosity	Time for 946 cm^3 (1 U.S. quart) of the 1500 cm^3 volume to drain from a standard cone, or time for 500 cm^3 of the 500 cm^3 volume to drain from a cone (Japan)	Marsh funnel viscometer
Marsh cone gelation	Time for the remainder of the 1500 cm^3 to drain from same cone	March funnel viscometer
Initial gel strength	Minimum shear stress to produce flow designated as τ_s	Rotational viscometer
10 minute gel strength	Shear strength obtained by allowing 10 minutes to elapse between stirring and reading	Rotational viscometer
p^H	Logarithm of the reciprocal of the hydrogen ion concentration	p^H electrometer, p^H
Filtration or fluid loss	Volume of fluid lost in a given time from a fixed volume of slurry when filtered at a given pressure through a standard filter	Filter press test, but this procedure does not permit exact estimation. Stagnation gradient test is more appropriate
Filter cake	Thickness and strength of filter cake for standard or actual conditions	Thickness measured in fluid loss test, strength estimated from triaxial tests
Sand content	Percentage of sand greater than 200 mesh in suspension	API standard sand content test using a sand-screen set

TABLE 2

CONTROL LIMITS FOR THE PROPERTIES OF SLURRIES

Property / Function	*Average Bentonite concentration (%)	Density and specific gravity (lb/ft^3)	Plastic viscosity (cP)	Marsh cone viscosity	10 min gel strength (Fann) ($lb/100\ ft^2$)	p^H	Sand content (%)
Face support	$>$ 3-4	$>$ 64.3 (1.03)	-	Limits established by soil type	**	-	$>$ 1 (optional)
Sealing process	$>$ 3-4	-	-		-	-	1
Suspension of detritus	$>$ 3-4	-	-		$>$ 12-15	-	-
Displacement by concrete	$<$ 15	$<$ 78 (1.25)	$<$ 20		-	$<$ 12	$<$ 25
Separation of noncolloids	-	-	-		-	-	$<$ 30
Physical cleaning	$<$ 15	$<$ 78 (1.25)	-		-		$<$ 25
Pumping of slurry	-	-	-		Limits vary	-	-
Limits	$>$ 3-4 $<$ 15	$>$ 1.03 $<$ 1.25	$<$ 20		$>$ 12-15	$<$ 12	$>$ 1 $<$ 25

* Should be expected to vary widely because of different bentonite brands.

** The shear strength of filter cake is more applicable to peel off control (also the time required for its formation.

Controls are not considered necessary for apparent viscosity and yield stress. Whereas fluid loss commonly is judged by standard filtration test and a max. film thickness of 2 mm, better control limits are established by stagnation gradient tests.

REFERENCES

Hutchinson, G. P. et. al., "The Properties of Bentonite Slurries Used in Diaphragm Walling and Their Control," Conference on Diaphragm Walls and Anchorages, Institute of Civil Engineers, London, 1974.

Jefferis, S. A., The Composition and Uses of Slurries in Civil Engineering, University of London, 1972.

Sliwinski, A. and Fleming, W. G. K., "Practical Considerations Affecting the Construction of Diaphragm Walls," Conference on Diaphragm Walls and Anchorages, Institute of Civil Engineers, London, 1974.

Xanthakos, Petros P., Slurry Walls, McGraw-Hill, New York (in print), 1975.

PAPER 15

Analytical Modeling of Diaphragm Walls for Excavation Support

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PAPER 15

ANALYTICAL MODELING OF DIAPHRAGM WALLS FOR EXCAVATION SUPPORT

by

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INTRODUCTION

The title of this presentation is Analytical Modeling of Diaphragm Walls. The unfortunate usual connotation of the phrase "analytical modeling" is of something associated with machine generated solutions of problems which have little to do with reality. A typical scenario in this case has the speaker rising, a few analytical types sharpening up their attention span and the rest of the audience going out for coffee or sleeping with their eyes open. However, what I would like to present hopefully avoids this stereotype. Instead of delving into analytical techniques, per se, I would like to show, via examples, what can be learned from analyses of the diaphragm wall problem and how this type of information can be used to improve decisions which require a substantial amount of engineering judgement. First, a short, preliminary discussion is needed to define the problem we are attempting to model and the means available for modeling.

DEFINITION OF DESIGN PARAMETERS

Design parameters associated with the diaphragm wall are as follows:

1. Stability of the slurry trench
2. Stresses in bracing or tie-back elements
3. Moments and shears in the wall
4. Earth pressures on the wall
5. Water pressures on the wall
6. Movements of the wall and soil system
7. Required embedment

The following discussion will concentrate on items (3) through (7) because more confusion seems to exist concerning them. Analysis techniques in these cases are also more variable, leading to a diversity in "answers" for any given problem.

FACTORS WHICH INFLUENCE BEHAVIOR

In the last section the parameters important to design were defined. The influence which can be exerted upon these parameters in design and the accuracy required of a design analysis is a function of the types of factors which influence behavior. Some of the more prominent factors which influence behavior of diaphragm walls are:

1. Initial construction constraints
2. Construction sequence
3. Weather
4. Potential legal problems
5. Surcharge loadings behind wall
6. Labor problems
7. Time
8. Geology
9. Soil properties
10. Ground water
11. Wall characteristics (stiffness, permeability and continuity)
12. Support system (number, stiffness, inclination)
13. Connections
14. Adjacent structures
15. Prestress loads

A review of this list reveals a rather surprising fact; namely, that about seven of the parameters which can influence behavior cannot be completely assessed during the design period. For example, construction changes which occur after design is completed may significantly affect behavior but usually cannot be anticipated. It requires careful planning, scrupulous control, and good communications to avoid detrimental influences from such events. In design analysis this means that adequate factors of safety should be used, and that the predictions from analysis be used more as guides and not exact information. The other seven or so parameters which can be defined during design can be manipulated by the designer to effect the safe and economic design of the system. It is to utilize these parameters to best advantage that analysis should be directed.

ANALYSIS TECHNIQUES

Analytical modeling of the diaphragm wall problem can follow a number of different approaches. There are basically three different categories, (1) conventional, (2) discrete element, and (3) finite element. The conventional approach involves analyzing each design problem as if it were a separate entity and not coupled with the rest of the problem [9]. For example, the wall is usually treated as an isolated member loaded by earth and water pressures which are assumed by an empirical or semi-empirical method. The reverse influence of the wall and its support system on the earth loadings is only marginally considered. Also structural deflections are usually not calculated and soil movements are estimated by judgement. This conventional type of technique is attractive because of its very simplicity. Unfortunately, there is no way to evaluate the actual degree of safety of a system so designed nor to effectively compare different system capabilities insofar as reducing soil or structural movement.

The discrete element formulation provides answers to some of the problems with the conventional approach. Here the structure is represented as a finite element system which is supported by a bed of springs, the springs representing the soil [6]. This method allows for interaction effects between the soil and the structure, but unfortunately introduces new problems. Foremost among these is the definition of artificial spring constants for the soils. Additionally, no capabilities exist to simulate effects of construction sequence or surcharge loading and soil movements are not predicted.

The finite element method however, with proper adaptations for the excavation problem, serves to resolve most of the analytical difficulties which exist for the discrete element solution. It can be programmed to realistically model soil behavior, construction sequences, foundation nonhomogeneity, and soil-structure interaction. The solution yields estimates of all important design parameters, including soil deformations. This writer and others have used the method in numerous studies of hypothetical, model and full-scale walls [1, 2, 3, 4, 5, 9, 10, 11, 12, 13, 14] and results have been shown to be acceptably accurate. Costs of the analyses are not unreasonable for a moderate size project. Time required to perform the analyses is not a problem since a design study can be completed within the span of about two weeks. The primary desadvantage of the method lies in its inaccessibility to the average engineer.

All three of the modeling techniques mentioned will find applications in future design with the conventional method being the most widely used. It also now seems clear that in certain cases the finite element method will prove especially useful in (1) comparing the performance of different systems; (2) developing more economic and rational designs; (3) providing solutions where problems go beyond the state-of-the-art; and (4) providing predictions of deformation fields in the soil mass which can be used to supplement and help analyze field instrumentation results.

THE "BIG" QUESTIONS

For this presentation analysis perhaps best serves design in answering questions which are difficult to tackle within the day to day time constraints most engineers work under. In the following discussion, analysis for some everday common sense* will be used to address questions which seem to commonly surface which have an important bearing on excavation system performance. These "big" questions are:

1. What is the influence of prestress loads on movements?
2. How does wall stiffness affect movements?
3. What is system stiffness and its influence on movements?
4. What is the earth pressure distribution on diaphragm walls?
5. How do water pressures affect design?
6. What is the best method for predicting moments and shears in walls?
7. What is the best technique for predicting movements?

---

* Analysis and common sense should be used together contrary to what seems to often occur.

INFLUENCE OF PRESTRESS LOADS ON MOVEMENTS

It is generally accepted as a fact that prestressing of the braces or tie-backs of a support system reduces movement. However, it is not often clear how much prestress load nor what distribution should be used. Recently published data from finite element analyses [4], field data studies [2] and model tests [7, 8] are in general agreement on this question. First, increasing prestress loads reach a point of diminishing returns. The largest benefit of increasing prestress load seems to be obtained by using a resultant load only slightly larger than that proposed by Peck [9] for design loadings on braced walls. The Peck [9] loadings actually are slightly larger than those of the conventional Rankine or Coulomb active earth pressure analysis. Thus, prestressing above these values in effect limits the development of active conditions in the soil and thereby limits movements.

Optimal design distributions for prestress loads appear to be trapezoidal or rectangular shapes, not triangular [4, 7]. Triangular distributions lead to higher forces on lower struts or tie-backs where these forces are then opposing the movement of a large mass of soil, a mass usually large enough to be unaffected by prestress loads. Trapezoidal or rectangular distributions however lead to higher loads near the top; in the early stages of excavation, where significant movements can occur, these forces can be effective in restraining the movements of the smaller soil mass involved in the deformations.

EFFECT OF WALL STIFFNESS ON MOVEMENTS

As with prestress loads, it is generally accepted that increasing wall stiffness decreases movements, but quantitatively defining how much is more difficult. Finite element results suggest that relatively large increases in stiffness are required to effect moderate reductions in movements [4]. In one example of a 30 foot cut in clay, an increase in wall stiffness of eight times (from a street-pile to a diaphragm wall), resulted in a reduction in movement of about 40%. It seems likely that in practice even larger reductions might occur since (1) there are no gaps between the slurry wall and the soil which lead to movements, as can occur in a soldier pile and lagging wall; and (2) large amounts of water will not escape through a slurry wall which can lead to consolidation of the soil whereas this can occur in other wall types. Practical behavioral differences such as these may be the most significant reason for a reduction of movements by slurry walls; the effect of wall stiffness is often overemphasized, particularly when slurry walls are depicted as "rigid."

SYSTEM STIFFNESS

The excavation support system involves not only the wall but also the support elements such as braces, wales, rakers, and tie-backs, and connections between all of the elements. Effective reduction of movements requires that the entire system stiffness be considered, not just the wall, a fact commonly overlooked when the slurry wall concept is being sold. For example, increasing tie-back stiffness has been shown in finite element analyses to have, in some cases, as significant an effect in reducing movements as increasing wall stiffness (4). It is apparent then that the positive effects of using a relatively stiff diaphragm wall can be negated by using flexible supports or poor connections. The concept of system stiffness should take precedence over that of wall stiffness.

EARTH PRESSURE DISTRIBUTION ON DIAPHRAGM WALLS

A myth which needs to be dispelled is that there is one earth pressure distribution for diaphragm walls which is different from that of other walls. The actual distribution has been shown in model test and finite element studies to be a function of many factors, not just wall stiffness. The most important factor is probably prestress load, not wall stiffness. The effect of prestress load depends strongly on whether or not the total load transmitted to the wall is greater than the active pressure resultant. If it is, and it usually is for tied-back walls, then the earth pressure loading problem is akin to that of the slab-on-elastic foundation problem. The earth pressures on the wall will reflect the distribution of the prestress loads but is somewhat influenced by spacing of the loads and the wall-soil stiffness ratio [13]. It is generally conservative to assume the earth pressures are the same as the prestress pressure diagram.

If the resultant of the prestress loads is less than that of active loading conditions then the problem is different from the foregoing case since the prestress loads will not prevent active loadings from developing. However, the system stiffness may be adequate to minimize movements to the point of preventing active loadings from fully developing. In this case the earth pressures will likely be distributed triangularly and lie between at-rest and active pressures. Some reduction from at-rest will occur since only small movements are needed to drop pressures all the way to active values. In fact, if the system stiffness is not adequate, the wall cannot be made stiff enough to prevent active loadings from developing.

WATER PRESSURE EFFECTS

Attention of designers is often focused on earth loadings on walls; water pressures are seldom mentioned in the literature. In fact, water loads can be larger than earth loads and in many cases are a significant factor, especially for impervious diaphragm walls. In such instances, nuances concerning small changes in shape of the earth pressure diagram become unimportant.

Water loads most commonly need to be considered in silts and sands. In clays, water effects are implicitly accommodated through the use of total unit weights in calculations of earth pressures. For silts and sands, however, effective unit weights should be employed in earth pressure calculations below the water table. Water pressures must then be explicitly added to the loading.

In cases where the wall is impervious and the underlying soils or rocks do not allow downward seepage, the water loading will by hydrostatic. The diaphragm wall with well-sealed joints should be near impervious, but many soils and rocks allow seepage to develop under the wall with time. Downward seepage will lead to a reduction in water loadings but increased earth pressures. The increase in earth pressures is less than the decrease in water pressures, thus, the downward seepage effect leads to a reduction in loads on diaphragm walls. Neglect of this effect leads to overconservative design.

STRUCTURAL ANALYSIS OF DIAPHRAGM WALLS

Methods of analysis of excavation wall systems in the literature are varied. The simplest approach involves assuming plastic hinges to develop at each support point and analyzing the wall as a series of earth and water loaded simple beams which interact only at the support points. The earth and water loads are assumed based on experience and engineering principles. In this case only positive moments are calculated and they are a maximum at the center span between supports. Other procedures suggest using a continuous beam technique for the analysis, assuming that the beam is fixed at support points. This method yields smaller moments than those of the simple beam technique, moments which are negative and a maximum at the support levels.

Still less conservative results can be obtained by (1) using the continuous beam technique and assumed loadings but accounting for support flexibility, an item most important for tied-back walls; or (2) analyzing the system using a discrete element or finite element approach. The discrete element approach involves analysis of the wall as if it is a continuous beam which rests on a bed of springs which simulates the soil response. This allows consideration of wall-soil interaction effects and support flexibility (6). The finite element approach models the soil as a continuum and provides a better representation of the system than the discrete element approach. Analyses of diaphragm walls using finite element techniques have been developed and tested against full scale and model problems [1, 2, 3, 4, 5, 11, 12, 13, 14]. These analyses demonstrated very good correlation with observed behavior. Research in this area is ongoing at Stanford University.

In the opinion of the writer, however, the continuous beam analysis with assumed loadings and allowances for support flexibility is the best all-round approach for structural analysis of diaphragm walls in conventional settings. This analysis technique is more accessible than finite element procedures and generally reduces conservatism in analyses to an acceptable level. Where project sensitivity or cost justifies it, finite element analyses should be performed.

PREDICTION OF WALL AND SOIL MOVEMENTS

Methods for predicting diaphragm wall and soil movement are not generally available. Typically, estimates are made on the basis of past experience or using highly simplified models. Fortunately, because movements of slurry wall systems are often small, these estimates are not particularly important. This is not always the case, however, as recent observations of slurry wall excavations in San Francisco in deep soft clays have shown [2]. In instances where unusual conditions are encountered or the deformation of adjacent structures are particularly important, an accurate method of predicting movements is useful.

The finite element technique provides the only rational analytical approach to this problem. Comparisons of finite element predictions to observed behavior show that adequately developed finite element solutions can accurately model movements of the diaphragm wall supported excavations [2, 4, 14]. This writer has employed the technique in design studies of excavation walls (9); the results gave confidence in the use of a more economic design and yielded important data which was used to evaluate the instrumentation results obtained from the excavation. Costs of the studies were well within the project capabilities.

REQUIRED EMBEDMENT

Analysis of required embedment to prevent collapse of the system can follow conventional limit procedures. However, the embedment required to limit movements can be greater, especially in clays, and the finite element method is probably the only tool which can be used for this purpose. In a number of finite element studies of documented case histories the degree of embedment has been shown to be an important element in the amount of movement experienced by the system [3, 11]. Reductions of movements using greater embedment can sometimes be more effective than increasing wall stiffness.

CONCLUSIONS

The purpose of this presentation was more to demonstrate the uses of analytical modeling techniques rather than to delve into the techniques themselves. It appears that, at least into the near future, simplified analysis tools will provide the bulk of the design information for diaphragm walls. More sophisticated techniques can fill an important and growing role, however, in: (1) providing useful additional data where difficult engineering judgement decisions exist about designer controlled parametric influences; (2) developing a rational basis for more economic solutions; and, (3) providing solutions where problems go beyond the state-of-the-art.

The most likely technique to fill this role is the finite element method. It allows for the most realistic modeliing of the excavation system and yields predictions of all important design parameters in a single solution. Further, it has been developed to an immediately usable stage and has been proved to yield acceptably accurate predictions of behavior.

REFERENCES

1. Bjerrum, L., Clausen, C. J. F., and Duncan, J. M., "Earth Pressures on Flexible Structures -- A State-of-the-Art Report," Proceedings of the 5th European Conference on Soil Mechanics, Madrid, 1972, Vol. 2, pp. 169-196.

2. Clough, G. W., "Deep Excavations and Retaining Structures," to be published in the Proceedings, Conference on Foundations for Tall Buildings, Lehigh University, August 1975.

3. Clough, G. W., Weber, P. R., and Lamont, J., "Design and Observation of a Tied-Back Wall," Proceedings of the Specialty Conference on Performance of Earth and Earth Supported Structures, Purdue University, Vol. 1, Pt. 2, June 1972, pp. 1367-1390.

4. Clough, G. W. and Tsui, Y., "Performance of Tied-Back Walls in Clay," Journal of the Geotechnical Division, ASCE, Vol. 100, No. GT 12, December 1974, pp. 1259-1274.

5. Denby, G. M., "Temporary Berms for Minimizing Settlements Behind Braced Excavations," A Thesis Presented to Duke University in Partial Fulfillment of the Requirements for the M. S. Degree, 1975.

6. Haliburton, T. A., "Numerical Analysis of Flexible Retaining Structures," Proceedings, Journal of the Soil Mechanics and Foundations Division, ASCE, Proc. Paper 6221, November 1968.

7. Hanna, T. H. and Matallana, G. A., "The Behavior of Tied-Back Retaining Walls," Proceedings, Canadian Geotechnical Journal, Vol. 7, No. 4, 1970, pp. 372-396.

8. Hanna, T. H. and Kurdi, I. I., "Experimental Studies on Multi-Anchored Flexible Retaining Walls in Sand," Report, Department of Civil and Structural Engineering, University of Sheffield, Sheffield, England, 1974.

9. Murphy, D. J., Clough, G. W., and Woolworth, R. S., "Temporary Excavations in Vorved Clay," Journal of the Geotechnical Division, ASCE, Vol. 101, March 1975.

10. Peck, R. B., "Deep Excavations and Tunneling in Soft Ground," Proceedings, 7th International Conference on Soil Mechanics and Foundation Engineering, State-of-the-Art Volume, 1969, pp. 225-290.

11. Stroh, D., "Discussion - Performance of Tied-Back Walls in Clay," Journal of the Geotechnical Division, ASCE, Vol. 101, No. GT 8, August 1975, pp. 833-836.

12. Tsui, Y., "A Fundamental Study of Tied-Back Wall Behavior," A Thesis Presented to Duke University in Partial Fulfillment of the Requirements for the Ph.D. Degree, 1974.

13. Tsui, Y. and Clough, G. W., "Plane Strain Approximations in Finite Element Analyses of Temporary Walls," Proceedings, Conference on Analysis and Design in Geotechnical Engineering, Austin, Texas, Vol. 1, 1974, pp. 173-198.

14. Wong, I. H., "Analysis of Braced Excavations," Thesis Presented to the Massachussetts Institute of Technology in 1971, in Partial Fulfillment of the Requirements for the Degree of Doctor of Science.

PAPER 16

Analysis, Design and Installation of Tiebacks and Ground Anchors

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PAPER 16

ANALYSIS, DESIGN AND INSTALLATION OF TIEBACKS AND GROUND ANCHORS

by

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INTRODUCTION

The use of ground anchors has become widespread during the last five years or so, particularly with respect to the support of deep excavation retaining walls. Despite the many successful schemes where ground anchors have been used, little is understood at present of the mechanics of an anchor either singly or as part of a structure. Consequently design methods are approximate and must be confirmed by full scale field testing.

The development of anchors goes back to the last century at least. Frazer (1874) describes tests on wrought iron anchorages for the support of a canal bank on the London-Birmingham railway, while screw piles have been used to restrain floor slabs against flotation, e. g. Anderson (1900). During the 1930's anchors were successfully used by Coyne for the strengthening of the Cheurfas dam in Algeria. Following the demonstrations of Coyne improved techniques in the manufacture of high tensile steel wire and strand and improvements in grouting and drilling led to the post war development of ground anchors in France, Germany, Sweden and Switzerland and during the 1950's anchors were first used to support deep excavations. The use of ground anchors in Great Britain did not start until the mid-1960's. Today ground anchor use is common in most parts of the world for both rocks and soils. Construction techniques enable high capacity anchors to be produced in stiff clay soils as well as in fine sands and silts.

There are numerous applications for ground anchors which include slopes and cuttings, cliffs, dry docks, basement slabs, preconsolidation of soils, underpasses and marine structures, as well as providing reaction for pile tests.

ANCHORS IN USE IN U.K.

In the U.K. there are several different anchor systems most of which have been developed elsewhere and are constructed under licence. These systems may be classified as follows:

(a) a grout filled cylinder
(b) a cylinder enlarged by grout under high pressure
(c) a multi-belled cylinder

In general cement grout is used to transfer the tensile force in the tendon to the soil. It also provides corrosion protection for temporary anchors. All three methods can be used in clay and/or rock but method (b) must be used in granular soils where permeation of grout into the pore spaces is required. In special cases where granular soils are too fine to permit the required penetration of cement-based grouts, chemical grouts are used, but on a very limited scale at present.

With the Bauer Injection Anchor the same construction procedure is used in soil and rock. The 76-mm hole is drilled with hollow tubes, the drilling bit being tack welded to the leading drill tube and on completion of the hole remains in the ground. The tendon, usually a Macalloy bar, is introduced and screwed into the sacrificial end bit. The bit is broken away from the drill tubes, the hole washed out under pressure and grout pumped through the drill tubes. In cohesionless type soils the grout permeates the voids and passes up the hole around the outside of the drill pipes. When the grout reaches the surface via the outside of the tubes the drill tubes are withdrawn 10 to 15 cm and this process repeated over the fixed anchorage length. After grouting the anchorage length, the remaining free anchorage length is grouted at low pressures. In fine sands the grouted zone may be double the hole diameter while in gravels a four-fold diameter increase may result. Corrosion protection comprises the tendon being coated with a synthetic adhesive resin, "Dicosol", and a plastic sheath is then shrunk around the tendon. For temporary anchors the bar is wrapped with "Denso" tape, a grease impregnated tape.

The Cementation Company install an alluvium anchor and a clay anchor. The alluvium anchor hole is advanced by a percussive drill but inside a rotating casing. This produces a 75 to 100-mm hole into which the tendon and a grout pipe are placed. The anchor is grouted as the hollow drill pipe is slowly withdrawn.

For the clay anchor a rotary rig with continuous flight auger is used to advance the hole. Where gravel overlies clay, a casing is used. After completion of drilling an expanding brush developed by Cementation is slowly rotated while water is circulated to remove soil cuttings. Thus a bell is cut. The process is repeated at 750-mm to 1-m intervals. The tendon is then installed and the shaft filled with grout and injected under a low pressure.

The Ground Anchors Ltd. hole is advanced by a rotary percussive rig which is kept free of cuttings by the circulation of compressed air through the drill stems. After the tendon and grout pipe has been installed grouting commences as the casing is progressively withdrawn.

The M.V. Anchor system is constructed by driving a tendon with an enlarged driving shoe by means of a diesel hammer. The tendon is pitched in the same way as a driven pile. As the tendon is driven grout is continuously pumped down to fill the space behind the driving shoe. When driving is complete grouting under pressure is continued for some time. The Soletanche anchor makes use of the tube a manchette method of grouting. The anchor hole is drilled by rotary rig or percussive methods. The hole is flushed clean with air or water and filled with grout under gravity. The anchor tendon with the P.V.C. tube a manchette packer, used to separate the fixed anchor from the free zone, is inserted. The packer is inflated. The double packer in the tube a manchette is aligned with the bottom series of ports and grout is pumped into the double packer under pressure. The process is repeated in stages by drawing the double packer up within the tube a manchette and grouting. After a period of 2 hours the process is repeated. Grout comsumption is carefully monitored.

The Universal Anchorage Company (UAC) installs two basic anchor types. The clay anchor comprises 2, 4 or 6 cones excavated in the fixed anchor length by and underreaming device. To achieve this, the anchor hole is drilled with a rotary auger using either a continuous flight auger (dry hole) or a drill bit on hollow rods. The underreaming device is then lowered to the bottom of the anchor hole. Two blades in this device are opended by mechanical means to enable the cones to be cut in the anchor hole wall. One or two further pairs of cones may be cut at a higher level in the anchorage zone. The tendon is then inserted and grout is injected at pressures of 300 to 800 kN/m^2 until it flows out at the working face. The sand and gravel type anchor is formed by drilling a hole using rotary or rotary percussive methods with a bit on hollow drill rods working within and just ahead of an outer casing. The tendon is then inserted and the casing connected to the drill head, grout being injected under pressures of 400 to 1200 kN/m^2.

A somewhat similar multi-belled anchor is constructed by Fondedile Ltd., but here all of the underreams are constructed in one operation. Quite naturally a higher torque capacity drill rig is required.

The anchor capacities which are achieved by the different systems described above vary depending primarily on the soil conditions and the grouting technique with safe working loads of up to 75 tons or more in stiff clays. In the early stages there were a number of failures when extending proven systems to new soil conditions. Each of the above systems has its particular choice of tendon material, stressing system and corrosion protection.

LIMITATION IN UNDERSTANDING OF ANCHOR BEHAVIOR

In use an anchor has to perform many duties. It must carry its design load with safety for the duration of the scheme of which it is a part. It must not corrode excessively. It must not "creep" too much. Because, in general, anchors have been constructed by specialist subcontractors and the site investigation programs have been directed to the needs of the main contract, it is not surprising that there are many questions for which we have few answers. Anchors stress the ground locally to high stress levels. Anchor construction methods may require sophisticated grouting techniques. If possible at the site investigation stage consideration should be given to the needs of a ground anchor project. In clays the more important factors are the undrained shear strength, the presence of fissures and discontinuities, boulders, and the potential of the clay to soften during drilling. With soft rocks data on permeability, strength and capacity to soften during drilling are necessary. In granular soils one might think that no problems occur and that the only need is to quantify the position of the water table and classify the soil. This is not so. Information on grain size distribution and degree of compactness are also required. Where the granular soils are fine grained there is also the need to ensure that careful sampling techniques are employed otherwise some of the fines may be lost. Very often it is these fines which control the success or otherwise of the anchor construction program.

When a structure such as a dry dock floor or a retaining wall is anchored, the anchors are placed close together and they are tensioned against the structure. The spacing may be such that the zones of stressed ground interact and "grouping" action results. Laboratory scale tests have shown that such grouping action results in a deterioration of the load carrying capacity of an average anchor in a group. How do we make proper allowances for this at the design and testing stage? The effects of prestressing an anchor against a structure are also unclear. It is known that prestressing modifies the state of stress in the soil mass and laboratory scale tests show that, in general, prestressing improves the performance of an anchor in sand. However there is much that we do not understand about this topic, particularly the effects which prestressing have on the overall stability of a multi-anchored wall.

In some situations anchors may experience slow or fast repeated load applications. Again there appears to be little data on this topic. Fairly sophisticated laboratory tests on both plate and cylinder-shaped anchors in sands of different overnconsolidation ratios have shown that repeated loading can, indeed, be significant and that after large number of repeated loadings failure may result. It appears that the most important factors are load level and change in loading. For most designs repeated loading is not a problem, but extrapolation from static conditions to conditions of repeated loading could be very unwise at this stage.

Because the anchor stresses the ground locally to relatively high stress levels creep must be expected both in sands and in clays.

Very often test anchors are placed in a vertical direction yet anchors on the site may be placed at very different inclinations. How does one extrapolate test data and experience from one inclination to another?

ISOLATED ANCHOR DESIGN FORMULAE

Most specialist contractors and consultants have evolved semi-empirical formulae based on experience with particular anchor systems in particular soils. Littlejohn has suggested the following:

Coarse Sands and Gravels:

$$T_{ult} = L\ n\ \tan \phi'$$

where L is the fixed anchor length in metres, n = 40 - 60 tonnes/m, ϕ' is the angle of internal friction and the anchor diameter is 400 to 600 mm, the fixed length < 4 m.

Fine to Medium Size Sands:

$$T_{ult} = L \quad n \quad \tan \phi'$$

where n' = 13 to 16 tonnes/m and is applicable in the range of anchor diameter 200 mm fixed length up to 4 m and a depth of overburden above the fixed anchor length of 6 to 10 m. Littlejohn suggests that the working load should not exceed 40 tonnes.

Stiff Clays:

When a smooth cylinder is drilled the adhesion which can be mobilized is 0.3 to 0.45 Cu. To increase this value pea gravel is occasionally injected into the hole sides to give

$$T_{ult} = \pi D L \quad 0.7 \overline{Cu} + \frac{\pi}{4} (D^2 - d^2) \; Cu_b Nc$$

where D and d are the diameters of the fixed anchor and shaft respectively, L is the length of the fixed anchor, Cu and Cu_b are the average undrained shear strength of the clay along the anchorage zone and at the top of the anchorage. For such anchors the working load is usually about 25 tonnes. Where a series of underreams are in use the carrying capacity is given by

$$T_{ult} = \pi D L \quad Cu \quad 0.9 + \frac{\pi}{4} (D^2 - d^2) \; Cu \; N_C \quad + 0.3 \; d \; \ell$$

where D is the diameter of the underream, d the shaft diameter, L the length of the fixed anchor, 'ℓ the length of the shaft and N_C = 9. For such anchors working loads are usually less than 75 tonnes.

From the above it will be seen that very simple formulae are in use at the present time. They do not make allowances for changes in carrying capacity especially in clays.

So far large diameter anchors have not been used in the U.K. although there appears to be many advantages. Recent field research has shown that small diameter underreamed piles may have some merit, but practical problems will limit their use on the majority of sites in the U.K. and elsewhere. In general what we require for most tied back structures is small load capacity anchors which can be formed reliably.

FIELD TESTING OF ANCHORS

There are three basic types of test to be carried out. First, it is necessary to establish whether an anchor system is fundamentally acceptable for use. In such a test, after the anchor has been load tested, it is exhumed to check that the fixed anchor is formed as specified, that the grouting is adequate and that the tendon is in a central position in the fixed anchor zone. General guidance on the execution of such tests is given in DIN 1425. Having shown that an anchor system is fundamentally sound tests may be required on site to ensure that the particular anchor chosen is appropriate. During testing the carrying capacity of the anchor and the residual displacement on unload are studied. The third type of test is a quality control test which is performed on every production anchor. Usually loads are applied in increments of the working load and the resulting movements measured. It is normal practice to load every anchor to the working load + 10 to 20% and to load every twentieth anchor to 1.4 or 1.5 times the working load. The results of these loading tests are then assessed.

Very useful guidance can be found in DIN 4125 Vols. 1 and 2, the French Code on anchor testing and in a number of Conference Proceedings such as Diaphragm Walls and Anchorages, 1974.

In addition to the routine field testing and evaluation of anchors there is a need for construction control. It is accepted that it is difficult to log the anchor holes accurately but I believe that if attention is given to the spoil brought to the surface during drilling and if some on site tests are carried out to quantify the ground, then our knowledge of anchor behaviour will improve. Control of grout is needed and perhaps small cubes or cylinders of grout should be taken for each anchor. Control of the quantities of grout pumped into the hole must be made otherwise heave of the ground can result, for example.

How accurately can anchor holes be drilled. On most sites this is not critical but where the anchors are long and closely spaced it is important. Tests to measure borehole slope and position may be required as well as tests to monitor the movements in the ground near to a test anchor.

OVERALL STABILITY OF TIED-BACK WALL

The development of ground anchors for retaining wall support has been influenced to a large extent by the work of Kranz who brought forward a practical method of designing bulkheads which has been applied to walls supported by ground anchors. The design of a multi-anchored wall may be considered in three parts. First, the anchors must be sufficiently long that they stitch the wall which retains the soil to the zone of stable ground behind the wedge which tends to slip into the excavation. The Kranz method and modifications of it enable this to be estimated with reasonable accuracy although Finite Element Idealizations of the complete excavation/wall/anchor system will, one day, be in general use. Secondly, estimates have to be made of the total pressures exerted by the ground against the wall. This includes the distribution of the pressures, their changes with time, the effects of repeated loading on the ground surface. Once the distributions of pressure have been arrived at, they have to be distributed to the rows of anchors. Here most engineers tend to follow the empirical procedures evolved for the analysis of strutted walls, although there are attempts to refine this empirical approach, e.g. Jack (1971). Thirdly, a stress analysis of the wall member is required to provide not only the stresses it will be required to carry but also a measure of the axial load. In certain cases much more sophisticated analyses may be necessary where the movements of the wall and the retained soil are required. Here use can be made of the Finite Element Method but many difficulties still have to be overcome in the modelling of the wall construction sequences, for example.

PERFORMANCE OF TIED WALLS

The construction of a multi-anchored wall causes changes in the stress state in the soil mass. Attempts are made to balance the horizontal stress changes by the anchor forces but it is not possible to balance the vertical stress change. Consequently the soil mass undergoes a complex three dimensional deformation causing redistribution of pressures on the wall during and after construction. Attempts are now being made by Mr. N. Arber to accurately "model" the construction of a wall using the Finite Element Method. Because of the great computational difficulties involved it is necessary to observe full scale and laboratory scale structures. At Sheffield we have concentrated on the laboratory work and have examined a large number of variables during the last 8 years. Our main findings are that:

1. The earth distribution on the wall is more trapezium shaped than triangular shaped, lending support to the view that such walls are somewhat similar to strutted walls in that their movement is restricted and consequently complete failure of the retained soil mass does not occur. It has also been found that surcharge loading and strip loading does not alter this trend of pressure distribution.

2. Wall and soil movements are small at shallow depths of excavation but increase rapidly as the excavation approaches the height of the wall. The magnitude of these movements is regulated by the inclination of the anchors and the vertical bearing capacity of the wall base. Overconsolidation of the sand, in general, reduced the movements.

3. With very flexible walls arching occurs between the levels of anchor support. It has been found that the moment reduction/wall flexibility relation proposed by Rowe for sheet pile walls applies provided it is suitably modified to allow for the effective height of the wall.

4. Two main problems have been highlighted by these tests. First we have little or no knowledge about how to check the load carrying capacity of the base of a wall such as a diaphragm type or contiguous bored pile supporting several rows of inclined anchors perhaps in association with an axial load at its top end. Secondly, it is still uncertain what the overall factor of safety of a wall is, how reliable is our method of anchor length selection, and whether a more economic and safer structure would result if the anchorage zone were extended up to the face of the wall. It should be noted that such a technique is followed in use of reinforced earth retaining structures. I am sure that in many of our anchor supported walls the anchorage zone may extend up to the wall face because of bad site control.

MONITORING OF TIED-BACK WALLS

Advances in field instrumentation in recent years have made it possible to monitor the performance of structures during and subsequent to construction with a high degree of accuracy. The main quantities to be measured are forces in the anchors and the wall member; water pressures in the retained soil; normal pressures and shear stresses on the wall; displacement of the wall and retained soil. Rarely is it possible to measure all of these quantities because of the resource involved. However, it is relatively simple to monitor movements at the ground surface in three dimensions. Forces in the tendons require load cells to be installed with remote reading facilities. Movements in the ground require inclinometers, extensometers, etc. while pressures in the soil and against the wall require quite difficult installation techniques to be followed. It must be appreciated that in a study of a tied-back wall system it may be necessary to instrument a large volume of ground extending several excavation depths away from the wall because the digging of an excavation causes stress changes over a large area especially in heavily overconsolidated clays.

CONSTRUCTIONAL FACTORS

The success of any anchoring project must depend on the control exercised on site. Difficulties do arise because of the variable and unknown nature of the ground. Difficulties encountered by the author include: (1) Inability of the anchors to carry the design load with safety. Very often this is due to the selection of the wrong anchor construction technique for the particular ground. Difficulties of this nature were encountered in the early anchor projects in London clay, for example. (2) Drop off in anchor load with time has been noted on numerous tied-back walls. This drop off is due to several causes which include redistribution of load within the anchor group; movements of the wall, creep of the anchor, consolidation of the retained soil mass. Losses are generally small and allowances, if necessary, can be made for them. (3) Permeation of grout along fissures and discontinuities in the ground. This can be overcome by control of the volume of grout pumped into the anchor hole. (4) Distress to adjacent property such as old sewers, water mains. Very often it is difficult, if not impossible, to guarantee that such structures can sustain the small movements that must occur during the construction of a shored excavation. (5) Inability of wall base to carry its axial load. Gross penetration of wall base can occur with sheet piles, soldier piles or diaphragm walls where the anchors are steeply inclined and inadequate toe-in of the wall has been provided. In my view the latter phenomenon is very difficult to quantify. (6) Continuing movement of a tied wall. In heavily overconsolidated clays it is possible for the wall system to keep on moving towards the excavation despite the presence of anchors. This is because the anchors are located in the zone of soil which has been subjected to a stress change. Problems of this nature are to be expected in the future where ground anchors have been used to shore deep permanent excavations in clay. (7) Accommodation of lateral movement of the anchor head. In some designs it may be necessary to allow for lateral movement such as might occur in a dry dock floor slab. Here the anchor head detail must be such that it can accommodate these movements. Techniques such as rocker heads, thick rubber sleeves and a plastic cement/bentonite grout have been used.

TOPICS FOR POSSIBLE DISCUSSION

1. Large diameter v small diameter anchors
2. Straight shafted v multi-belled anchors in clay
3. Site investigation requirements for an anchoring project
4. Installation, testing and monitoring

5. Limitations in overall stability analysis
6. Site problems and difficulties
7. Problems in soft rocks
8. Adequacy of Existing Codes of Practice

REFERENCES

Proceedings, Conference on Diaphragm Walls and Anchorages, Institution of Civil Engineers, London, 1974.

Proceedings, Conference on Ground Engineering, Institution of Civil Engineers, London, 1970.

Proceedings, 7th International Conference on Soil Mechanics and Foundation Engineering, Speciality Session No. 15, Anchorages, especially in soft ground, Mexico City, 1969.

Anderson, C. W., "Hydraulic Pile Screwing," Proceedings, Institution of Civil Engineers, Vol. 139, pp. 302-307, 1900.

Bauer, K. "The injection Bauer anchor system," Civil Engineering Journal, Zurich, No. 47, pp. 1265-1273, 1965.

Clough, G. W. et al., "Design and observation of a tied-back wall," ASCE Speciality Conference on the Performance of Earth and Earth Supported Structures, Purdue University, Vol. 1, Part 2, pp. 1367-1389, 1972.

Clough, G. W. and Tsui, Y., "Performance of tied-back walls in clay," Proceedings, American Society of Civil Engineers, Vol. 100, GT 12, pp. 1259-1274, 1974.

Frazer, T., "Experiments on the holding power of earth and the strength of materials," Corps of Royal Engineers papers. New Series, Vol. 22, pp. 80-89, 1974.

Hanna, T. H. and Matallana, G. A., "The behavior of tied-back retaining walls," Canadian Geotechnical Journal, Vol. 7, pp. 372-396, 1970.

Hanna, T. H. et al., "Anchor behaviour in sand," Proceedings, American Society of Civil Engineers, Vol. 100, GT 10, pp. 1107-1122, 1974.

Hanna, T. H. and Kurdi, I.I., "Studies on anchored flexible retaining walls in sand," Proceedings, American Society of Civil Engineers, Vol. 100, GT 10, pp. 1107-1122, 1974.

Hanna, T. H., "Foundation Instrumentation," Trans Tech Publications, 1973.

Kranz, E., "Ueber die Verankerung von Spund wanden," W. Ernst & Sohn, Berlin, 1939.

Littlejohn, G. S. et al., "Anchored diaphragm walls," Ground Engineering, Vol. 4, No. 5, pp. 14-17, 1971.

"Recommendations concernant la conception, le calcul, l'execution et le controle des tirants d'ancrage," Bureau Securitas, 1972.

DIN 4125 "Verpressanker für vorübergehende Zwecke in Lockergestein," Vol. 1, 1972.

PAPER 17

Diaphragm Walls and Secant Piles in Subway Construction

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PAPER 17

DIAPHRAGM WALLS AND SECANT PILES IN SUBWAY CONSTRUCTION

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INTRODUCTION

Over the last 12 years London Transport has carried out a number of projects where diaphragm walls, contiguous or secant pile walls have been formed from ground level. In every case the wall was used as the structural load bearing wall. The brief detail of some of these is as follows:

1. A diaphragm wall at Westminster for a station extension. This was sunk close to the foundations of an existing building (New Scotland Yard) and immediately outside and parallel to the brick arch sub-surface railway tunnel. The wall was used as the permanent structural wall and carried a considerable load from the building.

2. Diaphragm walls cast to form a sub-surface approach tunnel from Northumberland Park Depot to the Victoria Line. The site was fairly open at the commencement of the works and the walls were not very close to any buildings. Again the wall formed a permanent part of the structure. Some difficulty was experienced in this construction with the sides of the trenches peeling and causing blisters in the walls.

3. Contiguous bored piles used to form structural walls for underpinning the Merchant Navy War Memorial at Tower Hill. The piles were bored piles formed with a tripod rig driven through dry gravel into London blue clay. Some trouble was experienced with settlement of the monument while the piles were being formed adjacent to its foundation. The piles formed the structural wall of the tunnel and also carried the 1,500 ton load of the memorial.

4. Secant piles to form the upper machine chamber at Stockwell station (Brixton Extension to the Victoria Line). These were formed in water-bearing sand and gravel close to some substantial properties. Again they formed the structural wall and carried an overlying development.

5. Secant piles and diaphragm walls in the construction of the 3-1/2 mile extension of the Piccadilly Line to London Airport.

The most pertinent of these to the work to be carried out in Chicago is the recent Heathrow Extension tunnel construction and this paper will emphasize these works.

GENERAL DESCRIPTION OF THE HEATHROW EXTENSION WORKS

Work on the ground on this project started in April, 1971. The railway is now open to Hatton Cross, the intermediate station, and is expected to be open to Heathrow Central in 1977. (See Fig. 1)

The project is to extend the Picadilly Line from Hounslow West to a new station in the central area at Heathrow with an intermediate station at Hatton Cross, the total length of the extension being approximately 3-1/2 miles. Of this, 2-1/4 miles are basically cut and cover type of construction except for a short length where the railway comes into the open to cross a small river. The remaining length between Hatton Cross and Heathrow Central is in twin bored tunnels passing under the operational airfield. The central station is of cut and cover construction, although the railway here is at a depth of some 14m. Most of the cut and cover part is in a typical urban area, part residential and part commercial.

At Hounslow West the point of connection to the existing surface railway is just to the east of the existing station. This suits the new alignment and gives sufficient distance for the railway to gain depth to pass under the roads. This involved building a new station at Hounslow West, although for the time being the existing ticket hall is retained. The railway is then aligned on the south side of the Bath Road, being as far as possible located under the verge and service road.

At Henlys Corner the railway passes under the road junction on a 15 chain curve bringing it onto the north side of the Great South West Road where it is located partially under the carriageway. After a short distance, a reverse curve takes the line under a wide verge where the tunnel was built with minimum interference to traffic. The first tunnel section ends just west of Parkway and the railway then passes over the River Can and again into tunnel directly under the Eastern Perimeter Road, passing in front of the B.O.A.C. maintenance building and Champions Factory to Hatton Cross station situated just west of Hatton Road. The remaining length is under the operational airfield entering the central area just under the Queens Building. The alignment in the central area and that of the overrun tunnels is so designated that the railway could be further extended to Perry Oaks should the need ever arise.

ROUTE PLANNING

In planning the route of the cut and cover railway every effort was made to follow existing roads and, where possible, to site the new railway under verges and service roads, thus leaving the main carriageway as free as possible. At the Hounslow West end it was necessary to buy up some 20 residential properties to allow the construction of Hounslow West station. It should be noted, however, that on completion of the works there is now a substantial site which can be redeveloped with either houses, office or other commercial premises. The station is specifically designed to take some building loads. Apart from this there was no direct interference with any private property.

DESIGN CONSIDERATIONS FOR CUT-AND-COVER WORK (Excluding Central Station)

In designing the cut and cover work careful thought was given to the type of construction that would give the least interference to the public and residents whilst actually carrying out the work. Also, as the result of an undertaking entered into through the Parliamentary stages, it was necessary to design the railway to be mounted on track trays carried on rubber blocks to prevent ground-borne vibrations reaching the properties.

During the design stages several basic methods of construction were considered but the method chosen was believed to give the least possible surface interference. It was considered that only comparatively short lengths would be open at any one time and then the surface interference would be for the shortest possible duration.

The basic procedure was to construct secant pile walls from surface level using benoto piling rigs (Fig. 2). Secant piles are so called because they intersect each other during construction with the resulting advantage of a nearly water-tight joint. On the project, 890mm dia. piles were bored generally at 800mm centers giving a 380mm contact interface between adjacent piles. Initially, alternate piles only are cast, the intermediate male piles being completed on the following day when the concrete previously placed has set, yet is still green. Generally only the male piles are reinforced to avoid difficulties when cutting into the adjacent piles.

The method of operation of the piling rig is to force a steel casing into the ground by means of hydraulic rams; a secondary transverse ram imparts a twisting motion to the casing at the same time. A grab removes material from inside the casing while sinking progresses, the casing always being kept a little in advance of the grab. When the excavation reaches the required depth, reinforcing steel is placed in the casing and concrete is then poured in to form a normal insitu pile. As the concrete rises, the casing is gradually withdrawn, again with the twisting motion which leaves a distinctive zig-zag finish on the concrete.

In order to construct the walls it was only necessary to occupy the surface over the top of one wall at a time, although obviously the whole width was occupied where this was possible. Also, during this operation it was quite practicable to restore temporary crossing for householders to get their cars to their garages overnight. When both walls were completed a shallow excavation (about 5ft. on average) was taken down to the level of the top of the secant piles (Fig. 3) and a continuous capping beam constructed. The roof, which is designed to act as a prop between the walls, was constructed next using precast, prestressed, inverted 'T' beams with an insitu concrete infill and topping (Fig. 4). The waterproofing and protective concrete screed were laid next, followed by backfilling. At this stage the surface reinstatement was carried out and traffic restored (Fig. 5). All excavation and the rest of the construction was then carried out underneath the roof from selected openings left for this purpose (Figs. 6 and 7). By this method it was possible to restore the surface about 5 or 6 weeks after work in a location was started. In fact in certain localized sections, it was possible to restore the surface 4 weeks after work had begun. Obviously this only happened where it was necessary as a longer period enabled a longer length of roof to be completed in one operation.

Ideally the work progressed in sequence working along a section of tunnel. Two piling rigs worked first, one on each wall, this was followed by the excavation and a gang forming the capping beams. A further gang followed on with the roof construction while the final gang back-filled and reinstated the surface. In a length of about 100m of tunnel, all the operations could be seen taking place.

Apart from minimizing the duration of surface occupation, the benoto pile method had several other notable advantages:

1. It completely eliminated traditional sheet piling which is unacceptably noisy in built up areas.

2. The benoto piling rigs were extremely quiet in operation, especially after a second silencer had been fitted to each machine.

3. The method was extremely versatile in piling for road diversions and service diversions. It was possible to occupy a small area in the initial stages of the work, drive two short lengths of wall say 30ft. long, construct the roof on the walls and reinstate the surface. At this stage roads crossing the line of the tunnel and services could be diverted across this short length of roof giving freedom for constructing long lengths of tunnels between these short sections.

At the planning stage the initial concept was to use diaphragm walls (slurry trench walls) but after some consideration this was abandoned in favour of secant pile walls. There were a number of reasons why the decision was reached:

(a) On a long working site adjacent to a main road and footpaths the spillage of bentonite mud can be a major problem. An accidental spillage on the road could turn it into a skid pan.

(b) With nearly every process of forming slurry trench walls there is some splash from the machine. Footways have to be sited far enough from the machine to ensure that passers-by do not get splashed. With the benoto pile rig there is no such limitation.

(c) Because of the presence of a large number of Statutory Undertakers' services it was necessary to pre-construct a number of short lengths of tunnel. Also to get the best programme advantage, lengths of wall that did not interfere with existing services had to be constructed at an early stage. This meant that over the 1-1/2 mile length of cut and cover construction a considerable number of isolated lengths of wall were constructed. To move a slurry trench wall rig complete with its mud station is a major operation. To move a benoto rig, which is fairly mobile, is a comparatively easy operation

(d) With a benoto rig there is no need to preconstruct a guide trench; a pair of 12" x 12" timbers being quite sufficient. The time of occupation of a particular site is therefore shortened.

(e) The presence of underground services can be a great nuisance with a slurry trench wall construction. Certainly with the most usual 'grab methods' the presence of an old block of, say, telephone ducts surrounded with concrete will bring the rig to a halt. It is usually necessary to locate such obstructions and carry out a pre-excavation to remove them. With such things as sewers they may well be deep and their removal in water-bearing ground will be a problem. The benoto rig will bore through brickwork and any concrete that is likely to be surrounding services. In extreme cases a man can be sent down the pile with a concrete breaker. At Heathrow this was important as for about 500 yds. a nest of telephone ducts 14ft. deep and surrounded with concrete were along the line of one of the walls.

Another problem commonly encountered is the sudden loss of bentonite through an undetected obsolete sewer or service duct. Even known service ducts or sewers can be a great nuisance as to deal with them it is generally necessary to fill them with grout beforehand; not always an easy job. On the job at Westminster great trouble was experienced initially with bentonite seeping through the brickwork of the brick arch tunnels in considerable quantities; enough to flood the track and stop the trains. Before work could proceed it was necessary to drive lances in the ground and treat the back of the tunnel with a cement/bentonite grout injected into the ground.

(f) Some lengths of wall on the Heathrow Extension were very close to the foundations of some buildings (about 4ft. from the wall). With a slurry trench there is obviously some danger of settlement of a structure where the pressure bulb from the foundation is anywhere near the top of the trench where the hydrostatic pressure is small. This is probably particularly true for a plastic material such as medium soft clay. With the benoto rig as each bore is caused to the bottom and at no time is any ground unsupported, the danger of settlement of adjacent buildings during the construction of the wall is practically eliminated.

(g) On a cost basis the slurry trench wall is probably a little cheaper than a secant pile wall placed in open ground without obstruction. Tenders were received for both slurry trench walls and secant piling on the Heathrow job. On a straight comparison making no allowance for short length working, the benoto piles were 12% more expensive than the diaphragm wall. However, when all factors such as pre-excavation to remove obstructions, standing time, and construction of short lengths were taken into account it was apparent that the benoto piling was competative. Actual difficulties encountered during construction confirmed the assessment that in fact slurry trench walling would have been more expensive than costs actually incurred on secant piling.

The majority of these arguments did not apply to the walls for Heathrow Central Station which was in a defined site free of known services. Here slurry trench walls were used.

COST COMPARISON TO DEEP TUNNEL WORK

In the planning stage the question of whether to build a cut and cover railway or a deep tube railway to Hatton Cross was looked at very carefully. Even using modern construction methods, cut and cover work clearly results in greater surface disruption than bored tunnelling. In examining the initial proposals for a deep tube level there were also several severe disadvantages.

The first was that at Hounslow West, to gain depth and to bring the railway on its correct alignment, it would have been necessary to reconstruct the railway almost back to Hounslow Central. Another point which of course must be taken into account, is the fact that a deep tube station is not so convenient for passengers. Escalators or lifts have to be installed and the time taken from the train to the surface level is considerably increased.

A further consideration was that specialist tunnel labour, which is often in short supply, was not required for the cut and cover work except for a small gang to build sewer headings. In the event this was important as the cut and cover works coincided with the Fleet Line tunnelling works.

A cost assessment was also carried out and at that time it was estimated that to build in tunnel between Hounslow West and Hatton Cross would be 10% more expensive than building in cut and cover. This allowed for the cost of all the service diversions in the cut and cover work which amounted to about 1/7 of the cost of the civil engineering works. This figure was subsequently verified by comparing the actual cost of cut and cover work with the estimated cost of the tunnel work using rates extracted from the tender for the tunnel portion of the works under the operational airfield.

CONSTRUCTION TECHNIQUES FOR CUT-AND-COVER TUNNELS

One of the first problems to be faced in the construction of the cut and cover portion of the works near to and alongside main roads in a residential area was the presence of a very large number of statutory undertakers' services and Council sewers. The G.P.O. services were particularly difficult being the main routes leading to the airport. It was considered that the successful undertaking of the works would be largely dependent on very careful co-ordination and planning for the diversion of services. With this in view a senior engineer led a small section dealing only with service diversions. Because of the length of time required for some of the service diversions - the G.P.O. required two years in some areas - a lot of this planning was carried out prior to the letting of the contract and certain service diversion work was commenced some months before the main contract.

It was decided that a number of short lengths of tunnel wall and roof would be constructed in advance of the main work at suitable points and the services diverted over them thus leaving lengths of tunnel free for unimpeded progress. During the first year the main contractor's work was controlled entirely by service diversions. No fewer than ten statutory authorities had to move their equipment and at times all were on the site at once.

Although each statutory authority carried out their own work, careful planning by the Resident Engineer ensured that work by all authorities went on within defined areas at any one time and kept interference to the public to a minimum.

In addition to the service problems traffic had to be kept flowing smoothly at all times and this necessitated considerable negotiations with the relevant authorities on the many road diversions. Where roads crossed the line of construction it was necessary to build short lengths of wall and roof to the tunnels so that the roads could be diverted over a piece of finished roof and construction could be carried out on the site of the original road. In all, seven short sections were constructed in advance for this purpose.

Constructing the secant pile walls posed its own special problems. During the peak construction period 5 secant piling rigs were working on the site and each rig averaged 6 or 7 piles per day with an average length of bore about 30 ft. This meant that each rig constructed 5.5m of wall per day. Sections of tunnel came available over a period of about a year as service diversions were completed and a very careful programming exercise took place to ensure continuity of work for the rigs on site. Some trouble was experienced with the reinforcement cages rising as the pile was concreted. This problem was tackled by adding an air extraining agent to the concrete to improve its workability and by putting cruciform bars at the toe of the cage.

An interesting constructional problem arose at a point some 33m to the east of Hatton Cross Station. Here the railway passes under an existing sewer to give a cover of some 5m. In the deepest section conventional sheet piling methods were used but in the approach to this section where the railway was getting deeper, it was necessary to use secant piles as the railway was immediately adjacent to the Champion Spark Plug Factory where sheet piling was not acceptable.

Since the whole of the ground above roof level was water-bearing gravel there would have been considerable problems in excavating a 5m cut to place the roof of the tunnel on the secant piles. The contractor asked if it would be possible to lift the roof to a higher level to avoid this problem. By increasing the reinforcement and including it in every pile instead of alternative piles it was found to be structurally possible and the proposal was agreed. It also had the effect of saving some 100,000 on a high voltage cable diversion that would have been necessary if the original scheme had been implemented.

Initially the 890mm dia. secant piles were driven at 840mm centres giving a 50mm overlap, but as the extremely wet conditions of most of the gravels became apparent, the centres were reduced to 800mm to give a 90mm lap. This also gave some protection against 'out of plumb' piles becoming unlapped. In fact, it was found that the completed walls were remarkably watertight, even with the wider spacings. In all some 6,100 piles were sunk on this contract and on the exposed parts of the pile there was not one single faulty pile. Where leaks did occur it was mostly at a junction between one section of wall and another constructed at a different time. These leaks were dealt with by cement grouting using a grout pan.

After long sections of the wall and roof were completed the main mucking out and concreting of the invert were carried out from a number of pit heads spaced about half-a-mole apart. The contractor chose to install a belt conveyor system at the pit head capable of being extended in 9m lengths to a maximum of 335m. The conveyor system was designed to remove the muck on the lower belt, both operations capable of being carried out simultaneously. Excavation at the face was by means of a traxcavator with a side tipping bucket. Initially, considerable trouble was experienced with fumes from the traxcavator and to overcome this two roof beams were omitted at selected locations and a portable housing containing two 9 horse power electric extractor fans placed over the top. The foul air was then drawn from in front of the face through the small gap between the roof and the top of the unexcavated material.

The majority of the excavation was with the interface between the blue clay and the water-bearing gravel at about half face level. A problem which was encountered was the due to variations in level of clay/gravel/interfaces a series of 'ponds' were trapped between the secant piles where there were depressions in the clay. Generally, it was possible to pump these but, coupled with the continued ingress of water from the occasional leaks between the secant piles, it was sufficient to make the clay sticky and tend to clog the conveyor. Partly due to these difficulties more conventional methods, using face shovels feeding to roof access points, were used for some of the later sections, the conveyor being used mainly for concrete placing.

It is interesting to note that in all cases the secant piles with the roof in position were designed to form a structure capable of taking road loading and was not dependent on the invert slab being placed. Also the design of the piles allowed all the ground to be removed without the need for struts, there being about 5ft. of pile below the formation level. In some areas where the ground was suspect the toe in was increased. In all areas a 3" blinding was placed on the formation as soon as it was cut. The invert slab followed as soon as practicable.

In one area it was deemed wise to make the invert slab act structurally with the walls owing to the presence of a 9ft. dia. brick barrel sewer. In casting the pile, additional bars for bending out were included in the cage and a polystyrene box out formed. This method was found to be entirely satisfactory.

DESIGN CONSIDERATIONS FOR HEATHROW CENTRAL STATION

The basic station construction is a reinforced concrete box approximately 122m long by 24m wide formed by the slurry trench wall method and excavated by conventional methods from the surface. The ticket hall will be approximately 6m below ground level with platforms at a depth of 14m. There will also be an intermediate mezzanine floor containing staff accommodation (See Fig. 10). The British Airports Authority are also building their pedestrian subways to connect each terminal to the new station.

The ground strata consisted of sandy gravel for the first 8m followed by blue London clay. The water table stood about 4m below ground level.

The slurry trench walls were 1m thick and taken down to a level about 6m below rail level. They were designed as a row of discontinuous panels 6m long with a knuckle joint between each panel. Continuity at the top of the panels was achieved by a capping beam.

The walls were designed as propped vertical slabs with continuous bearings formed by means of 100m deep rebates provided for the ticket hall and mezzanine floors. The openings for the tunnels to enter the station had to be broken out and the reinforcement was suitably arranged to facilitate this.

The diaphragm wall is the permanent finished structure but will be faced with a brick skin wall. A 4" cavity will be left between the skin wall and piles for drainage purposes.

CONSTRUCTION OF HEATHROW CENTRAL

The formation of the slurry trench wall progressed smoothly although in one area trench fall-in did occur, possibly caused by ground water movement leaching out the fines or from vibration caused by some short sheet piles being driven elsewhere on the site.

As the slurry trench walls were designed as propped walls it was necessary to provide temporary propping during the construction period. Randex No. 6 steel struts were used spanning across the station onto twin universal beam walings at ticket hall and mezzanine floor level. Timber struts were used at platform level.

Because of adjacent services, the deflection at the top of the wall was limited to 5mm. This was achieved by jacking the struts at ticket hall level immediately after installation using flat jacks. The next level of struts were shimmed and welded but not jacked. A number of the struts were monitored with strain gauges and generally behaved as expected. The mezzanine level struts, however, took rather more load than expected. This was possible because the timber struts at the lower level did not take their full load.

Excavation was carried out with a traxcavator moving muck to selected areas where it was lifted out with a grab.

At formation level a centre strip was excavated first leaving two side berms of muck against the toe of the wall. When the concrete in the centre block had been cast, timber struts from the centre block to the wall were placed. These timber struts and starter bars made the excavation of the side beams very difficult. In hindsight it may have been better to design the walls so that the excavation could have been taken down to formation level over the whole area.

The technique of excavating to each floor level and then casting the floor, with access holes, using the muck as a soffit shutter would be appropriate and economical for some works of this type. At Heathrow, however, it was not possible as the station had to be designed to carry an overlying building development. The roof therefore had to be able to carry column loads anywhere and was consequently 2m thick. It was also necessary to have heavy columns from platform level to the soffit of the roof.

THE BENOTO RIG

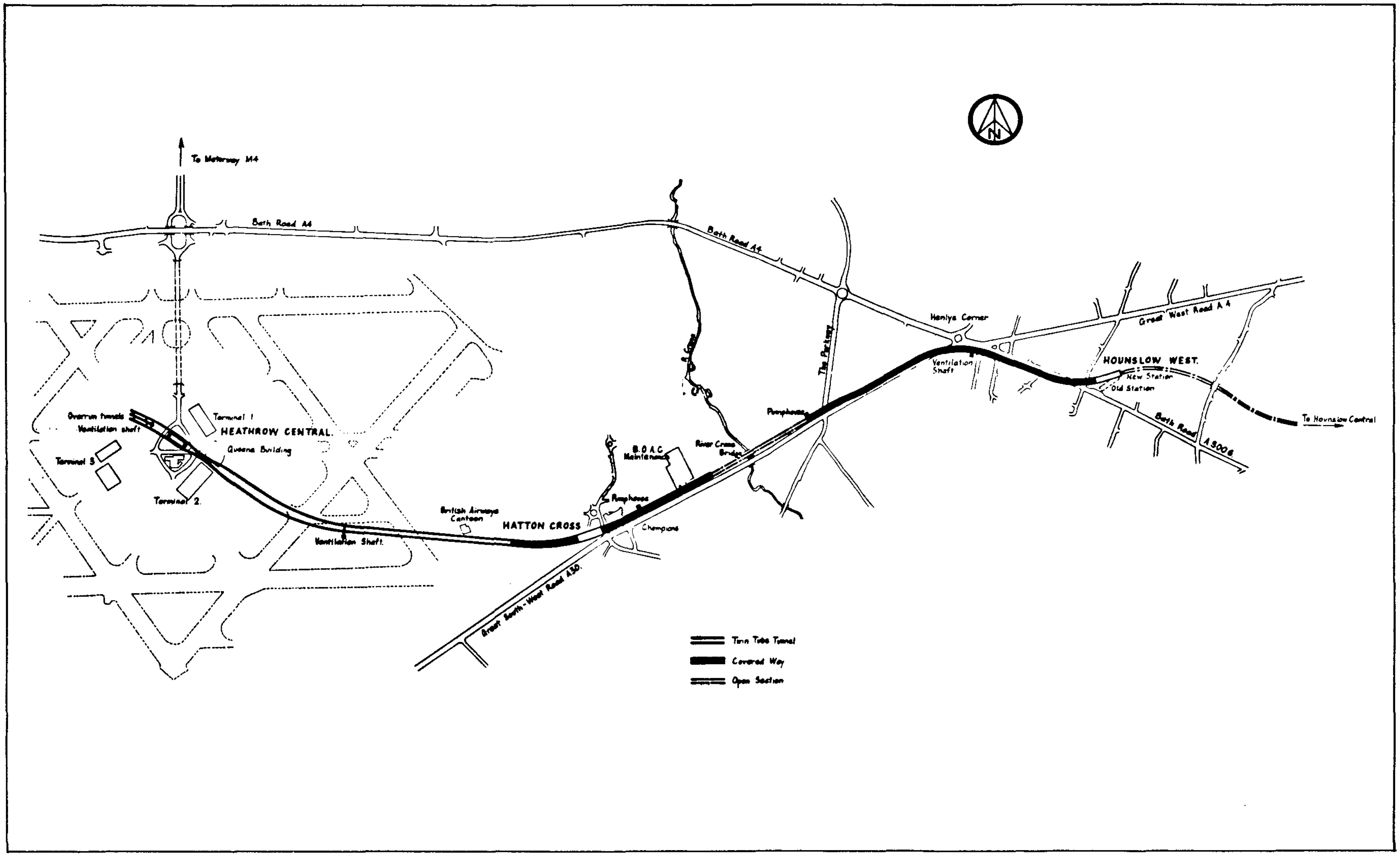
The benoto rig can travel under its own power economically up to a distance of about 300m. Over this it is usual to lower the mast and fit some road wheels, it can then be towed to a new location. For long moves such as to a new site a low loader would normally be used.

The rig moves on a rail track mounted on a steel plate all of which forms part of the machine. It lifts itself up on four hydraulic legs so that the rail track is hanging on the machine and may be moved either backwards or forwards. The machine then lowers itself onto the track and continues its travel.

Because of the large area, the bearing pressure is very low when the machine is sitting on its track.

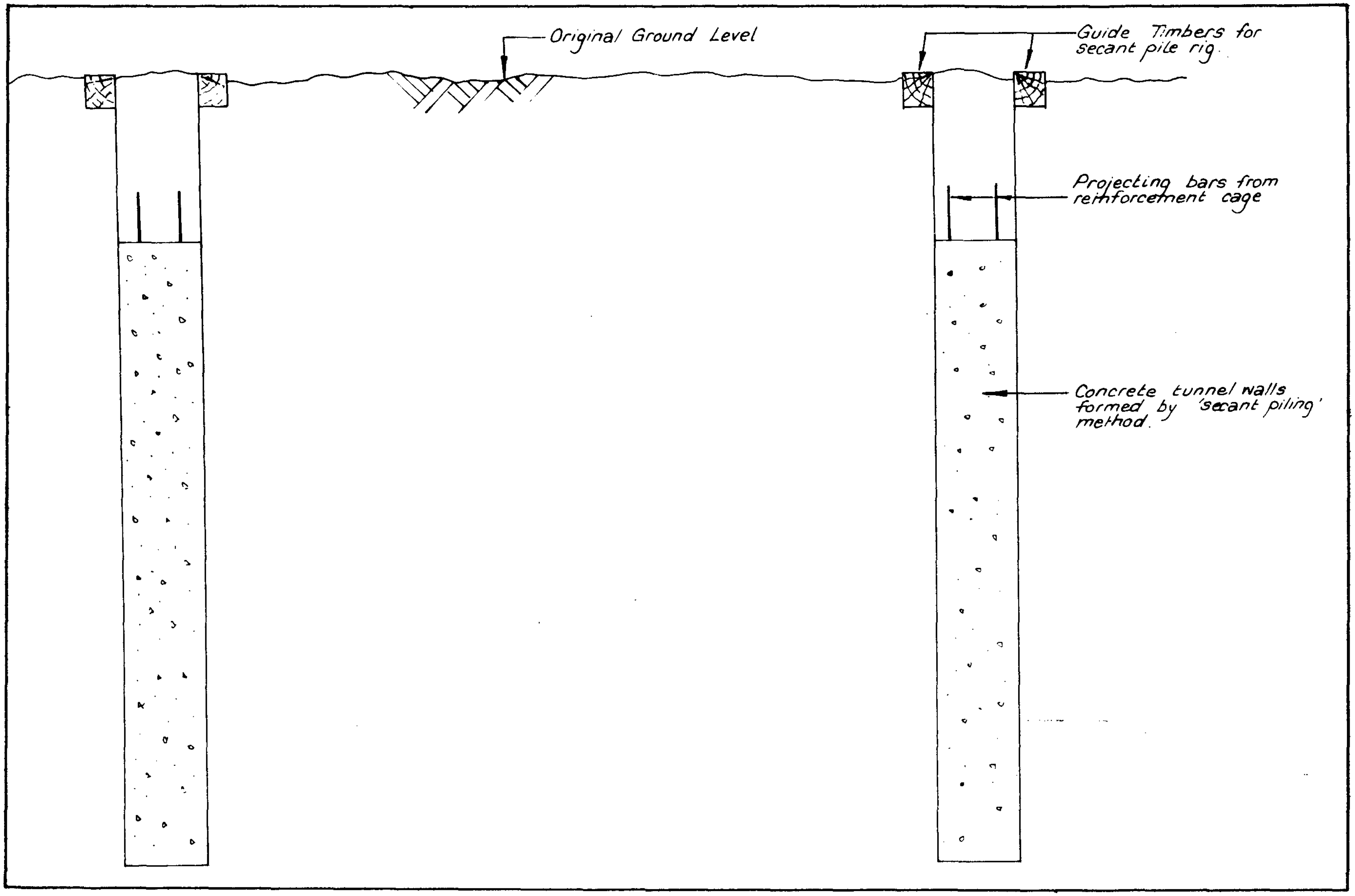
The attendant plant with a rig consists of a light crane for handling the bore casings and the reinforcement and a small traxcavator for handling the spoil and levelling the ground for the guide timbers. When two rigs are working in tandem they can be serviced by one set of attendant plant.

Figs. 11 through 14 show the benoto rig in operation, and various phases of the construction by cut-and cover.



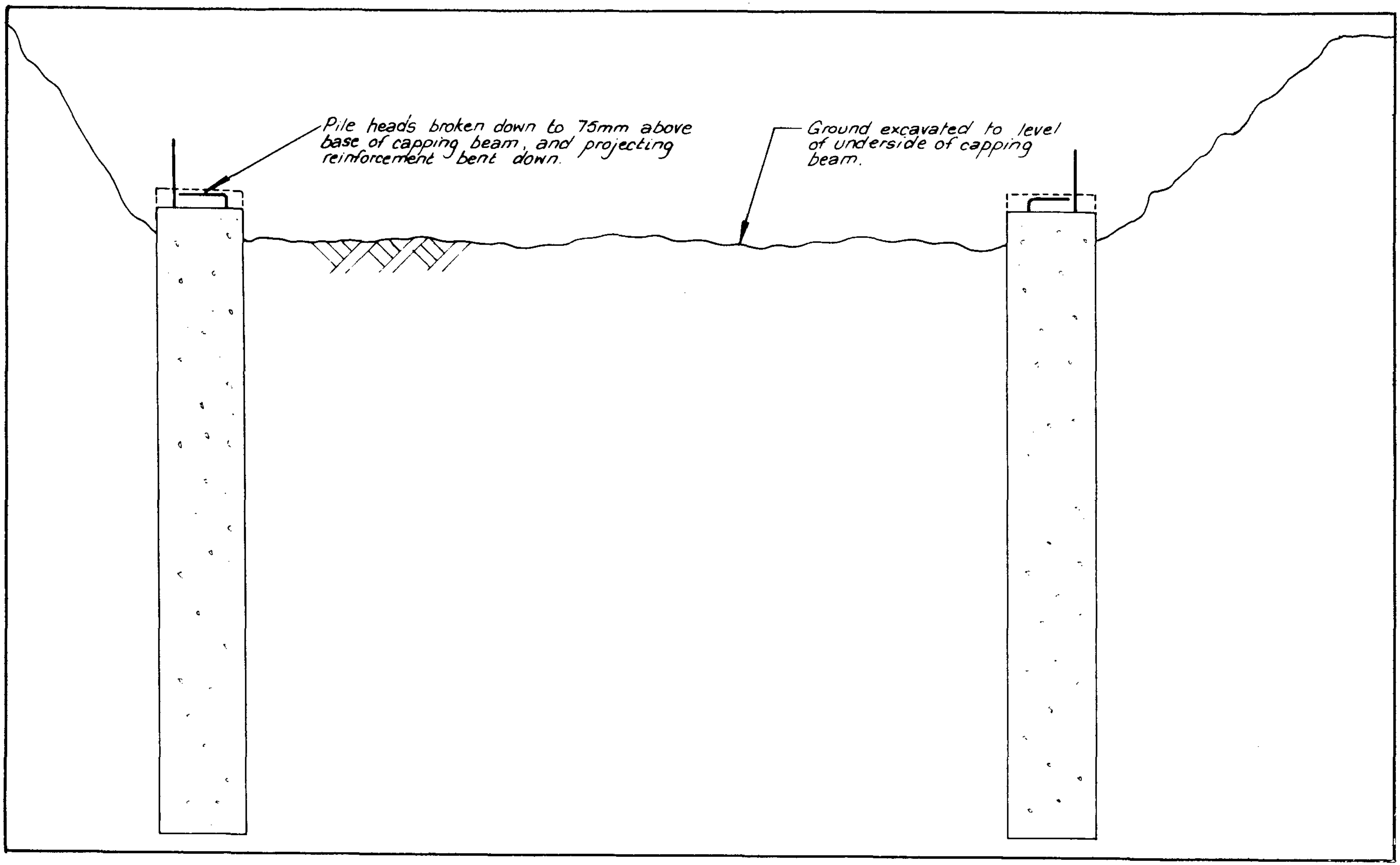
Route Plan

FIGURE 1



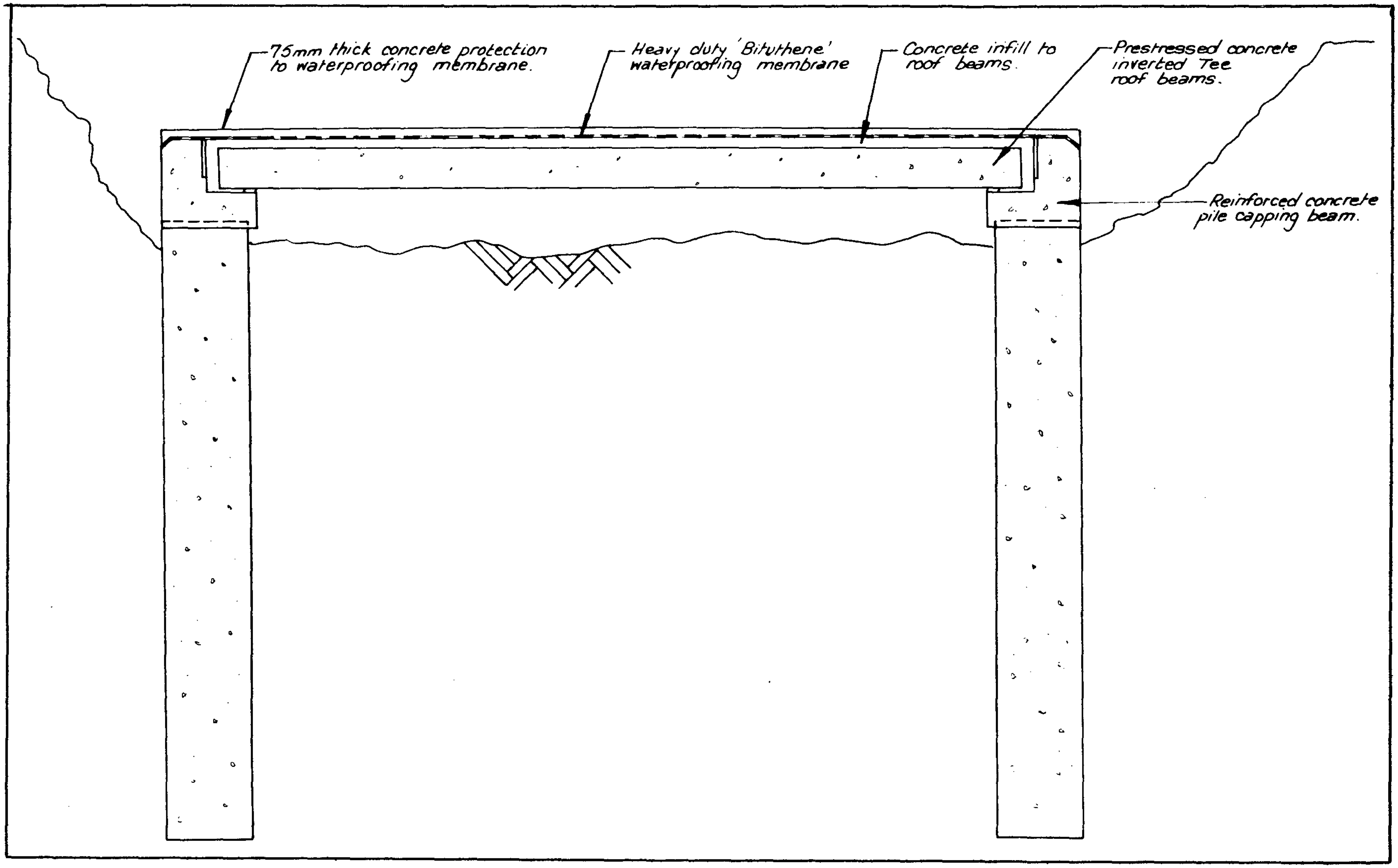
Stage 1 - Secant Piling

FIGURE 2



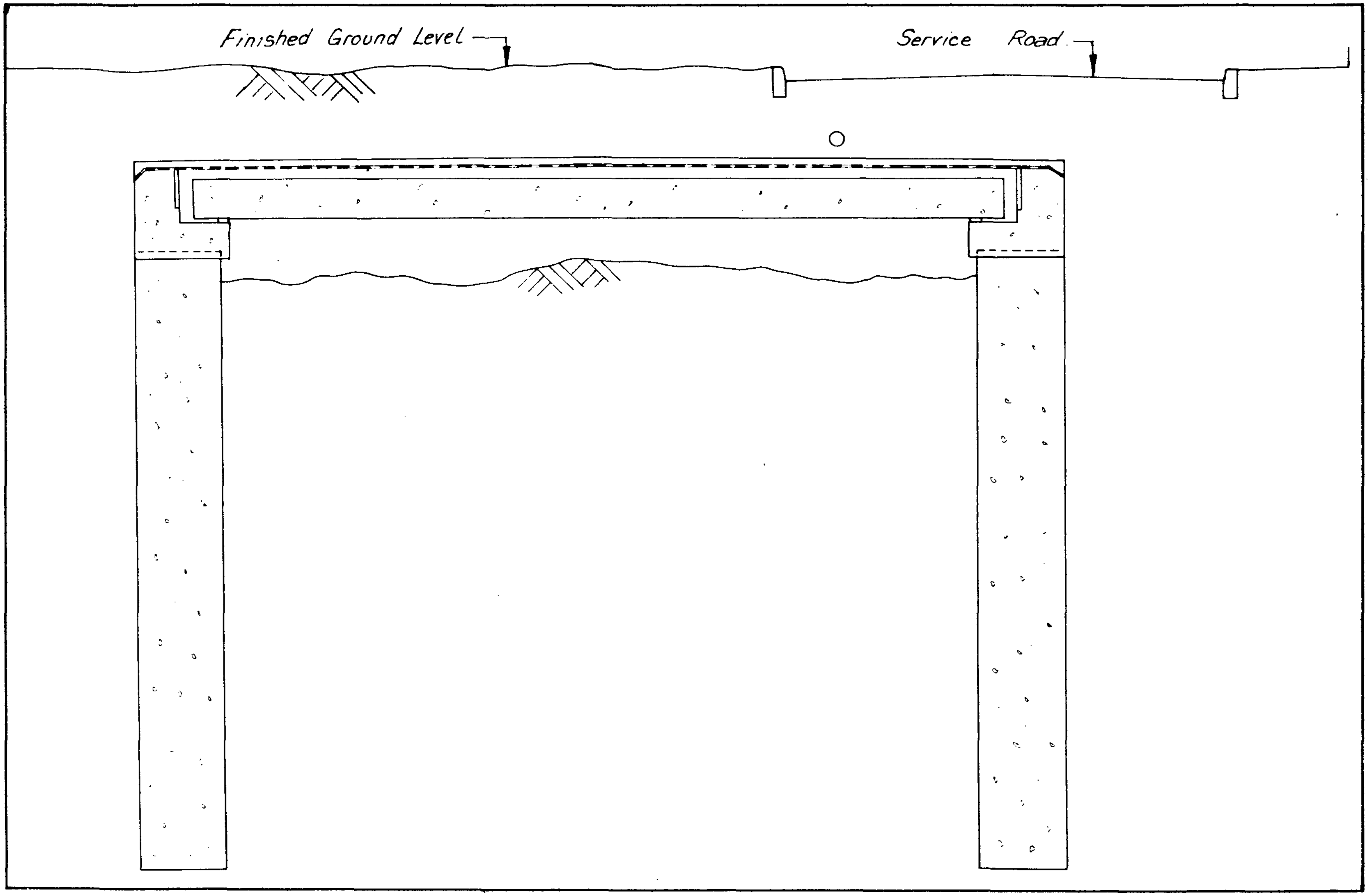
Stage 2 - Excavate to Pile Heads

FIGURE 3



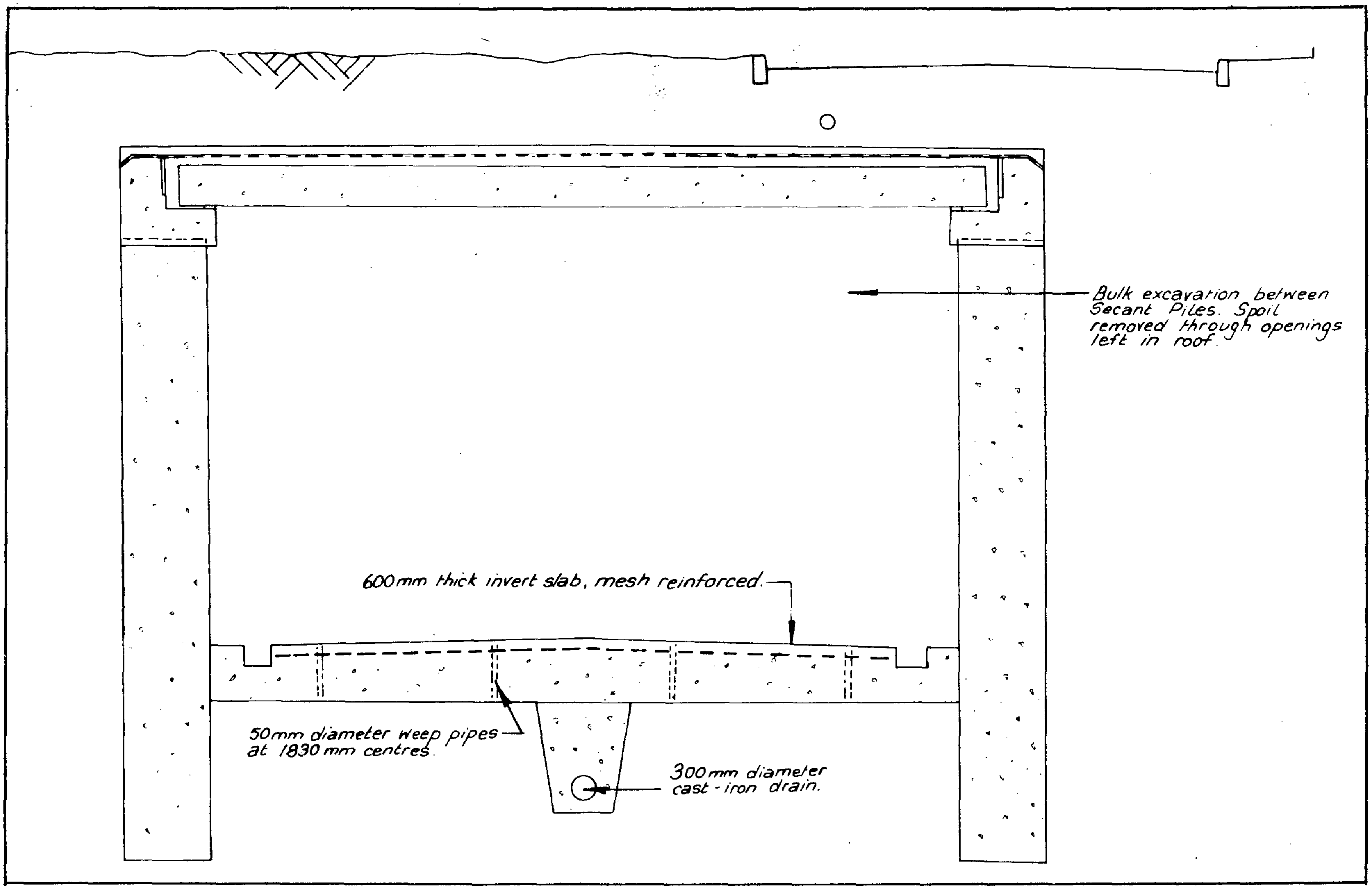
Stage 3 - Capping Beam and Roof Construction

FIGURE 4



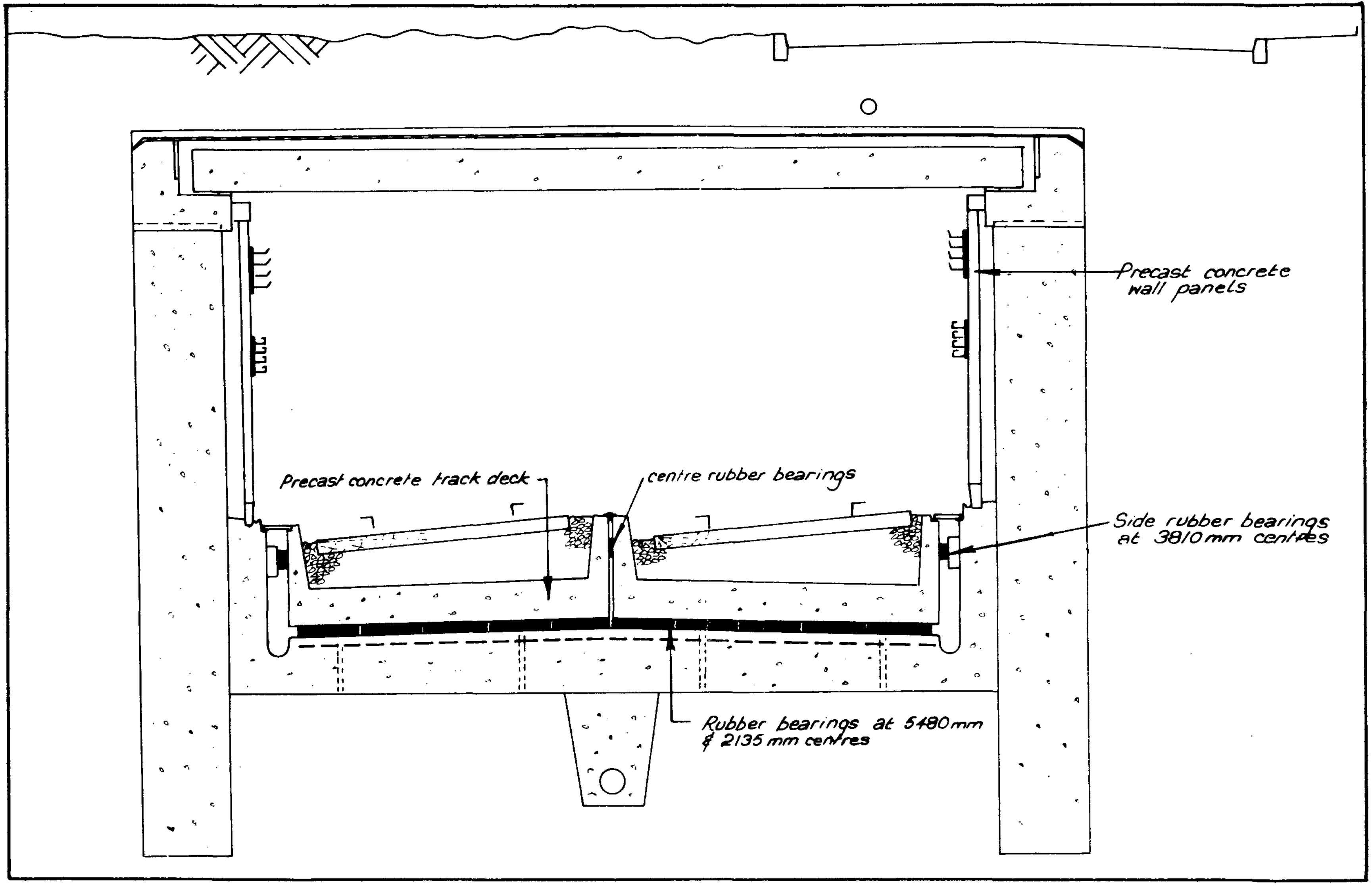
Stage 4 - Backfill and Reinstate

FIGURE 5



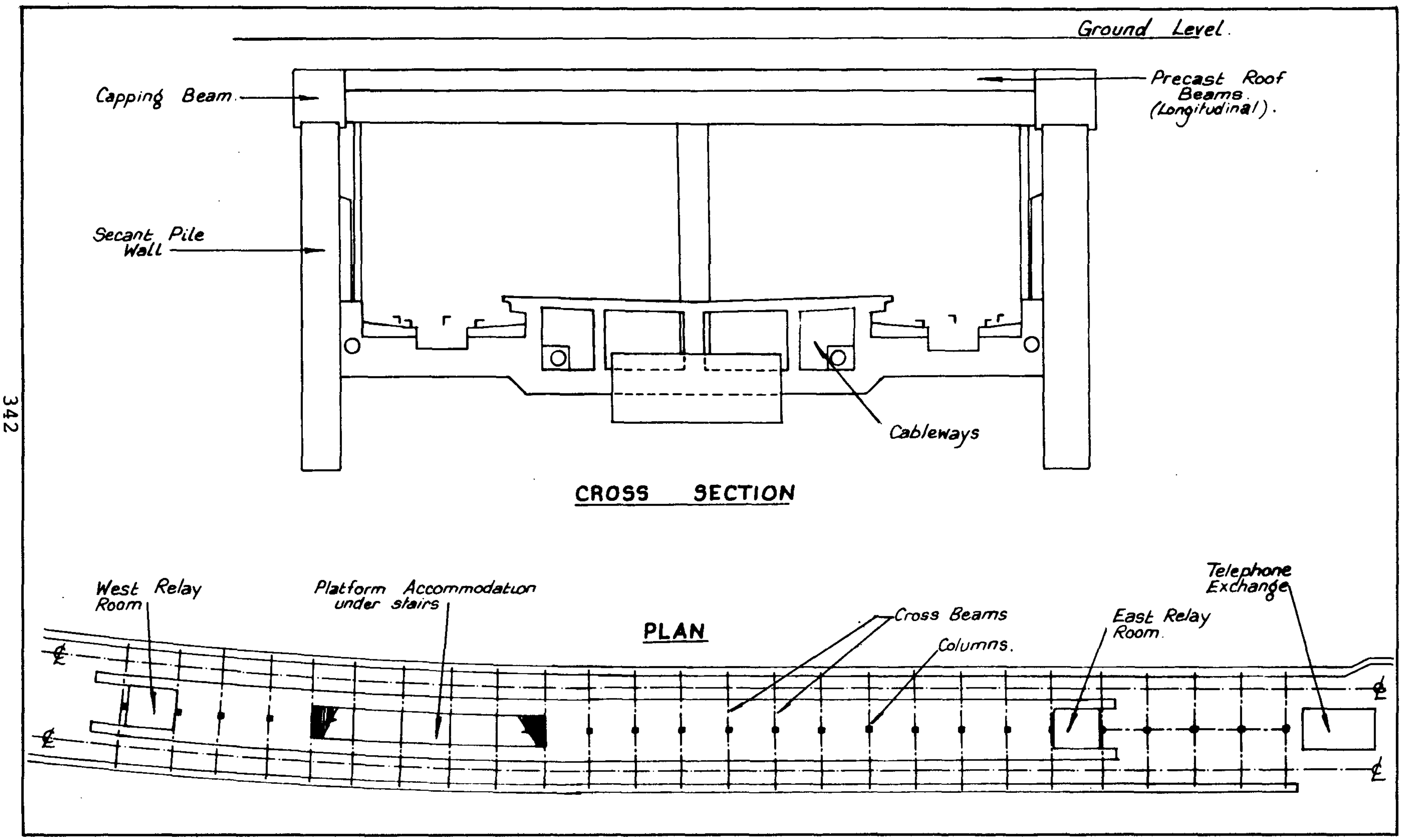
Stage 5 - Bulk Excavation and Invert Slab Construction

FIGURE 6



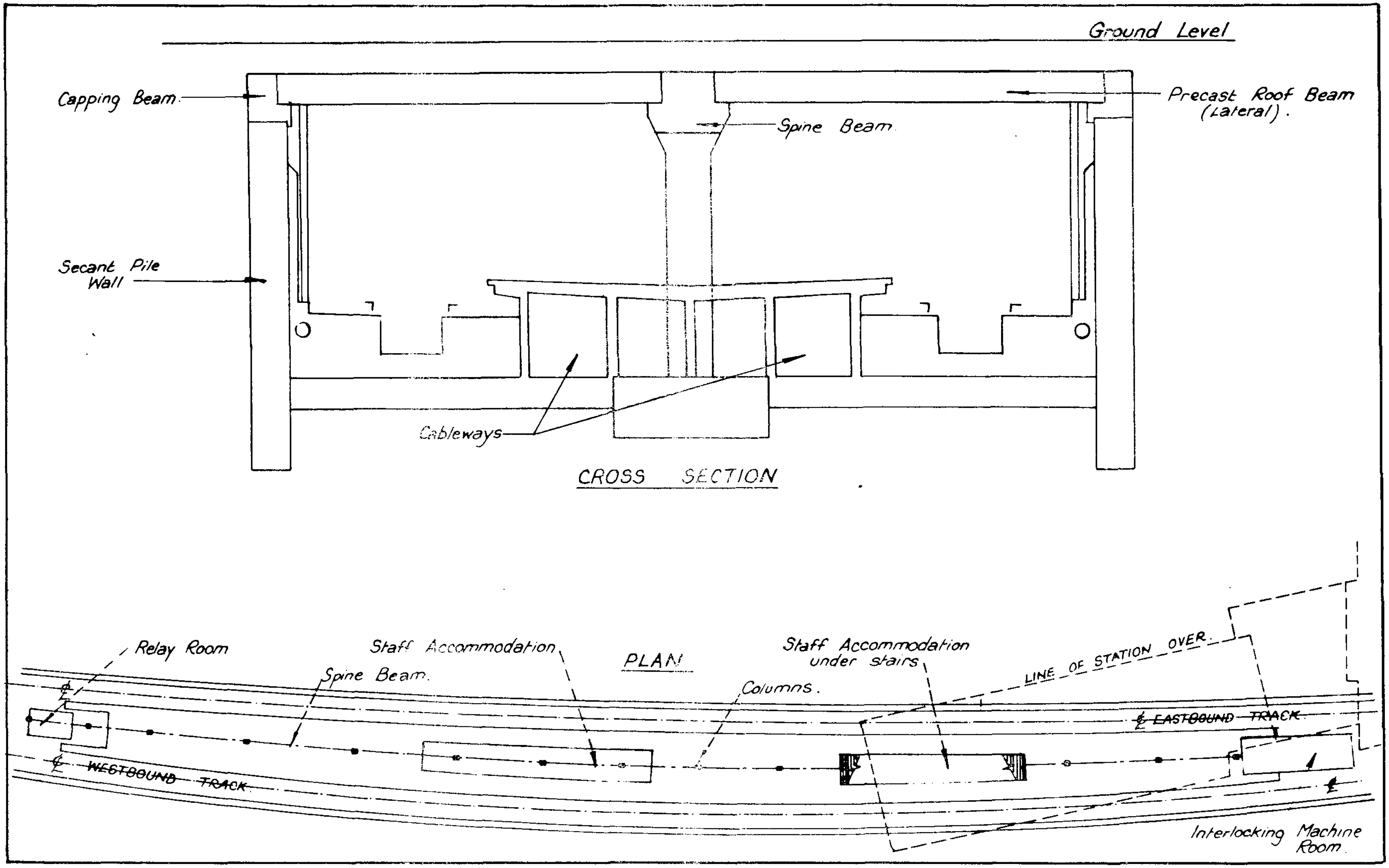
Stage 6 - Tunnel Interior Construction

FIGURE 7



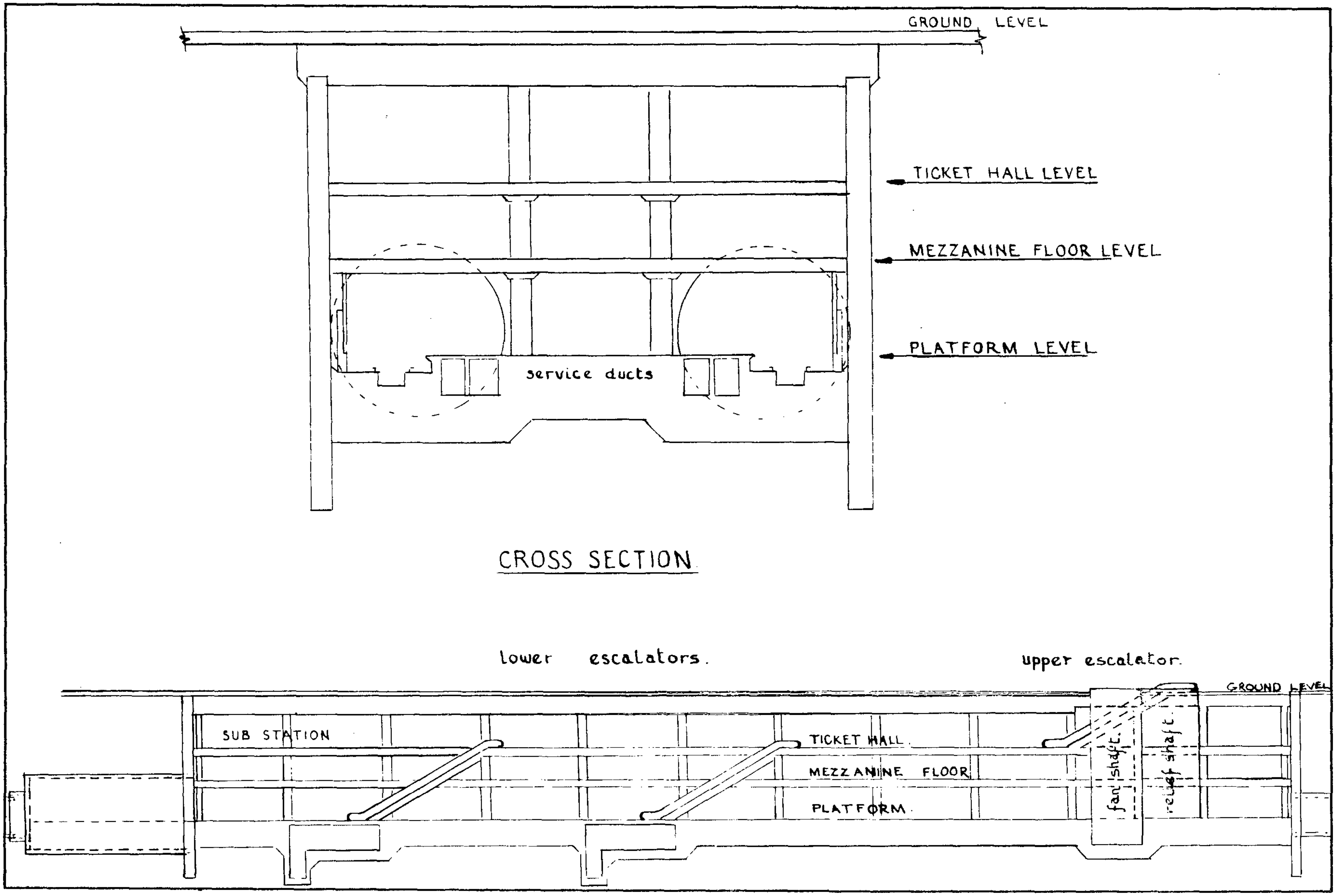
Hounslow West Station

FIGURE 8



Hatton Cross Station

FIGURE 9



Heathrow Central Station - Longitudinal Section

FIGURE 10

FIGURE 11

A Benoto piling rig working on forming a subway tunnel wall close to the boundary of some residential properties. No complaints of noise or vibration were received from these houses while this wall was being built.

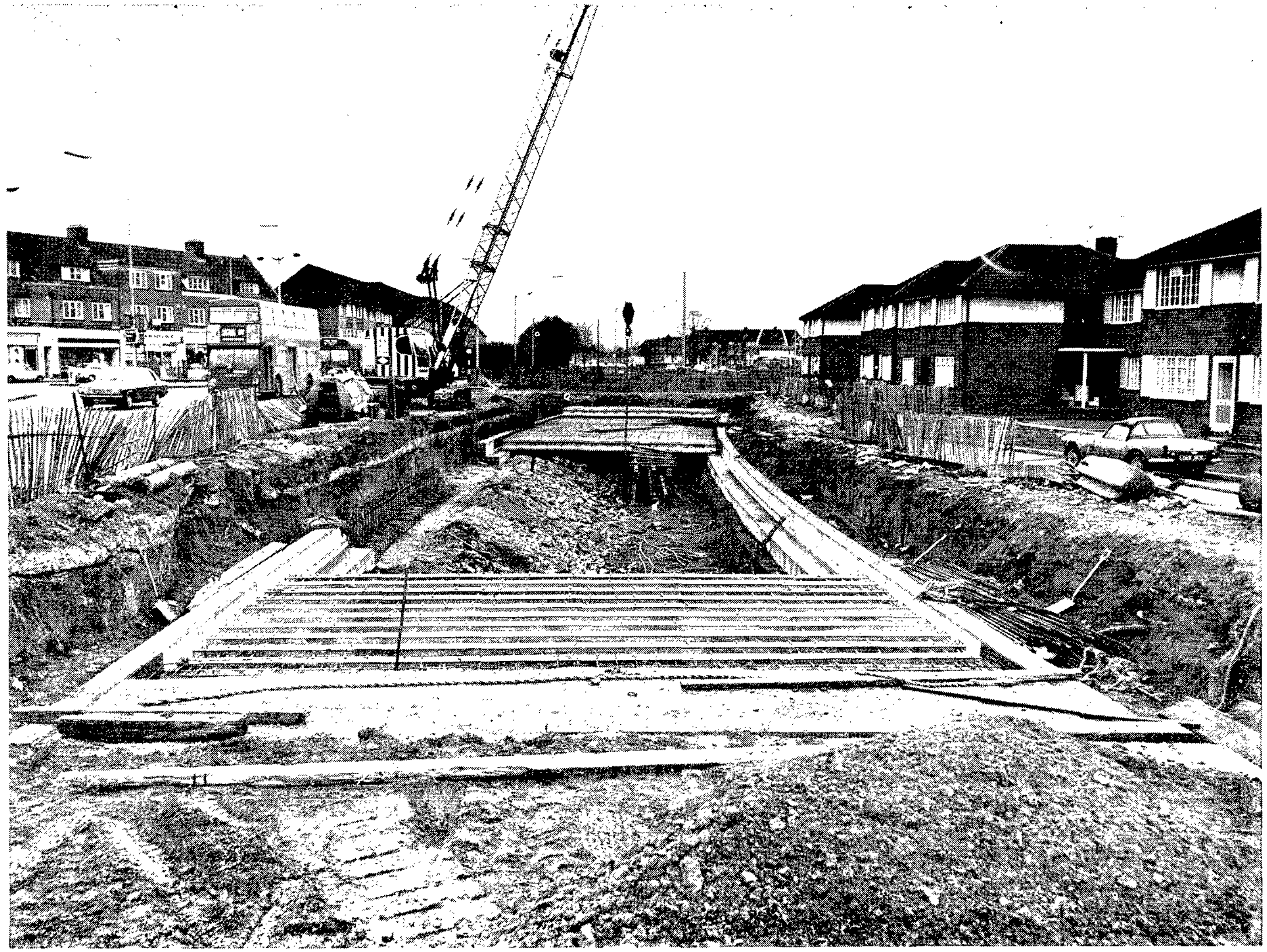


FIGURE 12

Placing a section of tunnel roof. Note in the foreground the backfill taking place immediately behind the roof construction.

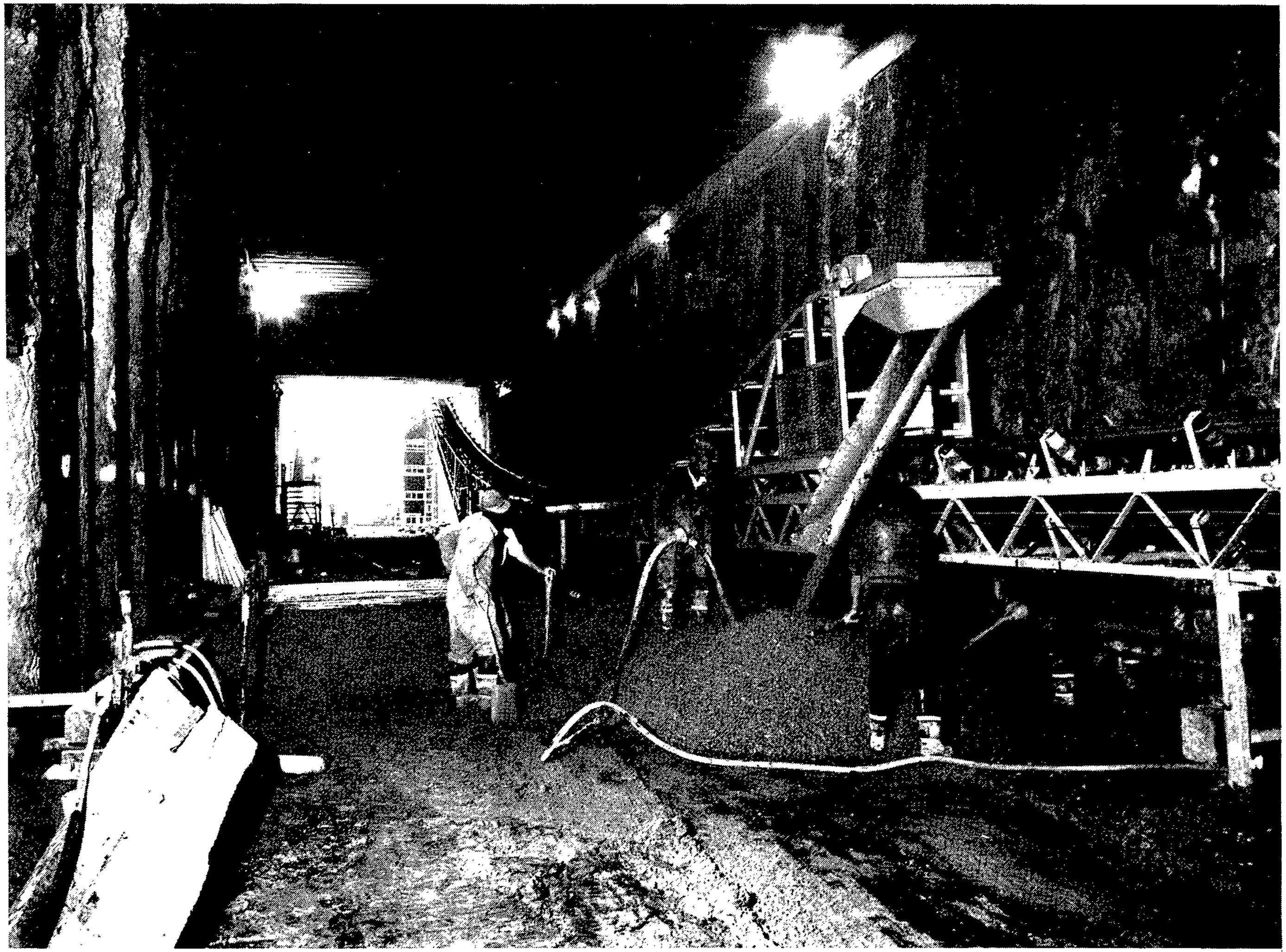


FIGURE 13

Invert concrete being placed by means of a conveyor. The delivery chute can be moved along the conveyor so that the position of delivery of the concrete is independent of the length of the conveyor.

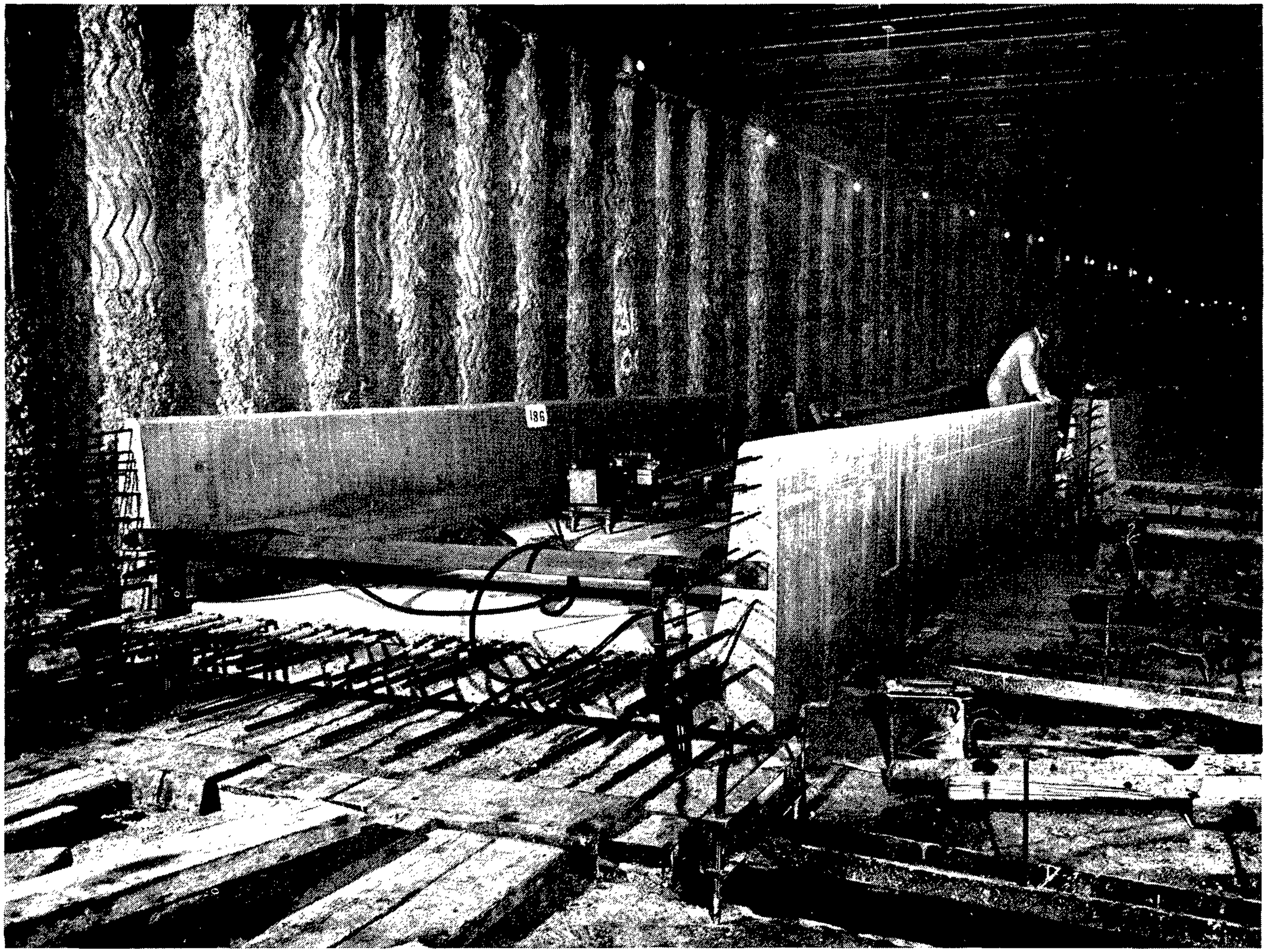


FIGURE 14

A track tray being placed in the completed tunnel. Although the ground water table was only about 300mm below the capping beam, the walls are remarkably dry.

PAPER 18

Construction Fundamentals of Diaphragm Walls

Part I

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PAPER 18

CONSTRUCTION FUNDAMENTALS OF DIAPHRAGM WALLS

Part I

by

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This paper is the first of two papers addressing the subject, "Construction Fundamentals for Diaphragm Walls". The paper will focus on the construction details which directly affect the preparation of contract drawings and specifications, and those elements of diaphragm wall construction which are of special consequence in the actual application in the field.

GUIDEWALLS

Guidewalls are essential for the accurate control of line and grade of a diaphragm wall. They serve as a guide for the excavation and then as the support for the cage prior to and during the concrete pour. The excavation for guidewalls should be carried down to the elevation of the lowest utility or to the level of the footing of the adjacent structures. All terminated utilities should be removed and plugged and all openings in walls of adjacent structures should be sealed. In the cases where no guide-walls are provided against existing structures, those structures should be protected by a cement plaster coat or a lining of plywood or steel plate. The trench should be backfilled to the underside of the guidewalls with a lean concrete or a clean well compacted backfill.

Guidewalls are usually six to twelve inches thick and from three to six-feet deep and are normally reinforced with only four #6-bars running longitudinally. As a practical matter, only the inside face of the guidewalls should be formed while the backfaces of the guidewalls should be cast against the soil sides of the trench. In many locations guidewalls need not be removed after final construction is completed. In these cases the guidewall on the unexcavated side of the wall should remain in place, while the guidewall on the inside (or excavated) side of the wall is removed during the general excavation.

PANEL DIMENSIONS AND ARRANGEMENT

Panel dimensions are usually controlled by both the technical requirements of the work and the type and size of equipment that is available to the contractor performing the work. It is obvious that panel lengths can be no shorter than the length of the bucket and no thinner than the width of the bucket. Short panel lengths usually, in the range of seven feet, would be used in loose unstable materials or in areas where very high surcharge pressures occur because of adjacent structures. Longer panels, ranging up to thirty feet in length, can be used in cohesive soils or other stable materials. Panel lengths can vary anywhere from the seven foot to thirty-foot panel lengths available. However, specific panel sizes are dependent upon the location of the panel joints since the panel joints must be coordinated with the permanent framing system, the temporary bracing scheme or the location of adjacent footings.

Wall thicknesses are usually twenty-four or thirty inches. However, if it is necessary for the diaphragm wall to carry a high lateral load, to span large distances between supports or to carry substantial vertical loadings, the thicknesses can be carried to thirty-six inches, and even up to sixty inches, if necessary.

The "Bottom of Wall" elevations are governed by the location of the top of rock, by the need to embed the wall in an impervious strata, by the need to prolongate flow lines through the pervious strata, to secure a stable bottom during the construction period; or by the need to provide lateral support for the wall during the excavation and bracing process.

The "Top of Wall" elevation is governed by the details of future subgrade and superstructure construction, by the location and the elevation of adjacent footings, by the location of existing and future utilities, and by the need to trim or remove sections of the wall at a later time.

Construction joints between panels are achieved in a variety of ways. The most basic and simplest methods are the half-round joint formed by a stop end pipe or joints formed with steel wide-flanged sections. Less often used are joints formed by square-end buckets and joints incorporating sheetpile sections or break-away keys set into the pour. Needless to say, complicated joint details are expensive, difficult to install and perform unsatisfactorily in less than ideal conditions.

REINFORCING STEEL CAGES

In general, reinforcing steel cages should be detailed as simply as possible and the same details should be repeated as often as possible. The designer should require only one layer of horizontal and vertical reinforcing steel on each face of the wall. Splices should be minimized and, if possible, totally eliminated. Provision should be made for field alterations of all cages.

Unforseen site conditions usually require a greater or lesser depth of wall; the cage may also have to be altered in order to accommodate a revised panel length. Accessories may be added, removed or relocated during construction.

Welding of cages is to be discouraged. Erection stresses often cause even the best prepared weld to fail. Cages and accessories should be securily tied with tie-wire.

A minimum three-inch cover should be required over each face of reinforcement and wheel. Wheel spacers, skids or other spacing devices should be used in order to guarantee the specified cover. Cages should be securely suspended from the guidewalls and kept at least six inches clear of the bottom of the excavation.

Shop drawings should plan for and accurately show the details and locations of all bracing plates, tieback anchor trumpets, beam seats, keys and pipe sleeves. These accessories should be detailed and located in such a manner that they will not interfere with the placement of the concrete. The location and details of grout pipes, tiedown anchor sleeves, slope indicator tubes and even the tremie pipe must be studied in detail and shown on the shop drawings. Complicated, poorly conceived details will guarantee problems in the field.

DESIGN OF CONCRETE MIXES

Because of variations in materials, mixing equipment and skilled labor, it is difficult to recommend a specific concrete mix design; however, there are several rules which should be followed in the design of the mix. With hard gravel being preferred over crushed gap-graded stone, the aggregates used in the mix should be well-graded. A sandier mix similar to a "pumpcrete" mix will flow better in a tremie pipe and throughout the panel and is therefore preferred. Plasticizers and air entrainment mixtures are recommended. Design mixes should range between 3,000 psi and 5,000 psi ultimate strengths and should be designed and tested with enough water to guarantee that an eight inch slump will be achieved. It is important that all personnel involved in the execution of this work understand that an eight inch slump is essential for the proper casting of these panels and that the people in the field are not to be permitted to tamper with the mix.

A four-inch slump concrete gives fine results in a laboratory but guarantees improperly cast walls. The surface will be honeycombed, the joints will be irregular and improperly concreted and an occasional cold joint will occur due to the difficulty of the placing of this material through a tremie pipe into a long, narrow excavation.

CONCRETE PLACEMENT

If excavations are performed in fine sand, or with percussion tools used to drill boulders or bedrock, it is imperative that the panel be desanded prior to the placement of concrete. Otherwise the fine sand particles, held in suspension by the bentonite, will mix the concrete and form pockets of "mud" in the panel. These mud pockets usually flow to the edges of the panels or get "hung up" in the cage.

A single eight-inch or ten-inch diameter tremie pipe centrally located within the panel is recommended. The tremie hopper should be large enough to receive the occasional surge of concrete and prohibit the spillage of concrete from the hopper into the trench. It is wise to require the placement of a four to six-inch mesh screen at the hopper in order to prohibit the entry of large balls of concrete which occasionally occur with high cement contents concretes.

A concrete pour should proceed as rapidly as possible. However, it should always be timed in such a manner that a continuous concrete pour is maintained. Delays in the delivery of concrete guarantee a cold joint in the panel and a future source of leakage.

At the conclusion of the concrete placement the stop end pipe must be extracted from the excavation; it must be removed at a rate slow enough that it is never raised above the level at which the concrete has already set and a rate fast enough that the pipe will not become stuck within the excavation.

QUALITY OF THE IN-PLACE CONCRETE

Once a properly designed mix has been properly placed it is almost impossible for the concrete to not achieve its designed strength. It has been our experience that cylinders taken during the pour usually show concrete strengths ten to fifty percent greater than the strength specified, and that cores taken from a wall even under the most disadvantageous placement conditions are apt to have strengths equal to or upwards to fifty percent greater than the strength specified.

The question of permissible bond stresses still remains unanswered; some researchers and designers recommend the use of allowable stresses as much as seventy-five percent of the ACI code allowables while other researchers and designers recommend no reductions at all. In most cases, the argument about allowable bond stresses is a moot argument since actual computed bond stresses are far less than the allowable stresses.

ACCURACY, TOLERANCES AND FINISH

Construction accuracy and finishes usually are dependent upon the geology of the site and the contractor's skill and tools. However, properly executed diaphragm wall construction should fall within the following tolerances:

> The vertical joint and the end of a panel formed with an end pipe should fall within six inches of the specified location, while the location of a panel formed by a steel beam should be within three inches of its theoretical location. Inserts within a cage should be within three inches horizontal and three inches vertical of the specified location. Keys and dowels should be permitted a three inch variation in vertical location and should not be required to develop the moment capacity of the member framing into the wall. Greater accuracy can be achieved in the construction of diaphragm walls. However, the amount of the additional cost necessary to achieve a small improvement in the accuracy of construction must be weighed against the cost of corrections performed on the wall upon exposure.

Tolerances should be increased by fifty to one-hundred percent in the cases where walls are excavated through loose boulder-laden soils or fills consisting of piles, timbers and other loose debris.

The finish of the wall is a direct reproduction of the soil surfaces against which the wall is cast. A loose boulder removed from the excavation leaves an indentation in the site of the excavation. Concrete placed within the excavation will fill that indentation and will appear as a protrusion on the finished wall. The occasional bump should be trimmed from the face of the wall and any voids should be filled. Occasionally a wall is parged, and most often the as-cast wall serves as the final wall finish.

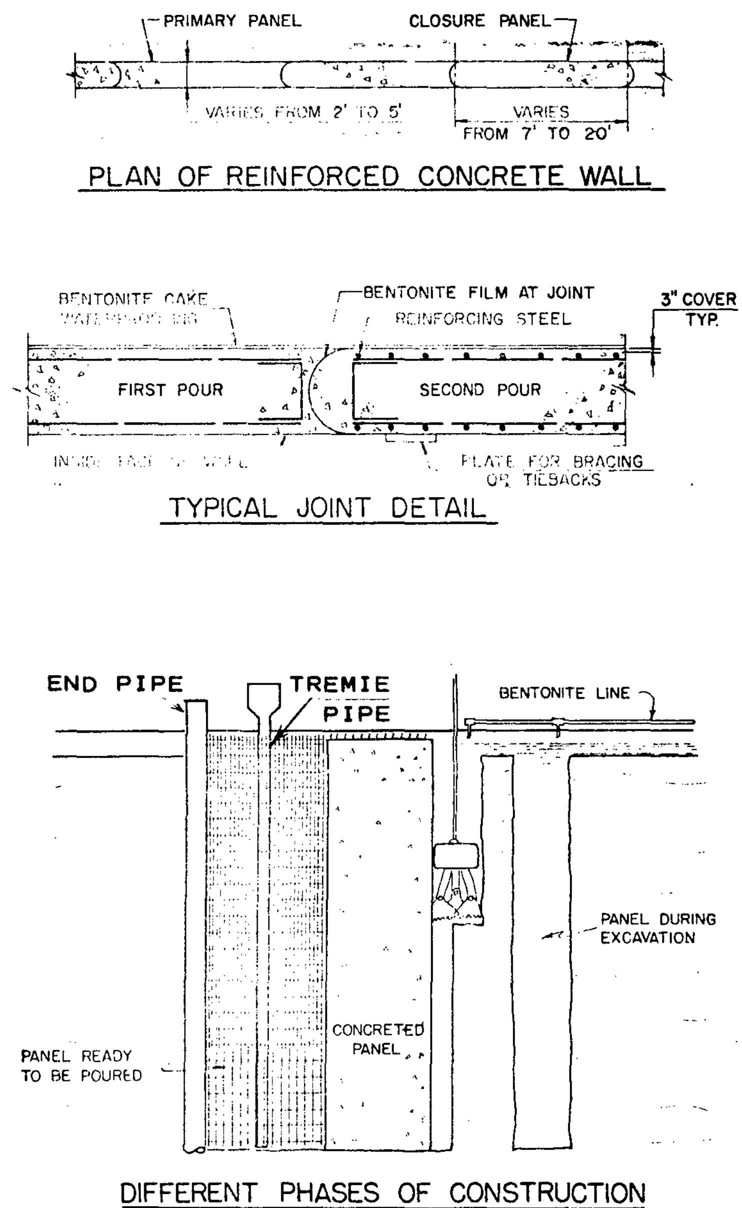
Some designers specify the construction of an independent masony wall anywhere from three inches to three feet from the face of the diaphragm wall. In the case where larger distances separate the masonry wall and the diaphragm wall, the space between two walls is used as a pipe chase, as an electrical conduit run, as a plennum and as a drainage channel for any water which might seep through the wall.

Properly executed slurry walls are water-tight throughout the panel. Occasionally seepage will occur at the vertical joint between panels, at cold joints, or at tieback trumpet locations. The first two problems, seepage through a vertical joint or through a horizontal cold joint, are the full responsibility of the slurry wall contractor and will be sealed by him after the wall is exposed. The joint or crack is chipped out and cleaned and then packed with rapid-setting grout mixes. Occasionally it is also necessary to chemically or cement grout the soil directly behind the wall at the location of the leak.

CONCLUSION

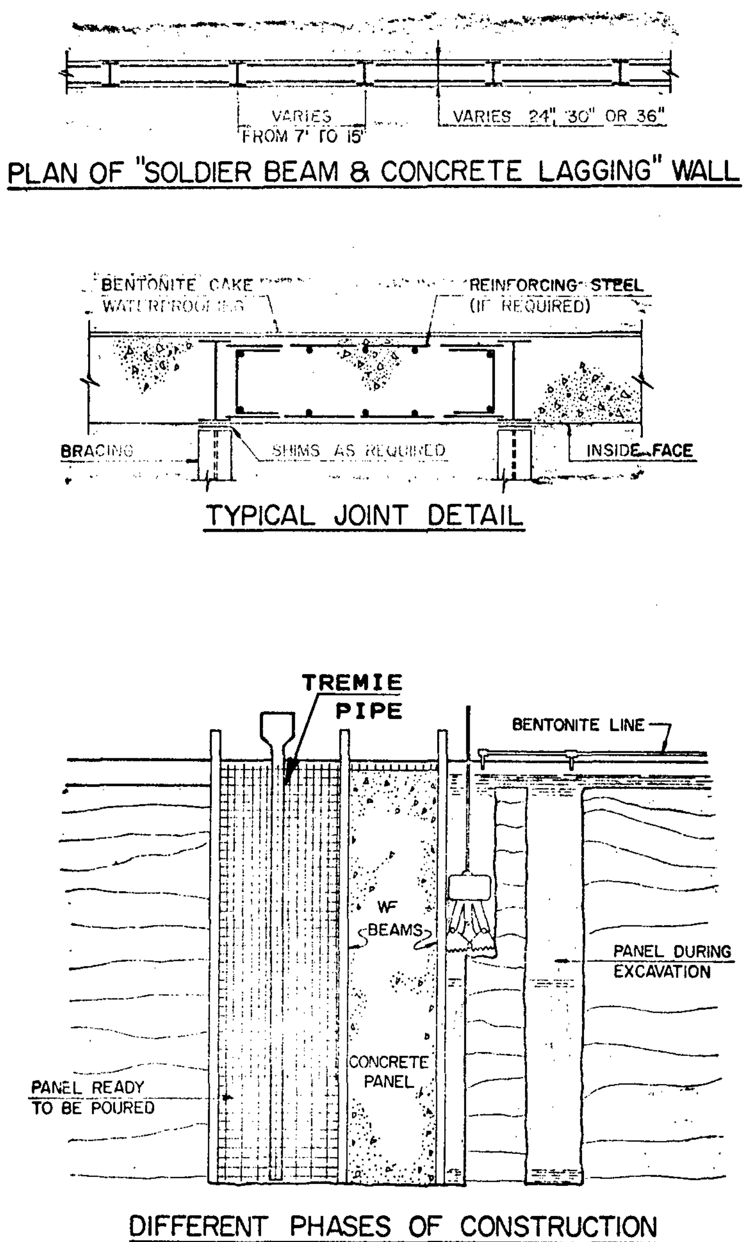
This paper has outlined the basic features of the design and the construction of diaphragm walls. It has warned you of some of the problems that can be expected in the use of diaphragm walls and has offered you guidance in the planning and the construction of sound, practical, economical walls. Regretfully, the paper has not at all covered the many benefits and advantages of diaphragm wall construction. This task is left to others to fulfill.

Figs. 1 through 17 show construction details of the various construction sequences of diaphragm walls.



CONVENTIONAL REINFORCED CONCRETE DIAPHRAGM WALL

FIGURE 1



SPECIAL "SOLDIER BEAM AND CONCRETE LAGGING" DIAPHRAGM WALL

FIGURE 2

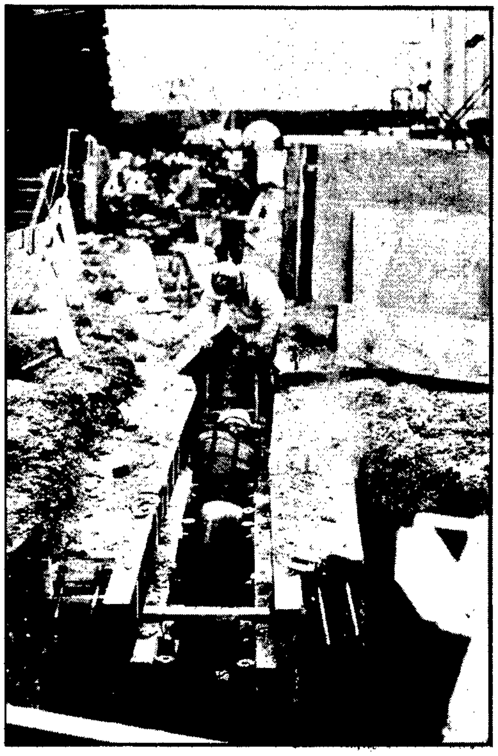
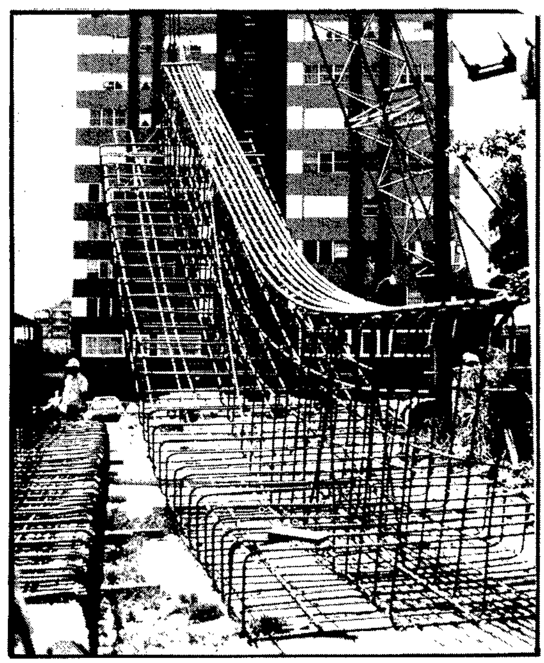


FIGURE 3

FORM WORK & ALREADY CAST GUIDEWALLS - D4B PROJECT WASHINGTON, D.C. SUBWAY

FIGURE 4

SPECIAL GUIDEWALLS FOR "T" SHAPED COUNTERFORT WALL - DE DIEGO EXPRESSWAY, SAN JAUN, P.R.



SPECIAL T SHAPED CAGE FOR COUNTERFORT WALL - DE DIEGO EXPRESSWAY

FIGURE 5

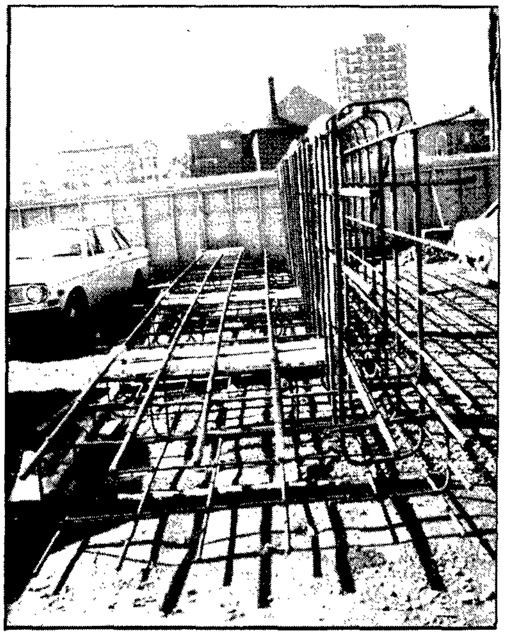


FIGURE 6

REINFORCING CAGE FOR "CORNER PANELS"
"LE CITE" PROJECT, MONTREAL

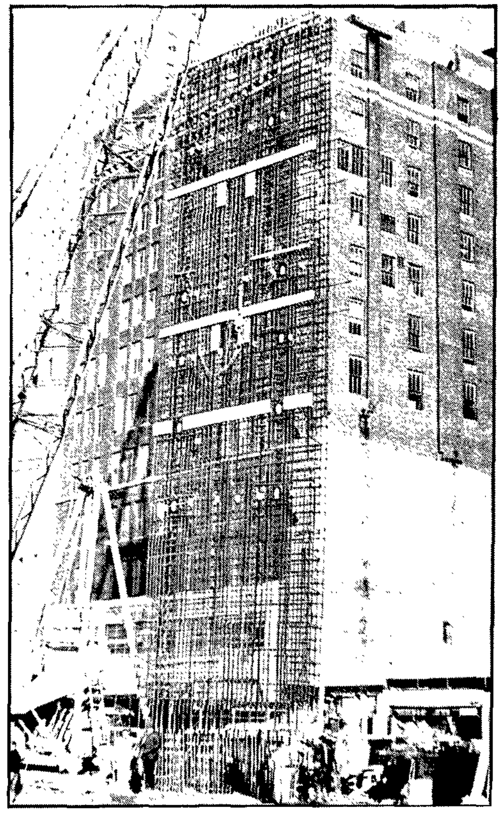


FIGURE 7

35 TON UPPER SECTION OF REINFORCING CAGE BEING SPLICED TO LOWER SECTION OF CAGE ALREADY SET INTO PANEL - PHOTO SHOWS WOOD SHEARKEY INSERTS, TIEBACK TRUMPETS & SPACER WHEELS - WORLD TRADE CENTER, NEW YORK

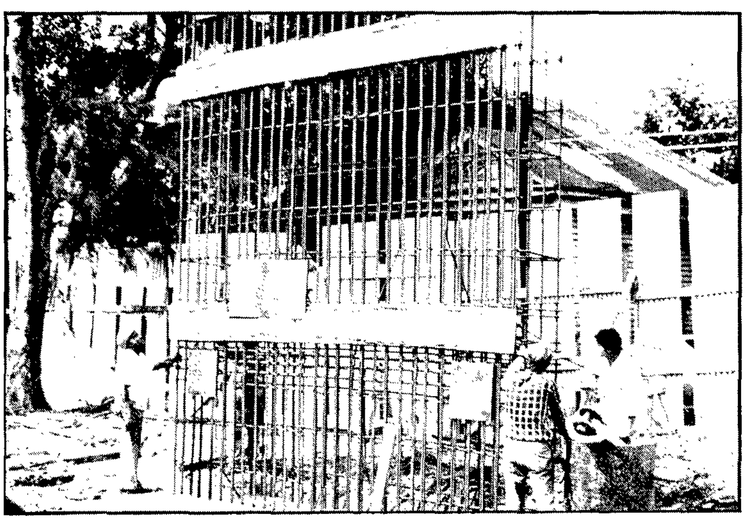


FIGURE 8

TYPICAL REINFORCING CAGE WITH STYROFOAM KEY INSERTS, BEND DOWEL BARS & BEARING PLATES FOR BRACING - COBIAN'S PLAZA PROJECT, SAN JUAN, P.R.

FIGURE 9

EXTRACTION OF STOP END PIPE

FIGURE 10

TYPICAL 'HALF ROUND' JOINT FORMED IN PRIMARY PANEL. NOTE DOWELS FOR CONTINUATION OF WALL BY CONVENTIONAL CONSTRUCTION

FIGURE 11

PREPARED TO PLACE CONCRETE - NOTE TREMIE HOPPER, PUMP TO REMOVE EXCESS BENTONITE & PROTECTION TO FACE OF BUILDING. DIAPHRAGM WALL WAS CAST DIRECTLY AGAINST BASEMENT WALL - ARCHER AVENUE SUBWAY, NEW YORK

80' LONG PRIMARY ELEMENT CONSISTING OF TWO WIDE-FLANGED BEAMS & LIGHT-WEIGHT REINFORCING CAGE FOR 'SOLDIER BEAM & CONCRETE LAGGING' METHOD.
D4B SUBWAY, WASHINGTON, D.C.

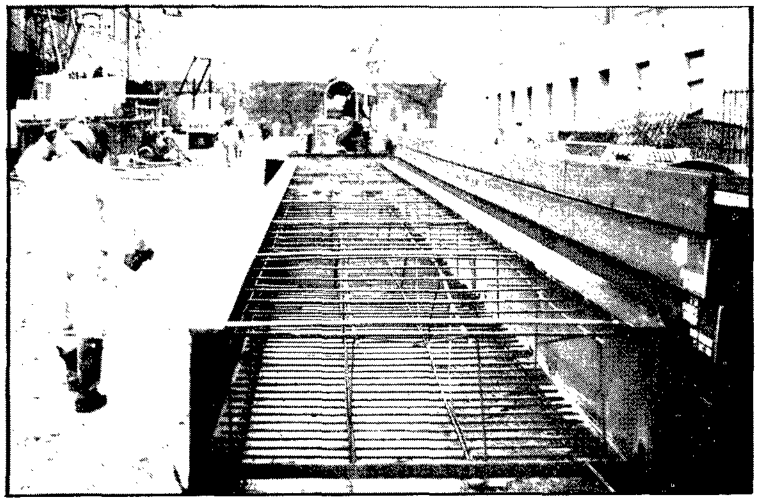


FIGURE 12

FIGURE 13

INSERTION OF PRIMARY ELEMENT INTO EXCAVATED PANEL
D4B SUBWAY, WASHINGTON, D.C.

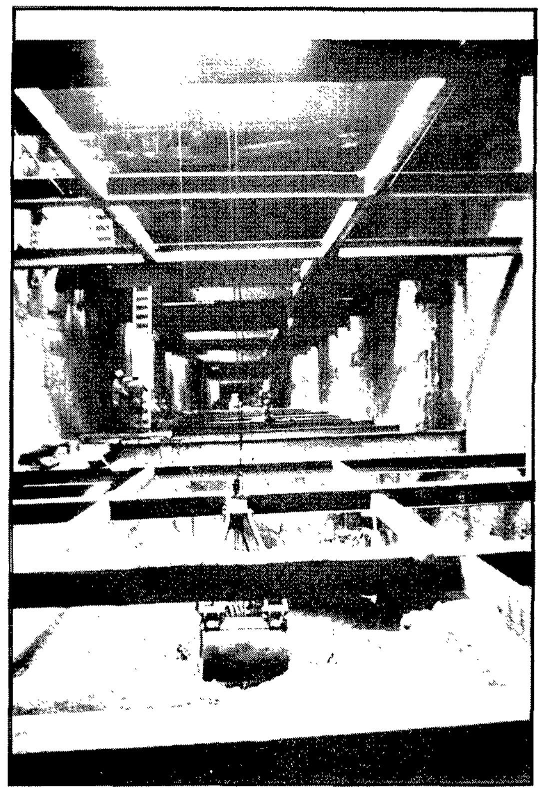


FIGURE 14

DIAPHRAGM WALLS LINE BOTH SIDES OF THE CUT - NOTE WIDE 16' VERTICAL BY 15' HORIZONTAL BRACING SPACING
ARCHER AVENUE SUBWAY
NEW YORK, NEW YORK

END PRODUCT - WALL WAS BRUSHED CLEAN OF SOIL & BENTONITE, PATCHED AS NECESSARY & THEN PAINTED WHITE - COBIAN'S PLAZA PROJECT SAN JUAN, P.R.

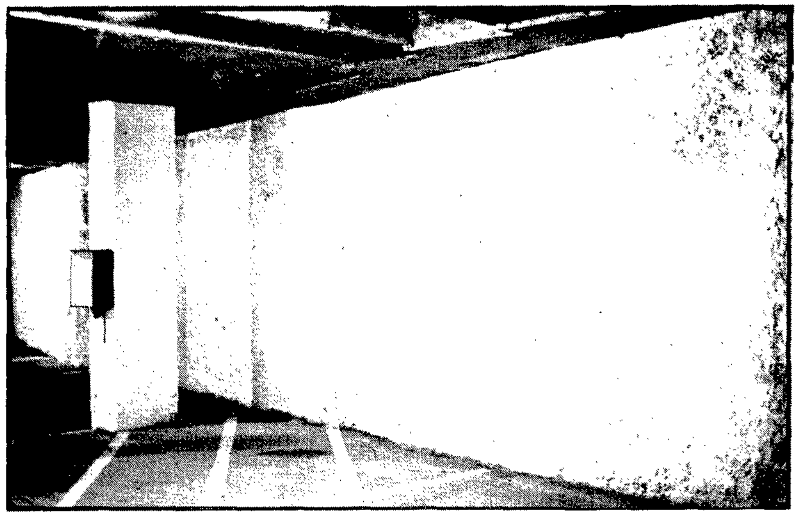
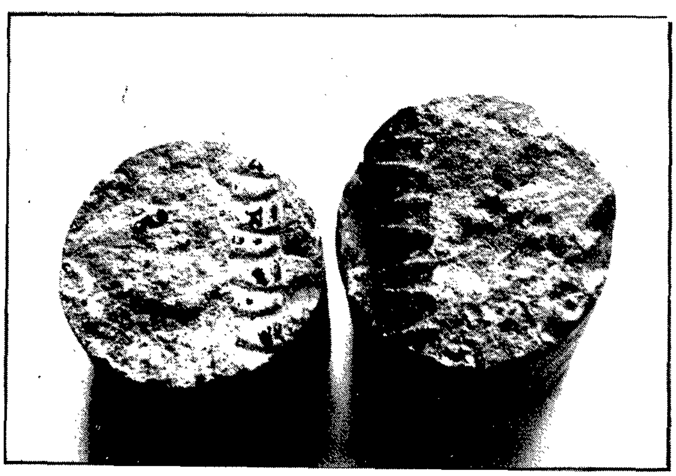


FIGURE 15

CLOSEUP OF COBIAN'S PLAZA WALL

FIGURE 16



CORE TAKEN FROM DIAPHRAGM WALL - BAR IS STILL BONDED TO CONCRETE - NOTE MILL SCALE ADHEARING TO FIRST SURFACE OF CONCRETE - FIRST WISCONSIN CENTER MILWAUKEE, WISCONSIN

FIGURE 17

PAPER 19

Construction Fundamentals of Diaphragm Walls

Part II

Arturo L. Ressi di Cervia, Ph.D.
Executive Vice President
ICOS Corporation of America
New York, New York

PAPER 19

CONSTRUCTION FUNDAMENTALS OF DIAPHRAGM WALLS

PART II

by

ARTURO L. RESSI di CERVIA, Ph.D.

Executive Vice President
ICOS Corporation of America
New York, New York

INTRODUCTION

After the paper of my friend and colleague, George Tamaro, who told you everything there is to know about slurry walls from the design and engineering viewpoint, I will make some comments on the more down to earth aspects of the business - namely, I will address myself to how to build diaphragm walls and how to utilize the technique to the best financial advantages.

MANAGEMENT

Construction companies specializing in foundations share with all other business activities a very unfortunate trait: the need of being managed. I say "unfortunate", since I believe that management, or lack of it, is the single most important cause of business failure - and particularly so in the foundation world. When a company specializes in an unusual type of construction like slurry walls and slurry trenches, it is forced to operate on a market limited in volume but farflung to the four corners of the world. Furthermore, it must employ people who are highly specialized, highly mobile and, even people in my field would cry in anger if they heard this last remark, highly paid.

The achievement of a high degree of professionalism and the ability to swiftly channel man and resources in the most productive way, and to the most remote places, is the mark of success of a company engaged in this field of work.

As a last general remark I want to add that companies specializing in this field of work are only a handful - since this type of company can only prosper if it maintains a reputation of high quality standards which is difficult to achieve and very easy to loose. A failure on one construction site is quickly known throughout the industry and a specialized company is only as good as the worst of its project managers. That is why it is of primary importance that field people be well trained, highly qualified and thoroughly supported by main office staff ready to help, advise and intervene when difficulties arise in the conduct of a given job.

SITE CONDITIONS

Whenever we are faced with the problem of estimating the feasibility or the cost of a diaphragm wall project one must take several actions in order to guarantee the proper evaluation of the construction costs.

1. Site Inspection. A thorough understanding of all factors relating to a particular site must be achieved in order to avoid surprises at a later date. Weather factors like precipitation and average temperatures must be considered together with means of access to the site for personnel, equipment and supplies. Headroom problems created by overhanging structures, power lines and other limitations must be examined together with all general logistic problems including accommodations for personnel, sources of supplies and availability in the general area of basic services.

2. Ground Conditions. Only where all the above investigations are completed, a detailed inquiry must be made on the geology of the area and, most specifically, a clear understanding must be achieved of the actual soil wall. Beside the constraints above the ground, one must consider all limitations below ground starting with the most obvious obstacles like structures interfering with the alignment of the wall, but continuing with the analysis of other situations like the presence of adajacent basements, large drain sewers or pipelines close to the wall, etc.

3. Utilities. Utilities and Services can become a major problem, particularly in urban job sites and their influence on the total outcome of the job must be thoroughly understood since it tends to be underestimated. When utilities or services cross the alignment of a slurry wall, there are three basic ways to cope with the problem.

1) While guide walls are being constructed a crew disconnects and relocates old utilities interfering with the construction of the wall. This is the best way and the least expensive for the slurry wall contractor if it can be accomplished ahead of the construction of the wall.

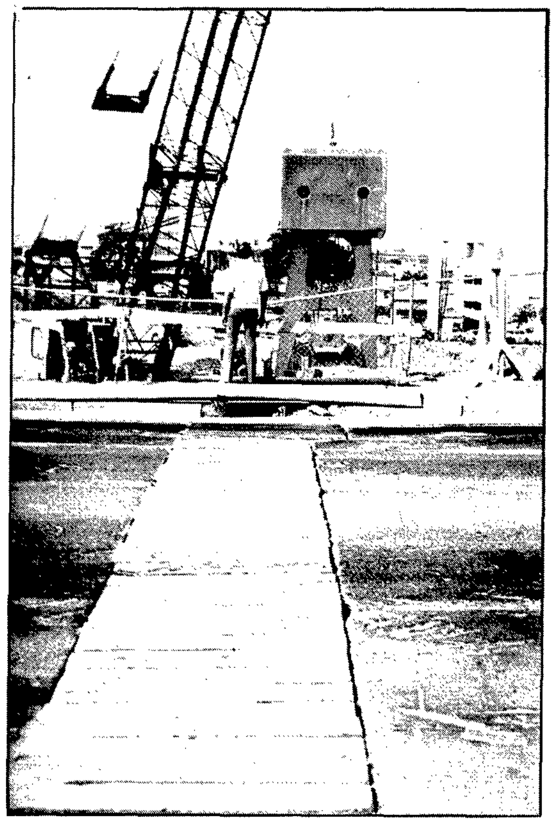
An alternate way consists in moving a utility laterally just far enough so that a panel can be completed and then swing it back in place over the completed section. This second way necessitates a continuous coordination between crews moving utilities and crews excavating the wall, and generally results in substantial delays in both operations. Such delays can become of gigantic proportions if the relocation of utilities must be done by agencies like the telephone company or some utility companies which require advance notice and do not allow for any flexibility in change of schedules. Furthermore, the presence of live utilities adjacent to existing excavation is a potential source of mistakes, and since some panels are clear of utilities and others are not, this always results in a number of accidents where live utilities are damaged.

A very important precaution to be followed whenever utilities are relocated is the plugging of any old pipe sleeve left in place to avoid bentonite losses through those conduits during excavation.

2) When relocation is not possible or economically feasible, certain types of equipment, namely free-hanging clamshells, can work around existing utilities and complete a panel leaving the utilities in place. The usual procedure consists in exposing the utilities so that they are visible in the trench, protecting them with a cap and then offsetting the clamshell below the level of the utility in order to complete the excavation. Reinforcing steel cages can be inserted, if required, by making them in vertical sections.

3) When neither of the previous solutions is possible or economical, the wall is stopped on either side of the utility and the remaining gap is closed during the excavation phase by conventional logging or underpinning methods.

Figure 1 shows a job in Puerto Rico where the wall, after hugging an existing building, crosses a thoroughfare where utilities were left in place and then acts as a sidewall for the under-the-roof type of excavation typical of the "Milan Method".



Wall crossing a street in Puerto Rico; the wall is completed and covered to allow traffic resumption.

FIGURE 1

METHODS AND EQUIPMENT

Only when all these factors are understood and evaluated and the estimator has a clear picture of the job and its problems, a preliminary selection of equipment and construction methods is done. In order to give an idea of how this selection is made I am going to list under three different categories the most commonly used type of equipment with their most important characteristics as they relate to the above mentioned problems:

1. For Slurry Trench Equipment. In order to understand the terms, I am defining Slurry Trench as a continuously excavated diaphragm wall filled with a plastic mixture of soils and bentonite. This type of diaphragm wall is the fastest and cheapest way of achieving a water-tight cut-off under certain circumstances, and it can utilize to a great extent standard excavation equipment.

Shallow cut-offs (up to approximately thirty feet in depth) can be most easily excavated with backhoe type of equipment (See Figure 2), while deeper ones (up to approximately eighty feet) are most effectively done with drag lines (See Figure 3). Only when the depth exceeds eighty feet, the use of clamshells is required (See Figure 4).

2. Equipment for Slurry Walls in Soft Medium Soils. The three types of equipment most commonly used in slurry wall construction in soft to medium soils are:

a) Free-hanging clamshells of the hydraulic or mechanical type (See Figures 5, 6 and 7). These clamshells are almost totally trouble free and can be easily transported, even to remote localities. Furthermore, they allow any positioning of the excavating rig with respect to the excavation progress, as long as it is within the radius of the reach of the crane.

b) Another common type of excavating tool is the hydraulic or mechanical clamshell operated by kelly bar (See Figure 8). While this equipment has a higher production rate it is more cumbersome, more expensive to operate, and it can less easily cope with space limitations in a job site.

c) The third type of commercial excavating tool is a reverse circulation drill of Japanese manufacture (See Figure 9). This tool requires special conditions of the site and its applicability is limited usually to urban sites.

FIGURE 2

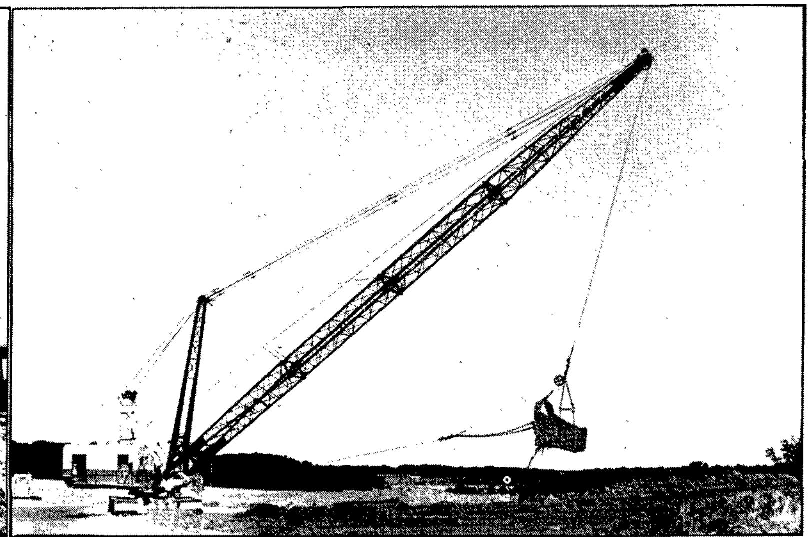


FIGURE 3

▲ Excavation with dragline bucket for a slurry trench cutoff wall.

◀ A conventional backhoe, used for slurry trench cutoff walls.

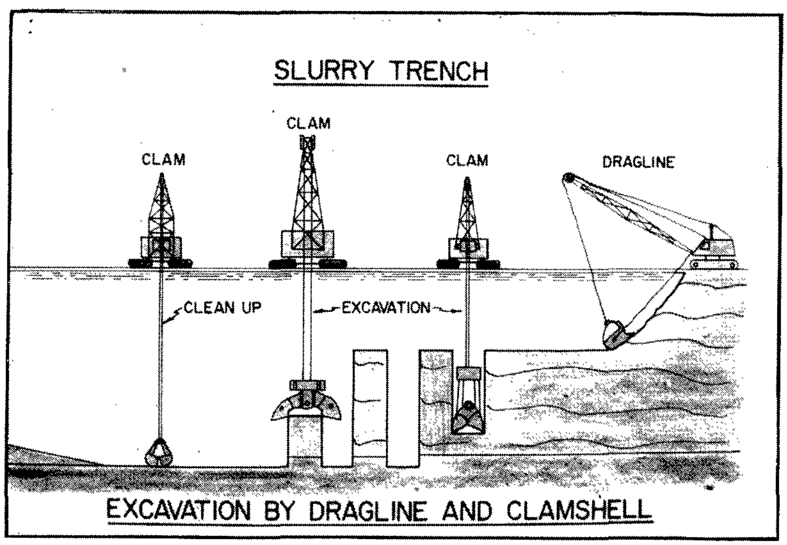


FIGURE 4

Excavation with dragline and clamshell.

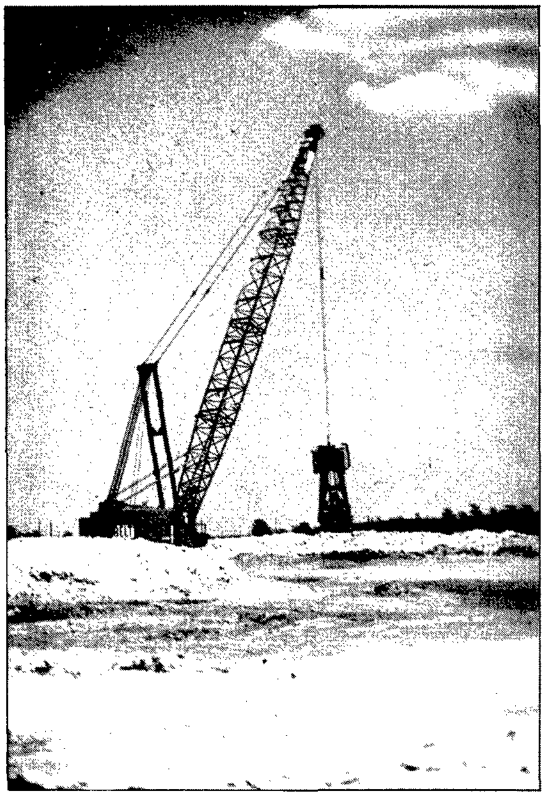


FIGURE 5

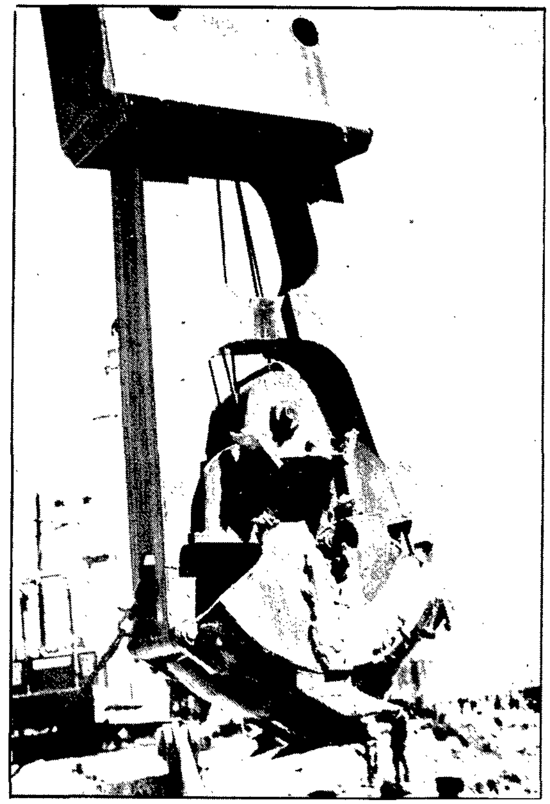


FIGURE 6

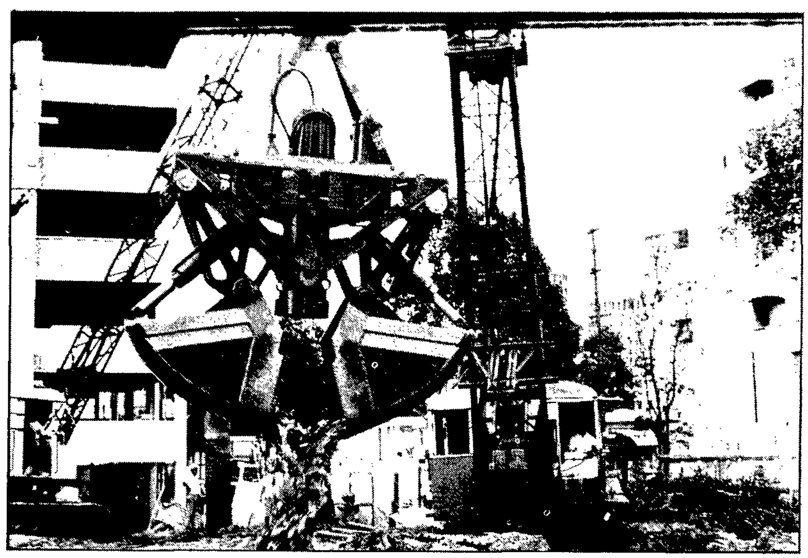


FIGURE 7

Figures 5, 6, and 7: Free hanging clamshells, either hydraulically or mechanically operated.

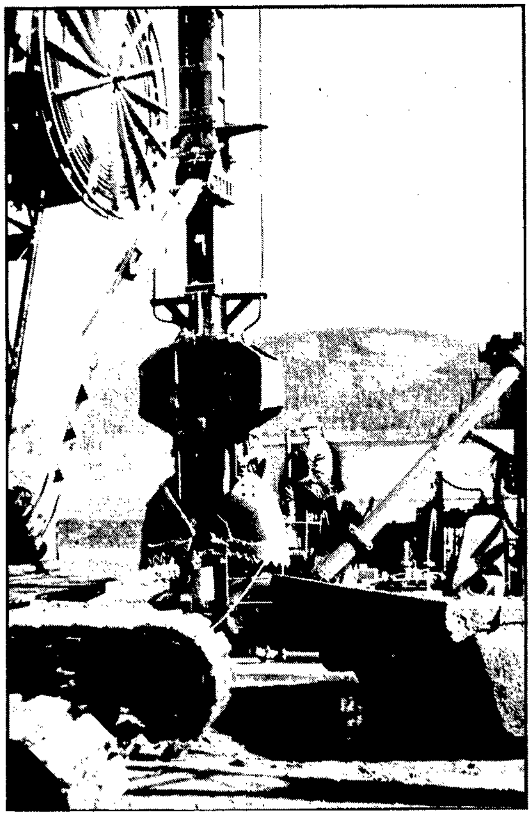
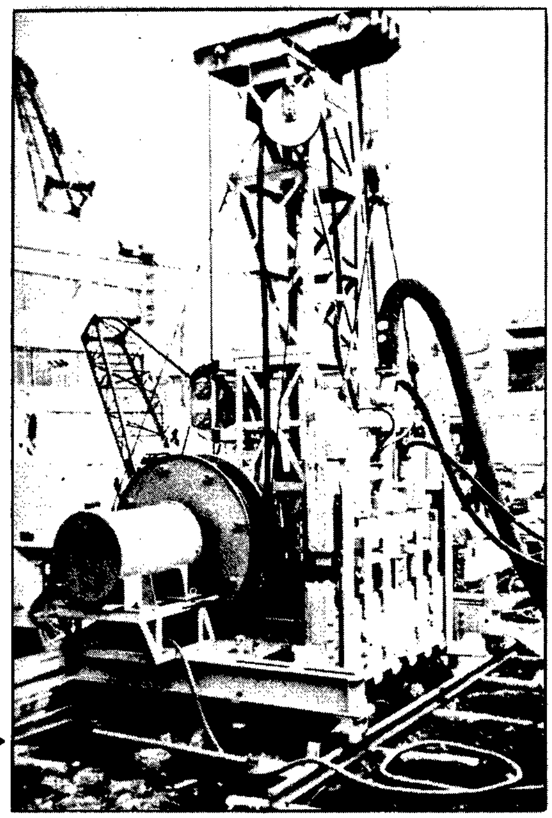


FIGURE 8

▲
A clamshell operated by kelly bar.



Reverse circulation rotary ▶
drilling equipment made in
Japan.

FIGURE 9

3. Equipment for Excavation in Hard Soil Formations or Rock. When the excavation has to be carried out in harder soils, percussion or rotary methods have to be employed. From simple chisels operated by a crane (Figure 10) to reverse circulation percussion drills, the progression of equipment follows a progression from less to more difficult soil to be drilled. When massive quantities of rock or boulders like the one shown in Figure 11 have to be confronted, the use of rotary methods is often recommended.

Conventional chisel operated by a crane.▶

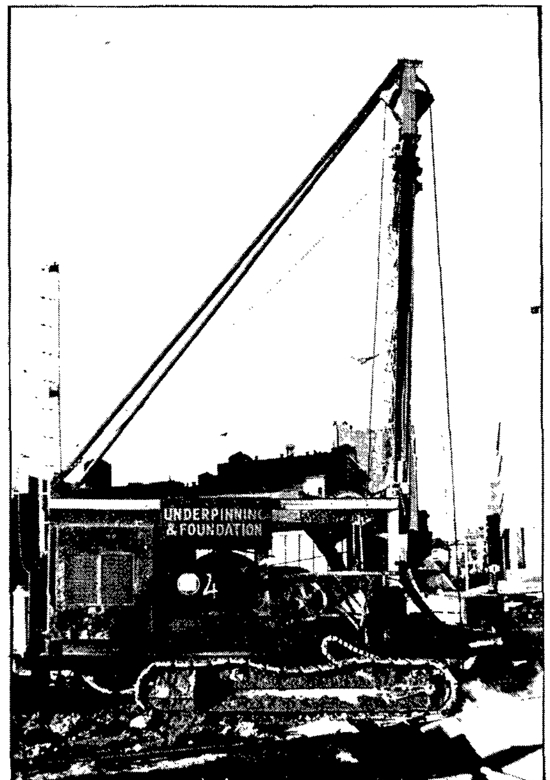


FIGURE 10

Massive boulders requiring special rotary-type equipment.▼

FIGURE 11

With all these tools at his disposal the estimator must choose one of them, or any combination of them, to achieve the most economical result within the required time frame and within the existing restrictions.

SITE ORGANIZATION

The proper organization of a worksite can be a significant factor in the economic success of a job. A properly laid out site plan where offices, shops and work areas, fabrication space, material storages and parking areas are intelligently laid out, can avoid costly delays during the conduct of the job. An intelligent way to handle and remove waste materials, a proper layout of bentonite and water lines, an efficient bentonite plant - are all important factors in the economic equation.

Last but not least, one must study an environmentally acceptable and economically viable system of disposing the bentonite mud. It is our experience that closed circulation mud systems give great flexibility, reduce mud costs and require only occasional waste of used mud during the course of the job. Since gone are the days where mud could be just pumped into a sewer or into a river without thinking of the long range consequences, today one must plan to dispose the mud either by carting it or pumping it to designated disposal areas - or, more efficiently, breaking it down chemically and wasting it together with other solid wastes.

COSTS

From the brief panorama which I have just described, you can probably surmise that the cost of such walls varies greatly from one project to the next. To give you a frame of reference, our company is presently constructing a slurry trench in New Jersey at an approximate cost of $3.00 per square foot, and a diaphragm wall in Kentucky at an approximate cost of $200.00 per square foot. The purpose of both these installations is strictly to obtain a waterproof barrier. It goes without saying that while a majority of the projects will fall within a much narrower bracket of prices, it is still very difficult for the layman to properly appreciate even the approximate costs of a diaphragm wall unless he secures the services of contractors or consulting engineers who are familiar with this type of construction.

I trust that all of the above has given you some insight into the problems faced by a contractor operating in this field.

PAPER 20

Review of New Technology for Underground Construction in Japan

Toshio Kato
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PAPER 20

REVIEW OF NEW TECHNOLOGY FOR UNDERGROUND CONSTRUCTION IN JAPAN

by

TOSHIO KATO

Manager
Sumitomo Construction America, Inc.
Chicago, Illinois

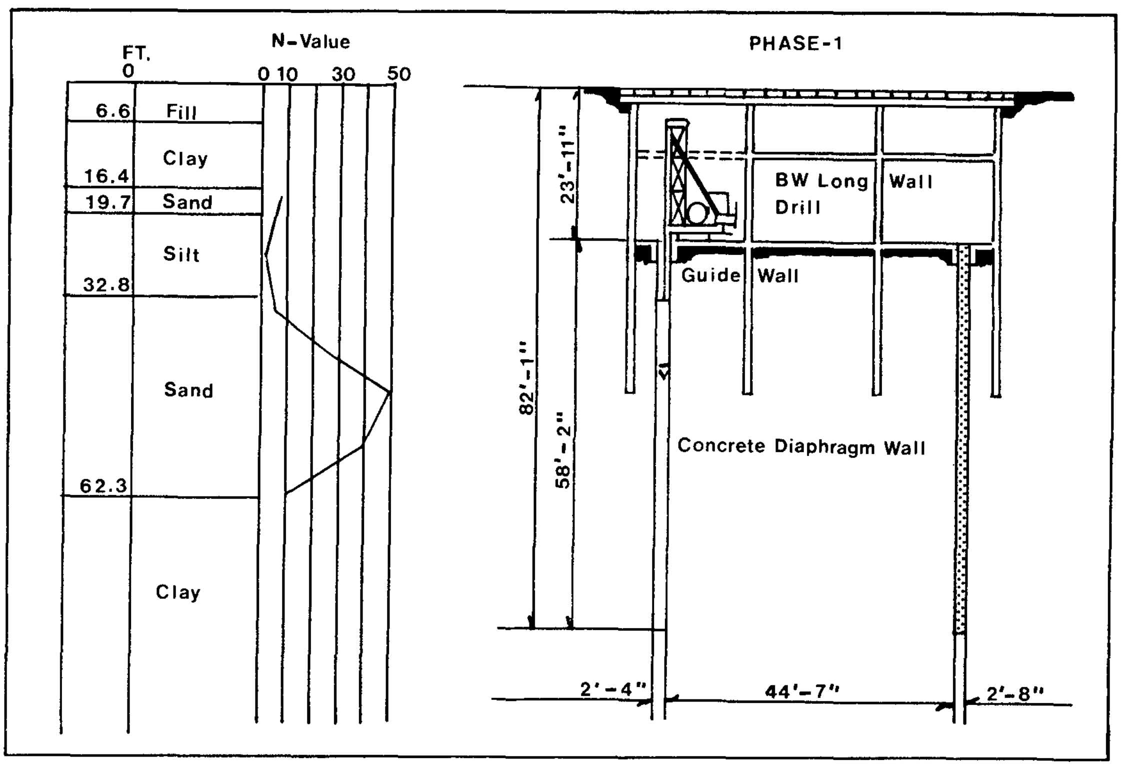
The purpose of this brief paper is to review diaphragm wall construction in Japan with emphasis on excavating techniques, uses and applications. The first underground wall using the slurry trench process was first built in 1959 at a dam site to provide a cutoff. Since then the total area of diaphragm walls constructed up to the present time may well be in excess of 1.5 million square meters at a rate of more than 200,000 square meters per year. The explanation for this volume is an unprecedented demand for underground space, environmental restraints in the noise and vibration levels, and improvements in the technical aspects of the technique. Problems in the disposal of excavated materials and used slurries have affected construction costs, but have resulted also in the development of slurry systems that can readily be converted into disposable muds. As underground construction reached record levels, the associated problems were intensified due to lack of space and restrictions in scheduling the operations.

In view of these unfavorable conditions, a concerted effort was made to control the economic aspects and ensure that underground construction would continue to be feasible from the cost-benefit viewpoint. It thus became necessary to consider, try, and often combine every alternative and type of construction placing emphasis on new technology and techniques in order to increase feasibility and decrease costs. For example, more than 15 excavating systems are available, some brought from abroad and some developed independently by the local market. These include the conventional clamshells and chisels suspended on wire-rope rigs. Cutter types are widely used and include rotary drilling systems. Bentonite is not as plentiful as it is in North Africa, Europe and the United States. This shortage forced the development of new colloids for use in slurries, which almost have replaced bentonite in the local market.

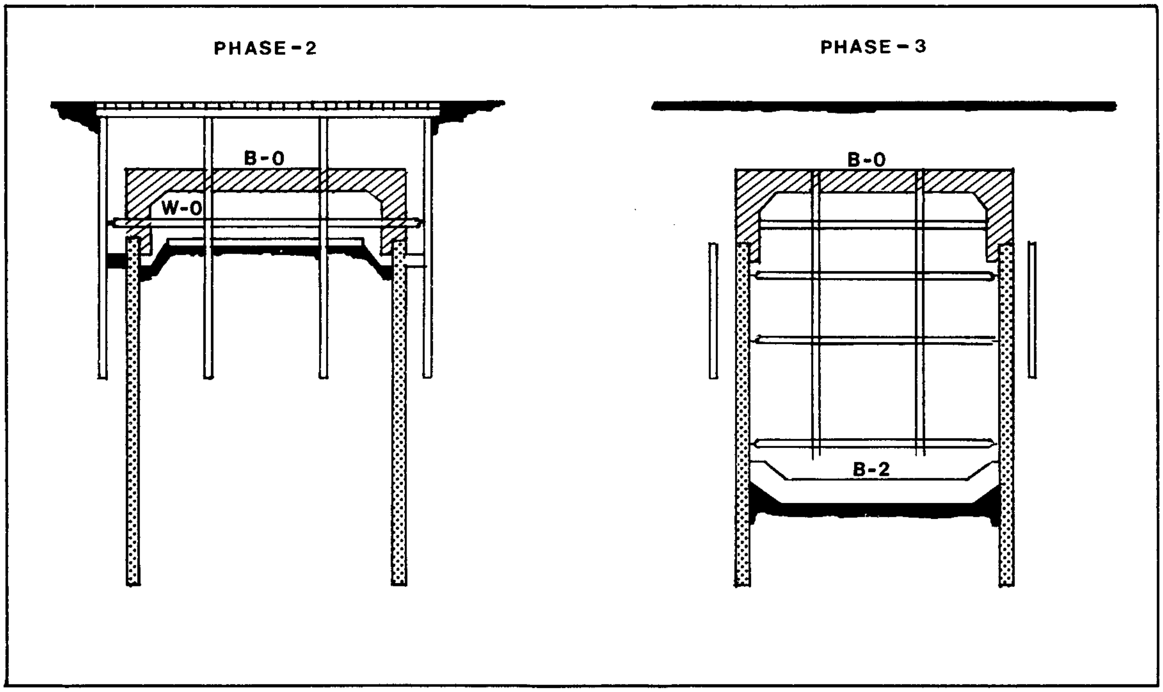
THE REINFORCED CONCRETE (RC) SLURRY WALL

The RC slurry walls, also called diaphragm walls, have been developed in three basic steps. In the first stage (1955-1960) the RC slurry walls were used primarily as temporary support systems in lieu of more conventional walls such as sheet piling and soldier piles with lagging. The merits of this application were in providing a more rigid earth retaining system, protecting the excavation more effectively from underground water, and reducing the excavation volume since no space or clearance were necessary between the temporary wall and the permanent structure.

During the second stage (1961-1967) the diaphragm wall was introduced and tried as part of the structure, and in this respect it was constructed as permanent wall. An example is the diaphragm wall shown in Fig. 1, illustrating the three phases of construction for a subway section. In this case, the RC slurry wall was installed from a lower construction level first as temporary support, but was later incorporated in the permanent structure. During the second stage many problems and questions had to be studied, solved and answered. These included the vertical accuracy of the wall as part of the construction tolerance, surface finish, vertical construction joints for watertightness, and horizontal connections.

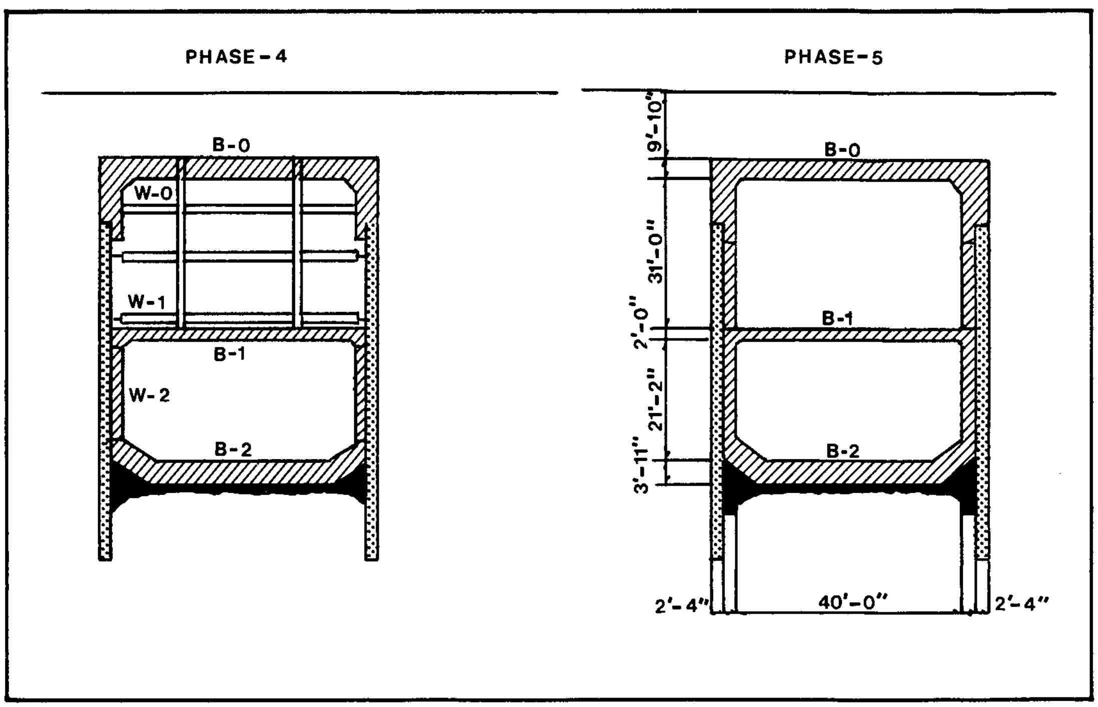


Construction Sequence of Composite Wall **FIGURE 1(a)**



Construction Sequence of Composite Wall

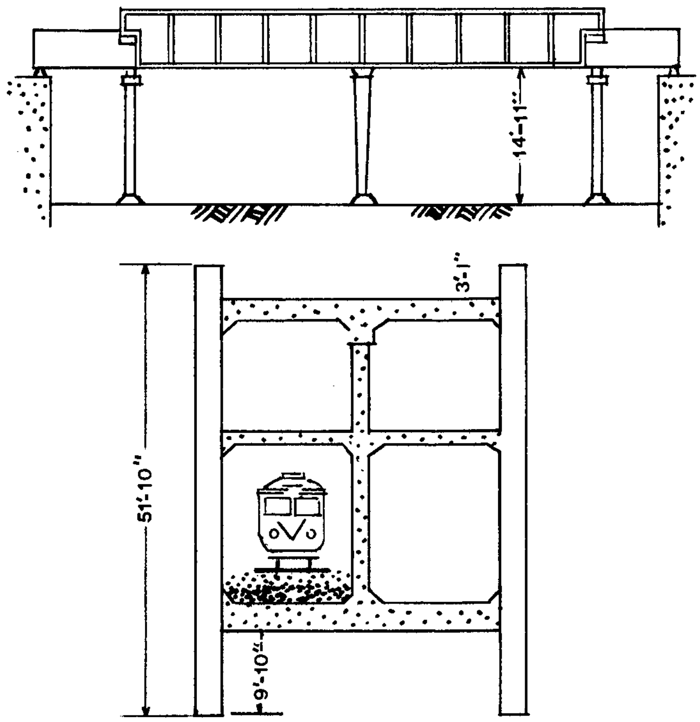
FIGURE 1(b)



Construction Sequence of Composite Wall

FIGURE 1(c)

CROSS SECTION



Ft. In.	Ground Formation	N
5'-7"	Surface Soil	
10'-2"	Sand/Gravel	40
16'-5"	Fine Sand	25
42'	Silty Fine Sand	20
51'-2"	Sandy Silt/ Gravel	20
	Sand/ Gravel	30

Structural Wall

FIGURE 2

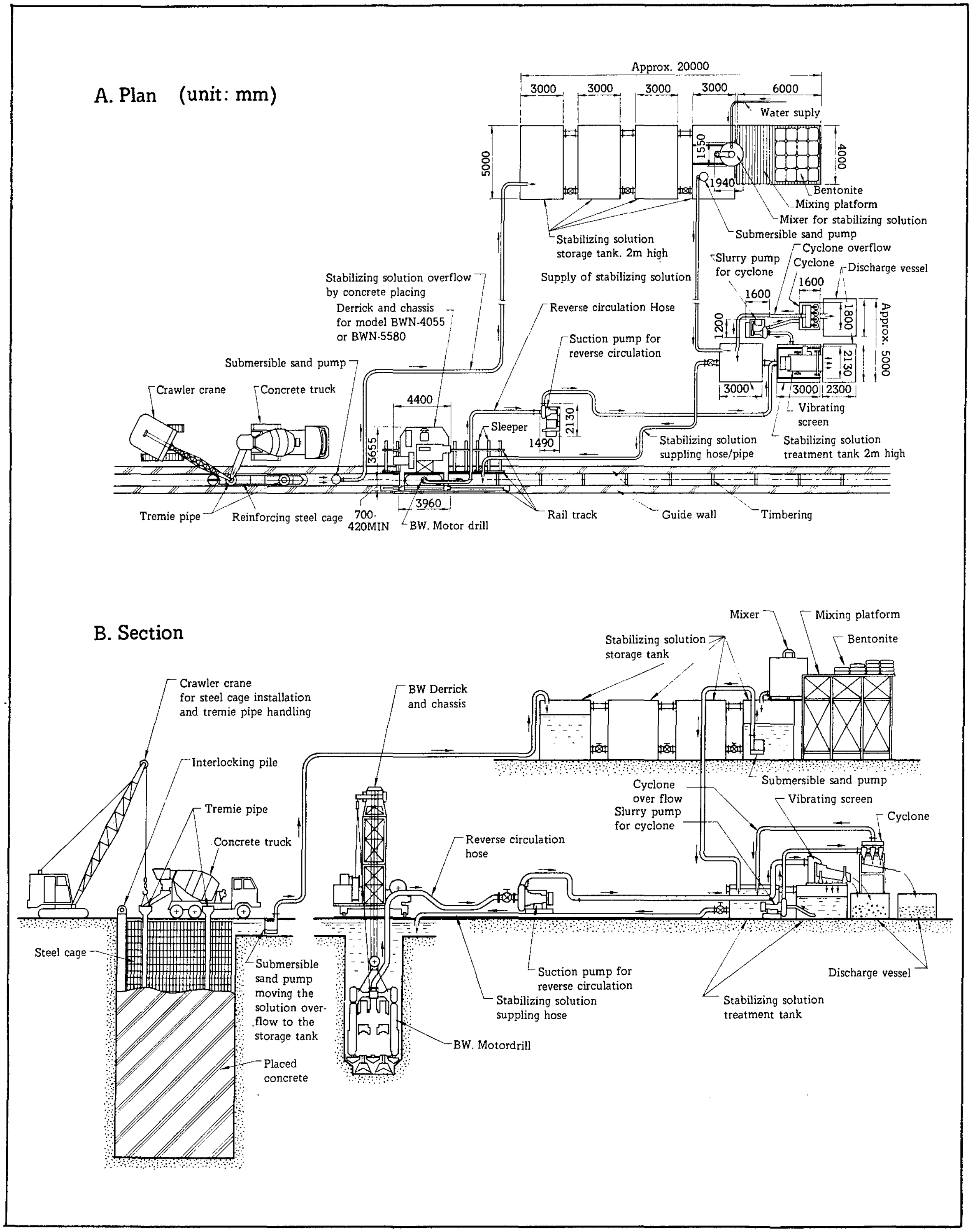
The third stage (1968-present) was probably the most significant and decisive in the history of new technology, and the RC slurry wall was adapted as permanent structure. Appropriate provisions were drafted and included in most government and municipal codes and in the Japanese structural standards. An example of a diaphragm wall built as permanent structure is shown in Fig. 2, in which the wall provides the final side wall of the section and is an integral part of the structural system. During the third state, the all important interaction with different structural elements was given proper attention. Long research programs had to be initiated, and still are ongoing, to determine various technical and construction aspects such as the resistance of diaphragm walls to seismic forces, allowable concrete stresses, bond requirements and bar splice details, structural connections, methods of concrete placement, methods of load transfer, capacity of diaphragm walls as load bearing elements, waterproofing requirements, excavating techniques, and related functional and architectural aspects.

EXCAVATING TECHNIQUES

As mentioned there are various techniques and excavating systems. Of these cutter-type rigs and rotary drills have been very popular, especially when they are equipped with reverse circulation.

The selection of an excavating system often must be based on the type of ground to be excavated. Soft clay and, loose sand can be excavated with almost any machine but difficulties arise where it is necessary to excavate in hard cohesive layers, consolidated gravel and bedrock. There is no excavator currently available which is suited to work under these ground conditions, and some systems may show a serious drop in efficiency or even complete inability to perform. On the other hand, the ideal endeavor for any contractor is to use a process and machine that will enable excavation to be completed according to the design and at the least possible cost.

In the category of rotary drilling systems are drilling rigs and machines for slot or circular excavation. Several rotary drill bits loosen the soil through simultaneous action aided with side cutters which move vertically to cut and trim soil not reached by the bits. The soil is consolidated into individual grains or cuttings as opposed to grabs and clamshells which excavate soil in bulk. The cuttings are mixed with slurry at the cutting face and removed by reverse circulation.



General Plant Arrangement of BW System (Example) **FIGURE 3**

An important item was the finish of the exposed wall, and the requirements for final treatment. Usually, the architectural standards require a separate finish and treatment for the interior of buildings and subway stations, but in covered subway lines and underground parking areas no finish is required except shotcrete and sprayed mortar in some instances.

During the final stage in the development of the technique the advantages were better understood and more explicitly defined; thus whenever the RC slurry wall can be used as permanent structure maximum benefits will be attained by eliminating the temporary shoring, underpinning, dewatering and formwork, and by corresponding savings in construction time.

In very deep projects, particularly those involving excavations deeper than 100 ft, the exceptionally high earth lateral stresses in the lower part of the structure often require unusually thick walls, sometimes 4 to 5 ft. In this case it is neither necessary nor economical to use this thickness for the entire depth of the wall. The most advantageous construction is obtained if a nominal wall thickness is used for the diaphragm wall (2 to 3 ft), and then a separate wall is attached to the initial by means of shear connectors in the deeper part of the structure as excavation progresses to this point.

USES AND APPLICATIONS

Of all uses and applications more than 50% by volume are for deep building basements, parking facilities, and new subway lines in cut-and-cover construction. In building construction more than 70% of the wall are permanent installations and are incorporated in the final structure, but a smaller percentage is for permanent construction in subway uses. The extent of usage is, however, not dependent upon different standards and codes, but is rather a function of the technical aspects of the project and mainly the configuration of the section. The permissible uses are determined jointly by pertinent building codes and regulatory agencies.

Diaphragm walls are also required to function as seismic walls to resist these forces. For this purpose various systems in which diaphragm walls are integrally treated have been studied and adapted in practice Usually these include a combination of exterior walls, bored piles, beams and columns, built to form a closed structural framing unit, with special emphasis on the design and execution of the connections. The entire process, including the construction stage, is carried out by special licensed organizations.

Other uses and applications in Japan include diaphragm walls and related elements in box culverts, sewers, bridge and tank foundations, and cutoff walls for water supply and pollution control.

ASPECTS OF SLURRY WALL TECHNOLOGY

Considerable effort has been made to improve and rationalize the various technologies involved in the construction of diaphragm walls and their applications. Regarding the incidental aspects of the process, research has been undertaken for the pollution-free disposal of excavated soil and used slurry, methods to prevent the escape of slurry in very pervious ground, and the development of new colloid-type materials to cope with the shortage of bentonite in the local market and improve the efficiency of slurries. Additionally, the verticality of the excavation and its measurement and control is important in trenching operations in order to ensure high degrees of accuracy in diaphragm walls and improve the construction tolerances. Two different developments may serve as good examples.

Firstly, a promising step in the technology of slurries was towards the development and commercial production of a new stabilizing agent. Almost invariably, properly proportioned slurries are based mainly on the use of bentonite as the main colloid material. This is common practice throughout the world. Although bentonite is quite suitable for technological applications, it has certain limitations in that the efficiency of slurries is reduced upon exposure to salt and cement. A new agent known as Telmarch, basically a polymer stabilizer, has been introduced in the Japanese market, and is totally free from any change in its viscosity when mixed with cement. This material also facilitates segregation of slurry and contaminants without causing serious changes in the flow properties, hence the recycling of used slurries is possible to a greater extent. Telmarch can produce slurries of suitable flow properties at a fraction (usually 10-15%) of the bentonite content, hence it replaces seven to ten times the quantity of bentonite in slurries.

A second example of current research and development is a measuring apparatus for the control of verticality in the trench. Ultrasonic devices have been produced and are now used to check the face of the excavation, record the properties of slurry, and measure the volume of concrete deposited at a given time. These devices are ultrasonic waves and record automatically the trench conditions.

Excavation with rotary drilling systems is shown in Fig. 3, also showing the general plan layout at the site.

THE BW SYSTEM

The BW long drill is a power operated excavator having reverse circulation as standard feature. The machine (shown in Fig. 4 (a) and (b) is an extension of multiple auger-drill excavation, which is however aided by side cutters as shown in part (b) of Fig. 4 to excavate and trim soil not reached by the rotary drills and thus give a continuous slot configuration.

The system has operational characteristics which to great extent were dictated by construction conditions and requirements usually encountered in urban sites. Thus, the excavator was developed to provide: (a) manageable size and overall height to make it workable under limited overhead and confined space; (b) nominal overall weight properly distributed to avoid excessive live load surcharge which often is the source of trouble in soft and loose ground; (c) adjustable parts and components to accommodate variations in panel width and length; and (d) freedom from noise and vibrations.

The motor drill usually is operated from the standard rig shown in Fig. 9, but for increased mobility around corners and irregular excavations the drill can be mounted on crawler tractors. The complete assembly includes the following units and instrumentation: (a) control system, providing the hoist, cable reel, switch board, feed indicator, and deflection indicator; (b) suction pump, used to eject soil and cuttings mixed with slurry instantaneously and directly through the reverse circulation hose, and convey them to the mechanical separation units; (c) mud circulation units, usually a vibrating screen to intercept coarse soil and clay cuttings, and a cyclone to intercept finer soil particles; and (d) slurry mixer and storage tanks according to the job requirements.

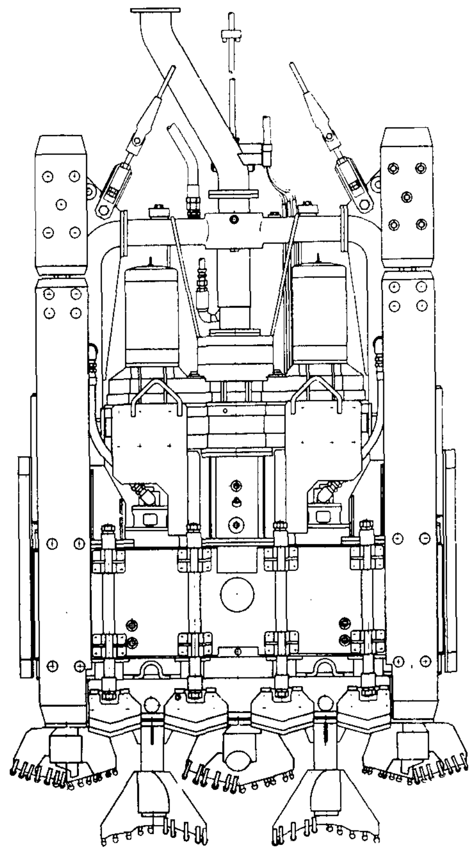


Figure 4 (a): Structural Scheme of motor-drill, Model BWN-5580

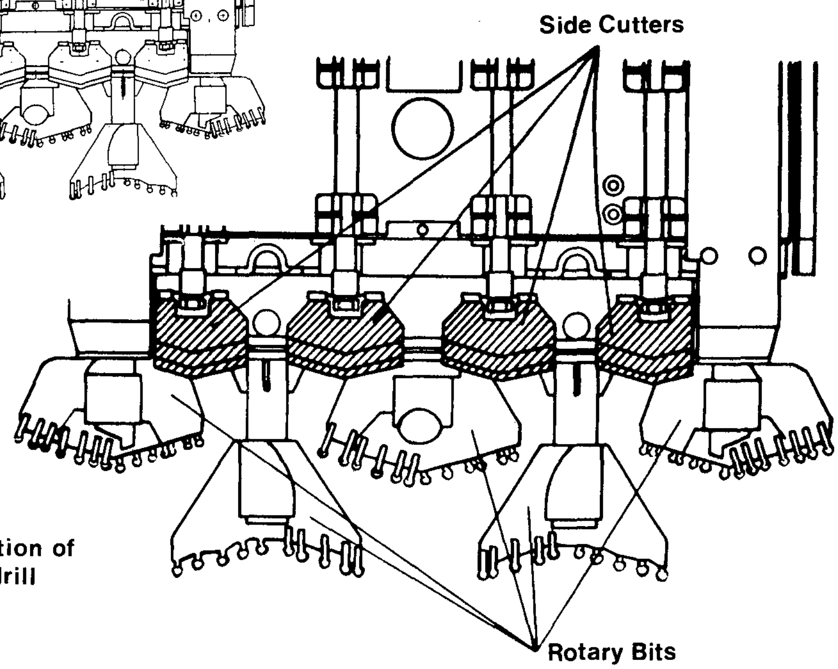


Figure 4 (b): Section of excavation by BW drill

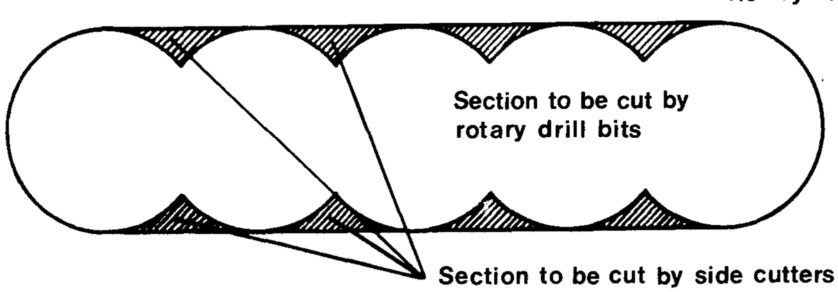
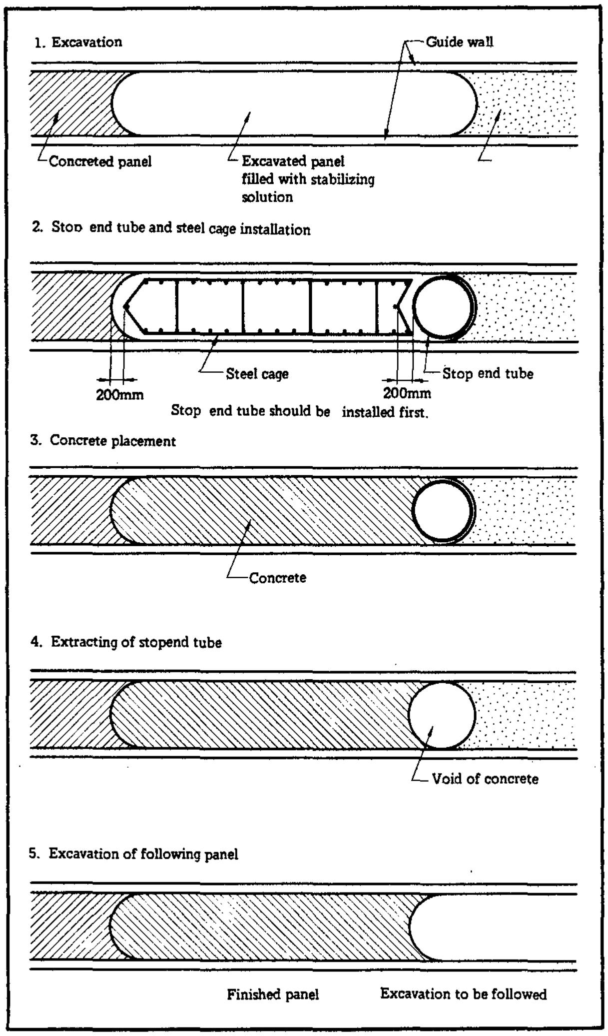
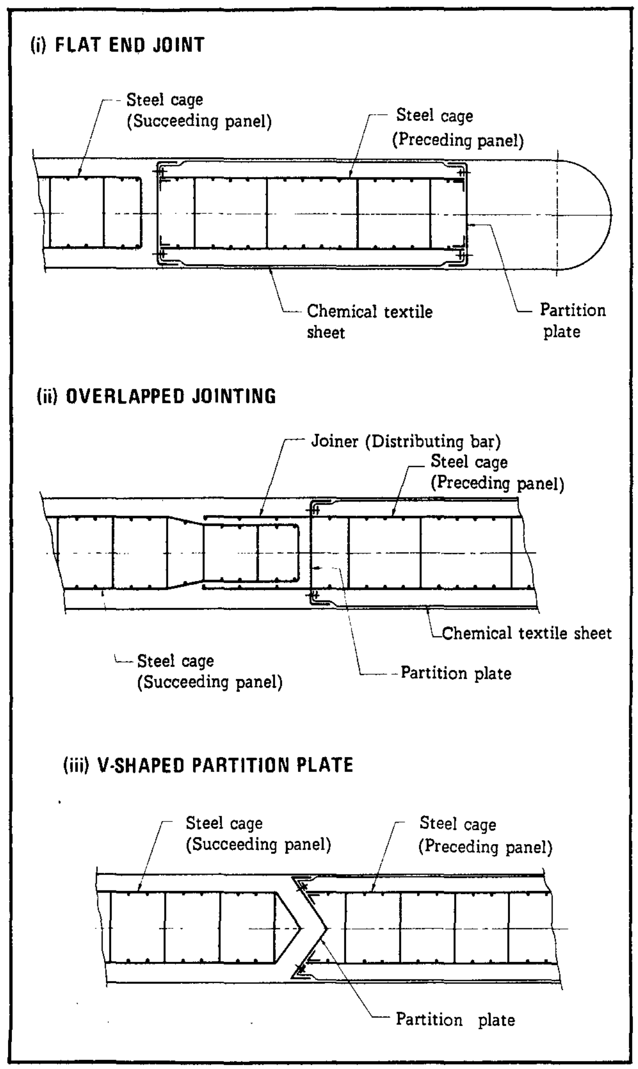


FIGURE 4



Sequence of Panel Jointing by stop-end tube method

FIGURE 5

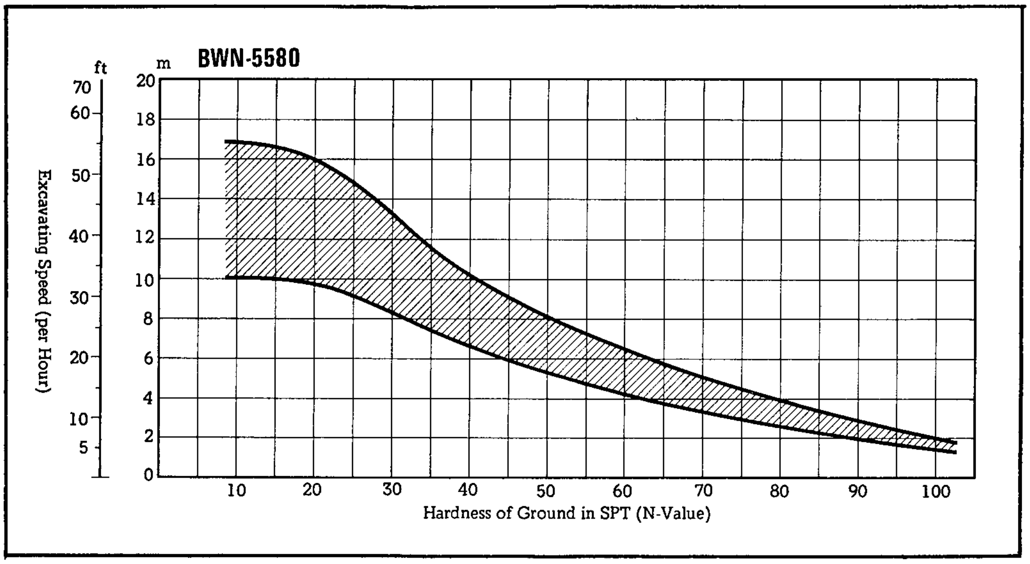


Example of Partition Plate Joint

FIGURE 6

Construction Joints. Two frequently used types of construction joints are shown in Figs. 5 and 6. Fig. 5 shows a very common and perhaps the simplest method to form a joint by inserting a round steel tube at the end of a panel. The stop-end tube must be slowly extracted after the pour is completed but before the fresh concrete begins to set, giving a half-round concrete key at the end of the unformed panel. This joint detail is suitable for excavations carried out with rotary drilling equipment, and is basically a shear connection.

Examples of the so-called partition-plate joints are shown in Fig. 6. The steel partition plate with the chemical textile sheet is used to stop the fresh mix from leaking into the end space. If structural continuity is desired through the joint, the reinforcement can be spliced as shown in the overlapped joint detail.

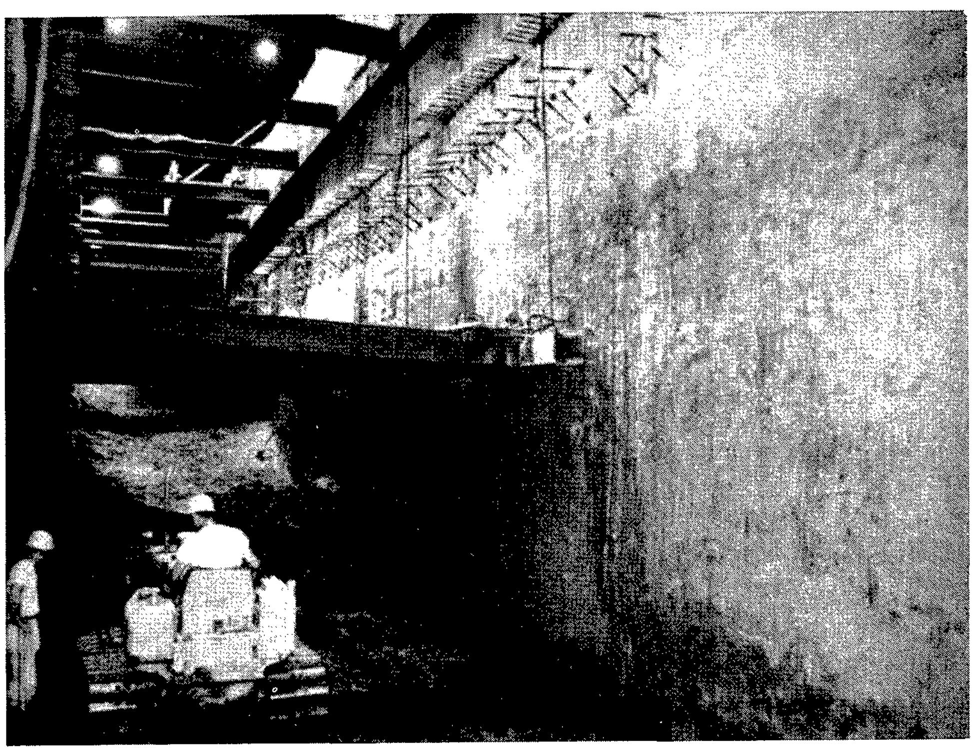


Performance Curve of Net Excavation by BW Drills **FIGURE 7**

Excavation. Fig. 7 shows the average performance record of excavation carried out with the BW system. These rates are for the vertical advance. Evidently, the best speed is attained in soft clay and loose sand, but this does not necessarily mean that the construction as a whole is more economical in these types of soil unless, of course, materials handling proceeds at comparable rates. On the other hand, the drilling cost becomes the most important single factor affecting the total construction cost where the excavation involves very hard layers and solid rock.

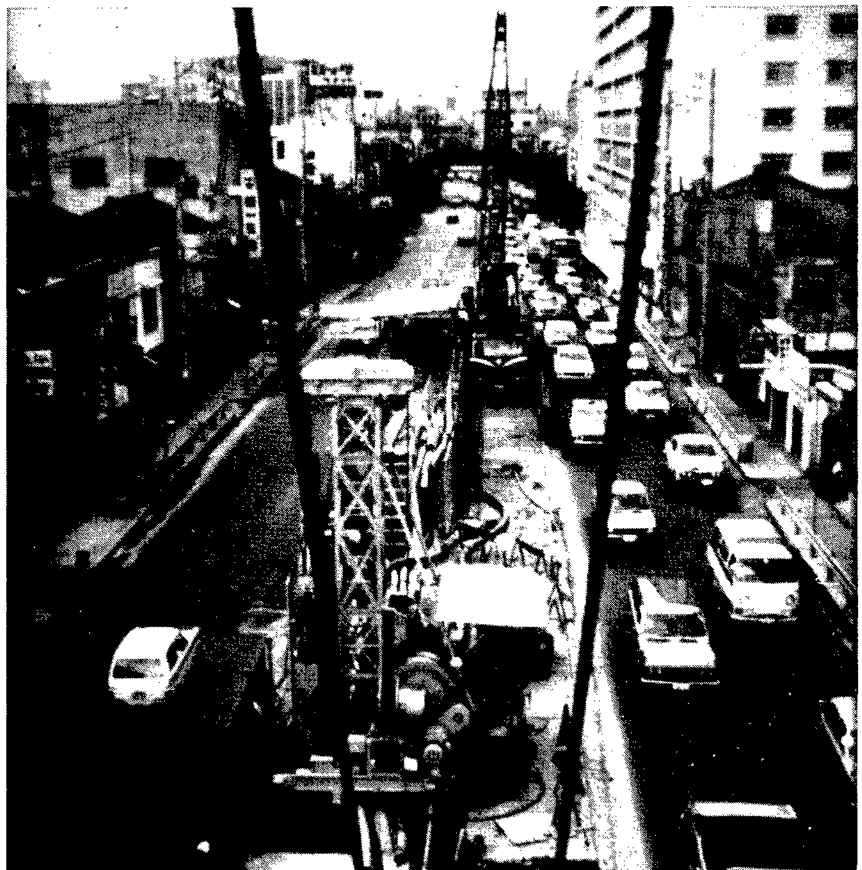
A most important phase of the excavation process is the separation of the excavated materials. This requires the use of suitable separation units or settling tanks. As mentioned the circulation initially goes through vibrating mud screens which intercept the coarser fraction of the materials and well preserved clay cuttings. Particles not intercepted at the screens must go through cyclones. This process is feasible if the specific gravity of the slurry (including the excavated materials) is less than 1.15 and the viscosity does not exceed 60 seconds, hence cyclones must be used together with mud screens for maximum efficiency.

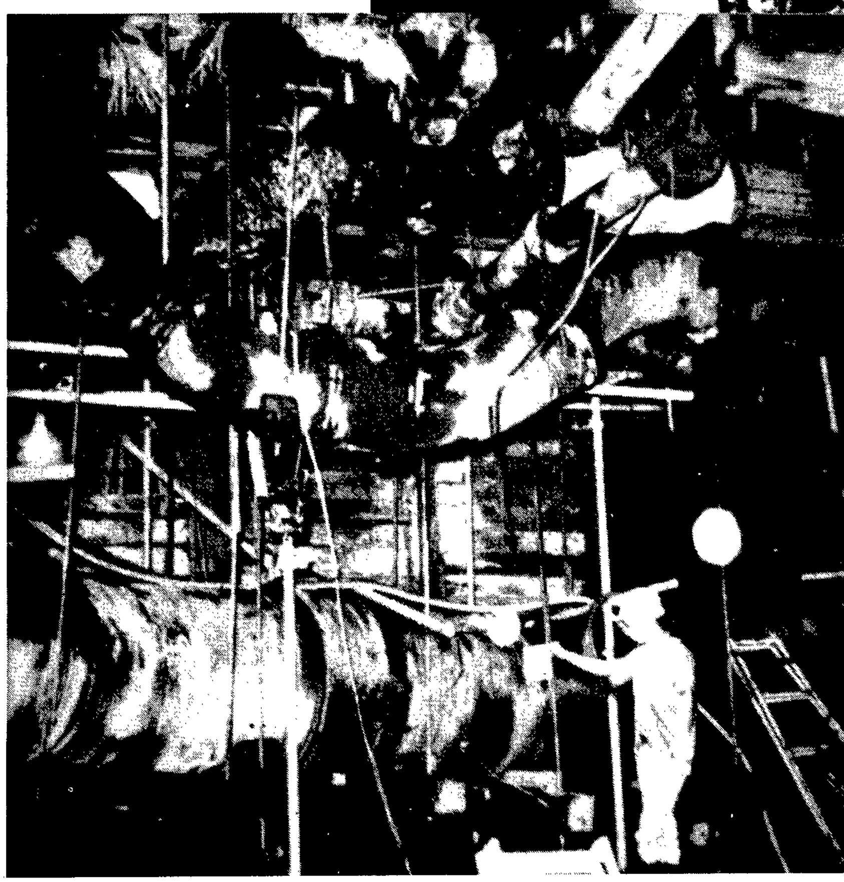
Uses and Applications. Figs. 8, 9 and 10 show three common cases of diaphragm wall construction, all in urban sites and in congested areas. The first example (Fig. 8) shows diaphragm walls built with the BW system as permanent walls of a subway section in Tokyo. Fig. 9 shows a site where diaphragm walls had to be constructed under partial disruption of traffic. Note that in this case the minimum space was determined by the requirements of assembling the reinforcing cages, which also dictated the maximum panel length. Coordination in the excavation and materials handling process gave a clean and basically unobstructed site. Fig. 10 shows a most frequently encountered problem in urban construction: the problem of maintaining existing utilities.



Permanent Structural Wall for Subway Construction **FIGURE 8**

Slurry Wall Construction Maintaining Traffic ▶

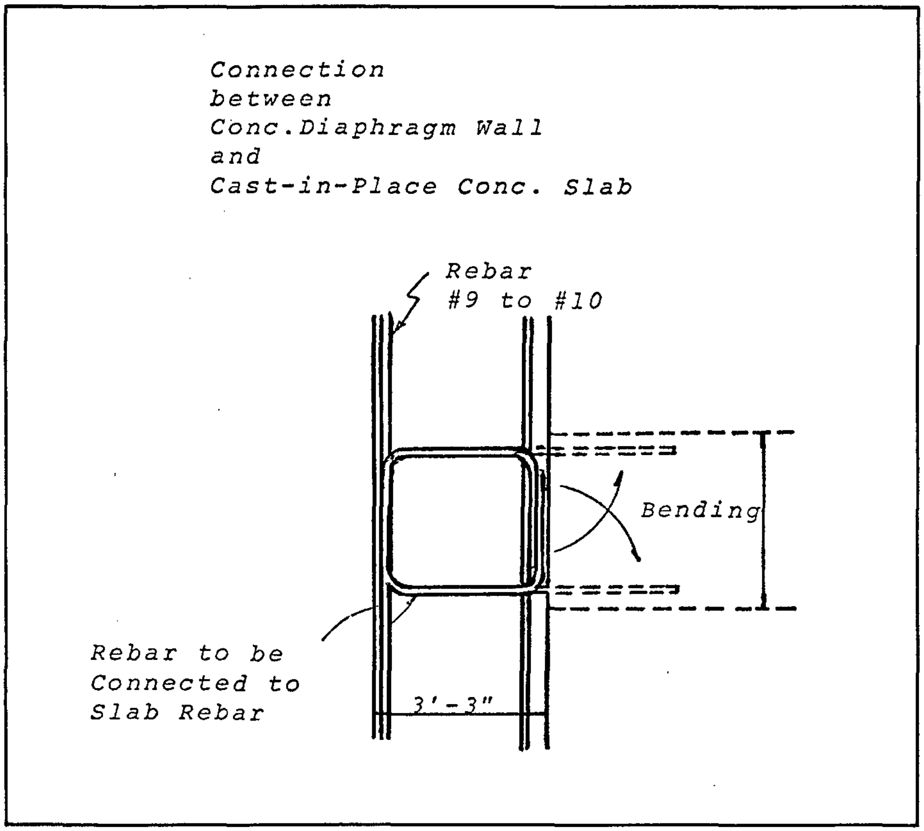




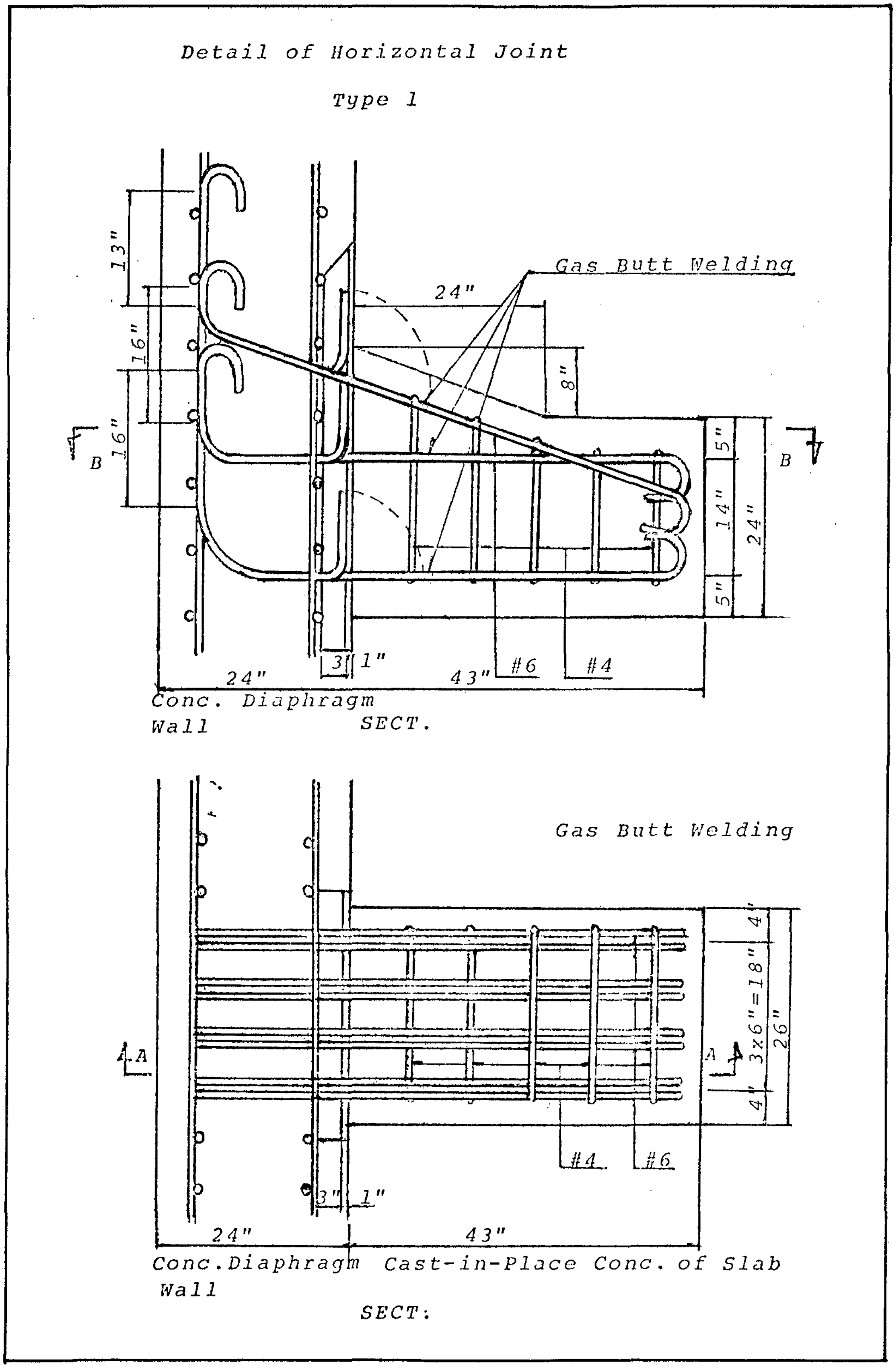
◀ Maintaining Congested Utilities

FIGURE 10

Structural Connections. For general information Fig. 11 shows methods for executing structural connections with adjoining elements. These connections are intended to provide shear and moment transfer and ensure rigid joints.

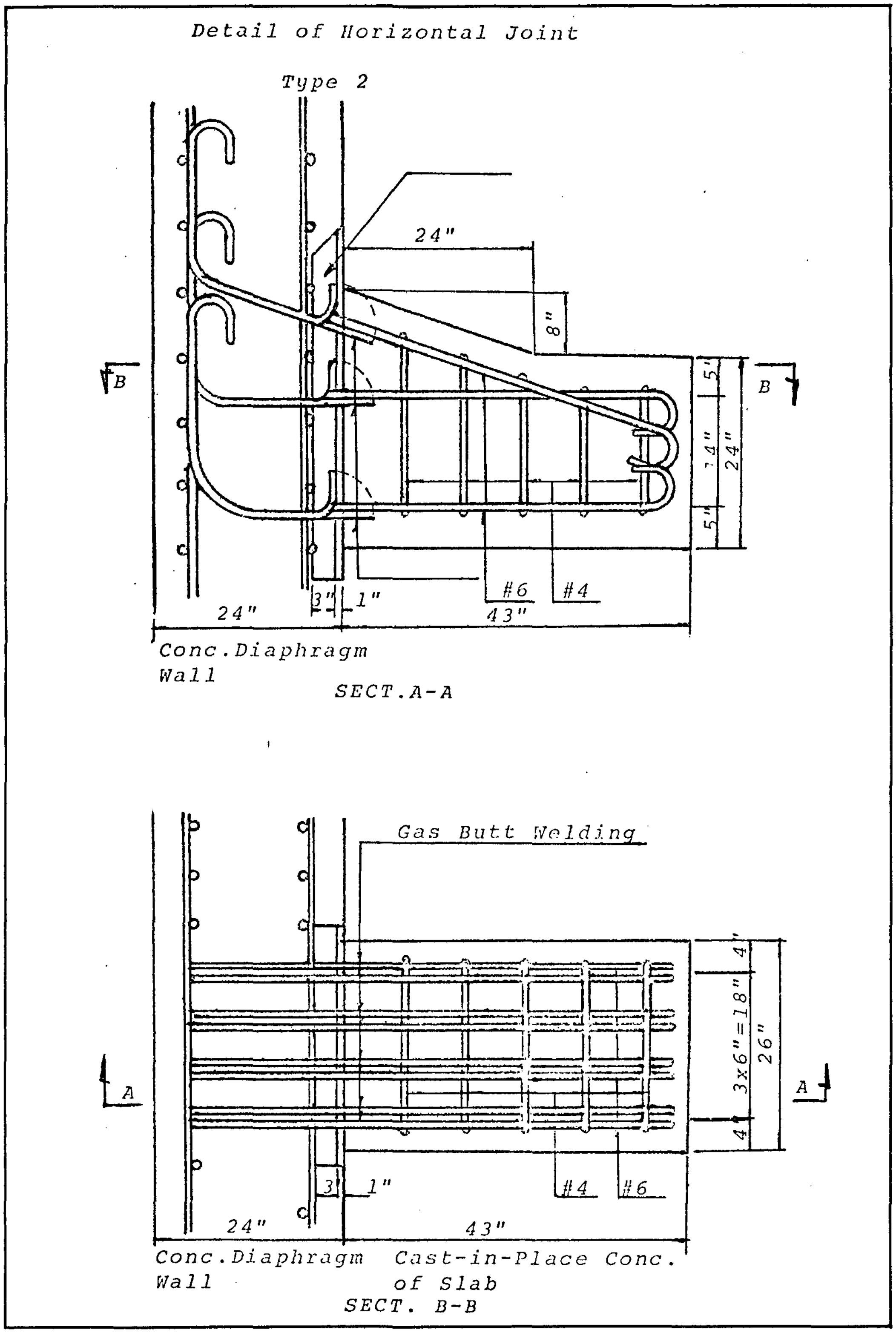


Variety of Horizontal Connection as Main Structure **FIGURE 11(a)**



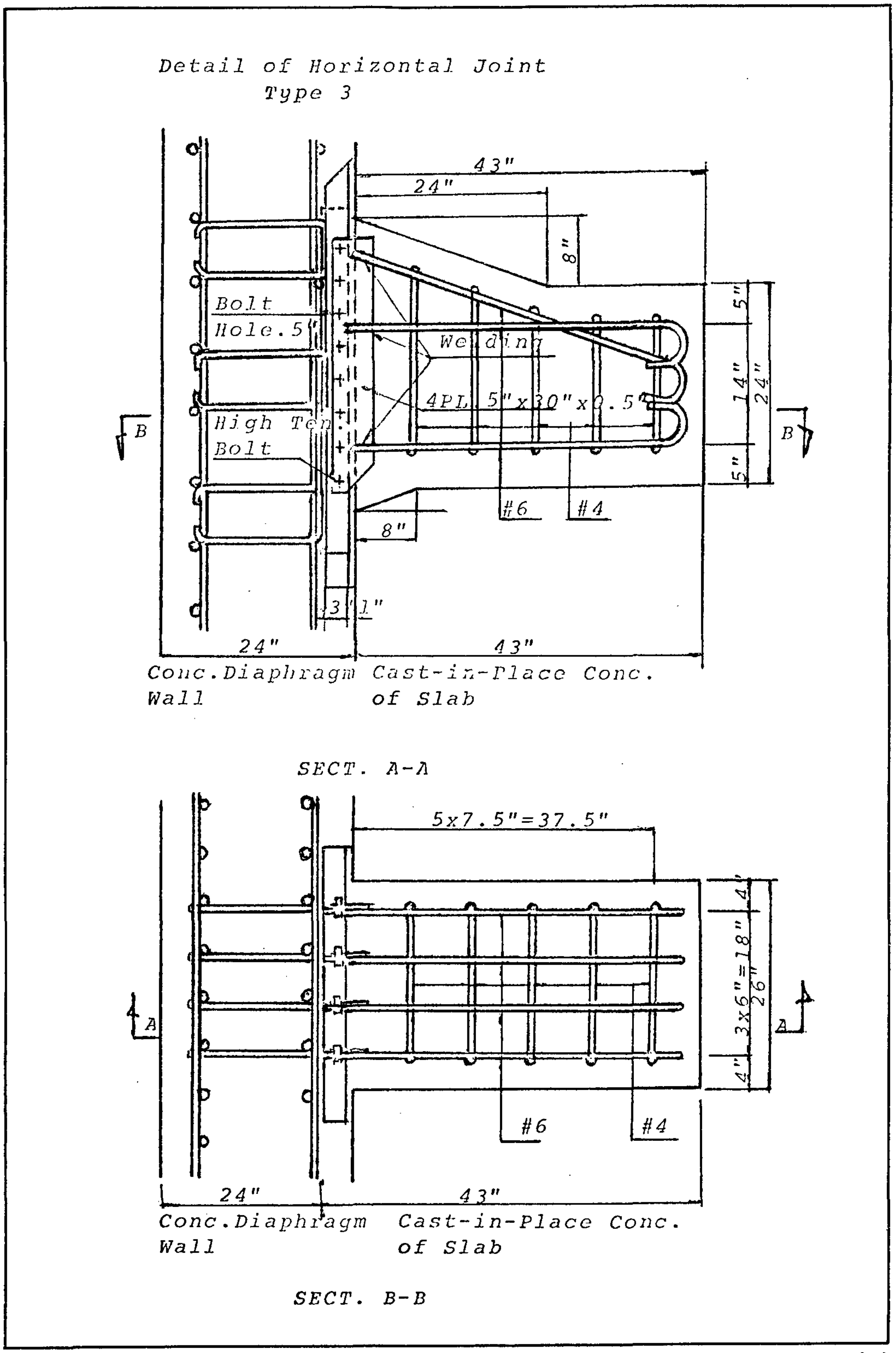
Detail of Horizontal Joint
Type 1
Gas Butt Welding
24"
13"
16"
16"
8"
5"
14"
24"
5"
B
B
3"
1"
24"
43"
#6
#4
Conc. Diaphragm Wall
SECT.
Gas Butt Welding
4"
3x6"=18"
26"
4"
A
A
#4
#6
3"
1"
24"
43"
Conc.Diaphragm Wall
Cast-in-Place Conc. of Slab
SECT.

FIGURE 11(b)



Detail of Horizontal Joint
Type 2
24"
8"
5"
14"
24"
5"
B
B
3"
1"
#6
#4
24"
43"
Conc.Diaphragm
Wall
SECT.A-A
Gas Butt Welding
4"
3x6"=18"
26"
4"
A
A
3"
1"
#4
#6
24"
43"
Conc.Diaphragm
Wall
Cast-in-Place Conc.
of Slab
SECT. B-B

FIGURE 11(c)



Detail of Horizontal Joint
Type 3
43"
24"
8"
5"
14"
24"
5"
Bolt
Hole.5"
Welding
4PL 5"x30"x0.5"
High Ten Bolt
B
B
#6
#4
8"
3"
1"
24"
43"
Conc.Diaphragm
Wall
Cast-in-Place Conc.
of Slab
SECT. A-A
5x7.5"=37.5"
4"
3x6"=18"
26"
4"
A
A
#6
#4
24"
43"
Conc.Diaphragm
Wall
Cast-in-Place Conc.
of Slab
SECT. B-B

FIGURE 11(d)

Detail of Horizontal Joint
Type 4

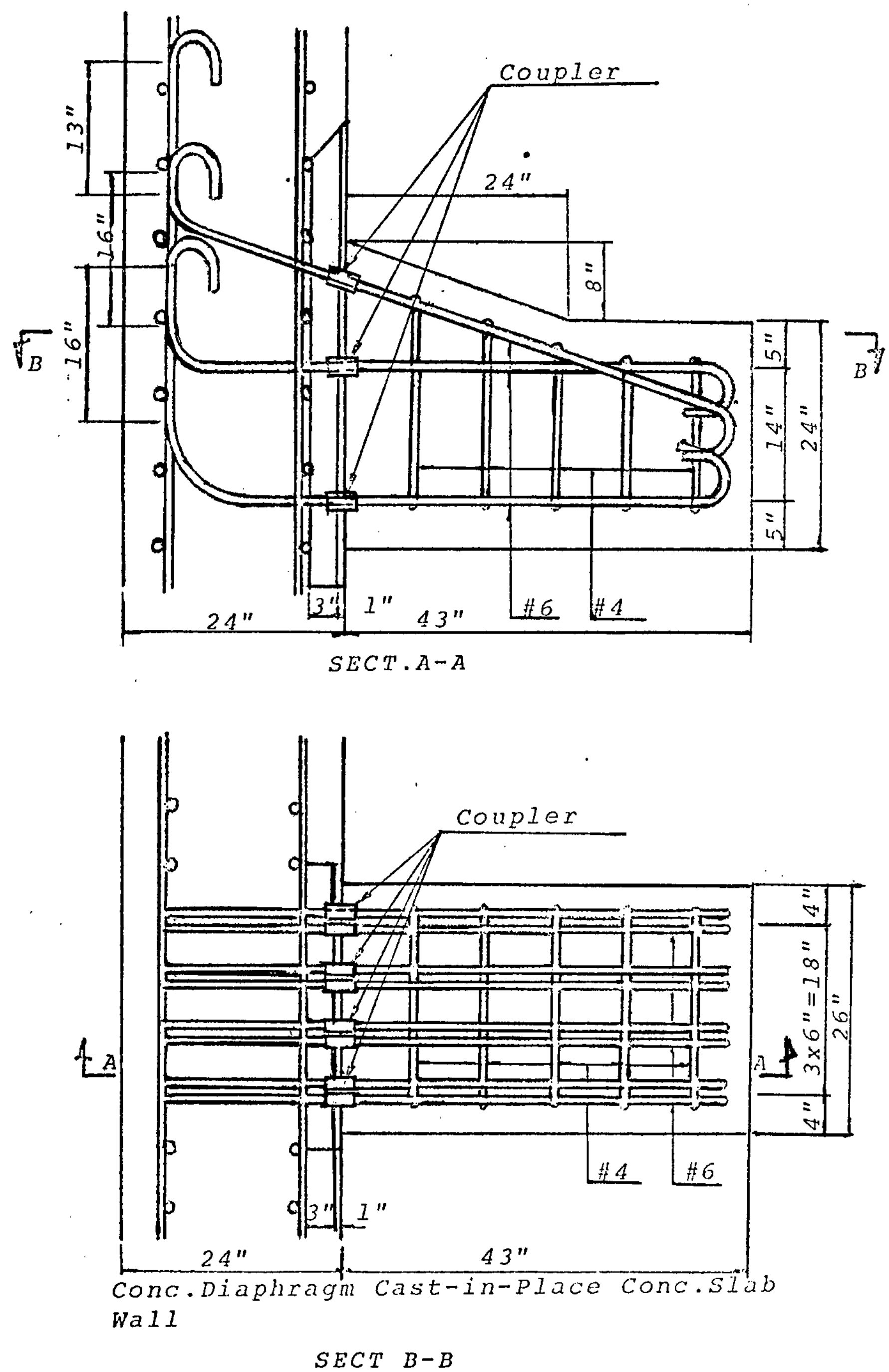


FIGURE 11 (e)

PANEL DISCUSSION

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Panelists at session on October 22, 1976. Left to right: Moderator, Harold E. Nelson, Chicago Urban Transportation District; James Birkmyer, Bechtel Corp.; Jean Francois Bougard, New Works Department, Paris; Thomas H. Hanna, University of Sheffield; David Jobling, London Transport; T. Kato, Sumitomo Construction; Robert S. Mayo, Mayo Associates; Jorj O. Osterberg, Northwestern University; Gilbert Tallard, Soletanche & Rodio, Inc.; and George Tamaro, I.C.O.S. Corp. of America.

PANEL DISCUSSION

WEDNESDAY, OCTOBER 22, 1975

Panel Moderator: Harold E. Nelson, Executive Director
Chicago Urban Transportation District

Panelists: Jorj O. Osterberg, Jean Francois Bougard, Gilbert Tallard, Robert S. Mayo, James Birkmyer, Thomas H. Hanna, David Jobling, George Tamaro, and Toshio Kato

Mr. Nelson began the panel discussion by asking the panelists to be specific and brief in their replies and comments, and by encouraging all those in the audience to respond and participate in the discussion. His first question was: In what area or areas of soft ground tunneling is research mostly needed? What additional research is necessary to improve the application of cut-and-cover and tunneling construction: excavation systems, materials handling, construction techniques, soil stabilization or others?

The question brought mixed response from the panel and the audience. Mr. Mayo, for example, commented that the next step is to improve the interaction between the tunnel equipment, and develop new lining systems. Regarding research, Mayo said, we should concentrate on tunneling because in his opinion tunneling is the answer to mass transit systems in cities with intensive development. The most essential areas of research should be the sealing of tunnel liners, and probing ahead of the excavation to find out what is in front of the tunnel face.

Professor Osterberg emphasized the matter of instrumentation; it is quite important, he said, to fully instrument deep excavations and use this information in the design process to obtain semi-empirical solutions. Mr. Kato, on the other hand, placed the emphasis on research for cut-and-cover; for example, how to use the side walls with the permanent structure and how to analyze the interaction of structural members built with different construction techniques; other important areas of cut-and-cover construction, Kato said, are the development of better solutions to remedy the traffic problem and the street congestion, and the interaction of construction techniques with the site conditions.

Mr. Birkmyer said that based on his contacts with tunnel contractors, especially in Detroit, Chicago and Milwaukee, some research should be continued to develop precast concrete lining segments to eliminate the secondary lining. Another area of research should concentrate on better geological forecasts for tunneling, and on some special details, for example the cutting teeth for the tunnel boring machines. Perhaps, a reevaluation of the present limitations in the use of compressed air might change the current restrictions and allow a more realistic use of the method.

Mr. Nelson commented at this point to say that what is also needed is a better mining method for soft ground tunnels. Mr. Nelson referred to the comment made by Mr. Robbins that better methods are available but for relatively long tunnels, and pointed out that in urban areas there are many short tunnels, of the order of 1000 ft.

Mr. Tallard said that as a contractor practicing various techniques in underground construction he felt that research should not go too far ahead. Much improvement may be possible if we organize the various systems available to bring the most optimum use and combinations therefrom. Mr. Tamaro, on the other hand, focusing on cut-and-cover suggested that a comprehensive system performance study is lacking, and all the associated aspects and problems of cut-and-cover should be attacked on a systematic basis.

At this point, Mr. Nelson raised the following question before the panelists: Should research be done by private resources, by public resources or by both? One panelist replied that the only way to institute private research is to have public authorities refuse new concepts until they are fully explained, otherwise the private industry will not accept the cost of research. Mr. Nelson asked, then, this question: Were some of the techniques we have seen here developed by the government of the countries represented, or by private resources? The panelists replied that it was done by private resources, but in response to economic necessity. The moderator, then, asked how many thought that research should be done by both groups, and the majority of individuals in the audience replied positively.

Mr. Xanthakos commented that in many instances research conducted by the private industry was not disseminated, but remained proprietary.

Mr. Bougard explained that in France the problem of organizing and administering a research program presents a difficult task. Stimulated by their own experience, contractors, owners and engineers are tempted to undertake research, beginning with what is most urgent to some. Thus, practically everyone participates in research; as for funds, the financial resources are always found.

Mr. Nelson continued by saying that one of the purposes of this seminar was to identify the needs for research, and he was quite pleased to have Messrs. Russell McFarland and Gil Butler of the U.S. Department of Transportation who are concerned about advancing the state of the art in attendance. The moderator reflected on two comments made by the panel: (a) the emphasis on instrumentation; and (b) some strong comments on construction techniques. This brought the question of basic versus applied research, and the moderator asked the panel and the audience where the emphasis should be placed. The response was that it probably should be on both, and that both basic and applied research should be developed on a parallel basis.

At this point Mr. Jobling took the opportunity to express his views on the subject. Research, he said, is primarily the development of techniques, and in this respect public authorities have a great deal to gain. As far as the London Transport is concerned, for instance, the shield machine shown in the movie last night required an investment of almost half a million pounds on experimental work. For the short term there is no justification for this expenditure, but for the long term, ten to twenty years, this money will be recovered many times if the machine proves to be successful. The same should apply to other organizations that have done experimental work on tunneling; the advantages are not short terms gains, but long term benefits.

Mr. Nelson replied that this was a very good comment, and agreed that in talking about applied research he was talking about development.

Mr. Nelson continued the discussion with specific questions taken from the audience. The first question was: How can we have the U.S. tunneling industry to design, produce and adapt precast tunnel lining systems? Mr. Bougard replied that in Paris contractors work with Robbins machines; for a medium size project precast concrete lining segments are economical compared to double primary and secondary linings. Some difficulties have been experienced with the joints, and complimentary contracts are necessary with precast liners. Mr. Nelson asked, then, how we might convince the construction industry to use more precast lining systems than presently in use. The panelists offered the following suggestions: (a) make it mandatory to have a test section, say 10% of the total length, to try the effectiveness of precast linings; (b) demonstrate to the construction industry the cost benefits, if any, resulting from the use of these systems; and (c) convince owners and engineers to specify them in the design.

The last comment raised the question whether reluctance on the part of engineers to accept something new poses a real problem, and Mr. McFarland took the opportunity to elaborate on this matter and explain that very often this is indeed a problem. A further problem is the contractor's reluctance that the use of a new system may decrease profits and increase costs; this, however, has been solved by using two contractors, one for supplying the concrete liners and the other for installing them. The use of segmented concrete liners may result in an appreciable saving, as Mr. Birkmyer pointed out.

At this point, Mr. E.A. Tillman, Project Director for the Baltimore Subway, expressed the opinion that the main reason why contractors and engineers are reluctant to try something new centers around the element of risk, so the owner who wants to promote new systems must assume part of the risk. This is probably the best way to achieve success. The interesting thing about the distribution of risk is that everybody agrees on the distribution as long as it does not affect the client-owner. In case of elected government officials, they are not in a position to accept risks.

Mr. Nelson responded to these comments by saying that if he could convince the members of the Board and the Service Consultants to recommend segmented concrete liners, the CUTD would be using them. Mr. Tillman, then, continued by saying that in Baltimore the possibility of using precast concrete liners was considered, and the question was raised about the actual watertightness of the system in the field. An industry-wide survey among contractors showed little interest on their part, and instead they indicated a preference towards steel liners. Thus, the contractors felt that if they were to adapt this system, the client-owner should supply it, and they would not assume responsibility for leakage.

Mr. Nelson asked Mr. McFarland to comment at this point because of his direct involvement in this matter. Mr. McFarland said that to his knowledge there have been as many contractors favoring precast liners as there have been against them, hence this is not the real issue. The question is whether precast sections can provide a structure that would satisfy the owner as being structurally competent and watertight. An objection to concrete liners in Baltimore stemmed from lack of previous experience with this system in similar water bearing ground. The survey conducted by Bechtel, Inc., revealed 22 projects either completed or under construction where precast sections were used without secondary liners, in many cases under many meters of ground water, and where the seal was quite effective. The real problem, therefore, is the initial risk involved, including the difficulties of changing construction practices.

Mr. Tillman, then, spoke more specifically about the Baltimore project and indicated that a test section has been recommended. This should not mean that the Senior Consultants are against the system, but under the present state of development it might be wise not to adapt it for general use without a sufficiently long test section from which direct experience could be gained.

Mr. Nelson concluded this matter by noting that in the near future similar decisions will have to be based on available resources for production. One of the problems CUTD faces is the availability of materials, unless CUTD makes them. CUTD also has steel problems, and if one attempts to balance energy and production in the concrete and steel industry with other techniques and labor, he will find out that these have not been explored to the extent that they should. Probably, the only ones who have really explored these matters are contractors, and this usually is reflected in their bids or reluctance to accept alternatives.

The moderator moved on to other questions and asked what types of seals are available to keep the slurry from flooding the excavated area in slurry-faced tunnels. A reply from Mr. Mayo is that he was aware of a seal that failed, but because of a weak part of the machine. For the Warrenton project which will have a slurry-faced machine, the seal has been completely redesigned.

A person from the audience described a seal used in Japan for slurry-faced machines which is slightly different from those used locally, but which gave good results. This is an inflated type seal with a double seal mechanism, and according to Japanese engineers it performs satisfactorily.

A question for Mr. Bougard was related to the problems caused by utilities in diaphragm wall construction for the Paris metro. Mr. Bougard explained that invariably there is interference and problems, but are unsolved. The usual procedure is to make a cost comparison between cut-and-cover plus utility relocation versus other techniques. In diaphragm wall construction, most of the time utilities must be diverted, and this is possible by moving cables, conduits and sewers on both sides of the street.

Replying to the question of how to space grouting wells to ensure uniform penetration of the zone being grouted, Mr. Tallard said that this is very much dependent upon the grain size and distribution of the soil. For example, for chemical grouting the spacing of grout holes is from 4 to 6 ft; for cement grouting this space can be from 6 to 15 ft. For a complete experience one must consider the type of grout, and possible combinations for the most economical application. For mixed grouts, which are quite common, the grout holes usually are spaced at 8-ft intervals.

The next question was for Mr. Birkmyer, about the special problems in the installation of precast concrete sections that contractors should anticipate. Mr. Birkmyer replied that from his experience and from discussion with tunnel contractors in the Washington, D.C. area, no problems are anticipated other than changing the method of handling the segments with the erector arm, and perhaps developing a different caulking technique. The latter will probably be solved by experience. According to the record, breakage is very small provided reasonable care is taken so that the segments are not thrown.

Responding to the question of how much extra reinforcement is needed in reinforcing cages for diaphragm walls for handling, Mr. Tamaro said that this extra steel is a very minor item, and represents a negligible cost of the total cost of construction. Usually, a few extra bars are attached to the cage, mainly to keep the main bars together, and the cage is better handled if it is made flexible. What is more important is that very often the contractor's engineer must check the adequacy of the reinforcement for the temporary excavation and bracing stages to find that more reinforcement is sometimes needed for the temporary condition than was provided.

A second question from the audience was about the use of diaphragm walls for deep shaft construction, say 100 ft, as compared to grouting, and whether diaphragm walls should be considered for the construction of the shafts of the Chicago Underflow System. Mr. Tallard, besides economy, focused on the safety aspects of diaphragm wall construction, and mentioned several occasions where he had used the technique in construction shafts for tunnels.

Mr. Xanthakos responded to the same question, and indicated that economy is in this case a relative concept, but judging from the wide use of diaphragm walls in deep shafts we must conclude that contractors find this solution more economical and advantageous. The next question is whether or not one can design and build a shaft without reinforcement. Taking, for example, a large diameter structure, it can be readily converted in a polygonal configuration without departing a great deal from the circular section, so that the assumption of compressive (hoop) stresses is still valid. However, one must consider possible construction imperfections, for example tendency of a panel to move inwards, and this may dictate the need for some reinforcement although theoretically this is not necessary. Medium and small diameter shafts have been built to depths of 80 and 90 ft without reinforcement, and withstood the stresses satisfactorily although the assumption of hoop stresses only was not valid. Among construction imperfections, unsymmetrical loading, and wall base restraint, the latter is more critical to the actual distributing of stresses. If one considers all these factors together with the minimum practicable wall thickness, it may be concluded that relatively large diameter shafts sometimes are more economical than small diameter shafts for the intended function.

Mr. Mayo replied to a question about soft ground tunneling versus cut-and-cover construction. In soft, but firm, ground the important aspect is the depth of excavation. If the cover above the box section is, for example, 20 to 25 ft without too many utilities, cut-and-cover may be cheaper. For deeper construction tunneling should be cheaper, unless the ground conditions are very poor and the control of ground water requires compressed air. In this case, Mayo said, the economics very probably will favor a slurry wall type of construction.

Commenting on the cost of service disruption, Mr. Jobling discussed the Heathrow airport subway extension, part of which was done in cut-and-cover. An analysis was made of the cost of cut-and-cover versus the cost of tunneling, which (tunneling) would have about 60 ft of cover. This indicated that cut-and-cover, 25% of the cost of which was for services and utilities, would cost less than tunneling (the cost of tunneling was in fact 10 to 20% higher). When the actual construction costs became available, they showed that tunneling was about 15% more expensive than cut-and-cover. In bad ground conditions, Mr. Jobling continued, cut-and-cover would be more economical even if the surface diversion cost was as high as the cost of cut-and-cover work itself. Besides these data, Mr. Jobling explained that a point that should not be overlooked is that the right place for transit tunnels is as close to the surface as possible. Deep transit tunnels, say 100 ft, are not desirable because of the escalator requirements, operating defects and troubles, and inconvenience to passengers. Adding to these comments, a gentleman from the audience indicated that for open-cut construction of the Paris metro, the cost of utility and service diversion was in some cases 40% of the total cost of construction. A panelist continued this subject saying that New York City, because of the recent experience with the cost of tunneling particularly for mixed face excavations, is reevaluating the entire tunneling program, and plans to return to cut-and-cover techniques placing the subway as close to the surface as possible.

At this point the question was again raised from the audience if a diaphragm wall, for instance a shaft structure, should be extended through water-bearing rock to make the excavation watertight. Mr. Tallard responded from the panel saying that a slurry wall placed through rock for water control only is not a good solution. Instead, grouting through preinstalled pipe casings is cheaper and effective, and can provide dry conditions.

The next question was about the use of grouting in tunneling to improve the performance of the system, and about experience or ongoing research on ground strengthening around the tunnel to reduce the pressure on the concrete arch and produce savings in the cost of tunneling. Mr. Tallard replied that it is feasible to strengthen the soil around the borehole in conjunction with the use of a lighter lining system, or perhaps shotcrete, but some doubts have been expressed about the permanence of grouts, especially chemical grouts. Mr. Tallard, said that his answer to these doubts is that chemical grouting is a permanent treatment, since the improvement of the strength characteristics of the soil is permanent.

A question was addressed to Mr. Jobling, if the cost difference he presented for cut-and-cover and tunneling took into consideration the cost of business disruption, and factors such as the effect on the public. Mr. Jobling replied that it is very difficult to quantify the cost of business disruption, and one can only look at the situation at hand. One consideration that sometimes leads to the choice of tunneling is the fact that cut-and-cover is not always acceptable by the public. On the other hand, one advantage of tunneling is more flexibility; for example it is easier to accommodate several lines in one line, provide better interchanges between stations, and have three or four lines at one station. In relatively deep tunnels, this can be given three-dimensional solutions. Nonetheless, Mr. Jobling said that the choice between tunneling and cut-and-cover is a very complex matter, and there is no simple answer; yet, it is fair to say that one should always start looking at cut-and-cover, and if it is impossible to use it then it is time to consider tunneling.

A question was asked if anyone on the Panel had experienced a cut-and-cover excavation using a diaphragm wall on one side and a more flexible ground support on the other, with bracing between them. Mr. Jobling responded to the question and mentioned a project that included bored piles along one side and sheet pile walls along the other, and this presented no particular problem except that some extra bracing had to be used along the sheet piling.

Mr. Tamaro commented that it is difficult to conceive of situations where a mixed group support system would be suitable. If, for example, a diaphragm wall is used on one side because of an adjacent tall building, and a sheet pile wall is installed on the other side, the engineer should be very careful when analyzing the performance of this construction and the interaction between the different support systems; the result can be a differential pressure, for instance more surcharge against the diaphragm wall and drained conditions for the sheet pile wall, that may cause passive resistance against the sheet pile wall. Mr. Tamaro mentioned a case of differential pressure on a subway job in Washington, D.C. These problems, he said, would require a great deal of engineering competence and judgment. The same panelist also explained how a rigid diaphragm wall is built around a utility line; in this case it is advantageous to provide some clearance by making a larger hole in the wall and fill it with a compressible material which will act as cushion. A good practice is also to keep the slurry level below the utility to allow some visual inspection and better protection during the excavation of the trench.

Mr. Nelson at this point, commented that the time had come to adjourn the panel session. He expressed his thanks to the panelists and other speakers who participated in this session, and particularly to Mr. William Barnes who is CUTD Manager of Professional Services, Mr. Petros P. Xanthakos for assisting in developing the original ideas and the final program, and Mr. George Guscott and his fine team. Mr. Nelson expressed his deep appreciation to Messrs. Russell McFarland, Gil Butler and others from the U.S. Department of Transportation for helping CUTD sponsor this seminar.

The important conclusion, Mr. Nelson said, is that the problems we had before are still with us and probably will always be with us. Somehow, the great movements in construction have taken place on the contractor's side, and engineers sometimes try to catch up with them. What is important now is to find a way to make useful the enormous experience and expertise that has been accumulated, and in this put on a piece of paper and assume the risk associated with construction.

The added dimension, Mr. Nelson emphasized, is the enormous social and economic impact of projects like the one we have been discussing here, and whether we use steel or precast concrete liners is among the questions to be answered by our engineers and contractors. Besides, there are other forces at work, especially in large urban centers, that may make these decisions for us, and the best thing for us to do is to recognize these forces and blend them with our technology as best as we can.

Seminars like this may help, and although they do not give us the answer they bring us together so that we can exchange ideas. For example, the idea of joint development and the idea of social interference through construction are concepts we have just begun to explore, and the decision of cut-and-cover versus tunneling might ultimately have to be made on that basis.

We are not going, Mr. Nelson said, to tear up the streets of Chicago and have them in this condition for five years, and this probably will never be done in any large city in this country. The experience in Washington has been disastrous for certain segments of the industry and the community, and this will not happen again. Although some may not believe that this problem can be avoided, we will try our best to avoid it. Mr. Nelson finished his closing remarks by expressing hope that he would see all the participants again when service is inaugurated in Chicago.

BIOGRAPHICAL SKETCHES OF SEMINAR SPEAKERS

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BIOGRAPHICAL SKETCHES

OF

SEMINAR SPEAKERS

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George J. Pastor. Mr. Pastor has been Associate Administrator for Research and Development of the Urban Mass Transportation Administration since July, 1974. Prior to his appointment with UMTA, he oversaw ground systems research programs at the Transportation Systems Center, Cambridge, Massachusetts. Before coming to government service, Mr. Pastor was an executive with Ford Motor Company's Philco Ford subsidiary and was involved in the many engineering and electronics design activities of that company. Born in Czechoslovakia, Mr. Pastor holds a BSEE degree from Columbia University and an MSEE degree from the University of Southern California.

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A. A. Mathews. Mr. Mathews is President of the consulting engineering firm of A. A. Mathews, Inc. In the over 22 years in this capacity, he has consulted on many difficult construction problems including a significant number of major tunneling projects. He has given talks and authored several articles on tunneling problems and contracting practices, and his work with the U.S. National Committee on Tunneling Technology on better contracting for underground construction has aroused much favorable interest in the industry. He is a member of many professional societies and a registered Professional Engineer in seven states. He holds a degree in Mining Engineering from the Michigan Technological University. Prior to the formation of his company, he had extensive experience as a field and resident engineer for tunnels, chief engineer for contractors on dams and tunnels, and in charge of quarries and gravel plants.

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Robert S. Mayo. Mr. Mayo has been a consultant over many years to government and local authorities, contractors, and engineers engaged in design on subway and tunnel construction. He is a well-known lecturer and the author of many technical papers and handbooks on tunnel construction and tunneling and shield-driven subways, probably the best known of which is the standard text on the subject "Practical Tunnel Driving", originally published by McGraw-Hill in 1941. His practical and consulting experience in major tunneling work goes back over 50 years. He is a member of numerous honorary and professional societies, a registered Professional Engineer from Illinois Tech and has done graduate work at the Michigan College of Mines.

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