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LATERAL SUPPORT SYSTEMS AND UNDERPINNING

Vol. I. Design and Construction

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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.

ACKNOWLEDGEMENTS

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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

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

Conversions

British Units	Metric Units
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 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	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	

<u>Symbol</u>	<u>Represents</u>	<u>Reference</u>
P	general symbol for Load or force	
p	general symbol for pressure	
pH	negative logarithm of effective hydrogen ion concentration	
R, r	general symbols for radius	
S, s	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 pressure (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

TRANSLATION

ROTATION
ABOUT BOTTOM

ROTATION
ABOUT TOP

FIXED

(b) WALLS DISPLAYING FLEXURE

TRANSLATION

ROTATION
ABOUT BOTTOM
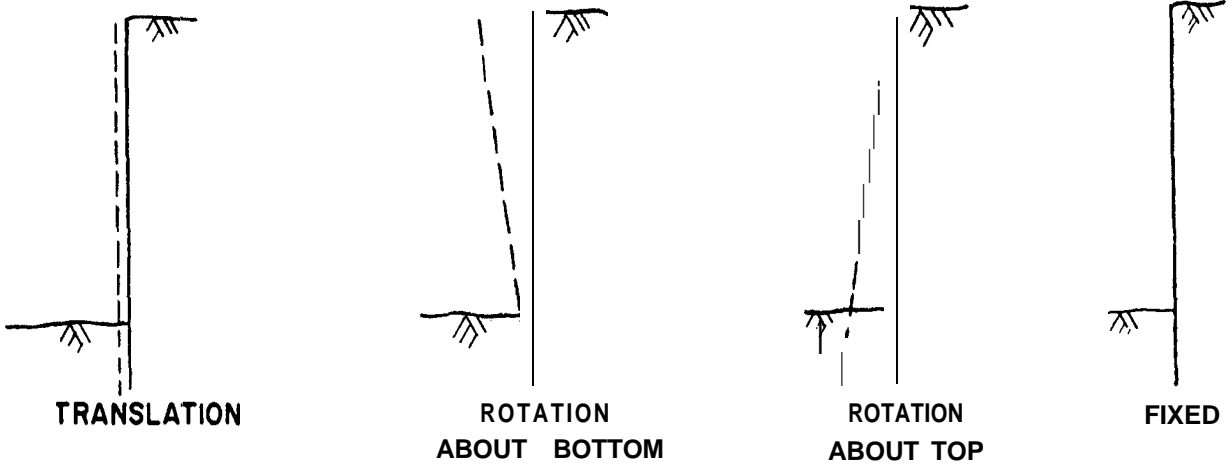
ROTATION
ABOUT TOP
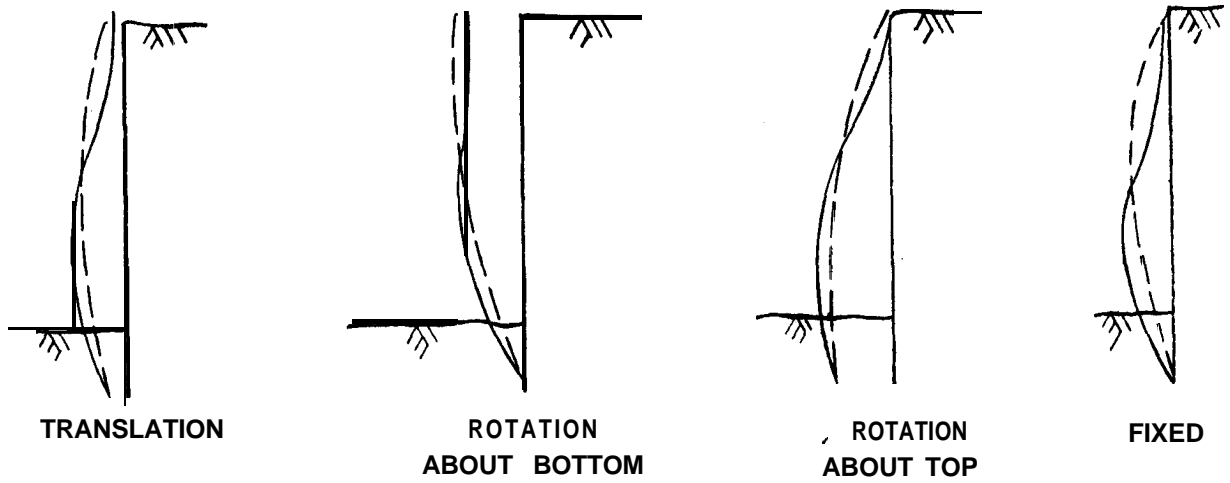
FIXED

Figure 1.   General   deformation   modes.

(a) TYPICAL FOR INTERNAL BRACING

**FIXED OR SLIGHT
TRANSLATION**

"BULGE"
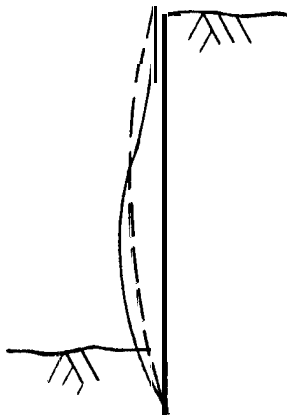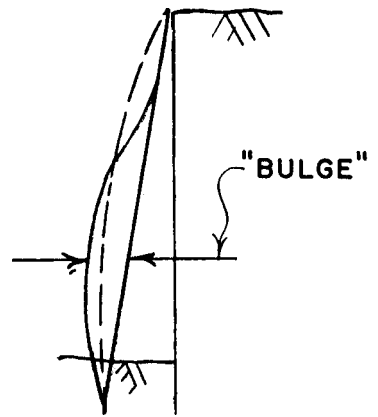
**ROTATION
ABOUT TOP**

(b) TYPICAL FOR TIEBACKS
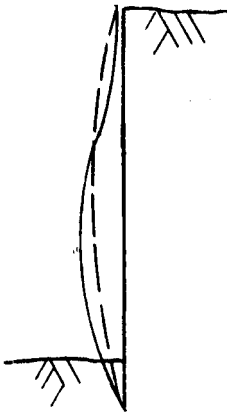
**FIXED OR SLIGHT
TRANSLATION**
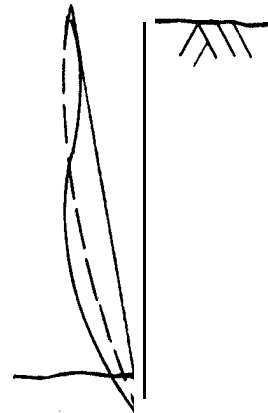
**ROTATION
ABOUT BOTTOM**

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.

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

SAND AND GRAVEL			VERY STIFF TO HARD CLAY $S_u \geq 2000\,psf$			SOFT TO STIFF CLAY $S_u < 2000\,psf$			OTHER SOIL CONDITIONS		
TIEBACKS	PRESTR. BRACING	BRACING (STAND.)	TIEBACKS	PRESTR. BRACING	BRACING (STAND.)	TIE BACKS	PRESTR. BRACING	BRACING (STAND.)	TIEBACKS	PRESTR. BRACING	BRACING (STAND.)

●28　　●19
●32　　　●3
●4　●33　●2

●25 ●31 ○52
●30　　　●8　　●9　○6
●15 ●55
○7*
●34　　　　　　　　　　　　　○36

○43
○5
　○38　○13
DIAPHRAGM WALL

●23b

●12

LIMITS OF ZONE I (PECK, 1969)

●16　●22　　●61
STEEL WALL
●10　●57
　●58
○39*

LIMITS OF ZONE II (PECK, 1969)

─── 75% OF CASES EXPERIENCED LESS MAXIMUM MOVEMENT.

───── 100% OF CASES EXPERIENCED LESS MAXIMUM MOVEMENT.

＊ ATYPICAL OR UNUSUAL CASES. REFER TO THE TEXT AND TABLE I.

NOTE: NUMBERS REFER TO REFERENCES IN TABLE I.

● ↓ 5% - DISPLACEMENT OFF SCALE, MAGNITUDE EQUAL TO 5%.

●14　○40*
●60
●59
ALL WALLS
●11

(ORGANIC SILT)　●1*

●23g ↓ 5%

26 ↓ 3.1%
(ORGANIC SOILS)

20 ↓ 35%
(SOFT CLAY AND PEAT)

Vertical axis: $\dfrac{\delta v_{max}}{H}\,,\ \%$ — 0.5, 1.0, 1.5, 2.0, 2.5

Figure 3.　Normalized vertical movements.

O = DIAPHRAGM WALL
● = SOLDIER PILE OR STEEL SHEETING

SAND AND GRAVEL			VERY STIFF TO HARD CLAY $S_u \geq$ 2000 psf			SOFT TO STIFF CLAY $S_u <$ 2000 psf			OTHER SOIL CONDITIONS		
TIEBACKS	PRESTR. BRACING	BRACING (STAND.)	TIE BACKS	PRESTR. BRACING	BRACING (STAND.)	TIEBACKS	PRESTR. BRACING	BRACING (STAND)	TIE BACKS	PRESTR. BRACING	BRACING (STAND.)

$\frac{\delta_{hmax}}{H}$, %

o 47 049 o 41 041
●25 ●2 ●3 08 042
●29 ●32 ●33 ●19 ●54 ●45 030 052 06 056 038 043
50 ●27 051 ●48 013 05 037 — DIAPHRAGM WALLS
●4 ●15 044 036 ●16
●24 033 (ORGANIC SILT)

0.5

●34 ●59

07 046 ●12 ●21 028 (ORGANIC SOILS) 039

1.0

●11 ●1 (ORGANIC SILT)

035 ●22 ●14 20 (SOFT CLAY AND PEAT

1.5 — 75% OF CASES EXPERIENCED LESS MAXIMUM MOVEMENT.

STEEL WALLS ●61

---- 100% OF CASES EXPERIENCED LESS MAXIMUM MOVEMENT. ●17 ●57

※ ATYPICAL OR UNUSUAL CASES. REFER TO THE TEXT AND TABLE I. STEEL WALL

2.0

NOTE: NUMBERS REFER TO REFERENCES IN TABLE I. ●59

● ↓ % = DISPLACEMENT OFF SCALE, MAGNITUDE EQUAL TO 5% 23 4.2%

2.5

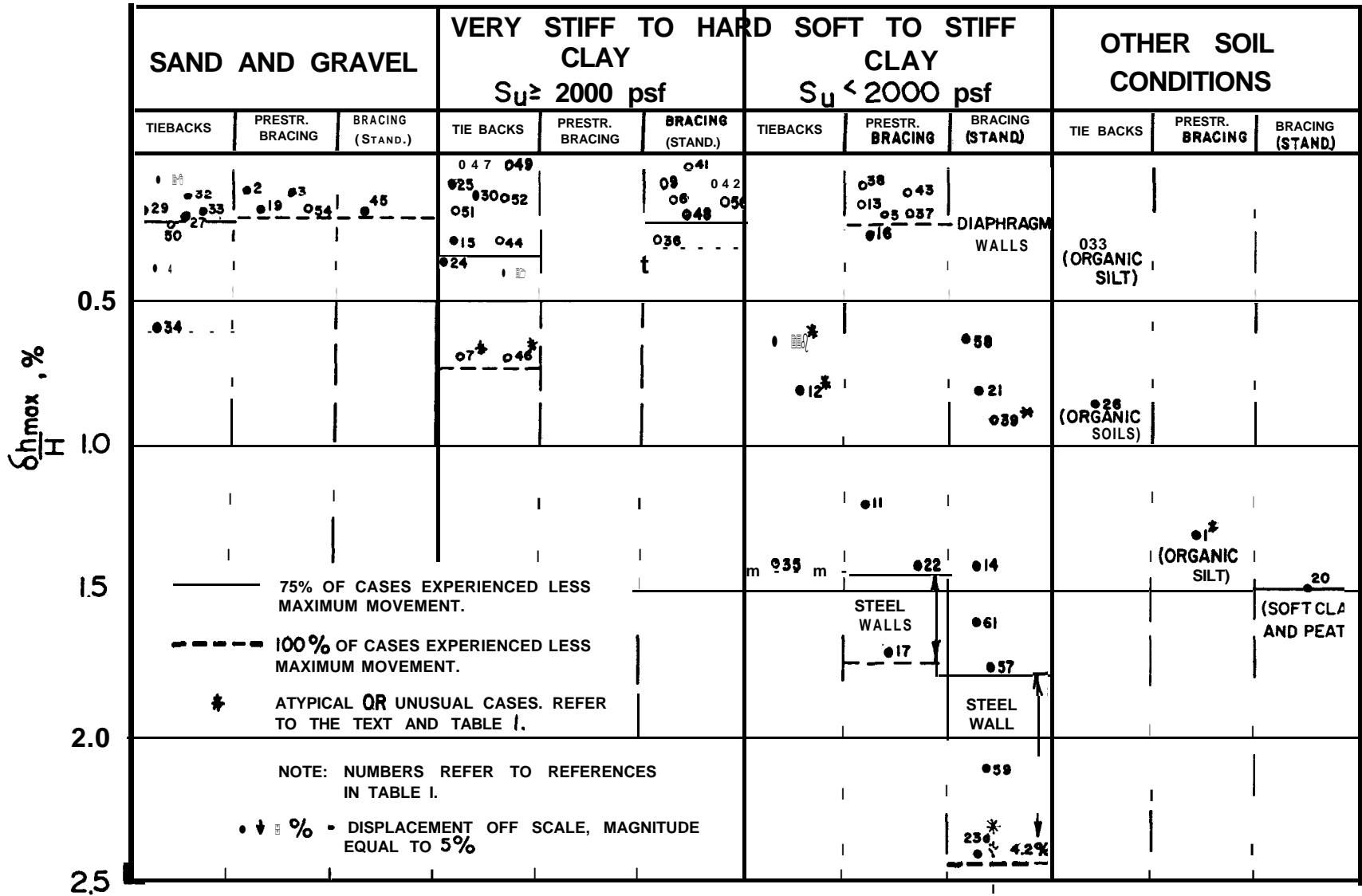**Figure 4. Normalized horizontal movements.**

O = DIAPHRAGM WALLS
. = SOLDIER PILES OR STEEL SHEETING

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	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 (1974) (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 (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 downdrag 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.
II	N.G.I. (1962) (Vaterland #2)	SSP	Struts (Prestressed)	Soft to medium clay	36' (11m)	8.9" (22.6cm)	5.1" (I3cm)	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 deflection.

See Sheet 5 for notes.

Sheet 1 of 5

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' ±. 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, Hoiman, 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, Hoiman, 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	δ_v max	δ_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	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" (IO.2cm)	Cracking in street indicated potential stability failure (6'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)	I"/_ (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)	I" (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)	I.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)	I" (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.

Sheet 3 of 5

-9-

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 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. 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. 1. (1962) Vaterland #1	SSP	Struts	Medium and soft clay	36' (11.0m)	7.9" (20.1cm)	9" (22.9cm)	
60	N.G.1. (1962) Grönland #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	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)	

Sheet 5 of 5

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.

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 (S_u), 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.
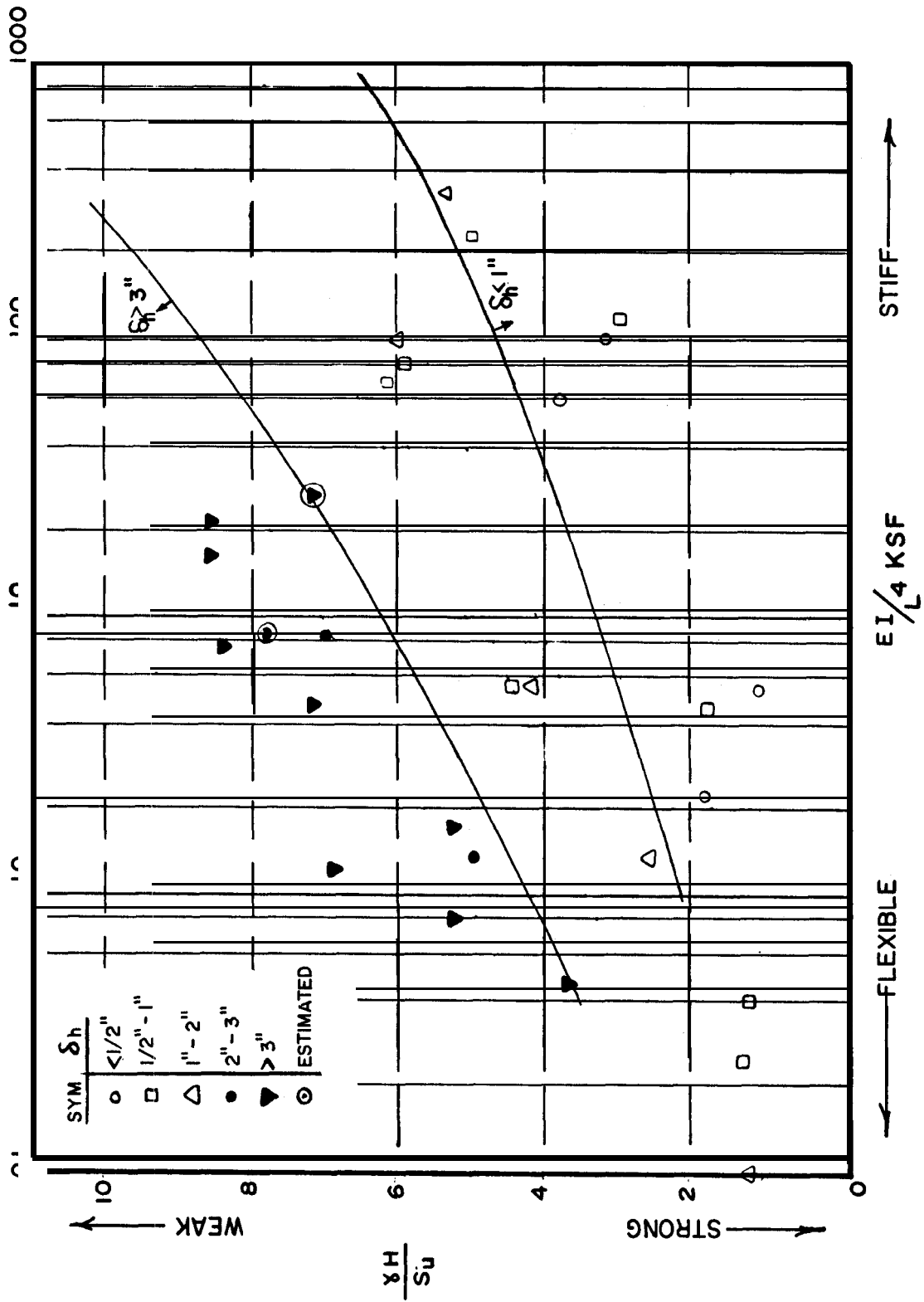
_____
*Ratio of overburden stress to undrained shear strength.

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.
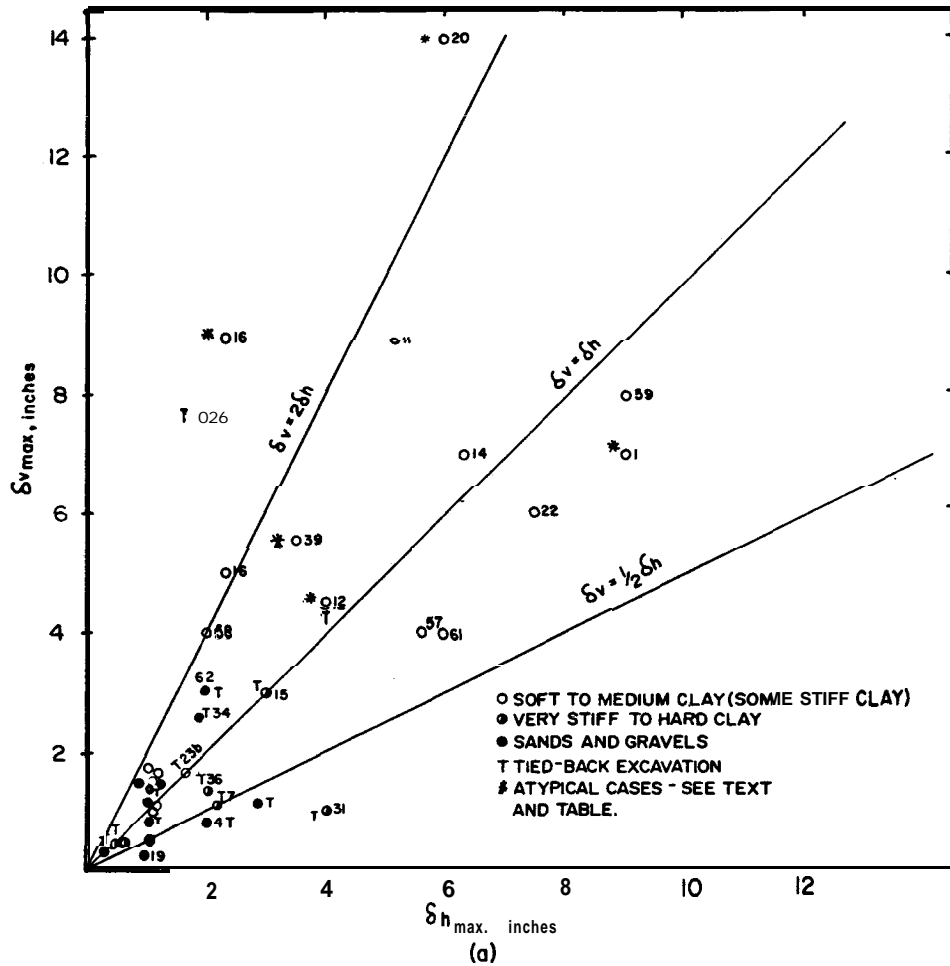
Very Stiff to Hard Clays

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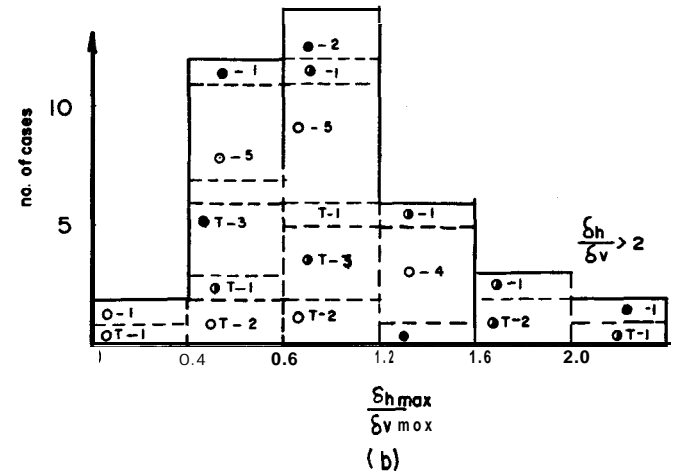
The finite element computer program used to develop thesedata considered only cohesive soils and internally braced excavations; See Volume II (Design Fundamentals) for soil properties and methods of analysis,

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.
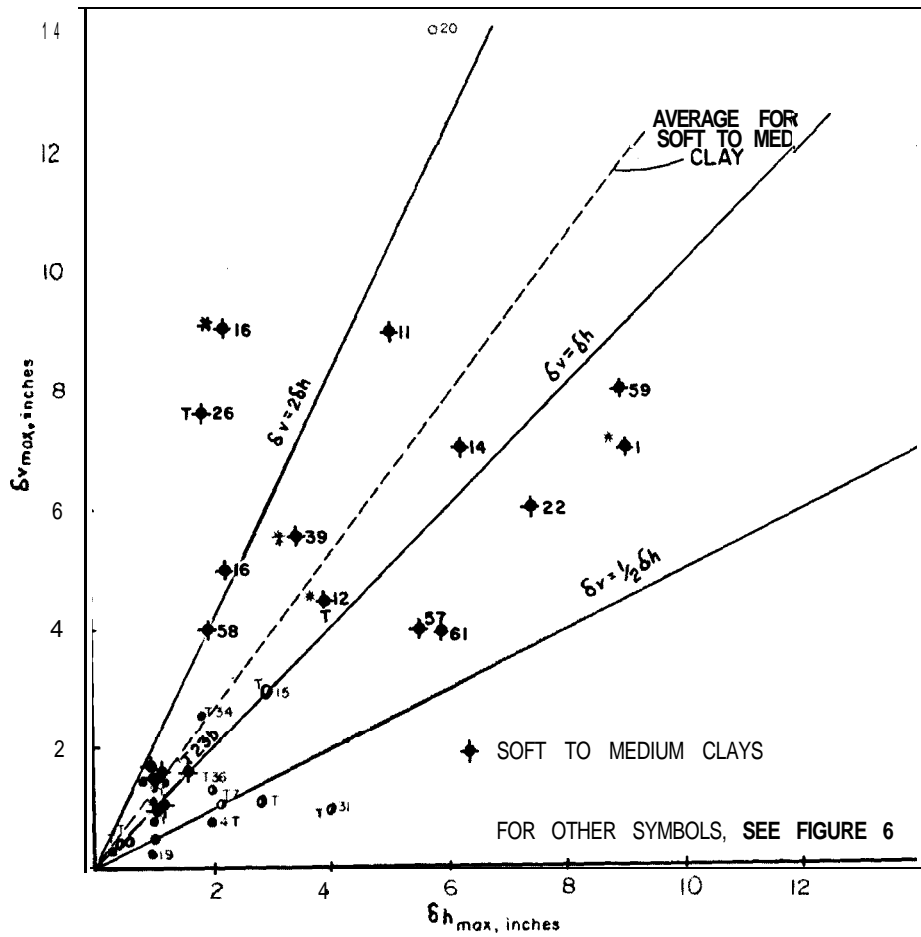
(a) MAXIMUM VERTICAL AND HORIZONTAL DEFORMATIONS

δv_{max}, inches (vertical axis): 14, 12, 10, 8, 6, 4, 2

$\delta h_{max.}$ inches (horizontal axis): 2, 4, 6, 8, 10, 12

* O20

$\delta v = 2\delta h$

$\delta v = \delta h$

$\delta v = \frac{1}{2}\delta h$

* O16

$\bar{\tau}$ O26

O59

O14

* O1

O22

* O39

O6

* O12
T

O58

O57 O61

62
T

T O15

T34

T23b

T36

T T

4 T   T

T O31

O19

O SOFT TO MEDIUM CLAY (SOME STIFF CLAY)
● VERY STIFF TO HARD CLAY
● SANDS AND GRAVELS
T TIED-BACK EXCAVATION
* ATYPICAL CASES - SEE TEXT AND TABLE.

(b) FREQUENCY DISTRIBUTION OF RATIO OF MOVEMENTS

no. of cases (vertical axis): 10, 5

$\frac{\delta h_{max}}{\delta v_{max}}$ (horizontal axis): 0.4, 0.6, 1.2, 1.6, 2.0

● -2
● -1   ◑ -1
◑ -5
O -5
● T-3   T-1   ◑ -1
T-1   ● T-3   O -4
O -1   O T-2   O T-2   ◑ -1   ● -1
O T-1   ◑ T-2   ◑ T-1

$\frac{\delta h}{\delta v} > 2$

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

Figure 6.   Comparison of maximum vertical and horizontal displacements,

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

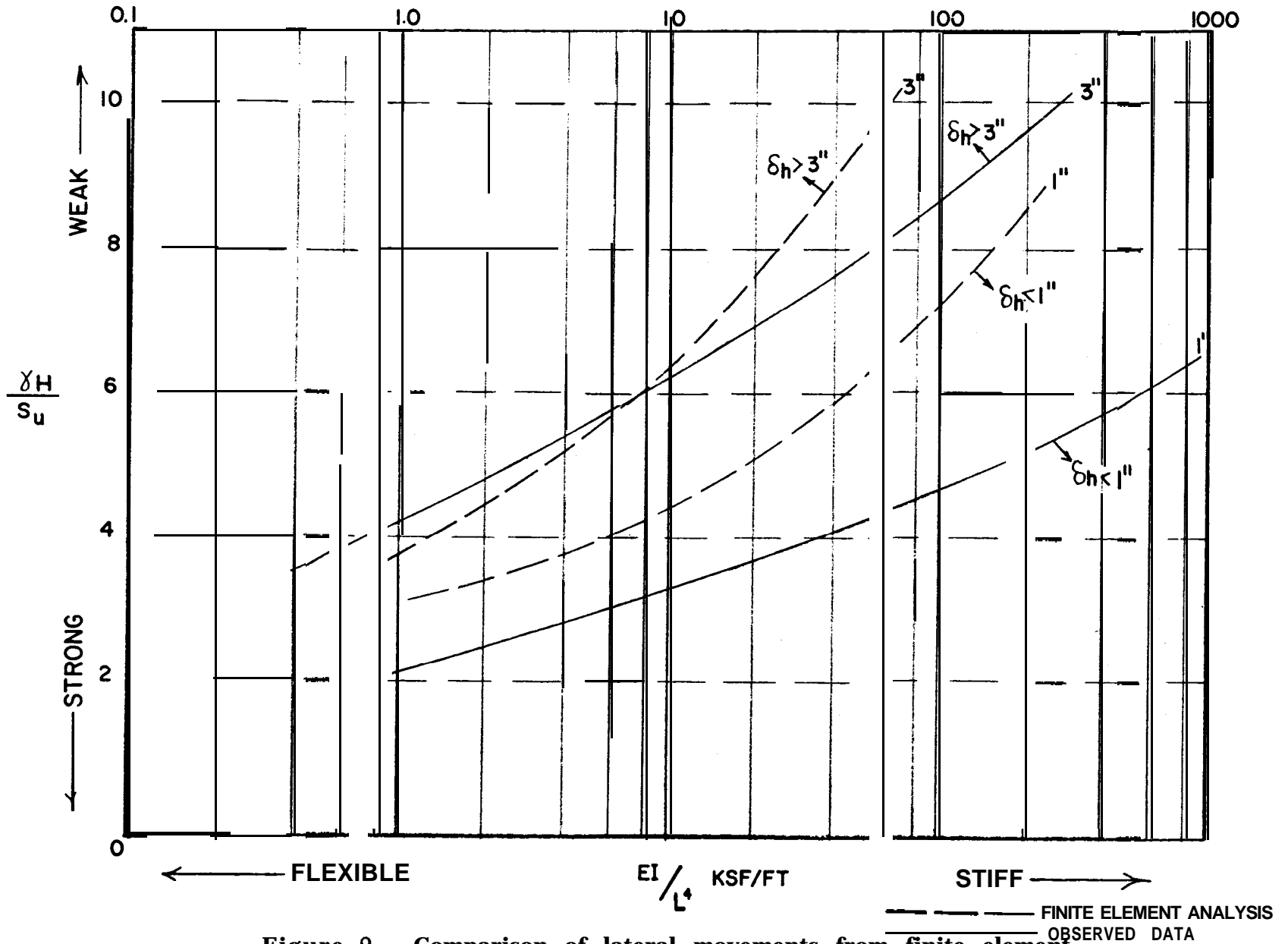Figure 8. Comparison of vertical and horizontal displacements for very stiff to hard clays.

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.

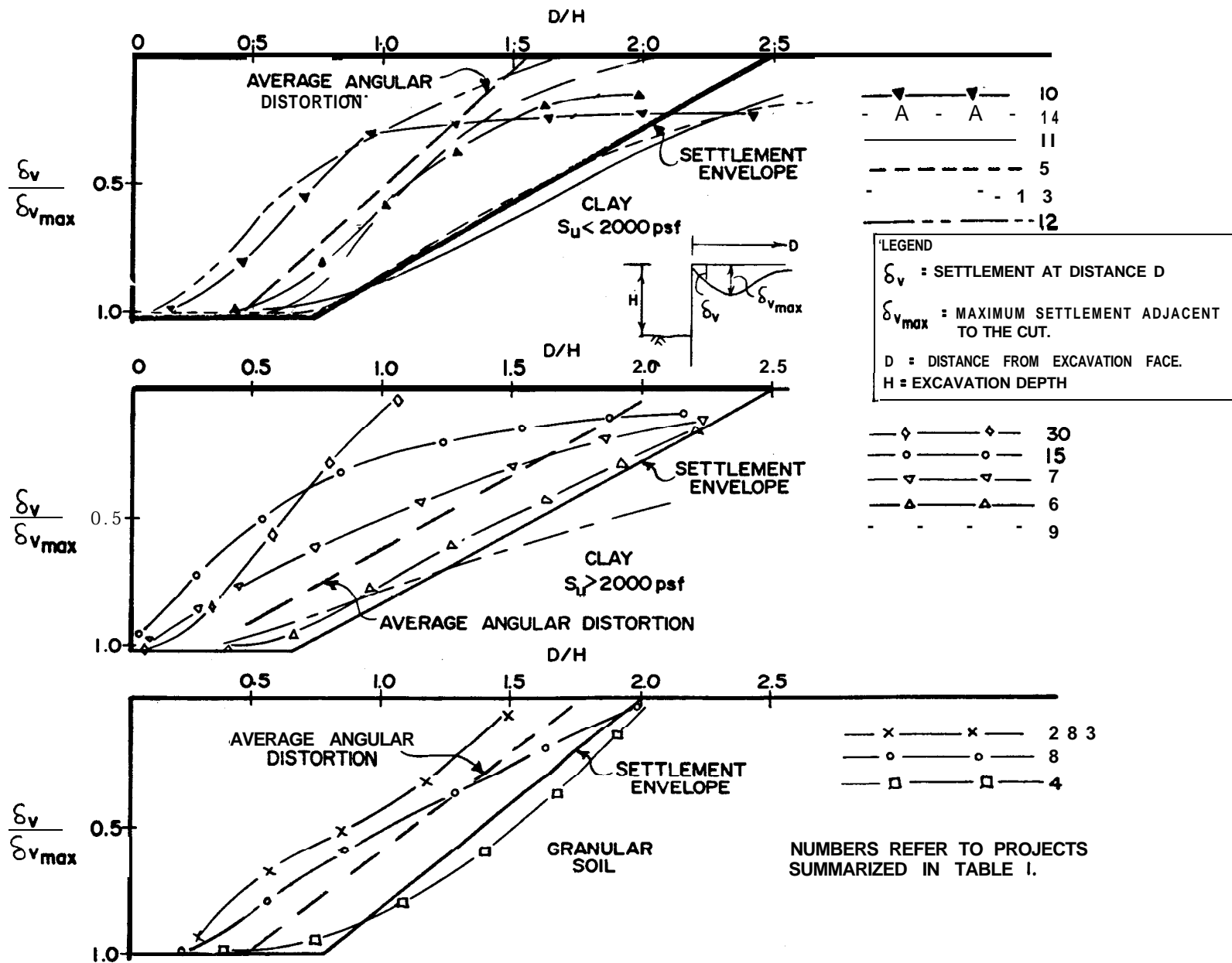Details of the finite element analysis are given in Volume II (Design Fundamentals).

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

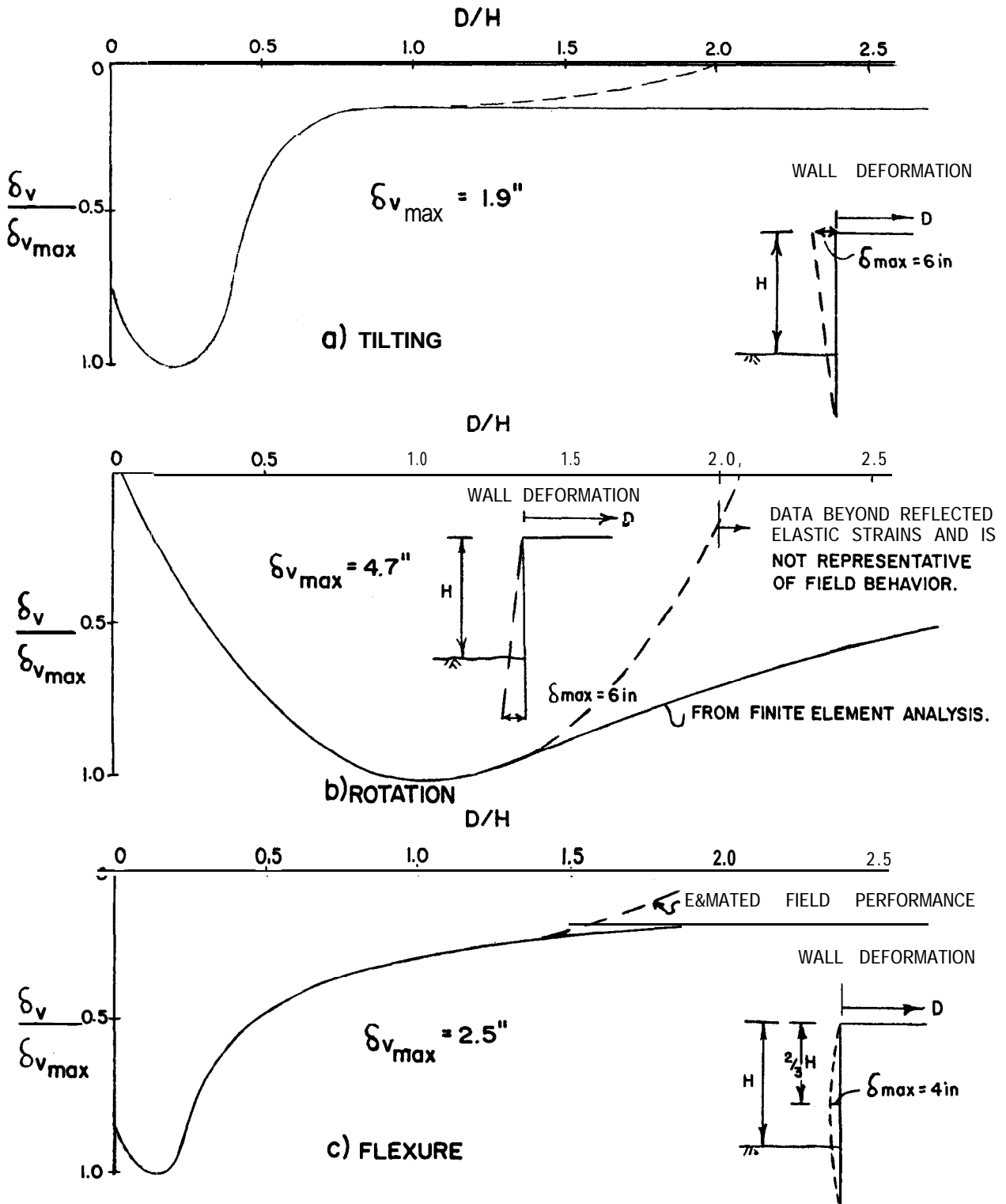Normalized contour plots of horizontal deformations are presented in Figures 13 and 14,

Figure 11. Finite element prediction of surface settlement profiles for normally consolidated clays,
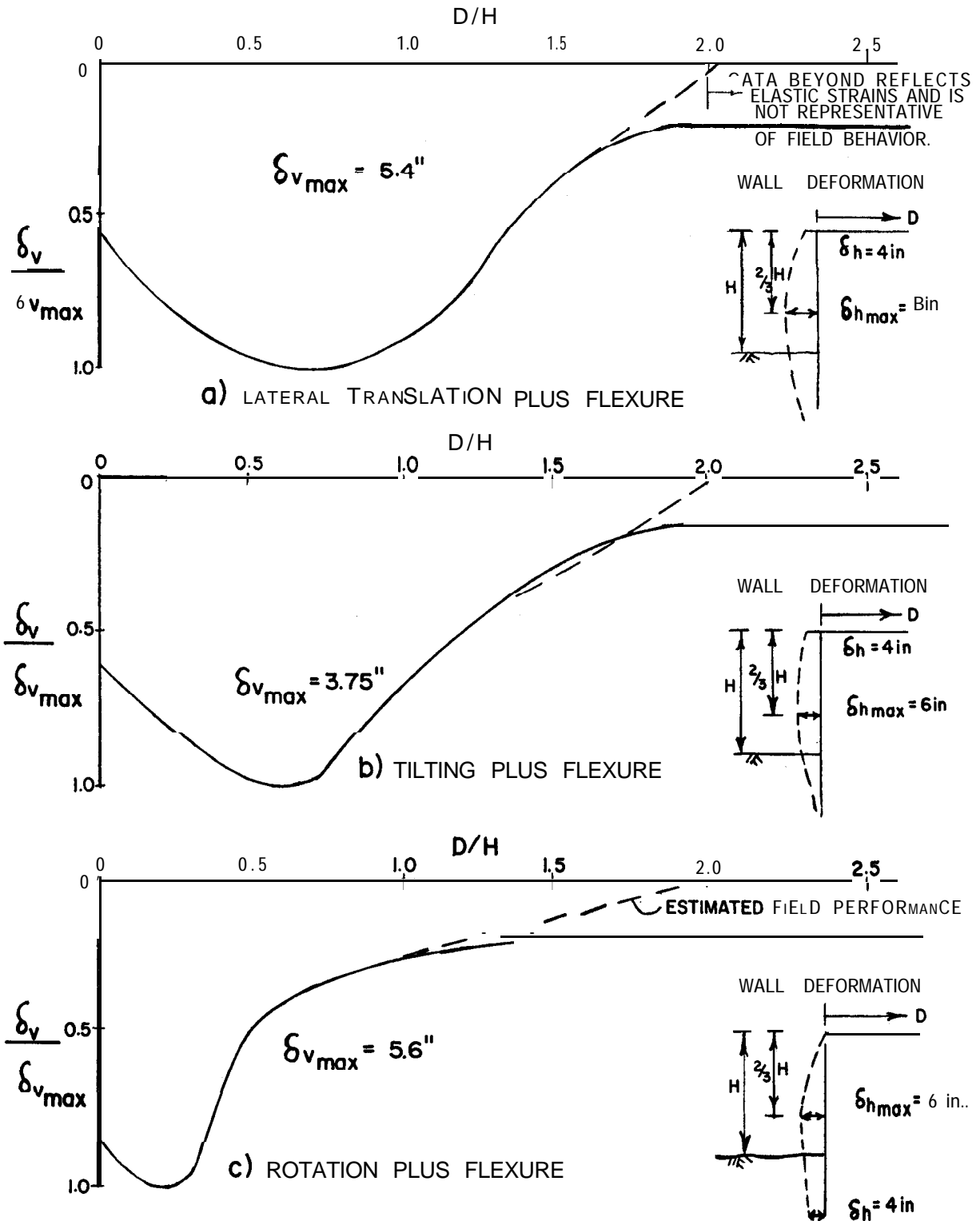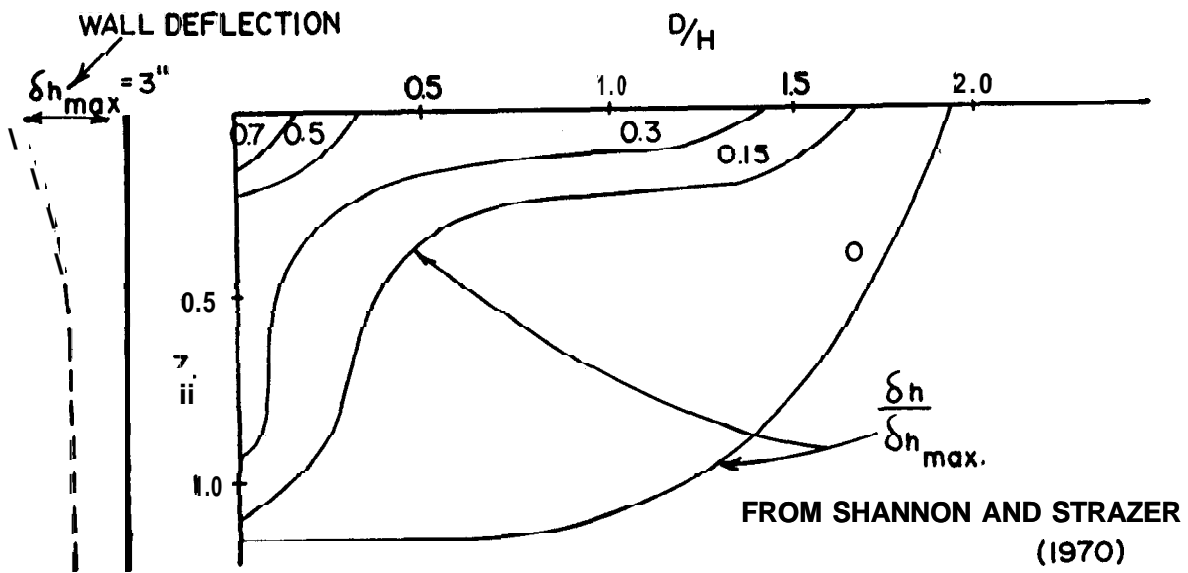
Figure 12. Finite element prediction of surface settlement profiles for normally consolidated clays.
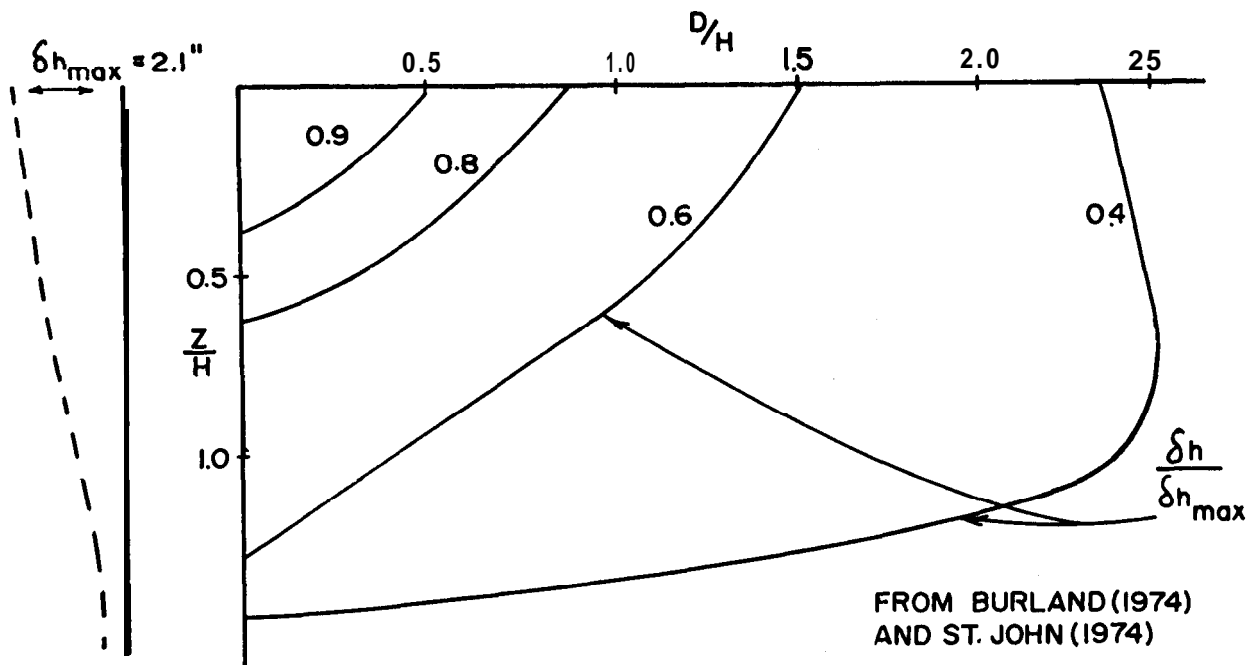
Figure 13. Normalized lateral movement for
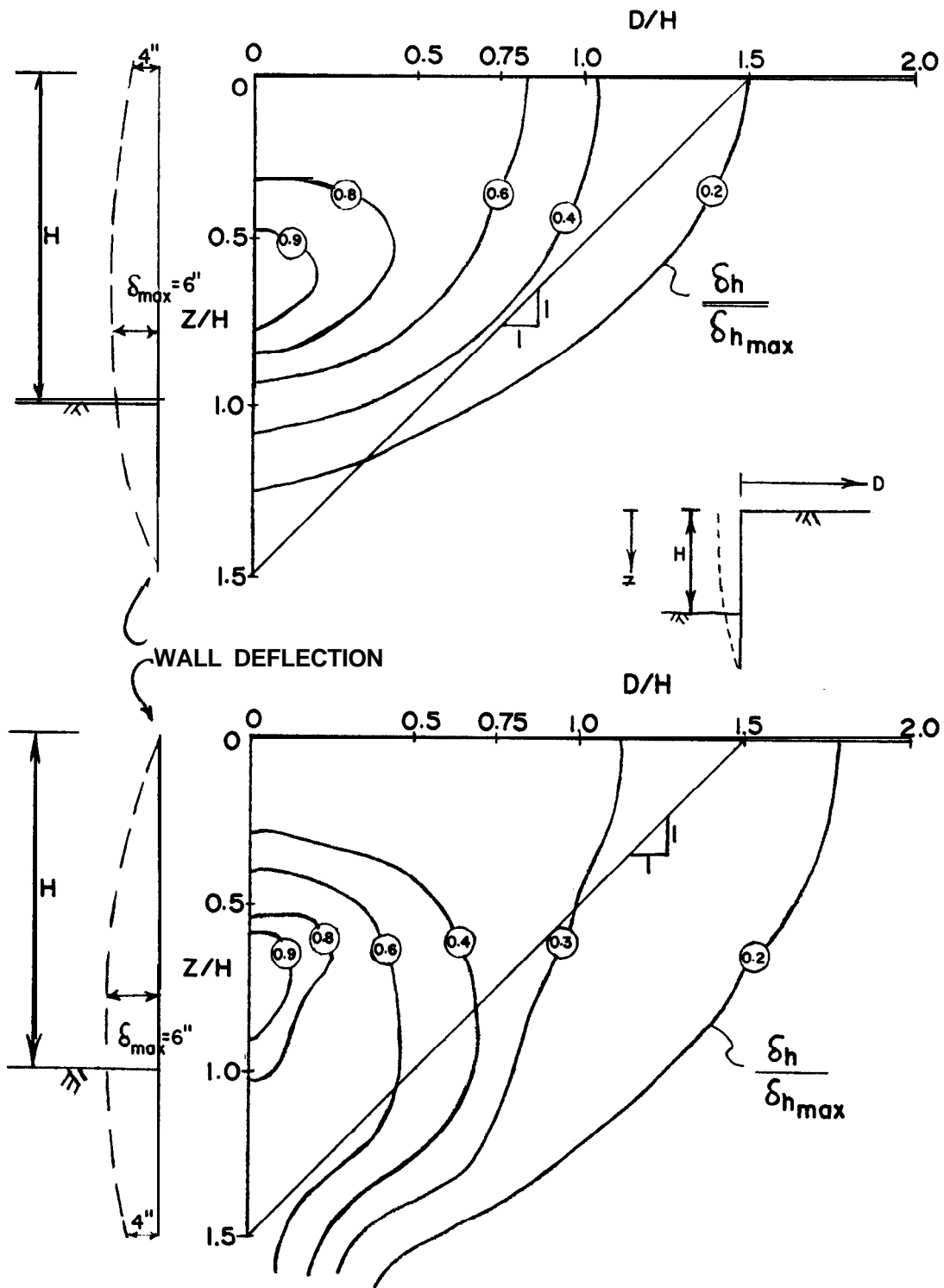tied-back excavation in heavily overconsolidated clays.

Figure 14. Normalized lateral 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 histories 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.
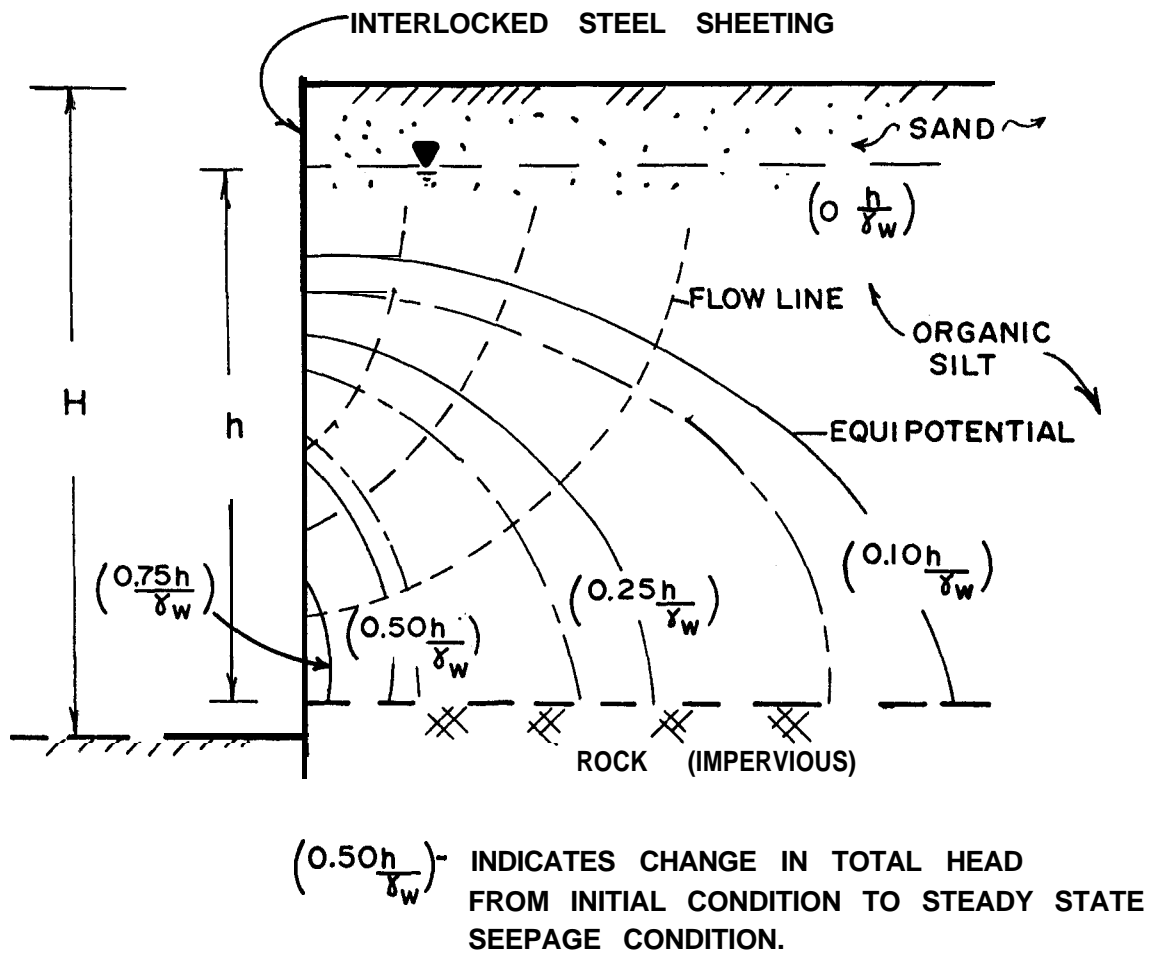
INTERLOCKED STEEL SHEETING

SAND

$\left(0 \dfrac{h}{\gamma_w} \right)$

FLOW LINE

ORGANIC SILT

EQUIPOTENTIAL

$\left(0.10 \dfrac{h}{\gamma_w} \right)$

$\left(0.25 \dfrac{h}{\gamma_w} \right)$

$\left(0.50 \dfrac{h}{\gamma_w} \right)$

$\left(0.75 \dfrac{h}{\gamma_w} \right)$

H

h

ROCK (IMPERVIOUS)

$\left(0.50 \dfrac{h}{\gamma_w} \right)$ — **INDICATES CHANGE IN TOTAL HEAD FROM INITIAL CONDITION TO STEADY STATE SEEPAGE CONDITION.**

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

CHAPTER - 3 LATERAL EARTH PRESSURE

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 refer-red 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

FULLY ACTIVE

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

AT REST

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

(b) ARCHING ACTIVE

FULLY ACTIVE

ARCHING ACTIVE

AT REST
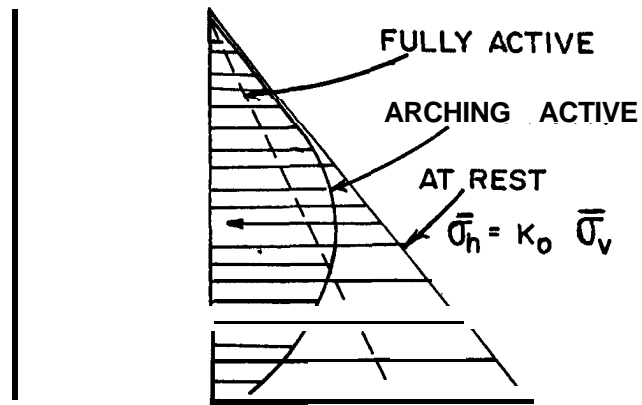
$$\bar{\sigma}_h = K_o \bar{\sigma}_v$$
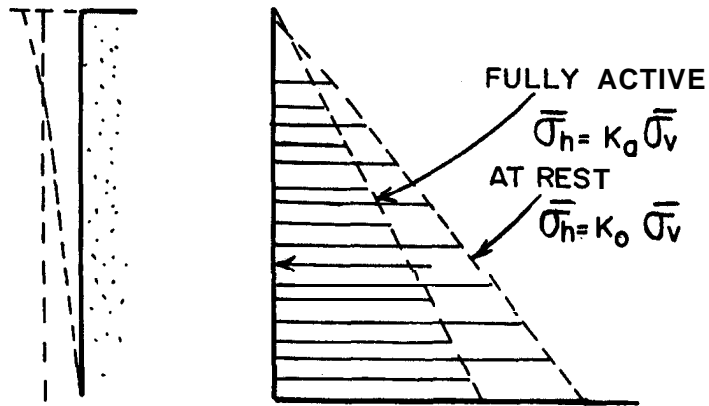
Figure 16.   Earth pressure distributions for active and arching active conditions.

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, $K_a = \tan^2 (45° - \bar{\phi}/2)$

For cohesive soils,

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

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

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

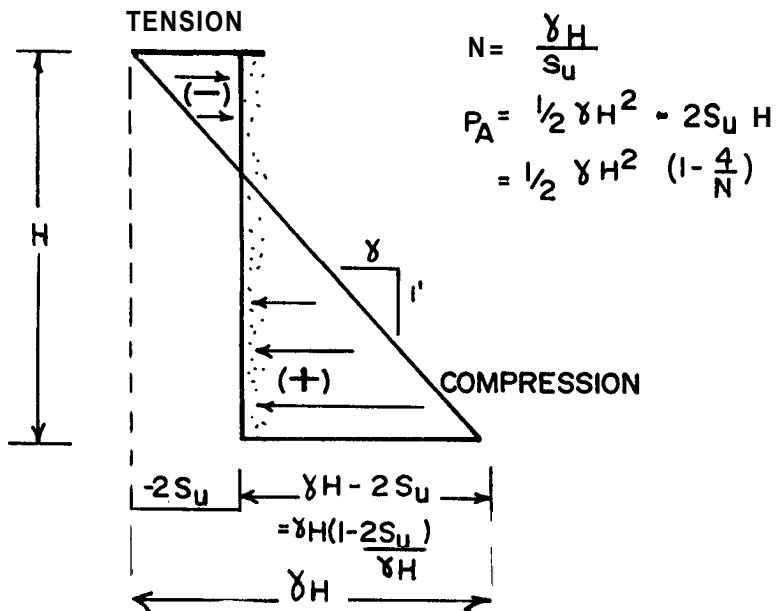$\bar{\sigma}_v$ = vertical effective stress

$\bar{\sigma}_h$ = 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

TENSION

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

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

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

$(-)$

γ

$(+)$ COMPRESSION

H

$-2S_u$ $\gamma H - 2S_u$

$$= \gamma H\left(1 - \frac{2S_u}{\gamma H}\right)$$

$$\gamma H$$

(b) TRIANGULAR PRESSURE DISTRIBUTION EQUIVALENT TO NET RANKINE FORCE

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

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

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

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

H

$\gamma H - 4S_u$

$$= \gamma H\left(1 - \frac{4S_u}{\gamma H}\right) = \gamma H\left(1 - \frac{4}{N}\right)$$
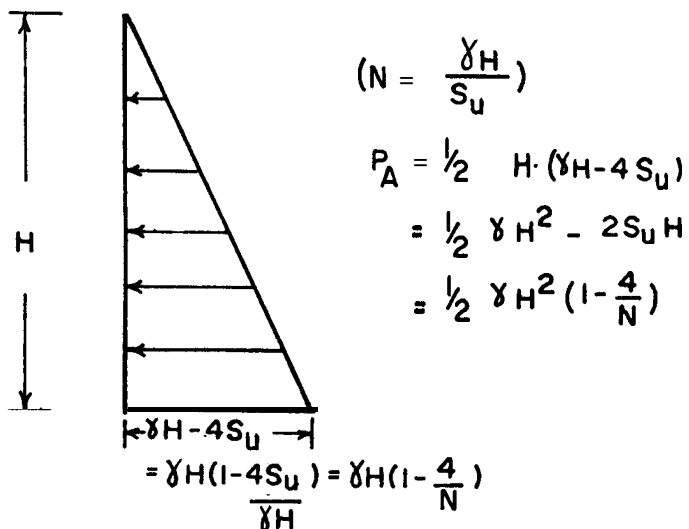
Figure 17   Earth pressure distribution for cohesive soil ($\phi = 0$).

(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.

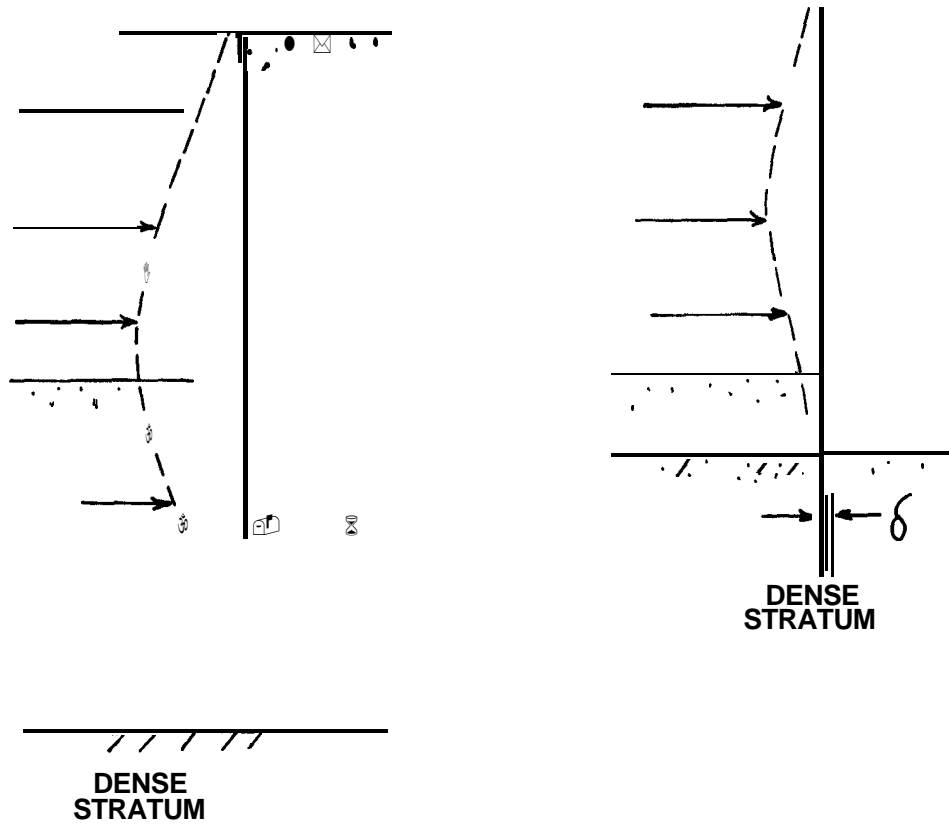The following discussion does not include the effect of surcharge (see Section 3.40).

DENSE STRATUM

DENSE STRATUM

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

REACTIONS
R_A, R_B, R_C

APPARENT EARTH
PRESSURES

$$P_A = \frac{R_A}{L_A}$$
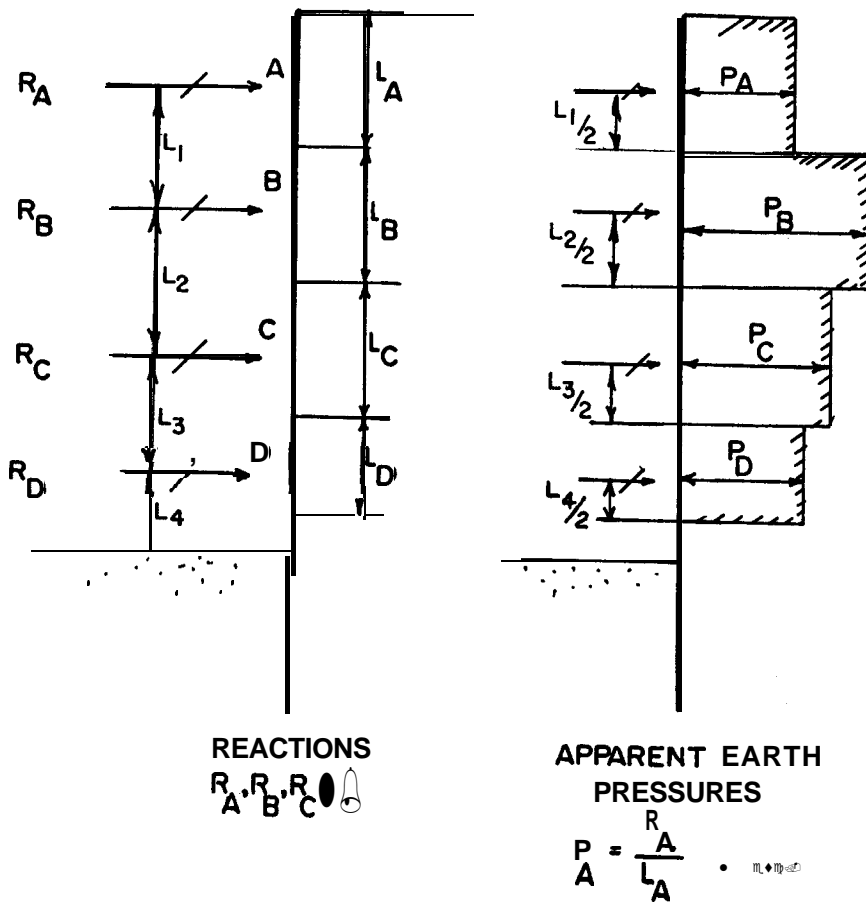
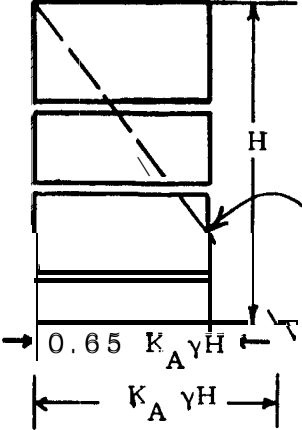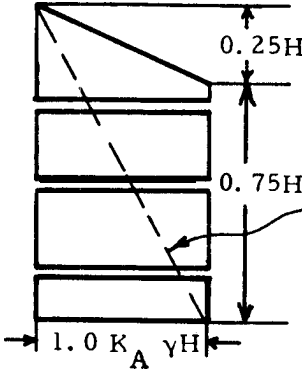Figure 19   Conventional procedure for development of earth pressure diagram,

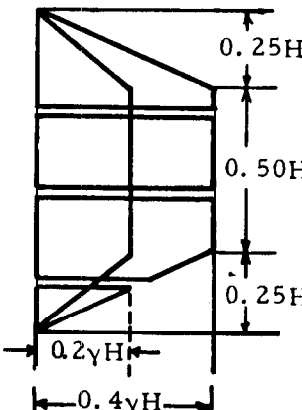| PRESSURE DISTRIBUTION	TOTALFORCE

a) Sands

$$K_A = \tan^2(45 - \phi/2)$$

Rankine Active

$0.65\ K_A \gamma H$

$K_A \gamma H$

P_t = Trapezoid

P_A = Rankine

$Pt = .65\ K_A \gamma H^2$

$P_A = .50\ K_A \gamma H^2$

$\dfrac{P_t}{P_A} = 1.30$

b) Soft to Medium Clays (N > 6)

0.25H

0.75H

For clays, base the selection on $N = \dfrac{\gamma H}{S_u}$

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

$1.0\ K_A \gamma H$

m = 1.0

$$P_t = .875\gamma H^2(1 - \frac{4}{N})$$

$$P_A = .50\gamma H^2(1 - \frac{4}{N})$$

$$\dfrac{P_t}{P_A} = 1.75$$

c) Stiff-Clays

0.25H

0.50H

0.25H

For N < 4

For 4 < N < 6, use the larger of diagrams (b) and (c).

$0.2\gamma H$

$0.4\gamma H$

$P_t = .15\gamma H^2$ to $.30\gamma H^2$

$\dfrac{P_A}{N} = 4,\ P_A = 0$

$N < 4,\ P_A < 0$

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 de-watered 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 $0.188\gamma H^2$

0.30 H

RANGE

0.50 H

H

0.20 H

$0.15\gamma H$

0.25h-l
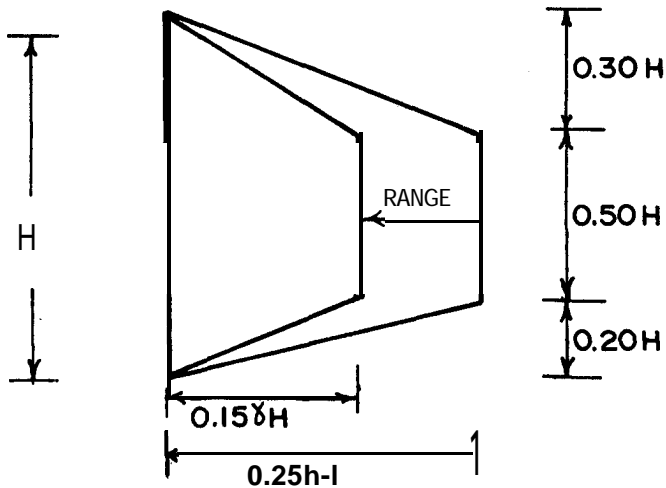
(b) UPPER THIRD OF CUT DOMINATED BY COHESIONLESS SOIL

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

H

0.80H

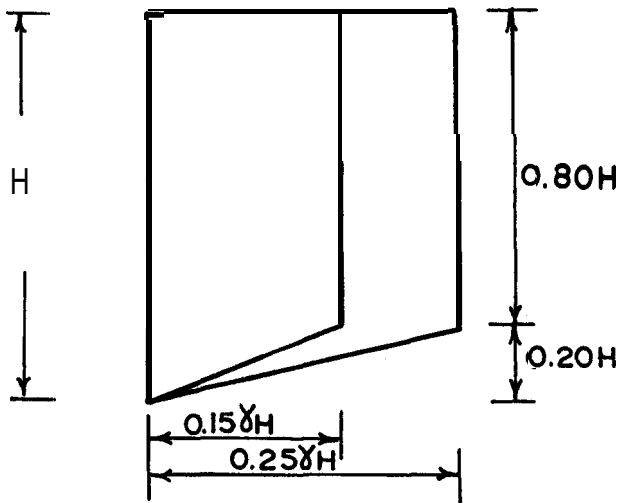0.20H

$0.15\gamma H$

$0.25\gamma H$

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. <u>Sands:</u> Where deformations <u>are critical,</u> and it is intended to prestress to <u>100 percent of design load,</u> 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 $K_{avg} = \dfrac{K_o + K_a}{2}$, that

is, a coefficient midway between active and at rest. A similar procedure was used by Hanna and Matallana (1970).

b. <u>Stiff to Very Stiff Clays:</u> 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.

$$\sigma_h^{'} = \sigma_h \cos^2(1.1\theta)$$
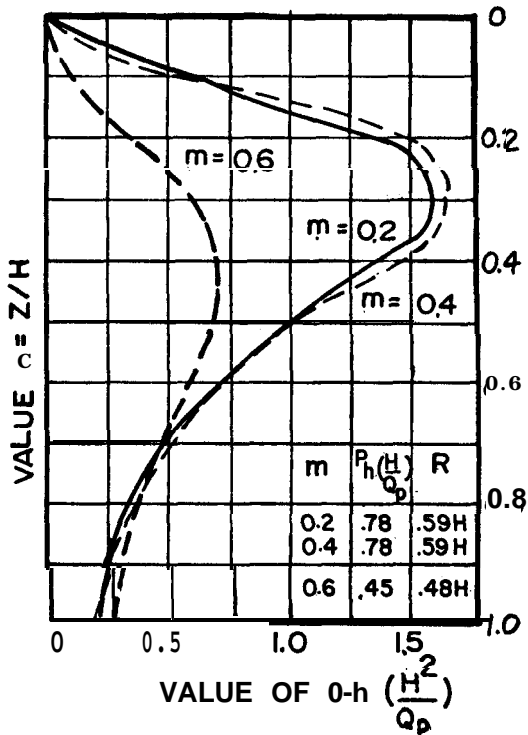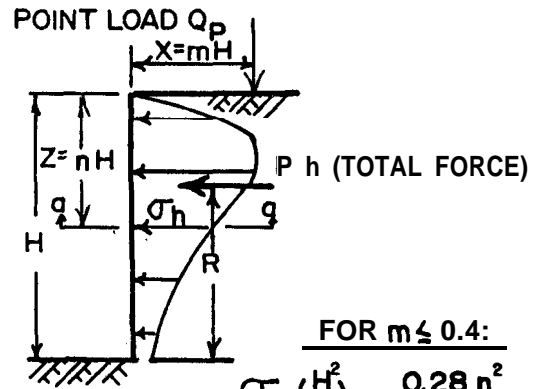
FOR m ≤ 0.4:

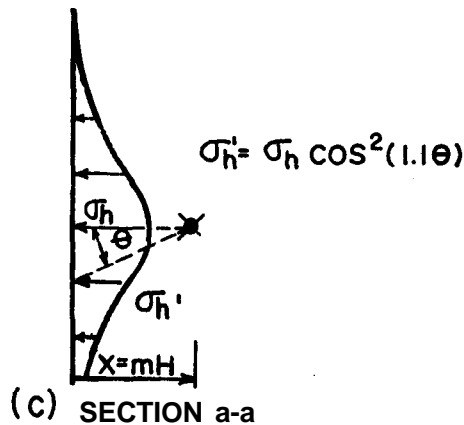$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{0.28\, n^2}{(0.16 + n^2)^3}$$

FOR m > 0.4:

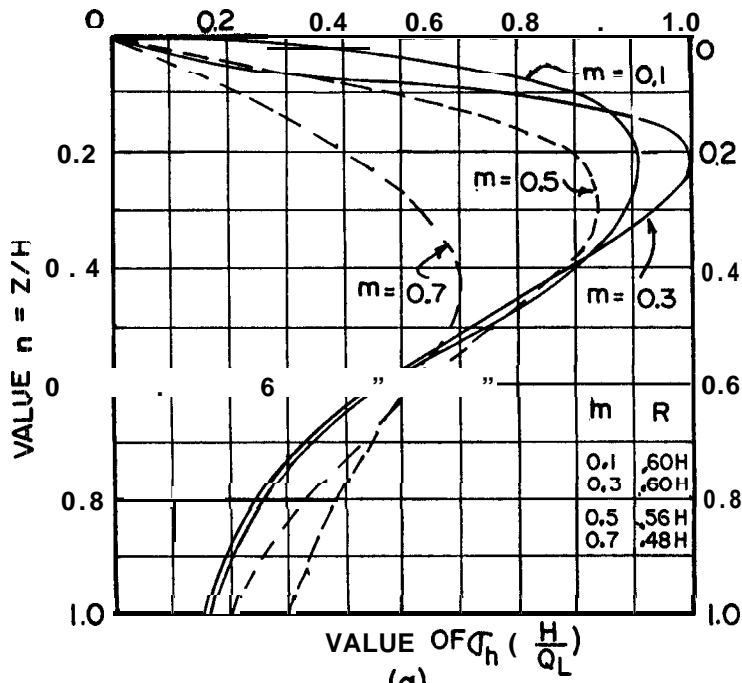$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{1.77\, m^2 n^2}{(m^2 + n^2)^3}$$

NOTE:

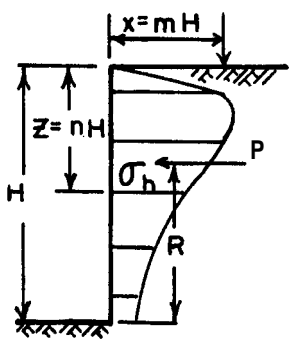$$m = \frac{X}{H}$$
$$n = \frac{Z}{H}$$

(b)

(a)

(c) SECTION a-a

m	$P_h\left(\frac{H}{Q_p}\right)$	R
0.2	.78	.59H
0.4	.78	.59H
0.6	.45	.48H

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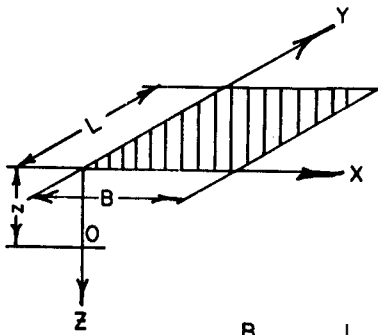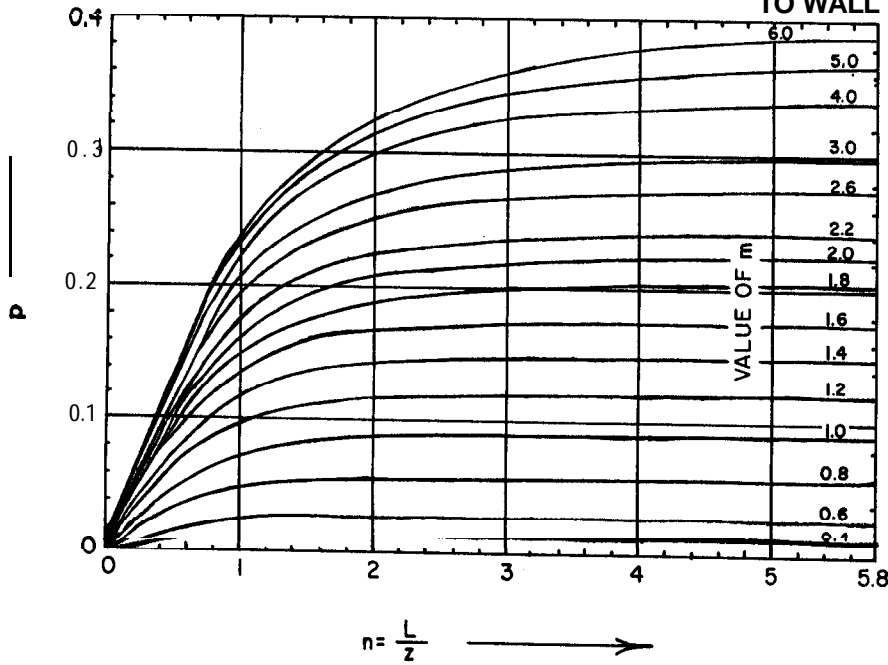
Figure 24   Lateral stresses on an unyielding wall due
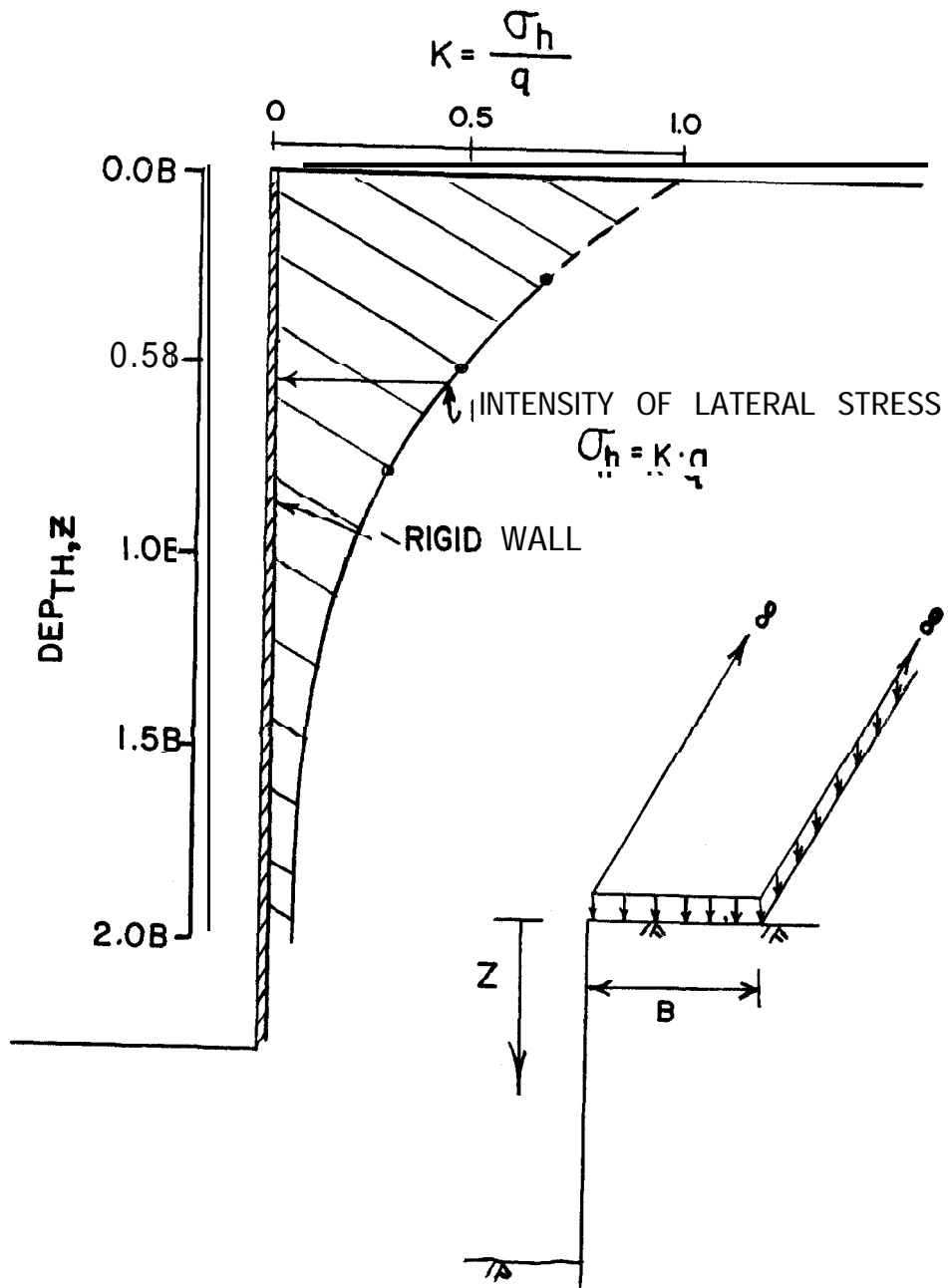to irregular surface loading (Sandhu, 1974).

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).

CHAPTER 4 - PASSIVE RESISTANCE BELOW BASE OF EXCAVATION

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 exce s sive 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,

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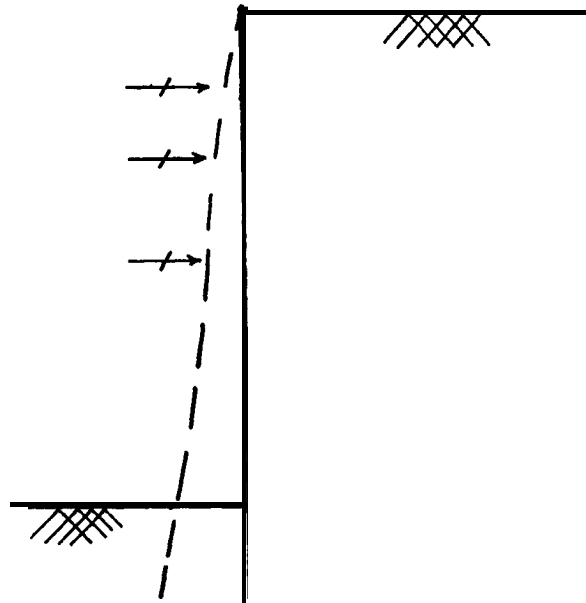
Figure 26   Movement at wall base due to insufficient passive resistance.

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 internal friction. The following table (from Terzaghi, 1954b) summarizes values 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:

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

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

$$\sigma_h = \bar{\sigma}_p + \mathbf{1}_w z$$

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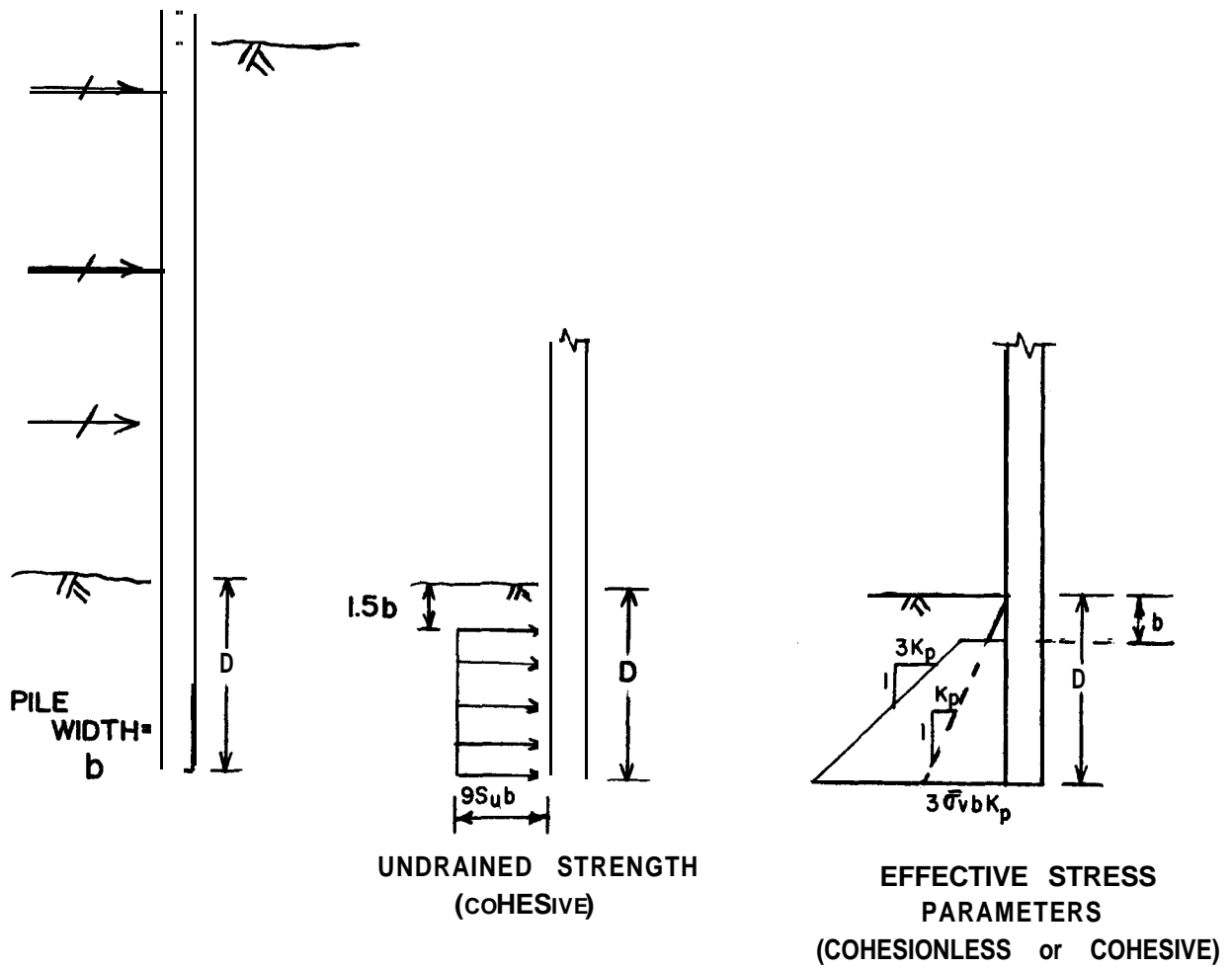
Figure 27   Passive pressure for soldier piles
(after Broms, 1965). (Modified.)

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 re-place the berm with an equivalent sloping plane and assign the appropriate passive coefficient (Terzaghi and Peck, 1968; NAVFAC, 1971).

CHAPTER 5 - DESIGN ASPECTS OF LATERAL PRESSURE

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 per-cent 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 :
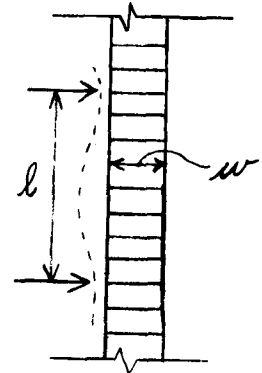
$$M = C w l^2$$

where:

M = moment

C = moment coefficient

w = distributed load on span

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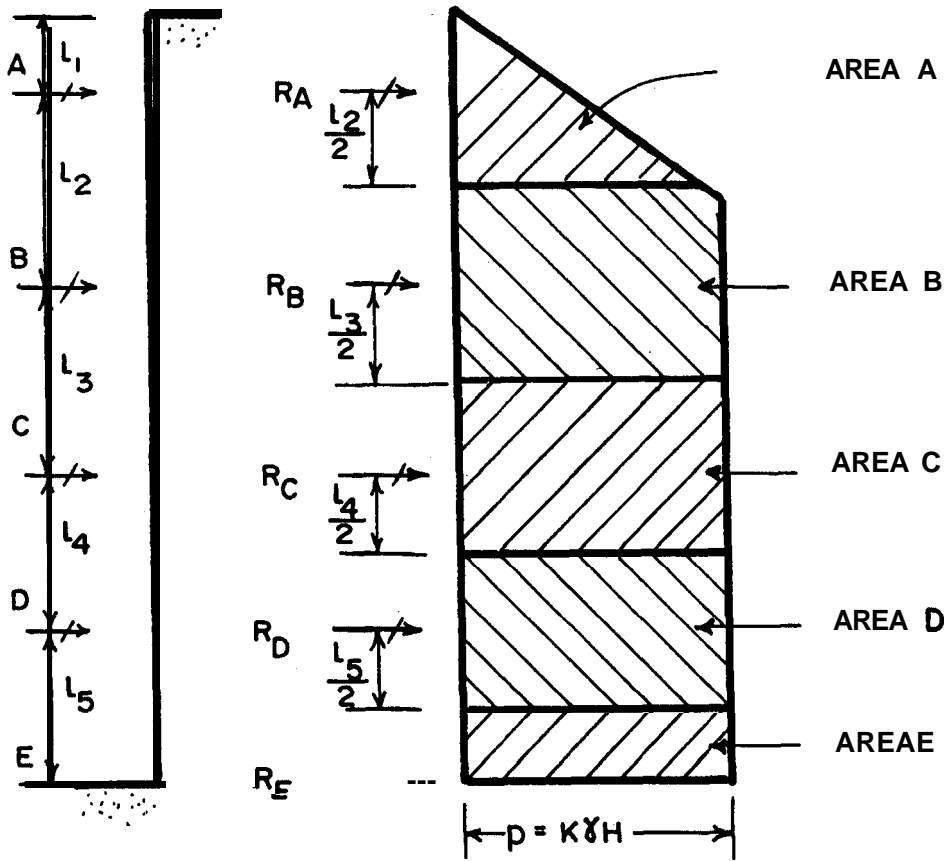
STRUT LOAD PER LINEAL FOOT OF WALL. IS EQUAL TO DESIGNATEDAREA

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.
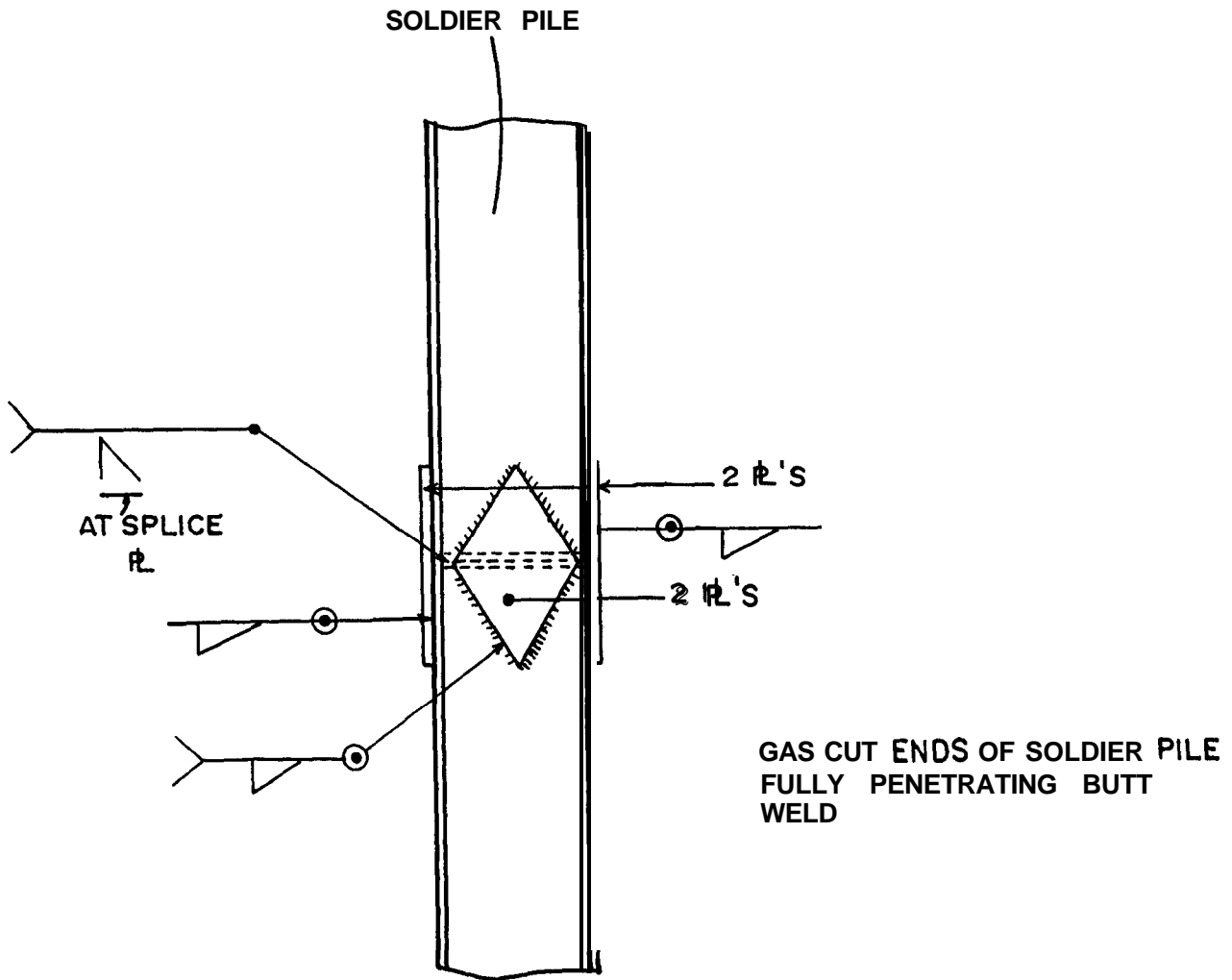
SOLDIER PILE

2 ℞'S

AT SPLICE
℞

2 ℞'S

GAS CUT ENDS OF SOLDIER PILE
FULLY PENETRATING BUTT
WELD

Figure 29.   Typical splice with butt welding,

1"CLEAR SPACE
IF NO BUTT
WELDING

PLATE

PLATE

w - w

h

SHEAR
PLATE

STIFFENER AT SOLDIER PILE

SPACER

PLATE IF STRUT
DEEPER THAN WALE

CONTACT
LAGGING

WALE

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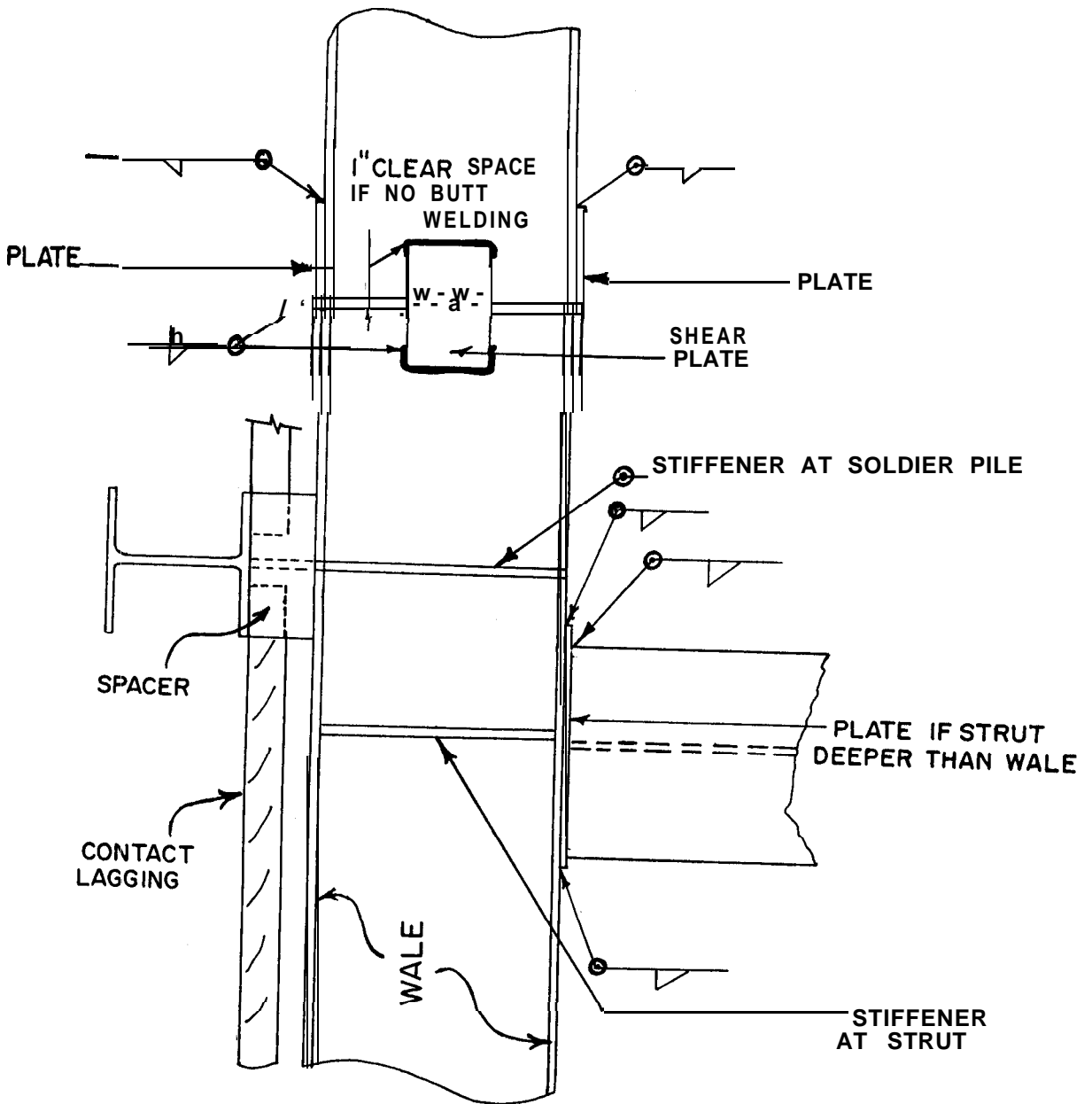
Figure 30. Plan view of typical wale splice and strut connection.

CONTACT LAGGING

ALL STIFFENERS

PLATE

SPACER
USED WITH
CONTACT LAGGING

STRUT
(PREFERABLY AT
SOLDIER PILE)

STIFFENER

ERECTION SUPPORT
AS REQUIRED

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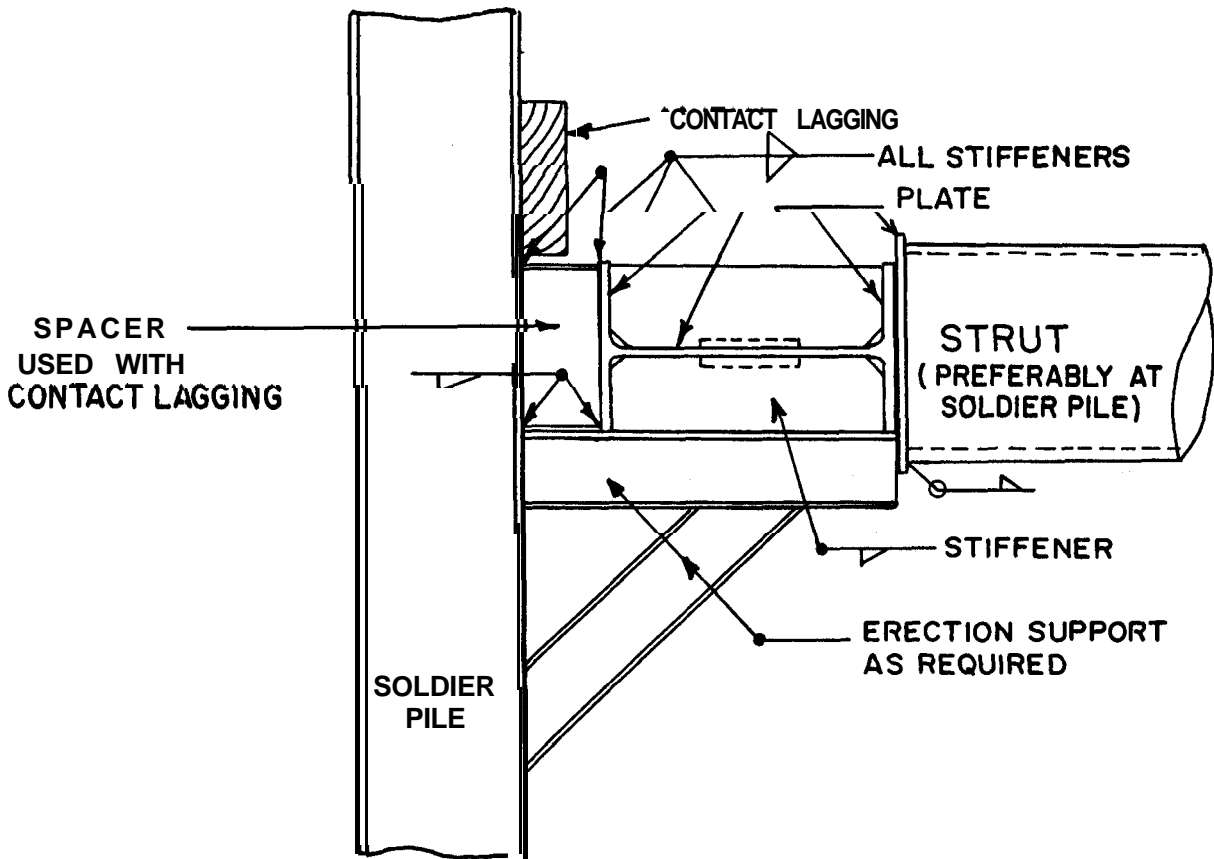
Figure 31. Typical strut-wale-soldier pile connections (elevation view).

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).
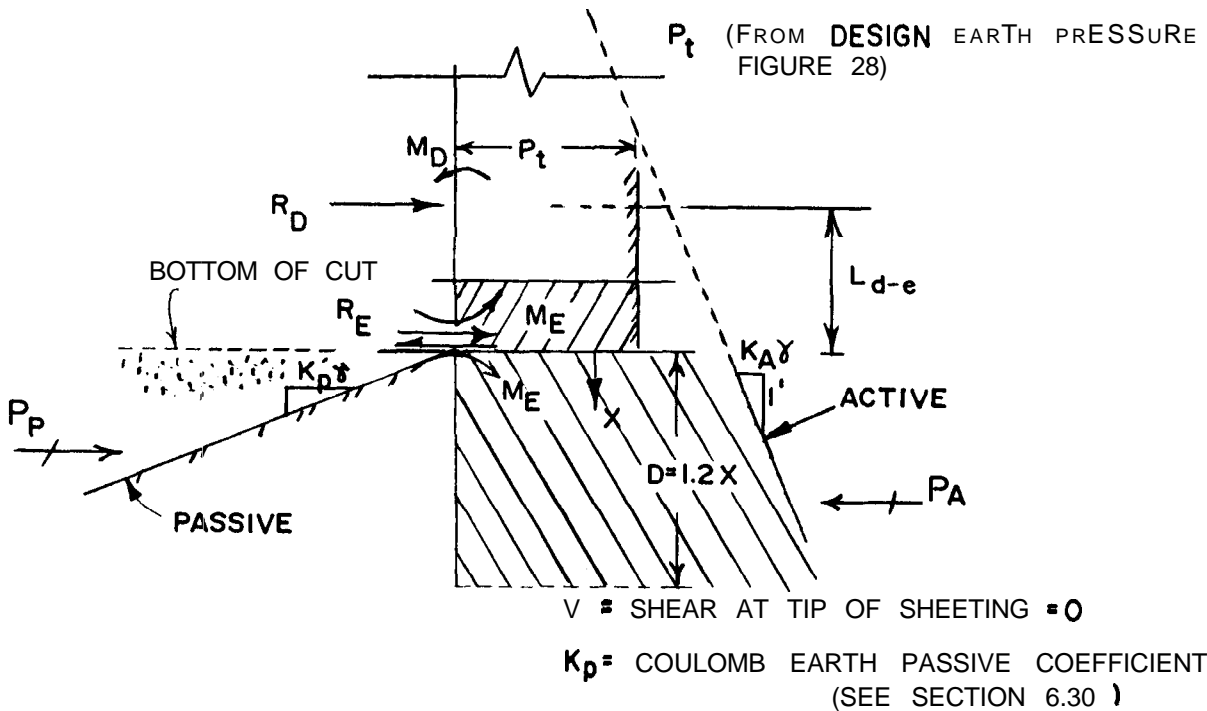
5.30   DEPTH OF PENETRATION BELOW CUT

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 re-quired 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).

P_t (FROM DESIGN EARTH PRESSURE FIGURE 28)

M_D

P_t

R_D

BOTTOM OF CUT

R_E

M_E

L_{d-e}

$K_p'\gamma$

M_E

$K_A\gamma$

P_P

ACTIVE

PASSIVE

$D = 1.2 X$

P_A

V = SHEAR AT TIP OF SHEETING = 0

K_p = COULOMB EARTH PASSIVE COEFFICIENT
(SEE SECTION 6.30)

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, $K_p' = \dfrac{K_p}{1.5}$
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.
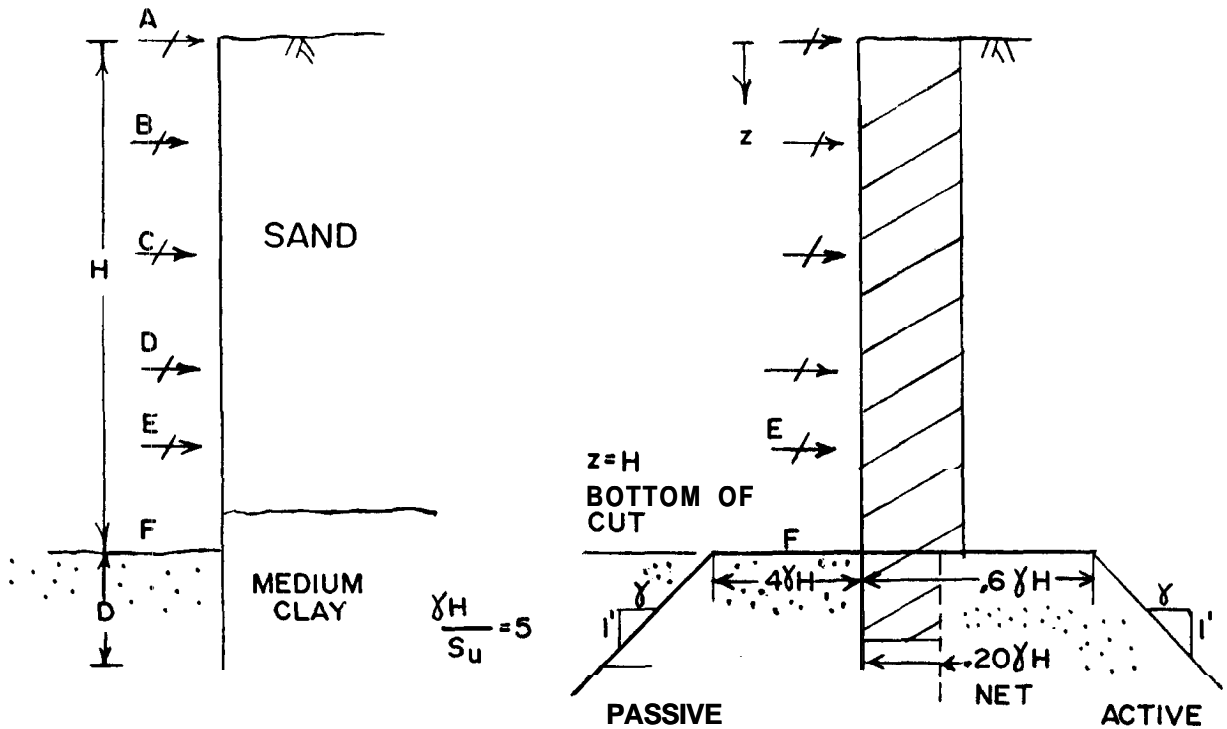
$$S u = \frac{\gamma H}{5} = 0.20 \gamma H$$

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.
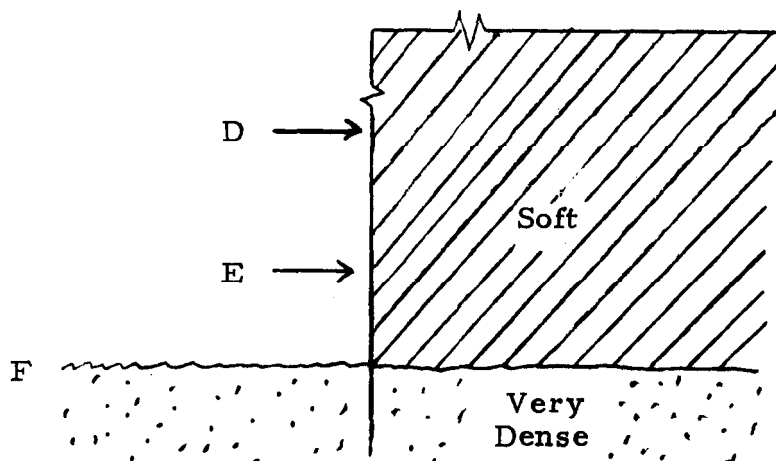
As the excavation proceeds below level D to level E, little passive resistance is provided because of the soft soil above F; hence, the wall deflects inward. Effectively, the wall spans from level D to F (the excavation base) with full active pressure applied and negligible passive resistance above F. 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 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 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.
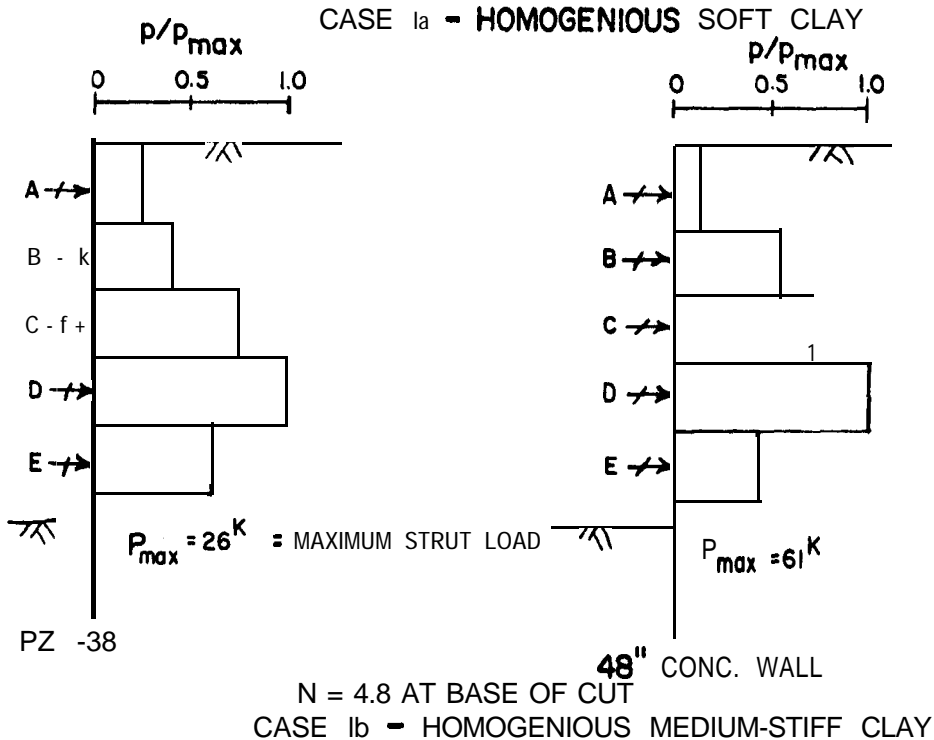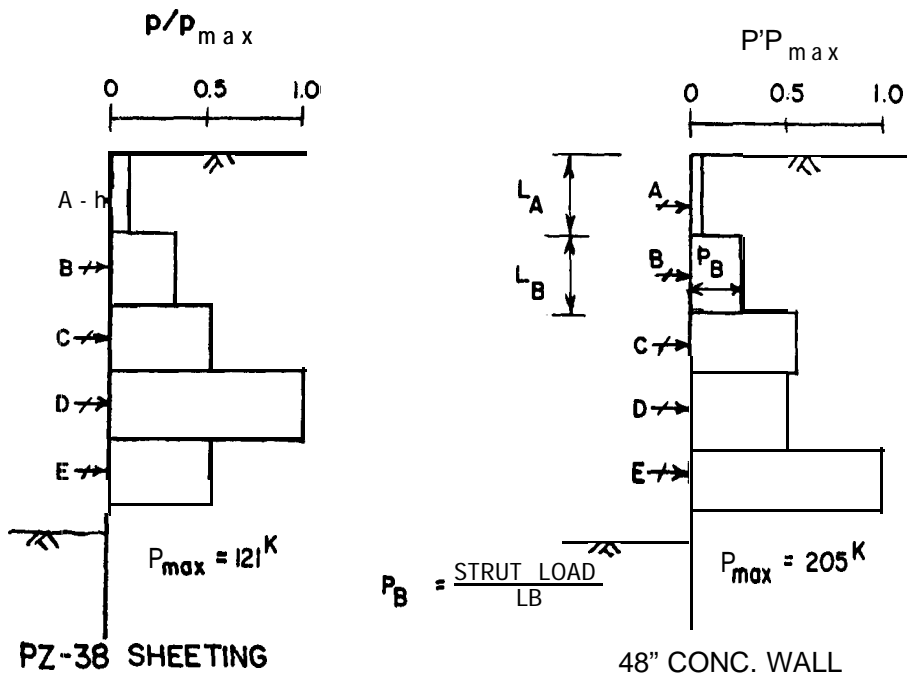
Figure 34. Normalized apparent earth pressure diagrams predicted by finite element analysis.
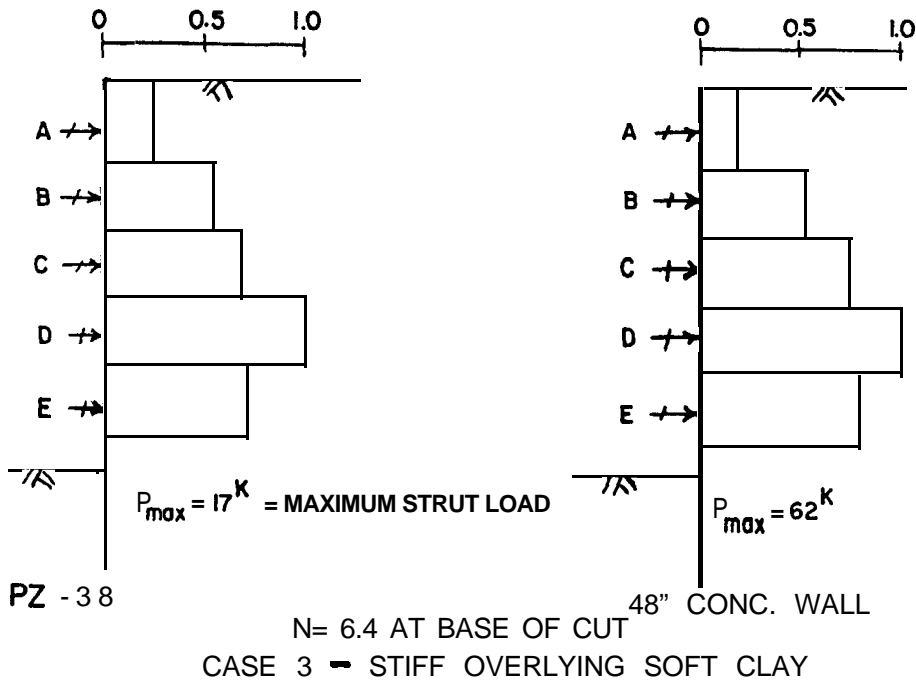
p/p_{mox}

p/p_{max}

A

B

C

D

E

$P_{max} = 50^K$

PZ-38   SHEETING

L_A $\dfrac{L_{AB}}{2}$

L_B $\dfrac{L_{BC}}{2}$

A

B

P_B

C

D

E

$P_B = \dfrac{\text{STRUT LOAD}}{L_B}$

$P_{max} = 73^K$

48" CONC. WALL

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

A

B

C

D

E

$P_{max} = 17^K$ = MAXIMUM STRUT LOAD

PZ -38

A

B

C

D

E

$P_{max} = 62^K$

48" CONC. WALL

N= 6.4 AT BASE OF CUT
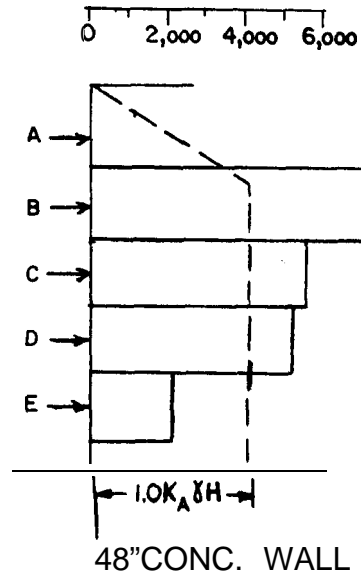CASE 3 — STIFF OVERLYING SOFT CLAY
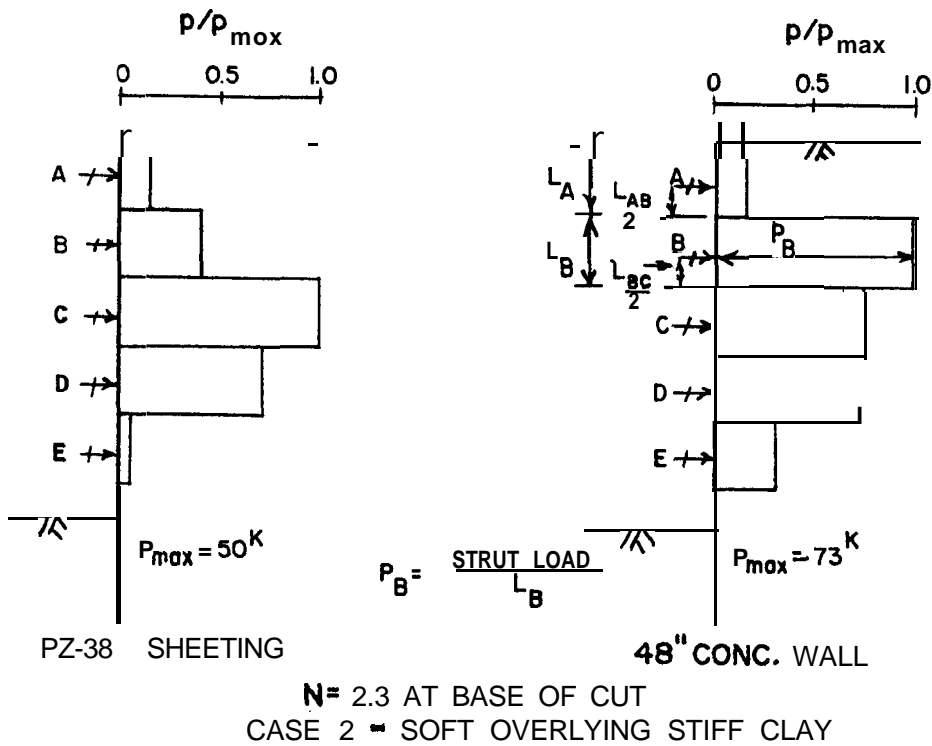
Figure 35.   Normalized apparent earth pressure diagrams predicted by finite element analysis.
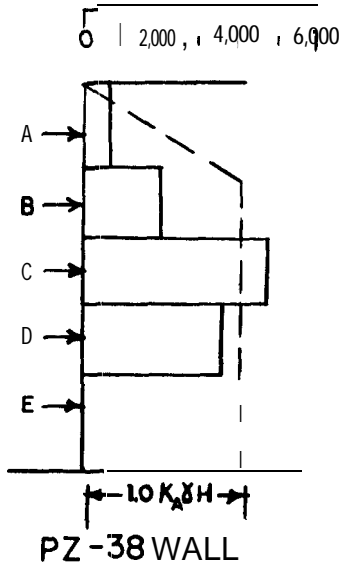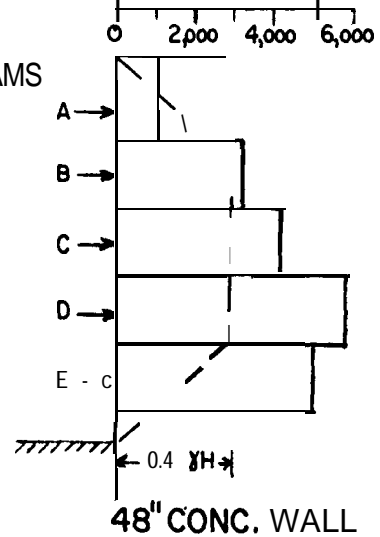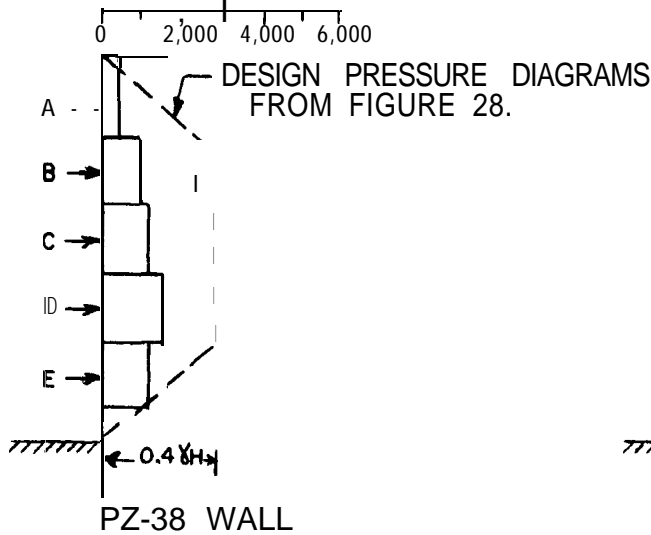
APPARENT LATERAL PRESSURE (psf)

PZ-38 WALL

48"CONC. WALL

AT BOTTOM OF CUT: N = 2.3

CASE.2 (SOFT'OVERLYING STIFF CLAY)

DESIGN PRESSURE DIAGRAMS
FROM FIGURE 28.

PZ-38 WALL

48"CONC. WALL

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:

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

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
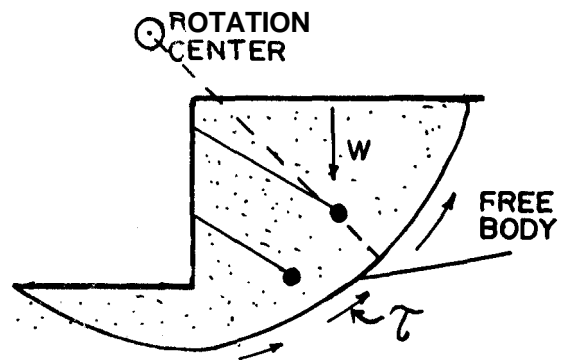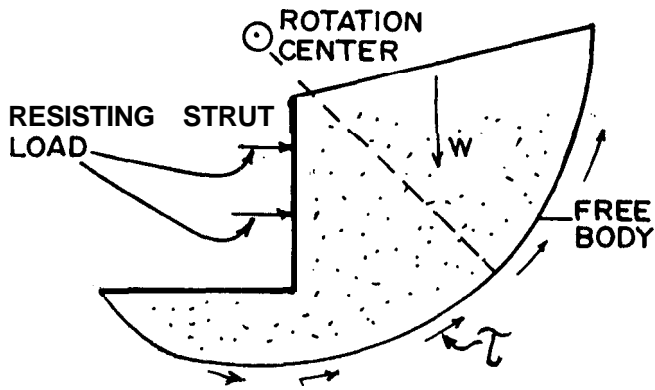
a) BOTTOM HEAVE FAILURE-

b) DEEP SEATED FAILURE
   INTERNALLY BRACED
   EXCAVATION

c) DEEP SEATED FAILURE
   TIED BACK
   EXCAVATION
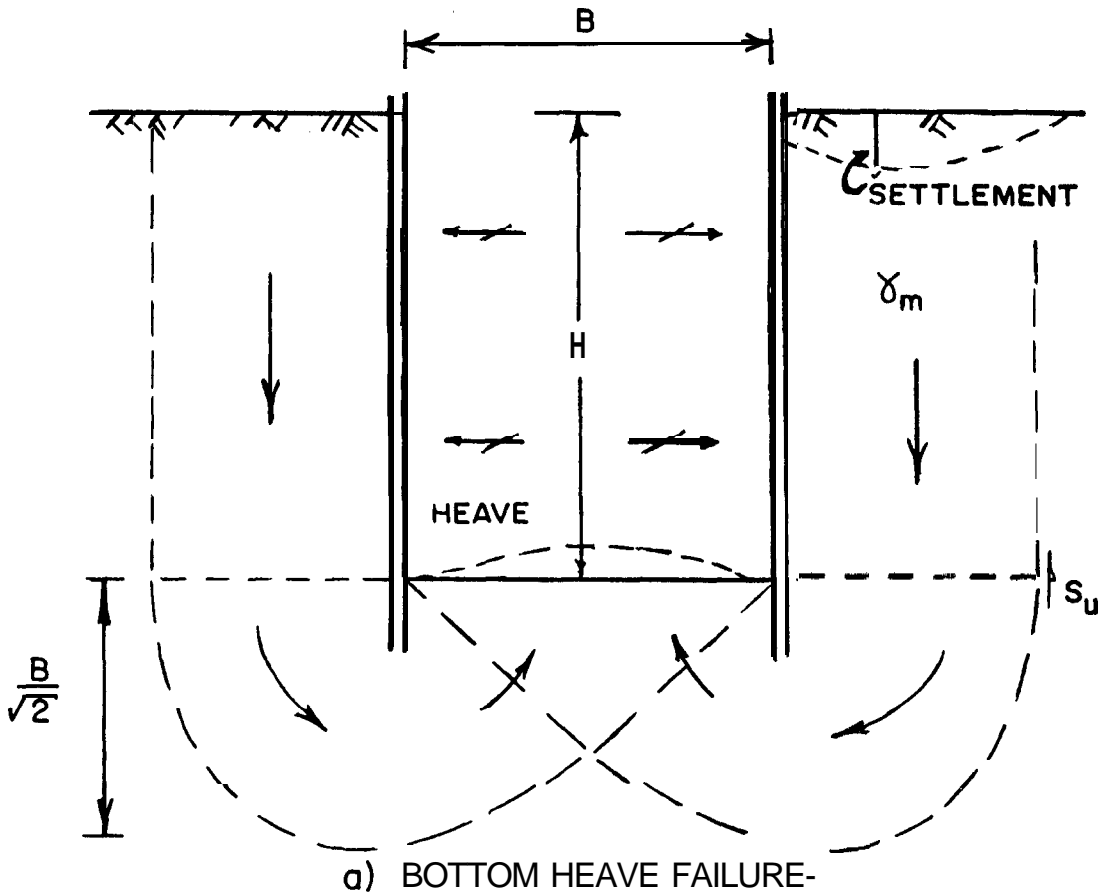
Figure 37. Potential failure surfaces.

$$H_{max.} = \frac{(S_u\ N_{cb} - q)}{\gamma_m}$$

SQUARE(B/L = 1)

B/L = 0.5

STRIP (B/L = 0)

L = LENGTH OF TRENCH

(FROM BJERRUM & EIDE, 1956)
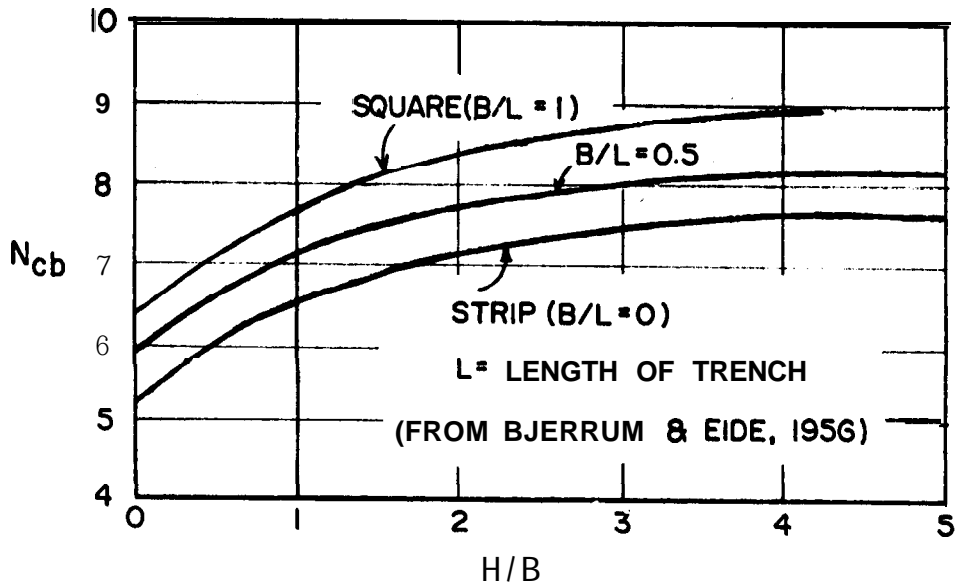
N_{cb}

H/B
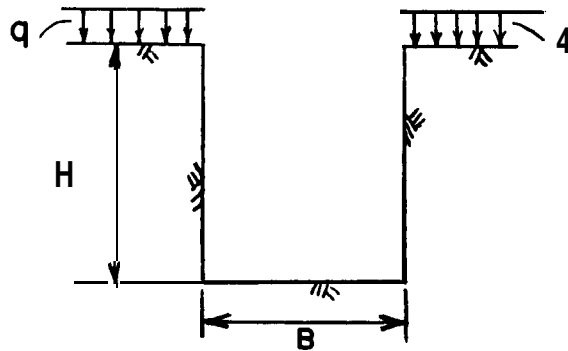
Figure 38.   Bearing capacity factors for bottom stability analy si s.

S_u. This average should be taken over a zone described between $\dfrac{B}{\sqrt{2}}$ 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
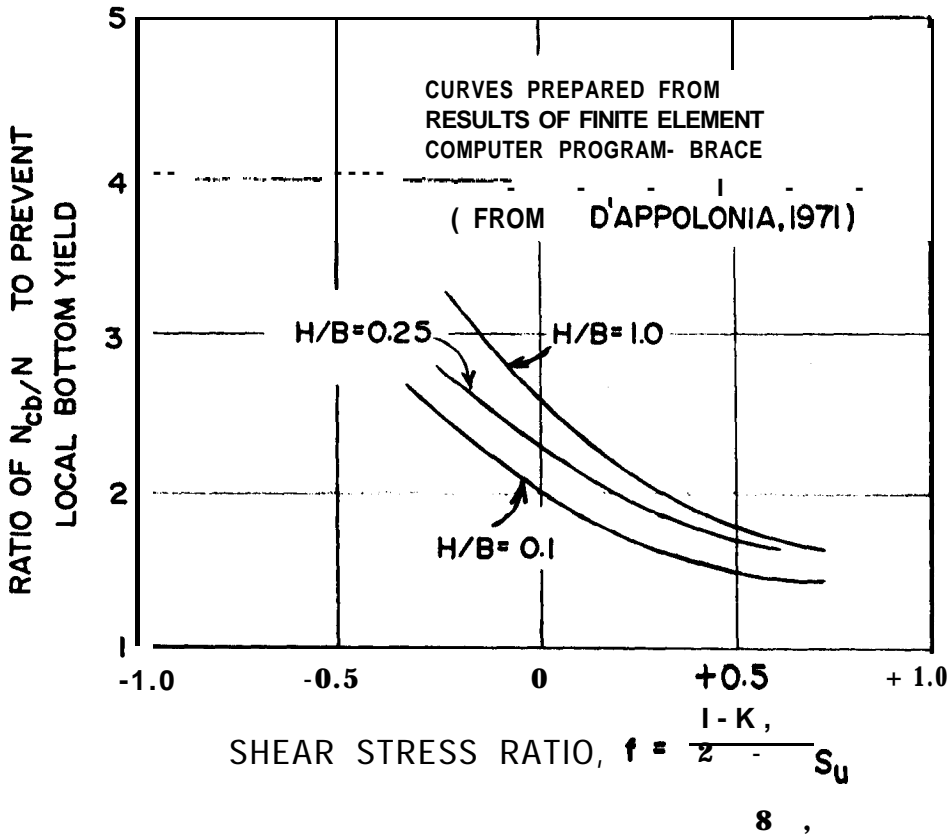
Figure 39. Factor of safety required to prevent local yield below bottom of excavation in clay.
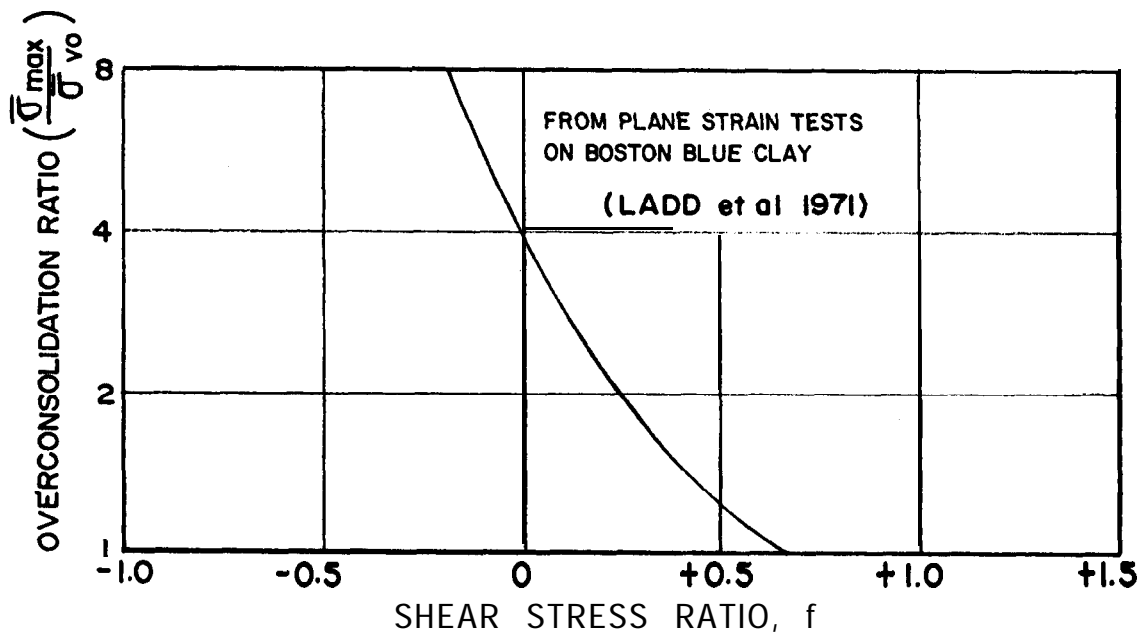
Figure 40. Shear stress ratio vs. overconsolidation ratio.

SYMBOL	L (FT)	EI x 10 (ksf)	K
⊙	15	4	0.79
△	9	4	6.25
⊡	9	40	62.5

(JAWORSKI, 1973)

FACTOR OF SAFETY: N_{cb}/N

H / B
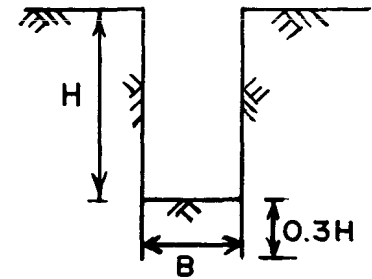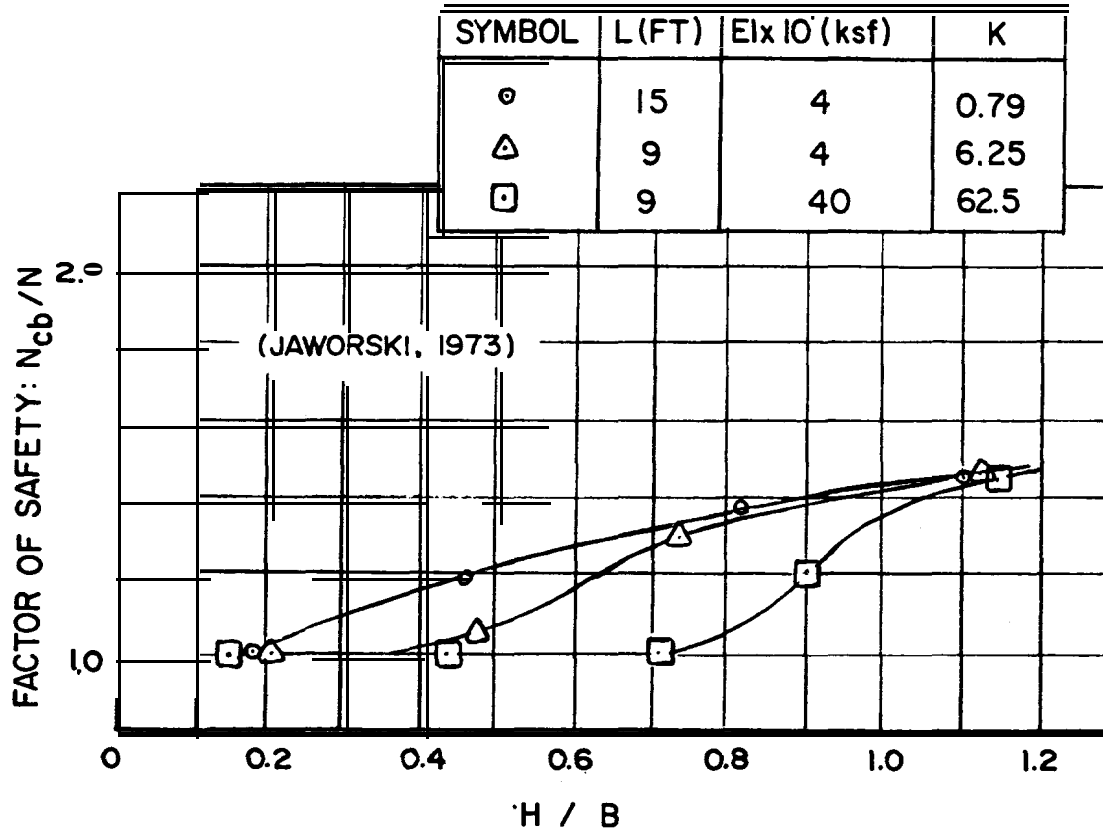
$K = \dfrac{EI}{L^4}$

L = STRUT SPACING

N_{cb} = BEARING CAPACITY NO.

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

$f = 0.75 = \left(\dfrac{1 - K_0}{2 \dfrac{S_u}{\overline{\sigma}_{vo}}} \right)$

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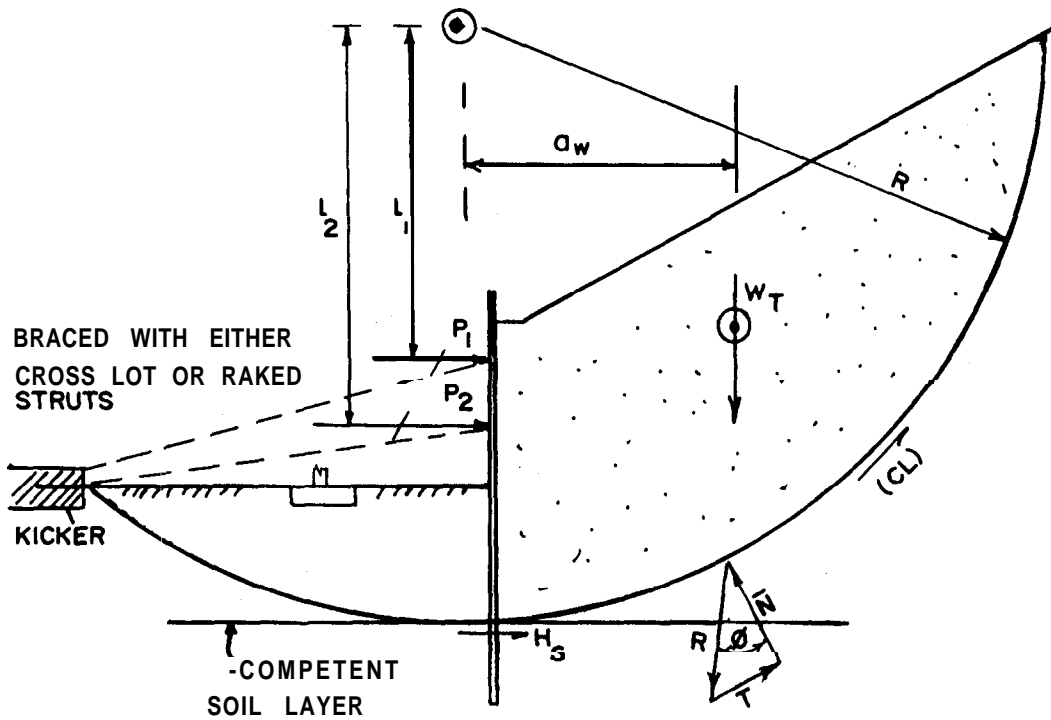
Consider overall stability:

Moments around center of rotation

Forces to consider:

1) Weight of driving mass (WT)
2) Resisting strut loads (P,, 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.

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

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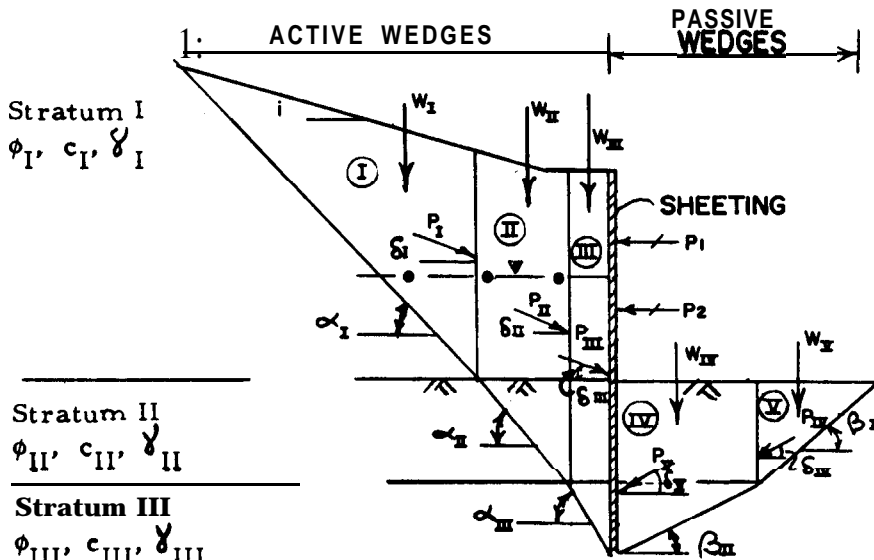
For general solution vary α, β, and&angles to obtain minimum value for factor of safety.

Method of Analysis:

1. Assume α, β, ϕ 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. $= \dfrac{P_1 + P_2}{P_{III} \cdot P_v}$
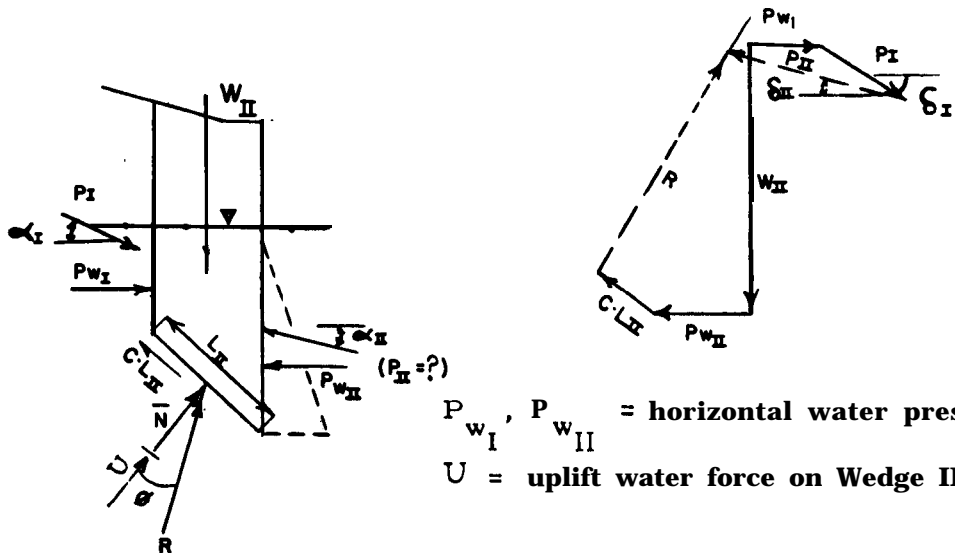
P_{w_I}, $P_{w_{II}}$ = horizontal water pressure

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. Accordingly, 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 Under- writers (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 - 121
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 - $0.1 \times N$, but not less than 6 ksf nor more than 12 ksf (where N= no. of blows in SPT)

3 - $0.1 \times N$, 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
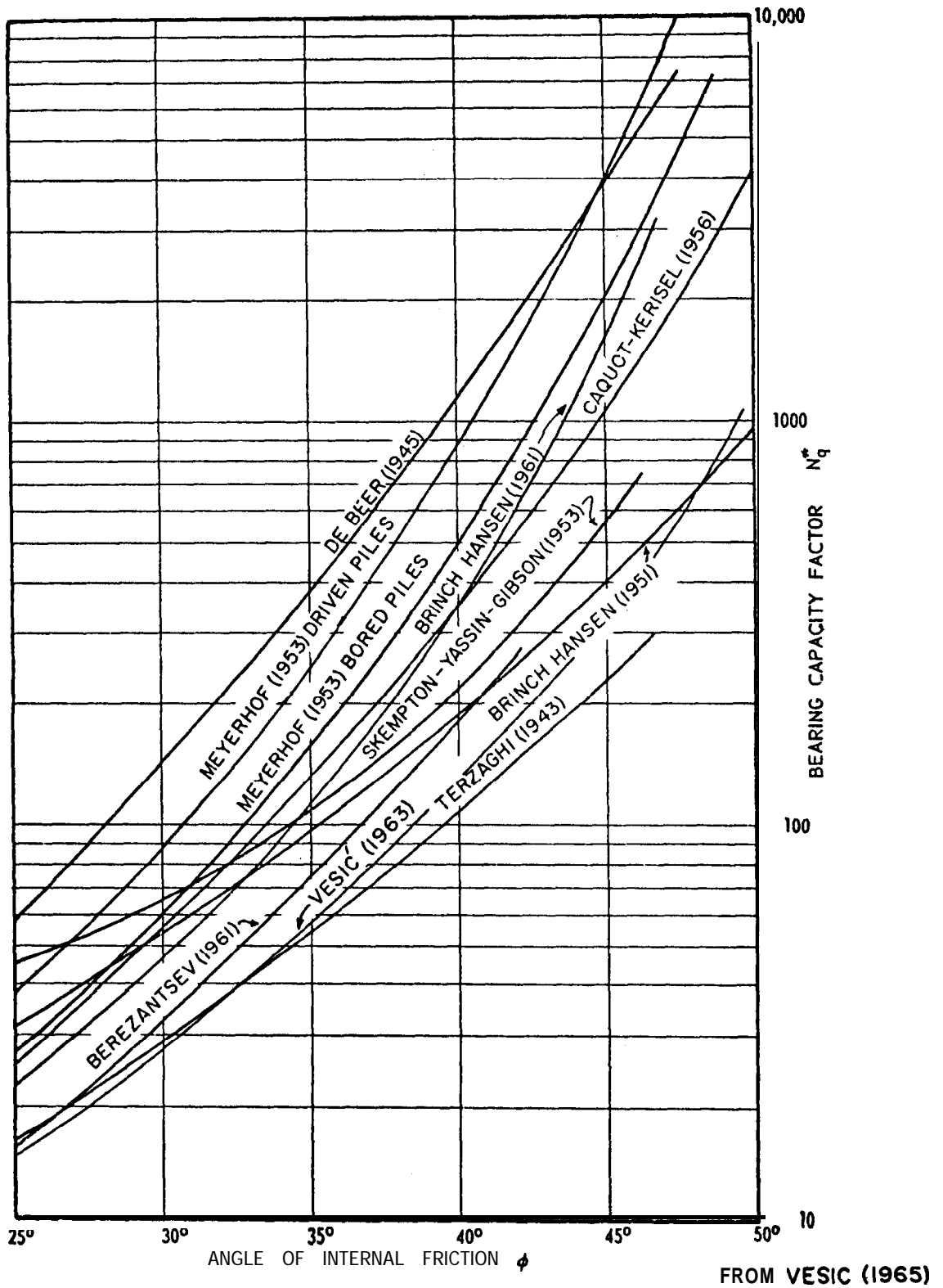   L = length

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

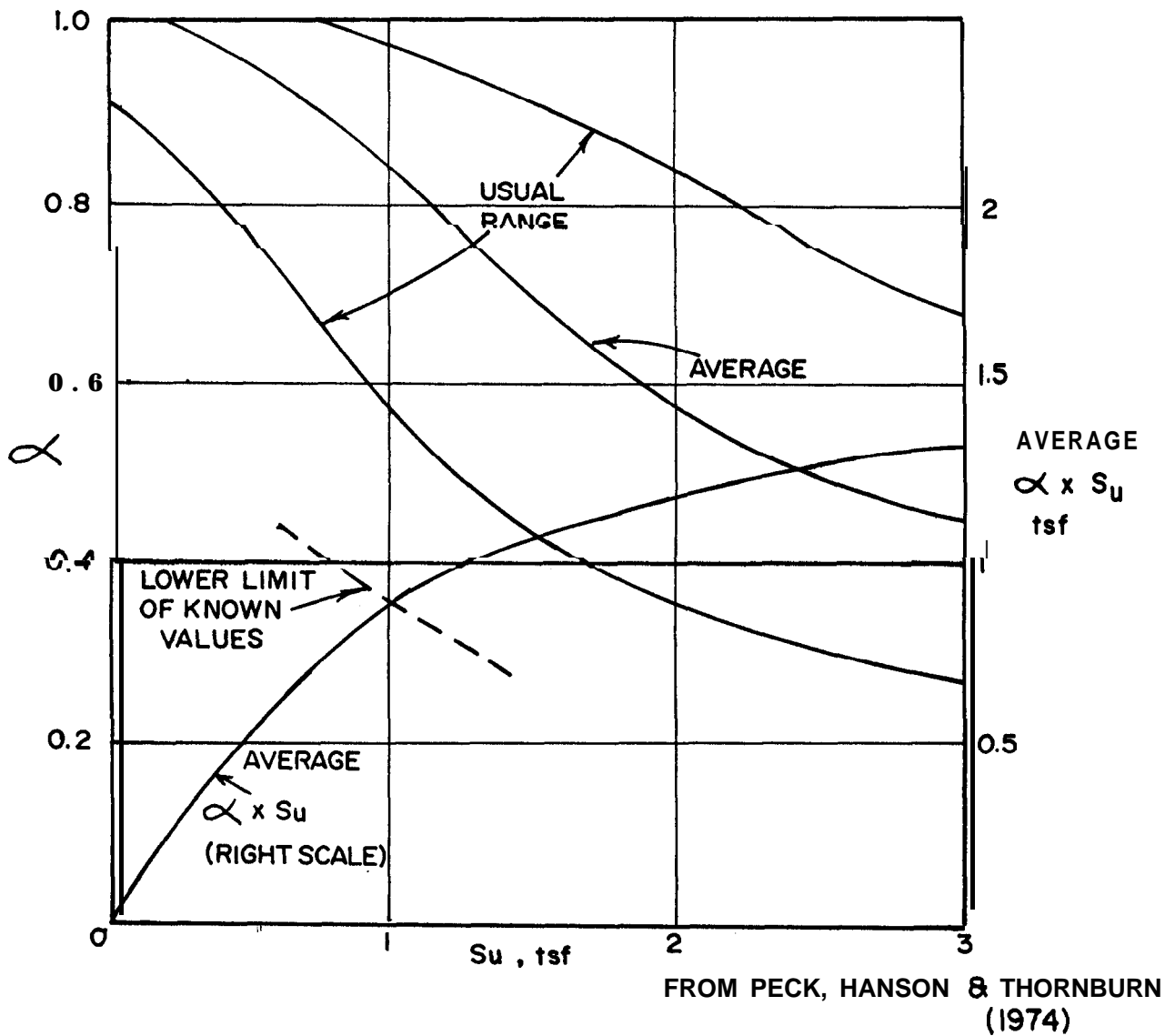$$P = q \, \frac{BI_\rho}{E} \, (1 - \nu^2) \qquad \text{Eq. 7.41.1}$$

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

FROM PECK, HANSON & THORNBURN (1974)

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where:

P = 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 (kl) 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 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:

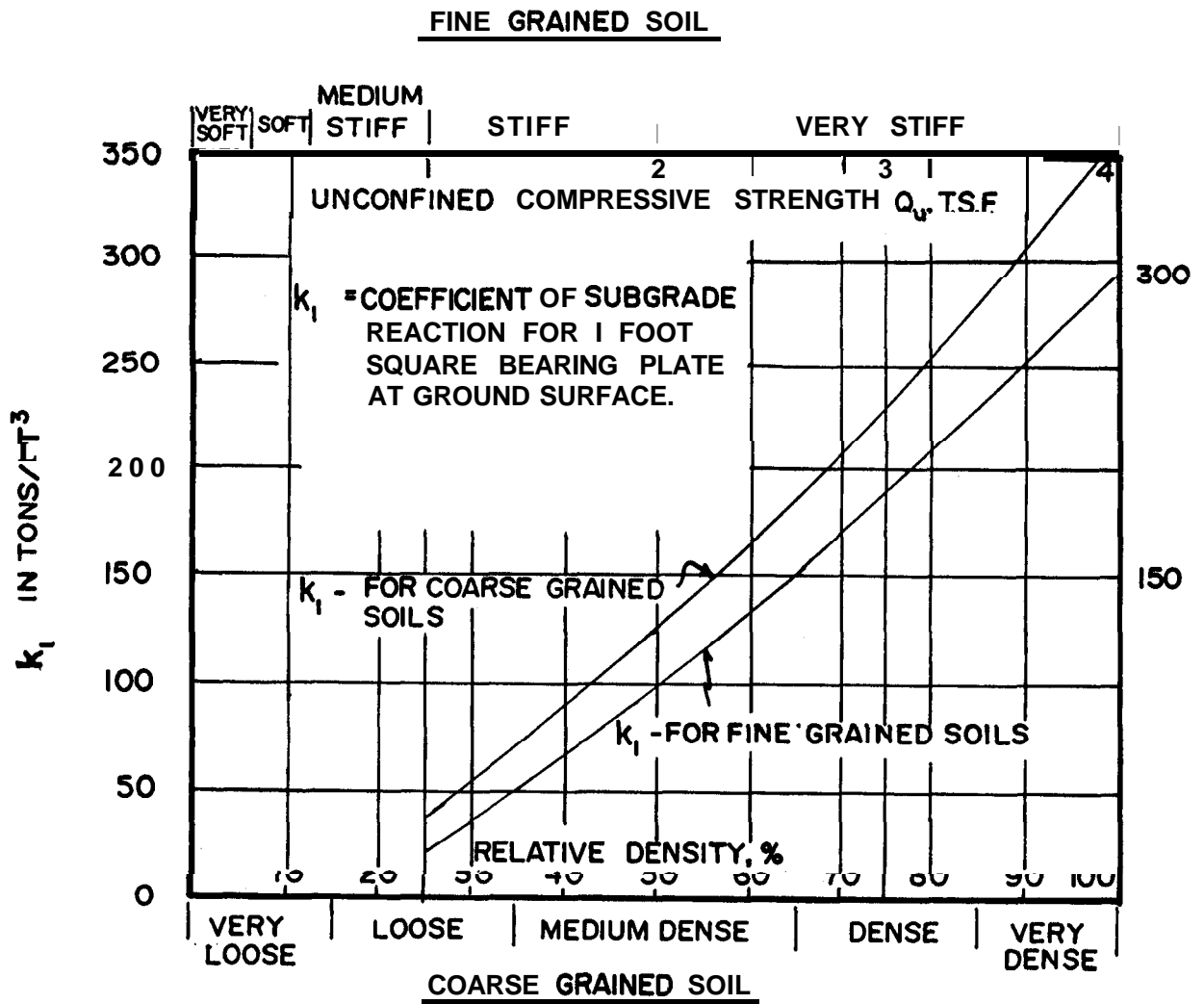k_B^S = coefficient of subgrade reaction for a footing (breadth B) at the surface

FINE GRAINED SOIL

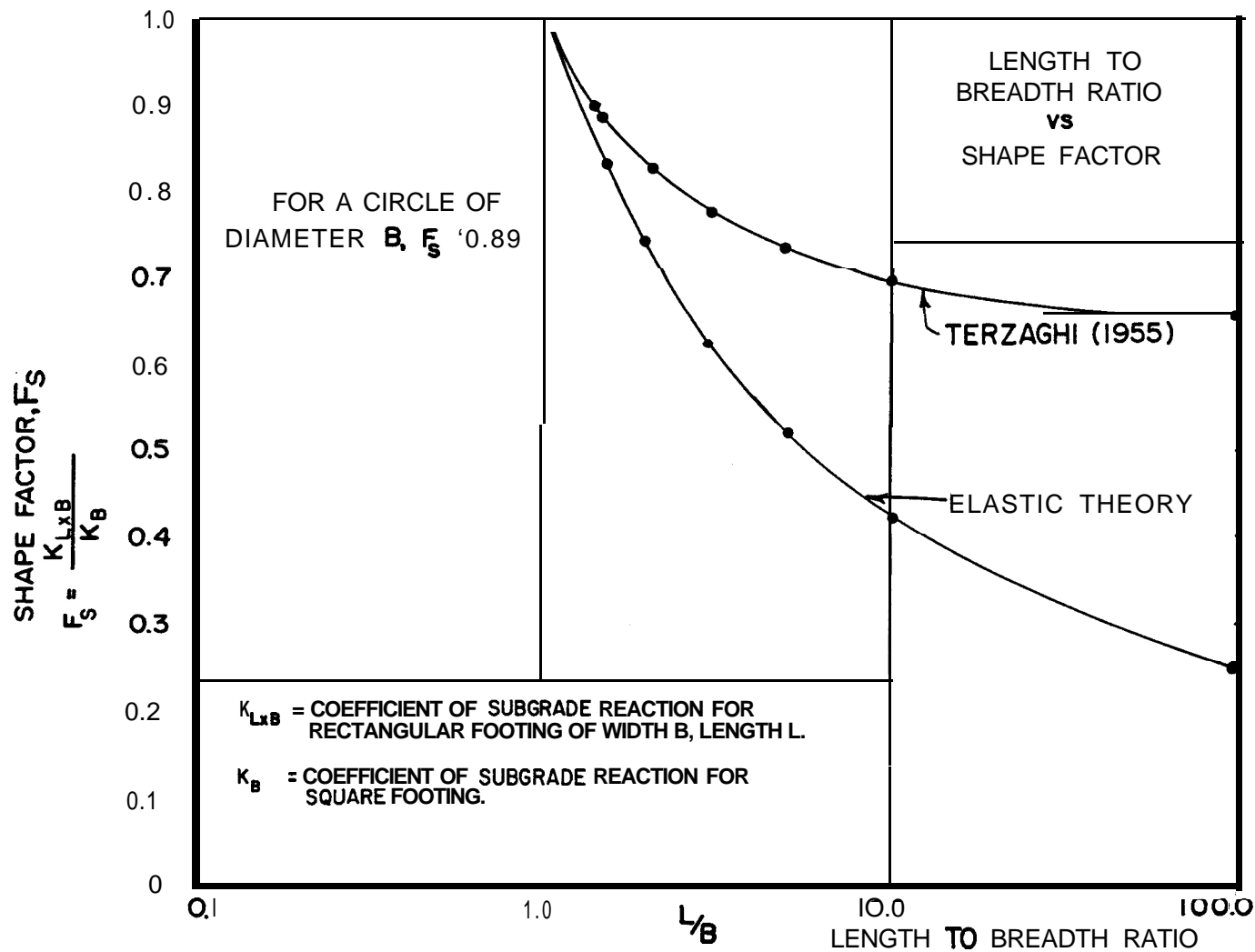Figure 46. Coefficient of subgrade reaction vs. in situ soil indices (NAVFAC, 1971).

Figure 47. Shape factor for rectangular footings.

$$k_B^D \quad = \quad \text{coefficient of subgrade reaction for a footing (breadth B) at depth D}$$

$$I_\rho^D \quad \simeq \quad \text{influence factor for footing at depth D}$$

$$I_\rho^S \quad = \quad \text{influence factor for footing at surface}$$

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.

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

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 :

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

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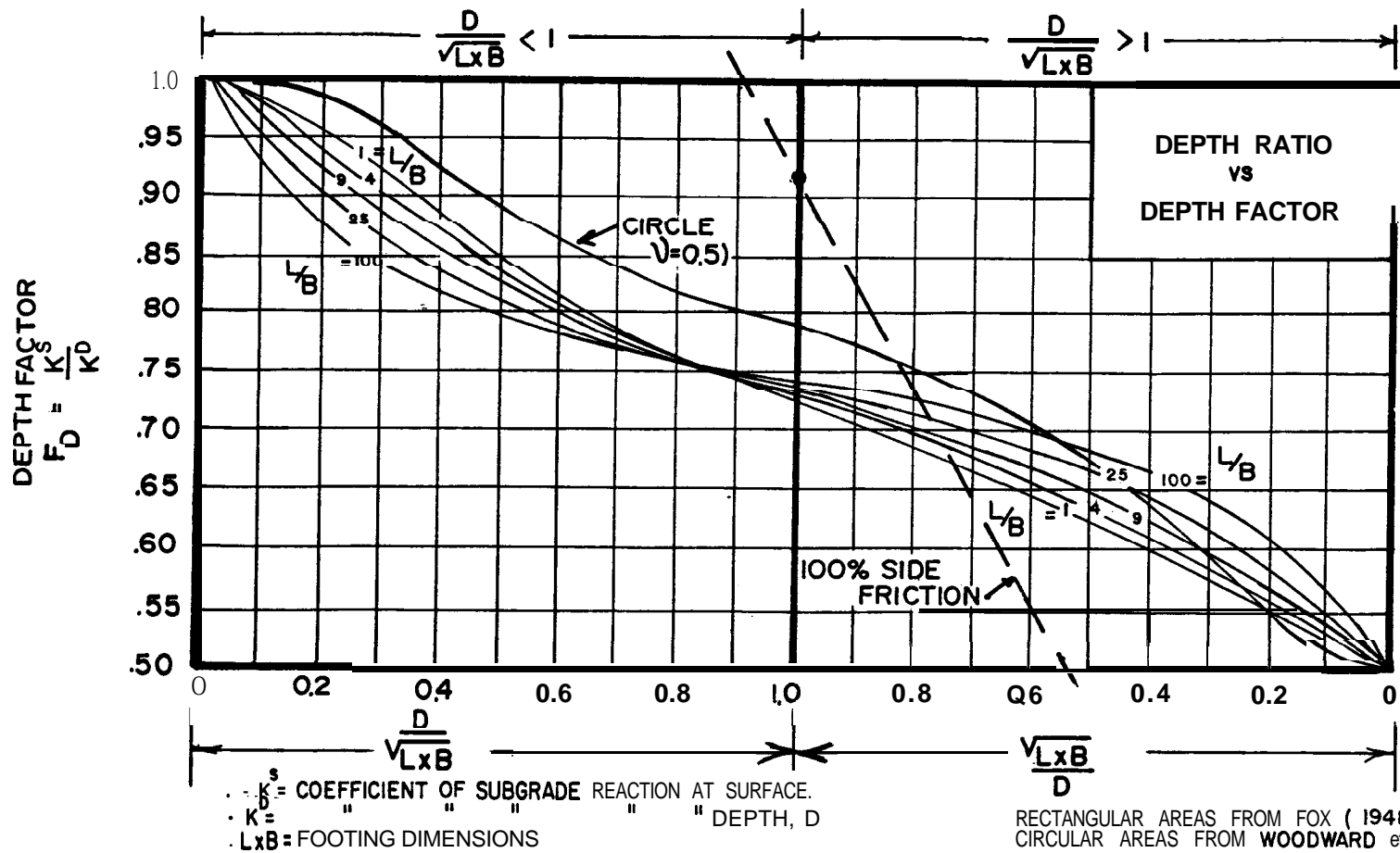
Figure 48. Influence of depth on coefficient of **subgrade** reaction (based on modulus of deformation that is constant with depth).

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}:

$$\frac{k_B^S}{k_B^D} = \left[\frac{\bar{\sigma}_3^S}{\bar{\sigma}_3^D}\right]^n = F_{DG} \qquad \text{Eq. 7.42.3}$$

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} = 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
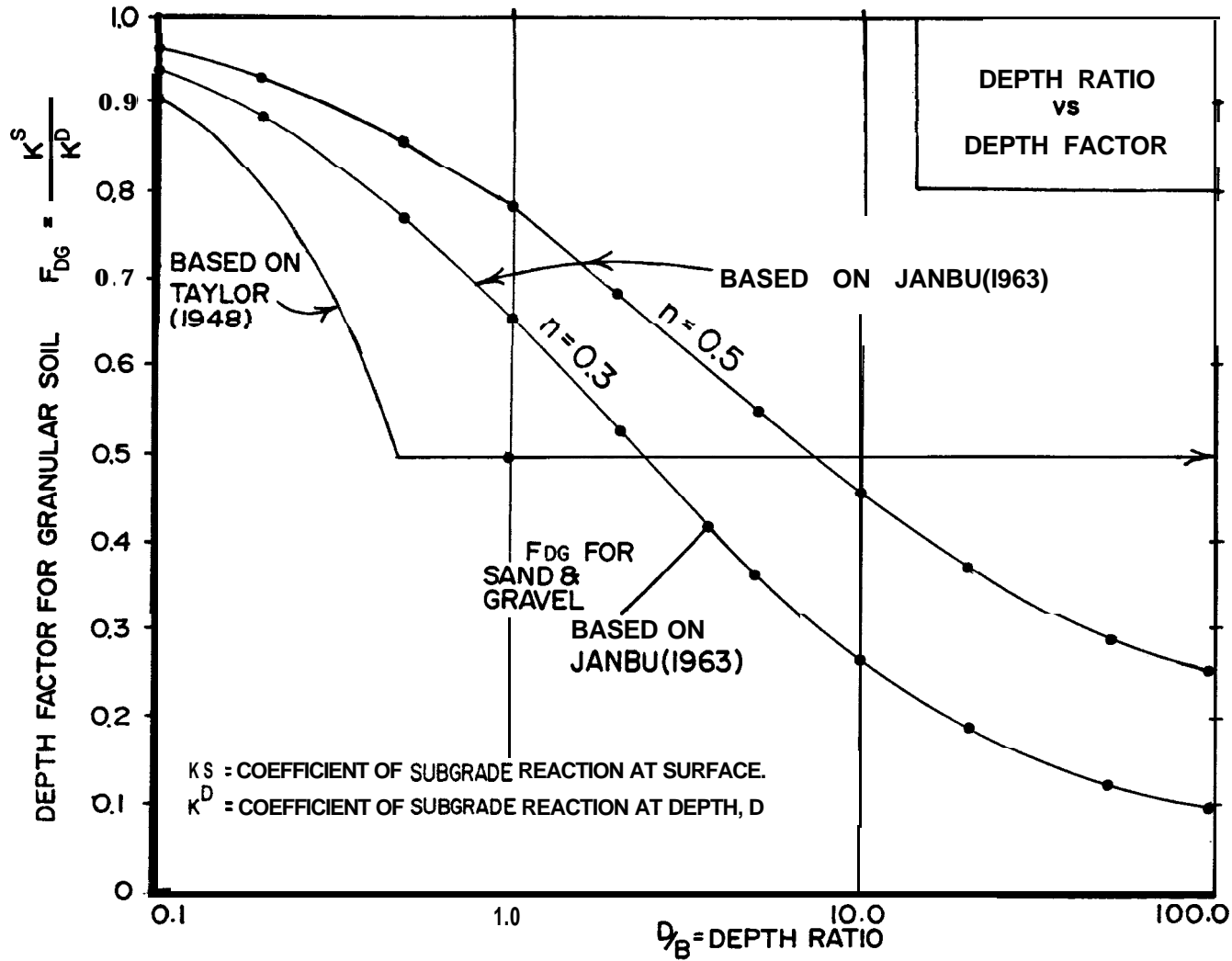
$$\rho = q/k$$

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

CHAPTER 8 - OVERVIEW OF CONSTRUCTION METHODS

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 in-sufficient 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 displace-ments 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 $\frac{EI}{L4}$) **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 diaphragm wall construction. 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 condi-tions 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 **pre-**stressing 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 proce-**dur es, and soil conditions. Water flow alone may Lead to a drop **in pie-zometric** 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 **character-istics,** 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 the 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   C O S T S

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 excava- tion 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:

	Cost per Square Foot[1]	
Depth (feet)	Soldier Piles and[2] Wood Lagging	Inter locked [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 pene- tration 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.
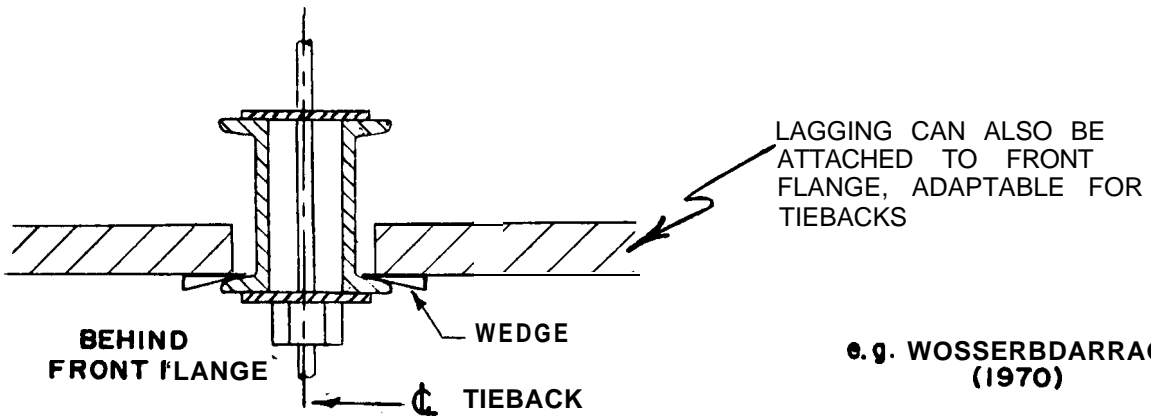
Spacers between the lagging boards (called "louvres") allow the introduction of material for backpacking boards and filter-

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*Contact Sheeting, Inc. , Nyack, New York.
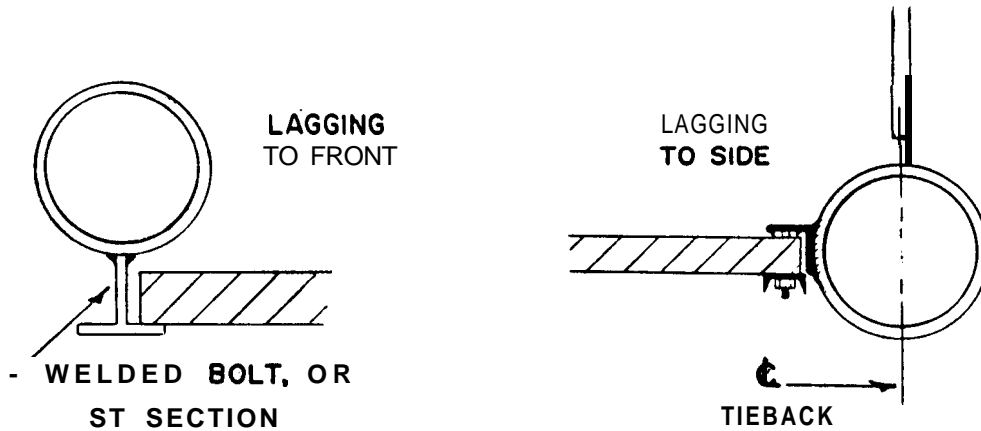
(a) WF SECTION OR H-PILE SECTION

LAGGING CAN ALSO BE
ATTACHED TO FRONT
FLANGE

WEDGED BEHIND FRONT FLANGE

(b) CHANNEL SECTION

LAGGING CAN ALSO BE
ATTACHED TO FRONT
FLANGE, ADAPTABLE FOR
TIEBACKS

BEHIND
FRONT FLANGE

WEDGE

e.g. WOSSERBDARRAGH
(1970)

TIEBACK

(c) PIPE SECTION

LAGGING
TO FRONT

LAGGING
TO SIDE

- WELDED BOLT, OR
ST SECTION

TIEBACK
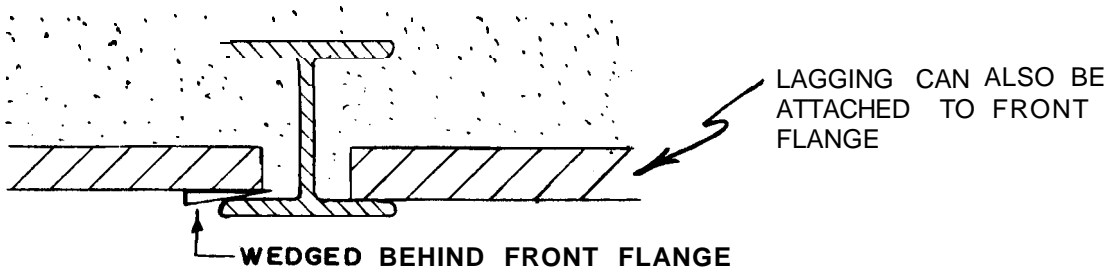
e.g. DONOLO (1971)

Figure 50. Steel soldier piles.

(a) CONTACT SHEETING
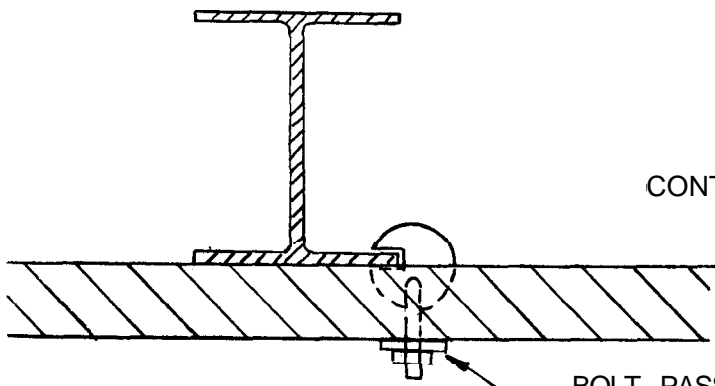
CONTACT SHEETING INCORPORATE0

(NYACK , N .Y.)

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

(b) BOLT
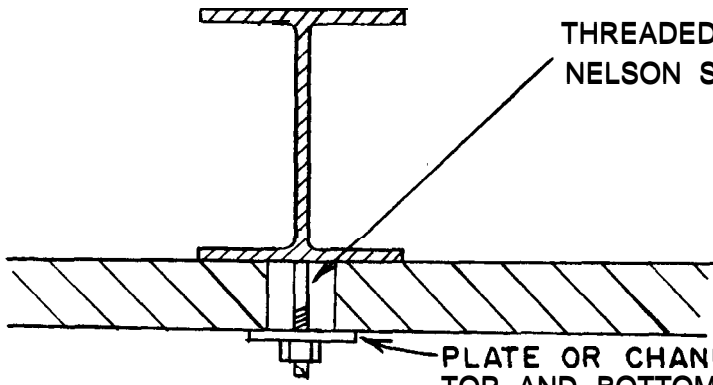
THREADED BOLT ATTACHED BY
NELSON STUD OR RAM SET,

PLATE OR CHANNEL SECTION HOLDS
TOP AND BOTTOM LAGGING.

(c) SPLIT T- SECTION
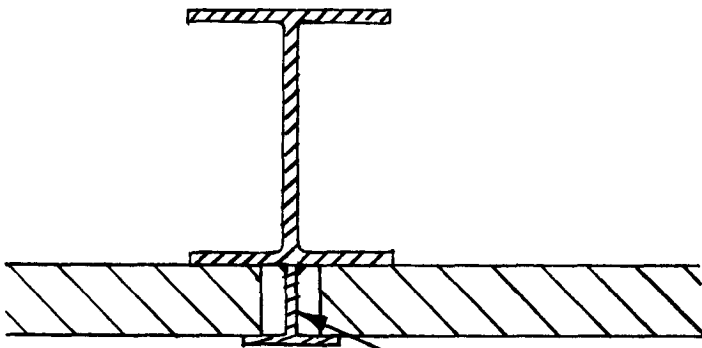
SPLIT "T" WELDED TO FACE

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.
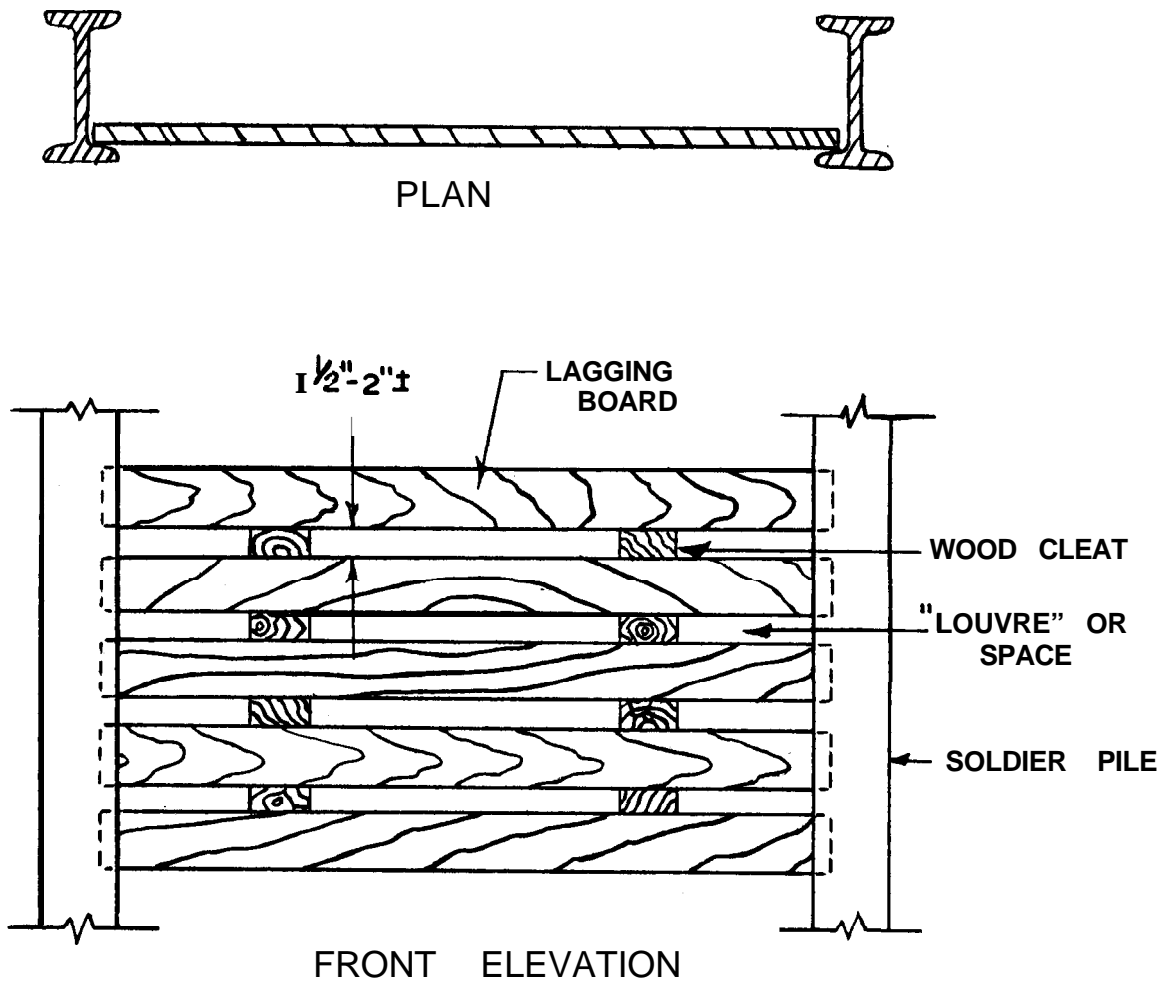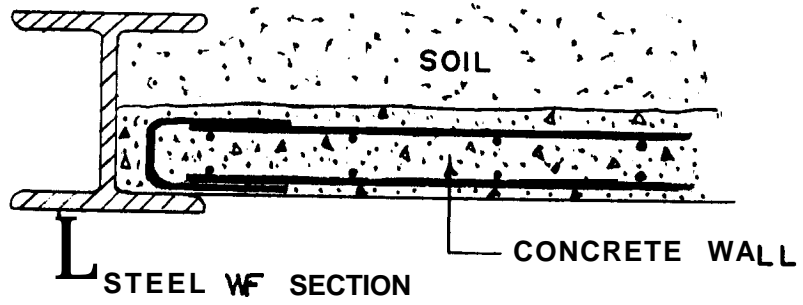
PLAN

LAGGING
BOARD

$I\frac{1}{2}"-2"\pm$

WOOD CLEAT

"LOUVRE" OR
SPACE

SOLDIER PILE

FRONT ELEVATION

Figure 52. Louvre effect for wall Lagging.

(a) CAST IN PLACE

SOIL

CONCRETE WALL

STEEL WF SECTION

(b) SHOTCRETE
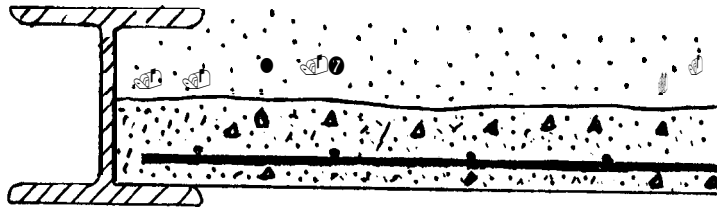
Figure 53. Concrete infill between soldier piles.

REINFORCING

PREFABRICATED ELEMENT

GROUT

GROUT

EXPOSE HORIZONTAL STEEL

SET REBARS AND FORMS AS EXCAVATION PROCEEDS DOWNWARD.

TIEBACK

CAST-IN-PLACE SECTION
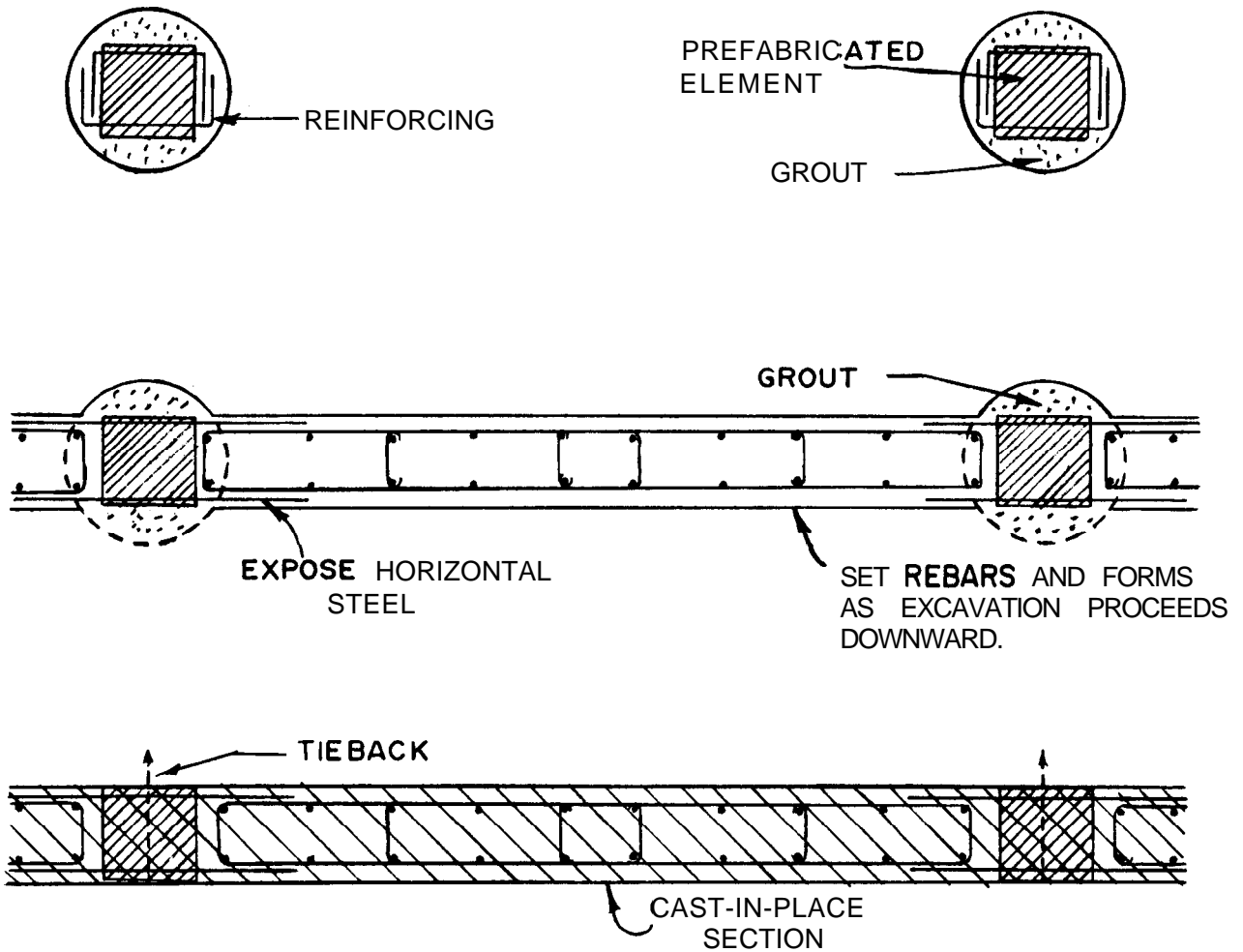
Figure 54. Parisienne wall, precast soldier piles with formed cast-in-place wall, (after Fenoux, 1974; Xanthakos, 1974; and D'Appolonia, et al, 1974).
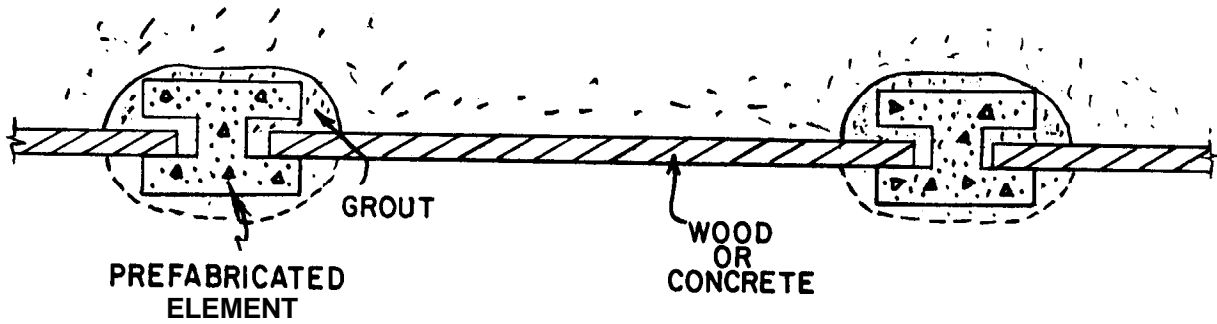
Figure 55. Berlin wall, precast soldier piles with
wood or precast concrete lagging (after Fenoux, 1974).

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.	M L	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.
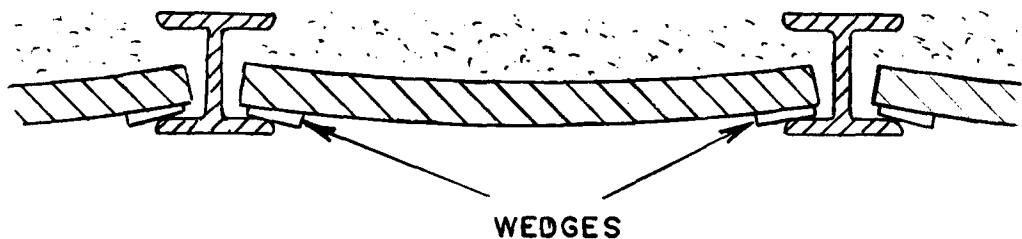
9.33.5   Pre-excavation for Soldier Piles

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.

WEDGES

(b) CONCRETE
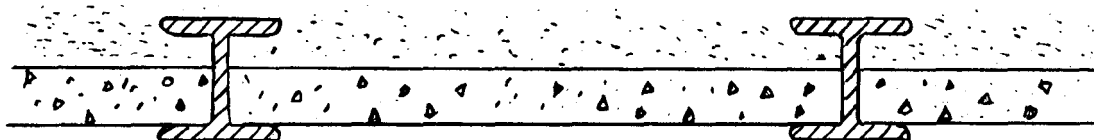
(c) VERTICAL STEEL SHEETS IN SOFT SOIL

DETAIL

A

WEDGE

STEELSHEET

WALE

SOLDIER PILE

DETAIL

STEEL
SHEET
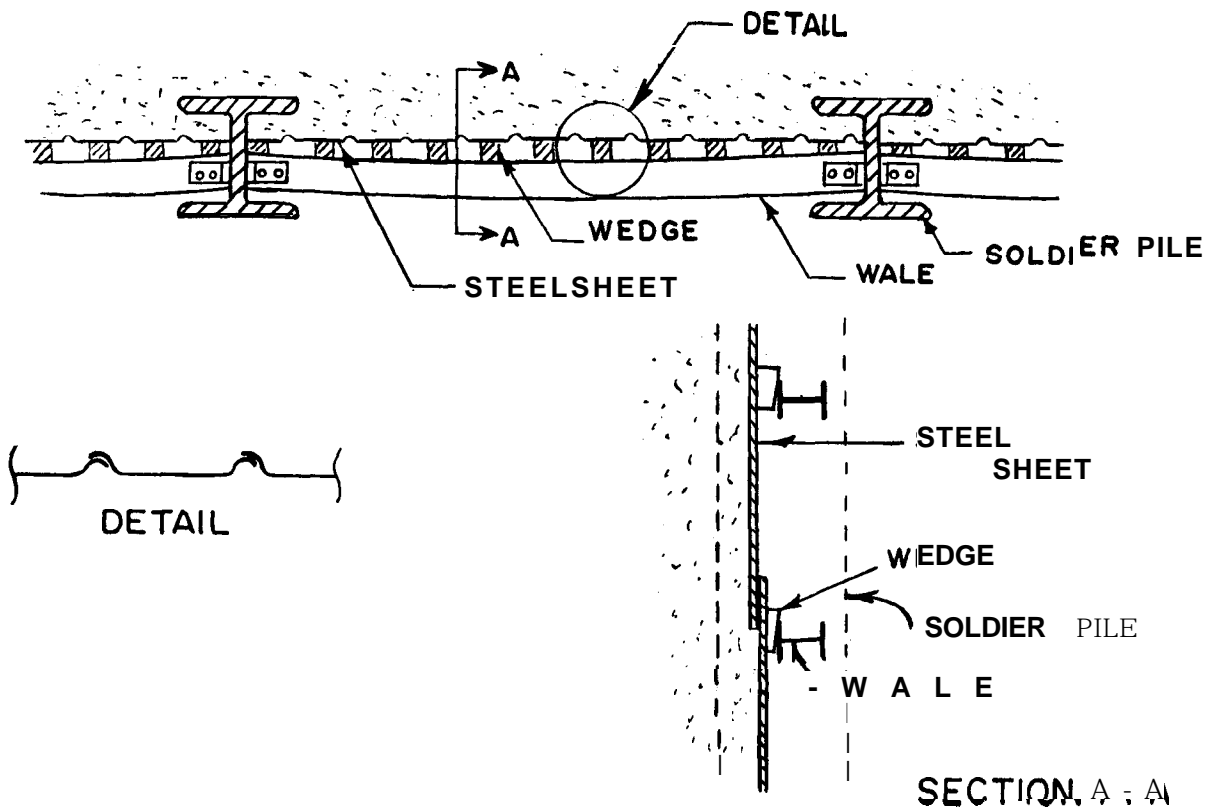
WEDGE

SOLDIER PILE

WALE

SECTION A - A

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

-123-

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 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.

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 filter-
ing 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
C hroma ted 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 inter-locked 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 piezo-metric 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 in-side 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 effec-tive in bending. European practice assumes interlock friction and
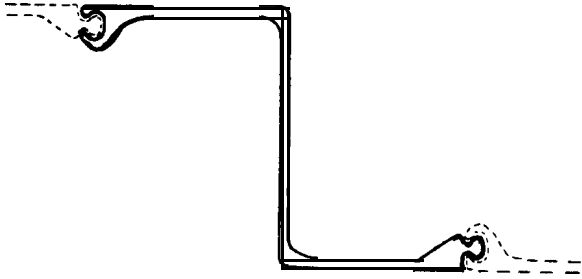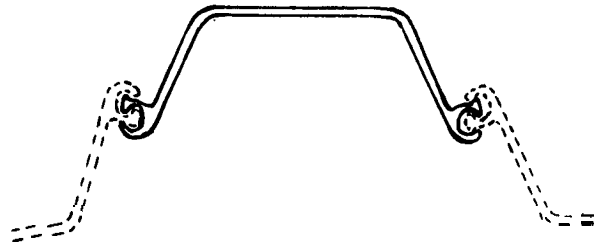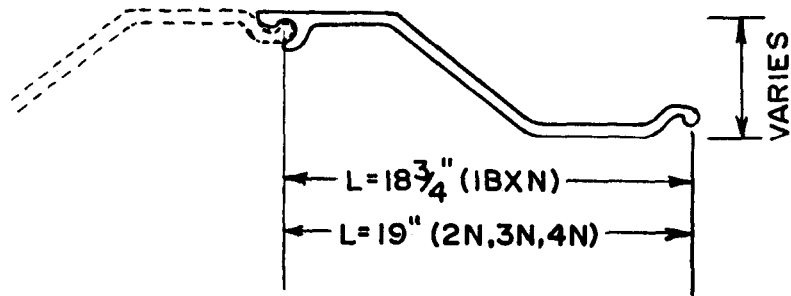
PZ 38
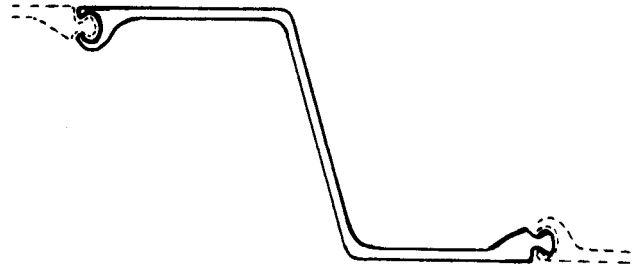
PZ 32 8 PZ 27
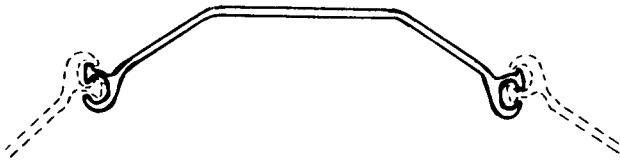
PMA 22

PDA 27

Figure 57. **Domestic sheet pile sections.**
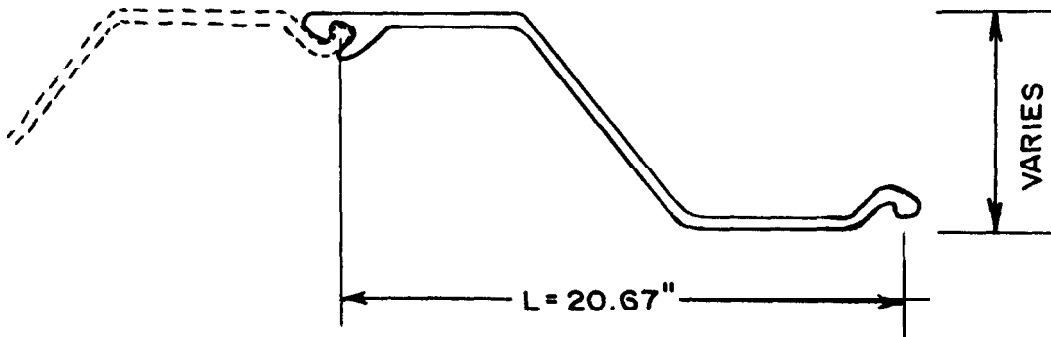
Table 6.   Domestic   steel   sheet   pile   sections.

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   IBXN,2N,3N,4N

L=18¾" (IBXN)
L=19" (2N,3N,4N)

VARIES

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

L=20.67"

VARIES

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

L=19.68"

VARIES
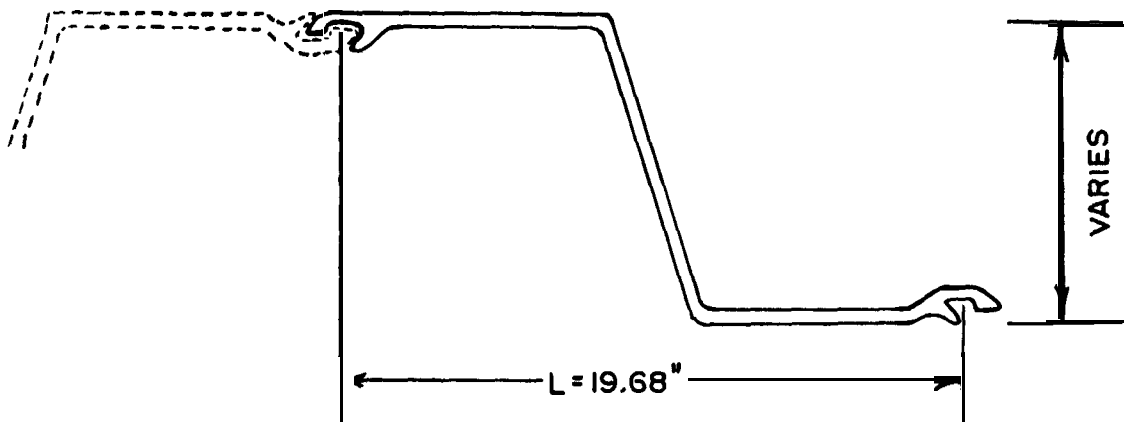
Figure 58. Foreign sheet pile sections.

Table '7. Foreign steel sheet pile sections.

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

(1) Data from L. B. Foster Company, Pittsburgh, Pa.
(2) Data from Skyline Industries, Port Kearny, N. J.

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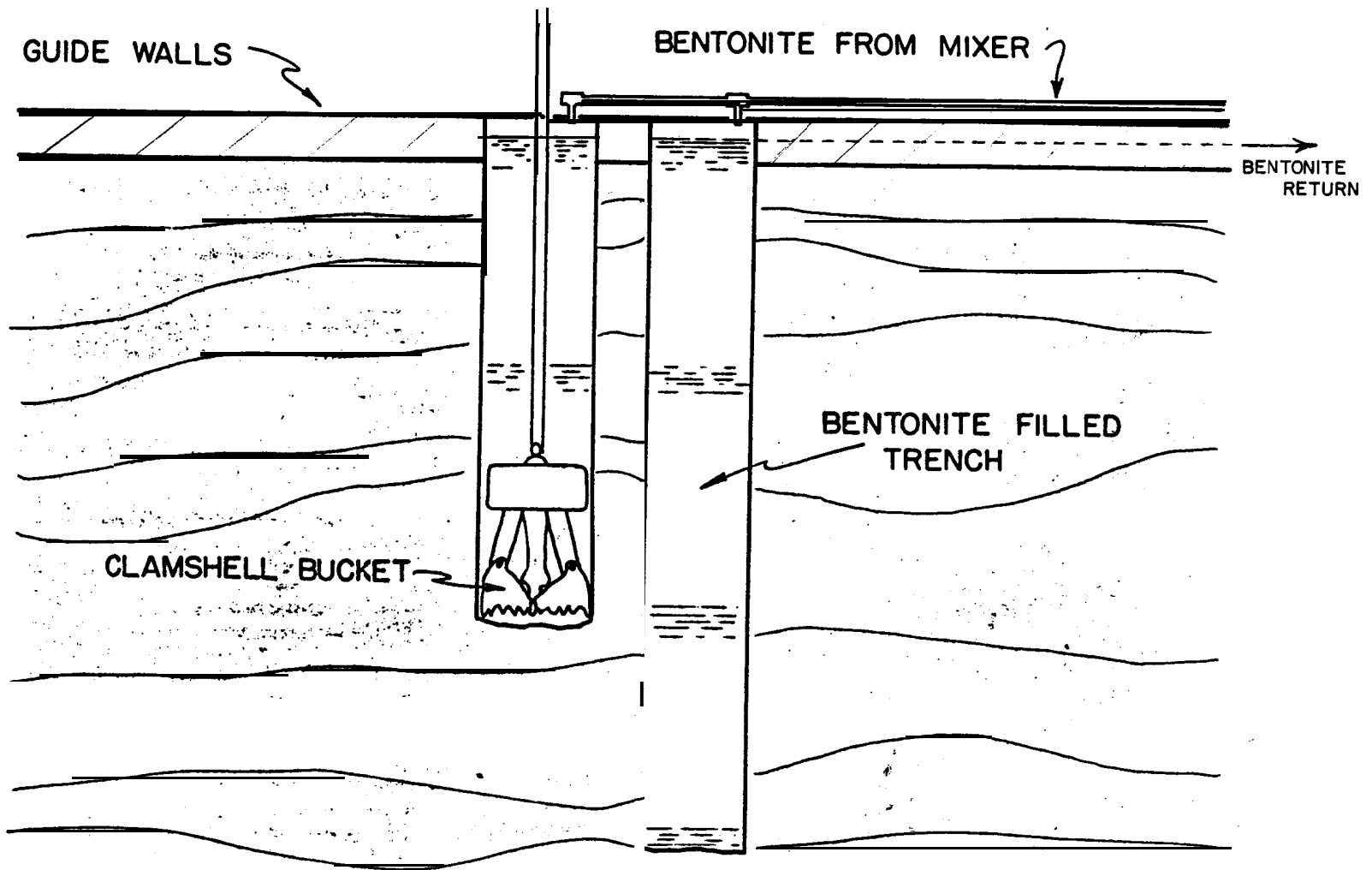
Figure 59. Excavation by clamshell bucket.
(Courtesy of ICOS Corporation).

REINFORCING STEEL CAGE BEING LOWERED
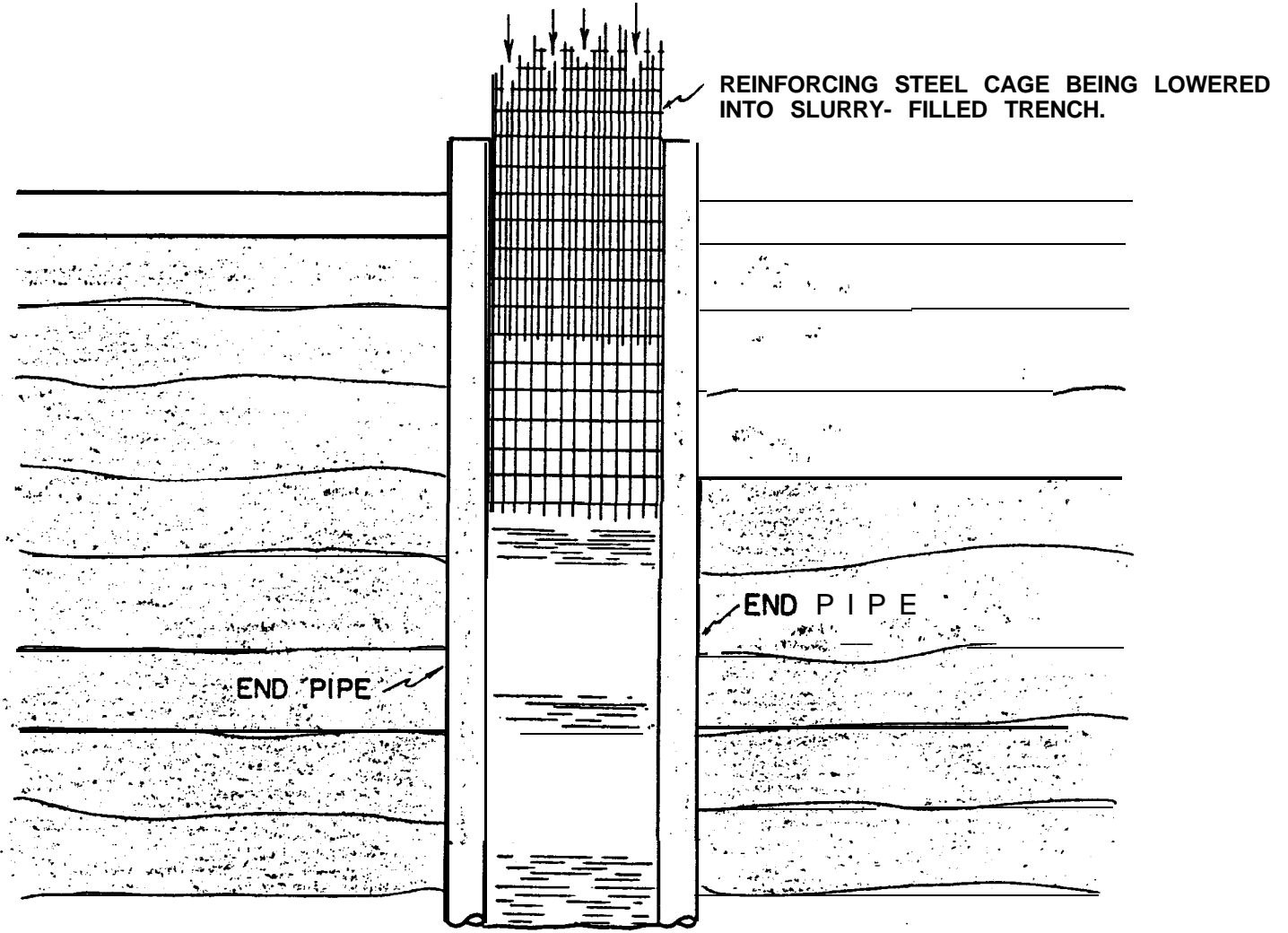INTO SLURRY- FILLED TRENCH.

END PIPE

END PIPE

Figure 60. Preparations for concrete placement.
(Courtesy of ICOS Corporation).
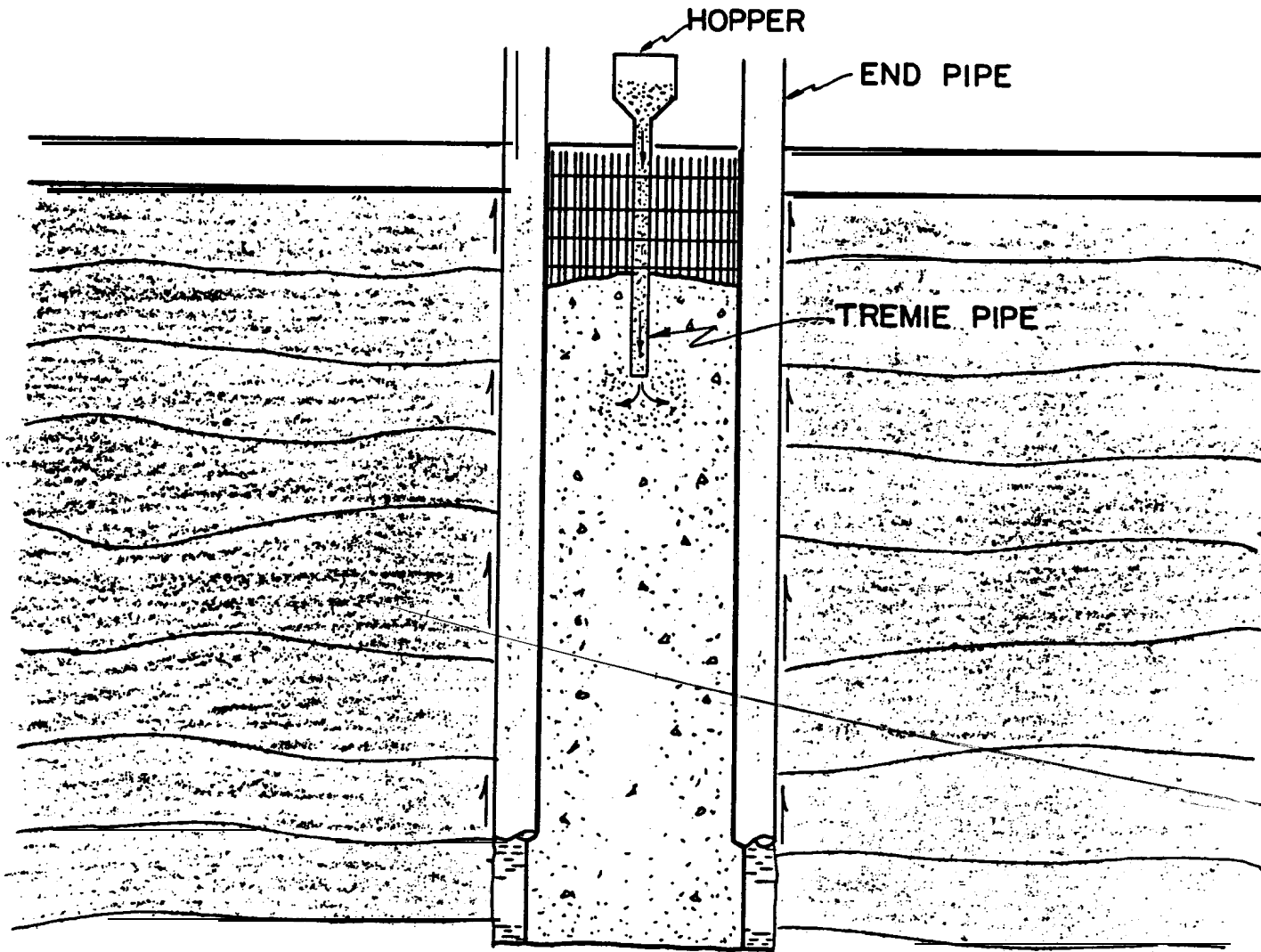
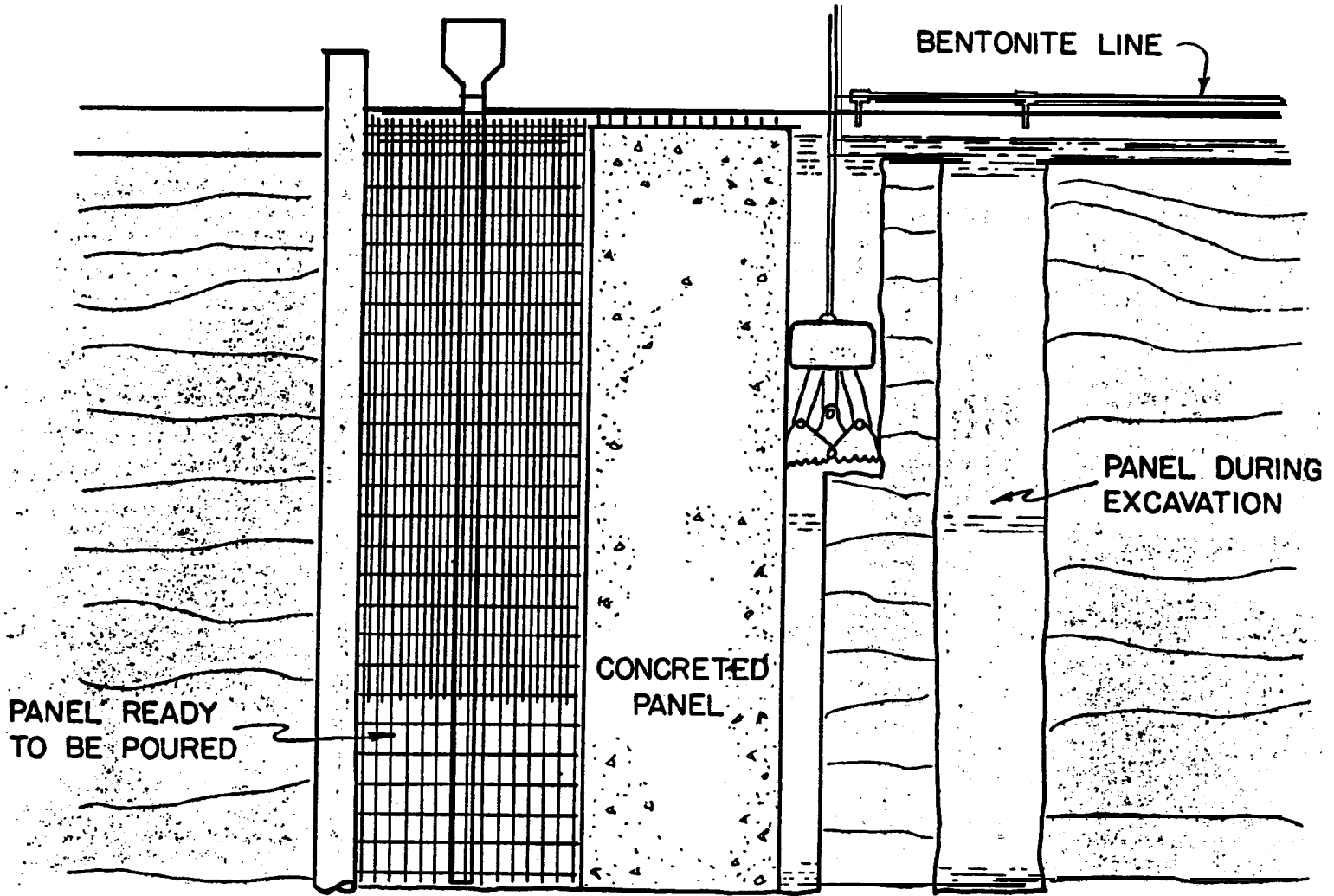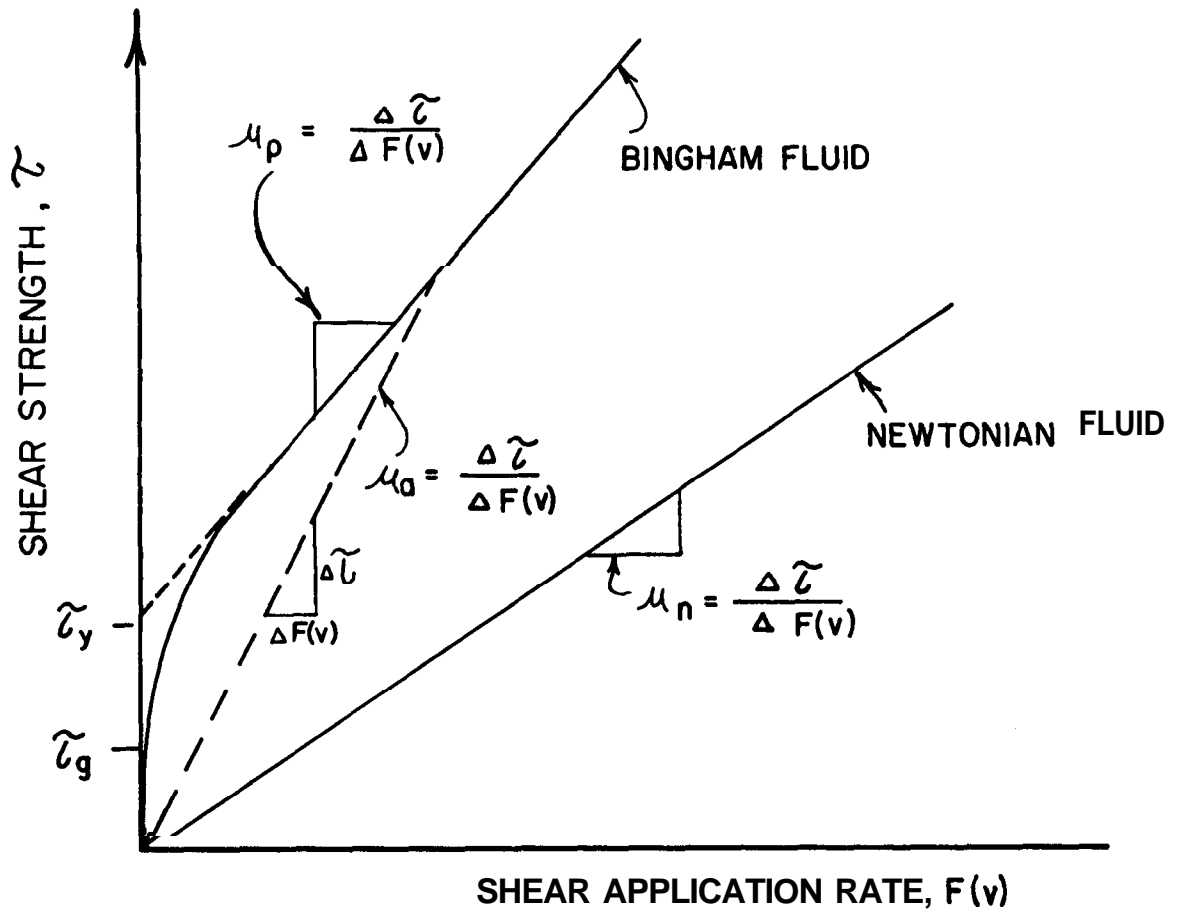Figure 61. Placement of concrete.
(Courtesy of ICOS Corporation).

BENTONITE LINE

PANEL DURING
EXCAVATION

CONCRETED
PANEL

PANEL READY
TO BE POURED

-140-

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

$$\mu_p = \frac{\Delta \tilde{\tau}}{\Delta F(v)}$$

BINGHAM FLUID

NEWTONIAN FLUID

$$\mu_a = \frac{\Delta \tilde{\tau}}{\Delta F(v)}$$

$$\Delta \tilde{\tau}$$

$$\mu_n = \frac{\Delta \tilde{\tau}}{\Delta F(v)}$$

$$\Delta F(v)$$

$\tilde{\tau}_y$

$\tilde{\tau}_g$

SHEAR STRENGTH, $\tilde{\tau}$

SHEAR APPLICATION RATE, F(v)

$\tilde{\tau}_g$ = Gel shear strength

$\tilde{\tau}_y$ = Yield shear strength

μ_n = Viscosity of Newtonian fluid

μ_a = Apparent viscosity of Bingham fluid

μ_p = Plastic viscosity of Bingham fluid

F(v) = A parameter which is for particular geometry. A function of shear application rate.

Figure 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 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 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).

<u>d. Fluid Loss in Highly Pervious Soils.</u>
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.

<u>11.42.4 Control Testing</u>

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

<u>Viscosity and Gel Strength</u>

<u>Fann Viscometer.</u>

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.

<u>Shearometer.</u> 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.

<u>Marsh Cone.</u> 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
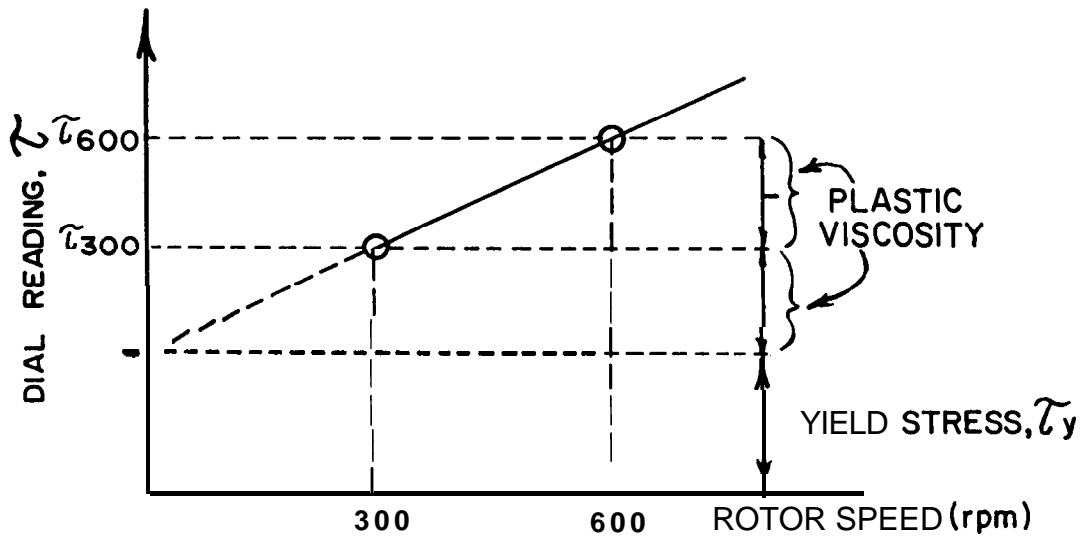
PLASTIC VISCOSITY $= \tau_{600} - \tau_{300}$

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

ROUND JOINT, USING STOP- END TUBE

PRIMARY PANEL   SECONDARY PANEL

PRIMARY PANEL
CONCRETED

STOP- END REMOVED

PRIMARY PANEL AND
SECONDARY PANEL
CONCRETED
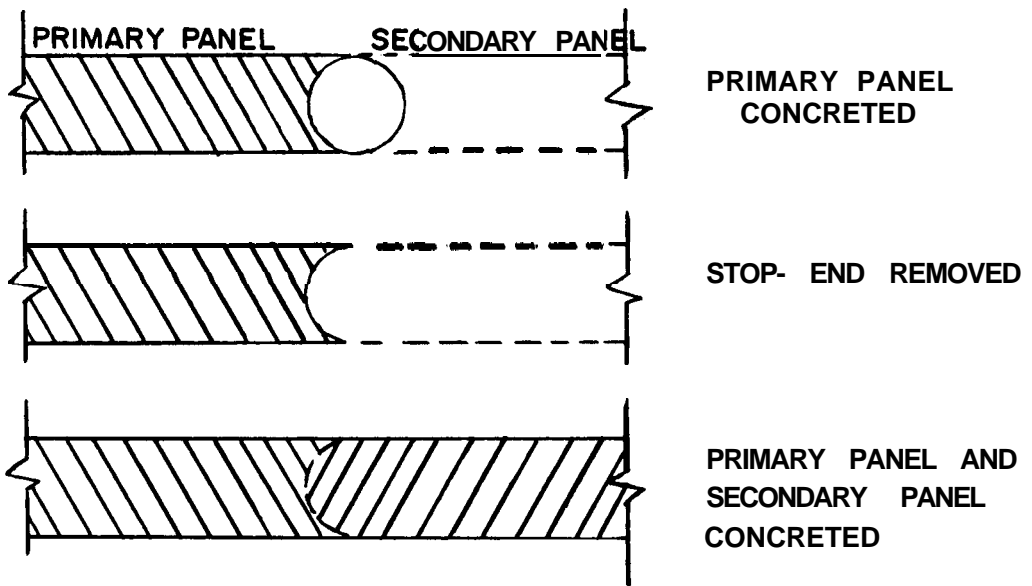
Figure 65. Panel joint with stop-end tube.

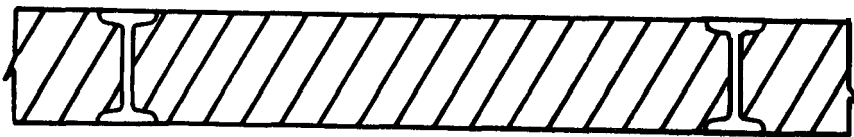TWO-STEP   EXCAVATION
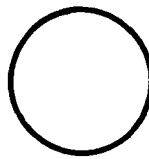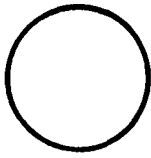
AUGER   PRIMARY
        HOLES.
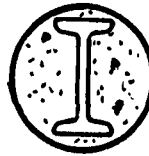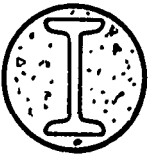
SET SOLDIER PILES;
FILL WITH LEAN
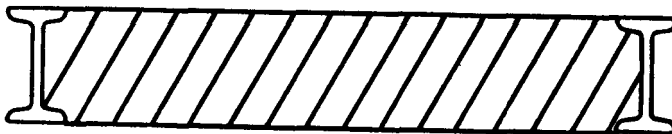CONCRETE OR GROUT,
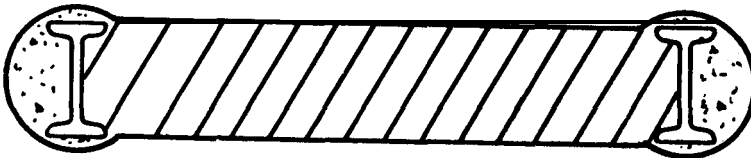
TREMIE   PRIMARY
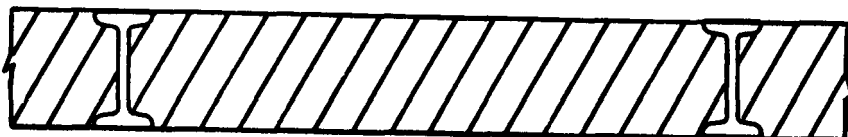   PANEL.

TREMIE   SECONDARY
         PANEL.

ONE -STEP EXCAVATION

EXCAVATE PRIMARY
PANEL AND SET
SOLDIER PILES.

TREMIE PRIMARY
        PANEL.

TREMIE SECONDARY
        PANEL.

Figure 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).

(a) COMPACT COHESIVE SOIL

CONCRETE WALL

COMPACTED BACKFILL

4'-5' ±

(b) LOOSE COHESIONLESS SOIL

CONCRETE WALL

COMPACTED BACKFILL

4'-5' ±
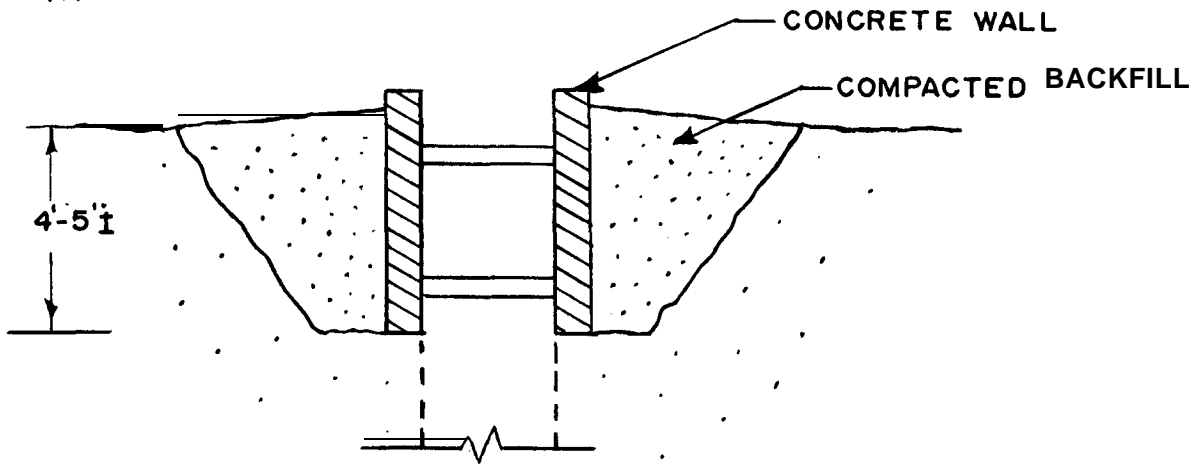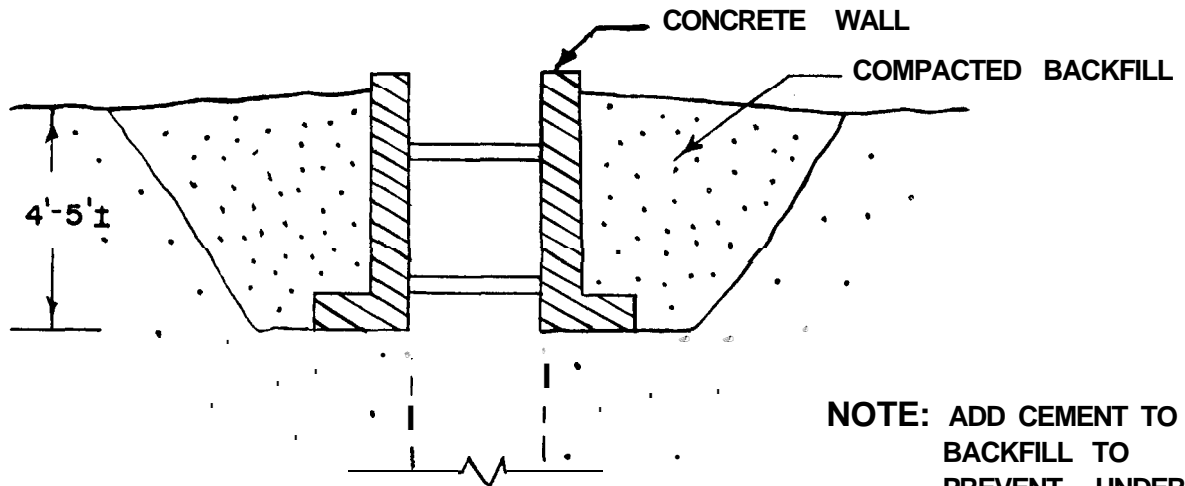
NOTE: ADD CEMENT TO BACKFILL TO PREVENT UNDER-MINING AND TO INCREASE STABILITY

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
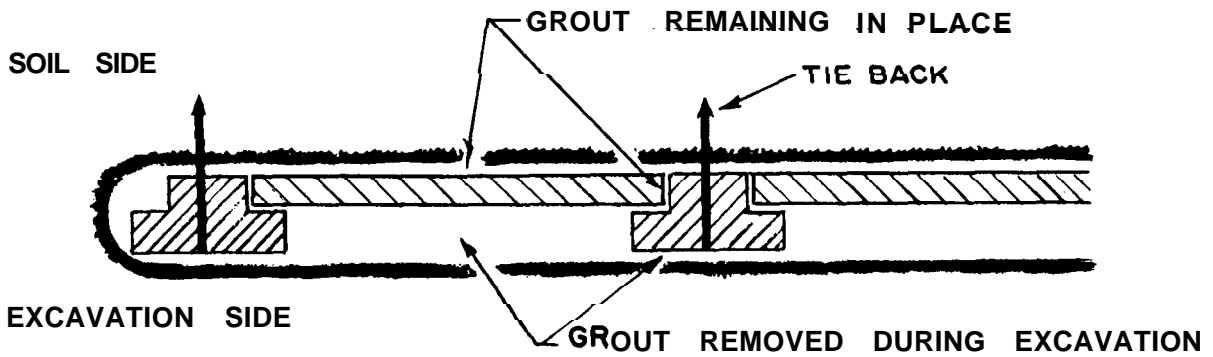
(a) TONGUE &GROOVE
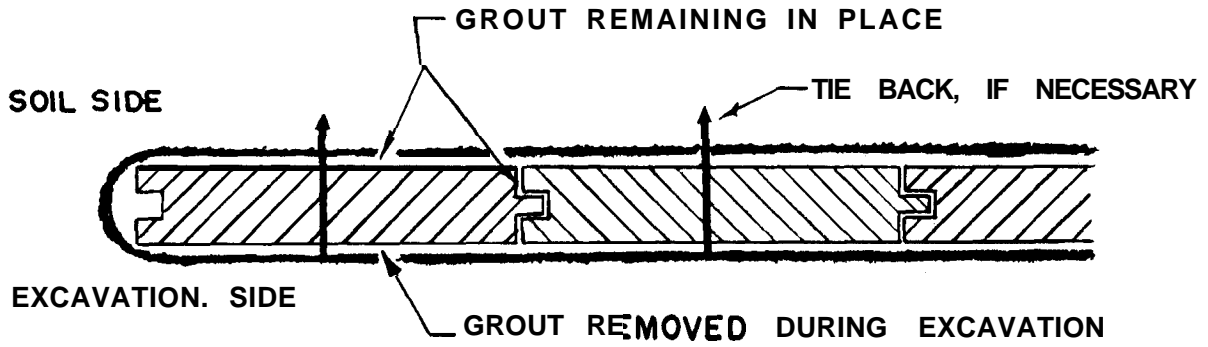
(b) T BEAMS & SLABS
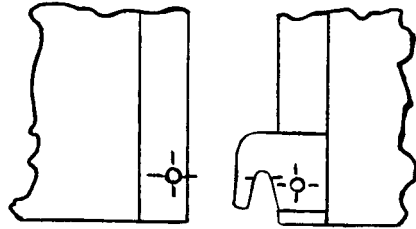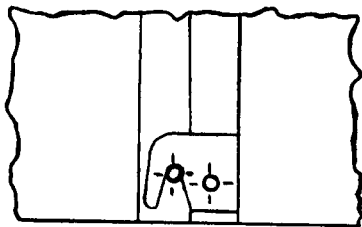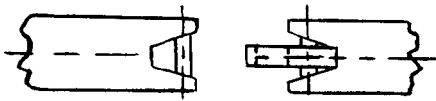

Figure 68. Panosol walls (Soletanchc, France).

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.

1

2

3

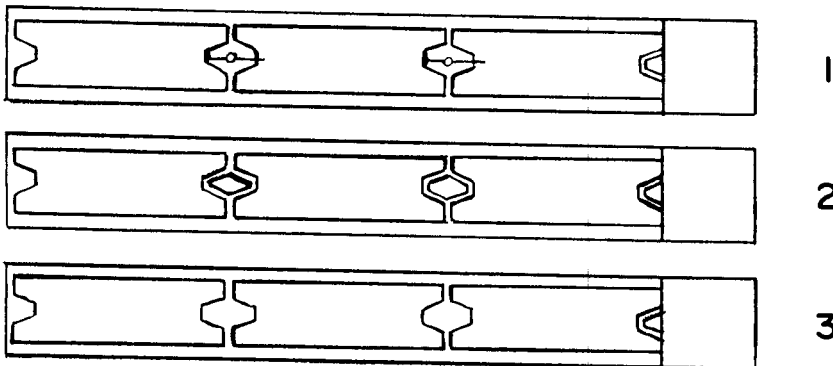Figure 69. Prefasif wall (from Bachy Enterprise, France).

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.
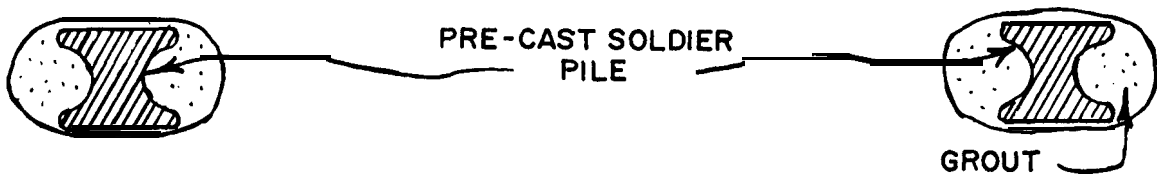
11. 74 Bored Pile Walls

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

PRE-CAST SOLDIER PILE

GROUT

2. EXCAVATE AND CONCRETE PANEL

CAST IN PLACE CONCRETE

REINFORCING STEEL

PREFABRICATED JOINT

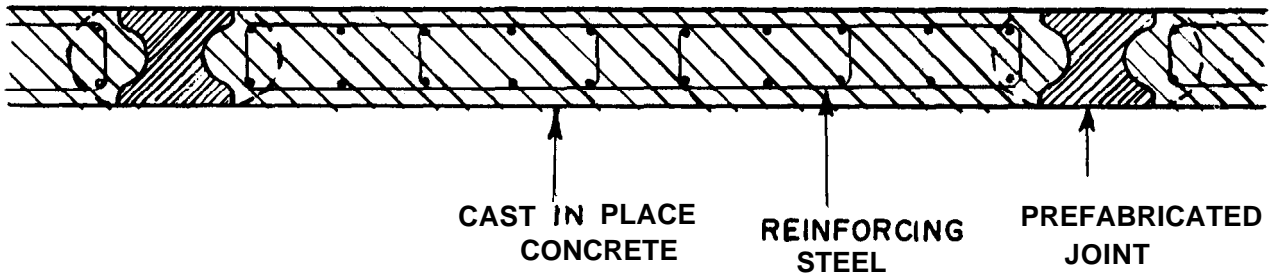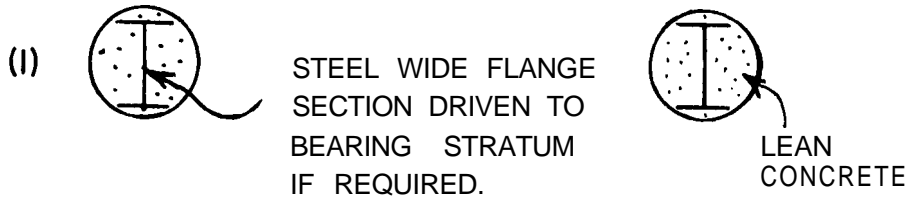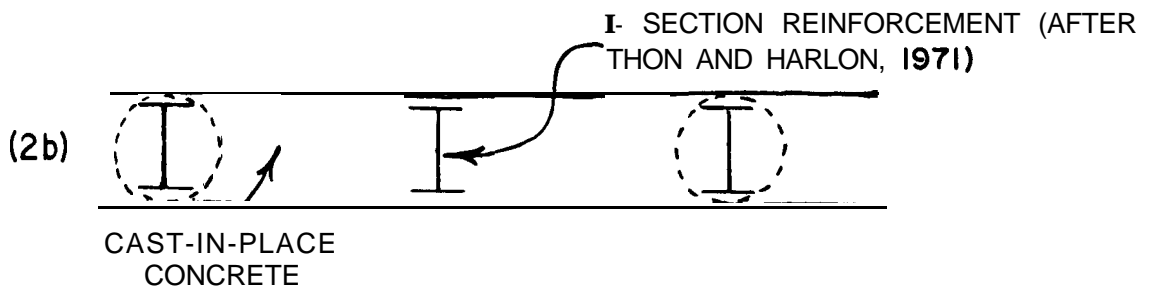**Figure 70. Two step excavation in slurry trench using precast soldier piles and tremie concrete.**

SET SOLDIER PILE IN PRE- EXCAVATED HOLE.

(1) STEEL WIDE FLANGE
SECTION DRIVEN TO
BEARING STRATUM
IF REQUIRED.

LEAN
CONCRETE

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

(2a)

CAST-IN- PLACE
CONCRETE

REBAR
REINFORCEMENT

I- SECTION REINFORCEMENT (AFTER
THON AND HARLON, 1971)

(2b)

CAST-IN-PLACE
CONCRETE
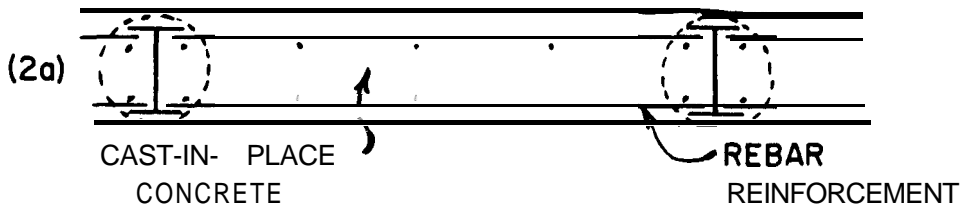
Figure 71. Two step excavation in slurry trench
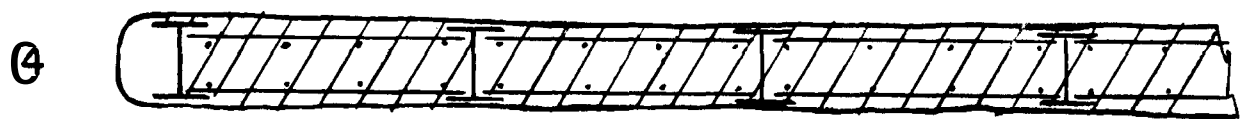using steel wide flange soldier piles and tremie concrete.

①
SET SOLDIER PILES AND REBARS
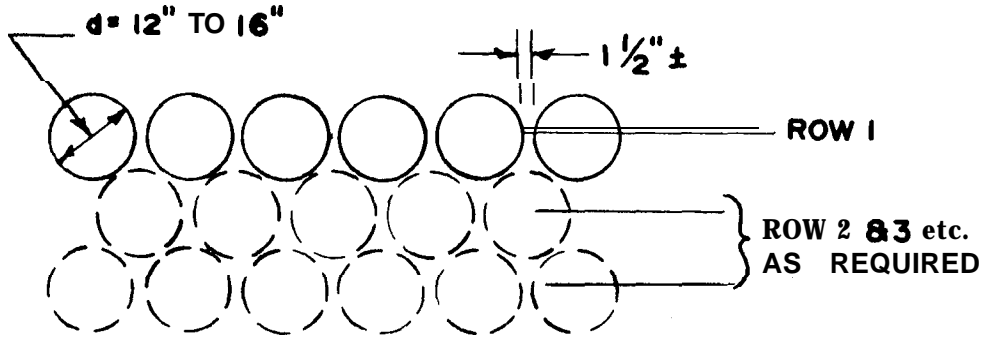IN ALTERNATE PANELS.

②
CONCRETE ALTERNATE PANELS.
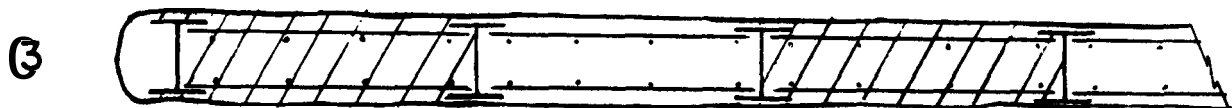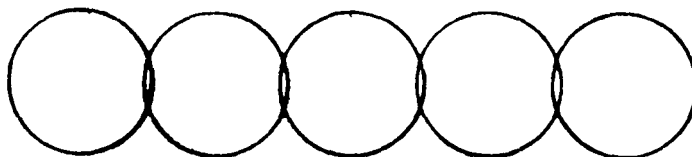
③
EXCAVATE INTERVENING PANELS
AND SET REBAR CAGE.

④
CONCRETE

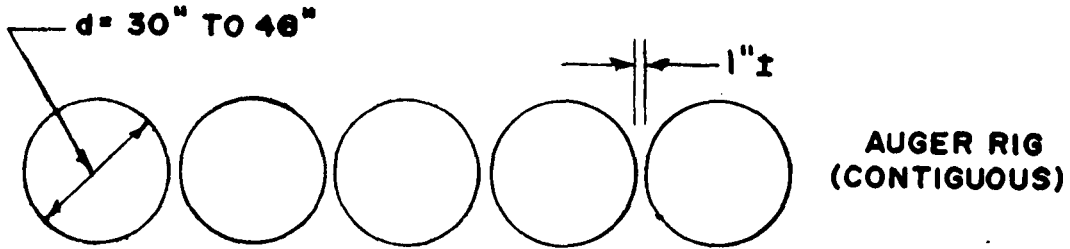Figure 72. One step excavation with soldier piles
(after Tamaro, 1974).

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

d = 12" TO 16"

1½"±

ROW 1

ROW 2 &3 etc. AS REQUIRED

GROUT FILLED WITH HOLLOW STEM AUGER

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

d = 30" TO 48"

1"±

AUGER RIG (CONTIGUOUS)

ROTATING CASING WITH CUTTING EDGE.

(SECANT)

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 s uppo rt , 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.
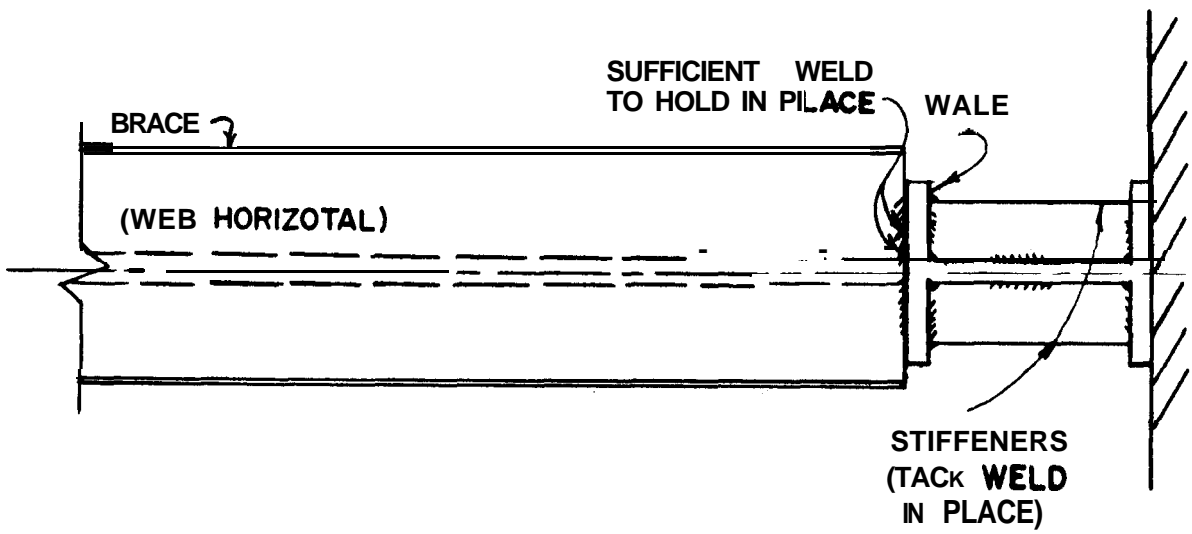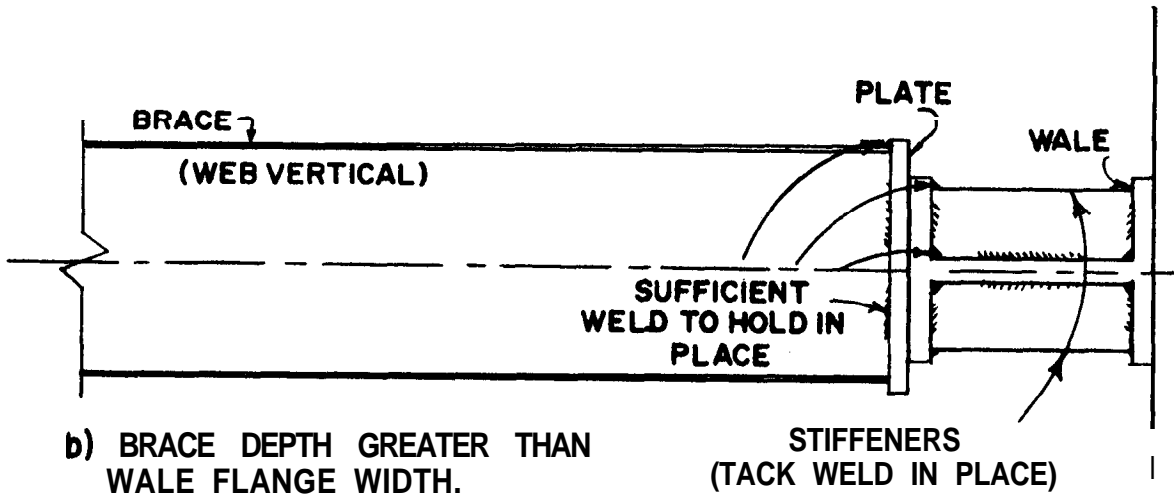
Figure 74. **Typical detail for horizontal brace with brace web horizontal.**

a) BRACE DEPTH SMALLER THAN WALE FLANGE WIDTH,

b) BRACE DEPTH GREATER THAN WALE FLANGE WIDTH.
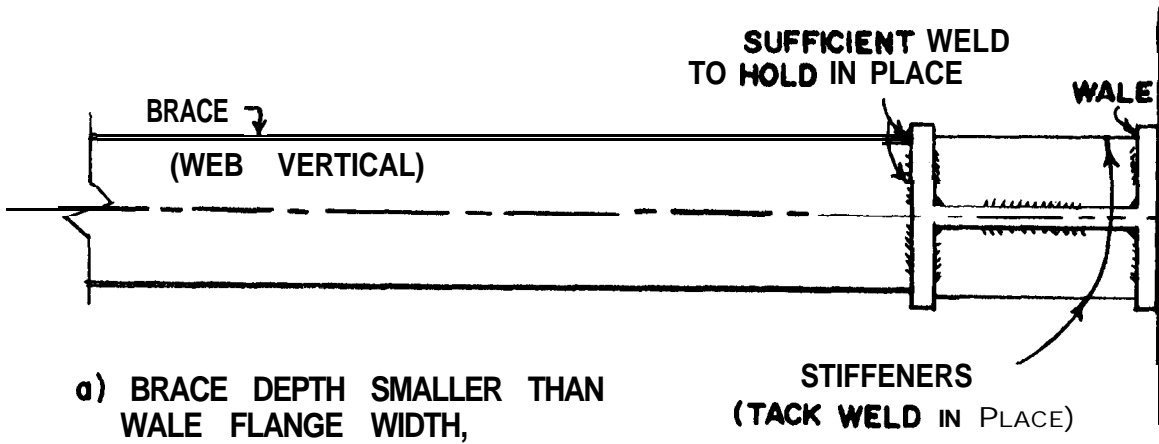
NOTE: DETAILS SIMILAR FOR HORIZONTAL BRACE.

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

INCLINED " KICKER" OR
"SPUR BRACE"

45°

WELD B

PLATE IF
NECESSARY

C

WALE

WELD A

BRACE

STIFF T & B
AT BRACE
($\frac{1}{2}$" THICK)

SOLDIER
BEAM

WELD A = WELD B & C =
VERTICAL COMPONENT
OF BRACE LOAD

HORIZONTAL WOOD
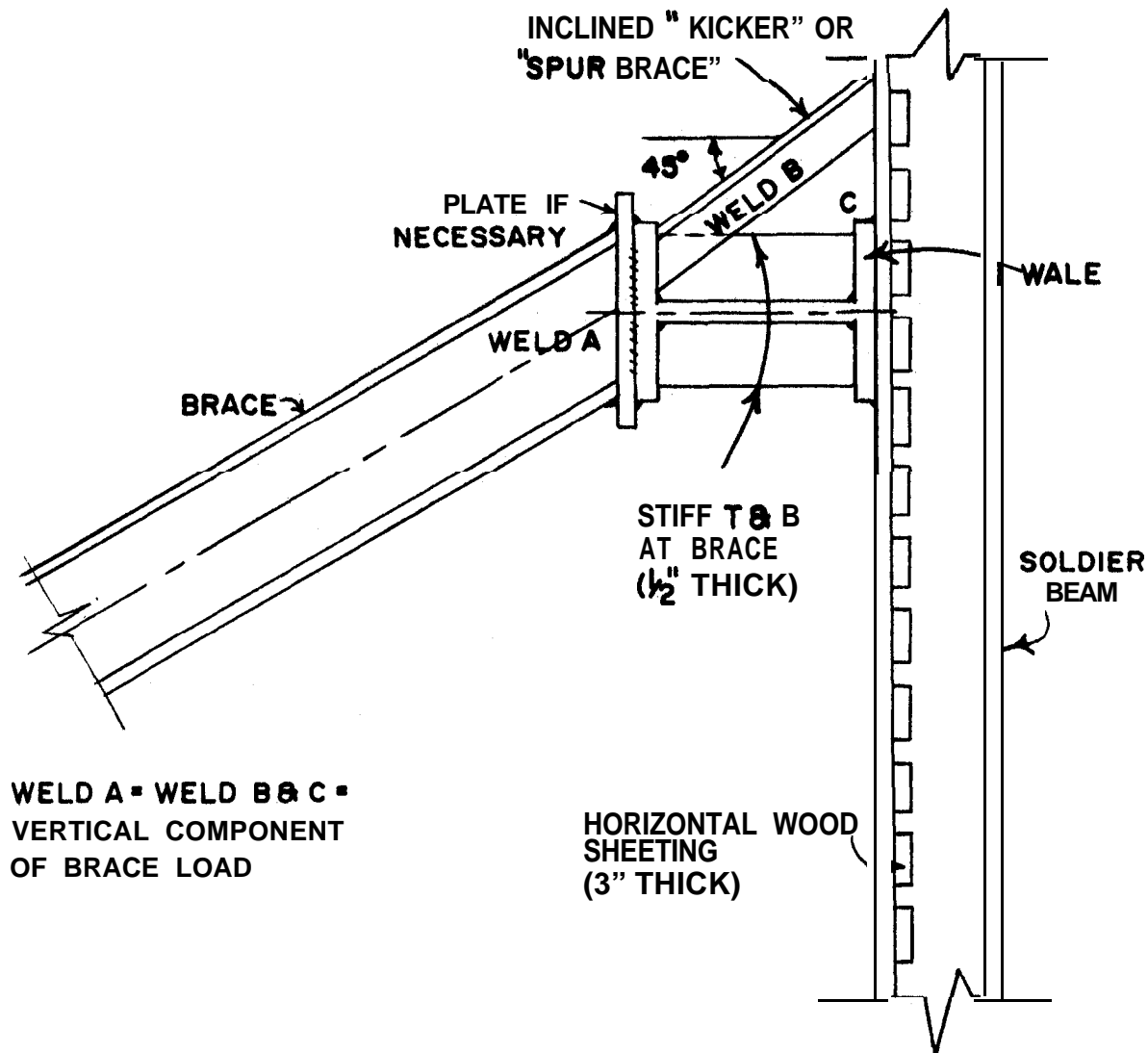SHEETING
(3" THICK)

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
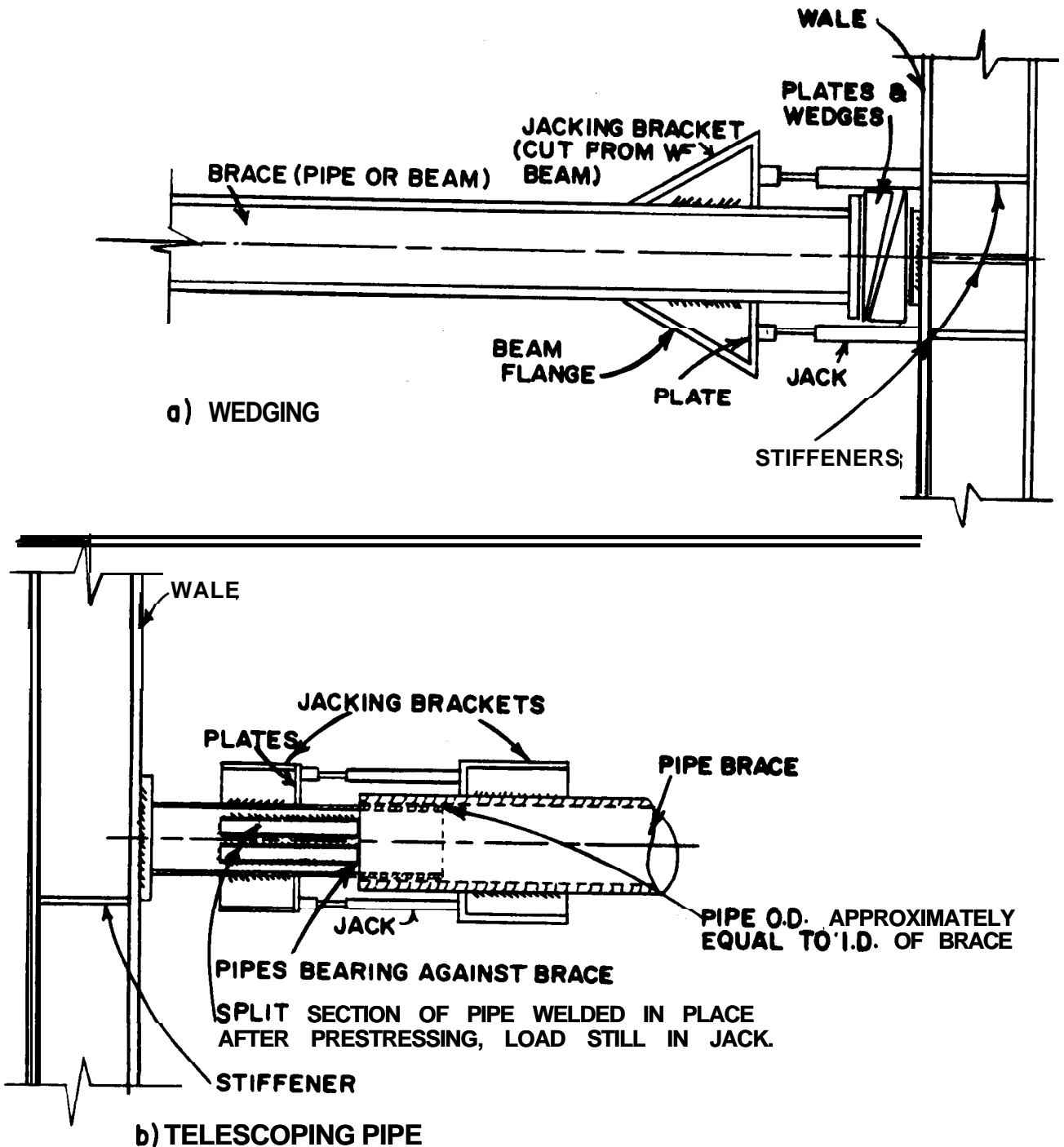
a) WEDGING

b) TELESCOPING PIPE
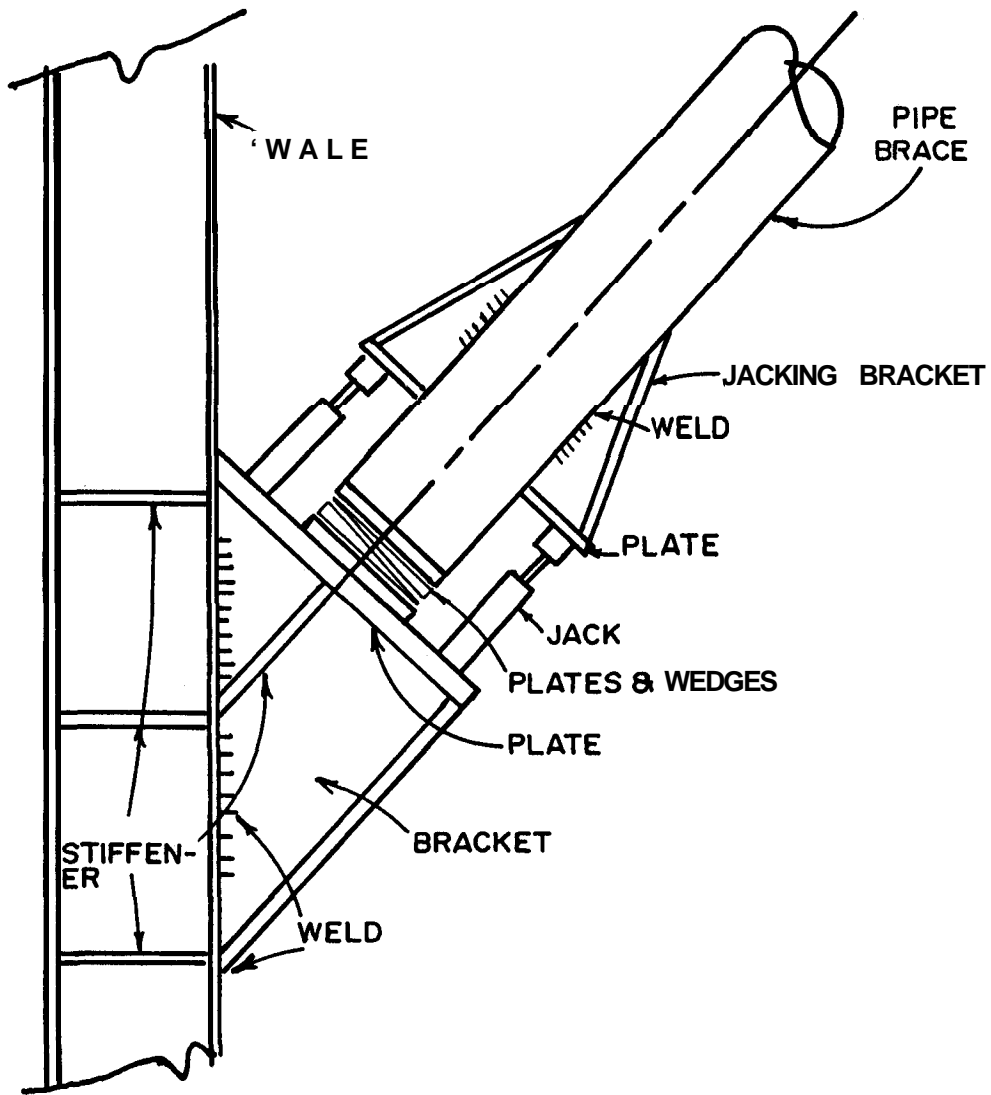
Figure 77. Preetreeeing details for braces.

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 be-cause 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 over-stressing.

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:

$$\Delta P \ = \ A_s E_s \ (\alpha \ x \ \Delta° \ F)$$

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)

$\Delta°F$ = 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/$^{\circ}$F
2. O'Rourke & Cording (1974)	Covered	0.5 kip/$^{\circ}$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. Tiebacks (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
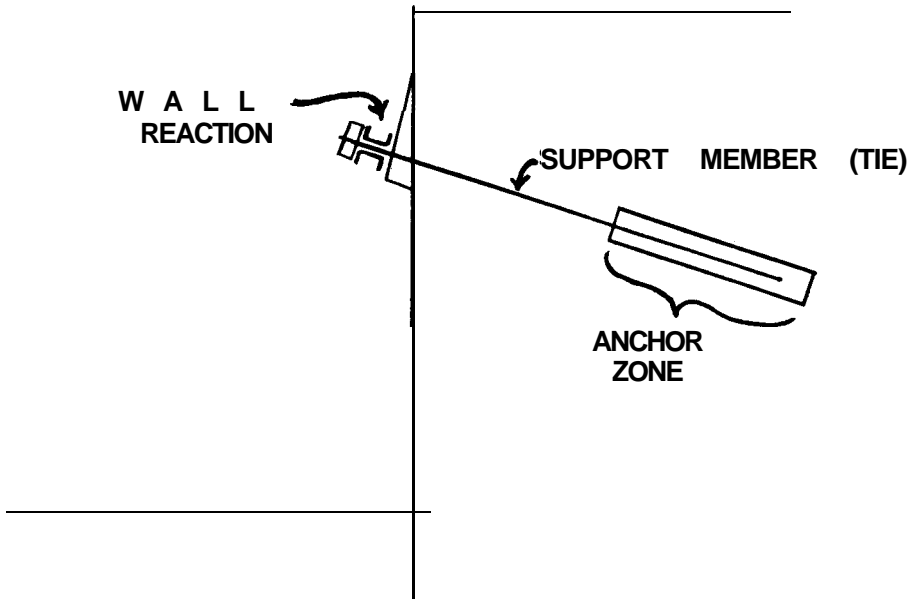
Figure 79. Major tieback components.

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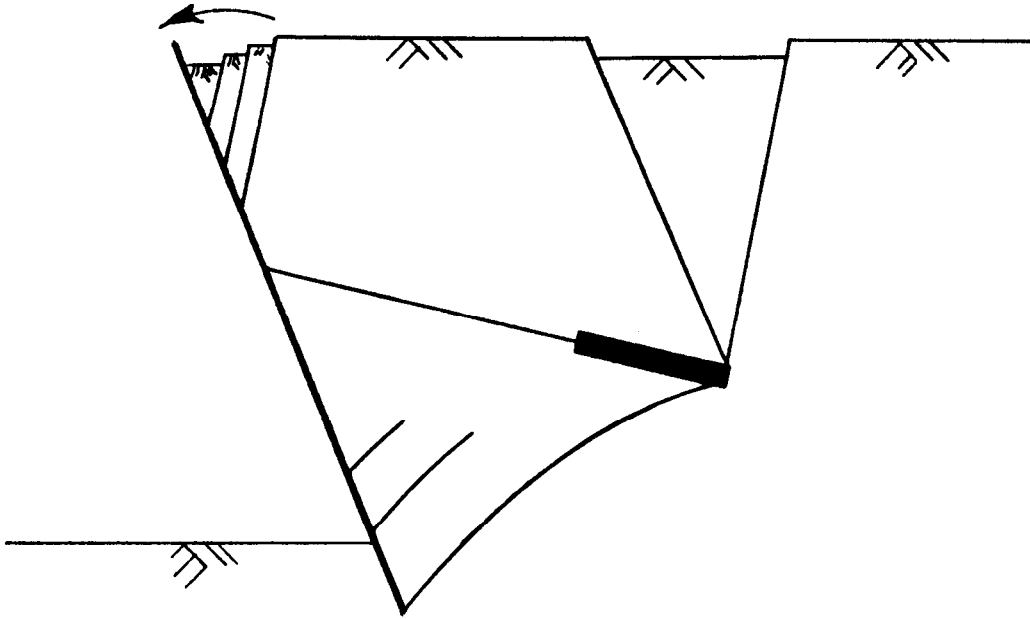
Figure 80. Sketch of tied-back wall failing by ove rtqrning.

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

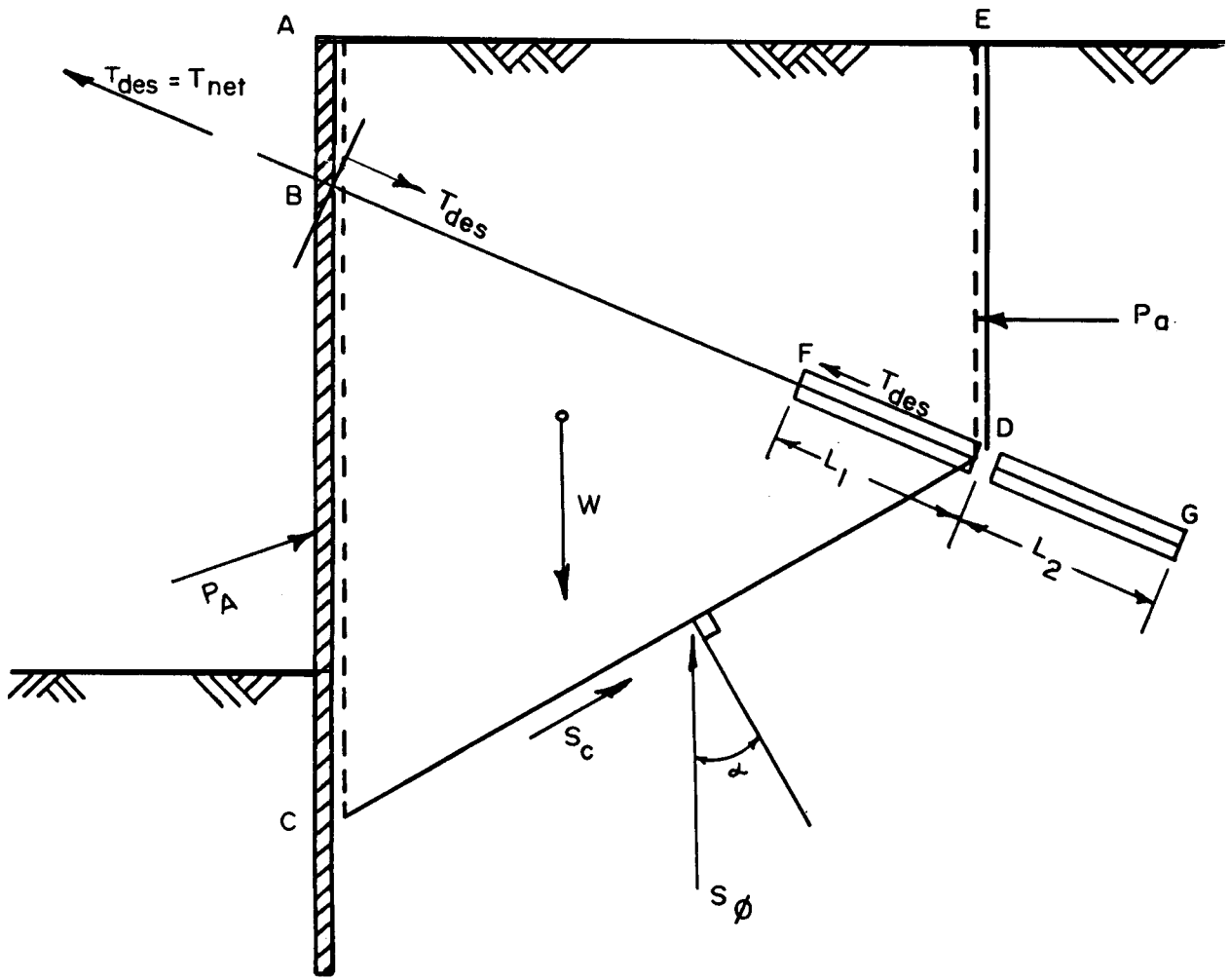The stability of the free body is evaluated in terms of

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. = $\dfrac{T_{max}}{T_{des}}$. 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.

$\alpha = \phi$ on failure plane
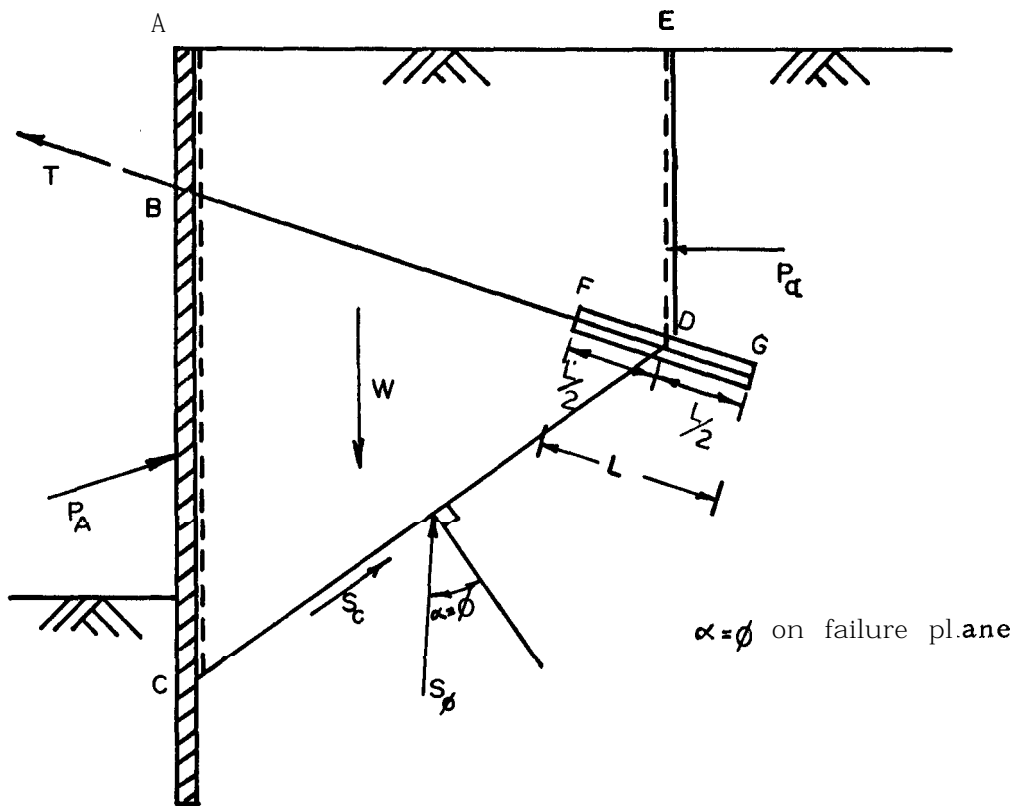
(a)
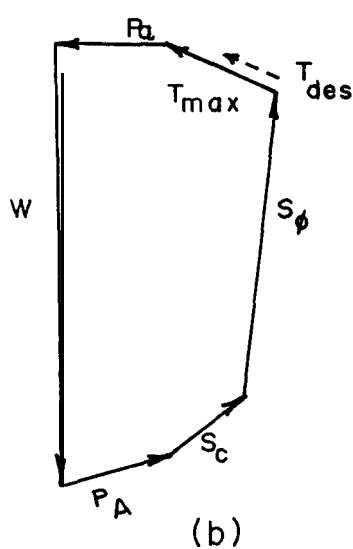
$$F.S. = \frac{T_{max}}{T_{des}} \geq 1.5$$

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

(b)

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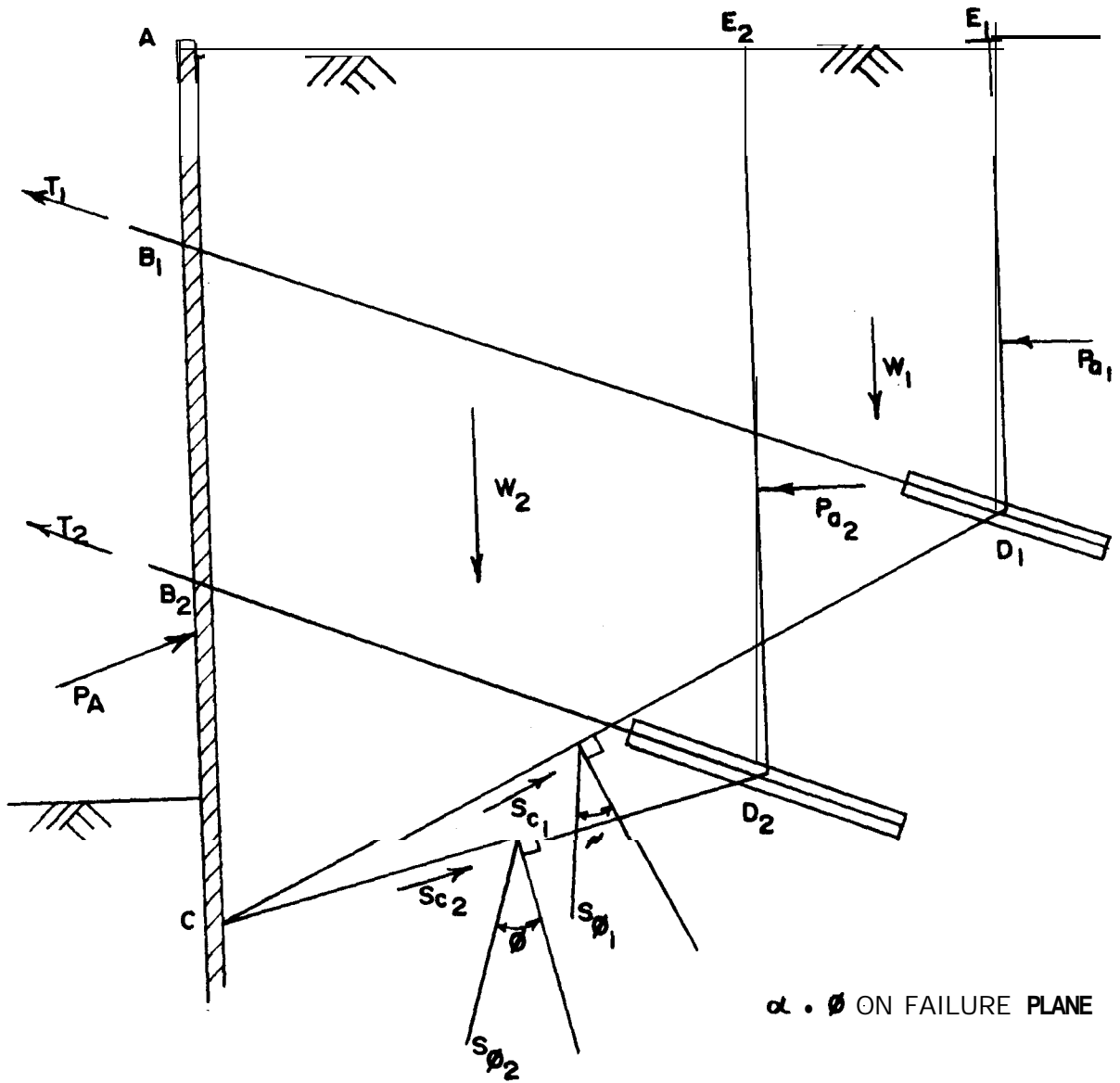(safety in terms of the tieback force).

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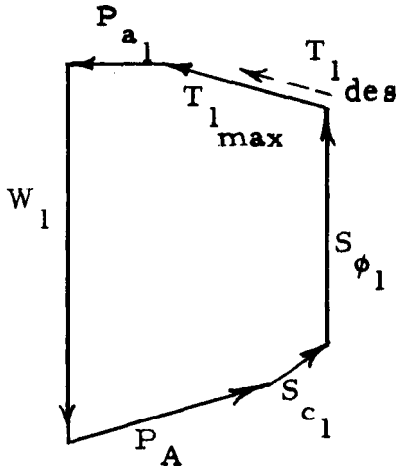
a. Upper Tieback

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

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

$\alpha \cong \phi$ on failure plane.

B. Lower Tieback

$$F.S. = \frac{T_{2_{max}}}{T_{2_{des}}} \geq 1 . 5$$
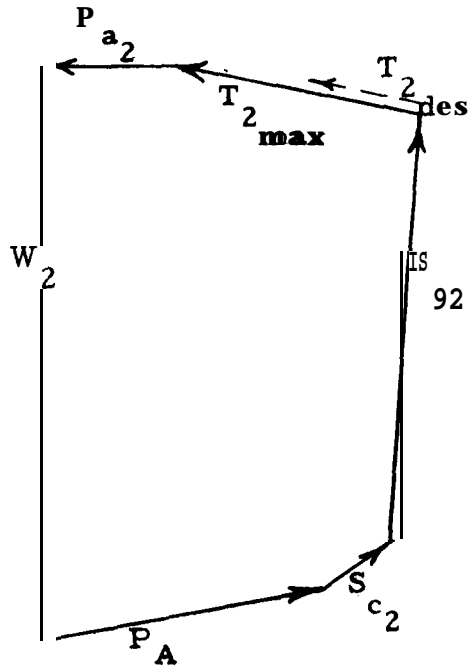
Figure 84.   Vector diagram for case of two completely independent anchors (safety in terms of anchor force) (after Ranke and Ostermayer, 1968).
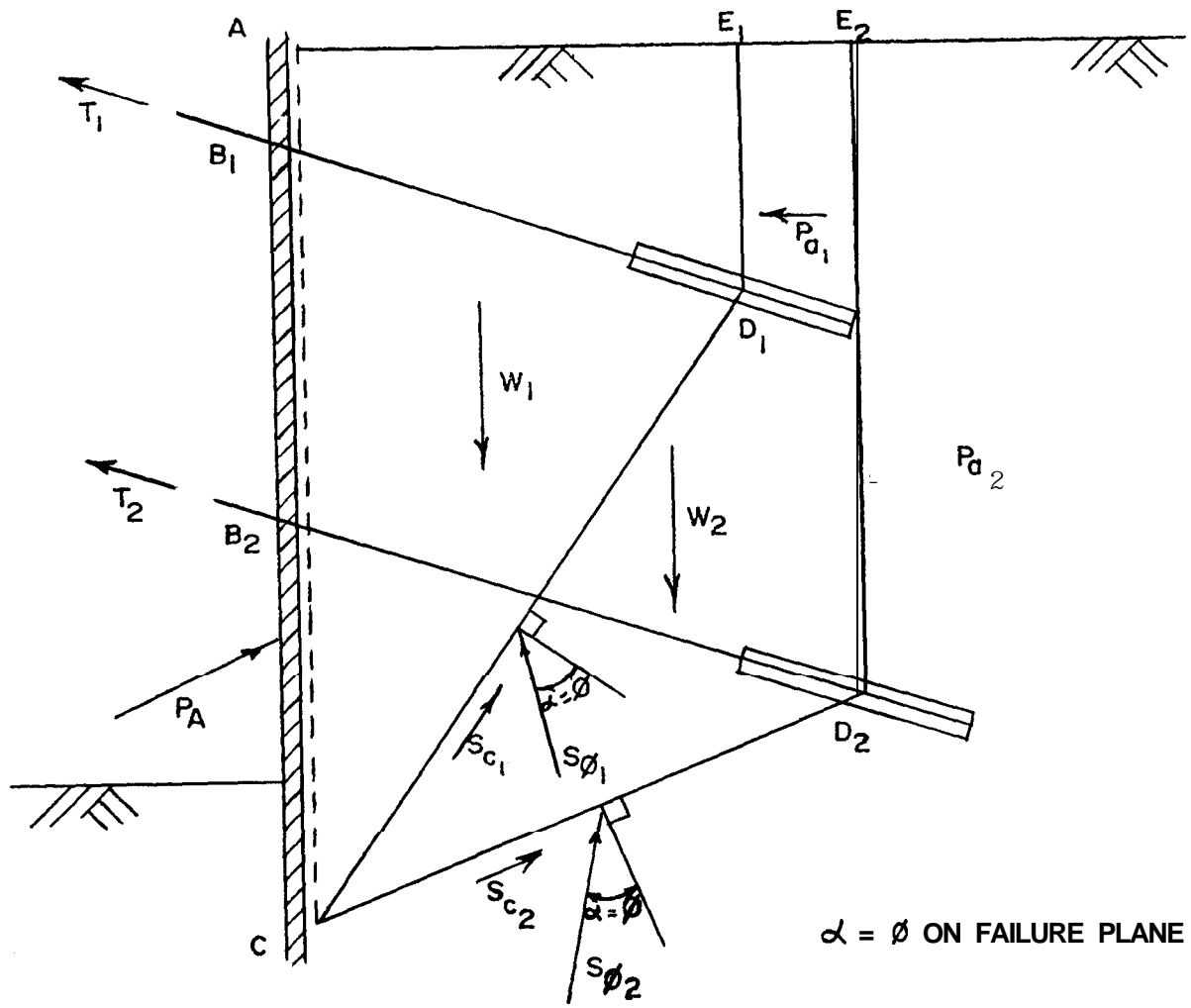
Figure 85. Free body diagram with forces acting on the bodies for the case of one independent anchor (safety in terms of the tieback force) (after Ranke and Ostermayer, 1968).
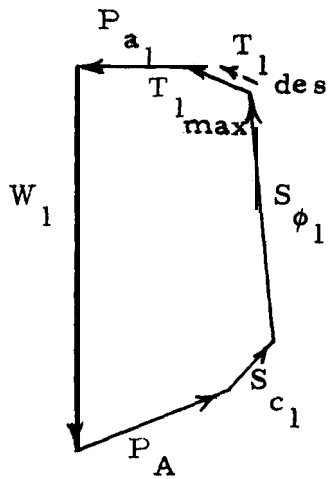
$$\text{F. S.} = \frac{T_{1_{max}}}{T_{1_{des}}} \geq 1.5$$

$\alpha \simeq \phi$ on failure plane.

$$\text{F.S.} = \frac{T_{1\text{-}2_{max}}}{T_{1_{des}} + T_{1_{des}}} \quad 1.5$$
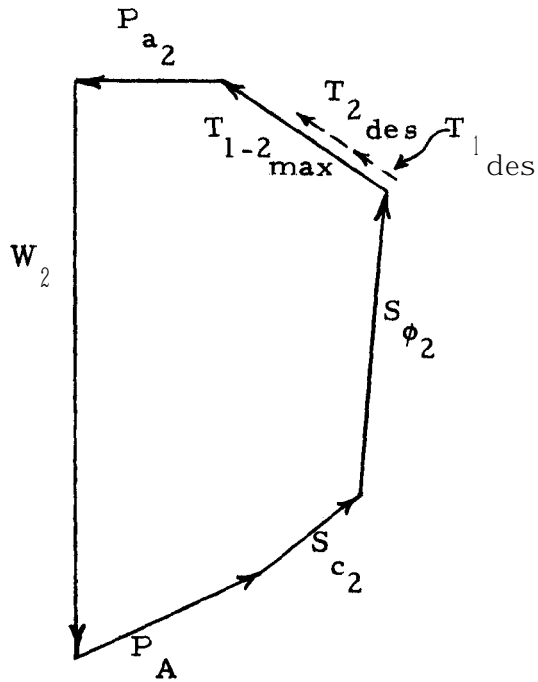
Figure 86. Vector diagrams used to evaluate the stability of case with one independent anchor (safety in terms of tieback force) (after Ranke and Ostermayer, 1968).
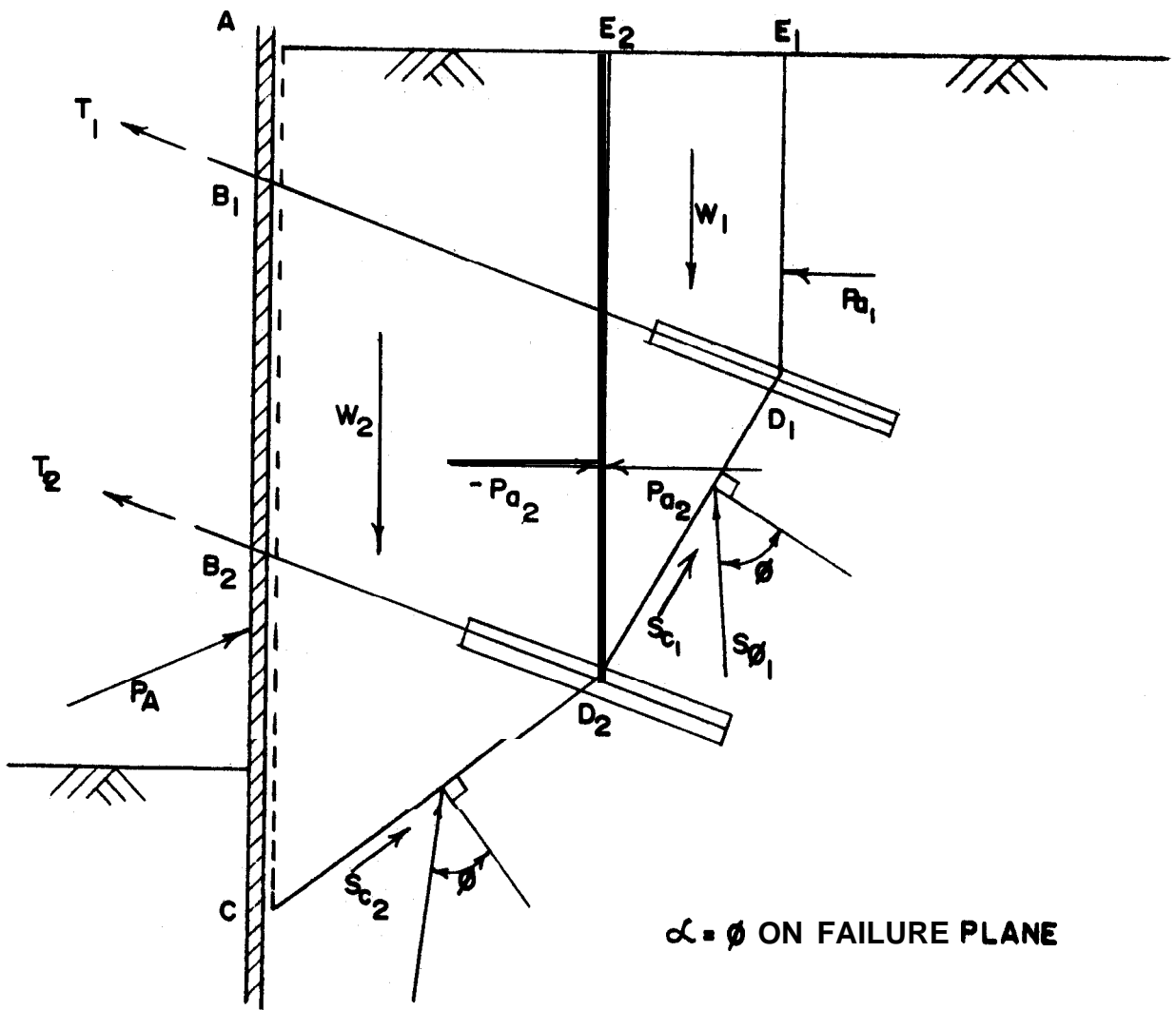
Figure 87. Free body diagram for anchor system with a complex failure surface (safety in terms of the tieback force).
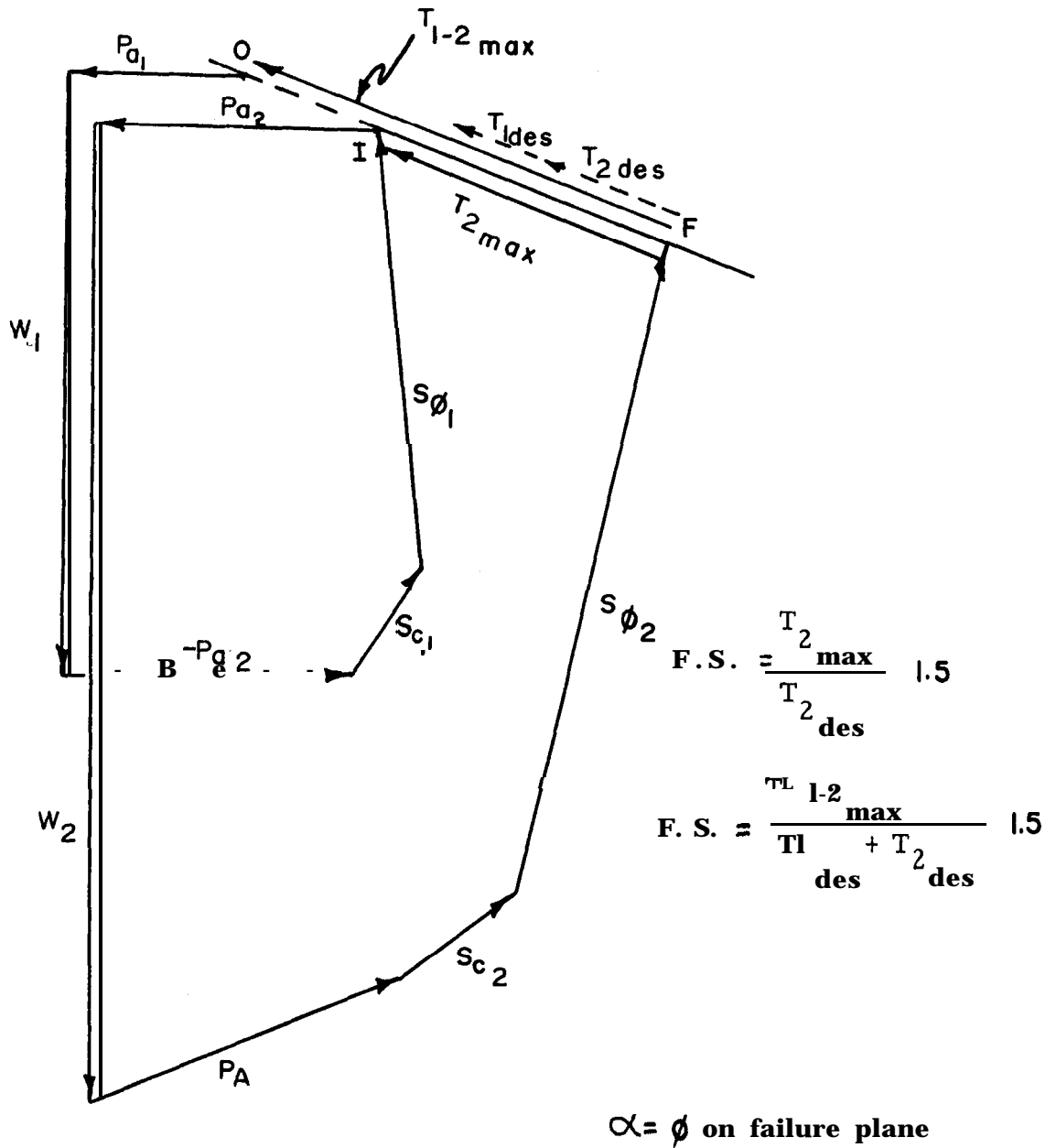
Figure 88. Vector diagram for a complex failure surface
(safety factor in terms of tieback force)
(after Kranz, 1953 and Ranke and Ostermayer, 1968).
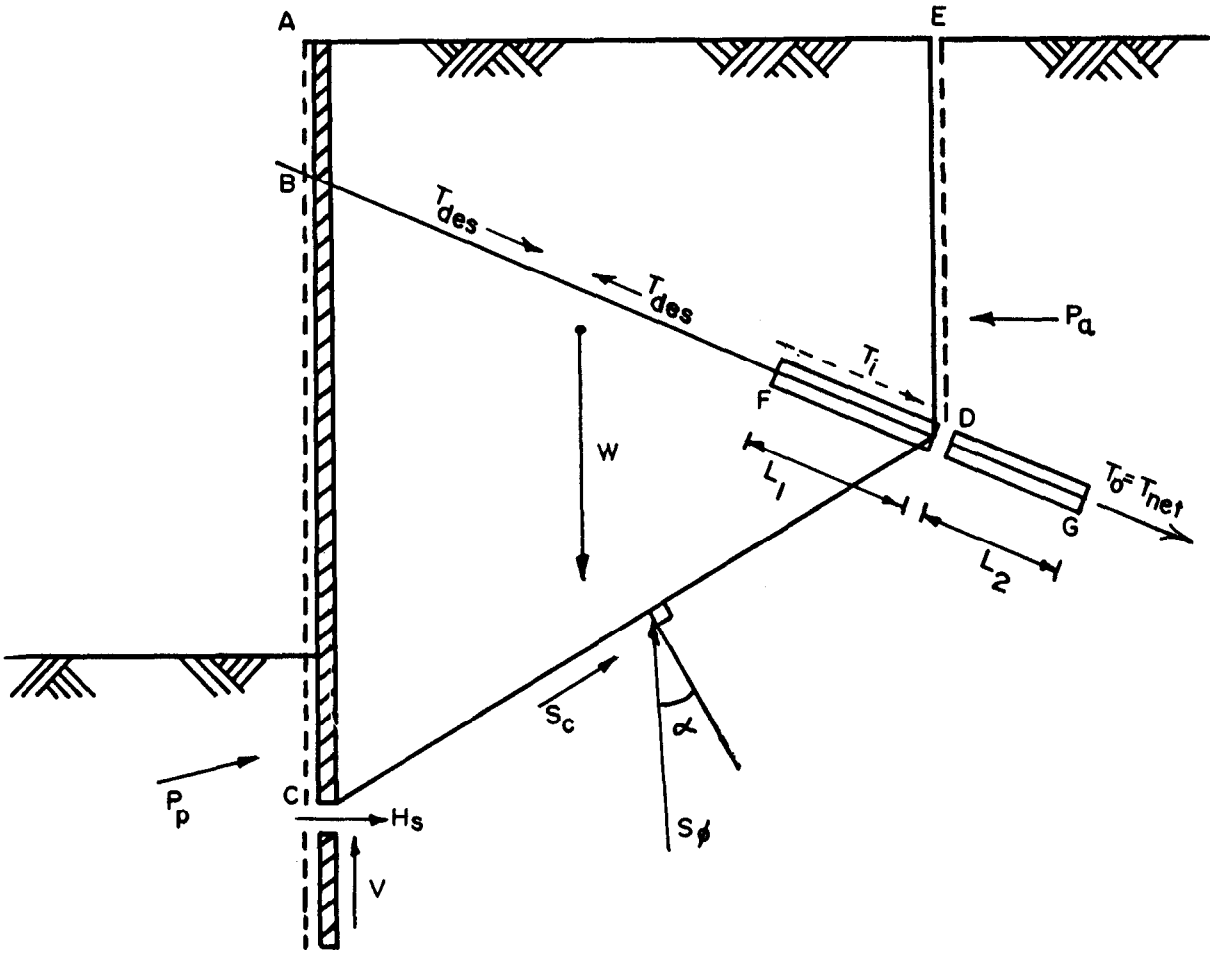
Figure 89. Free body diagram for a failure surface
in a single anchor tieback system
(free body outside of wall).

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. $= \dfrac{S_{avail}}{S_{mob}}$.

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. $= \dfrac{\bar{N} \tan \phi}{\bar{N} \tan \alpha} \approx \dfrac{\tan \phi}{\tan \alpha}$.

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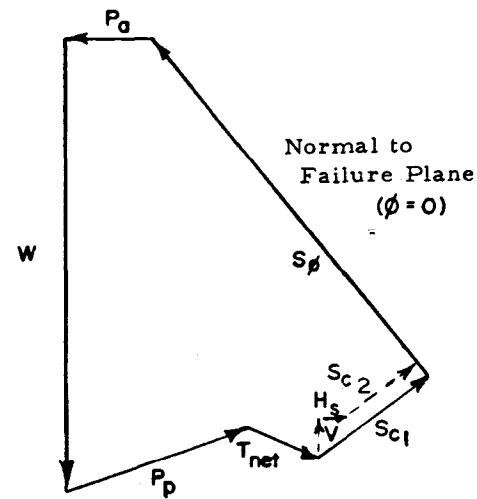
$$\text{F. S.} = \frac{S_c \text{ avail}}{S_c \text{ mob}} = \frac{S_u \times L}{S_{u \text{ mob}} \times L}$$

(a)

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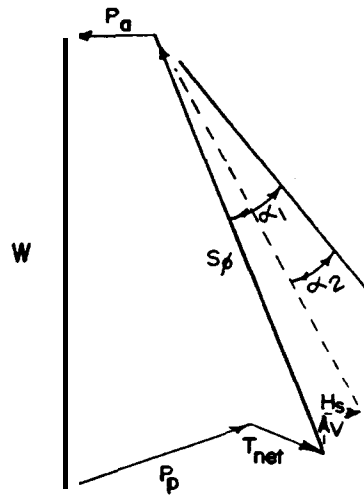
$$\text{F. S.} = \frac{\text{Shear \quad strength \quad available}}{\text{Shear \quad strength \quad mobilized}} = \frac{N \tan \phi}{N \tan \alpha}$$

(b)

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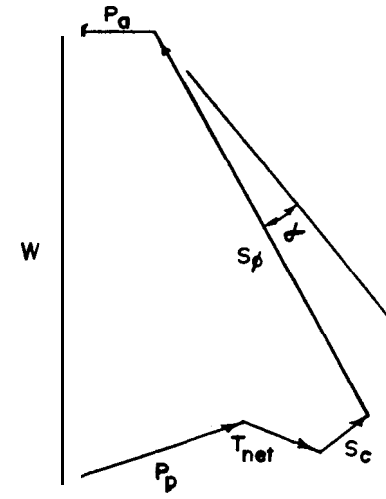
$$\text{F. S.} = \frac{S_u \times L}{S_{u \text{ mob}} \times L} = \frac{N \tan \phi}{N \tan \alpha}$$

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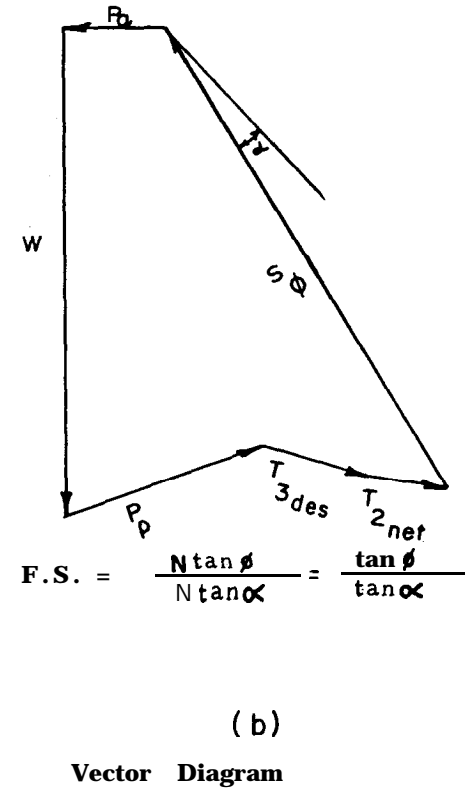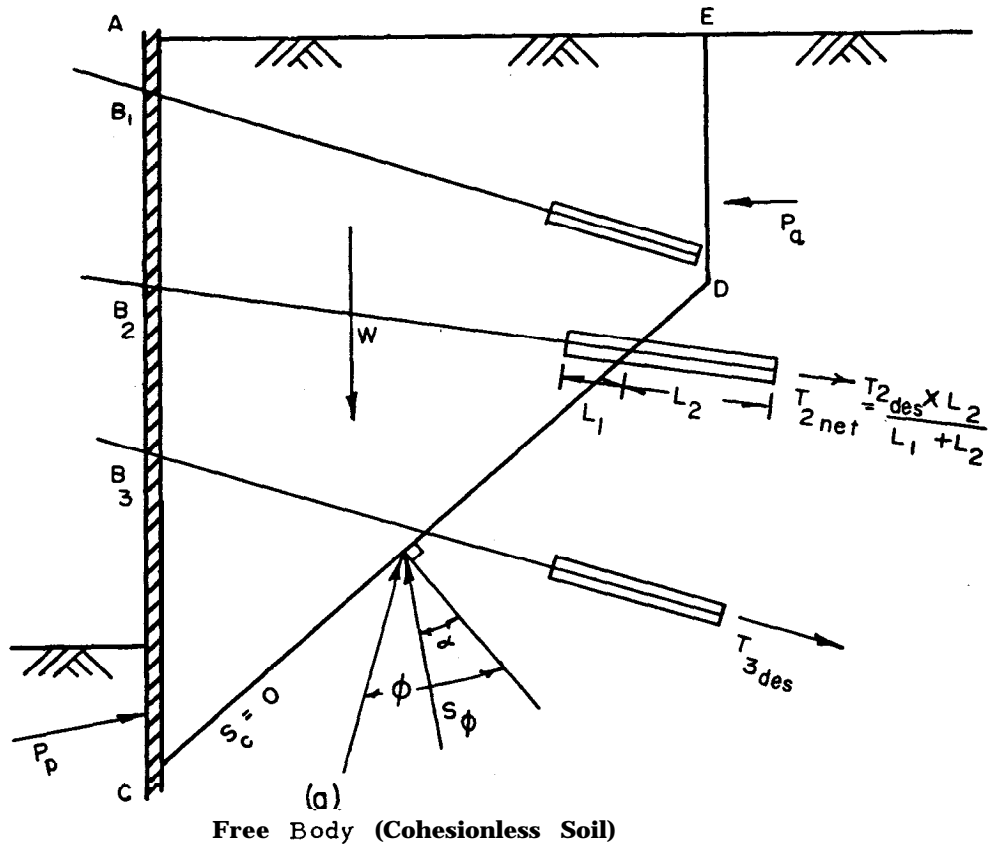
Figure 91. **Analysis of a multiple level anchor system (safety factor in terms of soil strength).**

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.
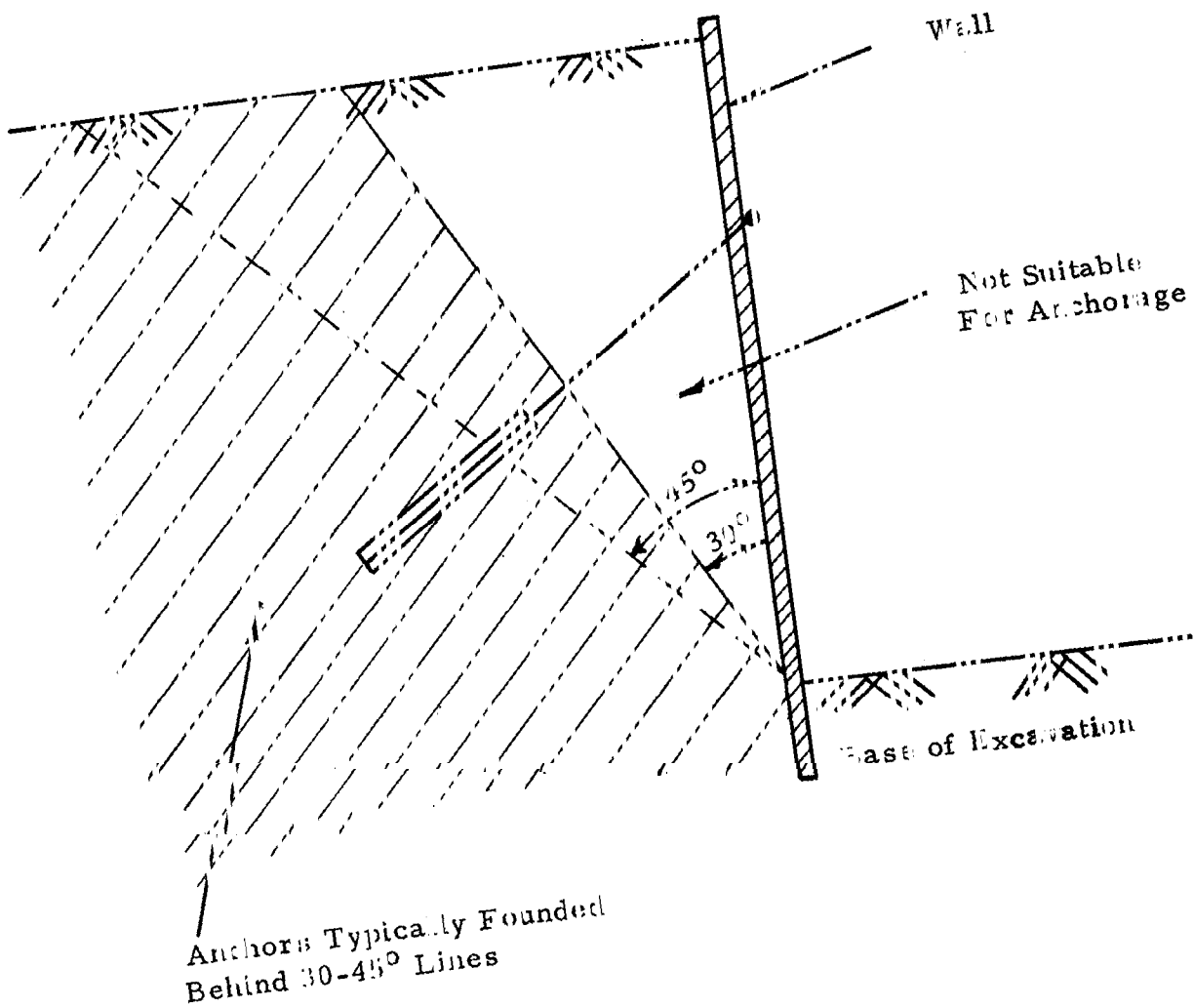
Figure 92. Typical location of anchors.

Labels in figure:
- Wall
- Not Suitable For Anchorage
- Base of Excavation
- 45°
- 30°
- Anchors Typically Founded Behind 30-45° Lines

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
. 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

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

where:

d_s = diameter of anchor shaft

L_s = length of anchor shaft

S_u = undrained shear strength of soil

$\propto$ = Reduction factor in S_u due to disturbance, etc. $\propto$ = 0.3 - 0.5 (Hanna, 1973a; Broms, 1968; Littlejohn, 1970a; Neely and Montague-Jones, 1974).

b. Belled Anchor

$$P_u = \propto S_u \pi d_s L_s + \pi/4 (D^2 - d_s{}^2) N_c S_u \quad \text{(Littlejohn, 1970a)}$$

where:

d_s, L_s, S_u and $\propto$ are as before

D = diameter of anchor bell

N_c = bearing capacity factor = 9
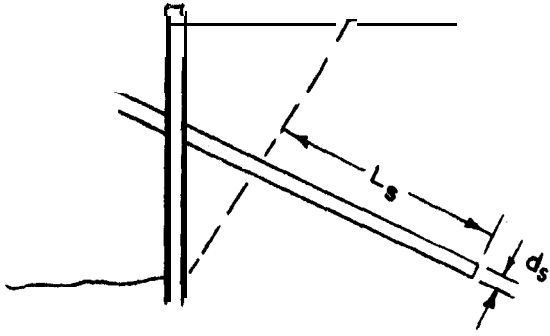
ESTIMATED LOAD FOR ANCHORS IN COHESIVE SOIL

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

$$\alpha = 0.3\text{-}0.5$$

(a)

Friction Anchor

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

$$\alpha = 0.3\text{-}0.5$$

$$N_c = 9$$

(b)
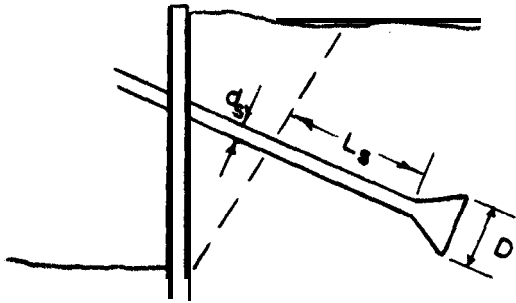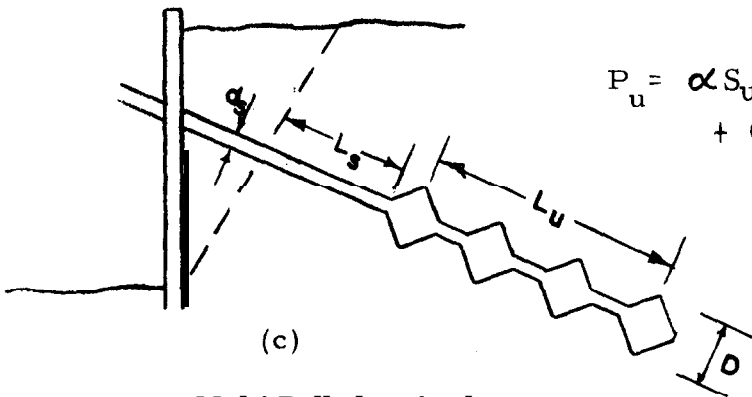
Belled Anchor

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

$$\alpha = 0.3\text{-}0.5$$
$$\theta = 0.75\text{-}1.00$$
$$N_c = 9$$

(c)

Multi-Belled Anchor

Figure 93. Schematic representation of large diameter anchors'

c. Multi-belled Anchor

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

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

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

where:

d_s = diameter of anchor shaft

L_s = length of anchor shaft

ϕ_e = effective friction angle between soil and grout

P_i = grout pressure

or $P_u = L_s n_1 \tan \phi_e$ (Littlejohn, 1970a)

where:

n_1 = 8.7 - 11.1 k/ft (127 - 162 kN/m)
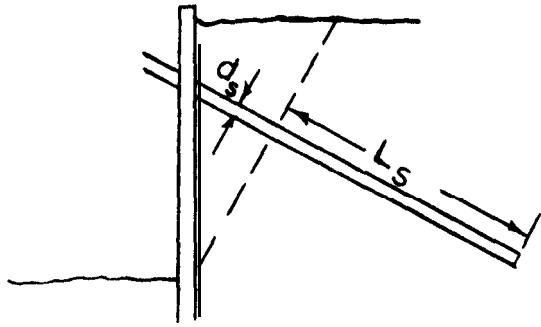
(a)

**Friction Anchor
(No Grout Penetration)**
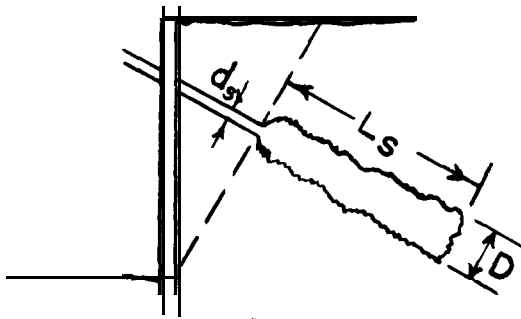
(b)

**Bulb Anchor
(Grout Penetration)**

(c)
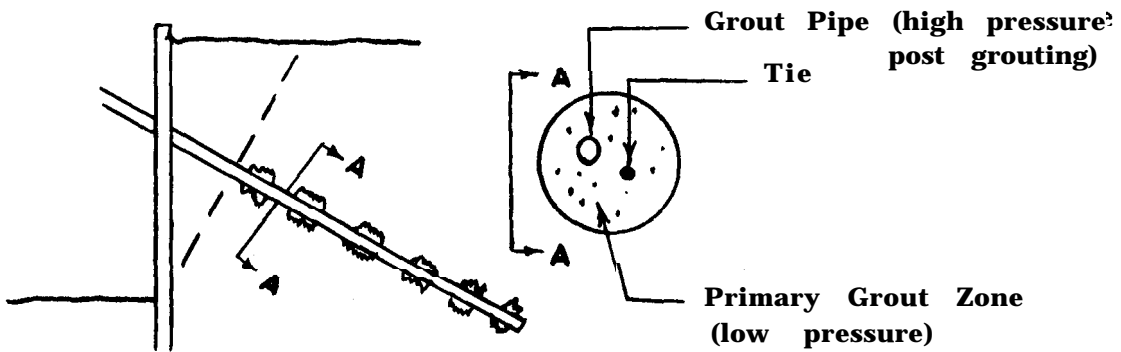
**Regroutable Anchor
(Local Grout Penetration)**

**Grout Pipe (high pressure
post grouting)**

Tie

**Primary Grout Zone
(low pressure)**

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

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

B = bearing capacity factor similar to N but smaller in magnitude. A value of ^{q}

$B = \dfrac{N_q}{1.3 - 1.4}$ 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.
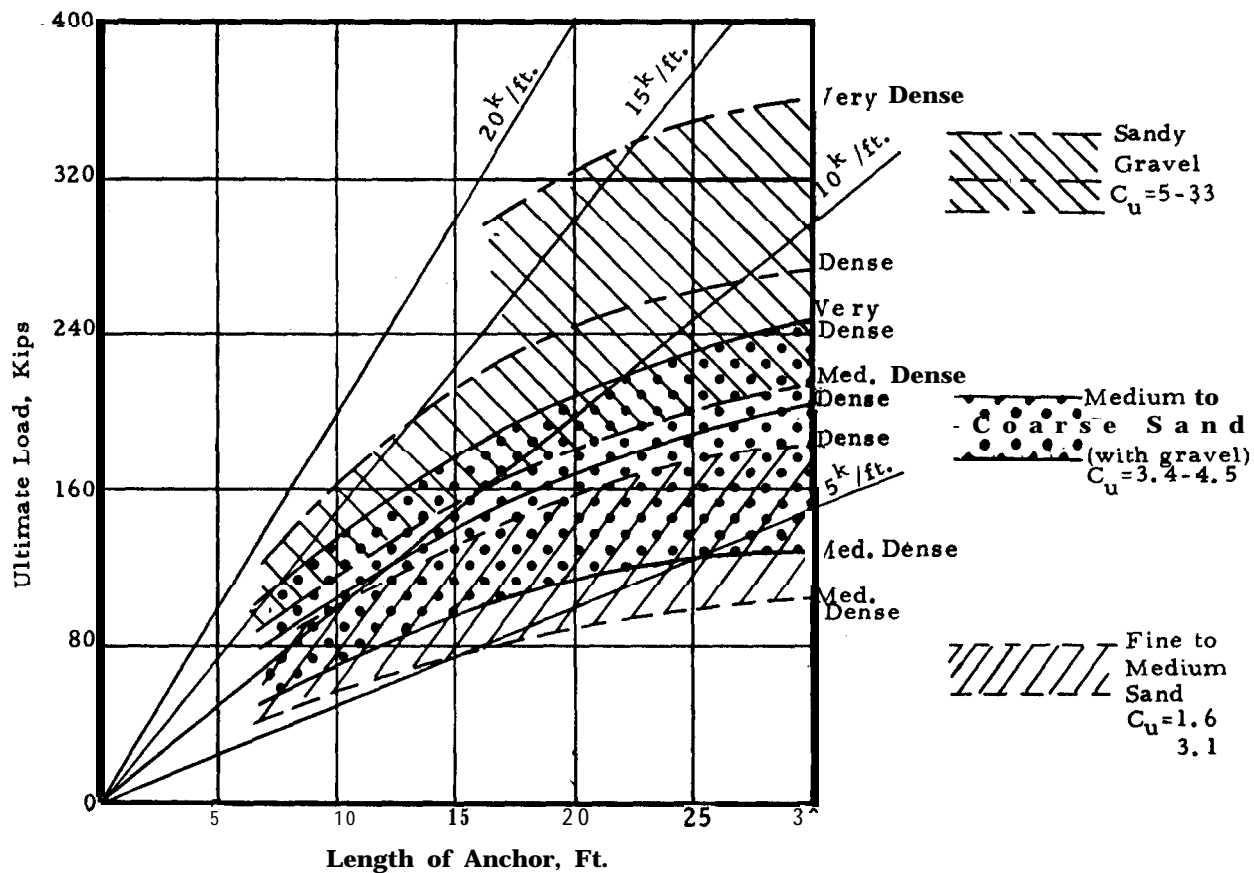
Figure 95. Load capacity of anchors in cohesionless
soil showing effects of relative density,
gradation, uniformity, and anchor length
(after Ostermayer, 1974).

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 (19 69) reported an improvement of anchor load capacity in both cohesionless and cohesive soils with a regrout-able 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 regrouta-ble 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
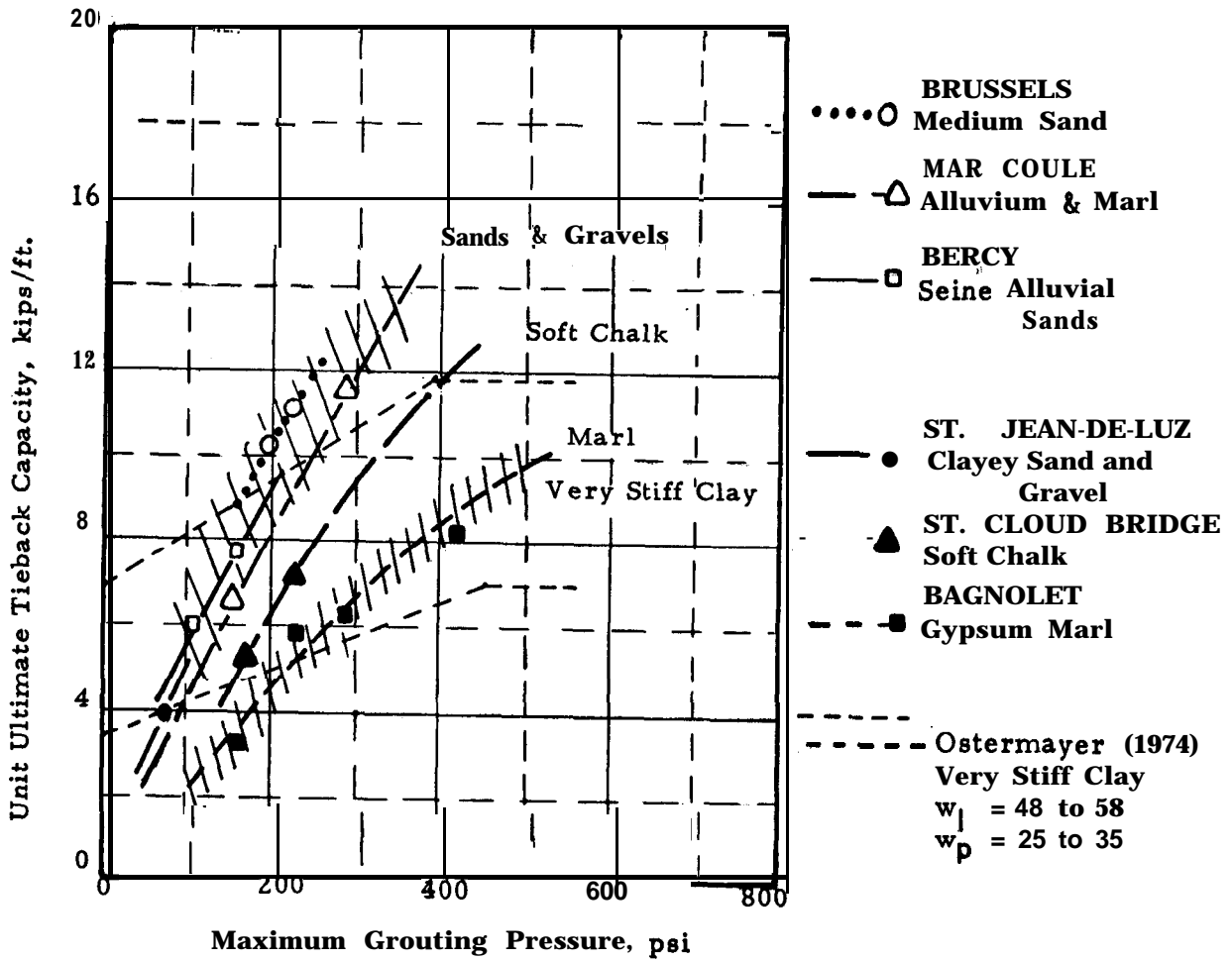
Figure 96. Ultimate anchor capacity as a function of grout pi-essure.

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

	Typical Skin Friction (per square foot of grouted zone)	
Soil Type	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

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° to 90° although in badly fissured or jointed rock the design criteria may be significantly different. If the weight of the rock within the contained cone is greater than the design anchor load, 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:

$$P_u = \pi \, d_s \, L_s \, \text{'skin}$$

where:

d_s = diameter of anchor shaft

L_s = length of anchor shaft

δ_{skin} = grout-rock bond strength

The values of skin friction, δ_{skin}, for various rock types are summarized in Table 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.

$\theta = 60^o - 90^o$

I? = Apex of Cone
(Varies from Midpoint
to Base of Anchor)

From Littlejohn (1975)

$P_u = \gamma \times$ VOLUME OF ROCK IN CONE
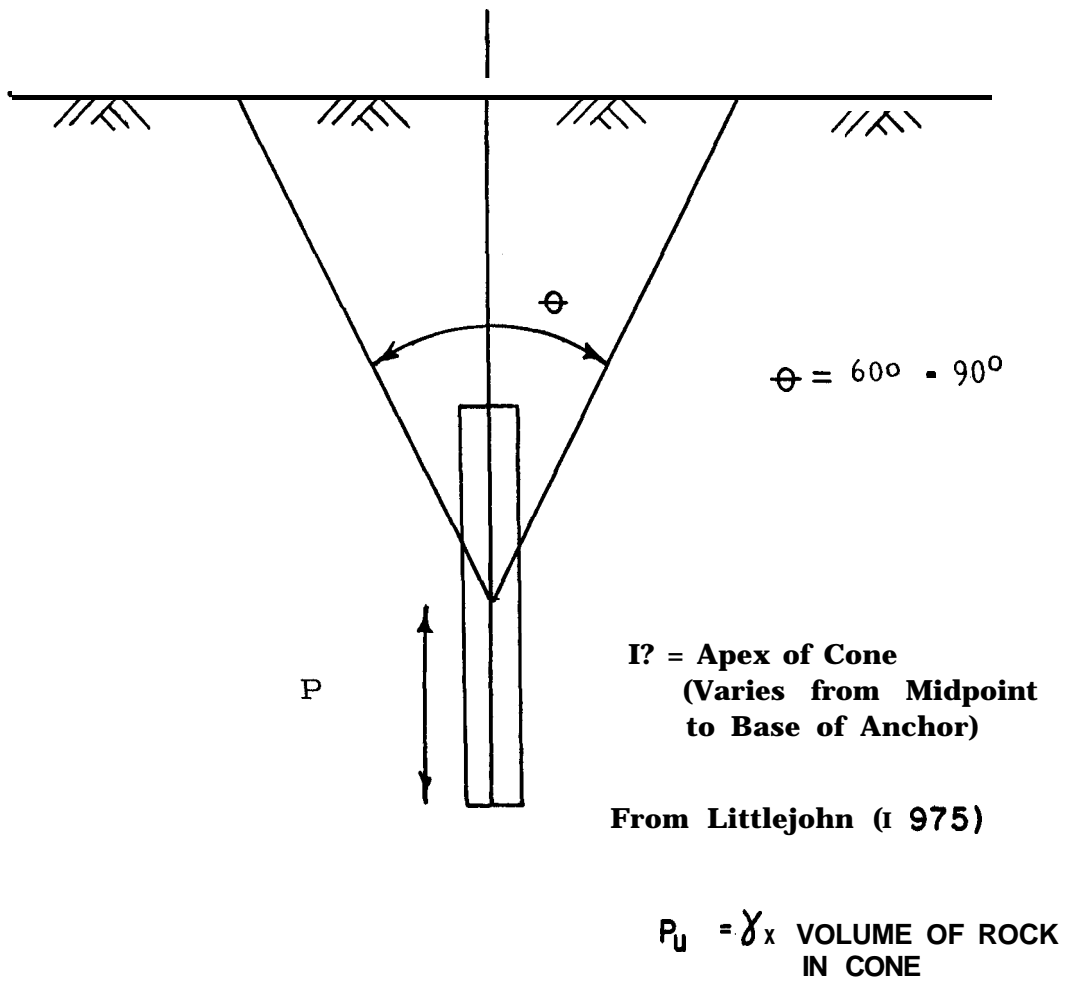
Figure 98.   Schematic drawing of design quantities for
failure in a rock mass.

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})[7]
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.

No Backfill Within 1 ' of Wall,

Wall

Sand or Lean Cement Grout

1'

1'

A

A

Tic Member

Plastic

Sheathing

Plastic
Sheathing
or Building
Paper

Tie Member

Sand or Lean Cement
Grout (Backfill)
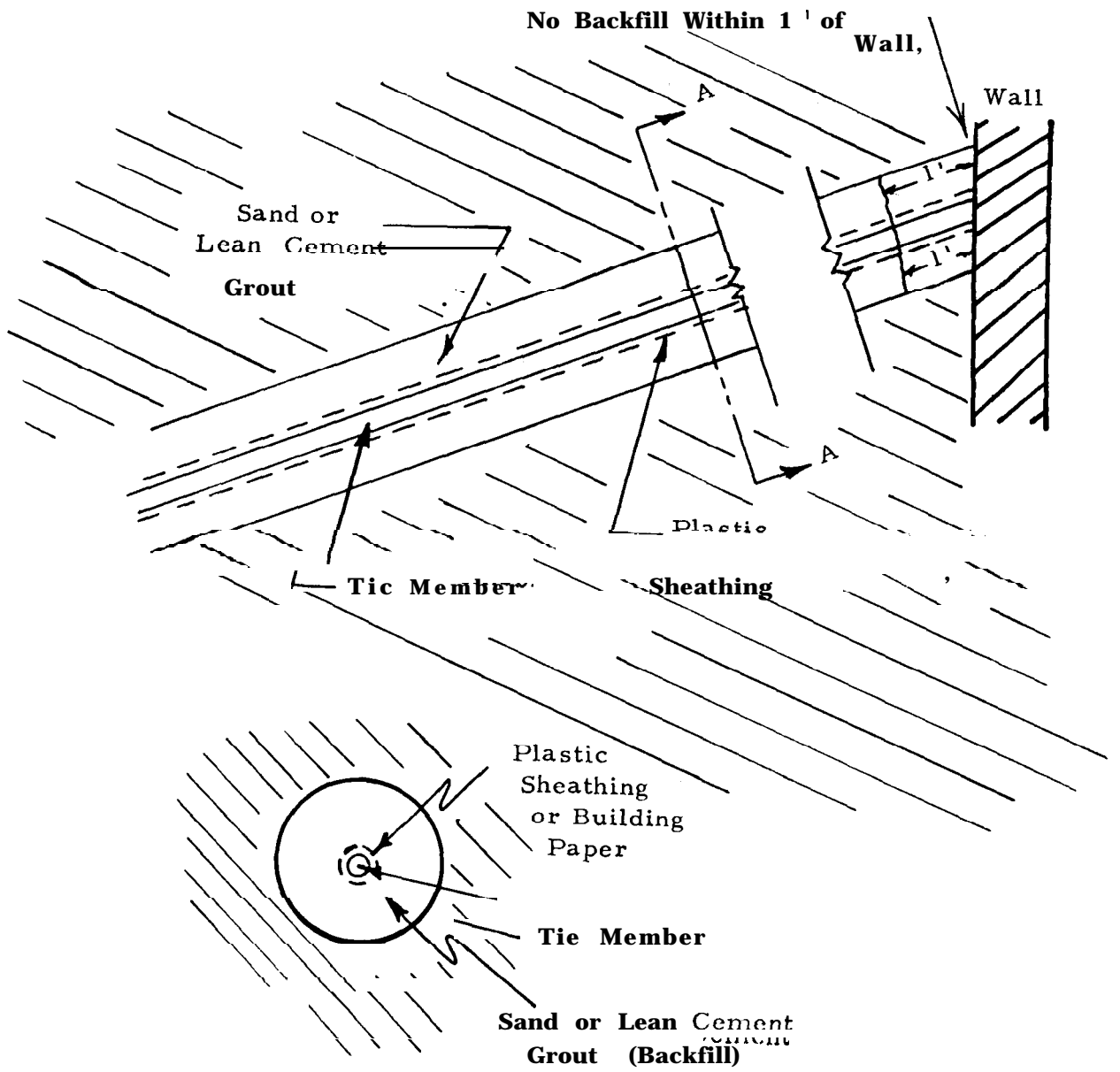
Figure 99. Recommended treatment for bond free zone.

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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/mm²)	80	70	55	55
Cable or Strand	270 (1.86 kN/mm²)	85	75	60	60
Bar or Rod	130 - 230 (0.897 - 1.59 kN/mm²)	85	75	60	60

[1]Maximum Design Stress, f_d, is equal to $\dfrac{f_t}{1.25}$ 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 cra cking 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:
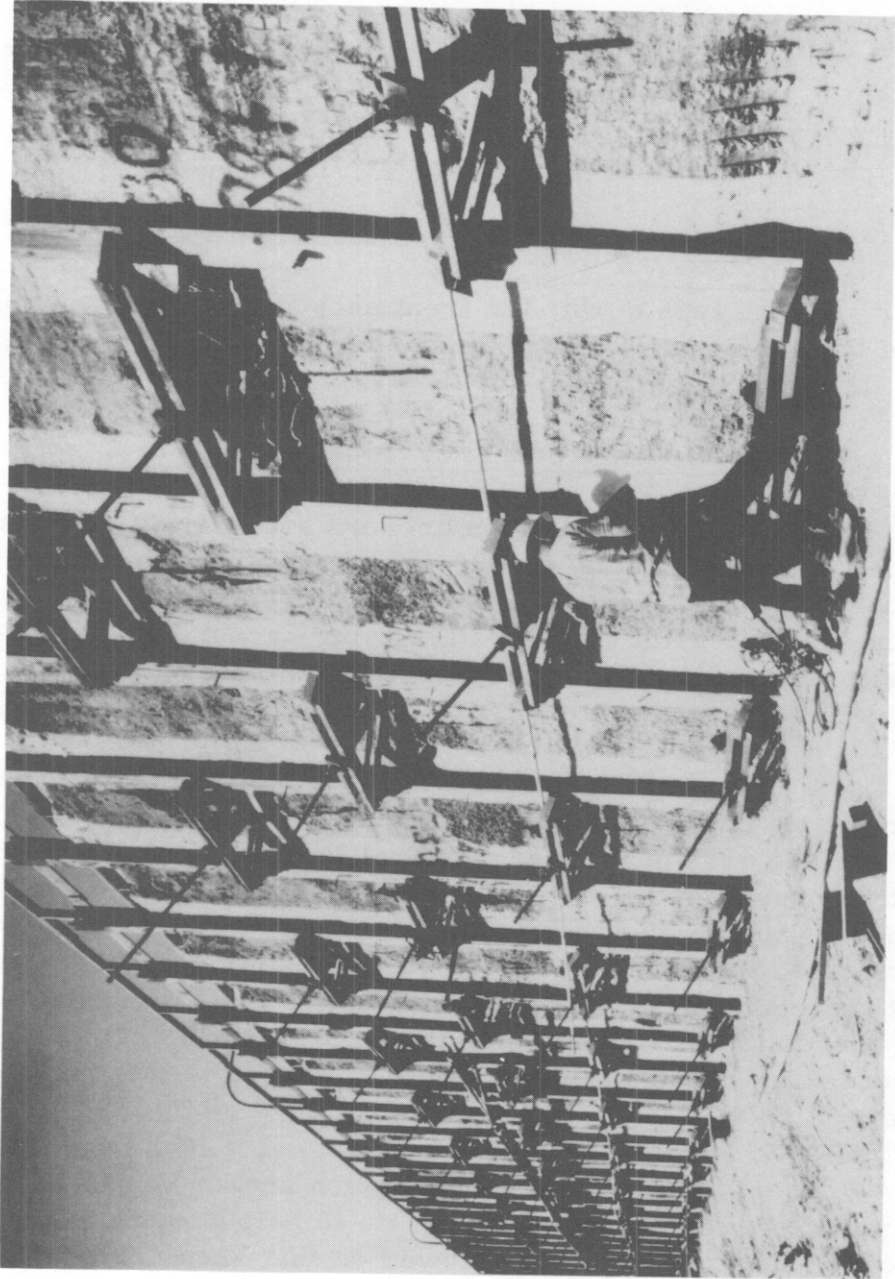
Figure 100. Example of tied-back wall using channel sections as wales. (Courtesy of Hughes Tool Company).

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
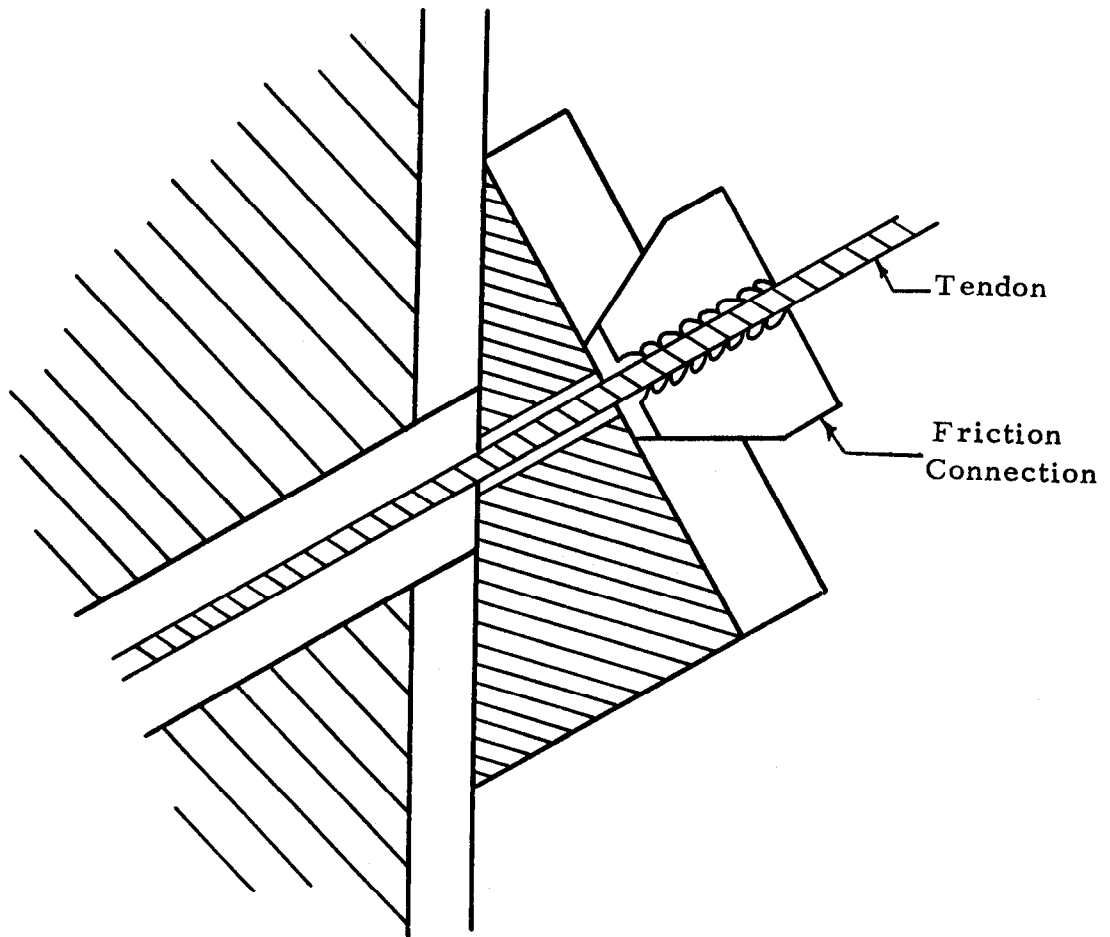
The major reasons for field testing are:

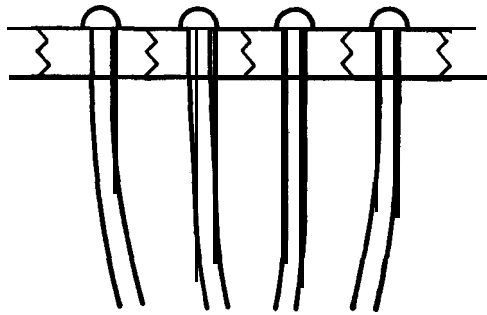Figure 101. Friction connection used to tie anchor to wall.

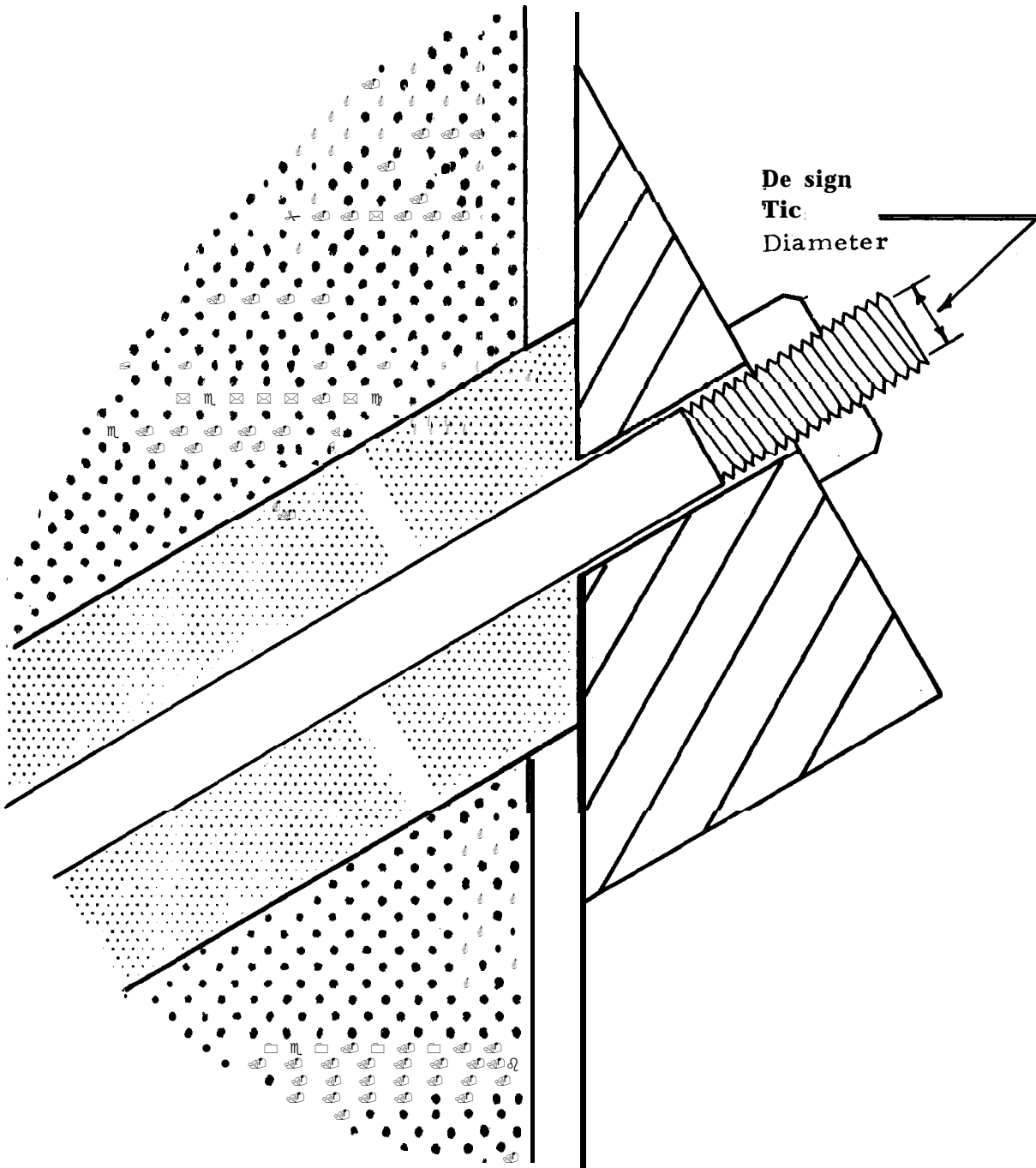Figure 102. Button-head connection for wire ties.

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° ▪ 90° (better at shallow angles)	Lean concrete or sand backfill. Plastic sheathing.
(2, Hollow stem **Augers**	Preferred in competent cohesive soils. Often used in sandy soils.	Truck-mounted crawler-mounted or crane-supported with guides.	6"- 18" (15 **cm-**45 cm) 12"- 14" most c **ommon**	Reported to 160'. (50 m)	High strength concrete pumped water **pressure** through hollow stem -150 psi or less. (1035 kN/m^2)	No spacer **necessary** since hollow stem **serves** as guide. Points are generally used in anchor.	0° ▪ 90° (proprietary methods may not be able to achieve lower angles)	Lean concrete or sand backfill. Plastic sheathing.
Belled Anchor	Competent cohesive soils which can remain open unsupported.	Truck-m&ted crawler-mounted or crane-supported augers with guides. Belling equip-**ment** same as used for caisson work.	12"-24" (30cm ▪ 60cm) Shaft 30"-42" (75cm ▪ 105cm) B e l l	Typical length to bell of approximately 50' **(15m).** Lengths up to 100'± **(30m)** in California,	Pumped concrete.	Spacers used to center ties. Plates or washers usually aid load transfer.	Generally installed at angle (30° ▪ 60°)	Lean concrete or sand backfill. Plastic sheathing.
Multi- Under-reamed Anchor (Multi-B e l l)	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) Under-reams $l_u \geq 1.5d_s$	Total lengths in excess of 50' **(15m).** Spacing between bells approximately 1.5 ▪ 2.0 **x** diameter bell.	cement grout or concrete. [Concrete for larger diameter anchors]	Spacers used to center ties. Plate used in some methods to transfer entire load.	Generally installed at angle (30° ▪ 60°)	Lean concrete, weak cement grout, **or** sand. Entire tie length except **for** plate is un-bonded 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 Regroutable) (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-Underreamed	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.
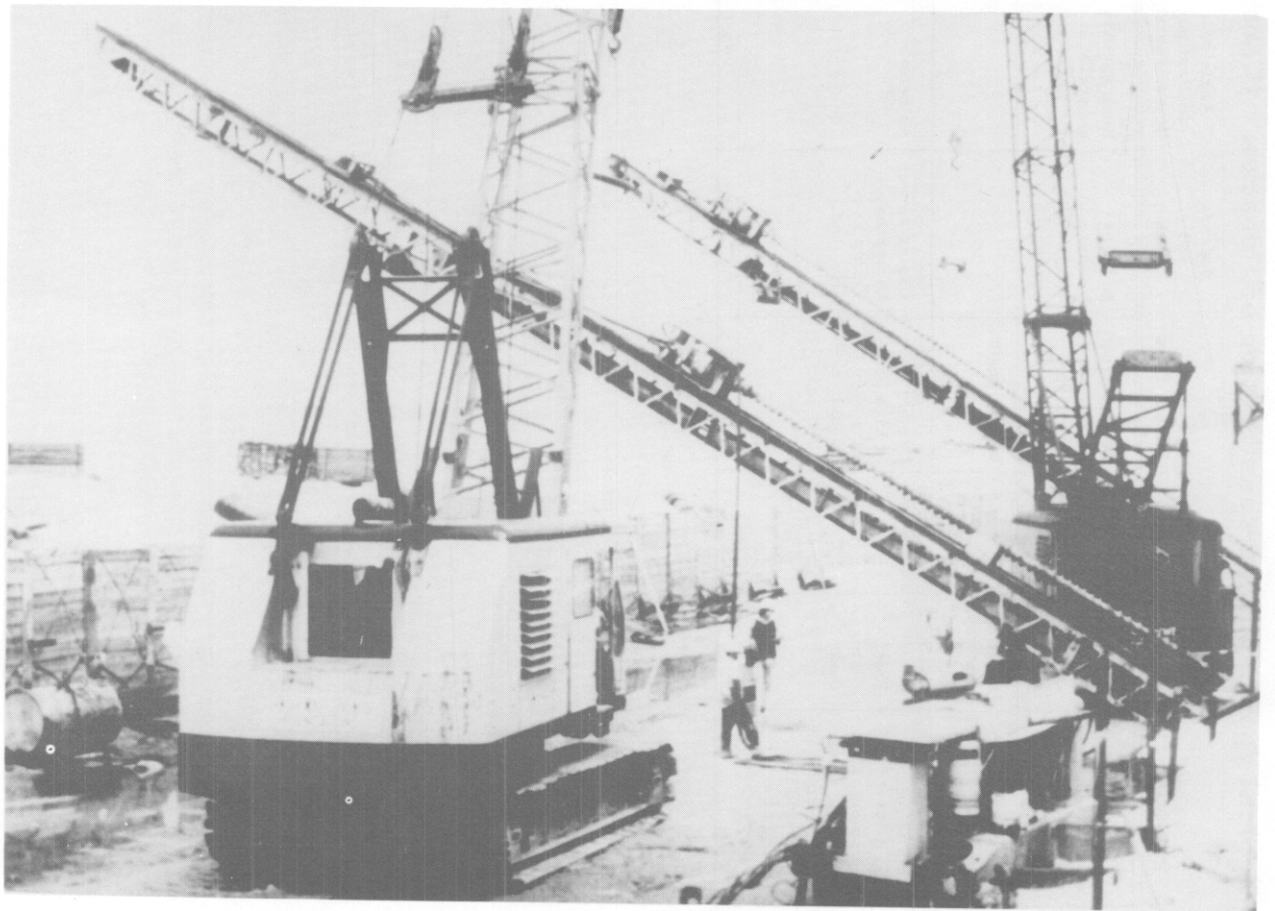
Figure 104. Crane suspended auger rig.
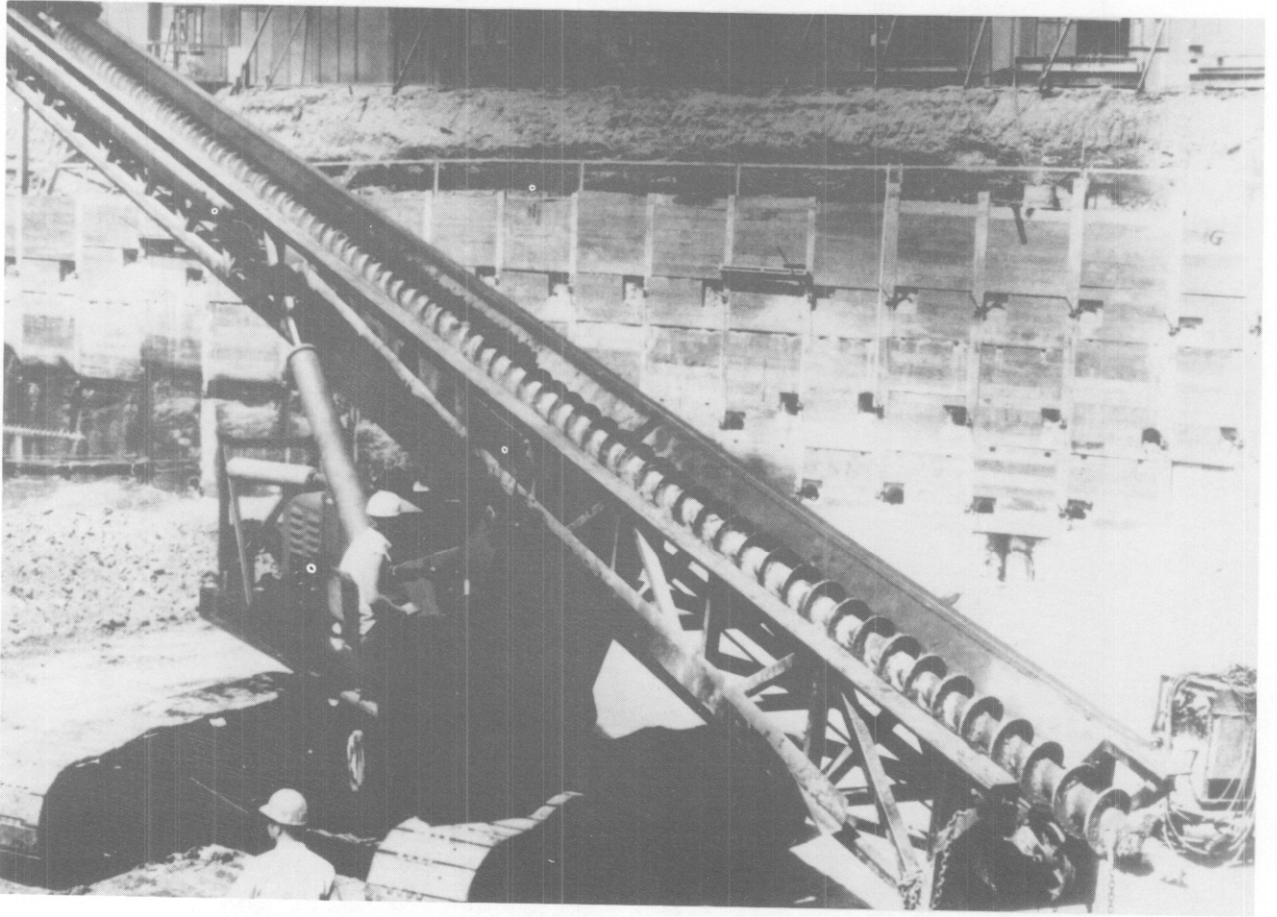(Courtesy of Spencer, White, and Prentis).

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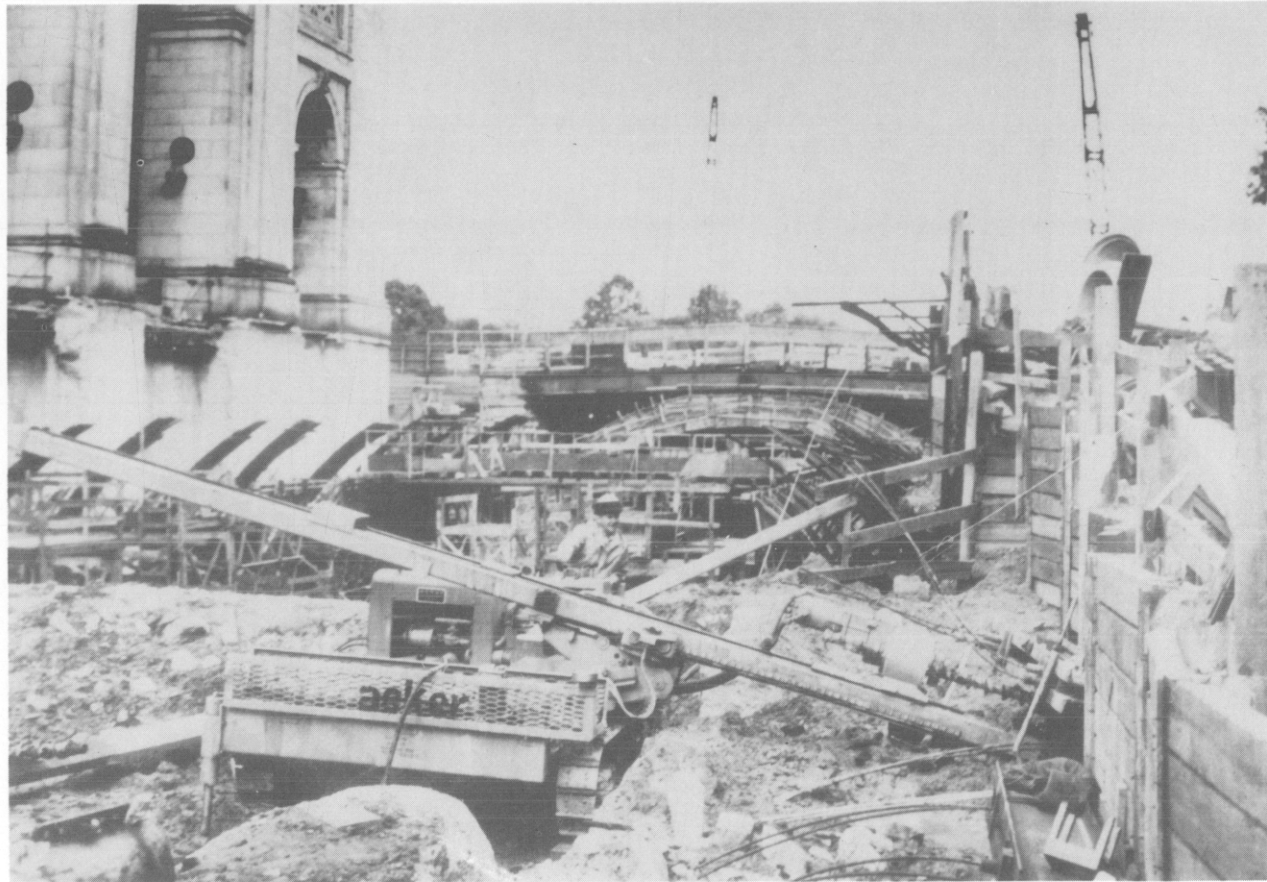(Courtesy of Spencer, White, and Prentis).

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 forma-tion should be tested to 200% of design.'
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).
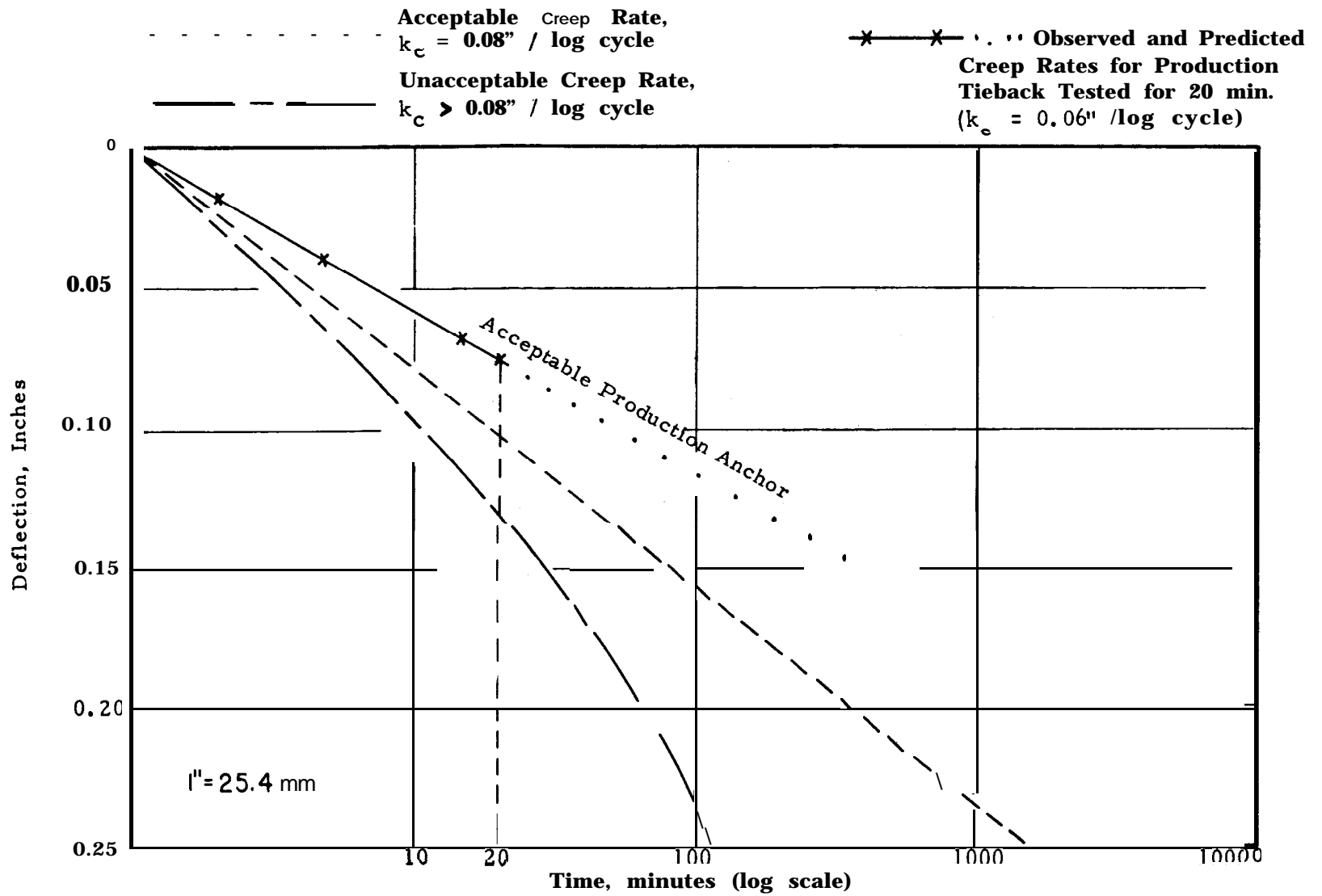
Figure 108. Example of recommended method of recording creep data.

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. $l_{eff} \leq l_g$).

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.
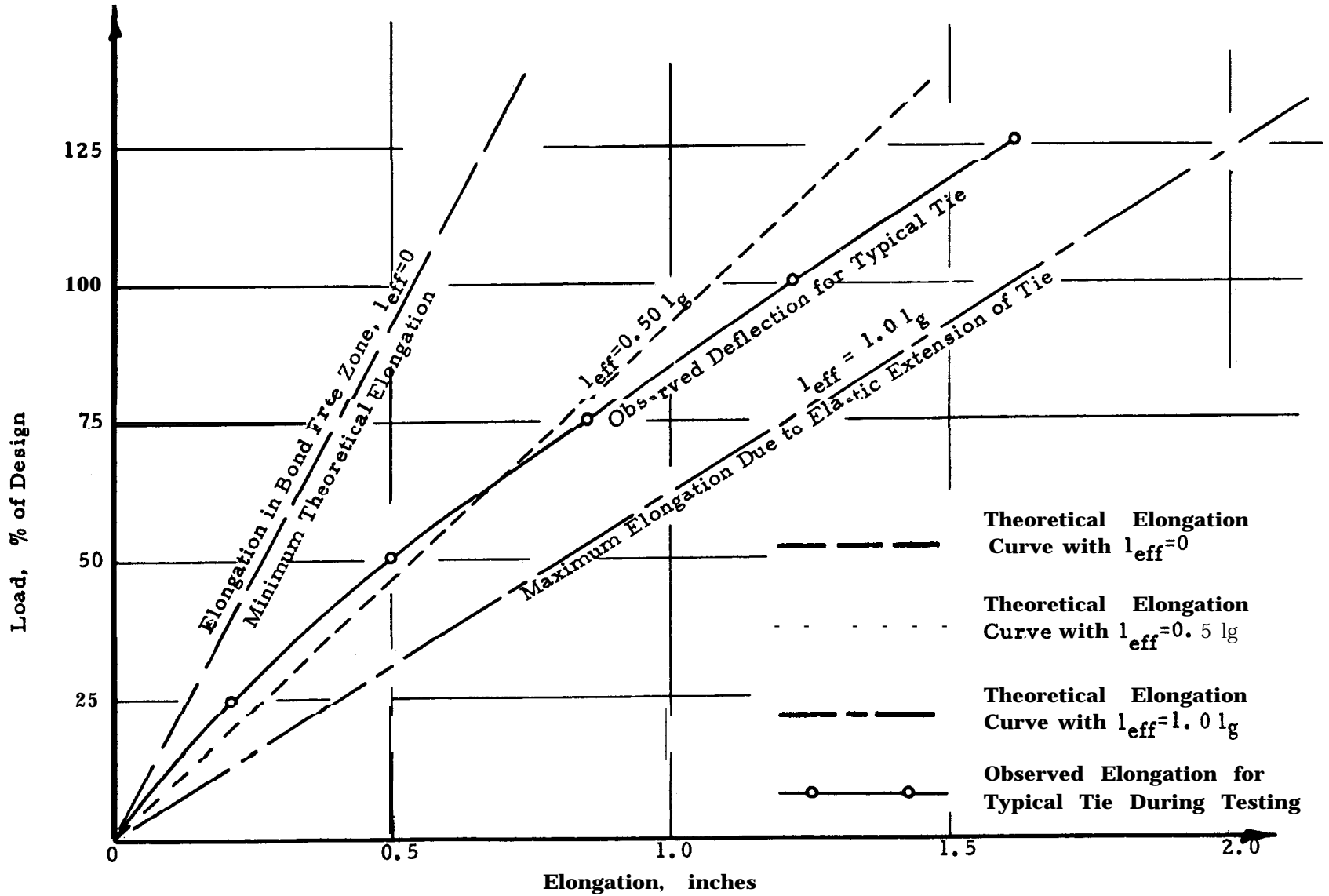
Figure 109. Typical plot of load vs. elongation during test loading.
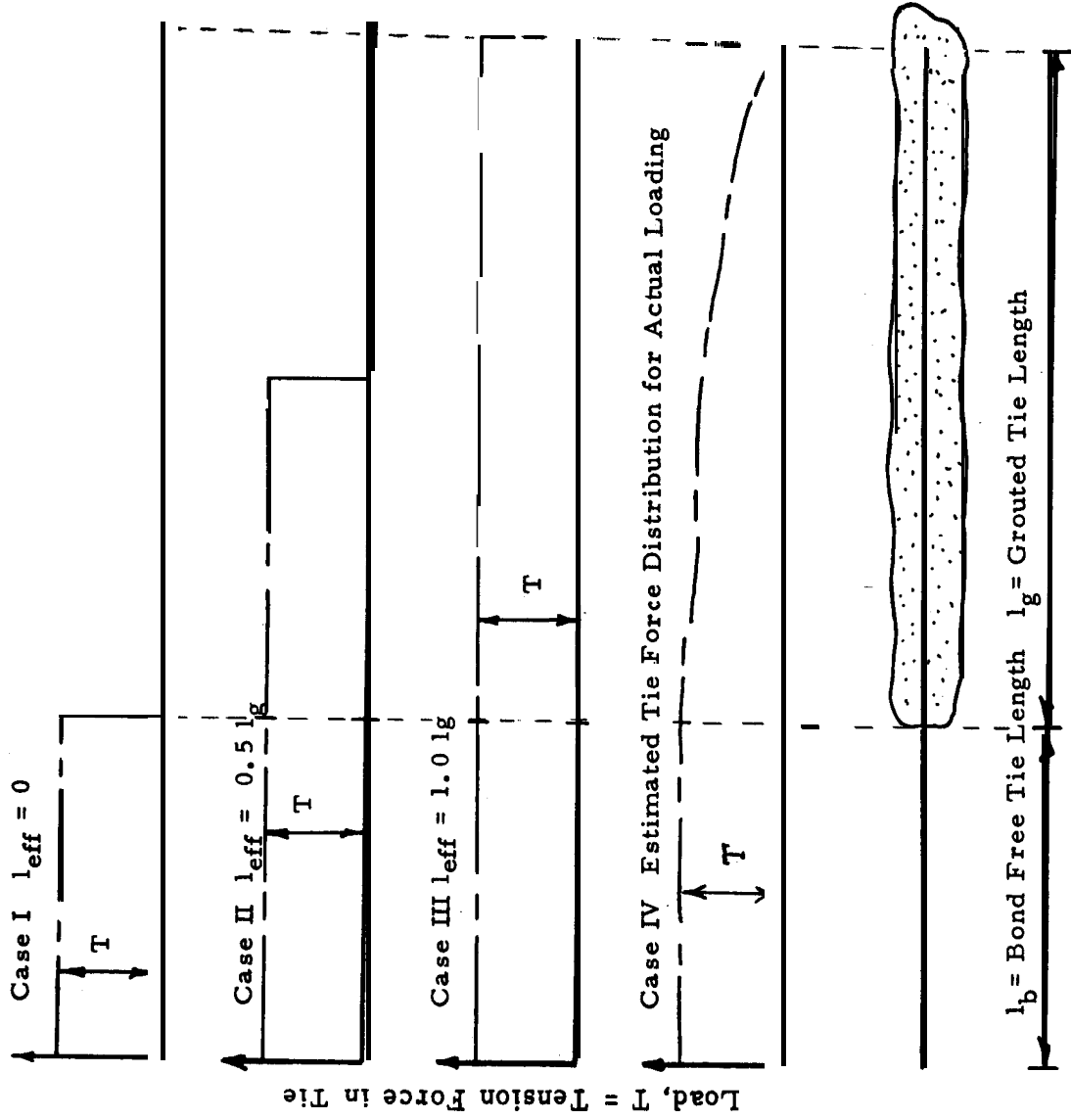
Figure 110. Idealized load distribution in tie.

See Figure 109.

See Figure 109.

See Figure 109.

See Figure 109.

Case I $l_{eff} = 0$

T

Case II $l_{eff} = 0.5\ l_g$

T

Case III $l_{eff} = 1.0\ l_g$

T

Case IV Estimated Tie Force Distribution for Actual Loading

T

T

Load, T = Tension Force in Tie

l_b = Bond Free Tie Length l_g = Grouted Tie Length

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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 under-pinning is a concrete filled pit or pier which has been excavated using horizontal wood sheeting to retain the earth. The construction pro-cedures 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

OPEN PIT

4'

ELEVATION

3'

4'

PLAN

(a)

SHEET PIT

2x4 CLEATS

ELEVATION

PLAN

(b)

SHEETED PIT UNDER FOOTING

ELEVATION

PLAN

(c)

ELEVATION

(d)

2"-3"

DRYPACK OR PLATES AND WEDGES
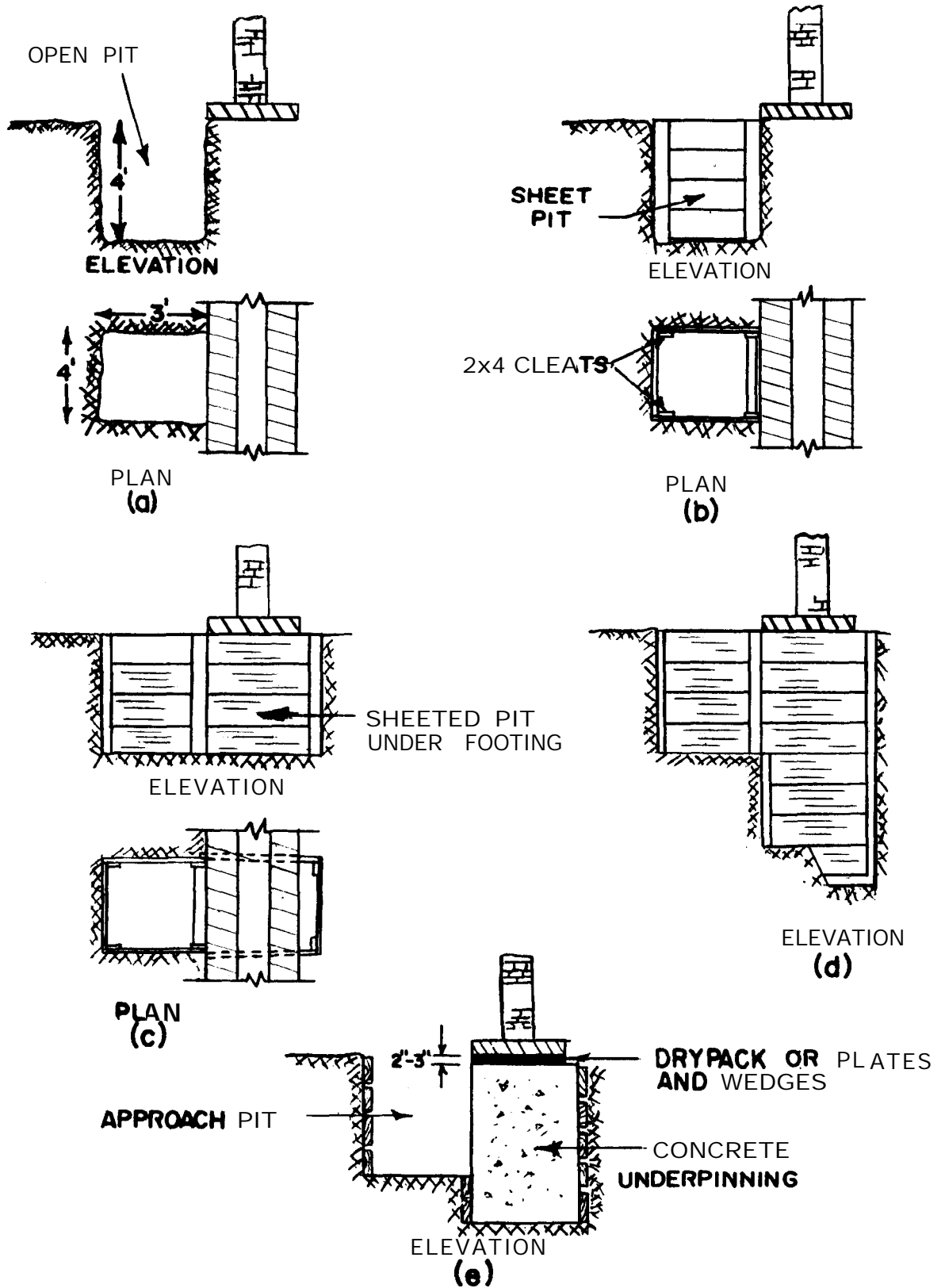
APPROACH PIT

CONCRETE UNDERPINNING

ELEVATION

(e)

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 into an 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
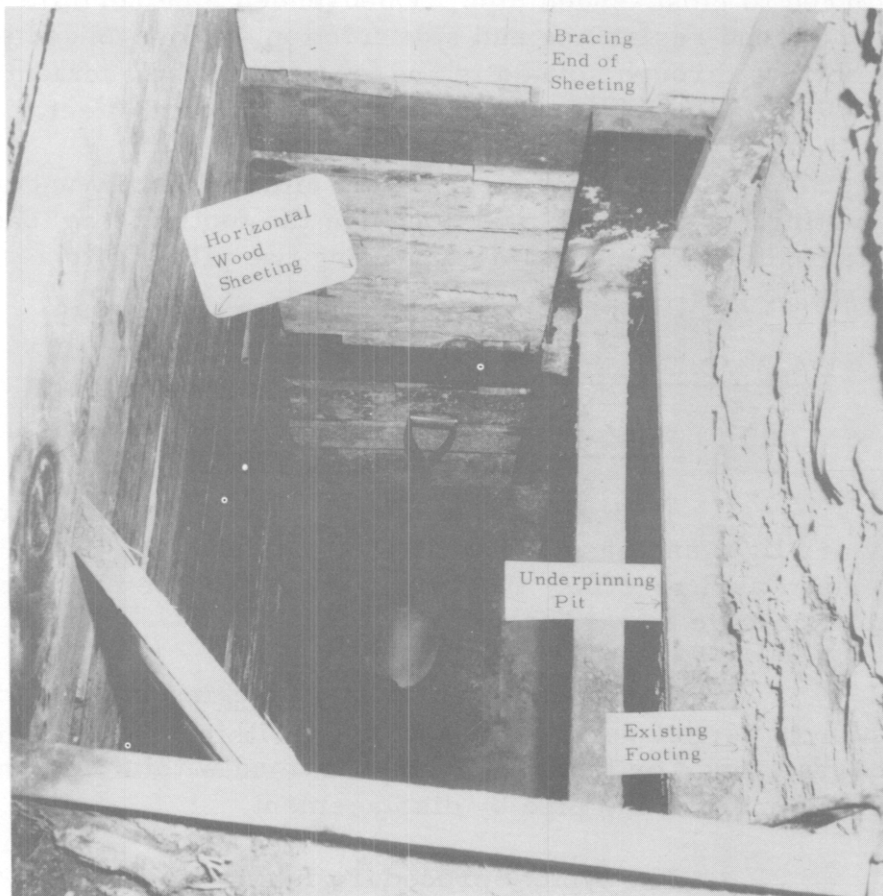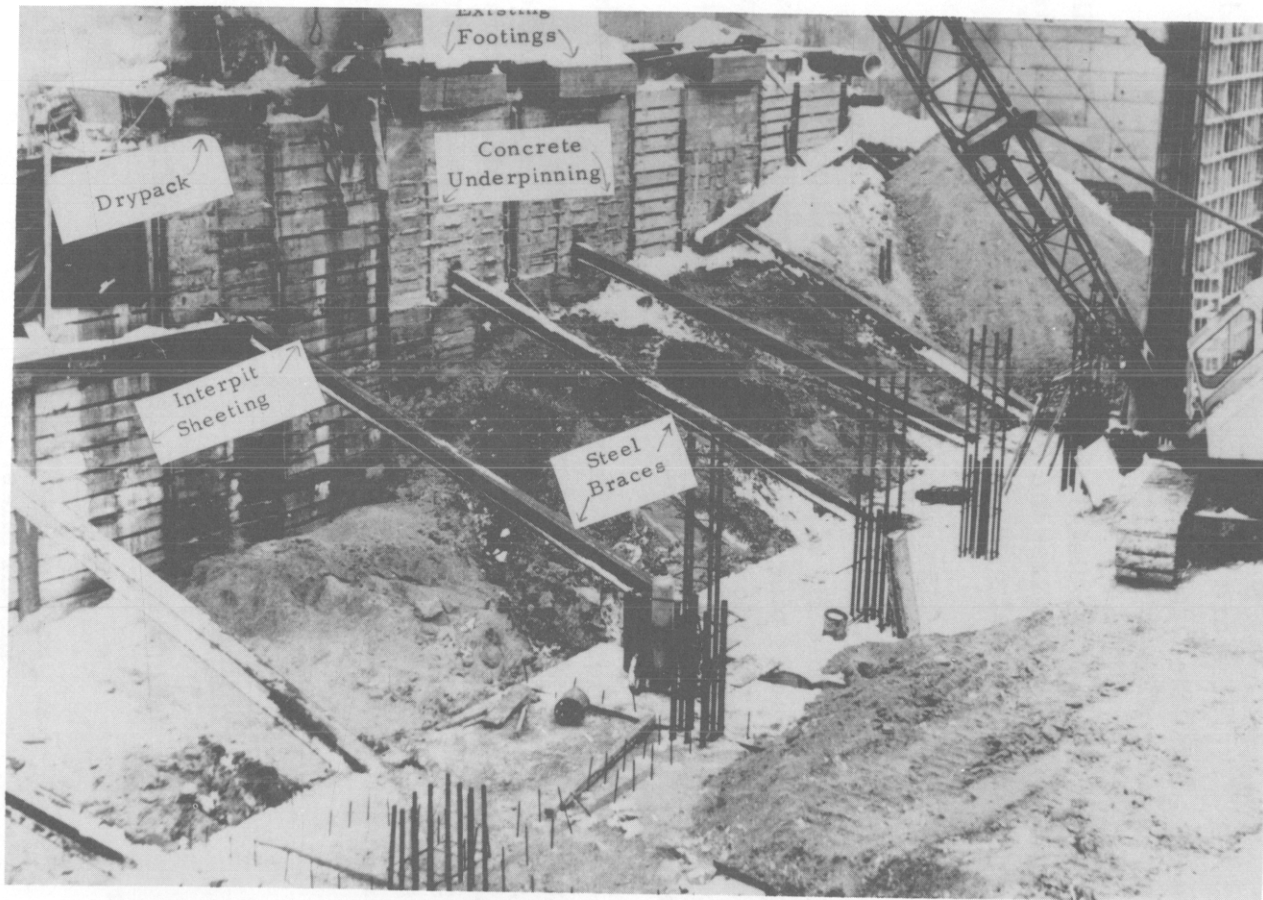
Figure 112. Details of pit underpinning.
(Courtesy of Spencer, White, and Prentis).

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.

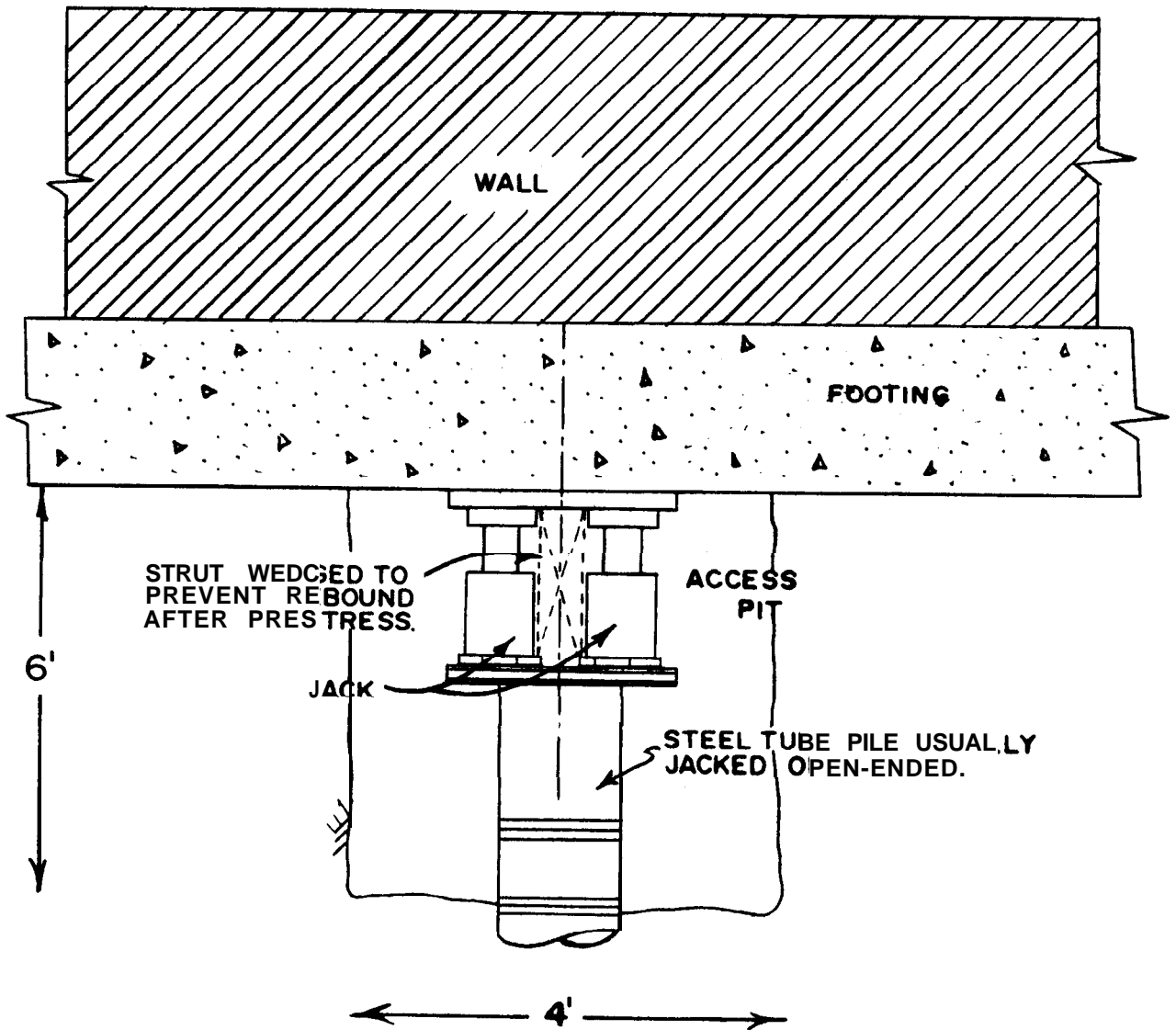b. Sequentially install a series of beams below the footing by excavating a sheeted trench.

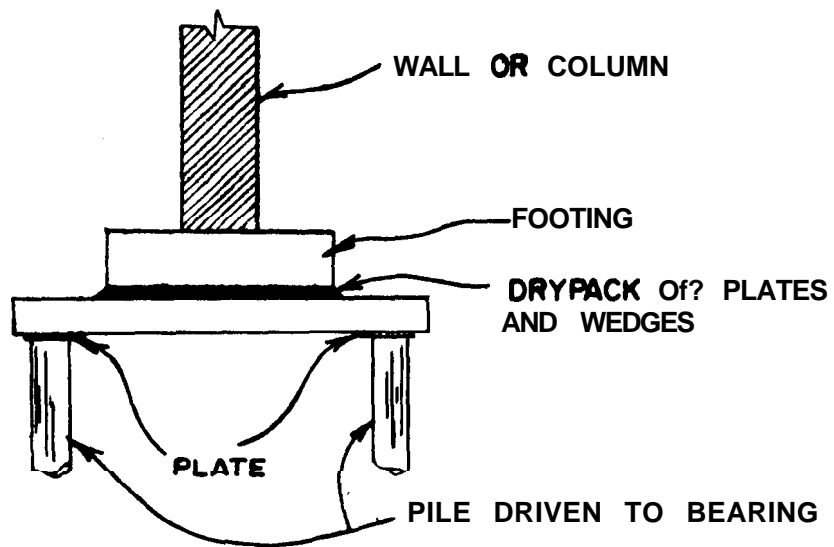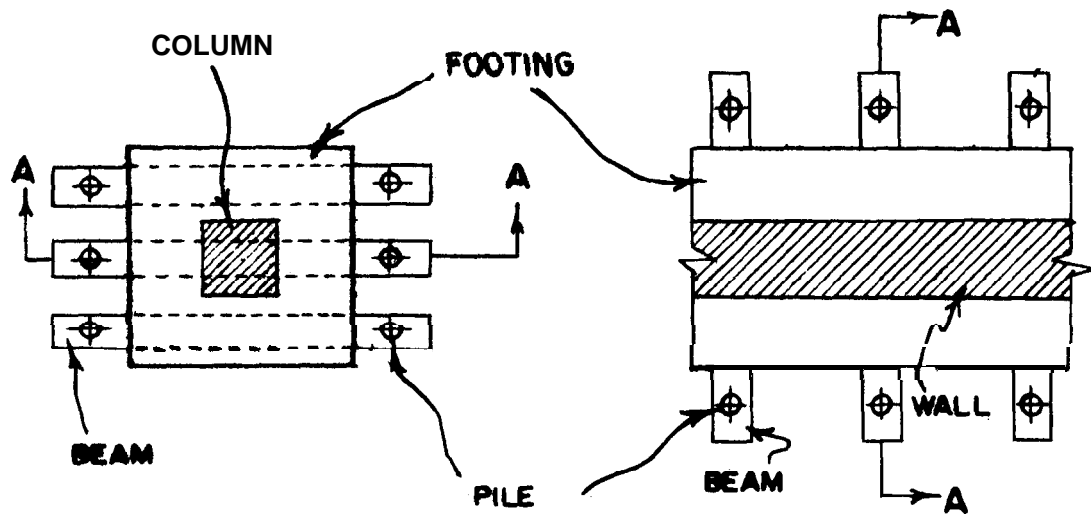**Figure 114. Jacked pile installation.**

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 normally used 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.

WALL

FOOTING

WELD

STEEL BEAM

DRYPACK OR PLATES AND WEDGES

WELD

BRACKET

STEEL BEAM

AUGER HOLE FILLED WITH LEAN CONCRETE.

BRACKET

PLAN

AUGER HOLE

ELEVATION
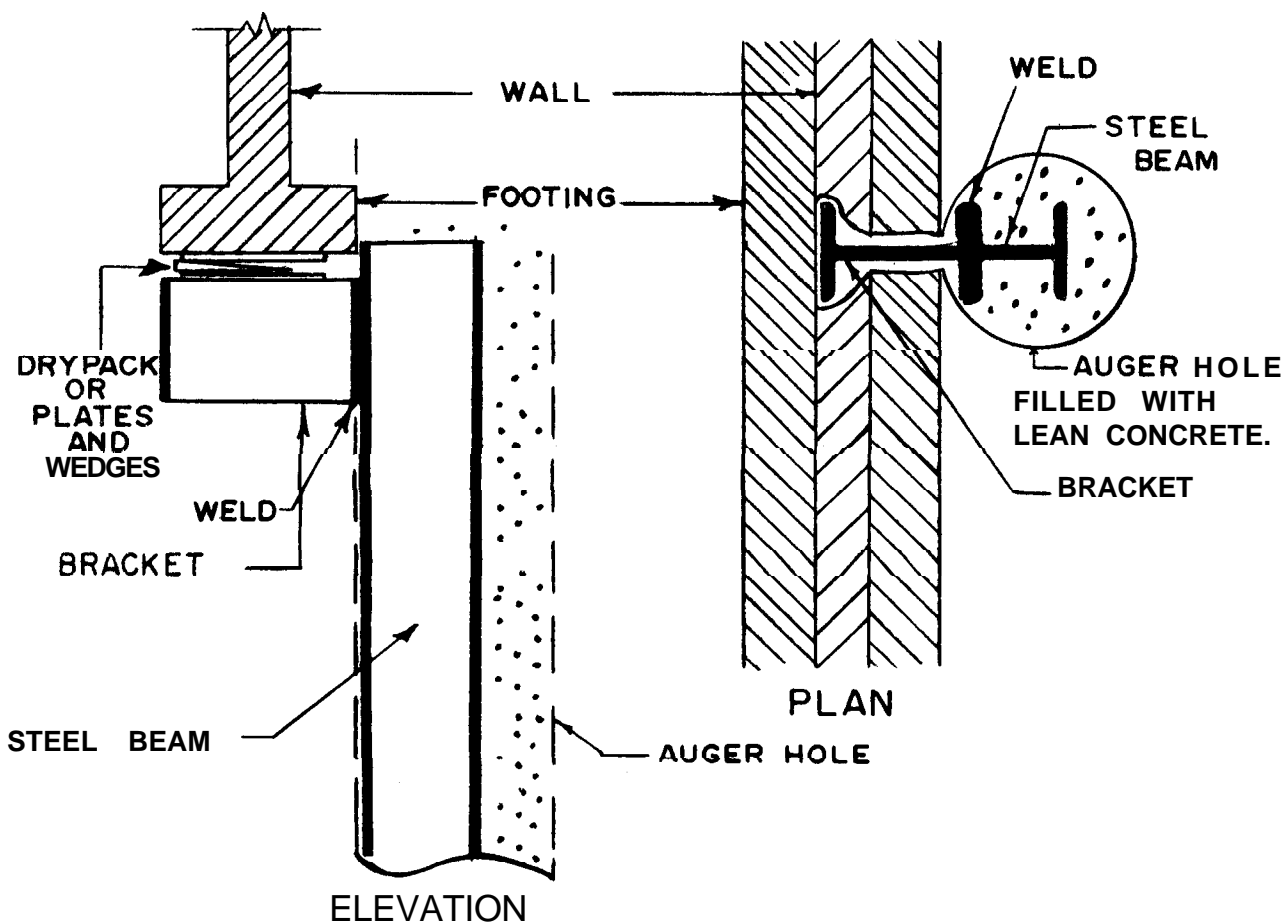
NOTE: SIMILAR DETAILS IF STEEL PILE IS DRIVEN IN PLACE.

Figure 116. Steel pile with steel bracket.
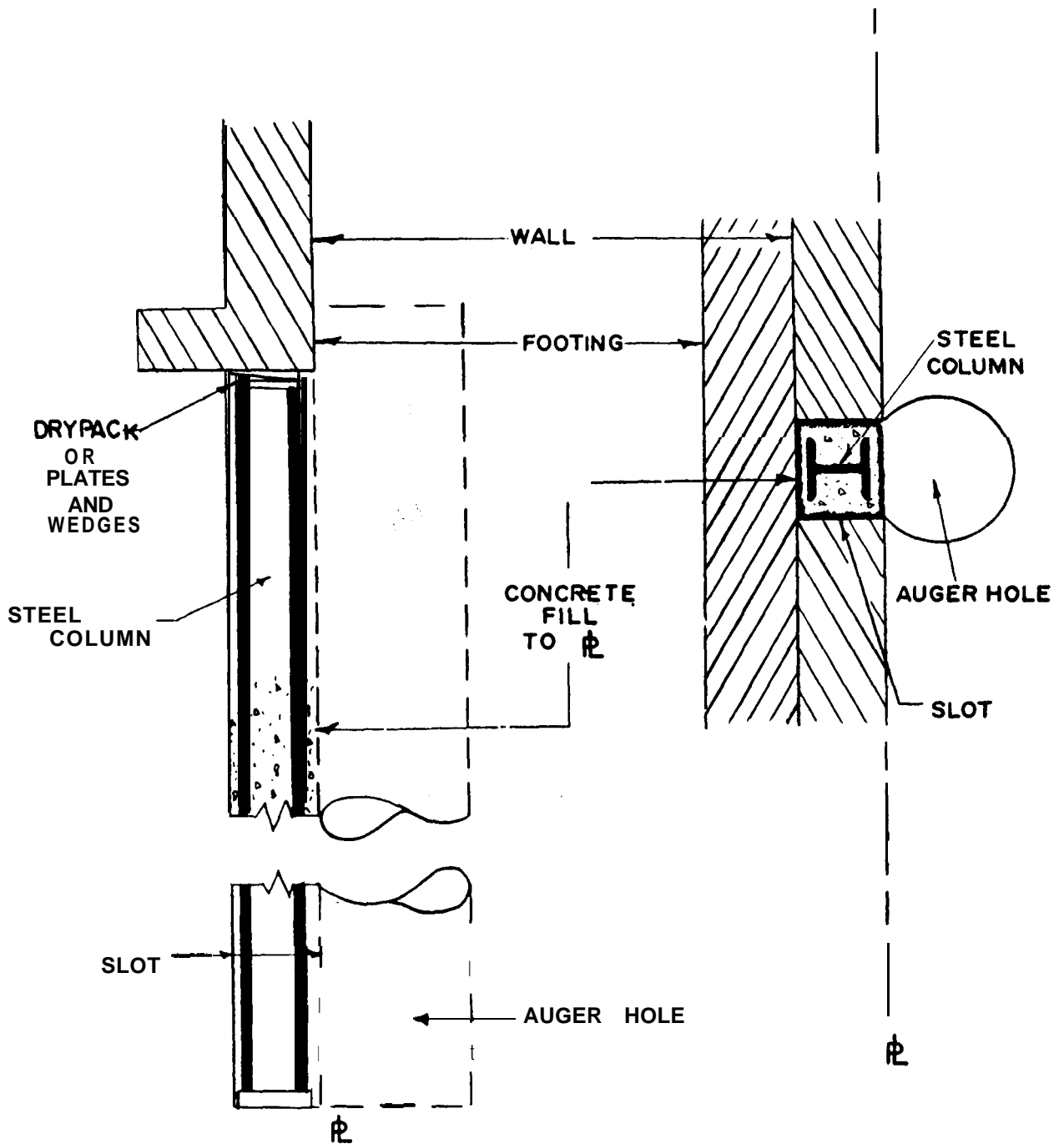
Figure 117. Auger hole with pile installed in slot.

WALL

FOOTING

BRACKET REIN-
FORCING STEEL

BRACKET

CAISSON

CAISSON REIN-
FORCING STEEL

(IF REQ'D)

PLAN

DRYPACK
OR
PLATES
AND
WEDGES

REIN. STEEL

CONCRETE--J
BRACKET

AUGERED
CAISSON

CAISSON REIN-
FORCING STEEL
(IF REQ'D)

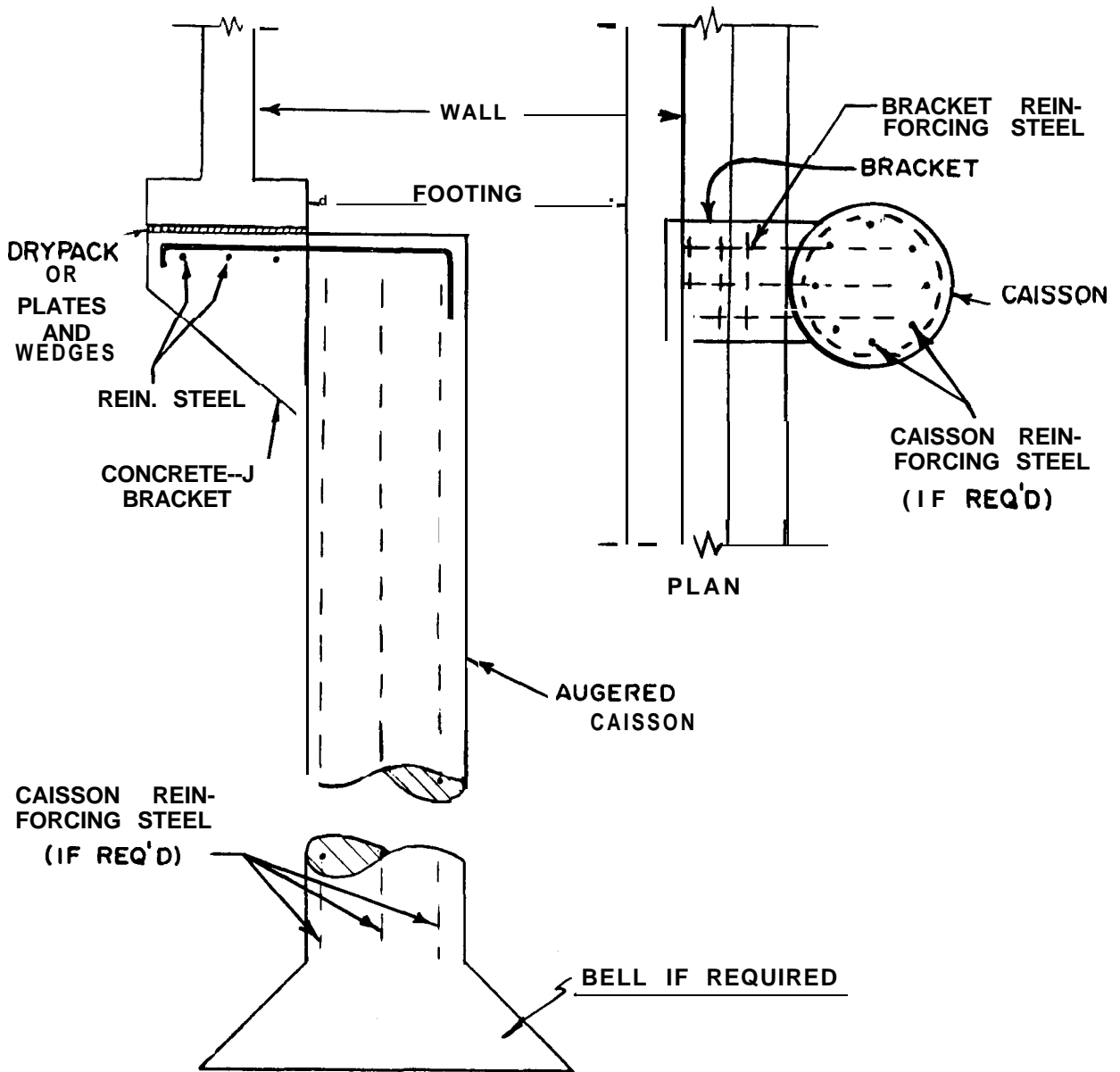BELL IF REQUIRED

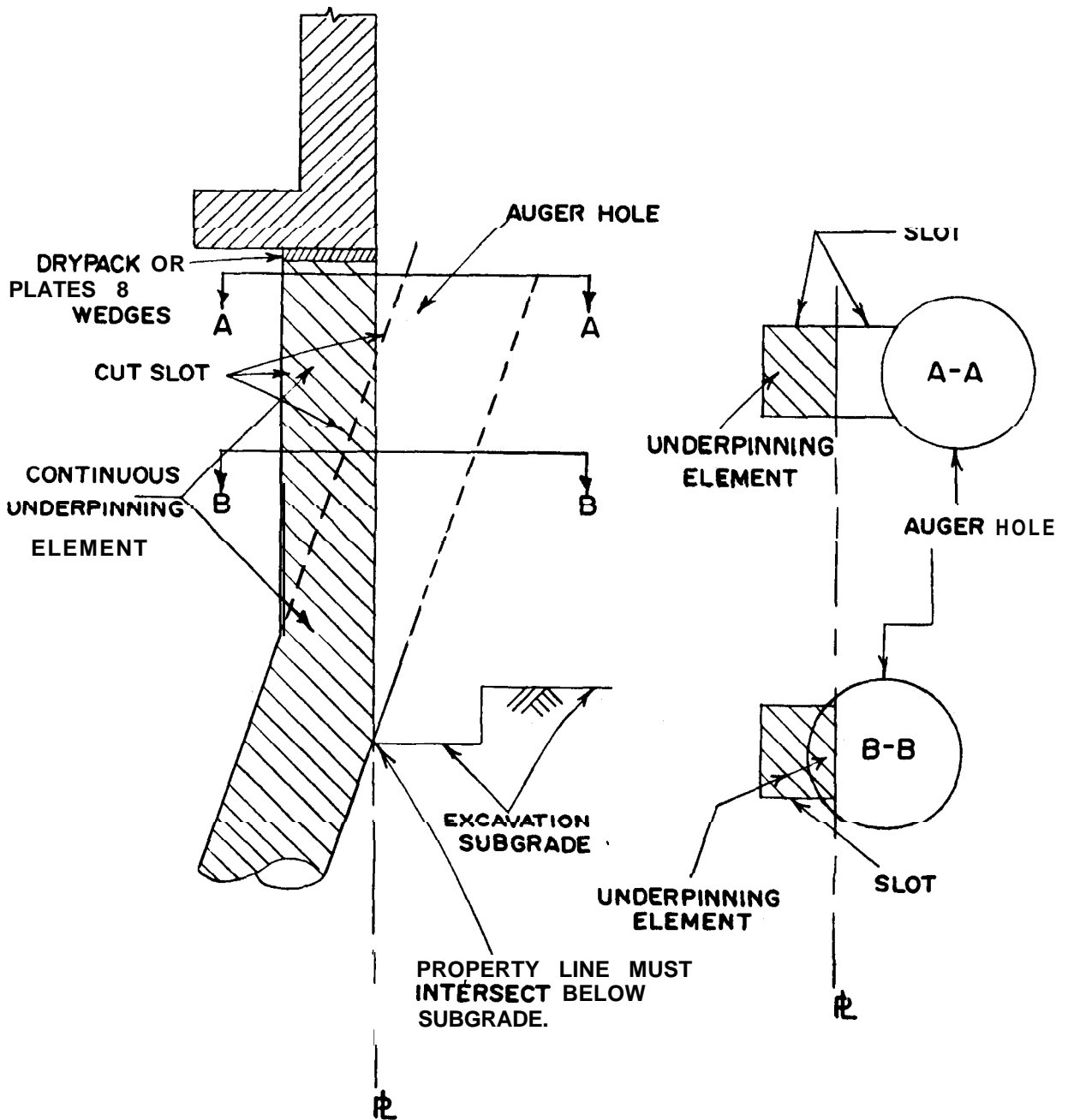Figure 118. Augered concrete caisson with concrete bucket.

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
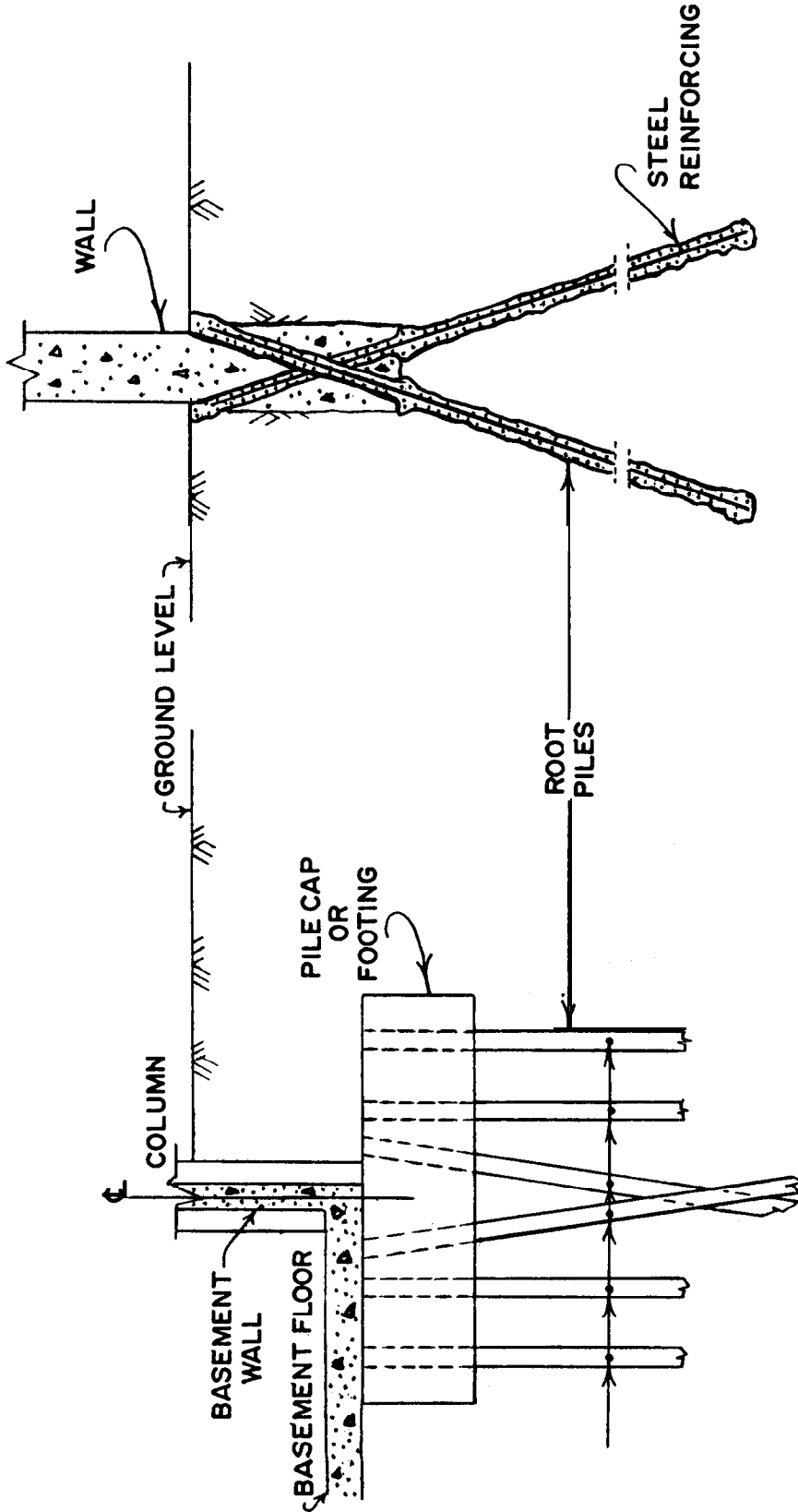
a) DIRECT SUPPORT OF FOUNDATION.
(EITHER FRICTION PILES OR END
END BEARING PILES).

b) UNDERPINNING OF A CONTINUOUS
WALL.

Figure 120.   Typical uses of root piles (pali radice).

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(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 2 (1) in-ft k ' t o n	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 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, DC., 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) $k = \dfrac{\rho}{P/L'D} = \dfrac{L'D}{P}$ (2) G = Granular; C = Clay; Si = Silt

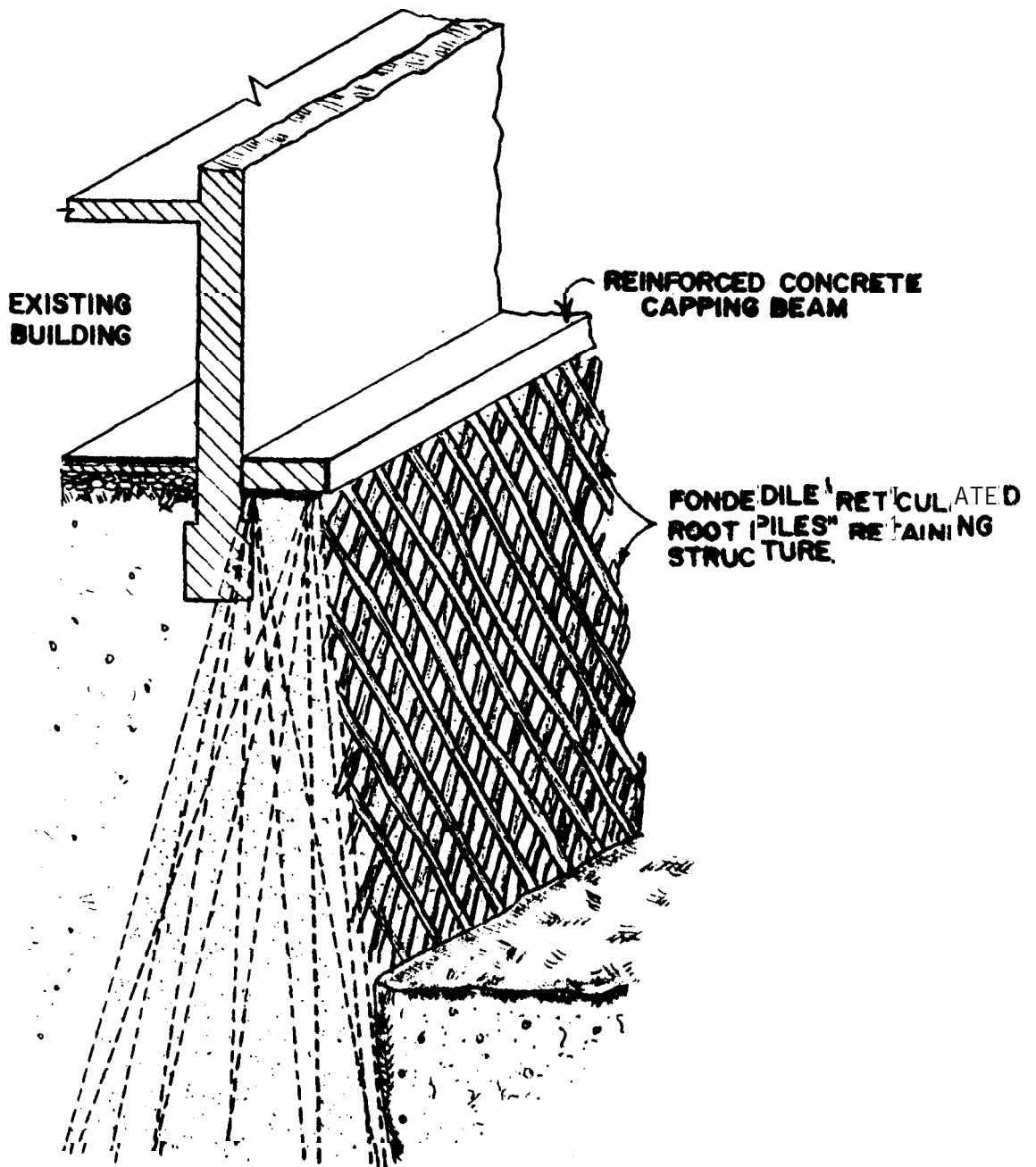* Bares, F.A.. Personal Communication, September 1975.
** Bares, F. A. (1974).

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 $k^{(1)}$, $\frac{in\text{-}ft^2}{ton}$	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	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**	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) $k = \dfrac{\rho}{P/L'D} = \dfrac{L'D}{P}$

(2)   G = Granular; C= Clay; Si = SiH

Bares. F.A. (1974)

EXISTING BUILDING

REINFORCED CONCRETE CAPPING BEAM
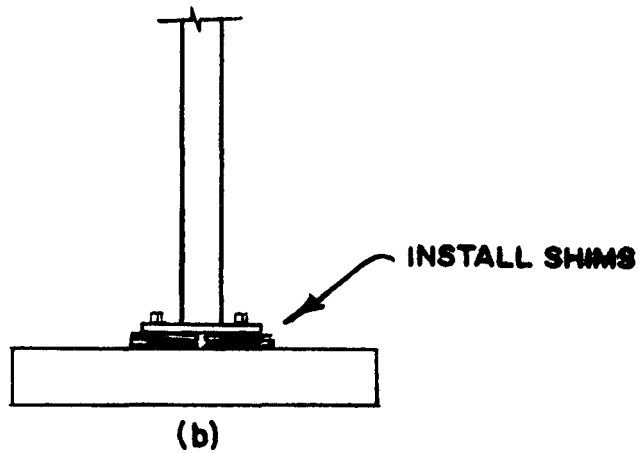
FONDEDILE "RETICULATED ROOT PILES" RETAINING STRUCTURE.

Figure 12 1. Schematic showing principle of
reticulated root piles.
(Courtesy of Warren-Fondedile, Inc.).

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(a)
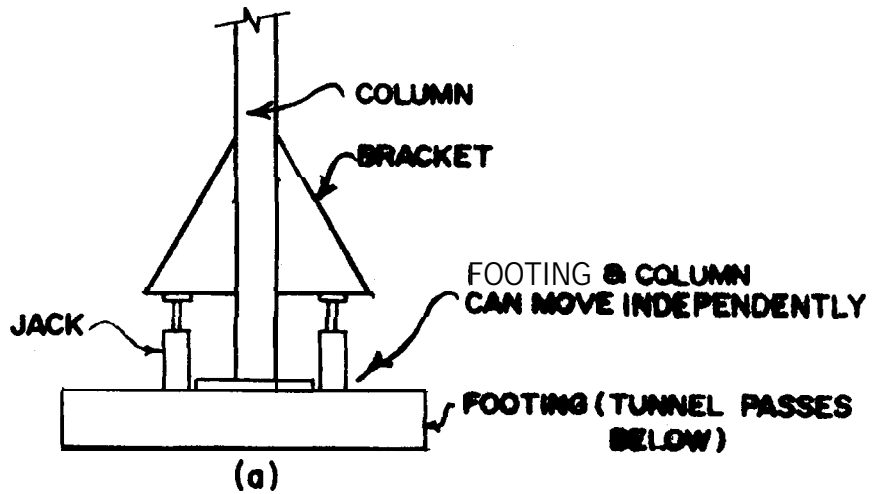
BRACKETS INSTALLED, ANCHOR BOLTS
LOOSENED, JACKS INSTALLED.

(b)

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
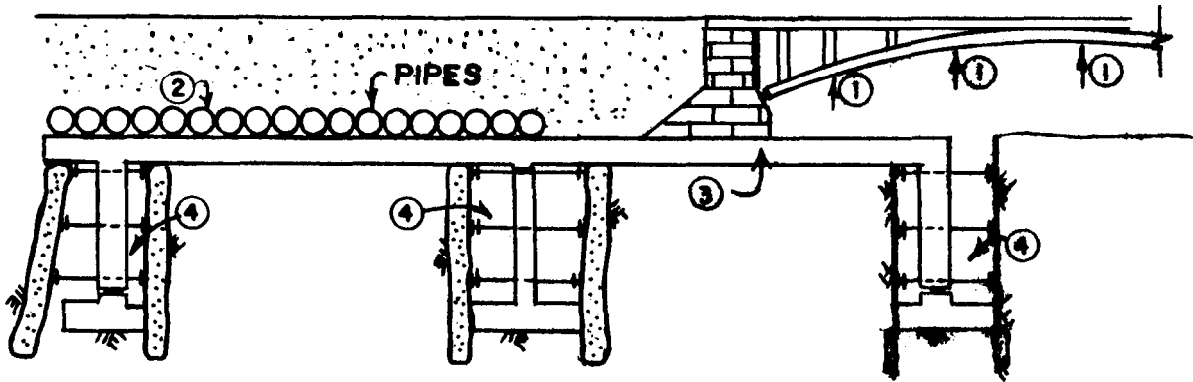
(b) AFTER COMPLETION
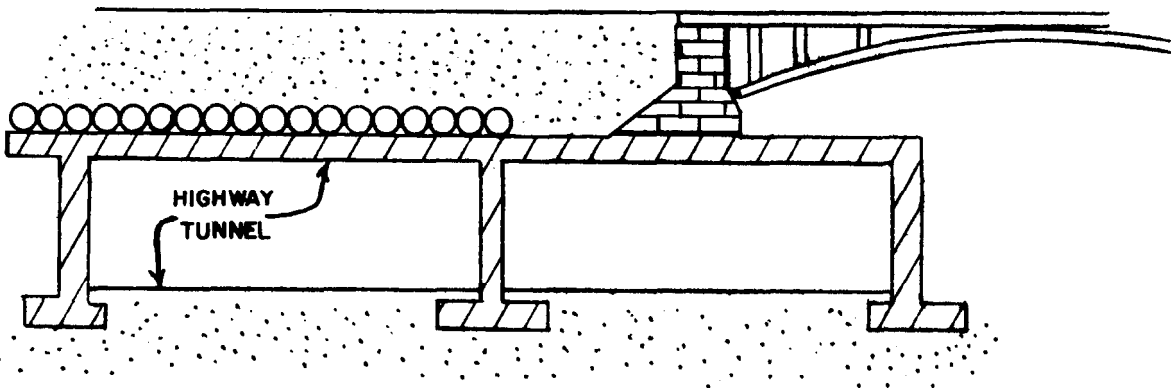
Sequence

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).

a) PLAN

CARRYING BEAM

BUILDING

PIPE SHIELD

TUNNEL

b) SECTION

BUILDING

4

RUBBISH
FILL
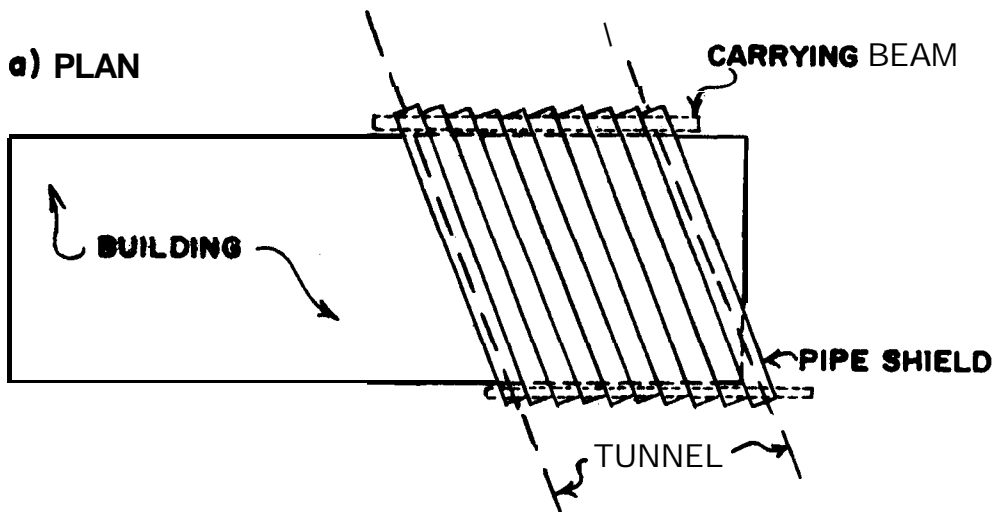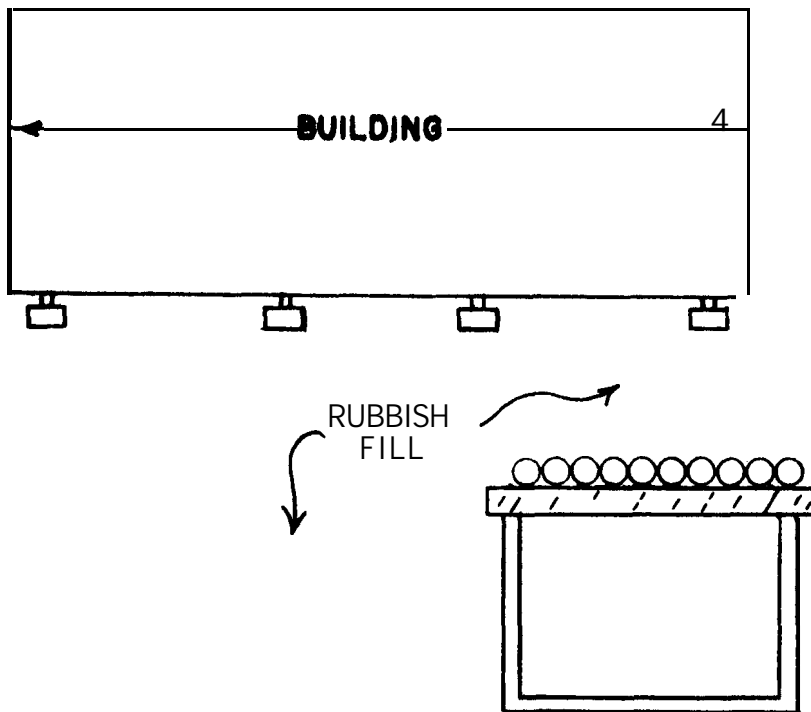
Figure 123b. **Pipe shield technique (after Rappert, 1970).**

ST. STEPHEN'S
CATHEDRAL

FILL
LOESS

CAPPING
BEAM

GRAVEL
SAND
&
SILT

BORED
PILE

STIFF FISSURED
CLAY WITH SAND
8 SILT LENSES

TUNNEL

Figure 124. Example of bored pile wall used to
pro tec t structure (after Braun, 1974).

Figure 125. Bridging.

14.62 Plates and Wedges

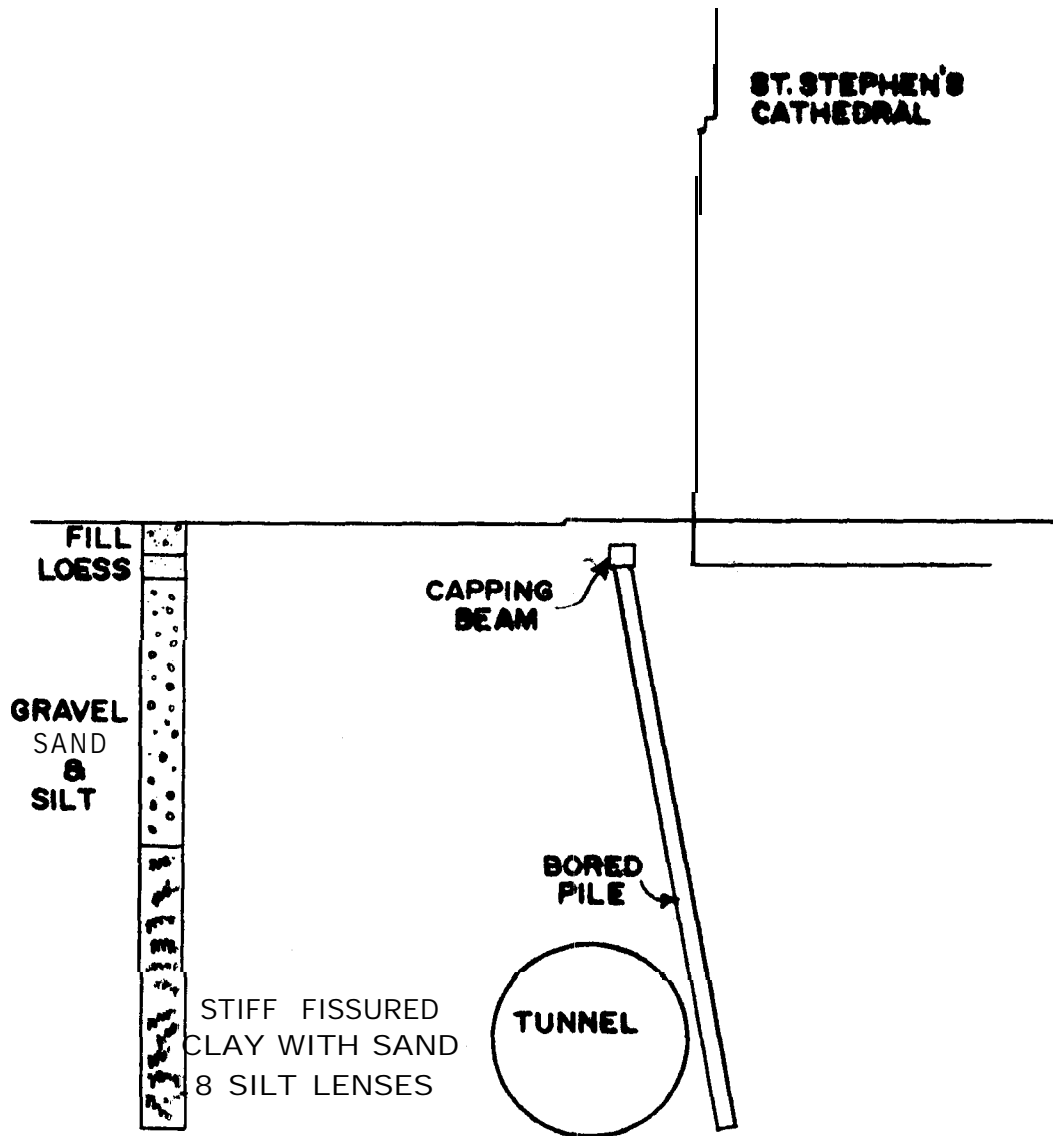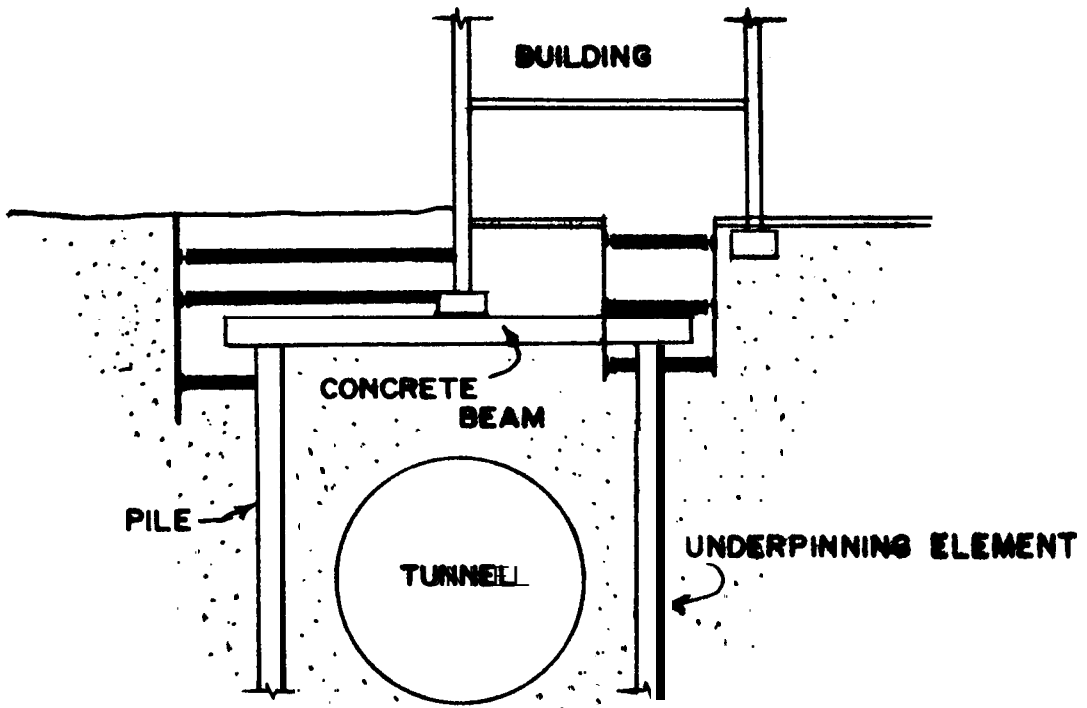This method consists of using pairs of steel (or wooden) wedges driven between steel plates in the void between the underpinning element and the footing. As the wedges are driven, their combined width increases. The footing then acts as a reaction, and the load in the underpinning element increases. 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.

WALL

DRYPACK & PLATES
OR WEDGES

CUT HOLE IN WALL

BLOCKING AND FOOTING
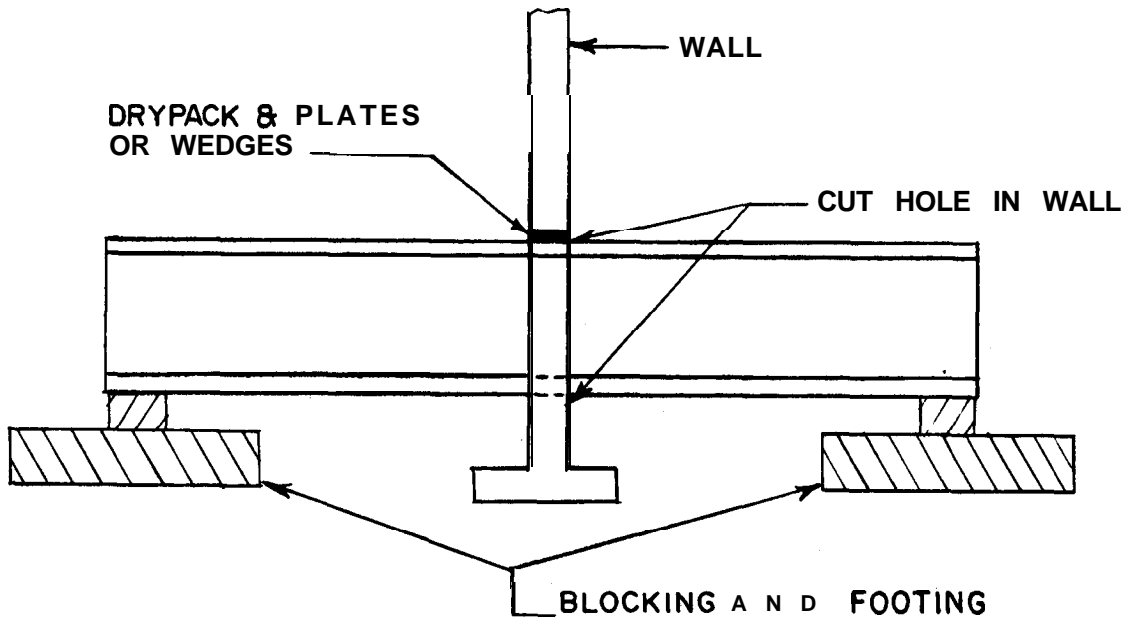
(a) NEEDLE BEAM THROUGH WALL
(SIDE VIEW)

BLOCKING AND FOOTING

COLUMN

FOOTING

FOOTING
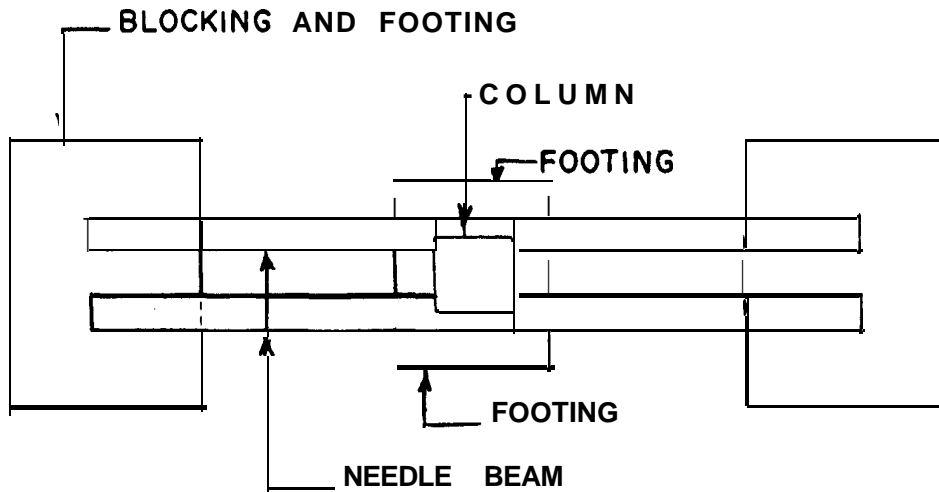
NEEDLE BEAM

(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.

CUT NOTCH IN FOOTING

DRYPACK, PLATES AND WEDGES

TEMPORARY SHORING

TEMPORAHY JACK IF REQUIRED

FOOTING

APPROACH PIT

TEMPORARY FOOTING (CONCRETE OR -WOOD)
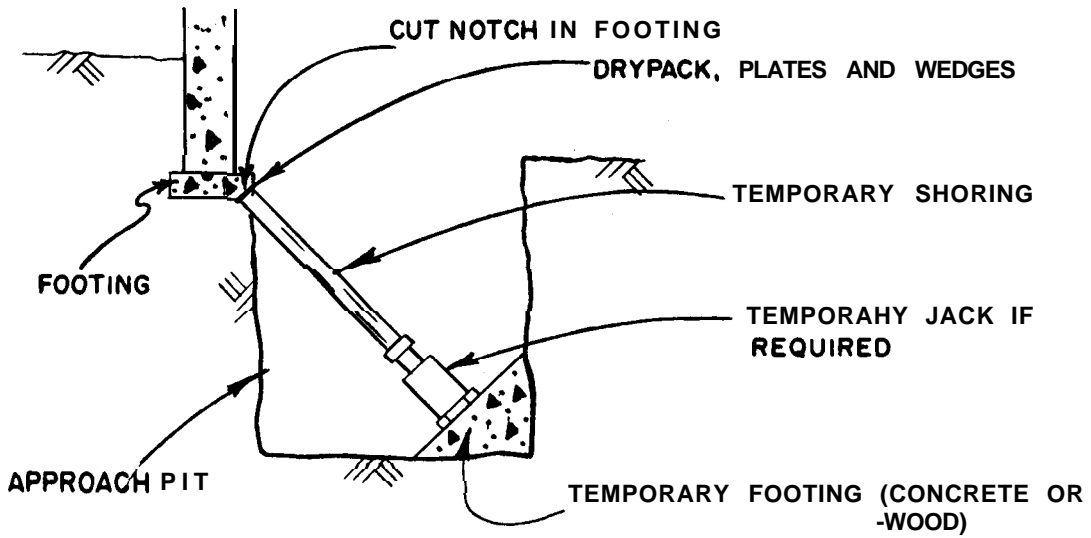
(a) INCLINED SHORING UNDER A FOOTING

COLUMN

FLOOR

FLOOR

BRACKET

INCLINED SHORE

FLOOR

FLOOR

FOUNDATION

WALL

FOOTING FOR SHORE

APPROACH AND UNDERPINNING PITS

PLATES AND WEDGES AS REQ'D
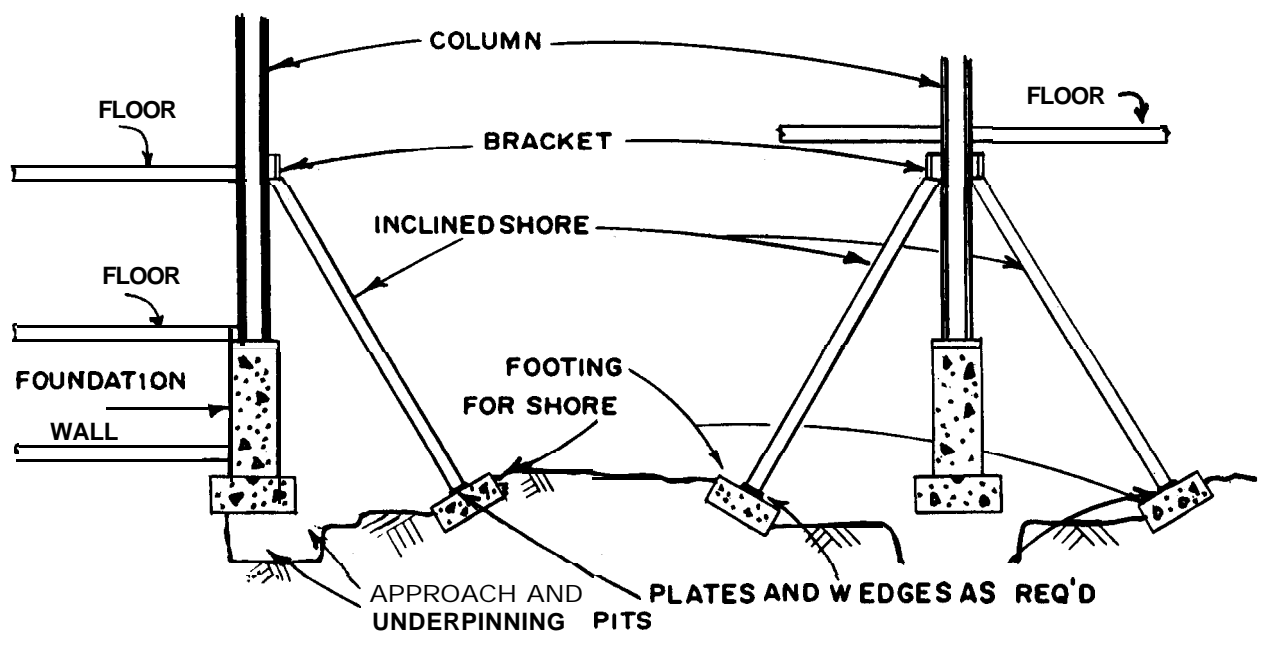
(b) SHORING A WALL OR COLUMN

Figure 127. Inclined shoring details.
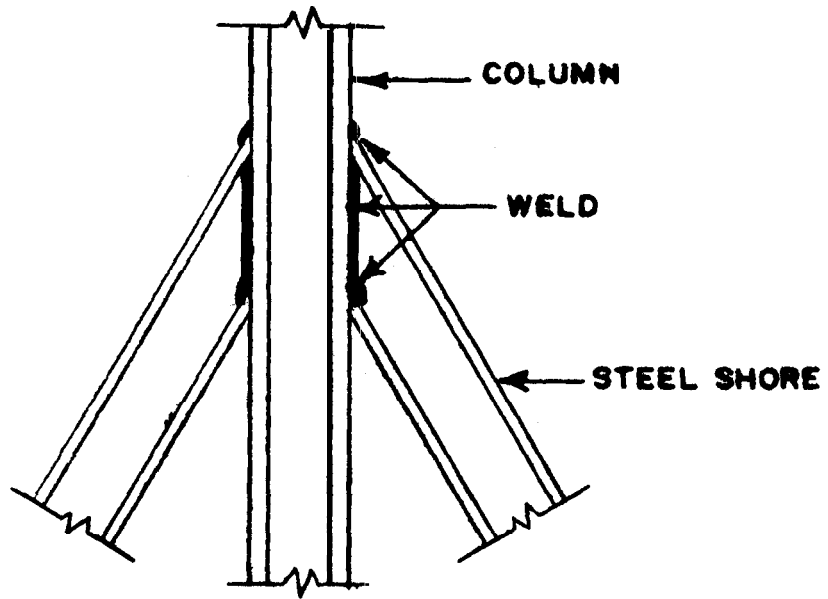
(a) WELD STEEL SHORES DIRECTLY TO COLUMN
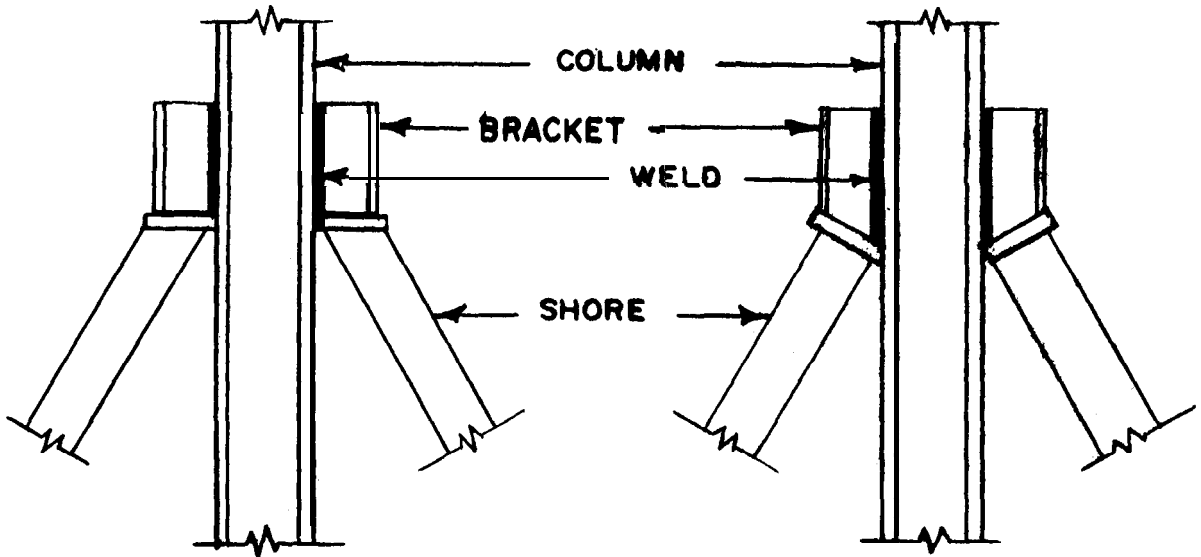
(b) SHORE TO BRACKETS WELDED ON COLUMN

Figure 128. Shoring details, steel column.

CONCRETE COLUMN

DRYPACK

NOTCH

STEEL
SHORING

REINFORCING STEEL

(a) LIGHTLY LOADED
COLUMN

CONCRETE
COLUMN

ROUGHEN
EXISTING
CONCRETE

BEARING

CONCRETE
COLLAR
(COMPLETLY
AROUND
COLUMN)
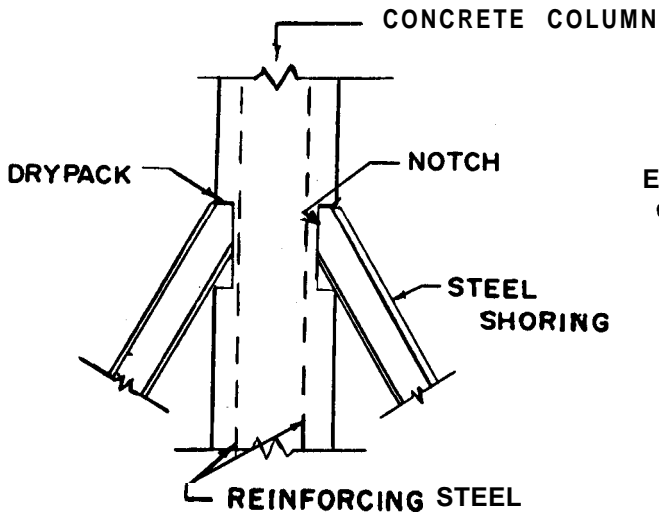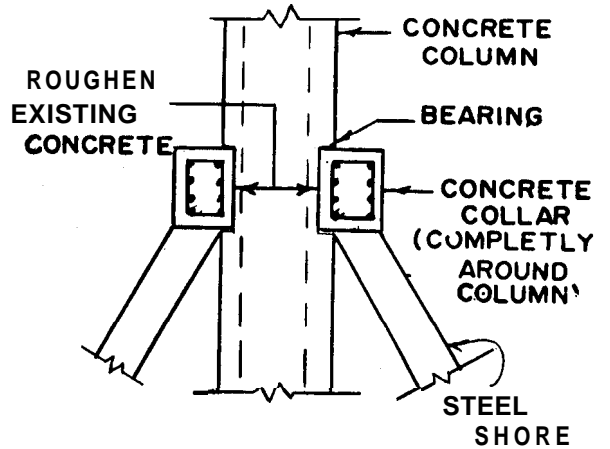
STEEL
SHORE

(b) SHORE AGAINST
CONCRETE
COLLAR

CONCRETE
COLUMN

GROUT
OR
DRYPACK

STEEL [OR I

STEEL SHORE
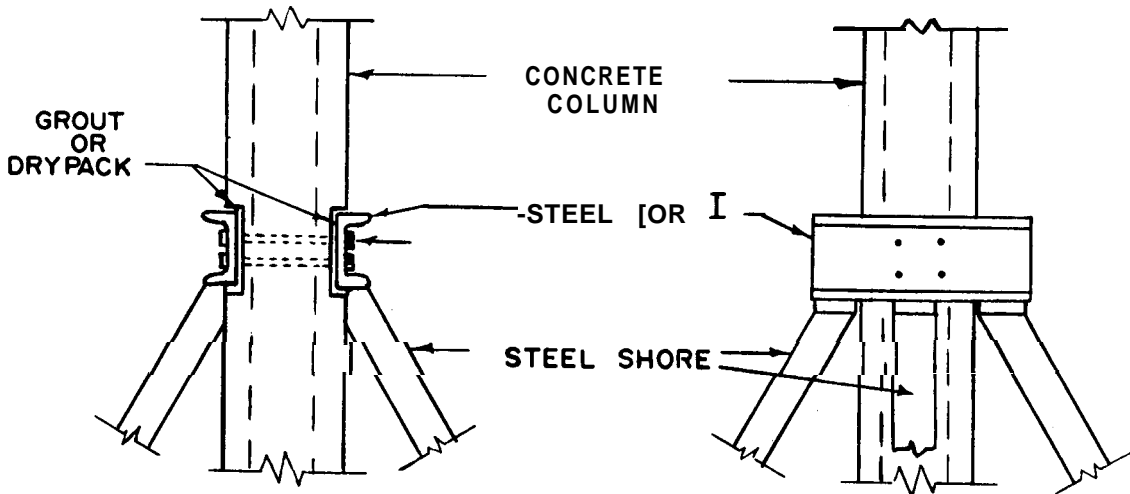
(c) CLAMP STEEL BEAMS OR CHANNELS TO COLUMN
AND SHORE AGAINST CLAMPED BEAMS.

Figure 129. Shoring detail, concrete column.

-273 -

CAST IRON COLUMN

GROUT HOLE

PIN

CLAMP

STEEL
SHORE

BOLTS

CLAMP TYPES
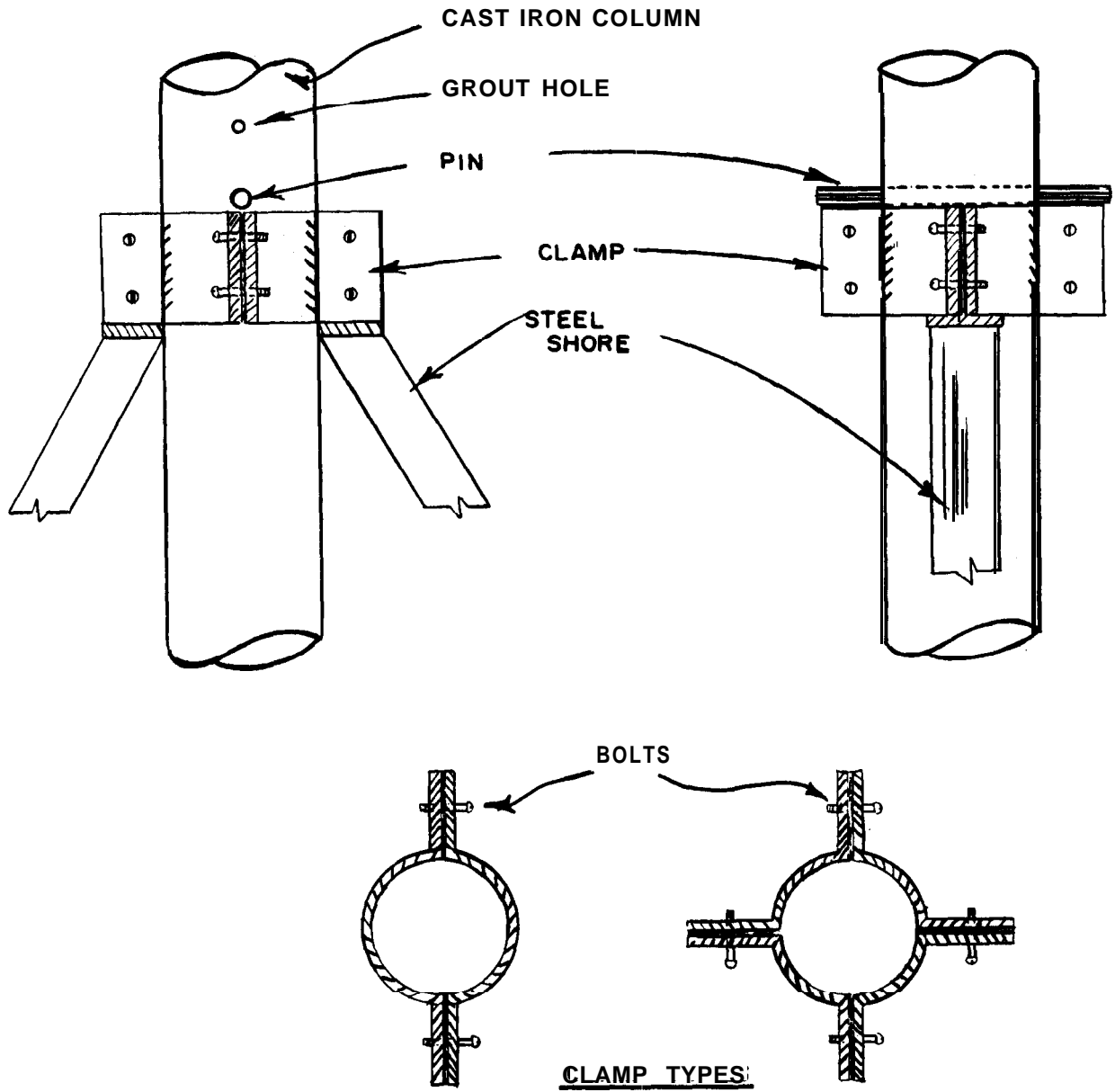
Figure 130. Pin and clamp details for a cast iron column.

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

CLAYEY SAND
(IMPERMEABLE)

SOLDIER PILE
WALL

PERVIOUS
SAND & GRAVEL

A

CLAY
(IMPERMEABLE)

GROUTED ZONE

CLAY

CLAY
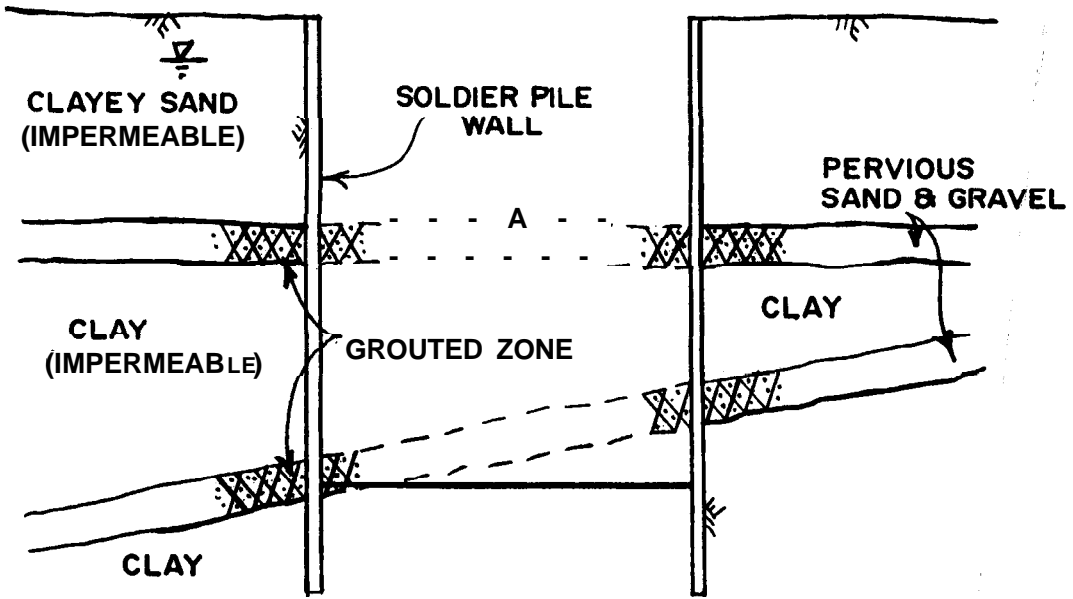
b) DEEP PERVIOUS DEPOSITS

DIAPHRAGM
WALL

Z

SAND & GRAVEL
(PERVIOUS)

X

(NOTE: $X \gamma_m > Z \gamma_w$)

HARDPAN (IMPERVIOUS)
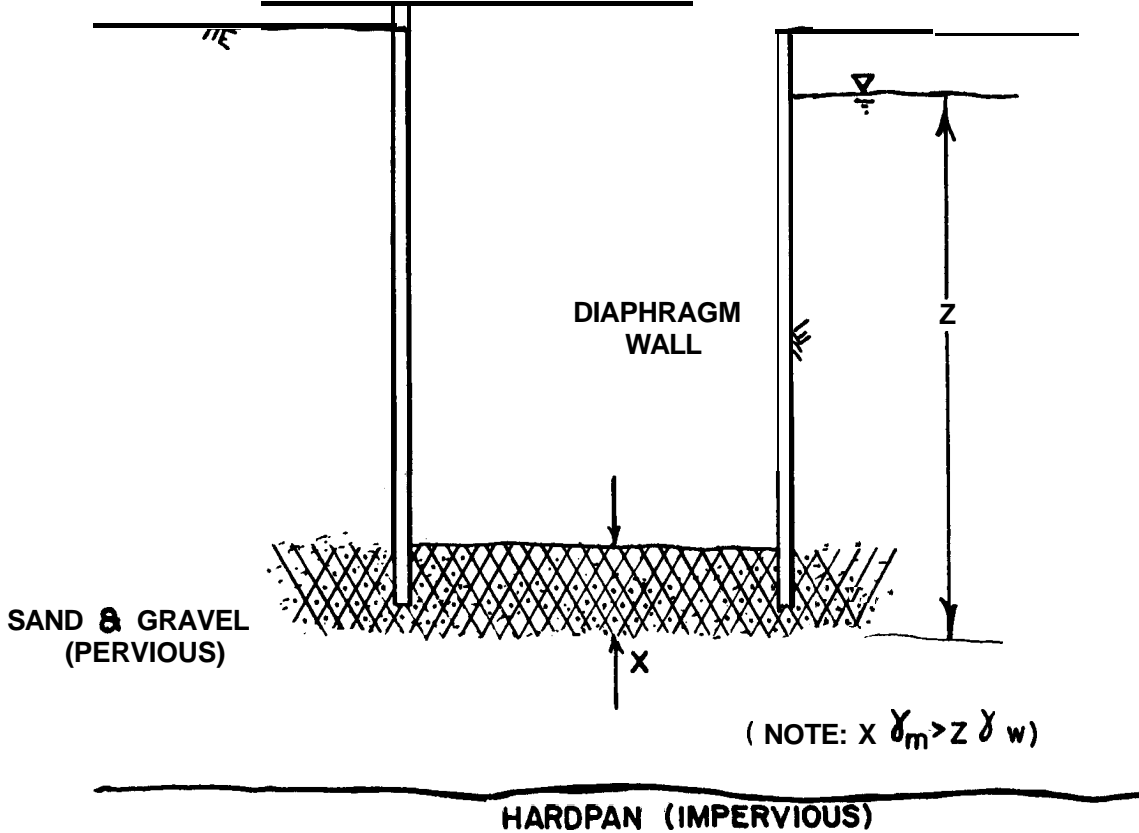
Figure 132. Grouting for ground water control.

c) BOULDERS OR SHATTERED ROCK

SAND

STEEL SHEETING

CLAY

SAND

SAND, GRAVEL
& BOULDERS

GROUT SOIL
STRATUM AND
ROCK

ROCK- BADLY
JOINTED &
FRACTURED

SHEETING
PENETRATION
UNCERTAIN

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.
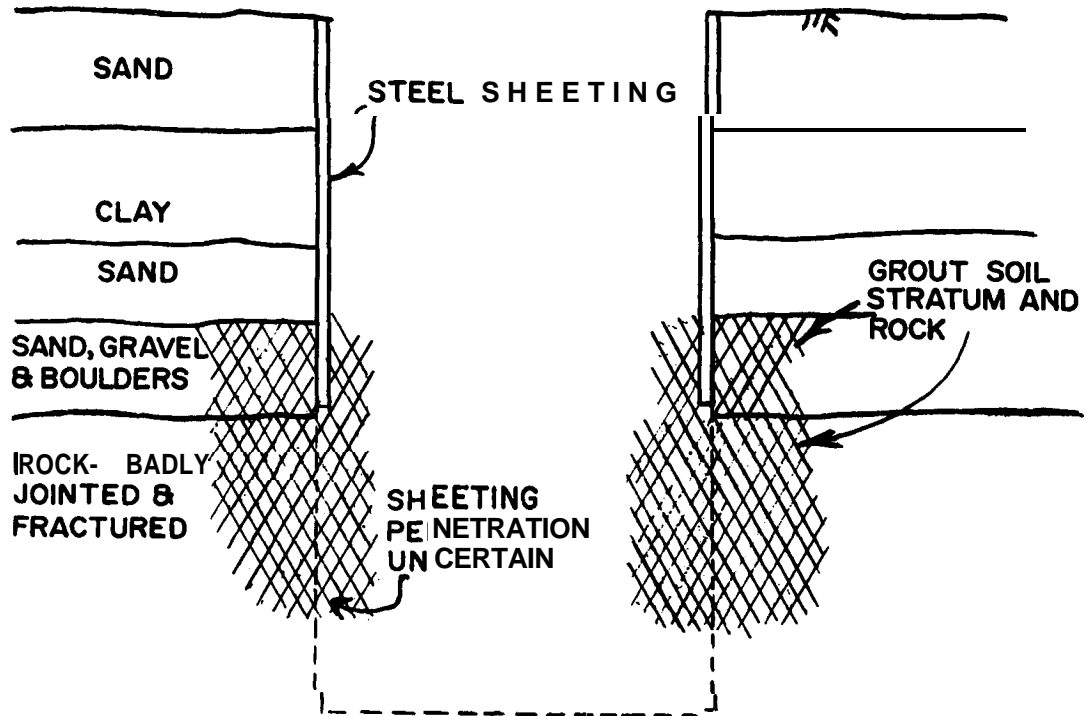
15.22 Soil Profile and Soil Type

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

a) **CUT AND COVER**

GROUTED ZONE

SOLDIER PILE WALL

LOOSE SAND

STIFF CLAY

b) **SOFT GROUND TUNNEL**
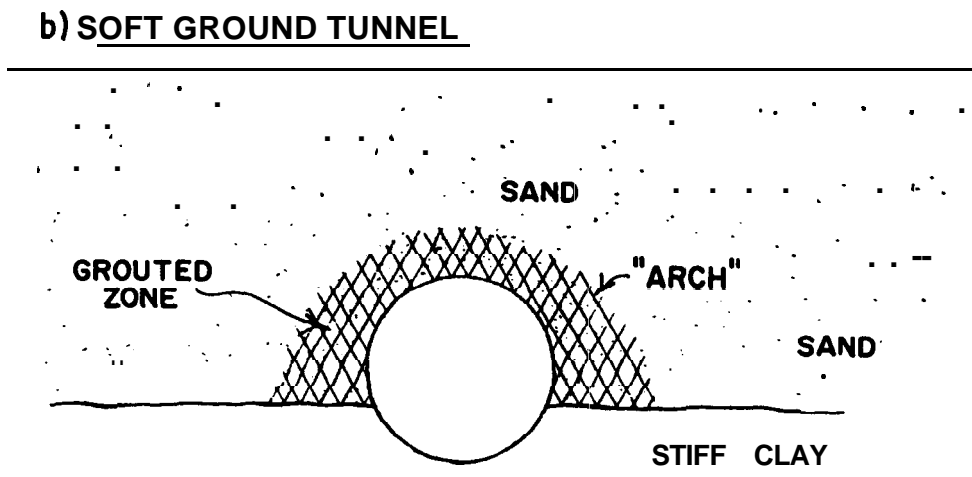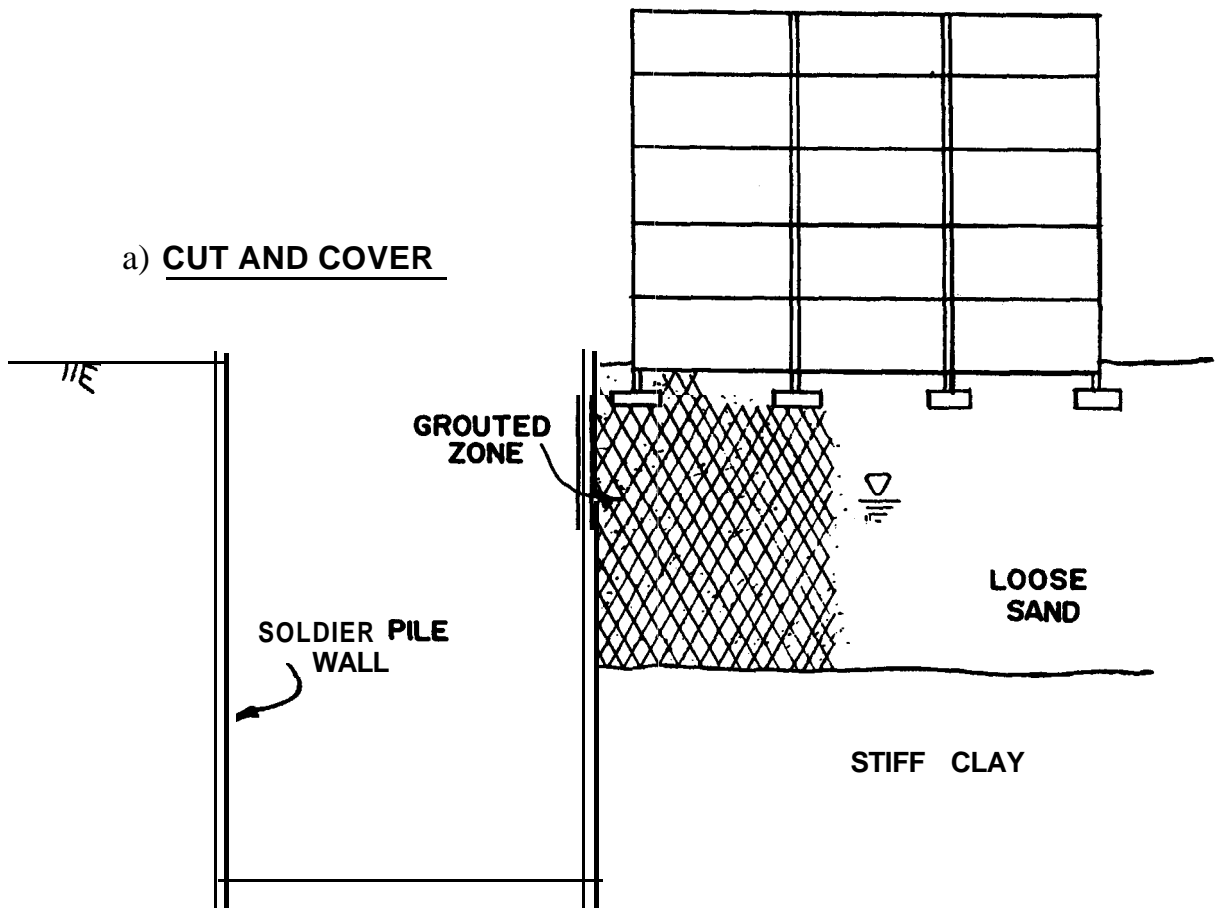
SAND

GROUTED ZONE

"ARCH"

SAND

STIFF CLAY

Figure 133.   Grouting   for   soil   solidification.
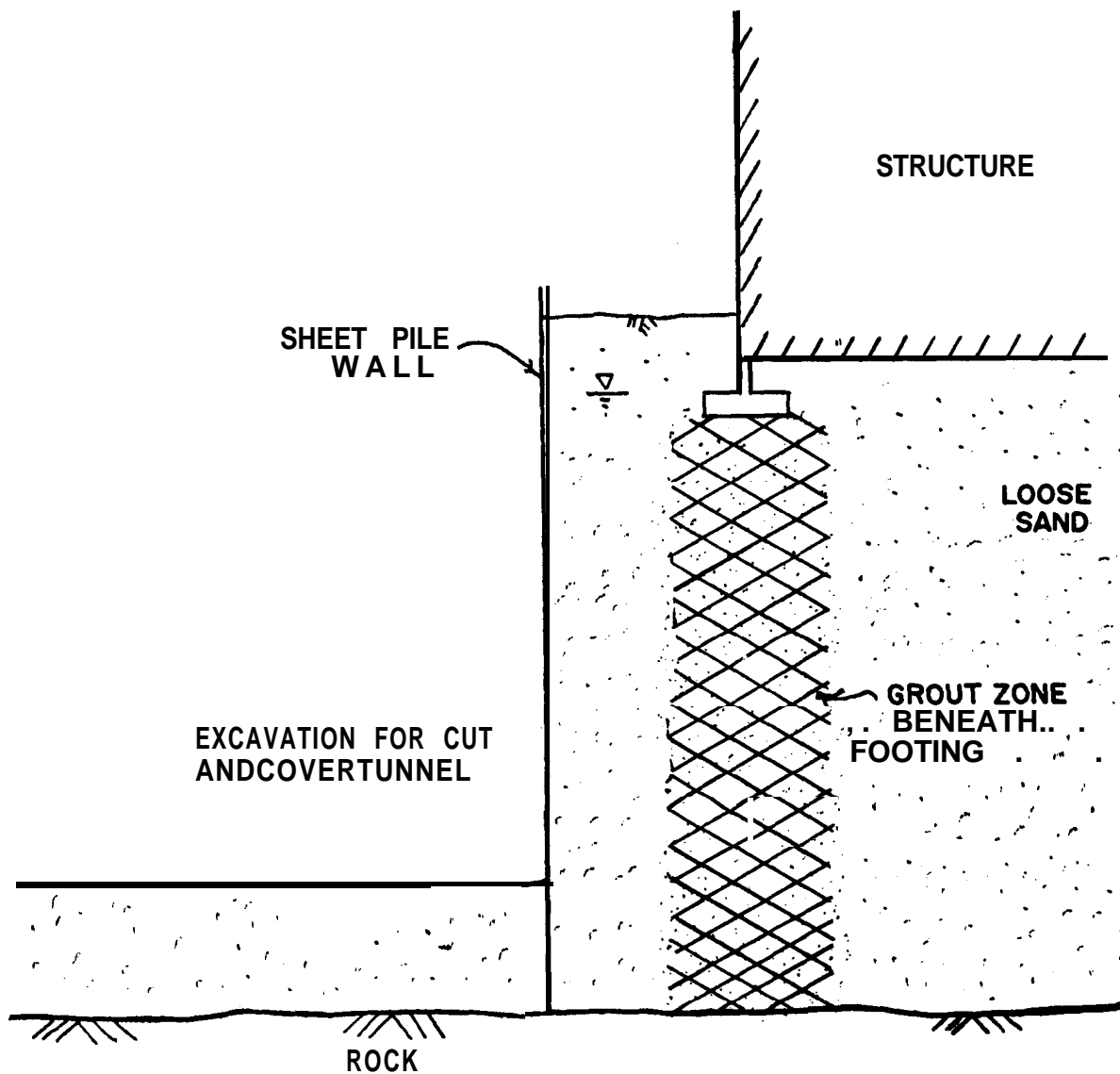
Figure 134. Example of grouting used to underpin an existing structure.

STRUCTURE

SHEET PILE
WALL

LOOSE
SAND

GROUT ZONE
BENEATH
FOOTING

EXCAVATION FOR CUT
ANDCOVERTUNNEL

ROCK

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.

$$\text{Groutability ratio} = \frac{D_{15}\,(\text{soil})}{D_{85}\,(\text{grout})} > 25$$

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 　**P rec** ip tat ion	Silicate Chemicals Aluminate Chemicals Chromelignosulfates	
Polymers	**Acrylamides (e.** g. **AM9)** Phenoplasts or Aminoplasts **(e.g.** recorcineformol, urea **-fo** rmol) **Epoxy** Polyester-resins	Injected in form of monomers 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:

$$ t = \frac{\alpha n}{3khr_0} (R^3 - r_0^3) $$

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 char-acteristics, 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
Grain diameter	$d_{10} > 0.5mm$	$0.02 < d_{10} < 0.5mm$	$d_{10} < 0.02mm$
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	Bing ham 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

(Soil Properties spans the Grain diameter, Specific surface, and Permeability rows.)

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.

AASHO CLASSIFICATION

GRAVEL	SAND			SILT
	COARSE	MEDIUM t FINE		

CEMENT SOIL

CLAY

SILICATE SOLUTIONS

CHROME - LIGNINS

POLYMERS(RESINS, PHENOPLASTS)

| 10 | 1.0 | 0.1 | 0.01 | 0.001 |

GRAIN SIZE MILLIMETERS
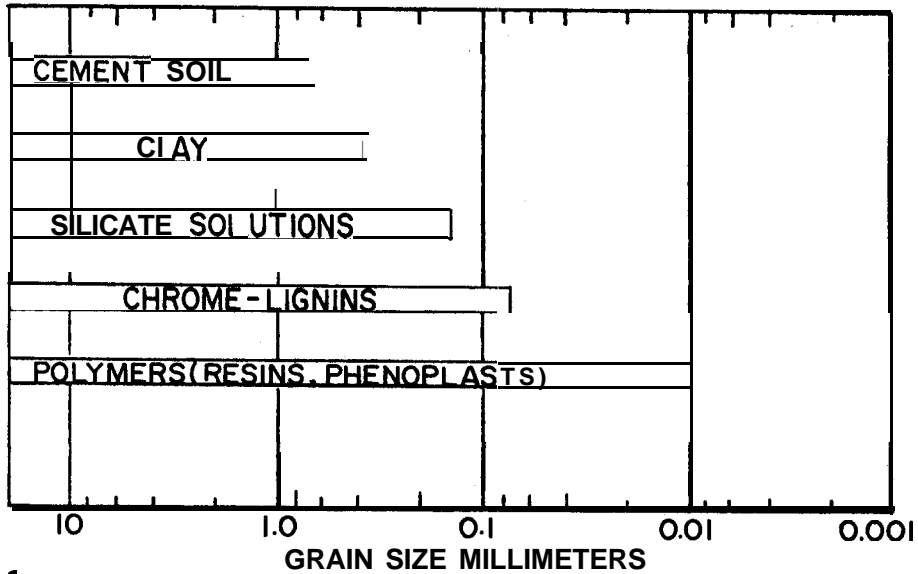
GRAVEL	SAND		SILT
FINE COARSE	MEDIUM	FINE	

UNIFIED'SOIL CLASSIFICATION

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.	Spe cifi c Gravity	Special Fields		
					Water - s topping	Consolidation	
					Fine Soil	Medium Strength	High Strength
Silica gel low concentra- tion	Silicate- bicarbonate	1. 5	0.1-300	1.02	X	X	
Silica gel high con- centration	Silicate- formamide	4-40	5.-300	1.10			X
Chrome li gnin	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	U F	6	5-300	1.08			X
Xl-based un- saturated fatty acid polymer s	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.
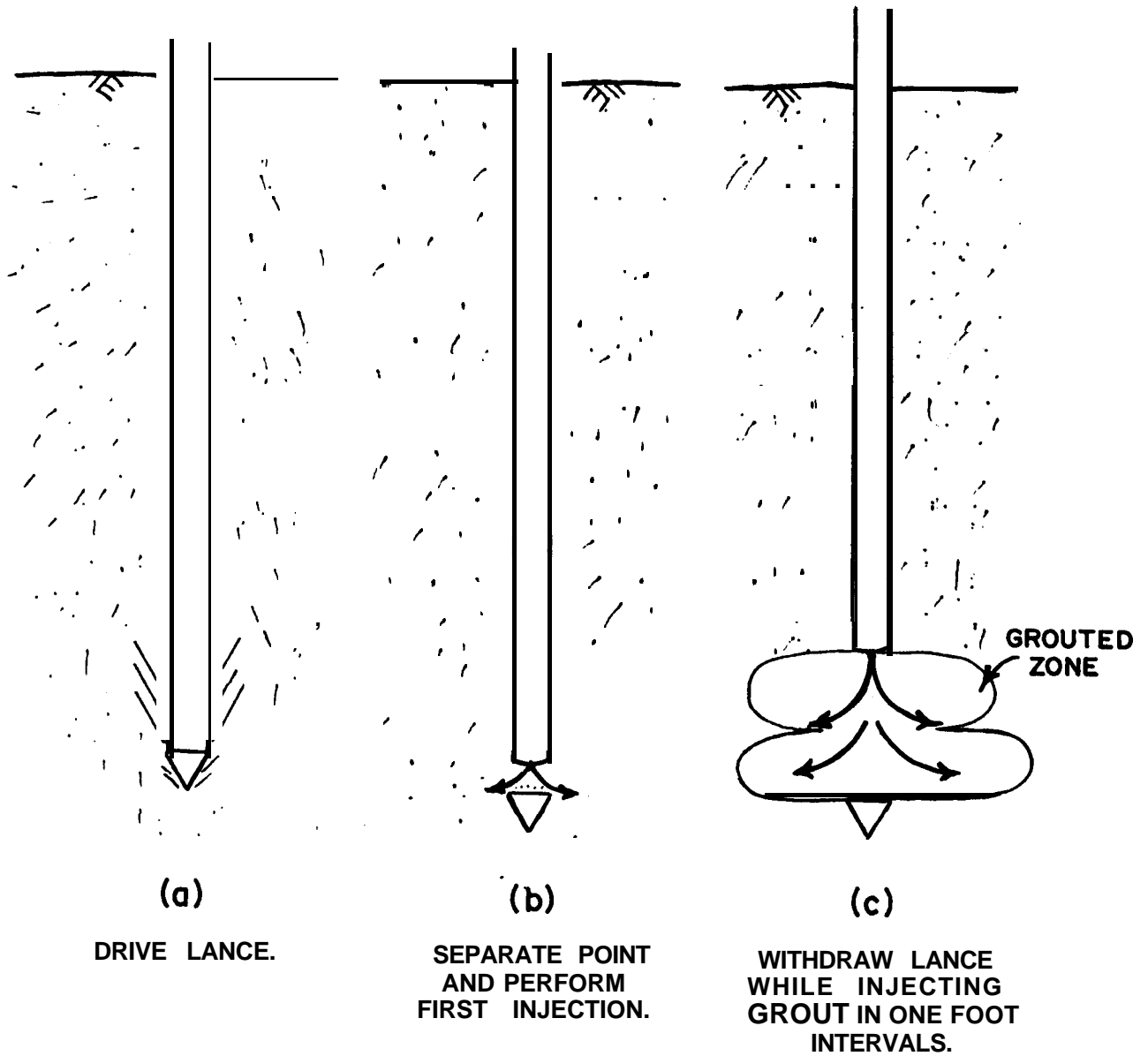
(a)

DRIVE LANCE.

(b)

SEPARATE POINT
AND PERFORM
FIRST INJECTION.

(c)

WITHDRAW LANCE
WHILE INJECTING
GROUT IN ONE FOOT
INTERVALS.

GROUTED
ZONE

Figure 136. Schematic of driven lance method.

DOUBLE PACKER

WALL OF GROUT HOLE

SEMI • PLASTIC SEALING SHEATH

PIPE SEALED INTO HOLE

RUBBER "MANCHETTE"

GROUTING ORIFICE

GROUTING PIPE

DOUBLE-PACKER

(a)

① BORING & CASING

② INSERTING THE TUBE à MANCHETTES

③ SEALING- IN OF TUBE à MANCHETTES AND WITHDRAWL OF CASING

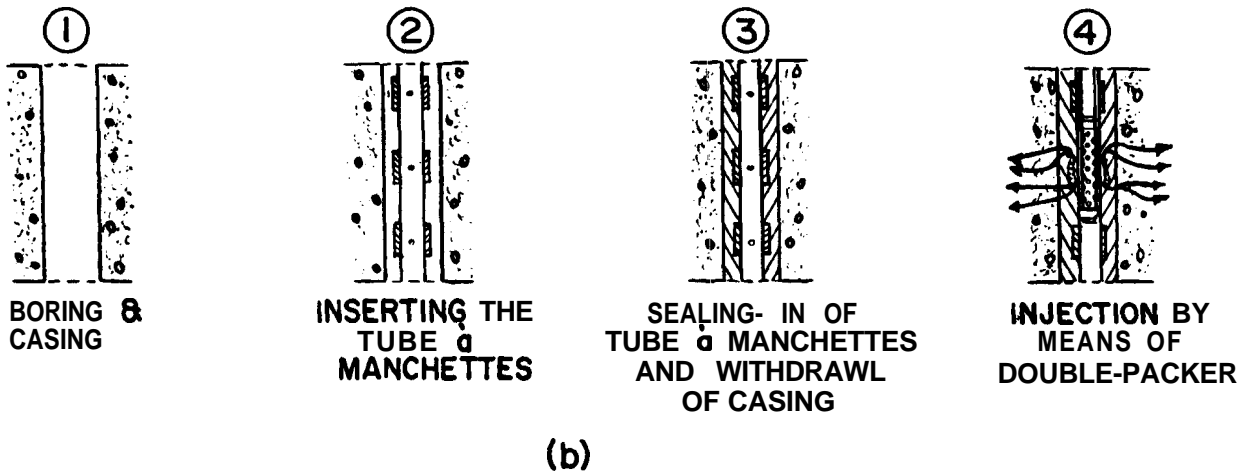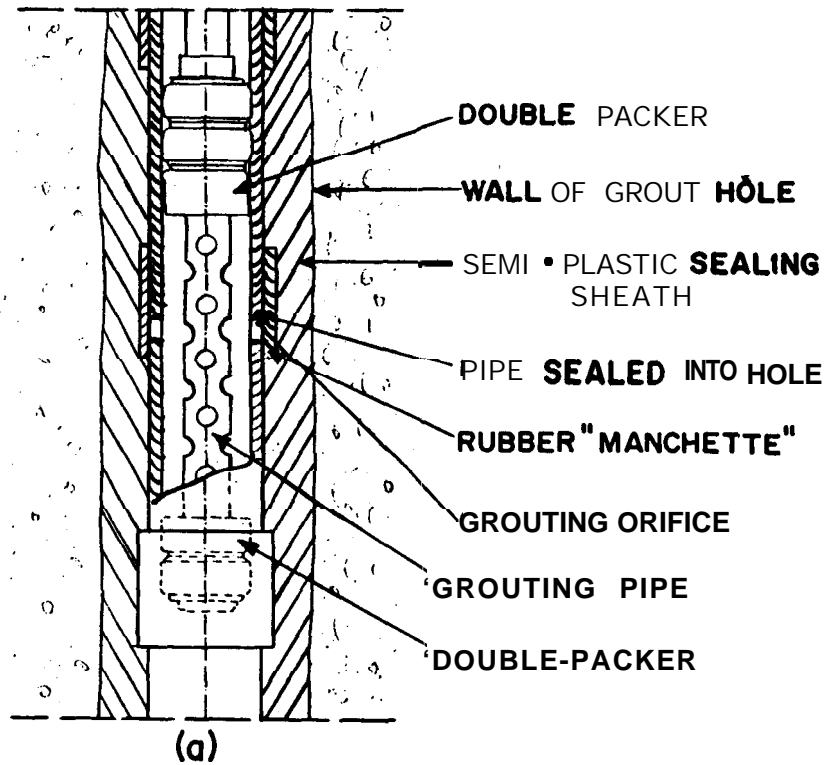④ INJECTION BY MEANS OF DOUBLE-PACKER

(b)

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.

CHAPTER 16 - GROUND FREEZING

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_0)

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:

$$C_u = \gamma_d (0.2 + \frac{w}{100}) \text{ in BTU/ft}^3/^\circ F \text{ (unfrozen)}$$

$$C_f = \gamma_d (0.2 + \frac{0.5w}{100}) \text{ in BTU/ft}^3/^\circ F \text{ (frozen)}$$

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).

$$L = \gamma_d \, 0.8w \text{ gm-cal/cm}^3 \; (\gamma_d \text{ in gm/cm}^3)$$

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 ($\propto$ =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.
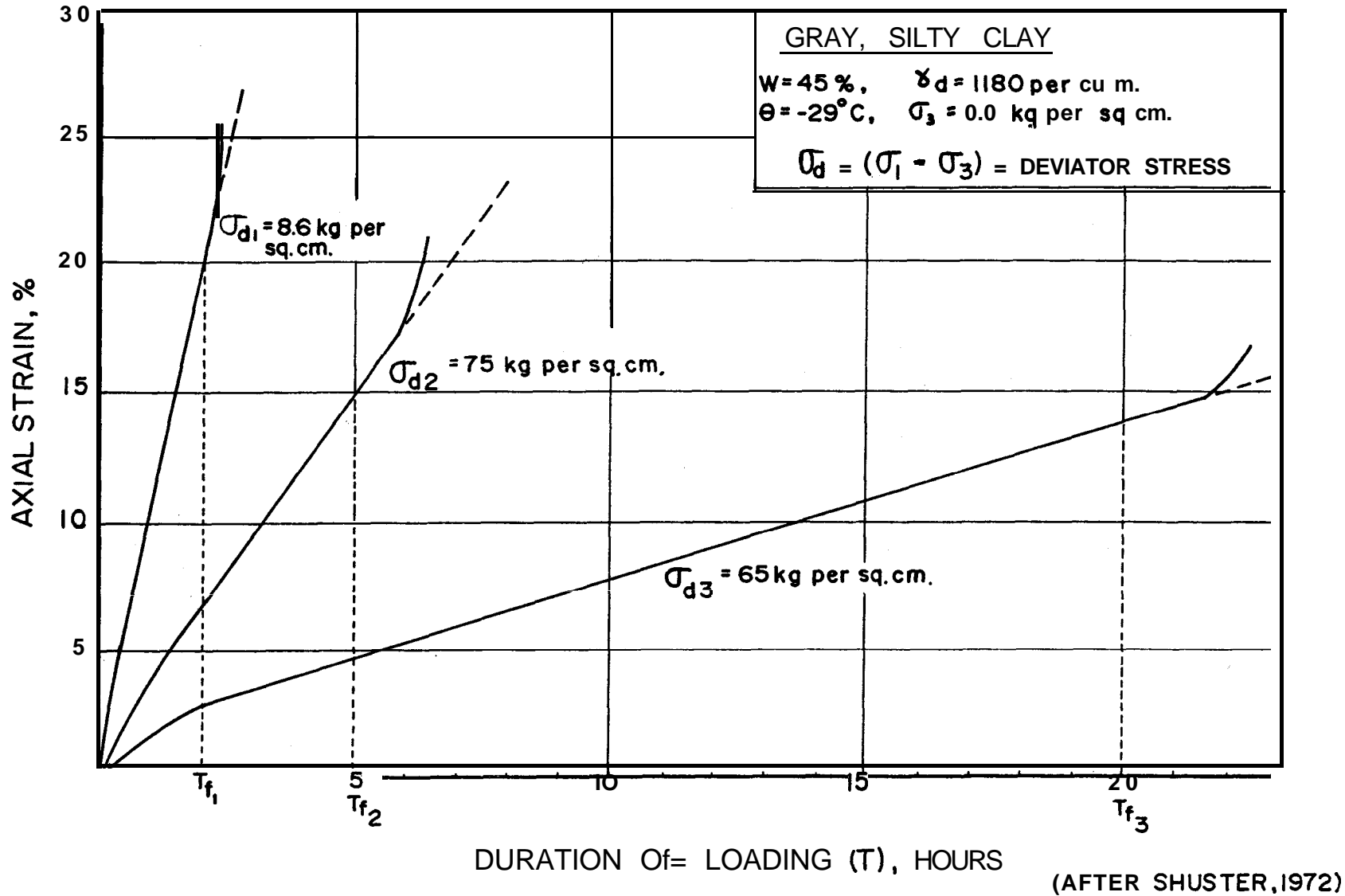
Figure 138. Strain versus time and loading for a frozen soil,
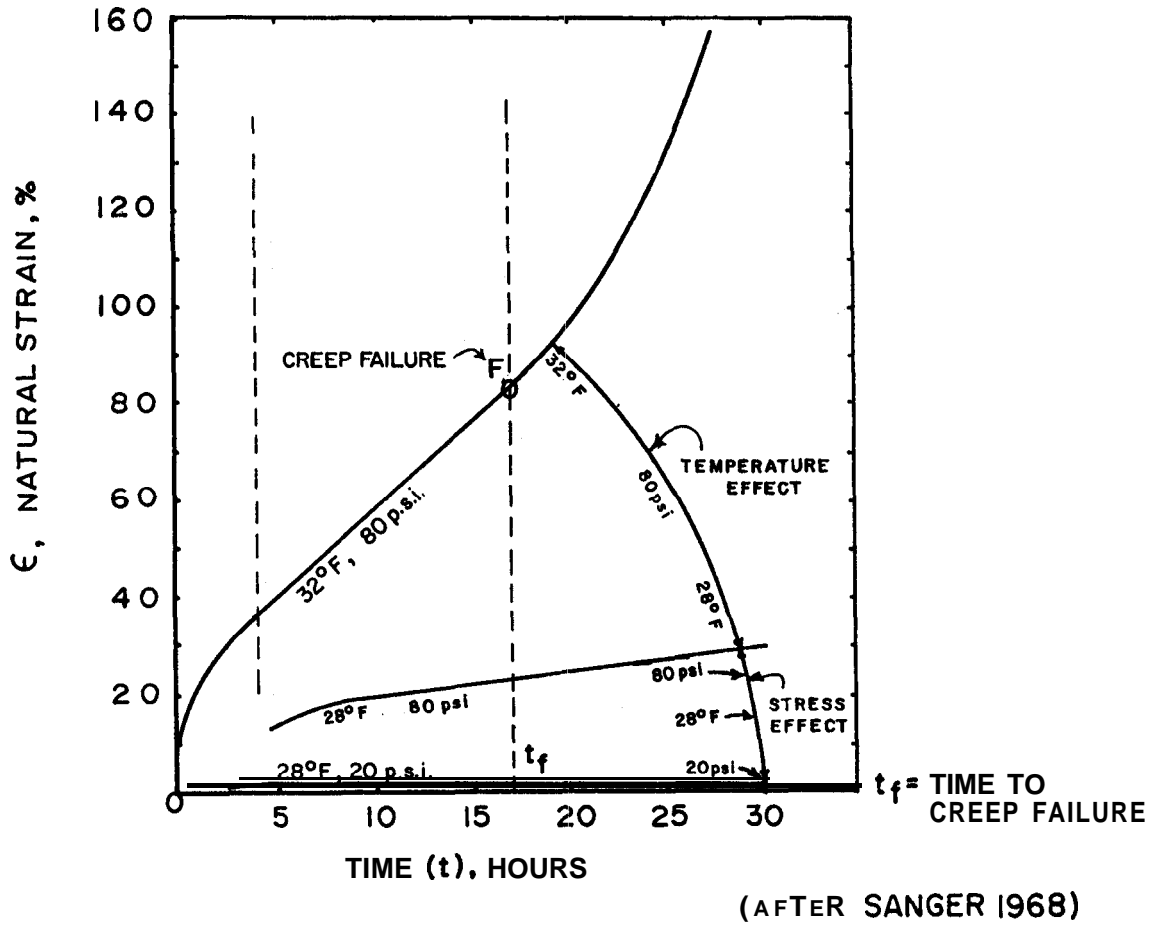
Figure 139. Creep curves for an organic silty clay with temperature influences.

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.

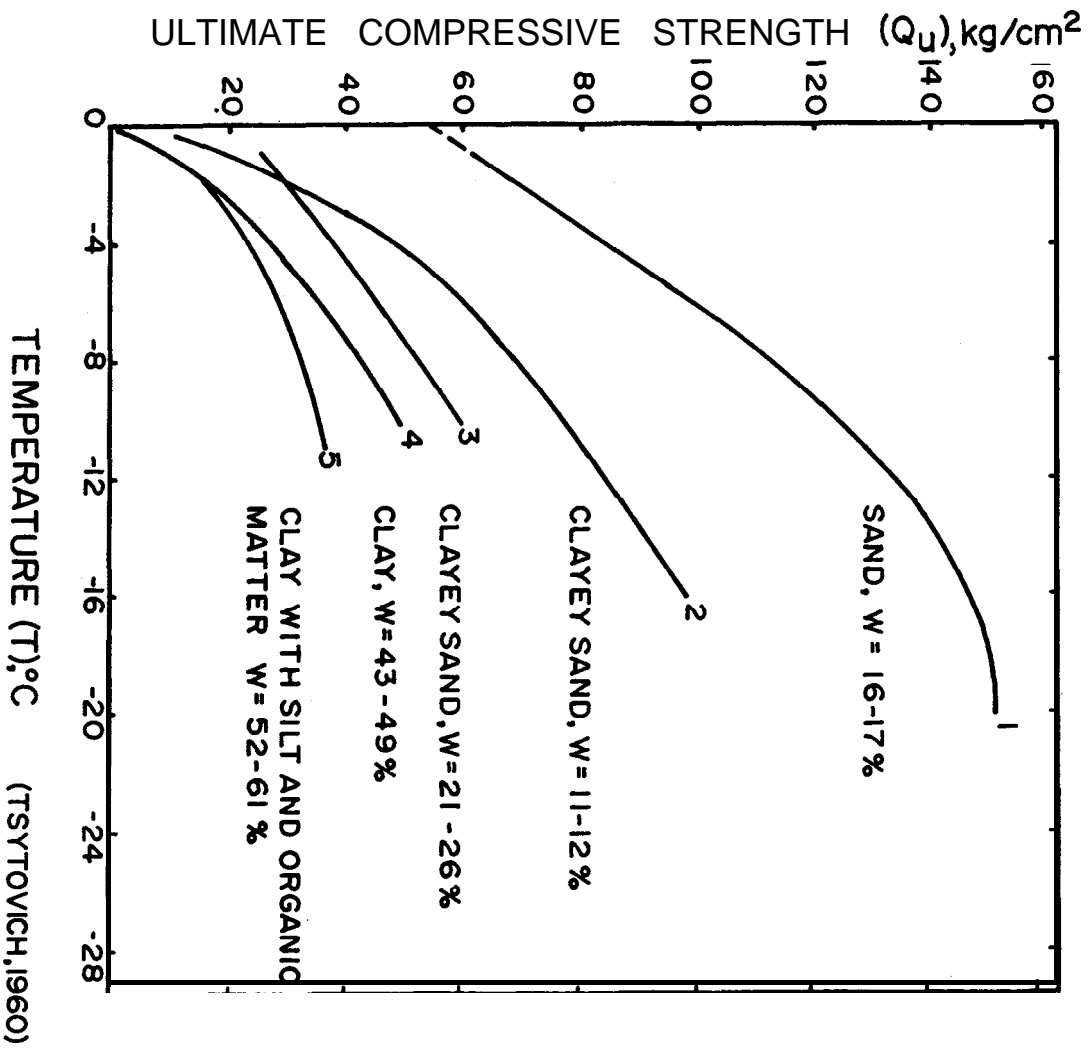4. Fringe losses at the freezing front due to ground water flow.

Figure 140. Ultimate short term compressive strength as a function of temperature.
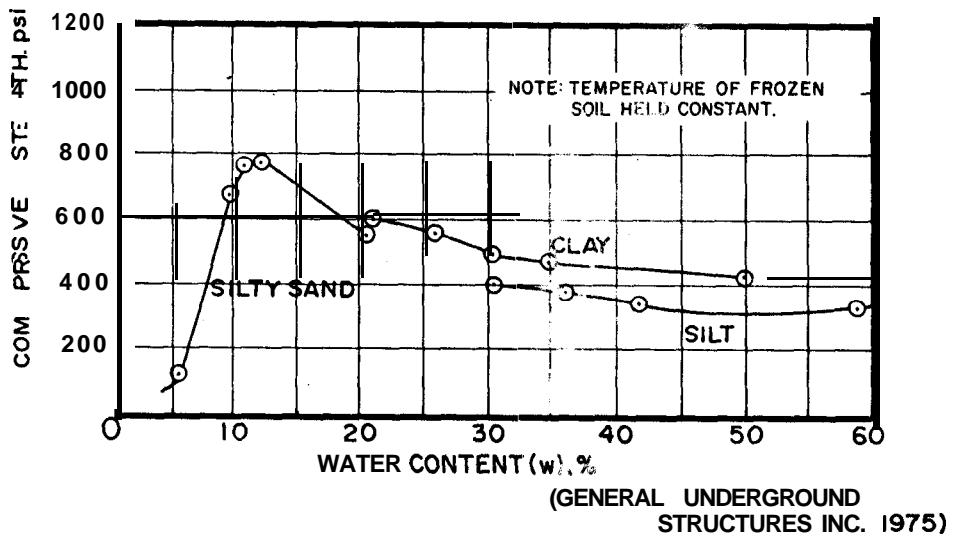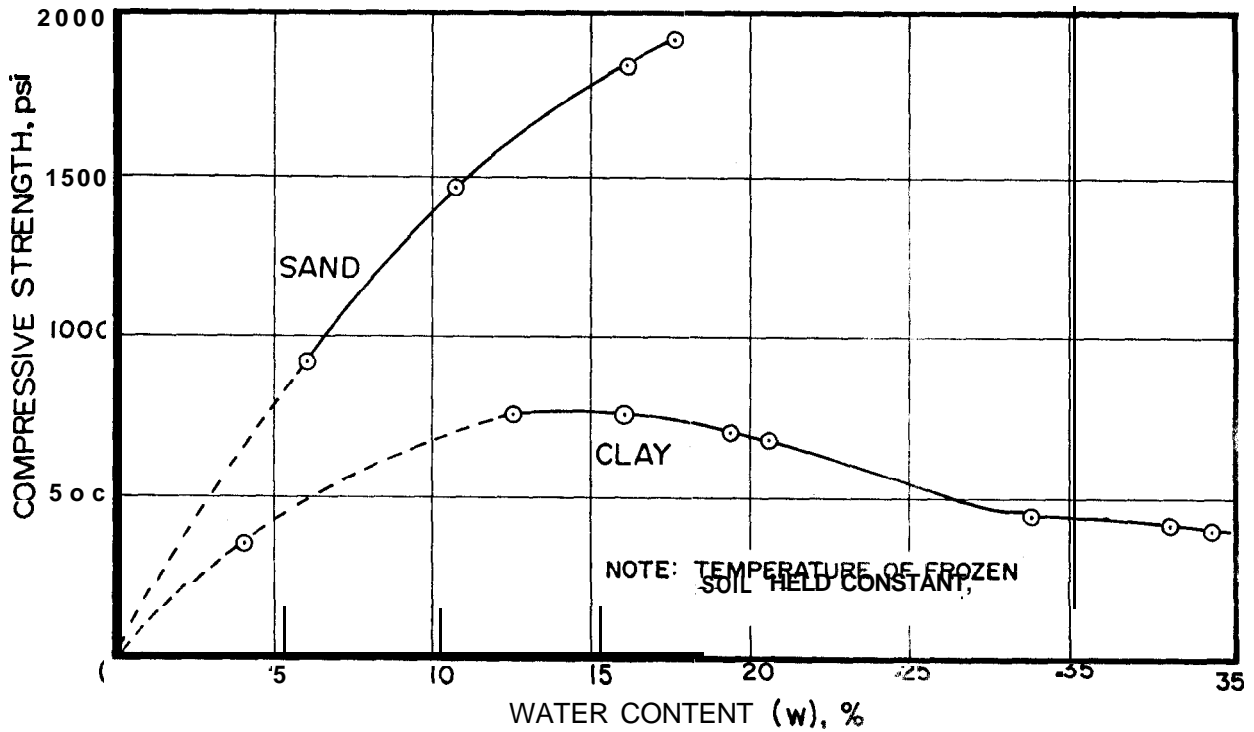
-301-

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_0 = 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 $-65^{\circ}C$ can be obtained. The most common system uses calcium chloride as the brine with a minimum temperature of $-40^{\circ}C$.

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.
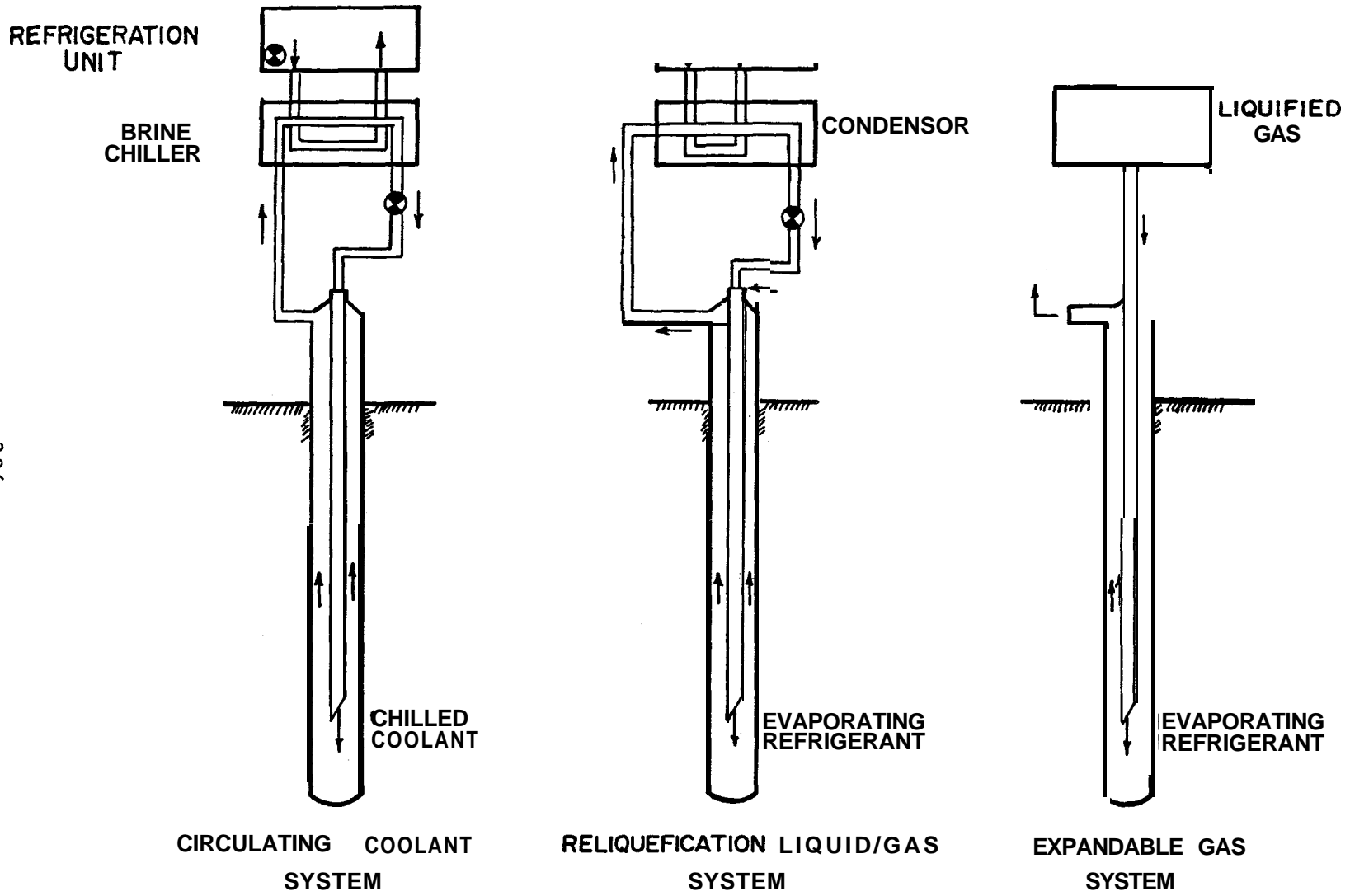
REFRIGERATION
UNIT

BRINE
CHILLER

CHILLED
COOLANT

CONDENSOR

EVAPORATING
REFRIGERANT

LIQUIFIED
GAS

EVAPORATING
REFRIGERANT

CIRCULATING   COOLANT
SYSTEM

RELIQUEFICATION LIQUID/GAS
SYSTEM

EXPANDABLE GAS
SYSTEM

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 $-20^{\circ}C$ to $-55^{\circ}C$. 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_0) 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
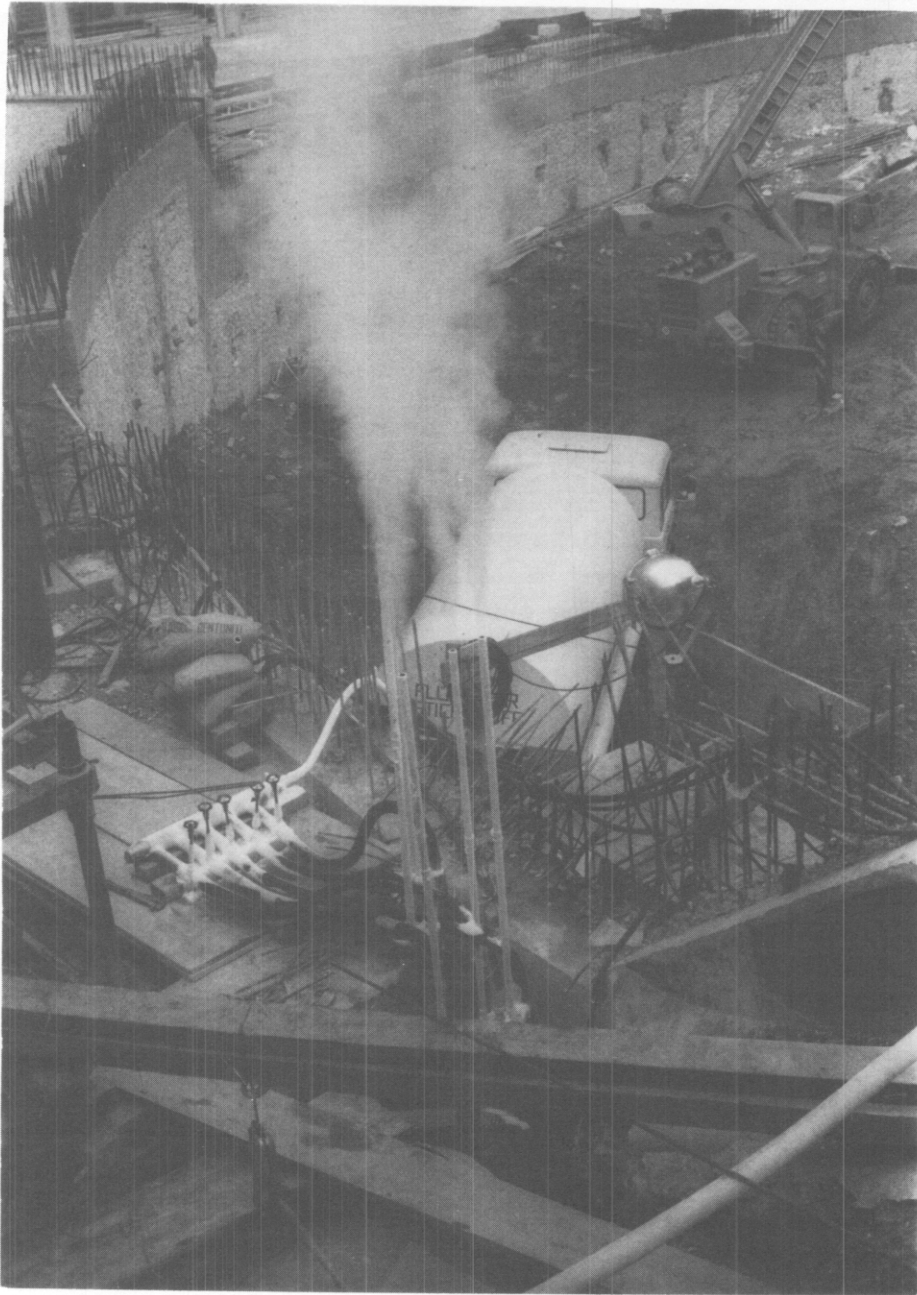
Note: Manifold for liquid nitrogen. Nitrogen gas being vented to atmosphere.

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 $32^{o}F$ isotherm remains in the insulation.
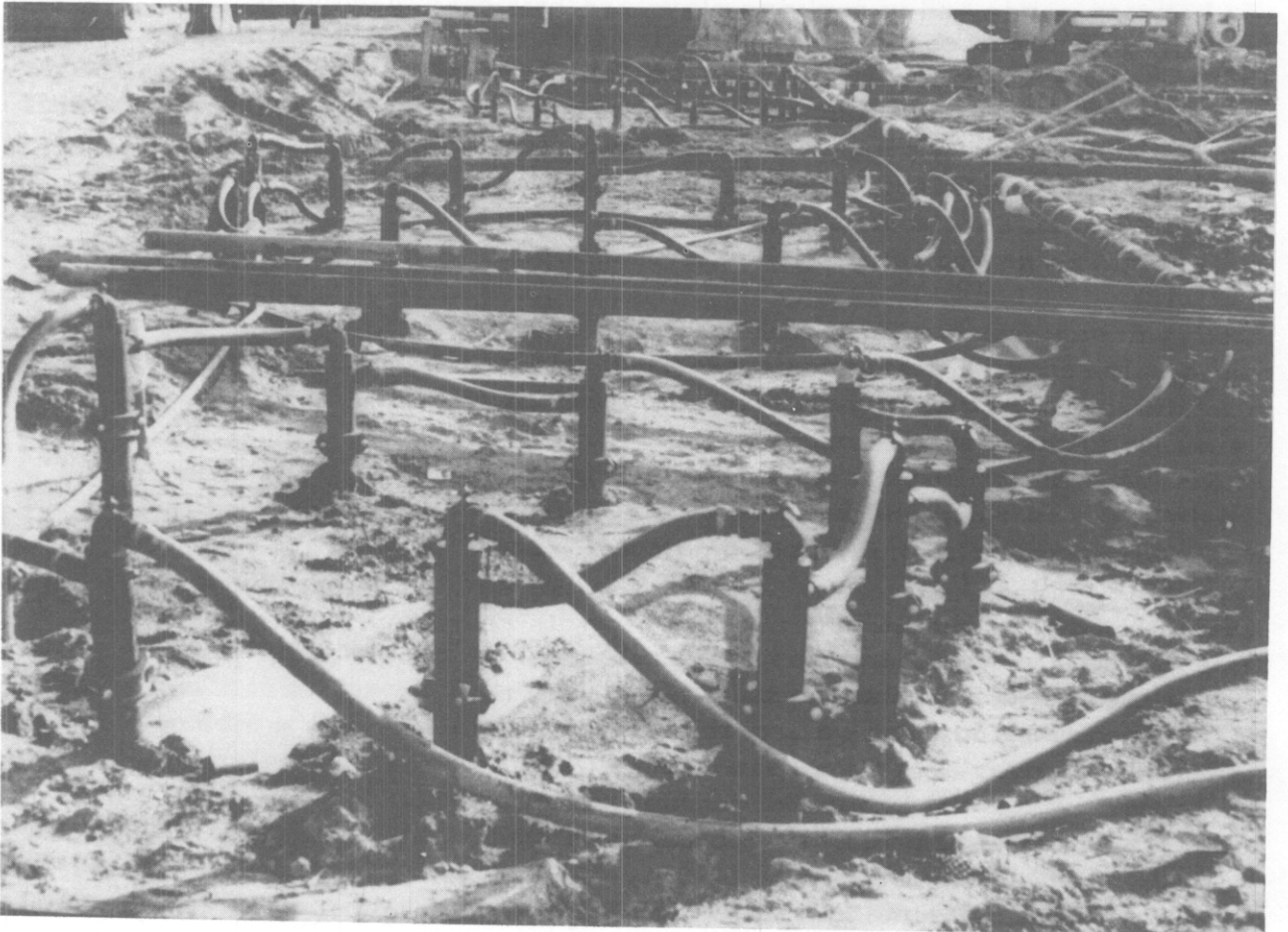
Note: Each group of freeze pipes forms a series of loops from brine supply line back to the return line.

Figure 144. Typical supply and return connections between group of freeze pipes using brine. (Courtesy of Terrafreeze Corporation).
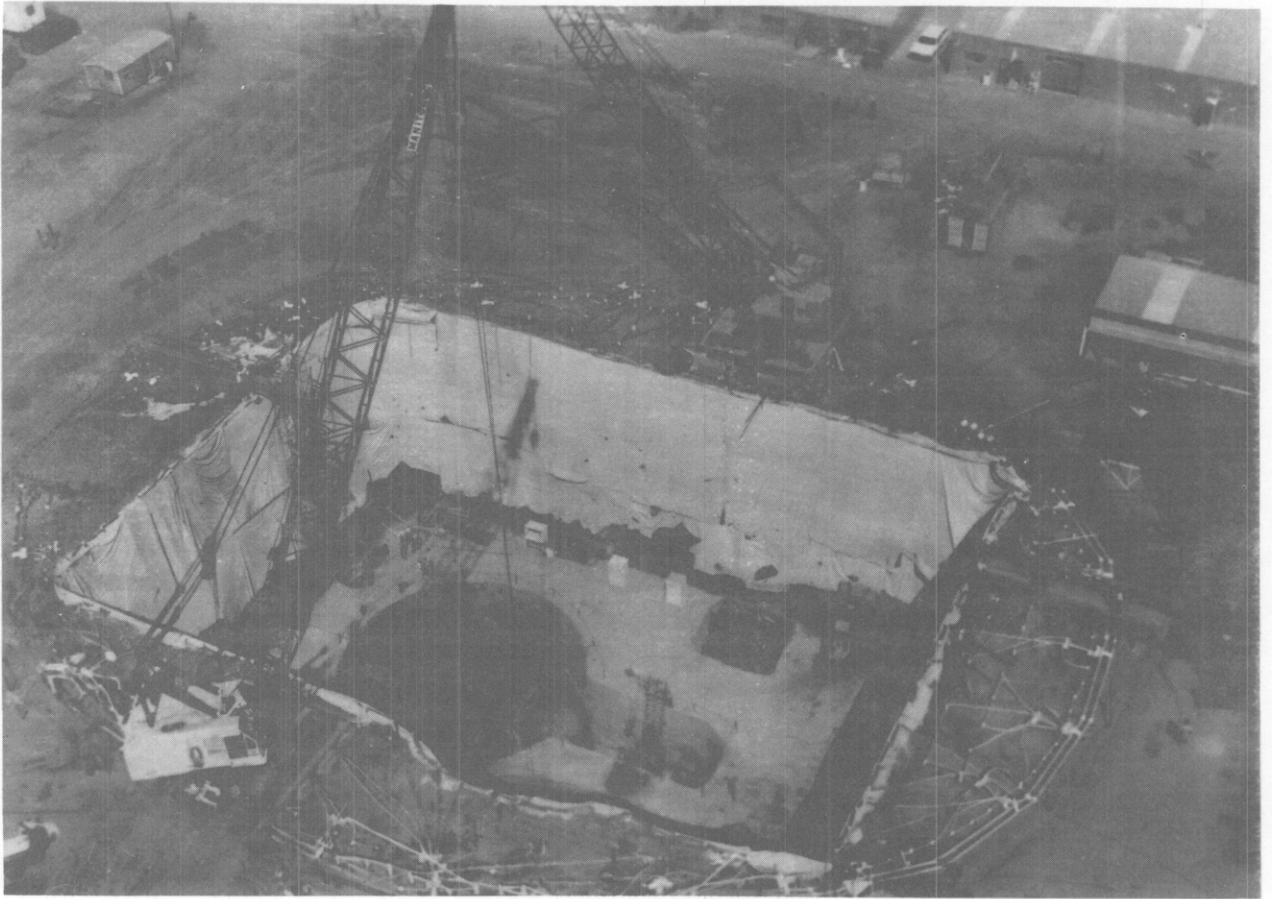
Note: Wall is protected by reflective thermal insulation.

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.