

LATERAL SUPPORT SYSTEMS AND UNDERPINNING

Vol. III. Construction Methods

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16. Abstract This provides specific design recommendations, design considerations, and construction techniques for the construction of lateral support systems and underpinning. The design considerations are presented for each technique or method (soldier piles, steel sheeting, diaphragm walls, internal bracing, tiebacks, underpinning, grouting, and freezing). The factors affecting the design or implementation of these schemes are discussed. Construction techniques are presented, and literature references are provided for those seeking even greater detail. An overview of the construction methods compares the applicability of the techniques and the construction costs of each. Other reports developed from the study are FHWA-RD-128, Volume I, Design and Construction; FHWA-RD-129, Volume II, Design Fundamentals; and FHWA-RD-131, Concepts for Improved Lateral Support Systems.					
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PREFACE

Volumes II and III of this three volume set present the current state-of-the-art on the engineering aspects of the design and construction of ground support walls and the closely related techniques of underpinning, ground freezing, and grouting. So that the reader will understand the rationale behind the subject matter, the text contains detailed discussions, especially in areas of controversial or technically new issues. On the other hand Volume I, a summary of Volumes II and III, is free from the detailed discussions embodied in the latter two. Its purpose is to provide a ready reference manual.

Overall, the primary intent is to provide information and guidelines to practicing engineers, in particular those engineers with an advanced background in the disciplines of Soil Mechanics and Foundation Engineering.

Volume II incorporates design fundamentals, primarily those of a geotechnical nature. It places considerable emphasis upon displacements of adjacent ground and adjacent structures and considers those parameters which are primary contributors to excessive displacements.

Volume III is directed toward the essential design and construction criteria associated with each of the following techniques: (a) Support Walls - soldier pile walls, sheet pile walls, concrete diaphragm walls; (b) Support Methods - internal bracing and tieback anchorages; (c) Underpinning; (d) Grouting; (e) Ground Freezing. Also, it presents an overview of these construction methods with regard to selection, performance, and relative cost. Throughout, an attempt has been made to provide a balance between the practical engineering considerations of construction and appropriate corresponding considerations of engineering fundamentals.

These publications are produced under the sponsorship of the Department of Transportation research program, a long range plan to advance the technology of bored and cut-and-cover tunnels, in particular those constructed in the urban environment.

Part of this program involves a synthesis and evaluation of existing knowledge and part involves a Research and Development effort. These volumes fall under the category of the former, "State of the Art", aspect of the program from which it is hoped that progress through development of bold innovative approaches will emanate.

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TABLE OF CONTENTS

INTRODUCTORY TEXT

	<u>Page Number</u>	
TITLE PAGE	i	
PREFACE	ii	
ACKNOWLEDGEMENTS	iv	
TABLE OF CONTENTS	v	
CONVERSION FACTORS	viii	
SYMBOLS	x	
 <u>TEXT</u> 		
CHAPTER 1	OVERVIEW OF CONSTRUCTION METHODS	1
	1. 10 Purpose and Scope	1
	1. 20 General Conclusions Concerning Displacements	1
	1. 30 Wall Type	3
	1. 40 Support Method	5
	1. 50 Underpinning	8
	1. 60 Stabilization Methods	8
	1. 70 Soil and Ground Water Conditions	10
	1. 80 Costs	11
CHAPTER 2	SOLDIER PILE WALLS	17
	2. 10 Introduction	17
	2. 20 Types of Soldier Pile Walls	19
	2. 30 Design Considerations	29
	2. 40 Construction Considerations	45
CHAPTER 3	STEEL SHEET PILING	50
	3. 10 Introduction	50
	3. 20 Design Considerations	50
	3. 30 Construction Considerations	57

TABLE OF CONTENTS (con't)

	<u>Page Number</u>	
CHAPTER 4	CONCRETE DIAPHRAGM WALLS	59
	4.10 Introduction	59
	4.20 Properties of Bentonite Slurry	60
	4.30 Slurry Trench Stability	67
	4.40 Practical Aspects of Slurry Stabilized Excavations	85
	4.50 Structural Aspects of Cast-in-Place Walls	99
	4.60 Excavation of Slurry Trenches	105
	4.70 Diaphragm Walls Other than Continuous Cast-in-Place Concrete	112
	Appendix A	125
	Appendix B	135
CHAPTER 5	INTERNAL BRACING	140
	5.10 Introduction	140
	5.20 Design Considerations	140
	5.30 Installation	144
	5.40 Temperature Effects	149
	5.50 Strut Removal and Re-bracing	154
CHAPTER 6	TIEBACKS	155
	6.10 Introduction	155
	6.20 Design and Theoretical Considerations	157
	6.30 Construction Considerations for Tiebacks	227
	6.40 Field Testing	244
CHAPTER 7	UNDERPINNING	267
	7.10 Introduction	267
	7.20 Design and Theoretical Considerations	269
	7.30 Classical Underpinning Procedures	272
	7.40 Grouted Piles	300
	7.50 Tunneling Below Structures	307
	7.60 Load Transfer	323
	7.70 Temporary Support or "Shoring"	324
	7.80 Performance	329

TABLE OF CONTENTS (con't)

	<u>Page Number</u>
CHAPTER 8	
GROUTING	335
8.10 Introduction	335
8.20 Design and Theoretical Considerations	335
8.30 Construction Considerations	350
8.40 Field Testing and Quality Control	358
8.50 Specific Applications	360
CHAPTER 9	
FREEZING	366
9.10 Introduction	366
9.20 Design and Theoretical Considerations	368
9.30 Construction Procedure	400
9.40 Typical Applications	410
BIBLIOGRAPHY	411

LIST OF CONVERSIONS

The list of conversions is designed to aid in converting from British units of measure to metric units. This section has been divided into two parts; general notation and arithmetic conversion.

General Notation

BTU	British Thermal Unit
cm	centimeter
cm ²	square centimeter
cm ³ , cc	cubic centimeter
cfs	cubic feet per second
ft	feet
ft ²	square feet
ft ³	cubic feet
fps	feet per second
gal	gallon
gpm	gallons per minute
g, gr	grams
hr	hour
in	inches
in ²	square inches
in ³	cubic inches
k	kilo (thousand)
kg	kilogram
m	meters
m ²	square meters
m ³	cubic meters
min	minute

mm	millimeters
mm ²	square millimeters
mm ³	cubic millimeters
ml	milliliters
N	Newton
lbs	pounds
pcf	pounds per cubic foot
plf	pounds per lineal foot
psf	pounds per square foot
psi	pounds per square inch
sec	second

Conversions

British Units

Metric Units

List Of Symbols

The following list of symbols has been prepared to aid the interpretation of symbol use in the text. This list identifies only the major symbols used in the text and their general meaning. Each symbol (with subscripts) is defined in the text for its particular usage. This list is not a complete list of all symbols or all symbol usage in the text but is a summary of major symbols and their usage.

<u>Symbol</u>	<u>Represents</u>	<u>Reference</u>
A	general symbol for area	
B, b	general symbols for width	
c	cohesion intercept	
C	heat capacity	Volume I, Chapter 16 Volume III, Chapter 9
D, d	general symbols for distance and diameter	
E	general symbol for modulus	
f	general symbol for stress	
F. S.	factor of safety	
H	depth of excavation; also general symbol for height	
K	general symbol for coefficient of lateral earth pressure	
K_o	coefficient of lateral earth pressure at rest	
K_a	coefficient of active earth pressure	
K_p	coefficient of passive earth pressure	
K	thermal conductivity	Volume I, Chapter 16 Volume III, Chapter 9
L, l	general symbols for length or distance	
N	general symbol for stability number or standard penetration resistance	
OCR	overconsolidation ratio	

<u>Symbol</u>	<u>Represents</u>	<u>Reference</u>
P	general symbol for load or force	
p	general symbol for pressure	
pH	negative logarithm of effective hydrogen ion concentration	
R, r	general symbols for radius	
S, s	general symbols for shear resistance or shear strength	
S_u	undrained shear strength	
u	pore pressure	
W	general symbol for weight	
w	general symbol for water content	
δ	general symbol for displacement or movement; also angle of wall friction	
δ_v (max)	vertical displacement (maximum)	
δ_h (max)	horizontal displacement (maximum)	
ϵ	general symbol for strain	
γ	general symbol for unit weight; total unit weight of soil unless otherwise specified	
γ_d	dry unit weight of soil	
γ_m	total unit weight of soil	
γ_{sub}	bouyant unit weight of soil	
γ_w	unit weight of water	
μ	Poisson's Ratio	
ν	Poisson's Ratio	
ϕ	general symbol for friction angle of soil	

<u>Symbol</u>	<u>Represents</u>	<u>Reference</u>
ρ	general symbol for settlement	
σ	general symbol for stress	
σ_v ($\bar{\sigma}_v$)	total vertical stress (effective vertical stress)	
σ_h ($\bar{\sigma}_h$)	total horizontal stress (effective horizontal stress)	
$\bar{\sigma}_{vm}$	maximum past vertical consolidation pressure (effective stress)	
τ	general symbol for shear stress or shear resistance	

Note: Line over symbols indicates effective stress parameters are to be used. (e. g. $\bar{\sigma}_v$ = vertical effective stress).

CHAPTER I - OVERVIEW OF CONSTRUCTION METHODS

1.10 PURPOSE AND SCOPE

This section presents a synthesis of the main conclusions concerning the performance of underpinning and of various techniques for supporting open excavations. Emphasis is placed upon the general applicability of each of the various techniques, and comparisons are made, when appropriate, in order to consider the influence of such variables as soil type, wall type, and method of lateral support. An attempt has been made to identify key operational contingencies which have the potential of contributing to excessive horizontal and vertical displacements in the adjacent ground. Finally, some general guidelines are provided concerning cost.

1.20 GENERAL CONCLUSIONS CONCERNING DISPLACEMENTS

1.21 Lateral Support Methods

"Competent Soils" (granular soils, very stiff clays, etc)

a. Displacements reported in the literature on well-constructed, well-documented cases are of insufficient magnitude to distinguish variations that may be inherent in wall type or in method of lateral support. Nevertheless, there is strong evidence to suggest that concrete diaphragm walls will exhibit less displacement than other wall types and walls supported by tiebacks will perform better than internally braced walls.

b. Maximum displacements are typically in the order of 0.25 percent to 0.35 percent of wall height. The lower range is associated with granular soils; the upper range is associated with cohesive soils.

c. Typically, maximum horizontal and vertical displacements are about equal.

"Weaker Soils" (soft to medium clays, organic soils, etc.)

d. Maximum displacements typically exceed 1 percent of depth of cut for flexible walls such as steel sheet piling. Concrete diaphragm walls dramatically reduce the magnitude of displacements to about 0.25 percent of the depth of cut -- or about the same as those observed for competent soils.

e. Typically, the maximum vertical displacements exceed maximum horizontal displacement.

f. When the excavation is underlain by deep deposits of weak soils, the cumulative total of all displacements occurring below the last placed strut level amounts to about 60 percent of the total measured movement.

"Wall Type"

g. With concrete diaphragm walls, displacements are typically less than 0.25 percent of wall height, regardless of soil type.

h. The stiffness of walls can be increased not only by using rigid concrete walls but by reducing spacing between support levels of soldier pile walls or steel sheet pile walls. Comparable wall stiffness (defined as $\frac{EI}{L^4}$) will result in comparable performance provided that the installations are carefully carried out and ground loss is minimal.

i. A comparison from observational data between soldier pile walls and sheet pile walls (of comparable stiffness) is not possible in very stiff to hard clays and dense granular soils because sheet piles are infrequently used under such hard driving conditions. Therefore, data are lacking.

Effect of Wall Stiffness in Cohesive Soil

j. The influence of wall stiffness (defined as $\frac{EI}{L^4}$) and of stability number of cohesive soil (defined as $N = \frac{\gamma H}{S_u}$) was examined

in some detail. The trends are clear, and the data indeed show increasing displacements with weaker soils and with more flexible walls. Displacements with sheet piling may exceed 4 to 5 inches, but in similar cases, diaphragm walls would control displacements to less than 1-1/2 inches.

1.22 Underpinning

Underpinning itself has an inherent source of deformation associated with the physical transfer of load from the existing to the new foundation. Well-executed construction procedures can normally control this vertical displacement to 1/2 inch or less.

Underpinning may also be influenced by the adjacent excavation because the underpinning elements will be installed within the zone of vertical and horizontal displacements. Thus, this creates the potential for additional displacements and/or additional load imposed upon the underpinning elements. Experience has shown that horizontal movements cause more damage than vertical movements.

1.30 WALL TYPE

1.31 Concrete Diaphragm Walls

1.31.1 Applicability

Diaphragm walls are applicable in practically every soil condition with the possible exception of very soft clays, peat, or cohesive hydraulic fill. They are used frequently to minimize displacements behind the wall. It is common in European practice to incorporate the diaphragm wall into the permanent structure; whereas in the United States, diaphragm walls have historically been used as a method of ground support without being incorporated into the permanent structure.

1.31.2 Operational Considerations

Precautionary measures should be taken to protect against fluid loss during excavation in highly pervious conditions (coarse sand, coarse sand and gravel mixtures). Other

contingencies lie in contamination of the fluid in soils with adverse pH, high salinity, or high calcium content. It is believed that most of these potential contingencies can be identified during the initial investigation and by proper quality control during construction.

Another potential problem is spalling (local collapse) of the trench wall near the ground surface. This may be caused by unstable soils or loose fill, particularly when containing miscellaneous rubble or old foundations. A well-constructed guide wall, sufficient head of slurry, and prevention of slurry flocculation are essential measures.

1.32 Soldier Pile Walls

1.32.1 Applicability

Soldier piles are applicable in all soils except perhaps soft to medium clays and in loose or soft dilatant soils of low plasticity below the water table. These soils have a tendency to run after exposure.

1.32.2 Operational Considerations

The following cited items have the potential of leading to additional displacements: deflection of lagging; overcut behind lagging; ground loss due to surface and ground water; and ground loss associated with pre-excavation for soldier piles. Additionally, there is the risk factor associated with open lagging due to an unusual occurrence which may cause heavy concentrations of water to flow toward the excavation. This may include broken water mains or flooding.

Pre-draining of saturated soils is essential, especially those which may have a tendency to run (silt or silty fine sand for example). A common, difficult situation is when such soils are underlain by rock or by impervious soil within the depth of excavation. This sequence makes it extremely difficult to fully dewater to the lowest extent of the water bearing formation.

1.33 Steel Sheet Pile Walls

1.33.1 Applicability

These are most generally used in soil types that are inappropriate for soldier pile walls, such as the soft clays, organic soils, and dilatant soils of low plasticity. Sheet piling is also used in situations where there is a desire to cutoff ground water or to reduce seepage gradients at the bottom of the excavation,

1.33.2 Operational Considerations

Steel sheet pile walls are relatively flexible with normal wale spacing, and they are frequently associated with relatively large displacements when installed in weak cohesive soils.

Contingencies lie in tearing of interlocks under hard driving conditions and associated ground loss occurring with ground water infiltration.

While interlocked steel sheet piling effectively intercepts ground water flow within previous layers, this is not necessarily a guarantee against depression of the piezometric level outside the excavation. Simply stated, relatively impervious soil types (including clayey sands, silts, and clays) are of equivalent permeability to the steel sheet pile wall itself. Therefore, as a practical matter, the presence of the interlocked steel sheet pile wall does not prevent a seepage pattern to the face of the excavation. Such a seepage pattern is accompanied by a drop in piezometric levels which may induce consolidation of compressible soils. Removal of steel sheet piling from cohesive soils may also remove soils with it and in turn lead to settlement of adjacent ground.

1.40 SUPPORT METHOD

1.41 Tiebacks

1.41.1 Applicability

Tiebacks are most applicable in very stiff to hard cohesive soils or in granular soils. In lower shear strength, cohesive soils, the regroutable tieback has been used successfully, while other anchor types have displayed relatively large movements.

1.41.2 Operational Considerations

Vertical Wall Movement

The vertical components of load may cause settlement of soldier pile walls and this may lead to horizontal displacement.

Excessive Prestressing

With a relatively flexible wall, excessive prestressing of the upper levels may cause inward movement of the top and outward bowing below. The magnitude of the bowing increases in response to excavation as the restraining force is removed on the inside of the wall. The problem is accentuated in a soil sequence of loose - hard - loose from the top to the bottom of the cut.

An unusual case was revealed in a paper by McRostie, et al (1972) which cites an excavation in a sensitive clay and tiebacks drilled into rock. The excessive prestressing induced horizontal stresses somewhat in excess of the at-rest earth pressure. This established a new stress condition which led to significant consolidation of the clay behind the wall.

Water Flow and Ground Loss into Drill Holes

Water flow through the drilled anchorage can result in ground loss particularly in loose fine sand. The magnitude of the ground loss is affected by the hydrostatic head, drilling procedure, and soil conditions. Water flow may also lead to a drop in piezometric level and consolidation of compressibles.

Lateral Creep

Lateral movement, several times greater than settlement and extending relatively large distances behind the face of the excavation, has been reported in highly overconsolidated clays and soft shales. The movement is believed to be associated with lateral expansion following stress relief from the excavation.

A other potential source of lateral creep is in the presence of a weak layer of cohesive soil below the excavation.

1.42 Internal Bracing

1.42.1 Applicability

Internal bracing is most applicable to situations in which a reasonably economical member section can be used without need of intermediate support or in cases where inclined rakers are feasible. As the distance between the sides of the excavation increases, internal bracing becomes less efficient, and therefore tiebacks become more attractive.

1.42.2 Operational Considerations

The most important contingency item is believed to be associated with improper connection details, especially with regard to alignment of members and welding.

Displacements may arise from slack in the support system (consisting of axial compression of the member, deformations in connections, bearing between wale and wall and the adjoining ground). However, this can be largely eliminated by preloading.

Brace removal is another source of displacement. However, this can be controlled by a combination of well planned restrutting and effective compaction of backfill between the wall and the structure.

Preloading to about 50 percent of the design load is common practice in areas where displacements are of concern.

Extreme temperature variations affect load. Reasonable precautions to prevent overstressing can be taken by covering steel members or by painting with reflective silver paint.

1.50 UNDERPINNING

1.51 Applicability

Underpinning of a structure transfers the load from its existing foundation to a new foundation bearing below the zone of influence of the adjacent excavation. Historically, decisions to underpin or not have stemmed largely from the subjective judgements of practitioners. A more rational assessment of related issues can be made on the basis of insight into anticipated displacements at adjoining structures and upon the traditional engineering assessment of cost, expediency, and risk.

1.52 Operational Considerations

It is axiomatic that a thorough study be made beforehand of the structure to be underpinned concerning its load and distribution of load. Temporary conditions that occur during underpinning will also require evaluation. Because the elements pass through a zone undergoing vertical and horizontal displacement, underpinning is not necessarily free from picking up downdrag forces, lateral forces, and/or moving. Lateral movements have proven to be a source of great damage.

A number of factors have the potential of causing ground loss. Lagged underpinning pits for construction of piers have many of the same contingencies mentioned previously for soldier pile walls, especially when aggravated by ground water conditions (see Section 1.32). The potential for ground loss also exists when "blow conditions" develop in open shafts or open-ended piles below ground water table.

1.60 STABILIZATION METHODS

1.61 Scope

This section makes a brief overview of grouting and freezing. Both of these methods are used to control ground water or to solidify a soil mass. Applications may be to create an "arch" over a tunnel or around a shaft or to solidify potentially unstable soils and badly jointed rock encountered within the excavation

Both methods are an "art" performed by specialty subcontractors often with proprietary equipment or material. Details of techniques are not highly publicized, although successful results of applications are.

Performance type specifications are believed to be the appropriate contracting procedure for both grouting and freezing.

1.62 Grouting

Basic soil classification, particularly grain size characteristics, is essential for selecting the type of grout and planning the grouting program. The 15 percent size of soil to be grouted is commonly used as a criterion for grout selection.

Least expensive grouts (cement and bentonite) are used in coarse sand and gravels. Silicates may be used in fine to medium sands. The most expensive are the chemical grouts, which are used for fine sands and coarse silts. In stratified deposits, multi-stage grouting consists of grouting with the cement or bentonite to reduce the permeability of relatively coarse soils followed by successive stages of finer grouts and/or less viscous chemical grouts to penetrate more fine-grained soils.

1.63 Ground Freezing

By and large, ground freezing methods have been used primarily in conjunction with shafts and small diameter tunnels. Frequently, it has been used in difficult situations of ground water where more conventional methods have failed or are inadequate. However, the use of ground freezing as a primary construction method is increasing and is expected to continue to increase in the future.

In evaluating energy requirements for freezing a given zone, the latent heat of fusion of the pore water usually represents the single most important parameter to be considered. It is directly proportional to the water content of the soil.

Creep characteristics of the frozen soil are of interest in deep shafts or tunnels. Creep is related to the stability of the ice structure and displacements outside the frozen zone,

1.70 SOIL AND GROUND WATER CONDITIONS

The following is a brief check list of those soil conditions that have the potential of contributing to additional displacement. Some of these were mentioned above.

1. Drawdown of ground water table: Ground settlement will occur if compressible soils are present,

2. Soft shale and highly overconsolidated clay: This may display lateral creep in tieback installations or may contribute toward load buildup in braced excavations. The high undrained strength of clay should not be counted on for permanent passive resistance on the inside face of the bottom of the cut. Rather, drained strength parameters should be used.

3. Rock within cut: A number of potential problems exist:

- a. Undermining of support wall from rock falls;
- b. Over-blasting below and behind wall;
- c. Difficulty in controlling flow at rock/soil contact or through joints;
- d. Inadquate toe restraint for soldier piles;
- e. Inability to completely dewater overlying soils to top of rock;
- f. Ground water flow through highly jointed zones in the rock: This may depress the ground water table and/or carry fines

(For further discussion see White, 1974).

4. Pervious soils underlain by impervious soil within depth of excavation: This will make it difficult to completely dewater to the bottom of pervious formations. This concern is most relevant to soldier pile walls.

5. Soft clay below excavation: Deformation characteristics of soil ("elastic" range) will cause flexure of the wall below the bottom of the excavation at intermediate stages and at final depths. These uncontrolled displacements represent about 60 percent of the total.

In deep excavations, the imbalance created by load removal causes excessive shear strains in the "plastic" range of stresses.

6. Seepage: Seepage at toe will weaken passive restraint and/or cause ground flow into the excavation.

1.80 COSTS

1.81 Purpose and Scope

This section is intended to provide some general guidelines to enable engineers to make a "first pass" approximation of costs or to make comparisons of alternate schemes. Obviously, these cost guidelines are not precise, and they will vary by geographic area and job conditions.

Costs have been developed on the basis of 1975 prices and labor conditions prevailing in the urban northeast.

1.82 WALLS

	Cost per Sq. Ft.	(Typical Conditions)
	Exposed Wall Only	Exposed with [*] Allowance for Toe
Soldier Piles and Wood Lagging	--	\$ 4 to \$ 7
Steel Sheet Piling		
PZ-27	\$ 6 to \$ 7	\$ 8 to \$ 9
PZ-38	\$ 8 to \$ 9	\$10 to \$11
Concrete Diaphragm Tangent Pile (single row)	\$15 to \$18	\$19 to \$23
Cast-in-place Slurry ¹ Wall (30"± thick)	\$20 to \$35	\$31 to \$44

*When applied to the exposed portion of the wall, this includes carrying the toe penetration to about 25 percent of exposed wall height below the bottom of the excavation.

1.83 Supported Walls

The following discussion presents costs of walls supported with tiebacks or bracing. The upper and lower limits of each do not represent corresponding situations and therefore do not represent the cost differential between the two support methods. In general, tiebacks are slightly more costly; however, many situations exist where tiebacks are less costly. Two examples are rock within the excavation and a wide excavation, such as at a station.

-
- (1) Price variation is relatively insensitive to variations in wall thickness in the range of 2 to 3 feet thick. Difficult excavation in hard materials (till, boulders, weathered rock) will raise costs to from \$40 to \$60 per sq. ft. (Tamaro, 1975).

1.83.1 Tiebacks

Typical tieback costs of small diameter (4 - 6 inches ±, usually percussion drilled) and large diameter anchors (12 - 18 inches ±, usually installed with auger equipment) do not vary greatly. The applicability of one type or the other will generally depend upon soil conditions.

Total cost of tiebacks, including installation and prestressing, is summarized below.

Easy job conditions	\$15 to \$20 per lineal foot
Average job conditions	\$20 to \$25 per lineal foot
Difficult job conditions	\$25 to \$30 per lineal foot

Assuming average tieback lengths of about 50 feet long at \$20 to \$25 per foot, this represents a cost of \$1000 to \$1250 each.

Costs for installed walls, supported by tiebacks and including the wale and connections, are as follows:

Depth (feet)	Cost per Square Foot ¹	
	Soldier Piles and ² Wood Lagging	Interlocked ² Sheet Piles
30 - 40	\$17 to \$22	\$20 to \$27
40 - 50	\$21 to \$26	\$25 to \$32
50 - 60	\$24 to \$30	\$30 to \$40
60 - 70	\$30 to \$40	\$35 to \$45

¹ When applied to the exposed portion of the wall, this includes toe penetration to about 25 percent of the exposed wall height below the bottom of the excavation.

² Water pressure is assumed to act on the sheeting, but is absent from the soldier piles.

1. 83.2 Internal Bracing

Costs for internally braced walls, including wale and connections are as follows:

Depth (feet)	Cost per Square Foot ¹	
	Soldier Piles and ² Wood Lagging	Interlocked ² Sheet Piles
30 - 40	\$15 to \$20	\$18 to \$23
40 - 50	\$20 to \$25	\$23 to \$28
50 - 60	\$25 to \$30	\$28 to \$35
60 - 70	\$30 to \$40	\$35 to \$45

¹When applied to the exposed portion of the wall, this includes toe penetration to about 25 percent of the exposed wall height below the bottom of the excavation.

²Water pressure is assumed to act on the sheeting, but is absent from the soldier piles.

1. 84 Underpinning

General guidelines are as follows:

a. Concrete Pit Underpinning

Installed cost is \$275 to \$350 per cubic yard of concrete.

b. Jacked Pile Underpinning

Installation cost includes cleaning out of piles

Soft material \$125 - \$175 per lineal foot
Hard Material \$150 - \$250 per lineal foot

c. Pali Radice

For piles 4 to 6 inches in diameter:

Easy job conditions	\$20 to \$25 per lineal foot
Average job conditions	\$25 to \$35 per lineal foot
Difficult job conditions	\$35 to \$60 per lineal foot

For piles 8 to 10 inches in diameter, add about 25 percent.

1.85 Ground Freezing

The main factors affecting costs are:

1. Geometry of excavation.
2. Earth and water pressures to be supported.
3. Amount of time available for completion of the excavation support system.
4. Duration of time for which the excavation is to be held open after completion.
5. Union or non-union work rules. (Union work rules, which demand round-the-clock manning of completely automated electrically powered equipment, frequently substantially increase the cost of ground freezing).

Installation of a cut-and-cover frozen excavation support and ground water control system might typically range from \$8 to \$16 per square foot of exposed wall. Maintenance of the system during subsequent excavation and subsurface construction might cost between \$.20 and \$.80 per square foot of exposed wall per week. Underpinning and tunneling costs vary too widely to allow any generalization. As a rule, circular, elliptical, or arch structures in which compression rather than shear or tension stresses govern are least expensive to construct.

1.86 Grouting

The specialized nature of grouting work prevents an accurate estimate of grouting costs. The cost data presented herein was obtained from Halliburton Services (1975).

The cost of the grout materials can be accurately estimated (cement grouts: \$0.50 - \$1.30/ft³; chemical grouts: \$1.50 - \$7.00/ft³); however, the installation costs are not as well known because

of the variables (time to grout, cost of equipment, etc.). Only the grouting contractor has an accurate idea of these costs, which will also vary depending upon the amount of competition. Halliburton (1975) also reports ranges in costs for final volumes of grouted soil (cement grouts: \$13.50 - \$35.00/yd³ of grouted soil; chemical grouts: \$40 - \$190/yd³ of grouted soil).

CHAPTER 2 - SOLDIER PILE WALLS

2.10 INTRODUCTION

Historically, the soldier pile and lagging method was developed in Germany in the latter part of the 19th century and is frequently referred to in Europe as the Berlin Method. The procedure is to drive or pre-excavate and set a vertical member of steel or concrete at spacings normally in the range of about 6 to 10 feet on center. The excavation proceeds in stages of about 1 foot to 5 feet depending upon the ability of the soil to stand in place before lagging is installed. Then horizontal sheeting, commonly called lagging, is placed between the previously installed soldier piles.

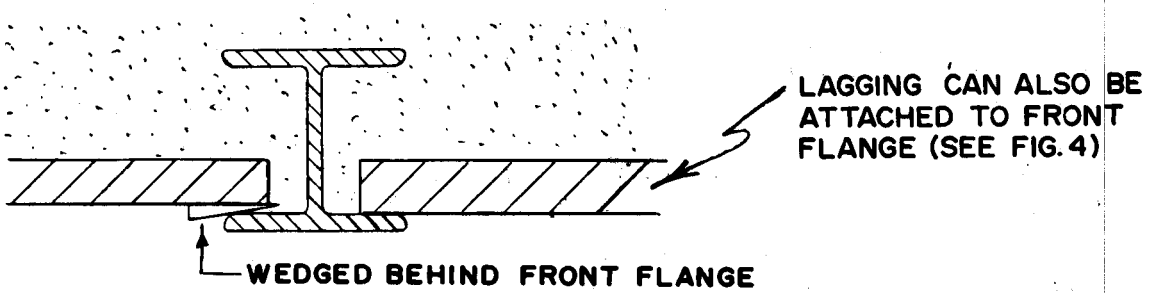
Soldier piles are either installed with pile driving equipment or are set in pre-excavated holes and then concreted in place. The most common soldier piles are rolled steel sections, normally wide flange or bearing pile. But soldier piles can be almost any structural member -- pipe sections, cast-in-place concrete, or precast elements.

When soldier piles are driven, a bearing pile section would normally be used because of the ruggedness of the member, in particular its resistance to twisting and bending. On the other hand, deeper wide flange sections are used where greater stiffness and flexural strength is required in the soldier pile. Conventionally, these are not driven; rather they are set in pre-excavated holes.

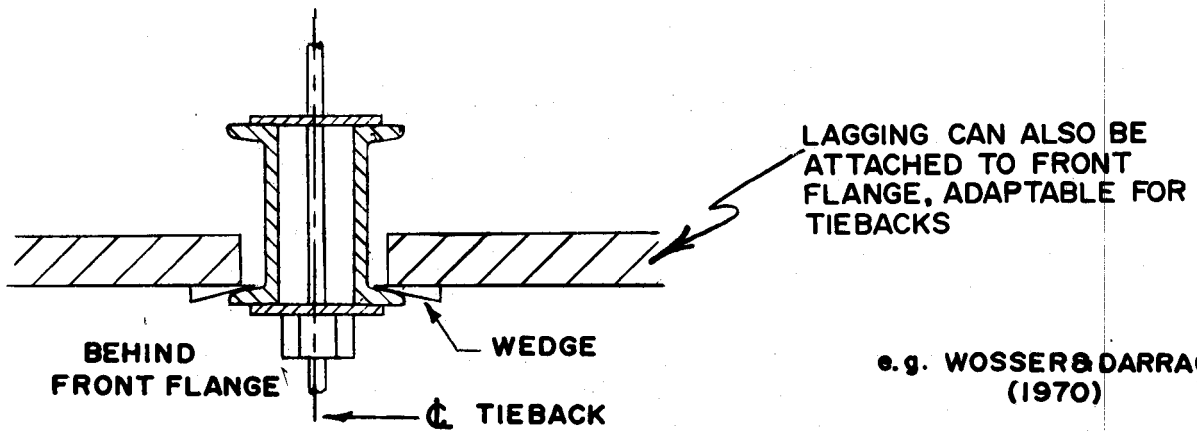
Figure 1 shows various types of steel soldier piles. In addition to wide flange and bearing pile sections, back-to-back channels or pipe sections are also used. Back-to-back channels allow tiebacks to be installed between the channels, thus eliminating wales. Such a setup could not be driven and would have to be installed in a pre-excavated hole as is the case for the wide flange section. An installation of this type was described by Wosser and Darragh (1970).

Pipe sections may be adapted as soldier piles by welding or bolting a T-section to the front of the member to permit the installation of the wood lagging. Pipe sections have also been adapted for installation of lagging along the side of the pipe pile section as described by Donolo (1971). In that case, 34-inch diameter pipe piles were installed in very hard ground with a Benoto caisson rig, and a tieback was drilled through the center of the pipe section.

(a) WF SECTION OR H-PILE SECTION

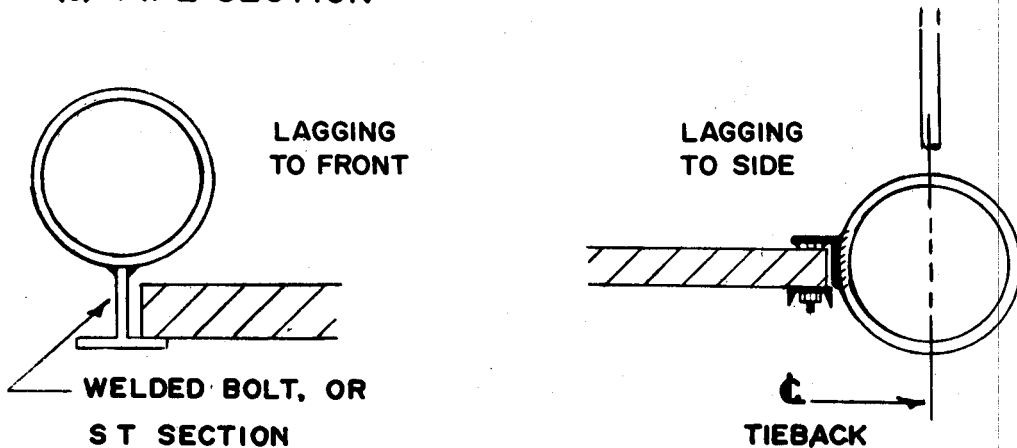


(b) CHANNEL SECTION



e.g. WOSSER & DARRAGH (1970)

(c) PIPE SECTION



e.g. DONOLO (1971)

Figure 1. Steel soldier piles.

2.20 TYPES OF SOLDIER PILE WALLS

2.21 LAGGING

Lagging is most commonly wood, but may also consist of light steel, sheeting, corrugated guard rail sections, or precast concrete.

Wood lagging is most commonly installed either behind or in front of the flange next to the excavation (front flange). It is technically possible to install the lagging behind the rear flange as well. This procedure is not recommended, because the arching action in the soil is destroyed by this process. As noted in Figure 1(a), the lagging can either bear directly against the soil side (back side) of the front flange or it can be wedged to make more intimate contact with the soil and thus reduce associated lateral displacement. Figures 2 and 3 show typical soldier pile wall installations.

Figure 4 shows various methods of attaching lagging to the excavation side (front side) of the front flange. The cases shown employ either a bolt or a T-section welded to the soldier pile or a proprietary method known as "Contact Sheeting"*. In all cases, the vertical plate which holds the lagging can extend up over several lagging boards so that the number of special attachments can be minimized. One distinguishing feature of attaching lagging boards to the front face is that the boards can run continuously across several soldier piles. This, of course, is not possible when installed behind the front flange.

Several examples of cast-in-place concrete soldier piles are shown in Figure 5. The hole is pre-excavated, a reinforcing cage is set, and concrete is poured. This method is uncommon in the United States but has been used in Europe. For instance, the figure shows an example of both a cast-in-place soldier pile and an arched reinforced concrete wall (by Gunitite method) that was used in Sweden and described by Broms and Bjerke (1973).

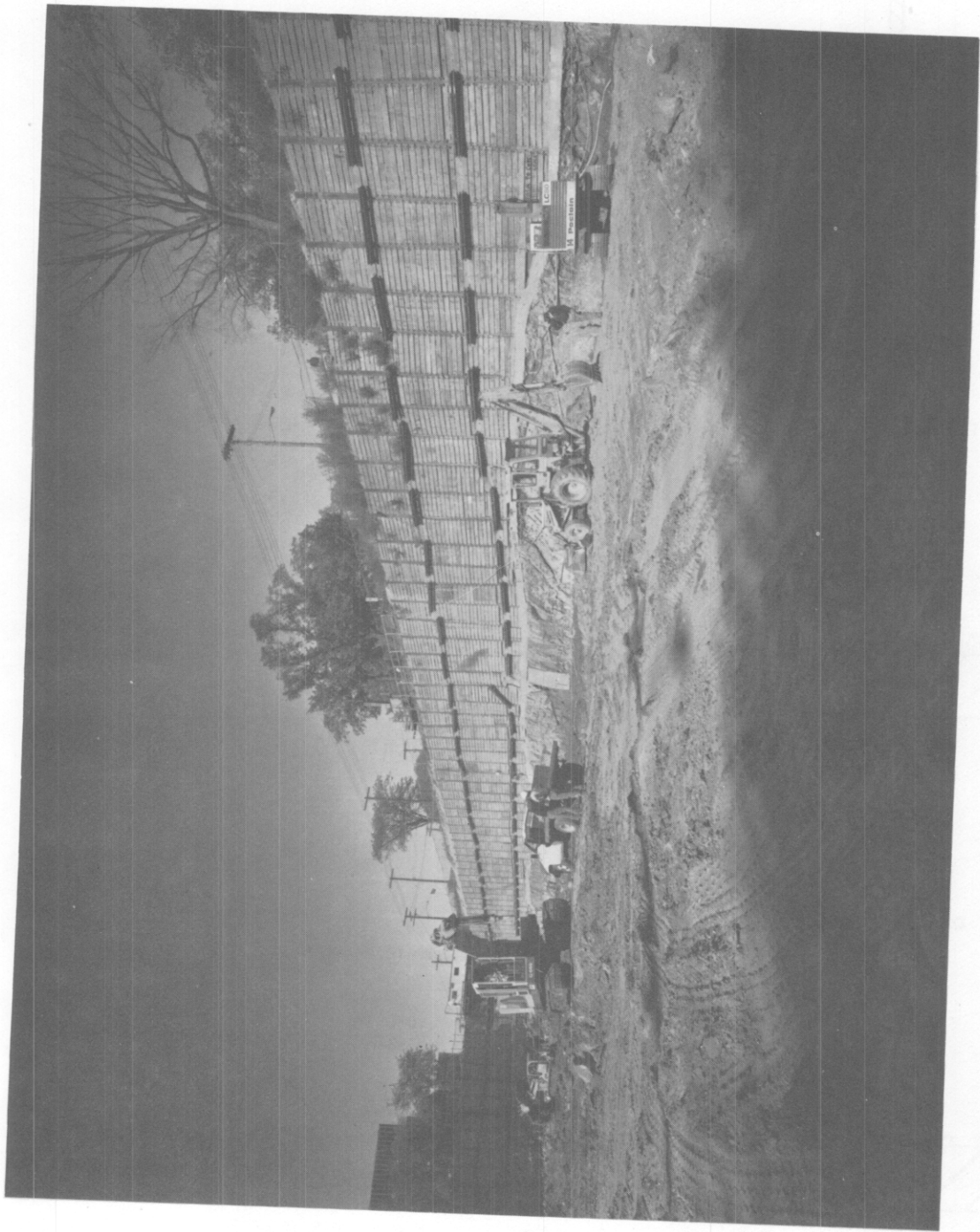
The use of spacers between the lagging boards (called "louvres") allows for the introduction of material for backpacking boards and filtering soil to protect against ground loss from erosion caused by seepage. In ground that is slow draining, the louvres are filled with salt hay. This material permits water to bleed through but also acts as a filter which prevents loss of ground (see Figure 6).

*Contact Sheeting, Inc., Nyack, New York.



Note: 1. Some driven soldier piles out of plumb, probably due to boulders or rock.
2. Hole spacer blocks and open lagging (shuttered lagging).

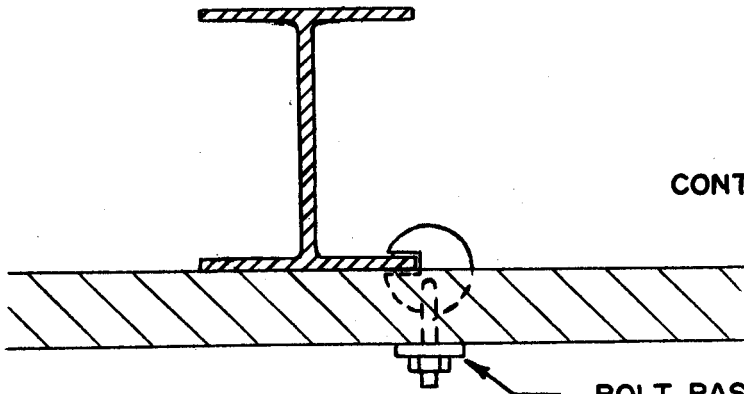
Figure 2. Soldier pile wall (lagging behind front flanges). (Courtesy of Urban Foundation Co., Inc.).



Depth is approximately 25 feet in soil and 15 feet deep in rock. Note upper portion of wall has lagging behind flange; lower part of wall has lagging attached to front of flange.

Figure 3. Soldier pile wall. (Courtesy of Schnabel Foundation Co.,).

(a) CONTACT SHEETING

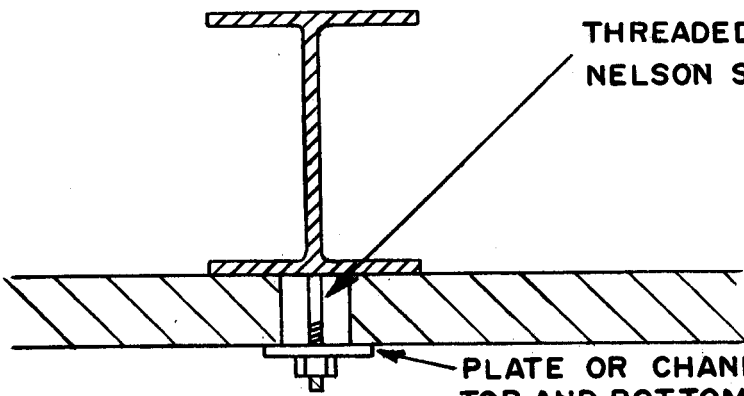


CONTACT SHEETING INCORPORATED

(NYACK, N.Y.)

BOLT PASSES BETWEEN AND PLATE HOLDS THE TWO LEVELS OF LAGGING BOARDS.

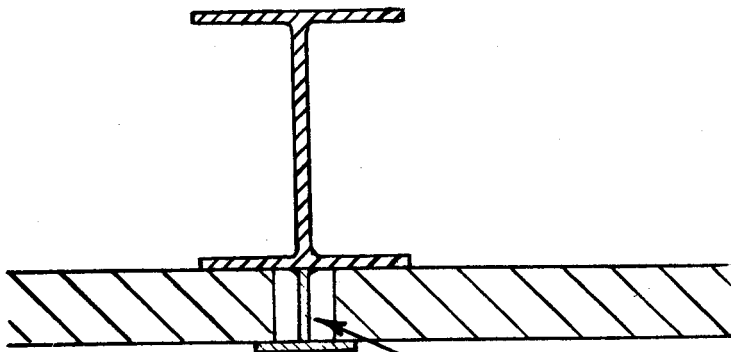
(b) BOLT



THREADED BOLT ATTACHED BY NELSON STUD OR RAM SET.

PLATE OR CHANNEL SECTION HOLDS TOP AND BOTTOM LAGGING.

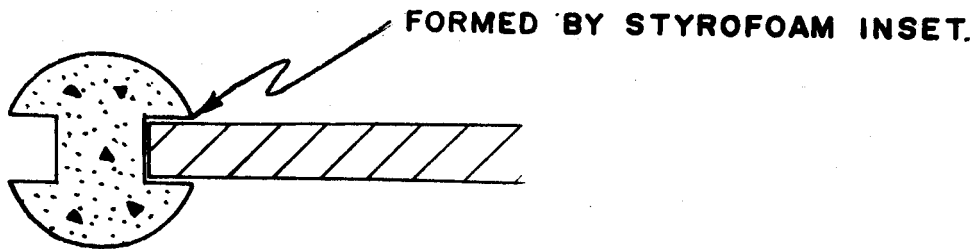
(c) SPLIT T-SECTION



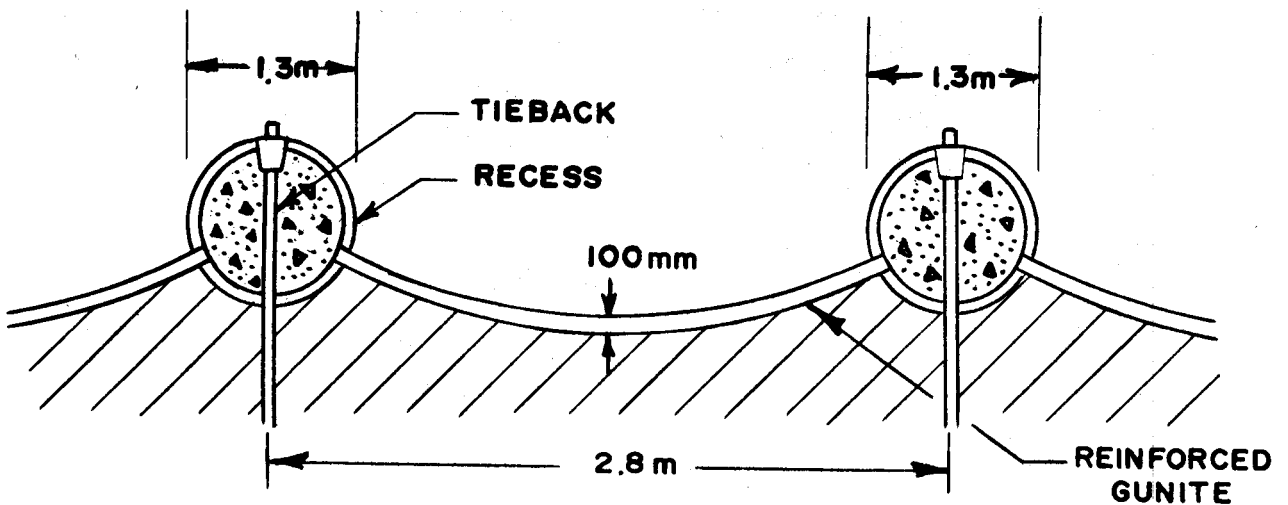
SPLIT "T" WELDED TO FACE

Figure 4. Wood lagging to front flange.

(a) WOOD LAGGING



(b) ARCHED REINFORCED CONCRETE



(AFTER BROMS & BJERKE
1973)

Figure 5. Cast-in-place concrete soldier piles.

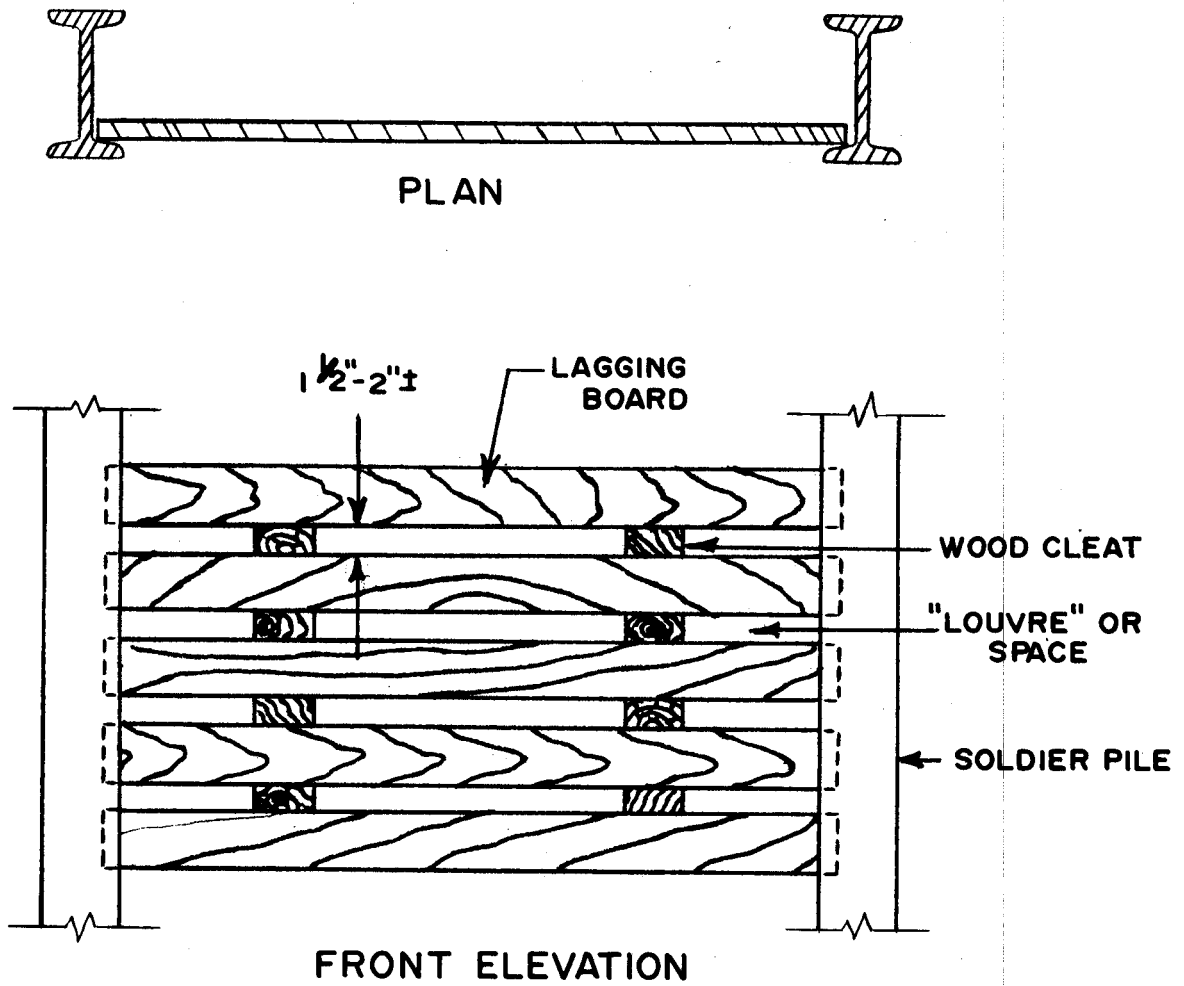


Figure 6. Louvre effect for wood lagging.

2.22 CONCRETE WALL

Examples of shotcrete or poured concrete wall constructed in conjunction with steel soldier piles are shown in Figure 7. An application with precast concrete soldier piles is shown in Figure 8. In general, the typical procedure is to expose about a 5-foot high section and to construct the wall by proceeding sequentially to the bottom of the excavation. In all cases, soil would have to have sufficient cohesion to stand up while the section of the wall is completed.

Figure 9 shows precast soldier piles shaped to receive either wood lagging or precast concrete lagging.

French literature refers to the reinforced concrete infill between soldier piles as a "Parisienne Wall". The wall with precast concrete or horizontal wood sheeting is referred to as a "Berlinoise Wall".

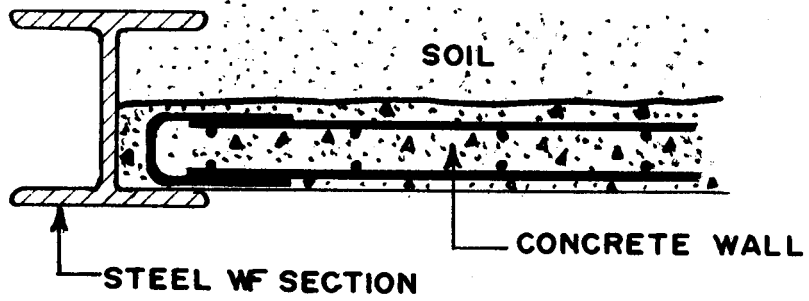
2.23 SOLDIER PILES ALONE

Lagging may not be necessary in hard clays, soft shales, or other cohesive or cemented soils, if the soldier piles are spaced sufficiently close together and adequate steps are taken to protect against erosion and spalling of the face. Examples of this were described by Shannon and Strazer (1970) and by Clough, et al (1972) for cases in cohesive soil in Seattle, Washington. In both cases, soldier piles were set 3 feet on center.

Erosion or ravelling caused by drying of the exposed soil can be inhibited by spraying the exposed soil face. Shannon and Strazer, for example, reported the use of Aerospray 52 Binder. In other cases, tarpaulins may be draped over soil to maintain moisture.

Workmen can be protected by welding wire fencing or wire mesh to the soldier piles to prevent material from falling into the excavation.

(a) CAST IN PLACE



(b) SHOTCRETE



Figure 7. Concrete infill between soldier piles.

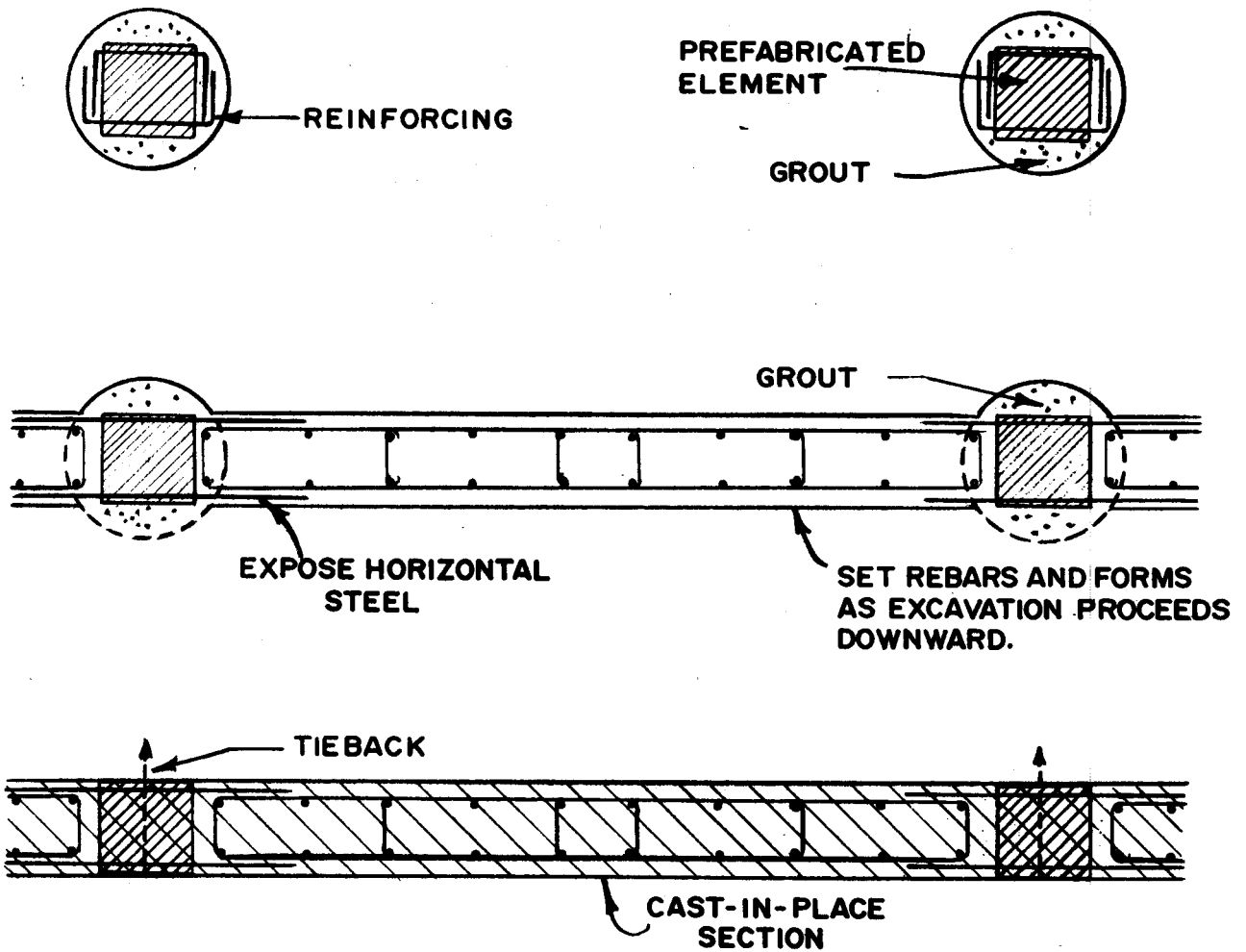


Figure 8. Parisienne wall, precast soldier piles with formed cast-in-place wall, (after Fenoux, 1974; Xanthakos, 1974; and D'Appolonia, et al, 1974).

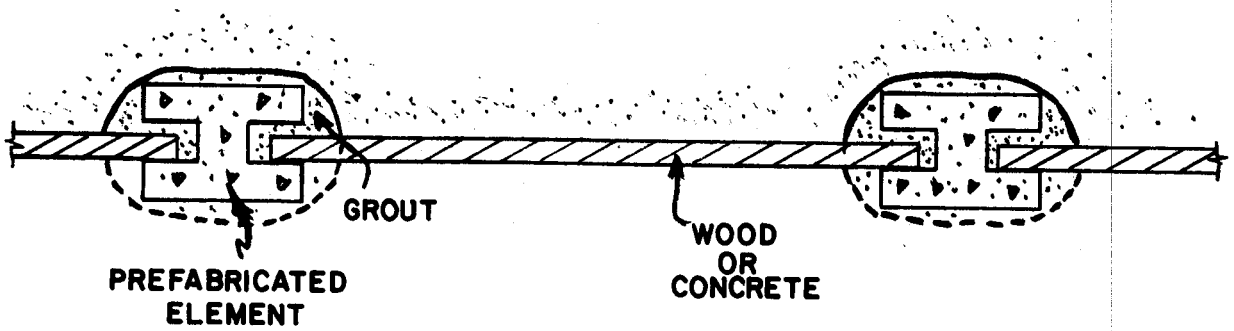


Figure 9. Berlin wall, precast soldier piles with wood or precast concrete lagging (after Fenoux, 1974).

2.30 DESIGN CONSIDERATIONS

2.31 SOLDIER PILES

In addition to their function as support for lagging, soldier piles must also develop vertical flexural strength, lateral resistance below the level of the last strut or tieback level, and in the case of inclined tiebacks bearing to support the vertical component of tieback force.

Design recommendations for soldier piles are presented in Volume II (Design Fundamentals).

2.32 WOOD LAGGING

2.32.1 Wood Materials

The most common wood used for lagging in the United States is construction grade, usually rough-cut. Structural stress-graded lumber may be specified though seldom used. Preferred woods are Douglas Fir or Southern Yellow Pine, both of which provide a desirable balance between flexural strength and deformation modulus. Hardwoods, such as oak, are less common. Although they are strong, they are also very stiff and heavy.

Table 1 lists the properties of some woods that may be used for wood lagging. The allowable flexural stress stated in the table is for normal or repetitive use construction.

2.32.2 Arching

Experience has shown that lagging installed in the conventional manner in most reasonably competent soils does not receive the total earth pressure acting on the wall. The lateral earth pressure concentrates on the relatively stiff soldier piles; less pressure is applied to the more flexible lagging between the soldier piles. White (in Leonards, 1962) discusses this point based upon many years of practical experience on a great number of jobs under different conditions.

This redistribution of pressure, known as arching, is inherently related to the usual manner of construction. The lagging is supported on the front flange; a slight overcut is made behind the lagging to facilitate placement of the boards; and the intervening space behind the boards is filled with soil. The soil should be packed tight; however, packing of the soil does not induce flexure. Flexure comes about as earth pressure builds up on the wall as the excavation deepens. This flexure causes a redistribution of load resulting in a decrease of pressure near the center where flexure is the greatest and a corresponding increase near the ends of the board near the soldier pile.

Table 1. Strength properties for typical grades of lumber.

Wood Type and Grade	Allowable Flexural Stress f_b , psi	Modulus of Elasticity E, psi
<u>Douglas Fir - Larch</u> , surfaced dry or surfaced green used at max. 19% M. C. Construction Select Structural	1200 2050	1,500,000 1,800,000
<u>Douglas Fir - South</u> , surfaced dry or surfaced green used at max. 19% M. C. Construction Select Structural	1150 1950	1,100,000 1,400,000
<u>Northern Pine</u> , surfaced at 15% moisture content, used at 15% max. 19% M. C. Construction Select Structural	1050 1750	1,200,000 1,500,000
<u>Southern Pine</u> , surfaced at 15% moisture content K. D., used at 15% max. M. C. Construction Select Structural	1300 2250	1,500,000 1,900,000
<u>Southern Pine</u> , surfaced dry, used at max. 19% M. C. Construction Select Structural	1200 2050	1,400,000 1,800,000

* American Institute of Timber Construction, "Timber Construction Manual", 2nd Edition, Wiley, 1974.

A related phenomenon is that the pressure on lagging is relatively unaffected by depth. It therefore follows that the greater forces associated with deeper excavations must be transmitted through piles. Again, this is attributed to arching.

To take advantage of arching, the excavation should not be made behind the rear flange of the soldier pile. During excavation behind the soldier pile, the point of load concentration is removed, and the stress conditions for arching are destroyed. Simply stated, the abutment of the arch is removed. (See Peck, 1969).

2.32.3 General Practice Concerning Lagging Thickness

Lagging thickness design is based primarily upon experience and/or empirical rules. One procedure is to vary the amplitude of the pressure diagram with maximum pressure at the soldier pile and minimum pressure midway between the soldier piles (see Lacroix and Jackson, 1972). Another procedure is to reduce the basic pressure diagram used in the design of bracing and/or tiebacks by applying a reduction factor. For example, Armento (1972) in designing lagging for the BARTD system; applied a 50 percent reduction factor to the basic trapezoidal earth pressure diagram used for strut design. The New York City Transit Authority uses the basic pressure diagram but allows 50 percent increase in the allowable flexural stress of stress-graded lumber.

Some examples of empirical design rules used in practice are listed below. The examples are presented to show a range of usage and are not intended to be final recommendations.

a. White (1973) suggests a 3-inch lagging thickness for excavations in sandy soils for soldier piles spaced from about 6.5 feet to 10 feet on centers. He also suggested a thickness of 4 inches when in soft clay for soldier piles spaced about 5 feet to 6.5 feet on centers. These recommendations apply to depths of about 50 feet.

b. Chapman, et al (1972) report the use of 3-inch lagging for soldier piles 9 feet on centers in Washington, D. C. soils. The typical soils include stiff clays and medium dense sands, and the excavation was 41 - 49 feet deep.

c. Ware, et al (1973) describe requirements for lagging for the Washington Metro System. For soldier piles 6 to 7 feet on center, the required thickness was 3 inches to 25 feet and 4 inches below 25 feet, using timber with an allowable 1100 psi flexural stress. The walls were primarily in competent granular soils to depths of about 30 feet and in stiff to very stiff clays below 30 feet.

d. Wösser and Darragh (1970) report lagging thicknesses from 3 inches to 6 inches using Douglas Fir with an allowable flexural stress of 2000 psi and with soldier piles 8 feet on center. The depth of the excavation was 60 feet, and the soils were typically sandy. Thickness of lagging was varied with depth, and 8 inch lagging was used near the bottom of the excavation in some of the clay areas.

e. In an excavation in soft clay, Insley (1972) reported using lagging thicknesses of 4 inches to a depth of 22 feet and 6 inches to a depth of 30 feet for soldier beams spaced 6 feet on center. Based upon data presented, the computed ratio of overburden stress to undrained shear strength was about 5.5.

2.32.4 Recommended Lagging Thickness

Based on the above discussion, upon other empirical rules that have been reported, and in consideration of the various soil conditions that may be encountered, recommended thicknesses are given in Table 2. Since the table has been developed on the basis of construction grade lumber, adjustments are required for stress graded structural lumber.

The recommendations given in the table are primarily for cases where there is a need to limit displacements to protect existing facilities adjacent to an excavation. They are therefore, by necessity, more conservative than what could be successfully used in cases where this criterion for protection did not exist.

The so-called "competent soils" shown in the table are typically either granular with relatively high angles of internal friction or stiff to very stiff clays. Medium clays included in the table have a ratio of overburden stress to undrained strength of less than 5.

The category of "difficult soils" includes loose, granular soils with low angles of internal friction, such as loose sands and silty sands. The table also includes soils which may pose some difficulty during construction below the ground water table. Some are clayey sands, cohesionless silts, and fine sands, all of which drain slowly and may have a tendency to run. Finally, this group includes heavily over consolidated fissured clay. Typically, this group of materials may have a K_0 value in excess of 2 or 3. Heavy over consolidated soils have a tendency to expand laterally especially when a deep excavation is made. Also, the fissuring may contribute to a loss of strength because of the affinity of the soil for water following a decrease of effective stress caused by the excavation

Table 2. Recommended thicknesses of wood lagging.

	Soil Description	Unified Classification	Depth	Recommended Thicknesses of Lagging (roughcut) for Clear Spans of:					
				5'	6'	7'	8'	9'	10'
COMPETENT SOILS	Silts or fine sand and silt above water table	ML SM-ML	0' to 25'	2"	3"	3"	3"	4"	4"
	Sands and gravels (medium dense to dense).	GW, GP, GM, GC, SW, SP, SM							
	Clays (stiff to very stiff); non-fissured.	CL, CH	25' to 60'	3"	3"	3"	4"	4"	5"
	Clays, medium consistency and $\frac{\gamma H}{S_u} < 5$.	CL, CH							
DIFFICULT SOILS	Sands and silty sands, (loose).	SW, SP, SM	0' to 25'	3"	3"	3"	4"	4"	5"
	Clayey sands (medium dense to dense) below water table.	SC							
	Clays, heavily over-consolidated fissured.	CL, CH	25' to 60'	3"	3"	4"	4"	5"	5"
	Cohesionless silt or fine sand and silt below water table.	ML; SM-ML							
* POTENTIALLY DANGEROUS SOILS	Soft clays $\frac{\gamma H}{S_u} > 5$.	CL, CH	0' to 15'	3"	3"	4"	5"	--	--
	Slightly plastic silts below water table.	ML	15' to 25'	3"	4"	5"	6"	--	--
	Clayey sands (loose), below water table.	SC	25' to 35'	4"	5"	6"	--	--	--

Note:

* In the category of "potentially dangerous soils", use of lagging is questionable.

The final grouping includes "potentially dangerous soils," which may run and lead to loss of ground. Normally, soldier pile walls are the least desirable alternative in these soils. Typical problem soils are:

a. Soft Clays. Soft clays with a ratio of overburden stress to undrained shear strength greater than 5. The consequences of excessive shear deformation increases with sensitivity of the soil. Assuming an average total unit weight of about 110 pounds per cubic foot, the approximate shear strength value associated with the ratio of 5 at depths of 15, 25, and 35 feet are respectively 330, 550, and 770 psf. Peck (1969) has shown that as the ratio of total overburden stress to undrained strength approaches 7, there is marginal safety, and the soil may be on the verge of incipient failure in tunnels without air pressure. Moreover, the tunneling proceeds without unusual difficulty provided the ratio is less than 5.

b. Dilatant Soils of Low Plasticity. This category includes slightly plastic silts and loose clayey sands below the water table. Both of these highly dilatant materials and upon disturbance would be expected to experience an increase in pore pressure. This would result in a loss of effective stress and therefore a loss of strength. Moreover, because of poor drainage characteristics, they may flow and lead to ground loss. Commonly, they are known as "running" soils.

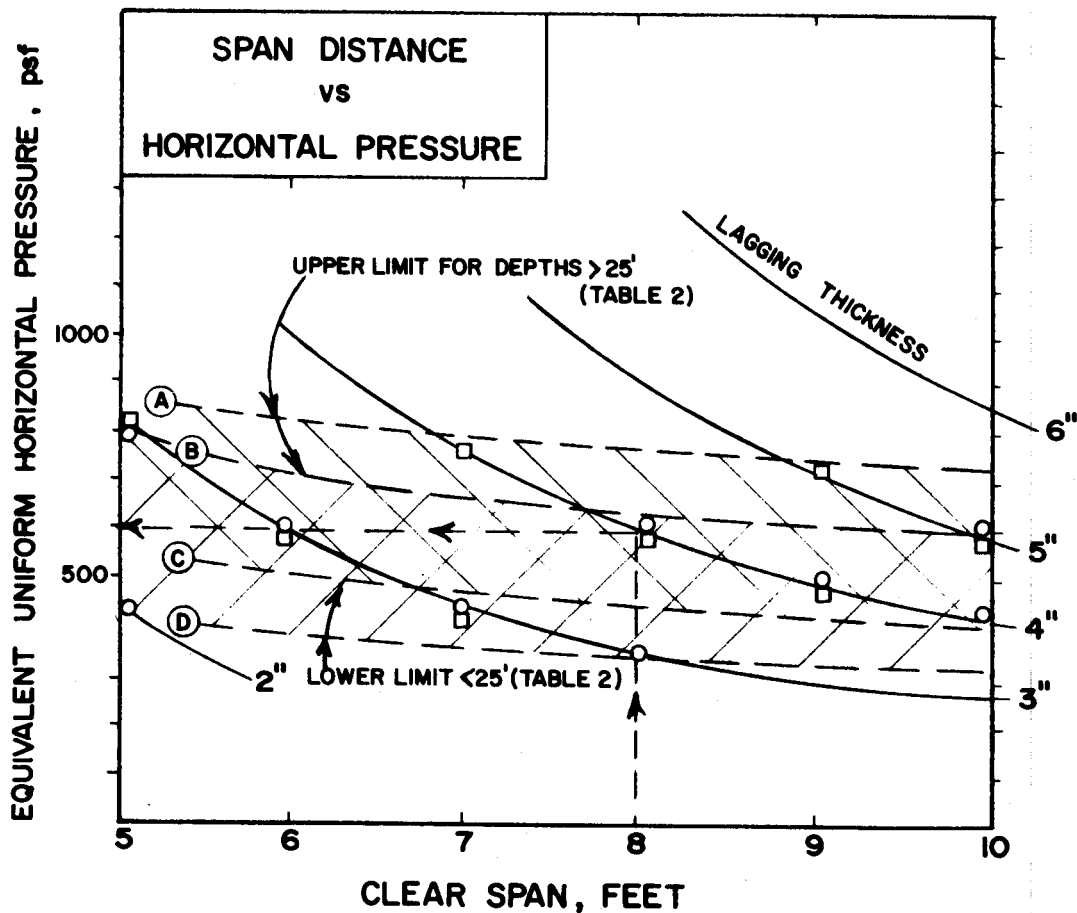
2.32.5 Equivalent Uniform Pressure

The concept of an equivalent uniform horizontal pressure acting on lagging is useful in illustrating the effectiveness of arching. With verification by field data, the equivalent uniform horizontal pressure could also be used as a basis for consideration of different grades of lumber and as an index of lagging deflection.

In Figure 10 the flexural stress has been arbitrarily assumed to be 50 percent above the normal working stress of construction grade lumber. This represents the approximate upper limit of what a designer would allow for temporary construction. Actual flexural stress could be more or less than the above limits, because the failure stress may be several times the normal working stress.

The process used in preparing Figure 10 was as follows:

a. A family of curves was developed relating the lagging thickness, required to limit flexural stress (1800 psi), to various clear spans. The 1800 psi figure is based on a 50 percent overstress value applied to the normal extreme fiber working stress of about 1200 psi for construction grade Douglas Fir or Southern Pine.



2. Given a lagging thickness and span distance, the equivalent uniform horizontal pressure causes a flexural stress of 1800 psi; e. g. given 4 inch lagging and 8 foot clear span, a uniform pressure of 600 psf causes a flexural stress of 1800 psi.

3. Implied range for "competent" soils (Table 2).

4. Implied range for "difficult" soils (Table 2).

Figure 10. Lagging thickness required to limit flexural stress.

b. The points were plotted from Table 2. These correspond to the recommended lagging thickness for different clear span values.

By comparing the theoretical computation from (a) with what works in practice, from (b) one can infer an equivalent uniform pressure.

2.32.6 Discussion

To illustrate the effect of arching, consider a 40 foot deep cut in "competent" soils given in Table 2.

$$\text{Let: } \gamma = 130 \text{ pcf}$$
$$K_a = 0.30$$

where:

$$\gamma = \text{unit weight}$$

$$K_a = \text{coefficient of active earth pressure}$$

Assuming active earth pressure conditions, the horizontal pressure at 40 feet would be as follows:

$$\sigma_h = 0.30 \times 130 \times 40 = 1560 \text{ psf}$$

Compare this pressure with the equivalent uniform horizontal pressure inferred from Figure 10. As an example, consider a clear span of 8 feet between soldier piles. From Figure 10, the equivalent horizontal pressure causing a flexural stress of 1800 psi is about 600 psf, which is less than one-half the active pressure.

To consider stress graded structural lumber, use the equivalent uniform pressure from Figure 10. To be consistent with the basic assumption of Figure 10; allow 50 percent above normal working stress.

As an example, consider the following:

- a. Given: 1. Sand and gravel;
2. Excavation 50 feet deep; and
3. Soldier piles to be set at 10 to 11 feet on center

- b. Find: 1. Lagging thickness for soldier piles at 10 feet and 11 feet on centers, using construction grade lumber.
2. Repeat for structural grade Douglas Fir (normal working flexural stress $f_w = 2000$ psi).
- c. Find lagging thickness for construction grade lumber from Table 2.

Soldier pile spacing (feet)	10	11
Approx. clear span (feet)	9	10
Lagging thickness for "competent soil" (inches)	4	5

- d. Find lagging thickness for structural grade lumber.
1. Compute moment using equivalent horizontal pressure from Figure 10.

$$M = \frac{w l^2}{8}$$

where:

w = uniform pressure, psf

l = clear span, feet

M = moment, foot-lbs.

Soldier pile spacing (feet)	10	11
Approx. clear span (feet)	9	10
w (psf), using upper limit* curve from Figure 10	600	600
Moment, foot-lbs.	6060	7500

* Note that the upper curve will produce conservative results for clear spans at 7 and 9 feet in "competent soil".

2. Compute section modulus for 3 and 4 inches thick by 12 inch wide lagging.

$$S = \frac{b h^2}{6}$$

where:

b = 12 inches

h = 3 or 4 inch thickness

$$\text{For 3 inch thickness, } S = \frac{(12)(3)^2}{6} = 18 \text{ in}^3$$

$$\text{For 4 inch thickness, } S = \frac{(12)(4)^2}{6} = 32 \text{ in}^3$$

3. Check stress

$$\text{Allowable} = 1.5 \times 2000 = 3000 \text{ psi}$$

Check 9 foot clear span

$$3'' \text{ inch lagging: } f = \frac{M}{S} = \frac{6060 \times 12}{18} = 4040 \text{ psi}$$

This exceeds allowable stress. Use 4'' lagging for a 9 foot clear span.

Check 10 foot clear span

$$4 \text{ inch lagging: } f = \frac{M}{S} = \frac{7500 \times 12}{32} \approx 2800 \text{ psi}$$

This is ok. Use 4'' lagging for a 10' clear span.

e. Summary

Soldier pile spacing (feet)	10	11
Approx. clear span (feet)	9	10
Lagging thickness (inches)		
Construction grade	4	5
Structural grade	4	4

f. Adopt

1. Construction grade: soldier piles at 10 feet on center with 4 inch lagging, or soldier piles at 11 feet on center with 5 inch lagging.
2. Structural grade: soldier piles at 11 feet on centers with 4 inch lagging.

2.33 Displacements and Loss of Ground

2.33.1 General

During construction of soldier pile and lagging walls the soil face must be exposed to install lagging and, in most instances, the lagging allows drainage of water behind the excavation. Because of the characteristics of a soldier pile wall, unfavorable soil conditions can lead directly to ground loss and deformation.

Important factors contributing to ground loss are the soil in the zones immediately behind the lagging and the flexure of the lagging board itself. The following discussion concerns ground loss caused

by the inherent characteristics of soldier pile walls, in particular the techniques used in their construction. The discussion does not deal with overall deformations of the retained earth mass.

2.33.2 Deflection of Lagging

The lagging board thicknesses recommended in Table 2 will generally maintain deflection to less than about 1 inch. Because of the empirical nature of Table 2 and Figure 10, the equivalent uniform pressure developed in the latter figure should not be used for a computation of the absolute value of deflection.

Because of arching, it is probable that the influence zone from lagging deflection is limited to the vicinity of the soldier pile wall in the "competent" soils listed in Table 2. When arching is not likely, such as for the "difficult" soils listed in Table 2, the influence zone from lagging is not limited to the locale of the wall.

2.33.3 Overcut

In order to physically install a lagging board, it is necessary to provide a clear space behind the board so that it can be fitted properly in place. Whenever there is concern about the effect of displacements on adjoining structures, this space must be filled (backpacked) to develop intimate contact with the soil.

An example of movement from overcut was reported by Prasad, et al (1972). In that case, during prestressing of tiebacks, the soldier pile and lagging wall moved about 1/2 inch to 2 inches toward the unexcavated soil. Similar behavior has been observed by many practitioners under similar circumstances which is caused by poor backpacking. The most effective way of backpacking is to ram the soil into the space from the upperside of the lagging board. If there is difficulty in obtaining sufficient cohesion in the material rammed in this manner and/or there is concern with future washout from ground water action, the soil can be mized with cement and dry packed. Louvres are also helpful because they allow backpacking from the top of the board as well as from the underside. Also, they provide an opportunity to take remedial measures to improve filtering or to correct for ground loss behind previously installed lagging.

2.33.4 Inherent Soil Properties

Those soils which, by virtue of their natural characteristics, may produce excessive strains during excavation are soft clays and loose soils of low plasticity below the water table.

The physical act of exposing a face below the last placed lagging board may result in deformation even while the excavation is being made.

An example of a rather dramatic failure in soft sensitive clay was reported by Broms and Bjerke (1973). The failure took place at a depth of approximately 30 feet where soft clay actually squeezed through the opening between adjacent soldier piles after the face of the clay had been exposed for a period of 1 to 3 days. The ratio of total overburden stress to undrained strength was about 6.3. In another case, Broms and Bennemark (1967) reported a shear failure through a 6.5 foot diameter opening in soft clay about 1-1/2 hours after exposure. The slide buried 3 men; one of whom was killed. The ratio of overburden stress to undrained strength exceeded six.

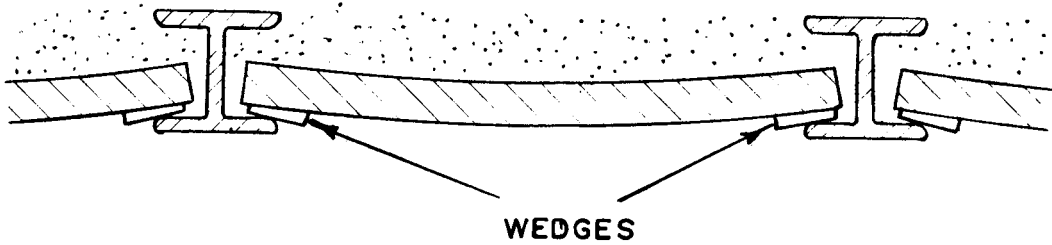
Examples of a German procedure for dealing with soft unstable soil are shown in Figure 11.

In (a) of the Figure the soil between the soldier piles is shaped in a slightly curved manner using a special steel form. Double wedging is used behind the flange of each soldier pile, and the lagging board is thus pulled tightly against the soil. The second case shown in (b) is an example of cast-in-place concrete to provide the stiffness necessary to limit deformation and to form intimate contact with the excavated soil. In (c) of the Figure the procedure is to drive short vertical sheets and to wedge behind horizontal wales attached to the soldier piles. This procedure effectively prestresses the soil.

The extent of stress relief from arching that occurs with very soft soils and soils subject to plastic creep is certainly in question. Therefore, the pressure used for lagging design should be determined directly from the basic pressure diagram used for design of struts and vertical members. Such a severe design condition would make it highly unlikely that lagging would be selected in the first place over sheeting or a diaphragm wall.

In dealing with interbedded silts and other soils that are difficult to drain, one obvious procedure is to dewater long in advance of excavation. An alternative procedure would be to continuously maintain a sloped berm from the inside face of the soldier piles and to pump from open sumps installed at the lowest portion of the excavation. If these procedures do not prove successful, then it may be necessary to employ special precautionary methods, such as the German technique shown in Figure 11c.

Dry cohesionless soil may also lead to difficulty, especially in hot, arid areas. Under these circumstances, one technique



(b) CONCRETE



(c) VERTICAL STEEL SHEETS IN SOFT SOIL

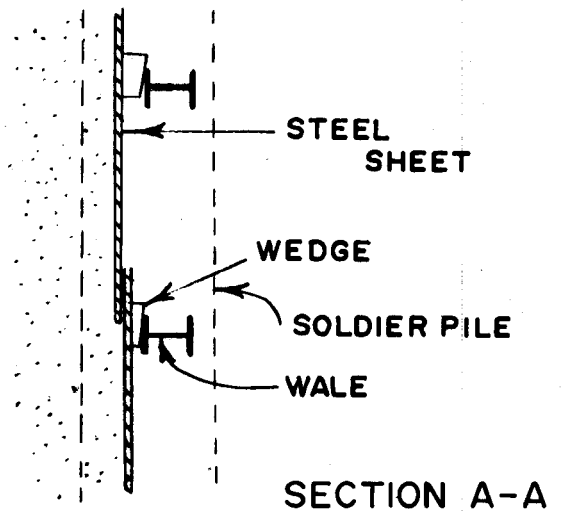
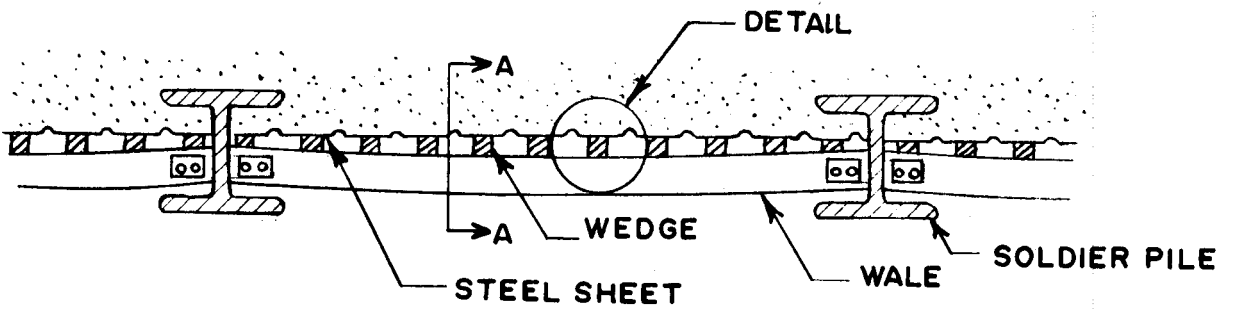


Figure 11. German techniques to prevent deformations (after Weissenbach, 1972).

is to moisten the face by spraying while placing the lagging. One may also use a board such as plywood to hold the soil temporarily in place while setting lagging.

2.33.5 Pre-excavation for Soldier Piles

There are several potential causes of material loss during pre-excavation.

One cause is from the suction effect that occurs during withdrawal of the auger. This may cause soil to squeeze into the hole. One way to prevent this is to provide ports within the auger which will prohibit the suction from developing below the auger. Another is to apply pressure to the inner hole of a hollow stem auger as it is withdrawn.

A second cause of ground loss is from collapse of the soil into the augered hole. This can be prevented by using a casing or a bentonite slurry suspension to stabilize the hole, especially when a positive slurry head is maintained above the ground water table.

A third possible cause of ground loss is from improperly filling the pre-excavated hole. Normally, the filling is done with lean concrete or grout. Cases have been observed in which ground water or surface water concentrated along improperly filled holes, flowed downward alongside the hole, emerged out from the space between lagging boards, and carried out a significant quantity of soil.

2.33.6 Surface Water and Ground Water

The importance of properly sloping the top surface so that surface water drains away from, rather than towards, the excavation cannot be overemphasized. Surface water tends to concentrate in local zones and become channelized once a path of flow develops. This in turn may lead to ground loss. In that connection, the German code (DIN 4123, 1972) calls for excavation walls at least 2 inches higher than surrounding ground in order to maintain drainage.

Other situations arise from leaky or broken sewers or backed-up municipal storm drainage during heavy rain. Such conditions are contingency items that may or may not be within the contractor's control. It is the responsibility of all parties to investigate the probability of such contingencies -- particularly where structures abut the excavation.

In any water-bearing formation it is absolutely essential that the ground be drained prior to exposing the face. The consequences of doing otherwise could be substantial ground loss. The depth of cut below the

water table, the porosity and permeability of the soil, and the presence of underlying or interbedded impervious layers must all be considered in devising a dewatering scheme.

In soils which drain very slowly, the excavation face can only be advanced about one foot at a time. The bottom of the cut is sloped in a V-shaped fashion to allow for surface drainage and to aid in depressing the phreatic surface at the side of the excavation. Such procedures have been used successfully in silt deposits in New York City (known locally as "Bull's liver").

When impermeable layers are interbedded with more pervious layers, ground water is more difficult to control. The ground water tends to flow for a relatively long period of time just above the impervious layer (or layers) or an interbedded formation. This condition is normally controlled by wells which intercept flow before it reaches the excavation.

In very severe instances, one possible protective measure would be to install a series of vertical drains which successively penetrate the various layers and to intercept horizontal flow before it emerges through the open lagging at the soldier pile wall. Overall these situations require the use of judgment to determine the feasibility of open lagging. Alternatives of interlocked sheeting or diaphragm walls must be considered.

Protection against water erosion through lagging is commonly done by a combination of effective backpacking and placement of salt marsh hay in the open space between the lagging boards to filter out the soil. Another way to prevent erosion is to use porous concrete as a filter behind the lagging. Such a procedure was reported by Mansur and Alizadeh (1970).

Figure 12 illustrates a case where water seepage through the soldier pile wall caused ground loss behind the wall. The ground loss was not severe and was controlled without damage.



Note: Shuttered lagging and packing of hay into spaces between boards. Running soil and ground loss below bottom board. (In this case total ground loss was not great and no damage was caused).

Figure 12. Soldier pile wall.
(Courtesy of Spencer, White, and Prentis).

2.40 CONSTRUCTION CONSIDERATIONS

2.41 SOLDIER PILES

2.41.1 Driven Soldier Piles

Conventional pile driving equipment may be used to drive soldier piles. Some of the drawbacks are as follows:

- a. The noise factor.
- b. Misalignment caused by deflection or twisting upon hitting underground obstructions or in penetrating hard ground.
- c. Vibration.

The more compact and heavy the steel section, the less likely twisting will occur. Therefore, bearing pile sections are the most desirable for driving. In hard ground these may be equipped with a driving point in order to help penetration through boulders and/or to get sufficient depth for adequate lateral resistance or bearing capacity. Bearing capacity is particularly important where soldier piles accept the vertical component of tieback force. Dietrich, et al (1972) report a case where the soldier piles settled more than 2.5" from the vertical component of the tieback load.

One possible means of avoiding the noise problem is by using vibratory hammers or impulse driving hammers specially designed to reduce the noise level. An impulse hammer is currently under development by Stabilator AB of Stockholm, Sweden, as reported in World Construction (April 1974).

With reference to potential settlement of the adjoining ground, there is some evidence to suggest that vibratory or double acting hammers may be more detrimental than single acting hammers. The latter delivers high energy per blow but acts at a lower frequency.

2.41.2 Soldier Piles Set in Pre-excavated Holes

Pre-excavated holes may be used for one or more of the following reasons:

- a. To reduce noise and vibrations.

- b. To penetrate a hard layer.
- c. To set a long soldier pile in the ground so that it can conveniently fit in the leads of a pile driving rig for further driving.
- d. To set the soldier pile at a precise location.
- e. To install certain types of soldier piles such as deep-web, torsionally flexible, wide flange sections, which otherwise may be difficult to drive.
- f. To minimize vibrations which could have an adverse effect on loose unconsolidated sediments and nearby structures.
- g. To penetrate sufficiently far below the bottom of the excavation to ensure lateral toe resistance and vertical bearing. Such considerations may necessitate percussion or rotary drilling to penetrate rock or boulders.

Badly fractured rock lying within the depth of excavation must be penetrated in order to avoid the risk of undermining the soldier pile during the process of rock excavation. Observations made, for example, in connection with the Washington Metro Project indicated the need to underpin certain soldier piles as a result of rock falling from below the soldier pile during excavation. Subsequently, soldier piles were required to penetrate below the bottom of the excavation to avoid such contingencies.

Pre-excitation is usually done with augers. Equipment used for augering may be bucket type augers at the end of a Kelly bar or continuous hollow stem augers. In either case, to avoid ground loss during withdrawal, a positive pressure should be applied to the inner hole of a hollow stem auger. Ports should be incorporated with the bucket type auger at the end of the Kelly bar to equalize pressure, and the hole should be maintained full with drilling mud. In hard ground, augers may not be practical. Percussion drilling or rotary drilling may be necessary.

Pre-excavated holes facilitate setting the soldier piles to a very close tolerance, both on line and with respect to verticality. When alignment is critical the soldier pile is set within the pre-excavated hole by means of a centering spider.

It is common practice to use structural concrete below the level of the excavation to assure vertical bearing the lateral resistance against kick out. Lean concrete can then be used for the rest

of the hole. However, it is believed that properly placed lean concrete can be just as effective below the level of excavation at the pile. Surely, lean concrete is at least equivalent in strength to most natural soil formations. Pouring concrete through water is totally unacceptable if ground loss during the course of future excavation is of concern; therefore, placement must be by tremie. Dry holes can be poured through a funnel that regulates placement rate. Rapid discharge without a funnel is discouraged because the concrete may "hang-up" by arching between the pile and outer wall, unless of course the concrete is placed first.

Lean concrete must be sufficiently strong to prevent collapse of the hole, yet weak enough to be excavated easily. A lean concrete mix is normally about 1 to 2 sacks of cement per cubic yard.

2.42 INSTALLATION OF LAGGING

Typical procedure is to dig below the last section of installed lagging, to remove the soil carefully, and then to slide the lagging boards in place.

To minimize over cut, hand tools should be used to shape the soil and to fit the lagging board in place. If necessary, wedges can be used to tighten up between the lagging board and its bearing area.

Depth of exposure below the last placed lagging may be as little as 1 foot, as in the case of saturated silts, or as much as 4 or 5 feet in cohesive hardpan. The German code (DIN 4124, 1972) allows an exposure of only 1/2 meter except in stiff cohesive soil where 1 meter is allowed.

In circumstances of adverse soil conditions, proper cutting of the soil bank, backpacking of soil behind the lagging, and filling the vertical space between lagging boards with a proper filtering and drainage material are all important details. Open, or louvered lagging, ensures proper drainage and at the same time, when properly installed, aids in preventing ground loss.

2.43 REMOVAL

There is a divergence of opinion among practitioners as to whether or not untreated wood can be left in place permanently above the ground water table. Some claim that deterioration of the wood leads to lateral movement of soil and therefore ground settlement. Others point to many examples of the wood remaining intact. If decay has occurred, it has been observed that the fabric of the wood remains strong enough to provide the necessary resistance to prevent closing the space occupied by the wood.

Given these diverse opinions, one has no alternative other than to be conservative when adjoining structures must be protected. Therefore, the viable options are to remove lagging that would be permanently above the ground water level or to treat with chemicals to prevent future deterioration.

When lagging is removed, the process should be in stages of a few feet at a time. Concurrently, backfill should be compacted. Soldier piles may be removed if it is practical to do so and provided that voids are not created below ground.

Treatment standards are shown in Table 3.

Table 3. AWWA minimum retention standards for sawn timber below ground.

	lbs/cu. ft. Retention
Creosote, creosote solutions, and oil-borne chemicals.	
Creosote	12
Creosote-coal tar solution	12
Pentachlorophenol	0.6
Water-borne inorganic salts (oxide basis).	
(1) Amoniacal copper arsenite (ACA)	0.6
(2) Chromated copper arsenate (CCA) type A	0.6
(3) Chromated copper arsenate (CCA) type B	0.6
Chromated copper arsenate (CCA) type C	0.6

Trade Names:

- (1) Chemonite
- (2) Erdalith, Green salt
- (3) Boliden K - 33
 Osrose K - 33

Note: This table presents minimum retention by assay in lbs. per cu. ft. for Southern Pine, Douglas Fir, or Western Hemlock.

* Data from AITC, "Timber Construction Manual", 2nd Edition, Wiley, 1974.

CHAPTER 3 - STEEL SHEET PILING

3.10 INTRODUCTION

This section concerns rolled Z-shaped or arch shaped interlocked steel sheet piling. Because of their greater resistance in bending, Z-shaped sections are more common in American practice than are the arch shaped sections.

3.20 DESIGN CONSIDERATIONS

3.21 General Applications

Typically, steel sheet piling is used in soils that are inherently difficult for placement of wood lagging such as soft clays, saturated silts, or loose silty or clayey sand. These soils are potentially unstable when they are exposed during excavation.

Interlocked steel sheet piling is highly effective in cutting off concentrated flow through pervious layers within or below the excavation and protecting against the possibility of a "blow" condition or other source of ground loss. On the other hand, the steel sheet pile wall does not necessarily prevent lowering of the piezometric level and accompanying consolidation when the excavation is made in relatively impervious soils. In these cases the steel sheet pile wall has approximately the same permeability as the soil in which it is driven, (clayey sands and clays would fall into this category of soil types).

In dense granular soils that can be relatively easily drained, soldier pile walls are normally selected over interlocked steel sheet piling. The selection of a soldier pile wall stems not only from cost considerations, but also from the fact that the soldier piles can be set in pre-excavated holes, thus minimizing the noise disturbance.

3.22 Available Sections

Figure 13 shows typical American steel sheet pile sections used for relatively deep excavations. Table 4 gives information concerning the properties of various steel sheet pile sections (see Figure 14 and Table 5 for foreign sections). Heavier sections are available in foreign steel sheet piling than in domestic piling.

The "Z" sections (PZ-27, PZ-32, and PZ-38) are most frequently used for deep cuts. These have a greater section modulus for corresponding weights than the medium arch and deep arch sections designated as PMA-22 and PDA-27.

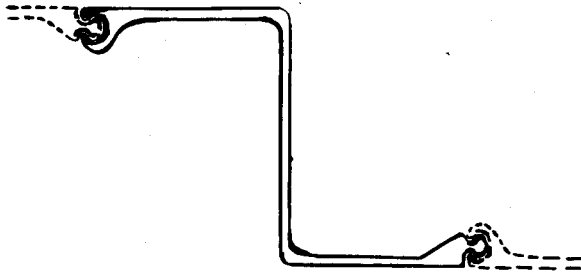
Note that the PDA section and PMA section interlock on the midline of the wall, whereas the "Z" sections interlock on the inside and the outside line of the wall. With regard to the deep arch and medium arch sections, it is conventionally assumed in American practice that shear cannot develop along the interlocks and therefore the two sheet piles which combine for the full wall depth cannot be considered effective in bending. European practice assumes interlock friction and therefore takes advantage of the full section modulus of both piles (Tschebotarioff, 1974).

3.23 Allowable Stresses

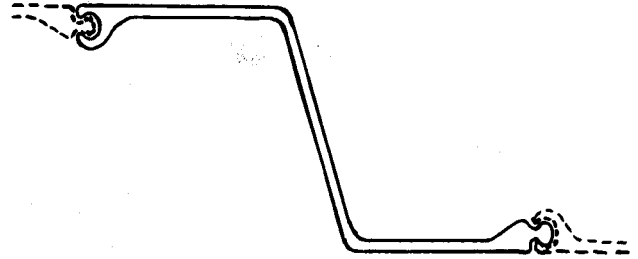
The conventional ASTM grade used for sheet piling is A 328, which has a minimum yield point of 38,500 psi. Some companies produce steel sheet piling in higher strength steel using ASTM grade A 572 in three types: 45,000; 50,000; and 55,000 psi yield point steel (see Table 6).

The AISC code allows an extreme fiber stress of 0.66 of the yield point, thus, the allowable stress in bending for A 328 steel is 25,400 psi or nominally about 25,000 psi. Proportionately higher values are used for A 572 steel.

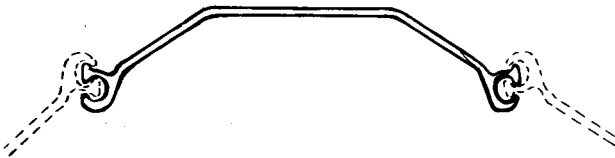
AISC allowable stresses may be used for the steel sheet pile wall at full depth. Temporary, intermediate conditions which exist during the course of excavation may be analyzed using a 20 percent overstress above the normal AISC allowable stress.



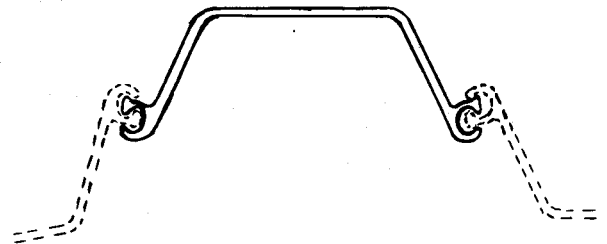
PZ 38



PZ 32 & PZ 27



PMA 22



PDA 27

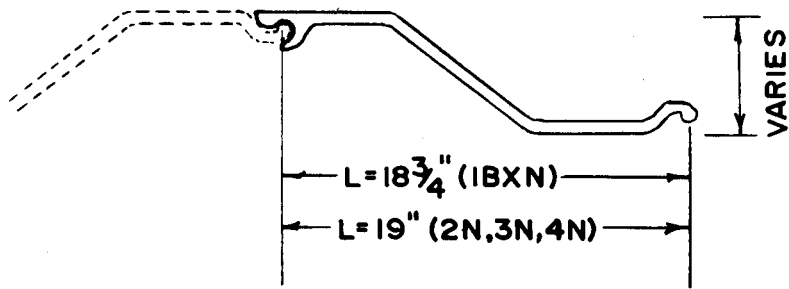
Figure 13. Domestic sheet pile sections.

Table 4. Domestic steel sheet pile sections.

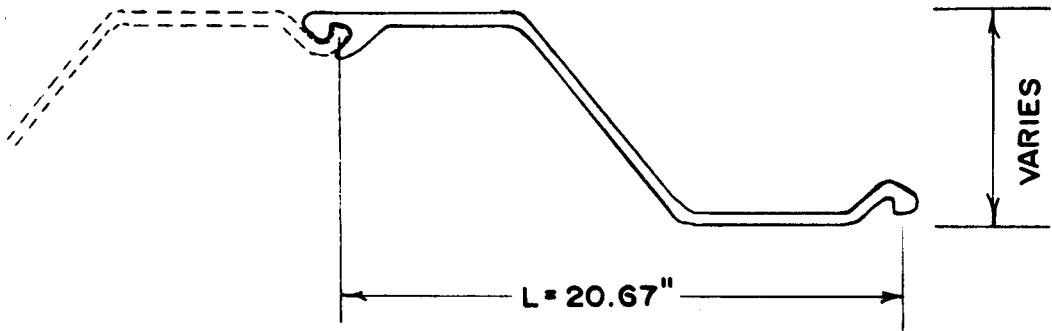
Section	Dimension (in)		Weight lb/sf	Moment of Inertia in ⁴ /ft	Section Modulus in ³ /ft
	D, depth	L, length			
PMA 22	3-1/2 x 2 = 7 ⁽¹⁾	19.6	22.0	16	5.4
PDA 27	5 x 2 = 10	16	27.0	40	10.7
PZ 27	12	18	27.0	183	30.2
PZ 32	11.5	21	32.0	220	38.3
PZ 38	12.0	18	38.0	281	46.8

(1) Single pile is 3-1/2" deep.
As driven, wall is 7" deep.

FRODINGHAM IBXN, 2N, 3N, 4N



HOESCH Nos. 95, 116, 134, 155, 175, 215



BELVAL BZ-250, BZ-350, BZ-450, BZ-550

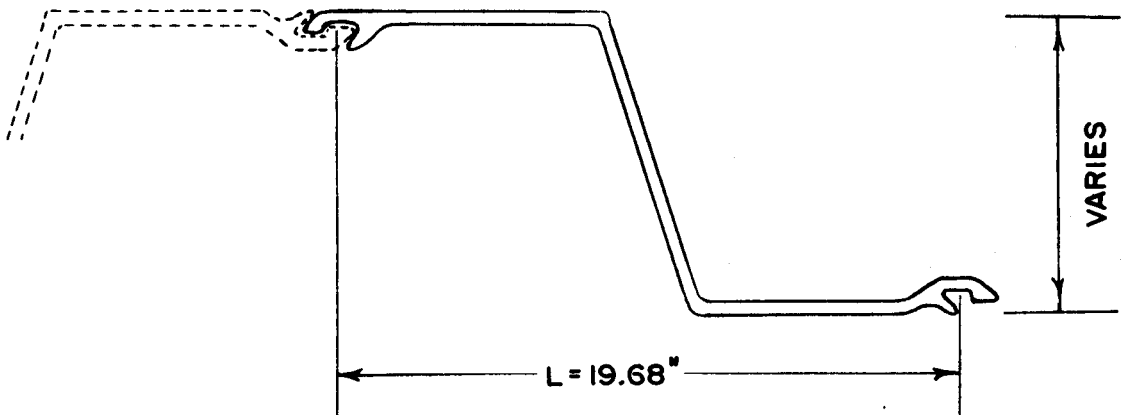


Figure 14. Foreign sheet pile sections.

Table 5. Foreign steel sheet pile sections.

Section	Dimension (in)		Weight lb/sf	Moment of Inertia in ⁴ /ft	Section Modulus in ³ /ft
	D, depth	L, length			
Frodingham ⁽¹⁾					
1B x N	5.63	18.75	27.00	36	12.9
2N	9.25	19.00	23.01	99	21.4
3N	11.13	19.00	28.08	175	31.4
4N	13.00	19.00	34.99	292	44.9
Hoesch ⁽¹⁾					
No. 95	7.48	20.67	19.46	52	13.95
No. 116	9.84	20.67	23.76	110	22.32
No. 134	11.80	20.67	27.45	187	31.62
No. 155	11.80	20.67	31.75	219	37.20
No. 175	13.38	20.67	35.84	324	48.36
No. 215	13.38	20.67	44.10	392	58.59
Belval ⁽²⁾					
No. 250	9.48	19.68	22.98	105	22.30
No. 350	11.40	19.68	26.75	180	31.10
No. 450	13.80	19.68	34.82	333	48.40
No. 550	13.80	19.68	55.71	547	78.50

(1) Data from L. B. Foster Company, Pittsburgh, Pa.

(2) Data from Skyline Industries, Port Kearny, N. J.

Table 6. Steel types used for sheet piles.

ASTM Grade	f_y , psi Yield Point	f_b , psi AISC* Design Flexural Stress
A 328	38,500	25,400
A 572		
Grade 45	45,000	29,700
Grade 50	50,000	33,000
Grade 55	55,000	36,000

$$* f_b = .66 f_y$$

3.30 CONSTRUCTION CONSIDERATIONS

3.31 Installation of Sheet Piling

Conventional pile driving hammers are used, and the hammer selection is usually a matter of cost and convenience to the contractor. The general technique is to drive the steel sheet piling in waves, always maintaining the tips of adjoining steel sheet piles no more than about 5 to 6 feet apart. The ball end (male end) should always lead to prevent plugging of the socket end (female end) with soil. This measure protects the interlocks from tearing.

Pile drivers may be impact type (single or double acting) or vibratory drivers. The vibratory drivers are run by hydraulic or electric motors which power eccentric shafts (Foster, 1971).

Concern over the noise factor in urban areas has led to the development of silent pile drivers. The one produced by the Taylor Woodrow Construction, Ltd., known as the Taywood Pile Master, operates on a hydraulic principle. Two hydraulic rams force the sheeting downward while the remaining six rams react against adjoining sheeting (Hunt, 1974). Stabilator AB, of Stockholm, Sweden has developed an Impulse Driver which operates by regulated pulses of compressed air, thus exerting a force on the piston. When reported (World Construction, 1974) the device was under development but was not available for general use.

If obstructions are encountered near the ground surface, they should be investigated and removed. If the obstruction cannot be removed easily, either because of its size or depth, then the procedure is to drive flanking sheets to their full depth. Later, lagging can be placed below the obstruction while the excavation is being made. Those sheets which cannot penetrate below the obstructions are cut off at the ground surface.

Under normal conditions, it is usually not practical to remove the obstruction during excavation and then drive the sheeting to its full depth. First, the pile driving rig is set within the interior of the excavation and there simply isn't sufficient room on the outside

to come back and redrive the sheet piling to its full depth. Second, the sheet pile line should be driven down continuously in waves as mentioned before. If this sequence is not followed, there might be a problem with piles ripping out of interlocks and with maintaining proper alignment.

Perhaps the single biggest potential for leakage of ground water and/or loss of ground is the sheeting ripping out of the interlocks as the result of poor alignment or hard driving conditions. Obviously, the potential for this rises with the density of the soil and with the frequency of boulders and obstructions below the surface.

3. 32 Removal of Sheet Piling

Conventional extractors can be used. Loose granular soils may, of course, consolidate as a result of vibrations during driving or extraction. It is believed, however, that the influence of such vibrations in loose granular soil will be confined to within about 10 to 15 feet of the sheet pile wall.

In cohesive soils the possibility exists that the clay may adhere to the sheeting, especially at the sharp angular bend in the corners of the PZ section. This would contribute somewhat to displacements in the adjoining ground.

Steps that can be taken to reduce the adhesion of clay include prior application of bituminous material to the steel and the application of direct electric current.

CHAPTER 4 - CONCRETE DIAPHRAGM WALLS

4.10 INTRODUCTION

The term concrete diaphragm wall as used herein applies to a continuous concrete wall built from the ground surface. One method of construction is by precast or cast-in-place concrete panels, both built within trenches stabilized by a slurry. Another method is to form the wall of continuous bored concrete piles. These piles, commonly referred to as secant piles or tangent piles, are not necessarily formed in slurry stabilized holes.

Diaphragm walls have been used more frequently in Europe than in the United States. The method was invented about 20 years ago and achieved early prominence when used for the Milan, Italy subway construction. Recent outstanding publications on diaphragm wall technology are Xanthakos (1974) and the Proceedings of the Diaphragm Walls and Anchorages Conference, 1974. * The London conference placed great emphasis upon the practical aspects of diaphragm wall construction.

By far the most common type of construction is the tremie concrete, diaphragm wall cast within a slurry stabilized trench. Reinforcement of such cast-in-place concrete walls is usually by a cage of reinforcing steel, either alone or in combination with vertical rolled steel sections or precast concrete sections. Closely spaced steel beams may eliminate the need for reinforcing steel.

The system using precast concrete panels lowered into a slurry stabilized trench has achieved a considerable degree of popularity in Europe. Bachy, Soletanche, and Franki have all installed precast panels in slurry stabilized trenches.

Typically, the excavation for cast-in-place diaphragm walls is in trenches about 10 to 20 feet long and about 24 to 36 inches wide. Panel lengths are excavated; end stops (usually pipe section) are placed; concrete is poured; and the end stops are removed. Once the end stop is removed, the neighboring panel can be excavated and concreted.

*Institution of Civil Engineers, London (September, 1974).

The diaphragm wall can be incorporated into the permanent wall of the substructure. Such applications, where feasible, are economical and fast. Also, a concrete diaphragm wall is much more rigid than either a soldier pile wall or an interlocked steel sheet pile wall, and therefore can be used for minimizing settlement and lateral movement of adjacent ground and structures during construction--especially in soft soils. This characteristic frequently provides an option to underpinning.

Figures 15, 16, 17, and 18 show various aspects of diaphragm wall construction.

4.20 PROPERTIES OF BENTONITE SLURRY

Bentonite slurries are normally in concentrations of 4 percent to 6 percent by weight (about 65 to 66 pcf). The primary functions of the bentonite slurry are as follows:

1. To maintain the excavated sand, silt, and clay particles in suspension so that these can be mechanically removed from the recirculated slurry.
2. To form an impermeable mudcake on the walls of the trench to prevent fluid loss and to transfer the hydrostatic fluid pressure in the trench to the soil.
3. To aid in stabilizing the walls of the excavated trench before concreting.

The bentonite contains the clay mineral sodium montmorillonite, which gives it high plasticity and swelling characteristics. When mixed with water, this forms a colloid suspension, or slurry.

Bentonite slurries, if allowed to set and remain undisturbed, will gel and develop shear resistance under static load. This is a characteristic of a Bingham body fluid as opposed to a Newtonian fluid such as water, which has no gel strength or shear resistance under static load. However, both Bingham fluids and Newtonian fluids display viscous shear resistance which is a function of the rate of shear application.

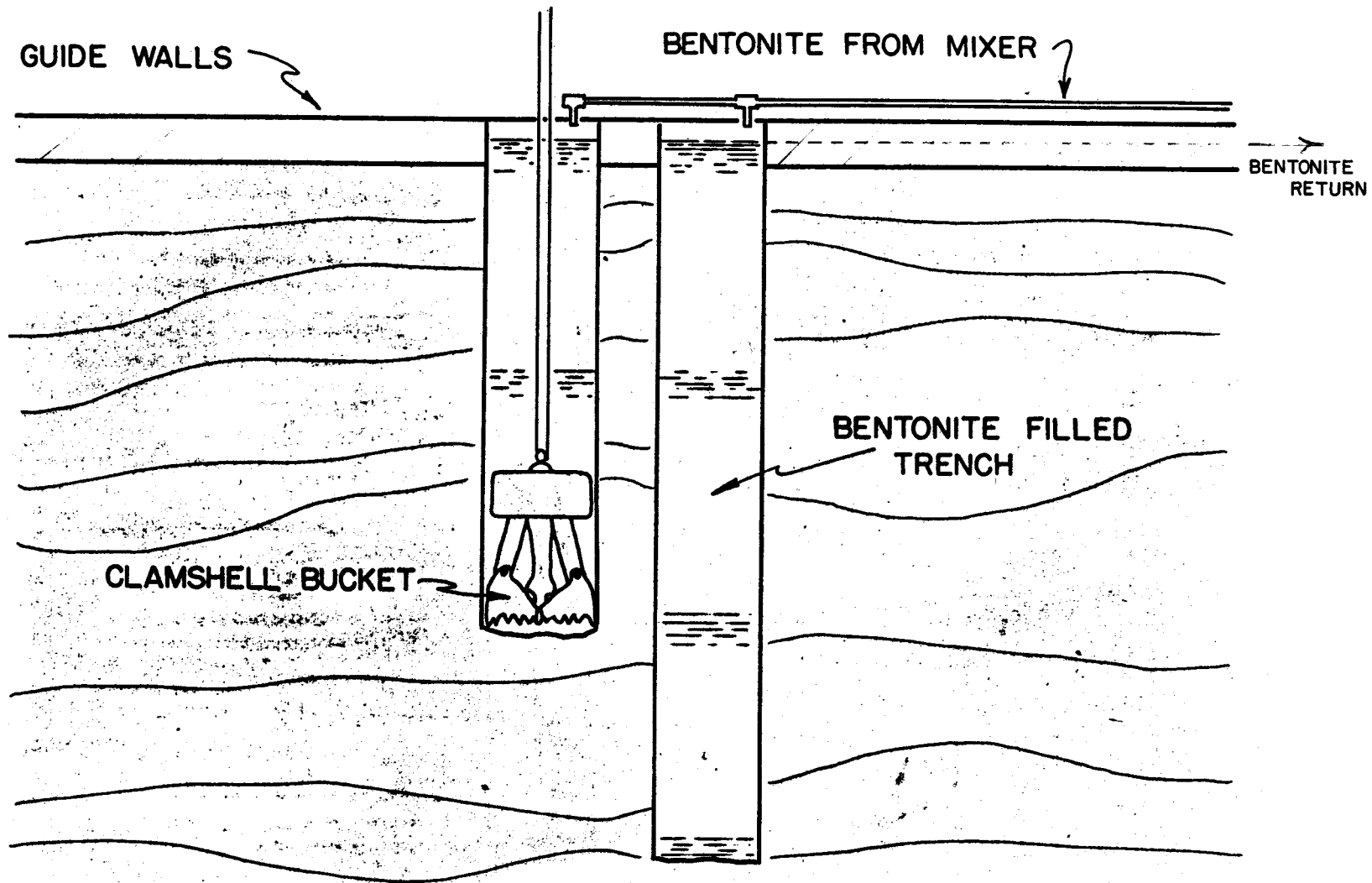


Figure 15. Excavation by clamshell bucket.
(Courtesy of ICOS Corporation).

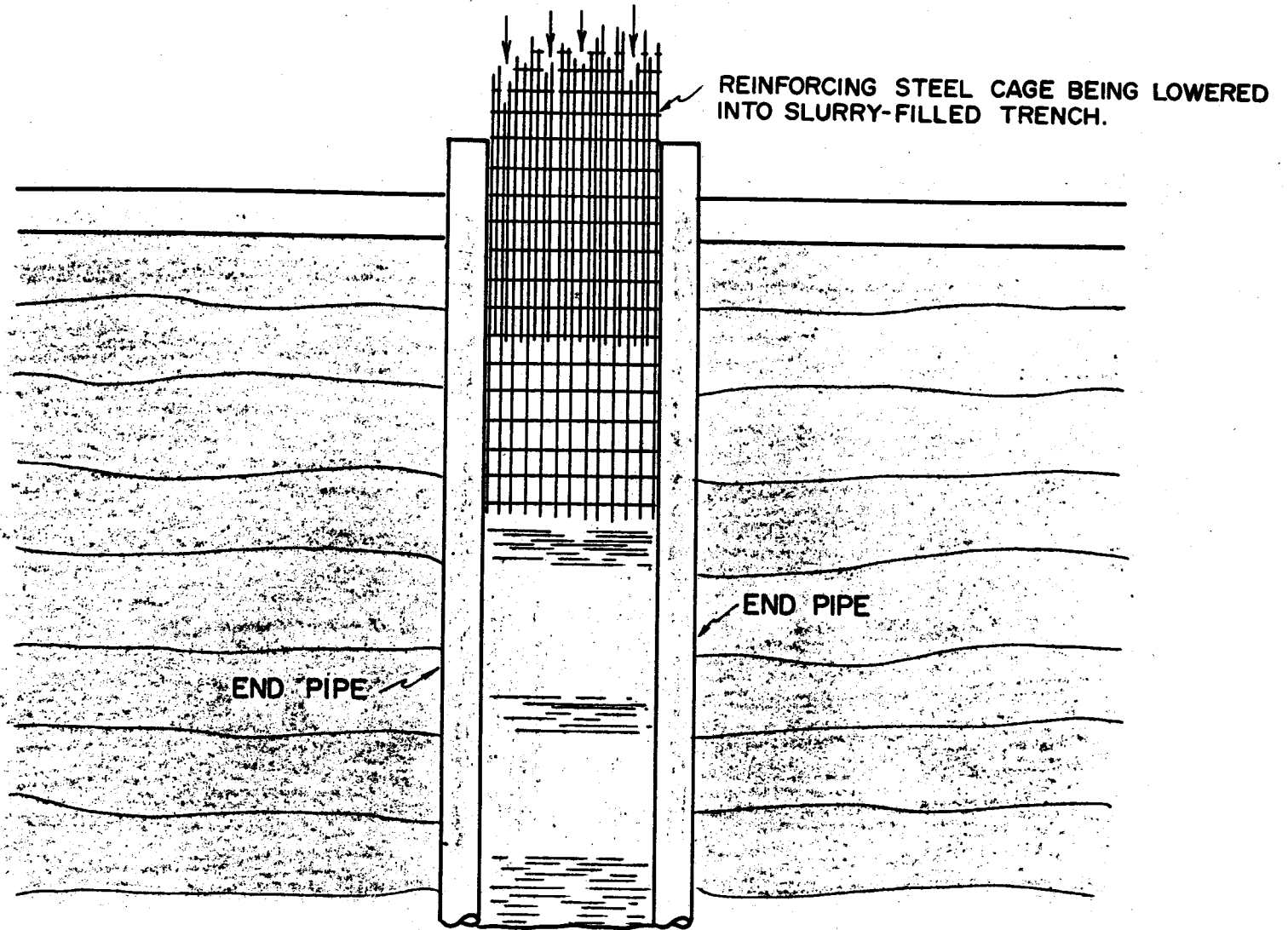


Figure 16. Preparations for concrete placement.
(Courtesy of ICOS Corporation).

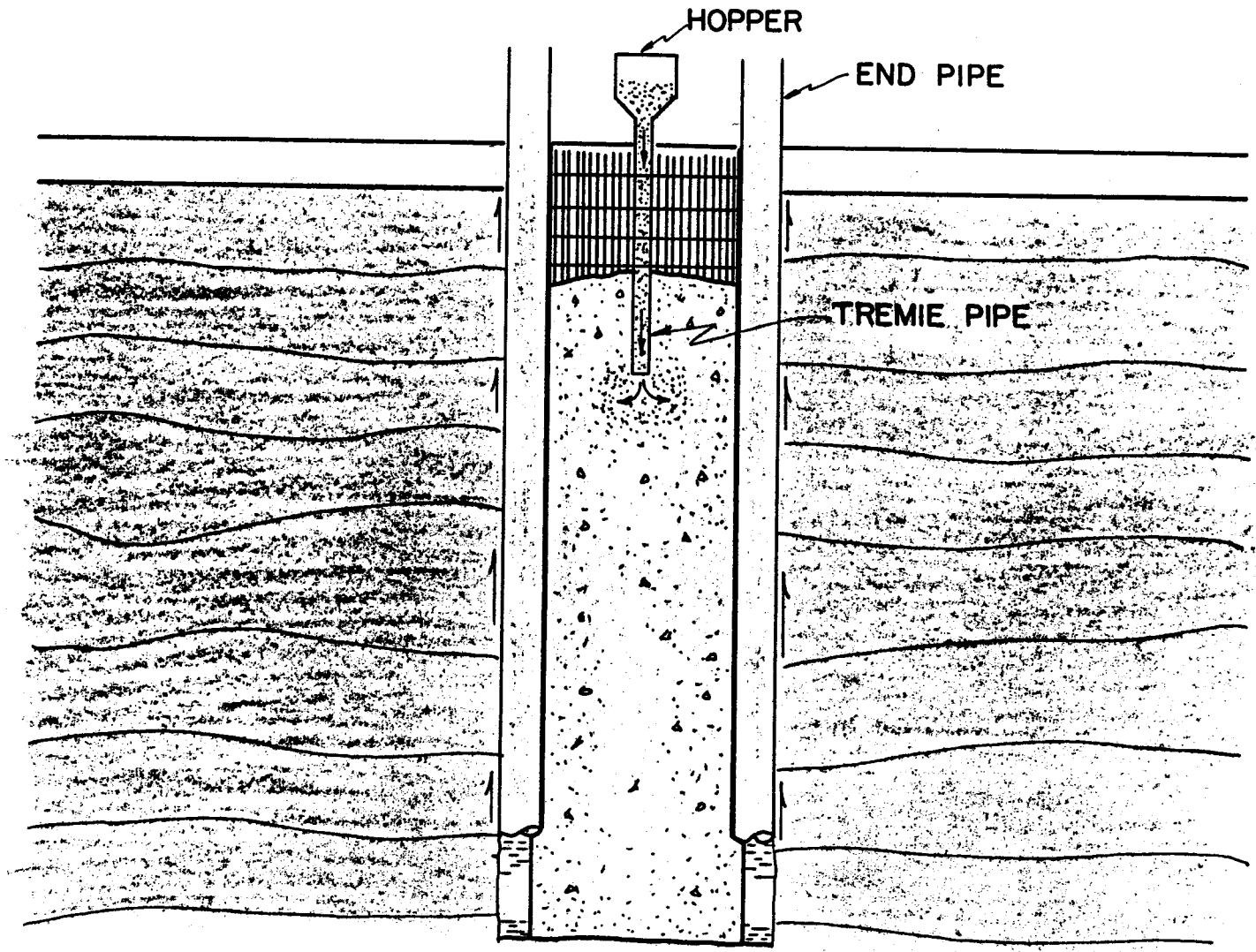


Figure 17. Placement of concrete.
(Courtesy of ICOS Corporation).

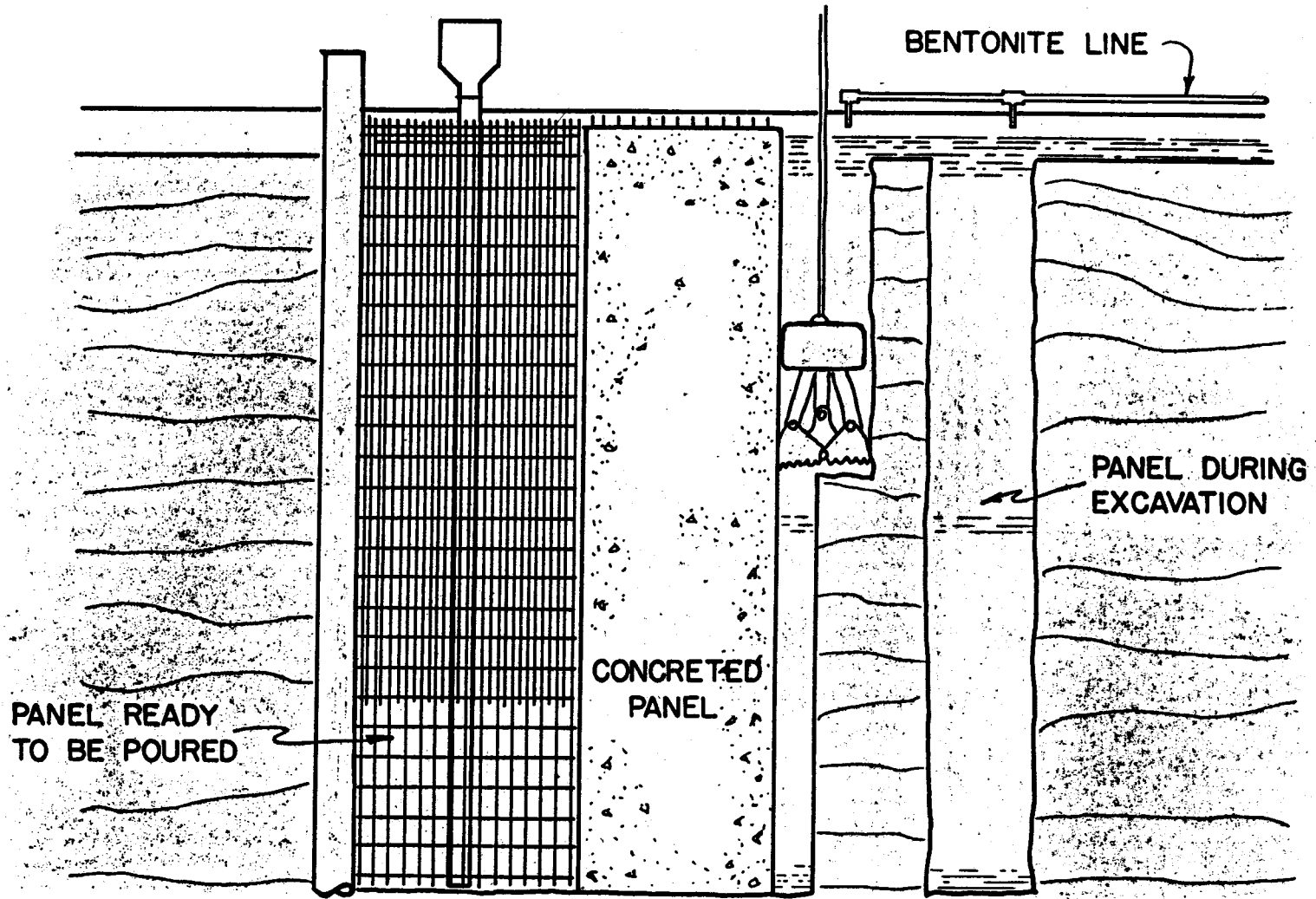


Figure 18. Different phases of construction.
(Courtesy of ICOS Corporation).

Figure 19 schematically shows the viscous character of Bingham and Newtonian fluids. This plot is shown for the sole purpose of advancing concepts, rather than for application.

For further discussion, see Rogers (1963).

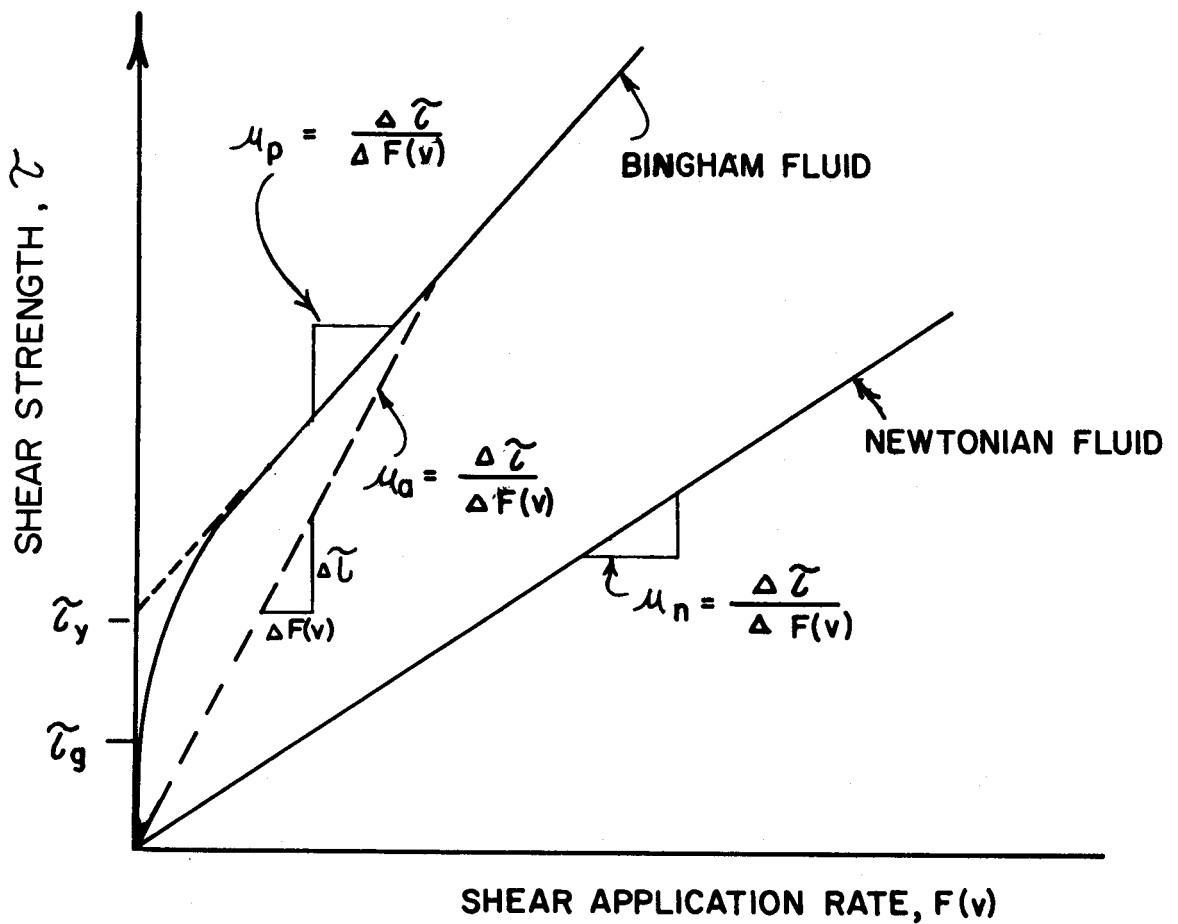
Viscosity and gel strength are used as indices for quality control testing of bentonite slurry. See Section 4.43.4 for further discussion.

Bentonite slurries are thixotropic--that is when left undisturbed, they gain strength with time. When disturbed or sheared again, they will lose strength. The process is reversible. A simple, practical application of this phenomenon is that slurry left in a trench will tend to stiffen up and will require agitation to become more fluid.

Xanthakos (1974) presents curves showing thixotropic strength gain with time of Fulbent 570 bentonite. These data show the following shear strength (g/cm^2) values.

Suspension by Weight	Setting Time (hours)		
	1	5	10
4%	0.12	0.20	0.22
6%	0.35	0.50	0.58
8%	0.80	>1.00	--

Comparable data are given in Weiss (1972) and Müller-Kirchenbauer (1972).



- $\tilde{\tau}_g$ ▪ Gel shear strength
- $\tilde{\tau}_y$ ▪ Yield shear strength
- μ_n ▪ Viscosity of Newtonian fluid
- μ_a ▪ Apparent viscosity of Bingham fluid
- μ_p ▪ Plastic viscosity of Bingham fluid
- $F(v)$ ▪ A parameter which is for particular geometry.
A function of shear application rate.

Figure 19. Viscous behavior of Newtonian and Bingham fluids.

4.30 SLURRY TRENCH STABILITY

4.31 General

4.31.1 Basic Considerations

The factors contributing to the stability of a slurry stabilized trench were discussed by Fernandez-Renau (1972) at the Madrid Conference* and were commented upon by Puller (1974). These are:

1. Penetration of slurry into voids of cohesionless soil.

Upon gelling, the slurry imparts cohesion to the soil and will prevent particles near the face from falling away. On the other hand, deep penetration, usually in coarse sands or gravels, will decrease effective stress and diminish wall stability.

2. Impermeable mudcake.

This membrane or "mudcake" prevents fluid loss and assures the maintenance of fluid pressure against the trench walls.

3. Pressure of slurry fluid.

The pressure of the fluid comes from two contributing factors. First, the density of the fluid itself is greater than that of water due to the bentonite concentration and suspended detritus. Second, and probably more important, the fluid level within the trench is maintained above that of the hydrostatic level within the ground water regime.

4. Arching.

The trenches are excavated in relatively narrow, short lengths which permits a redistribution of the earth thrust toward the ends of the panel and accompanying improvement of stability.

*5th European Conference on Soil Mechanics and Foundation Engineering

5. Slurry shear strength.

The slurry, being viscous and thixotropic, has inherent shear strength which theoretically serves to resist the lateral thrust of the earth.

6. Electro-osmotic pressure.

The particles within the bentonite colloid are attracted to the trench wall. A larger concentration of ions on the trench side of the wall creates electro-osmotic pressure.

Items 1, 2, and 6 (penetration of slurry, mudcake, and osmotic pressure) all relate to the mechanism occurring at the trench wall. This mechanism prevents fluid loss into the soil and prevents spalling of soil particles at the face. None of these items contribute to overall trench stability.

The two factors which are most important in controlling overall trench stability are fluid pressure (item 3) and arching (item 4). Finally, the effect of slurry shear strength (item 5) is believed to be small.

Much of the slurry trench work to date (1975) has been done successfully in situations where there are no theoretical analytical tools to explain why the method works. It is therefore a corollary that the theoretical criteria should not be applied strictly in the quantitative sense. Rather, their value lies in qualitatively understanding the factors contributing to slurry trench stability.

4.31.2 Field Experimentation

Deep Trench in Soft Clay (DiBiagio and Myrvoll, 1972)

A slurry stabilized trench 3.25 feet wide, 16 feet long, and 90 feet deep was made in soft clay, having an undrained strength of about 600 psf to 700 psf. Measurements included pore pressure, ground settlement, and lateral movement over a period of 31 days.

During this period, the specific gravity of the bentonite slurry was initially 1.24, then it was reduced in steps, to 1.10 and finally to 1.00 (water) prior to concreting. The average fluid level in the trench was maintained at about 3 feet below the top of the trench, which corresponds to 8 feet above the ground water level.

The authors concluded that settlement and lateral movement are small and that slurry trenches can be built successfully in soft clay. Specific conclusions were:

a. Settlement. Settlement was essentially negligible. Maximum occurred at the guide wall--0.2" following excavation and an increase of only 0.1" (total 0.3") 31 days later.

b. Lateral movement. Measurements were made for the full trench depth using an inclinometer. During excavation, maximum movement was about 1/4", which occurred at the bottom of the trench. Relative movement between the trench walls was monitored by sensors installed within the trench. In the zone of greatest movement near the bottom of the trench, the relative inward movement was about 1/4" after 3 days, about 1" after 15 days, and about 2" after 31 days. The horizontal sensors showed relatively greater deformation in the panel middle than near its ends, a clear indication of load redistribution by arching.

The data suggest that high bentonite concentrations are not essential to maintain the stability of trenches cut in clay -- even where the clay is soft.

Trench in Sand Next to Footing (ICOS Brochure, 1968)

ICOS reports a field test of a slurry trench with the edge of a loaded foundation, 16 feet long by 3 feet wide, 1.5 feet from the trench. The trench was excavated to a depth of 24 feet in preplaced washed sand and gravel that was carefully compacted to simulate the in situ density in the Milan area. The footing was 9 feet from the surface; therefore, the trench was 15 feet deep below the foundation.

The test procedure and the results are as follows:

1. 7.4 ksf applied on foundations with 1-1/4 inch settlement. The trench was unexcavated.

2. Excavate the central 6 foot long by 2 foot wide panel (opposite center of footing) while maintaining the 7.4 ksf load. Settlement during excavation was 1 mm.

3. Keep the central 6 foot long panel open, fill with bentonite, and increase the foundation load to 19.5 ksf. Settlement increased to 4-1/4 inches. There was no sign of collapse. The settlement curve was approximately linear above 4 ksf.

4. Decrease the load to 12 ksf and excavate the 6 foot long by 2 foot wide end panels. (Total length of excavated slurry filled trench is now 18 feet). Settlement increased to 7 inches. There was no sign of collapse.

Examination of the load settlement diagram shows little effect from excavation of the central 6 foot long panel. In other words, the slope of the settlement curve is about the same before and after excavation of this panel. On the other hand, the full 18 foot long open trench opposite the 15 foot long footing showed a dramatic settlement acceleration, albeit without collapse.

Apparently, arching was very effective in maintaining stability where the central panel alone was open. Then, excavation of end panels destroyed the arch and led to accelerated settlement.

4.32 Formation of Mudcake

With greater density and/or hydrostatic head as well as electro-osmotic pressure, the slurry is forced against the surrounding soil medium. As this occurs, the slurry may partially penetrate into the voids of the soil and build up an impervious layer or membrane on the face of the soil wall. In relatively pervious soil, it is fundamental that this impervious layer be formed in order to maintain the positive pressure against the soil and to prevent fluid loss.

Hutchinson (1974) reported that, in sand with permeability less than 10^{-2} cm/sec, the filter cake is about 15% by weight bentonite. For sand, with permeabilities between about 10^{-2} and 10^{-1} cm/sec. the bentonite slurry does effectively penetrate into voids of the soil to form a mudcake, but there may be some time lag associated with the development of a truly impervious mudcake. Finally, with very permeable soils such as coarse sands and gravels (10^{-1} cm/sec.), there could be free penetration of the slurry into the voids of the soil without the formation of a successful impermeable mudcake.

The distance of penetration is governed by the Bingham body characteristics of the bentonite suspension and the hydraulic driving head. Discussion of this issue appears in Section 4.37.

The chemical composition of ground water and soil, including such factors as pH, salinity, and calcium content, may also have an effect on the integrity of the bentonite slurry. Moreover, such conditions can adversely affect fluid specific gravity and viscosity. Therefore, chemical tests of both ground water and soil should be done as part of the soil investigation.

It is common practice to add various agents to plug voids of permeable soil so that an effective mudcake can develop. Further discussion concerning these additives will be made in Section 4.44.1.

4.33 Pressure of Slurry Fluid

4.33.1 General

Excess slurry fluid pressure is caused by the differential head of the fluid in the trench above that of the ground, water and the greater specific gravity of the slurry.

Typically, the bentonite concentration is about 4 - 6 percent by weight which corresponds respectively to the specific gravities of 1.023 to 1.034. As a practical matter, the slurry frequently contains suspended detritus (such as fine sand, silt, and clay particles) which increases the specific gravity above that of an idealized bentonite-water suspension.

With regard to excess head, it is common practice to maintain the water level in the trench at least 4 feet above the ground water level.

4.33.2 Stability Analysis

The following discussion presents several cases which examine trench stability on the basis of fluid pressure alone. None of these analyses consider arching, which as stated previously, is also of prime importance in maintaining trench stability.

For conventional panel lengths of 10 to 15 feet, these analyses are not a true representation of trench stability because arching is of relatively great importance. As panel lengths increase to 20 or 30 feet and more, arching is of little importance, and so the analyses becomes correspondingly more representative.

In summary then, the value of the analyses is as a means to temper judgement based upon experience. Since the state-of-the-art does not provide tools for evaluating arching, the analyses are not rigorous. However, the analyses provide a method of assessing the relative importance of fluid pressure, slurry height above water table, fluid density, and depth of trench on overall stability.

The following simple cases illustrate tools for analysis of trench stability. As stated above, all neglect arching, and therefore, are overly conservative for normal panel lengths.

Trench in Dry Cohesionless Soil (Xanthakos, 1974)

An idealized trench stability computation can be performed for an infinitely long slurry-filled trench in cohesionless soil. The ground water level is assumed to be below the base of the trench, and plane strain shear conditions are assumed. As shown in Figure 20, the most critical failure wedge rises at an angle of $45^\circ + \phi/2$ to the horizontal. The vector diagram shows boundary forces P_f and R_α in equilibrium with the wedge mass, W .

Forces are:

$$W = \text{Weight of wedge} = 1/2 \gamma_m H^2 (\tan 45 - \phi/2)$$

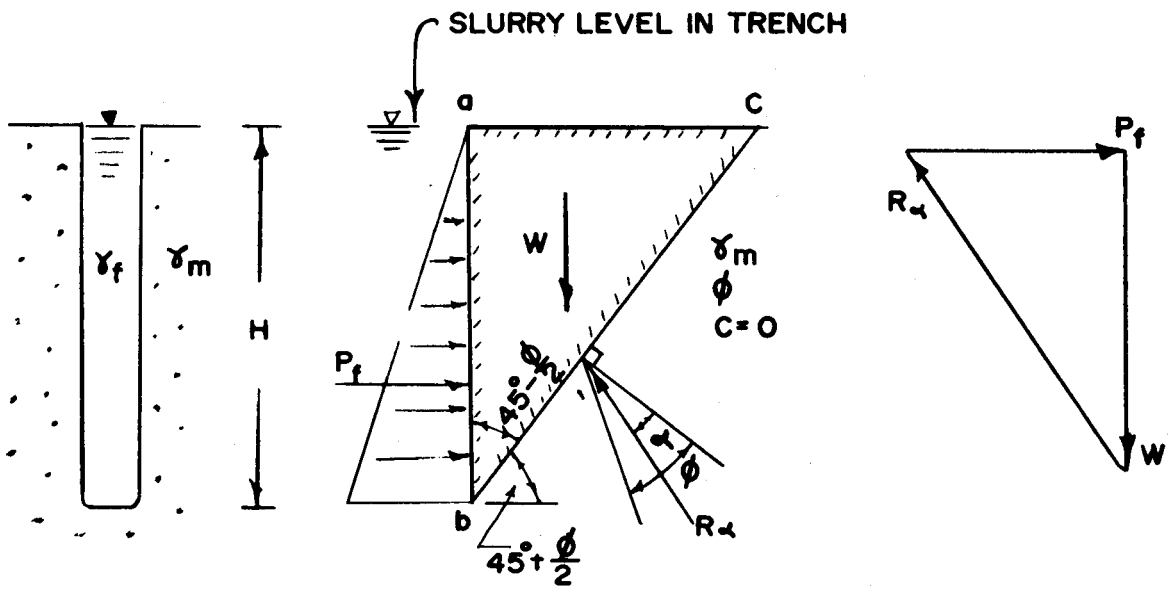


Figure 20. Force diagram for slurry trench in sand with ground water table below depth of trench.

$$P_f = \text{Lateral force from pressure of slurry fluid} = \frac{1}{2} \gamma_f H^2$$

R_α = Resultant force on boundary at obliquity α

γ_m and γ_f = Unit weight of soil and slurry, respectively

At failure, the obliquity at the boundary is ϕ , the angle of friction and therefore the maximum possible value. The safety factor is defined as $\frac{\tan \phi}{\tan \alpha}$, and it can be shown under this definition that:

$$F.S. = \frac{2\sqrt{\gamma_m \gamma_f} \tan \phi}{\gamma_m - \gamma_f}$$

Typically, γ_m for soil is about 125 to 135 pcf and that of slurry is about 65 pcf. By approximation $\gamma_m \approx 2 \gamma_f$. Substitution of this value in the above equation produces:

$$F.S. = 2\sqrt{2} \tan \phi = 2.8 \tan \phi$$

Cohesionless soils typically have friction angles of about 32° to 38° ; outer limits may vary from about 25° to 40° , corresponding to $\tan \phi$ of 0.46 and 0.84 respectively. The safety factor then, is always greater than unity, thus as a practical matter, stability will be assured in this special case of dry cohesionless soil.

An alternative way of expression the safety factor would be to assume full obliquity of the resultant, R_ϕ on the boundary, bc. Then the safety factor is expressed as the ratio of horizontal force required for equilibrium to the resisting horizontal force available from the pressure of the slurry. Under conditions of ϕ obliquity on the boundary, the earth mass would by definition be in the active state. A horizontal force to just balance active earth pressure would correspond to a safety factor of one. Safety factor is then:

$$F.S. = \frac{P_f}{P_a}$$

where:

P_f = force from slurry pressure

P_a = active force

$$F.S. = \frac{P_f}{P_a} = \frac{1/2 \gamma_f H^2}{1/2 \gamma_m H^2 \tan^2(45 - \phi/2)}$$

$$F.S. = \frac{\gamma_f}{\gamma_m} \times \frac{1}{\tan^2(45 - \phi/2)}$$

As before, $\frac{\gamma_f}{\gamma_m}$ is approximately equal to 1/2 thus:

$$F.S. = \frac{1}{2 \tan^2(45 - \phi/2)}$$

Comparing safety factor computation by the two methods shows little difference.

	$\phi = 30^\circ$	$\phi = 35^\circ$
F.S. = 2.8 tan ϕ	F.S. = 1.63	F.S. = 1.96
F.S. = $\frac{1}{2 \tan^2(45 - \phi/2)}$	F.S. = 1.50	F.S. = 1.85

The difference is of even less practical importance when one considers soil arching, and other factors that contribute to trench stability which cannot be analyzed rigorously.

Trench in Cohesive Soil, $\phi = 0$ case (Xanthakos, 1974)

A similar plane strain case for a slurry trench in cohesive soil is shown in Figure 21. Undrained strength conditions are assumed. Under these conditions, the failure wedge rises at a 45° angle to the horizontal. The vector diagram shows the boundary forces in equilibrium with the wedge weight, W.

Forces are:

$$W = \text{Weight of wedge} = 1/2 \gamma_m H^2$$

P_h = Horizontal force on boundary ab, required to maintain equilibrium. Note P_h is less than P_f .

$N = \bar{N} + U$, Resultant force acting normal to wedge boundary. \bar{N} and U are in terms of effective stress and water pressure.

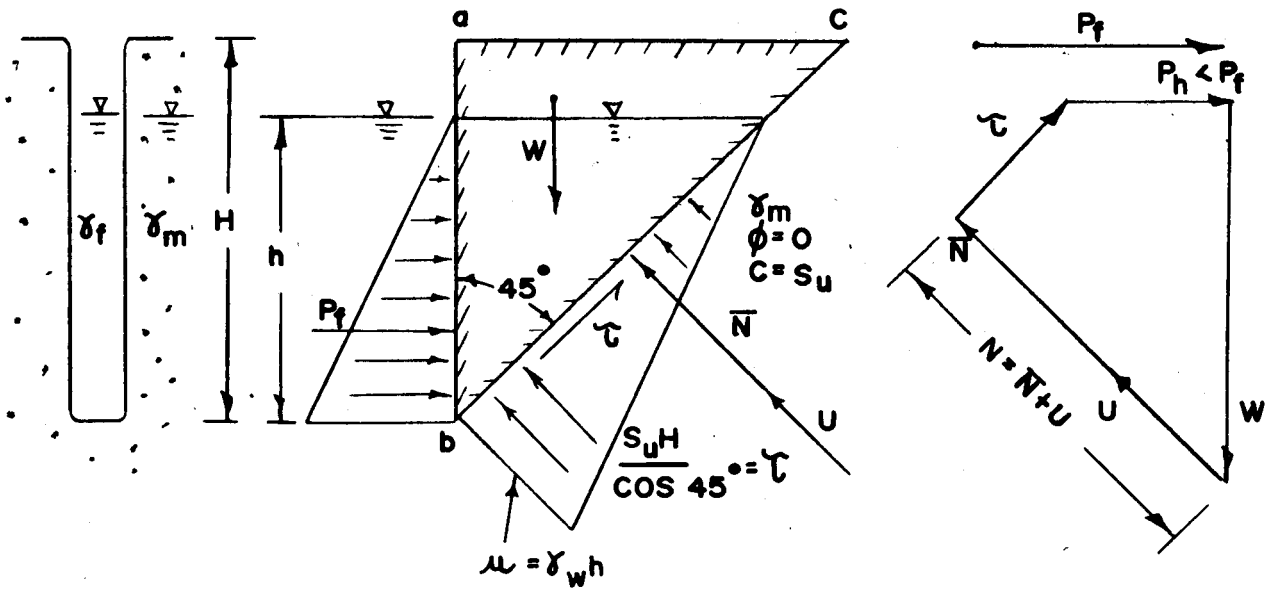


Figure 21. Force diagram for slurry trench in cohesive soil with water table and slurry at same level.

τ = Shear force from contribution of undrained shear strength, S_u , of soil.

γ_m and γ_f = Unit weight of soil and slurry respectively

In this case, the safety factor may be expressed as the ratio of P_f to P_h that is:

$$F.S. = \frac{P_f}{P_h}$$

where:

P_f = Force from slurry pressure

P_h = Horizontal force required for equilibrium

$$P_h = 1/2 \gamma_m (H^2 \tan 45^\circ) - S_u \frac{H}{\cos 45^\circ} \left(\frac{1}{\sin 45^\circ} \right)$$

$$= 1/2 \gamma_m H^2 - 2S_u H$$

$$F.S. = \frac{P_f}{P_h} = \frac{1/2 \gamma_f h^2}{1/2 \gamma_m H^2 - 2S_u H}$$

For the special case of fluid at the top of the trench, $h = H$, the expression reduces to:

$$F.S. = \frac{1}{\gamma_m / \gamma_f - 4S_u / \gamma_f H}$$

At failure, $F.S. = 1$; therefore, setting the equation equal to one yields:

$$H_{crt} = \frac{4S_u}{\gamma_m - \gamma_f}, \text{ where } H_{crt} = \text{critical depth} \\ \text{(see Nash \& Jones, 1963)}$$

Approximating, as before, that $\gamma_m = 2 \gamma_f$ and further approximating γ_f about equal to 64 pcf produces:

$$H_{crt} = \frac{4S_u}{64} = \frac{S_u}{16} \quad \text{for the special case of slurry and ground water at surface.}$$

Trench Below Water Level in Cohesionless Soil

Morgenstern and Amir-Tahmasseeb (1965), for the assumption of a Coulomb wedge, derived the following equation to analyze the stability of slurry trenches in cohesionless soils:

$$N^2 \frac{\gamma_f}{\gamma_w} = \frac{\frac{\gamma_m}{\gamma_w} \cot \alpha (\sin \alpha - \cos \alpha \tan \phi) + M^2 \operatorname{cosec} \alpha \tan \phi}{\cos \alpha + \sin \alpha \tan \phi}$$

where:

ϕ = the friction angle of the soil

γ_f = unit weight of the slurry

γ_m = total unit weight of soil

γ_w = unit weight of water

α = the angle between the horizontal plane and the failure plane.

N = (height of slurry in the trench)/(trench depth)

M = 1 - (depth to the ground water table)/(trench depth)

Figure 22 shows a plot of the factor of safety versus the ratio γ_m / γ_f derived from this equation.

Figure 23 shows the relative importance of slurry unit weight and slurry height above the water table for a 30 foot deep trench in cohesionless soil. While this case is for a particular set of boundary conditions, the case does reveal some practical considerations. In particular, it shows how important it is to maintain the slurry above static ground water level.

For example, a rise in slurry specific gravity from 1.0 to 1.10 increases the stabilizing force by about 3,000 lbs. per lineal foot. An equivalent increase in stabilizing force is achieved by a rise in fluid level of 2.8 feet (points a b c on the plot).

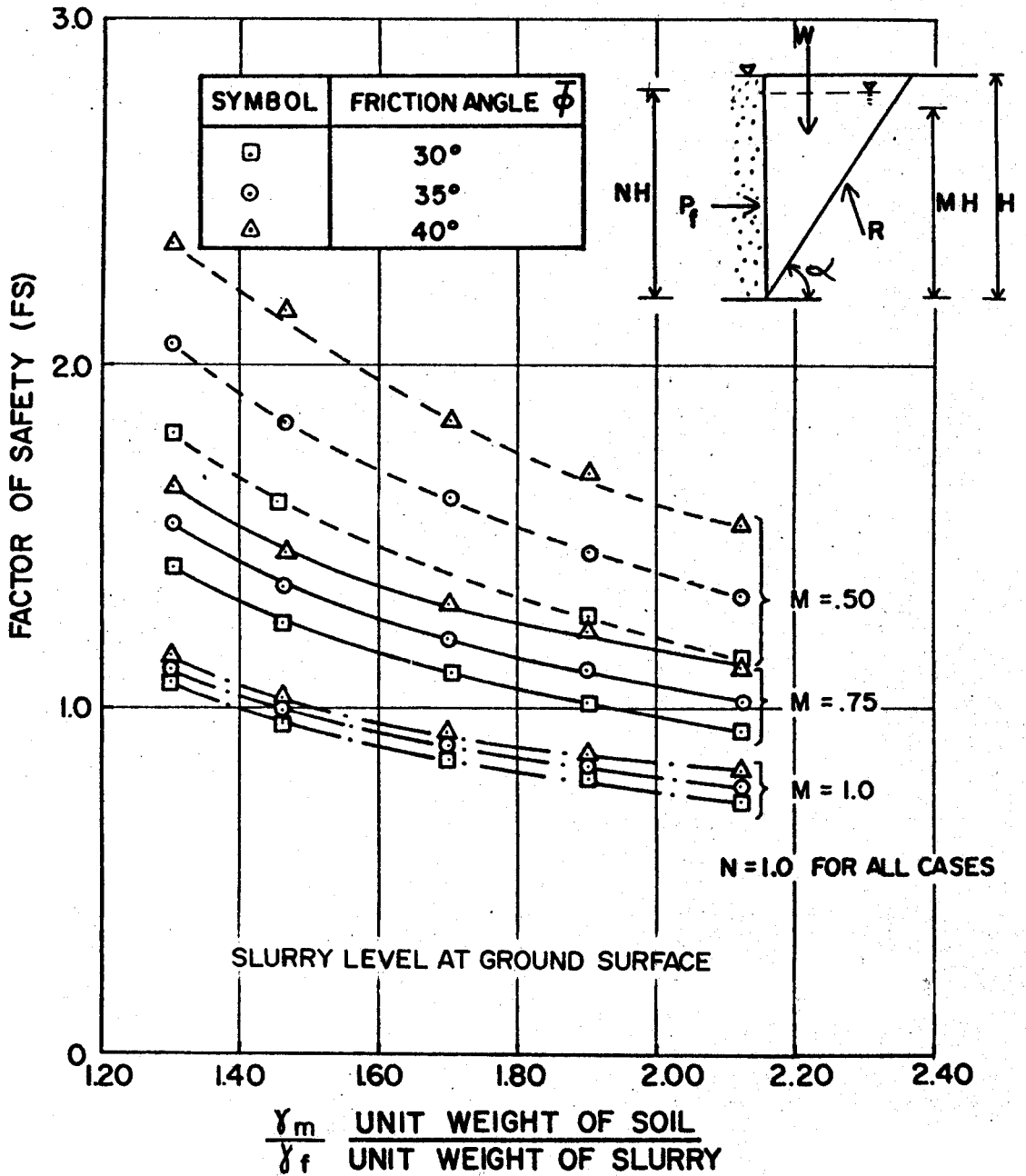


Figure 22. Stability of slurry trenches in cohesionless soils for plane strain conditions (from Morgenstern and Amir-Tahmassebi, 1965).

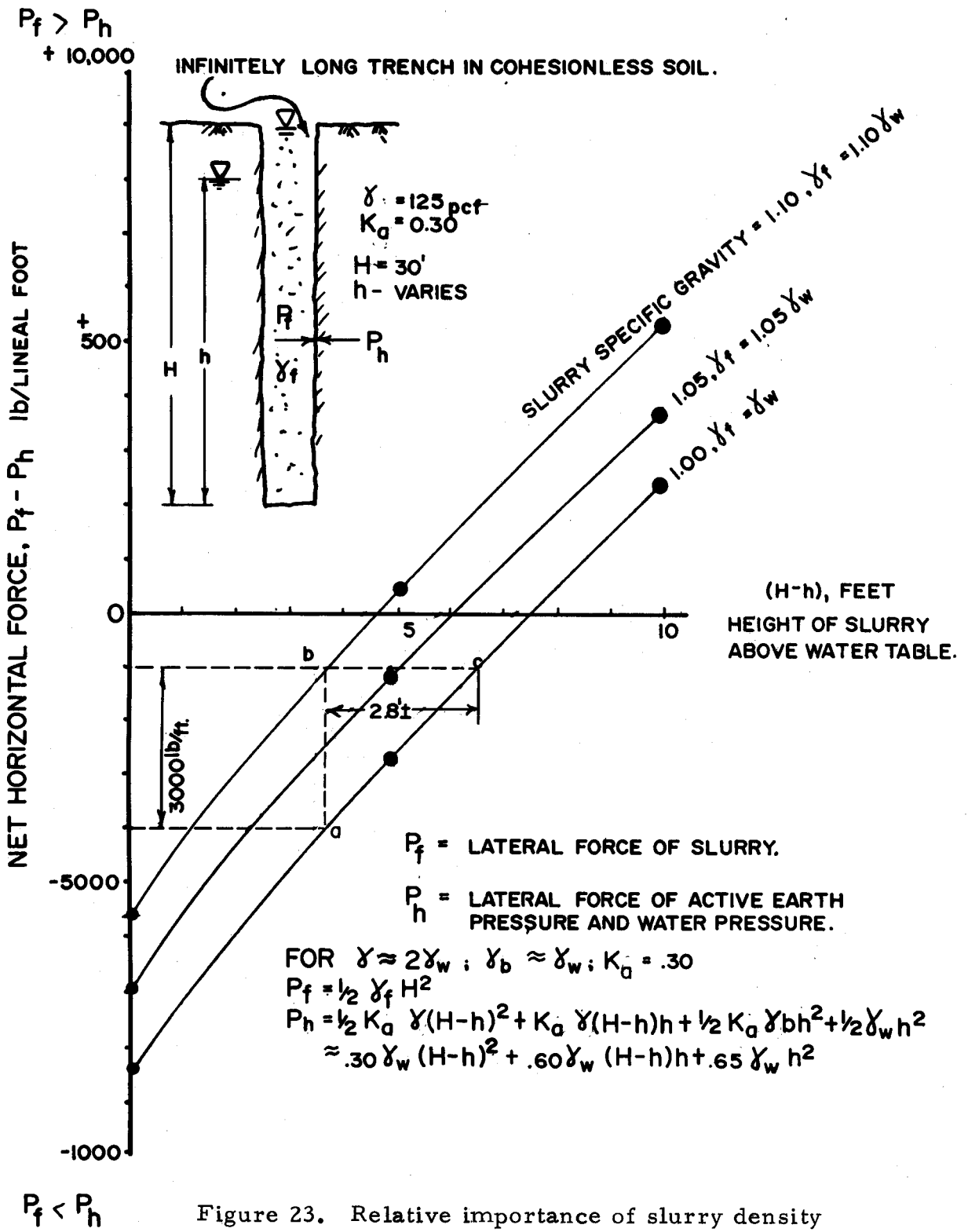


Figure 23. Relative importance of slurry density and slurry height for 30' deep trench.

However, there is little flexibility in varying slurry density in cohesionless soils. Normally the specific gravity is maintained around 1.05. To facilitate concreting the specific gravity should not exceed 1.10.

4.34 Arching

To understand the phenomenon of the redistribution of stress, referred to herein as arching, two conditions must be examined:

- a. The strain conditions at great depth below the surface.
- b. The strain conditions near the surface.

At great depth, strain is essentially a two dimensional condition acting in the horizontal plane outside the influence of local conditions at the top or at the bottom of the excavated panel. Horizontal strain is less near the ends of the panel than near the center of the panel. As a result, a redistribution of load takes place to the ends of the excavated panel, thus relieving the stress condition near the center and improving stability. This phenomenon, similar to that between soldier piles, is called arching.

The very top of the trench is restrained by a guidewall which is used to align the excavation process and to introduce recirculated slurry. The guidewall is essentially rigid and therefore restrains lateral movement so that arching action develops in the vertical plane. Similar arching occurs concurrently in the horizontal plane.

Experience has shown that a rigidly placed guidewall is an extremely important element in maintaining the stability of the top part of the trench. It acts as the top abutment of the arch with respect to strains taking place in a vertical plane. Inadequately constructed guidewalls frequently lead to a higher frequency of overbreaks immediately below the guidewall level, especially in cohesionless soils.

Without guidewalls, at trench depths equal to or less than panel lengths Schneebeli (1964) has shown that the condition is essentially one of plane strain (Rankine active). Other observations concerning the three dimensional behavior near the surface were made by Nash and Jones (1963). At depths greater than about the length of the panel, the arching action in the vertical plane rapidly reverts to a condition of arching in the horizontal plane.

The case reported by DiBiagio and Myrvoll (1972) discussed in Section 4.31.2 illustrates the influence of arching in soft clay. Measurements at a horizontal section within an excavated panel showed that horizontal movement increased gradually from the panel center and that the average movement was about 2/3 of the maximum movement at the center.

Piaskowski and Kowalewski (1965) and Meyerhof (1972) give theoretical treatment of arching; Piaskowski, for cohesionless material, Meyerhof for cohesive soils for side and bottom stability.

4.35 Slurry Shear Strength

As discussed earlier, the bentonite slurry has two components of shear strength. One is the shear strength due to gelation which is independent of the rate of shear application. The second is viscous shear strength which is dependent on the rate of shear stress application. Theoretically, if the slurry is allowed thixotropically to regain strength, it should offer resistance to movement of the soil mass into the trench.

An expression for the safety factor of an excavated trench in cohesive soil including the shear strength contribution of the bentonite slurry was presented by Xanthakos (1974).

While there is some theoretical basis for considering the shear strength contribution of the bentonite slurry, as a practical matter it is unlikely that this can be counted on for anything of significance. The slurry is in a continuous state of agitation and must remain sufficiently thin to allow placement of reinforcing steel and pouring of concrete. Under such conditions the slurry would have little time to gain significant strength by thixotropic action.

4.36 Electro-Osmotic Phenomenon

Xanthakos (1974) and Fernandez-Renau (1972) discuss the electrical phenomenon which occurs in a bentonite suspension and note that electro-chemical action is a contributing factor to the formation of the mudcake. Veder (1961) suggests that the mudcake formation is in part due to the electrical potential between the soil and the slurry. This creates a condition which attracts the electrically charged ions in suspension to the soil face thus forming a mudcake. This may take place in the absence of flow of fluid under hydraulic head. Such a cake has been observed in laboratory experiments by Veder (1963).

4.37 Penetration of Slurry into Cohesionless Soils

Two phenomena occur:

One is that the penetration of the slurry into the voids of the soil imparts cohesion in the penetrated zone by virtue of the yield shear strength of gelled fluid. This process prevents particles near the face from peeling away. Müller-Kirchenbauer (1972) relates the stability of these grains in terms of the yield shear stress of the slurry, the D_{20} size, and the bouyant unit weight of the soil.

The other phenomenon is a seepage gradient and resulting decrease in effective stress. This tends to lower the factor of safety within the zone of penetration.

The slurry will penetrate into the voids of the soil until the seepage force within the zone of penetration is in equilibrium with the shear resistance of the slurry. The shear resistance of the slurry acts in an opposite direction to the direction of seepage.

The gradient within the penetration zone is referred to as the stagnation gradient, defined as the hydraulic head loss within the zone of penetration divided by the thickness of the zone of penetration. The stagnation gradient can be computed theoretically:

$$i_o = C \left[\frac{2\tau_y}{r_e} \right] \frac{1}{\gamma_f}$$

where:

τ_y = Bingham yield shear stress

r_e = equivalent radius for the porous soil

γ_f = unit weight of slurry

C = a constant

Müller - Kirchenbauer (1972) describes an experimental technique used to determine the stagnation gradient, i_o :

$$i_o = \frac{h_f}{l}$$

where:

h_f = Hydraulic head

l = Distance of slurry penetration into the soil

With penetration of a few inches, an impermeable membrane effect is created; seepage pressures exist only in the membrane; and the soil within the membrane is easily held by the shear strength of the slurry in the soil voids. The weight of particles tending to fall away is small compared to the shear resistance of the soil. On the other hand, as the zone of penetration increases, a larger volume of soil is under the influence of the stagnation gradient. In this latter case, the weight of the soil mass within the zone of penetration is large compared to the shear resistance of the soil, and the condition becomes less stable. Müller - Kirchenbauer (1972) demonstrates analytically the decrease in safety factor due to slurry penetration.

The total shear resistance of the soil stems from yield shear of the slurry (analogous to cohesion) and from effective stress (ϕ obliquity). Elson (1968) suggests that negative pore pressure and dilatancy increase this latter component of shear resistance by about an additional 10 percent.

In terms of soil mechanics fundamentals, the seepage force (Taylor, 1948) per unit volume within the zone of penetration is:

$$\vec{j} = i_o \gamma_f$$

where:

\vec{j} = Seepage force per unit volume

i_o = Stagnation gradient

γ_f = Unit weight of slurry

A deeper penetration lowers the gradient, lowers the seepage force per unit volume, and diminishes the effective stress per unit volume of soil within the zone of penetration. A limiting case would be free penetration in open gravel, which would have a flat gradient approaching

zero. Such a condition would lead to collapse of the trench wall.

As a practical matter the preceding discussion is somewhat academic in nature, considering the present state-of-the-art. Müller-Kirchenbauer (1972) does draw some very significant qualitative conclusions relative to the fact that trench wall collapse (spalling) is far more common near the top of the trench in cohesionless soil than near the bottom. He points out that the slurry is relatively free of suspended soil particles when digging first commences; and so slurry penetration is primarily prevented by bentonite concentration. As the depth increases, the slurry gains in suspended soil and so is less likely to penetrate soil voids. For this reason, in pervious soil it is advisable to maintain a specified percentage of fine sand in the slurry as it is introduced into the trench (Hutchinson, et al, 1974). With depth, the slurry naturally gains in suspended soil particles which aid in forming a more effective mudcake by plugging soil pores.

Another reason for a higher incidence of instability near the top of the trench is that soil arching is frequently less effective. The zone just below the guidewall is most critical.

4.40 PRACTICAL ASPECTS OF SLURRY STABILIZED EXCAVATIONS

4.41 Scope

This section applies primarily to slurry stabilized excavation filled with tremie concrete. However, some of the considerations are also applicable to walls constructed of precast concrete panels.

In either case, there is always the common consideration of maintaining trench wall stability. In the case of tremie concrete, the requirements for proper concreting impose rather strict limitations on the characteristics of the bentonite slurry. For example, a highly viscous dense slurry is desirable for maintaining stability but may interfere with free flow of tremie concrete and adversely affect quality. This is especially true when the wall is heavily reinforced.

4.42 Water Level

It is common practice to maintain the trench fluid at least 4 to 5 feet above the ground water level. In soft clays, loose silts, and sands, cases have been reported where the level was maintained 8 feet or more above the ground water in order to assure stability. Under certain circumstances, this may necessitate the construction of dikes paralleling

the trench to maintain the slurry level at the desired elevation or alternatively, pumping to draw down ground water.

The site investigation must carefully identify highly pervious strata, through which slurry loss may occur, and also identify the potential for artesian conditions in confined layers.

4.43 Control of Bentonite Slurry

4.43.1 General

The bentonite powder is mixed with fresh water. Paddle mixers or high speed mixers are used to insure thorough wetting of the bentonite powder. After hydration the mix is checked for quality (e.g. viscosity, density, and pH) and then introduced into the trench slurry.

Normally, bentonite concentration is between 4 percent and 6 percent, corresponding respectively to densities of 1.023 and 1.034 g/ml where no constituents are present other than the bentonite.

It is essentially impossible to maintain uniform slurry density with depth within the trench. For example, contamination with concrete and detritus will lead to an increase in density near the lower portion of the trench excavation. Increases in density makes it difficult for the tremie to properly displace the slurry. This may lead to inclusions of bentonite within the concrete, poor bonding to steel, and associated loss of concrete quality. The FPS* specifications require that the density of the slurry should not be greater than 1.3 g/ml prior to placement of concrete. It is important that the sampling be taken near the lower portion of the trench (lower foot).

As discussed in Section 4.43.3, agents are added to the slurry in order to deal with specific field problems. The main problems arising from contamination of the slurry are an adverse affect upon tremie concrete placement (from high specific gravity and viscosity), fluid loss through ineffective mudcake development, or flocculation leading to spalling of the trench wall.

4.43.2 Source of Contamination

Contamination may be from detritus (the build-up of clay, silt, and sand particles within the suspension) or from chemical changes in the slurry. Chemical contamination may adversely alter pH,

* Federation of Piling Specialists, Great Britain, See Appendix B to this Chapter.

may alter electrolytic properties of the fluid, or may lead to ion exchange, usually replacement of sodium ions with calcium ions in the montmorillonite lattice (Sliwinski and Fleming, 1974; Xanthakos, 1974).

To summarize some of the effects arising from the contamination:

a. Detritus Contamination. This leads to an increase in slurry density. As a result of downward migration of particles, the density tends to increase with depth. The effect is to impair circulation of slurry and to adversely affect concrete placement.

b. Calcium Contamination. This causes flocculation of bentonite particles, rendering the slurry more viscous and more difficult to circulate. It causes an excessively thick mudcake which is relatively more difficult to displace by the rising tremie. Also, the cake is more permeable, thus creating the potential for fluid loss in permeable soils.

Calcium contamination comes from replacement of sodium ions with calcium ions, with associated increase of the latter in the montmorillonite lattice structure. It is commonly known that cement, in contact with slurry is the major source of calcium contamination. Fine soils or artificial fill containing concrete demolition debris may also be a source of calcium contamination.

c. Salt Contamination. Excessive salinity changes the electrolytic properties and may lead to flocculation of the bentonite particles. This makes it more difficult for the slurry to form an effective cake and may lead to fluid loss. Accordingly, the problem will be especially acute in relatively pervious granular soils.

4.43.3 Slurry Mix

As discussed above, the typical mix is about 4 to 6 percent by weight of bentonite. This will, of course, vary depending upon field conditions. For example, in highly pervious soils, the concentration may be increased to perhaps 8 percent. On the other hand, in competent stiff clays, where potential fluid loss is not a factor, the concentration may be decreased to 2 percent or less provided the soil is not stratified with sand.

Agents are added to the slurry to counteract chemical contamination, to decrease the viscosity of the slurry, or to aid in the development of an impermeable mudcake. These agents are discussed by Rogers (1963), Xanthakos (1974), Puller (1974), Sliwinski and Fleming (1974) and Hutchinson, et al (1974).

a. Viscosity. As discussed in Section 4.20, viscosity has two components -- the yield shear strength (essentially a static condition) and additional plastic viscosity dependent upon the rate of shear. High yield shear values are associated with an "edge-to-face" or "brush-heap" structure of the bentonite colloidal particles. This is more permeable than a dispersed structure which has particles aligned more parallel to one another.

In general, a "brush-heap" structure has higher yield shear strength, is more viscous, and is more permeable than a dispersed structure. Mud thinners, also called dispersants, change the colloidal structure from "brush-heap" to dispersed and aid in controlling fluid loss.

Rogers (1963) in discussion of chemical mud thinners classifies them in the following groups: molecularly dehydrated phosphates and polyphosphates, plant tannins, lignosulfonate wood by-products, and mineral lignins. He lists over 60 chemical mud thinners under these classifications.

Chemical mud thinners mentioned by Puller (1974), included "Dextrid", a trade name polysaccharide made by Baroid, and ferro chrome lignosulfonate (FCL). Puller (1974) reports on the results of fluid loss in standard API filter test in which Dextrid and ferro chrome lignosulfonate were used singly or in combination in concentration of 0.3 to 0.4 percent by weight in a 3 percent bentonite slurry.

Xanthakos (1974) discusses sodium ferro chrome lignosulfonate (FCL) usually in proportions of 0.1 to 0.3 percent, as a desirable mud thinner. FCL also has the additional feature of resisting cement contamination and being highly effective in resisting salt contamination.

Use of mud thinners requires experience, as well as laboratory test verification of their effect. As a minimum, such would include pH, viscosity, and standard API fluid loss tests in order to diagnose the problem and to determine appropriate treatment.

b. Cement Contamination. A common approach is to introduce sodium ions to retard ion exchange with calcium. Agents are: sodium ferro chrome lignosulfonate (FCL) (0.1 to 0.3 percent by weight), sodium bicarbonate, and other thinners.

c. Salt Contamination. A simple precaution to counteract salt contamination is to mix the slurry with fresh water and be sure that it is fully hydrated before introduction into the trench. Sodium

ferro chrome lignosulfonate (FCL) is remarkably effective in resisting excessive salinity (Xanthakos, 1974).

d. Fluid Loss in Highly Pervious Soils. Merely increasing bentonite concentration in soils having permeabilities greater than about 10^{-1} to 10^{-2} cm/sec. will not be effective (Sliwinski and Fleming, 1974). Hutchinson, et al (1974) proposes the addition of about 1 percent fine sand as a means to penetrate and block the pores of pervious soils having permeability greater than 10^{-1} cm/sec. Other additives include a whole range of inert plugging substances such as: nut shells, plant fibres, rayon, cellophane flakes, mica, ground rubber tires, etc. (See Section 4.44 for further discussion).

In extreme cases cement may have to be added to penetrate, plug, and set in the pores. Another approach is to excavate and backfill the trench with lean concrete. Both will require re-excavation in a normal slurry mix.

4.43.4 Control Testing

General

Testing to control the slurry is essential because first, the recirculated slurry may become contaminated and second, bentonite itself is of variable quality and character. Hutchinson, et al (1974) present a comprehensive overview of criteria for bentonite slurry quality and methods of testing. Those properties are obtained at 20°C and apply to slurry supplied to the trench unless stated otherwise.

Appendix A contains standard API procedures, equipment, and specifications.

Viscosity and Shear Strength

In fundamental terms the shear resistance of bentonite slurry is:

$$\tau = \tau_y + \mu_p F(\dot{\gamma})$$

where:

τ = Total shear stress

τ_y = Yield shear stress (static intercept)

$\mu_p F(\dot{\gamma})$ = Viscous shear stress (dynamic condition)

μ_p = Plastic viscosity

$F(\dot{\gamma})$ = Function of shear application rate

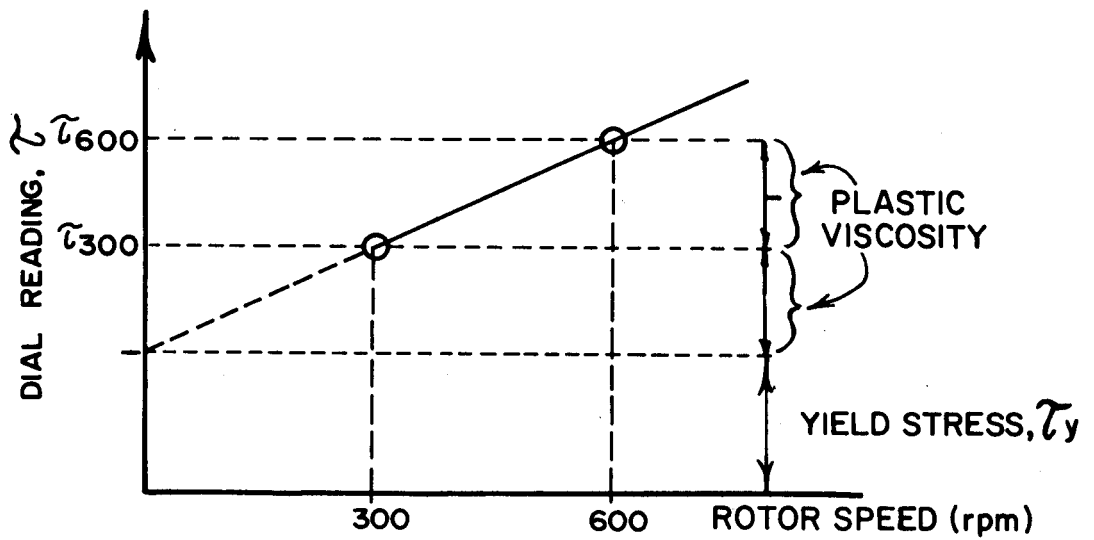
Fann Viscometer. The Fann viscometer, described in Appendix A, is used for measuring viscosity, yield shear strength, and gel strength. In this method, the slurry fills the annular space between a central circular core and an outer sleeve, and the device has a dial which enables one to measure the resistance while rotating the outer sleeve at a constant rate. The apparatus may be adjusted and calibrated in such a way that the viscosity in centipoises, and the yield shear stress, in lbs/100 ft², may be determined directly from the readings.

For determination of plastic viscosity and yield shear stress, the rotor is turned at 600 rpm and then at 300 rpm. The plastic viscosity in centipoises is the difference between the 600 rpm reading and the 300 rpm reading. The yield shear stress is the 300 rpm reading minus the plastic viscosity. (See Figure 24).

With the viscometer, the gel strength is defined by API as the maximum reading obtained at 3 rpm. Alternatively, the rotor may be turned very slowly manually; both are essentially a static condition which conventionally is obtained after 10 minute gel time.

Shearometer. The shearometer (Appendix A) is also used to obtain gel strength. This is a special cup, tube, and graduated scale. The scale is set in the cup along with the slurry. The cylindrical tube is slipped over the scale and allowed to sink into the slurry. After one minute, a reading is taken directly opposite the top of the tube on a scale graduated to read the shear strength value.

Because of differences in the equipment and procedures, the gel strength values from the shearometer are not the same as those from the Fann viscometer. Specifications must therefore identify procedure as well as control values.



$$\text{PLASTIC VISCOSITY} = \tau_{600} - \tau_{300}$$

$$\text{YIELD STRESS} = \tau_{300} - (\tau_{600} - \tau_{300})$$

Figure 24. Data from Fann viscometer.

The ten minute gel strength, following violent shearing of the slurry, can be used as an index of bentonite concentration and the degree of hydration. Hutchinson, et al (1974) recommend 0.05 g/cm² to 0.20 g/cm² using the viscometer. The FPS specification (1973) requires 0.014 to 0.10 g/cm² using the shearometer. Note that the results from the viscometer and shearometer are not the same.

Marsh Cone. A simple method for obtaining an index of viscosity, especially useful as a quick field method, is with the Marsh cone. The standard size cone is filled with slurry and the time for the funnel to drain is reported as Marsh funnel viscosity. Obviously, the more viscous the fluid, the longer the drain time.

The FPS specification (1973) requires that the Marsh cone drain time be between 30 and 60 seconds.

Density. Density (see Appendix A) is a simple measurement of a known volume of slurry using a Mud Density Balance. The FPS specification requires that the density of the mud supplied to the trench be less than 1.10 g/ml. Note that the slurry after re-circulation from the trench is not composed of pure bentonite but will still contain some suspended soil particles not removed by the cyclone.

Additionally, checks should be made of the slurry density within about 1 foot of the bottom of the trench. This should be less than 1.3 g/ml so as not to interfere with tremie placement (FPS specification, 1973). The density of freshly mixed bentonite slurry also can be used to check on the desired concentration. For example a 4 percent concentration of pure bentonite has a density of 1.023 g/ml, 5 percent has a density of 1.028 g/ml, and 6 percent has a density of 1.034 g/ml.

Cement contamination, which adversely affects the slurry, causing flocculation, increased viscosity, and more permeable mudcake, also raises the pH.

The FPS specification requires that the pH lie between 9.5 and 12. The pH can be determined with litmus paper strips or with a pH meter. The latter is preferred.

Filtering Performance

The device described in Appendix A is the API standard. Slurry (600cc) is placed over filter paper, 100 psi pressure is applied, and fluid loss is measured in a 30 minute time period.

Hutchinson, et al (1974) state that the fluid loss test is not strictly applicable to slurry trench work because the filter paper differs so radically from the soil. Veder (1974) suggested that the test be performed on samples of the actual soil to be encountered in the excavation. By this process, the effect of additives (such as fine sand) in reducing fluid loss can be assessed.

Excessive sand content may unfavorably raise the density of the slurry. On the other hand, fine sand may be added to the slurry being circulated into the trench to control fluid loss in permeable soils. (See Section 4.44.1).

In summary, the key tests are viscosity, density, pH, and 10 minute gel strength. Optional tests include filtering performance and sand content.

4.43.5 Cleaning the Slurry

Depending on the soil conditions and the method of excavation used, the procedure for cleaning the slurry of suspended detritus (gravel, sand, silt, etc.) may include sedimentation tanks, mechanical screening, and centrifugal separation using hydrocyclones.

A sedimentation tank is not common. Most generally it may be used in cases when the material is being removed by reverse circulation to allow the gravel and stone sizes to settle out as a first step in the process. However, this requires frequent unloading.

The more common method is first, to circulate the slurry over stationary or vibrating screens which remove the relatively coarse particles by mechanical process. Next, the slurry is circulated through centrifugal separators (hydrocyclones) which remove the sand. Finally, the slurry is discharged into a holding tank, tested for quality, treated with chemicals or additives if necessary, and recirculated back into its trench. See Figures 25, 26, and 27.

4.44 Some Potentially Difficult Soils

4.44.1 Highly Pervious Soils

Loss of ground water through highly pervious strata represents an obvious threat to the stability of the trench.

Hutchinson, et al (1974) point out that fluid loss in pervious soils rises sharply with bentonite concentration below about 4-1/2 percent, even in sands of relatively low permeability of about 5×10^{-3} cm/sec.

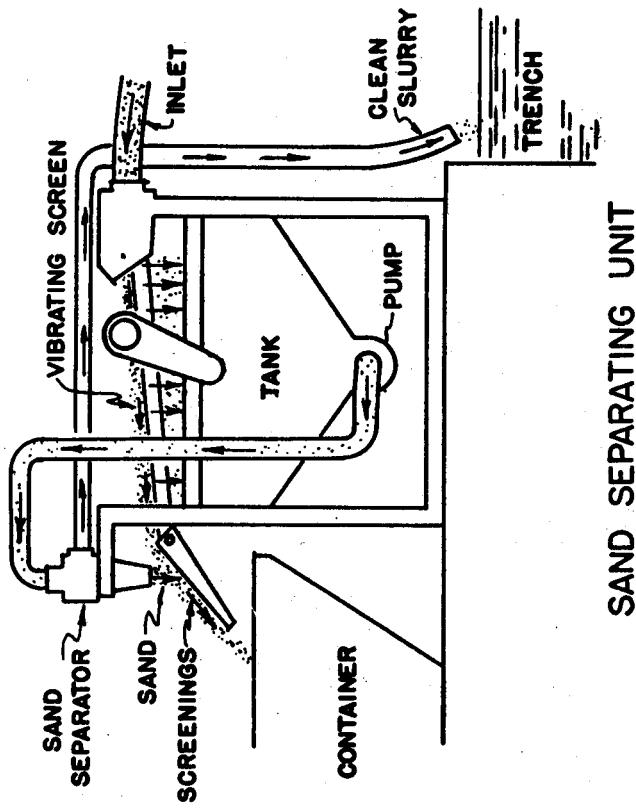
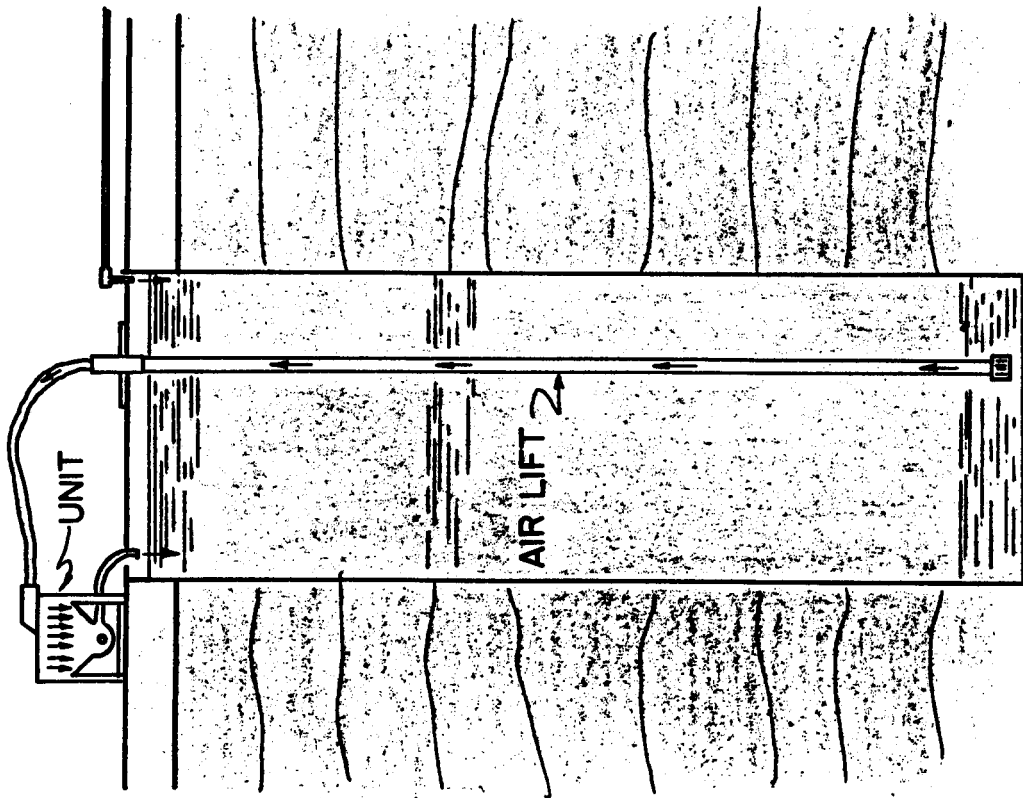
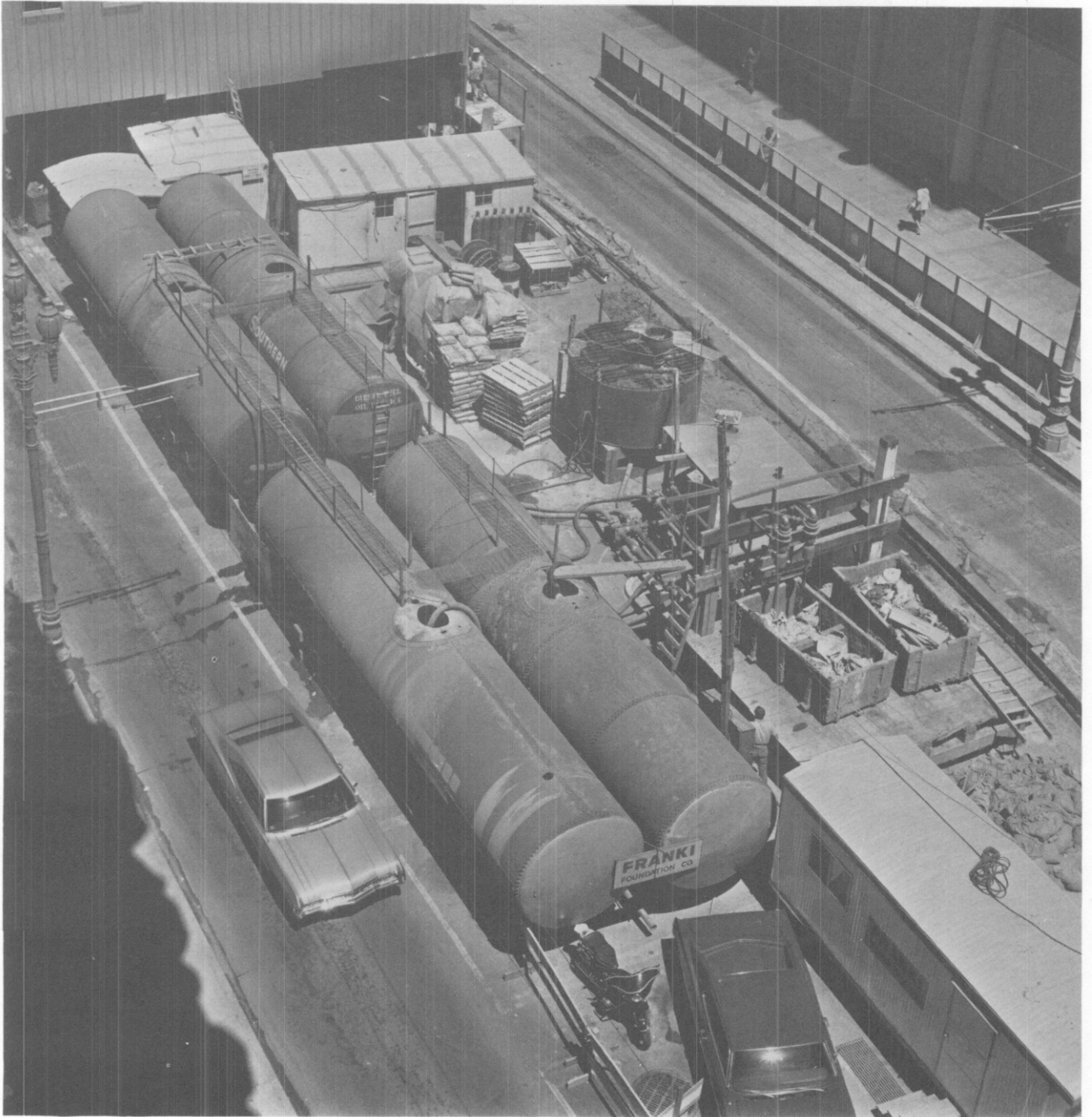


Figure 25. Clean-up with sand separator unit.
 (Courtesy of ICOS Corporation).



Note: Storage in tank cars.

Figure 26. Mud plant.
(Courtesy of Franki Foundation).

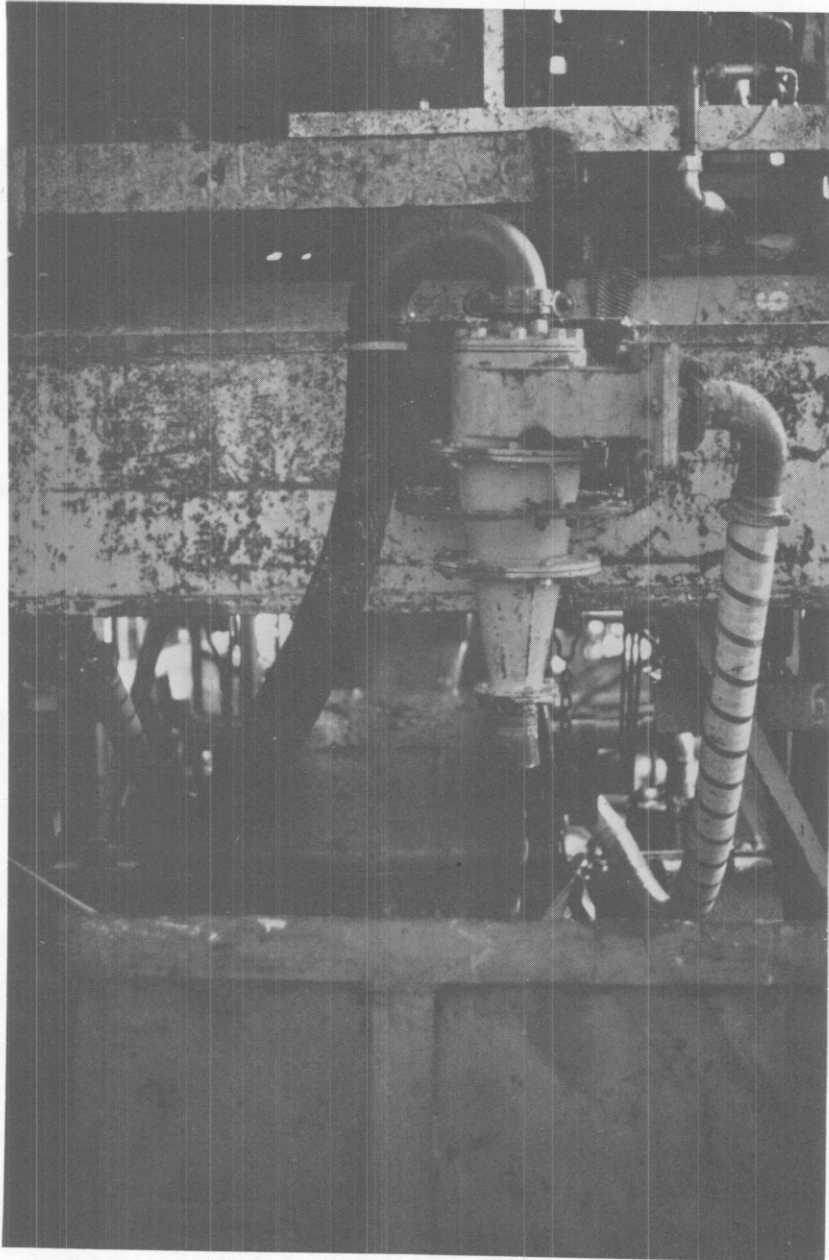


Figure 27. Hydrocyclone sand separator.
(Courtesy of ICOS Corporation).

They recommend that the bentonite concentration be greater than 4-1/2 percent to protect against fluid loss. However, in relatively impervious soils, such as clayey sands, compact glacial till, or clay where fluid loss is not a factor, there is no valid reason to adhere to the 4-1/2 per cent bentonite concentration criterion. Typically the effectiveness of the normal four to six percent bentonite concentration is limited to soils of permeability less than about 10^{-1} cm/sec. to 10^{-2} cm/sec. (Sliwinski and Fleming, 1974; Hutchinson, et al, 1974). More permeable soils may require a variety of measures such as increasing bentonite concentration and/or the addition of fine sand or various plugging agents to control seepage loss.

Some of these plugging agents as described by Rogers (1963) are:

Hay	Mica
Excelsior	Asbestos
Wood shavings or fibers	Shredded paper and bentonite
Wheat bran	Beet pulp
Beans, peas, rice	Flaxseed
Rubber pulp	Chicken feathers
Cotton	Chopped hemp
Cottonseed hulls	Cellular plastics
Sugar can fibers	Cellulose flakes
Rock Wool	Corn cobs
Nut hulls	Cork
Granular plastics	Ground tires
Bark fiber	Coke
Glass fiber	Rock
Perlite	Vermiculite
Textile fibers	

Certain soils (for example, open gravel or broken stone) may be so pervious that fluid loss cannot be controlled. Under such conditions it may be necessary to grout the pervious layer in advance of construction. At one project in Namur, Belgium, it was proposed to grout with a bentonite-cement mix (4 percent cement and 14 percent bentonite by weight) (Bauer, 1975).

4.44.2 Saline Soils

In general, this is not a severe problem, so long as the bentonite is hydrated with fresh water. Even in coastal sites where the land had been filled hydraulically with sand, the salt concentration was not found to be of sufficient concentration to cause adverse effects (Fuchsberger, 1974). Walls have been built in beach sand by the sea without difficulty.

Each case must be checked independently. Grab bucket excavation is less likely to result in salt contamination because pore water is removed with each bite, and so long as a positive head is maintained, ground water cannot enter the trench. Reverse circulation, on the other hand, will reintroduce the pore water of the soil into the slurry mix.

4.44.3 Soft Clays

The Norwegian Geotechnical Institute is conducting experiments on the stability of slurry stabilized panel excavations in soft clay. These data are in addition to that published previously by DiBiagio and Myrvoll (1972). At this writing the data are not available.

In the absence of more definitive research on soft clay, soils with a shear strength of less than 500 psf are suspect with respect to stability and excessive deformation. Under these conditions panel lengths and construction procedures must be verified by experimental test sections in early stages of construction. Such test excavations must be accompanied by careful monitoring of deformations in order to establish the constraints and controls that may be required to prevent damage to adjacent structures and settlement of adjacent ground.

4.44.4 Calcium Laden Soils

Calcium contamination comes from lime soils, gypsum, or anhydrite in the ground (Sliwinski and Fleming, 1974). It may lead to flocculation and an ineffective mudcake on the trench wall.

An example of trench collapse was reported by Mayer (1967). The trench was in fine sand and the failure resulted from flocculation of the bentonite slurry, because of a high lime concentration in the soil.

4.44.5 Organic Soils

Peat may overbreak and lead to an irregular wall. Also, it may float free into the slurry and become embodied within the concrete. Organic soils may also adversely affect the pH.

4.44.6 Residual Soils

Experience with residual soils in Brazil has shown severe pH contamination caused by the presence of iron oxides. The slurry became so thick and viscous that it was necessary to totally replace it before concreting.

4.44.7 Stiff Fissured Clays

Severe overbreaks and local collapses have been experienced in highly fissured overconsolidated London clay. This was attributable in part to an unfavorable joint pattern in the soil.

4.44.8 Soft Silts

Local liquefaction may occur in non-plastic soft silts, perhaps initiated by disturbance from excavation equipment.

4.45. Precautionary Measures

The site investigation must obtain sufficient data on ground water chemistry, soil strength, and pervious strata to permit an evaluation of slurry wall feasibility. Records of water loss during drilling operations are essential as are in situ permeability tests in suspect strata.

During construction, trial panels can be excavated, and the lengths of panels can be varied to determine the most efficient length to minimize the deformations and potential danger to adjacent ground.

In cases where fluid loss is likely, consideration should be given to stockpiling backfill material to fill panels in an emergency arising from a sudden loss of fluid. Such an instance was reported by Fleming, et al (1974) where the contractor was required to stockpile sufficient material to fill one or two panels. In another instance, the contractor was required to stockpile material and sacks of cement to mix with the backfill. Acceptable filling materials would be granular soils, gravelly soils, or crushed stone.

Where the source of leakage is near the surface, excavation has been carried out in two steps. The first step is to dig an oversized trench and refill with lean concrete; the second step is to make the slurry trench through the previously placed lean concrete and form the diaphragm wall in the conventional way.

4.50 STRUCTURAL ASPECTS OF CAST-IN-PLACE WALLS

4.51 Load Bearing

Provided that the slurry quality is adequately controlled, the tremie concrete will satisfactorily displace the bentonite mudcake and develop effective bond against the soil. British practice with cast-in-situ piling formed in slurry stabilized holes bears out the successful

development of soil adhesion. This is especially true in cohesive soils. With more pervious granular soils, the mudcake is more difficult to displace and may reduce side friction by about 10 to 30 percent (Sliwinski and Fleming, 1974).

It is commonplace in Europe to use load bearing diaphragm wall elements (also referred to as slot caissons). Examples of diaphragm wall load bearing elements were reported by Kienberger (1974) at a project in Vienna, Austria. The diaphragm walls, in this case, were about 80 feet deep and supported multi-story structures.

4.52 Concrete

4.52.1 Mix

The concrete must be free flowing mix which will displace the bentonite, and bond to the reinforcing.

An abstract of the FPS Specification (1973) is as follows:

Slump - minimum slump 150 mm (6");
desirable slump 175 mm to 200 mm
(7" to 8")

Water Cement Ratio - less than 0.6

Aggregate - naturally rounded gravel and
sand (if available)

Aggregate Size in Reinforced Walls -
less than 20 mm (3/4")

Sand Content - 35 to 40% of total weight of
aggregate.

Cement Content - at least 400 kg/cubic meter
for tremie concrete.

Many American contractors use, as a rule of thumb, one additional sack of cement per cubic yard when placing by tremie. This compensates for the cement loss which always occurs.

A retarder is often added to provide additional set time. Also, the retarder delays the development of bond to the stop-end tube.

Further elaboration on mix design is made by Xanthakos (1974). He suggests that for a 3/4" aggregate mix, water will be from 41 to 43 gallons per cubic yard if non-air entrained, and about 36 to 38 gallons per cubic yard if air entrained.

4.52.2 Placement

Concrete placement is simultaneously through one or more tremie pipes in each panel. Pipe diameters are normally 6 to 10 inches (Xanthakos, 1974).

The number of tremie pipes per panel will vary according to the panel size, amount of reinforcement, and slurry quality. General practice is to limit the lateral travel distance of the tremie to less than 8 to 10 feet (Fuchsberger, 1974). Thus, for panels less than about 15 feet long only one tremie pipe required, but two are frequently used to speed up the work.

Poor detailing, excessive reinforcement and excessive horizontal steel are all impediments to quality placed concrete. All of these points were emphasized at the London Diaphragm Wall Conference, 1974 (Fuchsberger, Sliwinski and Fleming).

4.53 Steel

4.53.1 General Applications

The reinforcing can be a cage of rebars, a combination of horizontal rebars and vertical wide flange sections, or wide flange sections alone. In this latter application, the wide flange soldier piles serve the dual purpose of vertical reinforcement and panel end stops; the horizontal steel transfers load to the wide flange elements.

Horizontal steel usually does not extend across the panel joints because of the installation difficulties.

Some general observations concerning European and American practice indicate more application of soldier pile reinforcing here than abroad. Post tensioned diaphragm walls have been used in Europe but, to the writers' knowledge, not in the United States.

4.53.2 Bond

Data reported in the literature concerning bond are not consistent. For example, Xanthakos (1974) cites conflicting test results as to the question of whether or not bond is affected by the bentonite.

and whether or not bond develops better on plain bars or on deformed bars. Haffen (1973) reports data which indicate that vertical bars have the same bond strength whether concreted in slurry or not, but horizontal bars show approximately 10 percent less bond strength.

The FPS Specification (1973) limits the bond stress on deformed bars to 10 percent more than the allowable bond on plain conventional bars used in structural concrete.

4.53.3 Cover

The FPS Specification (1973) suggests the following: Concrete cover over steel reinforcement should be at least 75mm (3 inches). Minimum clear spacing between main bars should be at least 100 mm (4 inches).

4.54 Panels and Joints

By far, the most common type of joint used in cast-in-place diaphragm wall construction is formed with a stop-end tube, a round pipe placed in the end of the panel prior to concreting. The stop-end tube is moved frequently, at about 1/2" at a time while concrete is curing, to prevent bond from developing. After the concrete has gained sufficient strength (usually after about three hours) the stop-end tube is removed, thus leaving a concave shape to the end of the panel. Figure 28a illustrates the joint configuration formed by a stop-end tube.

Another procedure is to use a steel wide flange beam or pre-cast I-beam to serve the dual purpose of providing a joint for both shear transfer and vertical steel reinforcement. Figure 29 is an example. The I-shaped soldier piles are installed in pre-excavated augered holes (as is the case shown in Figure 29) or they are simply set with the rebars in an excavated panel. In soft soils, they may be driven.

Joint watertightness is frequently an important criterion for satisfactory diaphragm wall construction. In that connection, a number of methods have been devised to accomplish joint watertightness. These methods are described thoroughly by D'Appolonia, et al (1974) and by Xanthakos (1974). The methods include installation of permanent water stops across joints, post grouting at the joint through plastic tubes left in the tremie concrete at the joint contact, and incorporation of sections of interlocked flat web steel sheeting across the

ROUND JOINT, USING STOP-END TUBE

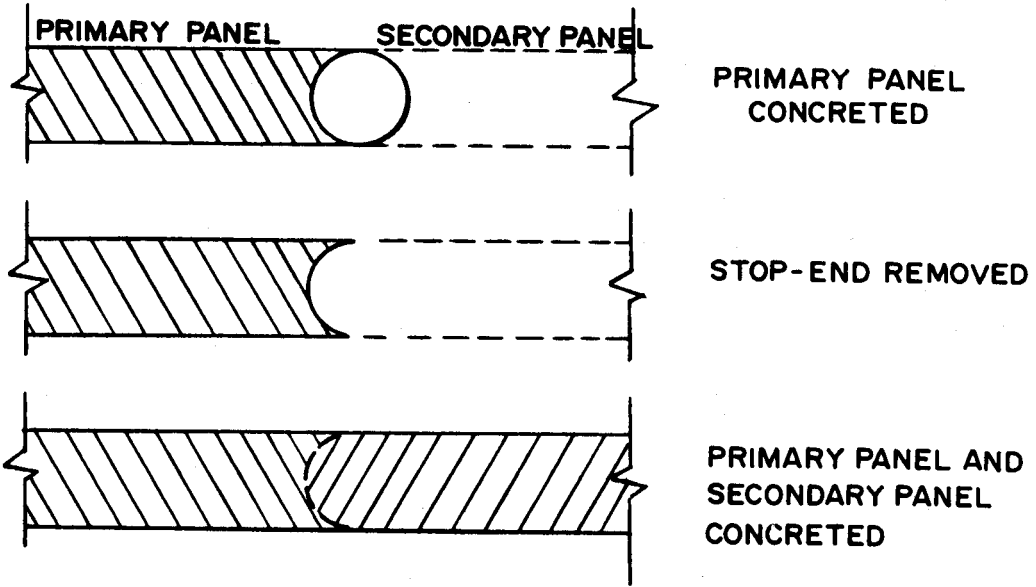
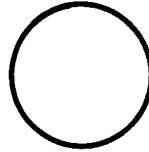
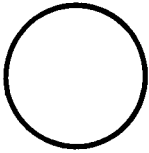
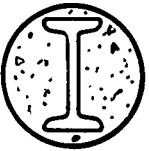


Figure 28. Panel joint with stop-end tube.

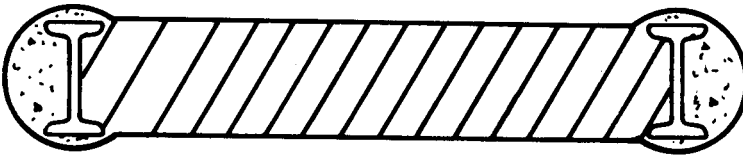
TWO-STEP EXCAVATION



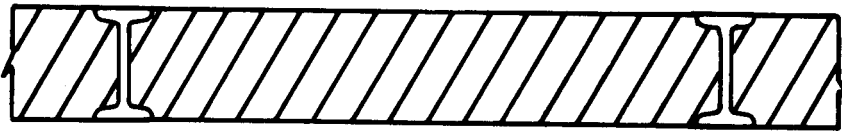
AUGER PRIMARY HOLES.



**SET SOLDIER PILES;
FILL WITH LEAN
CONCRETE OR GROUT.**



**TREMIE PRIMARY
PANEL.**

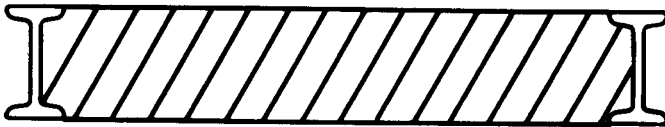


**TREMIE SECONDARY
PANEL.**

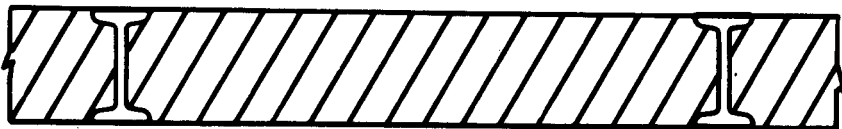
ONE-STEP EXCAVATION



**EXCAVATE PRIMARY
PANEL AND SET
SOLDIER PILES.**



**TREMIE PRIMARY
PANEL.**



**TREMIE SECONDARY
PANEL.**

Figure 29. Panel joints with I-beams.

joint. This latter method is believed superior to the others, which frequently are difficult to implement successfully.

Diaphragm walls, constructed of precast elements placed within slurry stabilized trenches, have inherent advantages with respect to water-tightness. In this case a grout mix of bentonite and cement sets up on the soil side of the precast wall and in the space that remains within the joint.

For a more detailed discussion on panels and joints, the reader is referred to Xanthakos (1974) and to D'Appolonia, et al (1974).

4.60 EXCAVATION OF SLURRY TRENCHES

4.61 Guide Walls

A well-constructed guide wall is essential to prevent caving of the trench wall in the uppermost part of the excavation. It not only serves to protect the integrity of adjacent structures but also insures the competency and the appearance of the uppermost part of the concreted wall. The guidewall serves additional functions: a) to align the trench, b) to contain the slurry, c) to suspend precast elements, and d) to suspend reinforcing steel in cast-in-place walls.

Figure 30 shows alternate concrete guidewall sections. One of the principal concerns is to prevent undermining of the wall caused by agitation of the bucket in the slurry. An L-shaped section is helpful in that regard. For additional stability cement may be added to the dry mix and compacted in place to impart permanent cohesion to the compacted backfill.

4.62 Trenching

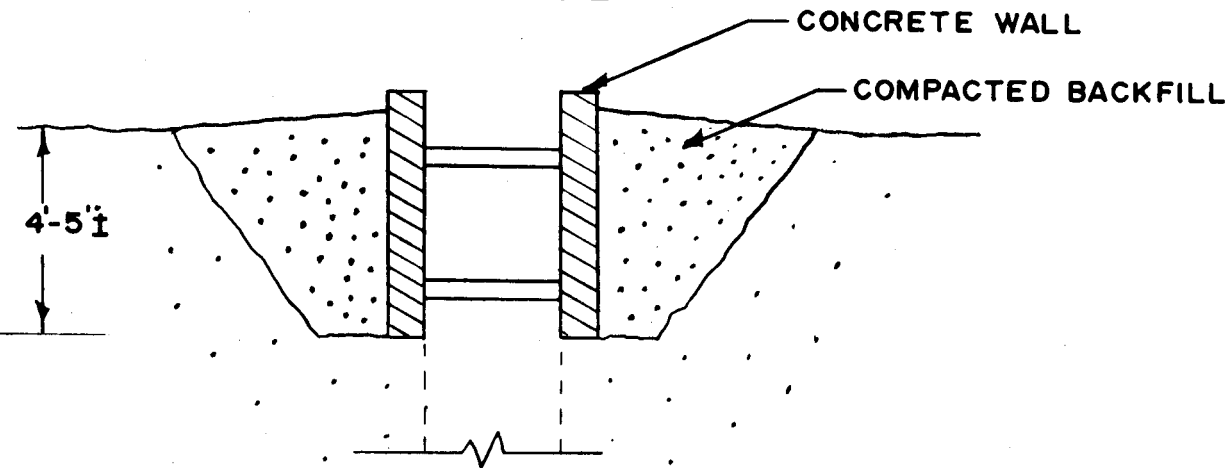
4.62.1 General

Procedures are:

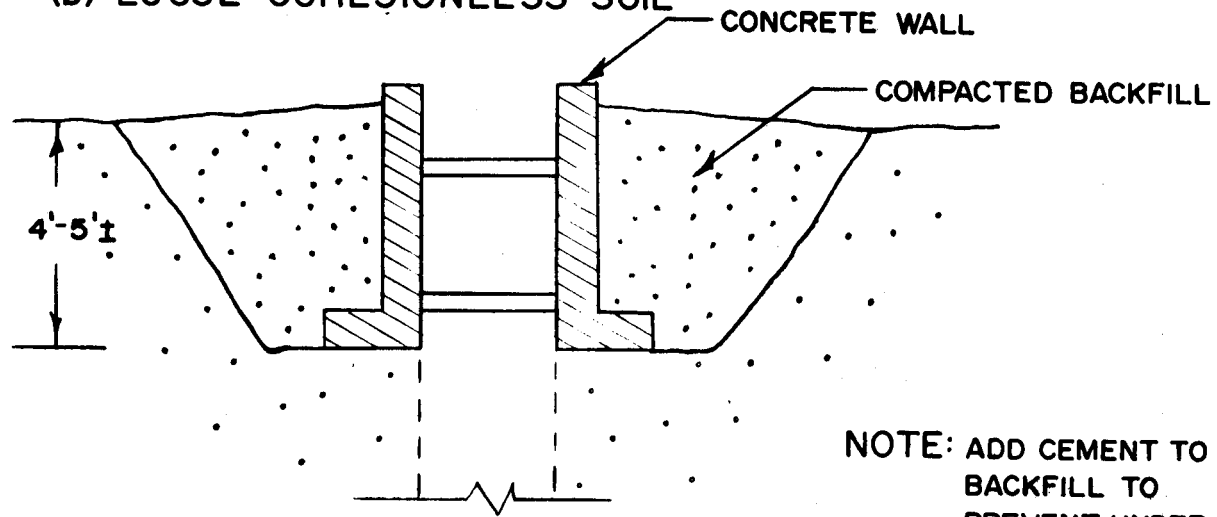
Excavation Buckets. These bring the material directly to the surface, discharge load, and then are reintroduced into the trench.

Direct or Reverse Circulation. These methods break up the material into smaller particles so that the material can be mixed with the bentonite slurry and circulated through piping back to the screening-desanding operation. Care must be taken to avoid clogging of circulation lines by stones or broken boulders.

(a) COMPACT COHESIVE SOIL



(b) LOOSE COHESIONLESS SOIL



NOTE: ADD CEMENT TO BACKFILL TO PREVENT UNDERMINING AND TO INCREASE STABILITY.

Figure 30. Guide walls.

Percussion tools or chopping bits may be used in hard ground. Devices used in conjunction with the circulation method include percussion techniques and rotary cutting devices which are maintained in the bottom of the trench and advanced to the required depth without necessarily being brought to the surface.

One practical consideration is the problem of disposal. Excavation buckets discharge relatively dry material, low in slurry contamination. In contrast, the discharge from reverse circulation is more fluid and so may require watertight trucks and special methods of disposal.

Typically, with cast-in-place walls, alternate panels are excavated and concreted between stop-ends. Then the remaining in-between panels are completed. Another procedure is to proceed continuously by excavating and concreting one panel at a time and always setting a stop-end at the leading edge. In this case, the work proceeds at two or more locations so that excavation equipment is busy during concreting.

4.62.2 Excavation Methods

ELSE Trenching Machine

An early excavation technique was the ELSE trenching machine which was introduced in Italy in 1958. This trenching shovel operates like a power shovel. The ELSE trenching shovel is a specially designed device which operates from a vertical mast that is advanced into the trench with the excavation. With each bite the shovel is brought to the surface to discharge its load.

This device is still used in Japan (Ikuta, 1974), but is rarely used in the United States. A detailed description of the operation of this device is provided by Xanthakos (1974).

Clam Shell

The most common types of excavation equipment are specially designed clam shell buckets, conventionally referred to as grabbing tools or grabs. Typically, the ends of the grab are rounded to effectively remove soil from the semi-circular shape of the previously constructed panel formed in contact with a stop-end tube. In cases where a wide flange section is used in the end of the panel, the bucket may be equipped with a square end to permit effective excavation. Figures 31, 32, and 33 show various types of grab buckets.



- Notes:
1. Clam shell operates by electro-hydraulic mechanism.
 2. Guide skirt above clam shell.
 3. Faneuil Hall (Boston) in background. Of historical interest to the cause of American independence.

Figure 31. Cable-suspended grab.
(Courtesy of Franki Foundation).



Note: Clam shell operates by cable mechanism.

Figure 32. Kelly bar suspended grab.
(Courtesy of Franki Foundation).



2'6" x 14'0" Hydraulic Clam Shell



2'6" x 7'6" Mechanical Clam Shell

Figure 33. Grab buckets.
(Courtesy of ICOS Corporation).

Vertical and horizontal alignment of the bucket is assisted by a guiding skirt (perhaps 15 or more feet high, 6 feet or more long, and slightly less wide than the grab bucket). The bucket extends just below the guide skirt.

The guide and the grab are suspended by cable or by Kelly bar. The decision of whether to use a Kelly bar or cable is governed by requirements for vertical and horizontal alignment and by the magnitude of downward force that must be developed in hard ground. At relatively shallow depth the Kelly is rigid, not easily deflected by hard strata, boulders, etc., and therefore generally preferred.

Fuchsberger (1974) states a preference for cable suspended tools to aid in maintaining verticality of the trench. Franki has used 16 inch diameter Kellys to achieve stiffness but prefers suspension below 100 feet. In contrast, Xanthakos (1974) reports that Soletanche conventionally uses a cable suspended grab to depths of about 65 feet but uses a Kelly bar at greater depths. Thus, it is clear that opinion varies concerning the use of Kelly bar or cable.

The jaws of the grab maybe operated mechanically or hydraulically. In the mechanical operation the equipment weight may not be fully effective and therefore the grab is less effective in hard ground. Hydraulic devices vary--they may work from a single central piston or from pistons on each side to close the jaws of the grab.

4.62.3 Direct and Reverse Circulation Methods

Devices are:

- a. Rotary cutter heads which rotate about a vertical axis.
- b. Percussion tools which chop up the material.
- c. Cutter heads which operate by rotation about the horizontal axis.

1. Soletanche. A Soletanche device, which operates on rails that are set along the trench, may use either the percussion or the rotary methods (about the vertical axis). The cutting tool benches back and forth between the ends of the panel, and cuttings are brought to the surface by suction and/or air lift through the tool itself.

2. The BW Drill. The BW drill is marketed through the Japanese firm, Mitsubishi International. Like the Soletanche device, it operates on rails. It is a self-contained excavation tool with four

rotary cutter heads at its base (rotation about the vertical axis). Slurry cuttings are circulated through the device in suction lines, desanded, and then reintroduced into the trench.

The machine has self-contained inclinometer instrumentation which senses and controls verticality. The BW drill comes in widths from 16 inches to 47 inches and in lengths from 8 feet to 11 feet. This machine is applicable in both sands and in cohesive soils but is difficult to operate if stones are larger than about 4 inches. It is not feasible to operate if cobbles or boulders are present (Ikuta, 1974).

3. TBW Excavator. Operation of this device is with cutter heads rotating about the horizontal axis. It is a product of the Japanese firm, Takanaka, and its use was reported by Ikuta (1974). The cutter heads chip out the soil and work the material into the bell mouth of the tool so that the soil can be removed by suction in the recirculation system. As in the case of the BW drill, the TBW machine is equipped with inclinometers which are used to control the verticality of the trench excavation.

4.62.4 Hard Ground

Obstructions are broken up by heavy chisels or chopping devices to facilitate removal by grab buckets, by percussion, or by rotary tools. In general, grab buckets or rotary devices are used in soils of normal density or consistency. Percussion methods are necessary in cemented soils, hard boulders, clays, and till.

Experience has shown that percussion methods used to advance trenches into rock may cause severe fracturing. Later when excavations are carried into the rock, this fractured zone may break away and undermine the wall. Moreover, the fractured rock can be a source of leakage in pervious soils. Precautionary measures are to dowel, core, or tieback into the rock.

Sliwinski and Fleming (1974) report a method to penetrate soft rock by first boring 30-inch diameter holes at regular spacing and then removing the material between the bored holes with a hydraulically operated grab tool. Tamaro (1974) reports a similar procedure used by ICOS to penetrate bouldery formations.

4.70 DIAPHRAGM WALLS OTHER THAN CONTINUOUS CAST-IN-PLACE CONCRETE

4.71 General

This discussion covers the following:

a. Diaphragm walls constructed of precast elements set within slurry stabilized trenches.

b. Hybrid techniques using pre-set steel or concrete soldier piles in combination with intervening cast-in-place concrete panels.

c. A wall composed of bored piles set in one or more lines.

4.72 Precast Concrete Methods

4.72.1 General

This general subject was discussed by Sverdrup and Parcel Associates (1973), D'Appolonia, et al (1974), and Xanthakos (1974). Precast concrete elements are normally set within a continuously excavated slurry stabilized trench. Figures 34 and 35 show schematics of the methods developed by Soletanche and Bachy, both French companies. Franki uses a similar method.

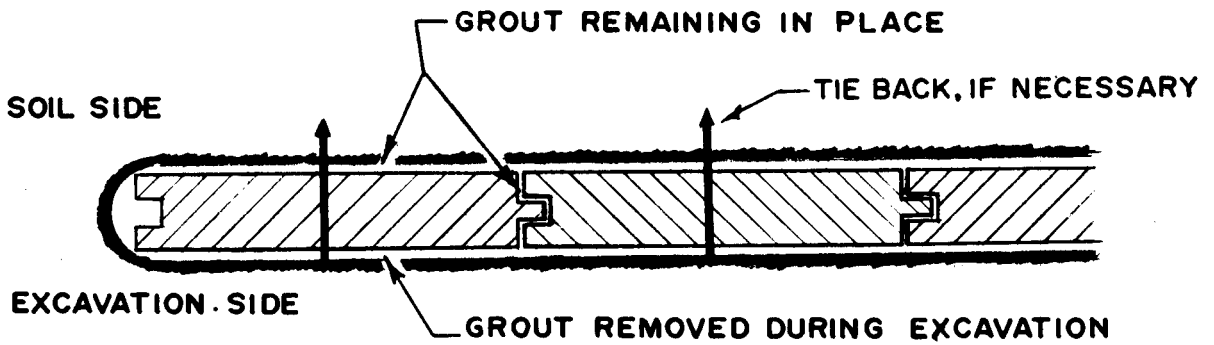
Precast elements are carefully aligned and suspended from the guide wall until the grout slurry (or cast-in-place concrete) below the elements has gained sufficient strength to provide vertical support. The elements can be used alone or in combination with an underlying conventional cast-in-place diaphragm wall.

The grout fills the space between the back side of the precast element and the soil, thus forming tight contact and an impervious membrane. The inside face of the wall is coated with a special compound which facilitates removal of the hardened grout during the excavation and ensures the satisfactory appearance of the inside face of the wall. Because the excavation is continuous, the grout must gain sufficient strength so that it will not flow into the subsequently excavated panel and expose an excessive length of unsupported trench to possible deformation during the excavation. For this reason, some contractors may work two sections of the wall concurrently allowing one to set up while excavating and setting panels in another.

The size of the precast elements is controlled by the load capacity of the crane. In urban areas the crane size may also be controlled by city ordinances thereby limiting panel size. Depending upon wall thickness, the depth limitation is normally in the range of 30 to 50 feet. Occasionally greater depths can be achieved with special equipment.

The T-beam/slab combination (Figure 34 b) offers flexibility with regard to depth. In this case the T-beam can be carried to a lower elevation to engage a bearing stratum or to provide additional passive resistance. Slab panels need only extend to the depths required for the permanent wall.

(a) TONGUE & GROOVE



(b) T BEAMS & SLABS

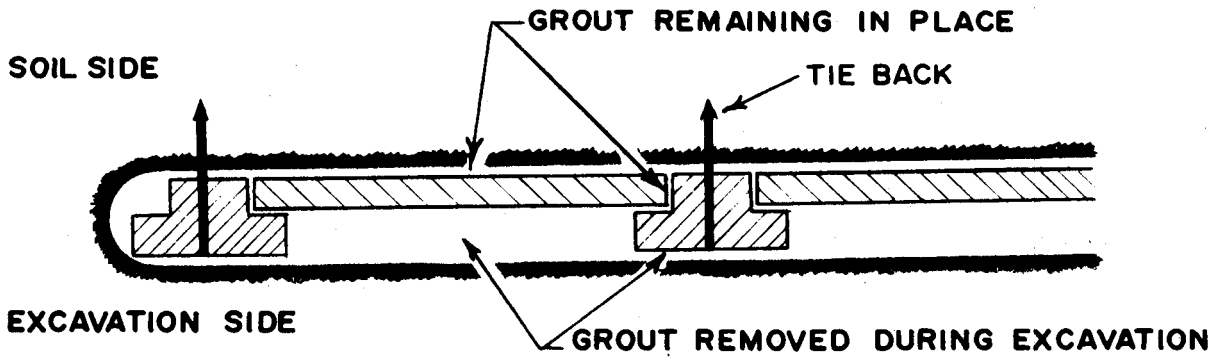
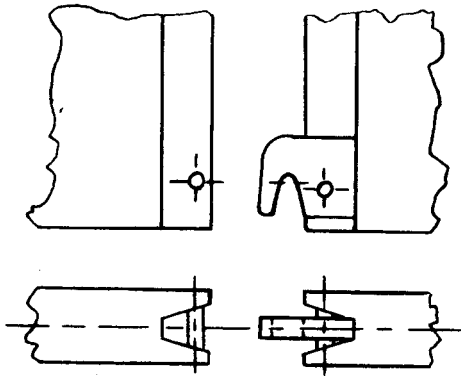
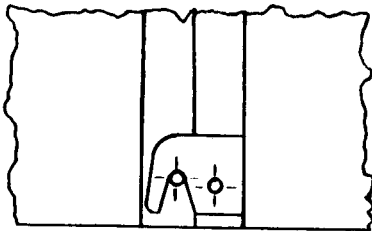


Figure 34. Panosol walls (Soletanche, France).



**PREFASIF SYSTEM:
SECURING THE FOOT:
THE HOOK ENGAGING
INTO THE LOCKING
BAR.**



**PREFASIF SYSTEM:
THREE EXAMPLES OF
USE OF THE SLOTS TO
GUARANTEE A WATER-
TIGHT JOINT BETWEEN
SECTIONS.**

**1- WITH THE WATERSTOP
JOINT.**

**2- WITH A REINFORCED
CONCRETE KEY.**

**3- WITH SEALING GROUT
ALONE.**

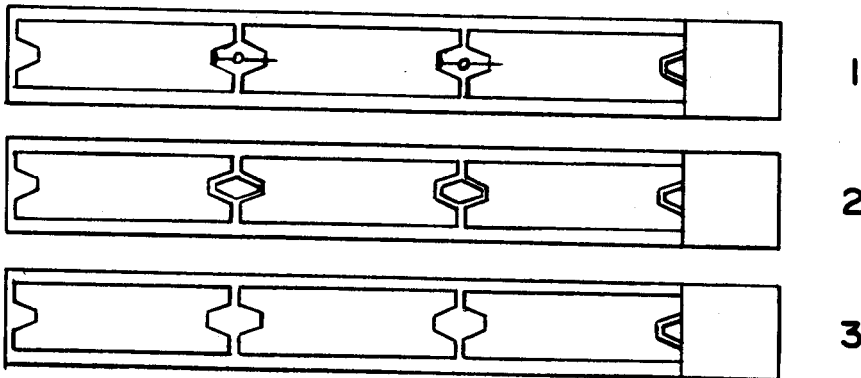


Figure 35. Prefasif wall (from Bachy Enterprise, France).

4. 72. 2 Grout and Slurry

The Soletanche method uses a special mix which serves the dual purpose of stabilizing the trench and then hardening in place. The base mix is cement and bentonite with additives to control setting time, viscosity, and strength. Bentonite and cement, without such additives, become viscous, sticky, and set up so rapidly that it would be impractical to allow the mixture to remain in the trench during excavation.

Other companies employ conventional bentonite mud slurries for trench stabilization during excavation but then introduce a cement-bentonite sealing grout (about 4 percent bentonite and 14 percent cement) into the bottom of the panel prior to placing the precast element. The panel then displaces the mud slurry so that only the cement-bentonite mix remains. Such a method, described by E. Colas Des Francs (1974), is the Bachy method. It is also used by Franki Foundation.

The sealing grout of the Bachy method hardens to form a tight contact between the wall elements and the soil and a satisfactory support below the base of the precast panels. As with the Soletanche method, additives may be used to control viscosity and setting time. Because the sealing grout is introduced separately, criteria for it and for the bentonite mud slurry for trench stabilization are not the same. This allows some flexibility in grout design without compromising the design of the mud slurry.

4. 72. 3 Discussion

The published documentation concerning performance of slurry stabilized trenches is based largely upon bentonite slurries used in connection with cast-in-place walls. Therefore, much of the technology, associated with maintaining the slurry to prevent fluid loss and with a variety of difficult soil conditions, stems from such experience.

Nonetheless, since the function of slurries for tremie concrete panels does not necessarily coincide with the function of grout used with precast panels, one cannot apply the same slurry requirements for both cases.

One of the main themes of this report is ground support and related protection of adjacent buildings and adjacent ground. Thus, there must be adequate assurance that the slurry and/or sealing grout will satisfactorily maintain trench stability. In difficult ground (such as open gravel, limey soils, organic soils, soft silts, or clays) test panels should be excavated and monitored to establish criteria for slurry mix, optimum length of the open trench, and construction sequence.

4.73 Soldier Pile Combination Walls

4.73.1 General

The techniques to be described in this section all use soldier piles at regular spacing along the wall in combination with poured concrete between the soldier piles.

Several techniques use soldier piles that are set in pre-augered holes. The intervening space is excavated and concreted. Normally, the augered hole is stabilized with a bentonite-cement slurry mix. Upon hardening this grout develops sufficient strength to provide competent contact with the soil. Later, during excavation between the soldier piles to permit concreting of the wall section, this hardened grout is removed.

One of the features of first setting the soldier pile in an augered hole and then concreting the panel is that the soldier pile can be carried to a lower elevation than the wall panel for the purpose of obtaining vertical bearing and/or increased lateral resistance in more favorable underlying strata. This feature is also common with the T-beam and slab combination used in the precast wall technique described in Section 4.72.

Another approach is to eliminate the extra step of augering and setting soldier piles separately. In this method, following excavation of the panel, the soldier piles are positioned together with the reinforcing cage, and then the panel is concreted.

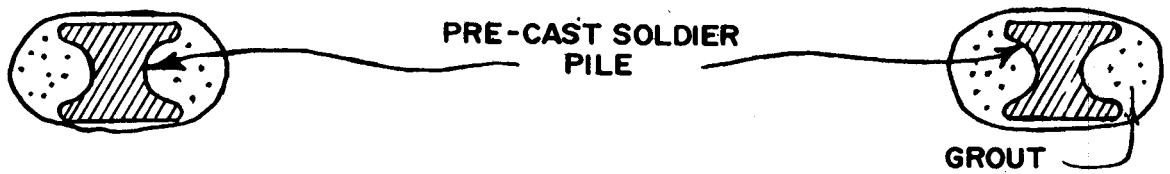
4.73.2 Two Step Excavation: First for Piles, Second for Panel

Two techniques are shown in Figures 36 and 37. Figure 36 shows the wall in combination with a precast concrete soldier pile, and Figure 37 shows the wall in combination with a steel wide flange member used, for example, in the BARTD subway construction in San Francisco.

This latter wall is also known as the SPTC (soldier pile tremie concrete wall). It was used at the San Francisco Civic Center Station and at the Embarcadero Station, using 95 foot long walls and excavating to 70 feet in soft bay mud. (See Thon and Harlan, 1971; and Armento, 1973).

In both cases the soldier piles are set within pre-augered holes which are subsequently filled with grout to form an intimate contact between the soldier pile and surrounding soil. Next, the space between the previously set soldier piles is excavated, and the panel is filled with cast-in-place concrete by the tremie method in a slurry stabilized trench.

I. SET SOLDIER PILE IN PRE-EXCAVATED HOLE



2. EXCAVATE AND CONCRETE PANEL.

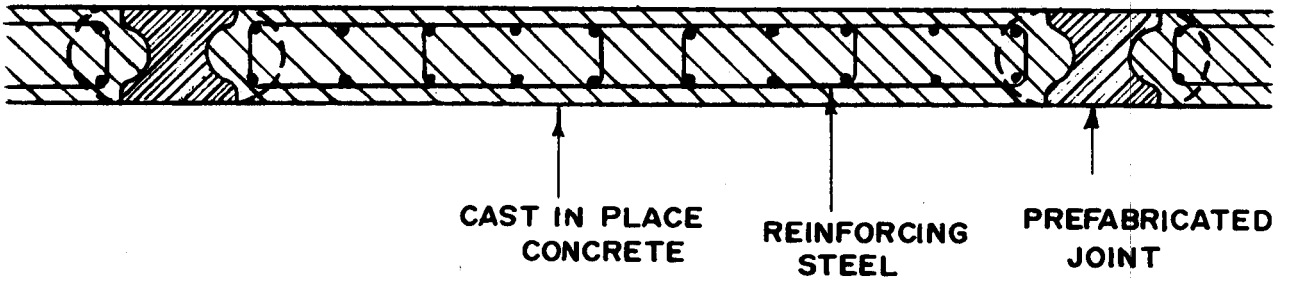
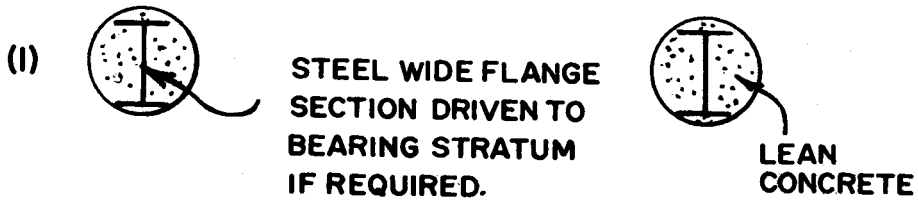


Figure 36. Two step excavation in slurry trench using precast soldier piles and tremie concrete.

SET SOLDIER PILE IN PRE-EXCAVATED HOLE.



EXCAVATE AND CONCRETE PANEL (REINFORCING IF REQUIRED BY REBARS OR I-SECTION)

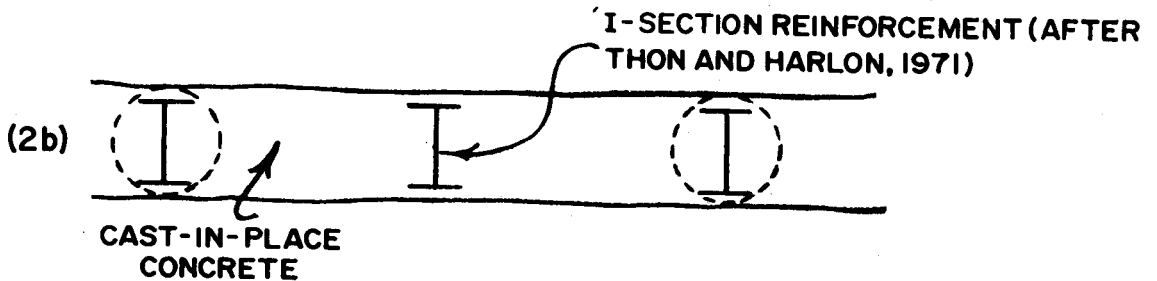
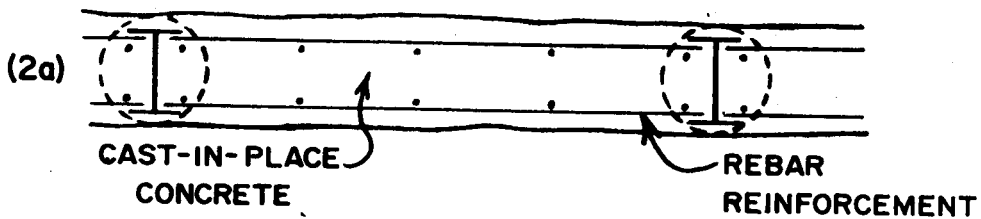


Figure 37. Two step excavation in slurry trench using steel wide flange soldier piles and tremie concrete.

4.73.3 One Step Excavation

In this method (see Figure 38) the entire panel is excavated at once, as is the case when stop-end tubes are used. Following the panel excavation the soldier piles and reinforcing are placed concurrently. Applications are described by Tamaro (1974). One of these jobs was the Federal Center Southwest Station, Washington, D. C.

4.73.4 Discussion

Cost considerations aside, preset soldier piles offer inherent advantages concerning protection of adjacent structures, especially in unstable or weak soils and/or in the presence of heavily loaded foundations. Risk exposure during setting of the soldier pile is minimal; subsequently during excavation of the intervening panel, the length between the soldier piles is relatively short -- in the case of BARTD, only about 6 feet. Thus, protection against movement (or worse still, collapse) is always maintained. As discussed earlier in this section, when soldier piles are installed separately, they can be extended to whatever depth is required to develop bearing and/or toe restraint.

4.74 Bored Pile Walls

4.74.1 General

These walls are built by forming grouted or cast-in-place concrete piles continuously along the line of the excavation. For purposes of discussion, the methods have been classified as "small-diameter piles", conventionally formed by grouting using hollow stem auger equipment, and "large-diameter piles", formed by excavation with a solid auger or with a bucket within a casing and then filling with concrete after withdrawal of the excavation equipment.

In both cases the piles are reinforced. Figure 39 illustrates these bored pile walls.

4.74.2 Small-Diameter Piles

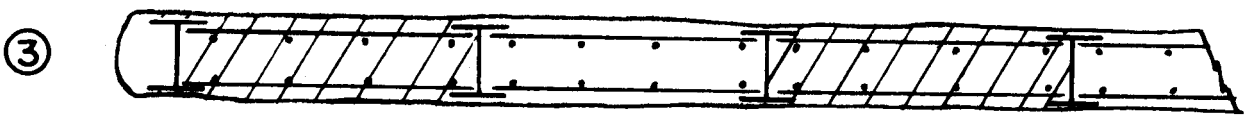
Piles are formed using hollow-stem auger equipment with outside diameters ranging typically from 12 to 16 inches. The procedure is to install alternate piles (primary piles) then, after the grout is set, to install the remaining piles (secondary piles). The piles may be augered in one or more lines, as necessary to achieve the desired watertightness and/or structural strength (see Figure 39).



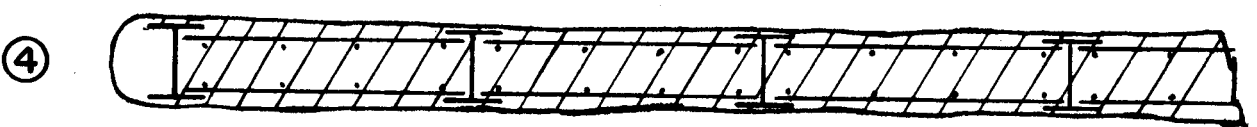
① SET SOLDIER PILES AND REBARS
IN ALTERNATE PANELS.



② CONCRETE ALTERNATE PANELS.



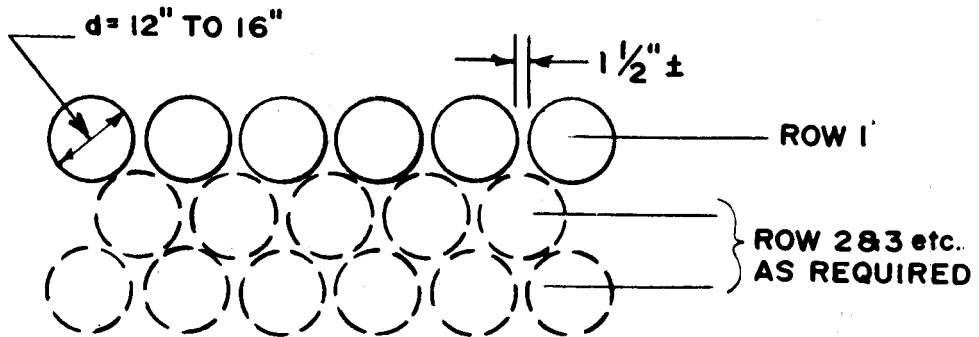
③ EXCAVATE INTERVENING PANELS
AND SET REBAR CAGE.



④ CONCRETE

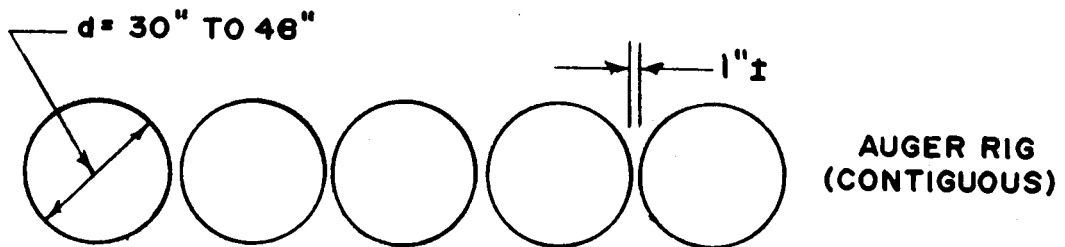
Figure 38. One step excavation with soldier piles
(after Tamaro, 1974).

(a) SMALL DIAMETER (COMMONLY "TANGENT" PILES)

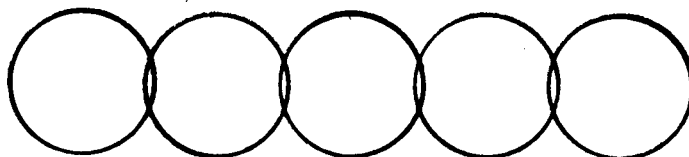


GROUT FILLED WITH HOLLOW STEM AUGER

(b) LARGE DIAMETER (COMMONLY "CONTIGUOUS" OR "SECANT" PILES)



AUGER RIG
(CONTIGUOUS)



CONCRETE FILLED

ROTATING CASING
WITH CUTTING
EDGE.
(SECANT)

Figure 39. Reinforced bored pile walls.

The grout is a mixture of Portland cement, fluidifier, sand, and water. Sometimes a mineral filler may be added as well. The grout is injected under pressure through the central hole as the auger is withdrawn, and soil cuttings are removed from the auger flights as they emerge from the ground. Immediately following grouting, a cage of reinforcing steel or a wide flange steel beam section is inserted into the wet mortar.

This method has been used in the United States by the Intrusion Prepack Co. and by the Turzillo Contracting Company. Also, the method was used in connection with the construction of the Tokyo Subway in a cut-and-cover operation.

4.74.3 Large-Diameter Piles

Shaft diameters typically range from about 2-1/2 to 4 feet. Depending upon the nature of the soil and ground water conditions, the excavation can be made with or without casing, either in the dry or in a slurry-stabilized hole. As in the case of the small-diameter piles, alternate piles are installed first, then the intermediate piles are installed.

Reinforcing is positioned following excavation, then the hole is filled with concrete. Several instances have been reported where the reinforcing cage included styrofoam inserts around certain bars. During the subsequent excavation the styrofoam is removed, and bars bent out to tie into structural deck, floor, or base slabs. Figure 39 shows two types of large-diameter bored pile walls.

Contiguous Pile Wall

Contiguous piles are made by a large-diameter auger rig, such as that conventionally used for drilling caissons. The contiguous piles are separated only by the thickness of the steel shell between adjacent piles.

In 1974, a contiguous bored pile wall was installed in connection with the A406 North Circular Road in London. In this case, a 35 foot deep excavation was made for a highway project. The wall was temporarily supported and then was framed into a concrete horizontal slab in order to achieve cantilever action. The gap between the piles was eventually gunited to waterproof the joint. Reported progress was at the rate of 7 to 8 completed piles per day.

Secant Pile Wall

The overlapping or "secant" piles, also shown in Figure 39, are made by back and forth rotation of casing with a bottom cutting edge. This cuts into the green concrete of previously placed alternate piles. Material is removed by a grab bucket operating within the casing.

The Sverdrup and Parcel report (1973) gives several examples of secant pile walls installed with a benoto rig and completing about 5 to 6 piles every day. Overlap was reported to be about 2 inches.

One of the examples discussed by Sverdrup and Parcel was the application for the Munich subway. This was also discussed by Weinhold and Kleinlein (1969). In this case, the piles were battered outward at 12 degrees to permit construction of the tunnel below foundations of abutting structures without need for other types of underpinning. Krimmer (1972) illustrates similar applications of battered piles to eliminate conventional underpinning for Frankfort subway.

The German applications described above had good success with watertightness. However, the authors stress that meticulous care is required to maintain the alignment tolerance to assure the desired overlap.

Deviations from the required alignment could create gaps in the wall and lead to ground loss--especially in previous soils below the ground water table. Such an instance was reported by Febesh (1975).

4.74.4 Discussion

A bored pile wall has inherent advantages because of the minimum exposure of excavated soil prior to concreting. This provides a measure of additional protection for heavily loaded foundations and/or when excavating in weak or unstable soil. Also, specific augered piles may be carried to a lower elevation for bearing or toe restraint. These characteristics are common to diaphragm walls utilizing "soldier pile" techniques, described in Section. 4.73.

APPENDIX A

API Recommended Practice -- Standard Procedure for Testing Drilling Fluids, API RP 13B

Reprinted from API RP 13B: Standard Procedure for Testing Drilling Fluids by permission of the American Petroleum Institute. Publication dated February 1974.

RECOMMENDED PRACTICE STANDARD PROCEDURE FOR TESTING DRILLING FLUIDS

Foreword

a. This recommended practice is under the jurisdiction of the API Committee on Standardization of Drilling Fluid Materials.

b. The purpose of this recommended practice is to provide standard procedures for the testing of drilling fluids. It is not a detailed manual on mud control procedures. It should be remembered that the agitation history and temperature of testing have a profound effect on mud properties.

c. Metric equivalents have been included in this publication in parentheses following the U. S. customary units.

d. Another publication under jurisdiction of this committee:

Spec 13A: Specification for Oil-Well Drilling Fluid Materials, covers specifications and test procedures for barite, bentonite, and attapulgite clay.

SECTION 1 DENSITY (MUD WEIGHT)

Equipment

1.1 Density may be expressed as pounds per gallon, pounds per cubic foot, grams per cubic centimeter, specific gravity, or pressure gradient (see Table 1.1). Any instrument of sufficient accuracy to permit measurement within ± 0.1 lb per gal, or ± 0.5 lb per cu ft (± 0.01 g per cm^3) may be used. The mud balance is the instrument generally used (see Fig. 1.1 and 1.2). The weight of a mud cup attached to one end of the beam is balanced on the other end by a fixed counterweight and a rider free to move along a graduated scale. A level bubble is mounted on the beam. Attachments for extending the range of the balance may be used.

Procedure

1.2 The instrument base should be set up approximately level.

1.3 Fill the clean, dry cup with mud to be tested; put on and rotate the cap until firmly seated. Make sure some of the mud is expelled through the hole in the cap to free trapped air or gas.

1.4 Wash or wipe the mud from the outside of the cup.

1.5 Place the beam on the support and balance it by moving the rider along the graduated scale. The beam is horizontal when bubble is on center line.

1.6 Read the density at the side of the rider toward the knife edge. Make appropriate corrections when a range extender is used.

1.7 Report the density to the nearest 0.1 lb per gal or 0.5 lb per cu ft (0.01 g per cm^3).

1.8 To convert to other units, use the following relationships:

$$\text{Specific gravity} = \frac{\text{lb per cu ft}}{62.3}, \frac{\text{lb per gal}}{8.33},$$

or g per cm^3

$$\text{Mud gradient in psi per ft} = \frac{\text{lb per cu ft}}{144},$$

$$\frac{\text{lb per gal}}{19.24}, \text{ or } \frac{\text{g per cm}^3}{2.31}$$

Calibration

1.9 The instrument should be calibrated frequently with fresh water. Fresh water should give a reading of 8.33 lb per gal or 62.3 lb per cu ft (1.00g per cm^3) at 70 F (21 C). If it does not, adjust the balancing screw or the amount of lead shot in the well at the end of the graduated arm as required.

**TABLE 1.1
DENSITY CONVERSION**

1	2	3	4		5
			Gradient,		
lb per gal	lb per cu ft	g per cm^3 or specific gravity	psi per 1,000 ft of depth	kg per cm^2 per 1,000 m of depth	
6.5	48.6	0.78	338	78	
7.0	52.4	0.84	364	84	
7.5	56.1	0.90	390	90	
8.0	59.8	0.96	416	96	
8.3	62.3	1.00	433	100	
8.5	63.6	1.02	442	102	
9.0	67.3	1.08	468	108	
9.5	71.1	1.14	494	114	
10.0	74.8	1.20	519	120	
10.5	78.5	1.26	545	126	
11.0	82.3	1.32	571	132	
11.5	86.0	1.38	597	138	
12.0	89.8	1.44	623	144	
12.5	93.5	1.50	649	150	
13.0	97.2	1.56	675	156	
13.5	101.0	1.62	701	162	
14.0	104.7	1.68	727	168	
14.5	108.5	1.74	753	174	
15.0	112.2	1.80	779	180	
15.5	115.9	1.86	805	186	
16.0	119.7	1.92	831	192	
16.5	123.4	1.98	857	198	
17.0	127.2	2.04	883	204	
17.5	130.9	2.10	909	210	
18.0	134.6	2.16	935	216	
18.5	138.4	2.22	961	222	
19.0	142.1	2.28	987	228	
19.5	145.9	2.34	1013	234	
20.0	149.6	2.40	1039	240	
20.5	153.3	2.46	1065	246	
21.0	157.1	2.52	1091	252	
21.5	160.8	2.58	1117	258	
22.0	164.6	2.64	1143	264	
22.5	168.3	2.70	1169	270	
23.0	172.1	2.76	1195	276	
23.5	175.8	2.82	1221	282	
24.0	179.5	2.88	1247	288	

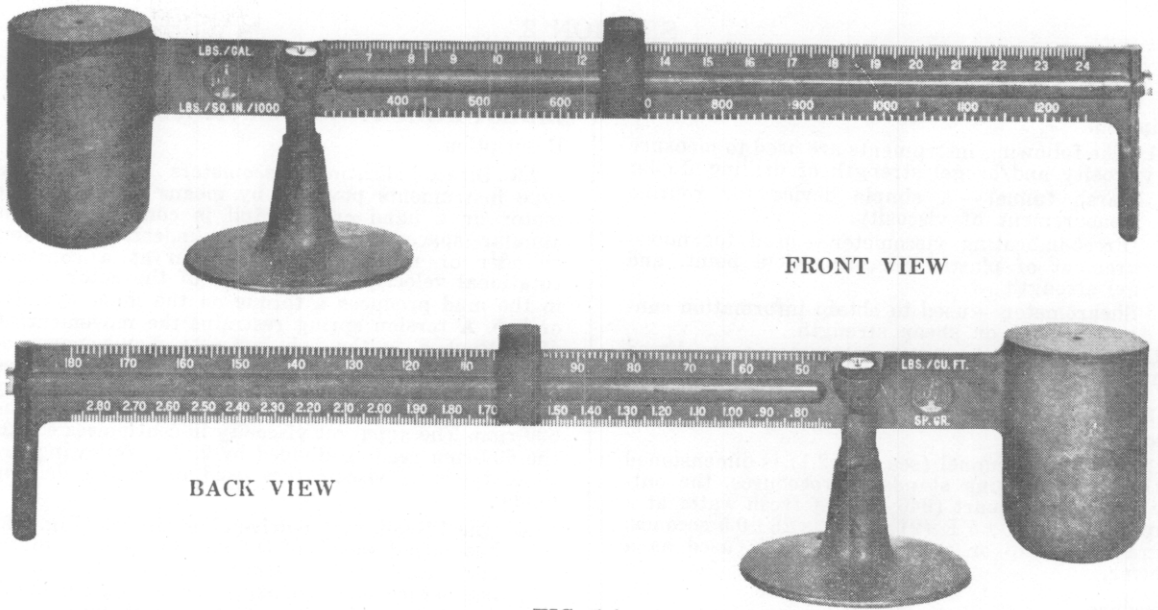


FIG. 1.1
MUD BALANCE

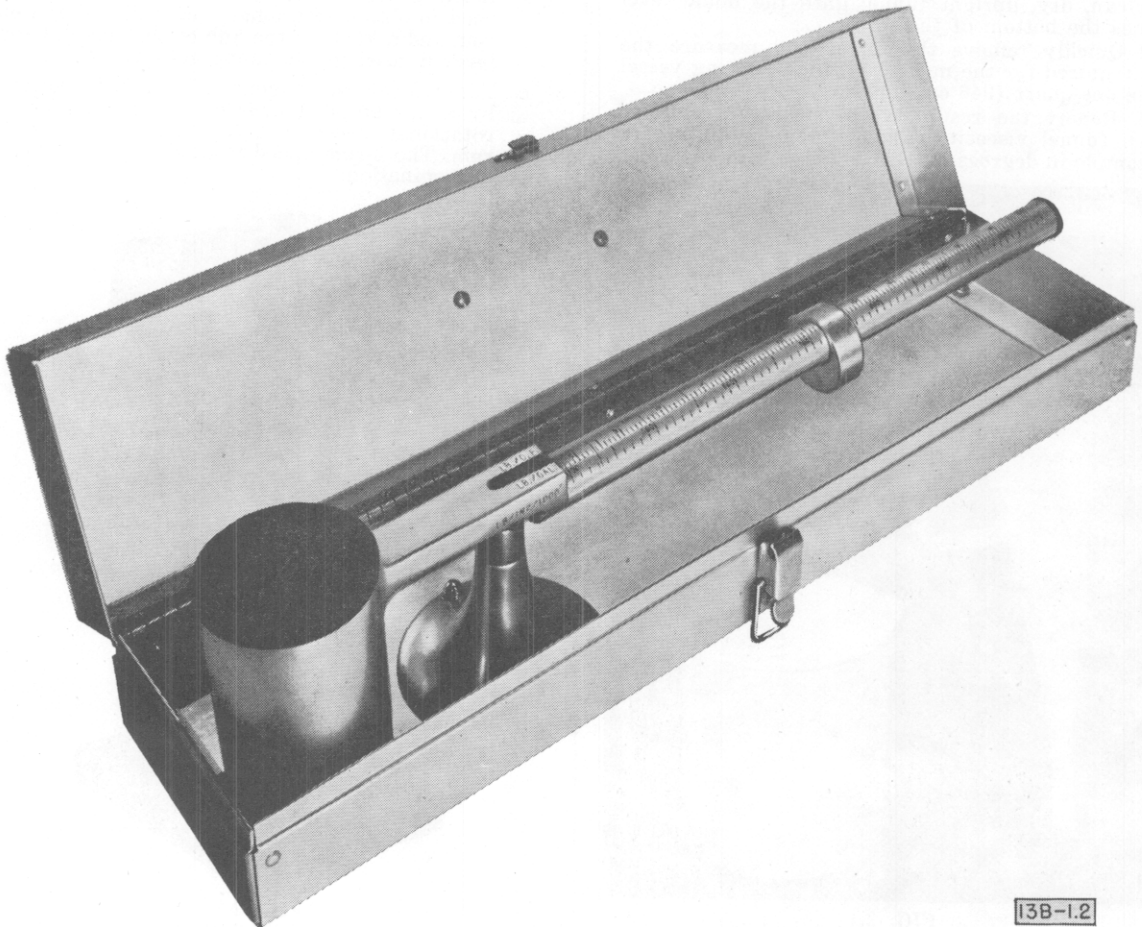


FIG. 1.2
MUD BALANCE

13B-1.2

SECTION 2

VISCOSITY AND GEL STRENGTH

Equipment

2.1 The following instruments are used to measure the viscosity and/or gel strength of drilling fluids:

- a. Marsh funnel—a simple device for routine measurement of viscosity.
- b. Direct-indicating viscometer—used for measurement of plastic viscosity, yield point, and gel strength.
- c. Shearometer—used to obtain information concerning gel or shear strength.

(Dimensions of the above instruments are listed in Par. 2.19.)

MARSH FUNNEL

Description

2.2 The Marsh funnel (see Fig. 2.1) is dimensioned so that, by following standard procedures, the out-flow time of one quart (946 cm³) of fresh water at a temperature of 70±5 F (21±3 C) is 26±0.5 seconds. A graduated cup or one-quart bottle is used as a receiver.

Procedure

2.3 Cover the orifice with a finger and pour a freshly taken mud sample through the screen into the clean, dry, upright funnel until the liquid level reaches the bottom of the screen.

2.4 Quickly remove the finger and measure the time required for the mud to fill the receiving vessel to the one-quart (946 cm³) mark.

2.5 Report the result to the nearest second as Marsh funnel viscosity. Report the temperature of the sample in degrees F (C).



FIG. 2.1
MARSH FUNNEL AND CUP

DIRECT-INDICATING VISCOMETER

Description

2.6 Direct-indicating viscometers are rotational type instruments powered by means of an electric motor or a hand crank. Mud is contained in the annular space between two cylinders. The outer cylinder or rotor sleeve is driven at a constant rotational velocity. The rotation of the rotor sleeve in the mud produces a torque on the inner cylinder or bob. A torsion spring restrains the movement. A dial attached to the bob indicates displacement of the bob. Instrument constants have been so adjusted that plastic viscosity and yield point are obtained by using readings from rotor sleeve speeds of 300 and 600 rpm. The apparent viscosity in centipoises equals the 600-rpm reading divided by 2. The following are three types of viscometers used in testing drilling fluids:

- a. The 12-volt, motor-driven instrument (Fig. 2.2) has output speeds of 300 and 600 rpm. A governor-release switch permits high intensity shearing before measurement, and a knurled hand-wheel is used to determine gel strengths.
- b. The hand-crank instrument (Fig. 2.3) is similar in design to the 12-volt unit. A hand-crank is used to obtain rotational speeds of 300 and 600 rpm and a knob on the hub of the speed-change lever is used to determine gel strength.
- c. The 115-volt instrument (Fig. 2.4) is powered by a two-speed synchronous motor to obtain rotational speeds of 3, 6, 100, 200, 300, and 600 rpm. The 3-rpm speed is used for gel-strength determination.

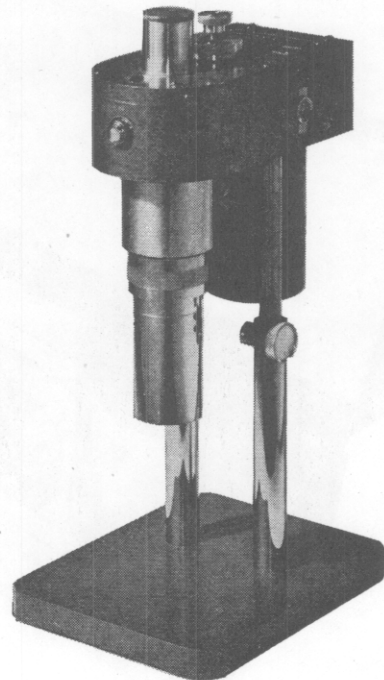


FIG. 2.2
12-VOLT MOTOR-DRIVEN VISCOMETER

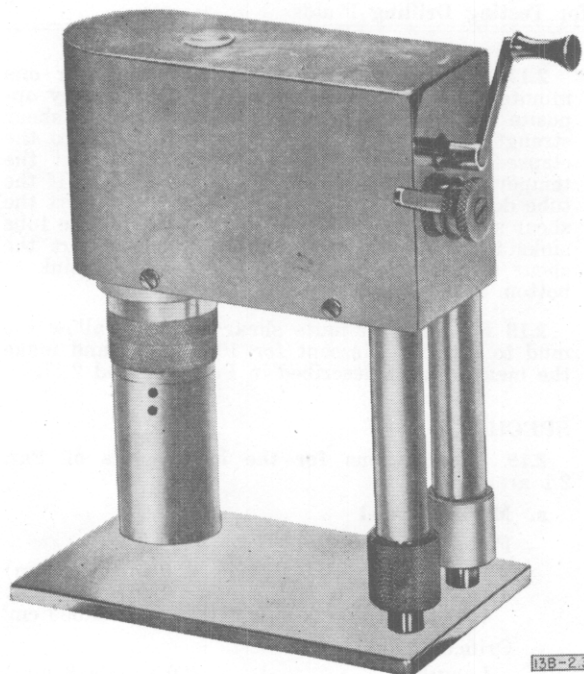


FIG. 2.3
HAND-CRANK VISCOMETER

Procedure: Plastic Viscosity and Yield Point

2.7 Place a sample in a suitable container and immerse the rotor sleeve exactly to the scribed line. Measurements in the field should be made with minimum delay (within five minutes, if possible) and at a temperature as near as practical to that of the mud at the place of sampling (not to differ more than 10 F, 6C). The place of sampling should be stated on the report.

2.8 With the sleeve rotating at 600 rpm, wait for the dial reading to reach a steady value (the time required is dependent on the mud characteristics). Record the dial reading for 600 rpm.

2.9 Shift to 300 rpm and wait for the dial reading to come to a steady value. Record the dial reading for 300 rpm.

2.10 The plastic viscosity (PV) in centipoises equals the 600-rpm reading minus the 300-rpm reading. The yield point (YP) in lb per 100 sq ft equals the 300-rpm reading minus the plastic viscosity.* Report the temperature of the sample in degrees F (C). The apparent viscosity in centipoises equals the 600-rpm reading divided by 2.

Procedure: Gel Strength

2.11 Place the mud sample in position as in Par. 2.7. Stir at high speed for 10 seconds.

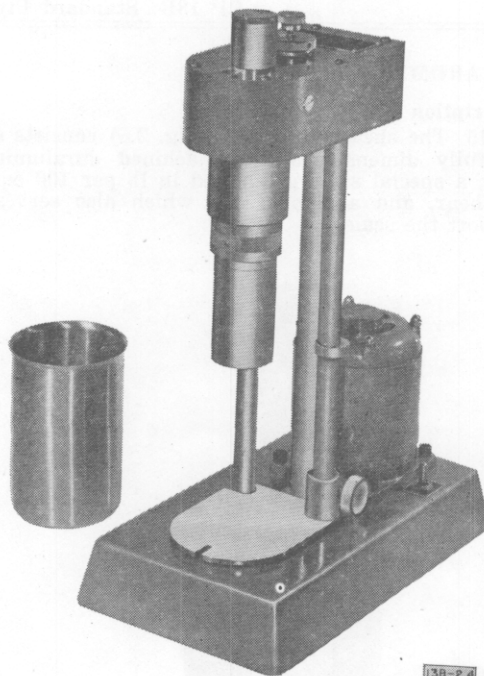


FIG. 2.4
115-VOLT MOTOR-DRIVEN VISCOMETER

2.12 Allow the mud to stand undisturbed for 10 seconds. Then slowly and steadily turn the handwheel in the direction to produce a positive dial reading. The maximum reading is the initial gel strength in lb per 100 sq ft.* For instruments having a 3-rpm speed, the maximum reading attained after starting rotation at 3 rpm is the initial gel strength. Report the temperature of the sample in degrees F (C).

2.13 Restir the mud at high speed for 10 seconds and then wait 10 minutes. Repeat the measurement as before and report the maximum reading as the 10-minute gel strength in lb per 100 sq ft.* Report the temperature of the sample in degrees F (C).

Calibration

2.14 Operation of the instrument as a direct-indicating viscometer depends upon maintenance of the correct spring tension and the correct speed of sleeve rotation. Procedures are available from the manufacturer to test spring tension and speed. Generally, however, a simpler test of reliability of the instrument can be made by measuring a Newtonian liquid of known viscosity (e.g., silicone liquids, sugar solutions, or petroleum oils of known viscosities at specified temperatures).

*The yield point or gel strength in kg per m² is calculated by multiplying lb per 100 sq ft by 0.05.

SHEAROMETER

Description

2.15 The shearometer (see Fig. 2.5) consists of a carefully dimensioned and machined duraluminum tube, a special scale graduated in lb per 100 sq ft* of shear, and a sample cup which also serves to support the scale.

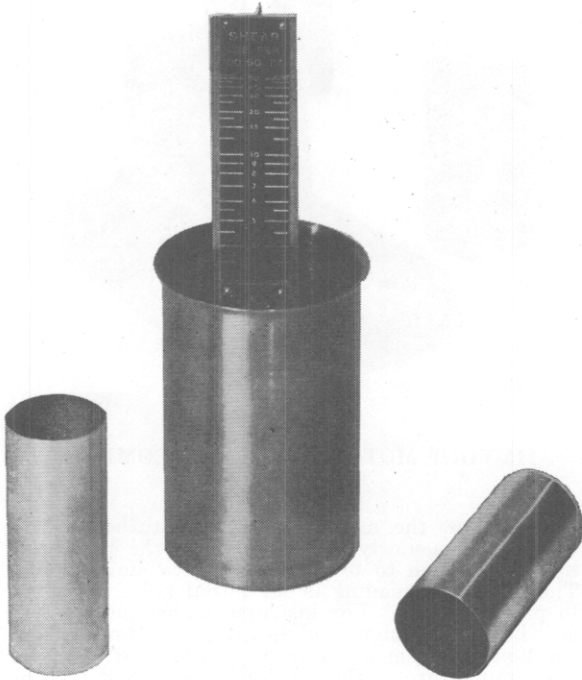


FIG. 2.5
SHEAROMETER SET

Procedure

2.16 Pour a mud sample into the clean, dry sample cup to the scribed line. The tube should be washed and dried just prior to use. For initial shear strength, quickly lower the tube over the scale support and place it on the quiescent surface of the mud. Allow it to sink vertically, guided by the fingers if necessary. With a stop watch, measure time from the instant the tube is released.

*The shear strength in kg per m² is calculated by multiplying lb per 100 sq ft by 0.05.

2.17 After permitting the tube to sink for one minute, report the reading on the scale directly opposite the top of the shearometer tube as the shear strength in lb per 100 sq ft* corresponding to the elapsed time before the measurement. Report the temperature of the sample in degrees F (C). If the tube does not penetrate the mud surface, report the shear strength as "too high to measure." If the tube sinks to bottom in 60 seconds or less, report the shear strength as zero; and show time to sink to bottom as a superscript.

2.18 For the 10-minute shear strength, allow the mud to remain quiescent for 10 minutes and make the measurement described in Par. 2.16 and 2.17.

SPECIFICATIONS

2.19 Specifications for the instruments of Par. 2.1 are:

a. Marsh Funnel

Funnel Cone

- Length12.0 in. (305 mm)
- Diameter6.0 in. (152 mm)
- Capacity to bottom of screen.....1,500 cm³

Orifice

- Length2.0 in. (50.8 mm)
- Inside diameter3/16 in. (4.76 mm)

Screen

Has 1/16-in. (1.6 mm) openings and is fixed at a level 3/4 in. (19.0 mm) below top of funnel.

b. Direct-Indicating Viscometer

Rotor Sleeve

- Inside diameter1.450 in. (36.83 mm)
- Total length3.425 in. (87.00 mm)
- Scribed line 2.30 in. (58.4 mm) above bottom.
- Two rows of 1/8-in. (3.18 mm) holes, spaced 120 deg (2.09 radians) apart, around rotor sleeve just below scribed line.

Bob

- Diameter1.358 in. (34.49 mm)
- Cylinder length1.496 in. (38.00 mm)
- Bob is closed with a flat base and tapered top.

Rotor Speeds

- High speed600 rpm
- Low speed300 rpm

c. Shearometer

Tube

- MaterialDuraluminum
- Length3.5 in. (89 mm)
- Inside diameter1.4 in. (36 mm)
- Weight5.0g

SECTION 3 FILTRATION

LOW TEMPERATURE TEST

Equipment

3.1 The filtration and wall-building characteristics of mud are determined by means of a filter press. Two standard makes are illustrated in Fig. 3.1 and 3.2. Essentially, the filter press consists of a cylindrical mud cell having an inside diameter of 3 in. (76.2 mm), and a height of at least 2½ in. (64 mm). This chamber is made of materials resistant to strongly alkaline solutions, and is so fitted that a pressure medium can be conveniently admitted into, and bled from, the top. Arrangement is also such that a sheet of 9-cm filter paper can be placed in the bottom of the chamber just above a suitable support. The filtration area is 7.1 ± 0.1 sq in. (45.8 ± 0.6 cm²). Below the support is a drain tube for discharging the filtrate into a graduated cylinder. Sealing is accomplished with gaskets. The entire assembly is supported by a stand.

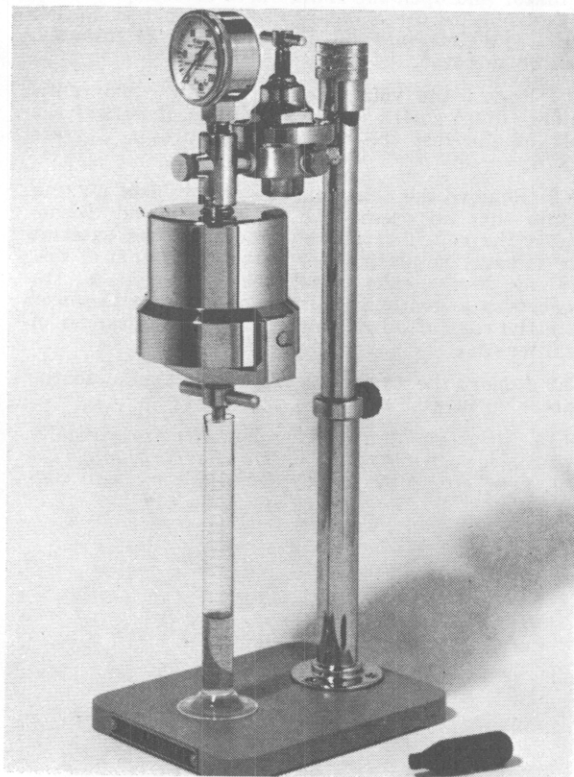
3.2 Pressure can be applied with any nonhazardous fluid medium, either gas or liquid. Presses are equipped with pressure regulators and can be obtained with portable pressure cylinders, midget pressure cartridges, or means for utilizing hydraulic pressure.

3.3 To obtain correlative results, one thickness of the proper 9-cm filter paper, Whatman No. 50, S&S No. 576, or equivalent, must be used.

Procedure

3.4 Be sure each part of the cell, particularly the screen, is clean and dry, and that the gaskets are not distorted or worn. Pour the sample of mud into the cell and complete the assembly.

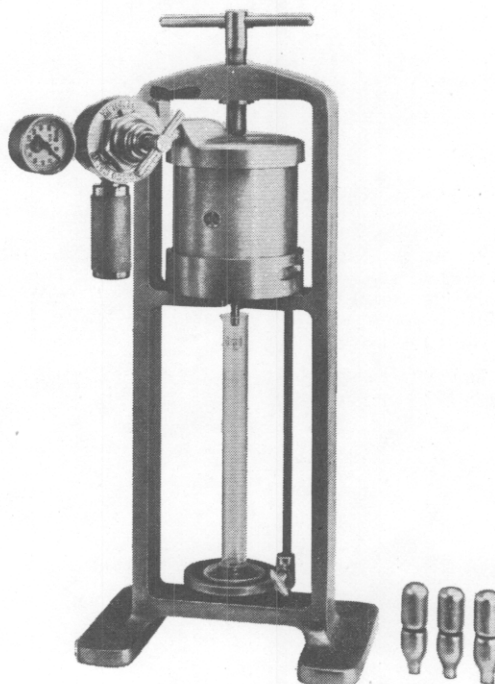
3.5 Place a dry graduated cylinder under the drain tube to receive the filtrate. Close the relief valve and adjust the regulator so that a pressure of 100 ± 5 psi (7.03 ± 0.35 kgf per cm²) is applied in 30 seconds or less. The test period begins at the time of pressure application.



With CO₂ Cartridge Pressurization
FIG. 3.1
FILTER PRESS



With Nitrogen Pressurization



With CO₂ Cartridge Pressurization

FIG. 3.2
FILTER PRESS

3.6 At the end of 30 minutes, measure the volume of filtrate. Shut off the flow through the pressure regulator and open the relief valve carefully. It may be desirable to use a one-hour filtration test for oil muds. The time interval, if other than 30 minutes, shall be reported.

3.7 Report the volume of filtrate in cubic centimeters (to 0.1 cm³) as the API filtrate. Report at the start of the test the mud temperature in degrees F (C).

3.8 Remove the cell from the frame, first making certain that all pressure has been relieved. Disassemble the cell, discard the mud, and use extreme care to save the filter paper with a minimum of disturbance to the cake. Wash the filter cake on the paper with a gentle stream of water or with diesel oil in the case of oil muds. Measure the thickness of the filter cake.

3.9 Report the thickness of the filter cake to the nearest $\frac{1}{32}$ in. (0.8 mm).

3.10 Although standard descriptions are virtually impossible, such notations as *hard*, *soft*, *tough*, *rubbery*, *firm*, *etc.*, may convey some idea of cake consistency.

SECTION 4 SAND

Equipment

4.1 Sand content of mud is estimated by the use of a sand-screen set (see Fig. 4.1). The set consists of a 200-mesh sieve $2\frac{1}{2}$ in. (63.5 mm) in diameter, a funnel to fit the screen, and a glass measuring tube. The measuring tube is marked for the volume of mud to be added in order to read directly the percentage of sand in the bottom of the tube, which is graduated from 0 to 20 percent.

Procedure

4.2 Fill the glass measuring tube to the indicated mark with mud. Add water to the next mark. Close the mouth of the tube and shake vigorously.

4.3 Pour the mixture onto the clean, wet screen. Discard the liquid passing through the screen. Add more water to the tube, shake, and again pour onto the screen. Repeat until the wash water passes through clear. Wash the sand retained on the screen to free it of any remaining mud.

4.4 Fit the funnel upside down over the top of the screen. Slowly invert the assembly and insert the tip of the funnel into the mouth of the glass tube. Wash the sand into the tube by playing a fine spray of water through the screen. Allow the sand to settle. From the graduations on the tube read the volume percent of the sand.

4.5 Report the sand content of the mud in volume percent. Report the source of the mud sample, i.e., above shaker, suction, pit, etc. Coarse solids other than sand will be retained on the screen (e.g., lost circulation materials) and the presence of such solids should be noted.

NOTE: Use diesel oil instead of water for oil muds.



FIG. 4.1
SAND-CONTENT SET

SECTION 6

pH

Equipment

6.1 Two methods for measuring the pH of drilling mud are used. These are: (1) a modified colorimetric method, using paper test strips; and (2) the electrometric method, using the glass electrode. The paper-strip method may not be reliable if the salt concentration of the sample is high. The electrometric method is subject to error in solutions containing high concentrations of sodium ions, unless a special glass electrode is used, or unless suitable correction factors are applied in using the ordinary electrode. In addition, a temperature correction should be made in the electrometric method of measuring pH.

PAPER TEST STRIPS**Description**

6.2 The test paper is impregnated with dyes of such nature that the color is dependent upon the pH of the medium in which the paper is placed. A standard color chart is supplied for comparison with the test strip. Test papers are available in a wide-range type, which permits estimation of pH to 0.5 unit, and in narrow-range papers, with which the pH can be estimated to 0.2 unit.

Procedure

6.3 Place a 1-in. (25 mm) strip of indicator paper on the surface of the mud and allow it to remain until the liquid has wetted the surface of the paper and the color has stabilized (usually not more than 30 seconds).

6.4 Compare the color of the upper side of the paper (which has not been in contact with the mud solids) with the color standards provided with the test strip and estimate the mud pH.

6.5 Report the mud pH to the nearest 0.5 or 0.2 unit, depending upon the scale of the color chart for the test paper used.

GLASS-ELECTRODE pH METER**Description**

6.6 The glass-electrode pH meter consists of a glass-electrode system, an electronic amplifier, and a meter calibrated in pH units. The electrode system is composed of: (1) the glass electrode, which consists of a thin-walled bulb made of special glass within which is sealed a suitable electrolyte and electrode; and (2) the reference electrode, which is a saturated calomel cell. Electrical connection with the mud is established through a saturated solution of potassium chloride contained in a tube surrounding the calomel cell. The electrical potential generated in the glass-electrode system by the hydrogen ions in the drilling mud is amplified and operates the calibrated meter which indicates pH.

Procedure

6.7 Make the necessary adjustments to put the amplifier into operation and standardize the meter with suitable buffer solutions, according to directions supplied with the instrument.

6.8 Wash the tips of the electrodes, gently wipe dry, and insert them into the mud contained in a small glass vessel. Stir the mud about the electrodes by rotating the container.

6.9 Measure the mud pH according to the directions supplied with the instrument. After the meter reading becomes constant, which may require from 30 seconds to several minutes, record the pH.

6.10 Report the pH of the mud to the nearest 0.1 unit.

APPENDIX B

Federation of Piling Specialists -- Specification for Cast-in-Place Concrete Diaphragm Walling

Reprinted from a reprint from Ground Engineering, Vol. 6, No. 4, July 1973 of Specification for Cast-in-Place Concrete Diaphragm Walling by permission of the Federation of Piling Specialists.



Specification for Cast in Place Concrete Diaphragm Walling

Design

1. All work shall be carried out in accordance with good engineering practice and related to an adequate site investigation. The recommendations of the codes of practice CECP. No. 2: EARTH RETAINING STRUCTURES and CP.2004: FOUNDATIONS, shall be followed in so far as they are applicable to the construction of diaphragm walling.

(Note for guidance: The site investigation should be designed to give the information required for the design of diaphragm walling and needs to be fully comprehensive).

(Note for guidance: All references to Codes of Practice and British Standards shall refer to the latest edition in print).

2. The maximum compressive stress in the concrete of a wall shall be that given in CP.114: REINFORCED CONCRETE, for the appropriate conditions of use (or CP.110: STRUCTURAL USE OF CONCRETE). Except with the approval of the Engineer, permanent direct compressive stress shall be limited to a value of less than 7.0 N/mm² and compressive stress due to combined bending and direct stress to 9.0 N/mm².

3. Steel reinforcement for use in diaphragm walls shall be designed in accordance with the recommendations of CP.114: REINFORCED CONCRETE (or CP.110: STRUCTURAL USE OF CONCRETE) except that if using deformed bars the increases allowed in permissible bond stress in the Codes may not be applied but a 10 per cent increase over equivalent plain bars may be allowed.

4. The minimum cover to the main bars of steel reinforcement is to be 75 mm and the minimum clear spacing between main bars shall be 100 mm.

5. The design of the wall shall take account of the stresses due to active and passive soil pressures, due to surcharges, due to the combined horizontal and vertical forces induced by ground anchors used to maintain stability of the wall, due to retained ground water where applicable, and due to the worst conditions arising in the stages of subsequent excavation, propping and anchoring and to other special conditions. The design shall take into account both the permanent and temporary states of stress which will arise during the life of the structure.

6. The assumptions made and the factors of safety

which have been used in the design of the wall are to be stated.

7. All the imposed loads including those arising from the soil taken into account in the design are to be clearly stated.

8. The design shall take into account the deflection of the wall. Consideration shall be given to the need for any underpinning, grouting or soil treatment required to maintain the stability of adjacent foundations during the construction and exposure of the diaphragm wall.

9. Walls constructed by diaphragm wall techniques may be used for the retention of earth, the provision of reaction to applied lateral forces and the support of vertical loads simultaneously, provided that evidence can be produced by testing or otherwise, that such loads can be supported in the ground conditions known to exist on the site.

(Note for guidance: Friction or adhesion on that part of any wall above the related main excavation level or where the contact between the soil and the wall face could be lost as a result of deflection should not be taken as contributing to the capacity of the wall to carry imposed structural loads).

10. All the panels in any continuous length of wall should be designed according to compatible principles.

(Note for guidance: For example, the use of panels spanning horizontally between alternate cantilever panels is generally to be avoided unless shear transference can be verified).

11. The thickness of wall and the provisional panel lengths required are to be as detailed on the drawings. Provision is to be made for all recesses, anchorage positions, inserts and special details as shown on the drawings, and steel reinforcement shall be fixed to accommodate these items.

(Note for guidance: Where close to adjacent structures the soil retained by a diaphragm wall is subject to surcharge loads, careful consideration should be given to the use of reduced panel lengths in order to increase the factor of safety and decrease the possibility of trench wall failures. The maximum panel excavation length acceptable should be stated by the Engineer in the tender documents. The minimum panel excavation length required to accommodate the excavating equipment should be stated by the Specialist Contractor with the tender. The effects of deflection of the wall on both adjacent structures and services must be considered).

(Note for guidance: Where boxes are required in a wall for the formation of recesses, consideration must be given to the effect of the boxes on the strength of the wall, the placing of rein-

flow of concrete during placing. Boxes must be positioned so as to prevent concrete from being forced through the boxes, and the effect of the boxes on the flow of concrete into the panel excavation with a clearance).

12. Guide walls are to be designed with continuous reinforcement and are to be constructed to comply with the drawings. They are to be cast on and against firm ground or alternatively, where it is desired to shutter both faces of the guide wall, all back-filling behind the wall is to be done using an approved lean mix concrete unless otherwise agreed by the Engineer.

(Note for guidance: The top of the guide wall should, preferably, be not less than 1.5 m above any standing ground water level, and guide walls must be capable of being constructed in the dry).

Materials

Concrete

13. Cement shall be Ordinary Portland cement complying with BS.12 or Sulphate Resisting cement complying with BS.4027.

14. Aggregates shall comply with BS.882. The shell content shall not be greater than the limits given in the table:

Nominal max. size of aggregate	Shell content max. per cent
40 mm	2
20 mm	5
Sand	30

The chloride ion content of the aggregate shall be such that the chloride ion content of the mixed concrete shall not exceed 1.2 per cent for unreinforced concrete or 0.2 per cent for reinforced or prestressed concrete.

(Note for guidance: Aggregate of a size in excess of 20 mm will normally only be used in non-reinforced concrete diaphragm walls).

15. Clean water, free from acids and other impurities and in accordance with the BS.3148 shall be used in the making of concrete.

16. The slump of the concrete shall normally be in accordance with the following standard:

Minimum slump	150 mm
Range	150 mm to collapse

Unless otherwise approved by the Engineer, a minimum cement content of 400 kg/m³ is to be employed in making concrete which is to be placed by tremie methods under a bentonite slurry, in accordance with CP.2004.

The concrete mix shall flow easily in the tremie pipe and shall be designed to give a dense concrete when placed by the tremie method.

Aggregates shall comply with gradings of Zones 2, 3 or 4 of BS.882 and shall preferably be of naturally rounded gravel and sand.

Water cement ratio shall not exceed 0.60.

(Note for guidance: The desirable range of slump is from 175 mm to 200 mm).

17. Any additive used in the concrete must be stated.

18. Ready mixed concrete may be used and shall comply with BS.1926.

(Note for guidance: BRMCA Reprint 71-1: "The Specification and Use of Ready Mixed Concrete for Cast in Place Piling" gives some useful information regarding the use of ready mixed concrete which is to be placed through a tremie pipe).

19. Test cubes shall be prepared and tested in accordance with BS.1881 as required in the contract.

(Note for guidance: Opinions vary as to the number of test cubes which should be required on a diaphragm wall contract but it is suggested that 4 cubes be taken for every panel).

20. In cold weather, ice and snow shall be excluded from the materials used in the manufacture of concrete for use in diaphragm walls.

Aggregates must not be heated to more than 38 deg. C, and the concrete when placed must have a minimum temperature of 5 deg. C.

Reinforcement

21. All reinforcing steel shall be in accordance with the appropriate British Standard unless otherwise agreed.

22. The welding of steel reinforcement required in the works shall be carried out only by techniques which can be shown to maintain the full strength of the structural reinforcement.

(Note for guidance: The drawings should show all the steel reinforcement necessary including that required for lifting stiffening and splicing. They should also show clearly the type of steel required. Mild Steel and High Tensile Steel of similar diameters and type should be avoided. The drawings should also indicate clearly the orientation of the cage in relation to the earth face and the excavated face. It may be advisable to leave the preparation of detail drawings of reinforcement, which should take into account all the tolerances stated in Clause 34, until after acceptance of tender when actual methods of construction are known).

23. The steel reinforcing cage shall be clearly marked to indicate its correct orientation for proper insertion into the trench.

Bentonite

24. Bentonite, as supplied to the site and prior to mixing, shall be in accordance with specification No. DFCP.4 of the Oil Companies Materials Association, London.

A certificate is to be obtained by the Specialist Contractor from the manufacturer of the bentonite powder, stating from which manufacturer's consignment the material delivered to site has been taken, and showing properties of the consignment as determined by the manufacturer. This certificate shall be made available to the Engineer on request.

(Note for guidance: The properties which should normally be given by the manufacturer are the apparent viscosity range (centipoises) and the gel strength range (N/m²) for solids in water).

25. The bentonite powder shall be mixed thoroughly with clean fresh water. The percentage of bentonite used to make the slurry shall be such as to maintain the stability of the trench excavation.

(Note for guidance: In the case of certain estuarine clays of very low strength, it may not be possible to produce a slurry which alone will maintain the stability of trenches. Care also needs to be taken in very permeable ground).

26. Control tests are to be carried out on the bentonite slurry using suitable apparatus, to determine the following parameters:

(a) Freshly mixed bentonite slurry

The density of the freshly mixed bentonite slurry shall be measured daily as a check on the quality of the slurry being formed. The measuring device is to be calibrated to read within ± 0.005 g/ml.

(Note for guidance: A satisfactory way of measuring the density of a bentonite slurry is by means of a mud balance.

The following table shows the relationship between the concentration, expressed as a percentage by weight, and the density:

Concentration per cent	Density g/ml
3	1.017
4	1.023
5	1.028
6	1.034

These figures relate to a typical bentonite material of British origin).

(b) Bentonite slurry supplied to trench excavation

In average soil conditions the following tests shall be applied to the bentonite supplied to the trench, and the results shall generally be within the ranges stated in the table below:

Item to be measured	Range of results at 20 deg C	Test method
Density	Less than 1.10 g/ml	Mud density balance
Viscosity	30-90 seconds	Marsh Cone method
Shear strength (10 min gel strength)	1.4 to 10 N/m ²	Shearometer
pH	9.5-12	pH indicator paper strips

Tests to determine density, viscosity, shear strength and pH value shall be carried out initially until a consistent working pattern has been established, taking into account the mixing process, any blending of freshly mixed bentonite slurry and previously used bentonite slurry, and any process which may be employed to remove impurities from previously used bentonite slurry.

When the results show consistent behaviour, the tests for shear strength and pH value may be discontinued, and tests to determine density and viscosity only shall be carried out as agreed with the Engineer. In the event of a change in the established working pattern, the additional tests for shear strength and pH value shall be reintroduced for a period if required by the Engineer.

(Note for guidance: Freshly mixed bentonite slurry should comply with the requirements of the table consistently, provided a normal concentration has been selected. Where bentonite slurry is used once only and then discarded, the tests set out in the table should not be necessary beyond a short initial period, unless some alteration is made to the concentration or mixing procedure.

Where bentonite slurry is re-used, and possibly blended with freshly mixed slurry, or has chemical additions made to preserve its properties, there will be a need for routine checking throughout the work, particularly in regard to the tests for density and viscosity. The frequency of testing may initially need to be on a panel by panel basis where bentonite slurry becomes heavily contaminated during its first use (eg fine sand soil conditions) and may in other cases (eg mainly clay soil conditions) be on a daily basis where contamination is slight. Subsequent frequency will need to be agreed between the Engineer and Specialist Contractor in the light of the test results obtained.

In those cases where a mechanical process is employed to remove contaminating solids from the slurry, the frequency of testing will depend on the circumstances and the equipment employed. The Specialist Contractor should indicate to the Engineer prior to the commencement of the contract, that he intends to employ such a method, and tests should be carried out as for re-used and blended slurries).

(c) Bentonite slurry in trench prior to placing concrete

Prior to placing concrete in any panel, the Specialist Contractor shall ensure that heavily contaminated bentonite slurry, which could impair the free flow of concrete from the tremie pipe, has not accumulated in the bottom of the trench. The proposed method for checking this item is to be stated with the tender, and is to be agreed with the Engineer prior to the commencement of the contract. If the bentonite slurry is found to exhibit properties outside the agreed appropriate range, then it shall be modified or replaced until the required agreed condition is achieved.

(Note for guidance: One method of identifying contaminated bentonite slurry is to take a sample of the slurry from near the bottom of the trench excavation (say about 0.2 m above the base of the trench) and to carry out a density test on this using a Mud Balance. Where this method is employed, the density determined should not be greater than 1.3 g/ml to enable satisfactory concrete placing).

(Note for guidance: Details of apparatus and test methods referred to in Clause 26 may be obtained from the following publication:

Recommended Practice: Standard by American Petroleum Institute, New York City, 1957. Ref. API RP29. Sections I, II and VI relate to the above mentioned tests).

(Note for guidance: The result of tests on bentonite slurry referred to in Clause 26 should be related to a temperature of 20 deg C approximately).

27. The temperature of the water used in mixing bentonite slurry, and of the slurry supplied to the trench excavation, is to be not less than 5 deg C.

28. During construction the level of bentonite slurry in the trench shall be maintained within the depth of the guide walls, and at a level not less than 1.0 m above the level of external standing ground water.

29. In the event of a sudden loss of bentonite slurry, the trench shall be backfilled without delay and the instructions of the Engineer shall be obtained.

30. Where saline or chemically contaminated ground water occurs, special measures shall be taken as required by the Engineer to modify the bentonite slurry.

(Note for guidance: The modification required depends on the nature of the contamination. In saline conditions it is frequently necessary to ensure that the bentonite is fully hydrated in fresh water before supplying it to the trench).

31. All reasonable steps shall be taken to prevent spillage of bentonite slurry on the site away from the immediate vicinity of the wall. Discarded bentonite slurry which has been pumped from the trench is to be removed promptly from the site.

Construction

32. The proposed method of excavation is to be stated by the Specialist Contractor at the time of tendering

(Note for guidance: The use of chiselling to overcome obstructions may cause difficulty in maintaining the stability of the trench and it is therefore an item to be treated with caution. It should also be allowed for in preparing the Bill of Quantities, where the possibility of its use is apparent).

33. Steps are to be taken to avoid damage to panels which have recently been cast. In deciding the sequence of panel construction, the Specialist Contractor shall take this into account.

(Note for guidance: If the Engineer requires some specific sequence of panel construction, this should be made known to the Specialist Contractor in the tender documents).

34. The construction shall be carried out in accordance with the following normal tolerances:

The finished face of the guide wall towards the trench and on the side of the trench nearest to any subsequent main excavation shall be vertical and shall represent the reference line. There shall be no ridges or abrupt changes on the face and its variation from a straight line or specified profile shall not exceed ± 15 mm in 3 m.

From this face the minimum clear distance between the faces of the guide walls shall be the specified diaphragm wall thickness plus 25 mm, and the maximum distance shall be the specified diaphragm wall thickness plus 50 mm.

(Note for guidance: Where curved walls are to be constructed, the clearance distance between the guide wall faces may have to be increased).

The wall face to be exposed and the ends of panels shall be vertical to within a tolerance of 1 : 80. In addition to this tolerance, a tolerance of 100 mm shall be allowed for protrusions resulting from irregularities in the ground as excavated, beyond the general face of the wall.

(Note for guidance: It should be borne in mind that, within the limits of the verticality tolerance specified, a wall panel may show an angular deviation at any level when viewed in plan. Such a deviation is usually only important in regard to the exposed face of the wall and will be a function of depth.

Tolerances are not normally necessary for this item, but where they are considered to be essential they should be agreed with the Engineer, taking into account the above factors, the panel length and the panel position, in relation to the particular site circumstances.)

(Note for guidance: Designers should have in mind that diaphragm walls normally consist of a series of panels and, especially in the case of deep walls, the wall thickness should be carefully considered in relation to the permitted tolerances for excavation).

(Note for guidance: The protrusion tolerance of 100 mm refers to homogeneous clays. In highly fissured clays, sands, gravels or loose or soft grounds the tolerance should be increased. Unless this tolerance has been taken into account in the design and setting out, provision needs to be made in preparing the Bill of Quantities for any cutting back required).

Where recesses are to be formed by inserts in the wall, they shall be positioned within vertical and horizontal tolerances of 150 mm.

(Note for guidance: Horizontal inserts cannot be placed continuously between panels in normal diaphragm wall construction, but must be curtailed at the end of the reinforcing cage).

The tolerances in positioning reinforcement shall be as follows:

Longitudinal tolerance of cage head at the top of the guide wall and measured along the trench: ± 75 mm.

Vertical tolerance at cage head in relation to top of guide wall: ± 50 mm.

The reinforcement shall be maintained in position during the casting of each panel.

(Note for guidance: In the design of diaphragm walls, the distance between reinforcement cages in adjacent panels must take into account both the longitudinal positional tolerance and the shape of the stop end in relation to the shape of the cage).

35. Stop ends, inserted prior to placing concrete in a panel shall be clean and have a smooth regular surface. They shall be adequately restrained to prevent horizontal movement during concreting.

36. Safety precautions shall be taken throughout the construction of diaphragm walls in accordance with the statutory requirements listed in CP.2004:
FOUNDATIONS.

Concrete placing

37. Concrete shall be placed continuously by one or more tremie pipes, and care shall be taken during

placing to avoid contamination of the concrete. Where two or more pipes are used in the same panel simultaneously, care shall be taken to ensure that the concrete level at each pipe position is maintained nearly equal.

38. The tremie pipe shall be clean, watertight and of adequate diameter to allow the free flow of concrete. The tremie shall extend to the bottom of the trench excavation prior to the commencement of concrete pouring, and care shall be taken to ensure that all bentonite slurry is expelled from the tube during the initial charging process. Sufficient embedment of the tremie pipe in concrete shall be maintained throughout concrete pouring to prevent re-entry of bentonite slurry into the pipe.

39. The concrete pour for any diaphragm wall panel shall be completed in such a manner and within such time that the concrete above the foot of the tremie remains workable until the casting of the panel is complete.

40. The effective trimmed final wall level shall generally be taken as 250 mm below the top of the guide wall when concrete is cast to the top of the trench.

For trimmed final wall levels below this level the tolerance of the cast concrete profile shall be a minimum of 150 mm and a maximum of 600 mm above the specified wall level plus an additional allowance of 150 mm over the maximum tolerance for each one metre of final wall depth specified below the top of the guide wall.

(Note for guidance: Special problems occur with deep specified final wall levels, when it becomes difficult to locate adjacent panels precisely and when backfill over previously completed panels cannot be retained without special measures such as backfilling above final wall level using lean concrete mixes. Such circumstances require appropriate items to be included in the Bill of Quantities).

41. The extraction of stop ends shall be carried out at such a time and in such a manner as to avoid causing damage to concrete placed against it.

42. The method of forming joints and the equipment used shall be such that all solids are removed from the end of the adjacent panel by the excavating equipment. The Specialist Contractor shall be responsible for the repair of any joint where, on full exposure of the wall, visible water leaks resulting from faulty materials or workmanship are found.

(Note for guidance: Seepage which may result from differential wall deflections or the installation of anchor points, are not considered to be included under this item. A provisional item should be included in the Bill of Quantities to allow for any special measures necessary to deal with such seepages).

Records

43. The following records shall be kept for each panel completed:

Panel number
Top of guide wall level
Bottom of guide wall level
Top level of wall as cast in relation to top of guide wall
Depth of base of panel from top of guide wall
Date panel excavated
Date panel concreted
Length of panel
Thickness of wall
Strata log
Cubes taken
Volume of concrete used
Details of steel reinforcement (cage type)
Details of any obstructions encountered and time spent in overcoming them.

CHAPTER 5 - INTERNAL BRACING

5.10 INTRODUCTION

This section discusses the design and construction aspects of internal bracing for lateral support of excavations. In cut-and-cover tunnel work, braces typically run cross lot without intermediate vertical support. Relatively wide excavations may require vertical support of the bracing member to decrease the bending moment caused by the dead load and to shorten the unsupported length. Also, the central portion of the invert slab may be poured first in order to use rakers (or inclined braces) from the lower levels.

Typical practice is to use a continuous horizontal wale to transfer loads from the ground support wall to the brace. Wale levels are normally set about 10 to 15 feet apart vertically, and brace positions are set at about 15 to 20 feet apart longitudinally along the cut. Recent excavation work in Washington used discontinuous wales to aid installation.

In general, internal bracing is most often used in relatively narrow cuts, where cross lot bracing can be used without intermediate support, or in wide excavations where suitable anchor strata are not available for tiebacks.

Representative examples of several internally braced walls follow in Figures 40, 41, and 42.

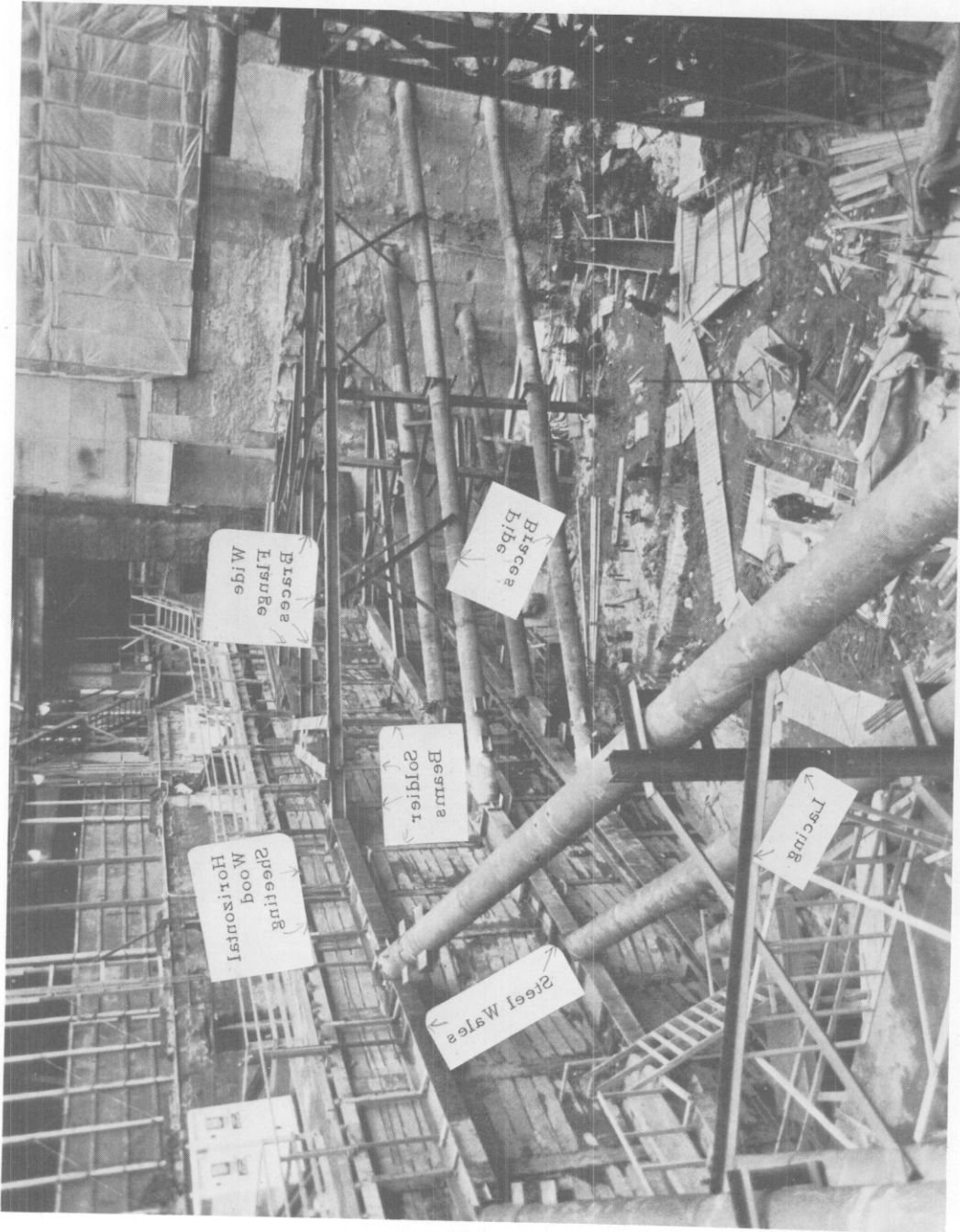
5.20 DESIGN CONSIDERATIONS

5.21 Types of Bracing

The most common sections used in the United States are wide flange or pipe. Concrete braces are uncommon, but their use has been reported for the subways in Cologne and Vienna (Haffen, 1973).

Conventional practice is to set the braces sequentially as the excavation proceeds. Excavation below the last placed bracing level is done with crawler equipment, usually front end loaders, feeding a clamshell. Caution is necessary because of possible damage to the braces.

A relatively recent support technique that has been used in Europe uses a waling slab constructed on the ground to support the walls. The waling slab later becomes the roof or intermediate floor of the structure. The excavation is carried out by mining beneath the "waling slab". The technique is also called "under the roof" construction. Examples of such projects are the Vienna Subway, House of Parliament



Note: Pipes and wide flange sections.

Figure 40. Corner bracing.
(Courtesy of Spencer, White, and Prentiss).



Note: Excavation in progress.

Figure 41. Internal bracing.
(Courtesy of Perini Corporation).



Note: Wood blocking between wale and steel sheet piling.

Figure 42. Internal bracing.
(Courtesy of Perini Corporation).

underground garage, and several building projects reported in the Diaphragm Wall Conference in London in 1974. Sverdrup and Parcel (1973) discuss the application of the technique in the Milan Subway.

5.22 Allowable Stresses

The controlling design criterion is the column-action combined axial and bending stress. In that regard, a pipe section is an extremely efficient section. Wide flange sections, especially when set with dead load bending against the weak axis (web horizontal) are relatively inefficient. However, this orientation is common because it is easily adapted to simple, economical connections at the wale.

AISC Code design stresses should be used for the completed braced wall at maximum depth. Temporary conditions arising from intermediate situations during the course of excavation will justify a 20 percent overstress above the AISC Code value.

5.23 Connections

Connections and details are critical elements in an internally braced excavation. Improper connections between strut and wale or between the wale and the support wall are perhaps the most frequent causes of difficulties in braced excavations. They can lead to twisting, buckling, and rotation of members. Figures 43, 44, and 45 present typical connection details.

5.24 Loads

Brace loading is computed on the basis of pressure diagrams presented in Volume II, "Design Fundamentals". Deep cuts in highly over-consolidated clays or in some clay chales should be designed and constructed with caution because of the expansion potential of these soils. A related phenomenon is lateral creep for tieback installation (see discussion in Chapter 2, Volume II).

5.30 INSTALLATION

5.31 General

Typically, the first step is to attach brackets to the wall for the purpose of supporting the wale. Measurements are taken to cut the bracing members to proper length, leaving a few inches of clear distance to facilitate placement. This extra space is taken up by plates and wedges when final connections are made.

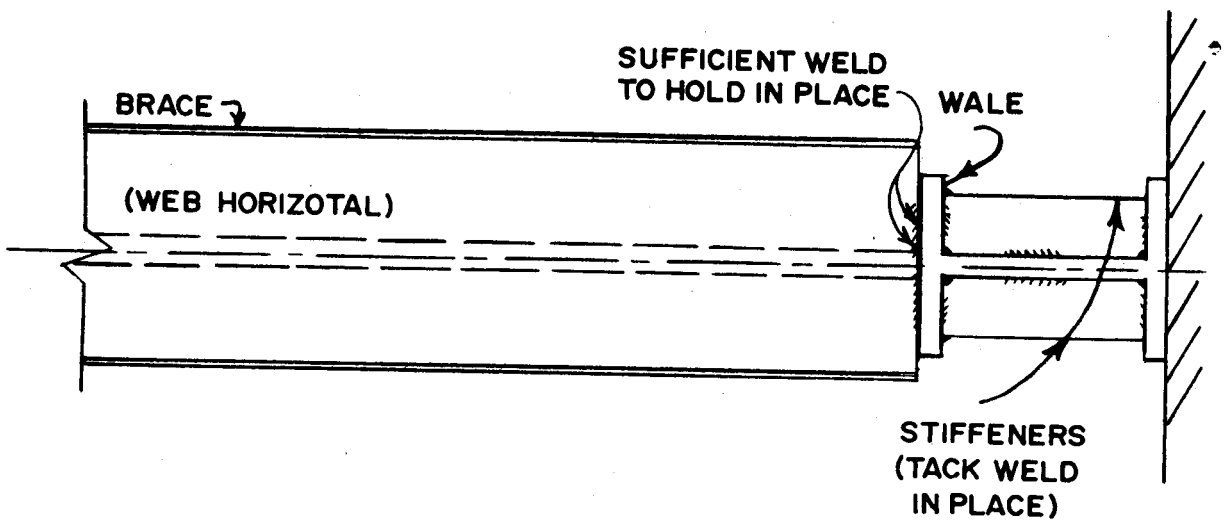
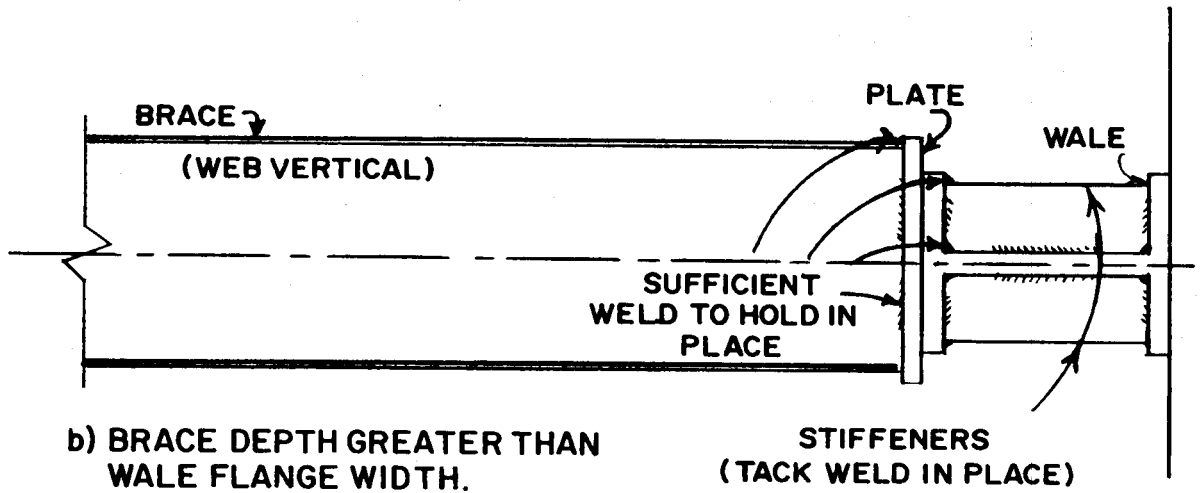
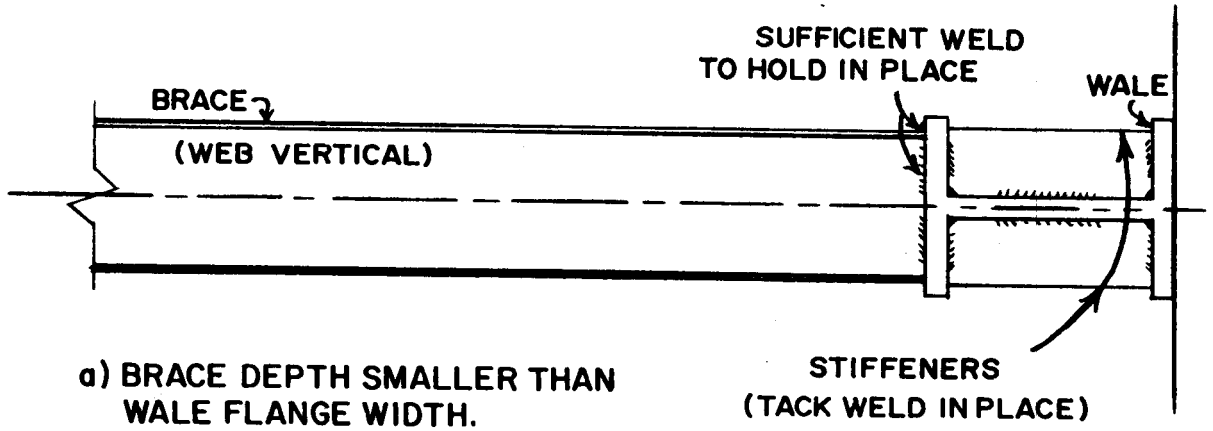


Figure 43. Typical detail for horizontal brace with brace web horizontal.



NOTE: DETAILS SIMILAR FOR HORIZONTAL BRACE.

Figure 44. Typical detail for horizontal brace with brace web vertical.

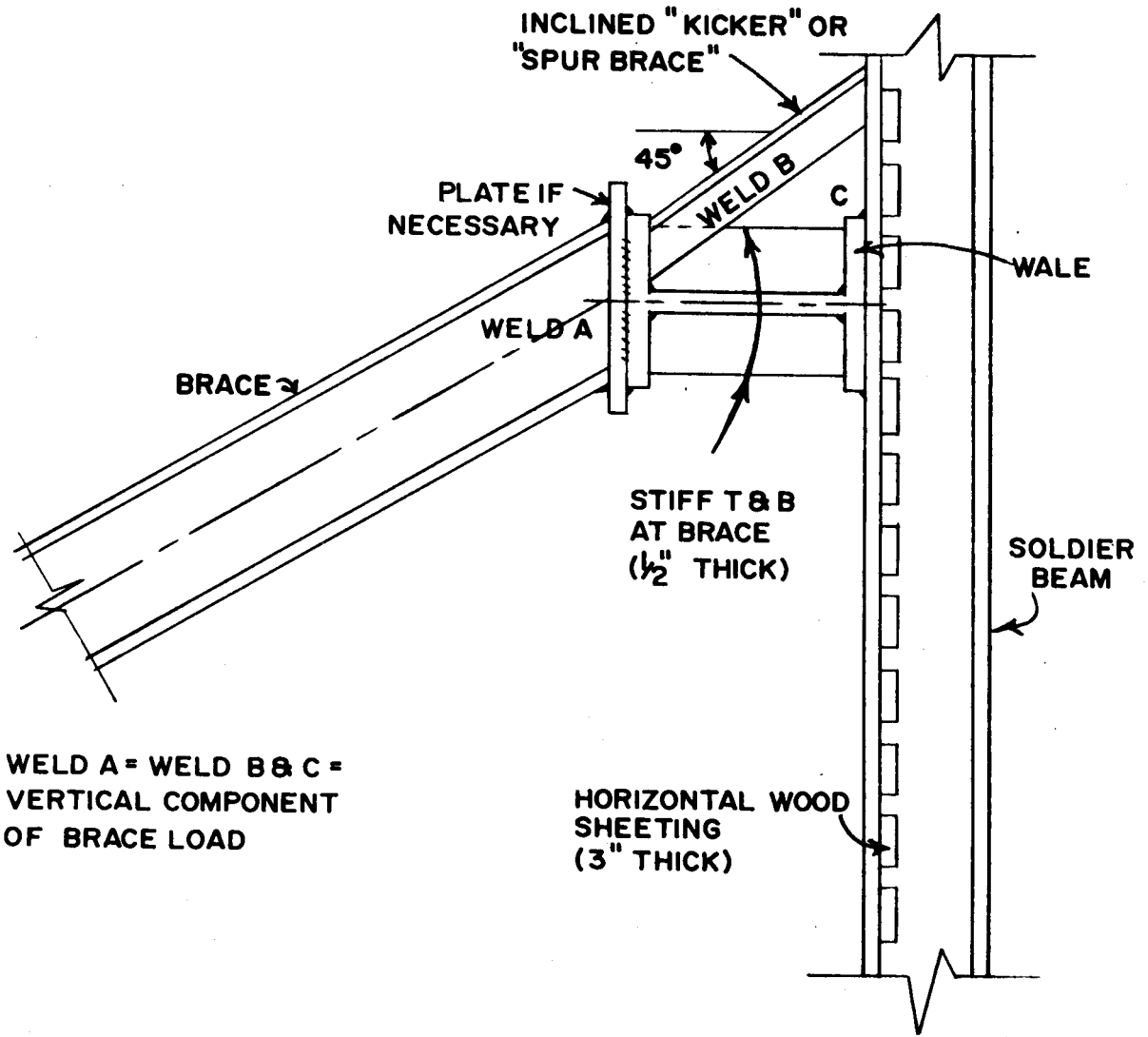


Figure 45. Typical connection for inclined brace and horizontal wale.

The space between the wale and the face of the support wall should always be taken up with appropriate blocking. In the case of soldier piles, a short piece of steel section is normally welded between the back flange of the wale and each individual soldier pile. In other cases, the space may be taken up with steel or hardwood wedges. Where there is concern about displacement in the adjoining ground, steel is preferred, and soft wood should not be used.

5.32 Installation without Preloading

In the case of cross-lot bracing, the member is welded at one end and blocked and shimmed at the opposite end. After the member is fitted in place, steel wedges and plates are tack welded to hold everything in place. In the case of an inclined brace (raker) the member is welded at one end (usually at the wale), and the reaction end may be cast into the concrete slab. An alternative procedure would be to weld at the wale end and use steel plates and wedges to make sure that the member is tight at the reaction end.

In cases where wall displacements must be held to a minimum, raker reactions against invert slabs are preferred to reactions against concrete deadmen. If deadmen are used, they should be used only in conjunction with preloading to remove slack and to assure that the reaction can be accepted without excessive movement.

The steel wedges that are driven between the member and the wale typically taper from about 1/8 inch thick at the knife edge to about 1/2 to 3/4 inch thick at the driving end. Common sizes are 14 to 20 inches long and about 2 inches wide.

5.33 Installation With Preloading

The procedure is to jack to the desired load, to make the connection, and then to remove the hydraulic jack. One procedure is first to jack to the desired load and then to drive wedges between the member and the wale until the jack load is down to essentially zero. This procedure effectively removes any slack or compression that may exist in the connection between the member and the wale. A second procedure is to weld the connection tight while maintaining the jack load, then to drop the pressure in the hydraulic jack, thus transferring the load through the connection to the wale. In the second method the connection undergoes compression following removal of the jacks.

The choice of method depends upon the relative magnitude of uncontrolled deformation that may take place in the second procedure.

In most instances the second procedure will be acceptable.

5.34 Preloading

Preloading of bracing is done for the purpose of removing elastic compression in the brace and the slack that may exist in the support wall between sheeting and wales, in connections of members, and between soil and wall. Preloading minimizes displacement of the adjacent ground but does not prevent displacement. Additionally, preloading assures relative uniformity in brace loads.

High preloads may cause overstressing of struts because of unforeseen job conditions or temperature effects. Accordingly, the general practice is to preload bracing members to about 50 percent of their design load. This satisfies the criterion of removing the slack from the support system and at the same time reduces the risk of overstressing.

Figures 46 and 47 show prestressing details for braces. Preloading is accomplished by means of hydraulic jacks followed by securing the member with steel blocking, steel wedges, and welding. In the case of pipe struts the connection can be made by use of a telescoping strut or by a split pipe which fits over the pipe brace.

5.40 TEMPERATURE EFFECTS

5.41 General Background

Several papers (Armento, 1972; Armento, 1973; Chapman, et al, 1972; O'Rourke and Cording, 1974a; NGI, 1962) have addressed the problem of strut load variation with temperature. Since temperature variations in strutted excavations may easily be as great as 50^o F and even more if unprotected, the changes in load accompanying such temperature variation can be large.

A limiting case, and obviously conservative approach, would be to assume a perfectly restrained strut (i. e. no movement). The increase in load would therefore be equal to:

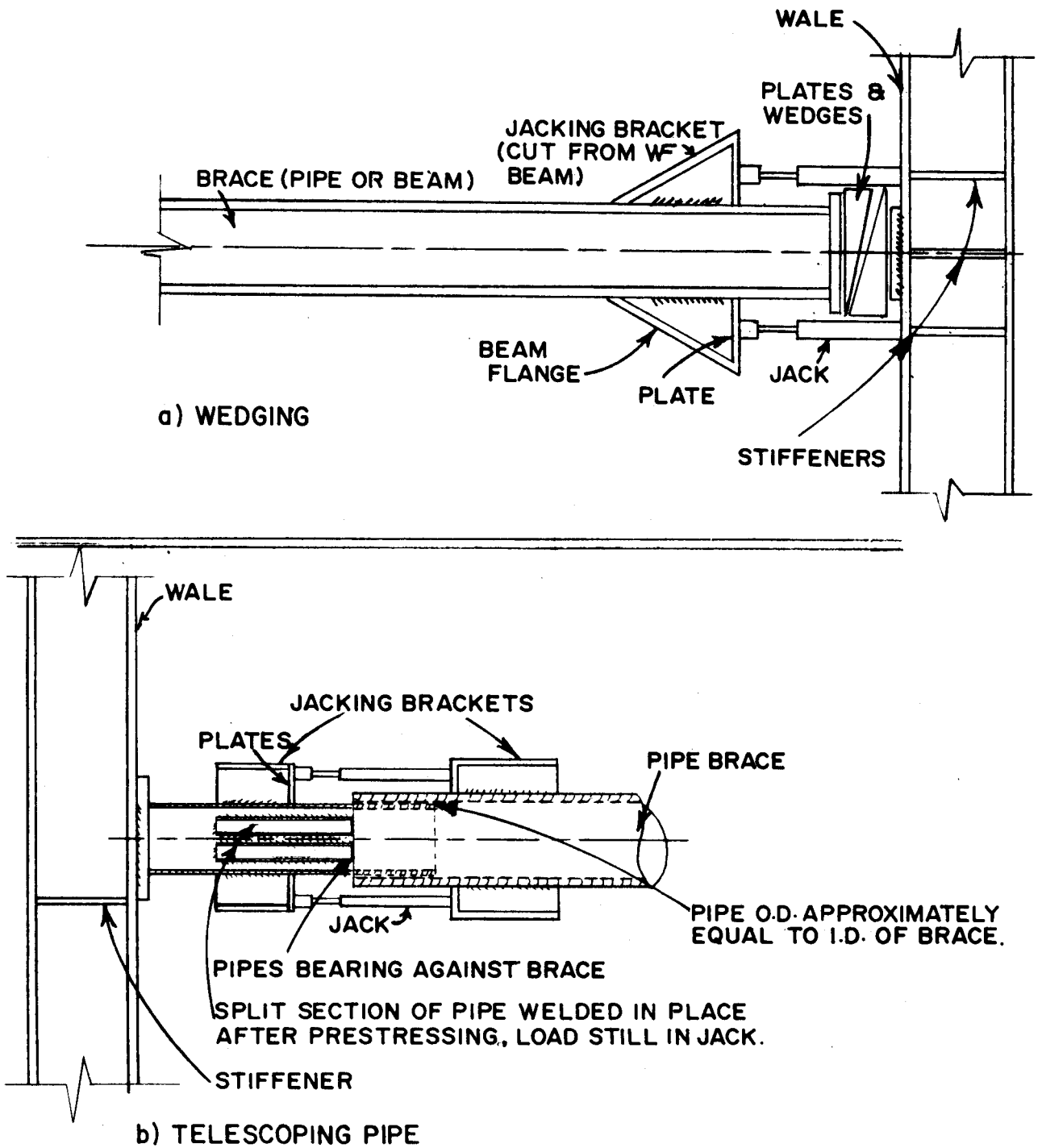


Figure 46. Prestressing details for braces.

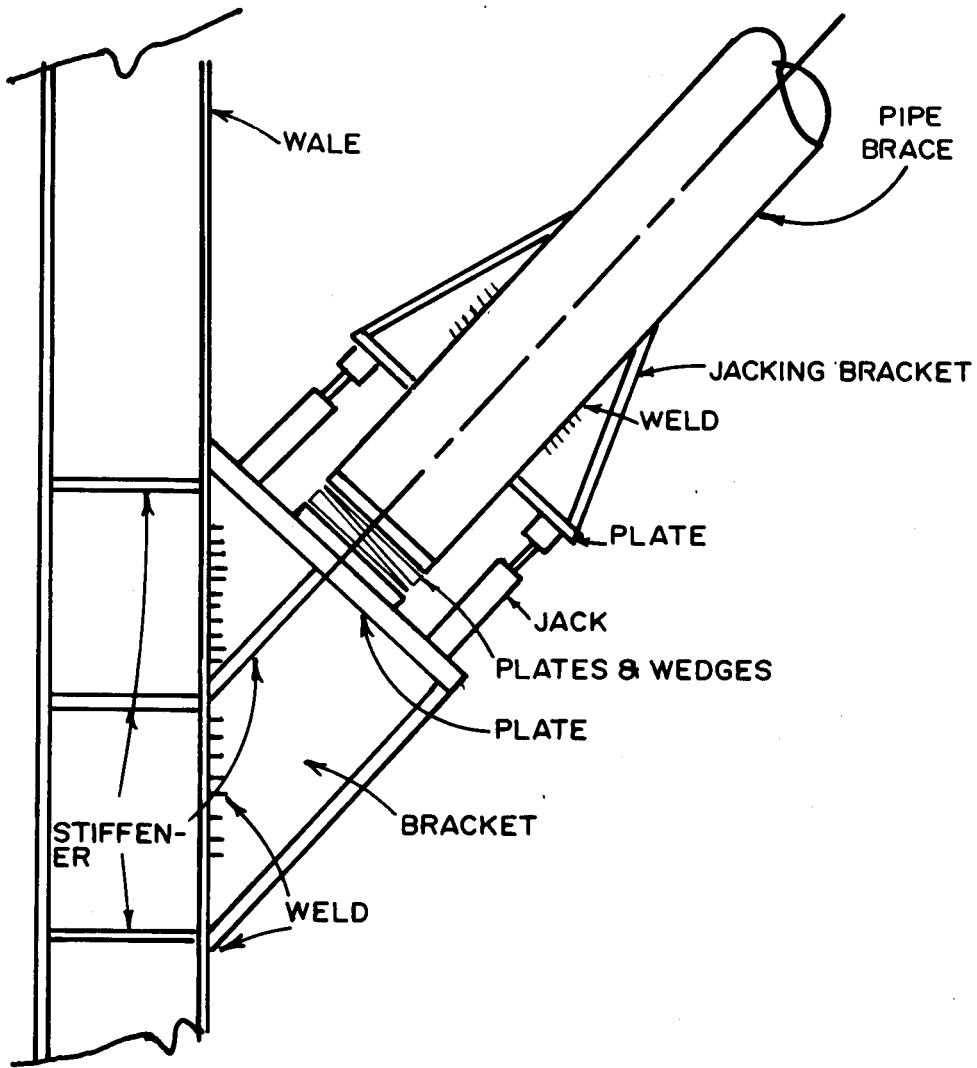


Figure 47. Prestressing of pipe brace at corners using brackets as reaction.

$$\Delta P = A_s E_s (\alpha \times \Delta^{\circ}F)$$

where

A_s = Area of Strut

E_s = modulus of strut (30,000 ksi)

α = thermal coefficient of expansion (6.5×10^{-6} in/in/ $^{\circ}$ F for steel)

$\Delta^{\circ}F$ = change in temperature in degrees Fahrenheit

In this case, a change in temperature of 40° F, for example, would result in a stress increase of

$$\frac{\Delta P}{A_s} = \Delta \sigma = 30,000 \times 6.5 \times 10^{-6} \times 40^{\circ} = 7.8 \text{ kips/in}^2$$

Actually, struts are not perfectly restrained, since the soil behind the wall yields under the increased loading. Chapman, et al (1972) measured the deflections and load variations in an open strutted excavation in Washington, D. C. For a 40° F increase, strut loads increased approximately 30 tons. The 30 ton load change represented approximately 30 percent of the total load. The theoretical increase in load due to a 40° F temperature change would have been 78 tons if the ends were perfectly restrained. The difference between 78 tons and the measured 30 ton change was attributed to some yielding of the soil behind the wall.

5.42 Some Case Studies

In the braced cut studied by Chapman, et al (1972) the strut load change due to a 1° F change in temperature was approximately 0.75 tons or 1.5 kips. In another excavation in Washington, D. C. (O'Rourke and Cording, 1974a) the strut load change was less than approximately 0.5 kips/ $^{\circ}$ F. In this case the excavation was decked over. The following cases are presented for the purpose of showing the order of magnitude of load variation that has been reported from field measurements.

<u>Case</u>	<u>Decked or Open</u>	<u>Load Variation</u>
1. Chapman, et al (1972)	Open	1.5 kip/ $^{\circ}$ F
2. O'Rourke & Cording (1974a)	Covered	0.5 kip/ $^{\circ}$ F
3. Jaworski (1973)	Open	20% \pm of measured average
4. Armento (1972)	Covered	10% \pm of measured average

5.43 Design and Construction Criteria

A generalized expression for predicting strut load variation was developed by Chapman, et al (1972):

$$P = A_s E_s (6.5 \times 10^{-6}) \left[\Delta^o F \times \frac{1}{1 + \left(\frac{3 \cdot n \cdot A_s \cdot E_s \cdot H}{A_{\text{cut}} E_d L} \right)} \right]$$

where:

A_s , E_s , and $\Delta^o F$ are as defined before

H = depth of cut

nA_s = total area of struts acting to brace wall

A_{cut} = area of excavation wall (tributary area to brace)

L = length of strut

E_d = Deformation modulus of soil

For cuts in Washington, D. C. in sand, gravel, and stiff clay a soil deformation modulus of from 5,000 psi to 15,000 psi was calculated on the basis of strut load changes due to temperature (O'Rourke and Cording, 1974a). Other methods of computing the field modulus are from plate bearing tests or from displacements measured during preloading at struts.

Much larger temperature variations can potentially occur in unprotected (undecked) excavations. Direct sunlight can cause the individual struts to reach temperatures far in excess of the measured air temperature. As a result it may become necessary to paint struts with a special reflecting silver paint or to spray water on the struts to keep them cool. These procedures are rarely necessary.

Wedging (or preloading) should be done at a time when strut temperatures are stable. Ideally, the temperature of the strut at the time of its installation should be at about the mean temperature anticipated during the course of the job. Natural variations of the actual temperature at the time of installation may be somewhat different from the reference temperature; and therefore, it will be necessary to make an adjustment in the preload force to account for the temperature differential. It may be desirable to monitor changes in strut load with temperature variation to provide an improved basis for establishing criteria for prestress loads on subsequent struts.

It should be noted that the Peck (1969) empirical earth pressure diagrams have been developed from measured maximum strut loads. Since the measured strut loads already reflect the effects of temperature, the Peck diagrams implicitly take temperature into account.

5.50 STRUT REMOVAL AND RE-BRACING

An additional source of displacement is the removal of braces (often accompanied by re-bracing) associated with construction of the structure within the cofferdam. Primary parameters controlling displacements are the wall stiffness, the deformation properties of the retained soil, the span distance between the remaining braces, and the quality and compaction of the backfill between the structure and the ground support wall.

Removal of struts in an excavation in Oslo resulted in 4 inches of additional lateral displacement of the sheet pile wall (NGI, 1962). The soft clay behind the wall influenced the magnitude of the lateral movements.

l
D'Rourke
ccurred
pile wall.

A well-documented case history of lateral displacement associated with strut removal was reported by Cording and Cording (1974a). The data show that the displacement is in direct response to removal of struts supporting the wall.

CHAPTER 6 - TIEBACKS

6.10 INTRODUCTION

During the last 20 years the use of soil and rock anchors to support side walls of excavations has increased significantly. Tiebacks (or anchors) have been used to support both temporary and permanent excavations.

A tieback consists of 3 major components (See Figure 48):

1. An anchor zone which acts as a reaction to resist the lateral earth and/or water pressures.
2. A support member which transfers load from the wall reaction to the anchor zone.
3. A wall reaction or point of support.

Since the wall reaction is the only part of the tieback in the excavation, a tieback system provides an open work area.

At present, the design of tied-back walls in the United States is based largely on empirical relationships obtained from successful tieback installations. This state-of-the-art report summarizes the practice of European and American designers and contractors. The design and construction recommendations are intended to serve as guidelines in practice and do not preclude the use of other established design or construction techniques.

The chapter has been organized into four major sections.

1. General design and theoretical considerations regarding tieback wall design and performance.
2. Specific design considerations including discussions of overall wall stability, anchor zone capacity, and tie member design.
3. Discussion of construction methods including typical equipment and installation procedures used.
4. Field testing criteria used to ensure adequate performance of a tieback system.

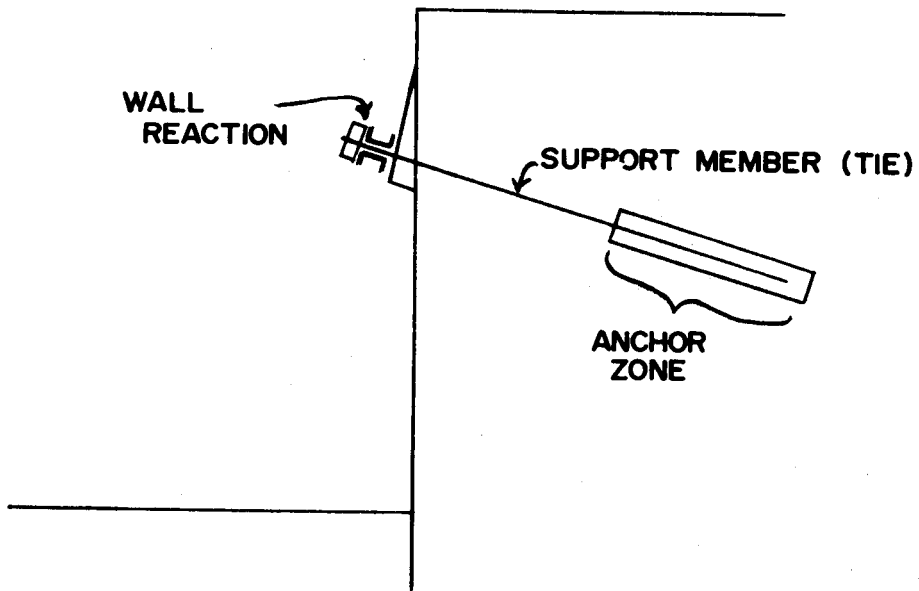


Figure 48. Major tieback components.

6.20 DESIGN AND THEORETICAL CONSIDERATIONS

6.21 General

The design of tied-back systems involves several major design considerations. First, an evaluation of the forces that must be resisted by the tiebacks must be made. This would include an evaluation of earth and water pressures acting on the excavation wall. Second, there must be a suitable stratum for anchorage. Third, the overall stability of the earth mass must be evaluated. Finally, vertical and horizontal deformations must be considered.

Since loads and deformations are interdependent, an analysis of these quantities is extremely complex. The state-of-the-art methods for determining these quantities rely heavily upon empirical procedures, supported qualitatively by theory and performance records. Volume II (Design Fundamentals) discusses the performance of internally braced and tied-back excavations and describes the design parameters used to determine loads on support walls.

6.22 Deformations

6.22.1 General

In recent years research into the area of tied-back wall-support interaction has been increasing in an effort to understand the factors affecting wall performance and design. This section discusses some factors affecting the performance of tied-back walls, particularly with respect to vertical and horizontal wall movements.

Several papers have been published (Hanna, 1968a; Hanna and Matallana, 1970; Egger, 1972b; Hanna, 1973b; Clough and Tsui, 1974) which present the results of both empirical and theoretical studies of tied-back walls. Some of the factors affecting wall performance, earth pressure distribution, and anchor loads are wall stiffness, amount of tieback prestress, design assumptions, and wall movement.

6.22.2 Vertical Wall Movement

Since most tiebacks are inclined at some angle to the support wall, a portion of the preload in the tieback is transferred to the wall as a vertical load, which may result in settlement of the wall. The steeper the angle of inclination, the greater the likelihood of settlement. This vertical load must be resisted by end bearing and frictional resistance in the wall whether the member be a soldier pile, steel

sheeting, or slurry wall. Several papers have addressed the problem of settlement of wall members. Dietrich, et al (1971) report a case where as much as 2.5" (6.3cm) of settlement of a soldier pile occurred. Ware, et al (1973) describe a project where inclined rakers were installed to prevent further settlement of a soldier pile. Shannon and Strazer (1970) report the case of a soldier pile that settled 3" (7.5cm).

During the course of construction the load at the base of the wall increases not only from the additional vertical component of force from the anchors but also the decrease in frictional resistance along the face of the wall caused by the removal of material. This would be particularly true for driven members.

The sketch in Figure 49 shows a relationship between vertical and horizontal deformations that may exist. If all other quantities are maintained constant, the horizontal movement accompanying wall settlement is:

$$\delta_h = \delta_v \tan \alpha$$

where:

δ_h = horizontal movement

δ_v = vertical movement

α = angle of tie to horizontal

If the integrity of nearby structures is to be maintained, little or no vertical movement of the wall can be allowed.

In severe cases the additional vertical load from tiebacks may cause a bearing capacity failure at the wall base and failure of the support wall. White (1974a) reports several cases where tied-back walls bearing on rock may be unstable. This is particularly true in those cases where the interior excavation extends below the base of the wall.

Most problems with tied-back walls have been caused by excessive vertical movements. Evaluation of the resistance of the wall to vertical movement is critical in any design. Obviously, load bearing competency of the wall must be assured. Another technique is to slope the sheeting (flared outward at the top) to reduce the downward component of load transmitted to the sheeting.

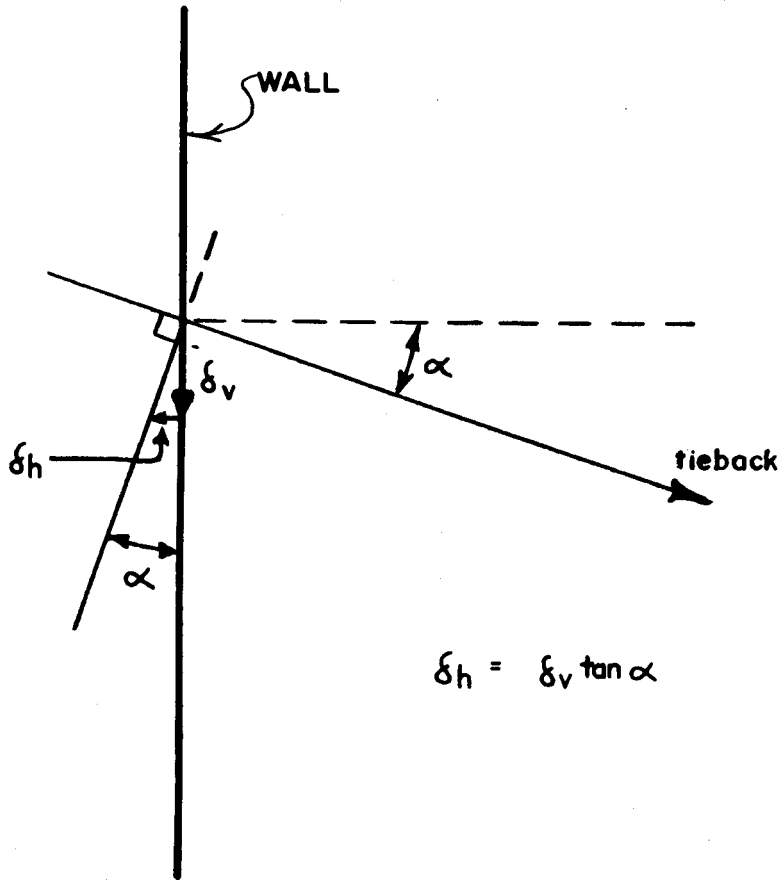


Figure 49. Horizontal deflection resulting from wall settlement.

6.22.3 Horizontal Wall Movement

In general, horizontal deformation of the ground support wall is controlled by the following factors:

- a. Relative stiffness of wall and soil.
- b. Tieback prestress.
- c. Deformation of soil block contained within the tiebacks.
- d. Movement of soil block.
- e. Settlement of support wall.
- f. Ground loss associated with construction method.
- g. Volumetric strain.

The effect of settlement of the support wall on lateral movements was discussed in the previous section. The other factors affecting lateral movements are discussed below.

Relative Stiffness of Wall and Soil

As is the case for internally braced walls, the wall initially moves inward during excavation. For internally braced walls, the placement of each strut or raker level ideally prevents any further inward movement of the wall at that point. Therefore, the inward movement of the wall is a function of the soil and water pressure acting on the wall, the stiffness of the wall, and the span between bracing levels below the lowest in-place strut.

In tied-back walls, prestressing of the first strut level may cause the upper part of the wall to move toward the retained soil (Hanna and Matallana, 1970; Clough, et al; 1972). The amount of movement during prestressing is influenced by the flexibility of the wall and the looseness of the soil immediately behind the wall. The movement would also be affected by overcut and improper backpacking behind lagging. In a qualitative sense the deformation is closely related to a beam on an elastic foundation. For example, excessive prestressing of upper ties in a relatively flexible wall-ground system would pull the upper part of the wall to the soil causing rotation of the elastic line

of the wall near the tieback anchorage. When subsequent excavation is made below the tieback, the wall at lower elevations would deflect toward the excavation. This effect is unlikely in a reinforced concrete wall because of its rigidity.

Egger (1972b) performed a finite element analysis of two walls that varied greatly in stiffness. The movements predicted for the stiffer wall were less than the movements predicted for the more flexible wall. This conclusion is supported by field experience. Concerning load, Egger found that pressure is more evenly distributed in the case of the stiffer wall. Hanna (1968a) developed similar conclusions.

Tieback Prestress

Insufficient prestress would result in load increase in ties accompanied by strain at the anchorage and elastic elongation of ties. To mitigate this situation ties are usually prestressed to about 80 percent or more of design load. The choice of the amount of prestress can have a marked effect on the movement of a wall. In his analysis Egger (1972b) found that wall movements decreased substantially with increased prestressing. This too is supported by field experience.

On the other hand, in certain cases excessive prestressing can cause movement. One situation is the case described above where the upper portion of a relatively flexible wall is pulled toward the soil and the lower portion deflects inward as the excavation proceeds. Also, it has been reported that prestressing of ties in rock induced settlement (consolidation) from overstress and yield of an overlying sensitive clay. In this case the movement of the wall was away from the excavation (McRostie, et al, 1972).

Deformation of Soil Block Contained by Tiebacks

If a tieback system retains its prestress, the wall and the prestressed soil behind the wall act together much as a gravity retaining wall might. It is possible to view the internal deformation of the soil block in the same way that internal shear development for stability of a cofferdam is viewed. In this case the movement is horizontal with the greatest movement of the earth mass occurring near the ground surface. Figure 50 illustrates the theoretical pattern of deformation for this case.

Movement of Soil Block

The entire soil block will also move in response to the removal of soil support on one side of the block. The movement

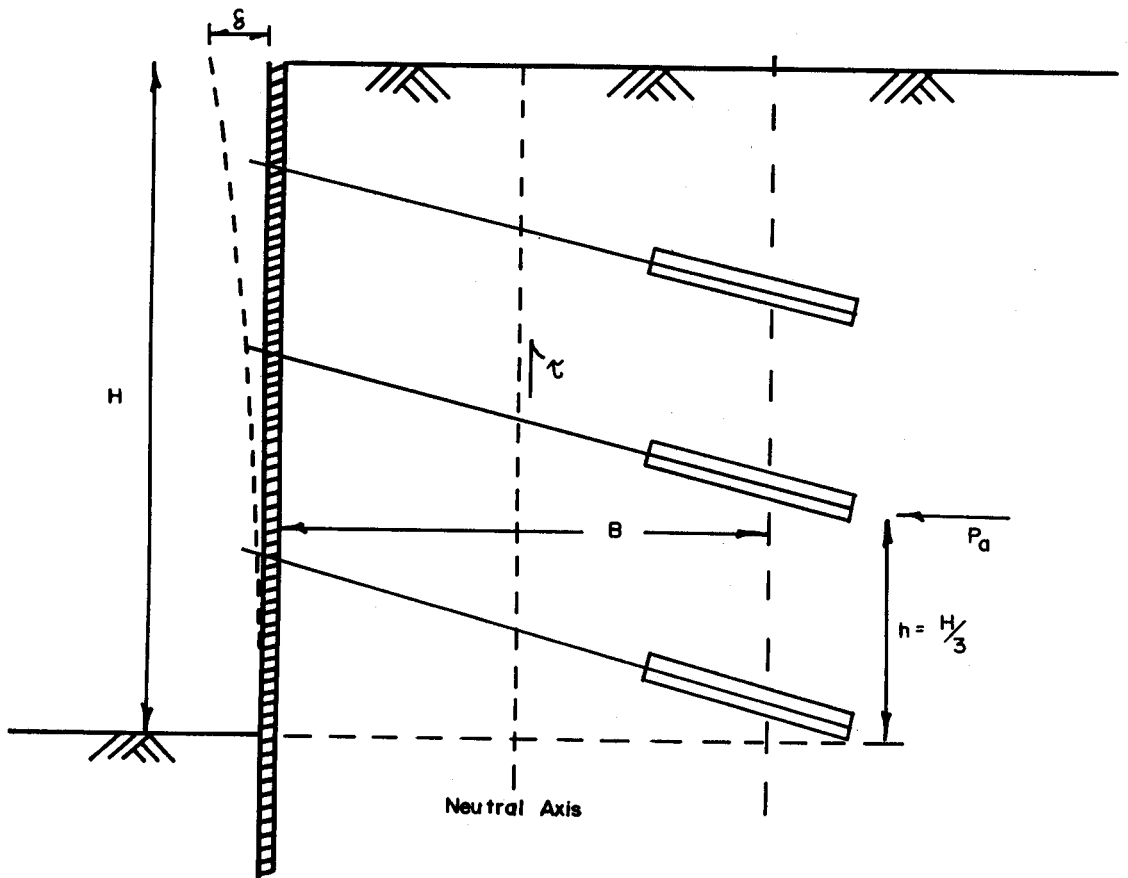


Figure 50. Sketch of equivalent cofferdam for tied-back wall.

of the soil block may be caused by strain required to mobilize soil strength for stability or by compression of the soil block below the base of the excavation.

Figure 51 illustrates the pattern of movements of the soil block (and wall) that may occur from the mobilization of shear strains to prevent a stability failure. The magnitude and pattern of the movements will depend upon the strength and stress/strain characteristics of the soil. The type of movement realized on a site will depend upon the soil conditions. In uniform soil conditions a rotational failure is more likely to occur, while in a layered soil profile a "sliding block" or translational failure may result.

Nendza and Klein (1974) and Breth and Romberg (1972) have presented discussions of the movements associated with tied-back walls. The authors have proposed a possible mechanism for tied-back wall movement that considers the movements associated with internal straining of the contained soil mass and lateral movement associated with pressure relief and compression of soil layers below the excavation base. Figure 52 illustrates how the various mechanisms proposed by Nendza and Klein (1974) would combine to result in an overall pattern of deformation. Clearly, the magnitude of the movement will depend on the stress/strain characteristics of the soil.

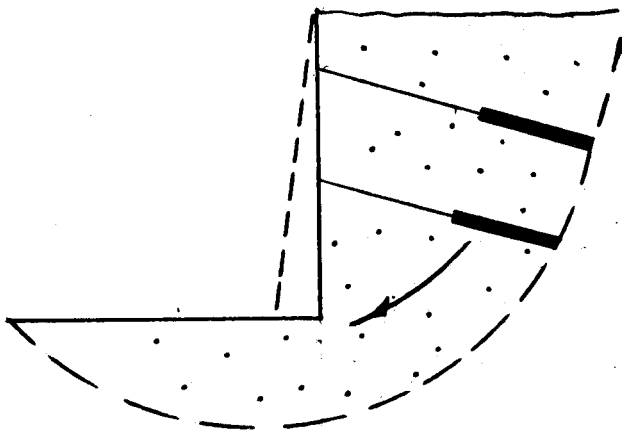
Ground Loss

Generally, each wall type or construction method used has associated with it a particular type of deformation or movement. Examples are: (1) the soil retained by a soldier pile wall will be subject to local sloughing and inward movement during placement of lagging and overcut; and (2) walls of a slurry trench may undergo local collapse during excavation.

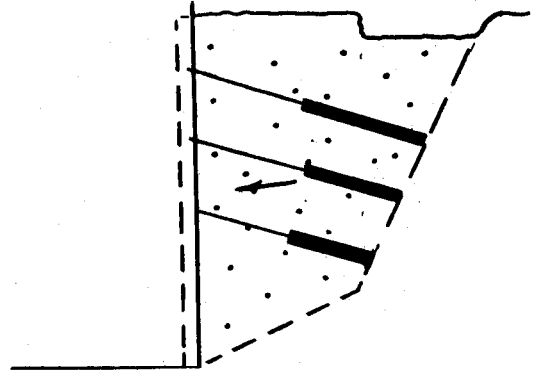
A specific potential for ground loss is when "running" ground flows through the tieback drill hole. This may occur if improper procedures are followed when tiebacks are installed through fine sand below the water table.

Volumetric Strain

In very stiff over consolidated clays there appears to be a tendency for the soil mass to move toward the excavation with time (St. John, 1974; Breth and Romberg, 1972). Some of this movement may be due to lateral soil expansion resulting from a decrease in lateral stress.



(a)
CIRCULAR ARC
ROTATIONAL MODE



(b)
SLIDING BLOCK
TRANSLATIONAL MODE

Figure 51. Possible stability failure modes for tied-back walls.

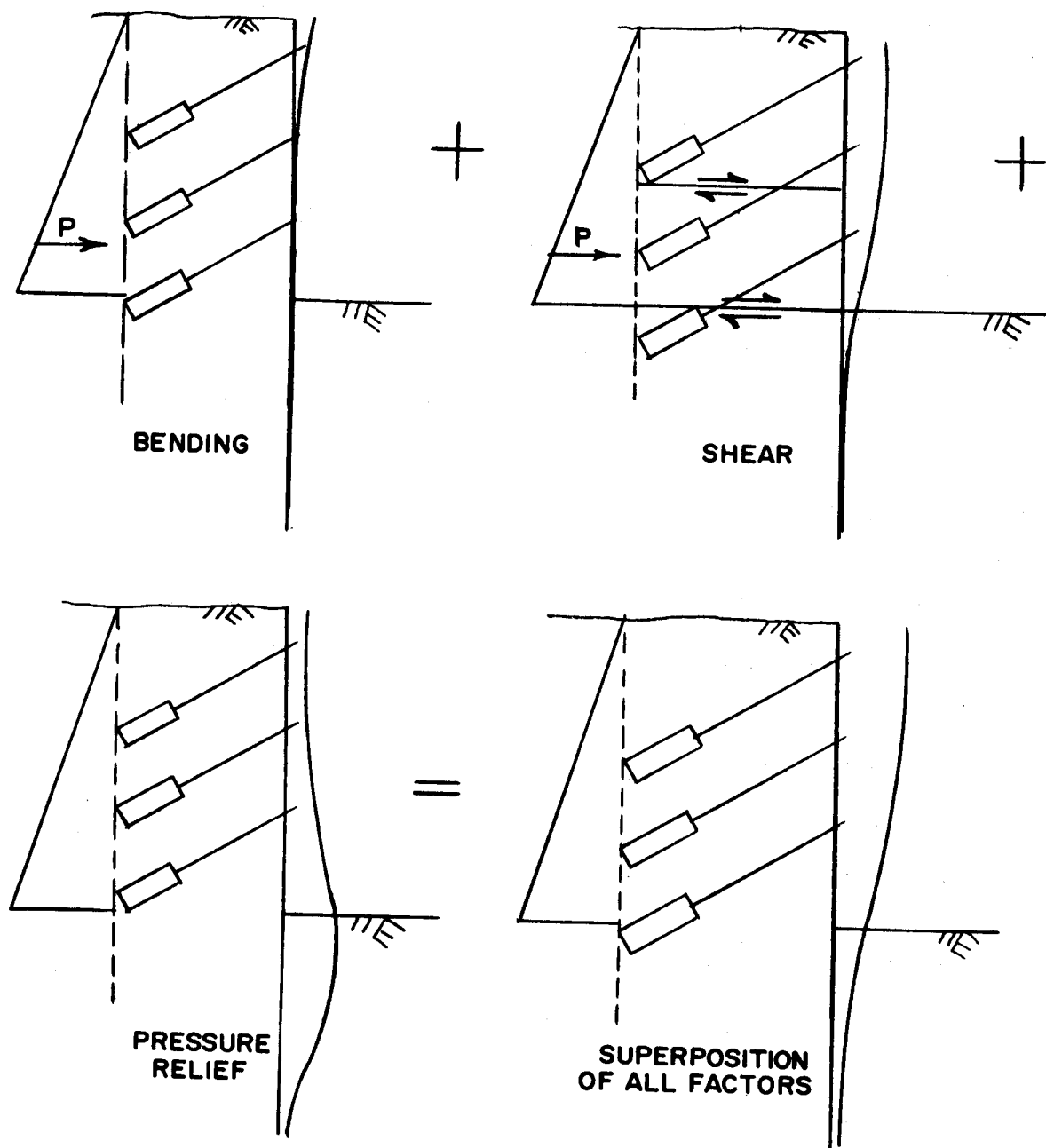


Figure 52. Idealized tieback wall deformation as proposed by Nendza and Klein (1974).

A decrease in lateral stress may also result in loss of strength in heavily overconsolidated soils.

6.22.4 Discussion

The movements that are likely to occur in a wall must be considered during the design phase. These movements must be evaluated in terms of the effects on adjacent structures and the stability of the excavation. There are a variety of factors that affect wall and adjacent soil movements including wall stiffness, tieback spacing, wall settlement, tieback prestress, internal deformation of the soil block, translation or rotation of the soil block, and movements associated with the particular wall type.

Although the precise nature of soil-wall interaction is unknown, all these factors combine to result in a final observed pattern of deformation. In a particular case any one of these factors may be the primary cause of the movements observed on the site. The discussion of movements in this section is intended to inform the engineer and/or contractor of the factors that affect tied-back wall movements and possible mechanisms controlling soil deformation behind the wall.

6.23 Overall Stability of Soil Mass

6.23.1 General

This section discusses the various methods used to analyze the stability of a soil mass behind a tied-back wall. Since it is assumed that the wall is stable (i. e. can resist earth and water pressures), this discussion focuses on the stability of the earth mass retained by the wall. The primary concern of these stability analyses is to determine whether the anchor location and soil shear strength provide adequate safety against failure of the soil mass and wall.

The possible modes of failure for a tied-back wall include:

1. Circular Arc Stability Failure
2. Overturning Stability Failure
3. Sliding Wedge Stability Failure
4. Internal Stability

The determination of the overall stability of a tied-back wall system generally involves the evaluation of the stability of the soil-wall system for several of these failure conditions.

6.23.2 Circular Arc Analysis

Circular arc stability analyses are widely used in practice and are discussed in soil mechanics texts and in Volume II (Design Fundamentals) of this report. When applied to tied-back walls, these analyses should specifically consider failure surfaces outside the tieback zone and below the base of the wall. Although this case is usually not critical, it should always be checked.

6.23.3 Overturning Analysis

In Europe two possible modes of failure are generally investigated. The recommended stability computation techniques are given by Ranke and Ostermayer (1968) who expanded upon the work performed by Kranz (1953). A circular arc analysis is performed to ensure the stability of the wall against failure of the soil mass outside the anchor zone and below the base of the wall. An analysis is also performed to determine whether the anchor locations are adequate to resist overturning moments on the soil mass. Figure 53 illustrates schematically a failure by overturning.

To simplify the analysis the failure surface at the base is assumed to be a straight line. The analysis therefore becomes a sliding wedge analysis with the free body taken on the inside of the wall. The German Design Codes (DIN 4125, 1972) and the Bureau Securitas (1972) recommend this method of analysis.

Free Body Diagram and Forces

Figure 54 illustrates the free body diagram and the forces acting on the free body. The wall is not considered part of the free body. Only the forces acting on the soil mass are considered. For this reason the forces, P_A and T_{des} , have been drawn in the directions shown. The choice of the free body (not including the wall) distinguishes this method from the more generalized sliding wedge approach discussed later.

The location of the free body is predetermined in this method of analysis. Points A and E are located at the ground surface immediately above points C and D. Point C is chosen as the point at which the shear forces in the wall are equal to zero. In other words, point C represents the point at which $P_{Ah} - T_{des} = P_{ph}$. Point

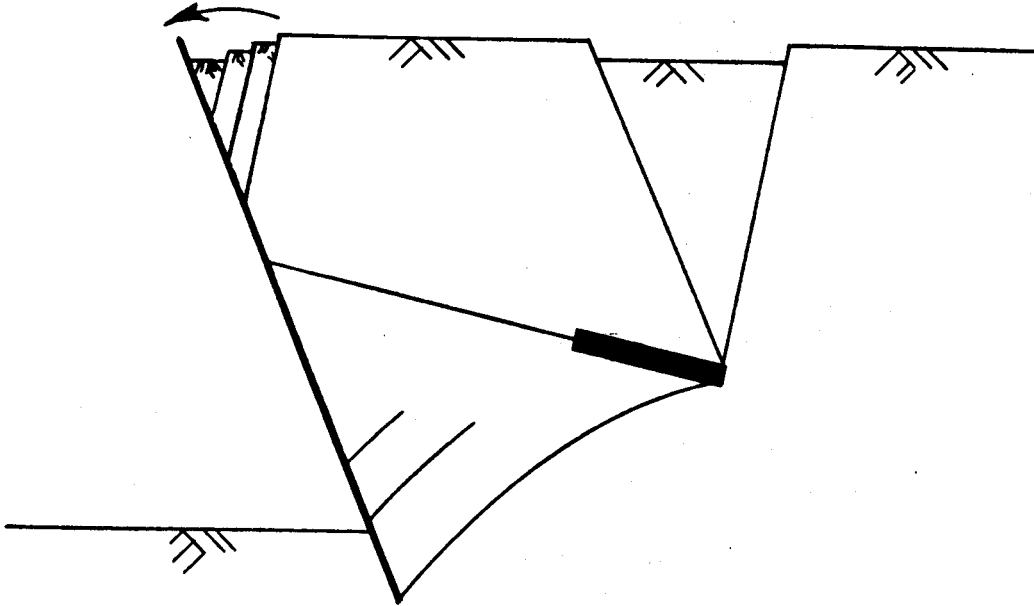


Figure 53. Sketch of tied-back wall failing by overturning.

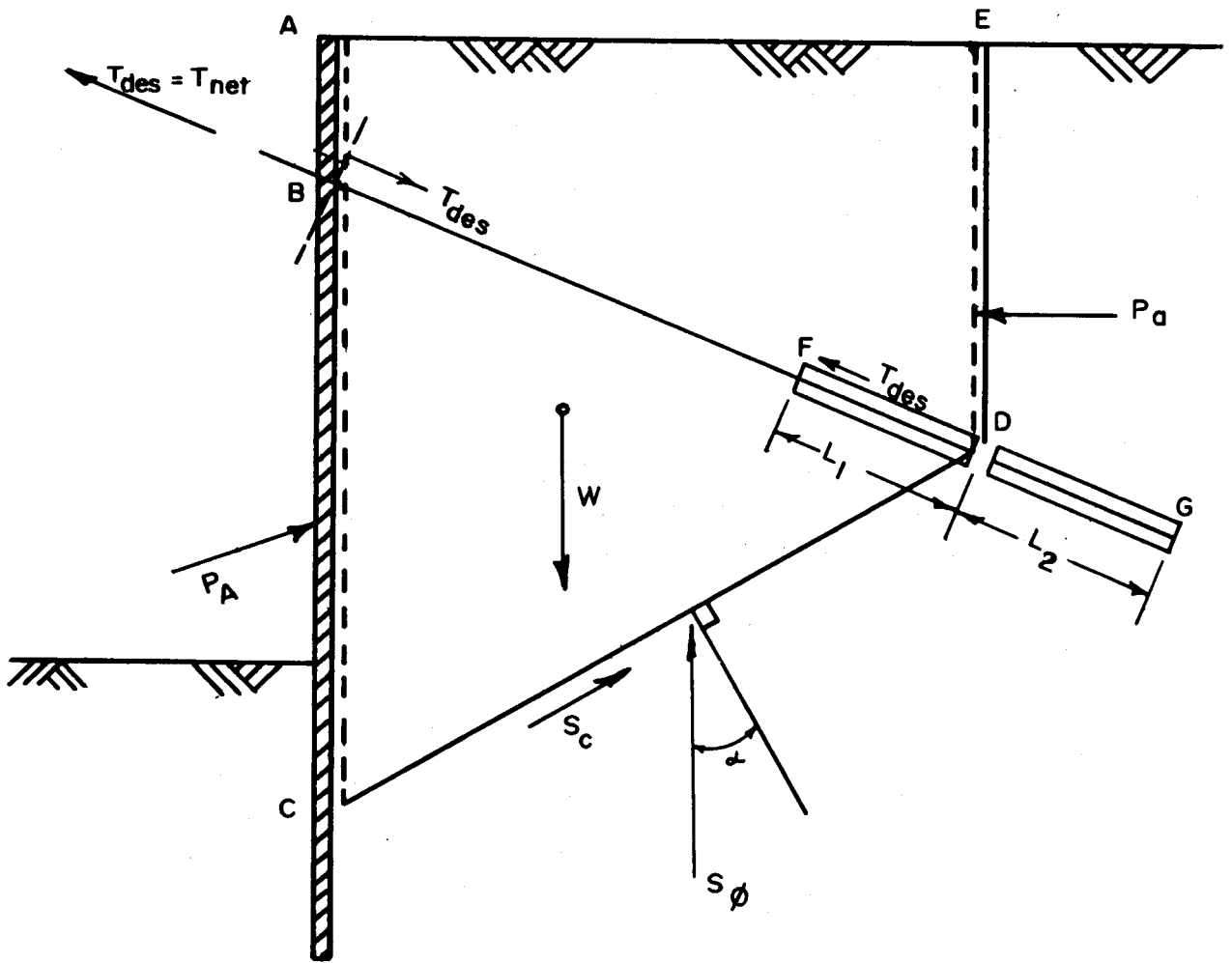


Figure 54. Free body diagram for a failure surface in single anchor tieback system (internal free body).

D is uniquely defined as the midpoint of the grouted anchor length. Therefore, in Figure 54, L_1 would be equal to L_2 . In this method of analysis the entire anchor load is assumed to be transmitted between points D and F.

The forces acting on the soil mass are:

a. P_a - the driving force on the face DE due to the soil pressure. Ranke and Ostermayer (1968) state that this force should be calculated as the active soil pressure. While P_a has been drawn horizontally, it can also be an inclined force. A driving force due to water must be considered when below the water table.

b. W - the weight of the soil mass within the free body.

c. P_A - the total active force acting along the face AC. This resultant is inclined at the friction angle between the soil and the wall.

d. S_ϕ - the frictional component of soil resistance. This force is applied at an angle, ϕ , to the normal to the failure surface. Full soil strength is mobilized.

e. S_c - the component of soil resistance due to cohesive soil strength.

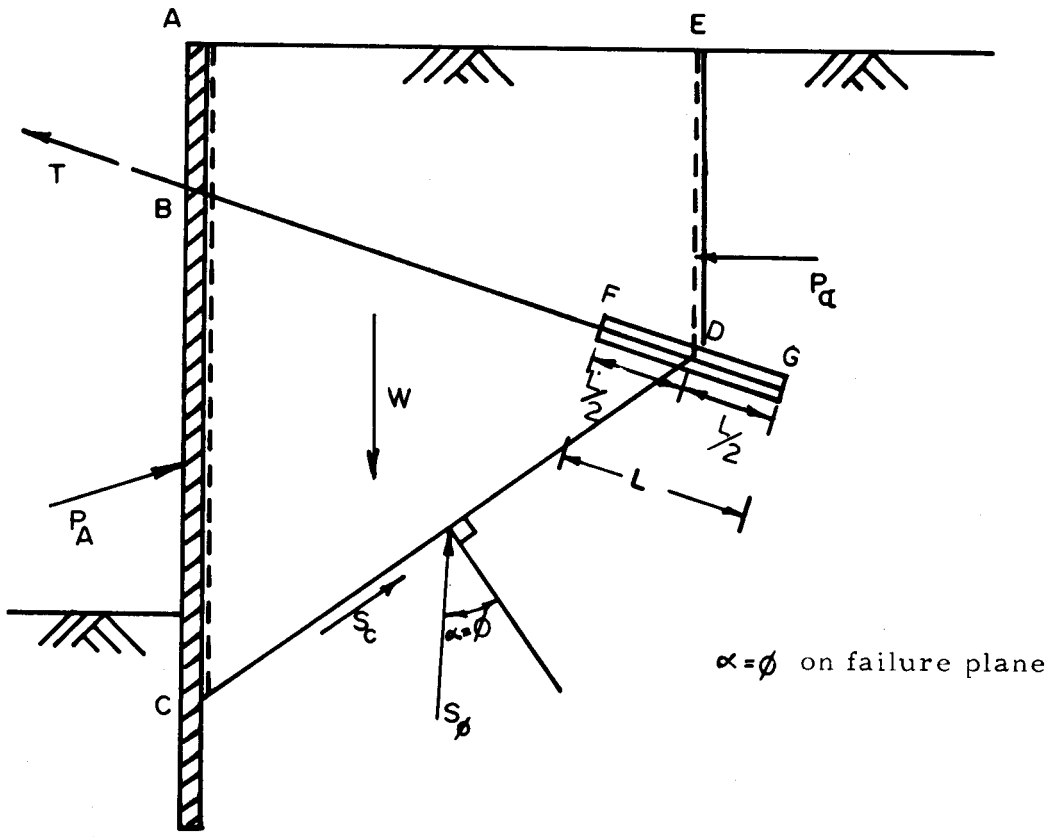
f. T - the tieback force. The free body cuts the tieback at points B and D. The force, T_{net} , (Figure 54) represents the vector sum of tieback force at point B and point D. Since the force at B must exceed the force at D, the force acts in the direction shown.

Safety in Terms of Tieback Force

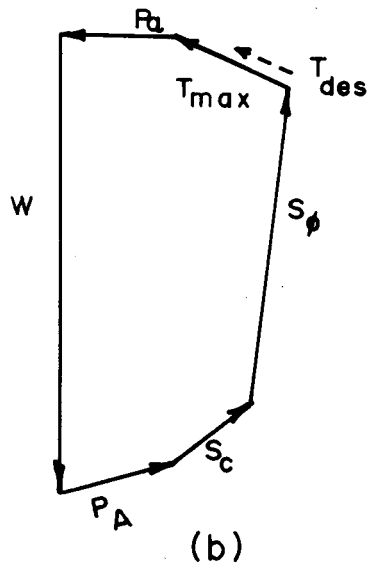
The force T_{max} is the maximum possible force acting in the direction of the tieback (see Figure 55). It should be noted that its magnitude will increase with increasing shear resistance on the failure plane. The overall stability is evaluated in terms of the ratio of T_{max} to the design tieback force; or,

$$F. S. = \frac{T_{max}}{T_{des}}$$

This method of analysis can be applied to both single and multiple anchor systems. A brief description of each of these cases follows.



(a)



$$F.S. = \frac{T_{max}}{T_{des}} \geq 1.5$$

After Kranz (1953) and
 Ranke & Ostermayer (1968)

(b)

Figure 55. Single anchor free body diagram with appropriate vector diagram (safety in terms of the tieback force).

a. Single Anchor. Figure 55 illustrates a single anchor tied-back wall, and the force diagram used to evaluate the stability of the system against overturning.

The vector diagram in Figure 55 defines the maximum tieback force consistent with the stability of the earth mass. The design tieback force must be less than this value, T_{max} . The Bureau Securitas (1972) and the German Design Codes (DIN 4125, 1972) recommend a minimum factor of safety of 1.5.

The method described to this point has been applicable to soil conditions where no water is present. If water is present, the pore water forces act on the free body, and the analysis should be consistent with other basic methods of stability analyses as described in many soil mechanics tests.

b. Two Independent Anchors. Figure 56 illustrates a two level anchor system for a wall. The forces acting on each free body are evaluated in the same manner as for a single anchor system. The stability of each failure surface must be evaluated. Figure 57 shows the vector diagrams for each free body.

Since each anchor is outside the free body of the critical failure surface for the other anchor, the stability of each anchor is evaluated separately. The second anchor has no direct influence on the stability of the chosen failure surface. It is recommended that the factor of safety for each anchor be at least 1.5.

c. One Independent Anchor. For the case shown in Figure 58, the stability of one anchor is independent of the stability of the other. However, the stability of the second anchor depends on the anchor force in the first. Figure 59 illustrates the vector diagrams used to evaluate the stability of the critical surfaces. As before, the critical surfaces are chosen to pass through the center of the anchor zone, and the frictional component of the soil resistance is assumed to act at full obliquity, ϕ , in the analysis. The full value of the cohesive soil resistance is also assumed to act. A similar analysis would be made if the top anchor were the independent anchor instead of the bottom anchor. The minimum recommended factor of safety for either case is 1.5.

d. Complex Failure System. Figures 60 and 61 show the free body diagram and vector diagram for a more complex failure surface. The analysis of the stability of this system is made by drawing a combined vector diagram for two assumed free bodies. The

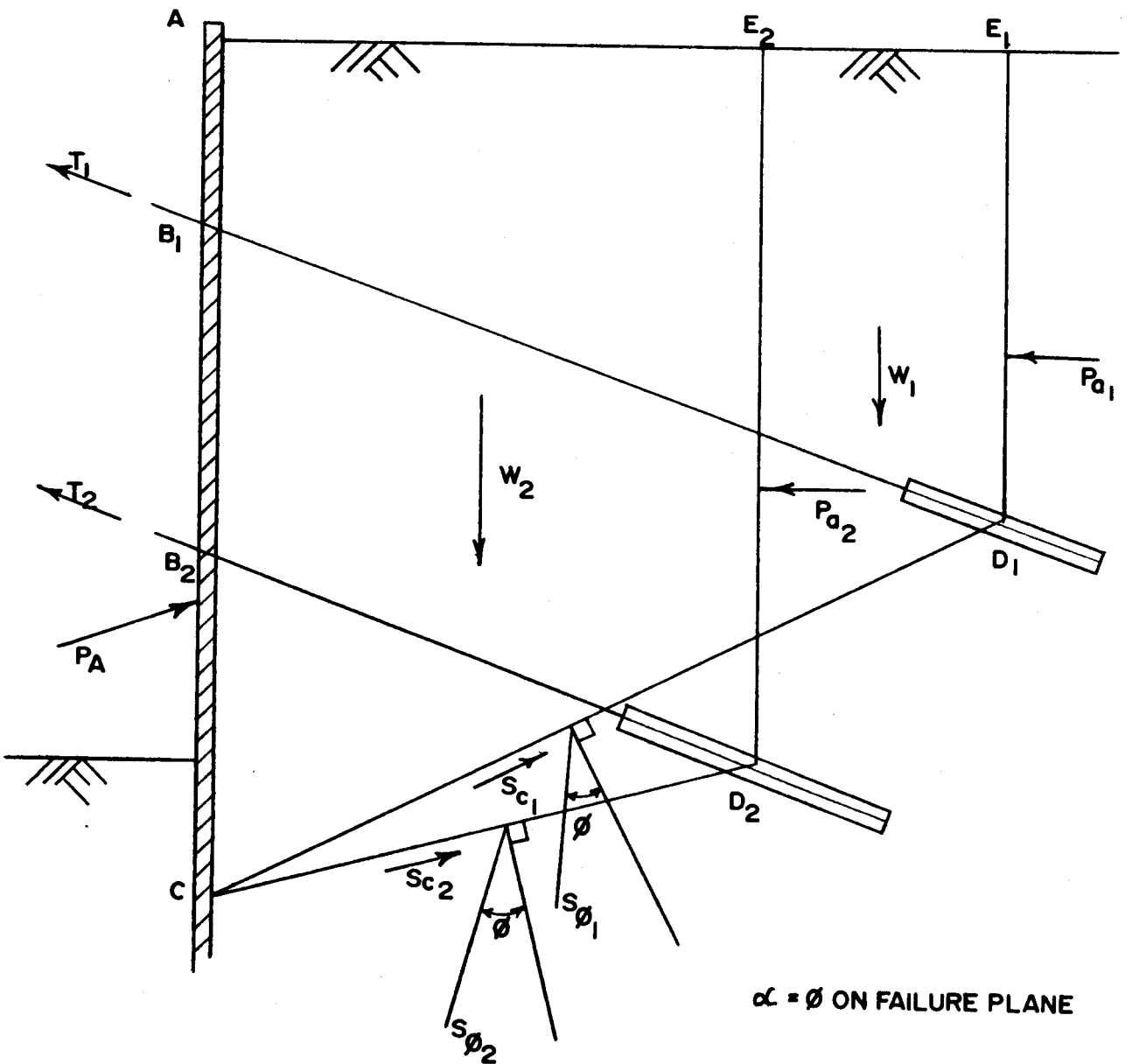
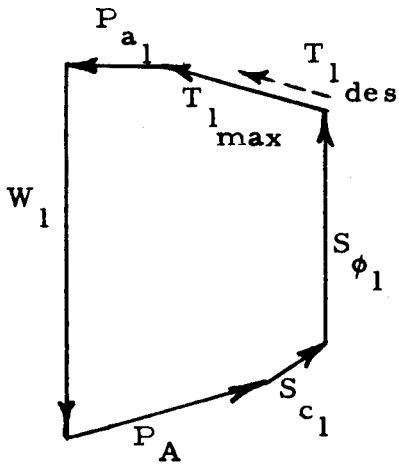


Figure 56. Free bodies and forces for two completely independent anchors (after Ranke and Ostermayer, 1968).



a. Upper Tieback

$$\text{F. S.} = \frac{T_{1\text{max}}}{T_{1\text{des}}} \geq 1.5$$

Note: only the directions of S_ϕ and T_{max} are known.

$\phi = \phi$ on failure plane.

B. Lower Tieback

$$\text{F. S.} = \frac{T_{2\text{max}}}{T_{2\text{des}}} \geq 1.5$$

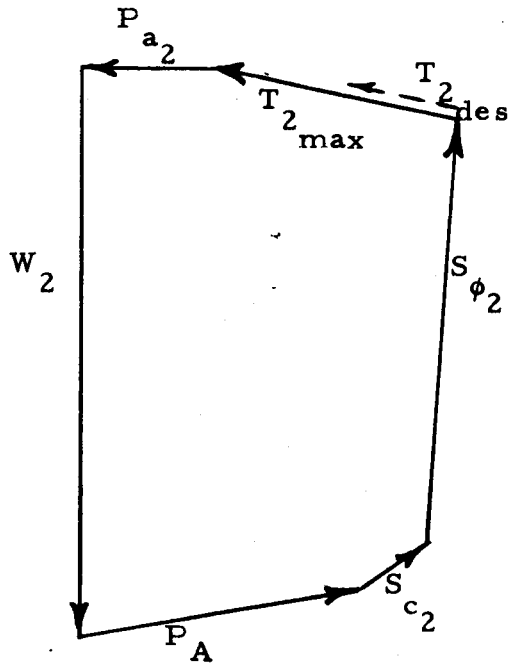


Figure 57. Vector diagram for case of two completely independent anchors (safety in terms of anchor force) (after Ranke and Ostermayer, 1968).

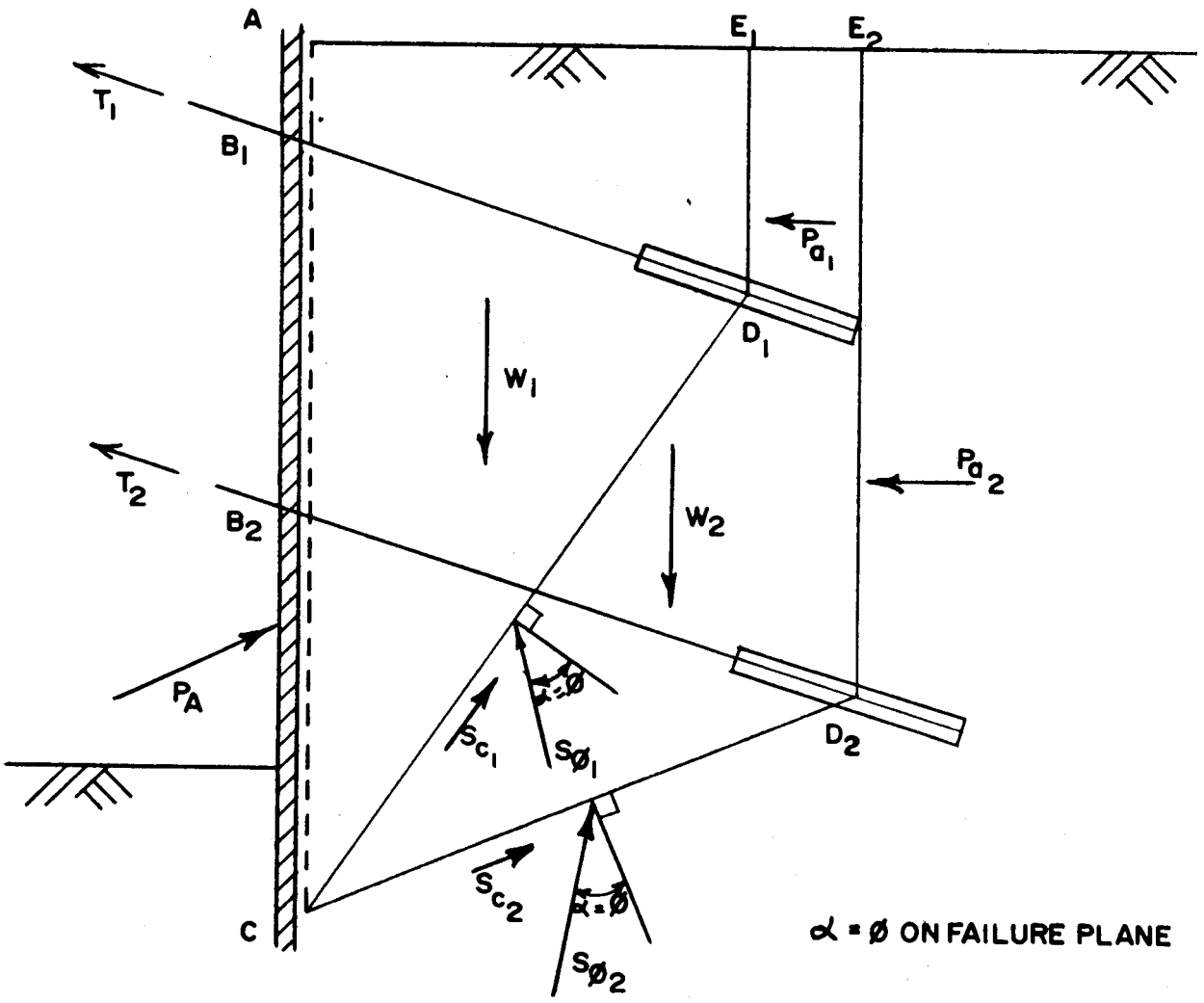
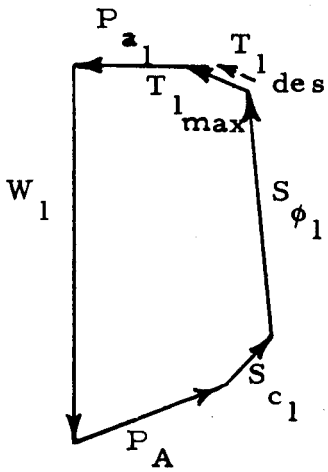


Figure 58. Free body diagram with forces acting on the bodies for the case of one independent anchor (safety in terms of the tieback force) (after Ranke and Ostermayer, 1968).



$$F. S. = \frac{T_{1max}}{T_{1des}} \quad 1.5$$

$\infty = \phi$ on failure plane.

$$F. S. = \frac{T_{1-2max}}{T_{1des} + T_{1des}} \quad 1.5$$

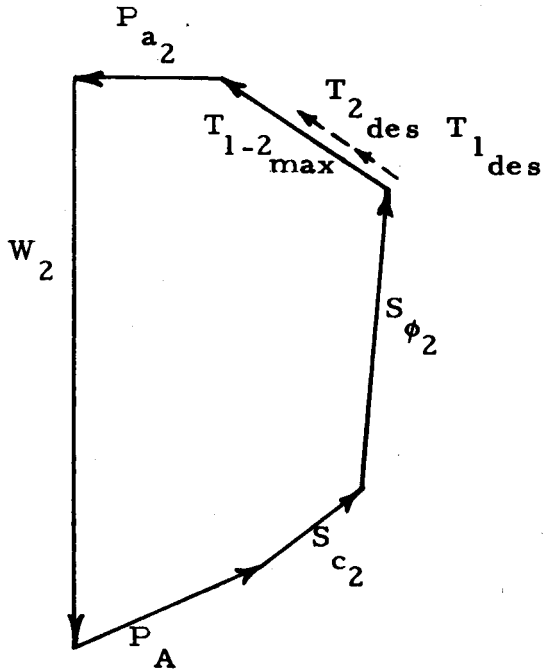


Figure 59. Vector diagrams used to evaluate the stability of case with one independent anchor (safety in terms of tieback force) (after Ranke and Ostermayer, 1968).

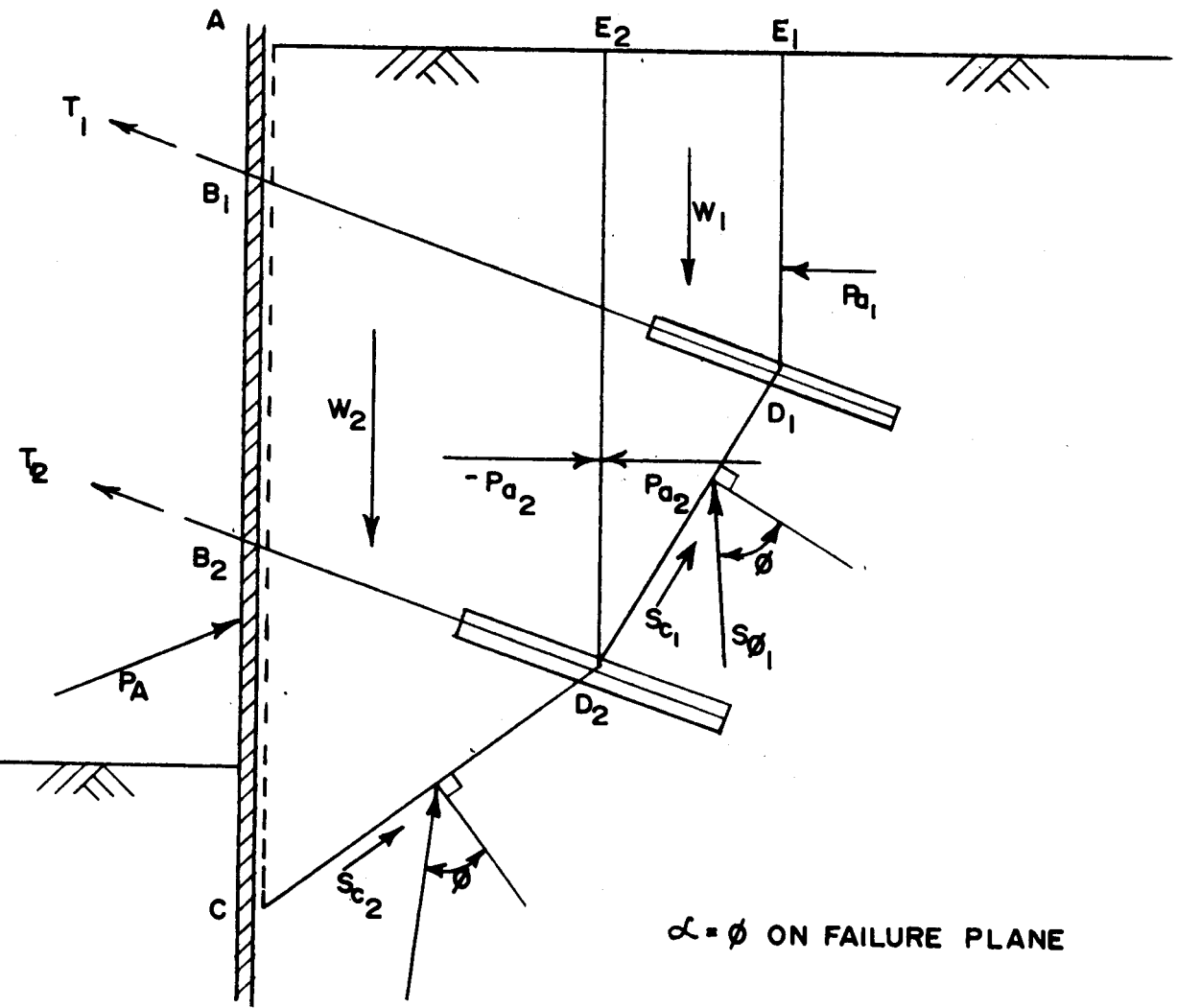


Figure 60. Free body diagram for anchor system with a complex failure surface (safety in terms of the tieback force).

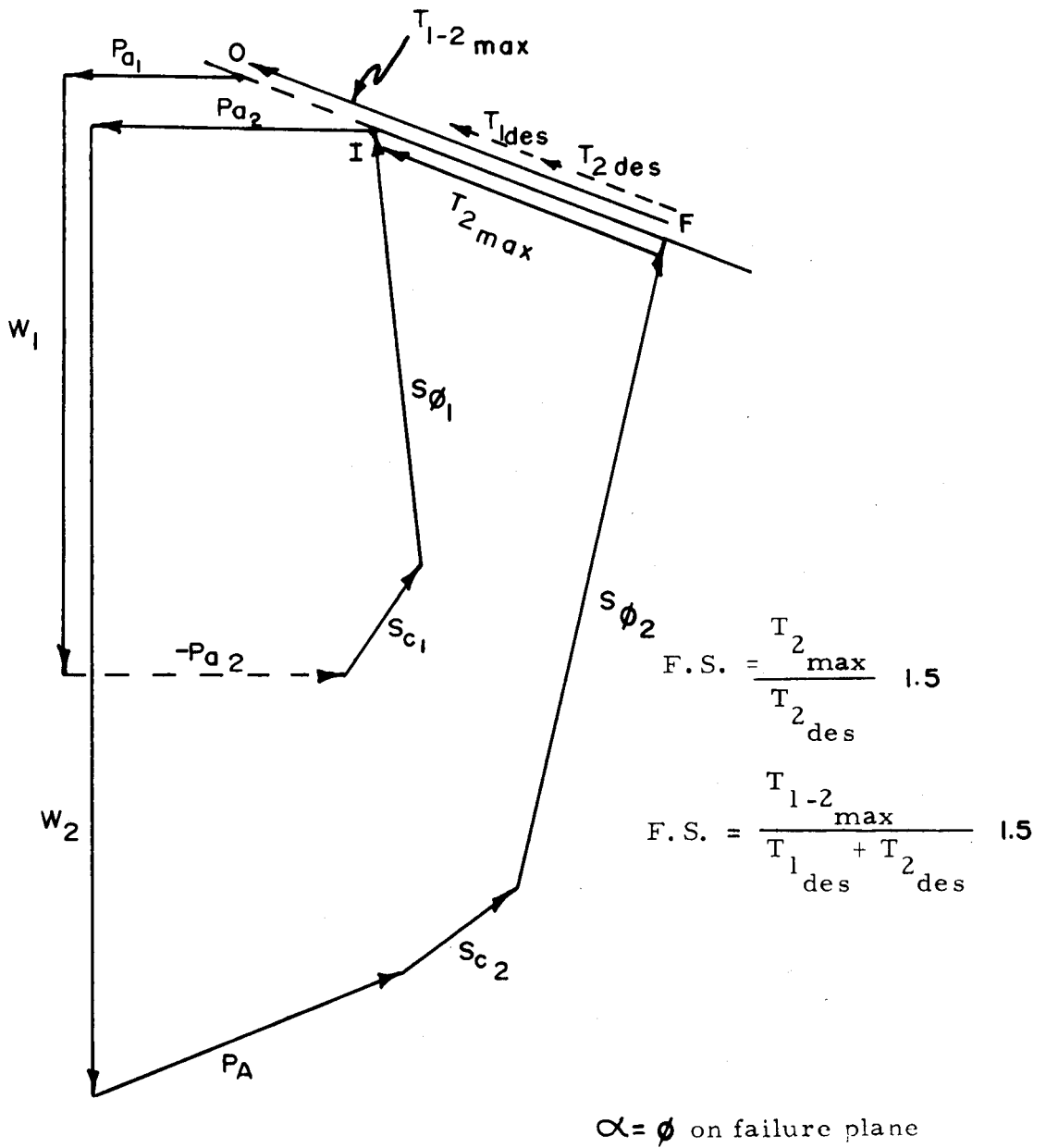


Figure 61. Vector diagram for a complex failure surface (safety factor in terms of the tieback force) (after Kranz, 1953, and Ranke and Ostermayer, 1968).

first free body (defined by points D_2, D_1, E_1, E_2, D_2) yields the forces, $P_{a1}, W_1, -P_{a2}, S_{c1}$, and S_{ϕ_1} . The vector diagram begins at point O in Figure 61. Vector summing of these forces results in an intermediate point, I. The second part of the vector diagram starts at this intermediate point and sums the vector forces acting on the second free body (points C, D_2, E_2, A, C). This vector diagram intersects the line of action of the tieback force at point F on the diagram (Figure 61). The stability of the earth mass is then defined in terms of the tieback forces,

$$\frac{T_{2 \max}}{T_{2 \text{ des}}} \quad \text{and} \quad \frac{T_{1-2 \max}}{T_{1 \text{ des}} + T_{2 \text{ des}}} \quad T_{2 \max}$$

is the vector distance between points F and I on the vector diagram while $T_{1-2 \max}$ is the distance between F and O. The recommended design criteria are

$$\text{both } \frac{T_{2 \max}}{T_{2 \text{ des}}} \quad \text{and} \quad \frac{T_{1-2 \max}}{T_{1 \text{ des}} + T_{2 \text{ des}}} \quad \text{to be greater than or equal to 1.5.}$$

This method has several apparent disadvantages. Among these is the rigid definition of the failure plane. However, because of the method's wide usage in Europe with satisfactory results, it is believed that the method can be to evaluate wall stability against overturning. The method should be used in combination with other methods evaluating sliding stability.

6.23.4 Sliding Wedge Analysis

General

A sliding wedge analysis involves evaluation of the driving and resisting forces acting on a designated free body. The forces are summed in a vector diagram to determine the magnitude of the unknown forces resulting in the calculation of the factor of safety. The factor of safety against sliding for a tied-back wall can be expressed in terms of the shear resistance of the soil or in terms of the passive soil resistance.

Free Body Diagram and Forces

A generalized free body diagram is presented in Figure 62. In this case the wall is part of the free body, and therefore, the wall forces, H_s and V , are included. Since the wall

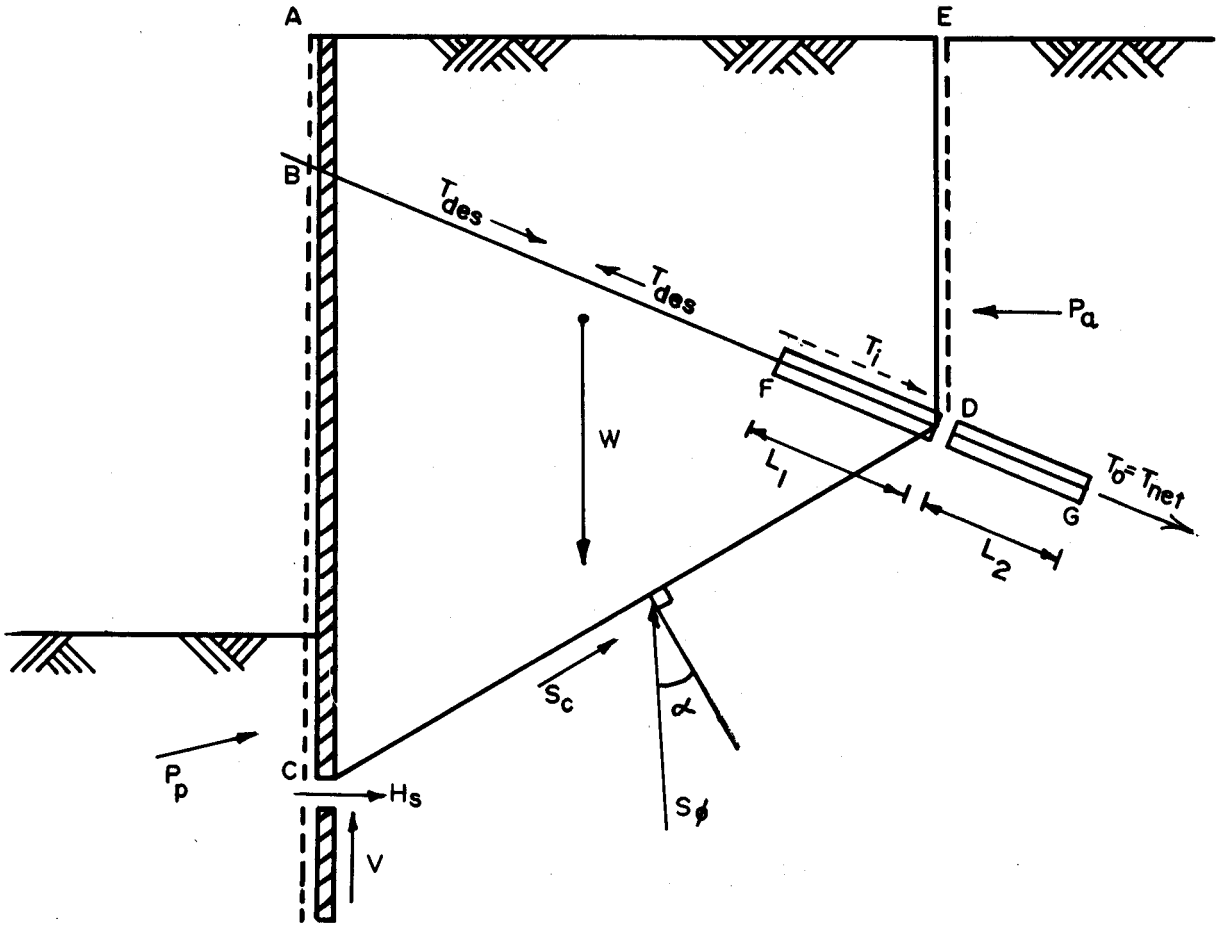


Figure 62. Free body diagram for a failure surface in a single anchor tieback system (free body outside of wall).

was not part of the free body in the method described before, the wall forces were not included in the analysis. Also, due to this change in the choice of the free body, the passive force must be considered; and the direction of the tieback force is changed.

The net tieback force, T_{net} is defined as the tieback force on the boundary of the soil mass which is equal to $T_{des} - T_i = T_o$; where T_{des} = design tieback force
 T_i = tieback force transferred to soil between points D & F
 T_o = tieback force at point D on boundary

The sliding wedge analysis does not specify the location of the failure surface as did the previous overturning analysis. Several failure surfaces can be analyzed for a given anchor geometry. The distribution of load in the anchor is assumed to be uniform over the entire length unlike the distribution assumed in the former analysis.

Safety Factor in Terms of Soil Strength

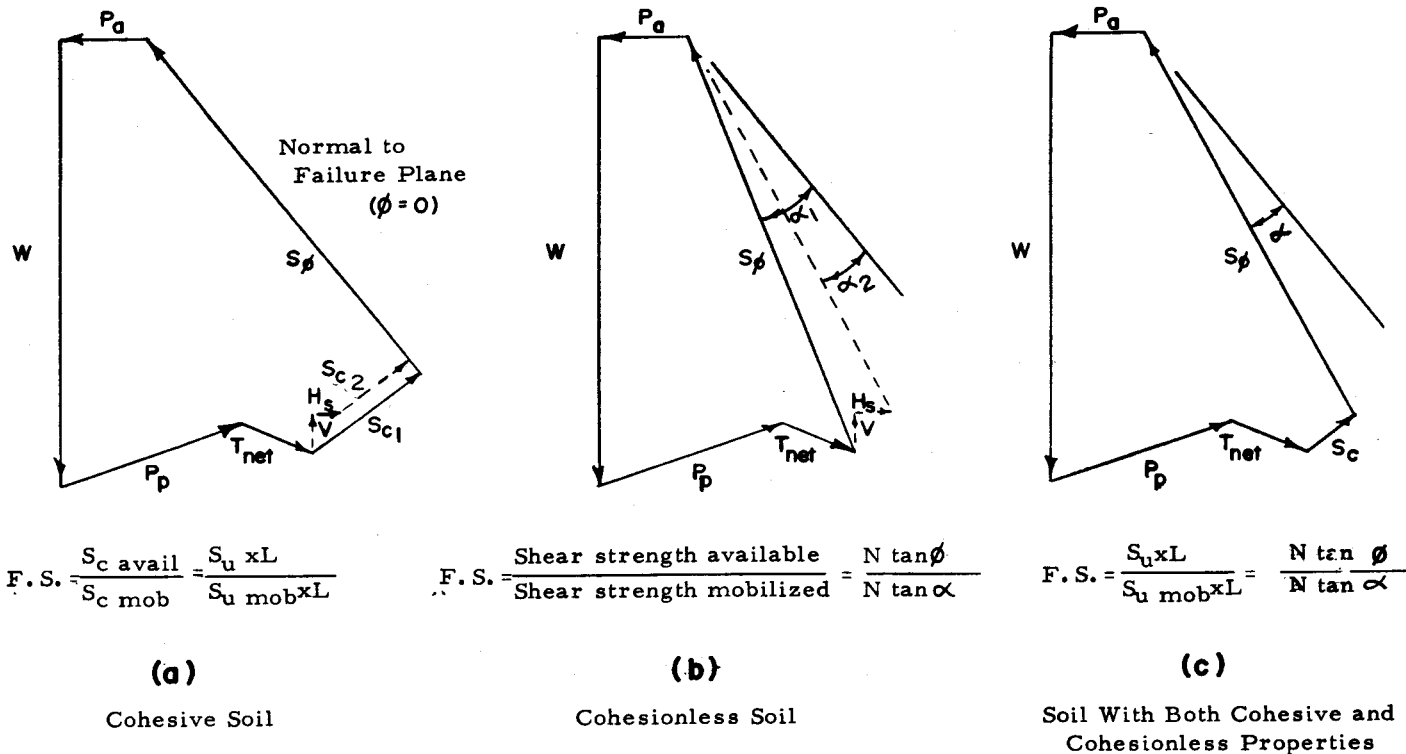
This section discusses a method of evaluating the stability of tied-back soil mass in terms of the available and mobilized

shear strengths, $F.S. = \frac{S_{avail}}{S_{mob}}$. Broms (1968) and Weissenbach

(1974a) also discuss similar methods of expressing the factor of safety of the soil mass.

a. Single Anchor. In order to evaluate the force, $T_{net} = T_o$ (see Figure 62), it is assumed that the anchor load is distributed evenly along the length of the anchor. Therefore, the forces T_i and T_o will depend upon the location of the failure surface with respect to the anchor zone. For example, the net anchor load, $T_{net} (=T_o)$, would be calculated to be $T_{des} \times \frac{L_2}{L_1 + L_2}$ and would act in the direction of the anchor.

Figure 63 shows the vector diagrams used to analyze a single anchor system. For a cohesive soil, the factor of safety can be defined as the ratio of the undrained shear strength to the mobilized shear strength along the failure surface:



Note: Consideration of Horizontal and Vertical Wall Forces Increases Factor of Safety

Figure 63. Vector diagrams used in analysis with factor of safety defined in terms of soil strength.

$$\frac{S_{c \text{ avail}}}{S_{c \text{ mob}}} = \frac{S_u \times L}{S_{\text{mob}} \times L}$$

where:

S_u = undrained shear strength of soil

L = length of failure surface

S_{mob} = mobilized shear strength

For cohesionless material the factor of safety will depend upon the angle (α) that the friction component of soil resistance is inclined at with respect to the normal to the failure surface. The angle is determined by closing the vector diagram shown in Figure 63 (b). The factor of safety is then defined as the ratio of the shear resistance available to shear resistance mobilized; or $F. S. =$

$\frac{\bar{N} \tan \phi}{\bar{N} \tan \alpha} = \frac{\tan \phi}{\tan \alpha}$ (where $\bar{N} = S_{\phi} \cos \alpha$). When a soil exhibits both cohesive and frictional components, the individual force components must be adjusted so that the same factor of safety is achieved for

each. For example, $\frac{S_{c \text{ avail}}}{S_{c \text{ mob}}}$ must equal $\frac{\tan \phi}{\tan \alpha}$. This will require

several iterations to determine the final vector diagram.

Although the vector diagram shows the forces, H_s and V , corresponding to horizontal wall load and vertical wall load, one can see that assuming these forces equal to zero is a conservative assumption. However, in special cases, where the wall is carried through a weak layer, these forces may be counted on to maintain stability and should be included.

b. Multiple Anchor Levels. Figure 64 illustrates a method of evaluating the stability of a three anchor level system. For simplicity, the example is for a cohesionless soil. In the vector diagram one can see that the individual tieback forces have been drawn to act along their angle of inclination. In the case of the second tieback level an even distribution of load along the tieback

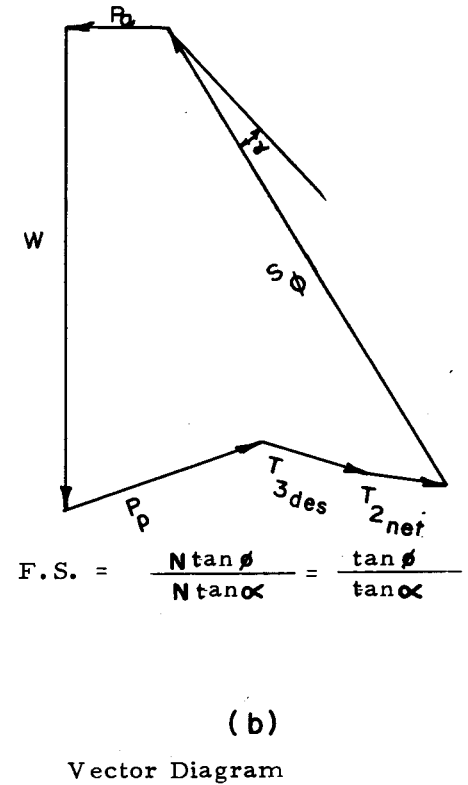
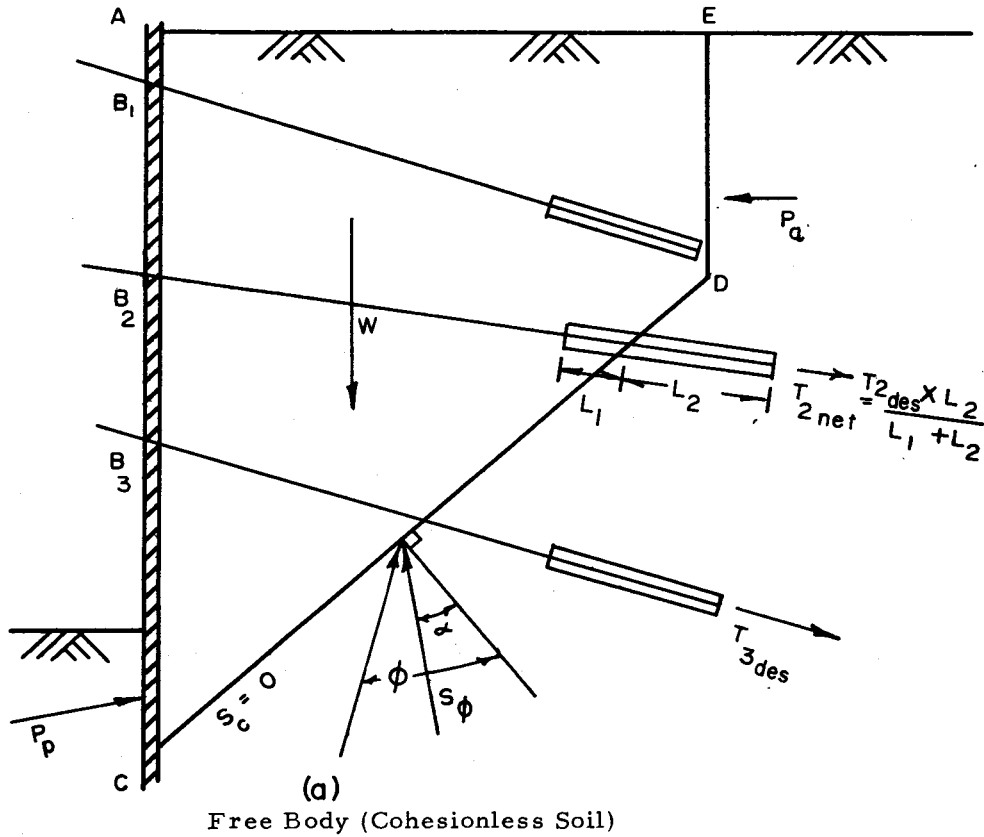


Figure 64. Analysis of a multiple level anchor system (safety factor in terms of soil strength).

length is assumed. These assumptions allow for the easy evaluation of many trial failure surfaces. The recommended factor of safety for this method of analysis is 1.5.

Safety in Terms of Passive Forces

This method is discussed by Broms (1968). The forces acting on the free body are as shown in Figure 62. However, the full soil strength is assumed to be mobilized for both S_c and S_ϕ with the passive force required for stability being determined by closing the vector diagram. The factor of safety is then defined as

$$F.S. = \frac{P_{avail}}{P_{mob}}$$

Broms (1968) recommends a minimum factor of safety of 1.5 when this analysis is used.

6.23.5 Internal Stability (Cofferdam) Analysis

This method is based on an analysis of the stability of cellular or double wall cofferdams as originally proposed by Terzaghi (1945) and as discussed in Teng (1962). The basic assumption of this analysis is that the prestressing action of the tiebacks embodies an earth mass. As shown in Figure 65, the earth mass can then be analyzed as a double wall cofferdam. Although the method is not conventionally used in practice, it does qualitatively illustrate some factors affecting tied-back wall stability and deformation.

As in the case of a beam in flexure, the maximum shear stress occurs on the neutral axis. A rigorous analysis of a cofferdam, however, indicates that both the location of the neutral axis and the direction of the maximum obliquity on the neutral axis are complex functions of the magnitude of external loading, the unit weight of backfill, and the strength and deformability of the backfill. Therefore, in engineering practice, the assumption is made that the maximum shear stress occurs on the vertical midplane of the cofferdam. Once this assumption is made, the magnitude of the total shear force can be determined from consideration of the loaded half of the cofferdam as a free body. The shear force thus computed is:

$$V_{max} = \frac{3M}{2B}$$

where:

$$M = \text{moment} = P_a \times H / 3 = \frac{\gamma H^3 K_a}{6}$$

$B = \text{effective width}$

$$K_a = \tan^2(45 - \phi/2)$$

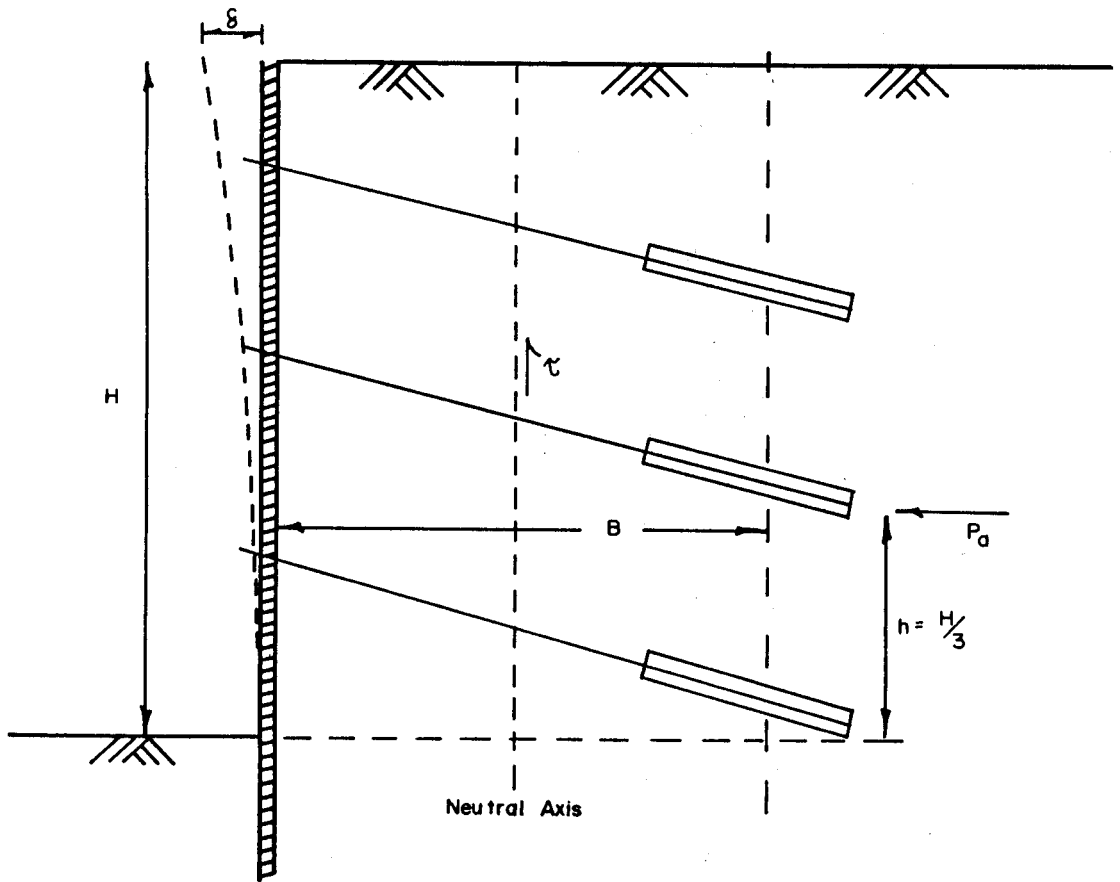


Figure 65. Sketch of equivalent cofferdam for tied-back wall.

The shear resistance at any point on the assumed failure plane is:

$$s = \bar{c} + \bar{\sigma}_h \tan \phi$$

where:

s = shear resistance

\bar{c} = cohesion intercept

ϕ = angle of internal friction (effective stress parameters)

$\bar{\sigma}_h$ = effective normal stress on the failure plane (horizontal stress)

Having once assumed a vertical failure plane, there now becomes a unique relationship between $\bar{\sigma}_v$ and $\bar{\sigma}_h$ (the vertical horizontal effective stresses respectively). For a cohesionless soil this relationship, which can be derived from the Mohr's circle at failure, becomes:

$$\bar{\sigma}_h = \frac{\cos^2 \phi}{2 - \cos^2 \phi} \quad \bar{\sigma}_v = \frac{1}{1 + 2 \tan^2 \phi} \bar{\sigma}_v = K \bar{\sigma}_v$$

where:

K is a coefficient of lateral earth pressure.

The shear strength at any point is therefore:

$$s = \bar{\sigma}_h \tan \phi = \frac{\tan \phi}{1 + 2 \tan^2 \phi} \bar{\sigma}_v$$

The total shear resistance for a backfill of unit weight, γ , and height, H , is as follows:

$$S = 1/2 \gamma H^2 \frac{\tan \phi}{1 + 2 \tan^2 \phi}$$

The factor of safety against internal shear failure therefore becomes:

$$F.S. = \frac{\text{available shear resistance}}{\text{maximum shear force}} = \frac{S}{V_{\max}}$$

From the expressions for S and V_{\max} that were previously developed, the factor of safety may be expressed as follows:

$$F.S. = \frac{1}{K_a} \frac{1/2 H^2 \frac{\tan \phi}{1 + 2 \tan^2 \phi}}{3/2 (H^3/6) (1/B)} = \frac{2B}{H} \frac{\tan \phi}{1 + 2 \tan^2 \phi} \frac{1}{\tan^2 (45 - \phi/2)}$$

This suggests that the factor of safety is directly proportional to the ratio of effective width (B) to height (H). If the analogy between a cofferdam and tied-back walls holds, then this also suggests that the stability of the tied-back wall increases with the length of the tiebacks.

Once a horizontal prestress is applied to the cofferdam, the unique relationship between the horizontal and vertical stress is violated. The ramification of this is that the failure plane can no longer be considered a vertical plane (if indeed it ever was vertical). A general relationship between the shear strength on the failure plane at failure (S_{ff}), the vertical effective stress ($\bar{\sigma}_v$), and the ratio of horizontal to vertical effective stress ($\bar{\sigma}_h / \bar{\sigma}_v = K$) can be derived from the Mohr circle at failure. This relationship is:

$$S_{ff} = \bar{\sigma}_v \sin \phi \cos \phi \left(\frac{1 + K}{2} \right)$$

This relationship suggests that as the ratio of horizontal to vertical effective stress increases the shear strength at failure increases. Again, if the analogy holds, this further suggests that stability increases with increased tieback prestress. This conclusion qualitatively verifies the analyses of Section 6.23.3.

Considering deformation in view of the cofferdam analogy two points become apparent. First, since the stress/strain behavior of soil is non-linear, it follows that if lesser portion of the strength is mobilized (i. e. factor of safety increased) the deformations will be less. Second, the deformability of soil is a function of confining pressure and as confining pressure increases, the modulus of deformation increases. Lambe and Whitman (1969) give the following approximate relationship between modulus and average confining pressure:

$$E = \bar{\sigma}_v \frac{1 + 2K_o}{3}$$

where:

E = Modulus of deformation

$\bar{\sigma}_v$ = Vertical effective stress

It can be seen that, as K_o increases as a result of prestressing, the modulus of deformation increases.

Applying these facts to the analogy suggests finally that both an increase in tieback length and an increase in tieback prestress will reduce deformations.

6.23.6 Discussion

The evaluation of the stability of a tied-back earth mass is a trial and error process involving the use of several analytical techniques. These techniques are based upon the forces acting on a free body and have been successfully used in tied-back wall design.

Circular arc stability analyses are used to evaluate the stability of the soil mass lying outside the tiebacks and below the wall. Sliding wedge analyses can be used to search out critical failure surfaces within the soil mass retained by the tiebacks.

The total evaluation of the stability will consist of the entire spectrum of possible failures to insure that the tiebacks are appropriately located and sufficiently long. No one method is applicable to all situations. All must be considered for a specific case.

6.24 Tieback Anchorage Design Considerations

6.24.1 General

The previous tieback discussions addressed the importance of movements and overall stability of the structure on tieback design. The design techniques presented previously are related to controlling deformations and maintaining a suitable factor of safety against failure for the entire soil mass and wall but do not deal with individual anchor resistance.

This section deals with the mechanics of anchor load transfer to the soil or rock formation, the determination of anchor load capacity, and the zones for anchor locations. Later sections

discuss tendon and grout considerations. A separate section of this chapter is devoted to the methods of installing tiebacks.

6.24.2 Suitable Anchorage Strata

Experience has shown that virtually all rock types can be used as anchorage zones; however, not all soil deposits are suitable. The following list summarizes the appropriateness of various soil and rock types for location of anchors.

1. Soft to medium clays are generally not suitable anchorage strata.
2. Stiff clays may or may not be suitable for anchorages depending upon the project particulars (allowable movements and loads).
3. Loose cohesionless soils have provided successful anchorages in some cases; however, other cases indicate that these soils are not satisfactory.
4. Very stiff to hard clays and medium to very dense granular soils are preferred anchorage strata.
5. Virtually all rock types provide suitable anchorages.

6.24.3 Location of Anchors

One of the criteria for determining the location of tiebacks is that the anchors be founded behind any zone of possible slippage. Internally, this would mean behind the "active wedge" zone. With respect to the entire soil mass, the anchors must be located at a sufficient distance behind the wall to ensure the overall stability. Section 6.23 deals with problems of overall wall and soil stability.

In U.S. practice, anchors are generally located beyond a line extending at a 30° - 45° slope to the wall from the base of the excavation to the ground surface (see Figure 66). In specific instances, the angle may be even greater as was the situation in the case illustrated in Figure 67 (ENR, 1973a). The rock was heavily jointed with a principal joint inclination at an angle of 33° to the horizontal. The tiebacks were anchored behind the possible zone of slippage. Recent cases indicate a more common use of 35° - 40° as an angle of inclination for the slip surfaces in granular soil deposits. However, anchors are often founded well behind 45° slip lines (Shannon and Strazer, 1970; Clough, et al, 1972) in cohesive soil deposits.

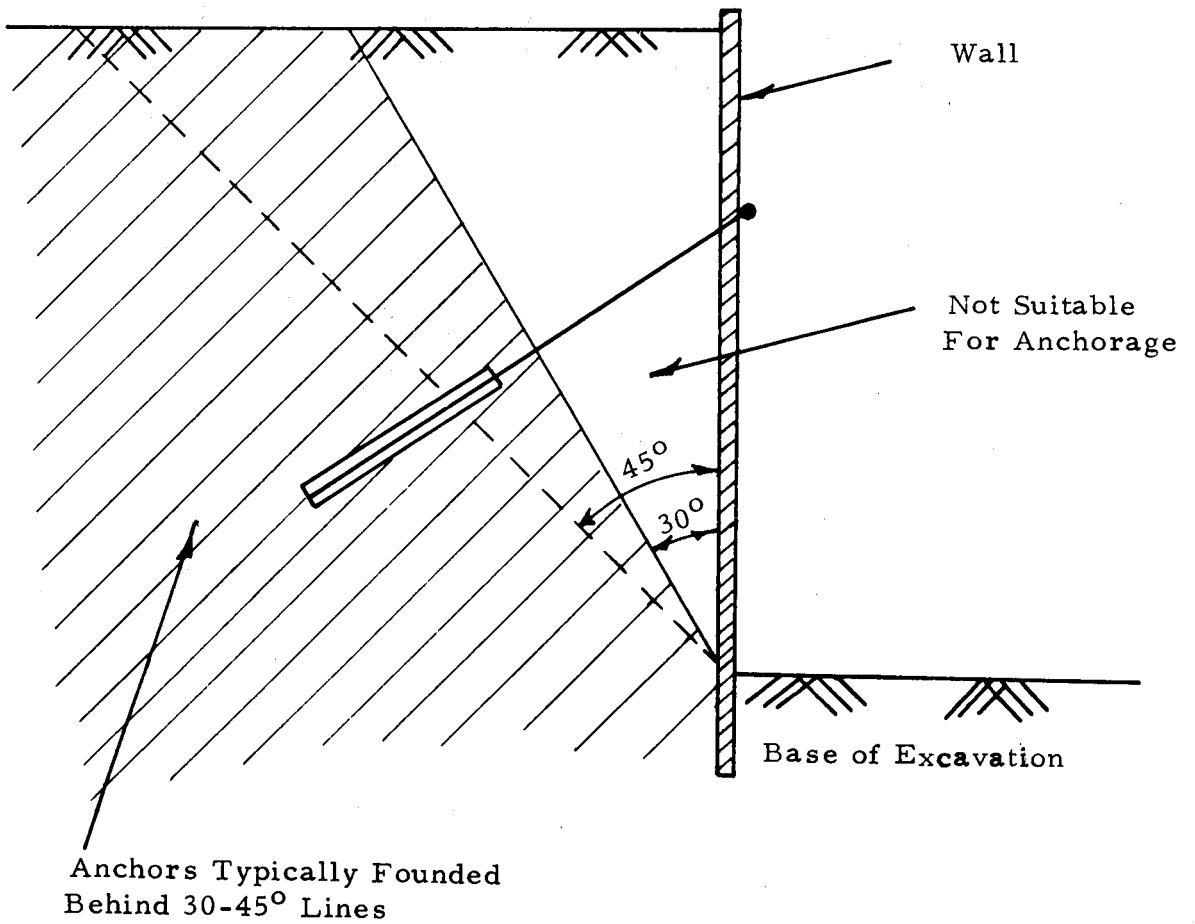
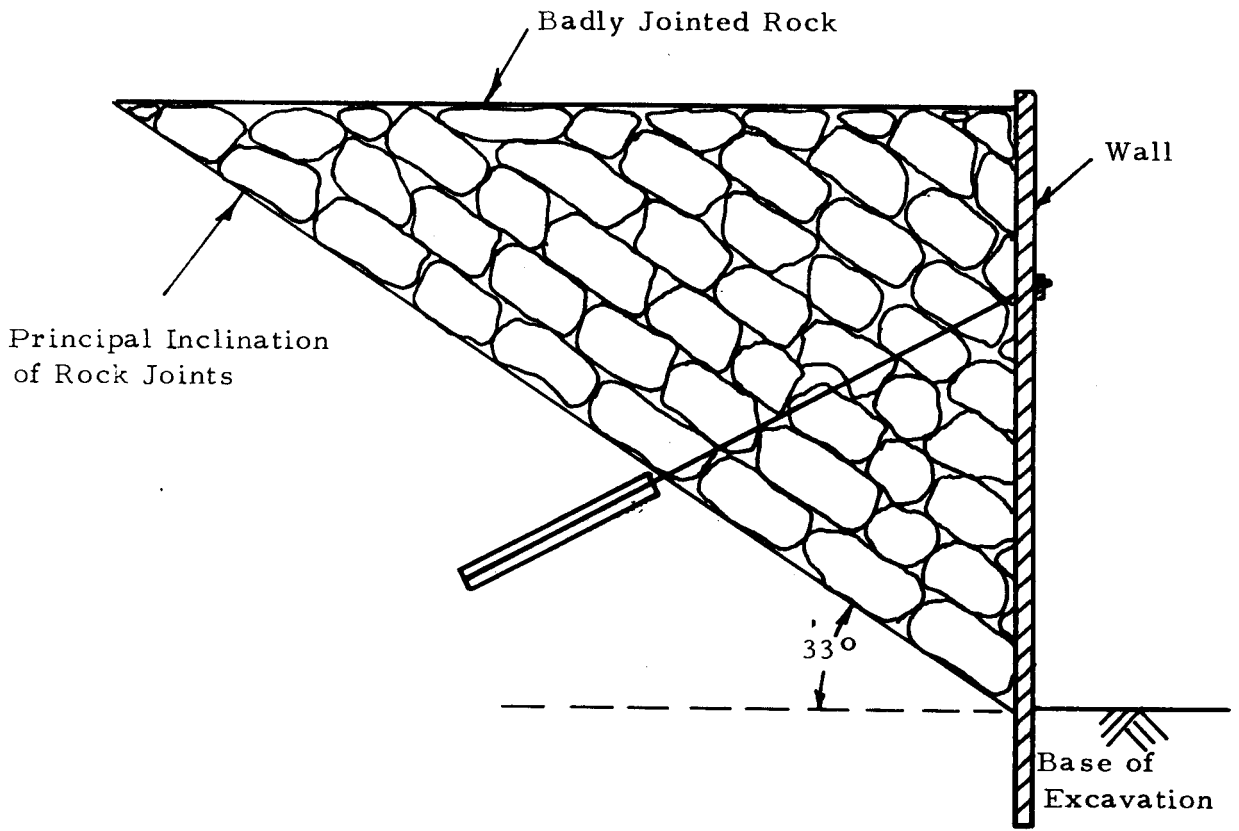


Figure 66. Typical location of anchors.



From ENR, January 11, 1973

Figure 67. Example case where geology controls anchor location.

European practice indicates more uniform tieback lengths with lower tiebacks somewhat longer than in the U. S.

6.24.4 Soil Anchors

General

The procedure used in selecting and setting length and load criteria for a soil anchor includes the following:

1. Initial estimate of anchor load based on past experience with soil and anchor.
2. In areas of relatively greater uncertainty, the procedure may also include pull-out testing of several anchors at the site to determine the appropriate design parameters for production anchors (i. e. load capacity per lineal foot of anchor).
3. Field testing of all anchors to ensure adequacy.

The most important aspect of any anchor installation is the prooftesting of the anchors after installation. Each anchor is loaded beyond the design load to ensure its adequacy to resist that load. Field testing requirements are discussed later in this chapter.

The theoretical and empirical load relationships presented in this section are intended to aid the designer in estimating load capacity of anchors, in interpreting field test data, and in understanding the mechanics of anchor load transfer. The relationships are not intended as a substitute for experience nor do they obviate the need for field testing. Field testing is required for virtually all anchor installations.

Soil anchors can be grouped into two principal categories: 1) large diameter anchors and 2) small diameter anchors. Generally, the larger diameter anchors are used in cohesive soils while small diameter anchors are more commonly used in granular soils. The following paragraphs briefly describe the basic anchor installation techniques and ranges in anchor size. Section 6.30 of this chapter describes the construction of anchors in greater detail.

a. Large Diameter Anchors

Large diameter anchors can be either straight shafted, belled, or multi-belled anchors. Belled anchors were among

the first anchors used in this country, but in recent years it has been found to be more economical to use straight shafted anchors. Multi-belled anchors have been used in the United Kingdom and South Africa.

In the United States, anchor shaft diameters are usually a minimum of 12" (30 cm) for straight shaft and belled anchors while multi-belled anchors may have shafts as small as 4" (10 cm) in diameter. In general, these anchors are installed in cohesive soils that are sufficiently competent to maintain open, unsupported holes or holes that are supported by hollow flight augers. Belled or multi-belled anchors must be installed in holes that will remain open when unsupported. Grout or concrete is then pumped (usually at low pressure) into the hole, and the anchor is formed. Figure 68 schematically illustrates what these anchors would look like.

b. Small Diameter Anchors

Small diameter anchors generally vary from 3" (7.5 cm) to 6" (15 cm) in size and are most frequently installed in granular soils. Anchors of this type are generally formed by grouting the anchor zone under large pressures. A special small diameter anchor which has the capability of grouting the anchor zone several times (regroutable anchor) has also been developed.

Often temporary casing is used to support the hole during its formation. After the hole has been formed, grout is injected under high pressure as the casing is withdrawn in stages. The final size of the anchor will depend upon the extent to which the grout can penetrate (pervious soils) or compact the soil. The mechanics of load transfer depend to some extent on the soil type, and these features will be discussed in more detail in the following sections. Figure 69 schematically illustrates what these anchors would look like.

Load Transfer Mechanisms

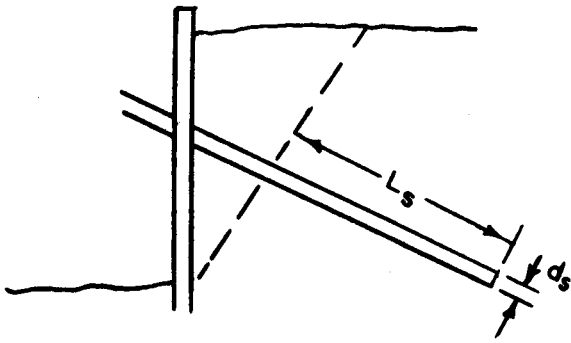
The anchor transfers the tieback load to the soil through two basic mechanisms: 1) frictional resistance at the anchor-soil interface and 2) end bearing where anchors have a larger diameter than the initial drilled shaft diameter. The actual load transfer mechanism(s) varies with anchor and soil type. The following list briefly describes the load transfer mechanisms for different anchor types.

1. Frictional anchors are those anchors in which the load transfer occurs along the grout-soil interface. These include both large and small diameter, straight-shafted anchors.

ESTIMATED LOAD FOR
ANCHORS IN COHESIVE SOIL

$$P_u = \alpha S_u L_s \pi d_s$$

$$\alpha = 0.3-0.5$$



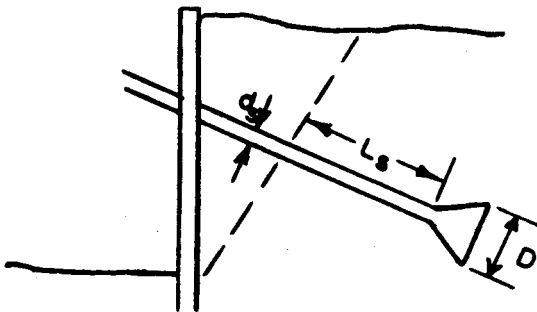
(a)

Friction Anchor

$$P_u = \alpha S_u L_s \pi d_s + \frac{\pi}{4} (D^2 - d_s^2) N_c S_u$$

$$\alpha = 0.3-0.5$$

$$N_c = 9$$



(b)

Belled Anchor

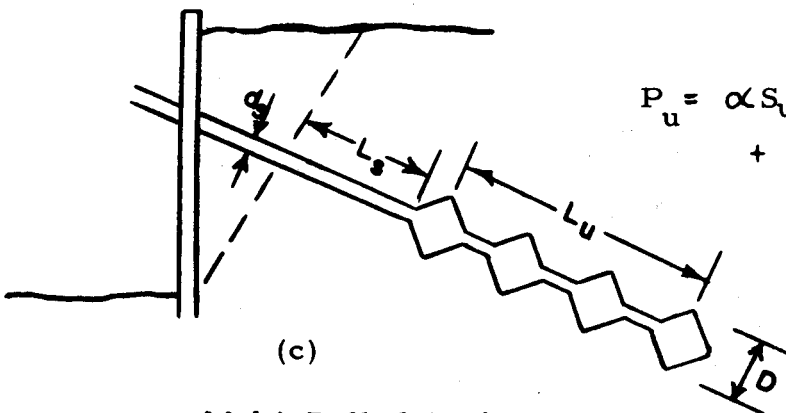
$$P_u = \alpha S_u L_s \pi d_s + \frac{\pi}{4} (D^2 - d_s^2) N_c S_u$$

$$+ \phi S_u \pi D L_u$$

$$\alpha = 0.3-0.5$$

$$\phi = 0.75-1.00$$

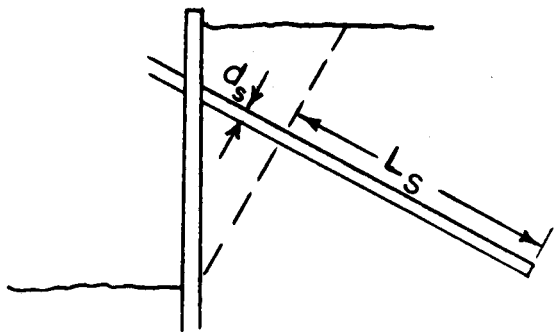
$$N_c = 9$$



(c)

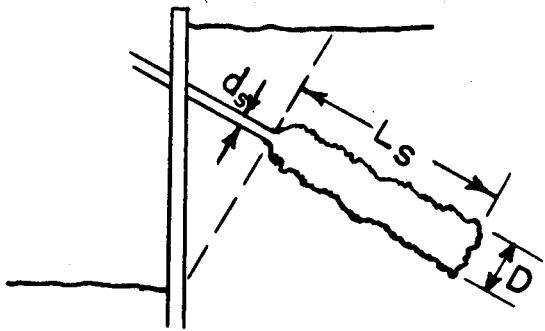
Multi-Belled Anchor

Figure 68. Schematic representation of large diameter anchors.



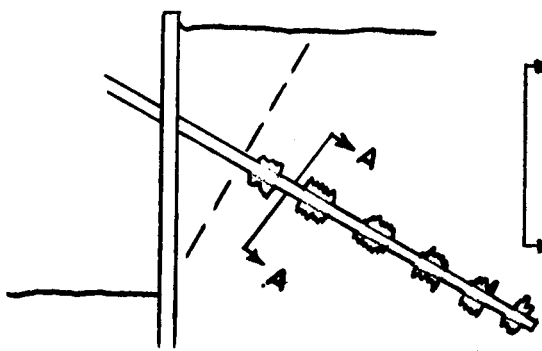
(a)

Friction Anchor
(No Grout Penetration)



(b)

Bulb Anchor
(Grout Penetration)



(c)

Regroutable Anchor
(Local Grout Penetration)

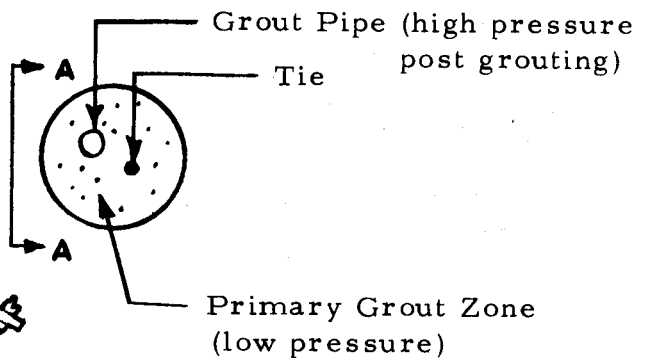


Figure 69. Schematic representation of small diameter anchors.

2. Bulb anchors derive their resistive strength from frictional resistance along an enlarged anchor diameter and from end bearing due to the larger diameter. These anchors can be formed in very previous granular soils with grout injected under pressure. (See Figure 69).

3. Belled anchors are both end bearing (belled portion) and frictional (shaft) anchors.

4. Multi-belled anchors are the same as belled anchors except that additional load transfer occurs due to the resistance of the soil between the tips of the bells.

5. Regroutable anchors transfer load through both frictional resistance and bearing. The bearing resistance is developed by the local penetration of grout through ports, generally about 3 feet apart in the grout pipe.

Table 7 summarizes the basic anchor types with respect to the soil types in which they can be used and the load transfer mechanism.

Large Diameter, Straight-Shafted Anchors

Large diameter, straight-shafted anchors transfer load to the soil by means of the frictional resistance developed along the grout-soil interface. Although these anchors can be formed in both cohesive and cohesionless soils, the anchors are most commonly used in stiff to hard clays. The distinguishing feature of this anchor type is that the final anchor diameter is essentially the same as the initial augered anchor diameter (see Figure 68).

Grouting of the anchor zone is generally performed by placing concrete at low pressures. However, it is possible to use grouting pressures of up to approximately 150 psi (1035 kN/m²) when hollow stem augering equipment is used. The main effect of grouting under pressure in these soil types is to recompact any zones that may have been loosened during the excavation stage. Grouting under pressure also insures that no voids will develop in the anchor zone.

The methods used to estimate the ultimate pullout capacity of friction anchors are largely based on the observed performance of these anchors and are, therefore, empirical in nature. The following equation is an idealized but common expression for the pullout capacity, P_u , of friction anchors in cohesive soils:

Table 7. Summary of tieback types and applicable soil types.

Method	Diameter (inches)		Gravity Concrete	Grout Pressure (psi) (1)	Suitable Soils for Anchorage	Load Transfer Mechanism
	Shaft Type	Bell Type				
1. LOW PRESSURE						
Straight Shaft Friction (Solid stem auger)	12-24" (30 - 60cm)	NA	A	NA	Very stiff to hard clays Dense cohesive sands	Friction
Straight Shaft Friction (Hollow stem auger)	6-18" (15 - 45cm) (12-14" most common)	NA	NA	30 - 150 (200 - 1035kN/m ²)	Very stiff to hard clays Dense cohesive sands Loose to dense sands	Friction
Underreamed Single Bell at Bottom	12-18" (30 - 45cm)	30-42" (75 - 105cm)	A	NA	Very stiff to hard cohesive soils Dense cohesive sands Soft rock	Friction and bearing
Underreamed Multi-bell	4-8" (10 - 20cm)	8-24" (20 - 60cm)	A	NA	Very stiff to hard cohesive soils Dense cohesive sands Soft rock	Friction and bearing
2. HIGH PRESSURE- SMALL DIAMETER						
Non-regroutable (2)	3-8" (7.5 - 20cm)	NA	NA	150 (1035kN/m ²)	Hard clays Sands Sand-gravel formations Glacial till or hardpan	Friction or friction and bearing in permeable soils
Regroutable (3)	3-8" (7.5 - 20cm)	NA	NA	200-500 (1380 - 3450kN/m ²)	Same soils as for non-regroutable anchors plus: a) stiff to very stiff clay b) varied and difficult soils	Friction and bearing

(1) Grout pressures are typical

(2) Friction from compacted zone having locked in stress.
Mass penetration of grout in highly pervious sand/gravel forms "bulb anchor".

(3) Local penetration of grout will form bulbs which act in bearing or increase effective diameter.

A - applicable

NA - not applicable

$$P_u = \alpha S_u \pi d_s L_s$$

where:

- d_s = diameter of anchor shaft
- L_s = length of anchor shaft
- S_u = undrained shear strength of soil
- α = reduction factor in S_u due to disturbance, etc.

The reduction factor, α , is applied to reduce undrained shear strength to a value consistent with the measured field performance of friction anchors. Reported values of α vary from 0.3 to 0.5. Figure 70 is a plot of the reduction factor, α , versus the undrained shear strength of the soil. The values of α plotted in the figure are those derived from measured values from friction piles as presented in Peck, Hanson, and Thornburn (1974). The values of α typically used in estimating tieback load are also shown.

Belled Anchor

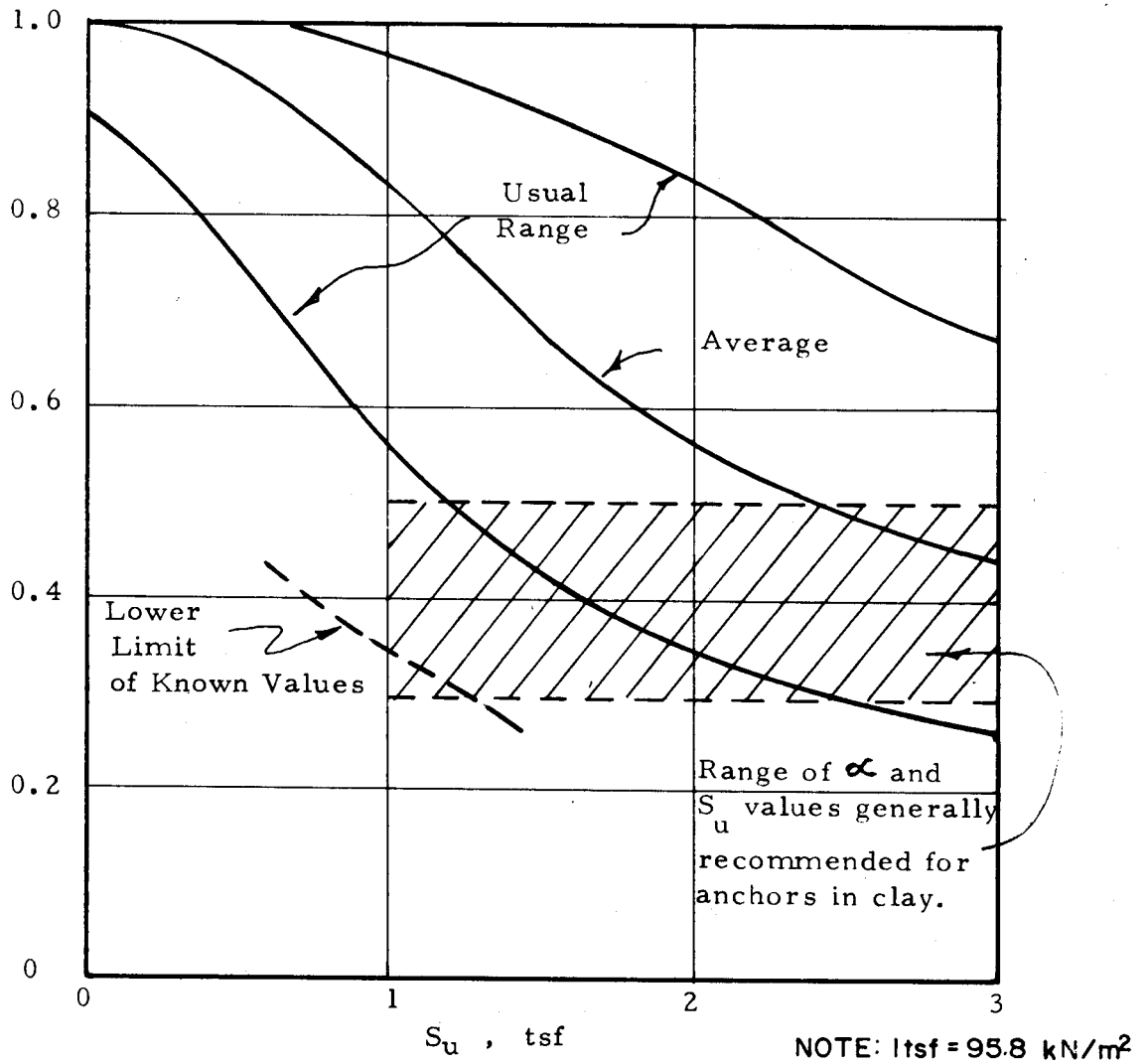
A belled anchor has two components contributing to its resistive strength, frictional resistance and end bearing resistance. The anchor shaft provides the frictional resistance while the bell at the base provides the end bearing resistance. Figure 68 illustrates the geometry of a belled anchor.

The equation used to estimate the ultimate load of a belled anchor includes both the frictional and end bearing components of resistive force. The equation presented in this section is an equation proposed by Littlejohn (1970a) for multi-belled anchors:

$$P_u = \alpha S_u \pi d_s L_s + \pi/4 (D^2 - d_s^2) N_c S_u$$

where:

- d_s = shaft diameter
- D = bell diameter
- L_s = length of straight shaft
- S_u = undrained shear strength of soil
- α = reduction factor for shear strength ($\alpha = 0.3-0.5$)
- $N_c = 9$



From Peck, Hanson & Thornburn (1974)

Figure 70. Reduction factor in S_u from observed capacity of friction piles.

Belled anchors can only be formed in competent cohesive soils since the hole must be capable of remaining open without support.

Multi-Belled Anchors

In addition to frictional resistance along the shaft and end bearing at the bell, a resistive component is developed between the tips of the underreams. The anchor consists of a shaft with a series of bells located at varying distances along the shaft. Figure 68 illustrates the geometry of a multi-belled anchor.

Typically, underream tips are spaced at 1.5 to 2.0 times the bell diameter with the bell diameter 2.0 to 3.0 times the shaft diameter. With these ranges in dimensions it has been observed that failure in the belled portion of the anchor will occur in the soil between the tips of the underreams. The following equation is proposed by Littlejohn (1970a) for the use in estimating the ultimate anchor load.

$$P_u = \alpha S_u \pi d_s L_s + \pi/4 (D^2 - d_s^2) N_c S_u + \beta S_u \pi D L_u$$

where:

d_s , D , L_s , N_c , S_u , and α are as before

L_u = length of underream portion of anchor

β = reduction factor for undrained shear strength in soil between underream tips

Since less disturbance of the soil between the underream tips occurs during the formation of the underreams than for a shaft, the value of β is greater than the value of α . Values of β ranging from 0.75 to 1.00 are typically used depending upon the amount of disturbance during anchor formation (Littlejohn, 1970a; Bassett, 1970; Neely and Montague-Jones, 1974). Underreamed anchors have been used primarily in very stiff clay and soft rock.

Small Diameter Anchors

The following discussion presents both theoretical and empirical methods for estimating anchor capacity. The former are presented primarily to gain a qualitative understanding of the load transfer mechanism. They are crude at best. Therefore, prime reliance must be placed upon empirical observations and experience.

No Grout Penetration in Anchor Zone

In soils ranging from clays to sands and gravels, except very coarse, practically silt-free, granular soils, cement grout is simply too coarse to penetrate the voids of the soil. Therefore, the effect of grouting anchors under pressure without grout penetration into the voids of the soil is to form a compacted zone immediately around the anchor which theoretically locks-in normal stresses acting on the anchor. Pressure grouting may cause a small increase in anchor diameter, but it is assumed that this small change in diameter results in a negligible increase in contact area. Grouting under excessively high pressures may also cause fracturing of soil and formation of discrete lobes or tongues of grout. Generally, excessive grouting pressures are avoided.

Broms (1968) and Littlejohn (1970a) noted that the ultimate capacity of anchors is often dependent upon the pressures used to inject the grout. As a result, the following equation has been used to estimate load for friction anchors in sand:

$$P_u = p_i \pi d_s L_s \tan \phi_e$$

where:

d_s = diameter of anchor

L_s = length of anchor

ϕ_e = friction angle between grout and soil

p_i = grout pressure

When high pressures are used to grout anchors in cohesive soils, the effect has been to increase the ultimate load capacity by virtue of an increase in the value α . However, the increase in α is generally small.

An alternative equation proposed by Littlejohn (1970a) in fine to medium sands is:

$$P_u = n_1 L_s \tan \phi_e$$

where:

$$n_1 = 8.7 - 11.1 \text{ k/ft (127 - 162 kN/m)}$$

b. Grout Penetration in Anchor Zone

In clean, coarse sands and gravels the cement grout flows into the voids and forms an effective anchor diameter which often is significantly larger than the original anchor diameter. Anchors of this type transfer load to the soil in both bearing and frictional resistance. Figure 69 illustrates, schematically, how such a bulb anchor might appear.

The methods used to calculate the ultimate load for bulb anchors are even more crude than those for friction anchors. The following defines the method proposed by Littlejohn (1970a) to predict the ultimate load in bulb anchors:

$$P_u = A \bar{\sigma}_v D L_s \tan \phi_e + B \bar{\sigma}_{v@end} \pi/4 (D^2 - d_s^2)$$

where:

d_s , D , L_s , and ϕ_e are as before

$\bar{\sigma}_v$ = average vertical effective stress over entire anchor length

$\bar{\sigma}_{v@end}$ = vertical effective stress at the anchor end closest to wall

A = contact pressure at anchor-soil interface
effective vertical stress ($\bar{\sigma}_v$)

Littlejohn reports typical values of A ranging between 1 and 2

B = a bearing capacity factor similar to N but smaller in magnitude. A value of q

$$B = \frac{N}{1.3 - 1.4} \text{ is recommended provided } h/D \geq 25; \text{ where } h \text{ is the depth to the anchor.}$$

There are many difficulties involved in trying to use this equation to predict anchor capacities. The values of D , A , and B cannot be predicted accurately, therefore an empirical equation has been proposed by Littlejohn (1970a) for use in these soil types.

$$P_u = n_2 L_s \tan \phi$$

where:

- ϕ = internal angle of friction of the soil
- L_s = length of shaft
- n_2 = 26 - 40 kips/ft (379 - 584kN/m) for $L = 3 - 12'$
(0.9 - 3.7m), $D=15 - 24''$ (400-600 mm),
depth of anchor = 40 - 50' (12.2 - 15.1 m)

More empirical relations based on anchors tested to failure will be discussed in greater detail in a later part of this section.

c. Regroutable Anchors

Regroutable anchors can be installed in virtually all soil types and are an extremely versatile anchoring system. The distinguishing feature of these anchors is that if the anchor fails to hold the initial load application, it can be regouted at higher pressures until the anchor can carry the higher loads. The details of regroutable anchor installation are discussed in Section 6.30; however, a brief description of the anchors follows.

A regroutable anchor requires the drilling in or driving of a casing to the desired length. After the holes is cleaned, a tie member attached to a grout pipe is placed in the hole. A cement grout is pumped into the hole (generally at low pressures) and allowed to set. After this initial grout has set, the implanted perforated grout pipe is then used to grout the zones along the grout pipe. The high pressures of the grout crack the existing grout and allow the grout to penetrate the soil mass forming bulbs. The regrouting process can be repeated several times until the desired anchorage capacity is achieved. Figure 69 is a schematic illustration of a regroutable anchor.

Usually, each zone is isolated by a pair of packers and grouted separately. Under some circumstances, the separate zones are not isolated individually; rather, the entire grout pipe is pressurized. In all cases, the grout pipe is cleaned out to permit regrouting.

The anchor capacity of a regrowable anchor cannot easily be estimated using theoretical formulae. Also, if the anchor proves inadequate under proof loading, it can be regrowed, and so the estimates of load carrying capacity do not need to be as precise as for other anchors. In practice empirical correlations and observations are used primarily to estimate anchor load and to determine the design for regrowable anchors.

d. Empirical Observations

Since the formulae presented in this section are relatively crude, theoretical attempts to estimate anchor load, several studies have been performed to try to relate anchor capacity directly to soil type, grouting pressure, anchor diameter, and anchor length. Littlejohn (1970a) presents some preliminary values for use in estimating anchor load for specific soil conditions. Some of these values have been presented in the previous sections.

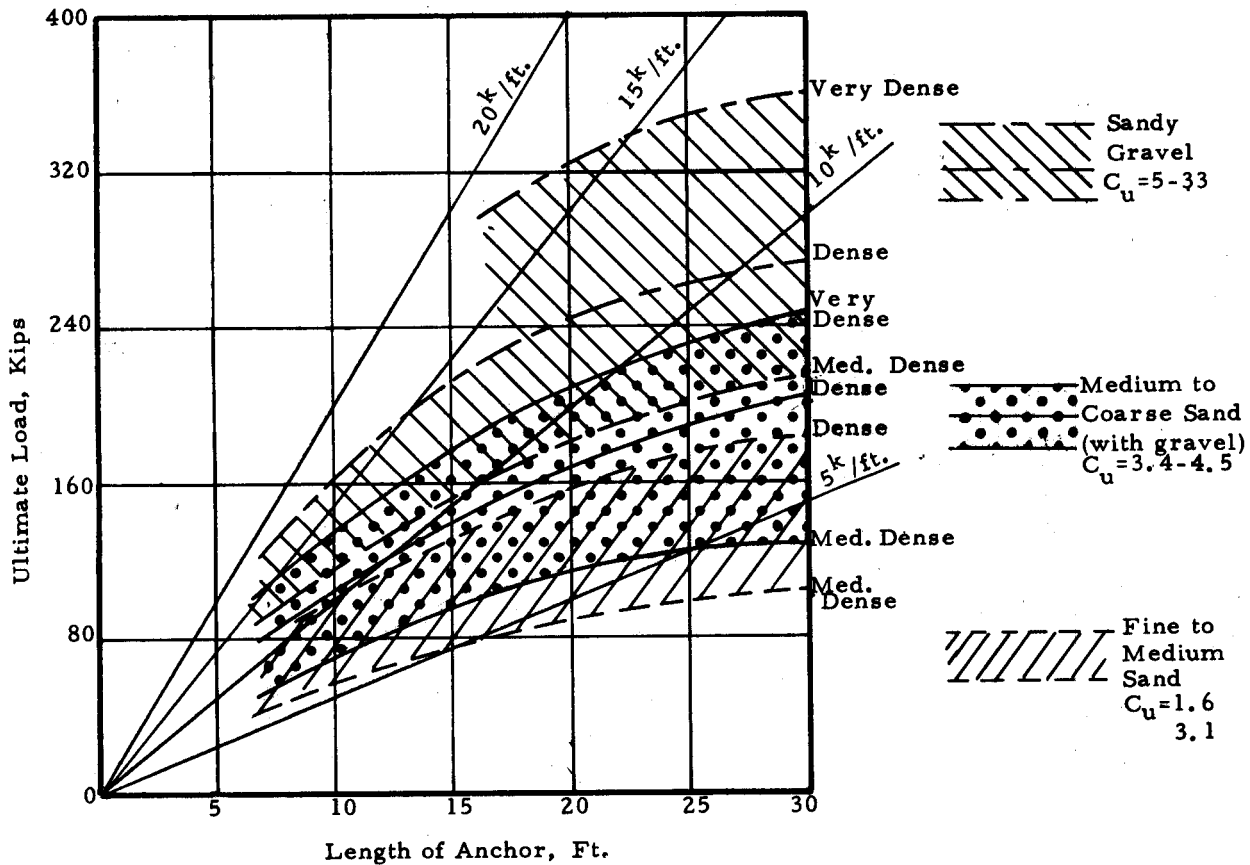
Ostermayer (1974) has recently reported the results of over 300 anchor pullout tests stemming from twenty-five years of German practice. Ostermayer has developed a series of empirical relationships that can be used to estimate anchor capacities on the basis of observed anchor performance.

Typically, the anchors studied were four to six inches (10 - 15 cm) in diameter and thirteen to twenty-six feet (4 - 8m) long in the grouted zone. Grout pressures of at least 150 psi (1035 kN/m²) are applied in cohesionless soils.

e. Cohesionless Soils

Figure 71 is an empirically developed plot showing the load carrying capacity of cohesionless soils considering relative density, gradation, and anchor length. The data show:

1. The carrying capacity increases with well-graded soils and with density.
2. The carrying capacity increases with increasing length of grouted zone, but at a decreasing rate. The author suggests that a length of twenty to twenty-five feet (6-7.5m) is about optimum. Above that, the increase in carrying capacity is substantially reduced.



NOTE: 1ft. = .305 m
 1in. = 2.54cm
 1k/ft = 14.6 kN/m

Diameter of Anchor 4"-6"
 Depth of Overburden \geq 13 feet

Figure 71. Load capacity of anchors in cohesionless soil showing effects of relative density, gradation, uniformity, and anchor length (after Ostermayer, 1974).

3. Ostermayer also concludes that capacity increases with holes up to about four inches (10 cm) in diameter but shows little or no increase above four inches (10 cm) in diameter.

4. The apparent value of the skin friction decreases with increasing anchor size.

Ostermayer (1974) suggests that the carrying capacity of the anchors in cohesionless soils can only be explained by normal stress in excess of the overburden stress which acts over the anchor length. This increased normal stress is due to the high pressure of grouting, and the value of this normal stress exceeds the effective overburden stress by a factor of from two to ten. This observation agrees in principal with the observations of Littlejohn (1970a) and Broms (1968) that grouting pressures control anchor capacity.

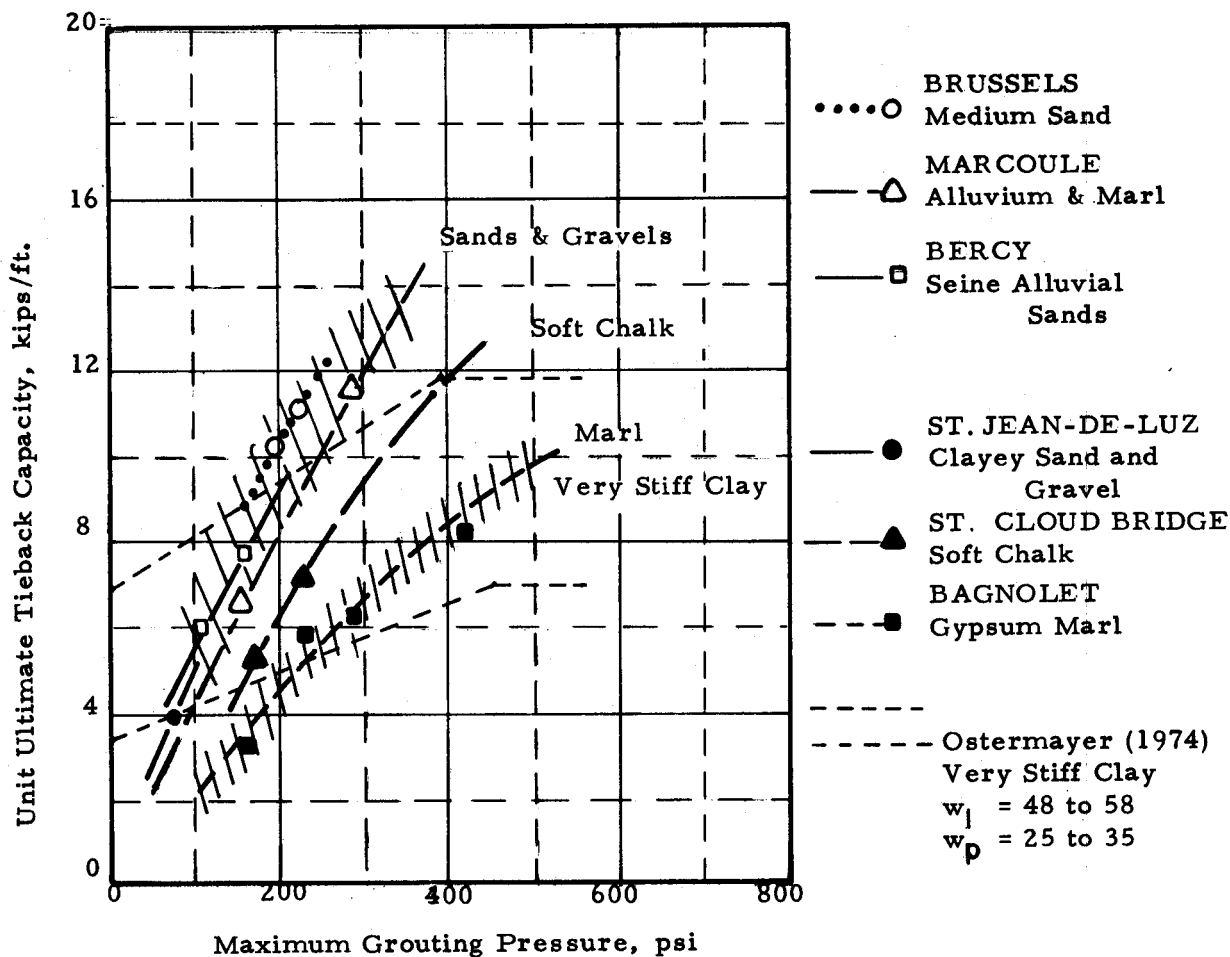
In summary, relative density, friction angle, gradation, and grout penetration into soil voids (soil permeability) will affect anchor capacity. As an approximation, considering Ostermayer's data and that of others, the following may be used as a rough guide for small diameter anchors installed without grouting at pressures of about 200 psi or more:

<u>Soil</u>	<u>Ultimate Load</u>	
	<u>(kip/ft)</u>	<u>kN/m</u>
Clean sand/gravel soils	10 - 20	145 - 290
Clean medium to coarse sands	7 - 15	100 - 220
Silty sands	5 - 10	70 - 145

In very clean gravelly soils it may be possible to exceed the ultimate loads as stated above; however, the range of values presented is believed to be representative of most soil conditions.

Jorge (1969) reported an improvement of anchor load capacity in both cohesionless and cohesive soils with a regroutable anchor. The initial grouting pressure was relatively low (70 - 130 psi) (480 - 900 kN/m²), and subsequent grouting was performed through the inner grout pipe at higher pressures. Figure 72 presents a summary of the results with data on very stiff clay from Ostermayer (1974).

The trend of the data for alluvium suggests an increase of ultimate capacity of approximately four kips per foot (58 kN/m) per 100 psi (690 kN/m²) increase in grout pressure. It should be noted that for sands and gravels subjected to post-grouting pressures of



NOTE: 1 k/ft = 14.6 kN/m
 1 psi = 6.90 kN/m²
 1 in = 2.54 cm

From Jorge (1969) 5" diam. and Ostermayer (1974) - 4-6" diam.

Figure 72. Ultimate anchor capacity as a function of grout pressure.

200 psi (1380 kN/m²) or greater a range of approximately 8 to 13 kips/ft (115 - 190 kN/m) in anchor load was observed. These are typical grouting pressures and ranges in anchor loads for nonregroutable, small diameter anchors.

f. Cohesive Soil

A summary of data presented by Ostermayer (1974) is given in Table 8.

The data in Table 8 and the data in Figure 73 show that the effective skin friction increases with an increase in consistency and decreasing plasticity of cohesive soil. The data also show an increase in skin friction with post-grouting of regroutable anchors. Ostermayer reported that on tests on nineteen anchors in very stiff medium to highly plastic clay a linear increase in skin friction with post-grouting pressure up to about 350 psi (2400 kN/m²) was observed. The skin friction associated with 350 psi (2400 kN/m²) post grouting was about 50 percent higher than the skin friction without post-grouting. Note, however, that no increase was observed above about 350 psi (2400 kN/m²).

Jorge's data, with marl, (Figure 72) show a near doubling of anchor capacity as a result of an increase in post-grouting pressure from 200 (1480 kN/m²) to 500 psi (3450 kN/m²). The data reported by Jorge (1969) and Ostermayer (1974) show the same basic trends and are of comparable magnitudes.

Moreover, these data are somewhat higher than the data in Table 8 on marl clay of stiff consistency, which shows skin friction of 2200 (105 kN/m²) to 3500 psf (170 kN/m²) for small diameter, high pressure tiebacks installed without post-grouting.

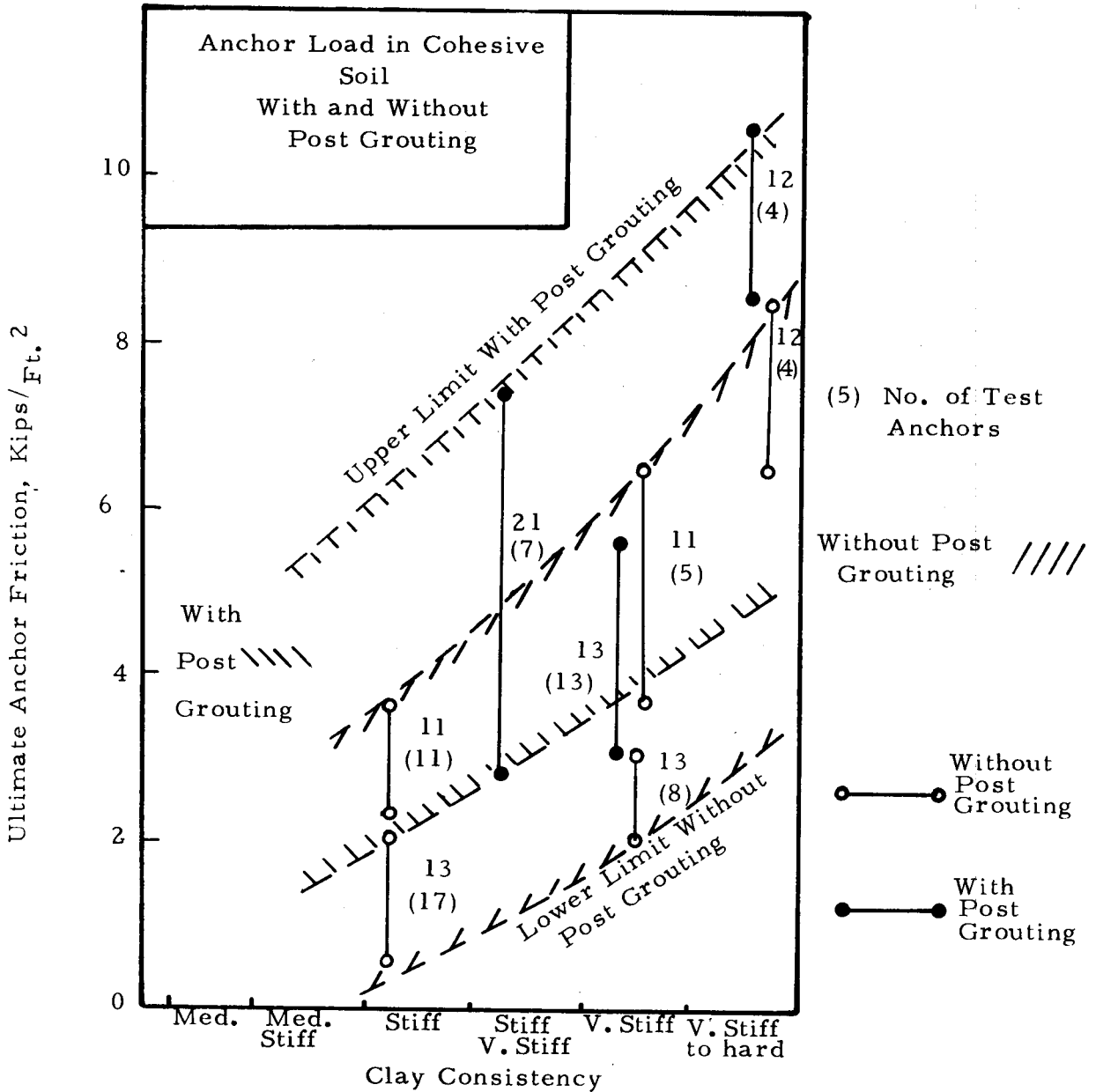
Ostermayer (1974) also reported data on the results of fifty-six tests to failure in which he developed creep rate coefficients for small diameter [3 - 1/2" ± to 6" (9 - 15cm)] anchors. The data was reported in terms of deflection per log cycle of time (creep rate) and percent of observed anchor failure loads for several soil types. The results indicate that clays of high plasticity will experience creep rates exceeding 1 mm per log cycle of time at 50% - 70% of their ultimate load. Clays of medium to high plasticity will experience these creep rates at 60% to 90% of the ultimate load. Tests on anchors in sand indicate that creep rates of 1mm (0.04") per log cycle will not be exceeded until approximately 80% to 95% of the ultimate load is achieved. As the load increases, the creep rates increase dramatically. The phenomenon

Table 8. High pressure small diameter tiebacks
in cohesive soil (after Ostermayer, 1974).

<u>Soil Type</u>	Typical Skin Friction (per square foot of grouted zone)	
	<u>Without Post-Grouting</u>	<u>With Post-Grouting</u>
Marl Clay - medium plastic ($w_1 = 32$ to 45 ; $w_p = 14$ to 25)		
Stiff	2200 - 3500	-----
Very Stiff	3500 - 6500	-----
Marl Sandy Silt - medium plastic ($w_1 = 45$; $w_p = 22$)		
Very stiff to hard	6500 - 8500	8500 - 10,500
Clay - medium to highly plastic ($w_1 = 45 - 59$; $w_p = 16 - 35$)		
Stiff	500 - 2000	
Very Stiff	2000 - 3000	3000 - 5500

Note:

1. Tiebacks 3-1/2" to 6" O.D.
2. Values are for lengths in marl - 15 to 20 feet and
for lengths in clay - 25 to 30 feet
3. $1 \text{ psf} = 0.48 \text{ kN/m}^2$
 $1 \text{ in} = 2.54 \text{ m}$
 $1 \text{ ft} = 0.305 \text{ m}$



Ostermayer (1974)

11 Marl Clay

$w_l = 32$ to 45 ;

$w_p = 14$ to 25

12 Marl Sandy Silt

$w_l = 45$;

$w_p = 22$

13 Clay-medium to highly plastic

$w_l = 45$ to 59 ;

$w_p = 16$ to 35

Goldberg-Zoino & Associates

21 Medium Plastic Clay

NOTE: $1 \text{ K/ft}^2 = 47.9 \text{ KN/m}^2$

Figure 73. Effect of post grouting on anchor capacity.

of anchor creep and its importance are discussed in greater detail in Section 6.40 of this chapter.

Gravel Packed Anchors

A gravel packed anchor is used on cohesive soils primarily to increase the value of the undrained shear strength coefficient, α . The original anchor hole is filled with angular gravel. A small closed-end casing is then driven into the hole displacing the gravel into the surrounding clay. Grout is then injected as the casing is withdrawn. The grout penetrates the gravel and increases the effective anchor diameter. The irregular gravel surface also improves the strength along the grout-soil interface. Figure 74 schematically illustrates the geometry of a gravel packed anchor.

Littlejohn (1970a) proposes that the following equation be used for determining the ultimate load of a gravel packed anchor. There are terms for both frictional resistance and end bearing. A substantial increase in the value of the undrained shear strength coefficient is recommended, and the anchor diameter is larger.

$$P_u = \alpha S_u \pi D L_s + \pi/4 (D^2 - d_s^2) N_c S_u$$

where:

d_s, D, L_s, S_u are as before and $N_c = 9$

$\alpha = 0.6 - 0.75 =$ undrained shear strength coefficient

6.24.5 Rock Anchors

Rock anchors have been used widely in engineering works for thirty years yet the design practices for rock anchors vary widely. The primary reasons for the lack of agreement on rock anchor design are the conflicting results of some tests and the nature of rock anchors. Even the weakest rock is generally capable of supporting large anchor loads. Since the additional cost of increasing the anchor length to ensure its ability to carry the load under even the most conservative criteria is generally small, this approach has been taken in rock anchor design. This section describes the basic procedures and criteria in rock anchor design.

Much of the data presented in this section has been obtained from papers by Littlejohn (1974a, 1975) on the design of rock anchors. The second paper (Littlejohn, 1975) is a state-of-the-art review of rock anchor design. Littlejohn (1975) summarizes

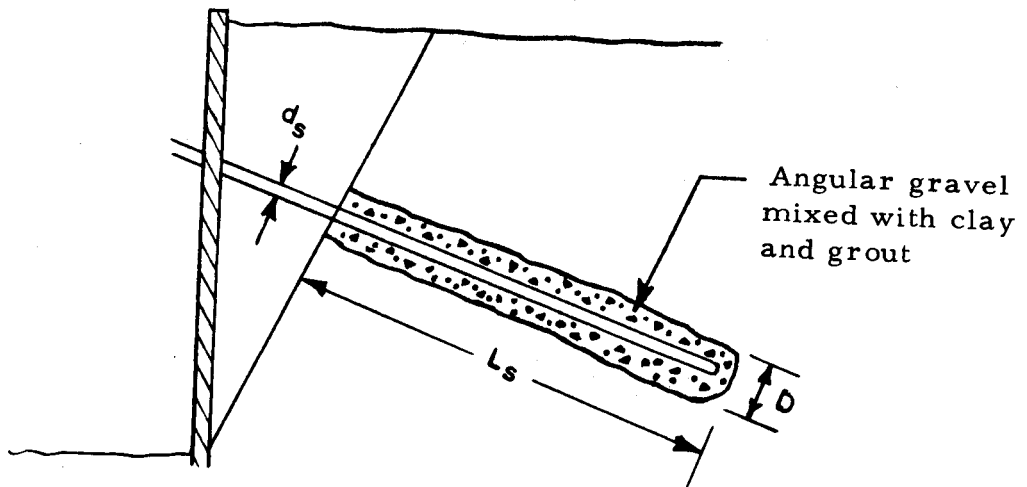


Figure 74. Schematic of gravel packed anchor.

the experiences and design criteria of rock anchor experts from around the world. The results presented in this section apply primarily to cement grout injected rock anchors.

Rock anchors may fail in any one of the following modes:

1. Failure of the rock mass
2. Failure of the grout-rock bond
3. Failure of the grout-steel bond
4. Failure of the steel tendon

The last two modes of failure are true of all anchors and will be discussed in Section 6.25.

Failure of the Rock Mass

The criterion for failure in a rock mass is based on the weight of the rock contained within a specified cone emanating from a point on the anchor and extending to the top of the rock. Figure 75 illustrates the geometry for this case. The criteria used to evaluate the value of the angle, θ , and the location of the apex of the cone vary with the type of rock, method of load transfer, and designer (Littlejohn, 1975).

Typically, the design value of θ will vary from 60° to 90° although in badly fissured or jointed rock the design criteria may be significantly different. If the weight of the rock within the contained cone is greater than the design anchor load, the anchor is generally believed to be safe since any cohesion or other rock strength properties have been ignored. However, a factor of safety can also be applied to the weight of the rock mass and the anchor load. This measure may be required if the rock is badly jointed.

Grout-Rock Bond

Most rock anchors are straight shafted friction anchors of 4" to 6" diameter. In the past it has been assumed that the load is transmitted uniformly along the grout-rock interface, and most anchor design has been based upon this assumption. However, Littlejohn (1975) reports the results of several studies indicating that the assumption of a uniform stress over the entire anchor is not necessarily valid. High stresses at the leading edge of the anchor are

are to be expected in harder rock formations (where $\frac{E_{\text{grout}}}{E_{\text{rock}}} < 10$).

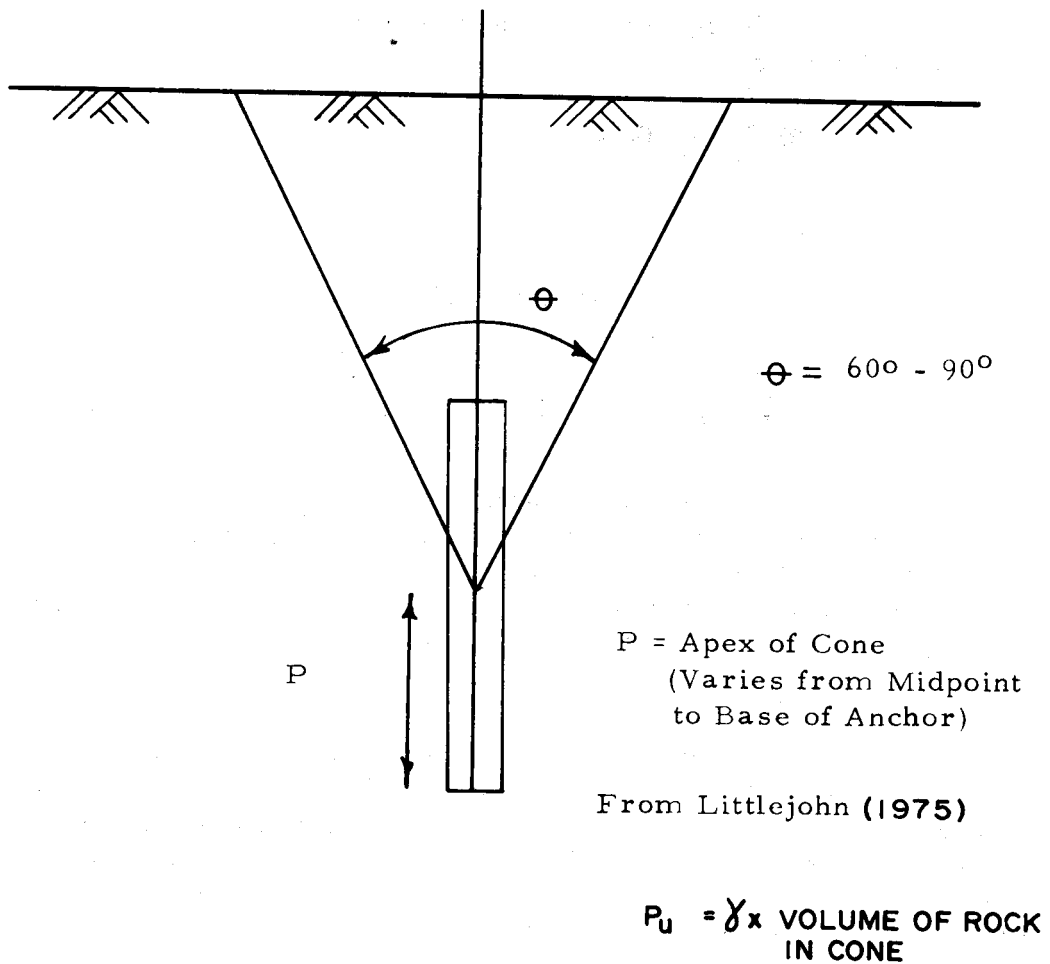


Figure 75. Schematic drawing of design quantities for failure in a rock mass.

In severe cases this may lead to debonding along the anchor length and load transfer towards the base of the anchor. To date there is little data available on the debonding phenomenon or how it affects anchor performance. Since the design of rock anchors has been based largely on the assumption of uniform load distribution, it would seem reasonable to continue using the relationships that have previously been derived while subjecting anchors to rigid field testing to assure their adequacy.

Using this method of determining anchor load the design equation becomes:

$$P_u = \pi d_s L_s \delta_{\text{skin}}$$

where:

d_s = diameter of anchor shaft

L_s = length of anchor shaft

δ_{skin} = grout-rock bond strength

The values of skin friction, δ_{skin} , for various rock types are summarized in Table 9. The data reported in this table represent a summary of results presented by an ad hoc committee of the ACI post-tensioning committee (March, 1974;) and Littlejohn (1970a, 1975). Littlejohn (1975) reports the bonding criteria used by designers in great detail.

In soft rock it is also possible to form belled or multi-underreamed anchors. Littlejohn (1970a) reports a case of using multi-underreamed anchors in marl. The equations governing the ultimate loads in these rocks are given in previous equations in Section 6.24.4. In these cases the cohesive strength of the rock becomes the controlling quantity.

6.24.6 Safety Factor of Soil or Rock

Safety Factor with Respect to Shear

The recommended factor of safety varies with the type of project, the soil conditions, previous experience in the soils, and the amount of field testing of the anchors. In practice, many successful jobs are planned on the basis of experience and with production testing to 120 percent of design load. Some anchors may be tested to 150 percent of design load, but pullout tests of anchors are not always performed. Therefore, the true factor of safety may never be known.

Table 9. Typical values of bond stress for selected rock types.

Rock Type (Sound, Non-Decayed)	Ultimate Bond Stresses Between Rock and Anchor Plug (f_{skin})
Granite & Basalt	250 - 800 psi
Limestone (competent)	300 - 400 psi
Dolomitic Limestone	200 - 300 psi
Soft Limestone	150 - 220 psi
Slates and Hard Shales	120 - 200 psi
Soft Shales	30 - 120 psi
Sandstone	120 - 250 psi
Chalk (variable properties)	30 - 150 psi
Marl (stiff, friable, fissured)	25 - 36 psi

$$1 \text{ psi} = 6.90 \text{ kN/m}^2$$

Note: It is not generally recommended that design bond stresses exceed 200 psi even in the most competent rocks.

Data is summary of results presented in:

1. Inland-Ryerson (1974 - ACI Ad Hoc Committee)
2. Littlejohn (1970)
3. Littlejohn (1975)

Since the formulae and empirical relationships presented in this section are relatively crude, considerable scatter could be expected between the predicted and actual anchor loads. However, these relationships can give estimates of ultimate anchor load and may be sufficient in design provided a suitably high factor of safety is applied and previous experience with the soils and anchor is available.

a. Soil Anchors

The methods of insuring an appropriate factor of safety for anchors will vary with the particulars of the project. In noncritical cases where it is not economically feasible to perform pull-out tests in the soil, the ultimate anchor load may be estimated using the empirical relationships presented. An appropriate factor of safety would then be applied to this predicted load, but it should be noted that these load predicting equations are crude estimates of actual load capacity. The magnitude of the safety factor would vary with the previous experience with the soil and anchor type and the field testing procedure.

In cases where there has been considerable experience with the soil and anchor type and where 5 percent or more of the anchors are to be proof-tested to 150 percent of design load, the anchors should be designed with a minimum factor of safety of 2. The design parameters should be based on previous pullout tests or the results of pullout tests performed on the site.

In special cases where a comprehensive field testing program is specified, the factor of safety may be reduced to 1.75. The general requirements for the reduction in the factor of safety are extensive experience with anchor in the soil type and a minimum of five carefully monitored pullout tests (or to 175 percent of design load). Production test monitoring of creep and load is also required.

b. Rock Anchors

The factor of safety that should be applied against pullout of a rock anchor depends upon the rock type and the type of failure. For failure in the rock mass itself a factor of safety of 1.1 applied to the weight of the rock mass inside the cone of rupture is considered adequate because of the beneficial contributions of rock shear strength. In heavily jointed rock the factor of safety may be increased.

The factor of safety applied to the grout-rock bond should be a minimum of 2.0. This factor of safety is recommended because of stress buildup and debonding.

Field testing of all production anchors to a minimum of 125% of design is recommended. Special test anchors (150% of design) and pullout tests are also recommended in critical tieback installations. Pullout tests can be performed on smaller diameter anchors or anchors of lesser length. The design parameters as described in the previous relationships can then be evaluated and used to determine the dimensions of the production anchors.

Safety Factor With Respect to Creep

In some cases, the anchor may have an adequate factor of safety against pullout, but not against creep. To date, the criteria for determining acceptable creep rates are based upon field observations. The values used may vary significantly depending upon the designer's experience.

With regard to permanent anchors, Ostermayer (1974) recommends that the working load should not exceed 2/3 of the load causing a creep rate of 1 mm (0.04") per log cycle of time. This is extremely small; it corresponds to a movement of 6 mm (0.24") between times of thirty minutes and fifty years. Twice this creep rate may be tolerated for temporary structures.

As a practical matter, the significance of the creep rate is as an index of potential progressive yielding during production testing. At a job in Boston in cohesive soil, if the creep rate exceeded 0.01" (0.25 mm) in the last five minutes of the specified twenty minute holding period under 125 percent of the working load, the contractor was required to maintain the test load for an additional thirty minutes to demonstrate satisfactory performance.

6.24.7 Discussion

This section presents the design criteria for determining the anchor capacities for various anchor types in differing soil conditions. The equations and figures presented are based largely on the results of empirical data and are far from a perfect means of determining anchor load capacity. It is for this reason that field testing of anchors be performed for all but very minor anchoring systems.

Large diameter anchors (straight shaft, belled,

multibelled) are most suitable in stiff to hard cohesive soils provided there are no installation difficulties. The large diameter of these anchors mobilizes a large surface area from which to derive resistive force. In some stiff cohesive soils there may be problems controlling the creep in belled or multi-belled anchors if stressed too highly.

Small diameter anchors are best used in cohesionless soils of moderate to high density. Large grouting pressures increase the normal stress acting on the anchor and therefore increase the load capacity. Very large capacities can be achieved in very dense, clean gravelly soils that allow the grout to penetrate the soil matrix.

Regroutable anchors are appropriate in difficult soil conditions such as loose cohesionless soils, clays of variable consistency, and soils with obstructions. The ability to regrout anchors is important in variable soil conditions where it is impossible to say how much effort will be required to install a suitable anchor.

The design of rock anchors generally is based upon the values of skin friction at the grout-rock interface. In actuality however, the steel tie member or the bond along the grout-tie interface are the most likely modes of failure in a rock anchor.

6.25 Tendon and Load Transfer

The previous section dealt with the transfer of load from the anchor to the surrounding soil (or rock). This section deals with the transfer of load from the concreted or grouted anchor to the steel tie member. The recommended design criteria are also presented.

6.25.1 Anchor Zone and Bond Free Zone

The anchor zone is that part of the tieback which is grouted in the soil and through which the tieback load is transferred to the soil. The transfer of load to the grout zone can be made either through bonding forces between the tie and the grout (tension anchor) or by plate rigidly attached to the tie at the base of the anchor (compression anchor). The plate reacts against the base of the anchor, the point at which all the load transfer occurs. The tie is debonded over the entire anchor length in this anchor type. These two anchor transfer mechanisms (tension anchors and compression anchors) will be discussed in Section 6.30 of this chapter.

The bond free zone refers to that portion of the anchor inside the theoretical or assumed slip line. Since anchor resistance

will not be developed in this area when the wall reaches its full depth, it is unconservative to test load the anchor if load can be transferred to the soil through this zone during testing. Therefore, the following methods are used to insure that all load is indeed carried in the anchor zone.

1. Prevent tendon load transfer
 - a. Wrap the steel tie in a plastic sheath to prevent bonding in this zone.
2. Prevent compressive force from developing
 - a. Do not backfill or wash out grout in the bond free zone.
 - b. Backfill the bond free zone with sand or a very lean cement grout to within a foot of the back face of the wall.

Although the technique of grouting to the back of the wall has been used, the technique is not as an effective a debonding technique as the others mentioned. Figure 76 illustrates the recommended treatment for bond free zones.

6.25.2 Steel Tie Member

Generally, the design of steel tie members depends on the ultimate load that the member can carry in tension. The exceptions to this rule would be where the bond between tie and grout is the controlling factor (rare) or where end connections cause a significant decrease in steel tie area. Bonding is not a significant problem unless large anchor capacities are required. Bonding may be critical in high capacity rock anchors. Bond strengths will typically be between 200 (1.38 N/mm^2) and 250 psi (1.73 N/mm^2) for cement grouts and concrete.

High strength steel wire strands, cables, and bars are most commonly used for tie members. Often the choice of the type of tie is controlled by the method of installation or convenience. Table 10 lists typical properties and dimensions of steel wires, strands, and bars for tie members.

6.25.3 Grout and Concrete

The choice between using a cement grout, resin grout, or concrete in the anchor zone often depends upon the type of

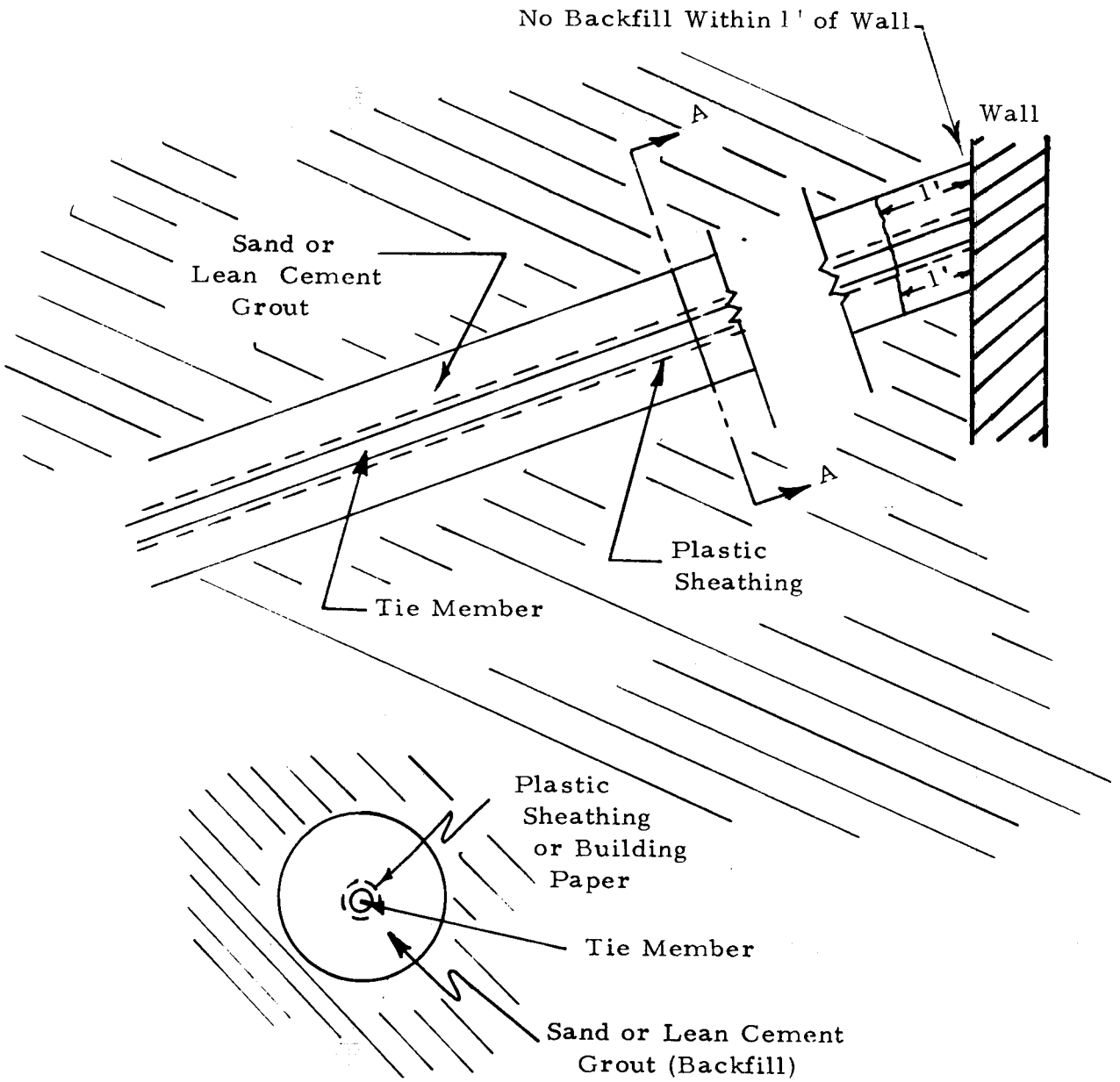


Figure 76. Recommended treatment for bond free zone.

Table 10. Typical steel properties and dimensions for ties.

Type of Tie	Diameters (inches)	Ultimate Stress f_u (ksi)	Yield Stress f_y (% f_u)	Ultimate Load (kips)	Yield Load (kips)
Wire (1)	.25"	240	.80	11.8	9.4
Cables or Strands (2)	.25"	270	.85	10.3	8.8
	.50"	270	.85	41.3	35.1
	.60"	270	.85	58.6	49.8
Bars or Rods (3)	.50"	160	.85	34.1	29.0
	.625"	230	.85	70.6	60.0
	1.00"	150	.85	127.8	108.6
	1.00"	160	.85	136.3	115.9
	1.25"	150	.85	187.5	159.4
	1.25"	160	.85	200.0	170.0
	1.375"	150	.85	234.0	198.9
	1.25"	132	.85	165.0	140.2

Wire Members: ASTM A-421
 Cable or Strands: ASTM A-416
 Bars or Rods: ASTM A-322

Note: 1 inch = 25.4 mm
 1 ksi = 6.898 N/mm²
 1 kip = 4.45 kN

- (1) Many wires are used in anchor to obtain load carrying capacity.
- (2) Several cables or strands are used in an anchor.
- (3) There are many bar or rod types and manufacturers. The data presented here is typical and is not meant to indicate the only bar types available.

anchor being installed. Resin grouts are not commonly used in tieback jobs, although their use may increase because of their quick setting times.

Resin Grouts

Resin grouts are used because of their quick setting times of ten to twenty minutes (for 80 percent to 90 percent ultimate strength). This allows anchor testing shortly after installation as opposed to other grouts which generally require 24 hours or more before testing. The strength of the resin grouts is comparable to that of concrete or cement grouts. The major disadvantage of resin grouts is their relatively high cost. One method of installation for these grouts is placement of the grout with packages of the activating agent in the anchor hole. The anchor tie is then pushed down the hole breaking packages containing the activating agent. The setting process starts as soon as the two compounds come in contact.

Cement Grouts

Cement grouts are most commonly used in small diameter anchors. Often the grout is injected under large pressures (150 psi (1035 kN/m²) or greater), but the grout can also be placed under relatively low pressures 30 psi (200 kN/m²). Generally, high early strength cement is mixed with water to form a neat cement grout.

The strength of the concrete is generally not critical provided the concrete or cement has a compressive strength greater than 4000 psi (27.6 N/mm²). The anchors are usually tested 24 to 72 hours after installation of the grout. Cement grouts are most common for both earth and rock anchors. While expansive additives have been used in grouts, recent experience has shown that such additives are not necessary to the satisfactory performance of the grout or anchor.

Concrete

In large diameter anchors (greater than ten inch [25cm] diameter) the anchor zone is generally grouted under low pressure with a mixture of high early strength cement, water, and sand or fine gravel. The sand or gravel filler is cheaper than cement and does not appreciably reduce the strength of the grout. The aggregate in the concrete may prevent grout penetration and therefore reduce anchor capacity in permeable soils. However, large diameter anchors generally derive their resistive force in friction or end bearing and do not rely upon grout penetration to increase resistive forces.

6.25.4 Factors of Safety

The strength of the grout and the tie-grout bond are generally not critical design quantities. The main item of concern is the strength of the steel tie member.

Two quantities are important in tie design. First, a suitable factor of safety with respect to the ultimate load of the tie must be maintained. Second, the yield stress of the tie should not be exceeded.

Table 10 has already presented the typical strength properties for tie members. Table 11 presents recommendations for stressing of steel ties. Important points from this table are:

1. Maximum test stress (f_t): This has been established at $f_y - 0.1 f_u$. The 10 percent margin with respect to ultimate stress ($0.1 f_u$) is to protect against rupture resulting from nicks or cuts in ties during construction.

2. Design stress (f_d): The magnitude of this stress is controlled by the design factor of safety against pullout and the production test stress. Thus, for a production anchor with a cable or rod tie stressed to 125 percent of design (see Table 11):

$$f_d = \frac{f_t}{1.25} = \frac{0.75 f_u}{1.25} = \frac{f_u}{1.67} = \frac{f_y}{(0.85) 1.67} = \frac{f_y}{1.42}$$

If these anchors are tested to higher loads, there will be a corresponding increase in the factor of safety against both ultimate failure and yield.

The German Design Codes for anchors (DIN 4125, 1972) allow the following steel stresses:

Active earth pressure design

$$f_d \leq 0.57 f_y$$

K_o design

$$f_d \leq 0.75 f_y \text{ or } f_d \leq 0.57 f_u$$

whichever is smaller

Field Testing

$$f_t \leq 0.9 f_y = 0.76 f_u, \text{ for } f_y = 0.85 f_u$$

Table 11. Recommended maximum stresses for tie members in anchor.

Type of Tie	Ultimate Stress, f_u (ksi)(typical)	Yield Stress f_y (% f_u)	Maximum Test Stress f_t (% f_u)	Design Stress, f_d^1 (% f_u)	Maximum Lockoff Stress f_w (% f_u)
Wire	240 (1.66 kN/mm ²)	80	70	55	55
Cable or Strand	270 (1.86 kN/mm ²)	85	75	60	60
Bar or Rod	130 - 230 (0.897 - 1.59 kN/mm ²)	85	75	60	60

¹Maximum Design Stress, f_d , is equal to $\frac{f_t}{1.25}$ which corresponds to the recommended factor of safety for production temporary anchors. For special test anchors or permanent anchors the design stresses will be lower due to the higher required design and tested factors of safety.

The design stress for the active earth pressure is smaller than that for earth pressure at rest (K_0) because the active pressure is the least pressure that is possible. At-rest allowable stresses do not differ substantially from those in Table 11.

For permanent anchors, it is recommended that a minimum factor of safety of 2 be applied to the ultimate stress in determining the design stress in the steel members. In other words, $f_d \leq 0.50 f_u$. The stresses during field testing should not exceed the values presented in Table 11.

6.25.5 Corrosion Protection

Corrosion protection for temporary earth or rock anchors is generally minimal. In those cases where the anchors are expected to be in use for two years or less, the only corrosion protection consists of greasing and sheathing the ties in the bond free zone. Where unusually corrosive soil and water conditions are encountered, specially treated grout, treated steel members, or extra steel may be used to insure that the anchors will perform adequately.

Radial cracking of the grouted portion of tension anchors is a source of corrosion. In the absence of measures to prevent corrosion, permanent anchors should not be used.

6.30 CONSTRUCTION CONSIDERATIONS FOR TIEBACKS

This section deals with the basic construction procedure and techniques used to install tiebacks. A brief general discussion of tieback wall construction precedes the descriptions of the construction techniques for each type of tieback. The differences between tied-back wall construction and internally braced wall construction are discussed briefly as are the construction procedures common to all tiebacks. Recommendations for field testing of anchors are given in Section 6.40.

6.31 Tied-Back Walls Versus Internally Braced Walls

The basic construction sequences and procedures are the same for both wall types.

1. Install wall (soldier piles, steel sheeting, slurry wall, etc.).

2. Excavate to support level.
3. Install tieback, strut, or raker.
4. Repeat steps 2 and 3 until excavation is complete.

The differences between the wall construction methods are very minor and primarily reflect ways of installing tiebacks through the walls. For example, one common procedure is to place tiebacks between back-to-back channels - set either vertically as soldier piles or horizontally as wales (See Figure 77).

6.32 Construction Techniques Common To Tiebacks

Stated very simply, the construction sequence for the installation of a tieback consists of the following steps:

1. Excavate a hole for the tieback.
2. Install the tendon (tie).
3. Grout the anchor to the specified point (usually to the "slip" line).
4. Tension and test the tie.
5. Make final anchorage at the wall.

The type of tie, the treatment of the bond free zone, the method of tensioning the tie, and anchoring of the tie at the wall are all virtually independent of the type of tieback.

Compression or Tension Anchors

Compression anchors are those where the entire load is transferred to the tie at the base of the anchor. The tie is connected to a plate or a point which is embedded in the anchor base. The plate or point transfers all of the anchor load to the tie with no bond allowed to develop between the tie and the grouted zone except at the very base of the anchor. The entire grouted portion of the anchor therefore acts in compression. Figure 78 illustrates the principles of a compression anchor.

In a tension anchor the load transfer from the anchor to the tie is accomplished through the steel-grout bond acting over the surface area of the tie. Both the tendon and the grout elongate due to elastic strain. Generally, the anchor geometry is such that no problems are encountered in obtaining the desired load in the tie through the steel-grout bond. However, when bonding problems are anticipated,



Figure 77. Example of tied-back wall using channel sections as wales.
(Courtesy of Hughes Tool Company).

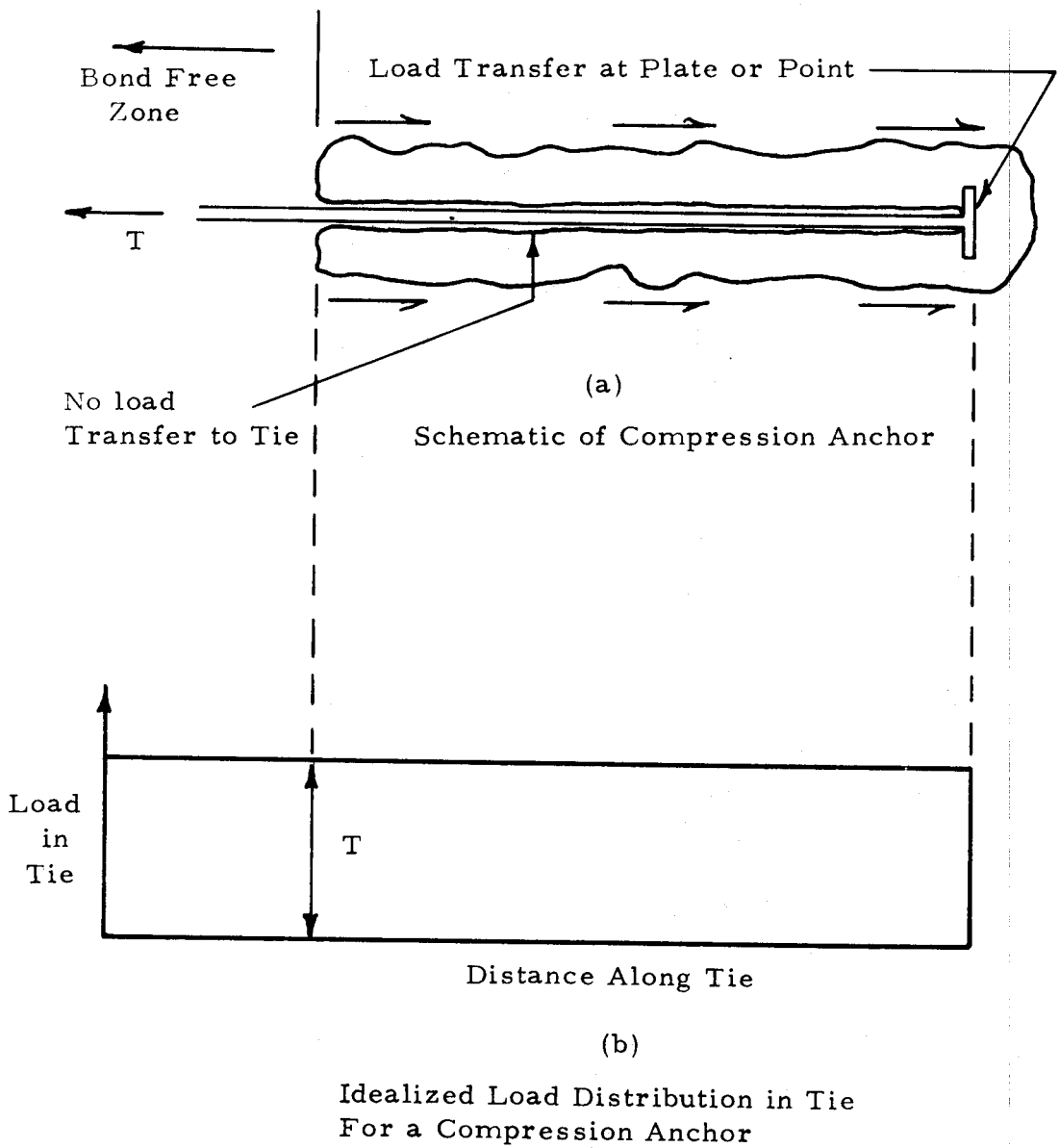


Figure 78. Schematic of compression anchor and resulting load distribution in the tie.

the wires or cables may be unraveled at the end to ensure that there is enough surface area for bonding. Hairline cracking in the anchors has been observed in these anchor types due to tensile strains (Ostermayer, 1974). Figure 79 illustrates a tension anchor.

A partial compression anchor is one in which a plate or point is fixed to the end of the tie to help transfer load. However, bonding of the tie to the grout is allowed so that such anchors have characteristics of both compression and tension anchors. Figure-80 illustrates the load transfer in a partial compression anchor.

Centering Ties

Spiders or other centering devices are required in larger diameter holes. This is particularly true for wire or cables because of their flexibility. In small diameter holes steel bars or rods often require centering while cables or wires generally will not because of their irregular surface.

Tendons

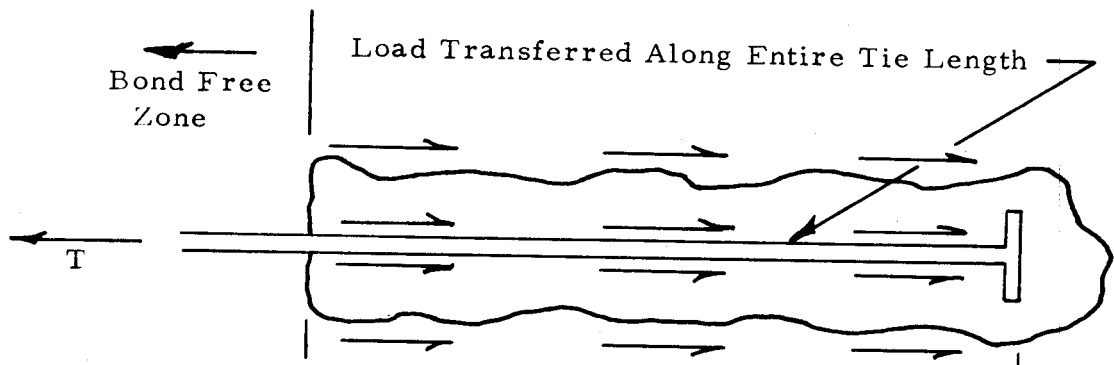
The different tendon (tie) types and their material properties have been described in an earlier section (Section 6.25). The choice of which tendon type to use (bar, strand, or wire) is virtually independent of anchor type. Bars and rods are used singly; strands or wires are wrapped together to form a bundle. High strength steel rods offer simplicity because they can easily be threaded into detachable points in the base of the anchor, allow for easy connections at the wall, and avoid the labor and time of bundling.

Anchorage at Wall

The method used to anchor tendons to the wall is independent of the tieback type although some methods are more suited to specific tendons. There are three basic types of connection: friction, button head, and threaded.

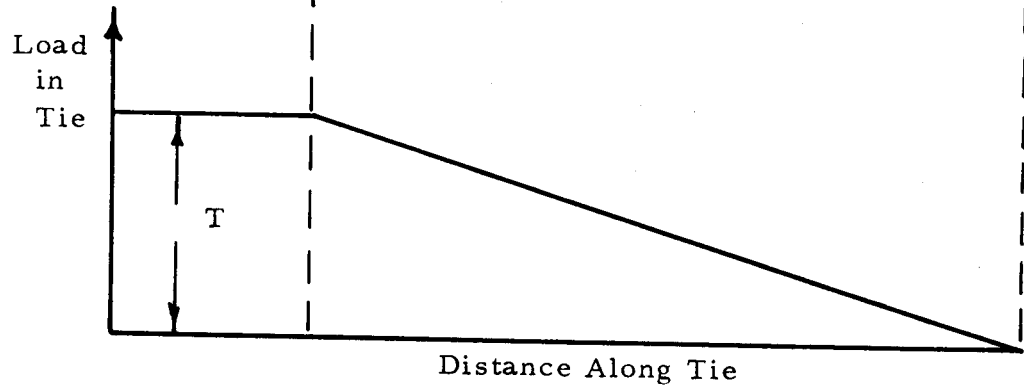
Friction connections have ridges or teeth that grip the tendon and cut into it slightly, thus causing stress concentration at that point in the tie. Figure 81 illustrates a typical friction connection.

Button head connections are generally preferred over friction connections where substantial retesting of anchors is anticipated. The connection is less likely to slip or cause damage to the tendons. Figure 82 illustrates a typical button head connection.



(a)

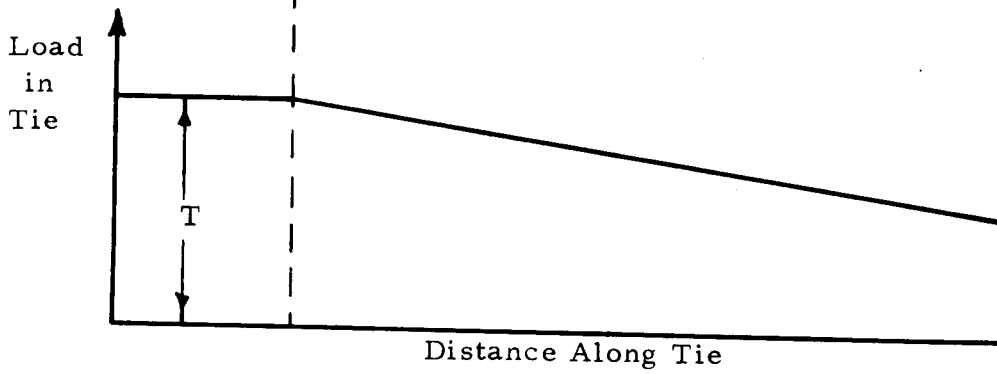
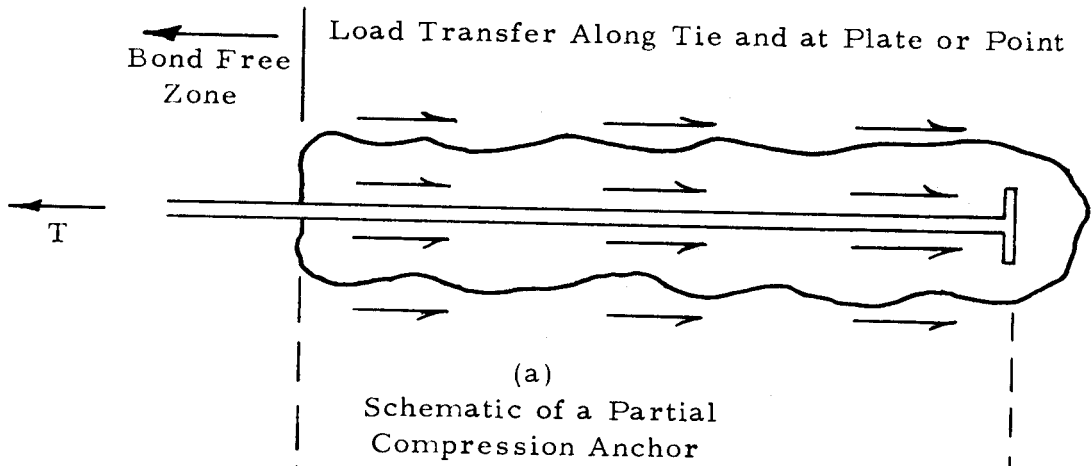
Schematic of Tension Anchor



(b)

Idealized Load Distribution in Tie For a Tension Anchor

Figure 79. Schematic of tension anchor and resulting load distribution in anchor.



(b)

Idealized Load Distribution in Tie
For a Partial Compression Anchor

Figure 80. Schematic of partial compression anchor and resulting load distribution in the tie.

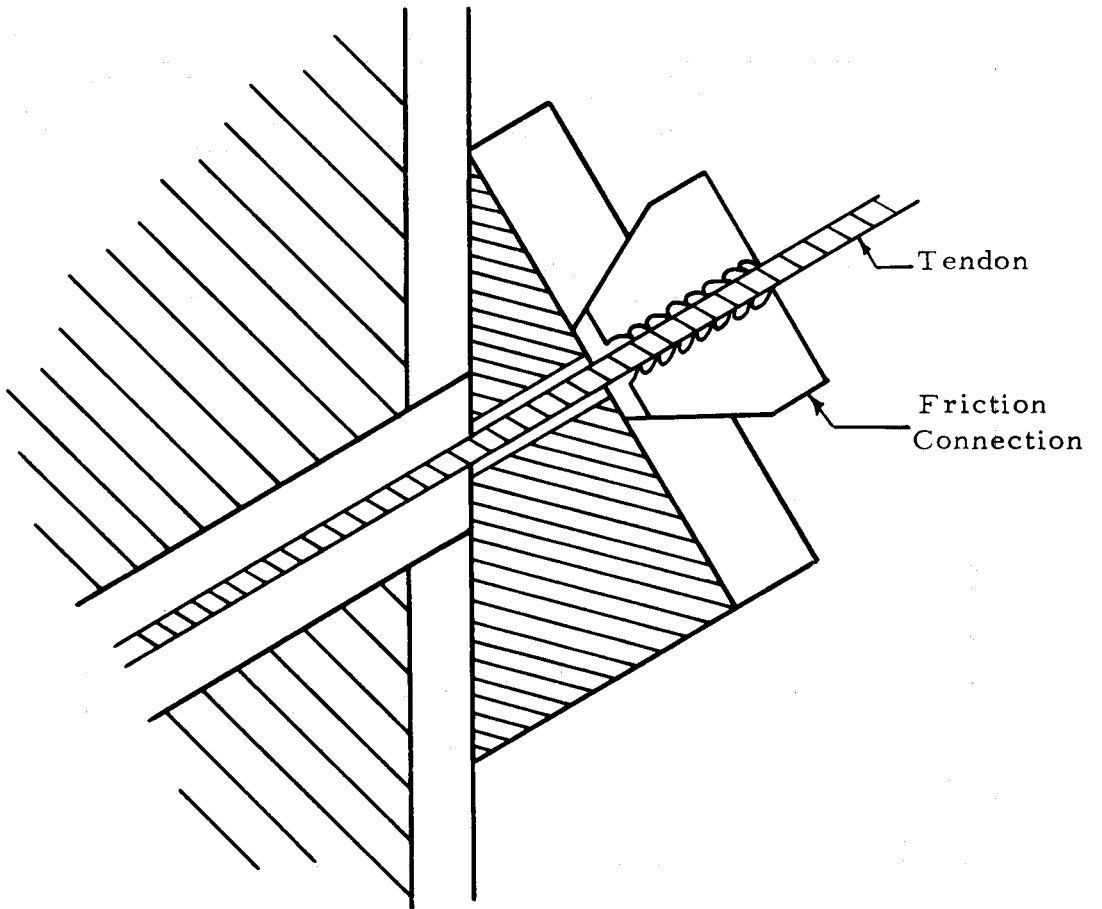


Figure 81. Friction connection used to tie anchor to wall.

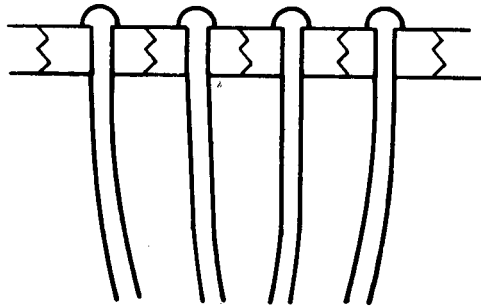
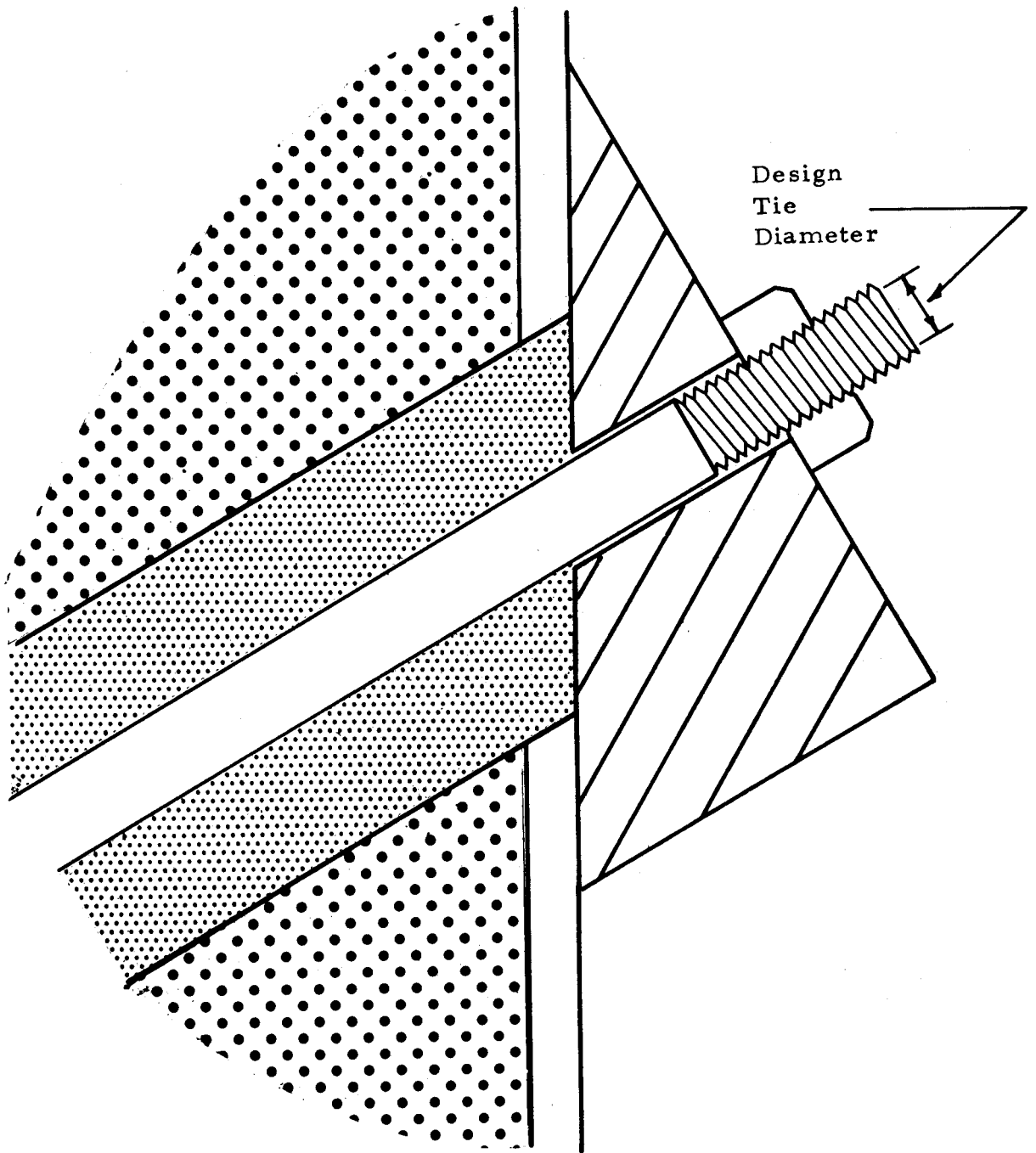


Figure 82. Button-head connection for wire ties.



THREADED CONNECTION FOR
TYING ANCHOR TO WALL

to wall.

Figure 3.20-5

Threaded connections also allow much retesting of anchors without damage to the tendon. The design steel area for the tendon is based on the interior area of the threads. Figure 83 illustrates a threaded connection. In practice, threaded connections are more commonly used than button head connections.

6.33 Construction Techniques and Procedures for Different Anchor Types

The following sections deals with the methods used to install the various types of tiebacks with emphasis upon essential differences and peculiarities between various types. Table 12 summarizes the main features of the construction of the different tieback types. Since tieback construction is a developing technology, not all procedures are listed in Table 12. The methods listed are intended to present representative installation techniques.

6.33.1 Straight Shaft Large Diameter Anchor

Solid Stem Augers

Large diameter anchors of this type require a large working area due to the size of the installation equipment. Continuous auger lengths of fifty feet and more are not uncommon. The augers are guided by a Kelly bar arrangement and have been used to install tiebacks up to 130 feet (40 m) in length. Some of this equipment was originally custom made for particular jobs. Many "early" drilling rigs used a bucket arrangement at the bottom to excavate rather than auger. All these rigs used the same basic installation equipment.

The basic method of installation is to auger a hole to the desired length, withdraw the augering equipment (assuming a competent, cohesive soil), install a tie member (usually with a plate or washer attached), and fill the hole with pumped concrete. These anchors derive all their resistance from the resistance along the grout-soil interface.

Hollow Stem Augers

The installation equipment is largely the same as for solid stem augers. The major difference is that the auger stem is hollow allowing the auger to remain in place during tendon placement. A detachable point is often located in the auger tip to which the tie is attached. The auger stem centers the tie in the hole. Grouting

Table 12. Typical equipment for construction of tiebacks.

Anchor Type	Preferred Soil Type	Equipment	Range in Diameter (typical)	Lengths	Typical Grout Type	Spacers and Plate	Angle of Inclination (to horizontal)	Bond Free Zone
Straight Shaft- Large Diameter	Competent cohesive soil which can remain open unsupported.	Truck-mounted crawler-mounted or crane-supported augers guided by Kelly Bars.	12"-24" (30cm-60cm)	50'-130' (typical) (15m - 40m)	Pumped concrete.	Spacers and plate generally used.	0° - 90° (better at shallow angles)	Lean concrete or sand backfill. Plastic sheathing.
(2) Hollow Stem Augers	Preferred in competent cohesive soils. Often used in sandy soils.	Truck-mounted crawler-mounted or crane-supported with guides.	6"-18" (15 cm-45 cm) 12"-14" most common	Reported to 160'. (50 m)	High strength concrete pumped water pressure through hollow stem ~150 psi or less. (1035 kN/m ²)	No spacer necessary since hollow stem serves as guide. Points are generally used in anchor.	0° - 90° (proprietary methods may not be able to achieve lower angles)	Lean concrete or sand backfill. Plastic sheathing.
Belled Anchor	Competent cohesive soils which can remain open unsupported.	Truck-mounted crawler-mounted or crane-supported augers with guides. Belling equipment same as used for caisson work.	12"-24" (30cm - 60cm) Shaft 30"-42" (75cm - 105cm) Bell	Typical length to bell of approximately 50' (15m). Lengths up to 100'± (30m) in California.	Pumped concrete.	Spacers used to center ties. Plates or washers usually aid load transfer.	Generally installed at angle (30° - 60°)	Lean concrete or sand backfill. Plastic sheathing.
Multi-Underreamed Anchor (Multi-Bell)	Competent cohesive soil or rock that can remain open unsupported. To date experience in United Kingdom.		4"-8" (10cm - 20cm) Shaft 8"-24" (20cm - 60cm) Underreams $D_u \geq 1.5d_s$	Total lengths in excess of 50' (15m). Spacing between bells approximately 1.5 - 2.0 x diameter bell.	Cement grout or concrete. [Concrete for larger diameter anchors]	Spacers used to center ties. Plate used in some methods to transfer entire load.	Generally installed at angle (30° - 60°)	Lean concrete, weak cement grout, or sand. Entire length except for plate is unbonded in some methods.

Table 12. Typical equipment for construction of tiebacks. (Continued).

Anchor Type	Preferred Soil Type	Equipment	Range in Diameter (typical)	Lengths	Typical Grout Type	Spacers and Plate	Angle of Inclination (to horizontal)	Bond Free Zone
Small Diameter Anchors (Not Re-groutable) (1) Driven	Sands and gravels preferred but can be installed in all soils except those with obstructions.	Crawler-mounted percussive driving equipment. Casing driven and then extracted.	4"-8" (10cm - 20cm) Shaft	Generally lengths about 70' (20m).	High early strength cement grout. Grout has high cement to water ratio. High pressure grouting (>150 psi) (1035kN/m ²)	Spacers may be used if ties are not attached to detachable points with threaded rods more common.	Generally installed at 15° - 60° angle.	Weak grout or sand used to backfill. In some cases, holes left open. Ties typically sheathed and greased.
(2) Drilled in Anchors	Sands and gravels. Generally used in soils with obstructions or where driving casing is difficult.	Crawler-mounted drilling equipment. Drill bit precedes casing or inside casing.	3"-8" (7.5cm - 20cm) shaft, if soils are permeable, bell may form.	Generally lengths less than 70' (20m).	High early strength cement grout with high cement to water ratio. Grouting pressure generally >150psi (1035kN/m ²)	Spacers may be required if flexible tie is used or no plate or point is used.	Generally installed at angle of 15° - 60°.	Weak grout or sand recommended or hole left open. Ties sheathed or greased or both.
Regroutable Anchors	All soil types. Usually used in softer soils, variable conditions, or where obstructions are encountered.	Same equipment as before for drilling or driving casing (depends on soil conditions). Grout pipe for each anchor.	4"-8" (10cm - 20cm) Shaft	As before for small diameter anchors.	Cement grout (1) 1st grout at 100 psi (690kN/m ²) (2) 2nd grout through individual packers at pressures up to 800 psi (5520kN/m ²) (3) successive grouts as needed.	Spacers not generally needed for bars although good for flexible ties. Points becoming common.	As before for small diameter anchors.	As before.
Rock Anchors (1) Multi-Underreamed	Used in softer competent rock.			See Section on Underreamed Soil Anchors.				

Table 12. Typical equipment for construction of tiebacks. (Continued).

Anchor Type	Preferred Soil Type	Equipment	Range in Diameter (typical)	Lengths	Typical Grout Type	Spacers and Plate	Angle of Inclination (to horizontal)	Bond Free Zone
(2) Drilled Anchors	All competent rock types.	As above for soil anchors. Rotary drilling equipment for rock drilling. Percussive drills also.	3"-8" (7.5cm-20cm) shaft depending on rock and load.	Generally <30' (9m) into rock.	Cement grout at high pressure 150psi (1035kN/m ²) or quick setting resin.	Bolts or washers in bottom with spacers.	Rock anchors generally at 45° angle.	Weak cement or sand backfill in soils above. Ties sheathed and greased.
Gravel Packed Anchors	Competent cohesive soils that will remain open when not supported.	Both augering and driving equipment is required. Drilling equipment for casing inserted after gravel in hole.	4"-8" (10cm - 20cm) Shaft	As before	Cement grout at high pressure >150 psi (1035kN/m ²)	Casing serves as spacers. Points usually used.	As before	As before.

with concrete or cement grout is done through the hollow stem while the augers are withdrawn. Grouting can be done under pressure, but the pressures are generally less than 150 psi (1035 kN/m²). This method of tieback installation can be used in soils that are not completely self-supporting since the augers provide partial support.

6.33.2 Belled Anchor

Belled anchors are generally installed using the following procedure:

1. A straight shaft is augered to the desired length.
2. The augering equipment is withdrawn and the belling equipment is put in place.
3. The bell is formed, and the equipment is withdrawn.
4. Tie placement and grouting is similar to that for solid stem augers.

Single belled anchors are becoming less popular due to the increased installation costs. Some contractors have found it more economical to extend straight shafts to greater lengths rather than to withdraw the augering equipment, put in belling equipment, etc.

6.33.3 Multi-Belled Anchor

Multi-belled anchors were developed to increase anchor capacity in competent cohesive soils and rock. Section 6.24.4 describes the theoretical reasons for the load increase observed in these anchors. Most of the installation techniques are proprietary; however, a few basics are true of all multi-underreamed anchors. A straight shaft of 4 inches to 8 inches (10cm - 20cm) diameter is augered (or cased) to the point of the first bell. A series of closely spaced bells (diameter of bell is 2.0 to 3.0 times diameter of shaft with a spacing between bells of 1.5 to 2.0 times the diameter of the bell) is then formed. Multi-underreamed anchors require that soils will remain open when unsupported in zone of bells. Although multi-underreamed anchors have been used successfully, the present trend is away from the use of these anchors. Problems with excessive creep in some formations, insufficient load capacity, and difficulties in forming the bells are the major reasons for using other anchors.

6.33.4 Small Diameter Anchor (Not Regroutable)

Driven Anchor

For this anchor type a casing is driven into the soil with a detachable point at the end of the casing. After the casing is driven to the predetermined anchor length, the tie is attached to the point, and the point is separated from the casing. Grouting, with pressures generally in excess of 150 psi (1035kN/m²), begins as the casing is withdrawn. High grout pressures are most effective in soils where the grout can penetrate into the soil matrix.

Drilled Anchor

Drilled anchors are essentially the same as driven anchors except that the hole is advanced by drilling instead of driving the casing. The soil inside the casing is removed by air or water as the casing is advanced. In cohesionless soils below the water table inflow of water and soil into the casing upon removal of the cutting bit could be a problem.

An advantage of small diameter anchors is that the installation equipment is readily available, very maneuverable, and usable in limited access and poor working conditions.

6.33.5 Regroutable Small Diameter Anchors

The installation procedures for this anchor type are very similar to those described above for small diameter anchors up to the point of tie insertion. Once the hole has been formed and the casing is in place, the tie is inserted in the hole with a grout pipe attached to the tie. When the tie and pipe are in place, grout is pumped in at low pressure to fill the anchor zone outside of the grout pipe. The casing is withdrawn as the grout is pumped.

After the grout in this initial grouting stage has set, a second grouting stage with higher grout pressures is performed from the grout pipe which has ports about three feet apart. The entire pipe can be grouted at once or the ports can be isolated by packers and grouted separately. The high pressures (often as great as 600 psi [4100 kN/m²]) crack the initial grout and allow localized grout penetration into the soil. Once the initial grout has been cracked, the grout pressure drops off markedly resulting in effective soil grouting pressures ranging from 100 psi (690 kN/m²) to 500 psi (3400 kN/m²) or more.

If the grout pipe is cleaned out, the preceding procedure can be repeated several times if necessary. Anchors that have failed to carry test loads after the first regrouting can be grouted several more times. Several regrouting stages may be required to achieve an anchor with the required load carrying capacity.

Regroutable anchors require special high pressure pumps, and the costs associated with these anchors are greater than for standard small diameter anchors. However, regroutable anchors allow for the improvement in anchor capacity even after installation is complete.

6.33.6 Gravel Packed Anchor

For this anchor type a hole is augered and then filled with angular gravel. A small casing with a detachable point is driven through the gravel displacing the gravel into the adjoining clay. A tie is connected to the point, and the point is then knocked out. Grout is injected into the gravel as the casing is withdrawn. The anchor is intended to improve the load carrying capacity of anchors in clay by increasing the adhesion between the clay and the anchor. The anchor has been used with success in hard clays and soft rock.

6.33.7 Rock Anchors

The equipment used to install rock anchors is the same as the equipment used for small diameter earth anchors (except for underreamed anchors in rock). Generally, a casing is advanced to the rock surface. Once the casing is firmly in contact with the rock, the rock is drilled out for an anchorage (3 to 8 inch (7.5 - 20cm) diameter). A tie is then founded in the hole and the hole is grouted.

6.33.8 Mechanical Anchors

The discussion of anchors in this section has been limited to grouted anchors. Many different types of mechanical anchors are available. The anchors may be simple rods or beams driven into the ground which derive their load capacity from frictional resistance. More complicated anchors are available which may include a plate (or plates) along the rod which extends out to form a bearing plate. Rock bolts would also be classified as mechanical anchors. Mechanical anchors in soil have limited capacities and will yield unpredictable load capacities. For this reason mechanical anchors are not discussed in detail in this report.

6.33.9 Examples

Figures 84 through 91 show photos of several tieback installations and some of the installation equipment. Generally, a tieback contractor will be able to install only one particular tieback type. This may be due to equipment costs or to the proprietary nature of some techniques.

6.40 FIELD TESTING

6.41 Reasons

The major reasons for field testing are:

1. Load

Theoretical bases for establishing design load are given in Section 6.24. These are crude at best and should only be used for a preliminary estimate of safe load. Field testing of anchors is the only method of assuring that the design anchor load can be carried by the anchor.

2. Quality and Safety

Proof testing of each production tie must meet general acceptance criteria to assure safety and to develop uniformity of the anchors.

3. Creep

Creep rates, inferred from long term tests, provide additional data for design and acceptance.

Field testing is an integral part of the design and should be performed on all anchors installed on a project. Since the additional costs of proofloading anchors is relatively small, field testing provides cheap insurance that the support system is adequate.

6.42 Criteria

The following quantities define the critical parameters in field testing of anchors:

1. Yield of steel tie
2. Ultimate capacity of steel to grout bond



Figure 84. Crane suspended auger rig.
(Courtesy of Spencer, White, and Prentis).



Figure 85. Crawler mounted auger rig.
(Courtesy of Spencer, White, and Prentis).



Figure 86. Crawler mounted auger rig.
(Courtesy of Acker Drill Company).



Note: Excavation has proceeded below tieback level.

Figure 87. Crawler mounted auger rigs.
(Courtesy of Hughes Tool Company).



Figure 88. Air trac drilling tiebacks.
(Courtesy of Spencer, White, and Prentis).



Note: Colcrete mixer in foreground.

Figure 89. Installation of small diameter anchor.
(Courtesy of Acker Drill Company).

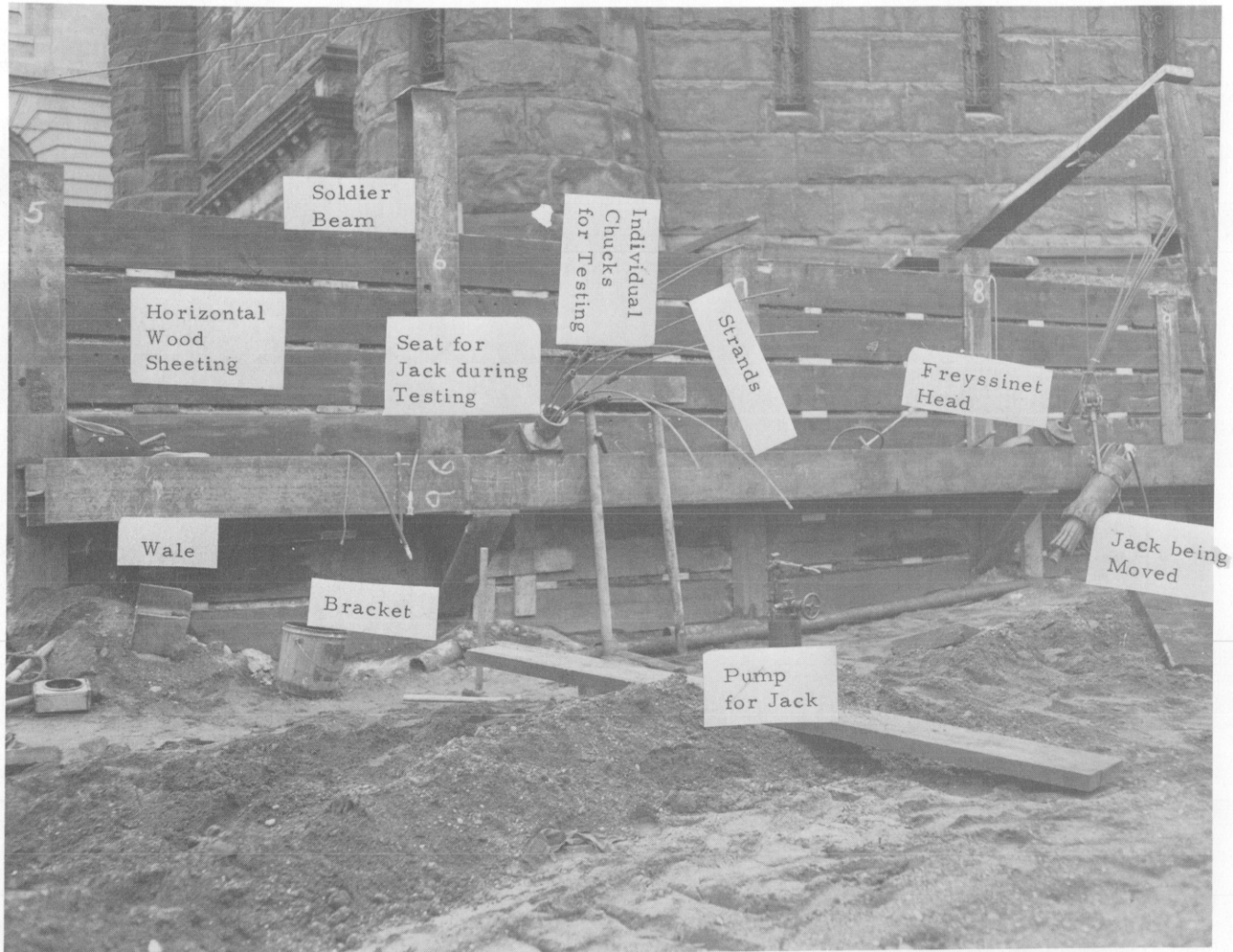


Figure 90. Tieback stressing details.
(Courtesy of Spencer, White, and Prentis).



In earth back-to-back channels have been set in holes augered to rock and filled with lean concrete. Poor quality rock is retained by sheeting and rock bolts.

Figure 91. Rock tiebacks - bottom of excavation in rock.
(Courtesy of Spencer, White, and Prentis).

3. Ultimate anchor load of the soil or rock formation
4. Lockoff load as a percentage of design
5. Production tie test load as a percentage of design
6. Special test load to a greater percentage of design than the production test load
7. Special test load to failure to assess true safety
8. Special test load of prolonged duration to assess creep

6.43 Range of Current Practice

6.43.1 General

The following discussion summarizes both published and unpublished opinion concerning field testing. While the field testing requirements on different projects are never the same, the basic range in testing requirements, as suggested by most publications, is quite similar. Special mention is made of the practices advocated by Littlejohn (Great Britain) and Ostermayer (Germany) both of whom have had a wide variety of experience with tiebacks.

General procedures for tieback testing include the testing of each production anchor to a load in excess of the design load (120 - 150 percent typical). In some cases additional and more stringent testing of specific production anchors is performed. This additional testing may involve loading test anchors to either failure or twice the design load. It may also include detailed load and deformation monitoring during test loading to 150 percent or more of design. Some of these techniques are described in more detail in several references (Bassett, 1970; Shannon and Strazer, 1970; Larson, et al, 1972; Osterbaan and Gifford, 1972).

Where there has been little experience with ground anchors in a particular soil deposit, anchors should be installed to determine load-carrying capacity. These anchors should be tested to failure, if possible, to determine the appropriate anchor design for the site. Several authors (Littlejohn, 1970a; Bassett, 1970; Booth, 1966; Hanna and Seeton, 1967; Prasad, et al, 1972; Nelson, 1973; Ostermayer, 1974) have described test anchor programs and the importance of installing test anchors on all tieback jobs.

Generally, it is not believed necessary to test anchors in groups because of the relatively large spacing of tiebacks.

The effects of group action are thought to be insignificant. Broms (1968) does recommend testing anchors if anchor spacing is less than 2.5 meters (8 feet). As a practical matter, several adjoining tiebacks are generally prestressed and tested at the same time. In this way the effect of group action is considered, even if not directly. Clearly, if group action is considered likely, anchors should be tested in groups.

Typically, tiebacks have been locked-off at loads varying from 80% to 100% of the design load, although Littlejohn (1970a) recommends locking in a load slightly greater than design to account for loss in the structural system, and measuring errors. The lock-off load often depends upon the design earth pressures assumed for the project. If the control of movements is critical, a larger design earth pressure and lock-off load is generally used.

The loss of load with time or the long term behavior of anchors is to a large extent an unknown quantity at this time. In temporary anchoring systems this is not usually a significant problem; however, in permanent anchoring systems it is. Littlejohn (1970a) recommends that the factor of safety be increased to account for the effects of creep, particularly in soils susceptible to creep or strength deterioration. Ostermayer (1974) recommends 24-hour load tests in cohesive soils. As would be expected, it has been observed that cohesive soils are more susceptible to creep and load loss than are cohesionless soils. In fact, cohesionless soils have been found to be remarkably insensitive to load loss with time (Ostermayer, 1974).

6.43.2 Some Specific Examples of Practice

Littlejohn (1970a and 1973) describes the testing procedures recommended by himself for both temporary and permanent tied-back installations. As a general rule, more rigid testing procedures are required for permanent tiebacks. However, if the consequences of a failure in a temporary tied-back installation are severe, more stringent testing procedures may have to be applied.

The following procedure outlines the criteria established by Littlejohn (1970a) for the testing of production anchors in a temporary tieback system:

- a. Test anchor to 128% of design for five minutes and unload.
- b. Restress anchor in steps to the lock off load and record movements. Lock-off load at design plus some nominal percentage (10%).

- c. Check load after 24 hours; if a loss of greater than 5% is recorded, restore to lock-off load.
- d. Repeat c.
- e. If a further loss of prestress is recorded, reduce anchor load until creep ceases. A safe lock-off load is 62.5% of load for which no creep occurs after 24 hours.

Littlejohn (1970a) also specifies that special testing procedures should be used on 10% of the anchors. He recommends that these anchors be installed and tested with extra steel in the tie such that the lock-off stress (f_w) [110% design (f_d)] is 50% of the ultimate steel stress (f_u). Each of these anchors should then be tested to 160% of design loading ($1.6 f_d$) prior to lockoff.

In addition, Littlejohn recommends that a minimum of three anchors of varying lengths be tested to failure to verify the design assumptions regarding ultimate anchor load. Failure at the grout-soil interface, rather than in the tie member or tie bond, should control,

Ostermayer (1974) recommends the following for temporary anchors:

1. Before construction starts, perform three tests to 150% of the design load and perform loading and unloading cycles to evaluate deformation characteristics. Study of the loading and unloading cycle will provide a basis for estimating the load transfer characteristics between the grouted anchor length and the soil or rock formation. To study creep effects the observation period for ties in cohesive soil should be 24 hours under 150% of the design load.
2. During construction, test production ties to 120% of the design load. Also test 5% of the anchors to 150% of the design load.

6.44 Recommendations

Considering the present state-of-the-art, the following recommendations are made for installation of temporary anchors to support excavations in the presence of nearby structures. These recommendations include requirements for special anchor testing, production anchor testing, methods of evaluating test loading data, and the proper lock-off loads for various design earth pressures and distributions.

a. Recommendations for Special Load Tests

Test Loads

<u>Soil and Site Conditions</u>	<u>Load</u>	<u>Remarks</u>
1. Reasonable experience with soil and anchor. Nearby structures outside "zone of influence".	150% of design	5% of production ties should be tested in this manner.
2. Reasonable experience with soil and anchor. Nearby structures within the "zone of influence".	150% of design	5% of production ties should be tested in this manner. In addition, 3 ties in each soil formation should be tested to 200% of design. (1)
3. Little experience or unsatisfactory experience with soils and/or anchor. Nearby structures within "zone of influence".	150% of design	10% of production anchors tested in this manner. In addition, 3 ties in each soil formation should be tested to failure or 250% of design. (2)

- (1) For ties loaded to 200% of design, the ties should be loaded to 150% of design and tested as other special test anchors. If the anchor passes the special test criteria, the anchor should then be loaded to 200% of design. If the anchors satisfy the creep criteria for special test anchors at this load, they may then be used as production anchors. However, it is recommended that these anchors be tested prior to actual construction to verify anchor design criteria (length of anchor, diameter, grouting pressure).
- (2) These anchors should be loaded to 150% of design and tested as special test anchors prior to increasing the load. If the anchor passes the special test criteria, the anchor should then be loaded to failure or 250% of design. The anchor design should be modified if failure occurs at less than 200% of design. It is recommended that these

anchors be installed and tested prior to actual construction. Anchors tested prior to construction should be of varying lengths and geometries to establish the appropriate design parameters.

Duration of Special Test Load and Criteria for Creep

Cohesionless Soil	Load duration of 1 - 2 hours depending upon prior experience with soil and anchor. The creep rate at a load of 150% of design should not exceed 2 mm (0.08") per logarithmic cycle of time. (See Figure 92)
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Cohesive Soil	Load duration of 24 hours for all cohesive soils. Creep rate should not exceed 2 mm (0.08") per logarithmic cycle of time. (See Figure 92)
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Method of Load Application

1. Load anchor in increments of 25% of design load to 125% of the design load.
2. Unload to zero
3. Reload in increments of 25% of design load to the desired load (or loads).
4. Maintain load for prescribed period.
5. Unload anchor to specified lock-off load.

b. Recommendations for Tests on Production Anchors

The following recommendations are the minimum test criteria that should be applied to any anchor. The recommended method of testing the production anchors is designed to be relatively easy to implement while still ensuring the adequacy of each tieback anchor. Depending upon the soil conditions and the nature of the excavation, it may be decided to use more rigid testing criteria.

Test Load

Load the anchor to 125% of the design load. Care must be taken to ensure that the recommended stresses in the ties are not exceeded.

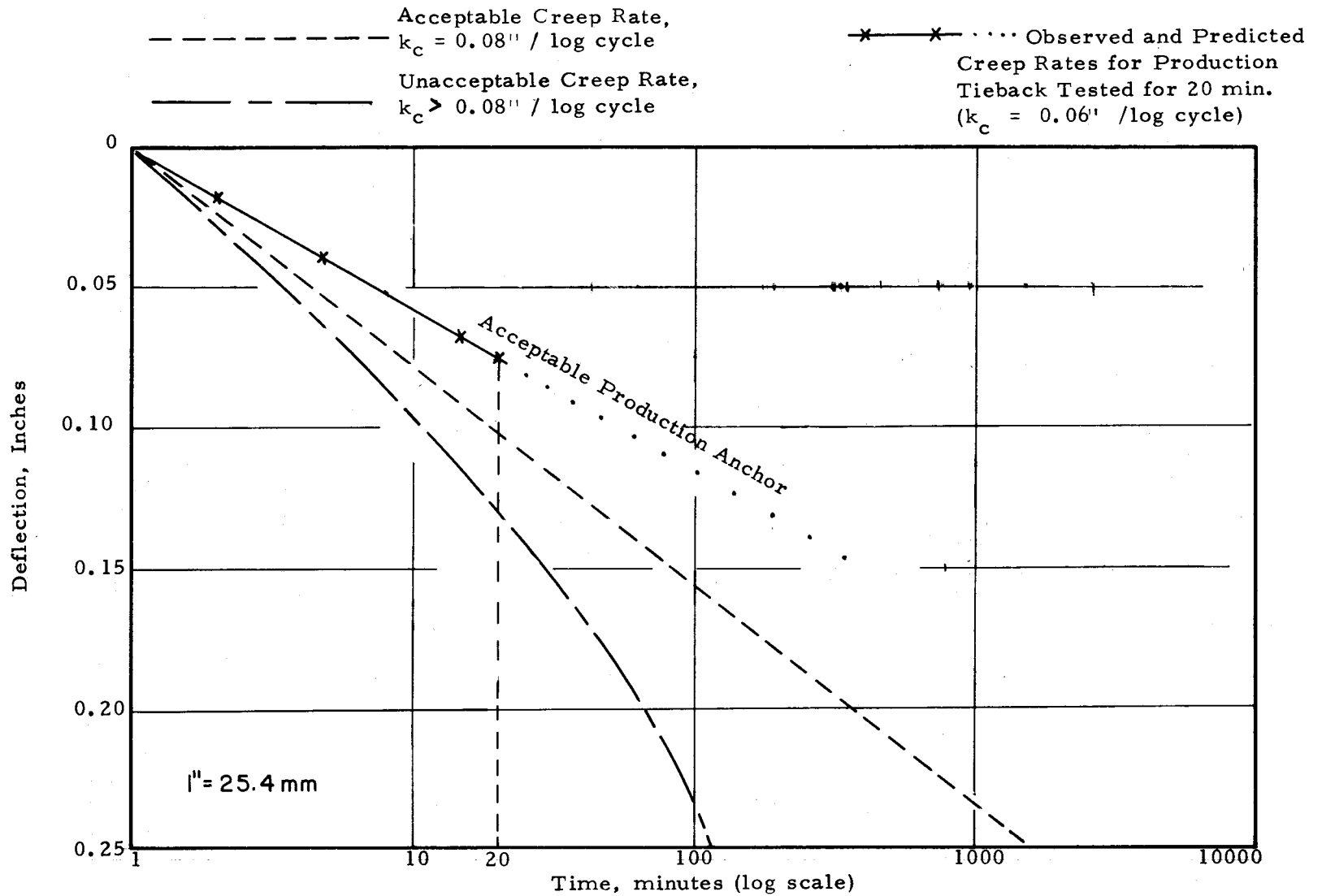


Figure 92. Example of recommended method of recording creep data.

Duration of Test Load

The load should be maintained for a minimum of 20 minutes or until a creep rate of less than 2 mm (0.08") per logarithmic cycle of time is achieved. This criterion for creep is applicable for both stiff clays and granular soils.

Method of Load Application

1. Load to 125% of the design load in increments of 25% of the design load.
2. Unload to zero.
3. Reload in increments of 25% of the design load to 125% of the design load.
4. Unload to desired lock-off load after completion of test.

c. Evaluation Anchor Test Loading

The evaluation of anchor performance necessitates the answers to two basic questions.

1. Can the anchor support the design load with an appropriate factor of safety?
2. Will excessive creep in the anchor result in a final anchor load that is unable to support the excavation?

The purpose of the special and production test loads is to determine whether the anchors are satisfactory with respect to these basic issues.

Anchor Capacity

The verification of anchor capacity is initially obtained when the applied load reaches the appropriate test level (125% - 150% of design load). However, this simple test may not be enough to ensure that the anchor capacity is sufficient. Any proof loading of ties should include a plot of load versus tie elongation. Figure 93 schematically illustrates one method that may be used to measure these movements.

Figure 94 shows a typical load vs. elongation plot for a tie. A comparison of the observed elongation curve can be made with theoretical elongation curves for several cases of "effective length"

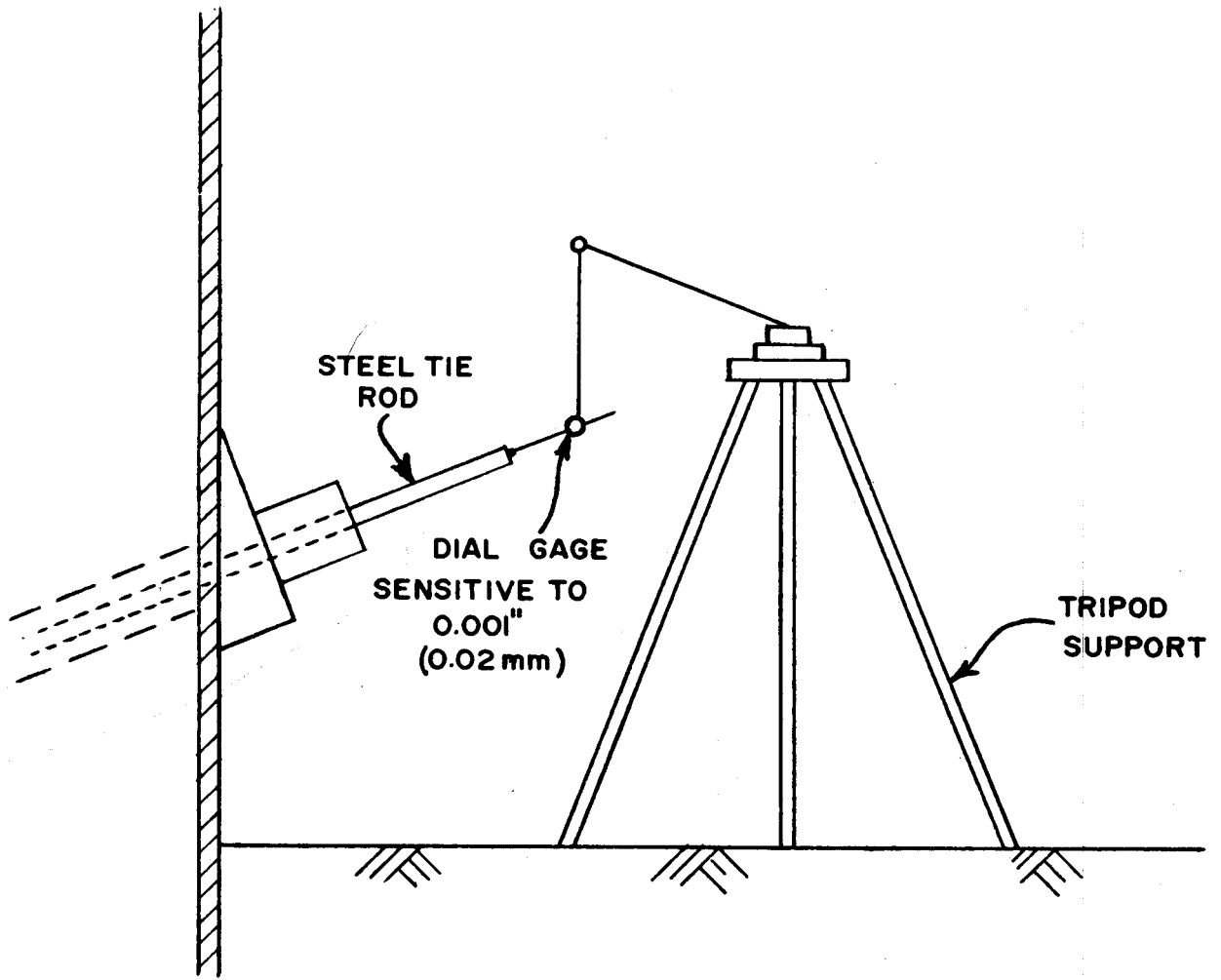


Figure 93. Schematic of method that can be used to detect creep movements in anchors.

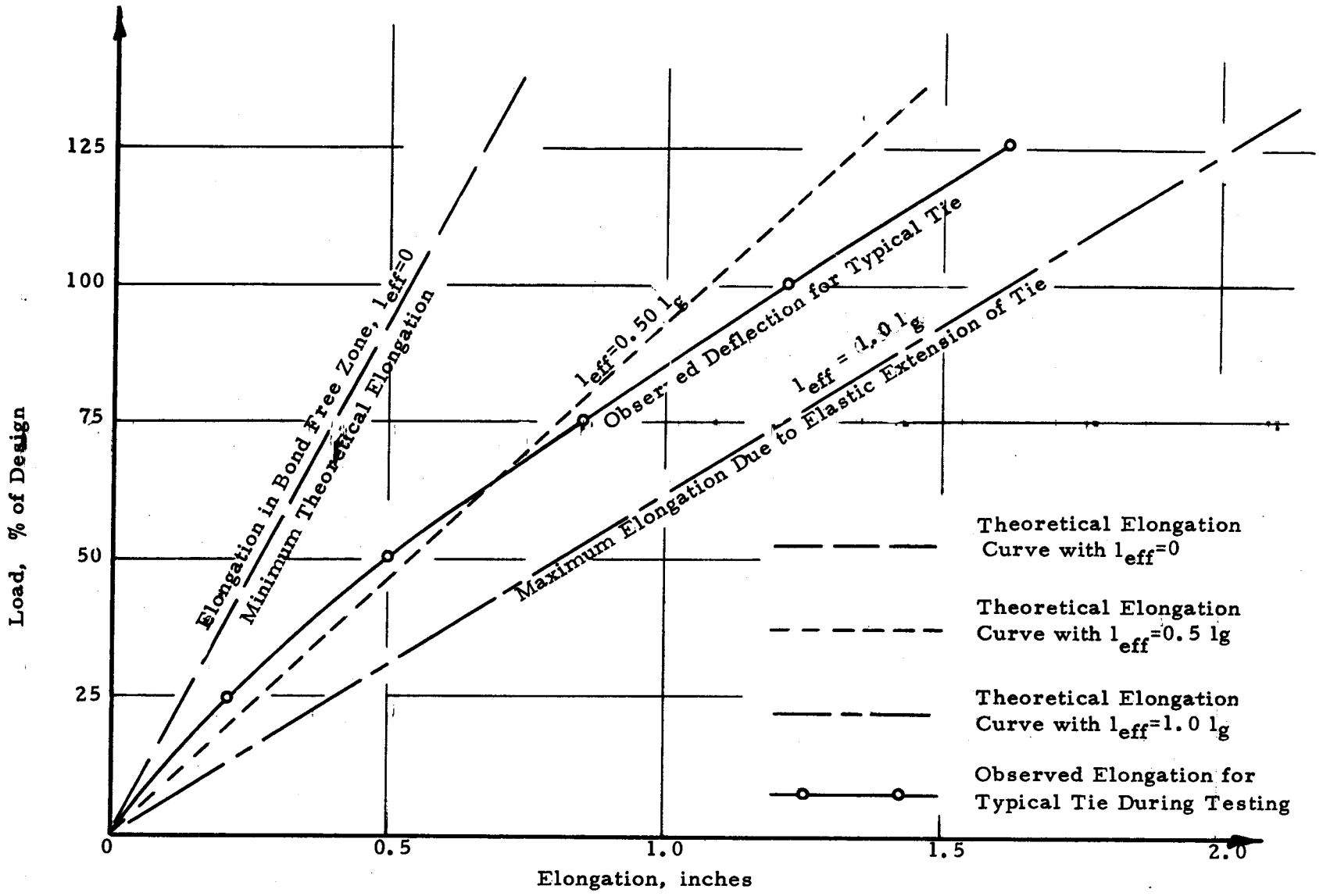


Figure 94. Typical plot of load vs. elongation during test loading.

in the grouted zone. The "effective length" can vary from zero ($l_{\text{eff}} = 0$) to the length of the tie in the grouted zone ($l_{\text{eff}} = l_g$). Zero effective length indicates an anchor in which the entire tie load is transferred at the end of the anchor nearest the bond free zone. In this case the elongation would be equal to the elongation of the tie in the bond free zone (l_b). The other limiting condition is where the entire anchor load is transferred at the base of the anchor zone. Figure 95 schematically illustrates the load distribution in the tie for several cases. In belled and compression anchors a larger elongation of the tie is expected because most (or all) of the tie load is transferred at the base of the anchor.

A comparison of this type provides some insight into the manner of load distribution in the anchor and in the soil. Since the data can be recorded and plotted directly, it is a convenient method for use in the field and during evaluation. The following equations define the important quantities in this evaluatory method.

$$\delta_b = \frac{Tl_b}{AE}$$

$$\delta_g = \frac{\int T dl_g}{AE} \quad \text{or} \quad \delta_g = \frac{Tl_{\text{eff}}}{AE}$$

$$\delta_E = \delta_b + \delta_g$$

where:

δ_E = total elongation of tie

δ_b = elongation of tie in bond free zone

δ_g = elongation of tie in grouted zone

T = tensile load applied to tie

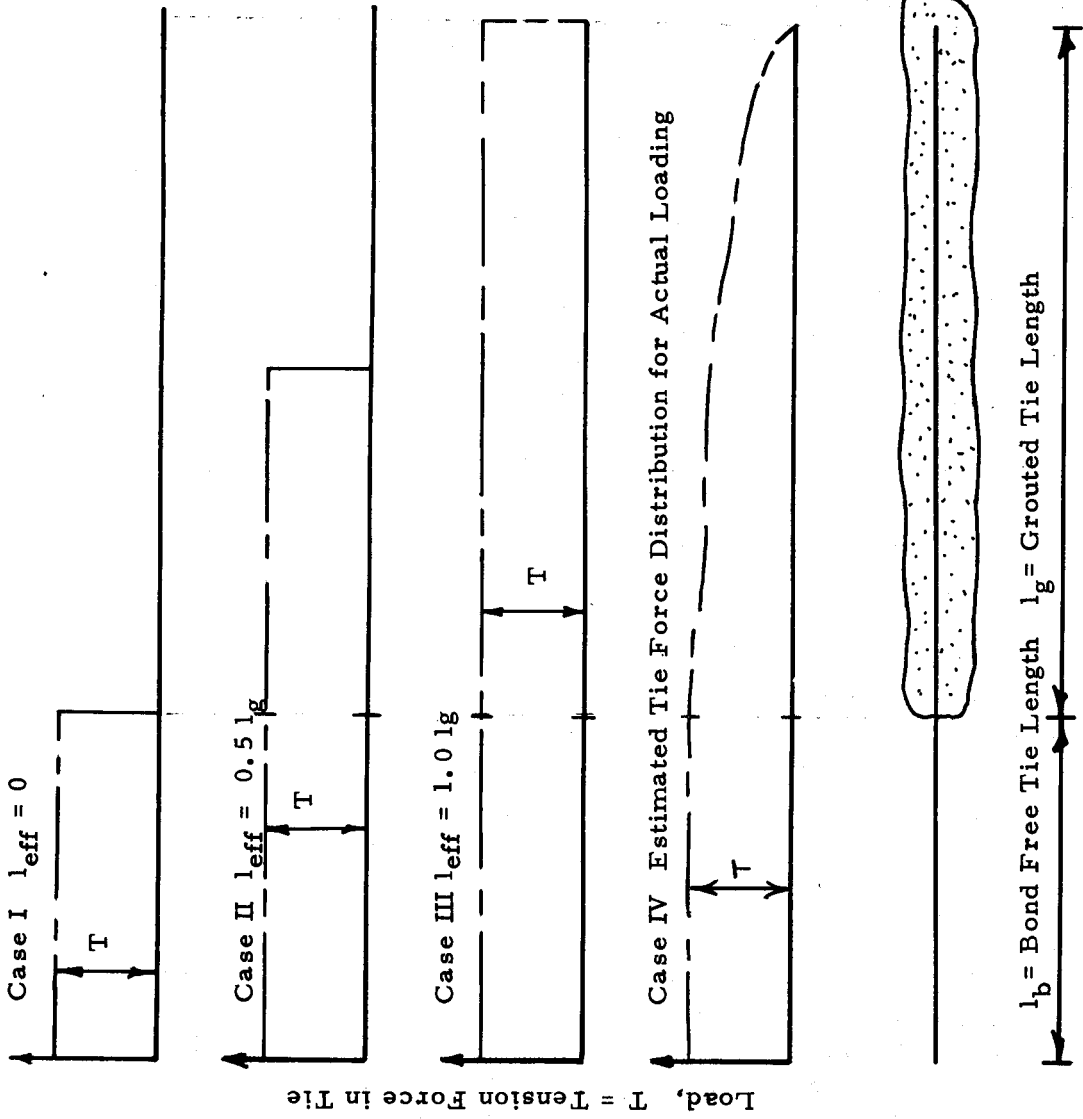
A = area of tie members

E = Young's modulus for tie members

l_b = length of tie in bond free zone

l_g = length of tie in grouted zone

l_{eff} = effective length of tie which yields same elongation as that observed under tie load



See Figure 94.

See Figure 94.

See Figure 94.

See Figure 94.

Figure 95. Idealized load distribution in tie.

As can be seen in these equations, the total elongation of the tie is dependent upon the quantities, δ_b and δ_g . While δ_b varies linearly with the tension in the tie, δ_g is a function of both the tie tension and the distribution of load in the tie. The effective length, l_{eff} , corresponds to the elongation that would be observed if the entire tensile force were applied over the effective length. As a general rule, it has been observed that as T increases, so does l_{eff} , which implies a change in the load distribution in the anchor and in the surrounding soil.

For production anchors loaded to 125 percent of design, the anchor is usually satisfactory provided the observed elongation is less than the maximum theoretical elongation of the steel tie ($l_{eff} = l_g$). Only in rare cases will anchors satisfying this condition be unacceptable (creep). However, this method does not directly consider the effects of movement of the anchor socket (along grout-soil interface). For this reason the anchor may be acceptable even though the measured elongation may be greater than the predicted maximum theoretical elongation of the steel tie. If the movement of the tie is greater than the maximum theoretical elongation, the load-deflection curve must be compared with the load-deflection curves of the special test anchors. The anchors may be evaluated on the basis of the linearity of the load-deflection curve. For example, Larson, et al (1972) established the additional criteria that the anchor was acceptable provided: 1) the deflection at 80 percent of the design load was less than the maximum theoretical elongation; 2) the deflection (Δl_2) from 100 - 120 percent of design load was less than or equal to 1.16 times the deflection

$$(\Delta l_1) \text{ from 80 - 100 percent of design load } \left(\frac{\Delta l_2}{\Delta l_1} \leq 1.16 \right).$$

Since the anchors should be preloaded to the test load and then unloaded the effects of anchor socket movement are minimized on the second load application. Therefore, unless there is a significant amount of anchor loading data from special test anchors, the requirement that the deflection at 125% of design is less than the theoretical maximum elongation is recommended.

Creep Considerations

Generally, the acceptability of a tieback is less dependent upon the ultimate load capacity in a short term loading than it is on the creep characteristics of the anchor. To assess the creep characteristics of an anchor, a plot should be prepared of anchor movement to an arithmetic scale, versus time, to a logarithmic scale. Figure 95

illustrates a plot and defines the creep coefficient, k_c , which must be less than 2mm (0.08") per logarithmic cycle of time.

An alternative to measuring the creep of the anchor in the manner described is to lock off the load and then measure the decrease in load with time. While either method can be used, the method described in detail above is preferred because the load is maintained constant while the deformation is measured. Otherwise, if the load is allowed to decrease with time, there would be an interaction between the variables of deformation and load that could not be easily assessed.

d. Lockoff Load

The amount of load locked into a tie depends upon the earth pressures and their distributions assumed for the wall.

The following recommended lockoff loads are intended to serve as a guideline for use. For design based on active earth pressures lock off load between 50% and 80% of design load. For triangular earth pressure distributions based on at-rest earth pressures, lock-off load at 100% of design. For trapezoidal and rectangular earth pressure distributions the ties in the upper one-fourth of the cut should be locked off at 80% of design; lower ties should be locked-off at 100% of design.

e. Permanent Anchors

At least three full scale pullout tests should be conducted for each soil type in which anchors are to be installed. Evaluation of the rate of creep at each stage of loading above the design load should be made. This information can be used to determine, more accurately, what the most appropriate value for use as the creep coefficient should be.

A conservative testing requirement for anchor failure under creep would be to maintain a creep coefficient, k_c , less than 1 mm (0.04") per logarithmic cycle of time at a test load of 150% of the design load. As a matter of routine all permanent anchors should be tested to a minimum of 150% of the design load as opposed to the 125% of design load recommended for testing of temporary production anchors.

Although evaluation of the creep characteristics of permanent anchors during test loading is important, it may not be sufficient to assure the safety of a permanent anchor installation. Therefore, it is

recommended that selected anchors (5%) from a permanent anchor installation be retested at later period after installation. The loads in these anchors should be checked to determine if the anchor load is being maintained or if there is a dangerous buildup of load in the anchors.

CHAPTER 7 - UNDERPINNING

7.10 INTRODUCTION

7.11 Definition

Underpinning is the insertion of a new foundation or support below an existing foundation and the transfer of load from the old to the new foundation.

The operation consists of constructing the new foundation (perhaps in stages) and then transferring load from the existing to the new foundation. Frequently, it is necessary to strengthen the existing structure or to remove the load from the existing foundation prior to installation of the underpinning elements.

7.12 Purpose

Principal reasons for underpinning are:

- a. Inadequate size or strength of the foundation or deterioration of the foundation.
- b. Inadequacy of the supporting ground.
- c. Intention to increase loads on a structure.
- d. Need for a foundation in lower, firmer material because of vibration in or near the structure.
- e. Construction of a tunnel or an adjacent deep excavation possibly causing displacements in the supporting ground.

7.13 Primary Source of Information

Literature on underpinning is sparse. The major reference is the book, Underpinning: Its Practice and Application (Prentis and White, 1950). Articles by White (1962), Tomlinson (1969), and Paterson (1970) complement the now classic work by Prentis and White. Mr. Melvin Febesh of Urban Foundation (New York) supplemented the textual information with his own insight and practical experience in underpinning of foundations. Mr. Febesh prepared much of the basic material, and this chapter reflects his

considerable experience in the field of underpinning.

Additional information was obtained from published accounts of specific underpinning applications. However, with few exceptions these discussions are qualitative and rarely report on performance. Emphasis is upon the "art" of the technique rather than upon engineering fundamentals. An exception is the work of Ware (1974) which presents quantitative data concerning performance of underpinning in connection with subway construction for the Washington Metro.

7.14 Execution

Before beginning underpinning operations, a careful examination should be made of the existing structure and (after it is exposed) its foundation should be made. Much information may be obtained by examining original building plans and by examining records available in building departments.

Since underpinning requires that a portion of the existing foundation be undermined, the structural integrity of the existing structure should be evaluated. This evaluation should include a determination of existing bearing pressures, soil conditions, ground water level, total loads on footings, and a determination of whether the existing foundation has some excess capacity. This investigation will determine the extent of the underpinning operation and determine the constraints which are required to maintain structural integrity.

While the purpose of underpinning is to prevent vertical displacements and strengthen the foundation through additional vertical support, the underpinned structure is not necessarily free of displacements. Even the best underpinning procedures will result in about 1/2 inch of settlement from the transfer of load. Finally, underpinning elements are embedded within the earth mass which undergoes both horizontal and vertical displacements -- thus, the elements will either move or will accept additional load.

7.20 DESIGN AND THEORETICAL CONSIDERATIONS

7.21 Load Computation

7.21.1 Existing Structures

The load of the existing structure can be determined from building drawings. Failure to locate the plans for the building (as is often the case in older structures) necessitates an analysis of the structure to estimate the existing foundation loads.

7.21.2 Load Distribution

The location of underpinning elements is often determined by the structural characteristics of the existing foundation. In addition, as the load is progressively transferred to the new foundation, the distribution of the foundation load changes. The existing foundation should be analyzed for each of the intermediate stages since the foundation could fail or settle excessively if allowable loads are exceeded.

7.22 Deformations

7.22.1 Displacements Resulting from Adjacent Construction

Even though a structure is successfully underpinned, it still may suffer damage from the adjacent excavation. Lateral displacement leads to cracking when one portion of the structure shifts relative to another portion of the structure. Vertical displacement below the bearing level contributes to additional load on underpinning elements. This may also cause settlement.

Lateral displacements of the soil mass will either cause the underpinning elements to move or will cause them to accept additional horizontal load. Tiebacks or braces may be employed to provide the resistance needed to withstand horizontal forces.

Vertical displacements may result in downward forces transmitted by friction along the side of the element. The resulting force is classically referred to as downdrag caused by negative skin friction. This vertical displacement may be associated

with consolidation of compressible soils or it may be associated with non-volumetric vertical strain within the earth mass bordering the excavation.

Several examples of settlement of underpinned buildings adjacent to excavations have been reported. Peck (1969) refers to settlement of a structure on piles adjacent to a cut and cover tunnel project. O'Rourke and Cording (1974a) cite settlements which might have been caused by downdrag on new underpinning elements. NGI (1962, No. 7) reports the case of an underpinned structure that moved substantially during construction of a cut and cover tunnel.

7.22.2 Settlement from the Underpinning Installation

Sources of settlement unique to each type of underpinning operation are discussed in Section 7.30. General sources are noted below:

a. Structural Elements. Settlements may be elastic in nature due to an increase in load. Non-elastic deformations may stem from creep and shrinkage of the concrete used for underpinning, as in pit underpinning.

b. Bearing Stratum. Settlements are caused by strain within the bearing stratum.

c. Construction Procedures. The two main sources of settlement during construction are loss of ground during excavation and the strain associated with load transfer. These will be discussed in detail for the various construction procedures.

d. The Structure. The integrity of the existing structure must be considered. Of special interest are old masonry walls, in which brick and mortar may have seriously deteriorated, and structural members (both walls and columns) that might not withstand the bending moments induced during load transfer.

7.23 Design of Underpinning Elements

7.23.1 General

While the actual design of the underpinning elements is relatively straightforward, the choice of an underpinning system and selection of a bearing stratum are more complex. Experience with the various types of underpinning systems is absolutely essential in choosing the best system.

7.23.2 Downdrag and Horizontal Forces

As discussed in Section 7.22, underpinning elements are influenced by displacements occurring in the soil mass within the zone of influence of adjacent excavations or tunneling. Underpinning elements may settle, may shift laterally, and/or may receive additional load.

The recognition of these factors and an assessment of their implications is vital.

7.23.3 Group Action

Because of interaction between piles, a pile group stresses soil to a greater depth than does a single pile. Thus, for a given load per pile, the settlement of a group of piles will be larger than for a single pile.

The concept of group action is important to gain an understanding of the mechanics of preloading pile underpinning elements to a desired locked-in load. Normally, piles are preloaded singly rather than in groups. This will cause elastic deformation of the pile and some compression of its bearing stratum. Subsequent installation and preloading of adjacent piles may cause additional strains in the bearing stratum and relief of load. Unless taken into account during preloading, piles in a group that are preloaded and locked-off separately may settle more than expected under the full structure load.

Whether group action is of significance will depend upon a number of variables -- the proximity of piles, characteristics of bearing stratum, and sequence of preloading. Normally, group action will not be important for piles spaced greater than 3 diameters apart or piles bearing on very competent granular soils or rock. Volume II (Design Fundamentals) discusses the bearing capacity of deep foundations in greater detail.

7.24 Prerequisites for Underpinning

Whether or not a structure should be underpinned will be controlled by one or more of the following criteria:

- a. Potential damage from displacements caused by the adjacent excavation.
- b. The cost of underpinning compared to the cost of protective measures to prevent excessive displacement (e. g. diaphragm wall, special techniques for lateral support, etc.)
- c. The cost of underpinning compared to the cost of the structure to be underpinned.
- d. Consideration of community reaction over damage to structures.

Empirical and theoretical tools for displacement prediction are presented in Volume II (Design Fundamentals). With regard to cost, underpinning is expensive; nevertheless, each case must be evaluated separately. In weighing underpinning and other viable options, experience and subjective judgment are essential, especially in evaluating the trade-offs between cost, risk, and community reaction.

7.30 CLASSICAL UNDERPINNING PROCEDURES

7.31 General Considerations

The objectives of underpinning are to transfer the foundation load to a firm bearing stratum with a minimum of movement. The underpinning operation must be coordinated with the overall construction project, especially when the underpinning system is incorporated into the lateral support system or the final new construction.

7.32 Pit or Pier Underpinning

7.32.1 General

Probably the most common method of underpinning is the use of concrete filled pits or piers which have been excavated using horizontal wood sheeting to retain the earth. The construction procedures for this method have not changed significantly since the technique was first used. The methods used for access below the foundation form the basis of other underpinning procedures.

7.32.2 Procedure

The basic procedure for installing a concrete underpinning pier is as follows:

1. Excavate a pit immediately adjacent to the footing to be underpinned. This pit should be approximately 4 feet long (along the length of the footing) 3 feet wide and 4 feet deep (see Figure 96a).
2. Sheet with horizontal wood sheeting, making the sheeting bear tightly against the ground. Pack behind the sheeting boards as required to obtain the bearing (See Figure 96b). The completed pit (commonly called an "approach pit") provides access below the existing footing.
3. Excavate beneath the existing footing to the depth of the approach pit.
4. Sheet the portion of the pit beneath footing, packing the earth as required. Make sure that the sheeting boards bear tightly against the earth (see Figure 96c).

Sheeting for pits is normally 2 inches thick, the width of the board (8, 10, or 12 inches) being determined by the nature of the soil being retained. The most common sizes of excavated pits are 3 feet x 4 feet or 3 feet x 5 feet; however, square pits, 4 feet x 4 feet, 5 feet x 5 feet, or 6 feet x 6 feet are not uncommon. Pits 10 or 12 feet on a side have been excavated, but thicker sheeting and sometimes supplementary bracing of the pits are required.

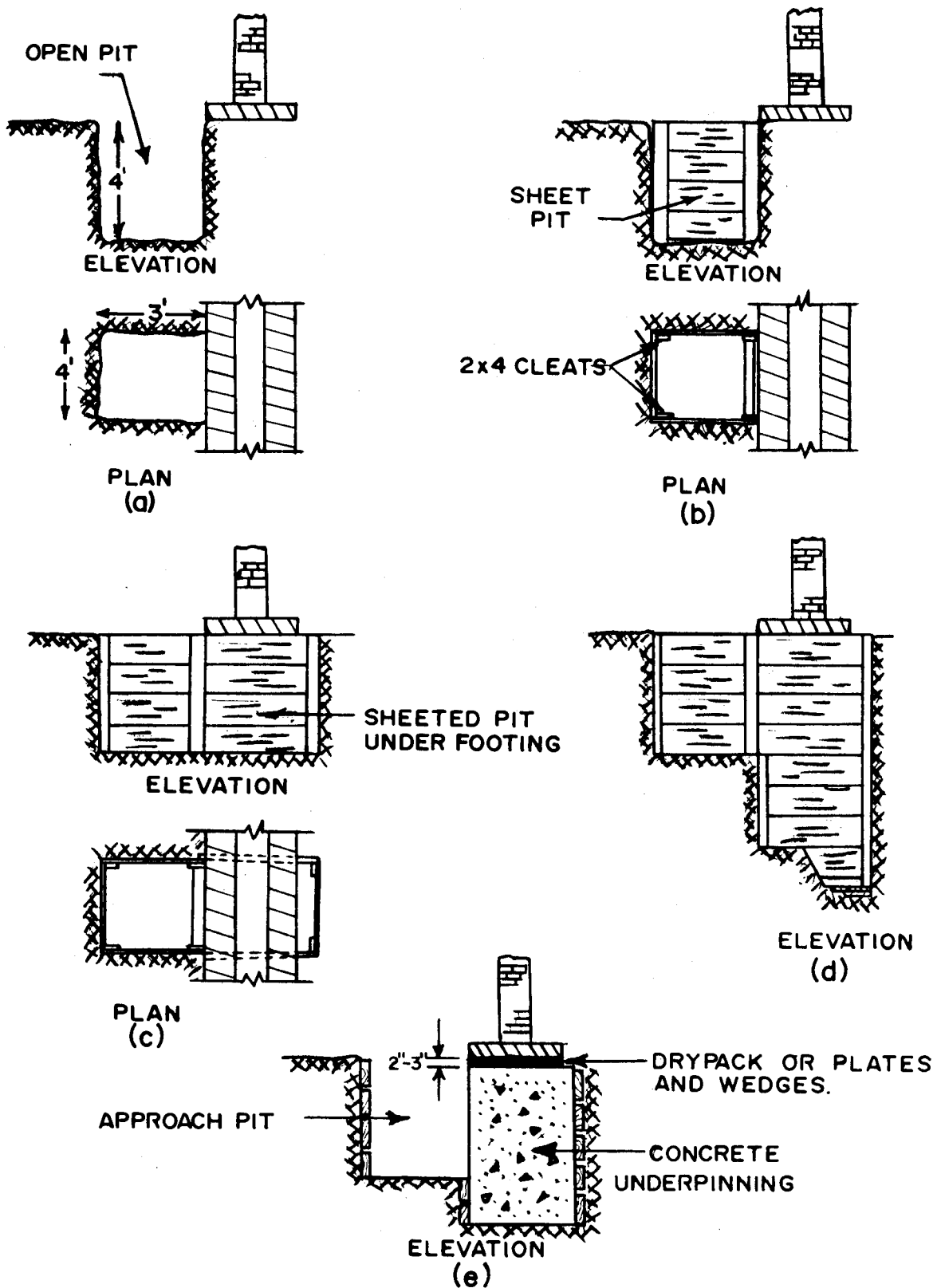


Figure 96. Pit or pier underpinning.

During the placement of the sheeting spaces are often left to permit packing of the soil behind the boards. These spaces are called louvres and are formed by nailing short pieces of wood between the sheeting boards. When sheeting is installed in a pit, the corners lap over each other, and the boards are toenailed in place. Alternate tiers of sheeting have the laps in alternate corners. Very often wood cleats are nailed in the corners after they have been toenailed in place.

5. Continue excavating the pit beneath the footing, excavating deep enough to install one ring of horizontal wood sheeting at a time. Each ring should be placed against the soil, packing the soil as required (see Figure 96d).

6. After the pit has been excavated and sheeted to the required depth, fill the pit with concrete to within 2 or 3 inches of the underside of the existing footing.

7. After the concrete has set, transfer the load from the footing using dry pack or plates and wedges. The time for setting of this concrete is typically 24 hours for high early cement and 48 hours for regular cement (see Figure 96e).

7.32.3 Discussion

Load Transfer

The space between the top of the pier and the foundation is normally filled with drypack -- a mixture of cement and moist sand. Dry pack is rammed in place with pieces of scrap lumber. It later hydrates and forms concrete.

Under certain circumstances settlement associated with load transfer may not be acceptable. In such cases jacks may be inserted between the top of the concrete piers and the underside of the footing (the jacks can also be placed in pockets formed in the underpinning piers), and loads maintained on the jacks. This would permit the concrete pits to settle while maintaining the structure at its original elevation.

Horizontal Wood Sheeting

The thickness of the sheeting is essentially independent of depth as the stresses in the soil are distributed by an

arching effect identical to that discussed in Chapter 2 on Soldier Pile Walls. The main exception to the spacing and sizing guidelines specified there is that in shallow pits (less than 8 to 10 feet deep) or in cohesive soils excavation and concreting can be done in one shift. Under these conditions sheeting requirements are less critical.

The material used for sheeting is commonly untreated wood. Occasionally, because of concern over future deterioration, specifications require treated wood, concrete planking, or steel sections. The issue of wood rotting is presently controversial. Many contractors have found that even with deterioration the fabric of the wood remains intact, thus preventing earth from filling the space occupied by the wood. (See Section 2.43 of Soldier Pile Walls).

Pit Size

The size of an underpinning pit is determined by several factors.

1. It must be large enough for a man to work in and to perform the sheeting and packing operations properly.
2. It cannot be so large that when the boards are in place they will deflect a large amount before the concrete has been placed.
3. The pit cannot be so large that it will undermine the footing to an extent that would cause settlement. (This assumes that the column or footing has not been temporarily supported).

Pit spacing and sequence of pit excavation must allow the remaining portion of the foundation safely to support the entire foundation load. Primary underpinning pits are completed at the selected spacing. A secondary sequence of pits is completed at the same spacing. The process continues until the required underpinning is installed. If the underpinning work will cause the foundation to be inadequate at any intermediate stage, then some form of temporary support will be necessary during the underpinning operation.

Soil Removal

When excavated pits are so deep that the man excavating the pit cannot throw the soil out of the hole, several methods of soil removal are used. A scaffold can be built part way up the pit so that the man at the bottom can throw the dirt up onto the scaffold and another man can throw the dirt from the scaffold out of the pit.

If the excavation depths are large, several tiers of these scaffolds can be used. An alternative is to use buckets filled manually. Excavated soil can be raised manually by pulley or by power winches. In some areas the unions will require an engineer to operate the power winches, making the cost of power winches excessive.

Belled Piers

Underpinning pits can be enlarged or belled at the bottom. There is a possibility for loss of ground if this operation (including sheeting of the bell) is not performed carefully. This is especially true in non-cohesive soils.

7.32.4 Source of Potential Settlement

General

The faster a pit is concreted, the less chance there is of having excessive settlements of adjacent footings or floor slab. Settlement may be caused by improper backpacking of horizontal sheeting, from excessive deflection of the sheeting, or from "loss of ground" -- that is, movement of soil into the pit excavation.

Weak Soils

Loss of ground may be caused by an outflow of "running soils" -- saturated non-cohesive soil such as silt or fine sand and silt, which are difficult to drain. Ground loss may also be caused by the movement of "squeezing soils". Weak cohesive soil, such as soft clays having a stability number greater than 5 are particularly susceptible. In both cases the threat of ground loss

exists during exposure of the soil face prior to placing lagging, after lagging placement by movement through open lagging, or by movement into an overcut zone behind the lagging.

Ground Water

Pit or pier underpinning is best suited for dry ground. If the bearing stratum is below the water table in granular soil, another type of underpinning method must be used or the ground water lowered in advance. Special techniques (vertical sheeting or tunneling methods) may be required in difficult conditions such as "running" ground.

If conditions do not permit the use of alternate methods, it may be necessary to resort to vertical wood or vertical steel sheeting to maintain the sides of the pit. This is both risky and expensive. The portion of the pit above the water level may have to be enlarged to permit the installation of the vertical sheeting inside the horizontal sheeting. If pumping is not properly performed, there is a risk of ground loss from behind the sheeting or of an unbalanced hydrostatic head causing a "blow" at the bottom of the pit.

A particularly sensitive situation is the case of sand or gravel formations that may be stratified with impervious layers which tend to support perched ground water levels even after dewatering with deep wells, well points, or sumps. Insufficient dewatering may result in erosion of soil by flow of water into the pit through open lagging.

7.32.5 Examples

Figures 97 through 102 illustrate examples of pit underpinning. Figure 97 illustrates a typical approach pit while the remaining photographs illustrate several pit underpinning installations.

7.33 Pile Underpinning

7.33.1 General

Piles are often used when the bearing stratum

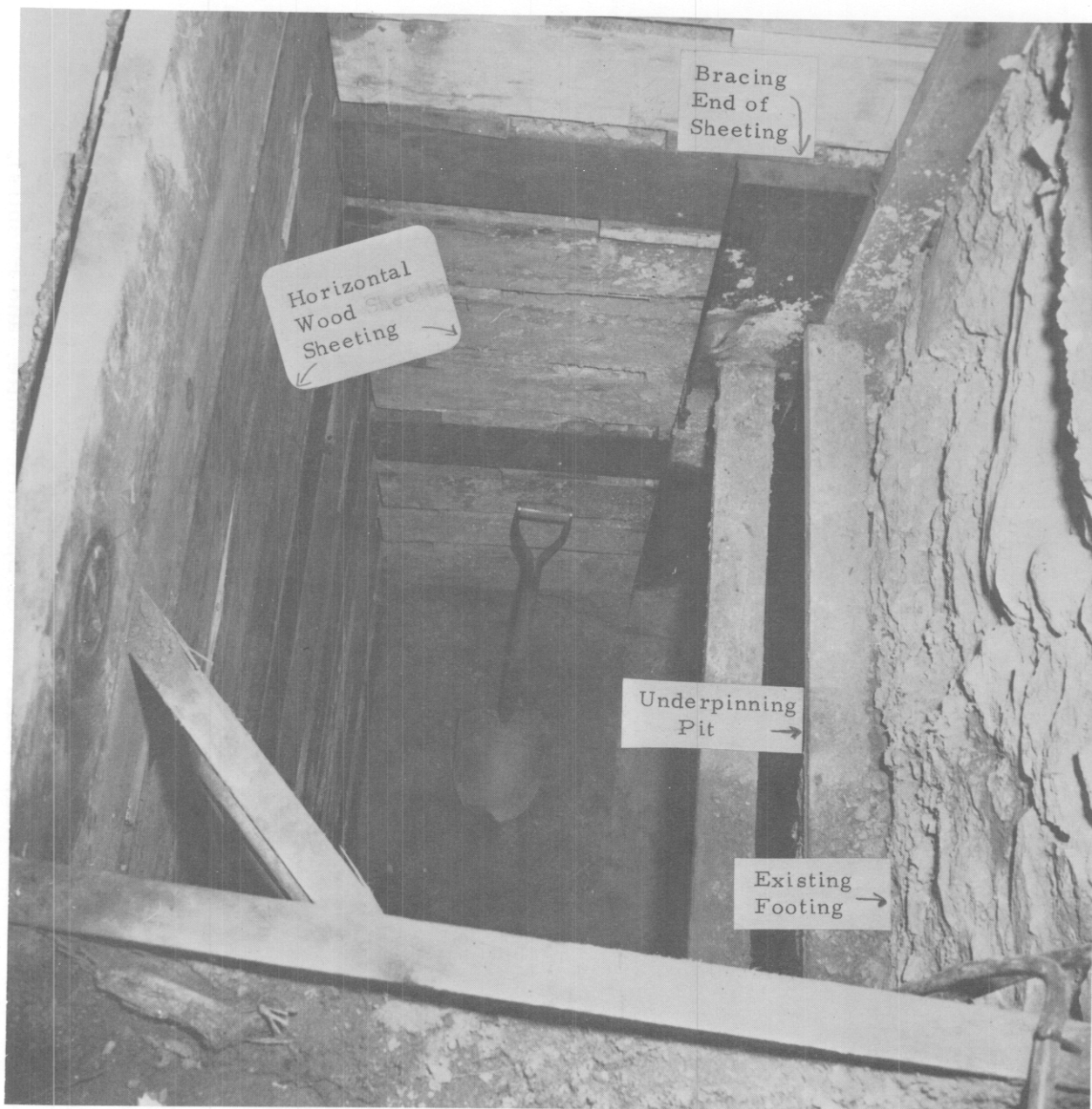


Figure 97. Details of pit underpinning.
(Courtesy of Spencer, White, and Prentis).



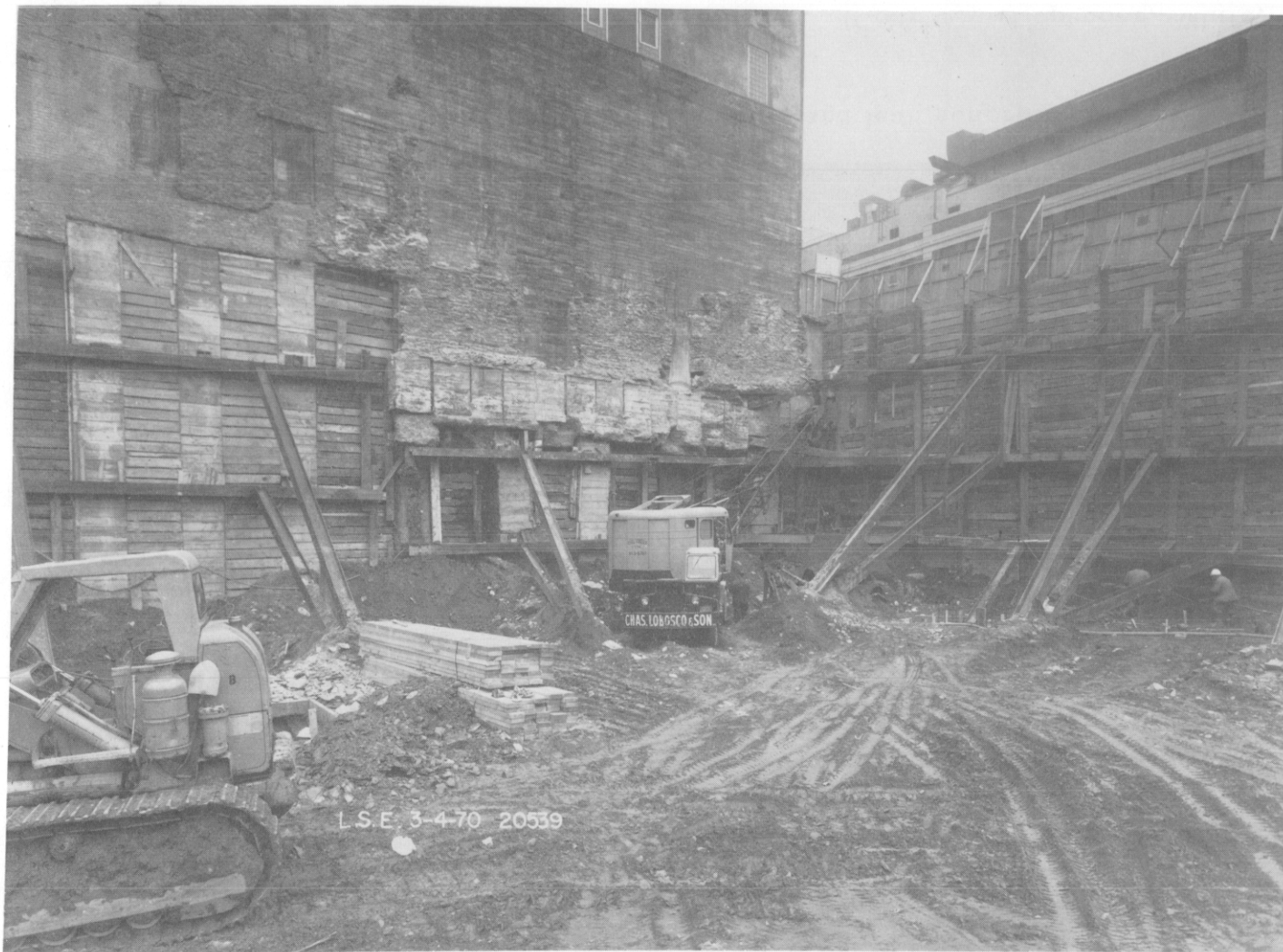
Note: Bracing for lateral support.

Figure 98. Pit underpinning. (Courtesy of Spencer, White, and Prentis).



Left side shows augered hole with steel soldier pile and lean concrete backfill. Right side shows pit underpinning and interpit sheeting.

Figure 99. Underpinning supported by earth tiebacks. (Courtesy of Spencer, White, and Prentis).



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- Note: 1. Wood lagging spanning between pits.
2. Spacer blocks and shuttered lagging.

Figure 100. Pit underpinning. (Courtesy of Urban Foundation Co., Inc.).



- Note: 1. Steel plates for bearing of tiebacks into rock.
2. Irregular rock surface and drill hole marks (left side photo).

Figure 101. Continuous pit underpinning.
(Courtesy of Urban Foundation Co., Inc.)



Note: Excavation underway for approach pits.

Figure 102. Pit Underpinning. (Courtesy of Urban Foundation Co., Inc.).

is at great depth, where ground water is a problem, or where column loads are relatively high. Any one of these factors or a combination of them might make pit underpinning too costly or too risky.

The materials for the piles and the basic installation procedures are the same as in conventional pile installations; however, underpinning piles are often installed from inside structures and as a result have unique problems. Commonly, piles are jacked in place. If piles are to be installed by driving, the hammer and pile sections must be short enough to be installed within the available head room. The need for short sections requires use of materials which can be easily spliced, thus eliminating wood piles.

Generally, H-beams or steel pipe piles (both open-and close-ended) are used in underpinning. H-beams and open-ended pipes are preferred in most cases. They are low displacement and therefore encounter relatively little resistance during driving. Open-ended pipe permits cleaning out soil to reduce end resistance and side friction. Close-ended pipe is used to penetrate through soft soils and/or where displacements and vibrations from pile driving do not have a significant effect.

Piles can be installed either directly under or alongside a footing. If the piles are alongside the footing, the load can be transferred either to a beam connecting two piles or to a bracket on a single pile. The load carrying capacity for the bracket pile is limited by the asymmetric loading on the pile and consequently can only be used for light loads. The use of a beam to carry the load is often restricted by the accessibility to either or both sides of the footing.

When excavations are made adjacent to an underpinned structure, it is not uncommon to use the underpinning as part of the earth support system. Piles are commonly used as soldier beams in a system with lagging. In this case, the pile will support lateral loads in addition to the axial loads of the foundation and must therefore be designed accordingly. For instance, welded splices would be necessary in a system employing H-beams, and welded splices or reinforcing steel might have to be added in a steel pipe pile.

7.33.2 Pile Installation

Jacked Piles

Typically, aluminum hydraulic jacks are used because they are light and easy to handle in a confined pit. The jacks are usually designed to retract automatically. The footing is conventionally used as the reaction, and the jacks are normally capable of developing 40 to 60 tons.

Jacking loads should be monitored to prevent an excessive upward force on the foundation before reaching the desired bearing level. In such cases, measures will have to be taken to reduce resistance. Coating the pile with lubricants can reduce resistance.

Except in soft material jacking is done with open-end pipe to permit removal of soil from within. In soft soils, a plug is formed using cinders, sand, or lean concrete. This plug permits advancing the pile through the soft strata without permitting the soft material to enter the pipe. When the soft material has been penetrated and jacking pressures start to build up, the plug can be cleaned out, and jacking and cleaning of the pile can commence in the normal manner.

The typical procedure for installing jacked piles is as follows:

1. Excavate a sheeted approach pit and a sheeted pit under the footing. The pit under the footing should be large enough for a man to work in, say 3 feet x 4 feet in plan and about 6 feet deep.
2. Fasten a steel plate to the underside of the existing footing, providing level bearing with drypack or mortar.
3. Stand a section of steel pipe in the bottom of the pit approximately 4 feet to 5 feet long. Place a steel plate on top of the pipe. Place the jack on the top of the plate and, if required, fill the space between jack and plate on the underside of the footing with steel blocking which may consist of pipe, plates, or H-sections.
4. Commence jacking the pipe into the ground, using additional steel blocking as required. When the top of the pipe is approximately at the bottom of the pit, remove the jack and blocking. Clean the pipe if required. Add a jacking sleeve and the next section of pipe. Replace the plate on top of the pipe, block, and commence

jacking. The basic configuration for the jacking is shown in Figure 103. If jacking pressures build up, cleaning may have to be done several times for each section of pipe installed.

5. Repeat jacking, cleaning, and blocking until required penetration is reached.

6. Clean out the pipe. Add additional sleeve and pipe so that the space at the top is approximately 1 to 1½ feet below the footing.

7. Fill the pile with concrete.

8. Test load as follows:

a. Put a plate on top of the pile large enough to accommodate two jacks. Place two jacks on top of the pile. Add plates on the underside of the footing if required for the jacks to bear against.

b. Test pile to 150 percent of design load.
(Note: Testing of the pile is often done before placing concrete in the pile).

9. Transfer load as follows:

a. With the full load on jacks, measure the space between the top and bottom plates, and cut an I-beam section approximately 1 inch shorter than the space between the plates.

b. Place the I-beam over the center line of the pile between the jacks. Place an additional plate on top and wedge between plates.

c. Drive wedges until pressure gages on the jack lines indicate load has been removed and is now going directly from the footing, through the I-beam (called a wedging strut), and into the pile. Remove the jack.

d. Backfill the jacking pit to approximately two inches below the underside of the plate on top of the pile.

e. Encase the wedging strut and plates in concrete.

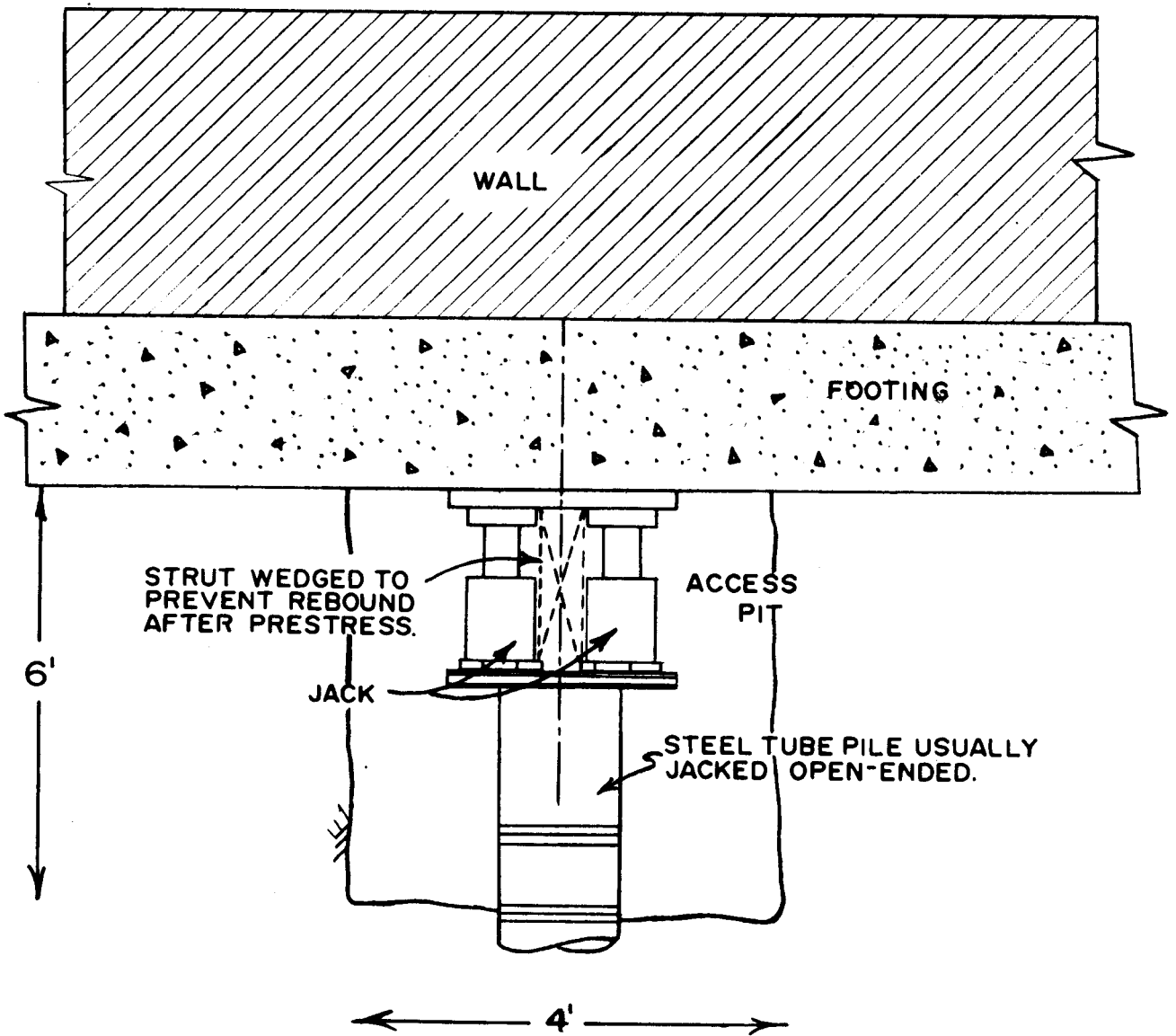


Figure 103. Jacked pile installation.

10. Complete backfill of jacking and approach pits as required.

When piles are installed in groups and there is the potential for additional settlement from group action, group testing should be considered.

When there is a question of the competency of the bearing material, the tops of the piles may not be encased after pre-testing but may be left open to allow retesting.

Two examples of jacked pile underpinning are shown in Figures 104 and 105.

Driven Piles

Conventional hammers or drop weights can be used to drive piles. When using a conventional hammer, the energy that can be developed by the hammer is often limited by the size of the pit that must be excavated beneath the footing to accommodate the hammer. In other words, a pit must be deep enough to accommodate a) the section of pile to be driven (say 5 feet or 6 feet long), b) the hammer, and c) the blocking, chain falls, etc. required to support the hammer.

Piles are driven in sections with splices made between successive lengths. Open-ended pipe may be cleaned out, if required, to reduce resistance.

When piles are installed below foundations, driven piles may be test loaded by jacking against the foundation. Load transfer is done in a fashion similar to that used for jacked piles.

Advancing Open-Ended Pipe

Reduction of side friction or end resistance during installation is accomplished by periodically cleaning the soil out from within open-ended pipe. Sections of pipe are connected by tight fitting sleeves generally fastened on the outside of the pipe. These outside sleeves are used (rather than using inside sleeves), because they do not create any interference on the inside which might make it more difficult to clean out or remove obstructions which might be encountered. The sleeves are not normally welded but are designed to keep the sections of pile in alignment.

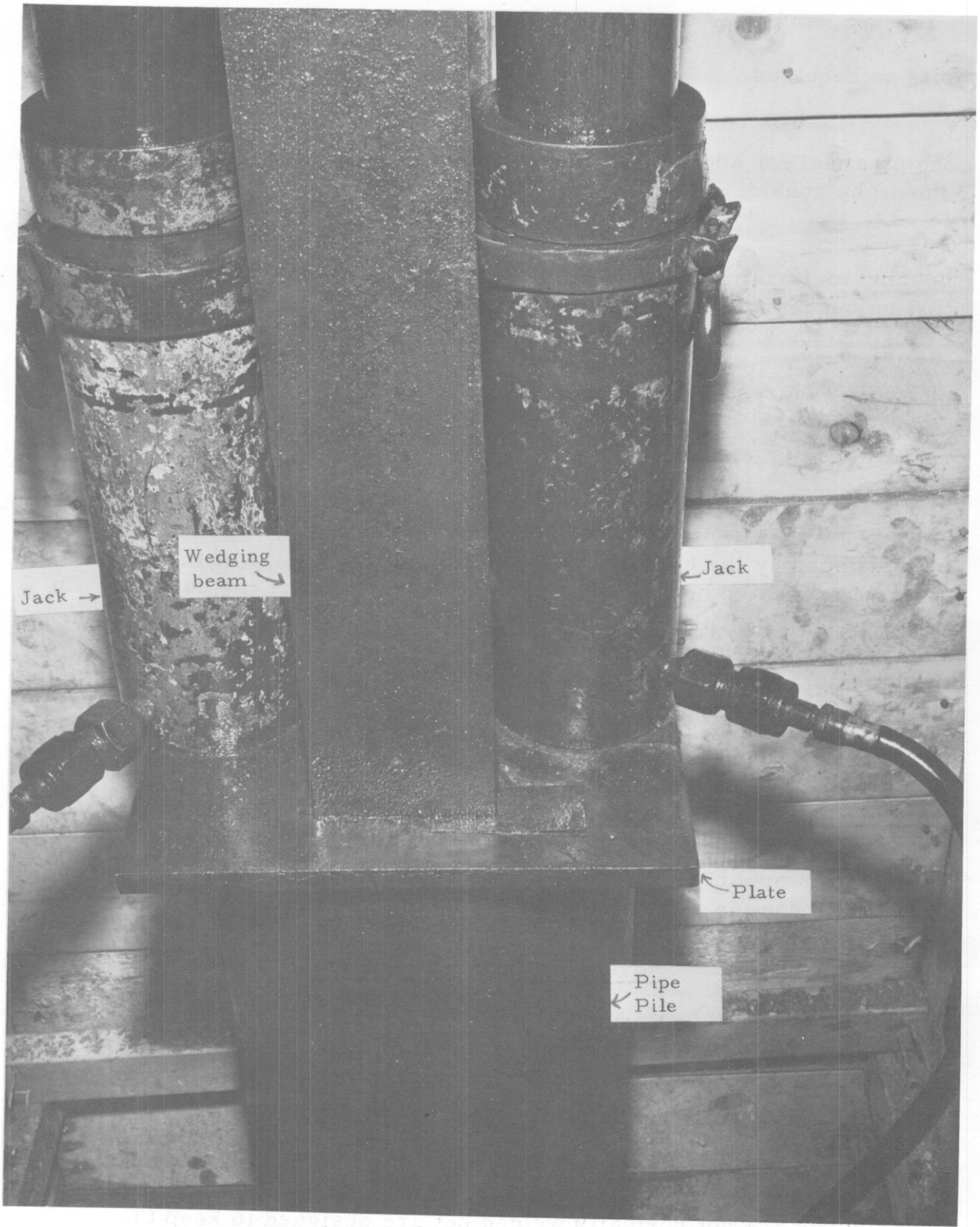


Figure 104. Prestressing of underpinning pile. (Courtesy of Spencer, White, and Prentis).

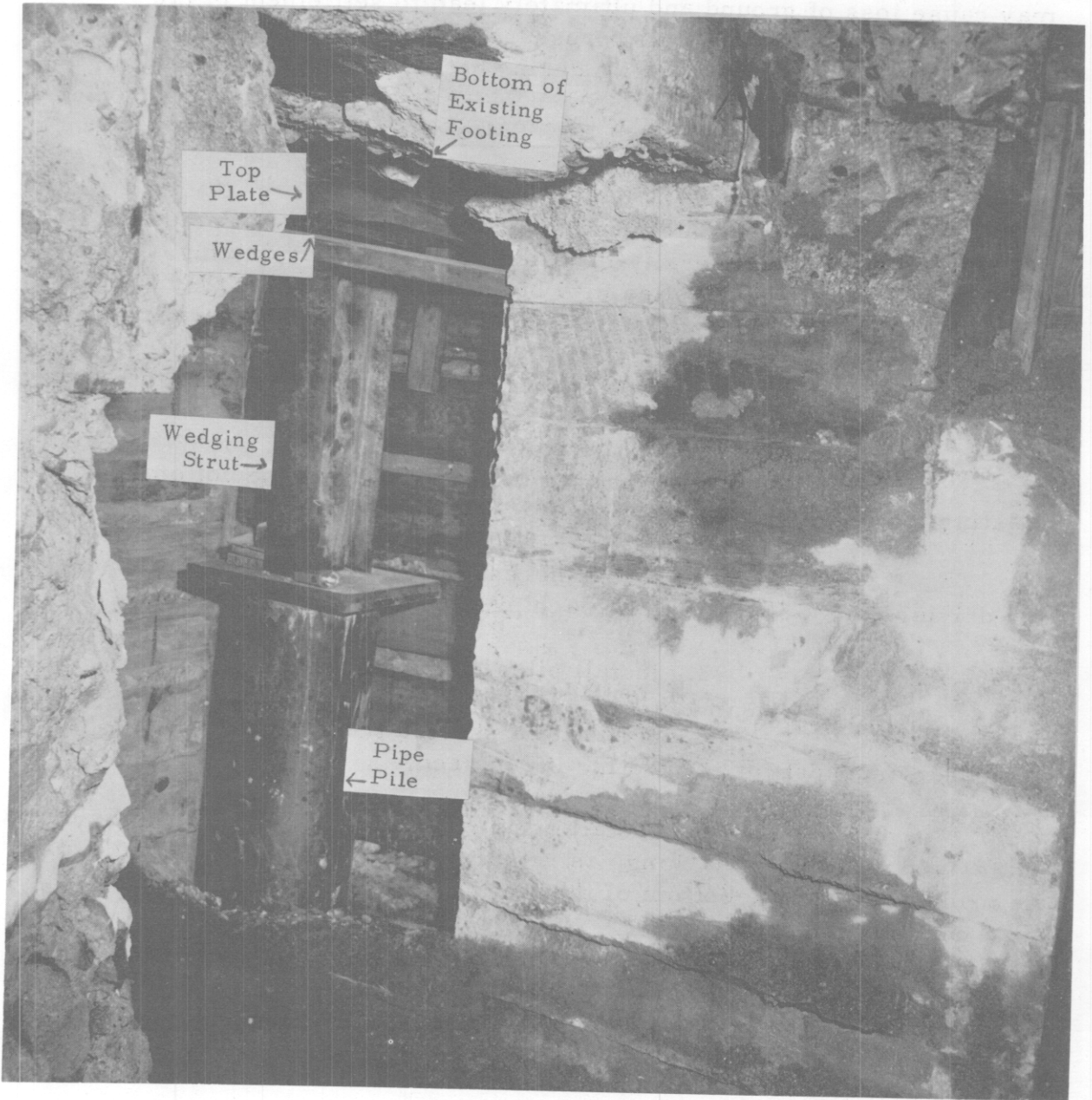


Figure 105. Jacked pile underpinning details.
(Courtesy of Spencer, White, and Prentis).

Piles can be cleaned using various tools such as pancake augers, flight augers, orange-peel buckets, water jets, air jets, or water/air jets. When using any of the jet cleaning methods, care should be taken not to clean below the bottom of the pipe as this may cause loss of ground and ultimately lead to settlement of the footing. A positive hydrostatic pressure must be maintained to prevent a "blow" at the bottom of the pile during both cleaning and driving.

7.33.3 Piles on Both Sides of Footing - Support with Beams

This method requires access to both sides of a footing. Piles are generally installed by augering or driving. In greatly restricted areas, piles may be installed by jacking if it is practical to provide temporary framing to develop the necessary reaction.

When piles are installed on both sides, the basic procedure is as follows:

1. Excavate to approximately the bottom of the existing footings.
2. If it is necessary to obtain sufficient headroom for driving, dig a sheeted pit at each pile location.
3. Install piles.
4. Excavate a sheeted trench for one beam using temporary support for the footing if required.
5. Install one beam and transfer the load by drypack, plates and wedges, or jacking, as required. This transfer of load can be made at either the bottom of the footing or at the top of the pile or a combination of the two.
6. Install additional beams, one at a time, completing the load transfer for each beam before the next trench is excavated. (Note: It may sometimes be necessary to provide temporary shoring during installations of the beams).

The beams can be steel, reinforced concrete, or post-tensioned concrete. If it is necessary to encase the steel beams in concrete, this can be done either after the load is transferred to one beam or when the entire footing is underpinned. Possible configurations for either a wall or column are presented in Figure 106.

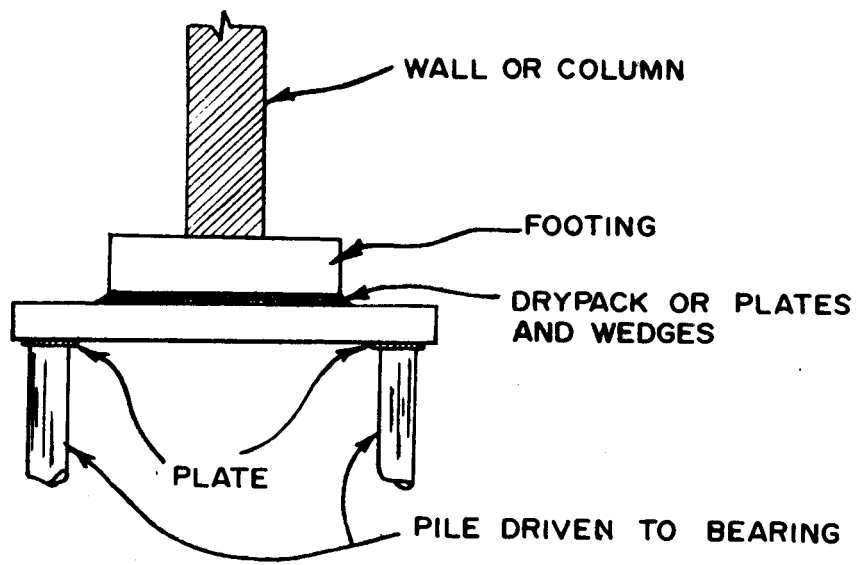
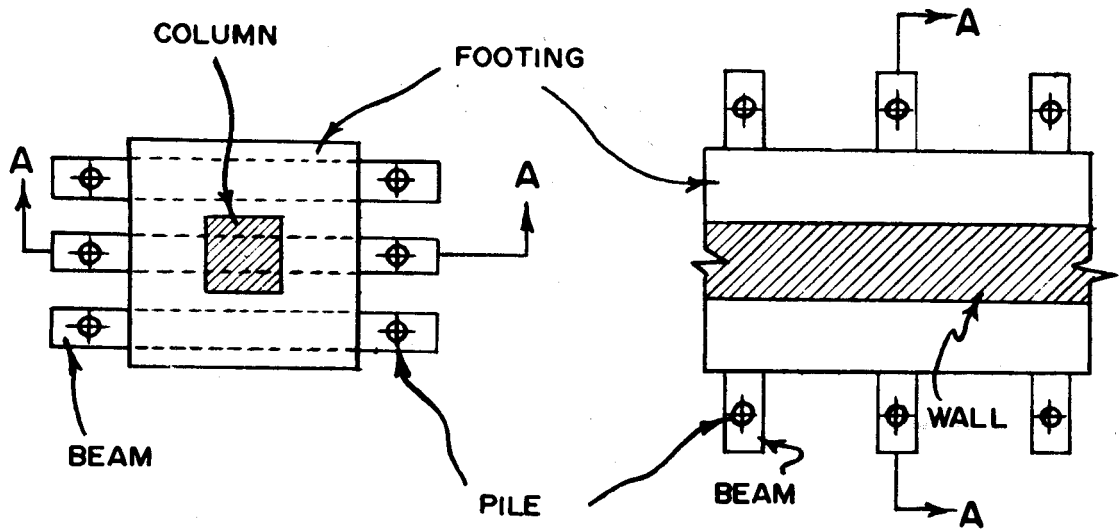


Figure 106. Piles driven alongside footing, support by beam.

If the piles are to be part of an earth support structure, the design of the piles must consider the lateral loads. Underpinning can also be performed by combining both driven and jacked piles as required by access limitations.

7.33.4 Piles on One Side of Footing - Bracket Pile Underpinning

This method is normally used for light structures. It is especially suited for exterior walls or continuous footings when brackets can be installed beneath a wall without fear of shearing off a footing. Bearing is developed either by a driven pile, usually an H-pile, or by a belled or straight-shaft caisson.

Driven Piles

When using driven piles, the typical procedure is as follows:

1. Excavate to expose bottom of existing footing.
2. If required, cut existing footing to permit the piles to be driven as close to the wall as the pile driving equipment will allow. (Special offset driving brackets may be fabricated to permit driving closer than would normally be possible).
3. Drive pile to required resistance.
4. Excavate a sheeted pit beneath the footing and behind the pile.
5. Install a bracket welded to the pile. Normally the flange width of the bracket is greater than the flange width of the pile to permit welding of the bracket from the outside.
6. Transfer load to the bracket with plates and wedges with the top plate drypacked against bottom of footing.
7. Encase bracket and top of pile in concrete if required.

Pre-excavated Vertical Piles and Caissons

Because large drilling equipment is usually used, most of the methods developed are for situations where the work can be

performed outside the building. The load transmitted by these drilled piers can be transferred to a bearing stratum by conventional means. They can be either straight or belled, to increase the end bearing area. Additional load capacity can be developed on the sides by friction.

A vertical hole is augered immediately adjacent to the footing to be underpinned. Then either of the following common methods may be used:

1. A steel beam is dropped into the hole. The hole is filled with lean concrete. A bracket is welded on the steel beam similar to driven bracket pile underpinning (see Figure 107).

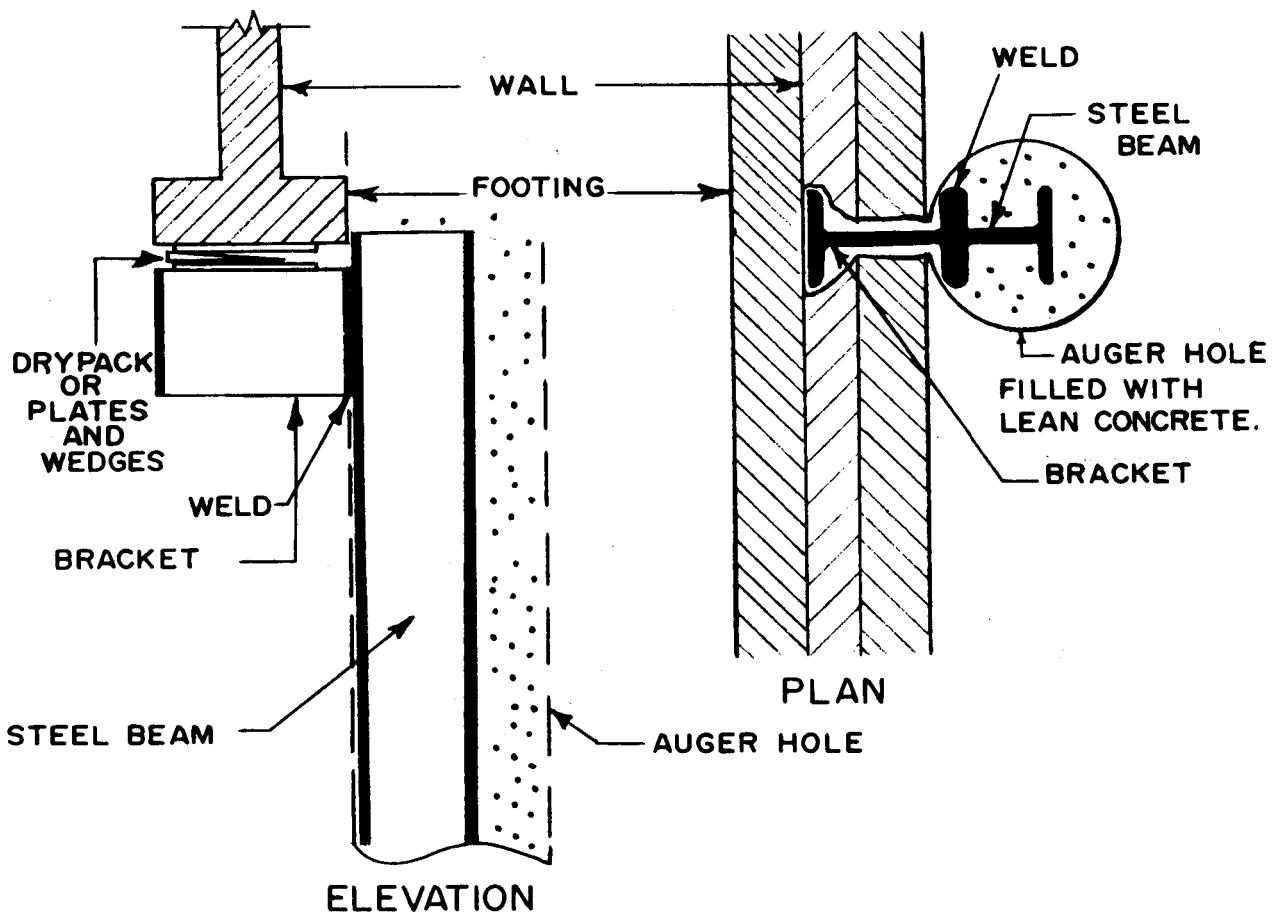
2. After the hole is augered, a hole is excavated under the footing for a bracket. The necessary reinforcing steel is placed, and the pile and bracket poured. The top of the bracket is left 2 to 3 inches below the footing. After the concrete has set, drypack is placed between the bracket and footing (see Figure 108).

3. An alternative to brackets is to auger a vertical hole next to the footing, cut a vertical slot under the footing for the entire depth of the hole, and insert a pile into the slot. After the pile is inserted into the slot it can be loaded with jacks as described previously. Figure 109 illustrates this technique.

Pre-excavated Battered Piles

This method uses "slant drilled" piles or battered piles and is often used when there is a great depth to the bearing stratum. This method is detailed on Figure 110 and consists of drilling a hole at a batter or a "slant" starting adjacent to the existing footing or as close as feasible to the footing, and continuing to the bearing stratum.

The actual underpinning is accomplished by excavating a vertical slot below the foundation down to the slant piles. Reinforcement in the pile and in the slot tie the pile and the slot together.



NOTE: SIMILAR DETAILS IF
STEEL PILE IS DRIVEN
IN PLACE.

Figure 107. Steel pile with steel bracket.

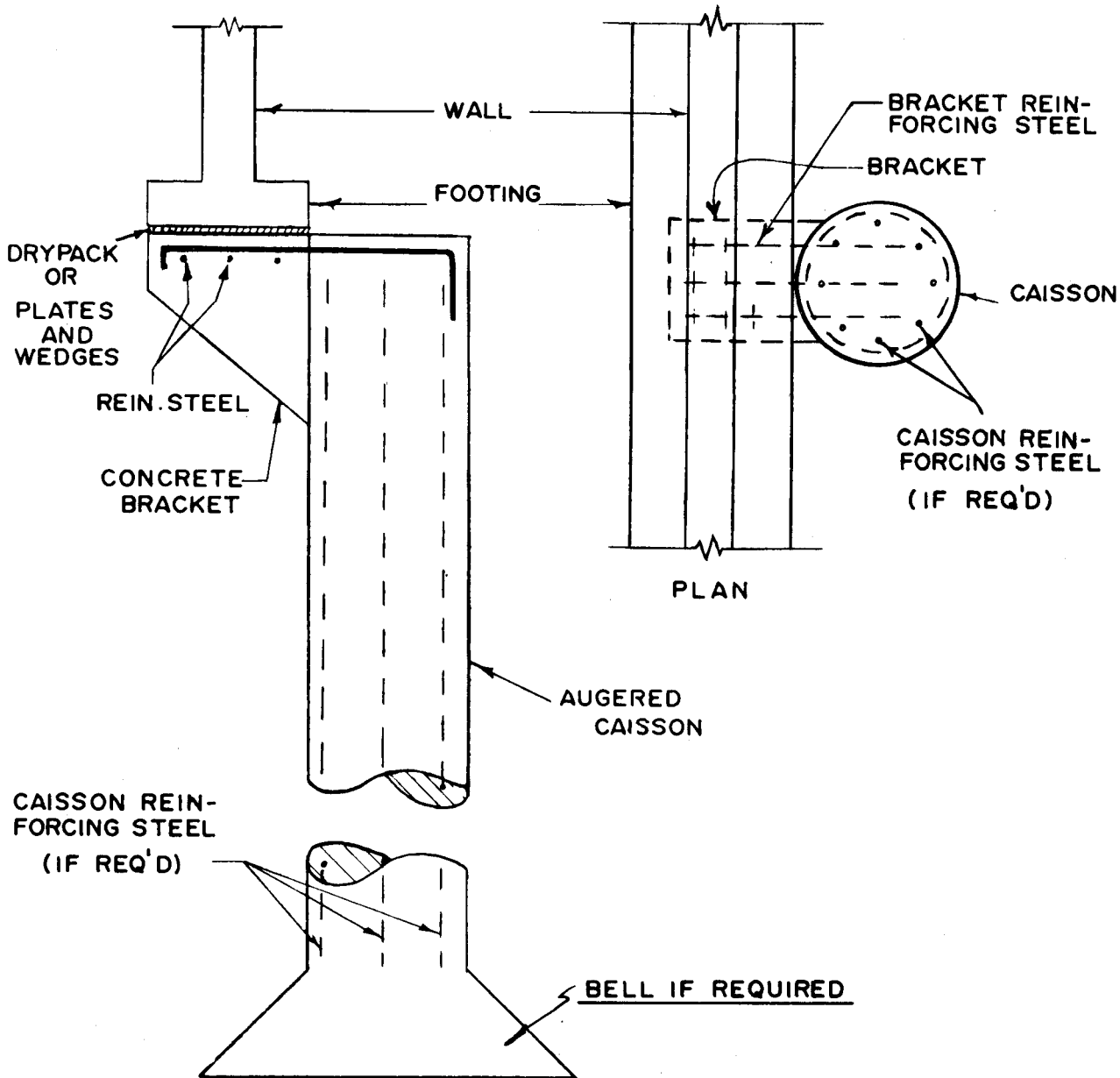


Figure 108. Augered concrete caisson with concrete bucket.

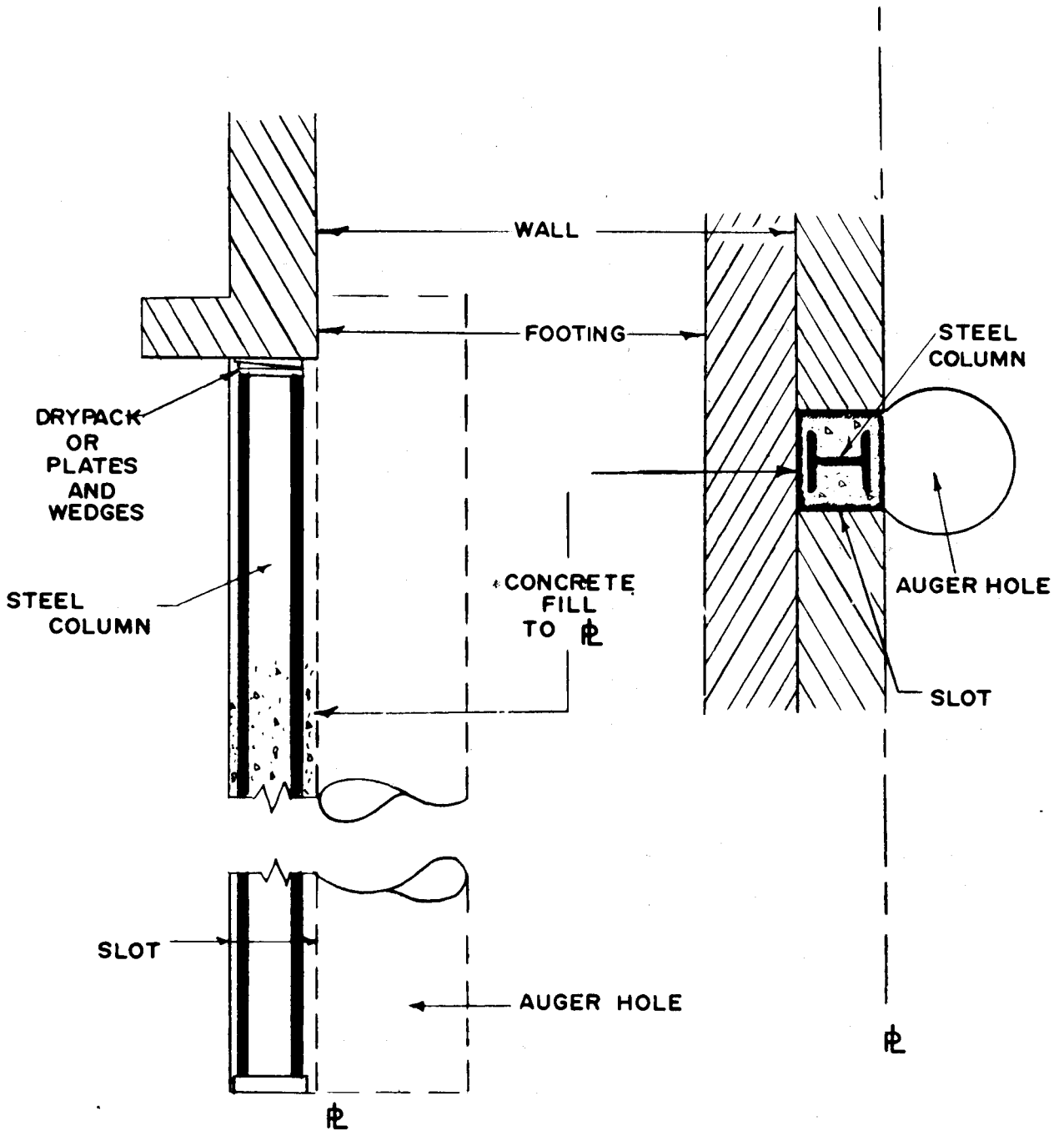


Figure 109. Auger hole with pile installed in slot.

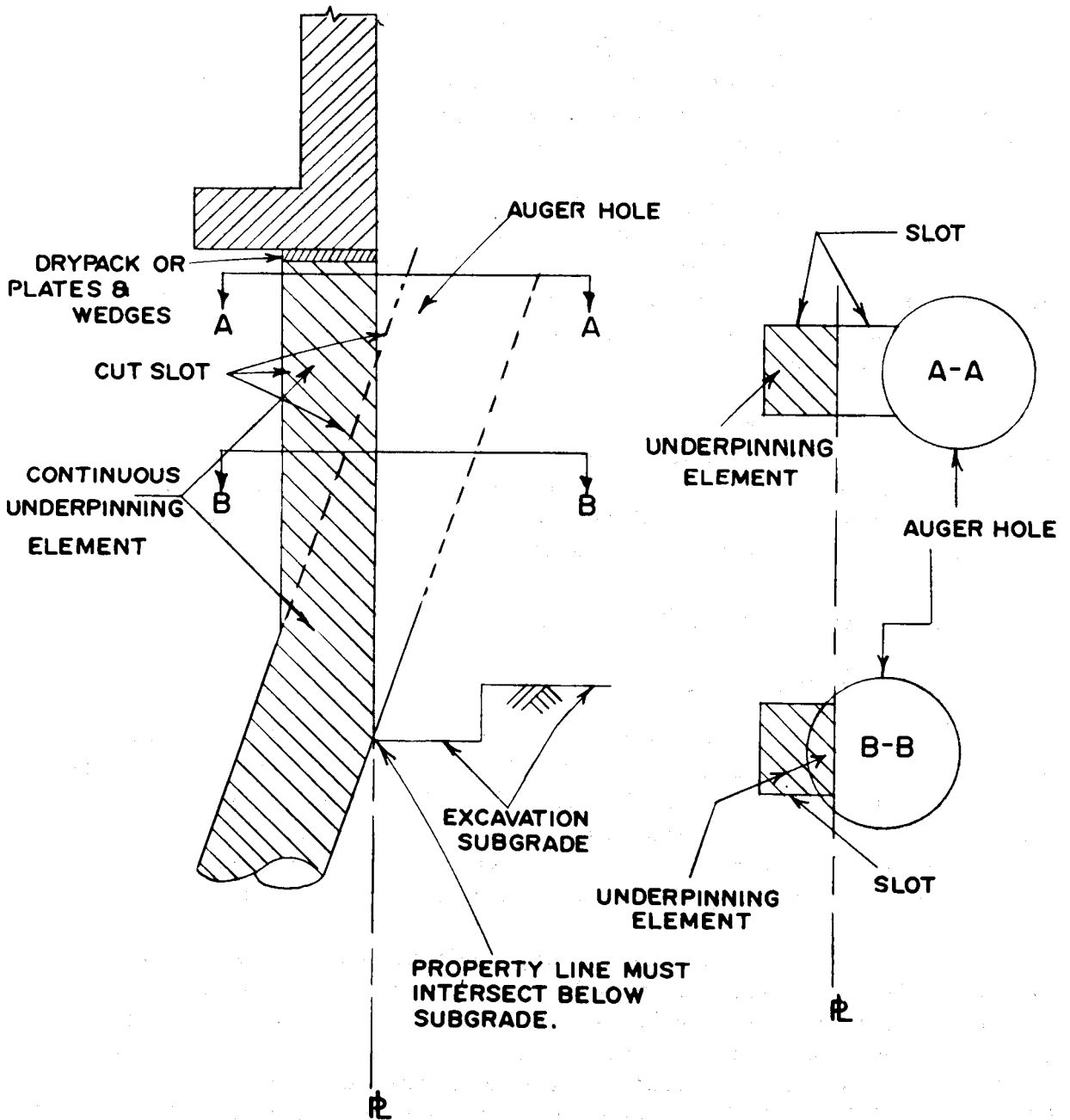


Figure 110. Battered pile underpinning.

7.40 GROUTED PILES

7.41 Hollow Stem Auger

When used as underpinning, the following procedure is generally used to place the piles. A continuous flight, hollow shaft auger is rotated into the ground to the specified pile depth. As the auger is withdrawn, high strength mortar is placed under pressure through its center to form a pile. A reinforcing cage is placed into the wet grout. Sizes typically range from 12 inch diameter to 16 inch diameter.

For different conditions, special mortar can be used. Special low headroom equipment permits installation of these piles inside buildings. These piles can be installed adjacent to or through existing footings, and loads can be transferred from the structure to the piles by beams or brackets or by making the piles integral with the footing through bond.

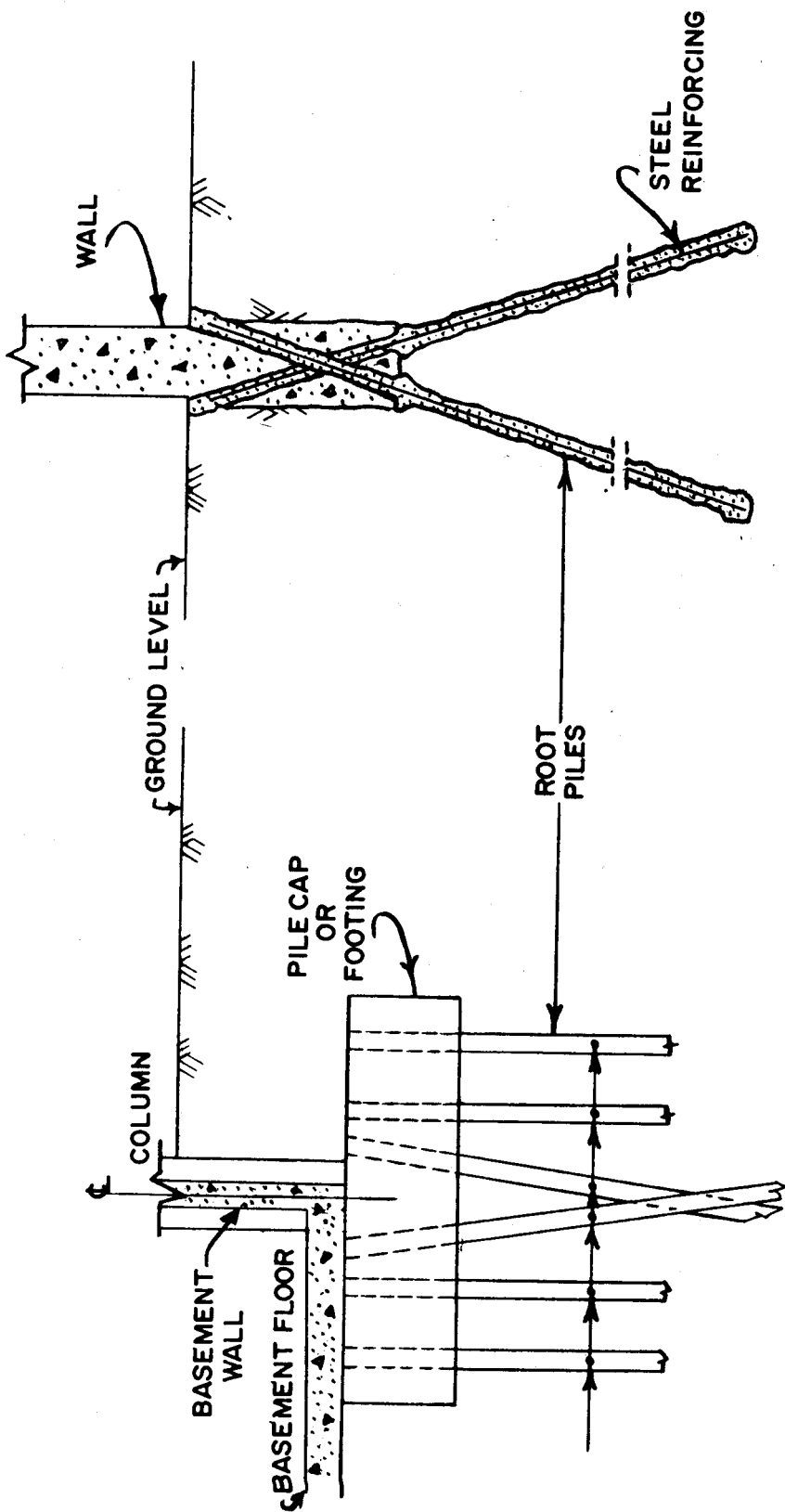
This method permits piles to be installed close to each other with minimum vibration and soil heave. If the auger is withdrawn too quickly, soil may fall into the hole before grout is injected and create a noncontinuous pile. Such a defect would not become evident until loads are imposed on the underpinning.

7.42 Root Piles (Pali Radice)

7.42.1 General

A relatively new development in the area of small to medium diameter friction and end bearing piles is a method developed by the Fondedile Corporation, known as the "Pali Radice" or root pile. This system is capable of providing vertical and/or lateral support to foundations and excavations (Bares, 1974) and can be used for underpinning and strengthening of existing foundations (see Figure 111).

The piles, ranging from 3-1/2 inches to 12 inches in diameter, are reinforced. Installation is done by rotary or percussion drilling of cased holes that are filled with concrete under pressure during withdrawal of the casing. A wide range of usage with good success has been recorded in Italy and other European countries (F. Lizzi, 1970 and 1974). Recently the method has been introduced in the United States (ENR, April 1972 and Bares, 1975).



a) DIRECT SUPPORT OF FOUNDATION
 (EITHER FRICTION PILES OR END
 BEARING PILES).

b) UNDERPINNING OF A CONTINUOUS
 WALL.

Figure 111. Typical uses of root piles (pali radice).

7.42.2 Root Pile Underpinning

Installation

When used for underpinning it is normally installed through existing foundations. The drilling muck or cuttings are brought up to the surface by direct circulation of the drilling fluid (bentonite slurry or water). The application in granular soils usually requires a casing throughout its entire length to prevent collapse of the hole. The drilling is done using a sharpened casing.

Concreting of the pile is accomplished by filling from the bottom with mortar placed through a pipe. Compaction of the mortar is achieved by blasts of compressed air (about 70 to 100 psi) done in stages as the casing is withdrawn. This improves the contact of mortar and soil and facilitates the withdrawal of casing.

Reinforcing consists of either a cage or a single bar. The smaller root piles (generally 4 to 5 inches nominal diameter) are reinforced by a deformed high strength bar while the larger piles (generally 6 to 12 inches nominal diameter) are usually reinforced with a spiral cage. The steel is placed in the smaller piles after concreting, but before concreting in the larger piles.

Design Considerations

The design of root piles should follow procedures for friction piles and end bearing piles modified by experience. The load carrying capacity is in the range of 10 to 15 tons for the smallest diameter piles and 40 tons or more for the larger diameter piles. Load is transferred to the soil through friction, end bearing, or a combination of the two, depending upon soil conditions.

Table 13 summarizes the results of load tests on root piles obtained from published and unpublished sources. In general, the tests were not carried to failure, and therefore, the data do not permit an evaluation of safety factors. However, since the settlement data were available, it was possible to develop, at least in crude fashion, a relationship between pile geometry, load, and settlement.

A pile settlement modulus was developed on the assumptions that the load is transferred to the soil primarily by skin friction and that settlement is inversely proportional to the average skin friction value. (See Figure 112). Thus:

Table 13. Results of load tests in Pali Radice. (Continued).

Case No.	Nominal Diameter D, inches	Length L, feet	Assumed Effective Length L', feet	Max. Test Load P, tons	Settlement at		Soil Type ⁽²⁾	Location
					Max. Load s, inches	Modulus (1) $\frac{\text{in-ft}^2}{\text{k} \cdot \text{ton}}$		
14**	8	66	66	58.7	0.037	0.0247	G	Railway Terminal, Naples (Corso A. Lucci)
15**	8	63	63	56.5	0.065	0.0483	G	Plant (Brindisi)
16**	8	60.5	60.5	56.5	0.028	0.0200	G	Plant (Brindisi)
17**	8	73.5	73.5	27.5	0.252	0.4490	C	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
18**	8	66	66	24.2	0.386	0.7018	C	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
19**	8	66	66	48.5	0.205	0.1860	C	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
20**	8	99	66	110.2	0.213	0.0850	G	Belt (Expressway) East-West, Naples
21**	8	99	66	88.2	0.127	0.0634	G	Belt (Expressway) East-West, Naples
22**	8	59.5	59.5	68.3	0.061	0.0354	G	Swimming Pool - Scandone Pool, Naples
23**	4	33	33	21.5	0.087	0.0445	G	Casa Albergo in Viace Piave
24**	8.5	82.5	82.5	69.7	0.148	0.1241	G	Port of Naples
25**	8.5	82.5	82.5	69.7	0.150	0.1258	G	Port of Naples

(1) $k = \frac{P}{L/D} = \frac{L/D}{P}$

(2) G = Granular; C = Clay; Si = SiH

** Bares, F.A. (1974)

Table 13. Results of load tests in Pali Radice.

Case No.	Nominal Diameter D, inches	Length L, feet	Assumed Effective Length L', feet	Max. Test Load P, tons	Settlement at		Soil Type (2)	Location
					Max. Load ρ , inches	Modulus k , in-ft/ton		
1*	4	21	21	22	0.04	0.013	G	School Building, Milan, Italy
2*	4	40	40	22	0.16	0.097	C	Olympic Swimming Pool, Rome
3*	12	90	90	50.6	0.32	0.570	G	Bausan Pier, Naples
4**	4	49	20	19.8	0.08	0.0270	Si, G	Italian State Railroad, Rome
5*	4	52	42	17.6	0.09	0.072	G	Bank of Naples
6*	8.5	99	66	108	0.22	0.087	G	Corps of Engineers, Naples
7**	5	65	24	50	0.32	0.062	G	Washington, D. C., Subway
8*	9	19.5	10	45	0.45	0.075	G	Queen Anne's Gate, London
9*	7	28	18	50	0.30	0.063	G	Queen Anne's Gate, London
10**	4	52.8	52.8	23.1	0.236	0.1798	C-G	Salerno-Mercatello Hospital, Salerno-Mercatello
11**	8	82.5	43	108	0.472	0.125	G	Marinella Wharf, Port of Naples, Naples
12**	8	47.5	47.5	59.4	0.035	0.0187	G	Main Switching Plant, Genoa
13**	8	73	73	62.5	0.065	0.0506	G	Mobil Oil Italiana, Naples

(1) $k = \frac{\rho}{P/L'D} = \frac{L'D}{P}$ (2) G = Granular; C = Clay; Si = Silt

* Bares, F. A., Personal Communication, September 1975.

** Bares, F. A. (1974).

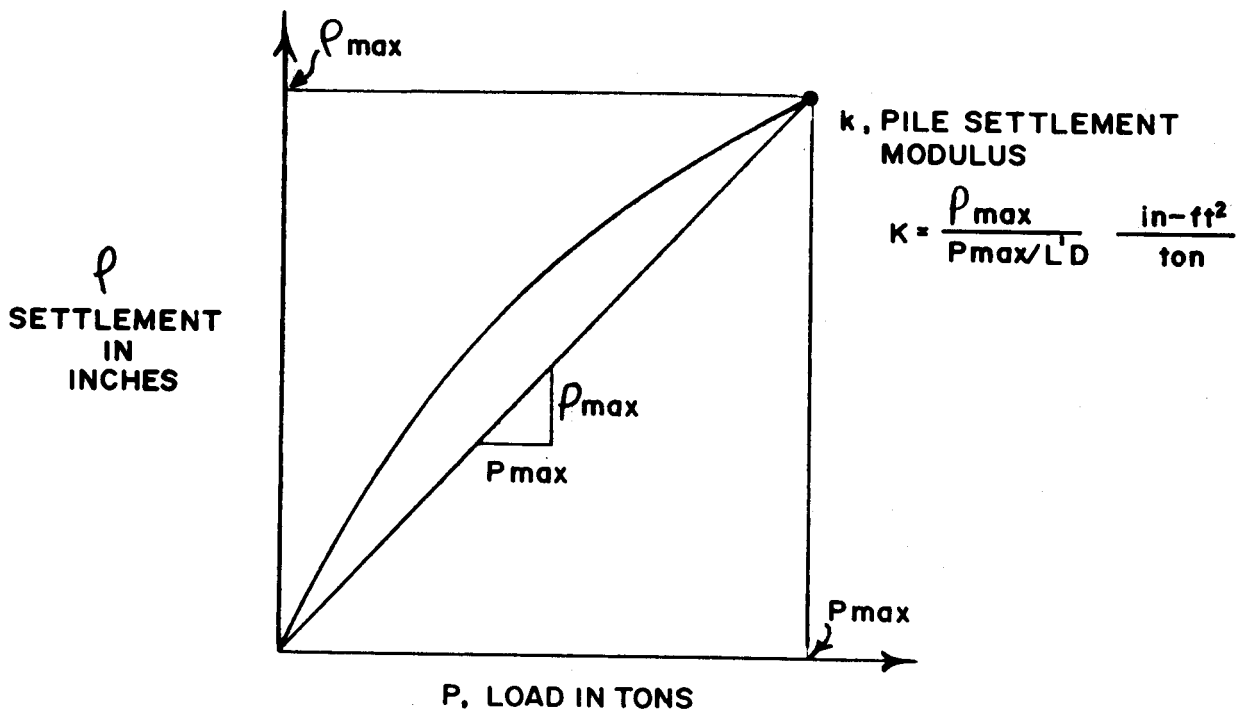
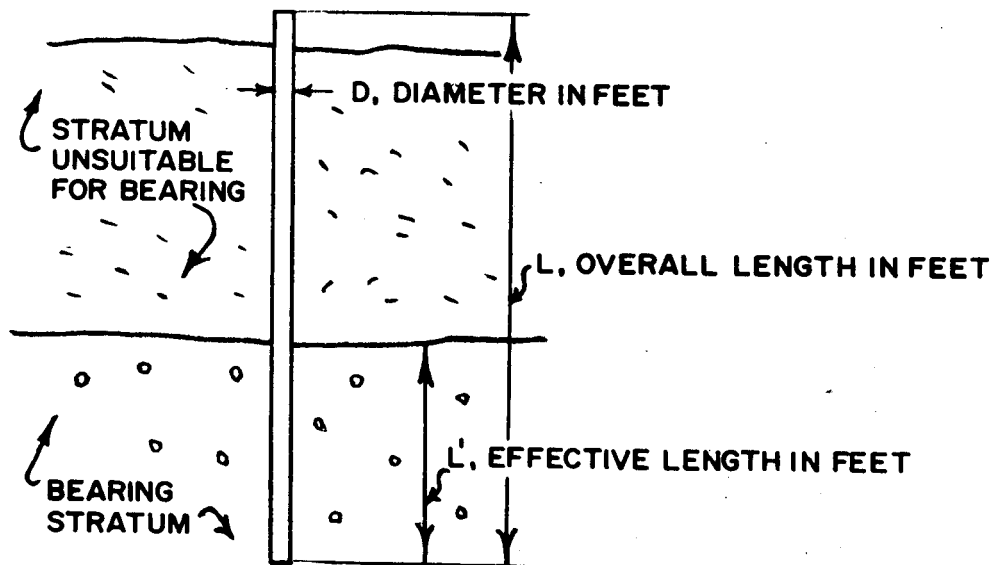


Figure 112. Development of pile settlement modulus.

$$\rho \sim P, \frac{1}{\pi DL'}$$

$$\text{or } \rho = k \frac{P}{DL'}$$

where:

ρ = settlement in inches

P = load in tons

D = pile diameter in feet

L' = length of pile in the load transfer zone

k = settlement modulus

Therefore, to establish a settlement modulus for the pile:

$$k = \frac{(\rho_{\max}) DL'}{P_{\max}}$$

where:

ρ_{\max} is the observed settlement under maximum load, P_{\max} .

Column number seven in the table presents the pile settlement modulus. The data for piles in granular soils indicates that the settlement modulus is generally less than 0.1 in-ft²/ton. In those cases where the settlement modulus is greater than this value it is usually only slightly larger. In clayey soils the pile settlement modulus can be significantly larger (0.7 in-ft²/ton in one case). This trend in the data is not unexpected and implies that the conditions in clayey soil should be carefully investigated. Load tests at all sites are recommended to determine the actual settlement characteristics. It should be noted that these data are not applicable to cases where end bearing represents a large portion of the total load transferred.

► As an example of a settlement computation, assume a value of 0.10 in/tsf, and compute settlement of a 20 ton pile with an effective length in the bearing stratum of 20 feet. Assume a 4 inch diameter pile is used.

$$\rho = k \frac{P}{DL'} = 0.10 \frac{20}{4/12 \times 20} = 0.3''$$

Cases described in the literature indicate that load is normally transferred gradually as the former foundation support

is removed in the process of excavation. This process differs from other methods in which the load is transferred to the underpinning by jacks or wedges at the time of installation.

Sources of settlement are strain at the contact between existing foundation and the piles and pile movement as it accepts load. The amount of settlement associated with this load transfer must be evaluated.

7.42.3 Reticulated Root Piles

The term "reticulated" is used by Fondedile to describe an application where the piles resist lateral displacement of the soil, as differentiated from the underpinning application where the piles support vertical load. In certain cases underpinning piles serve the dual purpose of carrying load and resisting soil displacement (Bares, 1974; Bares, 1975; and Lizzi, 1970).

The reticulated pile principle is to engage an earth mass by installing a root pile network at close spacing and in a particular pattern of pile batter and orientation. A lattice structure is thus constructed to encompass the soil, which consequently behaves monolithically. Design procedures involve analyses similar to those used for gravity walls, namely, evaluating the overturning moment, determining the position of the vertical reaction on the base, and checking for horizontal shear through and below the monolith. Figure 113 demonstrates the principle. Figure 114 shows reticulated root pile underpinning adjacent to a cut-and-cover tunnel. Applications demonstrated in Figure 115a, 115b, and 115c are in connection with bored tunnels. They provide underpinning as well as a network to resist soil displacement. The application shown in Figure 115c suggests that the network contributes to the development of arching over the tunnel.

7.50 TUNNELING BELOW STRUCTURES

7.51 General

This discussion concerns instances when tunnels pass beneath structures. As a result, it is not possible to use vertical underpinning elements directly below the foundations. Some applications using reticulated walls were illustrated in Section 7.42. Other examples follow:

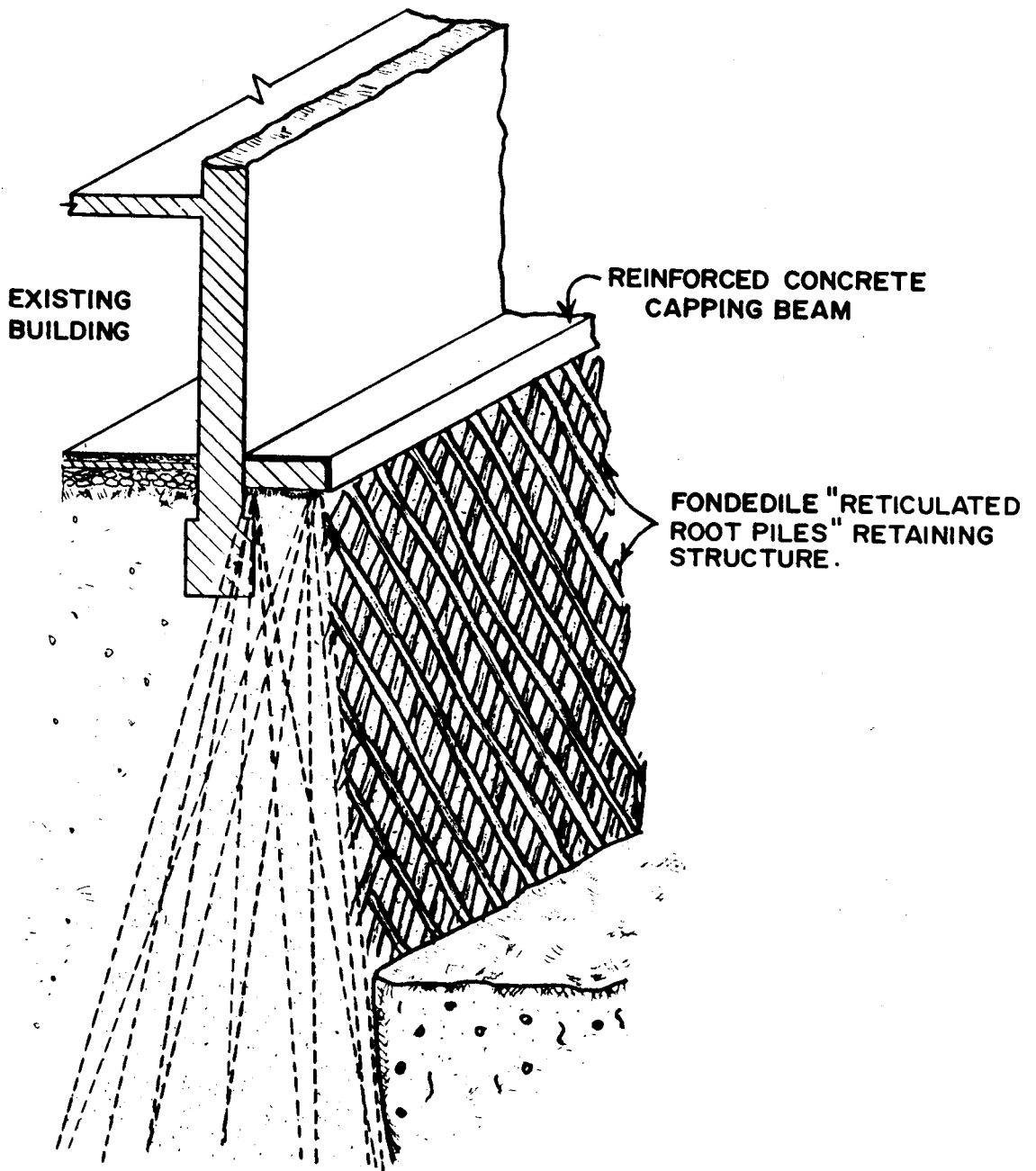
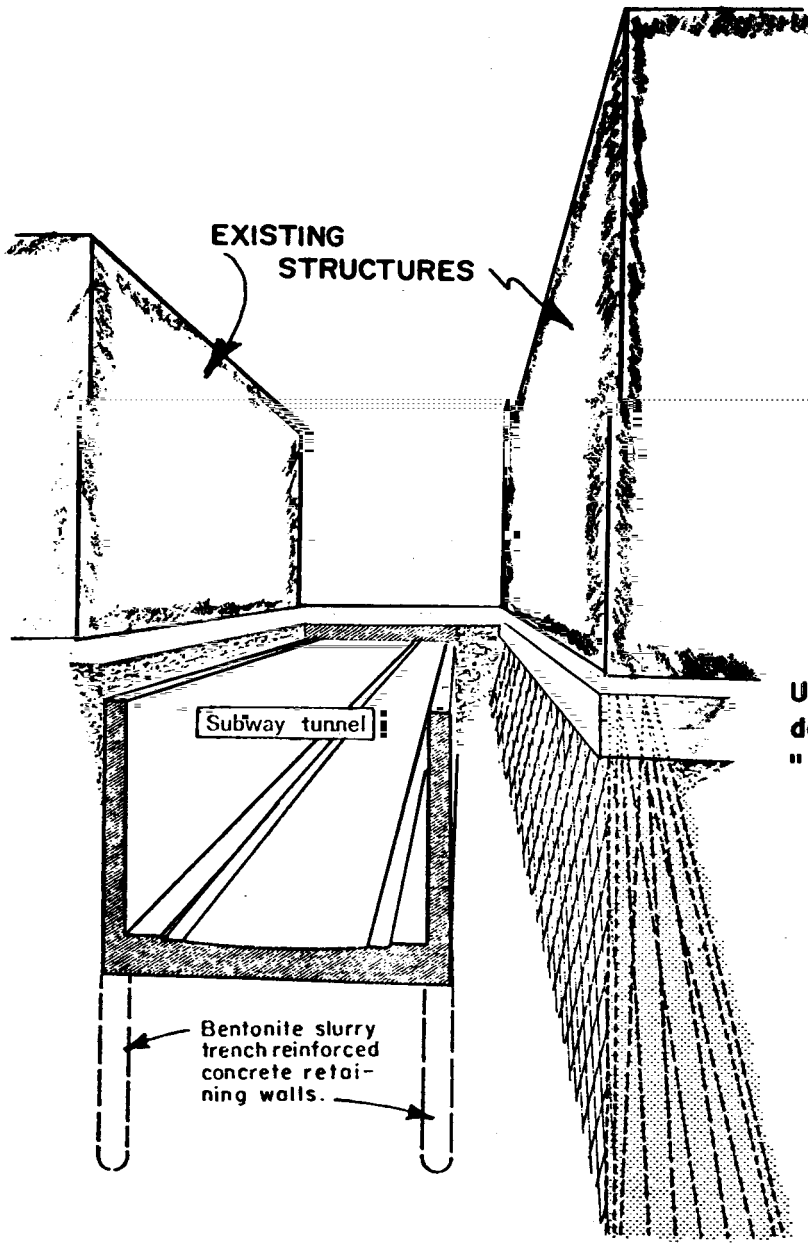


Figure 113. Schematic showing principle of reticulated root piles.
(Courtesy of Warren-Fondedile, Inc.).



**MILAN SUBWAY
CONSTRUCTION**

**UNDERPINNING OF OLD "LOGGIA
dei MERCANTI" by FONDEDILE
"RETICULATED ROOT PILES."**

Figure 114. Reticulated root pile underpinning.
(Courtesy of Warren-Fondedile, Inc.).

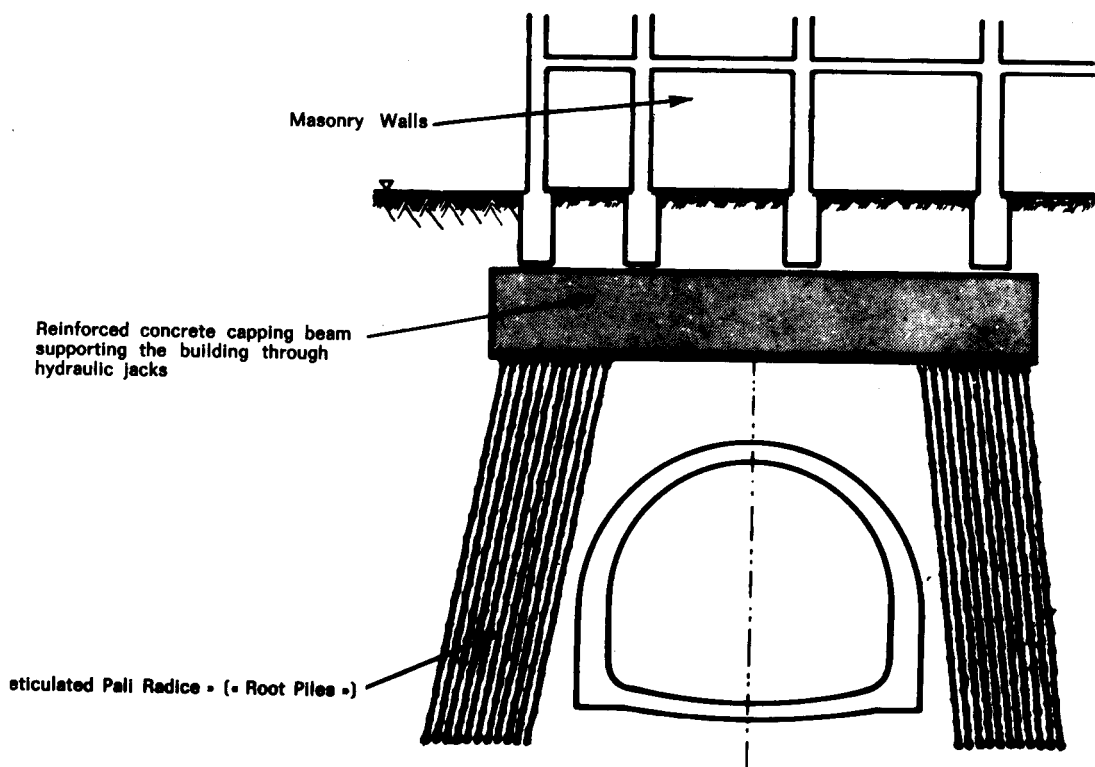


Figure 115a. Reticulated root pile applications.
 (Courtesy of Warren-Fondedile, Inc.).

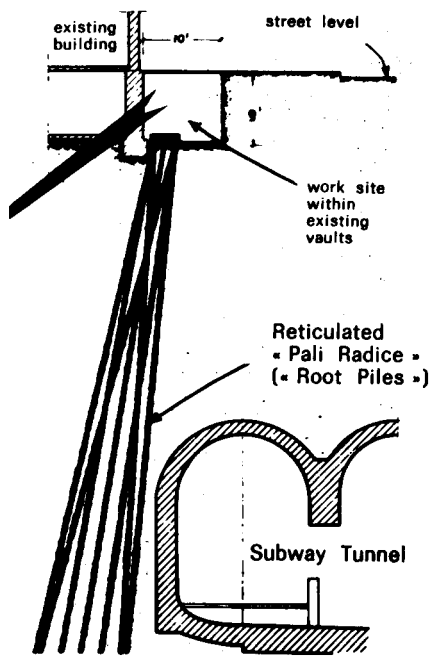
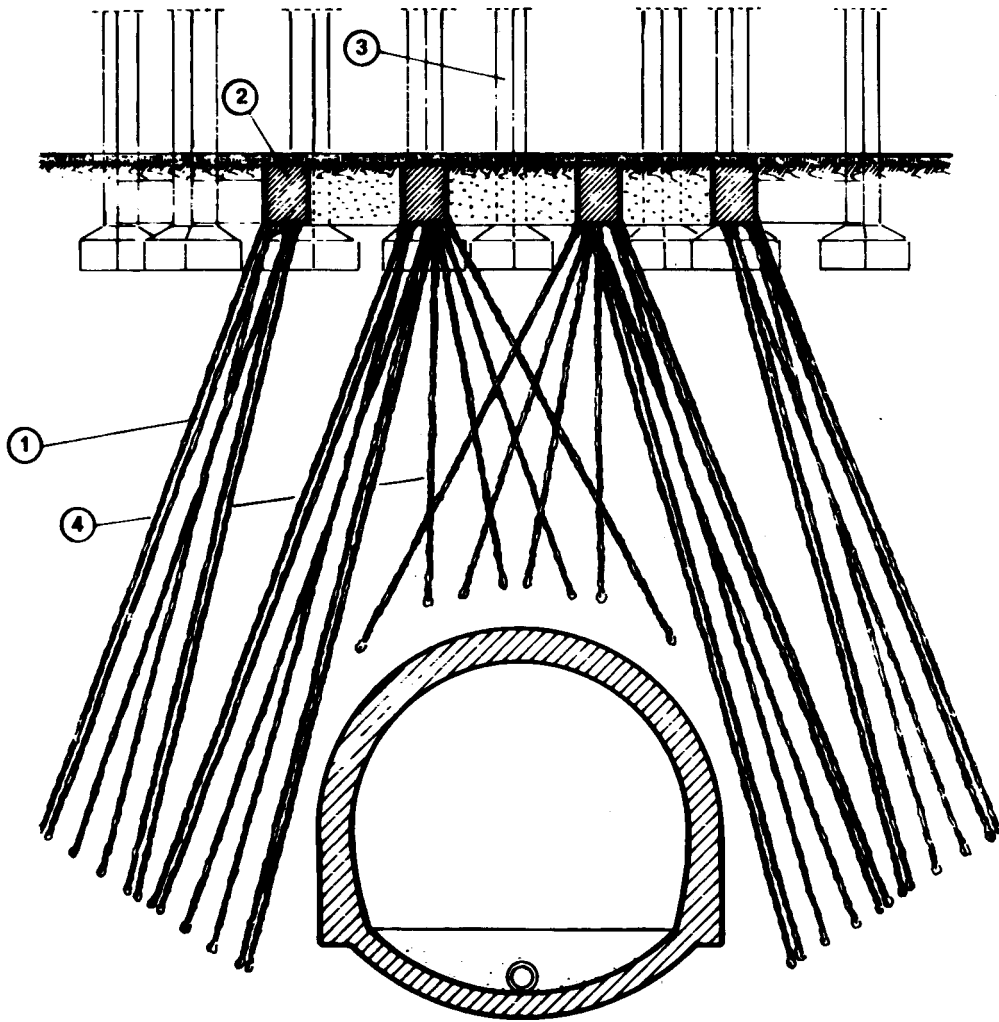


Figure 115b. Reticulated root pile applications.
(Courtesy of Warren-Fondedile, Inc.).



LEGEND:

- 1) - Reticulated Pali Radice - (= Root Piles -)
- 2) Network of reinforced concrete beams capping the - Reticulated Pali Radice - (= Root Piles -) and encasing footings of the building.
- 3) Existing Footings
- 4) - Reticulated Pali Radice - (= Root Piles -) for further soil strengthening.

Figure 115c. Reticulated root pile applications.
 (Courtesy of Warren-Fondedile, Inc.).

7.52 Column Jacking

A common method of protecting structures, when tunneling directly beneath the structure, is to maintain the structure elevation by freeing the column from the footing and jacking the column. The first step consists of installing brackets on the column, removing anchor bolts, and installing the jacks between the bracket and the footing. As the tunnel approaches, the jacks are activated, and the load is maintained on the footings. The jacks will allow the footing to settle while maintaining the column elevation. After completion of the tunnel the base plate is reshimmed, the anchor bolts are tightened, and the jacks are removed. Figure 116 illustrates the procedure used.

7.53 Pipe Shield Technique

The procedure is to install a series of contiguous horizontal pipe tunnels, on the order of 3 to 4 feet in diameter, which are later reinforced and concreted to provide a protective roof (or shield) above the vehicular or subway tunnel. Typically, the contiguous tunnels, called pipe shields, are installed by jacking pipe from an open cut or from the side of a primary drift tunnel if this is not possible.

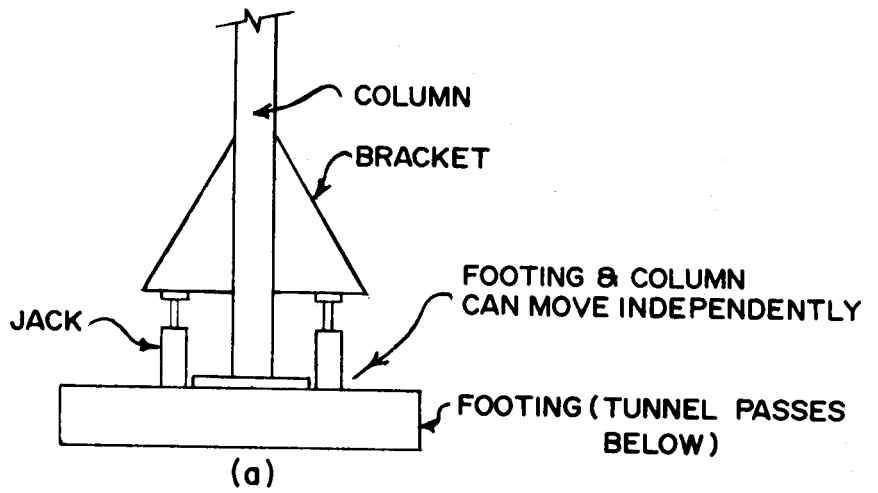
Figures 117a and 117b (Zimmerman, 1969 and Rappert, 1970) illustrate examples where jacking pits were excavated from the surface. In another case, reported by Maidl and Nellesen (1973), a subway passed beneath a heavy bank building. It was impossible to excavate a jacking pit from the surface. Therefore, a primary drift tunnel was advanced and then the pipe shields were jacked out transversely from the primary drift tunnel. The excavation was carried out below the pipe shield roof by a combination of secondary drift tunnels and general excavation.

7.54 Inclined Secant Piles

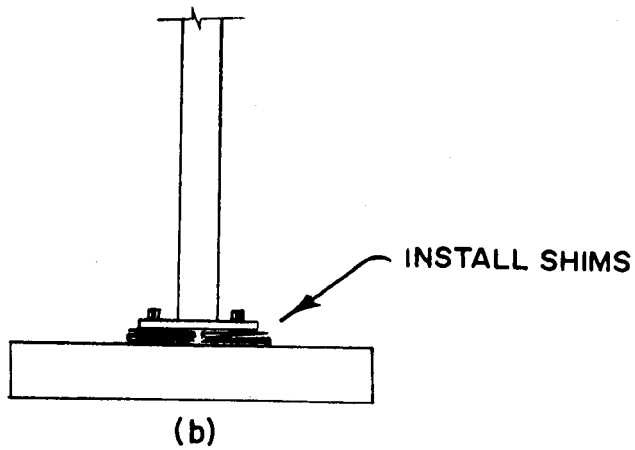
Refer to Section 4.70 (Diaphragm Walls) of this volume for a discussion of secant piles. Inclined secant piles in lieu of underpinning are applicable where there is a slight encroachment below utilities or structures (see Figure 118). This method was successfully used to protect the St. Stephen's Cathedral during construction of a subway tunnel in Vienna (Figure 119) (Braun, 1974).

7.55 Bridging

Figure 120 schematically illustrates measures that can be taken to bridge across the tunnel area.



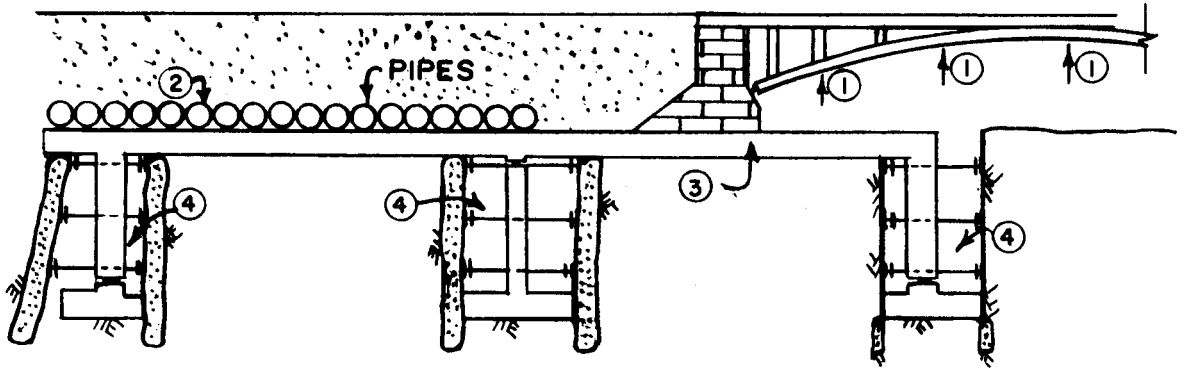
BRACKETS INSTALLED, ANCHOR BOLTS
LOOSENED, JACKS INSTALLED.



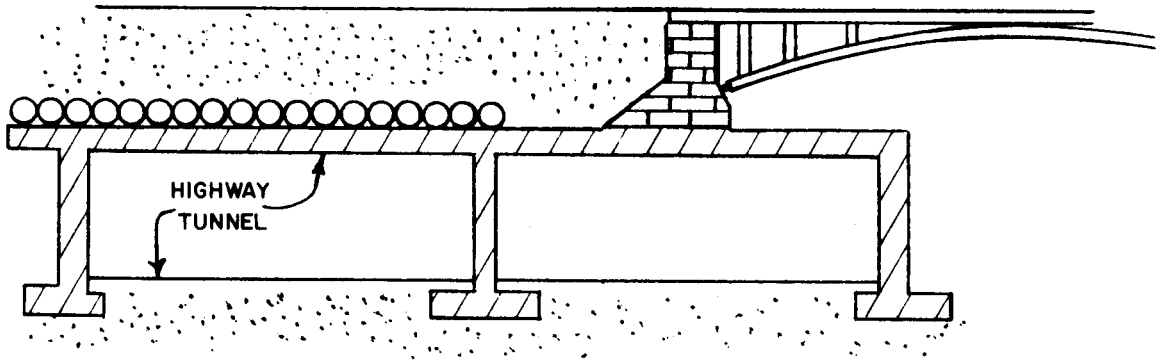
SHIMS INSTALLED, ANCHOR BOLTS
TIGHTENED.

Figure 116. Schematic of column jacking to prevent structure settlement during tunnel construction.

(a) CONSTRUCTION



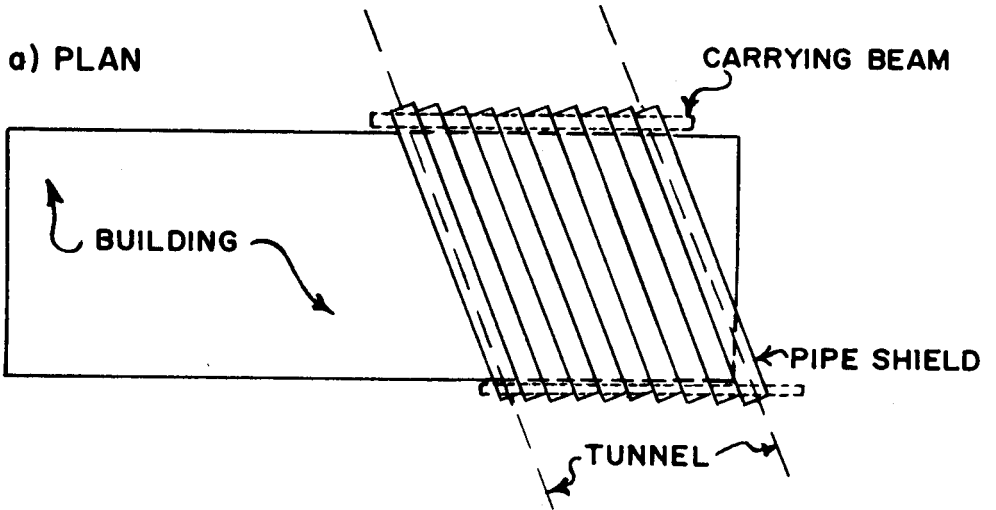
(b) AFTER COMPLETION



Sequence

1. Underpin bridge with steel piles and jacks to adjust for settlement.
2. Construct jacking pits on each side of highway, jack 1.2 m pipes and concrete pipes.
3. Construct 3 m wide x 2 m high tunnels below pipes. Concrete each tunnel before building next one.
4. Construct walls of highway tunnel.

Figure 117a. Pipe shield technique (after Zimmermann, 1969).



b) SECTION

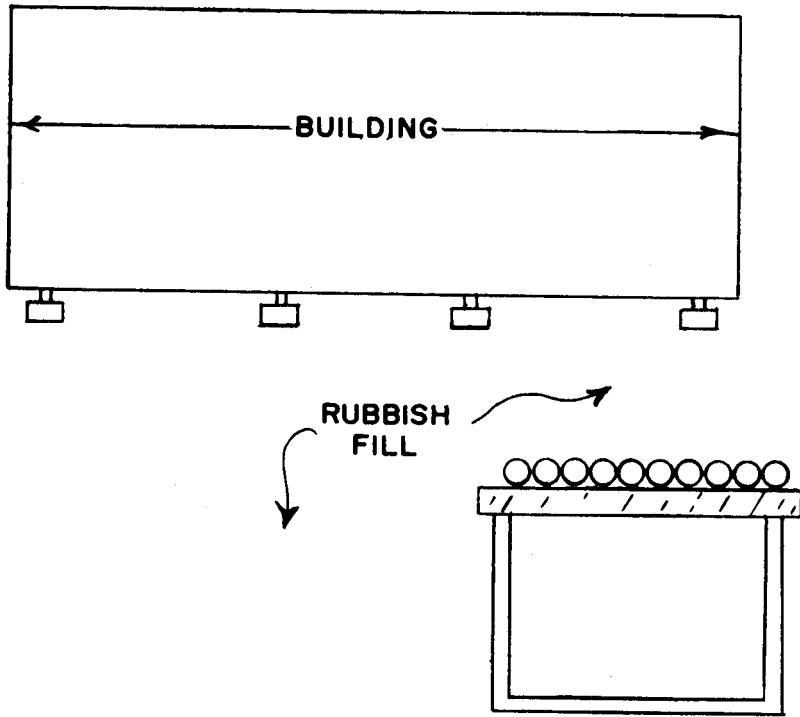


Figure 117b. Pipe shield technique (after Rappert, 1970).

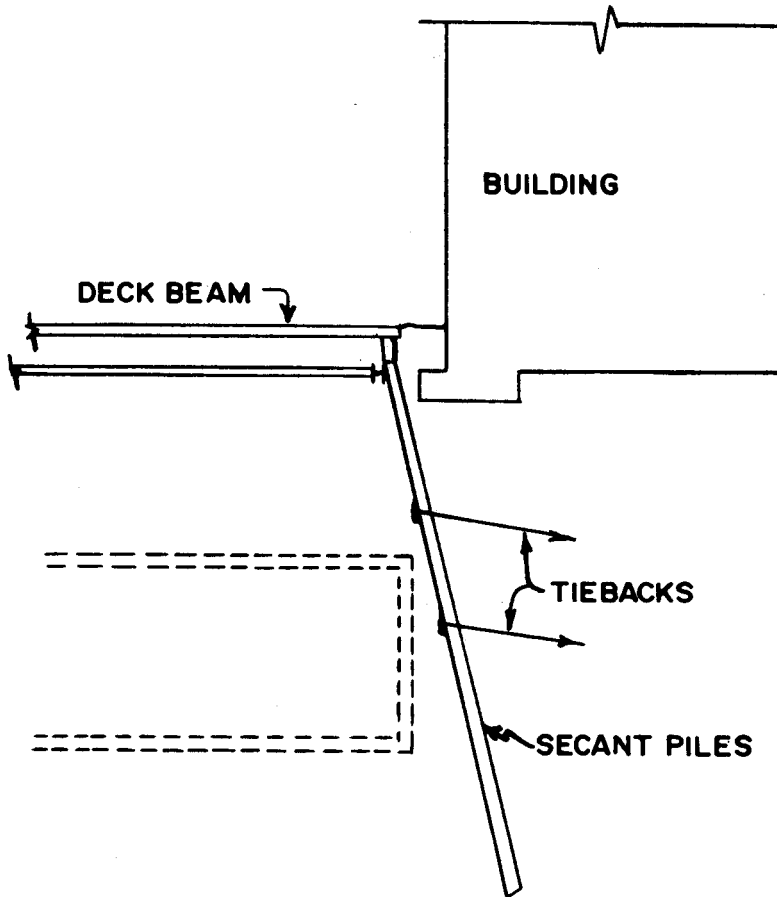


Figure 118. Inclined secant piles for protection of building (after Joas, et al, 1971; Weinhold and Kleinlein, 1969).

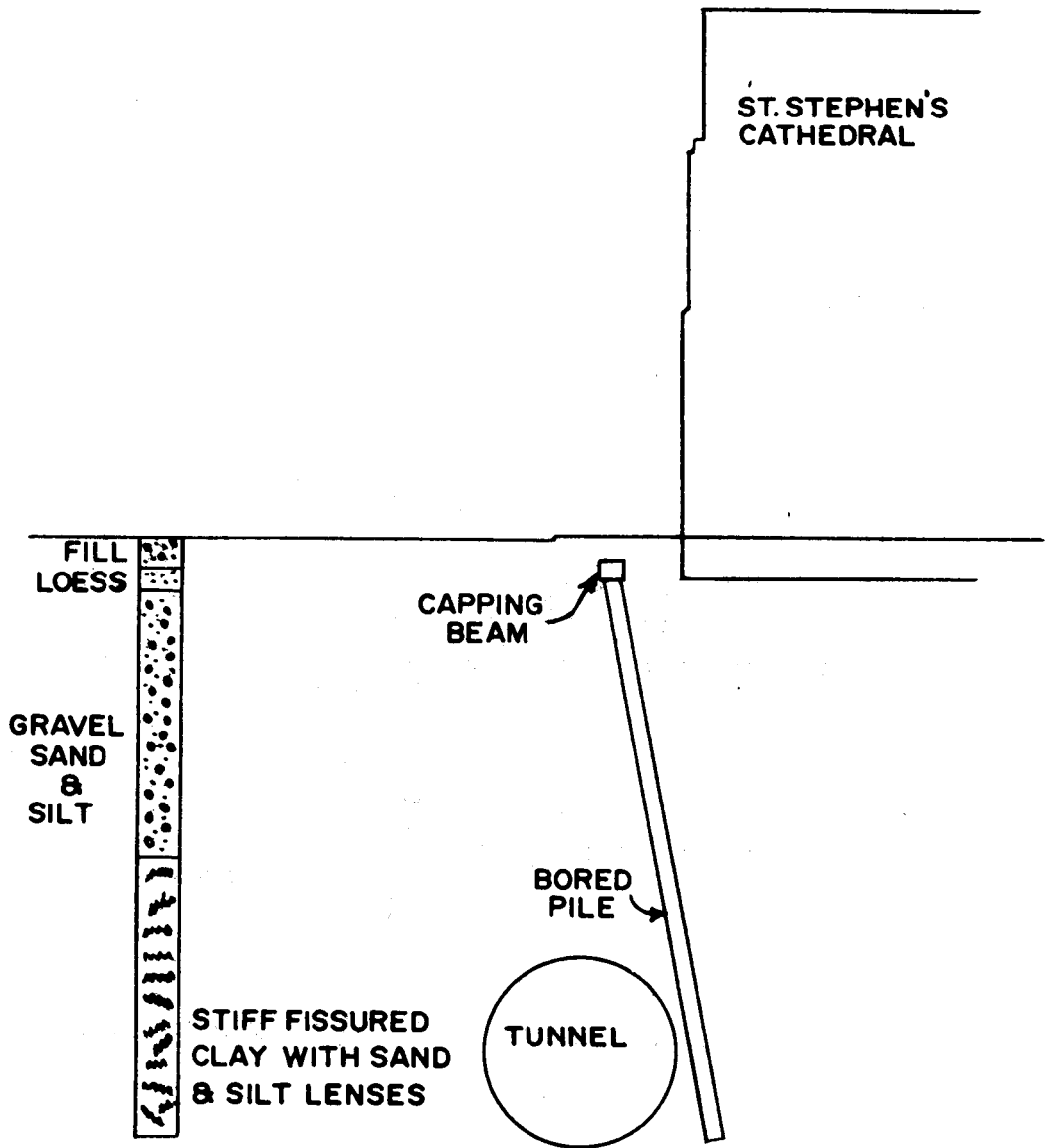


Figure 119. Example of bored pile wall used to protect structure (after Braun, 1974).

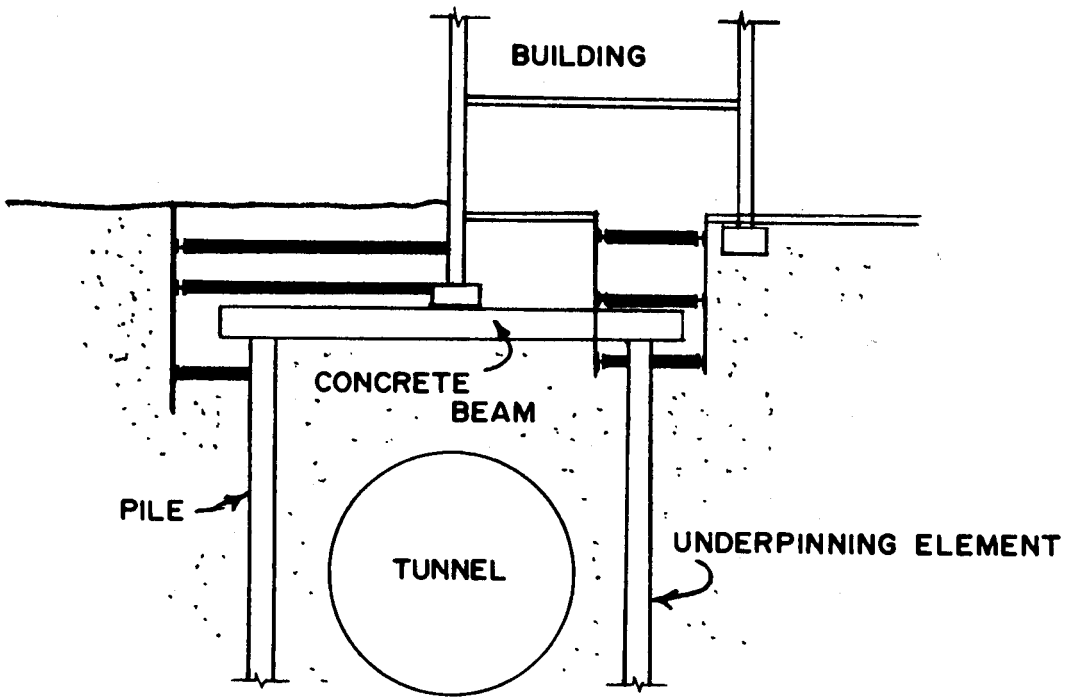


Figure 120. Bridging.

There are literally an infinite number of combination techniques that can be used. For example, the sketch shows a circular tunnel straddled by individual underpinning elements (Steps 1 and 3) and roofed by individual bridging beams. As an alternative, the excavation could be made in a box section and the tunnel formed within the box. The procedure would be to use a continuous roof (perhaps the individual beams in combination with grouting, pipe shields, or conventional mining techniques). In addition, rather than individual underpinning, a continuous wall could be constructed to retain the earth (concrete diaphragm walls or continuous pit underpinning). Lateral support would be provided by bracing or tiebacks.

7.56 Underpinning Elements As Part of Permanent Structure

Goldfinger (1960) describes the construction of a subway tunnel in New York that crosses immediately below an existing four track tunnel. Since there were only a few inches of clearance between the top of the new steel beams and the bottom of the old subway, placing temporary needle beams would be a problem. The problem was solved by increasing the size and length of the design roof beams to be able to transmit the subway tunnel and train weight to the exterior underpinning walls. The roof beams were then used as underpinning support for the existing subway during construction. Figures 121 and 122 illustrate the relative locations of the subway tunnels, and the construction procedure used on the project.

The new tunnel construction was accomplished by first underpinning the exterior track walls with jacked piles. Tunnel column loads were transferred to the piles through steel beams on top of the piles. The next step was to construct a 4 foot thick concrete retaining wall on either side of the new tunnel. The concrete walls were constructed using the pit method from access tunnels dug below the existing tunnel. The key to the procedure was to leave as little of the above track unsupported at any time as was possible. As each pit (5 feet x 4 feet) was completed, a steel post was installed to support the unreinforced slab. Jacked piles were installed under interior columns.

The final step was to install the roof beams to carry the subway load during general excavation. This was achieved by mining in approximately 6 foot sections and installing the beams on the 4 foot concrete walls. The subway load was transferred to each beam prior to excavating for the next beam. In all drift tunnels and excavations temporary shoring was installed to support the subway until the final support members were in place.

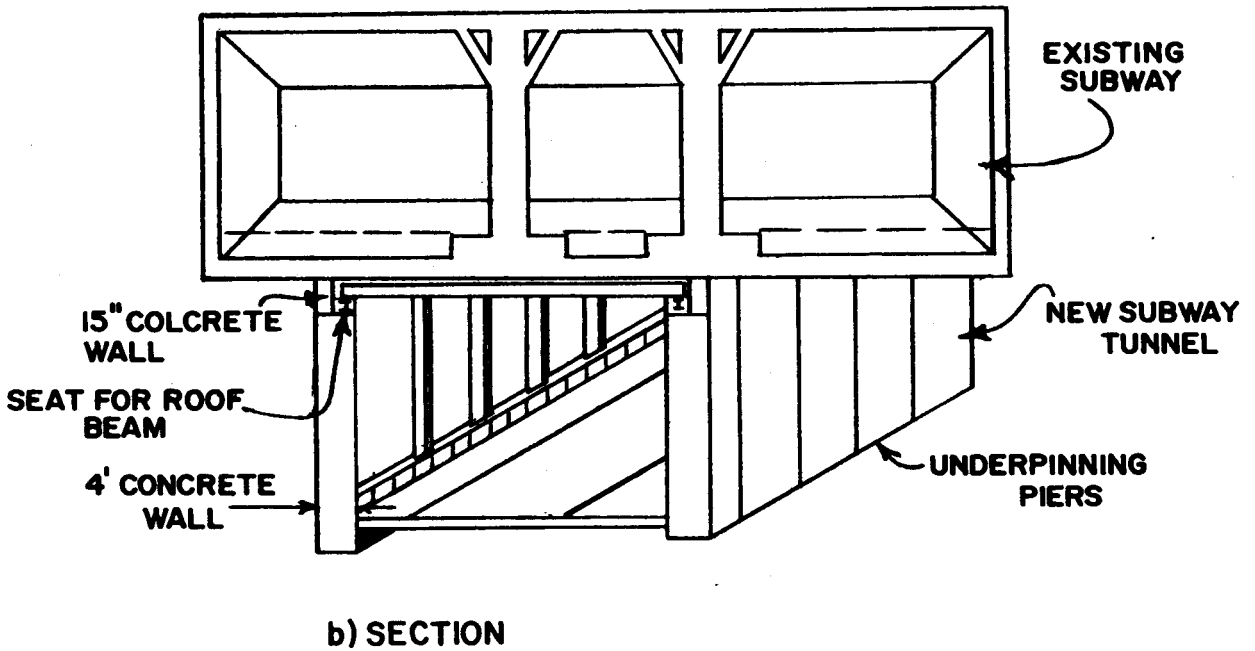
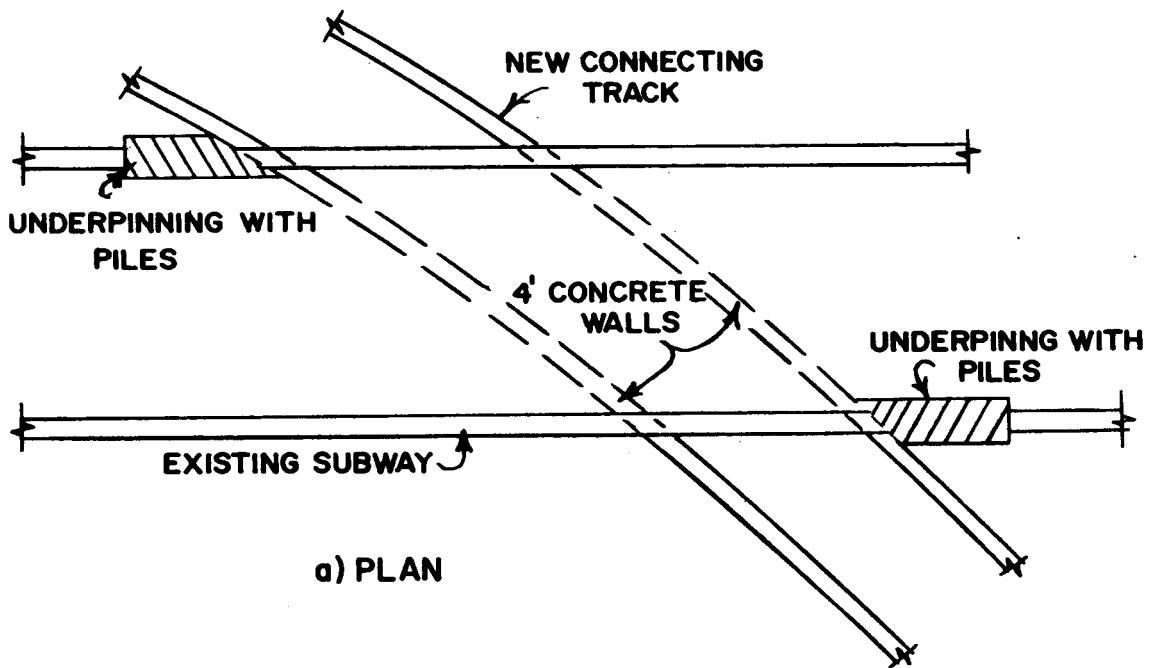


Figure 121. Location of new subway tunnel (after Goldfinger, 1960).

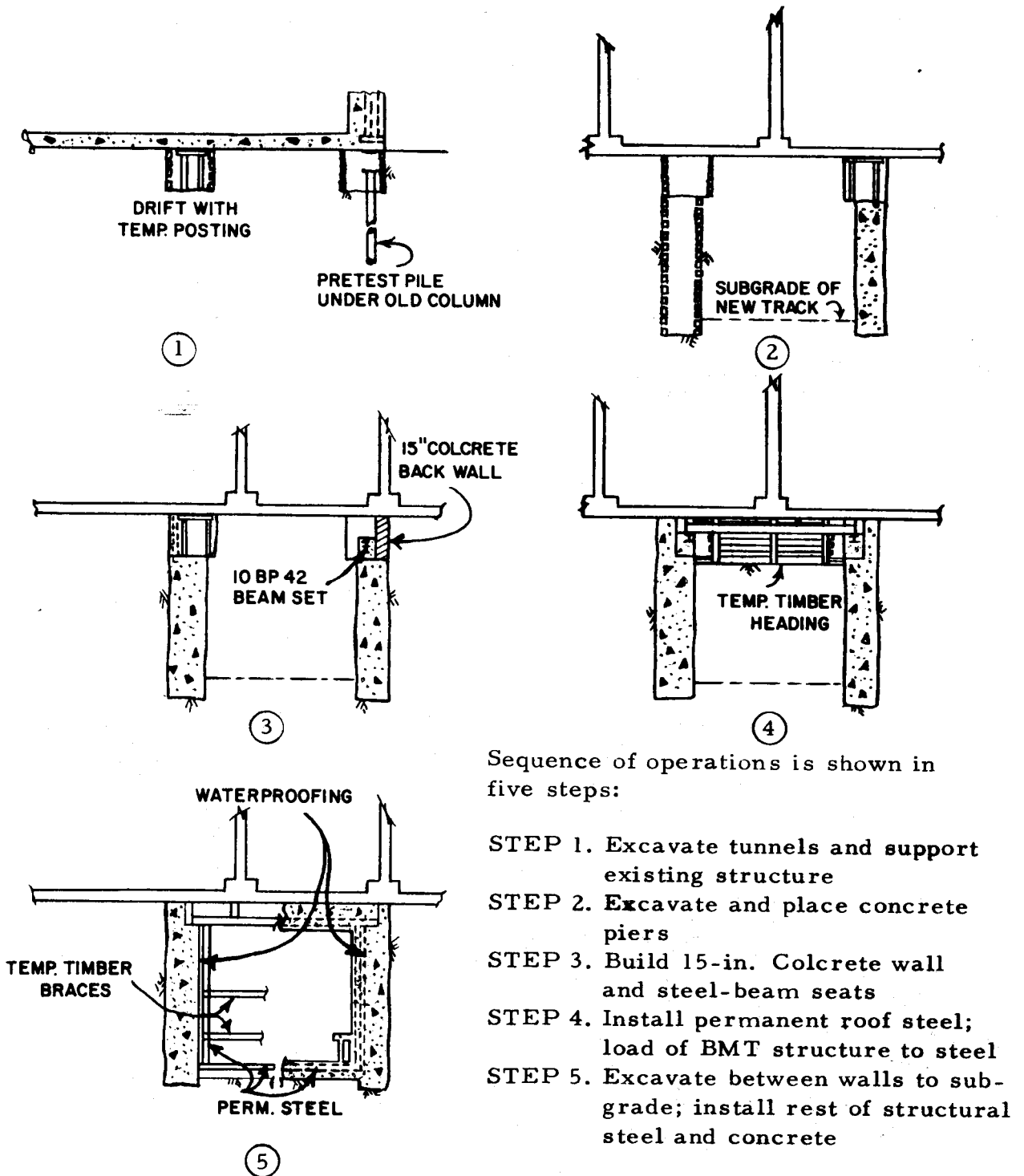


Figure 122. Construction sequence for subway tunnel (from Goldfinger, 1960).

7.60 LOAD TRANSFER

The transferring of the load from the old foundation or temporary shoring to the new underpinning elements is similar for all underpinning methods. Sources of potential settlement are compression of the underpinning member and displacement of the bearing stratum.

7.61 Dry Pack Alone

This is the simplest method but has the drawback that little if any of the elastic compression in the underpinning element or compression of the bearing stratum is accounted for prior to transferring the load. For this reason the use of dry pack alone is generally limited to pit underpinning since the pits are large enough that stresses are relatively small and elastic deformations are minimal. The dry pack is a dry mortar mix, generally consisting of one part cement, one part sand, and sufficient water to hold the mixture together. It is placed in the void between the underpinning element and the existing footing by ramming with a 2 x 4 and maul.

7.62 Plates and Wedges

This method consists of using pairs of steel or wooden wedges driven between steel plates in the void between the underpinning element and the footing. As the wedges are driven, their combined width increases. The footing then acts as a reaction, and the load in the underpinning element increases. For a permanent installation, dry pack may be used to fill voids. If the wedges are steel, they can be welded together to prevent future deformation.

7.63 Jacking

Jacking is done with mechanical jacks, hydraulic ram jacks, or with hydraulic flat jacks, where the space is too restricted to accommodate conventional jacks. Hydraulic jacks have the advantage that the hydraulic pressure can be monitored, and the load in the jack determined.

Where creep is minimal, the load can be transferred immediately by a steel or concrete plug. The plug is then dry packed, and the jacks removed. Where there is concern over settlement, the load can be maintained and periodically adjusted as needed.

7.70 TEMPORARY SUPPORT OR "SHORING"

7.71 Basic Considerations

The need for temporary support must be assessed for each structure to be underpinned. Generally, shoring will be required if:

a. The structural integrity of the structure being underpinned will be adversely affected during the underpinning operation. For example, old masonry walls with a poor footing might need temporary supports to prevent collapse.

b. The percentage of footing undermined will be sufficiently large to cause settlement from the increased loads on the adjacent soil.

The design of a temporary support system, in addition to geotechnical considerations, is a structural problem with the following items being individually designed:

a. New footings to transfer the loads from the shores to the soil.

b. Shores which transfer the load from the structure to the footing - - these shores can be beams, columns (either vertical or inclined), or combinations of both.

c. A method of transferring the load into the shoring system from the structure by welding, bearing, or friction.

d. A method of removing the elastic deformations so that when the load of the structure is transferred to the shoring settlement of the structure will not be excessive.

Shoring presents some special problems. First, when old walls are encountered, it is often not possible to "shore" these walls without reinforcing the footing. In some cases the entire footing must be rebuilt prior to both shoring and underpinning. In extreme cases entire walls have to be rebuilt.

A second consideration is the moment and shear capacity of the walls being underpinned. Asymmetric loading or load concentrations (such as from high capacity underpinning piles) are typical concerns. Lateral support and/or reinforcement is often necessary to alleviate this type of problem.

Temporary support is not always required in underpinning. If a structure has a sound foundation and if walls can arch without damage, portions of the foundation may be undermined for underpinning without structural damage. Additionally, if the material on which the foundation bears is relatively sound, settlement will generally be minimal.

While there are no hard and fast rules concerning tolerable undermining, under favorable conditions pits can be installed at about 16 feet on center below continuous walls. Below isolated footings about 20 percent of the bearing area can be removed at a time.

Usually it is very difficult, and often impossible, to predict the loads which the shores will carry. Accordingly, during transfer of load to the shoring, movements of the element being shored should be monitored throughout construction. The shoring can be jacked or wedged to compensate for settlement, if and when it occurs.

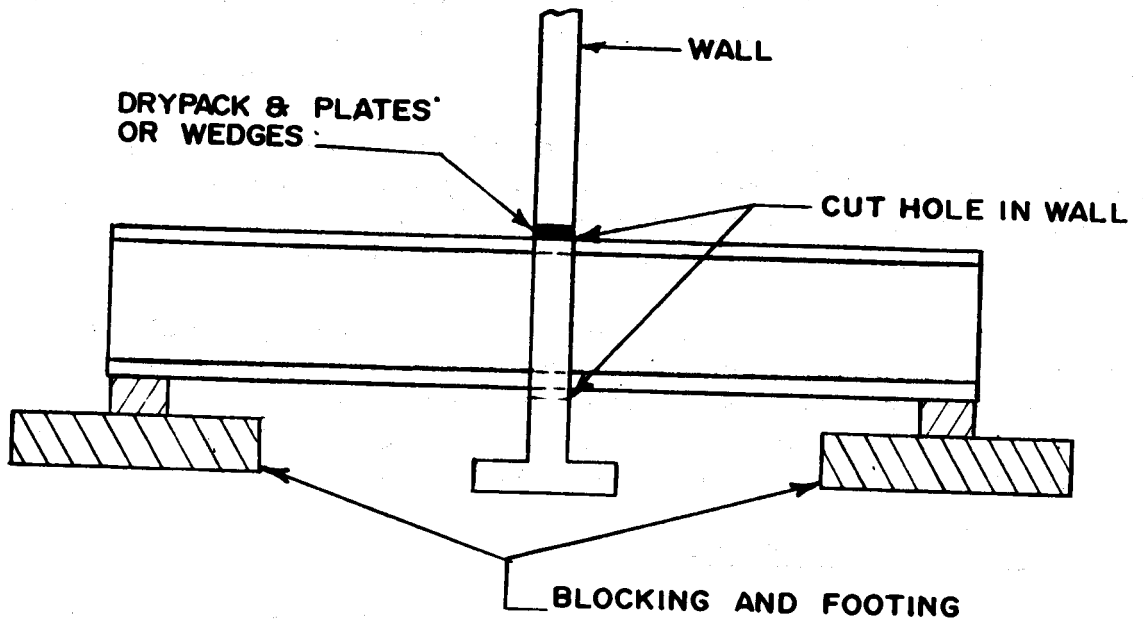
7.72 Needle Beams

The most commonly used method of shoring is the use of "needle beams". These "needle beams" can be used to shore both continuous walls and individual columns. Typical "needle beam" configurations are shown in Figure 123. The actual configuration can vary significantly depending on the requirements and the field conditions associated with the actual building. An elaborate system, where little settlement is tolerable, might consist of concrete pads and steel needles with jacks at the support points to control the movement of the structure. On the other hand, in less critical situations, the entire shoring system might consist of timbers. Again, the exact design must be made for the specific structure in question and the specific requirements of the entire construction operation.

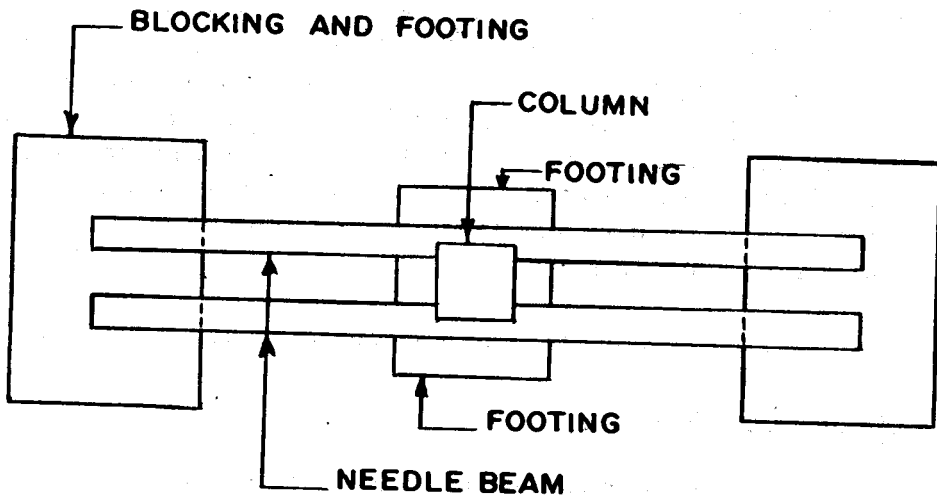
Figure 124 shows the underpinning of concrete columns.

7.73 Inclined Shoring

The use of inclined shoring is also common and is particularly applicable in cases where access is limited, where needle beams may be excessively long or deep, or where some lateral support is required. Configurations of inclined shoring systems vary greatly depending on the requirements and the structure being shored. Some typical configurations are presented in Figure 125. In all cases, the lateral loads transmitted through the shores must be resisted in the shore footings.



(a) NEEDLE BEAM THROUGH WALL
(SIDE VIEW)

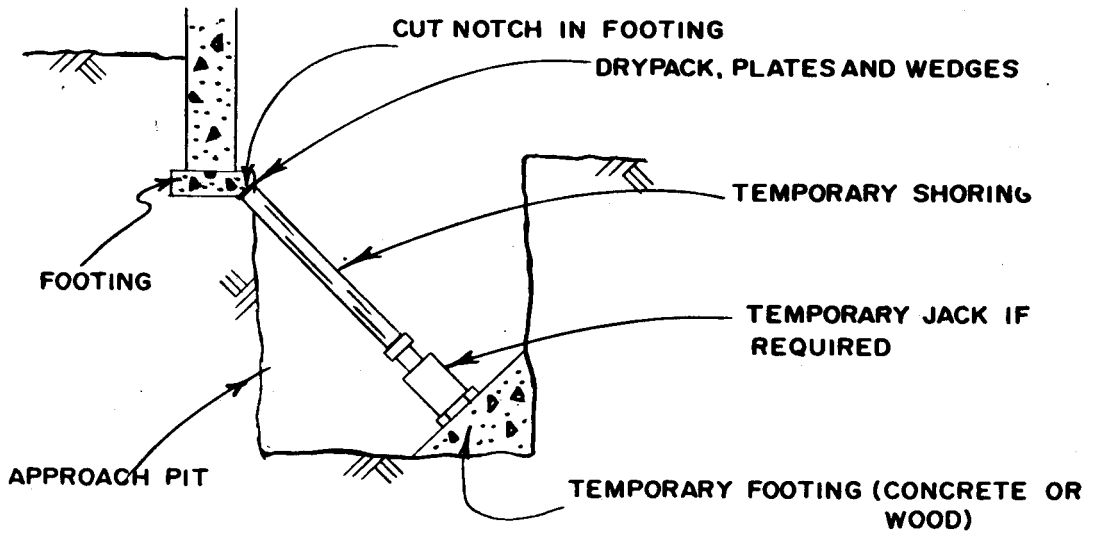


(b) NEEDLE BEAMS SUPPORTING A COLUMN
(PLAN VIEW)

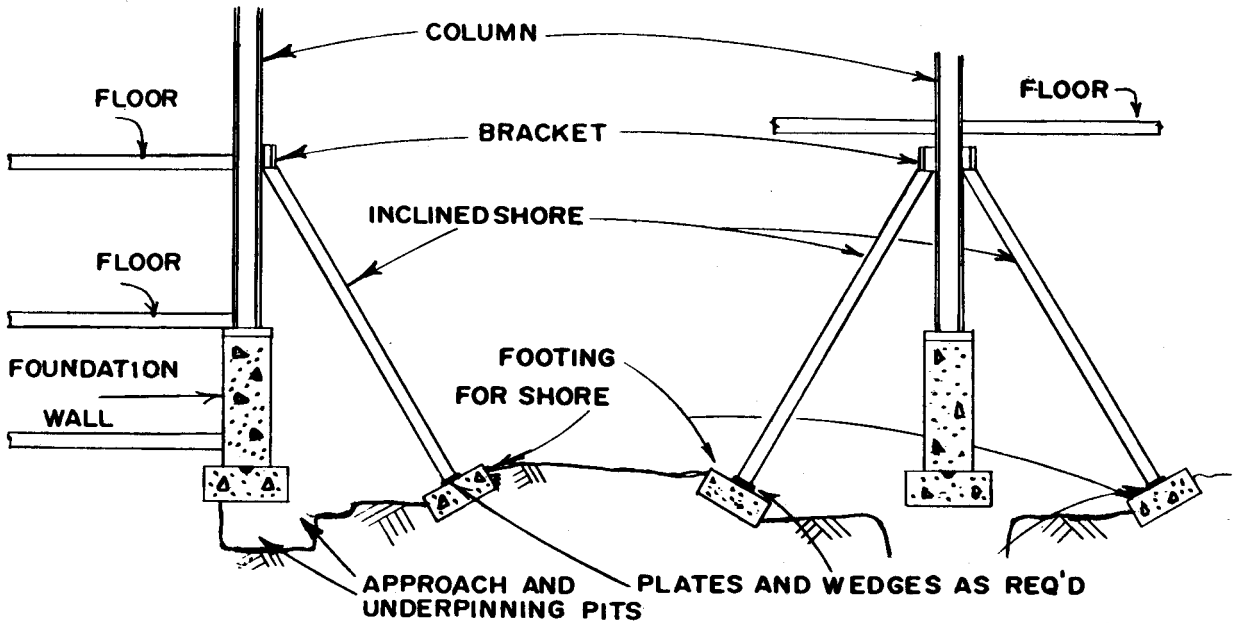
Figure 123. Needle beam detail.



Figure 124. Lateral bracing of wall prior to underpinning.
(Courtesy of Spencer, White, and Prentis).



(a) INCLINED SHORING UNDER A FOOTING



(b) SHORING A WALL OR COLUMN

Figure 125. Inclined shoring details.

Some common details of shoring connections are shown in Figures 126 and 127. When cast iron columns are encountered, special attention must be given to prevent damage to the column. Often it is necessary to fill the cast iron column with concrete. The pin and clamp method is presented in Figure 128. The shoring of cast iron columns might also be accomplished by the use of a concrete collar placed over either a roughened surface or welded shear connections on the column. Regardless of method, eccentric loadings should be avoided.

Masonry walls are also a special problem when shoring. Loading of masonry walls should be performed with care to prevent excessive lateral stresses in the wall. Concrete walls present a similar problem.

Figure 129 illustrates a case where inclined shoring was used to protect a structure.

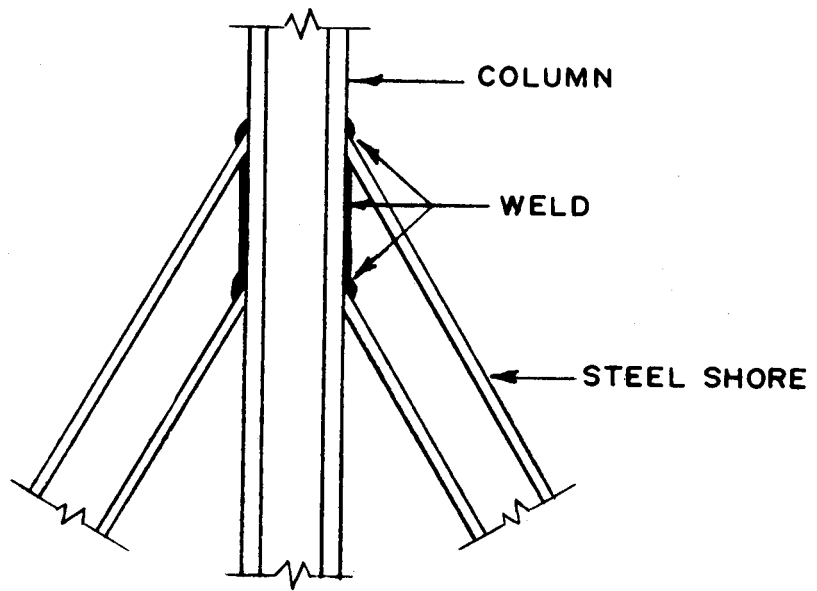
7.80 PERFORMANCE

Underpinning is no guarantee that the structure will be totally free from either settlement or lateral movement. About 1/4 - 1/2 inch of settlement should be expected during the underpinning process -- even under the best of conditions. Additional movements may be associated with the subsequent adjacent excavation, including lateral displacements occurring in the retained soil mass adjacent to the excavation.

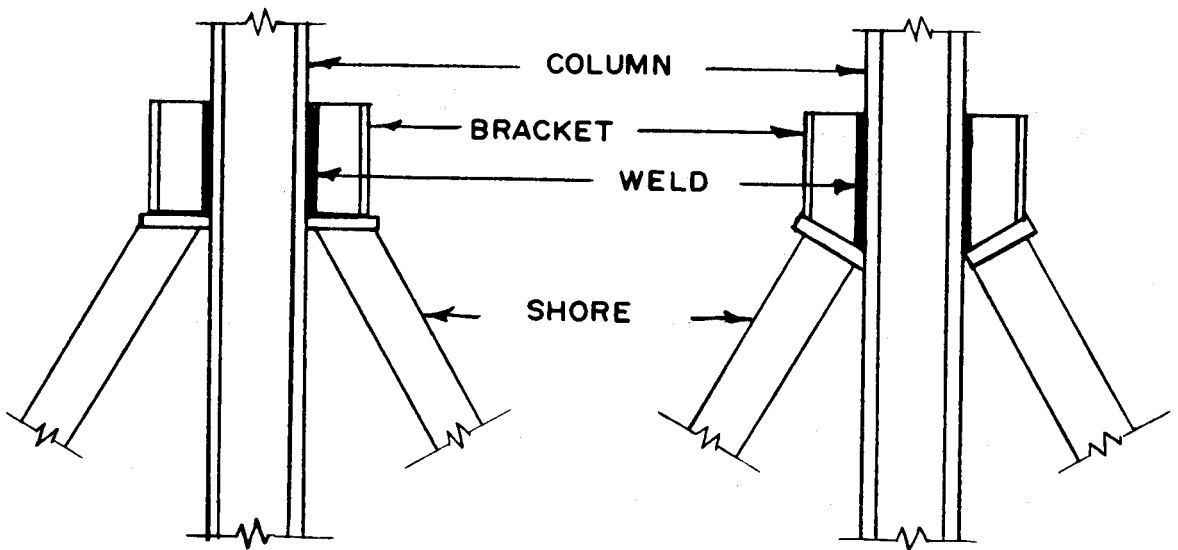
An extensive search of the literature produced little quantitative data on the performance of underpinning in connection with adjacent excavations. One exception to this general lack of performance documentation is the work by Ware (1974) which presents both settlements measured at the end of underpinning and overall settlements measured after the adjacent excavations were completed for various structures that were underpinned during construction of the Washington Metropolitan Area Transit Authority (WMATA) rapid transit system (METRO).

7.81 Pit or Pier Underpinning

Ware reports ten cases of structures underpinned by pit or pier underpinning. Settlements after underpinning were typically about 0.01 feet. In one case, the settlement was 0.03 feet. Total settlement after completion of the excavation was less than 0.03 feet except in two cases which experienced 0.04 feet and 0.05 feet.

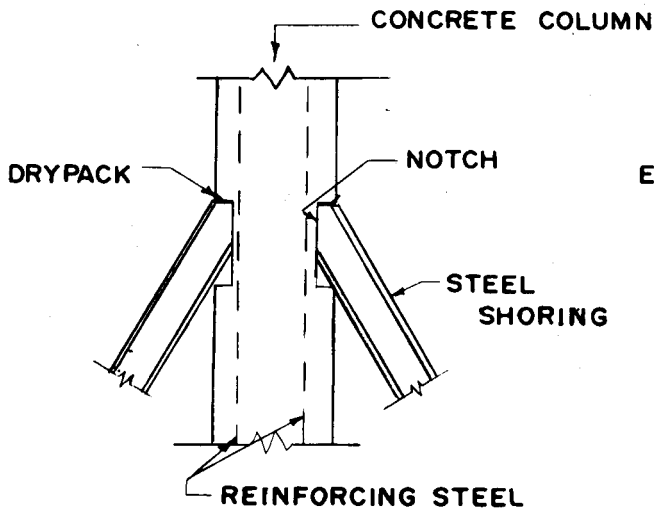


(a) WELD STEEL SHORES DIRECTLY TO COLUMN

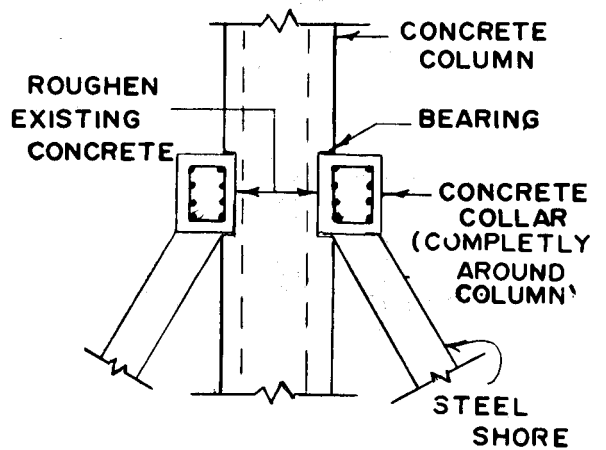


(b) SHORE TO BRACKETS WELDED ON COLUMN

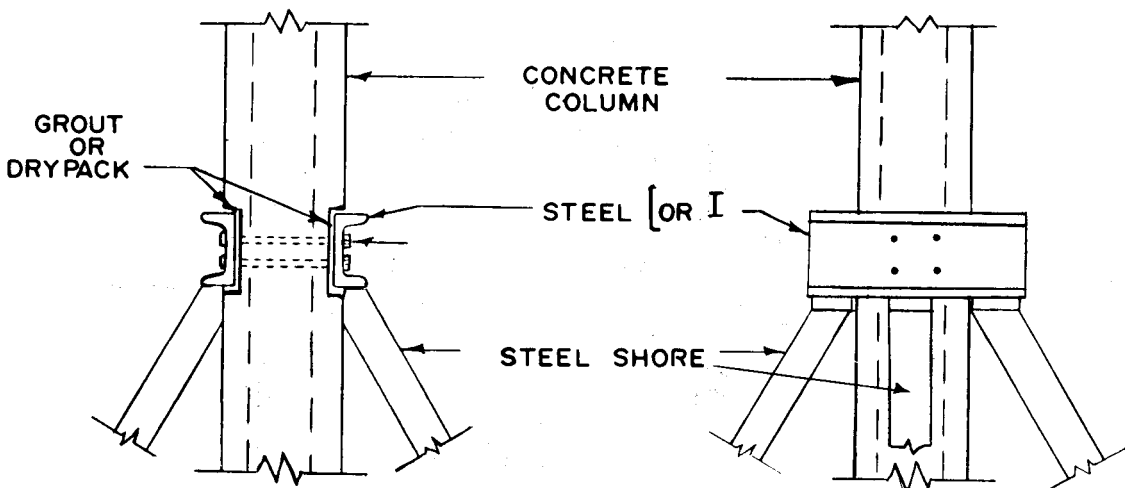
Figure 126. Shoring details, steel column.



(a) LIGHTLY LOADED COLUMN



(b) SHORE AGAINST CONCRETE COLLAR



(c) CLAMP STEEL BEAMS OR CHANNELS TO COLUMN AND SHORE AGAINST CLAMPED BEAMS.

Figure 127. Shoring detail, concrete column.

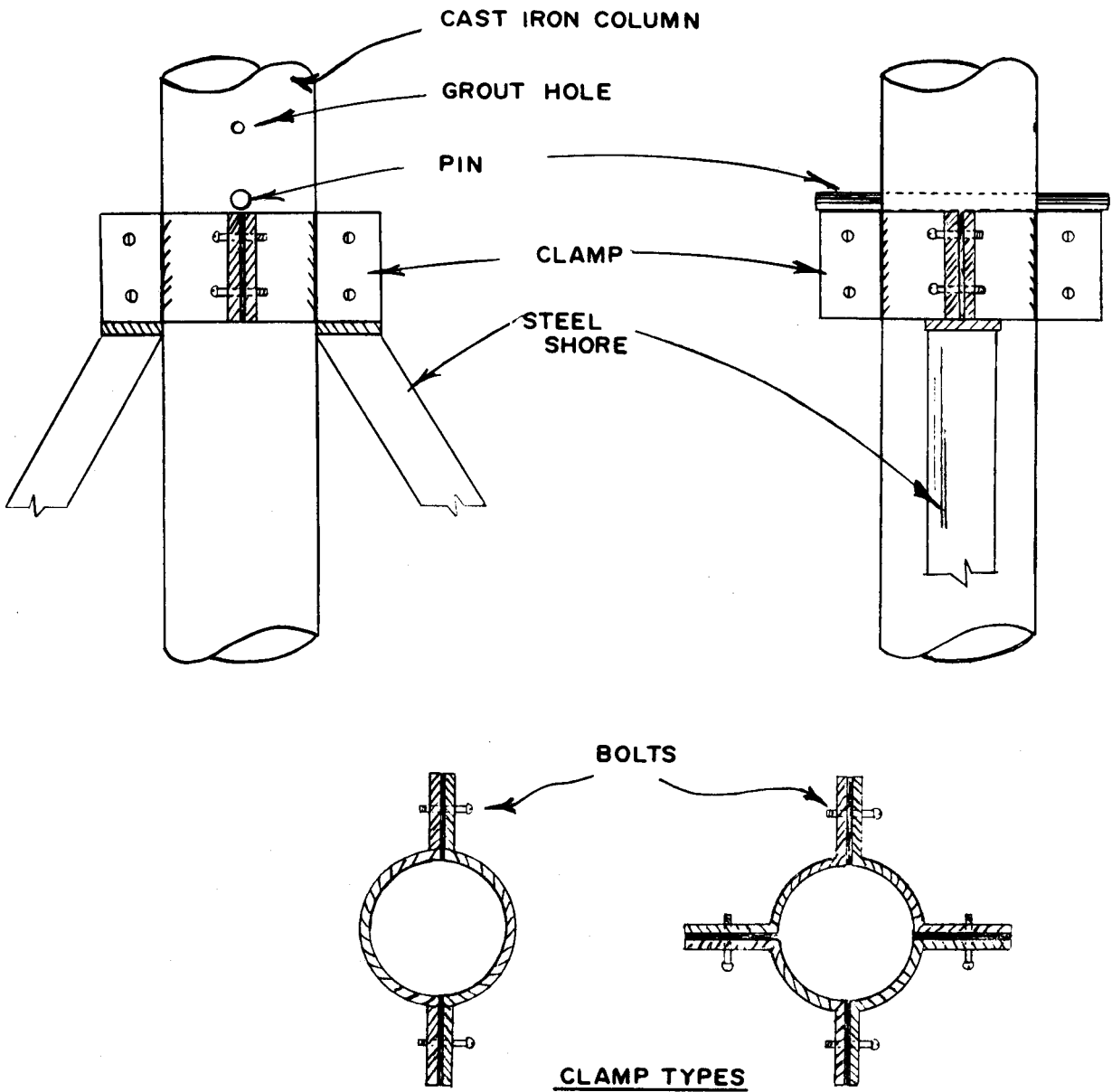


Figure 128. Pin and clamp details for a cast iron column.

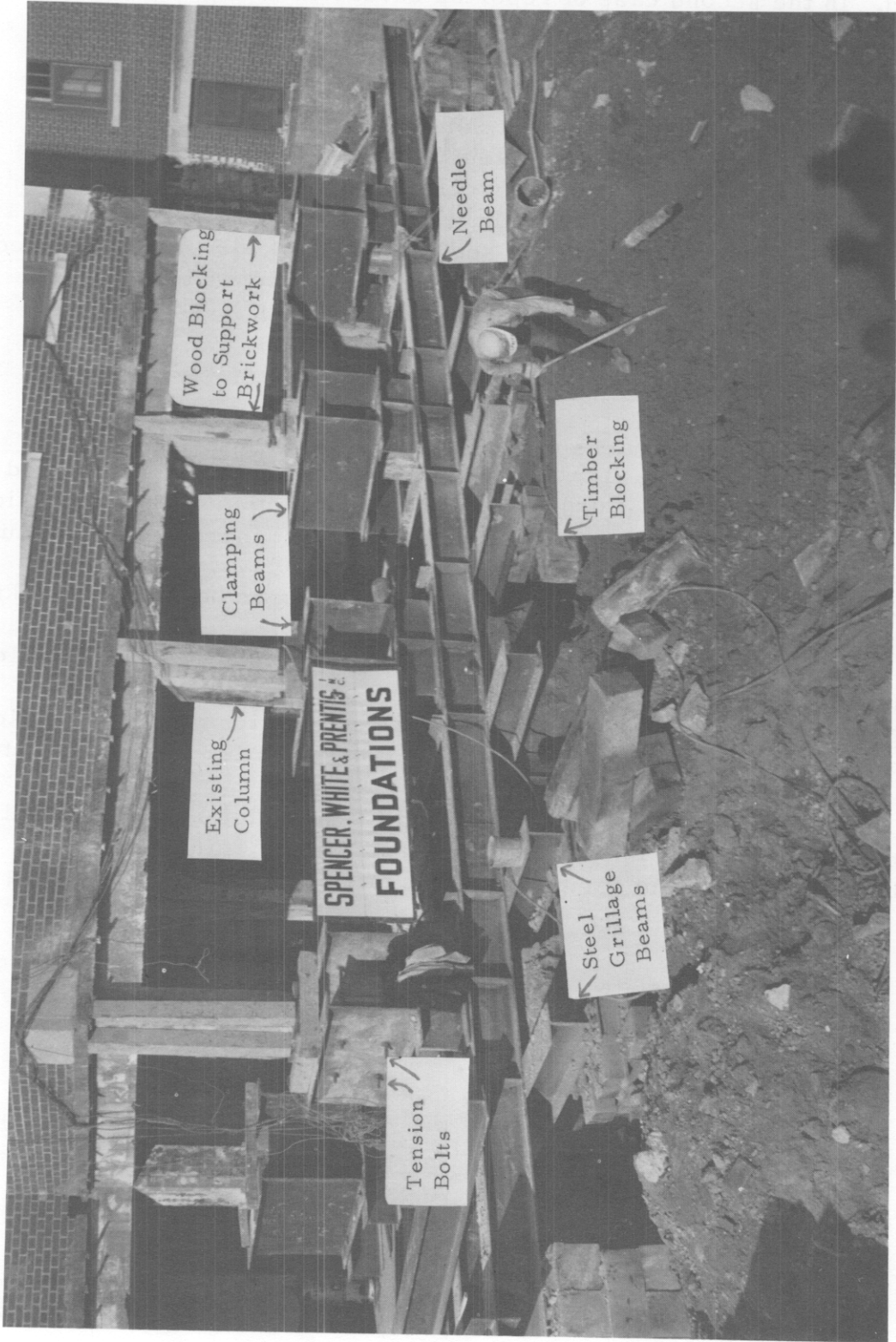


Figure 129. Shoring of concrete columns.
(Courtesy of Spencer, White, and Prentis).

this underpinning phase that settlement was experienced and damage occurred. In the second case of reported severe cracking the damage was reported to have been primarily a result of about one inch of lateral displacement.

The Norwegian Geotechnical Institute (1962, No. 7) reported the case of an underpinned structure on soft clay overlying bedrock. Because of nearby subway construction, the structure was underpinned. During the underpinning of the structure (jacked piles), approximately 6 cm (2.4 inches) of settlement of the structure occurred. The subsequent subway construction using cut-and-cover techniques resulted in settlements in excess of 10 cm (4 inches).

7.82 Jacked Pile Underpinning

Fourteen cases of underpinning using jacked piles are presented by Ware. In twelve of these cases settlement did not exceed 0.03 feet; most were about 0.01 feet. After completion of the excavations these cases exhibited less than 0.01 feet settlement increase. Maximum settlement after completion of the excavation was 0.03 feet. Nine structures had no noticeable damage; three structures had slight cracking.

The two remaining structures experienced greater settlement and had severe cracking. The average settlements after underpinning were 0.04 feet and 0.06 feet. After completion of the excavation the average settlements were 0.04 feet and 0.09 feet for the two structures.

One of these structures had load bearing brick walls with no footings. As part of the underpinning operation a reinforced concrete beam was placed under the wall in short sections. It was during

CHAPTER 8 -- GROUTING

8.10 INTRODUCTION

8.11 General

The practice of grouting was invented and first applied by Charles Bérigny in 1802 (Ischy and Glossop, 1962). The original process consisted of pumping slurried clay and hydraulic lime into sub-aqueous formations with a simple pump. Since the first use of grouting, improvements in methods, grouts, and applications have followed which have resulted in the development of a powerful tool in improving the engineering properties of soil and rock. Grouting has become a particularly valuable tool in urban areas where existing structures are founded on soils (or rock) that can be affected by nearby construction.

8.12 Purpose and Scope

This chapter provides the engineer and/or contractor with a general overview of the design and implementation of grouting systems. Special emphasis is placed on the use of grouting in cut-and-cover and soft ground tunneling situations. This section is primarily a condensed state-of-the-art review presenting basic design and construction features of grouting as well as examples of typical applications.

This chapter describes the basic design principles controlling the use of grouting techniques. This includes a discussion of the situations in which grouting is feasible, the soil types that can be grouted, and the type of grouts that should be used. In addition, simplified design criteria are presented which can aid in evaluating the feasibility of various grouting schemes.

This section is not a comprehensive design or construction manual on grouting. A comprehensive design and construction manual on grouting is being prepared by Halliburton Services of Duncan, Oklahoma, for the Federal Highway Administration and will be available through the National Technical Information Service. Other sources of general information on grouting are M. I. T. (1974), Sverdrup and Parcel (1973), and Cambefort (1964).

8.20 DESIGN AND THEORETICAL CONSIDERATIONS

8.21 General

In order to evaluate or design a grouting scheme, the

engineer must know the purpose of the grouting, the soil profile, specific soil data, the characteristics of the various grouts, and the behavior of the grouted mass. This information allows the engineer to evaluate the technical and economic feasibility of grouting schemes.

8.22 Purpose

The three basic reasons for grouting are to control ground water, to solidify or stabilize a soil mass, and to underpin an existing structure. For a given project, grouting may achieve one or all three of these purposes. The choice of the grout and the method of grouting will often depend upon the purpose of the grouting.

8.22.1 Control Ground Water

Grouts injected into a soil mass may reduce the permeability of the soil mass, and if properly designed and installed, effectively act as a cutoff. Cut-and-cover tunneling and soft ground tunneling in urban areas often require that the water level outside the construction area be maintained at its original level. Lowering of the water level may induce consolidation of compressible layers and result in settlement of existing structures. In other cases ground water lowering is difficult, and flow may cause washing and transporting of the soil into the excavation (through open lagging, for example). A grouted cutoff wall would prevent washing and transporting of soil during construction.

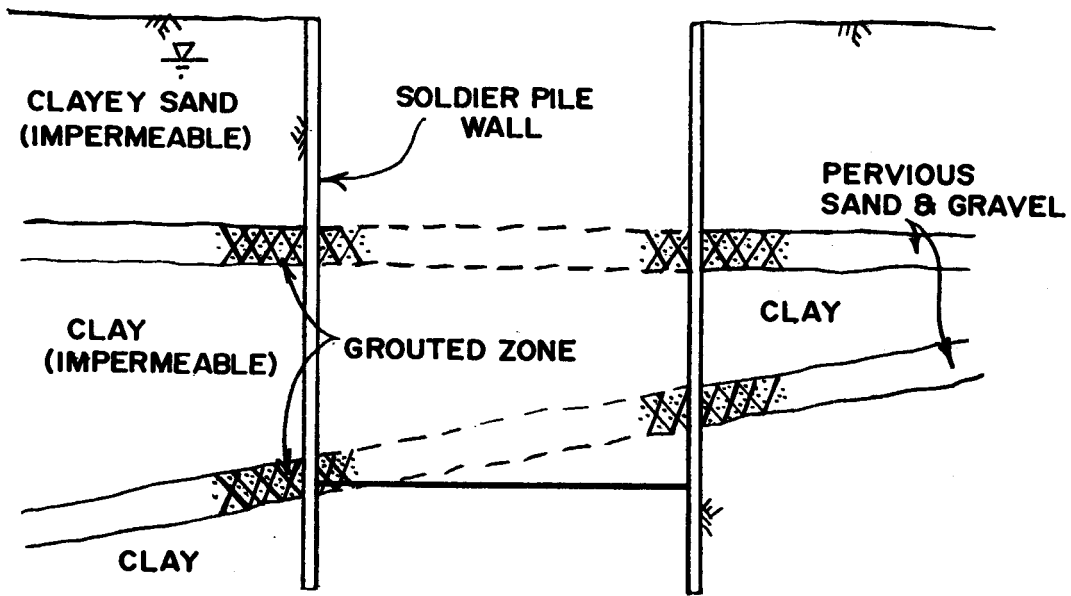
Grouting may be used to supplement an existing ground water control scheme. For example, in dense soils steel sheet piling may separate or become damaged. Water may then flow freely through the sheet pile wall. Also, as shown in Figure 130c, it may be impossible to obtain an adequate cutoff with sheet sheeting alone.

Figure 130 illustrates cases where grouting may be used for ground water control. Panel (a) of the figure shows a soldier pile wall with horizontal wood sheeting. Panel (b) illustrates a horizontal cutoff and the requirement for gravity resistance against hydrostatic uplift. This latter case was performed for a subway in Lyon, France. (Majtenyi, 1975).

8.22.2 Soil Solidification - (Stabilization)

Excavation of a tunnel (cut-and-cover or bored) through loose or running soils may result in large deformations in the soil mass. This is particularly true if these soils lie below the water

a) PERVIOUS LAYERS WITHIN DEPTH OF CUT



b) DEEP PERVIOUS DEPOSITS

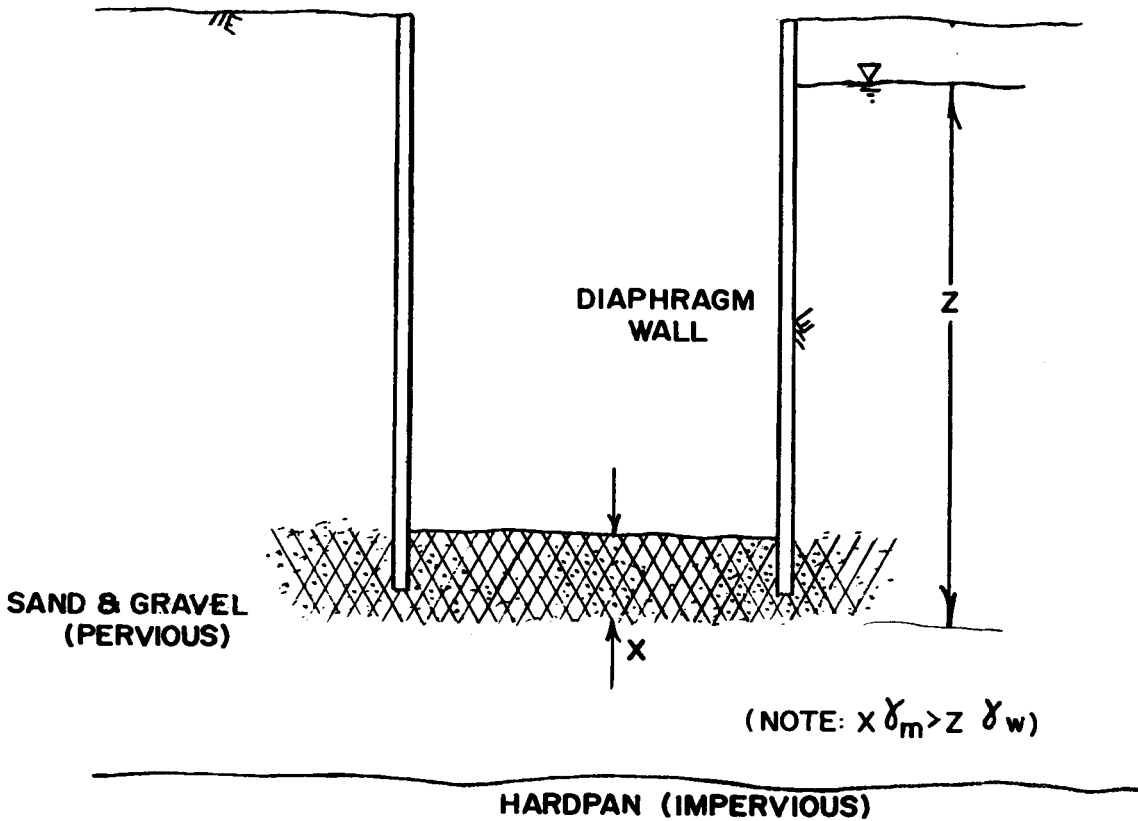


Figure 130. Grouting for ground water control.

c) BOULDERS OR SHATTERED ROCK

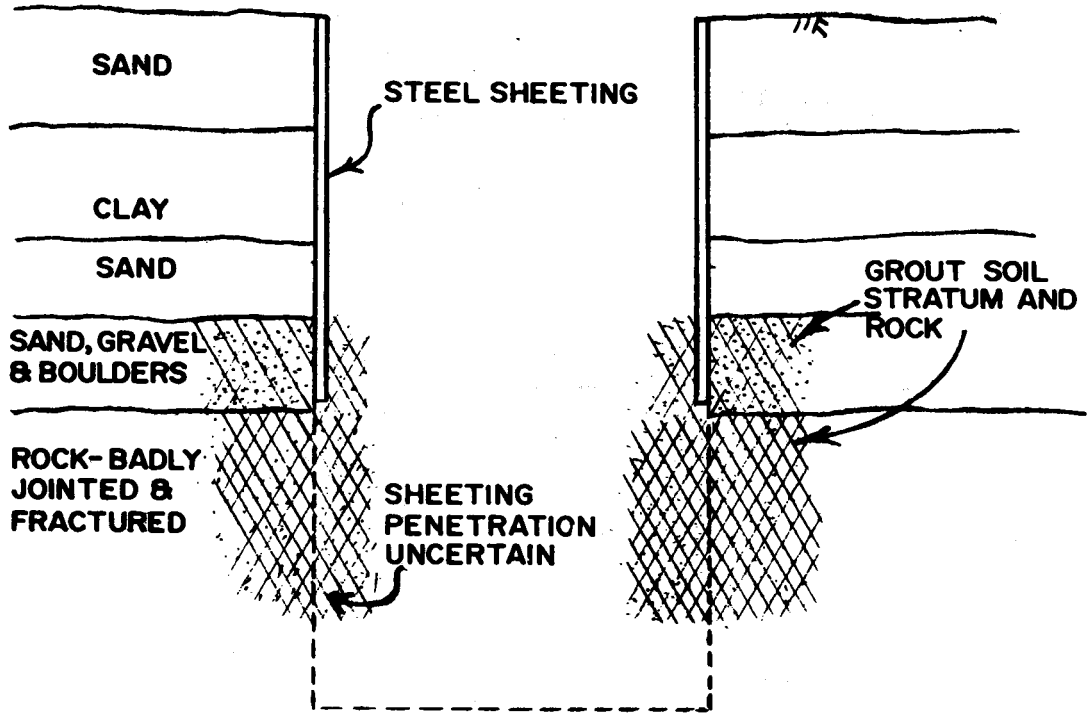


Figure 130c. Grouting for ground water control.

table. Grouting of soils can improve the strength and deformability characteristics of soils within the zone of influence of the excavation. Figure 131 illustrates two cases -- one to solidify loose soil in a cut-and-cover application, the other to penetrate and solidify sand to develop an "arch" over a tunnel. Passive resistance can also be improved by grouting, as is the case of the example shown in Figure 130 (b). The grouted soils may act as a lateral support wall.

8.22.3 Underpinning

Grouting to provide underpinning support for a structure is a specific application of grouting for soil solidification. Grouting might be used instead of conventional underpinning procedures if conventional procedures would cause untenable settlements during construction or if the grout can also serve another function (ground water cutoff or lateral support wall). Figure 132 illustrates a case where grouting may be used to underpin a structure.

8.23 Soil Profile and Soil Type

8.23.1 Field Investigations

Field investigations undertaken for a proposed grouting scheme fall into two phases. The first investigative phase would include obtaining an accurate definition of the soil profile, particularly of the soils to be grouted. Although much of this investigation may be encompassed within the normal site investigation, more detailed information on stratigraphy may be required.

The second phase may include field permeability tests and soil sampling for laboratory testing. The purpose of this phase of investigation is to obtain more data on the specific soil properties controlling groutability. The in situ soil permeability may be determined by various borehole procedures -- falling head or constant head flow from the borehole or rising head flow into the borehole or from pumping tests. Pumping tests are preferred since more reliable values of permeability are obtained. Siltation and limited flow quantities often adversely affect the permeability values obtained from borehole methods.

In rock, instances of water loss during drilling should be recorded, and rock core logging should reflect jointing, weathering, and RQD -- all of which bear a relationship to permeability.

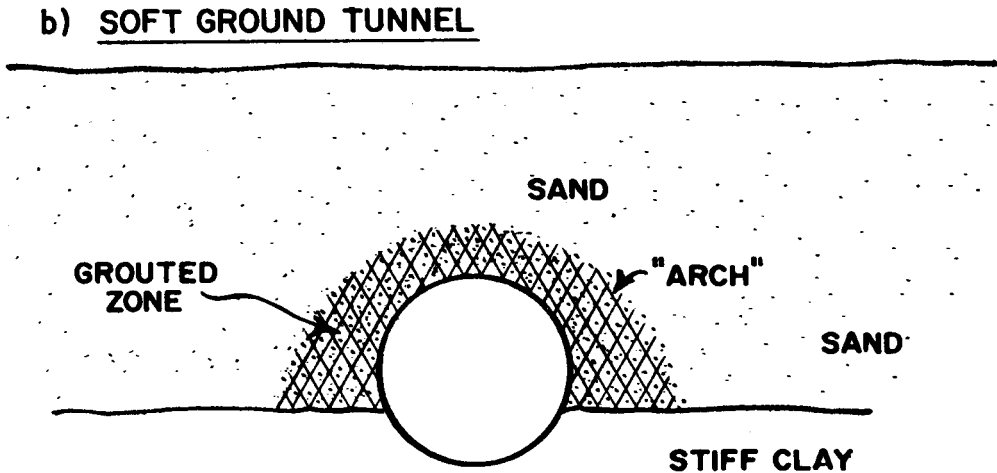
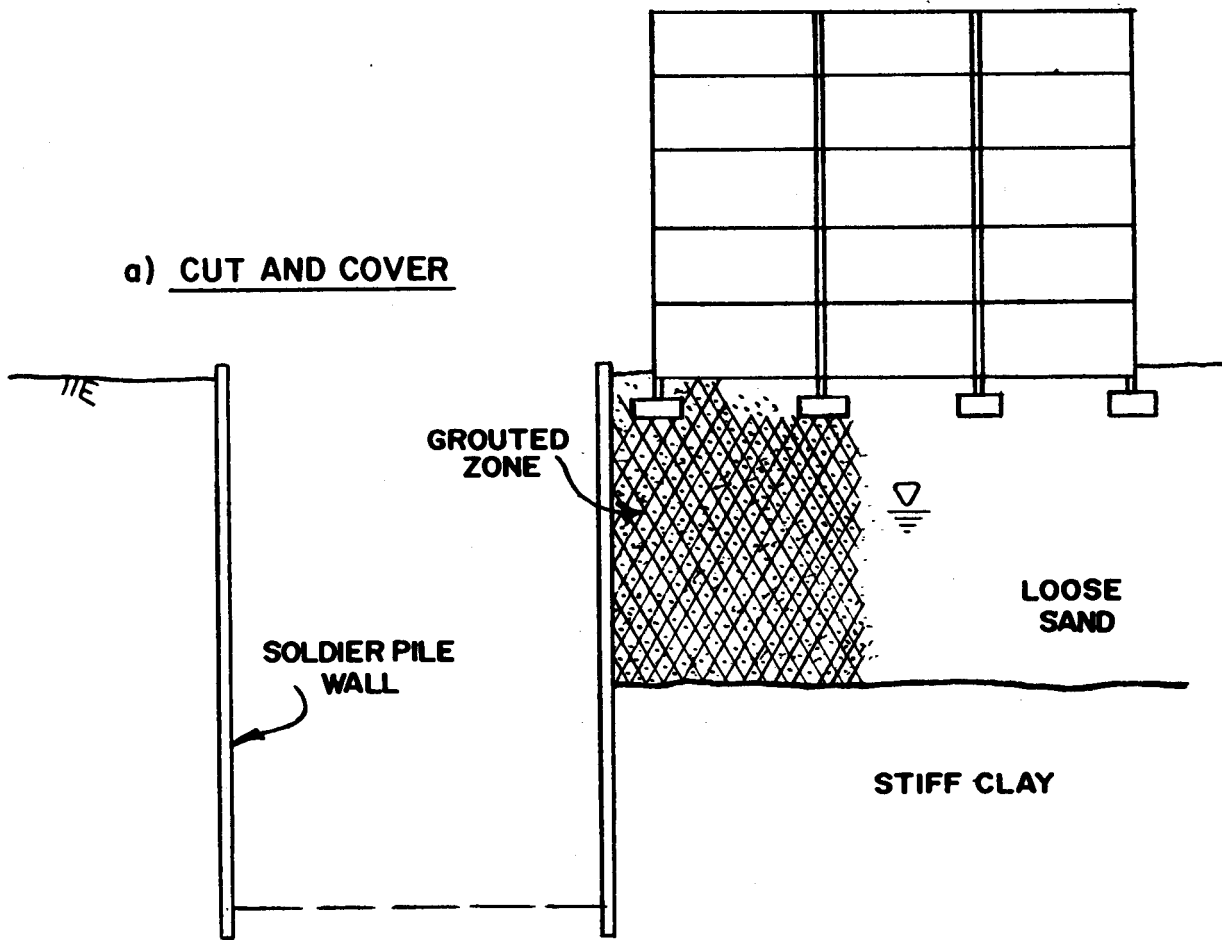


Figure 131. Grouting for soil solidification.

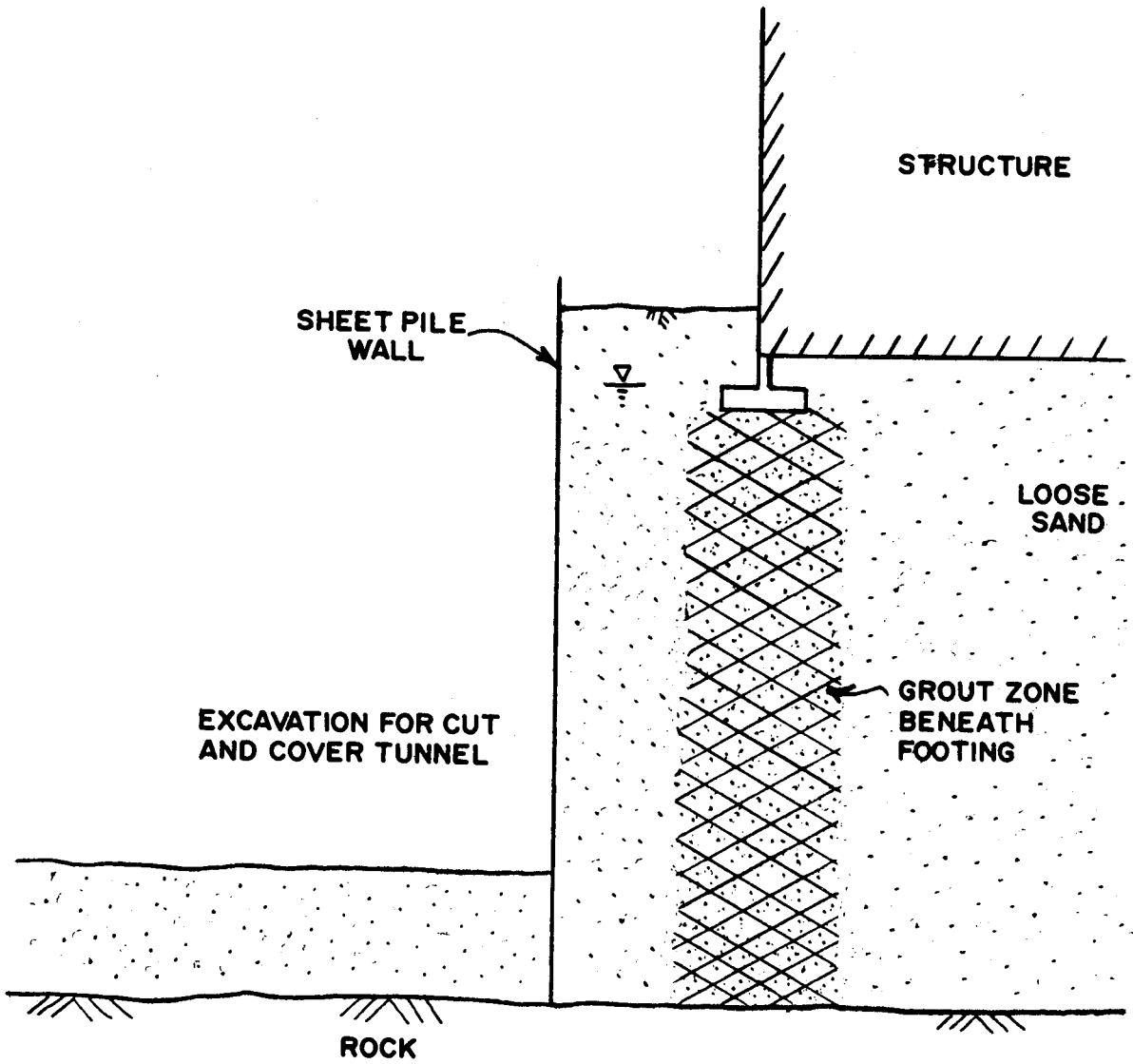


Figure 132. Example of grouting used to underpin an existing structure.

8.23.2 Laboratory Investigations

Laboratory testing of soils that are being considered for grouting will be limited primarily to detailed logging to map stratigraphy, grain size analyses, and laboratory permeability tests. Detailed knowledge of the stratigraphy will determine appropriate grouting methods and procedures. If the deposit is very homogeneous with little vertical variation, one grouting procedure may be most economical. However, if the deposit is highly stratified, an entirely different procedure or procedures may be more appropriate.

Grain size analyses provide an indication of the type of grout that can be used or, indeed, if the soil can be grouted at all. In granular soils, where less than 10 percent of the soil by weight passes the No. 200 sieve, grouting techniques can be used to stabilize the soil or to provide a ground water cutoff. Although it may be technically possible to grout finer soil deposits, grouting soils with greater than 10 percent by weight passing the No. 200 sieve is expensive and difficult.

Permeability values obtained from laboratory tests are useful, but their usefulness is limited since the tests are generally performed on reconstituted samples. Therefore, laboratory and field permeabilities may vary considerably. An assessment of all parameters -- grain size distribution, stratigraphy, laboratory permeability tests, field permeability tests -- provides a basis for judging whether a soil deposit can be successfully grouted.

8.24 Grout Type

8.24.1 General

Although there are many different proprietary grouts produced by a variety of manufacturers, grouts can be grouped into two major categories -- particulate and chemical. Bituminous emulsions have also been used as grouts although they are much less widely used than particulate and chemical grouts. This section presents a description of the major grout types as well as some basic design criteria.

8.24.2 Particulate Grouts

Particulate grouts are fluids with solid particles, such as cement, clay, a processed clay like bentonite, or a mixture of

these elements, suspended in the fluid. The groutability, or the ability of a grout to penetrate the soil, is limited by the size of the particle in suspension and the size of the voids in the material to be grouted. Mitchell (1968) defines a groutability ratio for soils as the ratio of the 15 percent size of soil to the 85 percent size of the particulate grout. For successful grouting the ratio should exceed 25.

$$\text{Groutability ratio} = \frac{D_{15}(\text{soil})}{D_{85}(\text{grout})} > 25.$$

In practice, normal cement based grouts are used only in coarse sands while a pure bentonite grout might be injected into a medium sand.

8.24.3 Chemical Grouts

Chemical grouts are frequently classified into two major groups : silica or aluminum based solutions and polymers. Metathetical precipitation processes (M.I.T. , 1974) generally use silicate solutions (with sodium silicate being the best known) although aluminates are also used. The basic process consists of adding acid to a soluble silicate to form a silicate gel and salt. Chromeligno-sulfates also fall into the general category of metathetical precipitation grouts.

Polymers are generally more fluid than the metathetical precipitation grouts. In these grouts monomers or partially polymerized polymers react to form macromolecules. The reaction can be triggered by catalysts or by application of heat, pressure, or radiation (M.I.T. , 1974).

Bituminous emulsions have also been used as grouts and are similar to polymer solution grouts. The reaction of these grouts consists of a removal of the carrier liquid (water) and the creation of bonds between the droplets of the emulsified material and the base material (M.I.T. , 1974). Table 14 summarizes the basic grout types and lists some of the common grouts according to these general groupings. Bituminous grouts differ from chemical grouts primarily by the reaction by which they solidify.

Chemical grouts are used to grout fine-grained deposits such as fine to medium sand and, in some instances, coarse silt. Unlike particulate grouts that are injected as suspensions, chemical grouts are injected as true solutions. Chemical grouts are therefore idealized to behave as Newtonian fluids exhibiting a characteristic viscosity. Viscosity, together with the permeability of the soil and the injection pressure, will control the groutability. E. Maag in 1938 (Ischy & Glossop, 1962) developed a simplified model of the behavior of a Newtonian fluid:

Table 14. Classification of common grout types
 (from Massachusetts Institute of Technology, 1974).

<p>Particulate Grouts</p>	<p>Cement Clay Bentonite</p>	<p>Suspensions</p>
<p>Chemical Grouts Precipitation</p> <p>Polymers</p> <p>Bituminous Emulsions</p>	<p>Silicate Chemicals Aluminate Chemicals Chromelignosulfates</p> <p>Acrylamides (e. g. AM9) Phenoplasts or Aminoplasts (e. g. recorcineformol, urea-formol)</p> <p>Epoxy Polyester-resins</p>	<p>Injected in form of monomers</p> <p>Injected partially polymerized</p>

$$t = \frac{\alpha n}{3khr} (R^3 - r_o^3)$$

Where:

R = The radius of grout distribution (idealized sphere)

r_o = the radius of the injection pipe

n = porosity of the soil

k = permeability of the soil

α = ratio of grout viscosity to that of water

h = piezometric head in the grout pipe

t = time of grouting

Maag's formula is based upon several simplifying assumptions -- a uniform homogeneous soil, spherical flow, radius of injection pipe small with respect to depth below water, and injection occurring above impermeable boundaries. In view of the many unknowns inherent in any soil mass, a more precise theoretical solution to the problem of rate of grout penetration is of questionable value. For a more precise determination of the rate of grout penetration field injection tests would be required.

8.24.4 Choice of Grout

The choice of a grout involves an evaluation of the grain size and permeability of the soil and the cost of grouting. In general, particulate grouts are used in coarse sands and gravels while chemical grouts are used in medium to fine sands and silts. The relationship presented in Tables 15 and 16 and Figure 133 provide a general guideline in choosing the type of grout to be used.

Since cement and clay suspension grouts are significantly less expensive than chemical grouts, these grouts are used whenever possible. In stratified deposits, particulate and chemical grouts may both be used. The particulate grout would be used to grout coarse-grained deposits while chemical grouts would be used to grout the finer-grained deposits.

More than one grout can be used to grout a soil mass. Less expensive grouts may be used to fill the larger voids while the less viscous (and more expensive) grouts are used in final grouting to assure complete grouting of the soil mass. The use of more than

Table 15. Limits of grouting ability of some mixes.

Type of Soils	Coarse Sands and Gravels	Medium to fine Sands	Silty or Clayey Sands, Silts	
Soil Properties	Grain diameter	$d_{10} > 0.5\text{mm}$	$0.02 < d_{10} < 0.5\text{mm}$	$d_{10} < 0.02\text{mm}$
	Specific surface	$s < 100\text{ cm}^{-1}$	$100\text{ cm}^{-1} < s < 1000\text{ cm}^{-1}$	$s > 1000\text{ cm}^{-1}$
	Permeability	$k > 10^{-3}\text{ m/s}$	$10^{-3} > k > 10^{-5}\text{ m/s}$	$k < 10^{-5}\text{ m/s}$
Series of Mix	Bingham Suspensions	Colloid Solutions (Gels)	Pure solutions (Resins)	
Consolidation Grouting	Cement ($k > 10^{-2}\text{ m/s}$) Aerated Mix	Double-shot silica-gels (Joosten) Single-shot silicate	Aminoplastic Phenoplastic	
Impermeability Grouting	Aerated Mix Bentonite Gel Clay Gel Clay/Cement	Bentonite Gel Lignochromate Light Carongel Soft Silicagel Vulcanizable Oils Polyphenol	Acrylamide Aminoplastic Phenoplastic	

After Janin and Le Sciellour, 1970

Table 16. Grout types for ground stabilization.

Soil Type	Particle Size Minimum	Grout Type
Fissured rock to coarse sand	5mm	Cement PFA Bentonite
Coarse sand to medium sand	1mm	Silicate
Medium sand to fine sand	0.1mm	Resins
Coarse silt	0.01mm	Acrylamide

After Flatau, et al, 1973

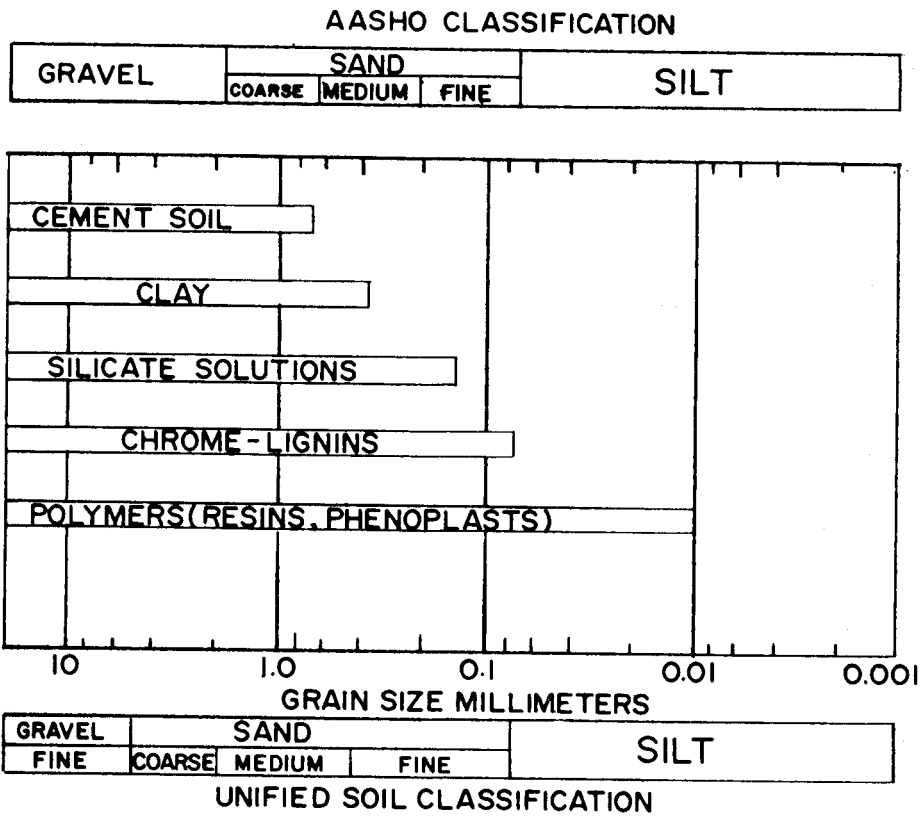


Figure 133. Range of usefulness of various grout types (from Mitchell, 1968).

one grout or injection depends on what the most economical procedure is. In some cases it may be less expensive to use the more expensive grout and have only one injection. Multiple injections are more common in European practice than U.S. practice.

8.25 Design Factors

8.25.1 General

The grouts selected for a grouting system may involve a combination of grouts, some of which are mixtures of individual grouts. While the final design of a grout system is done by a grouting specialist, the applicability of grouting and the factors that should be considered in design should be understood by the engineer and/or general contractor.

8.25.2 Grout Type

The choice of the grout type will be primarily controlled by its suitability for injection and its ability to do the required function (provide proper strength or provide ground water control). Mitchell (1968) describes some of the factors that must be considered when choosing a grout.

- a. Stability and the possibility of segregation within soil and cement grouts.
- b. Setting time; it is important to get the grout to the right place at the right time.
- c. Volume of set grout; a maximum volume with a minimum weight of material is usually desired.
- d. Adequate strength to prevent washing out and to support imposed loads.
- e. Viscosity; generally the lower the viscosity the better.
- f. Rheologic properties, yield stress, thixotropic properties, gelling characteristics.
- g. Particle size and distribution.
- h. Permanence.

8.25.3 Layout

Layout refers to the spacing and pattern of the grout holes during installation of the grout. The layout will depend upon the injection pressure, viscosity of the grout, soil type, and the gel time. Based largely upon experience, the grouting specialist will establish the configuration necessary to conform to the requirements of the job. The layout may need to be adjusted after grouting begins to accommodate unknown site conditions.

Figure 134 illustrates a scheme that was used to grout a cutoff wall for a tunnel project beneath an existing structure in Cologne, W. Germany (Sening and Klotschke, 1970). The grouting procedure involved installing the three grout rows, grouting the outer two rows, and then grouting the center row. The outer rows were grouted using the Joosten process while the inner row was grouted using the "Monodur" process. The "Monodur" process is used to grout finer soil deposits than the Joosten process. This technique minimized the amount of the more expensive grout used.

8.30 CONSTRUCTION CONSIDERATIONS

8.31 Materials

8.31.1 General

The basic grout types and their general range of applicability were presented in the design section on grouts. Tables 15 and 16 and Figure 133 classify the various grout types according to their possible uses and groutability. This section will discuss each of the specific grout types in more detail.

8.31.2 Particulate Grouts

Cement grouts are used primarily to increase strength but also have the added benefit of lowering permeability. Cement grouts can be used to grout soil deposits consisting of gravel and sand with a minimum particle size of approximately 0.6 mm. These grouts are the least expensive grout types and are often mixed with natural clay or bentonite to prevent cement segregation in coarser soil deposits.

Natural clays can be used as grouts, but they must be carefully studied before they are used. Generally, a clay grout will be used to fill voids to decrease permeability as it will give little or no increase in strength to the soil.

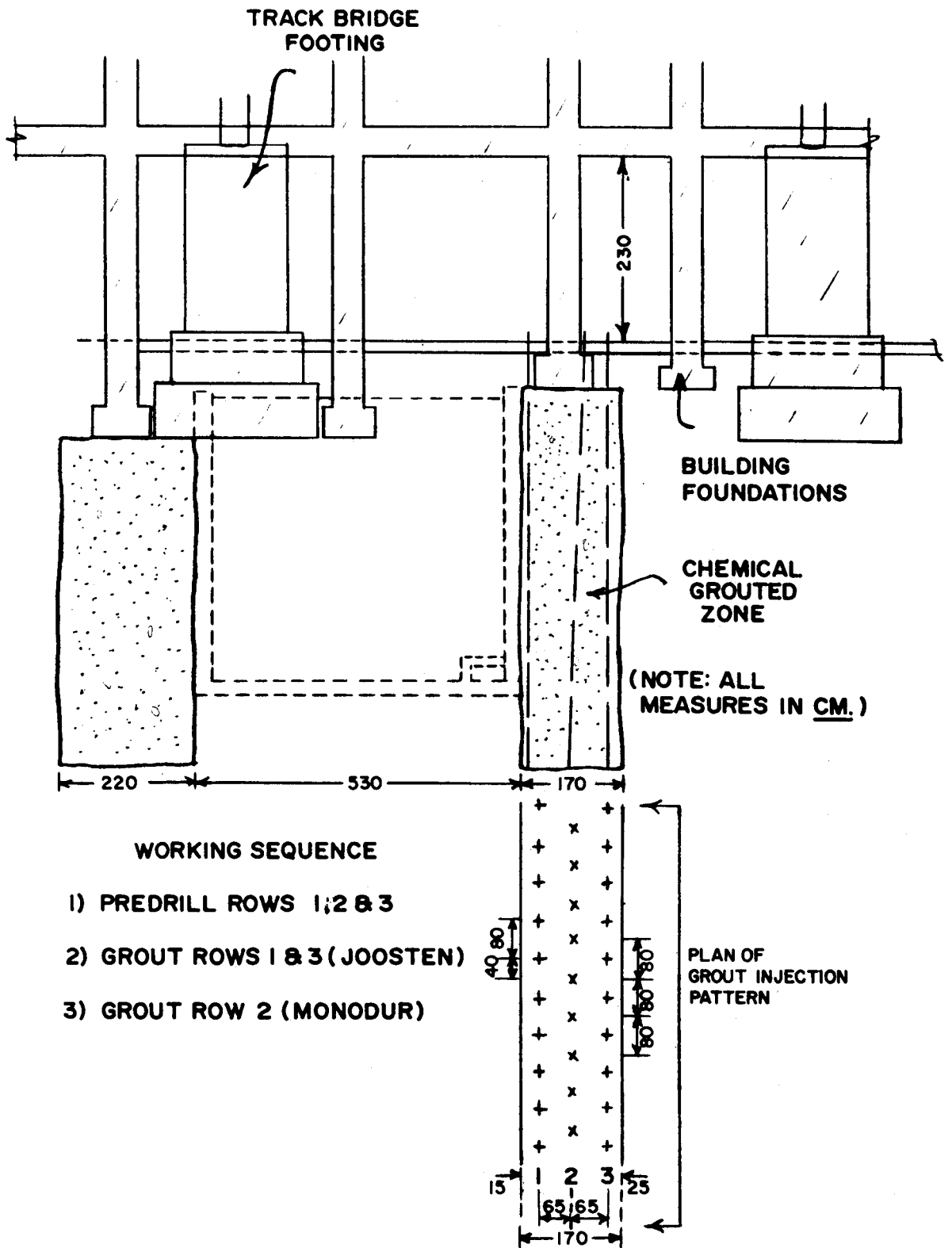


Figure 134. Lance system and working sequence for the grouting process (from Sening and Klotschke, 1970).

The use of bentonite as a grout is similar to that of clay. Unlike other clays, however, bentonite has very small particle sizes of limited size range. Therefore, its behavior is more predictable, and its ability to penetrate is superior to that of other clays. Bentonite forms a low-strength gel which is very effective in reducing permeability. It is sometimes used by itself but more often is mixed with cement, other clays, or chemicals to make the grout more suitable for a specific application.

8.31.3 Chemical Grouts

Chemical grouts are divided into groups according to their respective chemical processes, inorganic (methathetical precipitation) and organic (polymerization). Table 17 summarizes the basic types of commercial grouts available and their relevant mechanical properties.

Inorganic grouts are silica or aluminum based grouts. A great variety of these grouts exist and range from high strength, high viscosity grouts with little penetration to relatively low viscosity grouts with low strength and greater penetration. It is possible to mix these grouts with other grouts.

Organic chemistry has yielded several different grouts. High strengths can be achieved with these grouts, and in some cases it is possible to grout coarse silts. Gel times for some grouts can be set for a minute to a few hours after placement. A special installation technique using grouts of short gel time can be used to establish ground water cutoffs in the presence of flowing water.

Chemical grouts are generally combined or activated using one of the following techniques:

- a. A two-shot process in which two fluids are injected separately into the same mass. The grout sets when the fluids come into contact with each other. The classic Joosten process is an example of this.
- b. A one-shot process where a relatively low viscosity grout gradually gains strength with time and eventually forms a stiff gel.
- c. A one-shot process where the gel strength of a very low viscosity grout remains constant for a period of time (which is controlled by the mix); then the grout gels almost instantaneously.

Figure 17. Physical properties of chemical grouts (after Neelands and James, 1963).

Class	Example	Viscosity cP	Gel Time Range Min.	Specific Gravity	Special Fields		
					Water - stopping	Consolidation	
						Fine Soil	Medium Strength
Silica gel low concentration	Silicate- bicarbonate	1.5	0.1-300	1.02	X	X	
Silica gel high concentration	Silicate- formamide	4-40	5-300	1.10			X
Chrome lignin	TDM	2.5-4	5-120	1.10	X	X	
Vinyl polymer	AM-9	1.3	0.1-300	1.02	X	X	
Methylol bridge polymer	UF	6	5-300	1.08			X
Oil-based un- saturated fatty acid polymers	Polythixon FRD	10-80	25-360	0.99- 1.05			X

8.31.4 Discussion

Grouting is an art, requiring experience and judgment. In practice, the selection of grout will be governed by the type of soil, the performance required, and cost. Particulate grouts are the least expensive grouts followed by chemical solution grouts, polymer grouts, and resin grouts. There may be a factor of 2 to 3 between the cost of grouting with particulate grouts and chemical grouts. Grouts may be used in intricate combinations, and the precise design must be made by an expert. Frequently, grouts and grouting techniques are proprietary, and therefore not all grouting firms will be able to provide a particular function. Often many schemes will work, but a specific firm will only be able to use those grouts and techniques it is franchised to use.

8.32 Procedures

Methods for injecting the grouts are frequently proprietary. The basic techniques will be discussed here. The methods are similar for a one-shot or two-shot process with the only difference being that a second injection is made in the two-shot process.

8.32.1 Driven Lance

Probably the most widely used method for injection at shallow depths (10m - 12m) is the driven lance method (Dempsey and Moller, 1970). The method consists of driving a lance using a pneumatic hammer and extracting the lance by jacking. The injection may be through perforations at the end and may be done either during driving or withdrawal (or both in a two-shot process). Alternatively, a loose point may be used during driving, and upon withdrawal, injection can be made through the open end with the point remaining in place. A non-return valve may be installed to prevent influx of firm material when driving. In heterogeneous deposits, multiple injections can be accomplished by successive injections, through different lances, of grouts of successively lower viscosity. Figure 135 schematically illustrates the driven lance method.

8.32.2 Sleeved Grout Tube

The sleeved grout tube or tube-a-manchette method was introduced by Ischy and is the standard method for injecting grouts in deep or intricate grouting operations (Dempsey and Moller, 1970; Ischy and Glossop, 1962). The basic system consists of a tube, now generally of PVC, which is installed in a borehole and surrounded by a clay cement, sleeve grout which seals the tube into the ground. At

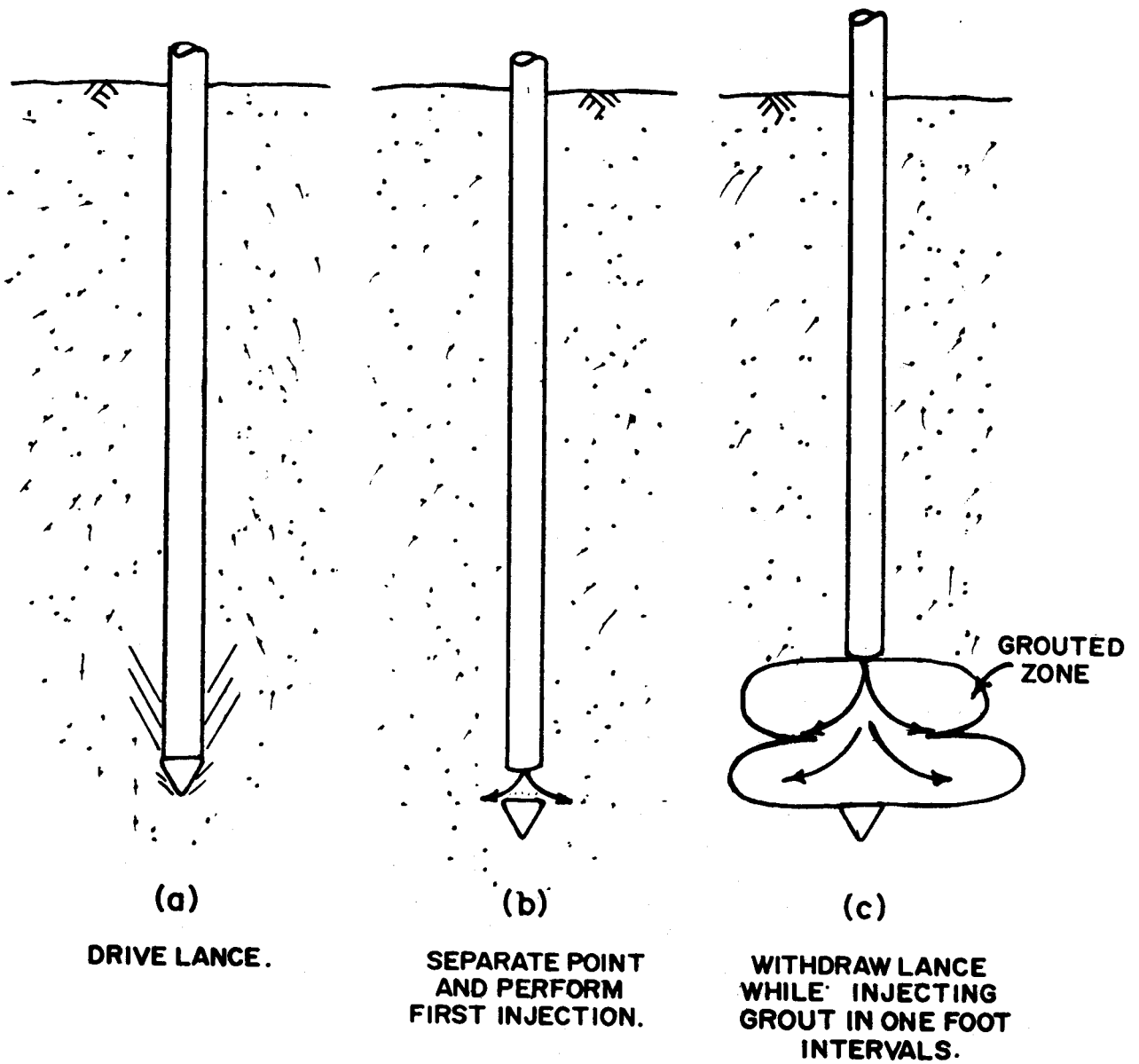


Figure 135. Schematic of driven lance method.

short intervals (approximately 300 mm) the tube is perforated, and rubber sleeves are used to cover these perforations. The grout is injected through a double packer arrangement which isolates each perforated zone. Under grout pressure the rubber sleeves are forced open, the sleeve grout ruptures, and the grout passes into the soil.

The primary advantage of this system is that multiple injections can be made from the same tube. This allows use of different grouts and better control of the grouted soil mass properties. Figure 136 shows the basic tube-a-manchette and the grouting procedures.

Other injection systems use the basic principle of the tube-a-manchette. The two most notable are the split tube method reported by Dempsey and Moller (1970) in which the grout tube is not perforated prior to installing, but rather, the tube is split with a knife edge after the sleeve grout is in place. The grout is then injected using a double packer.

Moller (1972) reported another method that uses a double packer system in which the packing is inflated by compressed air when the packer is in place. This method lends itself to greater flexibility in that the packer can be relocated without side constraints and flexible tubing can be used to work with the packer.

8.32.3 Injection Pressures

In general, injection pressures for normal grouting operations are limited to 1 psi injection pressure for each foot below ground surface. The purpose of limiting the injection pressure is to prevent fracturing of the ground. In specific instances where high confining pressures are known to exist (below heavy structures, for example), the 1 psi per foot of depth limitation may be raised.

8.32.4 Special Techniques

Vibratory Lances

Büttner (1974) reported a case in the Netherlands in which a horizontal cutoff below an excavation was placed using vibratory techniques to install the lances to the proper depths. A detachable point with a plastic pipe attached was connected to the vibrating lance. When the lance reached the required depth, the point was detached, and grout was pumped through the plastic pipe to form the horizontal cutoff. In this case the lances were installed to depths of 23 m or approximately twice the depth possible using driven lances.

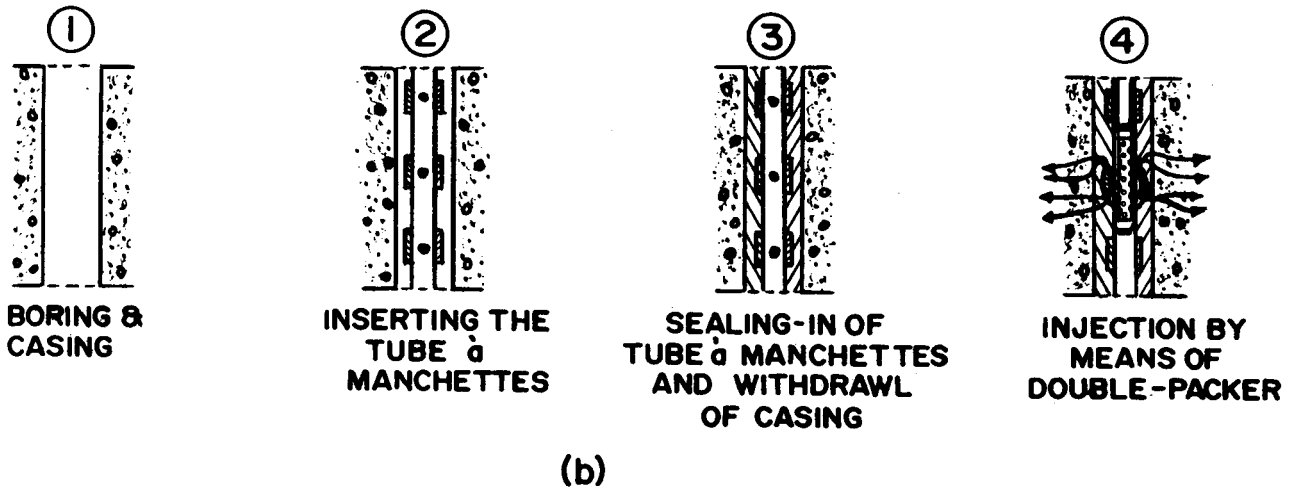
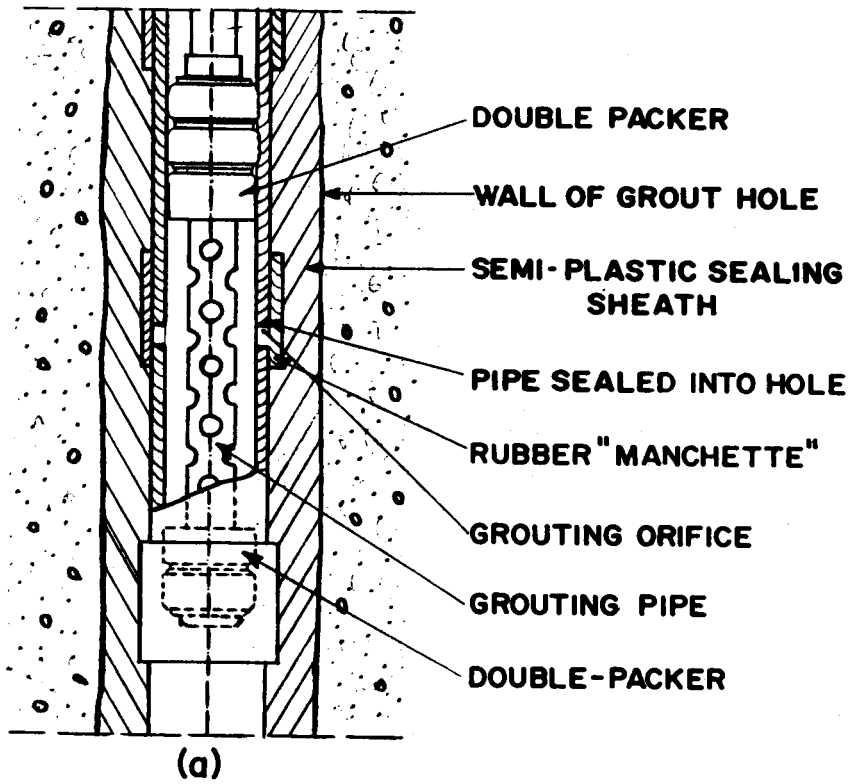


Figure 136. Sleeved grout tube (after Ischy and Glossup, 1962).

Short Gel Times

Karol (1968) reports the use of AM-9, an acrylamide grout, with a gel time less than the pumping time. Pumping of the grout continues after the initial grout has set, creating an ever-increasing size grout bulb. The mechanism controlling this behavior is still unknown; however, it has been found that it can be used to create a grouted formation in the presence of flowing ground water.

8.32.5 Compaction Grouting

Compaction grouting has limited application to tunnel construction. Rather, its use is principally restricted to the repair of damaged structures. Graf (1969) presented the basic theory and techniques of compaction grouting. Brown and Warner (1973) give a more detailed account of the process, which was updated by Warner and Brown (1974).

Compaction grouting had its beginnings in the mud jacking processes and can often be done with the same equipment. The grout used is a stiff mortar-like cement grout, which is injected under pressure at the desired location. Unlike injection grouting, the object of compaction grouting is not to penetrate voids of the soil but rather, to form a bulb of solidified material distinct from the soil. The effect is to displace and densify the soil.

8.40 FIELD TESTING AND QUALITY CONTROL

8.41 General

The success of the grouting operation is often not known until the excavation or other construction proceeds. An undetected, unsuccessful, grout installation could result in significant damage to surrounding structures before the problem is identified. It is becoming increasingly important to evaluate the quality of the grouting work after the grouting is completed but before construction begins. The extent of the field testing will depend upon the consequences of a failure of the grouting program and the ability to detect those failures during construction but before irreparable damage has occurred. This section will describe some of the methods that can be used to evaluate the in situ condition of a grouted soil mass. Halliburton Services (1976) discusses this feature of grouting in greater detail.

8.42 Ground Water Control

The basic purpose of grouting for ground water control is

to reduce the permeability of the soil mass. Several methods have been developed for evaluating the effectiveness of a grouted structure for ground water control.

8.42.1 Core Borings

This technique consists of drilling core holes into the grouted soil mass and recovering grouted soil samples. These samples can then be tested in a laboratory to determine the permeability characteristics of the soil. Since the samples are difficult to obtain and since there are no standardized procedures for testing grouted soils, this method is of limited value.

8.42.2 Pumping Tests

Pumping tests, similar to those preceding the grouting operation, can be performed. Perhaps the easiest test to perform is the test using water. The new permeability value can be compared to the permeability values calculated prior to grouting.

A slight variation of this test is to use a very low viscosity chemical grout and calculate the permeability based on the known flow and viscosity at the time of pumping. The grout will eventually gel and further reduce the permeability (Halliburton Services, 1975).

8.42.3 Flow Tests

In certain instances it may be possible to judge the effectiveness of a grouted soil mass by observing the flow through it. Two methods could be used to evaluate the grout curtain. One method is to pump on one side of the grout curtain and observe the loss of head on both sides of the curtain. Alternatively, dyes could be injected on the side of the curtain away from the pump, and the travel times observed.

8.43 Soil Stabilization

At present the methods of evaluating the effectiveness of grouting procedures to stabilize a soil mass are primitive. The only widely accepted method of determining the in situ strength is to take core borings and test the recovered samples in a laboratory. However, the same problems apply in this type of testing as in permeability testing (representativeness of sample, effects of disturbance, testing procedures).

8.50 SPECIFIC APPLICATIONS

8.51 General

Since grouting is often a special solution to a unique problem, an analysis of some of the projects that have used grouting will provide some insight into those situations that can effectively employ grouting. The specialized nature of grouting makes it impossible to say that grouting should definitely be used when a particular soil profile and project type are encountered. Grouting is simply one of the alternatives available to solve the problem and must be evaluated on the basis of economics, technical feasibility, and risk.

8.52 Soil Stabilization

The most commonly reported uses of grouts for soil stabilization have been for work associated with bored tunnels. The applications often combine underpinning and ground water control into a general stabilization function which allows tunneling to continue through loose, runny ground.

8.52.1 Auber Station, Paris

Janin and Le Sciellour (1970) report the use of grouting in connection with the construction of rapid transit tunnels in Paris including the Auber Station. The grouting was performed in a variety of granular deposits that were most economically grouted using a combination of grouts.

The Auber Station is located below Auber Street with structures located on both sides of the street and an existing subway tunnel located above the station. Figure 137 illustrates the geometry of the station and the grouting stages.

Initially, a small tunnel was constructed at the crown of the tunnel at approximately the level of the existing water table. From this gallery the side walls were grouted. Additional grouting galleries were constructed through these grouted side walls. The grouted side walls prevented water flow into the main excavation. The second grouting phase consisted of grouting a protective arch over the top of the tunnel and a grouted cutoff below the base of the station. The grouted arch above the station was installed to prevent sloughing or "running" of the ground into the excavation and thus to protect the overlying structures.

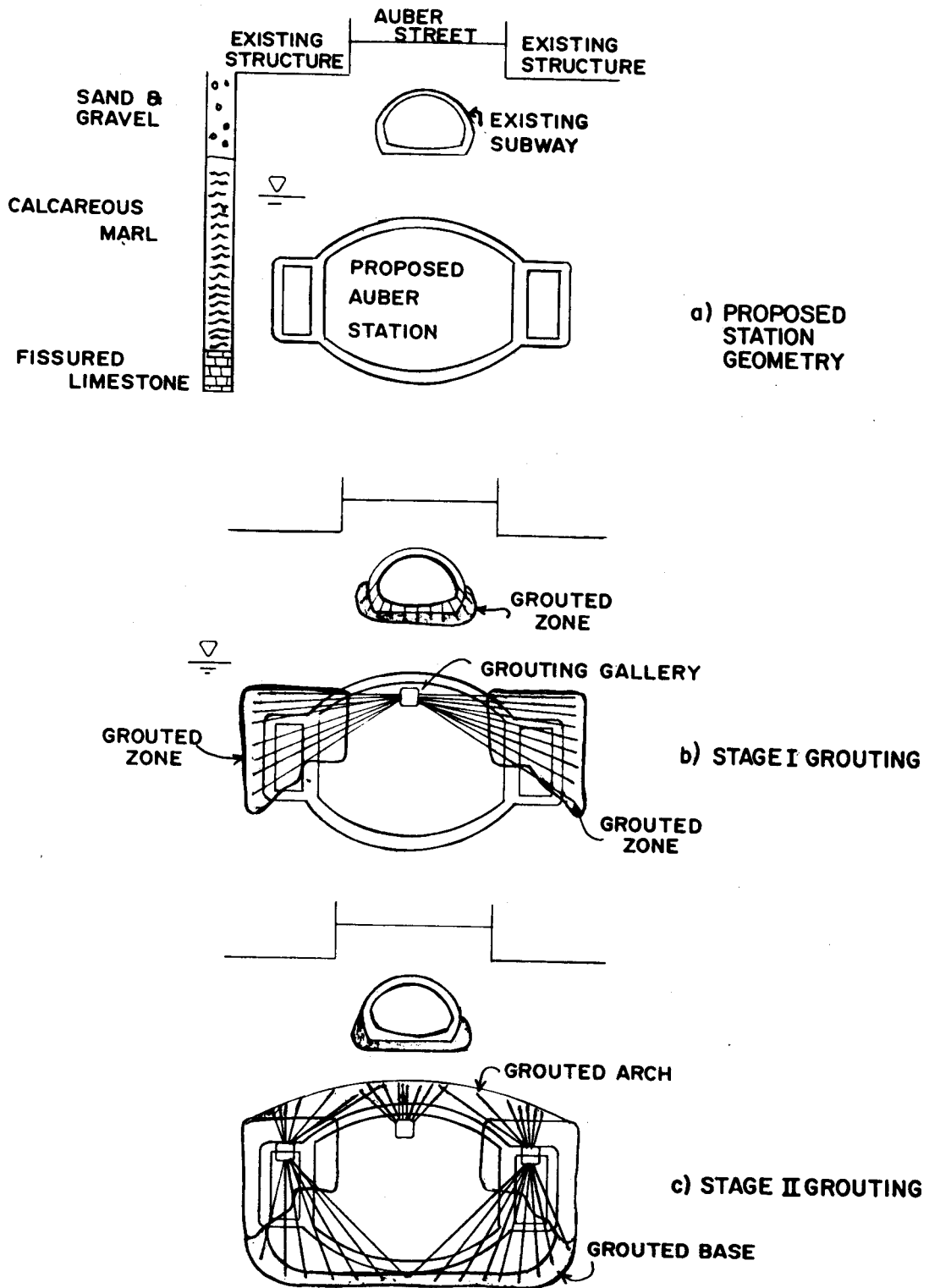


Figure 137. Grouting for Auber Station, Paris (after Janin and LeSciellour, 1970).

The grouting was performed using tube-á-manchette techniques and three basic grout types. A clay-cement grout was injected into the coarse, permeable deposits. This grout filled the larger voids. A second grout (Carongel) was used to grout the sand and gravel deposits while a phenoplastic resin grout was used in the fine sand deposits.

8.52.2 Victoria Line Extension

The construction of the Victoria Line Extension tunnel (Dempsey and Moller, 1970) to the underground railway system in London required the stabilization of water-bearing Thames Gravel. An arch around the top half of the tunnel in the gravel was grouted to stabilize the gravelly soils. This was done with the Joosten Process, and the tunnel was excavated with no noticeable movement of the gravelly soils.

8.52.3 Munich Tunnel, Roseheim Hill

In Munich, (Haffen and Janin, 1972) a section of tunnel was constructed under the River Isar and under Rosenheim Hill. An important building of historic interest also had to be protected. A pre-injection of bentonite cement was used prior to injection of the silica based gel to stabilize water-bearing sands and gravels.

8.52.4 Sewer Tunnels, Pontiac, Michigan

As reported by Halliburton Services (1975), two sewer tunnels were to be constructed under a series of railway tracks in Pontiac, Michigan. The soils consisted of very permeable soils at the upper levels and less permeable, but still groutable, soils at lower elevations. Since no disturbance to the railways could be tolerated, the 14 foot and 4 foot diameter tunnels were to be constructed after stabilizing the water-bearing soils with grout. The entire 4 foot tunnel was grouted while the 14 foot tunnel was only grouted around the periphery. Grouting of the interior of the tunnel was not required for the larger diameter tunnel. No settlement of the railroad tracks was observed.

8.53 Ground Water Control

8.53.1 Mangla Dam

Skempton and Cattin (1963) give a detailed presentation of the grouted cutoff for the closure dam at the Mangle Dam

construction site. The deposits consisted of alluvial material predominantly gravel and cobbles with 15 percent sand. The permeability of the material was determined to be 4×10^{-1} cm/sec at the top. The grout was injected using the tube-a-manchette technique. The grouts consisted of ~~Portland Cement, neutral clay, bentonite, and admixtures~~ with the mixture depending on the gradation at the point of injection. A high cement content was used in gravelly soils, and no cement was used in sandy regions. The chemicals used were sodium silicate with monosodic phosphate which acted as a gelling agent for the silicate and a deflocculating agent for the clay. By this method the permeability of the base was reduced to 5×10^{-5} cm/sec.

8.53.2 Backwater Dam, England

Geddes, et al (1972) report on a grouted curtain wall in sand and gravel ($k=10^{-3}$ cm/sec) under the Backwater Dam in England. Three different grouts were used. First a bentonite-cement grout, then a flocculated bentonite grout, and last a silicate-based grout were used. The grouting was done through a series of tube-a-manchettes, and the permeability was reduced to less than 10^{-5} cm/sec.

8.53.3 Keystone Tunnel, Alaska

Halliburton Services (1975) reports the case of a chimney of soil intersecting a rock highway tunnel in Alaska. Since water flow into the tunnel was a significant problem, it was decided to grout the soil to eliminate the flow of water and to strengthen the soil mass. The soil was grouted over a thickness of 15 to 16 feet and later observations indicated that the flow of water into the tunnel has been eliminated.

8.54 Underpinning

Grouting has been successfully used to underpin buildings adjacent to cuts, and several of these cases are reported here. In the construction of bored tunnels, grouting has often been used to stabilize the material to be mined as well as to protect structures. Conceptual applications of this technique were discussed under soil stabilization.

8.54.1 Brick Structure

Neeland and James (1963) report a case of the underpinning of an old brick structure adjacent to an excavation in water-bearing, sandy gravels. The excavation was supported using soldier piles and lagging. The grouting procedure included an initial injection of cement-clay using a driven lance followed by a TDM chromelignin

grout that was also injected with a driven lance. The grouting worked well as no damage to the building was reported.

8.54.2 Bank Excavation, Mannheim

Neumann and Wilkins (1972) report the underpinning of a structure adjacent to the excavation of a 3-story basement for a bank in Mannheim. The Joosten process and a one-shot silicate grout were used to consolidate the foundation soil which was primarily sandy. The grouted mass was tied-back using earth anchors and the face of the grouted structure was left exposed. The job was successful.

8.54.3 Walt Whitman Bridge, Philadelphia

Halliburton Services (1976) reports the case of the underpinning of the Walt Whitman Bridge in Philadelphia by Soiltech. The Broad Street Subway Extension required that a cut-and-cover tunnel be excavated near the East Pier of the bridge approach. The pier was founded on piles bearing on a fine sand and gravel layer. To protect the pier, chemical grouting of the soil in the bearing soils was specified. After injection of the grout a marked increase in the blow count was observed. Running of soils was not observed after grouting, and the cohesion of the soil was increased while the permeability was reduced. Figure 138 illustrates the grouting scheme for this case.

8.55 Discussion

The proper implementation of a grouting scheme relies in large part on the experience and ability of the grouting contractor. The many variables involved in a grouting scheme also imply a degree of risk for any such scheme. Better methods of determining the characteristics of the in situ grouted mass are required particularly when trying to evaluate the success of strength grouting.

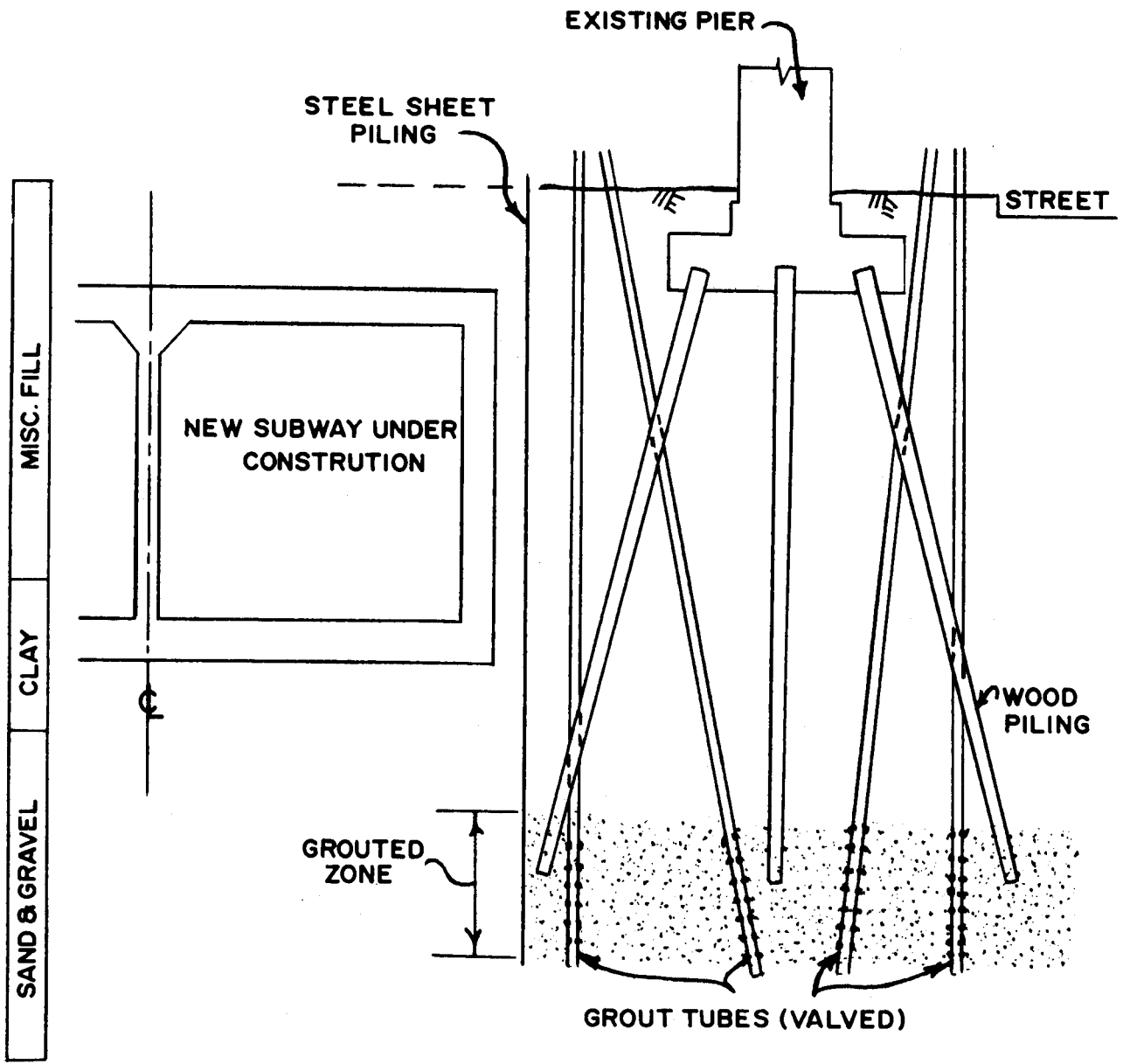


Figure 138. Underpinning of Walt Whitman Bridge, Philadelphia (from Halliburton, 1976).

CHAPTER 9 - FREEZING

9.10 INTRODUCTION

9.11 Scope

This chapter reviews and examines ground freezing as a stabilization method for use in cut-and-cover tunneling. Like grouting, ground freezing is most effectively done by specialty contractors who have technical capability to deal with engineering matters and the know-how to install and operate the equipment. Accordingly, this chapter is not intended to preempt the specialty contractor; his role is absolutely essential. Rather, the purpose of this chapter is to highlight the main issues so that practicing engineers understand the technique and are aware of factors which govern the economic and technical feasibility.

9.12 Background

The use of in situ ground freezing as a stabilization method was reported for a mine shaft excavation in South Wales in 1862 (Maishman, 1975). The process was patented in Germany by F. H. Poetsch in 1883. The basic method of circulating cooled brine through underground tubing described in the patent, known as the "Poetsch Process", remains the basic process in use today. The first reported use in the United States occurred in 1888 where a mining shaft in Louisiana was attempted by this technique (Jumikis, 1966).

Primary use and development of this method has been in the mining industry where excavation sites are selected on the basis of ore location and related factors rather than on a basis of economics and feasibility of designed excavations. A similar siting problem has now developed for other excavations, and "poor ground" becomes more common since the "good" sites have been used up in many locales.

In situ freezing for stabilization in both the mining and construction industries has been applied in two basic modes:

- a. As an emergency technique for stabilizing ground installations using traditional support methods (sheet piles, lagging, etc.).
- b. As the primary construction method of stabilizing the excavation openings.

Most use of the method by the engineering community, exclusive of mining engineers, has been as described in "a" above until recent years. However, in situ ground freezing as a primary method of stabilization in the initial design is increasingly used in the U. S.

Increased use of ground freezing for stabilization currently appears to be related to the following factors:

- a. Increasing costs of conventional construction procedures relative to the costs of ground freezing.
- b. Increased use of sites previously judged as "unsuitable".
- c. Advances in engineering technology providing new efficiencies in design and versatility of the freezing technique.

9.13 Basic Ground Freezing Process

The fundamental process in ground freezing is the removal of heat from the ground to cause lowering of subsurface temperature below the freezing point of moisture in the pore spaces. The frozen moisture acts as a cementing agent binding the soil particles together and as a structural support framework in the soil mass. Heat is removed by circulating coolants through pipes installed from the surface into the zone to be frozen, and the heat removed is transferred into the atmosphere by several different methods.

In practice, a designed pattern of freezing pipes or "probes" is emplaced in the zone to be frozen. The probes are commonly two pipes of different size, one within the other, so that the coolant can be pumped into one and extracted or allowed to escape from the other. Freezing in the soil progresses radially outward from the probes as a frozen cylinder along the length of the probe. The cylinders eventually coalesce between probes to form a wall or zone enclosing the area to be excavated with a mechanically strong and impervious barrier within the soil mass.

Closed systems, where the coolant is continuously circulated, cooled, and recirculated through the heat removal system, are the most common techniques used. Open systems are more direct. The cooling is accomplished by sublimating a solid or releasing pressurized liquefied gas to evaporate in the zone where cooling is

wanted. This permits the heat to be carried off directly to the atmosphere. A description of these techniques is provided by Sverdrup & Parcel (1973). Intermediate systems, where repressurization and re-use of the gas is done, are also possible (Maishman, 1975 and Shuster, 1972).

Photographs illustrating typical applications of ground freezing are shown in Figures 139 through 142.

9.20 DESIGN AND THEORETICAL CONSIDERATIONS

9.21 Design Parameters

Basic design parameters considered necessary for a ground freezing program include the thermal, hydrological, and mechanical properties of the soil mass to be frozen. The influence of these properties on the behavior of the soil mass must be weighed against performance criteria, cost factors, and time factors to achieve final design of the freezing plan.

9.21.1 Thermal Properties

For design of a frozen structure and the freezing program to be followed, several of the basic thermal properties of both the soil and pore water in the zone to be frozen are required. This information includes:

- a. Initial subsurface temperatures (T_0)
- b. Specific heat (C) of both the fluids and solids in the zone to be frozen, or the ratio of the amount of heat required to change the temperature of a unit mass of material one degree to the amount of heat required to raise the same mass of water one degree. Ordinarily, the approach taken is to use the term heat capacity for this quantity (they are numerically equal in the cgs system) and to consider both a mass and a volumetric heat capacity term. Mass heat capacity (C_m) is taken as reference, and for water, is 1.000 cal/gm⁻¹°C or 1.000 BTU/lb. -°F. Volumetric heat capacity (C_v) is sometimes more convenient:



Note: Wall is protected by reflective thermal insulation.

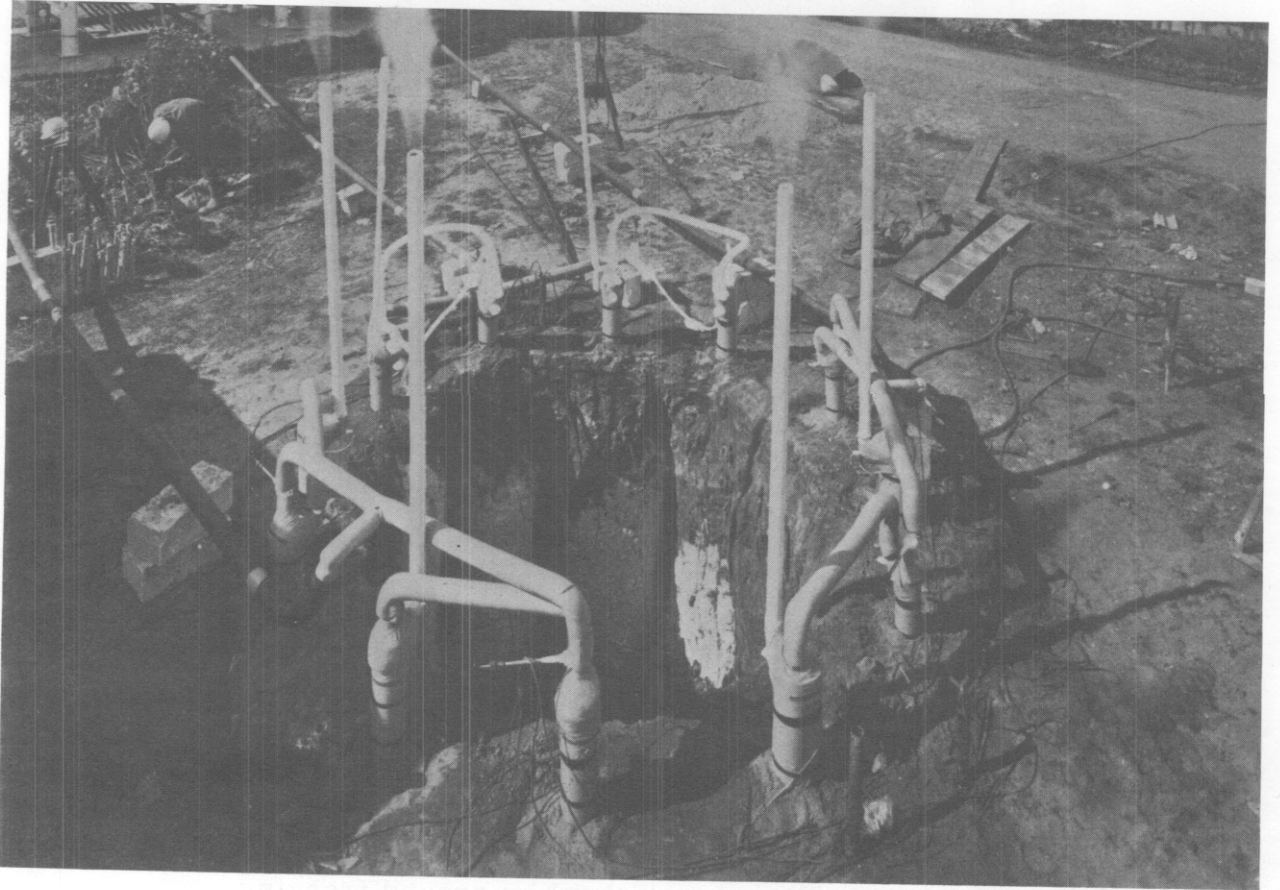
Figure 139. Aerial view of freeze wall surrounding deep excavation.

(Courtesy of Terrafreeze Corporation).



Note: Wall is protected by reflective thermal insulation.

Figure 140. Freeze wall surrounding open excavation.
(Courtesy of Terrafreeze Corporation).



Note: Stand pipes to exhaust nitrogen gas to atmosphere.

Figure 141. Small diameter shaft frozen with liquid nitrogen.
(Courtesy of Terrafreeze Corporation).

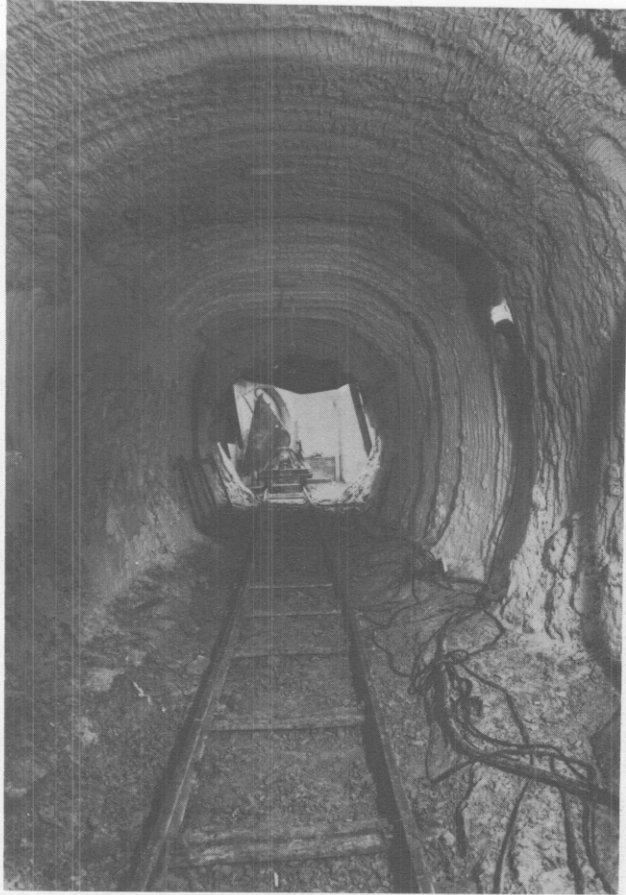


Figure 142. Tunnel stabilized by ground freezing.
(Courtesy of Terrafreeze Corporation).

$$C_v = C_m \gamma_d$$

where γ_d is the dry unit weight of the material.

Frozen and unfrozen soils have different heat capacities. Moisture content (w) (weight of water in percent of dry weight of soil) is the major factor that must be considered in calculating heat capacity. The approximate volumetric heat capacity of unfrozen soil is:

$$C_u = \gamma_d C_{ms} + w \gamma_d \frac{C_{mw}}{100}$$

and for frozen soil,

$$C_f = \gamma_d C_{ms} + \frac{w \gamma_d C_{mi}}{100}$$

where:

γ_d = dry unit weight of soil (in pounds per cubic foot, pcf)

C_{ms} = mass heat capacity of dry soil (varies with temperature). Typically about 0.2 BTU/lb-°F or 0.2 cal/gm-°C.

C_{mw} = mass heat capacity of pore water (1.0 BTU/lb-°F or 1.0 cal/gm-°C).

C_{mi} = mass heat capacity of ice (0.5 BTU/lb-°F or 0.5 cal/gm-°C).

Substituting the numerical values of C_{mw} and C_{mi} and simplifying, the volumetric heat capacity of unfrozen soil may be expressed as follows:

$$C_u = \gamma_d \left(0.2 + \frac{w}{100} \right) \text{ in BTU/ft}^3/\text{°F}$$

Similarly, the volumetric heat capacity of frozen soil is:

$$C_f = \gamma_d \left(0.2 + \frac{0.5w}{100} \right) \text{ in BTU/ft}^3/\text{°F}$$

Typical values for dry unit weight and water content of soils are given in Table 18.

Table 18. Water content and dry unit weight of typical soils.

Soil Type	Typical Values	
	w (% dry wt.)	γ_d (pcf)
Silty or clayey well-graded sand and gravel	5	140
Clean well-graded sand and gravel	8	130
Well-graded sand	10	120
Poorly-graded sand	15	110
Inorganic silt or fine sand and silt	15 - 25	110 - 85
Stiff to very stiff clay	20 - 30	95 - 80
Soft to medium clay	30 - 40	80 - 70

c. Latent heat of fusion (L) of the pore water is the amount of heat removal needed to convert the pore water to ice. This factor must be accounted for in the overall thermal requirements of the freezing program. Because latent heat is large compared to all other heat losses, it usually represents the most important factor in the freezing process. Removal of latent heat commences when the temperature of an element of soil is lowered to about 32°F. The temperature remains at approximately 32° while water is converted to ice. In fine-grained, brine-saturated, or chemically contaminated soils this phase transition occurs over a temperature range rather than at a single point. Approximately 144 BTU are required to convert one pound of water into ice (or approximately 80 cal/gm). For a body comprised of both solids and moisture, the latent heat of fusion is a function of the dry unit weight of the soil (γ_d) and the percent of water by dry weight (w):

$$L = \gamma_d 0.8w \text{ gm-cal/cm}^3 ; \gamma_d \text{ in gm/cm}^3$$

or

$$L = \gamma_d 1.44w \text{ BTU/ft}^3$$

Since the variation in dry unit weight is small, the water content is of far greater significance.

d. Thermal conductivity (K) expresses the quantity of heat transfer through a unit area in unit time under a unit thermal gradient. Typical values for soils are about 1.0 BTU/Hour-ft-°F and about 2.0 BTU/Hour-ft-°F for frozen soils. Thermal diffusivity (or temperature conductivity, α) is the quotient of conductivity and volumetric heat capacity ($\alpha = K/C$). Kersten (1949) provides a summary of thermal conductivities for typical frozen and unfrozen soils.

9.21.2 Hydrologic Properties

Hydrologic properties are interrelated with the bulk thermal properties and have a very strong influence on the final design. The most important hydrologic considerations include:

1. Moisture content.
2. Subsurface flow rates and direction of flow.
3. Permeability of the soil.
4. Pore water chemistry (i. e., brine, unusual composition).

9.21.3 Mechanical Properties

General

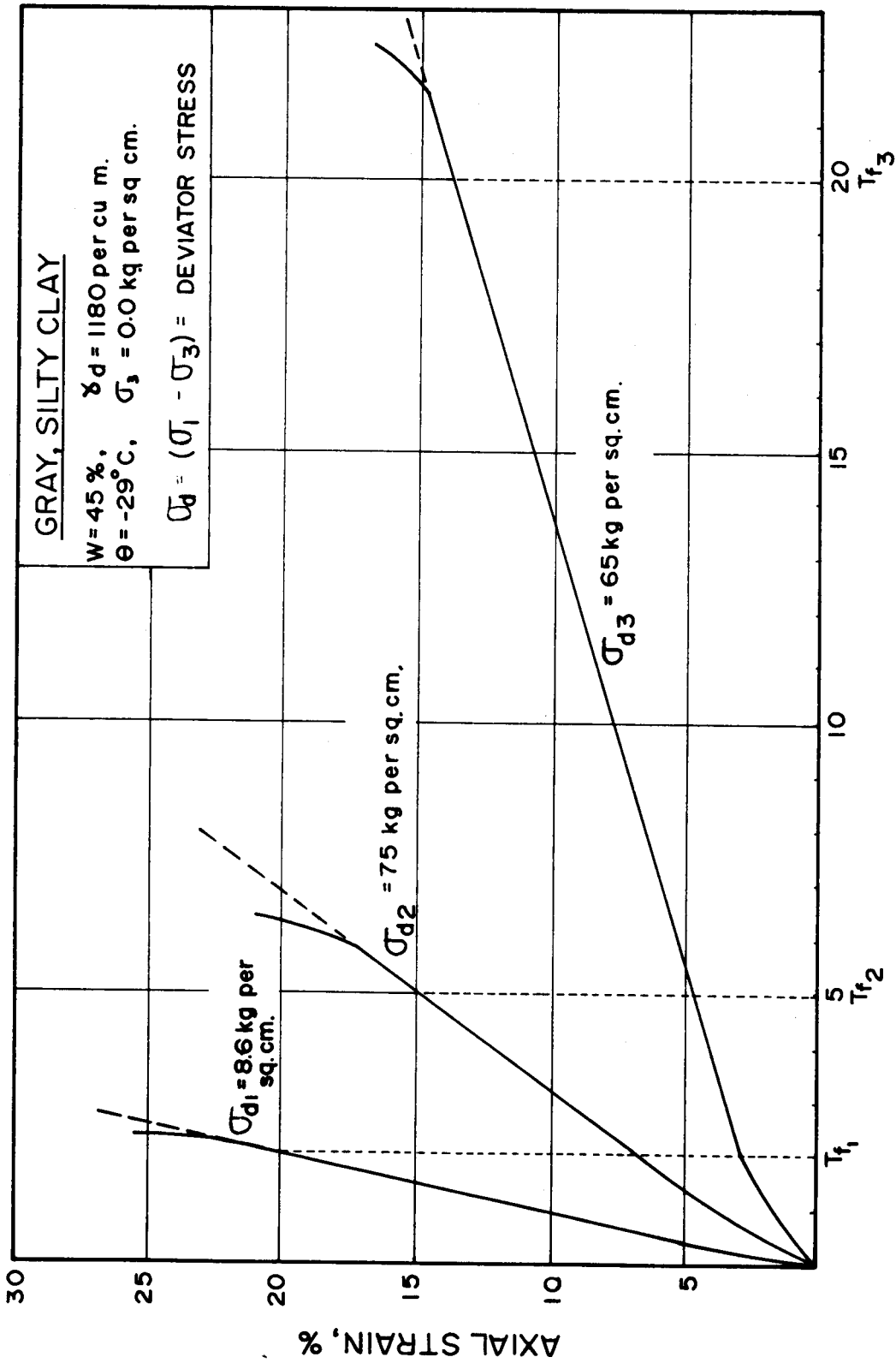
A frozen soil mass is a visco-plastic material in that it will creep under stress application. Normally the creep rate, rather than ultimate strength, will control the design. The latter, however, is a useful index parameter in assessing creep.

Tests are usually made to determine actual mechanical behavior using laboratory samples frozen to the temperatures expected in the field. The laboratory data can be correlated to field performance using empirical correlations to past performance. Critical problems ordinarily arise when heterogeneous deposits are encountered and the true in situ conditions are not represented in the laboratory investigations. In situ pressuremeter tests have been used to assess deformation characteristics of soil after freezing. This may also involve test sections prior to initiating the field program (Shuster, 1975).

Creep

Since the behavior of frozen soil is visco-plastic, its behavior with time is dependent upon stress level and temperature. Creep will increase with applied stress and will decrease with temperature below freezing. Typical behavior patterns are shown in Figures 143 and 144. Figure 143 shows the effect of increasing compressive stress on axial strain. Figure 144 shows strain increase with both higher stress and higher temperature. Stress is held constant for each of the three curves.

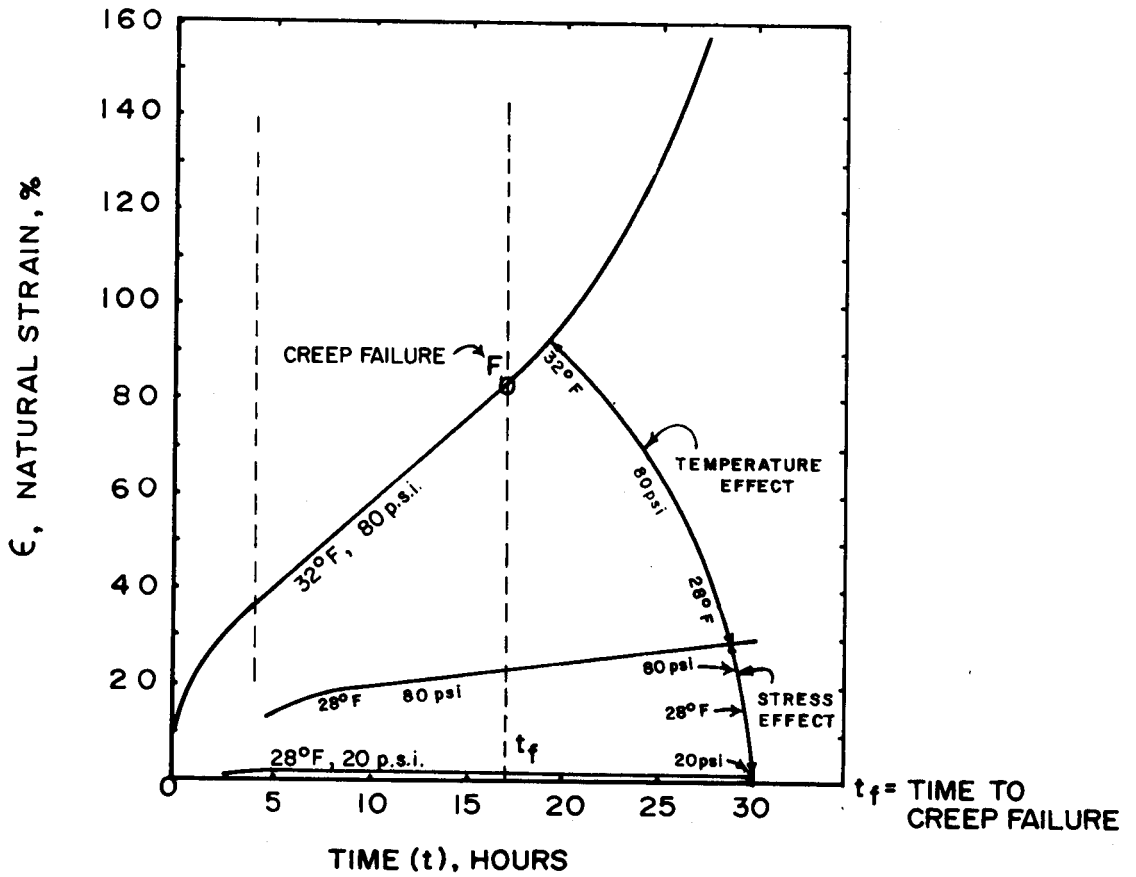
Point "F" in Figure 144 represents the line at which the rate of strain becomes increasingly greater with time. Sanger (1968) refers to this as creep failure.



DURATION OF LOADING (T), HOURS

(AFTER SHUSTER, 1972)

Figure 143. Strain versus time and loading for a frozen soil.



(AFTER SANGER 1968)

Figure 144. Creep curves for an organic silty clay with temperature influences.

Creep tests, such as those shown in Figure 144 are carried out under constant stress and temperature while measuring strain.

In Figure 145, the reciprocal of stress and corresponding time to creep failure are both plotted on families of curves of various temperatures. Note that as the temperature increases, the reciprocal of stress also increases. Also, the time to creep failure becomes exponentially longer as the reciprocal of stress increases. (Note, plot is of reciprocal of stress and therefore an increase of the reciprocal represents a lower stress value).

In any given installation, the designer must be assured that actual stress levels are safely below values that would produce excessive creep over the duration of the project.

Ultimate Strength

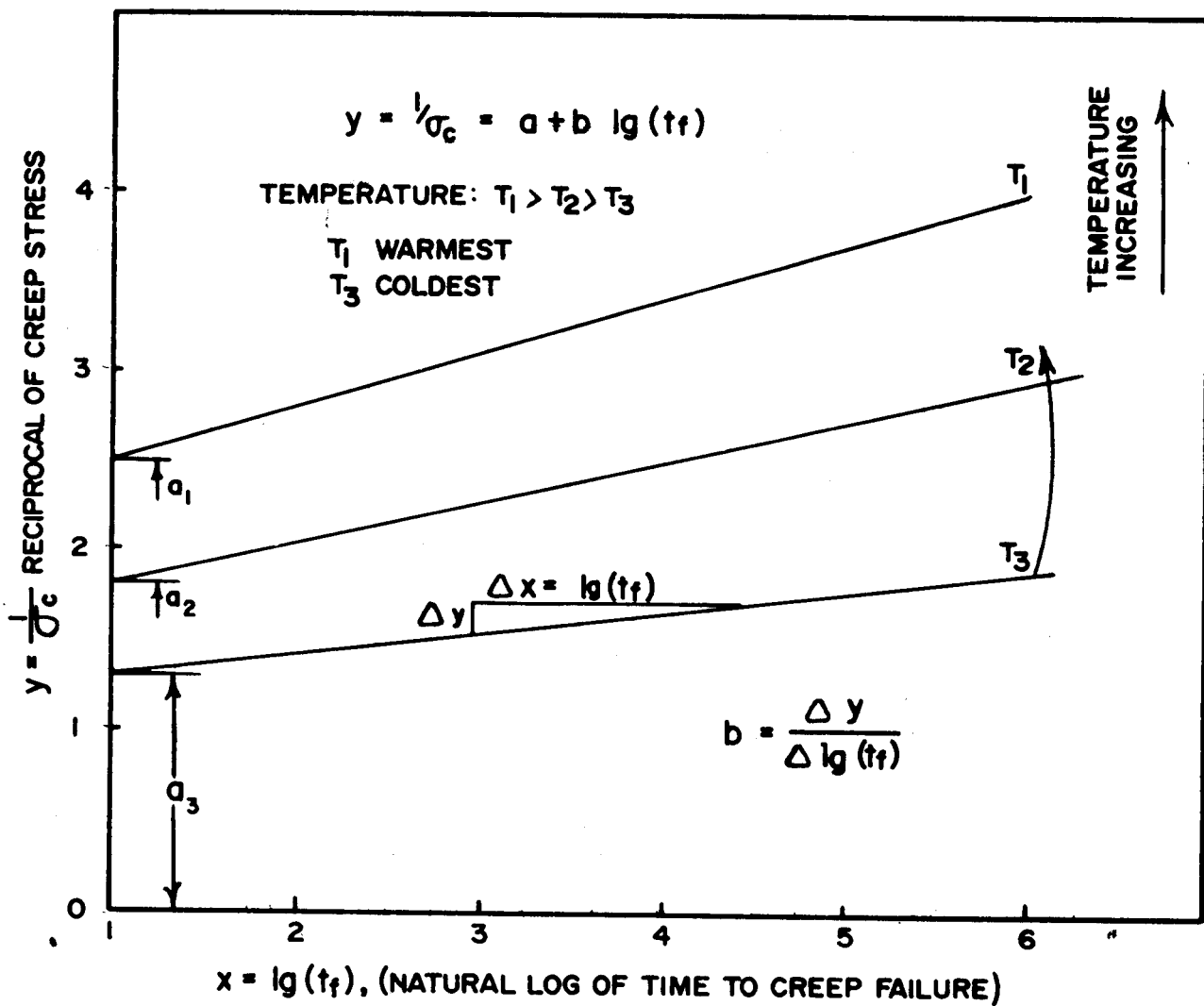
One standard method of judging the engineering suitability of materials is to measure the ultimate compressive strength. Jumikis, after Tsytoich (1960), presents a summary of ultimate compressive strengths of common soils as a function of temperature below the freezing point of water (Figure 146).

The figure shows that sandy soils have greater strengths than clayey soils. Porous, granular soils attain the greatest strengths through freezing since virtually all of the pore water is frozen. As the clay content of the soil increases, however, ultimate and shear strengths decrease, partially because the water in this clay may not be completely frozen and the total volume of interconnected water-filled pores decreases. Ice in interconnected pores provides a structural framework as well as a new element of strength in previously water-filled voids.

The strength of frozen granular soil at a given temperature increases as the moisture content increases. See Figure 147 showing ultimate compressive strength increase of sand. The figure also shows that the strength of a clay does not increase with moisture content.

9.21.4 Geometry and Capacity of the Freezing System

Cost and time factors for ground freezing programs are strongly influenced by both the geometric arrangement of



(AFTER SANGER, 1968)

Figure 145. Influence of stress and temperature on time of creep failure.

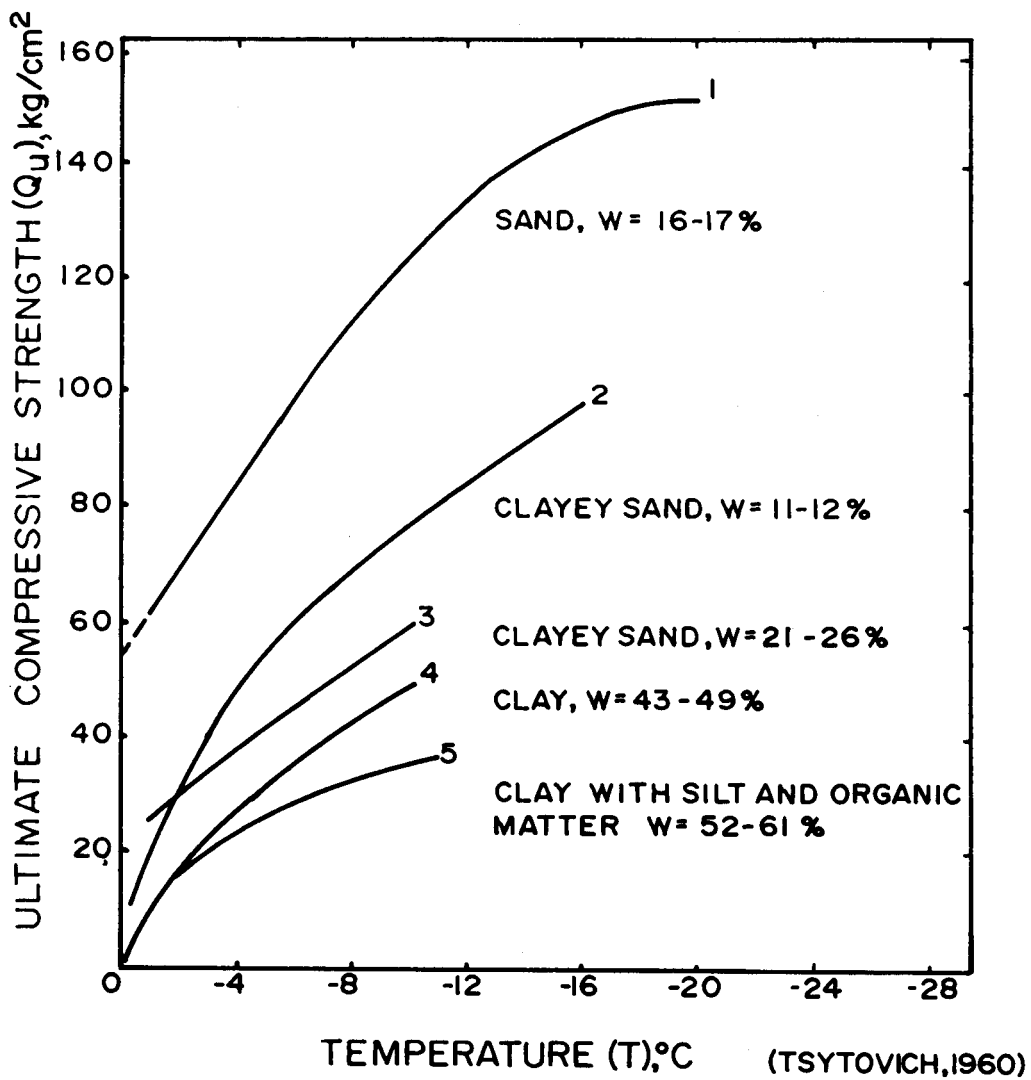


Figure 146. Ultimate short term compressive strength as a function of temperature.

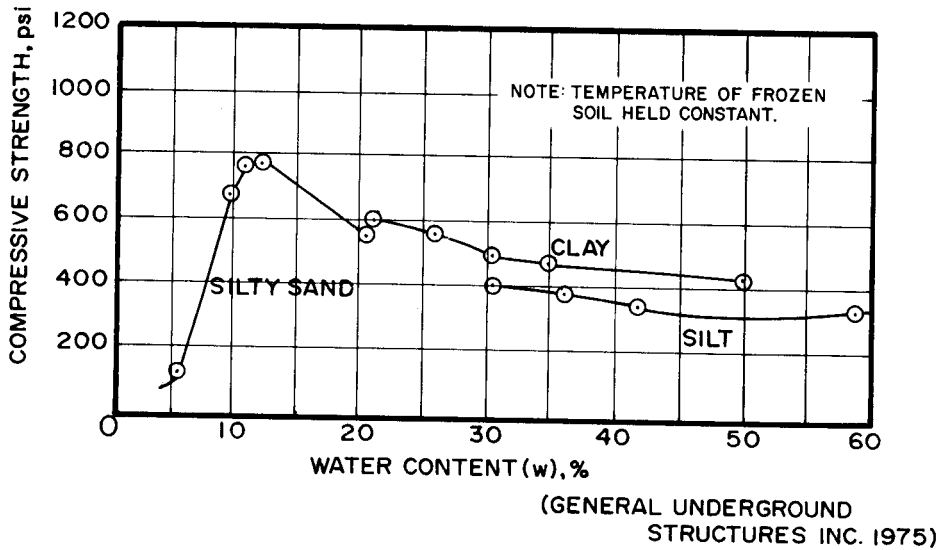
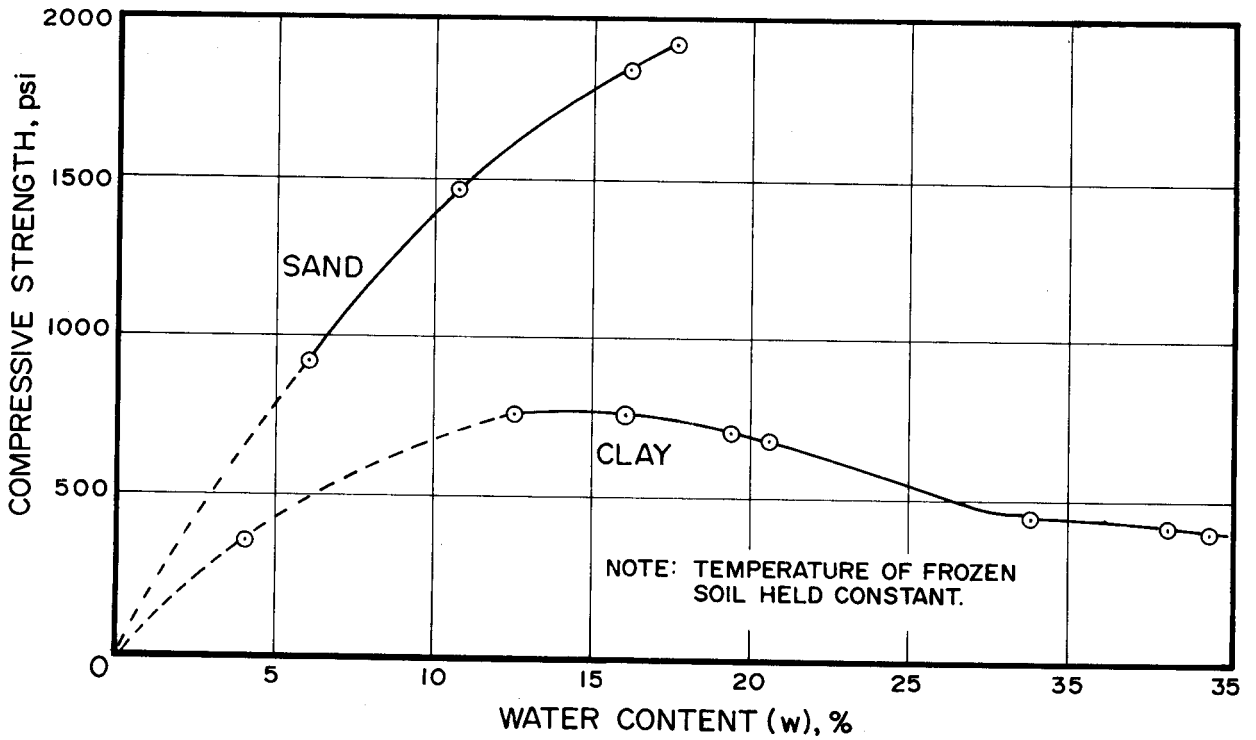


Figure 147. Ultimate short term compressive strength of ground vs. moisture content.

freezing probes and the capacity of the refrigeration equipment. The ground freezing process proceeds radially outward from each of the freezing probes, and the rate of progress is a function of:

1. The capacity of the equipment relative to the thermal load of all of the combined probes and surface piping.
2. The thermal gradient between the probe and surrounding materials.
3. The rate of heat transfer between the probe-frozen ground system and the unfrozen soil mass.
4. Fringe losses at the freezing front due to lateral ground water flow.

In the design process, increased freezing rates can be obtained by decreasing freeze element spacing and/or increasing the temperature differential by increasing the capacity of the cooling equipment.

Fringe losses are reduced as the radial freezing fronts converge between probes since both the frontal areas between frozen and unfrozen masses are reduced and thermal losses due to ground water movements through the freezing mass are effectively blocked.

9.22 Approaches to Design

9.22.1 Thermal Considerations

Fundamentals

Several approaches are available to the problem of determining the amount of refrigeration capacity required. The approaches all must consider two basic phases of operation including (1) reducing the temperature of the soil mass to a level where the required frozen ground behavior will be obtained, and (2) maintaining all or some part of the frozen mass at a temperature where the mass will behave in a satisfactory and predictable way during construction activities. Theoretical analyses may be undertaken by several methods. However, all methods are fundamentally an exercise in heat transfer from the ground to the atmosphere.

A rigorous solution would require an analysis of heat flow under thermal gradients in frozen and unfrozen zones. The rate of heat flow is a time dependent variable which is initially high under steep thermal gradient but becomes less with time as the gradient becomes flat. Figure 148 shows thermal gradients and sources of heat losses.

At any given instant of time, t , continuity requires that the total rate of heat flow from the ground be:

$$\Sigma q = q_f + q_L + q_u$$

where:

Σq = total rate of heat flow

q_f = rate of heat flow from frozen zone

q_u = rate of heat flow from unfrozen zone

q_L = rate of heat flow due to latent heat of soil element maintained at the freezing point.

All in heat units per unit of time (BTU/hr or cal/sec).

In time interval $\Delta t = t_2 - t_1$, the ice front advances from distance z_1 to distance z_2 and the thermal gradient changes from that shown by T_1 to that shown by T_2 . Additional heat is removed from the ground during this period by lowering of temperatures from T_1 to T_2 and by removal of latent heat. Note that the thermal gradient has decreased during time interval Δt . This is most evident in the frozen zone. Thus the rate of total heat flow, Σq , has also decreased.

This incremental heat loss during time Δt is:

$$\Delta Q = \Delta Q_u + \Delta Q_L + \Delta Q_f$$

where:

ΔQ = Incremental total heat loss

ΔQ_f and ΔQ_u = Heat loss required to drop temperature from T_1 to T_2 in frozen and unfrozen zones, respectively.

ΔQ_L = Incremental latent heat loss

(all heat quantities above in units of BTU or calories)

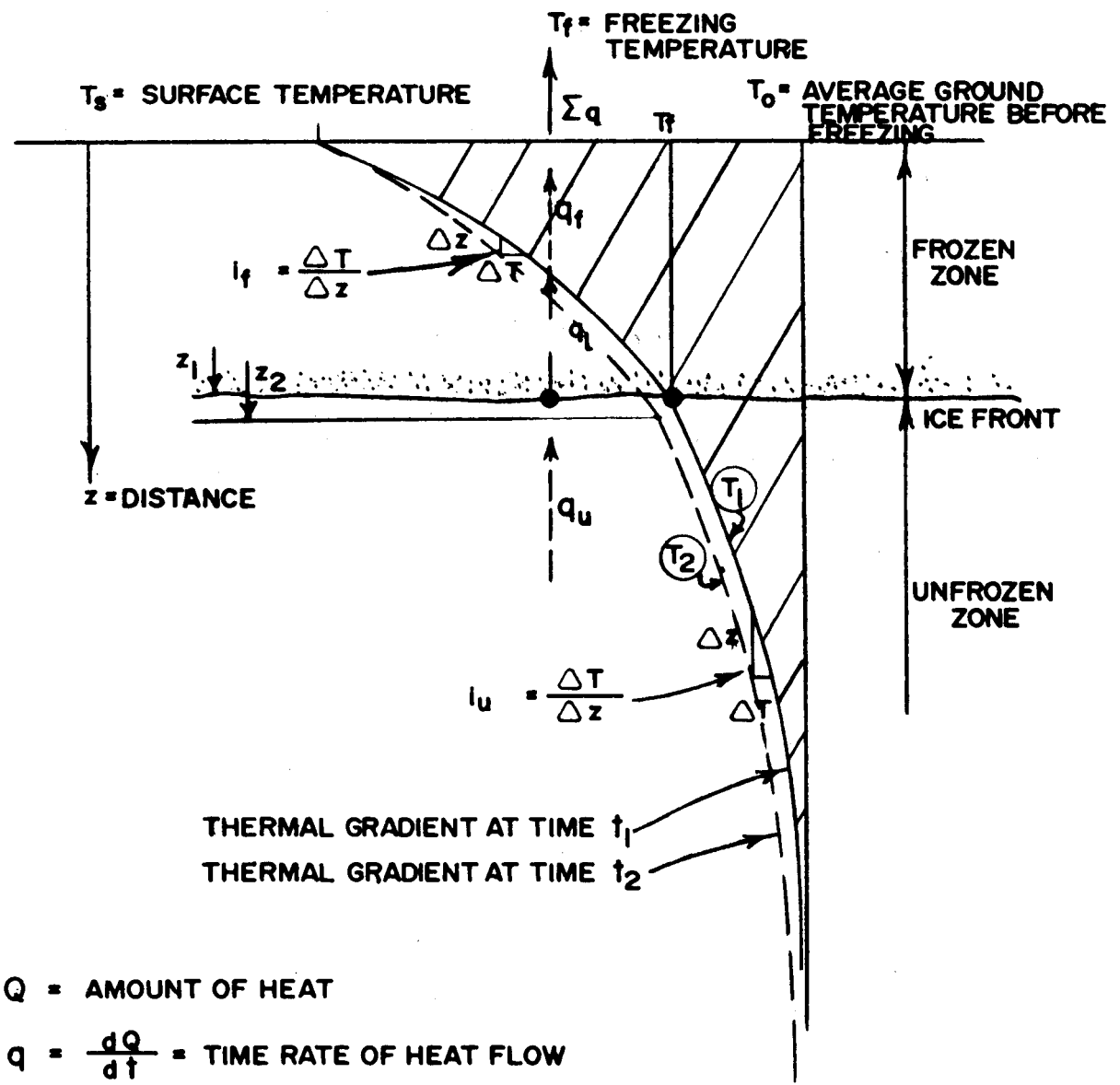


Figure 148. Heat flow under thermal gradient.

Rigorous Solution

Rigorous solutions are complex because the thermal gradient is changing with time. Sanger (1968) presents a discussion of the Russian procedures. These procedures result in a closed form solution of the energy removal and duration of time required to freeze a zone of specified size. The mathematical operation is complicated and the multitude of design variables makes this type solution cumbersome for all but the simplest cases. Computer models can be made using finite element techniques but this is quite costly. Even computerized modeling has significant limitations when it is necessary to design a freezing plan in heterogeneous deposits.

Simplified Solution

The basic approach to simplify the analyses is to (1) identify the zone to be frozen, (2) establish existing temperatures and temperatures after freezing, and (3) compute the amount of heat loss required to transfer the volume of soil in the zone from the existing condition to the frozen condition. This simplification neglects temperature drops (and therefore heat loss) at distances beyond the ice fronts. However, for practical applications the heat loss within the frozen zone is large compared to heat losses beyond the frozen zone.

The total heat losses that occur within the frozen zone are:

Q_u = Heat flow from soil, solids, and pore water required to drop temperature from initial soil temperature T_o to the freezing T_f .

Q_L = Latent heat flow to transfer from water to ice (occurs at constant temperature of T_f).

Q_f = Heat flow from soil, solids, and pore water required to drop temperature from freezing point, T_f , to the design subsurface temperature T_2 .
Therefore, the total heat loss from a unit volume of soil is:

$$Q_u = C_u (T_o - T_f)$$

$$Q_L = \gamma_d (1.44)w$$

$$Q_f = C_f (T_f - T_2)$$

where:

T_o = Initial ground temperature (usually mean annual temperature).

T_f = Freezing temperature.

T_2 = Final temperature.

C_u and C_f are as previously defined, heat required to drop temperature one degree per unit volume (volumetric heat of frozen and unfrozen soil).

Gail (1972) describes a design method for freezing ground based solely on the energy required to freeze a given body of soil. This method was used by engineers before modern heat transfer technology made much more accurate computations possible. The technique consists of assuming a value for the specific heat of the material to be frozen and a latent heat of fusion, then determining the amount of energy required to lower the temperature of the body of soil to the desired temperature. An empirical relationship based upon the amount of required energy, geometry of the design structure, and thermal conductivity of the soil mass provides the spacing of freezing elements, diameter of elements, and refrigeration capacity in this approach. The technique also requires that an allowance of not less than 100% of the initial calculated energy be included in the design to account for thermal fringe losses.

Typically the latent heat is large compared to the volumetric heat associated with temperature drop. Consider, for example, a saturated soil with water content, w , of 25% and dry unit weight, γ_d , of 105 pcf.

$$C_u = \gamma_d \left(.2 + \frac{w}{100} \right) = 105 (.45) = 47 \text{ BTU/ft}^3/\text{°F}$$

$$C_f = \gamma_d \left(.2 + \frac{.5w}{100} \right) = 105 (.325) = 34 \text{ BTU/ft}^3/\text{°F}$$

$$L = \gamma_d (144) \frac{w}{100} = 105 (1.44) (25) = 3800 \text{ BTU/ft}^3/\text{°F}$$

Assume an initial ground temperature (T_o) of 50°F and final average design temperature (T_2) of 10°F. Then,

$$Q_u = 47 (50 - 32) = 840 \text{ BTU/ft}^3$$

$$Q_L = 3800 \text{ BTU/ft}^3$$

$$Q_f = 34 (32 - 10) = 750 \text{ BTU/ft}^3$$

This simple example illustrates the overriding importance of latent heat relative to volumetric heat.

Ground Water Movement

Hashemi and Sliepcevich (1973) have developed a rigorous approach to evaluate the influence of ground water movement on the freezing process. Assumptions are made that the soil is homogeneous, that the latent heat of fusion is much greater than the specific heat (heat removal to further lower temperatures) of the frozen soil, and that the temperature varies only with time and radial position. These assumptions make it possible to develop a closed solution, but for a field application of multiple rows of closely spaced freeze pipes or for temperatures below -40°C the solutions cannot be applied.

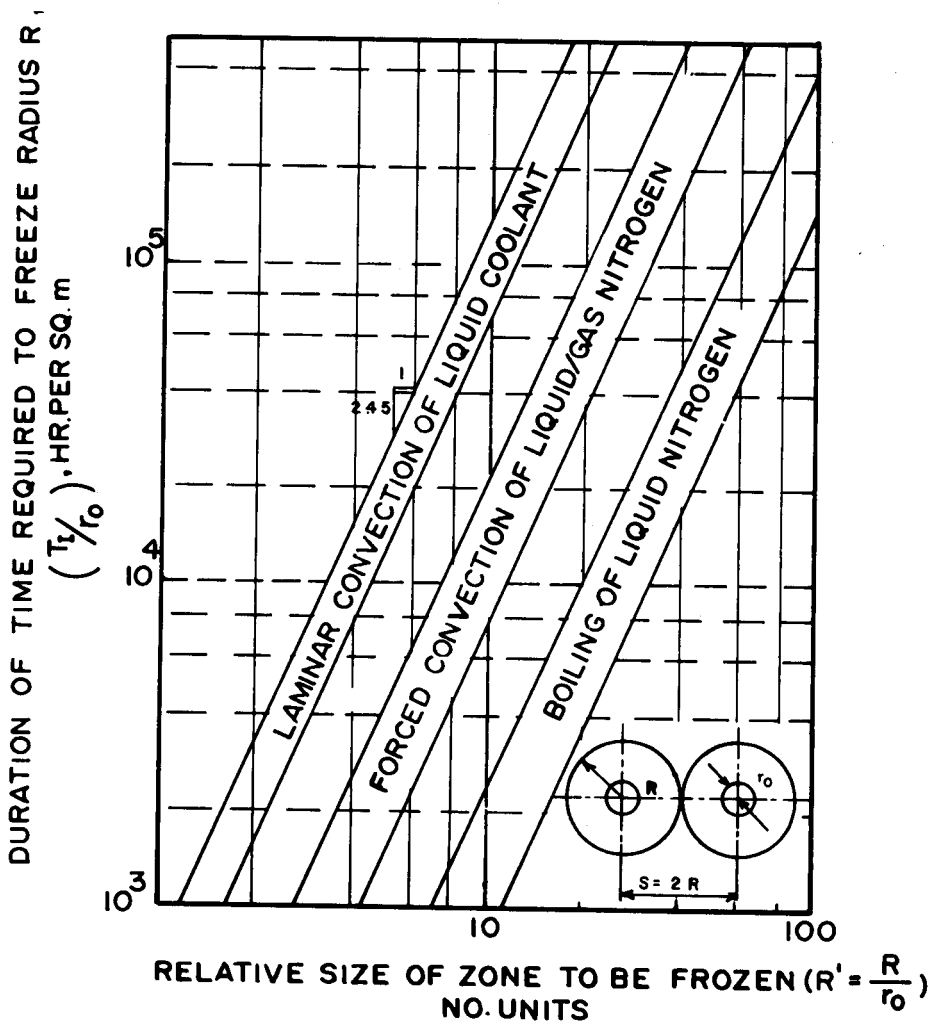
Discussion

In practice, the simplifications of the Gail technique lead to a conservative estimate of energy requirements. The same observation is true for a purely theoretical solution. Shuster (1972) presents an outline of the basic design considerations, illustrating empirically supported theoretical correlations between various parameters. The theory upon which these correlations is based was originally developed by Kamenskii (1971) for freezing with air convection.

Figure 149 compares typical freezing times for various coolants (Laminar Liquid Coolant is about -15°C to -40°C , and Boiling Liquid Nitrogen about -175°C to -190°C). The R' factor shown in Figure 149 illustrates the important effect of freezing element spacing. Figure 150 illustrates the effect of ground water flow on freezing time. The time for freezing decreases with decreasing temperature of coolant, decreasing spacing of elements, and decreasing flow of ground water. Shuster emphasizes the important point that the amount of energy and time required is governed chiefly by the latent heat of fusion of the pore water.

Synopsis

The state-of-the-art of thermal design is refined to the point where reasonably accurate theoretical solutions to the thermal requirements for freezing design are available for simple geometries in homogeneous soil. The theoretical energy requirements can be calculated for a proposed freezing application rather simply, but to bring the design to a workable, economical, and safe field operation still requires extensive use of empirical knowledge developed

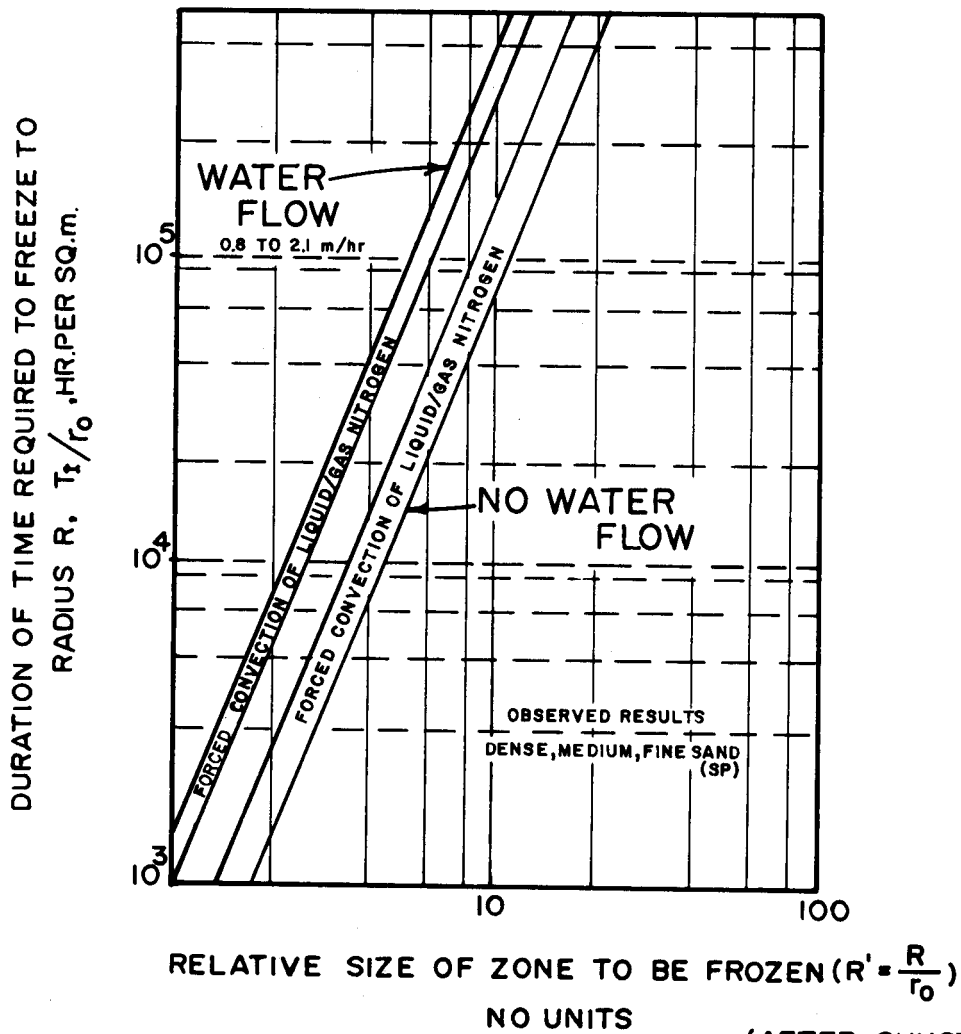


(AFTER SHUSTER 1972)

NOTES:

1. Indicated bands represent normal range of observed field and laboratory results; however, results with forced convection of N_2 may vary more widely than indicated due to variables in control of the freezing process.
2. R, r_0 in meters.
3. T_I = time of active freezing (hrs).

Figure 149. Determination of required freeze time
(as affected by coolant types).



NOTES:

1. Indicated bands represent normal range of observed field and laboratory results; however, results with forced convection of N_2 may vary more widely than indicated due to variables in control of the freezing process.
2. R , r_0 in meters.
3. T_1 = time of active freezing (hrs).

Figure 150. Duration of freeze time (as affected by ground water flow).

through the experience of past applications. This is especially true under complicated stratigraphic and site boundary conditions.

9.22.2 Mechanical Considerations

Structural design with frozen ground must consider the viscoplastic time and temperature dependent behavior of the material as described in Section 9.21.3. The appropriate creep theories and related laboratory test equipment were not available prior to the early 1960's. Because of this, earlier designers accounted for the creep of frozen ground by the use of elastic analysis and arbitrarily reduced values of the short term ultimate compressive or shear strength of the material. Figures 146 and 147 give some typical data on short term strength illustrating the combined effects of temperature and material type.

Arbitrarily reducing ultimate strength (perhaps by a factor from 2 to 10), without adequate understanding of the true rheological behavior of the materials is just as likely to produce unsafe as overconservative designs (Shuster, 1975).

Vialov (1962) developed creep models for the analysis of circular shafts in frozen ground, but there are still no closed form models for the majority of problems. The designer must either use finite element analyses with a time and temperature dependent modulus or he must use elastic analyses with material properties selected on the basis of their creep behavior. The latter procedure is conservative and is most commonly used today.

For circular shafts, Sanger (1968), as well as Jessberger and Nussbaumer (1973), provides simple analytic procedures. For shallow circular tunnels the procedures are more complex; Richards and Agrawal (1974) and Butterfield (1970) among other provide some guidance in the matter. There are no simple closed form elastic models for elliptical shafts or tunnels.

Open surface excavations with frozen walls are normally designed as simple massive gravity structures or cantilevered beams (depths typically less than 20 feet). Where possible, arching action is used to minimize the thickness of frozen earth required. A basic procedure for this type of elastic arch analysis is given by Davis (1952).

9.22.3 Ground Movement Considerations

Frost action beneath unconfined pavement has no direct correlation with confined movements during ground freezing,

but a brief review of the literature provides some insight into the nature of the problem.

Mitchell (1968) presents a state-of-the-art review of frost heaves and related problems. Sanger concludes that most but not all frost heave noticed on highways is caused by water migration from the unfrozen region toward the freezing plane in relatively fine-grained soil. Heaving is caused by the excess water freezing into layers of segregated ice, oriented at right angles to the direction of heat flow.

Linell, et al (1963) prepared an extensive tabulation of various soil types with frost susceptibility classification based upon their tendency for ice lens segregation and rate of heave. An old rule of thumb is that soils having more than 3 percent by weight finer than the 0.02 mm size are frost susceptible below pavement.

The Corps of Engineers criteria have been developed for nonsaturated soils in which the frost heave is associated with ice lens segregation. The primary mechanism of lens growth is by capillary migration of pore water to the ice lens. Clean, free-draining soils have insufficient fines to develop capillarity, and therefore are not frost susceptible. The frost susceptibility of silty or clayey sands and gravels generally increases with the percentage passing 0.02 mm. The most frost susceptible soils are silts, clayey silts, and sandy clays.

The direction of heat flow in a ground freezing system with vertical pipes is also perpendicular to the direction of the surface freezing, but the geometries of groundwater, stratigraphic sequence, capillarity, and permeability relative to the freeze surface are very different from the general frost heave model below pavement.

In free-draining, non-frost susceptible soils frost heave is not typically a problem because (1) excess water is expelled along the freeze front during the freezing process, and (2) even if the freezing front were essentially stagnant, ice lenses could not develop. Thus, there is no possibility for ice lens segregation or volumetric expansion in either partially saturated or non-saturated free draining soils.

In soils that are not well drained, heave is generally attributed to two separate phenomena. The first phenomenon is the expansion of pore water owing to the change of state from liquid to solid. Water expands a maximum of 9 percent in volume during

this process, and the maximum possible heave is therefore 9 percent of the pore volume if the soil is saturated and there is no drainage. The second phenomenon is frost expansion due to pore water migration and ice segregation with time at the freezing front or within the frozen zone in partially saturated soils. The first phenomenon occurs simultaneously at the freeze point, and the second continues after the earth mass is partially frozen. Neither phenomenon will produce ground movement if the confining pressure is greater than the pressure developed by the freezing soil-water system.

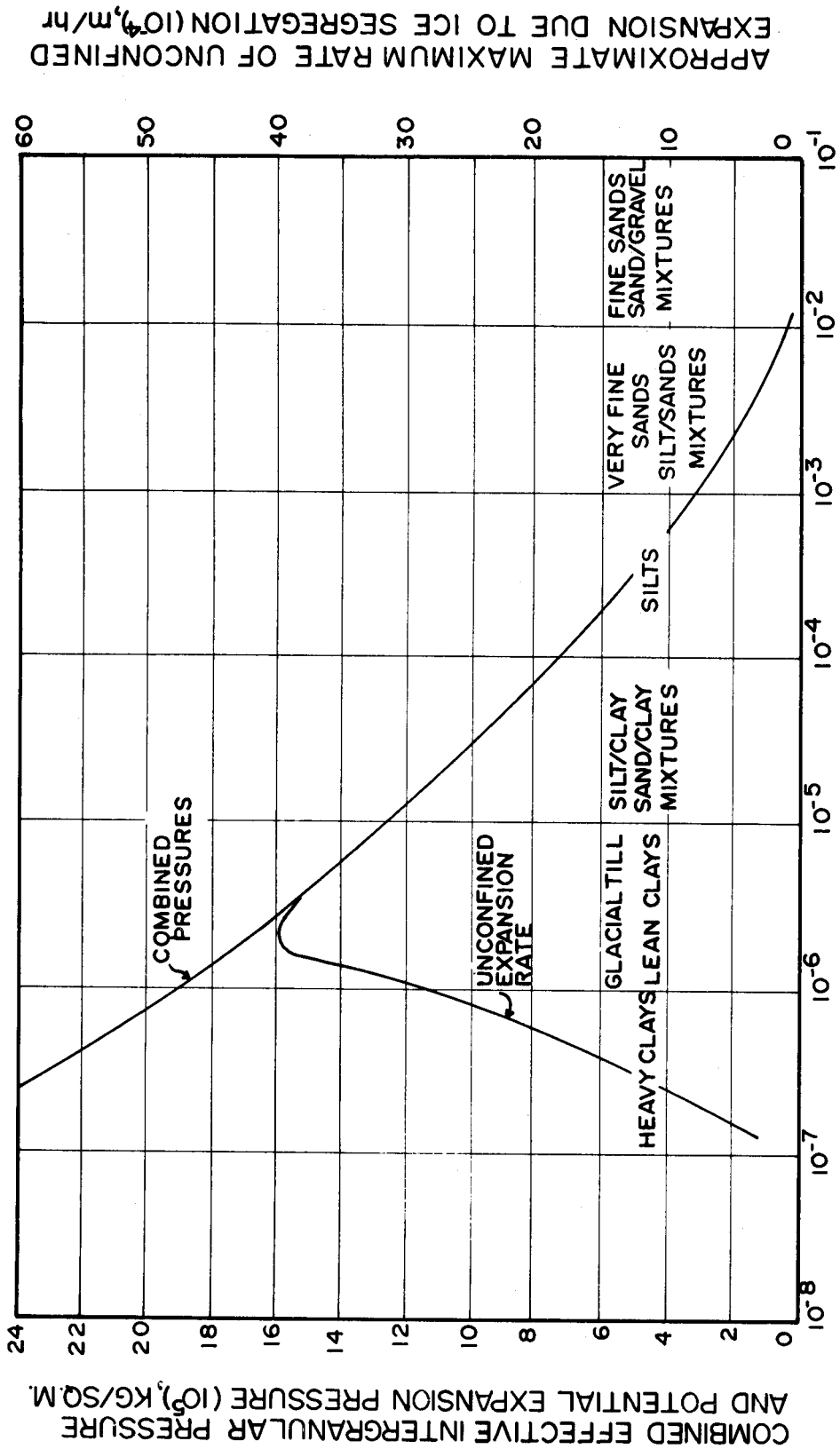
The occurrence and rate of vertical heave depends not only upon the pressures exerted by the water-ice expansion but upon the overburden pressures resisting expansion and the permeability of the freezing material. Figure 151 shows the combined expansion effects for various soil types (unconfined by overburden) assuming that water is available throughout the freezing process. From the figure, it is evident that potential expansion is much greater as clay content increases (or as permeability decreases). These potential expansion pressures must overcome overburden pressures before frost heave occurs. The unconfined expansion rate in heavy clays decreases from the general trend because of the extremely low permeability restrictions to capillarity and resulting restricted moisture migration. Heavy clays do not normally present a problem for ground freezing operations lasting a few months. Problem soils are lean clay and clayey silt.

Rapid freezing can be used as a device to minimize, if not eliminate, ice segregation in soils. However, after a period of time when the ice front advance slows down or stagnates, the possibility for ice segregation and associated heave will exist. Accordingly, in all such cases, careful monitoring is essential, especially where structures are adjacent to the excavation.

9.22.4 Selection of Freezing System

In designing a refrigeration system for a particular application, the critical factors which must be weighed are time, temperature, and cost. Generally, the lower the freezing temperature, the higher the cost and the shorter the time. Figure 152 shows the basic elements of some freezing systems that have been used.

The most common and least expensive method of soil freezing in use today is the Poetsch Process. The system consists of an ammonia or freon primary refrigeration plant to chill a secondary



(AFTER SHUSTER, 1972)

COEFFICIENT OF PERMEABILITY, m/hr

Figure 151. Typical effect of soil type on frost expansion pressures and rates.

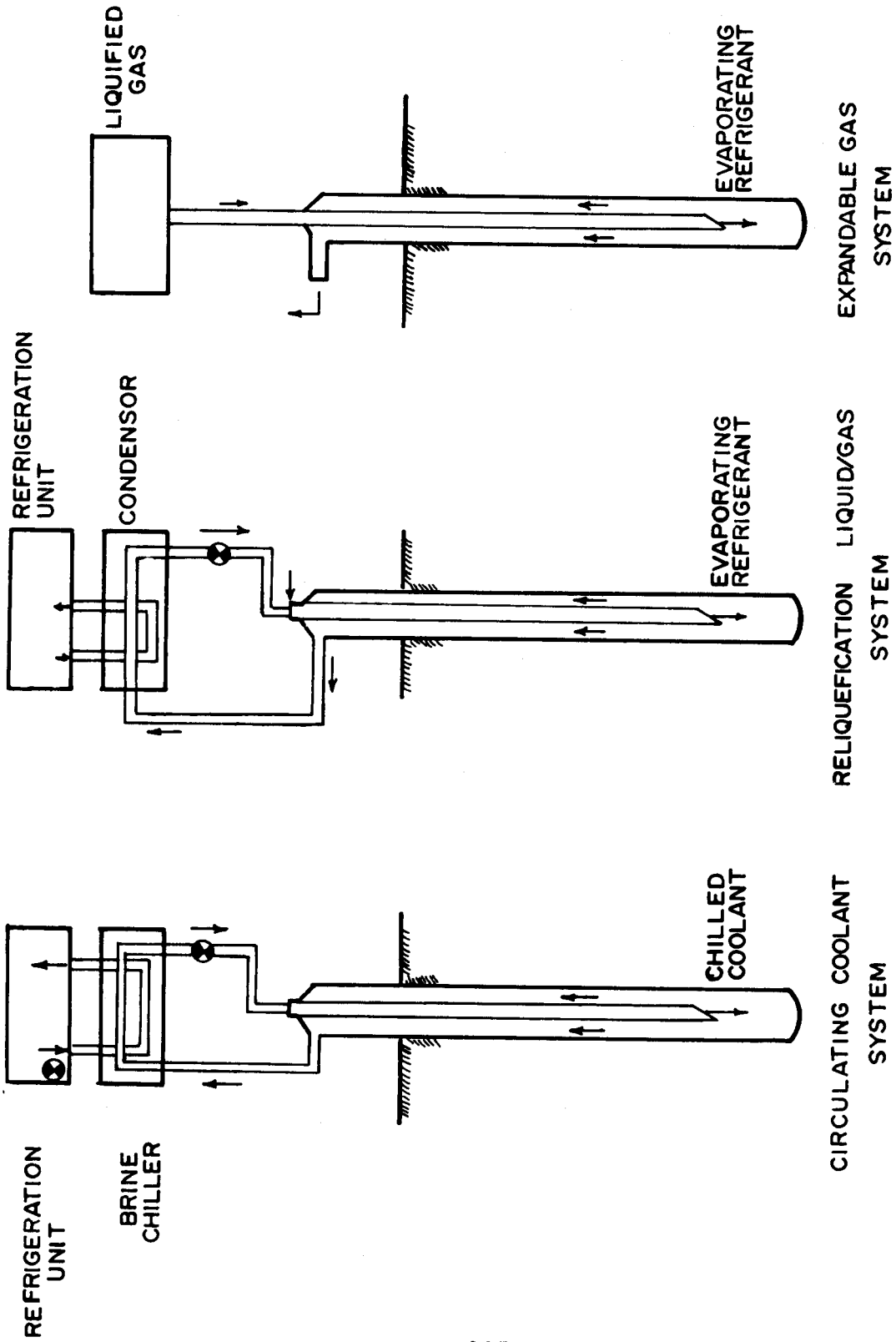


Figure 152. Basic refrigeration system elements for ground freezing.

coolant which is circulated into freeze pipes in the soil. Depending on the brine, temperatures to -65°C can be obtained. The most common system uses calcium chloride as the brine with a minimum of temperature of -40°C . See Figure 153 for view of a typical refrigeration plant.

Additional methods of freezing are now being used which have as their principal advantage a much lower operating temperature at the soil interface and a resultant much quicker freezing time. The methods are currently more expensive than the Poetsch Process, but often the time savings will justify the additional cost. As interest in the freezing process increases, the costs of alternative freezing process with probably become more competitive through refinements in technology. Specifically, the alternatives to the Poetsch Process can be broken down as follows:

a. On-Site Refrigeration Plant

This system, including an on-site refrigeration plant with the primary refrigerant pumped directly into the freezing pipes, has been tried using ammonia or freon. But because the system operates under a vacuum, leaks are undetectable. With carbon dioxide, the system operates under high pressure to keep the CO_2 liquid. Hence, expensive high pressure plumbing is required.

b. Primary and Secondary Refrigerants

A second alternative is to use a thermally cascaded system employing a primary refrigerant which can produce low temperature and a secondary refrigerant capable of transmitting this low temperature. A system using freon as the primary and CO_2 as the secondary coolant seems the most feasible and would be capable of temperatures of -20°C to -55°C . The problem with this system is that a field control of the secondary refrigerant is more expensive. Improved technology in the field, primarily in the direction of simple control units, will make this approach practical.

c. Expendable Refrigerants

Currently, the most feasible way to achieve very low temperatures in the freezing process is to use expendable refrigerants. Liquid Nitrogen (LN_2) is available commercially and can be used directly to freeze soil. No refrigeration plant is required since the liquified state is maintained by pressure. The cost of LN_2 is high enough to make a freezing program an expensive operation.

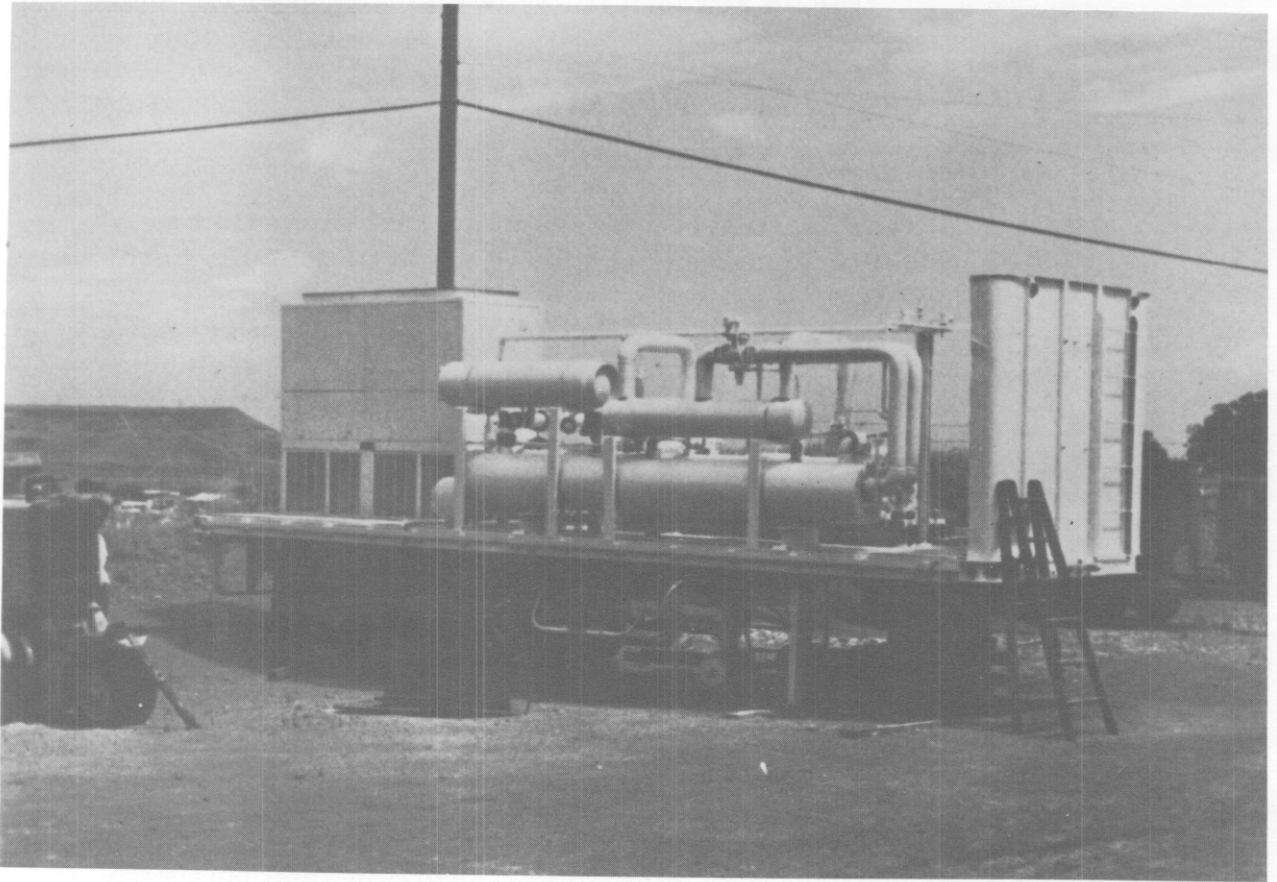


Figure 153. Small mobile freon or ammonia
refrigeration plant.
(Courtesy of Terrafreeze Corporation).

However, as freezing operations which normally might take a few weeks can be compressed to a few days; the direct use of LN₂ is sometimes economically attractive. Care must be taken in this type of operation to control the vented gas. A less efficient but cheaper alternative might be to use solid or liquid CO₂ as a refrigerant directly from a commercial supplier.

Liquid nitrogen (LN₂) is typically used for short term or emergency situations (see Figure 154 for example).

d. Carbonic Acid

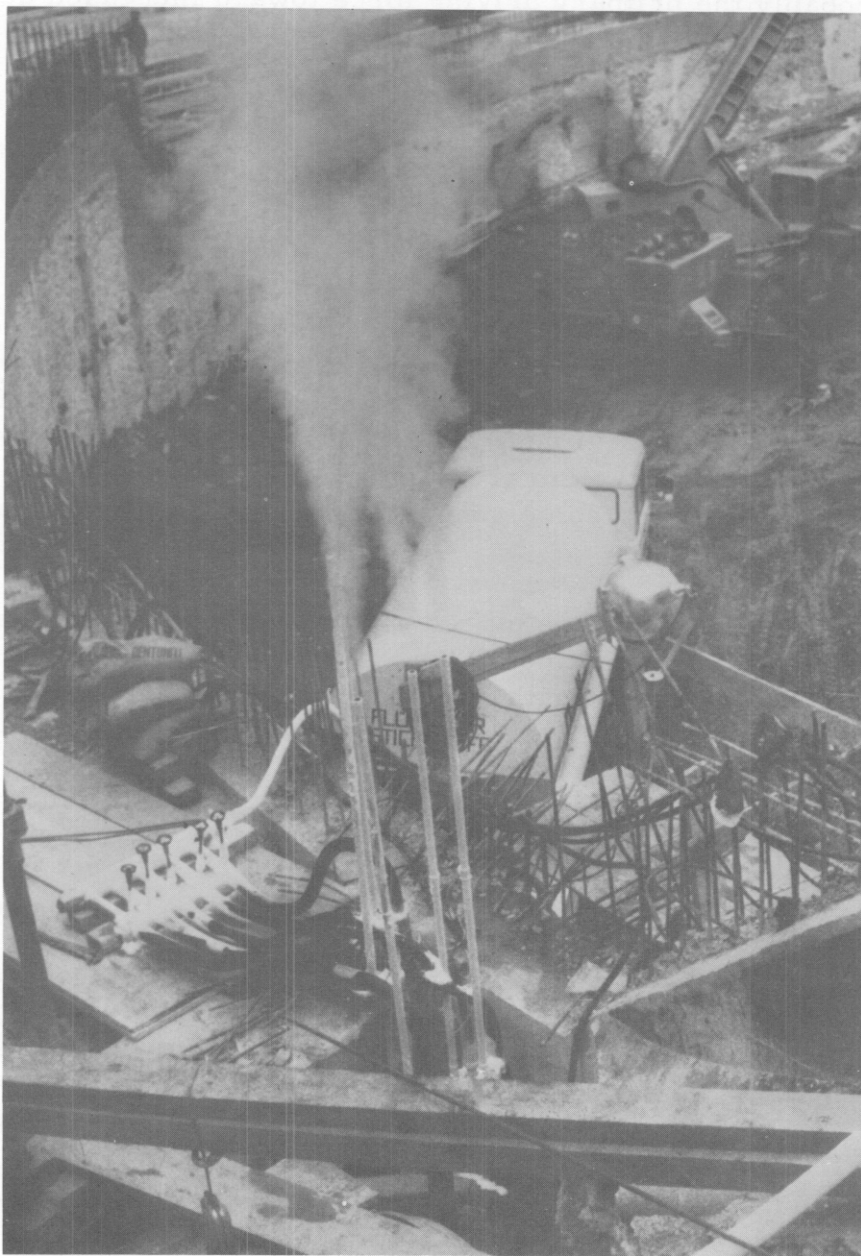
Fujii (1971) has noted that carbonic acid has been used as a refrigerant in Japan and its use might be feasible here.

The basic freezing method consists of choosing one of the freezing processes discussed above and drilling freeze holes into which the freezing pipes are installed. A cylinder of frozen material forms around the pipes and increases in size until the heat gain at the perimeter is equal to the heat taken out in cooling. The freeze pipes are installed in such a manner that the final frozen zones will overlap and a continuous barrier will be formed.

In the freezing process, the greatest amount of heat removal required is to actually change the phase of the water from liquid to solid, i. e. the latent heat of fusion in the soil mass. Once the desired size of the frozen zone has been reached, the only amount of heat removal required to maintain the frozen condition is the heat gain at the perimeter of the frozen zone. The amount is considerably less than the heat removal required when the frozen zone is expanding and more water is becoming solid. This means that the refrigeration load is much less in the maintenance of a frozen earth mass than it is in freezing it. Generally, the capacity of the refrigeration plant in use is reduced after freezing. Fujii (1971) has suggested that a freezing system of expendable LN₂ might be used initially, and after freezing a conventional brine type system might be used to maintain the frozen zone.

9.23 Design Summary

Past practice relied very heavily upon a relatively limited empirical base of experience and a not too sophisticated theoretical structure. For these reasons, many of the designs were necessarily inefficient simply to assure safety. Many successful ground freezing



Note: Manifold for liquid nitrogen. Nitrogen gas being vented to atmosphere.

Figure 154. Liquid nitrogen freezing to cut off leak in diaphragm wall.

(Courtesy of Terrafreeze Corporation).

designs have been constructed, however, and continued developments in theory, practice, and equipment will no doubt evolve a much more efficient design technique. The need for versatility in shaping the frozen structure to a variety of sizes, shapes, and strengths in "poor" ground is probably the primary driving force toward increased design efficiencies.

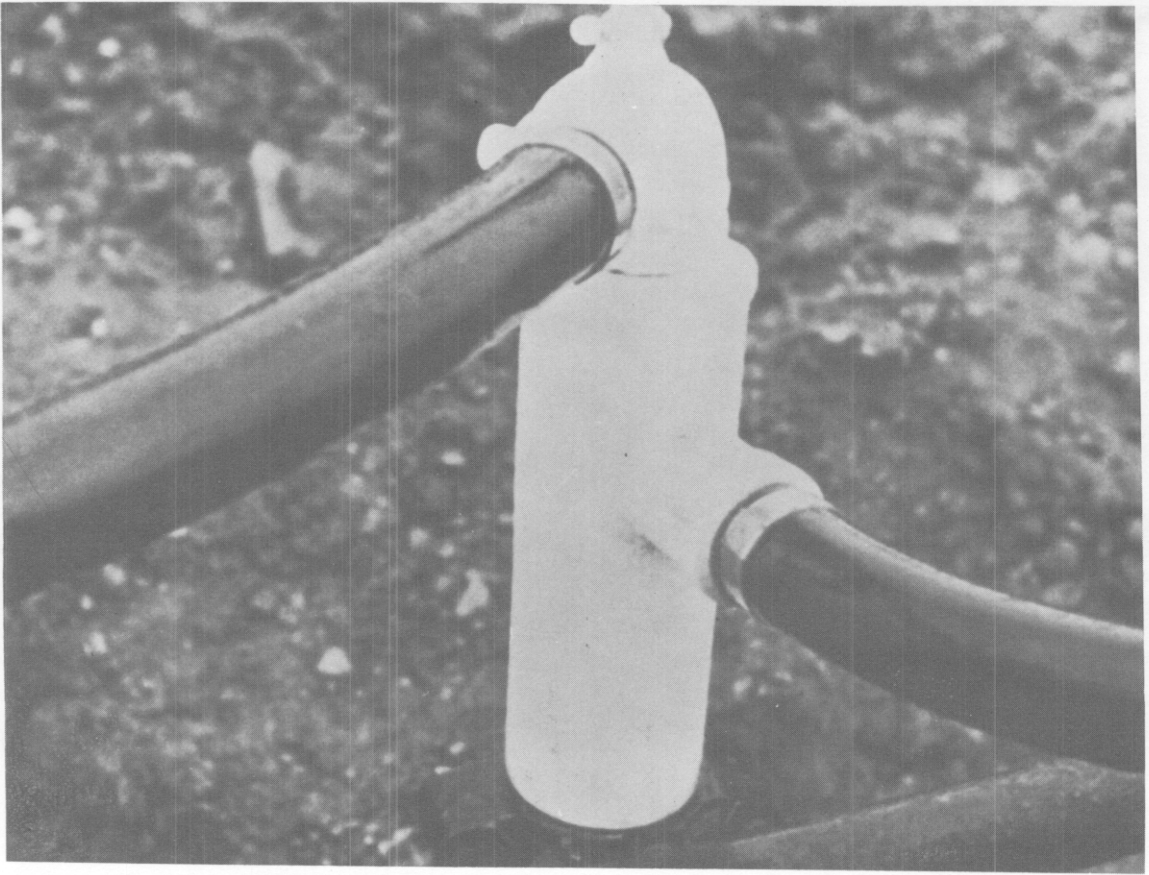
9.30 CONSTRUCTION PROCEDURE

9.31 General Approach

Construction procedures for a ground freezing operation are relatively straightforward once the design is selected. Some modification may be necessary as the process is actually accomplished to account for variations in freezing rate caused by variations in stratigraphy, groundwater movements, unforeseen subsurface conditions, and freezing system departures from ideal design. Design, installation, and operation of a ground freezing system are normally undertaken by specialized subcontractors. However, some general contractors and owners have used freezing in the past.

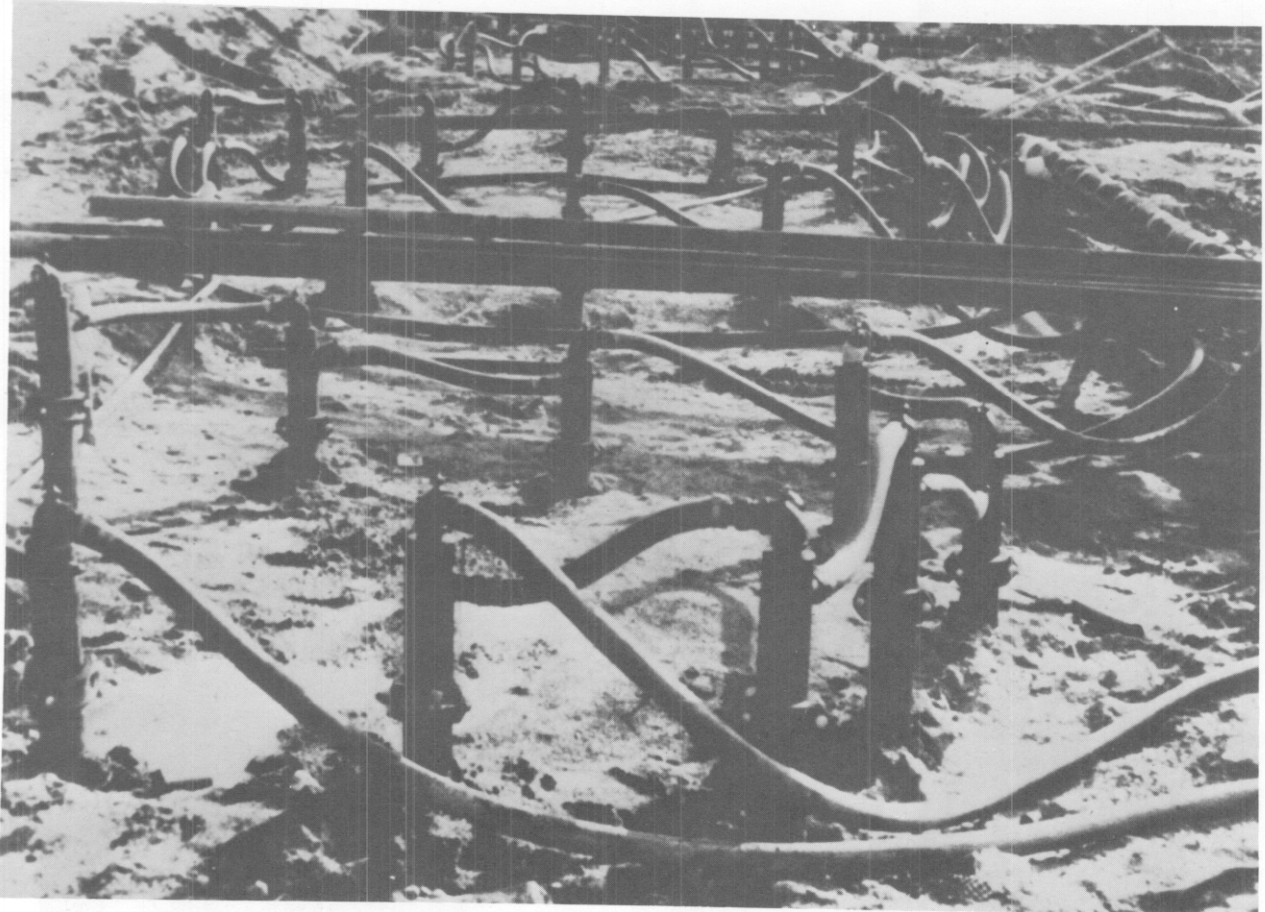
Freeze pipes are placed with spacing, s , and probe size, r_0 , according to time requirements (see Figures 149 and 150) and required freeze wall thickness for strength. Strength requirements are based upon the type of frozen structure (e. g. gravity wall); strength requirements determine the average temperature of the frozen mass. Typical piping and circuits used in connection with circulation of brine are shown in Figures 155 and 156.

Obtaining the required ice wall thickness is not usually a difficult problem unless groundwater flows in excess of about 6 feet/ day are encountered. Frequently, low temperature freezing techniques are employed to overcome heat losses to the moving water above this range. Jumikis observes that minimum wall thicknesses are typically no less than about 3 to 8 feet to 100 foot depths, and no less than about 8 to 15 feet below 100 feet. For example, wall thicknesses on the order of 10 feet in 120 feet of clay and silt beneath 40 feet of water in Lake Huron have been successfully constructed. A large excavation in sands and gravels 50 feet in depth was supported in Colorado by 5 foot thick straight ice walls surrounded by an elliptical 6 foot thick outer wall (see Table 19). A 9 foot thick wall for a 30 foot deep excavation in fine sand (80% passing #40 sieve) was used as an alternate to other wall support methods in downtown Minneapolis during a cut-and-cover tunneling program.



Note: Supply line enters on top; return line goes out side. Small bump on top of the header is a bleed valve.

Figure 155. Freeze pipe control head.
(Courtesy of Terrafreeze Corporation).



Note: Each group of freeze pipe form a series loop from brine supply line back to the return line.

Figure 156. Typical supply and return connections between group of freeze pipes using brine.
(Courtesy of Terrafreeze Corporation).

Table 19. Typical applications of ground stabilization by freezing.

DESCRIPTION	LOCATION	REFERENCE
<u>TUNNELS</u>		
5' - 6' diameter, 650' long	Germany	Deilmann-Haniel, 1965
12.5' diameter, 164' long	Germany	Deilmann-Haniel, 1968
8' diameter, 320' long in clayey silt, fine sand (below water table)	Germany	Deilmann-Haniel, 1968
6' x 18' x 100'	Germany	Braun, 1972
7' diameter, 130' long in saturated peat and fine-grained sand	Germany	Jessberger and Nussbaumer, 1974
12' diameter, 2700' long in saturated fine sand, clay	U. S. S. R.	Marsak, 1964
12-15' diameter, 410' long in saturated sands, silt	France	Foraky, undated
33' diameter, 232' long in saturated sand, "powdered" dolomite	Italy	Braun, 1974
7.5' diameter, 340' long in saturated sand, silt, clay	Germany	Braun, 1974
8' diameter, 180' long	South Carolina	Braun, 1974
8' diameter, 160' long in sand and clay	Germany	Braun, 1974
4' diameter, 3200' long in saturated medium sand	Germany	Sewig and Schebtz, 1969
24' diameter, 150' long in gravel and sand	London	Foraky, 1970

Table 19. Typical applications of ground stabilization by freezing. (Continued).

DESCRIPTION	LOCATION	REFERENCE
<u>OPEN CUT EXCAVATIONS</u>		
130' diameter, 130' deep in saturated gravel, sand, silt, and clay	England	Foraky, 1967
100' x 150' x 50-80' deep saturated sands, gravels (nuclear power plant) (ground water table at 20' depth)	Colorado	Braun, 1970
160' x 270' x 30' deep in sand, gravel, cobbles (basement and foundation, office building)	Arizona	Stoss, 1972
150' x 100' x 40' deep in undescribed deposits (power plant foundations)	Wyoming	Underground Structures, Inc., undated
120' x 50' x 11' deep in fill and clay (cement storage silos)	Utah	Underground Structures, Inc., undated
120' long, 35' deep storm sewer trench in undescribed deposits	Indiana	Underground Structures, Inc., undated
40' x 80' x 20' deep foundation excavation in soft clays	Maine	Underground Structures, Inc., undated
95' x 95' x 75' deep foundation excavation for radioactive waste building in sand deposits	Nebraska	Underground Structures, Inc., 1973/74
35' deep sidewall stabilization as an alternate to underpinning, highway tunnel (open cut) in fine sand	Minnesota	Osterby, 1971
4000' long, 65' deep ground water cutoff wall in sand and gravel	U. S. S. R.	
100' x 70' x 45' deep foundation excavations in sand for two pump stations	New Jersey	Terrafreeze, 1975

Table 19. Typical applications of ground stabilization by freezing. (Continued).

DESCRIPTION	LOCATION	REFERENCE
<u>OPEN CUT EXCAVATIONS (cont'd)</u>		
120' x 52' deep circular foundation excavations in clay and sand for two chimneys	Kentucky	Terrafreeze, 1975
<u>SHAFTS AND CAISSONS</u>		
25 - 5-7' diameter, 40' deep caissons in saturated sand, clays	New York	General Underground Structures, Inc., undated
9 - 14' diameter, 140 deep pumping shafts in glacial till	Michigan	Stoss, 1971
1 - 21' diameter, 230' deep shaft in clay and sand for a salt mine	Louisiana	Terrafreeze, 1974/75
5 - 4' diameter, 160' deep exploration shafts in riverbed sands and gravels	California	General Underground Structures, Inc., undated
20' diameter, 140' deep subbottom intake shaft in Lake Huron bottom mud and silt	Michigan	Hampton, 1974
48' diameter, 60' deep air shaft in saturated fill, gravel, silty sand, silty clay	Germany	Didlaukies, 1971
30' diameter, 50' deep bridge pier caisson in saturated fill	Italy	Jessberger and Nussbaumer, 1973
5' diameter, 110' (max.) deep belled caissons for column casts in sands and silts	Germany	Jessberger and Nussbaumer, 1973
20' diameter, 135' deep water supply shaft in saturated glacial till, decomposed bedrock	New York	Jumikis, 1966

Table 19. Typical applications of ground stabilization by freezing. (Continued).

DESCRIPTION	LOCATION	REFERENCE
<u>SHAFTS AND CAISSONS (cont'd)</u>		
12 - 5' diameter, 90' deep caissons in sands and silts	Germany	Deilmann-Haniel, 1968
2 - 7' diameter, 167' deep caissons in sands, silts, and clays	Africa	Deilmann-Haniel, 1972
15' - 24' diameter, (6 shafts) 80 to 250' deep in unstable silts and saturated chalk	England	Collins and Deacon, 1972

Special care must be taken when drilling the holes and placing the freeze pipes to insure proper alignment. This is a critical part of the operation, in that if one freeze pipe is out of line, closure of the freeze wall might not occur resulting in a leak or concentrated stress condition.

Closure of the wall is critical prior to construction because after excavation begins and the excavation is dewatered, significant pressure gradients will occur across the frozen zone. Any breaches in the freeze wall (even small ones) can lead to failure from ground water inflow. Under these circumstances excavation is normally stopped, the partially completed hole allowed to flood, and freezing continued until the leak is closed prior to further excavation. Boundaries at interfaces between soil and bedrock or sands and underlying clays must be accounted for because these zones are often quite pervious. A closely monitored freezing program is required to prevent any gap in the freeze wall.

It is common practice to design the frozen structure so that it either bottoms in an impervious stratum or a frozen bottom is part of the design. When the former procedure is followed, the freezing probes are commonly inserted several feet into the impervious zone to assure that watertight closure of the frozen structure is accomplished.

9.32 Protection of The System

During the construction process, care must be taken to avoid mechanical damage to the distribution system that might cause loss of refrigerants resulting in a leak in the frozen wall. Maintenance of the frozen mass of earth after it is formed is dependent on a constant removal of heat to compensate for any heat gain at the fringes of the frozen zone. It is necessary to protect the frozen mass from any gross inflow of heat, such as large areal exposure to the atmosphere or long-term localized heat sources (heavy equipment, stationary boiler, etc.). As may be noted in Figures 139 and 140, specific attention should be given to preventing long-term exposure to direct sunlight and excessive amounts of surface water (including rainfall) as well as to venting spaces where equipment generates excessive heat.

9.33 Special Construction Problems

Special details are necessary to work in areas containing existing utilities, especially steam, water, and sewage. Not only can these conduits be frozen and flows interrupted but if not frozen, they constitute a heat source and a potential leak in the freeze wall. One

possible solution is to temporarily reroute the utilities, or if freezing must proceed through the utilities, the utilities can be insulated prior to freezing so that the 32^oF isotherm remains in the insulation.

Shuster (1975) suggests that concrete greater than 1 foot in thickness may be poured directly against frozen earth. The warm concrete placed at 55-60^oF will thaw the surface of the frozen ground as it is placed and the developing heat of hydration furthers the thawing. Continued refrigeration will start refreezing the thawed zone, ultimately reaching and freezing the concrete. This will not occur until the concrete has attained its initial set. Normally no special additives are required, but a slightly richer concrete mix may be desirable.

Concreting against a freeze wall without any special precaution is normal, but it is also possible to place insulation on the frozen earth prior to concreting.

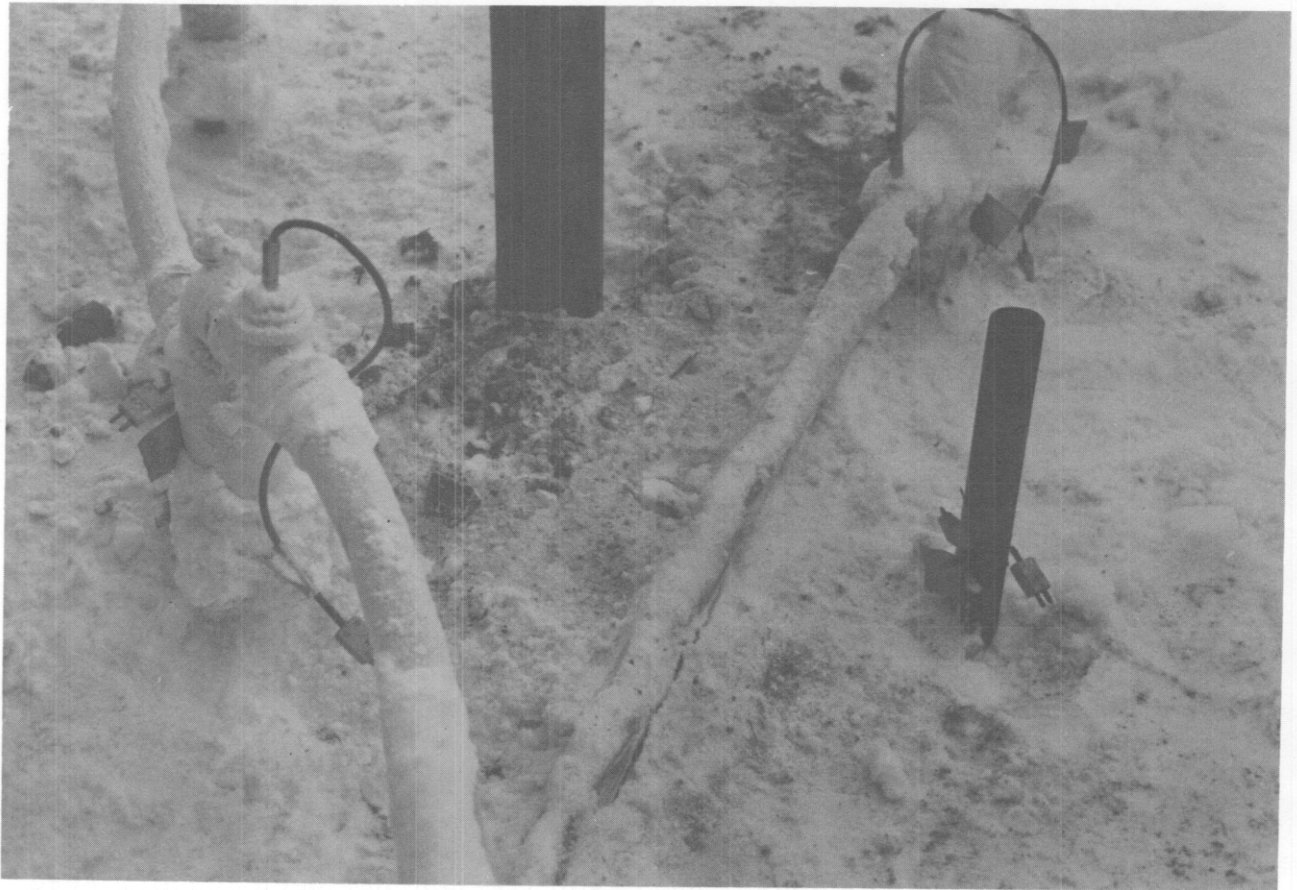
9.34 Construction Monitoring

Detailed monitoring of subsurface temperatures is a critical requirement during construction of a frozen ground structure. Extensive monitoring is required while the freeze walls are being built to establish the rate of progress and to assure that no breaches exist. The level of monitoring is decreased after the structure is complete, but it is usually continued through the excavation stages so that the temperature dependent ground strengths are known as excavation proceeds. A moderate level of monitoring is maintained until the freezing program is terminated.

Monitoring is usually accomplished by measuring the profile of subsurface temperatures in small diameter observation pipes (1"O.D., or so) distributed throughout the frozen zone. Commercially available thermistors or thermocouples are widely used as the temperature sensor, and relatively inexpensive readout devices are adequate for monitoring requirements. Whether a problem exists in the refrigeration system or in unexpected subsurface conditions, it can normally be detected with an accurate profile of subsurface temperatures together with routine coolant temperature data obtained during plant operation.

Figure 157 shows a thermocouple installation to monitor coolant temperature and ground temperature.

Under some soil conditions, surface heave may be an important measurement during the freezing process. As discussed in Section 9.22.3, the greatest deformations occur in fine-grained soils below



Note: Thermocouples are installed on brine return lines. Right foreground shows pipe containing two thermocouples to monitor ground temperature between freeze pipes.

Figure 157. Thermocouples to monitor temperature.
(Courtesy of Terrafreeze Corporation).

the water table, but a prudent contractor will usually maintain a heave monitoring program of moderate extent under any conditions. An adequate program can usually be conducted using conventional surface settlement platforms. The magnitude of frost heave varies according to soil conditions. If deformations of this kind are expected around sensitive structures either freezing should not be used, or care must be made in the design to try to reduce heave (rate of freeze, etc.).

In contrast to heave, lateral or settlement displacements associated with the construction process are much more complex. While a frozen mass of earth has much greater strength and stiffness than an unfrozen mass, it may be subject to creep deformations at high stress. If the frozen zone is to be used for high strength support, especially for a long period of time, laboratory tests should be made to determine behavior at the expected stress level and temperatures. For most projects, however, stress levels are kept low enough to eliminate this concern. Laboratory tests and detailed subsurface explorations are usually adequate to predict and minimize construction-related deformations through design procedures.

9.40 TYPICAL APPLICATIONS

The versatility of the ground freezing process has produced a history of large and small scale applications in over a century of use. Unfortunately little published information exists; therefore, documentation of the causes and extent of failures or misapplications of the technique is hard to find. Primary use of the freezing process has been for shaft and tunnel alignments in unstable ground, but there is currently a growing area of use for foundation and storage excavation stabilization. Some applications have been found for stabilizing potential landslide zones, for extracting samples of loose and running soils from the subsurface, and for "plugging" ground water leakages in excavations supported by other techniques. Successful freezing has been accomplished in water-bearing rock to 900 meters in depth (about 3000 feet), and unstable sediments have been successfully maintained to 600 meters in depth (about 2000 feet) (Maishman, 1975).

Table 19 provides a survey of the types of frozen ground structures that have been successfully completed. While the table is incomplete in displaying all uses, the versatility of the method under poor ground conditions is documented.

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Abbreviations:

ASCE	American Society of Civil Engineers
ICSMFE	International Conference on Soil Mechanics and Foundation Engineering
ECSMFE	European Conference on Soil Mechanics and Foundation Engineering
JSMFD	Journal Soil Mechanics and Foundation Division
GTED	Journal Geotechnical Engineering Division
SGDMEP	Symposium on Grouts and Drilling Muds in Engineering Practice

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