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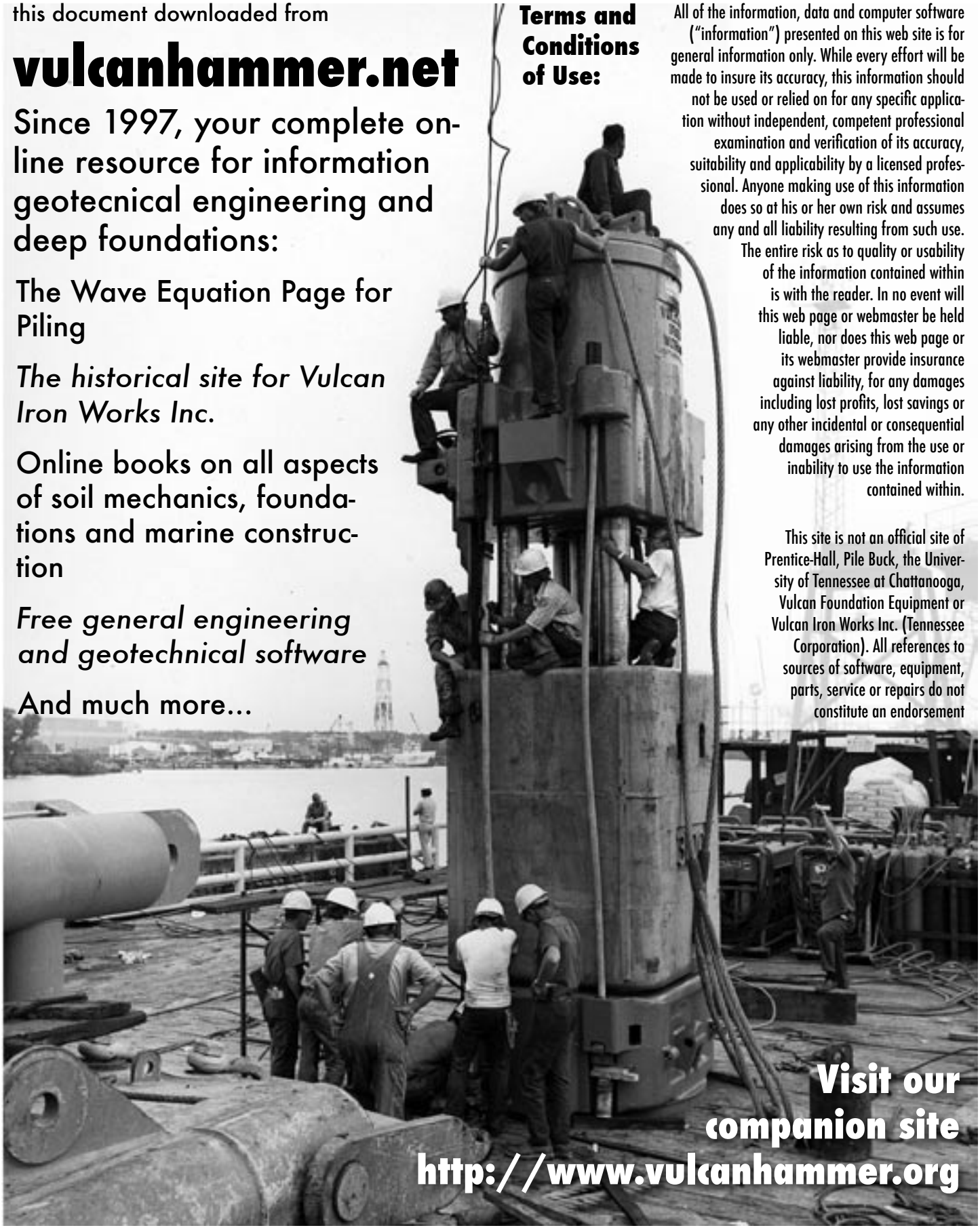
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Report No. FHWA-RD-75-128

LATERAL SUPPORT SYSTEMS AND UNDERPINNING

Vol. I. Design and Construction

D. T. Goldberg, W. E. Jaworski, and M. D. Gordon



April 1976
Final Report

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16. Abstract

This volume is a convenient reference on the design and construction of lateral support systems and underpinning which are often required in conjunction with cut-and-cover or soft ground tunneling. The design recommendations and construction methods described herein are a summary of the more detailed information1 presented in the companion volumes of this study. Included in this volume are discussions of displacements, lateral earth pressure, ground water, passive resistance, stability analysis, bearing capacity, soldier piles, steel sheeting, diaphragm walls, bracing, tiebacks, underpinning, grouting, and freezing. An overview compares the relative costs of the construction methods used in lateral support systems and underpinning.

Other reports developed from the study are FHWA-RD-75-129, Volume II, Design Fundamentals; FHWA-RD-75-130, Volume III, Construction Methods; and FHWA-RD-75-131, Concepts for Improved Lateral Support Systems.

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PREFACE

This volume summarizes the information presented in Volumes II and III reporting the results of the study. The purpose of this volume is to provide the design engineer and/or contractor with a convenient reference for everyday use in cut-and-cover tunneling. The basic design concepts presented in Volume II (Design Fundamentals) are presented with a minimum of discussion on the development of these concepts and design recommendations. The specific design considerations and construction methods for the various wall types, bracing types, and special techniques are also presented (summary of Volume III, Construction Methods). Volumes II and III provide greater detail on development of design recommendations and a more detailed description of the construction techniques and their performance with pertinent references listed.

The reports present information gathered from a state-of-the-art review of Lateral support systems and underpinning. The study was performed through a contract with the Federal Highway Administration as part: of their sponsored research program. The volumes reporting the results are designed to aid the practicing engineer and contractor participating in the design or construction of Lateral support systems or underpinning.

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The contributions of Dr. James P. Gould, of Mueser, Rutledge, Wentworth and Johnston, New York, through his review of the chapters in Volume I dealing with displacements, earth pressure, and design aspects of land cofferdams. Thanks also to:

To Mr. John Shuster of Terrafreeze Corporation, Bethesda, Maryland for the review and contributions for the section on the techniques of ground freezing.

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And finally, to Mr. J. R. Sallberg, contract manager for the Department of Transportation and Federal Highway Administration, for his patience, support, and welcome criticisms throughout the program.

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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
ps_f	pounds per square foot
psi	pounds per square inch
sec	second

Conversions

British Units	Metric Units
1 BTU	0.2520 kg = calories 107.5 kg = meters
1 in	2.540 cm = 25.4 mm
1 in^2	6.452 cm^2
1 in^3	16.103 cm^3
1 ft	30.48 cm = 0.3048 m
1 ft^2	929 cm^2 = 0.0929 m^2
1 ft^3	28,317 cm^3 = 0.0283 m^3
1 pcf (lbs/ft^3)	16.02 kg/m^3 = 0.01602 g/cm^3
1 psf (lbs/ft^2)	4.883 kg/m^2 = 47.9 N/m^2
1 ksf ($kips/ft^2$)	
1 psi (lbs/in^2)	
1 lb	4.45 N
1 in - lb	0.1127 N-m

List Of Svmbols

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.

Symbol	Represents	Reference
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
J-J, l	general symbols for Length or distance	
N	general symbol for stability number or standard penetration resistance	
OCR	over consolidation ratio	

Symbol	Represents	Reference
P	general symbol for Load or force	
p	general symbol for pressure	
pH	ne gative logarithm of effe ctive hydrogen ion concentration	
R, r	general symbols for radius	
S, s	genera.1 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	
$\delta_{v\,(max)}$	vertical displacement (maximum)	
$\delta_{h\,(max:)}$	horizontal displacement (maximum)	
ε	general symbol for strain	
γ	general symbol for unit weight; total unit weight of soil unless other wise 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	Pois son's Ratio	
V	Pois son's Ratio	
ϕ	general symbol for friction angle of soil	

Symbol	Represents	Reference
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 pr e s sure (effective stress)	
τ	general symbol for shear stress or shear resistance	

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

CHAPTER 1 - DISPLACEMENTS

1.10 GENERAL

The purpose of this section is to provide insight into displacements occurring adjacent to deep excavations -- specifically, into those factors influencing displacements and into the manner in which displacements occur.

While the magnitude of settlement is a useful indicator of potential damage to structures, the amount of settlement change with horizontal distance (settlement profile) is actually of greater significance.

Horizontal displacements have proven to be a source of severe damage, even in the presence of underpinned structures.

1.20 CHARACTERISTICS OF WALL DEFORMATION

General Mode of Deformations

Figure 1 shows the possible range of deformations for perfectly rigid walls and for walls displaying flexure. Basically, the range of behavior includes translation and either rotation about the base or rotation about the top. In addition, wall deformation will include some bulging as a result of flexure -- the amount of bulging depending upon the stiffness of the wall support system.

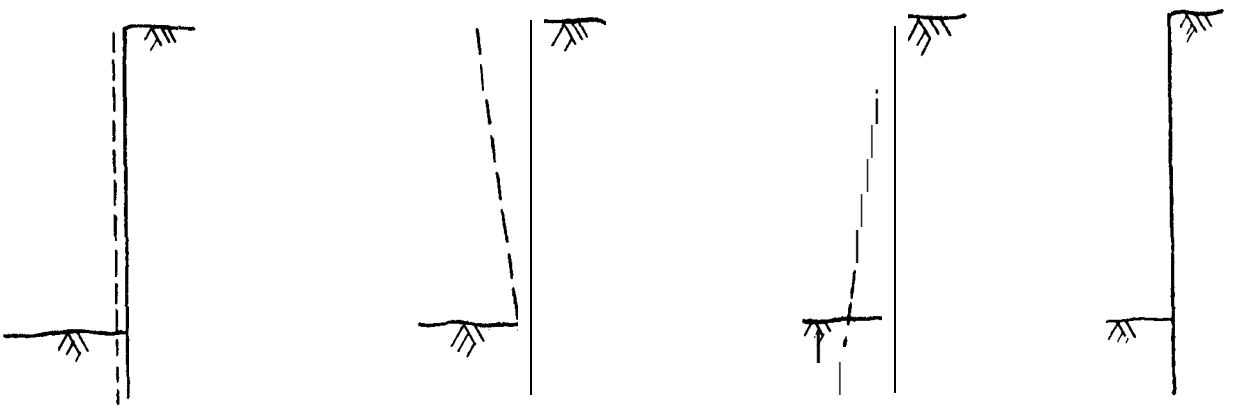
Internally Braced Walls

The upper portion of the internally braced wall is restrained from undergoing large horizontal movement especially when braces are pre-stressed and are installed at or close to the surface. This produces the typical deformation mode as shown in Figure 2a. The degree of rotation will depend upon the toe restraint below the bottom of the excavation.

Tied-Back Walls

If the top of the tied-back wall remains fixed, then the deformation mode is similar to that of an internally braced wall (see Figure 2b, left panel). On the other hand, settlement of the wall, partial yielding of the ties, gross movement of the soil mass, or shear deformation of the soil mass may result in inward movement of the top and rotation about the bottom as shown in Figure 2b, right panel.

(a) INFINITELY RIGID WALLS



(b) WALLS DISPLAYING FLEXURE

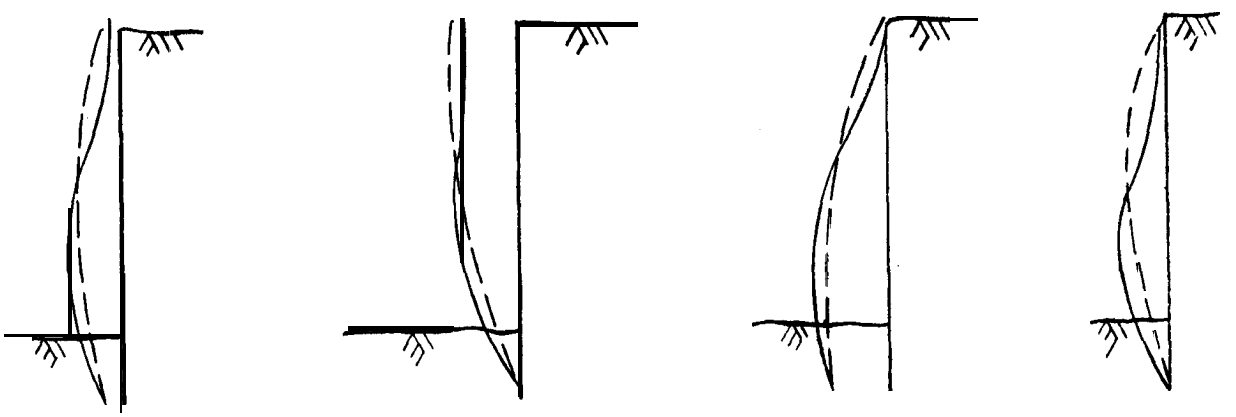
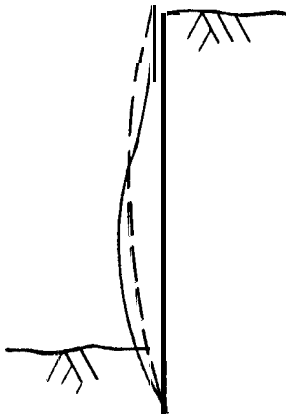
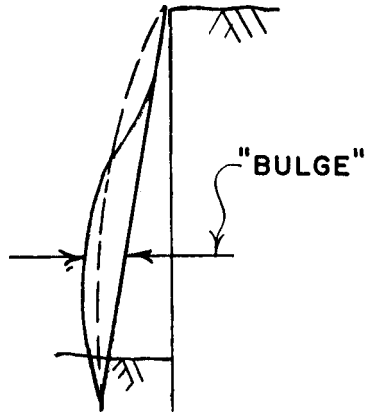


Figure 1. General deformation modes.

(a) TYPICAL FOR INTERNAL BRACING

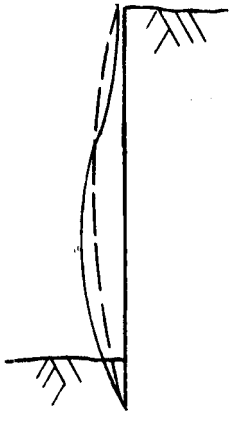


FIXED OR SLIGHT TRANSLATION

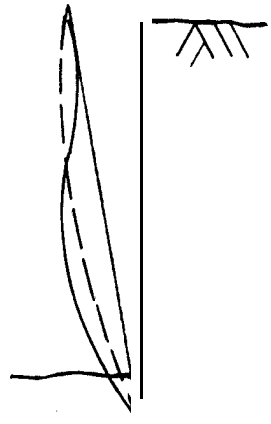


ROTATION ABOUT TOP

(b) TYPICAL FOR TIEBACKS



FIXED OR SLIGHT TRANSLATION



ROTATION ABOUT BOTTOM

Figure, 2. Typical deformation of tied- back and internally braced walls.

Comparison of Braced Walls with Tied-Back Walls

There are insufficient data for a meaningful comparison of deformations of internally braced walls and tied-back walls. In competent soils (e. g. granular deposits, dense cohesive sands, very stiff or hard clays, etc.) displacements are small and no significant differentiation can be made between tiebacks and bracing.

A number of factors indicate that a superior performance should be attained with tiebacks in competent soils:

1. In granular soils in which soil modulus increases with stress level, the prestressed soil mass, engaged by the tiebacks, is made more rigid and therefore less deformable.

2. Tiebacks are typically prestressed to about 125 percent of the design load and then locked-off between 75 percent and 100 percent of the design load. Prestressing in this manner prestrains and stiffens the soil and pulls the wall back toward the soil to remove any "slack" in the contact zone.

3. Internal bracing, if prestressed, is usually to about 50 percent of the design load. Typically, the. bracing gains in load as the excavation deepens. Elastic shortening of the strut continues after installation of the member.

4. Temperature strains are more important with bracing than with tiebacks because the latter are insulated in the ground.

5. Internal bracing is removed then rebraced to facilitate construction, whereas tiebacks do not have to be removed.

1.30 MAGNITUDE OF DISPLACEMENTS

1.31 Reported Horizontal and Vertical Displacements

The summary of data in Figures 3 and 4 is an extension of a similar procedure presented by D'Appolonia (1971).* The figures show normalized vertical and horizontal displacements (ratio of the maximum displacements to the height of the cut) versus three general categories of soil type and support type, The corresponding references are listed in Table 1. Diaphragm walls are distinguished from the relatively more flexible soldier pile or sheet pile walls by symbol.

* Complete references are given in the Bibliography contained in Volumes II and III of this report.

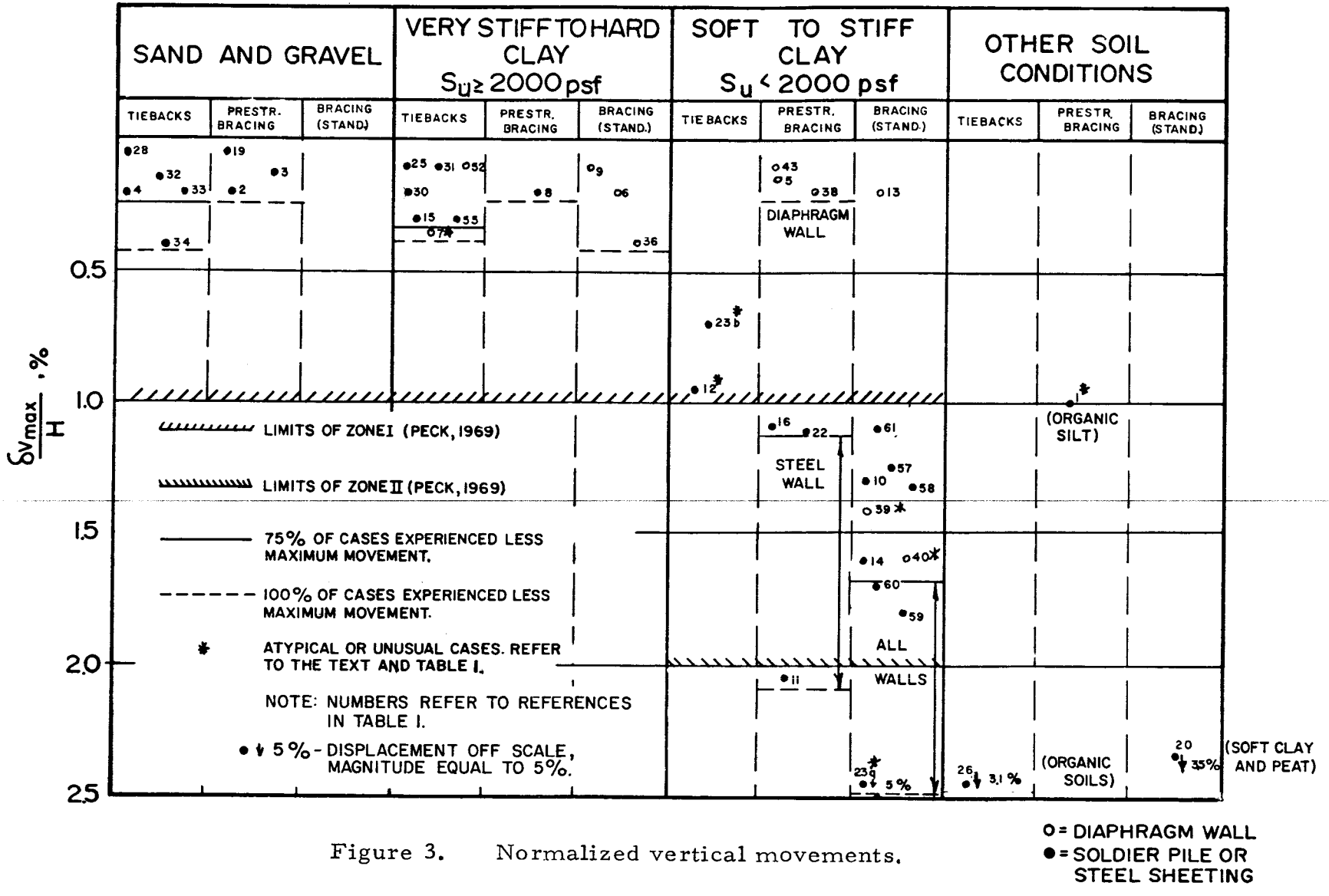


Figure 3. Normalized vertical movements.

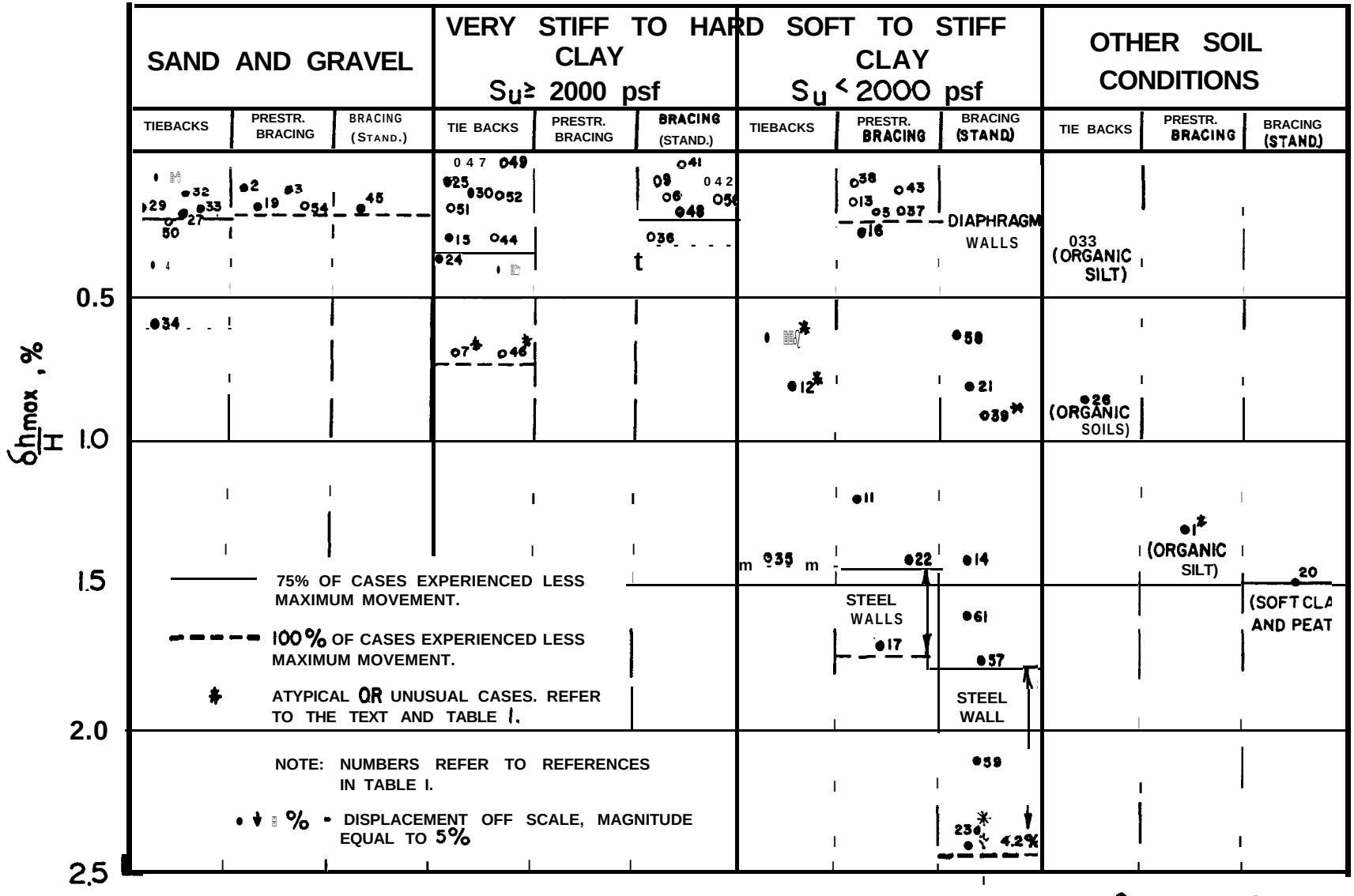


Figure 4. Normalized horizontal movements.

Table 1. Summary of references on displacement.

Ref. #	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of Cut	$\delta_{v_{max}}$	$\delta_{h_{max}}$	Comments
1	Lambe, Wolfskill, & Wong (1970)	SSP	Strut, (Prestressed)	Fill, Organic Silt, till, rock	5X' (17.7m)	7" (17.8cm)	9" (22.9cm)	Consolidation settlements significant. Settlements of 3" (7.6cm) up to 70' (21.3m) from excavation.
2	O'Rourke and Cording (1974) (G St. Excavation)	SP	Struts (Prestressed)	Dense Sand and gravel, Stiff clay	60' (18.3m)	I. 5" (3.8cm)	.9" (2.3cm)	Removal of struts increased settlement from 0.9"(2.3cm) to I. 5' (3.8cm).
3	O'Rourke and Cording (19741 (7th & G Streets)	SP	Struts (Prestressed)	Dense Sand and gravel, Stiff clay	82' (25m)	1.5" (3.8cm)	I. 25" (3.2cm)	Some time-dependent consolidation settlements.
4	O'Rourke and Cording (1974) and Ware. Mirsky, and Leuniz (19731 4th & G Streets)	SP	Tiebacks	Dense Sand and gravel, Stiff clay	40' (12.2m)	.7" (1.8cm) 2" (5.1cm)	2" (5.1cm)	Street settlements small while soldier piles settled due to down-drag from tiebacks. Soldier piles settled 2"(5.1cm) maximum.
5	Lambe, Wolfskill and Jaworski (1972)	DW	Struts (Prestressed)	Fill, hard to medium clay, till	50' (15.2m)	1" (2.5cm)	1.2" (3.0cm)	Minor consolidation settlements. School located 5'(1.5m) from wall.
6	Durland (1974) and St. John (1974) (New Palace Car Park)	DW	Struts (Slabs poured as excavation proceeded)	Gravel and very stiff clay	52' (15.9m)	.6" (1.5cm)	1" (2.5cm)	
7	Burland (1974) and St. John (1974) (Neasden Underpass)	DW	Tiebacks	Very Stiff clay	26' (7.9m)	1.1" (2.8cm)	2.2" (5.6cm)	Much of the wall movement was pure translation and continued with time. Extremely small vertical settlements except directly behind the wall.
8	O'Rourke and Cording (1974) (11th & G Streets)	SP	struts (Prestressed)	Dense Sand and gravel and stiff clay	--	.2%	--	Did not report depth of excavation or amount of settlement.
9	Burland (1974) and St. John (1974) (London YMCA)	DW	Slabs and Tiebacks	Gravel and very stiff clay	52' (15.9m)	.5" (1.3cm)	.6" (1.5cm)	
10	N.G.I. (1962) (Oslo Technical School)	SSP	struts	Soft to medium clay	19.5' (5.9m)	3" (7.6cm)	--	Consolidation settlements due to lowering of head in underlying sand.
11	N. G. 1. (1962) (Vaterland #2)	SSP	Struts (Prestressed)	Soft to medium clay	36' (11m)	8.9" (22.6cm)	5.1" (I 3cm)	Nearby underpinned structure settled significantly.
12	McRostie, Burn and Mitchell (1972)	SSP	Tiebacks	Medium to stiff Clay	40' (12.2m)	4.5" (11.4cm)	-4" (-10.2cm)	Excessive tieback prestressing pulled wall away from excavation. Sensitive clay consolidated due to shearing stresses.
13	DiBiagio and Roti (1972)	DW	Floor slabs used to support wall	Medium clay	62' (18.9m)	I. 6" (4. 1cm)	1-1.2" (2.5-3.0cm)	Structure < 2'(0.6cm) from wall. All settlement appeared to be due to lateral wall dcfloction.

See Sheet 5 for notes.

Table 1. Summary of references on displacement. (Continued.)

Ref. #	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of Cut	$\delta_{v_{max}}$	$\delta_{h_{max}}$	Comments
14	N.G.I. (1962) (Grønland #2)	SSP	Struts	Soft to medium clay	37' (11.3m)	7" (17.8cm)	6.3" (16.0cm)	Part of excavation performed under water.
15	Shannon and Strazer (1970)	SP	Tiebacks	Very stiff clay and sand	78' (23. 8m)	3" (7.6cm)	3" (7. 6cm)	Maximum settlement measured at wall. Settlement may be due to downward force exerted by tiebacks.
16	Swatek, Asrow, and Seitz (1972)	SSP	struts (Prestressed)	Soft to stiff clay	70' (21.4m)	9" (22.9cm)	2.3" (5. 8cm)	Large settlement attributed to localized heavy truck traffic. Typically settlements < 5"(12. 7cm).
17	Rodriquez and Flamand (19691	SSP	Struts (Prestressed)	Soft to medium clay	37' (11.3m)	--	7.9" (20. 1cm)	Staged construction to minimize movements. Dewatered to prevent bottom heave.
18	Scott, Wilson and Bauer (1972)	SSP	Struts	Dense fine sands	50' (15.3m)	--	8" (20. 3cm)	Poor performance attributed to poor construction techniques and dewatering problems. Nearby structures damaged.
19	Chapman, Cording and Schnabel (1972)	SP	Struts and Rakers (Prestressed)	Sand and gravel and stiff clay	45' (13. 8m)	. 25" (0.64cm)	1" (2. 5cm)	Running soil encountered in one section.
20	Boutsma and Horvat (1969)	SSP	struts	Soft clay and soft peat	33' (10. 1m)	14" (35.6cm)	6" (15.2cm)	Some settlement due to extensive dewatering for long time period. Affected structures 600' I, "", excavation. Liquefaction of back-fill during extraction.
21	Insley (1972)	SP	Rakers	Soft to medium clay	25' (7. 6m)	--	2.5" (6.4cm)	One section tested to failure.
22	Tait and Taylor, (1974)	SSP	Struts and Rakers (Prestressed)	Soft to medium clay	45' (13.8m)	6" (15.2cm)	7.5" (19. 1cm)	Larger movements attributed to lack of firm bottom for wall. Utility lines damaged; no major damage to adjacent structures.
23a	Hansbo, Hoiman, and Mosesson (1973)	SSP	Rakers	Soft clay	23' (7. 0m)	13.M" (35. 1cm)	11,8" (29.9cm)	Poor sheet pile interlocking. Long time between excavation of center portion and bracing. Disturbance during pile driving for foundation.
23b	Hansbo, Hoiman, and Mosesson (19731	SSP	Tiebacks and Rakers	Soft clay	23' (7. 0m)	2" (5. 1cm)	2" (5. 1cm)	Improved construction techniques.
24	Prasad, Freeman, and Klajnerman (1972)	SP	Tiebacks	Very stiff clay	45' (13.8m)	--	-2" (-5. 1cm)	Top of wall moved away from excavation. Maximum movement at top.
25	Mansur and Alizadeh (1970)	SP	Tiebacks	Very stiff to hard clay	45' (13.8m)	.5" (1.3cm)	.5" (1.3cm)	
26	Sandqvist (1972.1	SSP	Tiebacks	Sand and silt with organic soils	19.5' (5,9m)	7.9" (20. 1cm)	2" (5. 1cm)	Settlement in organics due to lowered ground water level. Pile driving also caused settlement.

See Sheet 5 for notes.

Table 1. Summary of references on displacement. (Continued.)

Ref. #	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of Cut	$\delta_{v\,max}$	$\delta_{h\,max}$	Comments
27	Sigourney (1971)	SP	Tiebacks	Clayey sand and hard clay	20-26' (6.1-7.9m)	--	1.5" (1.3cm)	
28	Goettle, Flaig. Miller. and Schaefer (1974)	SP	Tiebacks	Dense sand and gravel	23' (7.0m)	.25" (0.64cm)	.25" (0.64cm)	Structure with footings only 2' (0.62m) from wall was undamaged.
29	Sigourney (1971)	SP	Tiebacks	Very dense silty sand and gravel	35-43' (10.7-13.2m)	--	.1" (0.25cm)	
30	Clough, Weber, and Lamont (1972)	SP	Tiebacks	Very stiff clay	64' (19.6m)	1.25"± (3.2cm)	1"± (2.5cm)	Top of wall moved away from excavation.
31	Nelson (1973)	SP	Tiebacks	bandy over-burden, hard clay shales	90' (27.5m)	1" (2.5cm)	4" (10.2cm)	Cracking in street indicated potential stability failure ($\delta_{h\,max}$ 15.2cm) Maljian & Van Beveren (1974).
32	Liu and Dugan (1972)	SP	Tiebacks	Dense sand and gravel. very stiff clay	55' (16.8m)	.8"± (2.0cm)	1"± (2.5cm)	Tops of soldier piles pulled away from excavation during prestressing.
33	Larson, Willette, Hall, and Gnaedinger (1972)	SP	Tiebacks	Dense sand	50' (15.2m)	1" (2.5cm)	1" (2.5cm)	
34	Dietrich, Chase, and Teul (1971)	SP	Tiebacks	Silty sand	23-54' (7-16.5m)	2.5" (6.3cm)	1.8" (4.6cm)	Lateral movements measured at top of wall.
35	Cunningham and Fernandez (1972)	DW	Tiebacks	Medium clay under dense sand	23' (7.0m)	--	4" (10.2cm)	Tiebacks anchored to deadman.
36	Cole and Burland (1972)	DW	Rakers	Very stiff clay	60' (18.4m)	1.5" (3.8cm)	2.5" (6.3cm)	Most movements occurred while earth berm supported wall. Excavation in heavily overconsolidated clay.
37	Tait and Taylor (1974)	DW	Tiebacks, prestressed struts and rakers	Medium and soft clay	45' (13.8m)	--	.9" (2.3cm)	Minor settlements of nearby structures
38	Armento (1973)	DW	struts (Prestressed)	Sand and soft to medium clay	70' (21.4m)	1.7" (4.3cm)	1" (2.5cm)	Some settlement may have been caused by other excavations in the area.
39	Cunningham and Fernandez (1972)	DW	struts	Soft and medium clay	32' (9.8m)	5.5" (13.9cm)	3.5" (8.9cm)	Underpinning of nearby footings required after 5.5" (13.9cm) of settlement. 50-70% of movement during caisson construction.
40	Tan (1973)	DW	Basement slab as support	Soft clay	43' (13.2m)	6"± (15.2cm)	--	Settlement estimated on basis of substantial damage to structure 40' (12.2m) from excavation.

See Sheet 5 for notes.

Table 1. Summary of references on displacement. (Continued.)

Ref.#	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of cut	$\delta_{v\,max}$	$\delta_{h\,max}$	Comments
41	Breth and Wanoscheck (1969)	DW	Struts	Hard clay and limestone	60' (18.4m)	--	.4" (1.0cm)	
42	Huder (1969)	DW	Basement slabs as support	Slightly plastic silt and clay	65' (19.9m)	--	1.4" (3.6cm)	
43	Thon and Harlan (1971)	DW	Struts (Prestressed,	Soft to medium clay	78' (23.8m)	1" (2.5cm)	1.2" (3.0cm)	
44	Barla and Mascardi (1974)	SW	Tiebacks	Stiff clay	85 (25.9m)	--	2.6" (6.6cm)	Cracking in nearby structures.
45	Heeb, Schurr, Bone, Henke, and Muller (1966)	SP	struts	Sand	40' (12.2m)	--	.8" (2.0cm)	
46	Breth and Romberg (1972), Romberg (1973)	SP	Tiebacks	Stiff clay and sand	68' (20.8m)	--	5.9" (14.9cm)	Lateral movement of entire soil block.
47	Schwarz (1972) and Andra, Kunzl, and Rojek (1973)	SW	Tiebacks	Clayey marl (stiff clay,	97.5' (29.8m)	.2" (0.5 1cm)	.6" (1.5cm)	Many levels of tiebacks at very close spacing.
48	Corbett, Davies, and Langford (1974)	DW	Rakers	Very stiff clay; -- upper sand and gravel		--	2" (0.51cm)	Construction delayed after hole opened.
49	Hodgson (1974,	DW	Tiebacks and struts	Fill, gravel very stiff clay	26' (7.9m)	--	12" (0.3cm)	Special construction procedure used.
50	Corbett and Stroud (1974)	SP	Tiebacks	Fill, sand and marl	51' (15.6m)	--	8" (2.0cm)	Heave observed 18m from wall.
51	Littlejohn and MacFarlane (1974,	DW	Tiebacks	Gravel and very stiff clay	18' (5.5m)	--	.8" (2.0cm)	
52	Littlejohn and MacFarlane (1974,	DW	Tic backs	Gravel and very stiff clay	47 (14.4m)	.9" (2.3cm)	.9" (2.3cm)	
53	Saxena (1974,	DW	Tiebacks	Organic Silt and sand	55' (16.8m)	--	2.7" (6.9cm)	Tops of some wall sections moved toward soil by same amount.
54	Ware (1974, Personal communication	DW	struts (Prestressed,	Sand and gravel and stiff clay	62' (18.9m)	--	I.25" (3.2cm)	
55	Goldberg-Zoino & Assoc. Files	SP	Tiebacks	Fill, organic sand, stiff clay, till	45' (13.8m)	1.5" (3.8cm)	1" (2.5cm)	Vertical settlements due to lagging installation. Most horizontal movement away from excavation.
56	Burland (1974, and St. John (1974,	DW	Cantilever Wall	Very stiff clay	26' (7.9m)	Small settle-ments	.5" (1.3cm)	

See Sheet 5 for notes.

Table 1. Summary of references on displacement. (Continued.)

Ref. #	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of Cut	$\delta_{v_{max}}$	$\delta_{h_{max}}$	Comments
57	N. C. I. (1962) Telecommunications Center	SSP	Struts (Prestressed)	Medium and soft clay	26' (7.9m)	3.9" (9.9cm)	5.5" (13.9cm)	Significant movements after strut removal.
58	N.C.I. (1962) Enerhaugen South	SSP	struts (Prestressed)	Medium and soft clay	26' (7.9m)	4.2" (10.7cm)	2" (5.1cm)	Lateral deflections probably more than shown.
59	N. G. I. (1962) Vaterland #1	SSP	Struts	Medium and soft clay	36' (11.0m)	7.9" (20.1cm)	9" (22.9cm)	
60	N. G. I. (1962) Grønland #1	SSP	Slabs as support	Medium to soft clay	37' (11.3m)	75" (19.0cm)	--	Air pressure and upside down construction used.
61	N. G. I. (1962) Vaterland #3	SSP	Struts	Medium and soft clay	30' (9.2m)	3.9" (9.9cm)	5.9" (14.9cm)	
62	Maljian and Van Beveren (1974)	SP	Tiebacks	Stiff to very stiff clay and cohesive sand and silt	110' (33.6m)	3" (7.6cm)	2" (5.1cm)	Maximum vertical settlement atypical for the site--usually lateral movement greater than vertical.
63	Jennings (cases reported by Littlejohn and MacFarland [1974]) South Africa		Tiebacks	Firm	48' (14.7m)	--	3" (7.6cm)	Damage to utilities in street and building across street.
				Fissured	48' (14.7m)	--	1.5" (3.8cm)	Acceptable movements
				Clay	14' (22.6m)	--	1.5" (3.8cm)	
				Very stiff fissured clay	48' (14.7m)	--	.75" (1.9cm)	
				Soft Jointed rock	59' (18.0m)	--	1" (2.5cm)	

Notes:

1. SSP : Steel shoot piling
   SP : Soldier pile wall
   DW : Diaphragm wall
   SW : Secant wall

2. δ_h and δ_v are maximum horizontal and vertical displacements.

3. Reference # represents references listed by author in Bibliography.

Vertical and horizontal displacements in the ground outside the excavation arise from:

1. Horizontal and vertical displacement of the wall - - in general, this is rotation, translation, and flexure.

2. Movement of soil -- for example, loss of soil through lagging, overcutting and improperly backpacking of lagging, spalling of slurry trench walls, voids created from pulling of sheeting, etc. (See Volume III, Construction Methods, for more detailed discussion under various technique s) .

3. Consolidation of soil -- for example, densification of loose granular soils from vibration or consolidation of soft cohesive soils from lowering of ground water outside the excavation.

4. Base instability or near instability -- excessive shear strains set up by the imbalance created by removal of removal of load contribute to base heave and/or plastic conditions in soil.

5. Stress relief from excavation -- this reduces vertical stress below the base and relieves the K_o horizontal stress (earth pressure at rest). In turn, the possible modes are base heave, shear strains, and lateral strains.

The performance data indicate the following.

1. Sand and Gravel; Very Stiff to Hard Clay

Seventy-five percent of the excavations in this material experienced horizontal movements less than 0.35 percent of the excavation depth. Generally, the performance is not significantly affected by support method or by wall type.

One probable reason for little apparent difference between wall type and support method is the fact that the measured displacements are small (typically less than 0. 10 feet for a 50-foot excavation). Many construction factors can contribute to displacement variation o.f similar magnitude and therefore would mask such variation.

Two anomalous cases (no. 7 and no. 46, Table 1) reveal a potential source of extraordinary lateral movement of a tied-back wall retaining primarily very stiff or hard clays, Ward (1972) cites horizontal strains as two to three times as large as vertical strains in overconsolidated London clay.

2. Soft to Stiff Clay

Wide variations for both horizontal and vertical displacements are evident. Sixty-five percent of the cases experienced horizontal displacements which exceeded 1 percent for steel sheet pile or soldier pile walls, whether prestressed or not.

The largest benefit is derived from concrete diaphragm walls with prestressed bracing. Indeed, both horizontal and vertical displacements are no different from those typical for sands and very stiff to hard clays, being about 0.25 percent or less.

Another major cause of settlements in cohesive soils is lowering of the ground water table.

1.32 Effect of Wall Stiffness on Lateral Displacements in Clay

Wall stiffness refers not only to the structural elements comprising the wall but also to the vertical spacing between the support members. The measure of wall stiffness is defined as the inverse of Rowe's flexibility number for walls $\frac{EI}{L^4}$.

where:

E = modulus of elasticity of wall

I = moment of inertia/foot of wall

L = vertical distance between support levels or between support level and excavation base

A plot of observed displacements for stability number ($N = \frac{\gamma H^*}{S_u}$) and stiffness factor ($\frac{EI}{L^4}$) is developed on Figure 5. The stability number, which considers both overburden stress (γH) and the undrained shear strength (Su), is a measure of the relative strength or deformtability of the soil.

The data demonstrate what is intuitively obvious -- that deformations are functions of soil strength and wall stiffness. The contour lines of maximum lateral wall movements show this trend clearly. These data allow one to examine qualitatively the relative change in anticipated lateral displacement for a given change in wall stiffness and/or stability number of the soil.

*Ratio of overburden stress to undrained shear strength.

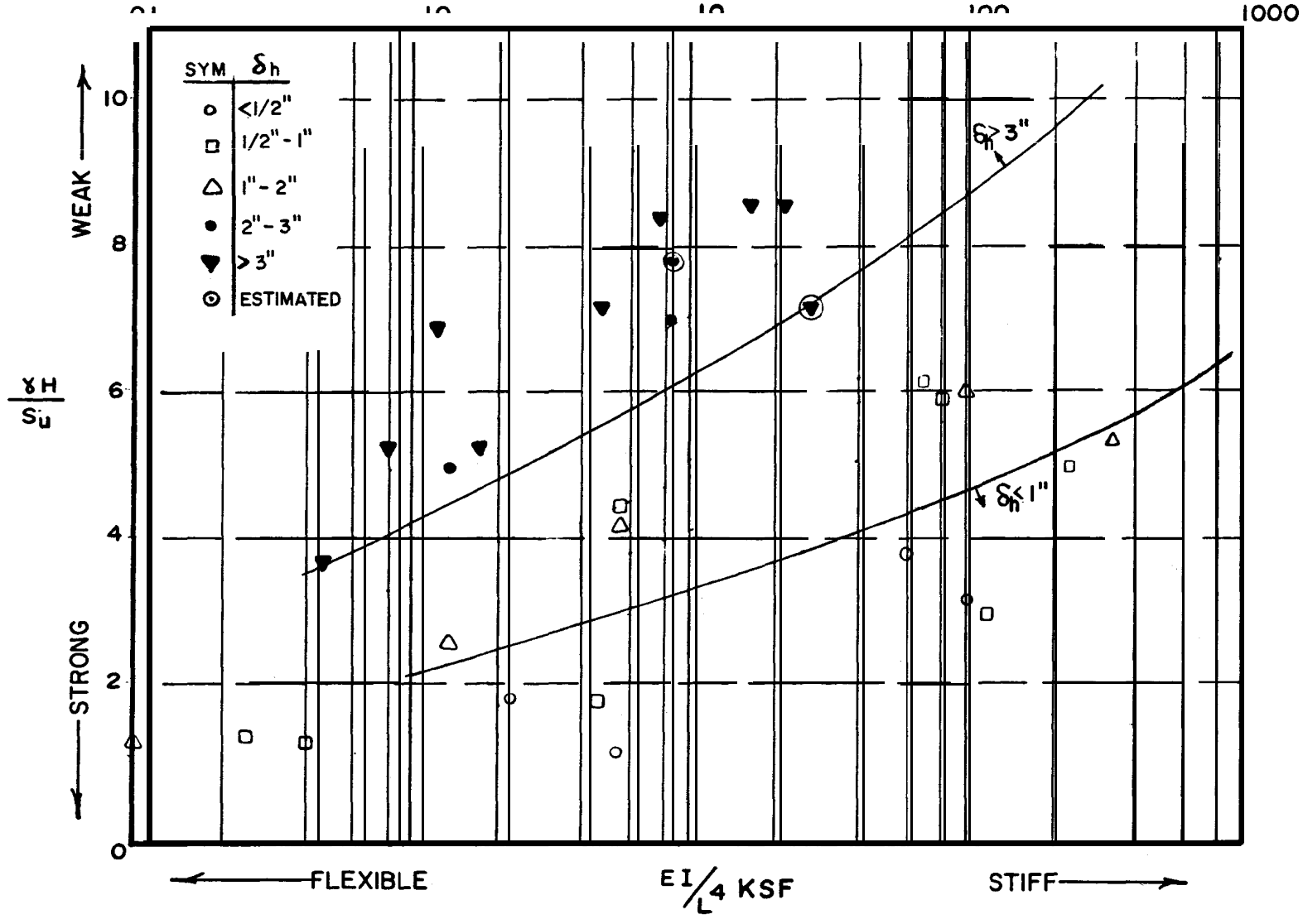


Figure 5. Effect of wall stiffness and soil strength on lateral wall deflection.

1.33 Wall Movement Versus Settlement

Comparison for all Cases

Figure 6 compares observed maximum horizontal and vertical displacements for all types of soils, support systems, and wall types. The absolute magnitude is shown in panel (a) and the frequency distribution of the ratio of the movements in panel (b). The figure shows that practically all the vertical displacements fall within a range of 1/2 to 1-1/2 times the horizontal displacements, with most of them lying in the range of 2/3 to 1- 1/3 times the horizontal movement.

Soft to Medium Clay

Figure 7 compares displacements for soft to medium clays. The average curve shows that the vertical displacements are generally well in excess of the horizontal displacements and that the range of displacements increases with the magnitude of the displacements.

The difference is believed to be directly attributable to consolidation settlements which are usually the result of changes in water levels adjacent to the excavation.

Very Stiff to Hard Clays

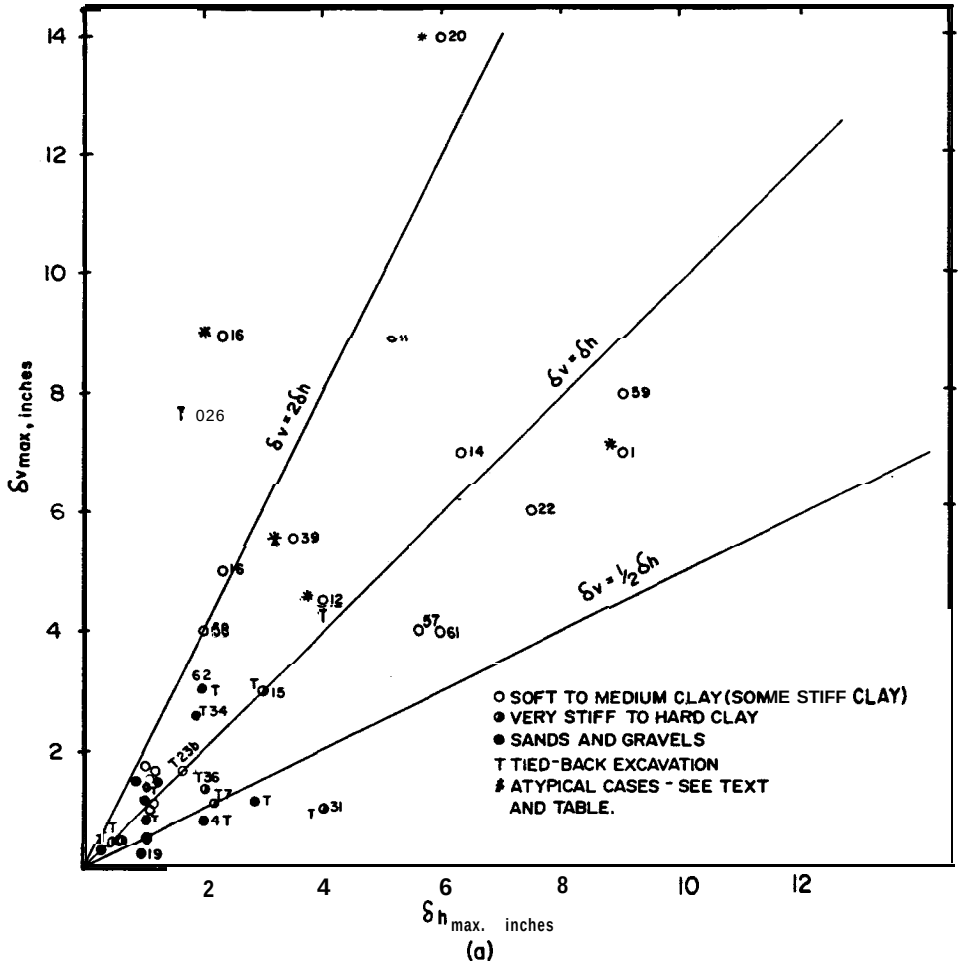
Figure 8 compares the displacements of these soils. As mentioned in a previous discussion, comparatively large lateral displacements have been reported in several tieback projects.

1.40 PARAMETRIC STUDIES

The results of a finite element study for evaluating the effect of wall stiffness on reducing deformations in various soil conditions is shown in Figure 9. Also shown for comparison are the lines defining deformation limits from Figure 5.

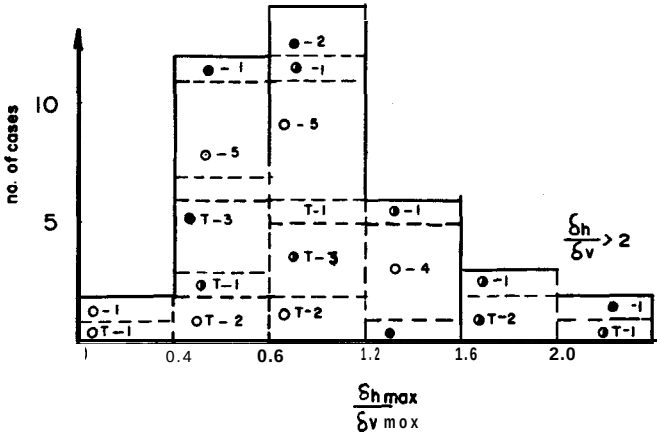
The finite element computer program used to develop thesedata considered only cohesive soils and internally braced excavations; See Volume II (Design Fundamentals) for soil properties and methods of analysis,

Figure 9 shows that the predicted lateral displacements are less than the observed values for a given condition. This difference is related to the inherent movements which are a function of the costruction process. Nonetheless, the theoretical results show a trend similar to that described by the field observations; that is, the stiffer walls result in lower movements for a given soil condition.



(a)

MAXIMUM VERTICAL AND HORIZONTAL DEFORMATIONS

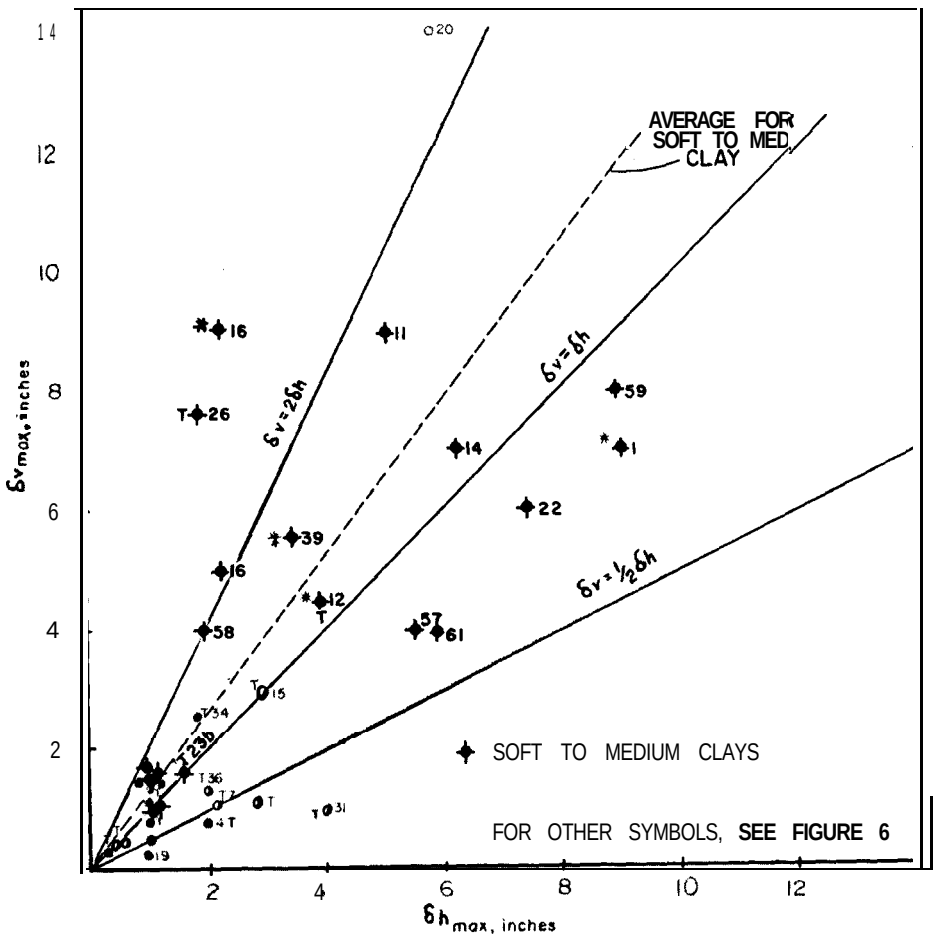


(b)

FREQUENCY DISTRIBUTION OF RATIO OF MOVEMENTS

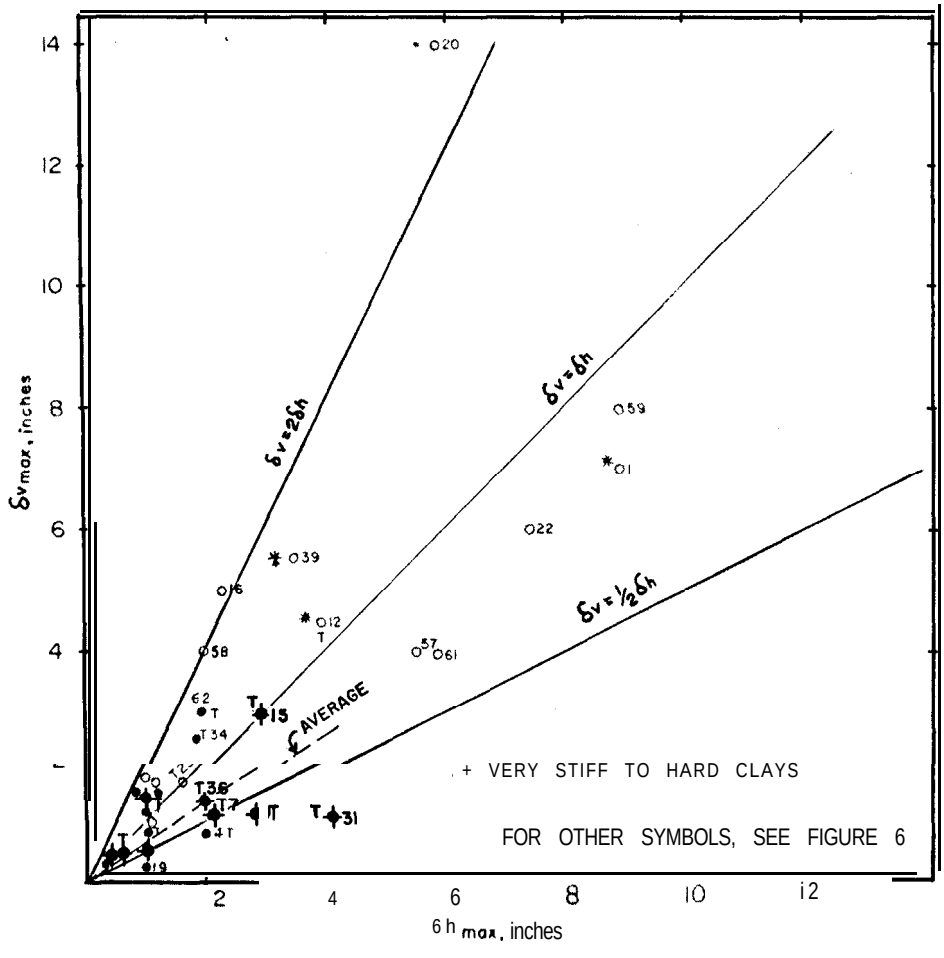
NOTE: NUMBERS REFER TO CASE STUDIES LISTED IN TABLE I.

Figure 6. Comparison of maximum vertical and horizontal displacements,



NOTE: NUMBERS REFER TO CASE STUDIES LISTED IN TABLE I.

Figure, 7. Comparison of vertical and horizontal displacements for soft to medium clays,



NOTE: NUMBERS REFER TO CASE STUDIES LISTED IN TABLE I.

Figure 8. Comparison of vertical and horizontal displacements for very stiff to hard clays.

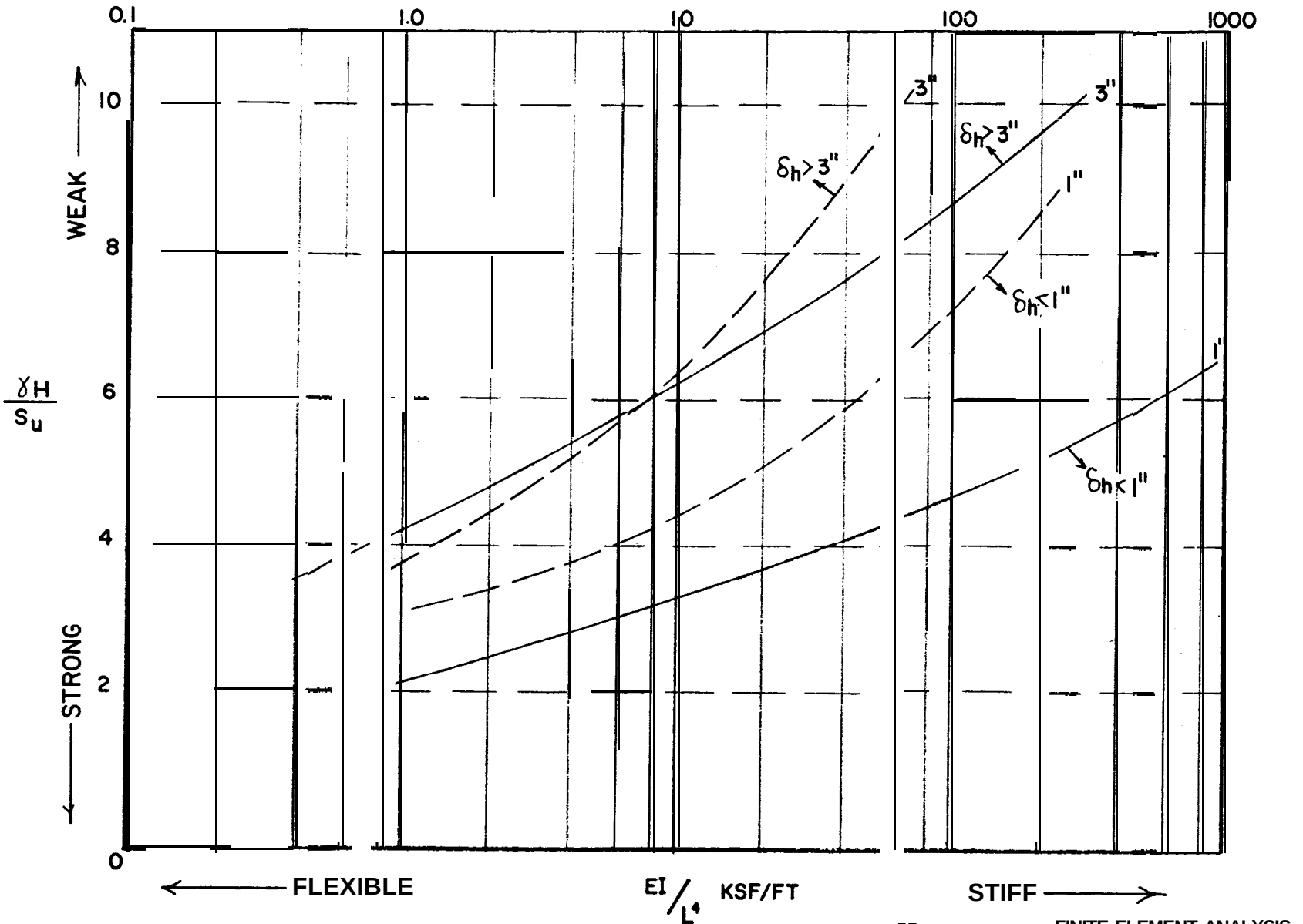


Figure 9. Comparison of lateral movements from finite element analysis and observed movements.

The results of the finite element analysis should not be taken in the quantitative sense. The intent is that such an analysis should be used as a guide in the design and in the consideration of various options for a bracing system.

1.50 DISTRIBUTION OF DEFORMATIONS

Currently, many engineers rely on judgment and experience in predicting deformation patterns adjacent to sheeted excavations. This section provides some information to aid the engineer in evaluating what deformation patterns might be expected adjacent to a cofferdam.

1.51 Vertical Deformations

Figure 10 illustrates how the observed maximum settlement patterns behind a wall varies with the soil conditions. The pattern of movements indicates that maximum movements occur immediately adjacent to the excavation. Also, one might expect significant movements a distance from the cut equal to twice the depth of the cut. At present, there are insufficient data to define any significant difference in settlement pattern based on soil type or support wall.

Comparing the settlement patterns of sand versus cohesive soil, the sands show essentially no settlement beyond twice the depth of the excavation whereas the cohesive soils do. This is most likely attributable to the consolidation experienced in the more compressible soils caused by lowering of the ground water table.

Reviewing Figure 10, it appears that both soft clays and the granular soils experience a significant agnular distortion outside a distance equal to the excavation depth (D/H = 1). The average lines of settlement ratio versus normalized distance, shown as dashed on the figure, may be used as a basis of comparison of this distortion. On the other hand, the stiffer clays (S> 2000 psf) seem to experience a more gentle distortion slope, even though the zone of influence extends further back from the excavation face.

1.52 Parametric Study On Zone Of Influence

Finite element studies were performed on several of the deformation modes shown in Figure 1. These analyses were aimed at obtaining some qualitative information on the settlement profile one might expect adjacent to the excavation.

Details of the finite element analysis are given in Volume II (Design Fundamentals).

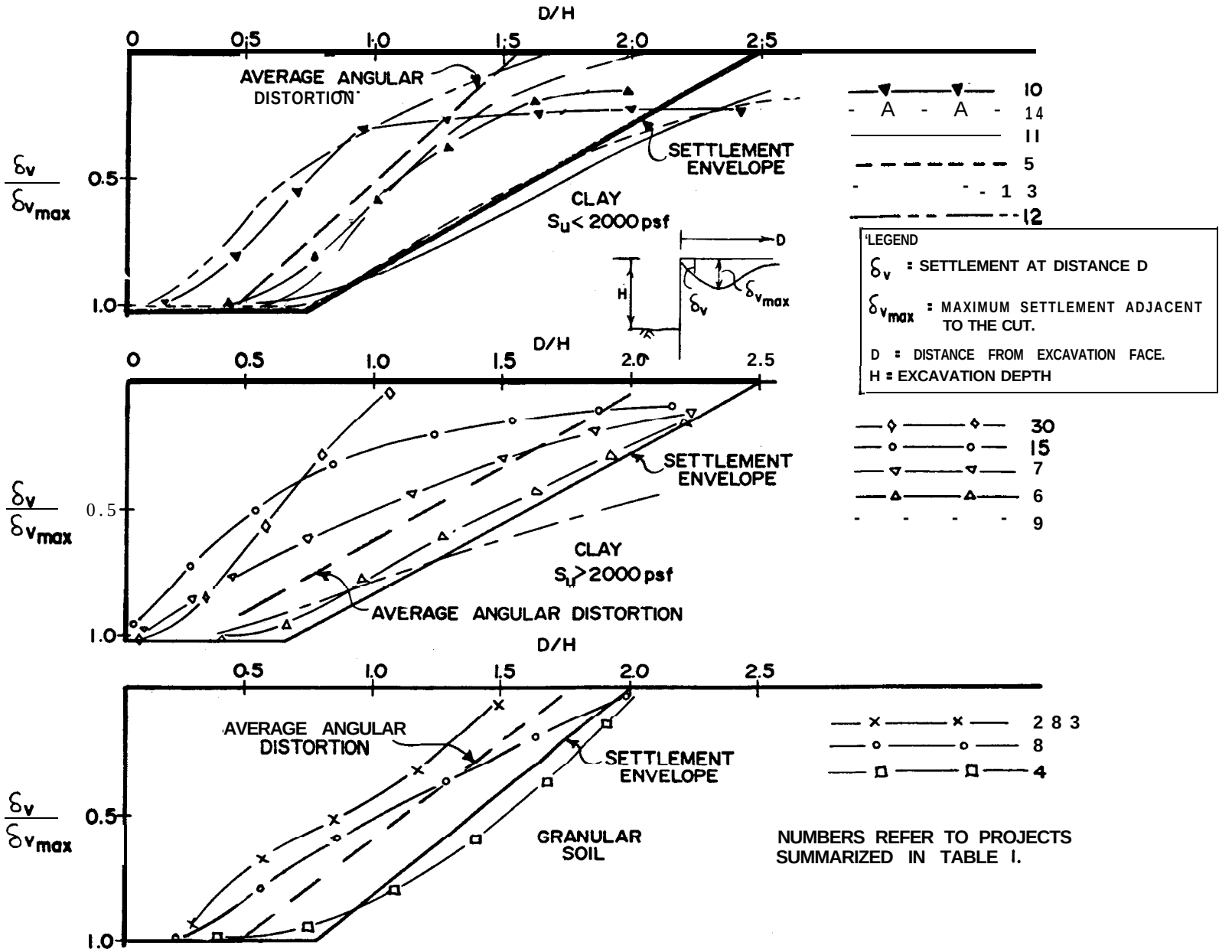


Figure 10. Normalized settlements adjacent to a wall.

Figures 11 and 12 show the wall deformations assumed and the corresponding settlement profiles predicted by the finite element program.

Two soil conditions were analyzed:

(a) Normally consolidated clay with both the soil strength and soil modulus increasing with depth.

(b) Elastic medium where the soil was assigned a constant modulus with depth.

Figure 11 illustrates settlement profiles for the ideal cases of tilting about base, rotation about top, and pure flexure. The first two conditions may be considered representative of rigid wall behavior, whereas the bulging cases represent deformations associated with a flexible wall.

The results indicate that for tilting about base and flexure the settlements are concentrated within a distance one-half the excavation depth. On the other hand, when rotation is the predominant mode of deformation, significant deformations may occur at distances up to 1.5 times the excavation depth from the excavation face.

Figure 12 shows the settlement profiles for wall deformations which are a combination of rigid wall displacement plus flexural deformations. The results show the zone of influence is greatly affected by the nature and volume encompassed by the horizontal wall movement.

The zone of influence demonstrated by finite element analysis ranged between 0.5H and 2.0H from the excavation face. This is consistent with data from field measurements (Figure 10) with the exception that field data are influenced by consolidation of softer cohesive soils. Consolidation settlements, which extend the zone of influence out further, are not accounted for in a finite element analysis.

The effect wall movement has on the zone of influence is another significant trend. Figure 11 and Figure-12 both show the importance of minimizing movement below the excavation base.

1.60 LATERAL DEFORMATIONS IN ADJACENT SOIL MASS

Tied-Back Walls in Heavily Overconsolidated Clay

Normalized contour plots of horizontal deformations are presented in Figures 13 and 14,

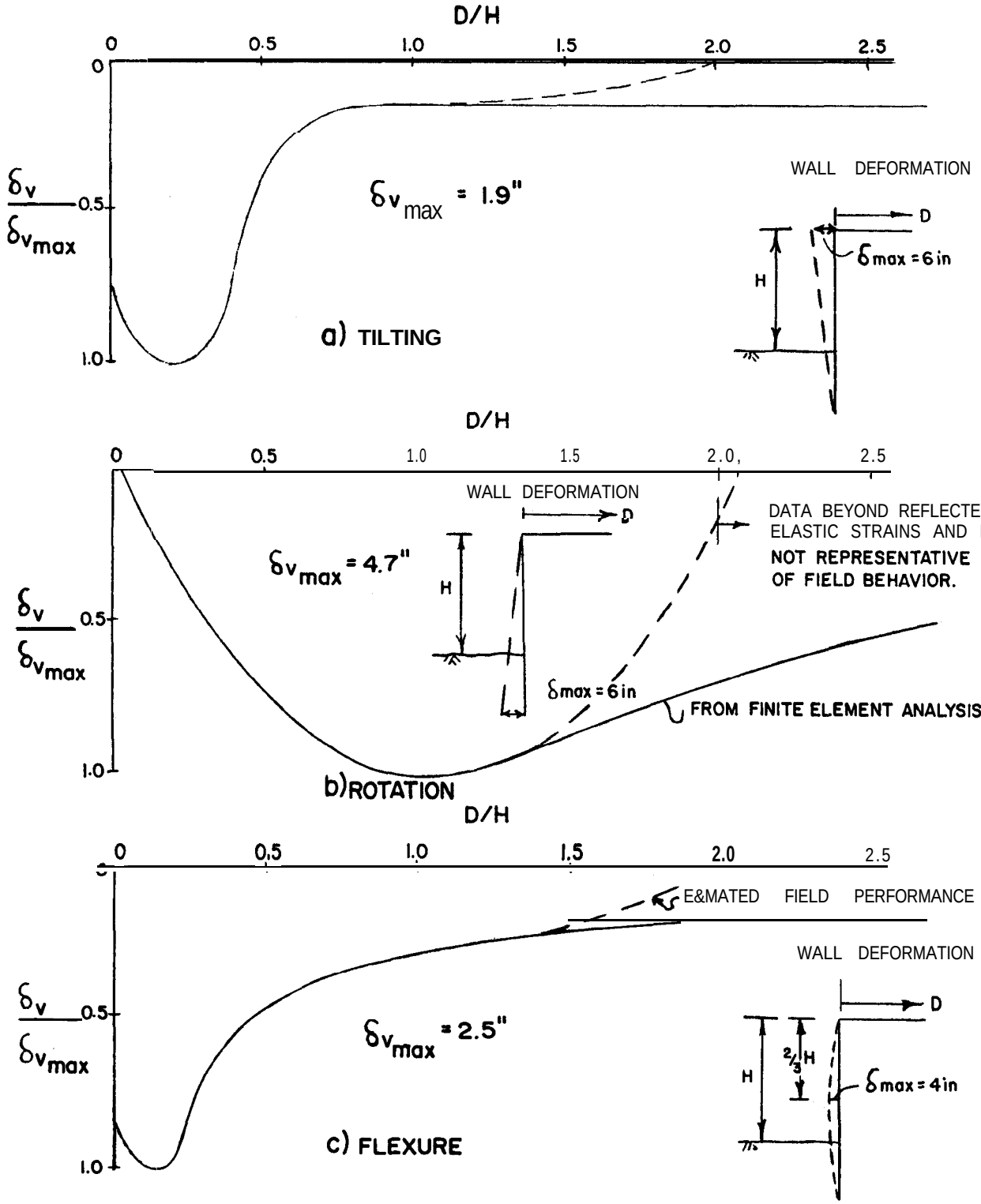


Figure 11. Finite element prediction of surface settlement profiles for normally consolidated clays,

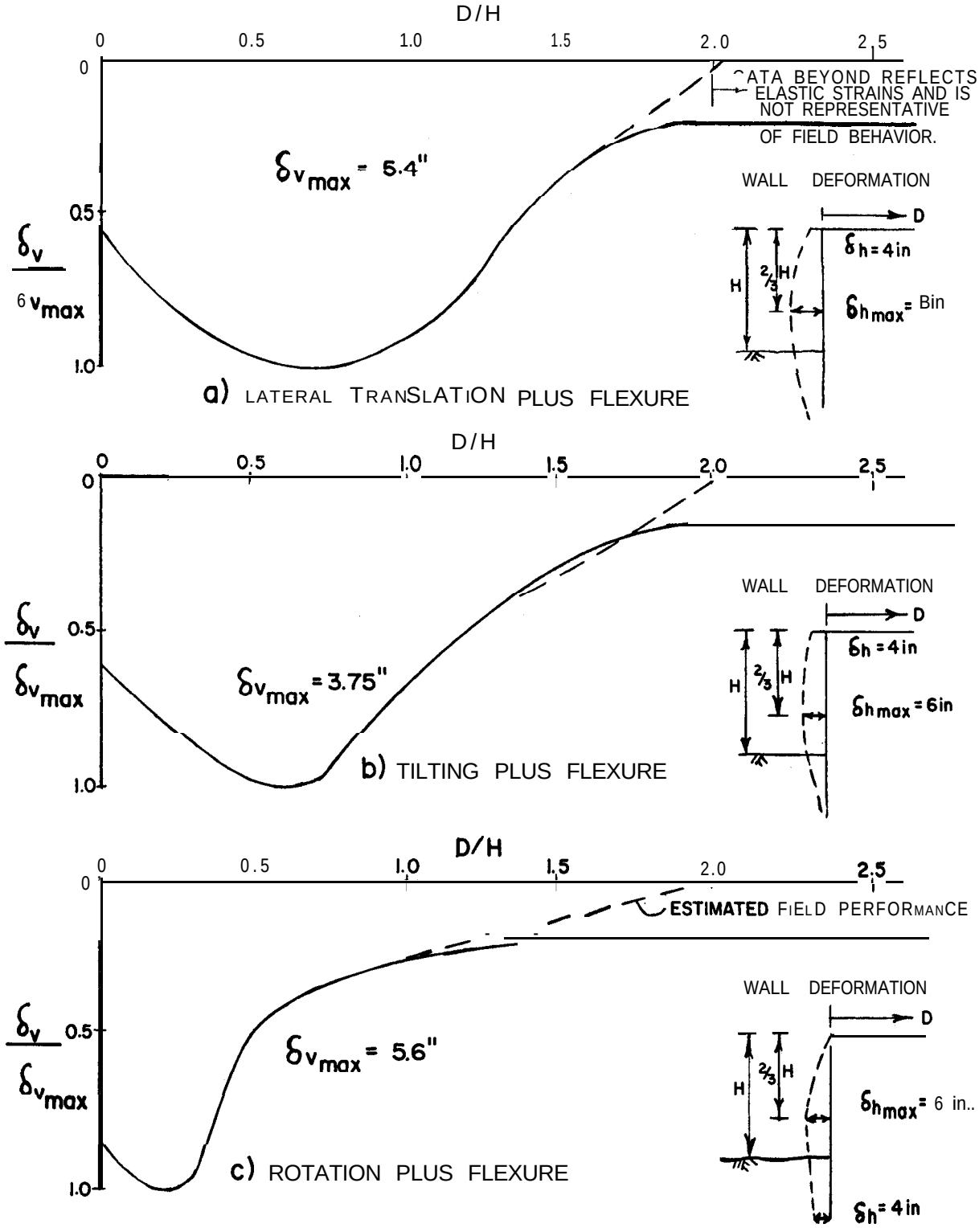


Figure 12. Finite element prediction of surface settlement profiles for normally consolidated clays.

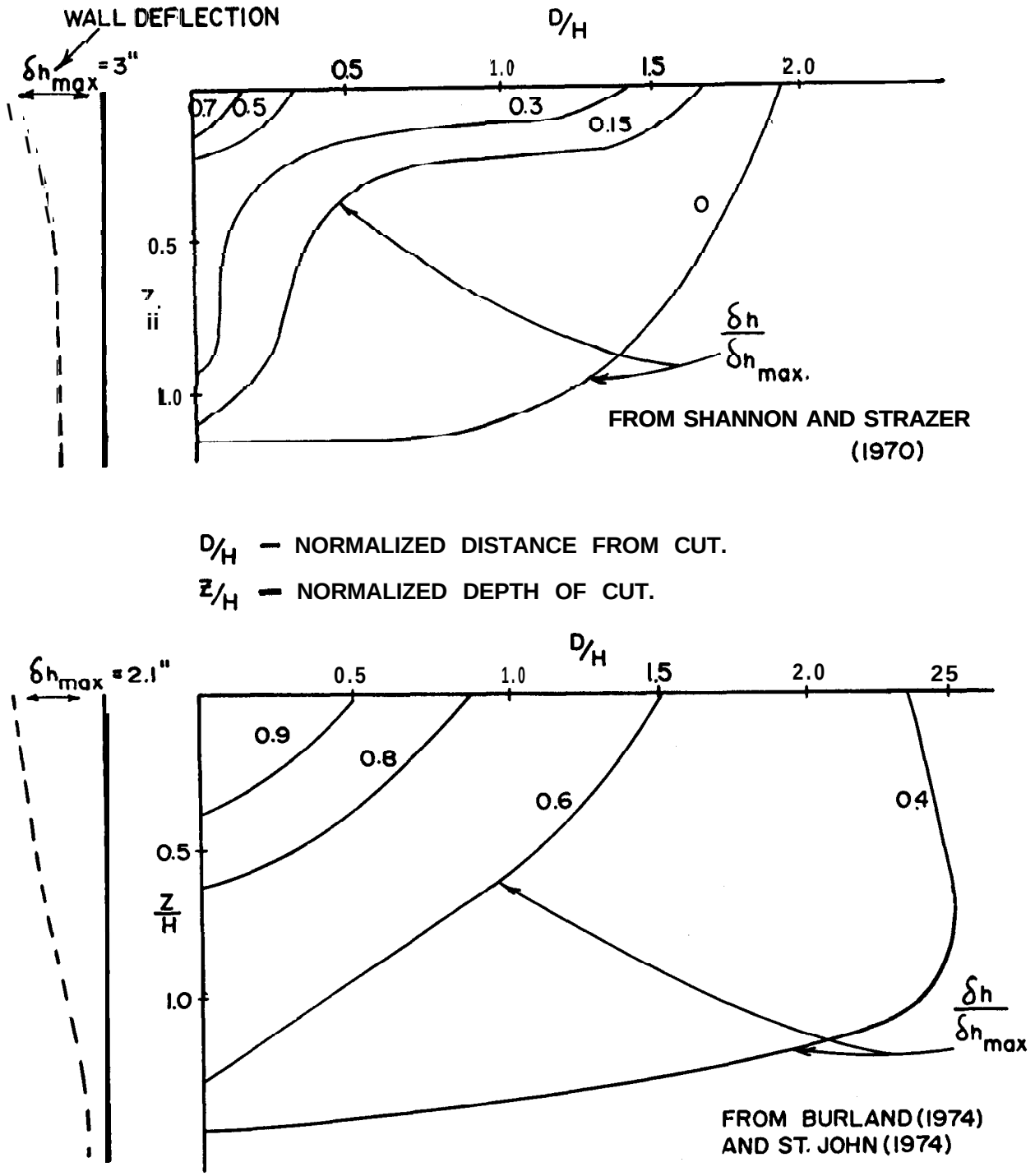


Figure 13. Normalized lateral movement for tied-back excavation in heavily overconsolidated clays.

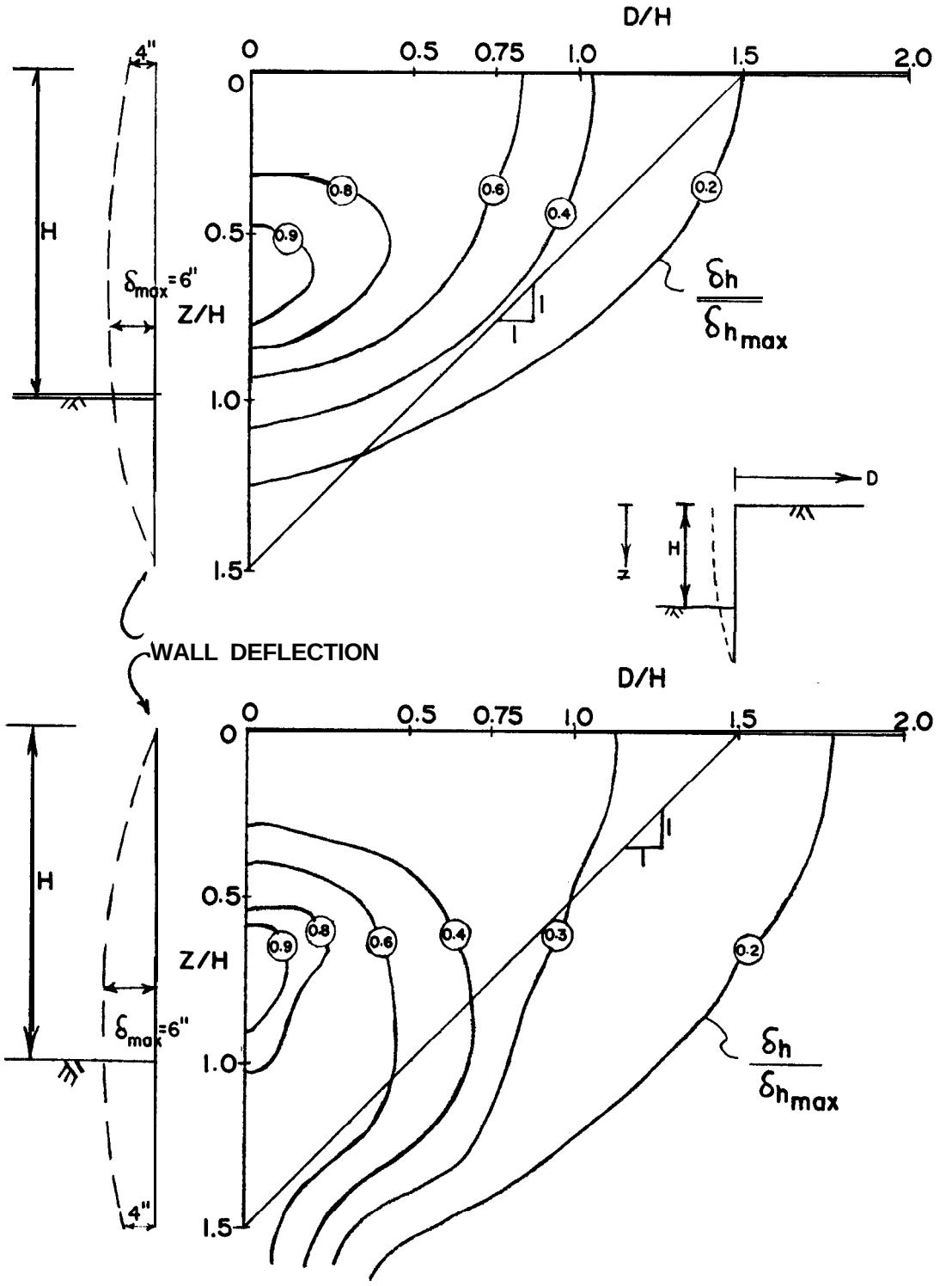


Figure 14. Normalized later al movements from finite element analysis for normally consolidated clays.

The aforementioned field data suggest two trends. First, the pattern of the lateral movement follows closely with the deflected shape of the sheeting. Second, the lateral movements can extend a substantial distance from the excavation face, and may involve general movement of the soil mass embodied by the tiebacks. Discussion of several case historties is made in Volume II (Design Fundamentals).

In overconsolidated clays and shales the movement is believed to be associated with lateral expansion following stress relief from the excavation. A weak layer below the excavation would add to the magnitude of movement.

Walls in Normally Consolidated Clay

There are little field data available regarding the distribution of horizontal displacements for excavations in a normally consolidated clay for comparison with the observed data for the heavily overconsolidated clays. Therefore, the results of the finite element studies used to develop Figure 12 were reduced to provide some insight to the distribution which might be expected for ideal conditions. These results, shown in Figure 14, indicate that the zone of significant movement is confined to an area described by a 1 on 1 slope from the base of the sheeting. As expected, it is within the theoretical yield zone. The movements are largely controlled by the sheeting displacement, with the zone of significant movements increasing with depth in the same pattern as the sheeting movements,

1.70 EFFECT OF CONSTRUCTION PROCEDURES

It is well known that construction procedures can have a significant effect on the performance of excavations.

Lowering of the ground water level either by pumping or by seepage into the excavation can result in significant settlements. These settlements could be associated with consolidation of the soil or, in the case of granular soils, the piping of soil into the excavation.

Poor installation techniques for tiebacks or struts can lead to surface settlements. Tiebacks should be carefully drilled to minimize the soil removed from holes, Also, any voids remaining after the tieback is installed should be filled with grout. Struts, rakers, and wales should be tightly wedged and preloaded to prevent movement of the wall. Earth berms when used to provide temporary support before installing a strut have been observed to be of little value in preventing wall movement.

Even though the entire support system may be in place, the sides of the excavation may continue to creep inward with time. This problem appears to be particularly acute in tied-back walls in very stiff to hard clays. There is also some evidence to indicate that lagging in soldier pile walls tends to pick up more load with time in all soils. Excessive bulging or even failure of some lagging has been observed.

1.80 ESTIMATING SETTLEMENTS

The data presented in this section may be used to obtain rough estimates of the ground movements which might occur adjacent to a support wall. The reason for making this estimate is to provide some additional input to aid in the decision of whether or not to underpin adjacent structures or utilities.

Settlements may be estimated using both Figure 3 and Figure 10. Once the soil type and excavation geometry are defined, an estimate of the maximum settlement may be made from Figure 3. Figure 10 provides a means of estimating the angular distortion and zone of influence of the ground movements. In the case of cohesive soils, Figure 5 may be used to estimate the wall stiffness necessary to limit the settlements.

2.10 GROUND WATER CUTOFF

General

Cutoff walls are used for the following purposes:

1. To avoid or to minimize dewatering of the excavation.

2. To lessen or to prevent lowering of ground water level outside the excavation.

3. Because it may be impractical to place lagging in soils that are extremely difficult to dewater in advance of excavation (such as silts and/or dilatant clayey sands).

4. To cut off pervious water bearing strata within or just below the bottom of the excavation; thus, protecting against the possibility of a "blow" condition or other source of ground loss.

Soldier Pile Wall

Inherently, a soldier pile and lagging wall is not watertight. If ground water is to be controlled it must be done by dewatering or by grouting or freezing. In "running" soils it is essential to maintain the ground water level below the working face in order to prevent in flow and subsequent ground loss.

Interlocked Sheeting

Provided that the steel sheet pile wall remains intact and penetrates into an underlying impervious stratum, the effectiveness of sheet piling as a cutoff will be very significant in pervious sands and gravel. On the other hand, in granular soils of low permeability (for example:, silty or clayey sands) interlocked sheeting will have relatively little effect on the flow into the excavation. In all cases, however, sheeting effectively cuts off flow in pervious interbedded layers, which in the case of soldier pile walls, may lead to ground loss at the face.

With regard to maintaining ground water level outside the excavation, interlocked sheeting is effective in pervious granular soils. For relatively impervious soils (such as clayey sands, silts, and clays) the sheet piling is essentially equivalent to the permeability of the soil and therefore, will have little or no effect on the seepage pattern toward the excavation or on lowering of piezometric levels.

The above discussion applies only to intact sheeting. The presence of boulders, difficult driving conditions, or obstructions can lead to ripping of the sheeting and/or jumping out of interlocks which will seriously impair if not destroy the effectiveness of the cutoff wall.

Another common problem is when the effectiveness of a cutoff in pervious soil depends upon achieving a tight seal on rock. This situation may be especially acute when rock occurs within the depth of excavation because of the threat of ground loss below the tips of the sheeting.

Concrete Diaphragm Walls

For all practical purposes, a well constructed concrete diaphragm wall is essentially impermeable. It will effectively cut off flow and prevent ground water lowering outside the excavation provided there is penetration into an underlying impervious formation.

2.20 SEEPAGE PATTERN TO EXCAVATION FACE

As mentioned previously, interlocked steel sheeting has relatively little influence on the seepage pattern in impervious soils. As a result, when cuts are made below ground water there will be flow to the face of the excavation. In clays, such a flow will be so small that it may not even be noticeable.

An example of a flow net for this type of situation is shown in Figure 15. During the initial process of excavation, deformation in the soil will generate shear strains and cause pore pressure changes. Eventually, these pore pressures will be dissipated and a steady state seepage pattern will develop as shown in the figure.

The equipotential lines shown in the figure demonstrate the changes in hydrostatic stress. Such changes in hydrostatic stress lead to a time dependent equivalent change in effective stress and consolidation of the soil. In soft normally consolidated clays or organic soils the associated amount of consolidation can be significant and will contribute to displacements behind the excavation.

The foregoing case is important because if illustrates that steel sheeting may not be effective in preventing consolidation of normally consolidated soils within depth of cut. Soil compressibility and rate of consolidation must be considered.

It would not be possible to recharge the ground water level in the cohesive soils of this example. Diaphragm walls should be considered in cases where there is a need to prevent displacement.

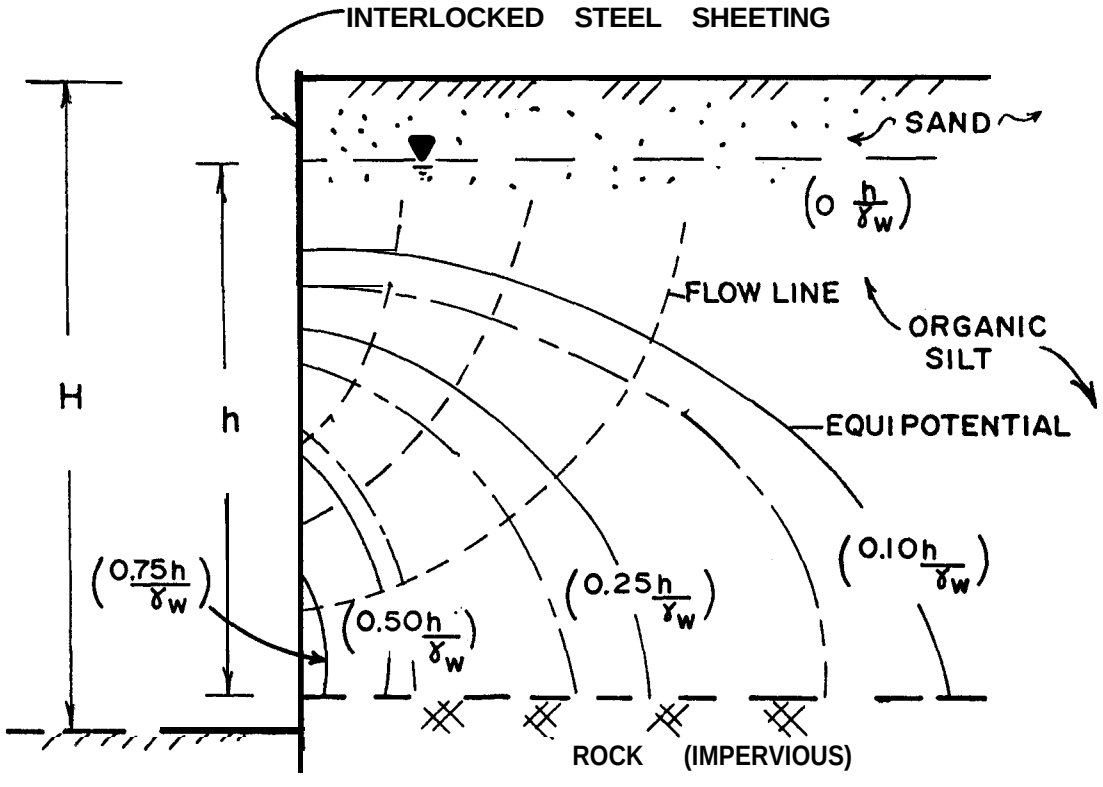


Figure 15. Change in pressure head for cut in impervious soil.

3.10 BASIC CONSIDERATIONS

Earth Pressure at Rest

The ratio of the geostatic horizontal to vertical effective stress of a natural soil formation is defined as:

$$K_o = \frac{\bar{\sigma}_h}{\bar{\sigma}_v}$$

where:

K_o = coefficient of earth pressure at rest

$\bar{\sigma}_h$ = horizontal effective stress

$\bar{\sigma}_v$ = vertical effective stress

For granular soils Terzaghi and Peck (1968) suggest K_o values of 0.5 for loose deposits and 0.4 for dense soils. Generally, K_o can be estimated for normally loaded soil deposits as:

$$K_o = 1 - \sin \bar{\phi}$$

where:

K_o = coefficient of earth pressure at rest

$\bar{\phi}$ = angle of internal friction in terms of effective stress.

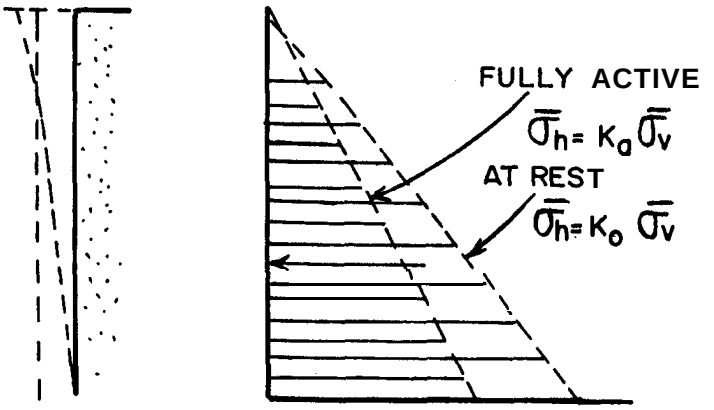
For cohesive soils, K_o is primarily dependent on the overconsolidation ratio (OCR). Normally consolidated clays typically have K_o values of 0.5 to 0.6; lightly overconsolidated clays (OCR $\leq$ 4) have K_o values up to 1; heavily overconsolidated clays (OCR $\sim$ 16) K_o may range up to a value of 2.

Active Earth Pressure

Lateral displacement (as shown in Figure 16) transforms the state of stress in the ground from the at-rest condition to the active condition. The mechanics of this process are the mobilization of full shear resistance within the soil mass -- a state of stress referred to as "plastic equilibrium".

Figure 16 shows the active earth pressure distribution associated with displacement modes. The fully active state stems from lateral translation, from rotation about the bottom, or from a combination

(a) FULLY ACTIVE



(b) ARCHING ACTIVE

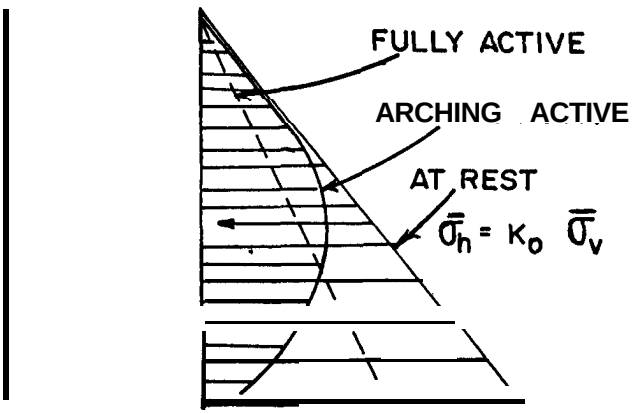


Figure 16. Earth pressure distributions for active and arching active conditions.

of both. The earth pressure distribution is triangular and the resultant occurs at the third height of the wall.

The direction and magnitude of active pressure depends upon whether or not there is wall friction. The particular case of horizontal surface and zero wall friction is the Rankine fully active condition, shown in Figure 16. For this case, the active stress acts horizontally on a vertical wall. The Rankine coefficient of active earth pressure Ka is the ratio of the effective stress.

$$\frac{\bar{\sigma}_h}{\bar{\sigma}_v} = \frac{\bar{\sigma}_a}{\bar{\sigma}_v} = K_a$$

For sands, $K_a = \tan^2 (45^o - \bar{\phi}/2)$

For cohesive soils,

General case ($\bar{\phi}$, c):

$$K_a = \tan^2 (45^o - \bar{\phi}/2) - \frac{2c}{H} \tan (45^o - \bar{\phi}/2)$$

Special case ($\phi = 0$, $c = S_u$):

$$K_a = 1 - \frac{2su}{H}$$

wher e :

K_a = coefficient of active pressure

$\bar{\phi}$, c = friction angle and cohesion intercept

$\bar{\sigma}_v$ = vertical effective stress

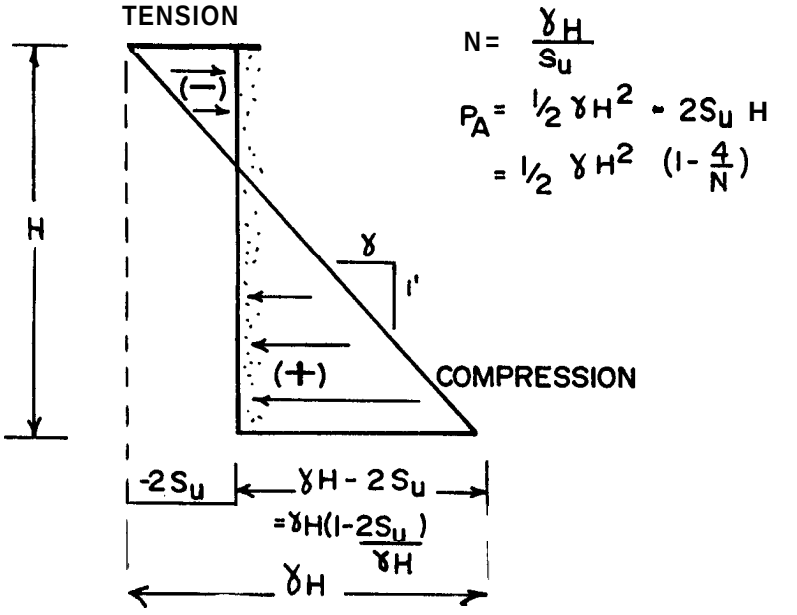
$\bar{\sigma}_h$ = horizontal effective stress

$\bar{\sigma}_a$ = active earth pressure (horizontal)

S_u = undrained shear strength ($\phi = 0$ case)

According to the Rankine expression, the pressure distribution for cohesive soils is theoretically in tension in the upper part of the wall as shown on Figure 17a. Frequently, adhesion simply does not

(a) RANKINE ACTIVE PRESSURE DISTRIBUTION IN COHESIVE SOILS

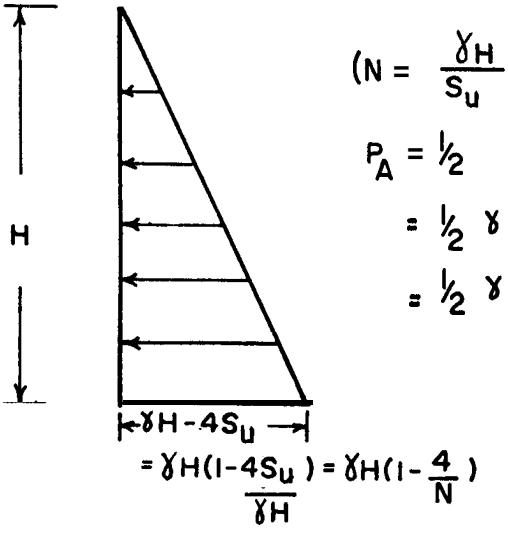


$N = \frac{\gamma H}{S_u}$

$P_A = \frac{1}{2}\gamma H^2 - 2S_u H$

$= \frac{1}{2}\gamma H^2 \left(1 - \frac{4}{N}\right)$

(b) TRIANGULAR PRESSURE DISTRIBUTION EQUIVALENT TO NET RANKINE FORCE



$\left(N = \frac{\gamma H}{S_u}\right)$

$P_A = \frac{1}{2} H \cdot (\gamma H - 4 S_u)$

$= \frac{1}{2}\gamma H^2 - 2S_u H$

$= \frac{1}{2}\gamma H^2 \left(1 - \frac{4}{N}\right)$

Figure 17 **Earth pressure distribution for cohesive soil (ϕ = 0).**

(or cannot) develop and therefore tension cannot occur. However, the net total lateral force on the wall is equivalent to that described by subtracting the "negative" pressure at the top from the positive pressure at the bottom. Assuming that this net force increases linearly with depth of wall, it can be represented by a net pressure diagram with a triangular distribution of the same force magnitude as shown on Figure 17b. The ordinate at the base of the wall is:

$$\bar{\sigma}_a = \gamma H - 4S_u$$

3.20 INTERNALLY BRACED COFFERDAMS

General

Initially, the internal bracing is set near or at the top, thus restraining inward displacement. With each stage of excavation and bracing there will be progressive inward displacement below previously placed braces. The net displacement profile typically takes the form shown in Figure 18 (after Bjerrum, et al, 1972).

Characteristically, there will always be some inward rotation about the top, at least in the upper portion of the cut. The degree of bulging and displacement. below the cut depends upon several factors -- the distance between braces, the stiffness of the wall, and the stiffness of soils near the base of the wall. In general, the resulting deformation pattern most closely resembles the arching active condition. Therefore, a parabolic, rather than triangular, pressure distribution is most Likely to act on the wall.

Figure 19 shows the conventional procedure for analyzing empirical load data. The resulting apparent earth pressure diagrams are used to develop an envelope encompassing the maximum distributed pressures. This design envelope then represents the maximum strut load that can be anticipated at any stage of construction.

Design Earth Pressure Diapram

Apparent earth pressure diagrams suggested by Terzaghi and Peck (1968) for design of braced walls are shown on Figure 20. Strut loads for a given Level are determined by reversing the procedure used for development of the diagram. A strut is designed to support a load described by the area between the mid points of the adjacent upper and lower support levels.

The following discussion does not include the effect of surcharge (see Section 3.40).

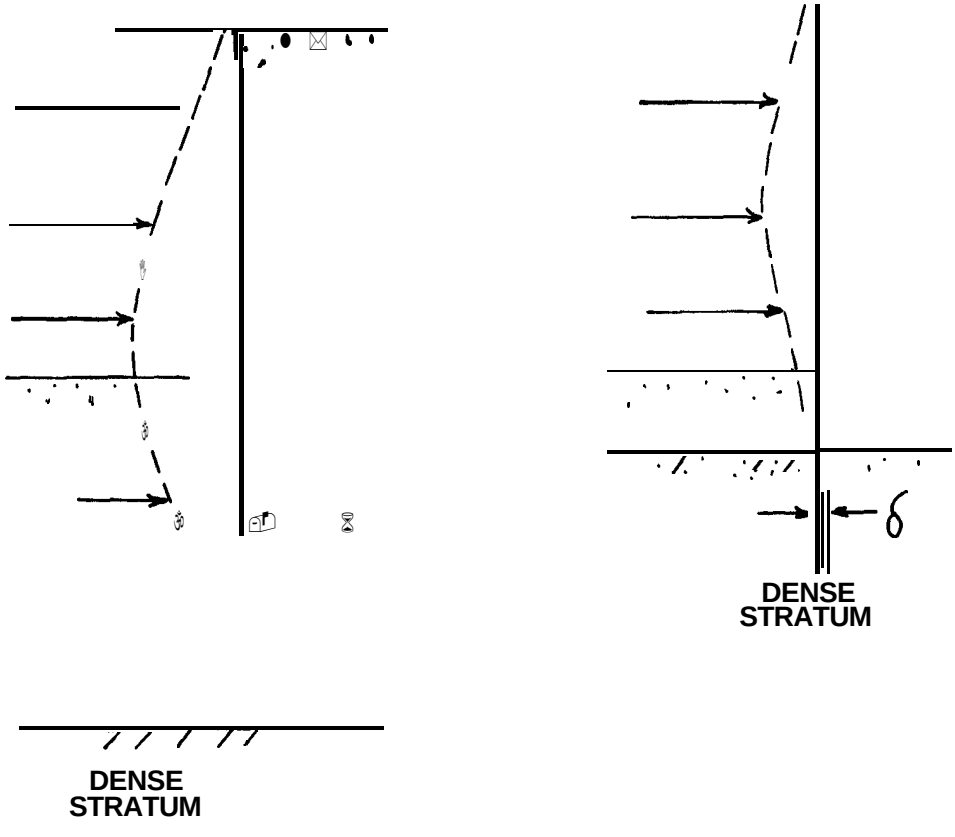


Figure 18 **Mode of deformation of internally.braced cofferdam (after Bjerrum, et al, 1972).**

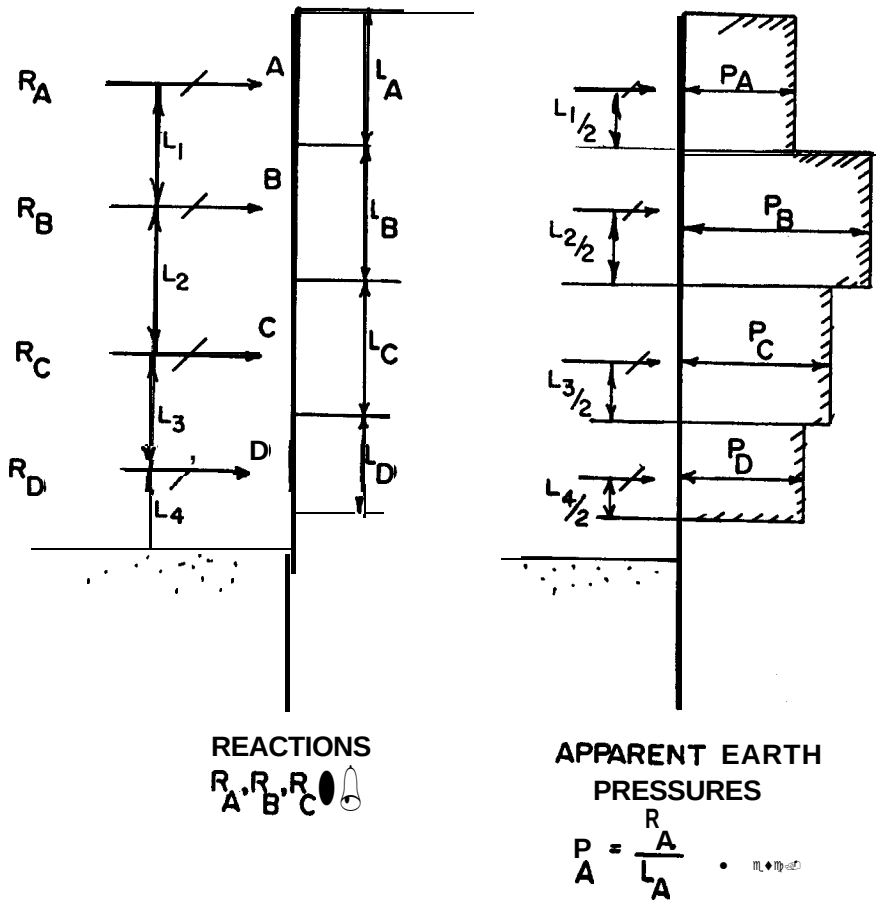


Figure 19 **Conventional procedure for development of earth pressure diagram,**

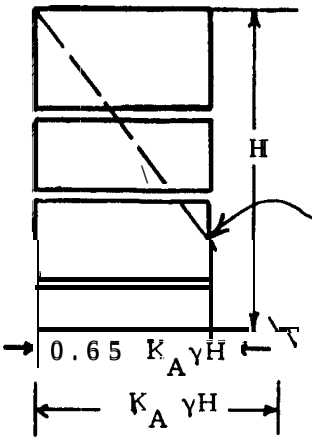
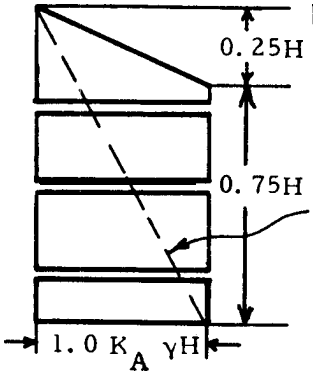
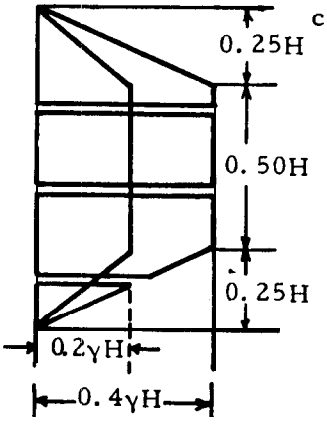
PRESSURE DISTRIBUTION	TOTALFORCE
 a) Sands $K_A = \tan^2(45 - \phi/2)$ Rankine Active	P_t = Trapezoid P_A = Rankine $Pt = .65 K_A \gamma H^2$ $P_A = .50 K_A \gamma H^2$ $\frac{P_t}{P_A} = 1.30$
 b) Soft to Medium Clays (N > 6) For clays, base the selection on $N = \frac{\gamma H}{S_u}$ Equivalent Rankine Active $K_A = 1 - m\frac{4s_u}{\gamma H} = 1 - \frac{4}{N}$ m = 1.0 except where cut is underlain by deep soft normally consolidated cla	m = 1.0 $P_t = .875\gamma H^2(1 - \frac{4}{N})$ $P_A = .50\gamma H^2(1 - \frac{4}{N})$ $\frac{P_t}{P_A} = 1.75$
 c) Stiff-Clays For N < 4 For 4 < N < 6, use the larger of diagrams (b) and (c).	$P_t = .15yH^2$ to $.30\gamma H^2$ $\frac{P_A}{N} = 4$, $P_A = 0$ N < 4, $P_A < 0$ NOTE: Equivalent Rankine Active = 0

Figure 20 Design earth pressure diagram for internally braced flexible walls (sands, soft to medium clays, stiff fissured clays), from Terzaghi and Peck (1968).

a. Sands : This diagram, which was developed from dewatered sites applies to cohesionless soils. If the soils outside the excavations remain submerged, then the earth pressure should be computed using the buoyant unit weight of the soil. Hydrostatic pressures are treated separately and added to the effect of the earth pressure.

b. Soft to Medium Clays: The recommended earth pressure diagram for these soils is shown in Figure 20b.

The value of 'm' used in the determination of the ordinate for earth pressure applies to situations where the cut is underlain by a deep deposit of soft clay. Its value can only be determined by empirical means from measurements and performance of an actual excavation. Experience thus far, reported by Peck (1969) from cases in Mexico City and in Oslo, Norway, leads to the conclusion that the value of 'm' is on the order of 0.4 for sensitive clays. For insensitive clays the value of 'm' may be taken as 1.0.

c. Stiff Clays : The recommended apparent earth pressure diagram for stiff clays is used when the stability number, N, is Less than 4. This empirical diagram is independent of the value of shear strength; the lateral earth pressure is a function of the gravity forces only.

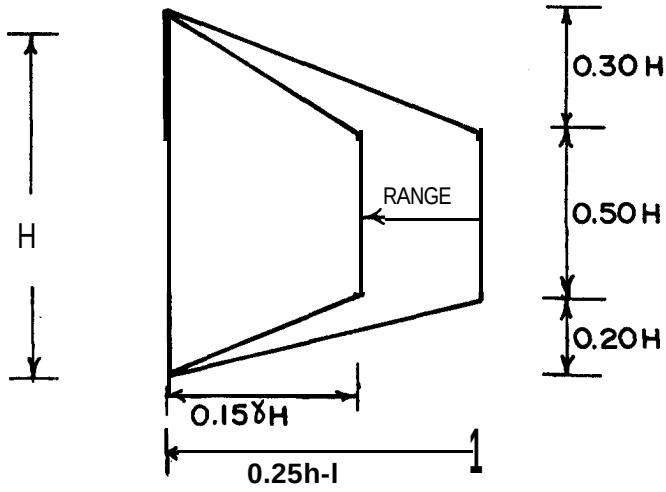
d. Heavily Over consolidated Very Stiff Fissured Clay: Several cases have been reported which suggest that stress relief from excavation leads to lateral deformation of these soils toward the excavation. Soil behavior would suggest that the potential for expansion increases with increasing overconsolidation ratio, increasing plasticity of clay, and for cuts below the water table with the intensity of fissuring in the soil. For strutted excavations this condition may lead to build up of strut load with time.

Design criteria for cases involving potentially laterally expansive soils are as yet undeveloped. Therefore, a Laboratory test program (possibly stress-path triaxial) should be undertaken to aid in evaluating the magnitude of the problem. Prototype test sections with construction monitoring are also recommended.

e. Dense Cohesive Sand; Very Stiff, Sandy Clay: Recommended design diagrams for dense cohesive sands and very stiff sandy clays are shown in Figure 21. The minimum pressure Line is associated with cuts less than about 30 or 40 feet deep, with reasonably consistent spacing between wale levels and relatively uniform soil conditions. The maximum pressure line is recommended to cover uncertainty

(a) RELATIVELY UNIFORM CONDITIONS

$P_t = 0.112\gamma H^2$ TO $0.188\gamma H^2$



(b) UPPER THIRD OF CUT DOMINATED BY COHESIONLESS SOIL

$P_t = 0.135\gamma H^2$ TO $0.225\gamma H^2$

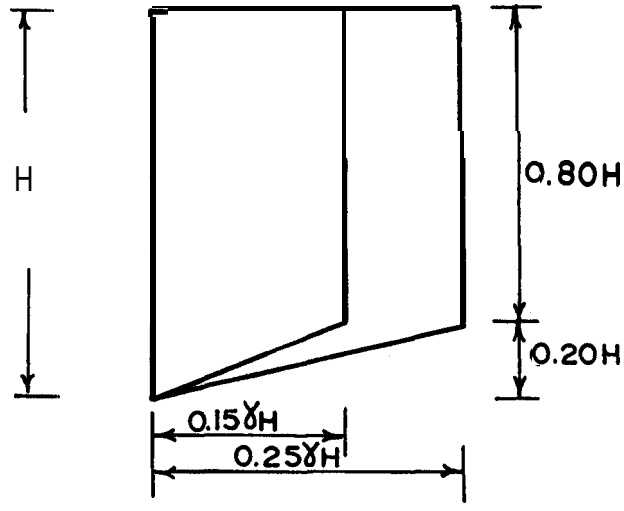


Figure 21. Proposed pressure diagram for internally braced flexible walls (dense cohesive sands, very stiff sandy clays).

regarding the effect of weak strata within the depth of cuts, contingencies arising from construction (for example, over -excavation below support level, or ineffective toe berms), and cuts in excess of 60 or 70 feet deep.

Cohesive soils near the top of the cut will justify pressure reduction as shown in Figure 21a. Absence of cohesive soils near the top of cut will require the higher pressures associated with Figure 21b.

f. Stratified Soils : The aforementioned cases apply to readily idealizedoil profiles. Actual soil conditions may have a stratigraphy which does not conveniently match these simplified cases. Moreover , an irregular ground surface or surcharge may complicate the analysis.

Under such circumstances, one approach is to determine the Lateral thrust either on the basis of classic active earth pressure or on the basis of trial planar sliding surfaces and wedge stability analysis. In this latter case the most critical wedge is used to determine the lateral thrust (see Chapter 6). In such cases, hydrostatic forces are treated separately.

Once the Lateral thrust is determined, it should be increased by the most appropriate value of P_t/P_A (ratio of force from the empirical diagram to the force determined from the analysis of active earth pressure or wedge equilibrium). The designer must choose the most appropriate ratio based upon a comparison of the actual case to one of the simplified cases presented in this section,

The final question is one of pressure distribution. Initially, the designer must compare the actual case with the simplified cases. Serious questions may need field measurements to provide data input during construction.

3.30 TIEBACKS

Background

Many practitioners have successfully applied the empirical rules developed for internally braced walls to tiebacks; others make variations for tied-back installations. In any event, at the present time there are no empirical methods for tied-back walls that have been accepted as universally as Peck's rules for internally braced flexible walls.

Recommendations for Tiebacks

The following discussion does not include the effect of surcharge (see Section 3.40).

Only limited documentation is available to quantify conclusions concerning the relative magnitudes of appropriate pressure envelopes for tiebacks and bracing. Accordingly, there is no present justification for a major departure from loading on internal bracing. In general, the force magnitude of the following proposals is similar, or the same, but the distribution has been changed slightly.

Soil type classifications are the same as for internal bracing, namely: sands , soft to medium clays, stiff clays, and finally, dense cohesive sands or very stiff sandy clays. A triangular pressure distribution, increasing linearly with depth, is recommended for soft to medium clay; a uniform pressure distribution is recommended for all other cases.

a. Sands: Where deformations are critical, and it is intended to prestress to 100 percent of design load, compute force using K_o. For dense sands $K_o = 0.4$; for loose sands $K_o = 0.5$. Thus, the uniform ordinate will vary from:

$$\text{Uniform Pressure, } p = 0.20\,\gamma H \text{ to } 0.25\,\gamma H$$

$$\text{Force, } P_t = 0.20\,\gamma H^2 \text{ to } 0.25\,\gamma H^2$$

Where deformations are not cirtical, use $K_{avg} = \frac{K_o + K_a}{2}$, that is, a coefficient midway between active and at rest. A similar procedure was used by Hanna and Matallana (1970).

b. Stiff to Very Stiff Clays: Use a uniform pressure ordinate varying from $0.15\,\gamma H$ to $0.30\,\gamma H$ to produce the same force magnitude as that for braced excavations. The higher value is associated witka stability number of about 4. The lower number is associated with very stiff clays where the stability number is less than 4. The force varies as follows:

$$\text{Stiff clays, } P_t = 0.30\,\gamma H^2$$

$$\text{Very stiff clays, } P_t = 0.15\,\gamma H^2$$

c. Cohesive Sand, Very Stiff Sandy Clays: Compute the total force associated with the diagram for braced excavations (Figure 2) and distribute uniformly with depth. Relatively uniform conditions :

$$\text{Force, } P_t = 0.112\ \gamma H^2 \text{ to } 0.188\ \gamma H^2$$

$$\text{Uniform Pressure, } p = 0.112\ \gamma H \text{ to } 0.188 \gamma H$$

Upper third of cut dominated by cohesionless soil:

$$\text{Force, } P_t = 0.135\ \gamma H^2 \text{ to } 0.225\ \gamma H^2$$

$$\text{Uniform Pressure, } p = 0.135\ \gamma H \text{ to } 0.225\ \gamma H$$

d. Soft Clays: It is unlikely that tiebacks would be used unless they could be embedded in an underlying denser stratum of soil or or in rock. Design with a triangular earth pressure diagram assuming at rest conditions and a K_o value between 0.5 and 0.6.

$$\text{Force, } P_t = 0.25\ \gamma H^2 \text{ to } 0.30\ \gamma H^2$$

In normally consolidated, sensitive clays, excessive prestressing should be avoided because of the potential for induced consolidation.

e. Stratified Soils: As with braced excavation, an approach based upon active earth pressure or wedge equilibrium should be investigated.

3.40 SURCHARGE LOADING

General Background

Surcharge near excavations may be the result of many different types of loading conditions including footings, structures, storage of construction materials, or traffic. The lateral pressure caused by a surcharge load on a retaining wall has been investigated for a variety of different loading and soil conditions (Spangler, 1940; Newmark, 1942; Terzaghi, 1954b). This pressure is in addition to the normal earth and water pressure.

Theoretical Considerations

The four basic loading conditions for which solutions of the lateral stresses in an elastic medium are readily available are:

1. Point Loading

2. Uniform line loading

3. Irregular area loading

4. Uniform area loading

Practical Considerations

With regard to surcharge loading from construction operations, it is common to take a distributed surface surcharge on the order of 300 psf to cover storage of construction materials and general equipment. Usually, this surcharge should be considered within a rather limited work area on the order of 20 feet to 30 feet from the coffer dam wall.

A second major consideration is the question of concentrated loads from heavy equipment (concrete truck, cranes, etc.). Lateral thrust from such equipment would easily be covered within the 300 psf sur charge, provided that the equipment were more than approximately 20 feet from the wall. On the other hand, such equipment within a few feet of the wall may create a concentrated surcharge loading which would be of far greater significance than a uniform surcharge loading. This must be accounted for separately.

Point Load and Line Load

Solutions, summarized by Terzaghi (1954b) are shown in Figures 22 and 23.

Irregular Area Loading

Figure 24 shows an influence chart for evaluating the lateral stresses acting on a rigid wall due to a rectangular loading (Sandhu, 1974). These charts assume a Poisson's ratio of 0.5 for the soil mass. Using the influence charts for point loadings, the lateral stress due to an irregular sur char ge loading can be calculated more easily.

Uniform Area Loading

The solution for lateral stresses on a rigid wall is presented in Figure 24. An example of the stress effect with depth is shown in Figure 25. Note that the stress influence below a depth of about 1.5B is negligible.

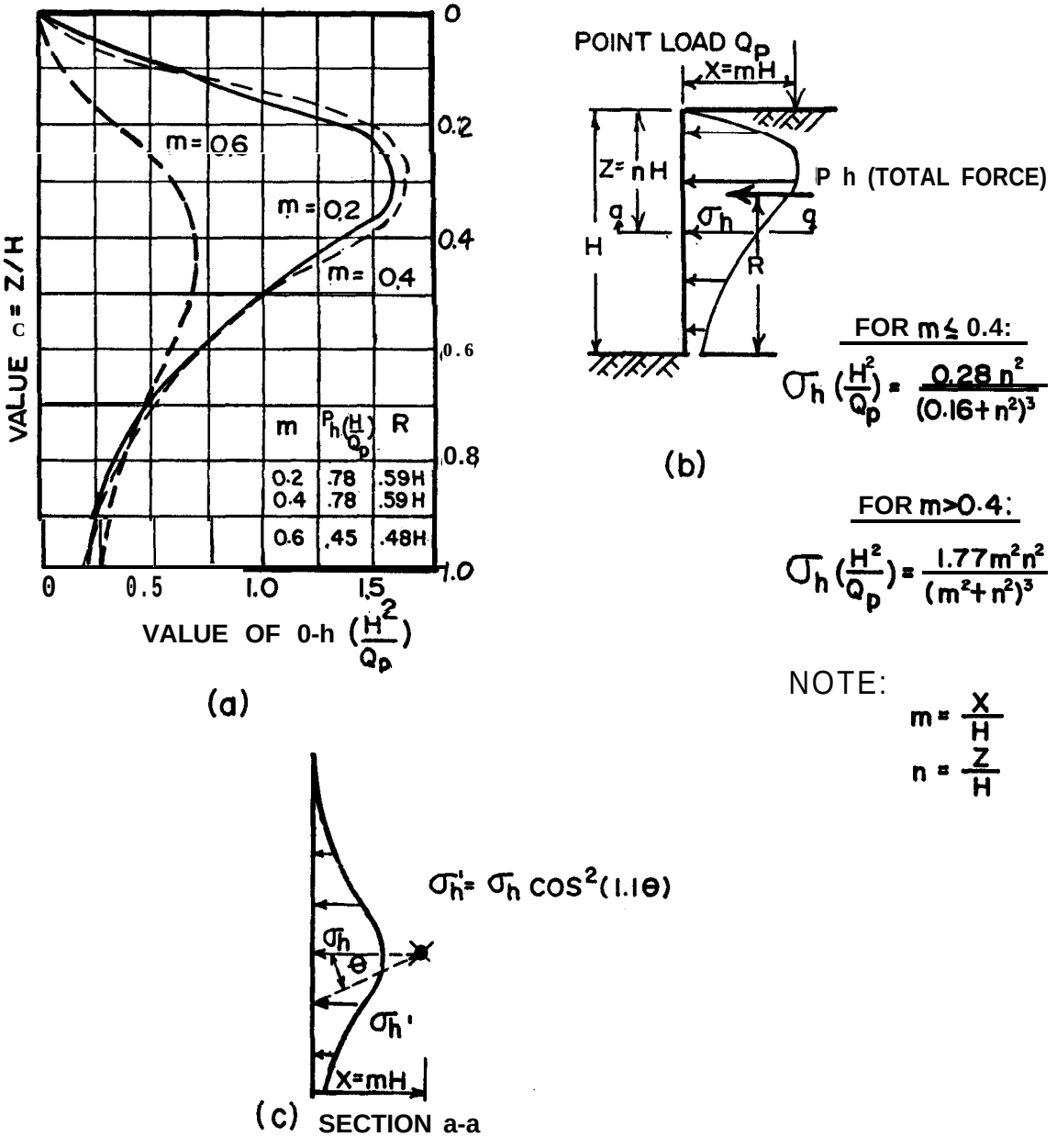


Figure 22 Lateral stresses on the face of an unyielding wall from a point loading (NAVFAC, 1971 and Terzaghi, 1954b).

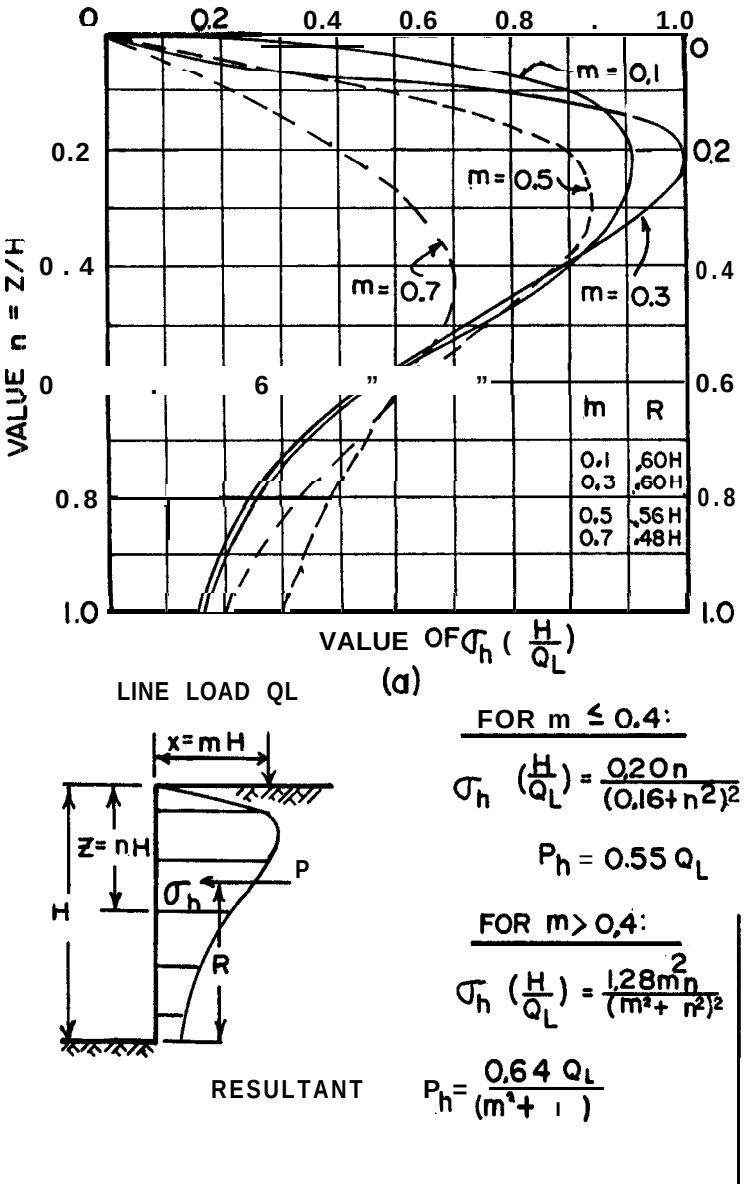


Figure 23 **Lateral stresses acting on an unyielding wall from a uniform line loading** NAVFAC, **1971 and Terzaghi, 1954b).**

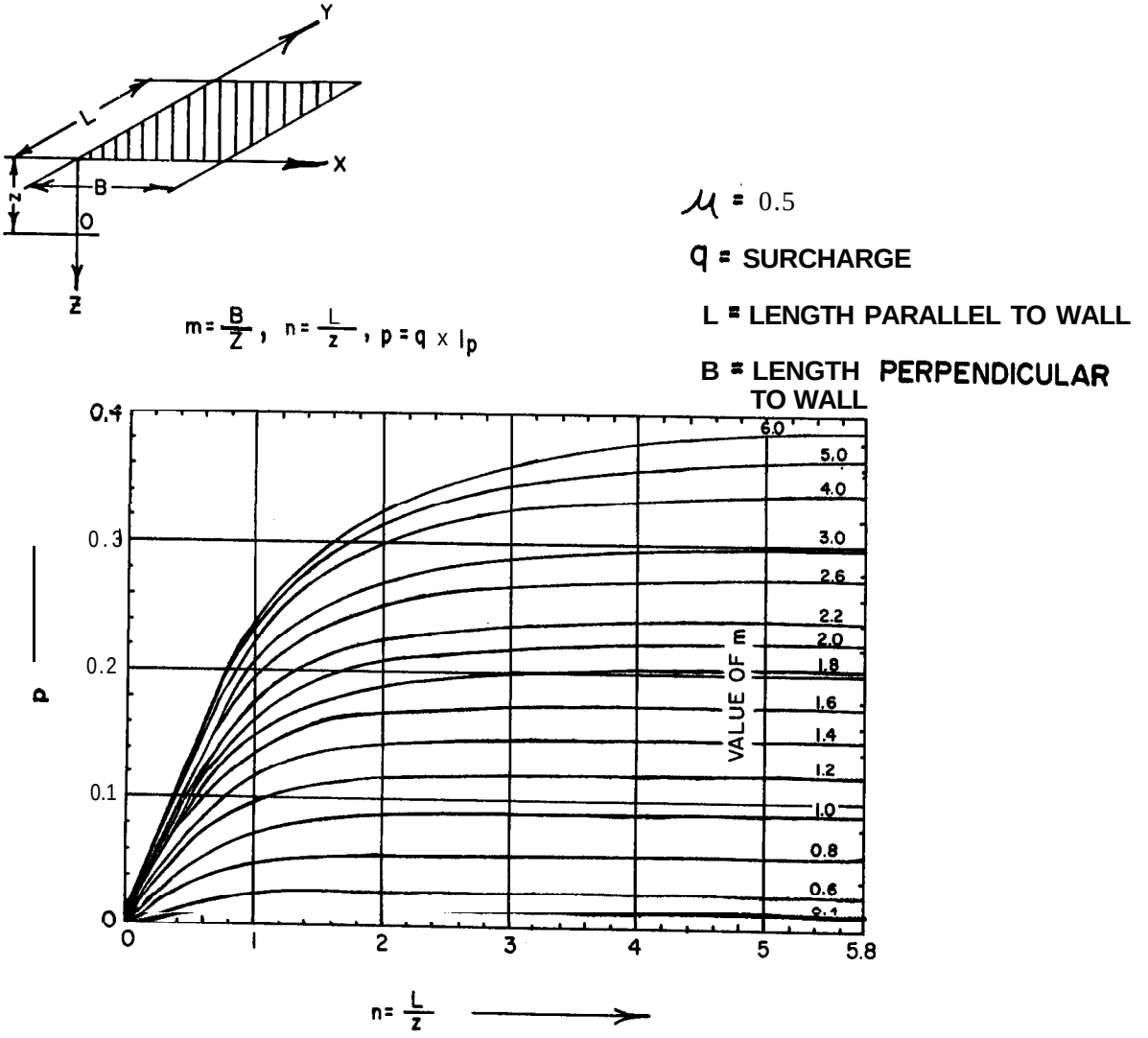


Figure 24 Lateral stresses on an unyielding wall due to irregular surface loading (Sandhu, 1974).

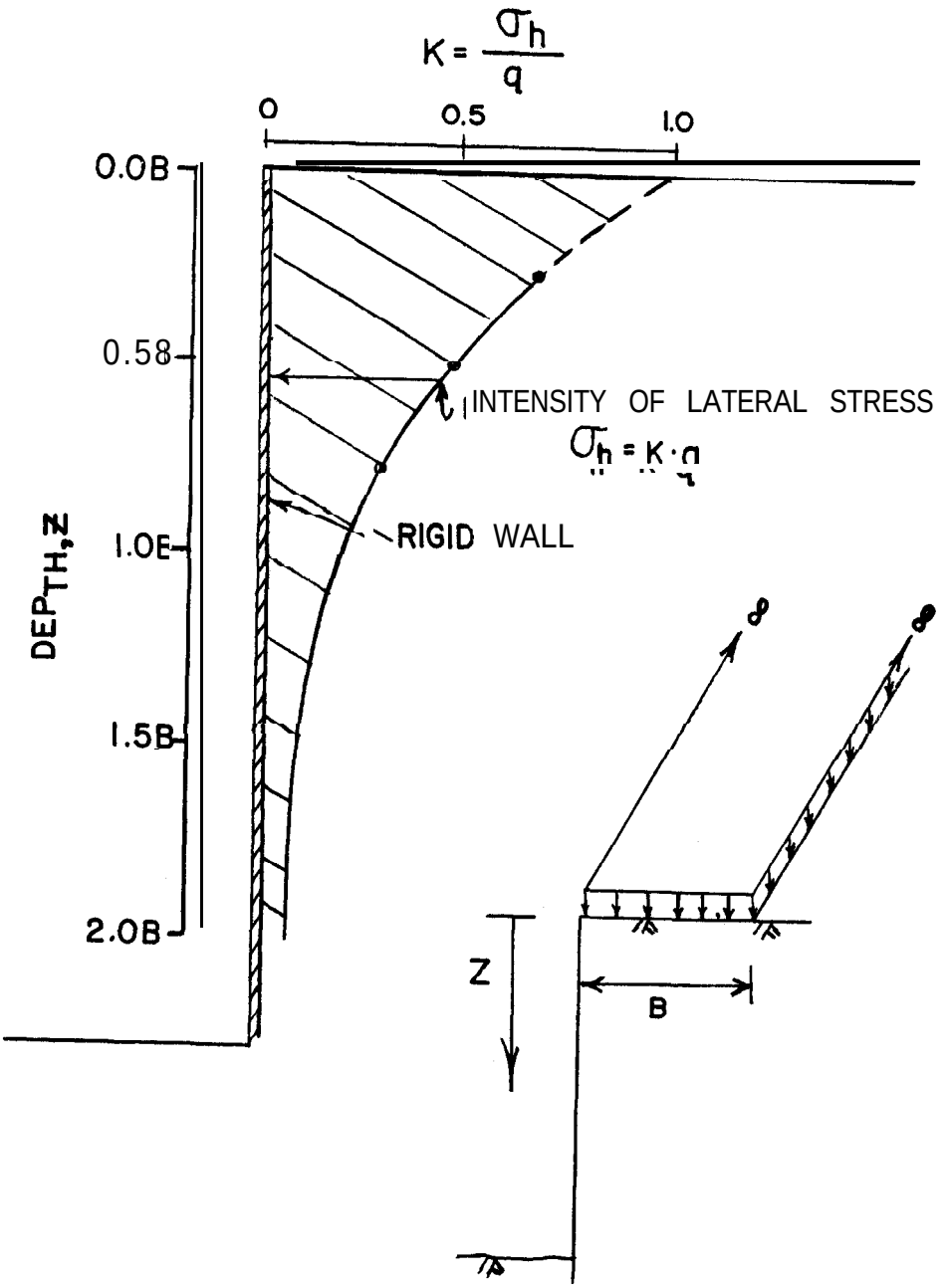


Figure 25. Lateral stress on rigid wall from surcharge of width B and infinitely long (solution from Sandhu, 1974).

A second approach is to apply an earth pressure coefficient, K, to the surcharge loading and to consider the surcharge effective within some portion of the cut. The magnitude of this coefficient will range from K_a (active earth pressure) to K_o (earth pressure at rest).

4.10 GENERAL

The design should provide that the soils below the base of an excavation mobilize sufficient passive resistance for force equilibrium or for limiting movement. The performance of the wall will depend upon the spacing of the support levels since the greater the spacing, the greater the passive resistance (and movement) that will be required below the lowermost support level. Figure 26 illustrates the case of a wall in which the passive resistance of the soil is insufficient to limit excessive wall movement.

This section deals with the selection of soil parameters and methods used to evaluate passive resistance. It does not deal with the depths of penetration required to maintain overall stability of the earth mass or to limit displacements in the earth mass.

4.20 SOIL PARAMETERS

Granular Soil

Granular soils are free draining and cannot sustain positive or negative pore pressures generated by strain or load changes for even a short period of time. Therefore, analyses of the stability of granular soils are performed on the basis of drained strength parameters and effective stresses in the ground. The appropriate soil strength parameter is the angle of internal friction, ϕ. For design, granular soils are assumed to have no cohesive strength component,

Cohesive Soil

Because of the load decrease from excavation, soils in the passive zone just below the excavation will initially experience a pore pressure decrease, Pore pressure may become negative. With time, the pore pressure will rise. This may be accompanied by heave, caused by swelling of the soil.

Limiting case strength parameters for passive pressure computation are:

a. Immediate Condition: Pore pressures generated by unloading and strain do not have time to dissipate. Use undrained strength of soil S_u at natural water content. Conventionally, this is determined from vane shear, unconfined compression, or unconsolidated-undrained compression tests.

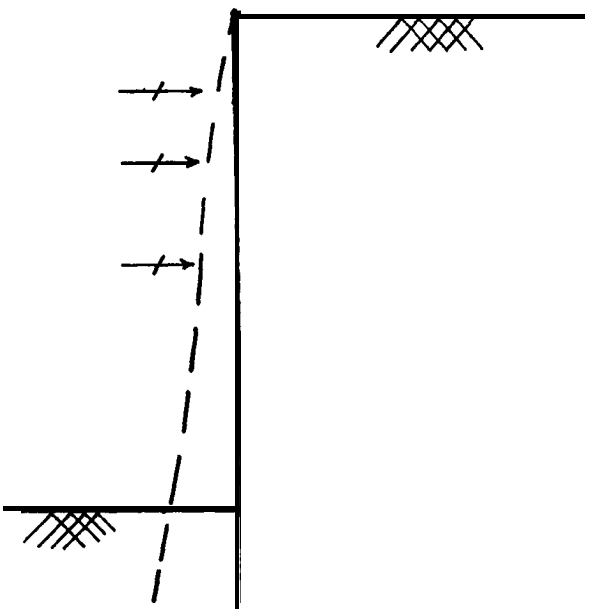


Figure 26 Movement at wall base due to insufficient passive resistance.

b. Ultimate Condition: Pore pressures generated by unloading and strain are dissipated by drainage. Effective stresses can be computed on the basis of static water levels. Use strength parameters from the effective stress envelope, $\bar{c}$ and $\bar{\phi}$.

General recommendations for strength relationships are to use undrained strength for the "during excavation" stage, and effective stress strength parameters for the final construction condition. Greater accuracy in determining strength values can be obtained by measuring pore pressures during construction and by appropriately modifying the strength values (either undrained strength or drained strength).

For overconsolidated soils, the undrained strength at natural water content may be greater than the drained strength. Therefore, indiscriminate use of undrained strength without regard to pore pressure dissipation may be unsafe.

4.30 ANALYSIS OF PASSIVE RESISTANCE

Several articles and texts address the problem of passive pressures that can develop behind a continuous wall (Terzaghi, 1954b; NAVFAC, 1971). In cohesionless soil, wall friction modifies both the direction and magnitude of the, passive resistance . Typically, the resultant of the passive pressure acts at an angle δ equal to 1/2 to 2/3 of the angle of inte rnal friction. The following table (from Terzaghi, 1954b) summarizes values of ϕ and δ .

ϕ	$\delta = 0$	$\delta = \phi/2$	$\delta = 2/3\ \phi$
25^o	2.46	3.00	3.20
30^o	3.00	4.20	4.80
35^o	3.70	6.50	7.30
40^o	4.60	9.20	11.00

The passive pressure for drained loading or in terms of effective stress at depth, z, will be:

$$\bar{\sigma}_p = \bar{\sigma}_v \tan^2 (45^o - \bar{\phi}/2) + 2\bar{c} \tan (45^o + \bar{\phi}/2)$$

where:

$\bar{\sigma}_p$ = passive pressure (effective stress)

$\bar{\sigma}_v$ = vertical effective stress = $\gamma_m z - u$

$\bar{\phi}$ = angle of internal friction (effective stress envelope)

c = cohesion intercept

For the above drained condition, in which by definition there is no excess pore pressure, the total stress at any depth, z, will be

$$\sigma_h = \bar{\sigma}_p + \gamma_w z$$

where:

σ_h = lateral stress

γ_w = unit weight of water

The passive resistance of cohesive soils in an undrained condition should be evaluated on the basis of the undrained shear strength, S_u, and the in situ total vertical stress, σ_v. For a continuous wall, the passive pressure at a given depth will equal.

$$\bar{\sigma}_p = \sigma_v + 2 S_u$$
$$= \gamma z + 2 S_u$$

where:

σ_v = total vertical stress = $\gamma_m z$

S_u = undrained shear strength of the soil

In this case, the water pressure is not added separately because pore pressure is, already accounted for in the determination of the undrained strength, S_u.

Soldier pile walls are not continuous walls, therefore the passive earth pressure coefficients must be modified from those used for continuous walls. Broms (1965) showed the passive resistance of laterally loaded piles based on pile width and on K_p values for continuous walls was too conservative, His study showed that soil arching and non-plane strain conditions increase the capacity of individual piles. Broms.! recommendations are given in the charts shown in Figure 27. It should be noted that for cohesive soils the lateral resistance of the soil should be neglected to a depth of 1. 5 Pile diameters. In cohesionless soils where the depth of penetration is greater than one pile diameter, soil arching causes an effective increase of 3.0 in the value of K_p.

A factor of safety of 1. 5 is recommended for use in passive pressure calculations.

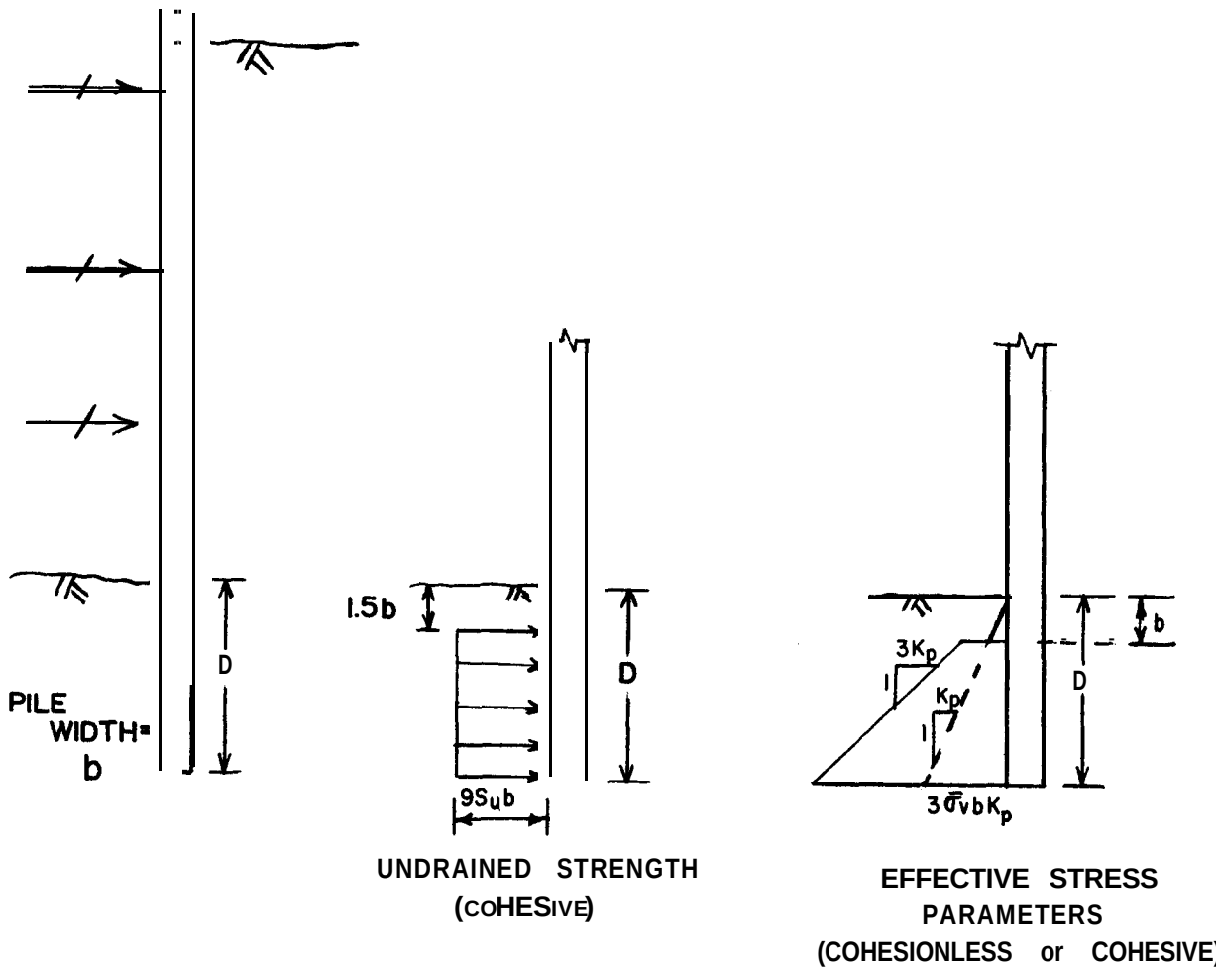


Figure 27 Passive pressure for soldier piles (after Broms, 1965). (Modified.)

4.40 OVERCUT DESIGN DETAILS

Over-excavation below the required support level depth is common either to obtain working room or to muck up the bottom. During intermediate excavation phases assume a minimum of two feet of overcut before strut placement. At final depth assume a minimum of one foot of overcut.'

4.50 BERMS

Lateral resistance of berms will, of course, be lower than the lateral resistance of a horizontal plane at the top elevation of the berm. One method of analysis is wedge or logarithmic spiral force equilibrium of trial failure surfaces. Another procedure is to replace the berm with an equivalent sloping plane and assign the appropriate passive coefficient (Terzaghi and Peck, 1968; NAVFAC, 1971).

5.10 ANALYSIS OF WALES AND SUPPORT WALLS

General

Deflection of structural members supporting soil causes arching of earth resulting in a reduction of pressure near the center of spans and a concentration of pressure at the supports. Hence, the actual bending moments in wall elements and wales is less than that which would be computed assuming a uniform loading on these flexural members.

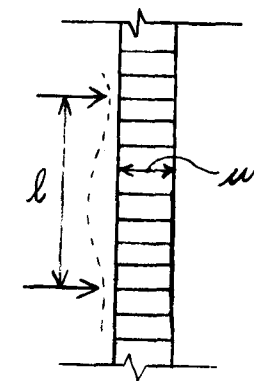
The approach used herein, for moment computation in wales and support walls, is to apply a uniform pressure equal to 80 percent of the loading diagram. For evaluation of loads in internal bracing and tiebacks, the full loading diagram (100 percent) is used. (See Figure 28).

When rigid walls support the earth, arching will be minimal; therefore, structural design of the wall as well as other elements should be based on the full pressure diagram.

Continuous Members

The following expression should be used for computing moments over continuous members (either wall member or wale) with uniformly applied loads :

$$M = C\, w\, l^2$$



where:

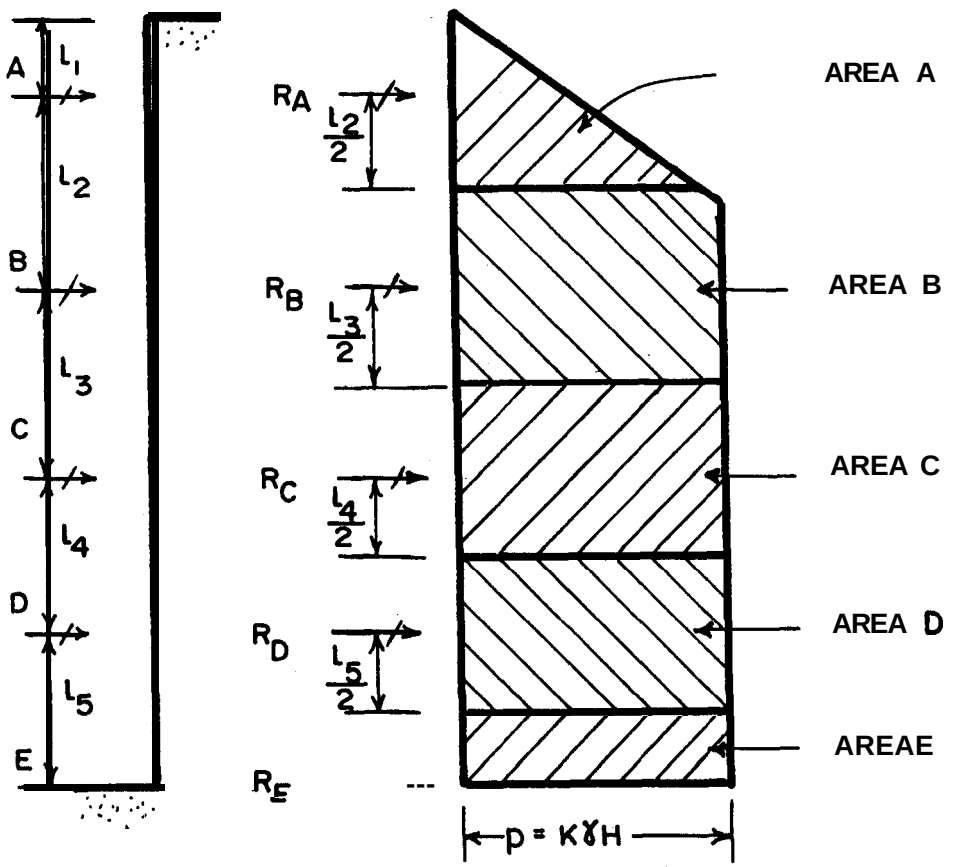
M = moment

C = moment coefficient

w = distributed load on span

1 = span length

Since construction methods greatly influence the position of the elastic line of members (especially vertical members), there is no practical way that the moment can be precisely analyzed. Therefore, a coefficient of C = 0.10 is recommended for continuous members supporting a uniform distributed load.



STRUT LOAD PER LINEAL FOOT OF WALL. IS EQUAL TO DESIGNATEDAREA

EXAMPLE: $R_C = p\left(\frac{l_4}{2} + \frac{l_3}{2}\right)$

Figure 28. Load determination from apparent earth pressure diagram.

Discontinuous Wales

The moment in the wale will depend on the splice detail. For splices which occur at a strut and which tie the wale together with a steel strap, which transfers shear but not moment, zero moment should be assumed at that point.

Wales supporting uniform load with moment splices over less than three spans should not be considered continuous. Three spans or more should be considered continuous using a moment coefficient, C = 0. 10.

The moment in wales supporting concentrated loads (as from soldier piles or tiebacks) should be calculated on the'basis of statics. Assume full continuity where moment splices are used; assume zero moment in other splices.

Member Connections

It is common to design splices for the full structural capacity of the member (both shear and moment). This is often done with a combination of fully penetrating butt welds and cover plates.

Figures 29, 30, and 31 show some typical details for splices and wale to strut connections. For splices that are butt welded it is often assumed that the butt weld is only 50 percent to 75 percent effective since the beveled edges at the splice are field cut. Hence, the cover plates are designed to carry 25 percent to 50 percent of the member capacity. In designing a strut to wale connection, stiffness must be provided to prevent web crippling. Also, if raked struts are used, a knee brace is required at the strut to prevent buckling of the wale from the vertical component of load.

Lagging

The determination of lagging size is largely based on the past experience. See Chapter 9 (Soldier Pile Walls).

5.20 BRACING AND TIEBACKS

Bracing and tieback loads must be determined for the most critical construction condition. This may be at an intermediate depth of cut or at full depth.

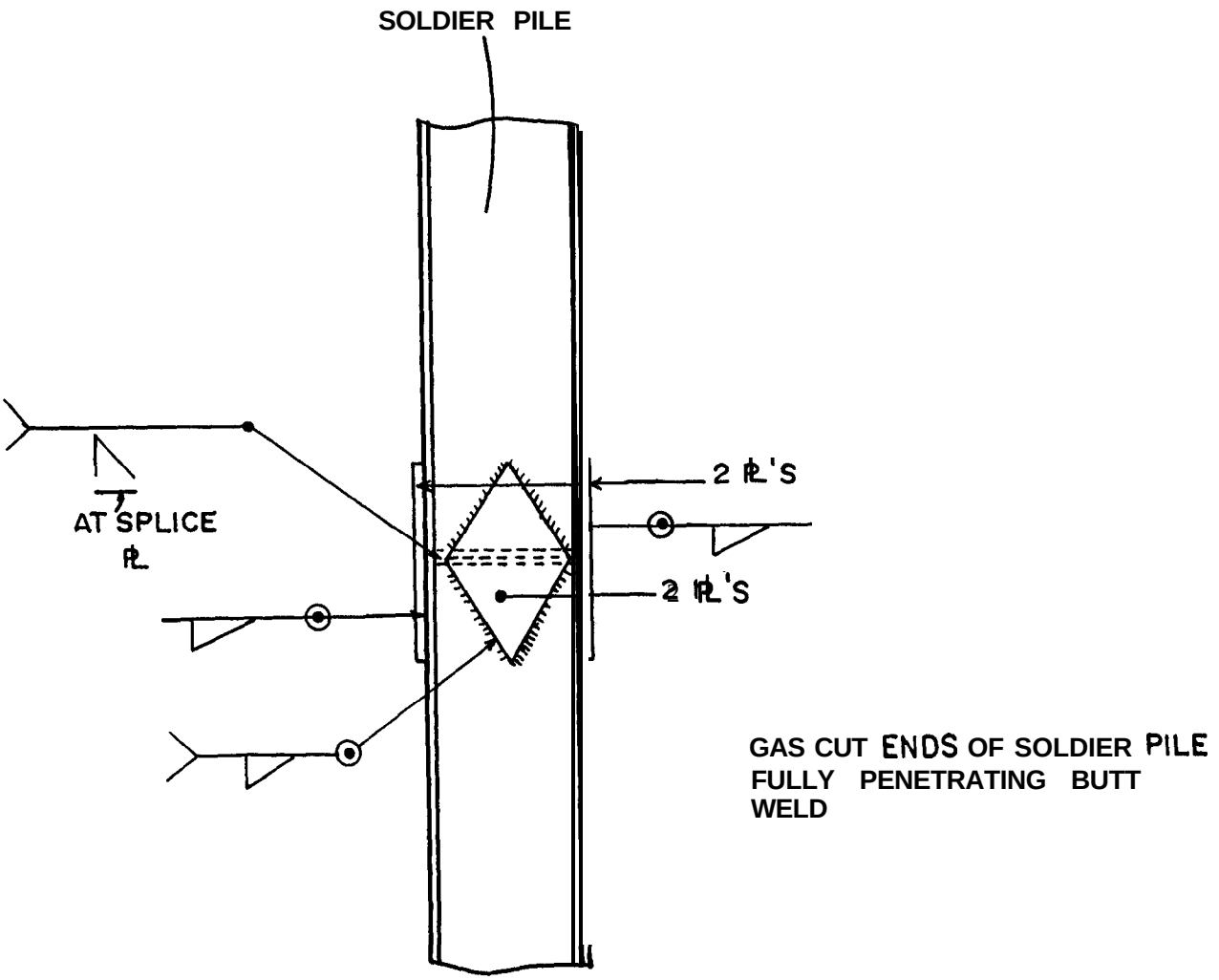


Figure 29. Typical splice with butt welding,

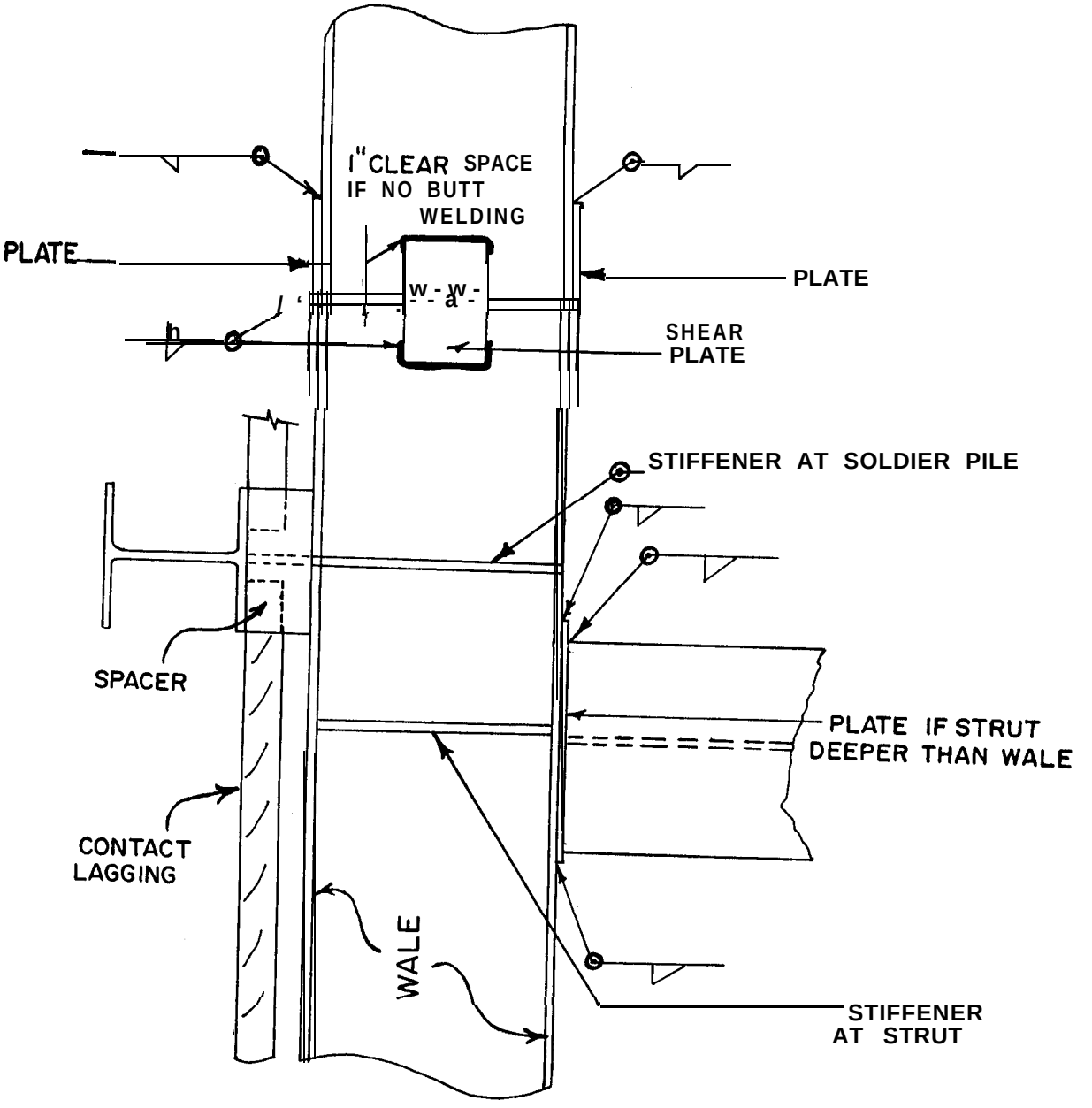


Figure 30. Plan view of typical wale splice and strut connection.

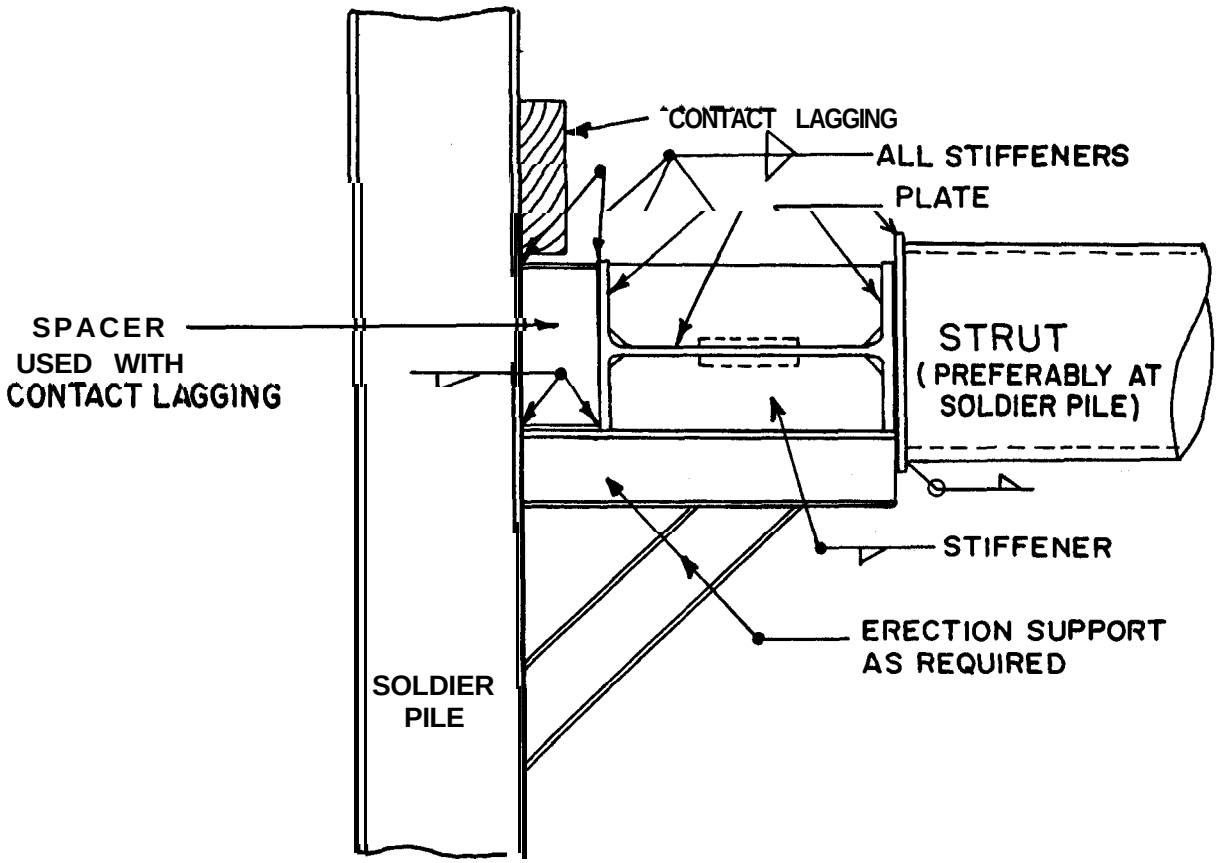


Figure 31. Typical strut-wale-soldier pile connections (elevation view).

For bracing:

a. At final depth, use allowable stresses by AISC Code.

b. For temporary conditions at intermediate depth of excavation use AISC + 20 percent.

For tiebacks use the stress values stated in Volume III, Chapter 6, (Tiebacks).

5.30 DEPTH OF PENETRATION BELOW CUT

Lateral Resistance

When use is made of the design pressure diagrams, a reaction at the base of the cut is assumed to exist which is equal to the lowest area shown in Figure 28. This reaction is provided by the passive resistance of the soil beneath the cut.

Figure 32 illustrates the method for determining the depth of penetration in competent soils that are capable of developing adequate passive resistance. Soils satisfying this condition are medium dense to dense granular soils and stiff to hard clays. The general method of analysis is:

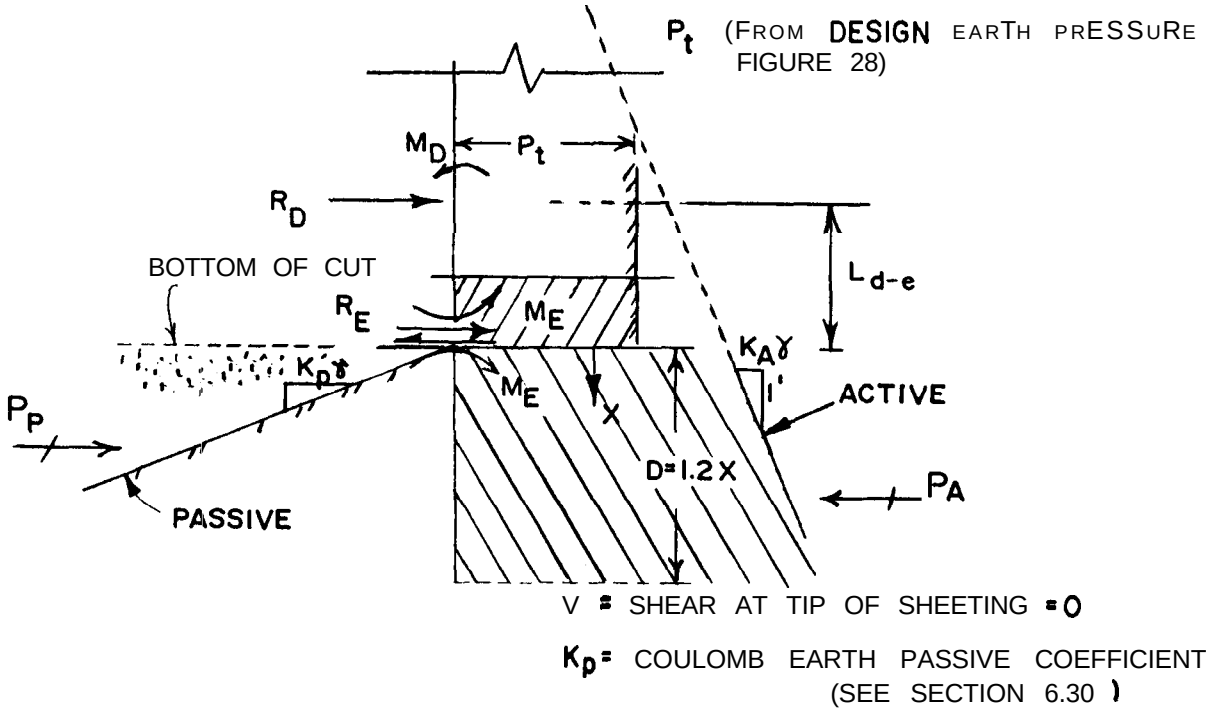
a, Compute the equivalent reaction at the base of the cut (R_E).

b. Determine the depth required to satisfy force equilibrium on the horizontal plane.

c. Check the maximum moment at or below RD against over-stressing of the support wall.

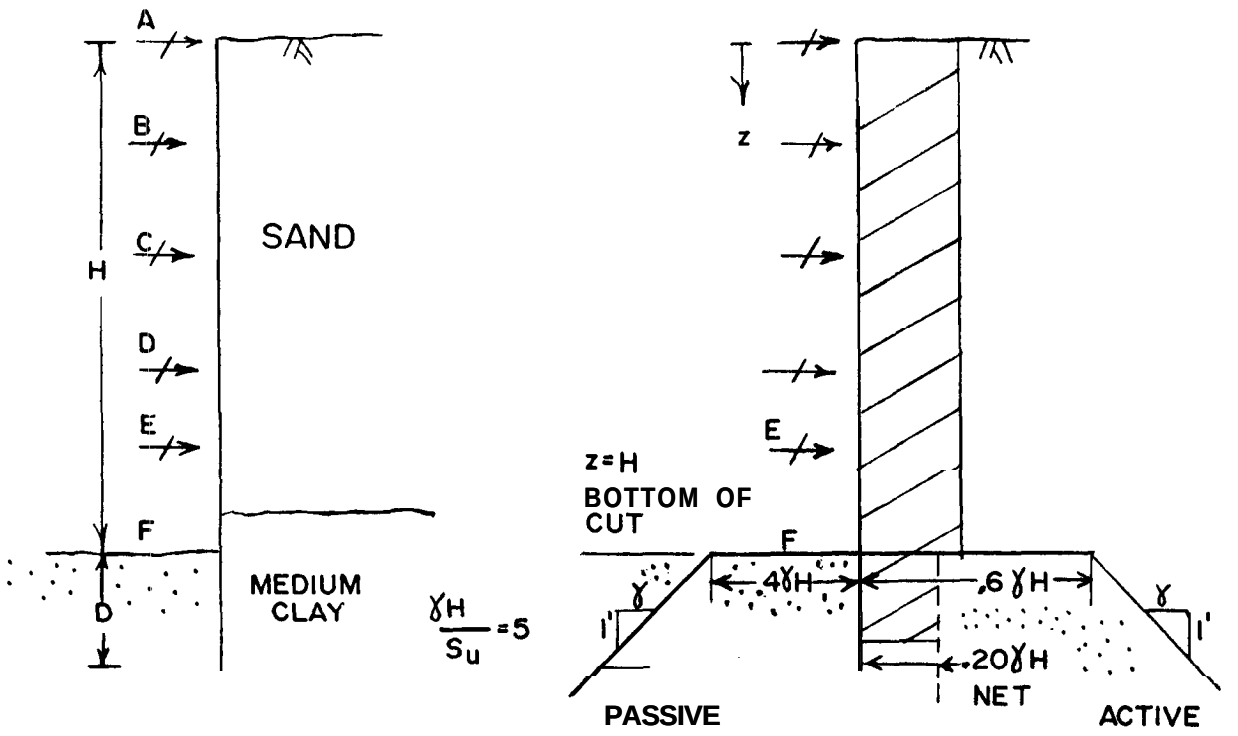
d. Drive sheeting to a depth 20 percent greater than that required for force equilibrium.

In cases where the soils below the base of the cut are soft clays the passive may never equal the active pressure, no matter how deeply it is driven. Since the passive resistance from the weak layer is small the sheeting acts much like a cantilever member; thus, a large load is developed in the lowest strut. For these conditions, where the base is stable against bottom heave, little is gained from driving the sheeting deeply below the bottom of the cut (see Figure 33).



1. Compute $R_E = 0.5\ p_t\ L_{d-e}$
2. Compute depth x such that: $P_p = R_E + P_A$
   Use minimum F. S. = 1.5 for passive coefficient, $K'_p = \frac{K_p}{1.5}$
3. Check $M_{max} \leq$ yield moment of sheeting
4. Drive to depth D = 1.2x

Figure 32. Procedure **for determining depth** of penetration in relatively uniform competent soil conditions.



$$S_u = \frac{\gamma H}{5} = 0.20\gamma H$$

RANKINE ACTIVE = $\gamma H - 2(.20\gamma H) =$ 0.601/H AT z = H

RANKINE PASSIVE AT z = H = 0 t $2(.20\gamma H) = 0.40\gamma H$

1. Theoretical passive resistance is not available below bottom of cut to develop horizontal reaction. In fact, the net force below cut is theoretically toward excavation, based on active and passive pressure.

2. Use nominal penetration of 0. 2 H or 5 feet whichever is greater, or penetration to cut off pervious layers.

3. Check base stability (see Chapter 6).

4. Design for cantilever condition below E.

Figure 33. Method for analyzing sheeting with weak underlying Layer.

Therefore, a minimal penetration of five feet or 20 percent of the excavation depth, whichever is greater, is recommended. In situations where the base is unstable, consideration may be given to deeper penetration and stiffer sheeting as a possible means to prevent bottom heave.

Bearing Capacity Considerations

Load capacity must be evaluated when there is a downward component of load, as is the case for inclined tiebacks. This may be accomplished by pile driving formulas or by the empirical and semi-empirical methods outlined in Chapter 7 (Bearing Capacity).

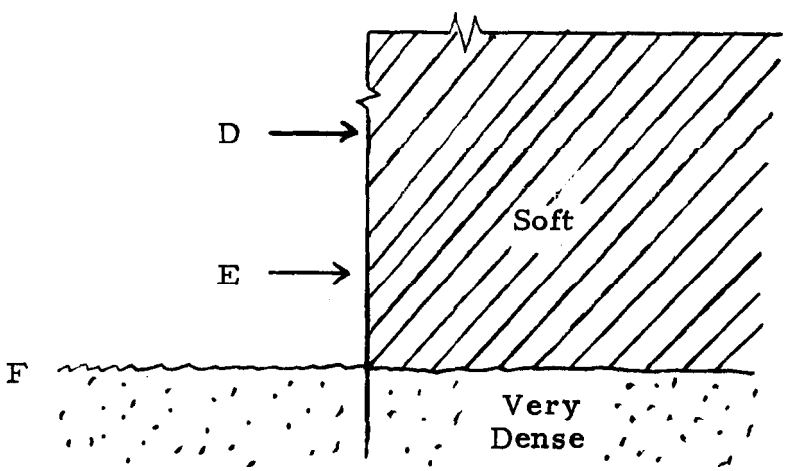
5.40 EXAMPLE SITUATIONS

Case I - Homogeneous Soil Profile

Case I is the analysis of a homogeneous soil profile which provides a basis for comparison of required penetration depth and strut load variations. It represents, most ideally, the conditions where the design envelope is appropriate. The method for analyzing soldier piles set in concrete-filled pre-augered holes is also presented,

Case II - Soft Soil Stratum to Base of Excavation Underlain by Dense Stratum

Let F represent an unyielding passive support at the base of the excavation. Let E represent the first wale level above the base. Let D represent the second wale level above the base.



As the excavation proceeds below level D to level E, little passive resistance is provided because of the soft soil above F; hence, the wall deflects inward. Effectively, the wall spans from level D to F (the excavation base) with full active pressure applied and negligible passive resistance above F. The deformation of the sheeting is such that during this excavation stage it resists essentially the same load over the span D to F whether or not strut level E is installed. This would be particularly true in the stiffer diaphragm walls. The effect of this large unsupported length is twofold:

a. Since the sheeting has already assumed an elastic line such that it resists the full active load, little load is transferred to strut level E. Hence, strut level D effectively takes a disproportionate share of the load.

b. The moment in the sheeting is greatly increased by the long unsupported length.

Case III - Soft Layer Underlying More Competent Soil

Design sheeting as cantilever below lowest strut with normal penetration below base (see discussion Section 5.30).

5.50 FINITE ELEMENT ANALYSES

Case Studies

Parametric studies by finite element analyses are powerful tools to examine qualitatively the effects of wall stiffness and soil conditions: on strut loads.

This section presents a brief summary. Details are contained in Volume II (De sign Fundamentals). Four soil conditions were analyzed:

Case la. Homogeneous soil profile of soft, normally consolidated clays.

Case lb. Homogeneous soil profile of medium-stiff clay.

Case 2. A soft soil stratum to base of excavation underlain by a stiff stratum.

Case 3. A soft soil layer underlying a more competent stiff soil.

Distribution of Earth Pressures

Figures 34 and 35 show normalized apparent earth pressure diagrams predicted by the finite element analysis for the four soil conditions .

Cornparing Case la with Case lb, the analysis shows that walls in the soft clay are expected to experience relatively higher pressures near the base of the cut than the wall in the medium-stiff clay. This trend is more obvious for the stiffer concrete walls. As Case lb shows, this behavior becomes Iess pronounced as the soil becomes stiffer, One possible remedy for reducing this effect in soft soils would be to prestress the second lowest strut and lock in a high residual compressive force,

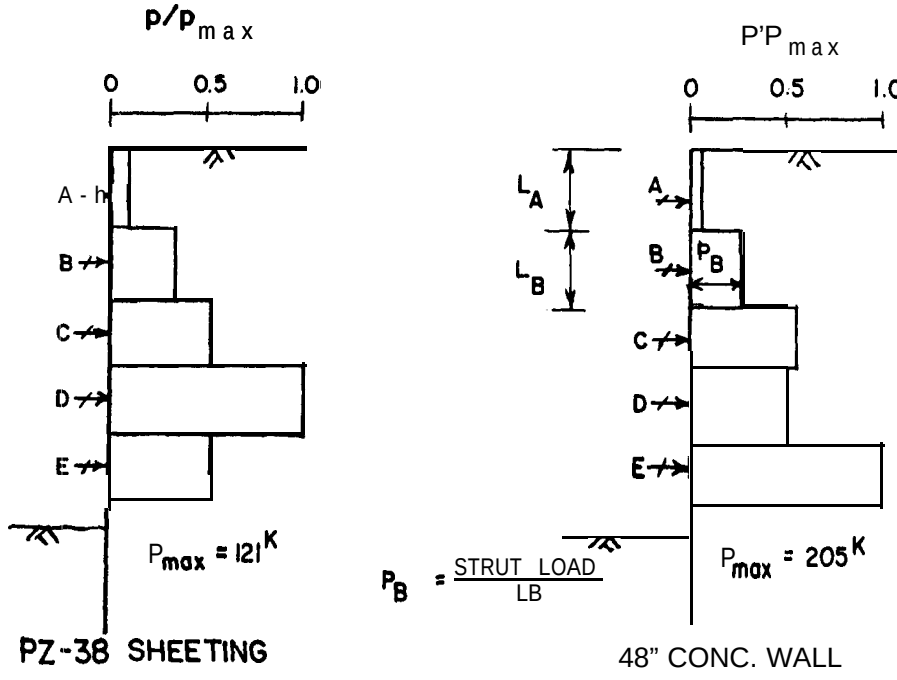
On Figure 35, Case 2 (soft clay overlying stiff clay) shows an opposite effect to that experienced in the homogeneous soil mass. This stiffer layer provides an adequate reation for the wall, restricting its inward deflection in the overlying soft clay. This leads to a larger strut load in the second to last strut and a reduction in the load received by the lowest strut. This results because the wall has already deflected inward close to its maximum amount before the last strut is installed and final excavation completed. Therefore, this last excavation stage results in little load transfer to the lowest strut. Stiffer walls push the center of gravity higher (Case 2, right panel).

Fo:r Case 3, where the soils within the depth of cut are stiff ($N < 4$) and soft soils exist immediately below the base of the excavation, the results show that the strut loads are greatest in the lower two struts. This occurs for the same reasons given for Case la, that is, lack of support below the excavation base. For this soil profile, the pattern of pressure distribution appears independent of wall rigidity since both give essentially the same normalized pressure diagram.

Magnitude of Strut Loads

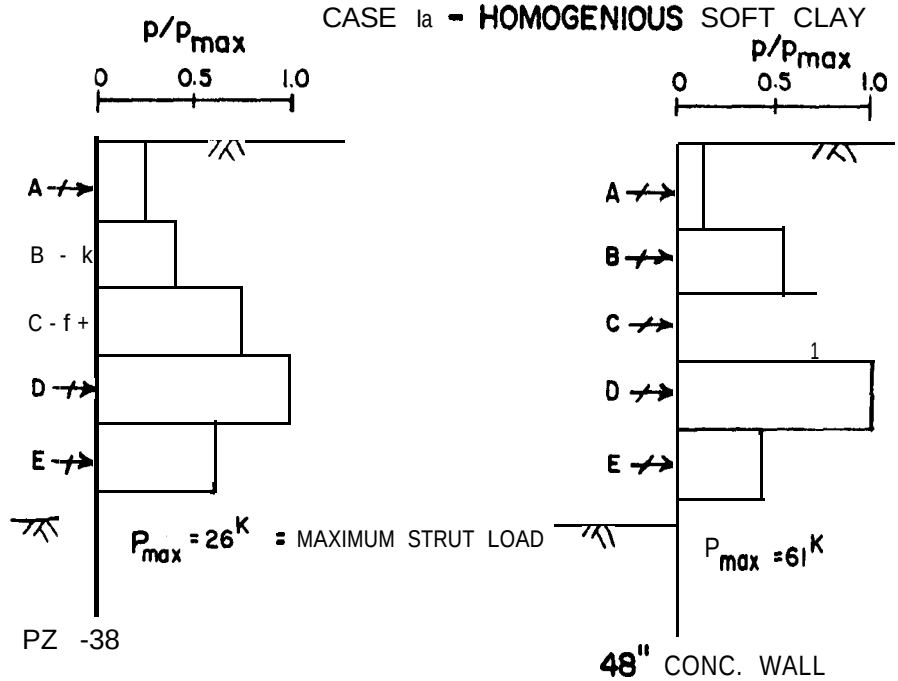
Figure 36 shows the magnitudes of the predicted loads for Cases 2 and 3. In both cases, the diaphragm wall receives much greater apparent pressures, on the order of 2 to 4 times that of the more flexible PZ-38 steel sheeting.

The higher apparent pressures in the concrete wall are attributed to snnaller lateral deformations, hence, less mobilization of shear strength in the soil adjacent to the wall. This behavior is particularly acute in the heavily overconsolidated soils such as those assumed for Case 3.



N = 6.8 AT BASE OF CUT

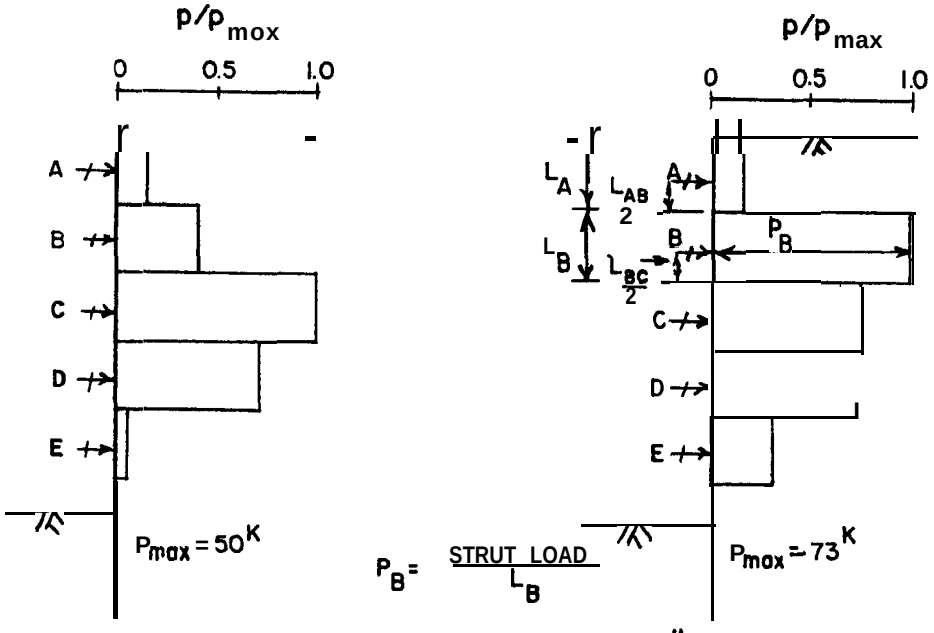
CASE Ia - HOMOGENIOUS SOFT CLAY



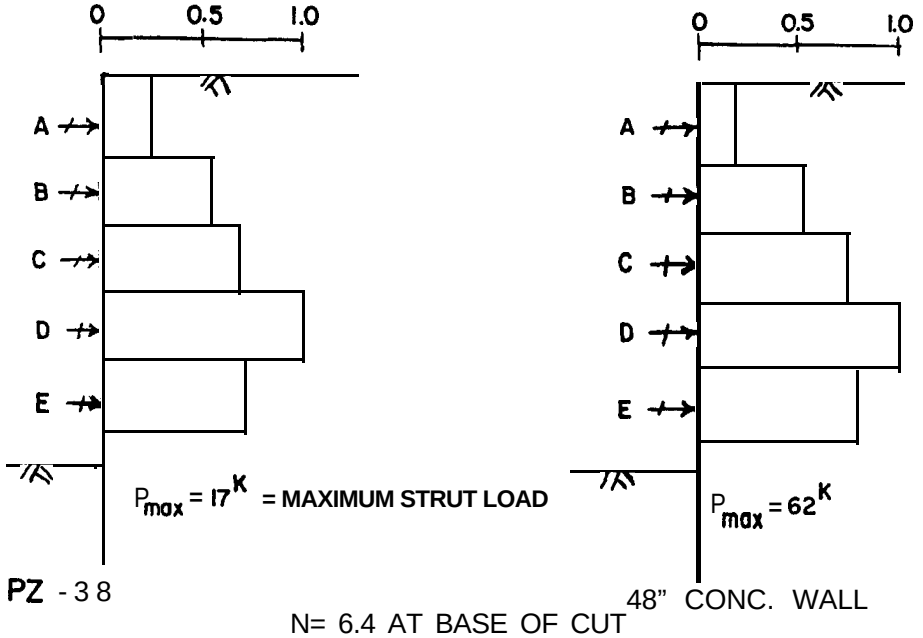
N = 4.8 AT BASE OF CUT

CASE Ib - HOMOGENIOUS MEDIUM-STIFF CLAY

Figure 34. Normalized apparent earth pressure diagrams predicted by finite element analysis.

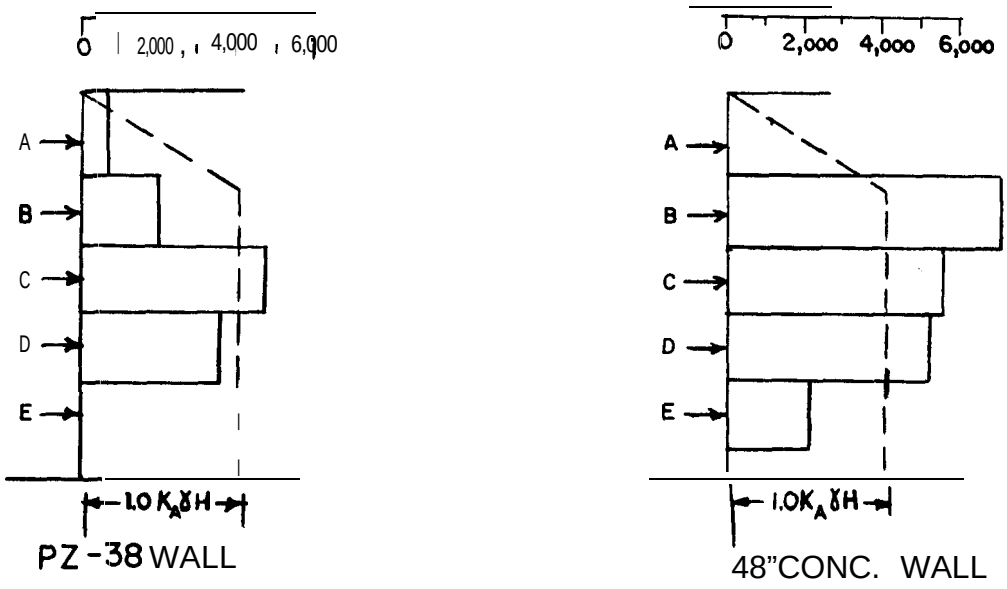


N= 2.3 AT BASE OF CUT
CASE 2 – SOFT OVERLYING STIFF CLAY

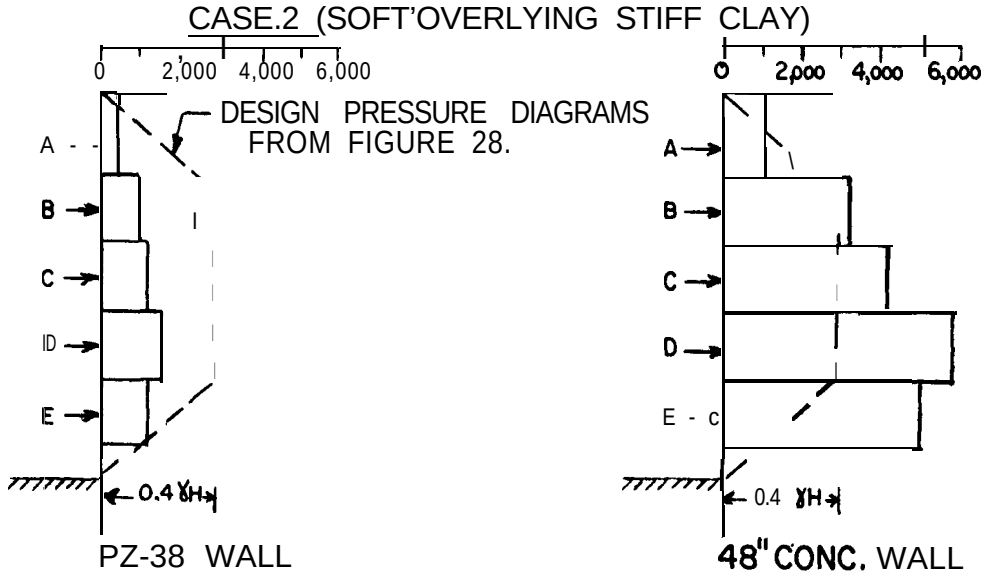


N= 6.4 AT BASE OF CUT
CASE 3 – STIFF OVERLYING SOFT CLAY

Figure 35. Normalized apparent earth pressure diagrams predicted by finite element analysis.



AT BOTTOM OF CUT: N = 2.3



NOTE: SEE TEXT FOR DISCUSSION OF LOADS.

AT BOTTOM OF CUT: N = 6.4

CASE. 3 (STIFF OVERLYING SOFT CLAY)

Figure 36. Comparison of predicted apparent earth pressures with finite element analysis on stratified soils.

There is scant field evidence to support the trends illustrated by the finite element analyses. One cannot accept these implications literally, but nevertheless, pending further advances in the state of the art, they are a matter of concern. Therefore, when overconsolidated soils are present, one should be aware that loads may build up on the support system causing overloading, especially if a relatively rigid wall is used which restricts the lateral swelling of the soil.

The Use of the Finite Element Method in Design

The finite element method (FEM) is a more realistic mathematical modeling of the complex soil profile and the soil structure system, thus making it a powerful tool in the analysis of supported excavations. However, it should be used with great care and only by experienced engineers with a strong soil mechanics background.

Until substantially more experience is gained with FEM as a design tool, it should be used primarily as an aid to guide engineering judgement.

6.10 GENERAL

The three primary modes of instability for excavations in clay are shown in Figure 37. Bottom heave and deepseated failure (Figures 37a and 37b) are related to the overall stability of the excavation and may even dictate the construction procedure.

Local failures (Figure 37c) are of concern where it is necessary to limit inward sheeting deformations. Failures of this type occur below the excavation level immediately adjacent to the sheeting, resulting in partial loss of lateral support.

6.20 BOTTOM HEAVE

Bottom heave is a problem primarily in soft to medium clays where the strength of the soil is nearly constant with depth below the base of the excavation. The failure is analogous to a bearing capacity failure; the difference being that stress is relieved. This mode of failure should be analyzed (Bjerrum and Eide, 1956) using the stability chart given in Figure 38. The factor of safety against a bottom heave is determined as:

$$F.S. = N_{cb} \left(\frac{S_u}{\gamma H + q} \right) = \frac{N_{cb}}{N}$$

where:

N = stability number $= \dfrac{\gamma H + q}{S_u}$

N_{cb} = bearing capacity factor from Figure 38

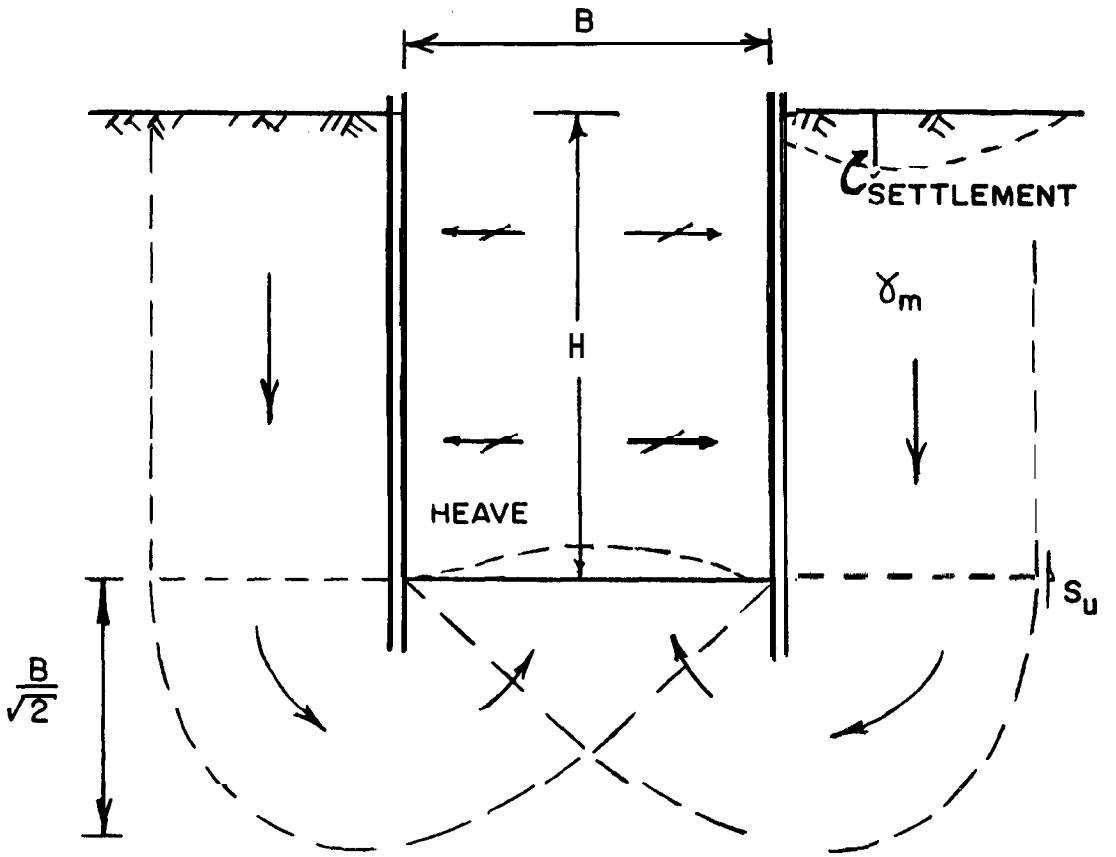
S_u = the undrained strength of the clay

γ = total unit weight of the soil

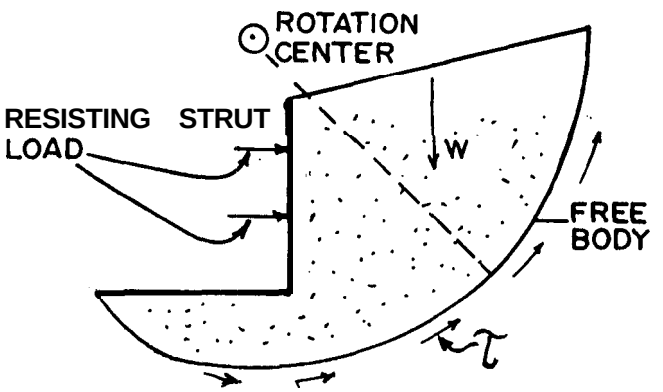
H = depth of excavation

q = uniform surcharge loading on the area adjacent to the excavation

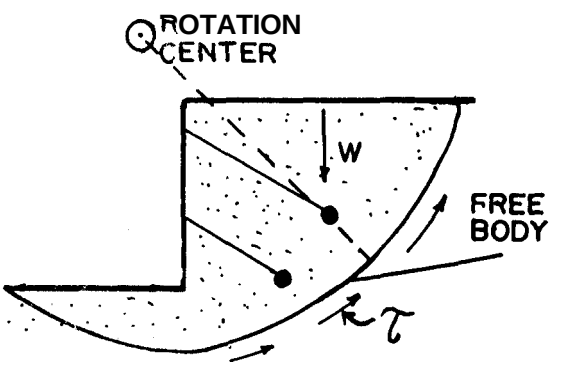
Where the soil is stratified within the depth of excavation and below, a weighted average of undrained strength should be used for



a) BOTTOM HEAVE FAILURE-

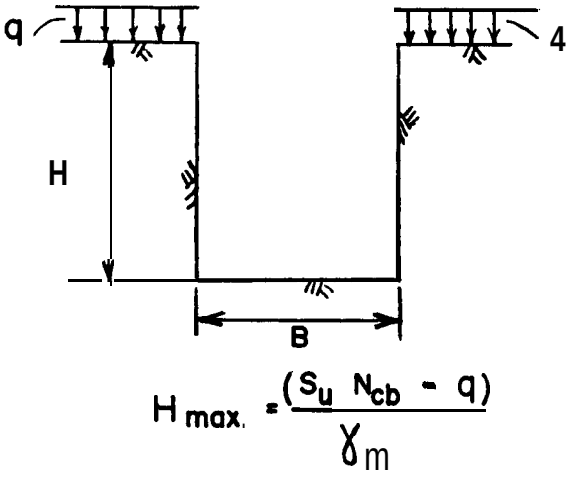


b) DEEP SEATED FAILURE INTERNALLY BRACED EXCAVATION



c) DEEP SEATED FAILURE TIED BACK EXCAVATION

Figure 37. Potential failure surfaces.



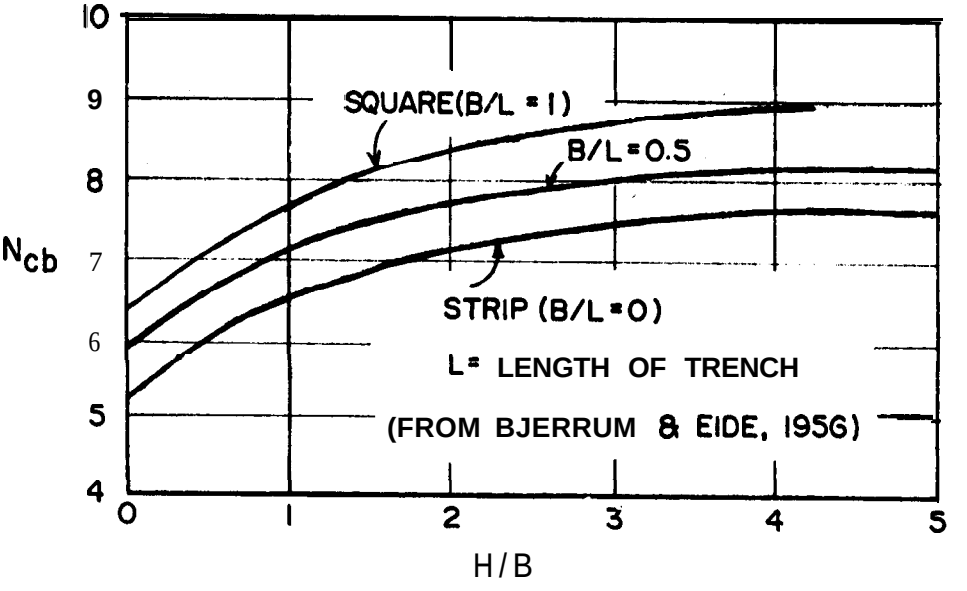


Figure 38. Bearing capacity factors for bottom stability analysis.

S_u. This average should be taken over a zone described between $\frac{B}{\sqrt{2}}$ below the excavation base and 2.5B above the base.

6.30 LOCAL FAILURE

Lateral pressure on the retaining wall coupled with the stress relief from the excavation can be of sufficient magnitude to cause local yielding of the soil immediately adjacent to the inside of the sheeting, This localized overstressing results in loss of passive resistance which in turn leads to uncontrolled inward movements of the sheeting, often amounting to about 50 percent of total movement.

Figures 39 and 40 can be used to estimate when local failure is imminent in cohesive soils where flexible sheeting is used. Figure 39 shows the factor of safety against bottom heave necessary to prevent local yield as a function of excavation geometry and the shear stress ratio.

The potential for local yielding is most prominent in the over-consolidated soils, that is, soils with a high value of $K_o(\bar{\sigma}_{ho}/\bar{\sigma}_{vo})$. Excavation in these soils relieves lateral stress which in turn leads to an extension type local failure near the base. Note, for example in Figure 39, that the ratio N_{cb}/N_c required to prevent local failure will increase with increasing K_o, 'all other variables being constant, Figure 41 shows the effect sheeting stiffness has on reducing local yielding in normally consolidated soils. Stiffening the sheeting reduces the factor of safety required to prevent local failure,

6.40 DEEP SEATED FAILURES

6.41 Internally Braced Excavations

Circular Arc Analysis

One way to analyze the stability is the classical circular arc analysis as illustrated in Figure 42, This involves a series of trial centers of rotation and failure surfaces to find the most critical condition.

The sum of the strut forces necessary to maintain a stable excavation should be compared to those predicted from the lateral earth pressure diagram. The greater of the two total loads should be used to establish the ordinate of the design earth pressure diagram.

In the cases where the retaining wall extends through a weak layer into a highly competent soil, the structural resistance of

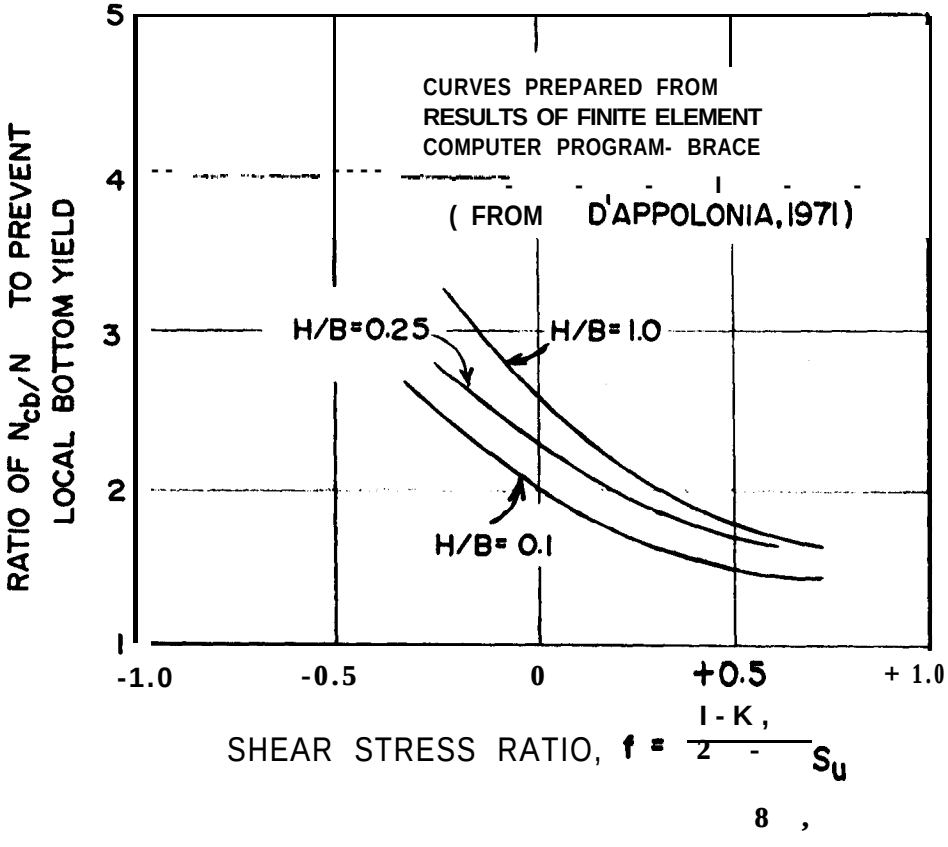


Figure 39. Factor of safety required to prevent local yield below bottom of excavation in clay.

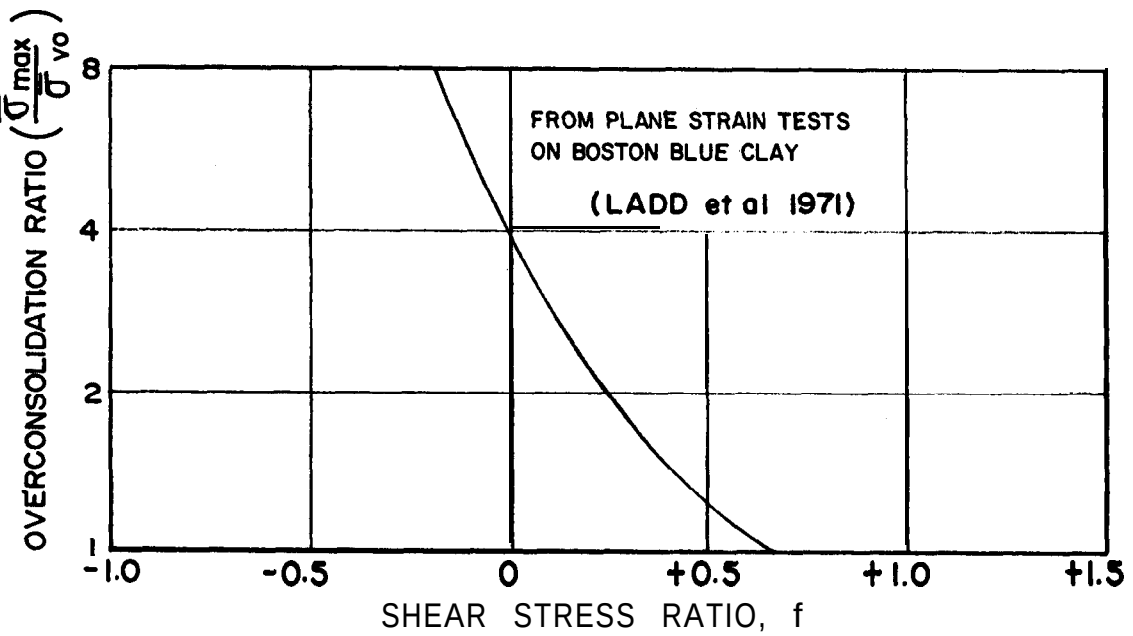


Figure 40. Shear stress ratio vs. overconsolidation ratio.

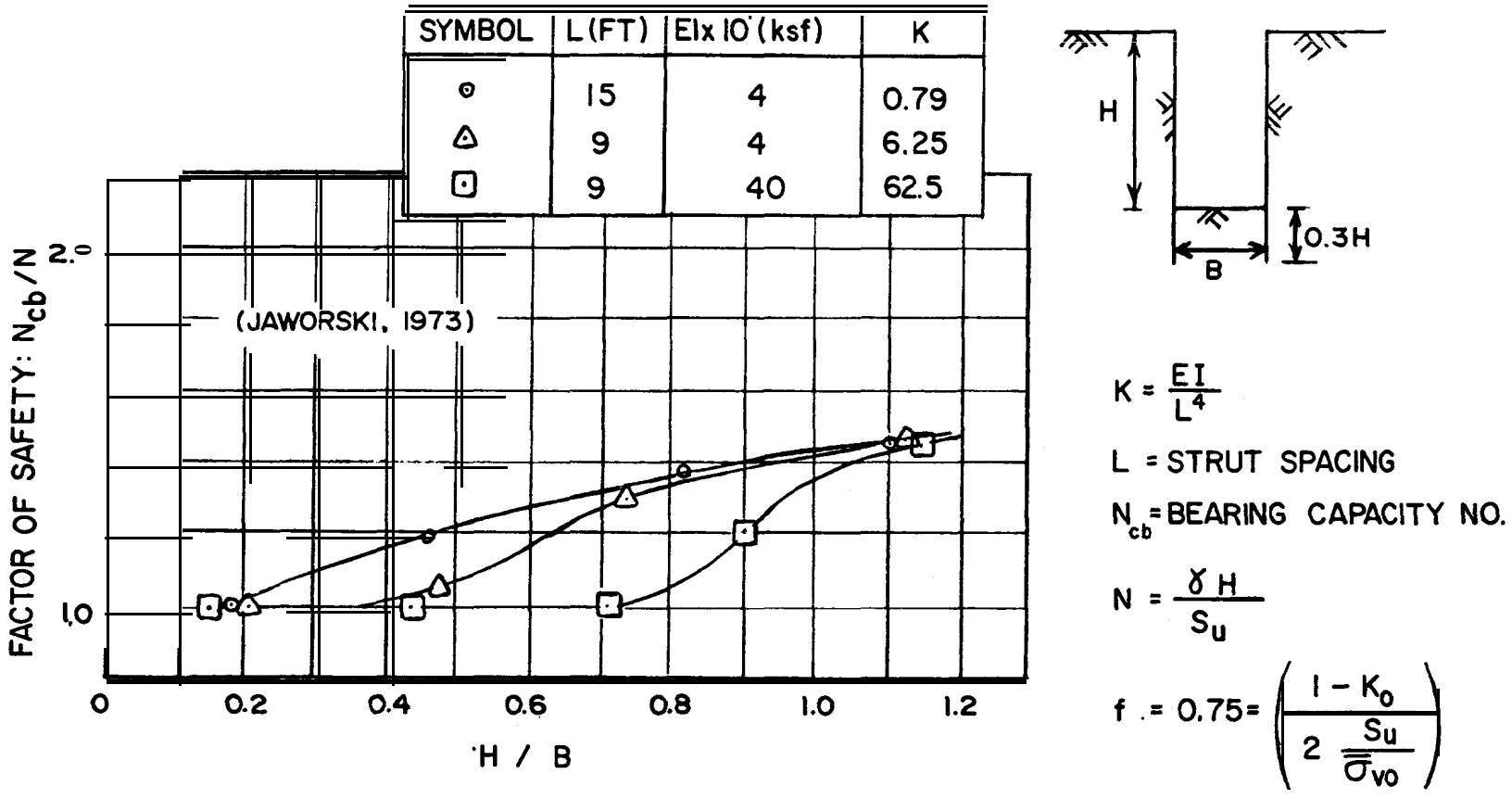
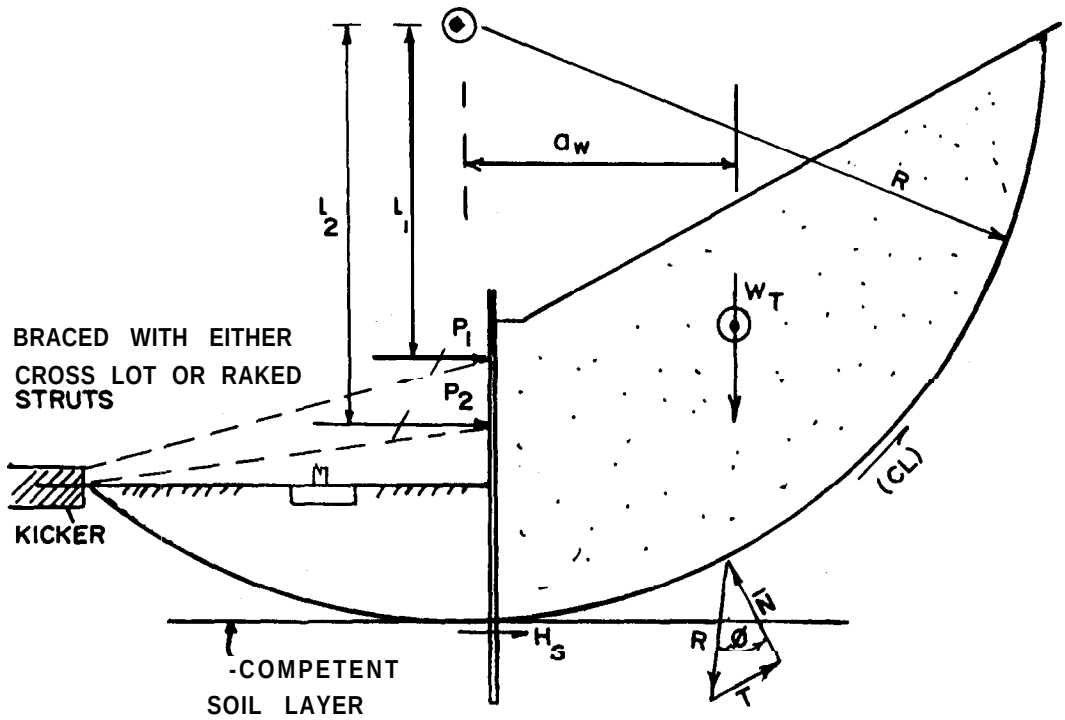


Figure 41. Effect of sheeting stiffness on factor of safety at which first yield occurs in normally consolidated clay.



Consider overall stability:

Moments around center of rotation

Forces to consider:

1) Weight of driving mass (WT)
2) Resisting strut loads (P_1, P_2) (Horizontal component of support load.)
3) Resisting shear capacity of wall (Hs) from competent soil layer.
4) Shear strength of soil, frictional component (T). and Cohesion, (c)

Note: If rakers used, kicker must be located outside failure mass for P_1 and P_2 to be considered in analysis.

$$\text{Safety Factor} = \frac{\Sigma M_R}{\Sigma M_D} = \frac{(\bar{N} \tan \phi \text{ t CL}) R}{W_T a_w + P_1 l_1 + P_2 l_2 + H_s R}$$

Figure 42. Stability of internally braced cut (circular arc method).

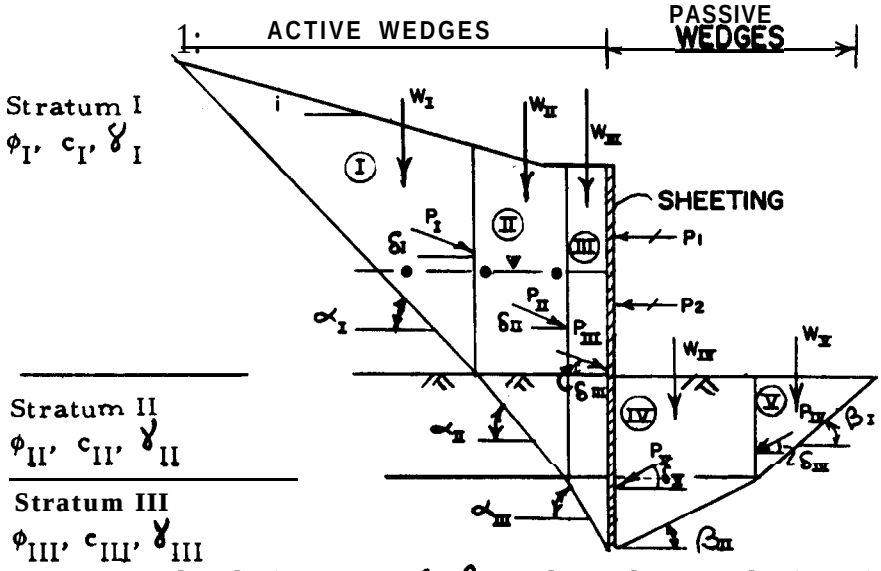
the retaining wall (Hs) should be considered in the analysis. The soil shear resistance should be taken equal to the passive force determined in accordance with Chapter 4, (Passive Resistance).

Wedge Stability Analysis

Figure 43 shows a wedge stability analysis based upon planar failure surfaces. Like the circular arc method, this involves a series of trial planes to find the most critical failure surface.

6.42 Tied-Back Walls

Detailed procedures for analyzing the stability of tied-back walls by a variety of methods employing trial planar surfaces and wedges are presented in Volume III. By and large, these methods place emphasis on failure surfaces passing through the zone of tiebacks. As such, the techniques may be used as a design tool for establishing the appropriate length of tiebacks.

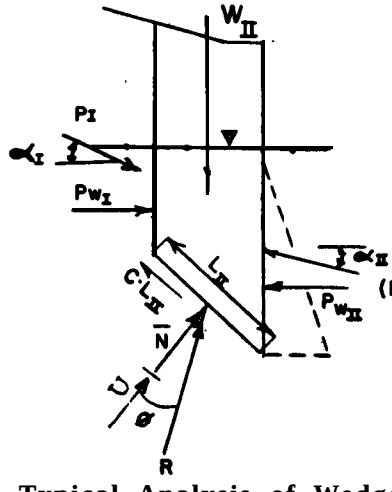


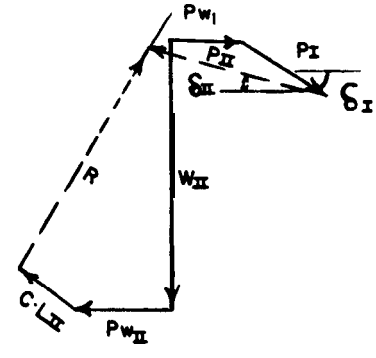
For general solution vary α, β, and&angles to obtain minimum value for factor of safety.

Method of Analysis:

1. Assume α, β, δ angles.
2. Sequentially analyze the active and passive segment for loads P_{III} and P_V, Include water pressure.
3. Sum forces in horizontal direction for factor of safety

   i.e. F.S. $= \dfrac{P_1 + P_2}{P_{III} + P_V}$





P_{w_I}, $P_{w_{II}}$ = horizontal water pressure

U = uplift water force on Wedge II

Typical Analysis of Wedge (Wedge II)

Figure 43. Wedge stability analysis for braced cut.

CHAPTER 7 - BEARING PRESSURE OF DEEP FOUNDATIONS

7.10 GENERAL

This section is directed toward those basic considerations used to establish bearing values for elements involved in cut-and-cover operations. Typically, the bearing stratum is deep--that is, it lies at great depth relative to the width of the bearing area. A ccor dingly, design rules developed for shallow foundations will be overly conservative.

Fundamentally, allowable bearing value must recognize two governing criteria --first, adequate safety against shear failure of the foundation and second, a limitation of settlement. Usually, as will be apparent further in this discussion, it is the former which controls for clays and it is the latter which controls for sands.

7.20 PRESUMPTIVE BEARING VALUE

Table 2 presents a summary of the range of allowable bearing values for building foundations resting on a variety of soil types. This tabulation is not intended to represent a recommendation for design but rather to aid in assessing the relative competency of different materials and to provide a crude initial guide. Because the values typically apply to shallow foundations, acceptable values for deep foundations will be somewhat higher.

7.30 BEARING VALUES BASED ON SHEAR FAILURE

7.31 General

The following represents a summary of theoretical procedures for calculating net ultimate bearing capacity using shear strength parameters, $\bar{\phi}$, of cohesionless soil, and undrained shear strength, S_u, of cohesive soils. A factor of safety of 2 to 3 should be applied depending upon risk and confidence level in data.

7.32 Sand

For deep piers in sand the end bearing load capacity is generally expressed as:

$$q_u = N_q \bar{\sigma}_v$$

Table 2. Abstract of presumptive bearing capacity, ksf.

	A Mass. State Code (1974)	B New York City (1968)	C Atlanta (1950)	D National Board of Fire Underwriters (1955)	E BOCA (1970)
Glacial Till*	20	--	--	--	--
Hardpan*	--	16 - 24	20	20	20
Gravel, well-graded sand and gravel*	10	8 - 20	8 - 12[1]	8 - 12[1]	8 - 12[1]
Coarse sand*	6	6 - 12[2]	--	6 - 8 1	6 - 8[1]
Medium sand*	4	--	--	--	4(loose)
Fine sand	2 - 4	4 - 8[3]	--	4 - 6 1	--
Hard clay	10	10	--	--	--
Stiff clay	--	--	4	5	--
Medium clay	2	4	--	5	--

*Massachusetts and New York Code allow 5 percent increase in bearing value per foot of additional embedment, but not more than twice tabulated value.

1 - Range reflects compactness, gradation, and/or silt content

2 - 0. 1 x N, but not less than 6 ksf nor more than 12 ksf (where N= no. of blows in SPT)

3 - 0. 1 x N, but not less than 4 ksf nor more than 8 ksf (where N= no. of blows in SPT)

where:

N_q = dimensionless bearing capacity factor that is a function of the shear strength parameter, $\bar{\phi}$, of the bearing material and shape of the loaded area

$\bar{\sigma}_v$ = effective stress in the soil at the bearing surface

q_u = ultimate bearing capacity (load per unit area)

Values of N_q vary depending upon assumptions made in the derivation. Vesic (1965) presents ranges for the values as shown in the Figure 44. In general, a safety factor of 3 is applied to these ultimate values.

7.33 Clay

In clays the undrained strength, S_u, rather than drained strength will control the bearing capacity of a foundation element. Skempton (1951) presents bearing capacity factors N_c for net ultimate bearing capacity on clays. In this case, "net" means pressure in excess of the effective overburn stress on the bearing level.

$$q_u = N_c S_u$$

where:

q_u = net ultimate bearing capacity (load per unit area).

N_c = dimensionless bearing capacity factor that is a function of the shape of the loaded area.

S_U = undrained shear strength of soil.

For deep foundations (at depth greater than 4 to 5 times the breadth of the loaded area), values of N_c are as follows:

Circle: $N_c = 9$

Strip: $N_c = 7.5$

Rectangle: $N_c = 7.5\ (1 + 0.2\ B/L)$

where: B = breadth
L = length

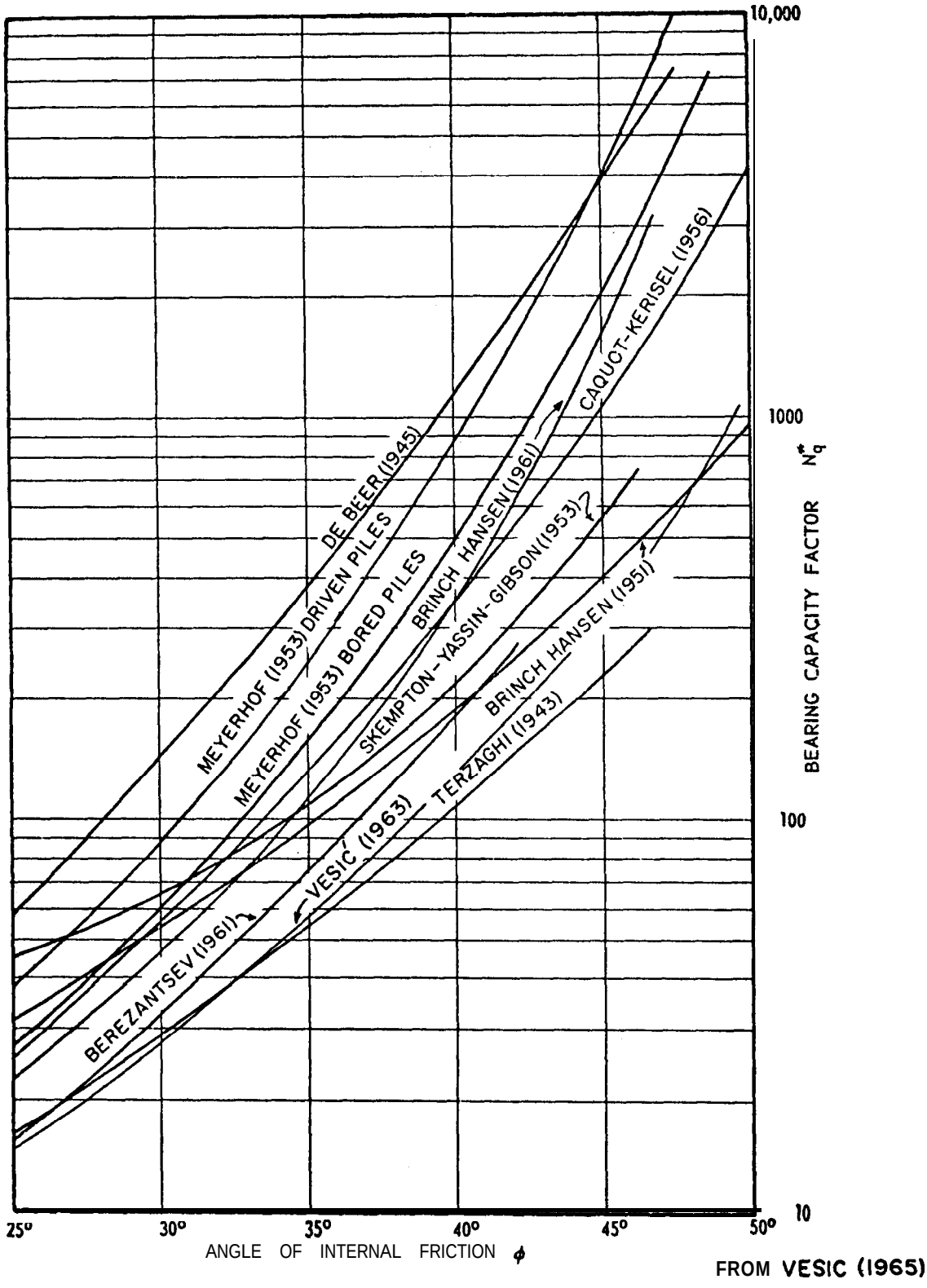


Figure 44 Bearing capacity factors for deep circular foundations.

Note that for clays the net ultimate bearing pressure is independent of depth (and therefore overburden stress). It is a function only of the shape of the loaded area and undrained shear strength of the soil.

In addition to the load bearing capacity at the base, the side friction may be determined on the basis of the embedded area and adhesion along the shaft. In soft clays, the adhesion is equal to or only slightly less than the undrained shear strength. However, in stiff to hard clays the adhesion is typically less than one-half the undrained strength.

The practice is to apply a reduction factor, α, to the undrained strength to estimate adhesion. Thus :

$$S_{eff} = \alpha S_U$$

where:

α = reduction factor

S_U = undrained shear strength, psf

S_{eff} = adhesion along shaft, psf

Figure 45 (after Peck, et al, 1974) shows that α decreases as the shear strength of clay increases.

7.40 BEARING VALUES BASED ON SETTLEMENT

7.41 Soils Havinp Constant Modulus of Deformation with Depth

Surface Loading

Theoretical procedures for determination of settlements have been 'developed based on integration of the Mindlin solution for a point load within an elastic half space. At a depth equal to zero, the Mindlin solution is identical to the familiar Boussinesq solution. These solutions all have the general form

$$P = q \frac{BI_\rho}{E} (1 - \nu^2) \qquad \text{Eq. 7.41.1}$$

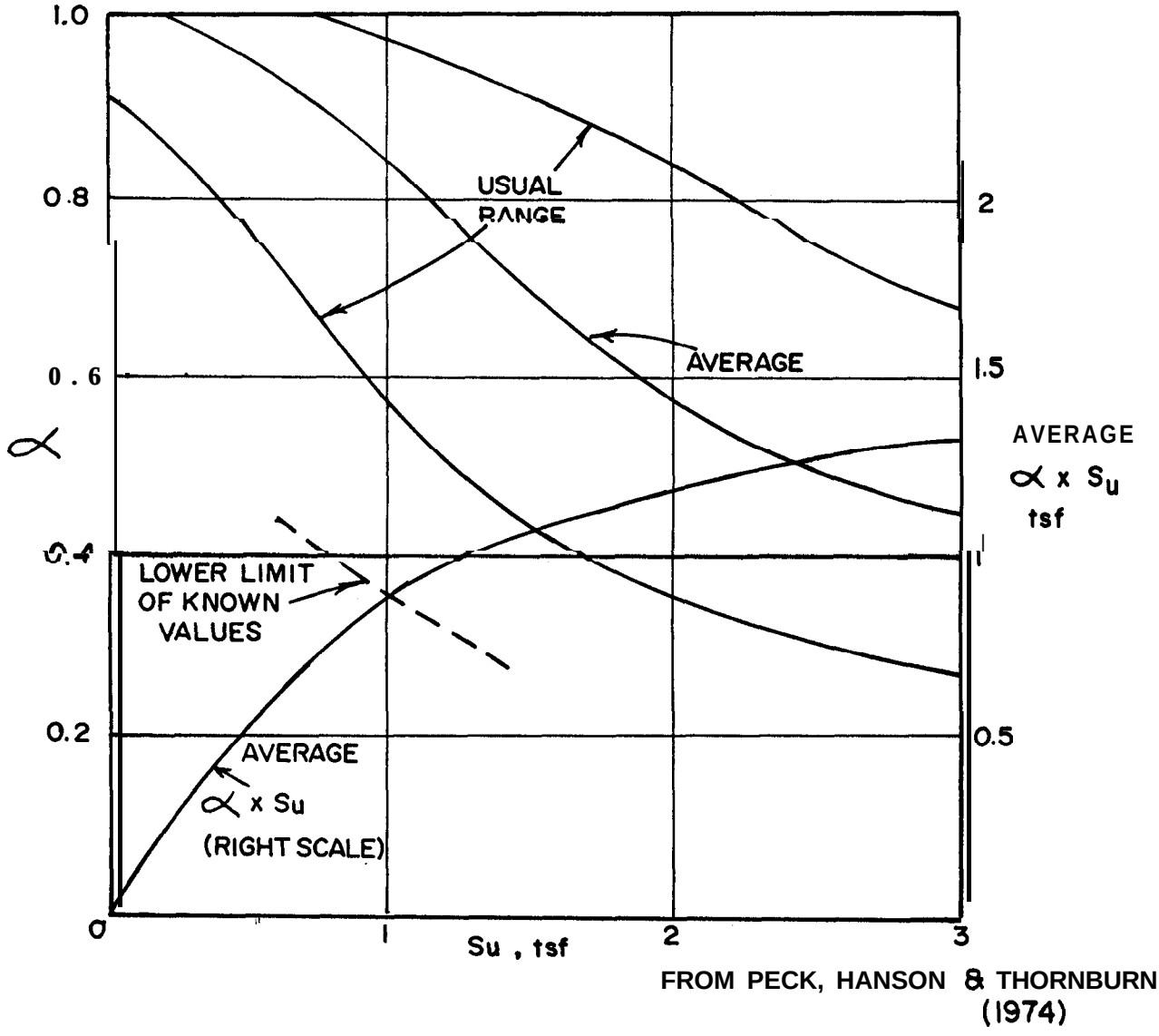


Figure 45. Reduction factor in S_u from observed capacity of friction piles.

where:

ρ = settlement

q = distributed load

B = least dimension of foundation unit

E = modulus of deformation

ν = Poisson's Ratio

I_ρ = influence factor which depends on rigidity of footing, shape of footing, and depth of footing

A simplified method for determining settlement at the surface is based upon a coefficient of subgrade reaction, defined as follows:

$$\rho = \frac{q}{k} \qquad \text{Eq. 7.41.2}$$

where:

ρ and q are defined as above

k = coefficient of subgrade reaction in general units of pressure per unit deflection

The value of the coefficient of subgrade reaction is commonly determined by plate loading tests or by correlation with in situ soil indices such as relative density and standard penetration resistance. By comparison of Eq. 7.41.1 and 7.41.. 2, the coefficient of subgrade reaction is related to the theoretical settlement as follows:

$$k = \frac{E}{B\ (I - V^2)\ I_\rho} \qquad \text{Eq. 7.41.3}$$

For a constant footing shape and depth and constant material properties, the coefficient of subgrade reaction for a footing of size B is therefore related to a footing of size B' as follows:

$$k_{B'} = k_B \left(\frac{B}{B'}\right) \qquad \text{Eq. 7.41.4}$$

It is common to express the coefficient of subgrade reaction in terms of the value for a 1 foot square plate (k1) as this is the size for conventional plate loading tests. Therefore,

$$k_B = \frac{k_1}{B}$$

Typical values for k1 are shown in Figure 46.

Rectangular Footings

Terzaghi (1955) has proposed the following empirical relationship for rectangular footings :

$$k_{L \times B} = k_B \frac{(1 + 0.5\ B/L)}{1.5} \qquad \text{Eq. 7.41.5}$$

where:

$k_{L \times B}$ = coefficient of subgrade reaction for footings of length, L, and width, B

k_B = coefficient of subgrade reaction for square footing of dimension, B

See Figure 47 for comparison of Terzaghi's empirical equation and elastic theory. Terzaghi's equation is recommended.

Effect of Depth

For a footing with constant loading, shape, and material properties, the subgrade modulus of that footing is inversely proportional to the influence factor (see Eq. 7.41.3). Thus, when the influence factor varies with depth, the ratio of subgrade modulus at the surface to the subgrade modulus at depth may be computed as follows:

$$\frac{k_B^S}{k_B^D} = \frac{I_\rho^D}{I_\rho^S} \qquad \text{Eq. 7.41.6}$$

where:

k_B^S = coefficient of subgrade reaction for a footing (breadth B) at the surface

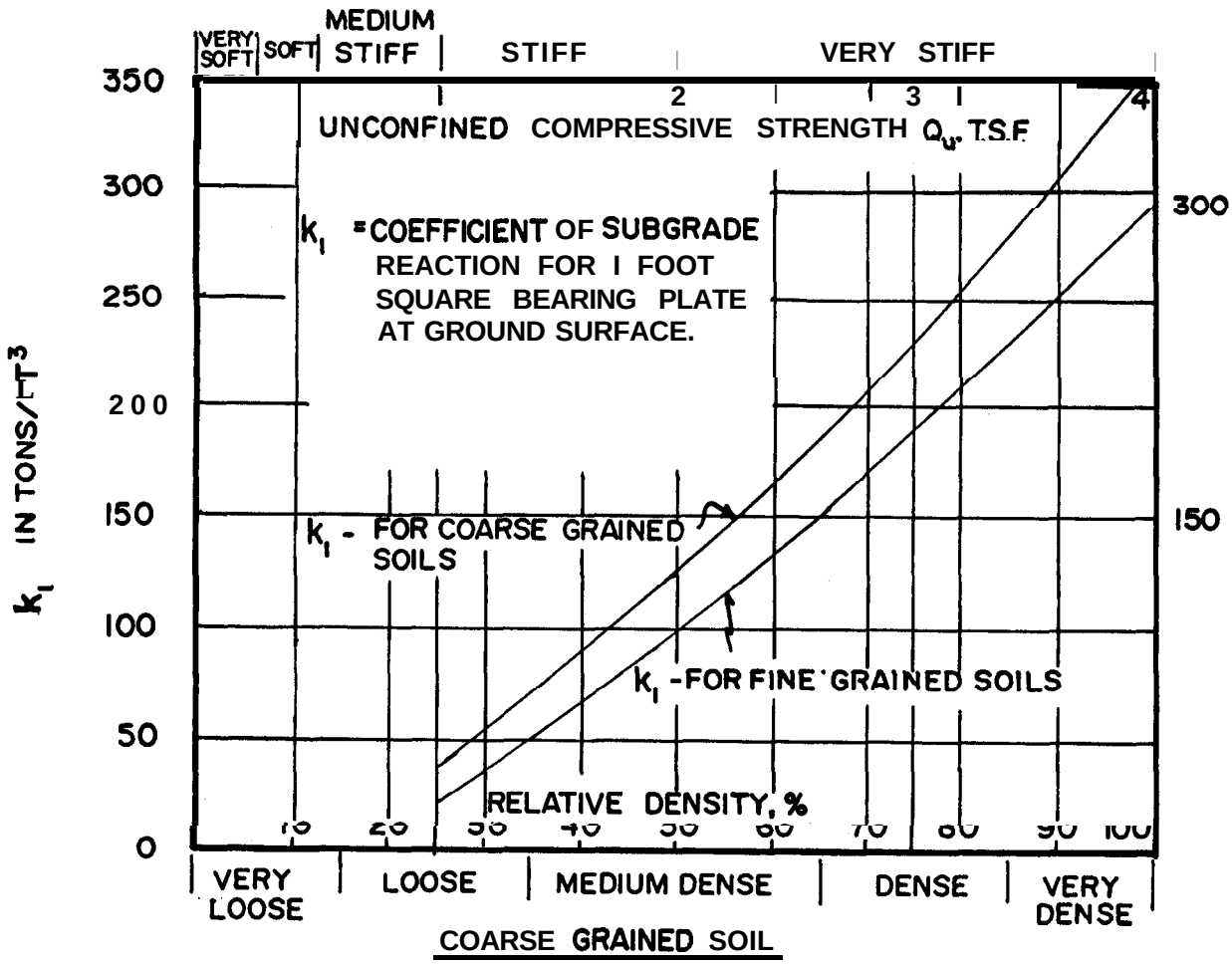


Figure 46. **Coefficient of subgrade reaction vs. in situ soil indices (NAVFAC, 1971).**

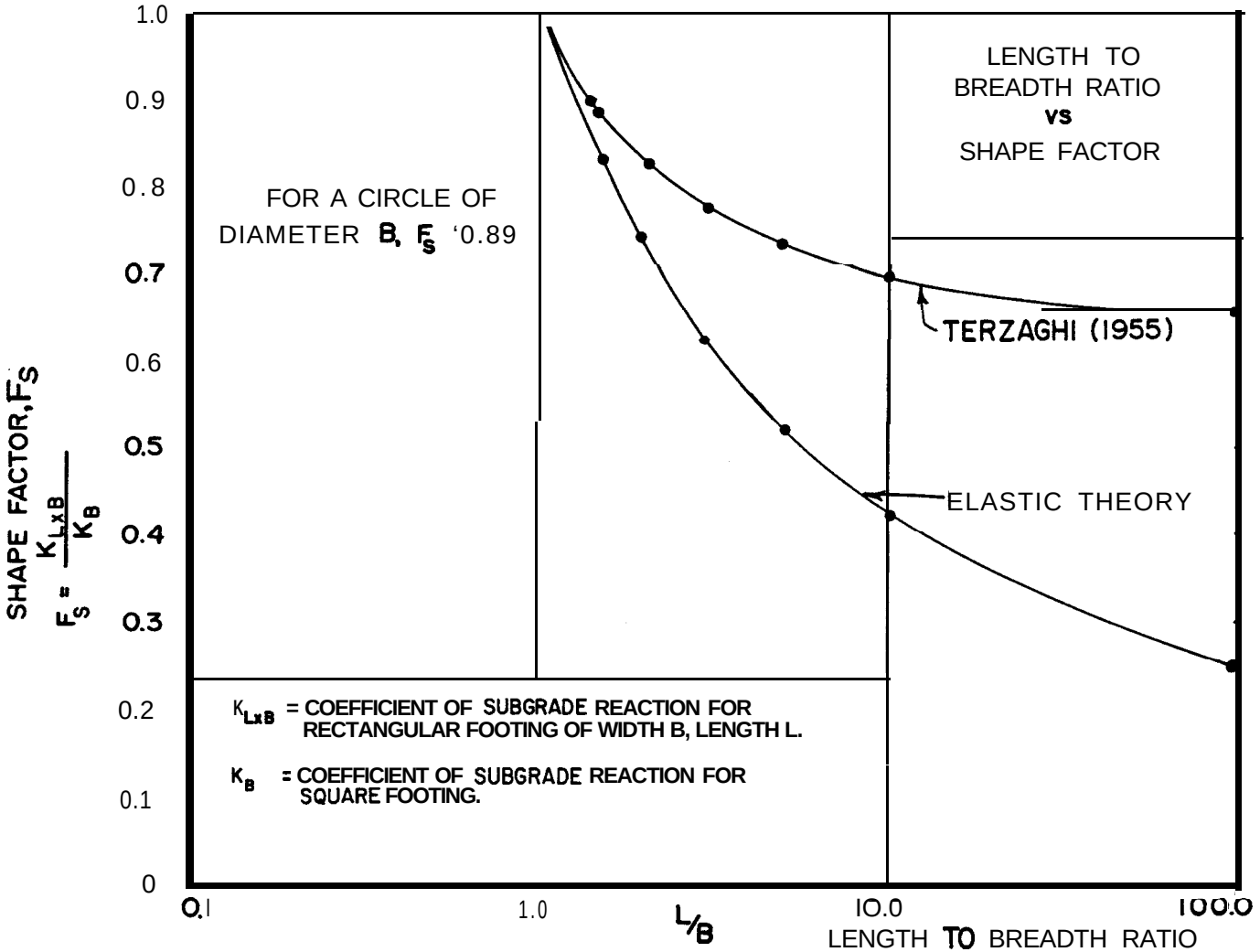


Figure 47. Shape factor for rectangular footings.

k_B^D = coefficient of subgrade reaction for a footing (breadth B) at depth D

I_ρ^D = influence factor for footing at depth D

I_ρ^S = influence factor for footing at surface

Elastic theory demonstrates that footings will undergo Less settlement with depth. This is illustrated in Figure 48 which shows that the subgrade modulus increases with depth below the surface.

7.42 Soils Having Modulus of Deformation that Increases with Depth

Surface Loading

Terzaghi (1955) has proposed the following empirical relationship to convert the coefficient of subgrade reaction for a 1 foot square area to an area B x B square.

$$k_B = k_1 \left(\frac{B + 1}{2B}\right)^2 \qquad \text{Eq. 7.42. 1}$$

Once k_B is determined for a square footing, the coefficient of subgrade reaction, $k_{L \times B}$, for a rectangular footing may be obtained from Figure 47.

Depth Effects

Taylor (1948) has proposed an embedment correction to account for the increase in modulus of deformation with depth as follows :

$$k_B^D = k_B^S \quad (1 \text{ t } 2 \text{ D/B}) \qquad \text{Eq. 7.42. 2}$$

where:

D is the depth of footing
B is the minimum footing dimension

A second approach is based on Janbu (1963) who demonstrated that the initial tangent modulus increases as a power function of confining stress.

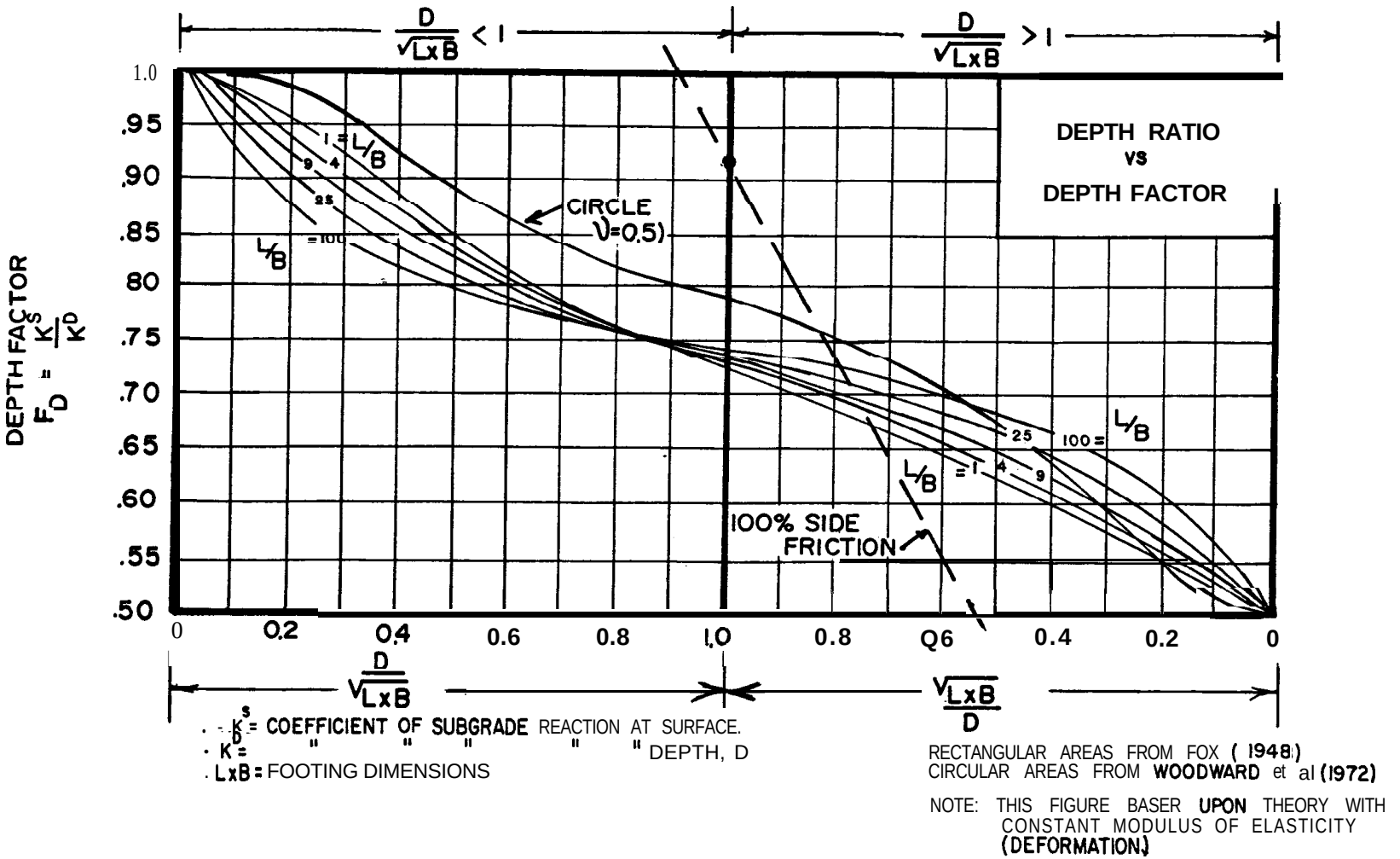


Figure 48. Influence of depth on coefficient of subgrade reaction (based on modulus of deformation that is constant with depth).

E_{it} is proportional to $(\bar{\sigma}_3)^n$

where:

E_{it} = initial tangent modulus

σ_3 = lateral effective stress

n = 0.3 for gravel and 0.5 for sands

From the assumption that k is proportional. to E_{it}:

$$\frac{k_B^S}{k_B^D} = \left[\frac{\bar{\sigma}_3}{\bar{\sigma}_{3D}}\right]^n = F_{DG} \qquad \text{Eq. 7.42. 3}$$

In normally consolidated deposits, $\bar{\sigma}_3$, is proportional to the overburden stress and therefore the depth.

The Taylor and Janbu methods for determining depth effects in soils with varying modulus of deformation are presented in Figure 49. Note that a limitation of $F_{DG} \geq 0.5$ has been set for the Taylor expression.

Water Table Effects

The presence of ground water in granular soils will effect the modulus of deformation by reducing the lateral effective stress. The plots in Figure 49 would therefore require corrections based upon reduction of effective stress level from submergence.

7.43 Recommended Procedure for Determination of Settlements of Deep Foundations

Clays

Assume modulus of deformation is constant with depth. Compute settlement for Eq. 7.41.2

$$\rho = q/k$$

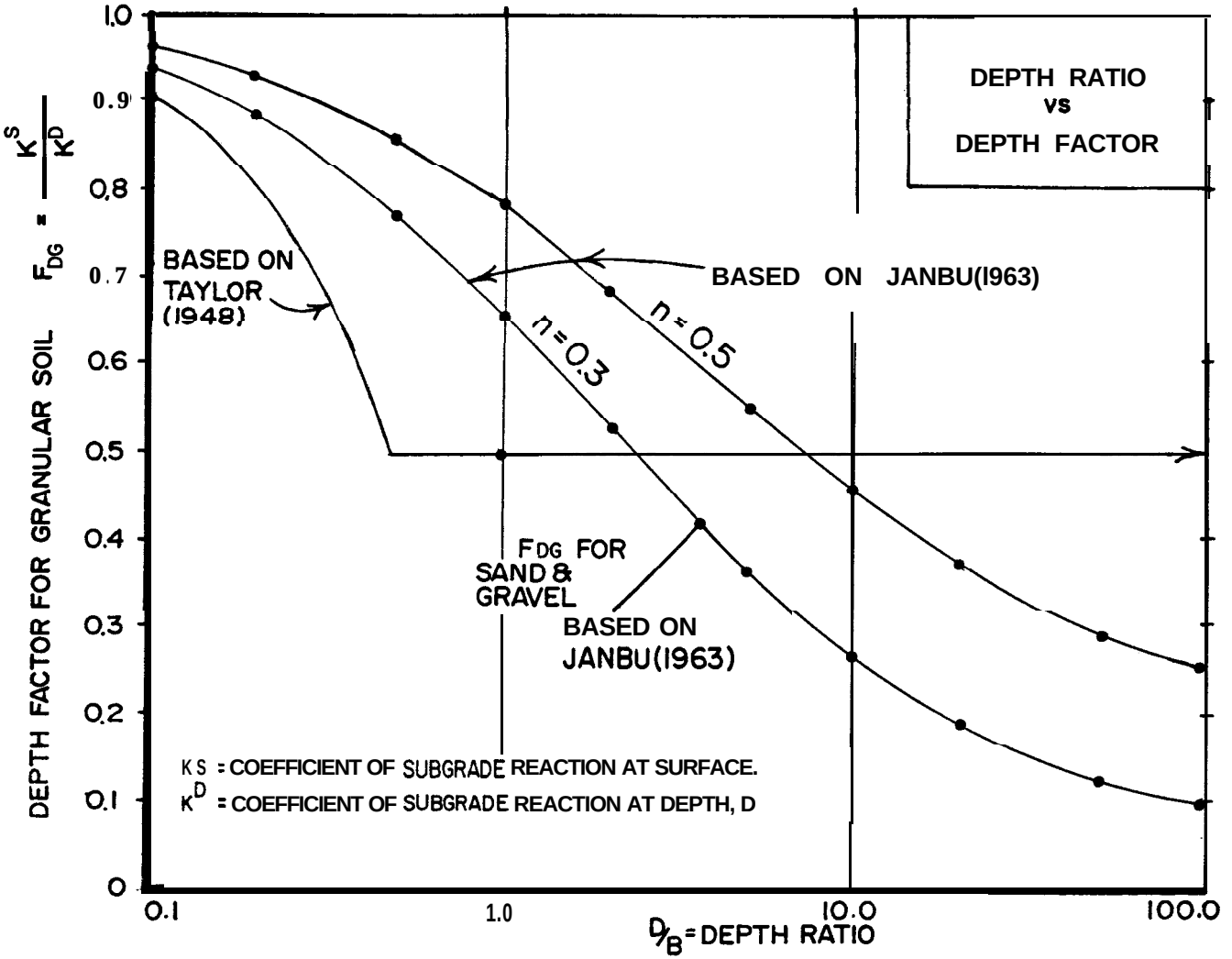


Figure 49. Influence of depth on coefficient of subgrade reaction for granular soils (based on modulus of deformation that increases with depth).

where:

q = load in tsf

k = coefficient of subgrade reaction in tsf/ft

ρ = settlement in feet

Determine k by first determining k_1 at the surface of the soil from Figure 46 or from plate load test. Modify k_1 as follows:

$$k = \frac{k_1}{B} \left(\frac{F_S}{F_D}\right)$$

where:

F_S = shape factor from Figure 47

F_D = depth factor from Figure 48

B = least dimension of bearing area in feet

Sands

Assume modulus of deformation increases with depth. Compute settlement from Eq. 7.41.2 as above. Determine k by first determining k_1 as above. Modify k_1 as follows to account for size, shape, and depth.

$$k = \frac{k_1 \times F_S}{F_D \times F_{DG}} \left(\frac{B + 1}{2B}\right)^2$$

where:

F_S, F_D and B as defined above

F_{DG} = depth factor for granular soil from Figure 49

8.10 PURPOSE AND SCOPE

This section is a synthesis of the main conclusions concerning the performance of underpinning and of various techniques for supporting open excavations. The general applicability of each of the various techniques is discussed, and comparisons are made, when appropriate, in evaluating 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 may contribute to excessive horizontal and vertical displacements in the adjacent ground. Finally, some general guidelines are provided on costs.

8.20 GENERAL CONCLUSIONS CONCERNING DISPLACEMENTS

8.21 Lateral Support Methods

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

a. For these soil types the displacements reported in the literature on well- constructed, well-documented projects are of insufficient magnitude to distinguish variations that may be caused by wall type or method of lateral support. Nevertheless, there is strong evidence to suggest that use of concrete diaphragm walls will result in less displacement than other wall types and some evidence that walls supported by tiebacks will perform better than internally braced walls

b. Maximum displacements are typically 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 the depth of the cut for flexible walls. The use of concrete diaphragm walls reduces the magnitude of displacements'to about 0.25 percent of the depth of the cut -- or about the same as those observed for competent soils.

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

f. When the excavation is in 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, Wall stiffness can be increased by using rigid concrete walls or by reducing spacing between support levels. It is believed that comparable watt stiffness (defined as $\frac{EI}{L^4}$) will result in comparable performance provided that the installations are carefully performed.

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 comparative data are not available. Sheet pile walls are rarely used in these soil types because of the hard driving conditions.

Effect of Wall Stiffness in Cohesive Soil

j. The influence of wall stiffness and of stability number of cohesive soil (defined as $N = \frac{\gamma H}{S_u}$) was examined in some detail. The data show increasing displacementswith 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.

8.22 Underpinning

The underpinning process has an inherent source of deformation resulting from the transfer of Load from the existing foundation to the new foundation. Well-executed construction procedures can normally control this vertical displacement to 1/2 inch or less.

The underpinning elements may also be influenced by the adjacent excavation because the underpinning elements will be installed within the zone of vertical and horizontal displacements. This may result in additional displacements and/or additional Load on the underpinning elements, Experience has shown that horizontal movements cause more damage than vertical movements.

8.30 WALL TYPE

8. 31 Concrete Diaphragm Walls

8, 31. 1 Applicability

Diaphragm walls can be used in virtually 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 generally been used as a method of ground support without being incorporated into the permanent structure.

8.31.2 Operational Considerations

Soil and water conditions can adversely affect diaphr a gm wall cons tr uction. Of particular concern are excavations in very pervious soils (fluid loss), contamination of the fluid (adverse pH, high salinity, or high calcium content), and spalling of the trench wall. Spalling of the trench wall may be caused by unstable soils or loose fill, particularly when containing miscellaneous rubble or old foundations. It is believed that most of the problems can be identified during initial investigation and controlled during construction.

8. 32 Soldier Pile Walls

8. 32. 1 Applicability

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

8. 32.2 Operational Considerations

The following items may cause displacements: deflection of lagging, overcut behind lagging, ground loss caused by surface and ground water flow, and ground loss associated with pre-excavation for soldier piles. Broken water mains or flooding may cause heavy water flow toward the excavation. This water flow is an additional risk in soldier pile walls.

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 soils within the depth of the excavation. This sequence makes it extremely difficult to fully dewater to the lowest extent of the water-bearing formation.

8.33 Steel Sheet Pile Walls

8.33. 1 Applicability

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

8.33.2 Operational Considerations

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

Tearing of interlocks under hard driving conditions may cause ground Loss because of ground water infiltration through the torn inter Locks .

While interlocked steel sheet piling effectively intercepts ground water flow within pervious layers, the piezometric level outside the excavation will often be depressed in impervious soil strata. 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 lead to settlement of adjacent ground.

8.40 SUPPORT METHOD

8.41 Tiebacks

8. 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 often experience relatively Large movements.

8.41.2 Operational Considerations

A number of operational contingencies are listed and commented on below.

Vertical Wall Movement

The vertical components of Load may cause settlement of soldier pile walls, and this may Lead to horizontal wall 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.

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 procedur es, and soil conditions. Water flow alone may 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 over consolidated clays and soft shales. The movement is believed to be associated with Lateral expansion following stress relief from the excavation.

Another potential source of Later al creep is in the presence of a weak Layer of cohesive soil below the excavation.

8.42 Internal Bracing

8.42. 1 Applicability

Internal bracing is most applicable to situations where a reasonably economical section can be used without need of

intermediate support, As the distance between the sides of the excavation increases, internal bracing becomes less efficient, and therefore tiebacks become more attractive. In some cases inclined rakers are economical alternatives.

8.42.2 Operational Considerations

The most important contingency is believed to be the connection details, especially alignment of members and welding.

Displacements may occur 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 them with reflective silver paint.

8.50 UNDERPINNING

8.51 Applicability

Underpinning elements transfer the load from an existing foundation to a new foundation bearing below the zone of influence of the adjacent excavation. The decision to underpinn a structure is based on several factors including the cost of underpinning, the cost of alternatives, expediency, and risk.

8.52 Operational Considerations

A thorough study of the structure to be underpinned should be made to determine load and Load distribution. Temporary conditions that occur during underpinning will also require evaluation. Because the elements pass through a zone undergoing vertical and horizontal displacement, underpinning elements may be subject to downdrag forces, lateral forces, and/or movement. Lateral movements have been a source of great damage.

A number of factors may cause ground Loss. Lagged underpinning pits for construction of piers have many of the same contingencies mentioned previously for soldier pile walls. The potential for ground loss also exists when "blow conditions" develop in open shafts or open-ended piles below gound water table,

8.60 STABILIZATION METHODS

8. 61 Scope

This section is a brief overview of grouting and freezing. 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 sub - contractors, 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.

8. 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,

Less expensive grouts (cement and bentonite) are used in coarse sands and gravels. Silicates may be used in fine to medium sands. The most expensive grouts are the organic grouts, which are used for fine sands and coarse silts. In stratified deposits multi-stage grouting consists of grouting with 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 - gr ained soils.

8. 63 Ground Freezing

Ground freezing methods have been used primarily in conjunction with shafts and small diameter tunnels. Frequently, it has

been used in difficult ground water situations where more conventional methods have failed or are inadequate. However, the use of ground freezing as a primary construction method is increasing.

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.

8.70 SOIL AND GROUND WATER CONDITIONS

The following is a brief check list of soil conditions that may contribute 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 over consolidated clay: These may display Lateral creep in tieback installations or may contribute toward load buildup in braced excavations.

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. Inadequate toe restraint for soldier piles;

e. Inability to completely dewater overlying soils to the top of the 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, 1974a).

4. Pervious soils underlain by impervious soil within the depth of excavation: 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. stresses.

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

8.80 COSTS

8.81 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 compare 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.

8.82 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) 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).

8. 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, tie - backs 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.

8.83.1 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 including the wale and connections, are as follows:

Depth (feet)	Cost per Square Foot[1] Soldier Piles and Wood Lagging[2]	Interlocked Sheet Piles[2]
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.

8.83.2 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' 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

[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.

8. 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 Materials150 - $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.

8.85 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 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 r ound- 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.

8.86 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 - $1. 30/$ft^3$; chemical grouts: $1.50 - $7. 00/$ft^3$); 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 - $35. 00/$yd^3$ of grouted soil; chemical grouts: $40 - $190/$yd^3$ of grouted soil).

CHAPTER 9 - SOLDIER PILE WALLS

9.10 INTRODUCTION

Soldier pile walls have two basic components, soldier piles usually set at 6 to 10 foot spacings and lagging which spans the distance between the soldier piles. The soldier piles must carry the full earth pressure load while the lagging must resist relatively minor earth pressure loads.

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. However, soldier piles can be almost any structural member- -pipe section, cast-in-place concrete, or precast concrete,

Figure 50 shows various types of steel soldier piles.

9.20 TYPES OF SOLDIER PILE WALLS

9.21 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 behind or in front of the flange next to the excavation (front flange). As noted in Figure 50a, 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.

Figure 51 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 continuouslyacross several soldier piles. This is not possible when lagging is installed behind the front flange.

Spacers between the lagging boards (called "louvres") allow the introduction of material for backpacking boards and filter-

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

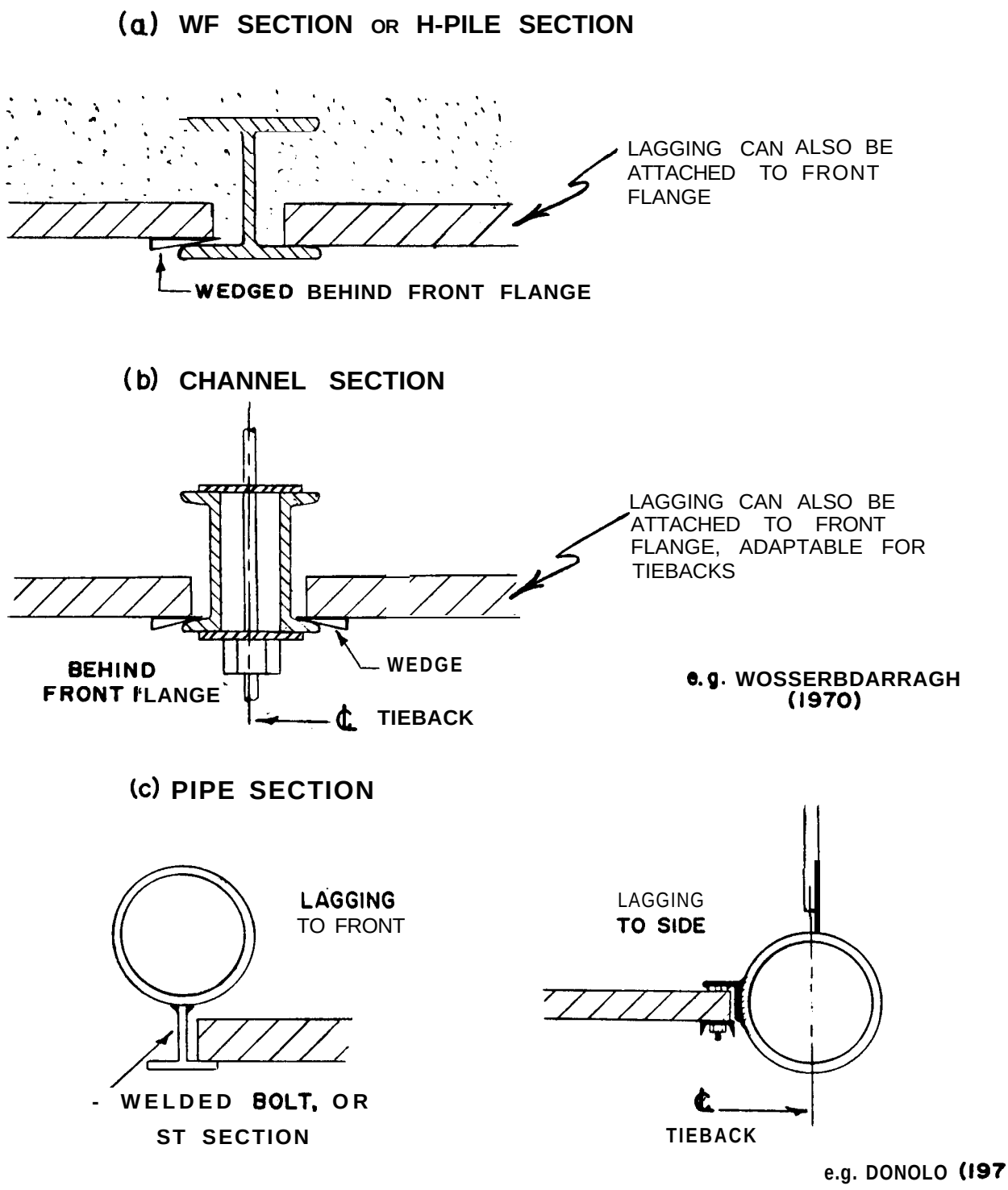
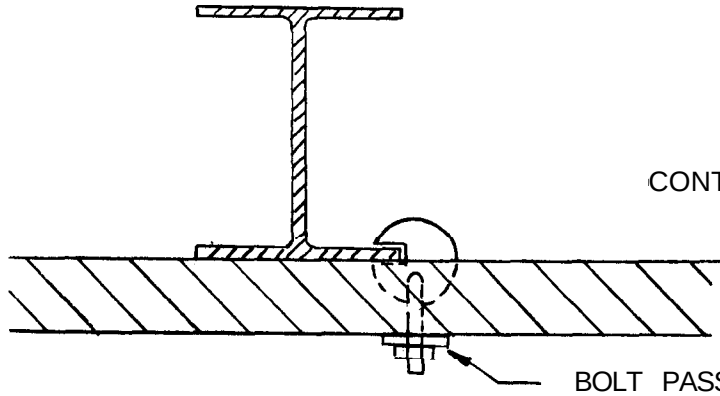


Figure 50. Steel soldier piles.

(a) CONTACT SHEETING

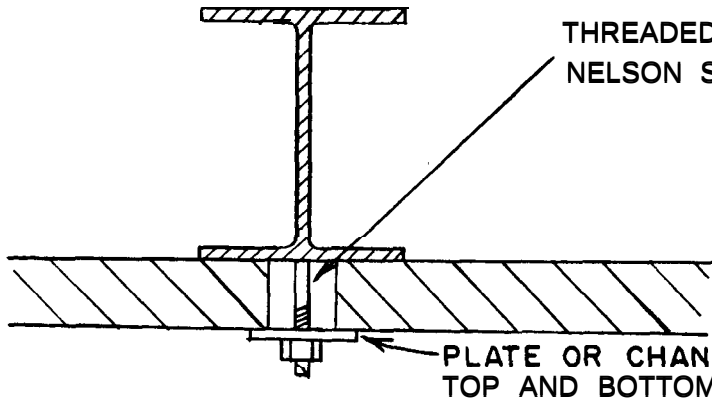


CONTACT SHEETING INCORPORATE0

(NYACK , N .Y.)

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

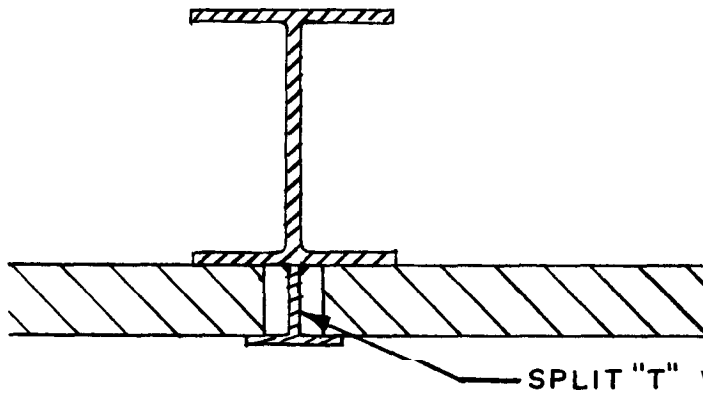
(b) BOLT



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 51. Wood lagging to front flange,

ing soil to protect against ground loss from seepage. In slow draining ground 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 52).

9.22 Concrete Wall

Examples of shotcrete or poured concrete walls constructed in conjunction with steel soldier piles are shown in Figure 53. An application with precast concrete soldier piles is shown in Figure 54. In general, the typical construction 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 the soil would have to have sufficient cohesion to stand up while the section of the wall is completed.

Figure55 shows precast soldier piles shaped to receive either wood lagging or precast concrete lagging.

9.23 Soldier Pile 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 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.

9.30 DESIGN CONSIDERATIONS

9.31 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.

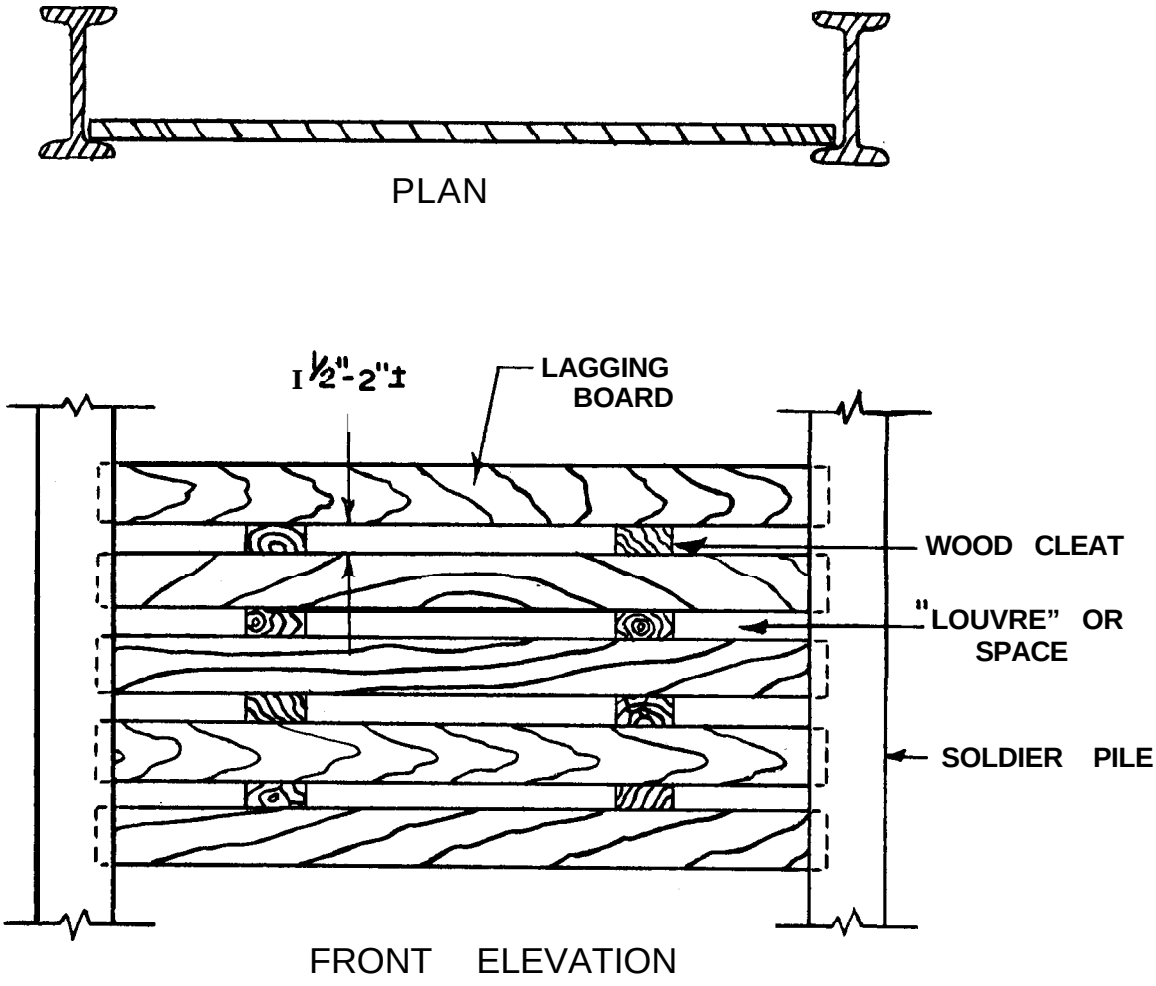
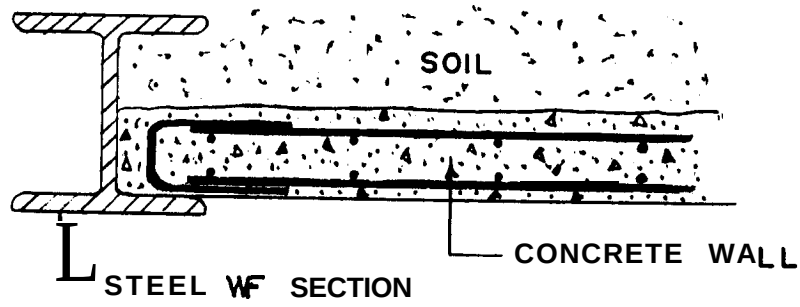


Figure 52. Louvre effect for wall Lagging.

(a) CAST IN PLACE



(b) SHOTCRETE

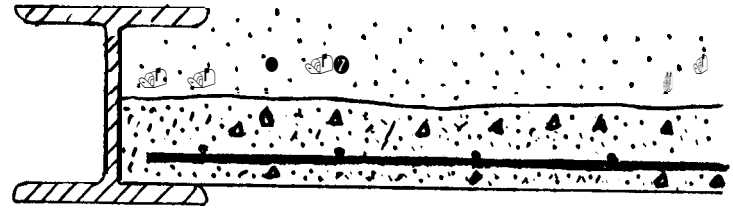


Figure 53. Concrete infill between soldier piles.

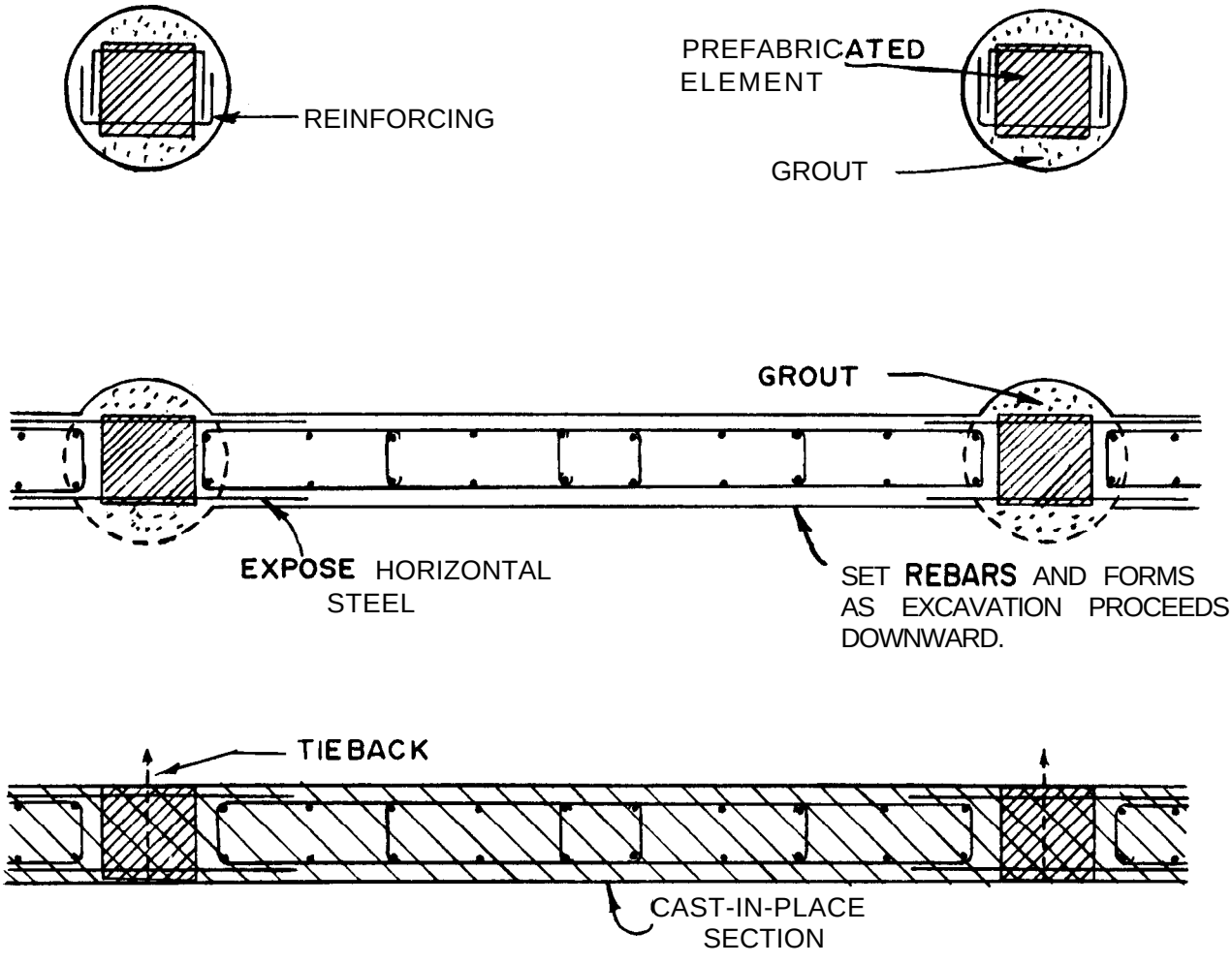


Figure 54. Parisienne wall, precast soldier piles with formed cast-in-place wall, (after Fenoux, 1974; Xanthakos, 1974; and D'Appolonia, et al, 1974).

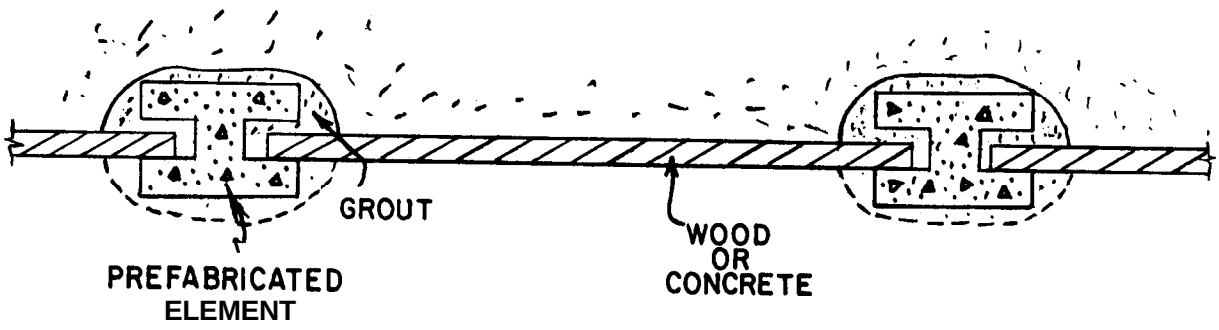


Figure 55. Berlin wall, precast soldier piles with wood or precast concrete lagging (after Fenoux, 1974).

9.32 Wood Lagging

9.32.1 Wood Materials

The most common wood used for lagging in the United States is construction grade lumber, 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.

Table 3 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,

9.32.2 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 pressure concentrates on the relatively stiff soldier piles; less pressure is applied to the more flexible lagging between the soldier piles,

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.

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 soldier piles.

9.32.3 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 pile (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 Transit Authority uses the basic pressure

Table 3. Strength properties for typical grades of timber.

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

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

diagram but allows a 50 percent increase in the allowable flexural stress of stress graded lumber.

9.32.4 Recommended Lagging Thickness

A table of recommended thicknesses has been developed and is presented as Table 4. Since the table has been developed on the basis of construction grade lumber, adjustments are required for stress-graded lumber.

The so-called "competent soils" shown herein are typically either granular with relatively high angles of internal friction or stiff to very stiff clays. Medium clays included in the table are those with 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 and soils having a tendency to run when saturated, Heavily overconsolidated fissured clays are also included because they have a tendency to expand laterally, especially in deep excavations.

9.33 Displacements and Loss of Ground

9.33.1 General

Important factors contributing to ground loss are the soil in 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 construction, The discussion does not deal with overall deformations of the retained earth mass.

9.33.2 Deflection of Lagging

The lagging board thicknesses recommended in Table 4 will generally maintain deflection to less than about 1 inch.

9.33.3 Overcut

Movements caused by overcut are best controlled by effective packing of soil behind lagging. The most effective way of backpacking is to ram the soil into the space from the upperside of each 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 mixed

Table 4. 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							
	Sands and gravels (medium dense to dense).	GW, GP, GM, GC, SW, SP, SM	0' to 25'	2"	3"	3"	3"	4"	4"
	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							
	Clayey sands (medium dense to dense) below water table.	SC	0' to 25'	3"	3"	3"	4"	4"	5"
	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.

with cement and dry packed. Louvres also aid in backpacking. Louvres also provide an opportunity to take remedial measures to improve filtering or to correct for ground loss behind previously installed lagging.

V. 33.4 Inherent Soil Properties

Soft clays and loose soils of low plasticity below the water table are of particular concern. The physical act of exposing a face in these soils below the last placed lagging board may in itself provide the opportunity for deformation.

An example of a rather dramatic failure after exposure of soft sensitive clay was reported by Broms and Bjerke (1973). Examples of a German procedure for dealing with soft unstable soils are shown in Figure 56.

The extent of stress relief caused by arching that occurs with very soft soils and soils subject to plastic creep is in que stion. It is recommended that the pressure used for lagging design be determined directly from the basic pressure diagram used for design of struts and vertical members.

A procedure for constructing walls in silts and other soils that are difficult to drain, is to dewater in advance of excavation. An alternative procedure would be to maintain continuously 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.

Dry cohesionless soil may also cause difficulties, especially in hot, arid areas. Under these circumstances, one remedial technique is to moisten the face by spraying while placing the lagging board. Another technique is to use a board such as plywood to hold the soil temporarily in place while setting lagging.

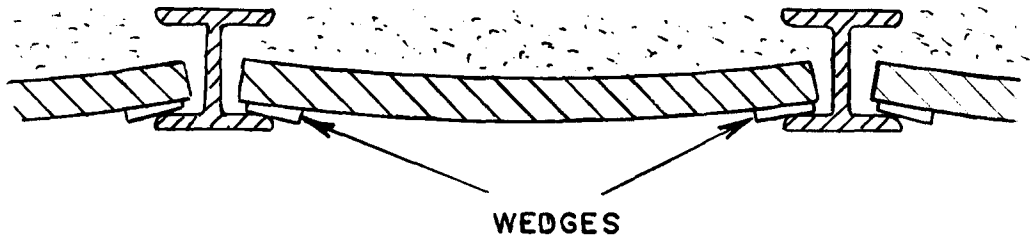
9.33.5 Pre-excavation for Soldier Piles

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

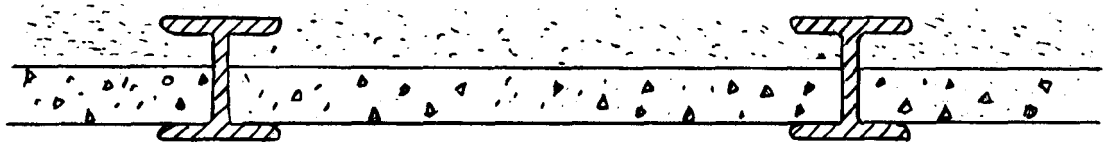
One cause is from the suction effect that occurs during withdrawal of the auger. One way to prevent this is to provide ports within the auger which will prohibit the suction from developing below the auger. Another is to apply pressure to the inner hole of a hollow stem auger as it is withdrawn.

A second cause of potential ground loss is from

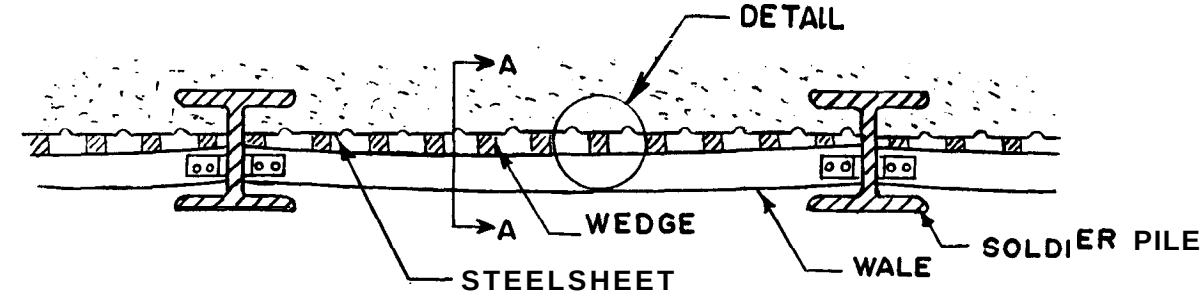
(a) CURVED SOIL FACE AND DOUBLE WEDGING.



(b) CONCRETE



(c) VERTICAL STEEL SHEETS IN SOFT SOIL





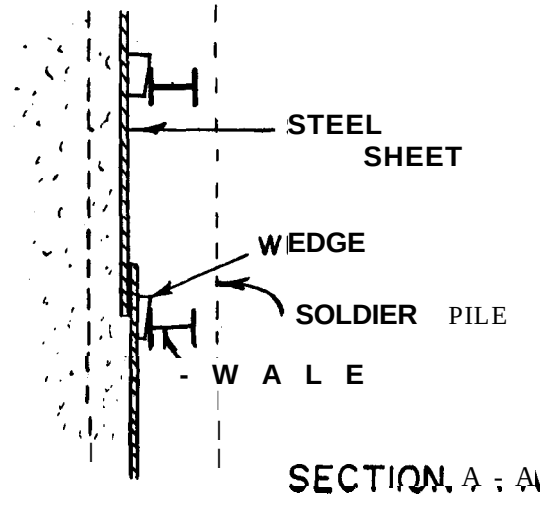


Figure 56. German techniques to prevent deformations (after Weissenbach, 1972).

collapse of soil into the augered hole. This can be prevented by using casing or by using a bentonite slurry suspension to stablilize the hole.

A third possible cause of ground loss is from improperly filling the pre-excavated hole following insertion of the soldier pile. 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 from the space between lagging boards, and carried out a significant quantity of soil.

9.33.6 Surface Water and Ground Water

In any water-bearing cohesionless formation it is absolutely essential that the ground water be drained piror to exposing the face. The depth of the cut below the water table, the porosity and permeability of soil, and the presence of underlying or interbedded impervious layers must all be considered in devising a dewatering s cheme.

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.

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).

Protection against ground 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 of preventing erosion is using porous concrete as a filter behind the lagging. Such a procedure was reported by Mansur and Alizadeh (1970).

9.40 CONSTRUCTION CONSIDERATIONS

9.41 Soldier Piles

9.41.1 Driven Soldier Piles

Conventional pile drivers may be used to drive soldier piles. Bearing pile sections are the most desirable sections

for driving. In hard ground, bearing piles may be equipped with a driving point to help penetrate boulders and/or to get sufficient depth for adequate lateral resistance or bearing capacity.

9.41. 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 conveninetly 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 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.

Pre-excavation 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 hard ground augers may not be practical. Instead, percussion drilling or rotary drilling may be necessary.

Pre-excavated holes facilitate setting the soldier piles to a very close tolerance, both vertically and in plan. Where alignment is critical, the soldier pile is set within the pre-excavated hole with a centering spider,

It is common practice to use structural concrete below the level of the excavation to assure vertical bearing and lateral resistance against kick out and to use lean concrete for the rest of the hole. It is believed that properly placed lean concrete can also be used below the excavation level.

9.42 Installation of Lagging

To minimize overcut, hand tools should be used to shape the soil and to fit the lagging board in place. If necessary, wedges can be used to close the space between the lagging board and its bearing area,

The depth of exposure below the last placed lagging board may be as little as 1 foot, as in the case of saturated silt, or as much as 4 or 5 feet in cohesive hardpan. The restriction in depth of unsupported cut is the height of cut that is stable. If the unsupported cut below the last board is unstable, excessive loss of ground may occur.

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. Louvered lagging is considered good practice,

9.43 Removal

There is a divergence of opinion among practitioners with regard to whether or not untreated wood can be left in place permanently above the ground water table. In this present state of diverse opinion, the preferred options are to remove lagging that would be permanently above the ground water table or to treat the wood with chemicals for the purpose of preventing 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 voids are not created.

Treatment standards are shown in Table 5.

Table 5. AWPA 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
Osmose 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 10 - STEEL SHEET PILING

10.10 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.

10.20 DESIGN CONSIDERATIONS

10.2 1 General Applications

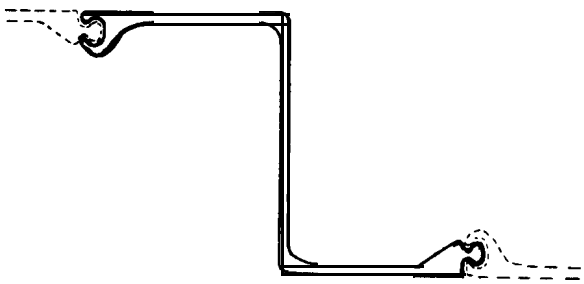
Steel sheet piling is typically used in soils that do not permit easy placement of lagging, such as soft clays, saturated silts, or loose silty or clayey sand. These soils are potentially unstable when they are exposed during excavation.

Interlocked steel sheet piling is very 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).

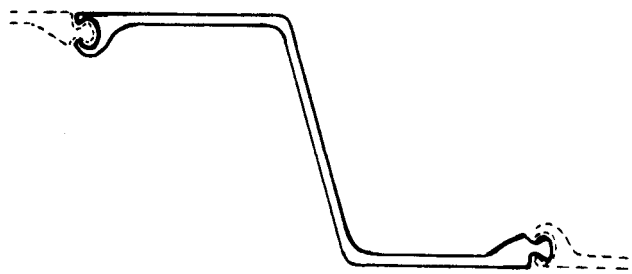
10.22 Available Sections

Figure 57 schematically shows typical American steel sheet pile sections used for relatively deep excavations. Table 6 gives information concerning the properties of various steel sheet pile sections (see Figure, 58and Table 7 for foreign sections,). Heavier sections are available in foreign steel sheet piling than in domestic piling.

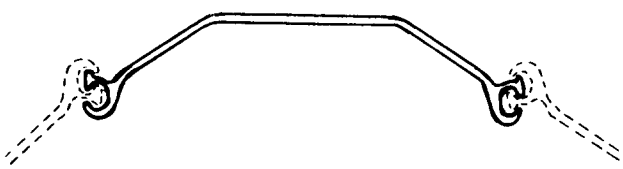
Note that the PDA section and PMA section interlock on the midline of the wall, whereas the "Z" sections interlock on the inside and the outside line of the wall. For the deep arch and medium arch sections, it is conventionally assumed in American practice that shear cannot develop along the interlocks 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



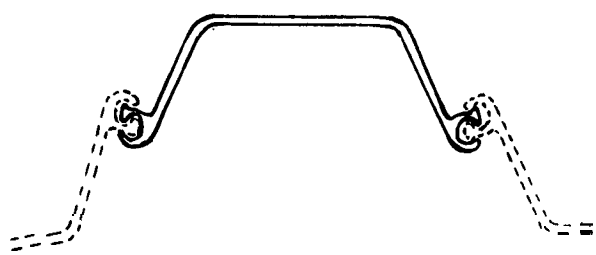
PZ 38



PZ 32 8 PZ 27



PMA 22



PDA 27

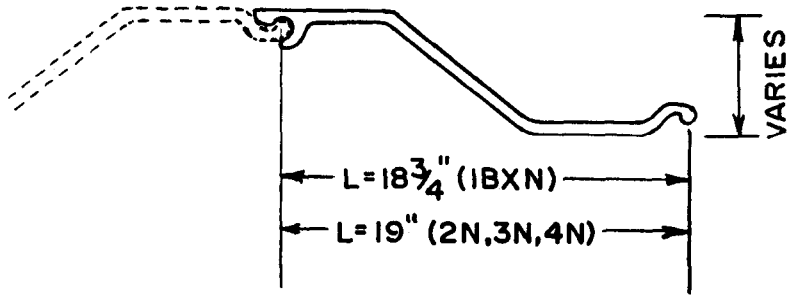
Figure 57. Domestic sheet pile sections.

Table 6. Domestic steel sheet pile sections.

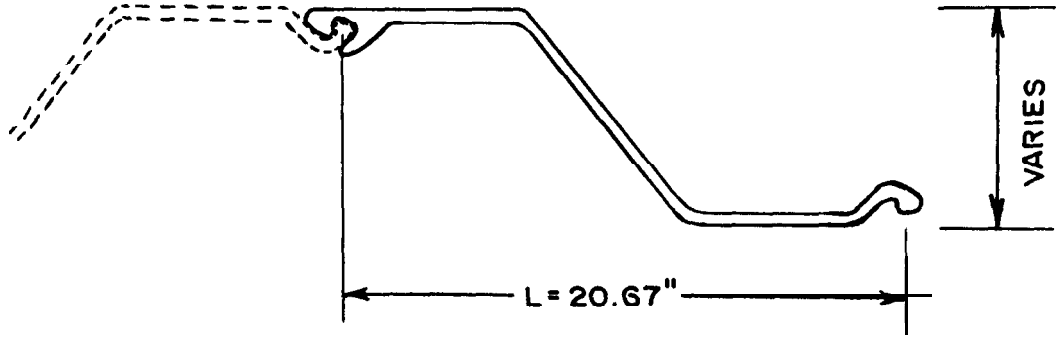
Section	Dimension (in)		Weight lb/ sf	Moment of Inertia in^4/ft	Section Modulus in^3/ft
	D, depth	L, length			
PMA 22	3-1/2 x 2 = 7 (1)	19.6	22. 0	16	5.4
PDA 27	5 x 2 = 1 0	16	27. 0	40	10.7
PZ 27	12	18	27. 0	183	30.2
PZ 32	11.5	21	32. 0	220	38. 3
PZ 38	12. 0	18	38. 0	281	46.8

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

FRODINGHAM 1BXN, 2N, 3N, 4N



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



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

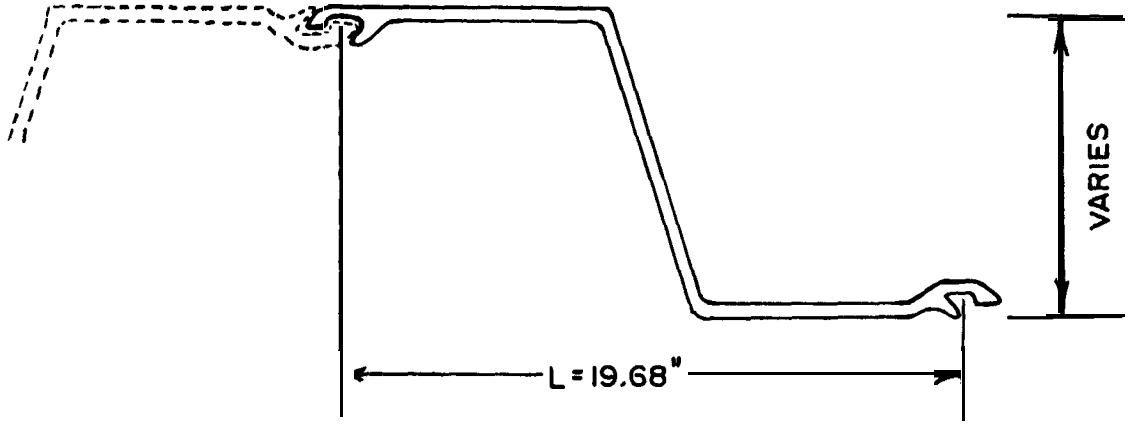


Figure 58. Foreign sheet pile sections.

Table 7. Foreign steel sheet pile sections.

Section	Dimension (in) D, depth	Dimension (in) L, length	Weight lb/sf	Moment of Inertia in^4/ft	Section Modulus in^3/ft
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.

therefore takes advantage of the full section modulus of both piles (Tschebotarioff, 1974).

10.23 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. 8).

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.

10.30 CONSTRUCTION CONSIDERATIONS

10. 31 Installation of Sheet Piling

The general installation 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 hammers , or vibratory drivers. The vibratory drivers are run by hydraulic or electric motors which power eccentric shafts (Foster, 1971).

Silent pile drivers have been developed by Stabilator AB of Stockholm, Sweden, and by Taylor Woodrow Construction, Ltd., of Great Britain. The former operates by compressed air, the latter by hydraulic rams.

Perhaps the greatest cause of ground water leakage and/or loss of ground is the ripping of sheeting 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.

10. 32 Removal of Sheet Piling

Conventional extractors can be used. Loose granular soils

Table 8. 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

*$f_b = .66\, f_y$

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 lead 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 11 - CONCRETE DIAPHRAGM WALLS

11.10 INTRODUCTION

The term concrete diaphragm wall refers to a continuous concrete wall built from the ground surface, The walls may consist of precast or cast-in-place concrete panels or contiguous bored concrete piles,

The most common wall type is a tremie concrete, diaphragm wall cast within a slurry stabilized trench. The trenches are usually about 24 to 36 inches wide and are excavated in 10 to 20 foot lengths. After the individual panels are excavated, end stops and reinforcing are placed. Concrete is poured, and the end stops are removed. Once the concrete has set, the neighboring panel can be excavated.

The system using precast concrete panels lowered into a slurry stabilized trench is quite popular in Europe. The use of this system and concrete diaphragm walls in general is expected to increase in the United States.

Figures 59, 60, 61 and 62 show various aspects of diaphragm wall construction.

11.20 PROPERTIES OF BENTONITE SLURRY

The most commonly used bentonite contains the clay mineral, sodium montmorillonite. When mixed with water, bentonite forms a colloidal suspension (slurry). Following agitation, a bentonite slurry will gel and develop shear resistance. Bentonite also displays plastic viscosity, which means additional shear resistance develops depending upon the rate of shear application.

Fluids which develop both gel shear strength and additional shear resistance from fluid viscosity are known as Bingham fluids. Fluids which have no shear resistance in the static conditions but do have viscosity characteristics are known as Newtonian fluids. Figure 63 illustrates the viscous character of Bingham and Newtonian fluids.

Bentonite fluid is thixotropic; that is, it will lose strength when disturbed but will gain strength and gel when left undisturbed. In diaphragm wall construction it is desirable to maintain a fluid slurry which requires that the slurry be circulated and agitated.

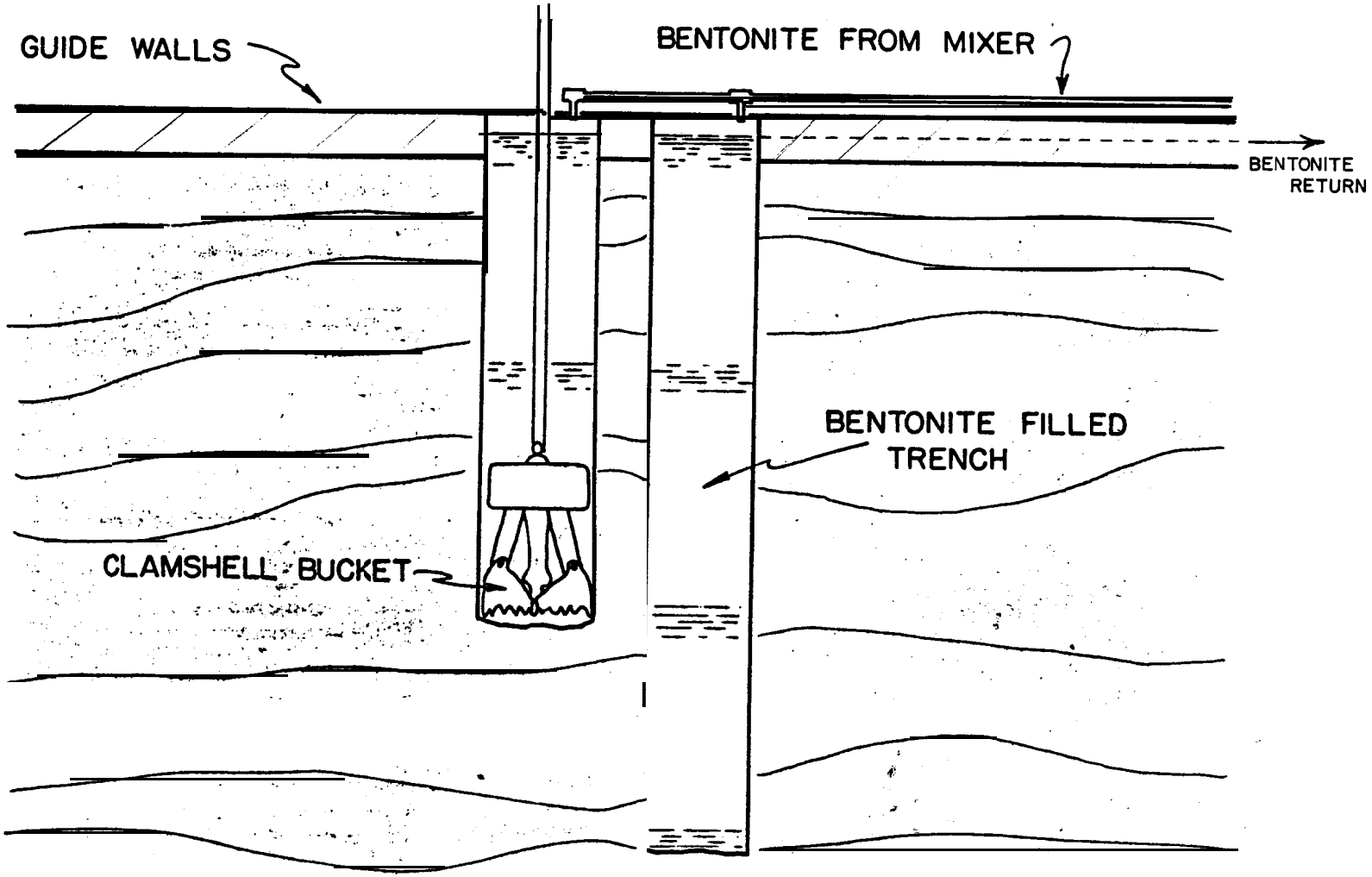


Figure 59. Excavation by clamshell bucket.
(Courtesy of ICOS Corporation).

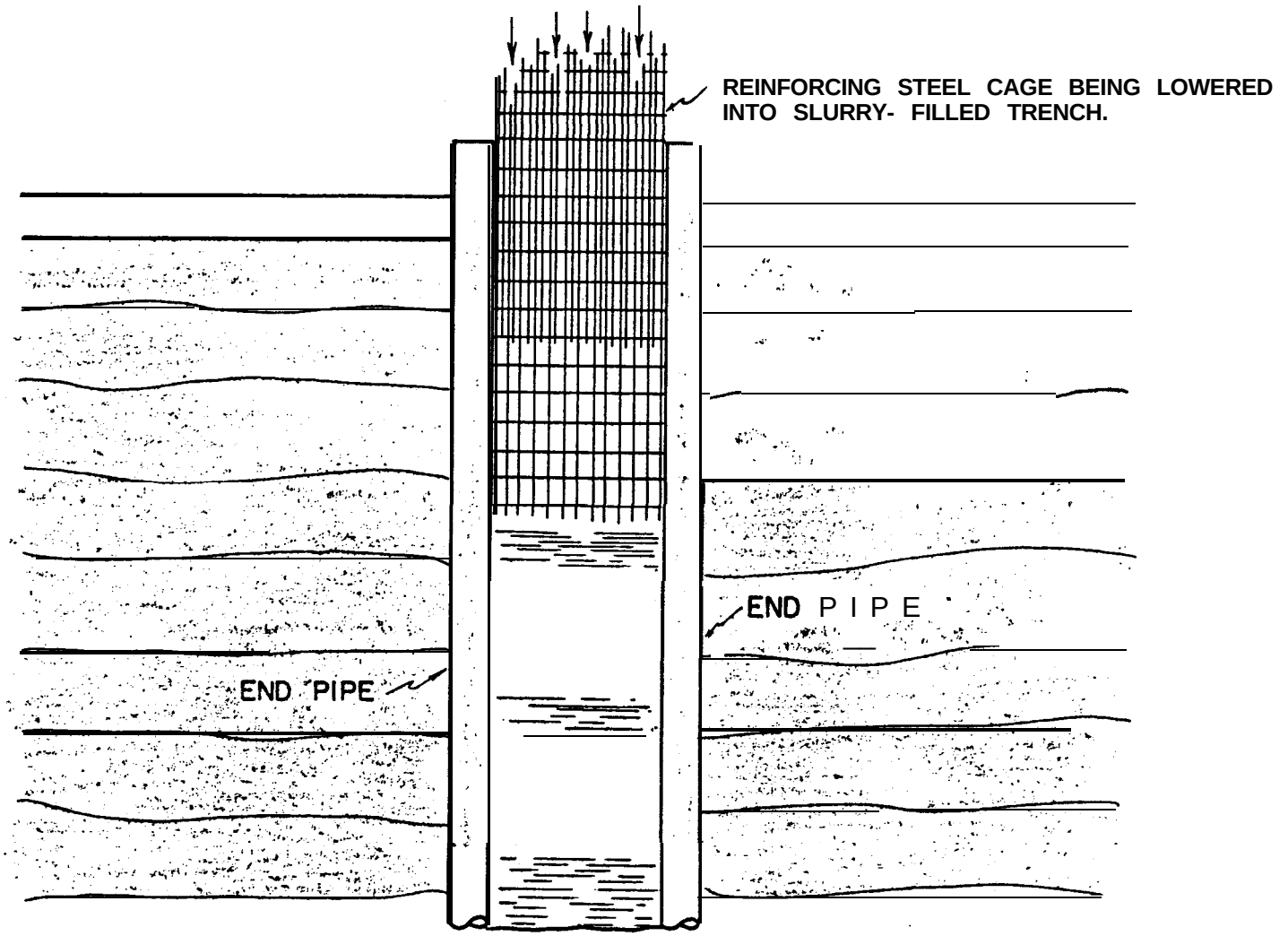


Figure 60. Preparations for concrete placement. (Courtesy of ICOS Corporation).

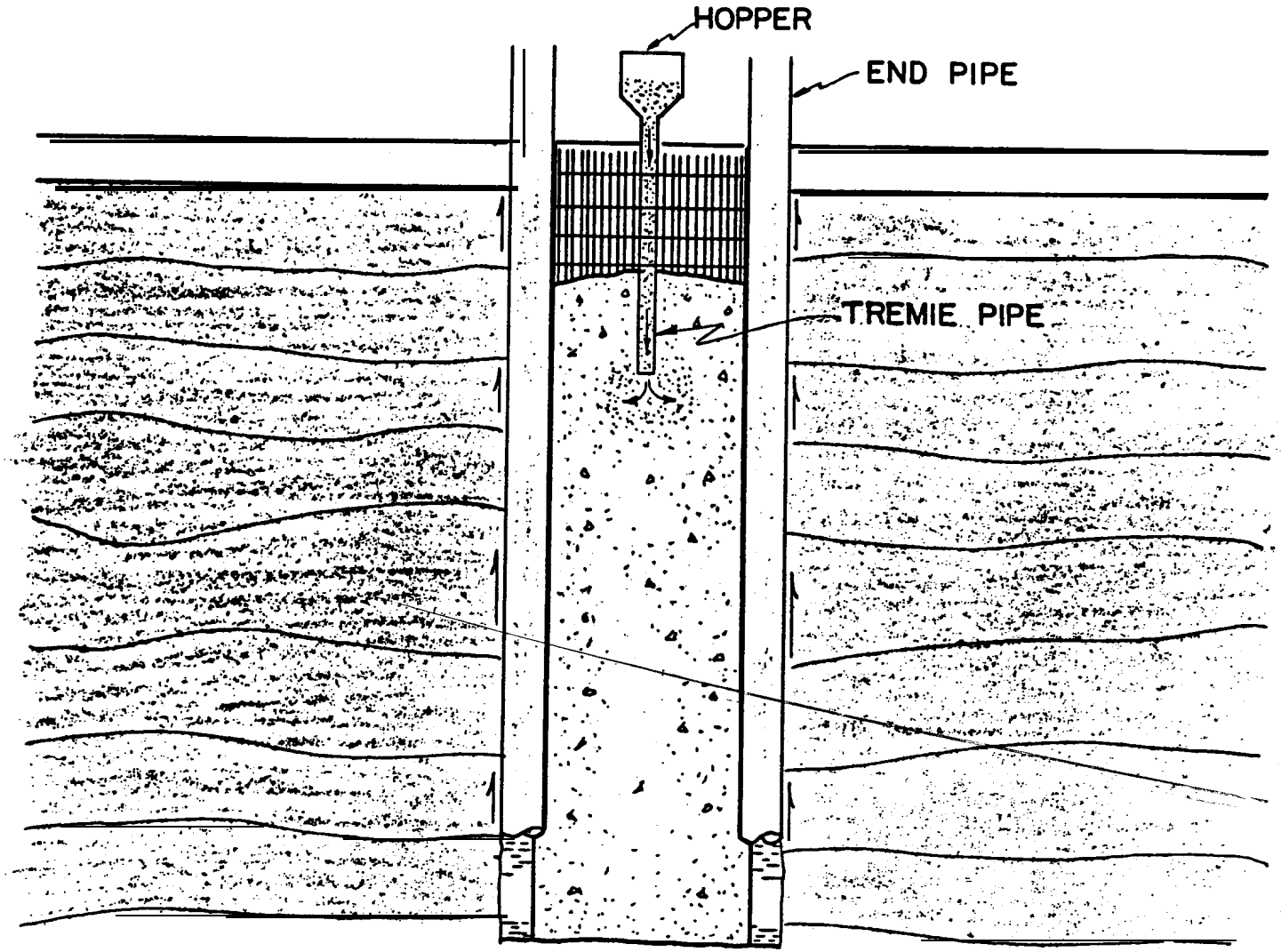


Figure 61. Placement of concrete.
(Courtesy of ICOS Corporation).

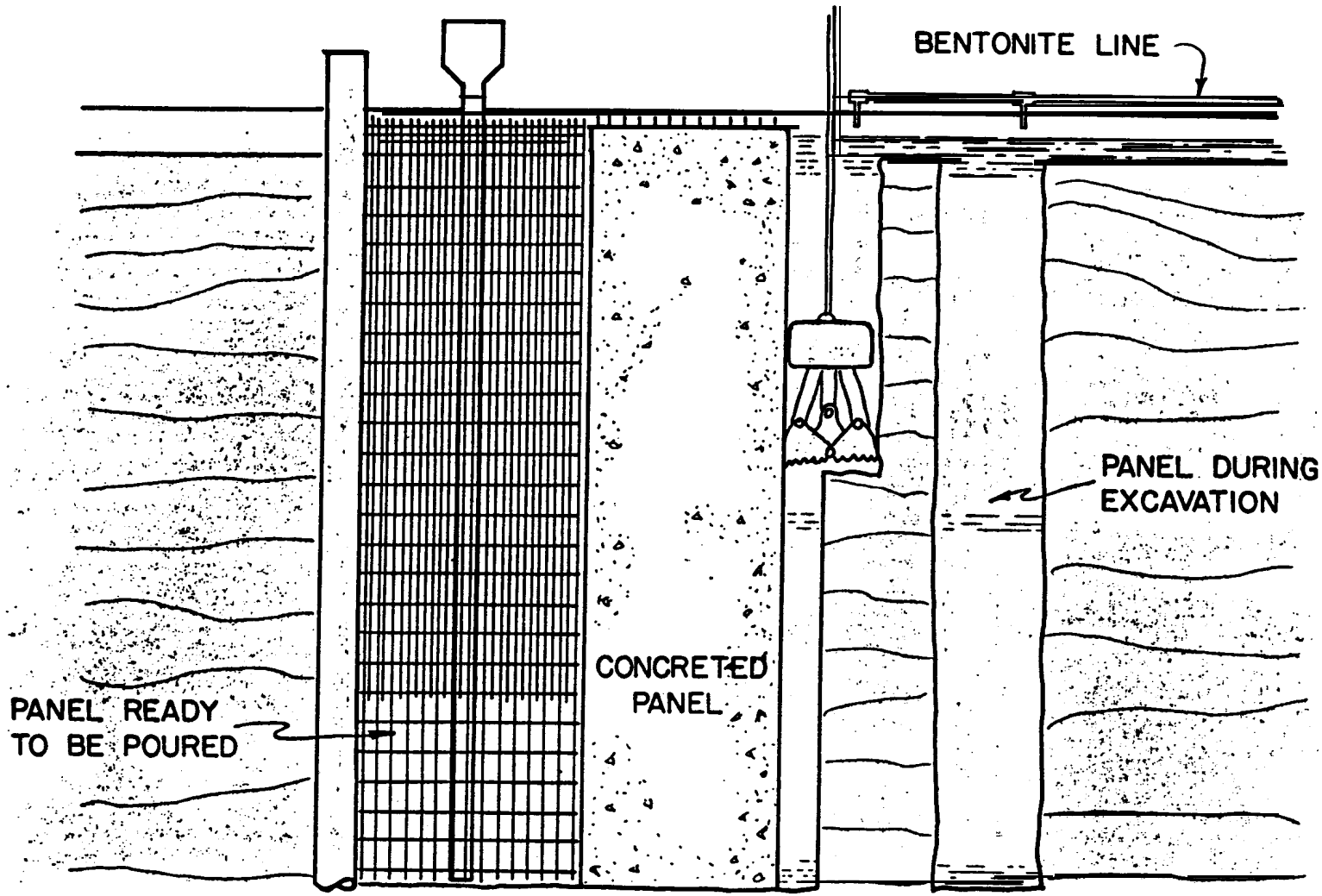
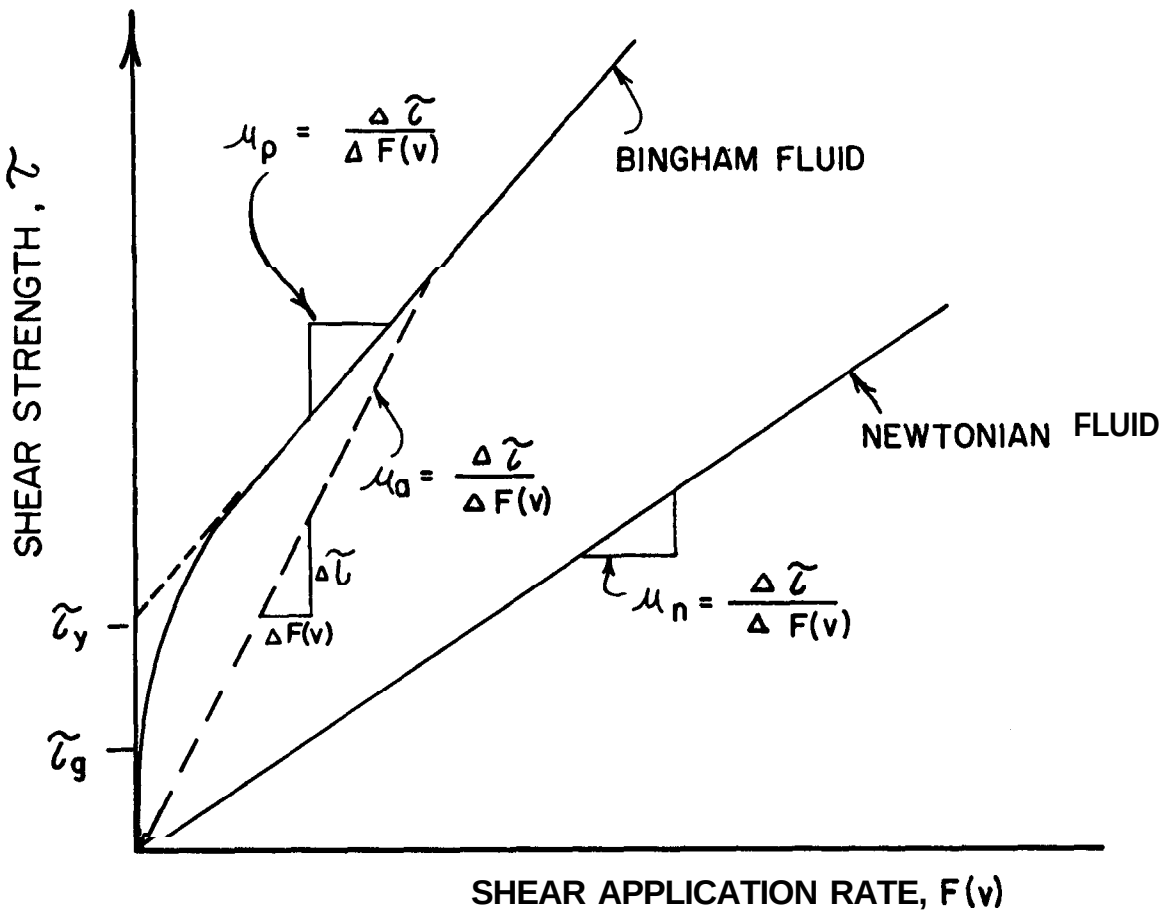


Figure 62. Different phases of construction.
(Courtesy of ICOS Corporation).



τ_g = Gel shear strength

τ_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 63. Viscous behavior of Newtonian and Bingham fluids.

11.30 SLURRY TRENCH STABILITY

11.31 General

Primarily, it is the fluid pressure of the slurry in combination with arching in the ground that maintains trench stability in cohesionless soil. In addition, some local penetration into the pervious, soil will impart cohesion to the soil and will prevent spalling.

The bentonite slurry in the trench is maintained at a higher elevation than the surrounding ground water table. By a combination of hydrostatic pressure, osmotic pressure, and electrolytic properties of the colloid, a membrane or "mudcake" forms against the walls of the trench. The effect of this "mudcake" is to prevent fluid loss and to maintain the fluid pressure against the trench wall.

11.32 Mudcake

The extent of fluid penetration into the soil voids depends upon the permeability of the soil and the properties of the colloid.' With very pervious soils such as sands and gravels, having permeabilities greater than 10^{-1} cm/sec, there could be free penetration of the slurry into the soil without the formation of a "mudcake". With soils having permeabilities between 10^{-2} and lb^{-1} cm/sec, there may be some time lag associated with the development of an impervious "mudcake". With soils having permeabilities less than 10^{-2} cm/sec, the depth of penetration for formation of an impervious "mudcake" is minimal and there is essentially no time lag (Hutchinson, 1974).

With impervious soils, such as clay, the bentonite need not form a "mudcake" because the clay itself is essentially impermeable. In these cases the bentonite protects against fluid loss through pervious seams that may be interbedded within the parent clay formation.

11. 33 Pressure of Slurry Fluid

11.33.1 General

It is common practice to maintain the water level in the trench at least 4 feet above the ground water level. This excess head in combination with the greater specific gravity of the slurry creates an unbalanced pressure on the trench walls which provides a force resisting a trench stability failure. Typically, the bentonite concentration is about 4 to 6 percent by weight yielding specific gravities of approximately 1.023 to 1.034.

11. 33.2 Stability Analysis

A number of simplified stability analyses are presented in detail in Volume III (Construction Methods). All of these cases examine trench stability on the basis of fluid pressure. None of the cases consider arching which is a significant stabilizing factor for panel excavations on the order of 15 feet or less.

Since panel lengths and fluid levels required to maintenance trench stability are established largely on the basis of experience and/or trial panel sections, stability analyses are not generally used to evaluate trench stability. Their main value is helping to assess the relative importance of fluid pressure and the depth of the trench on overall stability.

11.34 Arching

In order to understand arching, the redistribution of stresses away from plane strain conditions, 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. Horizontal strain is less near the ends of the panel than near the center of the panel. As a result, load concentrates at the ends of the excavated panel, thus relieving the stress condition near the center and improving stability.

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

Experience has shown that a rigidly placed guidewalt is an extremely important element in maintaining the stability of the top part of the trench.

11.35 Factors Contributing to Trench Stability

Fluid pressure and arching are the primary factors maintaining trench stability. Other factors that contribute to trench stability are the "mudcake" (formed by the electro-osmotic phenomenon) and slurry penetration into the voids of cohesionless soil.

11.35.1 Electra-osmotic Phenomenon

Electro-osmosis contributes to the formation of the impermeable "mudcake" which prevents fluid loss. The electrical potential at the slurry - soil interface causes a migration of colloidal particles to the trench wall even in the absence of fluid flow under hydraulic head.

11. 35.2 Penetration of Slurry into Cohesionless Soil

With slurry 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. In this case, the weight of particles tending to fall away is small compared to the shear resistance of the soil. The seepage forces and the slurry shear strength combine to prevent spalling.

As the zone of penetration increases, a larger volume of soil is influenced. In this 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. A deeper penetration lowers the gradient , lowers the seepage force per unit volume of soil within the zone of penetration. A limiting case would be free penetration in open gravel. Spalling of the trenches is typically the result of this phenomenon.

It has been observed that trench wall collapses (spalling) are more common near the top of the excavation than the bottom. Müller - Kirchenbauer (1972) points out that the slurry contains few suspended soil particles when excavation first begins and only the bentonite resists slurry penetration. As the trench depth increases, the suspended soil particle concentration also increases. The suspended particles aid in forming a more effective mudcake by plugging soil pores. For this reason, in pervious soils it is advisable to maintain a specified percentage of fine sand in the slurry to aid mudcake formation (Hutchinson, et al, 1974). Soil arching is also less effective in preventing spalling at the top of the excavation.

11.40 PRACTICAL ASPECTS OF SLURRY STABILIZED EXCAVATIONS

11.41 Water Level

It is common practice to maintain the trench fluid at least 4 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 conditions, 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.

11.42 Control of Bentonite Slurry

11.42.1 General

The quality of the bentonite slurry must be checked to assure that the design slurry properties are being maintained. Quality control includes a check on the freshly hydrated bentonite slurry prior to insertion in the trench and checks on the re-circulated slurry to assure that the slurry is not being contaminated.

11.42.2 Source of Contamination

a. Detritus Contamination. The clay, silt, and sand particles that build up in the slurry are the contaminants. These particles increase the slurry density with the effect most pronounced at the bottom of the trench. The increased slurry density impairs circulation and adversely affects concrete placement. The concrete can not displace the denser slurry as easily as a thinner slurry.

b. Calcium Contamination. Cement, in contact with the 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. Calcium contamination causes flocculation of bentonite particles, making the slurry more difficult to circulate. An excessively thick mudcake may form which is more difficult to displace during concreting then the thinner sodium bentonite mudcake. The calcuim bentonite mudcake is also more permeable which increases the chances for fluid loss in permeable soils. In some soil conditions the use of a calcium bentonite slurry may minimize the problems of calcium contamination.

c. Salt Contamination. Excessive salinity changes the electrolytic properties and may cause the clay particles to flocculate and settle. This makes it more difficult for the slurry to form an effective mudcake and may lead to a loss of fluid and stability. The problem would be especially acute in pervious granular soils.

11.42.3 Slurry Mix

The slurry must form an effective mudcake on the sides of the trench and be dense enough to provide adequate stability while still being thin enough to allow circulation and concreting. Agents may have to be added to the slurry to counteract chemical contamination, to decrease slurry viscosity, or to aid mudcake development. The measures that can be taken to preserve slurry properties are described below.

a. Viscosity. Flocculation of the bentonite particles will result in increased slurry viscosity. Mud thinners (dispersants) retard flocculation and help maintain the desired viscosity.

Rogers (1963) classifies chemical mudthinners in the following groups: molecularly dehydrated phosphates and polyphosphates, plant tannins, lignosulfonate wood by-products, and mineral lignins. "Dextrid" a trade name polysaccharide made by Baroid, and ferro chrome lignosulfonate are two chemical mud thinners mentioned by Puller (1974).

Use of mud thinners requires experience and laboratory test verification of their effect. As a minimum, such tests would include pH tests, viscosity tests, and standard API fluid loss tests in order to diagnose the problem and to determine approprate 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 to be sure that it is fully hydrated before introduction into the trench. Sodium ferro chrome lignosulphonate (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) propose 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 range of inert plugging substances such as; nut shells, plant fibres, rayon, cellophane flakes, mica, ground rubber tires, etc.

11.42.4 Control Testing

Appendix A to Chapter 4 of Volume III (Construction Methods) describes Standard API procedures, equipment, and specifications for control testing.

Viscosity and Gel Strength

Fann Viscometer.

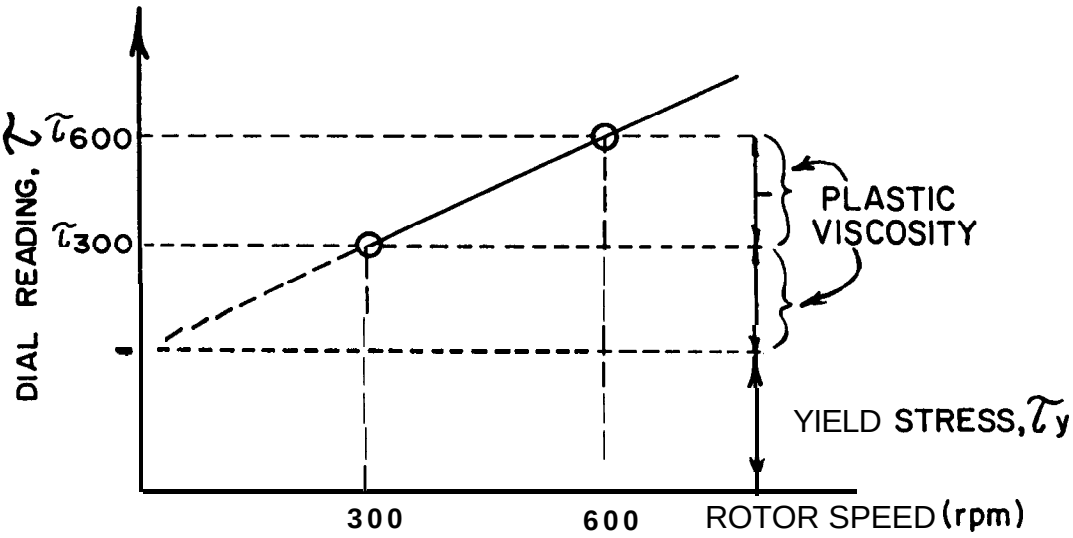
To determine plastic viscosity and yield shear stress, the rotor is turned at 600 rpm and then at 300 rpm. The plastic viscosity in centipoises is the difference between the 600 rpm reading and the 300 rpm reading. The yield shear stress is the 300 rpm reading minus the plastic viscosity. (See Figure 64.)

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 by hand. The tests are for an essentially static condition, conventionally obtained after 10 minutes of gel time.

Shearometer. The shearometer can be used to obtain gel strength. Because of differences between the equipment and procedures, the gel strength values from the shearometer are not the same as these from the Fann viscometer. Specifications must therefore identify the procedure to be used as well as the control values.

Hutchinson, et al (1974) recommend gel strength values of 0.05g/cm^2 to 0. 20g/cm^2 using the viscometer. The FPS specification (1973) requires 0.014 to 0. 10g/cm^2 using the shearometer.

Marsh Cone. The Marsh cone is a simple method for obtaining an index of viscosity, especially useful as a quick field method. The standard size cone is filled with slurry and the time for



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

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

Figure 64. Data from Fann viscometer.

the funnel to drain is reported as Marsh funnel viscosity. Obviously the more viscous the fluid, the longer the drain time. The FPS specification requires that the Marsh cone drain time be between 30 and 60 seconds.

Density. Density is a simple measurement of a known volume of slurry using a Mud Density Balance.

pH

Cement contamination, which adversely affects the slurry by 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 or with a pH meter.

Filtering Performance

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

Excessive sand content may unfavorably raise the density of the slurry so that it is difficult to displace during concreting, and it may result in sand pockets in cast-in-place concrete walls. On the other hand, fine sand may be added to the slurry being circulated in the trench to control fluid loss in permeable soils.

11.42.5 Cleaning the Slurry

Depending upon 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.

11.43 Some Potentially Difficult Soils

Diaphragm walls have been constructed in virtually all soil types. It is usually technically possible to install diaphragm walls, but the costs may be greater than other methods or combinations of methods. In severe cases treatment of the soils may be required prior to wall construction.

11.43.1 Highly Pervious Soils

Loss of ground water through highly pervious strata represents an obvious threat to the stability of the trench. Bentonite concentrations between 4 and 6 percent are satisfactory for soils with permeability less than about 10^{-1} cm/sec to 10^{-2} cm/sec. Beyond merely increasing bentonite concentration, more permeable soils may require a variety of measures such as the addition of fine sand or various plugging agents to control seepage loss. A more detailed discussion of plugging agents is presented in Chapter 4 of Volume III (Diaphragm Walls).

11.43.2 Saline Soils

In general, salinity not a severe problem provided 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 sufficient to cause adverse effects (Fuchsberger, 1974).

11.43. 3 Soft Clays

Soft clays with a shear strength of less than 500 psf must be approached with caution. Panel lengths and construction procedures must be verified by experimental test sections in the early stages of construction. Moreover, such test excavations must be accompanied by careful monitoring of displacements of the adjacent ground.

11.43.4 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.

11.43.5 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.

11.43.6 Residual Soils

Experience in residual soils in Brazil has shown

severe pH contamination due to presence of iron oxides. The slurry became so thick and viscous that it was necessary to totally replace before concreting.

11.43.7 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 clay (Puller, 1974).

11.43.8 Soft Silts

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

11.44 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 very permeable strata.

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

In cases where the potential for fluid loss is great, stockpiling of backfill material should be considered in order to fill panels in an emergency arising from a sudden loss of fluid. Acceptable filling materials would be granular soils, gravelly soils, or crushed stone.

Where the source of leakage is near the surface, the excavation can be carried out in two steps. The first step is to dig an over sized trench and refill with lean concrete; the second step is to make the slurry trench and form the diaphragm wall in the conventional way.

11.50 STRUCTURAL ASPECTS OF CAST-IN-PLACE WALLS

11.51 Load Bearing

Provided that the slurry quality is adequately controlled,

the tremie concrete will satisfactorily displace the bentonite mud-cake 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, especially in cohesive soils. In 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 common in Europe to use load bearing diaphragm wall elements (also referred to as slot caissons). Load is transferred by side friction and end bearing.

11. 52 Concrete

11.52.1 Mix

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

The FPS Specification (1973) for diaphragm walls is appended to Chapter 4 of Volume III (Construction Methods). Briefly, the requirements are as follows:

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

Water Cement Ratio - Less than 0.6

Aggregate - Naturally rounded gravel and sand (if available)

Sand Content - 35 to 40 percent of total weight of aggregate

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

11.52.2 Placement

Concrete placement is performed simultaneously through one or more tremie pipes in each panel. Pipe diameters are normally 6 to 10 inches. General practice is to limit horizontal travel distance of concrete to less than 8 to 10 feet to prevent significant segregation.

11.53 Steel

11. 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. The use of soldier pile reinforcing alone is more common in the U.S. than in Europe.

11.53.2 Bond

Opinion varies with respect to the reduction effect, if any, that bentonite slurry has on bond of concrete to steel.

The FPS Specification (1973) allows normal bond stress on plain bars but allows only 10 percent more bond on deformed bars.

11.53.3 Cover

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

11.54 Panels and Joints

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. Figure 65 is a schematic illustration of the joint configuration formed by means of the stop-end tube.

Another procedure for joint formation is to use a steel wide flange beam or precast I-beam to serve the dual purpose of providing a joint for both shear transfer and vertical steel reinforcement. Figure 66 illustrates this joint.

11.60 EXCAVATION OF SLURRY TRENCHES

11.61 Guide Walls

A well-constructed guide wall is essential to prevent caving of the trench wall in the uppermost part of the excavation. The guide wall serves additional functions: a) to align the trench, b) to contain the slurry, c) to suspend precast elements and d) to suspend

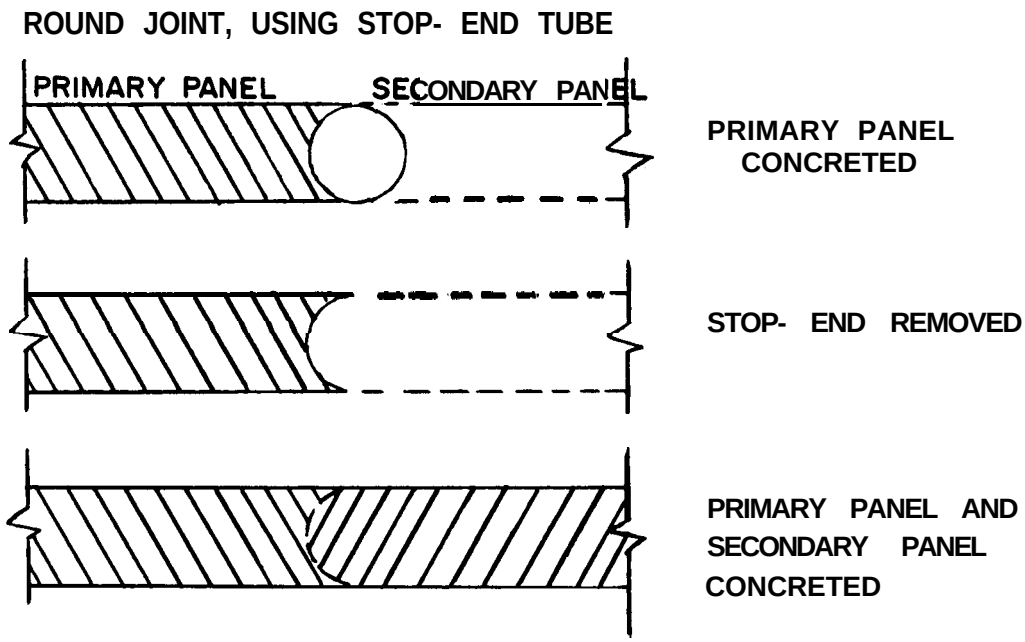
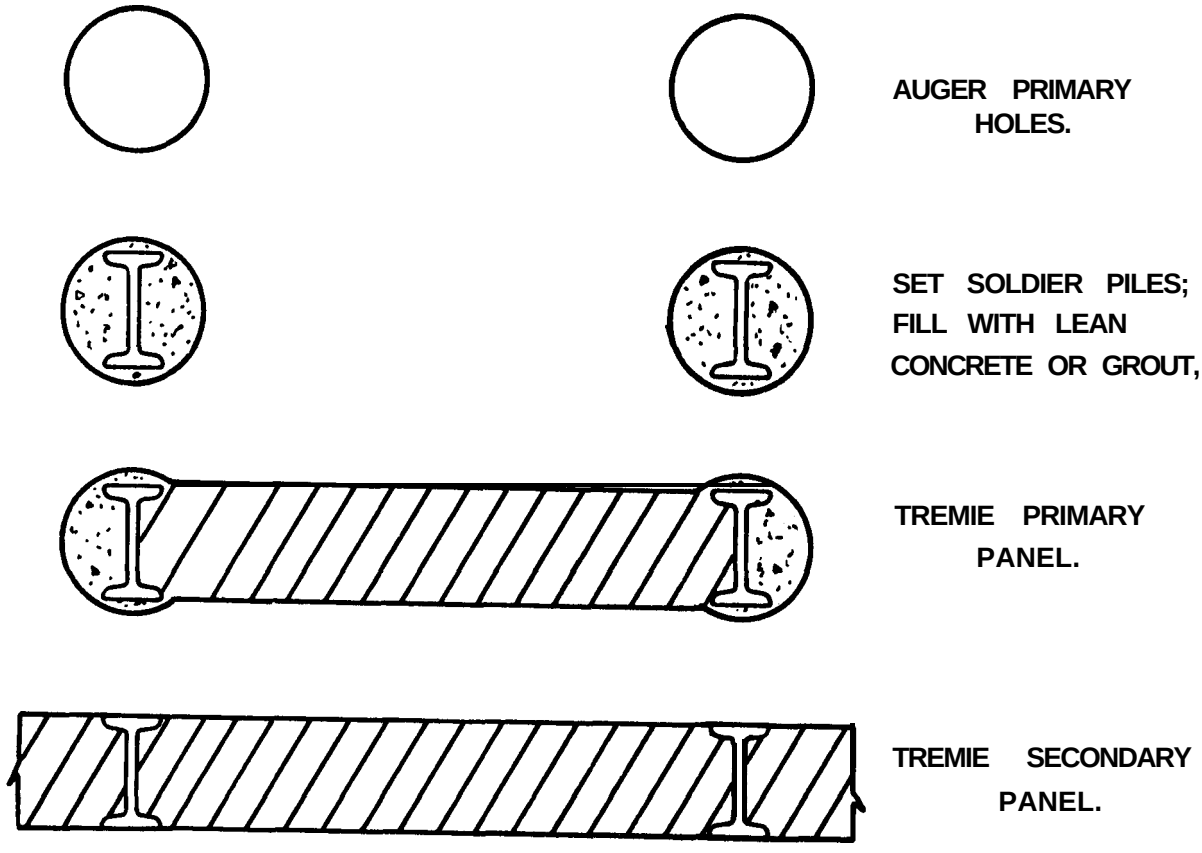


Figure 65. Panel joint with stop-end tube.

TWO-STEP EXCAVATION



ONE -STEP EXCAVATION

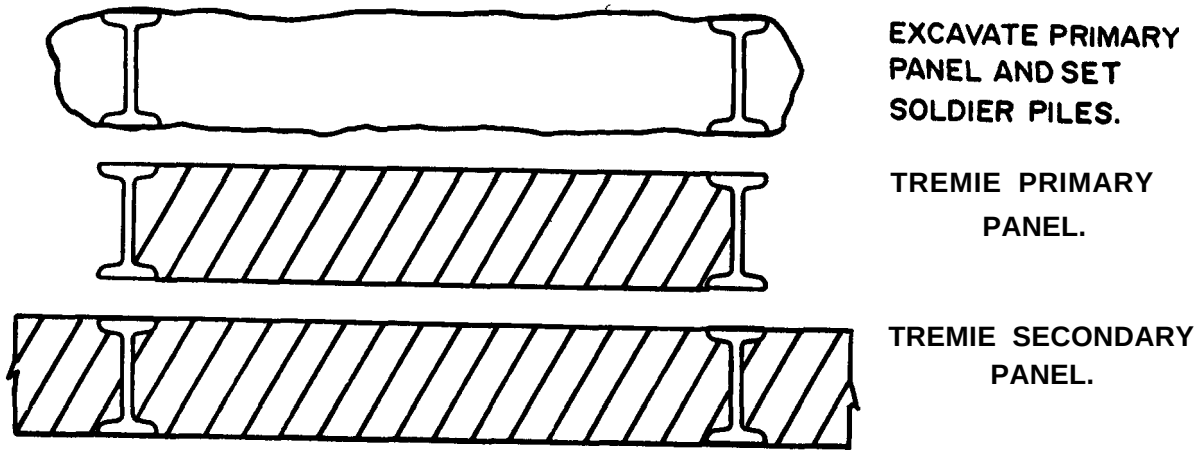


Figure 66. Panel joints with I-beams.

reinforcing steel in cast-in-place walls. Figure 67 shows guide wall, sections.

11.62 Trenching

11.62.1 General

Procedures are:

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

b. Direct or Reverse Circulation. These methods break up the material into smaller particles so that it may be suspended in the bentonite slurry and circulated to the surface to the screening-desanding operation. Care must be taken to avoid clogging of lines by boulders.

With cast-in-place walls, alternate panels are excavated and concreted between stop-ends. Then the remaining 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 the excavation equipment is busy during concreting.

Typically, with precast panels, the trench excavation proceeds continuously. However, the walls can also be constructed in alternate panels.

11.62.2 Excavation Methods

ELSE Trenching Machine

An early 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 conventionally used in Japan as reported by Ikuta (1974) but is rarely used in the United States. A detailed description of the operation of this device is provided by Xanthakos (1974).

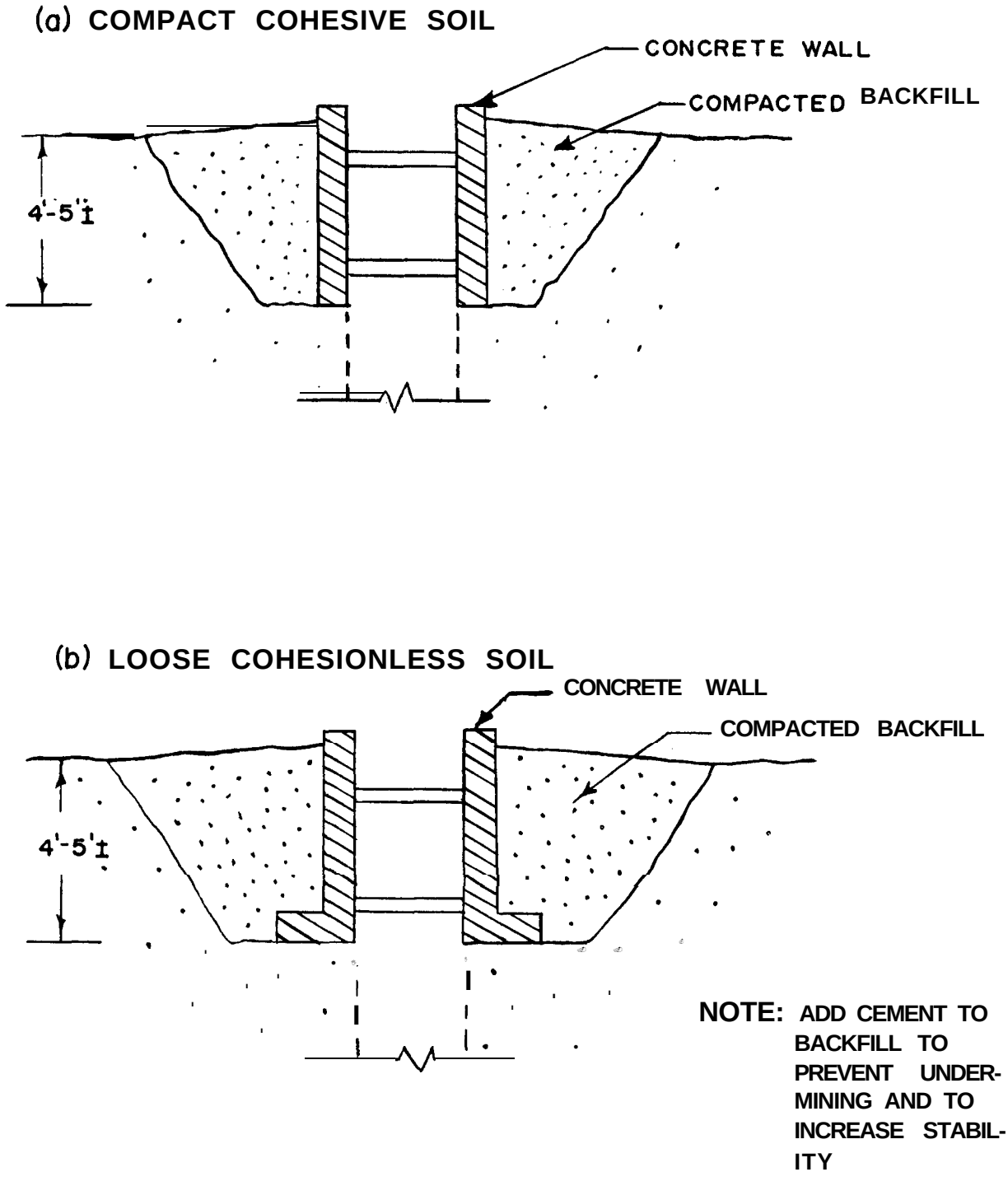


Figure 67. Guide walls.

Clam Shell

The most common types of excavation equipment are specially designed clam shell buckets, conventionally referred to as grabbing tools or grabs.

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 jaws of the grab may operate mechanically or hydraulically. In the mechanical operation, the equipment weight may not be fully effective and therefore 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.

11.62.3 Direct and Reverse Circulation Methods

a. Soletanche, A Soletanche device, which operates on rails that are set along the trench, is a reverse circulation device. The cutting tool benches back and forth between the ends of the panels and cuttings are brought to the surface by surface by suction and/or air lift through the tool itself. This device can employ either percussion or rotary drilling techniques.

b. 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 vertical axis). Slurry cuttings are circulated through the device in suction lines, desanded, and then reintroduced to the trench.

c. TBW Excavator. This device uses cutter heads rotating about the horizontal axis. It is a product of the Japanese firm, Takanaka, and was reported on by Ikuta (1974).

11.62.4 Hard Ground

Obstructions are broken up by heavy chisels or chopping devices (material removed by grab buckets), by percussion tools, 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.

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.

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

11.71 General

This discussion covers the following:

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

b. Hybrid technique 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.

11.72 Precast Concrete Methods

11.72. 1 General

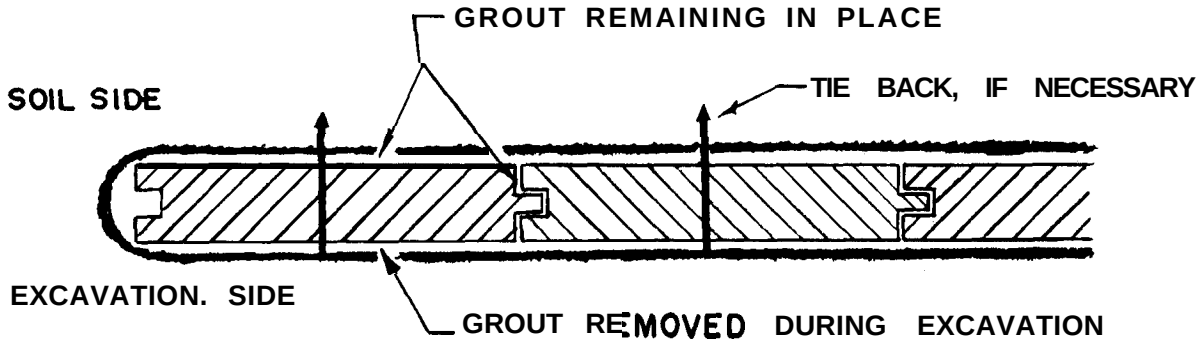
Precast concrete elements are normally set within a continuously excavated slurry stabilized trench. Figures 68 and 69 are schematic illustrations of the Soletanche and Bachy methods. Franki uses a similar method.

Precast elements are carefully aligned and suspended from the guide wall into the grout slurry (or cast-in-place concrete). The elements can not be set until the grout slurry 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.

Grout fills the space between the back side of the precast element and the soil, thus forming tight contact and an impervious membrane. Grout adhering to the inside face is removed during excavation.

The size of the precast concrete elements is controlled by the load capacity of the crane. In urban areas the crane size may be controlled by city ordinances thereby limiting

(a) TONGUE &GROOVE



(b) T BEAMS & SLABS

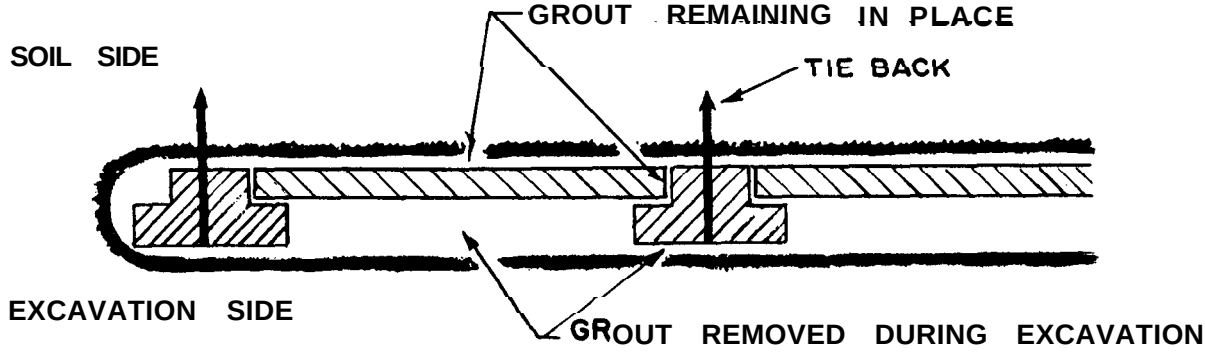
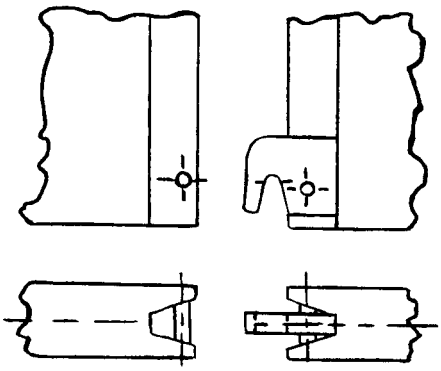
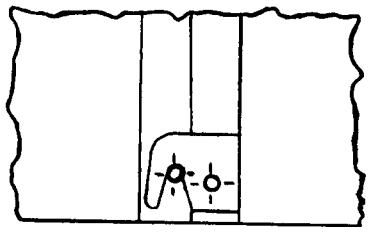


Figure 68. Panosol walls (Soletanchc, France).



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.

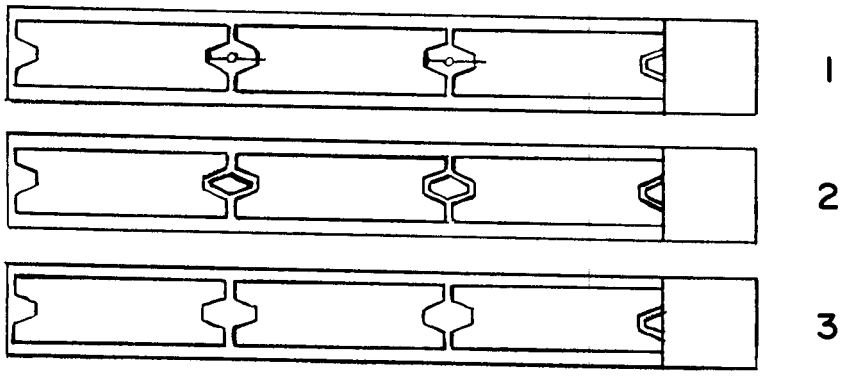


Figure 69. Prefasif wall (from Bachy Enterprise, France).

panel size. Depending upon wall thickness the depth limitation is normally in the range of 30 to 50 feet.

The T-beam/slab combination (Figure 68b) 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 develop additional passive resistance. Slab panels need only extend to the depths required for the permanent wall except where the wall must also act as a ground water cutoff.

11.72.2 Grout and Slurry

The Soletanche method uses a special grout 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,

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 was described by E. Colas Des Francs (1974) in the Bachy method.

11.73 Soldier Pile Combination Walls

11.73.1 General

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

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.

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

11. 73.2 Two Step Excavation: First for Piles: Second for Panel

Two techniques are shown in Figures 70 and 71. Figure 70 shows a wall formed using precast soldier piles while Figure 71 shows a wall with steel, wide flange, soldier piles.

11.73.3 One Step Excavation

In this method (Figure 72) the soldier piles and reinforcing are placed concurrently in the excavated panel.

11. 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 collapse, is always maintained. Also when soldier piles are installed separately, they can be extended to whatever depth is required to develop bearing and/or toe restraint.

11. 74 Bored Pile Walls

11.74. 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 this report, the methods have been classified as "small-diameter piles", conventionally formed by grouting using hollow stem augering equipment, and "large-diameter piles" formed by excavation with a solid auger and then filling with concrete after withdrawal of the auger. In both cases the piles are reinforced. Figure 73 illustrates these bored pile walls.

11.74.2 Small-Diameter Piles

Piles are formed using hollow stem augering equipment with outside diameters ranging typically from 12 to 16 inches. The procedure is to install alternate piles (primary piles) then after the grout has 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 73a).

1. SET SOLDIER PILE IN PRE-EXCAVATED HOLE



2. EXCAVATE AND CONCRETE PANEL.

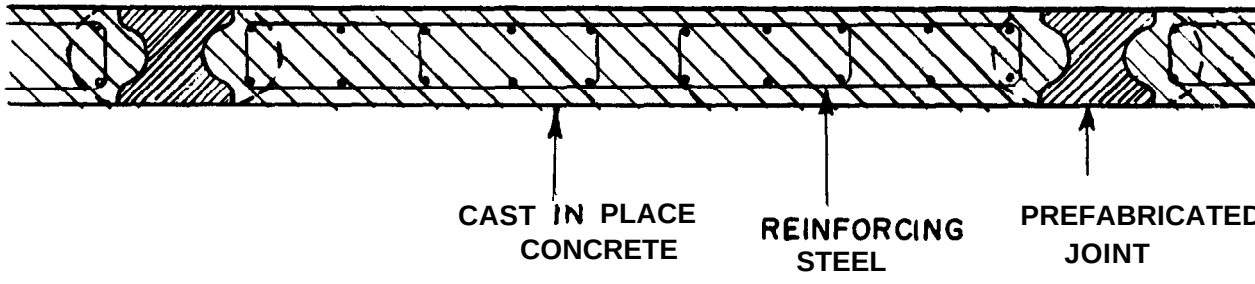
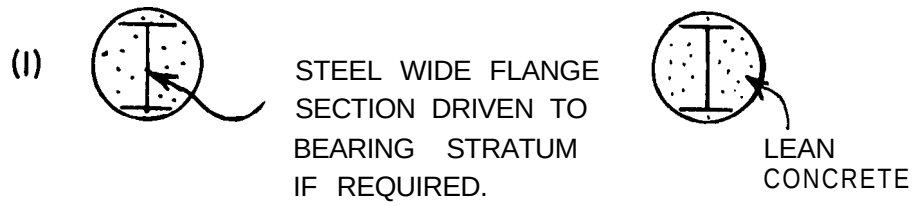
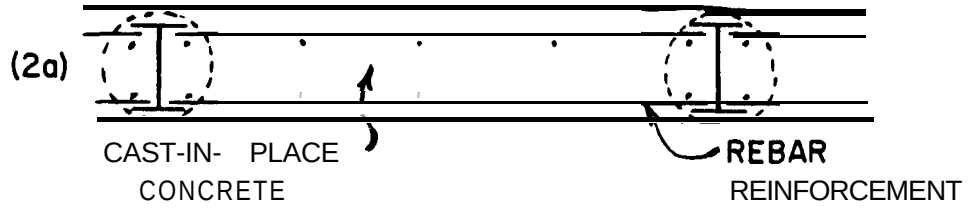


Figure 70. Two step excavation in slurry trench using precast soldier piles and tremie concrete.

SET SOLDIER PILE IN PRE- EXCAVATED HOLE.



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



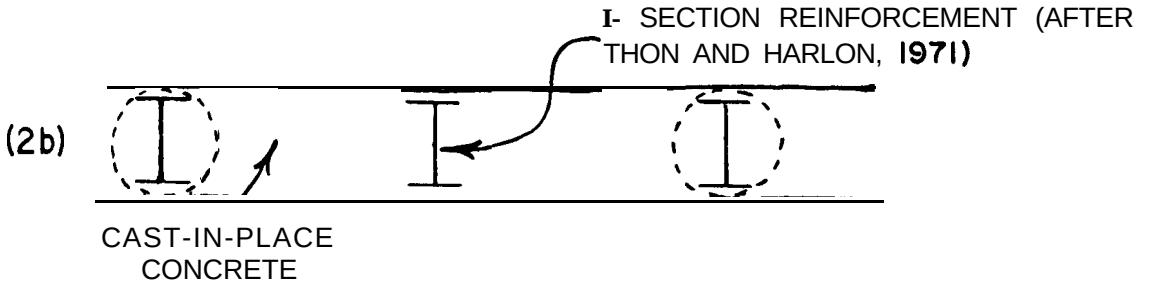


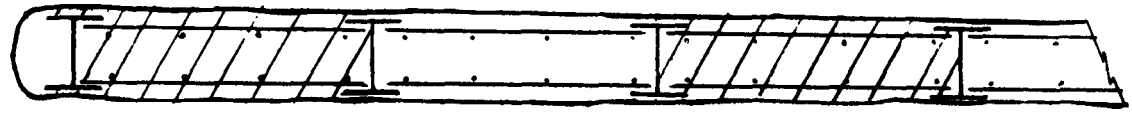
Figure 71. Two step excavation in slurry trench using steel wide flange soldier piles and tremie concrete.



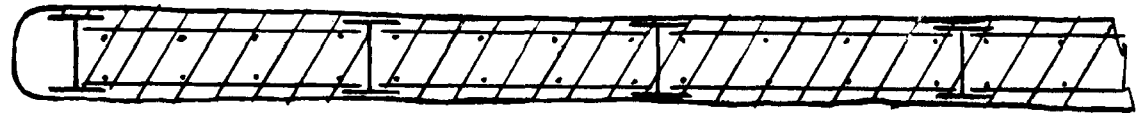
SET SOLDIER PILES AND REBARS
IN ALTERNATE PANELS.



CONCRETE ALTERNATE PANELS.



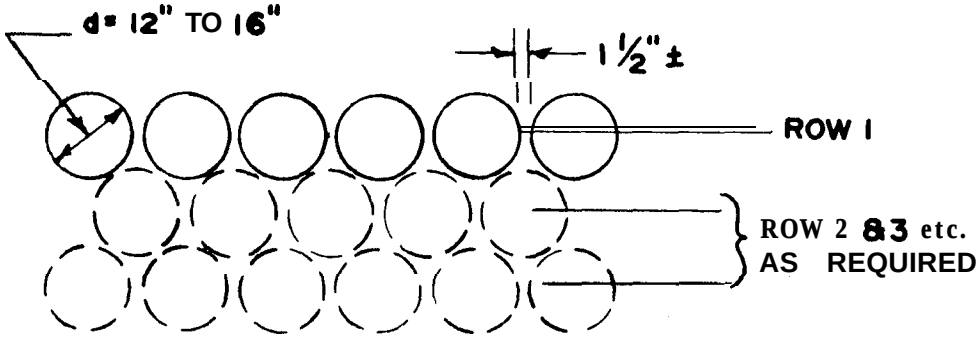
EXCAVATE INTERVENING PANELS
AND SET REBAR CAGE.



CONCRETE

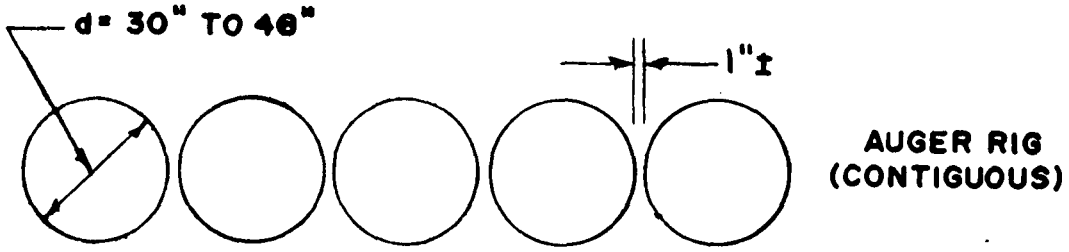
Figure 72. One step excavation with soldier piles (after Tamaro, 1974).

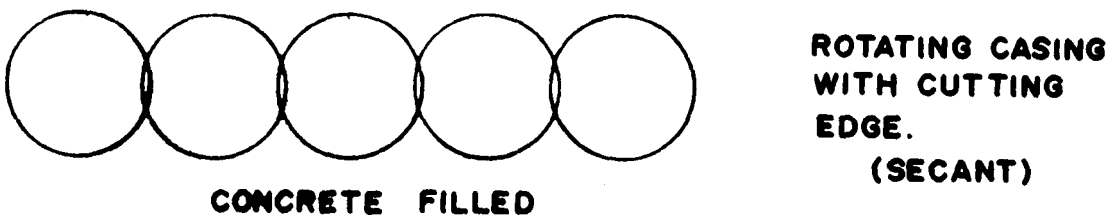
(a) SMALL DIAMETER (COMMONLY "TANGENT" PILES)



GROUT FILLED WITH HOLLOW STEM AUGER

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





CONCRETE FILLED

Figure 73. 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 beam section is inserted into the wet mortar.

11.74.3 Large-Diameter Piles

Shaft diameters range from about 2-1/2 feet 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 is the case for small diameter pile s, alternate piles are installed first, followed by installation of inte rmediate piles.

Reinforcing is positioned following excavation, then the hole is filled with concrete. Contiguous piles are used where there is not great concern over watertightness. Overlapping piles (secant piles) can be used to provide additional assurance of watertightness (see Figure 73b).

11. 74.4 Discussion

A bored pile wall has several advantages over walls cast in slurry trenches.' Because of the minimum exposure of excavated soil prior to concreting, additional protection is provided for heavily loaded foundations and/or in weak or unstable soils. Also, selected piles may be carried to a lower elevation for bearing or toe restraint.

CHAPTER 12 - INTERNAL BRACING

12.10 INTRODUCTION

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 anchorage strata are not available for tiebacks. In cut-and-cover tunnel work, braces typically do not require intermediate vertical support. A continuous horizontal wale is typically used 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 about 15 to 20 feet apart longitudinally along the cut. Recent excavation work in Washington used discontinuous wales to aid installation.

12.20 DESIGN CONSIDERATIONS

12.21 Types of Bracing

The most common braces are pipe or wide flange sections. Projects have been reported in Europe in which a concrete slab, poured on the ground, later serves as the roof of the structure or a floor level within the structure. The excavation is carried out by mining beneath the slab. The technique is often called "under the roof" construction.

12.22 Allowable Stresses

Controlling design criterion is the column-action of combined axial and bending stress. In that regard, a pipe section is more efficient than a wide flange section.

AISC Code design stresses are recommended for the completed cofferdam at its maximum depth. Temporary conditions arising from intermediate situations during the course of excavation will justify a 20 percent overstress above the AISC Code values.

12.23 Connections

Connections and details are of critical importance in an internally braced excavation. Improper connections between strut and wale or between the wale and the support wall are perhaps the most common causes of difficulties in braced excavations. They can lead to twisting, buckling, and rotation of members. Figures 74, 75, and 76 present typical connection details.

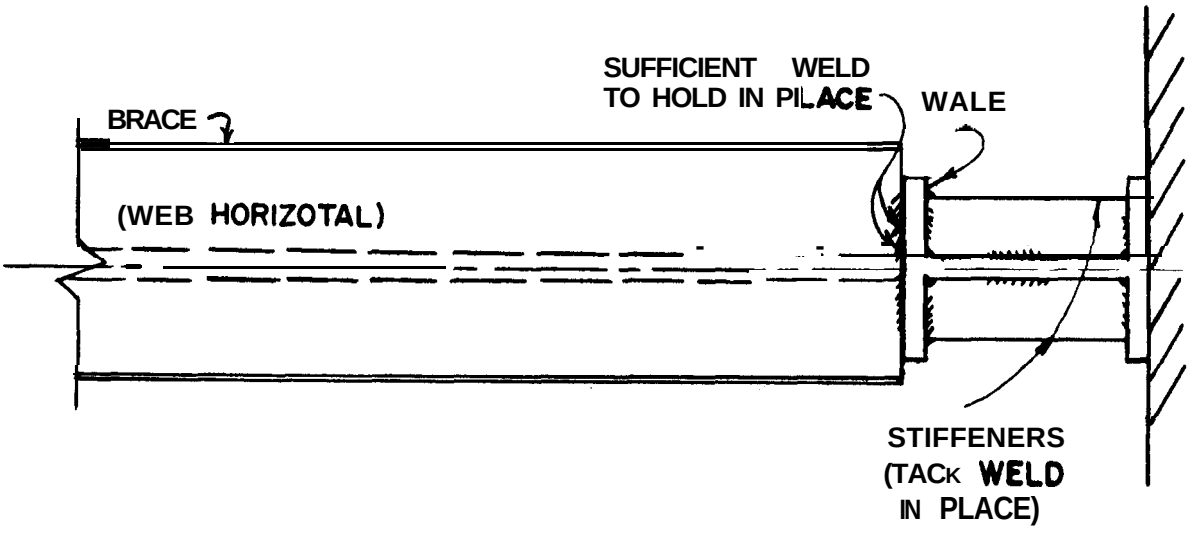


Figure 74. Typical detail for horizontal brace with brace web horizontal.

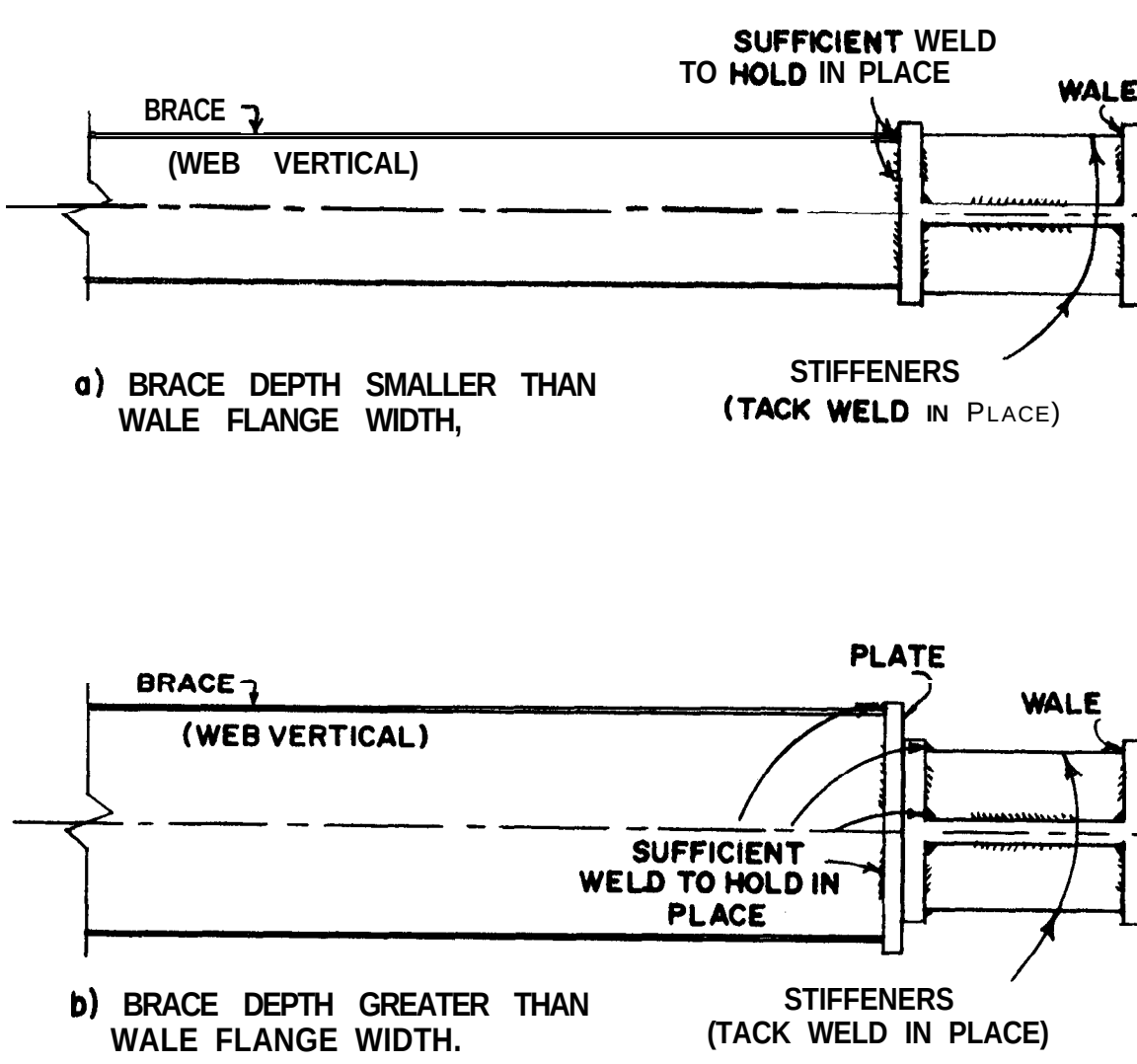


Figure 75, Typical detail for horizontal brace with brace web vertical,

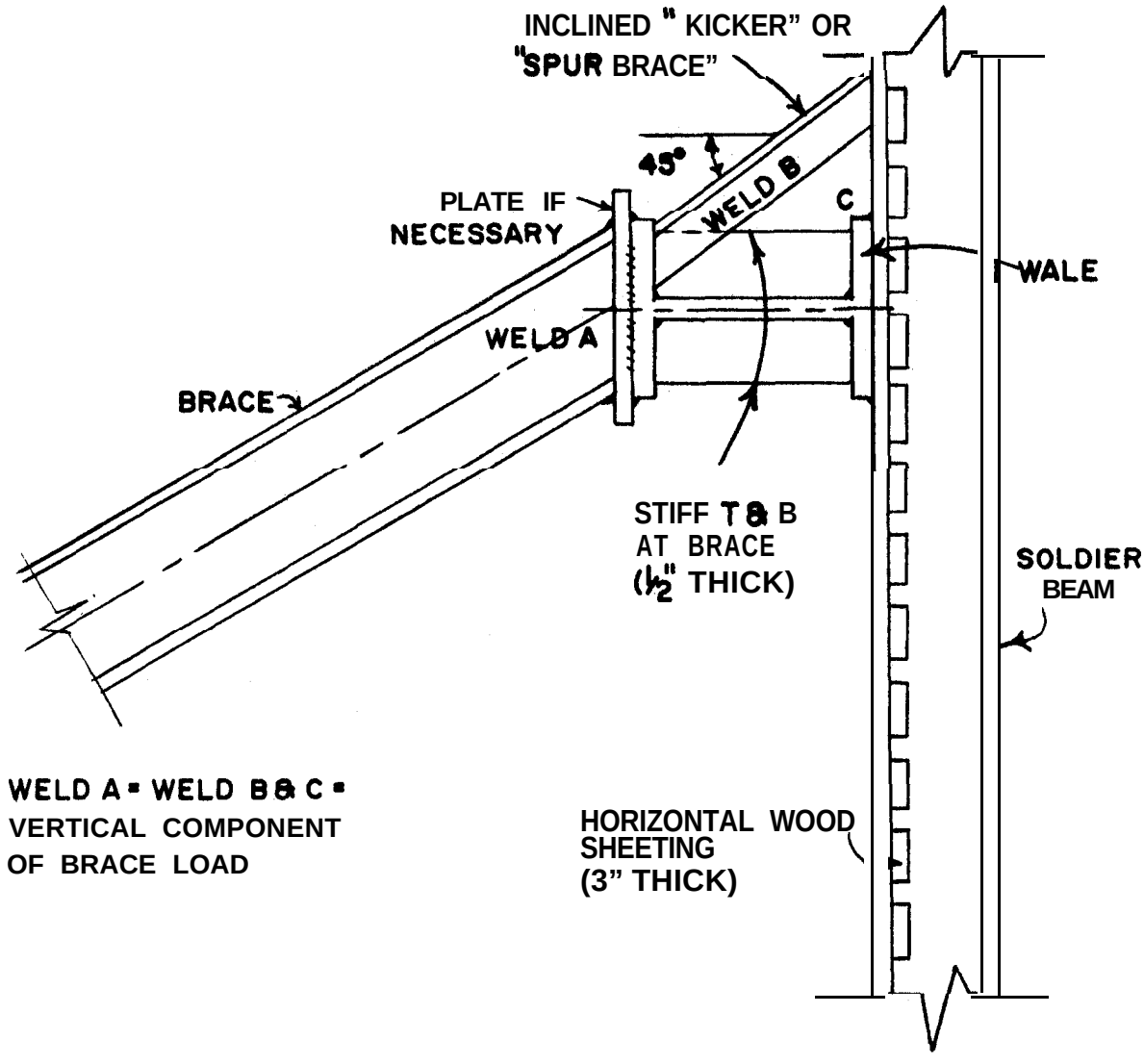


Figure 76. Typical connection for inclined brace and horizontal wale.

12.30 INSTALLATION

12.31 General

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

12.32 Installation Without Preloading

Cross-lot members are welded at one end and blocked and shimmed at the opposite end. After the members are fitted in place, steel wedges and plates are tack welded to hold everything in place. For inclined braces (rakers) the member is welded at one end (usually at the wale), and the reaction end cast into the concrete slab. An alternate 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 preloaded to remove slack and to assure that the load can be accepted without excessive movement.

12. 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 to jack to the desired load, and then to drive wedges between the member and the wale until the jack load is essentially zero. A second procedure is to weld the connection tight while maintaining the jack load and then to drop the pressure in the hydraulic jack, thus transferring the load through the connection to the wale. The second procedure may result in some wall movement as the load is transferred although the magnitude of movement is generally small.

12.34 Preloading

Figures 77 and 78 show prestressing details for bracing. Preloading is accomplished by loading hydraulic jacks to the desired loads followed by securing the member with steel blocking, steel wedges, and welding. In the case of pipe struts, the connection can be

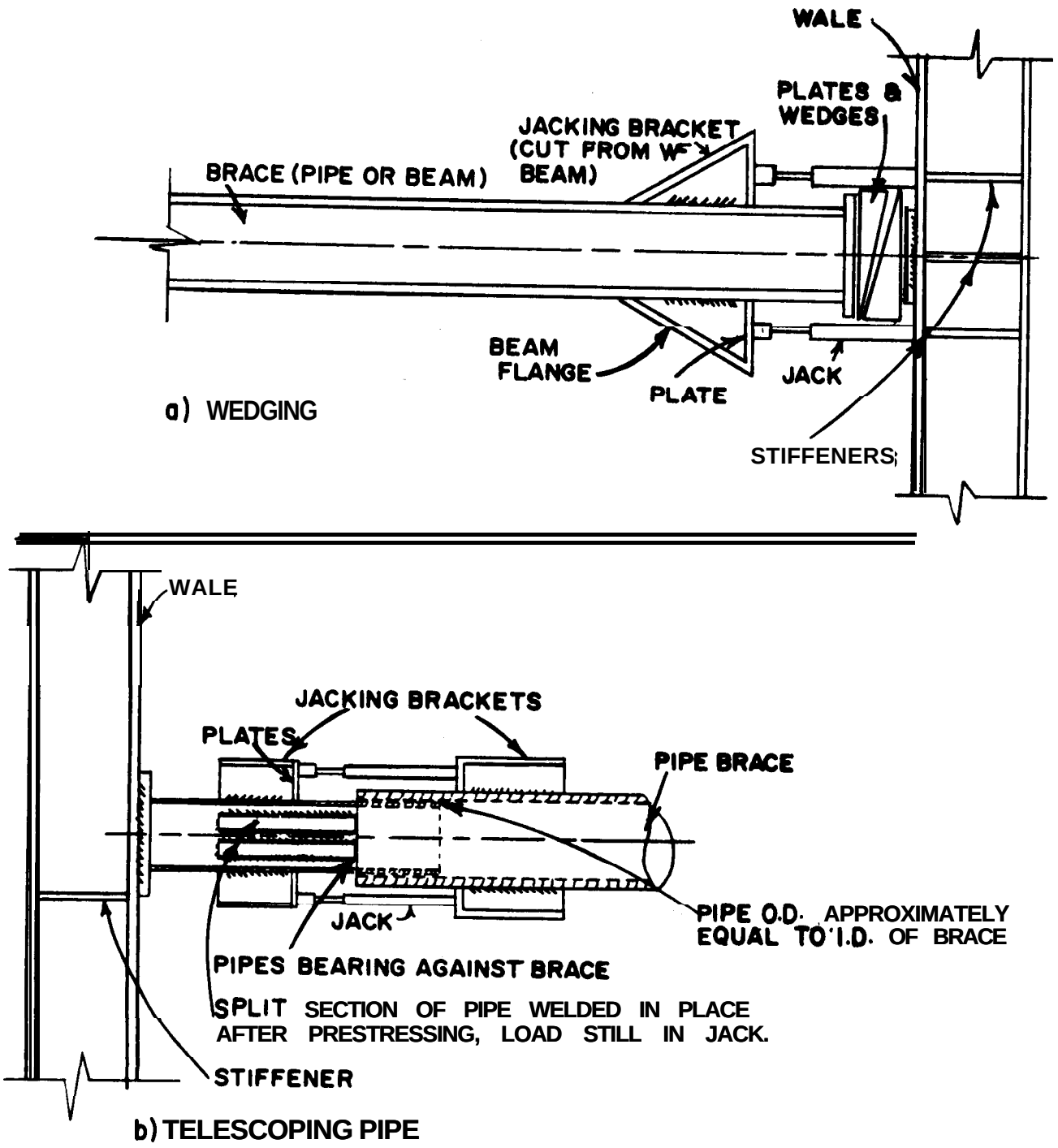


Figure 77. Preetreeeing details for braces.

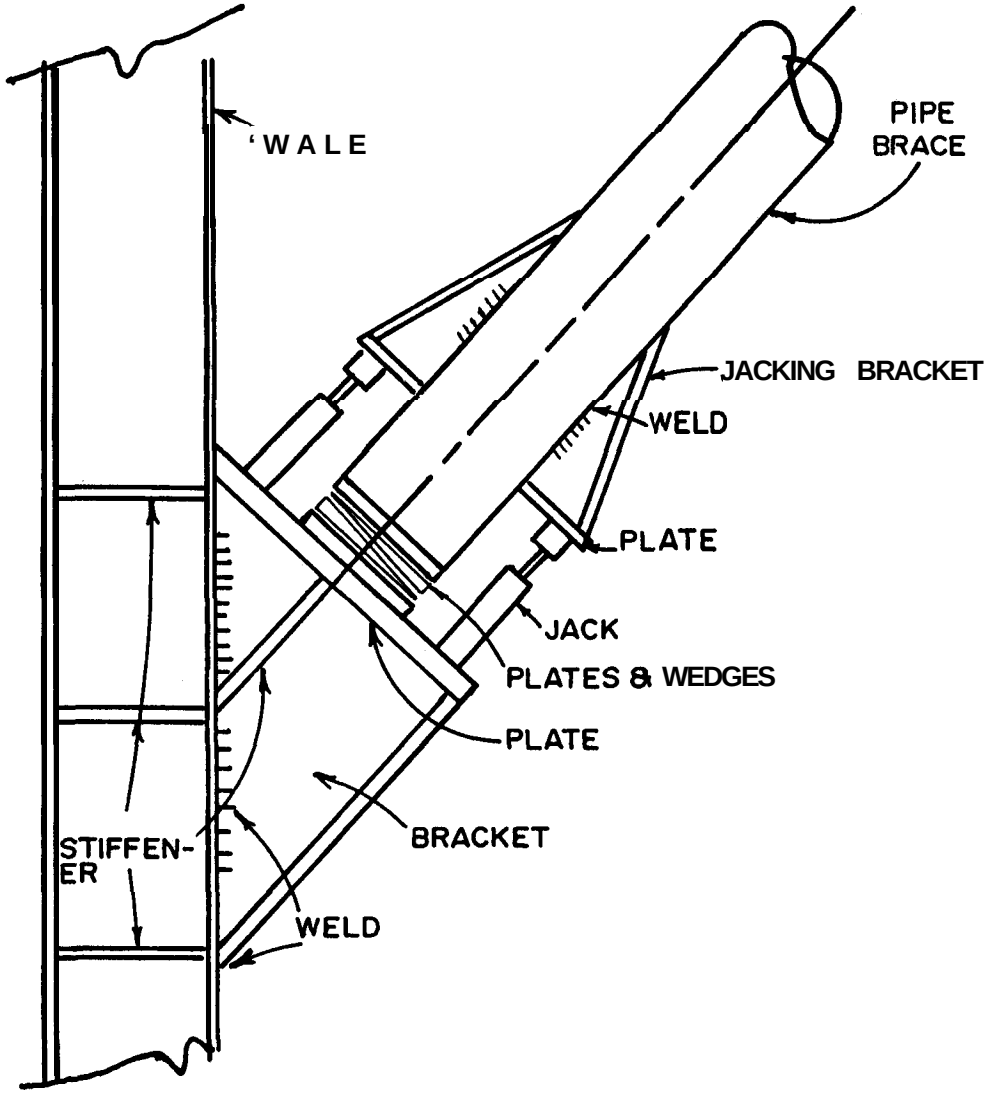


Figure 78. Prestressing of pipe brace at corners using brackets as reaction,

made using a telescoping strut or a split pipe which fits over the pipe brace.

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 preload removes the slack from the support system and at the same time reduces the risk of overstressing.

12.40 TEMPERATURE EFFECTS

12.41 General Background

Since temperature variations in strutted excavations may easily be as great as 50°F, and even more if unprotected, the changes in load accompanying temperature variation can be large. A limiting case would be to assume a perfectly restrained strut (i. e. no movement). The increase in load would therefore be equal to:

$$\Delta P = A_s E_s (\alpha \times \Delta^\circ F)$$

where:

A_s = area of strut

E_s = modulus of strut (30,000 ksi)

α = thermal coefficient of expansion (6. 5 x 10^{-6} in/in/°F for steel)

$\Delta^\circ F$ = change in temperature in degrees Fahrenheit

Since the soil behind the wall yields under the increased load, the actual stress increase will be less than that indicated by the limiting case condition.

12.42 Some Case Studies

The following table summarizes the load changes caused by temperature variations on four projects.

Case	Decked or Open	Load Variation
1. Chapman, et al (1972)	Open	1.5 kip/^{o}F
2. O'Rourke & Cording (1974)	Covered	0.5 kip/^{o}F
3. Jaworski (1973)	Open	20% ± of measured average
4. Armento (1972)	Covered	10% ± of measured average

12.43 Desipn and Construction Criteria

Since the Peck (1969) diagrams have been developed from measured maximum strut loads, the Peck diagrams implicitly consider temperature variations. In critical cases where large temperature variations are expected in an unprotected excavation, strut loads may be monitored to assure that overloading does not occur. Although rarely done, struts may be painted with special reflective paint or sprayed with water to prevent heat buildup in struts. Ideally, strut installation (and preloading) should be at about the mean temperature anticipated during the course of the job.

12.50 STRUT REMOVAL AND REBRACING

Strut removal (and rebracing) is an additional source of displacement. Factors controlling the amount of displacement are the wall stiffness, the properties of the retained soil, the span distance between remaining braces, and the quality and the compaction of the backfill between the structure and the ground support wall.

CHAPTER 13 - TIEBACKS

13.10 INTRODUCTION

During the last 20 years the use of soil and rock anchors to support side walls of excavations has increased significantly. Tie-backs (or anchors) have been used to support both temporary and permanent excavations.

A tieback consists of 3 major components (see Figure 79):

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 tied-back 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 tied-back installations. The purpose of this chapter is to present the techniques used in tied-back wall design and construction.

13.20 DESIGN AND. THEORETICAL CONSIDERATIONS

13.2 1 General

The design of tied-back systems involves several major design considerations. The earth and water pressures acting on the wall must be evaluated. There must be a suitable anchorage stratum. The soil-wall system must be stable with respect to overall sliding stability. The expected vertical and horizontal deformations must be evaluated.

13.22 Deformations

In response to excavation, the unexcavated soil mass will displace toward the excavation. Temporary support walls limit but do not prevent movements. The consequences of movement and the factors affecting soil and wall movements are discussed in greater

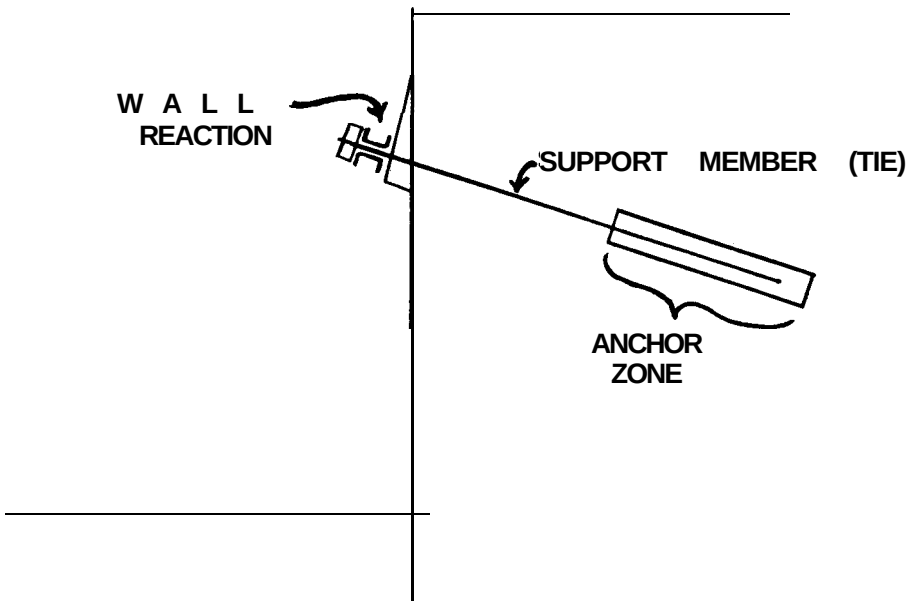


Figure 79. Major tieback components.

detail in Volumes II (Design Fundamentals) and III (Construction Methods) of this research.

The techniques used to predict movements are crude. Movements can be vastly influenced by specific conditions and construction techniques which makes prediction of movements difficult. However, certain factors in tied-back walls can be identified as affecting movements.

Some of the factors affecting performance of the wall and supported soil mass are listed below:

a. Vertical wall movement

b. Wall stiffness

c. Tieback prestress

d. Internal deformation of the soil block

e. Movement of the soil block

f. Ground loss associated with construction method

g. Volumetric strain

13.23 Overall Stability of Soil Mass

13.23.1 Circular Arc Analysis

A circular arc analysis of the stability of the soils behind the anchors and below the wall should be performed. The analysis should be performed using accepted circular arc methods of analysis as described in basic soil mechanics texts.

13.23.2 Overturning Analysis

This method of analysis in combination with a circular arc analysis is used to evaluate the stability of tied-back walls throughout Europe. The basic method was proposed by Ranke and Ostermayer (1968) who expanded on the work done by Kranz (1953). Figure 80 schematically illustrates a failure by overturning.

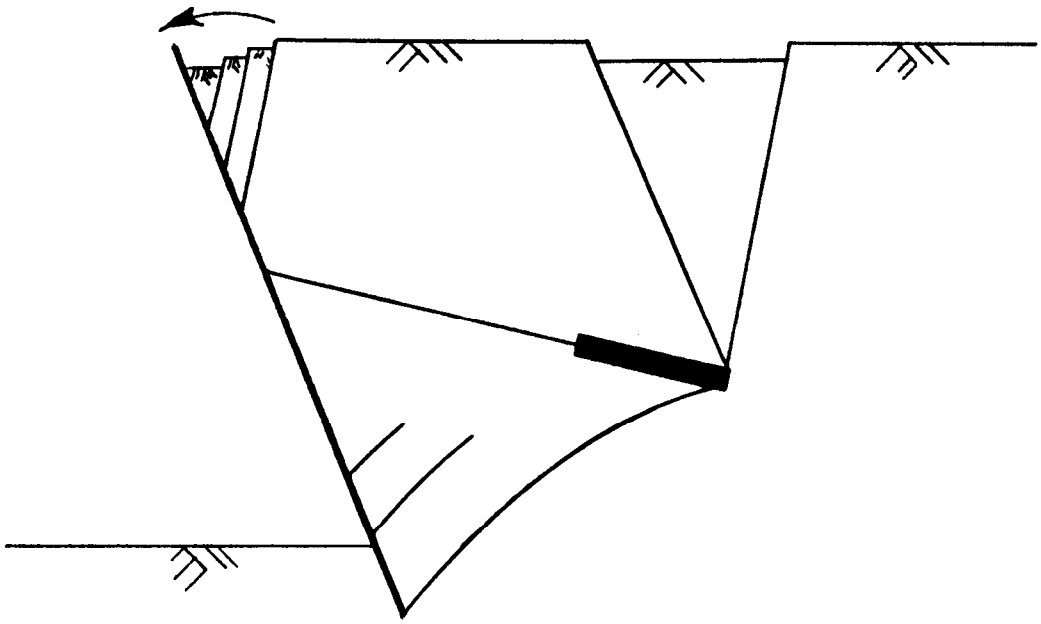


Figure 80. Sketch of tied-back wall failing by ove rtqrning.

Free Body Diagram and Forces

Figure 81 illustrates the free body diagram and the forces acting on the free body. The free body is defined by the points ACDE. Since the wall is not part of the free body, the forces P_A and T_{net} act in the directions shown.

The location of the free body diagram is predetermined in this method of analysis. Points A and E are located immediately above 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 $P_{A_h} - T_{des_h} = P_{p_h}$. Point D is uniquely defined as the midpoint of the grouted anchor length. Therefore; in Figure 81 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 described below:

a. P_a represents the active pressure driving force on the face DE from the soil pressure. The force can be assumed to be either inclined or horizontal.

b. W represents the weight of the soil mass within the free body.

c. P_A represents 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 represents the frictional component of soil resistance. Full soil strength is assumed to be mobilized.

e. S_c represents the full component of soil resistance from cohesive soil strength.

f. T represents the tieback force. The free body cuts the tieback at points B and D. The force, T_{net}, (Figure 81) represents the vector sum of the tieback forces 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 stability of the free body is evaluated in terms of

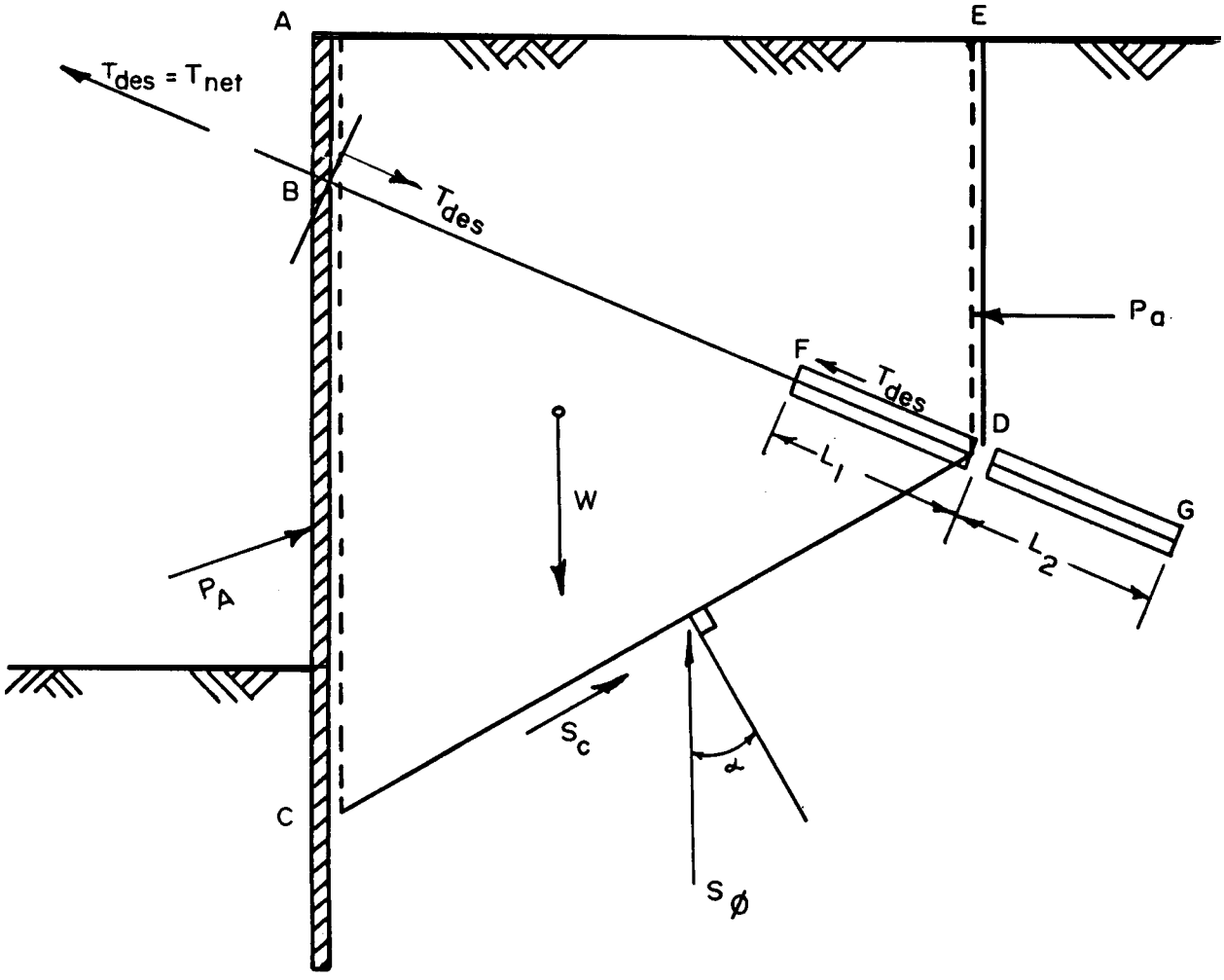


Figure 81. Free body diagram for a failure surface in single anchor tieback system (internal free body).

the ratio of the maximum possible tieback force to the design tieback force; F. S. = $\frac{T_{max}}{T_{des}}$. The maximum tieback force is determined from the appropriate vector diagram.

a. Single Anchor. Figure 82 illustrates a single anchor tied-back wall and the force diagram used to evaluate the stability of the system. The vector diagram in Figure 82 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}. A minimum factor of safety of 1.5 is recommended in design.

The method described to this point has been applicable to soil conditions where no water is present. If water is present, the free body must consider the forces due to pore pressure, and the analysis should be consistent with other basic methods of stability analysis as described in many soil mechanics texts.

b. Multiple Anchor Systems. Figures 83, 84, 85, 86, 87, and 88 illustrate the free body and vector diagrams for multiple anchor systems. The basic techniques are the same as for a single anchor with the factors of safety defined as shown on the figures.

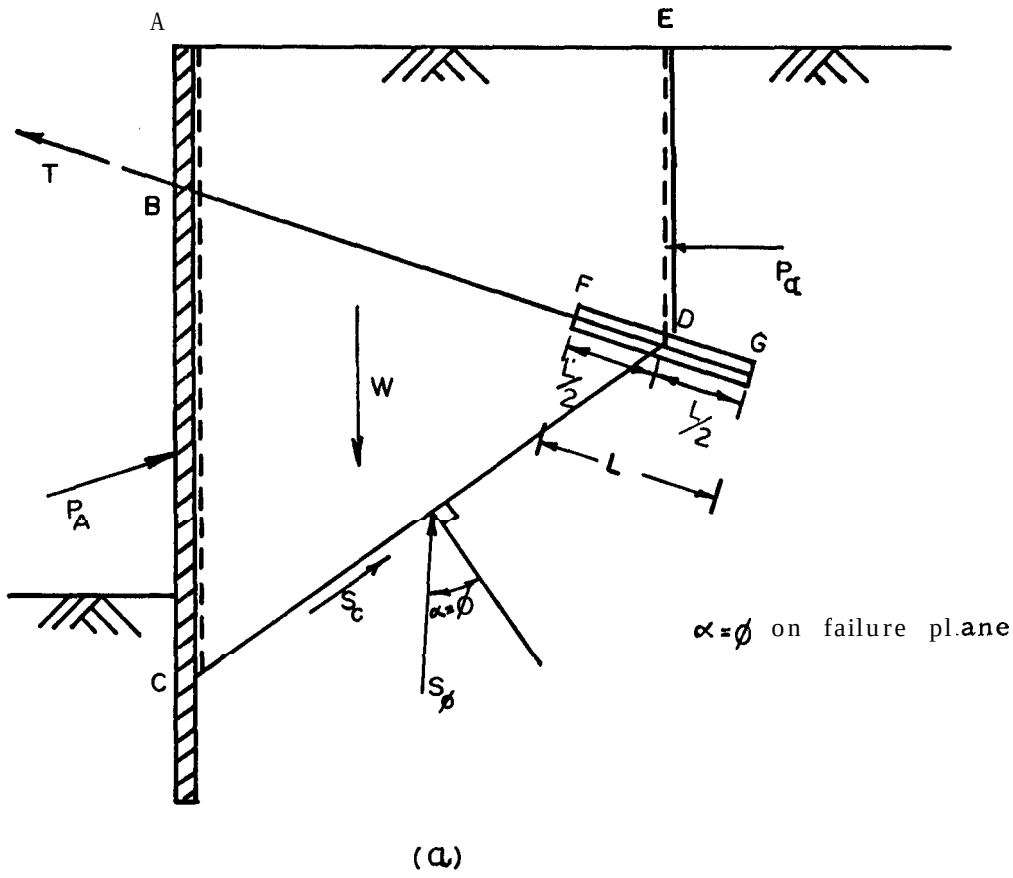
This method of stability analysis has several apparent disadvantages. Among these is the rigid definition of the failure plane. However, because of the method's wide usage in Europe with satisfactory results, it is believed that the method can be used to evaluate wall stability against overturning. The method should be used in combination with other methods evaluating sliding stability.

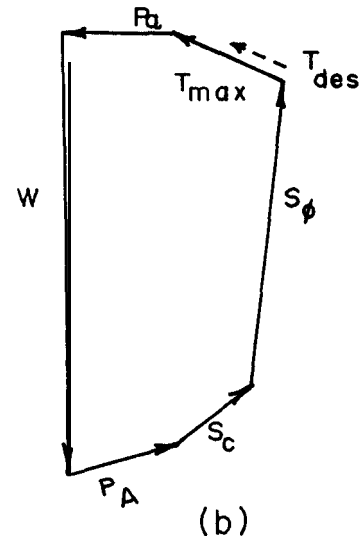
13.23. 3 Sliding Wedge Analysis

Free Body Diagram and Forces

A generalized free body diagram is presented in Figure 89. The wall is included as part of the free body, and therefore, the wall forces, H_s and V, are included. The passive soil resistance is included as is the net tieback force, T_{net}.

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.





$$F.S. = \frac{T_{max}}{T_{des}} \geq 1.5$$

After Kranz (1953)and
Ranke & Ostermayer (1968)

Figure 82. Single anchor free body diagram with appropriate vector diagram (safety in terms of the tieback force).

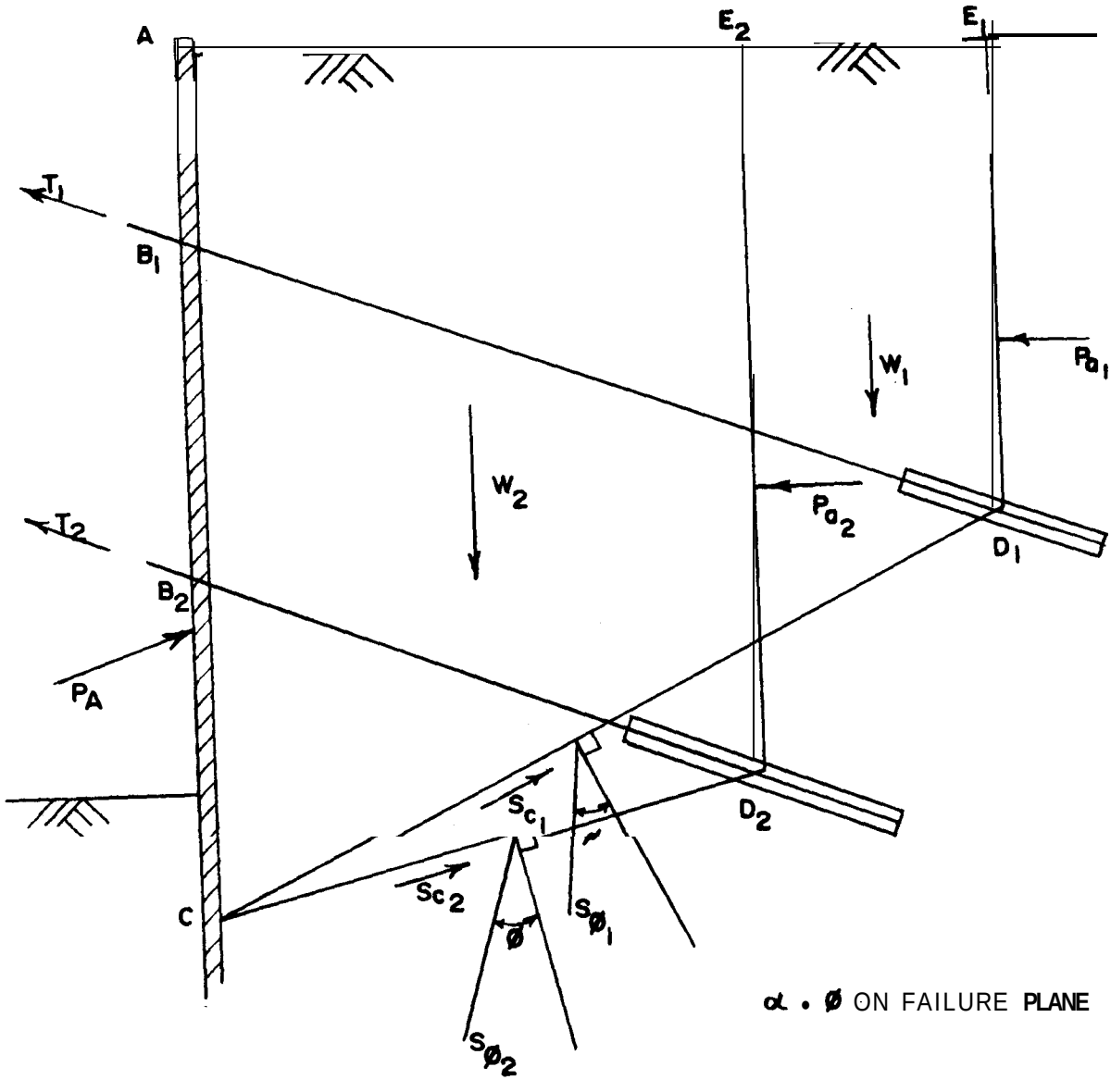
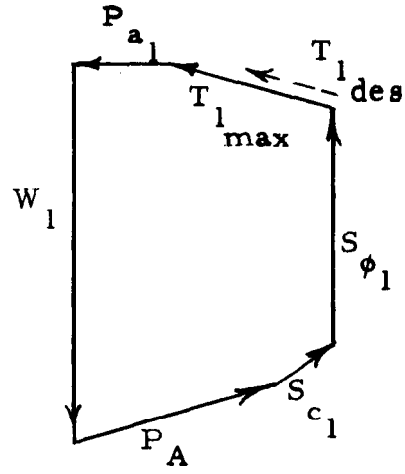


Figure 83. Free bodies and forces for two completely independent anchors (after Ranke and Ostermayer, 1968).



a. Upper Tieback

$$F.S. = \frac{T_{1_{max}}}{T_{1_{des}}} \geq 1.5$$

Note: only the directions of S_{ϕ} and T_{max} are known.

$\alpha = \phi$ on failure plane.

B. Lower Tieback

$$F.S. = \frac{T_{2_{max}}}{T_{2_{des}}} \geq 1.5$$

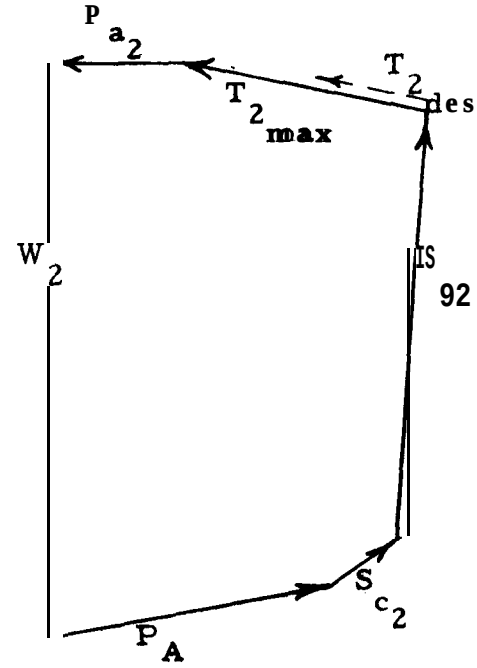


Figure 84. Vector diagram for case of two completely independent anchors (safety in terms of anchor force) (after Ranke and Ostermayer, 1968).

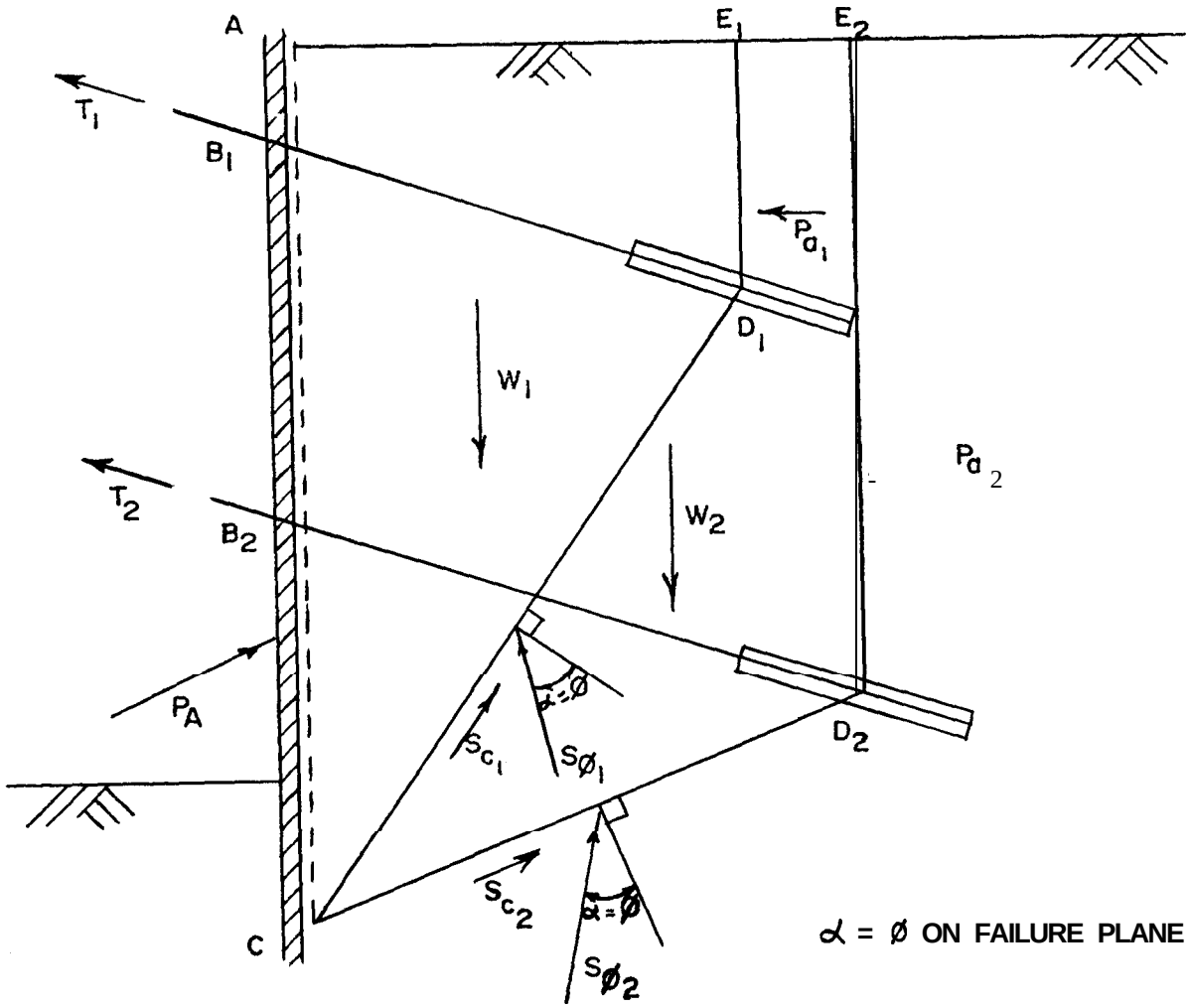
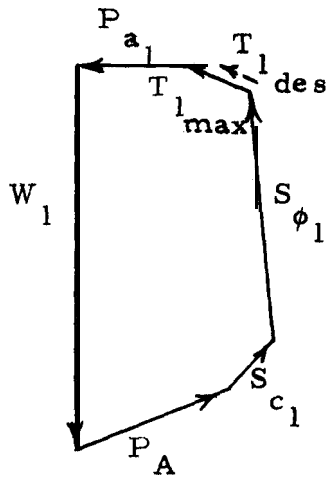
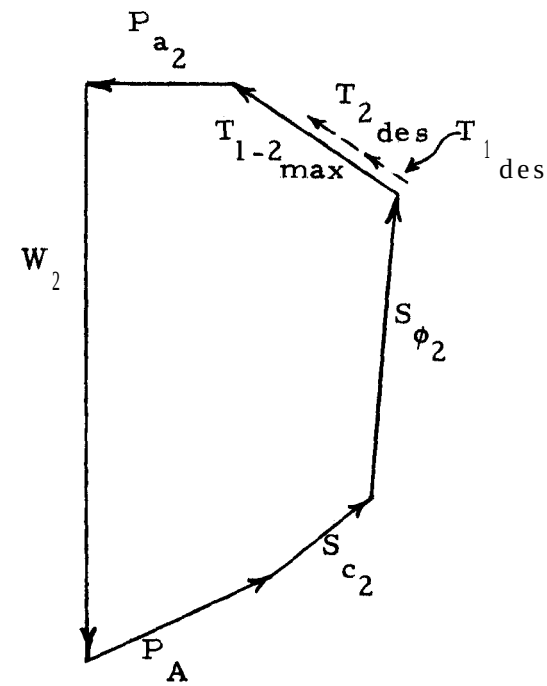


Figure 85. Free body diagram with forces acting on the bodies for the case of one independent anchor (safety in terms of the tieback force) (after Ranke and Ostermayer, 1968).



$$F.S. = \frac{T_{1_{max}}}{T_{1_{des}}} \geq 1.5$$

$\alpha = \phi$ on failure plane.



$$F.S. = \frac{T_{1\text{-}2_{max}}}{T_{1_{des}} + T_{1_{des}}} \quad 1.5$$

Figure 86. Vector diagrams used to evaluate the stability of case with one independent anchor (safety in terms of tieback force) (after Ranke and Ostermayer, 1968).

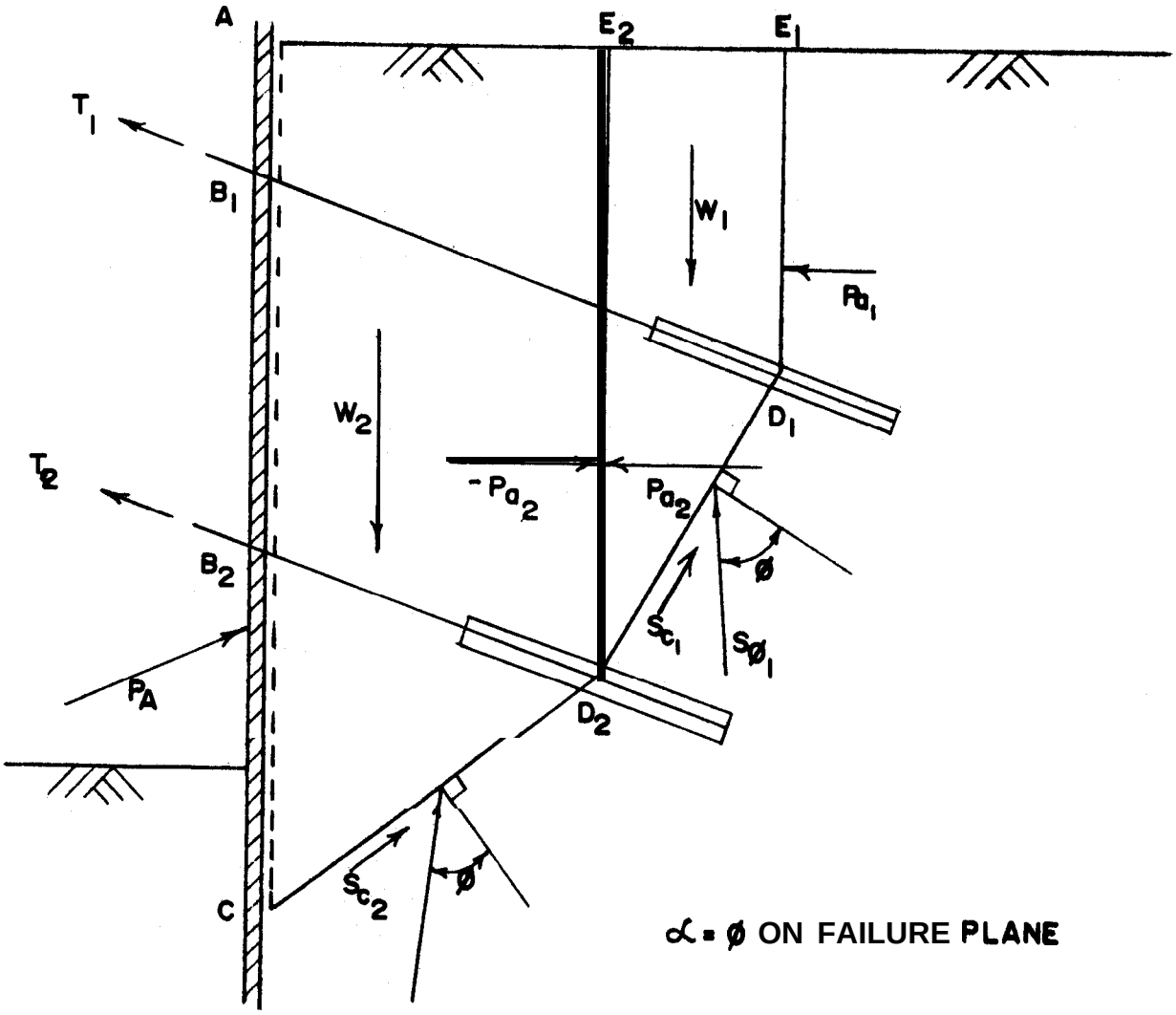


Figure 87. Free body diagram for anchor system with a complex failure surface (safety in terms of the tieback force).

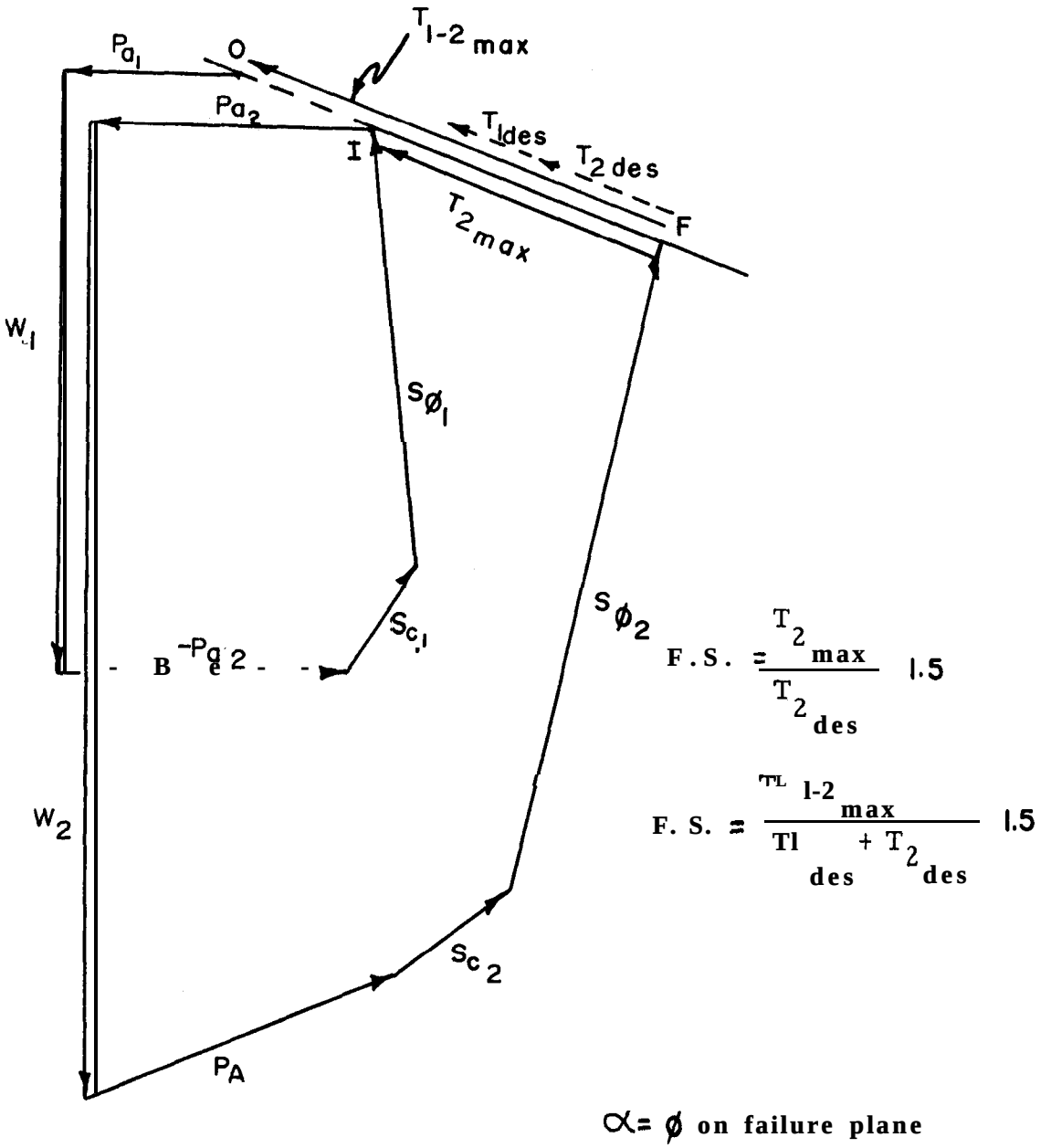


Figure 88. Vector diagram for a complex failure surface (safety factor in terms of tieback force) (after Kranz, 1953 and Ranke and Ostermayer, 1968).

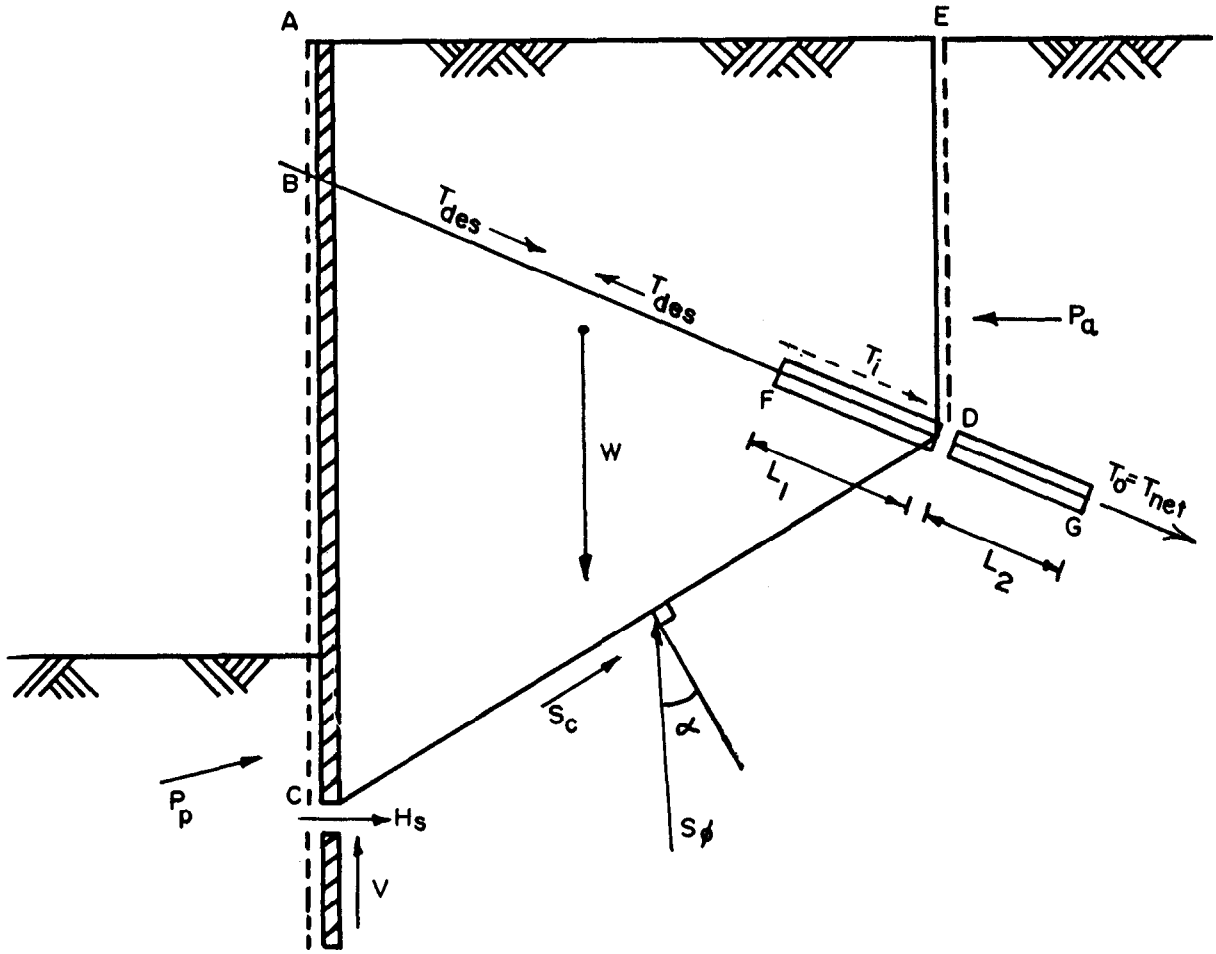


Figure 89. Free body diagram for a failure surface in a single anchor tieback system (free body outside of wall).

Safety Factor in Terms of Soil Strength

Traditional studies of the stability of soil masses express the factor of safety of the soil mass in terms of the available and mobilized soil strengths; F. S. = $\frac{S_{avail}}{S_{mob}}$.

a. Single Anchor. In order to evaluate the force, $T_{net} = T_o$ (see Figure 89), it is assumed that the anchor load is distributed evenly along the length of the anchor. Therefore, the magnitude of the forces, T_i and T_o, will depend upon the location of the failure surface with respect to the achor zone.

Figure 90 shows the vector diagrams used to analyze a single anchor system. For 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.

For cohesionless soils, the factor of safety becomes: F. S. = $\frac{\bar{N} \tan \phi}{\bar{N} \tan \alpha} = \frac{\tan \phi}{\tan \alpha}$.

b. Multiple Anchor Levels. Figure 91 illustrates how the stability of a three anchor level system would be calculated using this method. For simplicity, the example is for a cohesionless soil. Using this method of analysis several trial failure surfaces can be analyzed rapidly. The recommended factor of safety for this method of analysis is 1.5.

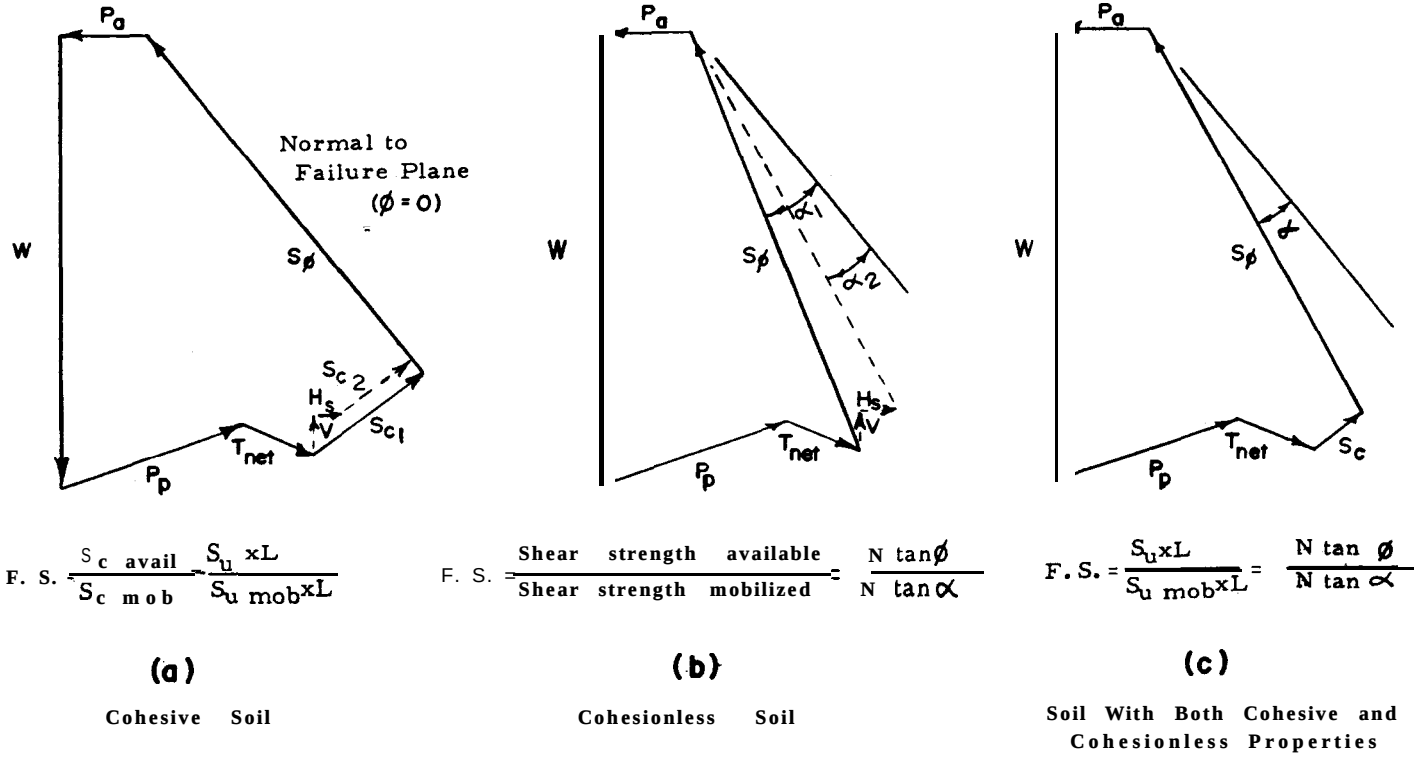
13.23.4 Discussion

The evaluation of the stability of a tied-back earth mass is a trial and error process involving the use of several analytical techniques. The stability methods presented in this section are those that have been commonly used in practice. It is recommended that several techniques be used to evaluate tied-back wall stability.

13.24 Tieback Anchorage Design Considerations

13.24. 1 Suitable Anchorage Strata

Experience has shown that virtually all rock types can be used as anchorage zones; however, not all soil deposits are suit-



$$\text{F. S.} = \frac{S_{c\ avail}}{S_{c\ mob}} = \frac{S_u \times L}{S_{u\ mob} \times L}$$

(a)

Cohesive Soil

$$\text{F. S.} = \frac{\text{Shear strength available}}{\text{Shear strength mobilized}} = \frac{N \tan \phi}{N \tan \alpha}$$

(b)

Cohesionless Soil

$$\text{F. S.} = \frac{S_u \times L}{S_{u\ mob} \times L} = \frac{N \tan \phi}{N \tan \alpha}$$

(c)

Soil With Both Cohesive and Cohesionless Properties

Note: Consideration of Horizontal and Vertical Wall Forces Increases Factor of Safety

Figure 90. Vector diagrams used in analysis with factor of safety defined in terms of soil strength.

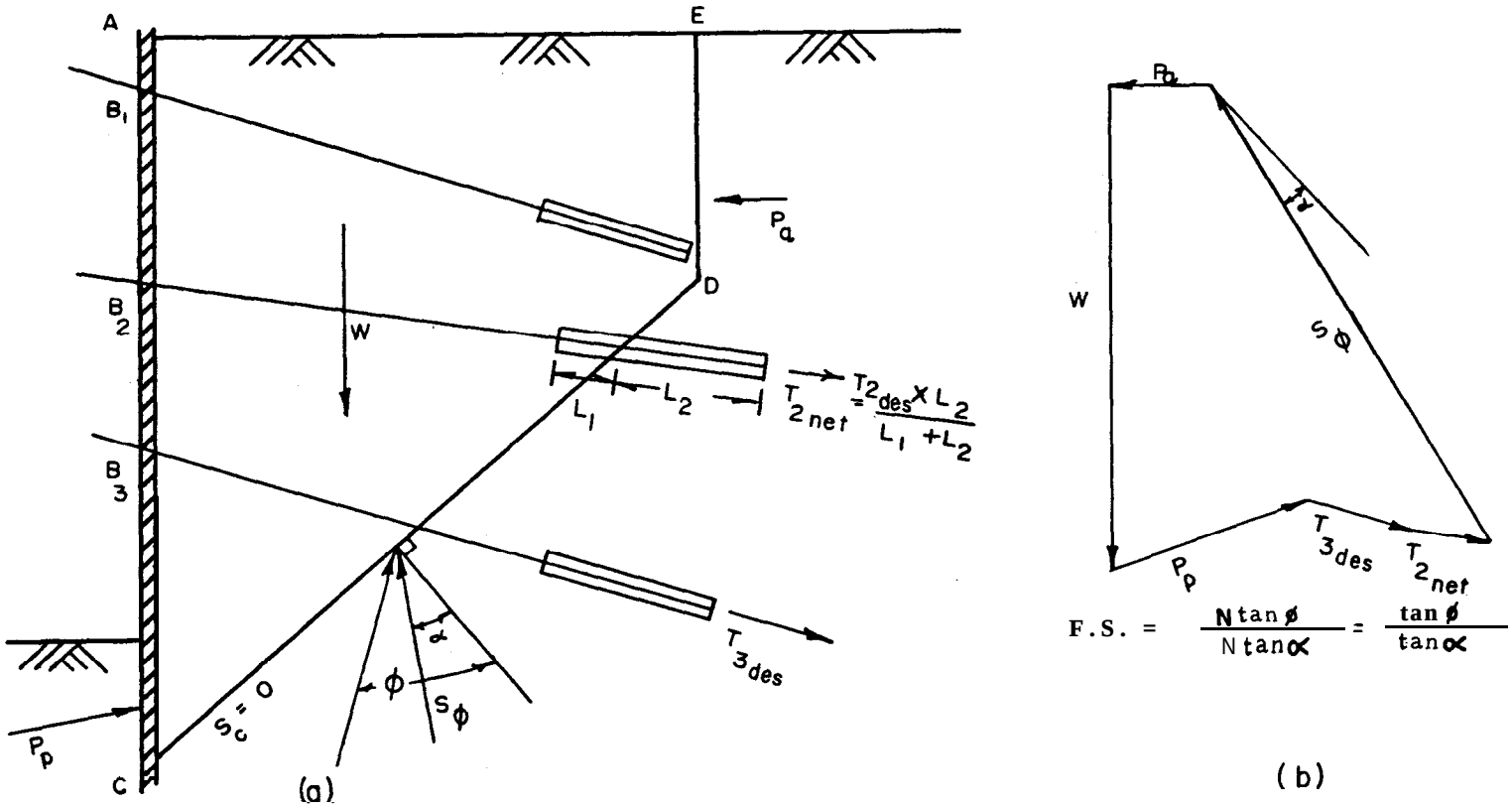


Figure 91. Analysis of a multiple level anchor system (safety factor in terms of soil strength).

able. 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.

13.24.2 Location of Anchors

In U. S. practice, anchors are generally located beyond a line extending at a 30° to 45° slope from the vertical from the base of the excavation to the ground surface (see Figure 92). Recent cases indicate a more common use of 35° to 40° as an angle of inlination for the slip surfaces in granular soil deposits. In cohesive soil deposits, anchors are often founded well behind 45° slip lines. For normal anchor lengths this procedure results in a stable soil mass.

13.24.3 Soil Anchors

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.

Table 9 summarizes the basic anchor-types with respect to the soil types in which they are used. The remaining sections discuss the specific methods used to estimate the load carrying capacity of each of the anchor types.

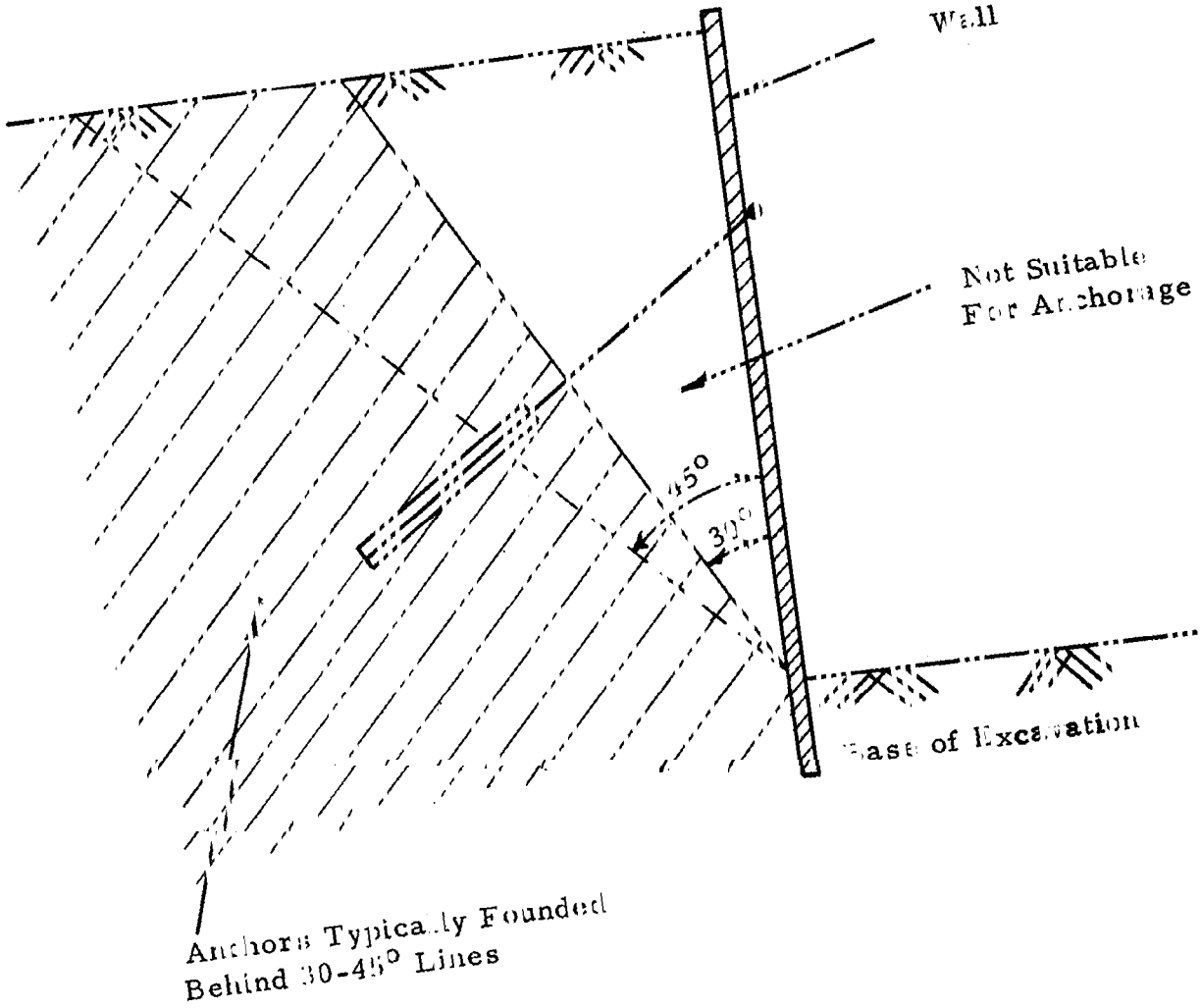


Figure 92. Typical location of anchors.

Table 9. Summary of tieback types and applicable soil types.

Method	Diameter (inches) Shaft Type	Bell Type	Gravity Concrete	Grout Pressure (psi) (1)	Suitable Soils for Anchorage	Load Transfer Mechanism
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

Large Diameter Anchors

Large diameter anchors can be either straight-shafted, single-belled, or multi-belled. These anchors are most commonly used in stiff to hard cohesive soils that are capable of remaining open when unsupported; however, hollow flight augers can be used to install straight-shafted anchors in less competent soils. Figure 93 schematically illustrates several large diameter anchors.

The methods used to estimate the ultimate pullout capacity of large diameter anchors are largely based on the observed performance of anchors and are, therefore, empirical in nature. The following equations can be used to estimate anchor load capacity; field testing of anchors is required to determine true anchor capacity.

a. Straight- shafted Anchor

$$P_u = \alpha S_u \pi d_s L_s$$

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. α = 0.3 - 0.5 (Hanna, 1973a; Broms, 1968; Littlejohn, 1970a; Neely and Montague-Jones, 1974).

b. Belled Anchor

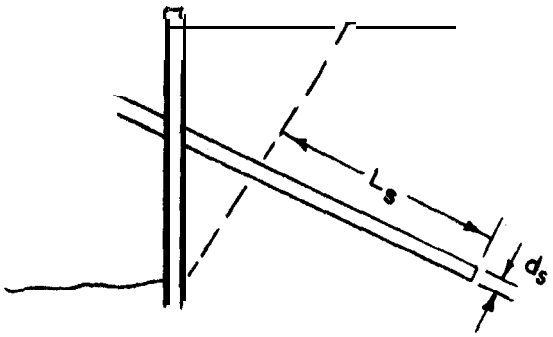
$$P_u = \alpha S_u \pi d_s L_s + \pi/4 (D^2 - d_s^2) N_c S_u$$ (Littlejohn, 1970a)

where:

d_s, L_s, S_u and α are as before

D = diameter of anchor bell

N_c = bearing capacity factor = 9



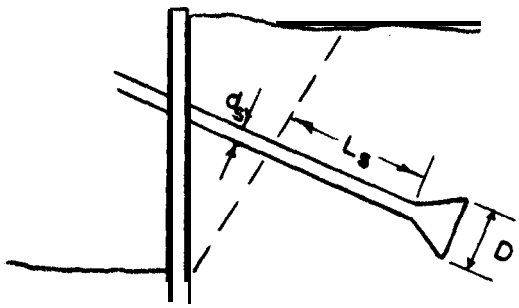
(a)

Friction Anchor

ESTIMATED LOAD FOR ANCHORS IN COHESIVE SOIL

$$Pu = \alpha S_u L_s \pi d_s$$

α = 0.3-O. 5



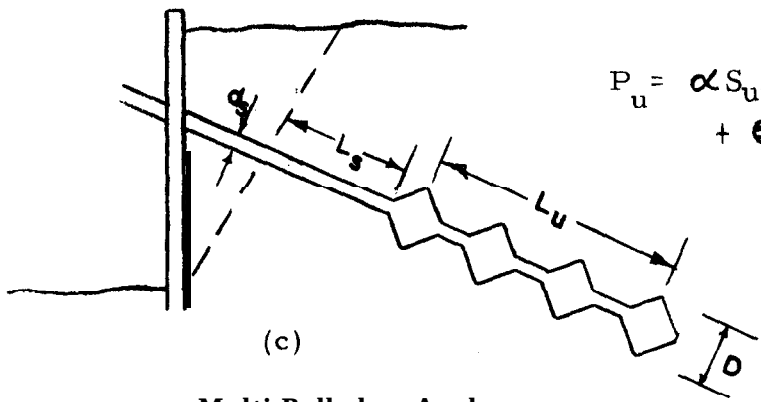
(b)

Belled Anchor

$$P_u = \alpha S_u L_s \pi d_s + \frac{\pi}{4} (D^2 - d_s^2) N_c S_u$$

α = 0.3-O. 5

N_c = 9



(c)

Multi-Belled Anchor

$$P_u = \alpha S_u L_s \pi d_s + \frac{\pi}{4} (D^2 - d_s^2) N_c S_u + \beta S_u \pi D L_u$$

α = 0.3-0.5

β = 0.75-1.00

N_c = 9

Figure 93. **Schematic representation of large diameter anchors'**

c. Multi-belled Anchor

$$P_u = \alpha S_u \pi d_s L_s t \quad \pi/4 (D^2 - d_s^2) N_c S_u + \beta S_u \pi D L_u$$

where:

d_s, L_s, S_u, α, N_c and D are as before

L_u = length of underreamed portion of anchor

β = reduction factor in S_u for soil between underream tips = 0.75 - 1. 0 (Littlejohn, 1970a; Bassett, 1970; Neely and Montague-Jones, 1974)

In order for failure to occur between the underream tips, the tips must be spaced at 1.5 - 2. 0 times the belled diameter with the bell diameter equal to 2.0 to 3.0 times the shaft diameter.

Small Diameter Anchors

Small diameter anchors are generally installed in granular soils with grouting taking place under high pressures (usually greater than 150 psi (1035 kN/m^2). The anchor capacity will depend upon the soil type, grouting pressure, anchor length, and anchor diameter. The way in which these factors combine to determine anchor load is not clear; therefore, the load predicting techniques are often quite crude. The theoretical relationships in combination with the empirical data can be used to estimate ultimate anchor load. Figure 94 schematically illustrates several small diameter anchors.

a. Theoretical Relationshins

1. No grout penetration in anchor zones

$$P_u = p_i \pi d_s L_s \tan \phi_e$$ (Littlejohn, 1970a; Broms, 1968)

where:

d_s = diameter of anchor shaft

L_s = length of anchor shaft

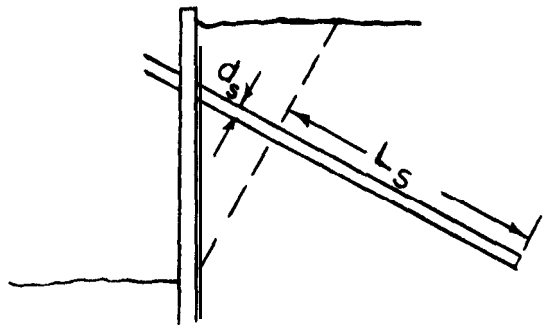
ϕ_e = effective friction angle between soil and grout

p_l = grout pressure

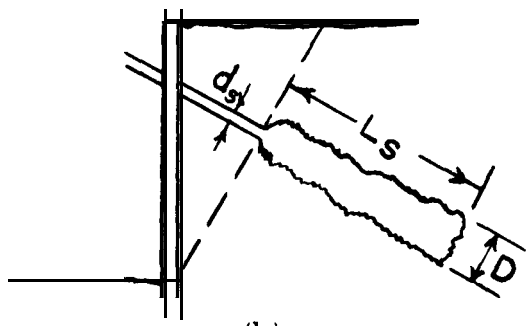
or $P_u = L_s n_l \tan \phi_e$ (Littlejohn, 1970a)

where:

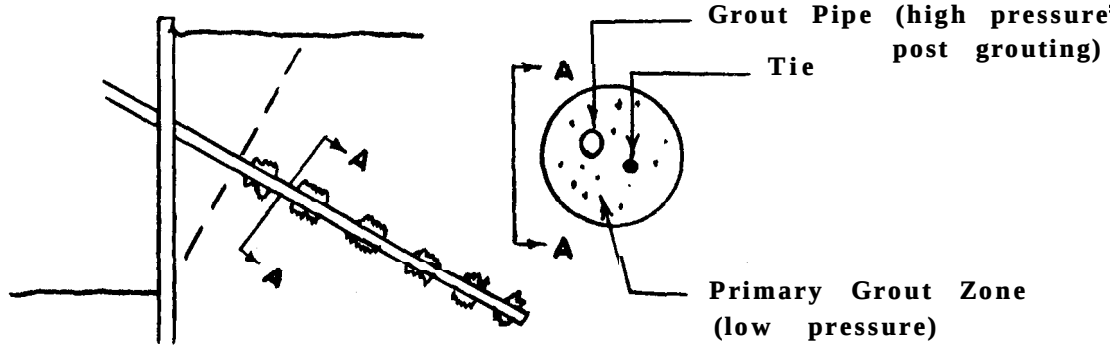
n_l = 8.7 - 11.1 k/ft (127 - 162 kN/m)



(a)
Friction Anchor
(No Grout Penetration)



(b)
Bulb Anchor
(Grout Penetration)



(c)
Regroutable Anchor
(Local Grout Penetration)

Figure 94. Schematic representation of small diameter anchors.

2. Grout penetration in anchor zone (very pervious soils)

$$P_u = A\,\bar{\sigma}_v\,\pi D L_s \tan\phi_e + B\,\bar{\sigma}_{v@\,end}\,\pi/4\,(D^2 - d_s^2)$$

(Littlejohn, 1970a)

where:

d_s, D, L_s and ϕ_e are as before

$\bar{\sigma}_v$ = average vertical effective stress at anchor entire anchor length

$\bar{\sigma}_{v@\,end}$ = vertical effective stress at anchor end closest to wall

$A = \frac{\text{Contact pressure at anchor soil interface}}{\text{effective vertical stress } (\bar{\sigma}_v)}$

Littlejohn reports typical values of A ranging between 1 and 2.

B = bearing capacity factor similar to N_q but smaller in magnitude. A value of

$B = \frac{N_q}{1.3 - 1.4}$ is recommended provided $h \geq 25D$; where h is the depth to anchor.

Since the values of D, A, and B are difficult to predict, Littlejohn (1970a) also suggests:

$$P_u = L_s\, n_2 \tan\phi_e$$

where:

n_2 = 26 - 40 kips/ft (380 - 580 kN/m)

L_s = 3 - 12 ft (0.9 - 3.7 m), D = 15 - 24 inches (400 - 610 mm).

depth to anchor = 40 - 50 ft (12.2 - 15. lm).

b. Empirical Relationships

Figure 95 presents an empirical plot of the load capacity of anchors founded in cohesionless soils. This figure was developed by Ostermayer (1974) and represents the range of anchor capacities that may develop in soils of varying densities and gradations.

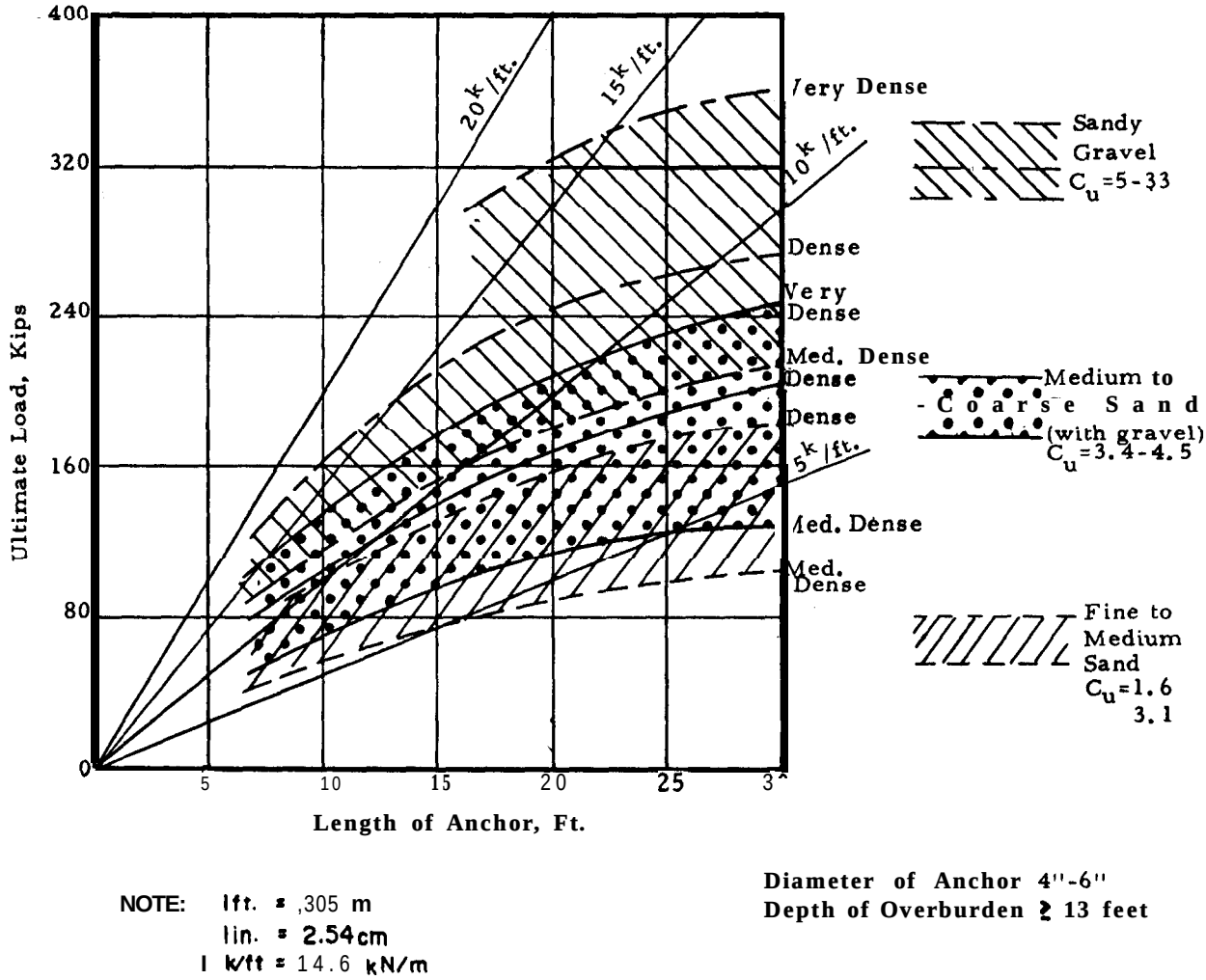


Figure 95. Load capacity of anchors in cohesionless soil showing effects of relative density, gradation, uniformity, and anchor length (after Ostermayer, 1974).

The following table summarizes the load capacity of single injection small diameter anchors :

Soil	Ultimate Load kips/ft	kN/m
Clean sand/gravel soils	10 - 20	145 - 290
Clean medium to coarse sands	7 - 15	100 - 220
Silty sands	5 - 10	70 - 145

c. Regroutable Anchors

Regroutable anchors are small diameter anchors that allow the Load carrying capacity of the anchor to be improved after installation and testing. Figure 94 schematically illustrates a re-groutable anchor.

Jorge (1969) reported an improvement of anchor load capacity in both cohesionless and cohesive soils with a regroutable anchor. Figure 96 presents a summary of the results with data on very stiff clay from Osterxnayer (1974).

A summary of data on cohesive soils for regroutable anchors is presented in Table 10 and Figure 97. These values can be used to estimate regroutable anchor loads.

13.24.4 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 13.25.

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

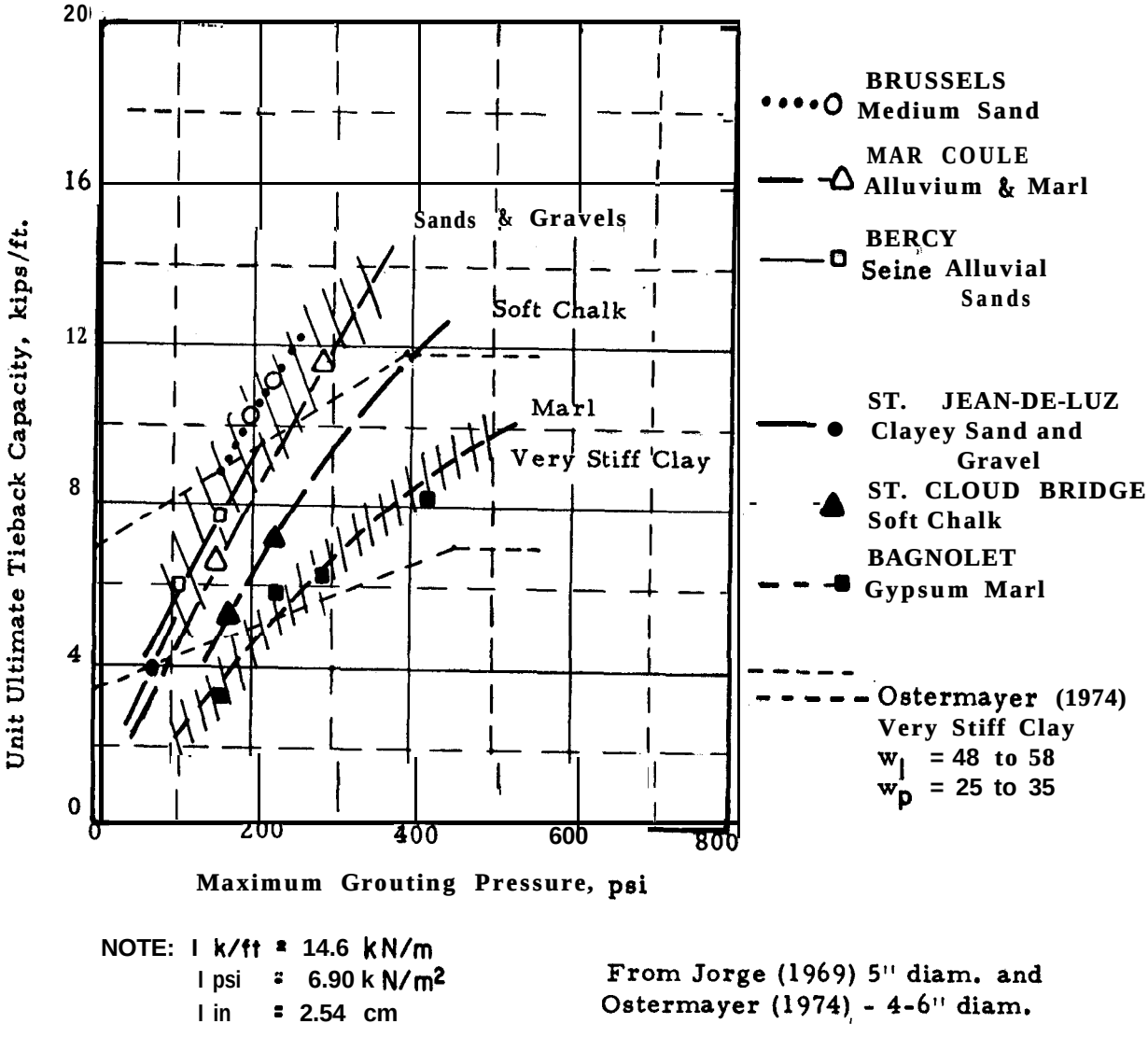


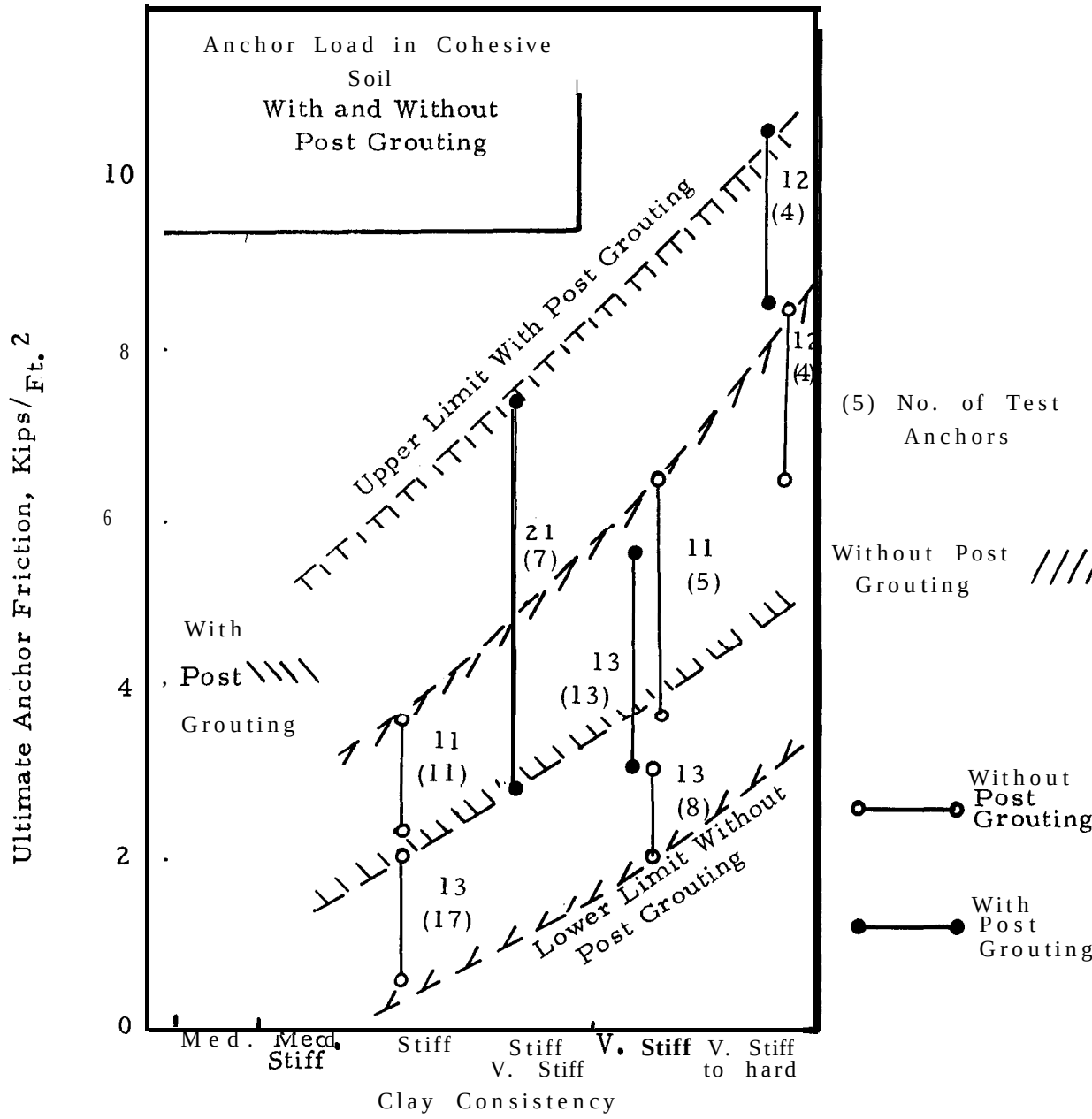
Figure 96. Ultimate anchor capacity as a function of grout pi-essure.

Table 10. High pressure small diameter tiebacks in cohesive soil (after Ostermayer, 1974).

Soil Type	Typical Skin Friction (per square foot of grouted zone) Without Post-Grouting	With Post-Grouting
Marl Clay - medium plastic (w_l = 32 to 45; w_p = 14 to 25)		
Stiff	2200 - 3500	----------
Very Stiff	3500 - 6500	----------
Marl Sandy Silt - medium plastic (w_l=45; w_p=22)		
Very stiff to hard	6500 - 8500	8500 - 10,500
Clay - medium to highly plastic (w_l = 45 - 59; w_p = 16 - 35)		
Stiff	500 - 2000	
Very Stiff	2000 - 3000	3000 - 5500

Note:

1. **Tiebacks** 3-1/2" to 6" 0. D.
2. Values are for lengths in marl - 15 to 20 feet and for lengths in clay - 25 to 30 feet
3. 1 psf = 0.48 kN/m^2
   1 in = 2.54 m
   1 ft = 0.305 m



Ostermayer (1974)

11 Marl Clay	w_l = 32 to 45;	w_p = 14 to25
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 K/ft^2 = 47.9 KN/m^2

Figure 97. Effect of post grouting on anchor capacity,

a point on the anchor and extending to the top of the rock. Figure 98 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^{o} to 90^{o} 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, then the anchor is generally believed to be safe since any cohesion or other rock strength properties have been ignored. However, if desired, 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 inches to 6 inches in 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. Littlejohn (1975) reports the results of studies performed by several authors that indicate that this assumption may not be valid. However, in the absence of more detailed information the established methods should still be used. The designer should be aware of the potential problems of local debonding, Rigid field testing should be establish anchor adequacy.

The equation used to estimate anchor capacity is:

$$P_u = \pi d_s L_s \, \delta_{skin}$$

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 11.

In soft rock, it is also possible to form belled or multi-underreamed anchors. The equations governing the ultimate loads in these rocks are given in previous equations (Section 13.24.3). In these cases, the cohesive strength of the rock becomes the controlling quantity.

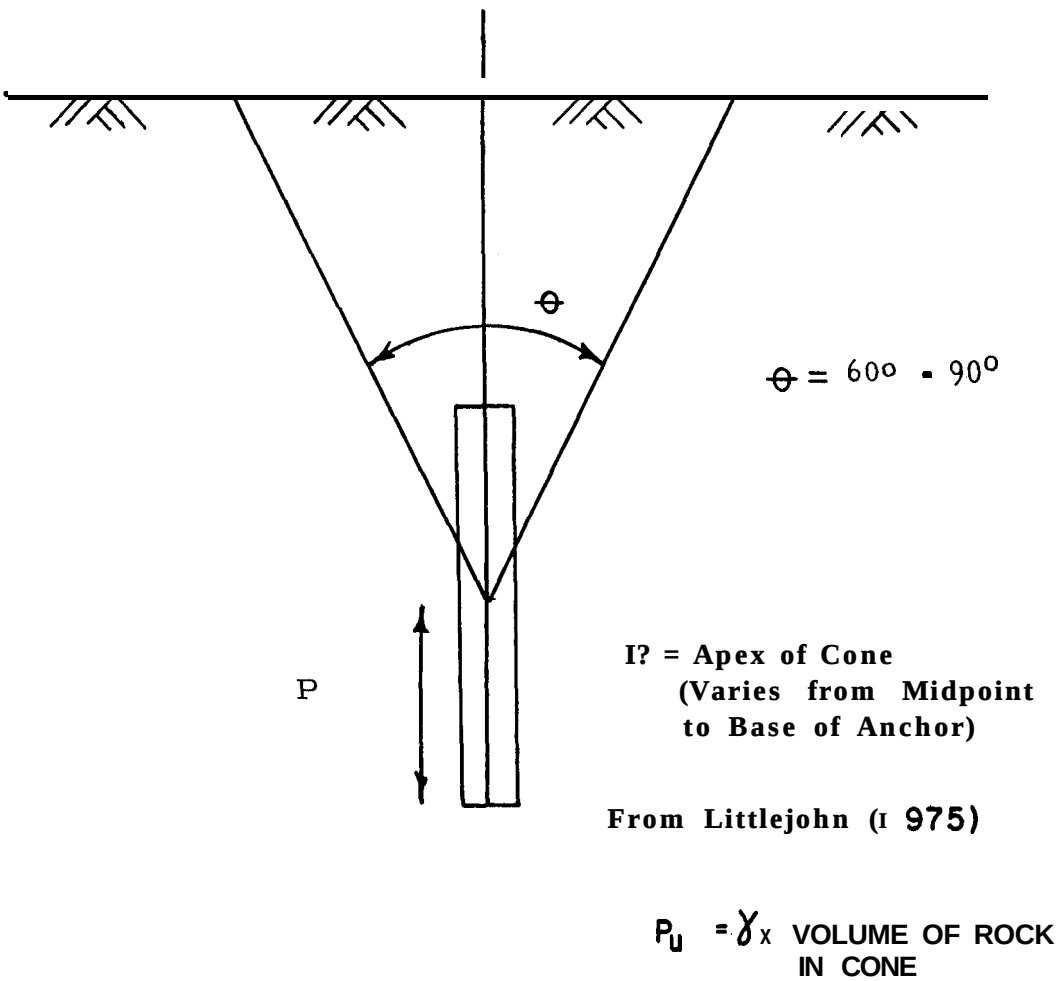


Figure 98. Schematic drawing of design quantities for failure in a rock mass.

Table 11. Typical values of bond stress for selected rock types.

Rock Type (Sound, Non-Decayed)	Ultimate Bond Stresses Between Rock and Anchor Plug (σ_{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 psi = 6.90 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)

13.24. 5 Safety Factor of Soil or Rock

Safety Factor With Respect to Shear

a. Soil Anchors. 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 the anchor in the soil type and a minimum of five carefully monitored pullout tests (or to 175 percent of design load) for the full anchor load. Production test monitoring including creep measurements 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.

Safety Factor With Respect to Creep

In some instances the design anchor load may be based on the creep of the anchor. At a particular load, the anchor may have an adequate factor of safety against pullout; however, the anchor may creep causing a loss of load and result in wall movement. The rate of creep of the anchor must be kept to acceptable values. To date, the criteria for determining acceptable creep rates are based upon field observations. Recommended creep criteria appear in the section on field testing of anchors (see Section 13.40).

13.24.6 Discussion

The theoretical and empirical methods for predicting anchor load are used as a first step in anchor design. Final anchor capacity should be verified by testing of each anchor beyond its design load.

13.25 Tendon Design and Load Transfer

13.25.1 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 done either through bonding forces between the tie and the grout (tension anchor) or by a device or plate at the base of the anchor rigidly attached to the tie (compression anchor).

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 through this zone during testing. Therefore, the following methods are used to insure that all load is 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 bond free zone.
   b. Backfill the bond-free zone with sand or 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 99 illustrates the recommended treatment for bond free zones.

13.25.2 Steel Tie Member

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 until large anchor capacities are required. Bond strengths will typically be between 200 and 250 psi (1.38 - 1.75 N/mm^2) for cement grouts and concrete.

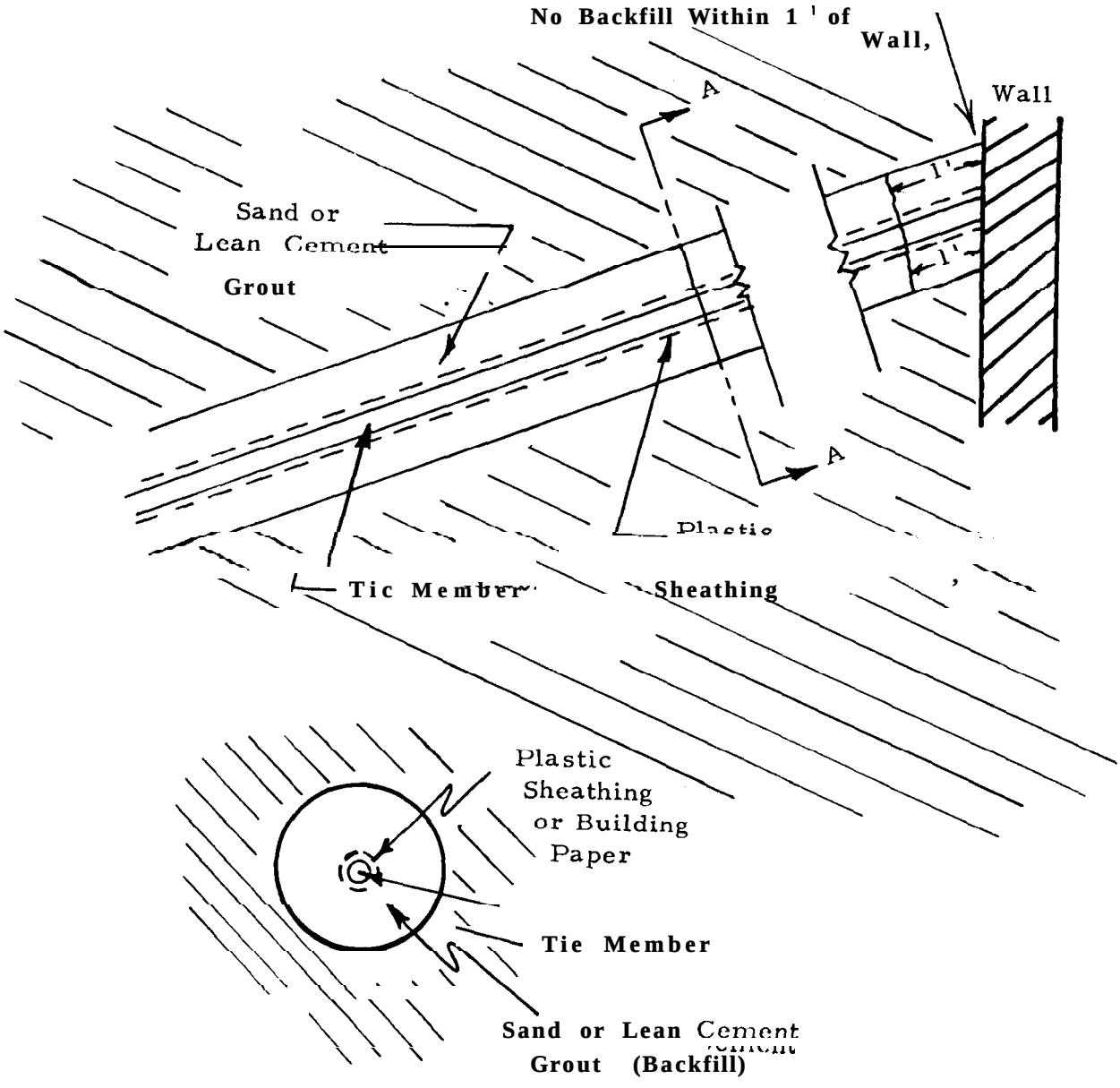


Figure 99. Recommended treatment for bond free zone.

High strength steel wire strands, cables, and bars are most commonly used for tie members. Often the choice of the type of tie is limited by the method of installation or convenience. Table 12 Lists typical properties and dimensions of steel wires, strands, and bars for tie members.

i3.25.3 Grout and Concrete

Resin Grouts

Resin grouts are used because of their quick setting times of ten to twenty minutes (for 80 per cent to 90 percent ultimate strength). This allows anchor testing shortly after installation. 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.

Cement Grouts

Cement grouts are most commonly used in small diameter anchors . Generally, high early strength cement is mixed with water to form a neat cement grout. A low water to cement ratio is used during the mixing process. The anchors are usually tested 24 - 72 hours after installation of the grout. 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 with a mixture of high early strength cement, water, and sand or fine gravel. This concrete mixture is used because the sand or gravel filler is cheaper than cement and does not appreciably reduce the strength of the grout.

13.25.4 Factors of Safety

Table 13 presents recommendations for design steel stresses in temporary ties. These values represent minimum factors of safety and should be increased for permanent tieback installations or critical temporary anchors. For permanent anchors, it is recommended that a minimum factor of safety against ultimate load of 2.0 be applied for the design loads in the steel members.

Table 12. Typical steel properties and dimensions for ties.

Type of Tie	Diameters (inches).	Ultimate Stress fu (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/mm^2
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.

Table 13. 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

[1] Maximum 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.

13.25.5 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, additional corrosion protection methods may be used. Specially treated grout, steel members or extra steel may be used to ensure that the anchors will perform adequately. Compression anchors have been used to prevent radial cracking of the grouted portion of the anchor. These cracks result in faster corrosion of the anchor parts.

13.30 CONSTRUCTION CONSIDERATIONS FOR. TIEBACKS

The purpose of this section is to describe the basic construction procedures and techniques used to install tiebacks. Since the major difference between tied-back wall construction and internally braced wall construction is the installation and testing of the tiebacks, this section is limited to a discussion of the construction considerations for each of the different tieback types.

13. 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 100).

13. 32 Construction Techniques Common to Tiebacks

Stated very simply, the construction sequence for the installation of a tieback consists of the following steps:

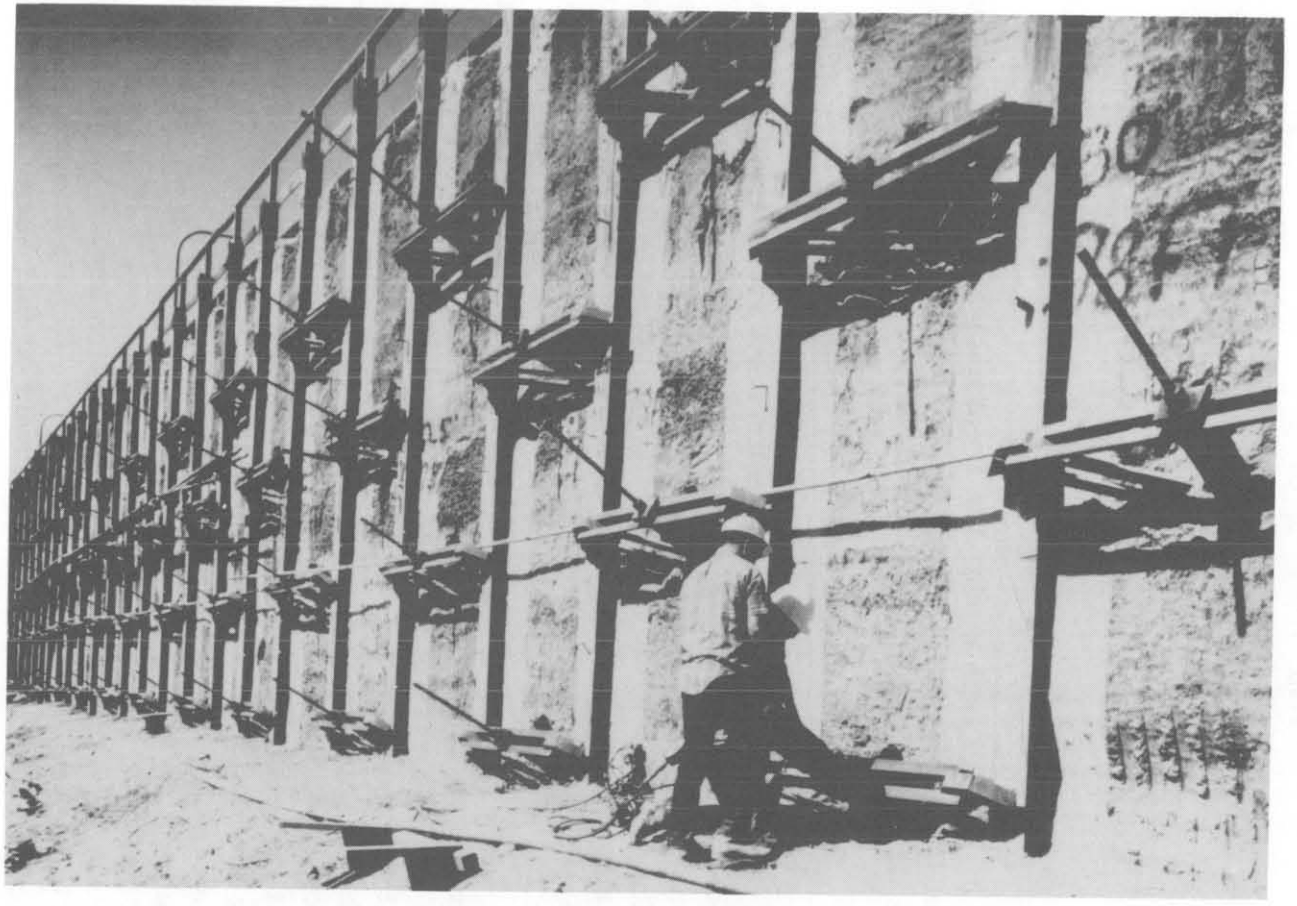


Figure 100. Example of tied-back wall using channel sections as wales. (Courtesy of Hughes Tool Company).

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. Backfill the bond free zone with a weak material to prevent bonding.

5. Tension and test the tie

6. 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.

13.32.1 Compression or Tension Anchors

Compression anchors are those where the entire anchor 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.

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. 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, the wires or cables may be unraveled at the end to ensure that there is enough surface area for bonding. The tensile and shearing forces in the concrete are larger for a tension anchor, and hairline cracking in the anchor has been observed in these anchor types (Ostermayer, 1974).

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.

13.32.2 Centering Ties

Spiders or other centering devices are required in larger diameter holes. This is particularly true for wire or cables be-

cause of their flexibility. In small diameter holes steel bars or rods often require centering while cables and wires generally will not because of their irregular surface.

13.32.3 Tendons

The different tendon (tie) types and their material properties have been described in an earlier section (see Section 13.25). The choice of which tendon type to use (bar, strand, or wire) is virtually independent of anchor type. Recently, the use of high strength steel rods has increased since these rods can be easily threaded into detachable points in the base of the anchor and allow for easy connections at the wall.

13.32.4 Anchorage at Wall

Friction connections have ridges or teeth that grip the tendon and cut into it slightly. The steel area is therefore reduced which leads to increased stresses at that point in the tie. Figure 101 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 102 illustrates a typical button head connection.

Threaded conections 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 103 illustrates a threaded connection. In practice, threaded connections are more commonly used than button head connections.

13.33 Construction Techniques and Procedures for Different Anchor Types

Table 14 summarizes the equipment installation techniques and preferred soil types for common anchor types. Figures 104 through 107 show several tieback installations and some of the installation equipment.

13.40 FIELD TESTING

13.41 Reasons

The major reasons for field testing are:

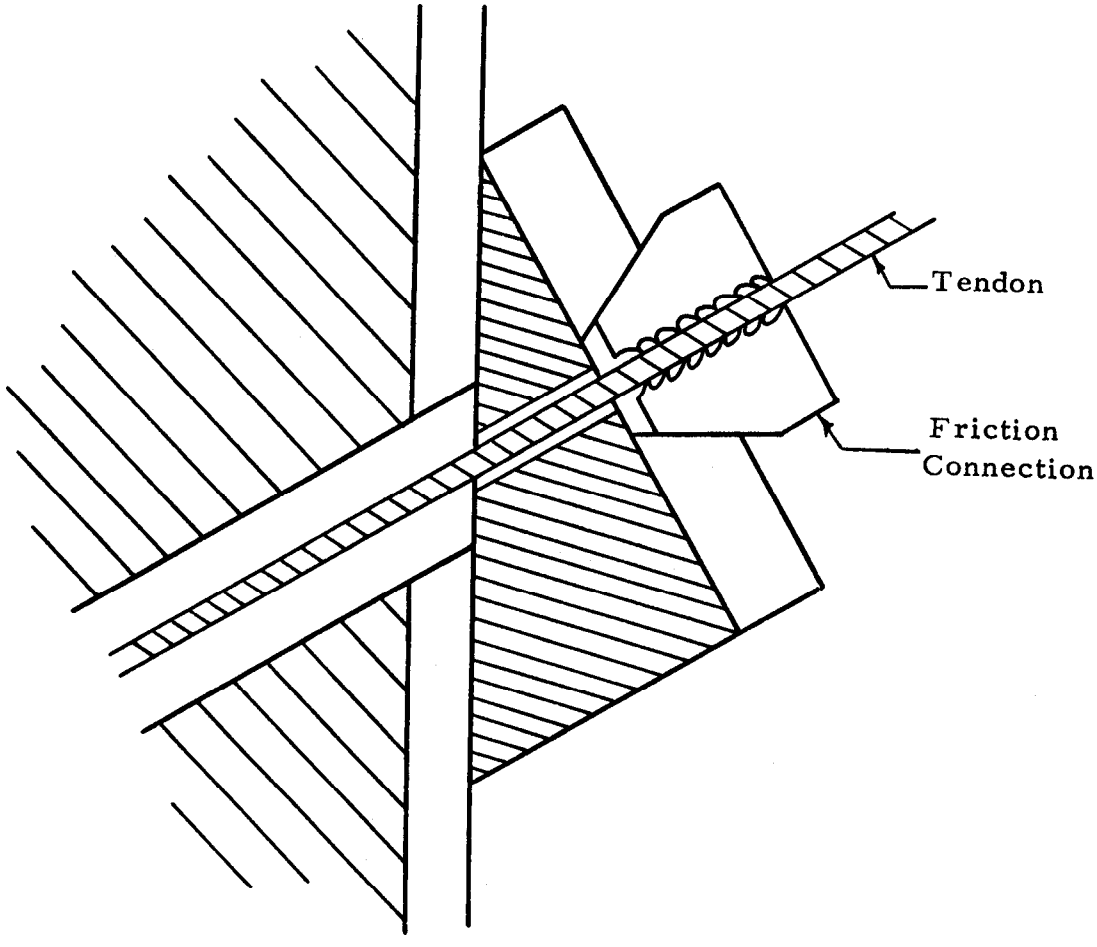


Figure 101. Friction connection used to tie anchor to wall.

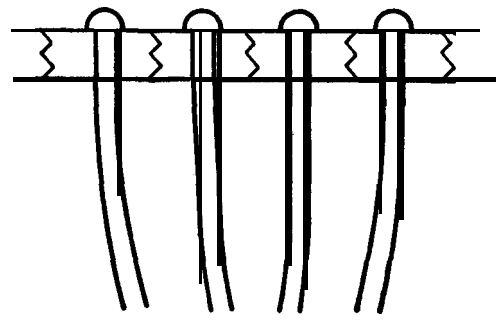


Figure 102. Button-head connection for wire ties.

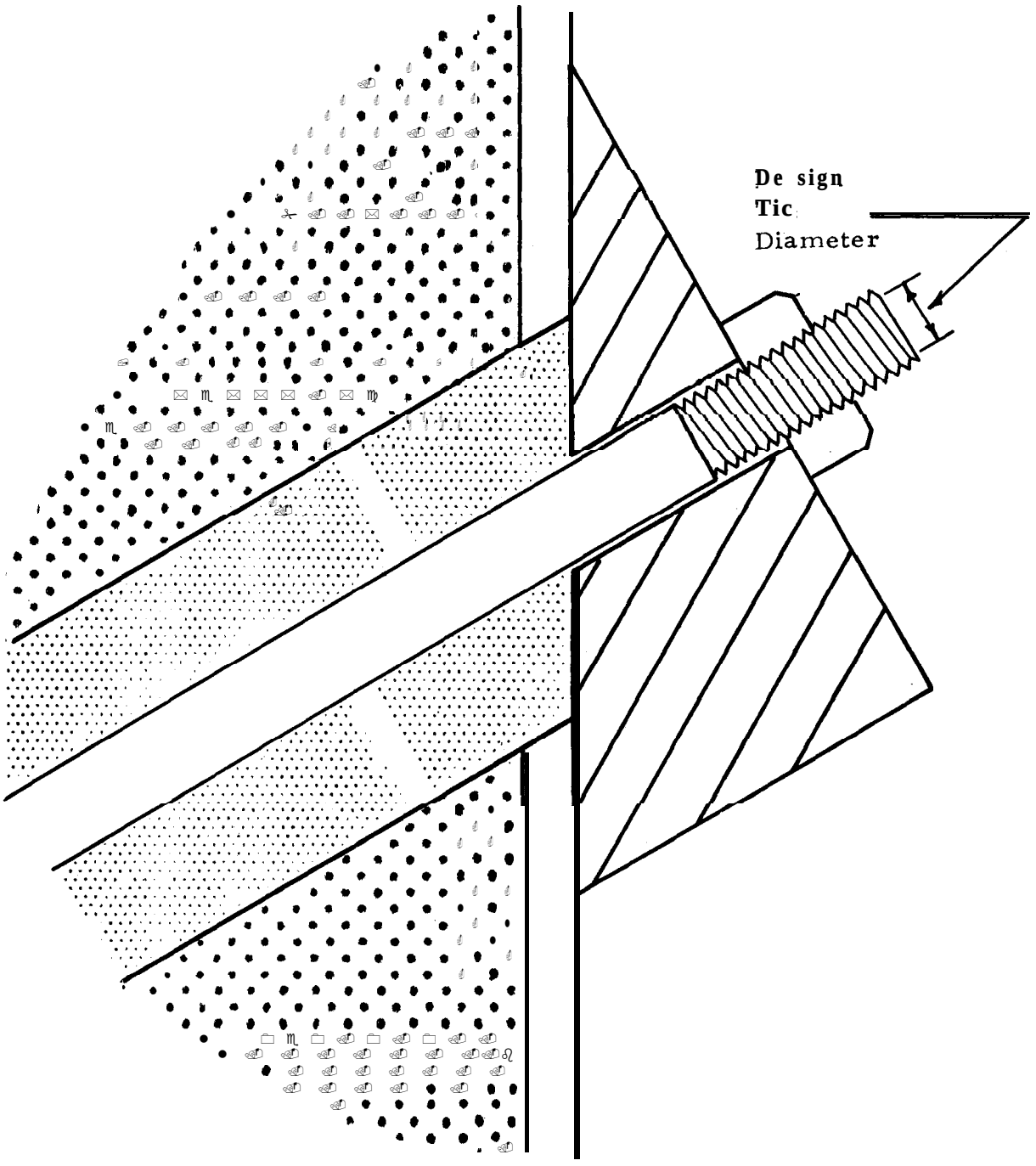


Figure 103. Threaded connection for tying anchor to wall.

Table 14. 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 (1) Solid stem Augers	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) (15x-n - 40m)	Pumped concrete.	Spacers and plate generally used.	0^{o} - 90^{o} (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^{o} - 90^{o} (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-m&ted 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^{o} - 60^{o})	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 $l_u \geq 1.5 d_s$	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^{o} - 60^{o})	Lean concrete, weak cement grout, or sand. Entire tie length except for plate is unbonded in some methods.

Table 14. 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.	Crawl&-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 lift 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 co-on.	As before for small diameter anchors.	As before.
Rock Anchors (1) Multi-Under-reamed	Used in softer competent rock.			See Section on Underreamed Soil Anchors.				

Table 14. 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. Driving 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.

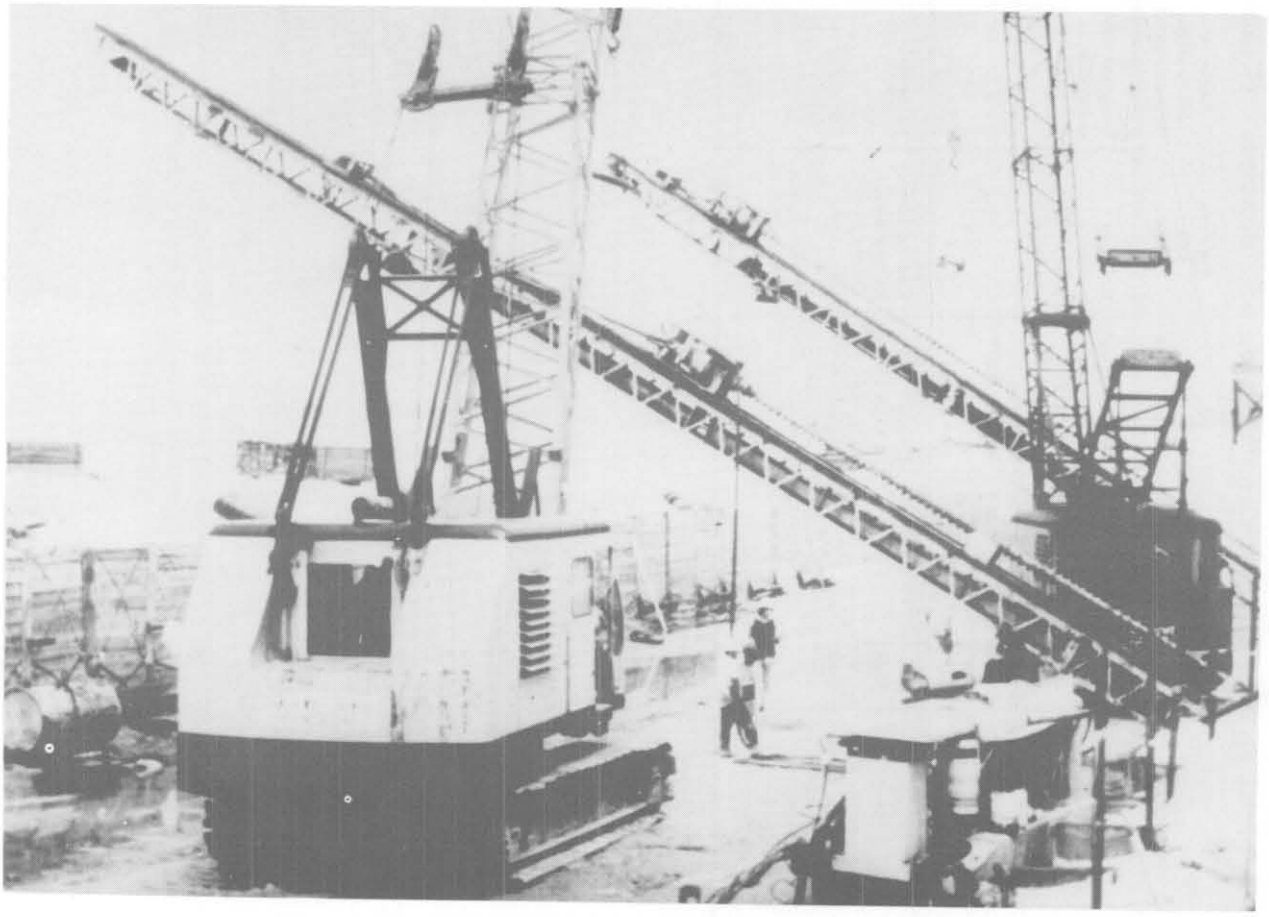


Figure 104. Crane suspended auger rig.
(Courtesy of Spencer, White, and Prentis).

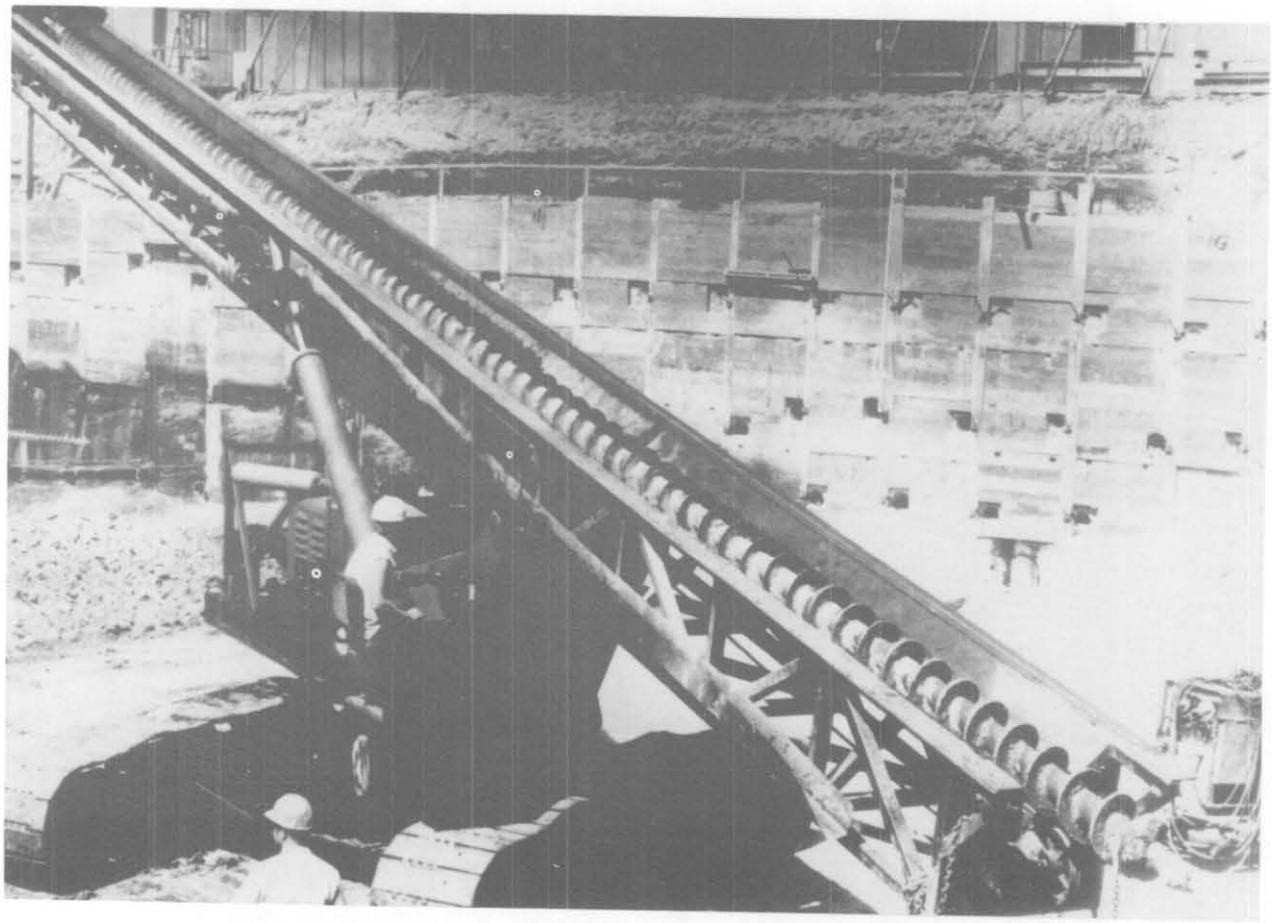


Figure 105. Crawler mounted auger rig.
(Courtesy of Spencer, White, and Prentis).



Figure 106. Crawler mounted auger rig.
(Courtesy of Acker Drill Company).



Note: Excavation has proceeded below tieback level.

Figure 107. Crawler mounted auger rigs.
(Courtesy of Hughes Tool Company).

1. Load - Theoretical bases for establishing design load are given in Section 13.24. These are crude at best and should only be used for 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 - Prooftesting 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.

13.42 Current Practice

The specific criteria used by engineers/contractors to field test anchors varies. However, it is generally recommended that pullout tests be performed on anchors to determine the design characteristics and anchor capacities. These may involve actual failures or loadings to some specified load greater than the design load (e.g. 200% of design). During anchor installation several special anchors may be tested to a load significantly in excess of design (e.g. 150 - 200%) to ensure that an adequate factor of safety against pullout is being maintained. Production anchors should be tk'sted to a load in excess of the design load (e. g. 120 - 150%) to prove the adequacy of every anchor. It is also recommended that the rate of anchor creep be observed during all anchor testing.

13.43 Recommendations

The following recommendations are made for installation of temporary anchors to support excavations in the presence of nearby structures.

13.43.1 Recommendations for Special Load Tests

Test Loads

Soil and Site Conditions	Load	Remarks
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.

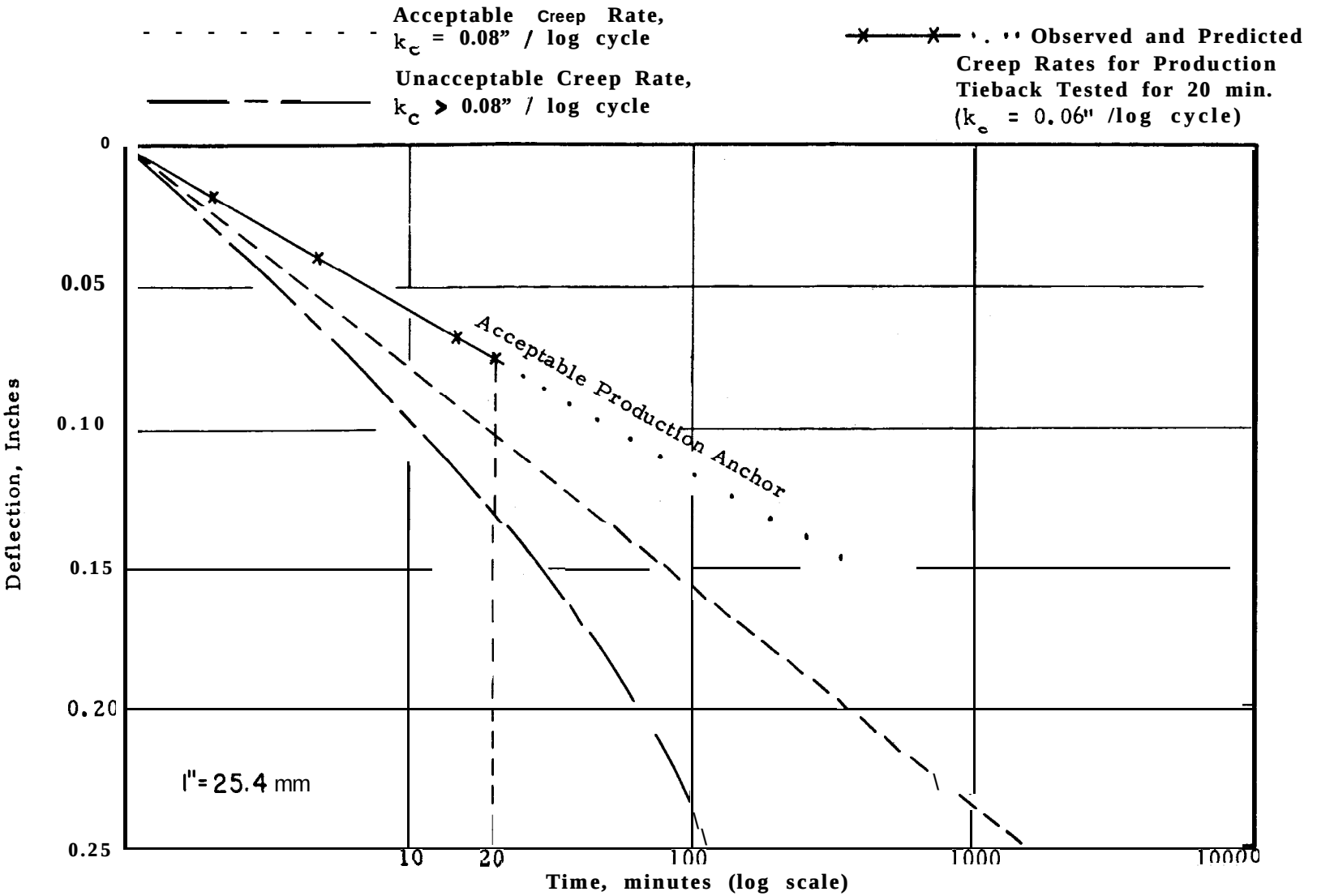
2. Reasonable experience with soil and anchor. Nearby structures within the "zone of influence".	150% of de sign	5% of production ties should be tested in this manner. In addition, 3 ties in each soil formation should be tested to 200% of design.[1]
3. Little experience or unsatis - factory experience 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 de sign.[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, 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 250% of design or failure. 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 appropriateness of the design a s sumption s .

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 2mm (0.08") per logarithmic cycle of time (see Figure 108).
Cohesive Soil	Load duration of 24 hours for all cohesive soils. Creep rate should not exceed 2mm (0.08") per logarithmic cycle of time (see Figure 108).



108. Example of recommended method of recording creep data.

Method of Load Application

1. Load anchor in increments of 25% of design load to 12 5% 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.

13.43.2 Recommendations for Production Anchor Load Tests

The following recommendations are the minimum test test criteria for anchors.

Test Load

Load the anchor to 125 percent of the design load. Care must be taken to ensure that the recommended stresses in the ties are not exceeded (see Section 13.25).

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.

13.43. 3 Evaluation of Anchor Test Loading

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 is not enough to ensure that the anchor capacity is sufficient. Any proof loading of ties should include a plot of load versus tie enlogation.

Figure 109 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" in the grouted zone. The "effective length" can vary from zero ($l_{eff} = 0$) to the length of the tie in the grouted zone ($l_{eff} = l_g$). 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 (lb). The other limiting condition is where the entire anchor load is transferred at the base of the anchor zone. Figure 110 schematically illustrates the load distribution in the tie for several cases.

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 ties will generally be acceptable if the measured elongation is less than the maximum theoretical elongation at 125% of design load (i. e. $l_{eff} \leq 1_g$).

Creep Considerations

To assess the creep characteristics of an anchor, a plot of anchor movement versus time should be prepared. Anchor movement should be plotted to an arithmetic scale while the time readings should be plotted to a logarithmic scale. Figure 108 illustrates a plot and defines the creep coefficient, k_c, which must be less than 2 mm (0.08") per logarithmic cycle of time.

If there is concern that even these creep movements may affect the performance of the tieback system, the testing criteria may be more rigid. For instance, the maximum allowable creep coefficient, k_c, may be reduced or the test load may be increased.

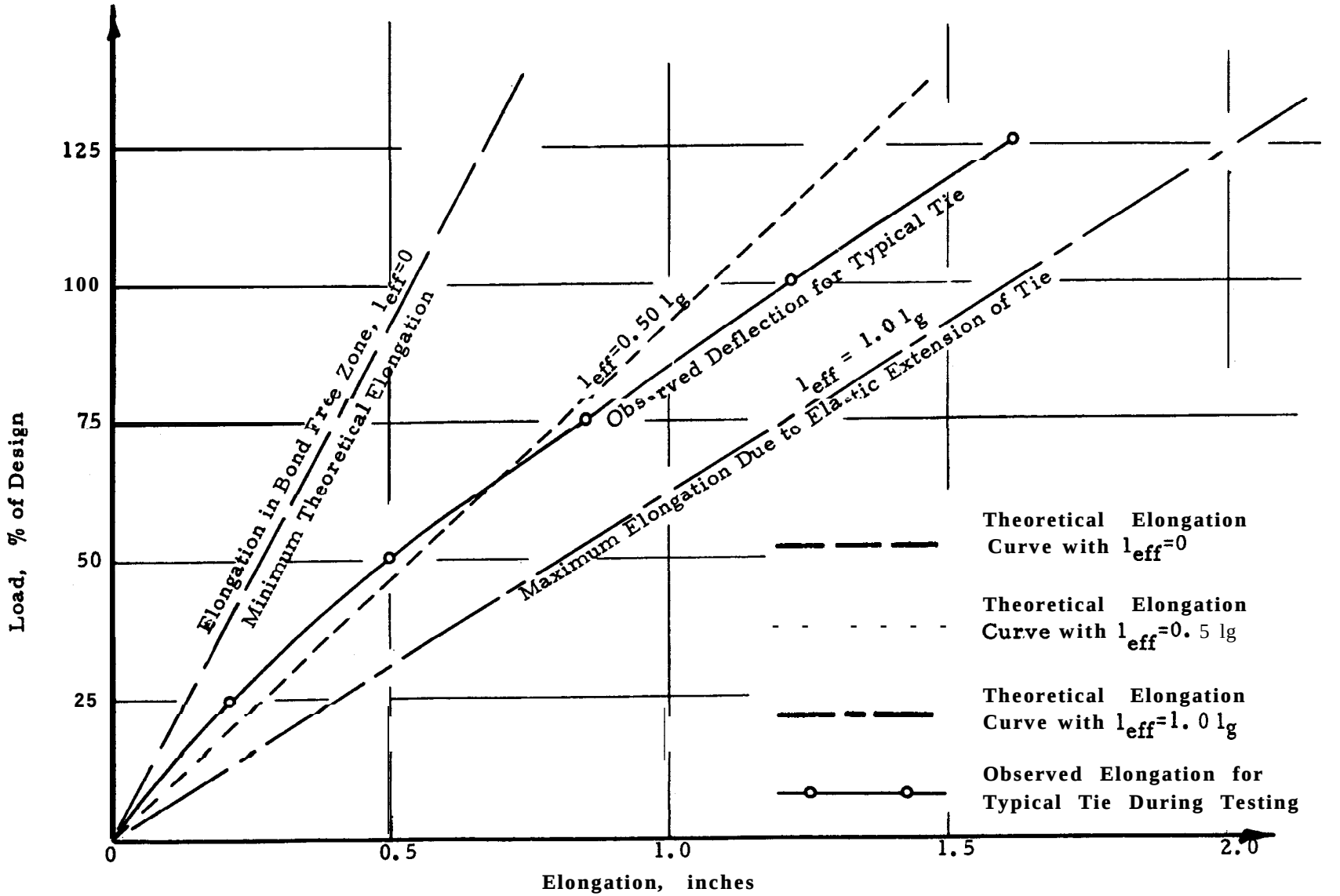


Figure 109. Typical plot of load vs. elongation during test loading.

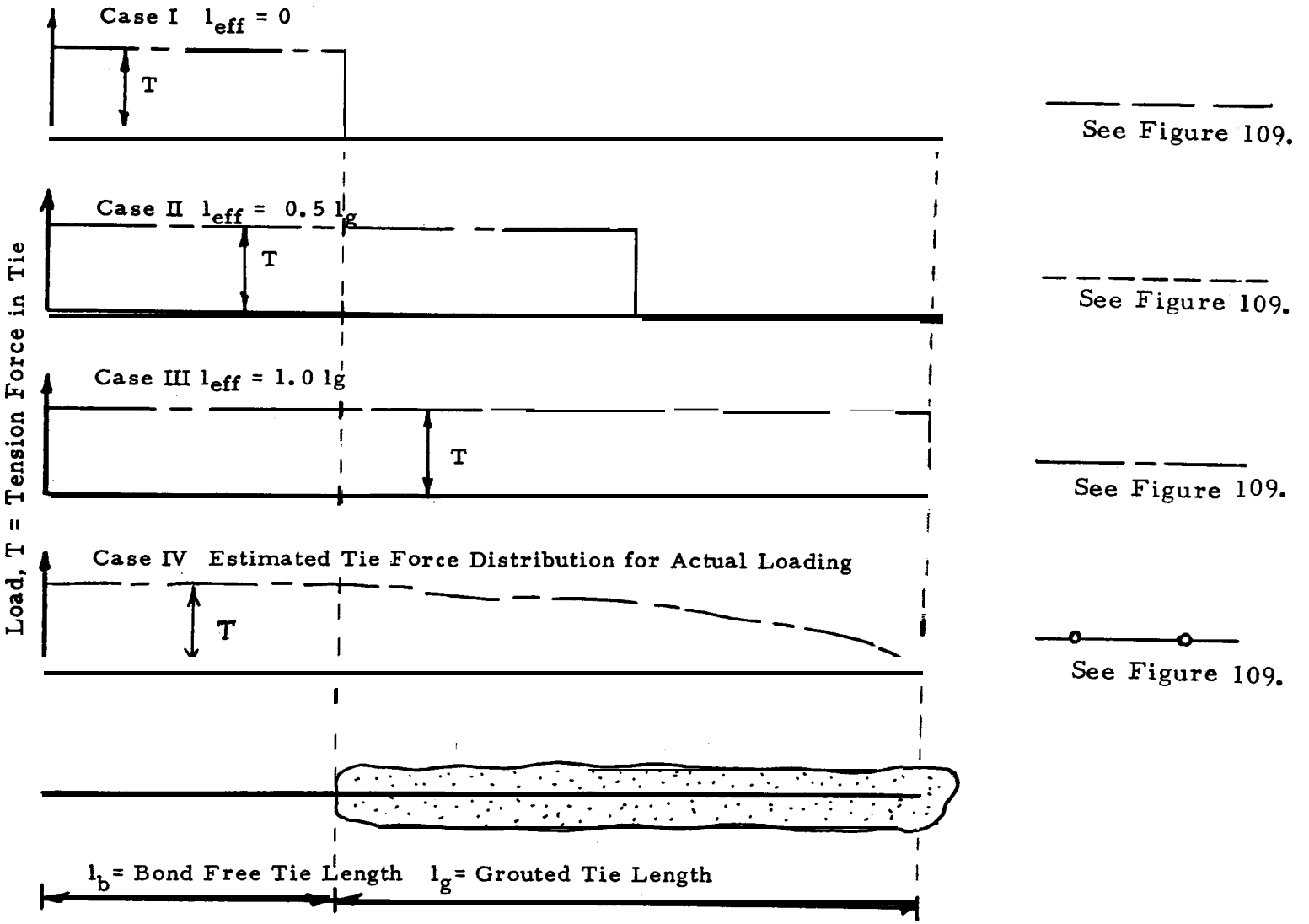


Figure 110. Idealized load distribution in tie.

13.43.4 Recommended Lockoff Loads

Earth Pressure	Lock-off % of design
For triangular active distributions	50 - 80%
For triangular at-rest distributions	100%
For trapezoidal and rectangular distributions	Upper ties - 80% Lower ties - 100%

13.43.5 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 creep rate 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 criterion 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 percent of the de sign load. As a matter of routine all permanent anchors should be tested to a minimum of 150 percent of the design load as opposed to the 125 percent of design load recommended for testing of temporary production anchors.

It is also recommended that selected anchors (5 percent) from a permanent anchor installation be retested at later periods after installation. The loads in these anchors should be checked to determine if the anchor'load is being maintained and if there is a possible dangerous buildup of load in the anchors.

CHAPTER 14 - UNDERPINNING

14.10 INTRODUCTION

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. In some cases underpinning elements may be installed on either side of the foundation, but in these cases cross beams or some other method is used to support the old foundation element.

A part of the underpinning process is to evaluate the existing structure for total footing loads, existing bearing pressures, soil conditions, ground water level, and possible excess foundation capacity. This investigation will determine the extent of the underpinning operation, the suitable underpinning techniques, and the constraints required to maintain structural integrity,

14.20 DESIGN AND THEORETICAL CONSIDERATIONS

14.21 Load Computation

14.21.1 Existing Structure

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.

14.21.2 Load Distribution

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.

14.22 Deformations

14.22.1 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. Tiebacks or braces may be used to proide the resistance needed to withstand horizontal forces. Vertical displacement below the bearing level obviously cuases settlement. Vertical displacements above the bearing level contribute to additional load on underpinning elements. This may also cause settlement,

14.22.2 Settlement from the Underpinning Installation

General sources of settlement are noted below:

a. Structural Elements. Increased loads in structural elements may cause elastic deformations. Non- elastic deformations may be caused by creep and shrinkage of the concrete used for underpinning,

b. Bearing Stratum, Settlements are caused by strain in the bearing stratum.

c. Construction Procedures. The two main sources of settlement during construction are loss of ground and the strains associated with load transfer (slack in plates and wedges, jacks, etc.).

14.23 Design of Underpinning Elements

14.23.1 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 in working with the various types of underpinning systems is absolutely e s sential.

14.23.2 Downdrag and Horizontal Forces

Underpinning elements are influenced by displacements occuring 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.

14.23.3 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 the group of piles will be larger than the settlement of a single pile.

The significance of group action depends upon a number of variables--the proximity of piles, the characteristics of bearing stratum, and the 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.

14.30 CLASSICAL UNDERPINNING PROCEDURES

14.31 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.

14.32 Pit or Pier Underpinning

14.32.1 General

Probably the most common method of underpinning is a concrete filled pit or pier which has 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 techniques used for access below the foundation form the basis for other underpinning procedures.

14.32.2 Procedure

The basic procedure for installing a concrete underpinning pier is to excavate an approach pit below the footing, advance the underpinning pit to the bearing stratum, and fill pit with concrete (see Figure 111).

14.32.3 Other Considerations

Load Tranfer

The space between the top of the pier and the

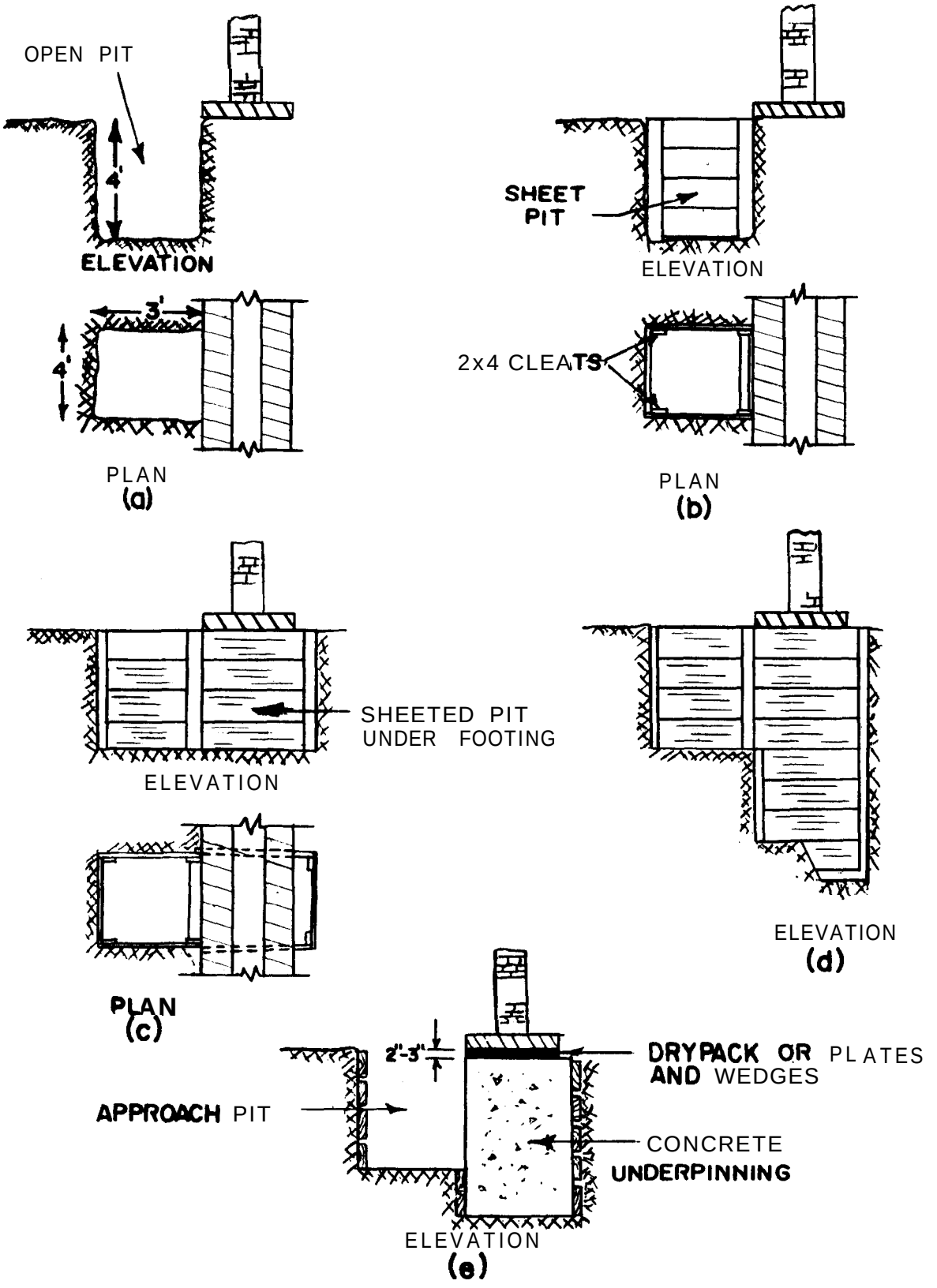


Figure 11 1. Pit or pier underpinning.

foundation is normally filled with dry-pack- -a mixture of cement and moist sand that is rammed in place. Plates and wedges may also be used.

Under certain circumstances the 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 and loads maintained on the jacks. This permits the concrete pits to settle while maintaining the structure at its original elevation.

Horizontal Wood Sheeting

The thickness of the sheeting (commonly 2 inches for wood sheeting) is essentially independent of depth as the stresses in the soil are distributed by arching. 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.

Belled Piers

If required, underpinning pits can be enlarged or belled at the bottom. There is a possibility for loss of ground if this operation is not performed carefully, especially in non-cohesive soils.

14.32.4 Sources of Settlement

Some causes of settlement are improper backpacking of sheeting, excessive deflection of the sheeting, and loss of ground. Pit excavations in weak soils or below the ground water level may cause significant movement of the adjacent soil. In general, fast excavation and concreting minimize movements of adjacent footings or slabs.

Weak soils, such as saturated silt or soft clay, tend to flow or squeeze into the pit excavations. Ground loss may occur during exposure of the soil face before lagging placement or after lagging placement by movement through open lagging or by movement intoan overcut zone behind the lagging.

Pit underpinning is generally limited to use in dry ground. If other underpinning techniques cannot be used, vertical wood or steel sheeting maybe required to maintain the sides of the pit. Settlement maybe caused by loss of ground behind sheeting, erosion of

soil through lagging, or an unbalanced hydrostatic head causing a "blow" at the bottom of the pit.

14.32.5 Examples

Figures 112 and 113 illustrate examples of pit unde rpinning .

14.3 3 Pile Underpinning

14.33.1 General

Generally, H-beams or steel pipe piles (both open-and close-ended) are used in underpinning. Open-ended pipe is usually preferred to close-ended pipe. 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.

14.33.2 Pile Installation

Jacked Piles

Typically, aluminum hydraulic jacks are used because they are light and easy to handle in a confined pit, Since the foundation is used as a reaction, the jacking load must be monitored to prevent an excessive upward force.

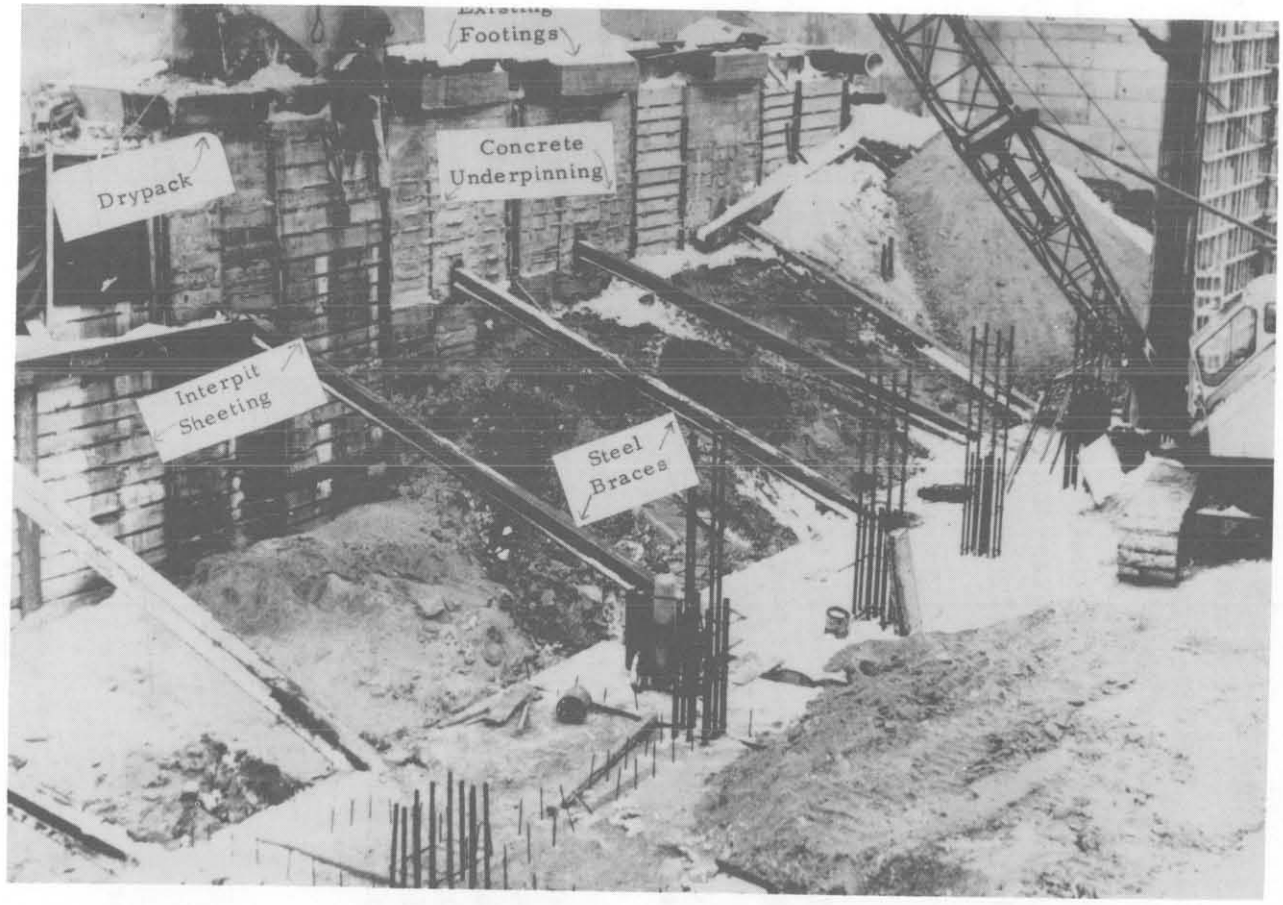
Except in soft material, jacking is done with open-ended pipe to permit removal of soil from within. In soft soils, a plug of cinders, sand, or lean concrete is formed within the pipe to permit advancement of the pipe by displacement.

The typical procedure is first to excavate an approach pit and then to jack the pile downward in approximately 5 foot long sections. When the required depth is reached, the pile is filled with concrete and test loaded to 150 percent of design load,

Load is transferred by inserting a wedging beam (e. g. I-section), plates, and wedges while maintaining the full



Figure 112. Details of pit underpinning.
(Courtesy of Spencer, White, and Prentis).



Note: Bracing for lateral support.

Figure 113. Pit underpinning. (Courtesy of Spencer, White, and Prentis).

load on the jacks. As a final step, the wedging beam and plates are encased in concrete (see Figure 114).

Driven Pile s

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 can be excavated beneath the footing.

Piles are driven in sections with splices made between successive lengths. Open-ended pipe may be cleaned out, if required, to reduce resistance.

Where installed below foundations, driven piles may be test loaded by jacking aginst the foundation, Load transfer is done in a fashion similar to that used for jacked piles.

Advancing Open-Ended Pipe

Side friction or end resistance is reduced during installation by periodically cleaning out the soil from within open-ended pipe. Sections of pipe are connected by tight fitting sleeves, generally fastened on the outside of the pipe to avoid interference during cleaning out. The sleeves are not normally welded and are designed to keep the sections of pile in alignment.

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 in the surrounding soils. While cleaning the piles and during driving, a positive hydrostatic pressure must be maintained to prevent a "blow" at the bottom.

14.33.3 Piles on Both Sides of Footing - Support with Beams

The basic procedure is shown in Figure 115. Main steps are:

a. Excavate to near bottom of footings and ‘install piles.

b. Sequentially install a series of beams below the footing by excavating a sheeted trench.

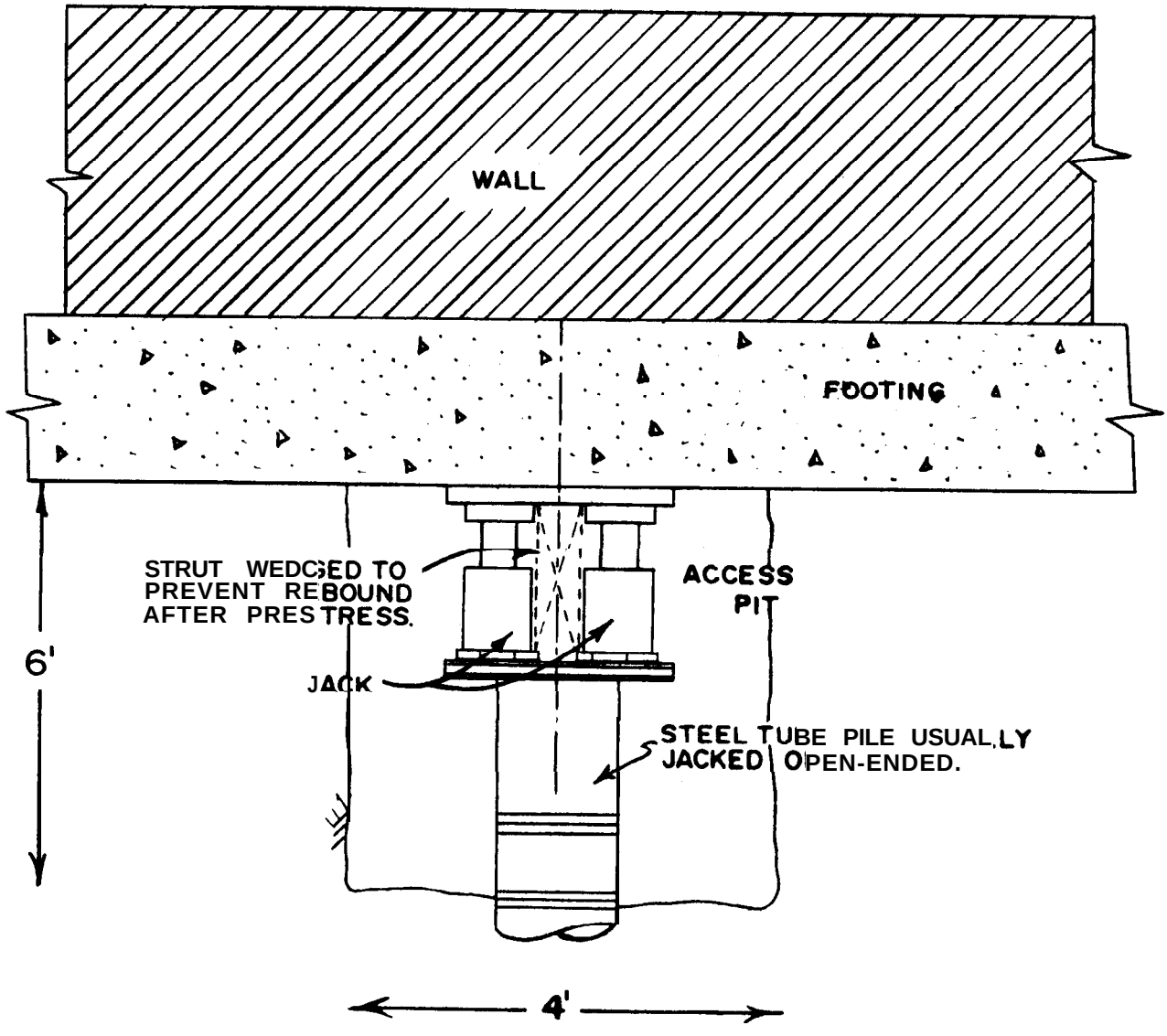


Figure 114. Jacked pile installation.

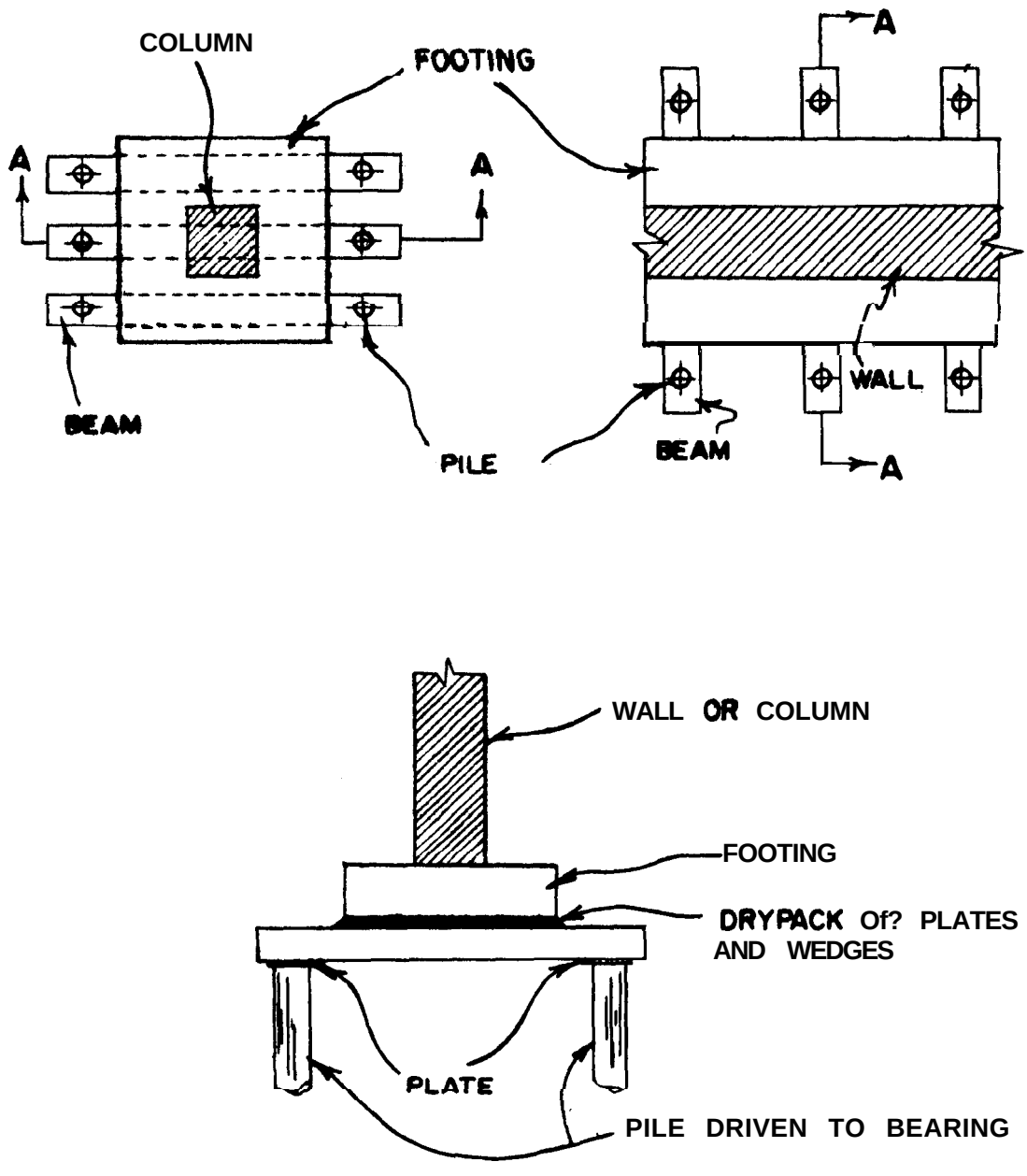


Figure 115. Piles driven alongside footing, support by beam,

Transfer load with drypack, plates and wedges, or jacks. This transfer of load can be made at either the bottom of the footing, the top of the pile, or a combination of the two.

14.33.4 Piles on One Side of Footing - Bracket Pile Underpinning

This method is normallyused for light structures, especially for exterior walls or continuous footings. Bearing is developed by a driven pile, usually an H-pile, or by a belled or straight-shaft caisson.

Driven Piles

The piles are usually installed alongside the footing and the load is carried by brackets welded to the pile, Plates and wedges or drypack are used to transfer load.

Pre-excavated Vertical Piles and Caissons

A steel beam or a concrete shaft may be used.

a. Steel Beam: The hole is filled with lean concrete and a bracket is welded on the steel beam similar to driven bracket pile underpinning (see Figure 116). An alternative procedure without brackets is shown in Figure 117.

b. Concrete Shaft: A hole is excavated under the footing, the necessary reinforcing steel is placed, and the pile and bracket are poured monolithically (see Figure 118).

Pre-excavated Battered Piles

This method, shown on Figure 119, 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 the hole to the bearing stratum. A vertical slot below the footing intersects the slant pile, and reinforcing ties the slot and pile together.

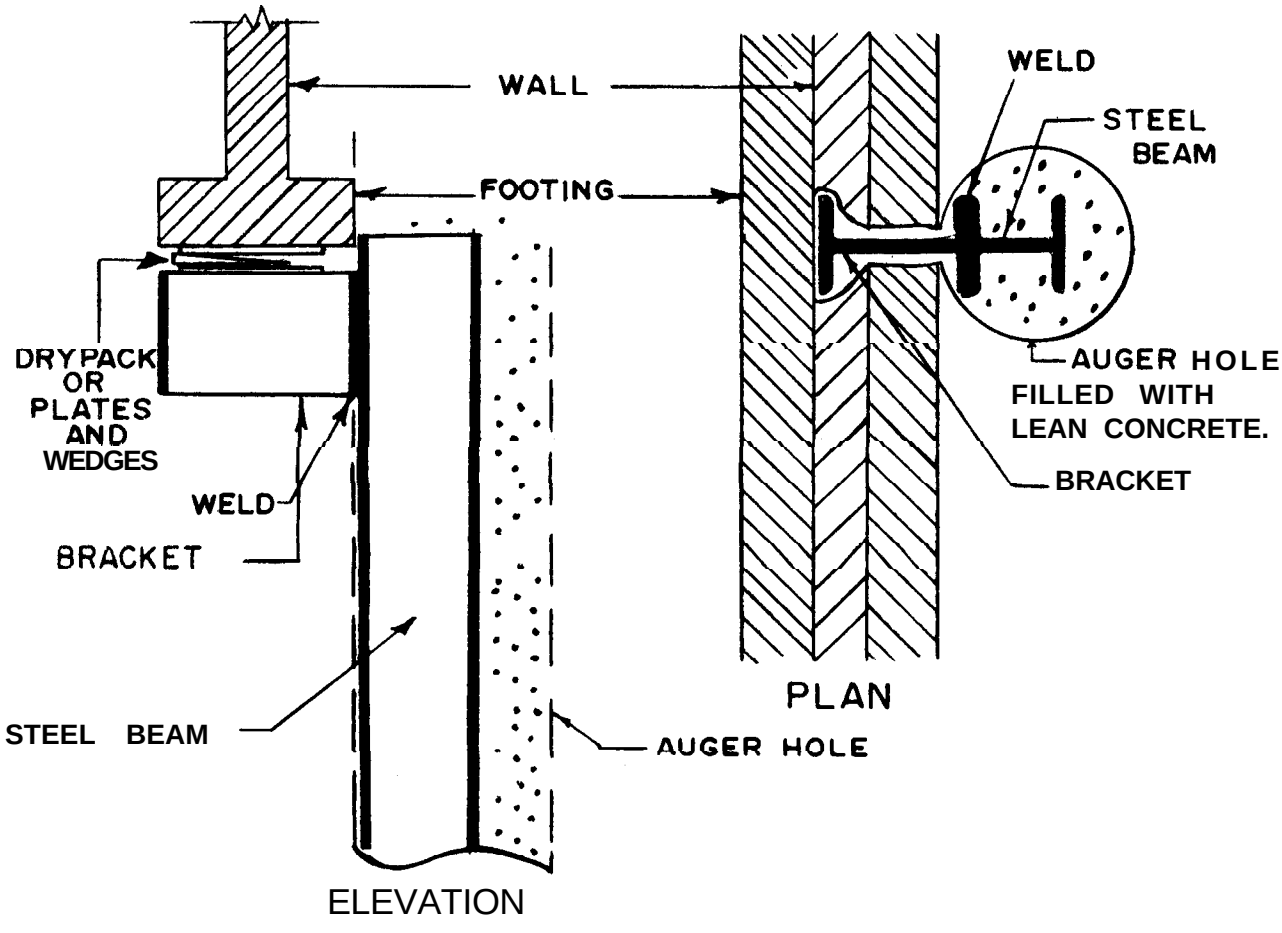


Figure 116. Steel pile with steel bracket.

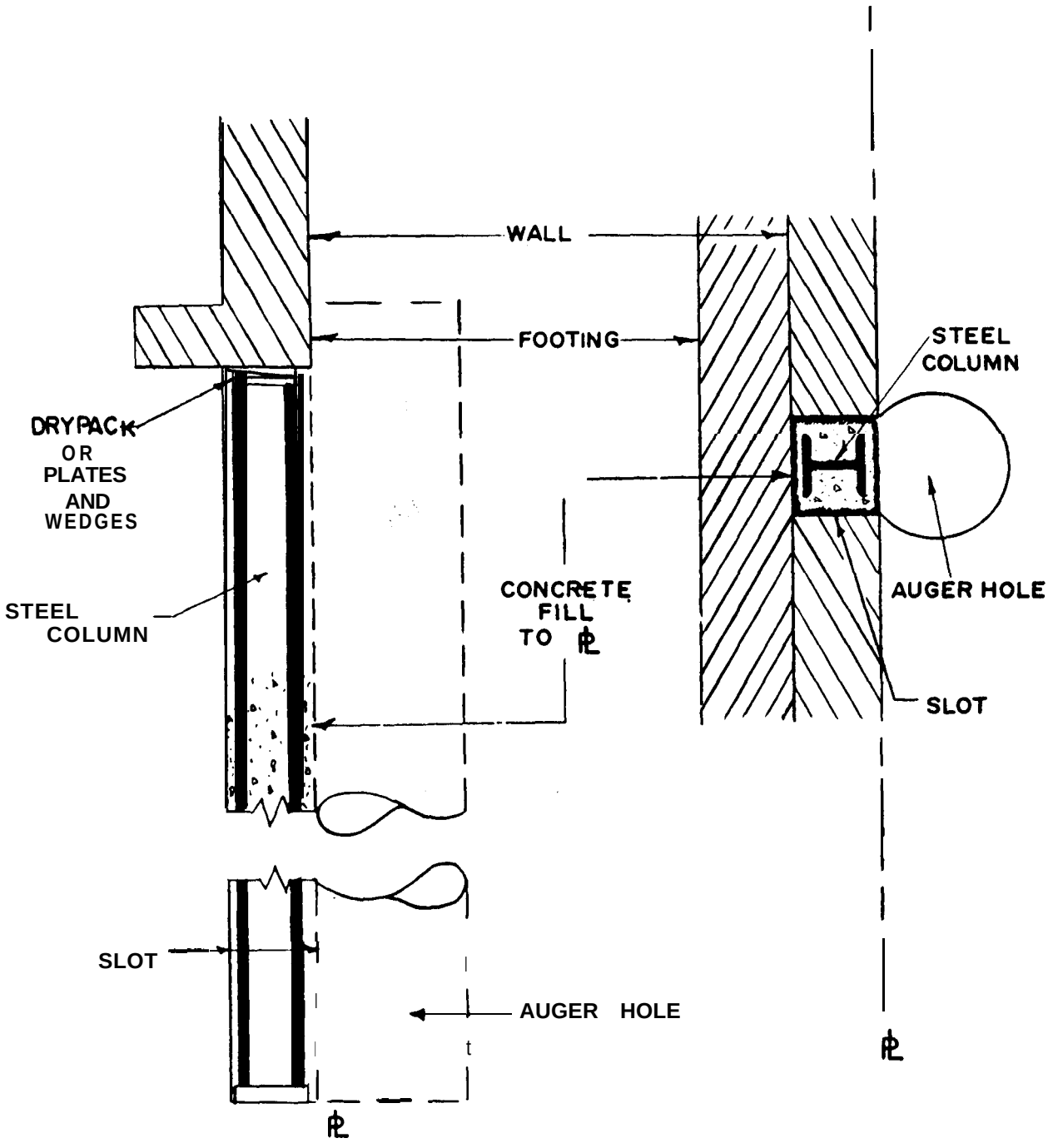


Figure 117. Auger hole with pile installed in slot.

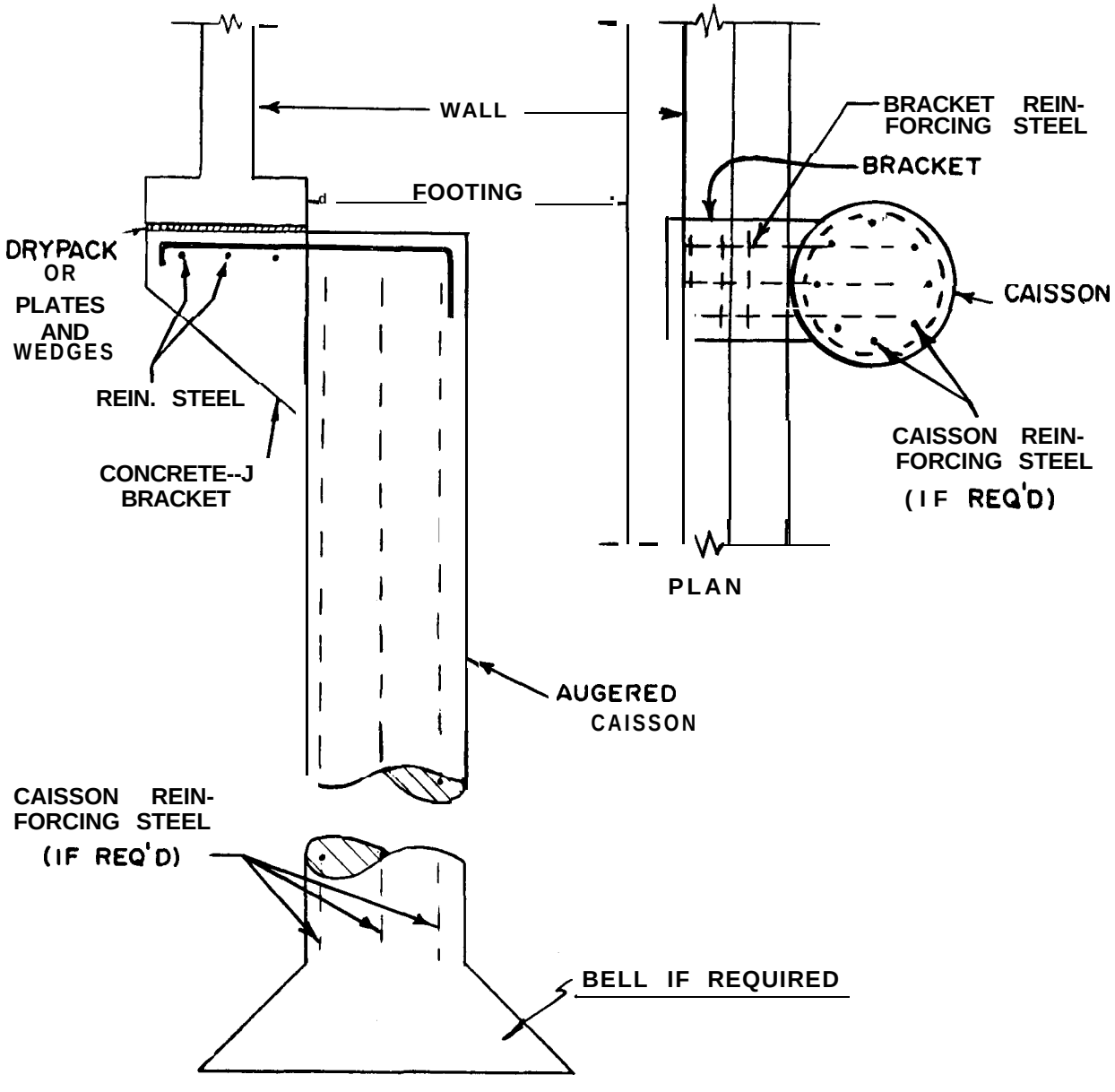


Figure 118. Augered concrete caisson with concrete bucket.

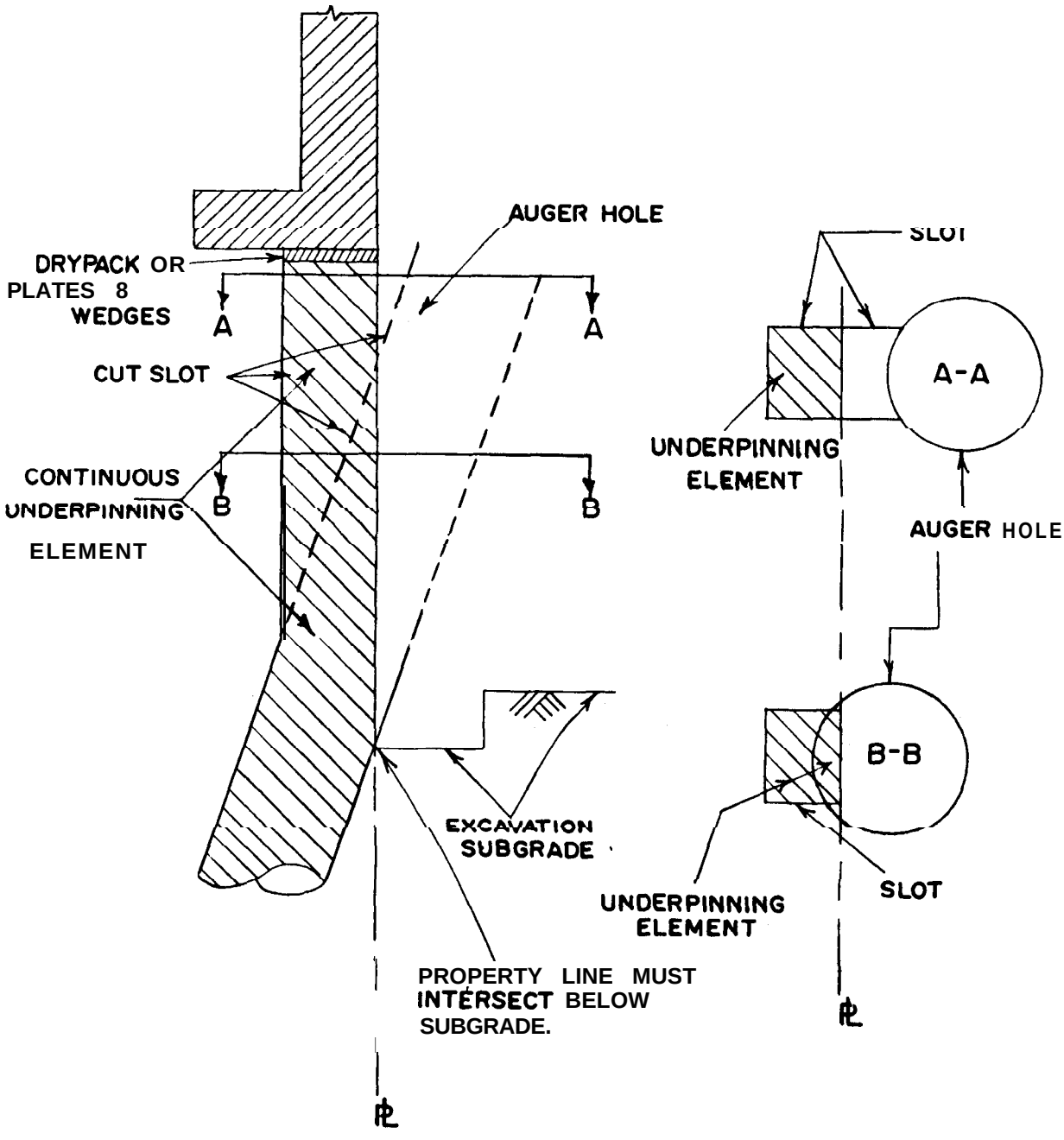


Figure 119. Battered pile underpinning.

14.40 GROUTED PILES

14.41 Hollow Stem Auger

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 of regular length and diameter. A reinforcing cage is placed into the wet grout. Typical sizes range from 12 inch to 16 inch diameter.

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.

14.42 Root Piles (Pali Radice)

14.42.1 General

This system is capable of providing vertical and/or lateral support to foundations and excavations (Bares, 1974) (see Figure 120). The piles range from 3-1/2 inches to 12 inches in diameter and are usually reinforced.

14.42.2 Root Pile Underpinning

Installation

When used for underpinning, root piles are 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). Installation in granular soils usually requires a casing throughout its entire length to prevent collaspe of the hole.

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 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

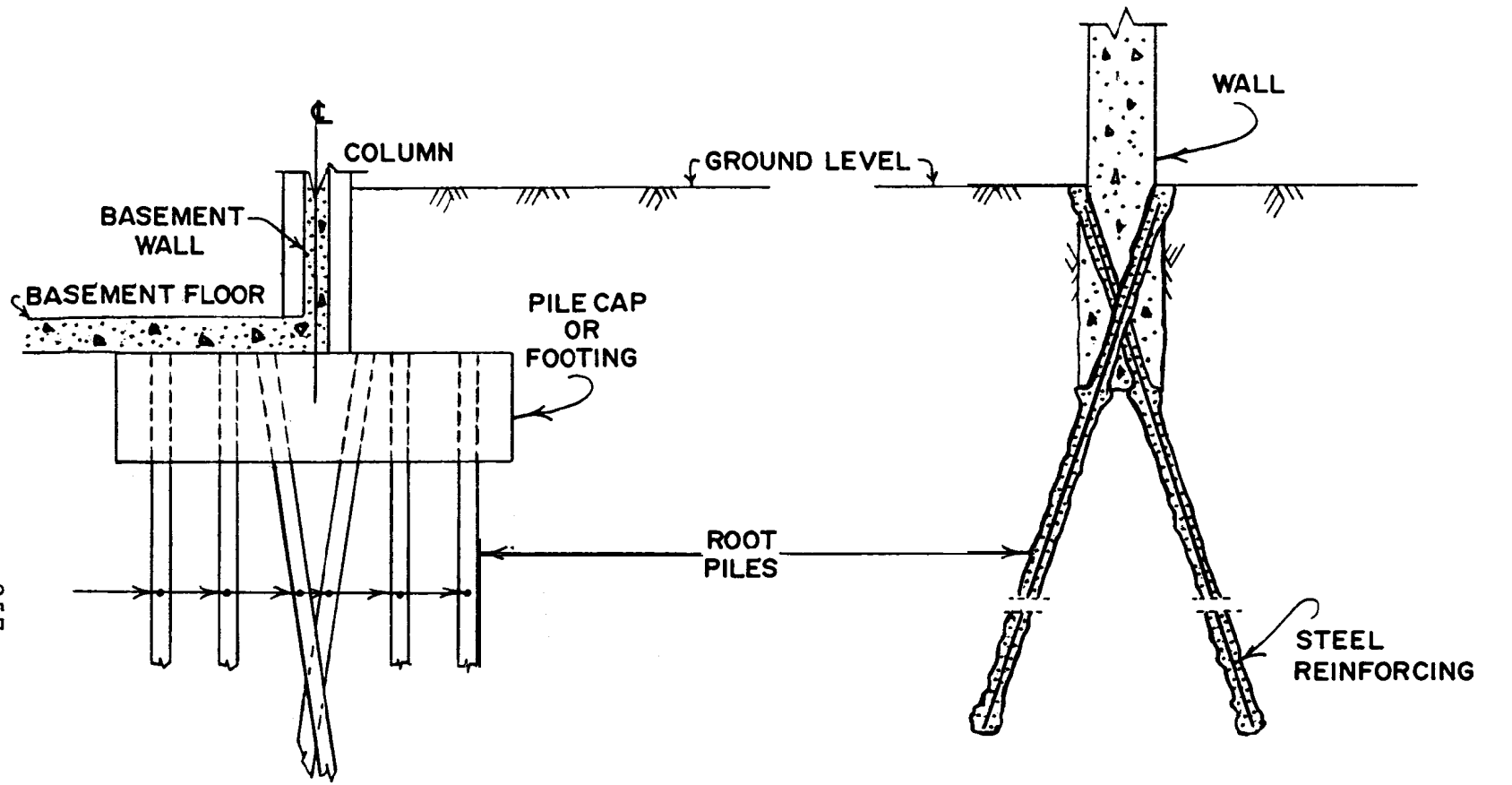


Figure 120. Typical uses of root piles (pali radice).

(generally 6 to 12 inches nominal diameter) are usually reinforced with a spiral cage. The steel is placed after concreting in the smaller pile and 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 15 summarizes the results of load tests on root piles obtained from published and unpublished data. In general, the tests were not carried to failure and therefore, the data do not permit an evaluation of safety factors. However, since settlement data were available it was possible to develop, at least in crude fashion, a relationship between pile geometry, load, and settlement.

14.42.3 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 these cases the underpinning piles carry vertical and lateral loads and resist soil displacement (Bares, 1974, 1975, and Lizzi, 1970).

The 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. See Figure 121 for an example.

14.50 TUNNELING BELOW STRUCTURES

14.51 General

This discussion concerns instances when tunnels pass below structures. In such instances, it is likely that vertical underpinning elements cannot be used directly below the foundations.

14.52 Column Jacking

Figure 122 illustrates a common technique used to support columns during below grade construction. The column is first isolated

Table 15. 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 Max. Load ρ, inches	Settlement Modulus (1) k, ton/in-ft^2	Soil Type (2)	Location
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 Railrod, 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 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 Gil 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).

Table 15. 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 Max. Load ρ, inches	Settlement Modulus (1) k, $\frac{in\text{-}ft^2}{ton}$	Soil Type (2)	Location
14**	a	66	66	58.7	0.037	0.0247	G	Railway Terminal, Naples (Corso A. Lucci)
15**	a	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**	a	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	0.7010	C	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
19**	a	66	66	48.5	0. 205	Q 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**	a	99	66	88.2	0. 127	0.0634	G	Belt (Expressway) East-West, Naples
22**	a	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) $k = \frac{\rho}{P/L'D} = \frac{L'D}{P}$

(2) G = Granular; C= Clay; Si = SiH

** Bares. F.A. (1974)

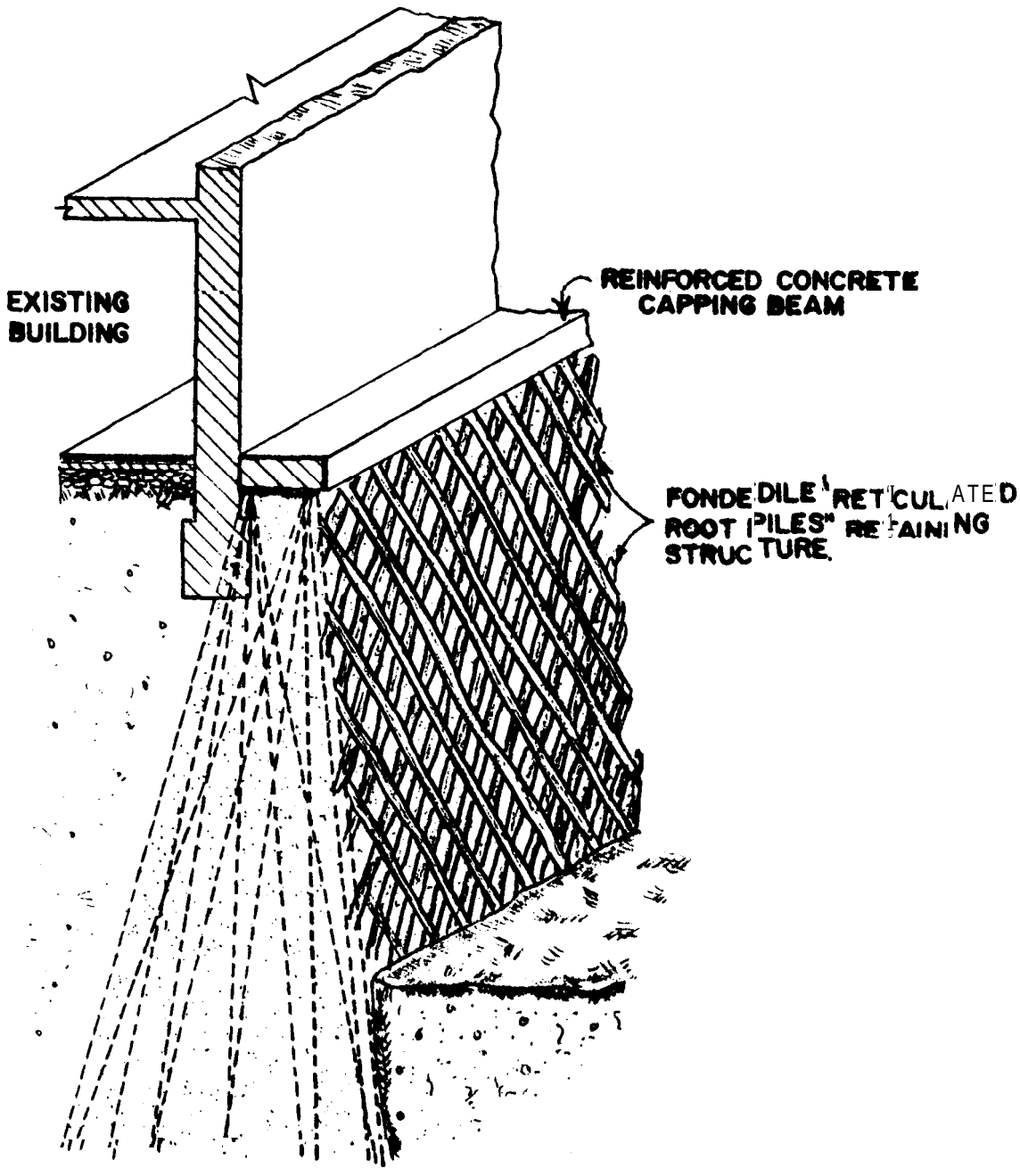
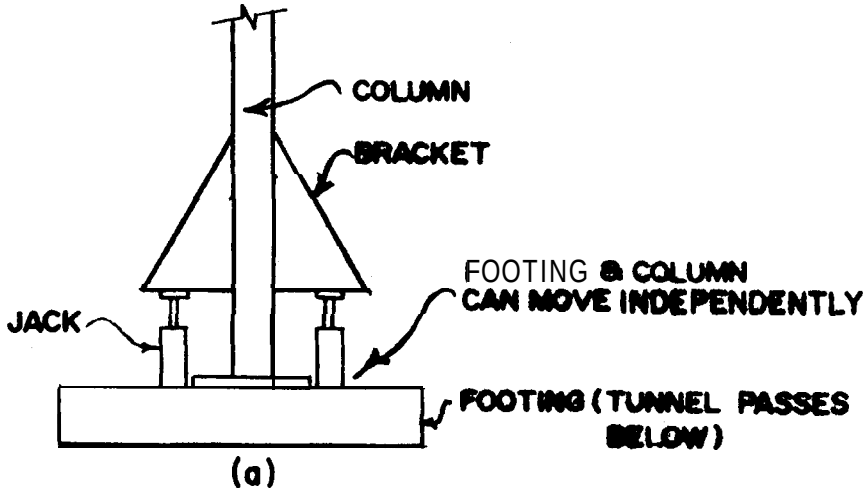
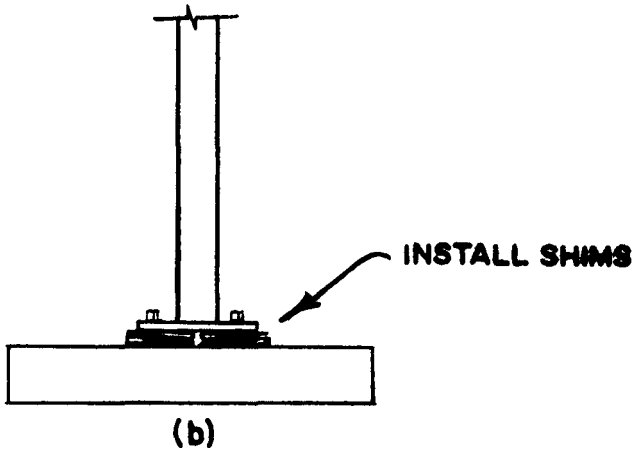


Figure 12 1. Schematic showing principle of reticulated root piles. (Courtesy of Warren-Fondedile, Inc.).



BRACKETS INSTALLED, ANCHOR BOLTS
LOOSENED, JACKS INSTALLED.



SHIMS INSTALLED, ANCHOR BOLTS
TIGHTENED.

Figure 122. Schematic of column jacking to prevent structure settlement during tunnel construction.

from the footing and is maintained in place by jacks, The footing can settle while the column remains in place. After construction the column is reconnected to the footing.

14.53 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 tunnel. Typically, the contiguous tunnels, called pipe shields, are installed by jacking pipe from an open cut jacking pit or from the side of a primary drift tunnel if this is not possible.

Figures 123a and 123b illustrate examples where jacking pits were excavated from the surface.

14.54 Inclined Secant Piles

Inclined secant piles in lieu of underpinning are applicable where there is a slight encroachment below utilities or structures (see Figure 124).

14.55 Bridging

Figure 125 schematically isllustrates measures that can be taken to bridge across the tunnel area.

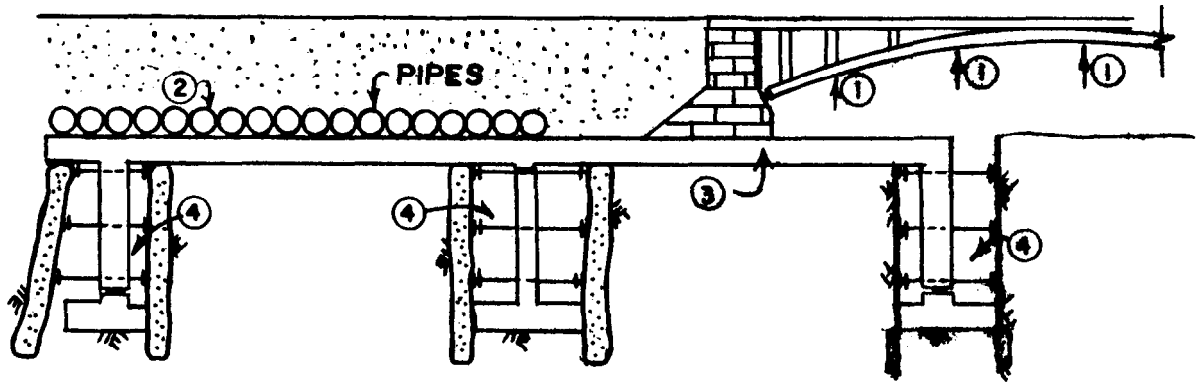
14.60 LOAD TRANSFER

The transferring of the load from the old foundation or temporary shoring to the new underpinning element is similar for all underpinning methods. Sources of potential settlement are compression of the under - pinning member, displacement of the bearing stratum, and compression. of plates and wedges or dry pack.

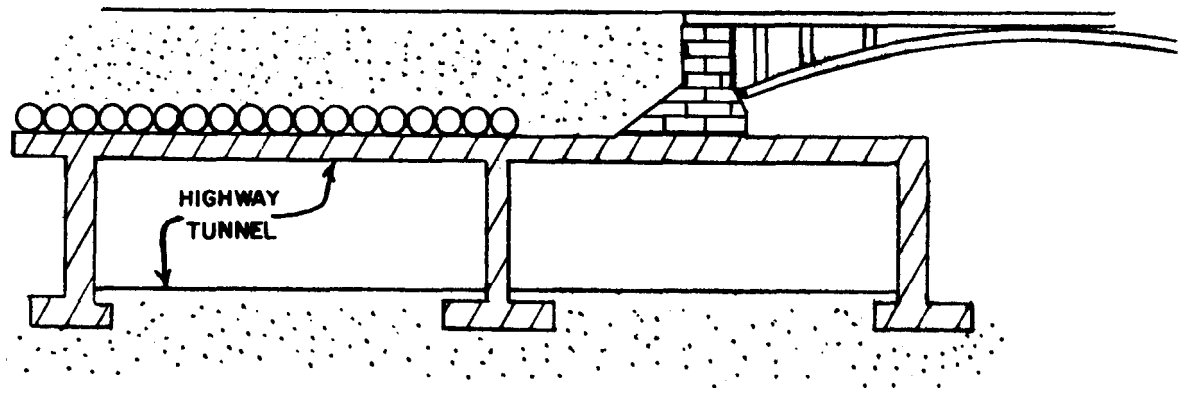
14.61 Dry Pack Alone

The use of dry pack alone is generally limited to pit underpinning. Preloading techniques may not be required because stresses and deformations are relatively small. 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.

(a) CONSTRUCTION



(b) AFTER COMPLETION



Sequence

1. Underpin bridge with steel piles and jacks to rdjurt for settlement.
2. Construct jacking pits on each side of highway, jack 1. 2 m pipes and conc rete pipes.
3. Construct 3 m wide x 2 m high tunnels below pipes. Concrete eac h tunnel before building next one.
4. Construct walls of highway tunnel.

Figure 123a. Pipe shield technique (after Zimmerman, 1969).

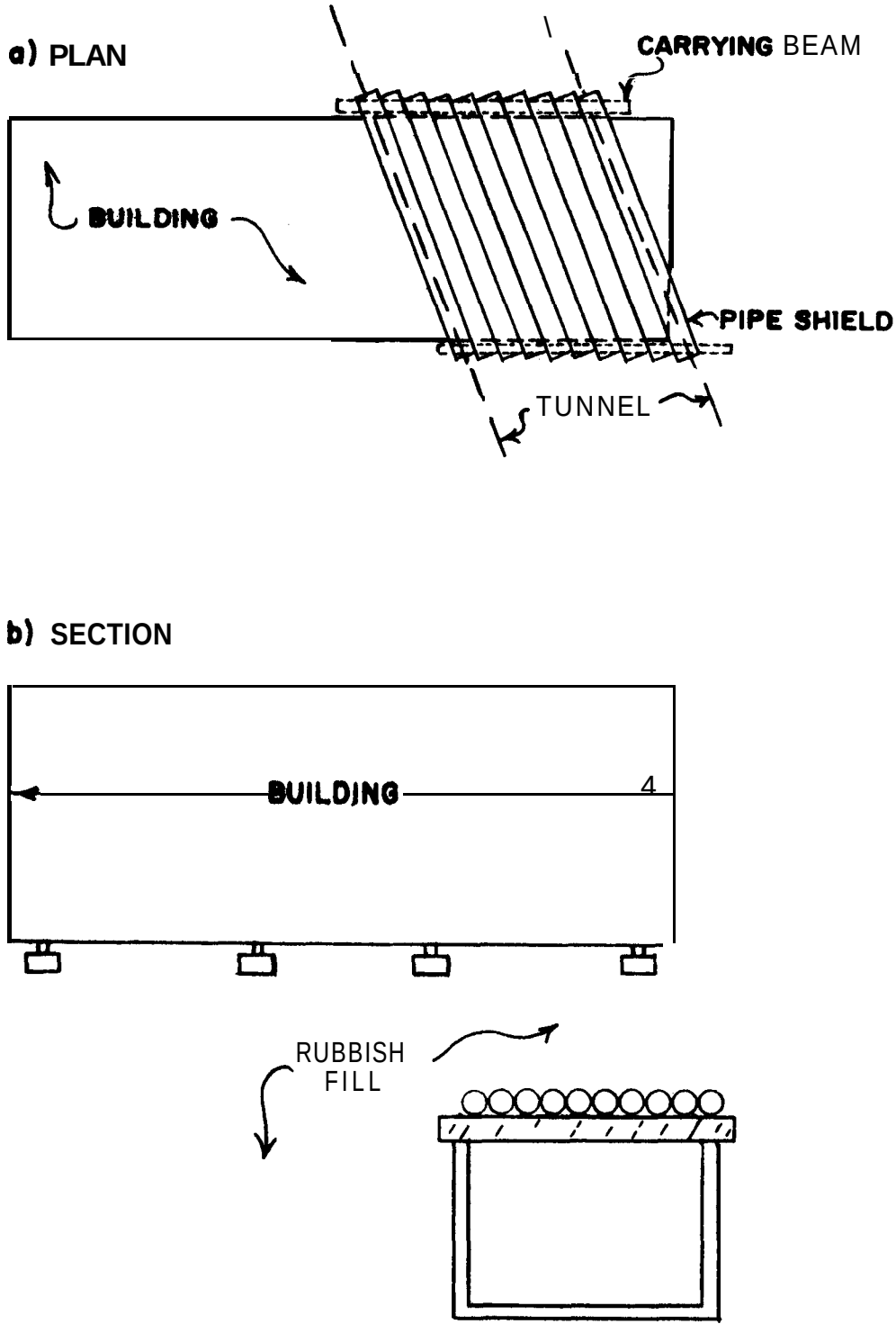


Figure 123b. Pipe shield technique (after Rappert, 1970).

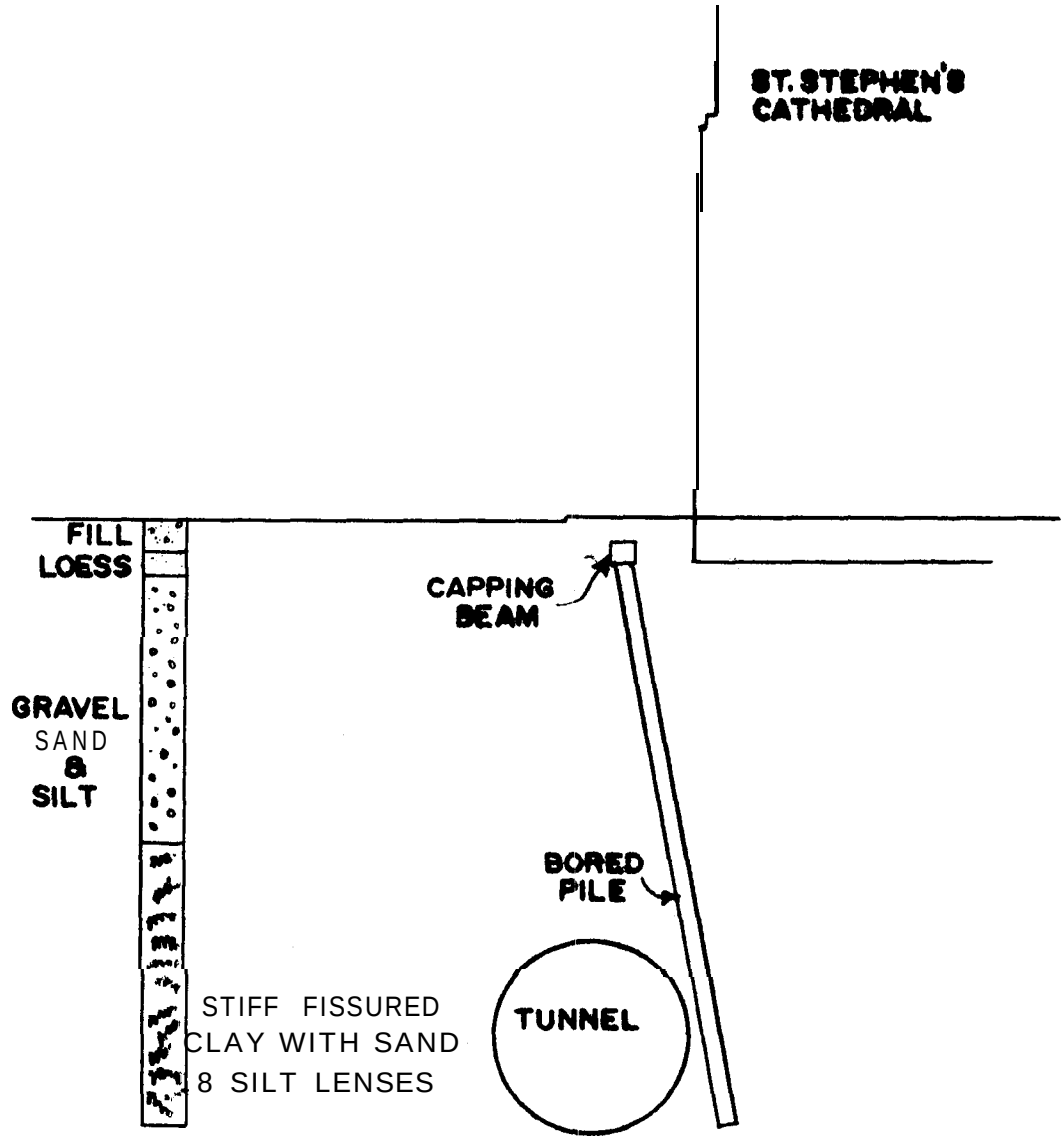


Figure 124. Example of bored pile wall used to protect structure (after Braun, 1974).

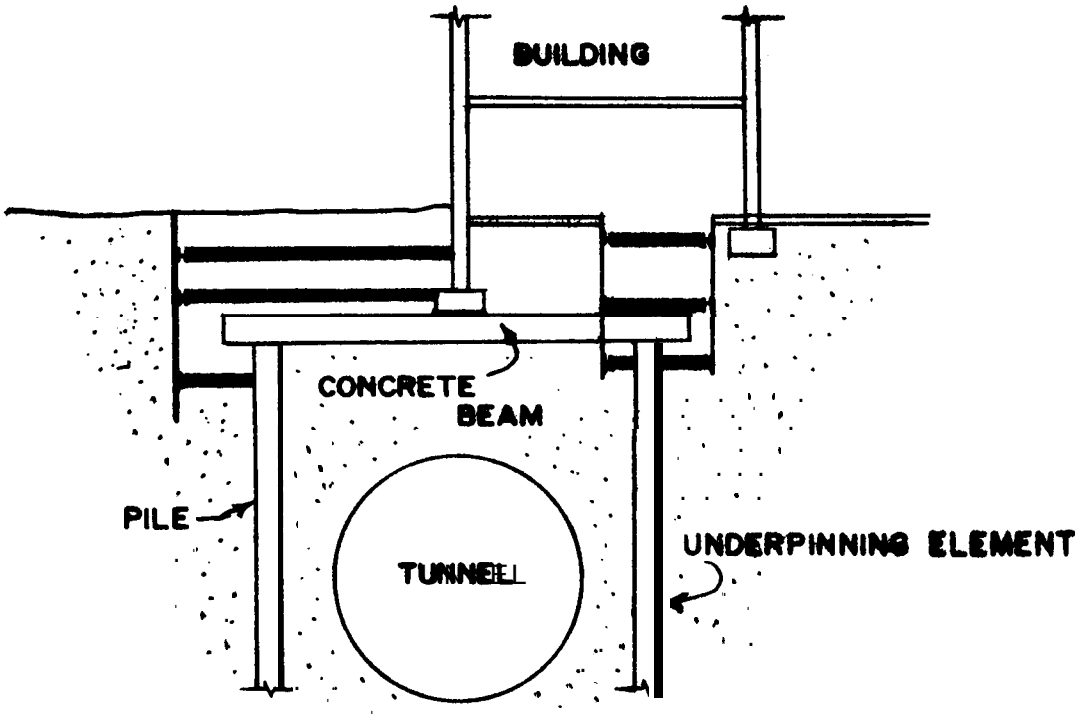


Figure 125. Bridging.

14.62 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.

14.63 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. Where creep is minimal, the load can be transferred immediately by a steel or concrete plug or dry packed. The jacks are then removed. Where there is concern over settlement, the Load can be maintained and periodically adjusted as needed.

14.70 TEMPORARY SUPPORT OR "SHORING"

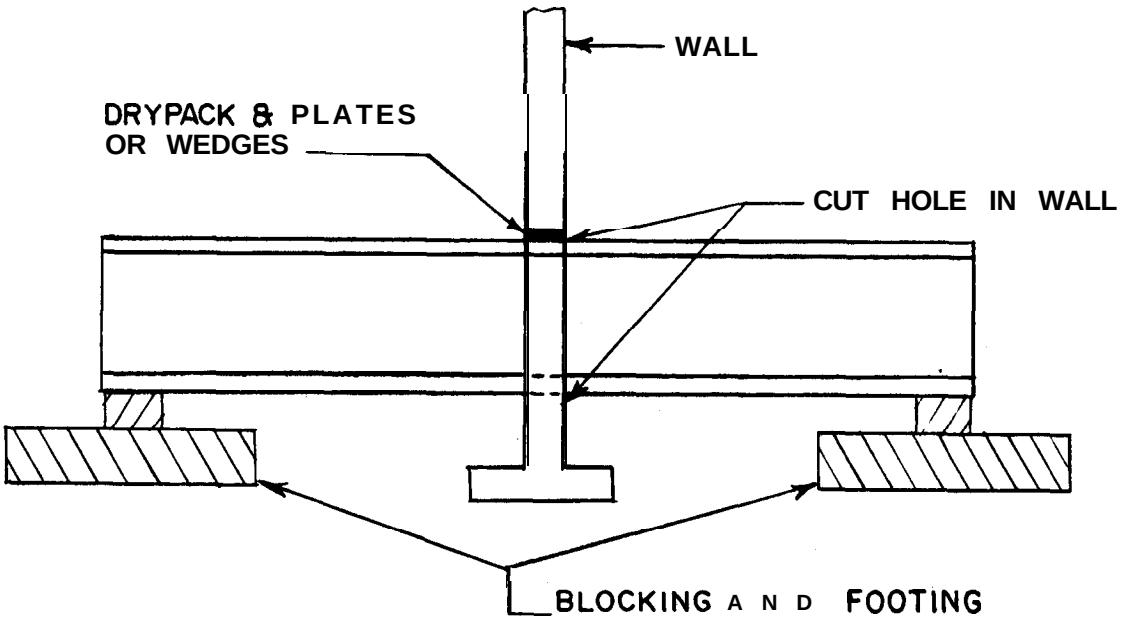
14.71 Basic Considerations

The need for temporary support during underpinning is controlled by the integrity of the structure being underpinned, the effect of a temporary bearing pressure increase adjacent to the underpinning operation, and the degree of foundation undermining.

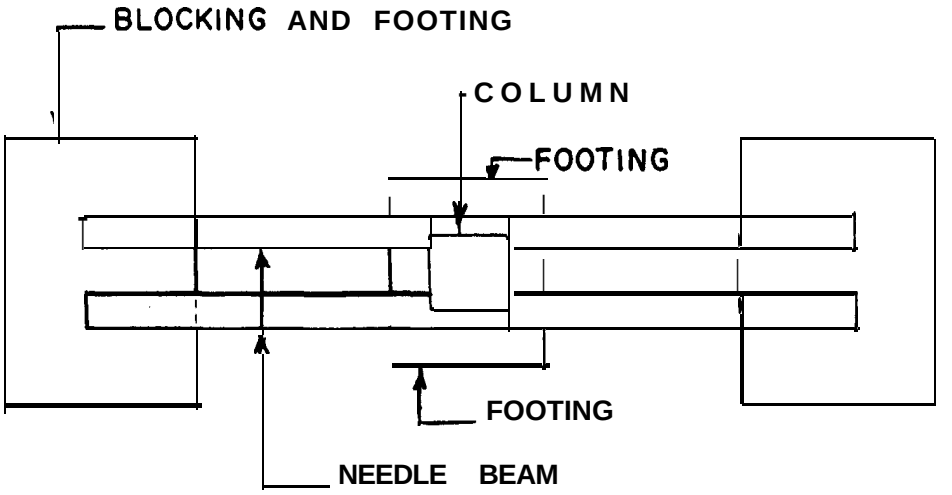
Usually it is very difficult and often impossible to predict the loads which the shores will carry. Movements of the shored element should be monitored throughout construction. The shoring can be jacked or wedged to compensate for settlement, if and when it occurs.

14.72 Needle Beams

Typical "needle beam" configurations are shown in Figure 126. More elaborate systems might consist of concrete pads and steel needles with jacks at the support points to control the movement of the structure.



(a) NEEDLE BEAM THROUGH WALL
(SIDE VIEW)



(b) NEEDLE BEAMS SUPPORTING A COLUMN
(PLAN VIEW)

Figure 126. Needle beam detail.

14.73 Inclined Shoring

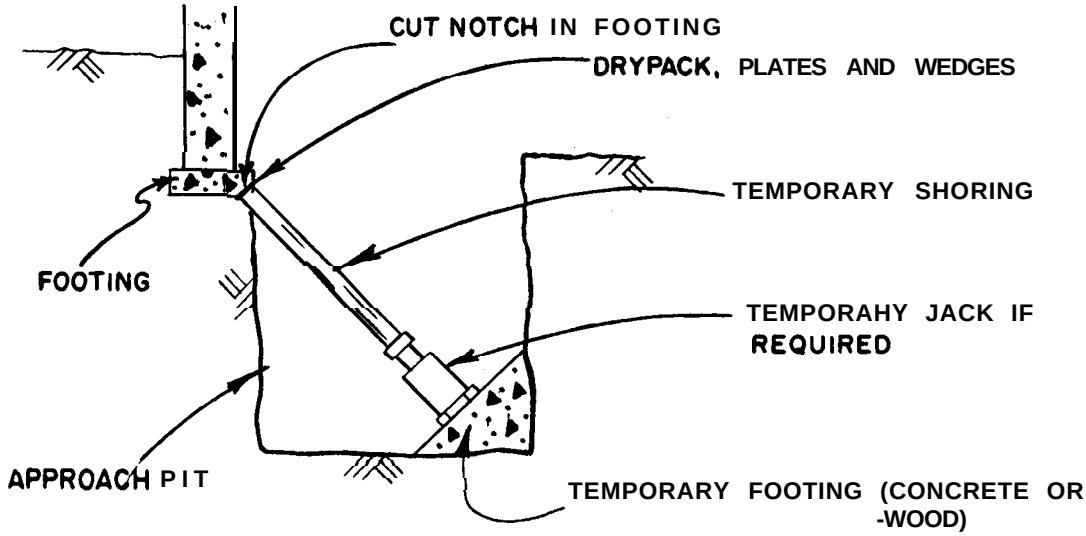
Some typical configurations are presented in Figure 127. In all cases, the lateral loads transmitted through the shores must be accounted for. Some common details of shoring connections are shown in Figure 128 and 129.

When cast iron columns are encountered, special attention must be given to prevent damage to the column. Often it is necessary to fill or encase the cast iron column with concrete. The pin and clamp method is presented in Figure 130. The shoring of cast iron columns might also be accomplished using a concrete collar placed over either a roughened surface or using welded shear connections on the column. Regardless of the method, eccentric loadings should be avoided.

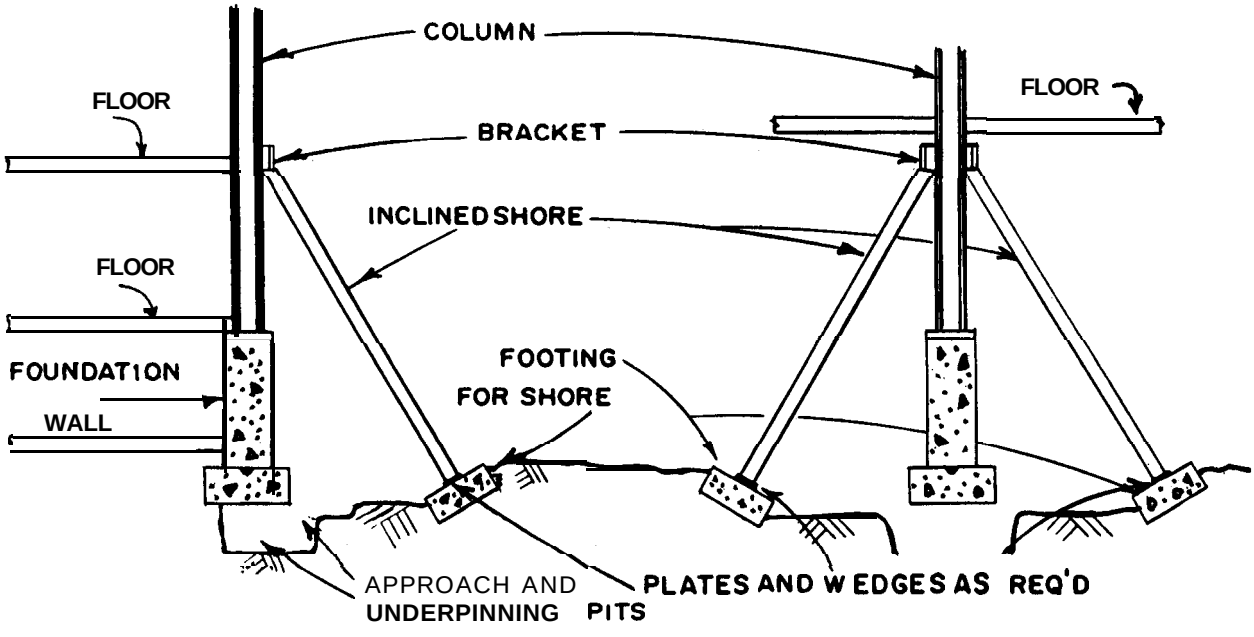
Figure 131 illustrates a case where inclined shoring was used to protect a structure.

14.80 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. Ware (1974) presents settlement and lateral movement data for underpinned structures in the Washington, D. C. area.

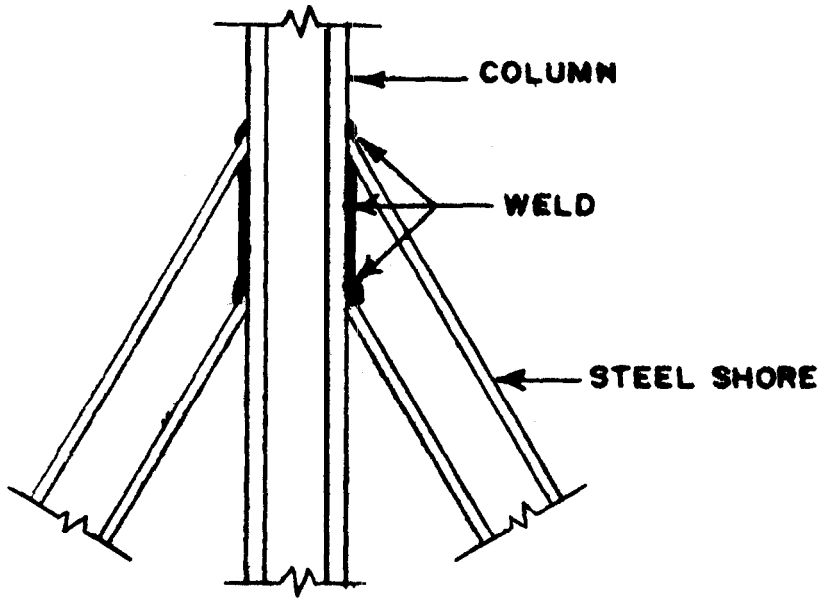


(a) INCLINED SHORING UNDER A FOOTING

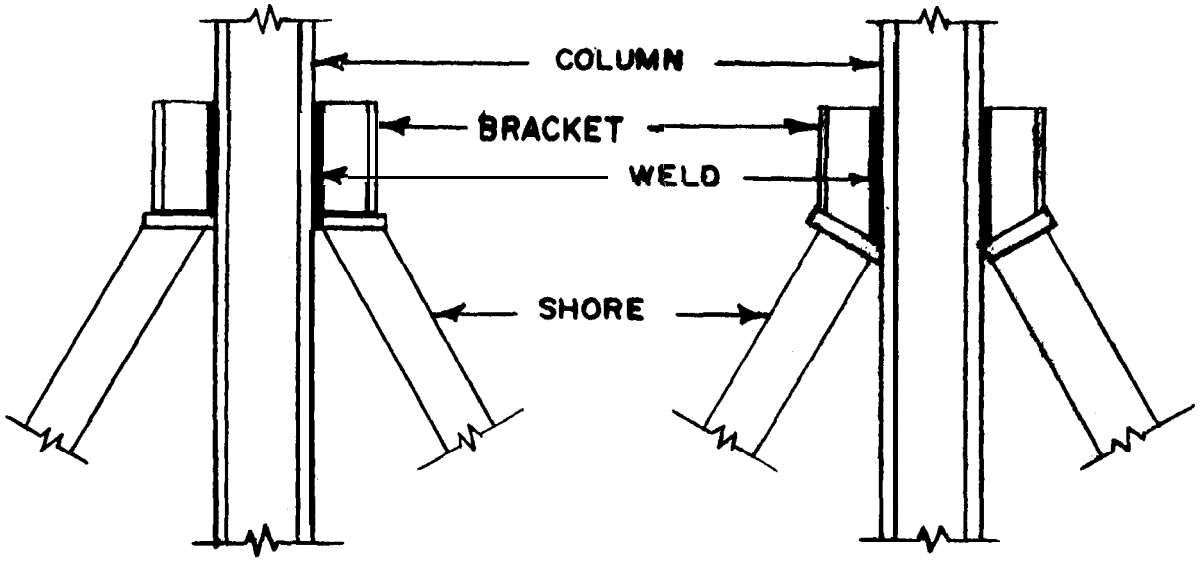


(b) SHORING A WALL OR COLUMN

Figure 127. Inclined shoring details.



(a) WELD STEEL SHORES DIRECTLY TO COLUMN



(b) SHORE TO BRACKETS WELDED ON COLUMN

Figure 128. Shoring details, steel column.

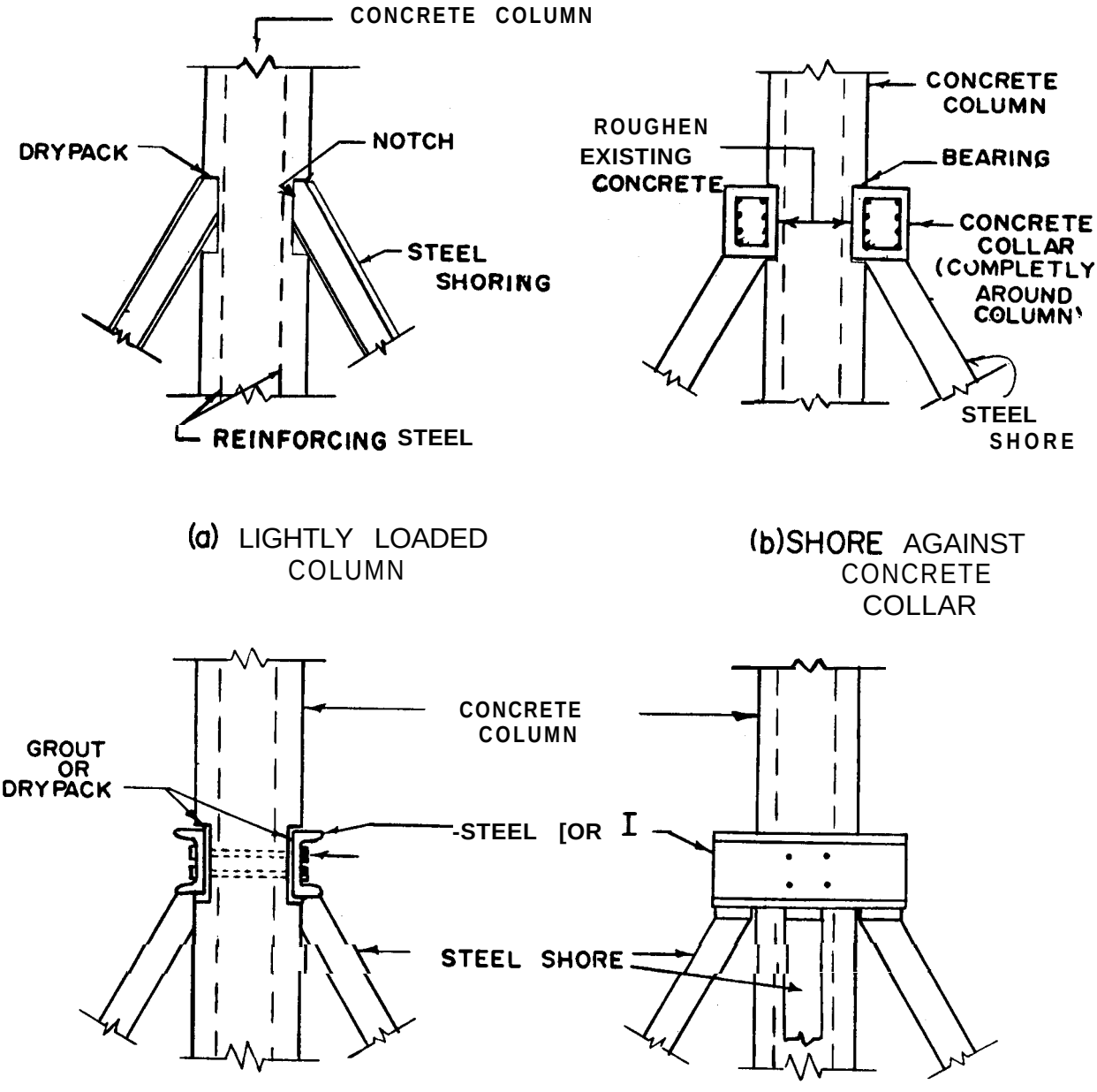


Figure 129. Shoring detail, concrete column.

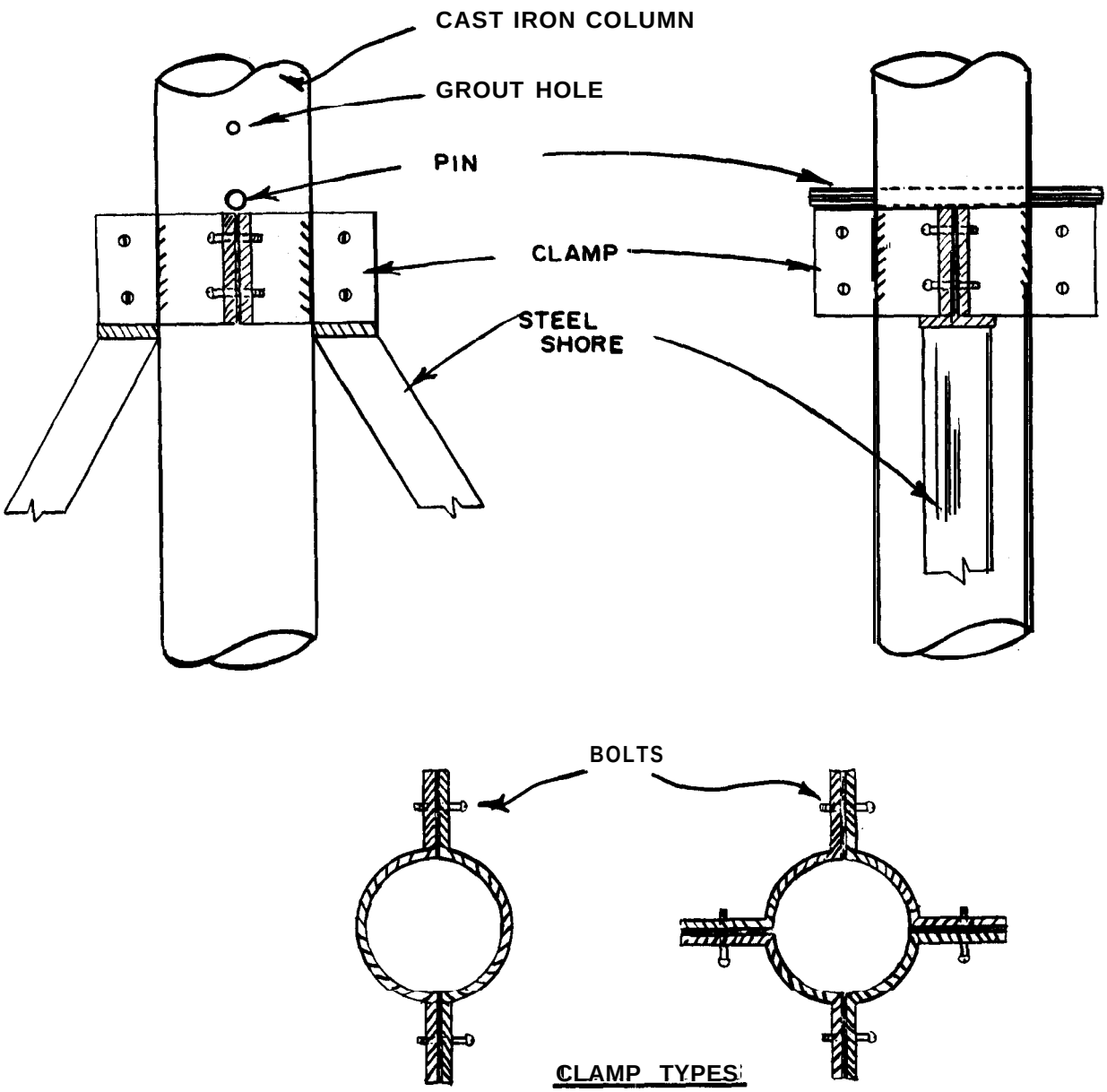


Figure 130. Pin and clamp details for a cast iron column.



Figure 131. Shoring of concrete columns.
(Courtesy of Spencer, White, and Prentis).

CHAPTER 15 - GROUTING

15.10 INTRODUCTION

Since 1802 when Charles Bérigny initiated the practice of grouting; the techniques, procedures, grouts, and applications of grouting have increased and improved. Today, grouting is used to stabilize soils, provide ground water cutoffs, and underpin structures. Any one or all three of these features of grouting may be required during cut -and- cover or soft ground tunneling in urban areas.

Injection grouting of a porous soil or rock mass may be done with particulate or chemical grouts. The principal advantage of injection grouting is that the engineering properities of a soil mass can be substantially improved with little or no disturbance to existing structures. Grouting can also be performed in locations where access limitations inhibit the use of conventional construction techniques.

The design and construction information on grouting presented in this report is provided to aid the engineer and/or contractor in deciding whether grouting is a feasible construction technique for the project being considered. Final design of a grouting system could not be performed on the basis of the information presented herein. A detailed design and construction manual on grouting is being prepared by Halliburton Services and will be available through the National Technical Information Service.

15.20 DESIGN AND THEORETICAL CONSIDERATIONS

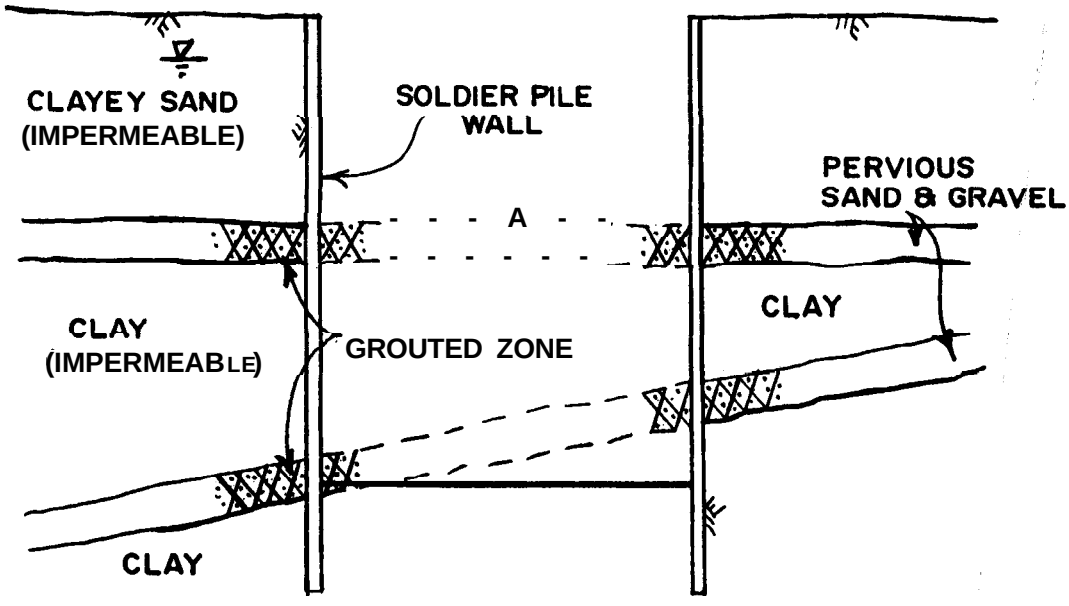
15.21 Purpose

Grouting can be used to control ground water, to solidify or stabilize a soil mass, or underpin an existing structure. For a given project, grouting may be used to achieve one or all three of these purposes.

Grouts injected into a soil mass reduce the permeability of the deposit. A ground water cutoff, either vertical or horizontal (Büttner, 1973), can be formed to replace or to supplement other dewatering s chemes. Selective grouting of specific strata may also be performed. Figure 132 illustrates several situations in which grouting techniques could be used to control ground water.

Grouting can significantly improve the strength and deformation characteristics of the soil mass. Strength grouting can be used to prevent large deformation behind lateral support walls, to

a) PERVIOUS LAYERS WITHIN DEPTH OF CUT



b) DEEP PERVIOUS DEPOSITS

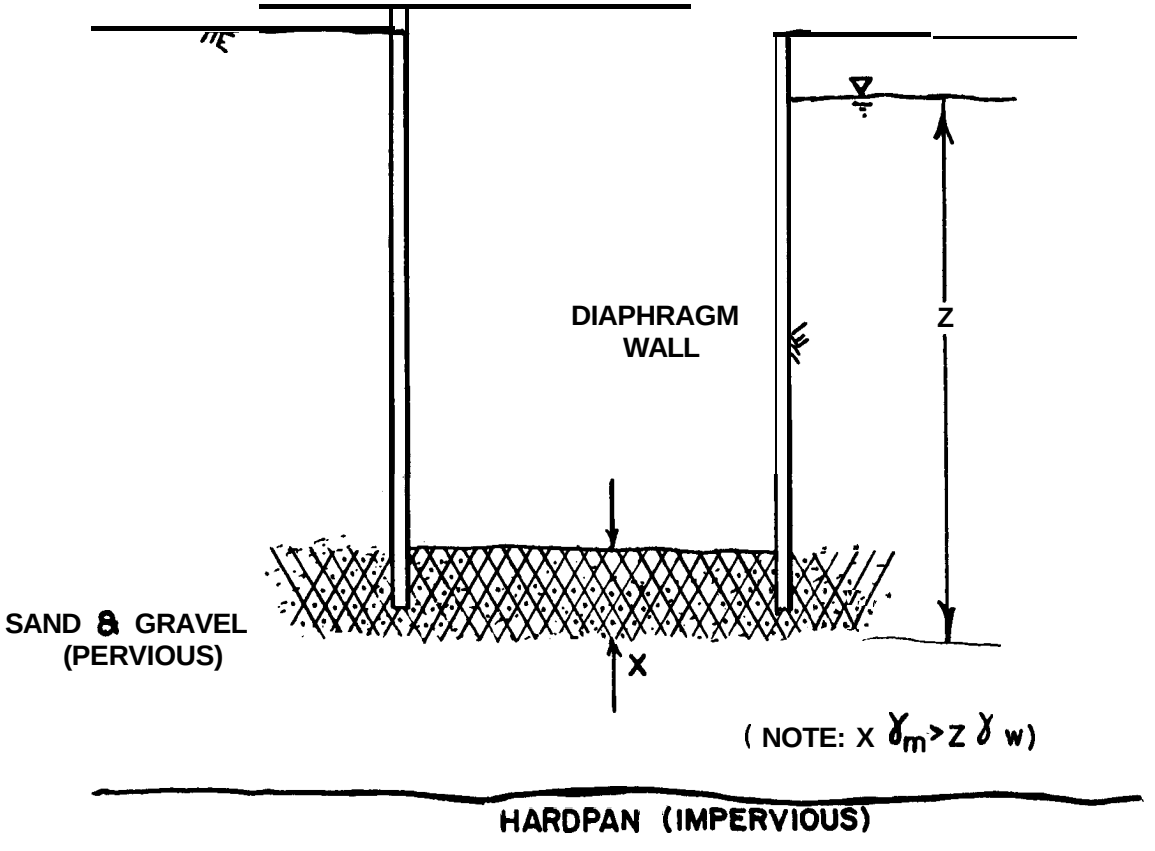


Figure 132. Grouting for ground water control.

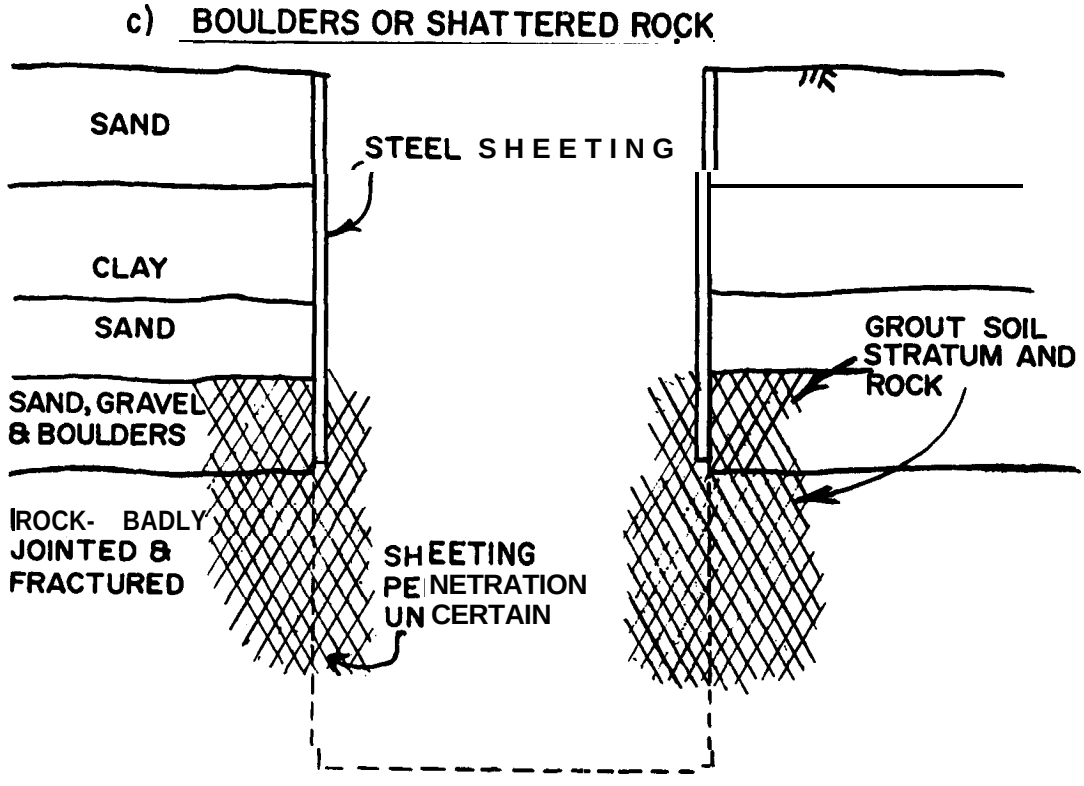


Figure 132c. Grouting for ground water control.

prevent running of soils, or protect overlying structures during bored tunnel construction. Figure 133 illustrates several examples of soil stabilization through grouting.

Grouting for underpinning is a special application of grouting for soil solidification. This technique may be particularly valuable if the grouted mass can also be used to control ground water or act as a lateral support wall. Figure 134 illustrates a case where grouting could be used to underpin a structure.

15.22 Soil Profile and Soil Type

15.22. 1 Field Investigations

Field investigations undertaken for a proposed grouting scheme fall into two phases. The first investigation phase would involve obtaining an accurate definition of the soil profile. This would include careful mapping of the depth and extent of strata.

The second phase includes obtaining more data pertaining to the specific soil properties controlling groutability, including field permeability tests and soil sampling or laboratory testing. The in situ soil permeability can be determined from borehole permeability tests or pumping tests. Pumping tests are preferred because they provide more reliable values of permeability.

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.

15.22.2 Laboratory Investigations

Laboratory testing will be limited primarily to detailed Logging to map stratigraphy, grain size analyses, and Laboratory permeability tests. The stratigraphy may determine the appropriate grouting methods and procedures. Grouting uniform soil deposits may be much different than grouting highly stratified deposits.

Grain size analyses may determine whether the deposit can be grouted. Although soils with greater than 10 percent by weight pas sing the No. 200 sieve can be grouted, it is generally very expensive to do so. Coarse silt deposits can also be grouted but more extensive analyses of the gradation and particle nature is required. Halliburton (1976) describes the more rigorous grouting Limit criteria.

Laboratory determination of permeability is of limited usefulness since the tests are generally performed on reconstituted

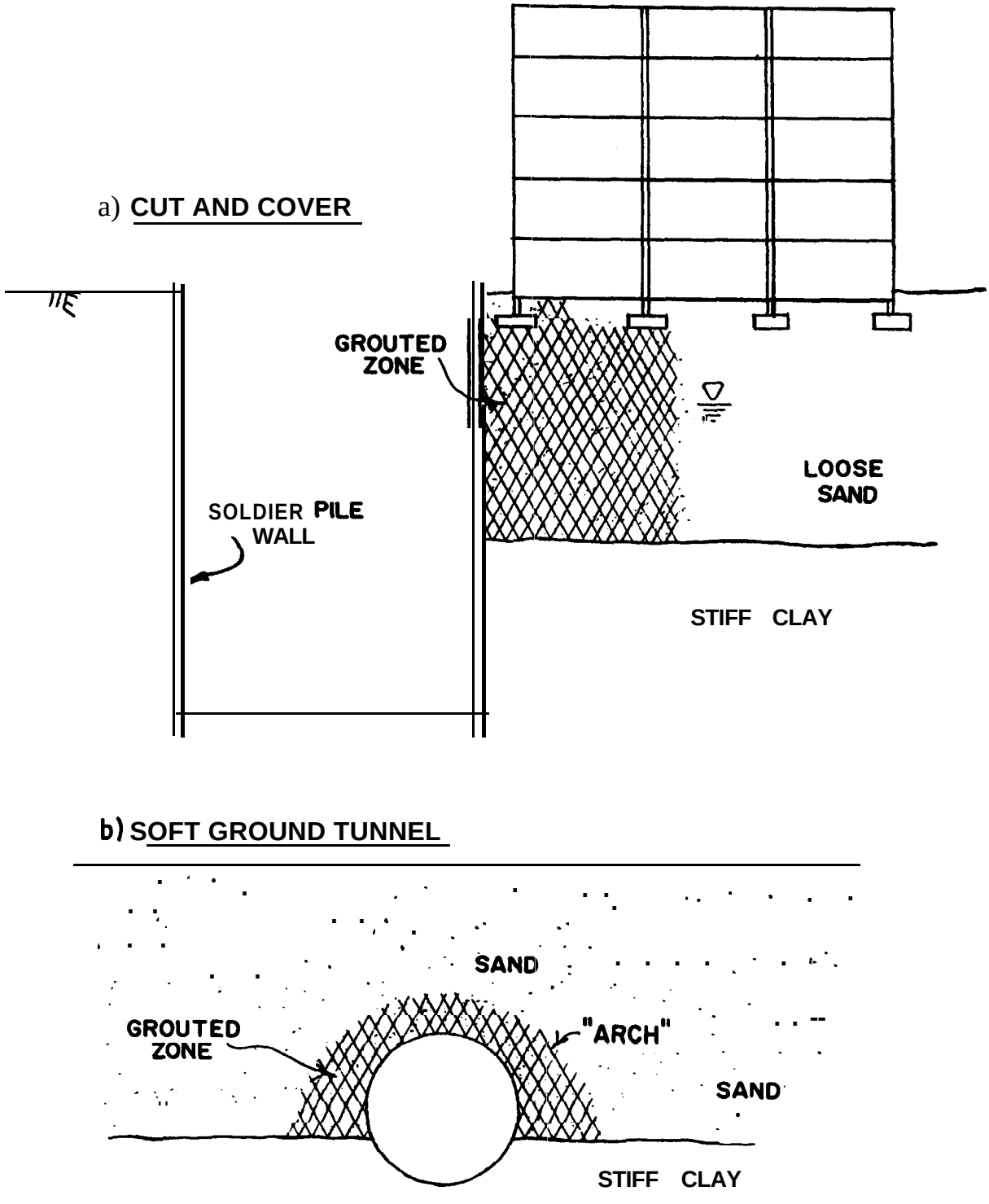


Figure 133. Grouting for soil solidification.

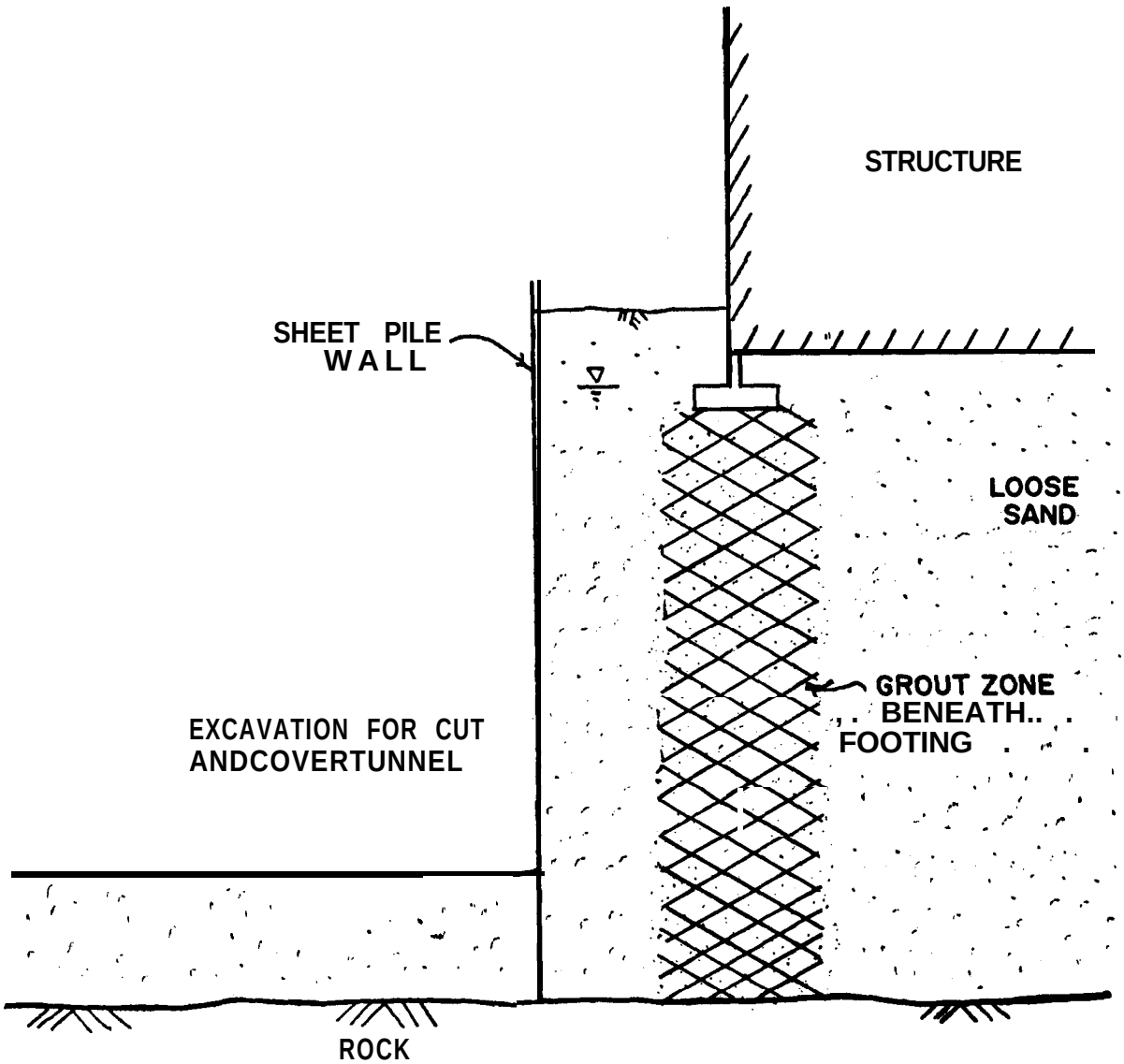


Figure 134. Example of grouting used to underpin an existing structure.

soil samples. Therefore, laboratory and field permeabilities may differ considerably. An assessment of all parameters --grain size distribution, stratigraphy, laboratory permeability tests--provides a basis for judging whether a soil deposit can be successfully grouted and what grouts are likely to be most efficient.

15.23 Grout Type

15.23. 1 Particulate Grouts

Particulate grouts are fluids that consist of a suspension of solid particles --such as cement, clay, a processed clay Like bentonite, or a mixture of these elements. The groutability, or the ability of a grout to penetrate, 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% size of soil to the 85% size of the particulate grout. For successful grouting the ratio should exceed 25.

$$\text{Groutability ratio} = \frac{D_{15}\,(\text{soil})}{D_{85}\,(\text{grout})} \geq 25$$

In practice, normal cement based grouts are limited in use to coarse sands while a pure bentonitic grout might be injected into a medium sand.

15.23.2 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 salt to form a silicate gel and salt. Chromelignosulfates also fall into the general category of metathetical precipitation type grouts.

Polymers are generally more fluid than the metathetical precipitation grouts and use a process by which 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). Table 16 summarizes the basic grout types and lists some of the common grouts according'to these general groupings .

Unlike particulate grouts that are injected as suspensions in a fluid, chemical grouts are injected as true solutions.

Table 16. Classification of common grout types (from Massachusetts Institute of Technology, 1974).

Particulate Grouts	Cement Clay Bentonite	Suspensions
Chemical Grouts		
Precipitation	Silicate Chemicals Aluminate Chemicals Chromelignosulfates	
Polymers	Acrylamides (e.g. AM9) Phenoplasts or Aminoplasts (e.g. recorcineformol, urea-formol)	Injected in form of monomers
	Epoxy Polyester-resins	Injected partially polymerized

Chemical grouts are therefore idealized to behave as Newtonian fluids of characteristic viscosity. Viscosity of the grout, together with the permeability of the soil and the injection pressure will control the groutability. E. Maag in 1938 (Ischy and Glossop, 1962) developed a simplified model of the behavior of a Newtonian fluid:

$$t = \frac{\alpha n}{3khr_o}(R^3 - r_o^3)$$

where:

R = radius of grout at time (t)

r_0 = 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 with respect to depth below water, and injection occurring above impermeable boundaries. In view of the many unknowns inherent in any soil mass, however, 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 are required.

15.24 Design Factors

The final grouting design is performed by a grouting specialist; however, the engineer and/or contractor should be aware of the features that influence grout selection and design. In some cases the grouting procedure may involve several injections with grouts of decreasing viscosity to achieve the desired product (most commonly done in Europe).

Grout selection must include evaluation of required soil strength and permeability as well as grout gel time, setting characteristics, volume of grout, and penetration. Tables 17 and 18 and

Table 17. 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	$d_{10} > 0.5$mm	$0.02 < d_{10} < 0.5$mm	$d_{10} < 0.02$mm
	Specific surface	$s < 100\ cm^{-1}$	$100\ cm^{-1} < s < 1000\ cm^{-1}$	$s > 1000\ cm^{-1}$
	Permeability	$k > 10^{-3}$ m/s	$10^{-3} > k > 10^{-5}$ m/s	$k < 10^{-5}$ m/s
Series of Mix		Bingham Suspensions	Colloid Solutions (Gels)	Pure solutions (Resins)
Consolidation Grouting		Cement ($k > 10^{-2}$ 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 18. 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 135 provide guidelines for grout selection.

Layout of grout injection pipes depends upon soil types, grout viscosity, injection pressure, and gel time. Spacing will depend upon grout penetration and the desired grouted soil properties. In Europe, less expensive grouts (coarser grouts) are often used as a first step in grouting to fill the largest voids and reduce the need for less viscous, but more expensive grouts. However, the labor costs of switching from more viscous to less viscous grouts may exceed the savings in materials. Using less viscous grouts for all grouting may be cheaper than using the sequential grouting system.

15.30 CONSTRUCTION CONSIDERATIONS

15.31 Materials

15.31.1 Particulate Grouts

Cement grouts are used primarily to increase strength but also have the added benefit of lowering permeability. 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 and processed (bentonite) clays can be used as grouts primarily to reduce permeability. The properties of natural clays must be carefully examined to determine their suitability for use. It is common to mix clays with cement to form the final grout.

15.31.2 Chemical Grouts

The basic divisions of chemical grouts are by their respective chemical processes, inorganic (metathetical precipitation) and organic (polymerization). Table 19 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 lower strength and greater penetration.

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.

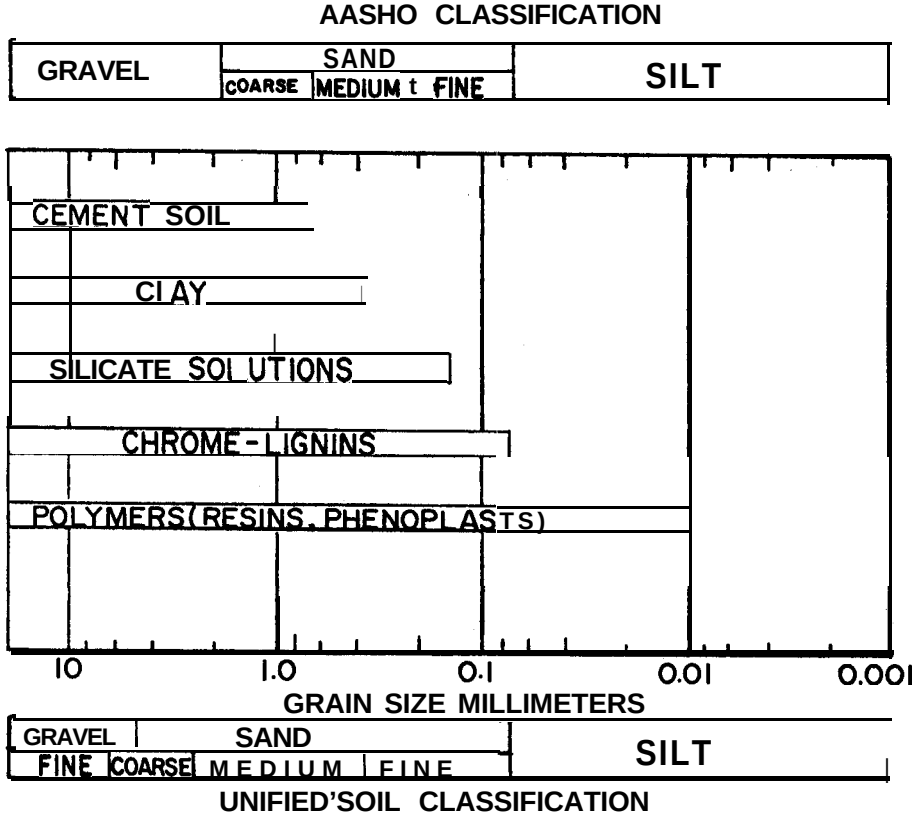


Figure 135. Range of usefulness of various grout types (from Mitchell, 1968).

Table 19. 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	High 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
Xl-based unsaturated fatty acid polymers	Polythixon FRD	10-80	25-360	0.99-1.05			X

b. A one-shot process where the gel strength of a very 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) and then gels almost instantaneously.

15.32 Procedures

15.32.1 Driven Lance

Probably the most widely used method for injection at shallow depths (10 - 12m) is the driven lance method (Dempsey and Moller, 1970). The method consists of driving a lan-ce using a pneumatic hammer and extracting the lance by jacking. The injection can be through perforations at the end done either during driving or withdrawal (or both in a two-shot process). Alternately, 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. Figure 136 schematically illustrates the driven lance method.

15.32.2 Sleeved Grout Tube

The sleeved grout tube or "tube-á-manchette" method was introduced by Ischy and is the standard method of injecting grouts in deep or intricate grouting operations (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 to seal the tube into the ground. At 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 the use of different grouts and better control of the grouted soil mass properties. Figure 137 shows the basic "tube-á-manchette" and grouting procedures.

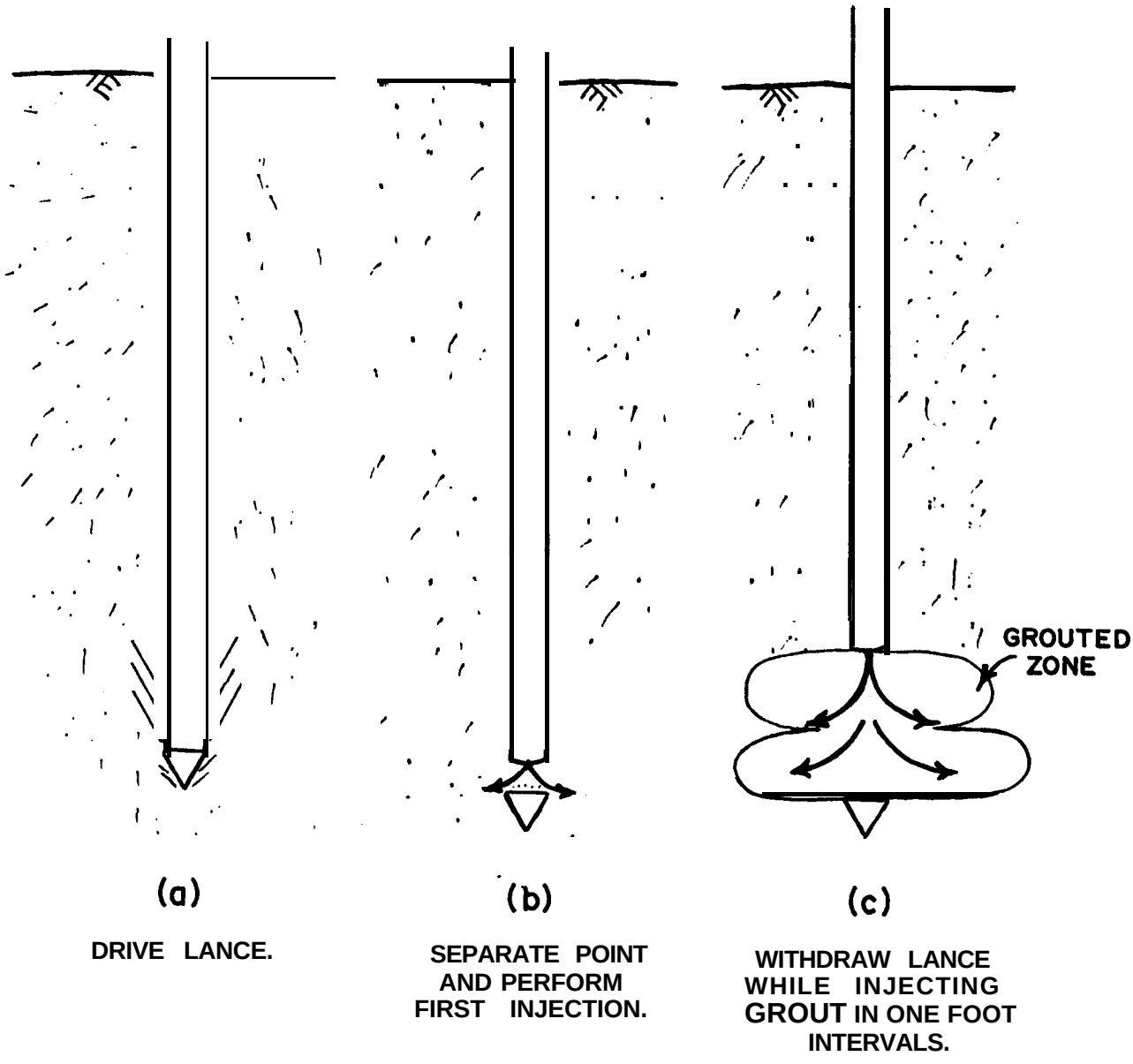


Figure 136. Schematic of driven lance method.

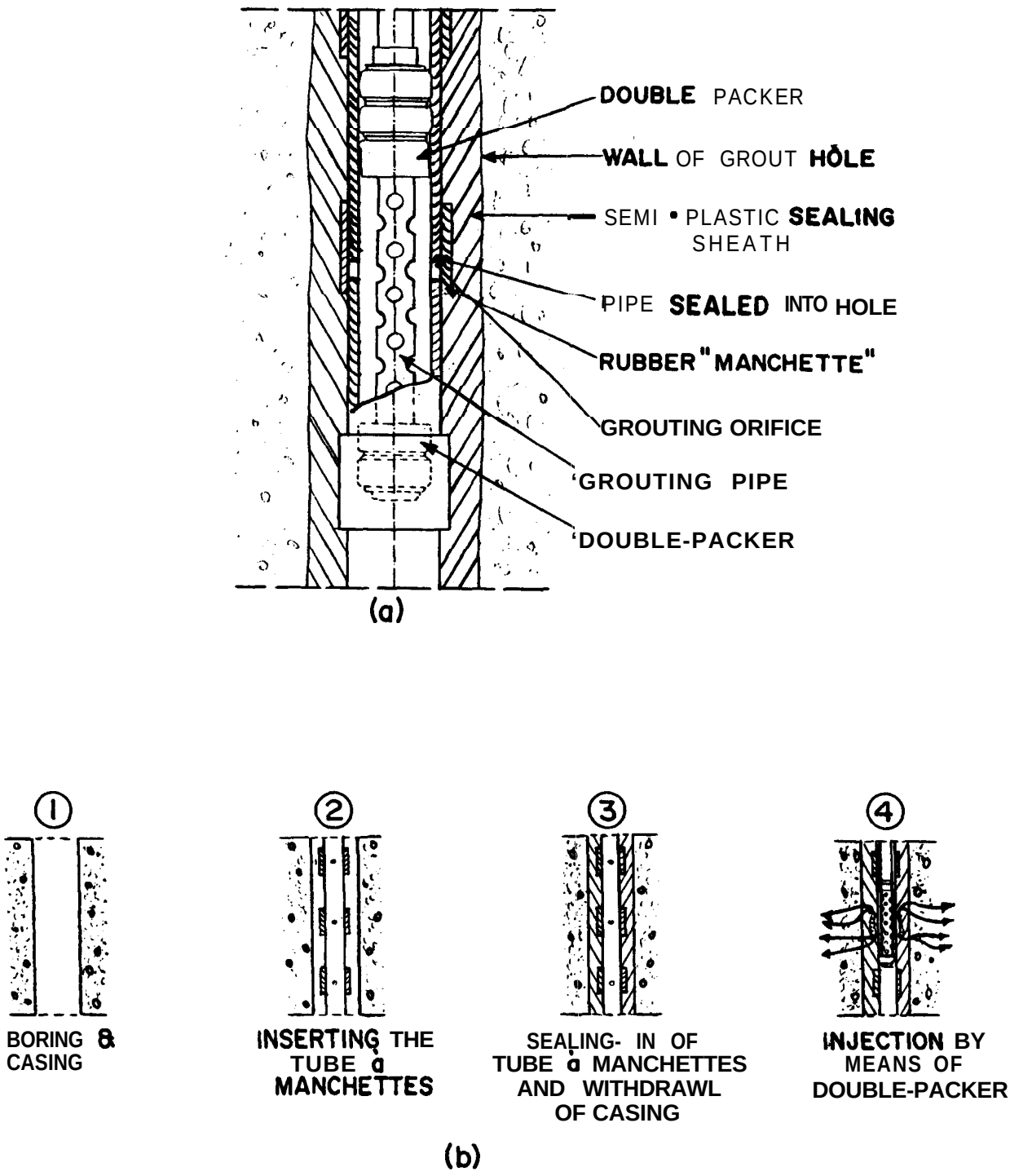


Figure 137. Sleeved grout tube (after Ischy and Glossup, 1962).

15.32.3 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.

15.32.4 Special Techniques

Vibratory Lances

Büttner (1974) reported a cage 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. The point was detached and grout pumped through the plastic pipe to form the horizontal cutoff. The primary advantage of this method is that the grouts can be injected at depths of up to 23 m or approximately twice the depth possible using driven lances.

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.

15.40 FIELD TESTING AND QUALITY CONTROL

15.41 General

Since it is becoming increasingly important to know how successful the grouting has been prior to the start of construction, techniques for evaluating grouted soil performance have been developed; but much improvement is needed. Some techniques are discussed in this section. Halliburton (1975) discusses field testing in greater detail.

15.42 Ground Water Control

15.42.1 Core Borings

This technique consists of drilling core holes into the grouted soil mass and recovering grout-soil samples. These samples

can then be tested in a laboratory to determine the permeability characteristics of samples. Since the samples are difficult to obtain and since there are no standarized procedures for testing grouted soils, this method is of limited value.

15.42.2 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 and calculating the resulting permeability. 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, 1975).

15.42.3 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. By pumping on one side of the grout curtain and observing the loss of head on both sides of the curtain the effectivenss of the ground water barrier could be determined. Alternatively, dyes could be injected on the side of the curtain away from the pump and the travel times observed.

15.43 Soil Stabilization

At present the methods of evaluating the effectiveness of grouting to stabilize a soil mass are pr imitive. 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.

16.10 INTRODUCTION

The first reported use of ground freezing as a stablization method was 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 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 to bind 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.

In practice, a designed pattern of freezing pipes or "probes" is placed 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. In open systems the cooling is accomplished by sublimating a solid (typically CO_2) or releasing pressurized liquefied gas to evaporate in the zone where cooling is wanted.

16.20 DESIGN AND THEORETICAL CONSIDERATIONS

16.21 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.

16.21.1 Thermal Properties

a. Initial subsurface temperatures (T_o)

b. Volumetric 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 pore water one degree. 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 is:

$$C_u = \gamma_d \left(0.2 + \frac{w}{100}\right) \text{ in BTU/ft}^3/^\circ\text{F (unfrozen)}$$

$$C_f = \gamma_d \left(0.2 + \frac{0.5w}{100}\right) \text{ in BTU/ft}^3/^\circ\text{F (frozen)}$$

where:

γ_d = dry unit weight of soil (in pounds per cubic foot, pcf)

Typical values for dry unit weight and water content of soils are given in the table below:

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. After the temperature of water is just lowered to 32°F, Latent heat of fusion (L) of the pore water is the amount of heat removal needed to convert the water to ice. Because latent heat is large compared to all other heat losses, it usually represents the most important factor in the freezing process. 144 BTU are required to convert one pound of water into ice (or approximately 80 cal/gm).

$$L = \gamma_d \, 0.8w \text{ gm-cal/cm}^3 \quad (\gamma_d \text{ in gm/cm}^3)$$

or

$$L = \gamma_d \, 1.44w \text{ BTU/ft}^3$$

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) is the quotient of conductivity and volumetric heat capacity (α =K/C). Kersten (1949) provides a summary of thermal conductivities for typical frozen and unfrozen soils.

16.21.2 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 may be performed in the laboratory or in situ. Schuster (1975) uses in situ pressuremeter tests for determination of short term deformation characteristics.

Creep

The creep rate of frozen soil is dependent upon stress level and temperature. Typical behavior patterns are shown in Figures 138 and 139. Figure 138 shows the effect of increasing compressive stress on axial strain. Figure 139 shows strain increase with both higher stress and higher temperature.

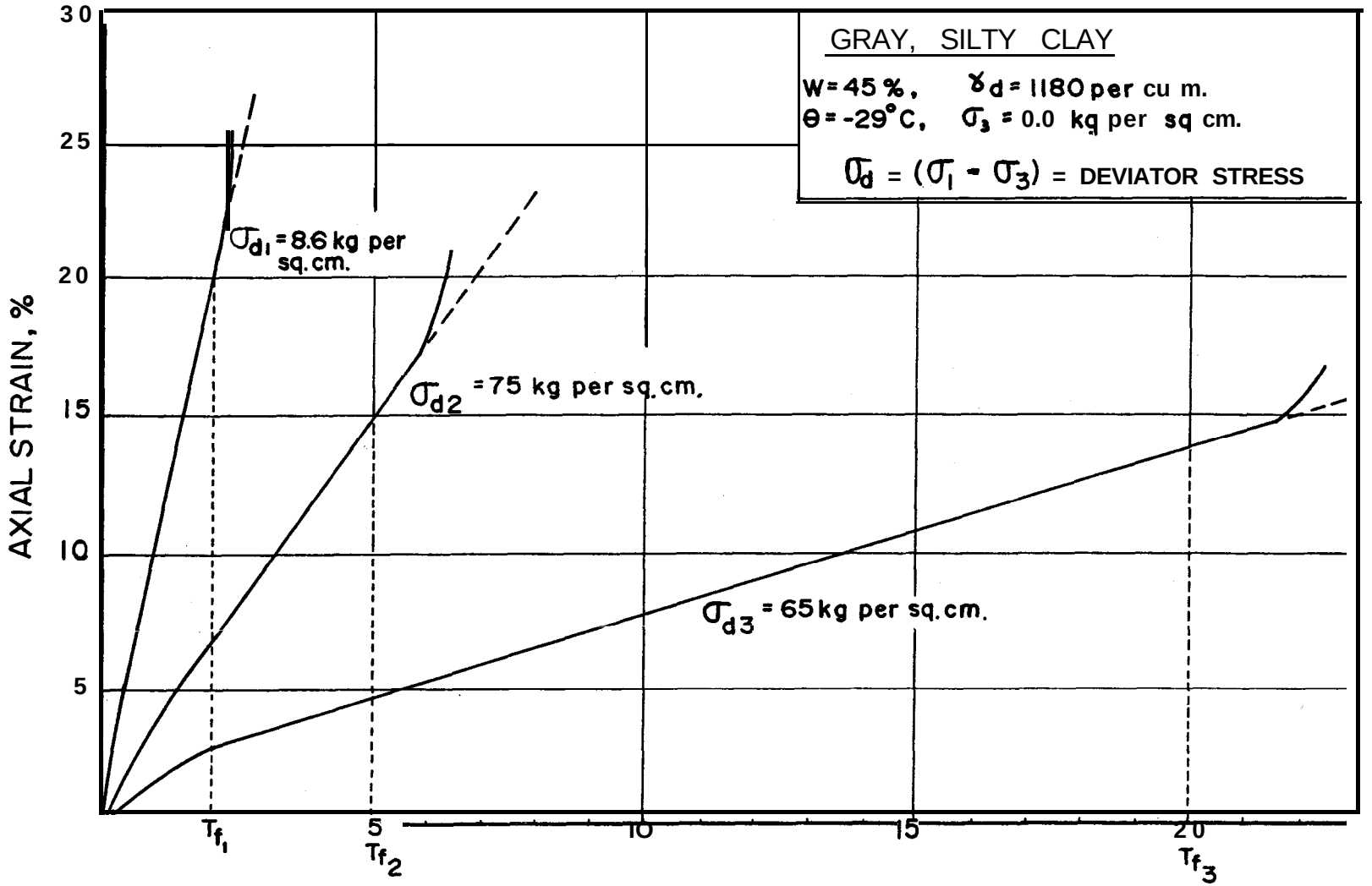


Figure 138. Strain versus time and loading for a frozen soil,

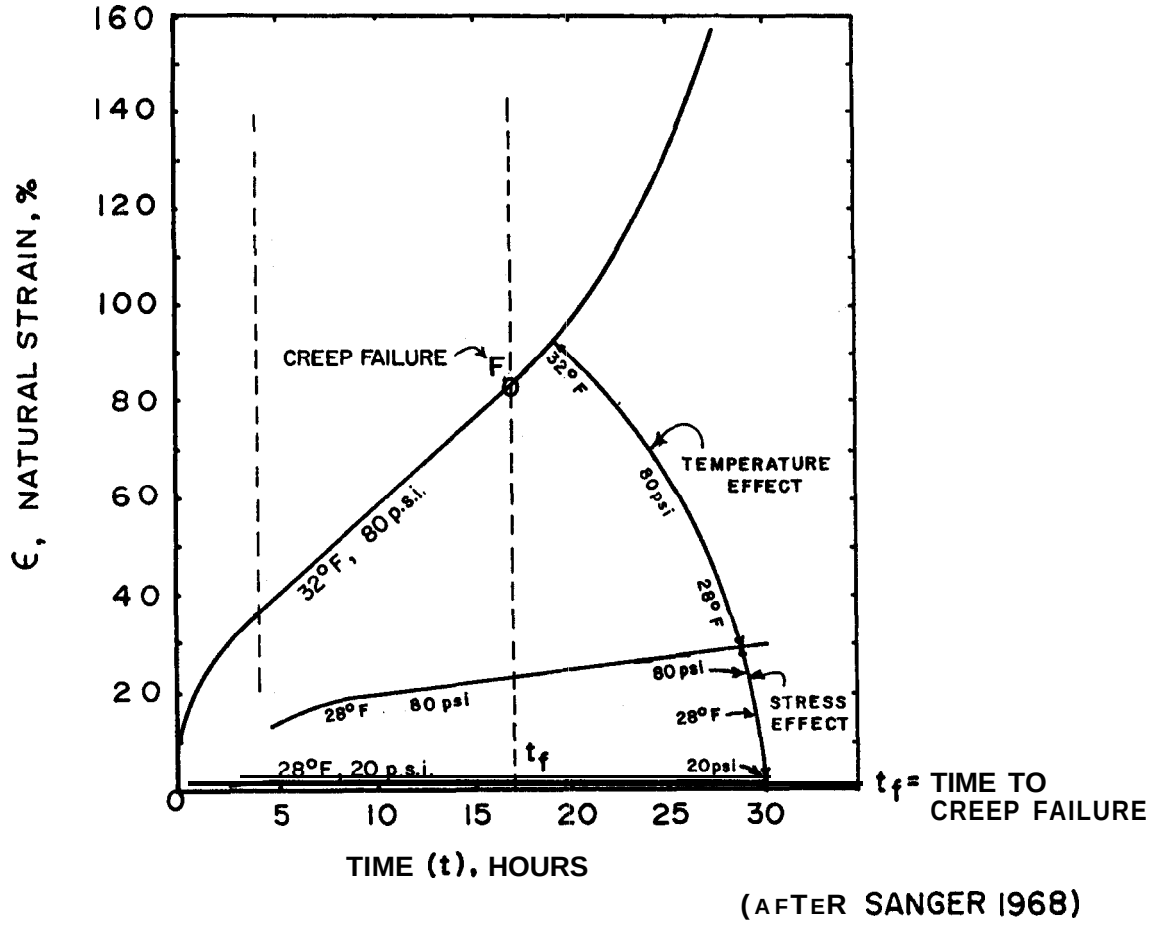


Figure 139. Creep curves for an organic silty clay with temperature influences.

Stress is held constant for each of the three curves.

Point "F" in Figure 139, represents the line at which the rate of strain becomes progressively greater. Sanger (1968) refers to this as creep failure.

Creep tests, such as those shown in Figure 139 are carried out under constant stress and temperature while measuring strain. 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

A summary of ultimate compressive strengths of common soils as a function of temperature below the freezing point of water is given in Figure 140. As may be noted, sandy soils have greater strengths than clayey soils. As the clay content of the soil increases, the shear strength decreases.

The strength of frozen granular soil at a given temperature increases as the moisture content increases. Figure 141 shows the ultimate compressive strength increase of frozen sand. The figure also shows that the strength of a clay does not increase with moisture content.

16.21.3 Geometry and Capacity of the Freezing System

Cost and time factors for ground freezing programs are strongly influenced by both the geometric arrangement of the 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 progess 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 ground water flow.

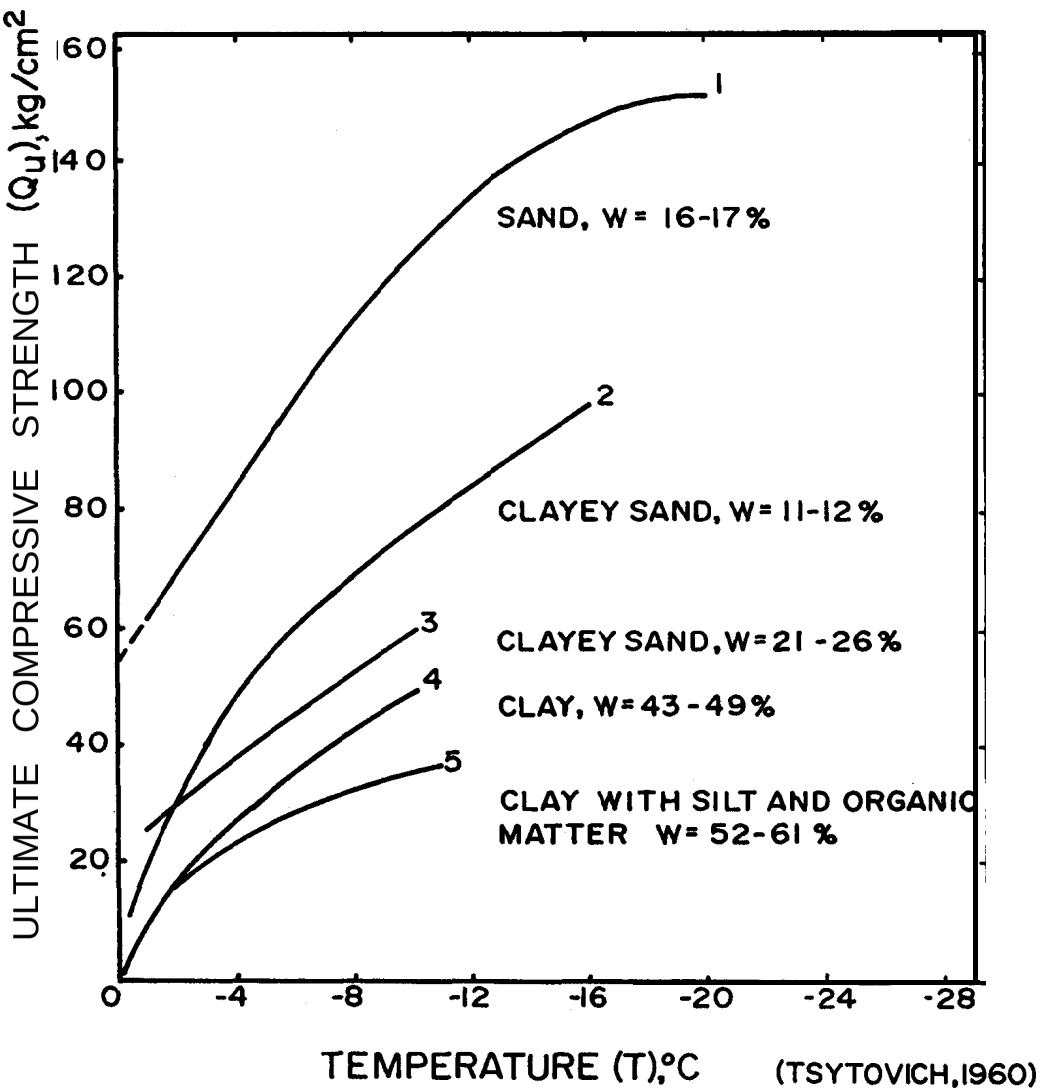
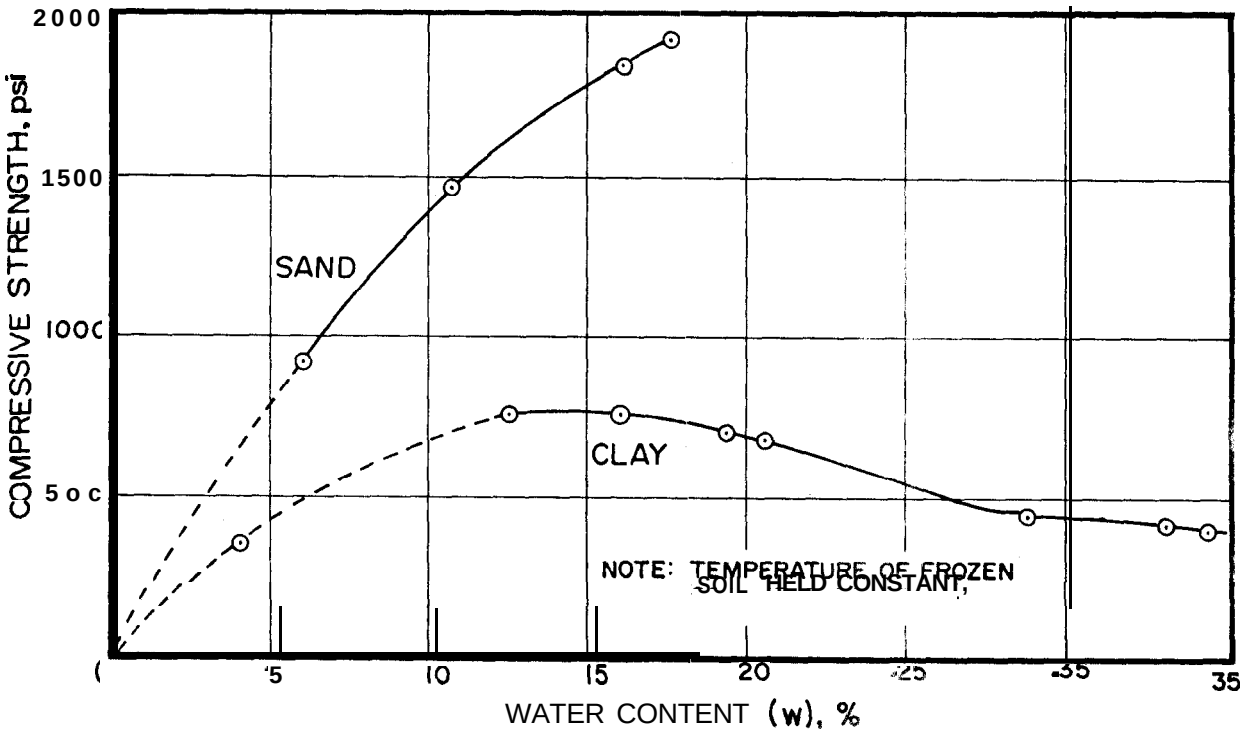


Figure 140. Ultimate short term compressive strength as a function of temperature.



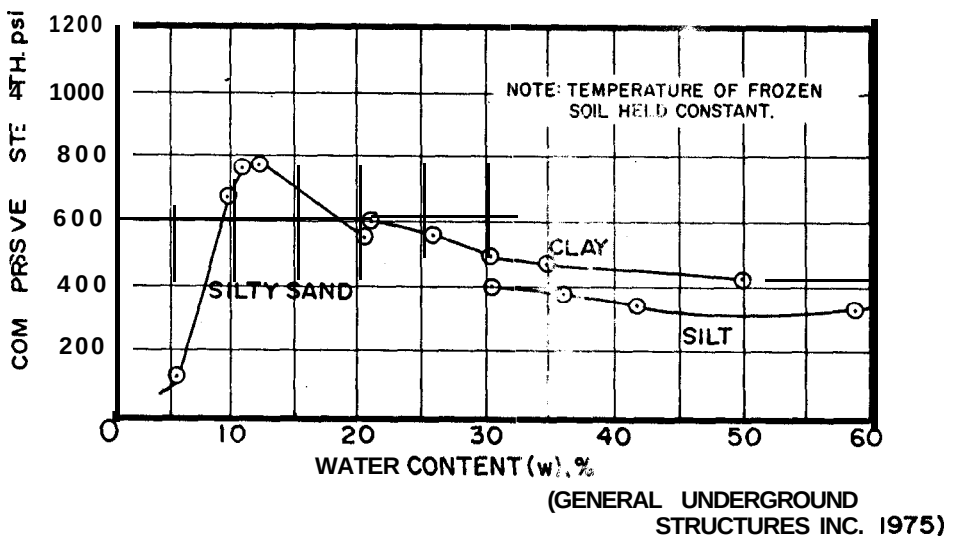


Figure 141. Ultimate short term compressive strength of ground vs. moisture content.

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.

16.22 Approaches To Design

16.22.1 Thermal Considerations

The analysis 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 a ctivitie s. All methods are fundamentally an exercise in heat transfer from the ground to the atmosphere.

The basic approach to simplify the analysis 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 existing condition to frozen condition. This simplification implicity neglects temperature drops (and therefore heat loss) at distances beyond the ice front. 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 soil temperature, T_o, to the freezing temperature, Tf .

Q_L = Latent heat flow to transfer from water to ice (occurs at constant temperature, of Tf).

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:

$$Q_u = C_u (T_0 - T_f)$$

$$Q_L = \gamma_d (1.44) w$$

$$Q_f = C_f (T_f - T_2)$$

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.

Typically the latent heat is large compared to the volumetric heat associated with temperature drop.

16.22.2 Mechanical Considerations

Consideration of creep is fundamental and indeed, for some cases special measures will be needed to offset contingencies associated with excessive creep which is especially important with frozen arches or tunnels.

Open surface excavations with frozen walls are normally designed as simple massive gravity structures or as cantilevered beams. The latter requires Less thickness of frozen'earth.

16.22.3 Ground Movement Considerations

Knowledge gained from studies of frost action below pavement indicates that clean, free draining soils have insufficient fines to develop capillarity and therefore do not develop ice segregation. 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.

Ground freezing below pavement differs from ground freezing as a construction method in several ways: (1) it is much slower; (2) the ice front is usually parallel to the stratigraphy; and (3) it is typically in the capillary zone above the water table.

Where ground freezing is used as a construction method, frost heave is not a problem with free draining, non-frost suspectible soils. In poorly drained soils, heave is generally attributed to two separate phenomena. The first is an approximate nine per cent expansion of pore water upon freezing. The second is expansion from pore water migration and ice segregation.

Rapid freezing can be used as a device tomitigate ice segregation. However, after a period of time when the rate of ice front advance slows down or stagnates, the threat of ice segregation and associated expansion will increase. In such cases, careful monitoring is essential, especially where structures are adjacent to the excavation.

16.22.4 Selection of Freezing System

Figure 142 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 and is essentially the same system used by Poetsch in 1883. The system consists of an ammonia or freon primary refrigeration plant to chill a secondary brine 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 temperature of -40°C.

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. Specifically, the alternatives to the Poetsch Process can be broken down as follows:

a. On-Site Refrigeration Plant

The first alternative is an on-site refrigeration plant with the primary refrigerant pumped directly into the freezing pipes. This system has been tried using ammonia. One disadvantage is that the system operates under a vacuum making leaks undetectable. With carbon dioxide, the system operates under high pressure to keep the CO_2 liquid. Hence, expensive high pressure plumbing is required.

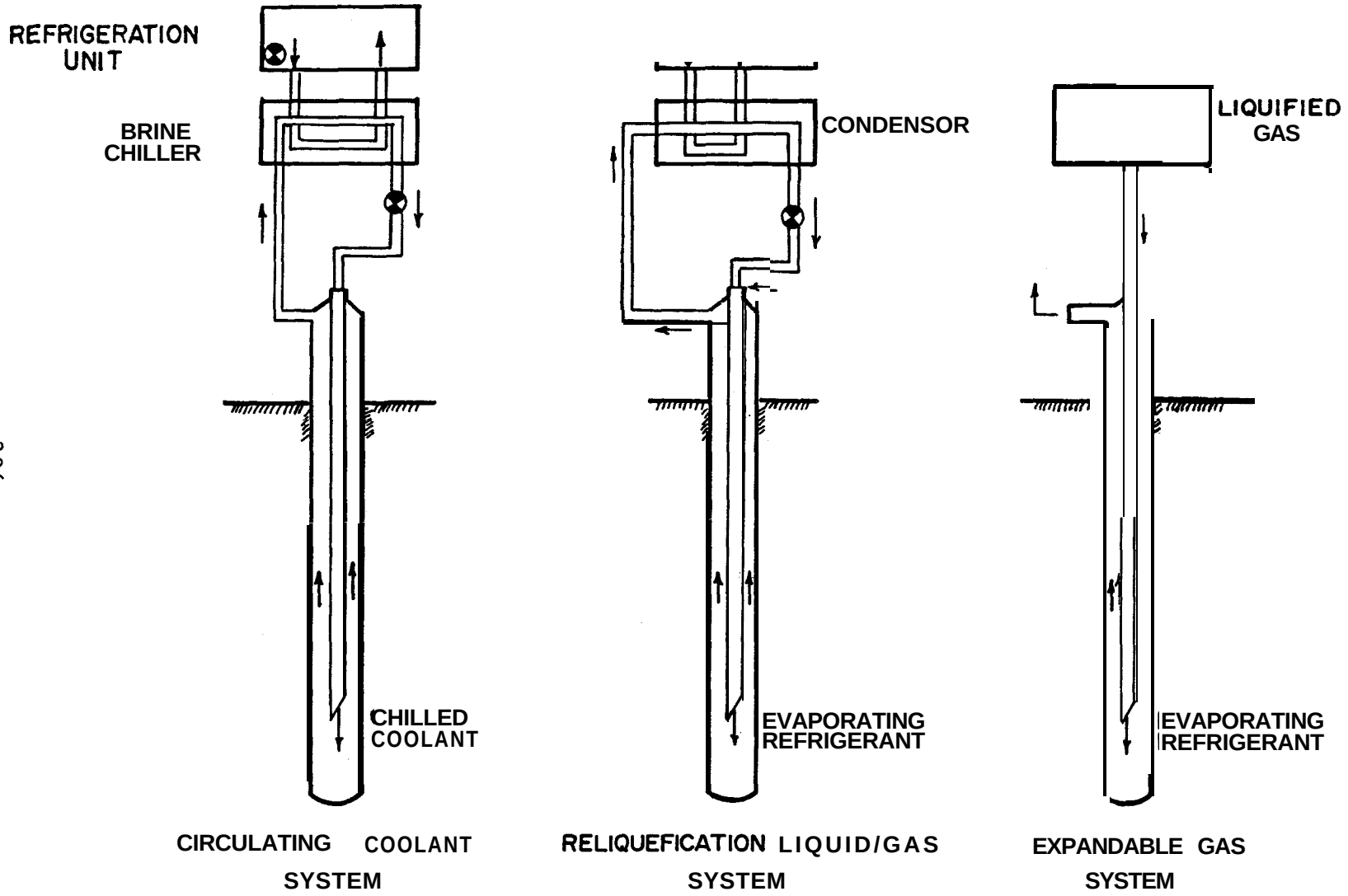


Figure 142. Basic refrigeration system elements for ground freezing.

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 CO2 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 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

A third technique is to use expendable refrigerants, such as commercially available liquid nitrogen. A less efficient but cheaper alternative might be solid or liquid CO_2. These materials are piped into the ground and then vented to the atmosphere. Expendable refrigerants are maintained at a lower temperature than can be achieved in the brine by on-site refrigeration units; and therefore, their rate of freezing will be more rapid. Typically, they are used for a short term and/or emergency situations. See Figure 143 for an example.

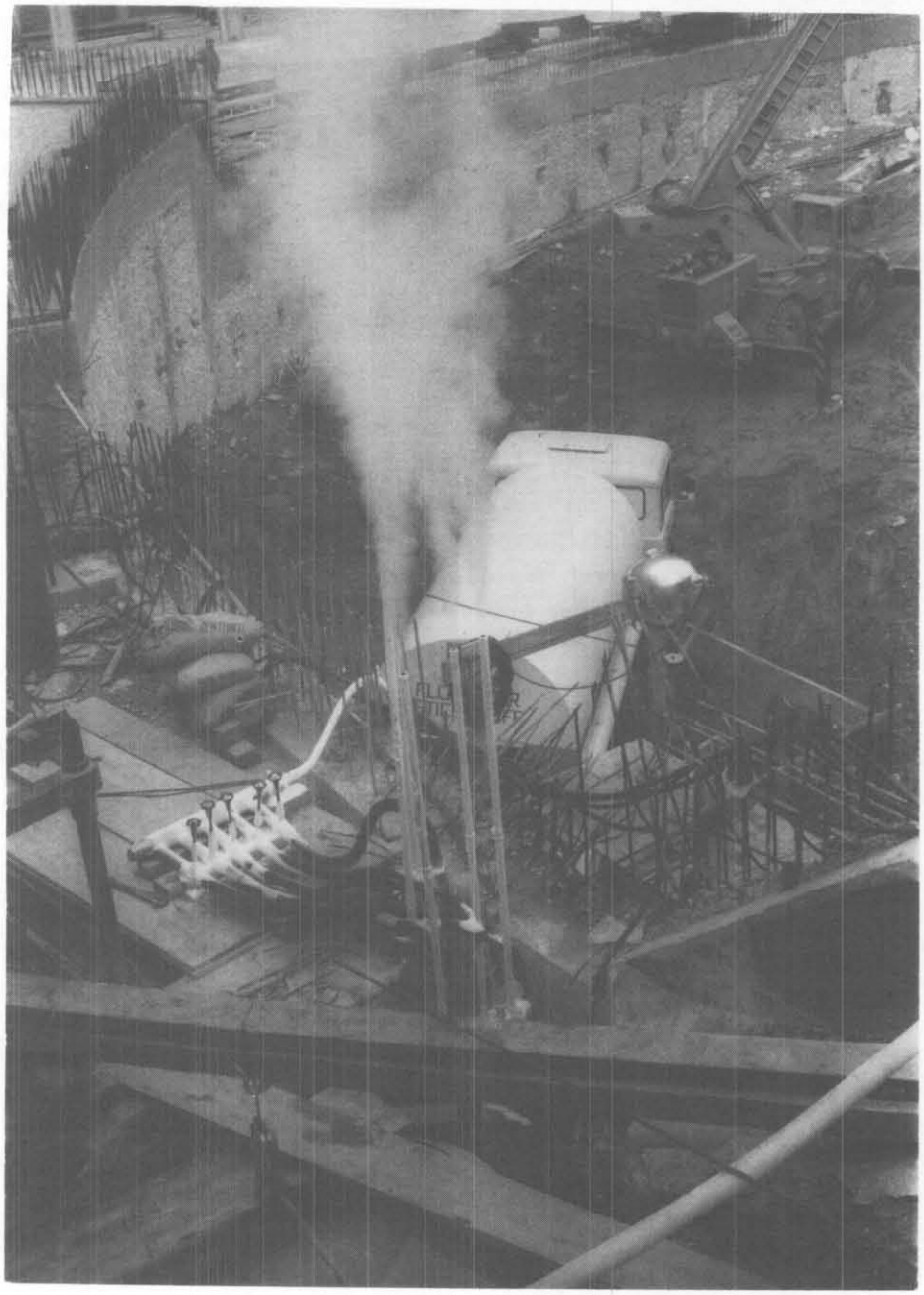
The basic freezing; method consists of choosing one of the freezing processes discussed above and drilling freeze holes into which the freezing pipes are installed. A cylinder of frozen material forms around the pipes and increases in size until the heat gain at the perimeter is equal to the heat taken out in cooling. The freeze pipes are installed such 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 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 energy requirement to maintain the frozen condition in equilibrium is considerably less than the energy required for initial freezing. Therefore, the capacity of the refrigeration plant can be reduced after initial freezing.

16.30 CONSTRUCTION PROCEDURE

16.31 General Approach

Freeze probes are emplaced with spacing ("s") and probe size (r_o) according to time requirements and required freeze wall thickness for strength. Strength requirements are based upon the type of frozen structure (i. e. gravity wall); strength requirements determine the average temperature of the frozen mass. A photograph showing piping



Note: Manifold for liquid nitrogen. Nitrogen gas being vented to atmosphere.

Figure 143. Liquid nitrogen freezing to cut off leak in diaphragm wall.
(Courtesy of Terrafreeze Corporation).

connections between a series of freezing pipes is shown in Figure 144.

Obtaining the desired ice wall thickness is usually not a problem unless goundwater flows in excess of about 6 feet per day are encountered. Frequently, low temperature freezing techniques are employed to overcome heat Losses to the moving water above this range.

Special care must be taken when drilling the holes and placing the freeze pipes to insure proper alignment. This is a very critical part of the operation. If the freeze pipes are out of line, closure of the freeze wall may not be adequate to prevent leakage of ground water. In this regard, the interfaces between soil and bedrock or between sands and underlying clays are critical. A closely monitored freezing program is required to prevent any gap in the freeze wall.

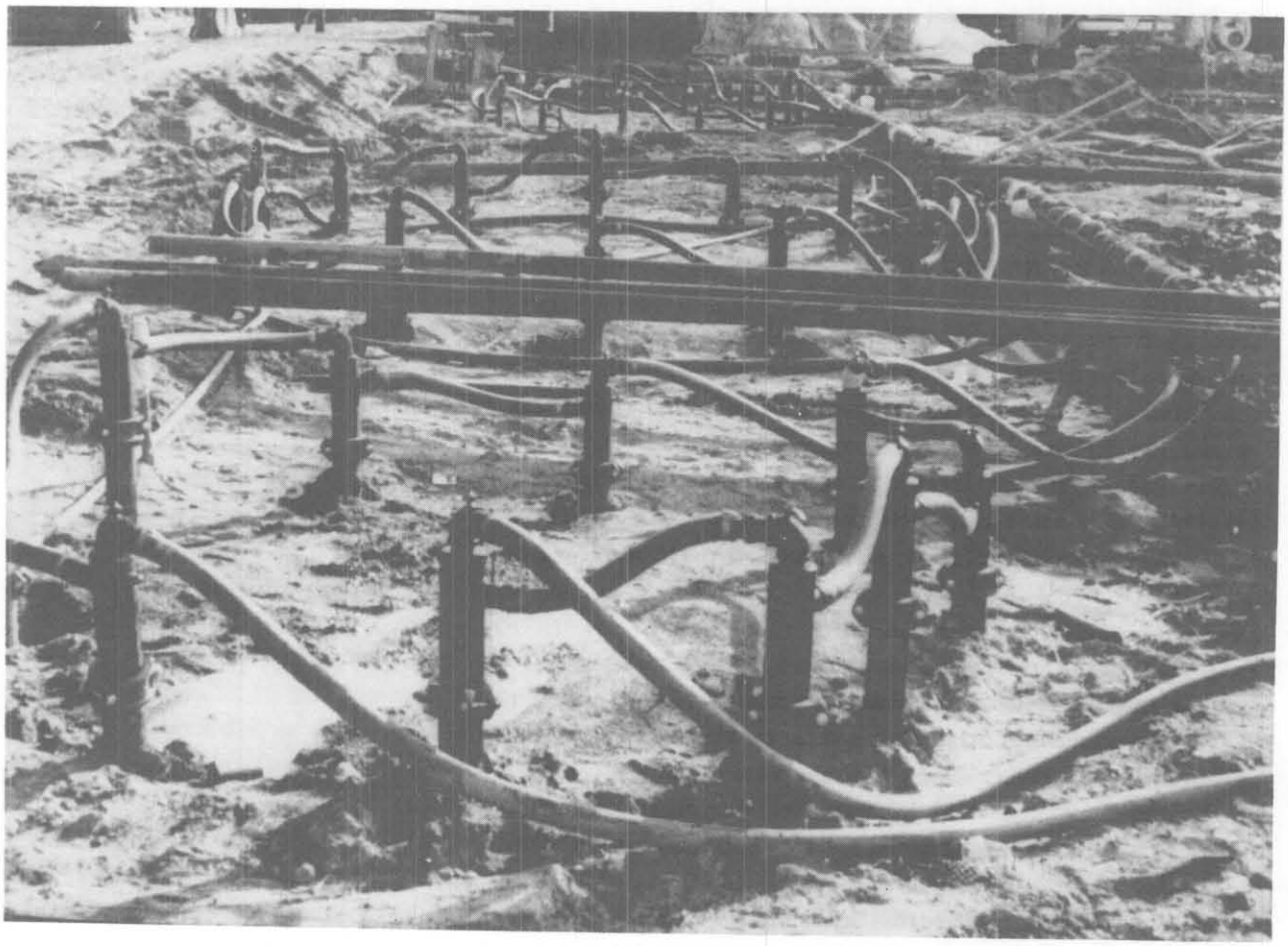
It is common practice to design the frozen structure so that it either bottoms in an impervious stratum or a frozen bottom is part of the design. When the former procedure is used, the freezing probes are commonly inserted several feet into the impervious zone to assure that watertight closure of the frozen structure is accomplished.

16.32 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 and leaks in the frozen wall. Maintenance of the frozen mass of earth after it is formed depends on a constant removal of heat to compensate for any heat gain at the fringes of the frozen zone. Open excavations are commonly covered with reflective thermal insulation that provides protection against sun and rain. An aerial view of a protected freeze wall is shown in Figure 145.

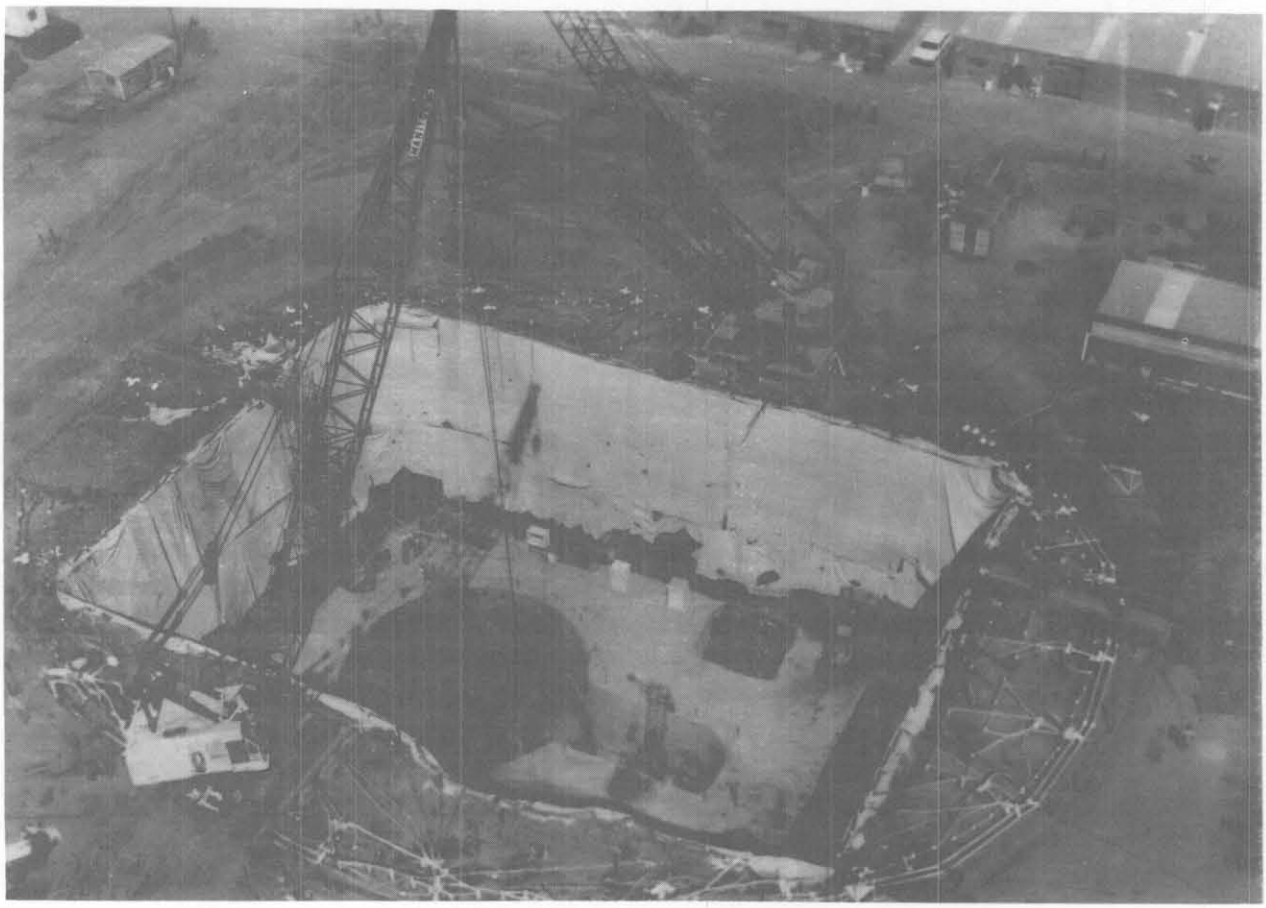
16.33 Special Construction Problems

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°F isotherm remains in the insulation.



Note: Each group of freeze pipes forms a series of loops from brine supply line back to the return line.

Figure 144. Typical supply and return connections between group of freeze pipes using brine. (Courtesy of Terrafreeze Corporation).



Note: Wall is protected by reflective thermal insulation.

Figure 145. Aerial view of freeze wall surrounding deep excavation.
(Courtesy of Terrafreeze Corporation).

16.34 Construction Monitoring

Monitoring subsurface and brine temperatures is a requirement during construction of a frozen ground structure. This is usually accomplished by measuring the profile of subsurface temperatures in small diameter observation pipes (1"O. D., or so) distributed throught the frozen zone. Commercially available thermistors or thermocouples are widely used as the temperature sensor, and relatively inexpensive readout devices are adequate for the monitoring requirements. Whether a problem exists in the refrigeration system, or in unexpected subsurface conditions, can normally be detected with an accurate profile of subsurface temperatures and routine coolant temperature data obtained during plant operation.

Report No. FHWA-RD-75-129

LATERAL SUPPORT SYSTEMS AND UNDERPINNING

Vol. II. Design Fundamentals

D. T. Goldberg, W. E. Jaworski, and M. D. Gordon



April 1976
Final Report

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16. Abstract

This report provides current information and design guidelines on cut-and-cover tunneling for practicing engineers. The main emphasis is on the geotechnical aspects of engineering. Included in this volume is a state-of-the-art summary of displacements and lateral pressure. Other topics are basic concepts of soil mechanics, ground water in open cut, passive resistance, design aspects of lateral earth pressure, stability analysis of sheeted excavations, bearing capacity of deep foundations, and construction monitoring. Detailed explanations of design methods and literature citations are included.

Other reports prepared as part of the study are FHWA-RD-75-128, Volume I, Design and Construction; FHWA-RD-75-130, Volume III, **Construction Methods; and FHWA-RD-75-131, 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.

ACKNOWLEDGEMENTS

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To Mr. John Shuster of Terrafreeze Corporation, Bethesda, Maryland for the review and contributions for the section on the techniques of ground freezing.

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To Mr. D. Maishman of Foraky, Ltd., Great Britain for allowing the authors the opportunity to review his yet unpublished manuscript on ground freezing.

To those many engineers, manufacturers, and contractors who graciously provided comments, data, and photographs.

To Mr. John Dunnicliff of Soil & Rock Instrumentation, Inc., of Newton Upper Falls, Massachusetts for the chapter on Construction Monitoring.

To those individuals associated with Goldberg-Zoino & Associates, Inc. who contributed greatly to the various chapters in these manuals: Dr. Stephen A. Alsup on Ground Freezing; Mr. Peter Riordan on Bearing Capacity; Mr. Jackson Ho for the outstanding effort on computer work; the efforts of Ms. Andrea Wizer and Mrs. Joan Jennings for their patience and endurance in the typing of these manuscripts.

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And finally, to Mr. J. R. Sallberg, contract manager for the Department of Transportation and Federal Highway Administration, for his patience, support, and welcome criticisms throughout the program.

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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	millimeter s
mm^2	square millimeter s
mm^3	cubic millimeters
ml	milliliters
N	Newton
lbs	pounds
pcf	pounds per cubic foot
pl_f	pounds per lineal foot
ps_f	pounds per square foot
psi	pounds per square inch
sec	second

Conversions

British Units	Metric Units
1 BTU	0.2520 kg - calories 107.5 kg - meters
1 in	2.540 cm = 25.4 mm
1 in^2	6,452 cm^2
1 in^3	16.103 cm^3
1 ft	30.48 cm = 0.3048 m
1 ft^2	929 cm^2 = 0.0929 m^2
1 ft^3	28,317 cm^3 = 0.0283 m^3
1 pcf (lbs/ft^3)	16.02 kg/m^3 = 0.01602 g/cm^3
1 psf (lbs/ft^2)	4, 883 kg/m^2 = 47.9 N/m^2
1 ksf ($kips/ft^2$)	
1 psi (lbs/in^2)	
1 lb	4.45 N
Lin- lb	0.1127 N-m

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.

Symb 1	Represents	Reference
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	over consolidation ratio	

Svmbol	Represents	Reference
P	general symbol for load or force	
p	general symbol for pressure	
pH	ne gative 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 fr i ction	
$\delta_{v_{(max)}}$	vertical displacement (maximum)	
$\delta_{h_{(max)}}$	horizontal displacement (maximum)	
ε	general symbol for strain	
γ	general symbol for unit weight; total unit weight of soil unless other wise 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	

Symbol	Represents	Reference
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 pr e s sure (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 1 - INTRODUCTION

1.10 PURPOSE AND SCOPE

This report, Volume II, discusses the general design aspects of cut-and-cover tunneling, in particular those factors affecting the design of the ground support wall, such as earth pressure, Lateral resistance , ground water, and bearing capacity. In addition, considerable emphasis is placed on displacements of adjacent ground and adjacent structures.

The intent of Volume II is to provide the basic theoretica. elements and design framework with which to approach the engineering of deep excavations and underpinning. Because emphasis is upon geotechni cal consider ations, comments on structural factors are included only when closely related. Legal and contractural relationships are not discussed.

1.20 ORGANIZATION AND USAGE

Including this introductory chapter, there are 10 chapters in this volume, With the exceptions of Chapters 3 and 10, each of the remaining chapters discusses analytical procedures or a particular aspect of design. Chapter 3 presents basic soil behavior concepts for reference, and Chapter 10 is an overview of construction monitoring as related to deep excavations and tunnels. Each of the remaining chapters is discussed briefly below.

1.21 Displacements - Chapter 2

The displacements occurring in the structures and soil mass adjacent to an excavation will have an effect on the choice of wall and the remedial or preventive measures required to protect the structures. The greater the amount of movement, the greater will be the protective measures required. An analysis of the performance data re cor ded at approximately 60 excavations revealed several important relationships between wall type, support type, soil type, and movements .

In sands and gravels and very stiff to hard clays, the wall type did not markedly affect performance. However, in softer cohesive soils the stiffer support systems (concrete diaphragm walls) limited movements to a much greater extent than did the more

flexible soldier pile or steel sheet pile walls. Some preliminary conclusions can be drawn from the displacement data analyzed which may be used as an aid in predicting movements behind a wall.

By improving the displacement prediction techniques for movements adjacent to an excavation, the engineer will be in a better position to evaluate the many factors involved in the decision to underpin structures. If displacements can be reliably predicted, particularly with distance behind excavations, the effect that the movements will have on structures can be evaluated, and the costs of repair can be compared to the costs of alternative procedures.

1.22 Basic Soil Parameters - Chapter 3

This chapter is a brief review of the basic soil parameters and soil behavior that affect lateral support wall design. Of particular interest are the differences in strength behavior between cohesionless and cohesive soils. In cohesive soils, it is important to identify the situations where undrained.. strength or draiaed strength is the critical controlling parameter. The chapter is not intended to be a complete review of soil properties and soil behavior; rather, it is intended to provide a general overview of factors affecting wall design.

1.23 Ground Water - Chapter 4

In many excavations, ground water control is often the most difficult aspect of wall construction. Lowered ground water levels may cause consolidation settlements in soil profiles where compressible soils are present. Ground water flow into the excavation may result in running soils and creation of voids behind a wall. This chapter summarizes the basic concerns in ground water as it relates to cut-and-cover tunneling; however, it does not provide specific details on design of dewatering systems.

1.24 Lateral Earth Pressure - Chapter 5

Lateral earth, water, and surcharge pressures on a wall represent the driving forces that must be resisted by the passive resistance of the soil below the base of the excavation and by the support members (internal bracing or tiebacks). This chapter reviews the established state-of-the-art, advances some new

concepts, and presents recommended design earth pressure distributions for internally braced and tied-back walls. Lateral pressures caused by surcharge are also discussed.

1.25 Passive Resistance - Chapter 6

The passive resistance mobilized in the soil below the cut may play a role in maintaining the stability of the wall and in controlling the amount of lateral deflection. Design parameters for passive pressures are presented, and specific design criteria are discussed,

1.26 Design Aspects of Lateral Pressure - Chapter 7

This chapter discusses the techniques used to evaluate strut loads, wale loads, and presents some typical design problems incorporating the principles advanced in the previous chapters. Specific design recommendations are made for allowable stresses in steel members, determination of passive resistance, 'and methods of analyzing loads in structural members. Recommendations are made relative to overcut, depth of embedment, and temporary overstressing.

1.27 Stability - Chapter 8

The base stability of excavations must always be examined, particularly in softer soil profiles. Methods of evaluating overall stability as well as shear strains inherent in localized zones of excessive shear, are also discussed.

1.28 Bearing Capacity of Deep Foundations - Chapter 9

This chapter is most useful in the design of underpinning members. However, this section may also be applicable to tied-back walls where the vertical load induced in the wall by the vertical component of tieback load can be significant. Design criteria and design charts are presented for analysis of failure load and settlement under various loading conditions.

1.29 Construction Monitoring - Chapter 10

Chapter 10 describes the basic considerations and reasons for monitoring the performance of a lateral support wall.

The reasons may be to verify design assumptions, assure structure stability, observe performance for possible legal litigations, or to advance the state-of-the-art. The general procedure to be followed in planning a monitoring scheme is outlined.

CHAPTER 2 - DISPLACEMENTS

2.10 GENERAL

2.11 Purpose and Scope

The purpose of this section is to provide insight into displacements occurring adjacent to deep excavations - specifically, into those factors influencing displacements and into the manner in which displacements occur.

This section describes the basic performance of excavations in terms of the magnitude and pattern of soil and wall movements, Empirical plots are derived from the measured performance and presented. In addition, finite element analyses have been used to help assess qualitatively the relative influence of the aforementioned parameters. Together, the empirical studies of performance data and accompanying computer analyses have provided new insight into the understanding and prediction of displacement.

Several other empirical analyses of the performance of laterally supported cuts have been performed. Peck (1969) and D'Appolonia (1971) are perhaps the most widely known, and their work has been most valuable in the preparation of this section. This present investigation incorporates data from the more recent cases, including many with tiebacks and concrete diaphragm walls.

2.12 Significance of Displacements

While the magnitude of settlement is a useful indicator of potential damage to structures, the amount of settlement change with horizontal distance (settlement profile) is actually of greater significance. This fundamental concept is related to the concept of differential settlement, as opposed to gross settlement.

Horizontal displacements have proven to be a source of severe damage (Gould, 1975). Therefore,, attention to the threat of settlement should not cause us to overlook what may even be a greater source of damage, Indeed, horizontal displacements are often of greater concern than are vertical displacements in the presence of underpinned structures (Febesh, 1975).

2. 13 Relationship to Underpinning

Historically, the decision whether or not to underpin has been a subjective judgement based upon experience - experience which reflects local soil conditions, contractors' practices, attitudes of engineers, and jurisdictional authorities. Rarely, if ever, have engineers attempted to base a decision concerning underpinning on a quantitative evaluation of displacements. Rather, structures within certain preestablished influence zones would be underpinned. Alternatively, if the cost of underpinning was disproportionate in relation to the value of the structure and there was no danger of collapse, one might accept the inherent risk of not underpinning and make necessary repairs afterwards.

Fundamentally, the amount and distribution of the movements in a soil mass adjacent to an excavation is governed primarily by soil type, stiffness of support wall, and construction procedures. A better understanding of how these parameters control displacement will lead to a more rational assessment of effects on adjacent buildings and to the development of improved techniques to minimize displacements. Ultimately, these efforts will contribute to the decisions concerning methods of support and underpinning of structures.

2.20 CHARACTERISTICS OF WALL DEFORMATION

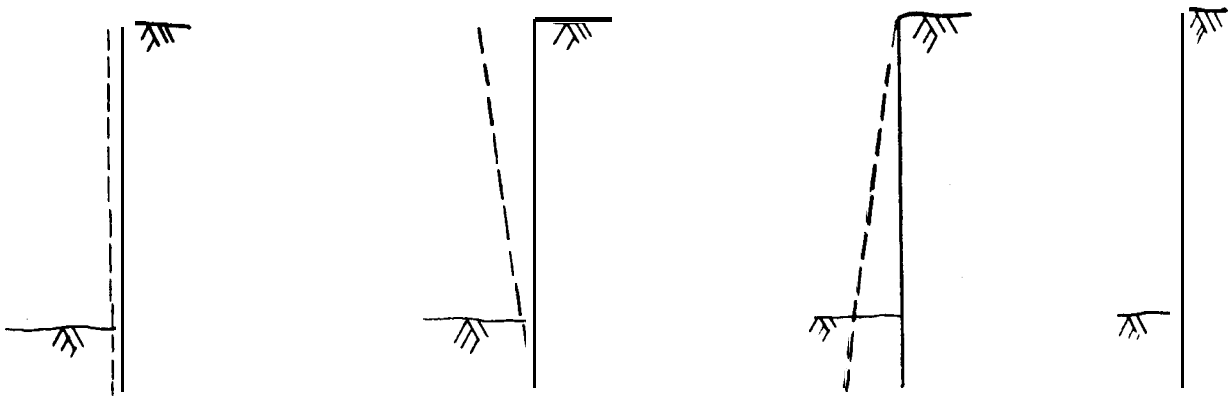
2. 21 General Mode of Deformations

Figure 1 shows the possible range of deformations for perfectly rigid walls and for walls displaying flexure. Basically the range of behavior includes translation and either rotation about the base or rotation about the top. In addition, wall deformation will include some bulging as a result of flexure -- the amount of bulging depending upon the stiffness of the wall support system.

2. 22 Internally Braced Walls

The upper portion of internally braced walls is restrained from undergoing large horizontal movement especially when braces are prestressed and are installed at or close to the surface. This produces the typical deformation mode as shown in Figure 2a. The degree of rotation will depend upon the toe restraint below the bottom of the excavation.

(a) INFINITELY RIGID WALLS



(b) WALLS DISPLAYING FLEXURE

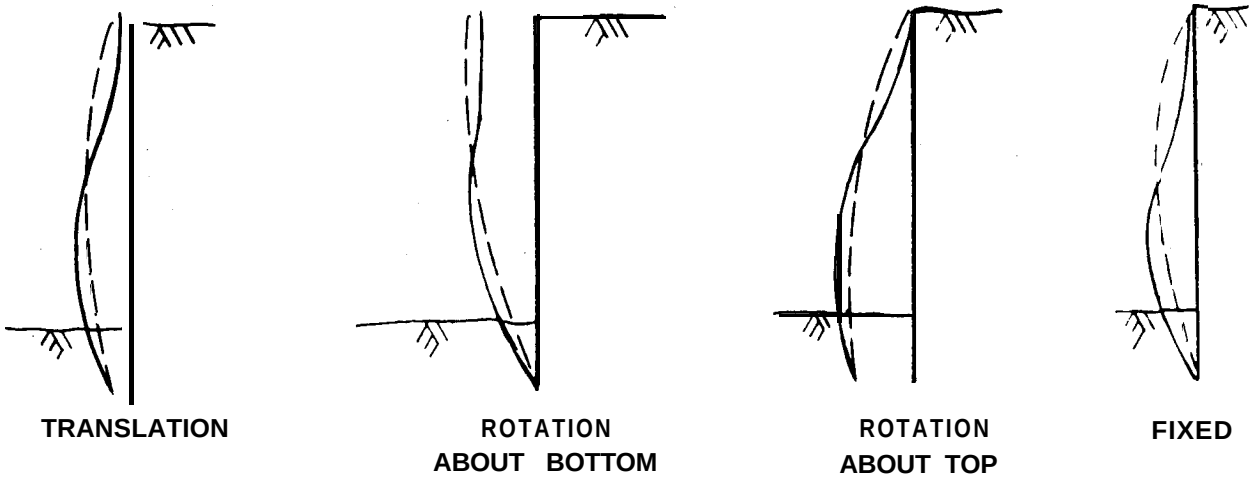
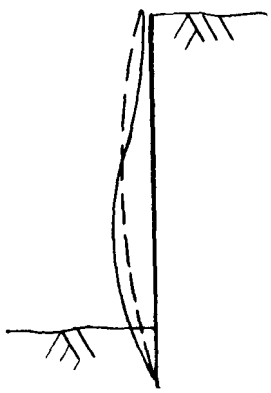
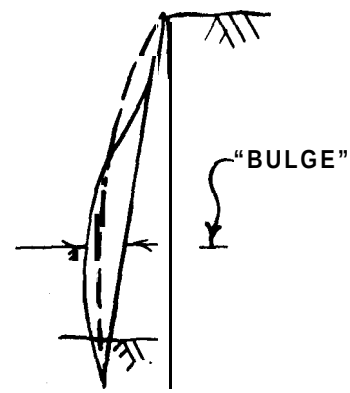


Figure 1. General deformation modes.

(a) TYPICAL FOR INTERNAL BRACING

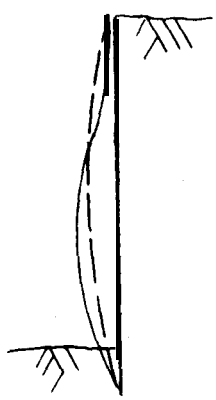


FIXED OR SLIGHT TRANSLATION

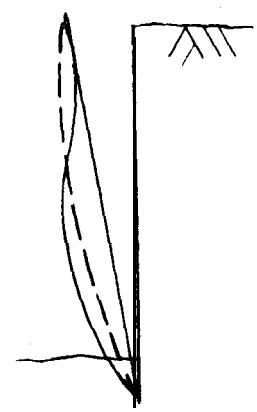


ROTATION ABOUT TOP

(b) TYPICAL FOR TIEBACKS



FIXED OR SLIGHT TRANSLATION



ROTATION ABOUT BOTTOM

Figure 2. Typical deformation of tied- back and internally braced walls.

2.23 Tied-Back Walls

If the top of the tied-back wall remains fixed, then the deformation mode is similar to that of an internally braced wall (see Figure 2b, left panel). On the other hand, settlement of the wall, partial yielding of the ties, gross movement of the soil mass, or shear deformation of the soil mass may result in inward movement of the top and rotation about the bottom as shown in Figure 2b, right panel.

Nendza and Klein (1974) attributed the deformation mode of Figure 2b, right panel, to a combination of shear deformation, which contributed to inward movement of the top, and flexure, which contributed to the bulging effect.

If the soil mass embodied by the tiebacks deforms somewhat as a unit, the pattern would be similar to that shown in Figure 3. Here, the top moves inward toward the excavation and the earth mass mobilizes internal shear. Such a concept was originally proposed by Terzaghi (1945) in connection with earth-filled cellular or double-walled cofferdams. Such a deformation mode is not true for all situations, but is very likely in cases of an unyielding base and with the bottom of the wall restrained from outward movement.

Overall, the deformation mode of a tied-back wall is complex in that various factors develop in different ways. For example:

1. High prestressing pulls the top of the wall into the soil, thus leading to a deformation mode of outward rotation about the bottom. In sensitive clays, this condition could induce consolidation (McRostie, et al, 1972).

2. Lateral translation of the entire soil mass occurs from shear strains within a weak underlying cohesive layer or from general lateral strain following relief of large residual horizontal stress in highly overconsolidated clay or soft shales. Observations show this may continue even after reaching full depth. (Burland, 1974; St. John, 1974; Breth and Romberg, 1972; Romberg, 1973).

3. Very stiff walls, such as diaphragm walls, will display less bulging from flexure. Therefore, horizontal movement at the top due to movement of the soil monolith will be large compared to the effect of flexure and therefore will assume relatively more importance.

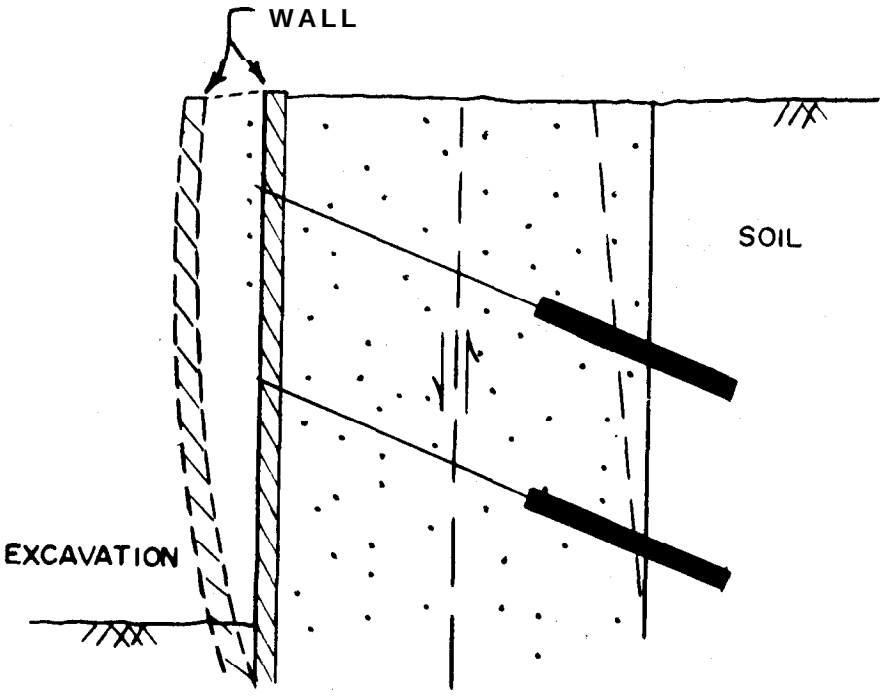


Figure 3. **Internal shear development, horizontal shift at top relative to bottom,**

2. 24 Comparison of Braced Walls with Tied-Back Walls

Overall, there are insufficient data to quantitatively compare deformations of internally braced walls with tied-back walls. Performance is highly dependent on construction methods and variables are many. In competent soils (e. g. granular deposits, dense cohesive sands, very stiff or hard clays, etc.) performance data suggest that tied-back walls display about the same deformation as internally braced walls.

Although the observational data demonstrate little difference between displacements with bracing or tiebacks, from a purely qualitative aspect, a number of factors suggest a superior performance should be attained with tiebacks in competent soils:

1. In granular soils in which soil modulus increases with stress level, the prestressed soil mass engaged by the tiebacks is made more rigid and therefore less deformable.

2. Tiebacks are typically prestressed to about 125 percent of the design load and then locked-off between 75 percent and 100 percent of the design load. Prestressing in this manner prestrains and stiffens the soil monolith. Further, the process pulls the wall back toward the soil to remove any "slack" in the contact zone.

3. Internal bracing, if prestressed, is usually prestressed to about 50 percent of the design load. Typically, the bracing gains in load as the excavation deepens. In contrast, tiebacks, being locked-off at higher loads, typically maintain the load or experience a slight loss of load with time. In the case of internal bracing elastic shortening of the strut continues after installation of the member.

4. Temperature strains are more important with bracing than with tiebacks because the latter are insulated in the ground. Temperature drop may cause a drop in load and/or contraction of the member. If load remains constant, a 35-degree Fahrenheit temperature drop from time of installation would cause a 75 foot member to shorten by about 0. 2 inches.

5. Frequently, internal bracing is removed then rebraced to facilitate construction, whereas tiebacks do not have to be removed.

Obviously, the flexure occurring with strut removal is affected by wall stiffness, span distance, and concurrent backfill and compaction. Past experience has shown strut removal to contribute significantly to settlement of adjacent ground. The settlement is the result of lateral wall deformation during the process of removing the support.

6. Contractors commonly overexcavate below bracing levels to facilitate removal of materials. This induces greater movements, especially in weak soils.' With tiebacks the contractor maintains the excavation at or slightly above the tieback level.

2.30 MAGNITUDE OF DISPLACEMENTS

2.31 Reported Horizontal and Vertical Displacements

Displacement of the soil retained by and adjacent to an excavation is a function of several factors including wall stiffness, construction technique, etc. Because of the inherent complexities of an actual installation, it is difficult to isolate all variables and analyze each separately on the basis of empirical data. However, some indication of the effect of some variables can be obtained by simplifying the primary characteristics of a cofferdam (soil type wall and bracing type) and summarizing and comparing them with the results of field measurements.

Figures 4 and 5 are an extension of a similar plot presented by D'Appolonia (1971). The figures show normalized vertical and horizontal displacements (ratio of the maximum displacements to the height of the cut) versus three general categories of soil type and support type. References for this data are summarized in Table 1. Diaphragm walls are distinguished from the relatively more flexible soldier pile or sheet pile walls by symbol.

Vertical and horizontal displacements in the ground outside the excavation arise from:

1. Horizontal and vertical displacement of the wall -- in general, these are rotation, translation, and flexure,

2. Movement of soil -- for example, loss of soil through lagging, overcutting and improperly backpacking of lagging, spalling of

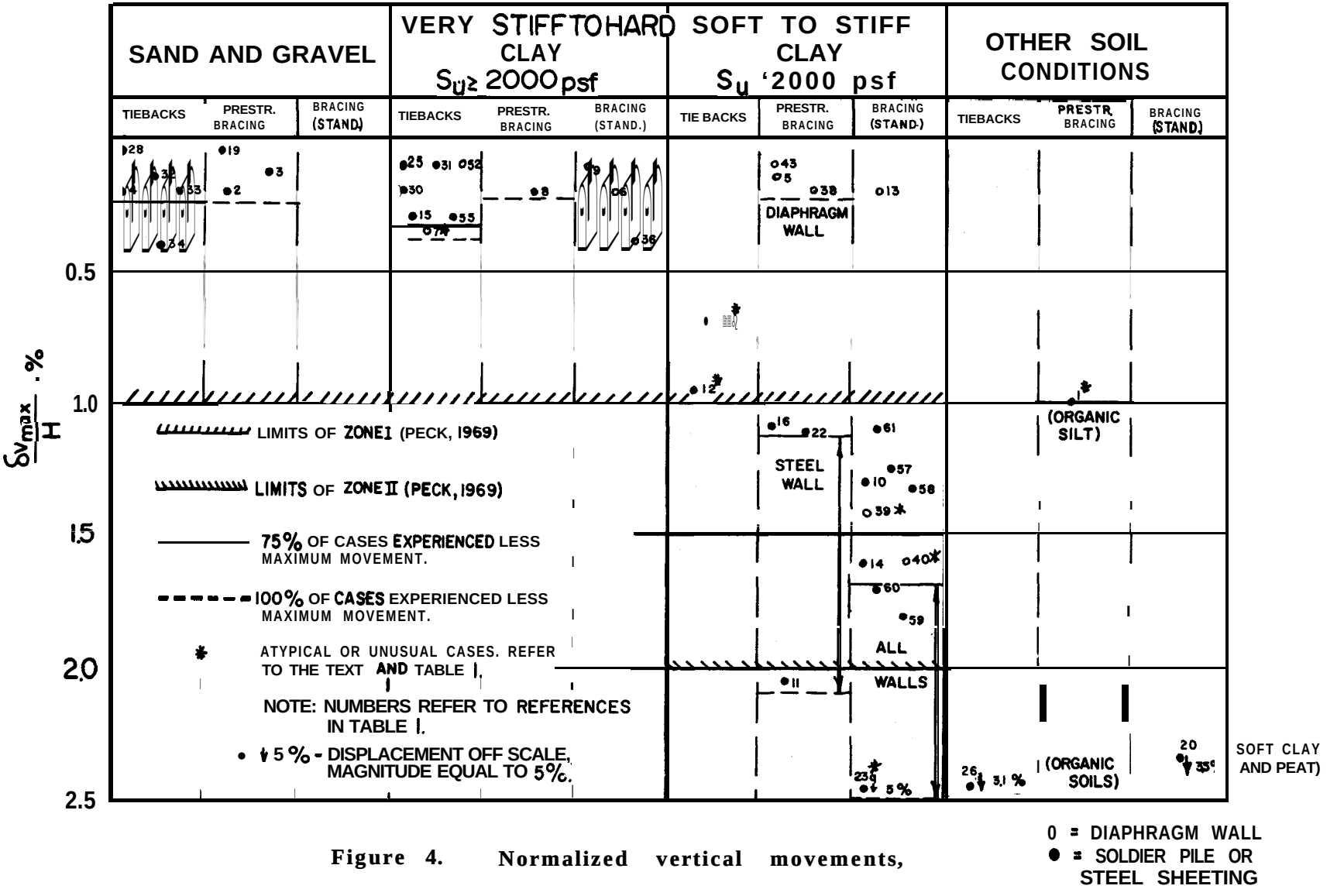


Figure 4. Normalized vertical movements,

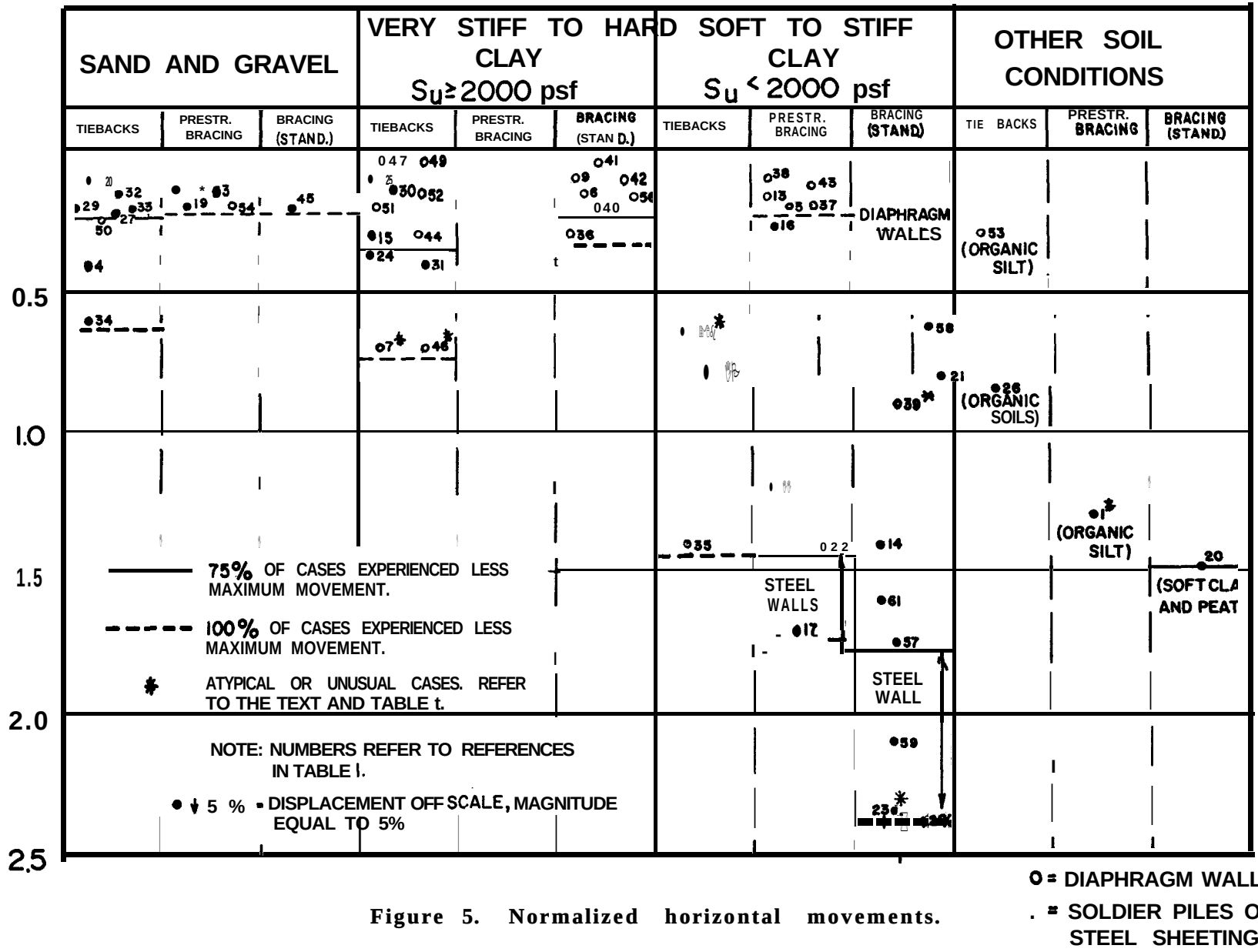


Figure 5. Normalized horizontal movements.

Table 1. Summary of references on displacement,

Ref.#	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of Cut	$\delta_{v_{max}}$	$\delta_{h_{max}}$	Comments
1	Lambe, Wolfskill, & Wong (1970)	SSP	Struts (Prestressed)	Fill, Organic Silt, till, rock	58' (17. 7m)	7" (17. 8cm)	9" (22.9cm)	Consolidation settlements significant. Settlements of 3" (7. 6cm) up to 70' (21.3m) from excavation.
2	O'Rourke and Cording (1974) (G St. Excavation)	SP	Struts (Prestressed)	Dense Sand and gravel, Stiff clay	60' (18.3m)	1.5" (3 , 8cm)	9" (2.3cm)	Removal of struts increased settlement from 0.9"(2.3cm) to 1.5" (3.8cm).
3	O'Rourke and Cording (1974) (7th & G Streets)	SP	struts (Prestressed)	Dense Sand and gravel, Stiff clay	82' (25m)	1.5" (3.8cm)	1, 25" (3.2cm)	Some tune-dependent consolidation settlements.
4	O'Rourke and Cording (1974) and Ware, Mirsky, and Leuniz (1973) (4th & G Streets)	SP	Tiebacks	Dense Sand and gravel, Stiff clay	40' (12.2m)	, 7" (1.8cm) 2" (5. 1cm)	2" (5 . 1cm)	Street settlements small while soldier piles settled due to down-drag from tiebacks. Soldier piles settled 2"(5. 1cm) maximum.
5	Lambe, Wolfskill and Jaworski (1972)	DW	struts (Prestressed)	Fill, hard to medium clay, till	50' (15.2m)	1" (2. 5cm)	1.2" (3. 0cm)	Minor consolidation settlements. School located 5'(1. 5m) from wall.
6	Burland (1974) and St. John (1974) (New Palace Car Park)	DW	Struts (Slabs poured as excavation proceeded)	Gravel and very stiff clay	52' (15,9m)	.6" (1.5cm)	1" (2.5cm)	
7	Burland (1974) and St. John (1974) (Neasden Under-	DW	Tiebacks	Very Stiff clay	26' (7. 9m)	1.1" (2,8cm)	2.2" (5. 6cm)	Much of the wall movement was pure translation and continued with time. Extremely small vertical settlements except directly behind the wall.
8	O'Rourke and Cording (1974) (11th & G Streets)	SP	struts (Prestressed)	Dense Sand and gravel and stiff clay	--	2 %	--	Did not report depth of excavation or amount of settlement.
9	Burland (1974) and St. John (1974) (London YMCA)	DW	Slabs and Tiebacks	Gravel and very stiff clay	52' (15,9m)	.5" (1.3cm)	.6" (1.5cm)	
10	N.G.I. (1962) (Oslo Technical School)	SSP	Struts	Soft to medium clay	19. 5' (5.9m)	3" (7.6cm)	--	Consolidation settlements due to lowering of head in underlying sand.
11	N. G. I. (1962) (Vaterland #2)	SSP	struts (Prestressed)	Soft to medium clay	36' (11m)	8.9" (22.6cm)	5.1" (13cm)	Nearby underpinned structure settled significantly.
12	McRostie, Burn and Mitchell (1972)	SSP	Tiebacks	Medium to stiff Clay	40' (12.2m)	4.5" (11.4cm)	-4" (-10.2cm)	Excessive tieback prestressing pulled wall away from excavation. Sensitive clay consolidated due to shearing stresses.
13	DiBiagio and Roti (1972)	DW	Floor slabs used to support wall	Medium clay	62' (18.9m)	1.6" (4. 1cm)	1-1.2" (2.5-3. Ocm)	Structure < 2'(0. 6cm) from wall. All settlement appeared to be due to lateral wall deflection.

See Sheer 5 for notes.

Table 1. Summary of references on displacement. (Continued.)

Ref. #	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of Cut	$\delta_{v_{max}}$	$\delta_{h_{max}}$	Comments
14	N. G. I. (1962) (Grønland #2)	SSP	struts	Soft to medium clay	37' (11.3m)	7" (17.8cm)	6.3" (16.0cm)	Part of excavation performed under water.
15	Shannon and Strazer (1970)	SP	Tiebacks	Very stiff clay and sand	78' (23.8m)	3" (7.6cm)	3" (7.6cm)	Maximum settlement measured at wall. Settlement may be due to downward force exerted by tiebacks.
16	Swatek, Asrow, and Seitz (1972)	SSP	struts (Prestressed)	Soft to stiff clay	70' (21.4m)	9" (22.9cm)	2.3" (5.8cm)	Large settlement attributed to localized heavy truck traffic. Typically settlements < 5" (12.7cm).
17	Rodriquez and Flamand (1969)	SSP	struts (Prestressed)	Soft to medium clay	37' (11.3m)	--	7.9" (20.1cm)	Staged construction to minimize movements. Dewatered to prevent bottom heave.
18	Scott, Wilson and Bauer (1972)	SSP	struts	Dense fine sands	50' (15.3m)	--	8" (20.3cm)	Poor performance attributed to poor construction techniques and dewatering problems. Nearby structures damaged.
19	Chapman, Cording and Schnabel (1972)	SP	Struts and Rakers (Prestressed)	Sand and gravel and stiff clay	45' (13.8m)	.25" (0.64cm)	1" (2.5cm)	Running soil encountered in one section.
20	Boutsma and Horvat (1969)	SSP	struts	Soft clay and soft peat	33' (10.1m)	14" (35.6cm)	6" (15.2cm)	Some settlement due to extensive dewatering for long time period. Affected structures 600' from excavation. Liquefaction of backfill during extraction.
21	Insley (1972)	SP	Rakers	Soft to medium clay	25' (7.6m)	--	2.5" (6.4cm)	One section tested to failure.
22	Tait and Taylor (1974)	SSP	Struts and Rakers (Prestressed)	Soft to medium clay	45' (13.8m)	6" (15.2cm)	7.5" (19.1cm)	Larger movements attributed to lack of firm bottom for wall. Utility lines damaged; no major damage to adjacent structures.
23a	Hansbo, Hofman, and Mosesson (1973)	SSP	Rakers	Soft clay	23' (7.0m)	13.8" (35.1cm)	11.8" (29.9cm)	Poor sheet pile interlocking. Long time between excavation of center portion and bracing. Disturbance during pile driving for foundation.
23b	Hansbo, Hofman, and Mosesson (1973)	SSP	Tiebacks and Rakers	Soft clay	23' (7.0m)	2" (5.1cm)	2" (5.1cm)	Improved construction techniques.
24	Prasad, Freeman, and Klajnerman (1972)	SP	Tiebacks	Very stiff clay	45 (13.8m)	--	-2" (-5.1cm)	Top of wall moved away from excavation. Maximum movement at top.
25	Mansur and Alizadeh (1970)	SP	Tiebacks	Very stiff to hard clay	45' (13.8m)	.5" (1.3cm)	.5" (1.3cm)	
26	Sandqvist (1972)	SSP	Tiebacks	Sand and silt with organic soils	19.5' (5.9m)	7.9" (20.1cm)	2" (5.1cm)	Settlement in organics due to lowered ground water level. Pile driving also caused settlement.

See Sheet 5 for notes.

Table 1. Summary of references on displacement. (Continued.)

Ref. #	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of cut	$\delta_{v\,max}$	$\delta_{h\,max}$	Comments
27	Sigourney (1971)	SP	Tiebacks	Clayey sand and hard clay	20-26' (6.1-7.9m)	--	.5" (1.3cm)	
28	Goettle, Flaig, Miller, and Schaefer (1974)	SP	Tiebacks	Dense sand and gravel	23' (7.0m)	.25" (0.64cm)	.25" (0.64cm)	Structure with footings only 2' (0.62m) from wall was undamaged.
29	Sigourney (1971)	SP	Tiebacks	Very dense silty sand and gravel	35.43' (10.7-13.2m)	--	.1" (0.25cm)	
30	Clough, Weber, and Lamont (1972)	SP	Tiebacks	Vary stiff clay	64' (19.6m)	1.25"± (3.2cm)	1"± (2.5cm)	Top of wall moved away from excavation.
31	Nelson (1973)	SP	Tiebacks	Sandy overburden, hard clay shales	90' (27.5m)	1" (2.5cm)	4" (10.2cm)	Cracking in street indicated potential stability failure ($\delta_{h\,max}$ [15.2cm]) Maljian & Van Beveren (1974).
32	Liu and Dugan (1972)	SP	Tiebacks	Dense sand and gravel, very stiff clay	55' (16.8m)	8"± (20.0cm)	1"± (2.5cm)	Tops of soldier piles pulled away from excavation during prestressing.
33	Larson, Willette, Hall, and Gnaedinger (1972)	SP	Tiebacks	Dense sand	50' (15.2m)	1" (2.5cm)	1" (2.5cm)	
34	Dietrich, Chase, and Teul (1971)	SP	Tiebacks	Silty sand	23-54' (7-16.5m)	2.5" (6.3cm)	1.8" (4.6cm)	Lateral movements measured at top of wall.
35	Cunningham and Fernandez (1972)	DW	Tiebacks	Medium clay under dense sand	23' (7.0m)	--	4" (10.2cm)	Tiebacks anchored to deadman.
36	Cole and Burland (1972)	DW	Rakers	Very stiff clay	60' (18.4m)	1.5" (3.8cm)	2.5" (6.3cm)	Most movements occurred while earth berm supported wall. Excavation in heavily overconsolidated clay.
37	Tait and Taylor (1974)	DW	Tiebacks, prestressed struts and rakers	Medium and soft clay	45' (13.8m)	--	9" (2.3cm)	Minor settlements of nearby structures
38	Armento (1973)	DW	struts (Prestressed)	Sand and soft to medium	70' (21.4m)	1.7" (4.3cm)	1" (2.5cm)	Some settlement may have been caused by other excavations in
39	Cunningham and Fernandez (1972)	DW	Struts	Soft and medium clay	32' (9.8m)	5.5" (13.9cm)	3.5" (8.9cm)	Underpinning of nearby footings required after 5.5" (13.9cm) of settlement. 50.70% of movement during caisson construction.
40	Tan (1973)	DW	Basement slab as support	Soft clay	43' (13.3m)	6"± (15.2cm)	--	Settlement estimated on basis of substantial damage to structure 40' (12.2m) from excavation.

See Sheet 5 for notes.

Table 1. Summary of references on displacement. (Continued.)

Ref. #	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of Cut	δ_v max	δ_h max	Comments
41	Breth and Wanoscheck (1969)	DW	struts	Hard clay and limestone	60' (18.4m)	--	.4" (1.0cm)	
42	Huder (1969)	DW	Basement slabs as support	Slightly plastic silt and clay	65' (19.9m)	--	1.4" (3.6cm)	
43	Thon and Harlan (1971)	DW	struts (Prestressed)	Soft to medium clay	78' (23.8m)	1" (2.5cm)	1.2" (3.0cm)	
44	Barla and Mascardi (19741	SW	Tie backs	Stiff clay	85' (25.9m)	- -	2.6" (6.6cm)	Cracking in nearby structures.
45	Heeb, Schurr, Bone, Henke, and Muller	SP	struts	Sand	40' (12.2m)	--	.8" (2.0cm)	
46	Breth and Romberg (1972), Romberg (1973)	SP	Tiebacks	Stiff clay and sand	68 (20.8m)	--	5.9" (14.9cm)	Lateral movement of entire soil block.
47	Schwarz (1972) and Andra, Kunzl, and Rojek (1973)	SW	Tie backs	Clayey marl (stiff clay)	97.5' (29.8m)	.2" (0.51cm)	.6" (1.5cm)	Many levels of tiebacks at very close spacing.
48	Corbett, Davies, and Langford (1974)	DW	Rakers	Very stiff clay; upper sand and gravel	--	--	.2" (0.51cm)	Construction delayed after hole opened.
49	Hodgson (1974)	DW	Tiebacks and struts	Fill, gravel very stiff clay	26' (7.9m)	--	.12" (0.3cm)	Special construction procedure used.
50	Corbett and Stroud (1974)	SP	Tiebacks	Fill, sand and marl	51' (15.6m)	--	.8" (2.0cm)	Heave observed 18m from wall.
51	Littlejohn and MacFarlane (1974)	DW	Tiebacks	Gravel and very stiff clay	18' (5.5m)	--	.8" (2.0cm)	
52	Littlejohn and MacFarlane (1974)	DW	Tiebacks	Gravel and very stiff clay	47' (14.4m)	.9" (2.3cm)	.9" (2.3cm)	
53	Saxena (1974)	DW	Tiebacks	Organic Silt and sand	55 (16.8m)	--	2.7" (6.9cm)	Tops of some wall sections moved toward sail by same amount.
54	Ware (1974) Personal communication	DW	struts (Prestressed)	Sand and gravel and stiff clay	62 (18.9m)	--	1.25" (3.2cm)	
55	Goldberg-Zoino & Assoc. Files	SP	Tiebacks	Fill, organic sand, stiff clay, till	45' (13.8m)	1.5" (3.8cm)	1" (2.5cm)	Vertical settlements due to lagging installation. Most horizontal movement away from excavation.
56	Burland (1974) and St. John (1974)	DW	Cantilever Wall	Very stiff clay	Small (7% settlements		.5" (1.3cm)	

See Sheet 5 for notes.

Table 1. Summary of references on displacement. (Continued.)

Ref.#	Author(s)	Wall Type	Bracing Type	Soil Type	Depth of Cut	$\delta_{v_{max}}$	$\delta_{h_{max}}$	Comments
57	N.G.I. (1962) Telecommunications Center	SSP	struts (Prestressed)	Medium and soft clay	26' (7.9m)	3.9" (9.9cm)	5.5" (13.9cm)	Significant movements after strut removal.
58	N.G.I. (1962) Enerhaugen South	SSP	struts (Prestressed)	Medium and soft clay	26' (7.9m)	4.2" (10.7cm)	2" (5.1cm)	Lateral deflections probably more than shown.
59	N.G.I. (1962) Vaterland #1	SSP	struts	Medium and soft clay	36' (11.0m)	7.9" (20.1cm)	9" (22.9cm)	
60	N.G.I. (1962) Grønland #1	SSP	Slabs as support	Medium to soft clay	37' (11.3m)	7.5" (19.0cm)	--	Air pressure and upside down construction used.
61	N.G.I. (1962) Vaterland #3	SSP	struts	Medium and soft clay	30' (9.2m)	3.9" (9.9cm)	5.9" (14.9cm)	
62	Maljian and Van Beveren (1974)	SP	Tiebacks	Stiff to very stiff clay and cohesive sand and silt	110' (33.6m)	3" (7.6cm)	2" (5.1cm)	Maximum vertical settlement atypical for the site--usually lateral movement greater than vertical.
63	Jennings (cases reported by Littlejohn and MacFarland [1974]) South Africa		Tiebacks	Firm	48' (14.7m)	--	3" (7.6cm)	Damage to utilities in street and building across street.
				Fissured	48 (14.7m)	--	1.5" (3.8cm)	Acceptable movements
				Clay	74' (22.6m)	--	1.5" (3.8cm)	
				Very stiff fissured clay	48' (14.7m)	--	.75" (1.9cm)	
				Soft jointed rock	59' (18.0m)	--	1" (2.5cm)	

Notes:

1. SSP = Steel sheet piling
   SP = Soldier pile wall
   DW = Diaphragm wall
   SW = Secant wall

2. δ_h and δ_v are maximum horizontal and vertical displacements.

3. Reference # represents references listed by author in Bibliography.

slurry trench walls, voids created from pulling of sheeting, etc. (See Volume III, Construction-Methods, for a more detailed discussion of various construction techniques.)

3. Consolidation of soil -- for example, densification of loose granular soils from vibration, or consolidation of soft cohesive soils from lowering of ground water outside the excavation.

4. Base instability or near instability -- excessive shear strains caused by the imbalance created by removal of load contribute to base heave and/or plastic conditions in soil.

5. Stress relief from excavation -- this reduces vertical stress below the base and relieves the K_o horizontal stress (earth pressure at rest). In turn, the possible displacement modes are base heave, shear strains, and lateral strains.

The tabulated performance data indicates the following:

1. Sand and Gravel; Very Stiff to Hard Clay

Seventy-five percent of the excavations in this material experienced horizontal movements less than 0. 35 percent of the excavation depth. On the average the clays experienced approximately 30 percent greater movement'(0.35 percent vs. 0. 25 percent of H) than the sands and gravels. Generally, the performance is not significantly affected by support method or by wall type. Sheet piling, however, is uncommon to these soil types due to difficulty in installation. One probable reason for little apparent difference between wall type and support method is the fact that the measured displacements are small (typically less than 0. 10 feet for a 50-foot excavation). Many construction factors can contribute to displacements of similar magnitude and therefore would mask the variation in displacement caused by wall support type.

Two anomalous cases (no. 7 and no. 46, Table 1) reveal a potential source of extraordinary lateral movement of a tied-back wall retaining a predominately very stiff or hard clay (see previous discussion in Section 2. 23 on tied-back walls). The mechanism causing this movement still is not clearly defined. However, the practical implications are to approach similar cases with caution. Ward (1972) cites horizontal strains as two to three times as large as vertical strains in overconsolidated London clay.

2. Soft to Stiff Clay

Wide variations for both horizontal and vertical displacements are evident. Sixty-five percent of the cases experienced horizontal displacements which exceed 1 percent for steel sheet pile or soldier pile walls, whether prestressed or not. The data suggest prestressing of these walls makes only minor difference.

The largest benefit is derived from concrete diaphragm walls with prestressed bracing. Indeed, both horizontal and vertical displacements are no different from those typical for sands and very stiff to hard clay, being about 0.25 percent or less.

A unique case is included in Table 1. In this case the wall was tied-back in a stratum stronger than the clay. The case reports on the performance of tiebacks (McRostie, et al - no. 12) anchored in underlying bedrock. The soil was a sensitive clay which experienced significant consolidation settlements due to excessive prestressing (McRostie, et al, 1972).

Another major cause of settlements in cohesive soils is due to lowering of the ground water table. These settlements can often be quite severe (Lambe, Wolfskill, and Wong, 1970; Boutsma and Horvat, 1969; NGI, 1962; Sandqvist, 1972).

2.32 Effect of Wall Stiffness on Lateral Displacements in Clay

Wall stiffness refers not only to the structural elements comprising the wall but includes the vertical spacing between the support members. The measure of wall stiffness is defined as the inverse of Rowe's flexibility number for walls $\frac{EI}{L^4}$

where:

E = modulus of elasticity of wall

I = moment of inertia. per foot of wall

L = vertical distance 'between support levels or between support level and excavation base

Concrete walls generally have a much higher EI value than soldier pile or sheet pile walls, and with comparable wale spacing, are much stiffer. On the other hand, soldier piles or sheet pile walls with closely spaced support levels may be stiffer than concrete walls with widely spaced support levels .

Figures 4 and 5 indicate that diaphragm walls reduce the magnitude of the movements in soft to stiff clay significantly below the magnitude of the movements for the more flexible sheet pile or soldier pile walls. In an attempt to further refine the effect of wall stiffness on displacements in cohesive soils, a plot of observed displacements versus corresponding stability number $(N = \frac{\gamma H^*}{S_u})$ and stiffness factor $(\frac{EI}{L^4})$ is developed on Figure 6. The stability number, which considers both overburden stress (γH) and the undrained shear strength (S_u), is a measure of the relative strength or deformability of the soil.

1. The maximum lateral displacement rather than the vertical has been plotted, since consolidation settlements would introduce a secondary variable.

2. The maximum value of N was calculated on the basis of the available strength data. H was taken as the depth of excavation to the lower Limit of an intermediate clay stratum where it intersected the wall.

3. The wall-support stiffness was based on the span distance, L, occurring where the "stability number, N, is a maximum. If N was a maximum at an intermediate excavation depth, L was calculated as the wale spacing plus 2 feet (for overcut). If N was a maximum at the excavation base, the span L was calculated from the lowest strut to the excavation base.

The data plotted in Figure 6 demonstrate what is intuitively obvious -- namely, deformations are functions of soil strength and wall stiffness. The contour lines of maximum lateral wall movement show this trend clearly. These data allow one to qualitatively

*Ratio of overburden stress to undrained shear strength.

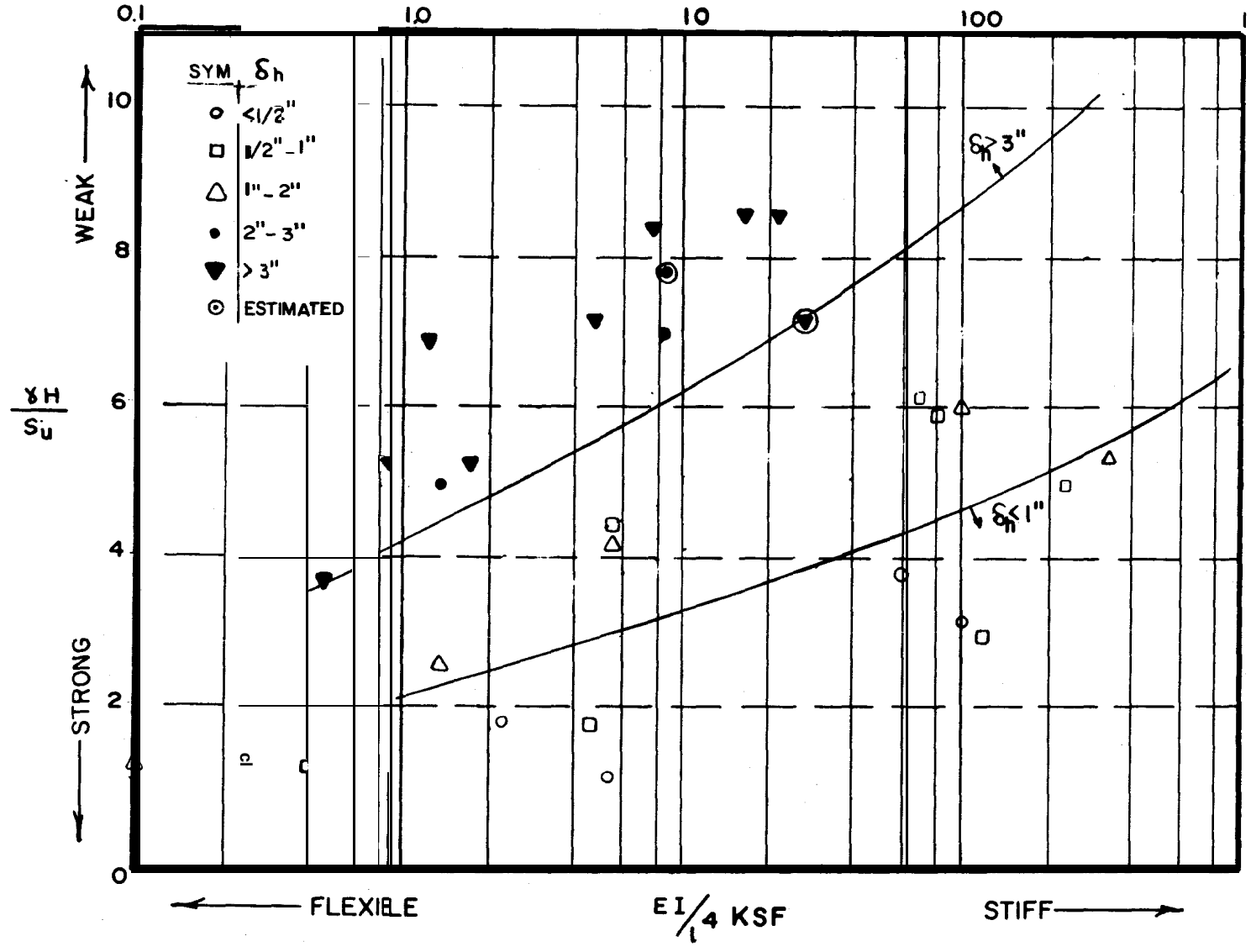


Figure 6. Effect of wall stiffness and soil strength on lateral wall deflection.

examine the relative change in anticipated lateral displacement for a given change in wall stiffness and/or stability number of the soil.

As an example, consider a soil with stability number of 6. Assume we are evaluating PZ-38 sheeting versus a 30-inch thick concrete diaphragm wall, both with 8-foot spans between wale levels. For an intermediate construction condition 2 feet is added to the span distance for overexcavation yielding a length, L=10 feet.

The stiffness factor of the steel sheeting (PZ-38) is:

$$\frac{EI}{L^4} = \frac{(30 \times 10^6)\,(281)}{(120)^4} = 40.7 \text{ psi} = 5.86 \text{ ksf}$$

Correspondingly, the stiffness factor of a 30-inch concrete wall is:

$$\frac{EI}{L^4} = \frac{(3 \times 10^6) \times 1/12\,(12 \times 303)}{(120)^4} = 391 \text{ psi} = 56.3 \text{ ksf}$$

The data in the figure show that the expected maximum lateral displacement for the PZ-38 is approximately 3 inches, whereas that for the stiffer diaphragm wall is approximately 1.5 inches.

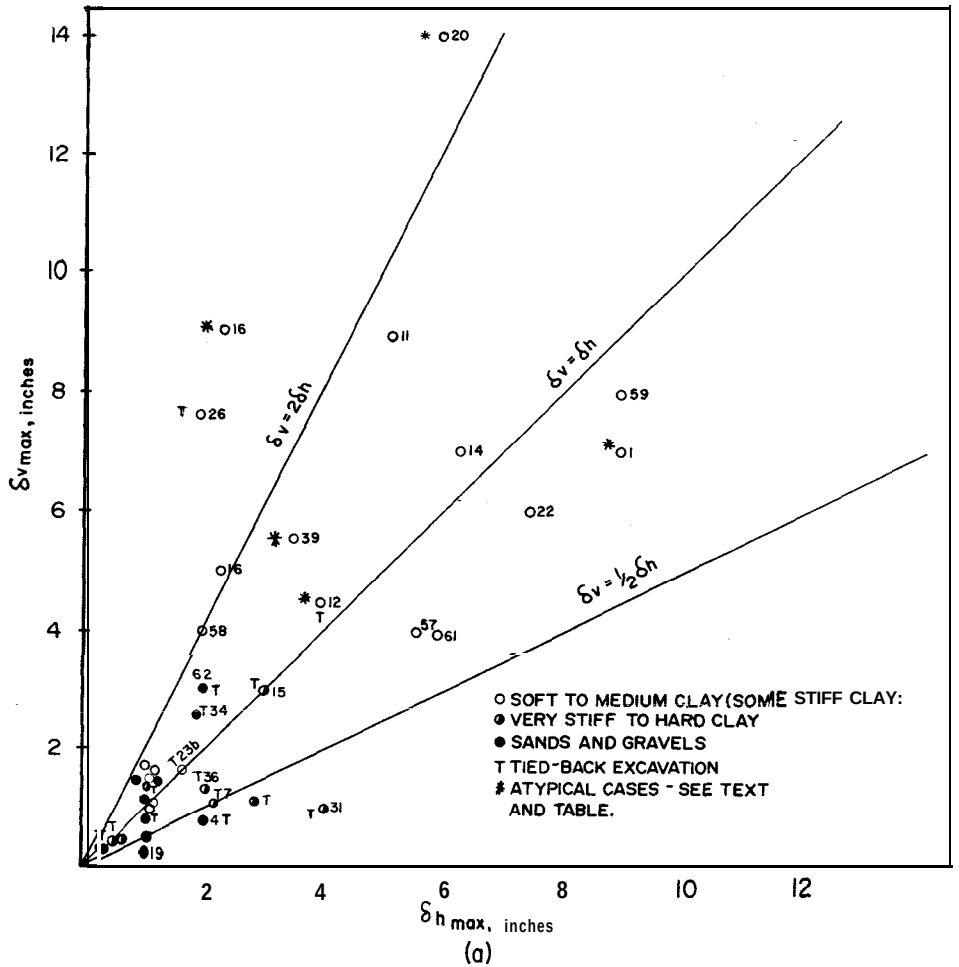
2.33 Wall Movement Versus Settlement

2.33.1 Comparison for all Cases

Figure 7 compares observed maximum horizontal and vertical displacements for all types of soils, support systems, and wall types. The absolute magnitude is shown in panel (a) and the frequency distribution of the ratio of the movements in panel (b). The figure shows that practically all the vertical displacements fall within a range of 1/2 to 1-1/2 times the horizontal displacements, with most of them lying in the range of 2/3 to 1-1/3 times the horizontal movement.

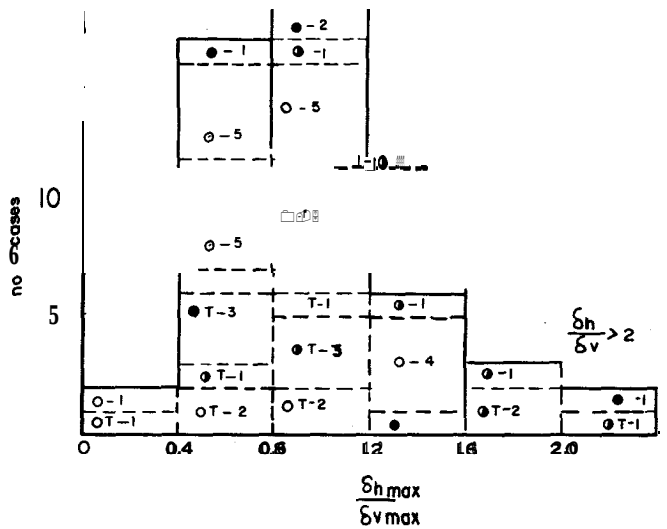
2.33.2 Soft to Medium Clay

Figure 8 compares displacements for soft to medium clays. The average curve shows that the vertical displacements are generally well in excess of the horizontal displacements and that the disparity increases with the magnitude of the displacements.



(a)

MAXIMUM VERTICAL AND HORIZONTAL DEFORMATIONS

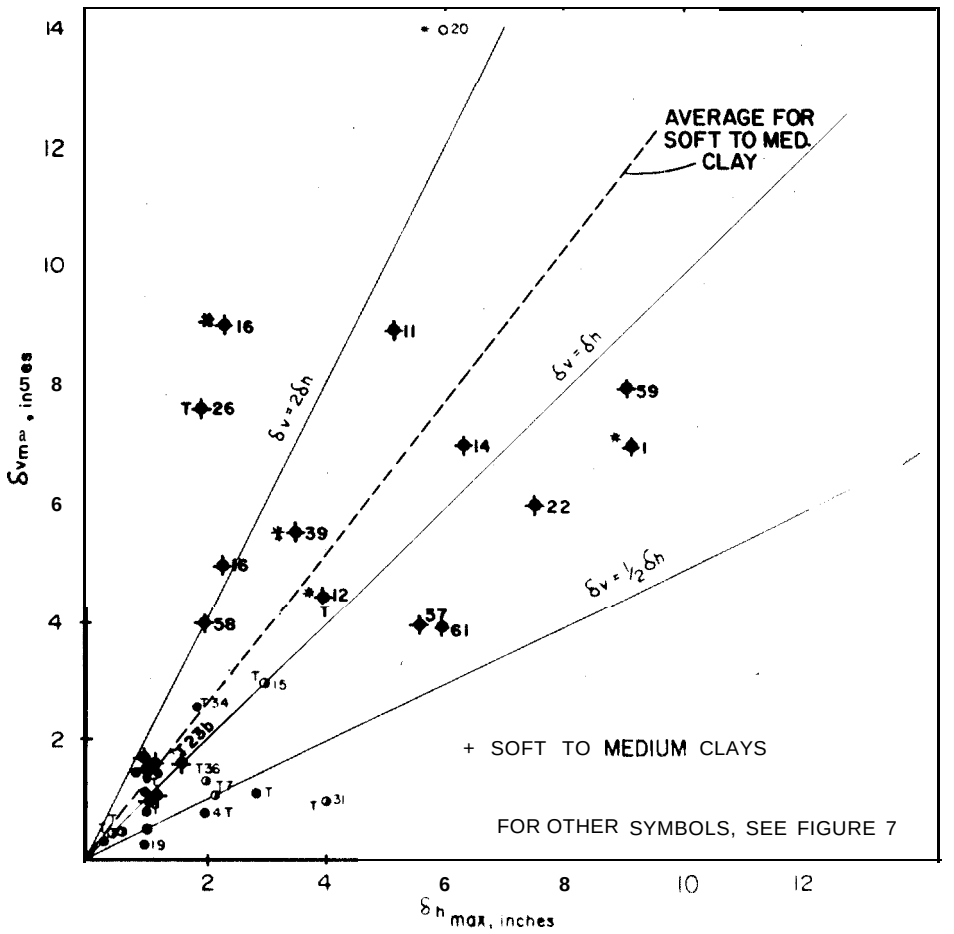


(b)

FREQUENCY DISTRIBUTION OF RATIO OF MOVEMENTS

NOTE: NUMBERS REFER TO CASE STUDIES LISTED IN TABLE I.

Figure 7. Comparison of maximum vertical and horizontal displacements.



NOTE: NUMBERS REFER TO CASE STUDIES LISTED IN TABLE I.

Figure 8. Comparison of vertical and horizontal displacements for soft to medium clays,

This difference is believed to be directly attributable to consolidation settlements which are usually the result of changes in water levels adjacent to the excavation. This situation becomes more acute where deep deposits of soft clay underlie the excavation.

2.33.3 Very Stiff to Hard Clays

Figure 9 compares the displacements of these . As mentioned in a previous discussion (Section 2.31), comparatively large lateral displacements have been reported in several tieback projects. Notable among-these are cases no, 7 (Burland and St. John), no. 31 (Nelson), and no.' 46 (Breth and Romberg),

2.40 PARAMETRIC STUDIES

Finite element studies are useful in providing qualitative information on the behavior of cofferdams. Several studies of this type have been undertaken for e'valuating the primary parameters affecting bracing loads and deformations (Wong, 1971; Palmer and Kenney, 1972; Jaworski, 1973; Clough and Tsui, 1974).

The results of a finite element study for evaluating the effect of wall stiffness on reducing deformations in various soil conditions is shown in Figure 10. Also shown for comparison are the lines defining deformation limits from Figure 6.

The finite element computer program used to develop these data considered only cohesive soils and internally braced excavations. A brief description of the program, its capabilities, and the soil properties used in the analysis is appended to this section.

Briefly, the conditions assumed in developing Figure 10 are:

1. The excavation was 60 feet deep, with a wall penetration of 30 feet. The wall was supported by five strut levels with approximately a 10 foot vertical span between each level.

2. Three wall stiffnesses were analyzed. The $\frac{EI}{L^4}$ ratios were 1. 00, 5. 8, and 230.

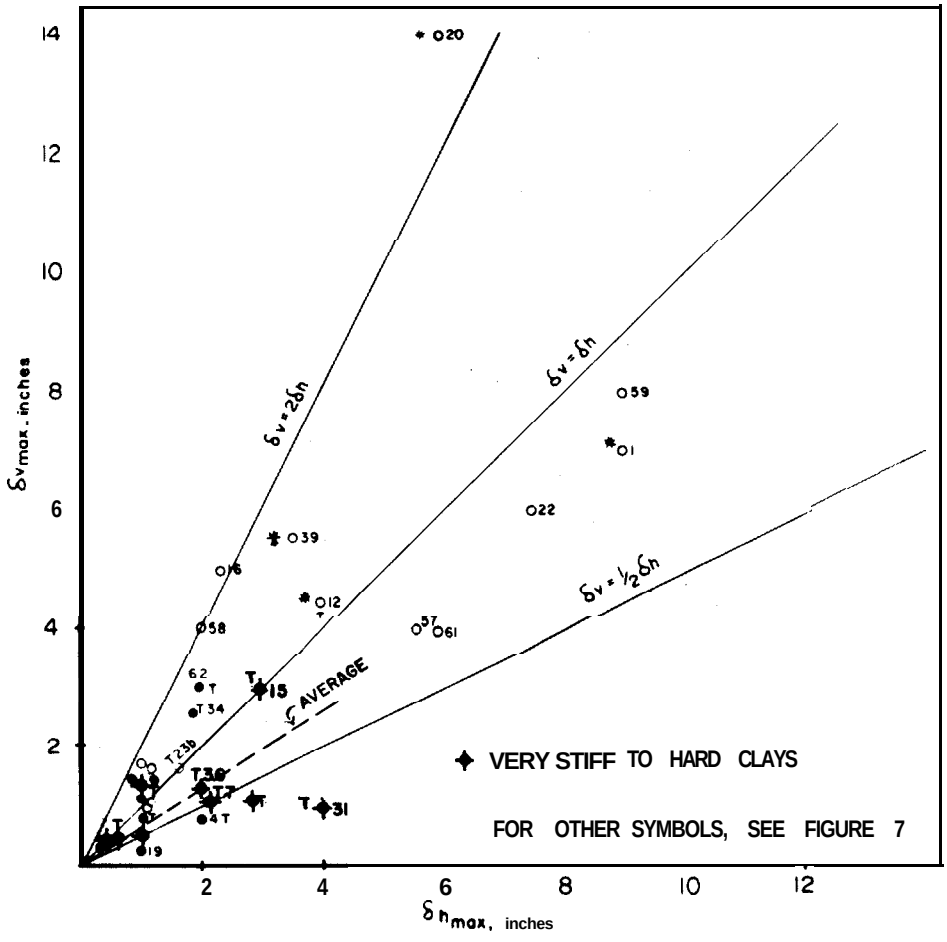


Figure 9. Comparison of vertical and horizontal displacements for very stiff to hard clays.

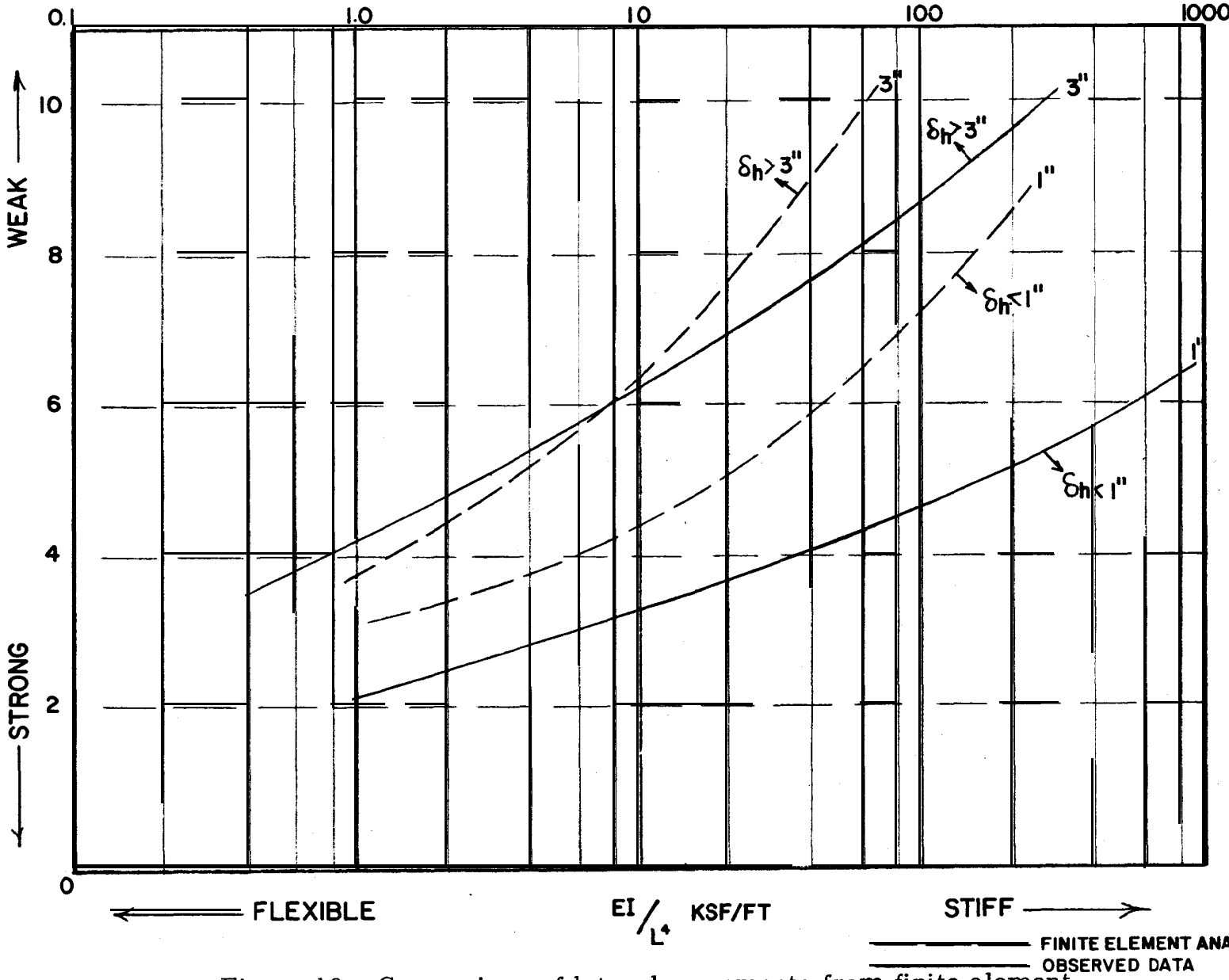


Figure 10. Comparison of lateral movements from finite element analysis and observed movements.

3. A uniform soil profile was assumed in which the shear strengths were varied to obtain different stability numbers (N). The value of N used to develop this plot was based on the shear strength of the soil at the base of the 60 foot deep excavation,

4. The finite element analysis models ideal conditions. It does not account for variations in construction procedure (such as overexcavation for a strut level) or anticipated construction events which are reflected in Figure 6.

Figure 10 shows the predicted lateral displacements are less than the observed values for a given condition. This difference is related to the inherent movements which are a function of the construction process. Nonetheless, the theoretical results show a trend similar to that described by the field observations; that is, the stiffer walls result in lower movements for a given soil condition.

The results of the finite element analyses should not be taken in the quantitative sense, The intent is that such analysis should be used as a guide in the design and in the consideration of various options for a bracing system.

2. 50 DISTRIBUTION OF DEFORMATIONS

In addition to knowing what the maximum lateral and vertical displacements will be for a cut, it is also important to know the influence zone of these deformations adjacent to an excavation. Primarily, this is related to the distortion the deformations may impart to a structure, for if it is anticipated that differential displacements will result in structural distress, then alternate procedures should be considered.

Currently many engineers rely on judgement and experience in predicting deformation patterns adjacent to sheeted excavations. It is the intent of this section to provide some information to aid the engineer in evaluating what deformation patterns might be expected adjacent to a cofferdam.

2. 51 Vertical Deformations

Peck (1969) suggested envelopes for the zone of influence of settlements behind an excavation based on field measurements. The envelope showed that significant vertical movements may occur up to a

distance of twice the excavation height from the excavation face depending on workmanship and soil profile, A refinement of this plot was undertaken to provide more information on settlement patterns adjacent to cofferdams. This was accomplished through a series of normalized plots of vertical deformations versus distance from the excavation face for three general soil classifications.

Figure 11 illustrates how the observed maximum settlement patterns behind a wall varied with the soil conditions. The pattern of movements indicates that maximum movements occur immediately adjacent to the excavation. Also, one might expect significant movements a distance from the cut equal to twice the depth of the cut. At present, there are insufficient data to define any significant difference in settlement pattern based on soil type or support wall.

Comparing the settlement patterns of sand versus cohesive soil, the sands show essentially no settlement beyond twice the depth of the excavation whereas the cohesive soils do. This is most likely caused by the consolidation in the more compressible soils from lowering of the ground water table. A factor to consider when viewing these results is that when settlements are small consolidation can be a large percentage of the total settlement; hence, in dimensionless plots the total settlement may appear to extend over a greater zone than is attributable to lateral movement or shear strain alone. This is evident in the data presented by Lambe, Wolfskill, and Jaworski (1972). The measured settlements due to drops in ground water table appear significant even though the settlements were small (less than 1 inch). On the other hand, when the maximum settlements are large (NGI, 1962 Oslo Technical School), consolidation settlements do not appear as significant. The data is further influenced by variations in the surface elevation caused by ground water fluctuations, freeze-thaw cycles, and other factors which will also be a greater percentage of the maximum observed movement in those cases where $\delta_{v_{max}}$ is small.

Reviewing Figure 11, it appears that both soft clays and the granular soils experience a significant angular distortion outside a distance equal to the excavation depth (D/H = 1). The average lines of settlement ratio versus normalized distance, shown as dashed lines on the figure, may be used as a basis of comparison of this distortion. On the other hand, the stiffer clays (S_u > 2000 psf) seem to experience more gentle distortion slope, even though the zone of influence

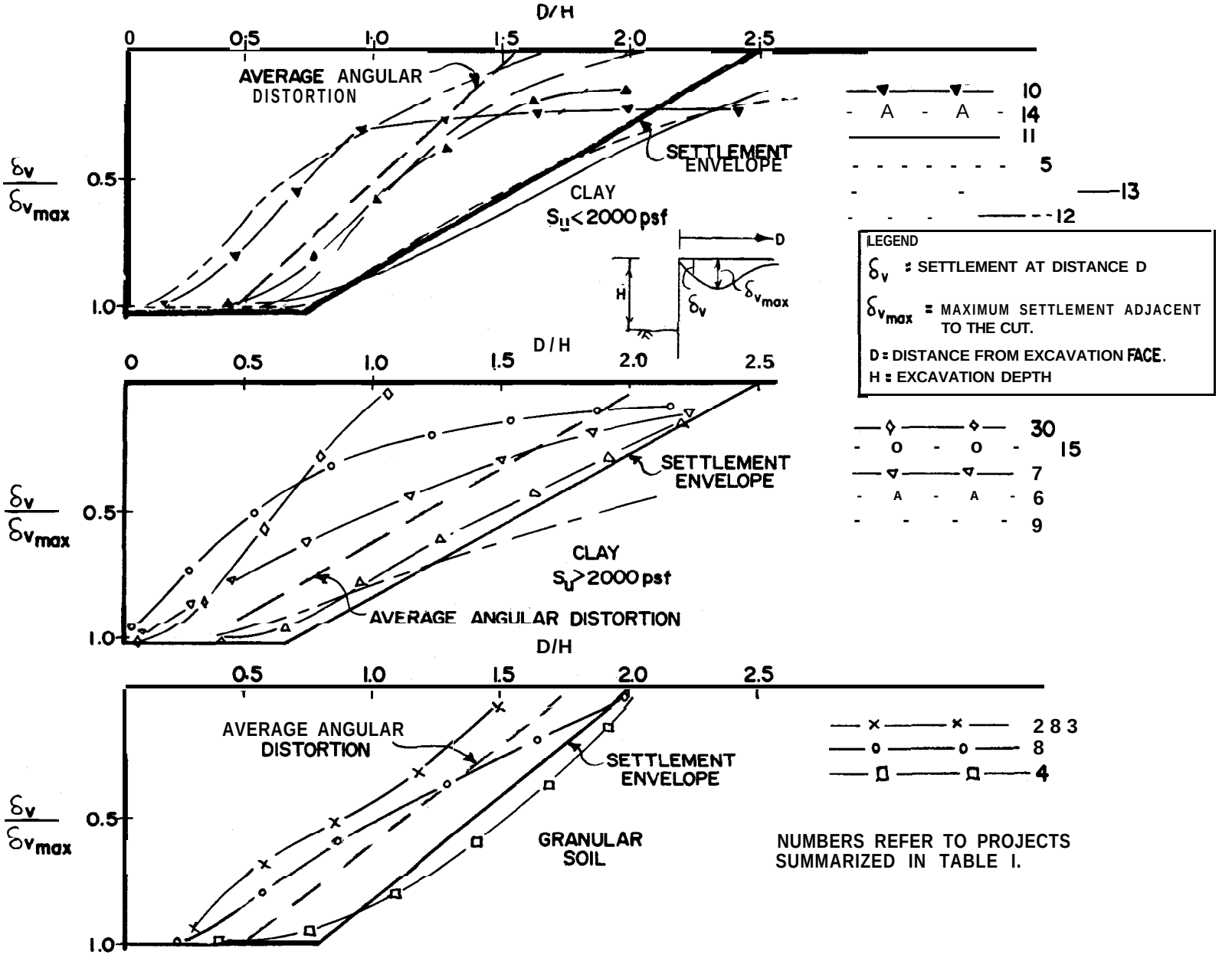


Figure 11. Normalized settlements adjacent to a wall.

extends farther back from the excavation face. Of course these figures do not show the absolute value of angular distortion. This is determined by estimating $\delta_{v_{max}}$ from Figure 4, based on excavation depth and method of wall support. Then the angular distortion is related to the differential settlement by:

$$\alpha = \alpha_n \times \delta_{v_{max}}$$

where:

α = differential settlement over distance D

α_n = normalized differential settlement from Figure 11

$\delta_{v_{max}}$ = maximum settlement from Figure 4.

2. 52 Parametric Study on Zone of Influence

Finite element studies were performed on several of the deformation modes shown in Figure 1. The se analyses were aimed at obtaining some qualitative information on the settlement profile one might expect adjacent to the excavation. The analytical approach used was to apply incrementally a specified mode of horizontal wall displacement to a soil profile and, using the finite element program BRACE II, determine the induced settlement profile. The results of this analysis portray only those constant volume settlements associated with the wall displacement and ignore settlements associated with consolidation of the soil.

Figures 12 through 14 show the wall deformations assumed and the corresponding settlement profiles predicted by the finite element pr 0 gram. The results are reported in dimensionless form. The basic patterns of deformation used were: tilting about the base, rotation about the top, symmetrical bulging, a combination of bulging with tilting, rotation and lateral translations.

The se patterns, although ideal, typify the more common modes of deformations experienced in braced and tied-back walls. In those cases where bulging of the wall was used to represent the flexural deformation of the wall, the bulging was assumed to be symmetrical around a depth equal to two-thirds of the excavation depth.

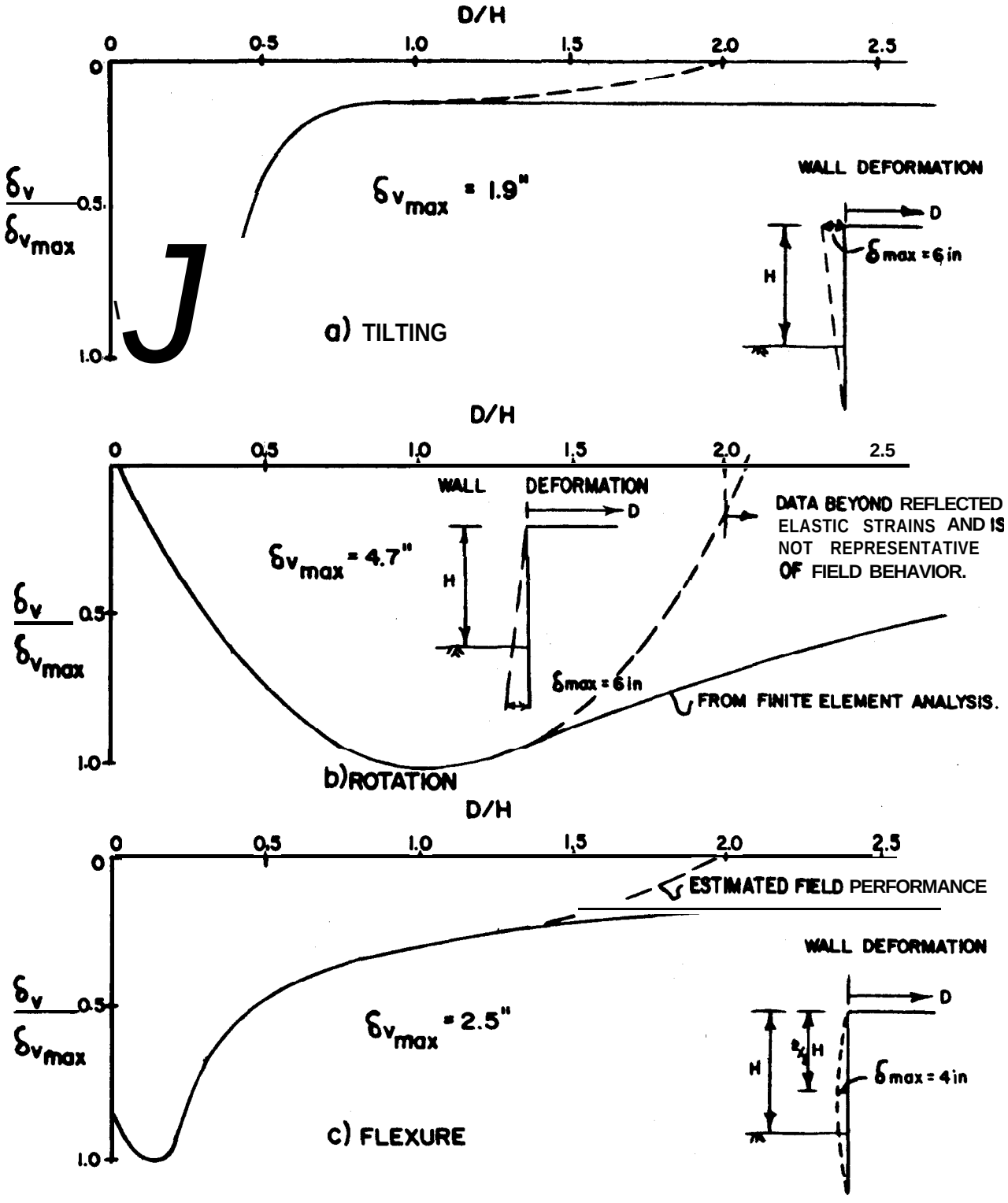


Figure 12. Finite element prediction of surface settlement profiles for normally consolidated clays,

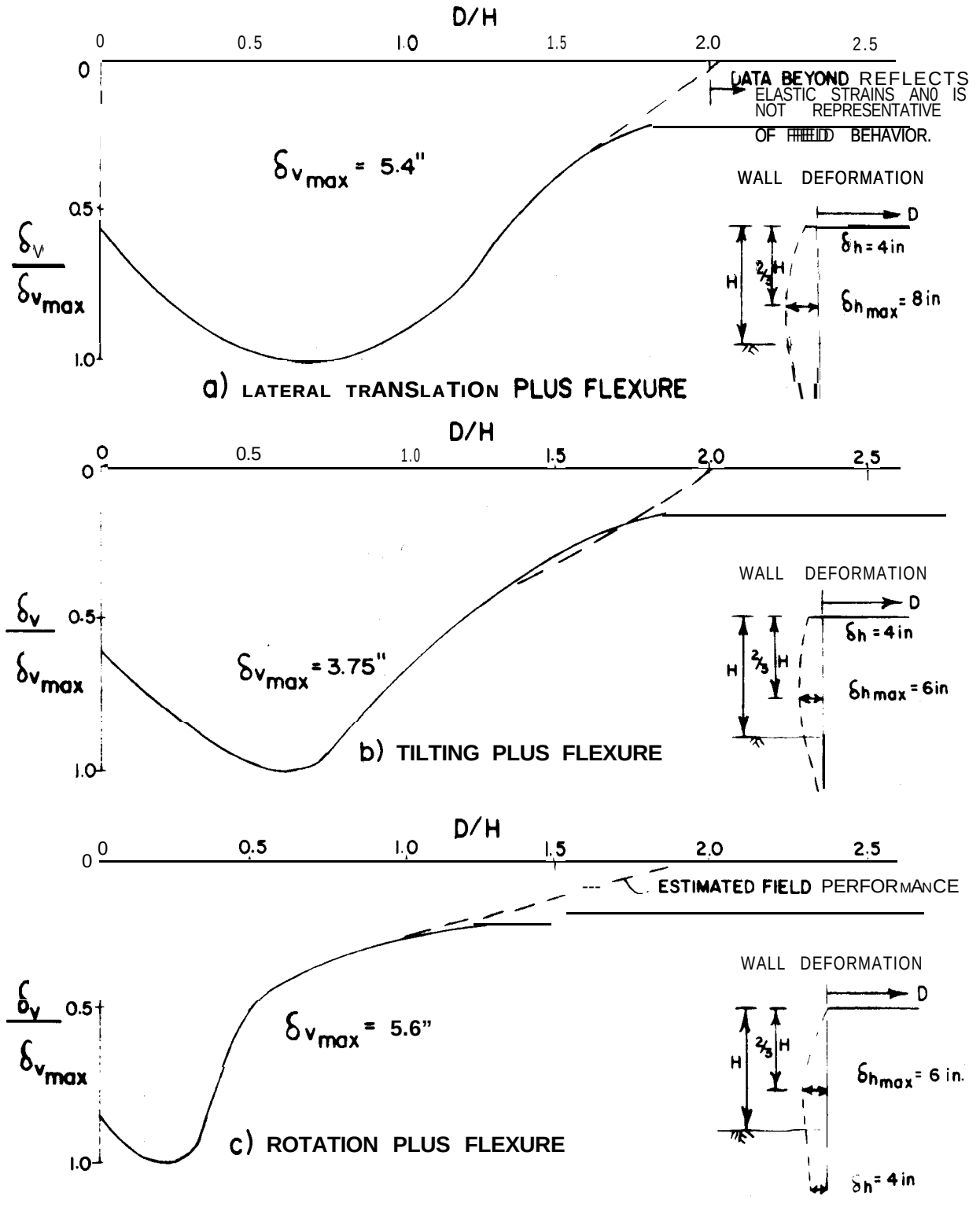


Figure 13. Finite element prediction of surface settlement profiles for normally consolidated clays,

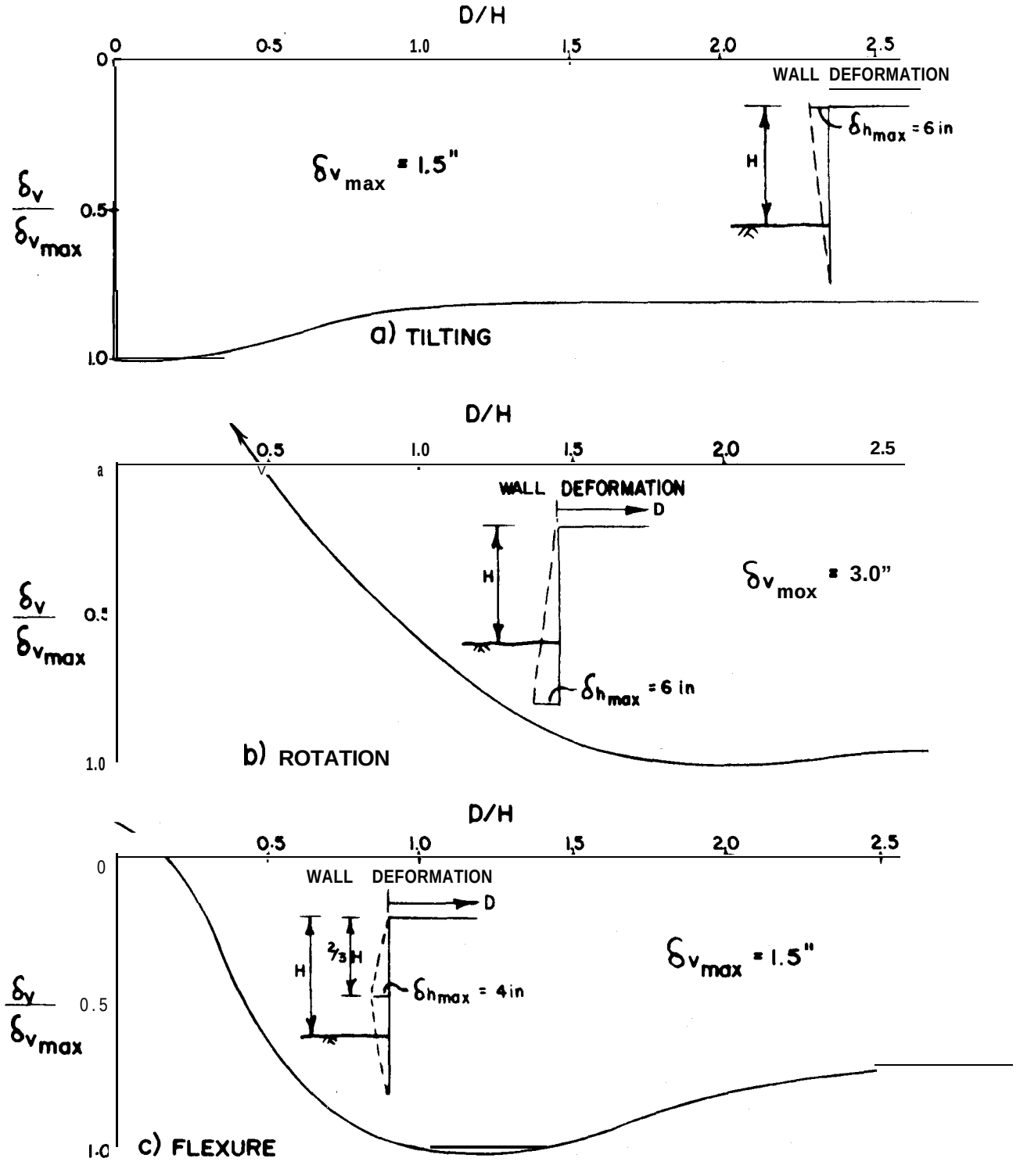


Figure 14. Finite element prediction of surface settlement profiles in elastic medium,

Two soil conditions were analyzed:

(a) Normally consolidated clay with both the soil strength and soil modulus increasing with depth as a function of the effective overburden stress.

(b) **Elastic** medium where the soil was assigned a constant modulus with depth. The elastic cases were analyzed primarily to provide background information to evaluate the extent to which the elastic strains influence the results from the yielding soil profiles of normally consolidated clay.

In addition, the elastic cases provide some insight into the settlement patterns for cohesive soil profiles where the stability number is less than 4. The parameters for the cases studied are summarized in Table 3 in the appendix to this section,

Figure 12 illustrates settlement profiles for ideal tilting, rotation, and bulging. The first two conditions may be considered representative of rigid wall behavior, whereas the bulging cases represent deformations associated with a flexible wall.

The results indicate that for tilting and flexure the settlements are concentrated within a distance **one -half the excavation depth. Settle**ments beyond this distance are elastic and probably not representative of actual field conditions. For both cases, the severe angular distortion occurs within this zone as expressed by the slope of the settlement profile. On the other **hand, when rotation is the predominant mode of** deformation, significant deformations may occur at distances up to 1. 5 the excavation depth from the excavation face, This should be expected since the maximum lateral deformation is deep. The over-stressed zone of soil is also deep, hence leading to a greater zone of **influence.** However, the settlement profile for this latter case suggests a less severe angular distortion along the ground surface.

Figure 13 shows the settlement profiles for wall deformations which are a combination of rigid wall displacement plus flexural deformations **(example,** tilting plus Qexure shown in Figure 13b). The results show the zone of influence is greatly affected by the nature and volume encompassed by the horizontal wall movement. This behavior agrees with the measured field performance of braced excavations (Flaate 1966). The data also indicate that as the zone of influence

increases from the excavation face, the settlementprofile will have a gentler slope. Hence, even though the area affected increases, the actual danger to a structure may not be as severe since the angular distortion is less.

Figure 14 shows the settlement pattern which results when the soil is assumed to behave elastically. For all three wall deformations the vertical displacements become essentially horizontal beyond the distance twice the excavation depth, Comparing Figures 12 and 13, the same trend of constant displacement occurs on the latter two figures beyond this same distance, thus suggesting these deformations are related to elastic strains. The elastic strains are transmitted to this outer zone by tensile stresses which are not capable of developing in soils. This idiosyncrasy of finite element analyses leads to behavioral trends which are not in keeping with field observations. Consequently, the settlements shown on Figures 12 and 13 beyond a distance of twice the excavation depth should be discounted as not representative of field conditions.

A comparison of Figure 13 with the field measurements summarized on Figure 11 shows some interesting trends. First, the finite element predictions give zones of influence and distribution of settlements similar to those recorded in the field. Considering the deformations beyond a distance 2. 0 D/H from the excavation as primarily elastic in origin, the finite element analysis gave a zone of influence which ranged between 0.5H and 2. OH from the excavation face, Correspondingly, the field measurements show a greater zone since their results do contain some consolidation settlements not accounted for in a finite element analysis, Nonetheless, it appears that the finite element programs can be used to give qualitative information on settlement profiles.

The effect wall movement has on the zone of influence is another significant trend. Figure 12 and Figure 13 both show the importance of minimizing movement below the excavation base which is often associated with base instability or local overstressing adjacent to the wall. (Chapter 8 discusses both types of instability in detail). For all cases it appears that the deeper the maximum movement is seated, the greater the zone of influence will extend from the excavation face.

2.60 LATERAL DEFORMATIONS IN ADJACENT SOIL MASS

Lateral movements of a structure have been observed to be more damaging than vertical movements. Therefore, one should attempt to evauluate the extent of lateral movement which may occur as the result of constructing a temporary retaining structure. These movements are most prevalent in heavily overconsolidated clays which have large residual horizontal stresses which are released as a result of the excavation.

Intuitively, one would think that the lateral movements would be a maximum at the face of the wall and decrease with distance from the wall. Also, the deflected wall shape would have some bearing on the distribution of lateral displacements in the soil mass. Unfortunately, very few measurements are available showing the distribution of lateral deflection behind a wall.

A few normalized contour plots of horizontal deformation are presented in Figures 15 and 16. The measurements were made in heavily overconsolidated clays where tied-back walls were used to support the excavation. The observations show that the lateral movements were time dependent. In another similar case, (Burland, 1974 and St. John, 1974) where only the Lateral movements at the surface were monitored, the measured horizontal movements were 20 per cent of the maximum movement of 0.5 inches at a distance 1.5H from the excavation.

The aforementioned field data suggest two trends. First, the pattern of the lateral movement follows closely with the deflected shape of the sheeting. Second, the lateral movements can extend a substantial distance from the excavation face as illustrated by the data from Burland (1974) and St. John (1974).

Another factor to consider with respect to tied-back walls in heavily overconsolidated clays is that the entire soil mass embodied within the tiebacks may move laterally (Burland, 1974; St. John, 1974; Breth and Romberg, 1972; and Romberg, 1973). Hence, in these types of soils the tied-back excavation may not be as successful in limiting wall movements as they would be in other soil types.

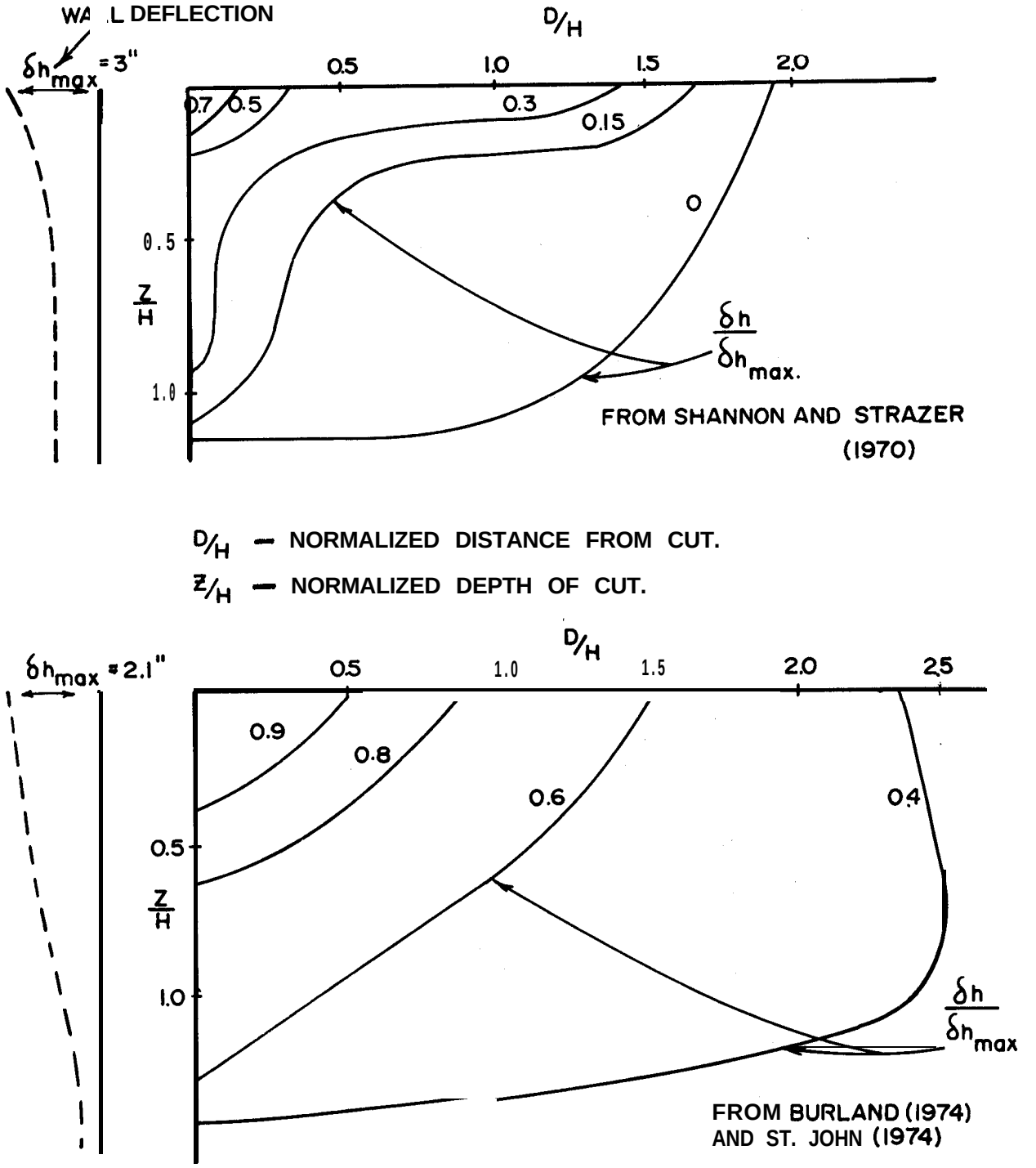


Figure 15. Normalized lateral movement for tied-back excavation in heavily overconsolidated clays.

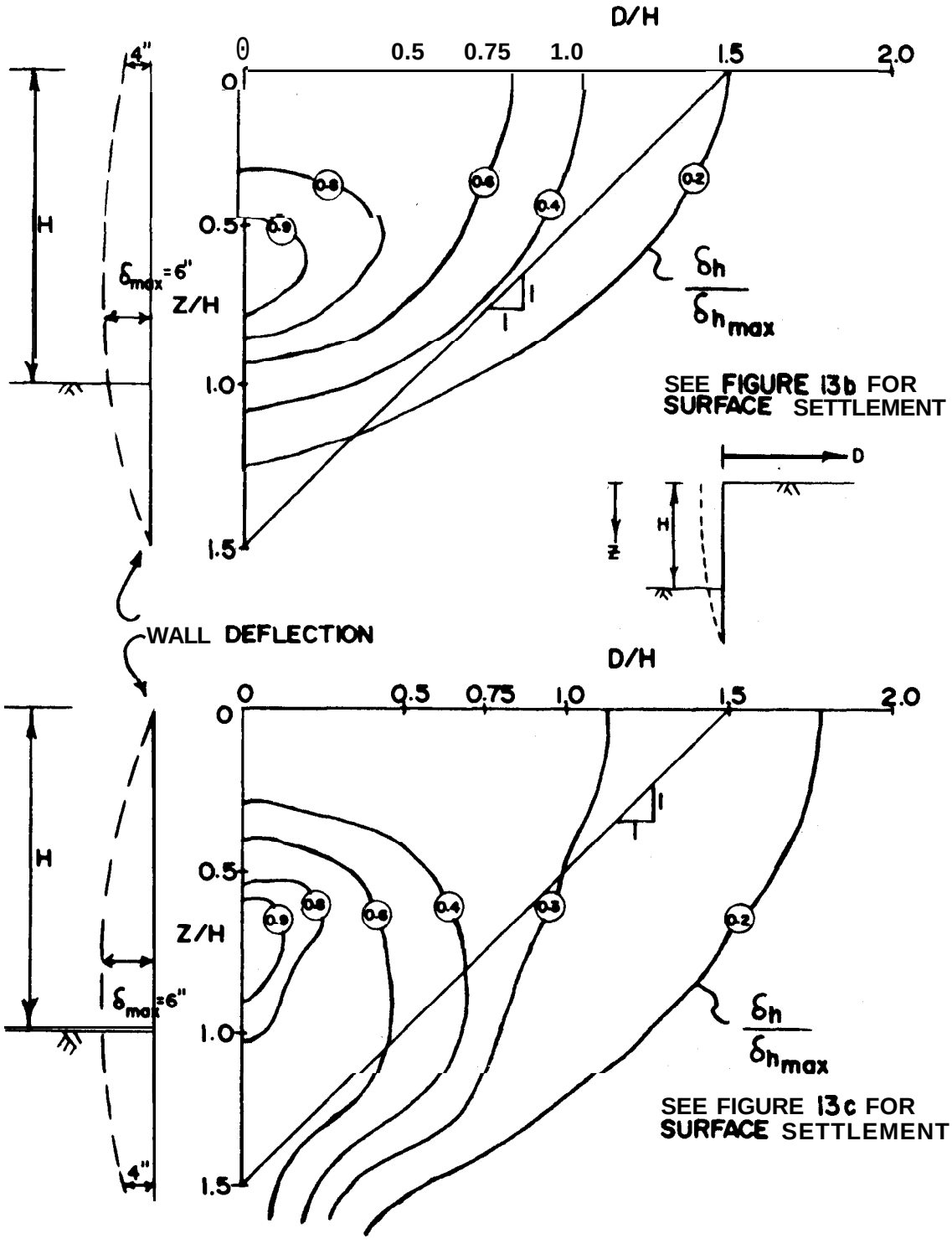


Figure 16. **Normalized Lateral movements from finite element analysis for normally consolidated clays.**

There are little data available regarding the distribution of horizontal displacements for excavations in a normally consolidated clay for comparison with the observed data for the heavily over-consolidated clays. Therefore, the results of the finite element studies used to develop Figure 13 were reduced to provide some insight as to the distribution which might be expected for ideal conditions. These results are shown in Figure 16. In contrast to the data from Burland (1974) and St. John (1974) for heavily over consolidated clays, the finite element analysis indicates that in this normally consolidated soil the zone of significant movement is confined to an area described by a 1 on 1 slope from the base of the sheeting. As expected, it is within the theoretical yield zone. The movements are largely controlled by the sheeting displacement.' The zone of significant movements increases with depth in the same pattern as the sheeting movements.

2.70 EFFECT OF CONSTRUCTION FROCEDURES

It is well known that construction procedures can have a significant effect on the performance of excavations.

Lowering of the ground water level either by pumping or by seepage into the excavation can result in significant settlements. These settlements could be associated with consolidation of the soil or, in the case of granular soils, the piping of soil into the excavation.

Poor installation techniques for tiebacks or struts can lead to surface settlements. Tiebacks should be carefully drilled to minimize the soil removed from holes. Also, any voids remaining after the tieback is installed should be filled with grout. Struts, rakers, and wales should be tightly wedged and preloaded to prevent movement of the wall. In addition, hard wood or steel wedges should be used for shimming to reduce movements caused by crushing. Earth beams when used to provide temporary support before installing a strut have been observed to be of little value in preventing wall movement. Cole and Burland (1972) and Hansbo, Hofmann, and Mosesson (1973) report cases where earth berms did little to restrict wall movement.

Even though the entire support system may be in place, the sides of the excavation may continue to creep inward with time. This problem appears to be particularly acute in tied-back walls in very stiff to hard

clays. There is also some evidence to indicate that lagging in soldier pile walls tends to pick up more load with time in all soils. Excessive bulging or even failure of some lagging has been observed,

2.80 ESTIMATING SETTLEMENTS

The data presented in the section may be used to obtain rough estimates of the ground movements which might occur adjacent to a support wall. The reason for making this estimate is to provide some additional input to aid in the decision of whether or not to underpin adjacent structures or utilities.

Settlements may be estimated using both Figure 4 and Figure 11. Once the soil type and excavation geometry are defined, an estimate of the maximum settlement may be made from Figure 4. Figure 11 provides a means of estimating the angular distortion and zone of influence of the ground movements. In the case of cohesive soils, Figure 5 may be used to estimate the wall stiffness necessary to limit the settlements.

2.90 SUMMARY

A review of available field measurements shows that wall and soil movement at the site of a temporary cut are influenced by the soil conditions, wall stiffness, vertical support spacing, prestressing, and construction procedures. For any given wall any one of these may be the most important factor. However, for situations were good construction procedures and typical wall types are used, Figures 4 and 5 indicate that the magnitude of maximum vertical and horizontal deflection is dependent on wall stiffness and method of support in soft to stiff clays, but independent of these factors for walls in sands and gravels or in very stiff clays.

In clays with an undrained shear strength of less than 2000 psf, flexible walls (soldier piles and lagging or steel sheeting) commonly experience vertical settlements in excess of 1.0 percent of the excavation depth. The magnitude of the se settlements can be reduced to less than 0. 75 percent of the excavation depth by strictly controlled construction procedures (Hansbo, Hofmann, and Mosesson, 1973). Where stiff support wall systems (such as diaphragm walls) are used in these soils, the settlements were less than 0.3 percent of the excavation depths.

The maximum settlements for all wall types in sands and gravels is typically less than 0. 25 percent of the excavation depth. For cuts in very stiff clays, the maximum settlements may be slightly larger although most of the maximum movements are still less than 0.25 percent of the height of the cut.

Concerning lateral wall deformations, the maximum for walls in sands and gravels is typically less than 0. 2 percent of the cut height. However, the lateral wall movements in very stiff clays are somewhat larger, reflecting the tendency for these soils to creep laterally with time. This behavior is most prevalent in situations where tiebacks were used in very stiff clays. For the very stiff clays, the maximum lateral movements for internally braced cuts are about 0.2 percent of cut height while maximum lateral movements for tied-back walls are generally less than 0.4 percent of cut height.

In general, Figure 4 shows that the stiffness of the wall-support system aids in controlling movements in virtually all soil types, although the effect is much less marked in sands, gravels, and very stiff clays.

Figure 11 shows how the vertical surface deformations vary with distance behind a cut. This data indicates that maximum settlements can occur at distances equal to the excavation depth from the support wall. To date, settlement profiles versus depth have not been measured. Also, it is not known to what extent adjacent structures affect the observed settlement profiles. In two cases (Lambe, Wolfskill, and Jaworski, 1972; DiBiagio and Roti, 1972), the settlement profiles were determined from the settlement of structures on shallow foundations. These settlement patterns are very similar to those for other cases.

APPENDIX A to CHAPTER 2

A. 10 INTRODUCTION

Finite element analyses of complex engineering problems are becoming more common each day in the engineering world. These analyses are often conducted to gain insight into the parameters which control the performance of a structure. As part of the development of this manual, computer analyses of braced excavations were made using the finite element program known as BRACE II (Jaworski, 1973). A brief description of this program's capabilities along with the details of the computer runs used to develop the data in this manual are presented in this appendix.

A. 20 BRACE 11 • FINITE ELEMENT PROGRAM

A. 21 General

The computer pro gram, BRACE II, was developed expressly for the purpose of analyzing internally braced excavations. The program simulates the construction process for a braced excavation in an isotropic, bilinearly-elastic material by performing a total stress analysis. It models sequentially the events of excavation and installation of struts. It can also consider such effects as prestressing of struts or additional movements at the strut levels after installation. Additional capabilities of the program include the facilities for handling anisotropic, bilinearly-elastic materials and the overstressing of the sheeting piling.

A, 22 Program Description

The use of the finite element technique in analyzing complex engineering problems is well described in the literature (Zienkiwiez, 1967). Briefly, this technique models a problem by an assemblage of discrete triangular to quadralateral elements. The forces $(\underset{\sim}{Q})$ at the nodes of these elements is related to the node displacements $(\underset{\sim}{U})$ and the global stiffness $(\underset{\sim}{K})$ of the element assemblage. A system of linear equations results which can be described by the equation:

$$(\underset{\sim}{K})\,(\underset{\sim}{U}) = (\underset{\sim}{Q})$$

This system of equations is solved to obtain nodal displacements. These displacements are then used to determine individual element strains and stresses.

This technique was used in the development of a computer program, BRACE II, for analyzing the behavior of braced excavations. This current program is a second generation of the original program, BRACE, developed by Wong (1971). The program models soil by discrete elements with bilinearly-elastic stress/strain properties. Within each element the strain is assumed constant. The retaining wall for the excavation is simulated by one-dimensional linearly elastic bar elements. The program simulates a specified excavation and bracing-construction sequence by applying the load relief due to an excavation stage or by applying a force at a node as a strut is prestressed. The loads from a particular construction operation are applied incrementally. Fbr each load increment a specified modulus is used until the yield strength of the soil is attained. Thereafter, a reduced modulus is used for each additional load increment.

Additional capabilities of BRACE II are: The retaining wall is allowed to develop a plastic hinge when a specified yield moment is exceeded; the soil behind the wall is allowed to slip unrestrained relative to the sheeting. Both capabilities are important in the analysis of braced excavations. In braced excavations in soft clay with large strut spacings, the sheeting may become overstressed and large movements will result. In cases where a concrete slurry wall is installed, a bentonite clay cake remains between the concrete and the soil. Since this clay has essentially no shear strength, restraint of slippage between the soil and concrete wall may be considered non-existant.

If, during a given excavation stage, the yield moment of the sheeting is exceeded at a bar element node, that node is made a plastic hinge for all additional incremental loads. Unrestrained slippage between the soil and the sheeting is modelled by setting the axial stiffness of the sheeting to zero.

The anisotropic variation of modulus of the soil is simulated based on the methods proposed by Christian (1971). He recommends the following appr oxirnate relation to account for the effect of stress reorientation on the undrained modulus:

$$E_\theta = E_{uh} - (E_{uh} - E_{uv}) \cos^4 \theta$$

where E_{uh} and E_{uv} are the undrained modulus for tests with the major principle stress applied to the soil in the vertical and horizontal directions respectively; θ is the orientation of the major principle stress from the vertical plane; and E_θ is the modulus used to compute the deformations of the soil.

To account for anisotropic shear strength properties the following yield criterion recommended by Davis and Christian (1971) was used:

$$\left(\frac{\sigma_x - \sigma_y}{2} - \frac{S_{uv} - S_{uh}}{2} \right)^2 + \tau_{xy}^2 \frac{a^2}{b^2} = a^2$$

where:

$$\frac{b}{a} = \sqrt{\frac{S_{u45}}{S_{uv}\, S_{uh}}}$$

$\sigma_x, \sigma_y, \tau_{xy}$ = conventional total stress components in the x, y plane

S_{u45} = shear strength of a soil sample oriented at 45 degrees from the vertical

S_{uv}, S_{uh} = shear strength for compression in the vertical and horizontal directions

A. 23 Summary of Parametric Studies

The details of the parametric studies used to generate the data in Chapter 2 and Chapter 7 are summarized in Tables 2 and 3. The tables show the sheeting stiffness and corresponding soil parameters used in each analysis. The soil parameters were selected based on synthesized data presented by Ladd, et al (1971) and Ladd and Vallaroy (1965). The soil parameters used in the analysis are not exactly those reported by Ladd, et al (1971), rather they reflect values which give stability conditions which would yield a broad spectrum of excavation performance.

The finite element grid used to model this excavation is shown in Figure 17. The soil mass was restrained against both vertical and horizontal deformations along the bedrock base (150 foot base) and at a distance of 250 feet from the excavation face. Excavation stages consisted of excavating approximately 3 feet below strut level. The struts were installed without prestressing; but, once the strut was installed, no further lateral movement was permitted at the strut level.

Table 2. Summary of case studies for analysis of strut loads and sheeting deformations.

Case	Soil Profile	Wall Type	Depth of cut (ft)	Penetration (ft)	$\frac{EI}{L^4}$ @ base	$\frac{\gamma H}{S_u}$ @ base	SOILS					
							Type	K_o	S_{uv}	S_{uh}	E_{uv}	E_{uh}
CASE I	Uniform	PZ-38 Steel	60	29	5.a3	6.a2	Soft Uniform	0.5	$0.28\bar{\sigma}_v$	$0.15\bar{\sigma}_v$	$200\bar{\sigma}_v$	$120\bar{\sigma}_v$
	Uniform	PZ-38 Steel	60	29	5.a3	4.78	Medium Uniform	0.6	0.406"	0.250,	$290\bar{\sigma}_v$	$180\bar{\sigma}_v$
	Uniform	26.5" Diaphragm Wall	60	29	38.40	6.a2	Soft Uniform	0.5	$0.28\bar{\sigma}_v$	0.156_v	$200\bar{\sigma}_v$	$120\bar{\sigma}_v$
	Uniform	26.5" Diaphragm Wall	60	29	38.40	4.78	Medium Uniform	0.6	0.406_v	$0.25\bar{\sigma}_v$	$290\bar{\sigma}_v$	$180\bar{\sigma}_v$
	Uniform	48" Diaphragm Wall	60	29	228.96	6.a2	Soft Uniform	0.5	$0.28\bar{\sigma}_v$	$0.15\bar{\sigma}_v$	$200\bar{\sigma}_v$	$120\bar{\sigma}_v$
	Uniform	48" Diaphragm Wall	60	29	228.96	4.78	Medium Uniform	0.6	$0.40\bar{\sigma}_v$	$0.25\bar{\sigma}_v$	$290\bar{\sigma}_v$	$180\bar{\sigma}_v$
CASE II	Uniform	PZ-38 Steel	60	10	5.a3	2.25	Stiff Uniform	1.0	3,200 psf	3.200 psf	3,000,000 psf	3,000,000 psf
	Soft Over Stiff	PZ-38 Steel	60	29	5.a3	2.25	Soft	0.5	800	540	800,000	500.000
							Stiff	1.0	3,200	3,200	3,000,000	2.500.000
	Soft Over Stiff	PZ-38 Steel	60	10	5.a3	2.25	Soft	0.5	800	540	800,000	500,000
							Stiff	1.0	3,200	3,200	3,000,000	2.500.000
	Soft Over Stiff	48" Diaphragm Wall	60	29	228.96	2.25	Soft	0.5	800	540	800,000	500,000
							Stiff	1.0	3,200	3,200	3,000,000	2,500,000
CASE III	Stiff Over Soft	PZ-38 Steel	60	29	5.a3	6.37	Stiff	1.0	3.200	3.200	3,000,000	2.500.000
							Soft	0.5	$0.3\bar{\sigma}_v$	$0.2\bar{\sigma}_v$	$360\bar{\sigma}_v$	$216\bar{\sigma}_v$
	Stiff Over Soft	48" Diaphragm Wall	60	29	228.96	6.37	Stiff	1.0	3,200	3,200	3,000,000	2,500,000
							Soft	0.5	$0.3\bar{\sigma}_v$	$0.2\bar{\sigma}_v$	$360\bar{\sigma}_v$	216%
	Stiff Over Soft	PZ-38 Steel	32.5	17.5	1.52	4.30	Stiff	1.0	2,000	2,000	2,400,000	2,400,000
							Soft	0.5	0.38"	$0.2\bar{\sigma}_v$	$360\bar{\sigma}_v$	$216\bar{\sigma}_v$
	Stiff Over Soft	48" Diaphragm Wall	32.5	17.5	59.60	4.30	Stiff	1.0	2,000	2.000	2,400,000	2,400,000
							Soft	0.5	$0.3\bar{\sigma}_v$	$0.2\bar{\sigma}_v$	$360\bar{\sigma}_v$	$216\bar{\sigma}_v$

Notes: For all cases γ_{sat} = 120 pcf
GWT = 5.0 ft. below GL
Vertical strut spacing = 10'

Table 3. Summary of case studies for analysis of zone of influence of deformations.

Type of Movement	Maximum Lateral Movement	K_o	S_{uv}	S_{uh}	E_{uv}	E_{uh}
Top tilt	6"	1.0	$0.3\bar{\sigma}_v$	$0.2\bar{\sigma}_v$	$300\bar{\sigma}_v$	$200\bar{\sigma}_v$
Rotation about top	6"	1.0	$0.3\bar{\sigma}_v$	$0.2\bar{\sigma}_v$	$300\bar{\sigma}_v$	$200\bar{\sigma}_v$
Flexure	4"	1.0	$0.3\bar{\sigma}_v$	$0.2\bar{\sigma}_v$	$300\bar{\sigma}_v$	$200\bar{\sigma}_v$
Top tilt and flexure	6"	1.0	$0.3\bar{\sigma}_v$	$0.2\bar{\sigma}_v$	$300\bar{\sigma}_v$	$200\bar{\sigma}_v$
Rotation and bulge	6"	1.0	$0.3\bar{\sigma}_v$	$0.2\bar{\sigma}_v$	$300\bar{\sigma}_v$	$200\bar{\sigma}_v$
Lateral shift and flexu re	8"	1.0	$0.3\bar{\sigma}_v$	$0.2\bar{\sigma}_v$	$300\bar{\sigma}_v$	$200\bar{\sigma}_v$
Top tilt	6"	1.0	elastic	elastic	3,000,000	2,500,000
Rotation	6"	1.0	elastic	elastic	3,000,000	2,500,000
Flexure	4"	1.0	elastic	elastic	3,000,000	2,500,000

Notes: For all cases:
γ = 0.01 psf
gwt = 150' below G. L.
depth of cut = 60'
depth of penetration = 29'
Poisson's ratio = $\nu_{hh} = \nu_{vh}$ = 0.499

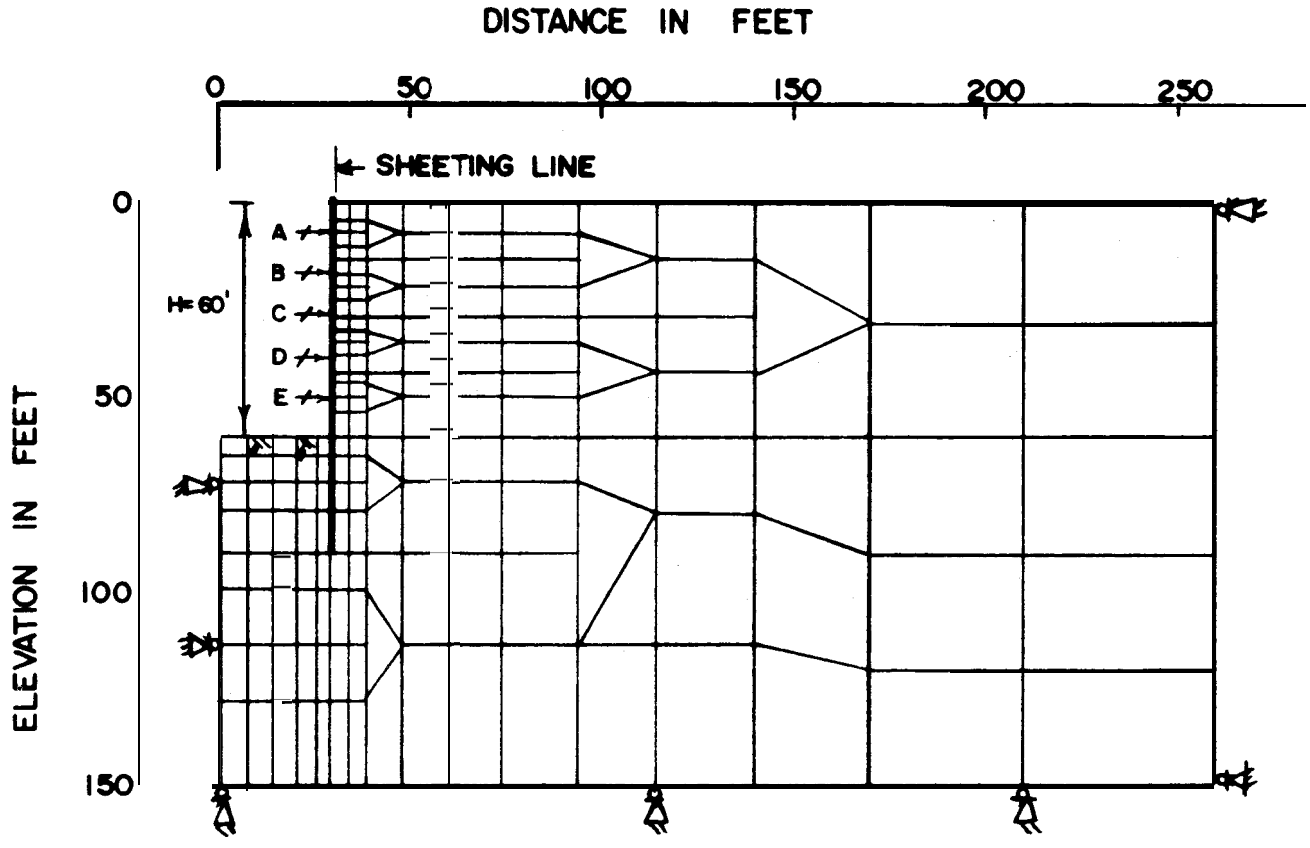


Figure 17. Finite element grid used in BRACE II analysis.

CHAPTER 3 - BASIC CONCEPTS OF SOIL MECHANICS

3.10 GENERAL

This chapter briefly summarizes typical properties and stress behavior of various soil types. Emphasis is placed upon basic concepts controlling behavior, particularly as related to the strength properties. This discussion should not be construed as a substitute for more comprehensive treatment given in soil mechanics texts, nor should the data concerning typical soil parameters obviate the need for soil testing. Rather, the data are given only as guidelines.

Soils may be classified under two broad categories, cohesive and cohesionless soils. In the cohesive category are clays which have low permeability and hence drain slowly. Most sands and gravels are classified as cohesionless. Silts and cohesive sands are an intermediate type whose engineering properties, especially strength, are largely controlled by their rate of drainage. Because of the general relationship between permeability and plasticity, the latter is a useful index for classification of these intermediate soils.

3.20 EFFECTIVE STRESS

Soil behavior is controlled largely by the effective stress in the soil. Effective stress is defined as follows:

$$\bar{\sigma} = \sigma - u$$

where:

$\bar{\sigma}$ = effective stress

σ = total stress

u = pore water pressure

Where $\bar{\sigma}_v$ is the vertical effective stress and $\bar{\sigma}_h$ is the horizontal effective stress, the in situ horizontal and vertical effective stresses are related by the at-rest coefficient of stress, K_o, defined as the ratio of the initial horizontal to the vertical effective stresses.

$$K_o = \frac{\bar{\sigma}_h}{\bar{\sigma}_{v_0}}$$

Soil strength parameters are governed by effective stresses which make the evaluation of pore pressure an important consideration in any engineering analysis.

Changes in pore water pressure generated by shear strain will dissipate immediately in cohesionless soil. Thus, the pore water pressure is controlled simply by the depth below the hydrostatic water level.

With cohesive soil, pore water pressure dissipates very slowly, in perhaps months or years. Moreover, one cannot accurately predict changes in pore water pressure caused by shear strain. As a corollary one cannot, with confidence, predict effective stress conditions in cohesive soil before the start of construction. On the other hand, pore water pressure measurements during construction do provide a basis for computing effective stress.

3.30 SOIL UNIT WEIGHTS

The effective stresses in a soil mass depend upon the total unit weight of the soil (γ_m), stress due to surcharge, and the pore pressures within the soil mass (often controlled by the water table). The unit weight of soil is a function of its specific gravity, void ratio, and water content.

The following table gives a range of unit weights for various soils for use in analysis.

Typical Values of Unit Weight

Soil Type	Moist Unit Weight (above water table) γ, pcf	Saturated Unit Weight* (below water table) γ_{sat}, pcf
Poorly graded sands	105 - 115	115 - 125
Clean well graded sand	115 - 125	120 - 130
Silty or clayey sands	120 - 130	125 - 135

Soil Type	Moist Unit Weight (above water table) γ, pcf	Saturated Unit Weight* (below water table) γ_{sat}, pcf
Silty or clayey sands and gravels	125 - 135	130 - 145
Soft to medium clay	100 - 115	100 - 115
Stiff to very stiff clay	110 - 125	110 - 125
Organic silt or clay	90 - 100	90 - 100

*Submerged unit weights = $\gamma_{sat} - \gamma_{w} = \gamma_{sub}$

3.40 SHEAR STRENGTH OF SOILS

3.41 General

Coulomb has defined soil shear strength in the general form of the effective strength parameters $\bar{c}$ and $\bar{\phi}$, where $\bar{c}$ is the cohesion intercept and $\bar{\phi}$ is the angle of internal friction as determined in laboratory tests. The shear strength of a soil is:

$$\tau_{ff} = \bar{c} + \bar{\sigma}_{ff} \tan \bar{\phi}$$

where:

τ_{ff} = shear stress on the failure plane at failure

σ_{ff} = effective normal stress on the failure plane at failure

Figure 18 illustrates the failure envelope.

3.42 Cohesionles s Soils

In cohesionless soils (sands and gravels) the cohesion intercept, c, is zero. Therefore, the shear strength can be expressed as:

$$\tau_{ff} = \bar{\sigma}_{ff} \tan \bar{\phi}$$

Figure 19 illustrates the failure envelope for a typical. cohesionless soil.

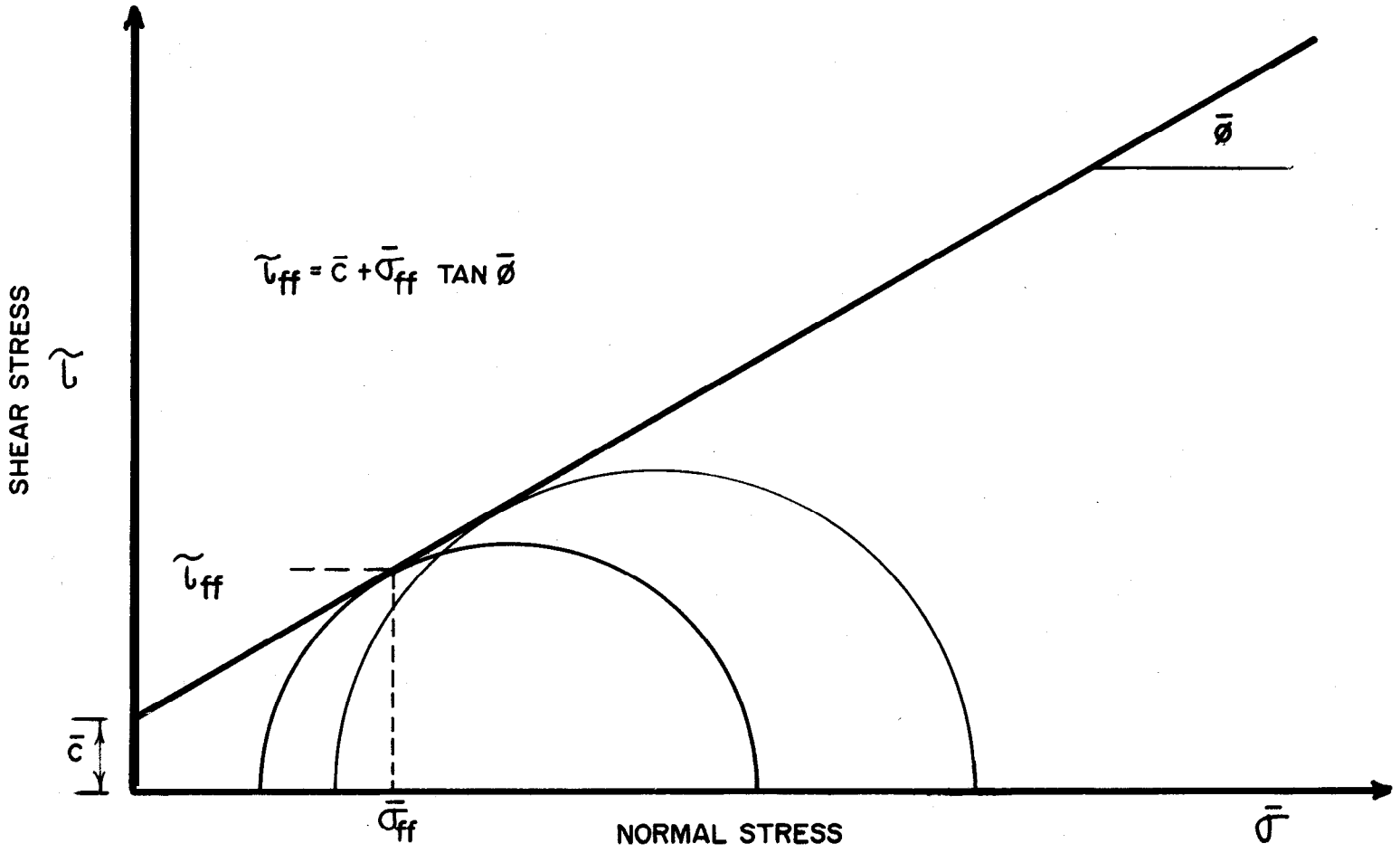


Figure 18. Failure envelope for soil, general Mohr-Coulomb failure criterion.

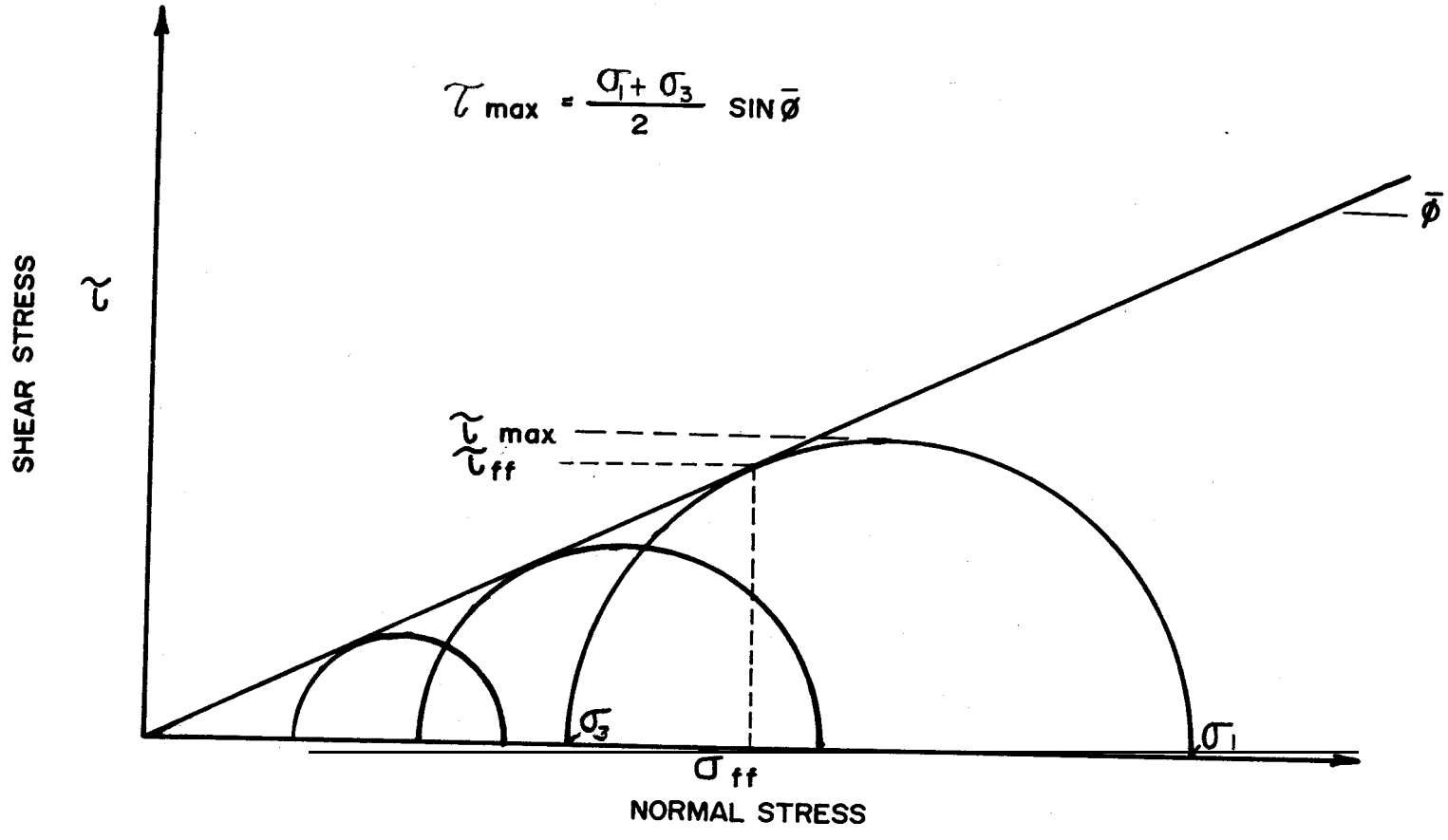


Figure 19. Strength envelope for cohesionless soils (based on effective stress).

As mentioned previously, sands and gravels are free drawing and therefore, pore pressure changes generated by loading or shear strains dissipate quickly. Therefore, the shear strength of the soil is related to the loading condition and the $\bar{\phi}$ value. For all practical purposes, the pore pressure remains unchanged during the loading.

The value of the angle of internal friction is determined from laboratory tests such as drained triaxial tests, direct shear tests, or from empirical correlations. A crude guide for making an initial estimate of $\bar{\phi}$ is the standard penetration test during sampling in the bore hole. Peck, Hanson, and Thornburn (1974), for example, relate $\bar{\phi}$ values for sands and gravels to the standard penetration resistance (N), as shown in the following table:

$\bar{\phi}$ vs N for Cohesionless Soil

Relative Density	$\bar{\phi}$	N* (blows/ft)
Loose	28° - 30°	5 - 10
Medium	30° - 36°	10 - 30
Dense	36° - 41°	30 - 50
Very dense	41° - 44°+	>50

*N, standard penetration resistance, is dependent on soil density, gradation, drilling procedures, sampling procedures, and depth below surface.

3.43 Cohesive Soils

Unlike cohesionless soils, which drain quickly, the strength of cohesive soils is related to the changes in pore pressure that occur during shear. The general expression for soil strength, $\tau_{ff} = \bar{c} + \bar{\sigma}_{ff} \tan \bar{\phi}$, is applicable to these soils; however, the evaluation of $\bar{\sigma}_{ff} = \sigma_{ff} - u$ can only be made with knowledge of the pore pressure, u. As stated previously, the current state-of-the-art does not provide a sufficiently accurate means to predict pore pressure, but it can be measured in situ.

Figure 20 illustrates the differences in the strength envelopes for normally consolidated and over consolidated clay. Note that the cohesion intercept ($\bar{c}$) is observed in overconsolidated clays, but not in normally consolidated clays.

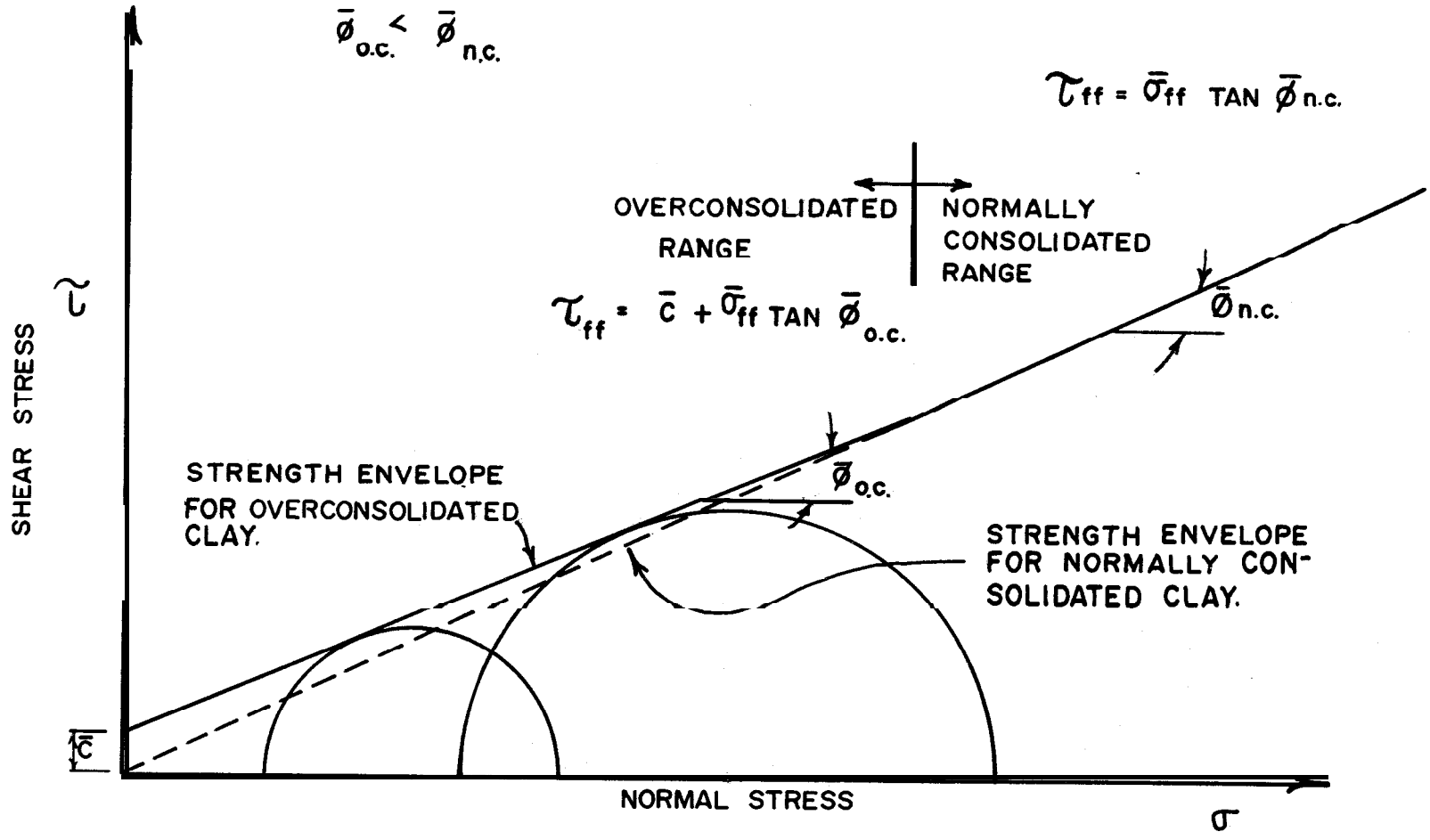


Figure 20. Strength envelopes for clay (based on effective stress).

3.43.1 Undrained Strength

Cohesive soils are not normally permeable enough to allow significant drainage during shear, and excess pore pressures do not dissipate quickly. Therefore, to be on the safe side during initial application of Load, the shear strength should be based on undrained strength of the soil at natural water content. For most loading conditions, normally consolidated soils will experience a pore pressure increase (positive). Over consolidated soils experience either a smaller pore pressure increase or may even experience a pore pressure decrease depending on the degree of overconsolidation.

When the change in pore pressure is positive, the effective stress in the soil is lowered; therefore, the shear strength is less than that which would be obtained using effective strength parameters based on static water table conditions. With time,as pore pressure dissipates, the effective stress, and therefore shear strength; increases.

When pore pressure decreases as a result of shear strain, as is characteristic of overconsolidated soils, the effective stress increases. Therefore, the shear strength is greater than that which would be computed using effective strength parameters based on the static water table conditions. With tirne, the pore pressure becomes increasingly positive (approaching the static water table condition), the effective stress decreases, and the strength becomes less.

Figure 21 shows two strength relationships. One is a drained strength envelope in terms of effective stress. The other is a plot of consolidation pressure vs. undrained strength. The plots in the figure show cases where the undrained strength is greater than the drained strength and, conversely, where it is Less.

Cohesive soils at their natural water content exhibit a unique shear strength for all undrained loadings (and unloadings) regardless of the total confining pressures. This value of shear strength is referred to as the undrained shear strength, S_u, of the soil. It is approximately equal to (and usually taken as) one half the compressive strength, $\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}$, as determined in a compression test. The undrained strength at natural water content can also be determined by a vane shear device, either in situ or in the laboratory.

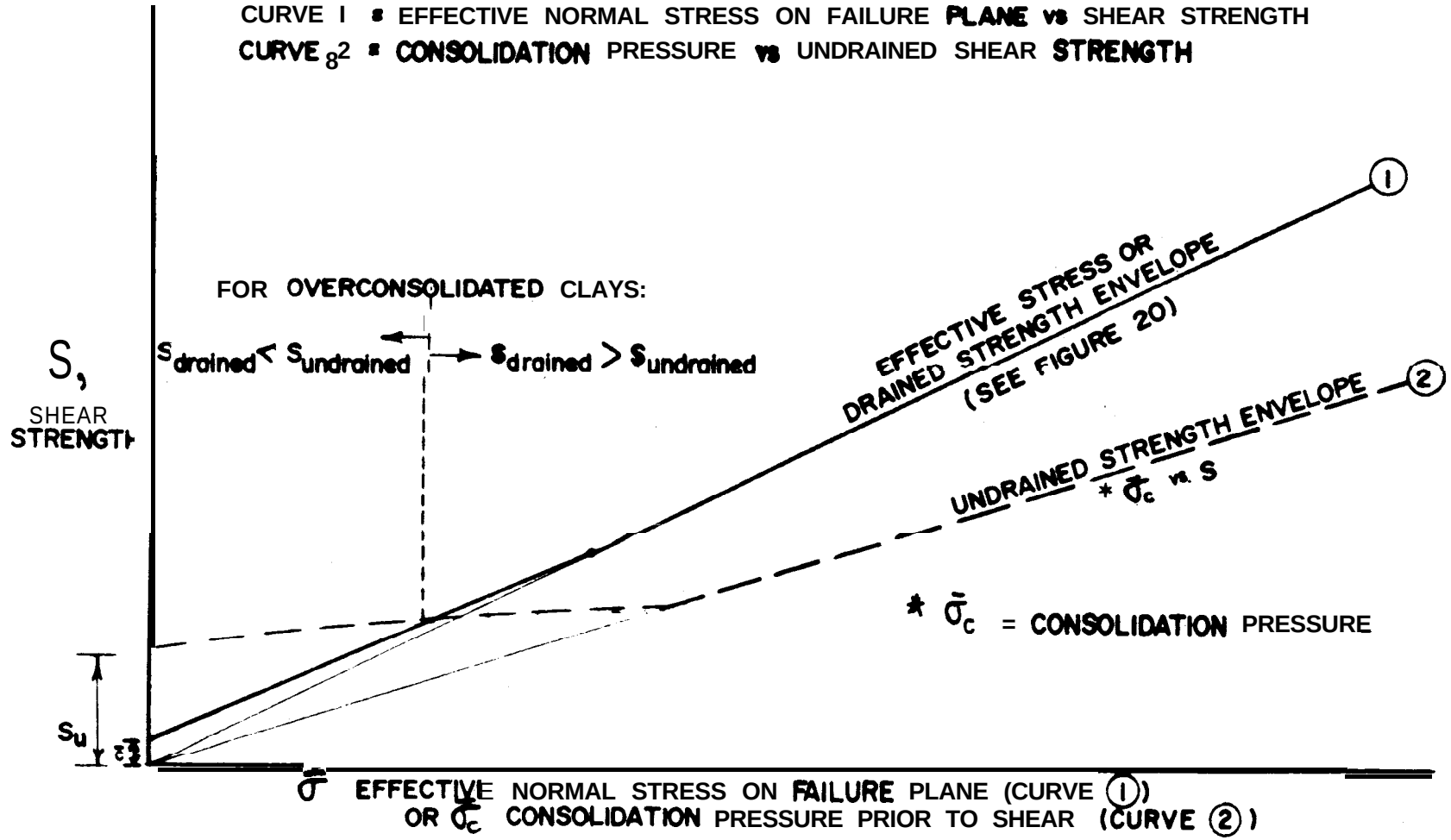


Figure 21. Comparison of drained and undrained strengths.

3.43.2 Drained Strength

Ultimately the excess pore pressures generated by shear strain in a cohesive soil will. drain, thus resulting in zero excess pore pressures and a return to hydrostatic conditions. Drained and undrained strengths present the limiting strength conditions for a cohesive soil. Because one does not necessarily know what the pore pressure conditions are, it is important to recognize which is the more conservative for a particular case. At low confining pressures (with respect to the degree of over-consolidation) the drained strengths control. On the other hand, the undrained strengths are more critical at higher confining pressures. A comparison of undrained and drained strengths is made in Figure 21.

3.43.3 Intermediate Cases

During drainage the strength lies between the drained and undrained strengths and is a function of the pore pressure dis sipation, i. e. it is related to the effective stresses at any given instant.

3.43.4 Cohesive Soil Strength During Excavation

Consider the case of an element of cohesive soil located at a depth inside an excavation. Initially, the shear strength of the soil will be equal to the in situ undrained shear strength, S_u, Assuming an immediate unloading of the soil element (due to excavation), the soil element will experience a reduction in pore pr es - sure, maintain a constant effective stress, and therefore, a constant strength value (except for minor changes due to loading conditions). However, as these negative excess pore pressures dissipate, the strength of the soil will decrease until the ultimate drained conditions are achieved.

This factor is particularly important for overconsolidated cohesive soils with high undrained strengths. Accordingly, upon initial excavation, the undrained shear strength at natural water content should be assumed. But if the excavation is open sufficiently long for excess pore pressure to dissipate, the drained strength should be the basis for an analysis of this long term case. See Chapter 6 for further discussion on the relationship to passive pressure.

3.43.5 Consistency

Cohesive soils are described in terms of their consistency, which in turn is directly related to their undrained strength, Standard penetration resistance may also be used as a rough measure of consistency for insensitive cohesive soils. See following table:

Consistency of Cohesive Soil	N* (blows /ft)	Undrained shear strength psf
Very soft	< 2	< 250
Soft	2 - 4	250 - 500
Medium	4 - 8	500 - 1000
Stiff	8 - 15	1000 - 2000
Very stiff	15 - 30	2000 - 4000
Hard	>30	> 4000

N* - Standard penetration resistance

3.43.6 Heavily Over consolidated Clay

For heavily overconsolidated clays with fissures and subject to strength deterioration with time, $\bar{c}$ should be set equal to zero in the calculation of passive resistance.

It is also possible that in heavily over consolidated clays, where $\bar{\sigma}_h > \bar{\sigma}_v$, a passive failure may occur in the soil due to the release of vertical soil pressure. These passive failures have been observed as base heave in excavations in heavily overconsolidated clays. In these soils, a detailed analysis of the effective stresses in the soil with excavation depth is required. Careful analysis of the in situ stress system is required for all clays.

Further dis cussion relative to strengths of highly over consolidated clays is made in Chapter 8 (Stability Analysis).

CHAPTER 4 - GROUND WATER IN OPEN CUTS

4.10 GROUND WATER CUTOFF

4.11 General

Cutoff walls are used for the following purposes:

1. To avoid or to minimize dewatering of the excavation.

2. To lessen or to prevent lowering of ground water level outside of the excavation because of possible settlement and damage to adjacent structures.

3. Because it is otherwise impractical to place lagging in soils that are extremely difficult to dewater in advance of excava- (such as silts and/or dilatant clayey sands).'

4. To cut off pervious water bearing strata within or just below the bottom of the excavation: thus, protecting against the possibility of a "blow" condition or other source of ground loss.

4. 12 Soldier File Wall

A soldier pile and lagging wall is not watertight. In order to control ground water; dewatering, grouting, or freezing must be performed. While excavating in "running" soils it is essential to maintain the ground water level below the working face to prevent inflow and subsequent ground loss.

4. 13 Interlocked Sheeting

The permeability of interlocked steel sheeting has been studied both experimentally and by observations of actual installations The results of these studies are quite variable, but as an approximation the seepage through intact interlocks may be assumed to be in the order of 0.01 gallons per minute per square foot of wall under a 1 foot differential head (Fruco & Associates, 1966). Sherard, et al (1963) suggest that flow through intact steel sheet piling is equivalent to about 30 to 40 feet of soil (probably relatively pervious granular soils), under the same hydraulic head.

Clearly, the effectiveness of an interlocked steel sheet pile cutoff wall depends upon the permeability of the soil in which the sheeting is driven. If the steel sheet pile wall remains intact and penetrates into an underlying impervious stratum, the effectiveness will be very significant in pervious sands and gravel.. On the other hand, in granular soils of low permeability, such as silty or clayey sands, the interlocked sheeting will have little effect on the relatively low flow into the excavation. In all cases, however, sheeting effectively cuts off flow in pervious layers that are interbedded within a parent stratum of impervious soil.

With regard to maintaining ground water level outside of the excavation, interlocked sheeting is effective in pervious granular soils. For relatively impervious soils (such as clayey sands, silts, and clays) the sheet piling is essentially equivalent to the permeability of the soil, and therefore will have little or no effect on the seepage pattern toward the excavation or on lowering of piezometric levels.

The above discussion applies only to intact sheeting. The presence of boulders, difficult driving conditions, or obstructions can lead to ripping of the sheeting and/or jumping out of interlocks which will seriously impair if not destroy the effectiveness of the cutoff wall.

Another common problem is when the effectiveness of a cutoff in pervious soil depends upon achieving a tight seal on rock. This situation may be especially acute when rock occurs within the depth of excavation. Settlement may resu'lt from ground loss due to water inflow or from lowering of ground water levels and thus induce consolidation of compressible soils.

In a case in Boston (Lambe, et al, 1970), the lowering of piezometric head in a sand and gravel deposit below organic silt contributed to consolidation of the organic silt contributed to consolidation of the organics and settlement of the adjacent ground. Interlocked sheet piling could not completely cut off water in a deposit of pervious sand and gravel over bedrock near the bottom of the excavation. A similar case was reported in Oslo (Hutchinson, 1964).

4.14 Concrete Diaphragm Walls

For all practical purposes, a well-constructed concrete diaphragm wall is essentially impermeable, It will effectively cut off flow and prevent ground water lowering outside the excavation provided that there is penetration into an underlying impervious formation. Nevertheless, lowering of ground water may be caused by several poor construction procedures, such as: (a) leaky joints,(b) water loss through drill holes made for tieback installations, (c) inadequate seal on bedrock, especially within the excavation.

4.20 SEEPAGE PATTERN TO EXCAVATION FACE

As mentioned previously, interlocked steel sheeting has relatively little influence on the seepage pattern in impervious soils. As a result, when cuts are made below ground water there will be flow to the face of the excavation. In clays, such a flow will be so small that it may not even be noticeable.

An example of a flow net for this type of situation is shown in Figure 22. During the initial process of excavation, deformation in the soil will generate shear strains and cause pore pressure changes. Eventually, these pore pressures will be dissipated, and a steady state seepage pattern will develop as shown in the figure.

The equipotential lines shown in the figure demonstrate the changes in hydrostatic stress. Such changes in hydrostatic stress lead to a time-dependent equivalent change in effective stress and consolidation of the soil, In precompressed cohesive soils, the amount of consolidation will be negligible; however, in soft normally consolidated clays or organic soils the associated amount of consolidation can be significant and will contribute to displacements behind the excavation.

The foregoing case is important because it illustrates that steel sheeting may not be effective in preventing consolidation of normally consolidated soils within depth of the cut. Soil compressibility and rate of consolidation must be considered.

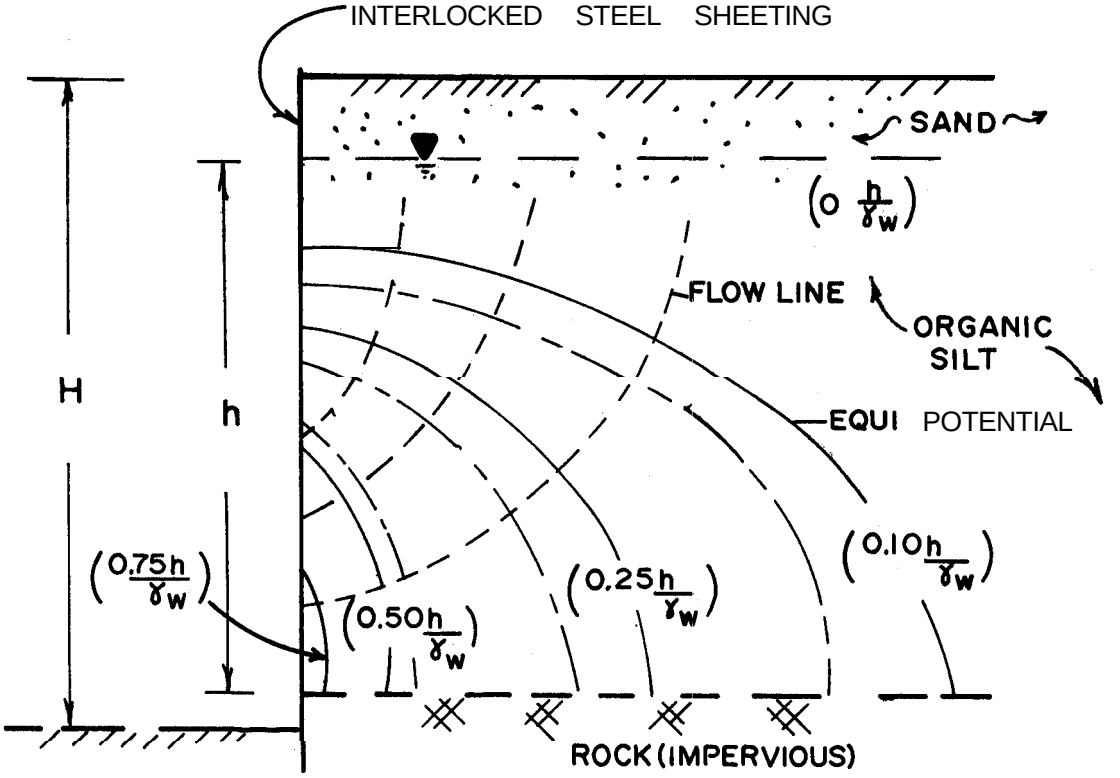


Figure 22. Change in pressure head for cut in impervious soil.

4.30 GROUND WATER RECHARGE

Lowering of ground water is accompanied by a corresponding decrease in hydrostatic pressure and an increase in effective stress. Such an increase in effective stress may possibly be accompanied by consolidation of compressible soils and settlement of surrounding buildings .

Compressibility of cohesive soils is time-dependent; compressibility of loose sand or non-plastic silt is immediate. Recharging of ground water can be accomplished through trenches, pits, or wells, in communication with pervious strata adjacent to the excavation. Most commonly, recharge wells are used in conjunction with excavations made in urban situations. Examples of recharge wells to maintain the ground water level outside of the excavation and to prevent settlement have been reported by Parsons (1959) and by Ball (1962).

One of the most difficult technical considerations in developing recharge wells is the problem of avoiding plugging of the well screen and surrounding soil. Contamination may develop from suspended particles, from corrosion, or by microorganisms which grow on the well screen. In addition, the recirculated water contains dissolved air which expands and plugs the soil pores after diffusion back into the soils, thus reducing the permeability of the soil.

Measures taken to counteract such contamination are to filter and chlorinate the water or to use cathodic protection to prevent corrosion.

A final consideration is to prevent buildup of excessive hydrostatic pressure by diffusion near the recharge well. Such a condition, especially in loose granular soils, can lead to loss of effective stress and settlement of adjacent foundations.

CHAPTER 5 - LATERAL EARTH PRESSURE

5.10 BASIC CONSIDERATIONS

The following discussion presents fundamentals concerning magnitude and distribution of lateral earth pressure when influenced solely by conditions within the depth of excavation. The influence of surcharge loadings are covered in Section 5.40.

In the case of excavations underlain by weak strata, the lateral pressure may be greatly increased as a result of shear deformations generated by marginal safety against base instability. These situations are addressed empirically in Section 5.22.

5.11 Earth Pressure at Rest

The ratio of the geostatic horizontal stress to vertical stress of a natural soil formation is defined as:

$$K_0 = \frac{\bar{\sigma}_h}{\bar{\sigma}_v}$$

where:

K_0 = coefficient of earth pressure at rest

σ_h = horizontal effective stress

$\bar{\sigma}_v$ = vertical effective stress

For granular soils Terzaghi and Peck (1968) suggest K_o values of 0.5 for loose deposits and 0.4 for dense soils, Generally K_o can be estimated for normally loaded soil deposits as:

$$K_o = 1 - \sin \bar{\phi}$$

where:

K_o = coefficient of earth pressure at rest

$\bar{\phi}$ = angle of internal friction in terms of effective stress

For cohesive soils, K is primarily dependent on the over consolidation ratio (OCR $= \bar{\sigma}^o_{vm} / \bar{\sigma}_v$) as shown on Figure 23. Normally consolidated clays typically have K_o values of 0.5 to 0.6; lightly over consolidated clays (OCR $\leq$ 4) have K_o values up to 1; for heavily over consolidated clays (OCR-16) K_o may range up to a value of 2.

5.12 Active Earth Pressures

5.12.1 Mobilization

Lateral displacement (as shown in Figure 24) transforms the state of stress in the ground from the at-rest condition to the active condition. The mechanics of this process are the mobilization of full shear-resistance within the soil mass - a state of stress referred to as "plastic equilibrium".

In their state-of-the-art report given at Madrid in 1972, Bjerrum, Frimann-Claussen, and Duncan summarized previous work concerning the amount of lateral displacement sufficient for mobilization of active earth pressure. They reported that lateral displacement of 0.1 percent of the wall height was sufficient to mobilize fully the active earth pressure of sands; whereas full active earth pressure develops in soft clays with displacements of 0.1 percent to 0.2 percent of the wall height.

5. 12, 2 Distribution

Figure 24 shows the active earth pressure distribution associated with displacement modes. The fully active state stems from lateral translation, by rotation about the bottom, or a combination of both. The earth pressure distribution is triangular and the resultant occurs at the third height of the wall.

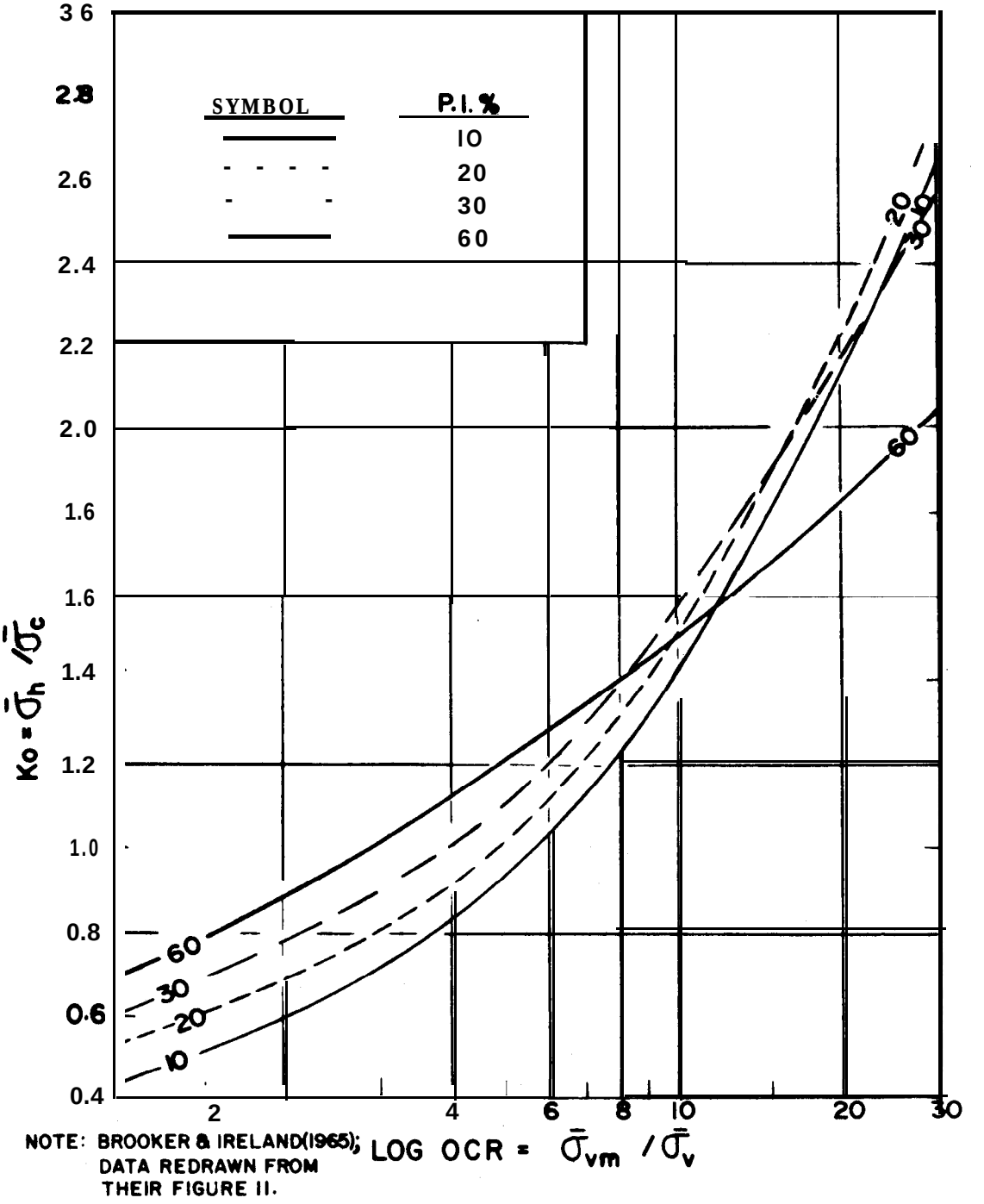
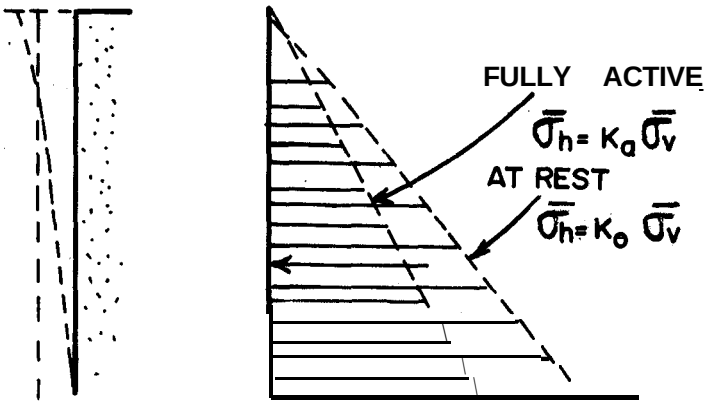


Figure 23. K o versus OCR for soils of varying plasticity, from Ladd (1968).

(a) FULLY ACTIVE



(b) ARCHING ACTIVE

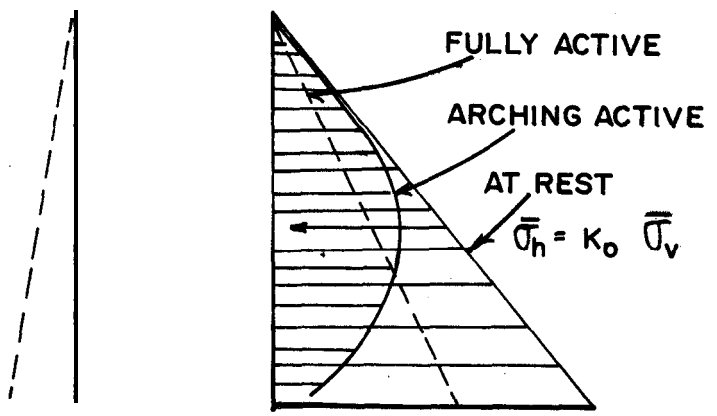


Figure 24. Earth pressure distributions for active and arching active conditions.

For a rigid retaining wall, the arching active case occurs by rotation about the top (Taylor, 1948). The resulting earth pressure distribution is parabolic in shape: it exceeds active near the top but is Less than active near the bottom of the wall. On a more flexible cofferdam wall the 'pressure distribution may be highly irregular due to the sequence of excavation and bracing or variation in the tightness of braces, both of which affect Load concentration.

5.12. 3 Coefficients

The direction and magnitude of active pressure depends upon whether or not there is wall friction. The particular case of horizontal surface and zero wall friction is the Rankine fully active condition, shown in Figure 24a. For this case, the active stress acts horizontally on a vertical wall. The Rankine coefficient of active earth pressure, K_a, is the ratio of the effective stresses.

$$\frac{\sigma_h}{\bar{\sigma}_v} = \frac{\bar{\sigma}_a}{\bar{\sigma}_v} = K_a$$

For sands, $K_a = \tan^2 (45^o - \bar{\phi}/2)$

For cohesive soils,

General case ($\bar{\phi}$, $\bar{c}$):

$$K_a = \tan^2 (45^o - \bar{\phi}/2) - \frac{2\bar{c}}{H} \tan(45^o - \bar{\phi}/2)$$

Special case ($\phi = 0$, $c = S_u$):

$$K_a = \frac{2S_u}{H}$$

where:

K_a = coefficient of active pressure

$\bar{\phi}$, $\bar{c}$ = friction angle and cohesion intercept

$\bar{\sigma}_v$ = vertical effective stress

σ_h = horizontal effective stress

$\bar{\sigma}_a$ = active earth pressure (horizontal)

S_u = undrained shear strength (ϕ = 0 case)

According to the Rankine expression, the pressure distribution for cohesive soils is theoretically in tension in the upper part of the wall as shown on Figure 25a. Frequently, adhesion simply does not (or cannot) develop and therefore tension cannot 0 ccur . However, the net total lateral force on the wall is equivalent to that described by subtracting the "negative" pressure at the top from the positive pressure at the bottom, Assuming that this net force increases Linearly with depth of wall, it can be represented by a net pressure diagram with a triangular distribution of the same force magnitude as shown on Figure 25b. The ordinate at the base of the wall is:

$$\bar{\sigma}_a = \gamma H - 4S_u$$

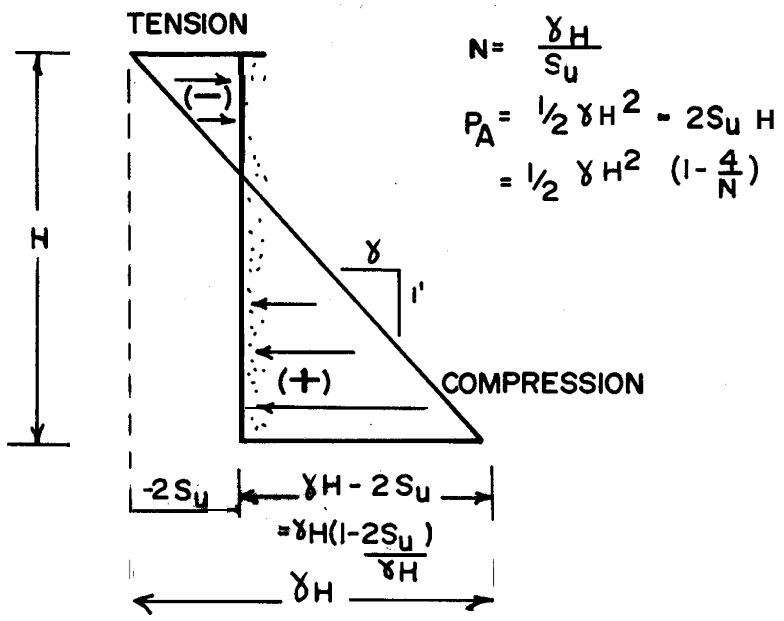
This procedure was described by Terzaghi and Peck (L968) as a means of comparing measured lateral forces with computed forces acting on braced cofferdam walls. White the method is reasonable for short term conditions, it is probably unrealistic to assume that undrained strength is mobilized over Long periods of time. Clearly, such an approach is unconservative with very stiff or hard clays. . In such cases and given the time for dissipation of pore pressure generated by shear strain, one should examine pressures based upon the effective strength angle, $\bar{\phi}$.

5.20 INTERNALLY BRACED COFFERDAMS

5.21 General

Initially the internal bracing is set near or at the top, thus restraining inward displacement. With each stage of excavation and bracing there will be progressive inward displacement below previously placed braces. The net displacement profile typically takes the form shown in Figure 26 (after Bjerrum, et al, 1972).

(a) RANKINE ACTIVE PRESSURE: DISTRIBUTION IN COHESIVE SOILS



(b) TRIANGULAR PRESSURE DISTRIBUTION EQUIVALENT TO NET RANKINE FORCE

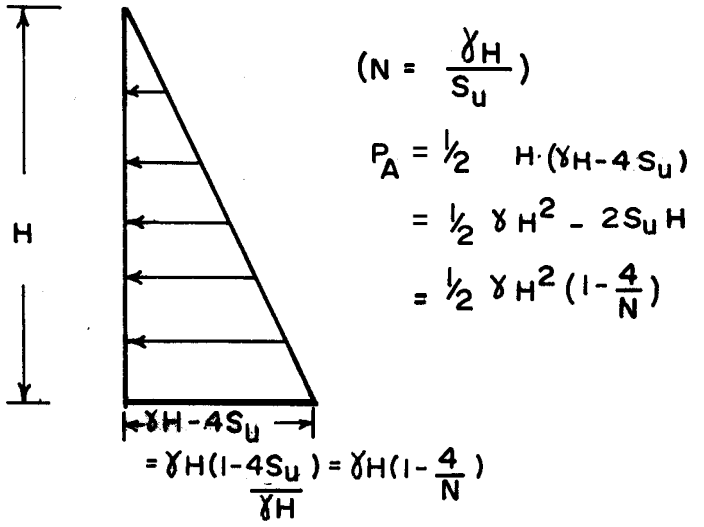


Figure 25. Earth pressure distribution for cohesive soil (ϕ = 0).

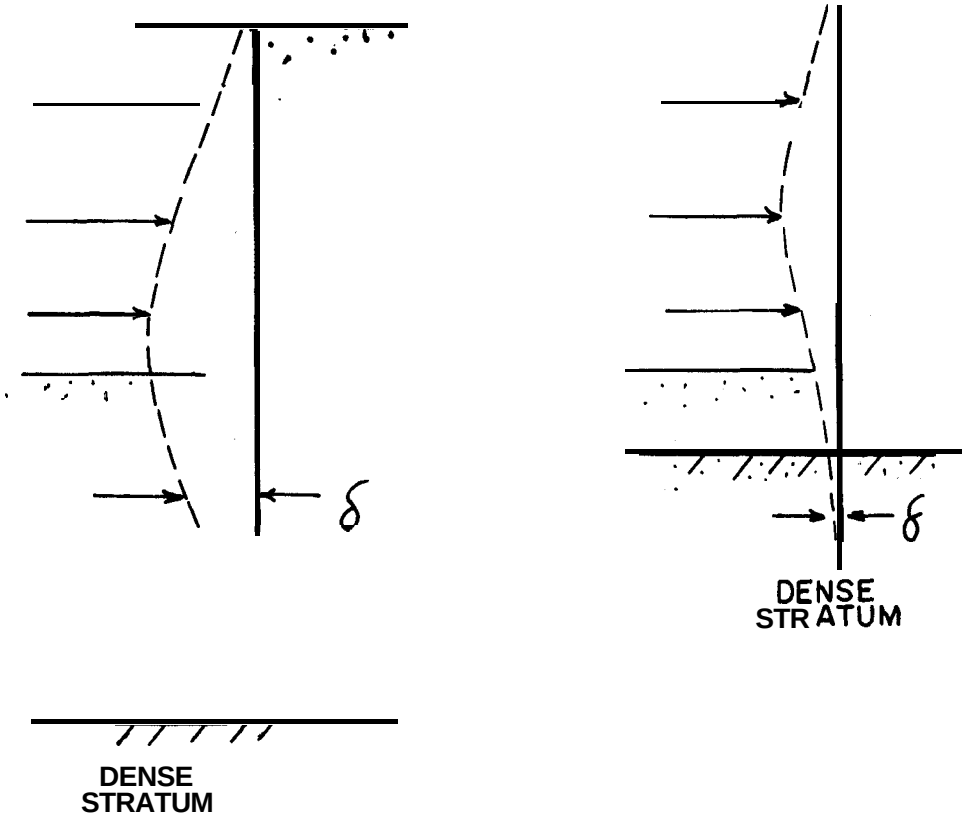


Figure 26. Mode of deformation of internally braced cofferdam (after Bjerrum, et al, 1972).

Characteristically, there will always be some inward rotation about the top, at least in the upper portion of the cut. The degree of bulging and displacement below the cut depends upon several factors - the distance between braces, the stiffness of the wall, and the stiffness of soils near the base of the wall. In general, the resulting deformation pattern most closely resembies the arching active condition. Therefore,- a parabolic, rather than triangular, pressure distribution is most Likely to act on the wall.

The Load concentrated on individual levels of lateral support is greatly affected by the construction procedure itself, envir onmenta l chan ge s, and design considerations.

Several factors that can affect the load in a strut are:

For bracing:

a. The tightness and consistency of the contact between struts, wales, and wall.

b. Whether or not prestressing was employed.

c. Temperature of braces during and following their installation. (Bracing loads may be significantly affected by temperature changes; especially in excavations which are not decked over.)

d. Excavation distance between lavels of support. (The relative consistency of Loads is directly affected by the variation in vertical distance between Levels of support. For example, if the upper 2 wale levels were spaced 10 feet apart and the third wale level were 15 to 20 feet below the second, the load on level 2 would-be relatively higher.)

For bracing or tiebacks:

e. Weak soils below depth of excavation. (For example, soft clay underlying the bottom of a deep cut would cause relatively high loads in the lower strut levels because of Lack of passive resistance below the excavation base. This occurs even though soils within the depth of cut are highly competent. Another example-of increased load on bottom struts would be an upward seepage gradient causing a loss of

passive resistance in front of the cofferdam wall.)

f. A stratum of weak cohesive soils within the depth of excavation. (Such a condition may selectively increase Loading on a particular wale Level unfavorably positioned in relation to the bracing sequence and the weak stratum.)

g. Concentrated construction surcharge.

h. Frost action causing additional lateral thrust on wall.

i. Erratic ground wat er conditions - per haps Locally perched zones combined with seepage.

Because of the number of variables affecting the distribution of Load on ground support walls, design procedures are Largely dependent upon empirical studies and correlations. The design procedures summarized in the two state-of-the-art reports by Peck (1969) and Bjerrum, Frimann-Clausen, and Duncan (1972) are based upon data primarily from internally braced, relatively flexible walls.

Figure 27 shows the conventional procedure for analyzing empirical load data. The approach has been to develop an apparent earth pressure diagram by distributing the maximum measured strut Loads during construction over an area described as midway between the upper and Lower adjacent spans of the particular strut Load measurement. The resulting apparent earth pressure diagrams are used to develop an envelope encompassing the maximum distributed pressures. This design envelope then represents the maximum strut Load that can be anticipated at any stage of construction.

5.22 Design Earth Pressure Diagram

Apparent earth pressure diagrams suggested by Terzaghi and Peck (1968) for design of braced walls are shown on Figure 28. Strut Loads for a given Level are determined by reversing the procedure used for development of the diagram. A strut is designed to support a load described by the area between the midpoints of the adjacent upper and Lower support Levels.

The following discussion does not include the effect of surcharge (see Section 5.40).

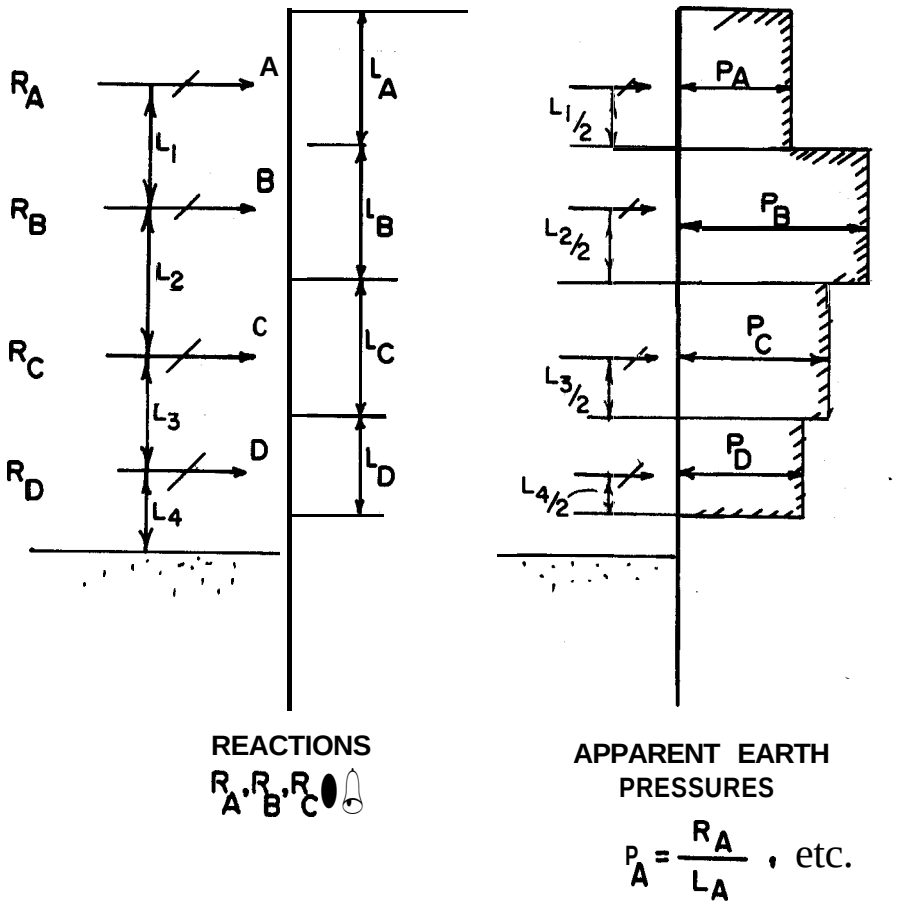


Figure 27. Conventional procedure for development of earth pressure diagram,

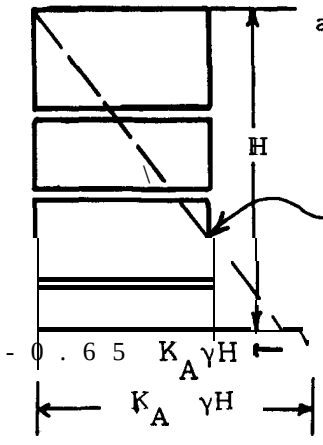
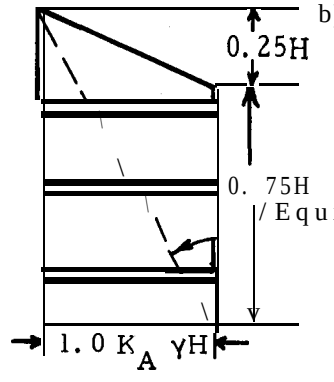
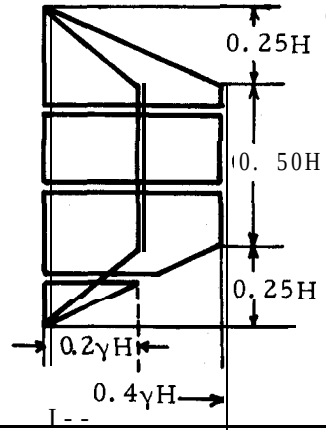
PRESSURE DISTRIBUTION	TOTALFORCE
 a) Sands $K_A = \tan^2(45 - \phi/2)$ Rankine Active	P_t = Trapezoid P_A = Rankine Pt = . 65 $K_A \gamma H^2$ P_A = . 50 $K_A \gamma H^2$ $\frac{P_t}{P_A}$ = 1.30
 b) Soft to Medium Clays (N > 6) For clays, base the selection on N = $\frac{\gamma H}{S_u}$ /Equivalent Rankine Active $K_A = 1 - m \frac{4S_u}{\gamma H} = 1 - \frac{4}{N}$ m = 1.0 except where cut is underlain by deep soft normally consolidated clay	m = 1.0 P_t = . $875\gamma H^2(1 - \frac{4}{N})$ P_A = . $50\gamma H^2(1 - \frac{4}{N})$ $\frac{P_t}{P_A}$ = 1.75
 c) Stiff-Clays For N < 4 For 4 < N < 6, use the larger of diagrams (b) and (c).	P_t = . 15yH^2 to . $30\gamma H^2$ $\frac{P_A}{N} = 4$, $p_A = o$ N < 4, P_A < 0 NOTE: Equivalent Rankine Active = 0

Figure 28. Design earth pressure diagram for internally braced flexible walls (sands, soft to medium clays, stiff fissured clays), from Terzaghi and Peck **(1968).**

5.22.1 Sands

This diagram, which was developed from dewatered sites applies to cohesionless soils. If the soils outside the excavations remain submerged, then the earth pressure should be computed using the bouyant unit weight of the soil. Hydrostatic pressures are treated separately and added to the effect of the earth pressure.

5.22.2 Soft to Medium Clays

The recommended earth pressure diagram for these soils is shown in Figure 28b. The selection of an appropriate design diagram is dependent upon the stability number, ($N = \gamma H/S_u$). The earth pressure computation for clays is based upon the total weight of soil, assuming undrained behavior. This follows from the fact that the data were empirically developed on the basis of total unit weights and the soils' initial shear 'strength.

Where the stability number (N) exceeds 5 or 6, shear deformation becomes significant. Note, by inspection of the empirical diagram for sands and for soft clays, that the latter is significantly greater than the equivalent Rankine pressure which is shown for comparison. The value of 'm' used in the determination, of the ordinate for earth pressure applies to situations where the cut is underlain by a deep deposit of soft clay. Its value can only be determined by empirical means from measurements and performance of an actual excavation. Experience thus far, reported by Peck (1969) from cases in Mexico City and Oslo, Norway, lead to the conclusion that the value of 'm' is in the order of 0.4 for sensitive clays. For insensitive clays the value of 'm' may be taken as 1.0.

5.22. 3 Stiff Clays

The recommended apparent earth pressure diagram for stiff clays is used when the stability number, N, is less than 4. This empirical diagram is independent of the value of shear strength, rather the lateral earth pressure is a function of the gravity forces only. Strains associated with excavations for cut and cover tunneling in these relatively strong soils are small, and the shear strength of the soil is only partially mobilized. However, the movement is sufficient to drop the lateral earth pressure below the K_o values (Gould, 1970).

5.22.4 Heavily Overconsolidated Very Stiff Fissured Clays

Several cases have been reported which suggest that stress relief from excavation leads to lateral deformation of these soils toward the excavation. The mechanism probably includes both elastic strain and volumetric expansion. The elastic deformations occur during the excavation process whereas the swelling is time dependent and is a result of the development of negative pore pressures caused by stress relief. Soil behavior would suggest that the deformations should increase with increasing overconsolidation ratio, increasing plasticity of clay, depth below the water table, and intensity of fissuring in the soil. For strutted excavations, this condition may lead to build up of strut load with time.

Pending the reporting of more field experience, design criteria for cases involving potentially, laterally expansive soils are as yet undeveloped. Therefore, a laboratory test program (possibly stress-path triaxial) should be undertaken to aid in evaluating the magnitude of the problem. Prototype test sections with construction monitoring are also recommended.

5.22.5 Dense Cohesive Sand; Very Stiff, Sandy Clay

Recent papers concerning measurements of loads in dense cohesive sands and sandy clays have been reported by Armento (1972), Liu and Dugan (1972), O'Rourke and Cording (1974) and by Chapman, et al (1972). Several cases involved cohesionless soils (either fill or natural deposits of sand) within the upper portions of the excavations. Others included interbedded strata of stiff clay. In all cases, the soils near or below the bottom of the excavation were extremely dense and highly over consolidated.

The cases reported that the following factors affect the load distribution:

a. Cohesionless soils within the upper portion of the cut.

b. The construction procedure.

c. The depth of the excavation and bracing sequence.

Chapman, et al (1972) report that for a number of cases studied in Washington, the ordinate of the apparent pressure diagram increased from 0.15γH for 30-foot cuts to 0.23 γ H for 60-foot deep cuts. The attribute the low pressures at shallow depth to the relative importance of soil cohesion. O'Rourke and Cording (1974) in their report to the Washington Metro also noted an increase in the ordinate of the apparent pressure diagram with depth.

Recommended design diagrams for dense cohesive sands and very stiff sandy clays are shown in Figure 29. The minimum pressure line is associated with cuts having reasonably consistent spacing between wale levels, relatively uniform soil conditions, and depths less than about 30 or 40 feet. The maximum pr e s sure line is recommended to cover uncertainty regarding the effect of weak strata within the depth of cuts, the contingencies arising from construction (for example, over-excavation below support level or ineffective toe berms), and cuts in excess of 60 or 70 feet deep.

Cohesive soils near the top of the cut will justify pressure reduction as shown in Figure 29a. Absence of cohesive soils near the top of cut will require the higher pressures associated with Figure 29b.

5.22.6 Stratified Soils

The aforementioned cases are for readily idealized soil profiles. Actual soil conditions may have a stratigraphy which does not conveniently match these simplified cases. Moreover , an irregular ground surface or surcharge may complicate the analysis.

Under such cir cumstances, one approach is to determine the Lateral thrust either on the basis of classic active earth pressure or on the basis of trial planar sliding surfaces and wedge stability analysis. In this latter case the most critical wedge is used to determine the Lateral thrust (see Chapter 8). In such cases, hydrostatic forces are treated separately.

Once the lateral thrust is determined,, it should be increased by the most appropriate value of P_t/P_A (ratio of force from the empirical diagram to the force determined from the analysis from active earth pressure or wedge equilibrium). The designer must choose the most appropriate ratio based upon a comparison of the actual case to one of the simplified cases presented in this section.

The final question is one of pressure distribution. Again, at least initially, this is a question of the designer's judgement by com-

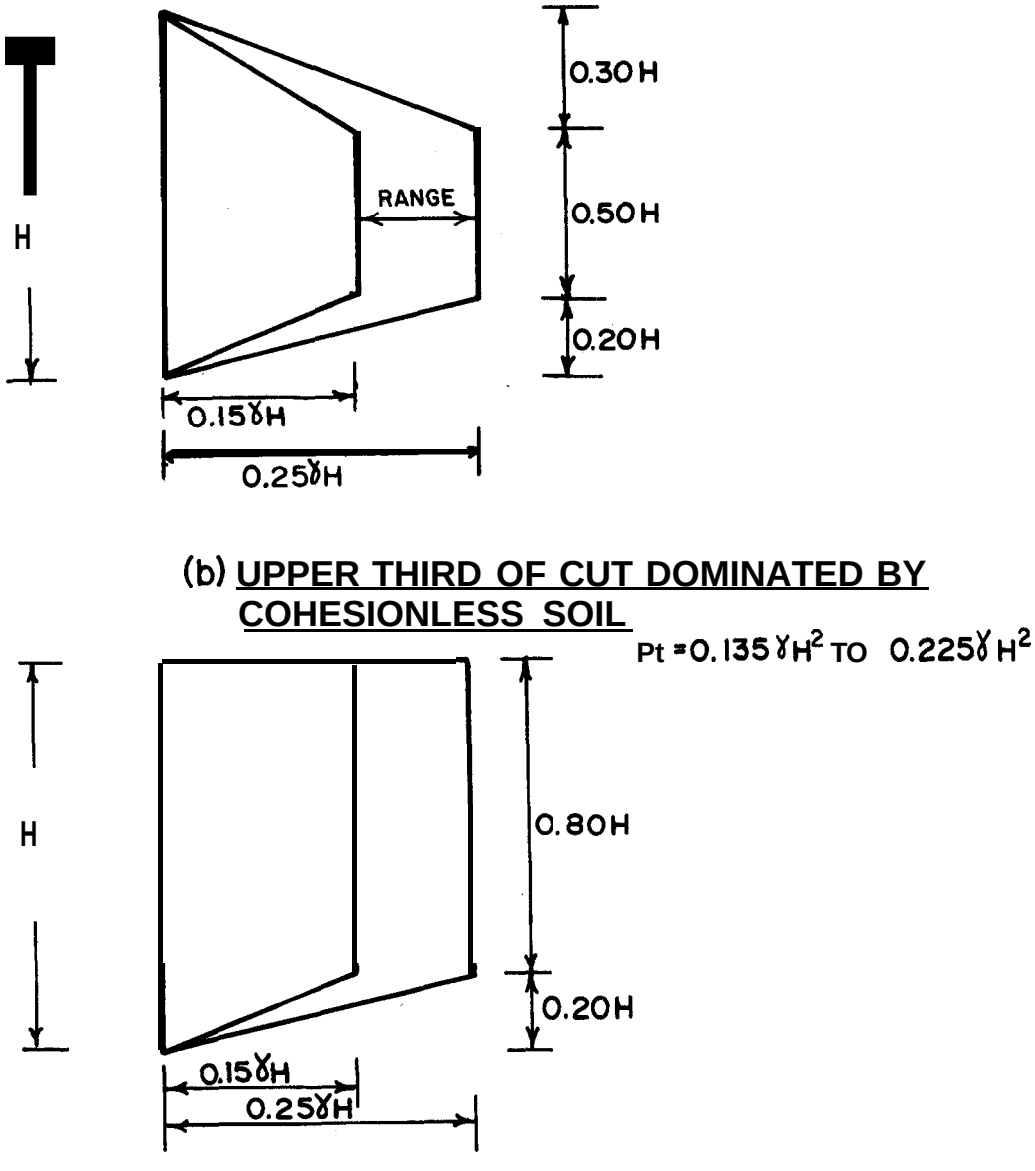


Figure 29. Proposed predsure diagram for internally braced flexible walls (dense cohesive sands, very stiff sandy clays).

parison with the simplified cases. Serious questions may need field measurements to provide data input during construction.

5. 30 TIEBACKS

5. 31 Background

Many practitioners have successfully applied the empirical rules developed for internally braced walls to tiebacks; others make variations for tied-back installations. In any event, at the present time there are no empirical methods for tied-back walls that have been accepted as universally as Peck's rules for internally braced flexible walls.

In a series of model tests, Hanna and MataLLana(l970) studied the effect of prestressing a wall to different design pressures and distributions. They observed with excavation that triangular distributions tended to redistribute Load to an apparent trapezoidal distribution. In addition, when ties were prestressed to loads corresponding to a trapezoida l distribution, there was Less Load redistribution and the movements were less than for the cases with triangular distributions. One problem with Hanna and Matallana's work was the Location of the ties. They were connected to a rigid wall of the experimental setup rather than embedded in the soil mass.

Apparently the tieback prestress has a significant effect upon the pressure distribution. Clough (1972) found, after studying sever al tied-back cases, that the pressure distribution suggested a parabolic shape; moreover, this was borne out by finite element analyses.

5. 32 Comparison Between Bracing and Tiebacks

Tied-back installations differ from internal bracing in their deformation mode, in the mechanics of stress conditions in the soil, and in various construction aspects.

a. Deformations associated with tiebacks and bracing are discussed in Chapter 2. Internally braced walls are restrained at the top and tend to move inward with depth by rotation about the top, whereas tied-back walls are more free to move inward at the top. Thus, the deformation mode often develops as inward rotation about the bottom. This latter mode is theoretically compatible with the "fully" active state and linear pressure increase with depth.

b. Temperature: Bracing loads may be significantly affected by temperature increase, especially by direct sunlight - to the extent that some projects require remedial measures to reduce thermal effects. Tiebacks are not subject to severe temperature changes since they are insulated in the ground. Thermal effects are more pronounced in prestressed struts when the strut is essentially between two unyielding supports.

c . Preload: Tiebacks are typically Locked off at 75 percent or more of the design Load. Observations suggest that tiebacks will either maintain their load or experience a slight load Loss with time. On the other hand, struts are generally preloaded to 50 percent or Less of the design Load and will gain Load as the excavation proceeds. (A greater preload in struts may risk excessive Load, especially from temperature rise.) These observations suggest that prescribed preloads for tiebacks are greater than the earth pressure wishes to impose. In effect, the tieback lockoff Load predetermines earth pressure rather than vice versa.

d. Mechanics: Tiebacks do not act by themselves, but in consort with the earth mass within which they are embedded. This behavior tends to dampen out Local variations in a given soil stratum and thus Leads to more uniform loading on the wall.

e. Load Variation: Overall, Load variation with tiebacks is Less than with bracing. Production testing of each tie above design Load, locking-off at 75 percent or greater of design Load, insulating from temperature effects, and engaging of an earth mass between the wall -- all tend to Lessen the variation in Load between individual tiebacks.

5. 33 Recommendations for Tiebacks

The following discussion does not include the effect of surcharge (see Section 5.40)

Because of the reasons cited above, the Load variation on individual tiebacks is believed to be Less than that on internal bracing. Thus, it follows that the design pressure envelope for tiebacks need not be as conservative as that for internal bracing. This does not mean that the resultant horizontal force on a section is Less with tiebacks. The actual resultant Lateral force must be differentiated from the empirical design envelope which is greater 'because it assures that no one tieback level is overstressed. Paradoxically, if one were to compare the actual forces on a given cross-section, the force on conventionally

installed tiebacks would probably exceed the force on a braced wall. This is because of the prestressing of tiebacks.

Only Limited documentation is available to quantify conclusions concerning the relative magnitudes of appropriate pressure envelopes for tiebacks and bracing. Accordingly, there is no present justification for significantly changing the pressure diagrams for tied-back walls from those used for internally braced walls. The following recommendations for tied-back walls yield similar total forces, but the pressures are distributed somewhat differently than for internally braced walls.

The soil classifications are the same as for internally braced walls shown in Figures 28 and 29 namely; sands, soft to medium clays, stiff clays, and dense cohesive sands or very stiff sandy clays. A triangular pressure distribution, increasing Linearly with depth, is recommended for soft to medium clay; a uniform pressure distribution is recommended for all other cases.

a. Sands: Where deformations are critical, and it is intended to prestress to LOO percent of design Load, compute force using K_o. For dense sands K_o = 0.4; for Loose sands K_0 = 0.5. Thus, the uniform ordinate will vary from:

Uniform Pressure, p = 0.20 γH to 0.25 γH

Force, P_t = 0.20 γH^2 to 0.25 γH^2

Where &formations are not critical, use $K_{avg} = \frac{K_o + K_a}{2}$, that is a coefficient midway between active and at rest. A similar procedure was used by Hanna and Matallana (1970).

Typical range is:

Loose sand:

K_a = 0.33; K_o = 0.50

K_{avg} = 0.42

Force, P_t = L/2 x 0.42 γH^2 = 0.21 γH^2

Uniform Pressure, p = 0.21 γH

Dense sand:

K_a = 0.24; Ko = 0.40

K_{avg} = 0.32

Force, P_t = L/2 x 0.32 γH^2 = 0.16 γH^2

Uniform Pressure, p = 0.16 γH

b. Stiff to Very Stiff Clays: Use a uniform pressure ordinate of 0.15γ H to 0.30γH to produce the same force magnitude as that for braced excavations. The higher value is associated with a stability number of about 4. The lower number is associated with very stiff clays where the stability number is less than 4. The force varies as follows:

Stiff Clays, P_t = 0. 30 γH^2

Very stiff clays, P_t = 0.15 γH^2

c. Cohesive Sand, Very Stiff Sandy Clays: Compute the total force associated with the diagram for braced excavations (Figure 28) and distribute the force uniformly with depth. For relatively uniform conditions use:

Force, P_t = 0.112 γH^2 to 0.188 γH^2

Uniform Pressure, p = 0.112 γH to 0. L88 γH

Where the upper third of the cut is dominated by cohesionless soil use:

Force, P_t = 0.135 γH^2 to 0.225 γH^2

Uniform Pressure, p = 0.135 γH to 0.225 γH

d. Soft Clays: It is unlikely that tiebacks would be used unless they could be embedded in an underlying denser stratum of soil or in rock. The walls should be designed with a triangular earth pressure diagram assuming at-rest conditions and a K_o value between 0.5 and 0.6.

Force, P_t = 0.25 γH^2 to 0.30 γH^2

In normally consolidated sensitive clays, excessive prestressing should be avoided because of the potential for induced consolidation (see McRostie, et al, 1972).

e. Stratified Soils: As with braced excavations an approach based upon active earth pressure or wedge equilibrium should be investigated. Section 5.22.6 generally describes a procedure for increasing the computed force in the same proportion as that of the most closely related simplified soil profile that exceeds active earth pressure. Use the force distribution most closely related to the simplified case.

5. 34 Effect of Wall Stiffness on Load Distribution

Theoretical analyses of the effects of wall stiffness on tieback loads (Egger, 1972; Clough and Tsui, 1974) and model tests (James and Jack, 1974) indicate that wall stiffness does affect anchor and wall load distribution.

Finite element analyses have shown that a more uniform load distribution occurs for a' stiff wall than for a flexible wall. In the more flexible walls the pressure distribution concentrates at the wale level due to arching. The difference between the load distribution for stiff versus flexible walls is greater with increased spacing between the wale levels. Clough and Tsui (1974) suggest that for typical spacing of tiebacks there is a relatively minor load distribution difference for the different wall types, Therefore, there appears to be no present justification for drawing a distinction in pressure distribution on the basis of wall stiffness.

5.40 SURCHARGE LOADING

5.41 General Background

Surcharge near excavations may be the result of many different types of loading conditions including footing6 structures, storage of construction materials, or traffic. The Lateral pressure caused by a surcharge Load on a retaining wall has been investigated for a variety of different loading and soil conditions (Spangler, 1940; Newmark, 1942; Ter zaghi, 1954b). This pressure is in addition to the normal earth and water pressure.

5. 42 Theoretical Cansiderations

The four basic Loading conditions for which solutions of the lateral stresses in an elastic medium are readily available are:

1. Point loading

2. Uniform line Loading

3. Irregular area Loading

4. Uniform area Loading

Typically, the stresses within a soil mass due to surcharge loadings are computed on the basis of elastic half-space theory;' When the wall is represented as a rigid boundary, it is necessary,to double all stresses obtained from half-space theory at the face of the wall in order to maintain compatible boundary conditions.

When lateral strains of the same magnitude as those in an elastic half-space do occur, it is not necessary to double the stress. In general, the true value of lateral pressure due to surcharges will be somewhere between these two cases. Since the assumption of an unyielding rigid boundary is conservative, uniform application of this rule should be questioned, and judgements made as to the appropriateness of the assumption for a given job condition (Gould, 1975).

5.43 Practical Considerations

With regard to surcharge loading from construction operations, it is common to take a distributed surface surcharge on the order of 300 psf to cover storage of construction materials and general equipment. Usually, this surcharge should be considered within a rather limited work area on the order of 20 feet to 30 feet from the cofferdam wall.

A second major consideration is the question of concentrated loads from heavy equipment (concrete trucks, cranes, etc.). Lateral thrust from such equipment would be easily covered within the 300 psf surcharge, provided that the equipment were more than approximately 20 feet from the wall. On the other hand, such equipment within a few feet of the wall may create a concentrated surcharge loading which would be of far greater significance than a uniform surcharge Loading. This must be accounted for separately. It may necessitate the designation of specific areas rather than designing the entire cofferdam for such Loading.

5.44 Point Load

While it is impossible to have a perfect point loading situation, the computed stress for an area Load or a point load is essentially the same when the distance to the wall is large compared to the size loaded area. The difference is small if a point load is assumed when the distance is greater than twice the average dimension of the Loaded area. There are several practical cases for which point loadings may

be as sumed. An isolated footing for a structure or a heavy object resting on a small base may be cases that can best be analyzed as point Loadings .

In the work of Spangler and Gerber, summarized by Terzaghi (1954b), it was shown that there is little change in the magnitude or distribution of Lateral stress from that determined by elastic theory until the point Load is located at a distance x Less than 0.4 H from the wall. This Leads to the following equations for evaluating the effects of a point Load on a rigid wall.

For m<0. 4:

$$\sigma_h = \frac{0.28\ n^2\ Q_p}{(0.16 + n^2)^3\ H^2}$$

where:

σ_h = horizontal stress at a depth, z = nH

Q_p = magnitude of point Load

H = height of cut

$$n = \frac{z}{H} = \frac{\text{depth to point on wall}}{\text{depth of cut}}$$

$$m = \frac{x}{H} = \frac{\text{distance of point Load from wall}}{\text{depth of cut}}$$

For m>0.4:

$$\sigma_h = \frac{1.77\ m^2\ n^2\ Q_p}{(m^2 + n^2)^3\ H^2}$$

Figure 30a presents solutions to these equations for selected values of m. The equation for m<0.4 has been derived from measured lateral pressures and does not correspond to results from elastic theory. For m>0.4, the equation gives values twice that of elastic theory to account for the wall as a non-yielding reflective boundary.

Figure 30c shows how the Lateral stress for a point Load varies along the Length of the wall. The calculation of σ_h gives the horizontal stress on a vertical plane lying perpendicular to the wall and through the point Load. The horizontal stresses along the wall vary as $\sigma_h' = \sigma_h \cos 2\ (1.\ 1\theta)$.

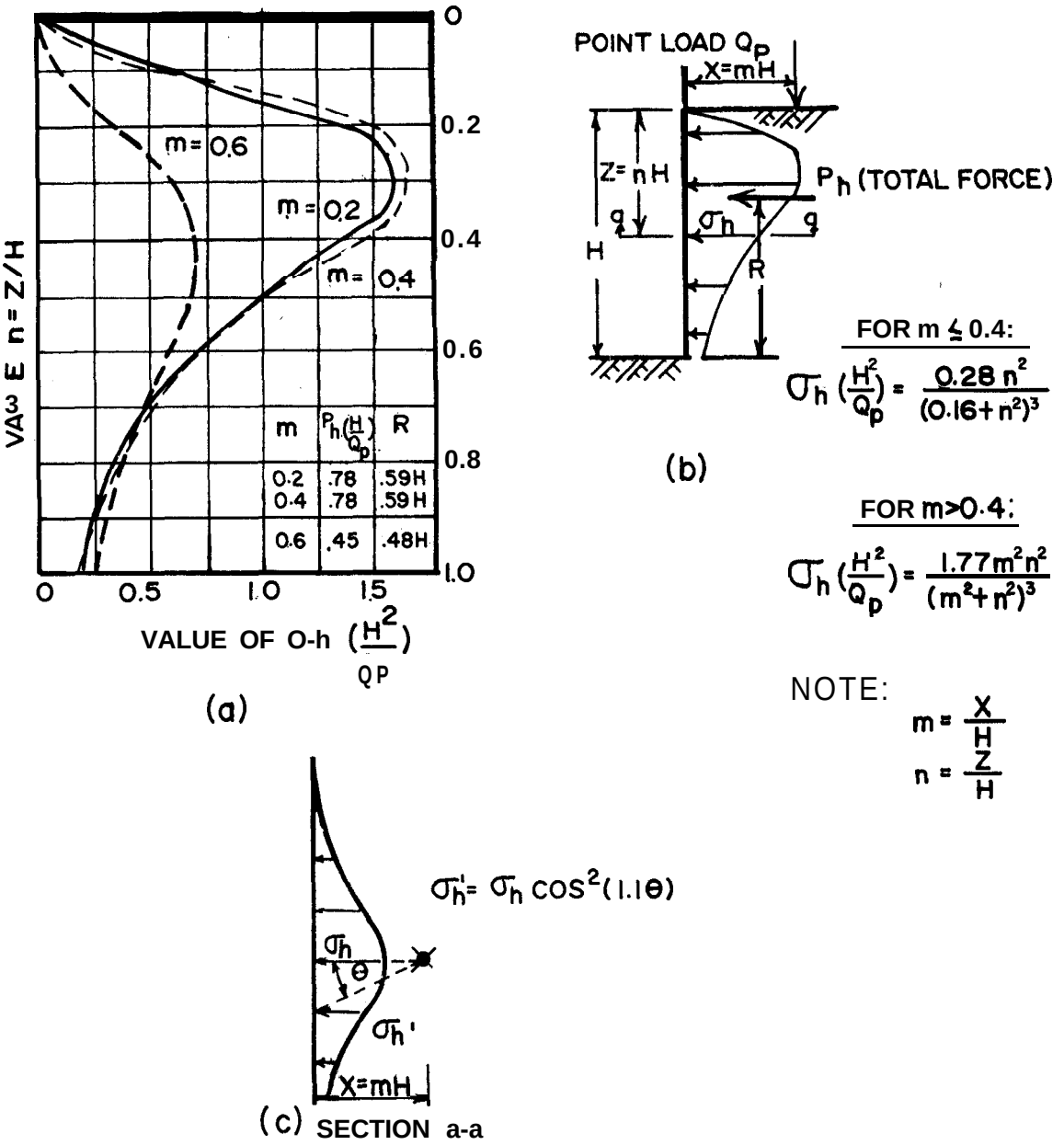


Figure 30. Lateral stresses on the face of an unyielding wall from a point loading (NAVFAC, 1971 and Terzaghi, 1954b).

5.45 Line Load

Terzaghi (1954b) also synthesized previous works on the effect of line load on rigid walls. Figure 31 presents simplified equations for evaluating horizontal stresses for a line load which is parallel to the direction of the wall and Located a distance, x, from the wall. In practice, such a situation may arise if a continuous strip footing runs parallel to an excavation. Again, these equations are for a non-yielding wall.

As for the case of the point loading, where $m<0.4$, the horizontal pressures predicted by elastic theory are too high. Hence, the equations given have been modified to correspond to measured lateral pressures. Where the load is located a distance less than 0.4 H from the wall, there is little change in the magnitude and distribution of lateral pressures from that computed at a distance x equal. to 0.4 H (see Figure 31). The variation in the location of the resultant, P_h, is also small until $x>0.4H$. The equations presented in this section have been adjusted to represent the boundary conditions of a rigid wall.

5.46 Irregular Area Loading

In some instances it may be unrealistic to assume a surcharge loading of infinite extent behind a wall. Theoretical solutions for area loading of limited (and irregular) dimensions have been developed for elastic half-spaces. Newmark (1942) presents an influence chart for use in determining the horizontal stress on a vertical plane. Although the chart was developed on the basis of Poisson's ratio $\nu = 0.5$, Newmark (1942) does give a method of converting these values to soils with other values of Poisson's ratio. The values of horizontal stress derived from Newmark's (1942) chart are for an elastic half-space. If the wall is assumed to be rigid, the values from the chart should be doubled.

Figure 32 shows an influence chart for evaluating the lateral stresses acting on a rigid wall from a rectangular loading (Sandhu, 1974). These charts assume a Poisson's ratio of 0.5 for the soil mass. Using the influence charts for point loadings, the lateral stress due to an irregular surcharge loading can be more easily calculated.

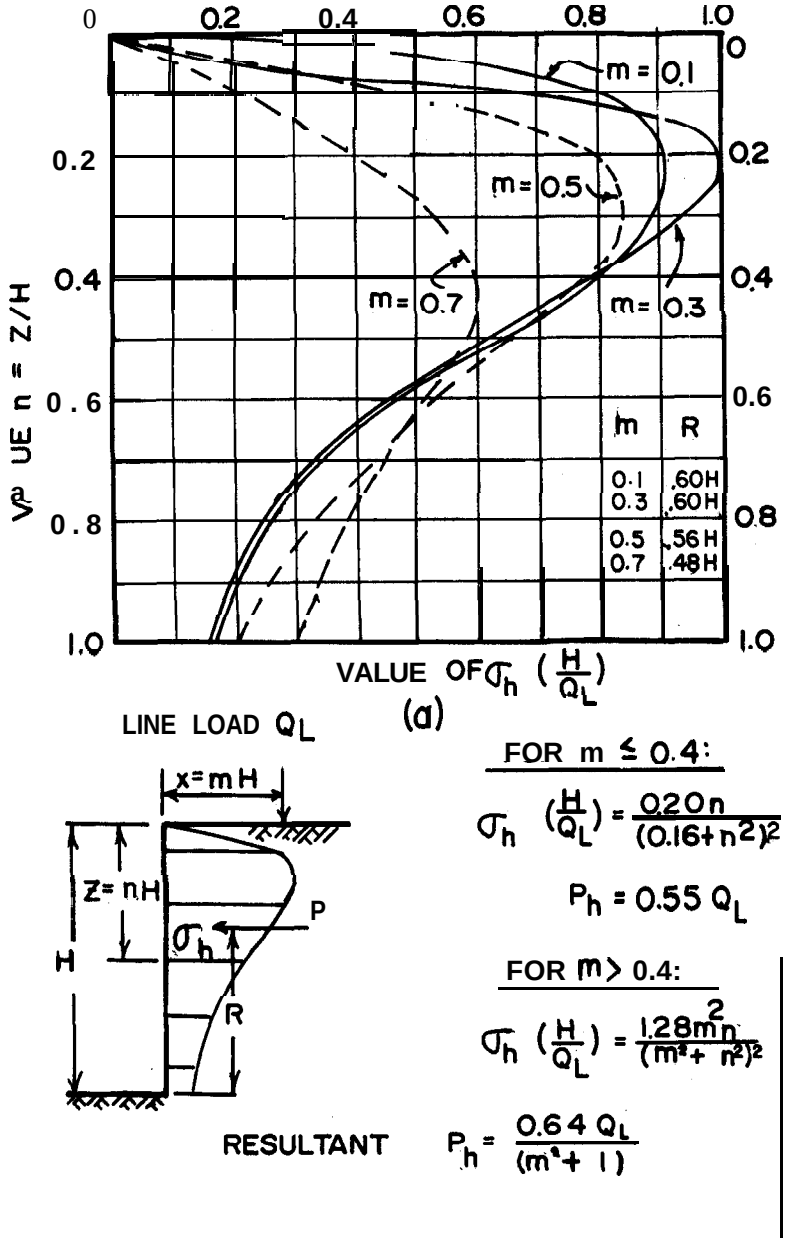


Figure 31. Lateral stresses acting on an unyielding wall from a uniform line loading (NAVFAC, 1971 and Terzaghi, 1954b).

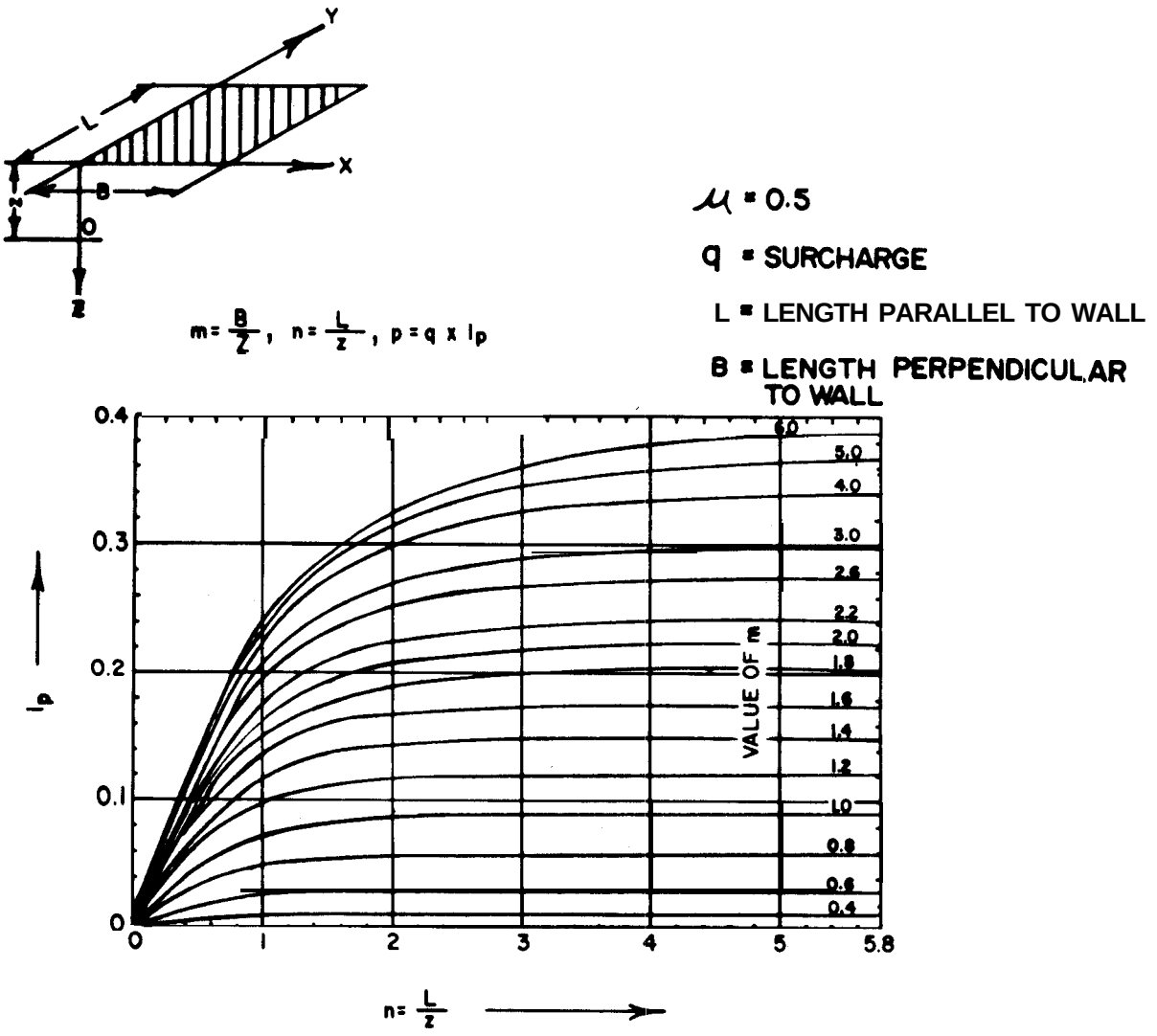


Figure 32. Lateral stresses on an unyielding wall due to irregular surface loading (Sandhu, 1974).

5.47 Uniform Area Loading

Intensity and distribution of Loading was discussed in Section 5.43. One approach is to treat the surcharge as a stress in an elastic medium. The solution for lateral stresses on a rigid wall are presented in Figure 32. An example of the stress effect with depth is shown in Figure 33. Note that the stress influence below a depth of about 1. 5B is negligible.

A second approach is to apply an earth pressure coefficient, K, to the surcharge loading and to consider the surcharge effective within some portion of the cut. The magnitude of this coefficient will range from K_a (active earth pressure) to K_o (earth pressure at rest).

In evaluating which of the above approaches to use, one should first establish whether or not there are significant design implications between the various methods. If there are then one must apply judgement concerning the relative rigidity of the wal.l (see Section 5.42). Moreover if the surcharge exists during the excavation process, then the appropriate coefficient is closer to K_a. If the surcharge is applied after excavation and bracing against a relatively unyielding wall, then one should use K_o or Figure 32.

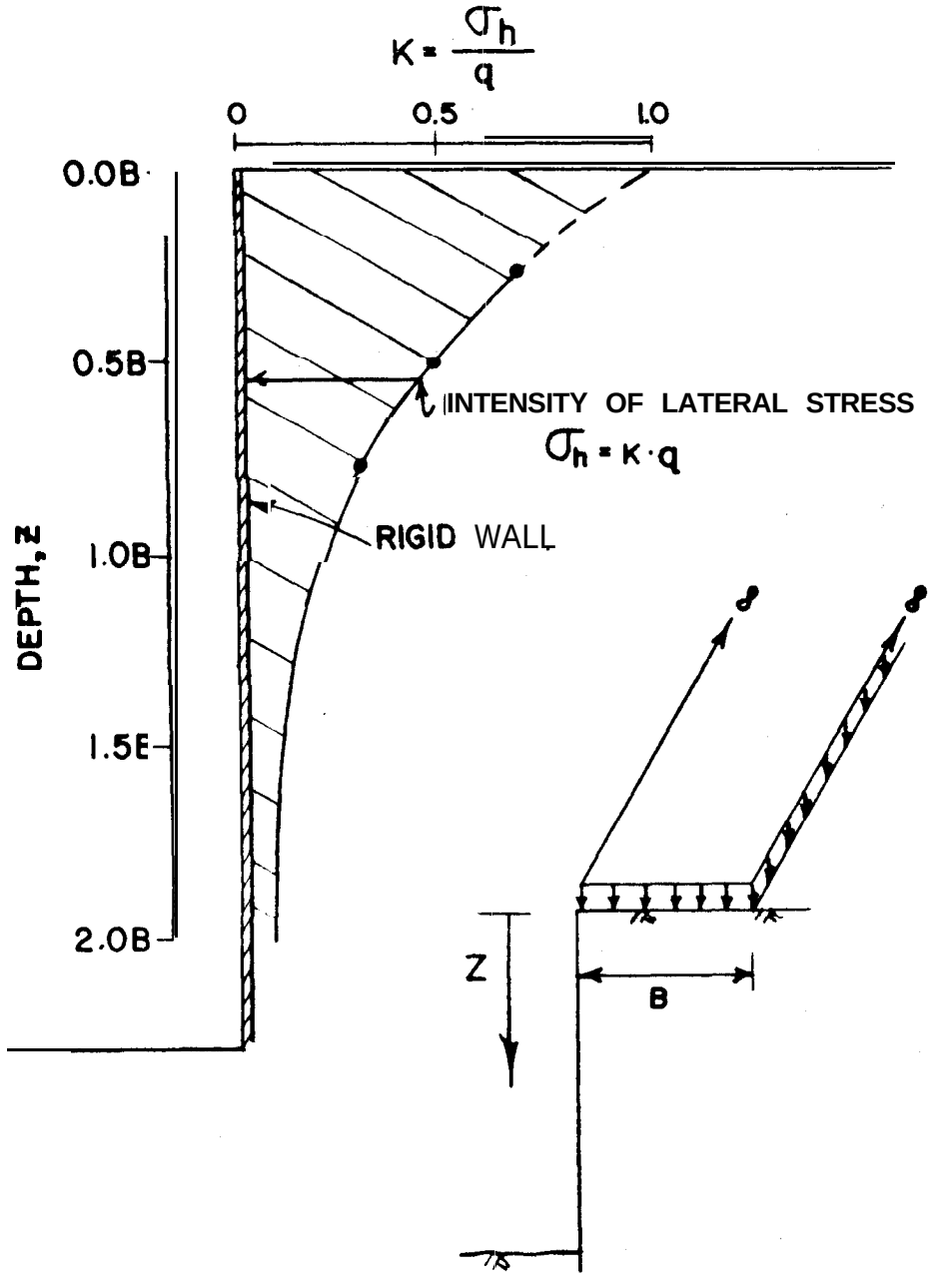


Figure 33. Lateral stress on rigid wall from surcharge of width B arid infinitely long (solution from Sandhu, 1974).

CHAPTER 6 - PASSIVE RESISTANCE BELOW BASE OF EXCAVATION

6.10 GENERAL

The design process frequently requires that the soils below the base of an excavation provide passive resistance for force equilibrium or to limit movement. The performance of the wall will depend upon the spacing of the support Levels since the greater the spacing, the greater the required passive resistance (and movement) below the Lowermost support level. Figure 34 illustrates the case of a wall in which the passive resistance of the soil is insufficient to limit excessive wall movements.

This section will describe the selection of soil parameters and methods used to evaluate passive resistance. This section will not consider the depths of penetration required to maintain overall stability of the earth mass or to limit displacements.

6.20 SOIL PARAMETERS

This section summarizes the soil properties relevant to the calculation of the passive pressures. Chapter 3 of this volume presented the basic concepts in more detail.

6.21 Granular Soil

Granular soils are free draining and cannot sustain positive or negative pore pressures generated by strain or Load changes for even a short period of time. Therefore, analyses of the stability of granular soils is performed on the basis of drained strength parameters and effective stresses in the ground referenced to the static water level. The appropriate soil strength parameter for the soil is the angle of internal friction, ϕ, for the soil. For design, granular soils are assumed to have no cohesive strength component.

6.22 Cohesive Soil

Passive stress conditions occur with excavation below the Last placed support Level. Because of the load decrease from excavation, soils in the passive zone just below the excavation will initially experience a pore pressure decrease. As a result, a gradient is set up which causes water to flow into the voids of the soil. This causes excess pore pressure to rise (i.e. become Less

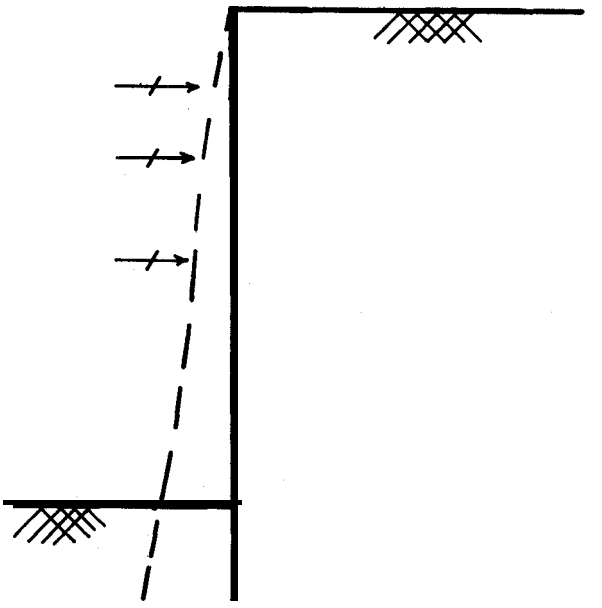


Figure 34. Movement at wall base due to insufficient passive resistance.

negative.) This may be accompanied by heave caused by swelling of the soil.

Limiting case strength parameters for passive pressure computation are:

a. Immediate Condition: Pore pressures generated by unloading and strain do not have time to dissipate. Moreover, pore pressure cannot be reliably predicted. Use undrained strength of soil at natural water content, S_u. Conventionally, this is determined' from vane shear, unconfined compression, or unconsolidated undrained compression tests.

b. Ultimate Condition: Pore pressures generated by unloading and strain are dissipated by drainage. Effective stresses can be computed on the basis of static water levels, Use strength parameters from the effective stress envelope, $\bar{c}$ and $\bar{\phi}$.

Specific cases obviously require soil testing and analysis in the light of soil properties, boundary conditions, and construction time. General recommendations for strength relationships are to use undrained strength parameters for the "during excavation" stage, that is, during the period of sequentially excavating and installing braces or tiebacks. For the fixed depth conditions, pore pressures will generally have sufficient time to dissipate, and therefore, effective stress parameters will apply for this limiting case condition. With in-situ pore pressure measurements during construction, the passive force can be assessed in terms of effective stress strength parameters based upon the computed effective stress conditions.

As was pointed out in Chapter 3, the undrained strength at natural water content may be greater than the drained strength of over consolidated soils. Therefore, indis criminate use of undrained strength without regard for pore pressure dissipation may be on the unsafe side.

Two factors that affect strength loss with pore pressure dissipation are the proximity of the soil element to the bottom of the excavation and the amount of unloading. Typically, the drained strength of cohesive soils in the passive zone of deep cuts will be the controlling strength.

6.30 ANALYSIS OF PASSIVE RESISTANCE

Several articles and texts address the problem of passive pressures that can develop behind a continuous wall (Terzaghi, 1954b; NAVFAC, 1971). In cohesionless soil wall friction modifies both the direction and magnitude of the passive resistance. Typically, the resultant of the passive pressure acts at an angle δ equal to 1/2 to 2/3 of the angle of internal friction. The following table (from Terzaghi, 1954b) summarizes values of K_p for various values of ϕ and δ.

Values of Passive Earth Pressure Coefficient as a Function of ϕ and δ

ϕ	$\delta = 0$	$\delta = \phi/2$	$\delta = 2/3\,\phi$
25°	2.46	3.00	3.20
30°	3.00	4.20	4.80
35°	3.70	6.50	7.30
40°	4.60	9.20	11.00

The passive pressure for drained loading or in terms of effective stress at depth, z, will be:

$$\bar{\sigma}_p = \bar{\sigma}_v \tan^2 (45^\circ - \bar{\phi}/2) + 2\bar{c} \tan (45^\circ + \bar{\phi}/2) \qquad \text{Eq. 6.30.1}$$

where:

$\bar{\sigma}_p$ = passive pressure (effective stress)

$\bar{\sigma}_v$ = vertical effective stress = $\gamma z - u$

$\bar{\phi}$ = angle of internal friction (effective stress envelope)

$\bar{c}$ = cohesion intercept

For this drained condition, in which by definition there is no excess pore pressure, the total lateral stress at any depth, z, will be:

$$\sigma_h = \bar{\sigma}_p + \gamma_w z \qquad \text{Eq. 6.30.2}$$

where:

σ_h = lateral stress

γ_w = unit weight of water

The passive resistance of cohesive soils in an undrained condition should be evaluated on the basis of the undrained shear strength, S_u, and the in situ total vertical stress, σ_v. For a continuous wall, the passive pressure at a given depth will equal:

$$\sigma_p = \sigma_v + 2 S_u$$
$$= \gamma z t \quad 2 s_u \qquad \text{Eq. 6.30.3}$$

where:

σ_v = total vertical stress = γz

S_u = undrained shear strength of the soil

In this case, the water pressure is not added because pore pressure effects are already accounted for in the determination of undrained shear strength, S_u. Therefore, the total lateral. stress at any depth, z, will be:

$$\sigma_h = \sigma_p = \gamma z + 2 S_u \qquad \text{Eq. 6.30.4}$$

Soldier pile walls are not continuous walls, therefore the passive earth pressure coefficients must be modified from those used for continuous walls, Broms (1965) showed that the passive resistance of laterally loaded piles based upon pile width and on K_p values for continuous walls was too conservative. His study showed that soil ar ching and non-plane strain conditions increase the capacity of individual piles. Indeed, the process is probably closely related to lateral bearing capacity. Broms' recommendations are given in the charts shown in Figure 35. It should be noted that for cohesive soils the lateral resistance of the soil should be neglected to a depth of 1.5 pile diameters. In cohesionless soils where the depth of penetration is greater than one pile diameter, soil arching causes an effective increase of 3.0 in the value of K_p.

A factor of safety of 1.5 is recommended for use in passive pressure calculations.

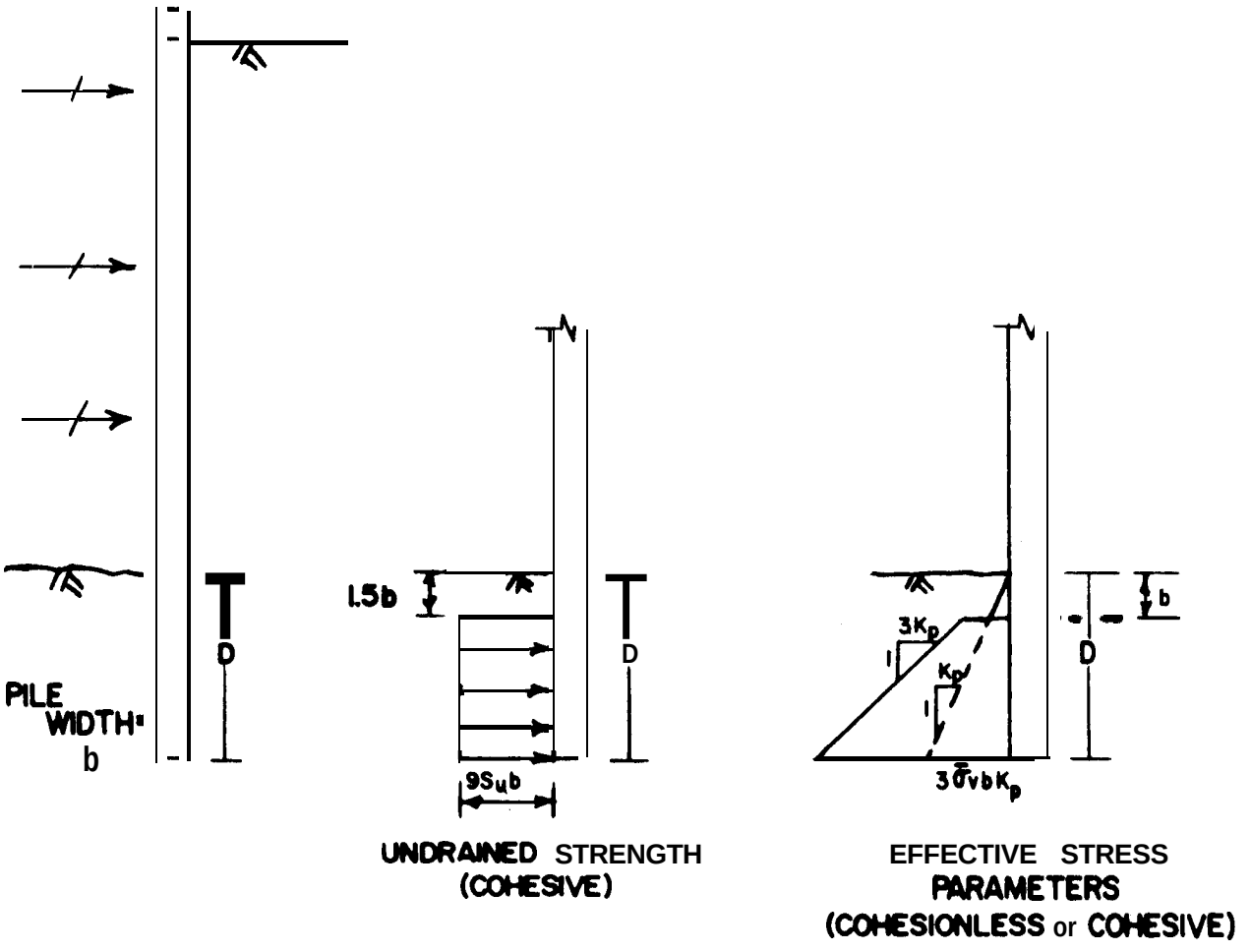


Figure 35. Passive pressure for soldier piles (after Broms, 1965). (Modified.)

6.40 OVERCUT DESIGN DETAILS

Over-excavation below the required support level depth is common either to obtain working room or to muck up the bottom, During intermediate excavation phases, assume a minimum of two feet of overcut before strut placement. At final depth, assume a minimum of one foot of over cut.

6.50 BERMS

Lateral resistance of berms will, of course, be lower than the case of a horizontal plate at the top elevation of the berm. One method of analysis is by wedge or logarithmic spiral force equilibrium of trial failure surfaces. Another procedure is to replace the berm with an equivalent sloping plane and assign the appropriate passive coefficient (Terzaghi and Peck, 1968; NAVFAC, 1971).

CHAPTER 7 - DESIGN ASPECTS OF LATERAL PRESSURE

The analysis of forces acting on a support wall, the related sizing of members, and the determination of wall penetration below the bottom of the excavation is related primarily to the wall stiffness and the type of wall. The wall stiffness is related to the ratio $\frac{EI}{L^4}$. For example, steel sheet piling and soldier pile walls with a typical wale spacing of eight feet or more and generally greater horizontal distance between support members are considered to be "flexible" walls. Design earth pressure diagrams should be determined in accordance with Chapter 5.

7. 10 LOAD ON SUPPORT LEVELS

Commonly,. wale loads are determined by area proportioning from the pressure diagrams developed from field measurements. This method for evaluating wale loads merely consists of reversing the procedure for developing the apparent earth pressure diagrams shown in Figure 28. This procedure is illustrated in Figure 36.

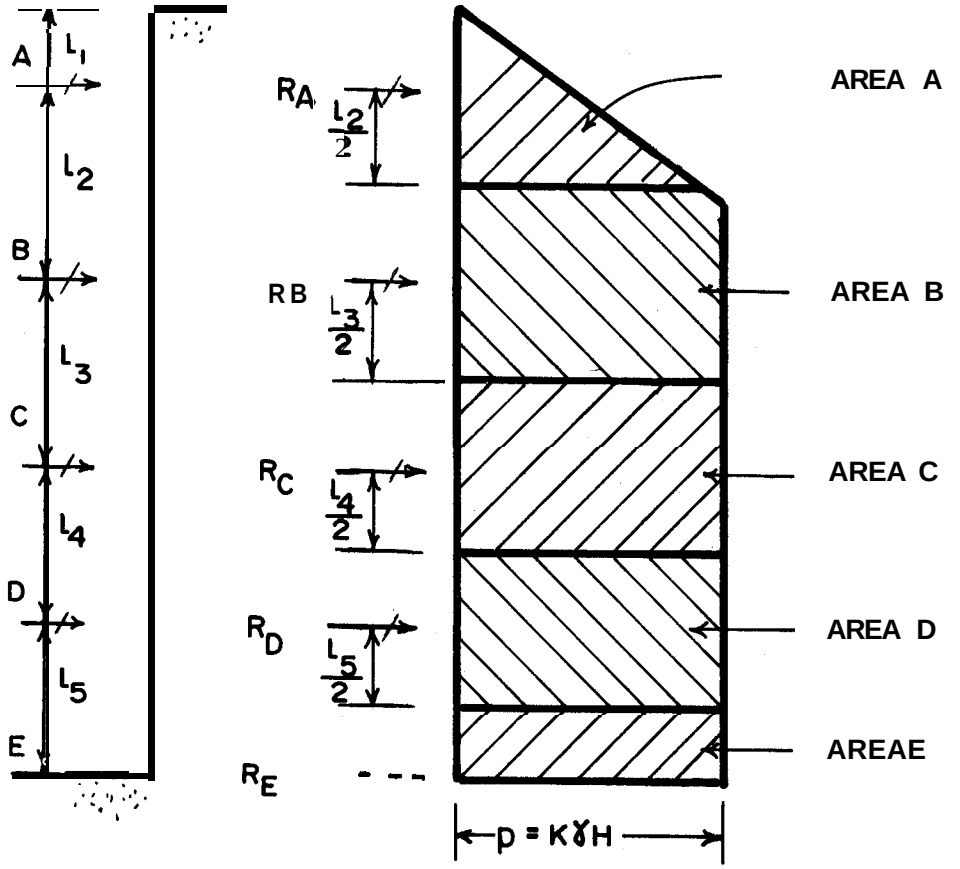
7. 20 ANALYSIS OF WALES AND SUPPORT WALLS

7. 21 General

Deflection of structural members supporting soil causes arching of earth resulting in a reduction of pressure near the center of spans and a concentration of pressure at the supports. Hence, the actual bending moments in wall elements and wales is less than that which would be computed assuming a uniform loading on these flexural members.

Several approaches have been used to determine moments in support members. Armento (1972), for example, used 80 percent of the uniform apparent pressure and computed moments assuming hinges at support levels. Peck, Hanson, and Thornburn (1974) propose using using 2/3 of the apparent pressure and assuming continuity over supports in computing moments.

The approach used herein, for moment computation in wales and support walls, is to use 80 percent of the loading diagram. For evaluation of loads in internal bracing and tiebacks, the full loading



STRUT LOAD PER LINEAL FOOT OF WALL IS EQUAL TO DESIGNATEDAREA

EXAMPLE: $R_C = p\left(\frac{L_4}{2} + \frac{L_3}{2}\right)$

Figure **36,** Load determination from apparent earth pressure diagram.

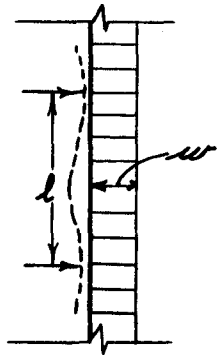
diagram (100 percent) is used. This recommendation is linked to a number of other associated design recommendations -- the pressure diagram itself, methods for moment computation, preload practice, allowable stresses, etc.

Where rigid walls support the earth, such as diaphragm walls with $\frac{EI}{L^4}$ greater than 50 ksf/ft, arching will be minimal; therefore, structural design of the wall as well as other elements should be based on the full pressure diagram,

7.22 Continuous Members

The following expression should be used for computing moments over continuous members with uniformly applied loads:

$$M = C w l^2$$



where:

M = moment

C = moment coefficient

w = distributed load on span

1 = span length

Hinged ends would have a coefficient, C = +0.125, with a maximum positive moment in the center of the span, Fixity at each support (no rotation) results in a maximum negative moment at the support and a moment coefficient, C= 0.087. Since construction methods greatly influence the position of the elastic line of members (especially vertical members), there is no practical way that the moment can be precisely analyzed. Therefore, a coefficient of C = 0.10 is recommended for continuous members supporting a uniform distributed load.

7.23 Discontinuous Wales

The moment in the wale will depend on the splice detail. For splices which occur at a strut and tie the wale with a steel strap,

to transfer shear hut not moment, zero moment should be assumed at that point.

Wales supporting uniform load with moment splices over less than three spans, should not be considered continuous. Three spans or more should be considered continuous using a moment coefficient, C = 0. 10.

The moment in wales supporting concentrated load (as from soldier piles or tiebacks) should be calculated on the basis of statics. Assume full continuity where moment splices are used; assume zero moment in other splices.

7. 24 Member Connections

It is common to design splices for the full structural capacity of the member (both shear and moment). This is often done with a combination of fully penetrating butt welds and cover plates.

Figures 37, 38, and 39 show some typical details for splices and wale to strut connections. For splices that are butt welded it is often assumed that the butt weld is only 50 percent to 75 percent effective since the beveled edges at the splice are field cut. Hence, the cover plates are designed to carry 25 percent to 50 percent of the member capacity. In designing a strut to wale connection, stiffness must be provided to prevent web crippling. Also, if raked struts are used, a knee brace is required at the strut to prevent buckling of the wale from the vertical component of load.

7. 25 Lagging

Arching of soil to the soldier piles results in substantial reduction of loads on lagging between soldier piles. This reduction depends on soil type and construction procedure, and it is not possible to predict by rational analysis. Therefore, the determination of lagging size is largely based on the past experience of the construction industry. The soldier pile section in Volume III (Construction Manual) summarizes recommended lagging sizes versus soil type and excavation depth.

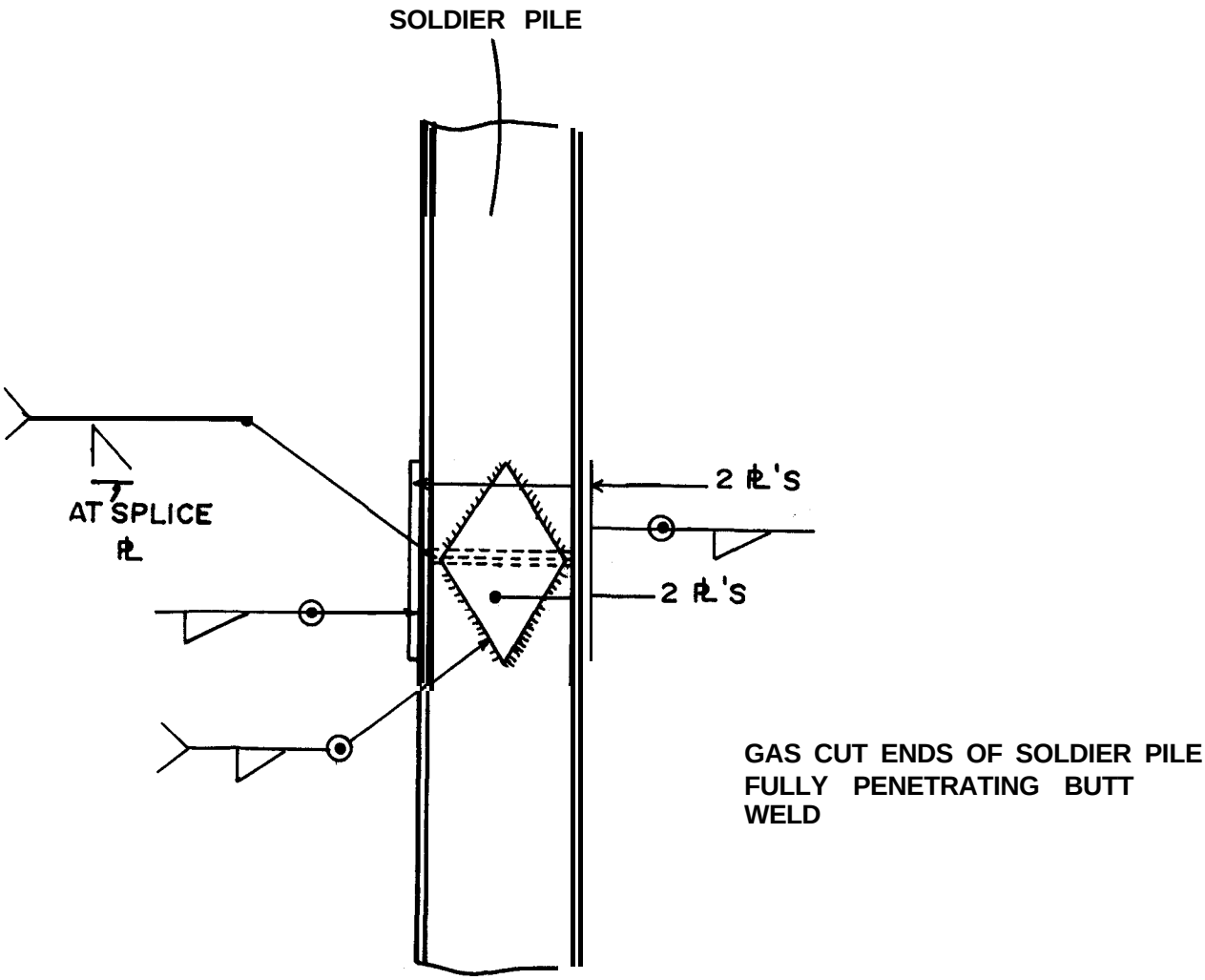


Figure 37. Typical splice with butt welding,

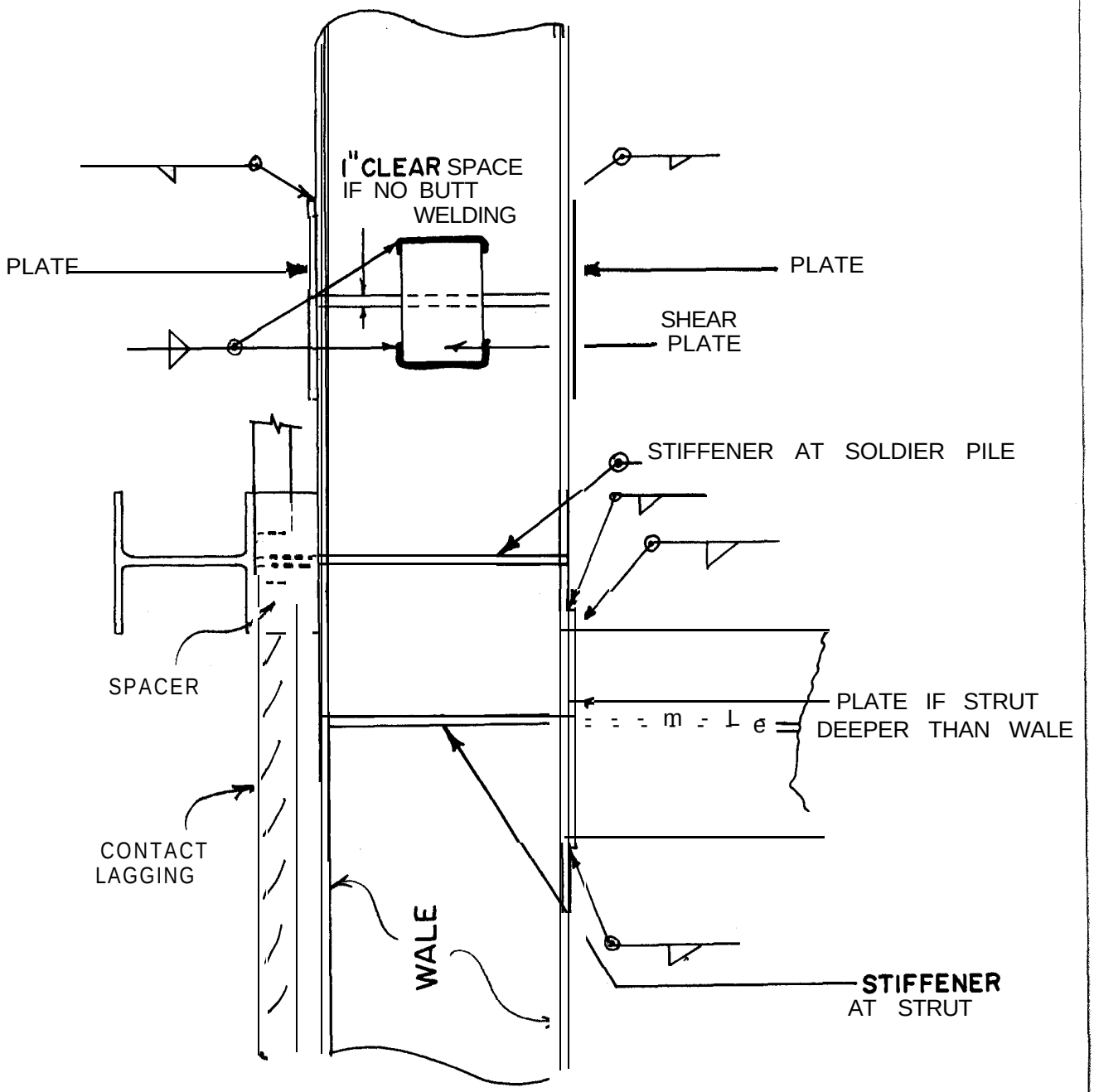


Figure 38. Plan view of typical wale splice and strut connection.

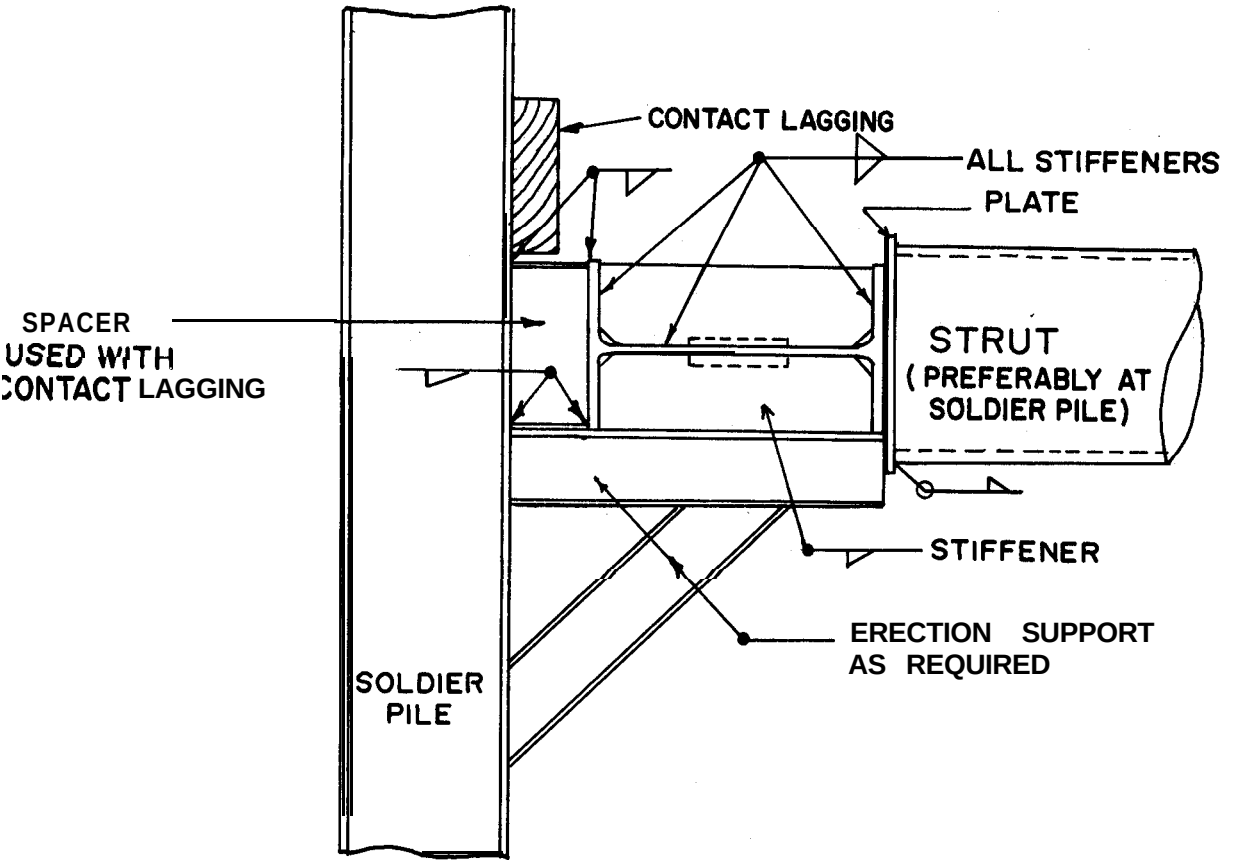


Figure 39. Typical strut -wale- soldier pile connections (elevation view).

7.30 BRACING AND TIEBACKS

Bracing and tieback loads must be determined for the most critical construction condition. This may be an intermediate depth of cut or at full depth.

For bracing, the allowable axial loads are governed by the member's slenderness ratio. Posting and lacing may be necessary to cut down on unsupported length to provide economical bracing members. Pipe sections may be utilized because of their efficiency as column members. Wide flange sections with vertical webs are also efficient, but this orientation may complicate wale connections.

For bracing:

a. At final depth, use allowable stresses by AISC Code.

b. For temporary conditions at intermediate depth of excavation use AISC t 20 percent.

For tiebacks, use the stress values stated in Volume III, Chapter 6 (Tiebacks).

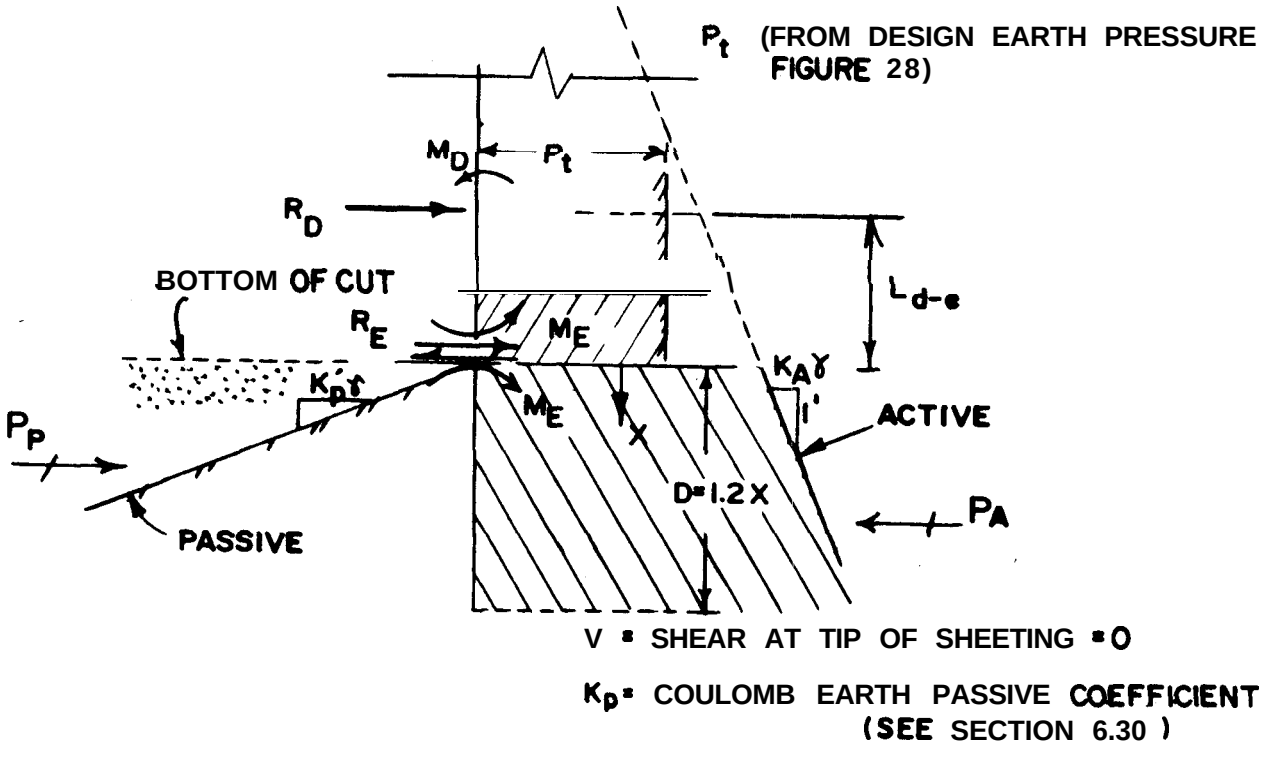
7. 40 DEPTH OF PENETRATION BELOW CUT

7. 41 Lateral Resistance

When design pressure diagrams are used, a reaction at the base of the cut is assumed to exist which is equal to the lowest area shown in Figure 36. This reaction is provided by the passive resistance of the soil beneath the cut. The magnitude of the passive resistance is analyzed for continuous walls using the modified Coulomb earth pressure coefficients given in Section 6. 30. Passive resistance for soldier piles reflects an added resistance due to soil arching as explained in Chapter 6.

Figure 40 illustrates the method for determining the depth of penetration in competent soils that are capable of developing adequate passive resistance. Soils satisfying this condition are medium-dense to dense granular soils and stiff to hard clays. The general method of analysis is:

a. Compute the equivalent reaction at the base of the cut (R_E).



1. Compute $R_E = 0.5\ p_t\ L_{d\ e}$
2. Compute depth x such that: $P_p = R_E + P_A$
   Use minimum F. S. = 1. 5 for passive coefficient, $K'_P = \frac{K_p}{1.5}$
3. Check $M_{max} \leq$ yield moment of sheeting
4. Drive to depth D = 1. 2x

Figure 40. Procedure for determining depth of penetration in relatively uniform competent soil conditions.

b. Determine the depth required to satisfy force equilibrium on the horizontal plane.

c. Check the maximum moment at or below RD against overstressing of the support wall.

d. Drive sheeting to a depth 20 percent greater than that required for force equilibrium.

In cases where the soils below the base of the cut are very loose to loose granular soils or soft to medium clays, the sheeting should penetrate to only a minimal depth of approximately 20 percent of the excavation depth and be designed as a cantilever below RD. The reason is that in loose granular soils the sheeting must experience large lateral deformation before building significant passive resistance. It is probable the sheeting will be overstressed before this deformation is attained. Hence, the sheeting will either act as a cantilever, or if it is driven to great depths it will act as a simple beam with a substantial span. In either case, the sheeting would most likely be overstressed at the lowest strut level. If the cut is underlain by soft to medium clays, the net pressure on the sheeting often is in the active state, hence there is theoretically a net active pressure. This situation can arise in deep cuts even when the base of the cut is stable as illustrated in Figure 41.

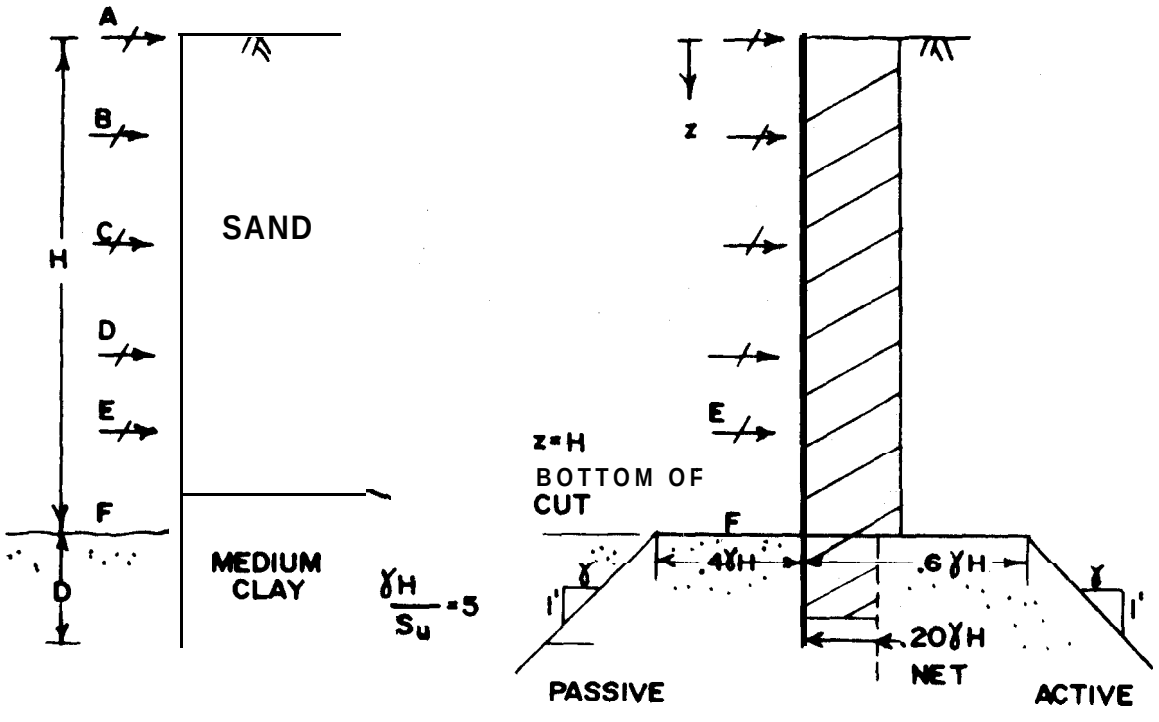
7.42 Bearing Capacity Considerations

Load capacity must be evaluated when there is a downward component of load, as is the case for inclined tiebacks. This may be accomplished using pile driving formulas or by the empirical and semi-empirical methods outlined in Chapter 9.

7.50 EXAMPLE PROBLEMS

7.51 Introduction

Three example problems are analyzed to illustrate methods of evaluating the depth of penetration required for sheeting stability and to show the effect soil stratification can have on the variation in strut load. The se example s, shown at the end of this chapter, consider the following three conditions:



$$s_u = \frac{\gamma H}{5} = 0.20\gamma H$$

RANKINE ACTIVE = $\gamma H - 2(.20\gamma H) = 0.60\gamma H$ AT $z = H$

RANKINE PASSIVE AT $z = H = 0$ t $2(.20\gamma H) = 0.40\gamma H$

1. Theoretical passive resistance is not available below bottom of cut to develop horizontal reaction. In fact, the net force below cut is theoretically toward excavation, based on active and passive pressure.

2. Use nominal penetration of 0.2 H or 5 feet whichever is greater, or penetration to cut off pervious layers.

3. **Check base stability (see Chapter 6).**

4. **Design for cantilever condition below E.**

Figure 41. Method for analyzing sheeting with weak under Lying layer.

Case I. Homogeneous soil profile.

Case II. "Soft" soil stratum to base of excavation underlain by a dense stratum.

Case III. A soft layer underlying a more competent soil.

Above the base of the excavation an empirical design pressure diagram is used as shown in Chapter 5. Below the base level Rankine active and passive pressures are used.

It is recognized that when using Peck's design envelopes the largest strut load for any condition is taken into consideration, However, a review of the development shows that in a few cases, strut loads gave apparent pressures which were greater than the de sign envelope s. The intent of this exercise is to illustrate a means by which a designer may estimate the magnitude of a strut load for unique conditions as well as providing a basis for judging whether or not to increase the design load on a given strut over that predicted by the design envelope. In addition, the analytical approaches will aid in the evaluation and understanding of observed strut loads obtained from instrumentation programs.

i'. 52 Results of Analysis

Case I is the analysis of a homogeneous soil profile which provides abasis for comparison of required penetration depth and strut load variations. It represents, most ideally, the conditions where the design envelope is appropriate. The method for analyzing soldier piles set in concrete-filled, pre-augered holes is also presented.

Case II analyzes the effect of a weak soil overlying a more competent one. It illustrates how the load in the second lowest strut can exceed that of the lowest strut.

As the excavation proceeds below level D to level E, little passive resistance is provided to the retaining wall; hence, the wall deflects inward. Effectively, the wall spans from level D to the excavation base with full active pressure applied and no passive resistance. The deformation of the sheeting is such that, during this excavation stage, it resists essentially the same load over the span D to F whether or not strut level E is installed. This would be particularly true in the stiffer diaphragm walls. The effect of this

large unsupported length is twofold:

a, Since the sheeting has already assumed an elastic line such that it resists the full active load, little load is transferred to strut level E. Hence, strut level D effectively takes a disproportionate share of the load.

b. The moment in the sheeting is greatly increased by the long unsupported length. As expected, the required depth of penetration for the sheeting will be significantly less than for Case I.

Case III depicts a method for evaluating the maximum strut load when a relatively weak soil layer starts immediately beneath the base of the excavation. Since little passive resistance can be expected from the weak layer the sheeting acts as a cantilever member; thus, a large load is developed in the lowest strut. For these conditions, where the base is stable against bottom heave, little is gained from driving the sheeting to any depth below the bottom of the cut. Therefore, a minimum penetration is recommended of five feet or 20 percent of the excavation depth, whichever is greater. In situations where the base is unstable, consideration may be given to deeper penetration and stiffer sheeting to prevent bottom heave.

7.60 FINITE ELEMENT ANALYSIS OF BRACED EXCAVATIONS

7.61 Introduction

In recent years several computer programs, based on the finite element methods of analysis, have been developed to analyze braced excavations (Wong, 1971; Palmer and Kenney, 1972; Jaworski, 1973; Clough and Tsui, 1974). Currently, the primary use of these programs is to provide insight into the behavioral trends of braced cuts. Using computer programs parametric studies can be conducted to evaluate, at least in a qualitative manner, the effect of wale spacing, sheeting stiffne s s, and soil stratification on strut loads and sheeting deformation. Further, these studies may be used to provide guidelines for engineering judgement and for obtaining some qualitative verification of design assumptions.

7.62 Case Studies

To illustrate how the finite element programs can be

used as an aid to the design engineer, the program BRACE II (see appendix to Chapter 2 for description) was used to analyze three different soil profile s. These profiles are similar in concept to those described in Section 7. 50, except that cohesive soils are assumed due to program limitations.

As was the intent of the design examples of Section 7. 50, the results of these analyses are used to give some insight into the effect soil stratification and sheeting stiffness m.ay have on the variation of strut load and sheeting forces.

Specifically, four soil conditions were analyzed:

Case la. Homogeneous soil profile of soft, normally consolidated clays.

Case lb. Homogeneous soil profile of medium-stiff clay.

Case 2. A soft soil stratum above the base of the excavation underlain by a stiff stratum.

Case 3. A soft soil layer underlying a more competent stiff soil.

For all cases, the ground water table was taken at a five footdepth, andundrained soil parameters were assumed for both the shear strength (S_u) and deformation modulus (E_u). For soft and medium soils, these parameters increased linearly with depth within a stratum as a function of the vertical effective stress ($\bar{\sigma}_v$). For the stiff soils, the strength and modulus were considered constant with depth. The soil parameters used in the analysis are summarized as follows:

Soil Type	K_o	Saturated Unit Weight (pcf)	Shear* Strength (S_u)	Soil* Modulus (E_u)
Soft	0.5	120	$0.28\bar{\sigma}_v$	$200\bar{\sigma}_v$
Medium-Stiff	0.6	120	$0.40\bar{\sigma}_v$	$290\bar{\sigma}_v$
Stiff	1.0	120	3200	3×10^6 psf

* The parameters for the soft and medium-stiff cohesive soils are from Ladd, et al (1971). These parameters are normalized against the vertical effective stress on the soil, i. e. they are directly related to the effective overburden pressure $\bar{\sigma}_v$. Hence, the strength and modulus increase linearly with $\bar{\sigma}_v$ and therefore with depth in a soil stratum.

For the purpose of providing a basis of comparison, the cofferdam geometry was the same for all cases:

Strut spacing (L) = 10' c/c
Depth of excavation (H) = 60'
Width of Excavation = 60'
Sheeting penetration = 30'

Also, two wall types were considered in order to provide some information on the effect of wall stiffness. The wall types were a steel sheet pile wall (PZ-38) and a 4 foot thick concrete diaphragm wall.

7. 63 Distribution of Earth Pressures

Figures 42 and 43 show normalized apparent earth pressure diagrams predicted by the finite element analysis for the four soil conditions outlined in Section 7. 62. These apparent earth pressures were obtained in the same manner as shown on Figure 27 for apparent earth pressures from field measurements, The pressures are normalized by taking the ratio of the predicted apparent pressure at a strut level (P) to the maximum apparent pressure (P_{max}), both computed from the finite element analysis. The diagrams were developed using the maximum strut load computed in any strut level and during any stage of excavation.

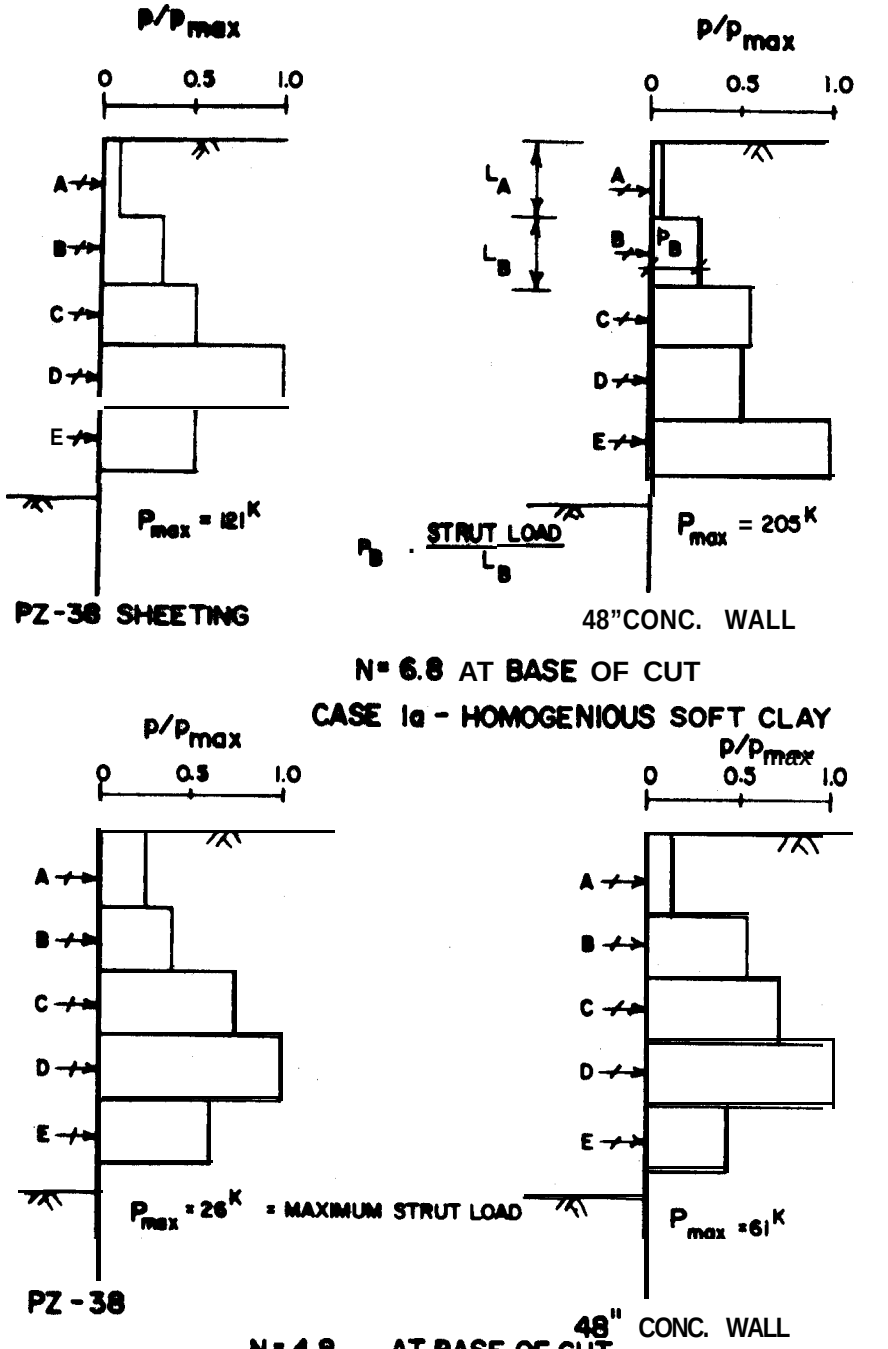
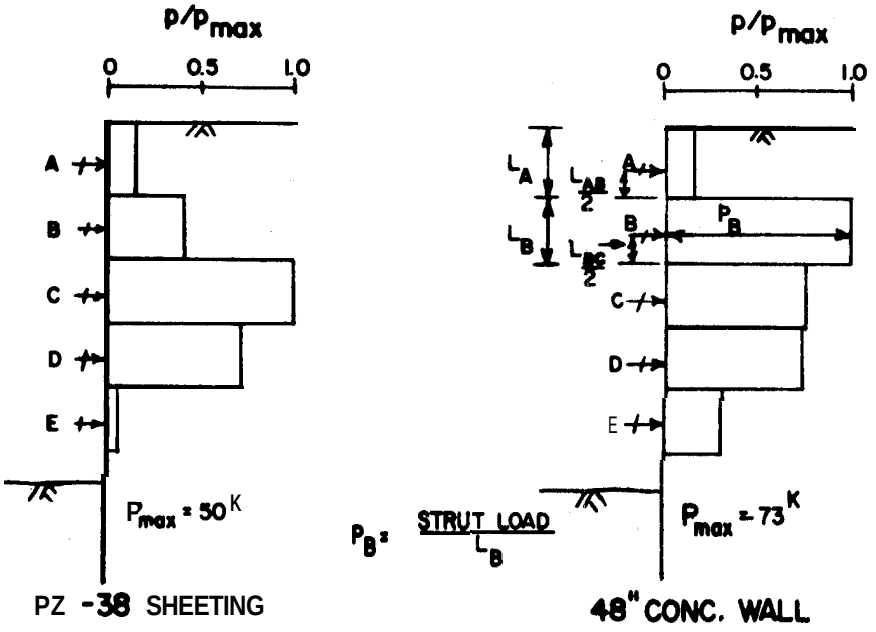


Figure 42. Normalized qpparent earth pressure diagrams predicted by finite element analysis.



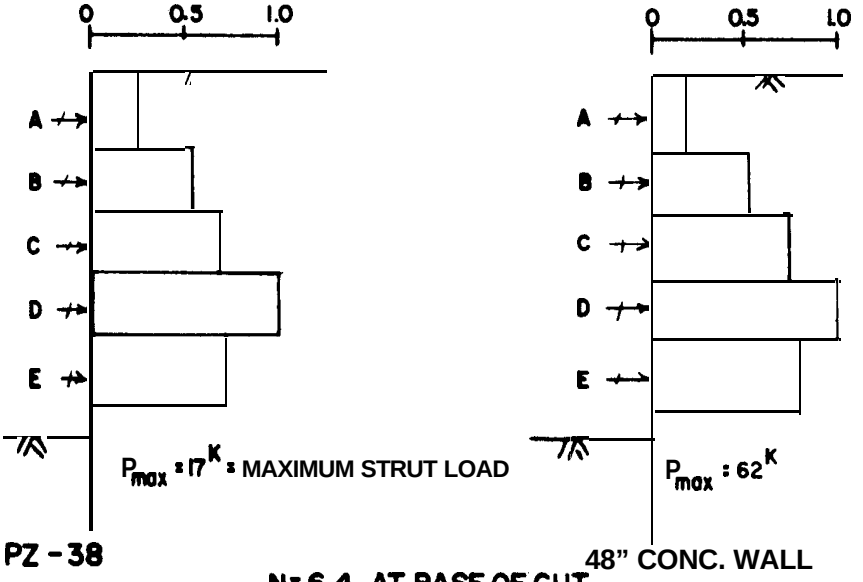


Figure 43. Normalized apparent earth pressure diagrams predicted by finite element' analysis,

Comparing Case la with Case lb in Figure 42, the analysis shows that walls in the soft clay should be expected to experience relatively higher pressures near the base of the cut than the wall in the medium- s tiff clay, This trend is more obvious for the stiffer concrete walls. In addition, the predicted earth pressures in the soft soil may be much higher, as shown by the maximum strut loads. The apparent reason for this behavior is the lack of lateral support below the base of the excavations; hence, an inward rotation around the lowest strut. The stability number of N = 6. 8 in Case la results in a factor of safety of less than 1. Hence, a bottom heave failure occurred which resulted in the Loss of passive resistance below the excavation Level. On the other hand, the stability number for Case lb is low (N ≃ 4. 8), resulting in a stable bottom. With increasing wall stiffness less curved deformation is expected, and the potential for unloading the second lowest strut and overloading of the lowest strut is increased. As Case lb shows, this behavior becomes less pronounced as the soil becomes stiffer . One possible remedy for reducing this effect in soft soils would be to prestress the second lowest strut and lock in a high residual compressive force.

On Figure 43, Case 2 (soft clay overlying stiff clay) shows opposite effect to that experienced in the homogeneous soil mass. This stiffer layer provides an adequate reaction for the wall, restricting its inward deflection in the overlying soft clay. This leads to a larger strut load in the second to last strut and a reduction in the load received by the lowest strut. This results because the wall has already deflected inward close to its maximum amount before the last strut is installed and final excavation completed. Therefore, this last excavation stage results in little load transfer to the lowest strut.

For Case 3, where the soils within the depth of cut are stiff, stability number N<4, and soft soils exist immediately below the base of the excavation, the results show that the strut loads are greatest in the second lowest strut. This occurs for the same reasons given for Case la, that is, lack of support below the excavation base. For this soil profile, the pattern of pressure distribution appears independent of wall rigidity since both give essentially the same normalized pressure diagram.

7.64 Magnitude of Strut Loads

Figure 44 shows the magnitudes of the predicted loads for Cases 2 and 3. In both cases, the diaphragm wall receives much greater apparent pressures, on the order of 2 to 4 times that of the more flexible PZ-38 steel sheeting,

The higher apparent pressures in the concrete wall are attributed to smaller lateral deformations, hence, less mobilization of shear strength in the soil adjacent to the wall. The stiffer wall tends to retain a large portion of the initial stresses, consequently its loading is more dependent on the K_o value of the supported stratum.

This behavior is particularly acute. in the heavily overconsolidated soils such as those assumed for Case 3. Considering the pressure diagrams for this case, the steel sheeting experienced a greater inward movement. Therefore, the high undrained shear strength of the soil was mobilized resulting in relatively low apparent pressures compared to those for the concrete wall which experienced little inward movement.

There is scant field evidence to support this trend. Observations of tied-back walls in heavily overconsolidated clays show that the walls move Laterally with time. The movement may be associated with the stress relief and subsequent lateral swelling of the soil. This swelling is time dependent and could result in the build up of strut Loads somewhere between the Rankine active stress and the initial horizontal stresses in the clay. Inward bulging of Lagging was observed in an internally braced cut made in overconsolidated soil in the Washington, D. C. area. The severity of the bulging increased with time suggesting a. load build up on the lagging and hence, an increase of load in the support system. In any case, when over consolidated soils are present, one should be aware that loads may build up on the support system with time causing overloading, especially if a relatively rigid wall is used which restricts the lateral swelling of the soil.

7.65 Structural Behavior

Table 4 compares the predicted location of zero moment and zero shear versus the sheeting stiffness for the three cases described in Section 7.61. For all conditions analyzed

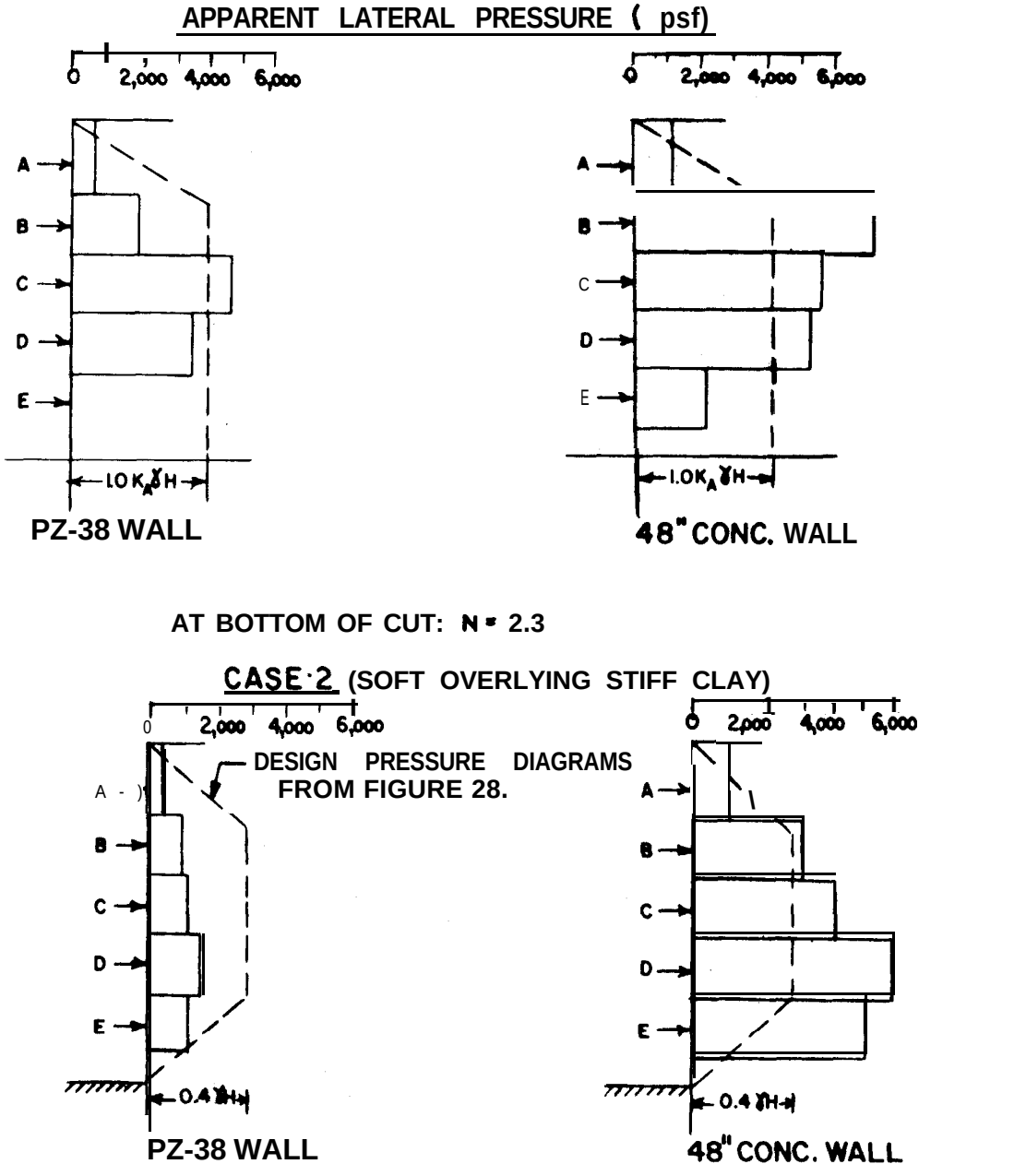


Figure 44, Comparison of predicted apparent earth pressures from finite element analysis on stratified soils.

Table 4. Summary of structural behavior for braced walls from finite element analysis.

Case	Wall Type	$N = \frac{\gamma H}{S_u}$ (1) At Base (2)	$N = \frac{\gamma H}{S_u}$ (1) Within Depth of Cut (3)	Wall Stiffness (4) $\frac{EI}{L4}$ (ksf)	Location of Zero Moment Above Base (4)	Location of Zero Moment Below Base	Depth Below Base to Zero Shear
Uniform Soil	PZ-38 Steel	6.8	11.8	5.8	9 ft.	>30 ft.	30 ft.
	PZ-38 Steel	4.8	8.2	5.8	>10 ft.	13 ft.	19 ft.
	4' Diaphragm Wall	4.8	8.2	229.0	>10 ft.	13 ft.	19 ft.
	4' Diaphragm Wall	6.8	11.8	229.0	>10 ft.	>30 ft.	>30 ft.
2 Stiff Soil Below Base of cut	PZ-38	2.3	9.0	5.8	2 ft.	>30 ft.	2 ft.
	PZ-38 (5)	2.3	9.0	5.8	3 ft.	>10 ft.	4 ft.
	4' Diaphragm Wall	2.3	9.0	229.0	>10 ft.	2 ft.	7 ft.
3 Soft Soil Below Base of Cut	PZ-38	6.4	2.2	5.8	7 ft.	11 ft.	19 ft.
	4' Wall Diaphragm	6.4	2.2	229.0	10 ft.	17 ft.	19 ft.

(1) All excavations 60 ft. deep; wall penetration 30 ft. below base of excavation.

(2) Based on S_u at base of excavation.

(3) Based on average S_u within the depth of the excavation

(4) Strut spacing 10 ft. c/c; lowest strut 10 ft. above base of excavation.

(5) Assumed depth of penetration below base was 10 ft.

except one; the sheeting penetration was assumed to be 30 feet, The exception was for Case 2 where, for one analysis, the PZ-38 sheeting penetration was assumed to be 10 feet to provide a comparison of the effect of sheeting penetration in stiff soils.

It is important to recognize that the finite element method is for analysis of a soil condition; therefore, the 30 foot depth of penetration was chosen to provide information on the effective depth of sheeting.

7.66 Implications of Finite Element Analysis

A comparison of the behavioral trends predicted by the finite element method (FEM) with the results of the simplified analytical method (SAM) described in Section 7.50 shows the benefits of performing finite element studies. Even though such a comparison is not theoretically justified since two different soil types are considered, the results from the FEM are indicative, in a general sense at least, of the type of behavioral pattern one might expect for the soil profiles considered,

For example, Figures 42 and 43 indicate that sheeting stiffness has little effect on the apparent normalized earth pressure diagrams. Figure 44, however, suggests that sheeting stiffness has a marked effect on the magnitude of the loads, The stiffer sheeting gives higher apparent earth pressures. This trend stems from the reduction in ground movement achieved by the stiffer wall versus the movements associated with the more flexible steel sheeting. This behavior is not inherently considered in the SAM. For this latter method, one must rely on engineering judgement to account for the effects of sheeting stiffness on strut loads.

Regarding Case 2, the SAM approach predicts higher strut loads on the second from the bottom strut and low strut loads at the lowest strut. The finite element analysis indicates a similar trend, i. e. , the predominance of load is in the upper strut levels. However, with the FEM the greatest loads are higher. For example, the maximum load is in strut B.

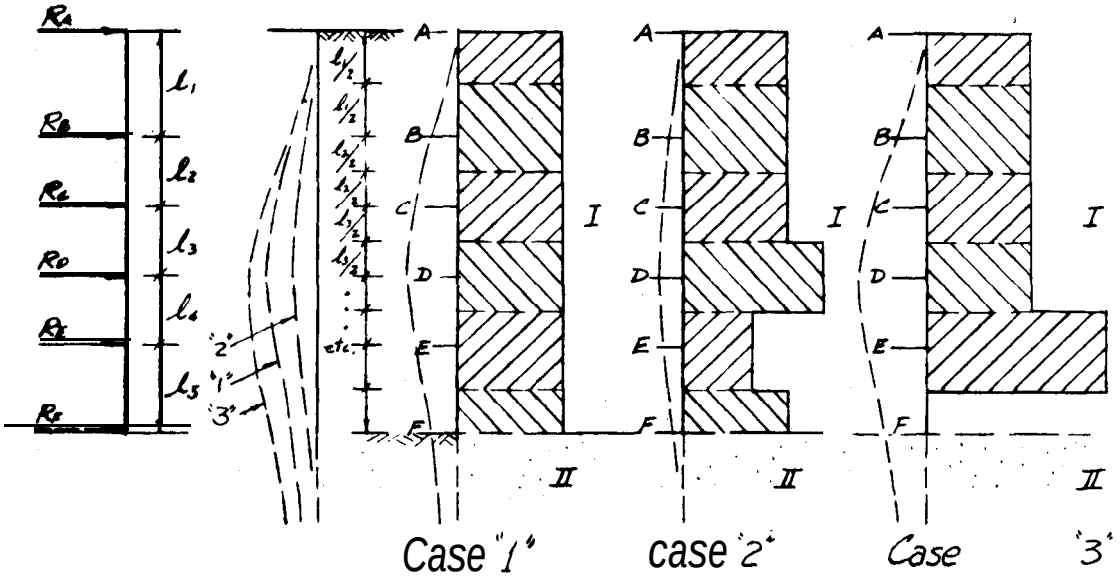
7. 67 The Use of the Finite Element Method in Design

The finite element method is a more realistic mathematical modeling of the complex soil profile and the soil-structure system, thus making it a powerful tool in the analysis of supported excavation s. However, this method should be used cautiously. The computer programs are developed by making various assumptions concerning soil behavior. Naturally, these assumptions have a large effect on the accuracy of the results. The results are only as good as the input, particularly the soil parameters assumed. Therefore, these programs should be used only by experienced individuals who are aware of the assumptions used in the program development and how the assumptions could possibly effect the results. Finally, when making a finite element analysis, the results should be carefully evaluated for consistency and behavioral trends. The results may appear correct, but because of inappropriate input or misapplication of the program, the analysis may be giving erroneous-results.

It is recommended that until substantially more experience is gained with the FEM as a design tool it be used primarily as an aid to guide engineering judgement.

GENERAL OVERVIEW OF EXAMPLES

Granular Soils Within Depth of Cut



Case "1" Uniform soil throughout

Case "2" Stratum II much denser and unyielding

Case "3" Stratum II weaker and more yielding than Stratum I.

BASIC APPROACH FOR DETERMINATION OF STRUT LOAD

With reasonably uniform spacing between levels of support:

Case "1" Apportion load by area distribution

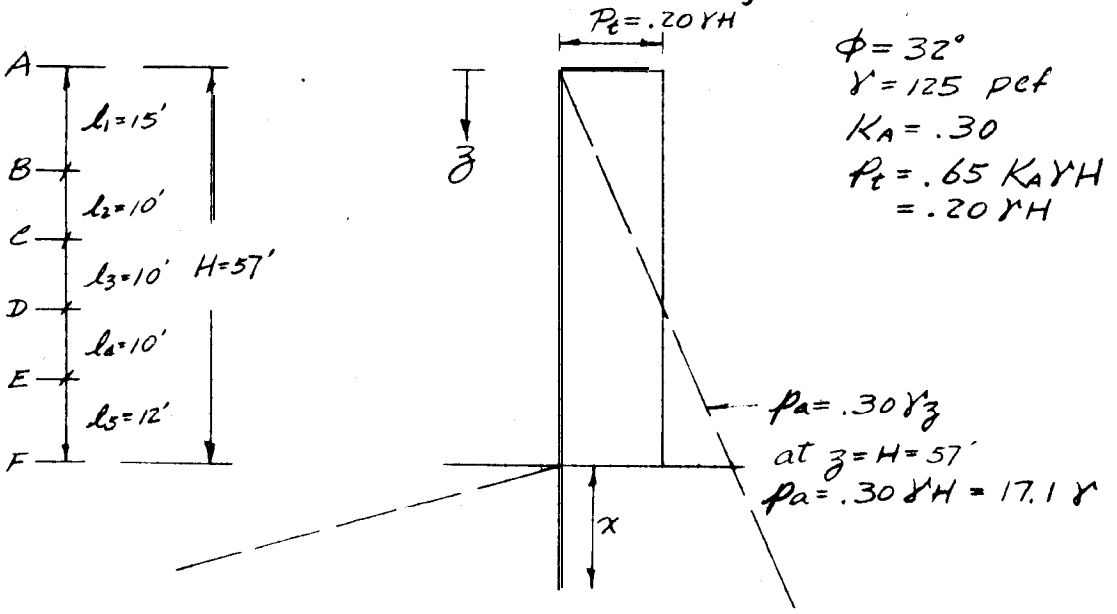
Case "2" Same as Case "1" except, analyse condition of support set at D and excavate to E. Assume concentrated reaction at F and at D. Neglect passive force in span EF.

Decrease force at E by the same amount that force at D is increased above Case "1"

Case "3" Same as Case "1" except, assume $R_F = 0$. Therefore reaction at E takes all pressure between E and F. Examine cantilever moment at E.

ANALYSIS OF CASE "1"

Uniform medium dense sand throughout



$R_A = (7.5)(.20\gamma H) = 1.5\gamma H$

$R_B = (\frac{15}{2} + \frac{10}{2})(.20\gamma H) = 2.5\gamma H$

$R_C = (\frac{10}{2} + \frac{10}{2})(.20\gamma H) = 2.0\gamma H$

$R_D = \quad " \quad = 2.0\gamma H$

$R_E = (\frac{10}{2} + \frac{12}{2})(.20\gamma H) = 2.2\gamma H$

$R_F = (6.0)(.20\gamma H) = 1.2\gamma H$

Depth of penetration for Continuous Wall

Follow procedure in Figure 40.

Let $\delta = \frac{\phi}{2} = 16°$ $\therefore K_p = 5.5$

Use F.S. = 1.5 $\therefore K_p' = \frac{5.5}{1.5} = 3.7$

$P_p = \frac{1}{2}\gamma x^2 (3.7) = 1.85\gamma x^2$

$P_a = 17.1\gamma x + \frac{1}{2}\gamma x^2(.30) = .15\gamma x^2 + 17.1\gamma x$

$R_F = 1.2\gamma H = 68.4\gamma$

$\Sigma H = 0$

$68.4\gamma + .15\gamma x^2 + 17.1\gamma x - 1.85\gamma x^2 = 0$

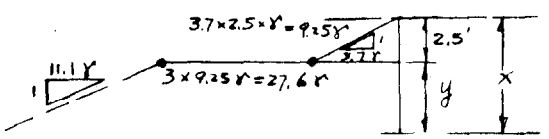
$1.7x^2 - 17.1x - 68.4 = 0$

$x = 13'$ $D = 1.2 \times 13 = 15.6'$

Say 16'

~~Depth of penetration for Soldier Pile~~

Assume soldier piles predrilled and encased in concrete, pile diameter $d = 2.5'$, For depth of 2.5' consider passive resistance reacts only on the width of soldier pile. $K_p' = 3.7$ $(P_p)_u = \frac{1}{2} \times 9.25\gamma (2.5)^2 = 29.0\gamma$



Below 2.5' K_p acts on 3 × width of pile diameter

$3K_p' = 3 \times 3.7 = 11.1$

$(P_p)_L = (27.6\gamma y + \frac{11.1}{2}\gamma y^2) \times 2.5$

$= 13.9\gamma y^2 + 69.0\gamma y$

Let soldier piles be at 8.0' O.C.

$\therefore R_F = 8 \times 68.4\gamma = 547\gamma$

$P_A = 8 \times (.15\gamma x^2 + 17.1\gamma x) = 1.2\gamma x^2 + 136.8\gamma x$

$P_p = 13.9\gamma y^2 + 69.0\gamma y + 29.0\gamma$

$x = 2.5 + y$

Try $x = 13'$, $y = 10.5'$

$R_F + P_A - P_p = 0$

$547\gamma + 202.8\gamma - 1532.5\gamma$

$1778.4\gamma - 724.5\gamma = +242.2$

$1981.2\gamma - 29.0\gamma$

2286γ

Try $x = 15'$, $y = 12.5'$ $\quad R_F + P_A - P_P = 0$

$$547\gamma + \begin{array}{r} 270\gamma \\ 2052\gamma \\ \hline 2322\gamma \end{array} - \begin{array}{r} 2171.9\gamma \\ 862.5\gamma \\ 29.0\gamma \\ \hline 3063.4\gamma \end{array} = -194.4$$

Say $x = 14.0'$

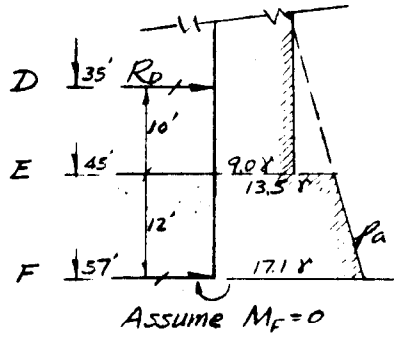
$D = 1.2x = 16.8'$ Say $17.0'$

ANALYSIS OF CASE "2"

Let stratum I be medium dense sand
stratum II be glacial till (very dense clayey sand some gravel) $\phi = 40°$

Reactions:

$R_A = 1.5\,\gamma H$ (same as before)
$R_B = 2.5\,\gamma H$ (" ")
$R_C = 2.0\,\gamma H$ (" ")
R_D Examine condition of support at D and excavate to E



@ 45' $p_t = 45 \times .20\gamma = 9.0\gamma$
$p_a = 45 \times .30\gamma = 13.5\gamma$

@ 57' $p_a = 57 \times .30\gamma = 17.1\gamma$

$\Sigma M_F = 0$ $\quad (R_D)_L = \frac{1}{22}\left[(9.0)(10)(17) + (13.5)(12)(16) + (3.6)\frac{(12)}{2}\times(4)\right]\gamma$

$= 117.7\,\gamma$
$= 2.06\,\gamma H$

$(R_D)_u = 1.0\,\gamma H$

$\therefore\ R_D = (R_D)_u + (R_D)_L = 3.06\,\gamma H$

$R_E = (2.0\,\gamma H + 2.2\,\gamma H) - (3.06\,\gamma H) = 1.14\,\gamma H$

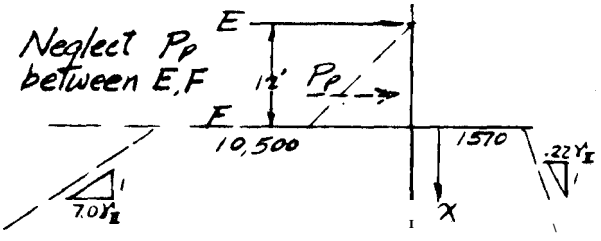
$R_F = 1.20\,\gamma H$ (same as before)

Depth of penetration for Continuous Wall

Follow procedure outlined in Figure 40.

(a) Check Temporary condition of brace at R_D and excavate to E

$$R_F = -(R_D)_L + (9.0)(10)\gamma_I + (13.5)(12)\gamma_I + (3.6)\frac{(12)}{2}\gamma_I$$
$$= -117\gamma_I + 90\gamma_I + 162\gamma_I + 22\gamma_I$$
$$= 157\gamma_I = (157)(125) = 19{,}600 \text{ lb.}$$



I Sand: $\phi = 32°$ $\gamma_I = 125$
$\delta = \frac{\phi}{2}$
$K_A = .22$, $K_p = 5.5$

II Glacial Till $\phi = 40°$ $\gamma_{II} = 140$
$\delta = 25°$
$K_A = .22$, $K_p = 10.5$

@ F

On active side $Z = 57'$

$P_a = (57 \times 125)(.22) = 1570$ psf

$K_A \gamma_{II} = .22 \times 140 = 30.8$

On passive side $Z = 12'$

$K_p' = \frac{K_p}{F.S. = 1.5} = \frac{10.6}{1.5} = 7.0$

$P_p = 7.0 \times 12 \times 125 = 10500$

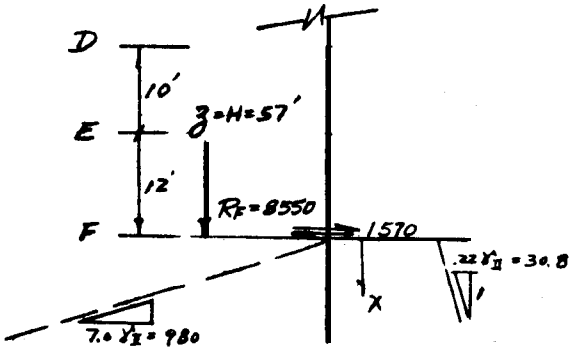
$K_p' \gamma_{II} = 7.0 \times 140 = 980$

$\Sigma H = 0$ $\quad (10{,}500 - 1{,}570)X + (980 - 30.8)\frac{X^2}{2} = 19{,}600$

$$474.6X^2 + 8930X - 19600 = 0$$
$$X^2 + 18.8X - 41.3 = 0$$
$$X = 2'$$

(b) Check full depth condition with brace at R_E and excavate to F

$$R_F = 1.20\gamma_I H = 1.2 \times 125 \times 57 = 8550$$



$\Sigma H = 0$

$$1570x + \frac{30x^2}{2} - \frac{980x^2}{2} + 8550 = 0$$

$$475x^2 - 1570x - 8550 = 0$$

$$x^2 - 3.3x - 18.0 = 0$$

$$x \approx 6'$$

$$D = 1.2 \times 6 = 7.2'$$

Say 8.0'

Note:

1. Full depth condition controls
2. $D = 8.0'$ Case "2"
   $D = 16.0'$ Case "1"

ANALYSIS OF CASE "3"

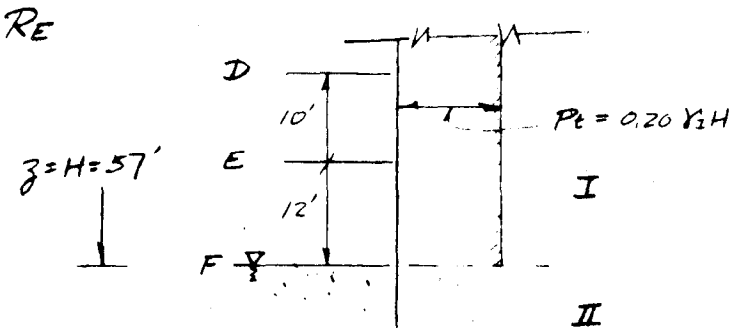
Let stratum I be medium dense sand
stratum II be loose stratified fine sand & silt
$\phi = 25°$ $\gamma_I = 110$ ground water cannot be lowered below top of silty soil

Analyze by procedure outline in Figure 41.

If limiting case condition of $R_F = 0$ is used, then theoretically no penetration is required below point F. Nominal penetration of 0.2 H (or ~ 5') should be used.

Reactions

$R_A = 1.5\ \gamma' H$ (Same as before)
$R_B = 2.5\ \gamma' H$ (")
$R_C = 2.0\ \gamma' H$ (")
$R_D = 2.0\ \gamma' H$ (")



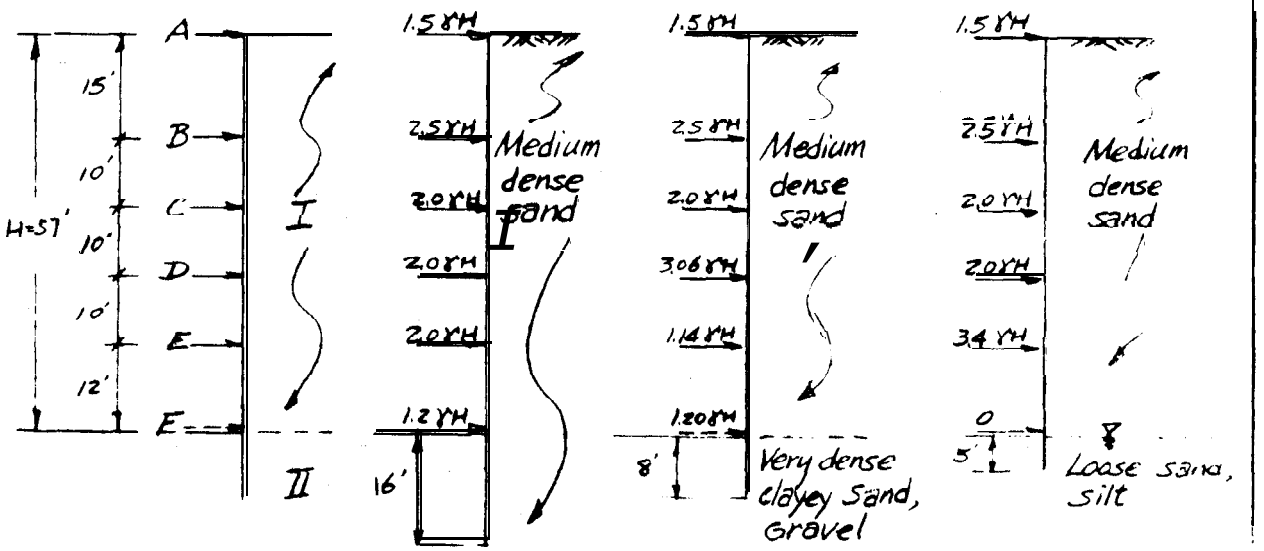
Assume $R_F = 0$

$$R_E = .20\,\gamma H\left(\frac{10}{2}+12\right) = 3.40\,\gamma H$$

$$R_F = 0$$

Must check cantilever moment at E provide nominal say 5' penetration, below F

SUMMARY AND COMPARISON Of THREE CASES



Case "1"

I & II the same

Case "2"

II much denser than I

Case "3"

II much weaker than I

(Assume cantilever below E)

CHAPTER 8 - STABILITY ANALYSIS OF SHEETED EXCAVATIONS

8.10 GENERAL

There are three primary modes of instability of concern in sheeted excavations in clay:

a. Bottom heave (Figure 45a).

b. Deep seated failures (Figures 45b and 45c)

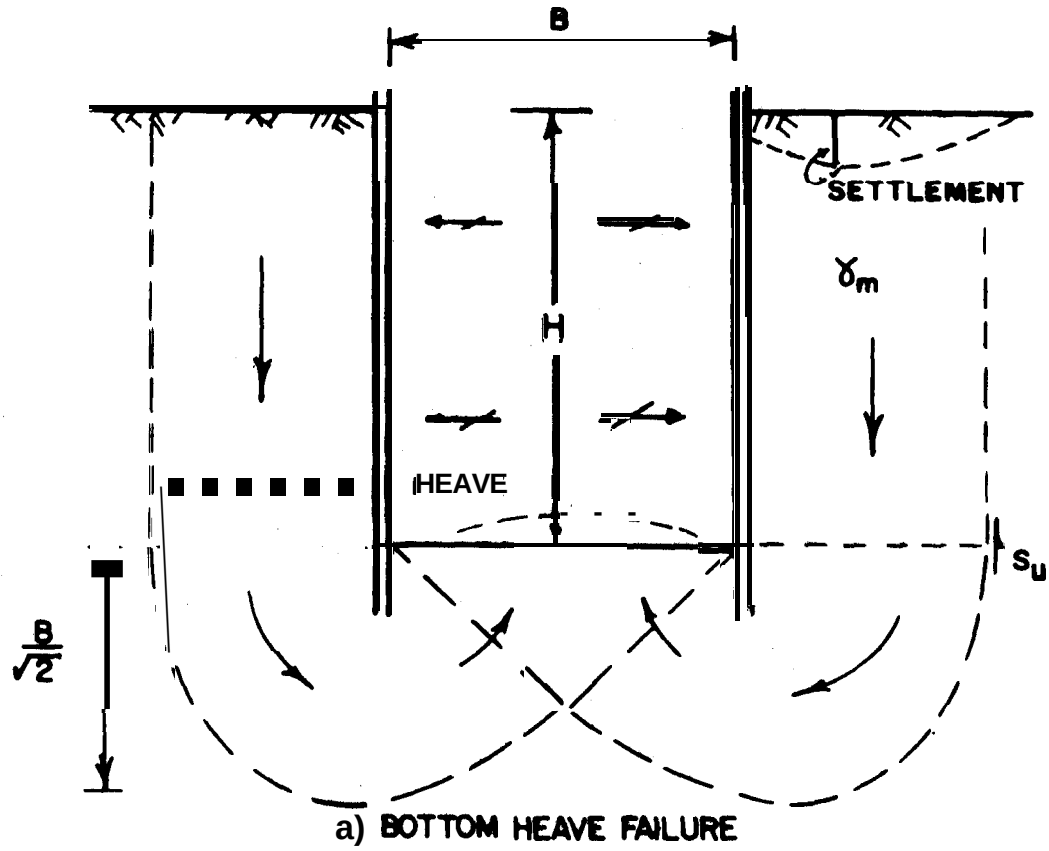
c. Local failures immediately adjacent to the support wall.

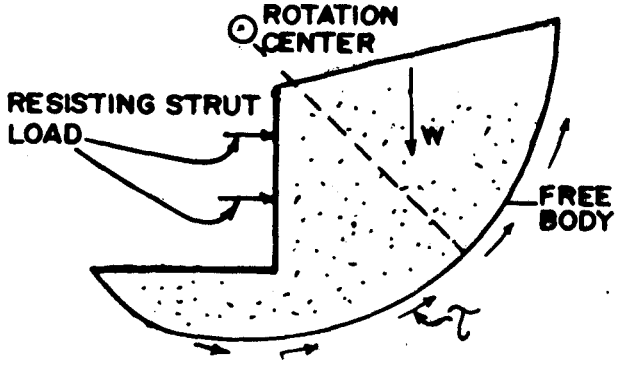
Of these modes, b). and c). are related to the overall stability of the excavations. They often will dictate the procedures to be used in constructing a cofferdam. These conditions of instability may also result in heavier strut loading than predicted by the method described in Chapter 5. An example of the potential for increased loading if bottom heave occurs is described in Section 7. 50.

Local failures (mode c) are of concern where it is necessary to limit inward sheeting deformations. Failures of this type occur below the excavation level immediately adjacent to the sheeting, resulting in partial loss of lateral support.. This loss of support creates a large unsupported length and can lead to excessive inward deflections of the sheeting.

8.20 BOTTOM HEAVE

Bottom heave is a problem primarily in soft to medium clays where the strength of the soil is nearly constant with depth below the base of the excavation. The failure is analogous to a bearing capacity failure, should be analyzed (Bjerrum and Eide, 1956) using the stability chart given in Figure 46. The factor of safety against a bottom heave is determined as:





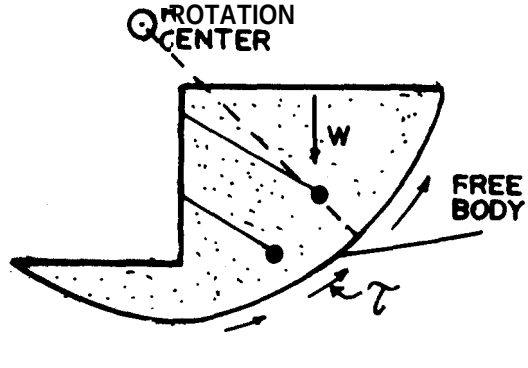


Figure 45. Potential failure surfaces.

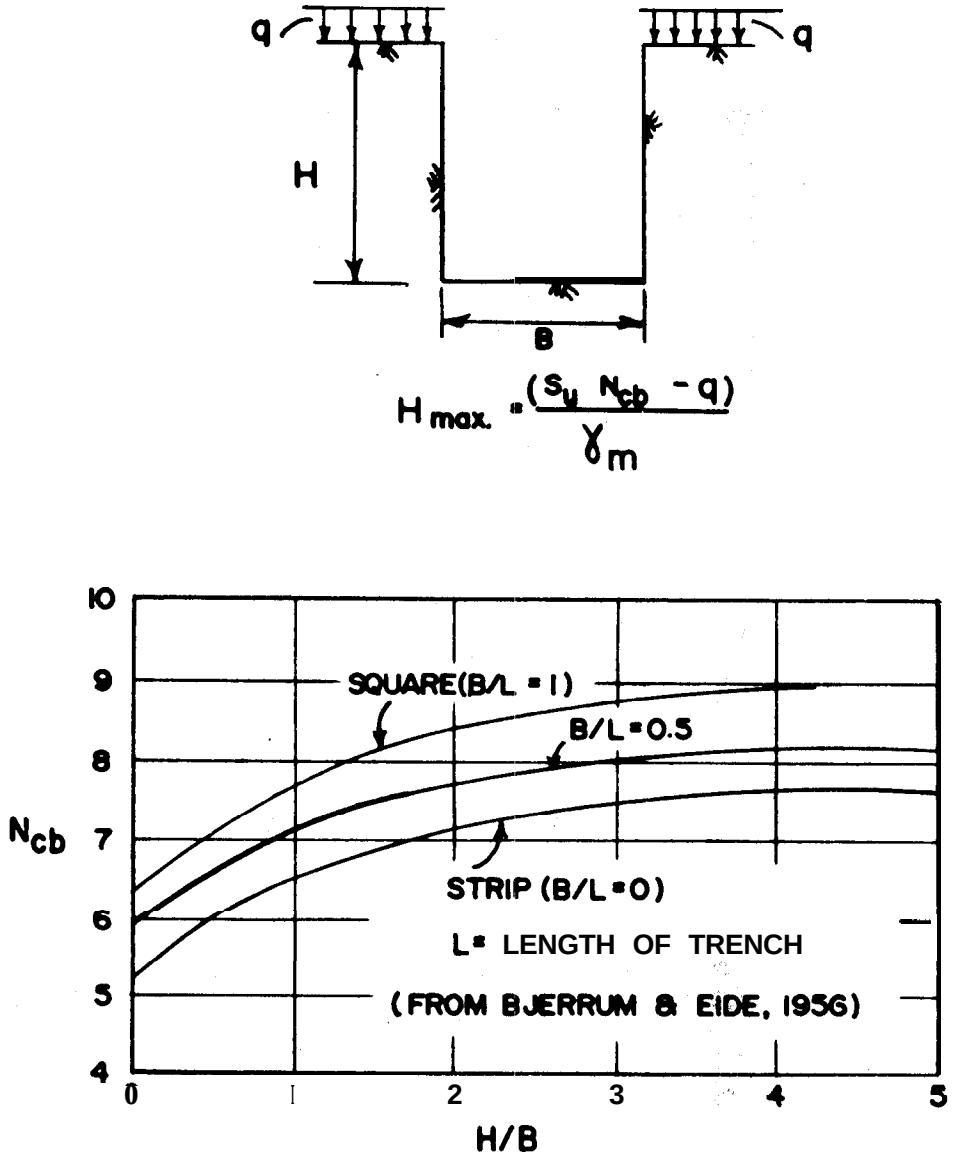


Figure 46. Bearing capacity factors for bottom stability analysis,

$$F.S = N_{cb}\left(\frac{S_u}{\gamma H + q}\right) = \frac{N_{cb}}{N}$$

where:

N = stability number = $\frac{\gamma H + q}{S_u}$

N_{cb} = bearing capacity factor from Figure 46

S_u = the undrained shear strength of the clay

γ = total unit weight of the soil

q = uniform surcharge loading on the area adjacent to the excavation

Where the soil is stratified within the depth of excavation and below, a weighted average of undrained strength should be used for S_u. This average should be taken over a zone described between $B/\sqrt{2}$ below the excavation base and $2.5B$ above the base.

8.30 LOCAL FAILURE

When braced excavations reach a certain depth in clay soils, the lateral pressures on the retaining wall coupled with the stress relief from the excavations can be of sufficient magnitude to cause local yielding of the soil immediately adjacent to the inside of the sheeting. This localized overstressing results in loss of passive resistance which in turn leads to uncontrolled inward movements of the sheeting. As the excavation proceeds, these inward movements become additive, resulting in large inward movements and a corresponding loss of ground adjacent to the excavation. D'Appolonia (1971), Jaworski (1973), and O'Rourke and Cording (1974a) all show data which indicate these uncontrolled movements can account for up to 50 percent of the loss of ground.

Figures 47a and 47b can be used to estimate when local failure is imminent in cohesive soils where flexible sheeting is used The failure is related to the shear strength (S_u) and to the initial state of stress in the ground. Figure 47a shows the factor of safety against bottom heave necessary to prevent local yield as a function of excavation geometry and the shear stress ratio. The shear stress ratio (f) is a convenient dimensionless parameter which defines the initial state of

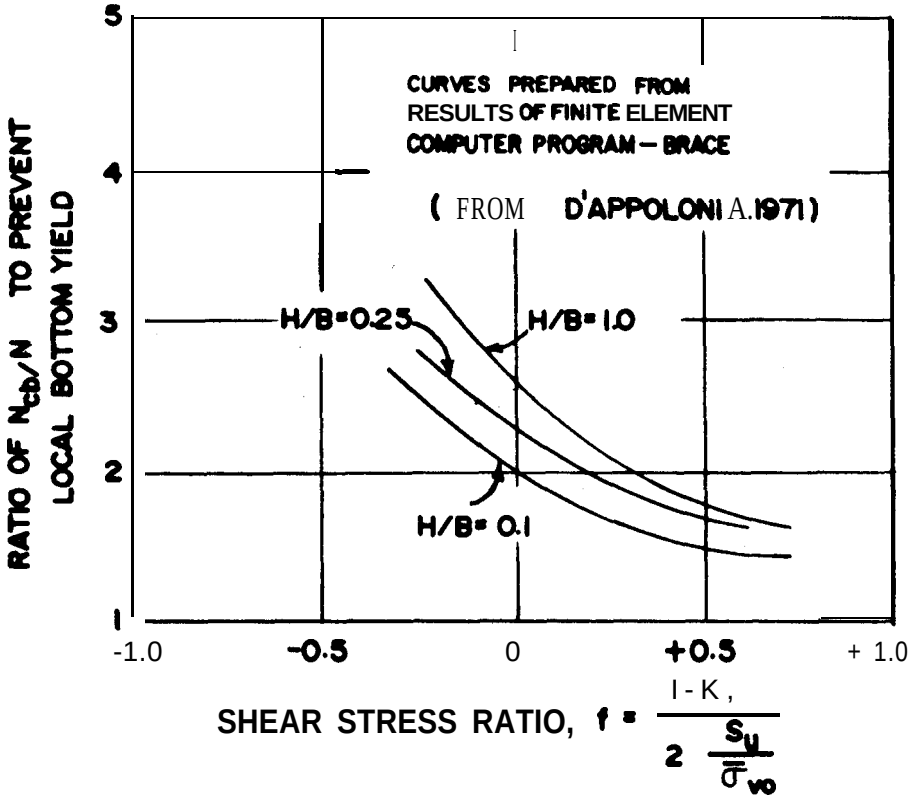


Figure 47a. Factor of safety required to prevent local yield below bottom of excavation in clay.

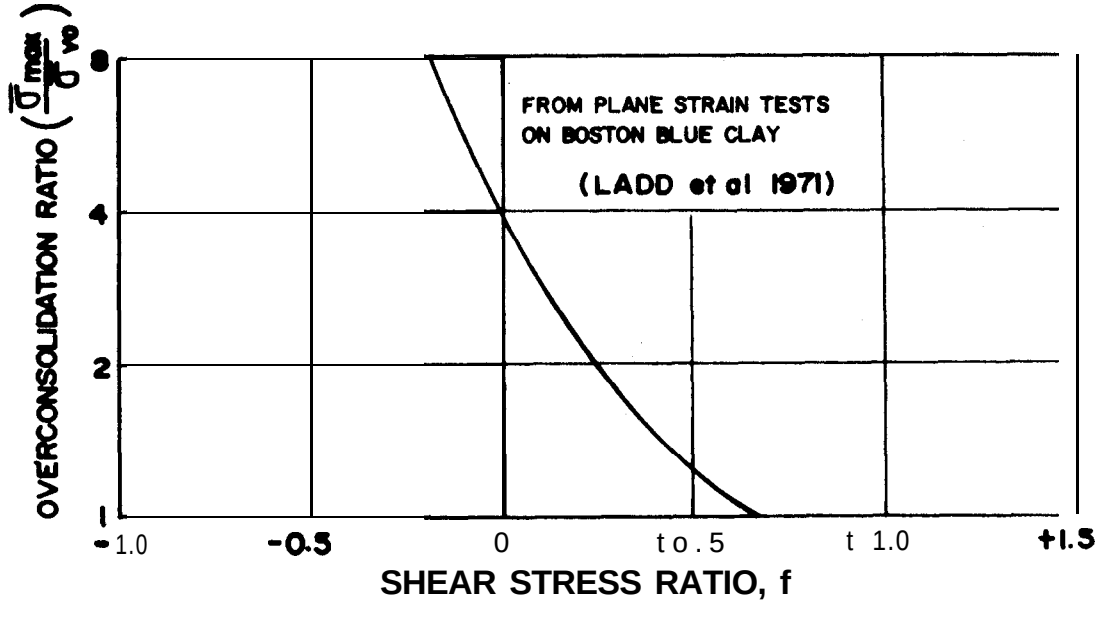


Figure 47b. Shear stress ratio vs. over consolidation ratio.

stress in the ground and the strength of the soil. Figure 47b gives the variation of the ratio (f) versus over consolidation ratio for Boston Blue Clay.

The depth at which local failures begin to develop is related to the shear strength of the soil and the initial state of stress in the ground. The potential for local yielding is most prominent in the over consolidated soils. The reason for this is the high value of K_o, ($\bar{\sigma}_{ho}/\bar{\sigma}_{vo}$), which is close to or can exceed 1. The failure at the base of the excavation is one of extension; that is, the shear stress (or deviator, stress) is increased by a decrease in the vertical stress. It follows that the higher the K_o value the closer the soil is to a failure condition. Hence, it takes less stress relief to cause overstressing of the soil.

Figure 47a shows that where the depth to width ratio of an excavation is 1 and the OCR is 6, a F.S. $\cong$ 3 may be necessary to prevent local yielding. On the other hand, for the same excavation in the softer normally consolidated soils (OCR = 1), a F. S. $\cong$ 1.5 is sufficient to limit local yielding. This lower factor of safety is associated with the low K_o value ($K_o \approx$ 0.5) in normally consolidated soils. Thus, they can experience much Larger stress release before failure.

Figure 48 shows when Local yielding starts in normally consolidated clays as a function of sheeting stiffness (K) and excavation geometry (H/B). These data were developed using a finite element program (see Appendix to Chapter 2). The results indicate that for a given excavation geometry (up to H/B $\simeq$ 1.0) stiffening the sheeting reduces the factor of safety required to prevent local failure. This trend is related to the ability of the stiffer sheeting to act as a cantilever wall while minimizing inward movement.

8.40 DEEP SEATED FAILURES

8. 41 Internally Braced Excavations

8.41.1 Circular Arc Analysis

In situations where internally braced excavations are either underlain by weak soils or the ground adjacent to the excavation slopes upward, the overall stability of the excavation should be analyzed.

One approach to analyze the stability is by the classical circular arc analysis as illustrated in Figure 49. It consists of assuming a series of centers of rotation and failure surfaces to find

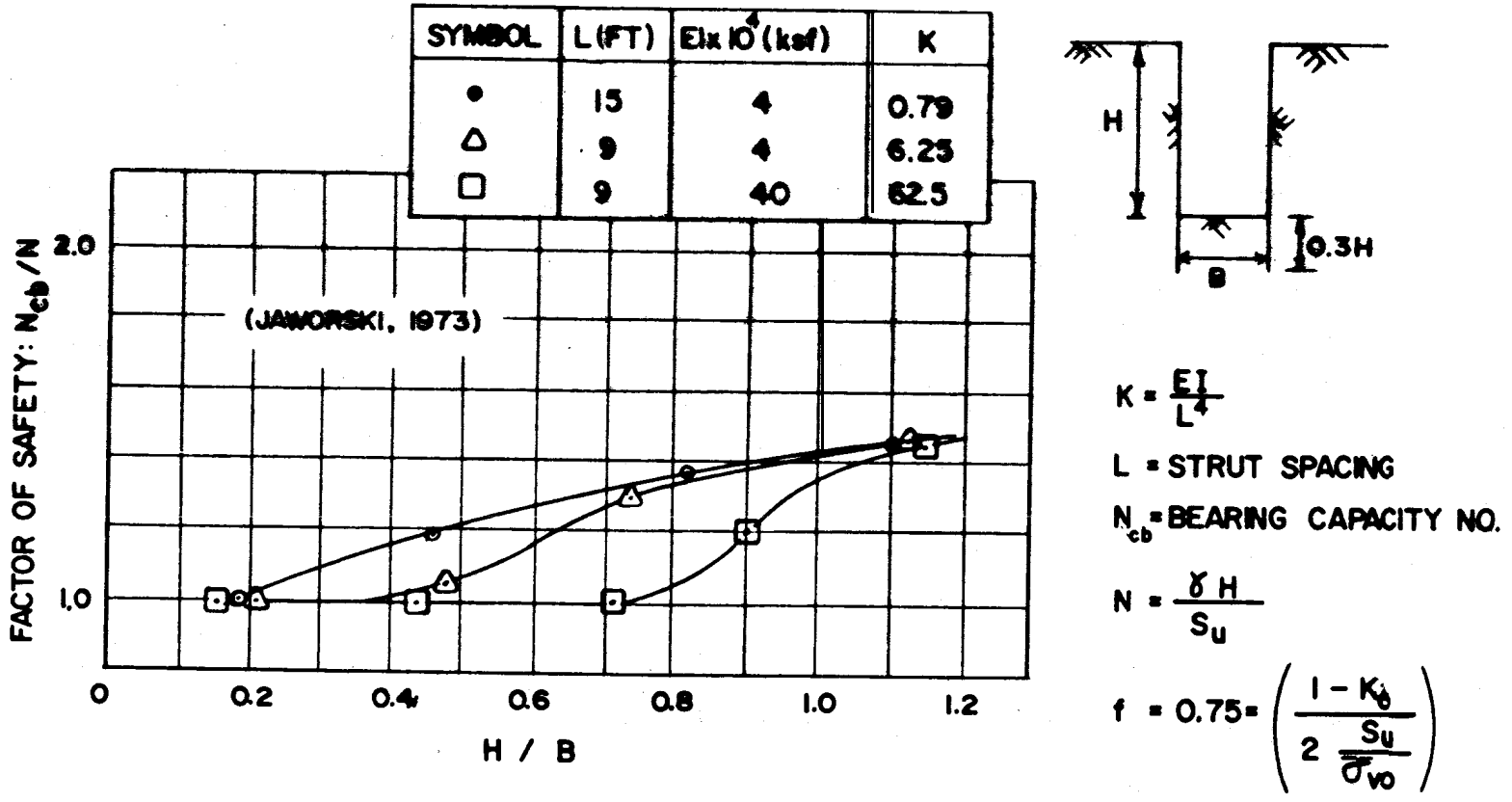
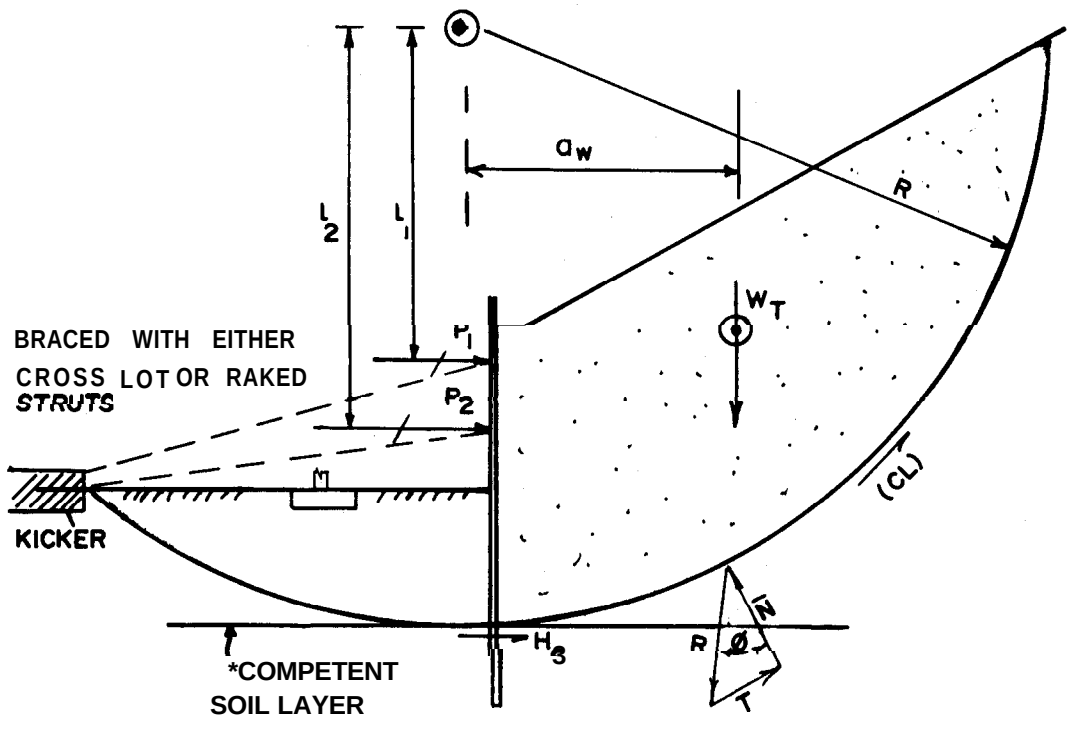


Figure 48. Effect of sheeting stiffness on factor of safety at which first yield occurs in normally consolidated clay.



Consider overall stability:

Moments around center of rotation

Forces to consider:

1) Weight of driving mass (WT)
2) Resisting strut loads (P_1, P_2) (Horizontal component of support load.)
3) Resisting shear capacity of wall (H_s) from competent soilkyer.
4) Shear strength of soil, frictional component (T). and Cohesion, (c)

Note: If rakers used, kicker must be located outside failure mass for P_1 and P_2 to be considered in analysis.

$$\text{Safety Factor} = \frac{\Sigma M_R}{\Sigma M_D} = \frac{(\bar{N} \tan \phi \, t \, CL)\, R}{W_T a_w + P_1 l_1 + P_2 l_2 + H_s R}$$

Figure 49. Stability of internally braced cut (circular arc method).

the minimum factor of safety against a rotational failure. The conditions shown are for the case of a homogeneous soil where the driving forces are the total weight of the soil mass plus any surcharge loads. Resisting forces consist of the strut loads (PL, P,), the shear strength along the failure arc, and the shear capacity of the sheeting below the failure arc. If the soil is stratified, then the stability analysis should be made using the classical "Method of Slices".

The analysis will determine the hypothetical failure surface bounding the failed soil mass. If rakers support the wall, care must be taken to insure their kicker support is outside the failure zone. Otherwise, the thrust forces from the rakers should not be considered in the analysis.

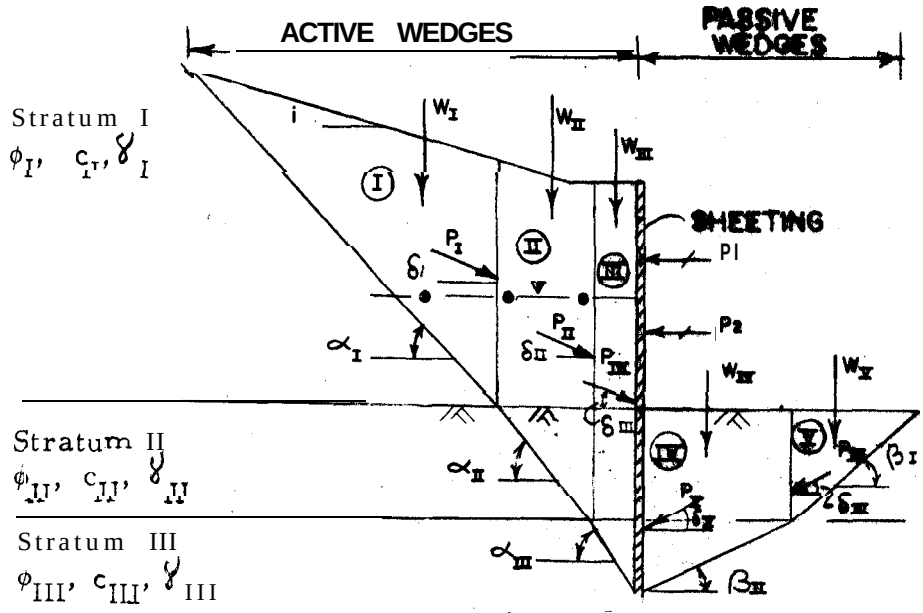
The sum of the strut forces necessary to maintain a stable excavation should be compared to those predicted from the Lateral earth pressure diagram as outlined in Section 7. 50. The greater of the two total Loads should be used to establish the ordinate of the design earth pressure diagram.

In the cases where the retaining wall extends through a weak Layer into a highly competent soil, the structural resistance of the retaining wall (H_s) should be considered in the analysis. The shear resistance should be taken equal to the passive force determined in accordance with Chapter 6.

8.41.2 Wedge Stability Analysis

An alternate means of evaluating the overall stability of an internally braced excavation is to make a wedge stability analysis. It is often a simple method for analyzing a stratified soil deposit for the maximum loads which might occur in a support system.

Figure 50 shows this method of analysis for an internally braced excavation. The analysis illustrated is general, with no assumption for either failure surface or direction of active and passive Loads. However, this Leads to a tedious analysis. A simple alternative for analyzing this condition is to assume Rankine conditions for failure surfaces and direction of Load. Although this Latter approach does not yield theoretically correct answers, the results will be adequate for most problems.

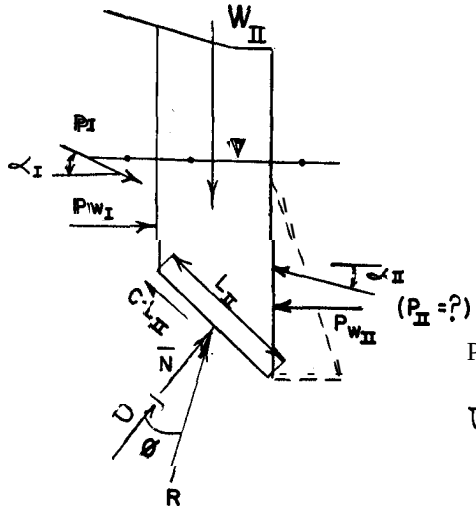


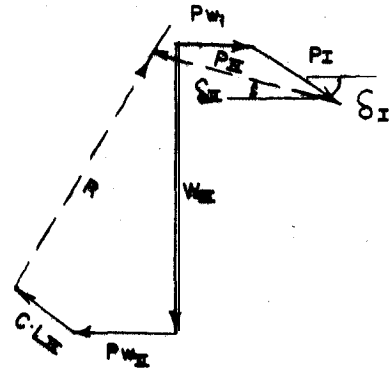
For general solution vary α, β, and&angles to obtain minimum value for factor of safety.

Method of Analysis:

1. Assume α, β, δ angles.
2. Sequentially analyze the active and passive segment for loads P_{III} and P_{V}, Include water pressure.
3. Sum forces in horizontal direction for factor of safety

i.e. F.S. $= \dfrac{P_1 + P_2}{P_{III} - P_v}$





P_{w_I}, $P_{w_{II}}$ = horizontal water pressure

U = uplift water force on Wedge II

Typical Analysis of Wedge (Wedge II)

Figure 50. Wedge stability analysis for braced cut.

8. 42 Tied-Back Walls

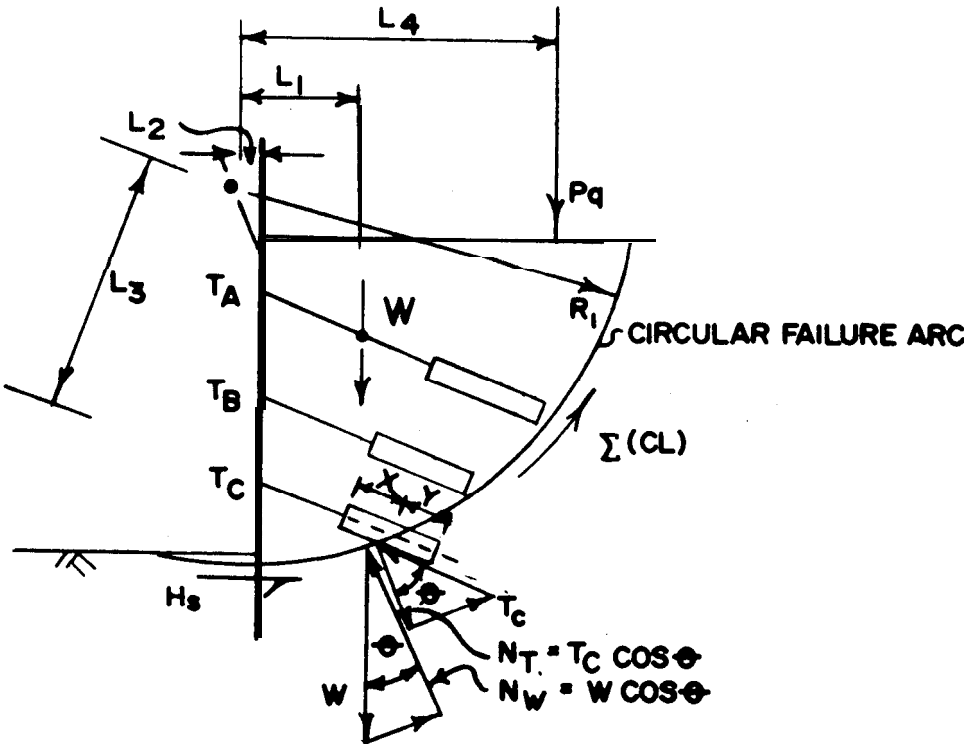
Detailed procedures for analyzing the stability of tied-back walls by a variety of methods employing trial planar surfaces and wedges are presented in Volume III. By and Large, these methods placed emphasis upon the failure surface passing through the zone of tiebacks. This technique may be used as a design tool for establishing the appropriate Length of tiebacks.

This section makes a simplified presentation of the circular arc method below as a means to examine overall stability for a failure surface passing beyond, or nominally through, the tieback zone. This concept is particularly appropriate when weak soils occur near or below the excavation base.

The analysis is quite similar to that used for internally braced excavations. Figure 51 illustrates the general approach for an assumed circular failure surface. The analysis must consider the position of the anchor relative to the failure surface. The example illustrated shows the surface cutting through the lowest anchor. The resisting force contributed by this anchor is a function of the amount of anchor outside the failed mass. If the failure surface passed before the anchor zone, then the tension force may be assumed to be the full design force in the tieback, T_c. For the surface shown, the failure plane passes through the anchor zone, therefore, it is necessary to make an assumption concerning tension force remaining in the tieback.' With ties having essentially uniform resistance in the anchor zone the load variation will be linear. Thus, the value of T to be used in the analysis may be taken as:

$$T = \frac{y}{x + y} \; T_c$$

where T_c is the total force in the anchor. In cases where ties are anchored in rock or belled anchors in highly competent soils are used, the full tension force may be assumed since the failure surfaces will not cut through these strata.



Take moments about center of rotation.

$$\Sigma M_0 = W l_1 + P_q l_4 - (T_C l_3 + H_s R_1)$$

$$\Sigma M_R = (W \cos \theta \tan \phi + cL) R1 + T_C \cos \theta \tan \phi$$

Safety Factor:

$$F. S. = \frac{\Sigma M_R}{CM_0}$$

Figure 51. Stability of tied-back excavation,

CHAPTER 9 - BEARING PRESSURE OF DEEP FOUNDATIONS

9.10 GENERAL

This section is directed toward those basic considerations used to establish bearing values for elements used in connection with cut and cover operations. These principles would be applied for underpinning units or for walls and soldier piles subjected to a vertical component of load.

Typically, the bearing stratum is deep -- that is, it lies at great depth relative to the width of the bearing area. Accordingly, those design rules developed for "shallow foundations" such as are presented in Terzaghi and Peck (1968) will be overly conservative.

Fundamentally, allowable bearing value must recognize two governing criteria -- first, adequate safety against shear failure of the foundation and second, a limitation of settlement. Usually, it is shear which controls for clays and settlement which controls for sands. The following discussion presents those basic tools required to assess the above stated criteria.

9.20 PRESUMPTIVE BEARING VALUE

Table 5 presents a summary of the range of allowable bearing values for building foundations resting on a variety of soil types. This tabulation is not intended to represent a recommendation for design; rather its purpose is to convey a means to assess the relative competency of different materials and to provide a crude initial guide. Because the values typically apply to shallow foundations, acceptable values for deep foundations will be somewhat higher.

9.30 BEARING VALUES BASED ON SHEAR FAILURE

9.31 General

The following represents a summary of theoretical procedures for calculating net ultimate bearing capacity using shear strength parameters, ϕ, for cohesionless soil., and undrained shear strength, S_u, of cohesive soils. A factor of safety of 2 to 3 should be applied depending upon risk and the confidence level in data.

Table 5. Abstract of presumptive bearing capacity, ksf.

	A Mass. State Code (1974)	B New York City (1968)	C Atlanta (1950)	D National Board of Fire Underwriters (1955)	E BOCA (1970)
Glacial Till*	20	--	--	--	--
Hardpan*	--	16 - 24	20	20	20
Gravel, well-graded sand and gravel*	10	8 - 20	8 - 12[1]	8 - 12[1]	8 - 12[1]
Coarse sand*	6	6 - 12[2]	--	6 - 8[1]	6 - 8[1]
Medium sand*	4	--	--	--	4(loose)
Fine sand	2 - 4	4 - 8[3]	--	4 - 6[1]	--
Hard clay	10	10	--	--	--
Stiff clay	--	--	4	5	--
Medium clay	2	4	--	5	--

*Massachusetts and New York Code allow 5 percent increase in bearing value per foot of additional embedment, but not more than twice tabulated value.

1 - Range reflects compactness, gradation, and/or silt content

2 - 0. 1 x N, but not less than 6 ksf nor more than 12 ksf (where N= no. of blows in SPT)

3 - 0. 1 x N, but not less than 4 ksf nor more than 8 ksf (where N= no. of blows in SPT)

9. 32 Sand

For deep piers, in sand, the end bearing load capacity is generally expressed as:

$$q_u = N_q \bar{\sigma}_v$$

where:

N_4 = dimensionless bearing capacity factor that is a function of the shear strength parameter, ϕ, of the bearing meterial and shape of the loaded area

$\bar{\sigma}_v$ = effective stress in the soil at the bearing surface

q_u = ultimate bearing capacity (load per unit area)

Values of N_q vary depending upon assumptions made in the derivation. Vesic (1965) presents ranges for the values as shown in Figure 52. The lower curves represent modes of failure in which the shear strength of the soil is developed below the footing. Higher values of N_q will be obtained by assuming that the failure surface extends above the plane of bearing, thus engaging shear resistance above that level,

As a practical matter, because of the high bearing capacity of sand there is little penalty in adopting a conservative value. For example, consider an extended underpinning pier bearing at a depth of 20 feet on a sand with $\phi = 35^{\circ}$. Assuming a unit weight of 125 psf, the effective stress would be:

$\bar{\sigma}_v$ = 20 x 125 = 2500 psf

N_q = 58 (Veeic)

= 75 (Berezantsev)

= 110 (Brinch Hansen, 1961)

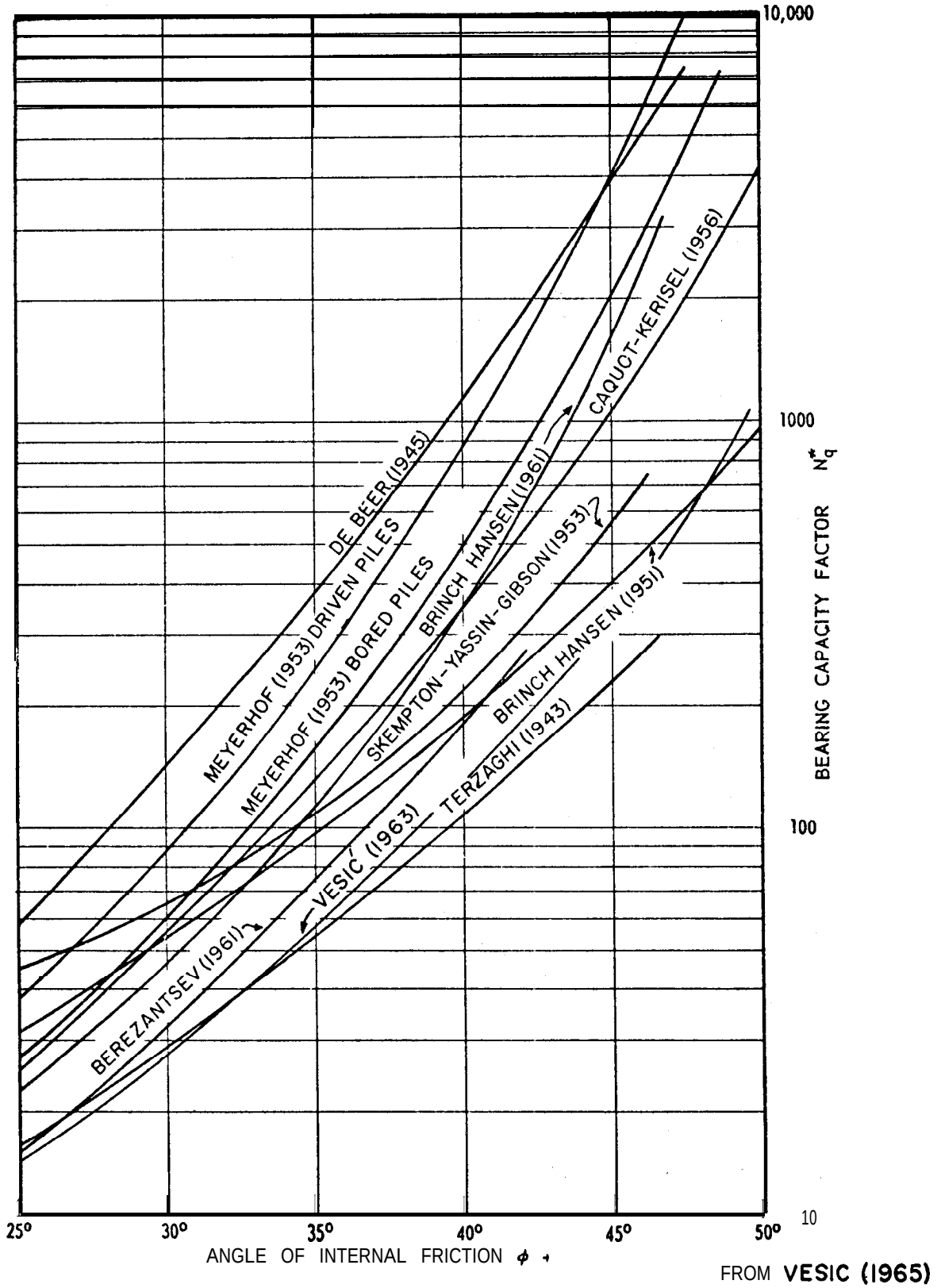


Figure 52. Bearing capacity factors for deep circular foundations.

Thus, the ultimate bearing pressure will range from:

Vesic: $q_u = 58 \times 2500 = 145{,}000$ psf

Berezantsev: $q_u = 75 \times 2500 = 187{,}500$ psf

Brinch Hansen: $q_u = 110 \times 2500 = 275{,}000$ psf

Obviously, all values are quite acceptable. Accordingly, there is Little to be gained in most applications by debating the appropriate value of Nq. In general, a safety factor of 3 is applied to these ultimate values. As stated above the settlement limitation usually controls in granular soils.

9.33 Clay

In clays the undrained strength, S_u, rather than drained strength will control the bearing capacity of a foundation element. Skempton (1951) presented bearing capacity factors N_c for net ultimate bearing capacity in clays. In this case, "net" means pressure in excess of the effective overburden stress of the bearing level.

$$q_u = N_c S_u$$

where:

q_u = net ultimate bearing capacity (load per unit area)

N_c = dimensionless bearing capacity factor that is a function of the shape of the loaded area

S_u = undrained shear strength of soil

For deep foundations (at depth greater than 4 to 5 times the breadth of the Loaded area) values of N_c are as follows:

Circle: $N_c = 9$

Strip: $N_c = 7.5$

Rectangle: $N_c = 7.5$ (1 t 0.2 B/L)

where: B = breadth and
L = length

Note that for clays, the net ultimate bearing pressure is independent of depth (and therefore overburden stress). It is a function only of the shape of the loaded area and undrained shear strength of the soil.

In addition to the load bearing capacity at the base, the side friction may be determined on the basis of the embedded area and

adhesion along the shaft. In soft clays, the adhesion is equal to or only slightly less than the undrained shear strength. However, in stiff to hard clays the adhesion is typically less than one-half the undrained strength. Tomlinson (1969) presents a summary of data for adhesion in both driven and bored piles.

Much of the data on shaft adhesion was developed on bored piles in London clay. The practice is to apply a reduction factor, α, to the undrained strength to estimate adhesion. Thus:

$$S_{eff} = \alpha S_u$$

where:

α = reduction factor

S_u = undrained shear strength, psf

S_{eff} = adhesion along shaft, psf

Figure 53 (after Peck, et al, 1974) shows that α decreases as the shear strength of clay increases. In general, shaft adhesion is counted on for load support in very stiff to hard clay, In this range, α varies from about 0. 3 to 0.5. For stiffer clays, the average developed adhesion, αS_u, shows Little variation with increasing shear strength. It varies from only about 1 tsf to 1. 3 tsf.

In areas where there is Little prior data, Skempton recommends a maximum adhesion of 1 tsf (Tomlinson, 1969) when using the chart. The total capacity of the shaft is equal to:

$$Q_{shaft} = \alpha S_u A$$

where:

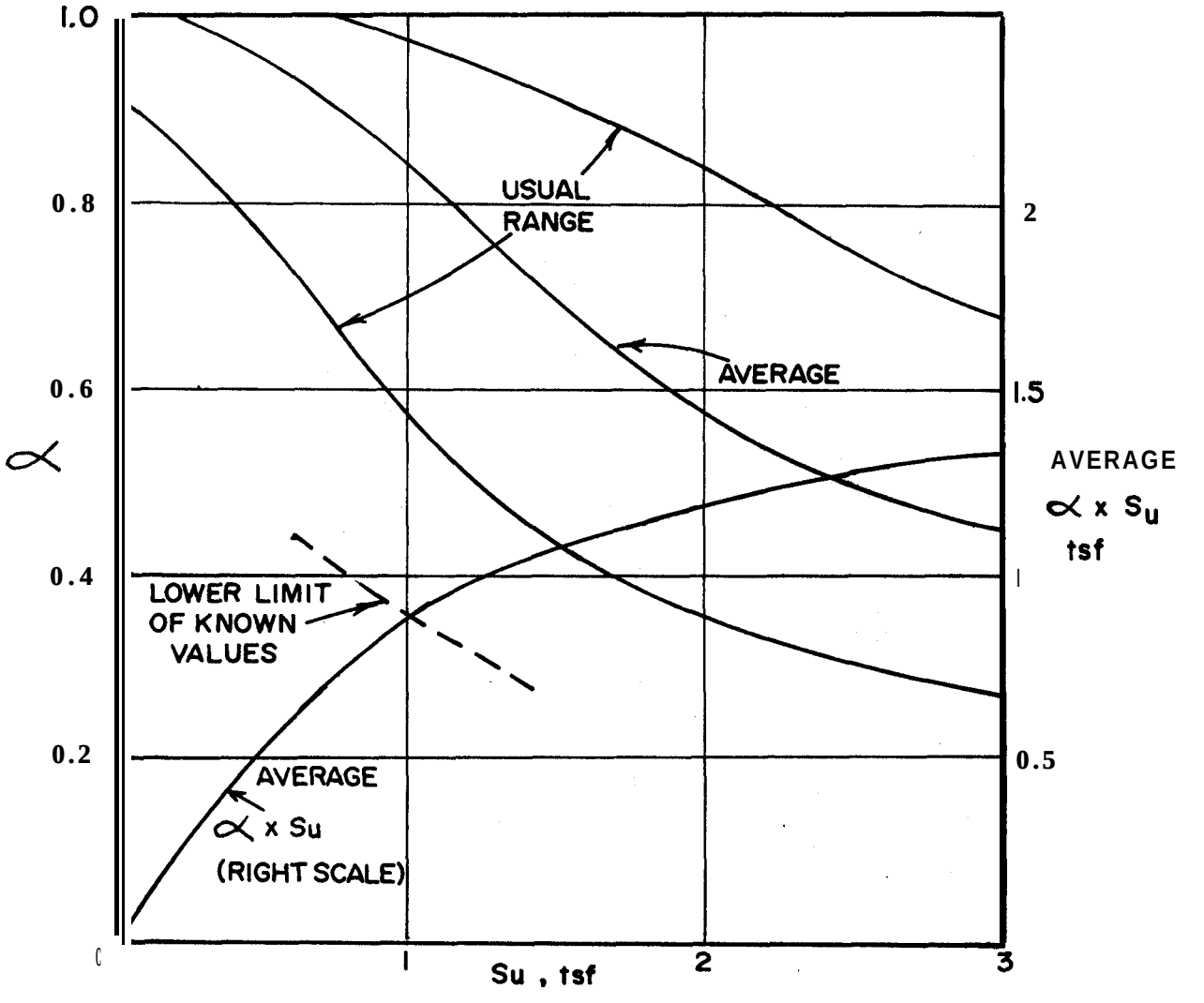
A = shaft area

Again, a safety factor of between 2 and 3 should be used.

9.40 BEARING VALUES BASED ON SETTLEMENT

9.41 General

The following presents the recommended procedures for estimating the settlements of deep foundations.



FROM PECK, HANSON & THORNBURN (1974)

Figure 53. Reduction factor in S_u from observed capacity of friction piles.

9.42 Soils Having Constant Modulus of Deformation with Depth

9.42. 1 Surface Loading

Theoretical procedures for determination of settlements have been developed by Fox (1948) for square and rectangular bearing areas and by Woodward, et al (1972) for round bearing areas. These are based on integration of the Mindlin solution for a point load within an elastic half space. At a depth equal to zero, the Mindlin solution is identical to the familiar Boussinesq solution. These solutions all have the general form.

$$\rho = q \frac{BI_\rho}{E} (1 - \nu^2) \qquad \text{Eq. 9.42. 1}$$

where:

ρ = settlement

q = distributed load

B = least dimension of foundation unit

E = modulus of deformation

ν = Poisson's Ratio

I_ρ = influence factor which depends on rigidity of footing, shape of footing, and depth of footing

A simplified method for determining settlement at the surface is based upon a coefficient of **subgrade** reaction defined as follows:

$$k = \frac{q}{\rho} \qquad \text{Eq. 9.42. 2}$$

And thus, the settlement is computed as follows:

$$\rho = \frac{q}{k} \qquad \text{Eq. 9. 42. 3}$$

where:

ρ and q are defined as above

k = coefficient of **subgrade** reaction in general units of pressure per unit deflection

The value of the coefficient of subgrade reaction is commonly determined by plate loading tests or by correlation with in situ soil indices such as relative density and standard penetration resistance. By comparison of Eq. 9.42. 1 and 9.42.3, the coefficient of subgrade reaction is related to the theoretical settlement as follows:

$$k = \frac{E}{B(1 - \nu^2) I_\rho} \qquad \text{Eq. 9.42.4}$$

For a constant footing shape and depth and constant material properties, the coefficient of subgrade reaction for a footing of size B is therefore related to a footing of size B' by the following relationship:

$$k_{B'} = k_g \left(\frac{B}{B'}\right) \qquad \text{Eq. 9.42.5}$$

It is common to express the coefficient of subgrade reaction in terms of the value for a 1 foot square plate (k_1) as this is the size for conventional plate loading tests. Therefore,

$$k_B = \frac{k_1}{B}$$

Typical values for kl are shown in Figure 54.

9.42.2 Rectangular Footings

Influence values for other than square footings can be determined from elastic theory. These values however, become very large for long narrow footings and in fact approach infinity (k approaches zero) for an infinitely long footing. These results directly follow from the fact that the Boussinesq solution does not approach the actual plane strain conditions when integrated to infinite limits. Therefore, the elastic solution is unrealistic for long footings. To solve the problem, Terzaghi (1955) has proposed the following empirical relationship for rectangular footings:

$$k_{L \times B} = k_B \frac{(1 + 0.5\, B/L)}{1.5} \qquad \text{Eq. 9.42.6}$$

where:

$k_{L \times B}$ = coefficient of subgrade reaction for footings of Length, L, and width, B

k_B = coefficient of subgrade reaction for square footing of dimension, B

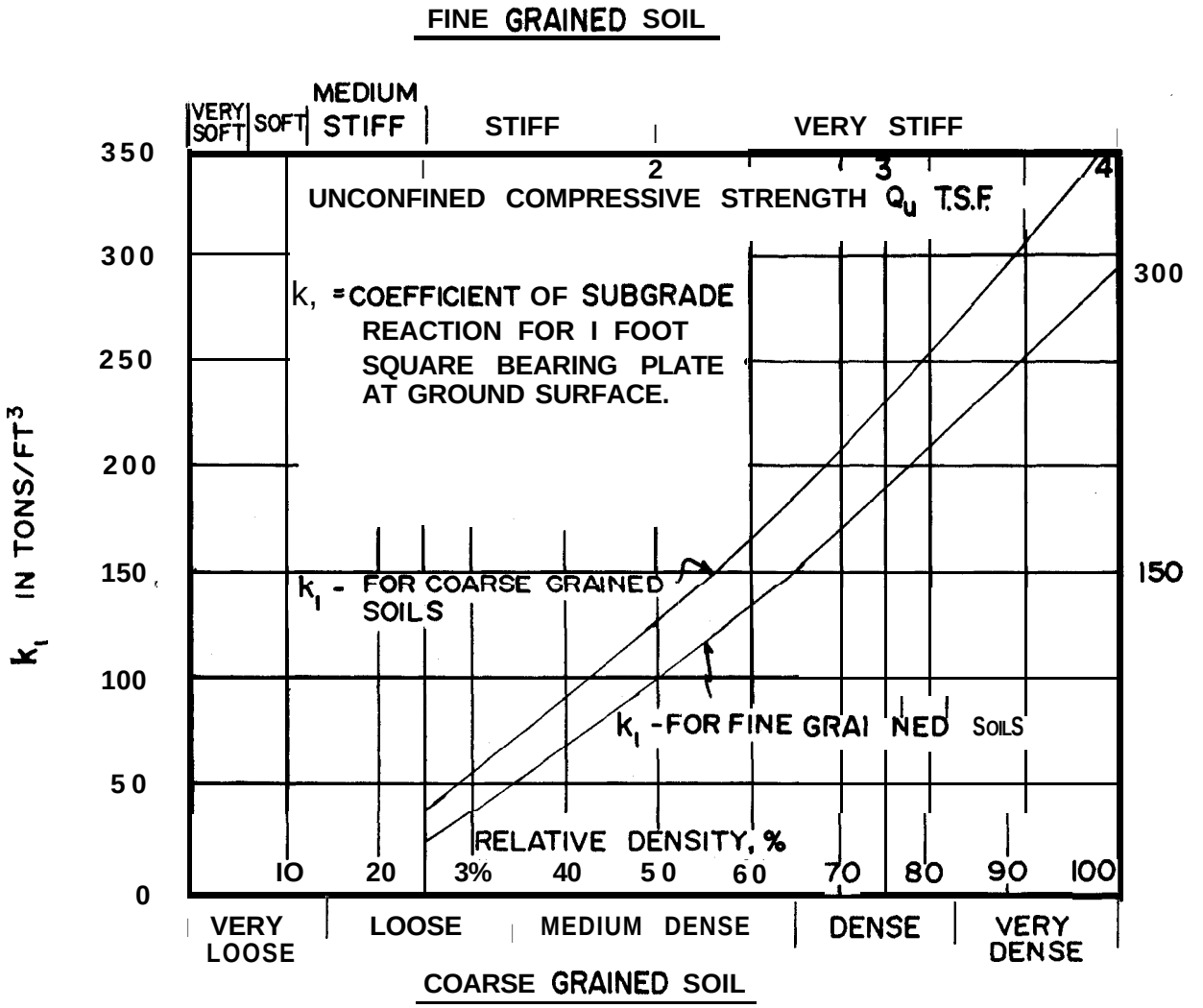


Figure 54. Coefficient of subgrade reaction vs. in situ soil indices (NAVFAC, 1971).

This relation suggests that the subgrade modulus for an infinitely Long footing approaches a value equal to 2/3 of that for a square footing.

Both the elastic theory and Terzaghi's empirical relationship are plotted in Figure 55. The recommended procedure is the Terzaghi relationship especially for Larger values of L/B.

9.42.3 Effect of Depth

For a footing with constant Loading, shape, and mater ial pr opertie s , the subgrade modulus of that footing is inversely proportional to the influence factor (see Eq. 9.42.4). Thus, when the influence factor varies with depth, the ratio of subgrade modulus at the surface to the subgrade modulus at depth may be computed as follows:

$$\frac{k_B^S}{k_B^D} = \frac{I_\rho^D}{I_\rho^S} \qquad \text{Eq. 9.42.7}$$

where:

k_B^S = coefficient of subgrade reaction for a footing (breadth B) at the surface.

k_B^D = coefficient of subgrade reaction for a footing (breadth B) at depth, D.

I_ρ^D = influence factor for footing at depth D.

I_ρ^S = influence factor for footing at surface.

Elastic theory demonstrates a variation of influence factor, I_ρ, with depth. On this basis, the depth factor FD has been plotted in Figure 56 for circular and rectangular footings. Note that this figure is for the special case of constant modulus of deformation.

Also shown on this figure is a plot of depth factor for a circular shaft which relies on 100 percent side friction and no end bearing. It can be seen that the depth factor is Less for this case than for the case of 100 percent end bearing for depth ratios. $\left(\frac{D}{\sqrt{L \times B}}\right)$ greater than approximately 1.25. For all cases, the effect of depth is to increase the subgrade modulus and thus to reduce settlement.

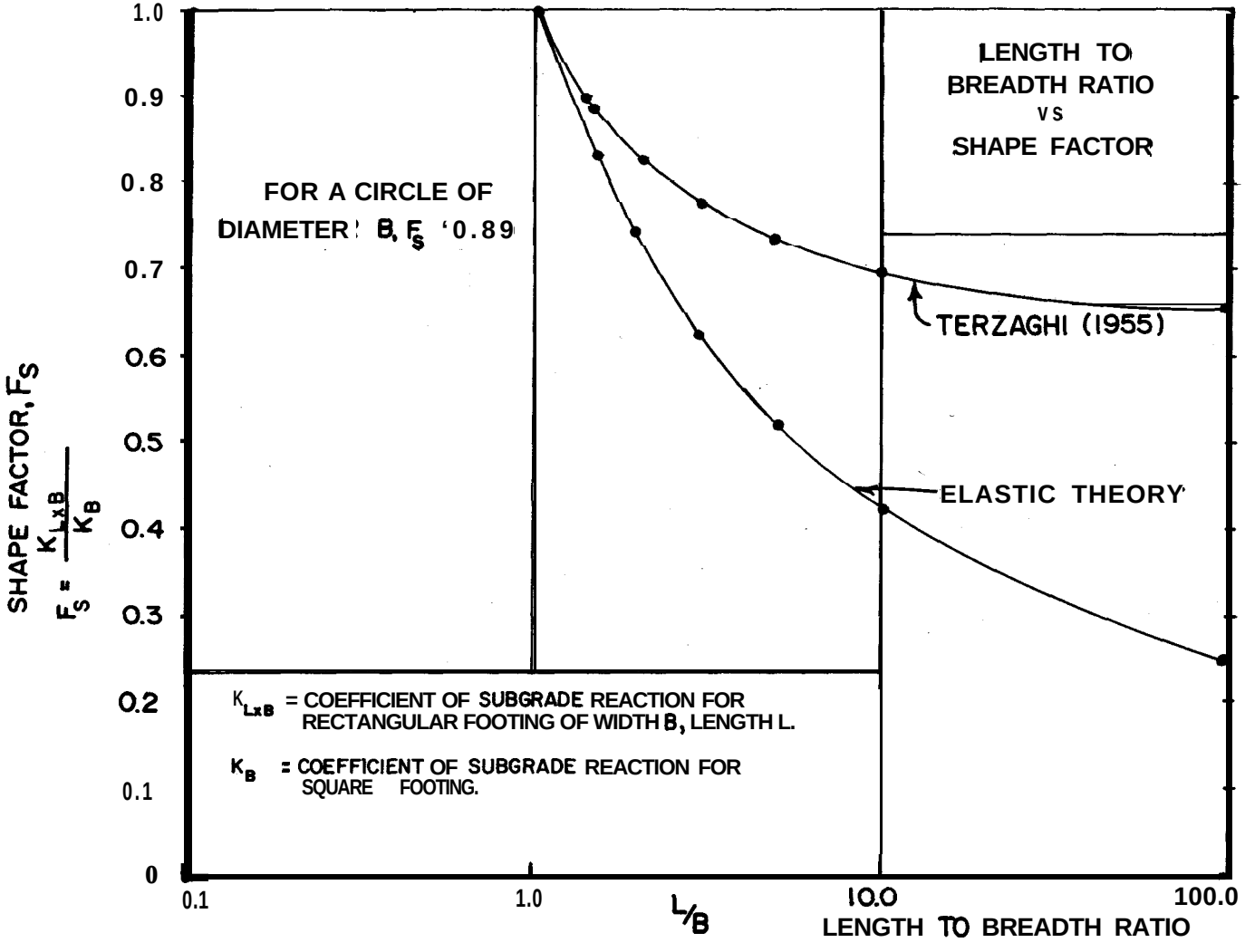


Figure 55. Shape factor for rectangular footings.

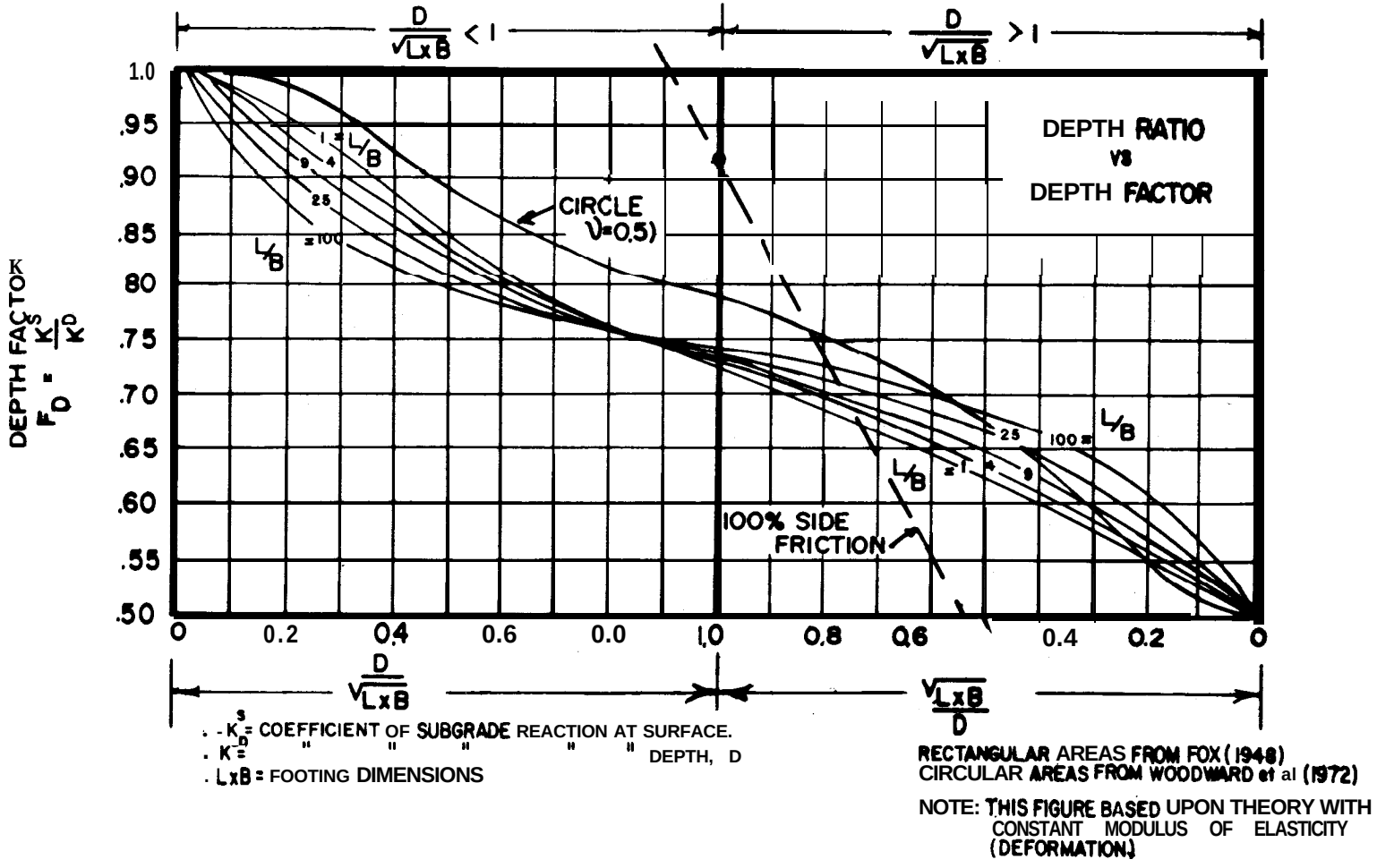


Figure 56. Influence of depth on coefficient of subgrade reaction (based on modulus of deformation that is constant with depth).

9.43 Soils Having Modulus of Deformation that Increases with Depth

9.43u1 r f a c e

In the case where the modulus of deformation increases linearly with depth, it can be seen that the coefficient of subgrade reaction will vary in direct proportion to the increase in modulus. This effect comes into play in two ways: first, for larger size footings a larger area is loaded and consequently a greater depth of stress influence is created; and second, for footings at depth, the deformation modulus E, will not be the same as at the surface, thereby invalidating the relationships in the preceding section.

For the former case, Terzaghi (1955) has proposed the following empirical relationship to convert the coefficient of subgrade reaction for a 1 foot square area to an area B x B square.

$$k_B = k_1\left[\frac{B+1}{2B}\right]^2 \qquad \text{Eq. 9.43. 1}$$

9.43.2 Rectangular Footings

Once k_B is determined, $k_{L \times B}$ at the surface can be obtained from Figure 55.

9.43.3 Depth Effects

The special case of constant modulus of deformation was discussed in Section 9.42. 3. In addition, Taylor (1948) has proposed an embedment correction to account for the increase in modulus of deformation with depth as follows:

$$k_B^D = k_B^S (1 + 2\,D/B) \qquad \text{Eq. 9.43.2}$$

Where: D = depth of footing
B = least breadth of footing

In using this relationship, care must be taken to assure that the value of k_1 used to determine k_B does represent the material at the surface. If the value of k_1 is determined from correlation with indices such as standard penetration resistance or relative density at the bearing level., the correction for increase in modulus would not be made.

A second method of evaluating the effect of increasing modulus of deformation is to consider the coefficient of subgrade reaction to be directly proportional to the initial tangent modulus E_{it}. Janbu (1963) shows that Eit for granular soils is proportional to a power function of stress Level. Specifically:

$$E_{it} \sim (\bar{\sigma}_3)^n \qquad \text{Eq. 9.43. 3}$$

where:

$\bar{\sigma}_3$ = lateral stress (assumed to be effective Lateral stress)

n = 0.3 for gravels, 0.5 for sands

Accordingly, it follows that:

$$\frac{k_B^S}{k_B^D} = \left[\frac{\bar{\sigma}_3^S}{\bar{\sigma}_3^D}\right]^n = F_{DG} \qquad \text{Eq. 9.43.4}$$

where: F_{DC} is defined as the depth factor for granular soil.

In normally consolidated deposits, $\bar{\sigma}_3$ is proportional to the overburden stress and therefore to the depth. The following equation results:

$$\frac{k_B^S}{k_B^D} \left[\frac{z^S}{z^D}\right]^n = F_{DG} \qquad \text{Eq. 9.43.5}$$

The recommended value of 'z' to be used in this expression is:

$$z = D_F \text{ t } 0.75B \qquad \text{Eq. 9.43.6}$$

where :

D_F = depth of footing from average ground surface

The results of both methods for determining depth effects in soils with varying modulus of deformation are presented in Figure 57. Note that a Limitation of F_{DG} = 0.5 has been set on the Taylor expression. The relationships shown in the figure are typically

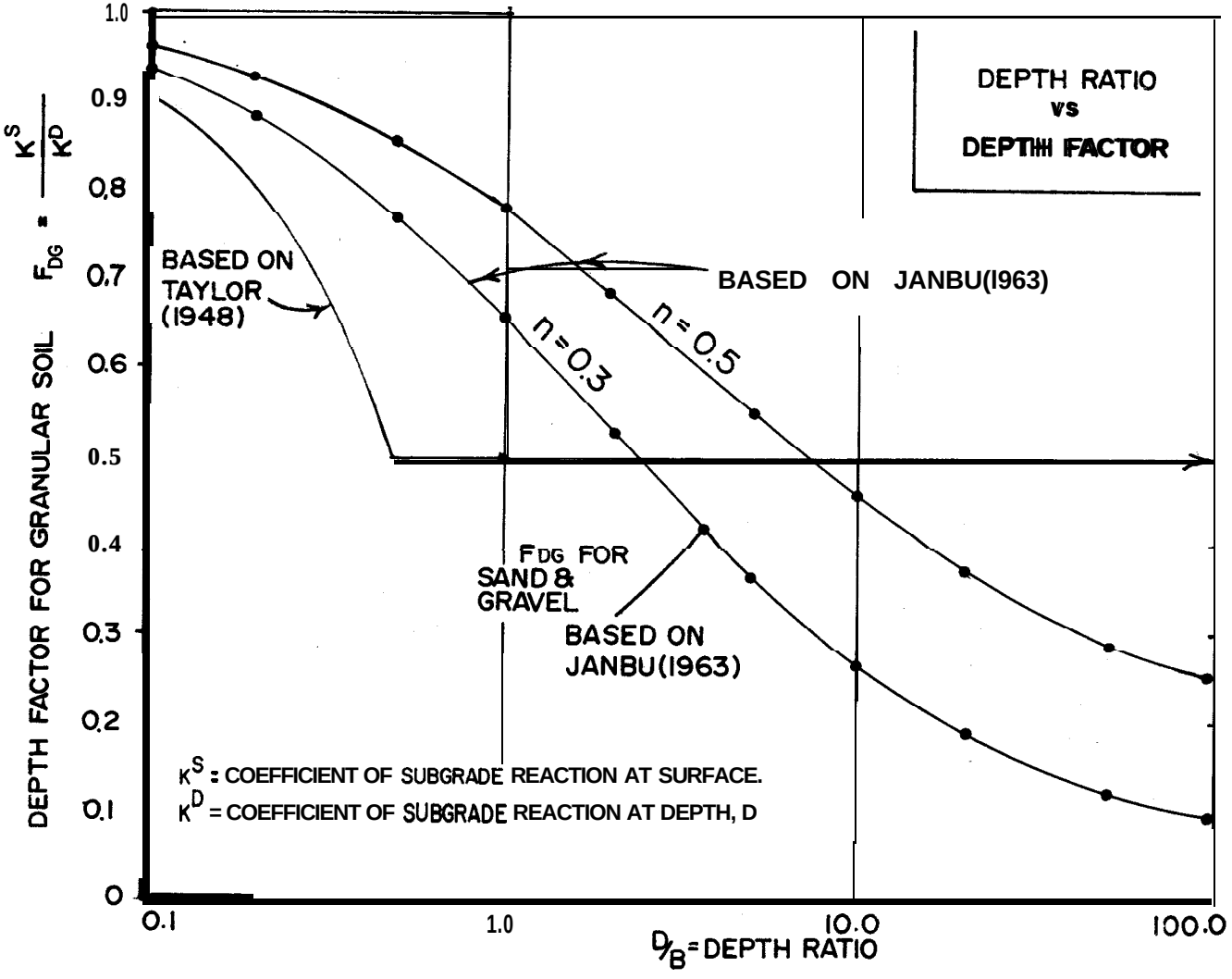


Figure 57. Influence of depth on coefficient of subgrade reaction for granular soils (based on modulus of deformation that increases with depth).

applicable to granular soils because their modulus of deformation is a function of stress level, and therefore, depth. Such soils rarely exhibit "over consolidated" behavior: however, they would only be applicable to cases where the determination of the basic value of subgrade modulus did not already include the effects of increasing modulus with depth. For instance, if k were based on the average standard penetration resistance in the zone of significant stress increase beneath the proposed footing,no correction for increasing modulus with depth would be made ($F_{DG} = 1$). On the other hand, if k were based on a plate loading test at the surface, the value of F_{DG} as determined from Figure 57 should be used.

9. 43. 4 Water Table Effects

The presence of ground water in granular soils will effect the modulus of deformation by reducing the Lateral effective stress. The effect of the ground water table can be estimated by considering equation 9.43.4 and computing the lateral effective stress at midpoint of the zone of significant stress increase. If the water table is below a depth of 1.5B beneath the base of the footing, then no water table correction would be necessary and Figure 57 could be used directly. If the water table is at or above the base of the footing, then F_{DG} would be computed using effective stress values at the average depth of significant stress increase substituted into Eq. 9.43.4. Where the water table lies between these limits, a pro-rated correction should be made.

The effects of the water table would only be considered where the water table effects have not been considered in determining the basic value of the subgrade modulus. For instance, if k is based on a plate loading test at the surface, with the water table also at the surface, Figure 57 could be used directly since water table effects would be accounted for in the value of k; however, if the footing and water table were at some depth D greater than the influence area of the loading test, F_{DC} would be computed as described above. Conversely, if the value of k were based on the average standard penetration resistance in the zone of significant stress increase, no adjustment to k would be necessary, either for water table effects or the increase in modulus with depth.

9.44 Recommended Procedure for Determination of Settlements of Deep Foundations

9.44.1 Clays

Assume that modulus of deformation is constant with depth. Compute settlement for Eq. 9.42.2.

$$\rho = q/k$$

where:

q = load in tsf

k = coefficient of subgrade reaction in tsf/ft

ρ = settlement in feet

Determine k by first determining kl at the surface of the soil from Figure 54 or from plate load test.

Modify k, as follows, to account for size, shape and depth:

$$k = \frac{k_1}{B}\left[\frac{F_S}{F_D}\right]$$

where:

F_S = shape factor from Figure 55

F_D = depth factor from Figure 56

B = least dimension of bearing area in feet

9.44.2 Sands

Assume modulus of deformation increases with depth. Compute settlement from Eq. 9.42.2 as above. Determine k by first determining kl as above. Modify k_1 as follows to account for size, shape and depth:

$$k = \frac{kl \times FS}{F_D \times F_{DG}}\left[\frac{B+1}{2B}\right]^2$$

where:

F_S, F_D, and B as defined above

F_{DG} = depth factor for granular soil from Figure 57

9.45 Example Problems

9.45.1 Problem Number 1

Determine the average settlement of a continuous footing for underpinning; 4 feet wide at a depth of 30 feet in sand. The water table is at a depth of 50 feet and the average load per linear foot of footing is 20 kips. The subgrade modulus of 200 tons/ft^3 was determined from a plate loading test at the surface prior to excavation. The unit weight is 125pcf.

a) Make correction for footing size and shape:

(1) Find k_4 (coefficient for 4' x 4' footing)

$$k_B = k_1 \left[\frac{B + 1}{2B}\right]^2$$

$$k_4 = 200 \left[\frac{-4 + 1}{8}\right]^2 = 78 \text{ tons/ft}^3$$

(2) Find shape factor for an infinitely long footing 4 feet wide. See Figure 55.

F_S = 2/3 (Tereaghi curve)

b) Make correction for footing depth.

(1) Find depth factor, based upon elastic theory and constant modulus. See Figure 56.

F_o = 1.0 for L = ∞ (by depth)

(2) Find depth factor due to increasing modulus with depth. See Figure 57.

$D/B = 30/4 = 7.5$

from Figure 57

$F_{DG} = 0.5$ (both Taylor and Janbu criteria)

c) Find corrected coefficient of subgrade reaction.

$$k = 78 \times \frac{F_S}{F_D \times F_{DG}}$$

$$= 78 \times \frac{2}{3} \times \frac{1}{0.5 \times 1.0}$$

$$= 104 \text{ tons/ft}^3$$

d) Settlement Computation

$\rho = q/k$

$q = \frac{20 \text{k/ft}}{4 \text{ft}} = 5$ ksf

$k = 104 \text{ tons/ft}^3 = 208 \text{ k/ft}^3$

$\rho = \frac{5}{208} = 0.024' = 0.288''$; say $\sim$ 0. 3''

9.45.2 Problem Number 2

Determine the settlement of a 3 foot square footing for an underpinning unit at a depth of 5' in stiff clay. The unconfined compressive strength of the clay was 1. 5 tsf,and the footing load is 45 k.

a) Determination of k_1

From Figure 54 for fine grained soils:

$k_1 = 60 \text{ tons/ft}^3$

b) Make correction for size

$k_3 = k_1 (1/B) = 60/3 = 20 \text{ tons/ft}^3$

c) Make correction for depth

1) Find depth factor based upon elastic theory and constant modulus. See Figure 56.

$$= \frac{5'}{3'\ B'} = 1.67$$

$$F_D = 0.64$$

d) Find corrected coefficient of subgrade reaction

$$k = 20 \times \frac{1}{F_D} = \frac{20}{0.64} \qquad 31 \text{ tons/ft.}^3$$

e) Settlement Computation

$$\rho = q/k$$

$$q = \frac{45\,k}{9\ ft.^2} = 5 \text{ ksf}$$

$$k = 31 \text{ tons/ft.}^3 = 62 k/ft.^3$$

$$\rho = \frac{5}{62} = .0.08' = 1.0"$$

9.45.3 Problem Number 3

Same as problem number 1 except that the water table is at 5 feet. The only difference will be the determination of F_{DG}.

a) Make corrections for footing size and shape

$k_4 = 78$ tons/ft.^{3} and $F_S = 2/3$ (from example 1)

b) Find depth factor (F_{DG}) due to increasing modulus with depth. In this case F_{DG} must be determined by Eq. 9.43.4, since the original estimate of subgrade modulus did not include the effects of the water table. Let K_o = at rest earth pressure coefficient.

Under a 4 foot wide load at the surface

$$\bar{\sigma}_3 = 125\ (0.75 \times 4)\ K_o = 375 K_o$$

Under a 4 foot wide load at 30 feet with the water table at 10 feet.

$$\sigma_3 = \left[(125 \times 5) \text{ t } (25 \times 62.6)\right] K_o = 2190 K_o$$

$$\frac{\bar{\sigma}_3^S}{\bar{\sigma}_3^D} = \frac{375}{2190} = 0.17$$

$$F_{DG} = \left[\frac{\bar{\sigma}_3^S}{\bar{\sigma}_3^D}\right]^n = (0.17)^{0.3} = 0.59$$

c) Find depth factor based on elastic theory and constant mo dulu s .

F_D = 1.0 from Example 1

d) Find corrected coefficient of subgrade reaction

$$k = 78 \frac{F_s}{F_{DG} \times F_D} = 78 \times \$ \times \frac{1}{0.57 \times 1.0}$$

91 tons/ft.^{3} = 182 k/ft.^{3}

e) Settlement computation

$$\rho = q/k = \frac{5}{182} = 0.027' = 0.324"; \text{ say } \sim 0.3"$$

9.45.4 Problem Number 4

Determine the settlement of a 5 x 5 footing at a depth of 10 feet in sand and gravel. The load is 80 tons and the coefficient of subgrade reaction is estimated to be LOO tons/ft^3 for a 1 square foot footing on the basis of the standard penetration resistance between 10 feet and 17. 5 feet. The water table is at 10 feet.

a) Correct for size effects

$$k_5 = k_1 \left(\frac{B+1}{2B}\right)^2 = 100 \left(\frac{6}{10}\right)^2 = 36 \text{ tons/ft.}^3$$

where: k_5 = coefficient of subgrade modulus for 5' x 5 ' footing at surface.

b) Shape effect - None required for square footing.

$F_s = 1.0$

c) Make corrections for depth effects

1) Consider elastic theory and constant modulus. See Figure 56

D/B = 5/10 = 0.5

$F_D = 0.85$

2) Consider the effect of increasing modulus with depth.

No correction for increasing modulus or water table effects since k_1 was based on data from the zone of influence of proposed footing. $F_{DG} = 1.0$

d) Find corrected coefficient of subgrade reaction

$$k = 36 \times \frac{F_s}{F_D \times F_{DG}} = 36 \times \frac{1.0}{0.85 \times 1.0}$$

$k = 42 \text{ tons/ft.}^3$

e) Settlement computation

$\rho = q/k$

$$q = \frac{80 \text{ tons}}{5 \times 5} = 3.2 \text{ tons/ft.}^2$$

$$\rho = \frac{3.2}{42} = 0.076' = 1.1''$$

CHAPTER 10 - CONSTRUCTION MONITORING

10. 10 INTRODUCTION

Geotechnical engineering, by its nature, involves contingencies from unforeseen conditions that are encountered during construction. Bold innovative designs may justify experimental test sections and then continuing re-assessment and verification during actual construction.

The preceding chapters in this volume are addressed to engineering during the design phase. Geotechnical engineering must extend beyond design into construction, and therefore it is essential that data be obtained for re-evaluation of design assumptions and implementation of appropriate modifications.

10. 11 General

This chapter presents an overview of the purpose of construction monitoring, what is measured, and how the task is planned and executed. Emphasis is placed on open cut deep excavations and adjacent structures. Construction monitoring case histories were reviewed by Schmidt and Dunnicliff (1974), who also describe construction monitoring of soft ground tunnels.

10. 12 Reasons for Construction Monitoring

If a construction monitoring program is performed for the right reasons, planned properly, and executed by diligent engineers, it can make a large contribution towards increasing safety,reducing cost, and reducing the impacts of construction on the environs. Some valid reasons for monitoring are:

Diagnostic:	To verify adequacy of design
	To verify suitability of construction techniques
	To diagnose the specific nature of an adverse event
	To verify continued satisfactory performance
Predictive:	To permit a prediction of behavior later on at the same job

Legal:	To establish a bank of data for possible use in litigation
Research:	To advance the state-of-the-art by providing better future design data

10.20 PLANNING CONSTRUCTION MONITORING PROGRAMS

Many construction monitoring programs fail to achieve their purpose because the engineer does not approach the program design in a logical sequence. There is a tendency among engineers to select an instrument, make some measurements, and then wonder what to do with the data (Peck, 1970). The essential elements required for successful planning to a construction monitoring program are presented in Table 6. This table is not a substitute for an experienced engineer, but if used as a guide by such a man it will help to minimize the the possibility of a monitoring program failing in its purpose.

10.30 PARAMETERS TO BE MEASURED

10.3 1 General

The most important parameters to be measured, irrespective of wall or support type, are load, pore water pressure (or ground water level), and horizontal and vertical displacement.

Temperature measurements have special application to excavations supported by internal bracing because of the influence that temperature may have on bracing load. In general, direct earth pressure measurements have been unreliable except where backfill has been placed carefully against an instrumented structure, which is generally not possible with a deep excavation. It is preferred to determine earth loading from load measurements in supports.

10. 32 Instruments

Types of instruments suitable for measuring the above parameters together with advantages and limitations are given in Table 7. Less suitable instruments, although available and occasionally used, have not been included in this table. Schematic diagrams illustrating instrument operation principles are given by Dunnicliff (1970, 1971, 1972).

Table 6. Steps for planning construction monitoring programs.

1. Define the Problem

Project type
Soil conditions
Ground water conditions
Status of nearby structures

2. Define the Purpose of the Instrumentation

Diagnostic:	To verify adequacy of design
	To verify suitability of construction techniques
	To diagnose the specific nature of an adverse event
	To verify continued satisfactory performance
Predictive:	To permit a prediction of behavior later on at the same job
Legal:	To establish a bapk of data for possible use in litigation
	To demonstrate a contractor's compliance with contract requirements
Research:	To advance the state-of-the-art by providing better future design data

3. Select Monitoring Parameters

Load or stress
Pore water pressure
Earth pressure
Settlement or heave (surface or subsurface)
Horizontal movement (surface or subsurface)
Tilt
Temperature

4. Make Predictions of Behavior and Define Specific Instrumentation Needs

Range
Accuracy
Duration of readings
Frequency of readings
Data evaluation schedule

5. Decide Who Will Do What

Who will procure the instruments?
Who will install the instruments?
Who will monitor the instruments?
Who will maintain the instruments?
Who will process the data?
Who will analyse the data?
Who will decide on implementation?
Who will implement?

6. Select Instruments, Components, and System

General Criteria
- Select each part of the system with equal care
- Will it achieve objective?
- Maximum simplicity
- Maximum durability in installed environment
- Minimum susceptibility to vandalism
- Appropriate accuracy, range, longevity
- Good past performance record
- Minimum cost (to furnish, install, read, process)
- Maximum environmental stability
- Calibration can be verified after installation
- Consistent with skills of available personnel as in 5. above
- Minimum interference to construction while installing and reading
- Minimum falsification of measured parameter

Selection
Refer to Table 7 and Section 10.40

7. Determine What Factors May Influence Measured Data (to permit an analysis of cause and effect)

Detailed record of all construction particulars, progress and other data
Incidence of any observed distress or unusual event
Environmental factors which may, in themselves, affect monitored data, e.g. temperature, nearby construction activities

8. Plan Procedures for Ensuring Reading Correctness

Consider necessary redundancy
Consider duplicate measuring system
Plan how instruments will be calibrated and corrected for environmental effects
Consider possibility of feature to check-calibrate in place

9. Determine a Numerical Value of Deviation from Anticipated Performance at which the Engineer Should:

Be concerned
Press the panic button

10. Plan Instrument Layout

How many?
Where?

11. Write Instrument Procurement Specifications

12. Plan Installation

Write installation specifications
Prepare field data sheet for recording details of installation
Examine every detail of the planned installation procedure and think through alternative methods in the event problems arise
Make detailed list of all materials and tools required

13. Plan Procedures Subsequent to Installation

Plan monitoring arrangements
Prepare field data sheets
Plan maintenance arrangements
Plan data processing arrangements
Plan analytical procedures
Plan remedial measures (in the event data indicates adverse event) or other methods of implementation, and forewarn all concerned parties

Table 7. Types of available instruments.

Parameter	Instrument	Advantages	Limitations
Surface movement of buildings and adjacent ground (horizontal and vertical).	Optical survey for settlement using settlement reference points.	Simple and direct.	Care required to prevent pin disturbance. Requires reliable benchmark.
	Optical survey for horizontal movement, using offsets from line of transit.	Simple and direct.	Requires immovable reference stations.
	Crack opening using portable mechanical gage.	Simple, inexpensive and direct.	Care required to prevent disturbance to reference points.
	Hose level for settlement within buildings.	Precise. Can monitor many points.	Requires skill to read. Manpower reading costs high.
	Tilt using "tidal quality resolution" tiltmeters.	Very precise, hence give useful data in short monitoring period.	Expensive and complex.
Subsurface settlement of adjacent ground.	Single or multi-point rod extensometer with mechanical readout.	Simple and reliable.	Rods can hang up within surrounding sleeves if many anchors in one hole, thereby falsifying readings. Requires manual access to read (may create traffic interference and danger to reading personnel).
	Magnet/reed switch vertical pipe gage.	Anchors follow pattern of settlement without falsification. Simple and reliable.	Requires manual access to read (may create traffic interference and danger to reading personnel).
	Single or multi-point embedded rod extensometer with electrical readout.	Can be read remotely, without traffic interference.	Rods can hang up. More prone to malfunction, damage and vandalism than mechanical readout.
Subsurface horizontal movement' of adjacent ground.	Horizontal or inclined extensometer.	Only few required to locate zone of no displacement.	Expensive. Must relate to datum for absolute movements.
	Inclinometer.	No embedded electrical parts. Gives full full depth profile of movement.	Expensive but usually cheaper than extensometers. Many required to locate zone of no displacement. Must relate to immovable reference station.
Movement of soldier piles, sheet piles, walers and diaphragm walls (horizontal and vertical).	Optical survey.	Simple and direct.	Requires reliable benchmark and immovable reference stations. No readings possible at depth until member exposed.
	Inclinometer installed on pile or in wall.	Readings at all depths available immediately after pile or wall installation.	Not suitable for driven piles. For soldier piles, must install inclinometer casing inside pipe for protection during pile installation. Best to weld pipe and install casing after pile is in place.

Table 7. Types of available instruments. (Continued).

Parameters	Instrument	Advantages	Limitations
Movement of tieback anchors.	During proof test: Dial gage, mounted on survey tripod.	Simple and direct.	
	After proof test: Sleeved unstressed telltale rod attached to anchor.	Simple and direct.	Measures only relative movement of soldier pile and anchor. Must relate to datum for good understanding of anchor load test.
Bottom heave of excavation.	Anchor embedded in borehole below eventual bottom Elevation read with probing	Simple. Inexpensive.	Requires survey crew. Risk of boring caving during excavation.
	Single or multi-point embedded rod extensometer with electrical readout.	Precise. Cann connect several sensors at different elevations to one anchor. Can become settlement gage.	Risk of electrical failure and damage during excavation. Bottom anchor must be deep enough to serve as benchmark.
Load and stress in struts, soldier piles, sheet piles, walers and diaphragm walls.	Mechanical strain gage.	Inexpensive. Simple. Easy to install. Minimum damage potential.	Access problems. Many temperature corrections required. Limited accuracy. Readings are subjective.
	Vibrating wire strain gage.	Remote readout. Readout can be automated. Potential for accurracy and reliability. Frequency signal permits data transmission over long distances. Gages can be re-used.	Expensive. Sensitive to temperature, construction damage. Requires substa· tial skill to install. Risk of zero drift. Risk of corrosion if not hermetically sealed.
	Electrical resistance strain gage.	Inexpensive. Remote readout. Readout can be automated. Potential for accuracy and reliability. Most limitations listed opposite can be overcome if proper techniques are used.	Sensitive to temperature, moisture, cable length change in connections, construction damage. Requires substantial skill to install. Risk of zero drift.
Load in tieback anchors.	Telltale load cell.	Inexpensive. Simple. Calibrated in- place.	Access problems. Cannot be used with all proprietary anchor systems.
	Mechanical load cell.	Direct reading. Accurate and reliable. Rugges and durable.	Expensive. Access problems.
	Electrical resistance strain gage load cell.	Remote readout. Readout can be automated.	Expensive. Sensitive to temperature, moisture, cable length, change in connections. Risk of zero drift.
	Vibrating wire strain gage load cell.	Remote readout. Readout can be automated. Frequency signal permits data transmission over long distances.	Expensive. Sensitive to temperature. Risk of zero drift.
	Photelastic load cell.	Inexpensive.	Limited capacity. Access problems. Kequires skill to read.

Table 7. Types of available instruments. (Continued).

Parameter	Instrument	Advantages	Limitations
Earth pressure on sheet piles and diaphragm walls.	Hydraulic, pneumatic or electrical interface stress Cdl.	Direct method.	Few successful case records.
	Backfiguring from strut load measurements.	As for strain gages above.	As for strain gages above.
Groundwater or piezometric level.	Standpipe piezometer or wellpoint.	Simple. Reliable, Long experience record. No elaborate terminal point needed. Heavy liquid version available for reducing response time and overcoming freezing problems.	Slow response time. Tubing must be raised nearly vertical. May create traffic interference and danger to reading personnel. Freezing problems.
	Pneumatic piezometer.	Level of terminal independent of tip level. Rapid response.	Must prevent humid air from entering tubing.
	Vibrating wire strain gage or semi-conductor pressure transducer piezometer. Suitable for automatic readout.	Level of terminal independent of tip level. Rapid response. High sensitivity.	Expensive. Temperature correction may be required. Errors due to zero drift could arise (although most manufacturers have overcome major problems).
Temperature	Thermistor	Precise	Delicate, hence susceptible to damage. Sensitive to cable length.
	Thermocouple	Robust. Insensitive to cable length. Available in portable version as "surface pyrometer"	Less precise than thermistor, but premium grade can give ±1° F.

10. 33 Related Parameters

To permit a cause and effect analysis, a complete record of other relevant parameters should be monitored:

a. A record of depth of excavation versus time, at close stations.

b. Time of installation of **all wales,** struts, and ties, with preload records if any, and depth of excavation below strut or tie at time of installation.

c. Incidences of extraordinary ground losses, ground water behavior, observed distress, or any other unusual event.

d. Complete as-built construction plans and records, including records of any pile driving.

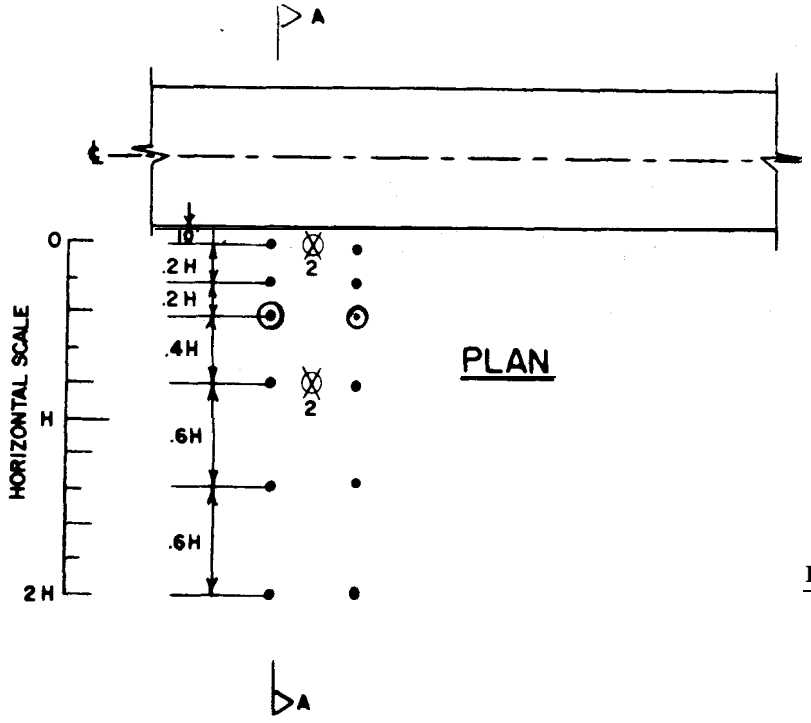
e. Environmental factors which may, in themselves, affect monitored data, e. g. temperature, nearby construction activities.

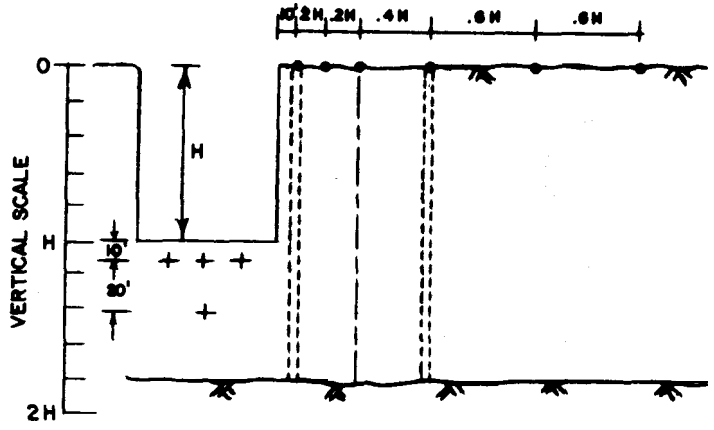
10.40 EXAMPLES

Two somewhat overly simplified situations are demonstrated in the accompanying Figures 58 and 59.

Figure 58 is at a test section well removed from adjacent structures that might be damaged by displacements caused by the excavation. The objective of the test section is to determine the magnitude and influence zone of displacements. Note that vertical and horizontal displacements can be measured at and below the surface.

Figure 59 is an example of monitoring performed for a building close to the excavation. Horizontal and vertical displacements are measured at and below the surface; also, settlement points are e s tabli s hed around the building. Concern over po s sible consolidation requires monitoring of piezometric levels above and below the clay.



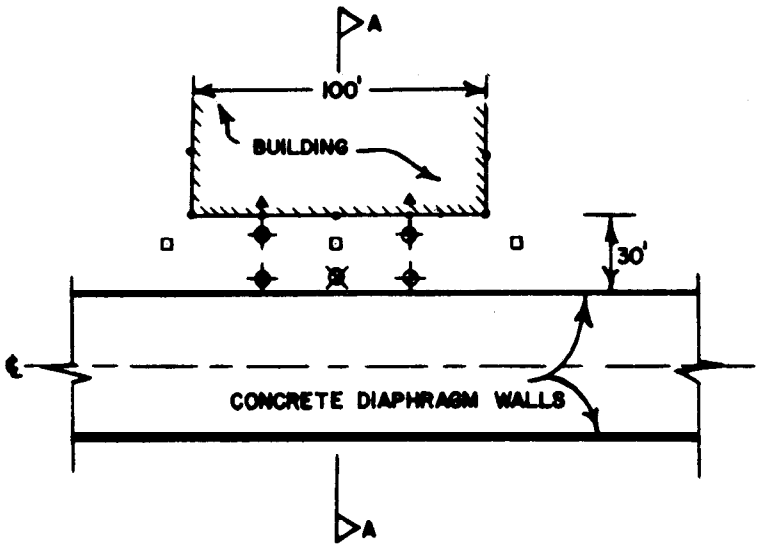


SECTION A-A

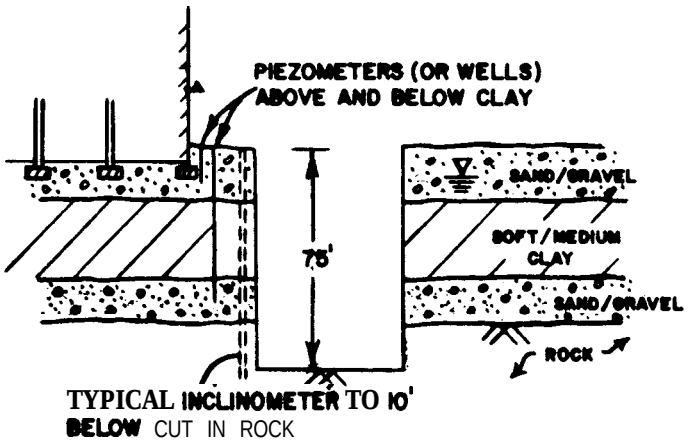
LEGEND

1. ⊙ δ_v - multi-point subsurface settlement system (e. g. magnet/reed switch gage)
2. ⊗ δ_v, δ_h - multi-point subsurface settlement and horizontal movement system (e. g. inclinometer with magnet/reed switch gage)
3. : δ_v - surface settlement point
4. + δ_v - heavepoint (use when underlain by cohesive soil; otherwise optional)

Figure 58. Example of instrumentation for measurement of displacement at test section.



PLAN



SECTION A-A

LEGEND

1. • δ_v - settlement point at surface
2. ✚ δ_h - inclinometer (for measurement of subsurface horizontal movements)
3. ✖ δ_h, δ_v - inclinometer with multi-point subsurface settlement system
4. ▫ - piezometer
5. 4 - tilt meter on building
6. Monitor horizontal movements at the face of the diaphragm wall by optical survey. Monitor settlement of diaphragm wall by optical survey.

Figure 59. Example of instrumentation adjacent to building and diaphragm wall.

lo. 50 RECOMMENDED METHODS OF MEASUREMENT

10.51 General

The selection of the measurement method depends upon case specific factors, and no general rules can be made. However, Table 7 provides basic information to assist in the selection process. The most difficult task, and the task with the poorest success record, is measurement of load in supporting members. The following sections therefore provide more detailed guidelines to assist in selecting a method for monitoring load in braced and tied-back excavations.

10. 52 Strut Loads

10. 52. 1 Instrument Type

For monitoring strut loads, strain gages are preferred rather than load cells, primarily because inclusion of a load cell will tend to create non-typical loading conditions and will interfere with the contractor's work. Strain gages permit measurement of bending stresses, whereas a load cell does not.

Since the most expensive feature of an instrumentation program is often the disruption to construction activities, remote readout is a desirable feature, and vibrating wire and electrical resistance strain gages are the preferred instruments. Selection between the two gage types should be based on the experience and skills of available personnel, rather than on any quality inherent to one or the other type.

In general, a backup system should be established, using mechanical strain gages, although of course, their use is limited by access restrictions. Each gage is discussed below, and advantages and limitations are sur-nmarized in Table 7. In general, an accuracy of no better than ± 10 percent of design load can be attained, and this accuracy is usually adequate.

10, 52. 2 Vibrating Wire Strain Gages

O'Rourke and Cording (1974b) provide a detailed and up-to-date technical quide in use of vibrating wire gages. Gages that are perfectly temperature compensated (equal thermal coefficients for gage and structural member) will provide optimum accuracy, and the vibrating wire itself should be as close as possible to the surface of the strut.

The problem of thermal response, in particular caused by temperature differential between gage and strut, and the problem of zero drift remain the most severe. Estimates of long term zero drift are necessary to judge the reliability of measurement. Individual gages set to different frequencies should be mounted on unloaded sections of strut steel and monitored throughout the life of the project. No-load gage readings should be taken after strut removal and then compared with the initial no-load readings taken before strut installation. Temperatures should be measured at the time of each recording, and readings should be appropriately corrected on the basis of temperature changes between the initial and subsequent values.

10. 52.3 Electrical Resistance Strain Gages

Electrical resistance (SR4) strain gages have been used very successfully to monitor strain in laboratories, but their use in field measurements has often yielded poor results, Largely because of the inexperience of personnel undertaking the monitoring program. Since an electrical property of the wire rather than a mechanical property is being measured, it is important that these personnel have experience in field electronics, and such personnel are rare among geotechnical firms.

Dunnicliff (1975) elaborates on key factors: gage selection and installation; sensor configuration; wiring; amplification and readout equipment. The task requires attention to many minute details, and wherever possible the gage installation work should be performed in a controlled labor environment prior to installation of the structural member.

When using electrical resistance gages zero drift tests should be made, as described above for vibrating wire gages.

Weldable gages have not yet been used on a widespread basis, perhaps because they are more costly than bonded gages, but the ease of installation and hermetic insulation of the gages are great advantages. Their sensitivity, although less than that of bonded gages, will normally be adequate.

10. 52.4 Mechanical Strain Gages

Although less accurate than the gages discussed above, use of mechanical gages provides valuable backup data provided three rules are adhered to. First, the Demec rather than the Whittemore type should be used (Schmidt and Dunnicliff, 1974). Second,

gage points must be rigidly attached to the structural member, by drilling into the member or by welding. Third, proper temperature correction procedures must be used, taking into account thermal coefficients of strut, gage, and gage reference bar.

10. 52. 5 Temperature Correction

Strain-gages are used to measure strain, which then has to be converted to stress and load by using a value for modulus. However, temperature change also causes strain, and any such strain must be subtracted from measured strain before the conversion to stress is made. In the absence of complete temperature compensation (possible using resistance gages, approachable using certain vibrating wire gages, impossible using mechanical gages) temperature must be measured and thermal strain accounted for. Temperature variations always contribute to inaccuracy, and any effort to minimize temperature variation is worthwhile.

10.53 Tieback Loads

Load cells have been more commonly used to monitor tieback loads than have strain gages. Strain gages are inapplicable for use on stranded wire tendons since no convenient method is available for attaching the gages. Furthermore, a single load cell with a central hole can surround an entire group of tendons. It is possible to attach strain gages to steel rods, although the rate of gage attrition is generally high. Advantages and limitations of the five basic types of load cell are given in Table 7. Portable ' calibrated' hydraulic jacks have also been used, but measurement error may be up to 30 percent.

Selection of cell type depends on the factors given in Table 6, on past personal experience of the engineers executing the monitoring program, and load cell availability. Dunnicliff (1975) describes an inexpensive home -made "telltale load cell" capable of monitoring 150 kip loads with an accuracy of ±5 kips, and in view of its simplicity and economy, it seems logical to use this method wherever feasible.

A backup system is desirable, although less necessary than for strut load monitoring. This can be done in one of two ways, although neither way is always practical. First, one or more Load cells, on a special test frame or rod, can be retained on the site in

a loaded condition. Readings can be examined for drift, and the cells can be checked periodically by calibrating in the normal way. Second, it may be possible to check and calibrate selected cells in place by pulling a tie to unload the cell and then releasing the tie. For this test, it is necessary to connect a calibrated load cell, in addition to the stressing jack, in series with the cell under test.

10.60 CONTRACTING FOR INSTRUMENTATION

Table 8 presents the three basic contracting methods for furnishing and installing instrumentation.

Sophisticated instrumentation should not be included as a bid item in the prime contract, as the task requires professional skill and dedication, usually unobtainable if the prime contractor shops between ' specialist' subcontractors. A separate contract between owner and a specialist firm is suitable for sophisticated instrumentation provided the specialist and prime contractor's work areas do not overlap. If sophisticated instrumentation is to be installed within the prime contractor's work area the only viable method is use of a cost plus item in the prime contract. The essential elements of this procedure are:

a. Work which is within the capability of the average prime contractor is bid in the normal way.

b. The prime contract specification defines the nature of special instrumentation work. This work is included in the bid schedule as an allowance item, with an estimate of cost, and the prime contract bidder bids a markup, carrying forward the marked up total to the amount column. The estimate is not an upset.

c. The owner selects an instrumentation specialist firm, using normal professional procedures for engagement of engineering services, and agrees on a basis of payment for the firm's services.

d. The owner instructs the prime contractor to enter into a subcontract with the specialist firm and to pay the firm in accordance with the agreed basis. The contractor is reimbursed by the owner at cost plus the bid markup.

This procedure requires a clear and thorough prime contract specification. It also requires close coordination, cooperation, and trust between owner and specialist to ensure that all expenditures are necessary,

Table 8. Contracting for instrumentation.

Type of Instrumentation	Location of Instrurrients	Example	Contract for Furnish and Install		
			, Bid Item in Prime Contract	Separate Contract With Specialists	cost Plus Item in Prime Contract
Simple	Outside or within contractor's work area.	W ellpoint s Optical survey	Suitable	Not necessary	Not necessary
Sophisticated	Outside contractor's work area	Inclinometer with top on sidewalk	Not suitable	Suitable	Suitable
	Within contractor's work area	Strain gages on struts Load cells on tiebacks	Not suitable	Not suitable	Suitable

thereby keeping costs to a minimum. If handled properly, it results in cooperation, flexibility to accommodate changes as the work proceeds, and a successful monitoring program at minimum cost to the owner.

10.70 THE KEY TO SUCCESSFUL CONSTRUCTION MONITORING

The key to successful construction monitoring may be stated as:

a. Have a valid reason for monitoring, and perform complete and logical planning (Table 6).

b. Select the most appropriate parameters and instruments (Table 7).

c. Establish workable contractual arrangements with experienced personnel who have a full understanding of the monitoring objective, and who have the patience and desire to ensure the success of the program (Table 8).

d. Achieve cooperation between all parties in the field. Cooperation can best be gained by explaining to the contractor's personnel the purpose of the program, gaining his respect by performing top quality work, then throughout the program being responsive to the effects of the.program on him/her and working with him/her to minimize any adverse effects.

e. Observe and record all relevant construction data (Section 10.33).

f. Make use of the data in the way intended.

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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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21

LATERAL SUPPORT SYSTEMS AND UNDERPINNING

Vol. III. Construction Methods

D. T. Goldberg, W. E. Jaworski, and M. D. Gordon



April 1976
Final Report

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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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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

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.

Symbol	Represents	Reference
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	

Symbol	Represents	Reference
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	
$\delta_{v_{(max)}}$	vertical displacement (maximum)	
$\delta_{h_{(max)}}$	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	

Symbol	Represents	Reference
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. Sheeting 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 solis, 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 whith 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 if 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 Wood Lagging[2]	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

[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.

1.84 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/ft^3; chemical grouts: $1.50 - $7.00/ft^3); 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/yd^3 of grouted soil; chemical grouts: \$40 - \$190/yd^3 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 in 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.

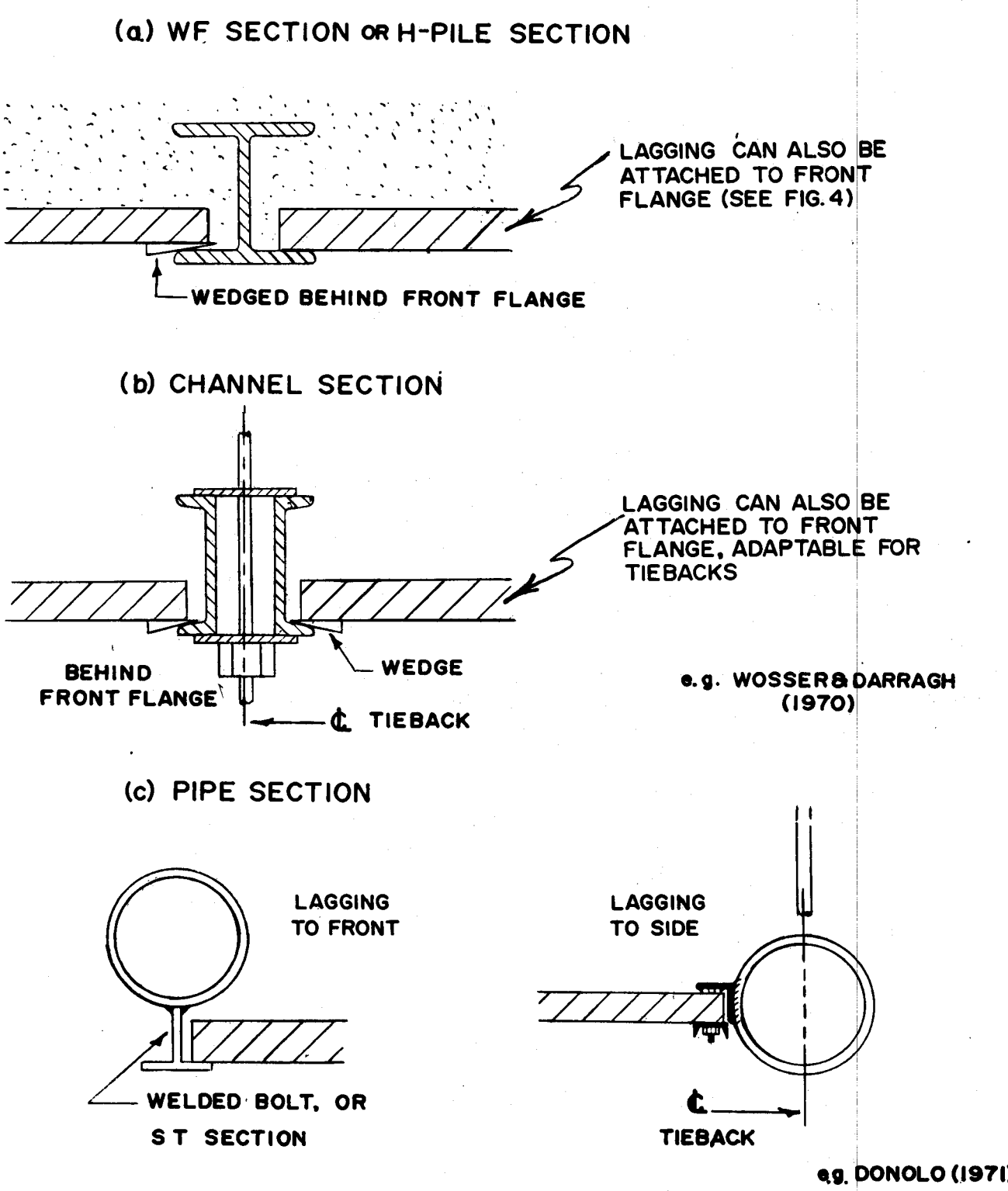


Figure 1. Steel soldier piles.

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 Gunite 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 erpsion 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.).

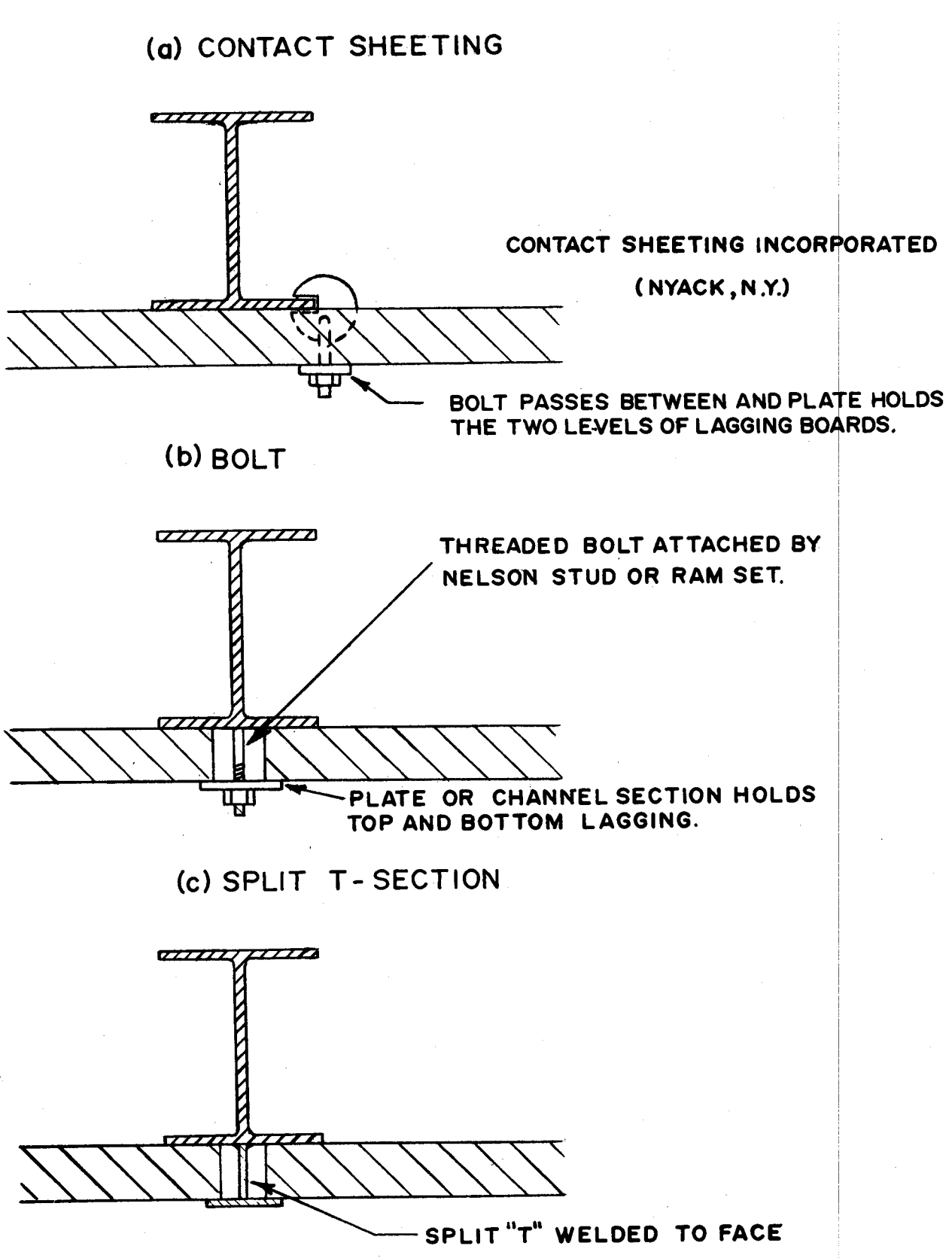
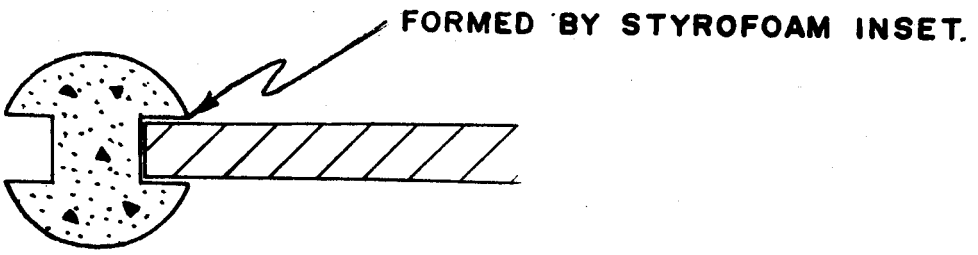


Figure 4. Wood lagging to front flange.

(a) WOOD LAGGING



(b) ARCHED REINFORCED CONCRETE

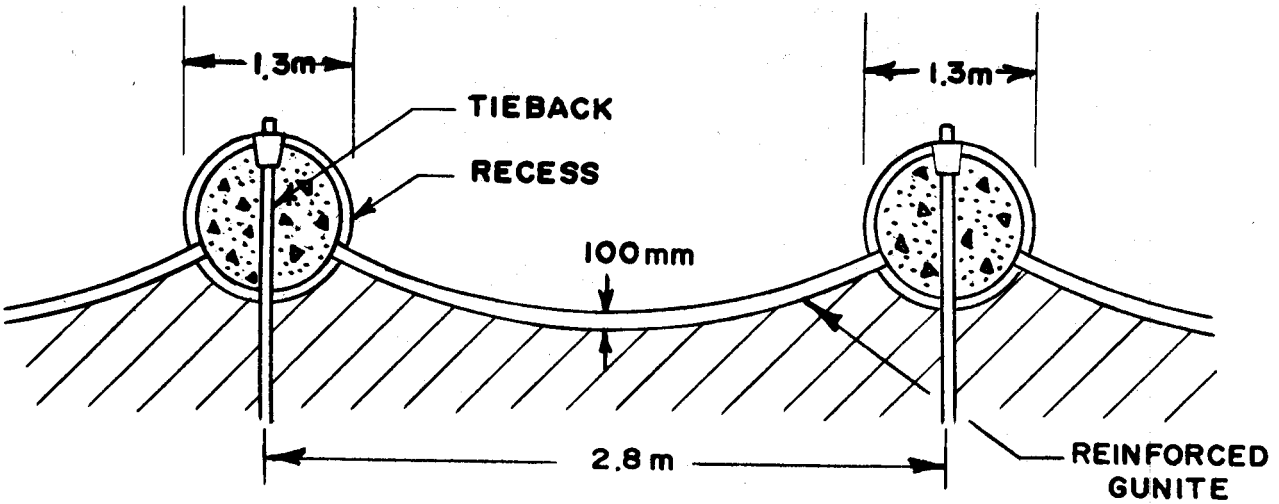


Figure 5. Cast-in-place concrete soldier piles.

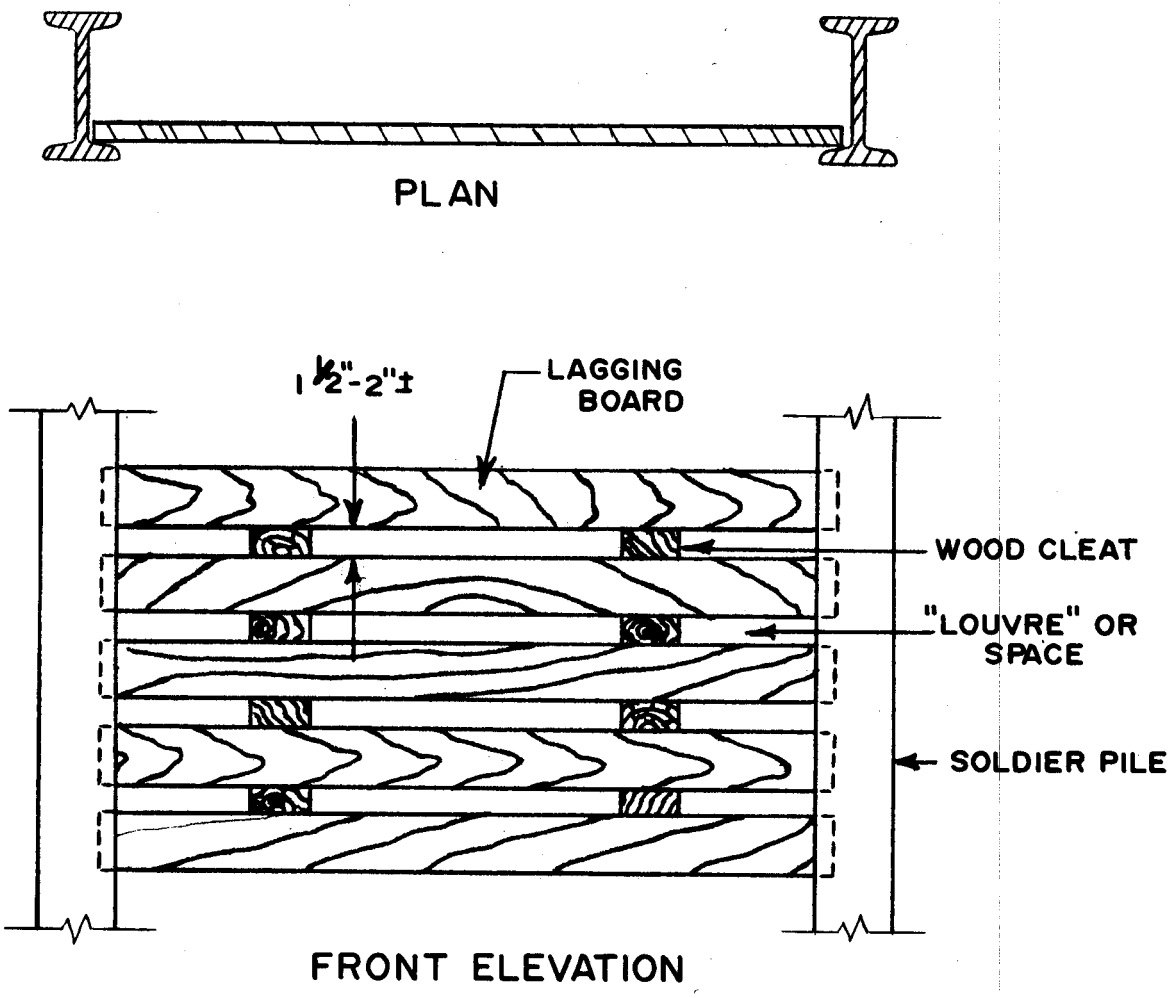


Figure 6. Louvre effect for wood lagging.

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". Wall".

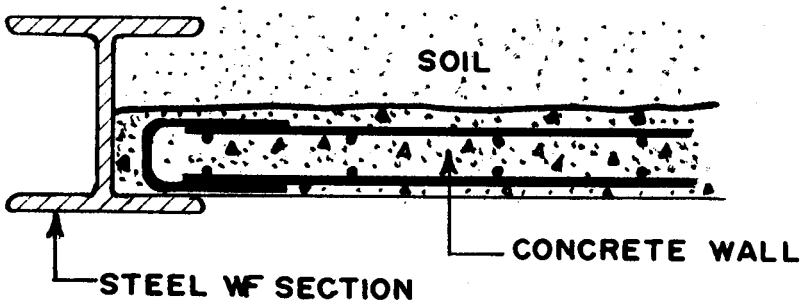
2.23 SOLDIER PILES ALONE

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

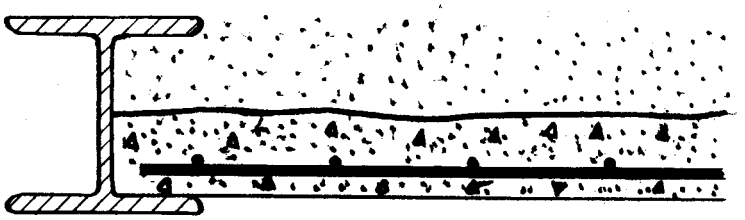


Figure 7. Concrete infill between soldier piles.

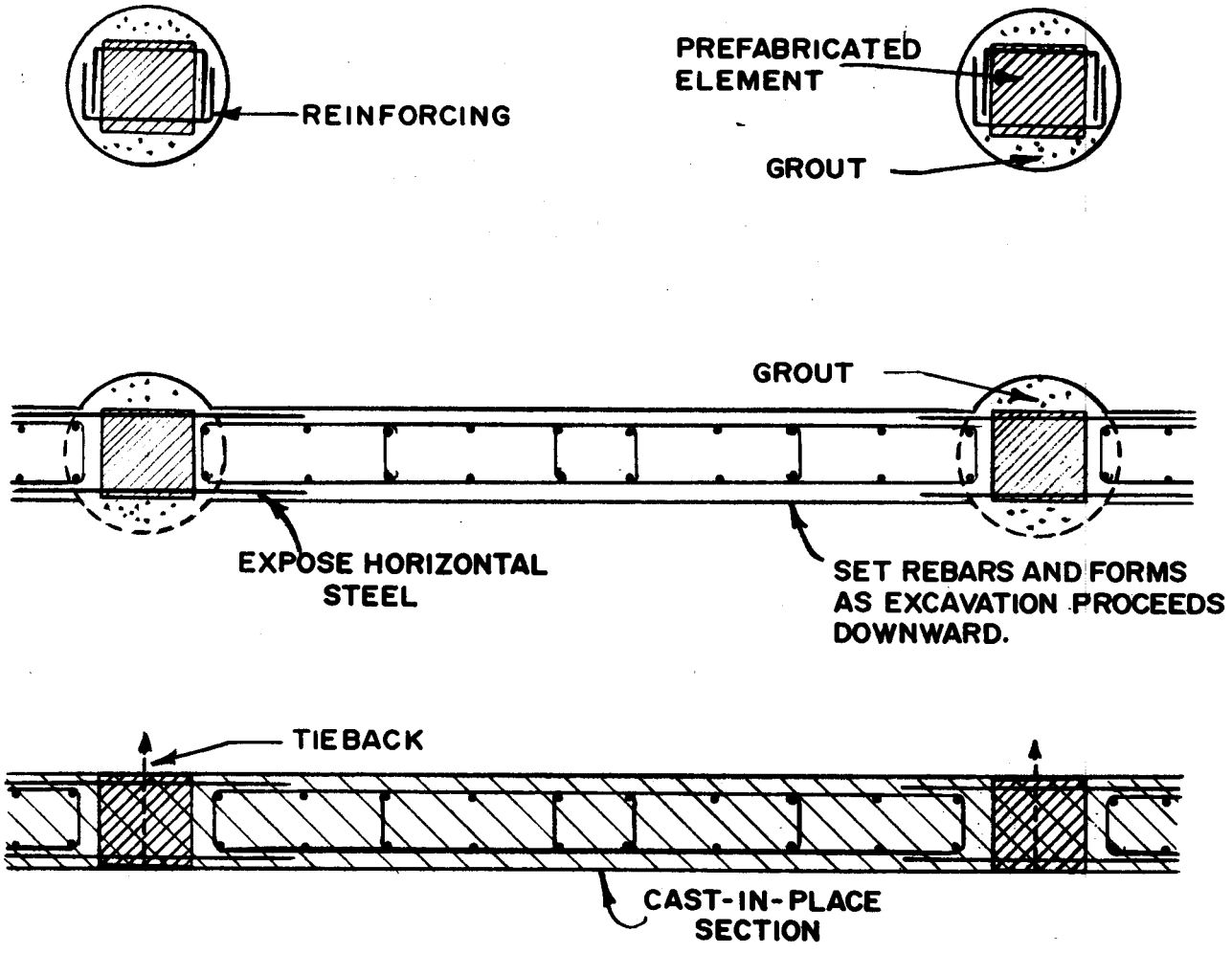


Figure 8. Parisienne wall, precast soldier piles with formed cast-in-place wall, (after Fenoux, 1974; Xanthakos, 1974; and D'Appolonia, et al, 1974).

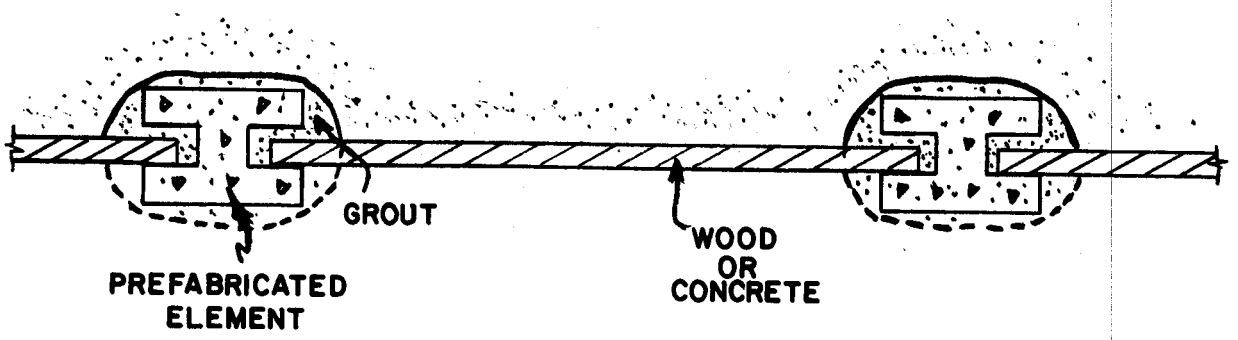


Figure 9. Berlin wall, precast soldier piles with wood or precast concrete lagging (after Fenoux, 1974).

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
Douglas Fir - Larch, surfaced dry or surfaced green used at max. 19% M. C.		
Construction	1200	1,500,000
Select Structural	2050	1,800,000
Douglas Fir - South, surfaced dry or surfaced green used at max. 19% M. C.		
Construction	1150	1,100,000
Select Structural	1950	1,400,000
Northern Pine, surfaced at 15% moisture content, used at 15% max. 19% M. C.		
Construction	1050	1,200,000
Select Structural	1750	1,500,000
Southern Pine, surfaced at 15% moisture content K. D., used at 15% max. M. C.		
Construction	1300	1,500,000
Select Structural	2250	1,900,000
Southern Pine, surfaced dry, used at max. 19% M. C.		
Construction	1200	1,400,000
Select Structural	2050	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_o value in excess of 2 or 3. Heavy overconsolidated soils have a tendency to expand laterally expecially 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.

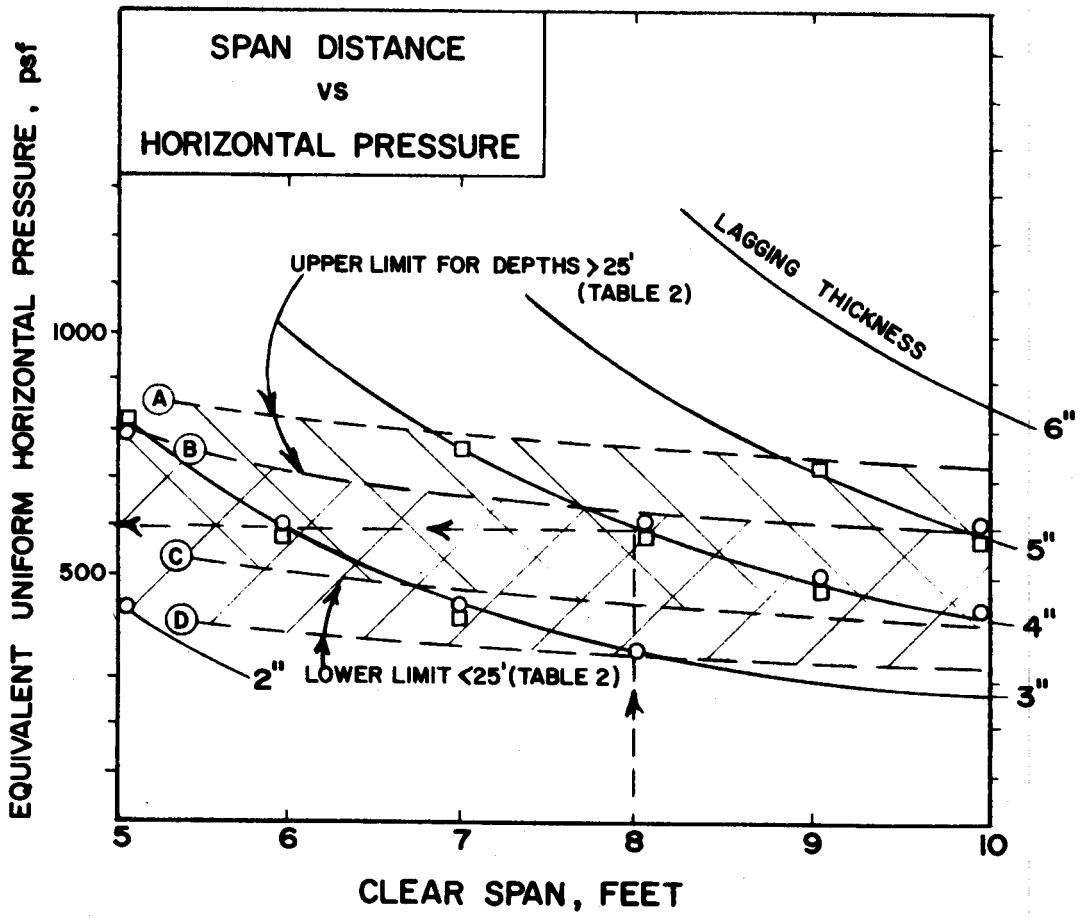
2.32.5 Equivalent Uniform Pressure

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.



1. 

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. ○ (A) (C) Implied range for "competent" soils (Table 2).

4. □ (B) (D) 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.

Let: γ = 130 pcf
K_a = 0.30

where:

γ = unit weight

K_a = 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

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

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

3. Check stress

Allowable = 1.5 x 2000 = 3000 psi

Check 9 foot clear span

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

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

Check 10 foot clear span

4 inch lagging: $f = \frac{M}{S} = \frac{7500 \times 12}{32} \approx 2800$ 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, the 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.

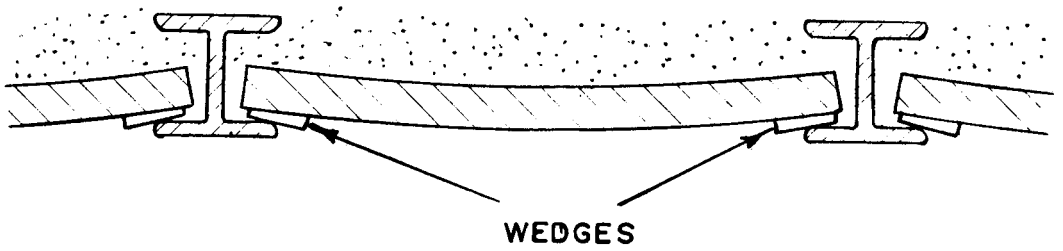
Examples of a German procedure for dealing with soft unstable soil are shown in Figure 11.

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.

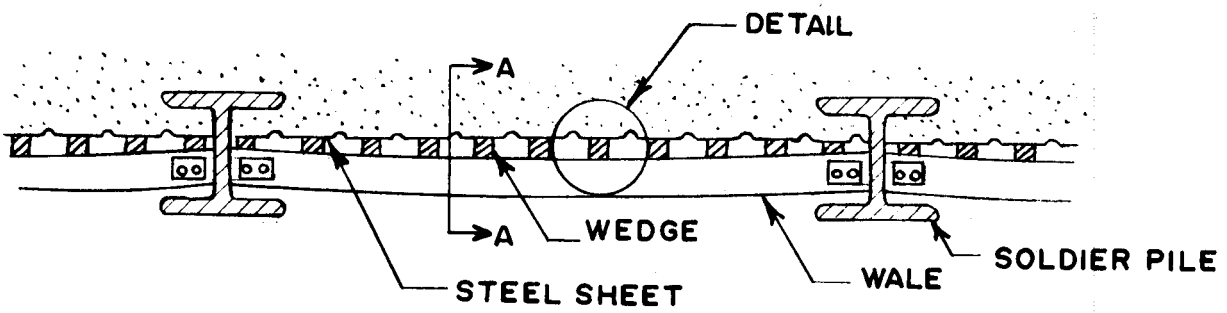
Dry cohesionless soil may also lead to difficulty, especially in hot, arid areas. Under these circumstances, one technique

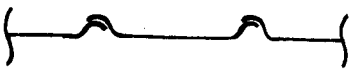


(b) CONCRETE



(c) VERTICAL STEEL SHEETS IN SOFT SOIL





DETAIL

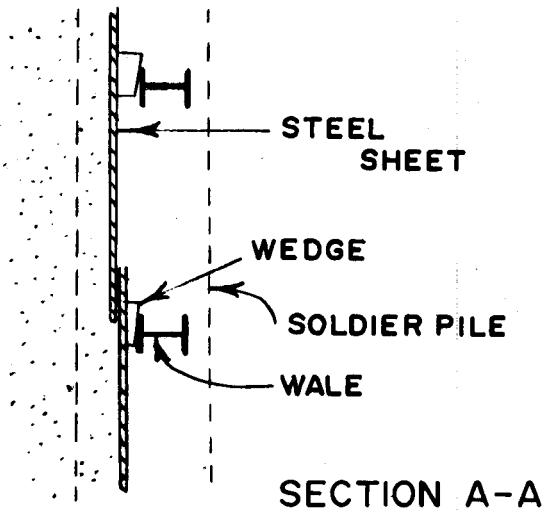


Figure 11. German techniques to prevent deformations (after Weissenbach, 1972).

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 frenquency.

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 nearly 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-excavation 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. AWPA 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
Osmose 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 nodulus for corresponding weights than the medium arch and deep arch sections designated as PMA-22 and PDA-27.

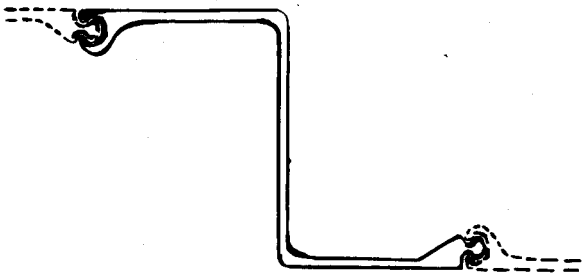
Note that the PDA section and PMA section interlock on the midline of the wall, whereas the "Z" sections interlock on the inside and the outside line of the wall. With regard to the deep arch and medium arch sections, it is conventionally assumed in American practice that shear cannot develop along the 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

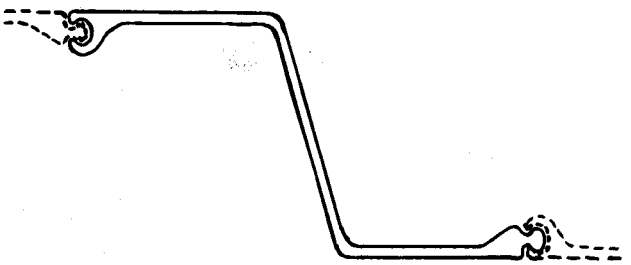
The conventional ASTM grade used for sheet piling is A 328, which has a minimum yield point of 38,500 psi. Some companies produce steel sheet piling in higher strength steel using ASTM grade A 572 in three types: 45,000; 50,000; and 55,000 psi yield point steel (see Table 6).

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.

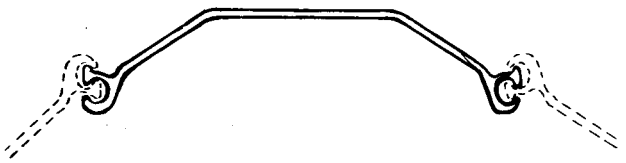
AISC allowable stresses may be used for the steel sheet pile wall at full depth. Temporary, intermediate conditions which exist during the course of excavation may be analyzed using a 20 percent overstress above the normal AISC allowable stress.



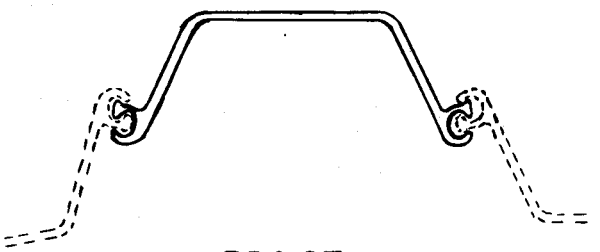
PZ 38



PZ 32 & PZ 27



PMA 22



PDA 27

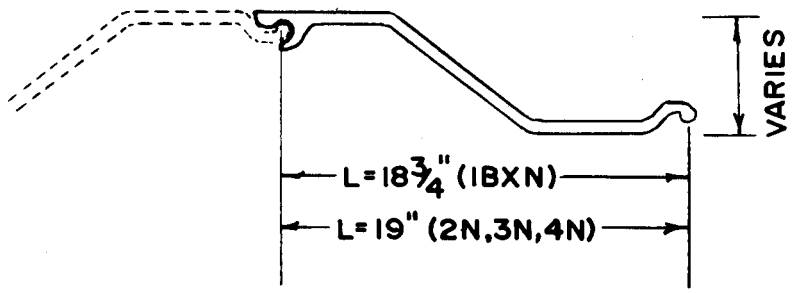
Figure 13. Domestic sheet pile sections.

Table 4. Domestic steel sheet pile sections.

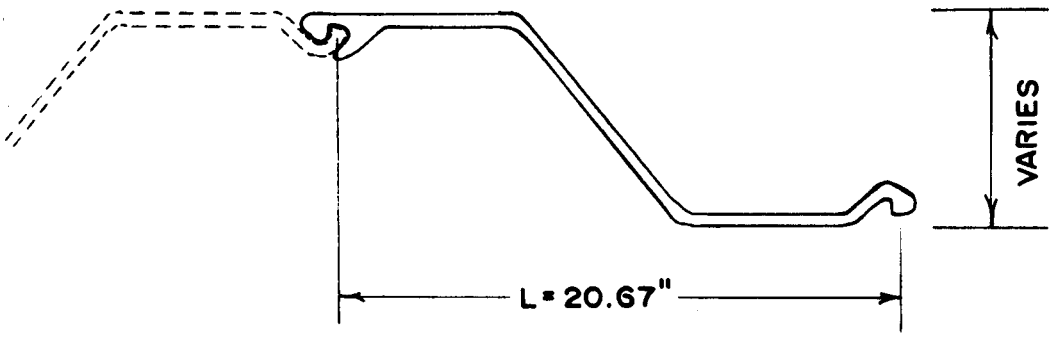
Section	Dimension (in)		Weight lb/sf	Moment of Inertia in^4/ft	Section Modulus in^3/ft
	D, depth	L, length			
PMA 22	3-1/2 x 2 = 7[1]	19.6	22.0	16	5.4
PDA 27	5 x 2 = 10	16	27.0	40	10.7
PZ 27	12	18	27.0	183	30.2
PZ 32	11.5	21	32.0	220	38.3
PZ 38	12.0	18	38.0	281	46.8

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

FRODINGHAM 1BXN, 2N, 3N, 4N



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



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

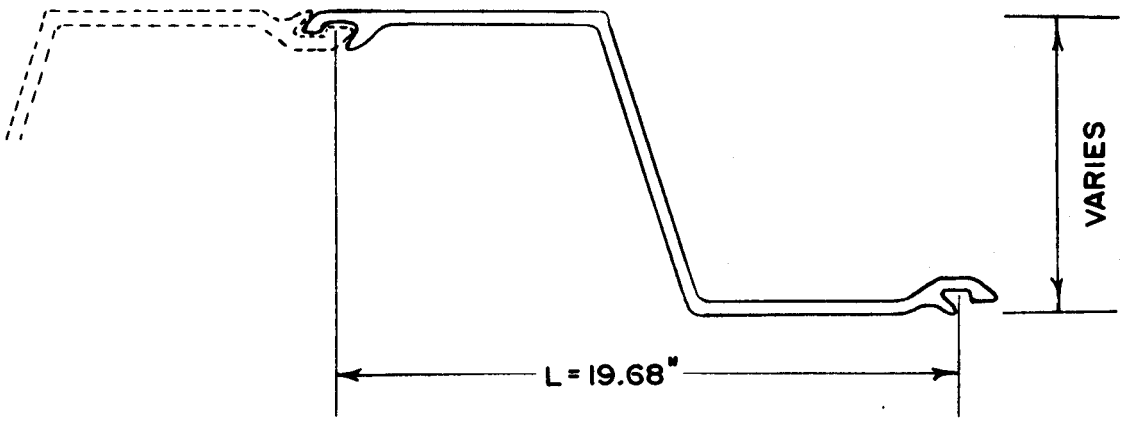


Figure 14. Foreign sheet pile sections.

Table 5. Foreign steel sheet pile sections.

Section	Dimension (in) D, depth	Dimension (in) L, length	Weight lb/sf	Moment of Inertia in^4/ft	Section Modulus in^3/ft
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 donward 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.

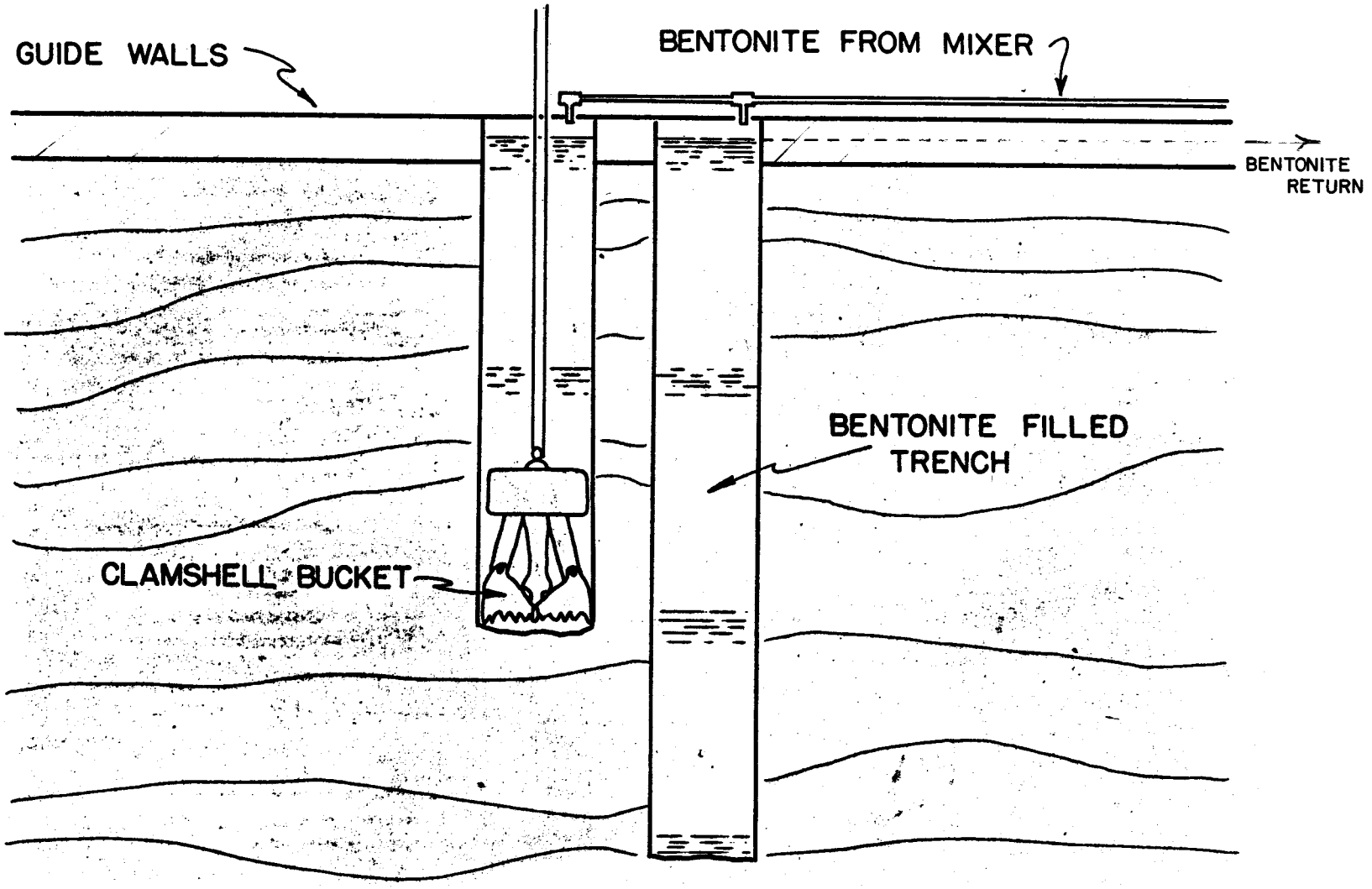


Figure 15. Excavation by clamshell bucket.
(Courtesy of ICOS Corporation).

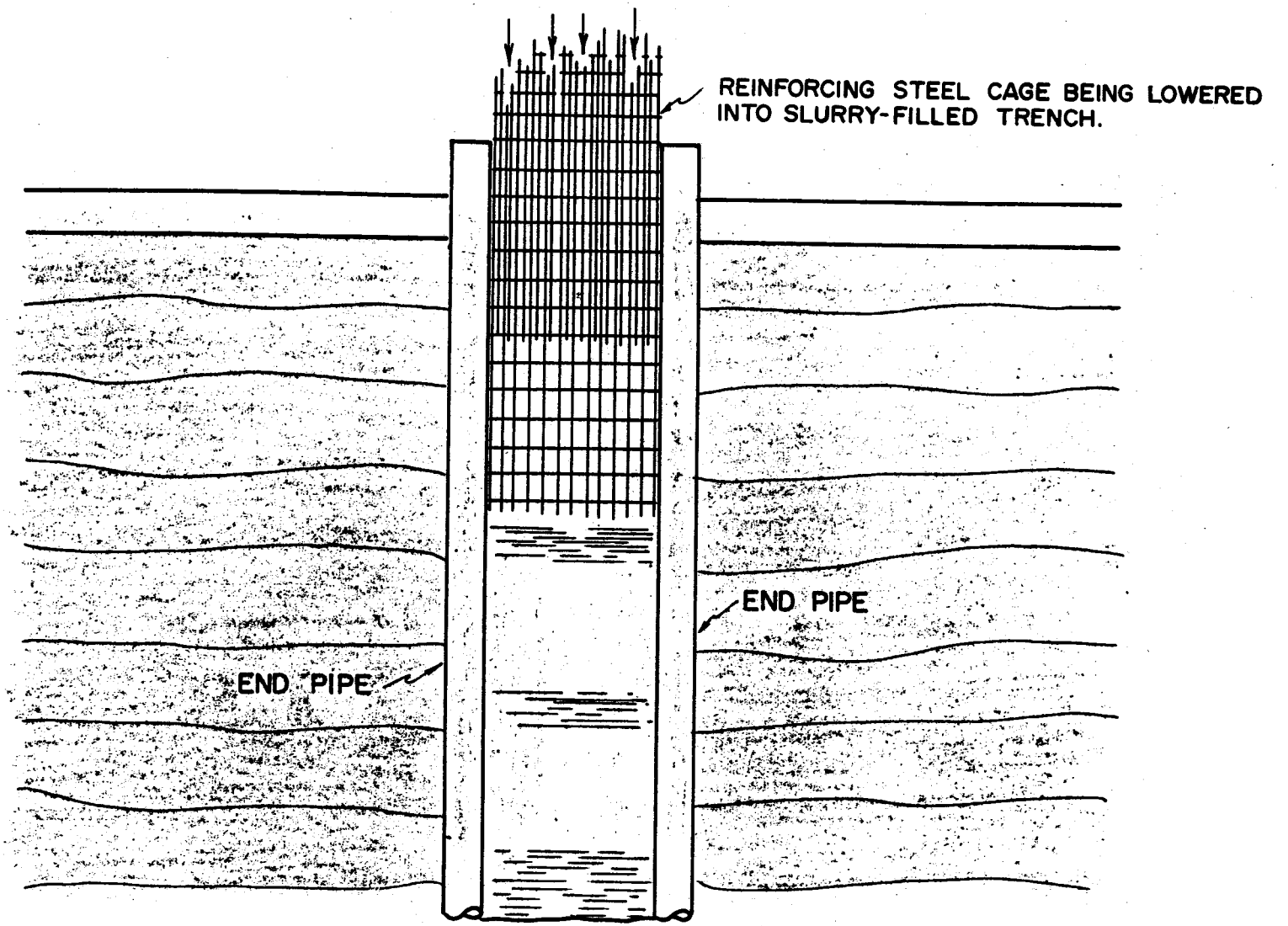


Figure 16. Preparations for concrete placement.
(Courtesy of ICOS Corporation).

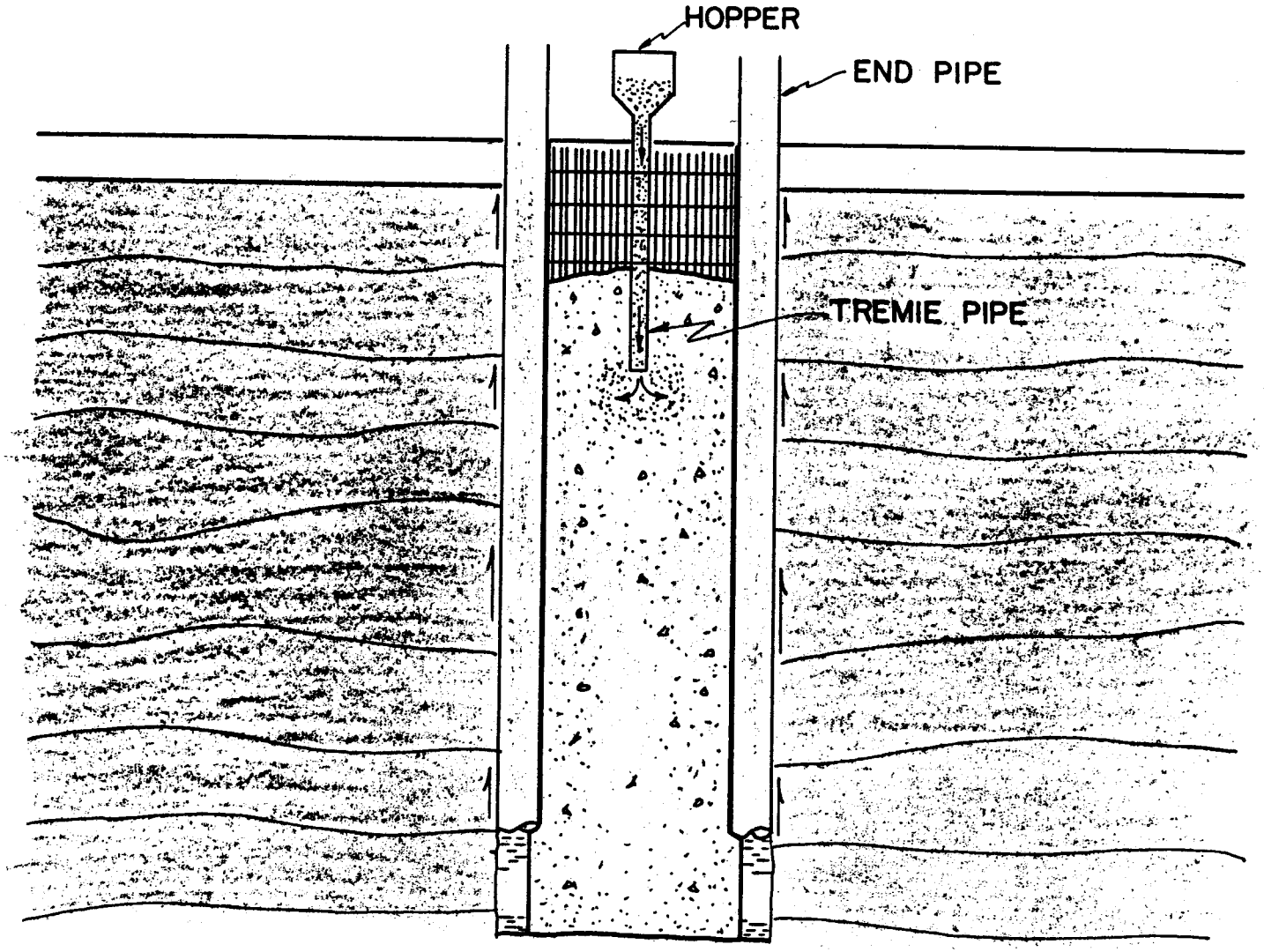


Figure 17. Placement of concrete.
(Courtesy of ICOS Corporation).

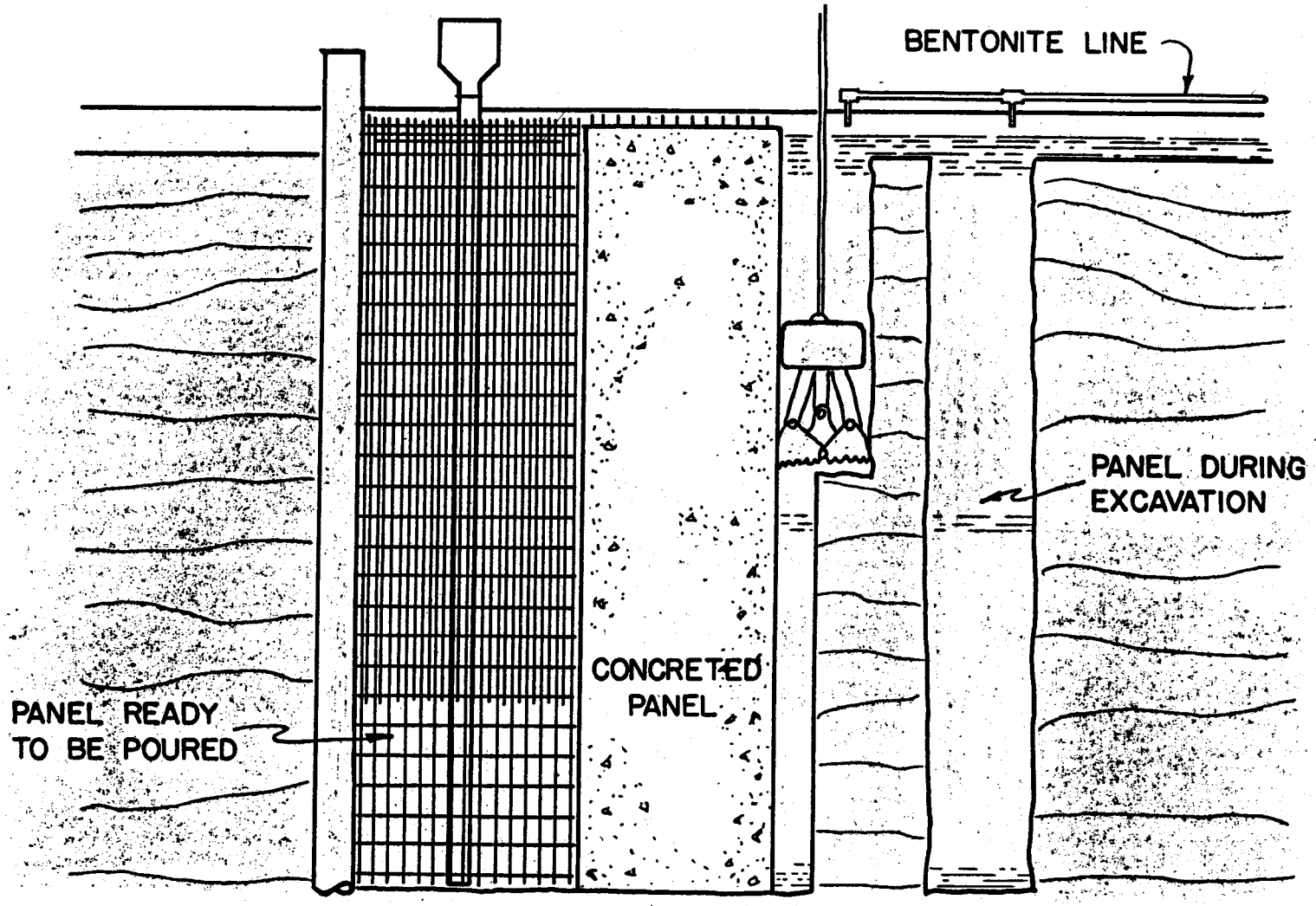


Figure 18. Different phases of construction.
(Courtesy of ICOS Corporation).

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 futher discussion, see Rogers (1963).

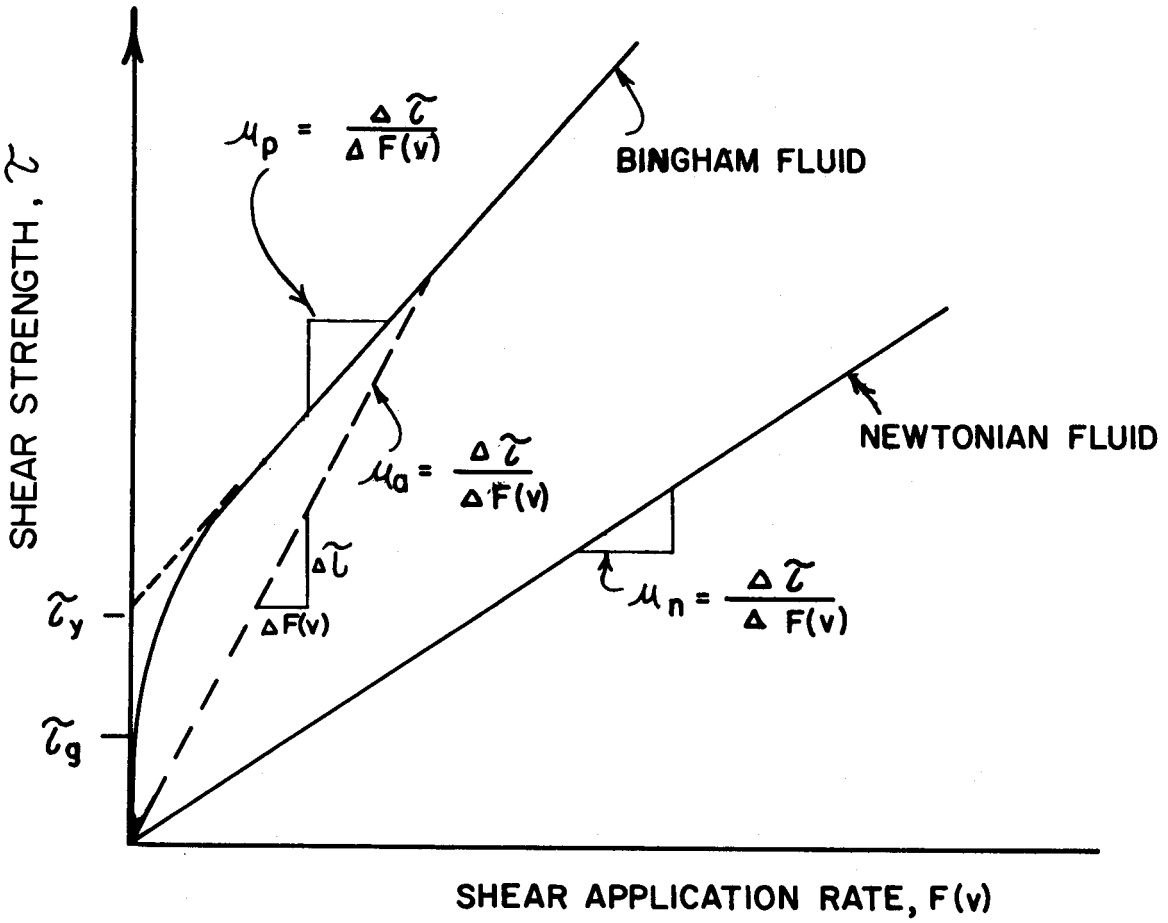
Viscosity and gel strength are used as indices for quality control testing of bentonite slurry. See Section 4.43.4 for further discussion.

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).



τ_g = Gel shear strength

τ_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:

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

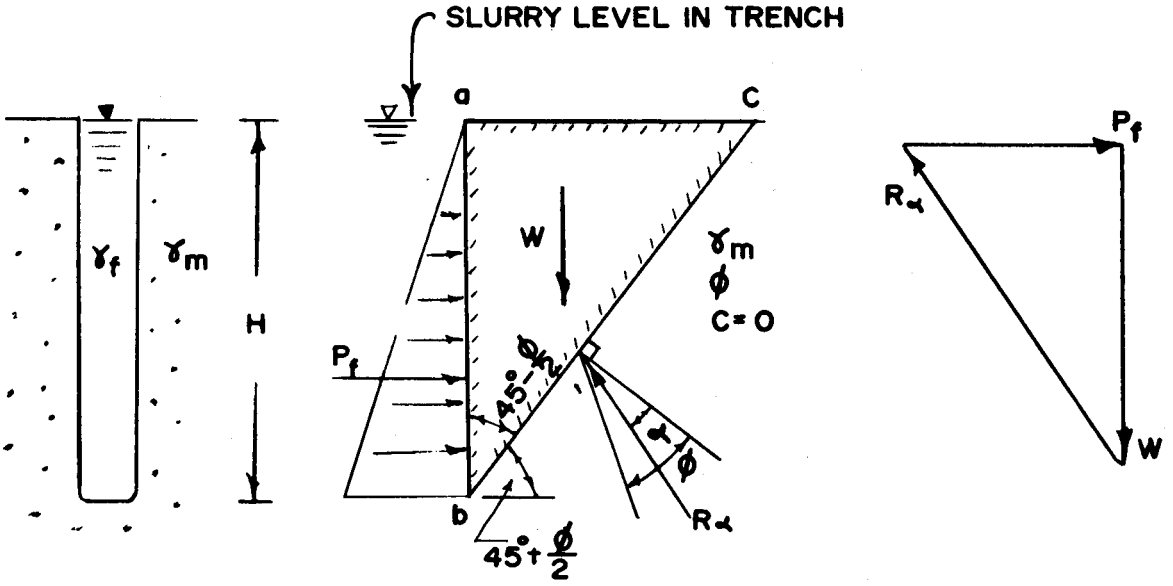


Figure 20. Force diagram for slurry trench in sand with ground water table below depth of trench.

P_f = Lateral force from pressure of slurry fluid = $1/2\,\gamma_f H^2$

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 ϕ 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.

	$\phi = 30^o$	$\phi = 35^o$
$F.S. = 2.8 \tan \phi$	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^o angle to the horizontal. The vector diagram shows the boundary forces in equilibrium with the wedge weight, W.

Forces are:

W = 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.

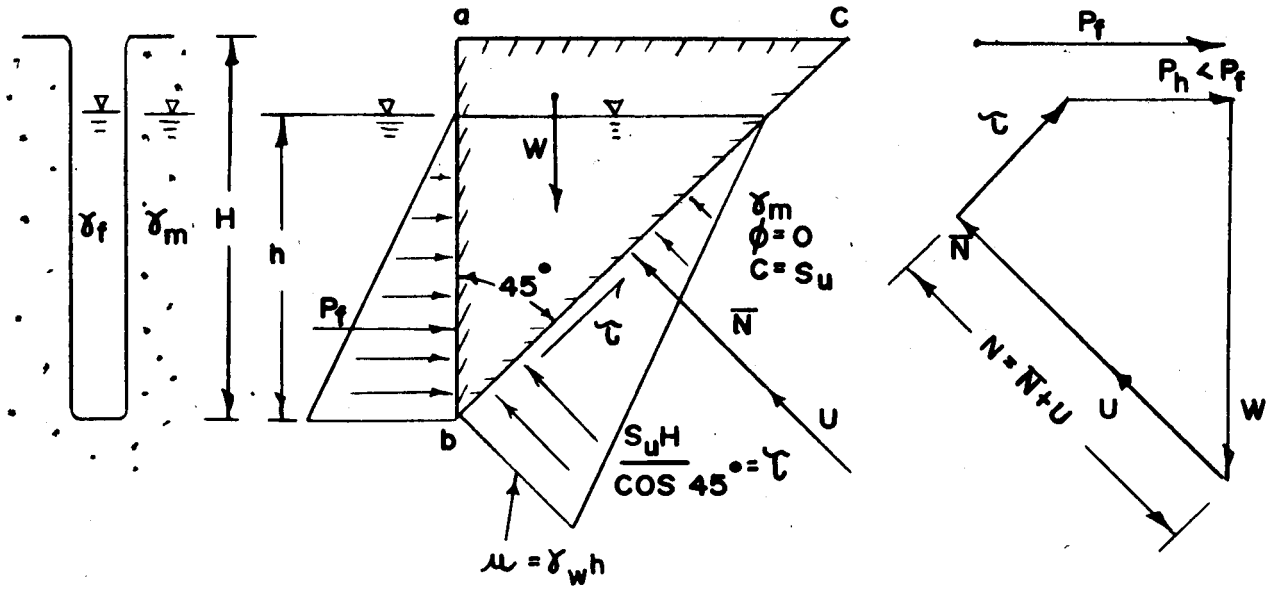


Figure 21. Force diagram for slurry trench in cohesive soil with water table and slurry at same level.

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

γ_m and γ_f = Unit weight of soil and slutty 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

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

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

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

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:

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

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

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

Trench Below Water Level in Cohesionless Soil

Morgenstern and Amir-Tahmasseb (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 slutty

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 tremch 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).

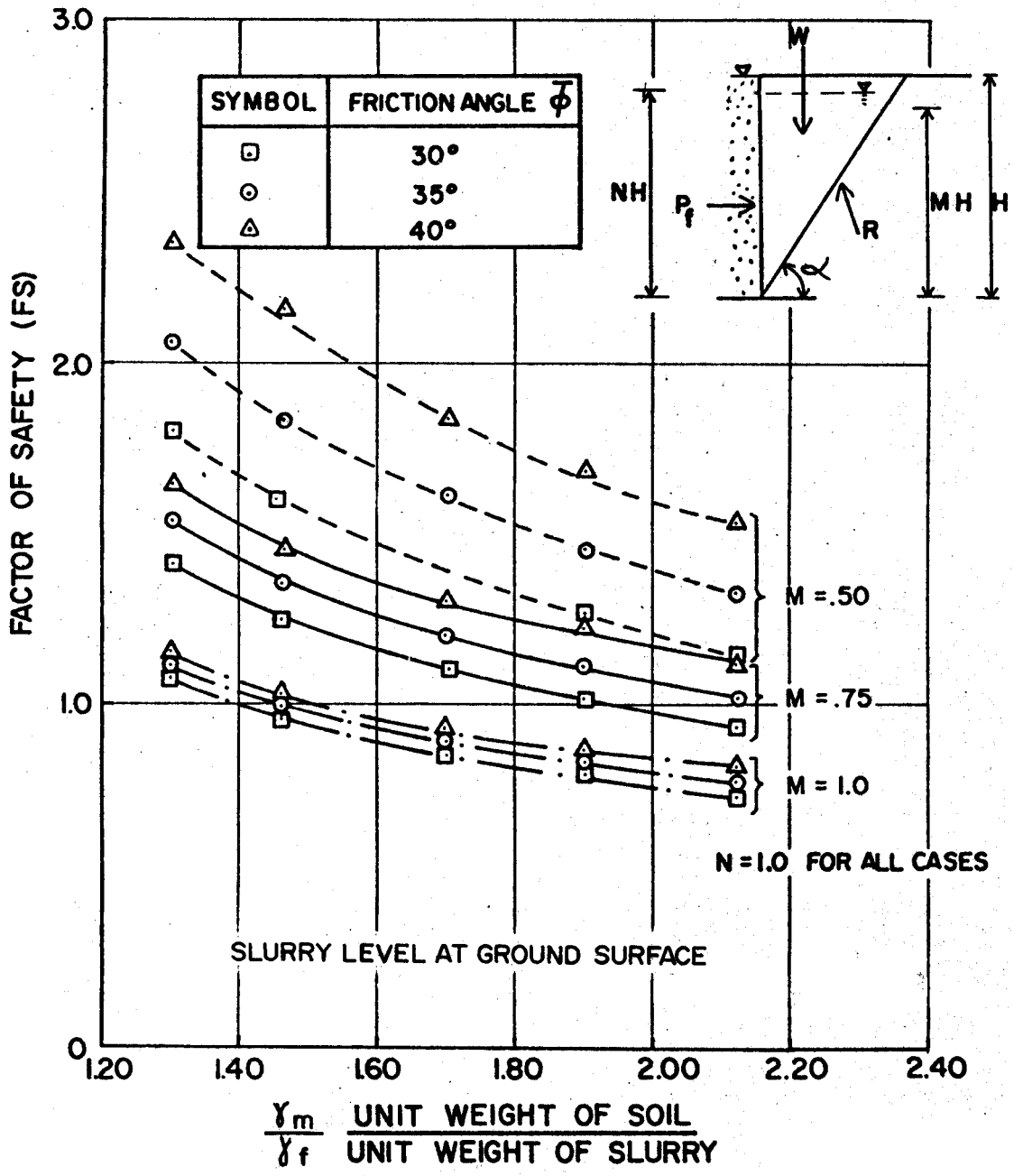


Figure 22. Stability of slurry trenches in cohesionless soils for plane strain conditions (from Morgenstern and Amir-Tahmasseb, 1965).

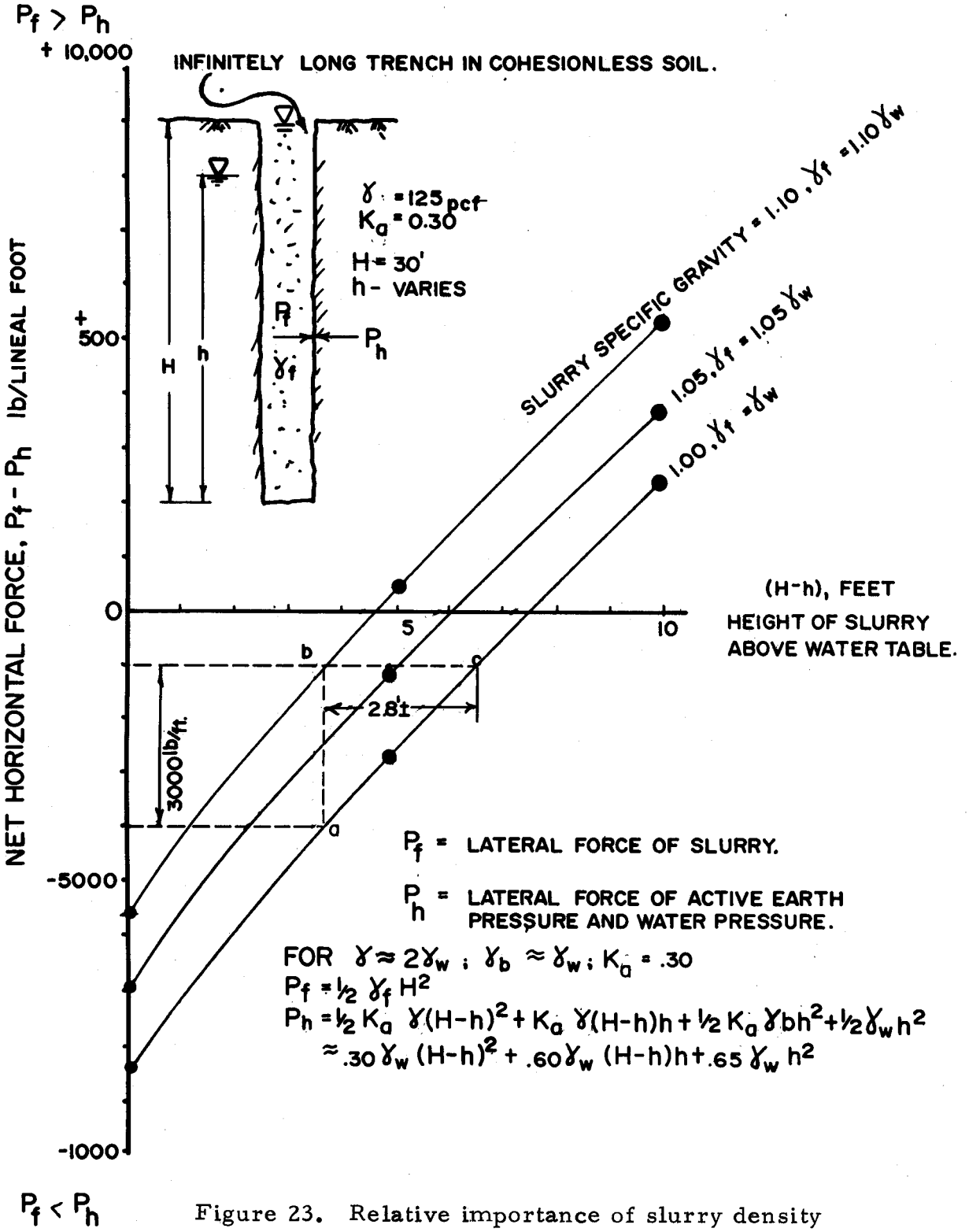


Figure 23. Relative importance of slurry density and slurry height for 30' deep trench.

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 charges 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 pressur 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 expecially 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 calcuim 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:

$$\tau = \tau_y + \mu_p F(\nu)$$

where:

τ = Total shear stress

τ_y = Yield shear stress (static intercept)

$\mu_p F(\nu)$ = Viscous shear stress (dynamic condition)

μ_p = Plastic viscosity

$F(\nu)$ = Function of shear application rate

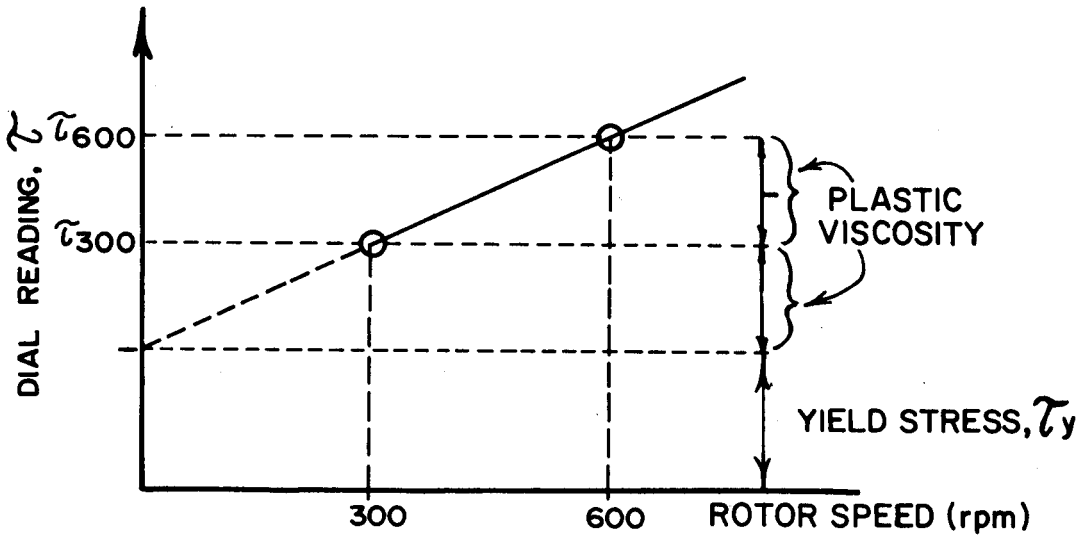
Fann Viscometer. The Fann viscometer, described in Appendix A, is used for measuring viscosity, yield shear strength, and gel strength. In this method, the slurry fills the annular space between a central circular core and an outer sleeve, and the device has a dial which enables one to measure the resistance 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 ft^2, 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/cm^2 to 0.20 g/cm^2 using the viscometer. The FPS specification (1973) requires 0.014 to 0.10 g/cm^2 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 denisty 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 at (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.

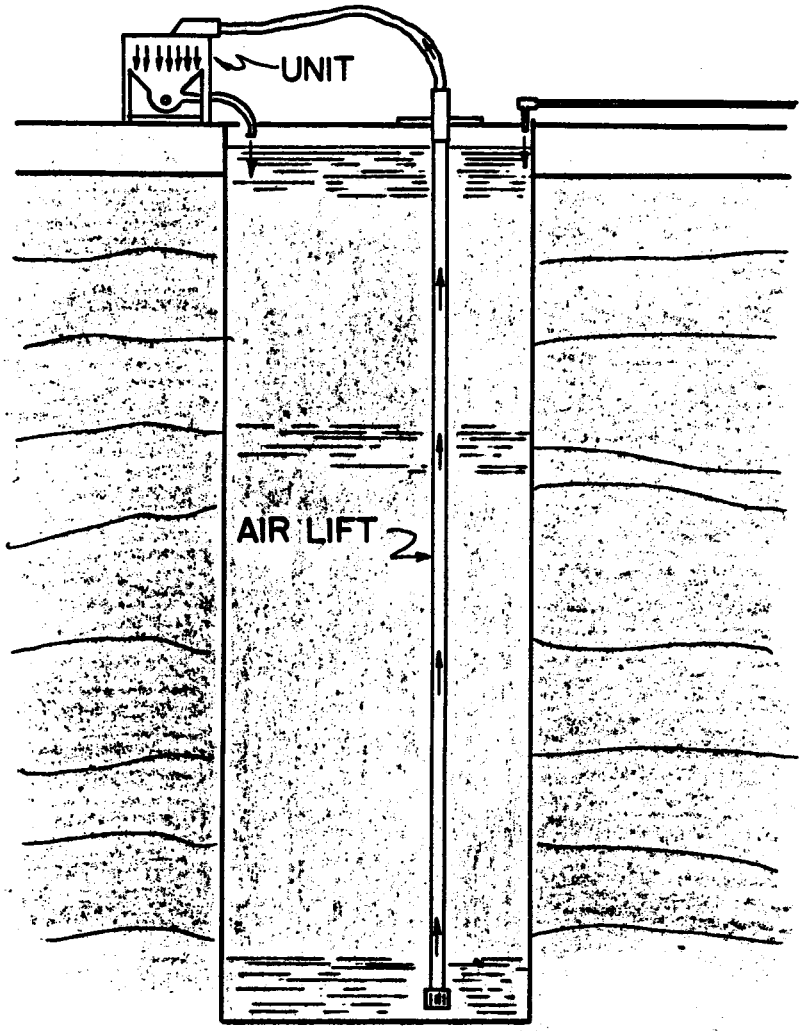
The more common method is first, to circulate the slurry over stationary or vibrating screens which remove the relatively coarse particles by mechanical process. Next, the slurry is circulated through centrifugal separators (hydrocyclones) which remove the sand. Finally, the slurry is discharged into 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.



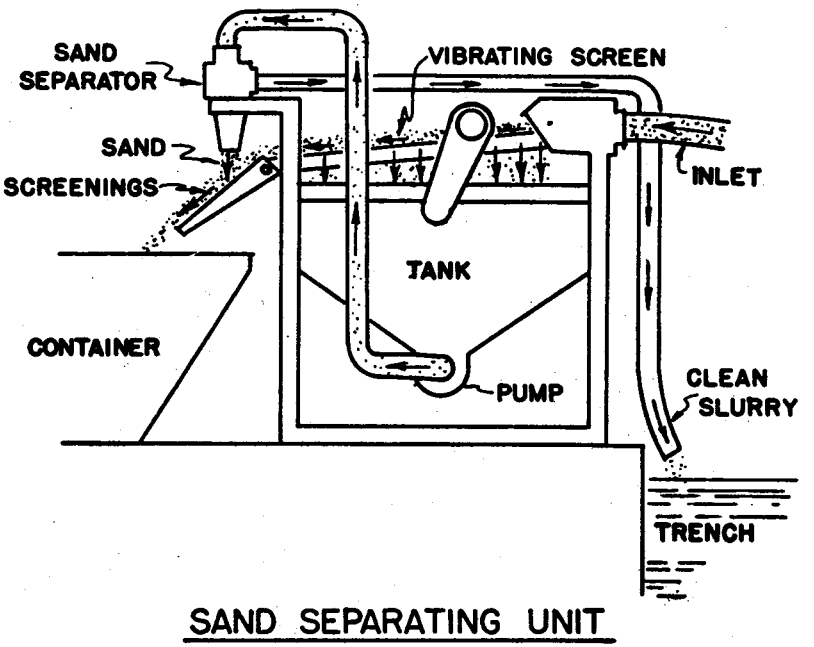
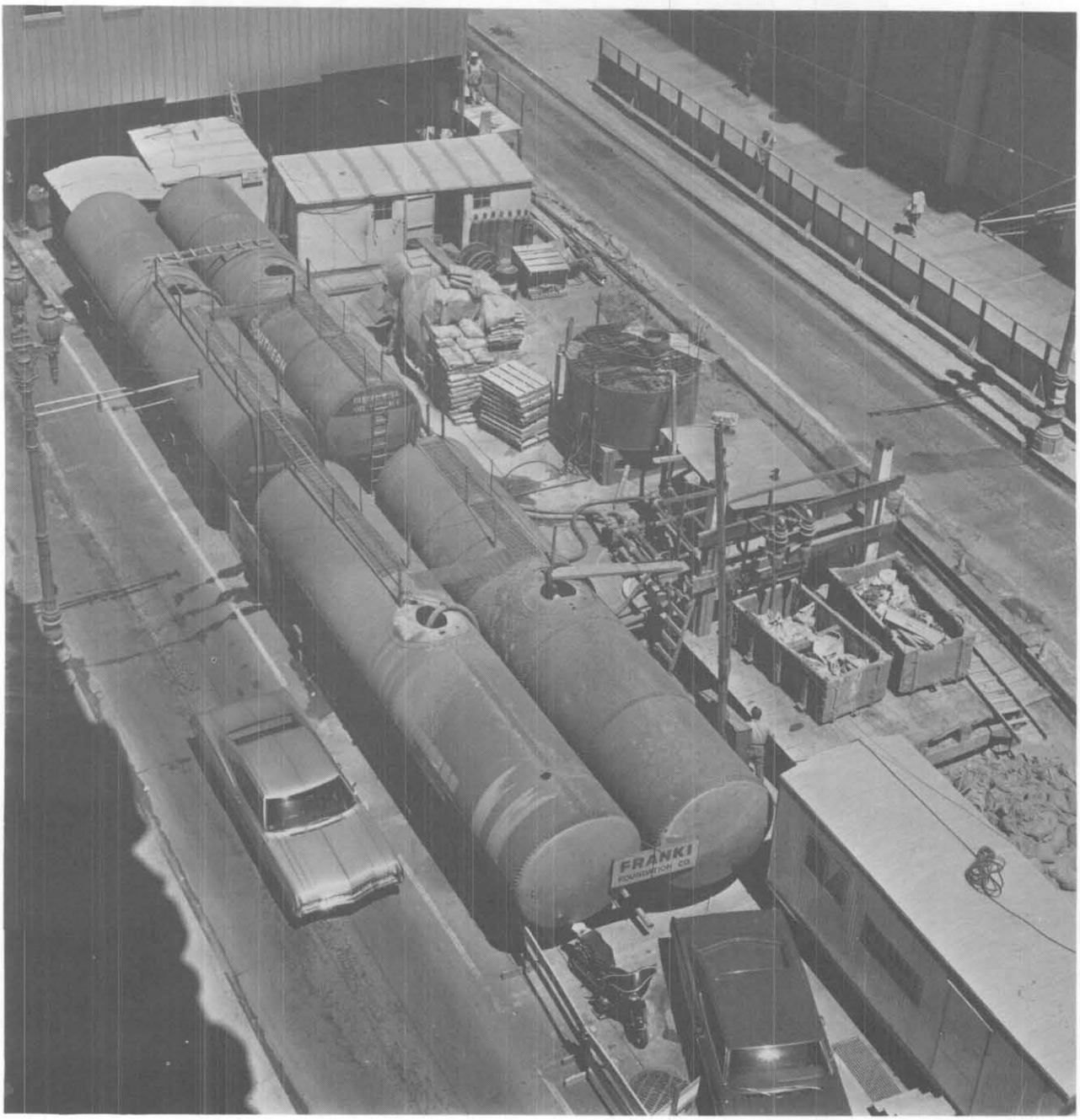


Figure 25. Clean-up with sand separator unit.
(Courtesy of ICOS Corporation).



Note: Storage in tank cars.

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

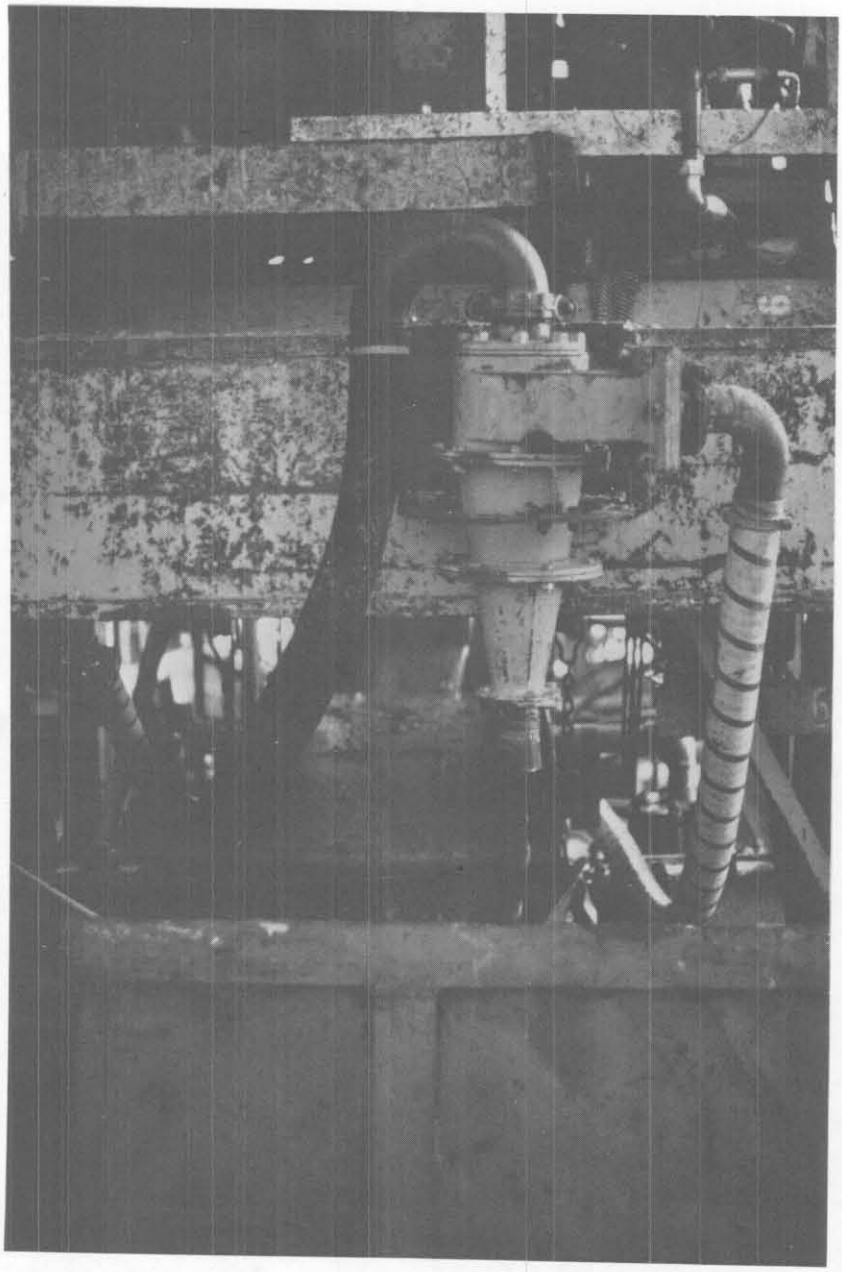


Figure 27. Hydrocyclone sand separator.
(Courtesy of ICOS Corporation).

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 per cent 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 at (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 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 precast 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

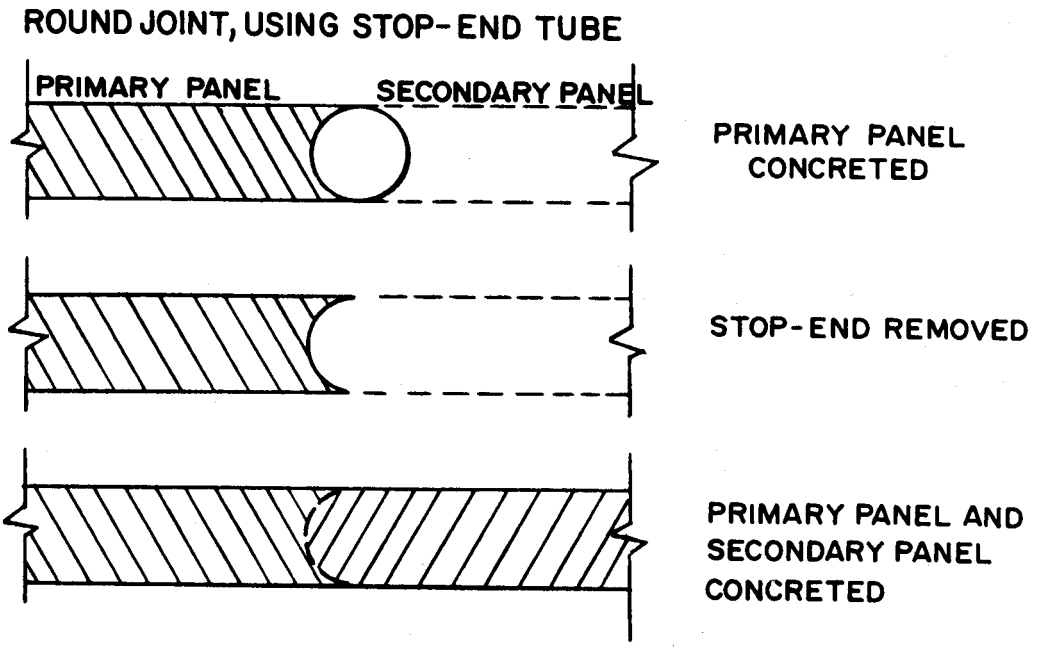


Figure 28. Panel joint with stop-end tube.

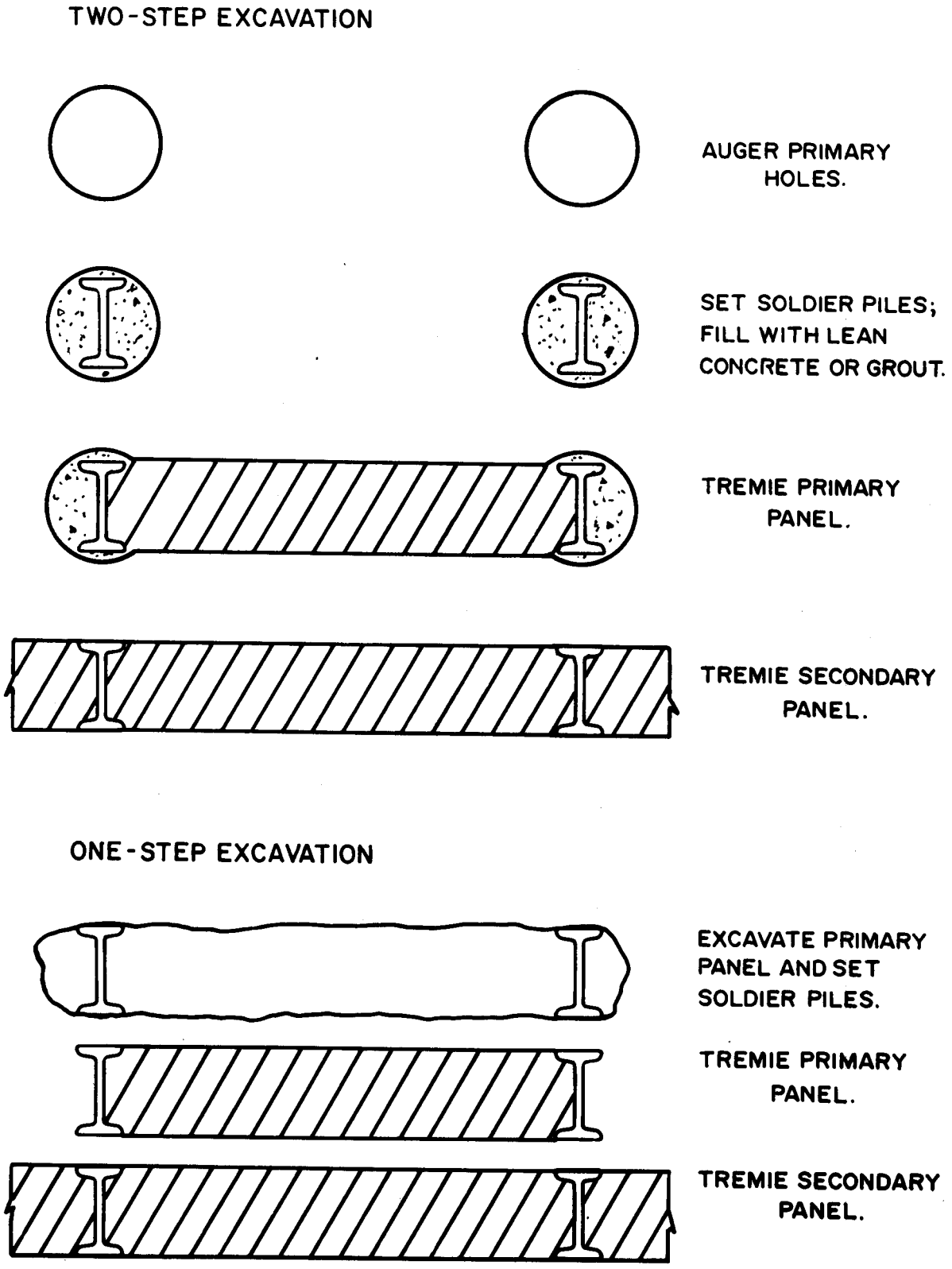


Figure 29. Panel joints with I-beams.

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.

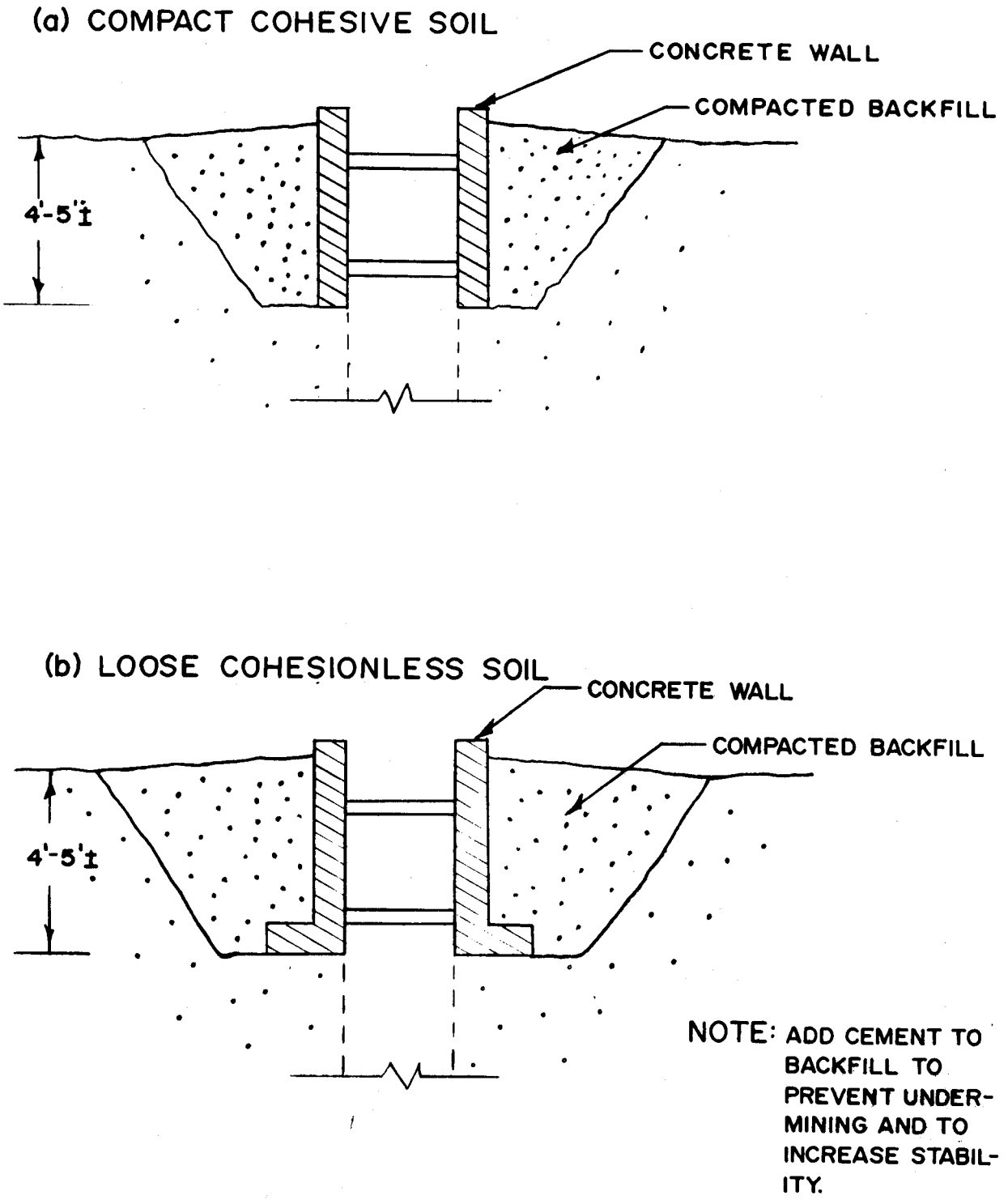


Figure 30. Guide walls.

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).

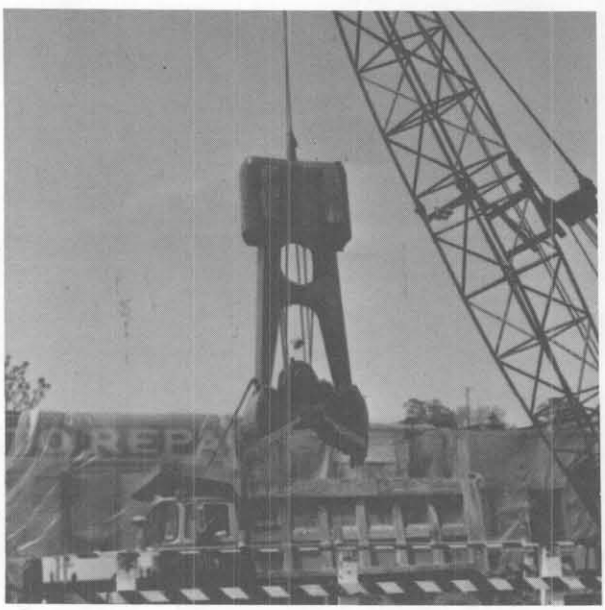


Note: Clam shell operates by cable mechanism.

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.

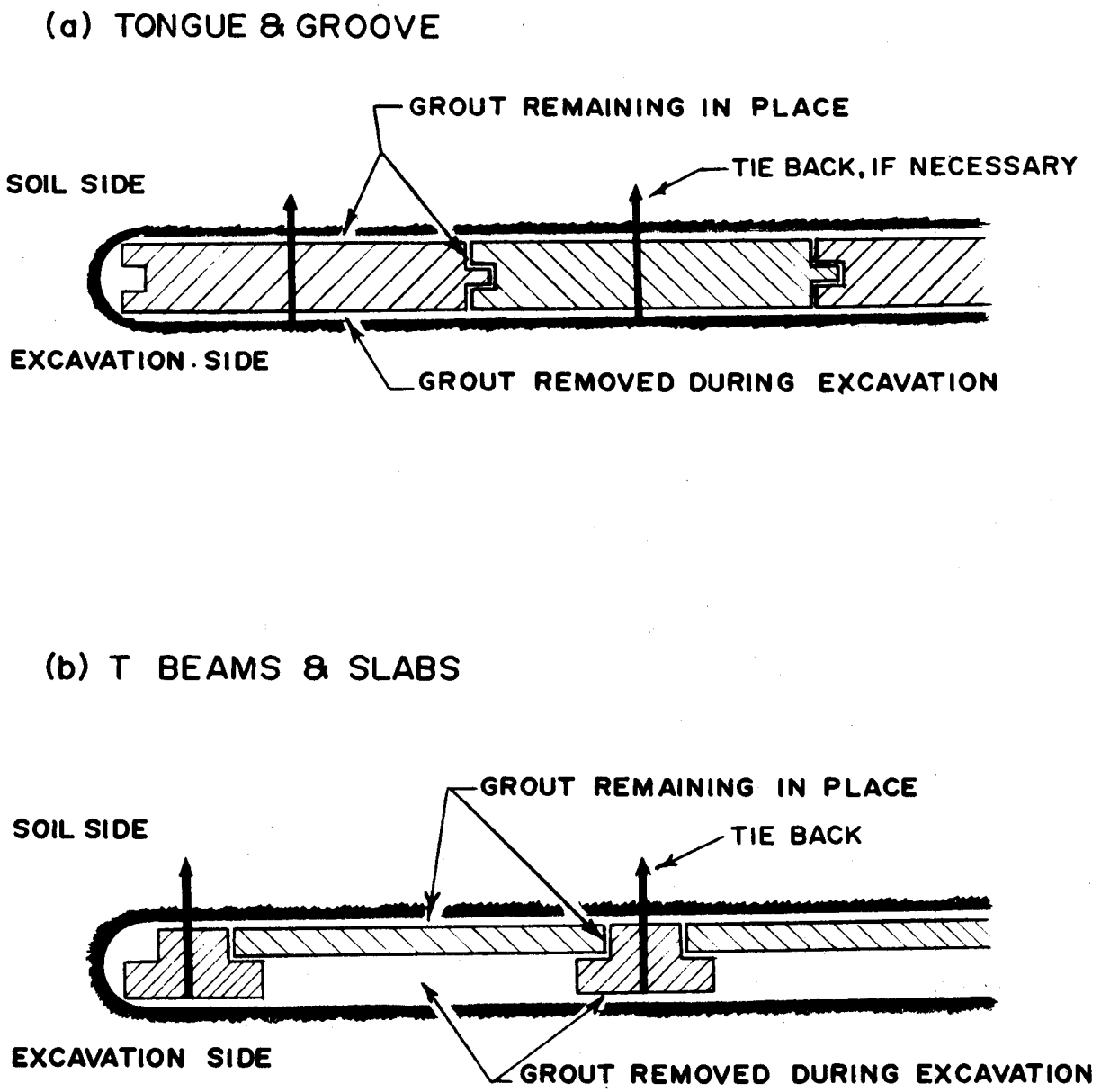
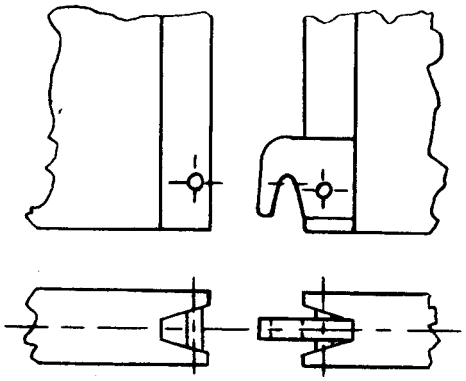
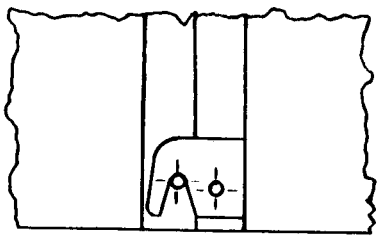


Figure 34. Panosol walls (Soletanche, France).



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.

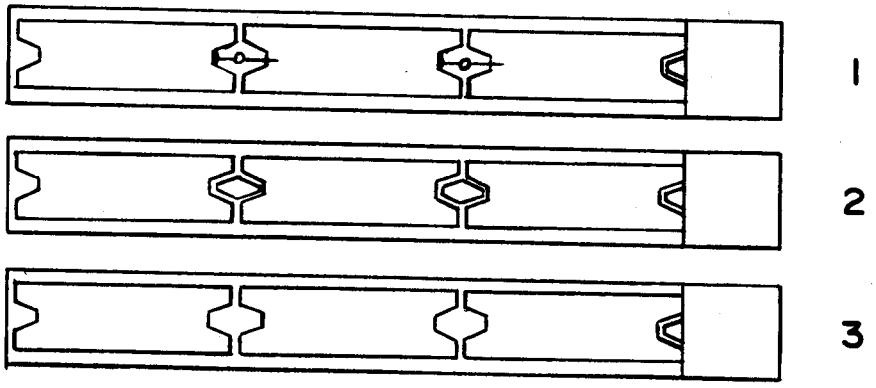


Figure 35. Prefasif wall (from Bachy Enterprise, France).

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.

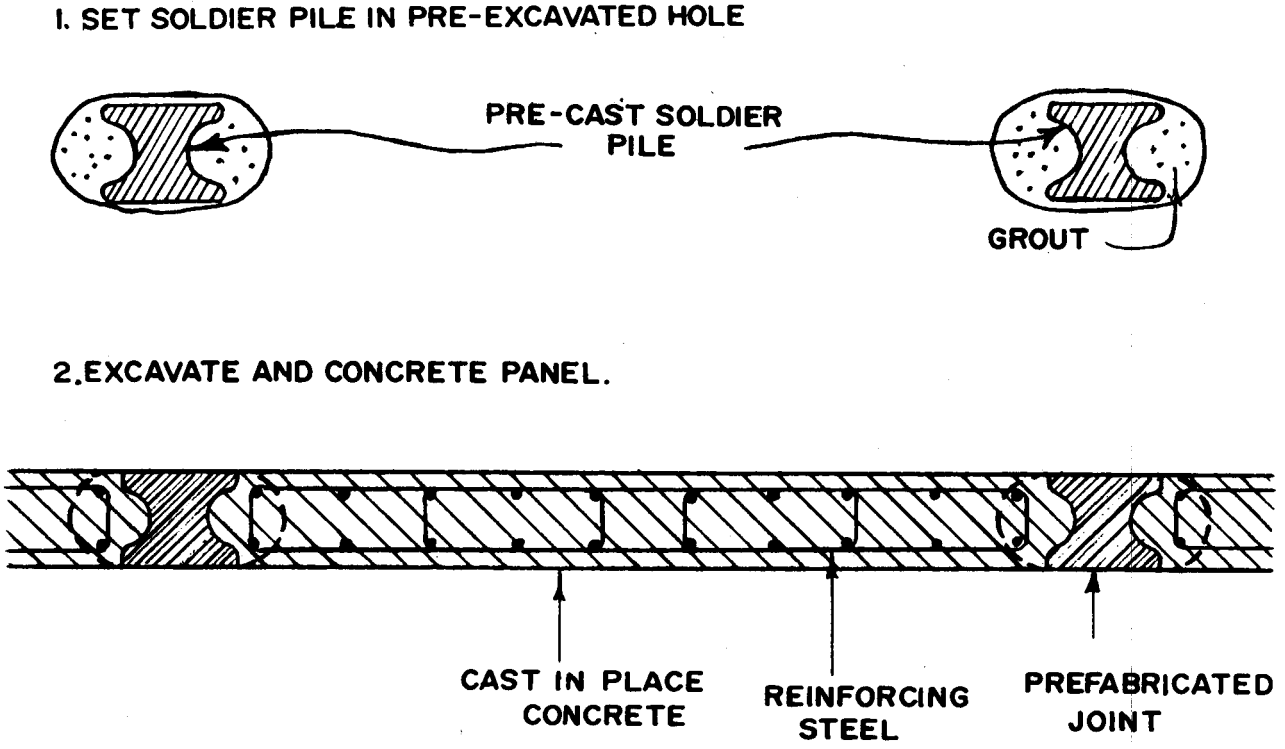


Figure 36. Two step excavation in slurry trench using precast soldier piles and tremie concrete.

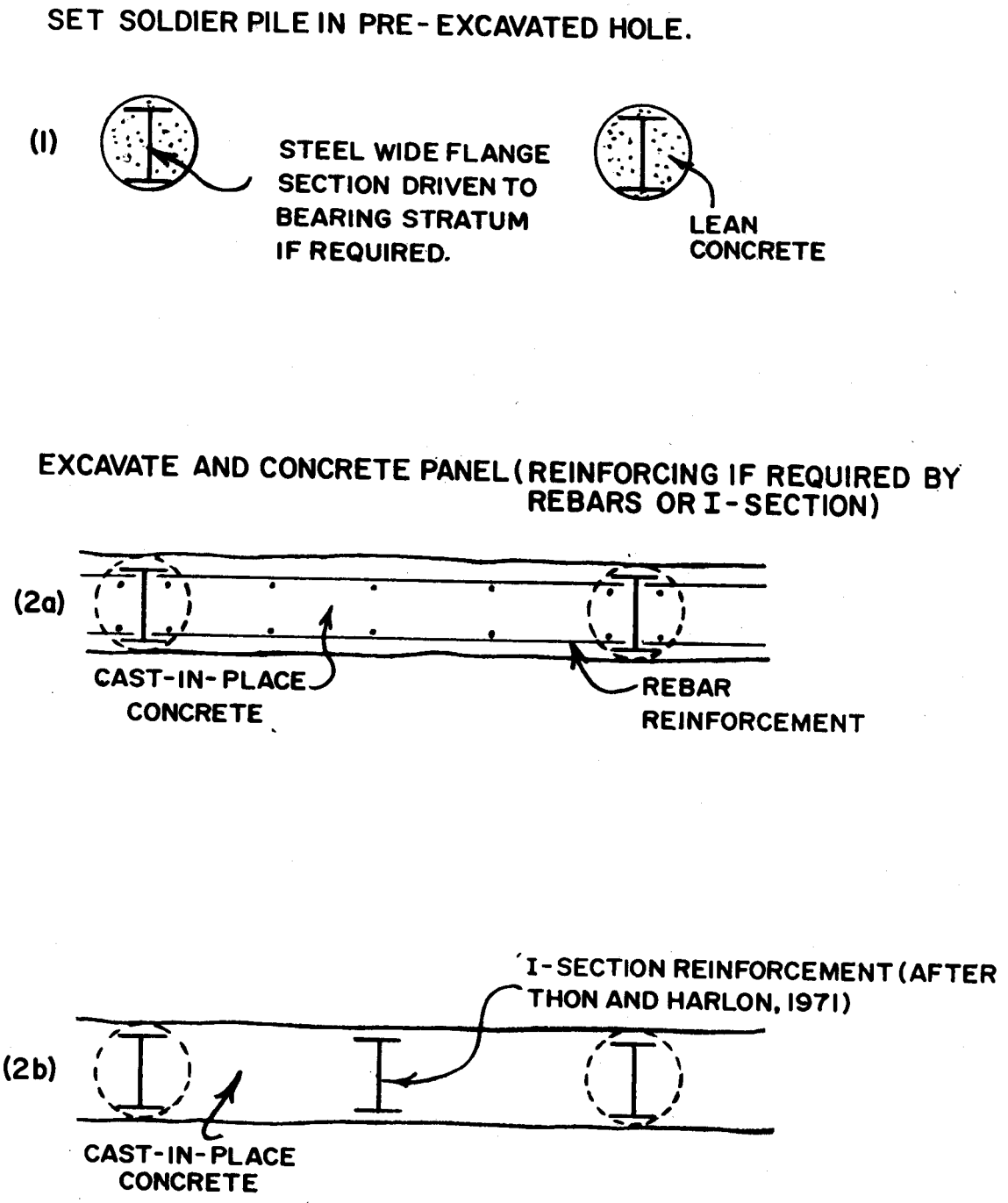


Figure 37. Two step excavation in slurry trench using steel wide flange soldier piles and tremie concrete.

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).

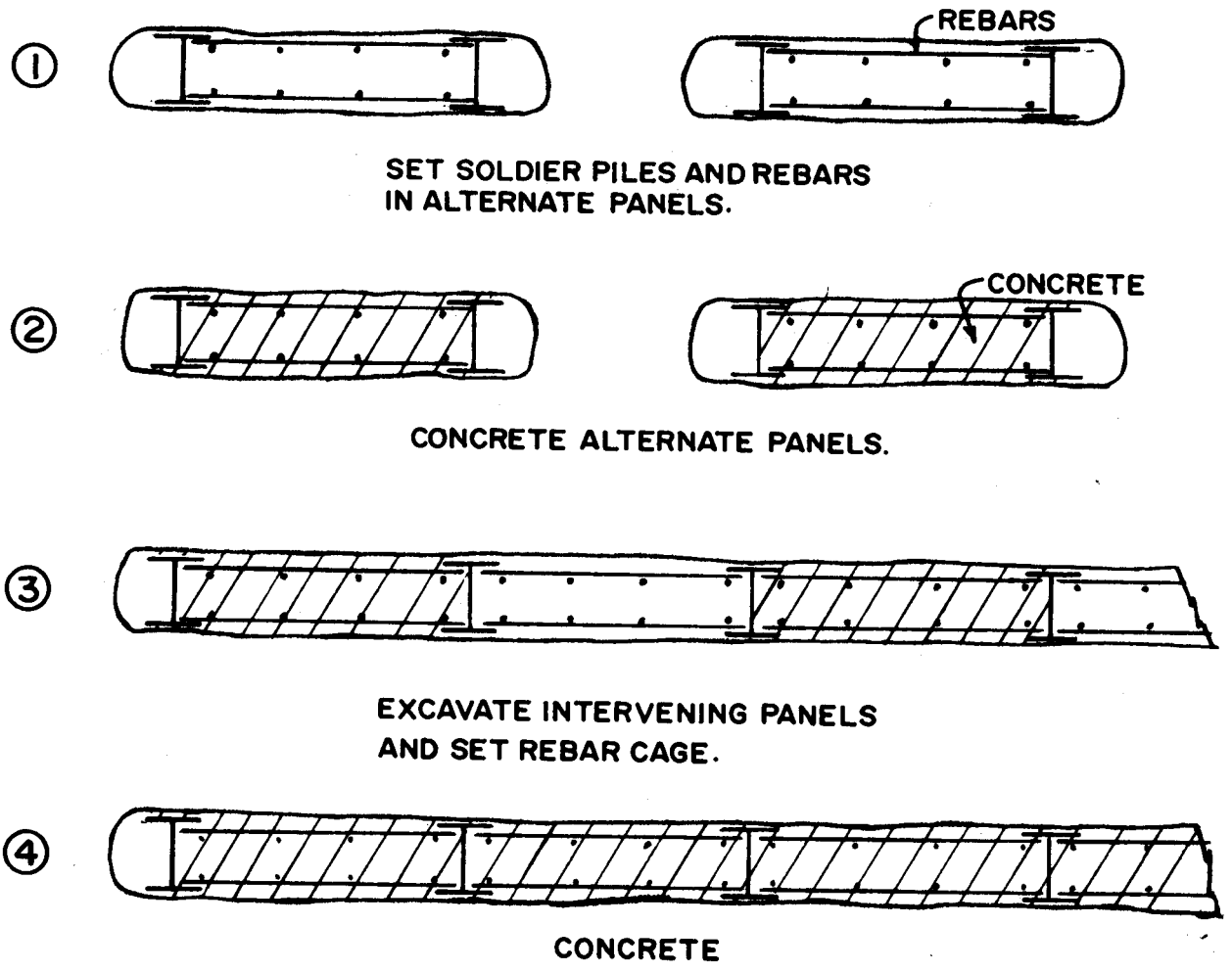
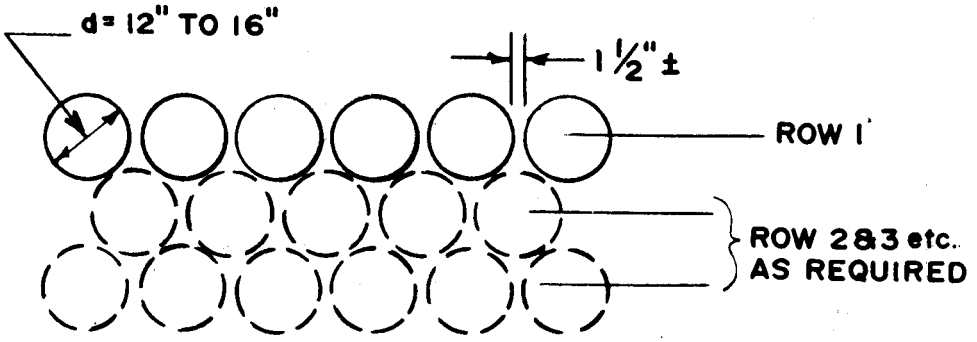


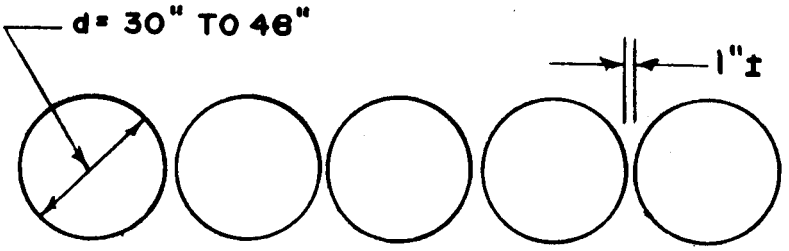
Figure 38. One step excavation with soldier piles (after Tamaro, 1974).

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

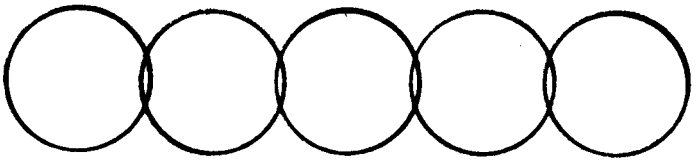


GROUT FILLED WITH HOLLOW STEM AUGER

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



AUGER RIG
(CONTIGUOUS)



ROTATING CASING
WITH CUTTING
EDGE.
(SECANT)

CONCRETE FILLED

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 Prepakt 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 cm³) 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 cm³).

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 cm³

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

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

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 cm³) 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
lb per gal	lb per cu ft	g per cm³ or specific gravity	Gradient, psi per 1,000 ft of depth	Gradient, kg per cm² 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

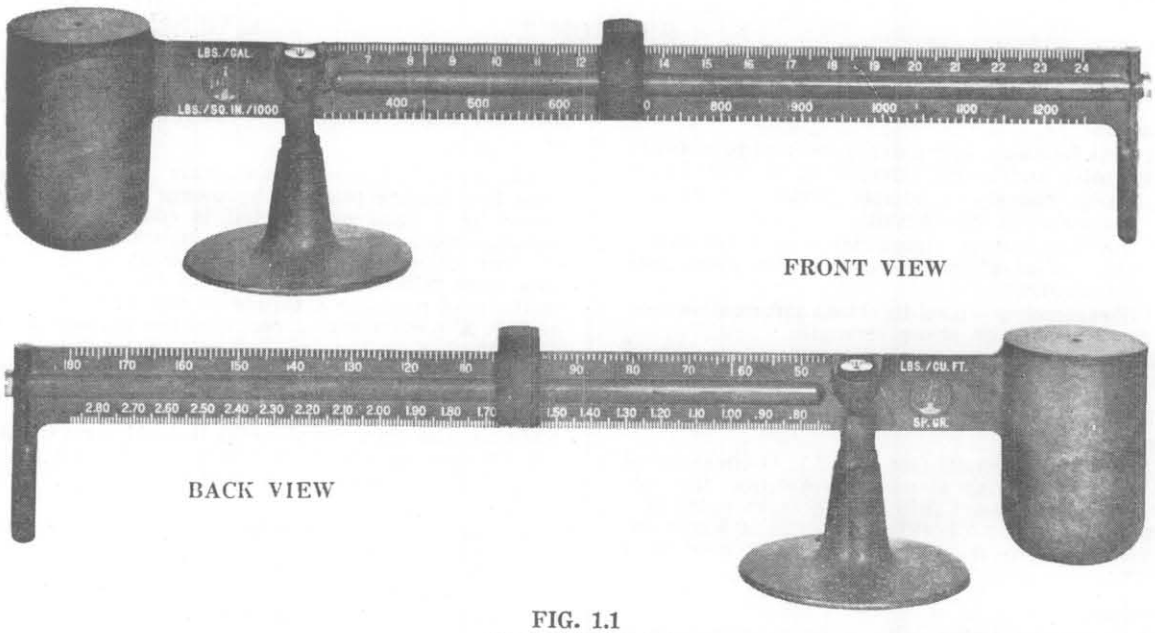


FIG. 1.1
MUD BALANCE

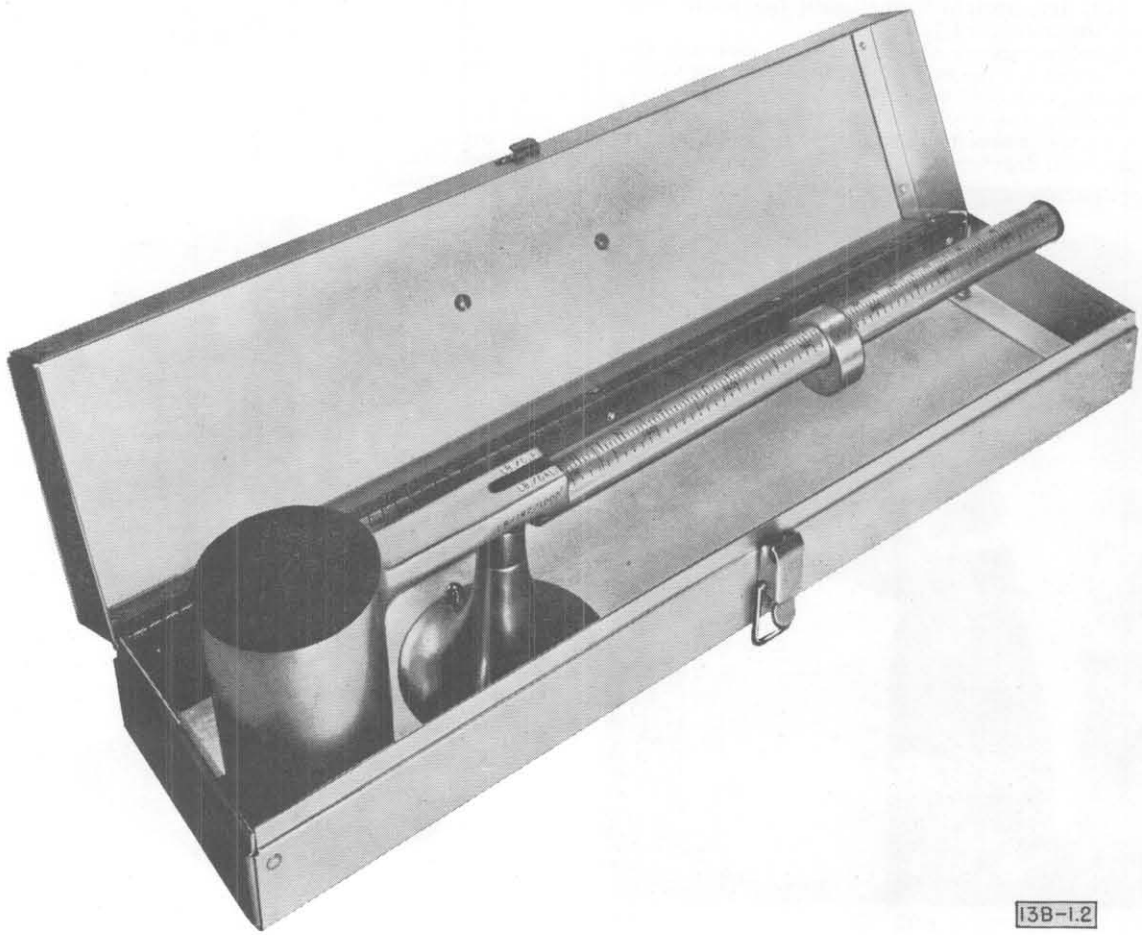


FIG. 1.2
MUD BALANCE

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 outflow 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 handwheel 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.

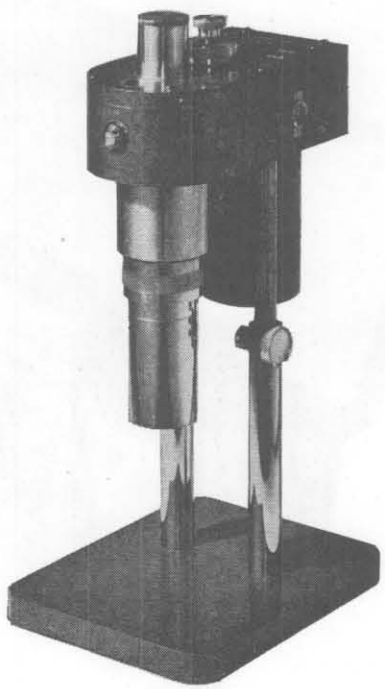


FIG. 2.2
12-VOLT MOTOR-DRIVEN VISCOMETER



FIG. 2.3
HAND-CRANK VISCOMETER

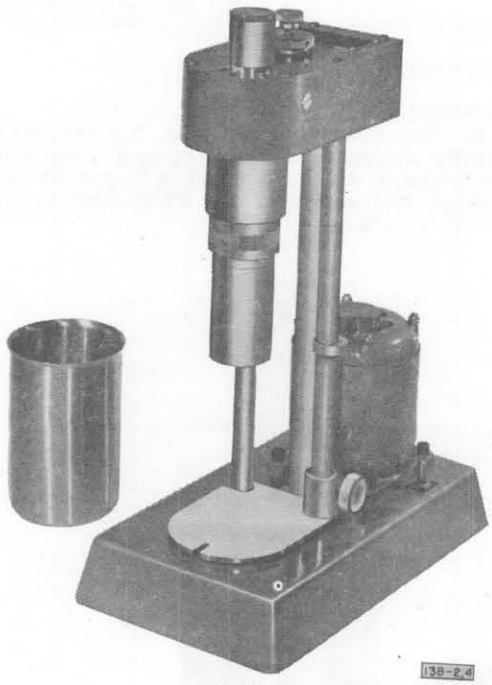


FIG. 2.4
115-VOLT MOTOR-DRIVEN VISCOMETER

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.

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.

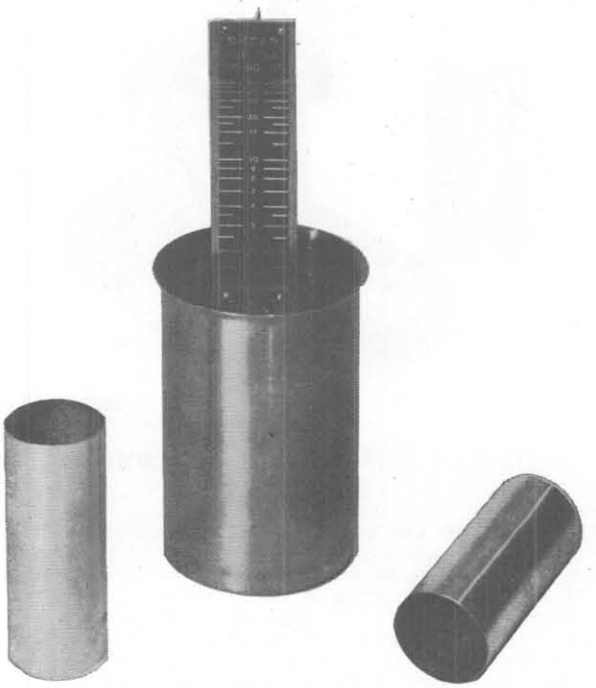


FIG. 2.5
SHEAROMETER SET

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

Length	12.0 in. (305 mm)
Diameter	6.0 in. (152 mm)
Capacity to bottom of screen	1,500 cm³

Orifice

Length	2.0 in. (50.8 mm)
Inside diameter	3/16 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 diameter	1.450 in. (36.83 mm)
Total length	3.425 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

Diameter	1.358 in. (34.49 mm)
Cylinder length	1.496 in. (38.00 mm)

Bob is closed with a flat base and tapered top.

Rotor Speeds

High speed	600 rpm
Low speed	300 rpm

c. Shearometer

Tube

Material	Duraluminum
Length	3.5 in. (89 mm)
Inside diameter	1.4 in. (36 mm)
Weight	5.0g

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.

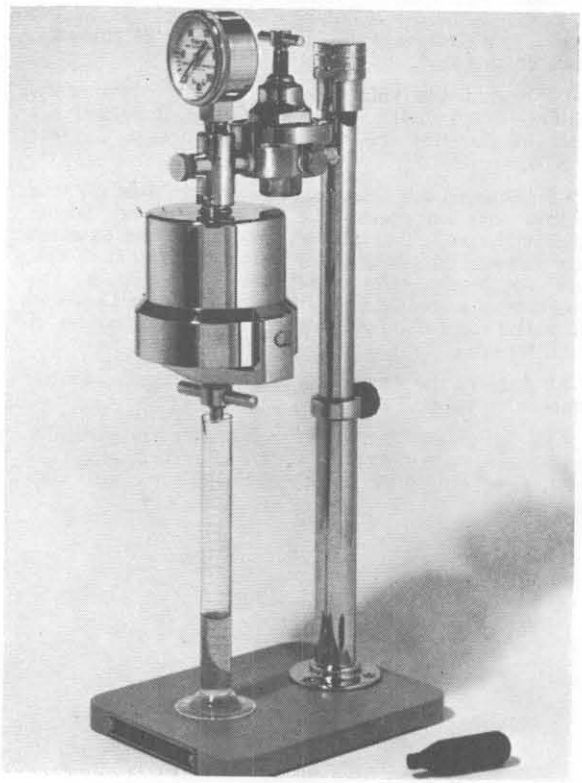
3.2 Pressure can be applied with any nonhazardous fluid medium, either gas or liquid. Presses are equipped with pressure regulators and can be obtained with portable pressure cylinders, midget pressure cartridges, or means for utilizing hydraulic pressure.

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.

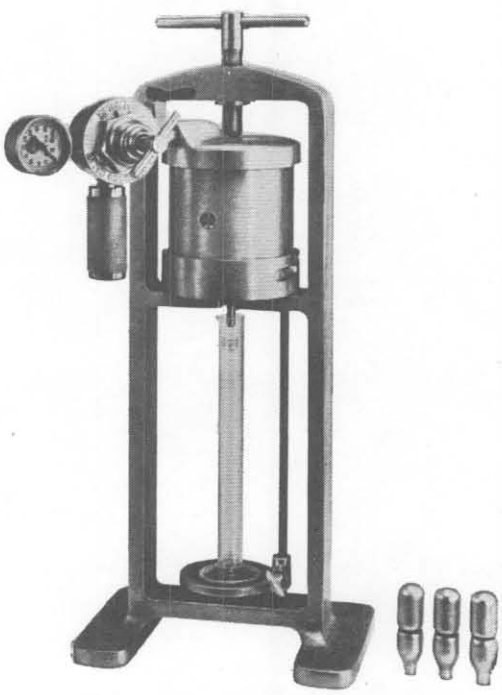


With CO_2 Cartridge Pressurization

FIG. 3.1
FILTER PRESS



With Nitrogen Pressurization



With CO_2 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 cm^3) 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 2½ 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.

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Reprinted from *Ground Engineering* Volume 6 No. 4 July 1973

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 torcement through the boxes, and the effect of the boxes on the to pass 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/m^3 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/m^2) 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 $\pm$ 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/m²	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 consistant 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.

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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.

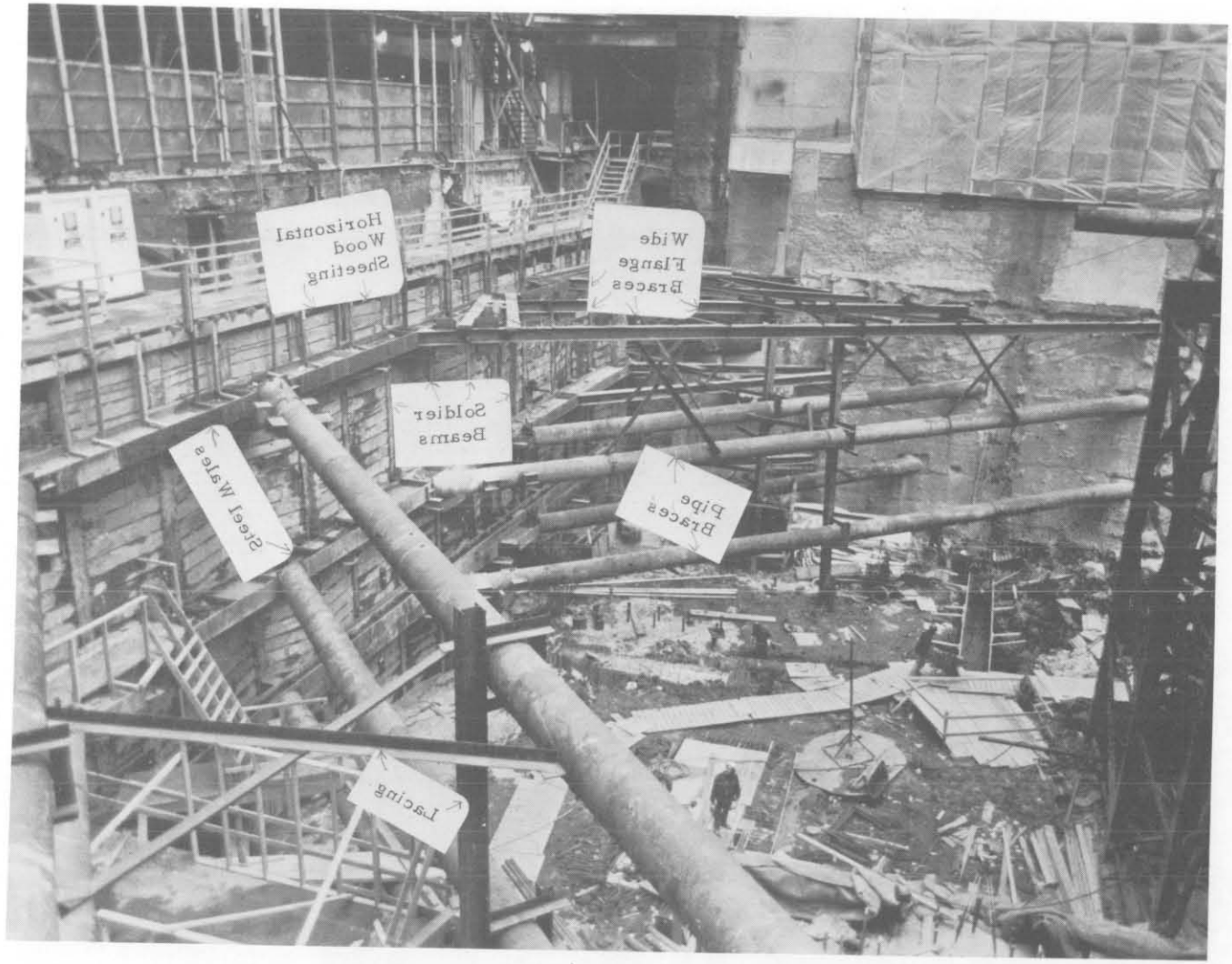
5.20 DESIGN CONSIDERATIONS

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 Prentis).



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 overconsolidated 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.

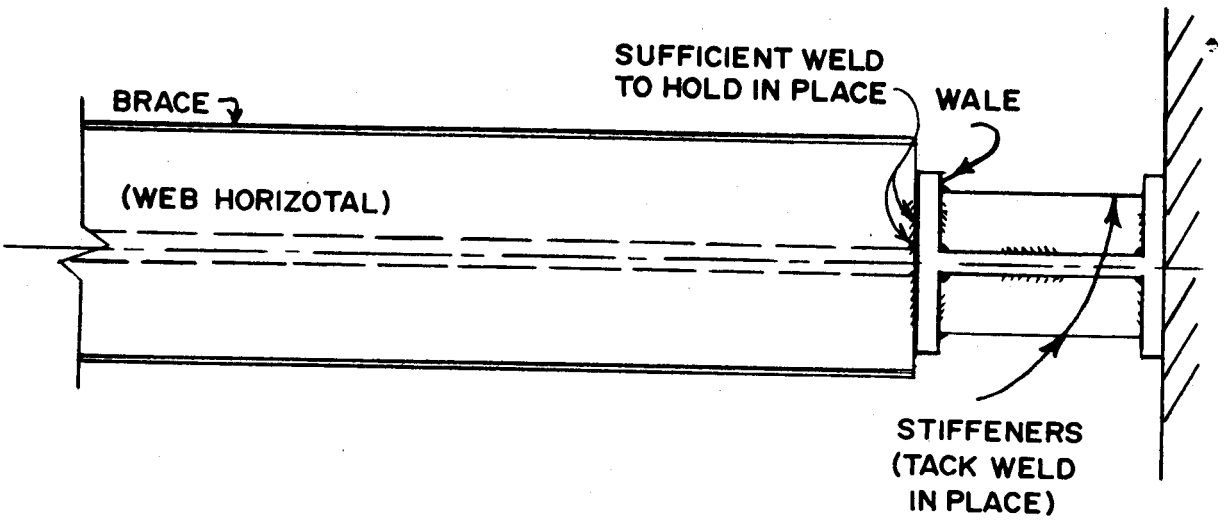


Figure 43. Typical detail for horizontal brace with brace web horizontal.

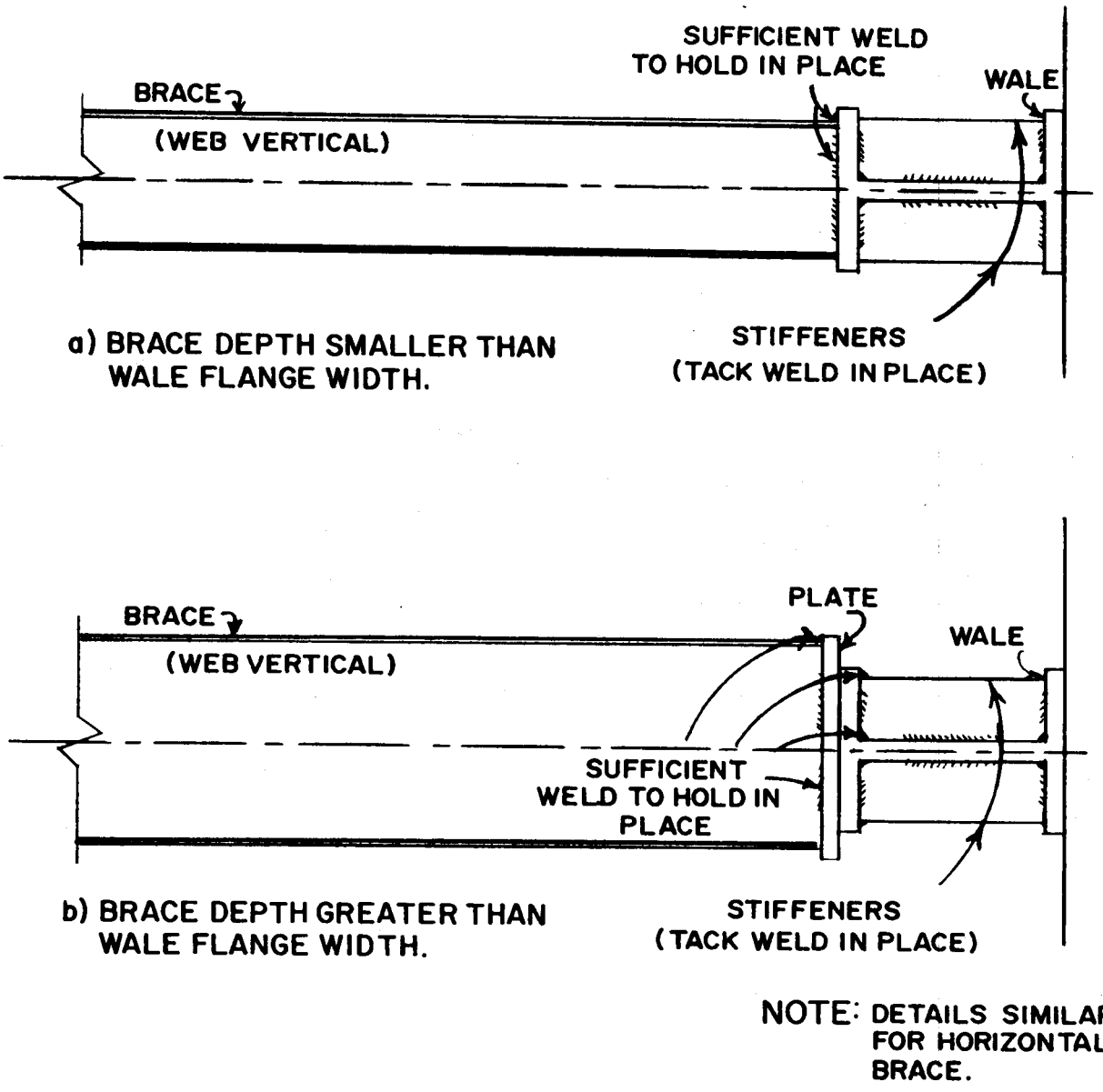


Figure 44. Typical detail for horizontal brace with brace web vertical.

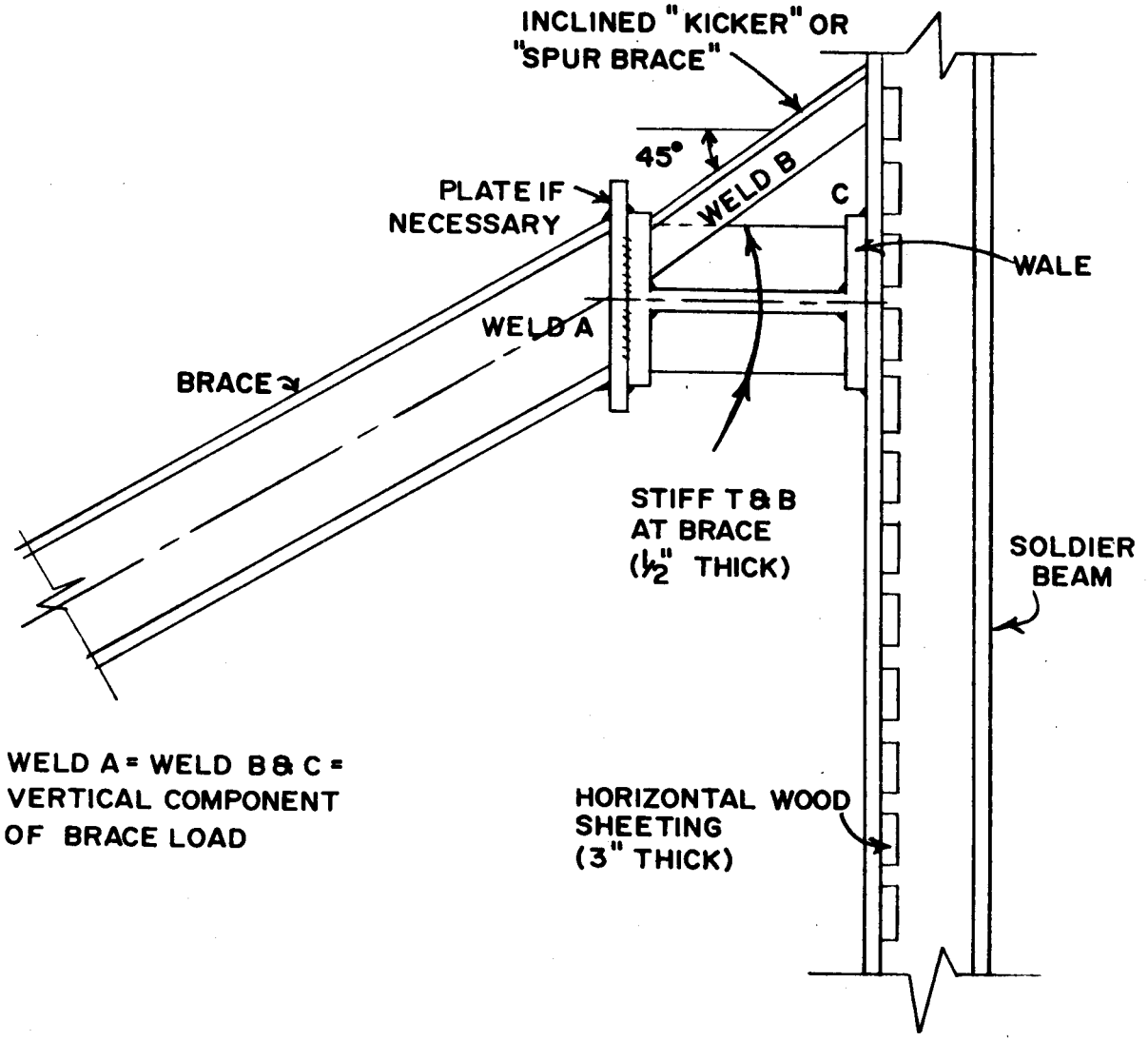


Figure 45. Typical connection for inclined brace and horizontal wale.

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 unforseen 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° 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:

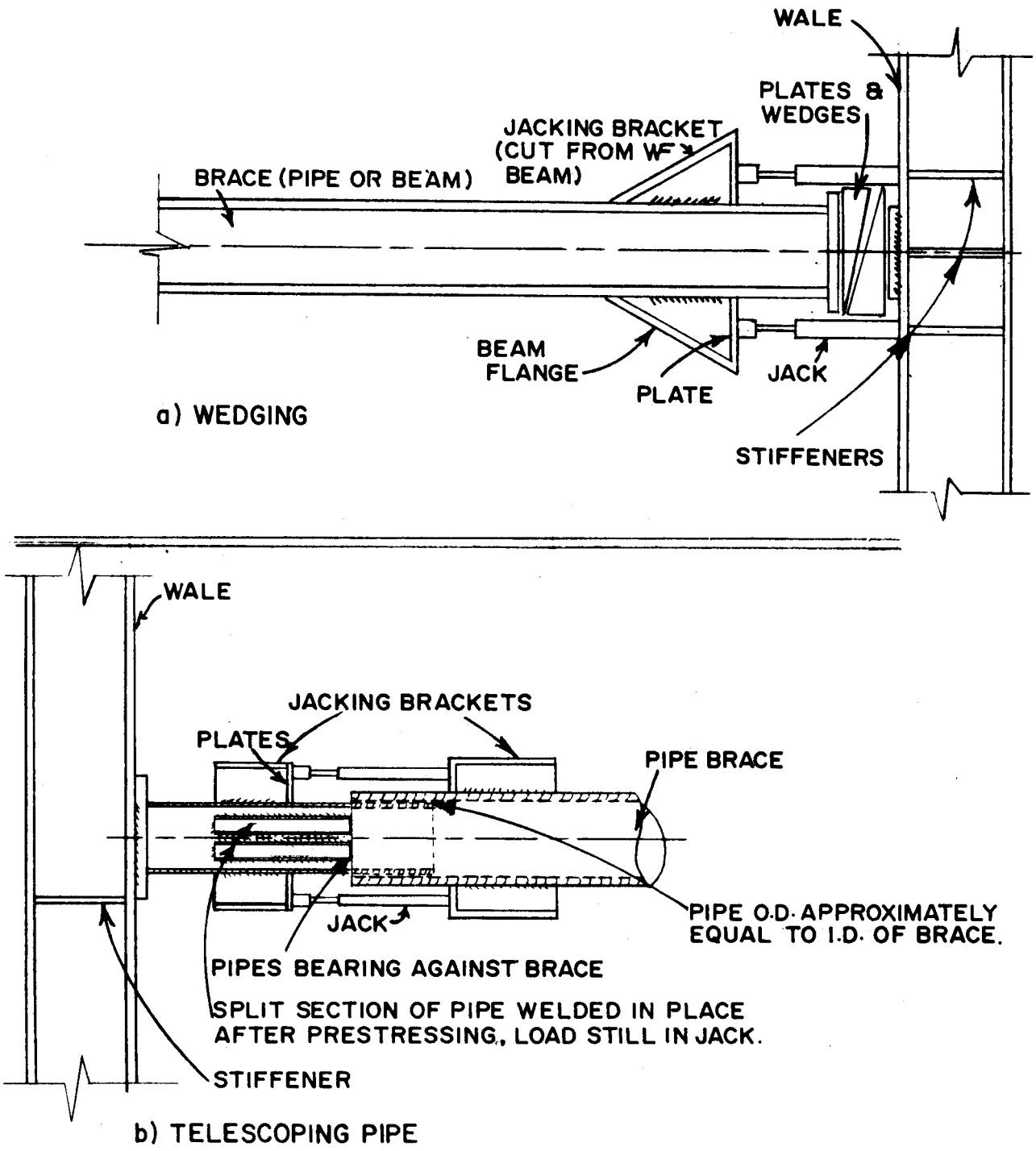


Figure 46. Prestressing details for braces.

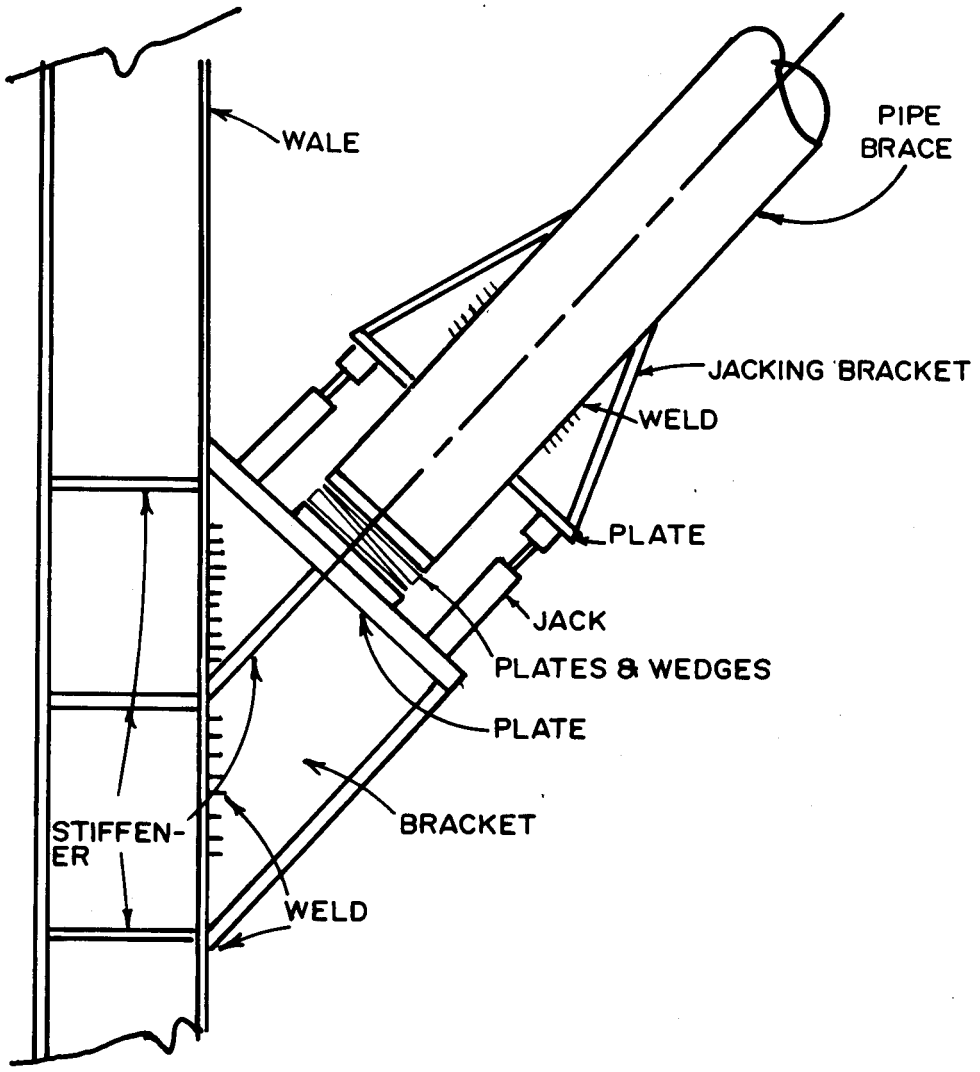


Figure 47. Prestressing of pipe brace at corners using brackets as reaction.

$$\Delta P = A_s E_s (\alpha \times \quad \Delta^{\circ}F)$$

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

$$\frac{\Delta P}{A_s} = \Delta\sigma = 30,000 \times 6.5 \times 10^{-6} \times 40^{0} = 7.8 \text{ kips/in}^2$$

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.

Case	Decked or Open	Load Variation
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% ± of measured average
4. Armento (1972)	Covered	10% ± of measured average

5.43 Design and Construction Criteria

A generalized expression for predicting strut load variation was developed by Chapman, et al (1972):

$$P = A_s E_s (6.5 \times 10^{-6}) \left[\Delta^{o}F \times \frac{1}{1 + \left(\frac{3 \cdot n \cdot A_s \cdot E_s \cdot H}{A_{cut} E_d L} \right)} \right]$$

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.

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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.

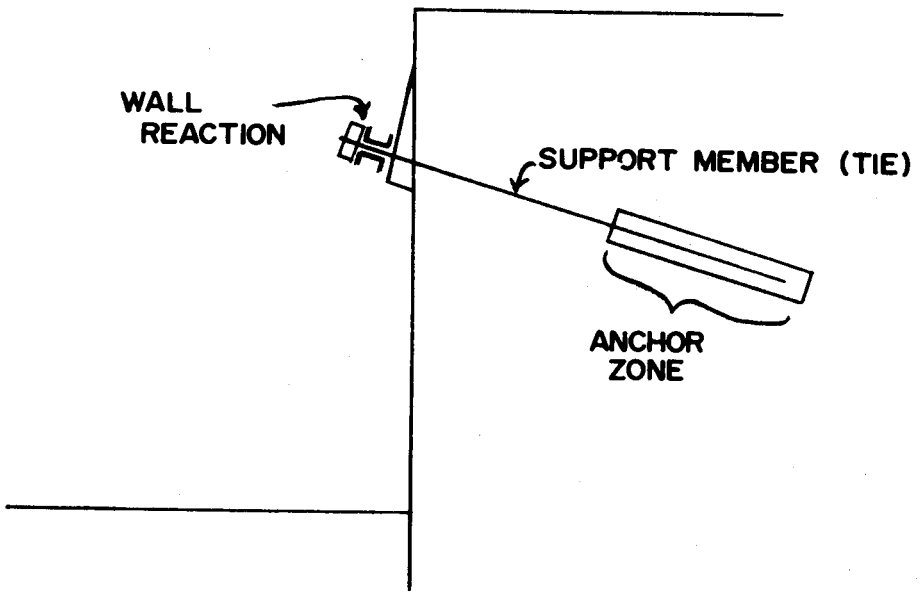


Figure 48. Major tieback components.

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 extemely 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.

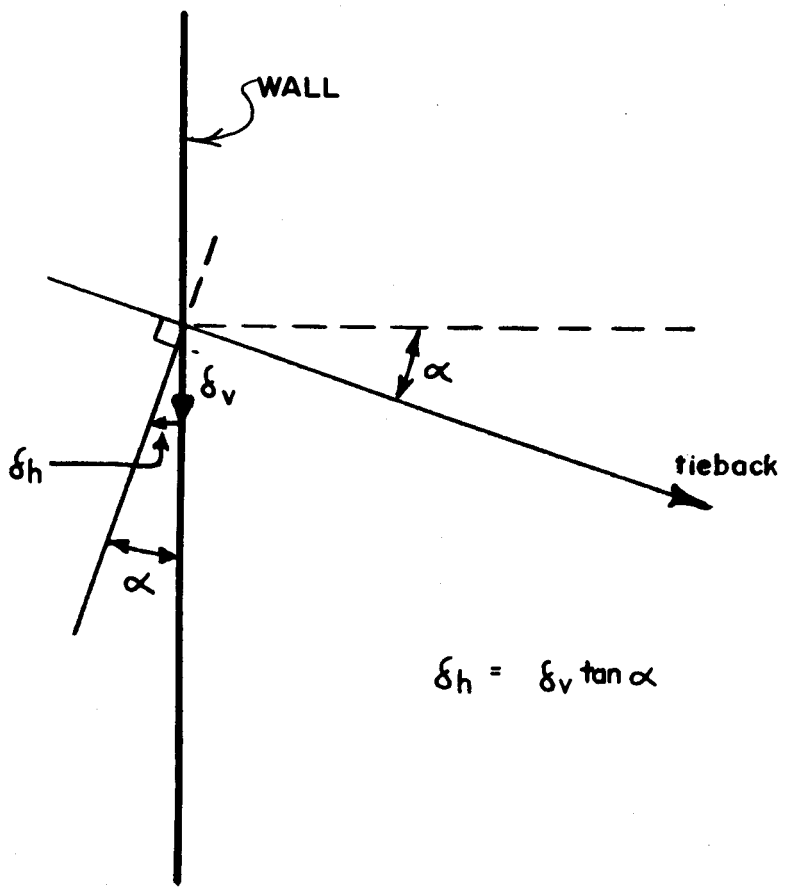


Figure 49. Horizontal deflection resulting from wall settlement.

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 procedes. 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

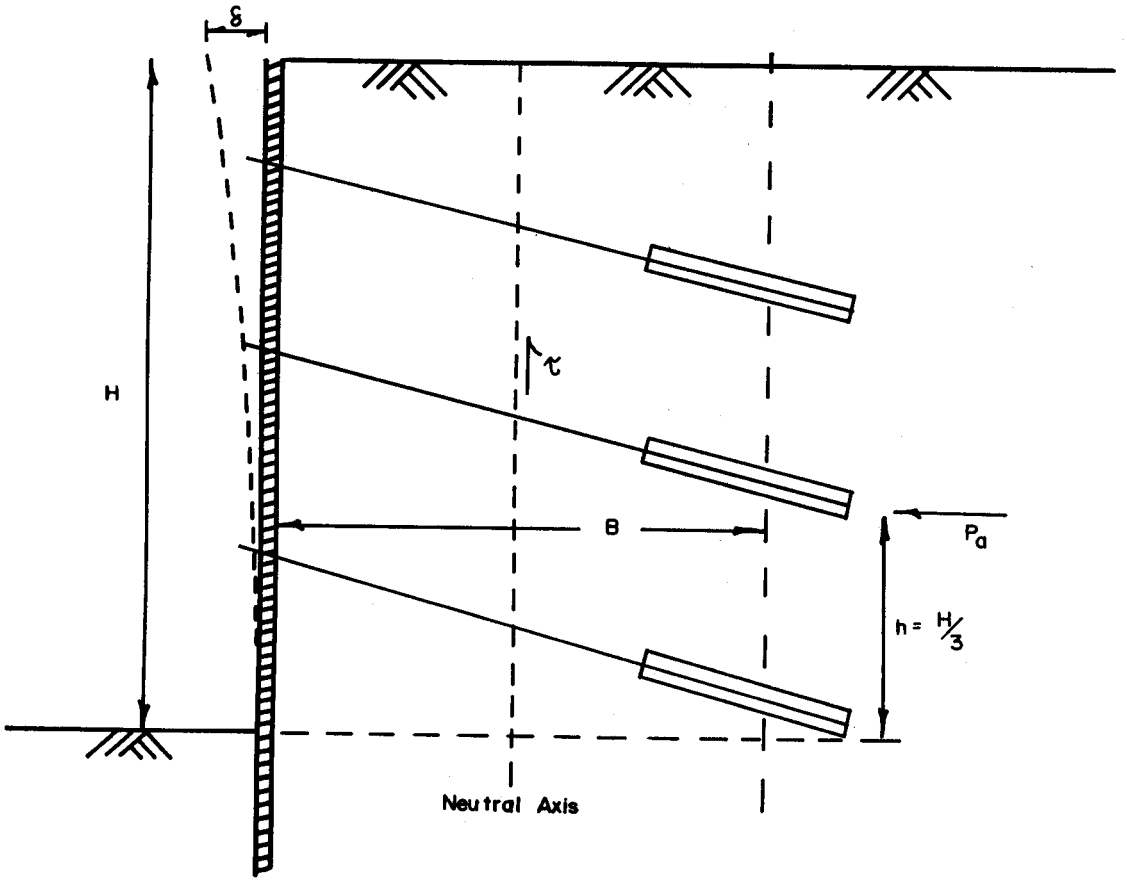


Figure 50. Sketch of equivalent cofferdam for tied-back wall.

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 overconsolidated 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.

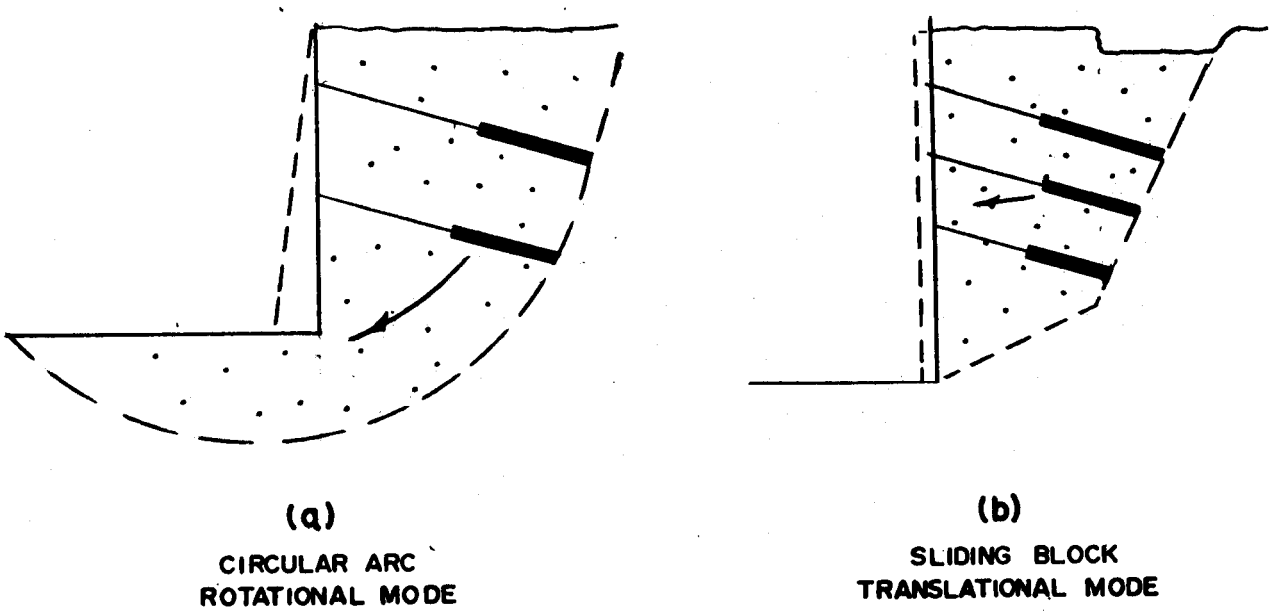


Figure 51. Possible stability failure modes for tied-back walls.

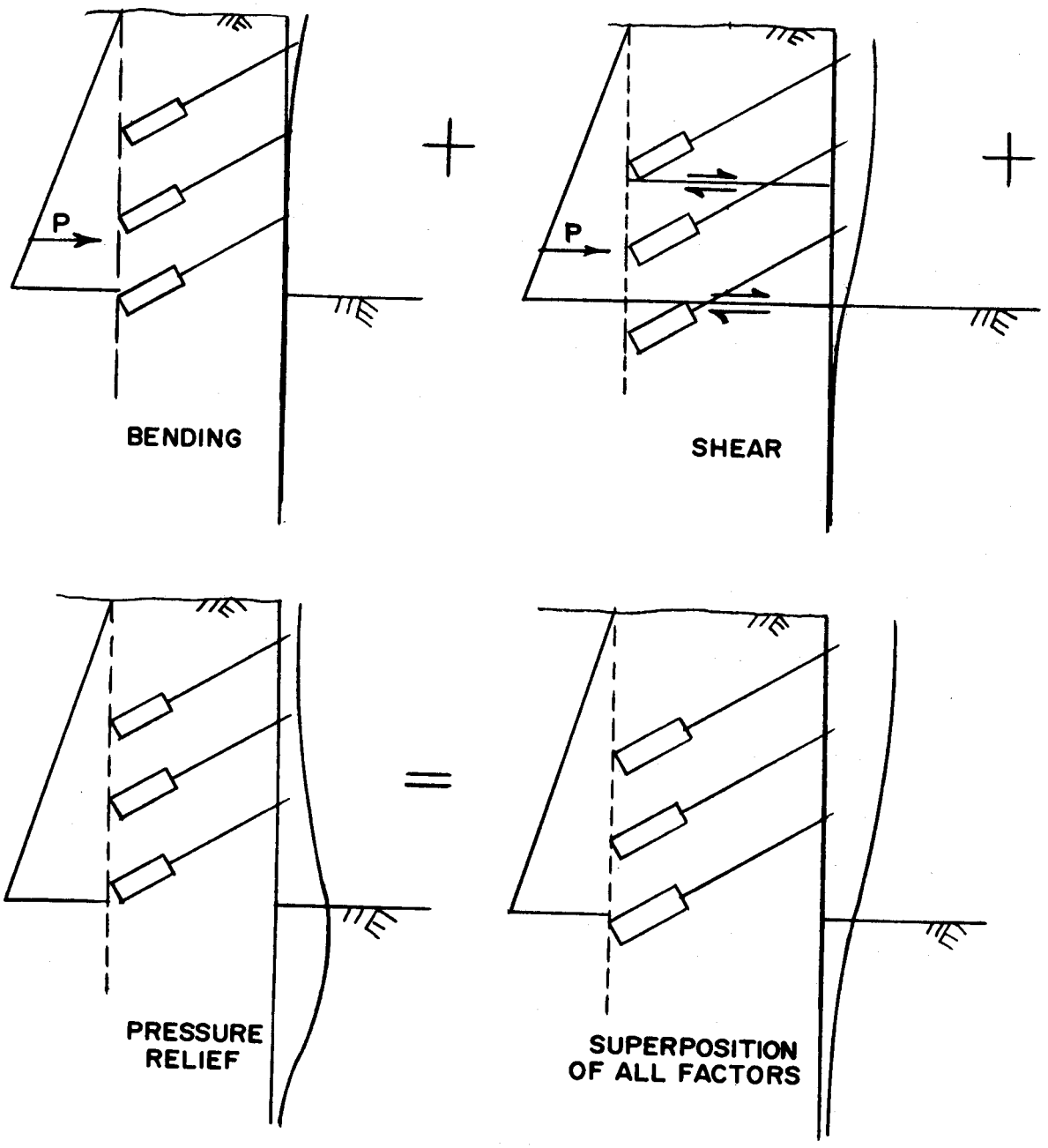


Figure 52. Idealized tieback wall deformation as proposed by Nendza and Klein (1974).

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 genralized 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 $P_{A_h} - T_{des_h} = P_{p_h}$. Point

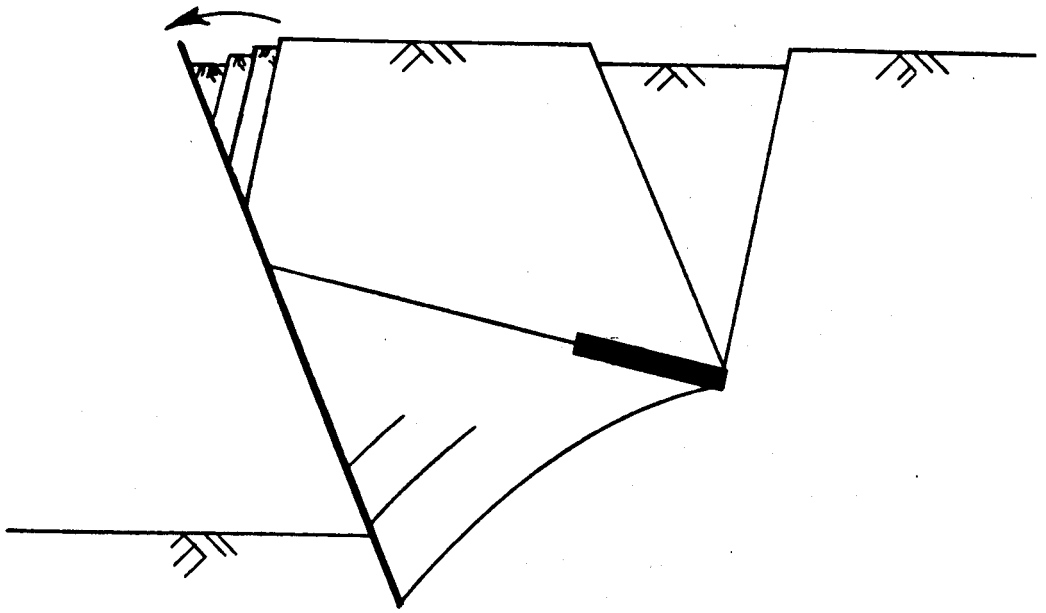


Figure 53. Sketch of tied-back wall failing by overturning.

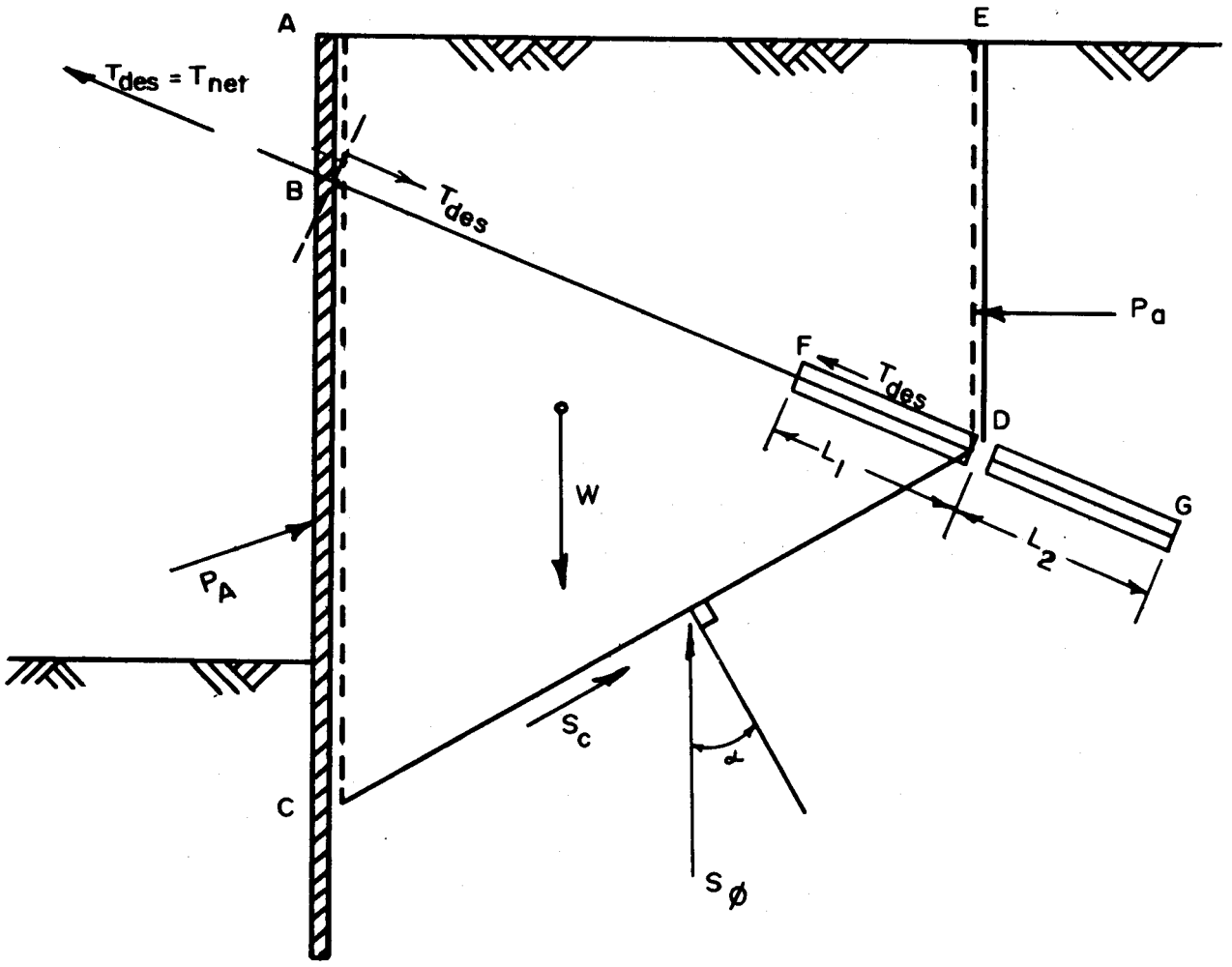


Figure 54. Free body diagram for a failure surface in single anchor tieback system (internal free body).

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.

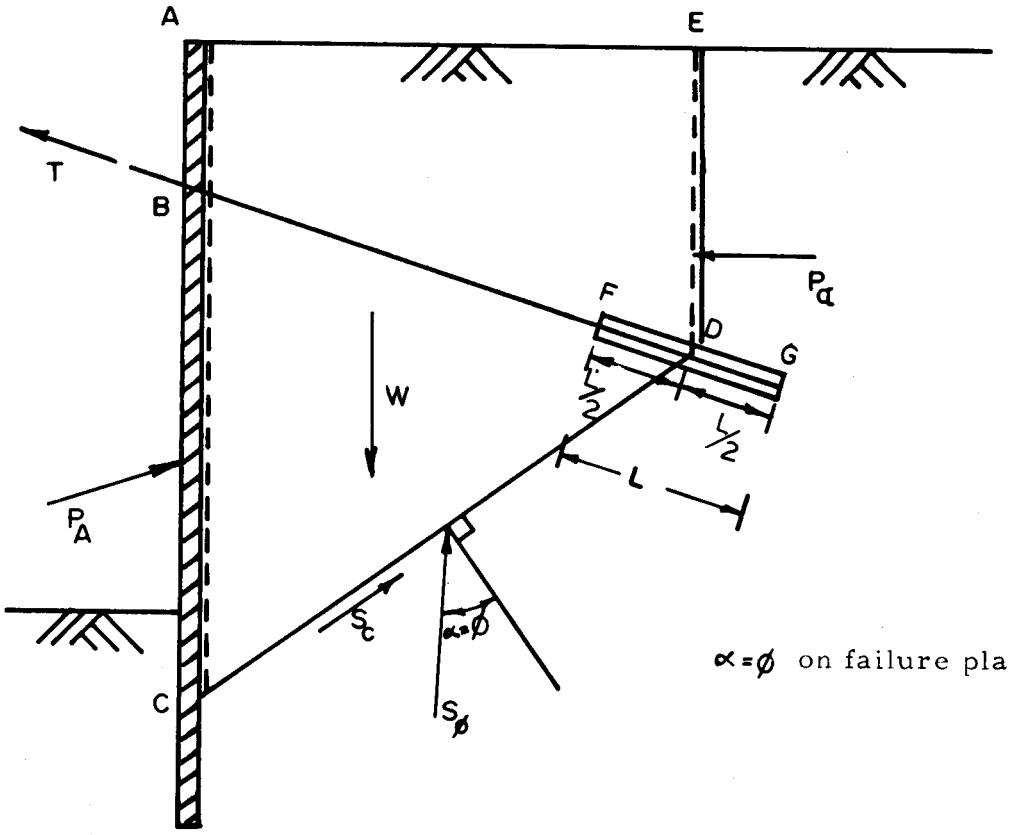
f. T - the tieback force. The free body cuts the tieback at points B and D. The force, T_{net}, (Figure 54) represents the vector sum of tieback force at point B and point D. Since the force at B must exceed the force at D, the force acts in the direction shown.

Safety in Terms of Tieback Force

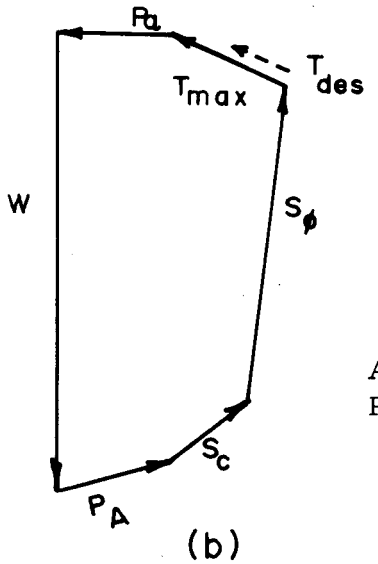
The force T_{max} is the maximum possible force acting in the direction of the tieback (see Figure 55). It should be noted that its magnitude will increase with increasing shear resistance on the failure plane. The overall stability is evaluated in terms of the ratio of T_{max} to the design tieback 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

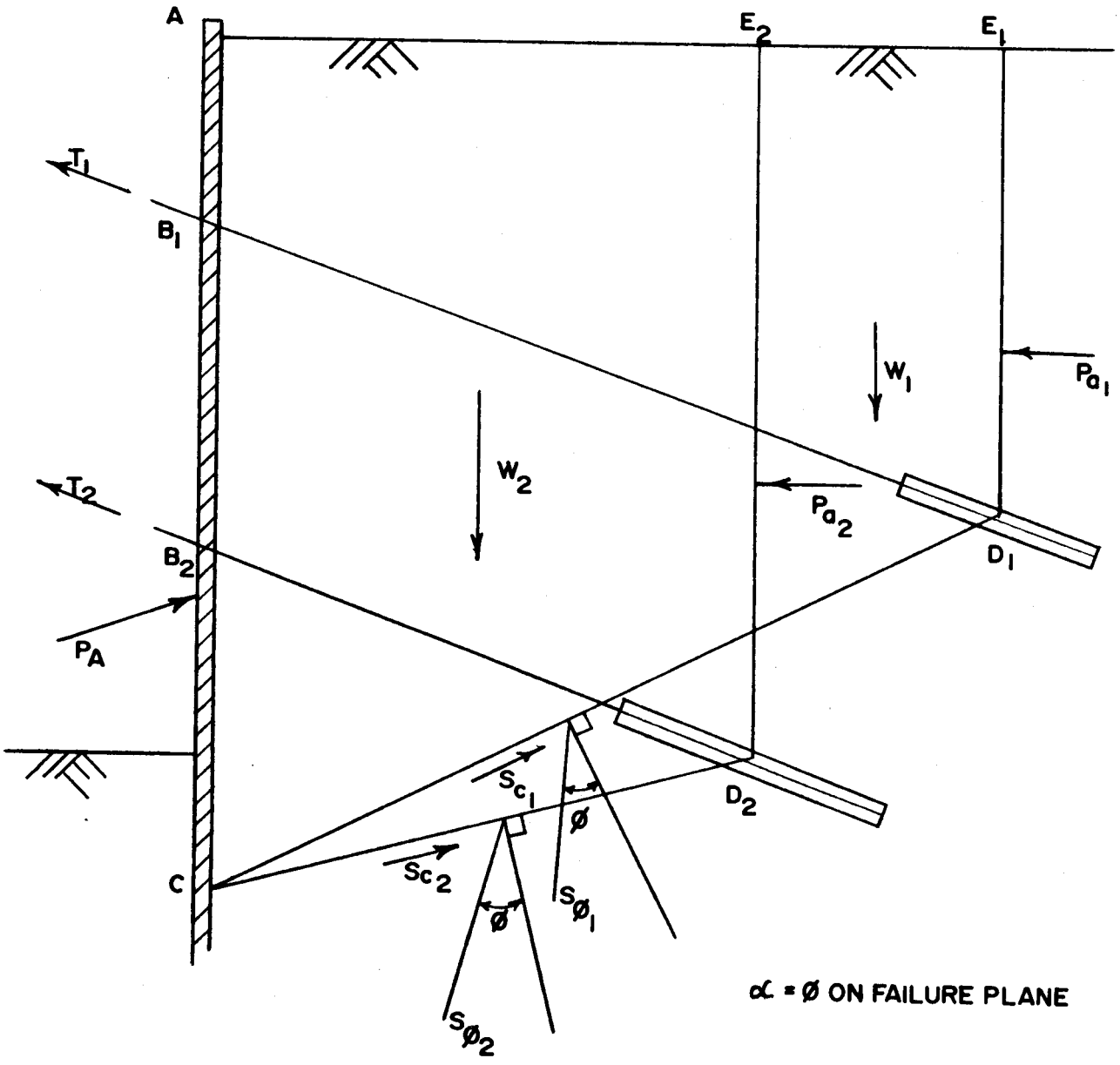
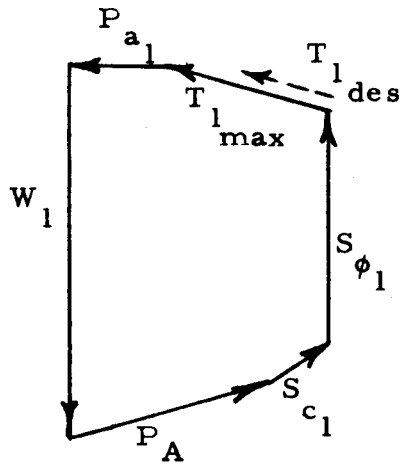


Figure 56. Free bodies and forces for two completely independent anchors (after Ranke and Ostermayer, 1968).



a. Upper Tieback

$$F.S. = \frac{T_{1_{max}}}{T_{1_{des}}} \geq 1.5$$

Note: only the directions of S_ϕ and T_{max} are known.

α = ϕ on failure plane.

B. Lower Tieback

$$F.S. = \frac{T_{2_{max}}}{T_{2_{des}}} \geq 1.5$$

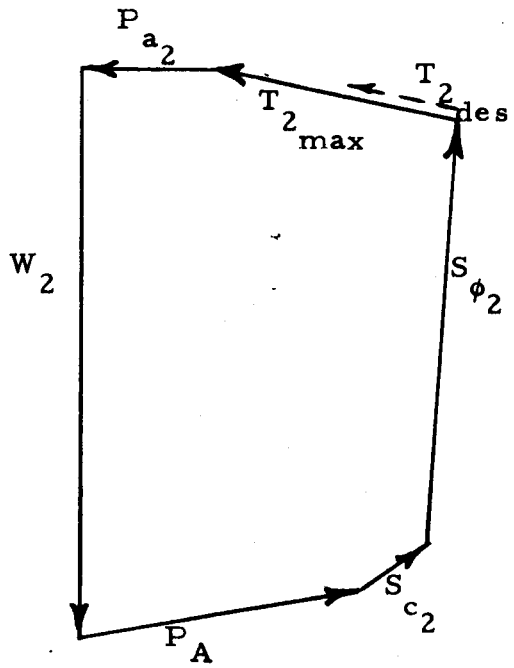


Figure 57. Vector diagram for case of two completely independent anchors (safety in terms of anchor force) (after Ranke and Ostermayer, 1968).

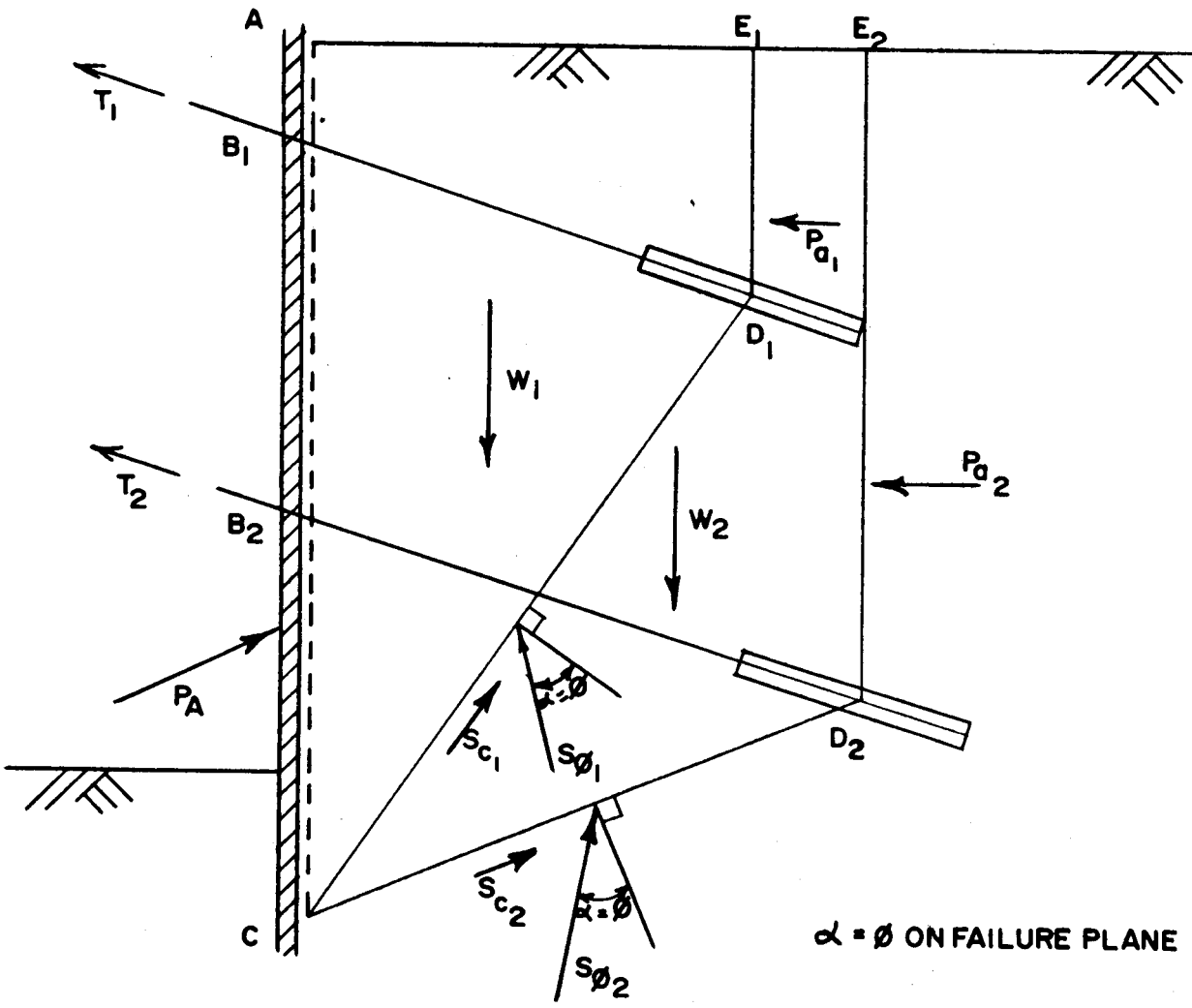
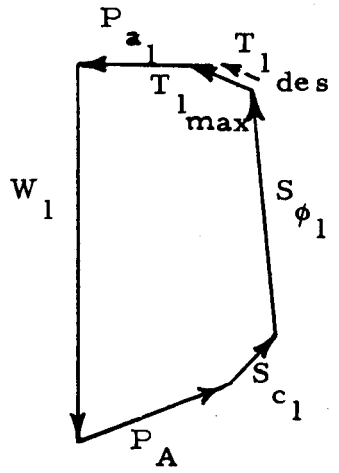
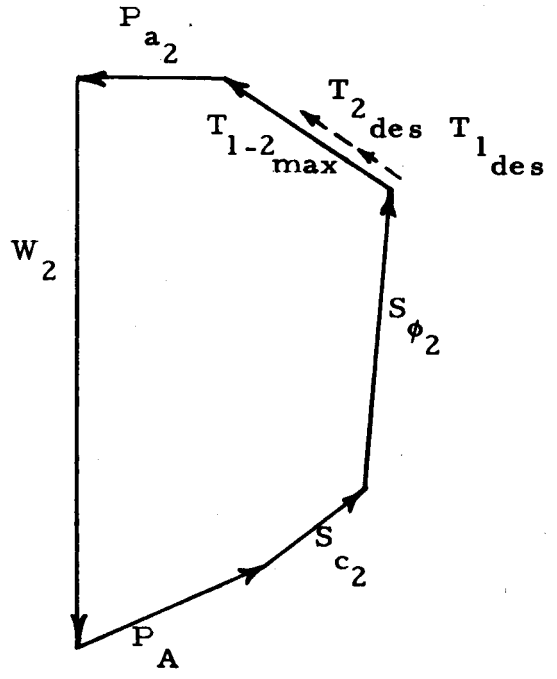


Figure 58. Free body diagram with forces acting on the bodies for the case of one independent anchor (safety in terms of the tieback force) (after Ranke and Ostermayer, 1968).



$$F.S. = \frac{T_{1_{max}}}{T_{1_{des}}} \qquad 1.5$$

$\alpha \equiv \phi$ on failure plane.



$$F.S. = \frac{T_{1\text{-}2_{max}}}{T_{1_{des}} + T_{1_{des}}} \qquad 1.5$$

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).

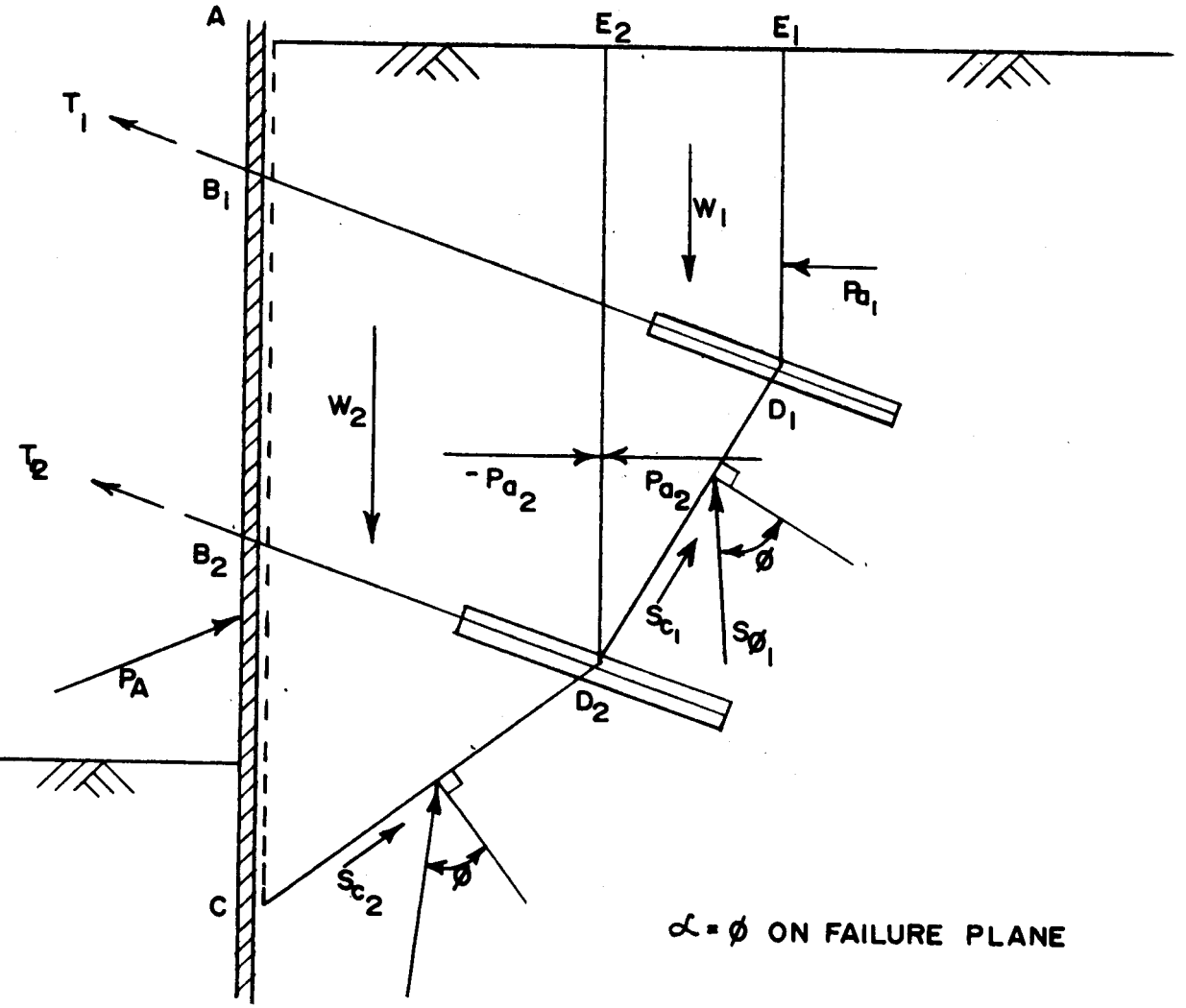


Figure 60. Free body diagram for anchor system with a complex failure surface (safety in terms of the tieback force).

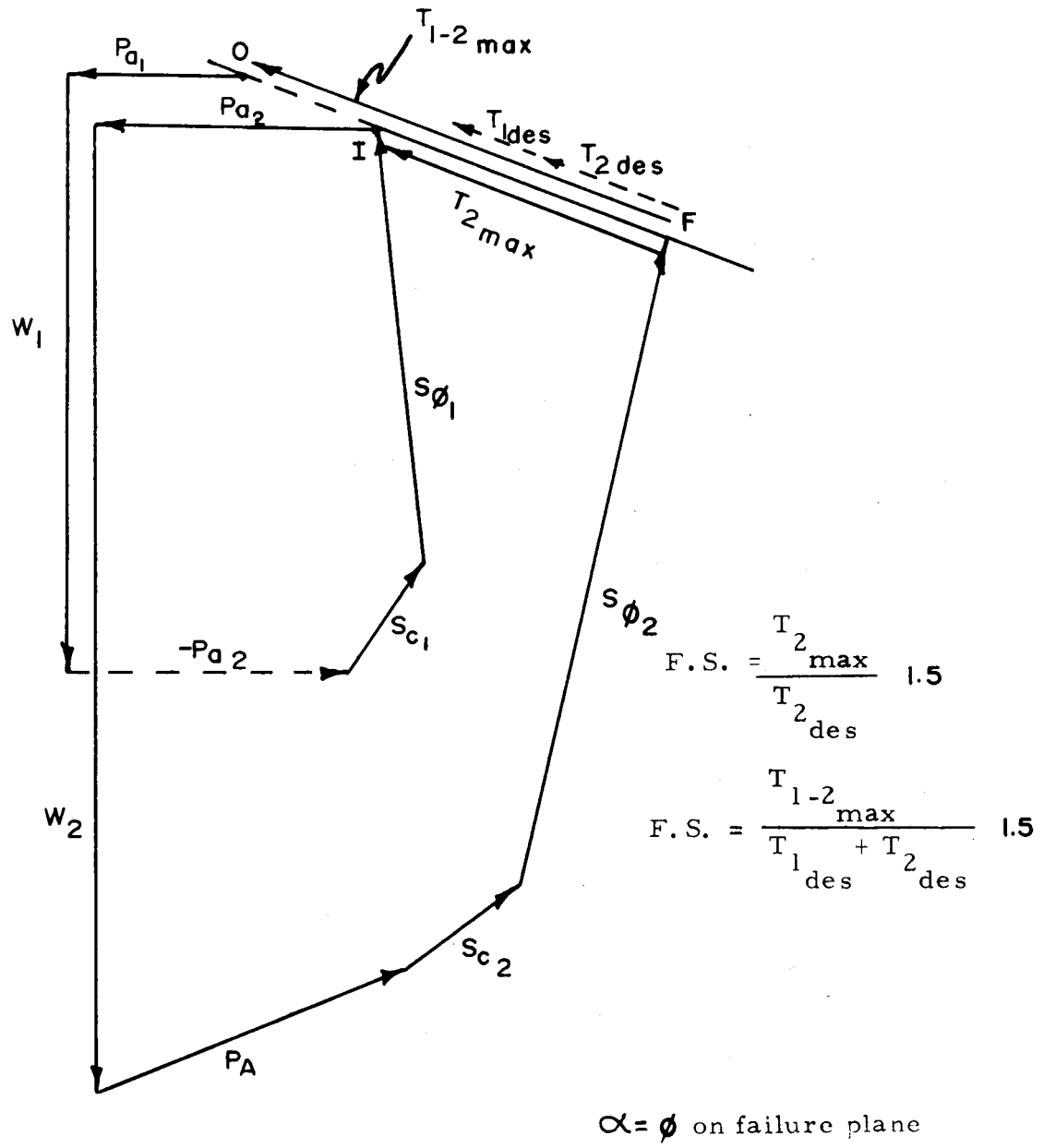


Figure 61. Vector diagram for a complex failure surface (safety factor in terms of the tieback force) (after Kranz, 1953, and Ranke and Ostermayer, 1968).

first free body (defined by points D_2, D_1, E_1, E_2, D_2) yields the forces, P_{a_1}, W_1, $-P_{a_2}$, S_{c_1}, and S_{ϕ_1}. The vector diagram begins at point 0 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, $\frac{T_{2_{max}}}{T_{2_{des}}}$ and $\frac{T_{1\text{-}2_{max}}}{T_{1_{des}} + T_{2_{des}}}$ $T_{2_{max}}$ is the vector distance between points F and I on the vector diagram while $T_{1\text{-}2_{max}}$ is the distance between F and O. The recommended design criteria are both $\frac{T_{2_{max}}}{T_{2_{des}}}$ and $\frac{T_{1\text{-}2_{max}}}{T_{1_{des}} + T_{2_{des}}}$ 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

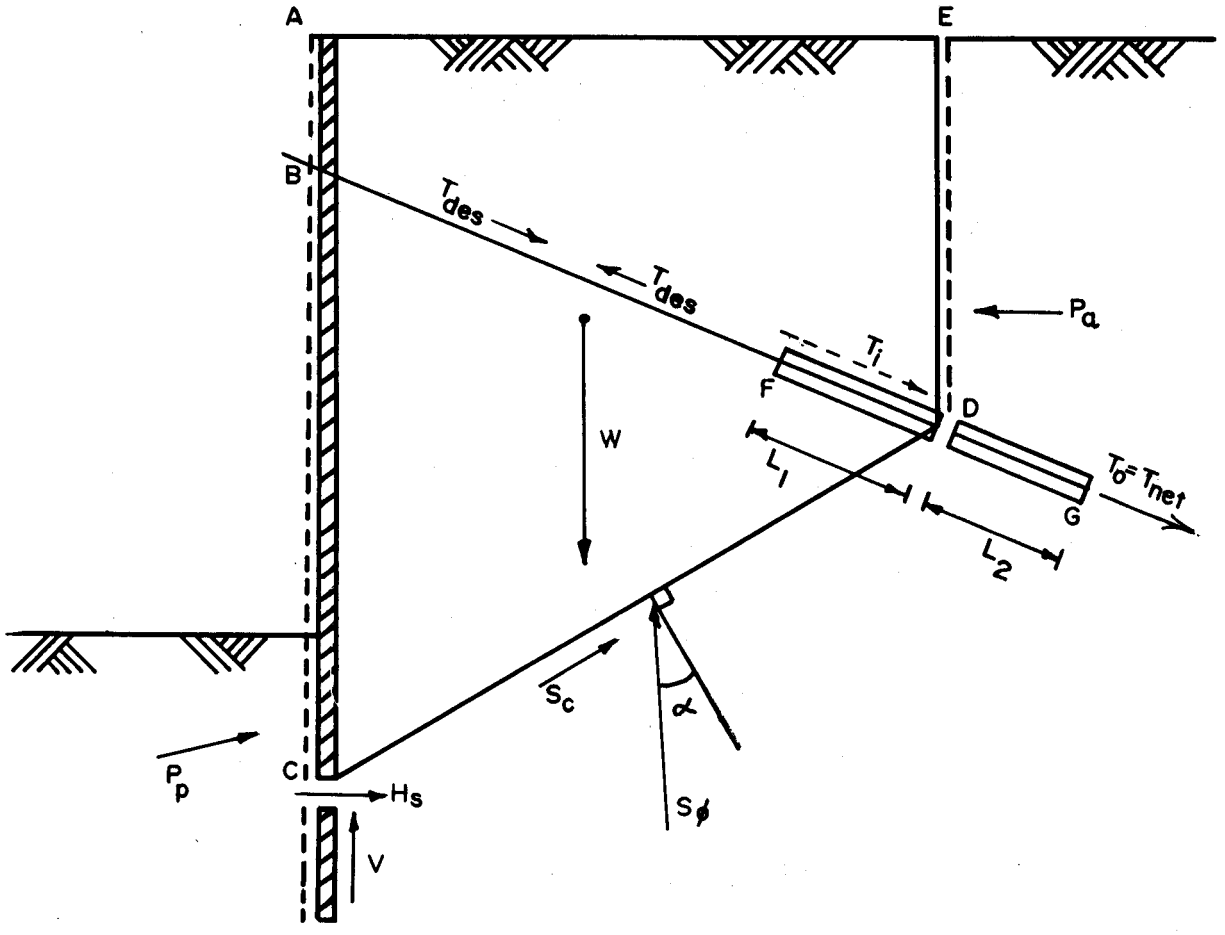


Figure 62. Free body diagram for a failure surface in a single anchor tieback system (free body outside of wall).

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

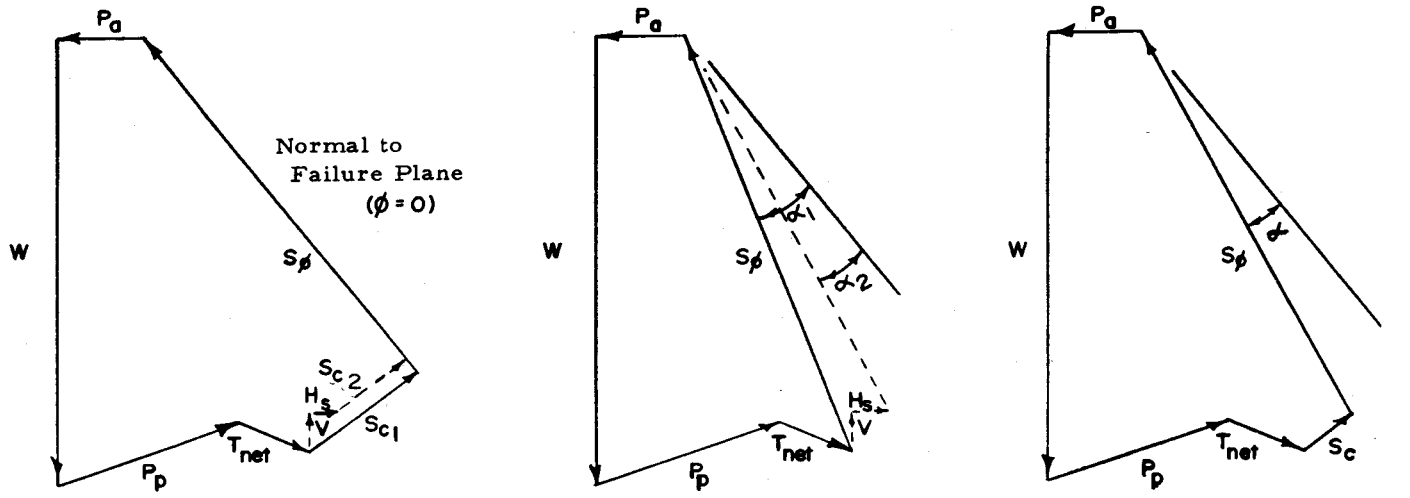
The sliding wedge analysis does not specify the location of the failure surface as did the previous overturning analysis. Several failure surfaces can be analyzed for a given anchor geometry. The distribution of load in the anchor is assumed to be uniform over the entire length unlike the distribution 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 mobi-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 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:



$$F.S. = \frac{S_{c\ avail}}{S_{c\ mob}} = \frac{S_u \times L}{S_{u\ mob} \times L}$$

(a)

Cohesive Soil

$$F.S. = \frac{\text{Shear strength available}}{\text{Shear strength mobilized}} = \frac{N \tan \phi}{N \tan \alpha}$$

(b)

Cohesionless Soil

$$F.S. = \frac{S_u \times L}{S_{u\ mob} \times L} = \frac{N \tan \phi}{N \tan \alpha}$$

(c)

Soil With Both Cohesive and Cohesionless Properties

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.

$$\frac{S_{c_{avail}}}{S_{c_{mob}}} = \frac{S_u \times L}{S_{mob} \times L}$$

where:

S_u = undrained shear strength of soil

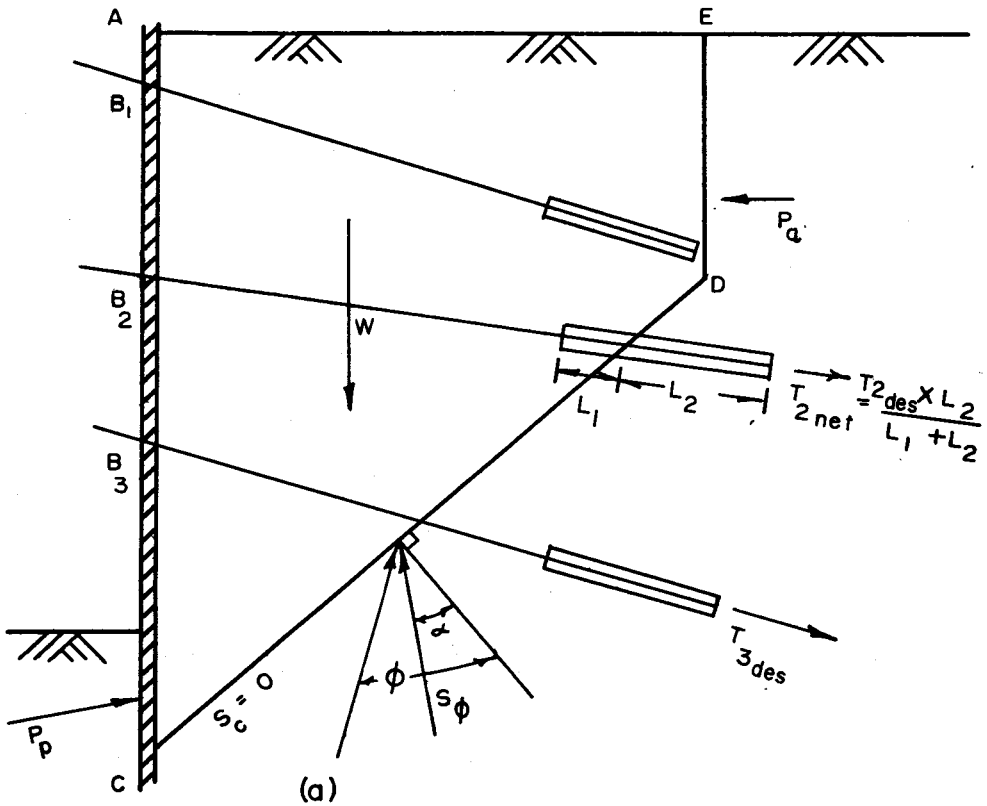
L = length of failure surface

S_{mob} = mobilized shear strength

For cohesionless materail 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, $\frac{S_{c_{avail}}}{S_{c_{mob}}}$ 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



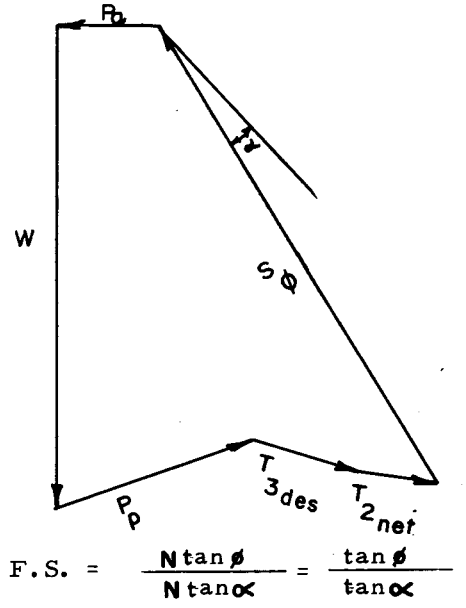


Figure 64. Analysis of a multiple level anchor system (safety factor in terms of soil strength).

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 $F.S. = \frac{P_{p_{avail}}}{P_{p_{mob}}}$. 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 = moment = $P_a \times H / 3 = \frac{\gamma H^3 K_a}{6}$

B = effective width

$K_a = \tan^2(45 - \phi/2)$

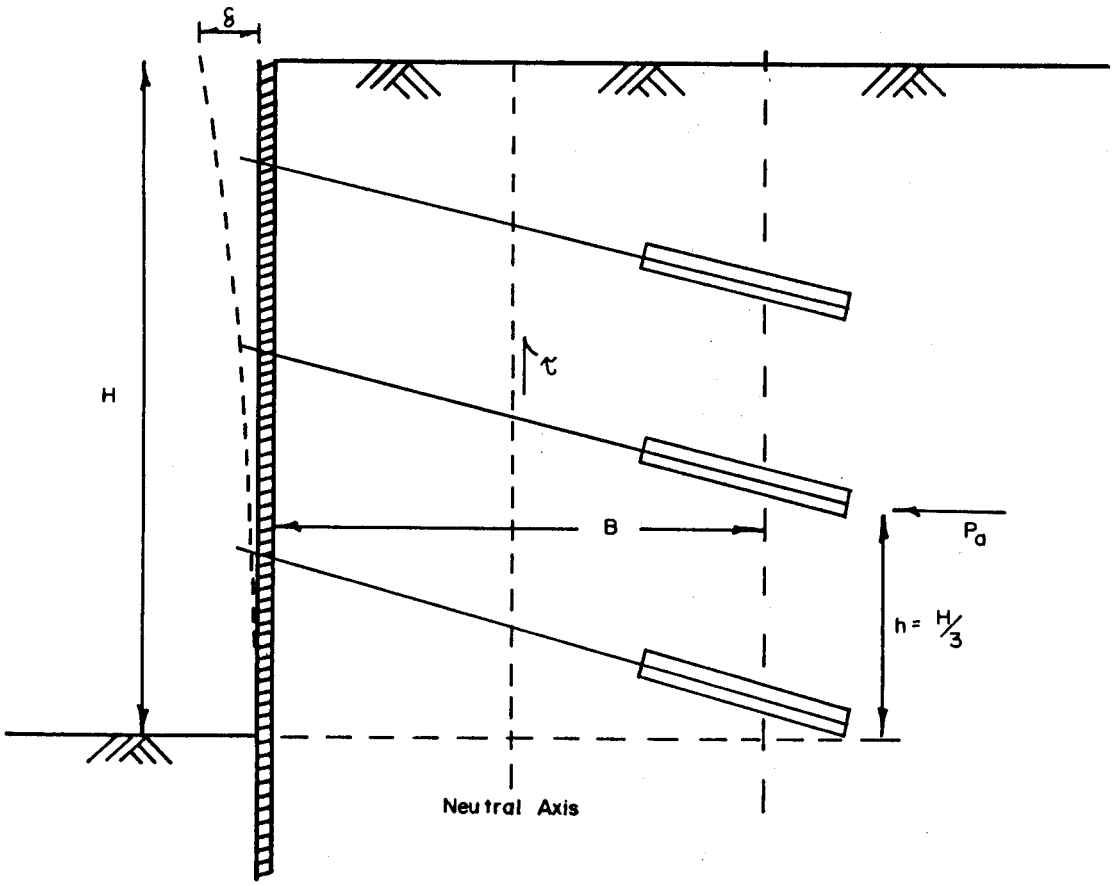


Figure 65. Sketch of equivalent cofferdam for tied-back wall.

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:

$$\bar{\sigma}_h = \frac{\cos^2 \phi}{2 - \cos^2 \phi} \qquad \bar{\sigma}_v = \frac{1}{1 + 2 \tan \phi} \bar{\sigma}_v = K \bar{\sigma}_v$$

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:

$$F.S. = \frac{1}{K_a} \quad \frac{1/2\ H^2 \dfrac{\tan\phi}{1 + 2\tan^2\phi}}{3/2\ (H^3/6)\quad (1/B)} = \frac{2B}{H} \quad \frac{\tan\phi}{1 + 2\tan^2\phi} \quad \frac{1}{\tan^2(45-\phi/2)}$$

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:

$$S_{ff} = \bar{\sigma}_v \sin\phi \cos\phi \ (\frac{1 + K}{2})$$

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 achorage 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.

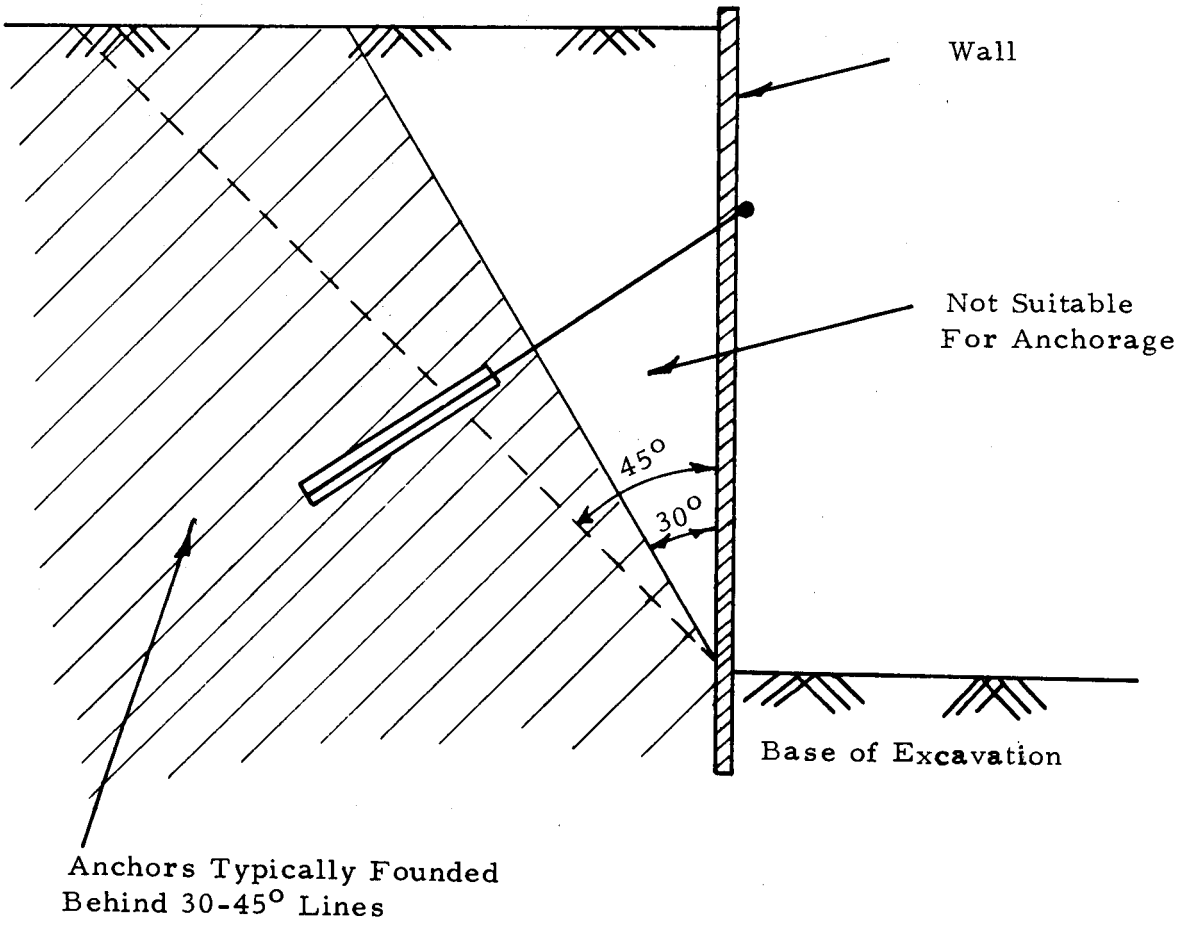
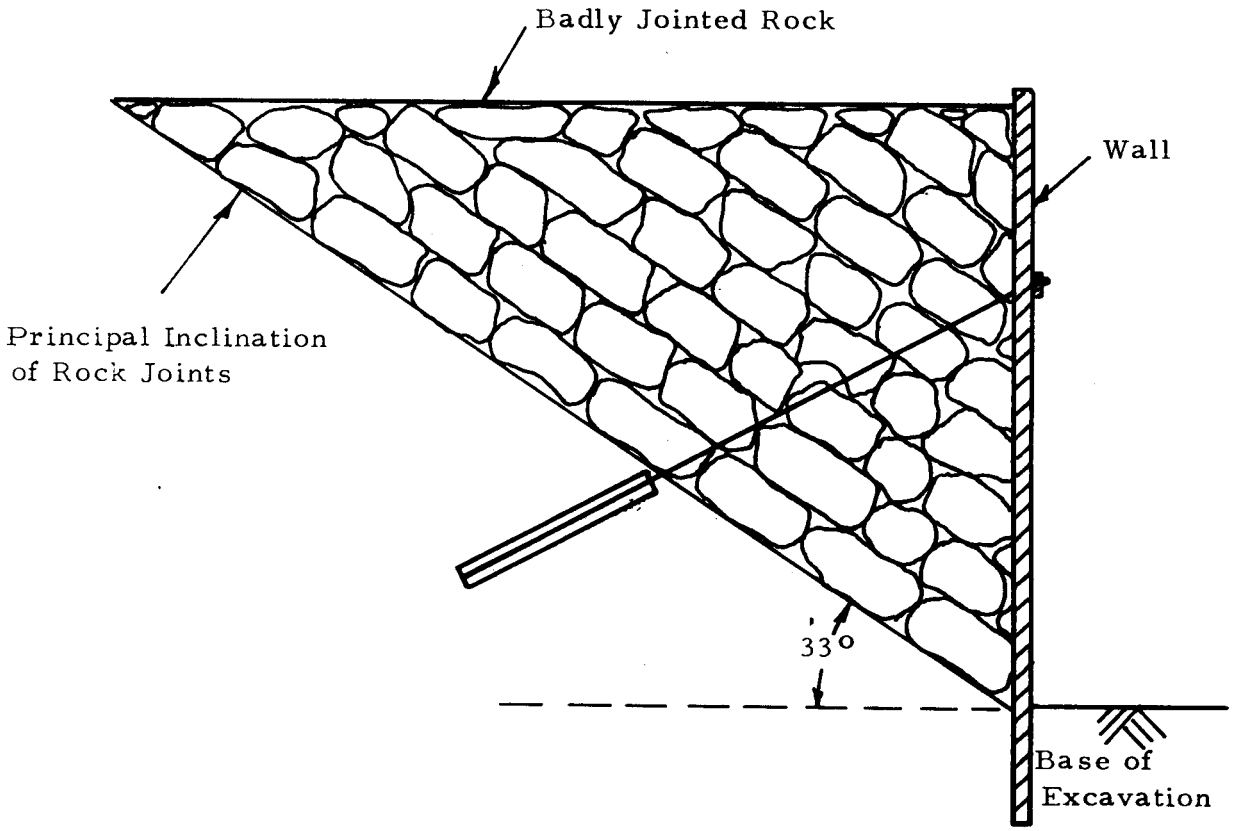


Figure 66. Typical location of anchors.



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 uncertainity, 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 Kindom 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

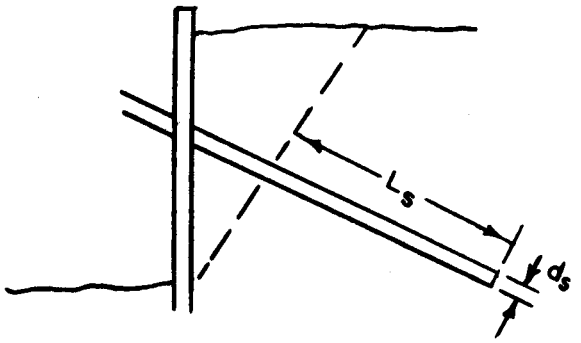
Small diameter anchors generally vary from 3" (7.5 cm) to 6" (15 cm) in size and are most frequently installed in granular soils. Anchors of this type are generally formed by grouting the anchor zone under large pressures. A special small diameter anchor which has the capability of grouting the anchor zone several times (regroutable anchor) has also been developed.

Often temporary casing is used to support the hole during its formation. After the hole as 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 anchorsoil 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.



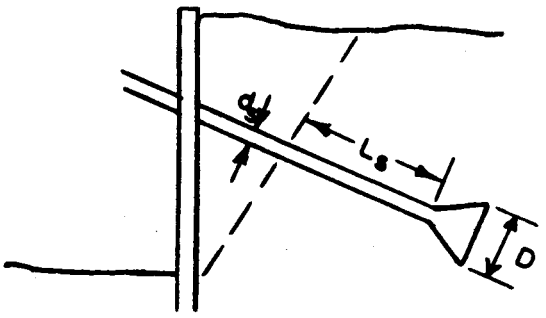
(a)

Friction Anchor

ESTIMATED LOAD FOR
ANCHORS IN COHESIVE SOIL

$$P_u = \alpha S_u L_s \pi d_s$$

$$\alpha = 0.3\text{-}0.5$$



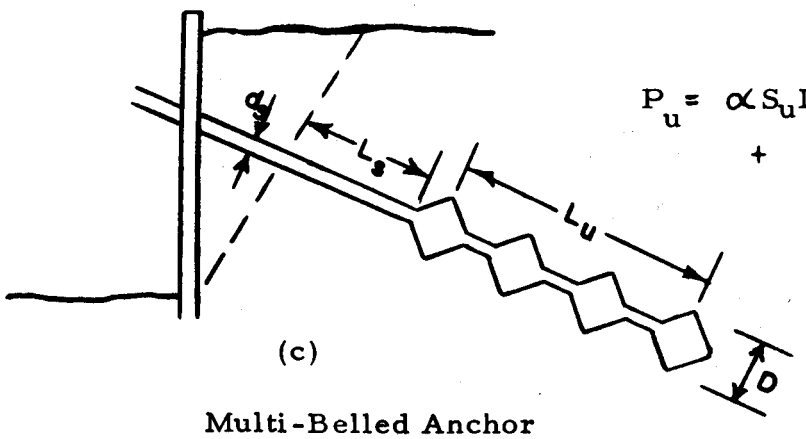
(b)

Belled Anchor

$$P_u = \alpha S_u L_s \pi d_s + \frac{\pi}{4}(D^2 - d_s^2) N_c S_u$$

$$\alpha = 0.3\text{-}0.5$$

$$N_c = 9$$



(c)

Multi-Belled Anchor

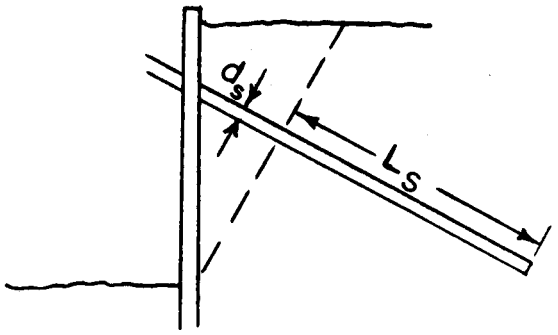
$$P_u = \alpha S_u L_s \pi d_s + \frac{\pi}{4}(D^2 - d_s^2) N_c S_u + \beta S_u \pi D L_u$$

$$\alpha = 0.3\text{-}0.5$$

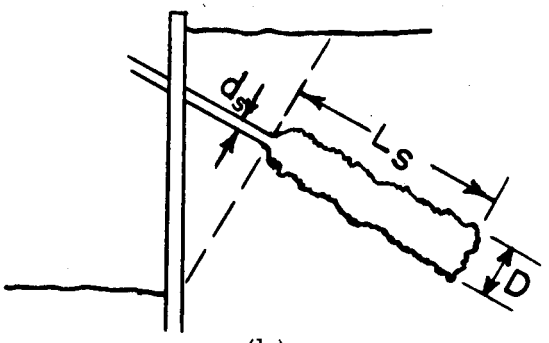
$$\beta = 0.75\text{-}1.00$$

$$N_c = 9$$

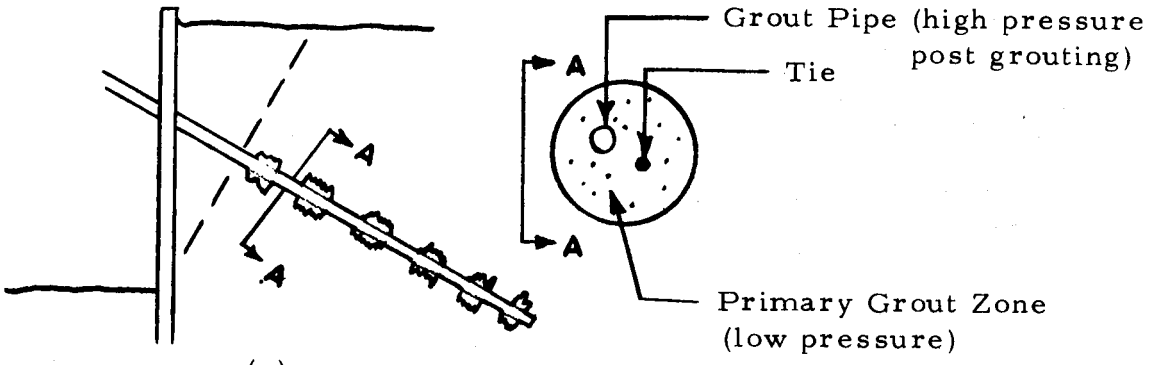
Figure 68. Schematic representation of large diameter anchors.



(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/m^2) 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) Shaft Type	Diameter (inches) Bell Type	Gravity Concrete	Grout Pressure (psi) (1)	Suitable Soils for Anchorage	Load Transfer Mechanism
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

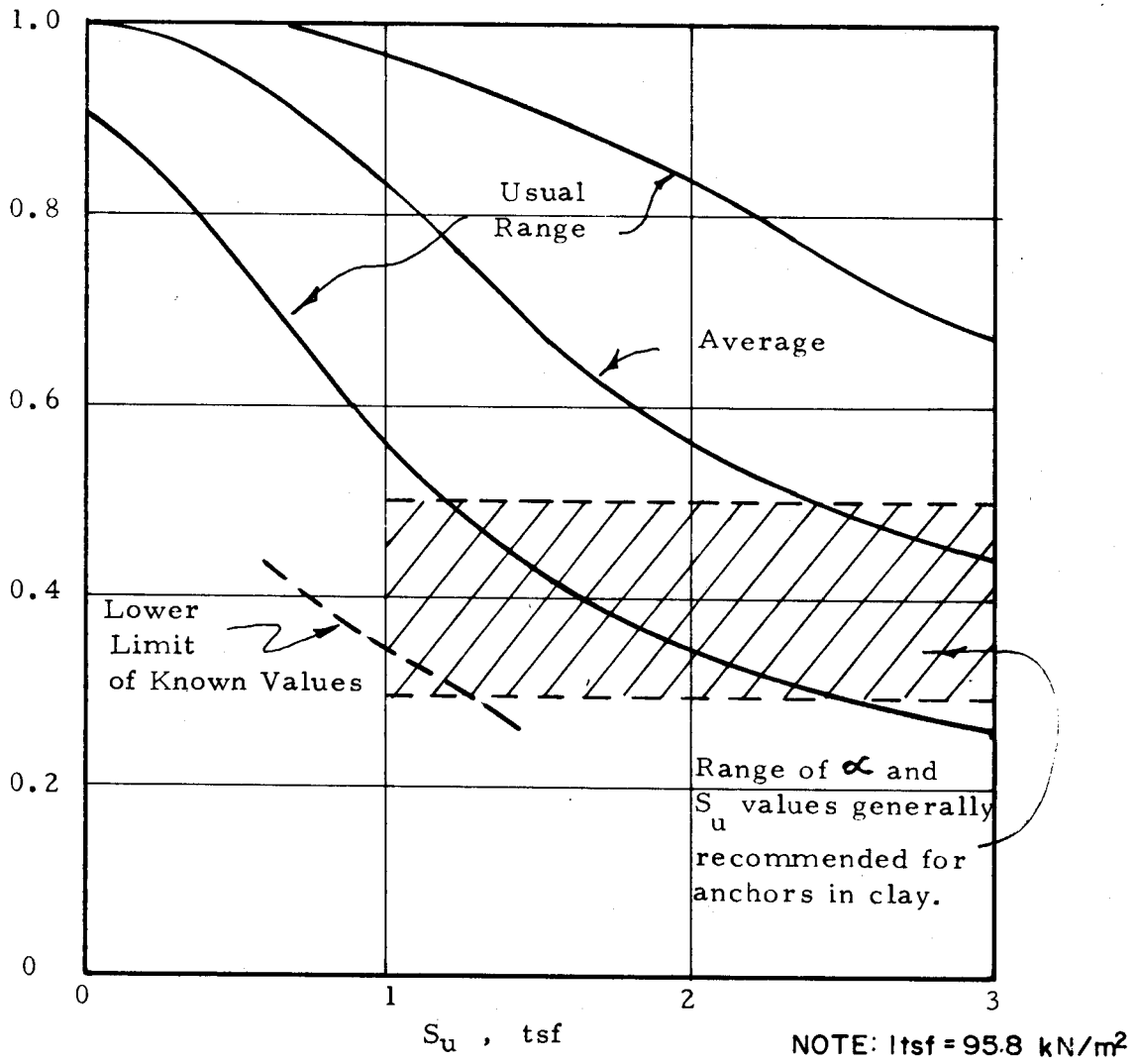
D = bell diameter

L_s = length of straight shaft

S_u = undrained shear strength of soil

α = reduction factor for shear strength (α = 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 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 = $\dfrac{\text{contact pressure at anchor-soil interface}}{\text{effective vertical stress } (\bar{\sigma}_v)}$

Littlejohn reports typical values of A ranging between 1 and 2

B = a bearing capacity factor similar to N_q but smaller in magnitude. A value of

$B = \dfrac{N_q}{1.3 - 1.4}$ is recommended provided $h/D \geq 25$; where h 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 regrouted 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 regroutable anchor cannot easily be estimated using theoretical formulae. Also, if the anchor proves inadequate under proof loading, it can be regrouted, 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 regroutable 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.

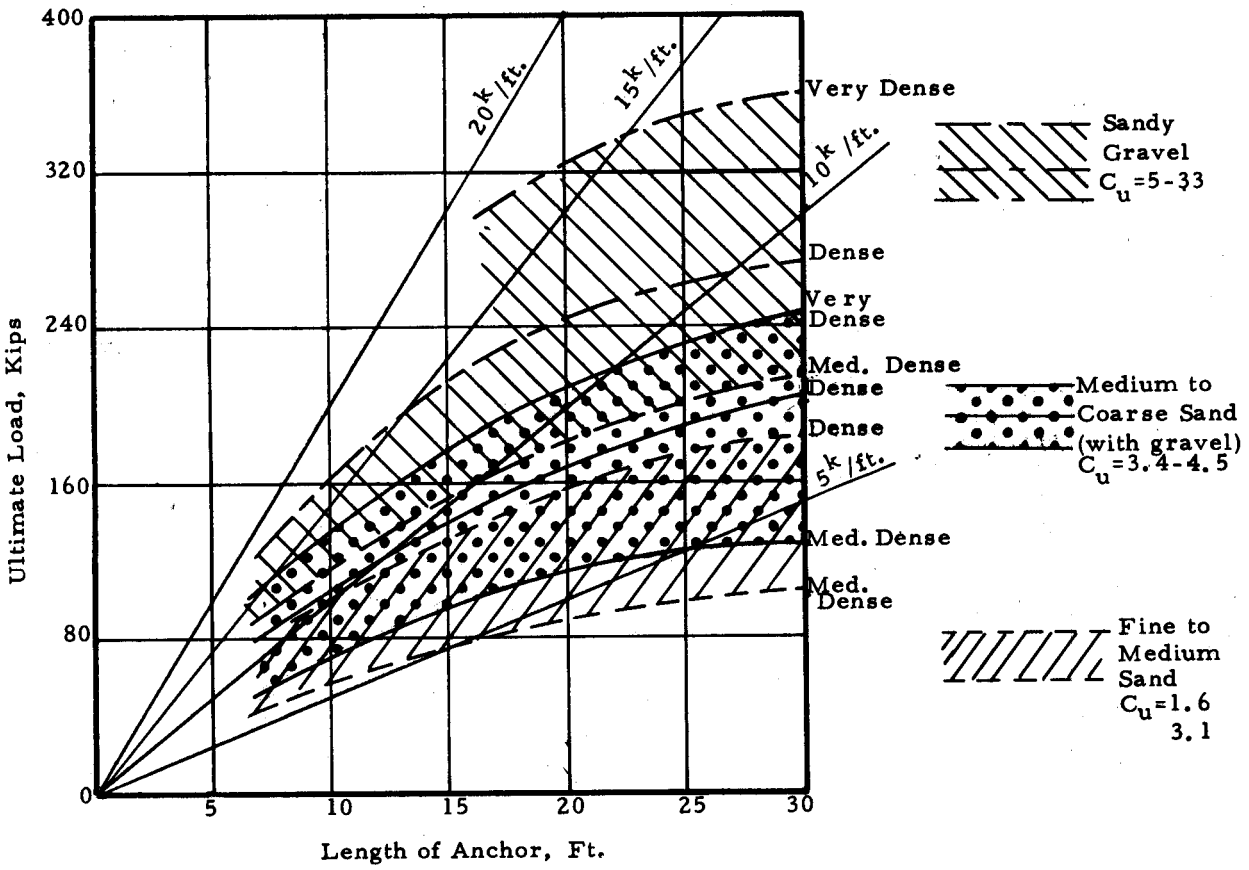
Typically, the anchors studied were four to six inches (10 - 15 cm) in diameter and thirteen to twenty-six feet (4 - 8m) long in the grouted zone. Grout pressures of at least 150 psi (1035 kN/m^2) are applied in cohesionless soils.

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 ≥ 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 capapcity 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:

Soil	Ultimate Load (kip/ft)	kN/m
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 inital grouting pressure was relatively low (70 - 130 psi) (480 - 900 kN/m^2), 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/m^2) increase in grout pressure. It should be noted that for sands and gravels subjected to post-grouting pressures of

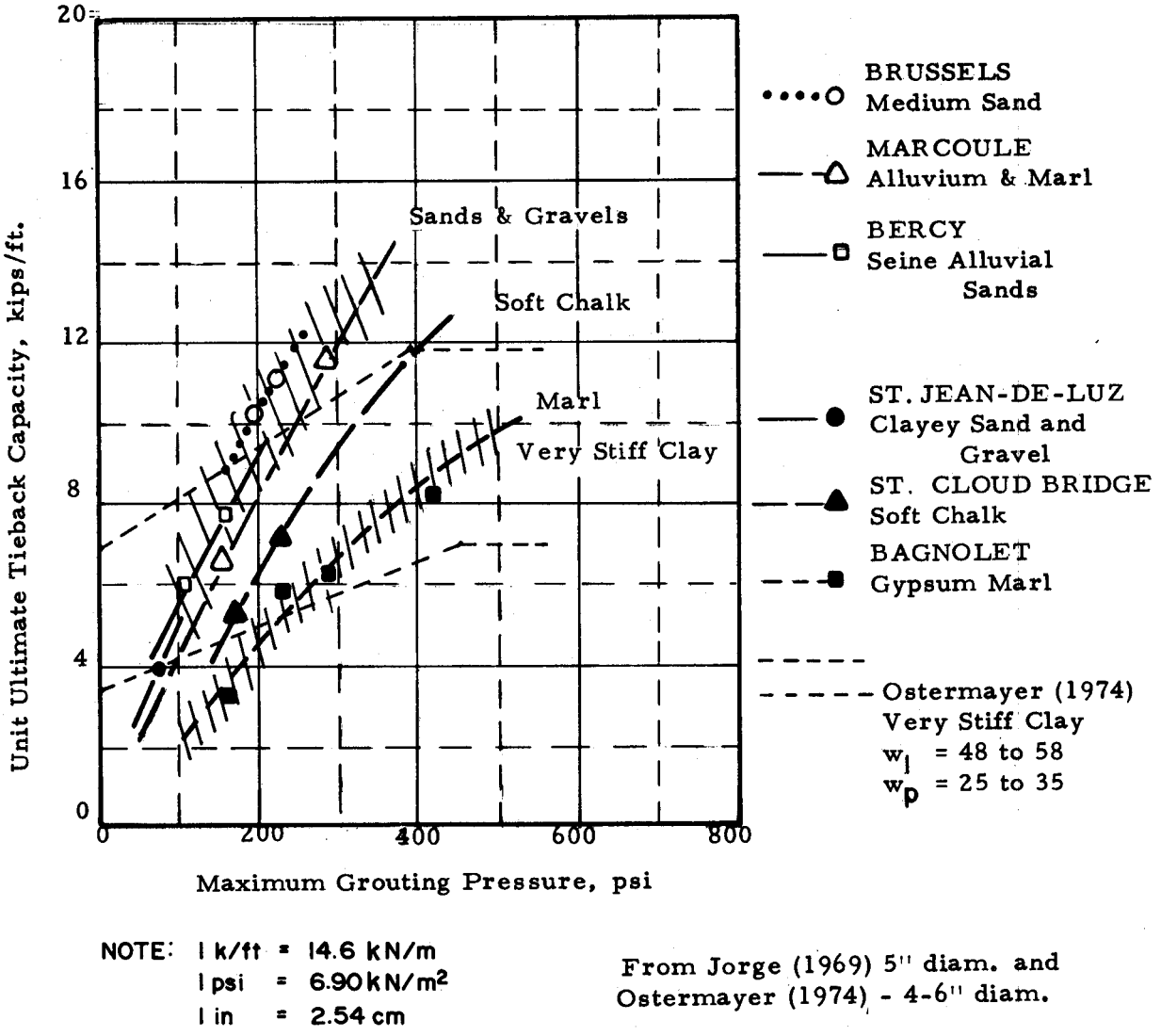


Figure 72. Ultimate anchor capacity as a function of grout pressure.

200 psi (1380 kN/m^2) 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/m^2) was observed. The skin friction associated with 350 psi (2400 kN/m^2) 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/m^2).

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/m^2) 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/m^2) to 3500 psf (170 kN/m^2) for small diameter, high pressure tiebacks installed without post-grouting.

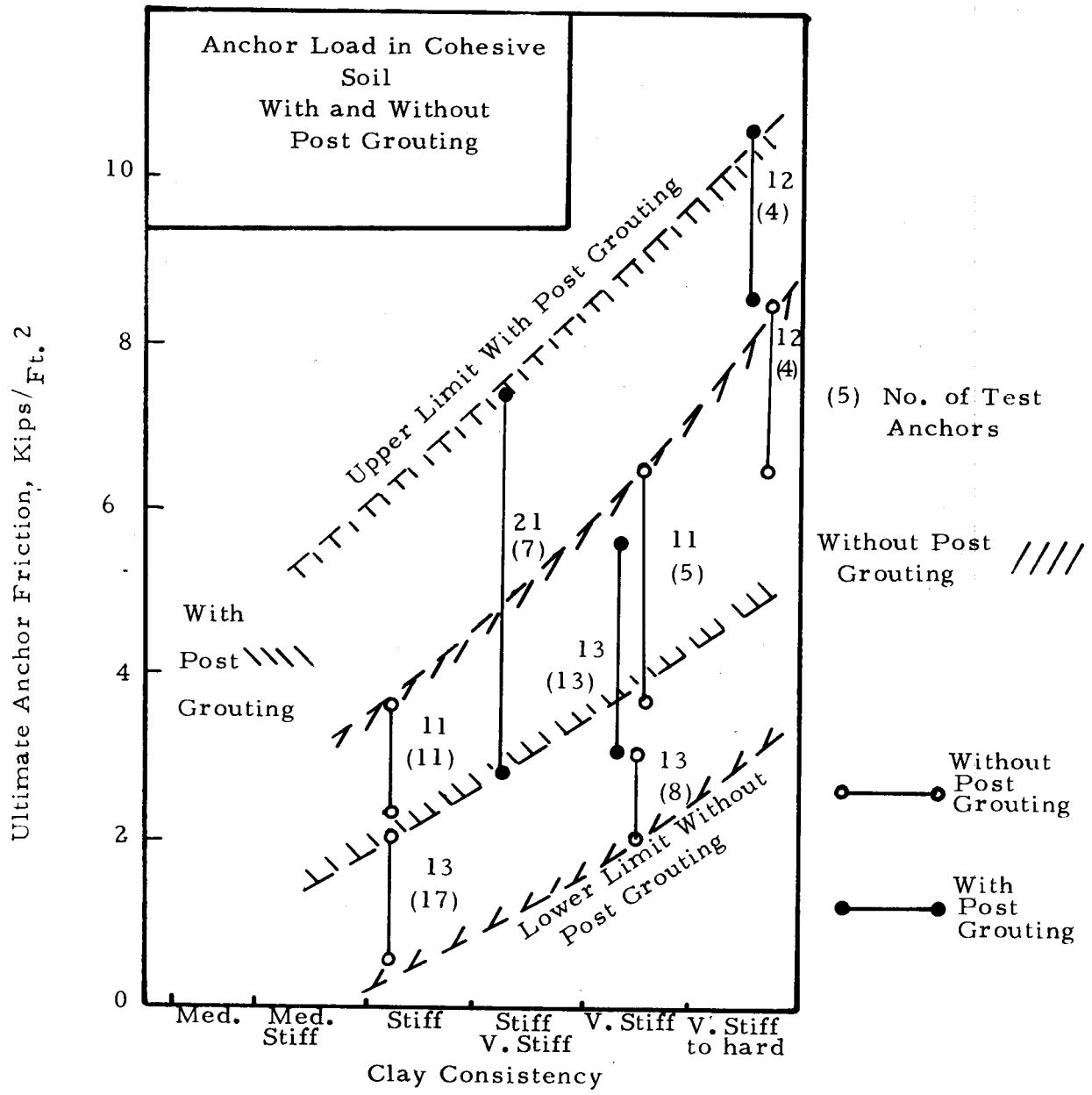
Ostermayer (1974) also reported data on the results of fifty-six tests to failure in which he developed creep rate coefficients for small diameter [3 - 1/2" ± to 6" (9 - 15cm)] anchors. The data was reported in terms of deflection per log cycle of time (creep rate) and percent of observed anchor failure 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).

Soil Type	Typical Skin Friction (per square foot of grouted zone) Without Post-Grouting	With Post-Grouting
Marl Clay - medium plastic (w_l = 32 to 45; w_p = 14 to 25)		
Stiff	2200 - 3500	----------
Very Stiff	3500 - 6500	----------
Marl Sandy Silt - medium plastic (w_l=45; w_p=22)		
Very stiff to hard	6500 - 8500	8500 - 10,500
Clay - medium to highly plastic (w_l = 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 psf = 0.48 kN/m^2
   1 in = 2.54 m
   1 ft = 0.305 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: $1\ K/ft^2 = 47.9\ KN/m^2$

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

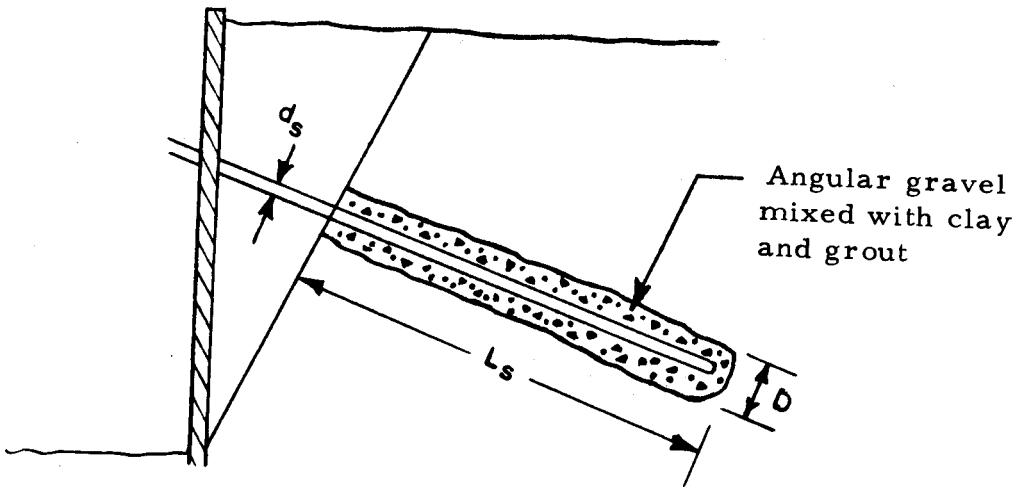


Figure 74. Schematic of gravel packed anchor.

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^{o} to 90^{o} 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 $\frac{E_{grout}}{E_{rock}} < 10$).

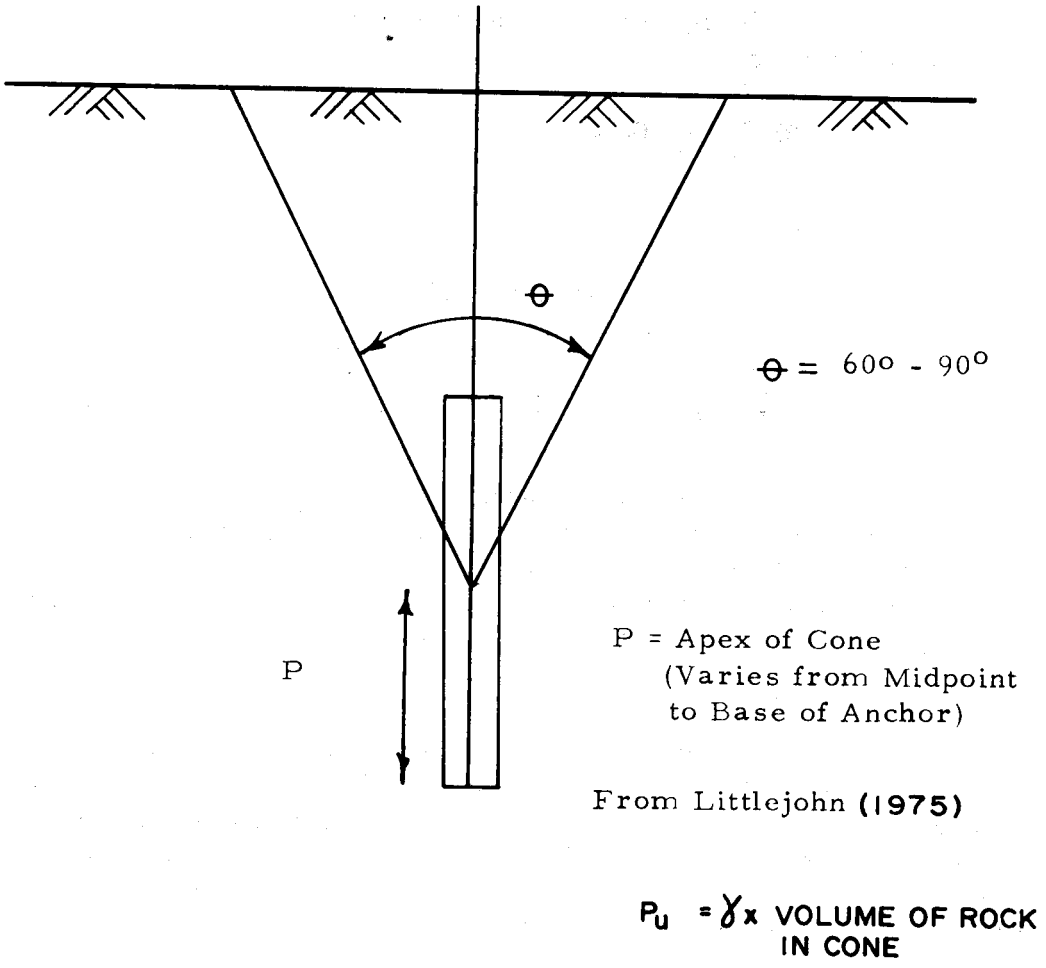


Figure 75. Schematic drawing of design quantities for failure in a rock mass.

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 determing anchor load the design equation becomes:

$$P_u = \pi d_s L_s \sigma_{skin}$$

where:

d_s = diameter of anchor shaft
L_s = length of anchor shaft
σ_{skin} = grout-rock bond strength

The values of skin friction, σ_{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 psi = 6.90 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 curde, 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 pullout 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 correpsonds 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 losse 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 significiant 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

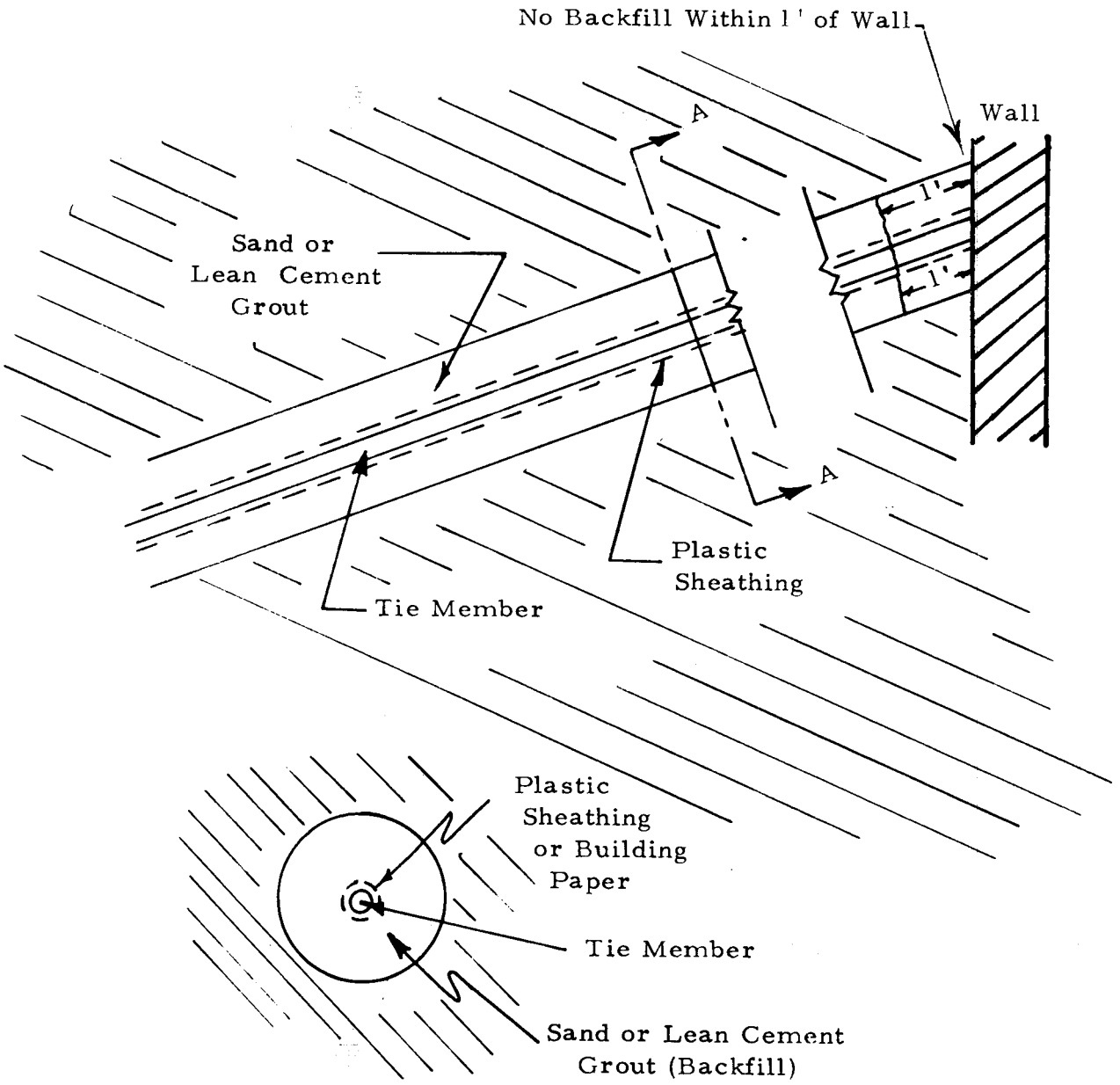


Figure 76. Recommended treatment for bond free zone.

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/mm^2
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/m^2) or greater), but the grout can also be placed under relatively low pressures 30 psi (200 kN/m^2) . 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/mm^2). 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	$f_d \leq 0.75 f_y$ or $f_d \leq 0.57 f_u$ whichever is smaller
Field Testing	$f_t \leq 0.9\, f_y = 0.76\, f_u$, 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

[1] Maximum 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_o) 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).

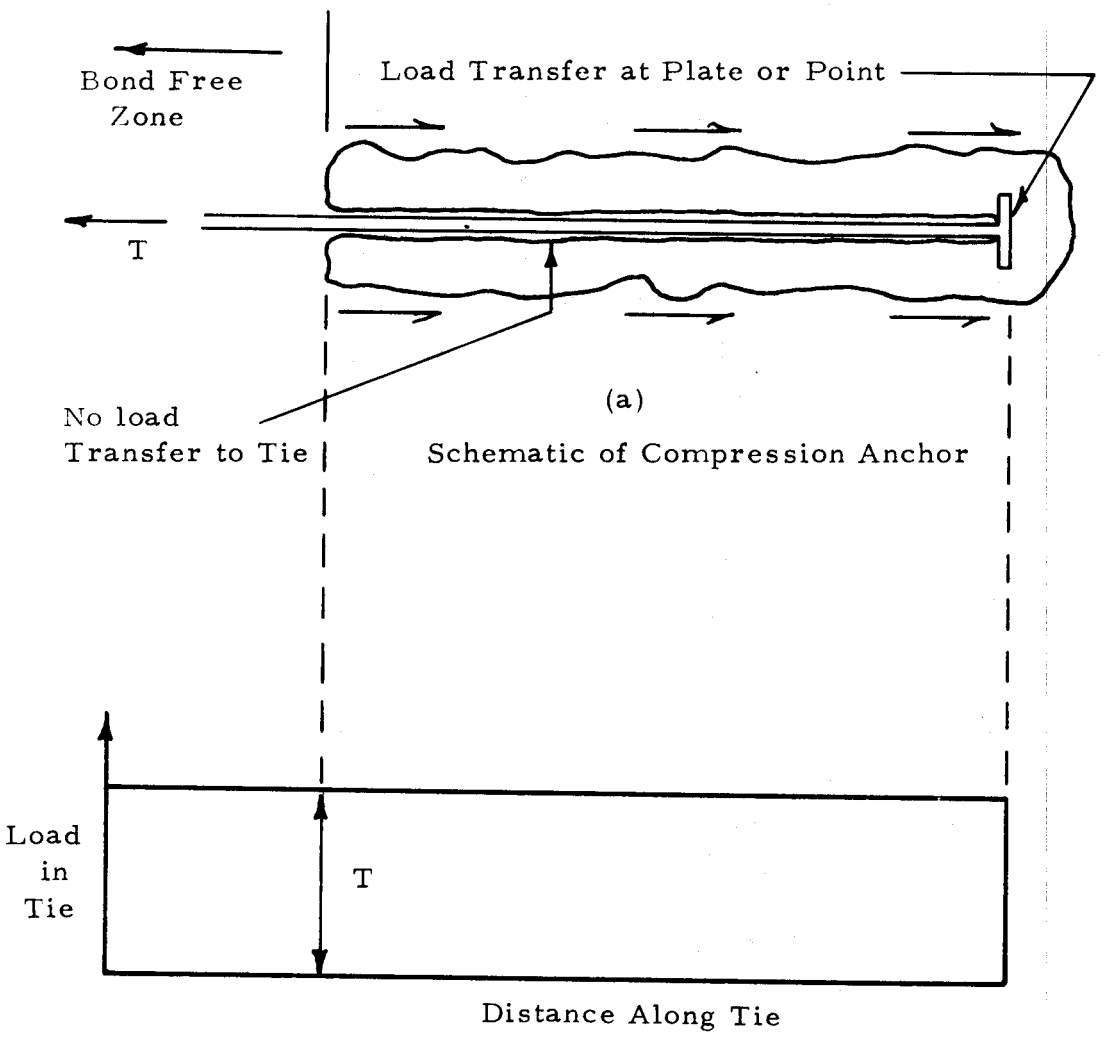


Figure 78. Schematic of compression anchor and resulting load distribution in the tie.

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 substantail 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.

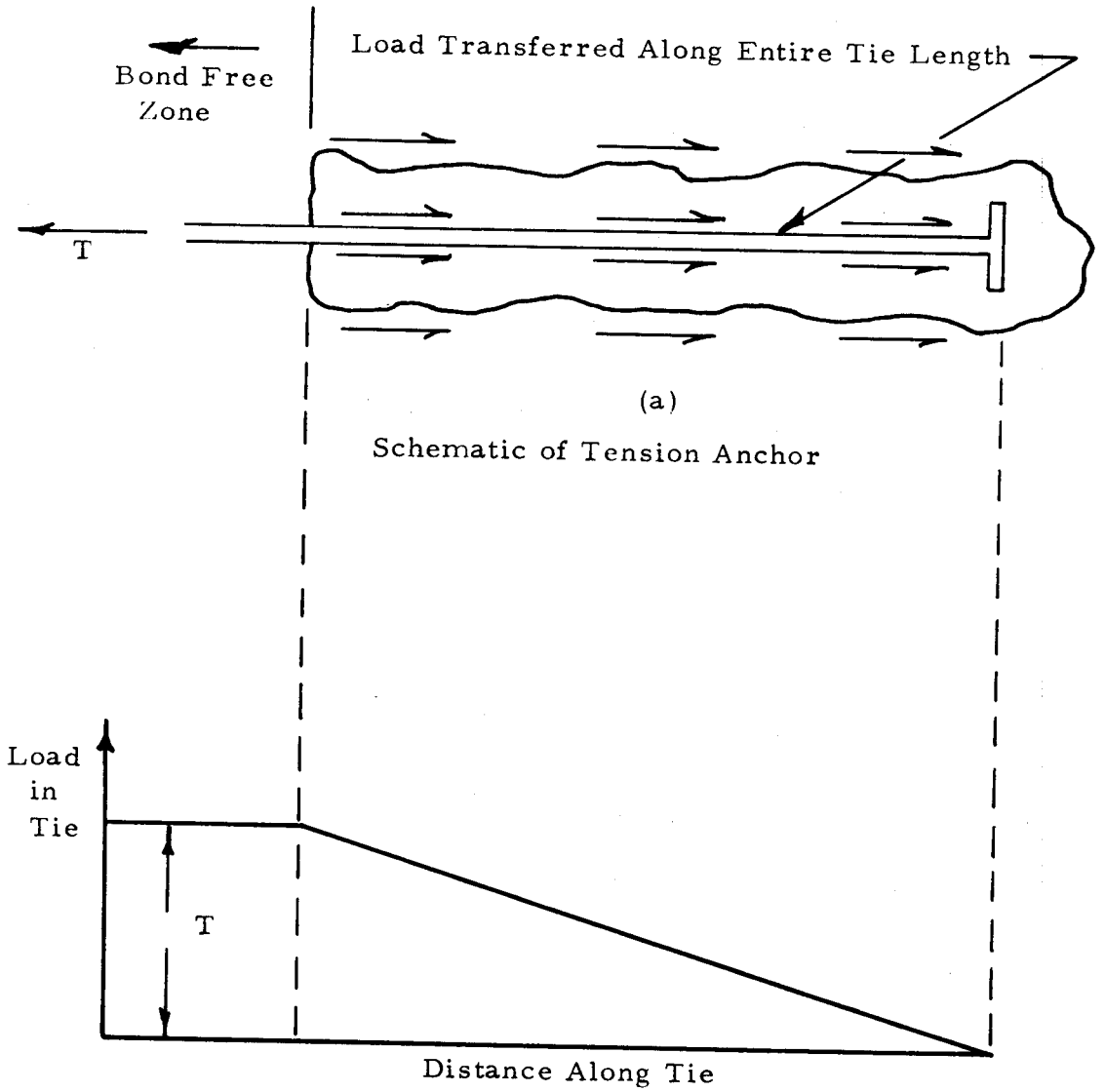


Figure 79. Schematic of tension anchor and resulting load distribution in anchor.

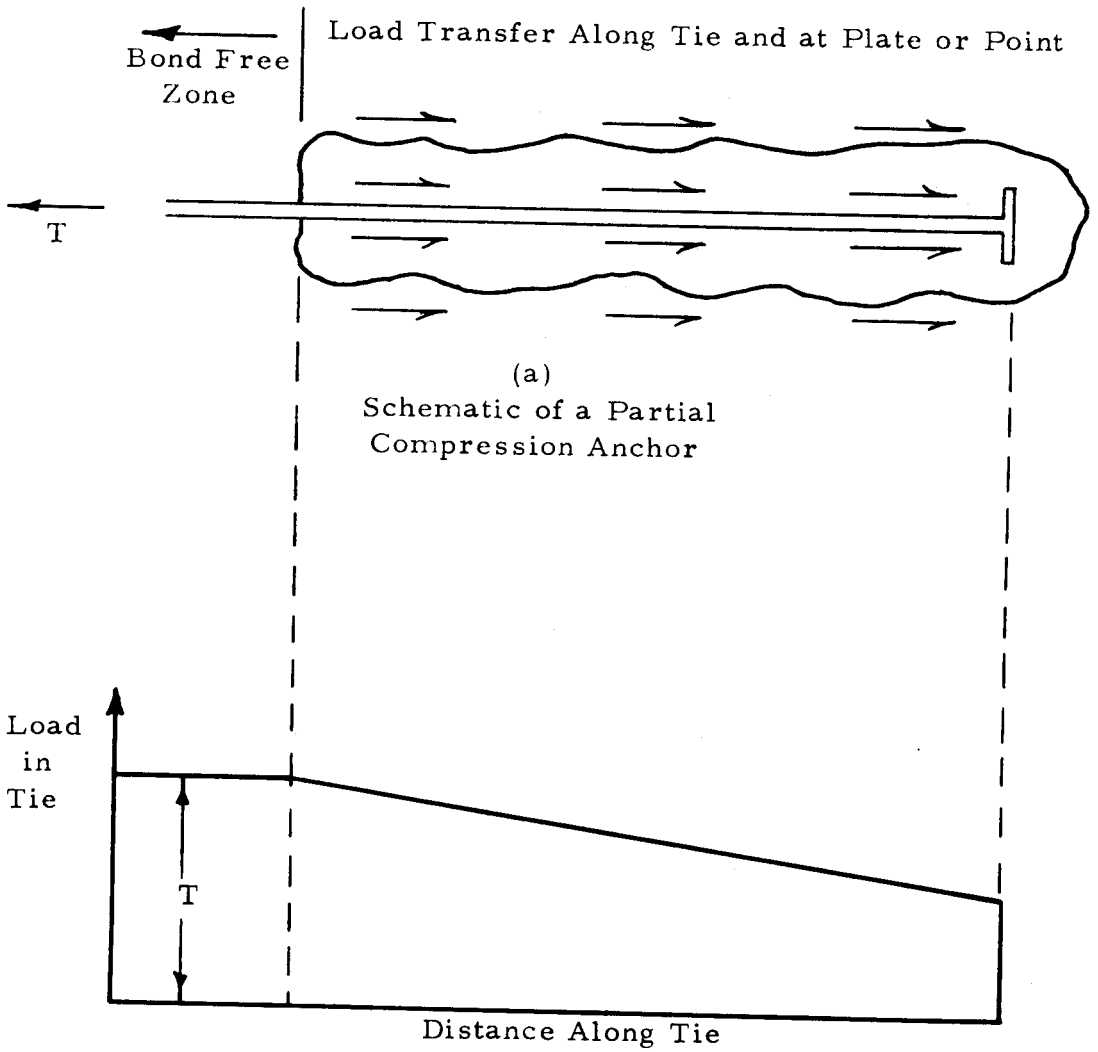


Figure 80. Schematic of partial compression anchor and resulting load distribution in the tie.

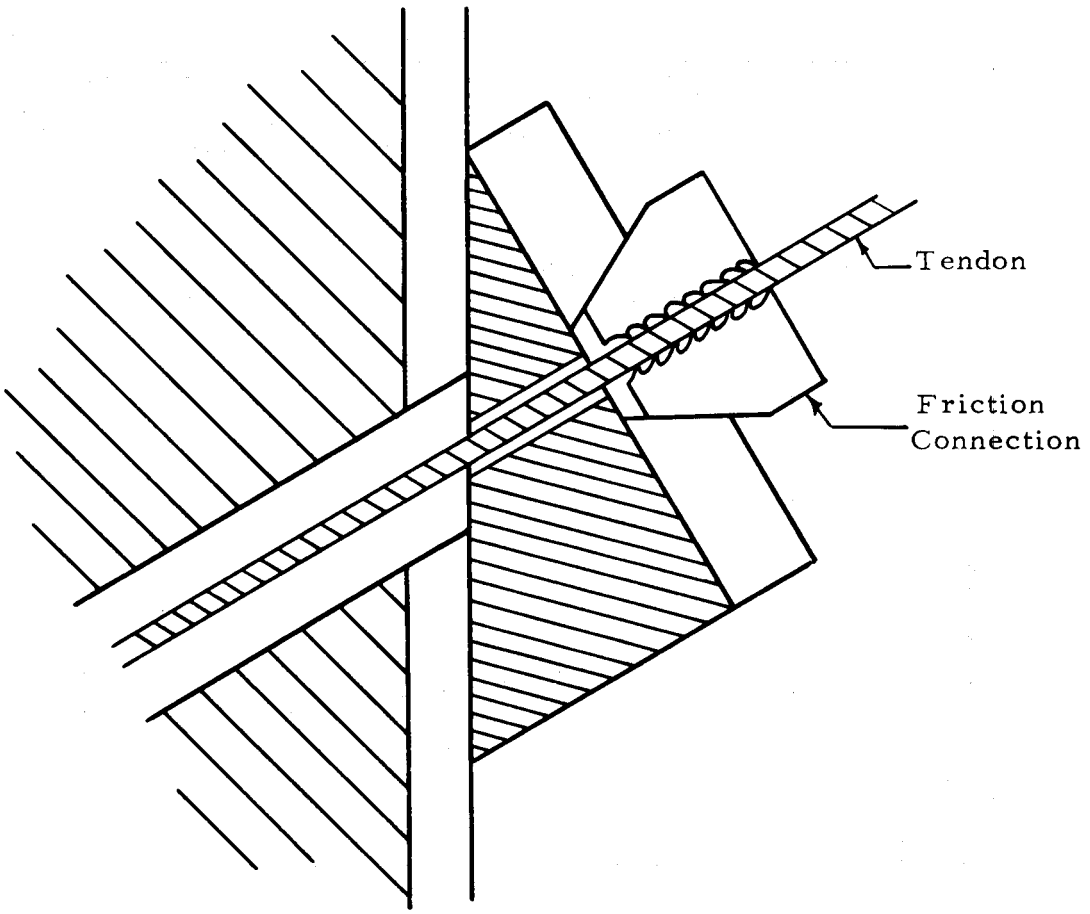


Figure 81. Friction connection used to tie anchor to wall.

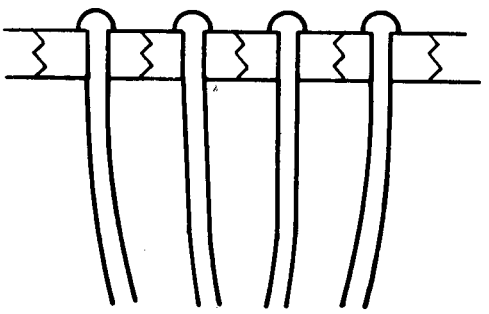
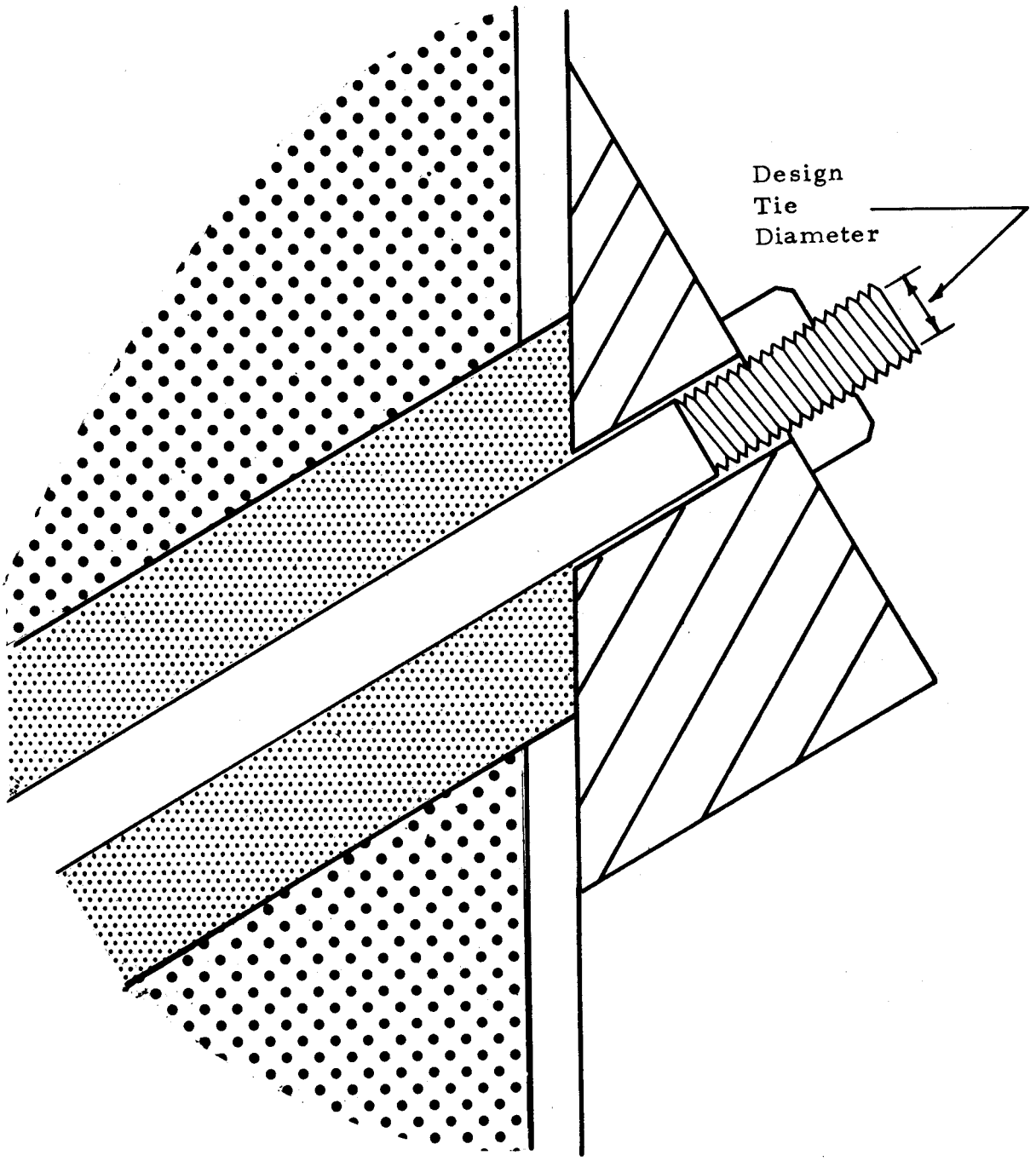


Figure 82. Button-head connection for wire ties.



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 (1) Solid Stem Augers	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^o - 90^o (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^o - 90^o (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^o - 60^o)	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^o - 60^o)	Lean concrete, weak cement grout, or sand. Entire tie 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 Regroutable) (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 lift 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 (690 kN/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. Driving 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/m^2). 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/m^2), 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/m^2]) 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/m^2) to 500 psi (3400 kN/m^2) 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).

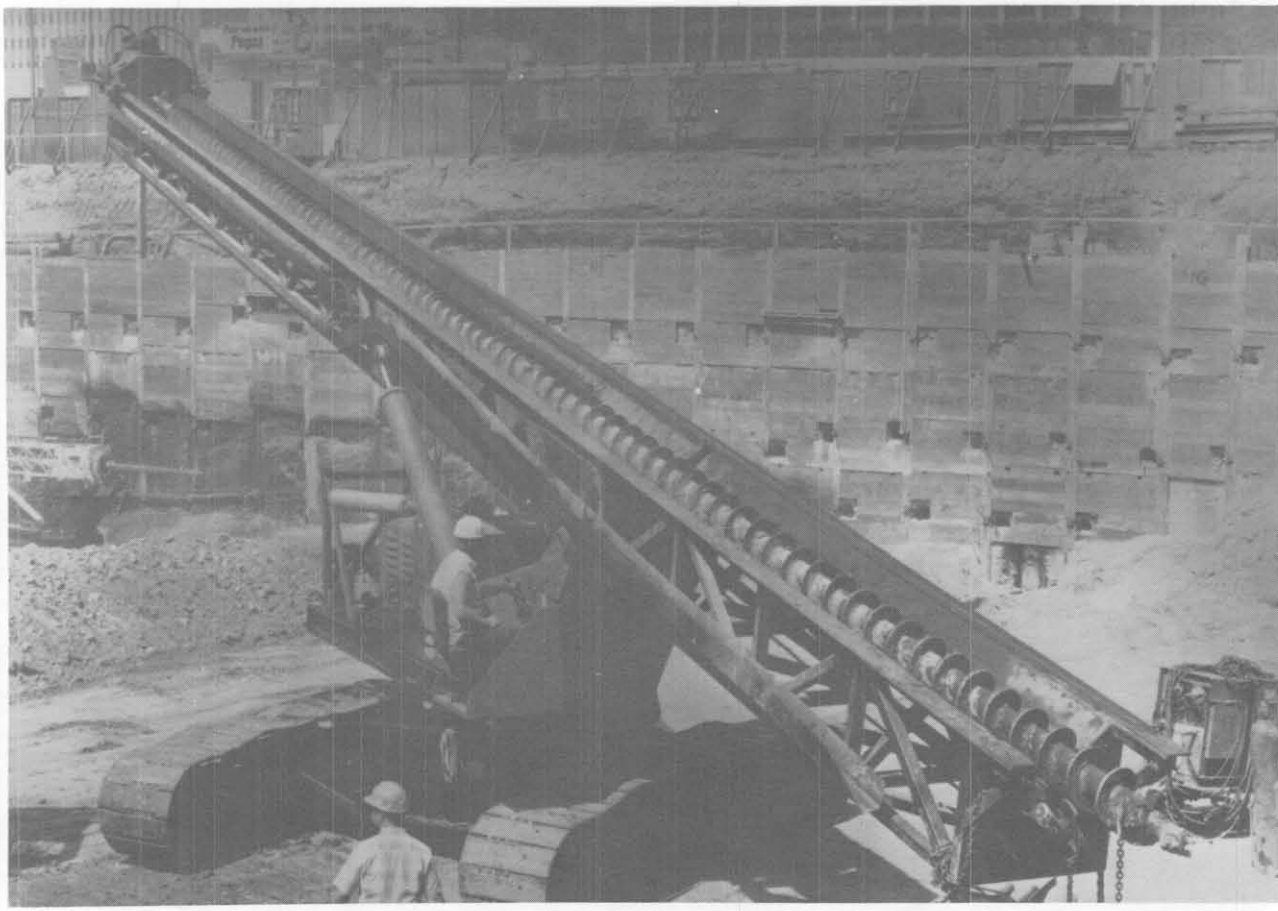


Figure 85. Crawler mounted auger rig.
(Courtesy of Spencer, White, and Prentis).



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).

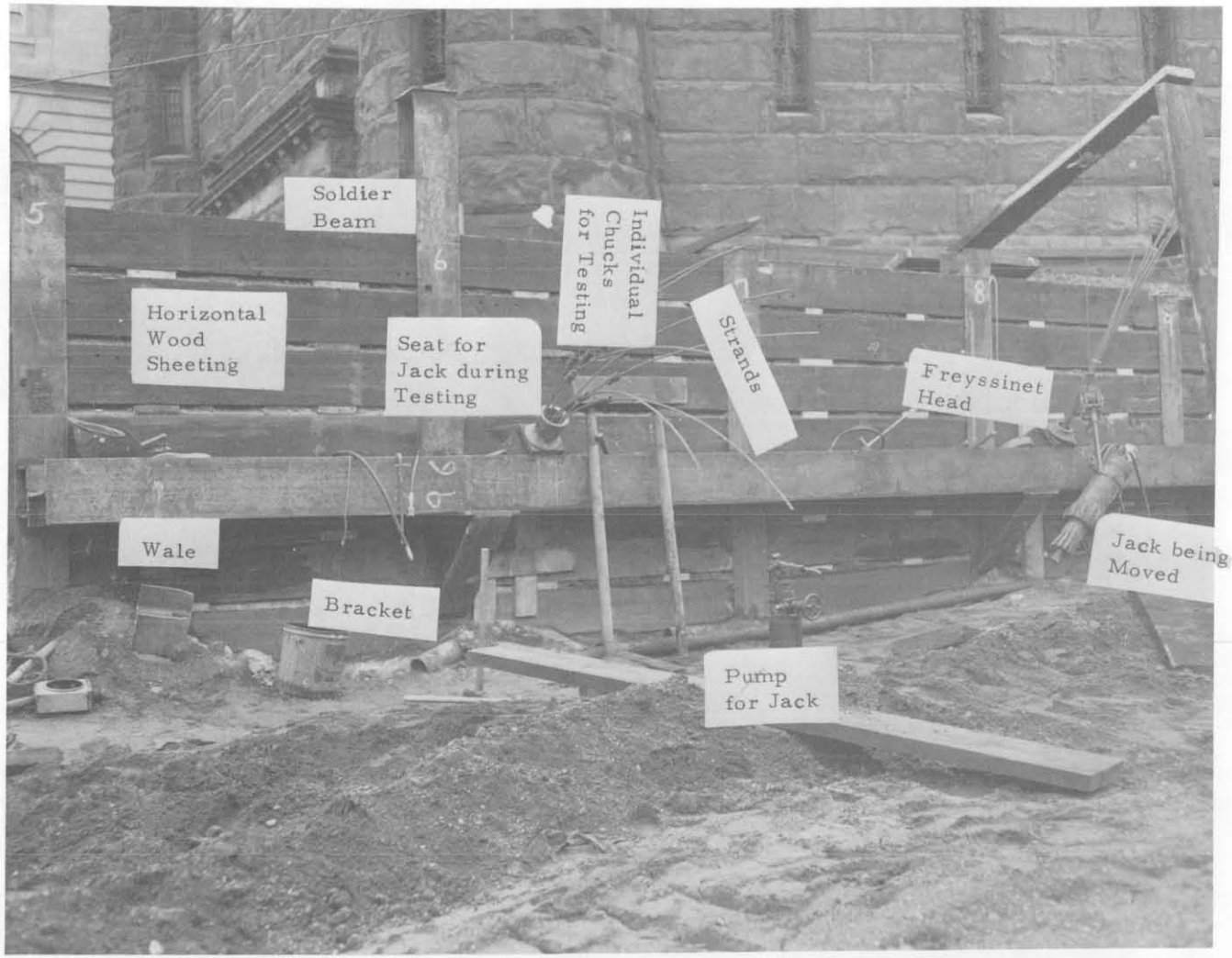


Figure 90. Tieback stressing details.
(Courtesy of Spencer, White, and Prentis).



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

Soil and Site Conditions	Load	Remarks
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)
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)

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 nautre 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.

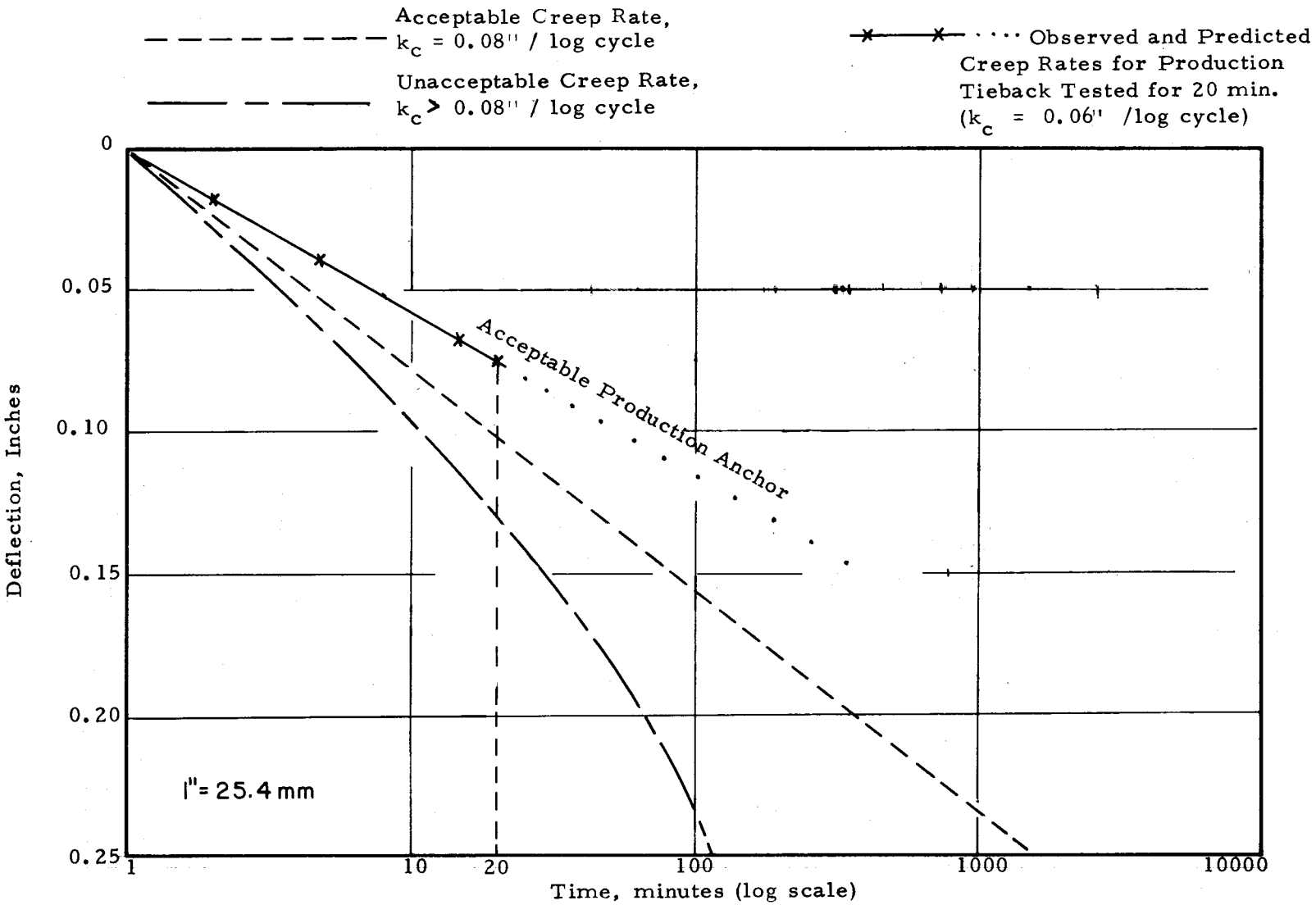


Figure 92. Example of recommended method of recording creep data.

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 repect 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"

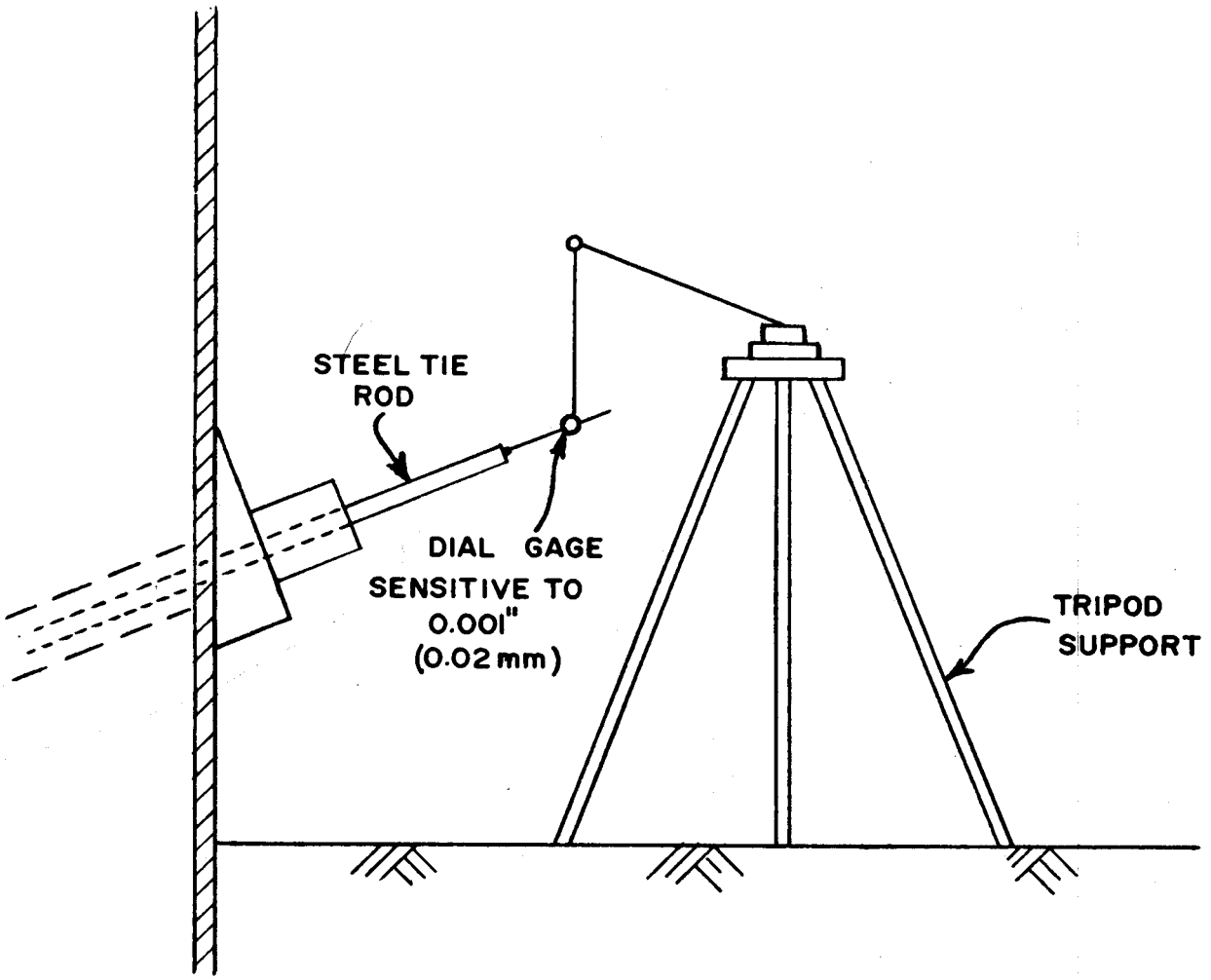


Figure 93. Schematic of method that can be used to detect creep movements in anchors.

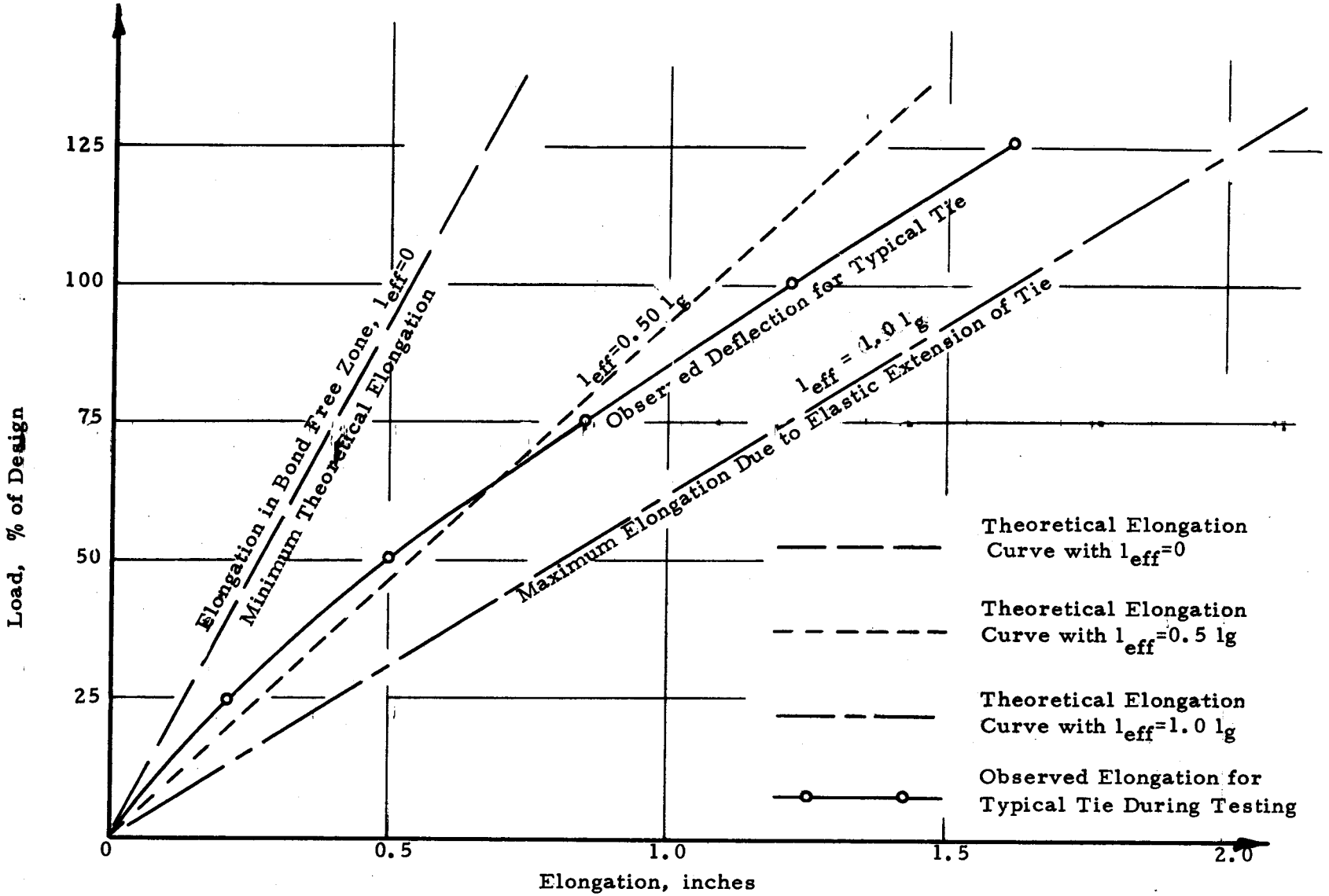


Figure 94. Typical plot of load vs. elongation during test loading.

in the grouted zone. The "effective length" can vary from zero ($l_{eff} = 0$) to the length of the tie in the grouted zone ($l_{eff} = l_g$). 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 quantites in this evaluatory method.

$$\delta_b = \frac{T l_b}{AE}$$

$$\delta_g = \frac{\int T \, dl_g}{AE} \quad \text{or} \quad \delta_g = \frac{T l_{eff}}{AE}$$

$$\delta_E = \delta_b + \delta_g$$

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

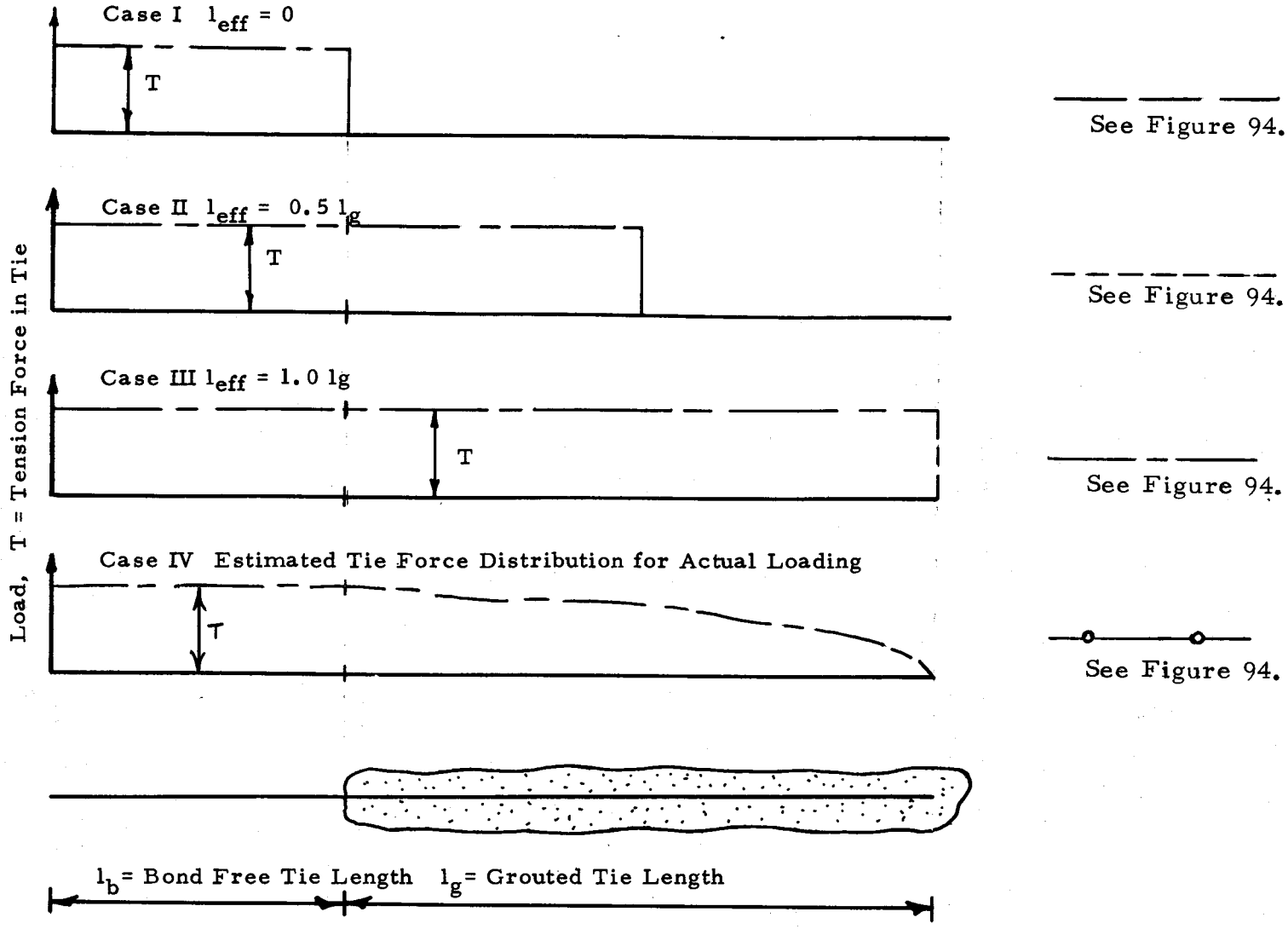


Figure 95. Idealized load distribution in tie.

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 (Δl_1) from 80 - 100 percent of design load ($\frac{\Delta l_2}{\Delta l_1} \leq 1.16$).

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. Therfore, 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 2inches 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.

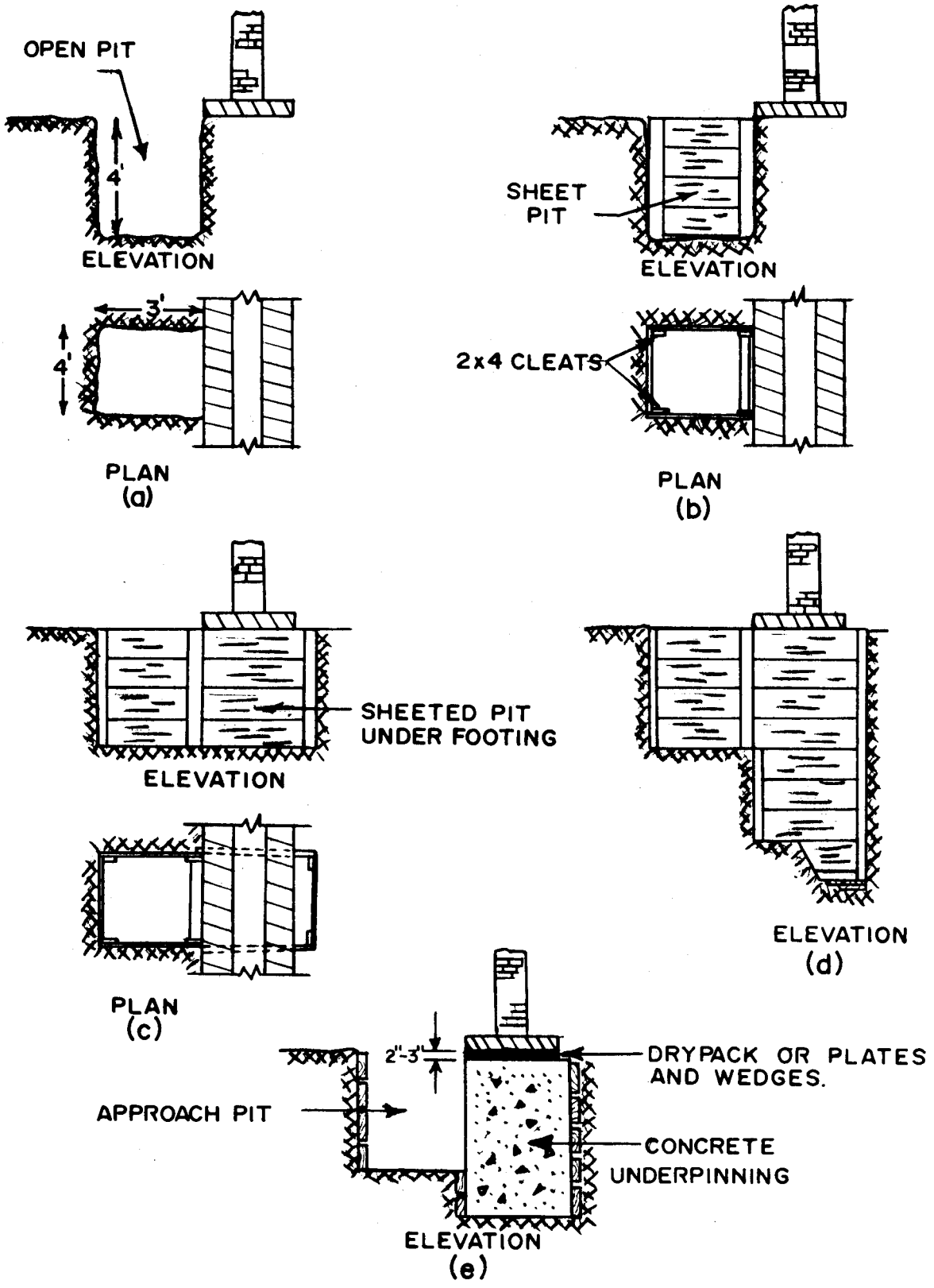


Figure 96. Pit or pier underpinning.

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

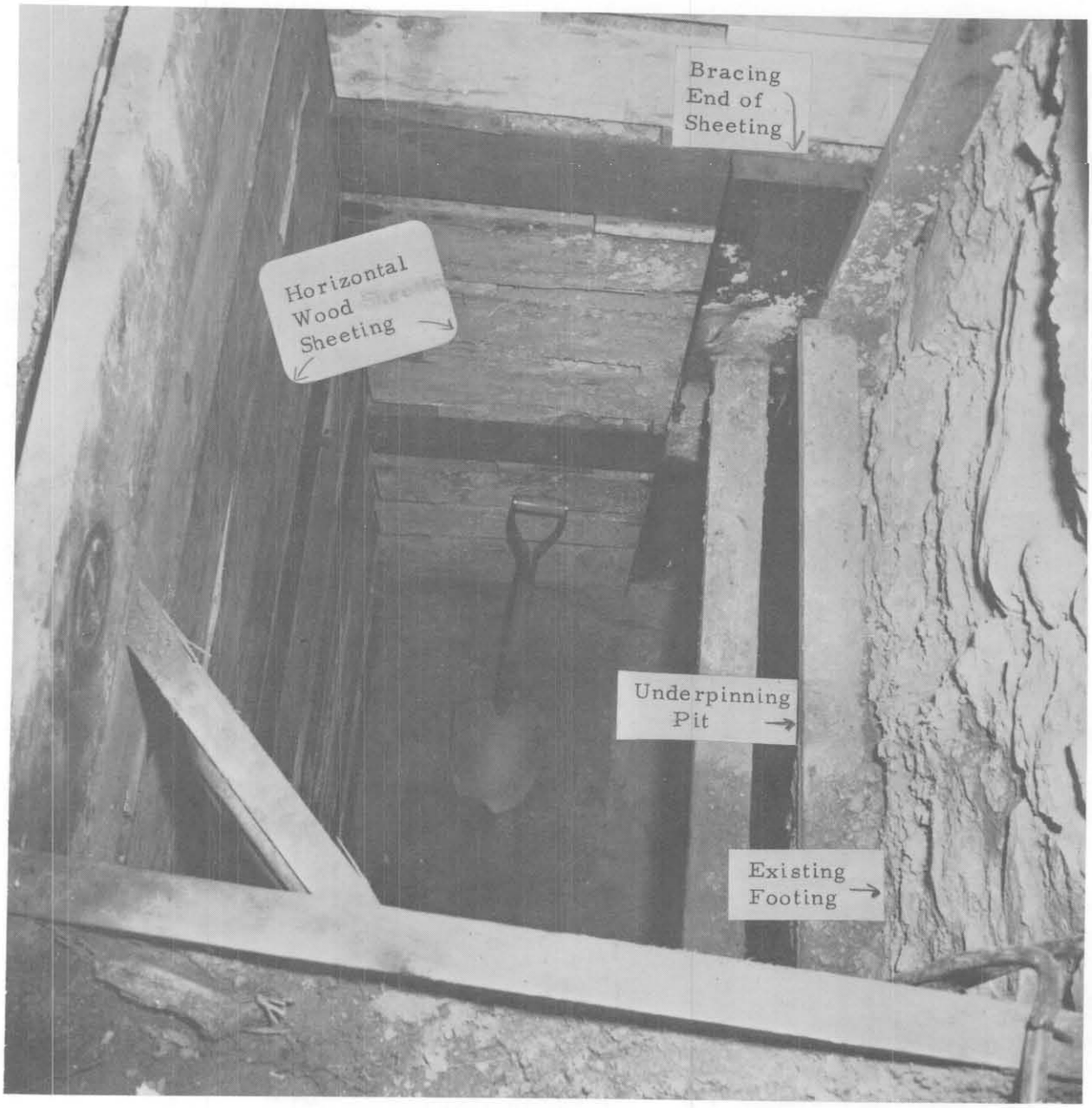
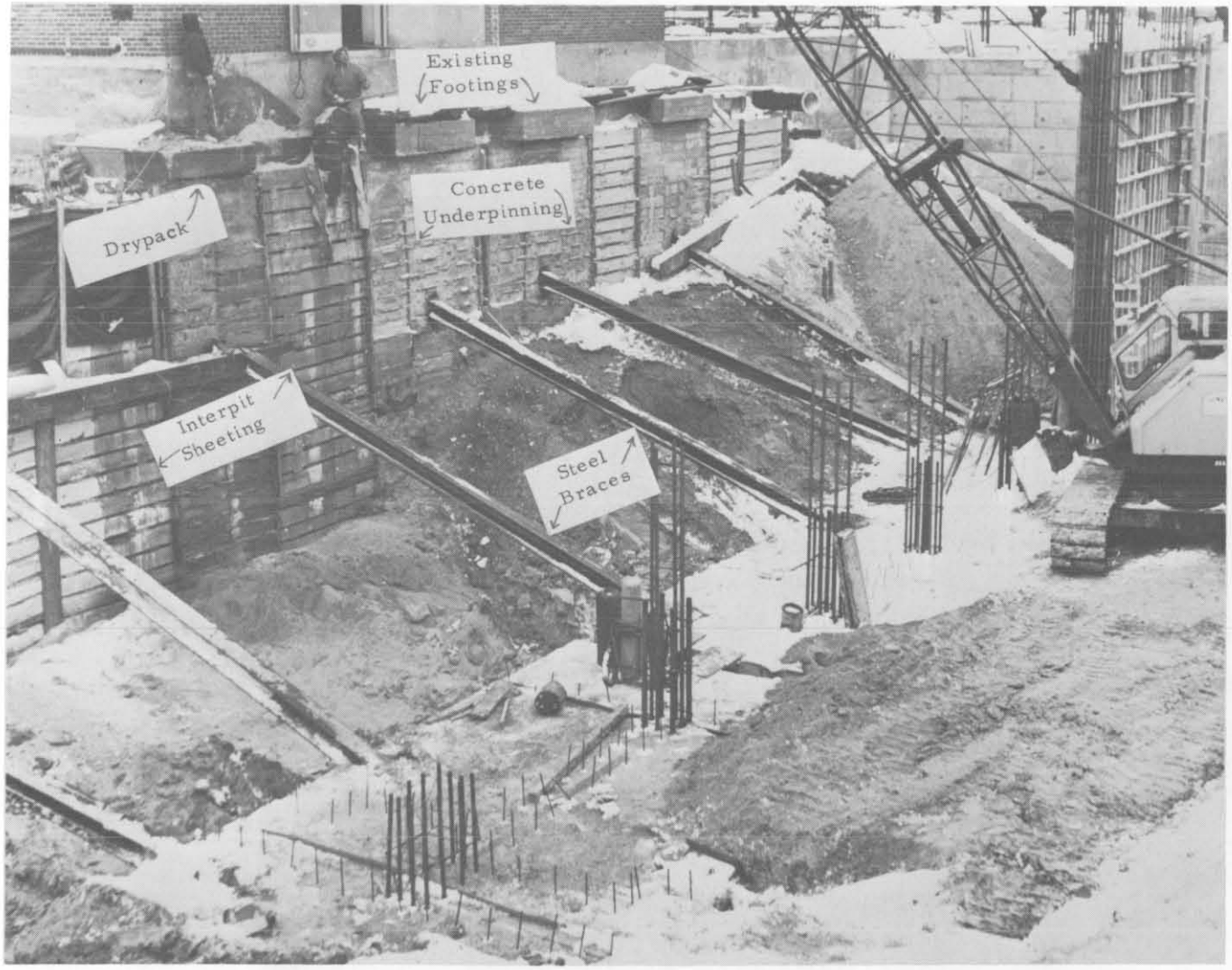
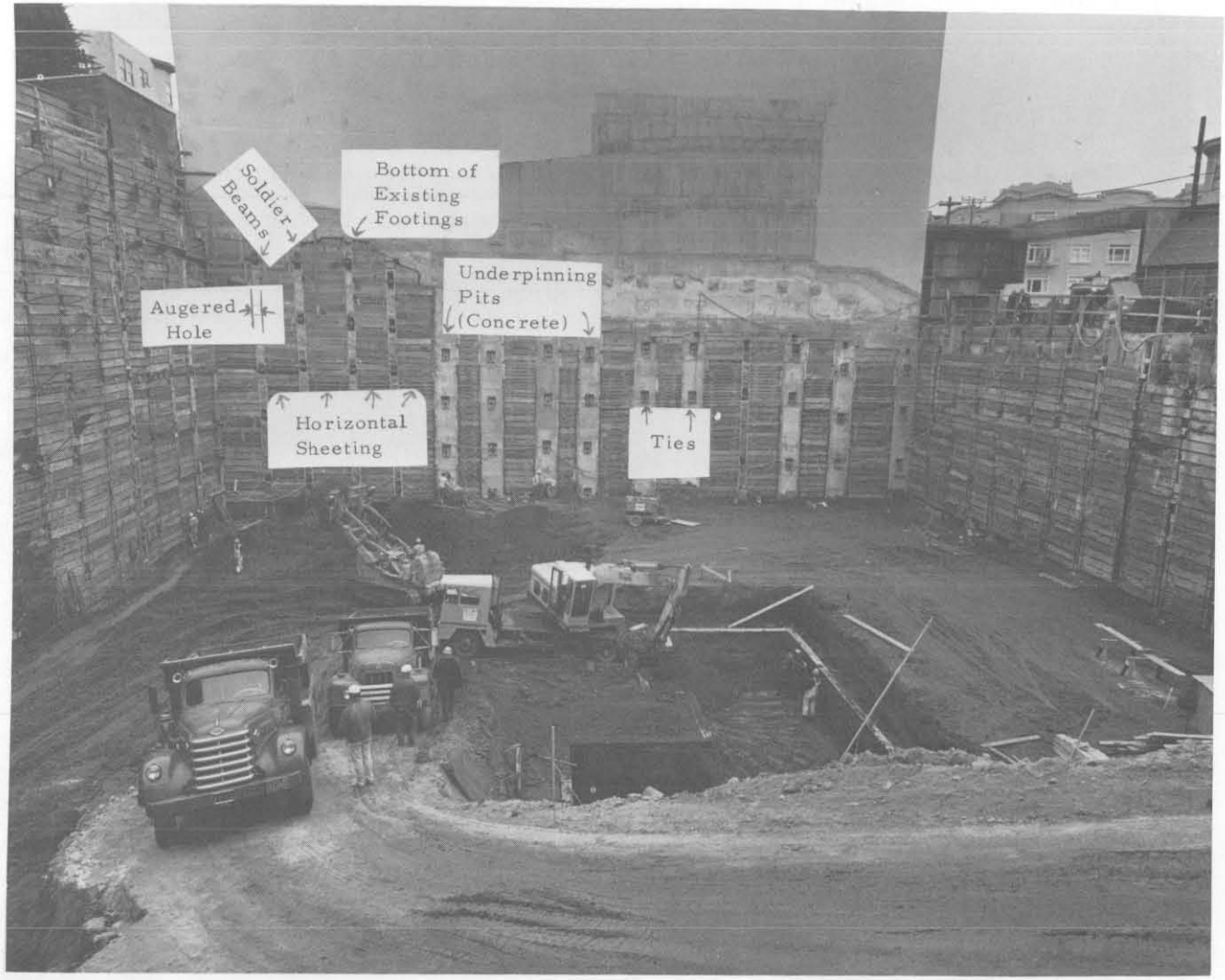


Figure 97. Details of pit underpinning.
(Courtesy of Spencer, White, and Prentis).



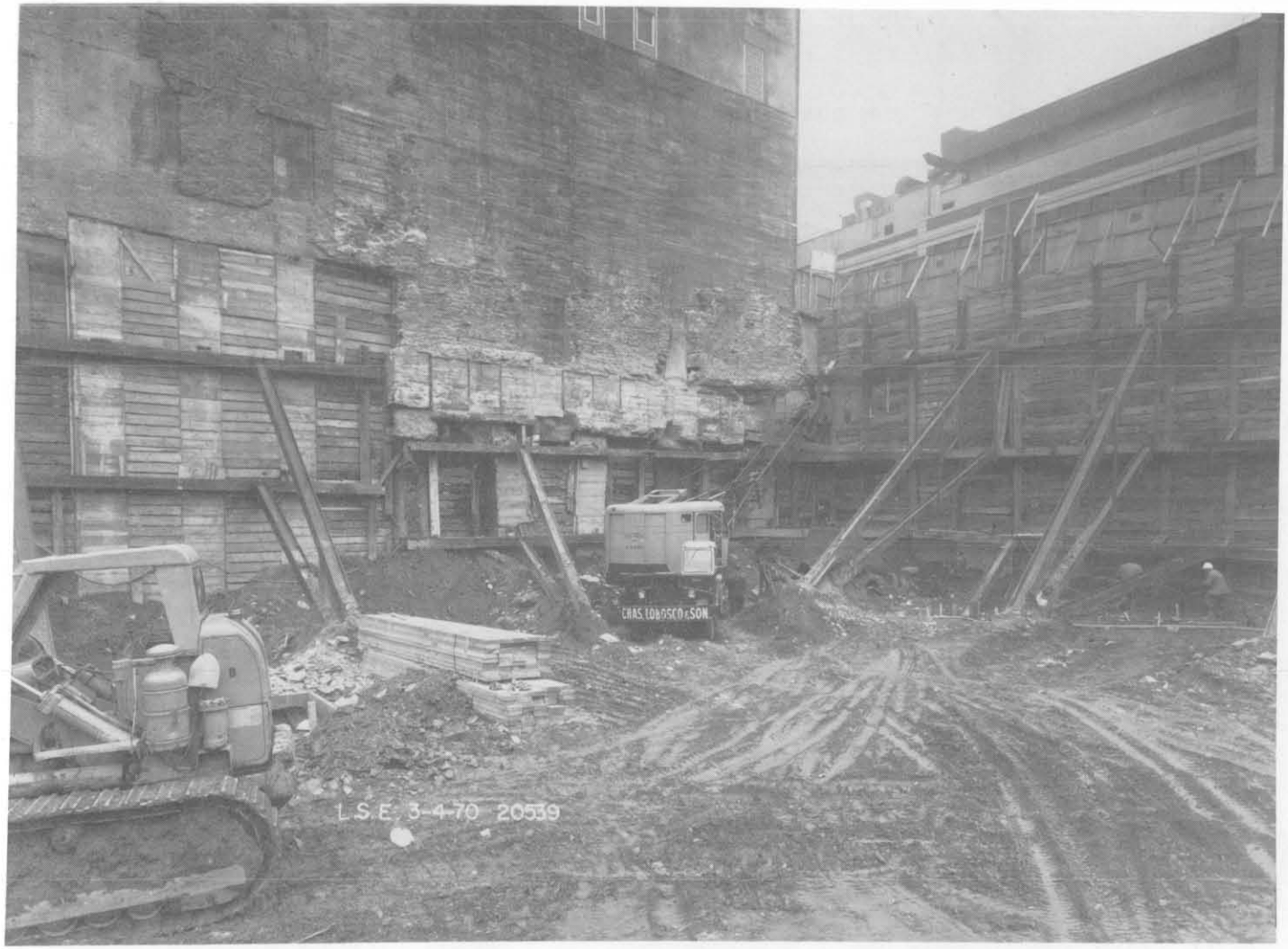
Note: Bracing for lateral support.

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).



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 $1\frac{1}{2}$ 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.

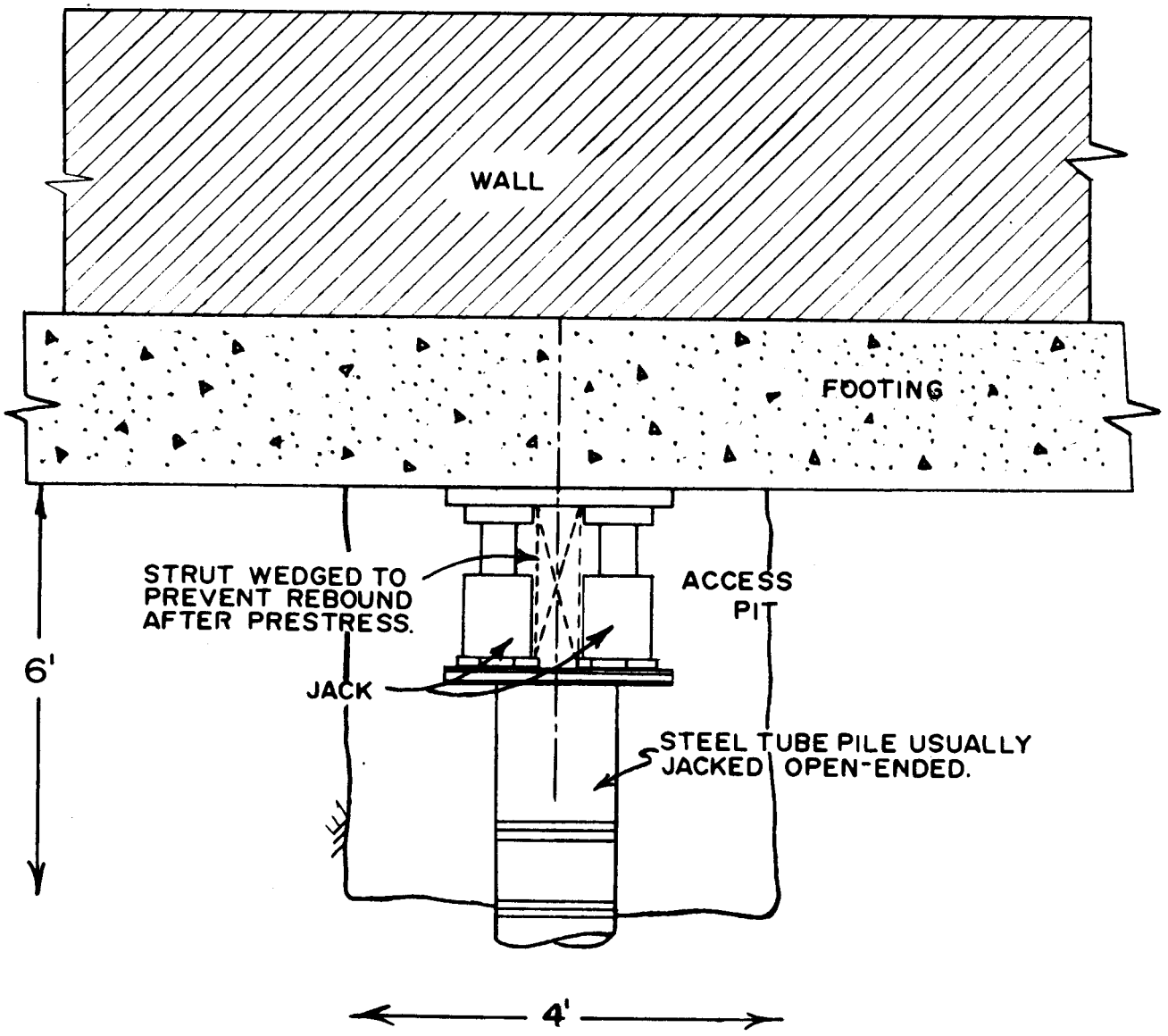


Figure 103. Jacked pile installation.

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.

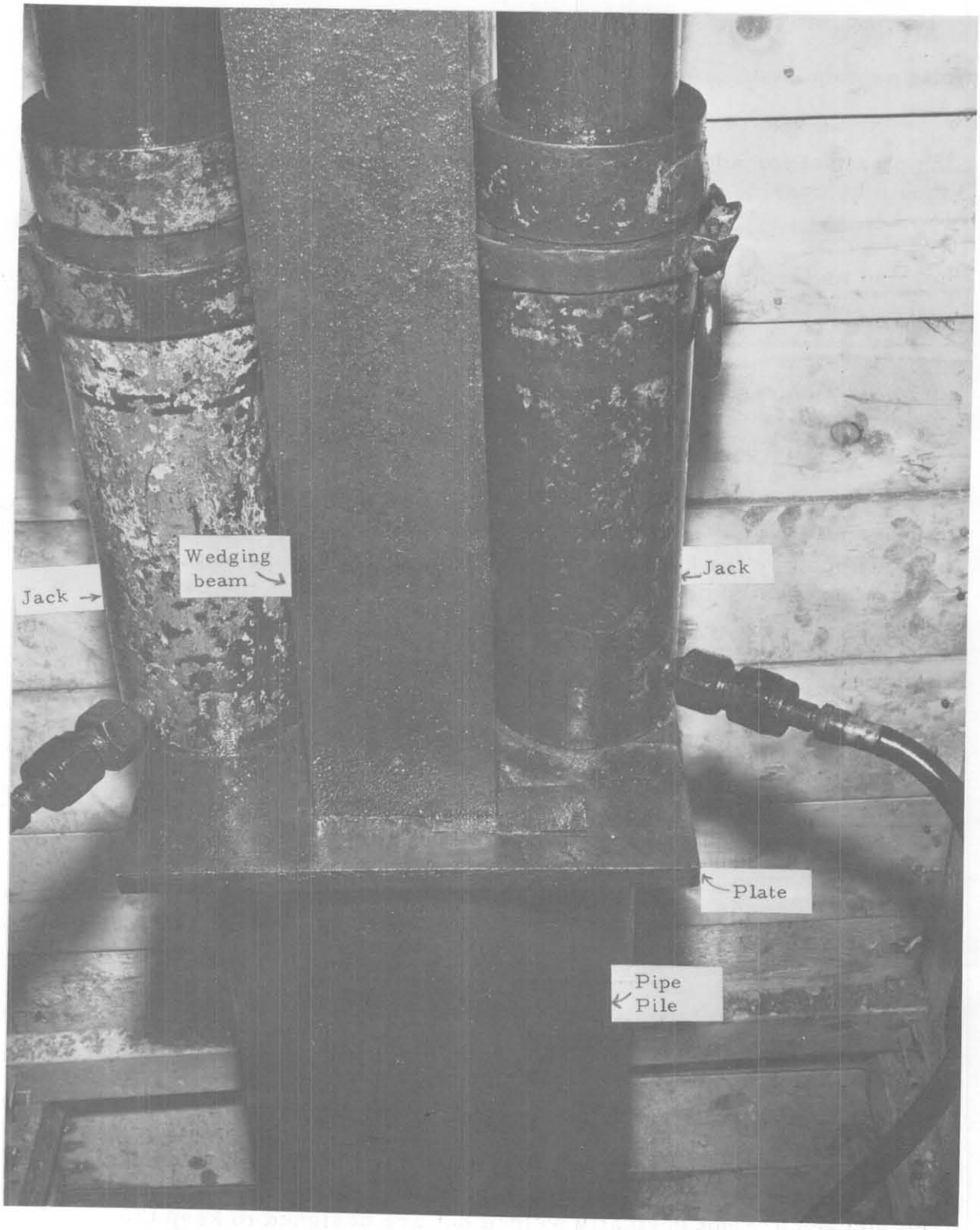


Figure 104. Prestressing of underpinning pile.
(Courtesy of Spencer, White, and Prentis).

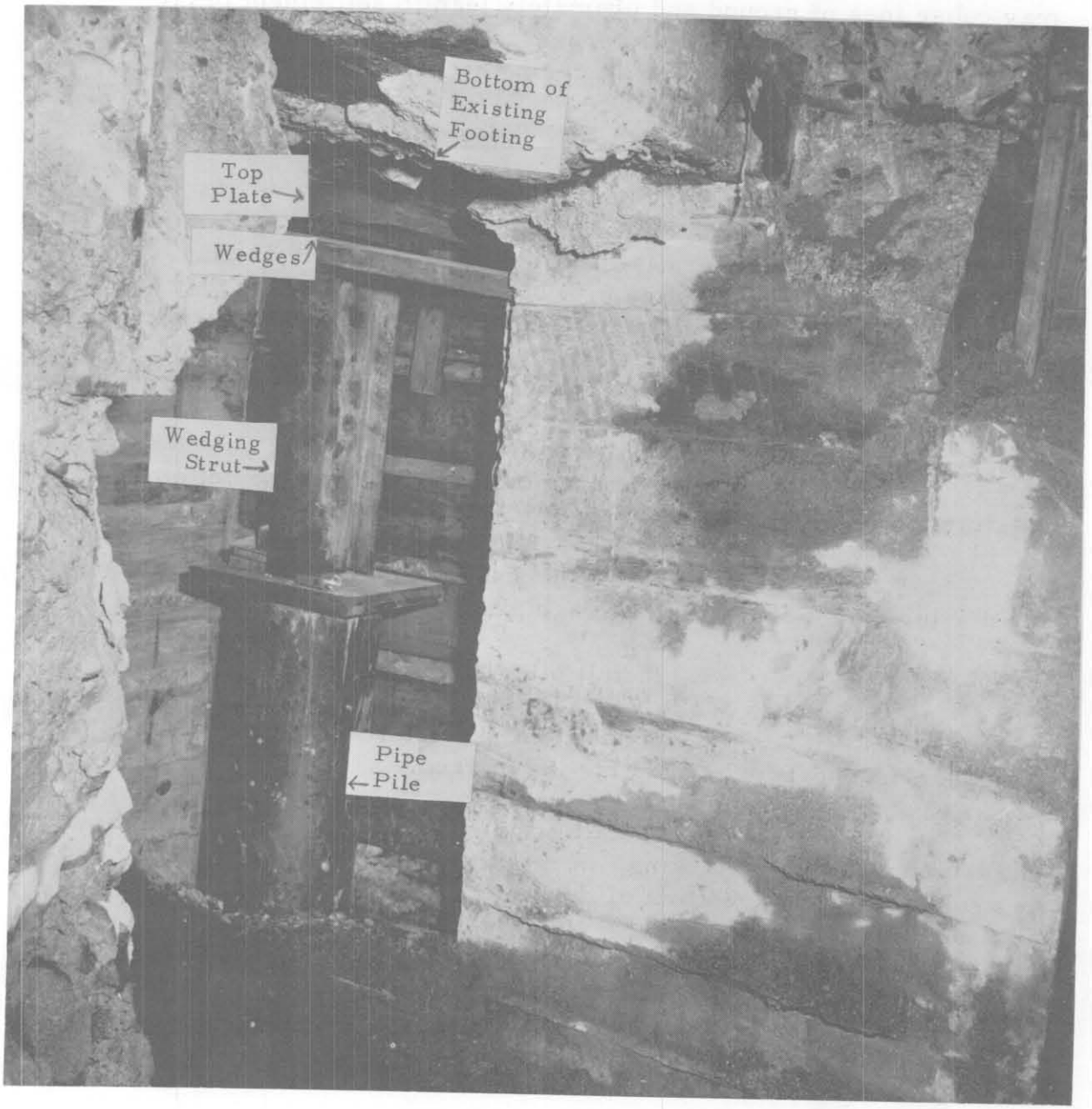


Figure 105. Jacked pile underpinning details.
(Courtesy of Spencer, White, and Prentis).

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.

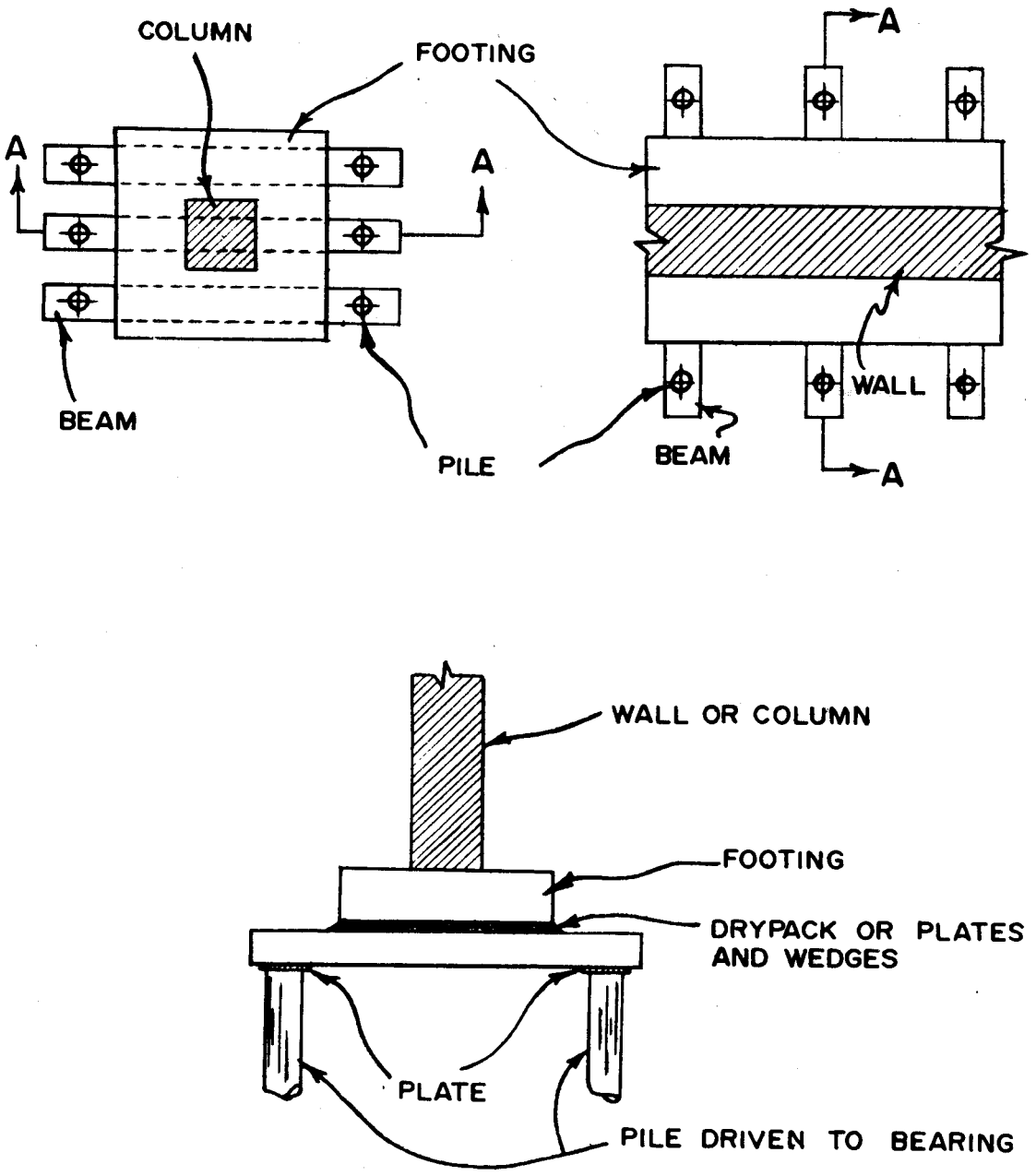


Figure 106. Piles driven alongside footing, support by beam.

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.

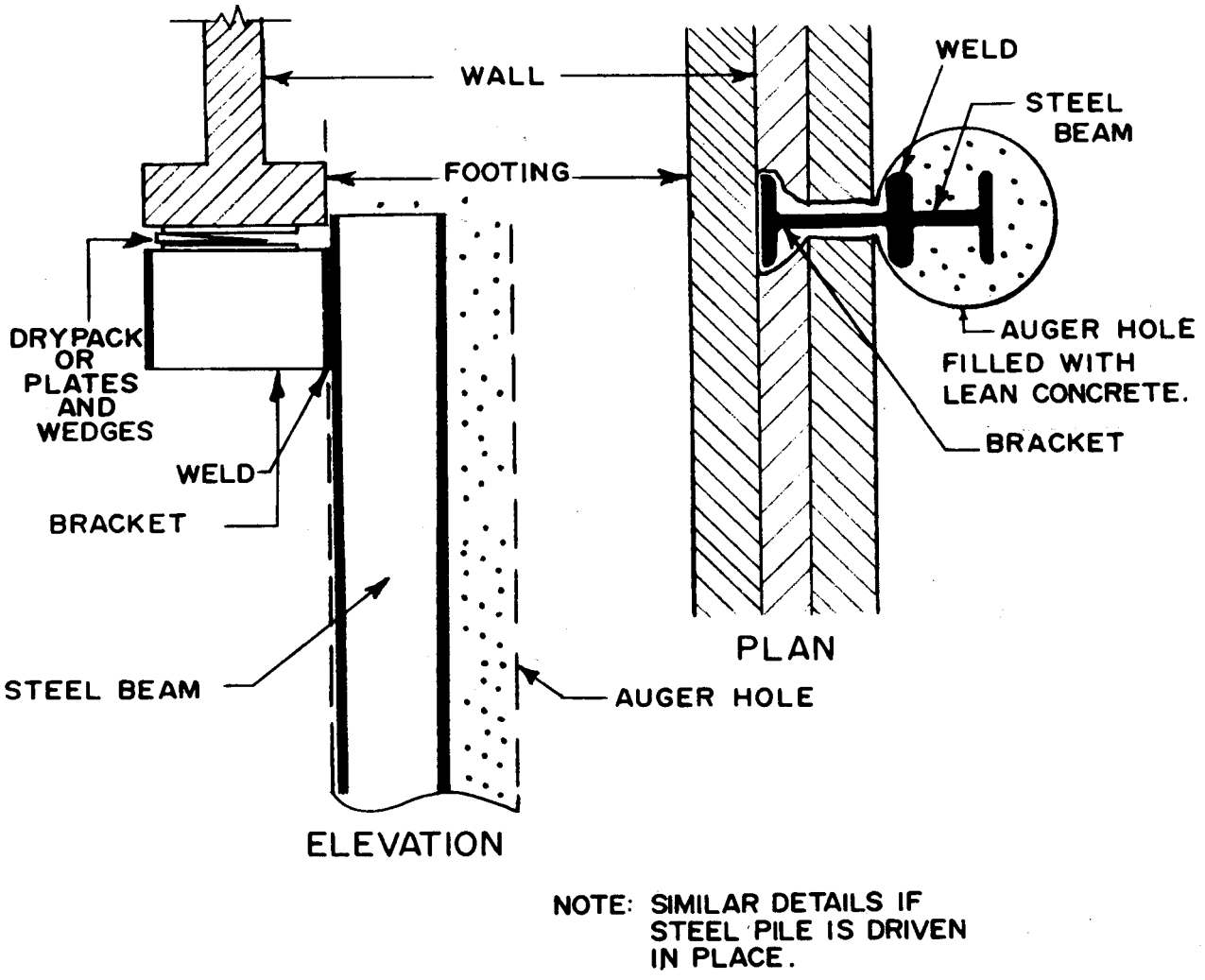


Figure 107. Steel pile with steel bracket.

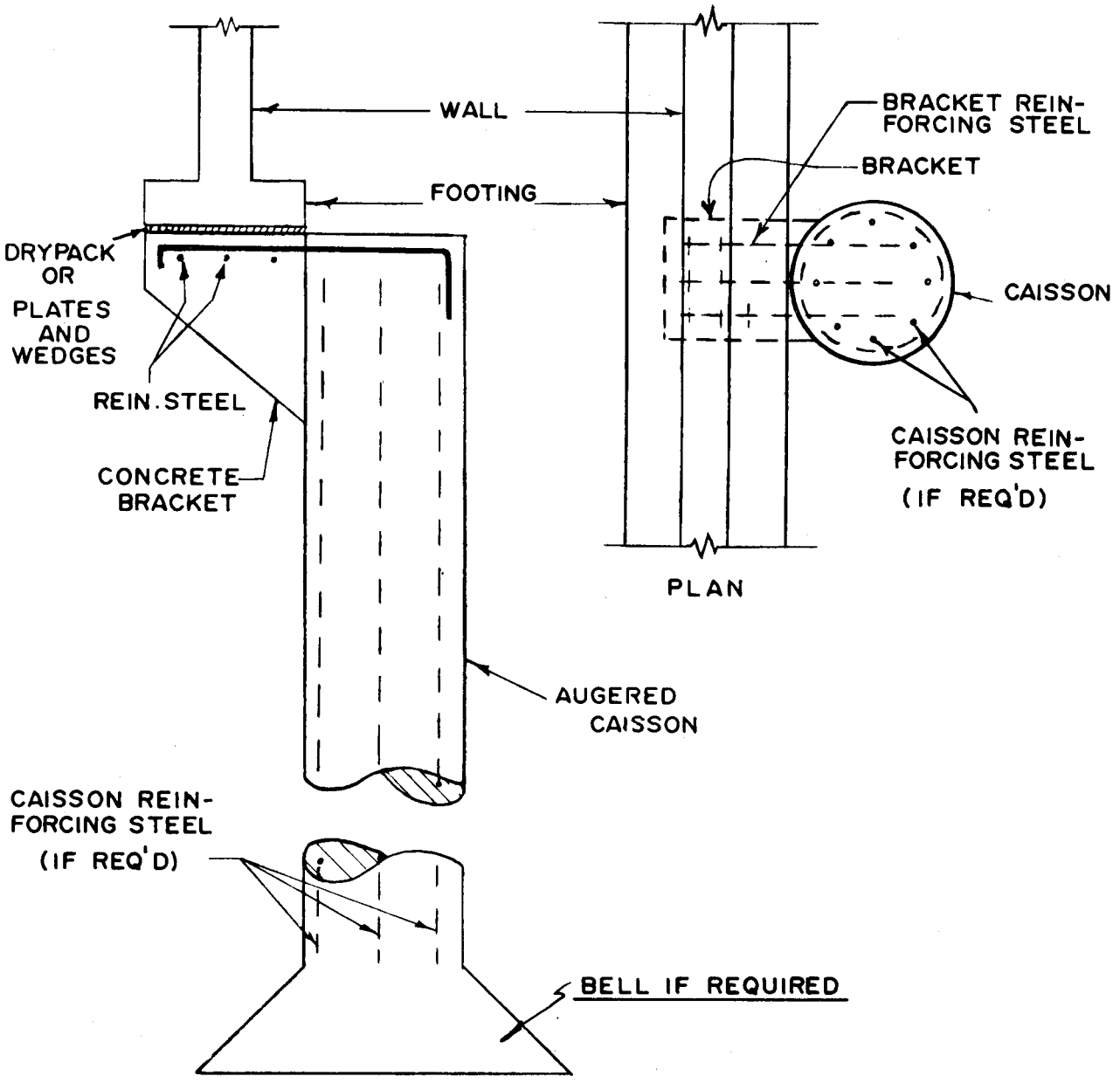


Figure 108. Augered concrete caisson with concrete bucket.

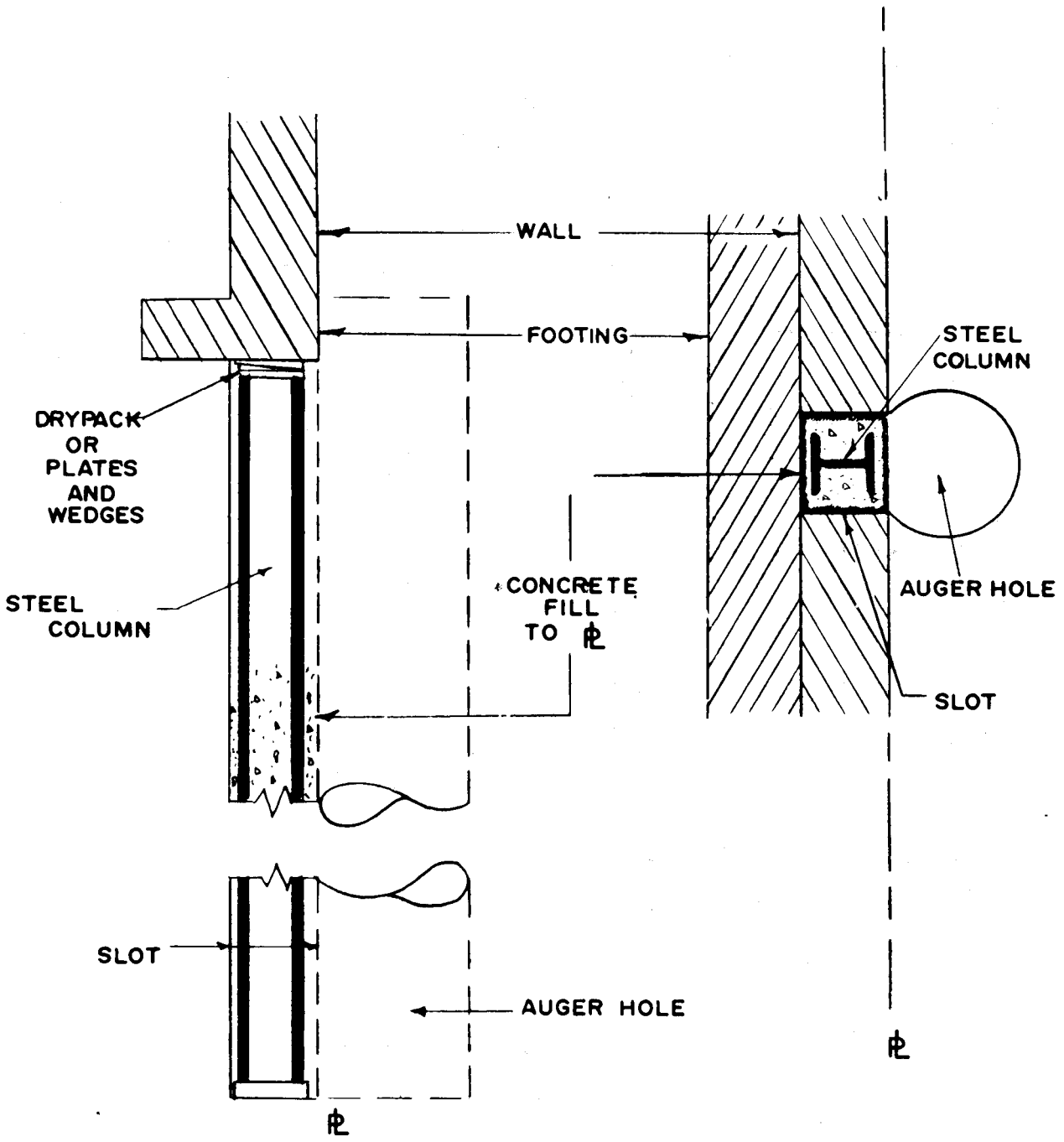


Figure 109. Auger hole with pile installed in slot.

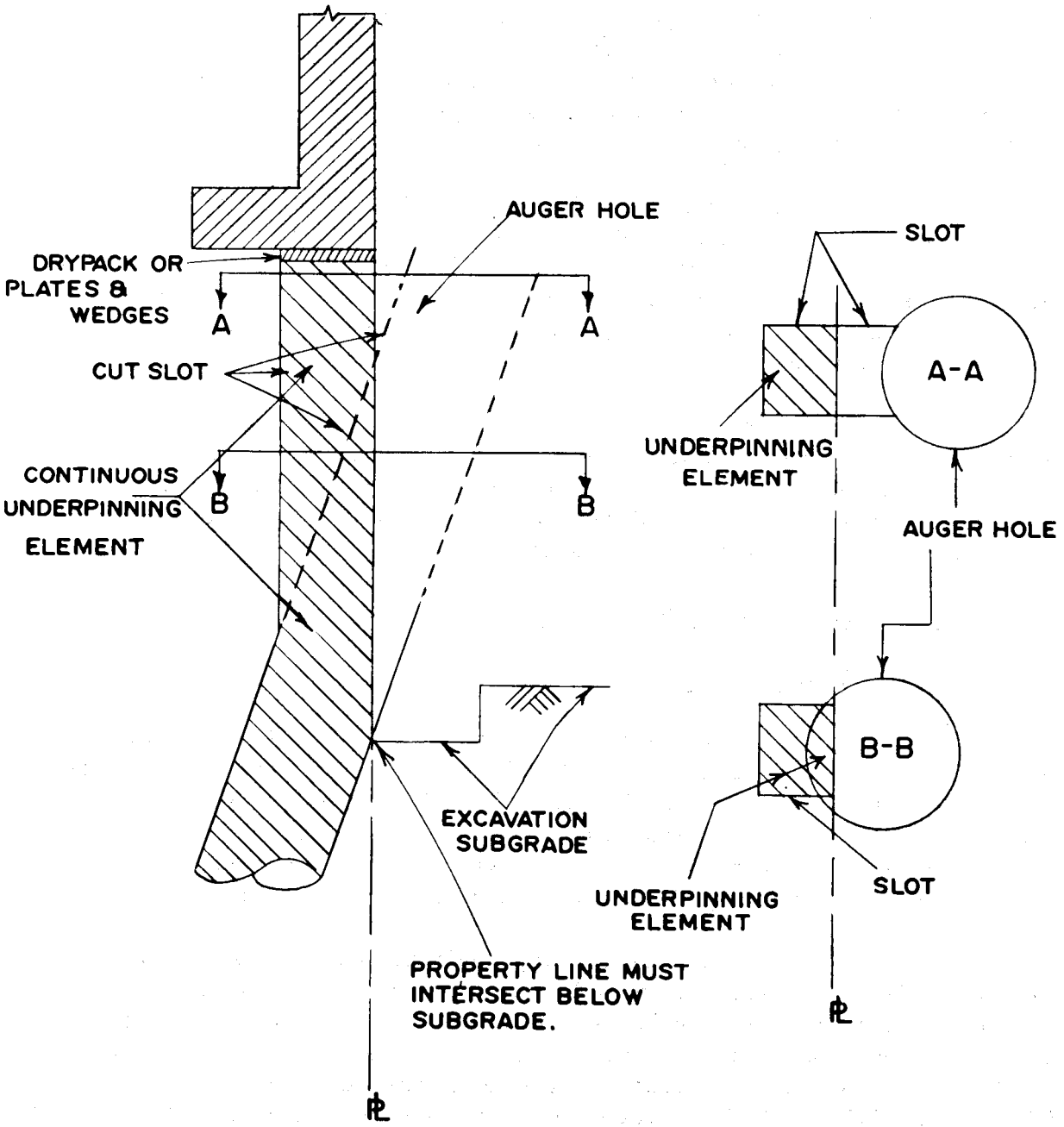


Figure 110. Battered pile underpinning.

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).

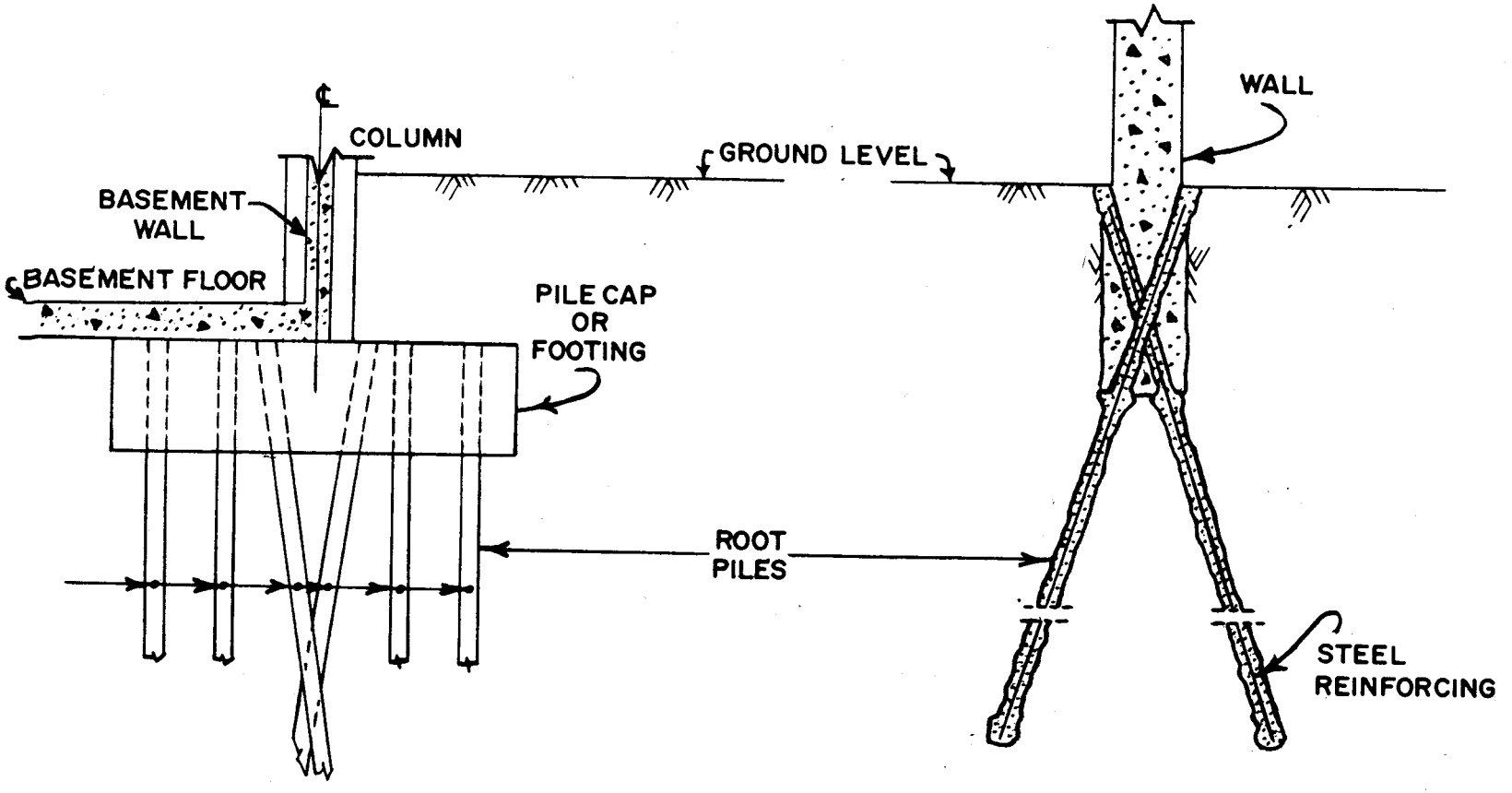


Figure 111. Typical uses of root piles (pali radice).

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 Max. Load ρ, inches	Settlement Modulus (1) k, $\frac{in\text{-}ft^2}{ton}$	Soil Type (2)	Location
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) $k = \frac{\rho}{P/L'D} = \frac{L'D}{P}$

(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 Max. Load ρ, inches	Settlement Modulus (1), k, $\frac{\text{in-ft}^2}{\text{ton}}$	Soil Type (2)	Location
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 Railrod, 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).

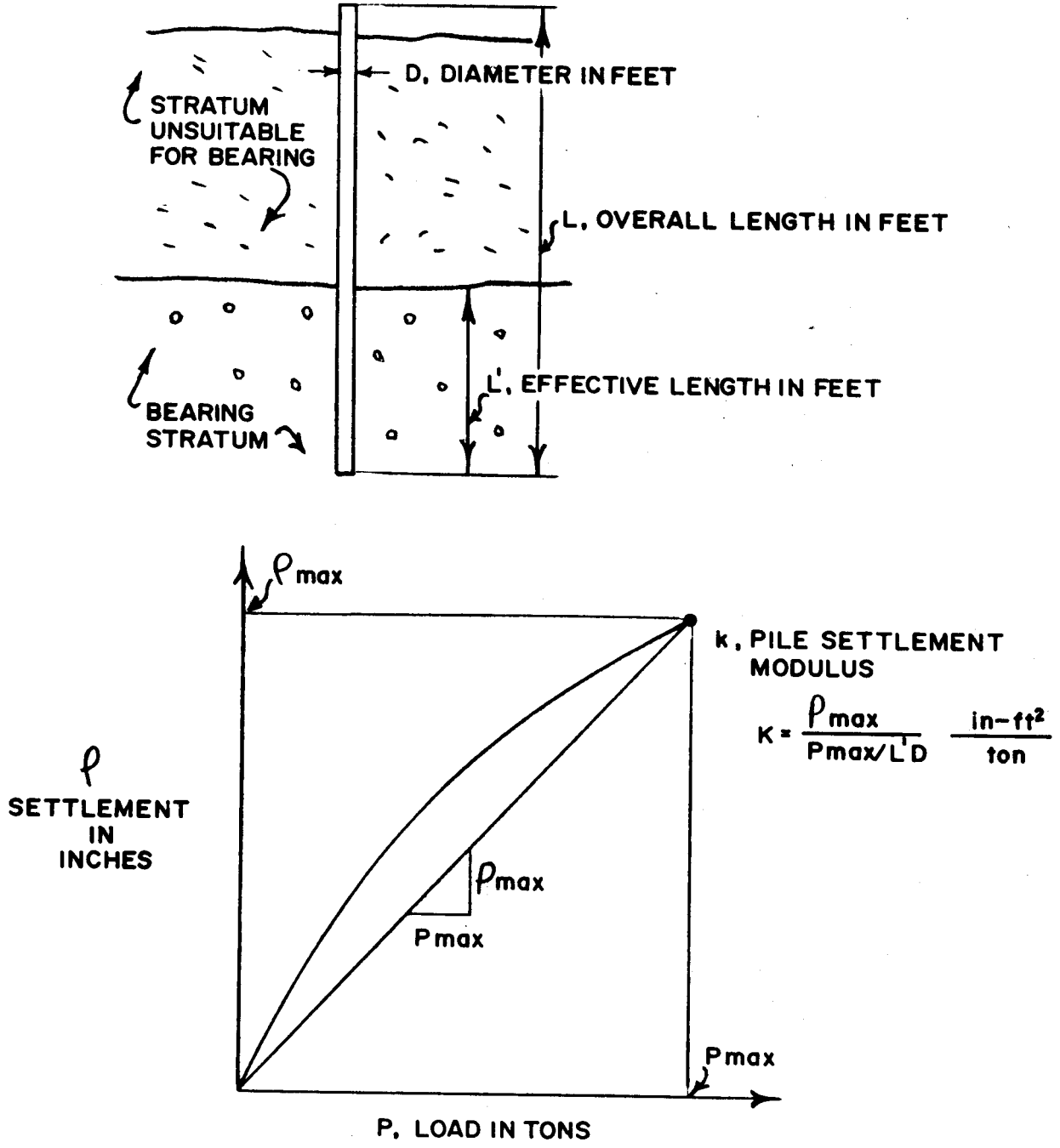


Figure 112. Development of pile settlement modulus.

$$\rho \sim P, \frac{1}{\pi DL'}$$

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-ft^2/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-ft^2/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 treansferred 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:

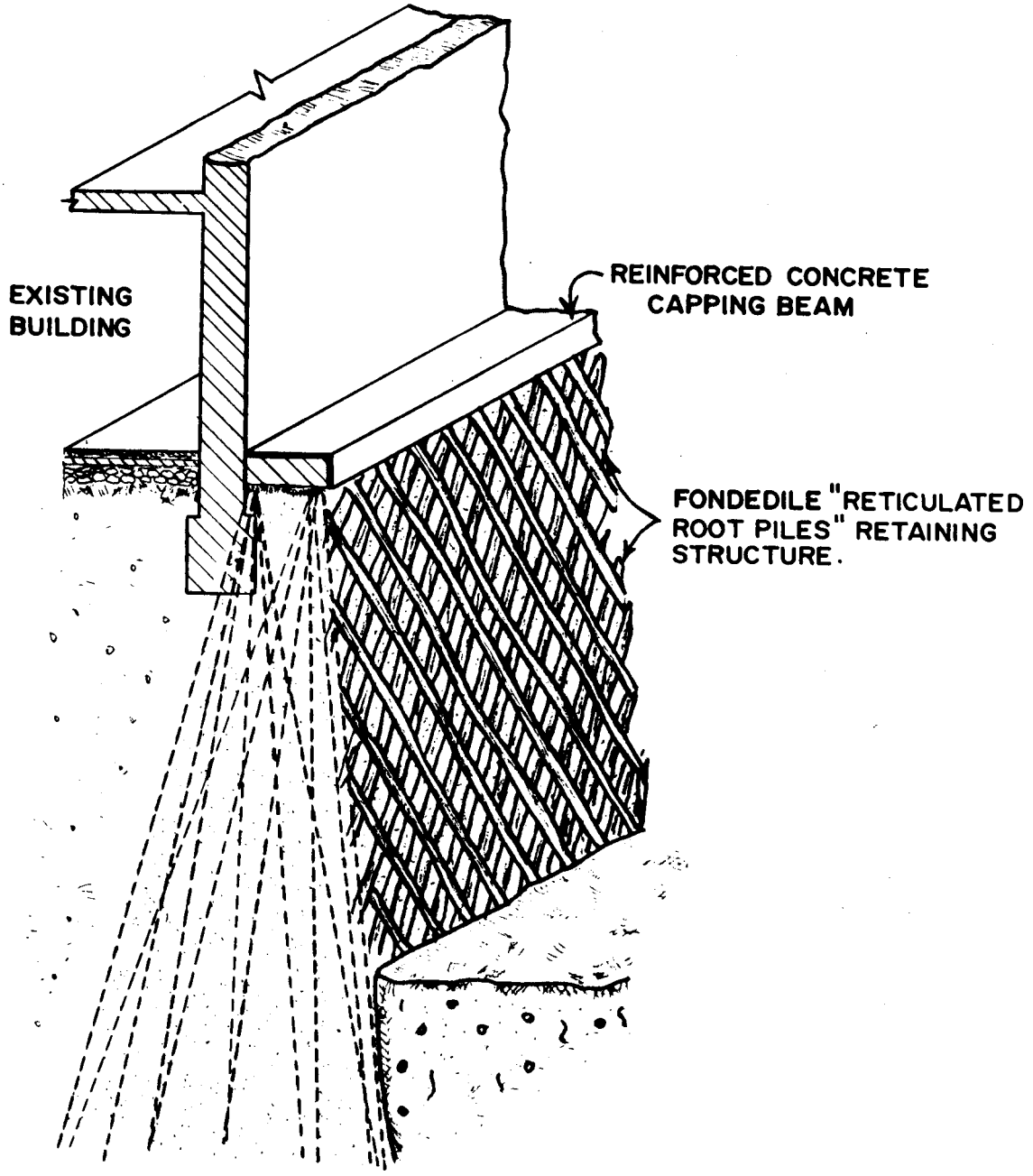


Figure 113. Schematic showing principle of reticulated root piles.
(Courtesy of Warren-Fondedile, Inc.).

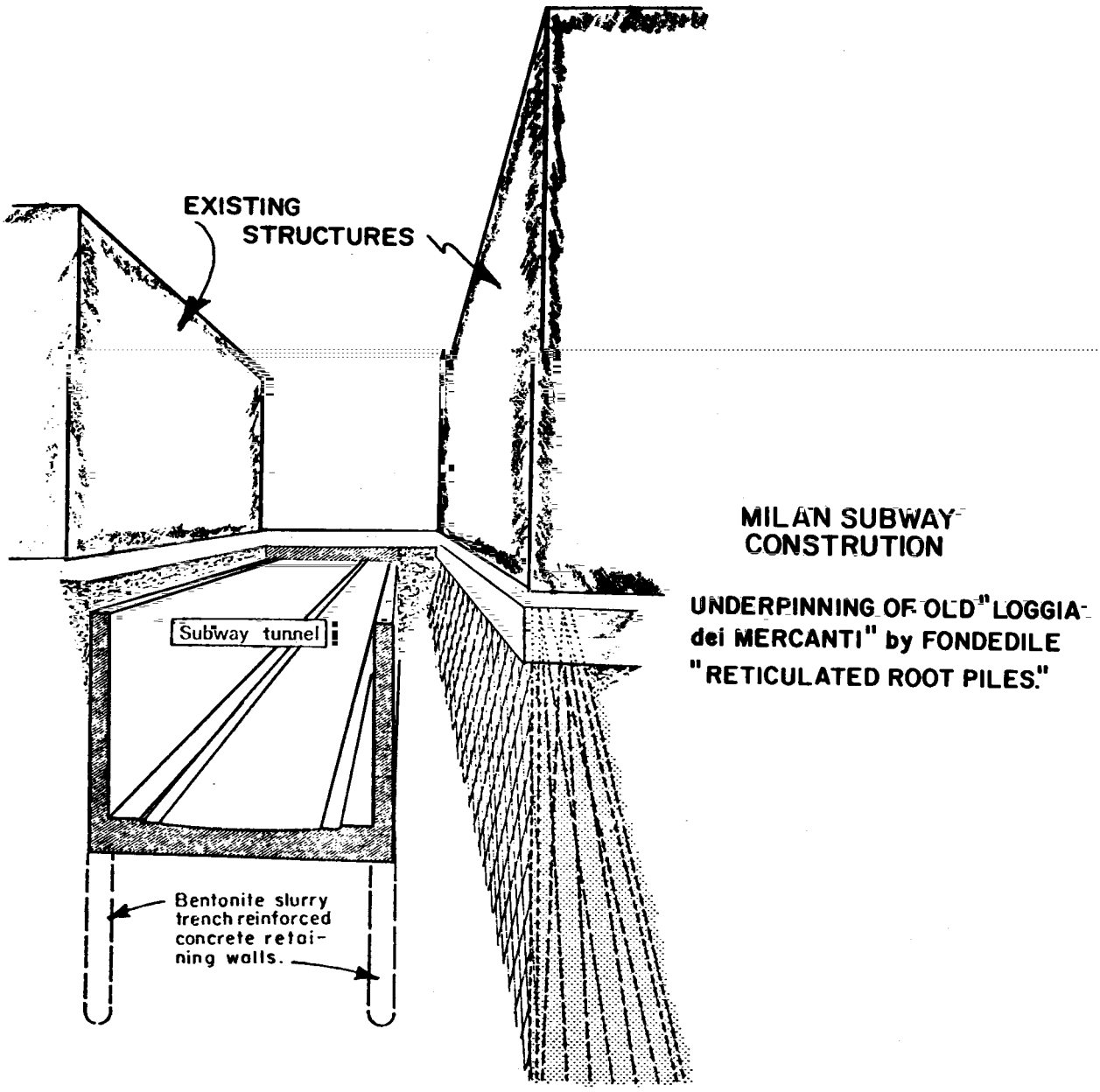


Figure 114. Reticulated root pile underpinning.
(Courtesy of Warren-Fondedile, Inc.).

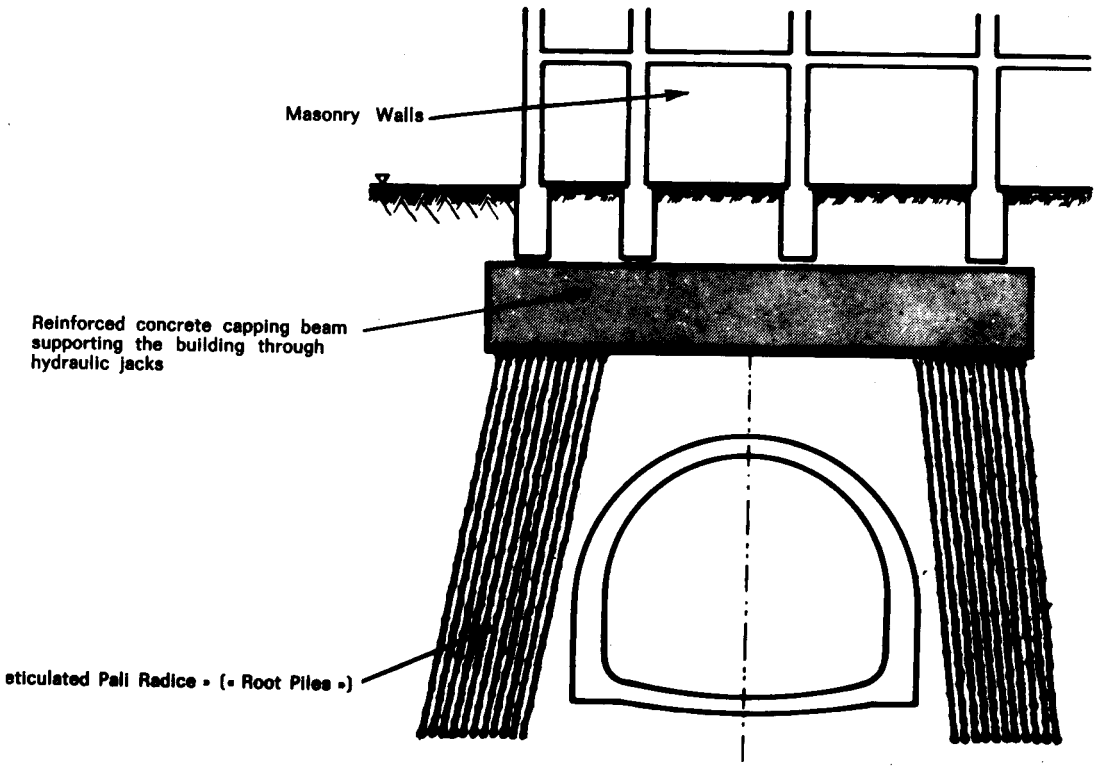


Figure 115a. Reticulated root pile applications. (Courtesy of Warren-Fondedile, Inc.).

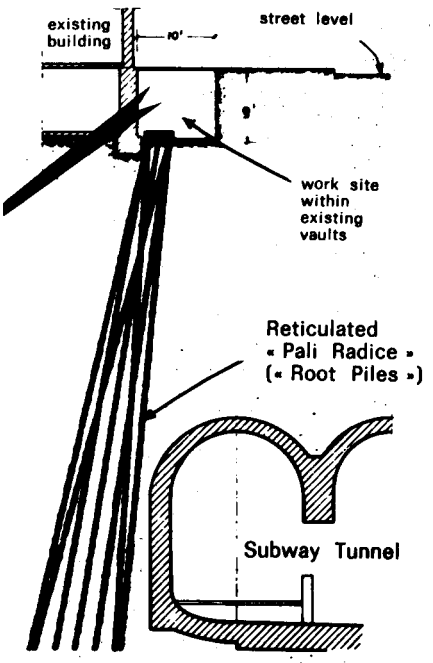
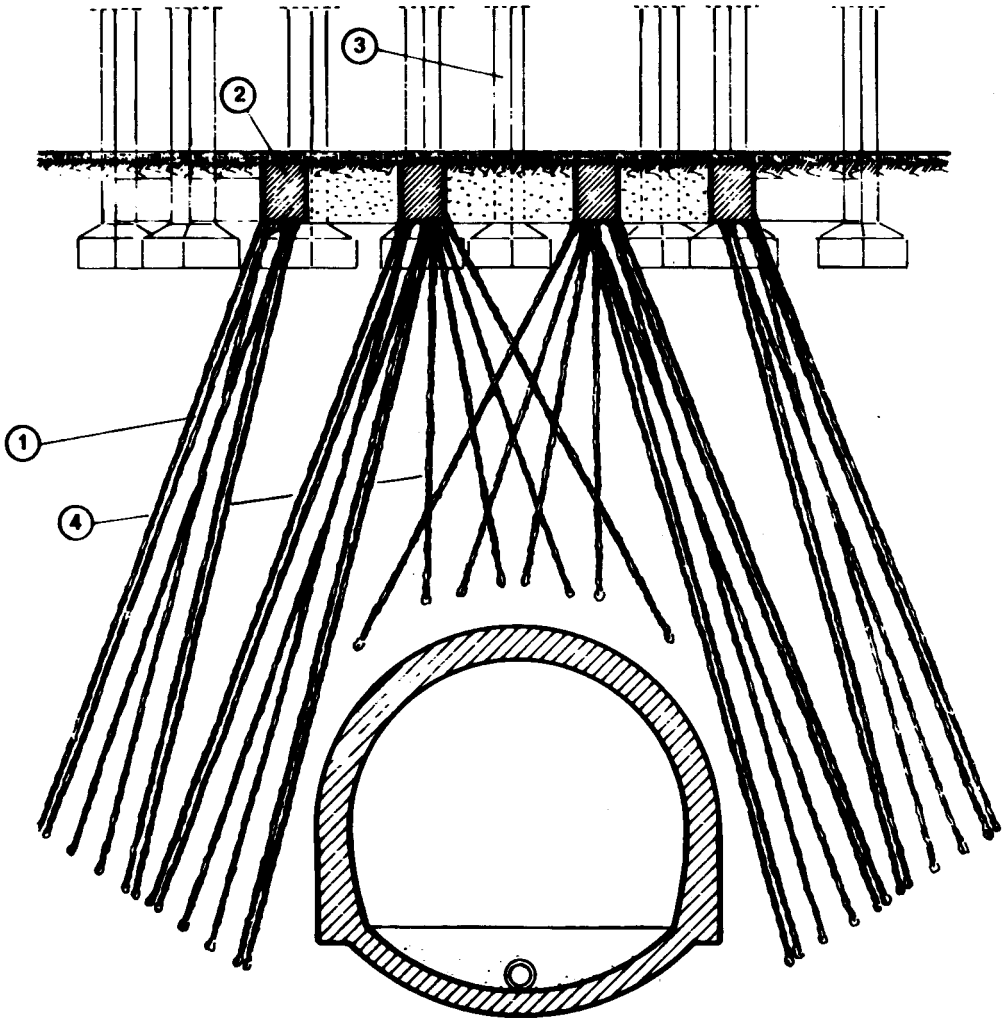


Figure 115b. Reticulated root pile applications. (Courtesy of Warren-Fondedile, Inc.).



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 accross the tunnel area.

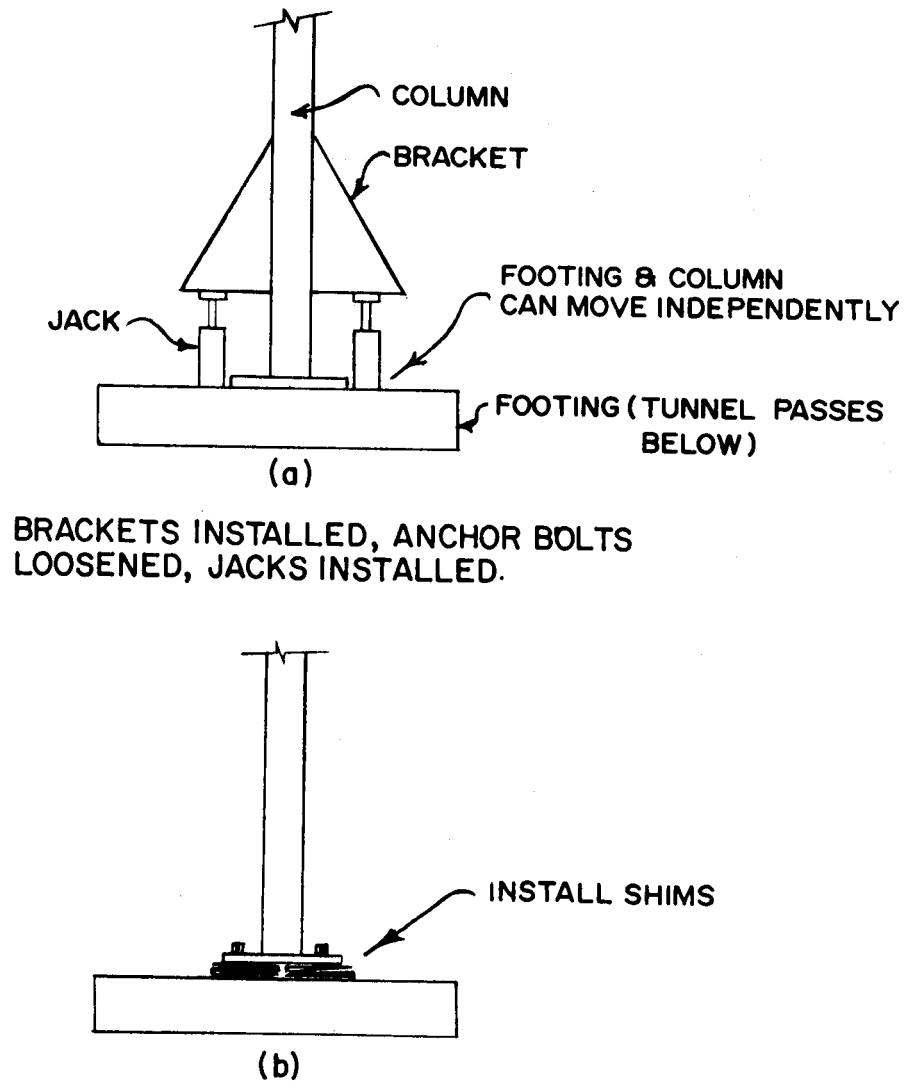
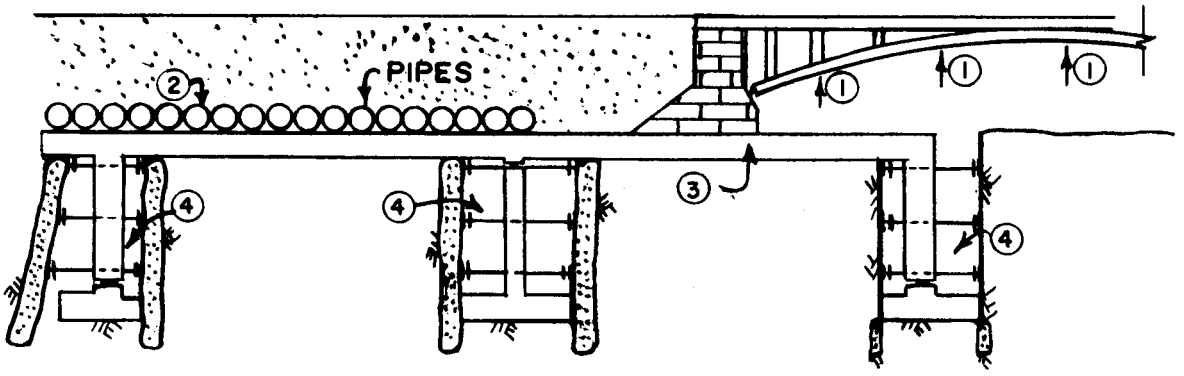
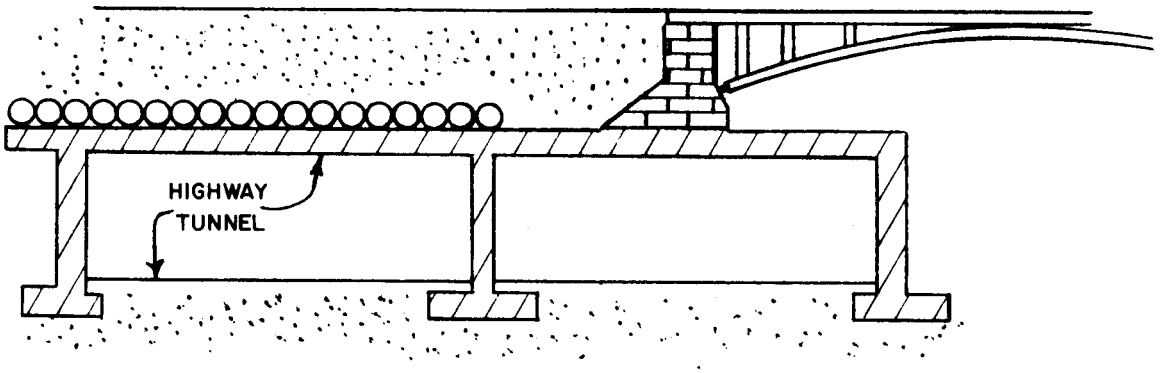


Figure 116. Schematic of column jacking to prevent structure settlement during tunnel construction.

(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).

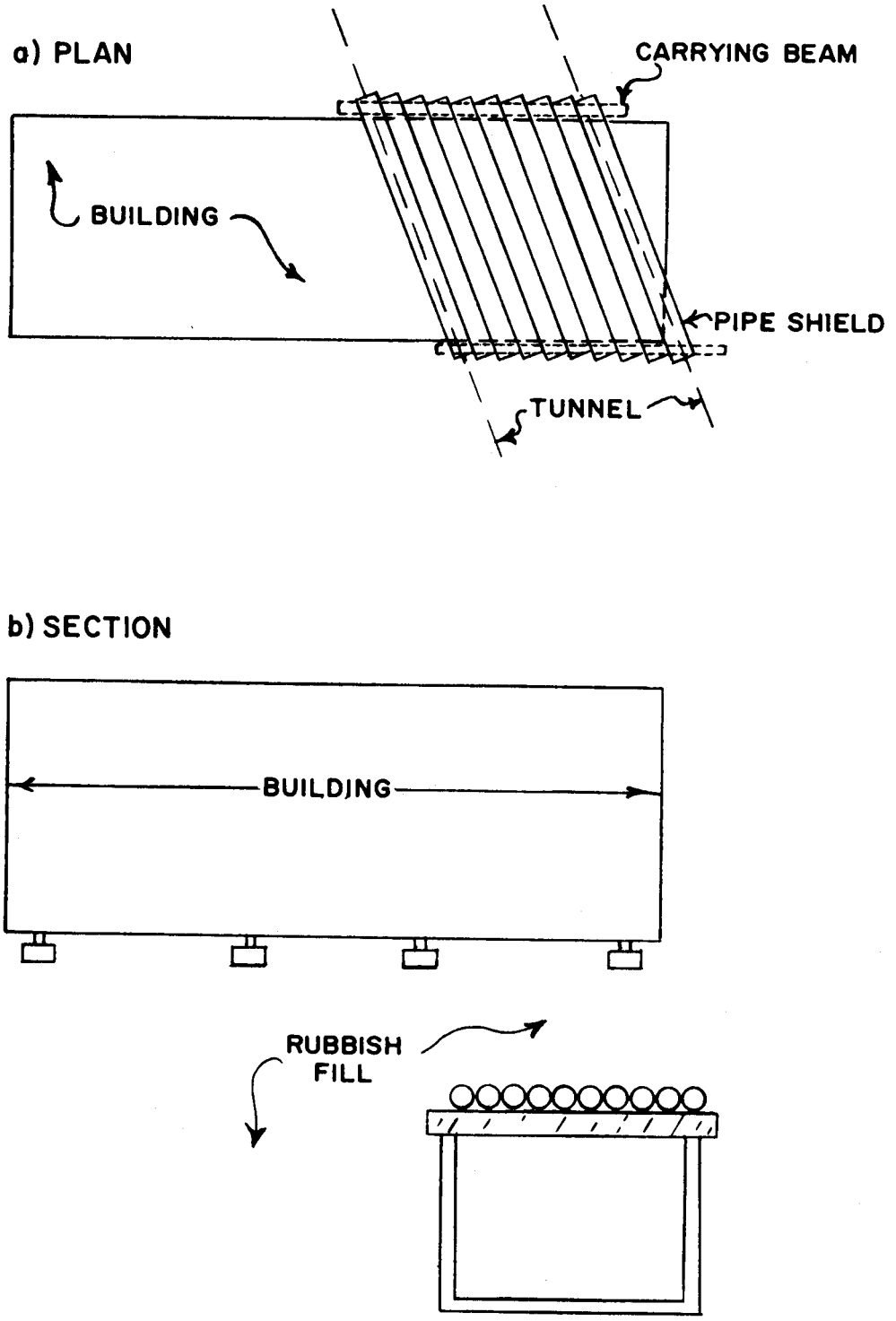


Figure 117b. Pipe shield technique (after Rappert, 1970).

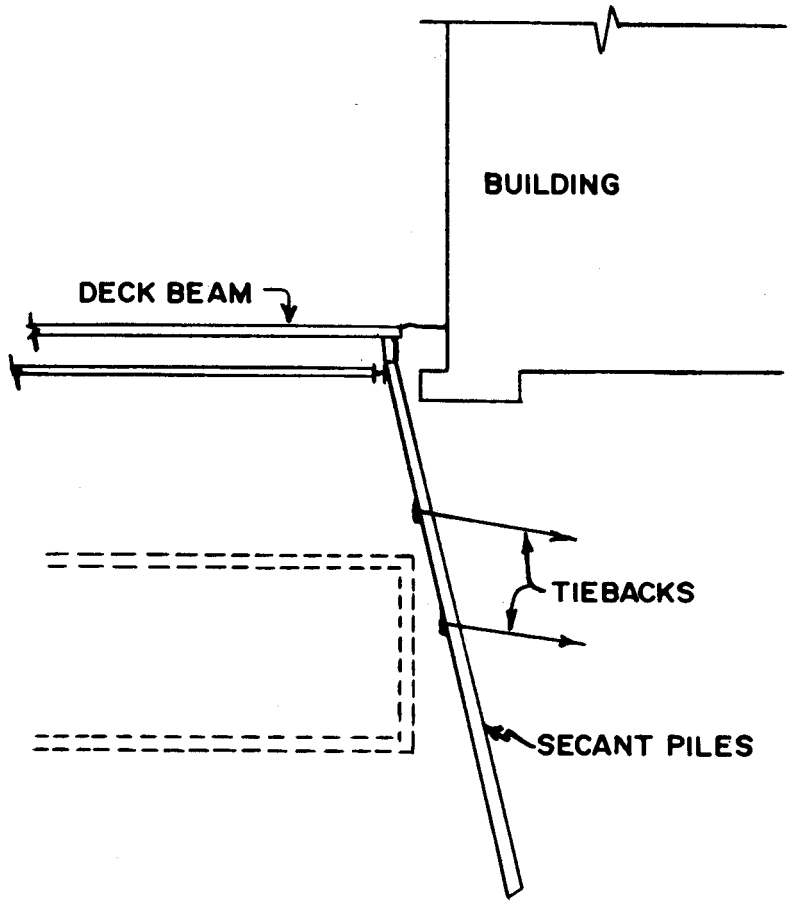


Figure 118. Inclined secant piles for protection of building (after Joas, et al, 1971; Weinhold and Kleinkein, 1969).

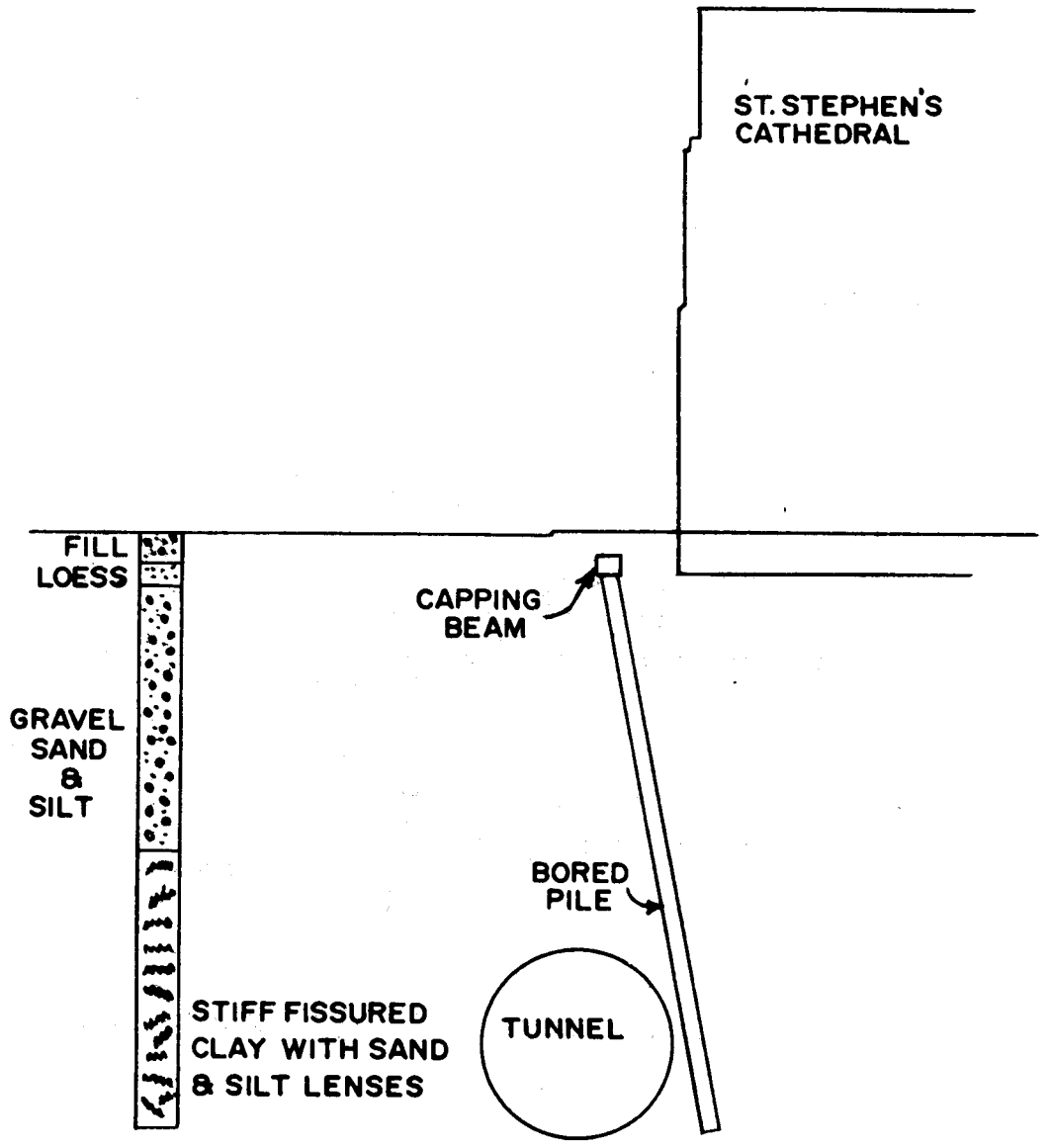


Figure 119. Example of bored pile wall used to protect structure (after Braun, 1974).

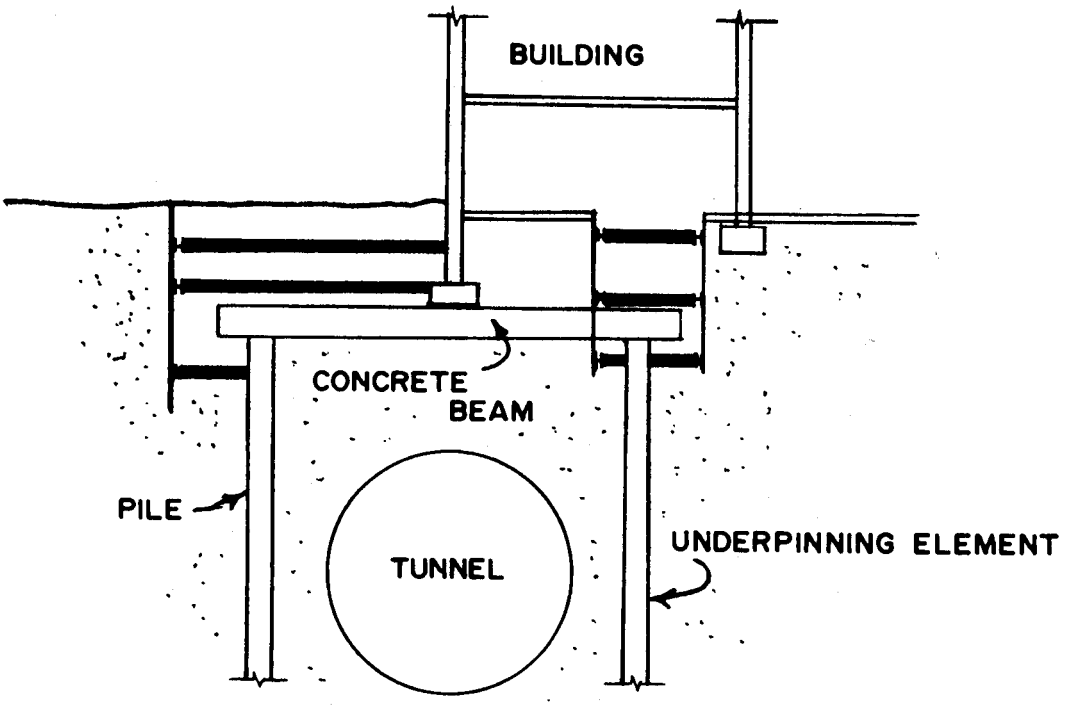


Figure 120. Bridging.

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.

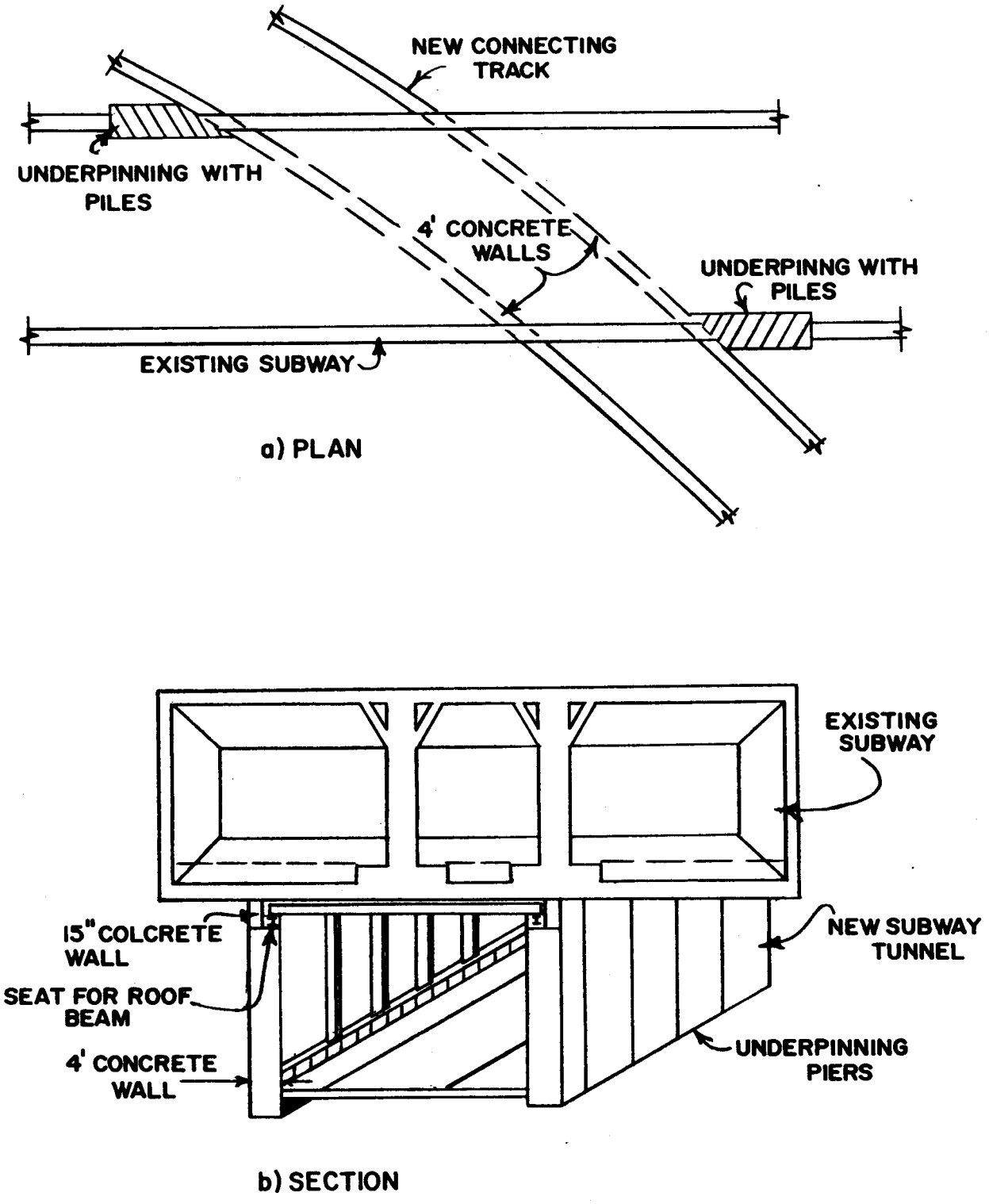
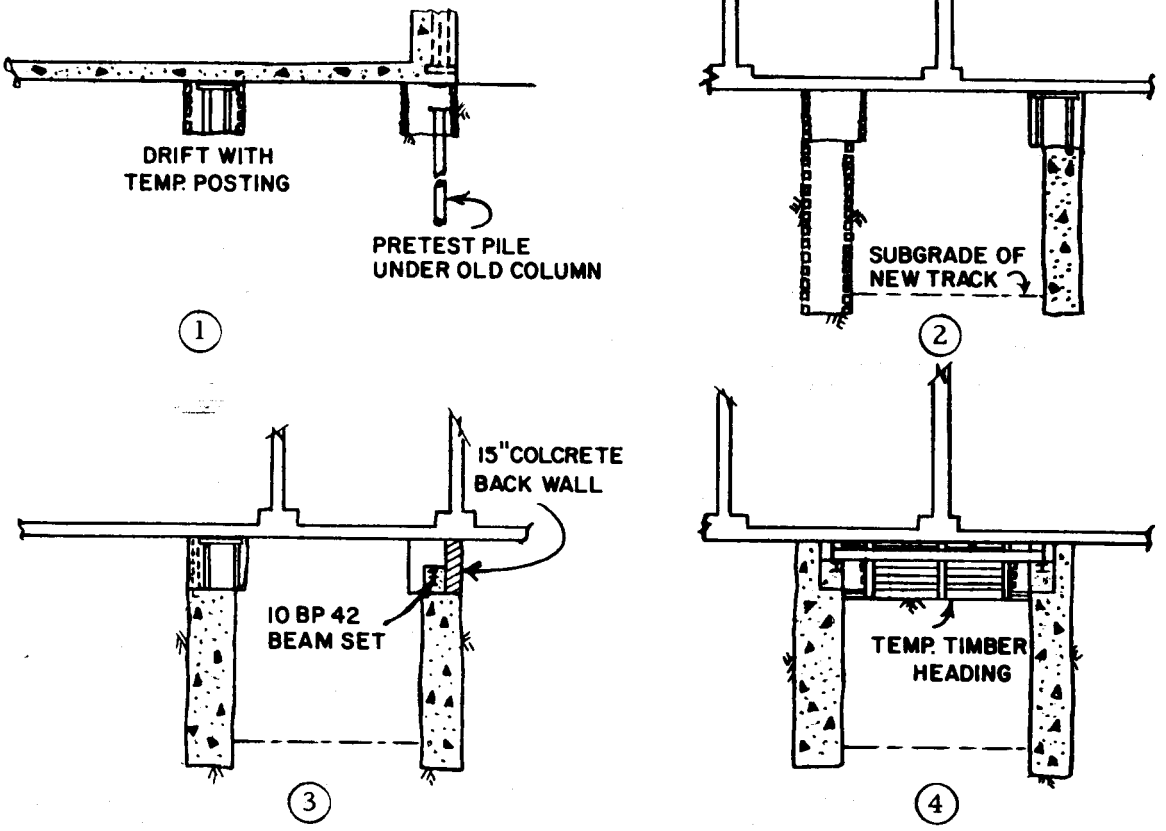


Figure 121. Location of new subway tunnel (after Goldfinger, 1960).



Sequence of operations is shown in five steps:

STEP 1. Excavate tunnels and support existing structure

STEP 2. Excavate and place concrete piers

STEP 3. Build 15-in. Colcrete wall and steel-beam seats

STEP 4. Install permanent roof steel; load of BMT structure to steel

STEP 5. Excavate between walls to subgrade; install rest of structural steel and concrete

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 speoial 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.

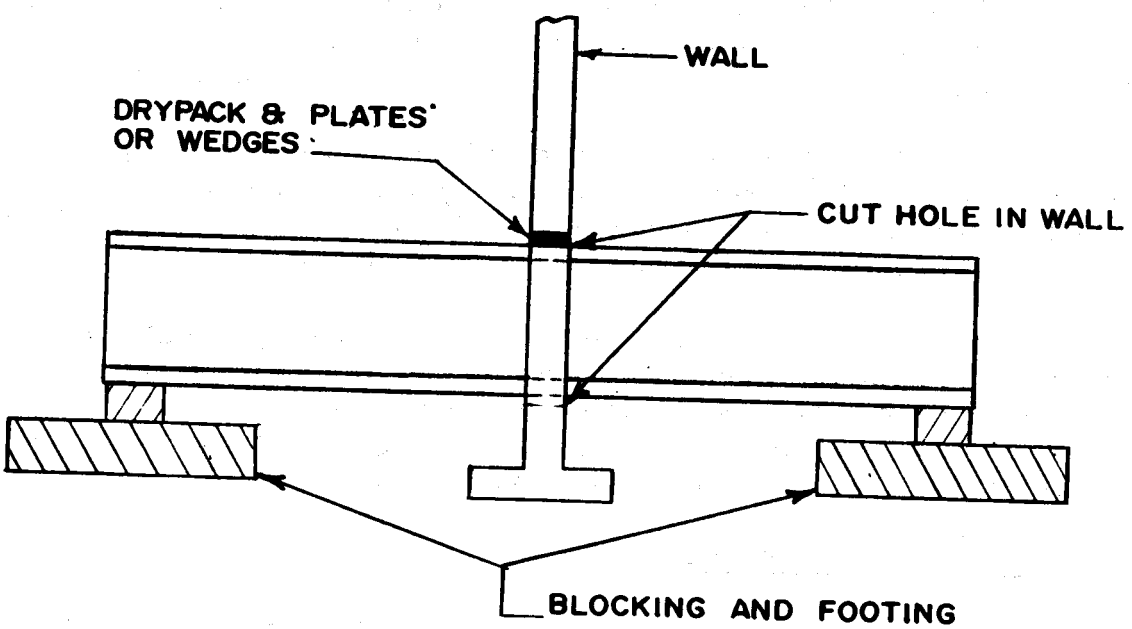
7.72 Needle Beams

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.

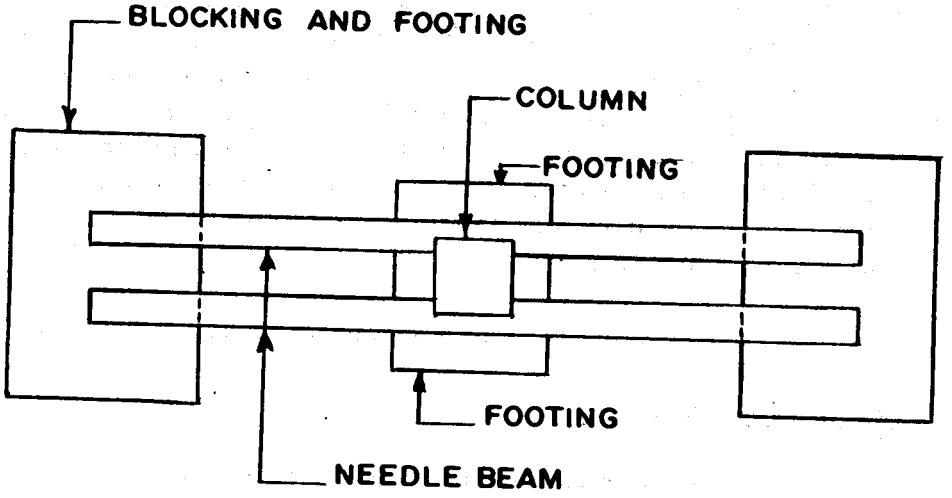
Figure 124 shows the underpinning of concrete columns.

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.

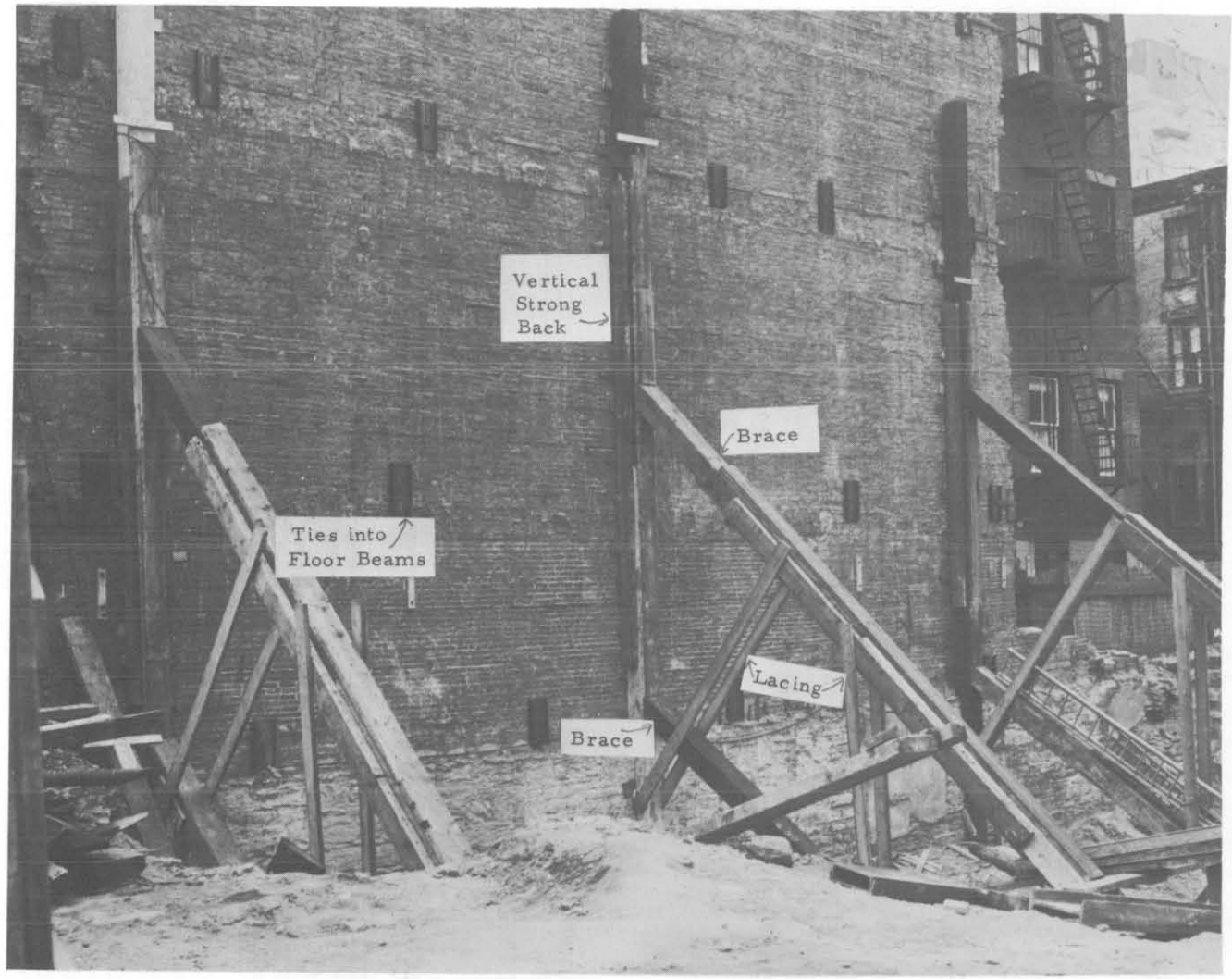


Figure 124. Lateral bracing of wall prior to underpinning.
(Courtesy of Spencer, White, and Prentis).

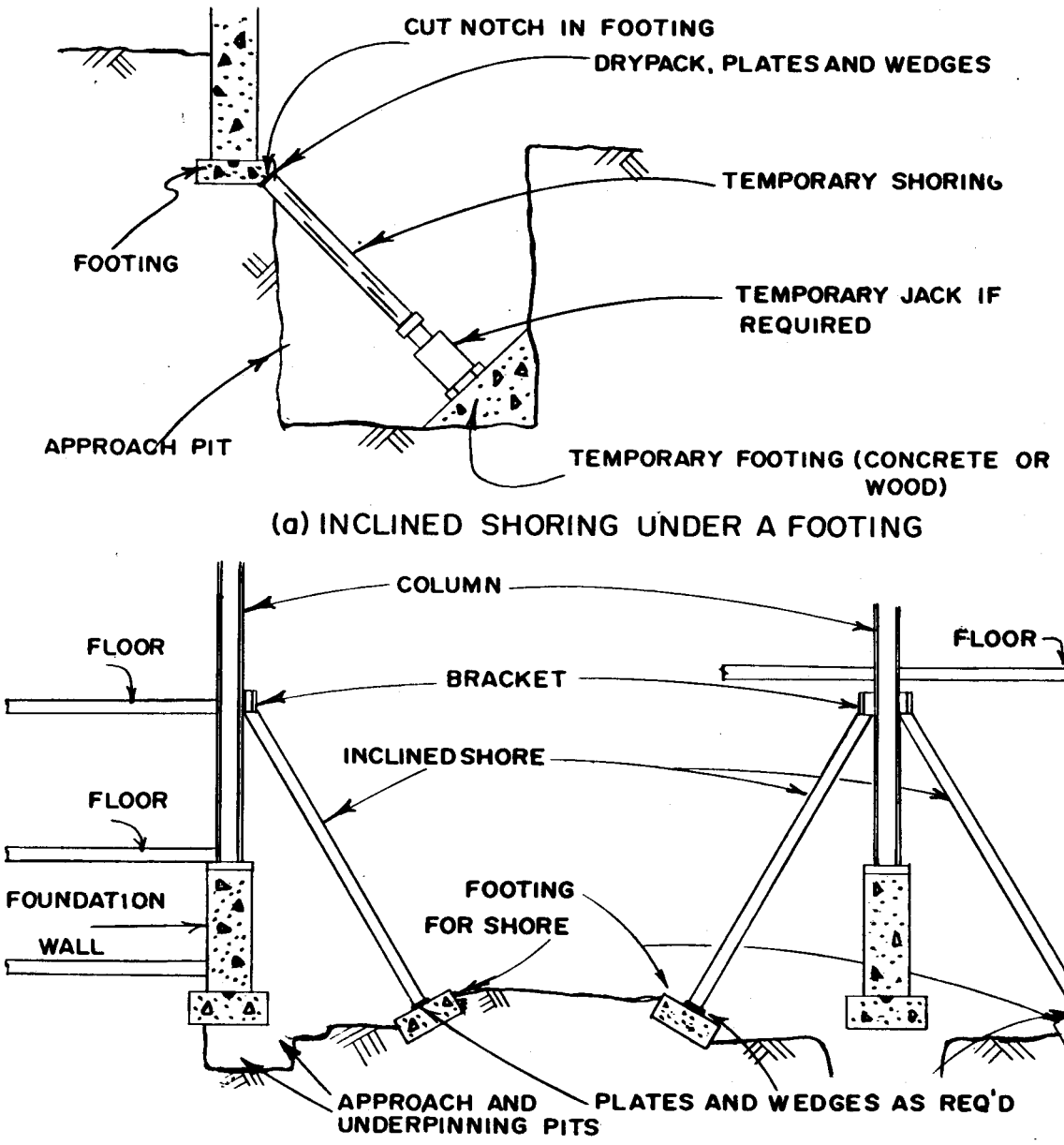


Figure 125. Inclined shoring details.

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.

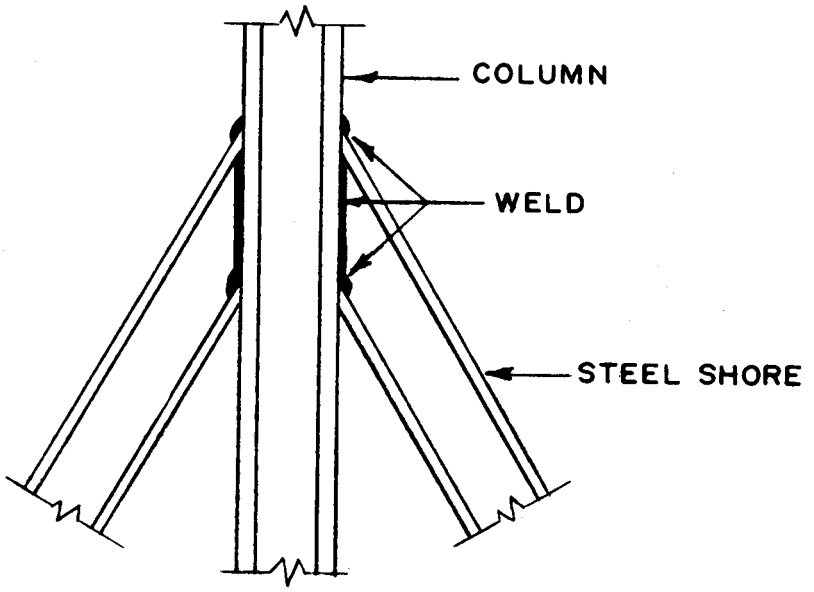
7.80 PERFORMANCE

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.

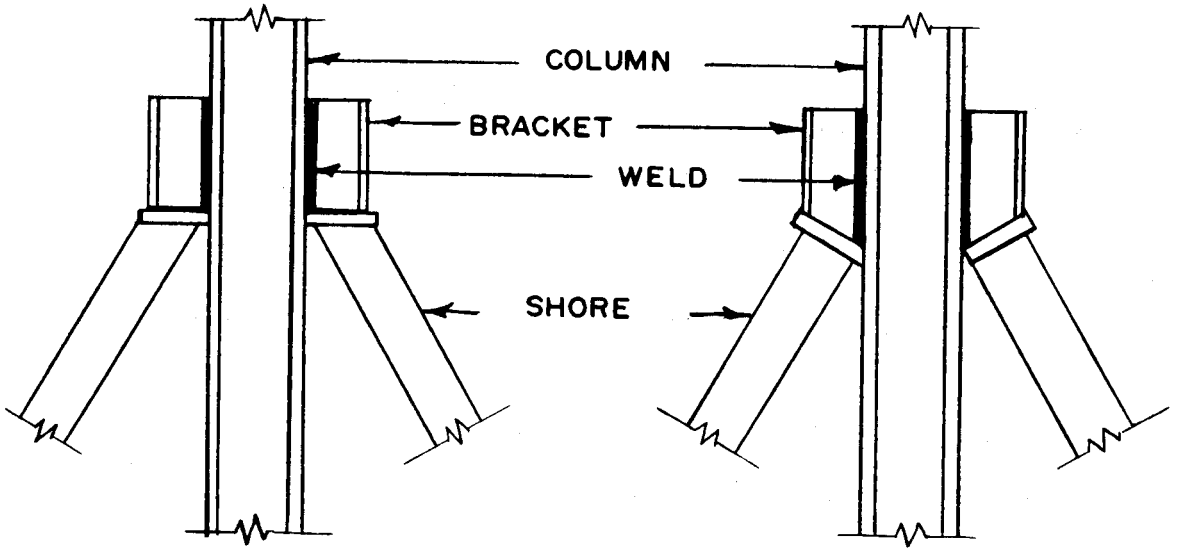
An extensive search of the literature produced little quantitative data on the performance of underpinning in connection with adjacent excavations. One exception to this general lack of performance documentation is the work by Ware (1974) which presents both settlements measured at the end of underpinning 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.

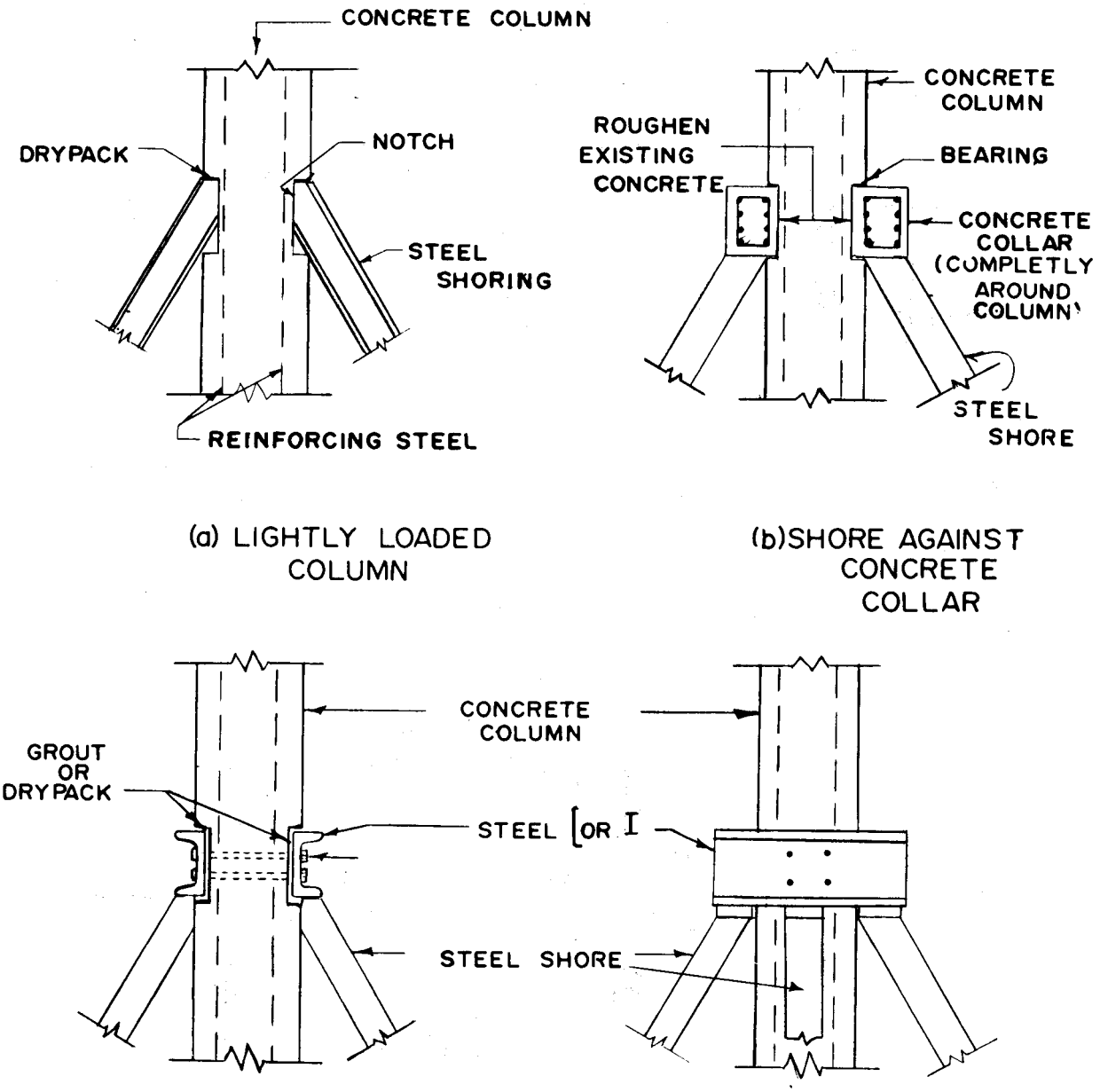


Figure 127. Shoring detail, concrete column.

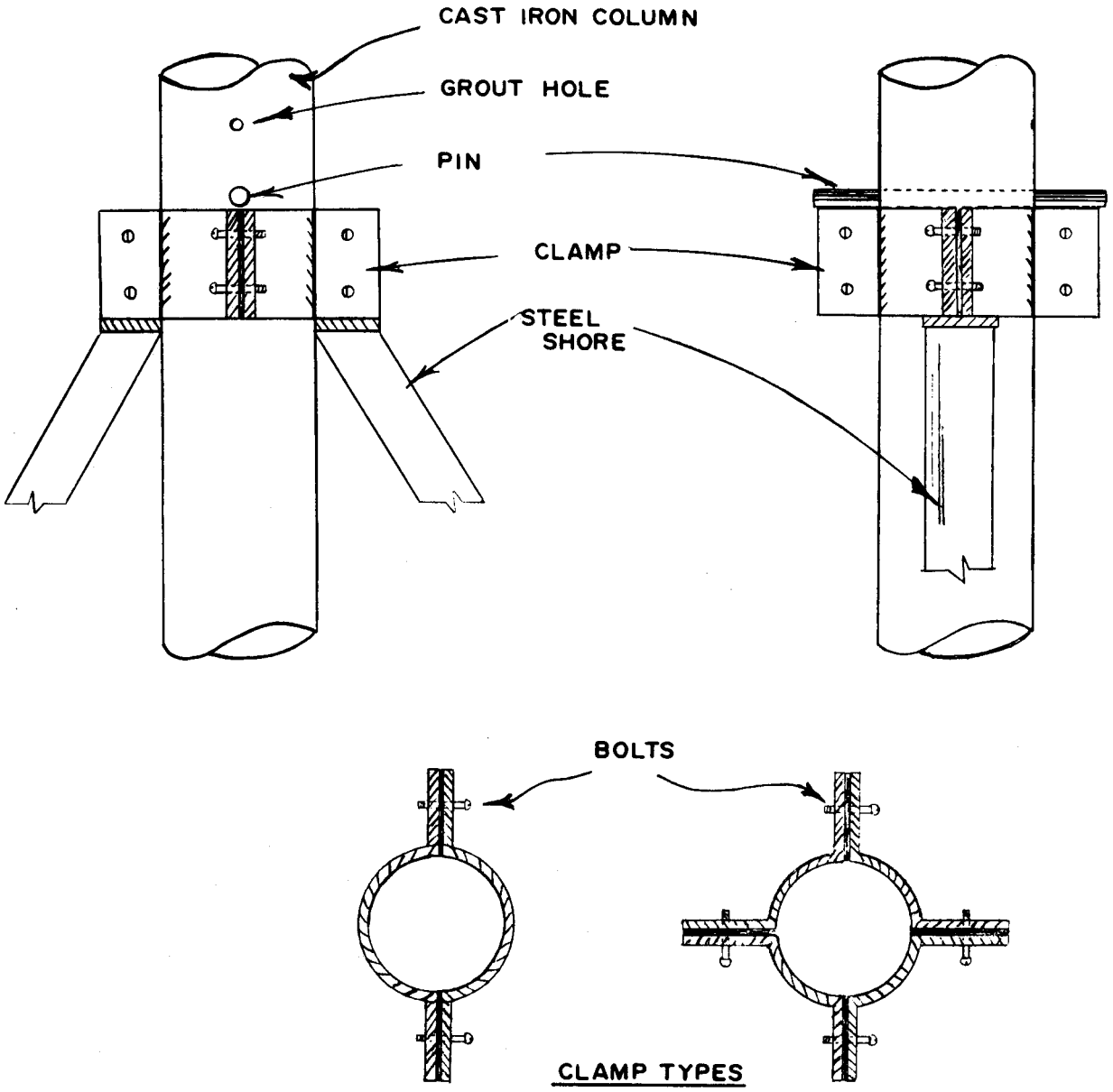


Figure 128. Pin and clamp details for a cast iron column.



Figure 129. Shoring of concrete columns. (Courtesy of Spencer, White, and Prentis).

this underpinning phase that settlement was experienced and damage occured. 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 subaqueous 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 gouting, 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

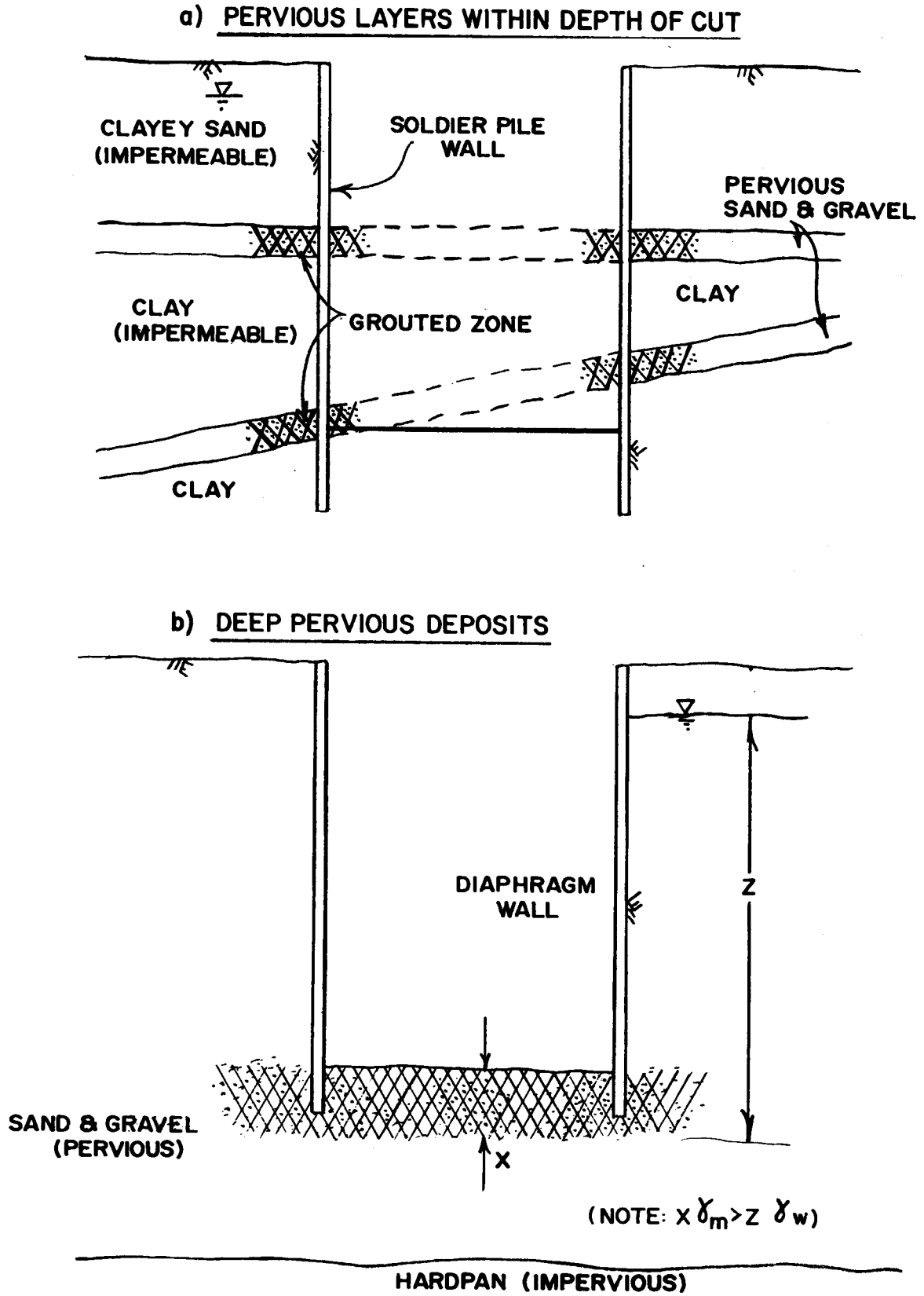


Figure 130. Grouting for ground water control.

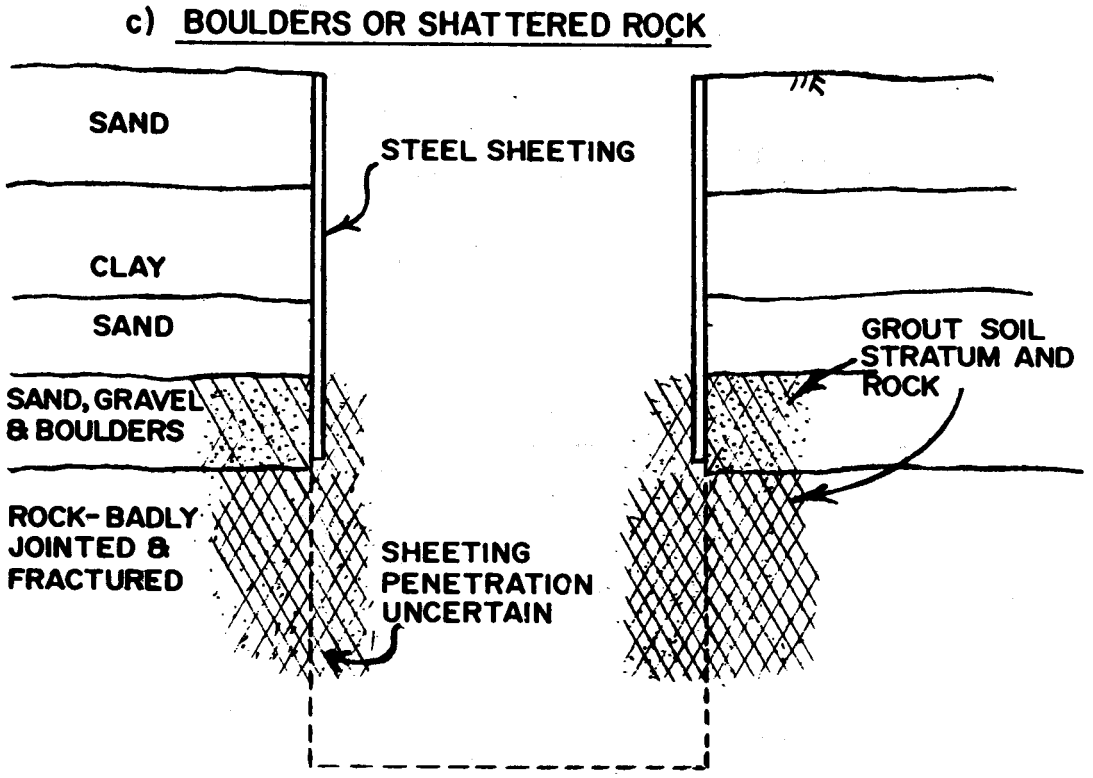


Figure 130c. Grouting for ground water control.

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.

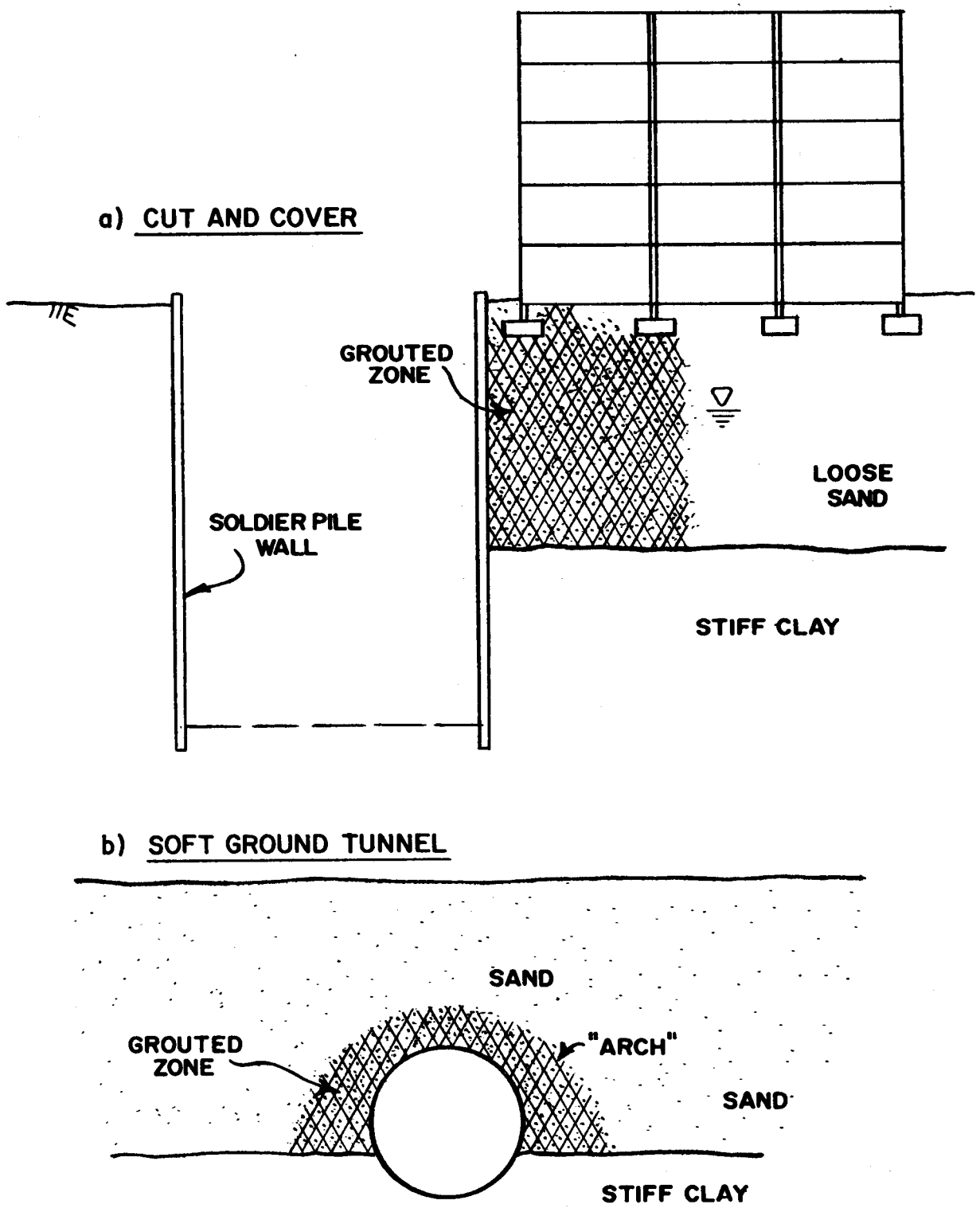


Figure 131. Grouting for soil solidification.

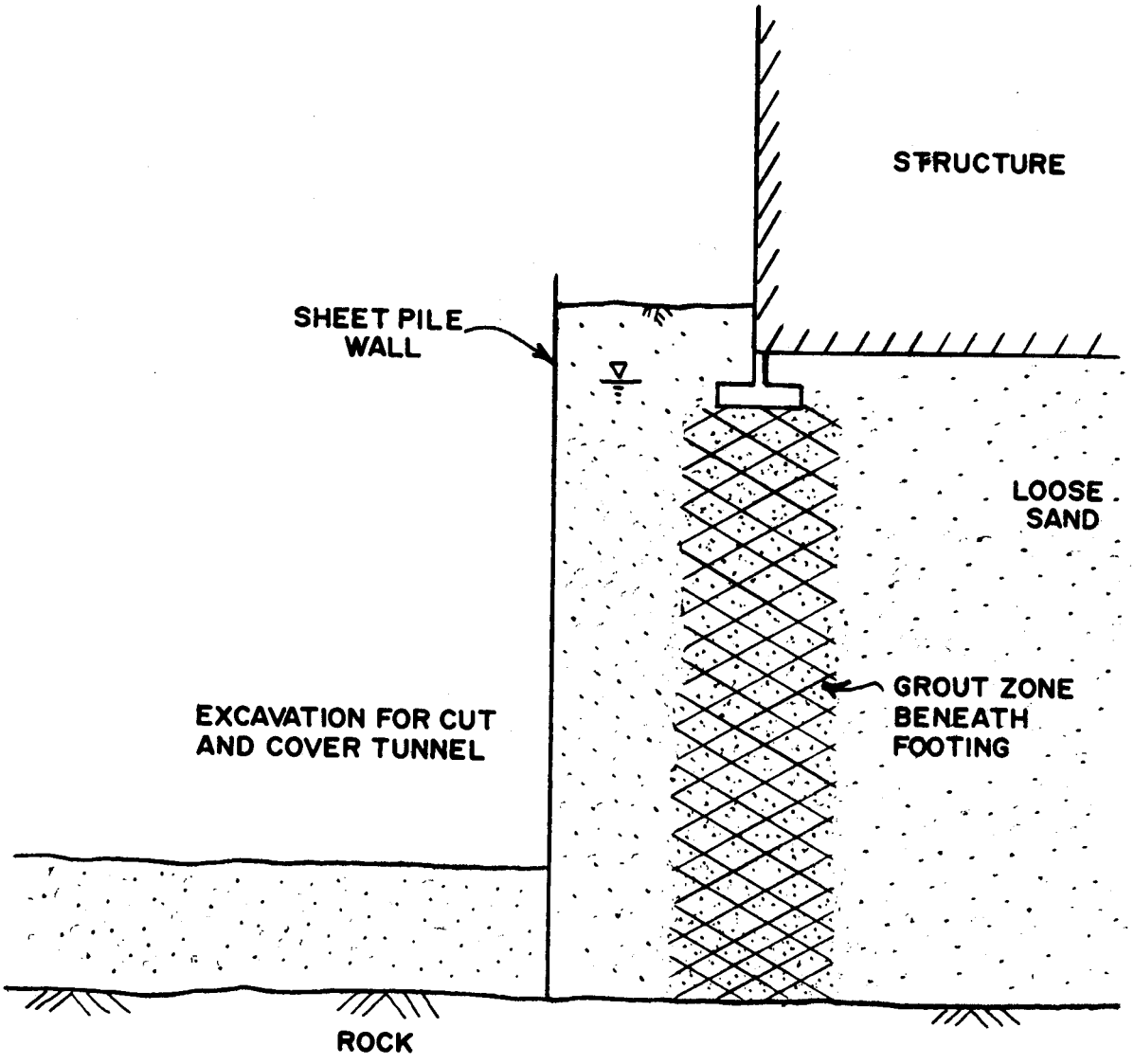


Figure 132. Example of grouting used to underpin an existing structure.

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.

$$\text{Groutability ratio} = \frac{D_{15\ (soil)}}{D_{85\ (grout)}} > 25.$$

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. Chromelignosulfates 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).

Particulate Grouts	Cement Clay Bentonite	Suspensions
Chemical Grouts		
Preciptation	Silicate Chemicals Aluminate Chemicals Chromelignosulfates	
Polymers	Acrylamides (e. g. AM9) Phenoplasts or Aminoplasts (e. g. recorcineformol, urea-formol)	Injected in form of monomers
	Epoxy Polyester-resins	Injected partially polymerized
Bituminous Emulsions		

$$t = \frac{\alpha n}{3khr_o} (R^3 - r_o^3)$$

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.

Soil Properties	Type of Soils	Coarse Sands and Gravels	Medium to fine Sands	Silty or Clayey Sands, Silts
	Grain diameter	$d_{10} > 0.5$mm	$0.02 < d_{10} < 0.5$mm	$d_{10} < 0.02$mm
	Specific surface	$s < 100\ cm^{-1}$	$100\ cm^{-1} < s < 1000\ cm^{-1}$	$s > 1000 cm^{-1}$
	Permeability	$k > 10^{-3}$ m/s	$10^{-3} > k > 10^{-5}$ m/s	$k < 10^{-5}$ m/s
Series of Mix		Bingham Suspensions	Colloid Solutions (Gels)	Pure solutions (Resins)
Consolidation Grouting		Cement ($k > 10^{-2}$ 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

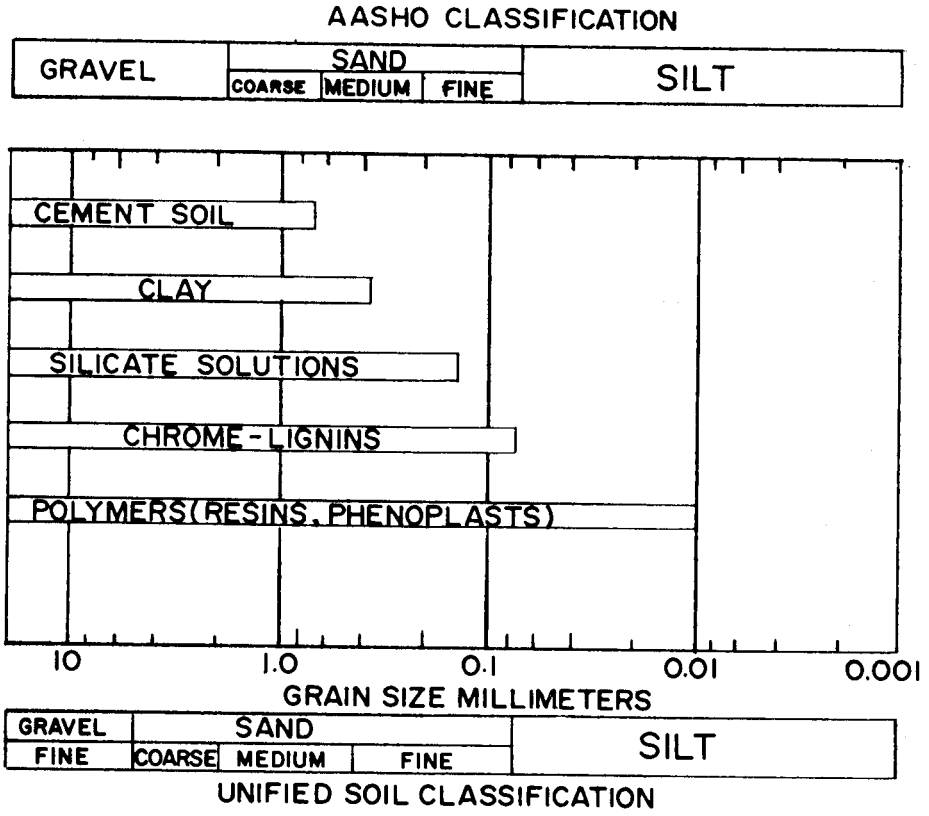


Figure 133. Range of usefulness of various grout types (from Mitchell, 1968).

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.

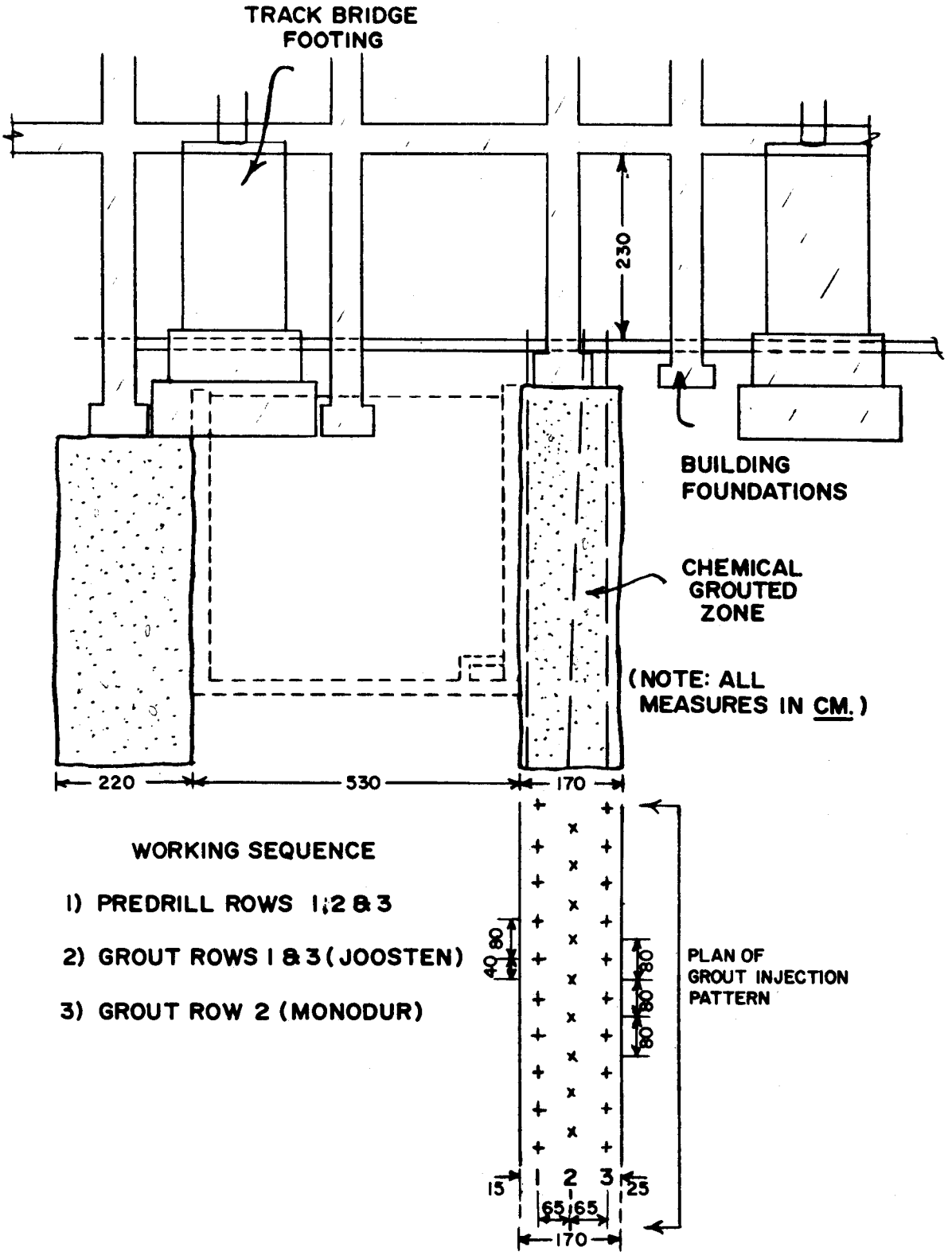


Figure 134. Lance system and working sequence for the grouting process (from Sening and Klotschke, 1970).

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	High 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 unsaturated 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 techniqes 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

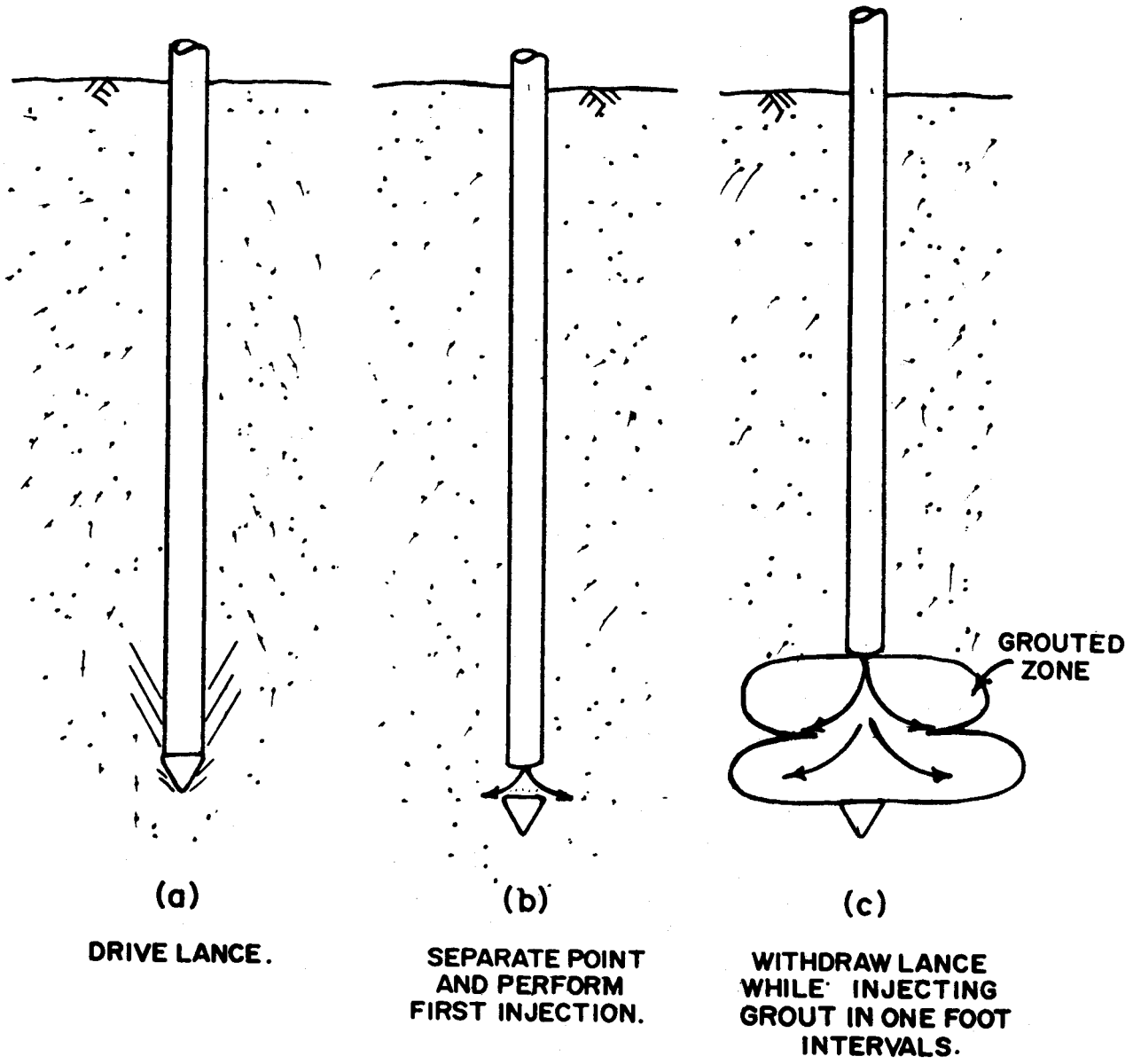


Figure 135. Schematic of driven lance method.

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-á-manchette and the grouting procedures.

Other injection systems use the basic principle of the tube-á-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.

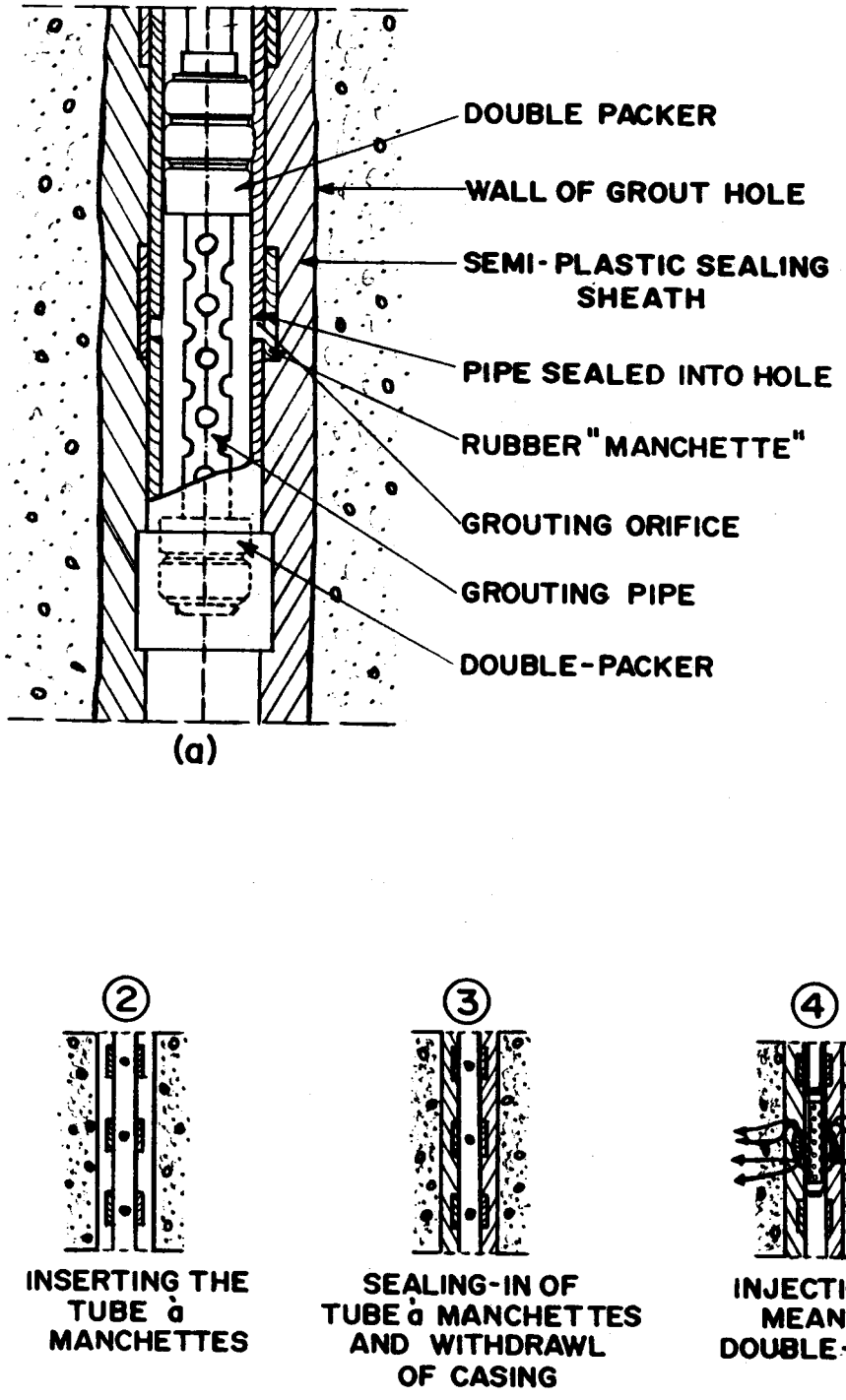


Figure 136. Sleeved grout tube (after Ischy and Glossup, 1962).

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.

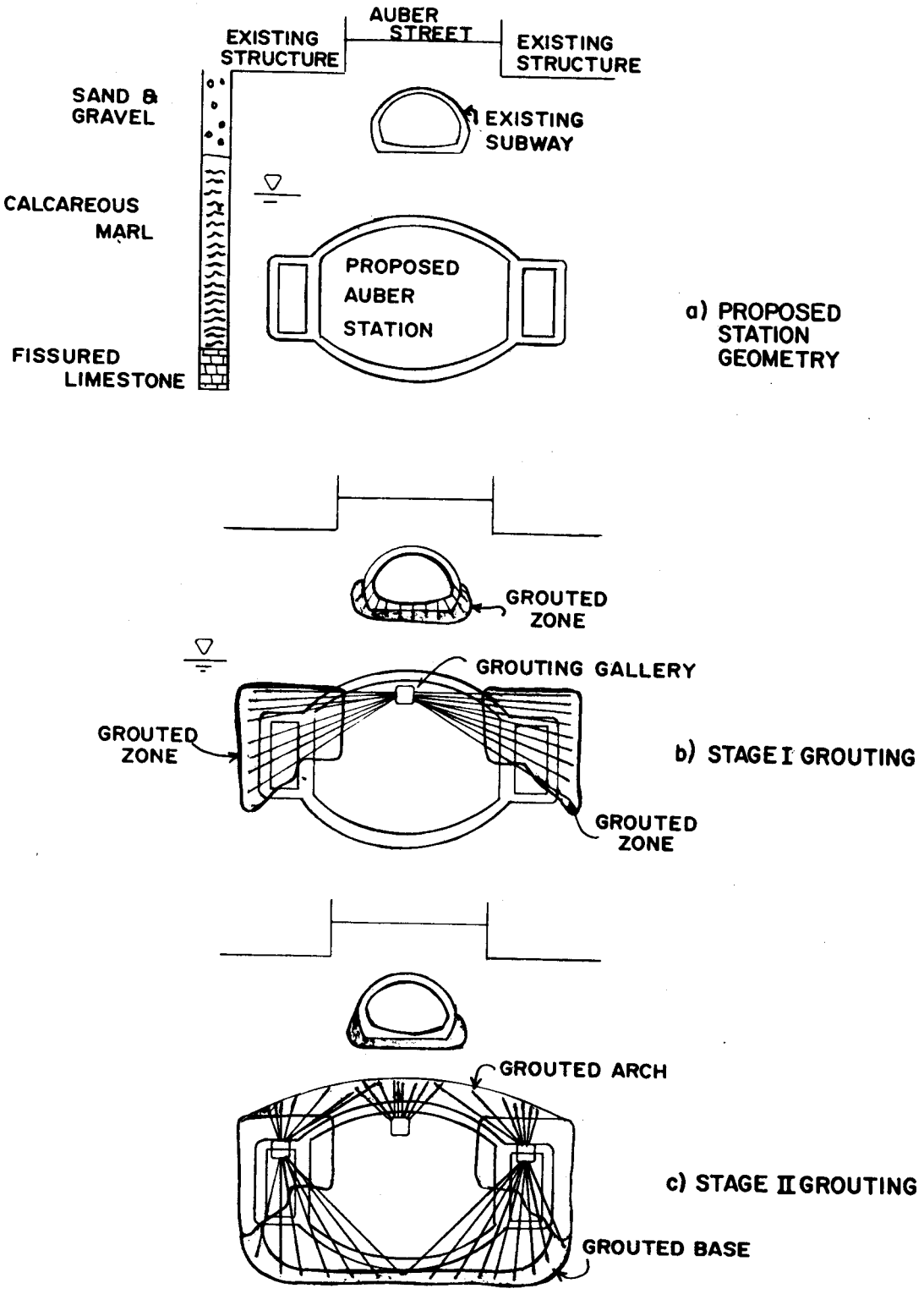


Figure 137. Grouting for Auber Station, Paris (after Janin and LeSciellour, 1970).

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 presen-tation 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-á-manchette technique. The grouts consisted of Portland cement, hydrated 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-á-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 runnel 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.

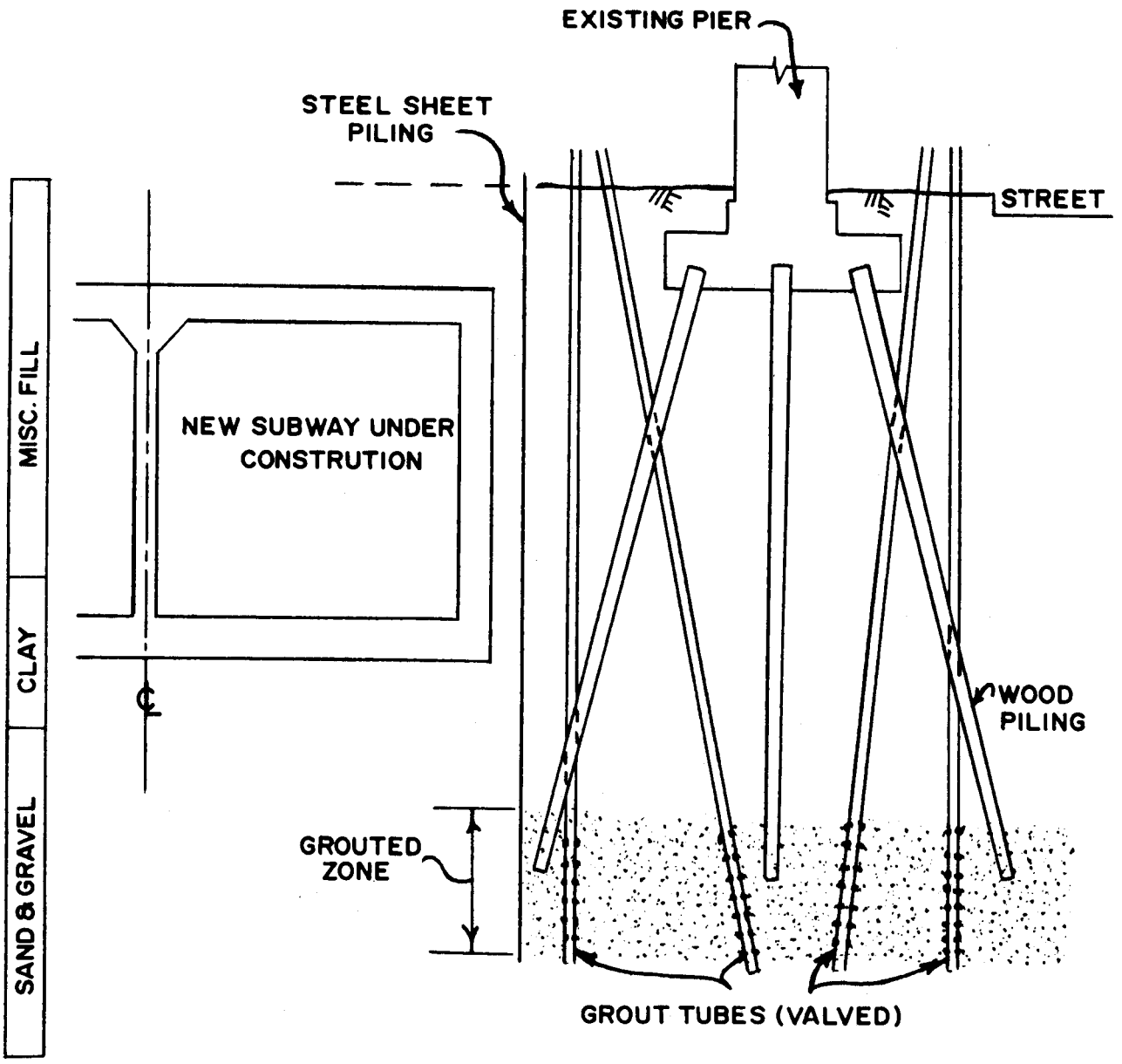


Figure 138. Underpinning of Walt Whitman Bridge, Philadelphia (from Halliburton, 1976).

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_o)

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-^oC or 1.000 BTU/lb. -^oF. Volumetric heat capacity (C_v) is sometimes more convenient:



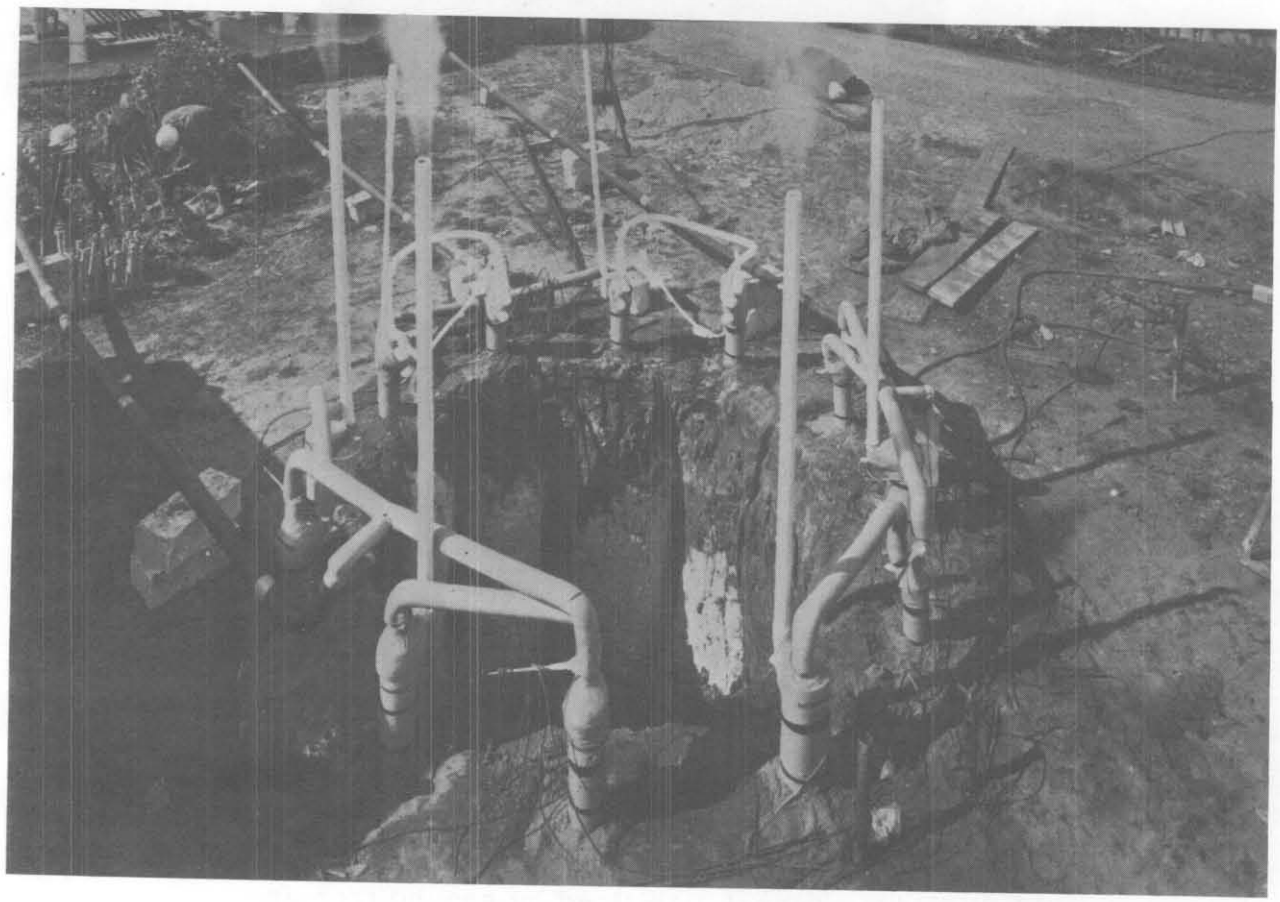
Note: Wall is protected by reflective thermal insulation.

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).

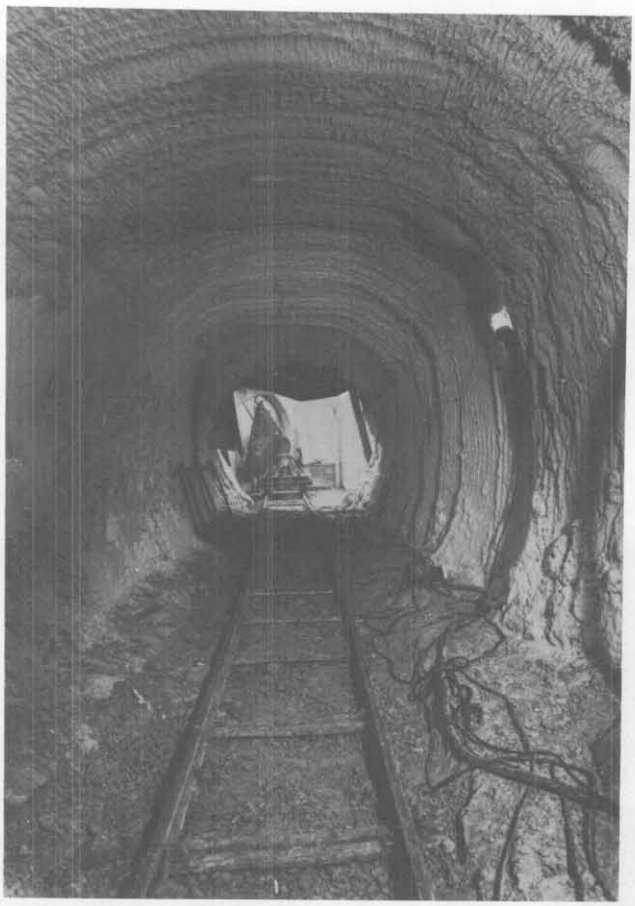


Figure 142. Tunnel stabilized by ground freezing.
(Courtesy of Terrafreeze Corporation).

$$C_v = C_m \gamma_d$$

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:

$$C_u = \gamma_d (0.2 + \frac{w}{100}) \text{ in BTU/ft}^3/^\circ\text{F}$$

Similarly, the volumetric heat capacity of frozen soil is:

$$C_f = \gamma_d (0.2 + \frac{0.5w}{100}) \text{ in BTU/ft}^3/^\circ\text{F}$$

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.)	Typical Values 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 materal 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.

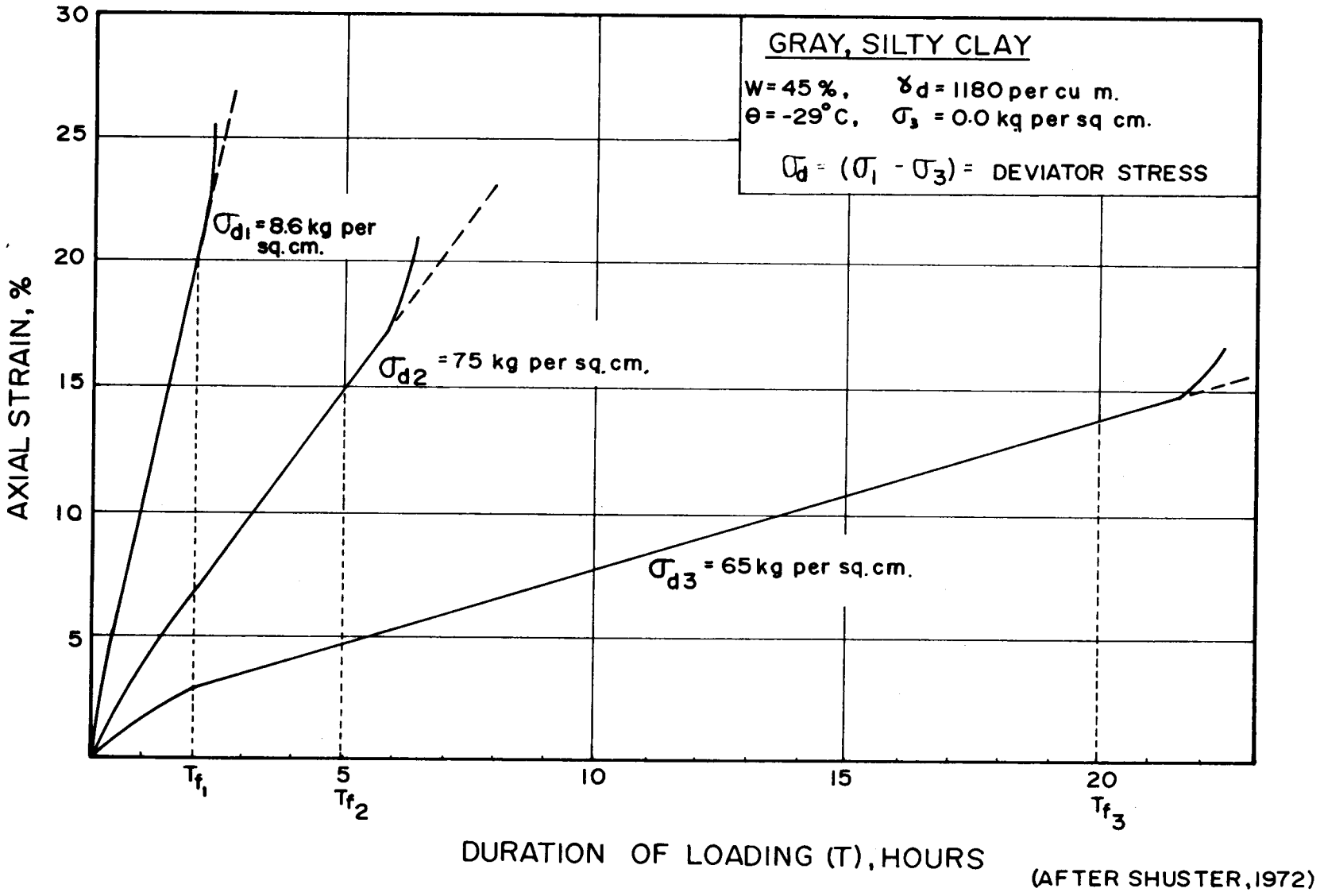


Figure 143. Strain versus time and loading for a frozen soil.

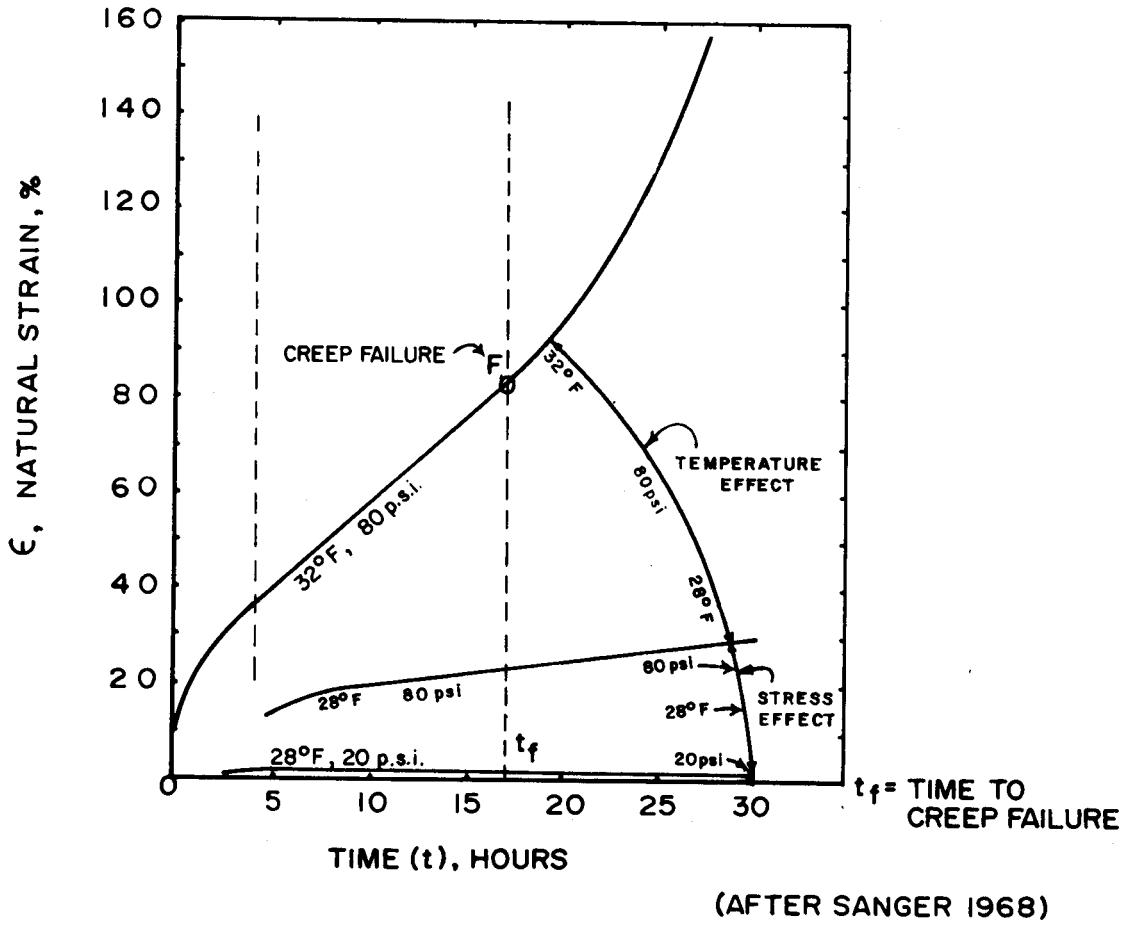


Figure 144. Creep curves for an organic silty clay with temperature influences.

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 Tsytovich (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

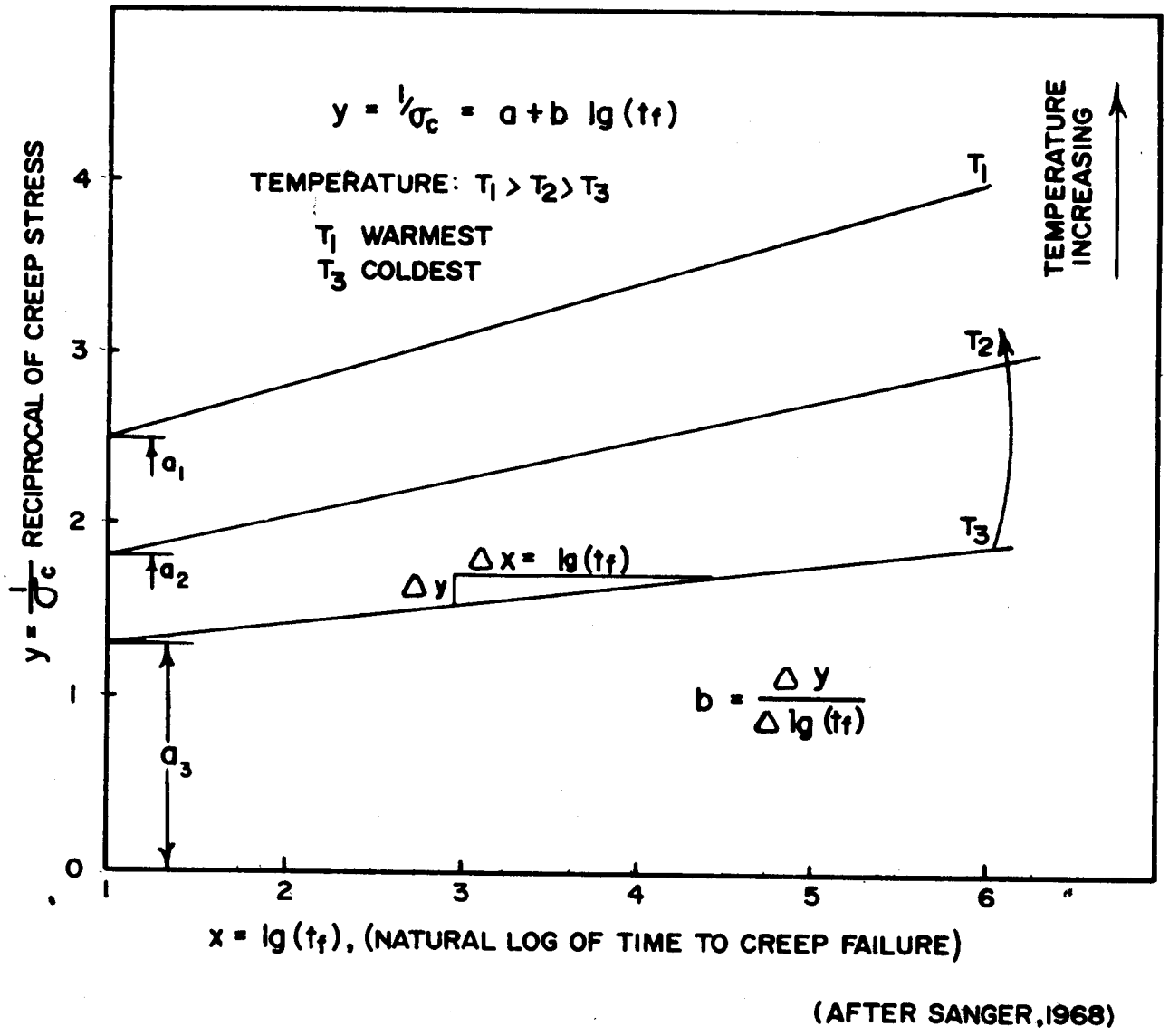


Figure 145. Influence of stress and temperature on time of creep failure.

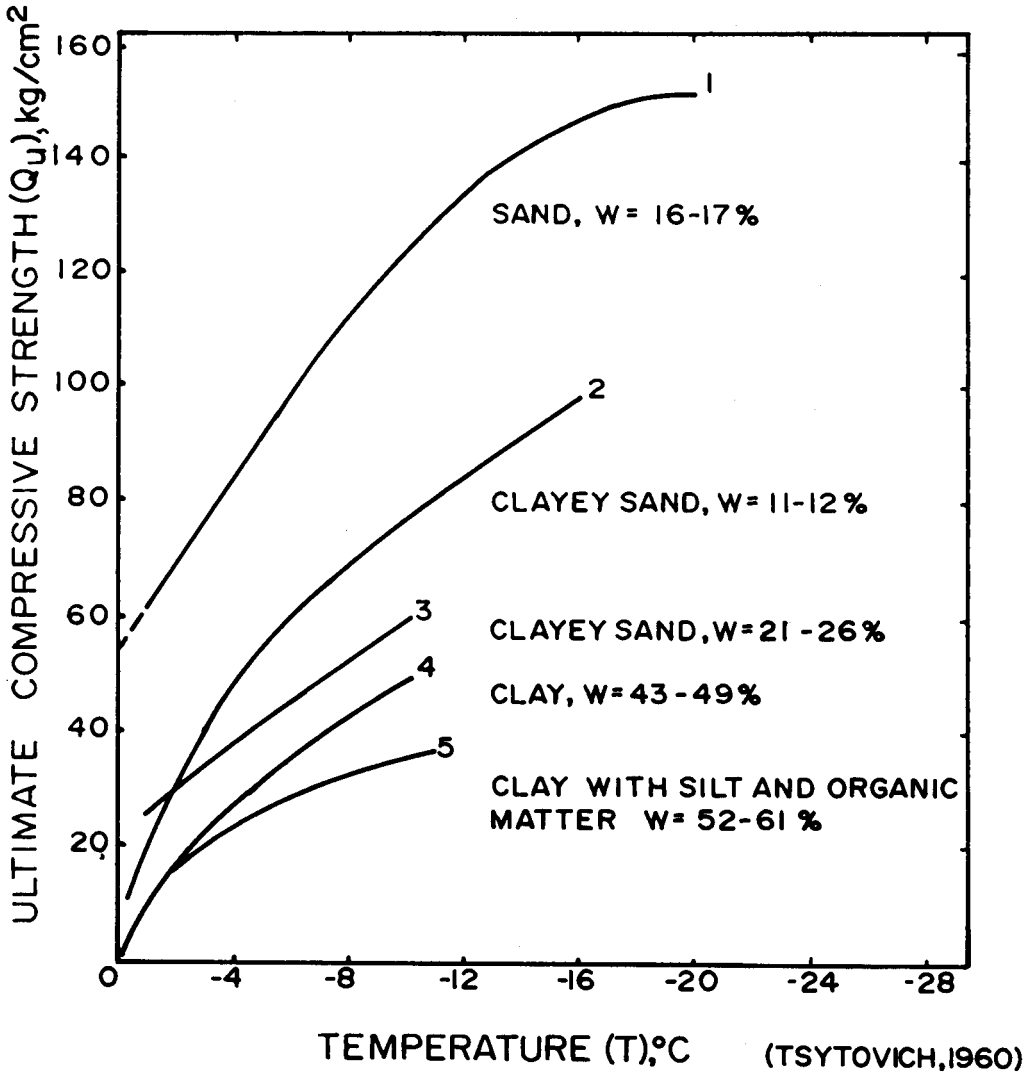
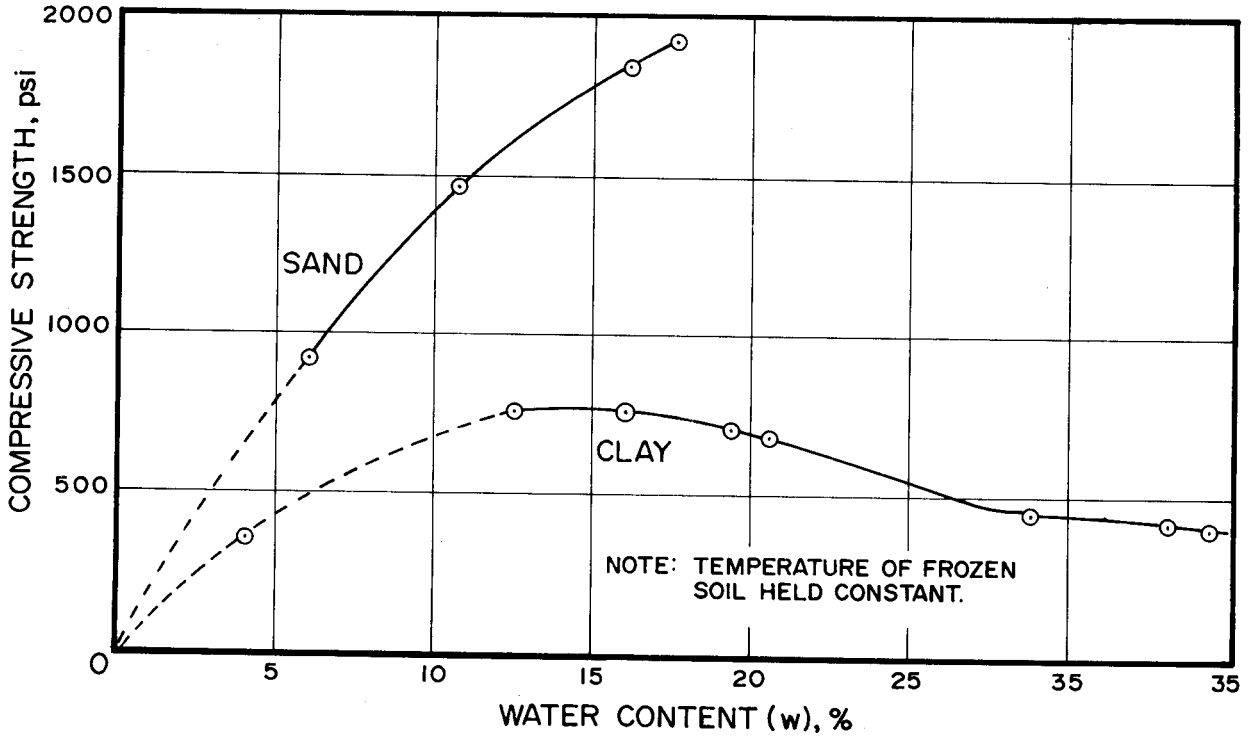


Figure 146. Ultimate short term compressive strength as a function of temperature.



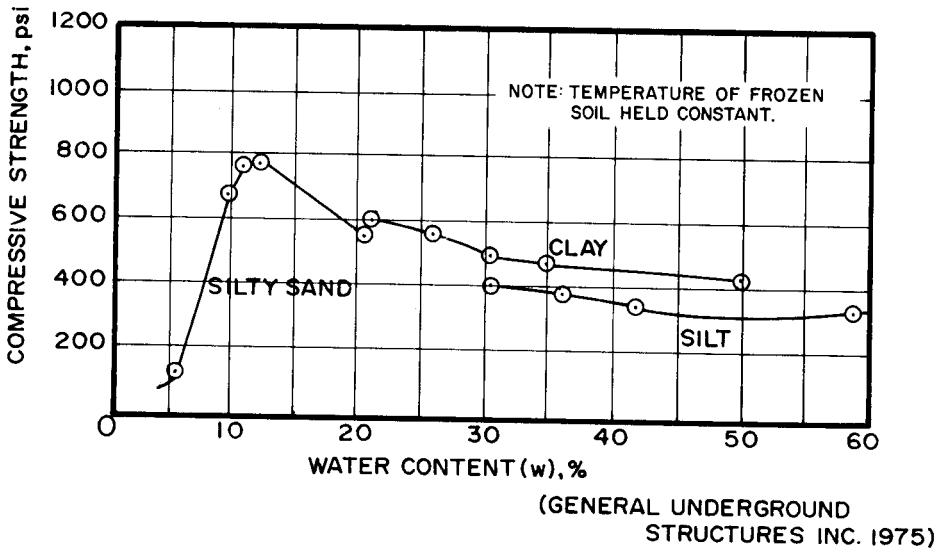


Figure 147. Ultimate short term compressive strength of ground vs. moisture content.

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 quanties above in units of BTU or calories)

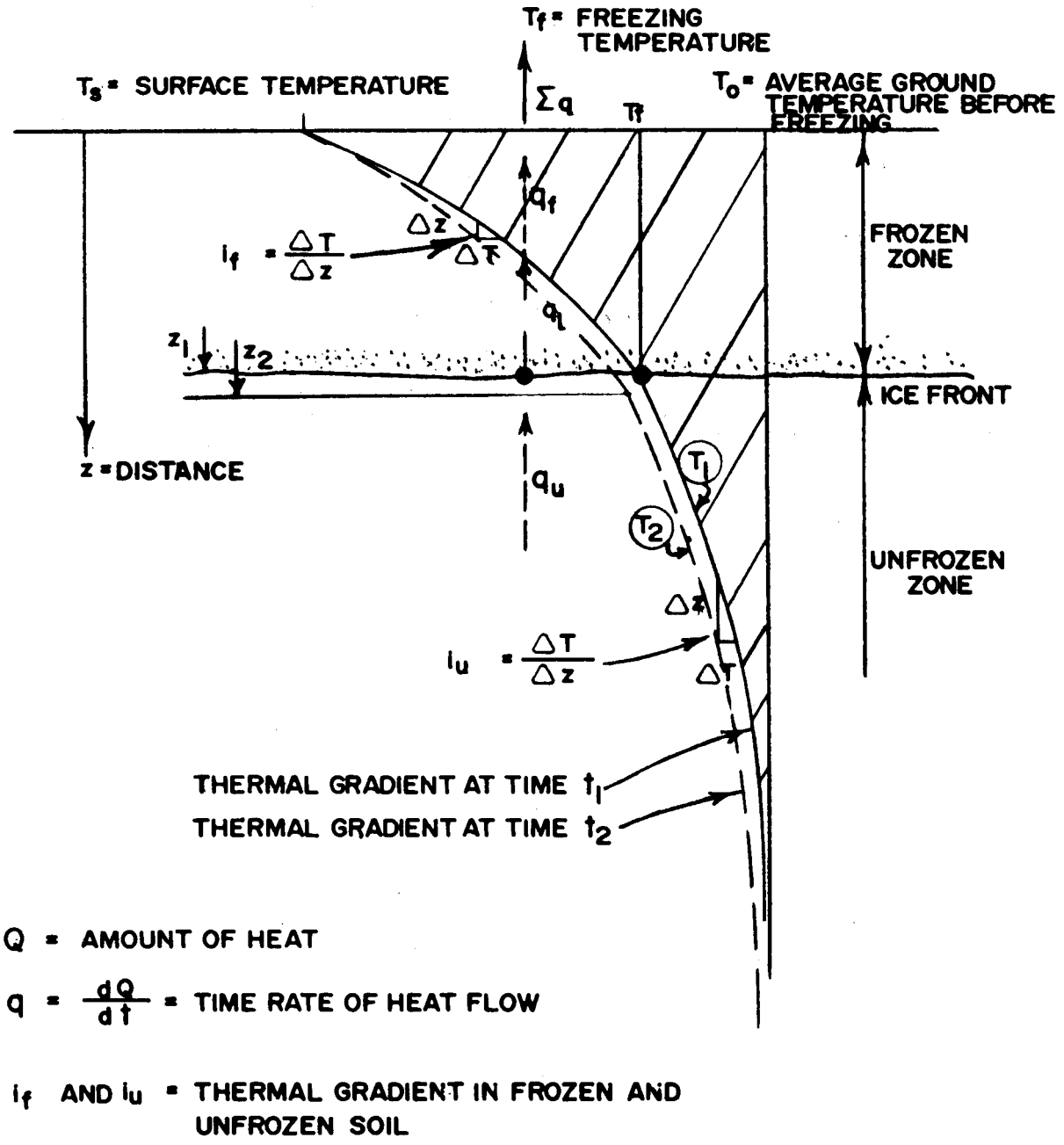


Figure 148. Heat flow under thermal gradient.

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 inital 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.

$$C_u = \gamma_d (.2 + \frac{w}{100}) = 105\ (.45) = 47\ BTU/ft^3/^oF$$

$$C_f = \gamma_d (.2 + \frac{.5w}{100}) = 105\ (.325) = 34\ BTU/ft^3/^oF$$

$$L = \gamma_d (144) \frac{w}{100} = 105\ (1.44)\ (25) = 3800\ BTU/ft^3/^oF$$

Assume an initial ground temperature (T_o) of 50°F and final average design temperature (T_2) of 10 °F. Then,

$$Q_u = 47\ (50 - 32) = 840\ BTU/ft^3$$

$$Q_L = 3800\ BTU/ft^3$$

$$Q_f = 34\ (32 - 10) = 750\ BTU/ft^3$$

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.

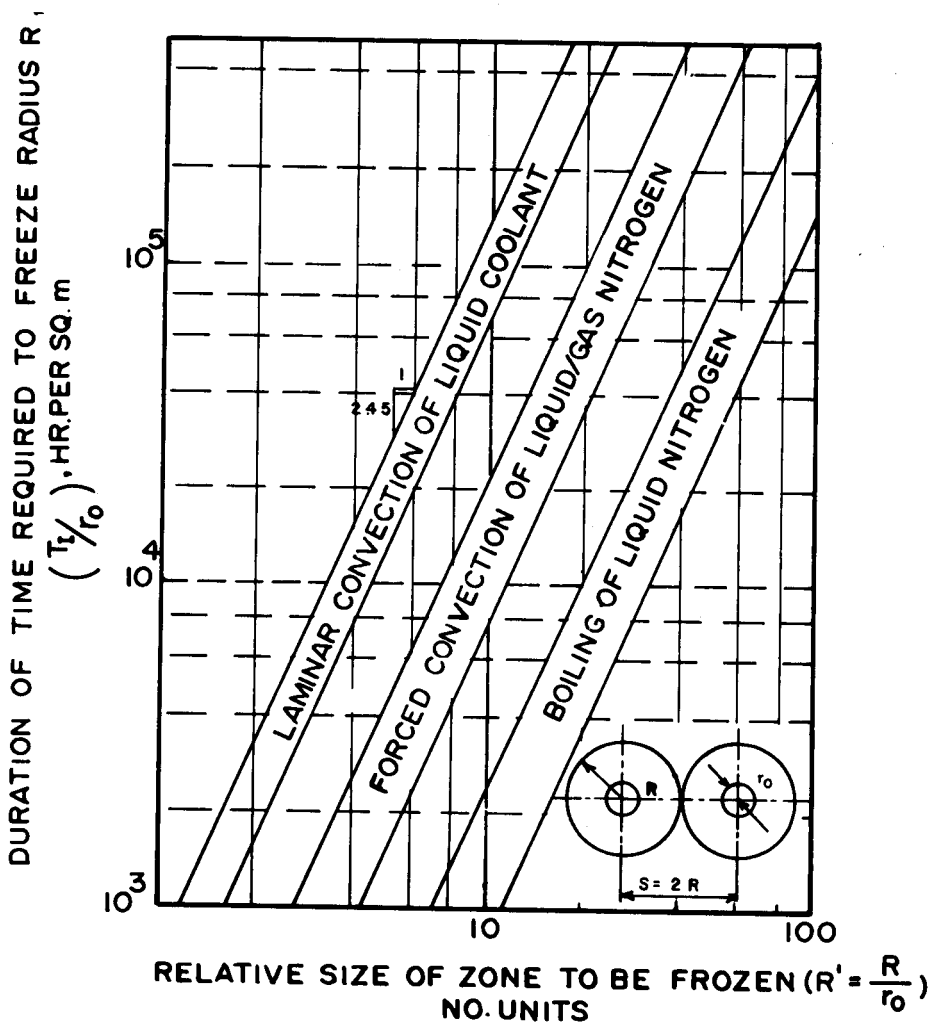
Discussion

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)

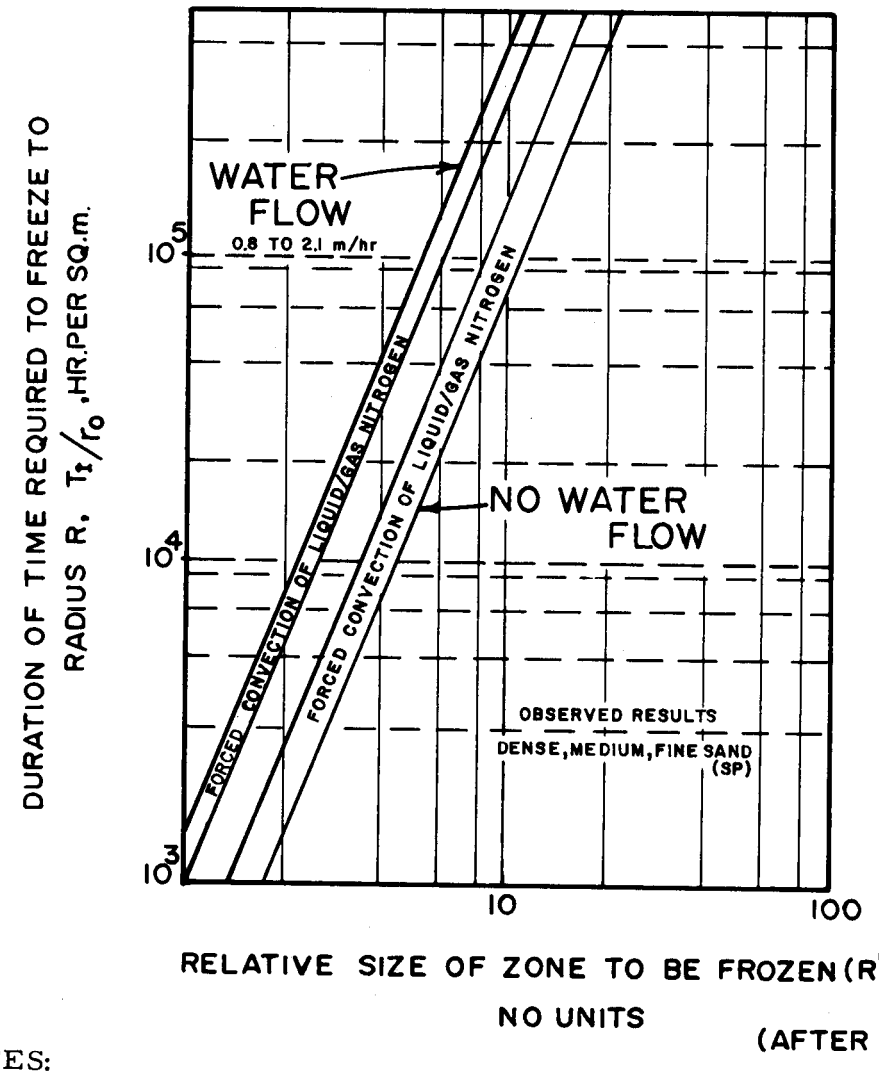
NOTES:

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_o 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_o in meters.
3. T_I = 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 avilable 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

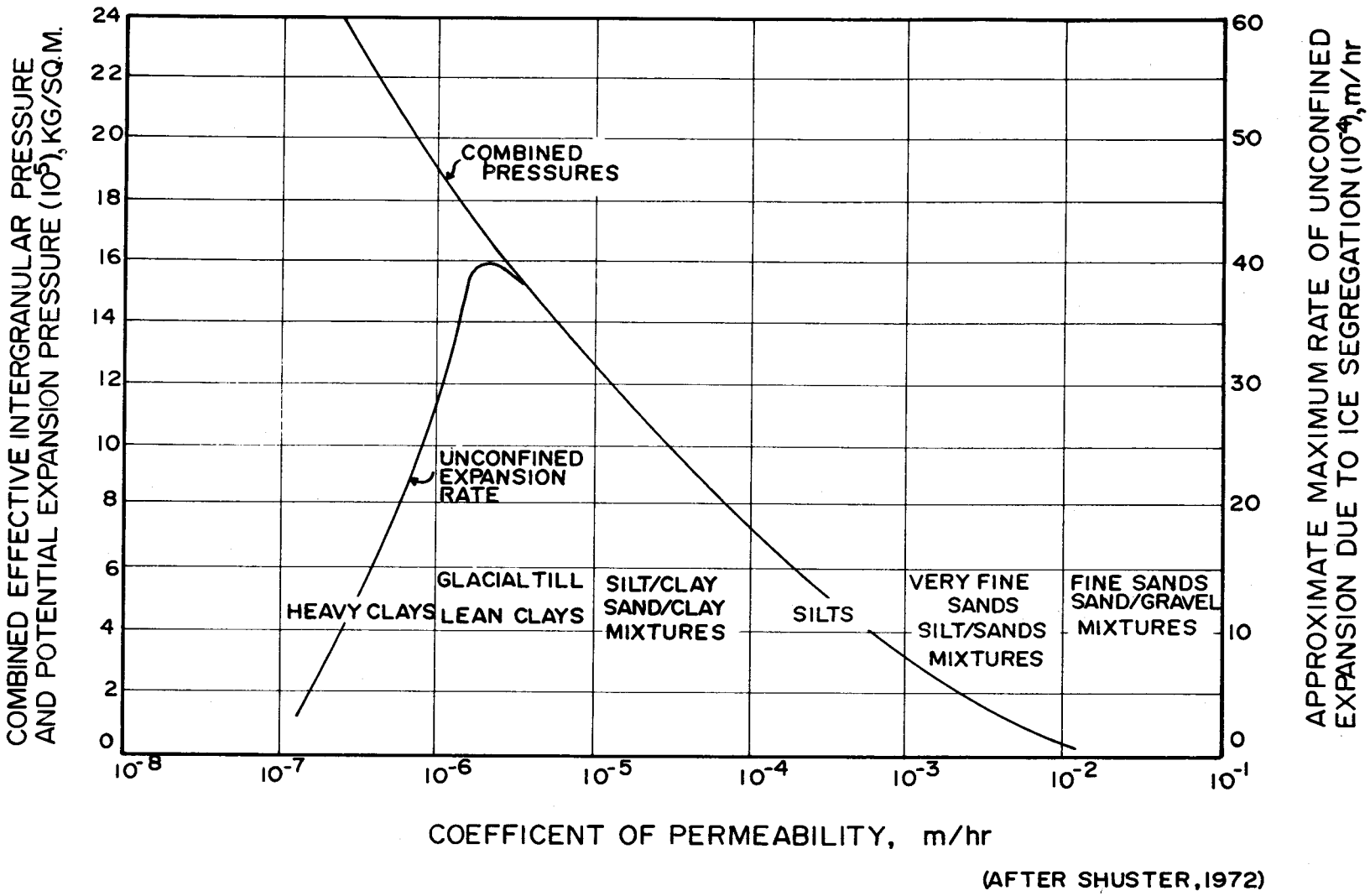


Figure 151. Typical effect of soil type on frost expansion pressures and rates.

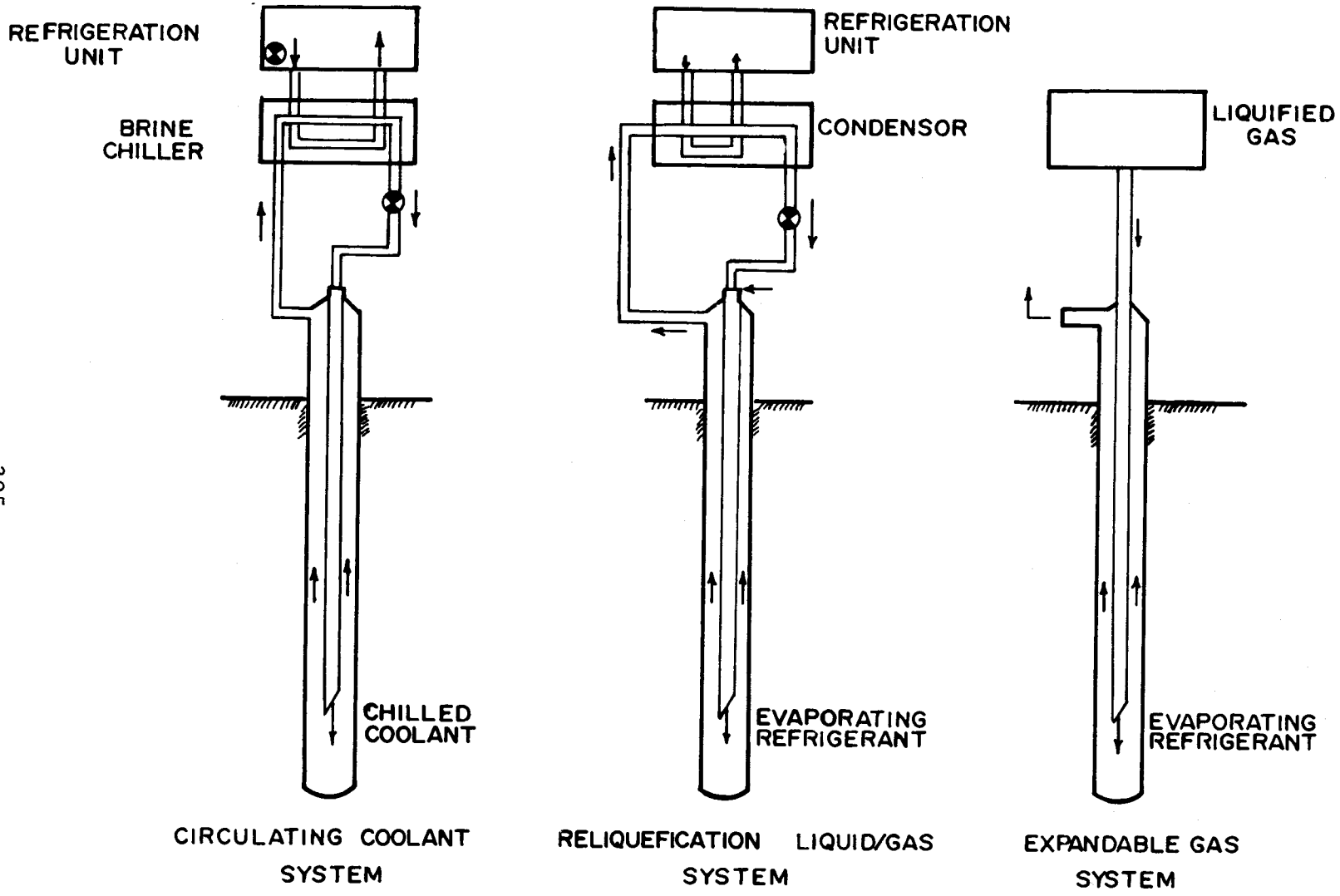


Figure 152. Basic refrigeration system elements for ground freezing.

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.

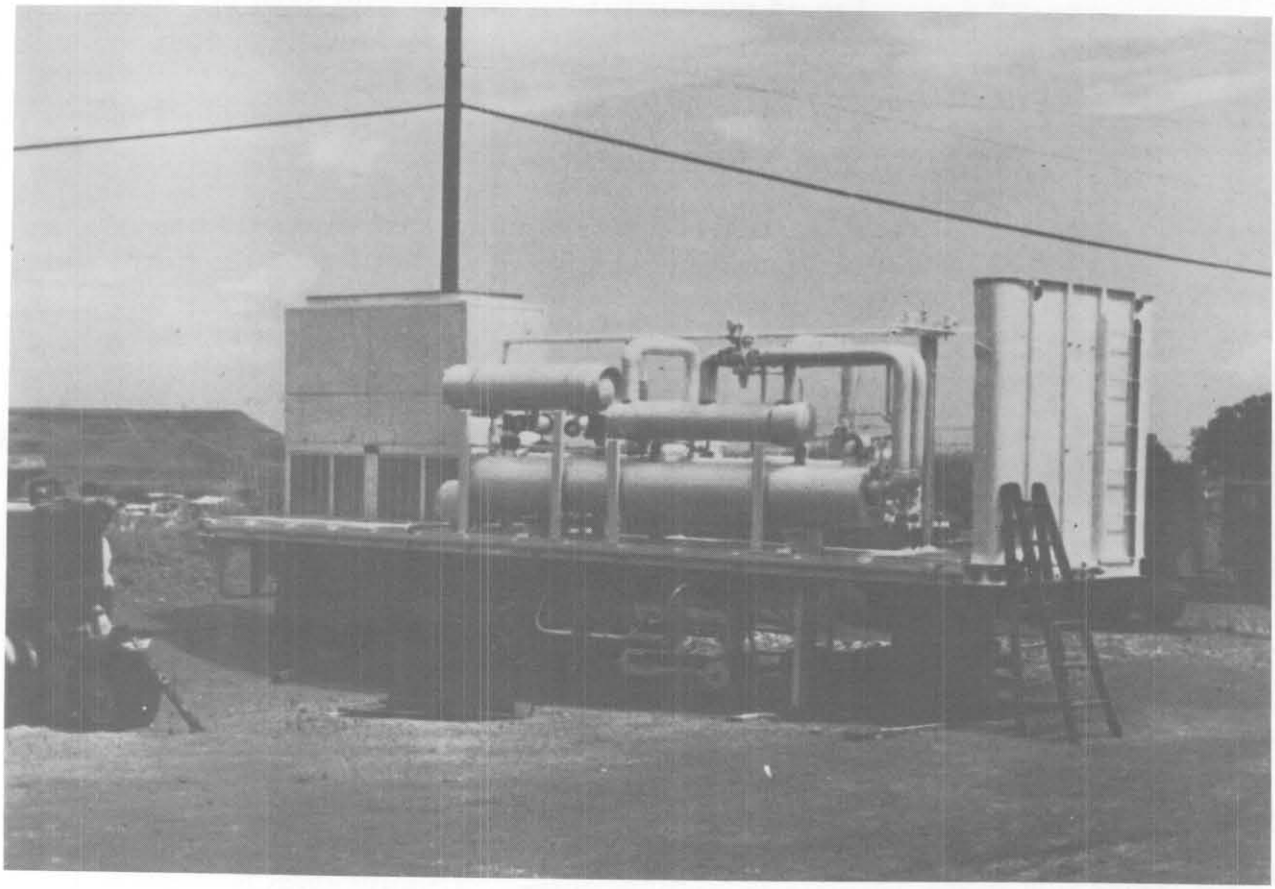


Figure 153. Small mobile freon or ammonia refrigeration plant.
(Courtesy of Terrafreeze Corporation).

However, as freezing operations which normally might take a few weeks can be compressed to a few days; the direct use of LN_2 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 CO_2 as a refrigerant directly from a commerical supplier.

Liquid nitrogen (LN_2) is typically used for short term or emergency situations (see Figure 154 for example).

d. Carbonic Acid

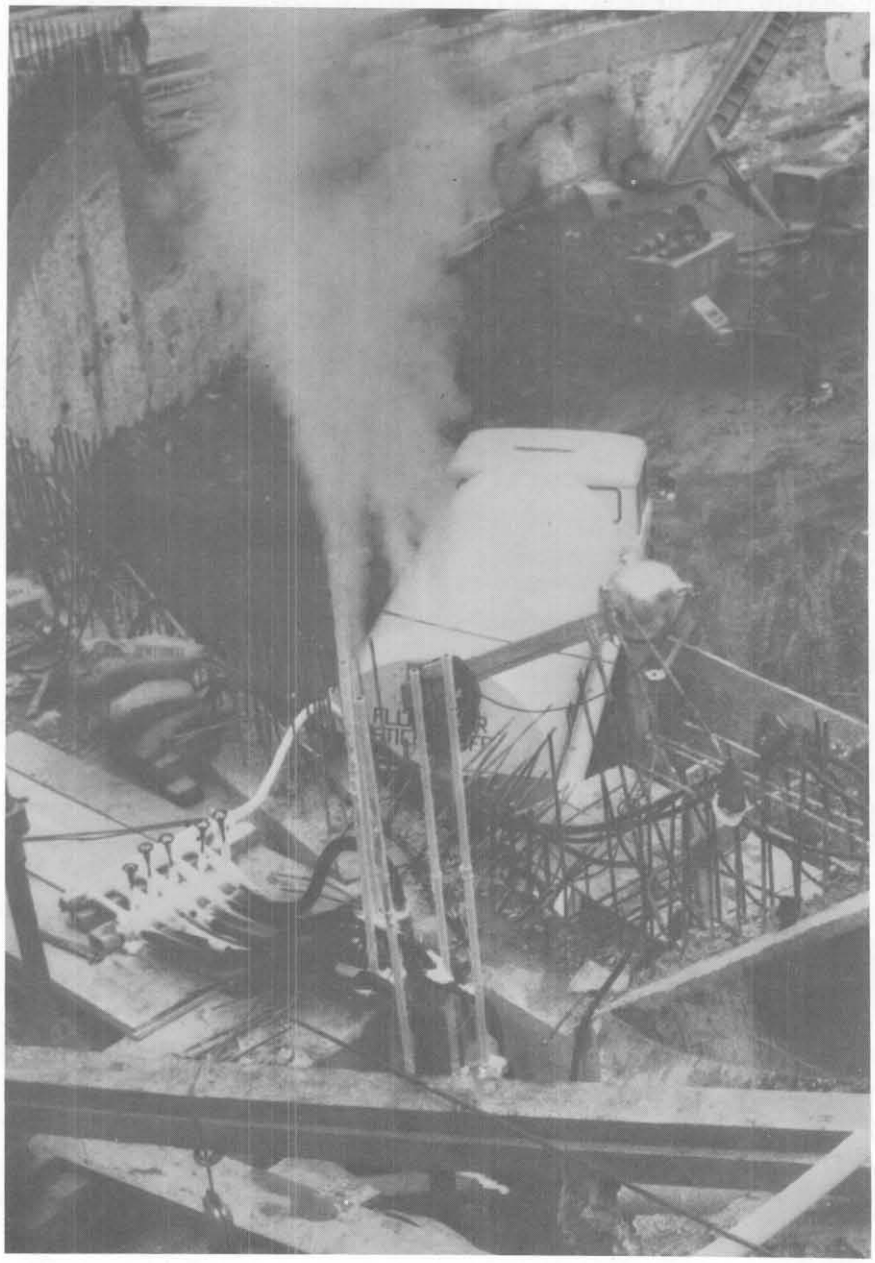
Fujii (1971) has noted that carbonic acid has been used as a refrigerant in Japan and its use might be feasible here.

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 LN_2 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.

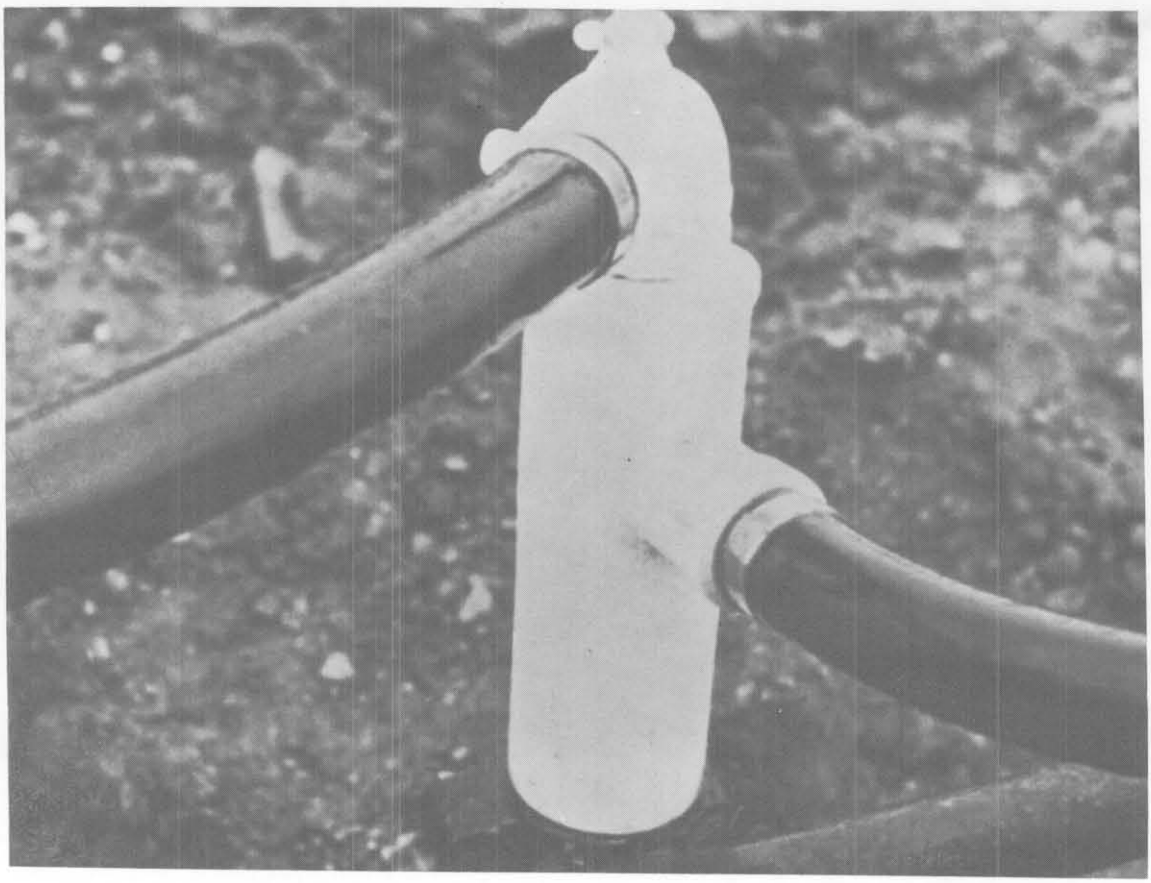
9.30 CONSTRUCTION PROCEDURE

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.

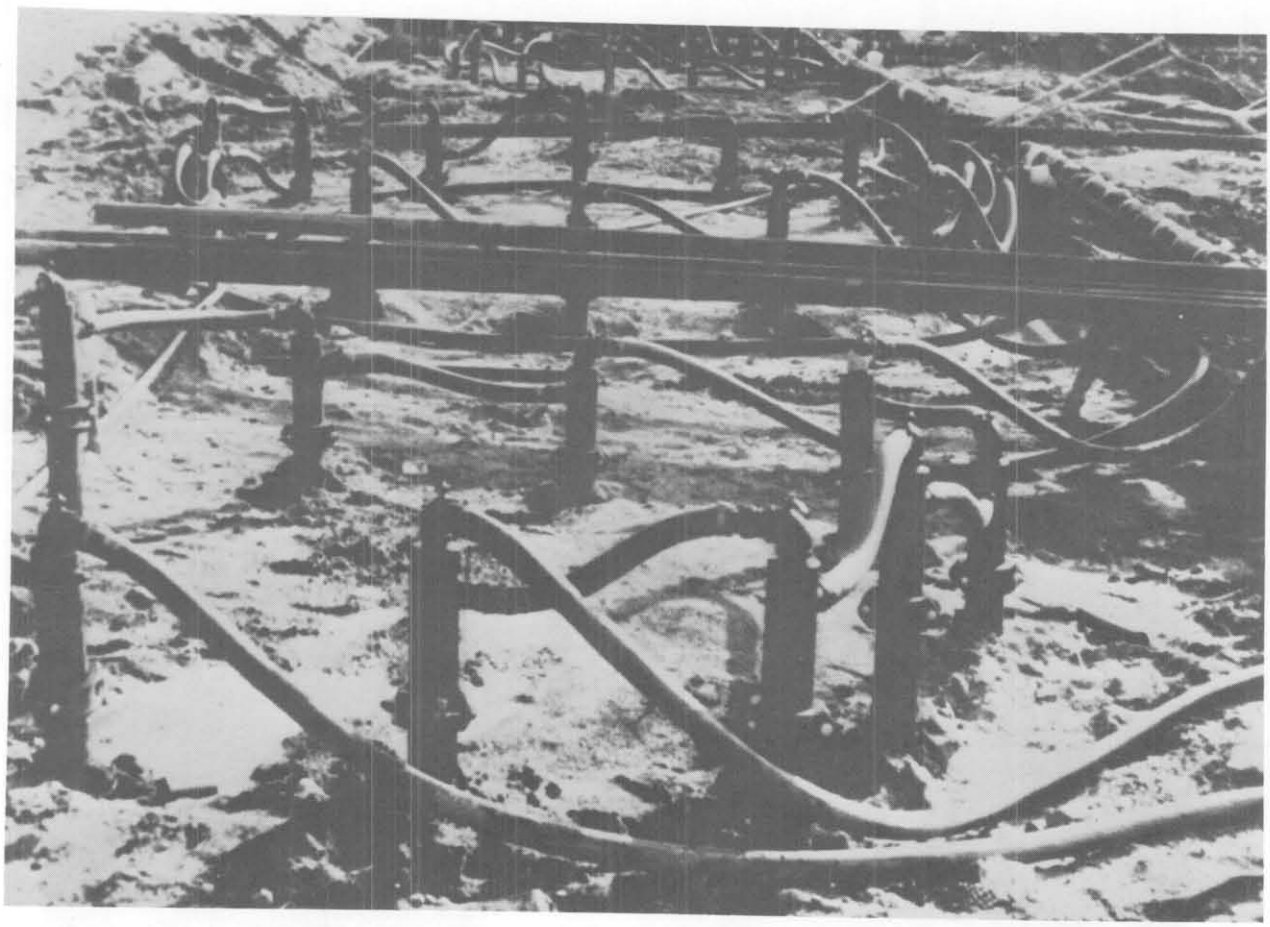
Freeze pipes are placed with spacing, s, and probe size, r_o, according to time requirements (see Figures 149 and 150) and required freeze wall thickness for strength. Strength requirements are based upon the type of frozen structure (e.g. gravity wall); strength requirements determine the average temperature 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
TUNNELS		
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
OPEN CUT EXCAVATIONS		
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
OPEN CUT EXCAVATIONS (cont'd)		
120' x 52' deep circular foundation excavations in clay and sand for two chimneys	Kentucky	Terrafreeze, 1975
SHAFTS AND CAISSONS		
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
SHAFTS AND CAISSONS (cont'd)		
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°F 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°F 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.

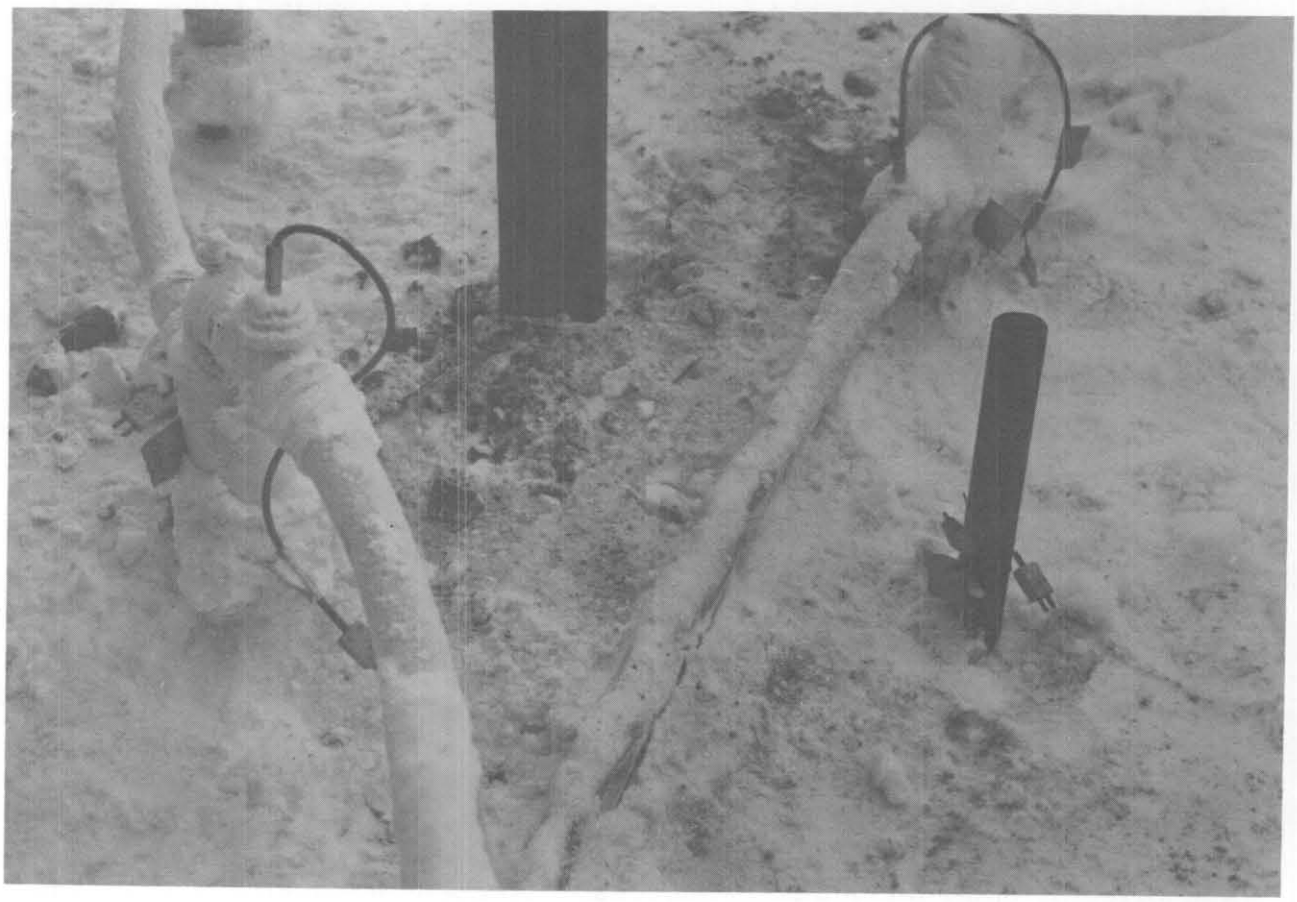
9.34 Construction Monitoring

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 contrator 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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