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A Study of the State of the Art

February 1973
Final Report

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16. Abstract

This study of cut-and-cover tunneling techniques in urban areas considers environmental quality, geotechnical investigation and analysis, ground support, ground water control, permanent structure, restoration, cost considerations and major problems. Both United States and foreign techniques are reviewed.

This volume summarizes the study findings. In addition to describing alternate methods of construction, it identifies key problems, needed construction improvements, and future research goals.

The other volume of this report is FHWA-RD-73-41, entitled "Appendix".

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PREFACE

This report is submitted in full compliance with Contract No. DOT-F-H-11-7803 dated June 30, 1971 between the Office of Research of the Federal Highway Administration, Department of Transportation and Sverdrup & Parcel and Associates, Inc., Engineers-Architects-Planners. It reviews the state-of-the-art in cut-and-cover tunneling construction and studies ways of improving operational technique.

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I INTRODUCTION

The Federal Highway Administration commissioned Sverdrup & Parcel to make a study on cut-and-cover tunneling techniques specifically related to urban areas. The object was to review the methods used in both the United States and abroad in an effort to define the factors that cause interference with surface traffic and disruption of business and communities, and to recommend methods of minimizing these factors. The report also points out the key problems that limit significant improvement and recommends areas for further research and further utilization of demonstration projects.

Cut-and-cover tunneling is a process of installing a 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 movement of traffic if the construction is in a street-area. The term cut-and-cover refers to what is done, and not to how it is accomplished.

The general method has been the most economical way to build underground tunnels, and, traditionally, the structure has been built inside a ground-wall support system of soldier piles and timber lagging. Utilities are generally relocated outside the excavation or are supported within the excavation. When the structure is complete, backfilling is done, utilities are repositioned, and surface restoration completes the work. Each of these steps results in disruption to the normal activity and flow of traffic at street level.

The below-ground structures which are normally suited to cut-and-cover construction are a highway of from two to four lanes in width (35- to 65-ft outside width of structure), rapid-transit structures (generally about 35 ft wide) and below-street-level parking structures. Construction of highway tunnels has in the past been limited in length, although today in urban areas longer highway tunnels are being considered. Construction by cut-and-cover is generally nearer the surface level (under 50 ft to bottom of excavation) than shield-driven tunnel construction.

The demand for tunnel construction over the next ten-year period has been discussed at length in the report of the Organization for Economic Cooperation and Development conference in Washington, D.C. in 1970. It is predicted that over 12,000 miles of tunnels not related to mining will be constructed in the decade 1970-1980. While much of this demand may be met by the use of shield-driven tunnels, economic considerations dictate that cut-and-cover techniques of an improved form be used to a large degree. While economics may rule out shield-driven tunnels in many instances, the public today may not be willing to accept the inconveniences imposed by conventional cut-and-cover techniques.

Many authorities believe that cut-and-cover construction will largely be supplemented in the next ten years by improvements in shield-driven tunnel technology. However, even if true, there will still be many areas that must be done by some form of cut-and-cover. The beginning and end of a tunnel where construction approaches the surface will nearly always be done by cut-and-cover. The same applies to intersections, connections with shallow-depth existing structures, station construction for rapid transit, and constructing underground garages over tunnels.

We believe the demand for an improved cut-and-cover tunneling technology will continue to increase over the next decade.

A. APPROACH

The approach to the study as defined in the contract "Statement of Work" was to review the state-of-the-art, summarize the best practices, list the key problems, recommend an improved construction method, and make recommendations for further studies.

Our approach to a review of the state-of-the-art consisted of reviewing the available literature from technical publications of the last ten years or so, although older material was reviewed for background information, in some cases. Our interviews showed that important practical experiences have not yet been written about in the professional journals. This especially includes negative data where construction techniques have not been successful. A bibliography of all the information reviewed is included as Appendix A, along with summaries of significant articles.

A questionnaire was also sent to Contractors and Consulting Engineers to determine their experience with cut-and-cover tunneling projects. The responses are summarized and analyzed in Appendix B.

Interviews were conducted with personnel from public agencies, contracting firms, consulting engineers, and educational institutions who have expertise in the tunneling field. Major projects under construction were visited while the study was in progress.

B. ACKNOWLEDGEMENTS

The cooperation and assistance of the following people in the tunneling industry both in the United States and abroad is gratefully acknowledged. Their contribution of time and information helped immeasurably in preparing this report.

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C. REPORT ORGANIZATION

Chapter II discusses the environmental quality considerations, Chapter III covers geotechnical investigations and analyses, Chapter IV covers groundwater control, while Chapter V discusses ground wall support. Chapters VI and VII cover excavation and structural control, and Chapters VIII and IX cover permanent structure and restoration. Cost considerations are discussed in Chapter X. Chapter XI presents our summary of major problems and recommended improvements in cut-and-cover tunneling techniques. The bibliography concludes Volume I. Volume II is the appendix to the report, and includes the summaries of important articles (organized according to appearance in the bibliography), the questionnaire evaluation, Canadian and New York noise-control laws, and the text of the Canadian Expropriations Act as amended in 1971.

II ENVIRONMENTAL QUALITY CONSIDERATIONS

Environmental quality considerations are discussed below under business disruption, regulations, traffic control, dust control, noise control, vibration control, safety, and ventilation. Some of these subjects, like dust and noise control, are particularly difficult to evaluate since the on-the-spot assessments of the degree of infringement are often only subjective judgments which may vary significantly from person to person. These are areas, however, which may vitally affect the contractor's public image and acceptance.

A. BUSINESS DISRUPTION

Most cut-and-cover construction projects in an urban area will affect the merchants and residents near the work as well as the motorists and pedestrians passing through the area.

While much can be done to alleviate the inconveniences, an effective public relations program directed and guided by the Owner to keep all interested parties informed is of prime importance. Local business groups and Chamber of Commerce organizations can do an effective public relations job and should be brought into the planning process in the early stages. Different businesses have entirely different requirements during construction. For example, a business primarily dependent upon telephone or mail for customers (i.e., a real estate agency) will be affected in an entirely different way from a business which is dependent upon drive-in or walk-in customers (i.e., an automobile agency), and the requirements of each must be considered.

Merchants must also be brought into the planning process so that street-closing times can be properly scheduled to allow deliveries and other necessary services. Construction operations and utility relocations can also be scheduled to minimize interference.

Buying off-street right-of-way through purchasing the buildings that are scheduled for redevelopment and constructing the tunnel clear of the street is a possible aid in redevelopment planning. This generally avoids most of the existing utilities and allows traffic to flow essentially uninterrupted. When construction is complete, the land over the tunnel may be sold for private or public development. Toronto has done this very successfully, in some cases realizing almost the cost of the original property purchased.

Laws or policies obligating a Construction Authority to pay for loss of business are becoming more common. Toronto has such a law, although its ramifications are not completely known at this time.

B. REGULATIONS

Federal regulations now require the Environmental Protection Agency to review any construction funded with Federal money, and require the preparation of environmental-impact statements. At the present time each Federal agency has specific regulations governing the preparation of environmental-impact statements; however, the regulations are generally directed at the permanent effects on the environment. For example the following basic requirements are taken from the Federal Highway Administration regulations PPM 90-1, paragraph 2A:

"a. Section 4332(2)(C), Title 42, United States Code (popularly known as Section 102(2)(C) of the National Environmental Policy Act of 1969, P.L. 91-190) states in part that all agencies of the Federal Government shall:

'include in every recommendation or report on proposals for legislation and other major Federal actions significantly affecting the quality of the human environment, a detailed statement by the responsible officials on---'

"(i) the environmental impact of the proposed action,

"(ii) any adverse environmental effects which cannot be avoided should the proposal be implemented,

"(iii) alternatives to the proposed action,

"(iv) the relationship between local short-term uses of man's environment and the maintenance and enhancement of long-term productivity, and

"(v) any irreversible and irretrievable commitments of resources which would be involved in the proposed action should it be implemented.

"Prior to making any detailed statement, the responsible Federal official shall consult with and obtain the comments of any Federal agency which has jurisdiction by law or special expertise with respect to any environmental impact involved. Copies of such statement and the comments and views of the appropriate Federal, State, and local agencies which are authorized to develop and enforce environmental standards, shall accompany the proposal through the existing agency review processes."

C. TRAFFIC CONTROL

A traffic-control plan showing street closings, changes in lane widths, detours, direction of traffic flow and signalization agreed to by the interested parties should be incorporated into the contract documents. This procedure still does not preclude the contractor from suggesting alternates after construction has begun.

D. DUST CONTROL

Dust is one of the most frequent sources of complaints during cut-and-cover construction. The Federal Occupational Safety and Health Act now regulates the amount of dust within working areas. Water, calcium chloride, frequent sweeping of streets adjacent to construction, frequent cleaning of vehicles (see Figure 1), and proper covers on the haul trucks are several ways to control surface dust.

Other control systems that seem to have considerable promise involve excavating in what is essentially a closed system.

E. NOISE CONTROL

Noise is also one of the most frequent sources of complaints. Construction near a hospital may require special considerations in addition to the normal requirements (see Figure 2). Subpart D of the Occupational Safety and Health Act (OSHA) gives control requirements for occupational-noise control based on sound levels (decibels, or, db) and the duration of exposure in hours per day. The maximum exposure is 90 db, based on an eight-hour day.

Many cities are considering enacting noise-control legislation, and New York City is one that already has such laws[1]. Such legislation may not only limit the noise level but may restrict the working hours except for special operations. However, equipment is available today with the following ratings:

1. Air compressors -- 85 db at 3 ft, or roughly the sound of ordinary street traffic
2. Small air compressors -- 75 db at 23 ft

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1. See Appendix for the New York City and Toronto laws.

FIGURE 1.   WASHINGTON, D.C. METRO, Truck Wheel-Washing Statio

FIGURE 2. STOCKHOLM, SWEDEN, NOISE CONTROL MEASURES NEAR A HOSPITAL. The construction was near a hospital, and rubber caps were put on the sheet-pile hammers. The drill hammers were enclosed and screens (at left in the picture) were placed between the machines and the hospital to reduce the noise.

3. Vibrating pile-drivers -- 92 db at a few feet, as compared with 140-160 db for an impact-driver

Noise levels are difficult to quantify in meaningful ways, and measurements in one number system often cannot be compared with others. Figure 3 diagrams the relationship between sound-pressure levels in decibels (db) and in microbars, and between loudness level in phons and in sones. Table I shows the relation between sound levels in db and relative loudness in phons.

Interpreting the numbers can also be confusing since the db scale is logarithmic, and the indicated noise level doubles with approximately each 7-db increase. Comparing phons with sones illustrates this problem. A sone is defined as a 1,000 cps pure tone at 40 db above the hearing threshold. The sound-pressure level generated by such a tone is defined as the loudness level in phons. A 10-phon increase in loudness level (or 10 db at 1,000 cps) doubles the loudness of a sound in sones.

The following general rules-of-thumb may be used as noise-control guidelines. Table II shows the recommended design criteria.

1. A 2 db change is barely perceptible.

2. A 5 db change is perceptible but in most cases will not solve noise problems.

3. A 10 db change is noticeable and will reduce or eliminate most noise problems.

4. A 20 db change is a dramatic change and will solve most serious noise problems.

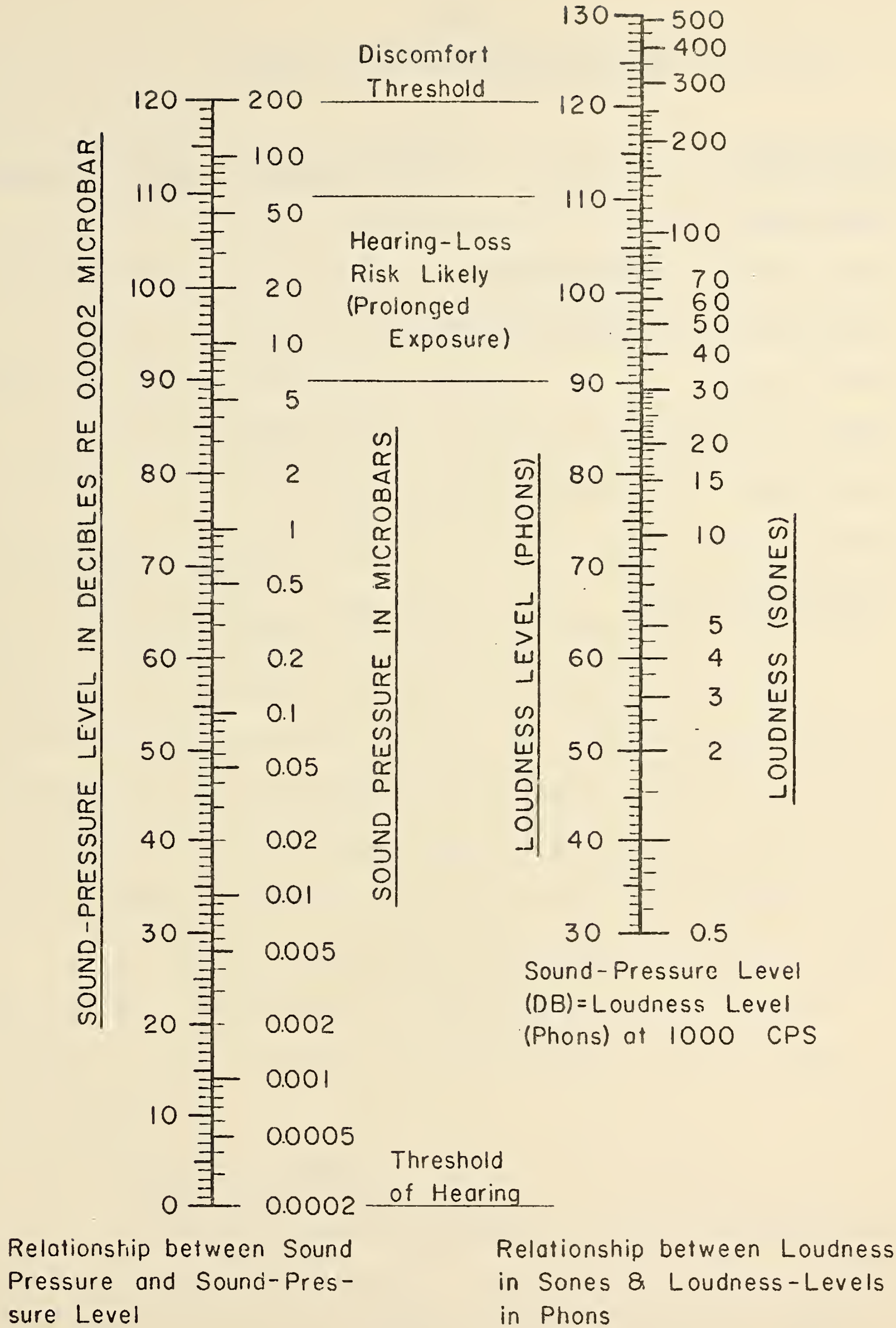

FIGURE 3. SOUND-PRESSURE LEVELS AND LOUDNESS.
(By permission of the Toronto Transit Commission)

TABLE I

NOISE LEVELS OF TYPICAL SOUNDS

NOISE SOURCE	SOUND LEVEL (db)	RELATIVE LOUDNESS
Whispering at 4 feet	66	7
Inside theatres - movie playing	68	11
Inside a commercial office building	70	11
Inside a factory	78	20
Inside a streetcar	87	33
Inside subway train (box structure)	92	48
Rush-hour street noises	96	65
Vibratory pile-driver	105	154

Sound-pressure levels may also be related to speech interference levels as follows:

1. 45 db - permits continuous conversation at about 10 feet.

2. 55 db - permits continuous conversation in a normal voice at about 3 feet and in a very loud voice at 12 feet.

3. 65 db - permits intermittent conversation in a raised voice at 2 feet and shouting at 8 feet.

4. 75 db - permits minimal communication by shouting at 2 or 3 feet.

F. VIBRATION CONTROL

Dynamic impact or vibrations induced by pile driving may disrupt human environment and may cause settlement and damage to adjacent structures. These structures are often underpinned when the foundation soils are loose granular materials, to prevent settlement and eliminate structural damage caused by pile-driving. Controls may still be applied for lesser disturbance to the human element, however.

Prohibiting or restricting the driving of piles in urban areas is fairly common now, and pre-boring the holes is usually necessary. However, driving the last few feet to seat the piles may be permitted.

1. <u>Vibration Estimation</u>

Some general rules-of-thumb may be used in considering the vibrations caused by pile driving. These rules are based on energy ratios (ER) which can be determined from the formula ER = $4\pi^2V^2$,

TABLE II

RECOMMENDED NOISE LEVEL DESIGN CRITERIA

Observer Category			Structure	L50* Day	L50* Night	L10* Day	L10* Night
1	Residences		Inside **	45	40	51	46
2			Outside **	50	45	56	51
3	Schools		Inside **	40	40	46	46
4			Outside **	55	--	61	--
5	Churches		Inside	35	35	41	41
6	Hospitals		Inside	40	35	46	41
7	Convalescent Homes		Outside	50	45	56	51
8	Offices	Inside	a) Stenography	50	50	56	56
			b) Private	40	40	46	46
9	Theaters		a) Movies	40	40	46	46
			b) Legitimate	30	30	36	51
10	Hotels, Motels		Inside	50	45	56	51

* All levels measured in dBA:
L_{50} - exceeded 50% of the time
L_{10} - exceeded 10% of the time

** Inside or outside design criteria can be used, depending on the utility being evaluated

From Digest 24, NCHRP - Research Results Digest, December 1970, p. 6

where V is the velocity of surface particles in feet per second (fps). An acceptable level of vibration is 1/6-fps peak particle-velocity, which corresponds to an ER of 1.

As a rule, damage is unlikely to occur where ER is under 3; where ER is under 6, the damage to adjacent structures from pile-driving operations is rare, although it may be enough to crack plaster, but buildings are in a danger zone where ER is over 6.[2]

The ER becomes negligible in sands beyond a distance equal to the pile length, although this distance might be two or three times the pile length in clays, while the vibration can be transmitted substantial distances in rubbish-fills. Also, vibration transmission is generally greater as the groundwater elevation rises.

Vibratory hammers with a 15- to 25-cps frequency range may cause settlement problems. The human response to vibration is a poor guide to possible damage. The human body is sensitive to one-tenth of the dynamic or impact hammer vibrations that will cause damage.

2. Liability

Some states, like New York, deny liability for damage resulting from concussion in the absence of proof the work was done in a negligent manner. Many states, however, impose liability for damages without proof of negligence on the theory that such work is inherently dangerous and constitutes a nuisance. Approaching the problem on the latter basis would seem to be the prudent course.

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2. Luna, Wm. A., "Ground Vibrations Due to Pile Driving" Foundation Facts, Raymond International, Volume III, No. 2, 1967, p. 3.

G. SAFETY

Safety requirements for excavations, trenching and shoring, tunnels and shafts; and caissons, cofferdams, and compressed air are defined in OSHA Subparts P and S.

A special problem in urban areas involves locations where petroleum products have leached into sands over a period of years. The danger comes not only from the gasoline-fume explosion-hazard, but from oxidation and the resultant reduction in oxygen level.

Utilities may have special requirements which must be observed (natural-gas lines are discussed elsewhere), and special precautions should always be taken in backfilling around lines, particularly natural gas.

H. VENTILATION

1. <u>Ventilation Rate</u>

Positive ventilation must be provided in cut-and-cover construction, especially if areas are decked over. Ventilation air should be drawn into the area from a point well above street level to prevent drawing contaminated street-level air into the excavation. At least 200 cfm of fresh air should be provided for each employee underground.

2. <u>Underground Motive Power</u>

Diesel engines are the only internal combustion engines that can be used underground, and the Bureau of Mines must certify all equipment used underground that is so powered.[3] No gasoline or liquefied petroleum gases can be taken, stored, or used underground.

3. Some local authorities require scrubbing of diesel exhaust.

III GEOTECHNICAL INVESTIGATION AND ANALYSIS

The geological stratifications and geotechnical characteristics, including the ground water conditions, must be determined by a comprehensive subsurface investigation and analysis. This data is necessary for designing temporary and final structures and for selecting the most feasible construction approach. The behavior of the selected structural system must be analyzed for the particular subsurface conditions at the project site.

The analysis methods are comparatively well developed in theory, and there is no doubt that the increasing understanding and application of elasticity theory in geotechnical analysis is a great leap forward in analyzing the type of problems associated with cut-and-cover construction work. However, the authors feel strongly that several areas of applied geotechnical engineering are lagging. Two of these are sampling and testing by commercial contractors and laboratories, and adaptation of the design to the actual construction operation. The course of this study confirmed these feelings, and this section has consequently been enlarged well beyond the area originally considered as being of direct interest for cut-and-cover construction work.

Both the type of geological formation and the intended approach for analysis govern the selection of the field procedures for the subsurface investigation. The feasibility of various ground-support, dewatering, underpinning, and excavation systems should be studied in the light of the preliminary information, and the final investigation should be directed toward solving the particular problems associated with the selected systems.

Additional subsurface investigations may be made when construction begins, or at any time during construction, either because of changes in the final design or to obtain knowledge for special construction methods, or because the actual ground conditions differ from those earlier predicted. Canadian and European contractors seem to more commonly make field checks to assure themselves that their equipment and methods will be suitable for the actual subsurface conditions than do their American counterparts.

With one exception, the contractors among the questionnaire respondents found the geotechnical information obtained by the engineer before construction was satisfactory both for planning the work and actual construction. The geotechnical information is generally given to the contractor as boring logs and soils profiles, and a narrative description or soils report is sometimes included.

The contractors are generally satisfied with the soils investigations, interestingly enough, probably because of the latitude they then have in contract renegotiations for extras. The soils engineers and designers for their part often feel the state-of-the-art needs to be improved in both field procedures and analysis methods. One gets the impression that working construction solutions result from using an overly conservative and costly design to compensate for the lack of good information and applicable theories.

Geotechnical studies are not generally concerned with locating underground utilities. The field methods specifically used for this are described in Chapter VII, par. C. The studies may produce some data on actual locations as a by-product, however.

A. COMMON FIELD-INVESTIGATION PROCEDURES

Soil boring and sampling are the major field procedures, and these may be supplemented by test pits, geophysical methods, visual inspections, aerial photographs, existing boring logs, and geological maps. Aerial photographs and surficial visual inspection in built-up areas have little value other than for planning purposes. Existing boring logs and geological maps will indicate the different soils that may be encountered, and will aid in planning the type of preliminary exploration program that will be necessary. Test pits complement other subsurface investigation methods, allowing the soil stratification to be seen and tested in-situ.

1. Borings and Sampling

The basic subsurface investigation methods consist of soil borings advanced by rotary drilling of solid or hollow-stem augers, by a cutting bit, by rotary or fixed cutting tools, or by wash-and-chop. The hole is, if necessary, stabilized by either casing or bentonite slurry. The cuttings are removed by augers, air, water, or slurry. Other methods, such as down-hole hammers or casing driven by pile-hammers, are used especially in hard, coarse, granular soils.

The standard penetration test with split-spoon samples and the 2- or 3-inch or larger diameter Shelby tubes for undisturbed samples are the most common field testing-and-sampling methods. In-situ shear-vane tests, which are frequently used in Europe to determine the shear strength of clays, are now more commonly used in the USA. Miniature shear-vanes and pocket penetrometers are used in the field for approximate cohesive-soils shear-strength determinations and to help check the continuity of zones.

The standard penetration test is regarded as a simple and practical field test, but numerous factors may influence the result and make the interpretations doubtful. The blow-counts have been correlated with various soil parameters. While the correlation factors vary between different geological formations, the standard penetration test is normally satisfactory for designing minor projects with some complementary information. For cut-and-cover projects, this test can at the best be regarded as a test indicative of relative changes.

Reasonably undisturbed samples of cohesive soils can be obtained with 3-inch-diameter Shelby tubes. The degree of disturbance partly depends upon the sampling procedure. Poor recovery and great disturbance are often obtained in soft clays or in highly-sensitive clays. The 3-inch Shelby tube is difficult to push in hard clays, and 2-inch tubes are sometimes substituted. Hard clays can sometimes be sampled by coring.

Up to 5-inch-diameter samplers for obtaining undisturbed samples are commercially available in the U.S. However, drilling contractors report that very few requests are made for samplers larger than 3-inch diameter. The larger samplers will require the use of correspondingly larger-diameter casings and augers, which means the contractors must tie-up a considerable amount of money to meet an infrequent demand. The larger contractors generally have both the larger-diameter samplers and the necessary auxiliary equipment. Piston samplers are also rather commonly available but, again, the requests for these samplers are limited. Consequently, few experienced drillers can use the more technically developed samplers.

The quality of the undisturbed samples may be affected by a number of factors, including the sealing, shipment, and laboratory handling methods, the storage of the tubes, and the extrusion procedure. The development of improved and standardized sealing devices and shipping containers for the various types of samples will help to improve the validity of the laboratory tests.

2. Geophysical Investigation Methods

Seismic refraction or reflection measurements can sometimes be very successfully used to continuously identify the stratification between bore holes. However, the ambient noise level may make such measurements difficult and the environment of built-up city areas precludes the use of sizable explosive energy-sources. Continuous signals generated by vibrating plates or other devices have been successfully used for reflection measurements in cities, but the method is best adapted to observation at greater depths than are of interest for cut-and-cover construction work. Radar-type equipment has also been used, as described in Chapter VII, par. C2 for utility locating. These measurements can also provide echoes from changes in soil stratification similar to seismic-reflection measurements.

B. GEOTECHNICAL ANALYSIS

The geotechnical analysis must develop the magnitude and the distribution of earth pressures on both the temporary supports and the permanent structure. The analysis must consider the ground-support system to provide an economical and safe excavation without excessive earth movements. The actual performance of the ground-support system depends upon the construction procedure, the workmanship, and local variations in the soil. These factors complicate evaluating the records from monitored excavations.

The accuracy of the estimate of the magnitude and distribution of the soil movements that will occur outside the excavation is of utmost importance when adjacent utilities and buildings must be protected. Both horizontal and vertical movement tolerances should be established for each unit, and underpinning or other protective measures should be planned if the allowable movement for the structure will be exceeded. The differential settlements that can be tolerated in buildings are shown in Table III. The accuracy of the settlement predictions should be in reasonable proportion to the amount of differential movements that a structure or utility can take.

The dewatering requirements and the dewatering system design must be part of or coordinated with the geotechnical analysis. The dewatering becomes especially critical if the groundwater drawdown can cause consolidation and settlement of the surrounding ground because of reduced buoyancy (see Chapter IV, Ground Water).

Measurement of earth-pressures and bracing loads in actual construction sites generally indicate considerable differences from

TABLE III

DAMAGE CRITERIA FOR SETTLEMENT OF BUILDINGS UNDER THEIR OWN DEAD WEIGHT

Angular Distortion	Damage Criteria
1 to 750	Difficulties with machinery sensitive to settlement are to be feared
1 to 600	Danger for frames with diagonals
1 to 500	Limits for buildings where cracking is not permissible
1 to 300	Cracking in panel walls and difficulties with overhead cranes are to be expected
1 to 250	Tilting of high rigid buildings might become visible
1 to 150	Considerable cracking in panel walls and brickwalls
	Limit where structural damage of general buildings is to be feared
	Safe limit for flexible brickwalls, $h/L < 1/4$

After Skempton and MacDonald (1956)
Bjerrum (1963)

the calculated values, see Figure 4. These discrepancies can basically be attributed to construction procedure and workmanship and to application of the theories. It seems the pressures in dense granular soils are sometimes conservatively overestimated while they are underestimated in clays. See Figures 5 and 6 showing measurements by the Toronto Subway Commission. The design engineer, of course, has only limited control over construction procedures and still less control over workmanship.

It takes great experience to accurately estimate the magnitude of total earth-pressure for design purposes, and it is still more difficult to estimate the load on individual struts. Redistribution of earth-pressure is caused by arching effects transferring loads from one area or elevation of the wall to another. The loads on some struts may be seriously underestimated by the classical earth-pressure theories based upon limited equilibrium conditions.

It is therefore prudent to use two or more rows of struts, according to the Toronto Transit Commission, because if an "overloading of any one row of struts occurs, causing buckling, the overload is then transferred to the second or third rows of struts, frequently preventing a total collapse."[4]

The analysis is made on the assumption that enough deflection of the support system will occur to reach the assumed

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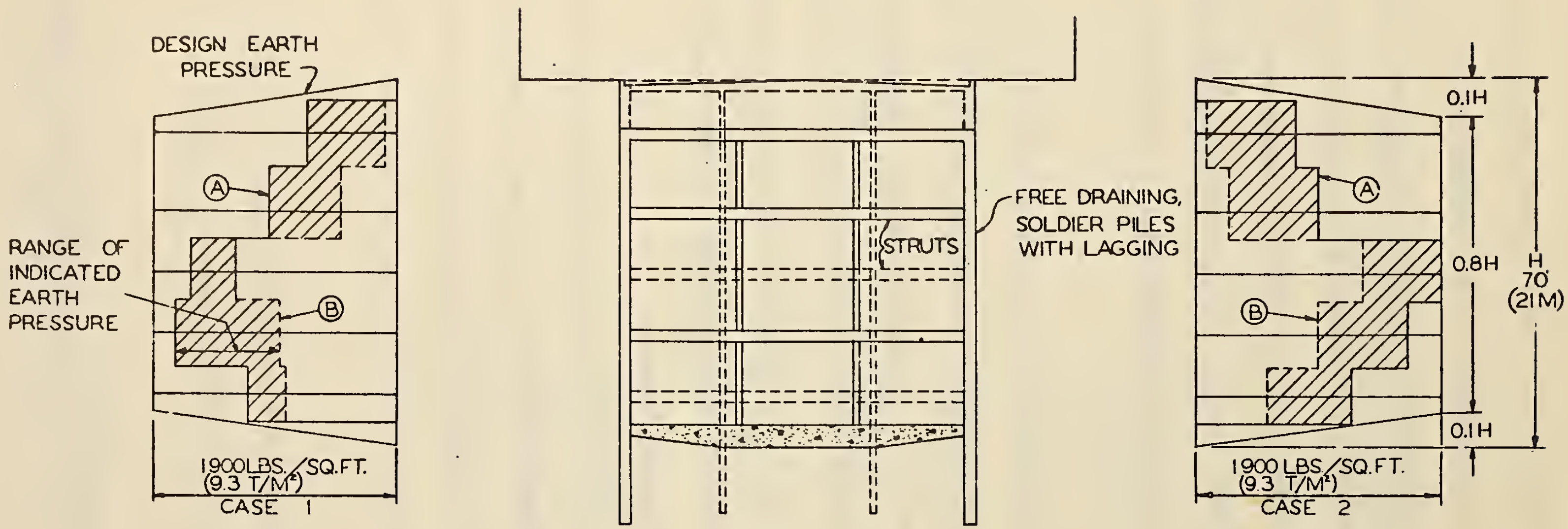
4. Lateral Earth Pressure Studies on Strutted Excavations of the Toronto Subway System, Toronto Transit Commission, July 1967, p. 7.



Ⓐ AVERAGE PRESSURE CALCULATED FROM LARGEST MAXIMUM STRAIN GUAGE READING
Ⓑ AVERAGE PRESSURE CALCULATED FROM SMALLEST MAXIMUM STRAIN GUAGE READING

MEASURED STRUT LOADS FOR TWO TYPICAL CASES AT MONTGOMERY STREET STATION (BART) (AFTER T. KUESEL 1969)

FIGURE 4. MEASURED STRUT LOADS FOR TWO TYPICAL CASES AT THE MONTGOMERY STREET STATION, BART. (By permission of the Sociedad Mexicana de Mecanica de Suelos, A.C.)

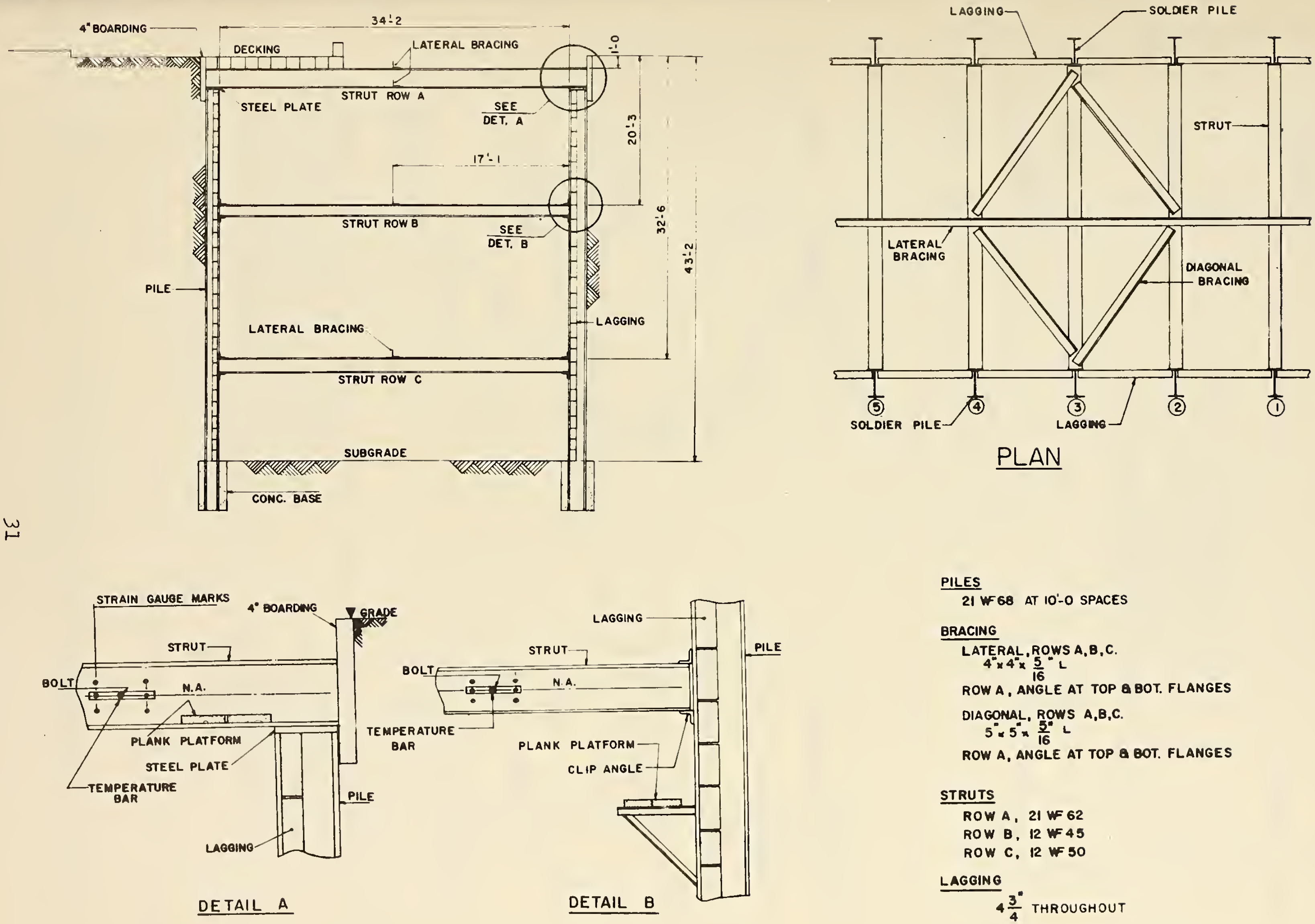
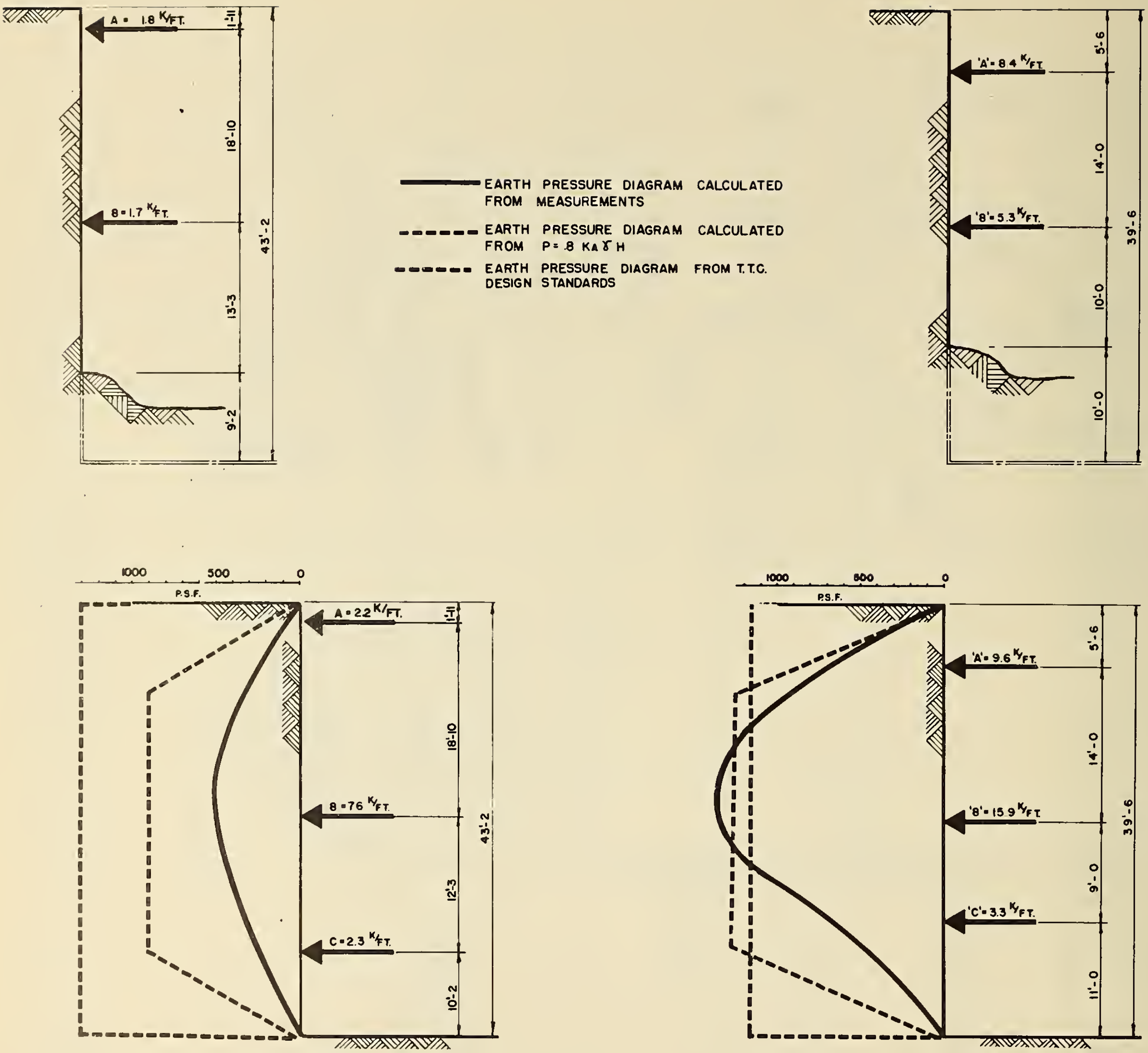


FIGURE 5. TORONTO TRANSIT COMMISSION TEST SECTION -- Bloor-Danforth subway ground-support system plan and cross section for Test Section D 7095. Strain gauge and temperature bar installations are shown in Details A and B. After the Toronto Transit Commission 1967 Report. (By permission)



Test Section B 4019
Medium sand with some coarse and fine fractions. The sand was in a very dense state, except in the upper 4-8 ft where it was loose-to-compact. The natural dry density was 95 pcf in the upper loose sand, and 108 pcf in the dense sand.

Test Section B 4233
Surficial 7-ft layer of dense sand, followed by a thick stratum of hard clay extending approximately to invert level, and then a layer of water-bearing sandy silt.

FIGURE 6. LATERAL EARTH-PRESSURES, Toronto Transit Commission 1967 Report. (By permission)

stress-condition on which the earth pressures are based. The settlement outside the excavation can eventually be estimated based upon the deflection of the wall, assuming the soil volume is constant. Consolidation of the soil due to the induced stresses or due to dewatering may give errors in these calculations. Predicting the settlement is further complicated if cohesive soils of low strength extend below the excavation bottom. The inward-upward soil movements which cause the bottom-heave phenomenon, may completely control the settlement outside the excavation and, as said above, the pressure distribution on the retaining structure. The best estimate can many times be obtained from experience with similar work.

The various methods for design of ground-wall support are described in Chapter V. There are, generally speaking, three basic methods to estimate the earth pressure against a retaining structure, i.e., the conventional limited-equilibrium and semi-empirical methods, to which the discussion in the preceding paragraph basically pertains, and the more recently developed elastic methods.

The analysis-method selected depends upon the magnitude of deformation of the ground-support system and soil. The ground-support system for cut-and-cover construction can in this respect be grouped in two categories. Sheet-pile walls and walls consisting of soldier piles and lagging can be characterized as flexible, even if the members are of heavy section. Slurry-trench walls, either bar-reinforced or reinforced by H-piles (SPTC-walls), can be characterized as semi-rigid.

A variety of limited-equilibrium methods have been developed which may be recognized as the Rankine, Coulomb, Sokolovski, Janbu, Brinch Hansen, and U. S. Navy (log-spiral) methods, which differ in the assumptions about the shape of the failure plane. The variations in total pressure between these methods may fall within ± 10%, which is rather insignificant considering the general uncertainty of the strength parameters and boundary conditions.

The deformations are not considered in the limited-equilibrium analysis further than to place the factor of safety sufficiently high to avoid "detrimental" earth movements. The deformation of a flexible ground-support system for a deep excavation is generally large enough to cause considerable redistribution of the earth pressure from the limited-equilibrium values. This may lead to dangerously high stresses on individual struts even if the total pressure may be rather correct. The magnitude of the wall deformation must be known if structures sensitive to ground movement are located within the area of potential influence. The limited-equilibrium methods are, for these reasons, less applicable for ground-support system designs for cut-and-cover construction work.

The semi-empirical methods are, to a large extent, originated from work by Terzaghi and Peck. The pressure distributions are based on actual field observations of construction using flexible ground-support systems. The basic idea in developing the earth-pressure envelopes is that the stress will never, at any point, exceed what is generated by the fictitious earth pressure (see Figure 7).

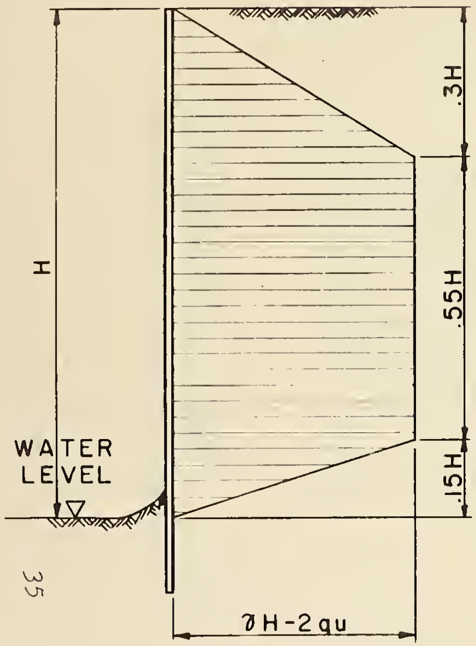


FIG. A

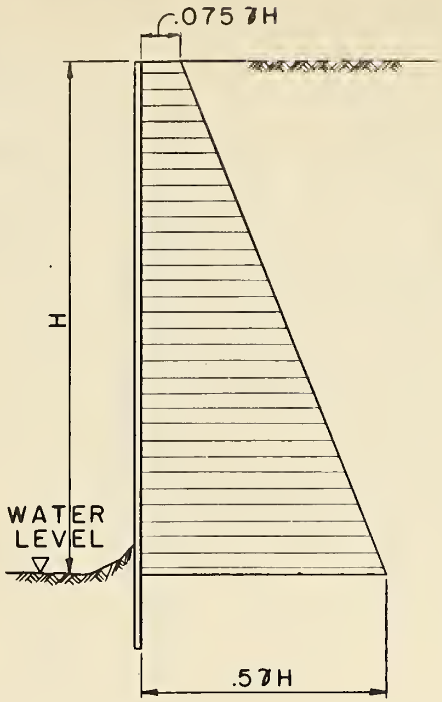
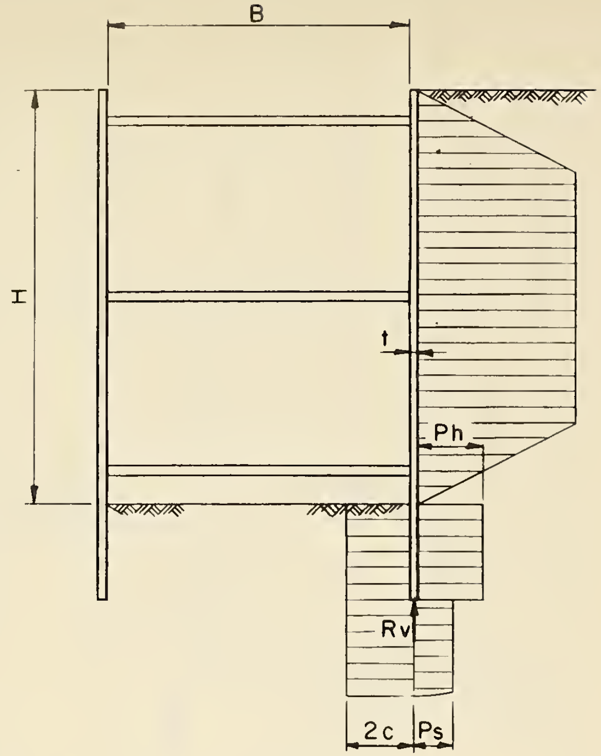


FIG. B



for $H_1 < \frac{2}{3} \times \frac{B}{\sqrt{2}}$ $\qquad Ph = 1.5\left[\gamma H - \frac{\sqrt{2}\,CH}{B} - \pi C\right]$

for $H_1 > \frac{2}{3} \times \frac{B}{\sqrt{2}}$ $\qquad Ph = \gamma H - \frac{\sqrt{2}\,CH}{B} - \pi C$

$$Ps = \gamma H - \frac{\sqrt{2}\,CH}{B} - \pi C$$

$$Rv = t(3.8C + \gamma H_1) + 2CH_1$$

FIG. C

Figures A and B show the earth-pressure envelopes when the material at the bottom of the excavation is firm. Figure C shows the pressure when clay extends below the excavation bottom. The factor of safety against bottom heave (Figure C condition) can be estimated as $F = \frac{cNc}{\gamma H}$, where Nc is a bearing-capacity factor which depends on the shape of the excavation. Hydrostatic pressure should be added to the envelopes if ground-water is present.

FIGURE 7. SEMI-EMPIRICAL EARTH-PRESSURE ENVELOPES FOR MEDIUM-STIFF TO SOFT CLAYS

Soft clays extending below the excavation bottom may seriously affect the pressure on the retaining structure, and the lower set of struts may be highly stressed, in particular.

As can be seen in Figure 8, the active earth pressure below the base of excavation is greater than the passive earth pressure to a considerable depth, and the bottom strut must resist this pressure. The soil will tend to move inward and upward into the excavation and may result in a bottom heave which may completely control the settlement outside the excavation.

The finite-element method, which is based on elasticity theory, holds the most promise for computing soil movements and stresses in the ground-support system. Many of the most important variables such as the deformation properties of the soil and retaining structure, the excavation and construction sequence, and the surcharge outside the excavation can be taken directly into consideration. Some of the variables can be predicted with considerable accuracy. However, the soil stress-strain properties are generally difficult to establish. The modulus of elasticity can be considered to be linear for small stress changes, but this does not hold true in material like loose sand. Furthermore, most geological formations are anisotropic and the modulus of elasticity increases generally with depth. Figure 9 shows how the ground-support system and adjacent ground deformations can be predicted, along with the strut loads.

The finite-element approach can be used at the present time with satisfactory results for semi-rigid retaining structures such as slurry walls where the deflection is relatively small. Research and development is urgently needed, however, to permit

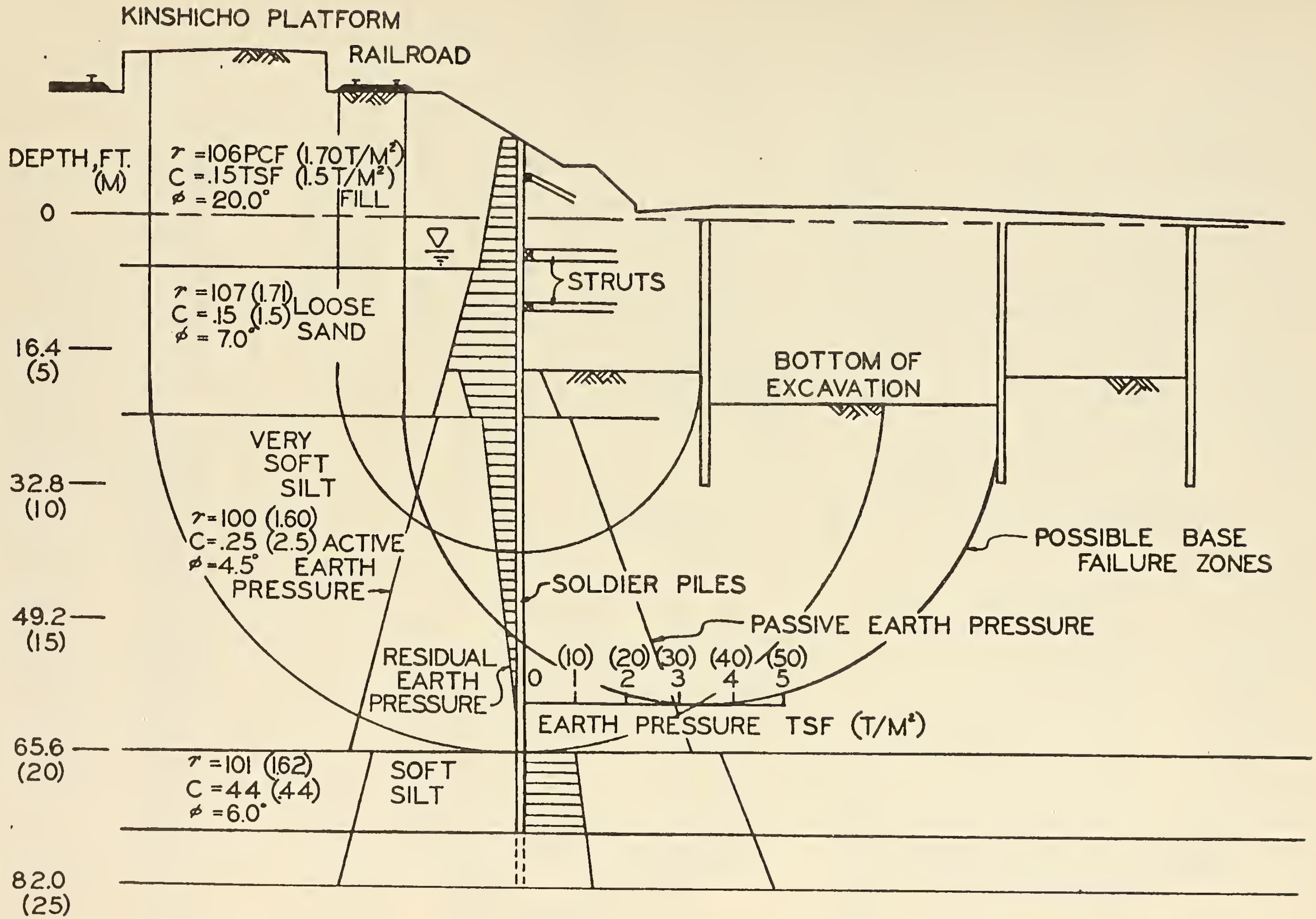
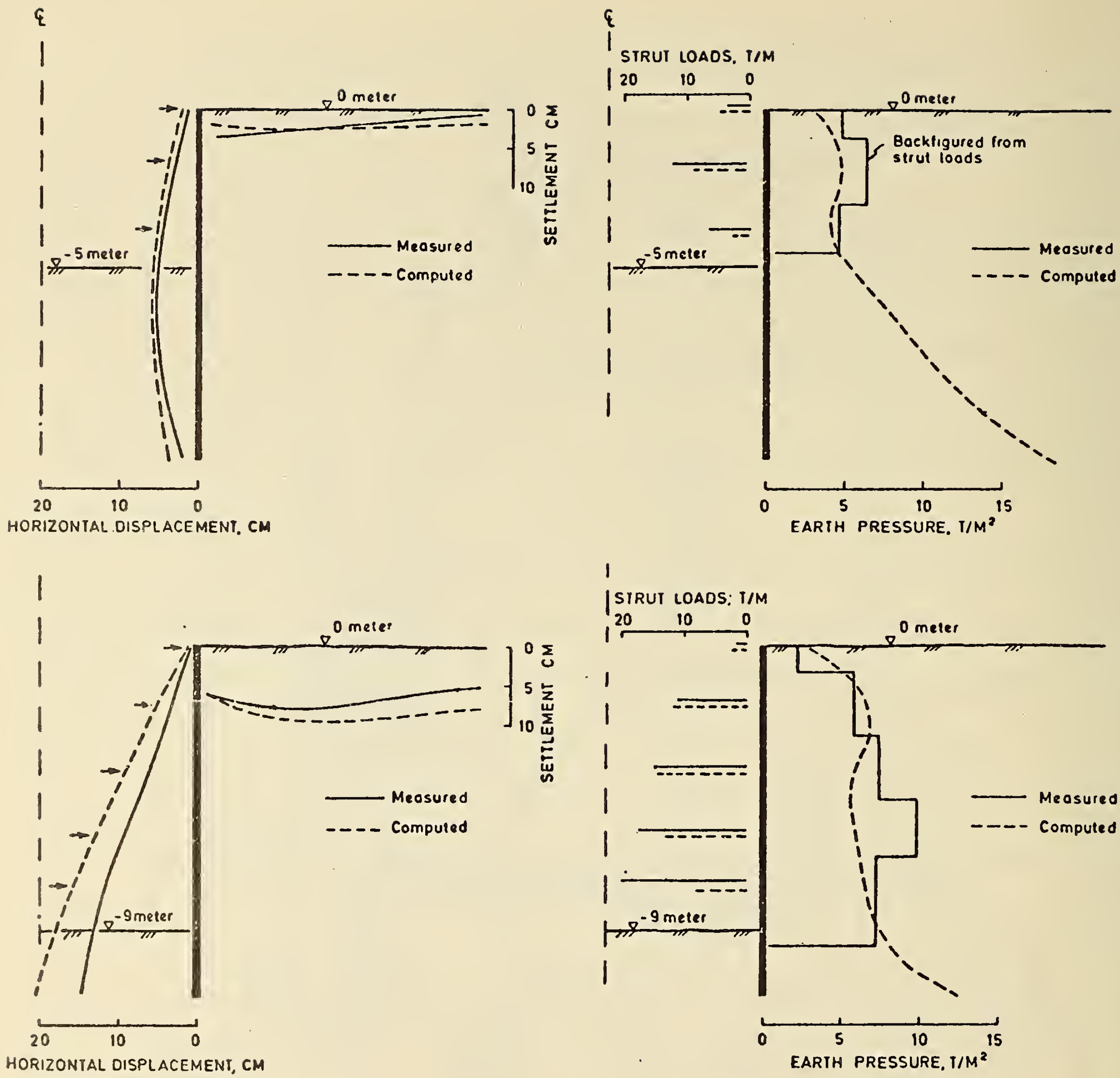


FIGURE 8. LATERAL EARTH-PRESSURE DISTRIBUTION, Kinischicho Station. (By permission of the Sociedad Mexicana de Mecanica de Suelos, S.A.)



Comparison between observed earth pressures, horizontal displacement and settlement and those calculated by the finite element method. Vaterland 3, Oslo. After Bjerrum.

FIGURE 9. EARTH PRESSURE BY THE FINITE-ELEMENT METHOD. (By permission of the Sociedad Espanola de Mecanica del Suelo.)

practical and reliable application in these cases as well as to flexible retaining-structures.

C. IMPROVED INVESTIGATION METHODS

The quality of the investigation must conform to the quality of the analysis. Much basic research and development are needed to provide data for the methods of analysis which are based on elasticity theories, e.g., the finite-element method. But several other limited-equilibrium and semi-empirical analysis methods also need standardization and improved adaptation to practical field conditions.

Earth-pressure analysis requires, among other things, determining the shear strength of the soil by *in-situ* shear-vane, unconfined compression, triaxial, or direct-shear test. The triaxial test costs are frequently a deterrent and more common use in the future may bring the price down. The need for standardized procedures for extension tests should be investigated. A need for a standardized direct-shear test procedure is also apparent. Figure 10 illustrates the adaptation of different test methods to the stress condition at various points along a potential failure-plane.

The field investigation of clays, especially soft or sensitive clays, should include *in-situ* shear-vane tests for determining the undrained shear-strength. Consideration should be given to the possible influence of the strain rate on the shear strength. High strain-rates in testing will produce larger undrained shear-strength

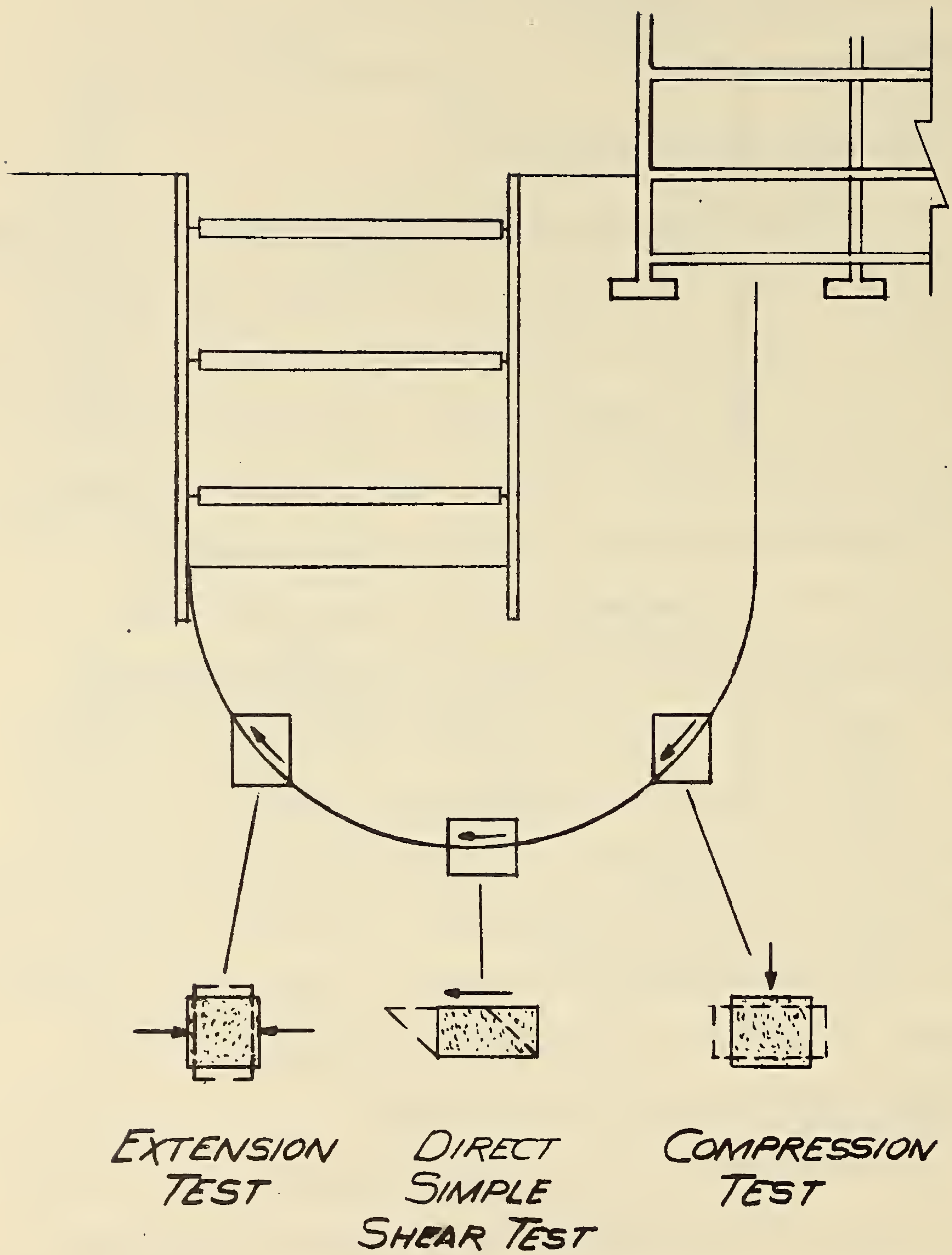


FIGURE 10. RELEVANCE OF LABORATORY TESTS TO SHEAR STRENGTH IN THE FIELD. (Adapted from Bjerrum. Reprinted by permission.)

than actually can be relied on in field during prolonged loading conditions.

The *in-situ* shear-vane test has been in common use in Europe for many years. It is becoming increasingly popular in the USA, but in many parts of the country neither the equipment nor the experienced field crews are readily available from drilling contractors.

The Norwegian Geotechnical Institute under L. Bjerrum has made great strides in understanding the properties and the behavior of soft sensitive clays. They are also progressing in their research of the anisotropic properties of soil* and in developing equipment and methods for determining at-rest pressures. The *in-situ* at-rest pressure in normally-consolidated clays where K_o is less than 1.0 can be determined by hydraulic fracturing. This method is reported by Bjerrum and Anderson*, who used an ordinary Geonor piezometer to measure the pressure. The pressure was measured when the crack closed, to correct for the disturbance caused by installing the piezometer. If K_o is greater than 1, a horizontal crack would be developed and thus only the overburden weight would be measured. This method is good only in fine-grained impervious soils in which the pore pressure can be built up hydraulically.

The at-rest pressures and elastic deformation properties of granular material can be determined by *in-situ* pressure-cell measurements. However, the field work and the interpretation of the field-test test data require experience. As described in paragraph A.1 of this Chapter, the standard method of obtaining "undisturbed" samples

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* See Geotechnical references 1 and 2 in the Bibliography.

in North America is with Shelby Tube samplers. The stress conditions within the sample are drastically altered during sampling and extrusion, which becomes a major influencing factor when determining the elastic properties by laboratory testing. The elastic properties can eventually be gradually restored by cyclic loading of the sample in triaxial machines and the *in-situ* modulus of elasticity be approximated. Further research and development of field equipment for *in-situ* determination of at-rest pressures and elastic properties are recommended.

The pore pressure has a decisive influence on the behavior of non-free draining soils and the pressures exerted by them. The pore pressure must be accurately predicted for the various construction stages, especially if attempts to take benefit of more sophisticated methods of analysis are made. Research is therefore needed to obtain improved methods for *in-situ* determination of the permeability and three-dimensional drainage patterns within the soil.

A cheaper method than soil boring should be developed for locating both the rock line and determine the rock quality. Where slurry-trench walls will be seated into rock one test per segment may be required if the rock surface is irregular. Modified rotary percussion rock-drilling rigs are used in some countries for this purpose.

The amount of additional information obtained by expanding a drilling program is commonly assumed to rapidly decline with an increase in the number of borings. We suggest a value-engineering model be developed based on statistical analysis to determine the optimum sampling and drilling program. Table IV shows the cost of obtaining different types of samples and the associated drilling cost.

These drilling costs are based on using augers. The prices will change with the size of the investigation program and the geological conditions, i.e., how hard the drilling is. The drilling contractor may transfer part of the sampling costs to the drilling cost or vice versa. The wide spread in cost for piston samples may decrease if piston samplers become more widely used and the drillers become more familiar with the operations.

The drilling costs to obtain 5-inch Shelby Tube samples will increase considerably if the samples have to be taken below sloughing soils because the smaller drilling contractors will not have large-diameter casing on hand, handling the large-diameter casing will decrease the production, and the cost of drilling per linear ft may increase to $20 or more. The costs for shear-vane tests may be as low as $2.00 per ft for continuous testing plus the cost of drilling. This is based on taking three shear-vane tests, clearing the hole, performing a standard penetration test, and repeating the cycle. The cost information about pressure-cell tests with a Menard pressure meter is very limited, but the price may be in the range of $30 to $50 per test, including engineering but excluding drilling.

D. FIELD OBSERVATIONS AND INSTRUMENTATION

Field observations include visual inspections and simple to complex measuring techniques. The simple methods will involve a tape

TABLE IV

APPROXIMATE COSTS FOR OBTAINING SOIL SAMPLES

Sampler	Per-foot drilling cost, approx. range	Cost of obtaining the sample
3" Shelby Tube	$2.50 to $4.50	$12 to $30
5" Shelby Tube	$4 to $6	$20 to $50
3" Piston Sample (fixed, floating, Osterberg)	$2.50 to $4.50	$14 to $50
Standard Penetration Tests	$2.50 to $4.50	$3.50 to $8

or a plumbline and the more complex techniques will involve sophisticated mechanical or electrical devices. The chances of success are improved if simple methods are used.

The two-fold purpose of the observations are (1) to provide information as a base for eventual changes in construction procedure, and (2) to provide data for recommending research and development for future projects. Immediate cost savings can be realized in both construction procedure and design through backfeeding data during the progress of the work. Whatever the purpose is, the key problems to be solved by observation must be well-specified in advance and the program tailored to provide the required data. Having only general goals may easily cause disillusionment about the justification for an observation program, including instrumentation.

Some observations should begin well before the start of construction to determine the ground-water level, consolidation rate, and other obvious features. An observation system usually includes some of the following components: settlement gauges at the surface, internal-settlement probes, piezometers, strain gauges for bracing and vertical wall members, inclinometers on wall members and in the soil outside the excavation, and pressure cells in eventually both the inside and outside face of the ground-support walls. The instruments, especially the piezometers, should be designed for instant reaction to a change in conditions.

The chance that instruments might be damaged or lost should be considered in establishing the number of instrumentation points, as should local variations at the instrumented site and the estimated area of influence. A comprehensive field diary and construction-procedure narrative should be kept as a part of the observation records. Even the most sophisticated recordings may be virtually worthless if they cannot be correlated with the construction activities.

The tolerable movement and stress limits should be established before construction begins. Alternate construction methods or, if needed, remedial measures to be taken should also be outlined in case the established limits are exceeded.

E. SUMMARY

Conventional soils-investigation methods and analysis are unsatisfactory for the final design of ground-support systems for cut-and-cover construction work in which the settlement of adjacent structures is critical. Methods must be developed allowing the practical determination of the <u>in-situ</u> deformation properties, shear strength, and at-rest pressures which will take into consideration the nonhomogenieties of the soil. Research into the magnitude and extent of the settlement caused by wall-deflections in the excavation or in clays from a combination of wall-deflection and bottom-heave is urgently needed because of the great costs involved in underpinning buildings.

Sampling tools giving better undisturbed samples and permitting safer handling of the samples should be developed and made commonly available. Less expensive subsurface exploration methods than conventional soil-boring methods also need to be developed for determining the rock line.

The commonly used limited-equilibrium or semi-empirical methods of analysis do not provide satisfactory knowledge of the soil deformations. The finite-element approach seems to be the most promising analysis method since all important parameters can be considered. Research is urgently needed on the stress-strain relationships of soils and, generally speaking, on the application and limitations of the finite-element method of analysis.

The ground-support systems should be monitored to determine the magnitude of the soil pressure, pressure distribution, and soil movements. However, no monitoring or observation program should be initiated without a well-defined purpose. Continuing research should be directed toward improving the methods for designing ground-support systems based upon these observations, especially for estimating the loads imposed on individual struts or parts of walls.

Case studies to be published in the above areas should include a comprehensive description of the construction methods and the effect the various conditions procedures have upon the instrumentation system.

"Both the designer and the contractor should more fully appreciate the importance of the method of construction in determining the actual distribution of pressure on the retaining structure" as stated in a report by the Toronto Transit Commission.[5]

This could be accomplished by a narrative of the soils investigation and analysis written in plain language and should be given to both the Contractor's Resident Engineers and the inspectors.

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5. Lateral Earth Pressure Studies on Strutted Excavations of The Toronto Subway System, page 7.

Special emphasis should be given to procedures that may improve the integrity of the ground-support system as such or on adjacent buildings.

IV GROUND-WATER CONTROL

Cut-and-cover construction frequently requires excavating below the water table, into water-bearing soils, or through perched water.[6] This may require lowering the water table below the bottom of the excavation to provide dry working conditions and a firm foundation for the construction operations. Other ground-water control objectives may be to relieve the lateral hydrostatic pressure on temporary supports for the excavation walls and to control uplift pressure in the excavation bottom.

The dewatering-system type or combination of types used for a particular project depends upon the soil and ground water conditions, the foundation conditions, and the movements that can be tolerated in adjacent buildings and structures. A thorough surface and subsurface investigation is required to determine first if a dewatering system is needed and, second, the most suitable system for the particular area. A guide for selecting a dewatering system on the basis of soil grain-size is shown in Figure 11.

The two most commonly used systems in cut-and-cover construction according to the replies to the S&P questionnaire are deep wells with submersible pumps, and open pumping from sumps. Other occasionally-used water-lowering systems are wellpoints and jet-eductors. Stabilization of the ground by electro-osmosis or accelerated drainage by sand drains may also be of potential use when the

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6. Water which lies above the normal water table because it has not yet percolated to that level, or because of a locally impervious layer beneath it.

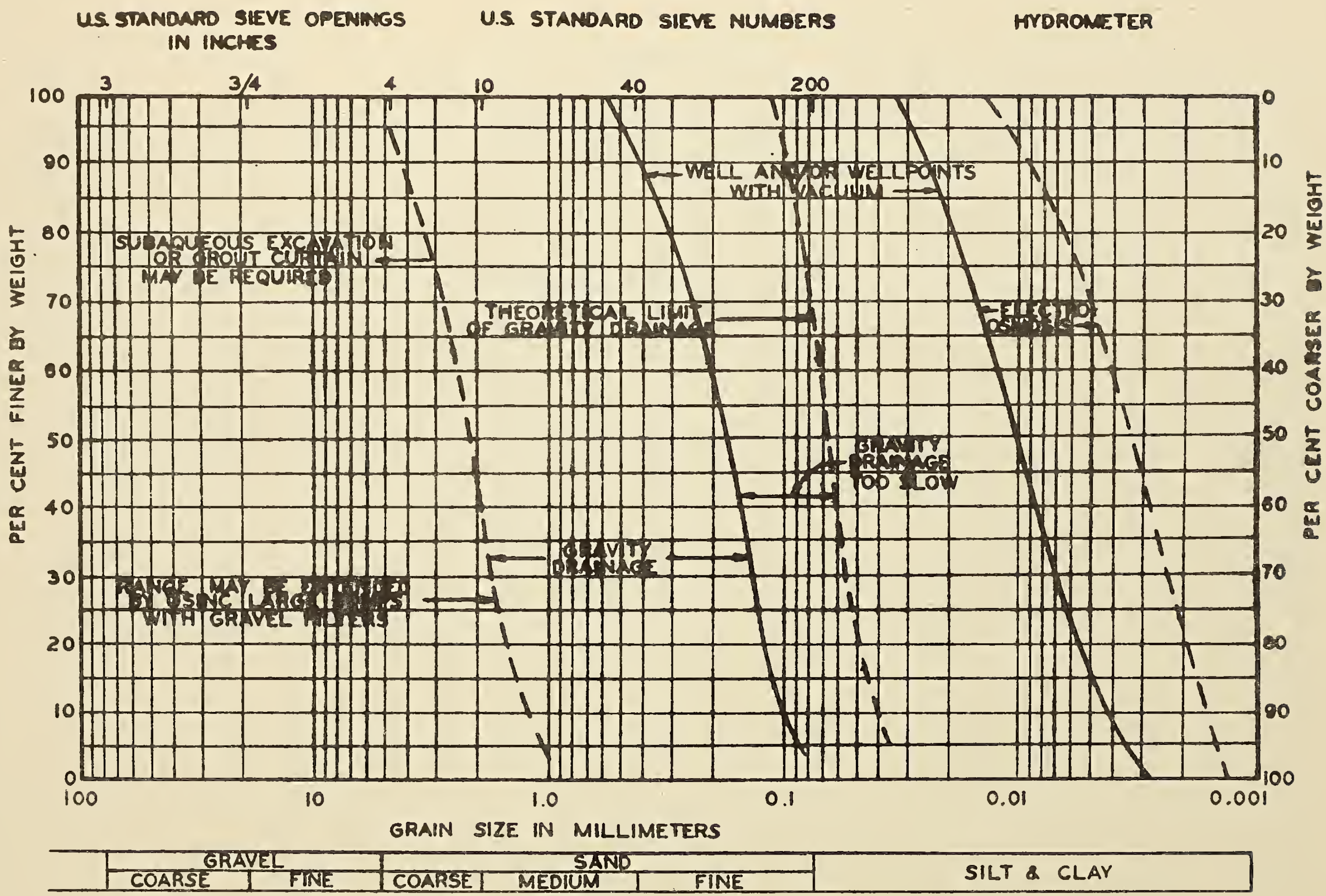


FIGURE 11. DEWATERING SYSTEMS APPLICABLE TO DIFFERENT SOILS. (By permission of the Moretrench American Corporation.)

boundary conditions permit. Examples are known where electro-osmosis has been used for direct ground-water-flow control outside a braced excavation. Several types of ground-support walls can serve as cut-off walls and thereby be used to control water flow into excavations. These walls include sheet piles, grout-curtains, slurry-trench cut-off walls and walls created by freezing.

Dewatering presents a two-fold problem for the engineer, that of how to eliminate or control the water, and also that of solving associated problems such as the settling of adjacent ground or increased pressures on temporary ground-support walls. The following paragraphs outline some items to be considered when using the various systems. The main uses, advantages, and disadvantages of the different water-control systems are summarized in Table V.

A. SUMP PUMPING

This is the most easily installed system because it consists of open ditches and pits. However, it does not relieve either the hydrostatic pressure on the ground-support walls or uplift pressure on the bottom of the excavation. The bracing system must be designed to resist the hydrostatic forces, and special construction features may be needed to control bottom heave in soft clays or boils (blow-ins) in granular soils. Construction activities may be hampered by this system, however, since the equipment is placed within the excavation.

B. DEEP WELLS

Deep wells are suitable for most types of permeable soils and are normally used for excavations over 15-ft deep. Although a

TABLE V

SUMMARY OF GROUND-WATER CONTROL SYSTEMS

Water Control System	Soil Types In Which Usable	Advantages	Disadvantages	Typical Costs per 100 ft of Excavation Length
Deep wells	Gravels to fine sands and layered soils	1. Can draw water from several layers throughout its depth 2. Vacuum can be applied to assist drainage of fine soils 3. No noise problem if submersible pump is used and electricity supply is available 4. Wide range of capacities 5 to 5,000 gpm. Most advantageous for large volumes	1. High installation cost 2. Requires individual power units 3. Occupies space outside excavation	8-inch diameter 40-ft deep, 20 ft apart $2,200 to install $3,200/mo pump rental and operation
Sump pumping	Gravels and coarse sands, clays	1. Simplest system 2. May require only intermittent pumping	1. Fines easily removed from ground 2. Encourages instability of excavation bottom 3. Sides of sump hole may need support 4. Sheeting and bracing must be designed for pressure head	$500 excavation $3,000/mo operation and pump rental
Wellpoints	Gravels to fine sands	1. Relatively quick and easy to install in most soils 2. Usually most practical and economical system for small excavation 3. More suitable than wells when well-screen submergence is limited	1. Stop cocks must be continuously adjusted to maintain a vacuum in the system 2. Single-stage lift limited to 15 ft (5m) 3. Pumping must be continuous	100 ft of 6-inch header 2-inch wellpoints on 5-ft centers $5,000/mo installed, operated 24-hr per day and removed

TABLE V - SUMMARY OF GROUND-WATER CONTROL SYSTEMS, continued

Water Control System	Soil Types In Which Usable	Advantages	Disadvantages
Jet eductor	Fine sands	1. Increases the height of wellpoint system lift 2. Can pump air as well as water 3. Useful when rate of pumpage is small	1. Requires two headers, a collector and a pressure supply 2. Efficiency is only one-third that of a wellpoint 3. Usually installed in gravel-packed holes
Electro-osmosis	Silts, clays	1. Only system for silts where gravity flow is slow or nonexistent 2. Increases stability of soil at anode	1. Installation and operating costs are very high 2. Used with wellpoints or deep wells 3. Requires more space outside excavation than other systems 4. Will increase corrosion of any nearby steel foundations
Sand drains	Fine sands, silts, stratified soils	Used in stratified soils to conduct water from upper strata to lower more pervious stratum in which wellpoint or well is located	1. Aids settlement of compressible soils which may damage nearby structures 2. Limited application in cut-and-cover construction
Cut-off walls (Steel sheetpiles) (Slurry trench walls) (Cement and chemical grouts) (Freezing)	All soils	1. Steel sheeting can become part of structure or can be recovered 2. Slurry-trench walls can also be incorporated in structure 3. Grout treatments are permanent	1. Pile driving is noisy and vibrations from driving may cause damage 2. Grout treatments are expensive and usually require extensive treatment to be effective 3. Chemical grouts are very expensive

wide range of filter-screen sizes is available, most deep wells are installed in a gravel-packed hole. This means driving or drilling a casing into the ground, cleaning out the hole, installing a pump and riser pipes, and backfilling with sand or gravel as the casing is removed. The sand or gravel backfill must be designed as a filter to prevent fines from being pulled into the pump and damaging it. Deep wells are the best system for handling artesian pressure. Pumping noise with deep-well systems can almost be eliminated if electric submersible pumps are used, and this is advantageous when around-the-clock pumping is necessary. Pumps range from 3 to 14 inches in size, with capacities ranging from 5 to 5,000 gpm. The diameter of deep wells may vary between 8 and 48 inches. This system may create operating or traffic problems in urban areas since all the equipment must be located outside the excavation.

C. WELLPOINTS

This is the most common dewatering system for open excavations and trenches used by U. S. construction contractors. The main reason for this is the relative ease of installation, since a wellpoint can be jetted into place by reverse circulation. The physical limits of suction-lift limit the amount of water lowering that is possible with a single-stage system to about 15 ft, see Figure 12. Wellpoint pumps range from 6 to 12 inches in size, with capacities ranging from 500 to 5,000 gpm. The wellpoints are usually of two-inch diameter. The approximate costs per 100 ft of tunnel are shown in Figure 13.

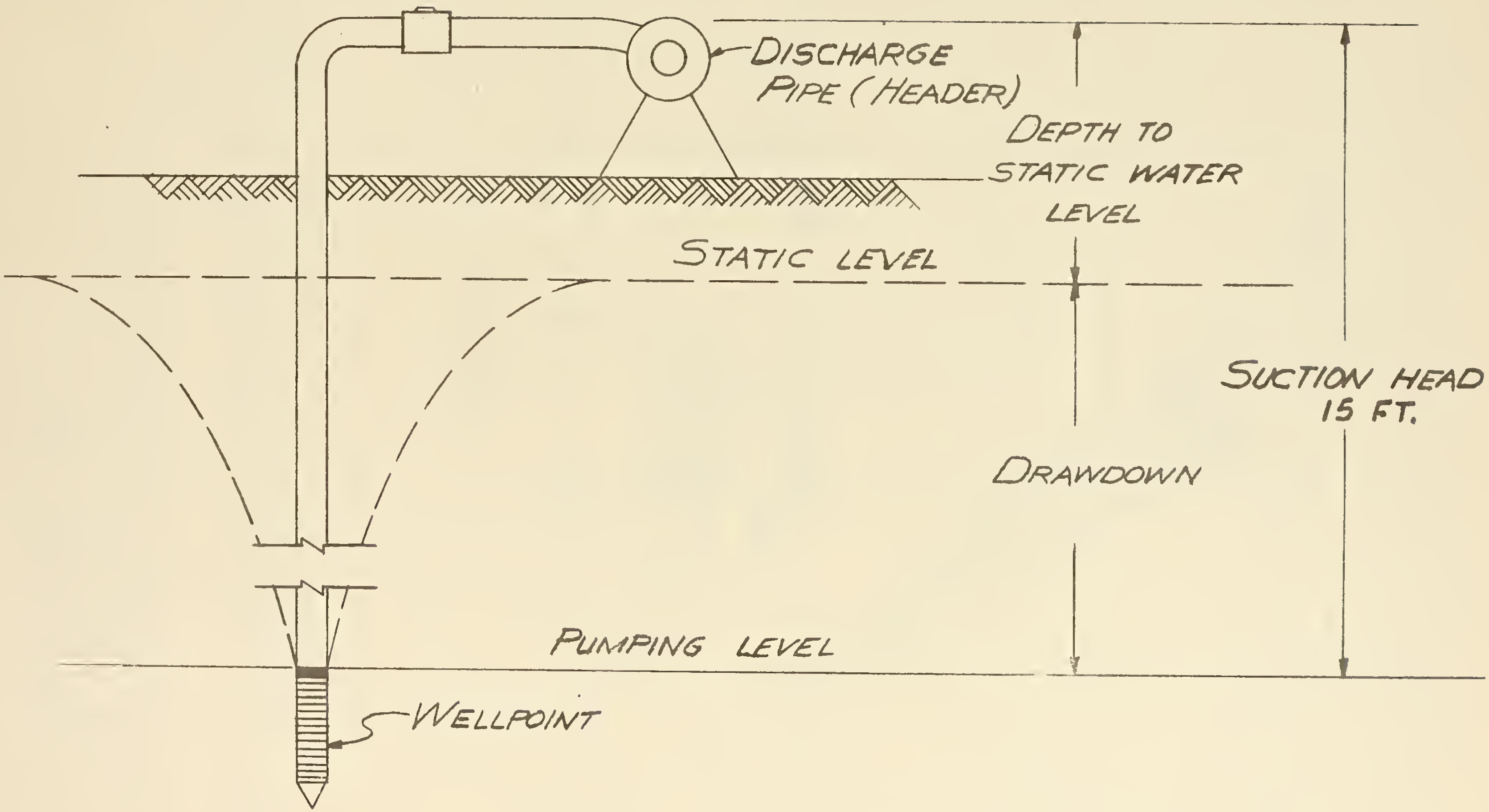


FIGURE 12. WELLPOINT SYSTEM

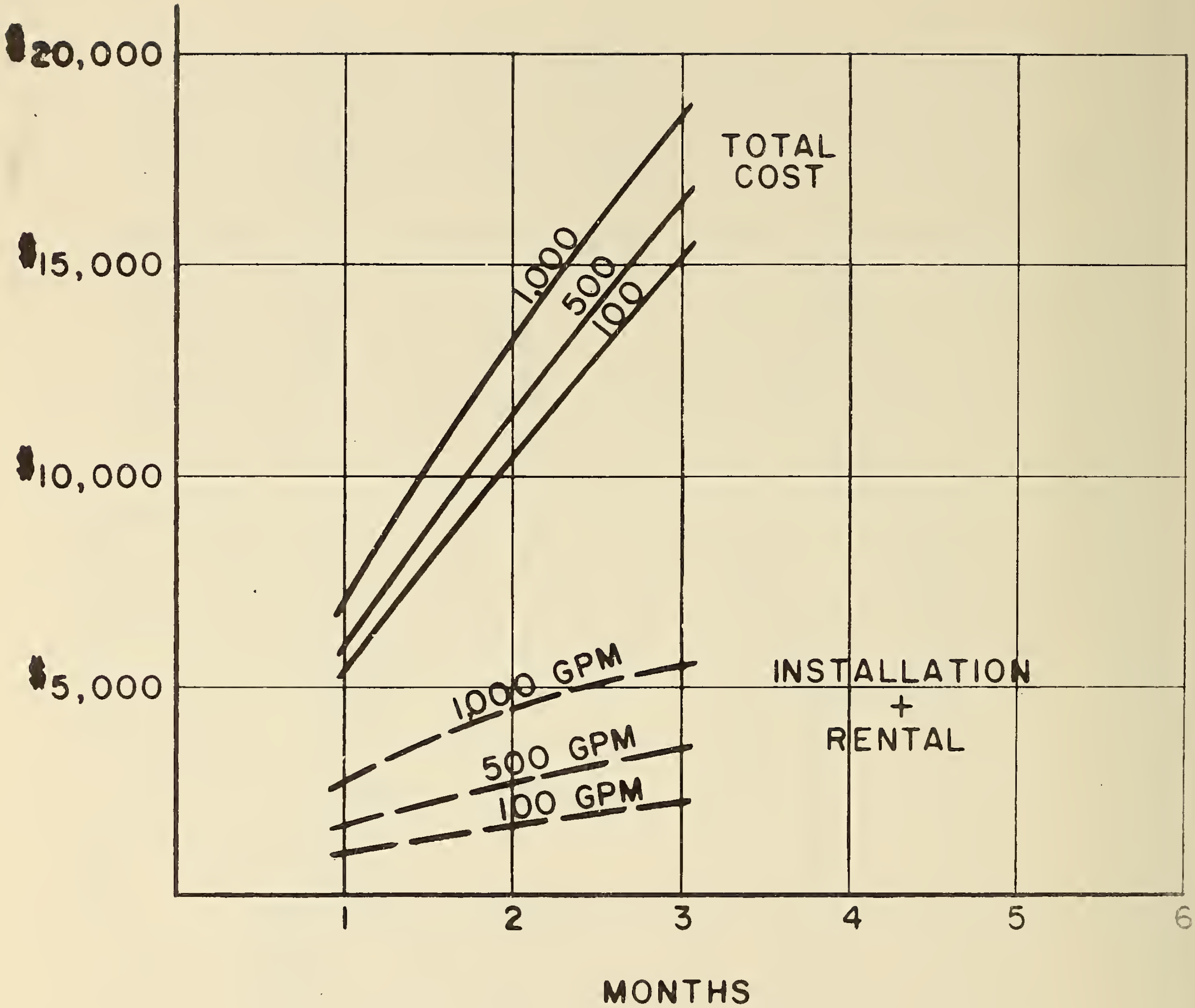


FIGURE 13. DEWATERING COSTS PER 100 FT OF CUT-AND-COVER TUNNEL CONSTRUCTION. The estimate is based upon a wellpoint system with:

1. 350 ft of header pipe on each side of the tunnel;
2. One operator for two pumps;
3. One stand-by pump; and
4. Pumping 24-hr/dy, 7 dy/wk.

Normally the first stage of wellpoints and headers is installed outside the excavation, which may hamper public traffic. Additional stages can be installed through the excavation walls at an angle. The header system is then attached to the ground-support walls.

D. JET-EDUCTORS

The eductor can be regarded as a deep-well system, since pumping can be done at considerably greater depths than is possible with regular suction pumps. However, a wellpoint is incorporated in the design and the system is therefore sometimes classified as a modified-wellpoint system. The eductor consists of a pipe orifice through which water is pumped at high velocity to create a vacuum into which air or water can be drawn, see Figure 14. Water can be lifted 50-100 ft in a single stage by this method. This system is also useful for small flows in which air would stop wellpoint or deep-well systems. Most eductors are installed in gravel-packed holes. A small eductor consisting of a 1-inch-diameter supply pipe inside a $2\frac{1}{2}$-inch-diameter discharge pipe is available which can be driven into the ground like a wellpoint. Pumping rates range from 3 to 50 gpm. Jet-eductors require one more header than the wellpoint system. However, these systems can also be hung from and installed through the ground-support walls.

E. SAND DRAINS

Vertical sand drains have been used for trenching work in combination with wellpoints or deep wells. They are used where a semi-pervious stratum overlies a pervious stratum, the sand drains

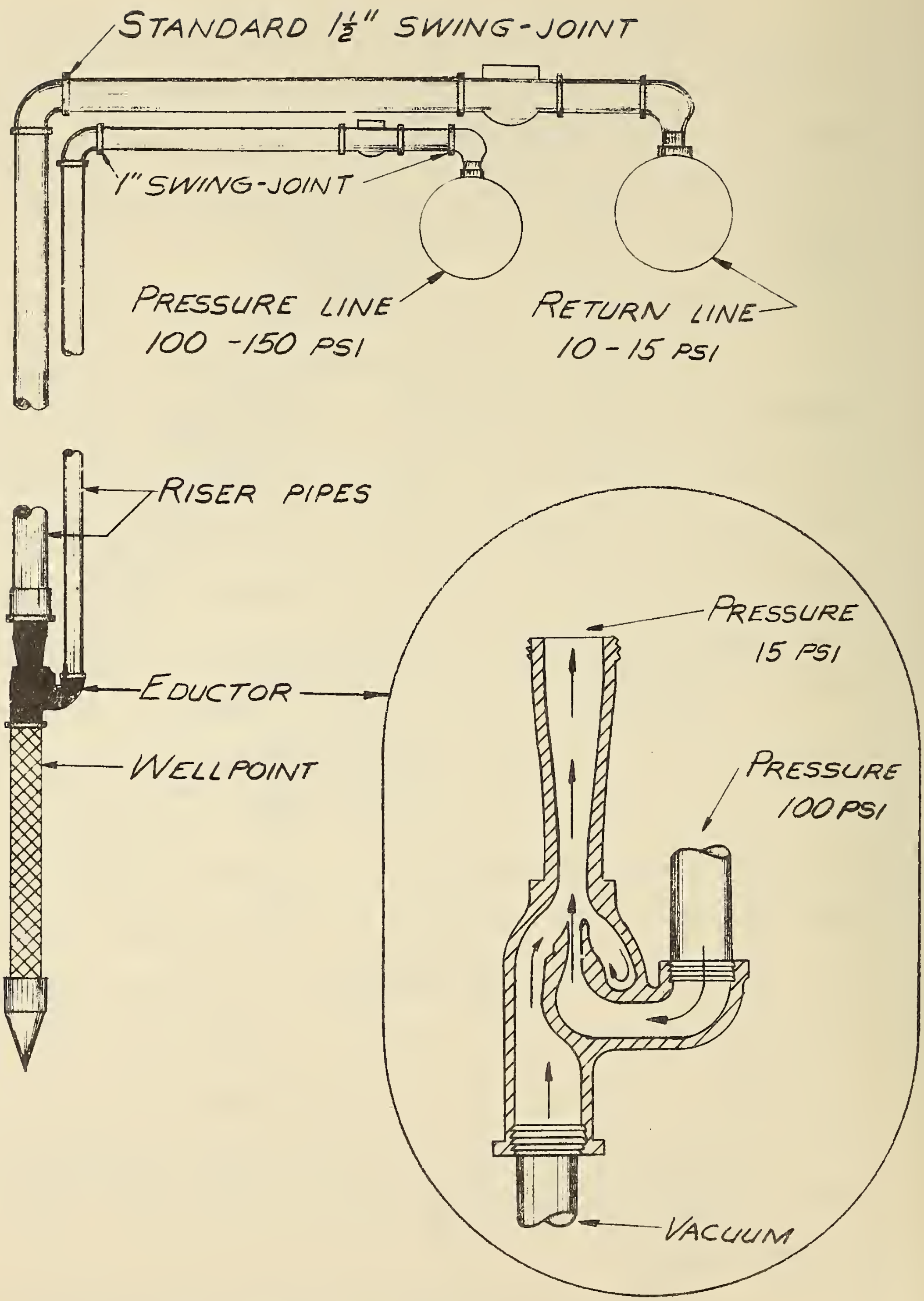


FIGURE 14. JET-EDUCTOR SYSTEM. (Engineering News-Record, October 1, 1959, p. 34, by permission.)

intercept water from the upper layer and transmit it to the lower pervious layer in which the wellpoint or deep well is located. Since the prime use of sand drains is to help consolidate soils, they have limited application for cut-and-cover construction because nearby structures may be damaged by settlement.

F. GROUND-WATER CONTROL BY COMPRESSED AIR

Ground-water inflow, as well as bottom heave, can be controlled by construction under compressed air. The method has proved to be technically very successful in other countries, especially Norway, where moderate air pressure (0.5 to 1 atm.) has been used. Restrictive medical criteria for working and decompression time has made the method less competitive, however. Figure 15 shows the construction procedure as applied in Oslo, Norway.

G. ELECTRO-OSMOSIS

Proper arrangement of the electrodes can induce seepage forces away from the excavation walls, thereby reducing the support-wall design pressures. This procedure, unfortunately, has several drawbacks with respect to cut-and-cover construction. Normally used to consolidate impermeable clays and saturated silts, severe settlement damage to nearby buildings may result in urban areas.

The moisture content tends to increase at the cathode, which may be associated with a loss of strength. The cathodes may, in unfortunate circumstances, be placed in the vicinity of the potential failure plane for active earth pressure. In this case, the soil strength will be increased at the anode immediately behind the excavation support-wall, but the overall stability will be reduced. Other

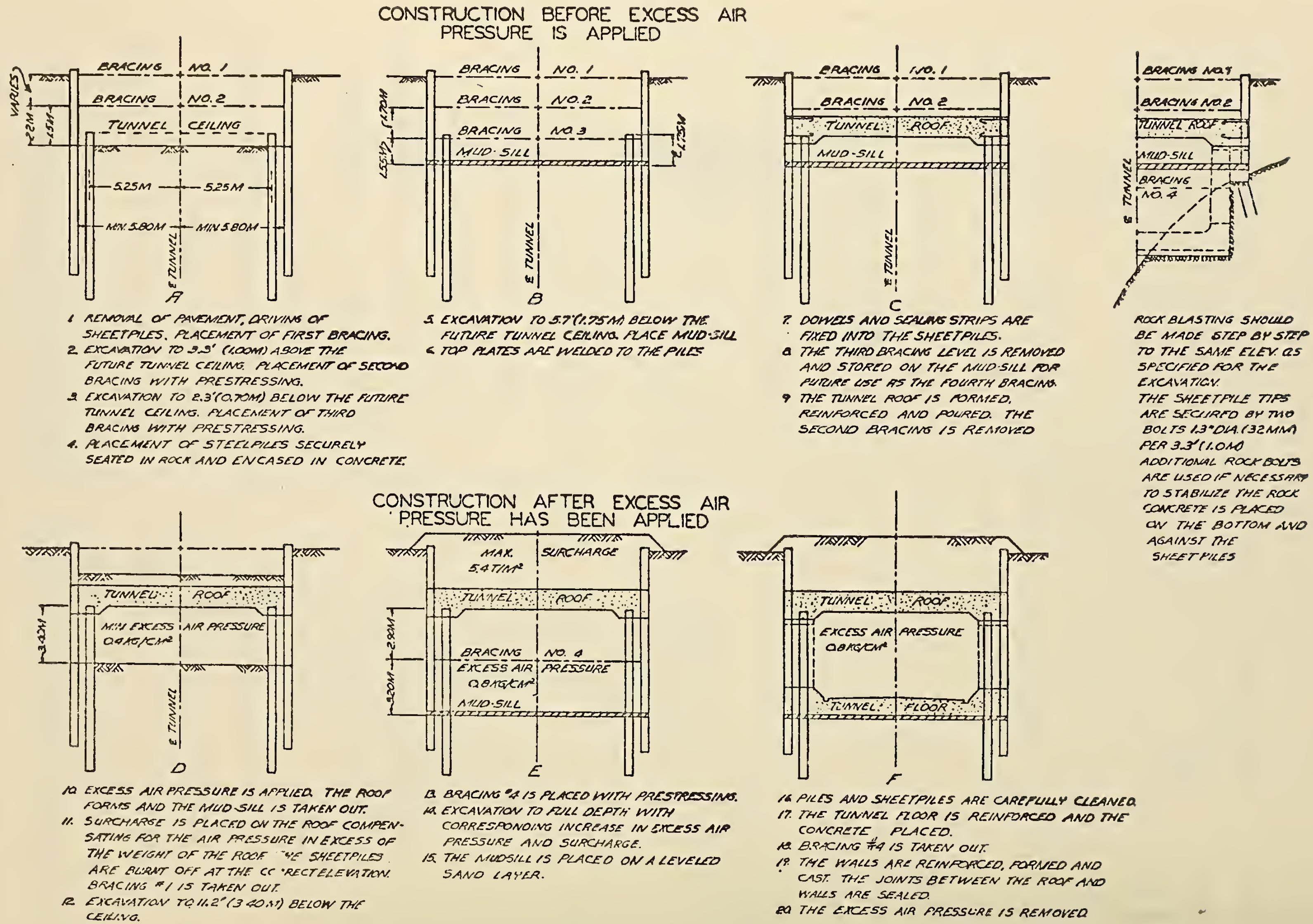


FIGURE 15. SUBWAY CONSTRUCTION PROCEDURE, OSLO, NORWAY. (By permission of E. Sunde.)

secondary effects include soil swelling from the creation of gas by the electro-osmotic process. While the swelling due to gas may offset the consolidation caused by the reduced moisture content, the end result is unpredictable. Another drawback is the stray currents that will be produced, which will increase the corrosion of any steel support-piles in the vicinity.

The electro-osmotic process has a time-lag which must be estimated and scheduled. About 30 days were required to lower the pore pressure some 3 ft in a Swedish experiment. Loss of anode material can be quite severe, but can generally be predicted. In the same experiment, 30 to 50% of the anode material was lost after 120 days of treatment.

H. CUT-OFF WALLS

Some of the ground-support walls (e.g. slurry-trench walls) are positive cut-off walls, and water can be removed from the excavation without affecting the ground-water level outside the walls. The bracing system for positive cut-off walls must be designed to support the earth pressure plus the full hydrostatic pressure. Freezing can also be considered a positive-type cut-off wall. Other ground support walls (sheet-pile, soldier-pile, and lagging) may allow some seepage. Eliminating water from excavations using semi-pervious walls as ground support will cause some drawdown behind the wall which, in turn, may cause settlement. Ground water recharging can be used to minimize or prevent any such settlement. Recharging involves pumping water back into the ground through infiltration wells or wellpoints located some distance outside the excavation.

If necessary, some construction can be done under water by placing the floor of a tunnel section by tremie techniques. This will then act as a positive seal against upward flow through the bottom. Uplift resistance may be provided by friction piles driven below the excavation bottom before the concrete seal is placed.

I. MONITORING

A monitoring or observation system is generally required in cut-and-cover construction. Observations of the ground water are necessary even if no dewatering system is installed. However, a monitoring system should not be installed if the purpose of the observations is not clearly defined.

The primary purpose of a monitoring system is to assure that no unpredictable situations will arise which would jeopardize the excavation stability or hamper construction activities. Another purpose is to compare the actual conditions with the design assumptions and to plan system changes or corrections, if necessary.

Piezometers are usually the only monitoring device used for ground-water control, but deep wells and wellpoints are occasionally used. The number of piezometers, locations, and tip elevations must be coordinated with the design assumptions and the installation must be scheduled so they are in place and equilibrium obtained before excavation and dewatering are begun. Limits must also be established before construction begins regarding the piezometric head the support walls can tolerate and what can be accepted without jeopardizing the stability of the excavation bottom.

J. CONCLUSIONS

The actual necessity for dewatering operations must be carefully considered before system selection is begun. Many items must be considered, such as ground water location, soil permeability, the type of ground-support system, and the consequences of not properly controlling the water level.

The consequences of not successfully controlling the water level or of a dewatering system failure may be severe. Probably the least that could happen will be a slowdown in work and this can be quite lengthy. If the excavation floods, for example, time will be lost because of the flood, installing or repairing the dewatering system, dewatering, and then, probably, more time to clean up the excavation. A catastrophic consequence would be loss of the temporary ground-support wall and resulting damage to adjacent buildings or property.

Another problem is scheduling the dewatering operation if ground-water control is needed before excavation begins. Sufficient time must be allowed to draw down the ground-water level and this will be a function of the soil permeability and the control-system capacity.

Disposal of the discharge water must be considered in both the time schedule and type of disposal system needed. Special permits may be required, for example, to discharge into sewer systems, and the quantities may exceed the capacity of the nearest sewer. The full capacity of the sewer cannot be used for the dewatering discharge since an allowance must be made for the runoff the sewer must carry during and after rainstorms.

The space requirements of each ground-water control system can be a problem in urban areas. Deep wells, wellpoints, and jet-eductors, require space outside the excavation, and thereby may interfere with public traffic. Sump pumping takes space from within the excavation and could therefore hamper construction activities. One solution to the outside space problem was to install a jet-eductor system at an angle through the ground-support wall from within the excavation (see Figure 16). The header system was hung from the support wall. This same procedure can be used with wellpoints, and only one step may be necessary if the pump and header system are placed at the ground-water level.

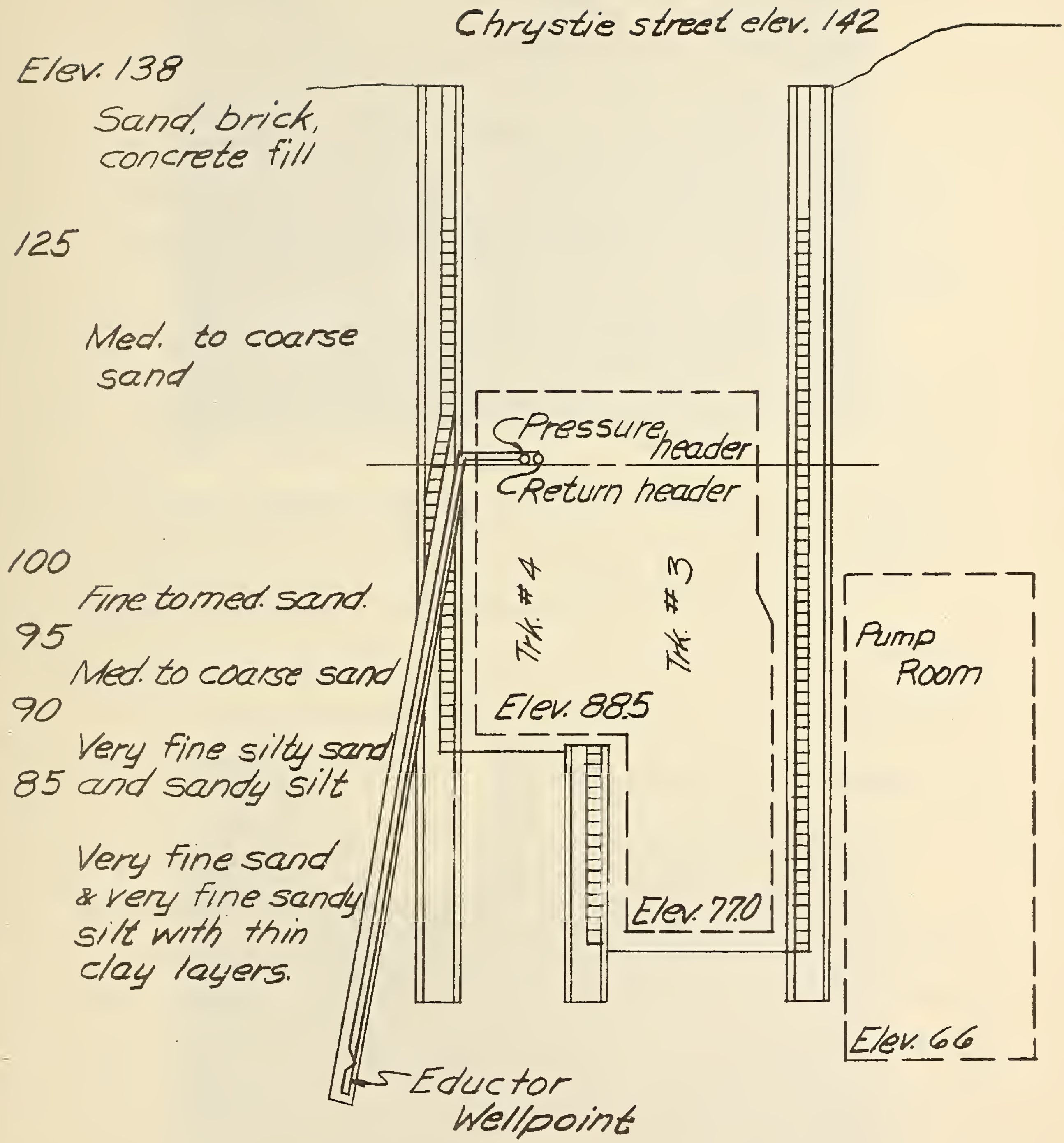


FIGURE 16. INSTALLATION OF JET-EDUCTOR SYSTEM WITHIN AN EXCAVATION. (Engineering News-Record, October 1, 1959, p. 34, by permission.)

V GROUND-WALL SUPPORT

This Chapter discusses the various components of ground-wall support which is herein defined as any method or combination of methods used to either temporarily or permanently support the excavation wall.

Braced or tied-back excavation, shotcrete, freezing, chemical injection and grouting are discussed in this Chapter. Other methods such as slurry walls and continuous bored-piles, which may also be used as permanent walls, are discussed in Chapter VIII, Permanent Structure.

This Chapter generally examines the construction material and methods, and evaluates deflection of the bracing system which can cause ground movement resulting in undesirable settlements.

A. BRACED OR TIED-BACK EXCAVATIONS

1. Soldier Beams and Lagging

The use of steel I- or wide-flange beams driven as vertical soldier beams or soldier piles with the space between filled with timber lagging dates from constructing the U-Bahn in Berlin in 1893. The Berlin method, as the technique is called, is still widely used today. Generally, steel wide-flange sections were driven at from 6- to 10-ft on-centers. Figures 17 - 20 show modern examples of this method.

Rough-hewn logs about 12 inches in diameter were originally used as lagging and were wedged between the beam flanges. The timber

FIGURE 17. TORONTO SUBWAY, Soldier-Pile Wedging System. This close-up shows the timber wedges used to keep the top of the soldier pile. The concrete was a one-bag mix to both keep the soldier pile in place and to obtain a good contact with native material.

FIGURE 18. TORONTO SUBWAY, Soldier-Pile Drilling Operations. The Contractor was having trouble augering through a boulder. The soldier pile in the foreground was used to break the boulder.

FIGURE 19. 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 20. 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.

lagging may be either new or used, and a variety of timber is used, with fir being fairly common. Precast concrete lagging has also been tried. Horizontal members called wales are installed at appropriate levels to carry the soldier beam loads to the more widely-spaced internal bracing.

a. <u>Materials</u>

The most common piling material is generally A-36 steel, which has been driven in lengths of 160 ft or longer.

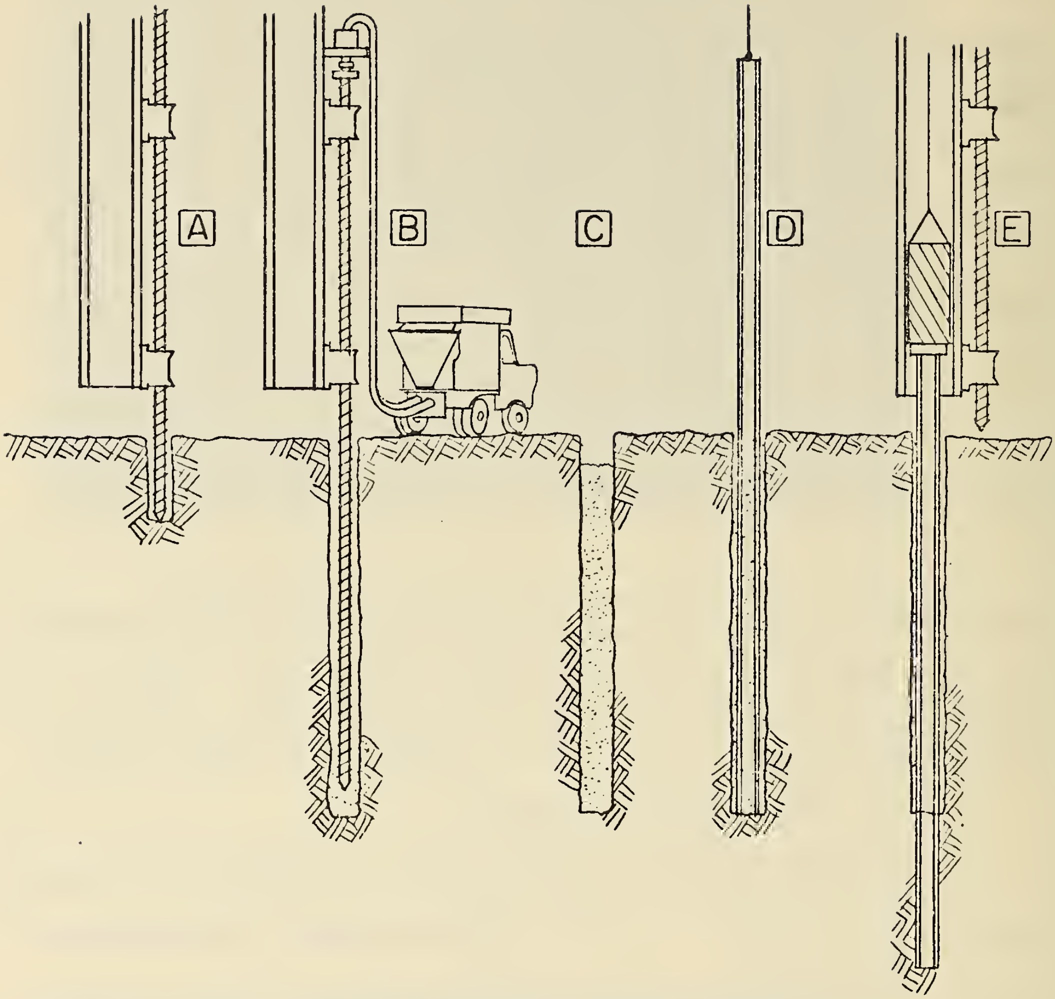
Concrete-filled steel-cylinder piles have also been used as soldier beams. Recently 34-inch-diameter steel-cylinder piles were used in Canada, with wood lagging between them. Special angle-brackets were welded to the piles to support the timber lagging.

b. <u>Construction Procedure</u>

Soldier beams are, traditionally, driven by conventional pile-driving equipment. In recent years, however, the noise and vibration problems have forced the development of other installation methods.

Some method of pre-boring the holes is generally used. One pre-boring technique used in San Francisco was to auger-drill the hole to the required depth; pump grout through a hollow-stem auger, filling the hole as the auger was withdrawn, computing the grout level to allow for the displacement caused by the H-beam; and lower the soldier H-beam into the grouted hole and drive it to the required depth.

Other methods are used, depending on the soil conditions and the groundwater level. The hole may be cased, if necessary, and



[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

FIGURE 21. INSTALLATION STEPS FOR GROUT SOLDIER-BEAM

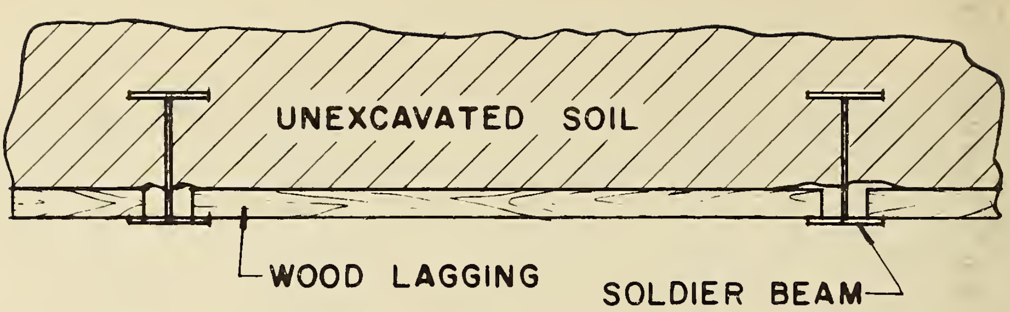
filled with slurry as the casing is withdrawn. A belled bottom may be installed by drilling machine to give a more positive anchorage for the pile base.

After setting the soldier beam in place, the last few feet are commonly driven, to achieve proper seating. Where this is not permitted, the bottom (straight-shaft or belled) is filled with concrete having a 28-day strength of 3,000 psi or more. This strength concrete is also used to fill the hole up to the bottom of the excavation. A lean concrete fill is used to fill the remainder of the cavity above the bottom of the excavation. This is necessary, since the flanges are normally exposed to receive the wood lagging as the excavation progresses.

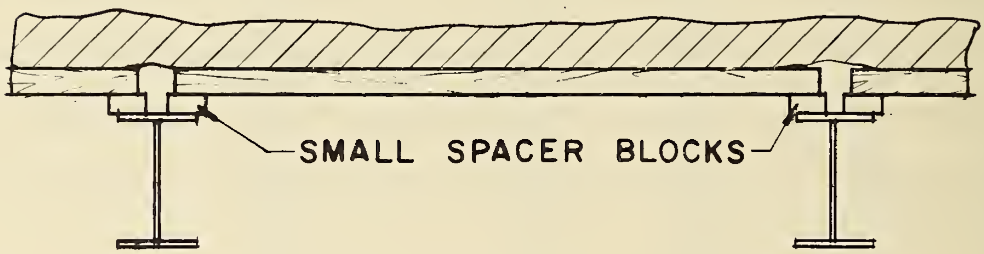
There are several methods of installing the wood lagging, three of which are outlined below and are shown in Figure 22.

1) The lagging may be wedged against the inside flanges of the soldier piles, as shown in Figures 17 and 20. This method is the most commonly used. This is a hand-operation in which the soil is cleared away to make room for the lagging. The variation in soldier-pile spacing and alignment often means that the lagging must be cut to fit. This is the biggest drawback to using concrete lagging. In clays, the earth pressure is transferred directly through the clay to the H-piles, with very little pressure being carried by the lagging.

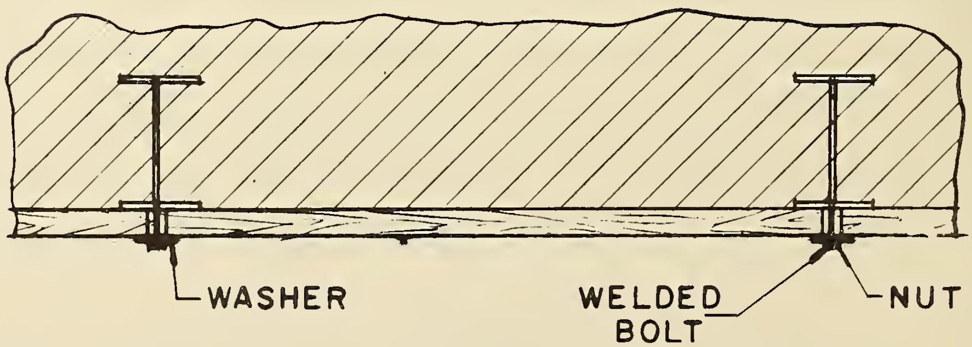
2) The lagging may be set behind the outside (soilside) flanges of the soldier piles, and wedged in place with spacer blocks. By this means the soldier pile may be incorporated into the permanent



LAGGING WEDGED AGAINST INSIDE FLANGES



LAGGING SET BEHIND OUTSIDE FLANGES



CONTACT LAGGING

FIGURE 22. METHODS OF INSTALLING LAGGING AT SOLDIER PILES

reinforcing of the structural wall. The settlement near an excavation made by this procedure may be considerably greater than by the other methods given. When the heavily-loaded soil behind the soldier pile is removed the soil mass may move inward, with a corresponding loss of ground. Space between the soil and lagging may permit additional loss of ground.

3) Contact lagging installed by welding studs to the face of the pile and bolting the lagging in place permits a tight fit because the surface of the exposed soil may be trimmed accurately. Installing the welded studs in accurate position also permits a possible use of concrete or steel lagging. This method is probably the most preferrable since fewer void spaces are created, but the cost is generally greater.

4) Grout or mortar is often installed behind the lagging to obtain a tighter fit and more nearly fill the voids.

Pile holes are generally hand-excavated to just below the utilities level (about 6 ft) because of the danger of driving piles through underground utilities whose location is either unknown or unsuspected.

Soldier piles may be either left in place or removed when the permanent structure has been completed. If left in place, they are generally cut off from 5 to 8 ft below the surface. Timber lagging is almost always left in place except for the top 5 to 8 ft, which is removed.

c) <u>Evaluation</u>

Soldier beams and lagging are probably the best known, the most adaptable, and the most proven ground-wall-support method in use throughout the world. While generally used in the United States, the many disadvantages should be recognized.

There is danger of a partial collapse of the hole, causing loss of ground if the pile holes are prebored. Also while the pile may be put into a partially-collapsed hole, a cast-in-place concrete plug which is counted on to anchor the base of the pile firmly in place may end up at mid-depth of the pile. This will probably not be discovered until excavation reaches the mid-depth where penetration of the concrete was stopped.

The contractor's tendency is generally to overdig when excavation reaches the bottom of the base slab, which may seriously reduce the penetration length and stability of the pile. Support-system designs should always allow a minimum of one foot for over-digging.

Improper alignment may occur whether the piles are driven or set in prebored holes. The tolerances allowed in San Francisco were zero inches inside, 12 inches outside, and 6 inches longitudinally. A pile misdriven to the inside may encroach on the structure, necessitating repair, removal, or redesign. Further, when deep wide-flange sections are used in deep excavations, the piles have a tendency to twist when obstacles or resistances are encountered. A pile starting to twist is further affected when the struts are installed and jacked into position. When this occurs steps must be taken to stop the twist, or the struts could be in danger of moving and the lagging could be displaced. Beams welded across the twisting pile and connected to several adjacent piles are often effective in stopping twisting.

Ground-loss can happen in several ways. Ground movements may be caused by infill of the hole around a pile in an augered hole and behind timber lagging. For instance, an infill of the bottom 20 ft of a 40-ft-deep 3-ft-diameter hole will cause a 4-inch settlement in a 20-ft square area. Surface and groundwater flowing through the lagging can cause loss of fine material and corresponding ground movements. Boils caused by groundwater can be a problem since the pile provides no seal below the bottom of the excavation. If the soldier piles are removed for reuse, the void left by the pile may collapse before it can be backfilled.

In almost all instances, the lagging is left in place and is removed to only a few feet below street level. In most cases this is still well above the permanent water table, leaving much of the timber to rot out in five to ten years. Very little consideration seems to be given to this problem although it seems to be reasonably important in contributing to future ground movements.

d. Comparative Costs

Soldier beams and lagging are still probably the most economical method of supporting an excavation wall and, in general, the system is more economical if the piles can be pulled and reused.

Other methods, which come closer to holding the soil in position while the excavation proceeds, enable different approaches to be made to the problem of underpinning or holding adjacent buildings in place. Also, savings can be realized if a temporary support-wall can be incorporated into the permanent structure. Thus, when the entire system and the interaction of all the components are considered we feel that other methods will in many cases displace soldier beams and lagging.

2. Sheet-Pile Walls

Sheet piling generally refers to steel sections with interlocking edges which are driven vertically to form a relatively impervious wall. This system has been used to build cofferdams for many years.

a. Materials and Design Standards

Sheet piling is available in a number of different types of steels, all of which are covered by ASTM specifications. The normal material specifications are ASTM A328 and ASTM A572, Grades 42 through 55. Concrete sheet-piling has also been used, primarily in Florida.

The designs for sheet-pile walls are based on the same principles, as are used for other types of ground-wall support.

b. Construction Procedure

Sheet piling is driven with either an impact- or a vibration-type hammer and, when used for temporary support of an excavation, it is generally pulled and reused. It cannot be used in urban areas where utilities frequently cross the excavation; other methods must be used to support walls in these areas.

The presence of boulders or other obstructions makes driving sheet piling difficult, if not impossible. A unique way for driving sheet piling in hard soil was used for subway construction in Stockholm by A. B. Stabilator. A hard till overlay the irregular bedrock surface. The grade was below the rock surface and the excavation was made in a tiedback sheetpile cofferdam to rock and then by blasting in the open cut. The till was too hard to permit driving the sheet pile to rock without damage.

The construction procedure was to drive soldier piles to rock and then drive sheet piles as deep as possible. Excavation was then done as deep as the stability of the sheets permitted. The soldier piles were tied back at this elevation and a concrete wale was cast directly on the excavation bottom. (See Figure 23). Cardboard was inserted between the wale and the sheet pile before casting to reduce the friction. The excavation was then carried down to the tip of the sheet pile, and the sheets were driven deeper as the excavation reduced the driving resistance. Wales were cast at predetermined elevations, or as required by either the irregular bedrock surface or the driving resistance.

The strength of the sections also generally limits sheet piling to shallower excavations. The largest section modulus available is 46.8 cubic inches per linear foot of wall while rolled sections are available with a section modulus in excess of ten times this figure.

FIGURE 23. CONCRETE WALES CAST ON THE GROUND, directly on the excavation bottom. Friction-reducing cardboard is inserted between the concrete and the sheets. The procedure of driving sheets, excavating, tieing-back the soldier piles and casting the concrete waler is repeated until bedrock is reached.

Methods have been developed to overcome the above difficulties. Sheet-pile walls in Holland are driven, and the excavation is done under compressed air to eliminate most of the requirements for internal bracing and hold the wall in position. A similar procedure has been applied in Oslo to counteract bottom heave in soft clays and eliminate the need for dewatering. The procedure has been to drive sheet piles, excavate down to the roof elevation and cast the roof. The excavation has then been continued below the roof under excess air pressure. The method has proved to be very feasible for excess air pressures up to 1 atm, see Figure 15.

In Hamburg, construction of the city railway (S-Bahn) was done by drilling continuous overlapping bored-piles. The holes were then filled with a mixture of sand, cement, and thixotropic clay (bentonite) which remains plastic for about twelve days. Sheet-pile walls made up of eight individual pile sections were hydraulically pressed into this replaced soil in a fixed order of succession. The wall depth was 20 m, and progress was reported as 4 to 5 m/day. While the technique eliminated the problem of boulders, it is hard to see any advantage over continuing the bored-pile technique and building a cast-in-place concrete wall.

c. Evaluation

Sheet-pile walls are effective for cutting off groundwater and for supporting relatively shallow excavations outside of which some movement can be tolerated. Other methods are more suitable for difficult soil conditions such as very dense or hard ground, till,

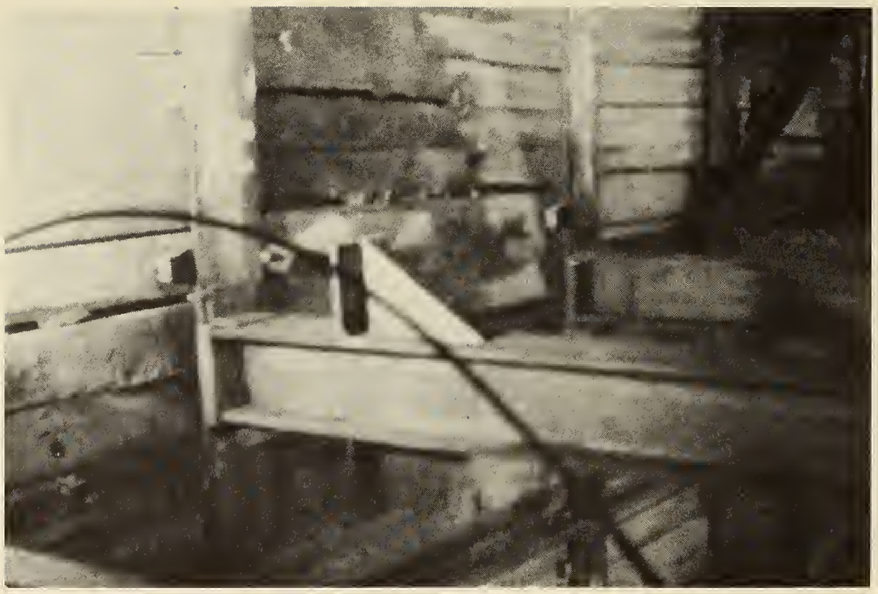


FIGURE 24. WASHINGTON, D. C. METRO, Cross-Lot Strut Jacking-Bracket

or boulders. Because of its inherent flexibility, sheet piling should not be used if ground movement must be very rigidly controlled. Extensive bracing of sheet-pile walls is needed if only limited ground movement outside the excavation can be tolerated. An example of a well-braced cofferdam with cross-lot struts is shown later in Figure 26.

3. Cross-Lot Struts

An excavation for cut-and-cover tunnel construction can be braced by struts extending across the excavation from one face to the other. (See Figures 24 through 27).

a. Materials and Design Standards

Steel members are generally used, and these may be either wide-flange shapes or tubular sections. Timber is unsuitable for a wide excavation. No examples using precast concrete have been found, although this would seem to be a possibility, especially if a strut could also serve as a permanent member incorporated into 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 visible. When precast concrete starts to fail, it can happen suddenly. Concrete walls cast in slurry trenches have been used as cross-diaphragms to brace excavation. (See Figure 27). The principle of using a cross-lot strut could be applied to any type of wall (i.e. soldier pile and lagging, slurry-trench, continuous bored-piles, etc.).

The struts should generally be preloaded to minimize ground movements. A common basis for design is to preload to 50 percent of

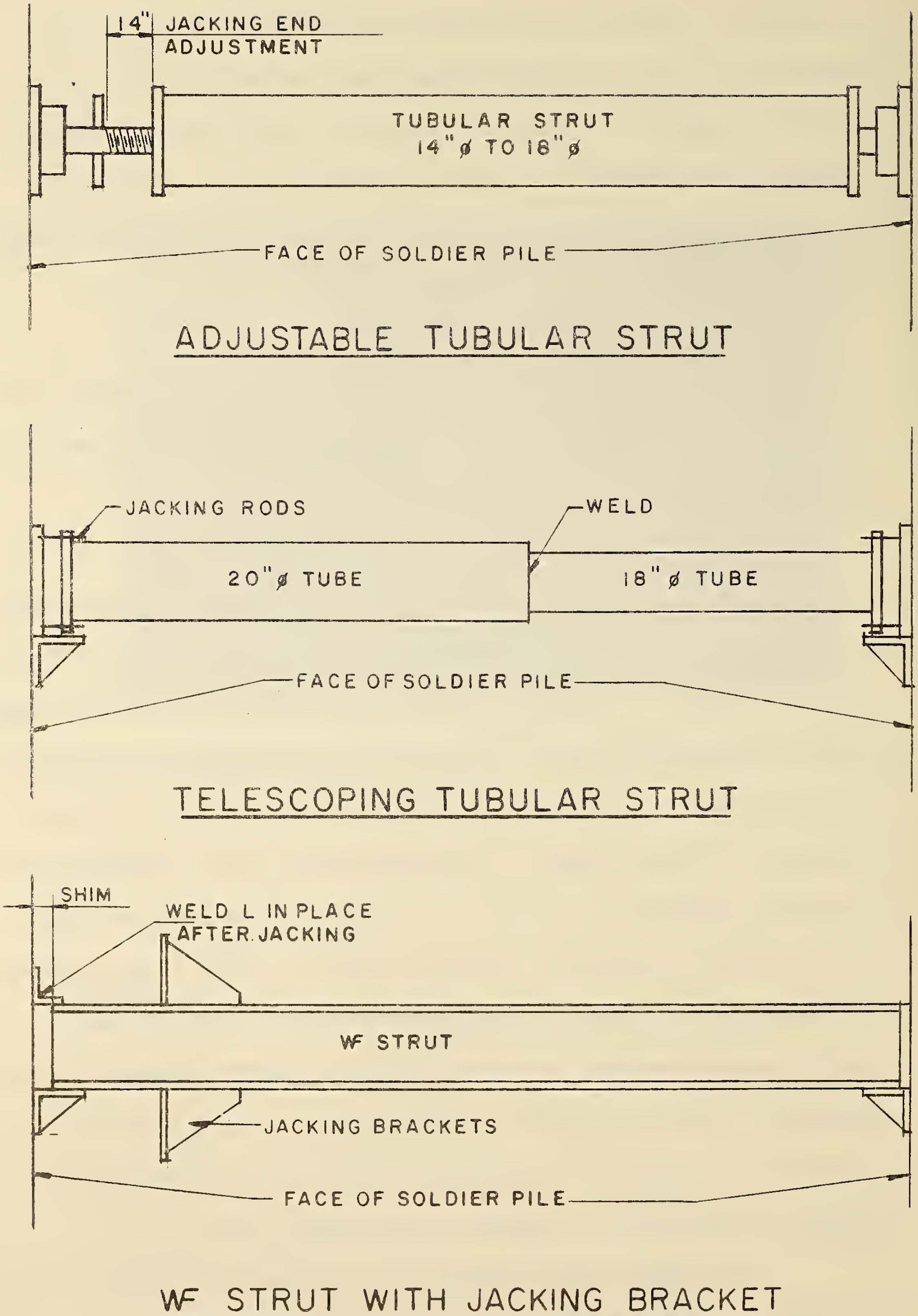


FIGURE 25. TYPES OF CROSS-LOT STRUTS

FIGURE 26. EXCAVATION FOR SUBWAY ENTRANCE AT OLAF KYRRES PLACE, Oslo, showing cross-lot braced sheet piles.

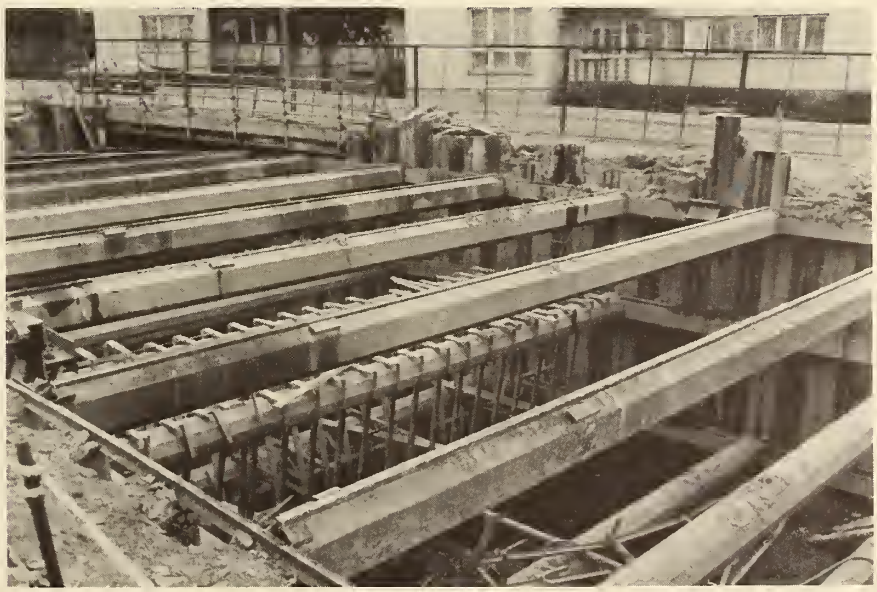


FIGURE 27. INSTRUMENTED STRUTS, OSLO. Four struts in each bracing level have vibrating strain gages. The struts from the second strut level are for building a transverse wall to increase the stability against bottom heave.

the design load, but sometimes as little as 25 percent of the design load is used. The struts should be preloaded to 100 percent of the design load, however, 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 percent or 40 percent of the design load is probably sufficient, however, 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. The amount of preloading should not cause an outward movement of more than 1-1/2 inches in either the top wale or the top of the wall.

b. Construction Procedure

Many attempts to improve the technique of installing and preloading struts have been made. Struts at first were cut short enough to fit, and were shimmed in position. A later improvement involved placing a jacking strut above the actual strut position, and jacking until the specified preload was obtained, after which the strut was wedged and shimmed into position and the jacking strut was removed. This not only increased the strut handling and the positioning operation, but the preload decreased when the jacking strut was removed because of elastic shortening of the bracing strut. The method is not satisfactory and should not ordinarily be used.

Telescoping tubular sections were used in San Francisco. The strut was put in place with the telescoping provision allowing for variations in length, a field-weld was made with the strut properly positioned, and then the preload was applied by threaded jacking rods. The necessity for a field-weld is the only disadvantage with this system. A later variation replaced the telescoping sections with a tubular section having a fitting on one end and an adjustable screw jack on the other. The screw jack allowed for up to 14 inches variation in length and could also be used for preloading the strut. The only disadvantage seems to be the uncertainity of the amount of preload since any calibration depends on the properties of the screw jack.

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 method works well, eliminates elastic shortening of the strut, and the preload force can be accurately evaluated. Another consideration when jacking struts into position is to shield the strut from direct exposure to sunlight. As an example, the change in length of a 40-ft-long 14-inch-round tubular strut loaded to 100^k is about the same as the change in length of the same strut caused by a 30^oF temperature change.

Either full- or partial-length wales may be used to increase the strut spacing. The wales can be eliminated if the strut can be placed at the same centers as the soldier piles; however, it is desirable to keep the excavation area as open as possible.

c. Evaluation

Tubular struts have definite advantages over wide-flange members--the sections are uniform about both axes and thus can allow wider strut-spacing. The tubular sections are readily adaptable to hanging forms and utilities and, from a safety standpoint, the round section discourages workmen from walking on them. There is also less likelihood of any construction equipment engaging the strut and either damaging or displacing it.

The correct placement of the top strut and waler is always very important to arrest the potential initial ground movement in areas where settlement may be a problem. The load of the different strut elevations will change as the excavation proceeds and the lower struts will be highly stressed, especially in soft clays.

The top struts and wales are the most important in controlling ground movements if the excavation is in soft clay.

Handling and putting long struts in place from the street level is a major consideration in decked-over construction.

4. Tie-Backs

Rock and earth anchorages for either temporarily or permanently supporting vertical excavations or tunnel approach-walls are increasingly used both in the United States and abroad. An unobstructed construction site is the primary advantage.

a. Materials and Design

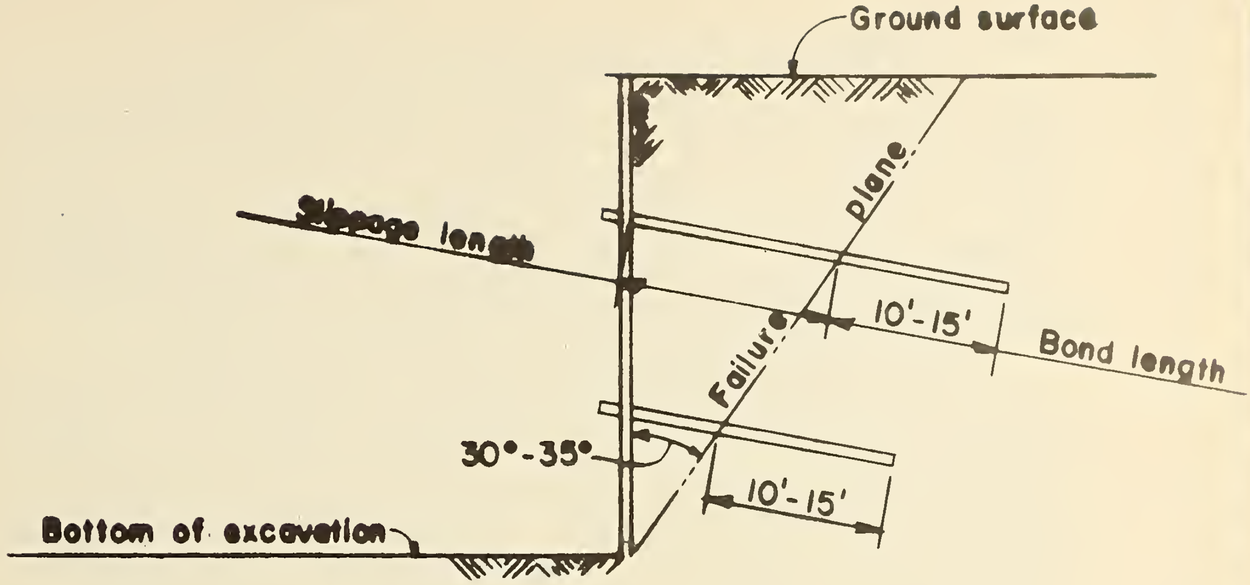
Grouted prestressed rods and cables are generally used as anchorages, although tension soldier-piles driven on a batter have also been used as for the Inner-Loop Tunnel in Washington, D.C. (See Figure 28). The location of the anchorage zone is dependent upon the soil or rock properties. Anchorage beyond a plane 30°- 35° from the

FIGURE 28. WASHINGTON, D. C., Soldier Beams Tied-Back with H-piles secure deep excavation. (From Engineering News-Record, October 19, 1967, p. 47, by permission.)

vertical is generally considered as safe. A 15-ft minimum grouted length is normally used.

From 50^k to 100^k capacities are used in the United States, and up to 200^k or more are used in Europe. The downward angle of the tie-backs should generally be kept as small as possible since the vertical component of the tie-back load is transmitted to the wall and may cause settlement. The length of tie-back required to pass through the failure zone increases as the angle approaches horizontal. Anchors are normally stressed to a proof-load, and then backed off to a working load. The proof-loading should be made on groups of at least three tie-backs to assure the anchors are not overstressing the soil or rock because of overlapping stress fields or unfavorable weakness planes. If an anchor fails and is replaced, the embedment may be increased until the loading is accepted. The possibility that an individual anchor may slip under long-term loading conditions should be considered in the design. Some design practices allow a 30-percent overstress for this case.

Tie-backs are used in both granular and cohesive soils, but the designs and installations vary considerably. In granular soils a small hole, generally about 3 inches in diameter is drilled and a pressure bulb is formed beyond the failure plane by pumping in cement and mortar grout at about 150 psi. The pressure bulb is formed around the bonded length which is generally 10 to 15 ft, using about five bags of grout per anchor. The length of tie-back which passes through the failure plane is greased or wrapped to prevent bond with the surrounding soil. (See Figure 29).



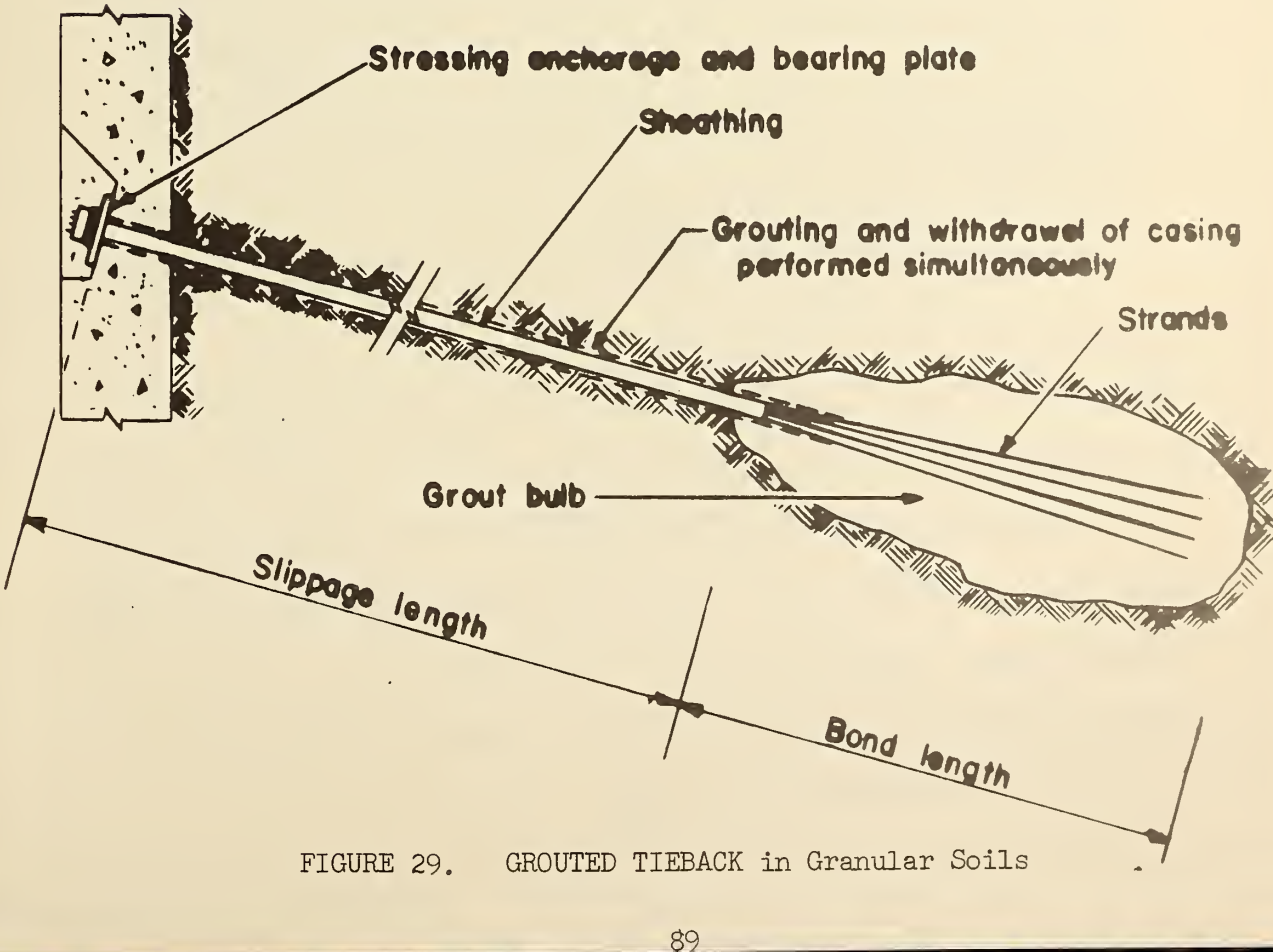


FIGURE 29. GROUTED TIEBACK in Granular Soils

In cohesive soils the anchor is formed by bond-stress between the cement grout and the soil. A 12-inch-diameter hole is augered to the proper length and the length beyond the failure plane (generally 20 to 25 ft) is filled (not pressure grouted) with cement grout. Sand is then used to fill the remainder of the hole.

For both types of soil only about 4 ft of anchor length is required to develop the tendon. The remaining length is required to develop the force between the grout and the soil. The installed cost for both types of anchors is from $9 to $15/lin ft of which $5 to $6/ft represents the drilling cost. Another means of estimating tie-back cost is an average price per tie which takes into account both long ties near the top of the excavation and shorter ties near the bottom. For a large number of ties (over 1,000) the average cost is about $500 per tie. For a small number (20 to 30) the price is about $1,000 per tie. For over 2,000 ties the price could drop to around $400 per tie.

b. <u>Construction Procedure</u>

Holes are drilled by various methods, some of which allow inserting the tie-rod and grouting the hole in one operation. Other methods use casings or slurry to keep the holes from collapsing. Soletanche uses an inflatable packer which is left in place, permitting regrouting of the anchor zone at any time. All the methods seek to grout from the anchor-end of the hole and to allow slippage within the failure plane. Temporary tie-backs installed in casing can be cut off above the grout and removed to reduce the amount of hardware left in the ground under adjacent properties.

c. Evaluation

Tie-backs, when properly installed, are probably the most appealing of the wall-bracing methods. The excavation area is unobstructed, and there is no danger of construction equipment moving or damaging a strut.

The disadvantages are the increase in cost over other methods and the necessity for obtaining easements under adjacent property. An easement obtained for tie-backs should recite and take into account the fact that the anchor material will remain in the ground permanently. However, the method may not be more costly when the various ease-of-construction factors are weighed. Research is needed about the long-term behavior of tie-backs, especially tie-backs anchored in unconsolidated soil. The main problem areas are the possibility of creep between soil and grout, and the corrosion of the steel rods or cables.

5. Rakers

Rakers are inclined struts used as internal bracing for a vertical excavation. (See Figure 30). Anchor blocks to take the thrust are cast in the floor of the excavation. Their use has generally been limited to building excavations where cross-lot struts are impractical, although other methods such as tie-backs are now more commonly acceptable. The drawback to the method is the danger that construction equipment or workers may dislodge the members since the working area is crowded with rakers.

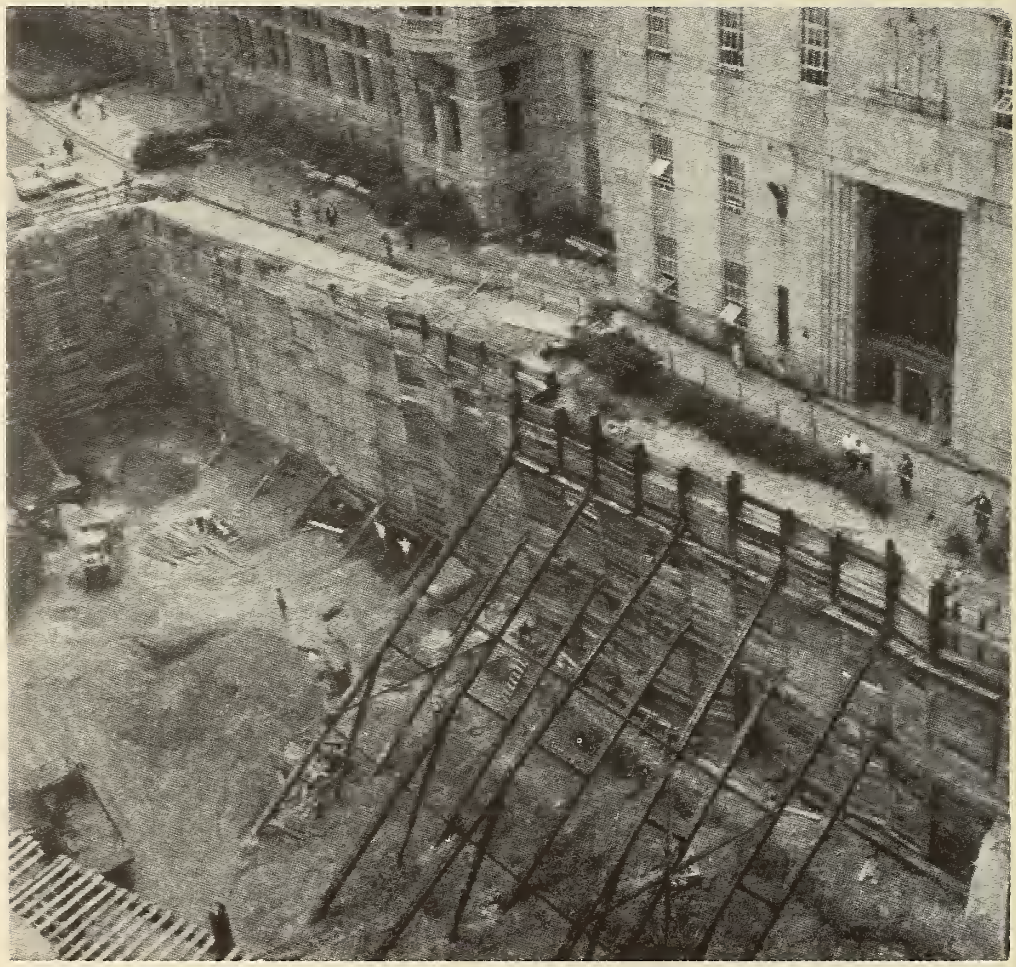


FIGURE 30. TWO CENTER PLAZA, BOSTON, MASSACHUSETTS. Rakers clutter the site with obstructions, while tiebacks simplify excavation. (By permission of Robert A. Lapointe.)

We do not see any significant use for rakers in cut-and-cover tunnel construction. Figure 31 shows some of the different types of excavation shoring methods.

B. SHOTCRETE

Shotcrete is a very dry concrete that is applied with pneumatic equipment. The general concept has been used in rock tunnels for several years, but there are increasing examples of its use for other types of soils.

1. Usage

The basic procedure is to line the exposed walls with a skin of shotcrete immediately after excavation, install wire mesh, drill and grout the tie-backs, and apply a finish layer of shotcrete to stabilize the wall.

Shotcrete has been used in Austrian and German tunnels, and loose gravel and even sand are claimed to have been controlled in this way. The only exceptions are highly plastic clay and quicksand.

Shotcrete was used experimentally in Washington, D.C., to stabilize 26-ft-high vertical cuts in some areas of the south ventilation facility during construction of part of the center leg of the Mall Tunnel inner loop. (See Figure 32). No buildings were immediately adjacent to the excavation.

2. Materials and Shotcrete Systems

Shotcrete may be applied by both wet and dry processes. The operational characteristics of the two processes are given below.

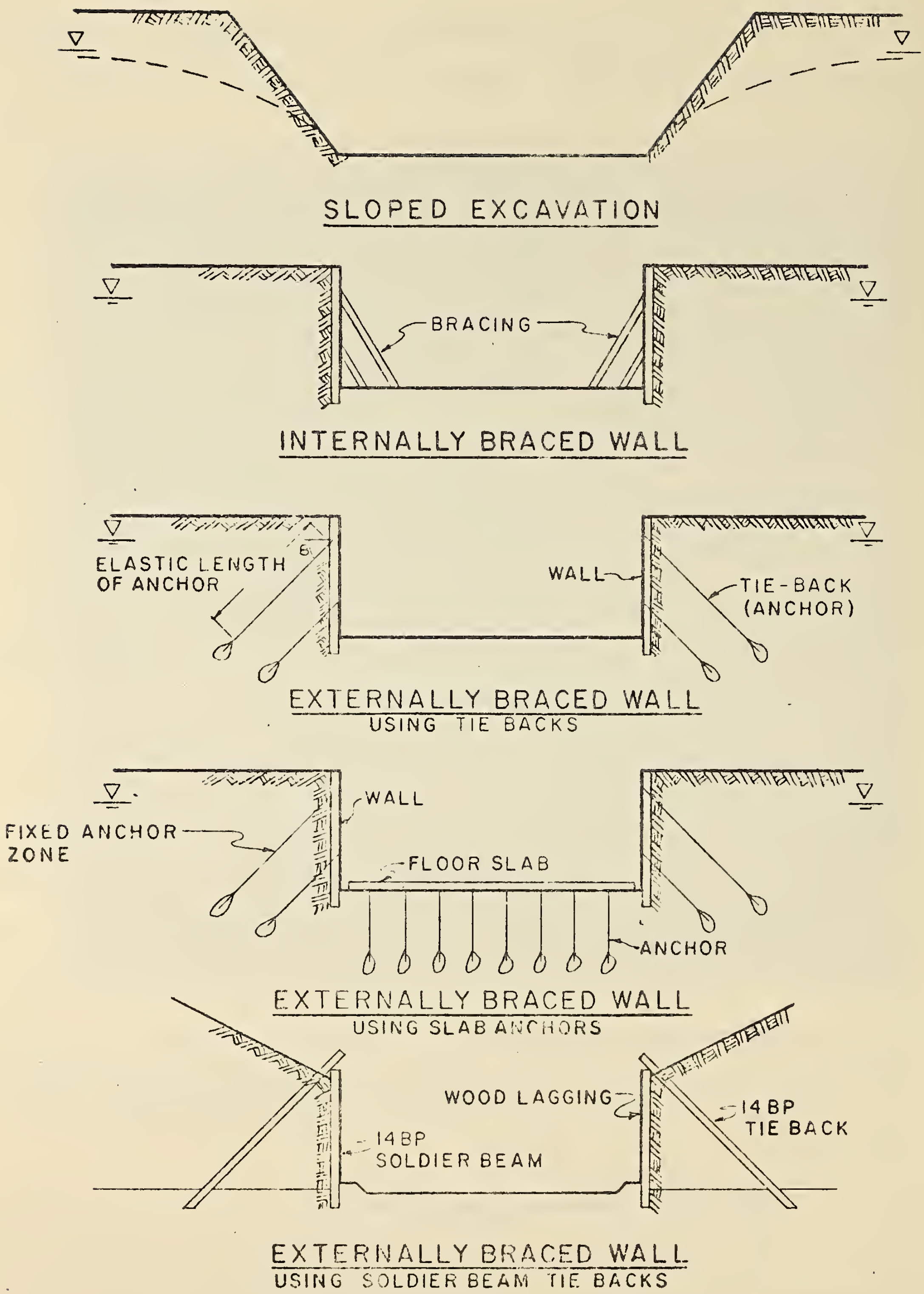
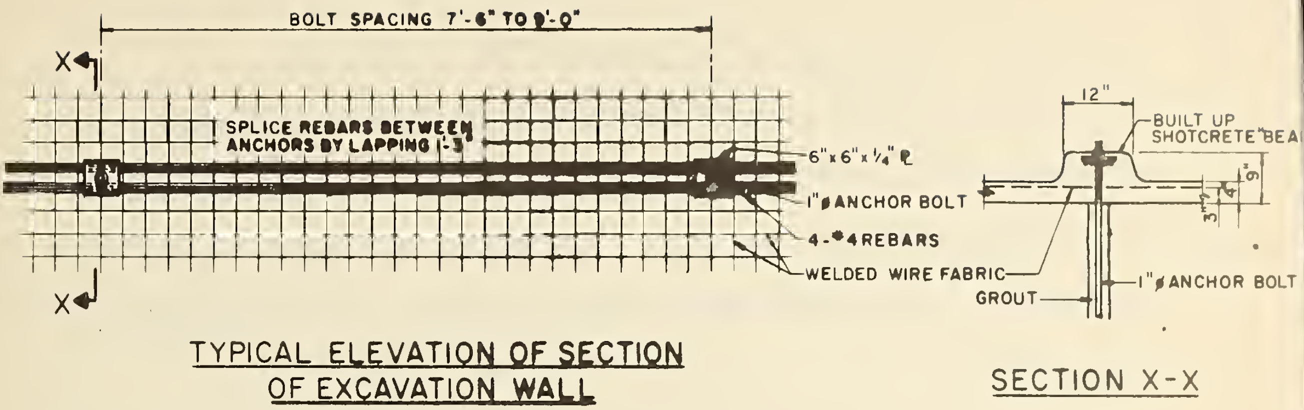
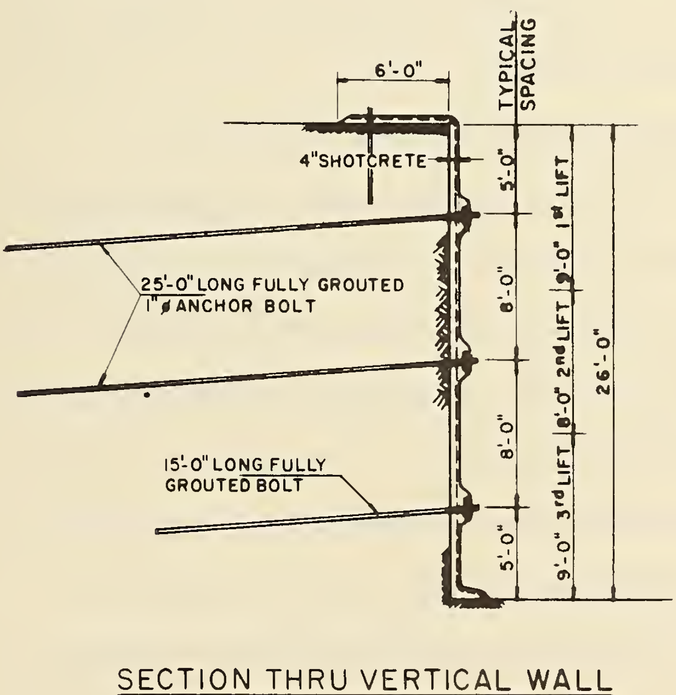


FIGURE 31. EXCAVATION SHORING METHODS





CONSTRUCTION PROCEDURE

1. *Excavate First Lift* to a depth of 9 Ft. following immediately with a 2" coat of Shotcrete. Apply second 2" coat of Shotcrete together with a layer of welded wire fabric. Drill anchor holes. Install and grout bolts, making sure that all of the surface of the bolt is in intimate contact with grout throughout its entire length. Start build up of Shotcrete "Beam". At required depth of "Beam" (see section X-X) install rebars and bolt plates, washers and nuts. Finish blowing "Beam" to cover rebars and plates with Shotcrete.
   Excavate Second Lift to a depth of 17 Ft. leaving alternate 20 ft. pillars between 20 ft. excavated sections in the first stage. Apply Shotcrete fabric, bolts, rebars and plates as for the first lift for the second stage, excavate the center pillars and stabilize the walls as for the first stage. For the third stage, excavate the end pillars, and stabilize.
   Excavate Third Lift to prescribed final depth and stabilize as for second lift.

2. All Shotcrete to test 5000 p.s.i. in compression at 28 days. Full test will be made on every 8th bolt in each row to a min. of 20,000 lbs. Bolt grout to test 6000 p.s.i. at 24 hours.

FIGURE 32. SHOTCRETE WALL CONSTRUCTION. (By permission of Dolmage, Mason & Stewart, Ltd., Vancouver, B. C.)

a. Wet-Process Shotcrete

Wet-process shotcrete equipment consists of a chamber into which measured quantities of aggregate, cement, and water are introduced. These materials are mixed to produce a low-slump concrete which is then discharged pneumatically through a hose with a special nozzle.

The wet-process shotcrete uses a maximum 3/8-inch aggregate size, which has been found to be the largest practical size. Type III cement is used in a seven- or eight-sack mix. The average unconfined compressive strength of the resulting shotcrete is 4,000 psi at three days and 6,000 psi at fourteen days.

A wet-process shotcrete system has very limited ability to adhere to wet surfaces. Since the material is mixed with water just before discharge through the hose and nozzle, fast-acting accelerator must be added in liquid form at the nozzle. This method does not normally produce good results, probably because it is impossible to thoroughly mix the ingredients at the nozzle.

Rebound-loss tends to be excessive and can run as high as 50 percent. This process can generally place only 2 to 3 inches on a vertical surface at one time, since thicker layers tend to slough off.

Wet-process equipment can convey material horizontally through about 200 ft of hose and vertically about 50 ft. Typical productivity for a five-man crew is about 3 cu yd of material in place per hour.

b. Dry-Process Shotcrete

The dry-process equipment consists of a special material-valve through which a dry mixture of aggregate and cement is continuously introduced into an air stream. A dry accelerator is metered into the material by a small adjustable auger-type feeder. The material is conveyed through a hose and discharged through a special nozzle, at which point water of hydration is added.

The dry-process shotcrete work is done using a maximum 3/4-inch aggregate size. Type III cement is used, and the average unconfined compressive strength of the resulting shotcrete is 3,000 psi at seven days and 5,000 psi at 28 days. (See Figure 33).

The natural water-content of the sand-aggregate material should be between 2 and 5 percent, and the process becomes extremely dusty when the water content falls below about 2 percent. The presence of an accelerator and the consequent plugging of equipment makes the process inoperable if the water-content rises over about 5 percent. Adding water or drying the material will be necessary to stay within these limits in some situations.

A dry, granular-form accelerator produces a rapid initial set and this reduces the amount of material lost through rebound. The accelerator also produces a relatively high-early-strength/time relationship. This is desirable in bad ground conditions where ground-support capability must be developed quickly. High strength after a period of a few hours is also desirable.

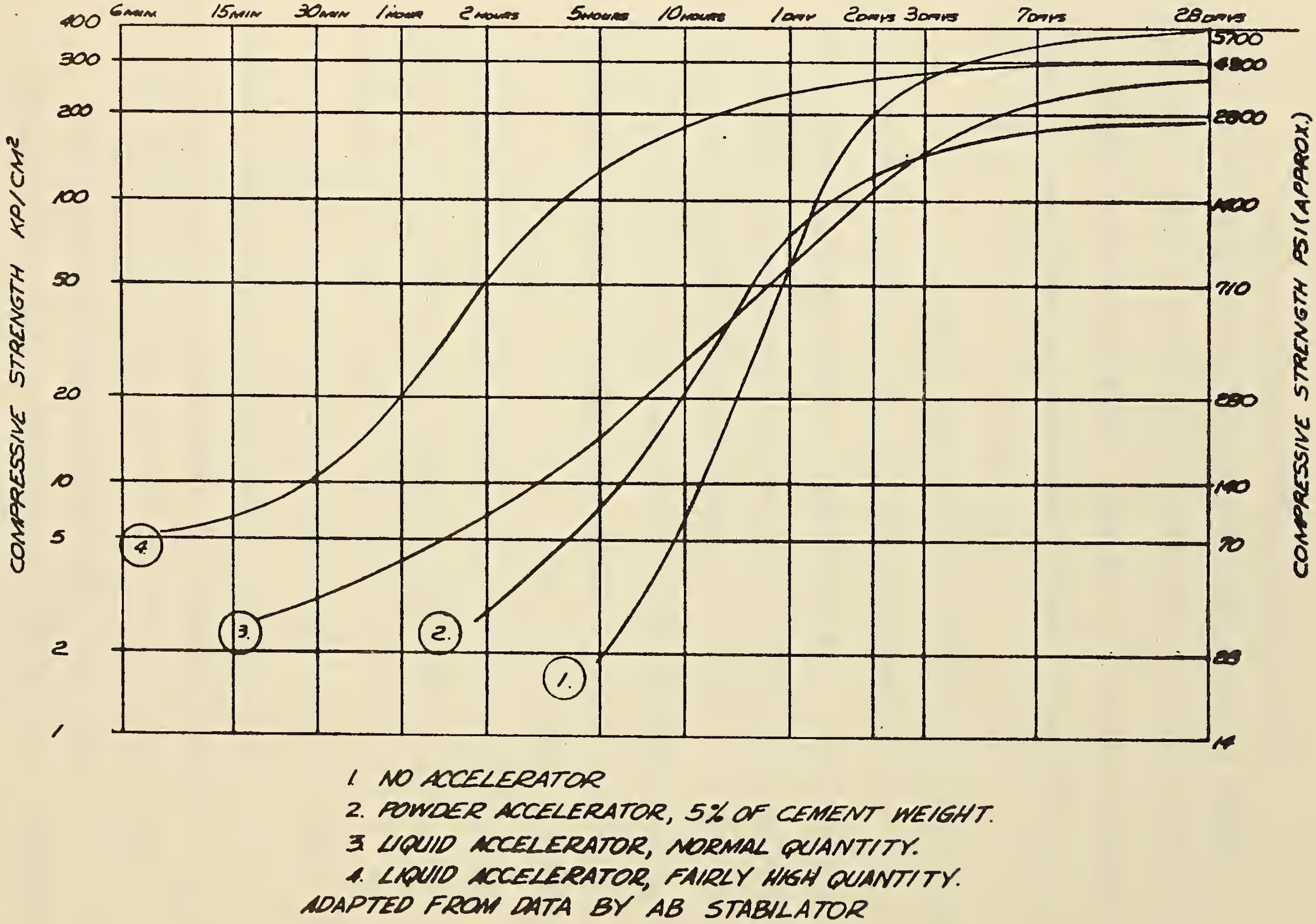


FIGURE 33. EFFECT OF ACCELERATORS ON SHOTCRETE/COMPRESSIVE STRENGTH.
(By permission of S. Backstrom of A. B. Stabilator, Bromma, Sweden.)

Sodium aluminate is the principal active ingredient in the accelerators now on the market. The composition of both the accelerator and the cement have a major influence on the set-times achieved. An initial set-time of 90 seconds and a final set-time of 12 minutes is generally considered desirable.

The accelerator and the cement are the two most expensive items in shotcreting. Careful evaluation of the possible combinations of these items is important in determining the pair that will produce the most economical result consistent with the specifications and the overall job conditions.

The dry-process equipment can apply at least 6 inches of material in one pass on vertical surfaces. When proper settings and nozzle techniques are used, there appears to be no tendency for the material to slough off. The average rebound loss of about 25 percent is considerably lower than with the wet process. As a general rule 1.5 cu yd of material through the machine will produce 1.0 cu yd in place. Typical productivity is about 5 cu yd/hr with an in-place cost from $70 to $90 per cu yd. Material can be conveyed up to 250 ft horizontally, and no problems have been encountered that can be attributed to excessive conveying distances.

c. Comparison of Wet and Dry Systems

The dry system is presently the most effective because of the ability to more accurately control the material proportioning and mixing, and because of the availability of accelerators that can be used with the dry process. This provides the ability to reduce rebound loss, achieve high-early-strength and handle difficult water conditions. These factors, in turn, result in higher productivity and substantially lower final in-place costs.

3. Evaluation

Shotcrete appears to be a promising area for additional research. Its greater adaptability to mechanization and the fact it is a continuous rather than a cyclical operation are its most attractive advantages.

The concept of excavating and immediately installing shotcrete as the ground-wall support in practically one continuous operation appears feasible. Such a system will probably be limited by the proximity and size of the existing buildings, and special attention will also be required to control both the surface water and the loadings near the excavation. In certain areas the strength of the shotcrete wall can be increased by using shotcrete in conjunction with soil injection. If this entire system can be constructed under a movable deck as discussed in Chapter VII, the total construction system will approach a continuous operation. We recommend that additional research be done in this area.

C. FREEZING

Freezing is normally considered for use where difficult soil conditions exist and may be used in conjunction with other methods of ground-wall support. It is applicable across the entire range of soil particle sizes. The freeze wall can be designed as a gravity retaining-wall requiring no other steps and no further bracing to permit the excavation to proceed.

The basic principle underlying the freezing process is changing the water present in the soil into the solid from the liquid state. The non-cohesive, or plastic, soil/water mixture gradually changes into an increasingly solid ice wall as the temperature decreases, and becomes completely impervious. Excavation can proceed inside such an ice wall without danger to the men working there, and the ice wall imparts stability and watertightness to the excavation walls.

Compressive strength in the order of 100 tons psf (1400 psi) can be obtained in saturated sands when the temperature of the frozen mass is at $0^{o}F$. (See Figure 34). The strength of frozen clay is about half the strength of sand at the same temperature. However, the compressive strength is dependent on the strain-rate.

1. Materials and Equipment

Two basic freezing systems are commonly used:

a. Closed System

The freezing liquid is recirculated, and freezing may be by either a direct system in which the compressor fluid is used directly for circulation in the freeze pipes or by an indirect system in which the compressor fluid circulates through a heat-exchanger and either brine or glycol is circulated through the freeze pipes.

The most common closed system uses calcium or magnesium-chloride brine having a specific gravity of 1.25, circulated through steel pipes embedded in the soil. The brine temperature, at $+5^{o}$ to $-15^{o}F$ is well below the freezing point of water. A refrigeration

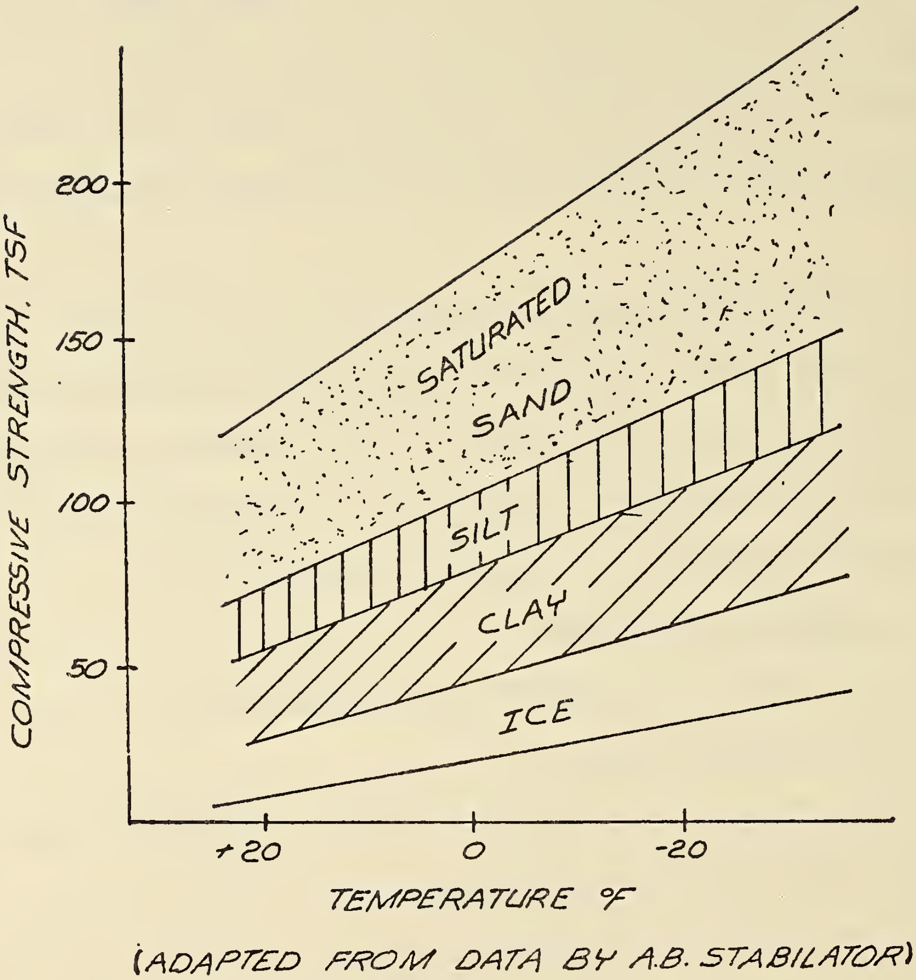


FIGURE 34. COMPRESSIVE STRENGTH VS TEMPERATURES FOR SAND, SILT, CLAY, AND ICE. (By permission of S. Backstrom of A. B. Stabilator, Bromma, Sweden.)

plant is required and the equipment consists of one or more reciprocating compressors, a heat exchanger, a condenser, a brine tank, freezing elements with valves to control the brine, and freeze pipes.

b. Open System

Liquid nitrogen or liquid carbon dioxide is used as the freezing material and the material vaporizes to a gas and is allowed to vent directly to the atmosphere. Liquid nitrogen is most commonly used and heat is extracted from the soil both by the heat of vaporization (at $-320^{\circ}F$) and by increase in temperature of the gas. Equipment includes special piping and valves (generally 9% nickel-steel) to avoid brittle fracture, and may require a storage tank unless the liquid nitrogen can be supplied directly from a tank truck.

2. Construction Procedure

The first step is exploring the area by drilling core holes and analyzing them for porosity, permeability, water content and salinity. Ground freezing is impossible in the presence of fast-moving water (the flow velocity should be less than 5 m/day).

The design has been done as a frozen gravity-wall and also proprietary designs using the tensile strength of the frozen mass have been used.

The second step is to sink the grid system of freeze pipes in the ground, with the pipes normally spaced 3-4 ft apart. The freeze pipes can be installed in two different ways. They can be sunk vertically or nearly vertically, or they can be placed horizontally through the shaft or pit walls in the direction of the tunnel alignment.

With the closed system (generally brine), each freeze pipe actually consists of two pipes, with a 2-inch-diameter brine feed-pipe inside a 3- to 4-inch diameter pipe which serves as the brine-return pipe to the heat exchanger. (See Figure 35). The brine is continuously circulated through the pipes at a temperature of about $-5^{\circ}F$. The surrounding soil temperature is measured regularly, and sounding-rod tests are used to measure the thickness of the ice walls. Excavation begins when sufficient ground is frozen. Brine circulation continues until construction is complete, after which the freeze pipes are removed and reused in another area. Sand is used to fill the cavities left after extracting the pipes to avoid settlement. The ground-freezing process requires between a few weeks and three months before excavation can start, according to the size of the project.

Open freezing using liquid nitrogen can be used for fast results, generally freezing the area in 72 hours or less. Obtaining large quantities of liquid nitrogen may be difficult and, as a result, may limit the size of the project where this type of freezing is possible. However, several years ago it was proposed to use liquid nitrogen to freeze a tunnel under the East River in New York City. Ventilation must be carefully considered when using liquid nitrogen as a person becomes "high" in about 30 seconds in a nitrogen-rich atmosphere and soon loses consciousness from lack of oxygen.

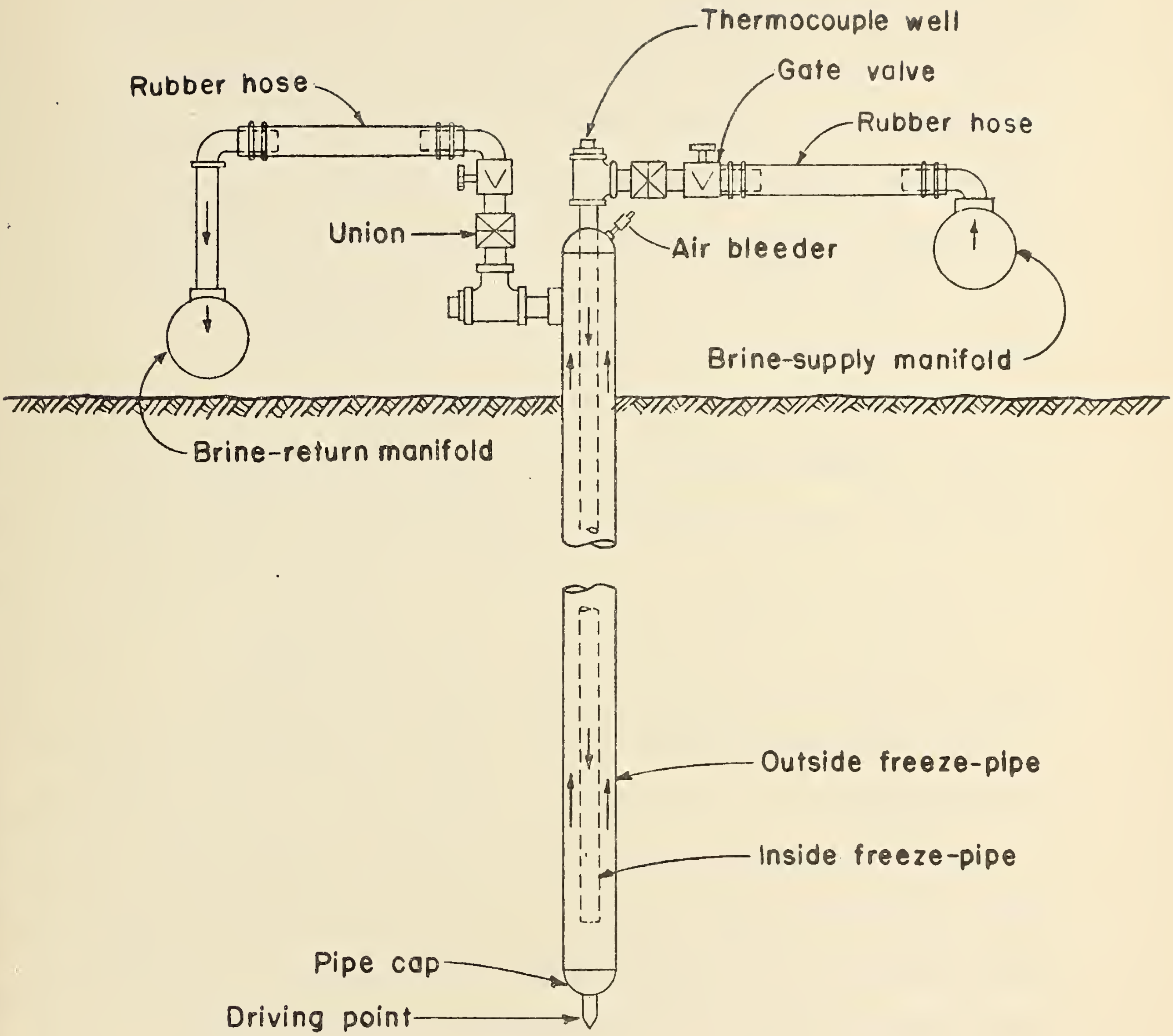


FIGURE 35. TYPICAL BRINE FREEZE-PIPE

3. Cost Data

a. Brine System

Cost for labor, materials, and equipment, not including operation and maintenance of plant is about $30 per cu yd of material frozen. This applies to ground-water flow velocities of under 1- to 2-m/day, and drilling and casing of freeze pipes 4 inches and under.

Other parameters that may be used in evaluating costs are:

Drilling and casing of holes - $4-$5 per ft

Refrigeration plant size - .02 ton of refrigeration per lin ft of freeze pipe in sand; range for other materials .01 to .03 t/lf

Surface distribution system	-	20% of total cost
Drilling and casing of holes	-	40% of total cost
Refrigeration plant	-	25% of total cost
Energy	-	15% of total cost

As an example, the cost for a large installation using three refrigeration plants was $630,000 which included $230,000 for material, $150,000 for labor, and $210,000 for equipment. In addition the weekly operation and maintenance cost of about $12,000 included $4,000 for power.

b. Liquid Nitrogen System

The material cost for freezing with liquid nitrogen (LN_2) is from $45 to $70/cu yd of material frozen, according to available data. In addition, the distribution piping and drilling the freeze holes costs about $25/cu yd of material frozen.

Factors entering into the cost are that LN_2 costs $40 to $80 per ton, or ($.14 to $.30/100 scf) depending on quantity required; 1 lb LN_2 will freeze 1.5 lb soil; a 6,000-gal storage tank (12-ft dia. x 28-ft high) costs about $28,000; standard tank truck carries 600,000 scf (or 22 tons) and costs about $300/day plus the driver; and the cost for piping and hardware is about twice the cost of drilling the freeze holes.

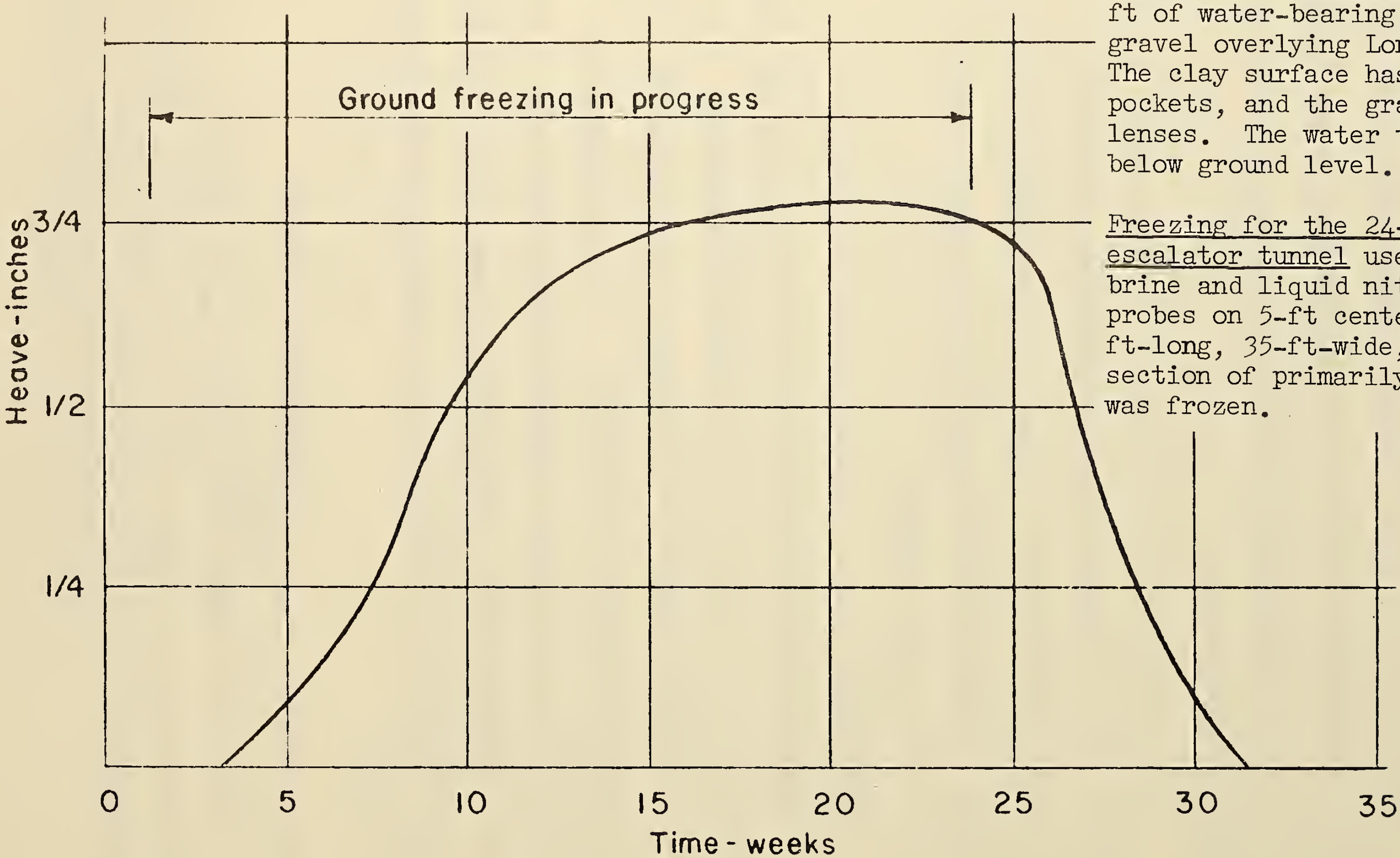
The advantage of this system lies in the elapsed time for freezing the soil. For example, only 33 hours were required in France for 13.5 tons of LN_2 to freeze 26 cu yd of material, and in Sweden only 72 hours were needed for 78 tons of LN_2 to freeze a 445-cu yd, 39-ft-long tunnel segment.

4. Conclusions

The main problems with the soil-freezing method are groundwater infiltration and volume change caused by frost-heave. (See Figure 36). The brine system is the cheapest ground-freezing method for stabilizing and supporting excavations, and is comparable in price with chemical injections. Liquid nitrogen freezes the ground much faster than conventional refrigerants because it circulates at a much lower temperature (-320°F), but it costs about 50% more than the brine method.

D. CHEMICAL INJECTION AND GROUTING

Chemical and cement grouting are used to stabilize and consolidate the soil and to control water. (See Figure 37). When consolidation grouting is used the need for underpinning may be minimized



<u>Subsurface conditions</u>: up to 19 ft of water-bearing dense sandy-gravel overlying London clay. The clay surface has silt-filled pockets, and the gravel has silt-lenses. The water table was 7 ft below ground level.

<u>Freezing for the 24-ft-diameter escalator tunnel</u> used ammonia brine and liquid nitrogen, and probes on 5-ft centers. A 75-ft-long, 35-ft-wide, 25-ft-deep section of primarily sandy gravel was frozen.

FIGURE 36. SURFACE MOVEMENT DURING GROUND FREEZING. (Adapted from <u>Proceedings of the Institution of Civil Engineers</u>, Supplement, Paper 7270 S, discussion, p. 340, 1970.)

FIGURE 37. PARIS, FRANCE, Chemical Injection. (By permission of Soletanche Entreprise, Paris.)

or eliminated, and the techniques may also be used in conjunction with other construction methods. Grouting to control ground water may also permit using other more-conventional construction methods.

1. <u>Application</u>

Cement grouts have had the longest history and have a wide range of applications, but their use is limited by the soil grain-sizes. Cement grouting has a consolidating effect in sands and other fine-grain materials because it forms lenses, columns, and bulbs, while a high degree of solidification is obtained in more-open soils such as gravels and porous rock formations. Cement grouts will generally only penetrate voids that are three times the diameter of the cement grain-size.

Chemical grouting is principally used for fine-grained granular soils into which the cement-based grouts cannot be effectively injected. (See Figures 38 - 43).

2. <u>Materials</u>

<u>Cement grouts</u>- material cost \$0.50 to \$1.30/cu ft

a.	Neat-cement grout:	cement and water - high shrinkage
b.	Cement grout with fillers:	mud-jacking and filling voids
c.	Cement grouts with admixtures:	soil consolidation, rock and masonry grouting

<u>Chemical grouts</u> - material cost \$1.50 to \$7.00/cu ft, low-viscosity and excellent control of gel-time.

a.	One-shot silicate:	Siroc,Carongel, Glyoxal viscosity 4 to 40 centipoises

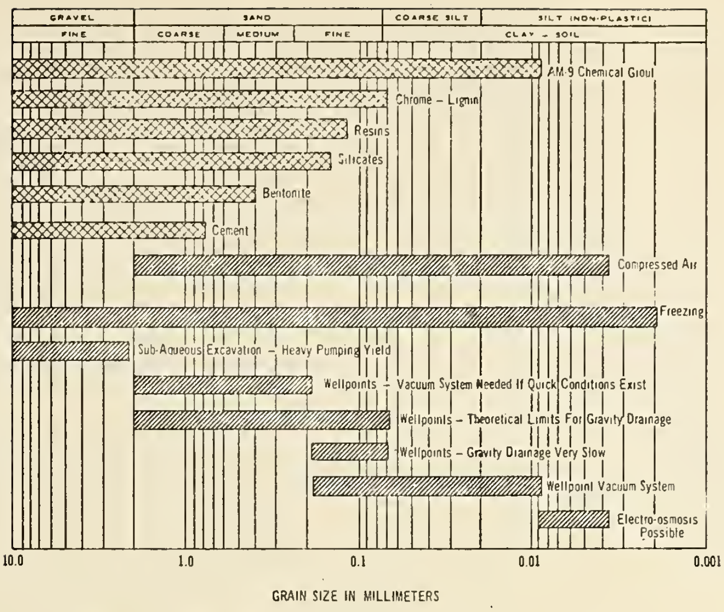


FIGURE 38. COMPARISON OF METHODS FOR STABILIZING AND DEWATERING SOILS. (By permission of American Cyanamid Company.)

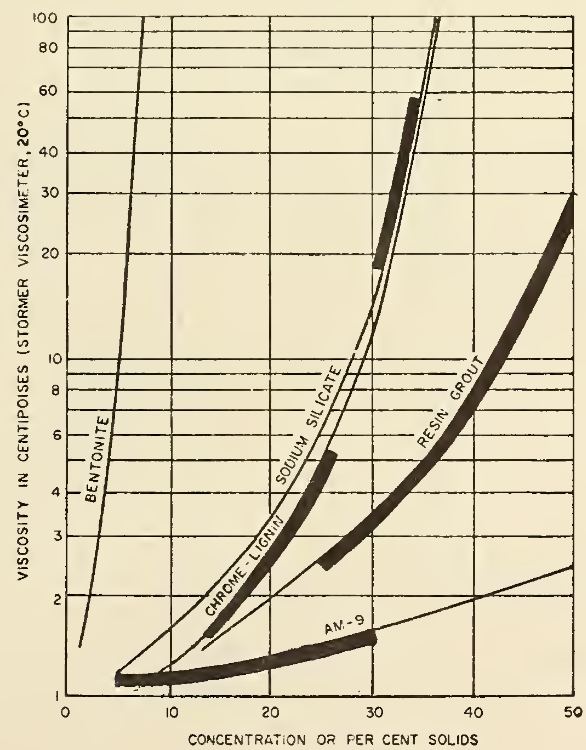


FIGURE 39. VISCOSITIES OF VARIOUS GROUTS. Heavy lines show the concentrations usually used in field work. (By permission of American Cyanamid Company.)

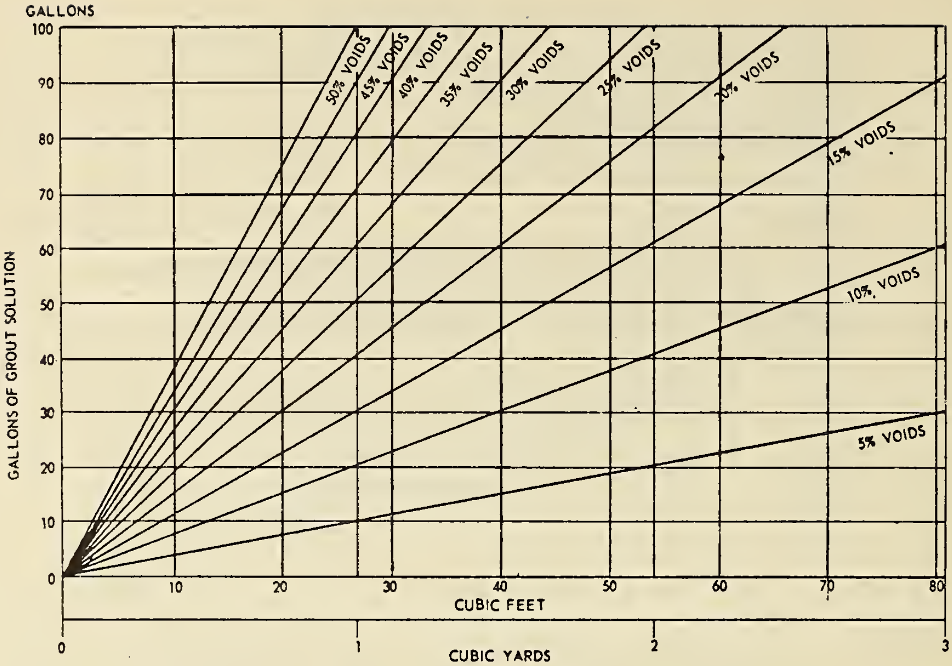


FIGURE 40. VOLUME STABILIZED IN SOIL OR ROCK RELATED TO GROUT VOLUME, AM-9

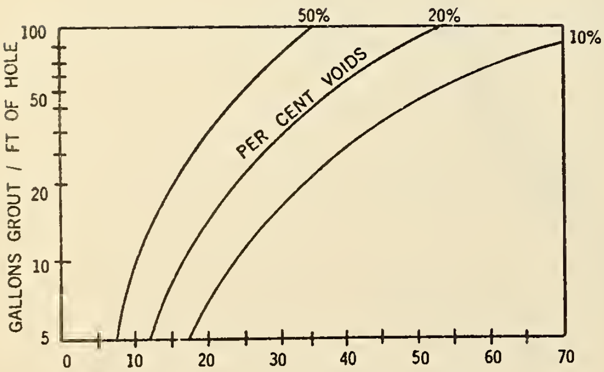


FIGURE 41. RADIUS OF GROUT SPREAD IN INCHES

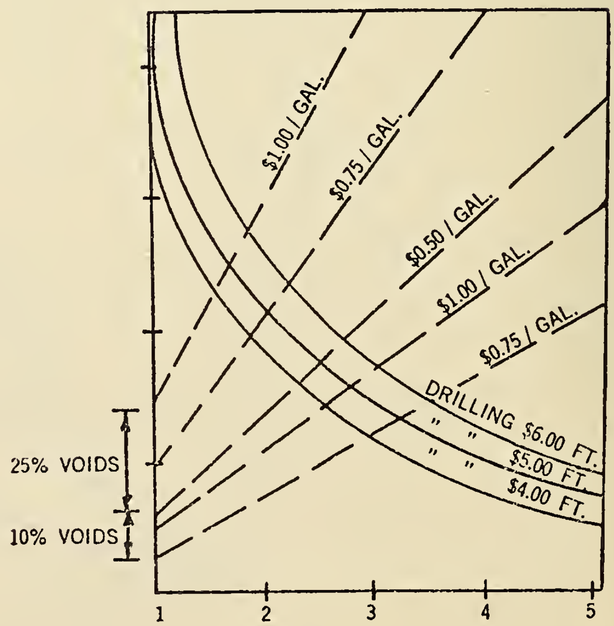


FIGURE 42. HOLE SPACING IN FEET

(Figures courtesy of American Cyanamid Company.)

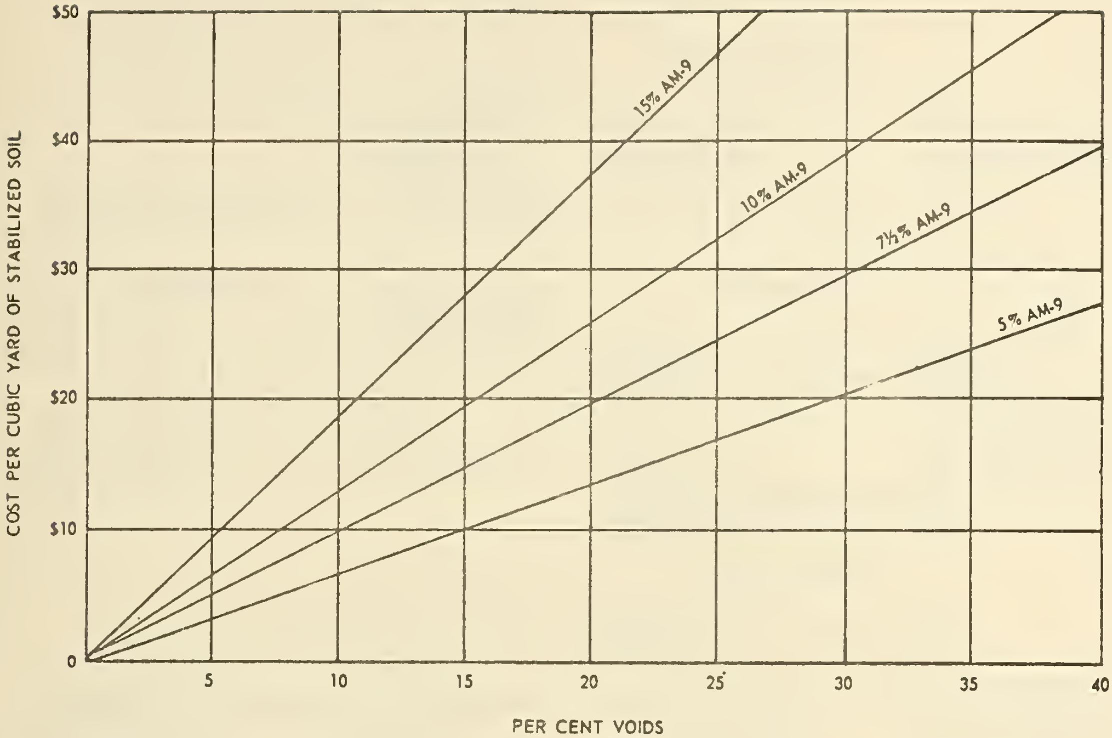


FIGURE 43. CHEMICAL COST PER CUBIC YARD OF SOIL STABILIZED WITH AM-9 GROUT. The unconfined compressive strength increases with the percent concentration of grout. (By permission of American Cyanamid Company.)

b.	Two-shot silicate:	Joosten process, instant-gel viscosity 30 to 50 centipoises
c.	Polyphenol:	Terranier - viscosity 2.8 to 75 centipoises
d.	Lignosulfonate:	Terrafirma - viscosity 2 to 20 centipoises
e.	Acrylamides:	AM-9 - high cost, viscosity 1.2 centipoises

The limits of grouting ability of some mixes are given in Table VI.

3. Construction Procedure

a. Driven Lance

This is the most widely used injection means for shallow depths. The lance is inserted into the ground by pneumatic hammer and extracted by jacking. The lance may be perforated over a short length at the end or it may have a loose-fitting point which is left in the ground. The normal penetration limit is 10-12 m.

b. Injections During Drilling

Injections may be made through the drill rod where the grouting depth is beyond a driven lance, or a casing may be used and injections made as the casing is withdrawn.

c. Sleeved Grout-tube

This is the most suitable and most widely used method for deep ground injections. A polyvinylchloride tube is installed in the bore hole and is surrounded by a clay/cement sleeve-grout to seal it into the ground. The tube wall has perforations at about 12-inch

TABLE VI GROUTING ABILITY LIMITS

	Type of Soils		
	Coarse sands and gravels	Medium-to-fine sands	Silty or clayey sands-silts
Grain size(mm)	0.5	0.02 - 0.5	0.02
Permeability(K)	10^{-3} m/s	10^{-3} - 10^{-5} m/s	10^{-5} m/s
Type of mix	suspension	colloid solution (gels)	pure solution (resins)
Consolidation or strength-grouting	cement (K 10^{-2} m/s)	double-shot silica-gels Joosten process (K 10^{-4} m/s)	aminoplastic phenoplastic
	aerated mix	single-shot silicate	
Impermeability grouting for water control	aerated mix bentonite clay-gel clay-cement	bentonite gel lignochromate soft silica-gel vulcanizable oils polyphenol	acrylamide aminoplastic phenoplastic

(From ASCE, Journal of the Construction Division, Paper 7382.)

centers which are covered with rubber sleeves. The grouting pipe is inserted into the tube to the desired depth, a double ram is actuated to seal off a segment, and grout is pumped in, forcing open the rubber sleeve. Thus the depth at which grout is injected can be accurately controlled (to one foot). The system can be installed in restricted quarters by threading the PVC tubes.

d. Split-Tube Method

Basically the same idea as the sleeved grout-tube, the opening for the grout is cut in the grouting rod by knives which are activated at selected depths.

e. Grouting with Valve Tube

Drilling and grouting are performed with the same tube. The drill assembly has an expendable ring bit attached to an inner extension drill rod. The bit is knocked off and the drill rod withdrawn when the full depth to be grouted has been reached. The tube has milled slots with an aperture. The soil is prevented from entering the tube through the apertures by leaf springs. The apertures can be spaced at variable intervals depending upon the actual conditions. The grouting is done using a double packer fitted to the valve spacing.

f. Monitoring and Testing

A monitoring and alarm system is generally used when grouting near buildings with the alarm sounding, for instance, when a building has raised 5 mm. Probe testing may be used to give some indication of the consolidation achieved, but a more accurate picture is probably obtained by recording and plotting the injection pressures and the grout quantities used.

4. Evaluation

Soil injection is becoming more widely used for both control of ground water and for consolidation or strength-grouting. In the United States the procedure has generally been used to overcome problems encountered in the field, while in Europe grouting is often planned as part of the initial design. A very extensive job of grouting was planned and carried out at the Auber Station in Paris, for example.

Grouting requires strict quality control to be successful. This means accurate batching of materials, control of pumping pressures, and measurement and recording of the amounts of grout injected. The pumping pressures are generally 1 to 2 psi per foot of distance from the exposed face or surface; however, it is generally desirable to pump at the highest pressure consistant with safety against formation uplift. When using chemical grouts the physical properties are affected by temperature, entrained air in the solution, metal in the soil, impurities in the mixing water, exposure of the solution to sunlight, the pressure of soluble salts, and the use of filler materials in suspension.

The method of payment used for grouting should be based on the estimated quantities with unit prices for more or less materials used. Items that should be included are as follows:

Mobilization and demobilization - lump sum

Drill and case holes	- per foot
Cement grout	- per cubic foot
Chemical grout, dry weight	- per pound
Placing grout	- per cubic foot
Connections to grout holes	- each

5. <u>Relative Costs</u>

Cement grouts range in cost from $13.50 to $35.00/cu yd and result in an in-place cost of material grouted of $46 to $77/cu yd.

Chemical grouts range in cost from $40 to $190/cu yd and an in-place cost of material grouted of $110 to $125/cu yd.

Drilling and casing grout holes is about $30/cu yd of material treated for a 3-ft grid to $15/cu yd for a 4-ft grid.

The following sample calculation for a cement grout and for AM-9 gives an example of the cost calculation.

<u>Cement grout</u> - cost of cement $.02/lb; 94 lb of cement produces 1.5 cu ft of grouted soil.

Cost = 94 x .02 x 1.0 x 27/1.5 - $34/cu yd

<u>Am-9 Chemical grout</u> - 10% solution; the material cost per 100 lbs of solution is $5.50, or $.44/gal. This is $90/cu yd of grout material, or about $95/cu yd of material grouted in-place.

Adding the drilling costs to the basic grouting cost:

Cement grout = $34 + $15 = $49/cu yd of soil in-place

AM-9 Chemical grout = $95 + $15 = $110/cu yd of soil in-place

The upper range of grouting cost is about the same as the cost of freezing with a brine system, and is less than the cost of freezing with an LN_2 system.

6. Conclusions

Chemical injections and grouting are valuable tools in cut-and-cover construction, both for controlling unexpected conditions in the field and for use in the initial design. Where soil injection is used, any easement required should recite and take into account the permanent change which may result in the soil.

This subject is discussed under the general headings of conventional methods, pneumatic conveying and hydraulic conveying.

Conventional methods are generally considered as using any of the diesel-powered earthmoving equipment, either alone or in conjunction with belt conveyors. Pneumatic conveying is essentially a closed system using air as the conveying medium while hydraulic conveying uses techniques similar to dredging operations.

A. CONVENTIONAL METHODS

1. Equipment

Most of the presently available conventional earthmoving equipment is used in cut-and-cover tunnel work. The type of equipment selected is based upon the types and total volume of material to be excavated, the types and sizes of hauling equipment, the load-supporting ability of the ground, the volume of excavation, the length of haul, and the urban or suburban character of the project area.

The equipment commonly used in open-cut construction where decking is not required may involve front-end loaders filling trucks which enter and leave by dirt ramps. This method is difficult to use if wall-bracing across the excavation is required since the bracing may severely limit the use of heavy equipment. Alternatively, a mechanical belt conveyor may be used to move the soil from the loader either directly to trucks or to a stockpile at surface level, see Figure 44. Another method is to use a crane at surface level to

FIGURE 44. BART, SAN FRANCISCO, Spoil Removal by Conveyor. (From Engineering News-Record, July 24, 1969, p. 30, by permission.)

remove the material from a stockpile in the cut made by either a bulldozer or a loader.

Open-cut construction presents additional problems if street decking is required. Hand-digging or a small backhoe must be used until the digging level is below the underground utility pipes and cables, see Figure 45. When headroom is available, crawler loaders can be used to load conveyors which move the material back to open areas where it can be trucked away.

2. Spoil-Material Disposition

Spoil disposition can be a problem in urban areas where large volumes of material may have to be hauled many miles. A number of options are available, however, since the material can be used as fill for a variety of construction projects in which the ground level must be raised. Besides stockpiling as future backfill if suitable, the material can be used for airports, highways, large industrial sites and reclaiming swampy areas; it might be used as aggregate for concrete if suitable.

3. Excavation Procedure

An example of the excavation procedure currently in use is San Francisco's BART project as given in the paragraphs below.

After a shallow excavation was made between the rows of soldier piles, 36-inch wide-flange deck girders and timbers were used to deck over the street, and wide-flange knee braces were welded to the girders and soldier piles 8 ft from their intersections to stabilize the deck and piles while excavation continued.

FIGURE 45. WASHINGTON, D. C. METRO, Exposing the Utilities

Excavation was done by working one station at a time. Earth work was handled with a 1-1/2-cu-yd Caterpillar 955 and a 2-1/2-cu-yd Caterpillar 977 tracklaying loader, supplemented with a backhoe digging a drainage trench. A Caterpillar D8 dozer pushed earth to the loaders.

Spoil was removed on a Cedarapids belt-conveyor system using up to five 40-ft-long, 36-inch-wide belts. The conveyors, fitted with deep-troughing rolls, traveled up to 450 feet per minute (fpm). Another belt at the surface loaded stockpiled material into dump trucks.

4. <u>Cost</u>

Excavation costs vary according to the composition of the material being removed, the ease or difficulty of handling and loading, the size of the trucks, and the haul distance. Excavation in earth varies between $10 and $25/cu yd, while rock varies from $25 to $40.

B. PNEUMATIC CONVEYING

Pneumatic conveyors handling fine dry materials have long been used in the cement, chemical, and food industries. The technique of handling broken shale for backfilling was developed in Europe where materials were conveyed several thousand feet before they were placed. Equipment is available today that will handle 3-inch-minus material at the rate of 300 tons per hour (tph). Excavation and backfilling are the main areas where pneumatic conveying, with its ability to convey horizontally, vertically, and around bends, shows promise of increasing the efficiency and productivity of the materials-handling systems.

1. System Description

A pneumatic conveying system consists of an air source connected to an airlock feeder that places the material into the pipeline, and a pipeline which transports the material to its point of discharge. The conveying air must be moving with sufficient velocity to keep the particles in suspension, or the material will settle out of the airstream and a plugged pipeline will result. Once the material is moving, the air supply must keep the material moving around any bends and vertical lifts, and overcome the frictional resistances of the pipe.

The higher the operating pressure of the pipeline, the smaller the diameter of the pipes required and the more economical the system. The system operating pressure increases with the pipeline length, the number of bends, the vertical distances to be lifted, and the frictional characteristics of the material being conveyed.

The particle velocity increases as the pipeline lengthens (80 percent of the maximum particle velocity is achieved in the first 40 ft of pipe). Large particles accelerate at a slower rate and, therefore, require a longer distance to reach conveying velocity. In backfilling, a minimum of 30 to 40 ft of pipeline should be used before the discharge point since compaction increases with the discharge velocity. At least 12 ft of pipe should follow a bend to let the material reaccelerate before it is discharged. A bend in the pipe may reduce the particle velocity by 30 percent.

When a pneumatic conveyor is used for excavation it is not necessary to move the material at the high velocities required for backfilling. The material velocity should be minimized to reduce the pipe wear-rate and the power consumption.

2. <u>Excavation</u>

The limits to moving granular material at present are 300 tph through 1,000 ft of pipeline, although recent work indicates the pipeline can be 3,000 to 4,000 ft long.

A pneumatic system could be installed just behind the excavation face for cut-and-cover. A section of telescopic pipe would allow the machine to move continuously as the excavated material is conveyed through the pipeline to either the infeed of a new system or to the surface. Oversize material would be passed through a small crusher before being fed into the pneumatic conveyor. A pneumatic conveyor would have the ability to ventilate and extract dust because the blower used has the ability to draw a vacuum to 15 inches of Hg.

The dimensions of the feed system are about 3- by 4- by 9-ft, with the pipeline size varying according to the desired system capacity. A 14-inch-diameter pipe is required to deliver 300 tph, for example. The system can be discharged either into a truck bin or a waste pile, and the ability to convey vertically to the surface and discharge into a truck bin is advantageous where surface space is difficult to obtain. The pipeline used in this type of system must be abrasive-resistant, and the pipe which has best suited the purpose is a two-layer pipe in which the inner layer is very hard (600 - 700 Brinnell), and the outer layer is of mild steel.

Pneumatic excavating equipment operating costs were reported as $2.10/cu yd in a recent study for an excavation system to move 200 tph through 5,200 ft of pipe.[7]

A pneumatic system can be easily automated and remotely controlled. The small space required for the pipe in an excavation results in more efficient operation.

The main disadvantages of the system are the large power requirements needed to produce the compressed air and operate the blowers. While the total power requirements of a conventional system are perhaps greater than those of a pneumatic system, it is distributed among several different types of equipment.

3. <u>Backfilling</u>

Backfilling was the main reason for introducing pneumatic conveyors below ground in coal mining.

Studies of backfilled material have indicated that high compaction can be achieved with pneumatic placement and the moisture content can be controlled within acceptable limits. The United States Bureau of Mines is presently analyzing this aspect.

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7. <u>Proceedings of the Second Symposium on Rapid Excavation</u>, Sacramento State College, October 16 and 17, 1969.

4. Evaluation

Pneumatic excavation and backfilling show promise of being successfully applied to cut-and-cover operations. The main advantages are that particle sizes of 3-inch-minus may be conveyed with only screening ahead of the machine required, and the system can be essentially automated. Pneumatic backfilling can also reach areas that may be impossible to reach by other means.

At present the high power requirements and air volume are the main disadvantages. There are also significant wear-rates on the machine components.

Additional research should be done to develop equipment and methods to apply this technique to cut-and-cover tunnel construction.

C. HYDRAULIC CONVEYING

Hydraulic pipelines for removing excavated material from tunnels have been used in both Holland and Japan. The basic techniques are similar to those used in dredging operations.

1. System Description

Most pipelines are constructed of steel pipes, and the power is derived from large slow-speed centrifugal pumps. The solids are broken into relatively small particles for long lines since the velocity required and achieved is in direct proportion to the particle size.

Dredging operations have proven the workability of pumping coarse solids. Material one-third the diameter of the pipe can be pumped several thousand feet. Velocities of about 20 fps are commonly used, and the solids concentration is normally maintained at around 20 percent by weight.

The factors which affect the pumping of solids are particle size and shape, particle gradation, the specific gravity of solids and liquids, the viscosity, and the gradient of the line. A minimum velocity is critical, and operation at less than this minimum results in a plugged pipeline.

The friction in the pipeline caused by particle drag in the pipe increases as the particle size increases. Particles of micron size are suspended in the liquid and can be transported at low velocities. Particles up to 1 mm or so remain in suspension under turbulent flow-conditions, while larger particles are moved by consecutive bounces along the bottom of the pipe. Slippage of the fluid past the particles depends on the difference in specific gravity between the liquid and the solids and the particle shape.

Small particles of clay- and silt-size increase the viscosity of the carrying liquid, thus increasing the carrying capacity and decreasing fluid slippage past the particles.

The operating costs greatly increase as the distance increases, but 2,000 to 4,000 ft is a practical range. For instance, a 12-inch pipeline, 4,000 ft long requires a 1,000 hp pump. The solids must be separated from the slurry at the discharge point, and desilting equipment will also be needed if the excavation is in clays, if most of the slurry is not to be wasted. Only a settling basin is required for large solids. Enough water must be returned to the excavation face to keep the solids concentration in the line below the design level of about 20 percent.

2. Evaluation

Hydraulic pipelines seem to have promise for limited use in cut-and-cover tunneling operations; however, there are several factors that must be considered.

a. The system works best as a continuous operation, and the pipeline must be flushed clear to prevent plugging whenever there is an interruption.

b. Large amounts of water are required, and a relatively large installation is required to separate the solids from the liquid.

c. Disposing of large quantities of slurry or partly-separated slurry can present a serious environmental problem.

VII STRUCTURAL CONTROL

This subject is discussed below under underpinning, street decking, and relocating and supporting existing utilities.

A. UNDERPINNING

Underpinning is a means of providing additional structural support for buildings which may be damaged by some aspect of tunneling operations. Underpinning for tunnel construction in urban areas has three purposes. These are to transfer the foundation load from buildings near the tunnel excavation to a level below the zone of potential soil movements; relieve the load from building foundations undercut by the tunnel excavation; and build support systems for structures to be constructed over the tunnel.

1. <u>General</u>

Underpinning buildings to be either undercut by or built over a tunnel presents very specific structural problems for each case. Prestressed concrete beams, steel girders, or articulated concrete arches have been used for this type of underpinning. The load can be carried down to the foundation elevation on individual supports, but the supports can also be incorporated into the final structure. Slurry-trench walls can be used as permanent underpinning, ground-support for the excavation, and finally become a part of the tunnel structures.

Supporting existing buildings close to the excavation is the most common underpinning work in connection with cut-and-cover construction. The need for underpinning is governed by the magnitude and distribution of soil movements outside the excavation in comparison with the tolerable movements of the structure within the zone of influence. The soil movements can be reduced by using a more-rigid ground-support system. A feasibility study should be made in developing the design approach, to determine whether a flexible weak ground-support system together with underpinning should be used, or whether a more-rigid system, eliminating or reducing the amount of underpinning, should be used.

Underpinning can be specified in the contract as either mandatory, or voluntary. In the first case the contractor is obliged to make the prescribed underpinning, while it may be done at the contractor's option in the second case. Zones can eventually be specified delineating areas near the excavation for mandatory underpinning, and for voluntary underpinning further out.

A thorough investigation of the structural integrity of the buildings that may be affected by the construction work is needed to determine the tolerable movements for each structure (see Table III). The permissible movement and the amount of temporary damage that can be accepted may not always be based upon tangible facts but rather upon individual personal subjective feelings. This is especially true of historical or monument buildings.

The analysis for determining how far away from the excavation the underpinning should extend and at what depth the new foundation should be seated is a part of the geotechnical analysis. The permissible differential settlement is generally most critical and is generally expressed as a gradient-not-to-be-exceeded.

The new foundation, installed by underpinning methods, should be designed to not only carry the load of the structure but also to resist the lateral forces developed because of soil-yield toward the excavation and because of drag forces that may develop from settlement of the soil behind the ground-support wall.

2. Underpinning Methods

Underpinning can either be made by installing new structural members or by improving the strength properties of the *in-situ* soil. Jacked piles are most commonly used in the United States, followed by augered piles. Pit walls, grouting, and freezing are also occasionally used. Improving the bearing capacity of the *in-situ* soil by grouting or freezing is described elsewhere in this report.

Underpinning by installing new structural foundation members can be divided into:

a. strengthening the existing structure so the load can be relieved from the foundation and transferred without damaging the existing structure;

b. relieving the load from the foundation, if necessary, to permit work below the old foundations;

c. installing the new foundation members; and

d. transferring the load to the new foundation.

Problems can occur when installing new structural elements if the building has a foundation with poor structural integrity and requires extensive strengthening. Boulders or other hard materials that must be penetrated to place the foundation below the zone of potential soil-movement may cause problems when jacking piles. Access to these work areas is generally limited, and the work has often to be done with limited headroom.

3. Monitoring

The structure should be carefully monitored during the underpinning procedure. Damage can be caused by loss of ground or settlement into excavations below the foundations, by heave caused by grouting, or by uplift to the structure from jacking the piles.

4. Underpinning Application

Underpinning should only be used if a feasibility study shows that other methods, such as a more-rigid ground-support system, are more costly. The reader should note that allowing a building to crack and repairing it when tunnel construction is finished may be much less costly than preventing the damage.

The underpinning approach must be evaluated for each individual structure. But, generally speaking, methods that do not require intensive work in limited space are to be preferred. Bored-pile walls and slurry-trench walls, installed flush with the outside face of the foundation are especially feasible if, in addition to supporting the structure, they can serve as ground-support and eventually become a part of the final structure. These types of walls can serve both as straight-wall underpinning (replacing pitwalls) or as column underpinning. Inclined bored-pile walls have been used in Europe. The piles for these walls are drilled at an angle under the structure, providing more space in the bottom of the tunnel excavation. The load transferred to underpinning elements placed outside or flush with the structure will be eccentrically applied, through brackets or other cantilever arrangements. Inclined piles in this position may have a more favorable loading condition than vertical piles.

In-situ improvement of the bearing capacity by grouting is attractive since the work can be done in the open, as from sidewalks. Grouting is not feasible or possible in all types of soils, and there are difficulties to making an exact check in the field to ensure that the designed strength and homogeneity of the treated soil have been obtained.

Underpinning can be done eventually under separate contract before mobilization for the other construction work, thus expediting the overall progress of the project.

B. STREET DECKING

1. General

Cut-and-cover construction in urban areas usually requires some form of street decking to maintain traffic through the area, and there is much variation in the type and quantity used. United States practice seems to favor completely decking the street as was generally done in San Francisco and Washington, D. C. for transit subway construction, see Figures 46 - 52. The Europeans tend to leave part of the street uncovered, generally the center, permitting easier access for construction equipment and materials; Figures 53 - 58 illustrate how this procedure has also been applied in the United States.

FIGURE 46. TORONTO SUBWAY, Seating the Timber Decking. A lean concrete was used to backfill behind the lagging and up to the timber decking. The lean concrete was then capped with 6 inches of 3,000-psi concrete.

FIGURE 47. TORONTO SUBWAY, Installing and Splicing the Deck Beams. The half-section of the tunnel on the left side of the picture is already decked, and decking is in progress on the right side. The soil in the middle is supporting the deck beams. The soil was removed when the deck beams were spliced with bolts.

FIGURE 48. TORONTO SUBWAY, Deck Beam Installation, looking south along Yonge Street. The laborers at right are manually exposing an oil line. The ramp to the excavation was previously north of the utilities. The Contractor had to jump over and start a new ramp.

FIGURE 49. TORONTO SUBWAY, Decking Installation, looking north along Yonge Street. The workers are installing timber decking. Only a quarter of the intersection was taken out of service at any one time. This picture does not show the policeman directing the traffic (two directed traffic during the rush hour).

FIGURE 50. WASHINGTON, D. C. METRO, Decking 11th and G Street

FIGURE 51. WASHINGTON, D. C. METRO, Decking and Access Opening

FIGURE 52. WASHINGTON, D. C. METRO, G Street Deck in Place

FIGURE 53. INSTALLING DECK BEAMS UNDER EXISTING TRACK and timber decking for shoofly support. Note the steel plates in the street centerline where 14-inch by 14-inch by $17\frac{1}{2}$-ft timber supports for the railroad were installed between deck beams under the existing ties. (Courtesy of Fruin-Colnon Contracting Company.)

FIGURE 54. SOLDIER PILES AND CAP BEAMS. Live water lines and two live telephone ducts were to be supported. (Courtesy of Fruin-Colnon Contracting Company.)

FIGURE 55. ACCESS DECK DISCONTINUITY
(Courtesy of Fruin-Colnon Contracting Company.)

FIGURE 56. EXCAVATING BEYOND PLATFORM at the west end of the job where the shoofly tied into the existing tracks. (Courtesy of Fruin-Colnon Contracting Company.)

FIGURE 57. EXCAVATING FROM THE LOAD PLATFORM (Courtesy of Fruin-Colnon Contracting Company.)

FIGURE 58. 33 WF DECK BEAMS and 14-inch square timber supports the Muni Railroad at the west end of the job. (Courtesy of Fruin-Colnon Contracting Company.)

2. Temporary Decking

This refers to a deck placed on temporary beams supported at the excavation side walls by either temporary walls (such as soldier beams) or the permanent structure (such as a concrete-slurry wall). The beams supporting the decking are also used to brace the side walls of the excavation and to provide support for utilities. The decking sections are installed as soon as practicable to normalize traffic, and the deck and supporting structure are completely removed when tunnel construction is completed.

Constructing and removing a temporary deck require the following steps.

a. Build the side-wall supporting structure (soldier piles, slurry walls, or continuous bored-piles) on both sides of the excavation and, possibly, in the center, with some degree of surface-traffic interruption.

b. Excavate the roadway area sufficiently to install the transverse supporting-beams. This requires a complete interruption of traffic for at least a short period of time. Only one-half to one-third of the traffic area will be disturbed by using a center wall or a center support of original soil to hold the support beams and then splicing the beams after a similar operation is done on the other part of the road.

c. Excavate beneath the deck, support the utilities lines, and construct the tunnel.

d. Backfill above the completed tunnel.

e. Remove the deck structure, complete the backfill, and restore the surface. This step requires interrupting street traffic a second time.

3. Permanent Decking

A permanent roadway deck is built in the same manner as the temporary deck, but the permanent deck-support is transferred to the permanent structure when the tunnel construction is completed. This transfer of deck support is unnecessary, of course, if the support is a slurry wall or other type of permanent wall. The permanent decking has not been seen in the United States but it has been used to a limited degree in Europe, one instance being in Stuttgart where precast prestressed-concrete decking was used (see Figure 59). Further development of a permanent deck is recommended.

4. Materials

Timber is the most frequently used material in the United States, with metal plates occasionally used for small areas. A few instances have also been noted (in Washington, D. C.) in which precast concrete was used. Precast concrete is more widely used in other countries, notably Japan.

A unique structure has been used in London, see Figures 60 and 61. Called an umbrella deck, it is a steel structure with steel-plate deck with its own asphalt wearing-surface which stands about 3.5 ft above the street surface. Designed to form a complete pavement umbrella over the busy intersection, it can be installed and removed over a three-day weekend. This will be discussed later in this chapter.

5. Placement Time

The time required for excavating a width of street, placing

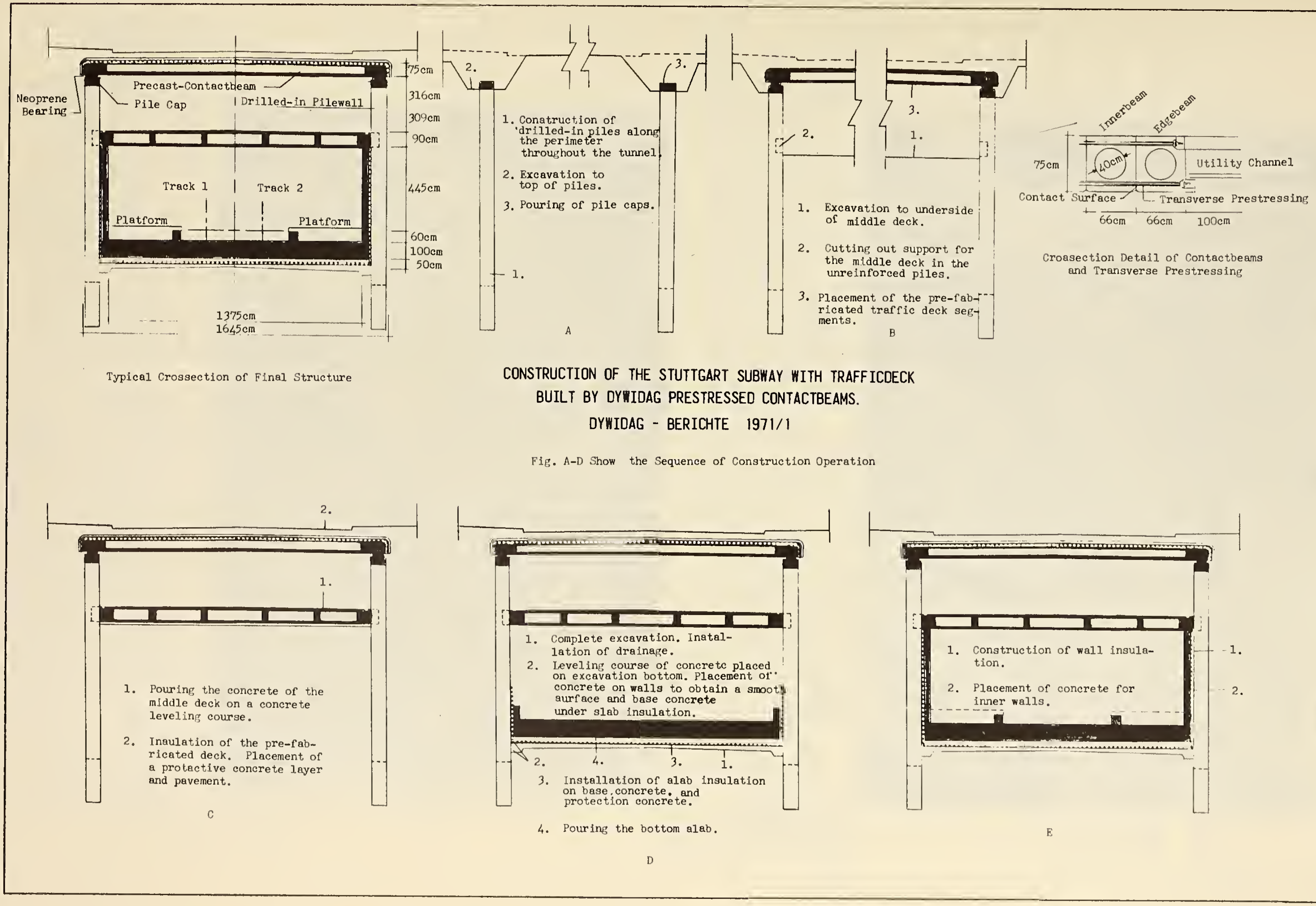

FIGURE 59. CONSTRUCTING THE STUTTGART SUBWAY WITH TRAFFIC DECK (By permission of Dyckerhoff & Widmann A. G., Munich.)

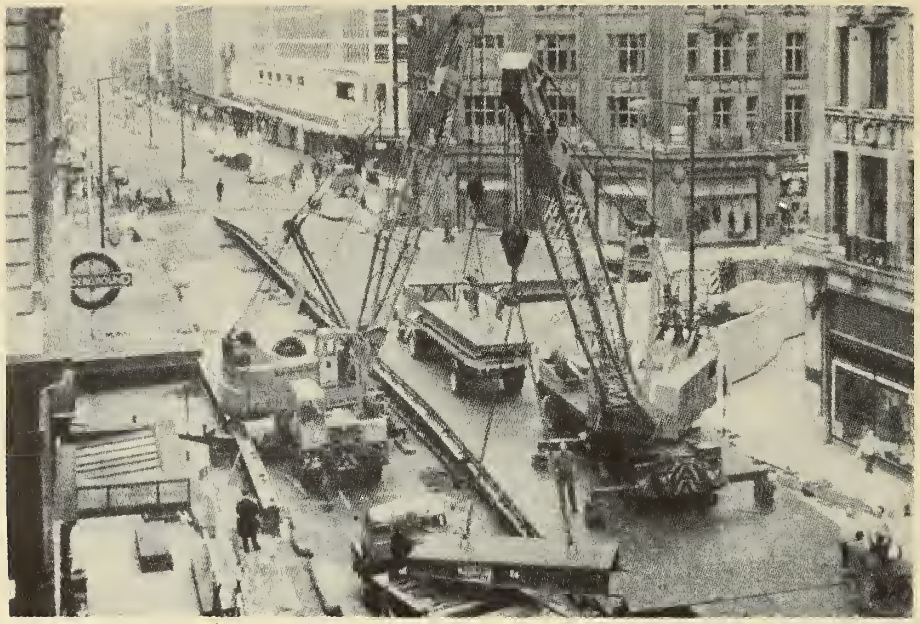

FIGURE 60. UMBRELLA DECKING, LONDON. Subsurface construction under busy streets can be done by covering the surface with temporary bridging. This busy intersection was completely covered during the weekend with an "umbrella" of steelwork panels complete with road surfacing. A ticket hall concourse was constructed under the temporary work, and the busy road traffic was carried without interference.

FIGURE 61. UMBRELLA DECKING, LONDON. The completed "umbrella" of steel and concrete panels of temporary roadway. 44 million vehicles were carried while the underground work was in progress. Upon completion, the steel and concrete panels were removed in one weekend, and the road intersection was re-opened with permanent roads which had been built under the decking. (Both pictures courtesy of H. G. Follenfant, Chief Civil Engr., London Transport. From "Underground Railway Construction".)

the support beams, and putting the deck into position for a construction length of roughly a city block will generally take three or four weeks under normal conditions. Timber panels are generally bolted together to form 10- to 12-ft-wide panels, and the panels are kept in position longitudinally with vertical plates welded to the tops of the supporting beams.

6. Problems and Limitations

a. Most timber and steel decks are subject to warping during use. This leads to additional noise and, if the deformation becomes great enough, additional maintenance during construction.

b. Bending of the steel plates is caused by fabrication inaccuracies, and by poor fit in the field.

c. Pavement noise under traffic can be generated by poor fit or deck warping. Rubber seating-arrangements have been tried (in Toronto) with some success.

d. The timber-deck surface becomes considerably more subject to skidding problems with time.

e. A certain amount of timber decking is replaced during the life of the job because of warping and excessive wear.

f. Almost no timber decking is reused, although this is theoretically possible. A contractor is often unable to move decking ahead because of the long periods during which streets must be decked while the utilities relocations are being made. A contractor will probably have only one tunnel segment, so the likelihood of reusing the decking is minimal.

7. <u>Suggested Development Areas</u>

The successful use of an umbrella-type structure in London to completely bridge an intersection with a deck raised above the road surface leads to another possible development. Moving such a structure on rubber tires or on steel wheels on rails and shifting the weight to fixed supports after it is properly positioned seems quite feasible. Work would proceed normally under the deck, but only surface reconstruction would remain to be done when the tunnel is completed since the deck would be moved to its next position. The techniques for moving such structures are readily available, for example, in the equipment for sports-stadium movable stands.

C. RELOCATING AND SUPPORTING EXISTING UTILITIES

1. General

Relocating the utilities was cited by many respondents as one of the most difficult of the problems and the one which causes the most delays.

Some of this undoubtedly comes from the fact that both public and private utilities operate in the United States, and the two tend to assign different priorities for relocations. The involvement of several agencies generally tends to make coordination more difficult. The practice in Europe seems to be somewhat different, probably because most utilities are under coordinated and unified management.

Hand excavation is generally required for the first few feet because the exact utility locations are not known, and unsuspected utilities may be present, resulting in a prolonged period of street disruption.

The utilities should, therefore, be located as accurately as possible before construction begins.

2. Methods for Locating Utilities

The utility-locating work normally begins with an inventory of the existing utility maps, which are checked in the field for accuracy and completeness.

The conventional field-location methods include digging and probing, as well as electromagnetic means. Regular seismic techniques are not suited for locating utilities because of their small dimensions. Acoustical, sonar-type equipment is under development.

A continuous profiling device operating with electromagnetically-generated radar pulses has been developed by Geophysical Survey Systems of Billerica, Massachusetts. It has reportedly been successfully used for locating utilities and for subsurface stratigraphy to 20-ft depths, but the recordings must be corrected for the variations in local electrical properties. Separating echoes seems to be the main evaluational problem, but depths can be determined within 5 percent limits of accuracy. Boring data should be used for the evaluation to assure optimum accuracy.

3. Storm and Sanitary Sewers

Existing storm and sanitary sewers are most preferably relocated outside the limits of the excavation. Very old sewers are often brick and mortar, while more recent ones are often vitrified clay pipe. These are very difficult to support without causing movement and creating leaks. The alignment and also the proper grade must be maintained for sewer lines.

If the sewer cannot be moved outside the excavation, one solution is to encase the entire sewer in a cast-in-place reinforced-concrete box. This will provide the additional advantage of giving substantial protection to the sewer during construction when there is danger of it being accidentally hit by construction equipment. Encasing is very expensive, however, and is generally used only where a sewer crosses the excavation. Several examples of this were used in Germany where 300-year-old brick sewers were totally encased where they crossed the excavation, at both right angles and diagonally, and the encasing was further supported by temporary structural-steel bents. As an additional benefit, these structures were fitted with handrails and were used as personnel bridges for crossing the excavation. Figures 62 and 63 show alternate construction solutions.

Another way to cross 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 building the remainder of the tunnel, see Figure 64. Figure 65 shows typical utility supports.

3. Water Lines

Both transverse and longitudinal water lines can be supported within the excavation, if necessary. This can be done with brackets supported from either the walls of the excavation support walls or the temporary street-deck structure.

If the water lines are supported from the structural beams used as part of the support for the street decking, consideration should be given to the effects repeated flexing by traffic loads will have on the water lines. Water lines have ruptured in several recent cases, apparently because of this deflection.

FIGURE 62. NEW YORK CITY TRANSIT AUTHORITY, Cutting a Sewer to Make Way for a Tunnel and Utilities. (From Engineering News-Record, October 20, 1969, p. 26, by permission.)

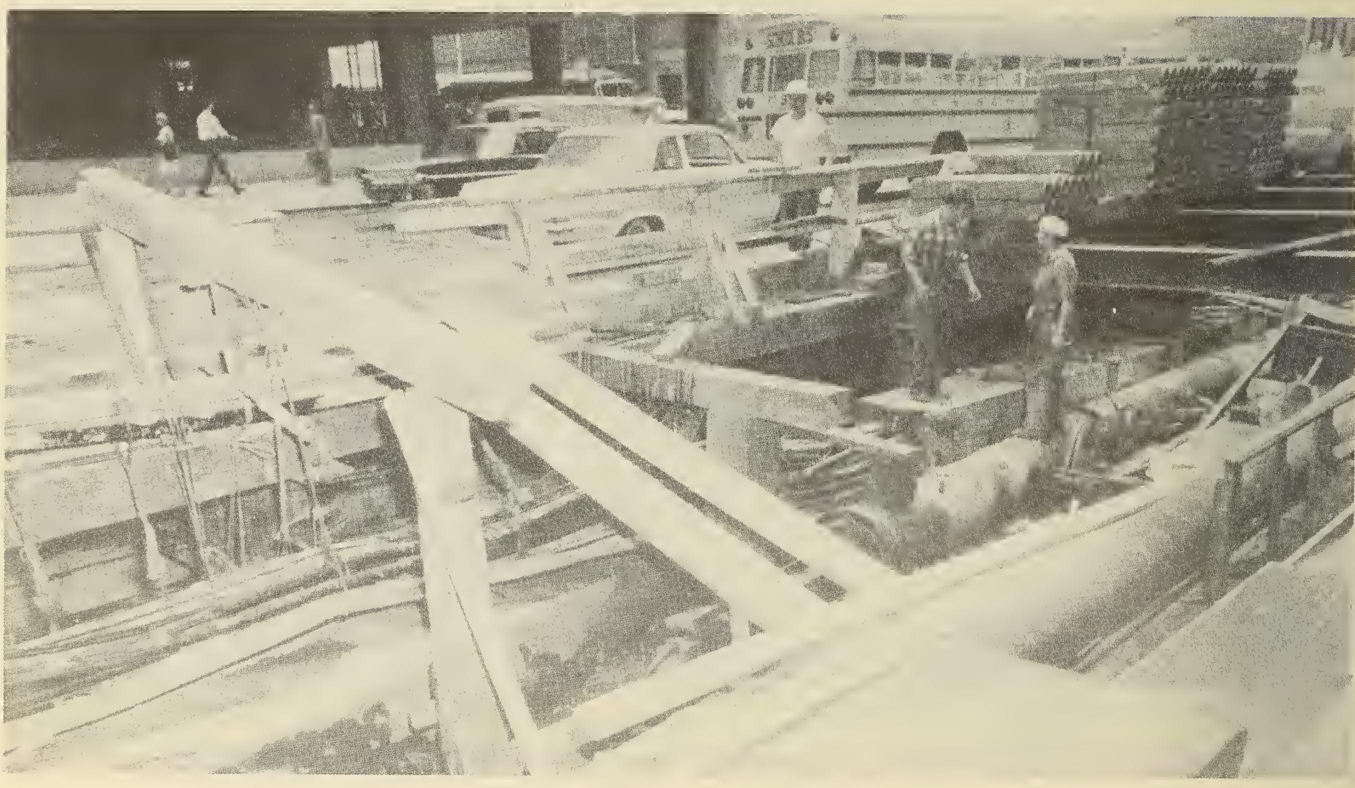

FIGURE 63. NEW YORK CITY TRANSIT AUTHORITY, Exposed Utilities. A maze of utility pipes and cables is close at hand for sidewalk superintendents. (From Engineering News-Record, October 20, 1966, p. 27, by permission.)

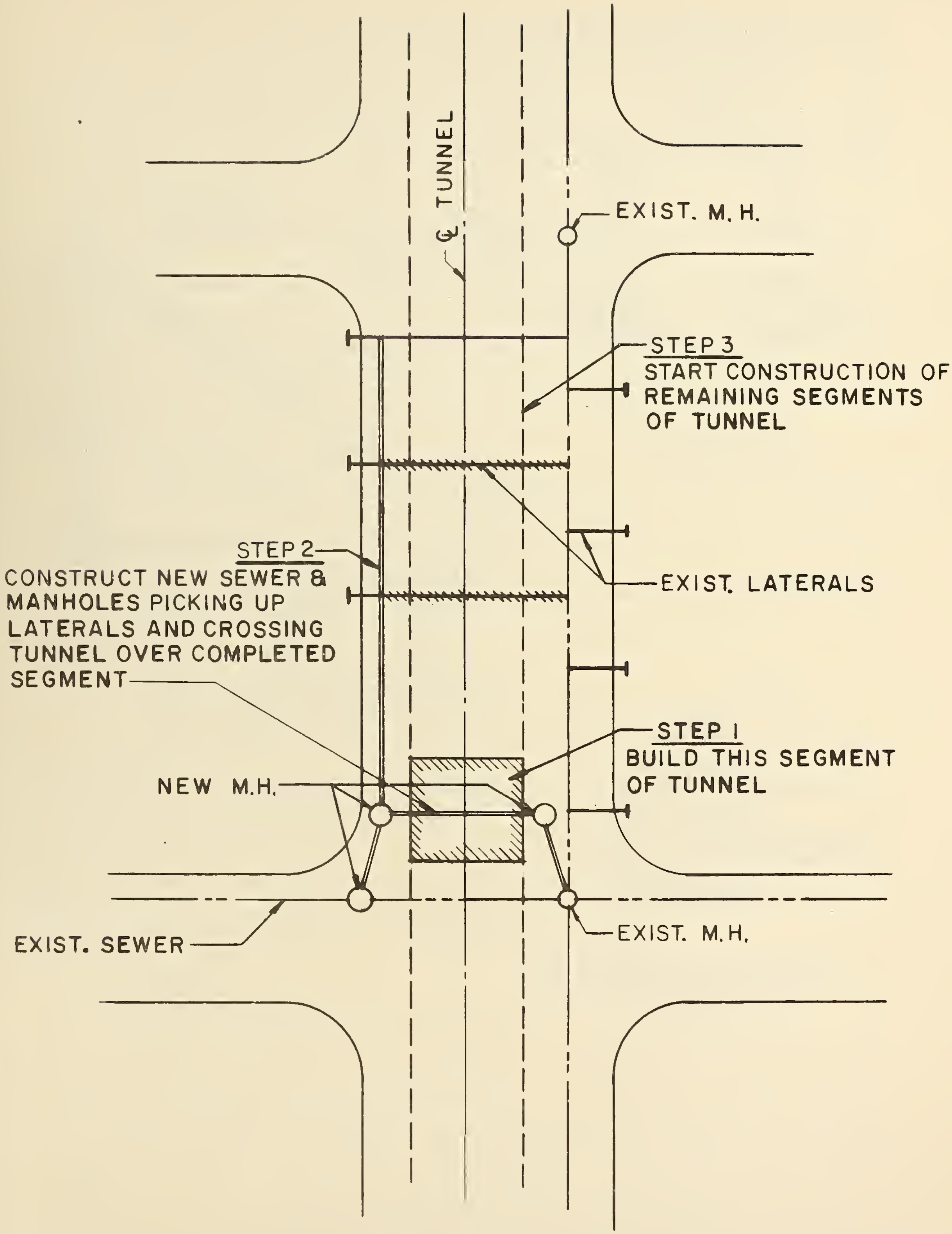

FIGURE 64. UTILITIES CROSSING CONSTRUCTION METHOD

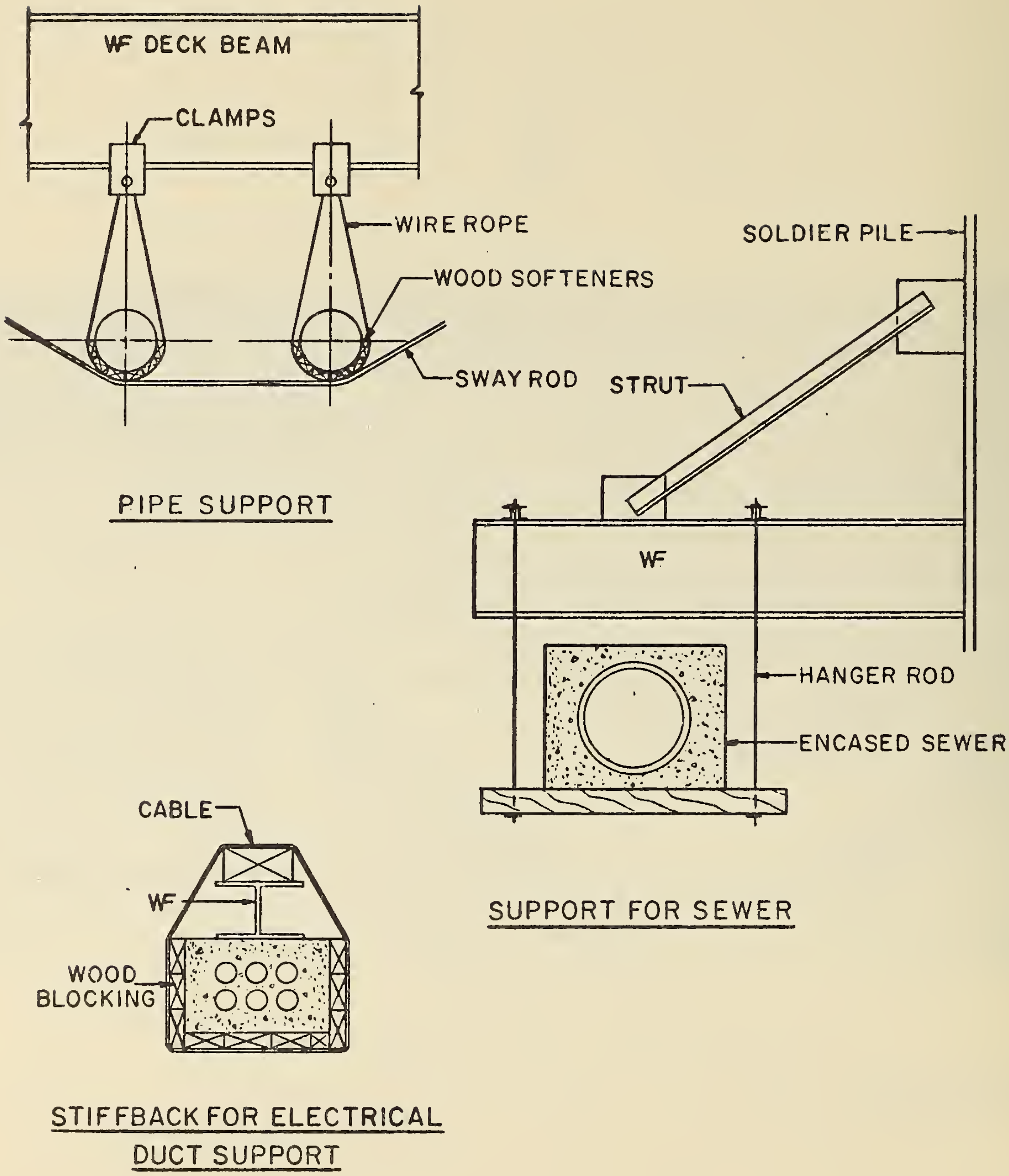

FIGURE 65. TYPICAL UTILITY SUPPORTS

Water-line hangers should preferably be placed as close as possible to the beam supports to minimize flexing, or supports should be considered that are independent of the street deck-supports.

4. Gas Lines

Supporting a gas line within an excavation is an extremely dangerous procedure which should be done only under the most rigidly controlled conditions. A gas line rupture in a decked-over excavation could lead to a catastrophe such as occurred in Japan in which 76 people were killed.[8]

Gas lines should be either permanently relocated outside the excavation or rerouted temporarily over the street near the excavation. If the latter is done, special precautions must be taken to protect the exposed line.

5. Telephone

Most telephone service occurs in duct banks made up of fiber ducts encased in concrete. Supporting such banks is not too difficult if they are easily accessible. If a bank must be relocated, however, several months' time may be required to install the new manholes and splice the possibly several thousand pairs of wires.

6. Electric

Major electric service also often occurs in duct banks, and the same criteria generally applies as for the telephone lines. There is, of course, the added danger of an accidental disruption of electrical lines.

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8. "Gas Explosion Rips Subway Construction Site," Engineering News-Record, April 16, 1970, p. 11.

7. Evaluation

The general procedure in the United States is to have the public utilities, (i.e., sewers and water lines) relocated by the general contractor and the private utilities (i.e., gas, electric, and telephone lines) relocated under separate contracts.

This tends to make the controlling and scheduling operation a very difficult procedure, and often leads to situations where the street is either disrupted several different times, or other work is delayed while waiting for a particular utility relocation.

Developing a local authority which can administer and coordinate all utility relocations in a timely manner will be a major improvement. This seems to be already true in Europe where all utilities are more in the category of public utilities.

Another major area where improvement can probably be made is in the field of utility tunnels or "utilidors," which are discussed in more detail in Chapter VIII of this report. The American Public Works Association has shown much interest in this subject, but the problem of gaining acceptance by all the utilities involved has deterred any real development in the United States. Nonetheless, other countries have successfully used this principle, and the idea of using the space above the tunnel as a permanent, accessible passage for all utilities seems to have merit.

Utilities relocations in older urban areas should be used as an opportunity to upgrade and modernize the existing sewers, as well as the electric, gas, and water lines and manholes. The storm and sanitary sewers might also be separated this way.

VIII PERMANENT STRUCTURE

This Chapter discusses permanent structures under the headings of: cast-in-place concrete and precast concrete elements, which are primarily components of the finished structure; slurry walls and continuous bored-piles (secant piles) which are used both in the permanent structure and in the ground-wall support; caissons in which the entire structure is installed at one time; and the miscellaneous items of under-the-roof construction, waterproofing, and use of space above the tunnel (Utilidors).

A. CAST-IN-PLACE CONCRETE

Concrete has long been the primary material for constructing below-grade structures. Ease of placing and resistance to deterioration have been the primary considerations.

1. Design Standards and Materials

Concrete is normally specified to have a 28-day strength of 3,000 psi or better. High-early-strength cement is often used to permit faster form-stripping and reuse, but the new expansive cements have not yet been used in tunnel construction.

The concrete structure is normally reinforced with conventional reinforcing-bars. Only a small amount of post-tensioning has been done to date. Experiments are currently under way on a material called "Wirand" concrete which was developed by the Battelle Development Corporation. It consists of a concrete mix to which fine ferrous wires are added (about 2 percent by volume, about one-inch long, and the diameter of a pin). The wire-reinforced concrete apparently acts in an entirely different manner from conventionally-reinforced concrete, and behaves practically as a homogeneous material.

The most common design for cut-and-cover construction consists of a continuous single- or double-box frame. Construction joints are spaced longitudinally at about 50-ft on-centers, and expansion joints are normally not required.

The so-called "jack-arch" type of construction used for many years by the New York City Transit Authority consists of closely-spaced steel frames (about 5-ft on-centers) encased in a concrete base-slab, and concrete walls and roof. Flat-arch metal-forms for the walls and roof span between the frames. The metal forms can be reused many times, concrete and reinforcement are kept to a minimum, while the completed structure may be readily altered if the need arises. The thin concrete causes such a structure to be more subject to cracking than a concrete frame, however, because of the planes of weakness introduced by the steel frames. These structures require more positive waterproofing treatment for this reason.

2. Construction Procedure

The concrete base-slab is cast first in the conventional approach, generally for the full width of the excavation between the ground-support structure (such as soldier piles and lagging), see Figures 66 and 67. This gives the lateral support required at the base of the excavation, often allowing removal of the bottom strut. The current practice generally uses an invert form which makes the top surface of the slab and the start of the side walls as well. The

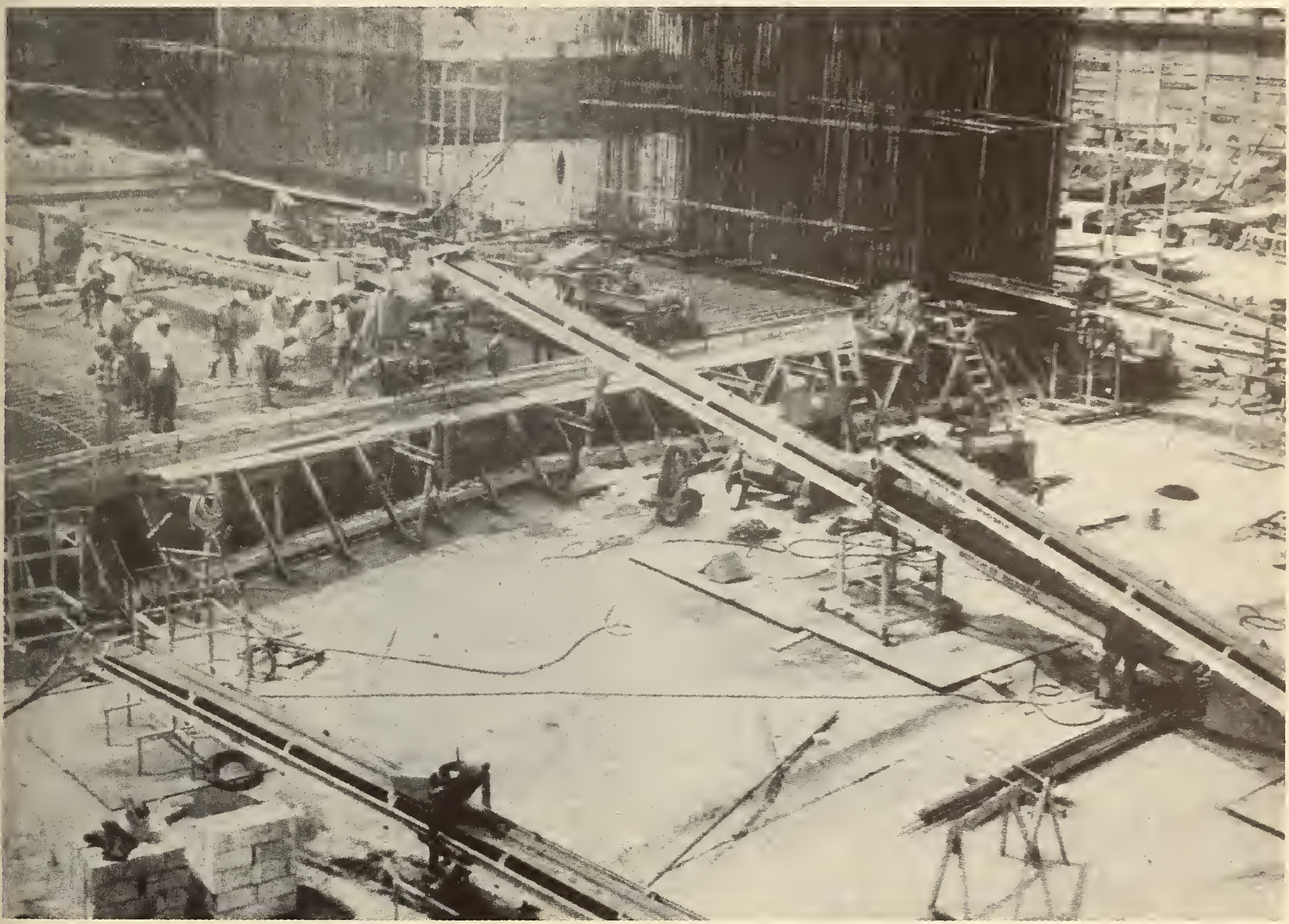


FIGURE 66. WASHINGTON, D. C., Casting the Four-Lane Floor Slab in the eight-lane Center Leg Freeway tunnel. (By permission of Stewart Bros. Photographers, Rockville, Maryland.)

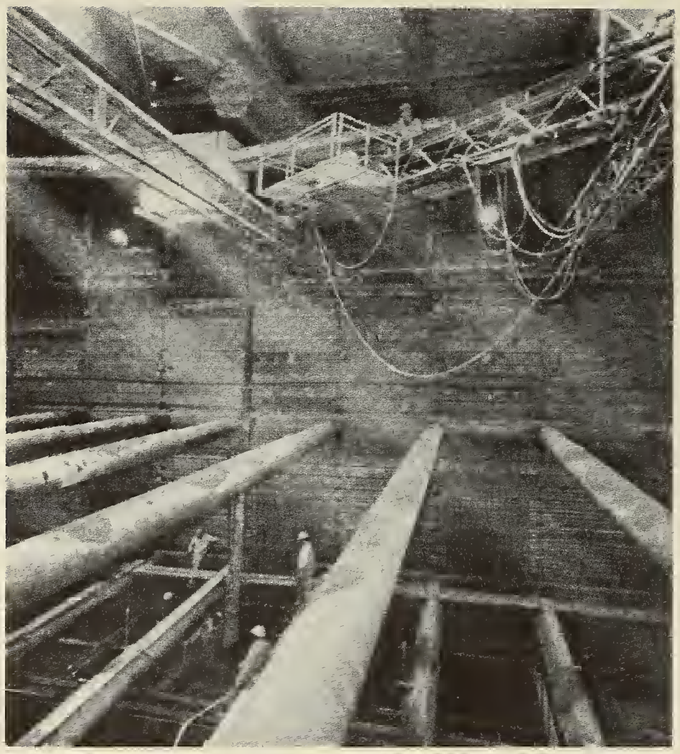


FIGURE 67. BART, SAN FRANCISCO, Gantry Rig with Pivoting Conveyor Truss Placing Concrete. (From Engineering News-Record, July 24, 1969, p. 30, by permission.)

slab is cast in about 50-ft sections, and 100 ft can be completed in two days.

The box forms for the walls and roof slab are installed about 300 to 400 feet behind the base-slab construction. These forms are generally equipped with telescoping-plate arrangements to permit retracting the forms for stripping and rolling ahead, (see Figure 68.) Two sets of forms are used--the first casting a 50-ft section and then skipping a space to move ahead and cast the next section. The second form casts the intermediate sections, thus reducing the total amount of shrinkage. Once the operation is under way 100 ft of tunnel can be completed in two days which is comparable to the rate for placing the base slab.

Since the operation normally takes place below the street level deck, the concrete is often either placed by conveyer with an elephant trunk (or tremie) to reach the bottom, or pumped and placed.

The tunnel walls may be either cast directly against the face of the ground-support structure or cast inside, leaving sufficient space to install waterproofing. Some designs incorporate the steel soldier-piles into the permanent wall.

The roof slab is cast first in under-the-roof construction, on top of the previously-installed sidewalls. A 4-inch leveling slab may be placed on the rough grade for use as a form for the roof slab and then removed from underneath as the excavation proceeds under-the-roof. The base slab is cast last, being either keyed into the sidewalls or fastened by reinforcing installed in the walls when they

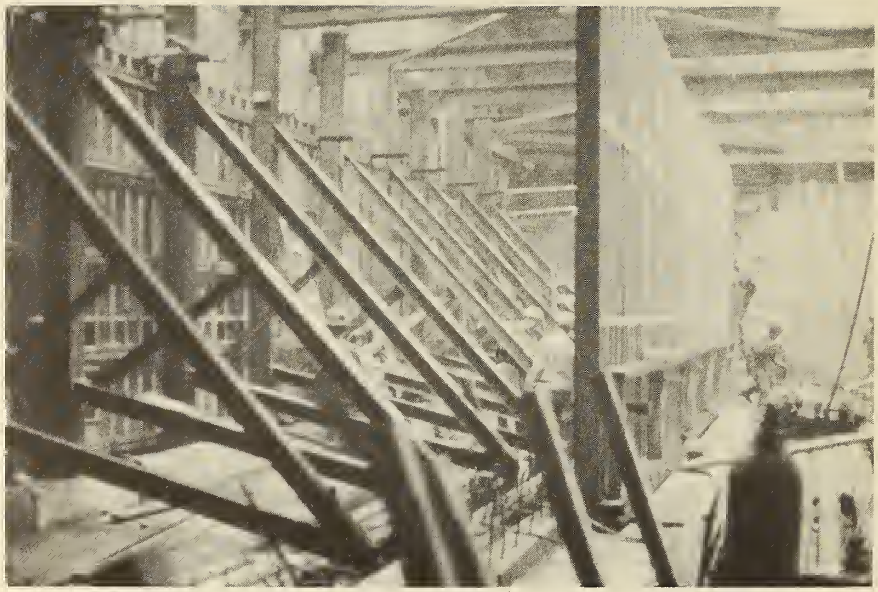


FIGURE 68. TORONTO SUBWAY, REUSABLE METAL FORMS at a location where the subway enters a station. All these forms are standard forms that were used for ten years. The Contractors purchase them from each other, thus minimizing the cost.

were cast.

3. Evaluation

The steel traveling forms used for conventional cut-and-cover tunnel construction are highly sophisticated, effective, and dependable. This area does not seem to be in need of additional development. The same may be said of the methods for placing concrete, in general. See Figure 69 for concrete placement costs and Table VII for data on lines for pumped concrete.

Areas that could have promise for future improvements include using wire-reinforced concrete, developing horizontal slipform techniques (which probably depend more on the development of regulated-set concrete), and the effective use of expansive cements to improve watertightness.

B. PRECAST CONCRETE ELEMENTS

Pretensioned or conventionally-reinforced precast elements can be prepared and stored off-site and then put in final position in a miminum amount of time, thus reducing the disruption of surface traffic.

The first major use of this method was in the Moscow Metro, in which precast elements, base slabs, wall, and decks were used. Precast prestressed beams about 30-inches deep and about 24-inches wide, span about 55 ft in Stuttgart to form the traffic deck over a subway tunnel. The beams are precast in contact with each other and are erected in the same position. Integral panels were made by post-tensioning from eight to fourteen beams as a single unit. The space

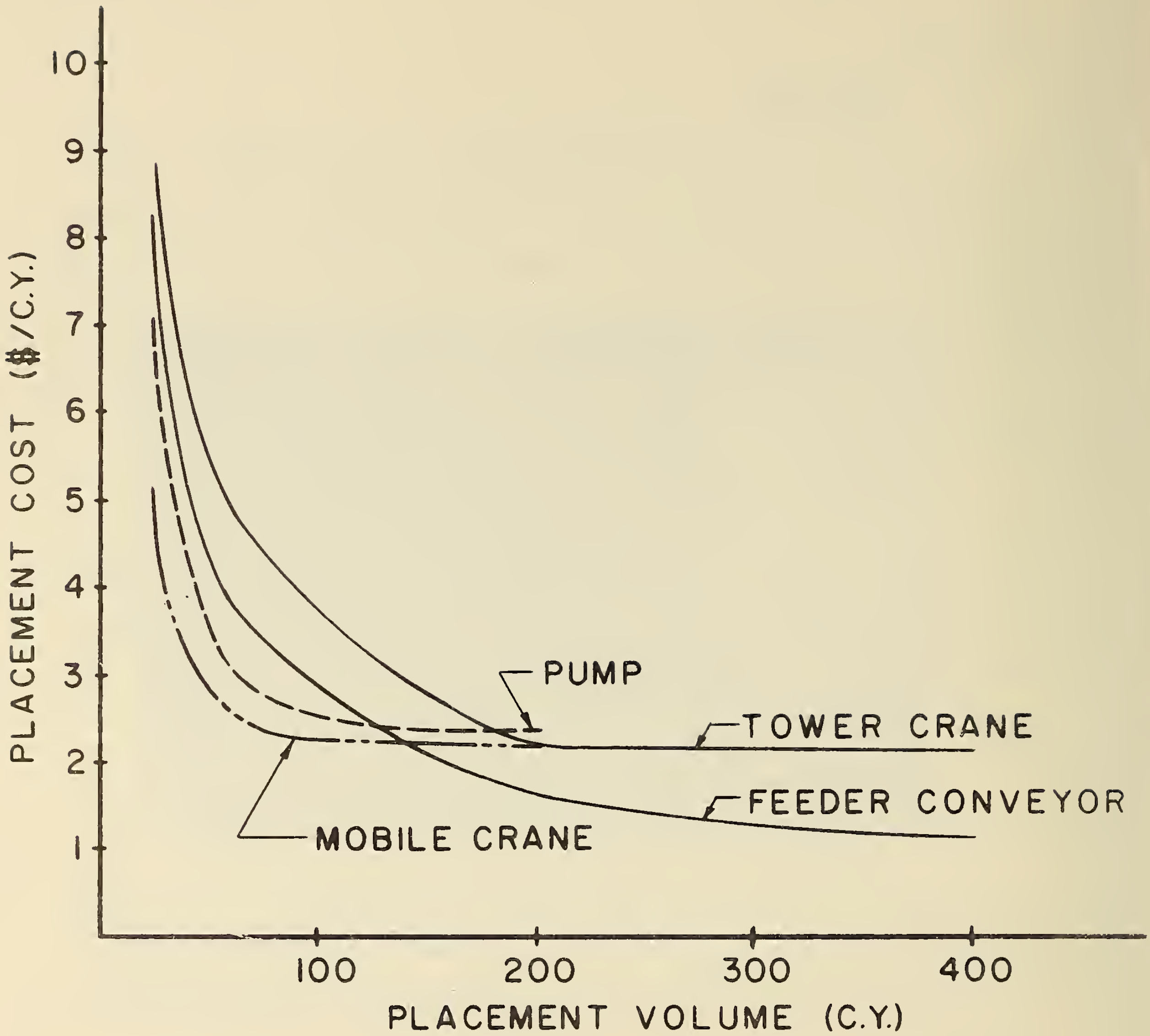


FIGURE 69. CONCRETE PLACEMENT COSTS

TABLE VII

DATA ON LINES FOR PUMPED CONCRETE

Pipe diameter in inches, nominal size		3	4	5	6	7	8
Cross-sectional sq in.		6.41	11.7	19.6	28.3	35.1	50.2
Nominal maximum size of aggregate (in inches)							
Rich mix		1	1	2	2	3	3
Lean mix		3/4	3/4	$1\frac{1}{2}$	$1\frac{1}{2}$	2	3
Volume of concrete per 100 ft of pipe, in cu yd		0.2	0.3	0.5	0.7	0.9	1.3
Pipe length per cu yd of concrete, in ft		625	333	200	137	111	78
Concrete weight per 10-ft section of pipe (in lbs)		66.8	123	204	294	366	524
Cu yd delivery capacity in fps for average (not peak) velocities indicated	fps						
	1	5.9	11	18	26	32	46
	2	12	22	36	53	65	93
	3	18	32	54	79	97	--
	4	24	43	72	100	--	--

(From Journal of the American Concrete Institute, May 1971, p. 331.)

between adjacent panels was used for utility crossing, see Figure 59.

Precast elements have been used in London to build pedestrian subways. Standard prestressed inverted T-beams have also been used to repair sections of the roof of the old District and Circle subway lines which were built with cast-iron beams in the 1860's. These same standard T-beams are presently being used to construct the roof deck of the Piccadilly Line extension to Heathrow Airport, (see Figure 70.) The beams are set side-by-side with their wide-bottom flanges forming a continuous surface, and concrete is poured between the webs and to a depth of about 4 inches over the top of the railhead-like top flanges. The top surface is waterproofed, and a protective slab is cast-in-place on the waterproofing. About three weeks is required to build a particular segment, from the beginning of excavation until the protective slab is in place.

Precast 54-ton T-beams in San Francisco's Embarcadero Station span 54 ft and bear in 16-inch-deep pockets in concrete-slurry walls and support the roadway slabs for municipal streetcars on the middle level. This is one of the few uses of precast members in the United States.

The potential timesaving from using precast deck elements is reduced by the necessary time for the cast-in-place elements to be poured and cured in the joint between the prestressed units. However, the possibility of significantly reducing construction time by using precast members warrants additional development. Use in the roof structure, especially in conjunction with some form of

FIGURE 70. PRECAST ELEMENTS USED FOR ROOF OF PICCADILLY LINE extension in London, England. Note the reinforcing bars through the section web. The step marks a change in the roof elevation.

slurry-trench walls or continuous bored-pile walls seem especially advantageous.

C. SLURRY WALLS

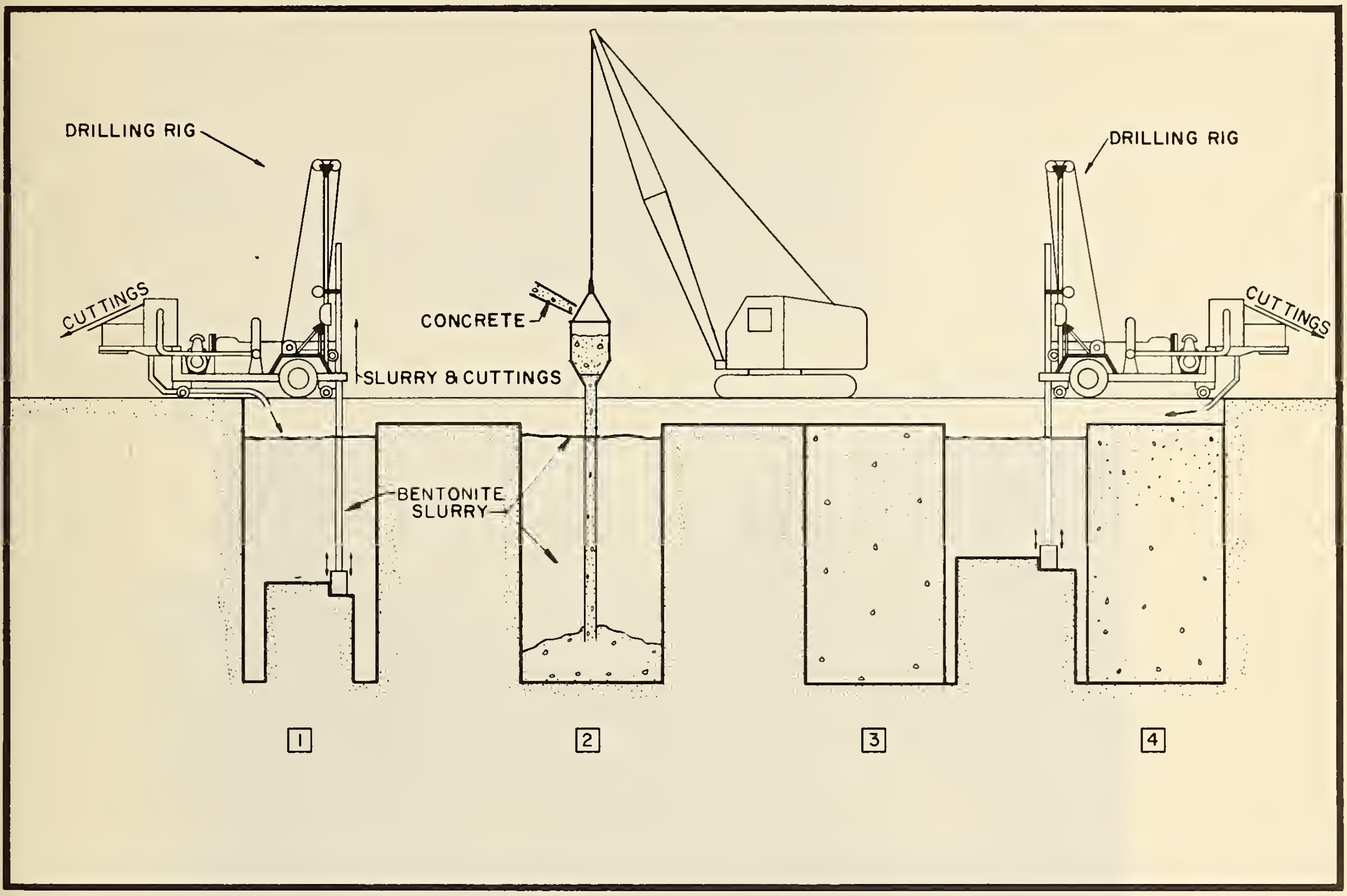
Slurry-trench construction has been used for highway and rapid-transit tunnels in many European cities (see Figures 71-77), and a test section of the Toronto subway was built by this technique in the late 1950's. The method has mainly been used in the United States for building foundations such as the World Trade Center in New York and for certain underground structures for the BART project in San Francisco, and a small section of the Washington, D.C. Metro system. A summary of slurry-trench construction in the United States is given in Table VIII which shows a steady rise in the number of projects in which it was used.

1. Technical Description

The slurry-trench technique consists basically of digging or drilling a trench which will be kept from caving in or collapsing by a bentonite (sodium-montmorillonite) slurry and constructing the structural member directly in the trench. Figure 71 diagrams this process.

The specifications for the slurry include limits for specific gravity, PH, viscosity, and the amount of solid particles. The slurry can generally be reused three to five times if the coarse particles are effectively removed.

The plumbness of the walls is sometimes specified. The specified vertical tolerance may vary between 1:80 and 1:200, depend-



1 EXCAVATION UNDER A CONSTANT HEAD OF BENTONITE SLURRY

2 TREMIE CONCRETE INTO THE SLURRY-FILLED TRENCH

3 SLURRY DISPLACED BY CONCRETE WALL

4 SLURRY RECIRCULATED FOR THE NEXT CONSTRUCTION

FIGURE 71. CONSTRUCTING A WALL BY THE SLURRY-TRENCH METHOD
(Soletanch equipment is shown)

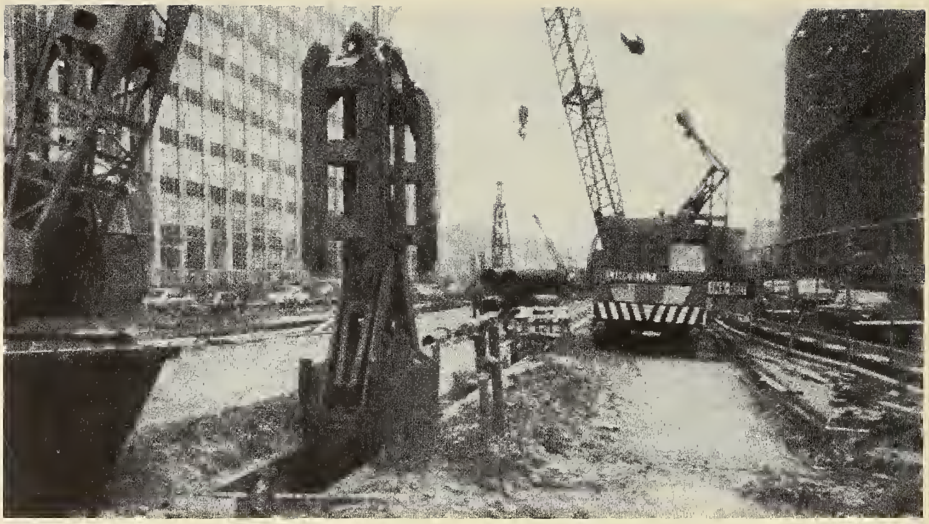


FIGURE 72. AMSTERDAM SUBWAY, Excavating for a Slurry-Trench Cut-off Wall (From Engineering News-Record, August 19, 1971, p. 31, by permission)

FIGURE 73. MEXICO CITY, MEXICO SUBWAY, Digging Utilities by Hand

FIGURE 74. MEXICO CITY, MEXICO SUBWAY, Concrete Lip Guides Excavators

FIGURE 75. MEXICO CITY, MEXICO SUBWAY, Casting Walls in a Slurry Trench

(All pictures from Engineering News-Record, June 27, 1968, p. 24, by permission)

FIGURE 76. SLURRY-TRENCH WALL FOR BANK BUILDING in Stockholm. The lower part of the wall was taken through boulders and broken rock by chiseling. The seepage was cut off in all places except in an area (lower right corner) where grouting and freezing outside the wall was used to stop the flow. The tiebacks are drilled at 45° and grouted into rock.

FIGURE 77. SLURRY-TRENCH WALL FOR BANK BUILDING in Stockholm. One adjacent multistory brick building (upper part of wall in picture) settled two inches during excavation.

TABLE VIII

SLURRY-TRENCH CONSTRUCTION IN THE UNITED STATES

Contractor	Year	Project	Wall area, (sq ft)
Bencor	71-2	Bank in Dallas, Texas	50,000
Bencor-Caisson	71	Standard Oil Building, Chicago	60,000
Franki	66	Somerset Hotel Garage, Boston	5,600
--	68	Boston Company Building, Boston	3,300
--	68	Spear Street Ventilation Shafts, BART, San Francisco	24,000
--	68	Embarcadero Relieving Platform, BART, San Francisco	15,000
--	69	Providence Office Building, Rhode Island	24,000
Ben Gerwick[1]	64	Bank of California Building, San Francisco	37,800
--	67	Civic Center Subway Station BART, San Francisco	136,000
--	68	Pacific Gas and Electric Office Building, San Francisco	78,472
--	69	Embarcadero Center Building San Francisco	73,304
ICOS	64	Kinzua Dam (Allegheny River)	118,000
--	66	World Trade Center telephone vaults, New York	15,000
--	67-8	World Trade Center, New York	250,000
--	70	CNA Building, Chicago	36,000
--	72	Cobian Center, Puerto Rico	62,000
--	72-3	Archer Avenue Subway Extension, New York	90,000
--	72-3	Federal Central S.W. Center, New Carrollton Route, Washington Metropolitan Area Transit Auth.	180,000
Mitsubishi Tone Boring Company	72-3	Fields Building, Chicago, foundation and building	80,000
P&Z	67	The Oakland "Y", BART	18,000

1. Now Santa Fe-Pomeroy

Table VIII, contd.

Contractor	Year	Project	Wall area, (sq ft)
P&Z (contd)	68	Montgomery and Powell Subway Stations, BART	124,800
--	69	University Hospital, Baltimore	40,000
--	72	City Center, Omaha	80,000
Santa Fe-Pomeroy	72-3	Lewiston Levee, Idaho	120,000
Soletanche	72-3	Hannah Street Pumping Station, New York City	20,000
Spencer-Soletanche	63	East River Tunnel Shaft, New York	3,550
--	64	Beacon Tower Apartments, Boston	26,140
Spencer, White & Prentis	70	Avenue C Pumping Station, New York City	30,000
--	71	Sears Building, Chicago	80,000
--	72	New York City Water Supply (PW 164)	30,000
Stang-Cofor	68-9	South Cove Tunnel, Boston Subway	34,000

ing upon the depth of excavation and the available space.

Over-excavation can be corrected by refilling the trench with lean concrete and re-excavating after the initial set. Neat-appearing walls are especially important where the slurry-trench wall will be an exposed part of the final structure.

The joints between wall segments are the weakest points of the walls with respect to both strength and watertightness. The segments should, therefore, be made as long as possible and not shorter than 5 meters (about 17 ft) or preferably 7.5 meters (30 ft).

The allowable width of the segments depends upon the soil properties. A time-dependent decrease in width was noted in a 28-m-deep, 5-m-wide trench segment in Oslo constructed in a marine clay having a shear strength of 2 to 4 tons/m^2 and a sensitivity ranging between 2 and 8. The first decrease occurred at about 2.3 mm/day for a slurry density of 1.24 t/m^3, and slowed down to 0.9 mm/day after two weeks. The slurry was then diluted to 1.10 t/m^3, giving an initial decrease-rate of 3 mm/day which slowed down to 1 mm/day after a week. The slurry was finally replaced by water, which gave an initial creep-rate of 2.9 mm/day which decreased to 1.2 mm/day after 11 days. An inclinometer casing placed outside the trench showed about 10 mm horizontal movement during excavation.

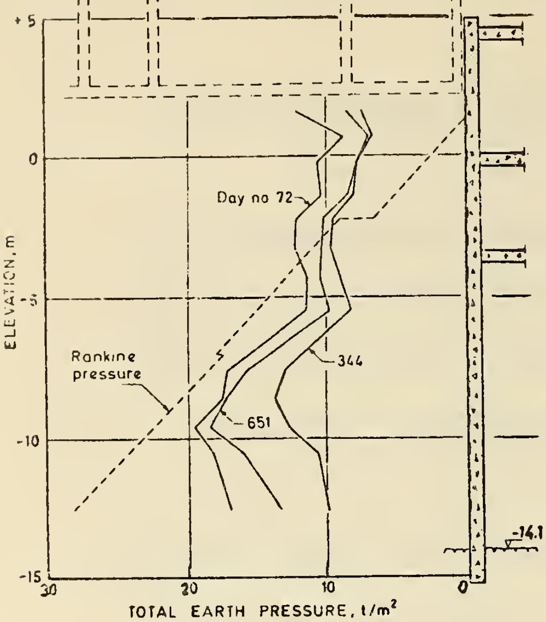
The total decrease in width after excavation was about 5.5 cm at about 22-m depth; the decrease was about half this value at the ends of the segment, which shows the end-restraint effects. The maximum settlement near the trench was about 8 mm. The settlement gauge farthest out (20 m from the trench) showed about 1 mm settlement.

Figures 78-81 give examples of the field observations in slurry walls.

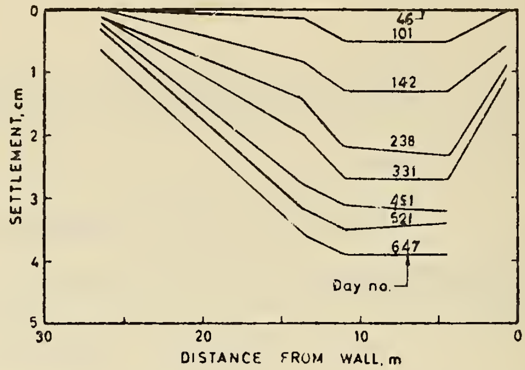
The reinforcement cages are normally tied horizontally, raised into position, and lowered into the slurry. The cages generally have spacers (sometimes rollers) to give the correct distance from the excavation walls; they also have blockouts for dowels, tie-backs, and other uses.

The concrete (which should have a slump of about 7 inches) is tremied from the bottom, displacing the slurry. The length of trench per tremie pipe should not exceed 3.6 m (12 ft) according to Japanese practices. Poor concrete quality occurred near the points in a case in which 5-m-long wall segments were tremied through only one pipe. These segments are permanently exposed in the building basement, and the extra work to make watertight joints raised the construction cost by about 50%. Numerous joint-sealing devices have been developed by Contractors--such as inserting strips of sheet metal or a double pipe (forming figure-eight cross-section) across the joint. Other methods include overcoring the joint or simply cleaning the concrete surface of the first cast segment. This does not presently seem to be a desirable method, and further development is apparently needed.

There is still much controversy about the effect of bentonite slurry on the bond-strength between the steel and the concrete. Japanese engineers claim there is no reduction in bond-strength if the slurry is well-controlled. The PH value is especially important

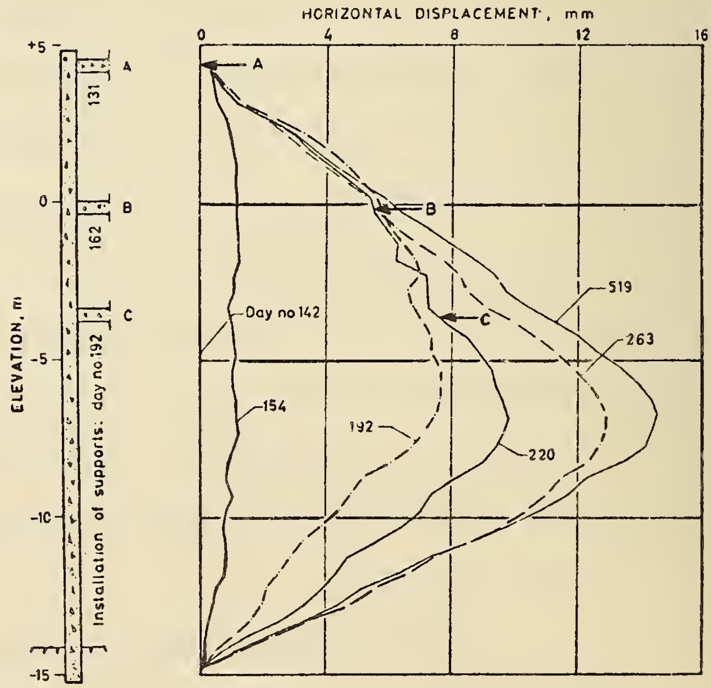


Distribution of Earthpressure



Settlement versus distance from wall

The soil is a normally consolidated marine clay of low sensitivity. The clay beneath the dry crust has an undrained shear strength of 2.5-4.0 t/m^2 and a sensitivity of about 5. The excavation started on Day 74 and was completed on Day 225. Support A, B, and C were installed on Day 131, 162 and 192 respectively.



Horizontal movement of wall determined by inclinometer measurements.

FIGURE 78. OBSERVATIONS MADE DURING CONSTRUCTION OF THE TELEPHONEHOUSE BASEMENT, OSLO, by the slurry-trench method. (After E. DiBiago and J. A. Roti. By permission of the Sociedad Espanola de Mecanica del Suelo)

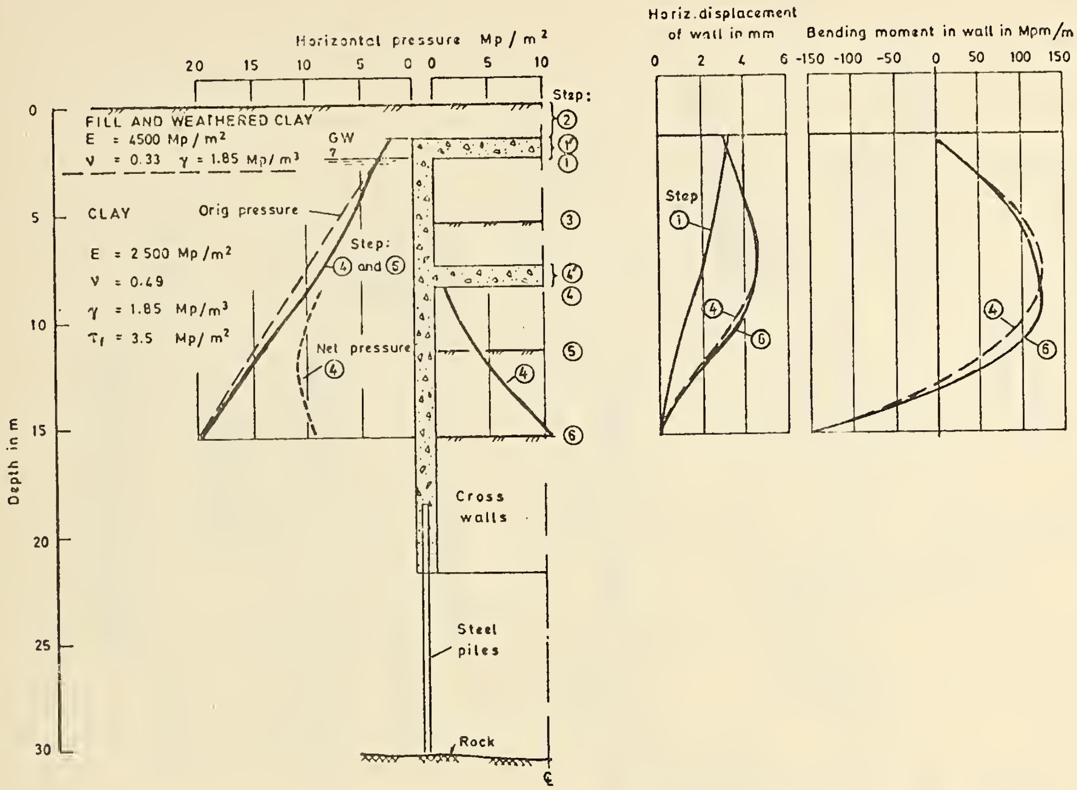


FIGURE 79. EARTH PRESSURES, DISPLACEMENT, AND BENDING MOMENT by the Finite-Element Method. Studenterlunden, Oslo. (After Eide, Aas, and Jøsang. By permission of the Sociedad Espanola de Mecanica del Suelo)

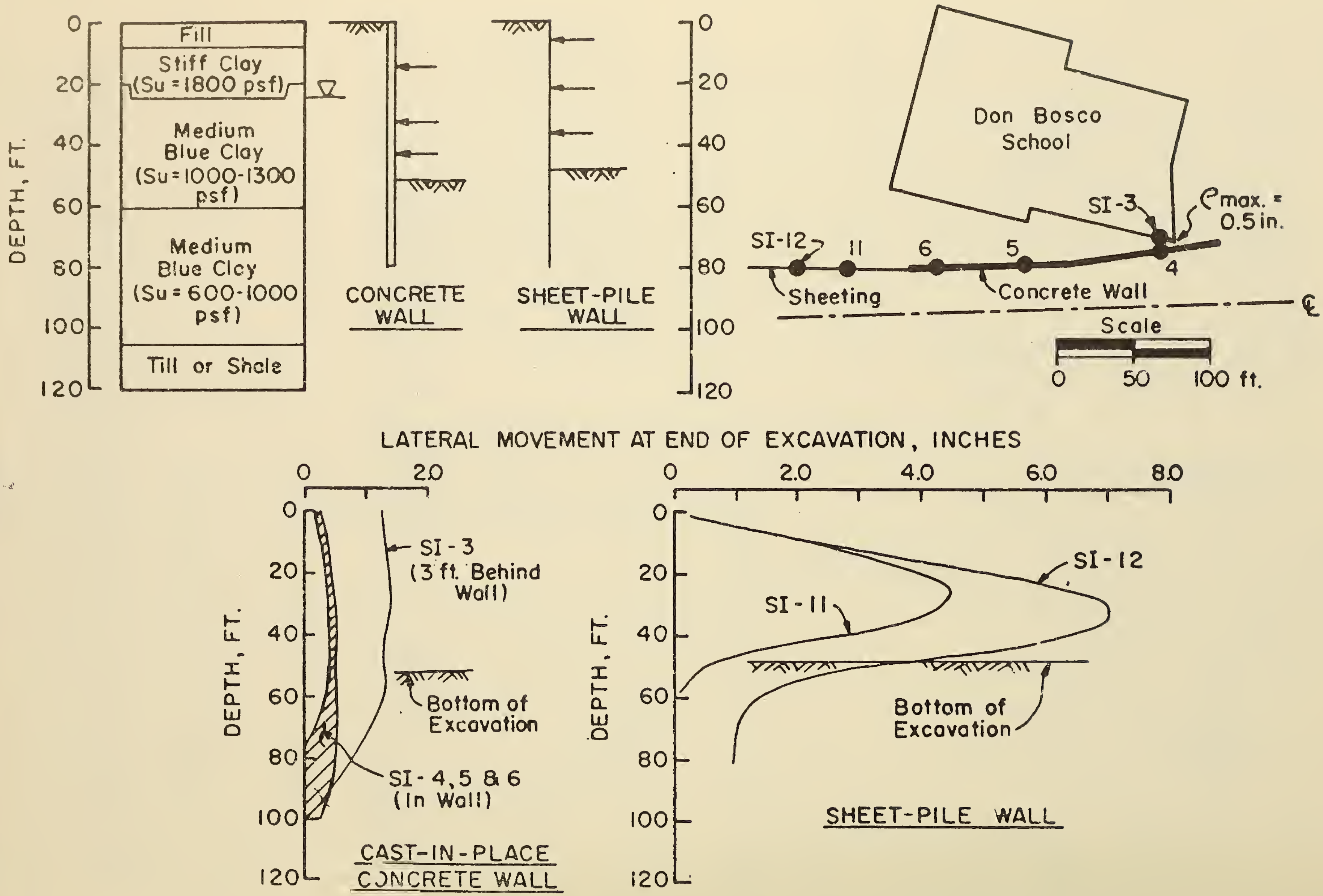


FIGURE 80. COMPARISON OF WALL DEFLECTIONS in Boston, Massachusetts for a cast-in-place concrete wall and a sheet-pile wall at nearby sections of excavation in medium clay. (After D. J. D'Appolonia, and with permission)

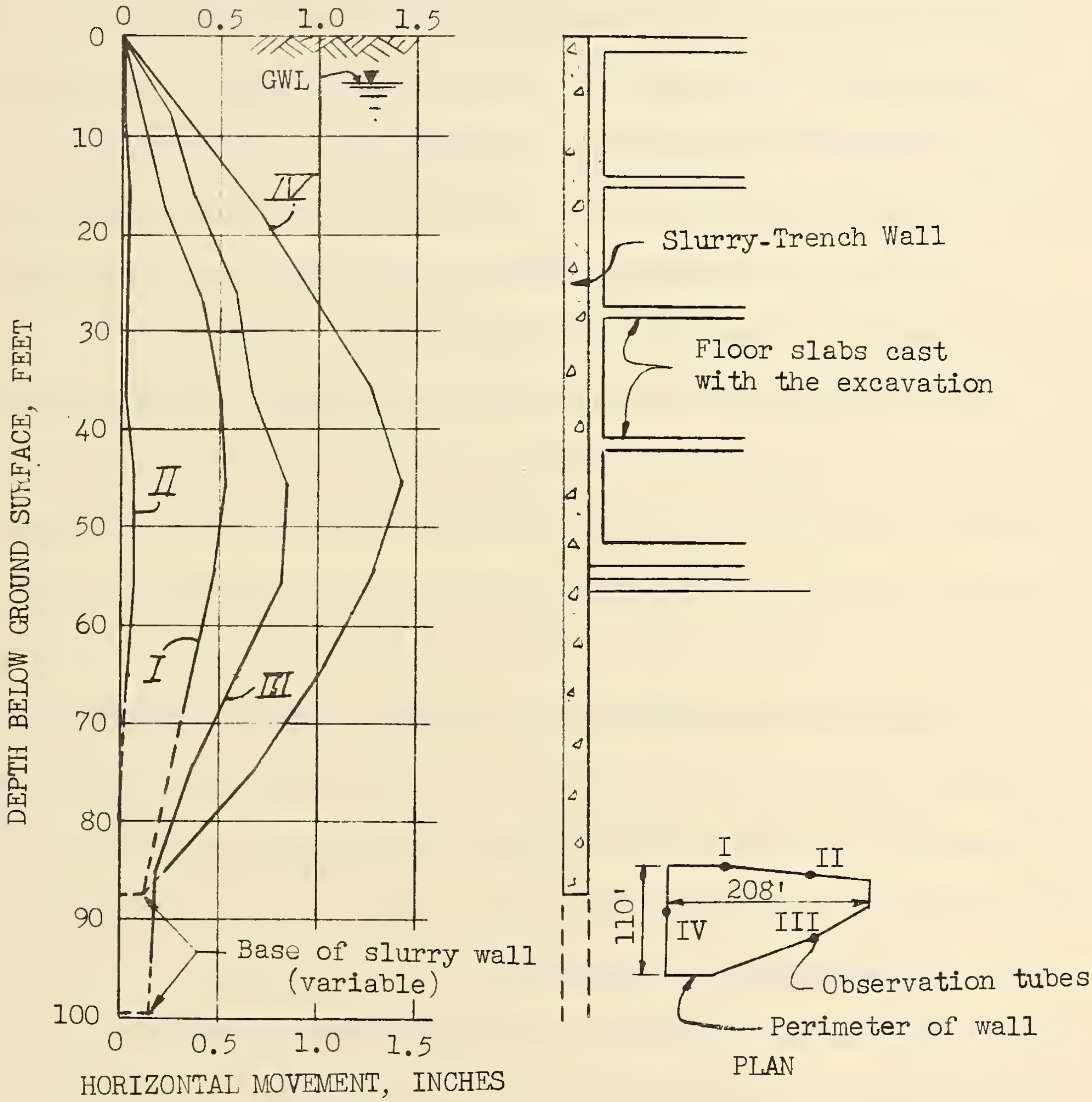


FIGURE 81. OBSERVED VARIATIONS IN DEFORMATION OF AN 80-CM THICK SLURRY WALL caused by different soil conditions. The wall was reinforced with 67-kg steel/m^2 (130 lbs/sq yd). Tubes I and III were in a decomposed moraine, tube II in a sound moraine, and tube IV in a lacustrine silt deposit. (After Huder. By permission of the Sociedad Mexicana de Mecanica de Suelos, A.C.)

in this respect and should not be allowed to get below 7.0. European experience is different, however. A reduction of 30 to 50% has been observed for horizontal reinforcement, and 10 to 30% for vertical. It has been proposed to use in design practice only 2/3 of the normally-allowed bond-strength for vertical reinforcement, and to design all horizontal reinforcements as closed loops. The splice lengths should be taken to 1.25 to 1.50 of the normally-used length.

The walls can be given different configurations in plan for increased stability, rigidity, and load-bearing capacity. Transverse walls can be cast below the future excavation bottom to increase the safety against bottom heave by providing a larger wall surface for the clay to adhere to. These walls may have to extend above the excavation bottom for deep excavations in soft clays. They can then be removed as the floor is poured.

Novel techniques involve prebending the walls with prestressing tendons before the excavation is made. The prebending reduces the wall deflection toward the excavation and, accordingly, reduces soil movements outside the excavation.

2. Development of the Technique

The earliest use was in the 1950's when digging was done by a cable-suspended clamshell weighing about 1-1/2 tons. The clamshell was attached to a Kelly bar to improve the production rate, and this had the main advantage of allowing extra weight to be added to the rig at the top of the Kelly bar. Some manufacturers have now gone to a heavier cable-suspended mechanical clamshell weighing 5 to 8 tons. As the weight increased so did the production rate. A still newer development uses a hydraulically-operated clamshell which, with good soil conditions, has even further improved the production rate.

The relative advantages of the cable-suspended method versus the Kelly-bar-supported-clamshell method are disputed by several of the manufacturers. The arguments for the cable-suspended method are that the cable forms a plumb line, maintaining the wall alignment; the digging depth is unlimited with a cable whereas a Kelly bar is limited to about 120-ft depth; and when a cable-suspended clamshell hits an obstruction it can work off alignment to get around it and then return to a plumb line. The arguments for the Kelly bar are that the alignment of a clamshell mounted on a rigid bar can be accurately controlled; an obstruction will not drive it off line; and greater weight can be applied to the Kelly bar without requiring larger and larger weights down the hole.

The development of percussion and circulating-type rigs for cutting through hard areas, rock, and boulders paralled the use of clamshell rigs. Bentonite is pumped down through the rig and the cuttings are pumped up to the surface in the returned bentonite. Both direct- and reverse-circulation types have been used.

The bentonite slurry was at first used only once and wasted; the current practice, however, is to reuse the slurry in a closed system. There is no need to continuously recirculate slurry; each recirculation and desanding results in about a 20-percent loss, however, so generally about five reuses are possible. The present cost of bentonite is about $15 per ton FOB mine in Wyoming, and $40 per ton FOB New York.

The changes that have been made to date have been directed at increasing the production rate; using labor that does not have to be specially trained; and having equipment available (chamshells, percussion rigs, circulating rigs, churn drills) that will perform under a variety of soil conditions. Most rigs today require only an operator and an oiler per rig, with a supervisor for the overall operation.

Representative costs in the eastern seabord area for up to 70-ft depths and over 4,000-sq-ft areas (including trenching, slurry, concrete, and reinforcing but no bracing) run about $13-15/sq ft in good granular or cohesive soil suitable for clamshell excavation; $18-20/sq ft in hard formations with boulders and some percussion work; and $60-100/sq ft in formations with embedded boulders requiring all-percussion work and churn-drilling the rock seats. The cost rises about 10 percent for each additional 50-ft beyond the 70-ft depth. Increasing the wall thickness to 3-1/2 from 2-1/2 ft has little effect on the costs since the greater production rates possible with the larger and heavier rigs offsets the rise in material costs. The indications are that recent slurry-trench jobs in New York and Washington, D.C., were bid at just over $13 per sq ft.

The main factors making a slurry-trench wall economical are eliminating the need for underpinning; using the wall as part of the permanent structure; and using the wall to solve a water problem.

3. <u>The Milan Method</u>

The so-called Milan Method is a special application of slurry-trench construction techniques to cut-and-cover tunneling which was developed in Italy. The steps in the method are as described below and shown in Figure 82.

a. Excavate a trench along the wall for constructing the guide walls for the excavation equipment. The guide walls are commonly about 5-ft deep and are spaced equal to the width of the final wall thickness. This construction is done in-the-dry.

b. Excavate the trench for the full depth of the wall, using bentonite slurry for wall support. Place the reinforcement and the concrete.

c. Excavate between the walls down to an elevation corresponding to the underside of the roof, installing lateral bracing as needed. Cut keys into the walls for roof support. Place a leveling-slab of lean concrete (mud sill) on the excavation bottom if a smooth roof underside is desired.

Backfill can then be placed on the roof and the utilities and traffic restored. For large spans, a center row of columns may have to be constructed before the backfill is made.

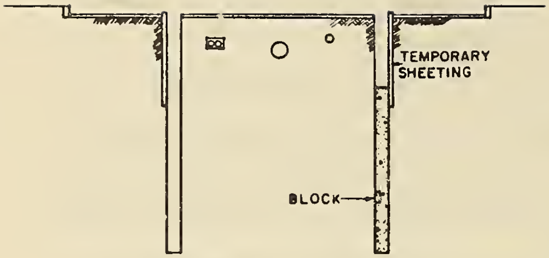
d. Continue excavation under the roof and between the earth retaining-walls.

e. Construct the invert and complete the tunnel structure.

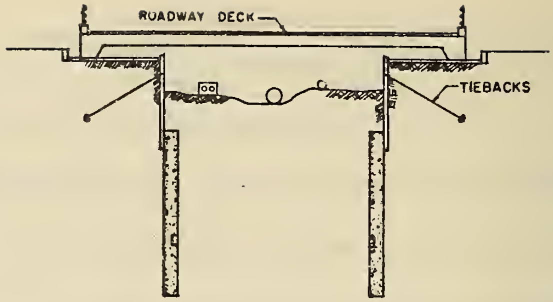
4. <u>Cast-In-Place Slurry-Wall Methods</u>

a. <u>Soletanche Method</u>

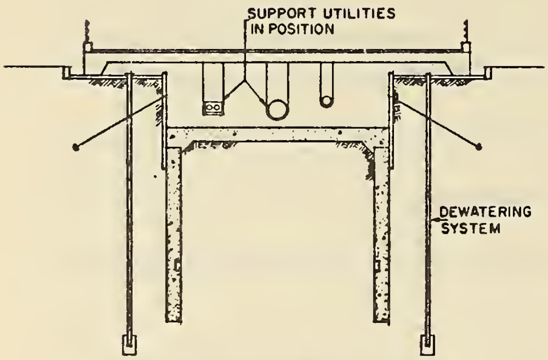
In this method, which was developed by Soletanche of Paris, the walls are constructed in .5- to 2-m thicknesses and up to 50-m heights. An oblong grab-bucket equal to the width of the trench is used to excavate material, at first simply suspended from a cable, and later with a Kelley bar attached to maintain alignment as the



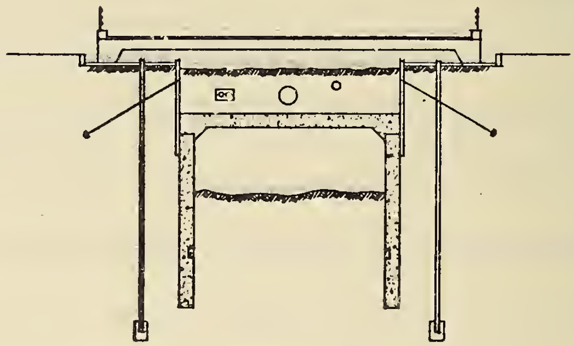
1 EXCAVATE WALL TRENCHES
PLACE CONCRETE SIDE WALLS



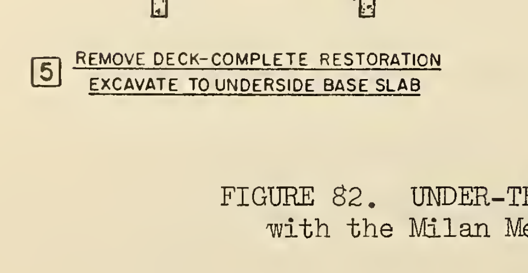
2 EXCAVATE TO DEPTH OF UTILITIES
PLACE DECK IN POSITION



3 EXCAVATE TO UNDERSIDE ROOF SLAB
PLACE CONCRETE ROOF SLAB



4 BACKFILL OVER ROOF SLAB
EXCAVATE UNDER ROOF SLAB



5 REMOVE DECK-COMPLETE RESTORATION
EXCAVATE TO UNDERSIDE BASE SLAB

FIGURE 82. UNDER-THE-ROOF CONSTRUCTION
with the Milan Method

depth increases. This method is suitable for soft or medium-dense ground with no boulders or other obstacles.

For hard ground, Soletanche uses a slurry circulation-type cutting-machine with a cutting-head diameter equal to the trench width, see Figure 83. Horizontal movement of the machine cuts the slot. Slurry is pumped through the tool, while both slurry and excavated material are sucked up through the tool. The cuttings and slurry are separated and the slurry is returned to the hole.

b. <u>ICOS-Icanda Ltd</u>.

An example of this construction method is the World Trade Center in New York, as shown in Figure 84. Three-ft-thick walls, from 50- to 70-ft tall were cast in 22-ft-long sections. The material was excavated by a clamshell bucket, except where obstacles required using churn drills. The churn drills were also used to cut a seat into the rock to anchor the bottom of the walls. When the excavation was complete a de-sanding machine (operating like a vacuum cleaner) was used to clean the bottom of the hole. When an overbreak occurred, that segment of the trench was filled with a one-bag-mix concrete and the trench was redrilled. The slurry specific gravity was 1.05.

The reinforcing cage was built at ground level, and rollers prevented fouling and gouging the sidewalls of the trench as the cage was lowered into place. These rollers were 7-inch-diameter precast concrete discs, strung on the reinforcing bars on 4-ft centers each way. 4,000-psi concrete was then tremied through the center of the cage. The end-bulkheads for each section were 3-ft-diameter steel pipes which were removed after the concrete had set. Styrofoam plugs

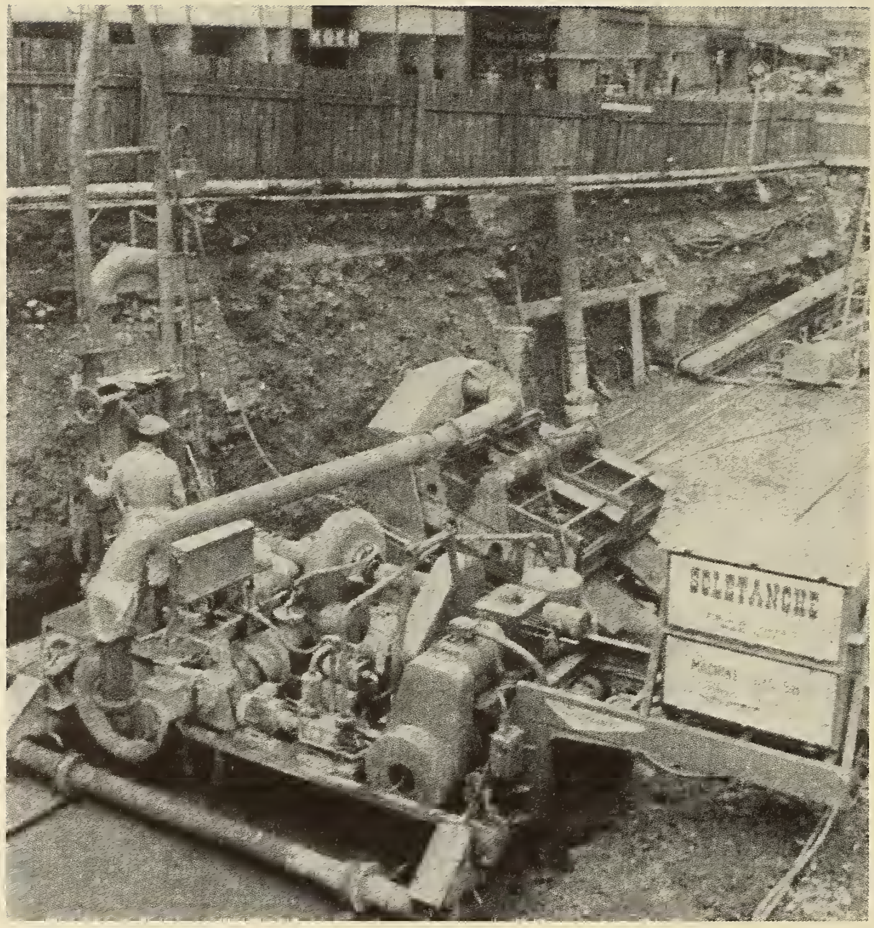


FIGURE 83. MONTREAL, CANADA, Slurry-Trench Excavation by the Soletanche method in shore-less excavation separates cuttings and recirculates bentonite slurry. (From Engineering News-Record, April 26, 1962, p. 100, by permission.)

FIGURE 84. WORLD TRADE CENTER, NEW YORK, Slurry-Trench Excavation Using the ICOS-Icanda Method. (By permission of ICOS Corporation of America.)

and castings (used for tie-back anchorages) were set and tied in position on the rebar cage before the cage was set in place. ICOS presently uses internal horizontal reinforcing to replace walers for wall bracing. Figures 85-87 show the ICOS equipment.

c. Tone Boring Company Method

Tone Boring is a Japanese company which is currently marketing slurry-trenching equipment in the United States. The trench is dug by either five or seven linearly arranged augers which are driven by two submersible motors. The excavated material is removed to a hopper by reverse circulation for separating the soil particles, so the slurry can be recirculated. Tone Boring claims to be able to maintain a vertical tolerance of 1/500 of the vertical depth. The total width occupied by the equipment is 16 ft. They recommend burying the tremie pipes at least 7 ft in concrete and that at least one tremie be used for every 11 ft of wall section, see Figures 88-91.

d. Soldier-Pile-Tremie-Concrete Method (SPTC)

The walls of the BART Civic Center Station were built by a method known as the soldier-pile-tremie-concrete method (SPTC), see Figures 92-94. Up to 100-ft-high walls were built as permanent load-carrying structural walls.

Steel wide-flange beams were set vertically in slurry-stabilized drilled holes. The slot between the flanges was excavated with a clamshell bucket under slurry, and concrete was tremied into the slot. The walls extended 20 ft below the bottom of the excavation, or into clay layers, and construction was then done by the inverted method.

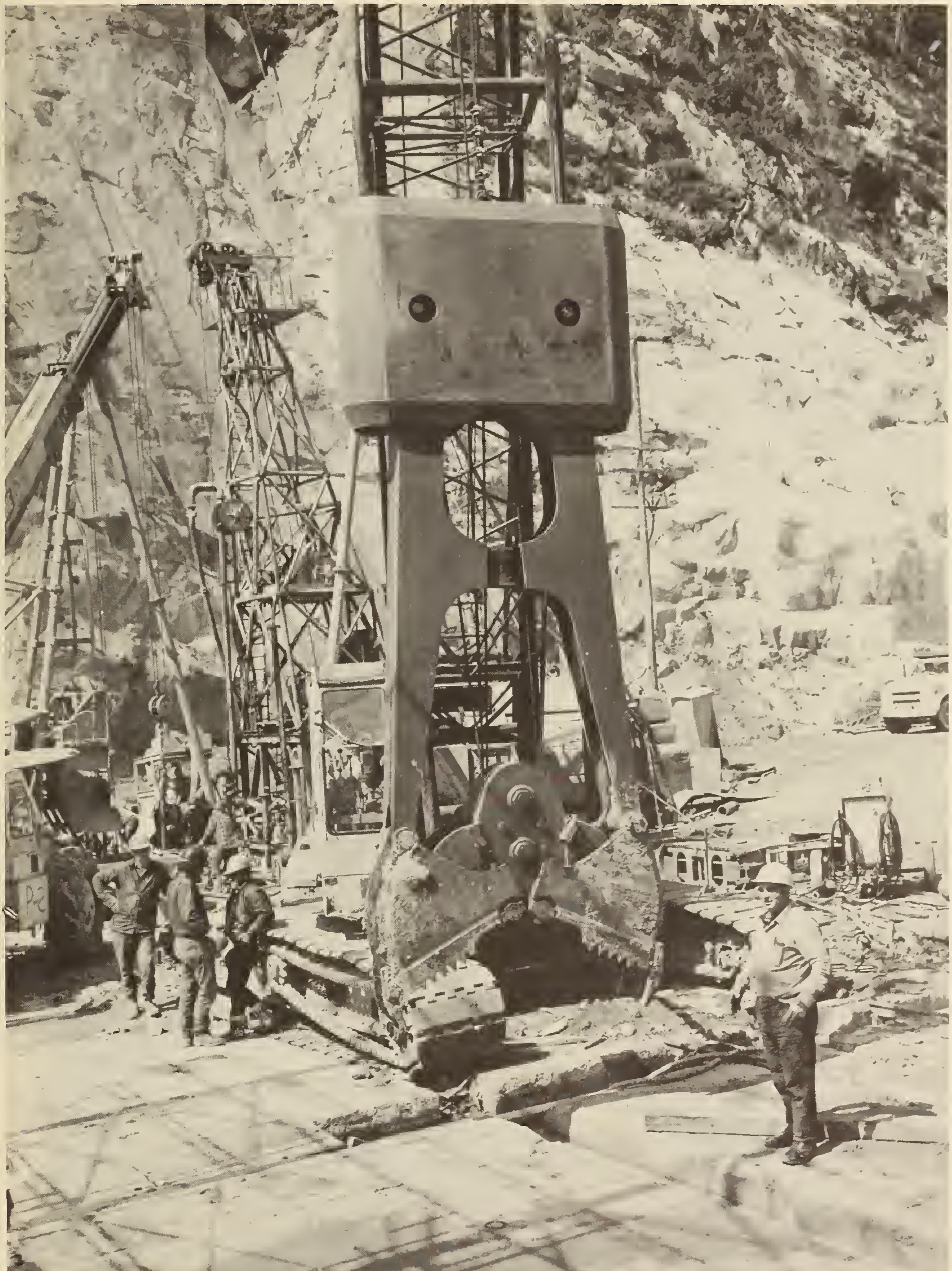


FIGURE 85. ICOS CORPORATION MECHANICAL CLAMSHELL. (By permission of ICOS Corporation of America.)

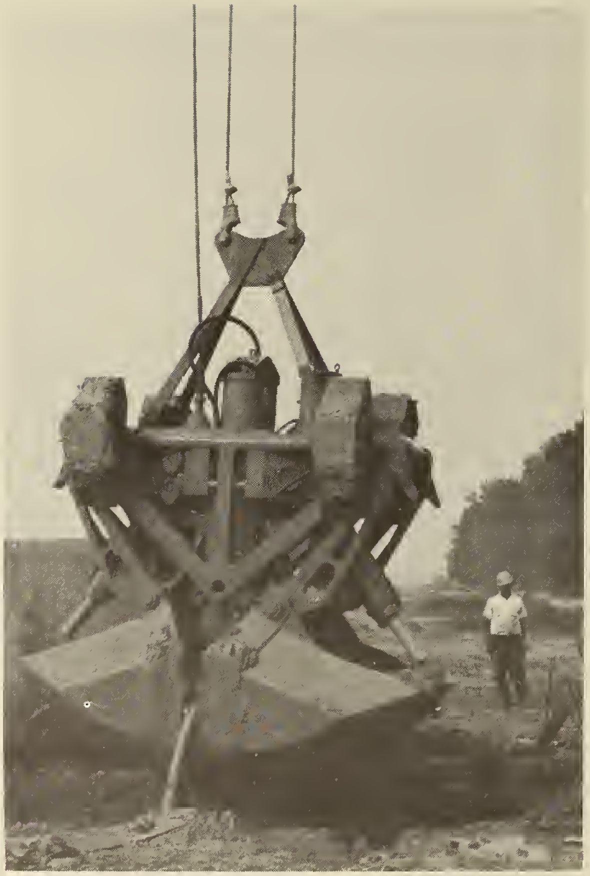


FIGURE 86. ICOS CORPORATION HYDRAULIC CLAMSHELL.
(By permission of ICOS Corporation of America.)

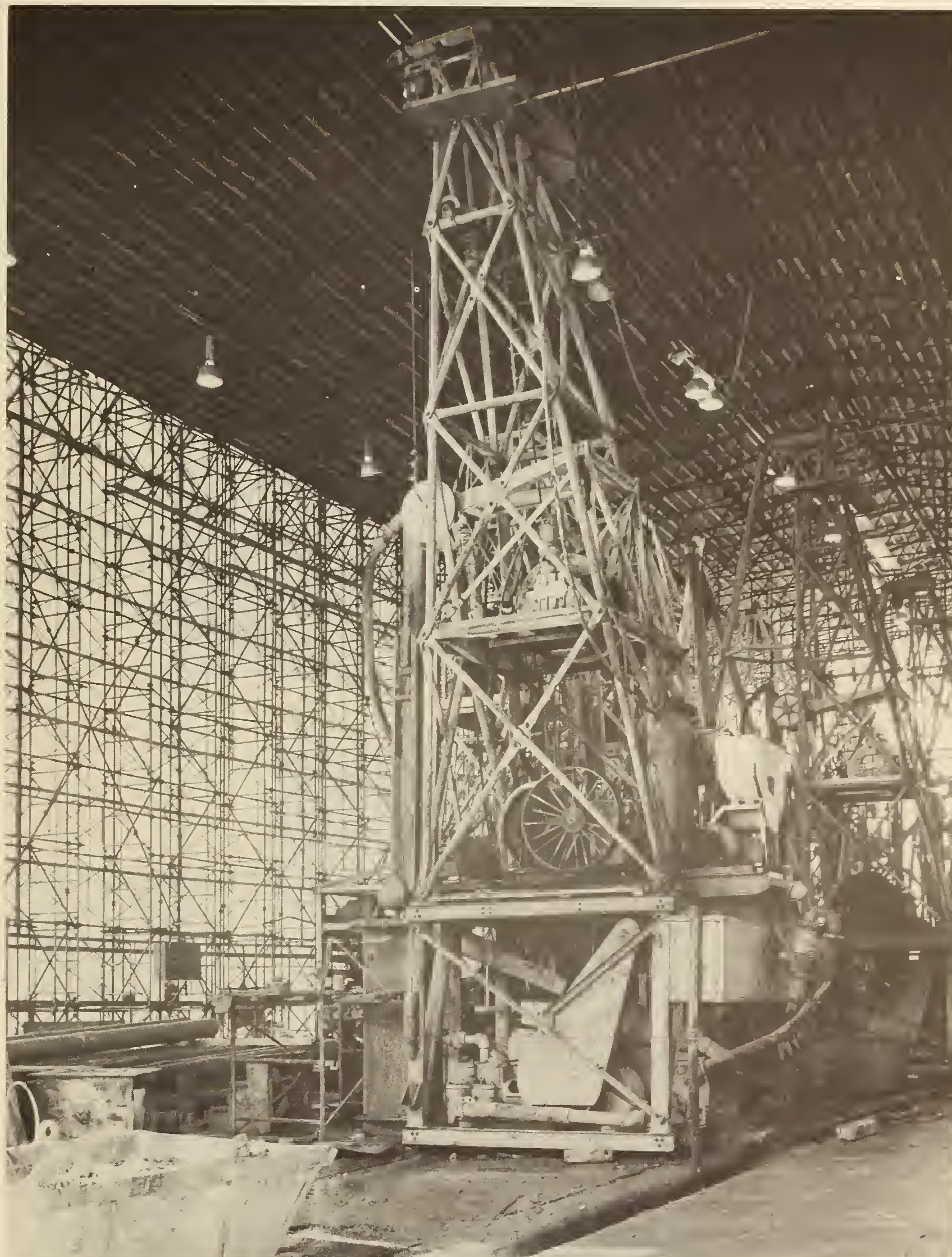


FIGURE 87. ICOS CORPORATION PERCUSSION RIG. (By permission of ICOS Corporation of America.)

FIGURES 88-89. TONE BORING CO. MULTIPLE-AUGER RIG. (By permission of Tone Boring Co.)

FIGURES 90-91. MULTIPLE-AUGER RIG AND EXPOSED SLURRY WALL. (By permission of Tone Boring Co.)

FIGURE 92. BART, SAN FRANCISCO, Soldier-Pile Tremie-Concrete Method (SPTC), Soldier Beams with Driving Shoes. (From Engineering News-Record, July 11, 1968, p. 44, by permission.)

FIGURE 93. BART, SAN FRANCISCO, SPTC METHOD. Backfilling a vertical H-beam.

FIGURE 94. BART, SAN FRANCISCO, SPTC Method. H-beam drilling-guide template.

(Both pictures from Engineering News-Record, July 11, 1968, p. 44, by permission.)

A detailed construction procedure is as follows:

1) Excavate a 12-ft ± deep trench to clear utilities and contain slurry.

2) Drill oversize slurry-stabilized holes and set soldier piles (24WF at 11'-8" c/c), and tremie grout.

3) Excavate slurry-stabilized slot, set intermediate pile (5'-10" c/c), and tremie concrete.

4) The pile position-tolerance was zero inside, 12 inches outside, and 6 inches longitudinally along the wall.

5) The measured horizontal wall deflection was 1-1/4 inches; the average settlement adjacent to the excavation was 1/2 inch. Soils were generally sands and clayey-silts.

Figure 95 gives production rates for some of the methods, and Figures 96-99 show construction examples.

5. <u>Grout Diaphragms</u>

A special use of the basic method of slurry trench-construction is to build thin grout walls, primarily for use as ground-water cutoff walls. Instead of removing the soil with a bentonite slurry and displacing the slurry with concrete, the soil is broken up and mixed in the trench with a cement and bentonite grout, which is then left to harden.

Six- to twelve-inch-thick cutoff walls can be constructed to 80 ft or greater depths in loose soils using this method. Two construction methods are described below.

a . <u>I-Beam Method</u>

An I-beam is driven into the soil by either percussion or

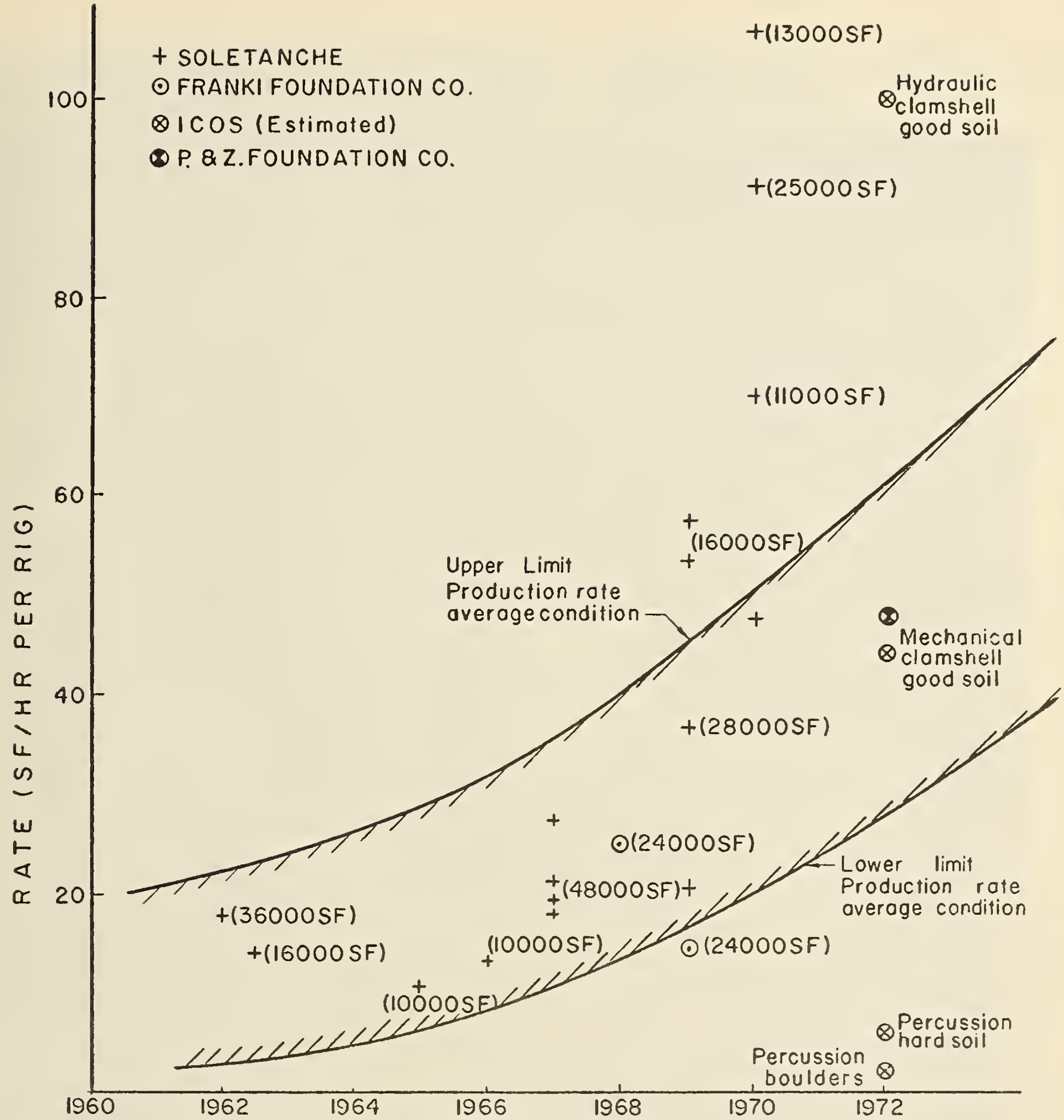


FIGURE 95. AVERAGE PRODUCTION RATE, Slurry-Trench Walls

FIGURE 96. EXCAVATION BETWEEN SLURRY WALLS, Milan,Italy. (By permission of ICOS Corporation of America.)

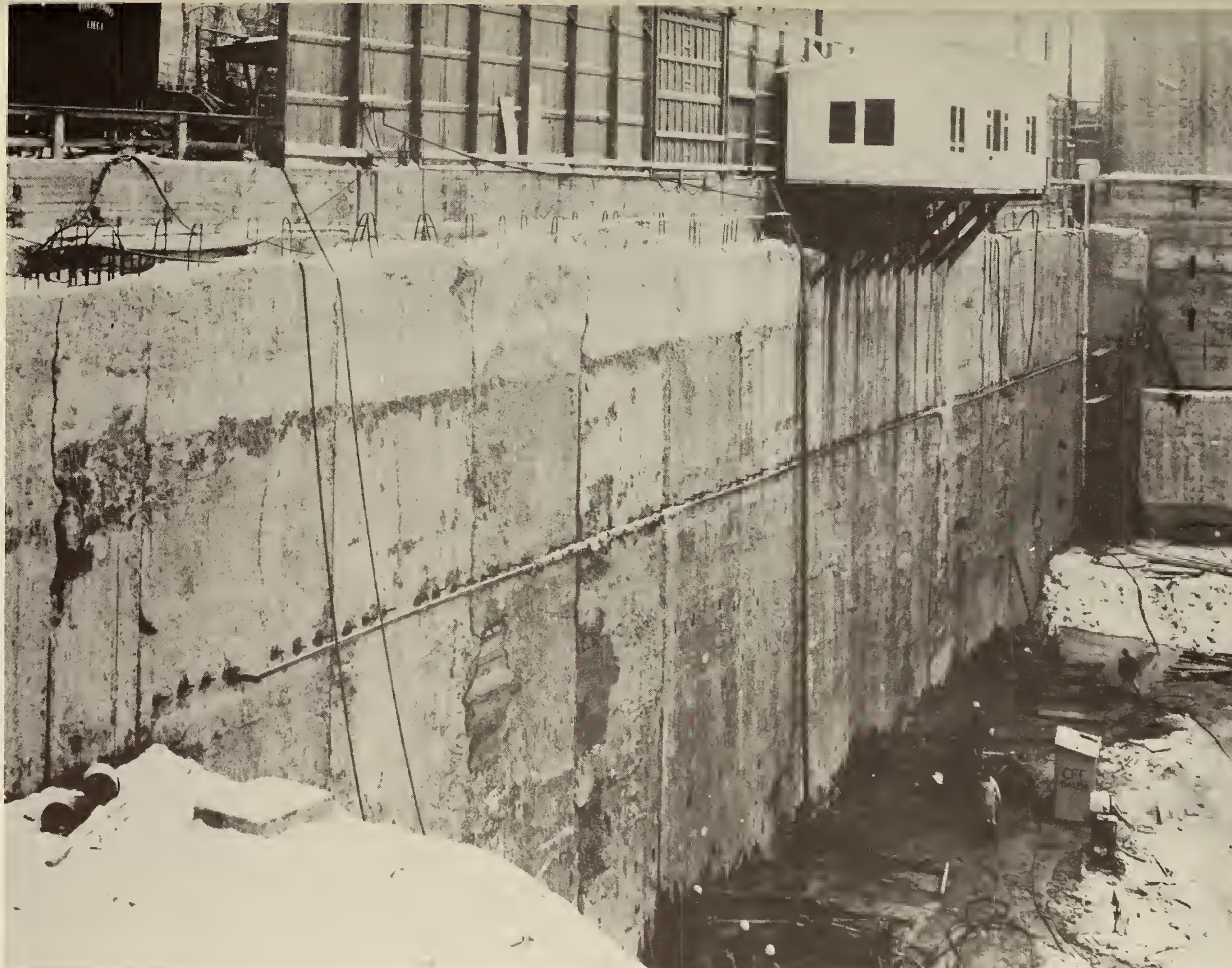


FIGURE 97. EXPOSED SLURRY WALLS, Brussels, Belgium, 20-m deep. (By permission of Franki Foundation Company, Boston.)

FIGURE 98. EXPOSED SLURRY WALLS, Brussels, Belgium. Note the tiebacks. (By permission of Franki Foundation Company, Boston.)

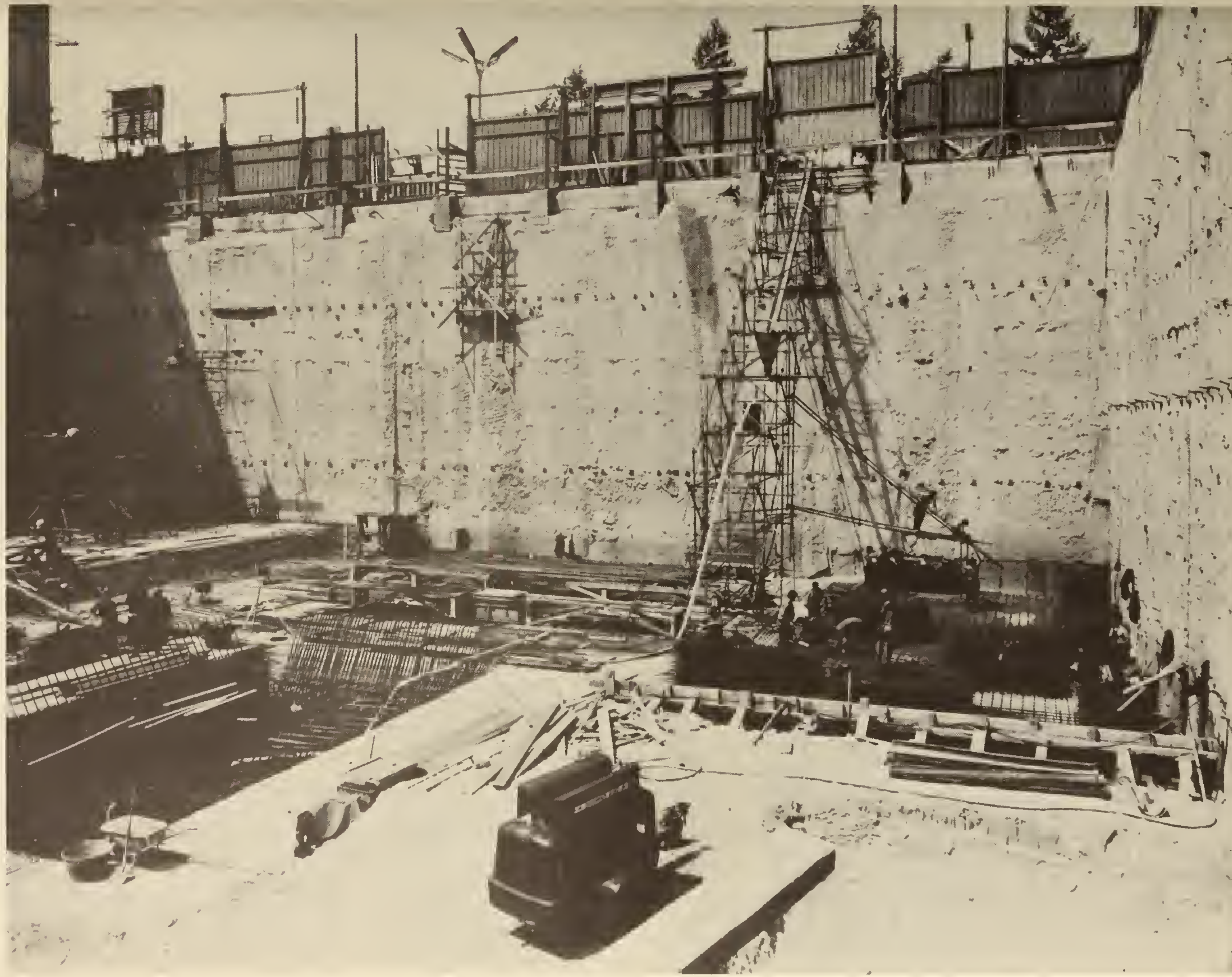


FIGURE 99. EXPOSED SLURRY WALLS, Brussels, Belgium. (By permission of Franki Foundation Company, Boston.)

vibratory hammers, while grout is simultaneously injected. The web of the I-beam is lined up with the line of the cutoff wall. The beam, about 3 ft deep, is withdrawn and driven in an adjacent position along the line of the wall with a 6-inch overlap.

b. Saw Method

A 6-inch-wide slot is cut by a chain saw to a maximum depth of 30 ft. The saw breaks up the soil and mixes it with grout which is simultaneously injected. This method is suitable for fine cohesionless soils. A saw of this type was used to cut a 22-ft-deep slot at 30 fph at the Dallas-Ft. Worth airport. The slot was not grouted, however.

6. Precast Slurry Walls

A frequently used method in Europe (developed and patented by Soletanche of Paris) is to insert prefabricated, precast, or prestressed elements into a trench dug and filled with a special type of slurry. The trench is dug in the same manner as for cast-in-place slurry walls, see Figure 100.

The prefabricated elements are inserted into a special sealing grout that differs from the normal bentonite slurry used in digging the trench in that it contains cement. The grout must remain fluid during the excavation and until the elements are lowered into the trench and suspended in position. The panels are generally inserted within one day of the drilling, although about two days maximum time is possible. The grout must then attain sufficient strength to permit the adjacent trench to be excavated without any displacement of the first panel, and yet the strength must not increase too

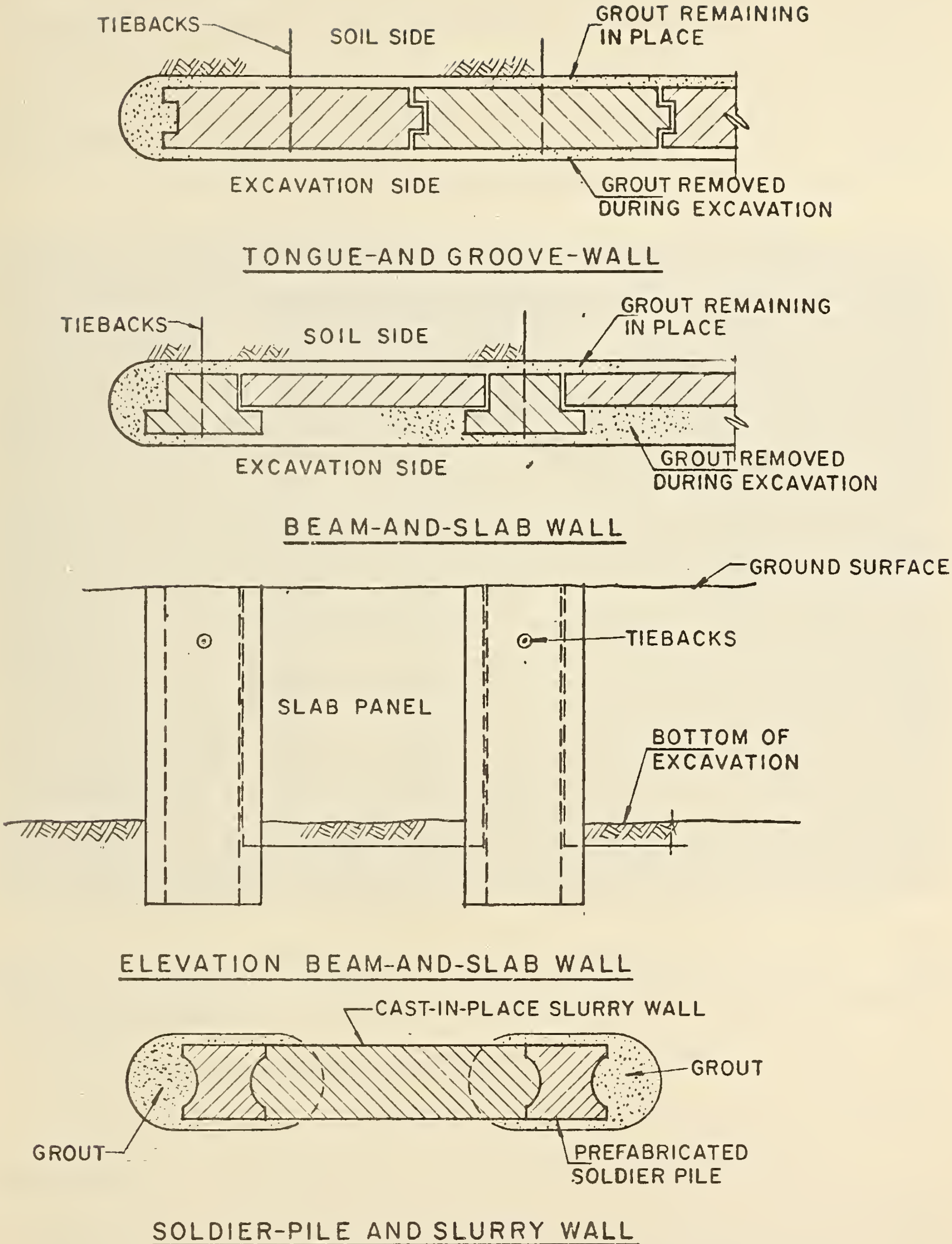


FIGURE 100. PREFABRICATED ELEMENTS IN SLURRY TRENCHES

rapidly so that the grout may be excavated for cleaning the joints. Generally, not less than five nor more than ten days are necessary before drilling for adjacent panels and the proper cleaning of the joints. After ten days the grout gradually increases in strength and equals or exceeds the strength of the soil around it.

The prefabricated elements are of various shapes, including tongue-and-groove panels which act like sheet piles; T-shaped beams with slabs between them; and I-shaped piles which act like soldier beams and may be used with a cast-in-place slurry wall or even conventional lagging. (See Figures 101 - 103.)

Whatever the shape, the prefabricated element has fittings for handling and for positioning by supporting it on top of the guide walls used to dig the trench. The elements may be designed for use with tie-backs, and may also have inserts for reinforcing bars and styrofoam pockets for beams.

The trench is excavated only a few inches wider than the panel, and the face of the panel to be uncovered receives a special coating to facilitate stripping off the grout as the excavation proceeds. The grout remaining on the soil-side contributes to the watertightness; however, a waterproofing compound can be incorporated in the concrete if necessary, or a waterproof membrane can be placed on the soil-side and a waterproofing treatment applied to the joints.

The appearance of the wall is far superior to any other type of cast-in-place trench wall, and accurate positioning and alignment are readily maintained. These walls are generally incorporated into the completed structure.

FIGURE 101. WALL CONSTRUCTION BY THE SOLETANCHE METHOD in Paris. Precast elements follow in a slurry trench. (By permission of Soletanche Entreprise, Paris.)

FIGURE 102. EXCAVATION FOR BUILDING FOUNDATIONS USING PRECAST ELEMENTS IN SLURRY TRENCHES, Paris, France, 20-m depth. Note the use of anchored tiebacks.

FIGURE 103. JUNCTION BETWEEN PRECAST AND CAST-IN-PLACE SLURRY WALLS, both built in a slurry trench, Paris. Note the tiebacks and styrofoam-filled pockets for the concrete floorbeams.

Panel lengths of 24 m or longer are not uncommon. One building was observed where the depth of excavation was 20 m. The system seems to result in very little ground movement and overcomes many of the disadvantages of cast-in-place slurry walls.

7. Evaluation

Walls constructed by the slurry-trench method can be incorporated into the final structure. The installation can be made with little disturbance to the environment. Traffic can be restored in a comparatively short time, and backfill can be made before the excavation between the tunnel roof and the invert is begun. The walls effectively cut off groundwater flow, and can be extended below the excavation to reduce seepage through the bottom.

D. CONTINUOUS BORED-PILES (Secant Piles)

This method of building ground-support walls consists of a series of overlapping circular cast-in-place concrete piles, see Figure 104.

1. Construction Procedure

The holes for odd-numbered piles are drilled first, and concrete having a controlled setting time is placed. The even-numbered piles are drilled while the concrete in the odd-numbered holes is still green, and overlap and cut into the previously-cast piles.

The normally-used pile diameters are from .8 to 1.5 m, and the overlap provides a 50- to 60-cm thickness at the intersections.

The drilling is done with a Benoto or other similar machine which forces a steel casing with a bottom cutting edge into the ground, using a downward pressure and a to-and-fro horizontal rotation, see Figure 105. A special grab-bucket removes the material inside the casing. The casing is normally left open, but it can be filled with bentonite slurry if the groundwater level requires. Concrete is placed as the casing is withdrawn with the same horizontal rotational motion, thus compacting the concrete and completely filling the hole.

2. <u>Design</u>

The piles may be reinforced, but the overlap requires using a square cage on the odd-numbered piles, while the even-numbered piles may have a round cage. Often only the round cage is used and only every other pile is reinforced. There is no practical way of providing longitudinal reinforcing for the wall.

Styrofoam blocks are sometimes attached to the reinforcing cage to provide a block-out to receive a cast-in-place bottom slab. The walls are adaptable to use with either tie-backs, or wales and bracing struts. Inclined bored-piles have been used in Munich, to also act as underpinning.

3. <u>Advantages</u>

Proper vertical alignment and horizontal positioning can be accurately maintained when the drilling rig is mounted on rails. For instance, in the drilling on the Piccadilly line extension to Heathrow Airport in London, (see Paragraph 6a, below) the center-to-center distances were reduced by 1 inch to provide a greater secant distance and improve the watertightness.

The method is suitable for most types of soil, and the cutting edge of the casing will go through brick, rubble, and small boulders.

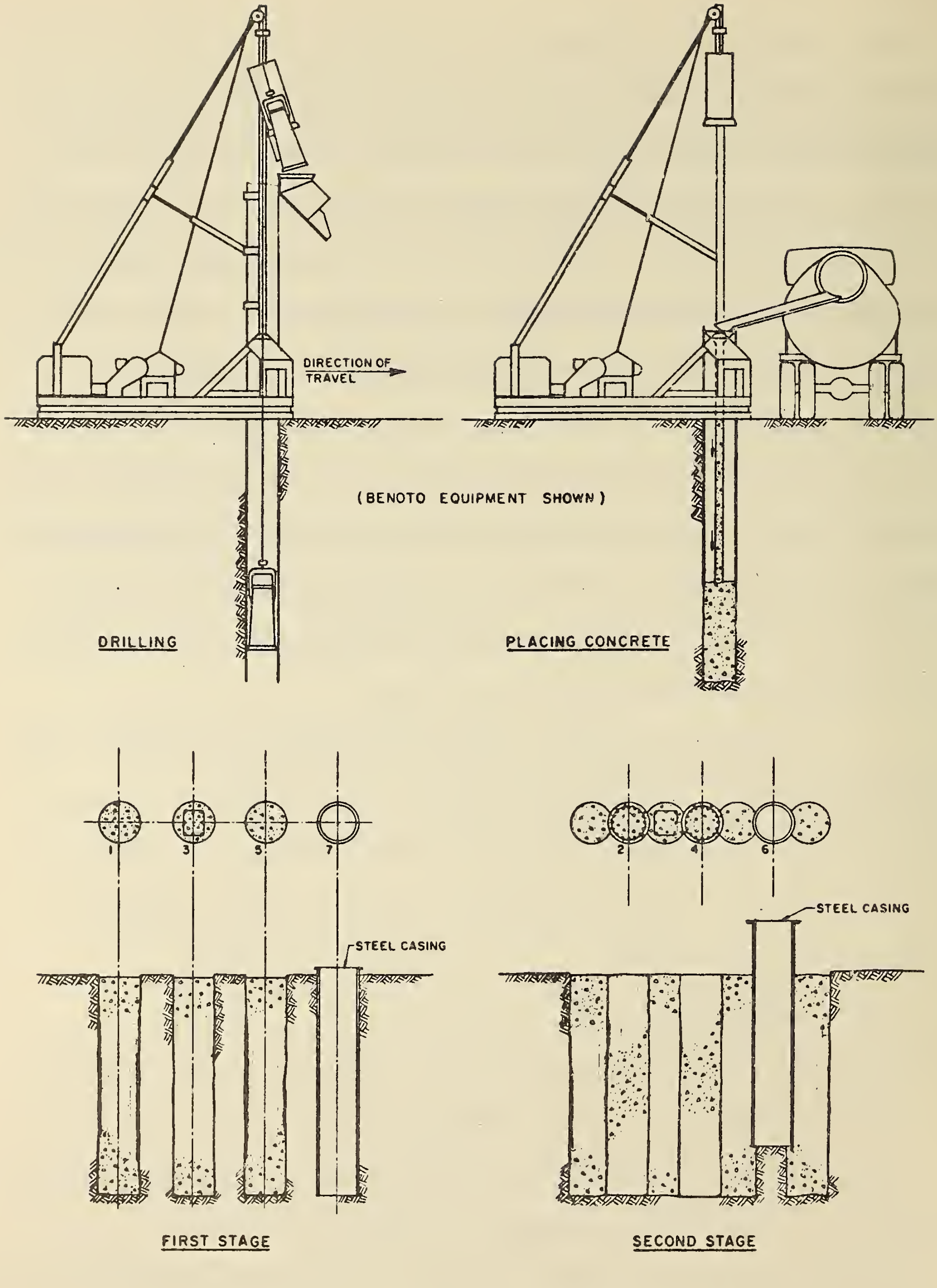


FIGURE 104. CONTINUOUS BORED-PILES

FIGURE 105. BENOTO RIG DRILLING HOLES, London, England.

There is no risk of wall collapse if an abandoned pipe or sewer is hit, such as is likely to occur with slurry walls when the slurry is lost. If drilling in a slurry is not required, each pile can safely be inspected by lowering a man to the bottom.

4. Disadvantages

The depth may be limited because of the resistance caused by sinking and raising the steel casing and maintaining the proper overlap. However, 40 ft or greater depths are usual, although continuous bored-piles have been sunk to 66 ft in Helsinki. The steel casing has a tendency to warp in very hard soils, and the casing can rupture. Reports indicate the walls also have a tendency not to be completely watertight, although several examples indicate this is not a problem.

5. Evaluation

The system is relatively quiet and clean, but it requires excellent maintenance and skilled operators.

All the methods are patented, which may tend to restrict the competition in bidding.

The rate of progress with this method under normal conditions is 5 to 6 piles per-machine per-day (up to 30 ft ± in depth). The drilling rig advances forward and requires a minimum 16-ft operational width. Drilling, material removal, and concreting can be a linear operation. The rig can also be turned perpendicularly to the wall line, and piles can be drilled practically flush against a building. This is a much slower operation, however.

The relative cost is somewhat difficult to evaluate, but it is generally conceded to be more expensive than slurry walls, and it is probably about 25 percent more costly than soldier piles and lagging.

6. <u>Examples of Use</u>

a. <u>Piccadilly Line Extension to Heathrow Airport, London</u>

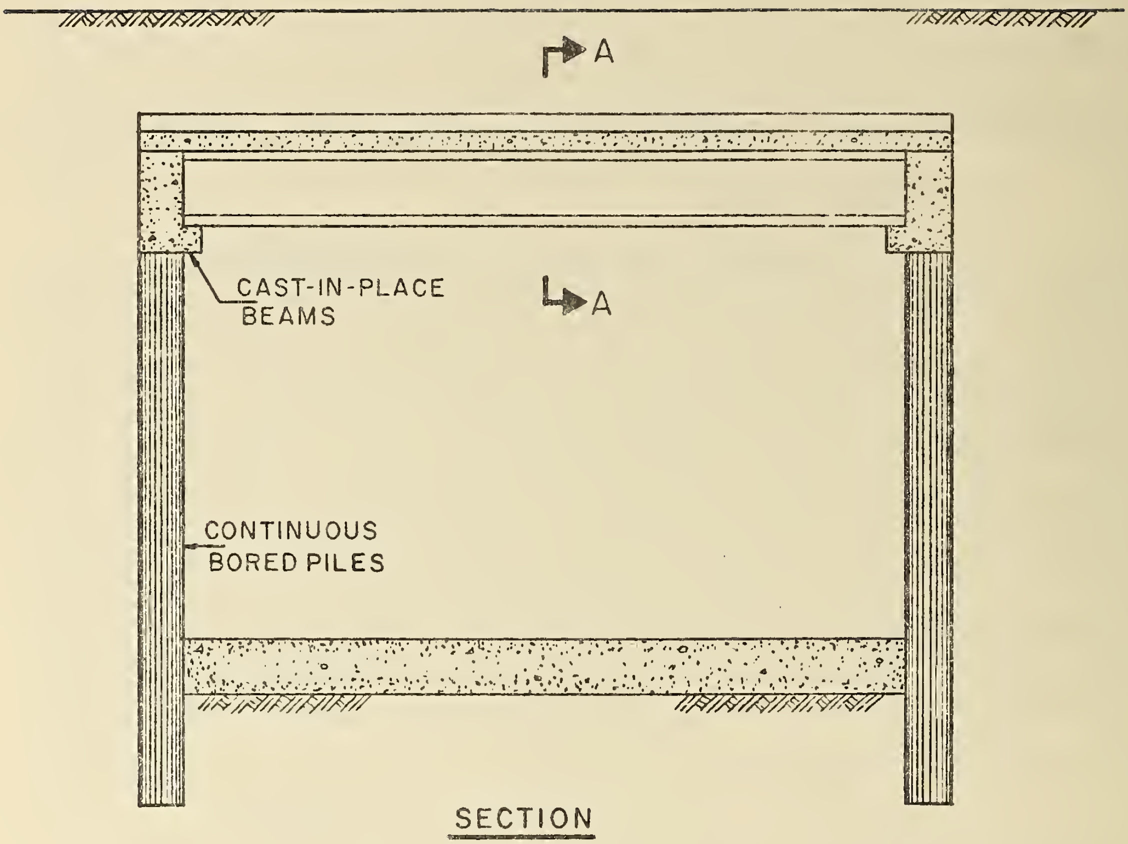
Four drill rigs were used to construct a two-track shallow cut-and-cover subway extension, see Figures 106 and 107. Continuous bored-piles were cast to within about 3 ft of the subway roof, leaving the reinforcing steel protruding. A continuous cast-in-place beam was constructed on top of the pile wall, with a shelf to receive the pre-cast, pretensioned inverted-T roof-beams. Concrete was then placed over the roof beams, and construction proceeded under-the-roof with no additional bracing for the pile wall. The base of the tunnel was below the water table but the area was practically dry. The wall was left exposed in the line sections, but finish-concrete walls were used at the stations.

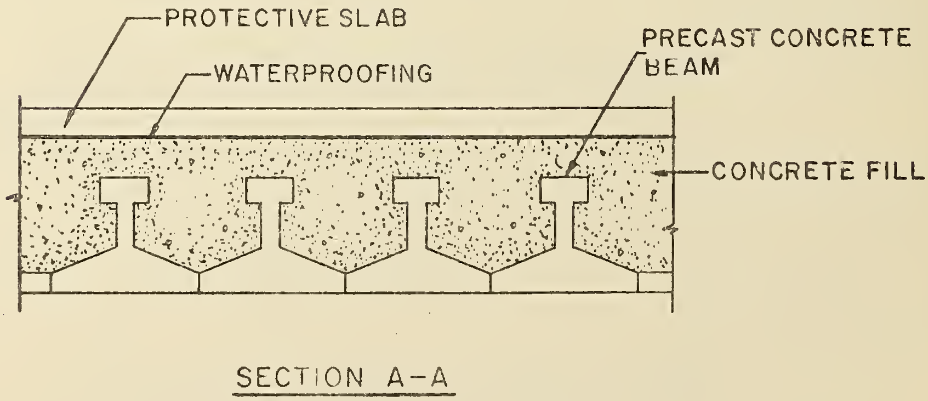
b. <u>Munich Subway, Goetheplatz Station to Harras Station</u>

Both vertical and inclined walls were used, see Figures 108 and 109. The double box-section of the subway was built inside the continuous bored-pile wall. Tie-backs, struts, and a combination of bracing were used.

c. <u>Munich-Tergernseer-Landstrasse Project</u>

Bored-pile walls were used for a depressed roadway having a cantilever roadway along each side. Tie-backs and a cast-in-place cantilever slab on top of the wall were used. Six rigs averaged six piles per-machine per-day.





PICCADILLY LINE EXTENSION
TO HEATHROW AIRPORT — LONDON, ENGLAND

FIGURE 106. PICCADILLY LINE EXTENSION to Heathrow Airport London, England.

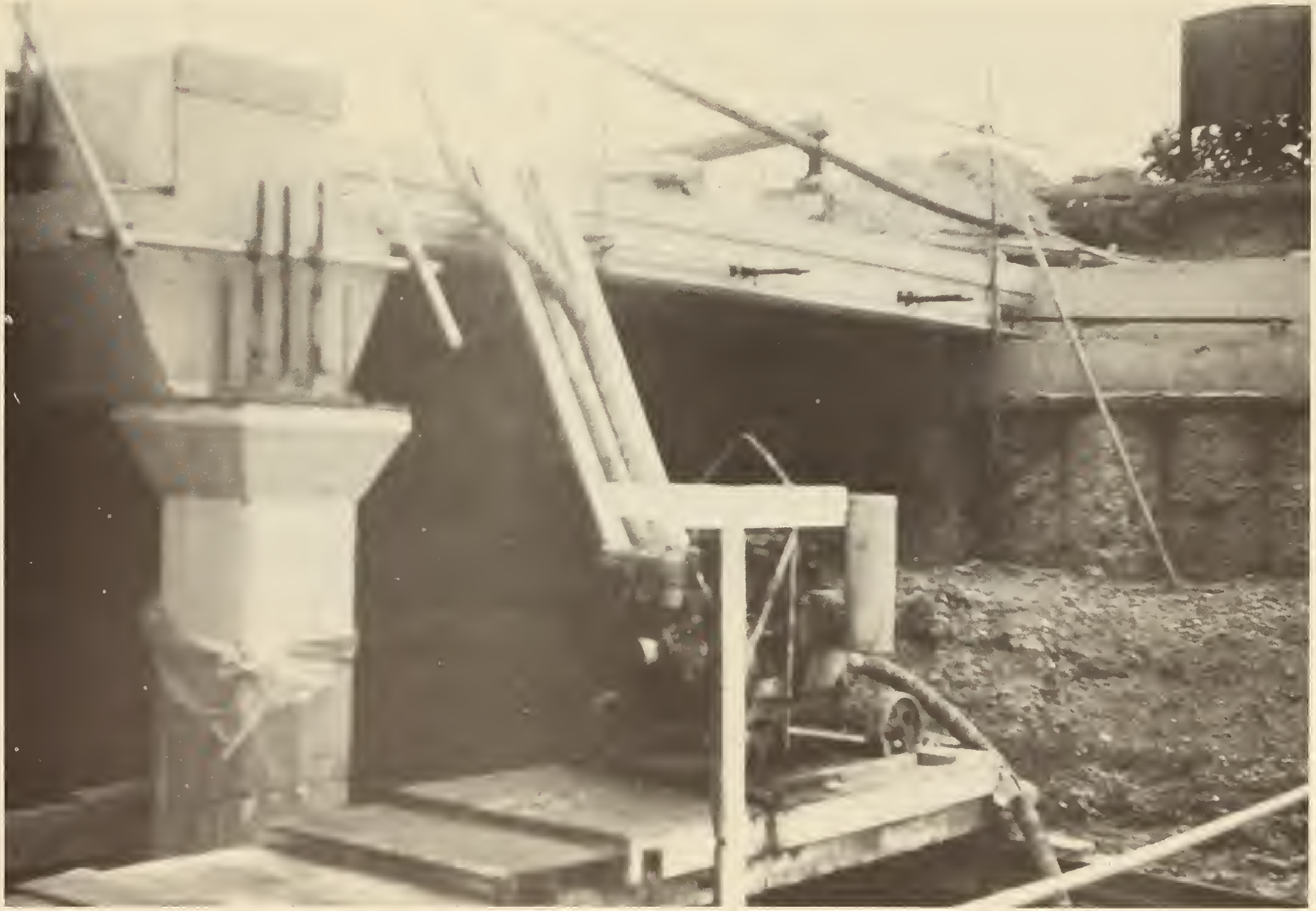


FIGURE 107. EXPOSED WALLS OF CONTINUOUS BORED-PILES, London, England. A precast roof structure post-tensioned longitudinally is also shown.

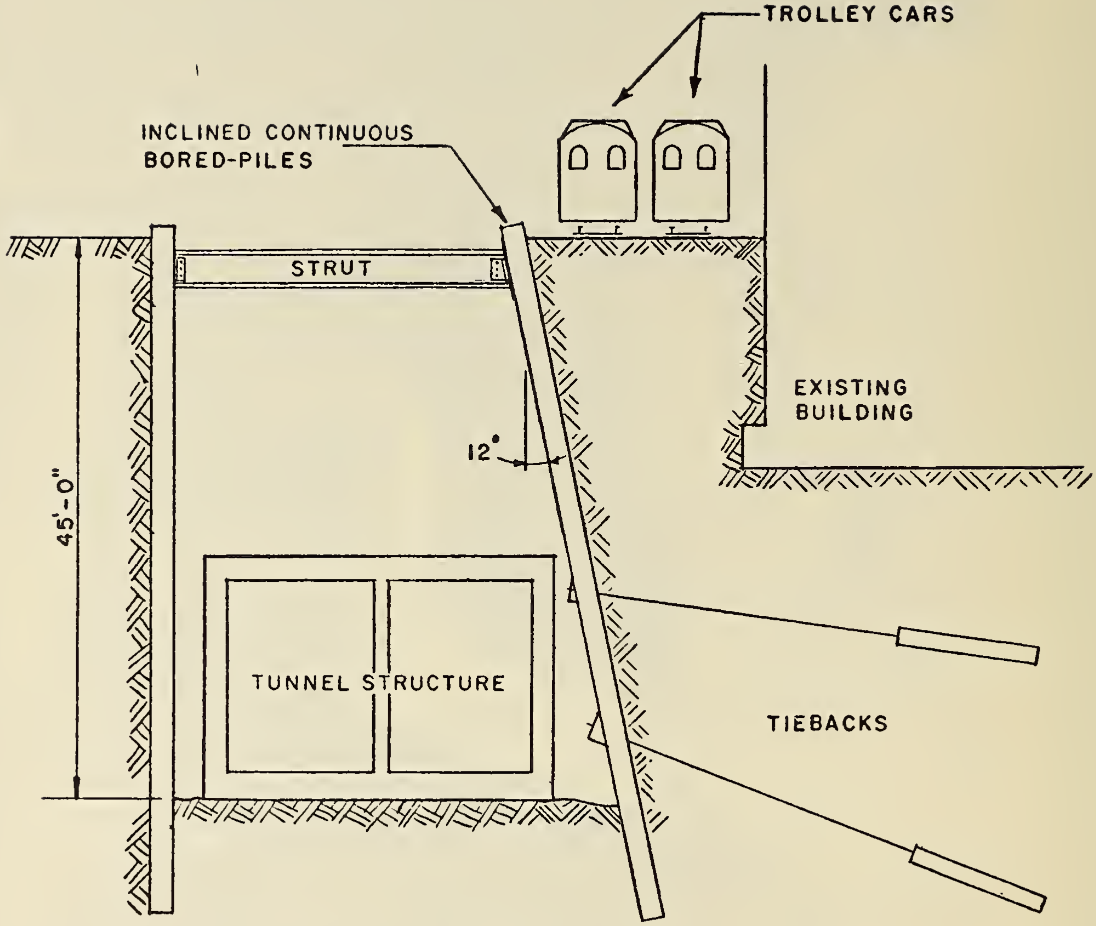


FIGURE 108. EXAMPLE OF INCLINED BORED-PILE WALL

FIGURE 109. MUNICH, GERMANY, INCLINED PILE WALLS protect a department store against a deep excavation. The wall is braced at the top, and it is tied back below by two rows of anchors grouted into the earth and prestressed. (From Civil Engineering, July, 1970, p. 67, by permission.)

d. Helsinki

The City Block Building was constructed using bored-piles reaching 66 ft to rock to make a watertight wall. Tie-backs and cast-in-place concrete wales were used.

E. CAISSONS

Some attempts have been made to construct building substructures as caissons to be sunk to their final position by internal excavation. One such example, in Mexico City, led to major disruption of the surrounding ground and to excessive settlements because the descending caisson exerted drag-down forces on the surrounding soil. Difficulties were also experienced in keeping the caisson vertical.

More recent examples have apparently met with considerable success. A five-story substructure was built above-ground in Tokyo in 1962, and then sunk into place by excavating beneath it. The cutting teeth were steel plates reinforced with angles and filled with concrete. Bentonite slurry was used to lubricate the outside of the walls, and progress averaged about 9 inches per day.

A seven-story garage was sunk in Geneva, Switzerland at about the same time. The techniques used were much the same, and the caisson-sinking averaged 8 inches a day. Again, bentonite slurry was used as a lubricant around the shell.

In Amsterdam, 98- to 131-ft-long subway sections were built on the surface and then sunk by tunneling underneath under compressed air. The tunnel is 30-ft wide and the tunnel floor is 38 ft below water level. Once the concrete tunnel sections were sunk in place, the

Contractor froze the ground around the joints with liquid nitrogen and cast the concrete joints.

F. UNDER-THE-ROOF CONSTRUCTION

The concept of constructing a tunnel from the top down is known by several different terms including under-the-roof, inverted construction, and construction by the Milan method. The construction method used is discussed above in Paragraph C, Slurry Walls. The technique involves casting the tunnel roof and any necessary supporting pillars as soon as the utilities are removed from the excavation and the excavation has reached the appropriate level. Utilities and street-level restoration then follow, after which the area under the tunnel roof slab is excavated and the remainder of the tunnel is constructed. The major advantage is that this technique disrupts surface traffic only once instead of twice.

G. WATERPROOFING

A watertight structure is necessary in most tunnel construction. The most commonly used methods of waterproofing cut-and-cover tunnels are bentonite panels, hydrolithic coating, brick in asphaltic-mastic, multi-ply membrane waterproofing, waterproof concrete, and chemical grouting.

1. <u>Bentonite Panels</u>

These are waterproofing panels made of fluted kraft paperboard and the flutes are filled with refined high-sodium bentonite. The panels are 4-ft square and generally 3/16- or 1/4-inch thick.

The panels are fastened to vertical walls by a Ramset tool and merely laid flat on the roof slab, lapping edges a minimum of 1-1/2 inches. A smooth surface is not necessary since the panels can be installed on a fairly rough surface, if it is dry.

When backfilling is completed, the kraft containers quickly decompose by moisture and anaerobic soil action, leaving a layer of bentonite adhered to the concrete surface. A break in the waterproofing is normally self-sealing.

The granulated bentonite material on contact with moisture expands to 10 to 25 times the original volume, creating a gel that is impervious to water. The force generated by expansion is on the order of 35 psi, so the walls of the structure must be designed to withstand this pressure. Bentonite panels have been proven effective at heads up to 70 ft.

2. <u>Hydrolithic Coating</u>

Coatings on the inside of the structural members consist of the plaster-coat and the iron-coat methods.

a. <u>Plaster-Coat Method</u>

A 3/4- to 1-inch thick layer of cement mortar consisting of cement, sand, and a waterproofing compound is troweled on the exposed surface.

b. <u>Iron-Coat Method</u>

Ironite is a material that includes finely ground 98-percent-pure iron to which other materials may be added to accelerate the oxidation of the iron particles. A trowel coat is applied to the structure.

3. Brick in Asphaltic-Mastic

This method has been traditionally used by the New York City Transit Authority. From one- to three-ply waterproofing is applied to the outside of the concrete surface. This is covered with a one-course thickness of brick laid up in asphaltic material, which is then covered with four inches of protective concrete. This method is very expensive, but it has been proven effective over many years of use.

4. Membrane Waterproofing

Waterproofing is applied to the outside walls and roof of the structure. The materials include bituminous-impregnated felts and butyl-rubber sheets. Butyl-rubber waterproofing is applied in one layer with the joints sealed or vulcanized, while other materials are normally applied in several layers or plies.

On sidewalls, insulation board is often applied over the waterproofing material to prevent damage from backfilling operations. A 2- to 4-inch protective slab is normally placed over the waterproofing on the roof.

5. Waterproof Concrete

The most common practice today is to construct the tunnel in such a manner that no waterproofing is required.

a. Principle Causes of Permeable Concrete

Permeable concrete results from improper or careless construction methods that result in honeycomb, voids, and improperly vibrated concrete. Inadequate foundation material, resulting in cracks caused by movement of the structure is another cause. Inadequate design of the concrete mix, and improper allowances for shrinkage and temperature forces are additional factors. Figure 110 shows the effect of the water-cement ratio and age on permeability.

b. Proportioning Watertight Concrete

The factors affecting the watertightness of concrete include surrounding the non-porous aggregate by a watertight portland-cement paste. The permeability of the concrete has direct relationship to the amount of water used and the time the concrete has cured.

The mixture should be proportioned to provide a plastic concrete which can be handled and placed without segregation. The exact proportions of aggregates to use will depend on their grading and the conditions of placing. The best procedure is to make trial batches, measuring the materials accurately and making due allowances for moisture.

The concrete should be mixed until it is uniform in color and consistency. This is especially important in watertight construction. Handling, placing, and compacting in the forms are some of the most important steps in concrete work.

The curing or hydration of cement is very important. In watertight construction the concrete should be kept wet for seven days if it is a normal portland cement, and for three days if it is high-early-strength cement.

Expansive cements are a possible development for use in obtaining watertight concrete. These expand after setting and counteract the effects of shrinkage.

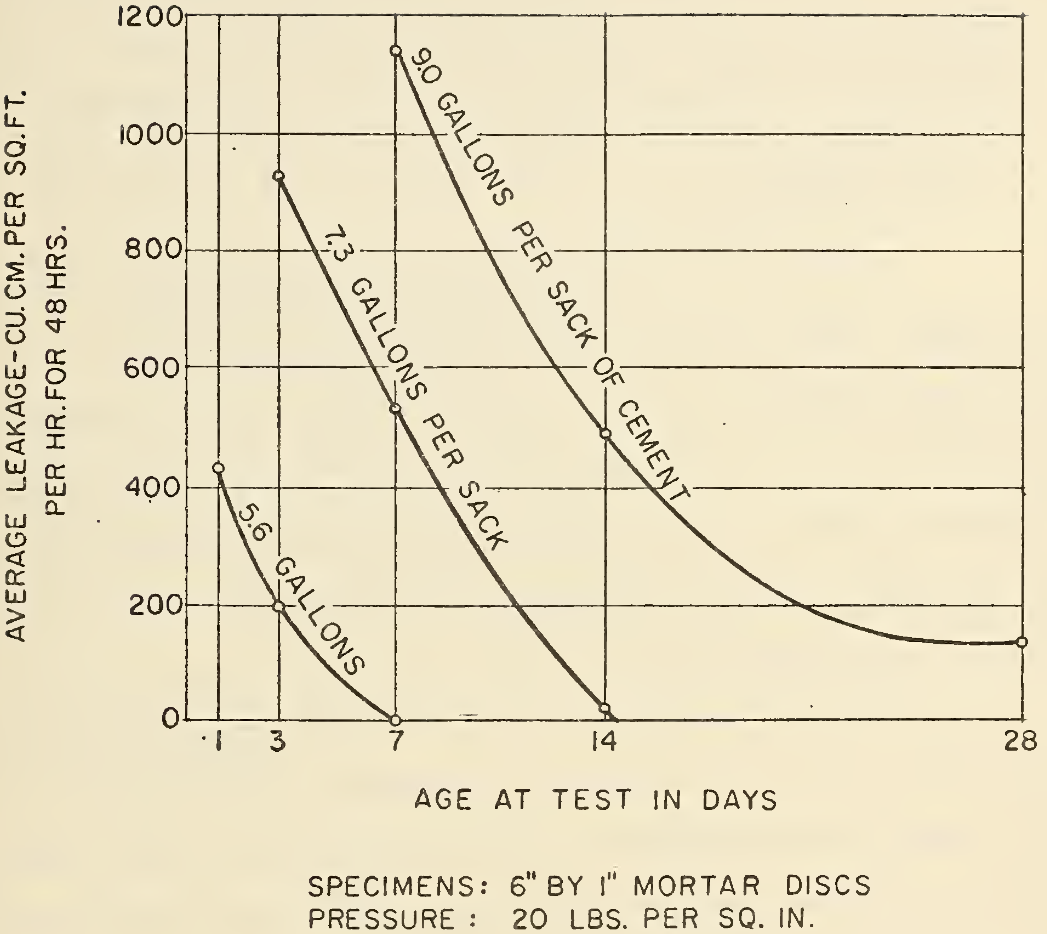


FIGURE 110. EFFECT OF THE WATER-CEMENT RATIO AND CURING ON PERMEABILITY. (By permission of the Portland Cement Association.)

6. Chemical Grouting

Chemical sealing is based on preventing water from flowing through the soil to the potential point of leakage. The flow of water is prevented by filling voids in the soil with fluid chemicals which will solidify in place and create permanent watertight barriers.

7. Evaluation

Waterproof concrete is the most economical means of waterproofing. Where the water table is higher than the tunnel axis, however, more-positive waterproofing means such as multi-ply membranes, asphalt-mastics, or bentonite panels are usually used. The most expensive of these is the brick-and-asphalt mastic, followed by the bentonite panels, the multi-ply membrane, and the asphalt mastic, respectively, in descending order of cost.

Chemical grouting is used only in real problem areas where its primary usage is to accommodate excavation. It is usually used with another more-conventional form of waterproofing to seal leaks or correct bad spots from inside the tunnel after construction is completed.

H. USE OF SPACE ABOVE THE TUNNEL

1. Utility Tunnels (Utilidors)

The space above a cut-and-cover tunnel should be readily and economically available for use as a utility tunnel. The advantages of locating utilities in such a space include easier accessibility for installation, inspection, maintenance, as well as alteration and expansion of the lines; the environment is less likely to cause deterioration and corrosion than direct-burial in the ground; less interference with surface traffic during installation, repairs, and utility-line expansions; and greater flexibility in installing new utilities or expanding existing ones to keep pace with growth.

The main deterrent to the utility tunnels comes from the utilities themselves. Proposals to use the space for BART in San Francisco and the Metro in Washington, D.C. were dropped because of opposition by both public and private utilities.

2. <u>Parking Structure Above Tunnel</u>

Where depth and width permit, the space above a tunnel can be used for vehicle parking. A garage was constructed in Paris, building down from the street-surface by under-the-roof construction methods. The two-track regional express was constructed at the bottom of six parking levels. A width of around 54 ft will be required for two parking spaces plus an aisle, which is not an unreasonable width to find in an urban tunnel.

IX RESTORATION

Restoration includes backfilling, and restoring the utilities and the street. This subject is discussed below under these headings, and an evaluation section summarizes the Chapter.

A. BACKFILLING

Backfill material for cut-and-cover construction is normally obtained directly from the excavation whenever possible, either from material stockpiled at the site or from excavation taking place ahead of the backfill operation. The backfill rate and the need to adjust the moisture content and separate unsuitable material generally make it necessary to stockpile substantial quantities.

Backfill should be placed in not more than 8-inch layers and compacted to not less than 95-percent Modified Proctor density. Backfill must be placed carefully around utilities so as to uniformly support the utilities throughout to line and grade. Tamping the backfill materials should be done under and around the utilities, paying particular attention to the materials in the bottom quadrant of the pipe.

If the excavation has been decked-over, a strip of decking along the center line can be removed and dump trucks can straddle the slit to end-dump the backfill in a continuous windrow. Spreading can then be done by hand or machine. Compacting the material over the full length and width of the construction when it is placed in only one row is a difficulty with this method; however, this is still preferable to removing large areas of decking with the resultant disruption of street traffic.

The clearance necessarily becomes smaller as the lifts approach street-grade until the street-deck and supporting beams must be completely removed. Backfilling operations are then more rapid and efficient, and a more uniform compaction can be achieved, although street-level traffic is more seriously interrupted. Figures 111 and 112 show a restored street in Toronto.

The street was closed to traffic for 45 days under one contract for BART in San Francisco while the Contractor removed the street decking and steel deck-beams, cut off the tops of the soldier piles, and completed the last 5 ft of backfill.

Concreting the street hatches, manholes, and other appurtenances also interferes with backfilling progress.

B. UTILITY RESTORATION

Utility restoration can begin as soon as the backfill is within about 5 ft of the street subgrade, which is about the backfilling limit with the deck still in place.

Trenching in the backfill for underground pipe systems (including electric, gas, water and sewers) can generally be done without shoring or dewatering since the water table was probably held well below the invert of the trench. Utilities supported in-place must have the supporting backfill carefully placed as described earlier.

To restore utilities, thrust blocks must be constructed for water lines, pipe must be welded, water lines sterilized, valve boxes constructed, and protective coating applied to lines. Electric and

FIGURE 111. TORONTO SUBWAY, YONGE STREET AFTER RESTORATION, looking south. Restoration is almost complete. The station is located to the right out of view. The standing water in the foreground shows where the Contractor had settlement problems.

FIGURE 112. TORONTO SUBWAY, YONGE STREET AFTER RESTORATION, looking south. Overall view of method of installing the soldier piles and how the traffic was rerouted. Note the locations in the foreground where the soldier piles were installed; the pavement has been restored.

telephone utilities require ducts, vaults, manholes, service boxes, and service conduits.

About 90 days can be expected to be taken up by utility restoration in any particular area under average conditions. During this time at least some interference with surface passage can be expected. This 90-day delay can be avoided if utilidors are used in conjunction with under-the-roof construction techniques since the utilities work can be done after backfilling is completed and while the tunnel work is being done.

C. STREET RESTORATION

All restoration work where conventional decking is used must be done after the decking has been removed. This includes placing the sub-base and base materials, placing the concrete base slab (if required), and placing the asphaltic-concrete street surfacing. This work again requires about 90 days; however, some of this time can be concurrent with utility restoration, and partial-width areas may be opened sooner.

Sidewalks, driveways, curbs, gutters, and other special construction items require additional time.

D. EVALUATION

The concept of under-the-roof construction appears to offer definite advantages since street traffic should be disrupted only about half the time of regular cut-and-cover operations.

Both the conventional cut-and-cover and the under-the-roof techniques require about the same construction time for building temporary walls (soldier piles, and lagging or other) and for the temporary or permanent relocation of utilities to clear the construction area. The under-the-roof technique can be completed to the point where the roof slab is cast and the street is permanently restored in about the same elapsed time the conventional cut-and-cover technique needs to advance construction to the point where the street can be reopened to traffic on the temporary roadway surface. Building a conventional temporary roadway deck takes about 30 days, as does (under ideal conditions) excavation for and casting the permanent roof slab. Backfilling to within 5 ft of the subgrade elevation can then be done quickly and the roadway reconstructed in the under-the-roof scheme. This latter operation will require the same amount of time for either construction method, although it will come much later in the project life-cycle in the conventional construction method. An alternate solution to the street-disruption problem if the conventional method is used is to build half the tunnel at a time, thus keeping half of the street-surface finished at all times.

Developing a method of quickly installing decking above the street level, and thus reducing the time the street is initially closed to traffic, seems to be a better solution to the problem. This is also the only means of allowing backfilling operations to get closer than 5 ft below street subgrade before disrupting traffic.

The umbrella-deck used in London allowed all but the final street resurfacing to be done before the deck was removed. While this method was only used at an intersection, only a three-day weekend was required for removing the deck and resurfacing the street.

X COST CONSIDERATIONS

The estimates usually used to evaluate the relationship between the cost for cut-and-cover and other tunnel construction methods are often unfair comparisions because important costs are not included. Cut-and-cover techniques are often compared with surface construction, but the land costs, the community tax-revenue losses, and a sum for the effect on the community are seldom considered as part of the surface construction costs. All costs involved in each construction method compared must be shown if a valid cost comparison is to result. The costs for building a highway with at-grade intersections, for example, can be legitimately compared with those for building a limited-access highway only if the costs for the latter's grade separations are included.

One paper[9] gives the ratio of construction costs as being surface roadway 1, cut-and-cover 13.4, and shield-driven tunnels with minimum ventilation 17.5. When all factors have been considered, the ratio becomes 1.0 for a surface roadway. 2.2 for a cut-and-cover tunnel and 2.7 for the shield-driven tunnel with minimum ventilation.

A. COMPARISON OF VARIOUS TYPES OF CUT-AND-COVER TUNNEL CONSTRUCTION COSTS

An accurate cost comparison for the differing construction methods is very difficult to get. Items are generally not bid as

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9. Brown, C. D., "Homes and Highways", The Consulting Engineer, London, England, November 1971.

alternates, so the actual cost-differentials are not known.

Soldier piles and lagging are generally conceded to be the least expensive ground-wall-support method.

If 100 percent for soldier piles and lagging is used as the reference point, then the slurry-trench method is about 10-to-15 percent more costly, and the continuous bored-pile wall is about 25 percent more expensive. The cost for precast elements in slurry-trench construction is unknown, but the method is probably 25-to-40 percent more costly. These comparisons are only for the construction costs, and other factors are not included. Temporary decking over the full excavation will probably add 10-to-15 percent to the total cost.

The reinforced-concrete continuous box is the most economical design for the permanent construction. The steel-frame and jack-arch construction method is about 40 percent more costly.

B. CUT-AND-COVER COST BREAKDOWN

Table IX is an actual breakdown for various construction contracts, and shows the most significant cost items. Figure 113, based on Table IX and on other Contracts, is given to show the cumulative percentages of average cut-and-cover construction cost. Note that some of the areas which have a major effect on disruption and inconvenience to the public have a relatively minor effect on cost. Street decking, utility relocation, and street restoration costs, for example, could be increased by a large percentage without significantly affecting the overall cost.

TABLE IX

CUT-AND-COVER COST BREAKDOWN

Item	BART Berkeley-Richmond	Washington Metro Contract B2	Washington Metro Contract A3
TOTAL COST	$5,935,000	$12,330,000	$21,690,000
Mobilization[a]	11.5%	3.5%	2.6%
Decking	1.5%[d]	10.6%	6.2%
Excavation[b]	30.0%	34.0%	30.4%
Permanent Structure[c]	48.0%	28.5%	39.6%[e]
Backfill	3.0%	5.5%	5.8%
Utilities	4.7%	15.7%	14.1%
Restoration	1.3%	2.2%	1.3%

a. Mobilization includes maintenance of traffic.
b. Excavation cost included ground-wall support.
c. Permanent structure includes underpinning.
d. Only 18% of original decking specified was installed.
e. Includes station cost.

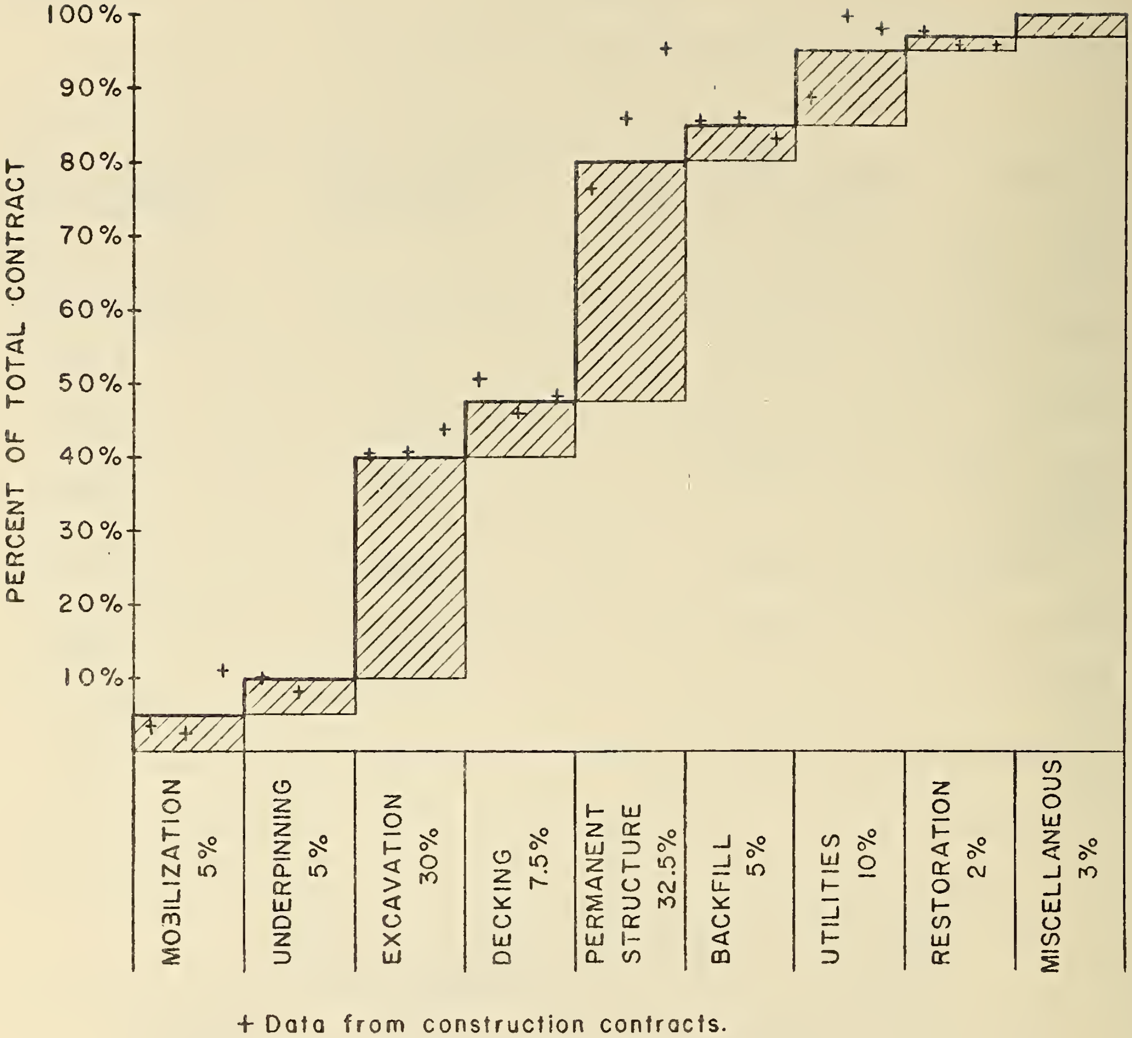


FIGURE 113. AVERAGE COST BREAKDOWN FOR CUT-AND-COVER CONSTRUCTION

Table X gives the relative costs for various types of ground-wall support. The calculation is based on currently available unit-cost figures and production rates, and does not consider other factors such as dewatering, underpinning, and using a ground-support wall in a permanent structure. This Table should be used as only a relative guide, and the conditions for the particular case should be included in any cost analysis.

Precast panels in a slurry trench have not been used in the United States and very limited use has been made of continuous bored-piles and LN_2 freezing, so these unit costs are undoubtedly less accurate. Slurry walls, tiebacks, and shotcrete walls are becoming more frequently used, but there is still a wide range of unit-costs, for these methods.

Several items such as the operating cost of freeze plants, and the crews required for drilling and grouting depend in a large degree upon local labor practices and the number of men required to operate and maintain equipment.

C. CASE STUDY

The cost of cut-and-cover construction with conventional construction (using soldier piles and lagging) and slurry-wall construction were studied to evaluate the two separate conditions.

Case I presumes construction in a medium-dense sand; Case II in a medium-stiff clay. An impervious till with good bearing capacity is presumed to underlie both the sand and clay.

Both cases presume a double-barrel vehicular tunnel with

TABLE X

RELATIVE COST OF VARIOUS TYPES OF GROUND-WALL SUPPORT

Ground-wall support	Advantages	Disadvantages	Cost/sq ft of exposed excavation wall (65-ft-wide by 45-ft-deep excavation) $
Soldier beams, lagging, and cross-lot struts	Minimum cost. Established method. Positive bracing from struts. Soldier piles can be reused. Will work under almost all soil conditions.	Does not control ground-water. Permits relatively large movements. Excavation area obstructed. Underpinning generally required. Dewatering required.	10.50
Soldier beams, lagging tiebacks	Excavation area unobstructed.	Required easements for tie-backs. Dewatering required.	11.00
Slurry-wall, tiebacks	Relatively rigid wall. Ground-water cutoff. Can be used as permanent wall. Minimizes underpinning. No dewatering required.	Danger of slurry-loss into voids. Disposal problem of slurry. Requires large operating area. Difficult if crossed by utilities.	24.00
Slurry-wall, precast panels, and tiebacks	Better-appearing finished wall. Wall can be installed plumb.	Storage and shipping of panels. Disposal of slurry grout. Uneconomical unless panels uniform. Trench unstable for longer period.	29.00

Table X, Relative Cost of Various Types of Ground-Wall Support, contd.

Ground-wall support	Advantages	Disadvantages	Cost/sq ft of exposed excavation wall (65-ft-wide by 45-ft-deep excavation) $
Continuous bored-piles, with tiebacks	Can be used as permanent wall. Narrow operating width. Dry process--no slurry required. No danger of intercepting voids. No dewatering required.	Equipment requires good maintenance. Tends not to be water-tight. Cost is higher. Longitudinal continuity difficult to obtain. Low production rate.	26.50
Soil injection	Control ground-water. Minimizes underpinning. May be used in conjuction with other methods.	Cannot be used in silts and clay. Limited strength attainable. High cost.	36.00 (6-ft wall)
Freezing, gravity wall using brine	No cross-bracing required. Minimal underpinning. Can be used as outside concrete form.	Ground-heave is a problem. Time required is large. Not applicable under high ground-water flow velocity. High labor costs for operating.	30.00
Freezing, tiebacks using brine	Minimizes underpinning.	Strength of wall uncertain. Heave near building a problem. Long period of time required for freezing. Continuing cost of operating plant.	21.00

Table X, Relative Cost of Various Types of Ground-Wall Support, contd.

Ground-wall support	Advantages	Disadvantages	Cost/sq ft of exposed excavation wall (65-ft-wide by 45-ft-deep excavation) $
Freezing, tiebacks using LN_2	Quick freezing possible.	High cost of material. Heave near building a problem. Hazardous in enclosed area.	32.00 (6-ft wall)
Shotcrete, with tiebacks	Installed in a continuous operation Can be used as outside concrete form.	Strength of wall uncertain Good quality-control required. Good workmanship required. Does not cut off ground-water. Depth of excavation limited.	6.00
Sheet piling, with cross-lot struts	Material reusable. Ground-water cut off.	Difficult to drive through hard material. Difficult if crossed by utilities. Relatively flexible system.	10.00

approximately 65-by-25-ft outside dimensions. The invert will be 40-ft below street-grade. The tunnel will be built in the center of a 60-ft-wide street with 15-ft-wide sidewalks on both sides, which gives 90 ft between building lines. Multistory buildings on both sides of the street are founded on spread footings, and utilities of different types are buried under the street.

1. Case I

a. Soil Conditions

Medium-dense sand overlying a dense, impervious till at 60-ft depth. The ground-water table is about 20 ft below street grade.

b. Basic Scheme

Ground support by soldier piles and timber lagging.

Dewatering by deep wells outside the excavation.

Intermediate support for bracing and street decking.

Underpinning by piles, penetrating to the till only under the first row of spread footings near the excavation.

Utilities will be suspended from the traffic decking.

No temporary members will be part of final structures; only struts, intermediate supports, and the upper 8 ft of soldier piles will be retrieved. Everything else will be left in place.

For estimating the earth pressure, use an apparent rectangular earth-pressure distribution calculated for $K_A = 0.3$ (ref. Terzaghi & Peck, 2nd ed., pp. 401 and 402).

Assume a dewatering capacity of 500 gpm/100 ft for the deep wells.

c. Alternate Scheme

The ground-support and intermediate-support walls will be built by the slurry-trench method. Street decking will be of timber and structural steel beams. The first excavation phase will go down to the tunnel roof level. The roof will be cast and used as permanent bracing. Excavation will continue under the roof.

The slurry-trench walls and the intermediate support will be used as part of the final structure. One set of temporary bracing will be used between the tunnel roof and excavation until the floor is cast.

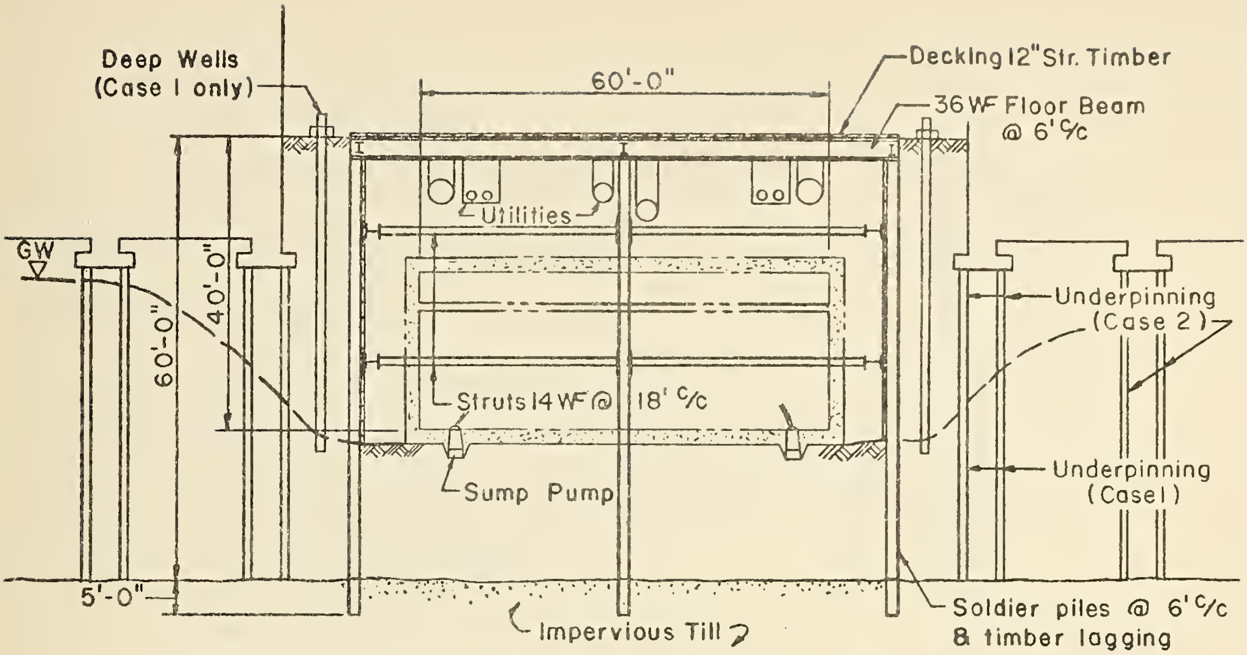
The rigidity of the slurry-trench wall will eliminate the need for underpinning.

Check the struts using a triangular earth-pressure distribution and an earth-pressure coefficient of 0.40.

Assume small sump pumps for dewatering having a capacity of 20 gpm/100 ft. The slurry-trench walls will cut off the ground water if well-seated in the presumably impervious layer. The sump pumps are only for removing minor seepage through the bottom and eventual leakage through the joints between segments in the slurry walls.

2. Case II

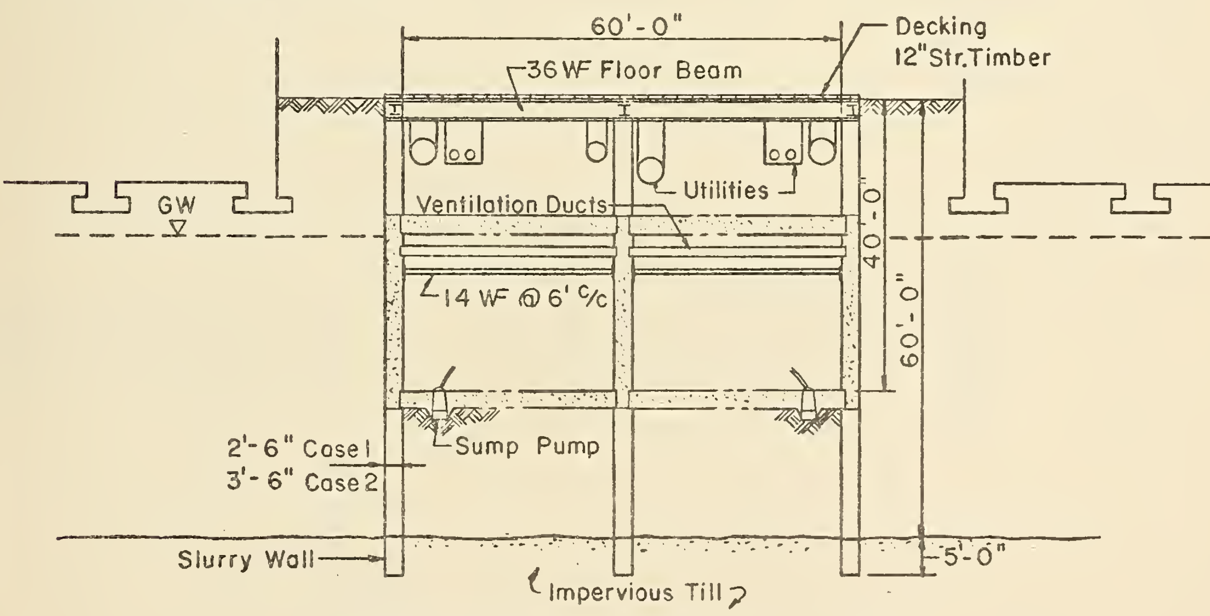
Soil Conditions: Medium-stiff clay overlying till at 60-ft depth. The ground-water table and all other boundary conditions will be as in Case I. Figure 114 shows the design assumptions used for the case study, and Table XI summarizes the cost estimate.



CONVENTIONAL CONSTRUCTION

Scale: 1" = 20'

CASE I—MEDIUM SAND
CASE 2—MEDIUM STIFF CLAY



SLURRY-WALL CONSTRUCTION

Scale: 1" = 20'

FIGURE 114. CASE-STUDY DESIGN ASSUMPTIONS

TABLE XI

Case Study Cost Estimate
For 100 ft of Cut-and-Cover Tunnel Construction

Items	Conventional Construction		Slurry-Wall Construction	
	Case I Medium Sand	Case 2 Medium-Stiff Clay	Case 1 Medium Sand	Case 2 Medium-Stiff Clay
Decking	27,600	$ 27,600	$ 27,600	$ 27,600
Dewatering	15,000	5,000	1,500	700
Excavation	124,000	124,000	91,000	87,900
Floor Beams	56,000	56,000	51,000	51,000
Gravel borrow	8,000	8,000	12,300	11,800
Reinforcing steel	65,500	65,500	42,000	40,900
Soldier piles	34,300	43,400		
Slurry walls			390,000	470,900
Temporary bracing and wales	35,000	86,900	44,300	71,400
Temporary intermediate support	16,300	16,300		
Timber lagging	22,400	29,800		
Tunnel concrete	228,000	228,000	158,000	154,000
Underpinning	65,000	130,000		
Utility Supports	10,000	10,000	10,000	10,000
Waterproofing	7,300	7,300		
TOTAL	$714,400	$837,800	$828,100	$926,200

a. Basic Scheme

As in Case I, except that underpinning is assumed to be needed both for the first and second row of spread footings under the adjacent buildings.

Select a heavier soldier-pile section and thicker lagging than in Case I. Check the bracing for a K_A value of 0.70.

Assume sump pumps having a capacity of 100 gpm/100 ft.

b. Alternate Scheme

As in Case I.

Check the bracing for a K value of 0.80.

Assume sump pumps having a capacity of 10 gpm/100 ft.

XI SUMMARY OF MAJOR PROBLEMS AND RECOMMENDED IMPROVEMENTS

This subject is discussed below under major problems, recommendations for further research, and recommended practices.

A. MAJOR PROBLEMS

Major problems contributing to increased construction time, construction that must be done in several separate operations, and factors contributing to increased costs all tend to act as a deterrent to the acceptance of cut-and-cover tunnel construction. A summary of these problems in given below.

1. Geotechnical Investigation and Analysis

A thorough understanding and analysis of the subsurface conditions is imperative, to minimize the effect of the construction work on adjacent buildings and utilities, and to assure the work will be completed as expediently as possible. Constructing the excavation support may be one of the most costly items in building a tunnel within a built-up area. The magnitude and distribution of soil movements outside the excavation (which may have detrimental effects on both existing structures and utilities) must be accurately predicted to establish the dewatering and underpinning criteria in connection with different types of support systems.

The theoretical know-how exists for precise analysis of stresses and deformations in the structural elements of the ground support and in the soil. However, the knowledge of the soil parameters, as obtained by common field and laboratory methods, is far from compatible with the quality of the analysis. Only limited knowledge about the construction procedure is generally available in the design phase, which may cause over-conservatism in selecting the soil strength parameters to assure that a catastrophic situation will not develop if the contractor deviates from the assumptions in the analysis.

Construction problems most frequently occur in soft clays. High stresses developing in the lower parts of the excavation support may cause excessive yielding, accompanied by settlement of the ground outside the excavation.

The frequency and size of boulders is often underestimated, which may either affect the choice of construction methods or render a selected method impractical.

Ground-water elevations may change considerably between different times of the year, and observations during the preconstruction drilling period may not be representative for the construction period.

2. Utility Relocations

The present method, in which several agencies (including the contractor) make the utility relocations, causes increased disruption and construction time. Better coordination must be obtained if improvements are to be made, and the responsible agency must exercise more effective control. For instance, improving a backfilling technique to permit more rapid restoration is pointless if the utility restoration procedures do not also change.

3. Other Considerations

While nonengineering considerations may be beyond the scope of an engineering evaluation of cut-and-cover techniques, several incidental problems were noted during this study which it may be helpful to mention.

The size and location of cut-and-cover projects magnify the problems normally encountered in lesser-sized projects in respect to responsibility for damages to property and persons, and the resulting adverse effect on project costs. For example, subsurface highway projects are likely to occupy the entire width of streets above the tunnel during construction. A central business district location with high-density population is normally involved, and this is an area where the lateral support of major structures and maintaining traffic and business-as-usual are most critical.

Immediately apparent is the problem of increased costs associated with responsibility for:

a. The accelerated construction schedule necessary to minimize traffic and business interruptions.

b. Selecting and using the most efficient construction methods.

c. The lateral support of adjacent property, including structures.

d. Easements on adjacent property for temporary and/or permanent construction.

e. Protecting the public and job safety.

f. Relocating and maintaining of utility lines and services.

While an exhaustive discussion of the possible ways to counter these problems is inappropriate here, the following suggestions may be considered:

a. An adequate subsurface investigation is indispensable and will minimize subsequent costs.

b. Choosing the construction approach most suited to conditions to be encountered will be advantageous if it is made early in the design phase. Collaboration between the design engineer and a construction consultant should permit a design consistent from the start with the most efficient construction methods for the project. This "value engineering" should expedite advance schedules and planning, and acquiring easements for utility relocations and for any tie-backs or underpinnings which may be involved.

While every encouragement should be given to using the new and more efficient construction measures and procedures, the selection must also consider that most of the new construction methods and equipment are covered by some form of patent. This includes almost all of the various types of equipment used for constructing slurry-walls, for drilling continuous bored-piles, for using precast elements in slurry trenches, for soil injections and grouting, and for installing tie-backs.

In general, enough options seem to be available so these systems are still reasonably competitive, and the existence of a patent should not deter their use. Legal liability for unexpected consequences, rather than patent-rights problems, is the reason for contractor-reluctance to use new methods.

c. If underpinning of structures cannot be avoided, the design engineer should be permitted to furnish a detailed design rather than provide a "performance"-type specification for the constructor as is quite commonly done.

d. The steps suggested in paragraphs a, b, and c above should enable prospective contractors to bid more realistically and with fewer contingency factors.

e. Project costs may also be reduced by the Owner providing the insurance coverage for the entire project under a "wrap-up" or similar policy protecting the contractors as well as the Owner.

f. Owners in some cases have procured and furnished certain construction materials to minimize the Contractor's capital requirements and reduce the size of the bids. Soldier piles and temporary decking materials are examples.

4. <u>Environmental Quality Considerations</u>

If cut-and-cover tunneling is to be a more useful technique in urban areas, a more meaningful approach must be made to the problem of properly complying with today's environmental regulations. It is no longer acceptable to give only superficial recognition to the problems involved: neither the public nor the regulatory agencies involved will accept such an approach. Establishing to a much greater degree just how the Contractor will be required to do the work will probably be necessary. The method of constructing the ground-support wall, how piles will be driven, the number and type of trucks permitted at street level, and noise- and vibration-control criteria will probably all be part of the designs.

B. RECOMMENDATIONS FOR FURTHER RESEARCH

The following items are those which we recognize as being worthy of future research and which will lead to significant improvements in cut-and-cover tunnel construction techniques.

1. <u>Contractual Procedures</u>

Some of the practices being initiated by the U. S. Bureau of Reclamation for the Stillwater Creek Project indicate promise in improving methods of contracting for underground construction. One phase of the negotiation on this project will apparently consist of a prebid conference to put all known facts on the table, and consideration of ideas suggested by the selected Contractors. These ideas will be evaluated and, after a second prebid conference, the selected ones will be incorporated in the final documents.

Contracting procedures in Europe allow more flexibility, based to some degree on a time-and-material or unit-cost approach and, to some degree, on built-in negotiation phases.

A study directed at a detailed analysis of these procedures should prove valuable.

2. <u>Geotechnical Investigation and Analysis</u>

Promising areas for research in this field are:

a. Continued research into the behavior of ground-support systems and soils as observed in actual construction cases.

b. Developing computer programs based on the finite-element method of analysis for designing ground-support systems and for predicting soil deformations.

c. Improving the methods for <u>in-situ</u> determination of the soils-deformation properties, shear strength, and at-rest pressure.

d. Adapting the geophysical exploration method to the environmental conditions of heavily built-up areas.

e. Developing a standardized method of design for semi-rigid ground-wall support-systems such as slurry walls.

3. Pneumatic and Hydraulic Material Handling

The technology of handling material by pneumatic or hydraulic pipelines is available, but it has never been applied to excavating for cut-and-cover tunnels. Research should include a demonstration project to determine actual problems in the field.

4. Movable Elevated Decks

Elevated decks have been successfully used in London and offer the only means of minimizing the time of disruption in the area between the street surface and a depth of about 5 ft. The idea of moving such a deck is new and requires additional research to study the problems and determine its feasibility under United States construction conditions.

5. Shotcrete

Shotcrete has been little-used for supporting excavations in urban areas. While obviously not the complete answer, it does appear to have potential under certain conditions. Research is needed to determine the proper methods of application to soils.

6. <u>Continuous Casting of Concrete</u>

Research on the continuous casting of concrete is currently under way at the University of Illinois, and is specifically directed toward use in shield-driven tunnels. The development to some degree depends on the successful application of wire-reinforced and regulated-set concretes. Continuous casting is analogous to a horizontal slip-form technique and requires the concrete to attain supporting strength by the time it leaves the form. The results of current research should be adapted and applied to cut-and-cover tunneling.

7. <u>Waterproof Concrete</u>

Structure backfilling could proceed more quickly if the need for additional waterproofing could be eliminated. Promising areas for accomplishing this are waterproofing admixtures, proper quality control, and expansive cements. The materials are already available and all that is needed is to adapt the use to tunnel construction. A demonstration project should be initiated to test these materials under actual construction conditions and to evaluate both the results and the costs.

8. <u>Joint-Use of Tunnel Space</u>

The American Public Works Association and others have long discussed placing a utility tunnel above a vehicular tunnel. The utility tunnel could accommodate all major utilities in a readily accessible space at very little additional cost. While most authorities agree in theory, the administrative and financial problems connected with reaching agreements with many utility companies have prevented any significant development. The opposition usually expressed by utilities is based on the possible hazards in the joint-use of tunnels, although there is sufficient evidence to the contrary.

C. RECOMMENDED PRACTICES

One of the purposes of this report is to recommend an improved cut-and-cover tunneling method. There is no single recommended improved method. In certain areas, however, we recommend that goals be established and direction given toward reaching what is admittedly an ideal situation.

1. Develop continuous, accurate subsurface data by other-than-direct physical means and correlate it with a conventional boring program.

2. To decrease damage to adjacent structures, hold the soil in its original position by permitting only tolerable ground-movements and minimizing underpinning. Use cast-in-place walls for ground support, tie-backs, and soil injection, and avoid using methods which will permit ground loss from infill of holes and infill behind lagging. Do not underpin except under unusual circumstances.

3. Use the ground-support wall as the permanent wall if possible.

4. Where conditions permit, use one continuous operation to excavate and support the ground-wall. Typically this would be an excavate-shotcrete operation in an area where loads adjacent to the excavation are either not high or can be effectively controlled.

5. Use a method of decking which permits rapid installation, provides for reuse, permits most of the excavation to start after the

deck is in place, and allows backfill to be placed close to the street level. The raised, movable umbrella-deck seems to satisfy these requirements.

6. Use a mechanized materials-handling system to excavate material, bring concrete into the excavation, and backfill to street level. This will minimize the number of trucks and other equipment required to be at street level.

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