Design Manual for Permanent Ground Anchor Walls

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FOREWORD

This report, Design Manual for Permanent Ground Anchors, gives recommendations and guidlines for improving the design and construction of permanent ground anchor walls. The reccommendations are based on detailed results reported in companion works (Summary Report of Research on Permanent Ground Anchor Walls. Volume I: Current Practice land Limiting Equilibrium Analysis; Volume II: Full-Scale Wall Tests and Soil Structure Interaction Model; Volume III: Model-Scale Wall Tests and Ground Anchor Tests; Volume IV: Conclusions and Recommendations). The manual includes design examples in coarse-grained and fine-grained soils.

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16. Abstract

This manual presents recommendations and guidelines to improve the design and construction of permanent ground anchor walls. The recommendations are based on research performed on two full-scale wall sections, four model-scale walls, 10 large-diameter ground anchors installed in a fine-grained soil, analytical studies, and experience. It focuses on tiedback soldier beam walls for highway applications. These walls are generally less than 25 ft high, and they are supported by one or two rows of permanent ground anchors.

Apparent earth pressure diagrams are recommended for walls supported by one or multiple levels of anchors. Laterally loaded pile relationships that consider interaction between adjacent soldier beams are used to determine the lateral resistance for the soldier beam toes. Recommendations for using limiting equilibrium analyses to evaluate internal and external stability, and to determine the required support load are made. Design examples in coarse-grained and fine-grained soils are provided.

The results of the research are presented in four volumes entitled:

FHWA-RD-98-065	Volume I	Current Practice and Limiting Equilibrium Analyses
FHWA-RD-98-066	Volume II	Full-scale Wall Tests and a Soil-structure Interaction Model
FHWA-RD-98-067	Volume III	Model-scale Wall Tests and Ground Anchor Tests
FHWA-RD-98-068	Volume IV	Conclusions and Recommendations

In addition, a computer program was developed. It is entitled:
FHWA-RD-98-093 TB Wall—Anchored Wall Design and Analysis Program for Personal Computers

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3

NOTE: Volumes greater than 1000 l shall be shown in m^3

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	l
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
psi	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	ac
ha	hectares	2.47	acres	mi^2
km^2	square kilometers	0.386	square miles	
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	megagrams	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	psi

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised August 1992)

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LIST OF ABBREVIATIONS AND SYMBOLS

A	=	Peak earthquake acceleration
A_S	=	Ground anchor tendon area
A_S	=	Surface area of the soldier beam
A_s	=	Block perimeter surface area of a driven soldier beam
A_s	=	Surface area of a drilled shaft
A_t	=	Block end bearing area of a soldier beam tip
A_t	=	Cross-sectional area of the drilled shaft
AASHTO	=	American Association of State Highway and Transportation Officials
AEP	=	Apparent earth pressure
AGC	=	Association of General Contractors
ARTBA	=	American Road and Transportation Builders Association
ASTM	=	American Society of Testing and Materials
B	=	Excavation width
B'	=	Width of soil block exerting a vertical stress at the level of the excavation
B/L	=	Ratio of the width to length of an excavation
b	=	Soldier beam width or drilled shaft diameter
c_a	=	Adhesion between the clay and the soldier beam
c'	=	Cohesion intercept
D	=	Depth to a stiff stratum below the bottom of the excavation
D	=	Toe depth
DOT	=	Department of Transportation
d	=	Depth internal stability failure plane passes below the bottom of the excavation

LIST OF ABBREVIATIONS AND SYMBOLS
(continued)

d	=	Toe depth
d_i	=	Distance where the failure wedge from two adjacent soldier beam toes intersect
d	=	Depth to a stiff stratum below the bottom of the excavation
d/H	=	Depth of weak soil below the excavation/height of cut
EI	=	Beam stiffness
EPF	=	Earth pressure factor
E_s	=	A Secant modulus on the soil's stress-strain curve
E_s	=	Young's modulus for anchor tendon
FHWA	=	Federal Highway Administration
FS	=	Factor of safety
F_p	=	Passive resistance
F_x	=	Force in the x-direction
F_y	=	Force in the y-direction
F_{yield}	=	Yield strength of the anchor tendon
f_s	=	Average unit skin friction
H	=	Excavation depth or height of wall
H/B	=	Depth/width ratio for an excavation
HP	=	H-pile
h	=	Height of wall
h_1	=	Depth of excavation to allow the installation of the upper ground anchor
I	=	Anchor inclination
K	=	A reduction factor to apply to s_u to give the adhesion between the soldier beam and the clay

LIST OF ABBREVIATIONS AND SYMBOLS
(continued)

K	=	Lateral earth pressure coefficient
K_A, K_a	=	Coefficient of active earth pressure
K_o	=	Coefficient of at-rest earth pressure
K_p	=	Coefficient of passive earth pressure
K_{reqd}	=	Required earth pressure coefficient
k	=	Anchor tendon stiffness
k_h	=	Horizontal acceleration from an earthquake
k_{max}	=	Maximum earthquake-induced acceleration
k_y	=	Earthquake acceleration where the external stability factor of safety is 1
L	=	Span distance
L_u	=	Effective elastic length of an anchor tendon
l	=	Effective facing span
M	=	Moment
MSE	=	Mechanical Stabilized Earth
m	=	Term in an equation for estimating the drained shear strength of an overconsolidated clay
NCHRP	=	National Cooperative Highway Research Program
N_c	=	Bearing capacity factor
N_q	=	Bearing capacity factor
N_s	=	Stability number
OCR	=	Overconsolidation ratio
P	=	Load
P_p	=	Passive resistance from the soil

LIST OF ABBREVIATIONS AND SYMBOLS
(continued)

P_{reqd}	=	External force required for stability
p	=	Unit load
pcf	=	Pounds per cubic foot
psf	=	Pounds per square foot
p_{AEP}	=	Apparent earth pressure
p_a	=	Active earth pressure
p_{active}	=	Active earth pressures below the bottom of the excavation
Q	=	Applied axial load
Q_s	=	Axial resistance due to skin friction
Q_t	=	Axial resistance due to end bearing
Q_{ult}	=	Ultimate axial load-carrying capacity
q	=	Unit end bearing resistance
q_b	=	Unit end bearing resistance
$q_{applied}$	=	Applied vertical pressure
R	=	Reaction vector in a force polygon
R	=	Resistance
R	=	Toe reaction from an apparent earth pressure diagram
R_a-y curve	=	Curve that describes the load and deflection relationship for the active side of the toe of a soil-soldier beam system
R_p-y curve	=	Curve that describes the load and deflection relationship for the passive side of the toe of a soil-soldier beam system
SPT	=	Standard penetration resistance
SP_h	=	Horizontal resistance from the wall
SP_v	=	Vertical resistance from the wall

LIST OF ABBREVIATIONS AND SYMBOLS
(continued)

s = Shear strength

s_c = Clear spacing between soldier beams

S_{cr} = Spacing between two soldier beams where the passive resistance at the toe changes from single soldier beam behavior to group behavior

s_u = Undrained shear strength

s_{ub} = Strength of the soil providing bearing resistance

T = Ground anchor load

T_s = Anchor lock-off load

T_u = Ultimate ground anchor tendon capacity

T_y = Ground anchor tendon capacity at its yield strength

T-y curve = Curve that describes the load elongation behavior of a ground anchor

tsf = Tons per square foot

W = Weight vector in a force polygon

y = Deflection

y_a = Active resistance deflection

y_b = Bulging deformations

y_c = Cantilever deformation

y_p = Passive resistance deflection

y_u = Wall deflection when the anchor tendon ruptures

y_y = Wall deflection when the anchor tendon yields

y_s = Wall deflection after anchor stressing (lock-off)

y_0 = Wall deflection when the anchor load is zero

α = Angle of the internal stability failure surface with respect to a horizontal plane

LIST OF ABBREVIATIONS AND SYMBOLS
(continued)

α = Adhesion factor

α = Anchor inclination

α = Angle with respect to a vertical plane that defines the shape of a failure wedge for the toe of a soldier beam in sand

β = Angle with respect to a horizontal plane that defines the shape of a failure wedge for the toe of a soldier beam in sand

β = Factor that defines the skin friction for a drilled shaft in terms of effective stress

γ = Total unit weight

γ_{ave} = Average total unit weight of soil

γ' = Effective unit weight of soil

δ = Angle of friction between soil and soldier beam or wall

θ = Angle that defines the shape of a failure wedge for the toe of a soldier beam in clay

ξ = Ratio of the depth of the internal stability failure surface below the bottom of the excavation to the final depth of the excavation

ϵ_{rupt} = Rupture strain

σ' = Effective normal stress

σ'_v = Effective overburden pressure

ϕ = Angle of internal friction, friction angle

ϕ' = Drained friction angle

ϕ_{mob} = Mobilized friction angle

CHAPTER 1: INTRODUCTION

1.1 PURPOSE, SCOPE, AND LIMITATIONS OF THIS MANUAL

This manual was developed to improve the design and construction of permanently anchored walls for highway applications. Guidelines for designing, specifying and inspecting permanent anchored soldier beam walls are provided. Existing design methods were evaluated and adopted where appropriate. New methods were developed on the basis of research, analytical studies, and experience. Long, et al. (1998), Weatherby, et al. (1998), Mueller, et al. (1998), and Weatherby, (1998) present the results of the research and analytical studies.

The scope of this manual includes:

Chapter 1	Descriptions of anchored walls and typical applications.
Chapter 2	Recommendations for site investigation, technical and economic feasibility evaluations, and cost estimating.
Chapter 3	Recommendations for selecting a contract delivery method.
Chapter 4	Apparent earth pressure diagrams are recommended for walls supported by one or multiple levels of anchors. The shape of the apparent earth pressure diagram is determined by the location of the supports. Using limiting equilibrium analyses to determine the lateral earth load.
Chapter 5	Designing a wall that can be built.
Chapter 6	Designing the soldier beam toe to carry axial and lateral loads.
Chapter 7	Recommendations for using soil-structure interaction analyses for the soldier beam walls.
Chapter 8	Determining the internal and external stability of a wall.
Chapter 9	Estimating wall and ground movements, selecting corrosion protection for ground anchors and soldier beams, preventing frost pressures, designing landslide stabilization walls, surcharge loads, barrier loads, facing design, and seismic design.
Chapter 10	Recommended design procedure and examples.
Chapter 11	Modifications to current specifications. Recommended specification sections for a contractor prepared detailed designs and soldier beam installation.
Chapter 12	Recommendations for construction inspection.
Chapter 13	Monitoring the performance of ground anchor walls.

Some of the research findings were incorporated in a computer code. The computer program is named *TB Wall—Anchored Wall Design and Analysis Program for Personal Computers* (Urzua and Weatherby, 1998).

The manual focuses on flexible earth retaining walls (walls with soldier beams or steel sheet piles). Apparent earth pressure diagrams apply to these types of walls. Some recommendations may not apply to stiff structural diaphragm walls, or walls where low-strength soils extend for considerable depth below the bottom of the wall. The recommendations presented are intended to apply to permanent ground anchor walls for typical highway applications. They were not developed for temporary earth support systems, but many principles presented apply to both permanent and temporary construction.

1.2 DESCRIPTION OF ANCHORED WALLS

Permanent ground anchor wall systems, often called tiedback walls, use tensile elements anchored in the ground to support earth retaining structures or stabilize landslides. These walls are built in excavated cuts from the top down. For most highway applications, ground anchor walls consist of anchored soldier beams with temporary wood lagging and a permanent cast-in-place concrete face. Figure 1 shows a typical anchored wall and identifies major components of the wall. Soldier beams distribute the ground anchor load to the ground and support the earth at the face of the cut. The components of a ground anchor are shown in Figure 2.

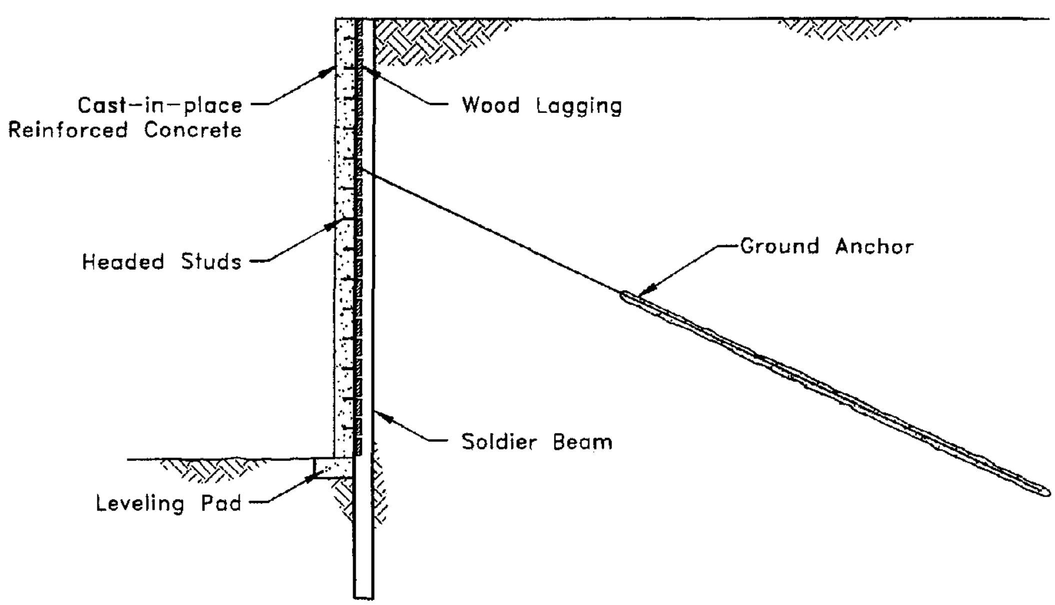


FIGURE 1
Permanent Ground Anchor Retaining Wall

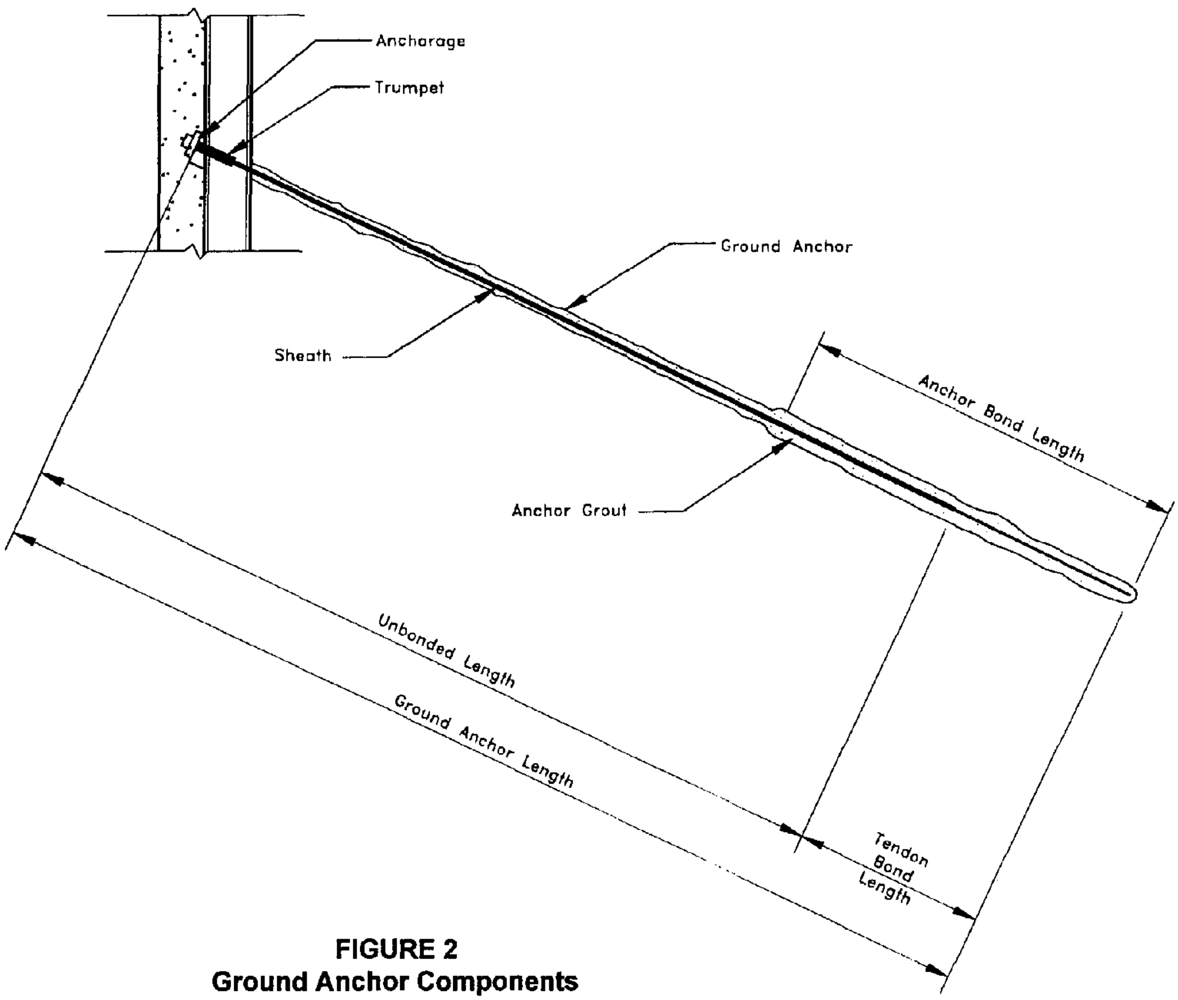


FIGURE 2
Ground Anchor Components

The steps involved in constructing a permanently anchored soldier beam wall are shown in Figure 3. First the soldier beams are driven or drilled into the ground from the existing ground surface. After the soldier beams are installed, the excavation proceeds to the first ground anchor level. As the excavation is made wood lagging or shotcrete is applied to support the ground between the soldier beams. Normally, the wood lagging or shotcrete supports the ground temporarily. Next, the ground anchors are installed. They are made by driving or drilling a hole into the ground behind the wall. After the hole has reached the desired depth, a prestressing steel tendon is grouted into the ground. A grouted anchor fixes the tendon to the ground at the far end. After the cement grout has cured, the ground anchor is load tested and locked-off to the soldier beam. Then the excavation and placement of lagging or shotcrete continues to the next anchor level or the bottom of the cut. If additional ground anchors are required, the steps described above are repeated.

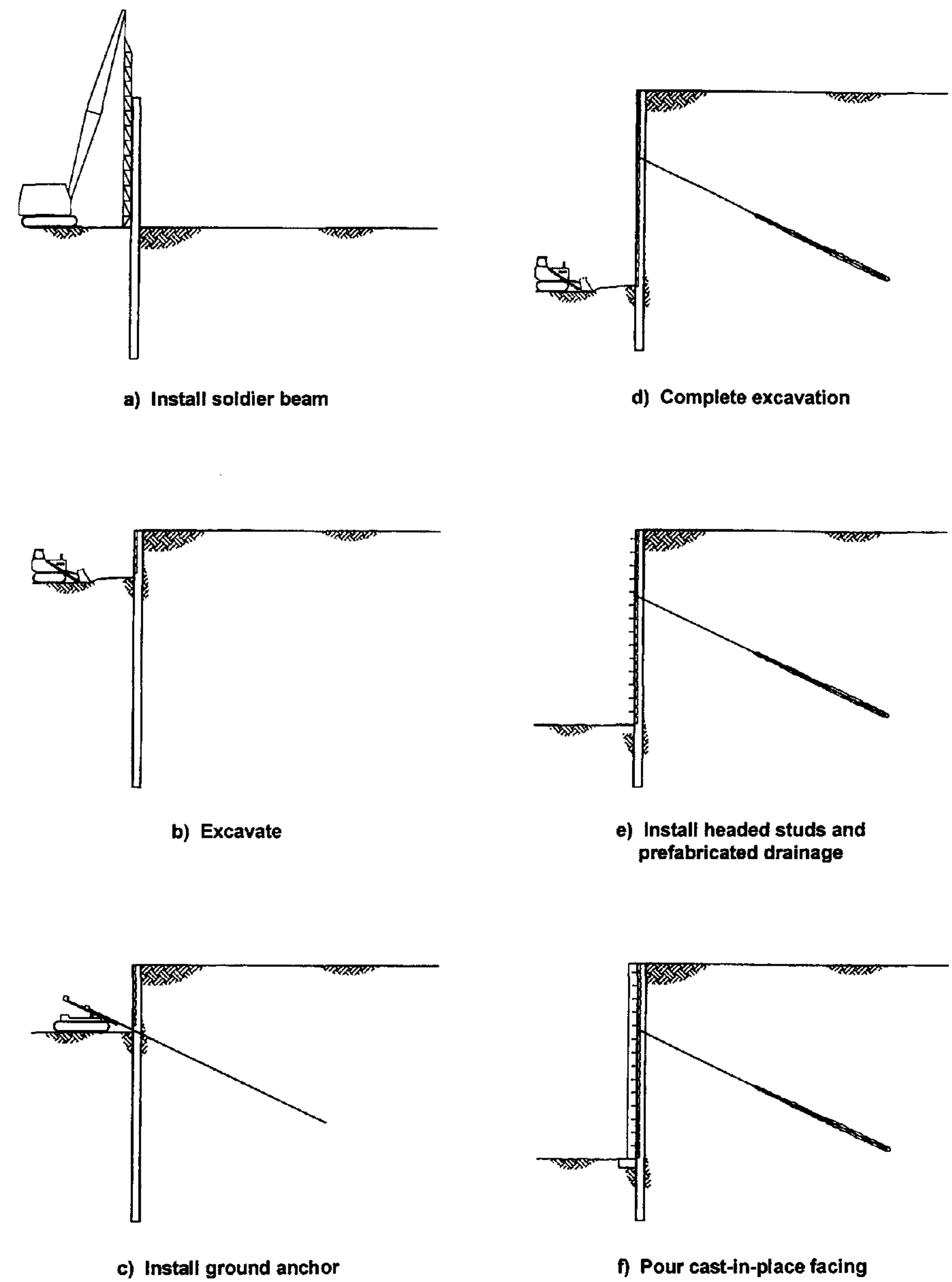


FIGURE 3
Construction Steps for Ground Anchor Wall

After the excavation is completed, prefabricated drains are attached to the lagging or shotcrete. An unreinforced concrete leveling pad is often cast at the bottom of the wall. The pad enables the wall forms to be easily set. A permanent, reinforced, cast-in-place concrete face is constructed from the bottom up. Headed studs are used to attach the concrete face to steel soldier beams. Grouted or epoxied dowels are used with drilled-in, reinforced concrete soldier beams.

Driven steel sheet piling, soldier piles in a deep soil mixed trench or structural diaphragm walls are occasionally used for the vertical elements of anchored walls. These walls are used when it is necessary to cut off groundwater from behind or under the wall. Sheet piling or deep soil mixed walls have been used when the ground between soldier beams will not support itself long enough to install lagging or shotcrete.

Stiff walls are used in attempts to minimize ground movements behind the wall. Wall stiffness is affected by the beam stiffness (EI) of the vertical elements and the spacing of the ground anchors. A wall with closely spaced anchors can be stiffer than a wall with a high EI and large anchor spacings. Large EI can be obtained by using cast-in-place diaphragm walls or closely spaced drilled-in soldier beams.

In competent ground, horizontal beams or isolated elements can be anchored. The beams or elements distribute the anchor force to the ground. Figure 4 shows a permanent ground anchor wall constructed for the realignment of State Route 91 near Brigham City, Utah. This patented wall uses anchored horizontal beams and precast concrete panels. Shotcrete and soil nails were used to temporarily support the ground between the horizontal beams.

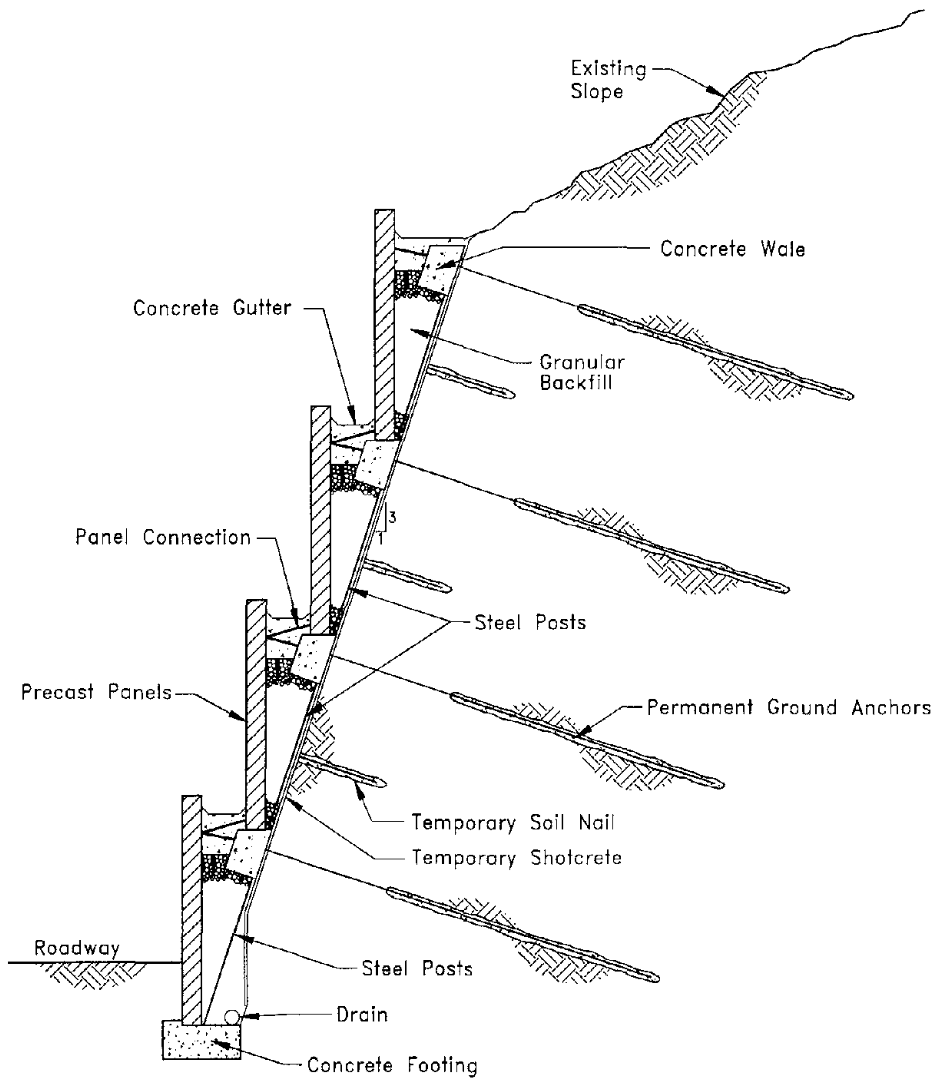


FIGURE 4
Anchored Wall with Precast Panels and Horizontal Wales
(U.S. Patent 5,551,810)

1.3 HIGHWAY APPLICATIONS

Permanent ground anchor retaining walls are a cost-saving alternative to cantilever retaining walls in cuts. Ground anchors are used to stabilize landslides, and anchored walls are used when the end slope under an existing bridge is removed during the widening of a highway. Ground anchors are used to strengthen existing earth retaining structures or when existing walls are replaced.

1.3.1 Retaining Walls

Ground anchor retaining walls are commonly used for roadway widening or realignment. When the new lane(s) or shoulder require that the roadway be constructed into the slope along an existing roadway, a permanent retaining wall is required. If temporary excavation support is required to support the cut for the retaining wall, then a ground anchor retaining wall will be cost-effective.

Figure 5 compares the construction of a conventional retaining wall with a permanent ground anchor retaining wall in a cut situation. The anchored wall has the following advantages over the conventional retaining wall:

- Eliminates the temporary excavation support system.
- Requires less excavation.
- Eliminates footing excavation and concrete.
- Eliminates deep foundations.
- Reduces the quantity of concrete for the wall facing (anchored wall facings are typically 12 in thick).
- Eliminates backfill behind the wall.
- Reduces construction disturbance since a footing is not required.
- Improves public and worker safety since the wall does not require wide construction easements.
- Allows quicker construction.
- Costs less.

Permanent ground anchor retaining walls are used for the construction of depressed roadways in urban areas. The advantages listed above apply to depressed roadways. Figure 6 illustrates how anchored retaining walls for depressed roadways reduce the disruption during construction and simplify the new construction. Construction of a conventional cantilever retaining wall requires closing traffic lanes behind the wall during construction. A permanent ground anchor wall only requires a construction easement of 2 to 3 ft behind the face of the finished wall. Even during soldier beam installation, the traffic behind the wall is not interrupted. A typical permanent ground anchor wall for a depressed roadway is shown in Figure 7.

Figure 8 shows a typical cross-section of a wall for the widening of I-285 and a parallel frontage road near Atlanta, Georgia. Soldier beams were driven through the slope separating the frontage road from the Interstate. As shown, a fill was required to widen the Interstate and a cut was required to widen the frontage road. The wall ranged in height up to 32 ft.

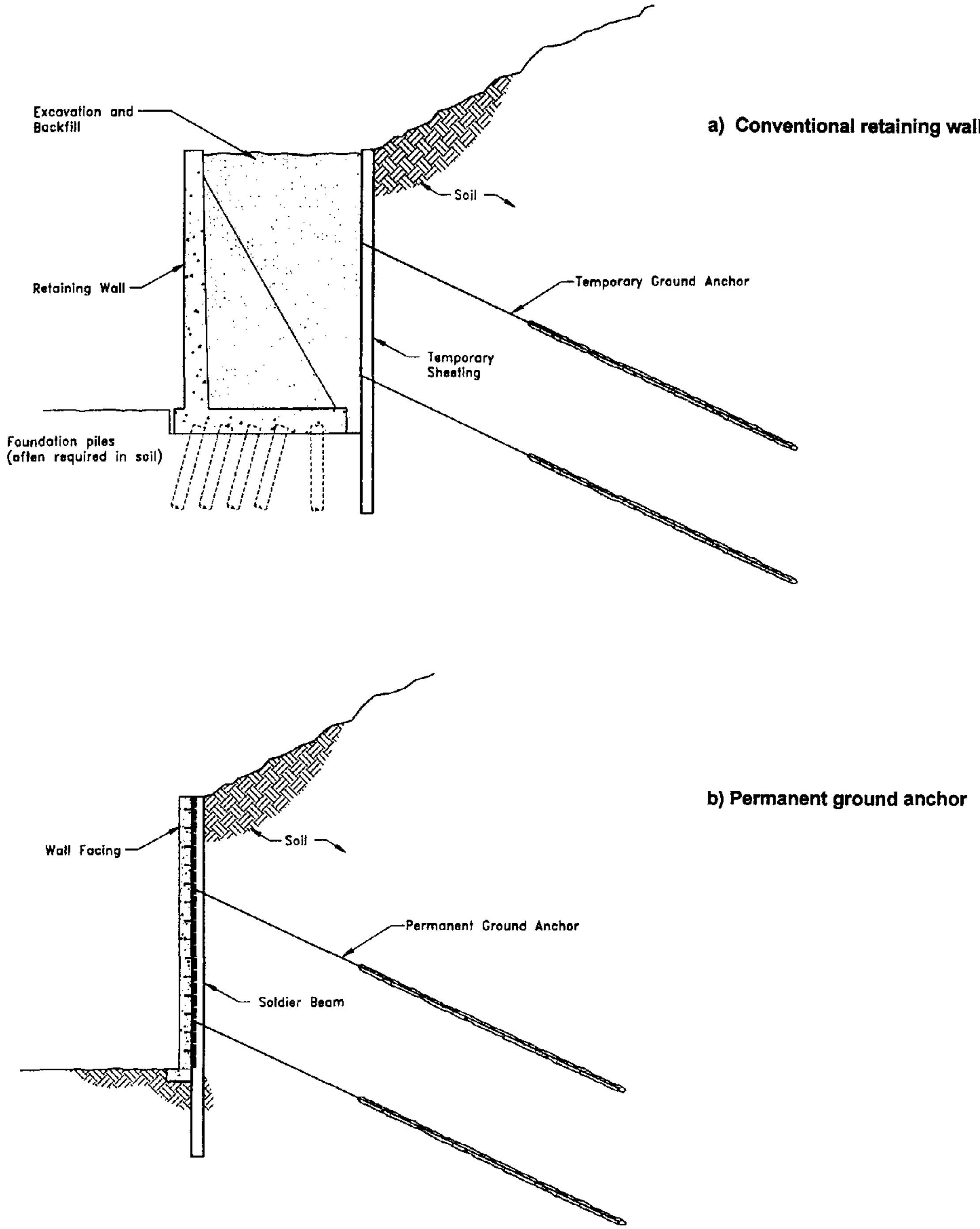
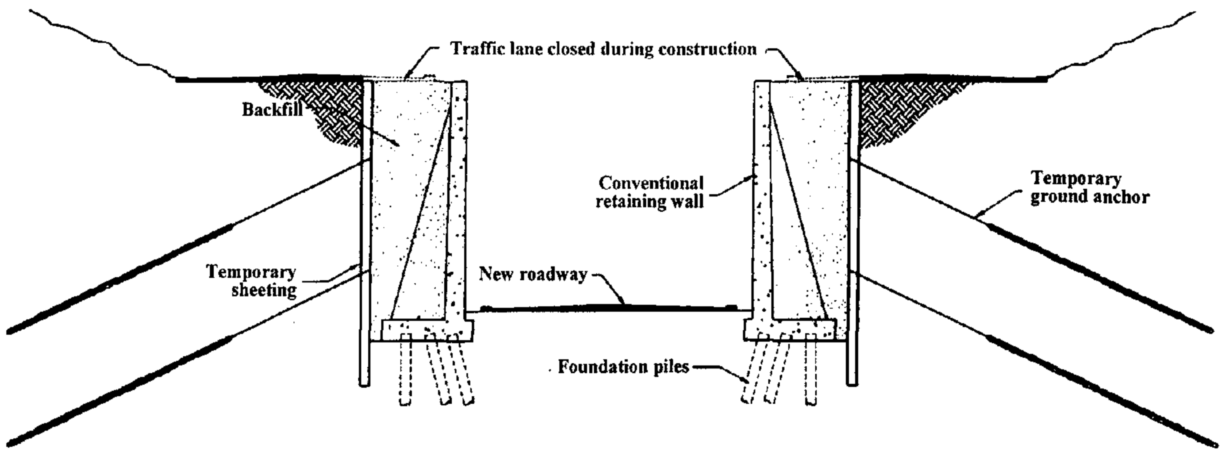
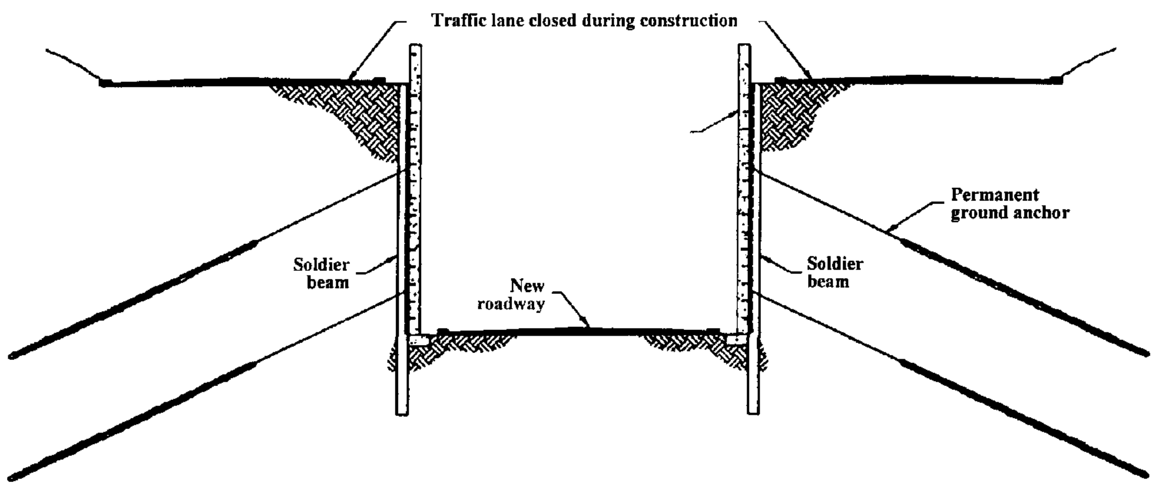


FIGURE 5
Comparison of a Conventional Retaining Wall with a Permanent Ground Anchor Wall



a) Conventional retaining wall



b) Permanent ground anchor

FIGURE 6
Comparison of a Conventional Depressed Roadway Construction and Ground Anchor Wall Construction

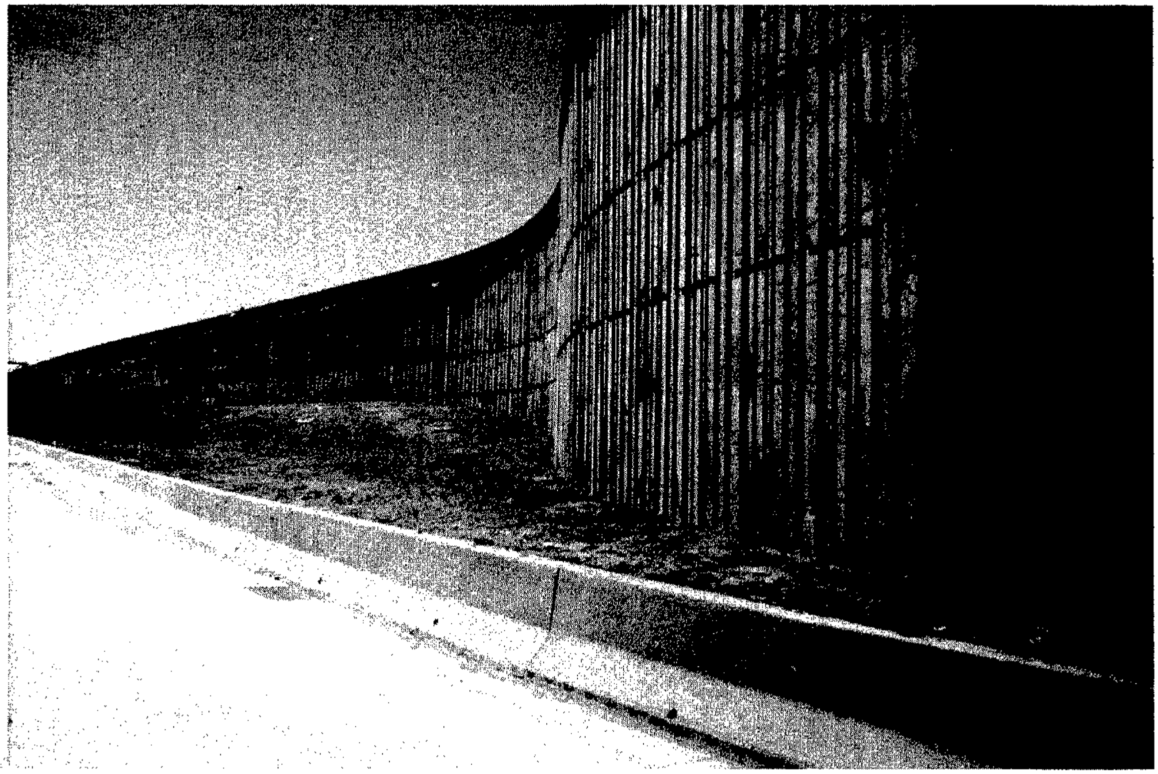


FIGURE 7
Permanent Ground Anchor Wall for a Depressed Portion of I-10, Phoenix, Arizona

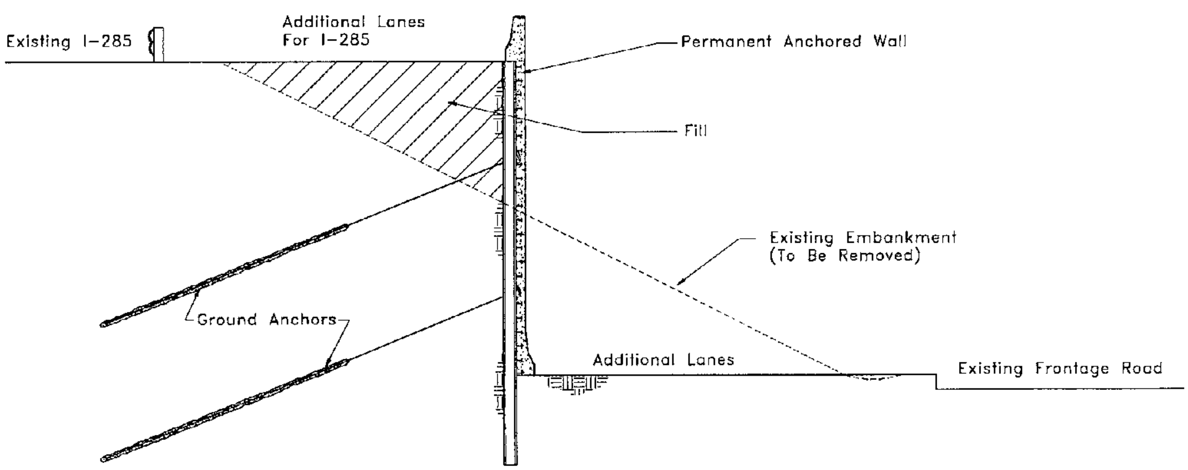


FIGURE 8
Permanent Ground Anchor Retaining Wall for I-285, Atlanta, Georgia

1.3.2 Landslide Stabilization Walls

Permanent ground anchor walls are routinely used to stabilize cut slope and fill slope slides (Figure 9) associated with roadways in mountainous regions. Landslides triggered by changes in the groundwater levels, removal of toe support by erosion or excavation, seismic activity, or reduction of shear strength are stabilized using anchored walls. Rock falls, soil or rock topples and some spread and flows can be stabilized using ground anchors or a combination of ground anchors and other measures.

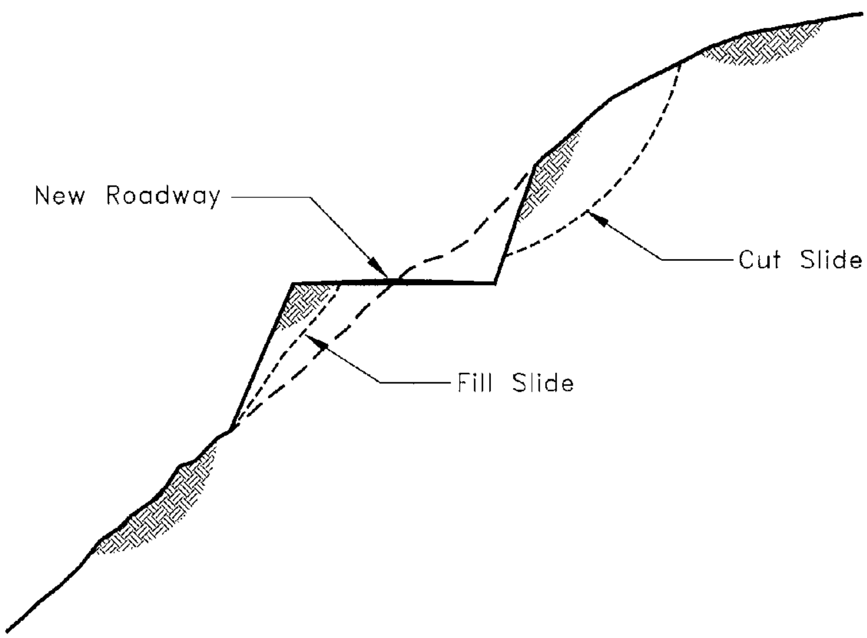


FIGURE 9
Cut and Fill Slope Landslides

1.3.2.1 Soldier Beam Walls

Anchored soldier beam walls are used to stabilize landslides. Driven or drilled-in soldier beams have been used. When access is limited, soldier beams have been placed in hand-dug pits. When the soldier beams do not penetrate the failure surface, multiple rows of ground anchors are necessary. If one row of anchors is used to support the wall, then the soldier beams must penetrate the failure surface. Figure 10 shows a permanent ground anchor wall used to stabilize a fill slide in Parker, Pennsylvania. Two rows of ground anchors supported the soldier beams, which did not penetrate the failure surface.

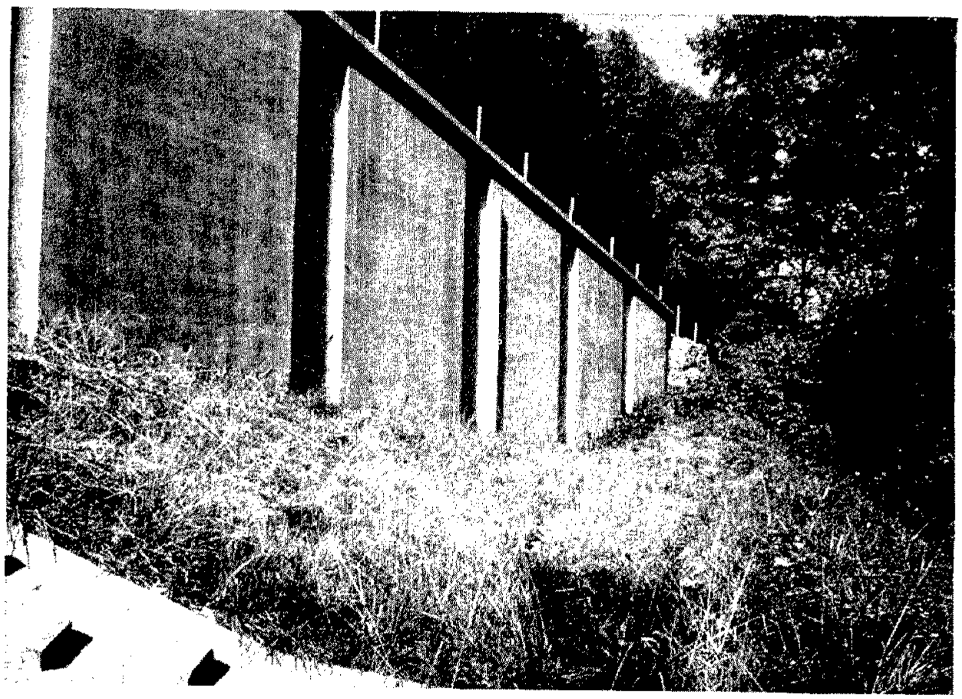


FIGURE 10
Landslide Stabilization Wall with Hand-dug Soldier Beams and Two Rows of Anchors

1.3.2.2 Horizontal Beams or Elements

Many landslides have a well-defined failure surface and competent intact ground above and below the failure surface. Anchored horizontal beams or elements can be used to stabilize these cuts. Figure 11 shows a landslide stabilization wall constructed on California Highway 36 for the Western Federal Lands Division of the Federal Highway Administration (FHWA) in Trinity National Forest, California. Two rows of horizontal shotcrete beams were constructed above and below the roadway. Permanent ground anchors were installed in each beam to stabilize a deep-seated failure. Beams visible from the roadway were encased in sculptured and stained shotcrete. The rock pattern cut into the shotcrete matched the rock structure on nearby outcrops.

FIGURE 11
Landslide Stabilization Using Anchored Horizontal Wales for State Highway 36 in Trinity National Forest, California

1.3.3 Bridge Abutments

The horizontal earth and surcharge loads applied to bridge abutments can be supported by permanent ground anchor walls.

1.3.3.1 New Construction

Permanently anchored bridge abutments are built when a new roadway will be depressed to eliminate an at-grade intersection with another roadway or rail line. Two types of anchored walls are associated with new bridge construction. The wall may be designed to support the superstructure and the earth behind the wall, or the wall may be separated from the bridge foundation. When separate deep foundations are used, the abutment piles or drilled shafts are located behind the wall and installed before the ground anchors.

Figure 12 shows an anchored wall and an abutment for a new bridge under construction for Davidson Freeway in Detroit, Michigan. The anchored wall carried the lateral earth loads and the impact loads, and the wall facing was design to transmit the vertical bridge loads to a spread footing. Anchored walls have been built that carry both the lateral earth loads and the bridge loads. This type of wall is not as common in the United States.

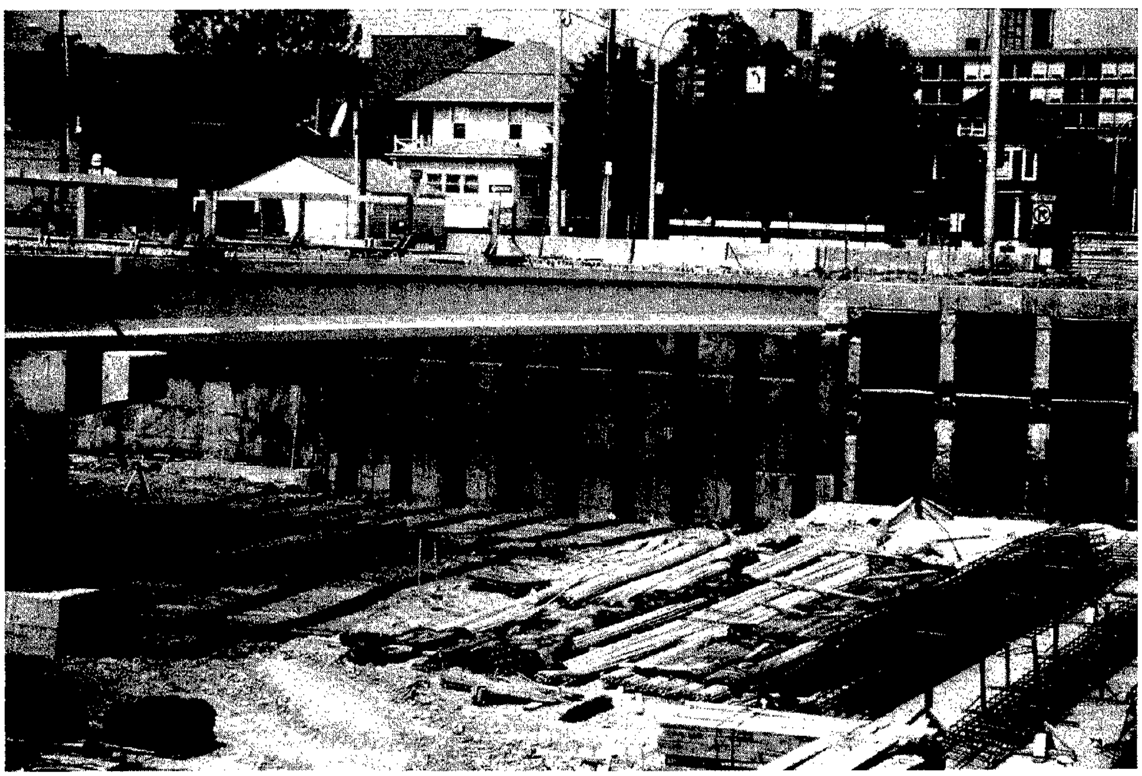


FIGURE 12
Anchored Wall and Bridge Abutment Under Construction on Davidson Freeway, Detroit, Michigan

1.3.3.2 End Slope Removal

Anchored walls are used to enable the slope in front of an existing abutment to be removed. Ground anchors are used when soil nailing is not suitable, when a spread footing abutment is used, or when the nature of the abutment fill is unknown. Soldier beams for these applications are drilled through the existing roadway or installed in hand-dug pits. Pits are dug from the top of the slope to eliminate the disruption of traffic on the bridge. Figure 13 shows a wall constructed under Greenbelt Road (Greenbelt, Maryland) for the extension of a Metro line. The soldier beams under the bridge were installed in hand-dug pits.

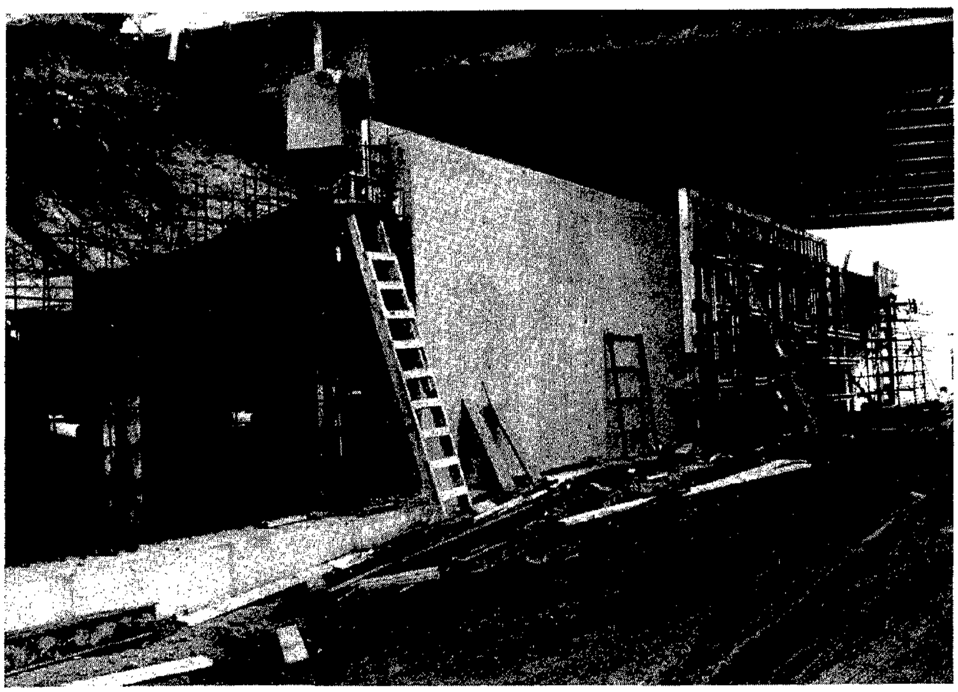


FIGURE 13
Permanent Ground Anchor Wall Used for End Slope Removal Under Greenbelt Road, Greenbelt, Maryland

1.3.4 Replacement or Strengthening of Existing Retaining Structures

Existing earth retaining structures that have deteriorated because of corrosion or require strengthening can be replaced or repaired using permanent ground anchors. Figure 14 shows a permanent ground anchor wall being constructed to replace a deteriorated bin wall. The replacement wall was constructed without disrupting the traffic behind the wall. Details of the replacement wall are protected by U.S. Patent No. 4,911,528. Figure 15 shows the installation of permanent ground anchors for the strengthening of an existing rubble bridge abutment. After the ground anchors were installed, the tracks were lowered, increasing the vertical clearance between the cars and the bridge beams.

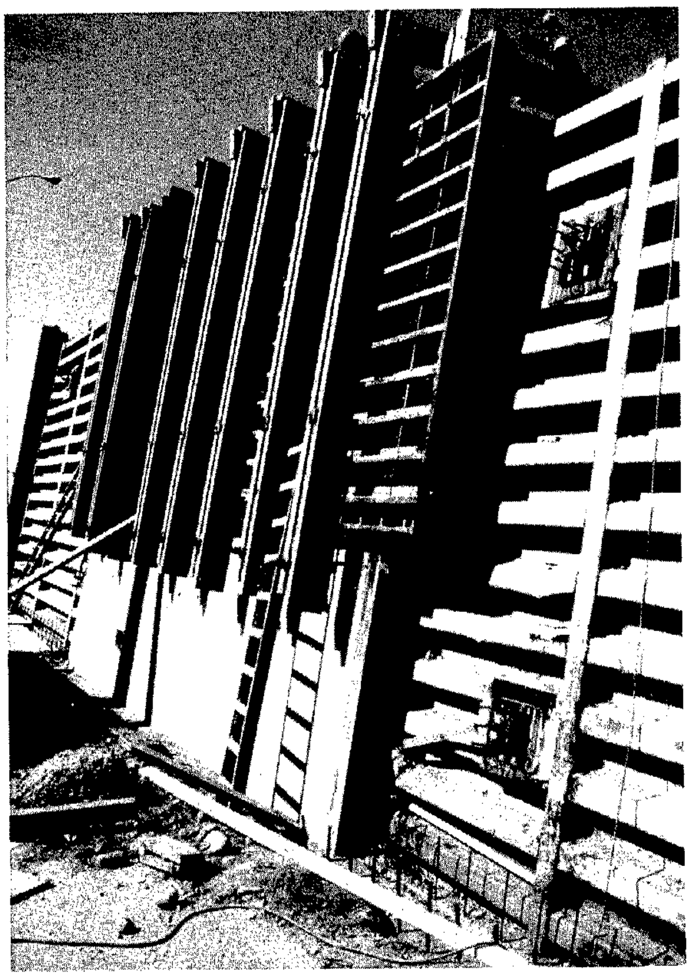


FIGURE 14
Replacing a Deteriorated Bin Wall with a Permanent Ground Anchor Wall
(U.S. Patent 4,911,528)

FIGURE 15
Permanent Ground Anchors Installed to Strengthen Rubble Bridge Abutments on the Orange Line, Boston, Massachusetts

CHAPTER 2:
SITE INVESTIGATION, FEASIBILITY EVALUATION, AND COST ESTIMATES

2.1 SITE INVESTIGATION

The feasibility of designing and constructing a reliable, permanent ground anchor wall depends upon topography, subsurface stratigraphy, physical properties of the soil/rock, groundwater conditions, and restrictions affecting the installation of the soldier beams or anchors. The topographical survey and the subsurface investigation must describe the block of ground that affects the wall. This block extends from behind the ground anchors to in front of the wall where the passive resistance for the embedded portion of the wall will be developed. The subsurface investigation should classify the ground and determine its engineering properties and its corrosion potential. General guidelines for characterizing the ground for an anchored wall are provided below. However, every project is unique and these guidelines should be adjusted depending upon ground variability, local knowledge, experience, and risk.

2.1.1 Field Reconnaissance

During project planning, potential ground anchor wall sites should be inspected in the field. Before making a site visit, a review of the available geologic and groundwater information should be done. Air photos, site surveys, route selection surveys, preliminary designs, geologic maps, and other reports or databases should be studied in preparation for the site visit. During field visits, the following should be accomplished (adapted from Cheney, 1988):

- Select the limits and intervals for cross-sections.
- Observe surface drainage patterns, seepage, flowing groundwater, and vegetation characteristics for estimating drainage requirements.
- Record deterioration and corrosion to existing concrete and steel facilities.
- Map surface geologic features, including rock outcrops, landslide features, and land forms.
- Record the location of above- and below-ground utilities and structures that may affect subsurface exploration or wall construction.
- Identify adjacent structures or properties for future right-of-way acquisition or easements.

2.1.2 Site Plan and Topographical Survey

A plan showing the proposed alignment of the highway, locations of potential earth retaining structures, and existing and future contours should be prepared. Cross-sections through the wall and referenced to the center of the proposed roadway should be prepared at 50-ft intervals. Intermediate cross-section may be required at 25-ft intervals if the ground surface varies or wall height changes abruptly. The plan and cross sections should be prepared as soon as possible during the design phase.

2.1.3 Subsurface Exploration

The goal of the subsurface investigation is to physically describe the block of ground that affects the design and performance of the wall and obtain samples for laboratory testing. The investigation must be complete enough to determine soil/rock stratigraphy and develop cross-sections for stability analyses. Figure 16 outlines the site exploration recommendations for a permanent ground anchor wall.

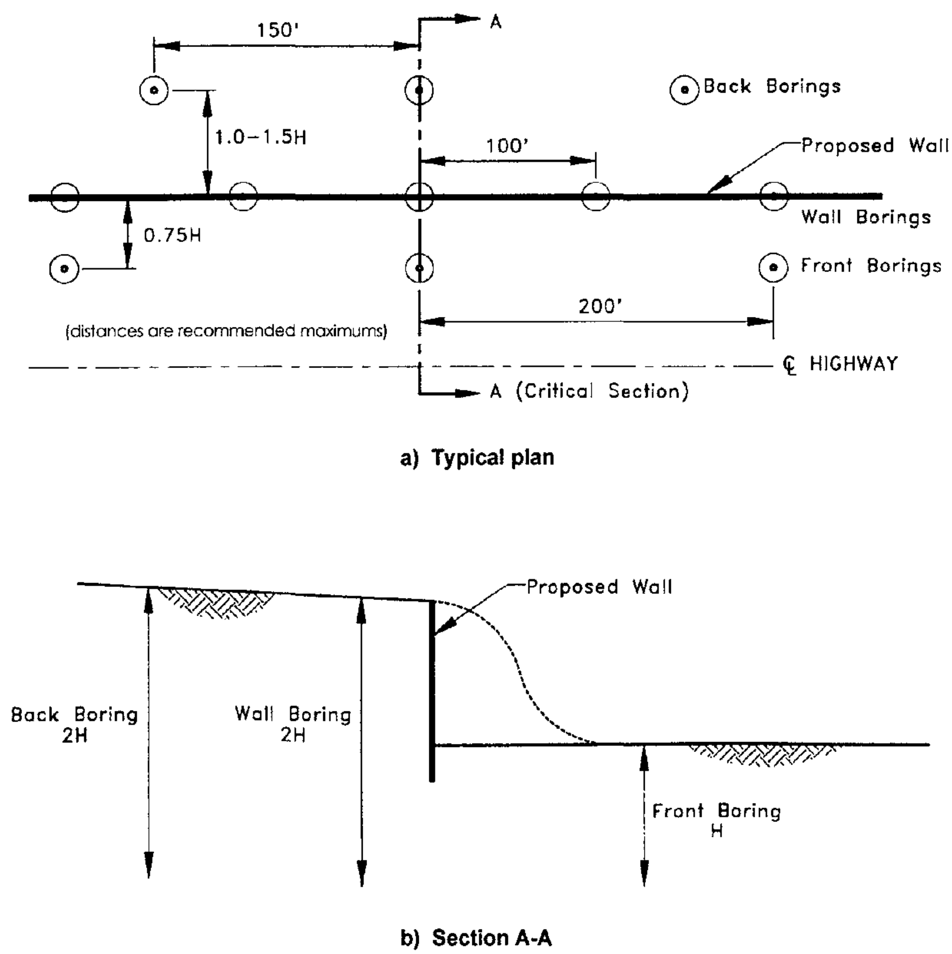


FIGURE 16
Recommended Locations of Subsurface Investigations for Anchored Walls (after Cheney, 1988)

"Wall Borings" should be spaced at approximately 100-ft intervals along the alignment of the structure. They are to extend down to a depth twice the height of the wall. If low-strength soil is found below the wall, the borings should extend completely through the poor soil and penetrate 10 ft into competent ground.

"Back Borings" are located to describe the ground where the anchor bond length will be made. They are spaced at a maximum of 150-ft intervals and located behind the wall at a distance equal to 1 to 1.5 times the height of the wall. If the ground behind the wall slopes up, the Back Borings may be located behind the wall a distance equal to 1.5 to 2.0 times the wall height. The Back Borings should extend down to a depth of at least twice the height of the wall. For landslide stabilization applications, the number and location of the Back Borings may change to describe the ground for limiting equilibrium analyses.

The "Front Borings" are located in front of the wall at a distance equal to 75 percent of the wall height. They are laterally spaced at a maximum of 200 ft center to center. Front Borings should extend down below the bottom of the wall to a depth equal to the height of the wall.

In soil, perform standard penetration tests (SPT) at 5-ft intervals and retain disturbed samples. Describe the soil in the driller's logs and send samples to the laboratory for visual identification and testing. Undisturbed soil samples should be taken in cohesive soils and sent to the laboratory for visual identification, classification tests, and strength tests. Field vane shear tests should be made in soft to medium clays. In soil, determine the aggressivity of the environment by performing field tests on at least one sample from each stratum (Weatherby, 1982). In ground with thin, low-strength soil layers, take continuous samples from the Front Borings for the first 10 ft below the proposed grade in front of the wall.

When rock is encountered, attempt to recover continuous cores. Describe the cores in the driller's logs and note drilling times, water flows and water pressures in joints, and the nature of the material infilling any discontinuities. Rock cores should be retained and sent to the laboratory for visual identification, study of discontinuities, and determination of rock quality designation (RQD).

Careful static water level determinations must be made after completion of each boring and before backfilling. A notation should be made upon removal of the tools and/or casing whether the hole stayed open or the depth of collapse. If groundwater may affect the wall, convert a Back Boring and Front Boring into an observation well and estimate the phreatic surface.

Soil samples and rock cores should be retained until the wall is built and accepted.

American Association of State and Highway Transportation Official's (AASHTO's) *Manual on Subsurface Investigations* (1988) provides additional guidance on subsurface explorations.

2.1.4 Physical Testing

Testing should focus on estimating the unit weight and the shear strength of the soil/rock. Soil samples and rock cores should be visually identified and described in the laboratory. Testing requirements vary depending upon the type of ground. After the testing is completed, soils should be classified following procedures described in Appendix E of the AASHTO *Manual on Subsurface Investigations* (1988).

2.1.4.1 Coarse-grained Soils

Particle size distributions for each stratum should be determined in the laboratory. Unit weights and angles of shearing resistance are estimated from correlations to standard penetration resistance (SPT) values for various types of coarse grained soils. Table 1 presents typical properties of coarse-grained soils related to SPT values and the Unified Soil Classification System (Casagrande, 1948).

TABLE 1
Typical Properties of Coarse-grained Soils
(after Hunt, 1983)

SOIL TYPE	RELATIVE DENSITY	SPT (blow/ft) (1)	VOID RATIO e	DRY DENSITY (pcf)	FRICTION ANGLE (2) ϕ (deg)
GP (poorly graded gravel, gravel sand mixture)	75% (dense)	70	0.33	127	38
	50% (medium dense)	50	0.39	120	35
	25% (loose)	<20	0.47	114	32
SW (Well-graded sand, gravelly sand)	75% (dense)	65	0.43	118	37
	50% (medium dense)	35	0.49	112	34
	25% (loose)	<15	0.57	106	30
SP (poorly graded sand, gravelly sand)	75% (dense)	50	0.52	110	36
	50% (medium dense)	30	0.60	104	33
	25% (loose)	<10	0.65	99	29
SM (silty sand, sand silt mixture)	75% (dense)	45	0.62	103	35
	50% (medium dense)	25	0.74	97	32
	25% (loose)	<8	0.80	93	29
ML (silt with little or no plasticity)	75% (dense)	35	0.80	93	33
	50% (medium dense)	20	0.90	88	31
	25% (loose)	<4	1.0	84	27

(1) *N* is blows per foot of penetration in the SPT. Adjustments for gradation are after Burmister (1962)
(2) Friction angle ϕ depends on mineral type, normal stress, and grain angularity as well as relative density and gradation (angular grains can increase ϕ by about 15 percent in the loose state and 30 percent in the dense state over rounded grains).

2.1.4.2 Fine-grained Soils

For fine-grained soils, the total and dry unit weight are determined from tests performed on undisturbed samples. Atterberg limits and natural moisture content for each soil stratum are determined in the laboratory. Undrained shear strengths shall be determined from unconfined compression tests or consolidated undrained triaxial compression tests. Drained strengths should be determined using consolidated drained triaxial compression tests or consolidated undrained triaxial compression tests with pore water pressure measurements. Estimates of normally consolidated drained shear strength can be made using Figure 17. Determine the overconsolidation ratio by performing consolidation tests on overconsolidated clays.

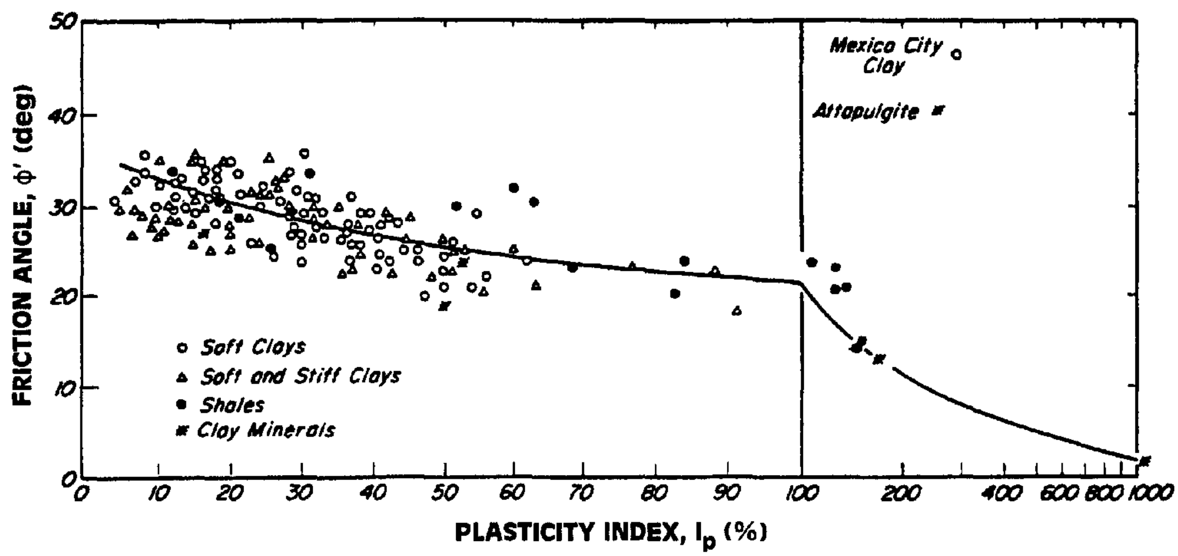


FIGURE 17
Values of Drained Angle of Shearing Resistance for Clays of Various Compositions as Reflected in Plasticity Index (after Terzaghi, et al., 1996)

2.1.4.3 Rock

RQD, unit weight, and unconfined compressive strengths should be determined in the laboratory for each rock stratum. If the field investigation determines that joints or discontinuities are continuous and dipping into the cut, then estimate the residual shear strength on these planes. Direct shear tests or recommendations from a qualified geotechnical specialist should be used to estimate the residual shear strengths.

2.2 FEASIBILITY EVALUATION

The owner determines if a ground anchor wall is feasible for a project. Depending upon the contracting method, the owner or the contractor will do the detailed design for the wall. A feasibility evaluation for each component of an anchored wall requires an understanding of the factors that affect the installation and design of each component.

2.2.1 Ground Anchors

Ground anchors are physically feasible if they can be installed, develop the specified load-carrying capacity, and maintain their load without excessive movement or load loss. An evaluation of the use of permanent ground anchors to support an earth retaining structure involves considering the range of load-carrying capacities, anchor lengths, anchor inclination, hole size, tendon type and size, right-of-way, and easements.

Permanent ground anchors for wall applications usually have design loads between 50 and 130 tons. Anchor tendons with load-carrying capacities in this range are compatible with the drills commonly used for ground anchor work. These tendons can be handled easily, and testing and stressing equipment for them can be handled without difficulty.

Permanent ground anchors in rock develop load-carrying capacities greater than 130 tons without significant loss of load or movement. Load-carrying capacity in rock varies with rock structure, compressive strength, and installation procedures. Even in weak clay shales, high load-carrying capacities are obtained with good hole cleaning and grouting practices. Highly fractured rock with open joints, cavernous limestone, and basalt formations with lava tubes should be avoided if possible since grouting is difficult in these formations.

Permanent ground anchors are routinely installed in coarse-grained soils with standard penetration resistances greater than 10 blows/ft or relative densities greater than 0.3. Ground anchors in coarse-grained soils that satisfy the load-tested acceptance criteria contained in the Post Tensioning Institute's (PTI's) *Recommendations for Prestressed Rock and Soil Anchors* (1996) do not experience significant time-dependent movements or load losses. Permanent ground anchors have been successfully made in natural and fill deposits with standard penetration resistance less than 10 blows/ft.. In these low-strength soils, local experience or a precontract test program may be used to evaluate anchor suitability and range of anchor design loads.

Since the early 1980's, hollow-stem-augered anchors have been used for permanent ground anchor walls in the United States. These anchors are routinely installed in soft rocks, clays, tills, and mixed soils. Recently, post-grouted anchors in clays have been used to support permanent earth retaining walls. Permanent ground anchors are not normally installed in soils with high organic content, normally consolidated clays, and cohesive soils with an unconfined compressive strength less than 1.0 tsf. Anchors installed in soils with a liquidity index less than 0.2 perform satisfactorily. Successful permanent anchor installations have been built in soils with liquidity indices greater than 0.2. In low-strength clays or soils with high liquidity indices, local experience or a precontract test program is recommended.

Ground anchor diameters vary between 3 and 16 in. Cased holes are normally 3 to 7 in in diameter. They are installed in ground that will not support itself during anchor installation. Uncased holes in hard clays, tills, and rock vary between 4 and 8 in in diameter. Hollow-stem-auger anchors are installed in fine-grained soils or in mixed soils. They are usually 12 in in diameter, but some contractors install 16-in hollow-stem-augered anchors. For a given wall, anchor type and drill hole size affects anchor design load, soldier beam type, and beam size.

Anchor design loads cannot exceed 60 percent of the minimum ultimate tensile strength of the prestressing steel tendon. For bar tendons, the maximum design load for a 1⅜-in bar is 140 kips. Normally, highway walls have design loads less than 140 kips. For ground anchor walls, the largest tendons commonly used consist of 7 or 8 0.6-in seven-wire strands. An eight-strand tendon has a maximum design load of 281 kips. Tendons with high design loads are used in landslide stabilization walls. To develop high load-carrying capacities, ground anchors should be anchored in competent soils or rocks.

Ground anchors in soil are at least 30 ft long. A minimum unbonded length of 15 ft is used to avoid significant load losses during seating of the anchorage, and a minimum bond length of 15 ft is used to mobilize load transfer. The unbonded length is selected to locate the anchor bond length behind the critical failure surface in ground suitable for developing the test load. Unbonded lengths rarely exceed 150 ft. A highway wall will usually have a total anchor length less than 60 ft unless the wall is very high, underlain by soft soils, or for a landslide correction.

Bar tendons are available in uncoupled lengths up to 60 ft. Couplers may be used to extend bar lengths beyond 60 ft or in situations where space constraints limit tendon length. Tendons longer than 60 ft are generally fabricated using seven-wire prestressing strands. Couplers should not be permitted for strand tendons except for tendon repair or for joining scrap lengths of bar tendons. Contractors select the tendon type on the basis of a variety of factors, and owners should not specify tendon type.

It is desirable to locate the center of the anchor bond length at least 15 ft below the ground surface. Experience has shown that shallow anchors do not develop as high a load transfer rate as deep anchors. Ground anchor length and angle are adjusted to obtain adequate overburden for the anchor bond length. Shallow anchors are used, but their load-carrying capacity may be less than expected.

Ground anchors are usually installed at inclinations between 10° and 30° down from the horizontal. To maximize the horizontal component of the ground anchor and minimize the downward load applied to the wall, ground anchors are installed at flat angles. When suitable anchoring strata lie below the bottom of the wall, or when underground structures or utilities prevent the installation of flat ground anchors, the ground anchor angle may be increased to 45°. Increasing the ground anchor angle applies a large vertical load to the wall.

During the acquisition of right-of-way for a proposed roadway, a geotechnical engineer should determine the right-of-way necessary for the permanent ground anchors. Permanent easements should be obtained from adjacent property owners prior to final design as right-of-way issues may be difficult to negotiate and control after the project is completed.

Highway departments must ensure that the permanent ground anchors will not be disturbed after they are installed. Procedures should be established to control above- and below-ground construction activities near the ground anchors. New roadway construction or fills behind the wall will not affect the individual capacity of the ground anchors but they may add additional loads to the wall. If plans call for construction behind the wall, expected loads should be included in the design of the wall.

2.2.2 Vertical Wall Elements

At many sites, driven or drilled-in soldier beams may be feasible. At these locations, cost should determine the method of installation. Driven soldier beams use H-pile (HP) or sheet pile shapes. Drilled-in soldier beams include double I-beams, double channels, single structural shapes, and cast-in-place reinforced concrete. When soldier beams must be installed in very dense coarse-grained soils, ground with cobbles or boulders, hard fine-grained soils, or rock, then soldier beams may have to be drilled in. Contractors have developed a variety of anchor to soldier beam and facing to soldier beam connections for driven and drilled-in soldier beams. Anchor type and load-carrying capacity is interrelated with the selection of soldier beam type and installation method. Specifications should allow the contractor to select soldier beam type and installation method. When the ground exposed by the cut is competent, anchored horizontal beams or elements may be a feasible alternative to soldier beams (Section 2.2.3).

Permanent ground anchor walls are not suitable for applications where soft cohesive soils lie below the bottom of the wall for considerable depths. When soft soils extend below the bottom of the wall, large ground movements will develop even when stiff walls and high ground anchor loads are used. In addition, the ground anchors must be installed at steep angles to extend through the soft soil to reach ground suitable for anchoring. Steep anchors apply large vertical loads to the wall. If the soft soil is underlain by competent ground near the bottom of the excavation, then the wall and anchors can extend into the competent ground. In this case, a permanent ground anchor wall should perform satisfactorily.

2.2.3 Horizontal Beams

Soldier beam installation can be very expensive. Soldier beam installation costs depend upon access to the work and the drilling costs. In bouldery ground or rock, soldier beam drilling costs can exceed $300/linear ft. On sloping sites, constructing access ramps for large drilling equipment may be difficult and expensive to build. When competent ground is present at the excavation face, anchored horizontal beams or elements may be used instead of soldier beams. Designs incorporating horizontal beams or elements should consider vertical loads from the anchors, support of the ground between the beams or elements, and aesthetics. Figures 4 and 11 show anchored walls built using horizontal beams. Figure 18 shows anchored elements for the construction of an exit ramp for U.S. Route 220 near Altoona, Pennsylvania.

FIGURE 18
Landslide Stabilization Along U.S. 220 Near Altoona, Pennsylvania

2.3 ECONOMIC FEASIBILITY EVALUATION AND COST ESTIMATES

When an earth retaining wall has to be built in a deep cut, a permanent ground anchor wall will usually be less costly than a conventional retaining wall. If temporary excavation support is required for the conventional wall, then an anchored wall will cost less than the conventional wall. Table 2 contains ranges of square-ft costs for permanent anchor walls. In developing the table, driven soldier beams were assumed to cost between $20 and $48/linear ft and to be on 8-ft centers. Drilled-in soldier beams were estimated to cost between $40 and $300/linear ft and to be on 10-ft centers. Ground anchor prices for the table ranged from $1000 to $6000 each and included the soldier beam connection and testing costs. Each ground anchor was assumed to support 100 sq ft of wall. Table 2 can be used during feasibility evaluations or for developing cost estimates if cost data are not available.

Indirect cost benefits from using ground anchor walls can be significant. When comparing the costs of a permanent ground anchor wall with other systems, the owner should include costs savings associated with faster construction time and minimized inconvenience to the traveling public. In addition, design costs and time can be saved when the walls are designed and built by specialty geotechnical contractors.

TABLE 2
Square-foot Costs for Permanent Earth Retaining Walls Using Soldier Beams and Ground Anchors (1997 costs)

	SOIL						BOULDERS AND/OR ROCK			LANDSLIDES					
	DRIVEN SOLDIER BEAMS			DRILLED SOLDIER BEAMS			DRILLED SOLDIER BEAMS			DRIVEN SOLDIER BEAMS			DRILLED SOLDIER BEAMS		
COST ELEMENTS	Min. $	Ave. $	Max. $	Min. $	Ave. $	Max. $	Min. $	Ave. $	Max. $	Min. $	Ave. $	Max. $	Min. $	Ave. $	Max. $
Soldier Beams	2.50	3.25	5.00	4.00	5.00	7.00	10.00	15.00	30.00	3.00	4.00	6.00	10.00	15.00	30.00
3" Temp. Lagging & Drainage	5.00	7.00	10.00	5.00	7.00	10.00	5.00	7.00	10.00	5.00	8.00	12.00	5.00	8.00	12.00
Anchors	10.00	17.00	30.00	10.00	17.00	30.00	15.00	20.00	40.00	15.00	20.00	60.00	15.00	20.00	60.00
Cast-in-place Facing	11.00	15.00	20.00	13.00	16.00	21.00	11.00	15.00	20.00	11.00	17.00	25.00	11.00	17.00	25.00
TOTAL COST/SF	28.50	42.25	65.00	32.00	45.00	68.00	41.00	57.00	100.00	34.00	49.00	103.00	41.00	60.00	127.00

CHAPTER 3: CONTRACTING PROCEDURES

Today, permanent ground anchor walls are routinely constructed for public and private owners by specialty geotechnical contractors. Prescriptive (procedural) specifications are used for most public projects and performance specifications are used in the private sector. Prescriptive specifications restrict specialty contractors from using their experience and specialized methods and equipment. Expensive walls, poor details, and unnecessary claims have resulted when prescriptive specifications are used. Public agencies, which recognize that specialty geotechnical contractors possess the design and construction expertise for anchored walls, have successfully used alternative contracting procedures for ground anchor walls. When used properly, these procedures encourage the contractors to select the best wall system, use constructible details, and continuously improve and innovate.

Ideally, the contracting procedures and specifications should allow contractors to use their experience, knowledge, proprietary methods, and specialized equipment. They also should enable owners to maintain control over the finished product and to develop an understanding of ground anchor wall work. Agencies with limited experience with permanent ground anchor wall work should consider beginning with small, straightforward projects and using prequalified contractors. Agency engineers should prepare the contract plans and specifications for the walls in-house, review the contractor's detailed wall design, and send their designers into the field during construction of the wall. Performance specifications can accomplish these objectives. A properly prepared performance specification can enable a positive relationship between the contractor and the owner, and equitably share the risk associated with the work. Performance specifications that have been successfully used on highway work are discussed in this chapter.

3.1 PREQUALIFYING CONTRACTORS

Experienced anchored wall contractors can design the work, adapt to the varying conditions encountered underground, and understand how to construct the walls efficiently. Some public agencies, which recognize the advantages of using wall specialists, prequalify wall contractors.

Prequalifying contractors before advertisement is done in two ways. An explicit prequalification is used to require the contractor to show that he or she satisfies specific written requirements. An implicit prequalification occurs when the contracting agency requires wall contractors to submit detailed design calculations and a complete set of working drawings for the wall for review and approval. After the contractors are prequalified, the contracting agency publishes their names in the bid documents and requires that they design and build the wall.

After a contract award and before commencement of the ground anchor work, contracting authorities have explicitly prequalified wall contractors by requiring them to show that the company and the key personnel satisfy requirements contained in the specifications. Ground anchor work cannot begin until the contractor and personnel are approved, and work is stopped if the contractor substitutes unqualified personnel for prequalified personnel.

Requiring pre-approved designs is the most effective way to prequalify the contractors, establish a common understanding of what will be provided, and educate agency personnel.

3.2 PERFORMANCE SPECIFICATION CONTRACTING PROCEDURES

The scope of the work included in a performance specification for ground anchor wall work will vary. Performance specifications can require the contractor to prepare detailed design calculations, working drawings, and construct the wall. A performance specification can be limited in scope and require the contractor to design and install the ground anchors for a wall shown on owner-prepared detailed working drawings. Each type of performance specification assigns different requirements and responsibilities to the owner and the contractor.

3.2.1 Contractor-prepared Working Drawings

Performance specifications that require the contractor to prepare detailed design calculations and working drawings for the anchored walls are sometimes called "design-build contracts." These contracts are not design-build contracts. A formal design-build contract is one where the project design and the construction contract are combined into one contract. When a contractor prepares detailed wall design calculations and working drawings, owners still use a design engineer and a traditional construction contract. The design engineer may work for the owner directly or for a consulting engineer, and he or she prepares the project design and specification. Design drawings prepared by the owner's design engineer establish location, layout, dimensions, and detail requirements for the walls. The design engineer also reviews the contractor's design calculations and working drawings. These contracts are traditional construction contracts and they preserve competitive bidding.

When the contractor prepares the working drawings, the owner should furnish the following:

- Geotechnical and groundwater investigations.
- Design criteria including seismic coefficients, soil or rock shear strength, unit weights, and required safety factors.
- Surcharge loads.
- Material specification requirements.
- Ground anchor tendon corrosion protection requirements.
- Wall finish and color requirements.
- Barrier, coping, and drainage requirements.

- Suggested wall location plans with beginning and end of wall stations, easements, and construction right-of-ways identified.
- Suggested wall elevation.
- Cross-sections defining the surface and subsurface conditions in front and behind the wall.
- Existing and finished grades near the wall.
- Maintenance of traffic requirements that affect wall construction.
- Construction tolerances for wall alignment.
- Locations for abandoned, existing, and future utilities.
- Location of existing and future structures.
- Construction monitoring requirements.
- Submittal requirements.
- Ground anchor testing procedures and acceptance criteria.
- Method of measurement and payment.

When a performance specification requires the contractor to prepare the working drawings, the contractor is responsible for:

- Providing a complete set of working drawings and detailed design calculations for review and approval.
- Building the wall according to the approved working drawings and the contract specifications.
- Performing ground anchor load tests and satisfying the acceptance criteria in the contract specifications.
- Performing required tests and construction monitoring.
- Redesigning work as required (depending upon the nature of the redesign and contract requirements, the contractor may or may not be compensated for the redesign and the extra work).

After the construction contract is awarded, the owner is responsible for the following when the contractor prepares the working drawing:

- Reviewing and approving the working drawing and design calculations.
- Reviewing and approving contractor submittals.
- Verifying that the materials satisfy the contract specifications, approved drawing, and submittals.
- Verifying that tendon corrosion protection requirements have been satisfied and the protection at the anchorage has been completed satisfactorily.

- Verifying that wall construction tolerances are satisfied.
- Verifying that wall facing requirements are satisfied.
- Ensuring that required tests and construction monitoring are done according to the agreed upon schedules.
- Observeing and verifying ground anchor test results.

The advantages and disadvantages of using contractor-prepared working drawings are presented in Table 3.

TABLE 3
Advantages and Disadvantages of Contractor-prepared Working Drawings

ADVANTAGES	DISADVANTAGES
Cost-effective	Owner gives up some control over design
Reduced claims and change orders	Limits competition to prequalified contractors
Encourages partnering and contractor-owner cooperation	Engineer and inspector's role is changed
Small owner staff and limited in-house technical expertise required	
Risk is equitably shared	
Contractors able to use special knowledge and equipment, patented systems	
Provides incentive for contractor to improve and innovate	

Different methods have been used to require contractors to prepare detailed design calculations and working drawings for the anchored walls. These approaches are presented below.

3.2.1.1 Pre-bid Wall Design

When a pre-bid wall design is used, owners contact qualified wall contractors and determine their willingness to prepare detailed design calculations and working drawings for the walls on a project that will be advertised in the future. If a sufficient number of contractors are interested in preparing the designs, the owner furnishes the geotechnical information, design requirements, performance criteria, and project design information to each contractor. The owner may require that an anchored wall be built or the owner may allow the contractor to select the type of wall. Then the contractors complete the design and prepare the working drawings. Normally, between 60 and 120 days are required for the contractors to complete the calculations and working drawings and obtained owner approval. If the owner's designer is a consultant, he or she reviews and approves the design and working drawings. The owner keeps each contractor's design confidential until the project is advertised. Approved working drawings for each contractor are usually included in the bid documents. When the project is advertised, the owner agrees to require the prime contractor to use the specialty contractor who prepared the design that the prime contractor selects. Occasionally, the owner provides a list of the approved wall contractors without including their working drawings in the bid documents. Then the prime contractor meets with the wall contractors individually and selects the wall system that best satisfies their requirements. The prime contractor is free to select the design that results in the lowest overall bid price. When the prime contractor submits a bid, he or she is required to list the pre-approved wall contractor who will construct the walls. When using pre-approved contractor designs, the public benefits since claims are reduced, construction time is shortened, wall costs are reduced, the owner and the contractor agree on what will be provided, and patented walls or methods can be incorporated in the work. Pre-bid approval is better than Value Engineering since the owner obtains the best ideas from a group of contractors who regularly design and build walls. Specialty wall contractors benefit by preparing the designs in advance. They know what the owner will accept and they can start work immediately since the working drawings have been approved.

3.2.1.2 Pre-bid Typical Section Design

Owners can reduce the work required to prepare and review a complete set of working drawings before bid by requiring the contractors to prepare detailed design calculations and working drawings for selected typical sections. The procedures are the same as those used for the pre-bid wall design (Section 3.2.1.1) but only selected portions of the work are designed. The contract documents include the typical working drawings and identify those contractors pre-approved to construct the walls. Prime contractors select the specialty wall contractor they want to use and bid the work. After an award the wall contractor completes detailed design calculations and working drawings using the pre-approved design procedures and details. When using this type of performance specification, claims and wall costs are reduced, and patented walls or methods can be incorporated in the work. This type of specification keeps pre-award design and review cost low while enabling the owner to benefit from the specialty wall contractor's experience.

3.2.1.3 Post-bid Wall Design

Public owners have developed contract plans and specifications that identify each wall and specify acceptable alternative wall types for each wall. These contracts list pre-approved wall supplies and prequalified wall contractors. Design requirements for each wall type are contained in the special provisions or standard specifications. Often the bid documents contain owner-prepared working drawings for a conventional cantilever wall, and they may contain owner-prepared working drawings for a ground anchor wall. The prime contractor decides whether to build the cantilever wall or the owner-designed anchored wall, or to select a wall system from a specialty wall contractor or supplier. Then the wall contractor or supplier prepares the detailed design calculations and a complete set of working drawings for owner review and approval. Upon approval, the walls are built in accordance with the working drawings. When owners use this type of contract, they benefit from the experience of the wall contractors or suppliers. However, they do not have as much control over the finished product as they do when they require the pre-bid approval of the working drawings, and they may not get the lowest price since the contractors/suppliers are uncertain what will be approved. Owners may not benefit from the best design and lowest price since prime contractors want to limit their risk and will not select alternate designs unless the cost savings is large and the time required to obtain owner approval for alternative systems can be quantified.

3.2.2 Owner-prepared Working Drawings

When the owner prepares the detailed design calculations and the working drawings the scope of the performance specification is limited. Here, the contractor selects the ground anchor installation method and is responsible for the anchor developing the specified load-carrying capacity. When preparing the design, the owner must make assumptions regarding the best type of anchored wall, ground anchor load-carrying capacity, soldier beam type and installation method, and wall facing and ground anchor connection details.

An owner-prepared design allows more control over the final product, but what is gained in control may be lost in economy.

When the owner uses a performance specification for the ground anchors and prepares the working drawings, the contract documents include the following:

- Complete working drawings with soldier beam and ground anchor schedules.
- Subsurface and groundwater investigations.
- Contractor submittal requirements.
- Required tolerances for soldier beams, ground anchors, and facing.
- Traffic relocation and maintenance plans.
- Material specifications requirements.
- Anchor tendon corrosion protection requirements.
- Wall facing reinforcing steel requirements, dimensions, finish, and color.
- Wall drainage details.
- Ground anchor testing requirements and acceptance criteria.
- Construction monitoring responsibilities and requirements.
- Methods of measurement and payment.

When the working drawings are prepared by the owner and a performance specification requires the contractor to select the ground anchor installation method and develop the specified ground anchor load-carrying capacity, the contractor is responsible for the following:

- Providing the required submittals.
- Selecting the ground anchor installation method and equipment.
- Providing materials that satisfy the contract specifications and the approved submittals.

- Installing the soldier beams and wall facings in accordance with the contract documents.
- Installing the ground anchor in accordance with the approved submittal.
- Meeting the specified construction tolerances for the soldier beam, ground anchors, and wall facing.
- Testing the ground anchor and obtaining the specified load-carrying capacity.

After a construction contract using owner-prepared working drawings is let, the owner is responsible for the following:

- Reviewing and approving the contractor submittals.
- Verifying that the materials satisfy the contract requirements.
- Verifying that the installation of the soldier beams and wall facing satisfy contract requirements.
- Verifying that the ground anchors are installed in accordance with the approved submittal.
- Verifying that soldier beam, ground anchor, and wall facing construction tolerances are satisfied.
- Verifying that tendon corrosion protection requirements have been satisfied, and the protection at the anchorage has been completed satisfactorily.
- Verifying that wall facing requirements are satisfied.
- Ensuring that required tests and construction monitoring are done according to the agreed upon schedules.
- Observing and verifying ground anchor test results.

The advantages and disadvantages of using owner-prepared working drawings and a performance specification for the ground anchors are presented in Table 4.

TABLE 4
Advantages and Disadvantages of Performance Specification with Owner-prepared Working Drawings

ADVANTAGES	DISADVANTAGES
Owner controls the final product	Owner must make assumptions regarding ground anchor capacity and wall construction
Risk is shared (owner assumes more risk than when the contractor designs the walls)	If wrong assumptions are made, wall cost will increase, and claims or change orders will result
Contractors able to utilize specialized ground anchor knowledge and equipment	Contractors cannot use specialized knowledge and equipment to construct the wall
Owners' staff gains experience	Owner should have trained experienced staff
	Patented and proprietary wall systems cannot be used

3.3 PRESCRIPTIVE SPECIFICATIONS

Traditionally owners have used prescriptive specifications in the public sector. They are perceived as satisfying the Federal Acquisition Regulations and delivering a satisfactory product at the lowest price. For many types of work the results have been satisfactory, but prescriptive specifications are not recommended for specialty geotechnical construction. When a prescriptive construction specification is used, the owner assumes full responsibility for the design and the construction of the work if the contractor has followed the prescription in the specifications. A prescriptive specification spells out every detail of the design and the construction process. The soldier beam size and installation method, ground anchor design loads, the ground anchor drilling and grouting methods, the ground anchor to soldier beam connection, and the facing to soldier beam connection are detailed in a prescriptive specification.

Prescriptive specifications do not ensure better work and they open the work up to contractors who are not familiar with ground anchor wall design and construction. When using a prescriptive specification, owners should be confident that they know more about ground anchor wall construction than the contractors, and that they can predict wall performance. The owner must be prepared to direct the contractor's operation if the specified installation procedures do not produce the desire results.

The advantages and disadvantages of using owner-prepared working drawings and prescriptive specifications are presented in Table 5.

TABLE 5
Advantages and Disadvantages of Prescriptive Specifications

ADVANTAGES	DISADVANTAGES
Owner controls the final product	Owner assumes most risk
Increases competition among contractors	Owner responsible for design and performance of the wall
	Owner must make assumptions regarding ground anchor capacity and construction techniques
	If assumptions are wrong, wall costs will increase, and claims or change orders will result
	Experienced contractors may be unable to use specialized knowledge and equipment
	Owner must have trained experienced staff
	Unqualified contractors may be awarded the contract
	Patented and proprietary wall systems cannot be used

3.4 RECOMMENDATIONS

Ground anchor wall technology is continuing to change as wall contractors develop new techniques, details, methods, and equipment. The continuously evolving nature of the work is shown when different contractors prepare working drawings for the same wall. Often the contractors select different types of soldier beams and ground anchors to build the same wall.

Performance specifications that enable prequalified contractors to prepare the detailed design calculations and the working drawings will ensure that the owner maintains control over the finished product while taking advantage of the specialty wall contractor's knowledge and experience. If the owner's staff has limited experience, they can require the pre-bid submission of complete detailed design calculations and working drawings or typical design calculations and drawings for selected sections of a wall. If the staff has sufficient experience, the owner can use a performance specification to obtain contractor-designed walls after the project is awarded.

Prescriptive specification should only be used in special circumstances. In those cases the owner believes that it will be in his or her interest to do the detailed design and to specify how the work will be installed.

CHAPTER 4: EARTH PRESSURES

4.1 INTRODUCTION

Apparent earth pressure diagrams distribute the total lateral earth load to permanent ground anchor walls for design. The magnitude and the shape of the diagrams are based on measured strut loads, limiting equilibrium analyses, experience, and an understanding of anchored wall behavior. Detailed discussions of earth pressure on soldier beam walls are contained in *Summary Report of Research on Permanent Ground Anchor Walls*, "Volume I: Current Practice and Limiting Equilibrium Analyses," (Long, et al., 1998) and *Summary Report of Research on Permanent Ground Anchor Walls*, "Volume III: Model-scale Wall Tests and Ground Anchor Tests,"(Mueller, et al., 1998).

4.2 DEVELOPMENT OF APPARENT EARTH PRESSURE DIAGRAMS

Apparent earth pressure diagrams or envelopes were originally developed to give the magnitude and distribution of earth pressures on braced excavation support systems. They were derived by measuring strut load increases on braced excavations as the excavation deepened. Little displacement was required for the stiff struts to pick up load.

Experience with strutted walls in the last 100 yr has shown that the total lateral force measured on the walls is close to values calculated from active earth pressure theory. However, the distribution of earth pressure on strutted walls does not fit the classical theories of Coulomb and Rankine. Instead of earth pressure increasing linearly with depth (triangular distribution), it has long been observed that high pressures develop in the upper part of the wall, as the supports are placed. The upper supports restrain the wall from rotating outward sufficiently to reduce the earth pressures to active (triangular distribution).

Field measurements of strut loads on internally braced excavations in sands (principally in Berlin, Munich, and New York City) and in clays (principally in Chicago) led to development of apparent earth pressure envelopes (Terzaghi and Peck, 1967). Apparent earth pressures were calculated by dividing measured strut loads by the area of the wall supported by each strut. Soil at the bottom of the cut was considered to be a strut, and the beam was hinged at the bottom of the excavation. The pressures were not directly measured, thus the name apparent earth pressures. Pressure distributions varied depending on the details of construction. For example, higher loads developed in some struts because they were more highly pre-loaded, or because they were installed quickly after excavating.

Apparent earth pressure diagrams were developed to be envelopes that encompassed the highest apparent earth pressures determined from the measured strut loads and, thus, predicted greater pressures than those calculated from most struts. Accordingly, the total load from an apparent earth pressure diagram was greater than the total measured earth load. Apparent earth pressure envelopes are rectangular or trapezoidal in shape. Typical apparent earth pressure diagrams used today are presented in Figure 19. These diagrams are discussed by Terzaghi, et al. (1996) and Schnabel (1982). An important assumption in the use of these diagrams is that the static water level is below the base of the excavation.

The apparent earth pressure envelope for sand (Figure 19a) is a rectangle with an apparent earth pressure (p_{AEP}) equal to

$$p_{AEP} = 0.65 K_A \gamma H \qquad \ldots [4.1]$$

where K_A is the Rankine active earth pressure coefficient ($K_A = \tan^2\{45-\phi/2\}$), γ is the total unit weight of the soil, and H is the height of the cut. Applying the apparent earth pressure along the full height of the cut produces a total lateral force that is 1.3 times the value predicted from Rankine active earth pressure theory.

Figure 19b is the apparent earth pressure envelope for soft to medium clays. The maximum apparent earth pressure for a soft to medium clay is expressed as

$$p_{AEP} = K_A \gamma H \qquad \ldots [4.2]$$

where $K_A = 1 - 4s_u/\gamma H$, γ is the total unit weight of the soil, H is the height of the cut, and s_u is the undrained shear strength of the soil. Soft to medium clays have a ratio of $\gamma H/s_u$ greater than about six. The distribution of apparent earth pressure varies from zero to full pressure at a depth of $0.25H$ (Figure 19b). The pressure remains constant below a depth of $0.25H$. Applying the apparent earth pressure along the full height of the cut produces a total lateral force that is 1.75 times the value that would be predicted from active earth pressure theory.

The apparent earth pressure distribution for stiff-fissured clays is shown in Figure 19c. The maximum apparent earth pressure for a stiff clay ranges between

$$p_{AEP} = 0.2\gamma H \text{ to } 0.4\gamma H \qquad \ldots [4.3]$$

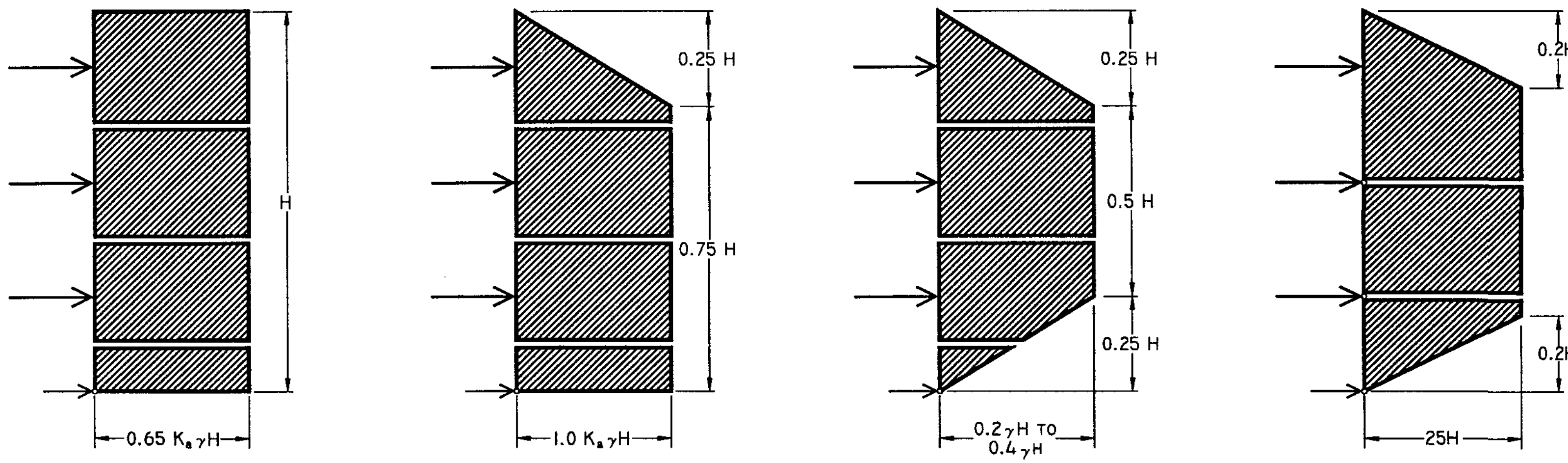
Stiff clays are identified as those with ratios of $\gamma H/s_u$ less than four. When $\gamma H/s_u$ is equal to four, K_A is zero, and the active earth pressure is zero. Significant loads were measured on struts supporting walls in stiff clay. These loads did not develop from a state of limiting equilibrium. Instead, the stresses and the deformations behind a wall in stiff clay correspond to a quasielastic state (Terzaghi, et al., 1996). In a quasielastic state the earth pressures depend on the at-rest pressures, the in situ modulus of elasticity, the stiffness of the supports, the depth of overexcavation before each level of support is installed, and the pre-load applied to the supports.

The total lateral earth load for cuts in clay having values of $\gamma H/s_u$ between four and six are determined using both the soft to medium clay (Figure 19b and Equation 4.2), and the stiff clay (Figure 19c and Equation 4.3) diagrams. The apparent earth pressure diagram that produces the greatest total earth load is used for design.

The Terzaghi and Peck apparent earth pressure diagrams (Figures 19a to 19c) assume that the wall is either in sand or clay. Frequently, excavation support systems and anchored walls are built in mixed grounds. In mixed grounds, selecting the apparent earth pressure diagram and estimating the intensity of the earth pressure is difficult. For more than 35 yr, Schnabel Foundation Company has successfully used a single apparent earth pressure diagram to design excavation support systems and anchored retaining walls in sands, stiff clays, and mixed grounds. Figure 19d shows the $25H$ trapezoidal apparent earth pressure diagram recommended by Schnabel (1982). The total lateral earth load estimated using Schnabel's diagram is approximately equal to the load determined using Terzaghi and Peck's diagram for a sand with an angle of friction of 35° or their diagram for a stiff clay with the pressure equal to $0.2\gamma H$. Measured strut loads in sands and stiff clays fit within the $25H$ envelope. This diagram is not used to design walls in low-strength cohesive soils, and the intensity of the pressure may be increased for stiff, fissured, heavily overconsolidated clays.

Today, apparent earth pressure diagrams developed from strut measurements are used to design permanent ground anchor walls. Implicit in the use of these diagrams is that the groundwater level remains below the bottom of the wall. Ground anchors, which are much more flexible than struts, are tensioned to loads near their design load to reduce lateral wall movements during excavation. Load cell measurements show that ground anchor loads do not change significantly during excavation. Measured ground anchor loads reflect the pre-load (lock-off load) rather than the load imposed by the ground during construction (as observed with strutted excavations).

Driven soldier beams typically are spaced 5 to 8 ft apart. When the soldier beams are drilled-in, the spacing often increases to 10 ft.



$K_a = \tan^2(45 - \Phi/2)$

$K_a = 1 - \frac{4s_u}{\gamma H}$

a) Sands ***b) Soft to medium clay*** ***c) Stiff-fissured clays*** ***d) 25H trapezoid***

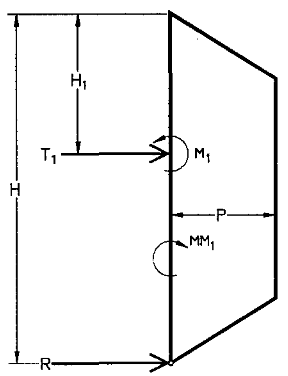
FIGURE 19
Apparent Earth Pressure Diagrams

4.2.1 Anchor Loads and Soldier Beam Bending Moments

Ground anchor loads and soldier beam bending moments are determined from the apparent earth pressure diagrams. Figure 20 shows how these methods are used in practice. Loads and moments are determined by the tributary area method or by dividing the beam into simple beams. Bending moments in the soldier beam down to the first ground anchor are calculated by summing moments. Below the upper ground anchor, current design methods conservatively predict the magnitudes of the maximum bending moments, but they do not predict their locations (Weatherby, et al., 1998).

For soldier beam and sheet pile walls, the actual distribution of earth pressures is different from the apparent earth pressure diagrams. Earth pressures increase at the ground anchor locations as the wall distributes the anchor load to the ground. Between rows of anchors or between the lowest row of anchors and the bottom of the excavation, the soldier beams deflect and redistribute the earth pressures to stiffer locations (ground anchors or subgrade) through arching. Redistribution of earth pressures results in the actual bending moments being less than the computed ones (Mueller, et al. 1998). Accordingly, Peck, et al. (1974) indicated that soldier beam bending moments may be computed using two-thirds of the apparent earth pressures. Consistently good results over many years have shown that apparent earth pressure diagrams are suitable for determining support loads and conservative in estimating bending moments.

Soldier beams deflect as the earth pressure increases, but they are not subject to progressive failure since the pressures redistribute to the supports. Strutted or anchored walls designed to resist appropriate apparent earth pressures have not failed in bending when base stability was adequate. Experience has shown that the exact shape of the apparent earth pressure diagram is not critical if the earth pressure result is near the mid-height of the wall and the total lateral earth loading is appropriate. The total lateral earth load is dependent upon the strength of the ground. Consequently, the emphasis should be placed on accurately determining the strength of the ground. Complicated bending moment calculations are unnecessary. The calculations and assumptions shown in Figure 20 are simple and they conservatively estimate ground anchor loads, bending moments, and toe reactions for soldier beam walls.



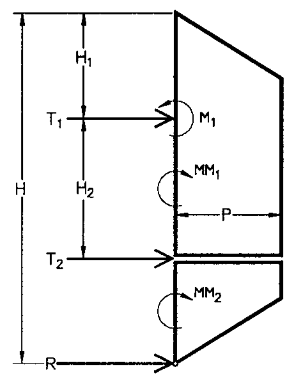
a) Single-tier walls

$T_1 = \Sigma$mom. @ R

R = Tot. Earth Pressure$-T_1$

$M_1 = \Sigma$mom. @ T_1

MM_1 = Max. moment where shear is zero



b) Multi-tier walls (simple beams)

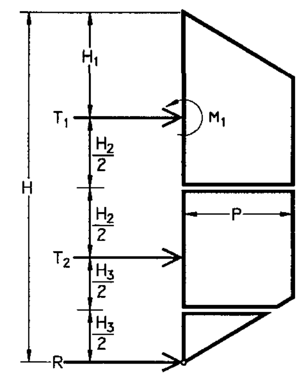
$T_1 = \Sigma$mom. @ T_2

$T_2 = \Sigma$mom. @ R

R = Tot. Earth Pressure$-T_1-T_2$

$M_1 = \Sigma$mom. @ T_1

Max. moment below T_1 = larger of MM_1 or MM_2



c) Multi-tier walls (tributary area)

T_1 = Load over span $H_1 + \frac{H_2}{2}$

T_2 = Load over span $\frac{H_2}{2} + \frac{H_3}{2}$

R = Load over span $\frac{H_3}{2}$

$M_1 = \Sigma$mom. @ T_1

Max. moment below T_1 = $\frac{1}{10}pL^2$ (L= larger of H_2 or H_3)

(Moments below T_1 often reduced)

FIGURE 20
Procedures for Determining Anchor Loads and Soldier Beam Moments from Apparent Earth Pressure Diagrams

4.2.1.1 Walls with a Single Level of Anchors

AASHTO's *Standard Specification for Highway Bridges* (1996) recommends that apparent earth pressure diagrams similar to those shown in Figure 19 be used to design ground anchor walls supported by multiple rows of anchors. For walls supported by one row of anchors, a triangular earth pressure diagram is recommended. A triangular earth pressure diagram assumes that the one-tier wall deforms similarly to an anchored bulkhead with yielding supports. Anchored bulkheads and ground anchor walls are built differently. Bulkheads are backfilled after the sheet piling has been driven and the deadman anchors are loaded as the earth pressures increase in response to raising the backfill. Ground anchor walls are built from the top down and the anchors are loaded when the excavation is at the anchor level. Deformations and stresses for the two walls are different and the earth pressure diagrams for the walls are different too. Research and experience has clearly shown that soldier beam and sheet pile walls supported by one level of anchors should be designed to resist the same apparent earth pressures as walls supported by multiple rows of anchors. Rowe (1952) demonstrated that bulkheads supported by one level of fixed supports (pre-loaded anchors) did not rotate outward sufficiently far enough to reduce the earth pressures to an active triangular distribution. Rowe's tests showed that the earth pressures were higher than active at the support and lower than active between the support and the bottom of the excavation. For more than 38 yr, Schnabel Foundation Company has used the same apparent earth pressure diagram to design walls supported by one or multiple levels of support. Weatherby, et al. (1998) and Mueller, et al. (1998) demonstrated that the same apparent earth pressure diagrams should be used to design walls supported by one or more rows of ground anchors. Weatherby, et al. (1998) also showed that the bending moments calculated assuming triangular earth pressures were unrealistically high near the bottom of the excavation.

4.2.1.2 Soldier Beam Toes

When base stability is adequate, a hinge and strut are assumed to exist in the soldier beam at the bottom corner of the excavation. A subgrade hinge and strut are consistent with the assumptions used in developing the apparent earth pressure diagrams (Terzaghi, et al., 1996). Weatherby, et al. (1998) showed that measured soldier beam bending moments near the bottom of walls supported by one and two tiers of anchors were smaller than those predicted assuming a continuous soldier beam. When a hinge is used at subgrade, the passive capacity of the toe must be adequate to support the computed load from the apparent earth pressure diagram. The ultimate lateral toe resistance of the soldier beam toe is at least 1.5 times the toe reaction determined from the apparent earth pressure diagram.

When base stability is poor (weak soil below the bottom of the wall), the wall is assumed to be continuous at subgrade and cantilevered about the lowest support. Temporary excavation support walls on the Central Artery Tunnel Project in Boston, Massachusetts, have been designed assuming that the embedded portion of the wall cantilevers around the lowest support.

4.2.2 Basal Stability

The apparent earth pressure diagrams discussed above and presented in Figure 19 are for conditions where the soil at the bottom of the wall is not near a state of plastic equilibrium (failure). Excavations in deep deposits of soft to medium clay have moved excessively because the weight of the retained soil exceeds the bearing capacity of the soil at subgrade or a deep-seated failure develops. Soldier beam walls in granular soils are not subject to basal instability since the walls are free draining, eliminating large unbalanced hydrostatic pressures, and the shear strength is adequate at the base of the wall. Special attention is given to assessing the base stability of cuts in soft to medium clay and the effect of base stability on apparent earth pressures.

Cutoff walls in granular soils will be subjected to unbalanced seepage forces when the groundwater level behind the wall is higher than the groundwater level inside the wall. If the wall does not penetrate below the bottom of the excavation sufficiently far, the upward flow of water inside the wall may create base instability. NAVFAC's *Foundations and Earth Structures Design Manual 7.2* (1982) presents procedures for determining the toe penetration required to prevent base instability resulting from unbalanced seepage pressures.

Significant basal heave (movements) and substantial increases in apparent earth pressures have resulted when the weight of the retained soil exceeds or approaches the bearing capacity at the base of the excavation. Figure 21a shows a cut in soft clay H deep and B wide. The block of soil *abcd* exerts a vertical pressure $q_{applied}$ on strip *cd* equal to its weight minus the shear resistance of the soil along plane *bd* ($q_{applied} = \{HB'\gamma - s_u H\}/B'$). The bearing capacity of a cohesive soil is equal to $N_c s_u$, where N_c is the bearing capacity factor and equal to 5.14. For cuts of infinite length, the factor of safety can be estimated as the ratio of the bearing capacity to the bearing pressure as

$$FS = \frac{5.14\, s_u}{H(B'\gamma - s_u)/B'} \qquad \text{. . . [4.4]}$$

Based on Equation 4.4, the factor of safety decreases as width B' increases. However, based on the geometry of the failure surface, B' cannot exceed $B/\sqrt{2}$. Thus, the minimum FS for Equation 4.4 is

$$FS = \frac{5.14\, s_u}{H\left(\gamma - \frac{s_u\sqrt{2}}{B}\right)} \qquad \text{. . . [4.5]}$$

The width, B', is restricted if a stiff stratum is near the bottom of the cut (Figure 21b). For this case, B' is equal to depth D. Substituting D for B' in Equation 4.4, the expression for FS becomes

$$FS = \frac{5.14\, s_u}{H(D\gamma - s_u)/D} \quad \text{(Limited depth to stiff stratum)} \qquad \text{. . . [4.6]}$$

For a very wide, infinitely long, excavation in a homogenous soft to medium clay of constant strength, the factor of safety in Equation 4.4 can be expressed as

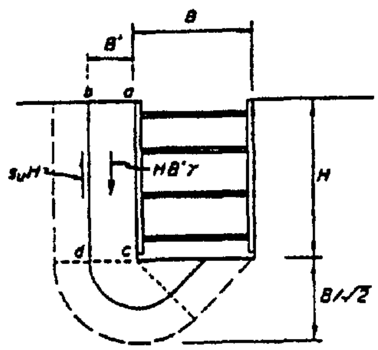
$$FS = \frac{N_c}{\gamma H/s_u} = \frac{5.14}{N_s} \qquad \text{. . . [4.7]}$$

where γ is the total unit weight and N_s is a stability number. The stability number ($N_s = \gamma H/s_u$) has been used to identify excavation support systems with potential for movement and basal heave problems. Small values of N_s (with respect to 5.14) indicate adequate basal stability and small ground movements.

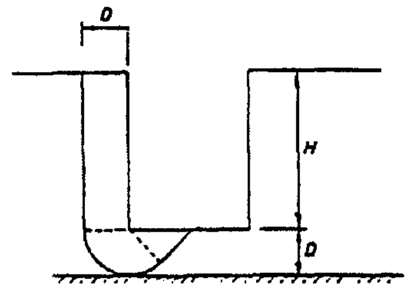
Significant ground movements will occur when the bearing capacity of the underlying soil is approached regardless of the strength of the supports. Current practice is to use a minimum factor of safety of 1.5 against basal heave. A more detailed discussion of the factor of safety against base heave is presented by Terzaghi, et al. (1996).

4.2.2.1 Excavation Geometry Effects on Basal Stability

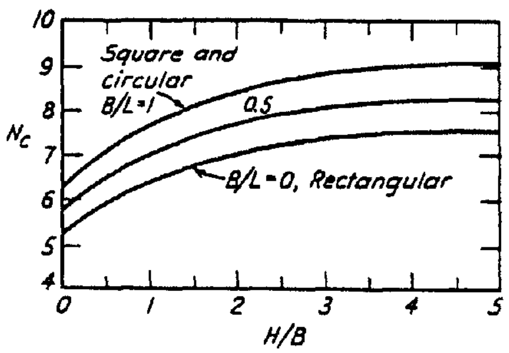
Bearing capacity factors used to evaluate base stability can consider excavation plan dimensions. The bearing capacity factor for the cut is taken to be identical to the factor for a footing with similar plan dimensions. Accordingly, the bearing capacity factor (N_c) is affected by the depth/width ratio (H/B) and the plan dimensions of the cut (B/L). Values of the bearing capacity factor, N_c, proposed by Janbu, et al. (1956) are shown in Figure 21c. Janbu's bearing capacity factor is substituted in the numerator of Equation 4.7 to give an estimate of base stability that takes into account dimensions of the cut. Figure 21c is used when evaluating how large an excavation can be opened at a time in soft ground. Terzaghi and Peck (1967) confirm the use of Janbu's chart, and Peck, et al. (1974) indicated that when N_s exceeded eight, collapse of the wall was likely.



a) Failure planes, deep deposit of weak clay



b) Failure plane, stiff layer below bottom of excavation



c) Bearing capacity factor, N_c

FIGURE 21
Base Stability for a Limited Excavation (from Terzaghi, et al., 1996)

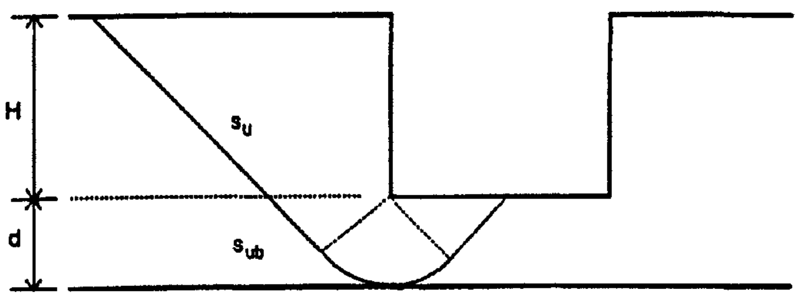
4.2.2.2 Effects of Basal Instability on Apparent Earth Pressures

Terzaghi and Peck (1967) observed that Equation 4.2 (with $K_A = 1 - 4\, s_u/\gamma H$) underestimated lateral pressures exerted on walls supporting cuts in deep deposits of soft to medium clay with poor base stability. Henkel (1971) developed an equation for estimating the active earth pressure coefficient that included deep-seated failures below the bottom of a cut in soft to medium clay. The equation for the earth pressure coefficient is

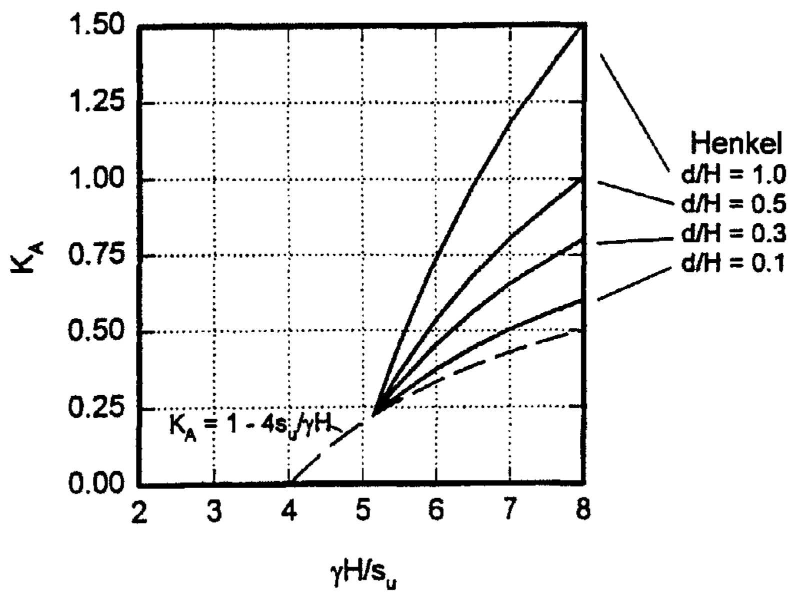
$$K_A = 1 - \frac{4 s_u}{\gamma H} + 2\sqrt{2}\,\frac{d}{H}\left(1 - (2+\pi)\,\frac{s_{ub}}{\gamma H}\right) \qquad \text{. . . [4.8]}$$

where d is the depth of the failure surface below the cut, s_u is the undrained shear strength of the soil through which the excavation extends, and s_{ub} is the strength of the soil providing bearing resistance (Figure 22a). Henkel's value of K_A is used in Equation 4.2 to estimate the apparent earth pressures for deep cuts in soft to medium clay with poor base stability.

Relationships between K_A and $\gamma H/s_u$ are shown in Figure 22b for Henkel's method. For purposes of illustration, the soil in which the excavation occurs is taken as uniform in strength, but with different ratios of d/H (depth of weak soil below the excavation/height of cut). Shown in solid lines are the relationships predicted with Henkel's method. Henkel's active earth pressure coefficients increase with the ratio $\gamma H/s_u$ and with d/H. The Rankine active earth pressure coefficient $(1-4s_u/\gamma H)$ defines the minimum values for K_A in Figure 22b. Agreement between observed apparent earth pressures and those predicted with Henkel's method is good (Christian, 1989).



a) Failure surface assumed for Henkel's method



b) Effect of $\gamma H/s_u$ and d/H on K_A

FIGURE 22
Henkel's Method for Determining K_A for Potential Base Failure

4.3 DETERMINING TOTAL LATERAL SUPPORT LOADS USING LIMITING EQUILIBRIUM METHODS

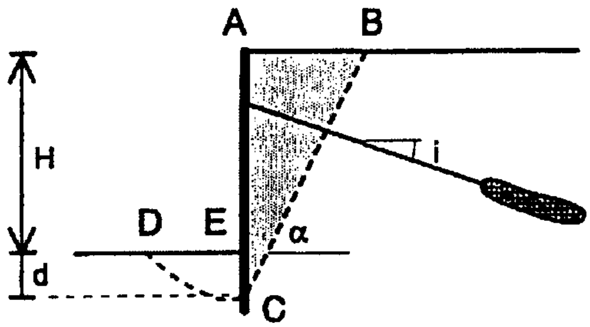
Apparent earth pressure diagrams for sand and soft to medium clays relate measured loads to loads determined from limiting equilibrium analyses (Terzaghi, et al., 1996, and Long, et al., 1998). Limiting equilibrium methods use simple free body diagrams or general purpose slope stability computer programs to calculate the total lateral earth load that must be supported by the ground anchors and the toe. Long, et al. (1998) demonstrated that the total lateral loads from Terzaghi and Peck's sand and soft to medium clay apparent earth pressure diagrams are equal to the total lateral loads determined using limiting equilibrium analyses with a factor of safety of about 1.3 on the shear strength. Therefore, a factor of safety of 1.3 on the shear strength of the soil is recommended when using limiting equilibrium methods to determine the total lateral support loads for the design of anchored walls and landslide stabilization walls. A force equilibrium method and a general purpose slope stability program are used to illustrate how limiting equilibrium methods can be used to determine the total lateral earth load.

4.3.1 Force Equilibrium Method

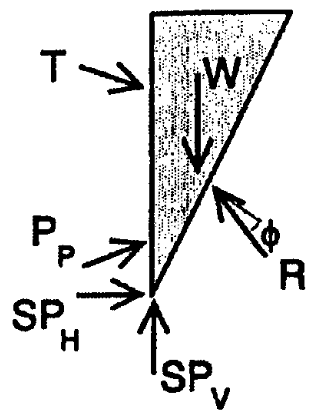
Free body and force diagrams for the force equilibrium method are shown in Figure 23. The anchored wall system retains a vertical cut in a sand with frictional strength, ϕ, average total unit weight, γ, and height, H. The unbonded length of the anchor extends far behind the wall to ensure the critical failure surface passes above the anchor zone and the full anchor load contributes to wall stability. The potential failure plane passes through the toe of the wall at depth, d, and mobilizes a passive resistance from the soil, P_p, and a horizontal and vertical resistance from the wall below the failure surface (SP_h, SP_v, respectively). SP_h will be the smaller of the shear strength of the wall or the lateral resistance of the wall below the failure surface.

For simplicity, the shape of the failure surface is assumed to be a straight line (BC) as shown in Figure 23a. Although many shapes could be used for the failure surface, a straight line approximation for the active portion of retained soil has been found adequate (Taylor, 1948, and Terzaghi, et al., 1996). Beneath the bottom of the cut, the failure surface is assumed to be shaped as a log spiral on the passive side of the soil. For soldier beam and lagging walls, significant soil to soil contact exists, thus interface resistance along the vertical face CE is assumed to be equal to the strength of the soil.

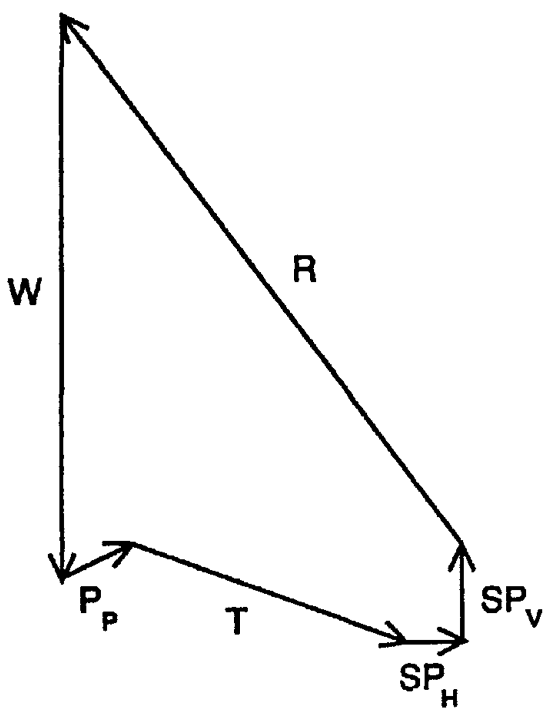
Passive resistance above the failure surface is $P_p = ½ K_p \gamma' d^2$, where γ' is the effective unit weight. Effective unit weight is used if the toe of the wall is submerged below the groundwater table. Passive earth pressure coefficients assuming a log-spiral failure surface in the passive zone are used (Terzaghi, et al., 1996). Passive earth pressure coefficients are shown in Figure 24.



a) Example of a tiedback wall system



b) Free-body diagram



c) Force vectors acting on area ABCE

FIGURE 23
Force Equilibrium Method for an Anchored Wall

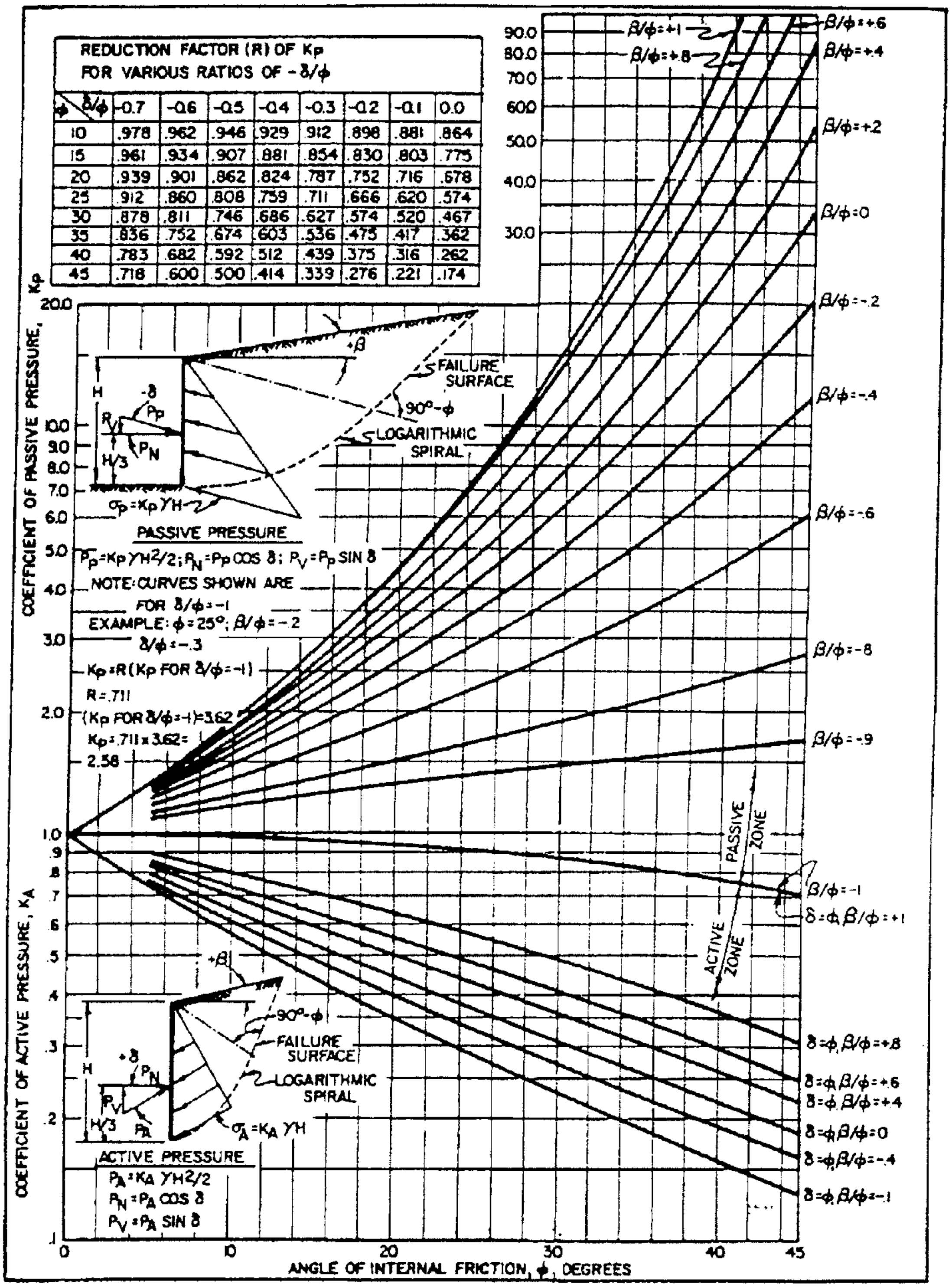


FIGURE 24
Passive Earth Pressure Coefficients (NAVFAC DM 7.2, 1982)

The contribution of forces from the anchor and soldier beam are shown as force vectors T, SP_h, and SP_v in Figure 23c. For simplicity and for ease of comparison with apparent earth pressure diagrams and the general purpose slope stability computer programs, the three forces are treated as a horizontal force with magnitude P_{reqd}. A wall of unit width is assumed. Thus, P_{reqd} represents the horizontal force required to provide stability to the vertical cut per unit width. Taking P_{reqd} as horizontal implicitly assumes that the vertical force in the soldier beam (SP_v) is equal in magnitude (and opposite in direction) to the vertical component of the anchor load ($T{\cdot}\sin(i)$). In addition, the groundwater table is well below the bottom of the cut, and the soil has the same shear strength and unit weight throughout the profile. This allows the forces acting on the soil to be considered in the equilibrium equations (Figure 25).

The solution for the external force required for stability (P_{reqd}) continues by summing the forces in the x-direction.

$$\Sigma F_x = P_p \cos(\delta) + P_{reqd} - R\sin(\alpha-\phi) \qquad \text{. . . [4.9]}$$

and summing forces in the y-direction (vertical) to get

$$\Sigma F_y = W - P_p \sin(\delta) - R\cos(\alpha-\phi) \qquad \text{. . . [4.10]}$$

Combining the two equations and solving for P_{reqd} results in the following expression

$$P_{reqd} = \frac{1}{2}\gamma H^2 \left[\frac{(1+\xi)^2}{\tan(\alpha)} - K_p \xi^2 \left(\sin(\delta) + \frac{\cos(\delta)}{\tan(\alpha-\phi)}\right)\right] \tan(\alpha-\phi) \qquad \text{. . . [4.11]}$$

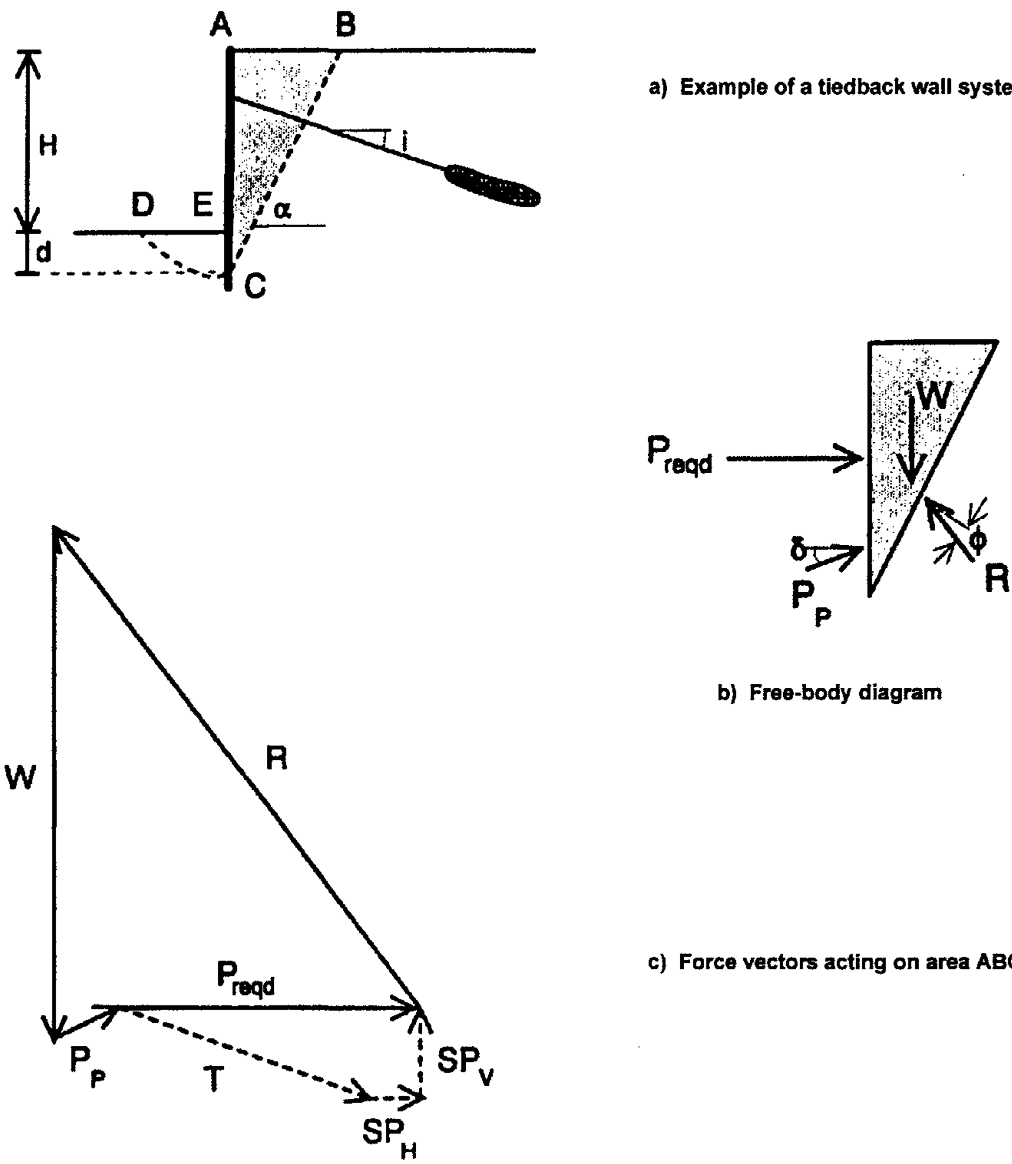
where ξ is the ratio of d/H, α is the angle of the failure plane with respect to the horizontal (all other parameters have been defined previously). The solution proceeds by varying values of ξ and α until a maximum for P_{reqd} is determined. Values of P_{reqd} are for a factor of safety of 1.0. Solutions were determined for soil friction angles of 20°, 30°, and 40°, and are presented in Table 6 in non-dimensional form as $K_{reqd} = P_{reqd}/(\frac{1}{2}\gamma H^2)$.

TABLE 6
Magnitudes of K_{reqd} for the Force Equilibrium Method (base failure)

ϕ (deg)	K_{reqd}	ξ	α
20	0.570	0.162	54
30	0.349	0.047	60
40	0.220	0.012	65

Thus, the total load required to support a vertical cut of height H is $P_{reqd} = \frac{1}{2}\gamma H^2 K_{reqd}$ with a factor of safety of 1.0.

The solution for Equation 4.11 includes failure surfaces that pass below the bottom of the cut. This explains why K_{reqd} for a soil with a friction angle of 30° is about 4 percent greater than the Rankine active earth pressure coefficient. For soils with a friction angle less than 30°, the difference between K_a and K_{reqd} increases since the failure surface drops farther below the bottom of the cut.



a) Example of a tiedback wall system

b) Free-body diagram

c) Force vectors acting on area ABCE

FIGURE 25
Force Equilibrium Method for an Anchored Wall with P_{reqd}

4.3.2 Slope Stability Computer Analysis

General purpose slope stability computer programs can be used to determine the total lateral earth load for the design of temporary and permanent ground anchor walls. These programs allow complicated surcharge loads, groundwater, and layered soil deposits to be modeled. They also can model ground whose shear strength is developed from frictional resistance and cohesion. However, many general purpose slope stability computer programs are unable to model the ground anchors as concentrated loads on the face of the wall. Apparently, they do not distribute the concentrated ground anchor loads properly throughout the slices to the failure surface.

Ground anchors apply large horizontal loads to the ground mass in the direction of the normal forces acting on the vertical sides of the slices. To solve the equilibrium equations for each slice and the overall mass, each analysis method makes assumptions regarding these interslice forces, and these assumptions may not be valid for anchored walls. Many of these programs can be used for ground anchor wall design by determining the lateral earth pressures. To determine the lateral earth pressures, apply a surcharge load to the face of the wall. The surcharge load will be directed toward the ground being supported. To distinguish the wall from a vertical slice and the surcharge load from an interslice force, the wall is battered slightly (usually 1 ft). Figure 26 shows the graphical output from a STABL5M (Achilleos 1988) analysis where a horizontal surcharge load equivalent to a total lateral earth load of 23,800 lb was applied to the wall, giving a factor of safety of 1.3. In the analysis the soil was assumed to have a friction angle of 30° and a unit weight of 115 pcf. A horizontal surcharge load was used so the load could be compared with that developed from the force equilibrium method and Terzaghi and Peck's sand diagram.

Table 7 compares the total lateral earth load computed using the force equilibrium method, a general purpose slope stability computer program, and Terzaghi and Peck's apparent earth pressure diagram. Each analysis assumed a 30-ft-high wall, a soil friction angle of 30°, and a soil unit weight of 115 pcf. A factor of safety of 1.3 was applied to the shear strength when computing the load using the force equilibrium method and the computer analyses. The mobilized friction angle for each limiting equilibrium analysis is $\tan^{-1}\phi_{mob} = (\tan\phi)/1.3$. The mobilized friction angle for the apparent earth pressure diagram was computed by solving Equation 4.12.

$$\phi_{mob} = 2\left(45° - \tan^{-1}\left[\sqrt{1.3}\,\tan\left(45 - \frac{\phi}{2}\right)\right]\right) \qquad \ldots [4.12]$$

The factor of safety on shear strength for each analysis was expressed as $FS = \tan\phi/\tan\phi_{mob}$.

Lateral loads and the locations of the failure surfaces were similar for the limiting equilibrium methods. Computed load from the apparent earth pressure diagram was about 7 percent lower than the loads determined using the limiting equilibrium methods. The difference in the loads is primarily a result of differences in the failure surfaces analyzed. Apparent earth pressure diagrams assume that the failure surface goes through the bottom corner of the excavation. Limiting equilibrium methods allowed failure surfaces to go below the bottom corner of the cut.

Long, et al. (1998) present a complete comparison of the force equilibrium method and apparent earth pressure diagram for different friction angles. The report shows that the results from the force equilibrium method and the apparent earth pressure diagram converge as the strength of the soil increases and diverge as the strength decreases. It also shows that the factor of safety on shear strength in the apparent earth pressure diagram varies depending upon soil strength. When the failure surfaces for both methods were similar, the factors of safety were about 1.3.

Before using a general purpose slope stability program, carefully check the program to ensure that it gives reasonable results. The force equilibrium method presented here can be used to check the calculations for several simple cases. The computer program and the force equilibrium method should give similar results for similar failure surfaces. A factor of safety of 1.3 on the shear strength of the soil will give lateral earth loads similar to those estimated using Terzaghi and Peck's apparent earth pressure diagrams. Terzaghi and Peck's sand diagram was developed for soils with friction angles between 35° and 40°. At these friction angles, the limiting equilibrium failure surfaces pass near the bottom corner of the cut and the factor of safety is about 1.3. Limiting equilibrium analyses will predict higher loads than the apparent earth pressure diagrams when the critical failure surface is below the bottom of the cut. The differences become larger as the strength of the ground decreases.

TABLE 7
Total Lateral Earth Loads Computed Using Limiting Equilibrium Methods and the Apparent Earth Pressure Diagram
(H = 30 ft, ϕ = 30°, γ = 115 pcf)

CALCULATION METHOD	TOTAL LATERAL Earth Load (lb)	MOBILIZED FICTION ANGLE (deg)	FS ON SOIL STRENGTH	DEPTH OF FAILURE SURFACE (ft)	ANGLE OF FAILURE SURFACE, α (deg)
Apparent Earth Pressure	22402	23.29	1.34	0.00	56.7
Force Equilibrium Method	24028	23.95	1.30	3.00	56.0
Computer	23800	23.95	1.30	4.38	58.9

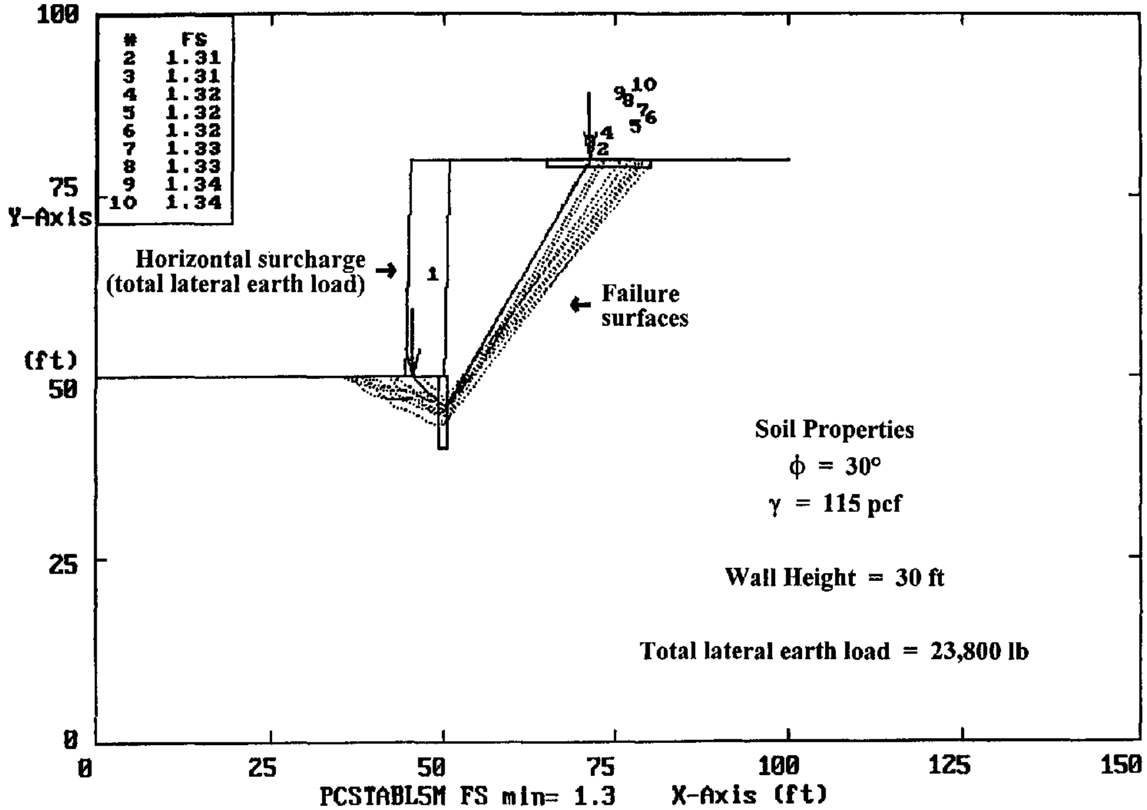


FIGURE 26
Graphical Output from a STABL5M Analysis, H = 30 ft, ϕ = 30°, γ = 130 pcf

4.3.3 Slope Stability Analysis Guidelines

When using a general purpose slope stability program to determine the total lateral earth load, do the following:

- In sandy ground, select an analysis method that uses force equilibrium and planar failure surfaces (Janbu's method).
- For clayey soils, select a moment equilibrium method (Bishop's method) and use circular failure surfaces for the analysis.

Model the total lateral earth load by applying a surcharge load to the face of the wall. When the wall penetrates the failure surface, use a horizontal surcharge load in the stability analysis. Here, the vertical component of the ground anchor load will be transmitted to the ground below the failure surface. If the toe of the wall does not penetrate the failure surface, the surcharge load should be inclined at the same angle as the ground anchor. When the wall does not penetrate the failure surface, the horizontal and vertical components of the ground anchor load are transmitted to the failure surface. Figure 27 illustrates the two different cases and shows how the ground anchor load should be modeled in the limiting equilibrium analyses.

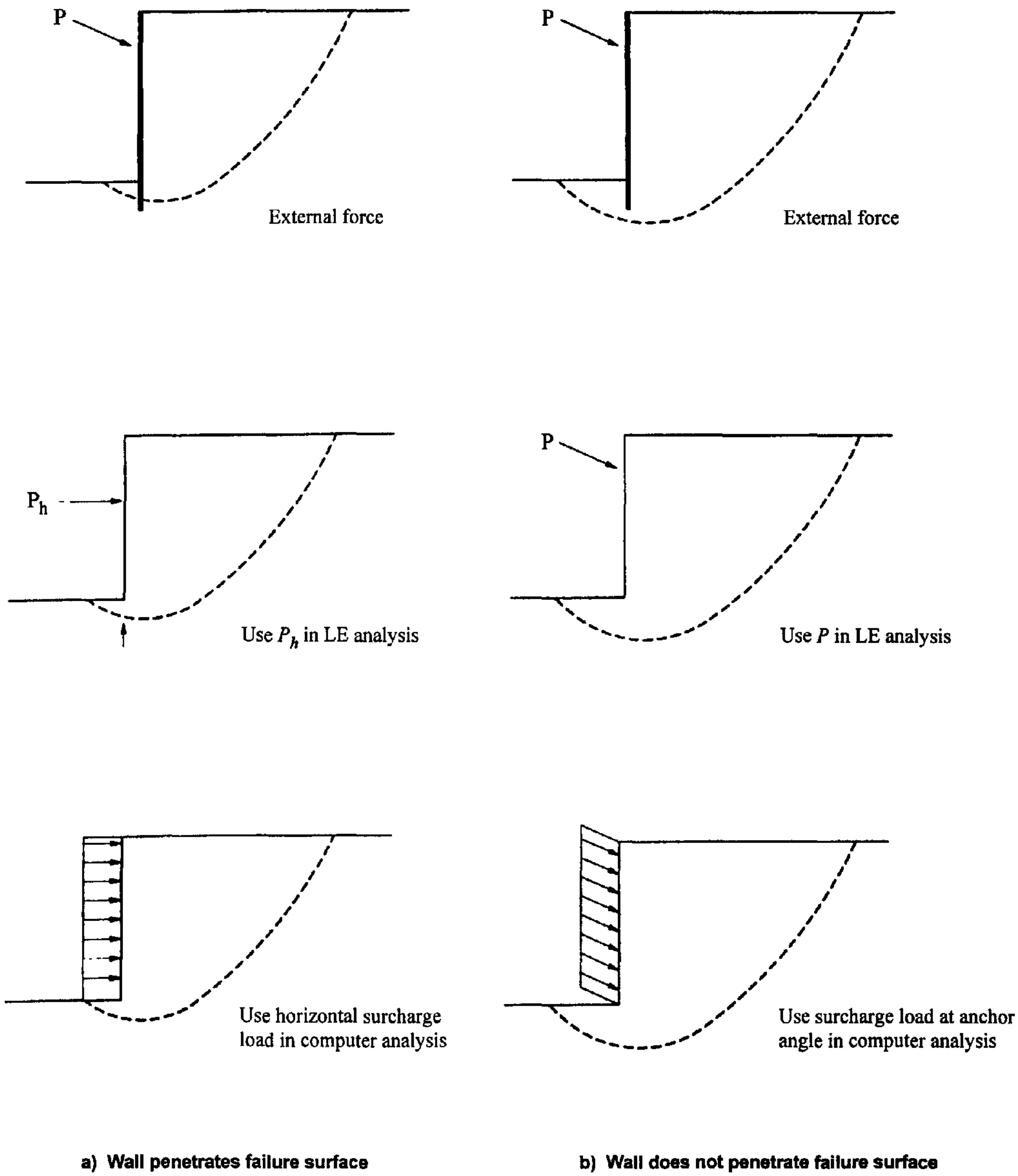


FIGURE 27
Modeling of the Ground Anchor in Limiting Equilibrium Analysis

4.4 APPARENT EARTH PRESSURE DIAGRAM FOR SAND

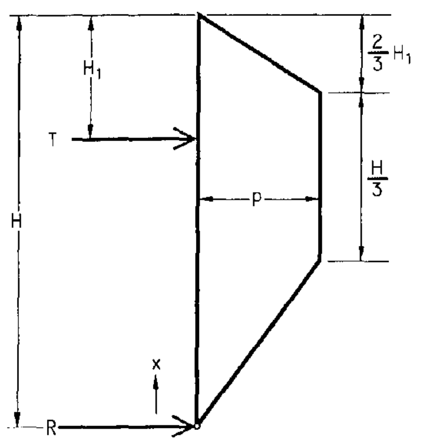
Figures 28 and 29 show new apparent earth pressure diagrams for coarse-grained soils. The diagram in Figure 28 is for a wall supported by one row of anchors and the diagram in Figure 29 is for a wall supported by multiple rows of anchors. The intensity of the pressure in these diagrams is calculated from the total lateral earth load. Total lateral earth load can be the total load from Terzaghi and Peck's sand diagram ($0.65K_a\gamma H^2$) or the load determined from a limiting equilibrium analysis. Cording (1996) called these pressure diagrams "non-symmetrical trapezoidal pressure diagrams." Additional discussion about the non-symmetrical earth pressure diagrams is contained in *Summary Report of Research on Permanent Ground Anchor Walls*, "Volume II: Full-scale Wall Tests and a Soil-Structure Interaction Model," (Weatherby, et al., 1998). The earth pressure in these diagrams increases to a maximum at a depth equal to two-thirds the distance to the upper ground anchor. For a wall supported by one row of anchors, the maximum pressure continues downward for a distance equal to one-third the height of the wall. Below that depth, the pressure decreases linearly to zero at the bottom of the excavation. The total lateral earth load is $0.65K_a\gamma H^2$, the total load from the Terzaghi and Peck diagram, and the intensity of the maximum pressure on the one-row wall is approximately $K_a\gamma H$. For a wall supported by multiple levels of ground anchors, the maximum earth pressure continues to a point below the lowest support equal to one-third the distance from the lower support to the bottom of the excavation. From there the pressure decreases linearly to zero at the bottom of the excavation. The total lateral load for the multi-tiered wall is the same as that for the one-tier wall, $0.65K_a\gamma H^2$. The non-symmetrical trapezoid is more appropriate than the rectangular diagram for the design of flexible, soldier beam walls supported by ground anchors since:

- Measurements show that arching concentrates the earth pressures at the ground anchor locations.
- The earth pressures in a sand deposit must be zero at the ground surface.
- Actual earth pressures increase from the ground surface to the ground anchor location.
- Bending moments predicted using the non-symmetrical diagram fit measured results better than those predicted by other apparent earth pressure diagrams.
- Ground anchor loads determined from the non-symmetrical trapezoid diagram are similar to those determined using other apparent earth pressure diagrams.

Equations for determining the ground anchor loads and the soldier beam bending moments are presented in Figures 28 and 29. These equations use the tributary area method for determining ground anchor loads. The equation for the bending moment at the upper anchor sums moments about the anchor. Bending moment equations below the upper support are based on a moment factor and the maximum intensity of the earth pressure. Locations of the lower moments are not determined. These equations are easily incorporated into a spreadsheet.

The total lateral earth load from Terzaghi and Peck's sand diagram is $0.65K_a\gamma H^2$. Total lateral earth load can be expressed as an earth pressure factor ($0.65K_a\gamma$) times the square of the wall height. Figure 30 is a plot of earth pressure factors versus standard penetration resistance for the coarse-grained soils in Table 8. Figure 30 assumes that the groundwater table is at or below the base of the wall and the unit weight of the soil is the total unit weight. The earth pressure factors for the soils in the table vary within a narrow range between 20 and 24 pcf. The narrow range of earth pressure factors reflects the relationship among friction angle, total unit weight, and density. When the deposit is dense, the friction angle and the total unit weight are high. The opposite is true when the deposit is less dense. Since the earth pressure factors were developed from Terzaghi and Peck's sand diagram, they include a factor of safety of about 1.3 defined on soil shear strength. Figure 30 can be used to determine the total lateral earth load to be used to develop the pressure for the non-symmetrical earth pressure diagram.

Limiting equilibrium analyses can be used to determine the pressure for the non-symmetrical trapezoid. Determine the total load that corresponds to a factor of safety of 1.3 on the shear strength. The total lateral earth load to be used to calculate the maximum apparent earth pressure is the horizontal component of the load from the limiting equilibrium analysis. Figures 28 and 29 contain the equations for the intensity of the earth pressure from the total lateral earth load.



$$M_1 = \frac{13}{54} H_1^2 p$$

$$T_1 = \frac{(23H^2 - 10HH_1)}{54(H-H_1)} p$$

$$R = \frac{2}{3} Hp - T_1$$

Solve for point of zero shear

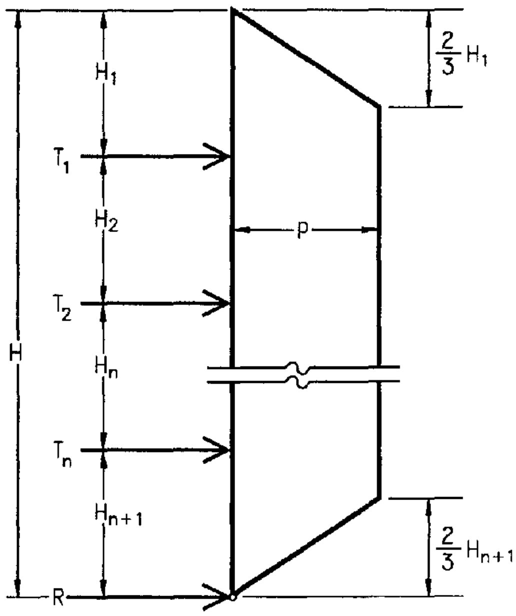
$$x = \frac{1}{9}\sqrt{26H^2 - 52HH_1}$$

$$MM_1 = Rx - \frac{px^3}{4(H-H_1)}$$

Earth Pressure "p" Determined from Total Load Required to Stabilize the Cut.

$$p = \frac{\text{Total Load}}{\frac{2}{3}H}$$

FIGURE 28
Recommended Apparent Earth Pressure Diagram for Wall Supported by One Row of Anchors



$$M_1 = \frac{13}{54} H_1^2 p$$

$$T_1 = \left(\frac{2}{3} H_1 + \frac{H_2}{2}\right) p$$

$$MM_1 = \frac{1}{10} H_2^2 p$$

$$MM_2 = \frac{1}{10} H_3^2 p$$

$$MM_n = \frac{1}{10} H_{n+1}{}^2 p$$

$$MM_{n+1} = \frac{1}{10} H_{n+2}{}^2 p$$

$M_2 =$ Larger of MM_1 or MM_2

$M_n =$ Larger of MM_n or MM_{n+1}

$$T_2 = \left(\frac{H_2}{2} + \frac{H_n}{2}\right) p$$

$$T_n = \left(\frac{H_n}{2} + \frac{23 H_{n+1}}{48}\right) p$$

$$R = \left(\frac{3}{16} H_{n+1}\right) p$$

Earth Pressure "p" Determined from Total Load Required to Stabilize the Cut.

$$p = \frac{\text{Total Load}}{H - \frac{1}{3} H_1 - \frac{1}{3} H_{n+1}}$$

FIGURE 29
Recommended Apparent Earth Pressure Diagram for Wall Supported by Multiple Rows of Anchors

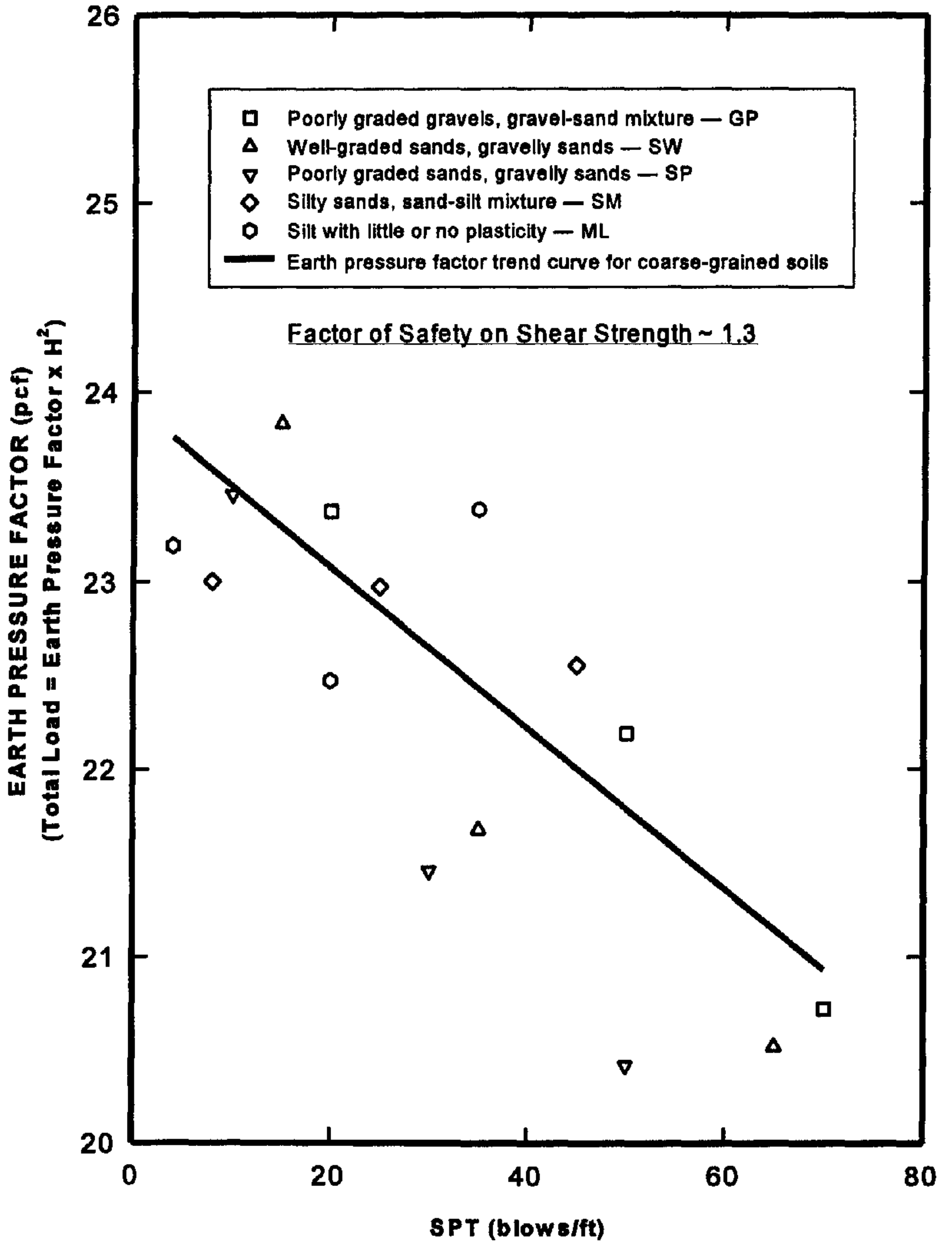


FIGURE 30
Earth Pressure Factors as a Function of Standard Penetration Resistance

TABLE 8
Earth Pressure Factors for Typical Course-grained Soils

SOIL TYPE	RELATIVE DENSITY	SPT (blow/ft) (1)	FRICTION ANGLE, ϕ (deg)	ACTIVE EARTH PRESSURE COEFFICIENT K_a	TOTAL UNIT WEIGHT, γ (pcf)	EARTH PRESSURE FACTOR (pcf)
GP (poorly graded gravel, gravel sand mixture)	75% (dense)	70	38	0.238	134	20.72
	50% (medium dense)	50	35	0.271	126	22.19
	25% (loose)	<20	32	0.307	117	23.37
SW (Well-graded sand, gravelly sand)	75% (dense)	65	37	0.249	127	20.52
	50% (medium dense)	35	34	0.283	118	21.68
	25% (loose)	<15	30	0.333	110	23.83
SP (poorly graded sand, gravelly sand)	75% (dense)	50	36	0.260	121	20.42
	50% (medium dense)	30	33	0.295	112	21.46
	25% (loose)	<10	29	0.347	104	23.46
SM (silty sand, sand silt mixture)	75% (dense)	45	35	0.271	128	22.55
	50% (medium dense)	25	32	0.307	115	22.97
	25% (loose)	<8	29	0.347	102	23.00
ML (silt with little or no plasticity)	75% (dense)	35	33	0.295	122	23.38
	50% (medium dense)	20	31	0.320	108	22.47
	25% (loose)	<4	27	0.376	95	23.19

(1) Adjustments for gradation are after Burmister (1962), no adjustment made for depth of overburden

4.5 APPARENT EARTH PRESSURE DIAGRAM FOR STIFF CLAY

A non-symmetrical apparent earth pressure diagram identical in shape to the one recommended for sand is recommended for the design of ground anchor walls built in stiff clay deposits when undrained conditions exist. An earth pressure factor of 20 pcf is recommended for these soils unless experience suggests that a higher factor should be used. Higher earth pressure factors may be warranted if the strength of the clay has been reduced along major discontinuities (see Section 4.8). An earth pressure factor of 20 pcf gives the same total lateral earth pressure as Schnabel's 25H trapezoid, and higher loads than the Terzaghi and Peck stiff fissured clay trapezoid with a pressure of $0.2\gamma H$.

The transition from using a stiff clay apparent earth pressure diagram to a soft to medium clay diagram does not occur at a unique undrained shear strength. For a given wall height or excavation depth, H, the undrained shear strength of the soil shall satisfy Equation 4.13 in order to use the stiff clay apparent earth pressure diagram.

$$s_u \geq \frac{H}{4}(\gamma - 22.857) \qquad \ldots [4.13]$$

Figure 31 is another way to determine whether a stiff clay apparent earth pressure diagram should be used for a particular soil and wall height. The figure shows earth pressure factors for typical clay soils as a function of undrained shear strength and depth of excavation or wall height. (The development of Figure 31 is presented in Section 4.6.) To decide if a stiff clay, non-symmetrical earth pressure diagram is appropriate, find the undrained shear strength and go upward to the horizontal line representing the minimum earth pressure factor, 20 pcf. If the height of the wall or the proposed depth of excavation is to the left of the intersection of the vertical line and the 20 pcf line, then the soil is considered a stiff clay. If the height of the wall is to the right of the vertical line, the soil is considered a soft to medium clay. Sections 4.6 and 4.7 discuss apparent earth pressures for soft to medium clays.

Limiting equilibrium analyses cannot be used to calculate the total lateral earth load for a wall built in a stiff clay. Loads on walls in a stiff clay correspond to a quasielastic state instead of a state of limiting equilibrium (Section 4.2). Limiting equilibrium analyses suggest that the ground is strong enough to support itself, or give loads that are too low based on measured strut loads.

Long-term earth pressures for stiff-fissured clays may depend upon the drained shear strength of the soil, and could be higher than those determined using the stiff clay apparent earth pressure diagram. Therefore, compute earth pressures using drained sheared strengths and reasonable porewater pressures. Design the wall to support the larger of the loads determined using undrained or drained shear strengths. See Section 4.8 for a discussion of the drained analysis.

Poor drilling techniques using air or water to clean the drill hole may fracture the soil and reduce the soil's shear strength or pressurize the drilling fluid in open fractures. The strength reduction or the effect of pressuring the drilling fluid is not considered in design. Fracturing the ground is controlled by preventing "collaring" of the hole when drilling with air or water. A collar occurs when the hole becomes blocked and cuttings no longer return up the drill hole to the surface. If a collar occurs, the pressurized drilling fluid (air or water) is forced into the ground, disrupting the formation. Auger drilling methods will not disrupt the soil where collaring is likely.

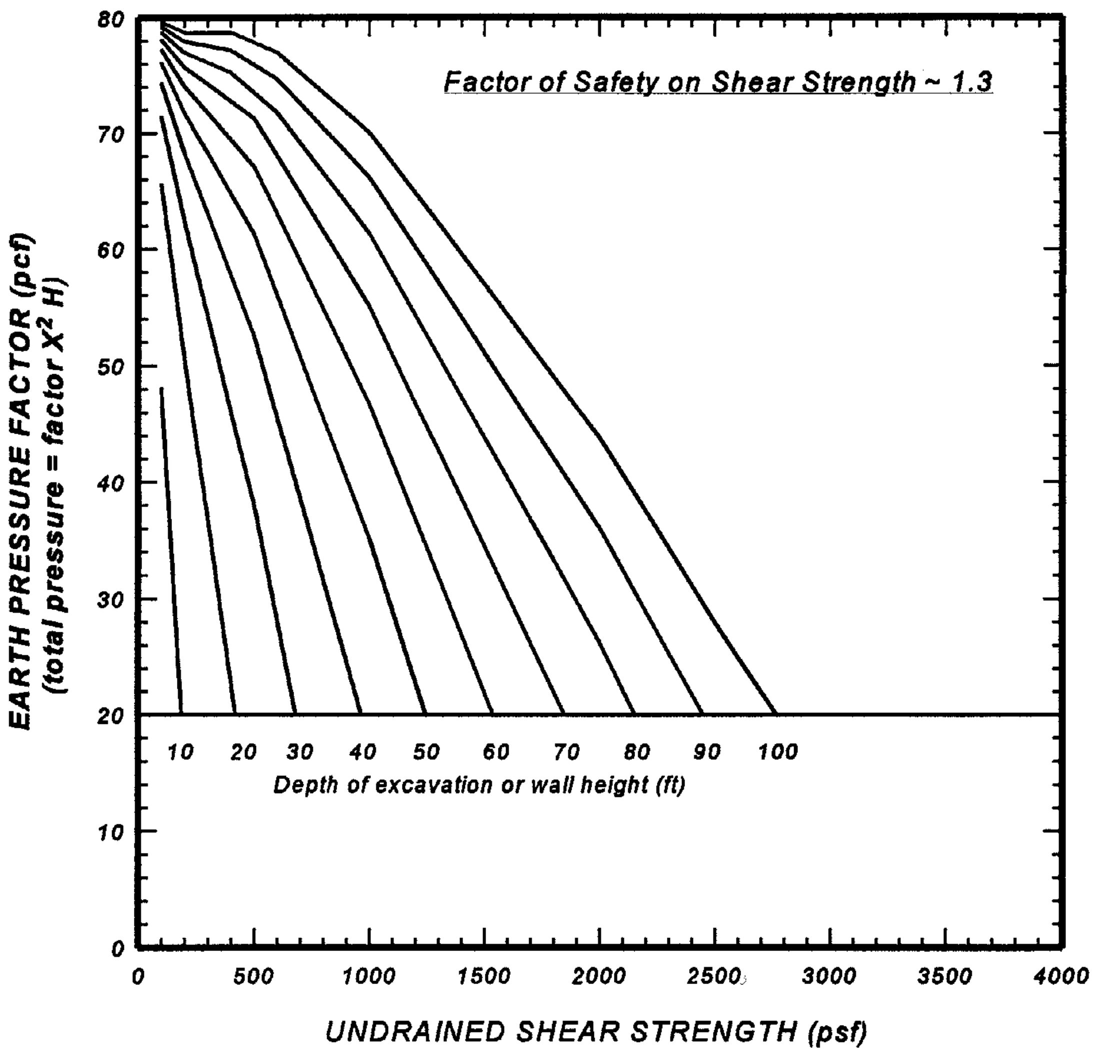


FIGURE 31
Earth Pressure Factors for Fine-grained Soils as a Function of Undrained Shear Strength

4.6 APPARENT EARTH PRESSURE DIAGRAM FOR SOFT TO MEDIUM CLAY, NO DEEP-SEATED FAILURE

Temporary and permanent ground anchor walls in soft to medium clay must resist the short-term lateral earth pressures determined using undrained shear strengths and total unit weights. This section discusses the determination of these pressures when competent ground is at or near the bottom of the excavation. Section 4.7 discusses undrained analyses when weak soils lie below the bottom of the excavation. For permanent ground anchor walls, long-term earth pressures determined using drained shear strengths and effective unit weights may be greater than pressures determined using undrained shear strengths. Section 4.8 discusses apparent earth pressure diagrams developed using drained shear strengths.

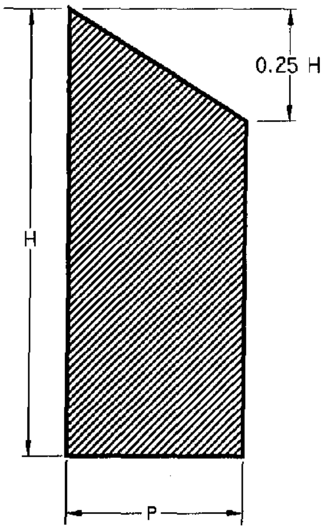
Terzaghi and Peck's soft to medium clay diagram forms the basis for determining the lateral earth pressures based on the undrained strength of a clay. Earth pressure factors, similar to those developed for sand, were determined for the typical clay soils identified in Table 9 at different wall heights. Earth pressure factors for clay are equal to $0.875(1-4s_u/\gamma H)\gamma$. Undrained shear strength of a fine-grained soil can vary greatly for small changes in the total unit weight. Calculate the earth pressure factor using the preceding equation if the undrained shear strength or the total unit weight are significantly different from those given in Table 9. Earth pressure factor curves for different wall heights are plotted in Figure 31 as a function of undrained shear strengths. To determine the total lateral earth load for a soft to medium clay, select the undrained shear strength that best represents the ground and go vertically upward until the curve for the desired wall height is reached. (If the wall height curve is to the left of the vertical line, then the soil is considered a stiff clay (Section 4.5).) Then go horizontally to the ordinate to select the appropriate earth pressure factor. The total lateral earth load is the earth pressure factor times the square of the wall height. The total lateral earth load is distributed to the wall using an earth pressure diagram with the same shape as Terzaghi and Peck's soft to medium clay diagram. Figure 32 shows the earth pressure diagram and contains an equation for determining the intensity of the earth pressure. The total load determined by the earth pressure factors or the Terzaghi and Peck soft to medium clay diagram has a factor of safety defined on undrained shear strength of about 1.3 (Long, et al., 1998).

Earth pressures for soft to medium clay correspond to a state of limiting equilibrium. Therefore, limiting equilibrium analyses can be used to calculate the total lateral earth loading that should be distributed to the wall using the apparent earth pressure diagram in Figure 32. A factor of safety on the shear strength of 1.3 should be used in the analysis.

Poor drilling techniques using air or water to clean the drill hole may fracture the soil and pressurize the drilling fluid in open fractures in weak fine-grained soils. The effect of pressuring the drilling fluid is not considered in design. Fracturing the ground is controlled by preventing collaring of the hole when drilling with air or water. A collar occurs when the hole becomes blocked and cuttings no longer return up the drill hole to the surface. If a collar occurs, the pressurized drilling fluid (air or water) is forced into the ground disrupting the formation. Auger drilling methods will not disrupt the soil where collaring is likely.

TABLE 9
Typical Undrained Shear Strengths and Total Unit Weights for Clays

UNDRAINED SHEAR STRENGTH (psf)	TOTAL UNIT WEIGHT (pcf)
100	95
200	98
250	100
300	102
400	106
500	110
600	112
700	114
800	116
900	118
1000	120
1100	121
1200	122
1300	123
1400	124
1500	125
1600	126
1700	127
1800	128
1900	129
2000	130
2500	132
3000	135
4000	140



Total Lateral Earth Load

Determined Using:

a) Earth Pressure Factor From Figure 31 $(EPF)(H^2)$

b) From Limiting Equilibrium Analysis

Intensity of Apparent Earth Pressure Diagram

$$p = \frac{\text{Total Lateral Earth Load}}{0.875H}$$

FIGURE 32
Apparent Earth Pressure Diagram for Soft Clay

4.7 APPARENT EARTH PRESSURE DIAGRAM FOR SOFT TO MEDIUM CLAY, DEEP-SEATED FAILURE

Permanent ground anchor walls are not recommended for sites where the bottom of the wall is underlain by deep deposits of weak soils, but temporary earth retaining walls are built at these locations. Earth pressures computed from the soft to medium clay diagram (Figure 19b) or from earth pressure factors (Section 4.6) underestimate the total lateral earth load when weak soils extend below the bottom of the wall. Loads higher than those predicted develop when the soil below the wall yields plastically. Terzaghi, et al. (1996) recommended that Henkel's method (Section 4.1) be used to calculate the value of K_a to be used in the soft to medium apparent earth pressure diagram. Limiting equilibrium analyses are recommended for determining the total lateral earth load on temporary earth retaining walls constructed in soft to medium clays and subject to deep-seated failures. Limiting equilibrium methods account for plastic yielding, basal heave, and the failure mechanism analyzed by Henkel.

Use moment equilibrium methods with circular failure surfaces for limiting equilibrium analyses in soft to medium clay soils. Undrained shear strengths are used in the analysis. A factor of safety of 1.3 on the shear strength should be used. Since moment equilibrium methods are recommended, each ground anchor will have a different moment arm (Figure 33). Consequently, the stabilizing effect of each anchor will depend upon the magnitude of the anchor load and its moment arm. Two limiting equilibrium analyses are necessary to develop reasonable earth pressure diagrams for walls subject to a deep-seated failure. First, run an analysis forcing the failure surfaces to go through the bottom corner of the wall. The second analysis is run allowing the failure surfaces to go below the bottom of the wall. In the second analysis, keep the surcharge load from the first analysis over the upper half of the wall, and apply a second surcharge load to the bottom half of the wall. Increase the lower surcharge load until the desired factor of safety is obtained. Construct an apparent earth pressure diagram from the two surcharge diagrams. Figure 34 illustrates how these two analyses are done. Bending moments and ground anchor loads are computed from the composite diagram.

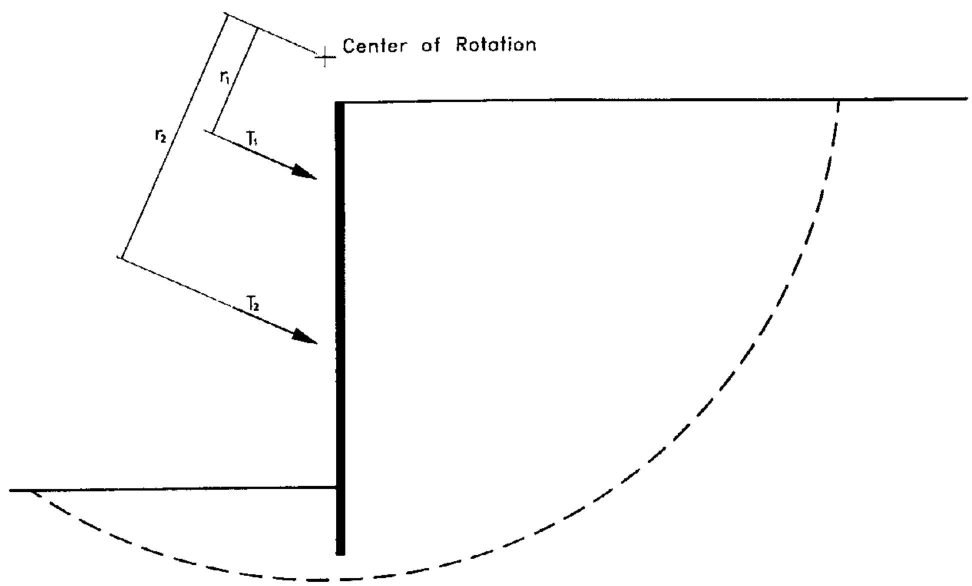
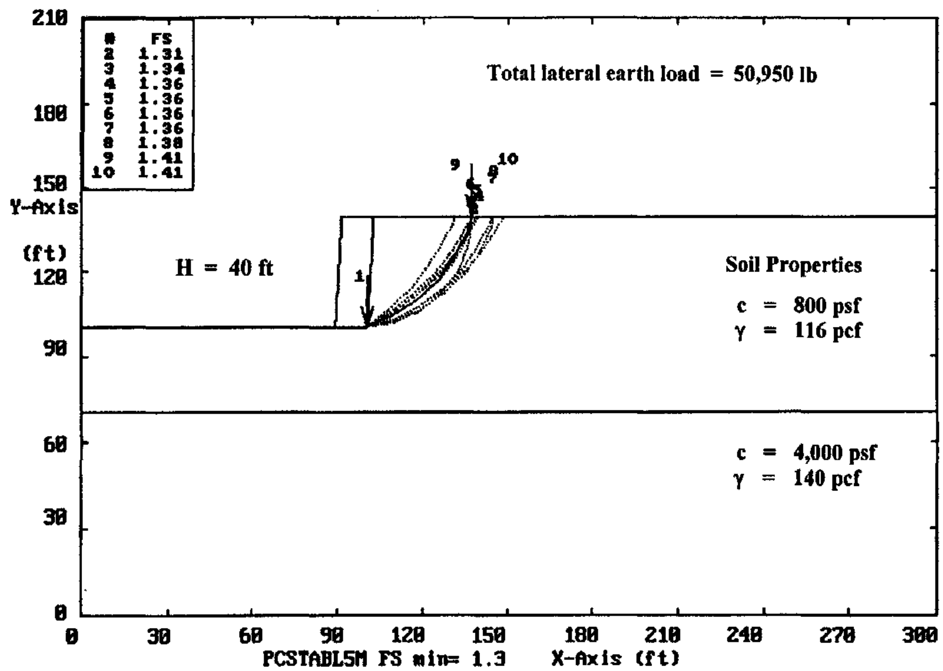
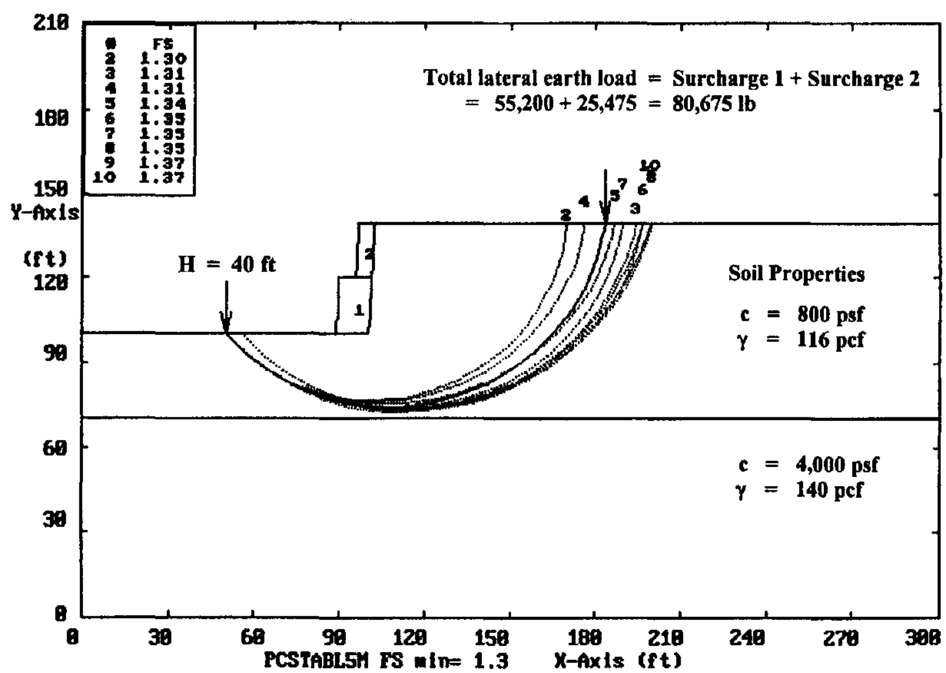


FIGURE 33
Moment Arm for Ground Anchor



a) Load determined by analyzing failure surfaces through the bottom corner of the excavation



b) Load determined by analyzing deep-seated failure surfaces

FIGURE 34
Example of Limiting Equilibrium Analyses for Determing Lateral Earth Pressures for Cuts in Soft to Medium Clay with Deep-seated Failures

4.8 APPARENT EARTH PRESSURE DIAGRAM FOR CLAY, DRAINED SHEAR STRENGTH

Terzaghi and Peck's soft to medium clay apparent earth pressure diagram relates the earth pressures to the undrained shear strength and the total unit weight of the soil. Their diagram for stiff fissured clays uses a factor times the total stress at the bottom of the excavation. Experience has shown that Terzaghi and Peck's or Schnabel's diagrams are valid for temporary earth support systems. They have been successfully used for the design of permanent ground anchor walls. However, anchored walls have not been in service long enough to determine whether fully drained conditions will develop in the soil behind the wall. Since permanent ground anchor walls have a design life greater than 50 yr, AASHTO (1996) recommends that permanent ground anchor walls in cohesive soils be checked for earth pressures associated with drained shear strengths and effective stresses. Design using drained strengths requires the selection of the correct shear strength parameters and the determination of the equilibrium porewater pressures within the ground behind the wall.

The drained shear strength of a given cohesive soil depends upon stress history (degree of overconsolidation), discontinuities (fissures, slickensides, joints, and shears), conditions during geological unloading and associated swelling, weathering, and level of effective normal stress. Drained shear strengths for a cohesive soil may be expressed as the normally consolidated (fully softened) shear strength, the intact strength of an overconsolidated clay, the destructured strength of an overconsolidated clay, or the residual strength (Terzaghi, et al., 1996).

The drained shear strength of a normally consolidated cohesive soil depends on the drained friction angle ϕ' and the effective normal stress σ', and is expressed by the relationship

$$s = \sigma' \tan \phi' \qquad \text{. . . [4.14]}$$

The effective normal stress σ' on the shear plane is the total normal stress, γz, on the plane less the porewater pressure after equilibrium is reached. Friction angle ϕ' depends on the clay content of the soil, clay mineralogy, and arrangement of the clay particles. Figure 35 (from Terzaghi, et al., 1996) shows how ϕ' varies with the plasticity index for normally consolidated clays. Data points far above the line represent soils that have an effective normal stress less than 1,000 psf and a clay content less than 20 percent, and data points well below the line represent soils having effective normal stresses greater than 8350 psf and clay contents greater than 50 percent.

Permanent ground anchor walls are seldom built in normally consolidated clay deposits. Anchored walls are routinely built in overconsolidated clays and many of these walls have been designed using the Terzaghi and Peck stiff clay pressure diagram or Schnabel's 25H trapezoid. The drained strength of an overconsolidated clay should be greater than the drained strength of a similar soil in a normally consolidated state. The drained shear strength of a saturated overconsolidated clay is called the intact shear strength, and is defined with respect to the cohesion intercept c' and the friction angle ϕ' of a Mohr failure envelope by Equation 4.15.

$$s = c' + \sigma' \tan\phi' \qquad \text{. . . [4.15]}$$

The drained friction angle for a normally consolidated clay and an intact overconsolidated clay are not the same (Terzaghi, et al., 1996). Friction angles for the intact overconsolidated clay will be higher at effective stresses lower than the preconsolidation pressure, and trend toward the normally consolidated friction angle at high effective normal stresses. Terzaghi, et al. (1996) used Equation 4.16 to express the drained strength of an overconsolidated clay in terms of the drained strength of the same soil in its normally consolidated state, the overconsolidation ratio, OCR, and a term m that depends upon the extent of the fissures in the soil.

$$s = \sigma' \tan\phi' \, OCR^{1-m} \qquad \text{. . . [4.16]}$$

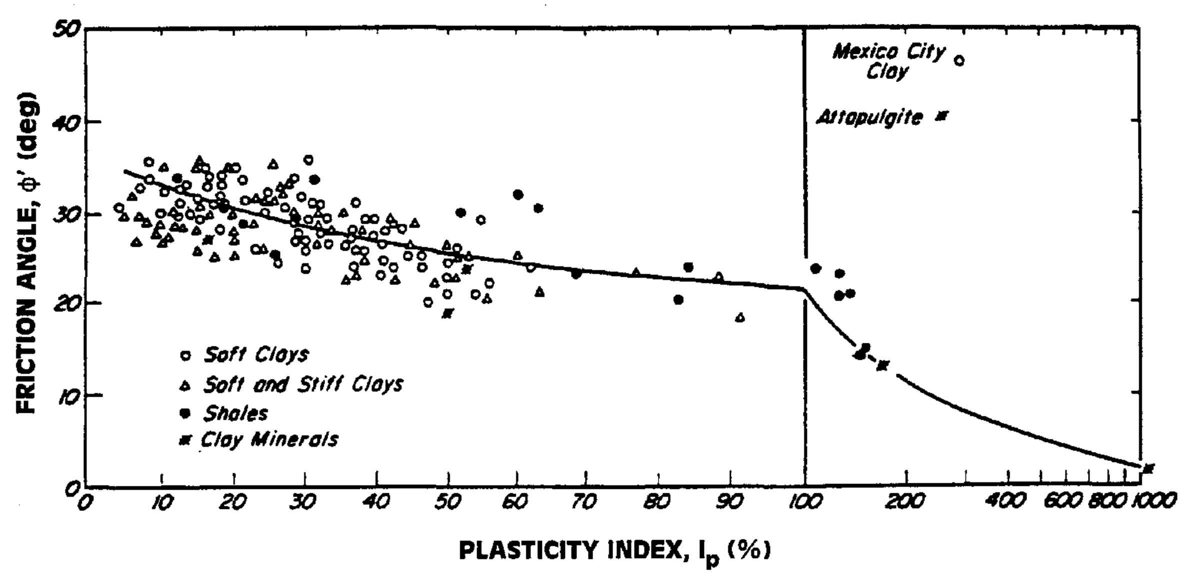


FIGURE 35
Undrained Friction Angle ϕ' for Normally Consolidated Clays in Terms of Plasticity Index (from Terzaghi, et al., 1996)

The preconsolidation pressure used to determine the OCR in Equation 4.16 is the effective normal stress where the Mohr diagram failure envelope for the overconsolidated clay joins the failure envelope for the normally consolidated clay. The exponent m for clays and shales are given in Table 10. Terzaghi, et al. (1996) defined intact soils as soils that are undisturbed and unfissured, and destructured soils as slightly fissured stiff clays and shales and soft clays sheared to a large-strain condition. Destructed soils are stronger than fully strained softened stiff clays or shales or completely remolded soft clays. Fully strained softened or remolded clays will have an m around one, and their drained shear strength will approximately equal the normally consolidated shear strength.

TABLE 10
Values of *m* in Equation 4.16

SOIL DESCRIPTION	*m*	
	Intact Soil	**Destructured Soil**
Stiff Clays and Shales	0.5 – .6	0.6 – 0.8
Soft Clays	0.6 – 0.7	0.7 – 0.9
from Terzaghi, et al. (1996)		

Drained shear strength of a heavily overconsolidated clay depends upon the condition of the clay after unloading and swelling. A badly fissured and jointed clay's drained shear strength may be reduced to its fully softened shear strength (strength in its normally consolidated state). If large displacements have occurred within a heavily overconsolidated stiff clay in the geologic past, the drained friction angle may be reduced to a residual value along planes where the displacements occurred. These planes must be continuous for a considerable distance for the shear strength to be reduced to a residual value. The residual friction angle is equal to or lower than the drained friction angle of a normally consolidated clay (fully strain softened). When the displacements occur, the clay particles are reoriented parallel to the direction of shearing. The magnitude of the friction angle reduction depends upon the clay content and the shape of the clay particles. The residual friction angle will be low for soils that have a high percentage of plate-shaped clay minerals. For an anchored wall, residual shear strength is mobilized only when displacements occur along pre-existing shear surfaces. These surfaces have to be oriented in a direction that will affect the stability of the anchored wall or the behavior of the wall will not be dependent upon the residual shear strength of the soil. Figure 36 from Patton and Henderson (1974) give drained residual friction angles for rock gouge material as a function of plasticity index. Terzaghi, et al. (1996) present the residual friction angle as a function of the friction angle of normally consolidated clays (Figure 37). Both figures show the strength reduction that can occur when a stiff, heavily overconsolidated clay is sheared, reducing the strength to a residual value.

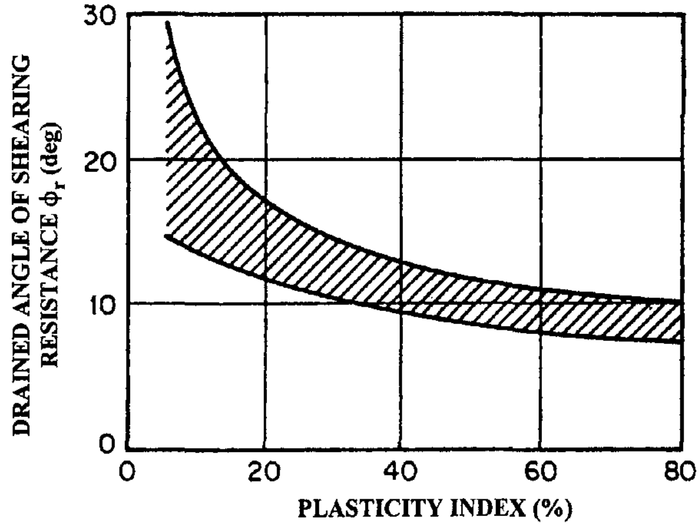


FIGURE 36
Approximate Relationship Between the Drained Residual Fraction Angle and Plasticity Index for Rock Gouge (from Patton and Henderson, 1974)

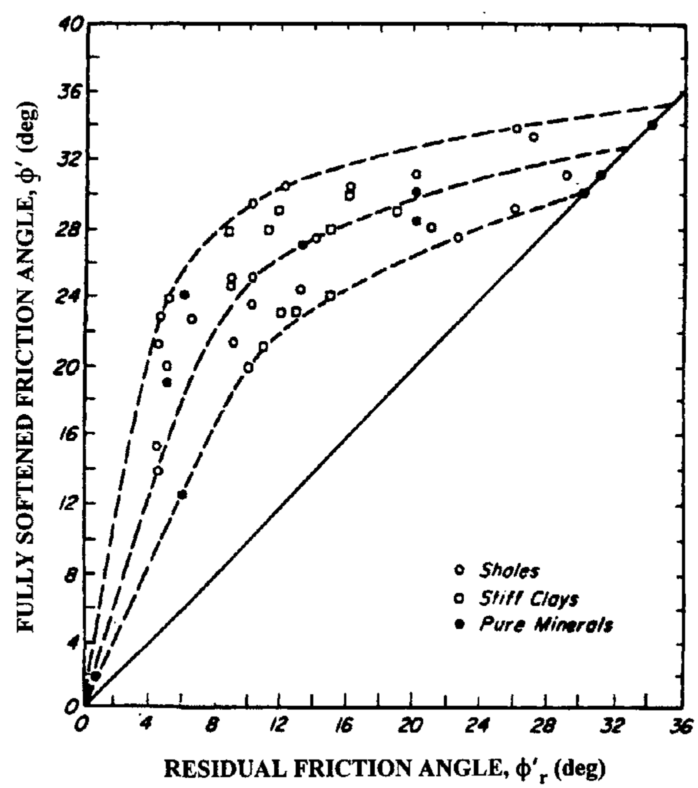


FIGURE 37
Relationship Between Fully Softened ϕ' and Residual ϕ' (from Terzaghi, et al., 1996)

Figure 38 combines the relationships from Figures 35 to 37 and Equation 4.16, and is a guide for estimating the drained friction angle for fine-grained soils in different states of stress or disturbance. The line representing the normally consolidated state is the trend line from Figure 35. Lines representing the overconsolidated soils were determined by setting Equation 4.14 equal to Equation 4.16 and solving for ϕ' in Equation 4.14. Values selected for m in Equation 4.16 are presented in Figure 38. Curves representing intact and destructured soils were drawn for clays with an OCR of two. Only curves for destructured soils were plotted for soils with OCR's greater than two. The range for the residual friction angles was developed from Figures 36 and 37.

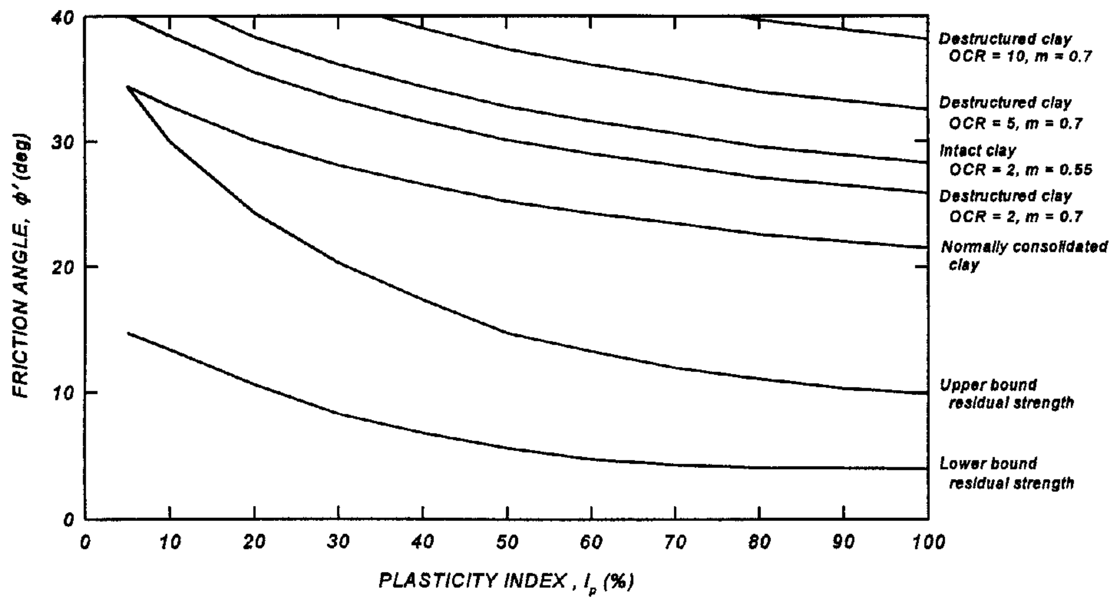


FIGURE 38
Friction Angle ϕ' for Clays in Different States as a Function of Plasticity Index

Setting the total earth load from Schnabel's $25H$ trapezoid or Terzaghi and Peck's stiff-fissured clay diagrams to $0.65K_a\gamma H^2$, the equation for the total earth pressure assuming drained conditions, enables the drained friction angle associated with each diagram to be computed. A drained friction angle of approximately 37° would give the same total load as the $25H$ trapezoid for a soil with an effective unit weight of 120 pcf. The drained friction angle associated with the Terzaghi and Peck stiff-fissured clay diagram with an intensity of pressure of $0.2\gamma H$ is approximately 38.7°. This demonstrates that short-term, apparent earth pressures can be greater than the pressures computed using drained shear strength parameters. It also shows that the short-term apparent earth pressure diagrams will give adequate lateral earth pressures for the long-term drained conditions for low plasticity clays if the groundwater level is at or near the bottom of the wall.

Atterberg limits for the clay, the OCR, the extent of fissuring, and the nature and orientation of joints or shears are needed to use Figure 38 for estimating the drained friction angle. After estimating the drained friction angle, determine the earth pressures associated with the drained condition and the porewater pressures and compare them with the earth pressures associated with the undrained shear strength. The pressures that give the greatest total load should be used for design. Often the undrained earth pressures will be larger than the drained earth pressures plus water pressure. When the wall is going to be built in a heavily overconsolidated deposit, local experience should guide in determining the degree of disturbance and the soil strength. Laboratory tests can be used to determine drained shear strength parameters, but tests done on samples recovered from the deposit may not accurately represent the strength of a fissured soil. In addition to testing, local experience, an understanding of the geologic events that have affected the soils at the site, and the relationships in Figure 38 should be considered when estimating the drained friction angle.

Stress relief in heavily overconsolidated fine-grained soils may result in a strength reduction. How this reduction affects anchored walls is not clear. Sills, et al. (1977) reported that stress relief in a 26-ft-deep excavation in London clay resulted in deep-seated movements behind ground anchors that were twice the height of the wall, but no increase in anchor load. If there is a concern that wall movements will cause stress relief in the ground, then the measured drained strength can be reduced. If stress relief occurs, the strengths will likely be greater than the normally consolidated drained shear strength (Figure 38). Drained shear strengths should not be reduced below the normally consolidated strengths unless deposit has been sheared in the geologic past and the discontinuities are oriented in a direction that affects the stability of the wall.

Poor drilling techniques using air or water to clean the drill hole may fracture the soil and reduce the soil's shear strength or pressurize the drilling fluid in open fractures. The strength reduction or the effect of pressuring the drilling fluid is not considered in design. Fracturing the ground is controlled by preventing collaring of the hole when drilling with air or water. A collar occurs when the hole becomes blocked and cuttings no longer return up the drill hole to the surface. If a collar occurs, the pressurized drilling fluid (air or water) is forced into the ground, disrupting the formation. Auger drilling methods will not disrupt the soil where collaring is likely.

4.9 SYNTHESIZED APPARENT EARTH PRESSURE DIAGRAM

Schnabel Foundation Company has successfully used the $25H$ trapezoidal apparent earth pressure diagram (Figure 19d) to design thousands of temporary walls and hundreds of permanent walls in sands, stiff clays, and mixed grounds. The total lateral earth load from this diagram is $20H^2$. Using one diagram with a constant pressure is appropriate considering how apparent earth pressures were developed and the variability of the ground. The non-symmetrical trapezoidal apparent earth pressure diagram is similar to the $25H$ trapezoid, and it can be used as a synthesized apparent earth pressure diagram for sands and stiff clays (undrained and drained conditions). This diagram also is satisfactory for mixed grounds composed of these soils. Sections 4.4, 4.5 and 4.8 show that the earth pressure factors for most ground fallsbetween 20 and 23.5, and includes a factor of safety of at least 1.3 on the shear strength.

4.10 EARTH PRESSURE FROM SOIL-STRUCTURE INTERACTION ANALYSES

Soil-structure interaction analyses for ground anchor walls relate the earth pressure at a location to a deflection at that location through earth pressure-deflection curves. Soil-structure interaction computer codes based on finite difference methods or simple finite element methods are used to analyze structural beam-columns supported by linear elastic supports and non-linear earth pressure-deflection curves. These computer codes are called beam-column programs. BMCOL76 (Matlock, et al., 1981), a finite difference program, and CBEAMC (Dawkins, 1994), a finite element program, are common beam-column computer programs. Both computer programs model the earth pressure-deflection behavior of the system identically and give the same results for similar problems.

The earth pressure deflection relationships used in these analyses are idealized elasto-plastic curves, where P is the horizontal earth pressure acting on the wall and y is the deflection of the wall. Figure 39 shows a typical earth pressure-deflection curve. The minimum and maximum pressures on the wall are assumed to be related to the active and passive pressures, respectively. The minimum and maximum pressures are reached after the wall has moved sufficiently to mobilize the active or passive states of stress. A modulus, E_s, or reference deflections, y_a and y_p, are necessary to construct the earth pressure-deflection curves.

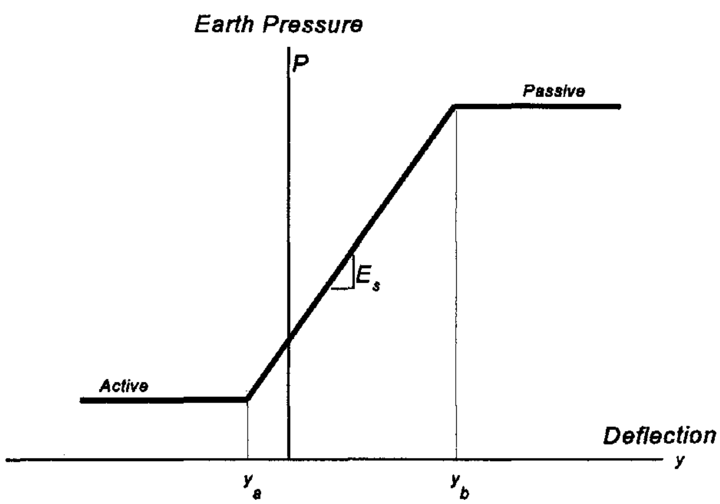


FIGURE 39
Idealized Earth Pressure-deflection Curve

The actual earth pressure-deflection relationship at a given location along an anchored soldier beam and sheet pile wall is affected by:

- Construction sequence.
- Flexibility of the wall.
- Discontinuous nature of the wall.
- Simplifications in modeling three-dimensional behavior using a two-dimensional model.
- Arching within the soil.

Earth pressure-deflection curves can account for some of the above behaviors, but they cannot account for arching within the soil. Back-calculated earth pressure-deflection curves from a test wall at Texas A&M University had an active earth pressure coefficient of 0.15 (Weatherby, et al., 1998) as compared with a Rankine pressure coefficient of 0.307. A relationship to account for the redistribution of earth pressures resulting from arching was not identified during the research at Texas A&M.

Apparent earth pressure envelopes include the effects of arching and the other aspects of flexible soldier beam and sheet pile wall behavior. They are better able to describe the behavior of the wall above the bottom of the wall than complicated soil-structure interaction models. Below the bottom of the excavation, earth pressure-deflection curves describe the behavior of the soldier beam toe. The behavior of the toe is discussed in Chapters 6 and 7 of this report.

4.11 RECOMMENDATIONS

Apparent earth pressure diagrams are recommended for the design of ground anchor walls. Experience has shown that the exact shape of the apparent earth pressure diagram is not critical if the earth pressure result is near the mid-height of the wall and the total lateral earth load is appropriate. When using apparent earth pressure diagrams the following guidelines should be considered:

- The same apparent earth pressure diagram can be used for walls supported by one or many rows of ground anchors.
- Ground anchor loads and the maximum bending moments can be computed using tributary areas and moment equations for a continuous beam, or by dividing the beam into a series of simple beams.
- Apparent earth pressure diagrams predict the magnitude of the maximum bending moments, but they do not necessarily predict their location.
- Assuming a hinge at subgrade gives a conservative prediction of the bending moments in sands and stiff clays.

- Limiting equilibrium methods can be used to determine the total lateral earth load for an anchored wall. Use a factor of safety of 1.3 on the shear strength of the soil. Distribute the load to the wall using apparent earth pressure diagrams.
- The total lateral earth load can be expressed as an earth pressure factor times the H^2. Earth pressure factors for granular soils and stiff clays generally range between 20 and 24.
- Use a non-symmetrical trapezoid apparent earth pressure diagram in sands and stiff clays.
- In soft to medium clay, use an apparent earth pressure diagram with the shape of Terzaghi and Peck's soft clay diagram. The total lateral earth load can be determined using earth pressure factors or limiting equilibrium analyses.
- Apparent earth pressure diagrams have a factor of safety on the shear strength of approximately 1.3.
- Apparent earth pressure diagrams can be effectively used in mixed ground. The total lateral earth load can be estimated using an earth pressure factor or computed using limiting equilibrium analyses .
- Compute earth pressures using drained and undrained shear strengths and use the largest pressure to design the wall.
- Good estimates of the porewater pressures are necessary to do a drained analysis.

CHAPTER 5: CONSTRUCTIBLE DESIGNS

Design calculations for a permanent ground anchor wall are straightforward. The difficult part of a permanent ground anchor wall design is developing a design that is flexible and constructible.

Permanent ground anchor walls are built by combining the following components:

- Soldier beams.
- Ground anchors.
- Ground anchor to soldier beam connections.
- Concrete wall facing.
- Facing to soldier beam connections.

How these components are combined affects construction, appearance, cost, and quality of a finished permanent ground anchor wall.

5.1 SOLDIER BEAMS

Soldier beams can be installed by driving, drilling, or in hand-dug pits. The selection of the installation methods depends upon the nature of the ground, access for the equipment, and costs. Sometimes soldier beam installation is expensive and unnecessary. Anchored horizontal beams or anchored elements can be used to replace soldier beams when excavations for the beams or elements can be made with almost no support.

5.1.1 Driven Soldier Beam Considerations

HP shapes or steel sheet piles are used for driven soldier beams. Driven soldier beams must penetrate to the desired depth without significant damage. Drive shoes or "points" may be used to improve the ability of the soldier beams to penetrate a hard stratum. High-strength steels also improve the ability of the soldier beams to withstand hard driving. If the soldier beams cannot penetrate to the desired depth, then the beams should be drilled-in. The design must allow for some misalignment if the soldier beams are driven. Thru-beam connections or horizontal wales are used to connect ground anchors to driven soldier beams.

A thru-beam connection is a connection cut in the beam for a small-diameter ground anchor. Thru-beam connections are usually fabricated before the beam is driven. This type of connection is designed so the ground anchor load is applied at the center of the soldier beam in line with the web. Large-diameter ground anchors, greater than 6 or 7 in, cannot be used with thru-beam connections. Thru-beam connections are used, when few ground anchor failures are anticipated. When a ground anchor fails, the failed anchor has to be removed from the connection or a new connection has to be fabricated. Both options can be expensive.

Today, "sidewinder connections" are not used for permanent ground anchor walls. A sidewinder connection is offset from the center of the soldier beam, and the ground anchor load is applied to the flange some distance from the web. Sidewinder connections subject the soldier beams to bending and torsion in maximum moment regions (Weatherby, et al., 1998). When soldier beam stresses resulting from this type of connection can be reliably calculated, sidewinder connections may be developed for permanent ground anchor walls.

Horizontal wales can be installed on the face of the soldier beams, or they can be recessed behind the front flange. When the wales are placed on the front flange, they can be exposed or embedded in the concrete facing. If the wales remain exposed, then the ground anchor tendon corrosion protection may be exposed to the atmosphere and ultraviolet radiation. Some plastic components of the anchor corrosion protection are not designed for prolonged exposure to UV rays and direct sunlight. Corrosion protection for the tendon anchorage must be well designed and constructed if the anchorage remains exposed. Exposed wales must be protected from corrosion and they are unattractive. Therefore, they are not recommended for permanent ground anchor walls. If the wales are placed on the front face of the soldier beams, encasing them in the concrete facing is possible. To encase the wales requires a thick cast-in-place concrete facing. If a wale is added during construction, the horizontal clear distance to the travel lanes should be checked before approval of the change. Wales can be recessed to allow a normal thickness concrete facing to be poured. Recessed wales must be individually fabricated and the welding required to install them is difficult and expensive.

U.S. Patent No. 4,561,804 describes a patented ground anchor wall that allows large-diameter ground anchors to be used with pairs of driven steel sheet piles. This wall has been successfully used by Schnabel Foundation Company on highway projects. It allows the ground anchor connection to be recessed and a normal thickness cast-in-place concrete facing.

When cast-in-place concrete facings are used with driven soldier beams, headed studs are used to attach the concrete to the soldier beams. The poured facing must allow for soldier beam misalignment. Experienced wall contractors know what tolerances can be achieved in different types of ground. Precast panels are not frequently used with driven soldier beams because the connection must allow for soldier beam misalignment. U.S. Patent No. 4,913,594 describes a system to connect segmental precast panels to driven soldier beams.

5.1.2 Drilled-in Soldier Beams

Drilled-in soldier beams normally use pairs of channels or wide-flange sections. Drill holes can be backfilled with lean mix (Weatherby et al., 1998). Individual shapes can be used, but the connection costs may be more expensive when a single section is used. Efficient structural shapes can be selected since drilled-in soldier beams are not required to withstand driving stresses. Ground anchors are installed between the structural sections and the distance between the sections depends upon the type of ground anchor used. Small- or large-diameter anchors can be installed. Drill hole diameters for the soldier beams depend upon the structural shape and the diameter of the anchor. Replacement anchors are easily installed between the two sections at any location along the soldier beam. Drilling is economical when the drill hole does not require casing. Reinforced concrete drilled shafts are occasionally used for soldier beams. When a reinforced concrete drilled shaft is used, blockouts have to be installed in the rebar gauge to permit anchor installation and stressing, and care has to be taken to position the cage so the blockouts are oriented in the right direction. Ground anchor and wall facing connections for reinforced concrete soldier beams are different from drilled-in soldier beams with structural shapes. Reinforced concrete soldier beams are expensive.

The ground anchor to soldier beam connection for drilled-in soldier beams can be installed on the front face of the structural sections or between the sections. For small-diameter ground anchors, the connection maybe prefabricated before the soldier beams are installed. The connections for large-diameter anchors are made after the anchors have been installed. Connections are designed so a 12-in-thick cast-in-place concrete facing can be attached to the soldier beams.

Precast concrete panels or cast-in-place concrete facings can be used with drilled-in soldier beams. Cast-in-place walls are attached to the soldier beam using headed studs. Precast panels are attached to the soldier beams using a cast-in-place closure pour.

5.2 GROUND ANCHORS

Ground anchors for permanent walls can be anchored in rock, clays, or sands. A variety of installation equipment and methods are used to make the anchors. Anchor diameter, load-carrying capacity, and cost depends upon equipment access, installation method, contractor's experience, soil type, and soil strength.

Specialty geotechnical contractors use empirical relationships and experience to estimate load-carrying capacity. Prescriptive values for load-carrying capacity, such as AASHTO's *The Standard Specification for Highway Bridges* (1996), are conservative and may result in expensive designs. Additional information regarding estimating anchor capacities is presented by Weatherby (1982) and the Post-Tensioning Institute (1996).

Allowable ground anchor loads are commonly estimated by dividing the ultimate ground anchor load-carrying capacity by a safety factor of two.

5.2.1 Rock Anchors

Rock anchors are drilled using rotary or percussion techniques. They are normally less than 8 in in diameter. High load-carrying capacities can be developed by small-diameter rock anchors. In clay shales and weathered rocks, load-carrying capacity is often affected by hole cleaning and grouting techniques. Casing may be required to maintain an open drill hole when caving overburden is encountered. Installing the ground anchors at 45° may be the most efficient way to reach a deep rock stratum, but such steep angles result in large vertical loads applied to the wall. The soldier beam must support this load. Developing the axial capacity to support the vertical component of the ground anchors can be expensive. Installing the ground anchors at flatter angles is often a better solution.

5.2.2 Anchors in Cohesive Soils

Contractors have developed a wide range of installation methods for anchors in cohesive soils. Their load-carrying capacity depends upon soil strength, anchor diameter, anchor bond length, and grouting method. Anchor diameters range from 4 to 16 in. Grouting methods include: tremie, low pressure, post-grouting, and post-grouting with a packer. A variety of anchors can develop adequate load-carrying capacities for a wall. Since the anchor type may affect the soldier beam and the connection, the specifications should require the contractor to select the best combination when preparing detailed design drawings.

5.2.3 Anchors in Granular Soils

Anchors in granular soils develop high load-carrying capacities in 3 to 6 in in diameter drill holes. They are installed using drilling methods that support the ground during installation. Casing or drilling fluids are common methods. In sand, anchor capacity is dependent upon the strength of the ground, anchor bond length, and grouting method. Grouting methods are classified as low pressure, less than 150 psi, and high pressure. High-pressure grouted anchors in dense sand develop very high load-carrying capacities. Pressure-grouted anchors in sand should have at least 12 ft of overburden over the anchor bond length. Contractors have developed a variety of installation methods for anchors in granular soils. For the same project, one contractor might use a few high-capacity ground anchors and drilled-in soldier beams, and a second contractor might select many lower capacity anchors and driven soldier beams with thru-beam connections. Because of the variety of options, the specifications should require the contractor to select the best combination and prepare the detailed design drawings for the owner's review.

5.3 FACINGS

Cast-in-place concrete or precast concrete facings should be used for permanent ground anchor walls. Walls using treated timber or precast concrete lagging are not recommended since they will require considerable maintenance during their life and they are unattractive. Cast-in-place facings are compatible with driven or drilled-in soldier beams. Precast facings are used with drilled-in soldier beams. The connection between the soldier beams and the facing must be flexible enough to allow for installation tolerances. Sharply curved walls or walls with varying heights generally have a cast-in-place concrete face. High walls next to roadways are less threatening to motorists if they are battered. Battered walls are easier to build using a cast-in-place concrete face. Precast concrete panels allow a variety of architectural treatments to be applied to the face of the wall. Full height, precast concrete panels are economical for long walls. Precast panels are made using reinforced concrete or prestressed concrete. Handling and lifting stresses are considered when designing tall panels. Precast panels can be cast on site or in a plant. If transportation costs are significant, site fabrication is cost-effective. Many highway departments require that the panels be fabricated by an approved precaster. Requiring an approved precaster may affect the selection of the type of facing on some projects.

A wide range of form liners can be used with cast-in-place concrete and precast concrete panels. Owners should specify the type of finish that they require since finish costs can vary by $2 to $4 per square foot.

5.4 RECOMMENDATIONS

A constructible design requires an understanding of the interrelationship between the different components of a ground anchor wall. The design must be flexible and allow for replacement anchors if failures are expected. On most projects, ground anchor failures rates are less than 1 to 2 percent. Most anchor failures occur at the beginning of a project, or in low-strength ground, or when ground conditions change. Specialty contractors, who design and build anchored walls, understand how the different components of an anchored wall fit together. To obtain a constructible wall at the best price, the contract documents should clearly establish the design requirements and require the contractor to prepare detailed design drawings. The contractor should select the soldier beam type and installation method, ground anchor type and load-carrying capacity, connection details, and facing type.

CHAPTER 6: DESIGN OF THE WALL TOE

The embedded portion of a ground anchor wall, the toe, must resist vertical and lateral loads. Vertical loads are caused by the ground anchors and other applied loads, and lateral load results from the earth pressures. Figure 40 illustrates skin friction and end bearing mobilized to resist the axial loads in the wall, and lateral resistance mobilized to resist the toe reaction from the apparent earth pressure diagram.

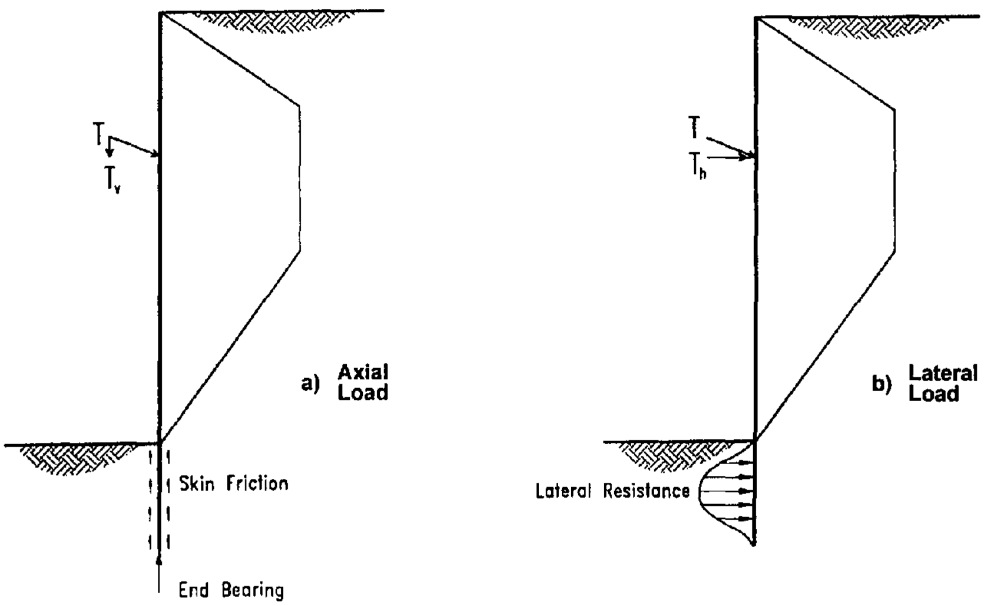


FIGURE 40
Loads on Anchored Wall Toes

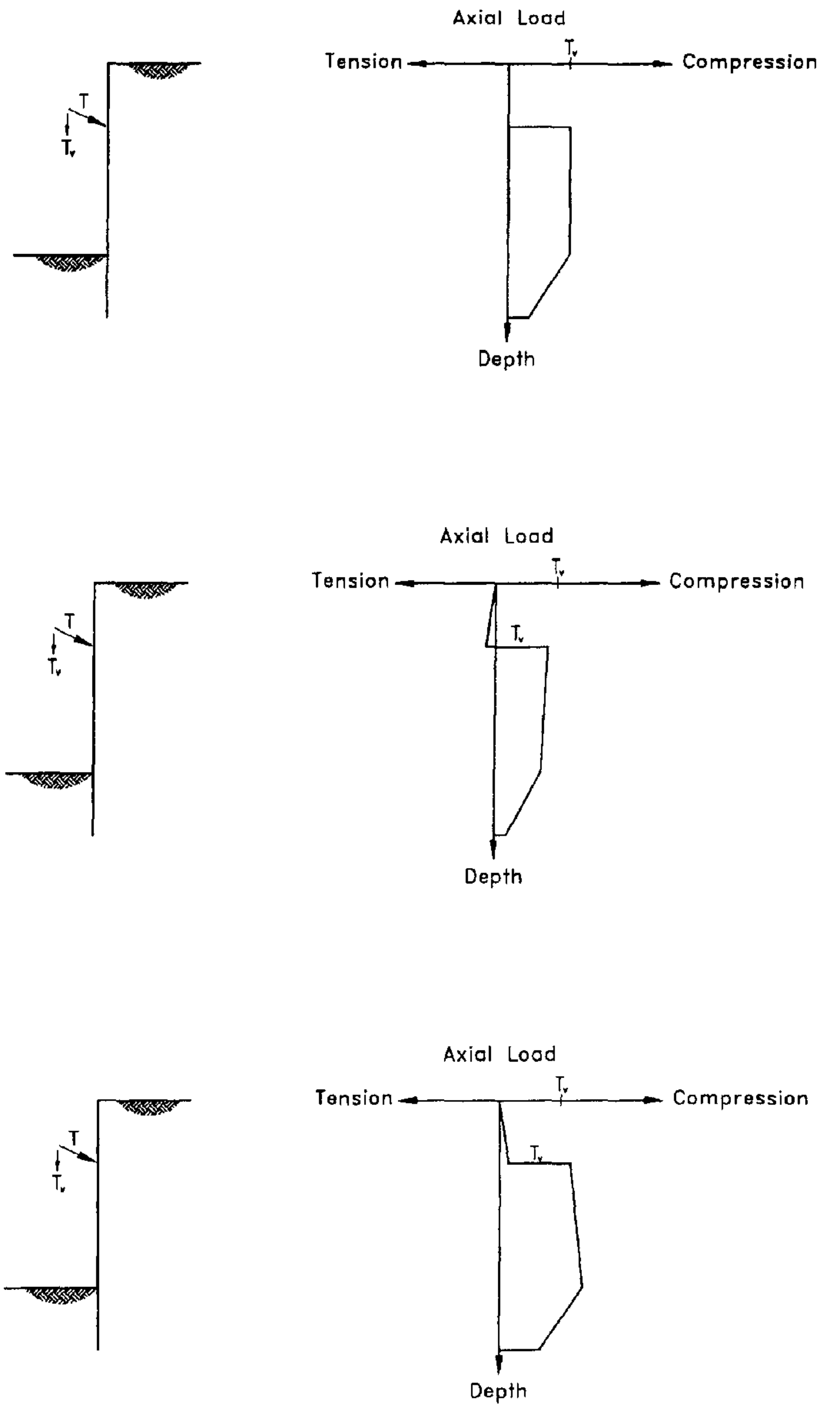
6.1 AXIAL LOAD DESIGN

Before presenting the design recommendations for axial load, observations from four projects with instrumented soldier beams will be reviewed. The projects in granular ground were monitored for less than 100 days after construction was complete. Short-term and long-term conditions were considered similar in the granular soils. In cohesive soils, monitoring continued for 168 days on one project and for 731 days on another project. Complete information concerning the axial load behavior on the projects is contained in *Summary Report of Research on Permanent Ground Anchor Walls*, "Volume II: Full-scale Wall Tests and a Soil-Structure Interaction Model," (Weatherby, et al., 1998). Axial load transferred to a soldier beam toe can be less than or more than the vertical components of the anchor loads (Figure 41). Axial loads in the wall are greater than the vertical components of the ground anchor loads if the ground behind the wall settles relative to the wall. When the wall settles relative to the ground, the axial load in the wall is less than the vertical component of the ground anchor. Axial load and ground movements are interrelated. The magnitude of the axial load depends upon: the vertical components of the ground anchor loads, the strength of the supported ground, vertical and lateral movements of the wall, the relative movements of the ground with respect to the wall, and the axial load-carrying capacity of the toe.

In dense sands or stiff to hard clays, the axial load measured in the instrumented soldier beam toes was less than the vertical components of the ground anchor loads. In dense sands, the load transferred to the ground above the bottom of the excavations was equal to the horizontal components of the ground anchor loads times the tangent of a wall friction angle. Back-calculated wall friction angles ranged between one-quarter and one-half the soil friction angle. This observation leads to the conclusion that the axial load transferred to the toe will be zero or very small if the ground anchors are installed at an angle equal to half the soil friction angle. Axial load transferred to the ground above the bottom of the excavations in stiff to hard clays was equal to $A_s(0.25s_u)$. A_s was the surface area of the soldier beam in contact with the ground above the bottom of the excavation, and $0.25s_u$ was the back-calculated adhesion. At the cohesive soil sites, the load transferred from the soldier beam to the ground above the bottom of the excavation appears to be valid for the long-term condition. Using an adhesion equal to 25 percent of the undrained shear strength gives a lower load transfer rate than a rate based on drained shear strengths.

In medium dense sands, the axial load measured in the toes of the instrumented soldier beams was equal to the vertical components of the ground anchor loads when wall movements were typical of other soldier beam walls (maximum vertical movement = $0.0017H$, and maximum lateral movement = $0.002H$). When soldier beam settlements were small, between 0.0009 and $0.0017H$, the measured axial load in the toes was greater than the vertical components of the ground anchor loads. The additional axial load is believed to be the result of downdrag as the ground settled with respect to the wall. In sands, the downdrag load can be expressed as the Rankine active earth pressure times the tangent of a wall friction angle. When downdrag was observed in the medium dense sand, the maximum angle of wall friction was equal to half the soil friction angle.

The case histories studied did not include a wall built in a soft to medium clay. In soft to medium clay, downdrag loads may develop if the toe is taken to a stratum with suitable bearing. In this report, soft to medium clays have an undrained shear strength less than $\gamma H/4 - 5.71H$. Downdrag loads in soft to medium clays are assumed to be $0.05\gamma H^2$. This value seems reasonable when compared with the downdrag loads in sandy ground.



a) No relative movement between wall and ground

b) Wall settles relative to the ground

c) Ground settles relative to the wall

FIGURE 41
Idealized Axial Load Distributions for Soldier Beam Walls

6.1.1 Applied Axial Loads

Determine the axial load to be resisted by the toe of an anchored wall using the guidelines in Table 11. Axial load transferred to the toe includes the vertical components of the ground anchor loads plus applied loads minus the load transferred to the ground above the bottom of the excavation. Load is transferred to the ground above the bottom of the excavation when the shear strength of the ground is high. The recommendations in Table 11 do not include downdrag loads. If downdrag loads develop, they will be transferred to the ground after small wall settlements. Limited case history data were used to develop the recommendations in Table 11. The recommendations are believed to be conservative.

TABLE 11
Guidelines for Estimating the Axial Load Applied to the Toe

SANDS		CLAYS	
Medium Dense	**Dense to Very Dense**	**Soft to Medium**	**Stiff**
$10 \leq SPT \leq 30$	$SPT > 30$	$s_u \leq \gamma H/4 - 5.714H$	$s_u > \gamma H/4 - 5.714H$
I	II	I	III

I	Design toe to resist vertical components of the ground anchor loads plus applied axial loads
II	Design toe to resist vertical components of the ground anchor loads plus applied axial loads minus the horizontal components of the ground anchor loads times $\tan\delta$ (δ between $\phi/4$ and $\phi/2$)
III	Design toe to resist vertical components of the ground anchor loads plus applied axial loads minus $A_s(0.25s_u)$ (A_s = surface area of steel in contact with the ground and s_u = undrained shear strength)

6.1.2 Axial Load-carrying Capacity

Axial load-carrying capacity of a soldier beam toe does not increase dramatically with depth. Economical designs require good estimates of the ultimate load-carrying capacity of the toe and a reasonable factor of safety. Soldier beams are installed by driving or they are drilled-in. Both methods are used in sands and clays. When the soldier beams are drilled-in, the toe can be backfilled with either structural concrete or lean-mix backfill. The axial load-carrying capacity of the soldier beam toe will depend upon the type of beam selected, the strength of the ground, whether the beams were drilled-in or driven, and the type of backfill if the beams were drilled-in. If lean-mix backfill is used to backfill the toe of a drilled-in soldier beam, determine the axial capacity for a beam punching through the lean mix and the capacity for a drilled shaft, and use the smallest capacity in the design. Use the block perimeter area of the soldier beam and an average friction angle of 0.83ϕ when computing the punching capacity of the soldier beam.

6.1.2.1 Driven Soldier Beams in Sand

Relationships for estimating the axial capacity of driven soldier beams in sand were developed from procedures in *Design and Construction of Driven Pile Foundations* (Hannigan, et al., 1996). Verification of these relationships for soldier beams is presented by Weatherby, et al., 1998. The ultimate axial load-carrying capacity of driven H-beams in sands is given by:

$$\begin{aligned} Q_{ult} &= Q_s + Q_t \\ &= f_s A_s + q A_t \end{aligned} \qquad \text{. . . [6.1]}$$

where:

Q_{ult} = ultimate pile capacity
Q_s = resistance due to skin friction
Q_t = tip resistance due to end bearing
f_s = average unit skin friction resistance
A_s = block perimeter surface area of the soldier beam toe
q = unit end bearing resistance
A_t = block area of the soldier beam tip

The average unit skin friction resistance, f_s, is determined using Equation 6.2:

$$f_s = K\sigma'_{vave}\tan(\delta) \qquad \text{. . . [6.2]}$$

where:

K = lateral earth pressure coefficient (recommended range 1 to 2)
σ'_{vave} = average effective vertical stress along the toe of the soldier beam
δ = angle of friction between soil and beam (recommended range 0.67ϕ to 0.83ϕ)

If the groundwater is below the bottom of the beam, then

$$\sigma'_{vave} = \gamma(h+d)/2$$

where:

γ = total unit weight
h = height of wall
d = depth of toe emdedment

The effective overburden pressure in Equation 6.2 is determined using the average of the wall height plus the toe penetration. Figure 42a illustrates how the effective overburden pressure is determined. The effective overburden pressure on one side of the soldier beam toe depends upon a depth of embedment from the ground surface to the midpoint of the toe. On the other side of the soldier beam the effective overburden pressure depends upon a depth of embedment from the bottom of the excavation to the midpoint of the toe. Lateral loads on a soldier beam toe are great-er than those computed using this procedure since passive pressures develop on the excavation side of the soldier beam.

Embedment depth, d, is used to compute the point bearing resistance, since bearing capacity is controlled by the shallow failure surface that would develop in front of the wall (Figure 42b).

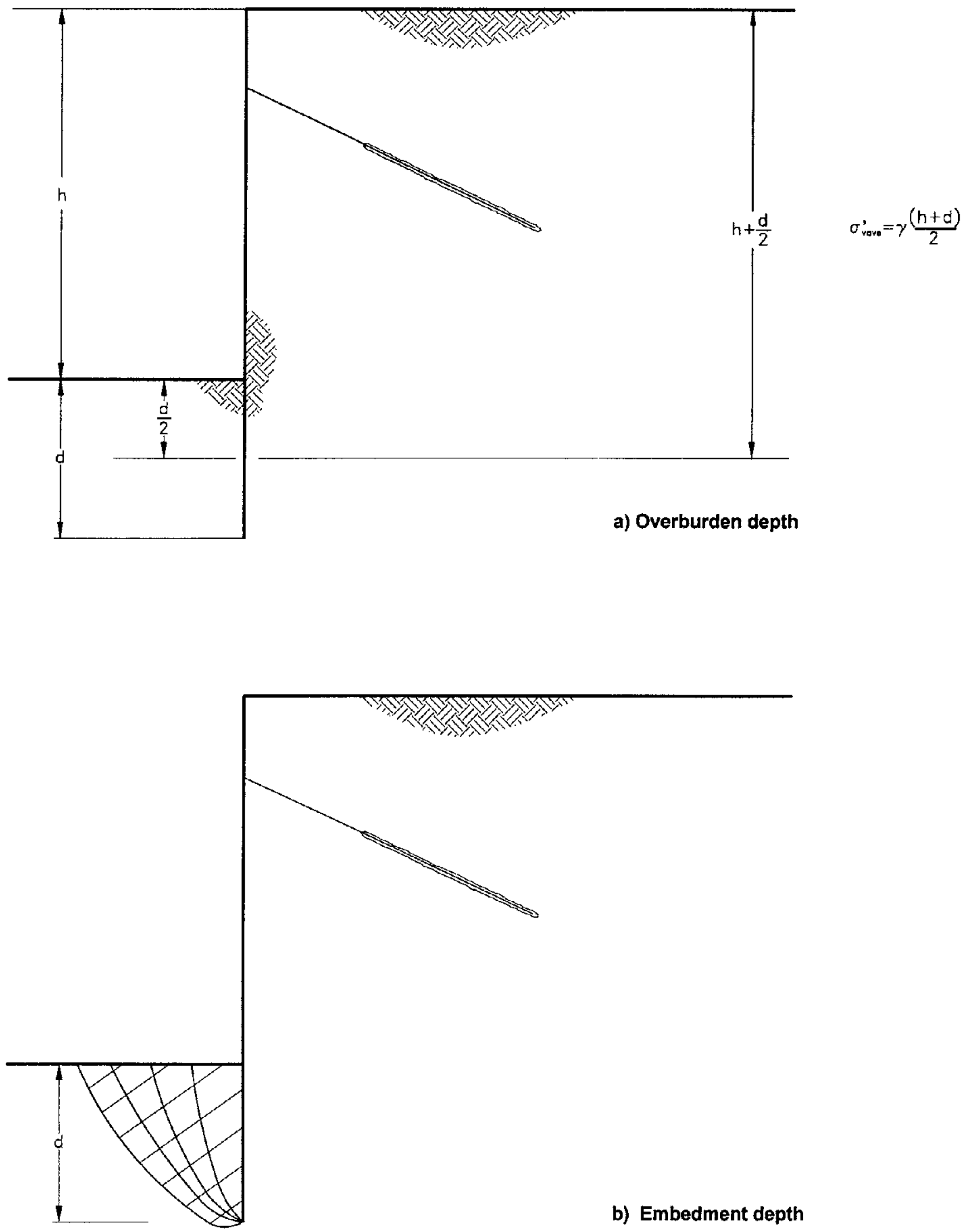


FIGURE 42
Overburden and Embedment for Skin Friction and End-bearing Calculations

Unit end bearing resistance q, is given by the Equation 6.3:

$$q = \sigma_v' N_q \qquad \text{. . . [6.3]}$$

where:

σ_v' = effective overburden stress at depth, d

N_q = bearing capacity factor from Figure 43

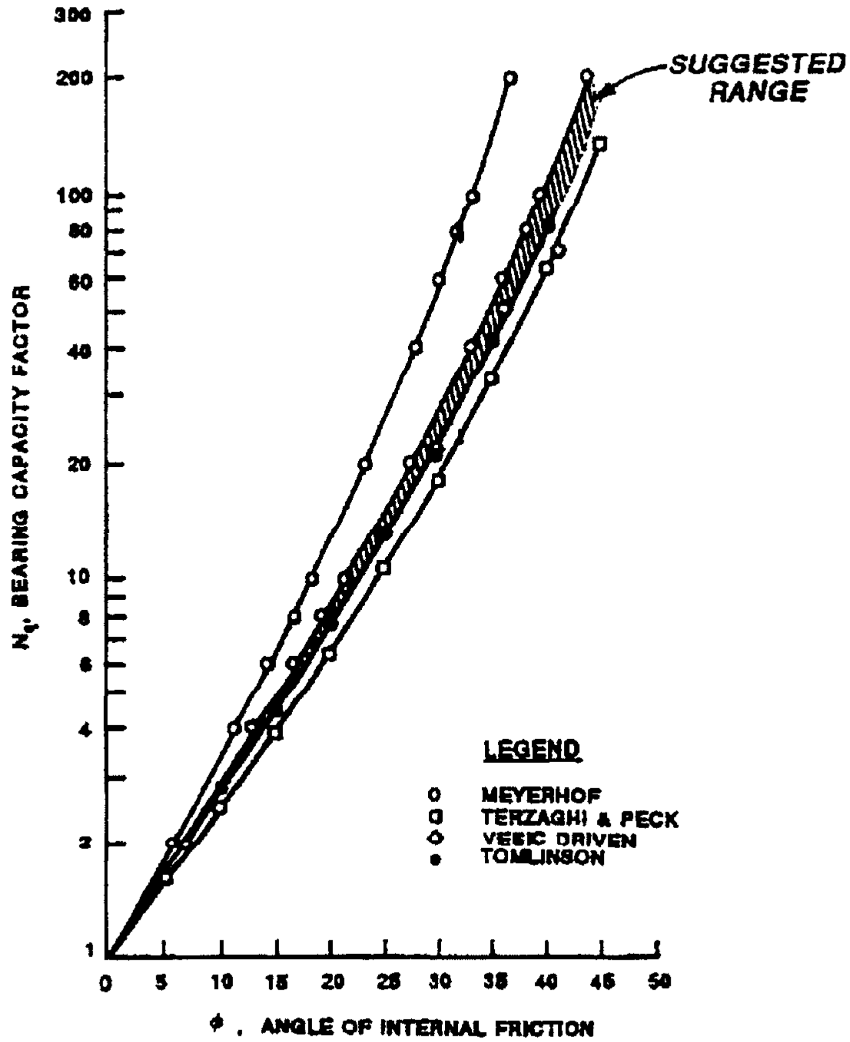


FIGURE 43
Bearing Capacity Factor, N_q (ASCE, 1993)

To match the predictions with the capacities measured at the Texas A&M University test wall, a lateral earth pressure coefficient, $K = 2$, was used. The design procedure recommends using an average of the friction angle, δ, for steel against sand and sand against sand. The best prediction of skin frictions occurred using a value of δ equal to 0.83ϕ. A value of N_q in the middle of the range recommended by Meyerhoff gave the best estimate of the end bearing capacities.

The axial load-carrying capacity of a drilled-in soldier beam backfilled with lean mix is estimated using Equations 6.1 to 6.3. These equations assume that the soldier beam will punch through the backfill rather than transfer the load through the backfill to the ground. When estimating the capacity, use $K = 2$ and $\delta = 35°$ in Equation 6.2. If the drilled shaft bears on clay, the unit end bearing resistance will be $9s_u$ (Equation 6.6) instead of the value determined by Equation 6.3. Compare the ultimate axial capacity with the capacity determined from either Section 6.1.2.3 or 6.1.2.4. Determine the toe depth using the lowest capacity.

6.1.2.2 Driven Soldier Beams in Clay

Axial capacity of driven soldier beams in clay is also based on the procedures in *Design and Construction of Driven Pile Foundations* (Hannigan, et al., 1996). The ultimate axial load-carrying capacity of a driven H-beam soldier beam in clay is given by Equation 6.4:

$$\begin{aligned} Q_{ult} &= Q_s + Q_t \\ &= f_s A_s + q A_t \end{aligned} \qquad \ldots [6.4]$$

where:

Q_{ult} = ultimate axial capacity
Q_s = resistance due to skin friction
Q_t = tip resistance due to end bearing
f_s = average unit skin friction resistance
A_s = block perimeter surface area of the soldier beam toe
q = unit endbearing resistance
A_t = block area of the soldier beam tip

The average unit skin resistance, f_s, in Equation 6.4 is:

$$f_s = c_a = \alpha s_u \qquad \ldots [6.5]$$

where:

c_a = adhesion between the clay and the soldier beam
α = adhesion factor
s_u = undrained shear strength of the clay

Values of α are given in Figure 44.

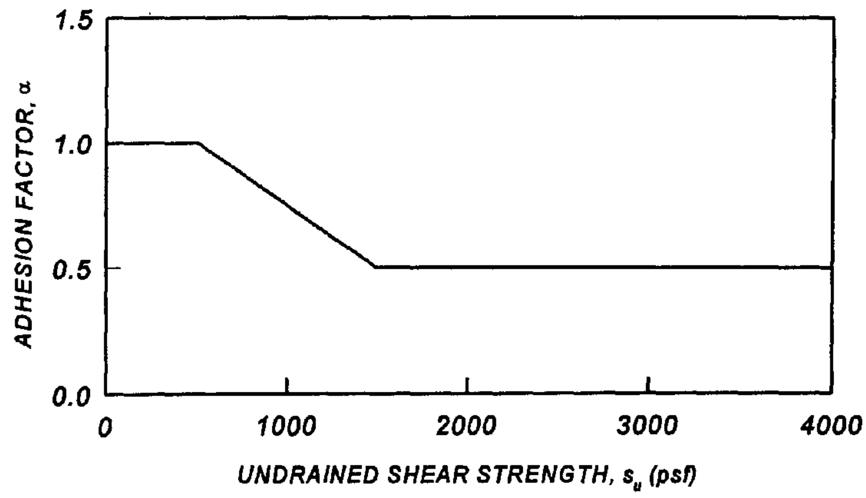


FIGURE 44
Adhesion Factor Versus Undrained Shear Strength

Unit tip bearing capacity is given by Equation 6.6:

$$q = 9s_u \qquad \text{. . . [6.6]}$$

Tip capacity is seldom relied upon for driven foundation piles in cohesive soils. To fully mobilize tip capacity in a clay requires a movement of 10 percent of the beam depth. Tip capacity is considered when computing the ultimate axial load-carrying capacity of a soldier beam under certain conditions. When normal soldier beam settlement ($0.0015H$) can be tolerated and at least 50 percent of the axial load-carrying capacity results from skin friction, then tip capacity can be included in the ultimate axial capacity of the soldier beam. If soldier beam settlements must be kept to a minimum, then the ultimate axial capacity of driven soldier beams in clay should be computed using only skin friction.

6.1.2.3 Drilled-in Soldier Beams in Sand

Relationships for estimating the axial load-carrying capacity for drilled-in soldier beams in sands are based on equations developed by Reese and O'Neill (1988). The ultimate axial load-carrying capacity of the drilled shaft is given by Equation 6.7:

$$Q_{ult} = Q_s + Q_t = f_s A_s + q_b A_t \qquad \ldots [6.7]$$

where:

Q_{ult} = ultimate axial capacity
Q_s = resistance due to skin friction
Q_t = tip resistance due to end bearing
f_s = average unit skin friction
A_s = surface area of the drilled shaft
q_b = unit end bearing resistance
A_t = cross sectional area of the drilled shaft

The average unit skin friction, f_s, for a drilled shaft is:

$$f_s = \beta \sigma'_{vave} \leq 4.0\ ksf \qquad \ldots [6.8]$$

where:

$\beta = 1.5 - 0.135((h+d)/2)^{0.5}$, $1.2 \geq \beta \geq 0.25$
σ'_{vave} = average effective vertical stress along the toe of the soldier beam
γ' = effective unit weight of soil
h = height of wall
d = depth of toe embedment

Section 6.1.2.1 discusses the computation of the average effective vertical stress. Reese and O'Neill (1988) stated that β in Equation 6.8 is independent of soil strength because drilling disturbance reduces the friction angle to a common value regardless of initial soil strength.

The unit tip bearing capacity, q_b, is given in Table 12.

TABLE 12
Recommended Values of Unit Tip Bearing Capacity for Drilled Shafts in Sands

RANGE OF UNCORRECTED STANDARD PENETRATION RESISTANCES (SPT) (blows/ft)	VALUES OF q_b (ksf)
0 to 75	1.2 (SPT)
Above 75	90

6.1.2.4 Drilled-in Soldier Beams in Clay

The axial load-carrying capacity for a drilled-in soldier beam in clay is estimated using equations developed by Reese and O'Neill (1988). The ultimate axial load-carrying capacity of the drilled shaft is given by Equation 6.9:

$$Q_{ult} = Q_s + Q_t = f_s A_s + q_b A_t \qquad \text{. . . [6.9]}$$

where:

Q_{ult} = ultimate axial capacity
Q_s = resistance due to skin friction
Q_t = tip resistance due to end bearing
f_s = average unit skin friction
A_s = surface area of the drilled shaft
q_b = unit end bearing resistance
A_t = cross-sectional area of the drilled shaft

The average unit skin resistance, f_s, in Equation 6.9 is:

$$f_s = \alpha s_u \leq 5.5\,ksf \qquad \text{. . . [6.10]}$$

where:

α = 0.55
s_u = undrained shear strength of the soil

The unit tip bearing capacity, q_b for clays is given by Equation 6.11:

$$q_b = N_c s_u \leq 80\,ksf \qquad \text{. . . [6.11]}$$

where:

N_c = 6.0 [1 + 0.2 (d/b)] $\leq$ 9
s_u = undrained shear strength at the tip of the drilled shaft
d = depth of the toe embedment
b = diameter of the drilled shaft

6.1.3 Factor of Safety

The factor of safety against axial load failure is Q_{ult}/Q, where Q_{ult} is the capacity computed using the equations in Section 6.1.2, and Q is the applied load computed using the recommendations in Section 6.1.1. Use a factor of safety of 2.0 for the axial design of the toe of permanent walls. This factor of safety is smaller than those recommended for foundation piles and drilled shafts in AASHTO's *Standard Specification for Highway Bridges* (1996). A smaller factor of safety is recommended since the wall will remain serviceable if the applied axial load exceeds the axial capacity of the toe. If the toe of an anchored wall is overloaded, the wall will settle slightly and transfer load to the ground until equilibrium is reached.

6.2 LATERAL LOAD DESIGN

Anchored wall toes must carry the lateral loads resulting from the earth pressures with an adequate factor of safety. Apparent earth pressure calculation methods in Chapter 4 determined a concentrated lateral load, called the toe reaction, at a hinge at the bottom corner of the excavation. The methods in Chapter 4 conservatively estimate the bending moments and the toe reaction for walls when the ground offers sufficient passive resistance to support the toe reaction. Passive resistance mobilized in front of the toe must be adequate to resist the toe reaction with a factor of safety of 1.5. Permanent ground anchor walls should not be constructed in ground that does not have adequate lateral support for the toe of the wall. If a temporary excavation support system is used in ground with insufficient strength to resist the subgrade reaction, then the wall must be designed to cantilever around the lowest support.

AASHTO's *The Standard Specifications for Highway Bridges* (1996) recommends computing the lateral resistance of soldier beams using laterally loaded pile design relationships developed by Broms (1965). In granular soils, Broms computes the lateral resistance at a given depth to be equal to three times the beam width times the Rankine passive earth pressure at that depth. In cohesive soils the lateral resistance was equal to the soldier beam width times the soil's undrained shear strength times nine. Lateral resistance to a depth equal to 1.5 times the beam width was assumed to be zero.

McClelland (1998) reported that the Texas Department of Transportation (DOT) currently designs cantilevered drilled shaft walls using soil-response curves (P-y curves) developed by Wang and Reese (1986). Similar relationships have been widely used for laterally loaded piles, and they are recommended for computing the ultimate lateral resistance of a soldier beam toe. They also will be used to develop non-linear soil springs for soil-structure interaction analyses (Chapter 7). Weatherby, et al. (1998) reported that soldier beam bending moments and toe depths computed using the Wang-Reese relationships were close to those measured in the instrumented soldier beams. Table 13 illustrates the difference between the Rankine passive resistances and the Wang-Reese resistances for a 30-ft-high wall in a loose to medium-dense sand having an effective unit weight of 108 pcf and a friction angle of 29°. The soldier beams were 12 in wide and spaced on 8-ft centers. Rankine active earth pressures from the opposite side of the soldier beam were subtracted from the passive resistances.

TABLE 13
Differences Between Rankine and Wang-Reese Passive Resistance for a 30-ft-high Soldier Beam Wall in a Loose to Medium-dense Sand

TOE DEPTH (ft)	WANG-REESE PASSIVE RESISTANCE (kips/ft of depth)	RANKINE PASSIVE RESISTANCES (kips/ft of depth) es
0	-1.12	-1.12
1	-0.60	-0.23
2	0.40	0.67
3	1.90	1.56
4	3.90	2.47
5	6.38	3.36
6	9.34	4.25
7	12.61	5.15
8	16.06	6.05
Positive resistances result from passive pressures and negative resistances result from active pressures		

Wang and Reese's relationships compute the ultimate passive resistances for a drilled shaft wall. They considered three modes of failure and developed equations for the passive resistance at any depth for sands and clays. To incorporate their equations in the design of anchored walls, the ultimate resistances for each failure mode are determined, and the smallest resistance is used to describe the passive resistance of the toe at any depth. The different failure mechanisms in sand are presented to illustrate the concept. Figures showing similar failure mechanisms for clay are presented in Section 6.2.2. One mode of failure assumes that the passive resistance results from a wedge failure in front of an individual soldier beam (Figure 45). When the soldier beams become too close or too deep, the individual wedges will overlap and the lateral resistance for an individual beam will be reduced (Figure 46). At some depth, the soil in front of the beam will be confined and the lateral resistance will not depend upon a wedge failure, but it will be limited by flow of the soil around the beams. Flow resistance will control when the soil plastically flows (Figure 47) between the soldier beams rather than a wedge failure up to the surface. Lateral resistance can be limited by a fourth failure mode not considered by Wang and Reese. At no point can the passive resistance be greater than that computed for a two-dimensional failure surface (Figure 48).

Equations for each failure mode are presented here and those interested in studying their derivation are directed to the work by Wang and Reese (1986) and the *COM624 Manual* (Wang and Reese, 1992).

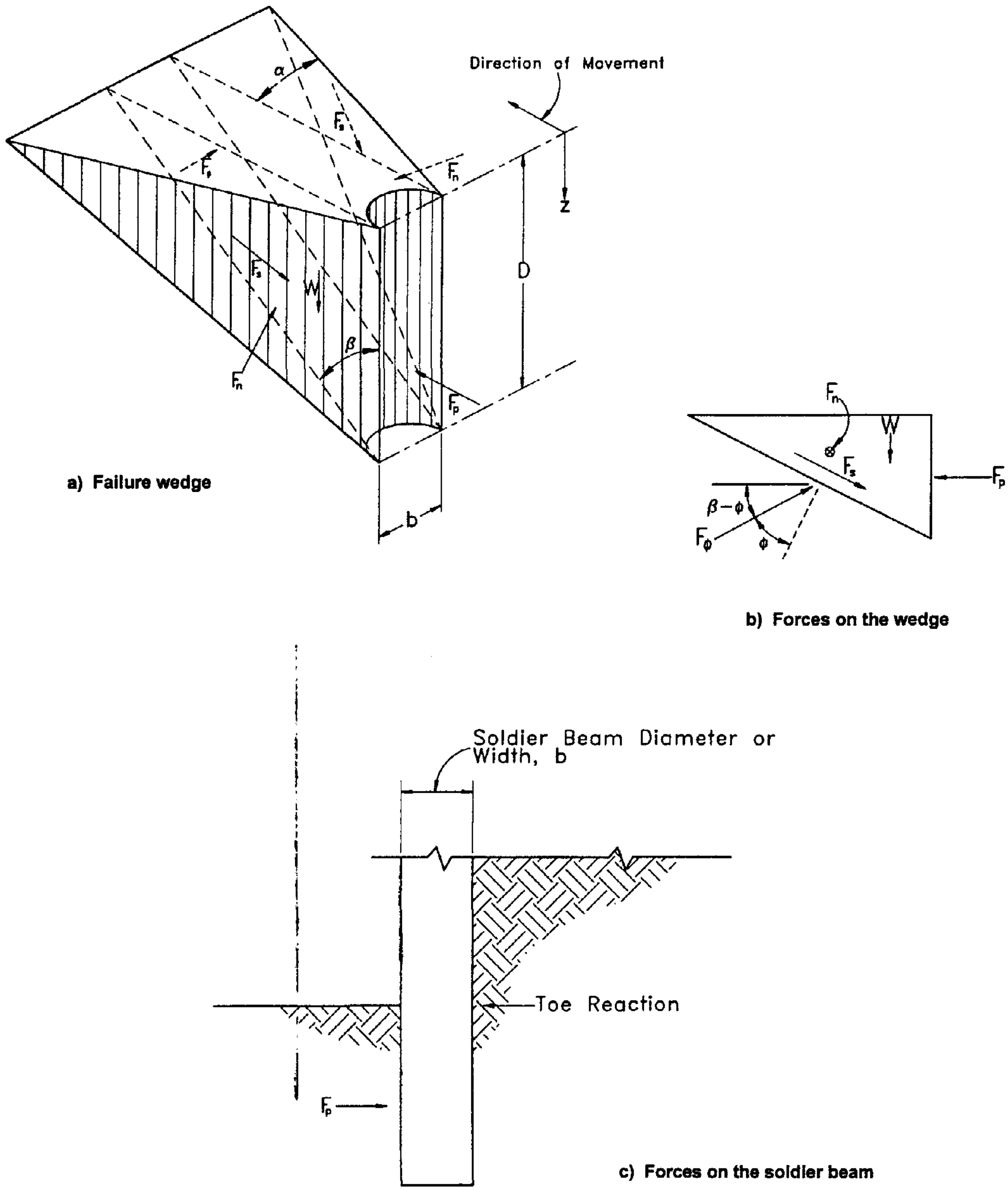


FIGURE 45
Passive Wedge Failure for a Soldier Beam in Sand (after Reese, et al., 1974)

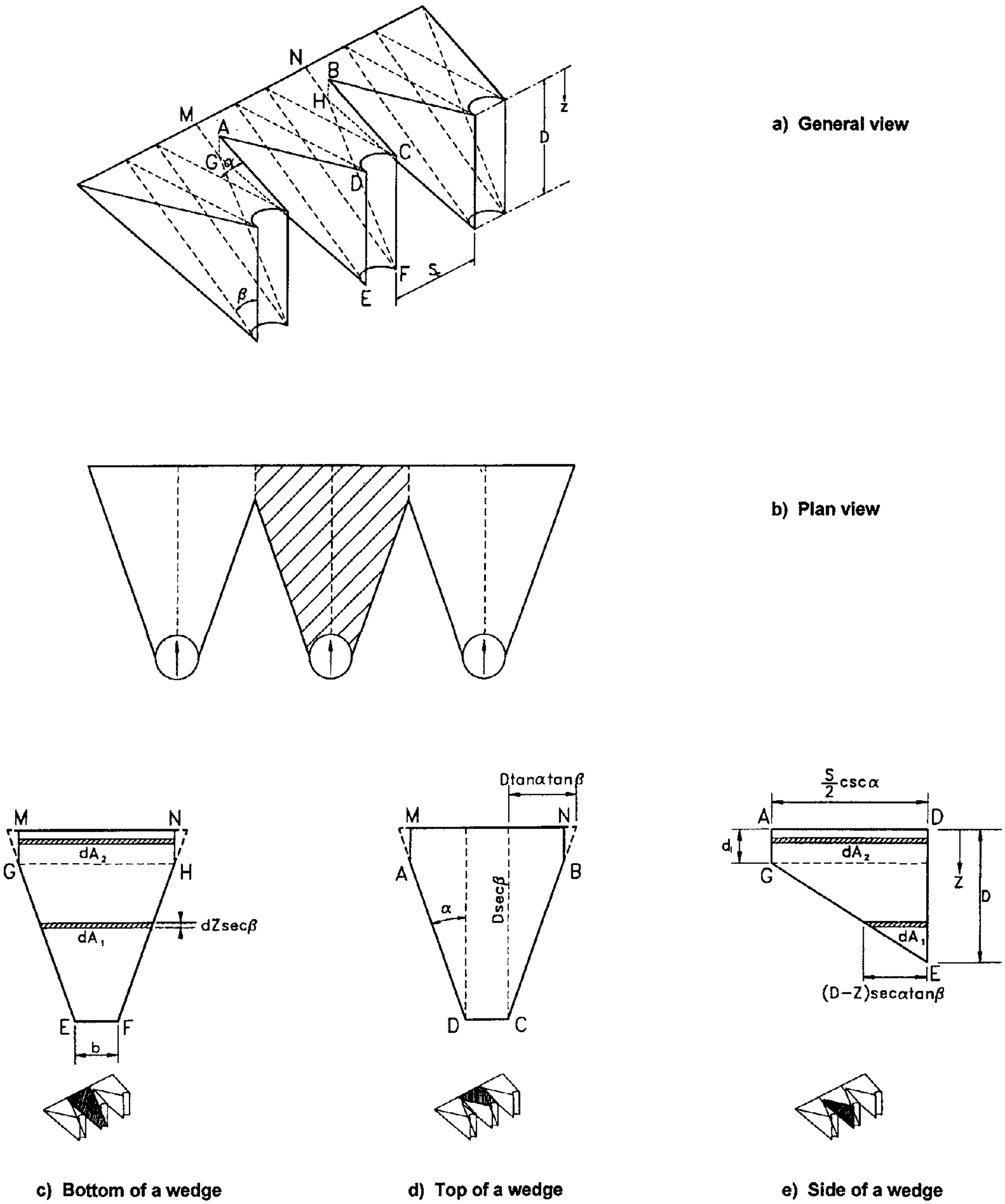


FIGURE 46
Intersecting Failure Wedges for Soldier Beams in Sand (after Wang and Reese, 1986)

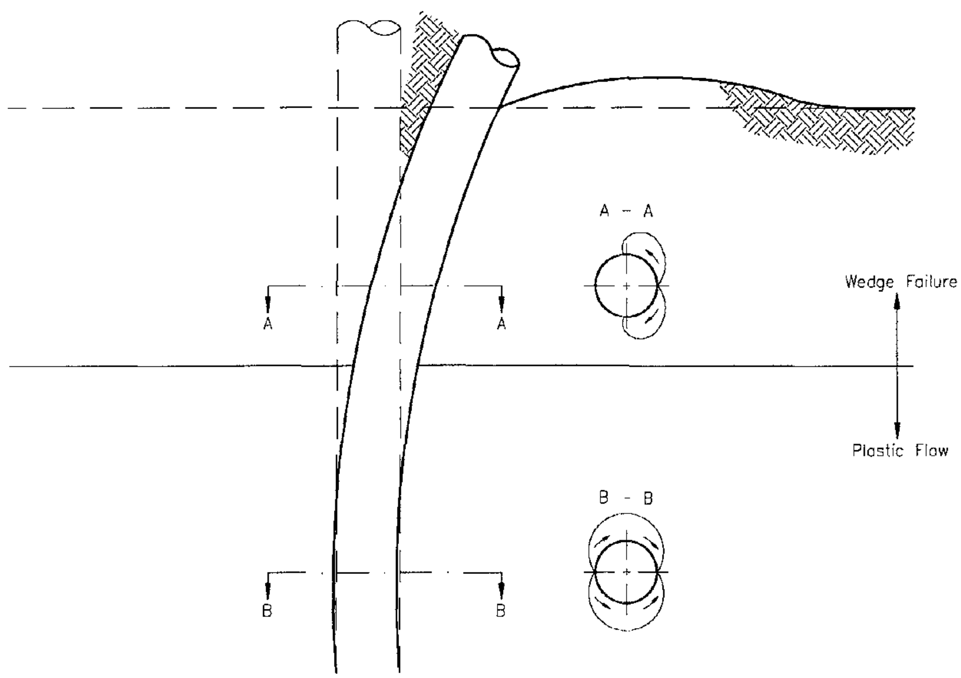
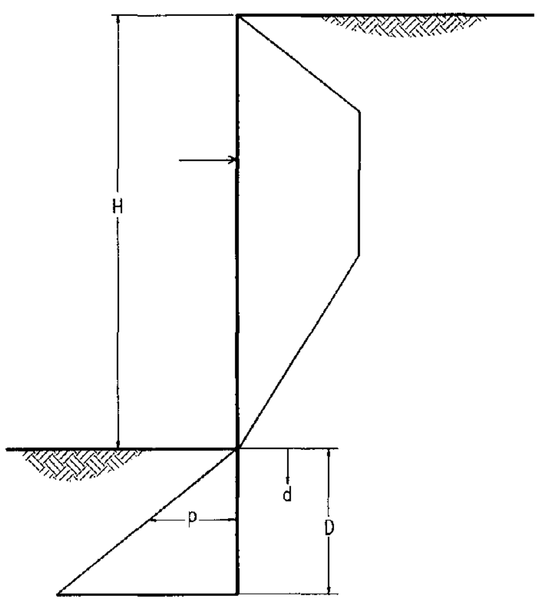


FIGURE 47
Plastic Soil Flow Around a Soldier Beam Toe (after Wang and Reese, 1986)



$p = (K_p \sigma'_v d)(s_c + b)$

K_p = Passive earth pressure coefficient

$\sigma'v$ = Effective vertical stress

d = Distance

D = Toe penetration

s_c = Clear spacing between soldier beams

b = Soldier beam width or diameter

FIGURE 48
Passive Resistance for a Continuous Wall in Sand

6.2.1 Passive Resistances in Sands

Figure 45 shows the wedge failure for a single soldier beam in sand. The passive force, F_p, is given by Equation 6.12 when the groundwater is below the tip of the soldier beam.

$$F_p = \gamma_{ave} D^2 \left[\frac{K_o D \tan\phi \sin\beta}{3\tan(\beta-\phi)\cos\alpha} + \frac{\tan\beta}{\tan(\beta-\phi)} \left(\frac{b}{2} + \frac{D}{3}\tan\beta\tan\alpha \right) + \frac{K_o D \tan\beta}{3}(\tan\phi\sin\beta - \tan\alpha) \right] \quad \text{. . . [6.12]}$$

where:

γ_{ave} = average total unit weight
K_o = at-rest earth pressure coefficient
K_a = active earth pressure coefficient
β = 45 + ϕ/2
α = ϕ for dense sands, ϕ/3 - ϕ/2 for loose sands

Equation 6.12 is differentiated to give the ultimate soil resistance at depth, d (Equation 6.13).

$$p = \gamma d \left[\frac{K_o d \tan\phi \sin\beta}{\tan(\beta-\phi)\cos\alpha} + \frac{\tan\beta}{\tan(\beta-\phi)}(b + d\tan\beta\tan\alpha) + K_o d \tan\beta(\tan\phi\sin\beta - \tan\alpha) \right] \quad \text{. . . [6.13]}$$

Figure 46 shows the individual failure wedges intersecting as the soldier beams get closer or as the toe depth increases. Equation 6.14 gives the depth of the intersection of adjacent wedges.

$$d_i = D - \frac{s_c}{2\tan\alpha\tan\beta} \quad \text{. . . [6.14]}$$

where:

D = toe depth
s_c = clear spacing between soldier beams

When d_i is positive, the failure wedges intersect. If d_i is negative, the failure wedges do not intersect. At depths greater than d_i the passive resistances are not affected by adjacent soldier beams, and they are computed using Equation 6.13. Above the point of intersection, the passive resistances are reduced to account for the intersection of the failure wedges. To account for the intersection of the wedges, the passive resistances determined from Equation 6.13 are reduced by the resistances determined for a wedge with a height, d_i, and a soldier beam with a width of zero. The resistances down to the depth, d_i, are given by Equation 6.15.

$$p = \gamma d \left[\frac{K_o d \tan\phi \sin\beta}{\tan(\beta-\phi)} \left(\frac{1}{\cos\alpha} - 1 \right) + \frac{d\tan\beta\tan\alpha}{\tan(\beta-\phi)} - K_o d \frac{\sin^2\beta}{\cos\beta} \tan\phi(\tan\alpha + 1) \right] \quad \text{. . . [6.15]}$$

where:

$d \le d_i$

At depth, the ultimate lateral resistance will be limited to the resistance that can develop before the soil flows between the soldier beams (Figure 47). Equation 6.16 gives the ultimate lateral flow resistance.

$$p = K_a b \gamma d \tan^8 \beta + K_o \gamma d \tan\phi \tan^4 \beta \qquad \ldots [6.16]$$

Figure 48 shows the two-dimensional failure wedge. Lateral resistances cannot exceed the value given by Equation 6.17.

$$p = K_p \gamma d (s_c + b) \qquad \ldots [6.17]$$

where:

s_c = clear spacing between soldier beams

b = soldier beam width or shaft diameter

Wang and Reese's sand equations included an active earth pressure term subtracted from the passive resistance to give a net resistance at a given depth. The active earth pressure term was dropped from each equation since the ground surface for a wall is not level. Equations 6.13 to 6.17 give the passive resistance at a location, and the Rankine active pressures must be applied to the other side of the wall when computing the capacity of the toe (Figure 49).

For drilled-in soldier beams backfilled with lean mix,use the steel soldier beam width, not the drilled shaft diameter, in computing the passive resistance of the toe. If structural concrete is placed in the toe, the diameter of the drilled shaft can be used in the calculations.

Equations 6.13 to 6.17 can be implemented in a spreadsheet. Figure 50 shows a spreadsheet developed to determine the failure mode that controls the lateral resistance at any depth, the pressures for each foot of depth, and the total passive capacity assuming a given wall height and a toe embedment.

Broms (1965) recommended that the passive resistance in front of the wall be reduced to zero for a depth equal to 1.5 times the width of the soldier beam. This reduction is apparently an attempt to account for disturbances near the ground surface. Soldier beam toes are initially far below the original ground surface, and the depth of disturbance in front of the wall is different from the depth of disturbance assumed by Broms. For a wall, the depth of disturbance does not depend on soldier beam width. Instead, the designer should select the depth of disturbance based upon construction activities. It is possible for the depth of disturbance to be zero. The spreadsheet allows the depth of disturbance to be selected, and assumes the passive resistance is zero over the depth of disturbance.

When the groundwater level is near the bottom of the excavation, use buoyant unit weights in the equations and the spreadsheet for passive toe resistance. If the soldier beam toe is long and the groundwater is found a reasonable distance from the bottom of the excavation, assuming the groundwater to be at the bottom of the excavation is conservative. Then, compute an apparent passive resistance factor from the selected toe penetration depth. For example, at a toe penetration depth of 7 ft, the total passive force from the spreadsheet is 35.84 kips, and the apparent passive resistance factor for this case is $K_{ap} = 35.84/(0.5\gamma d^2) = 13.54$. Apparent passive resistance factors are a unique value that depends upon geometry and the toe depth and they will not be constant with depth. The apparent passive resistance factor can be used like a passive earth pressure coefficient, allowing effective unit weights to be used to calculate the passive resistance of a partially submerged soldier beam toe. Figure 51 illustrates the use of the apparent passive resistance factor for a 30-ft-high soldier beam wall with a 7-ft toe, and the groundwater table located 5 ft below the bottom of the excavation.

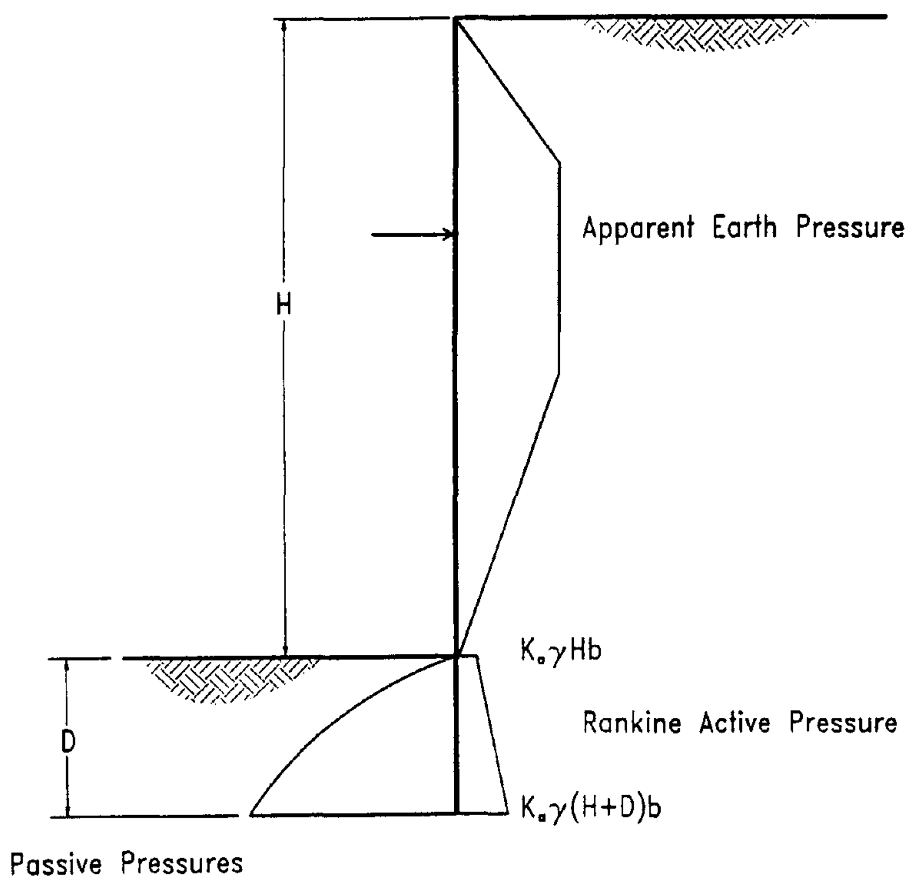


FIGURE 49
Diagram Illustrating the Active and Passive Pressures on a Soldier Beam Toe

uw	108	(pcf)	phi	29	phtn	0.554309	ko	0.52				
ht	30	(ft)	bet	59.5	betn	1.697663	ka	0.347	Toe Reaction =	15.29	(kips)	(from AEP calculations)
b	1	(ft)	alp	19.33333	altn	0.350848	kp	2.882	Depth of disturbance at toe =	0	(ft)	
s	8	(ft)	btmph	30.500	bmptn	0.589045						
scl	7	(ft)										

Ultimate Passive Resistances and Forces

			Passive Resistance Wedge or Flow Conditions									Passive Capacity					
Toe Depth (d)	Wing Width	(Eq. 6.14) Intersection Height (hi)	(Eq. 6.13) Wedge Resistance (kips/ft)	Wedge Resistance, d = hi (kips/ft)	Wedge Resistance d = hi, α = 0 (kips/ft)	(Eq. 6.15) Intersecting Wedge Resistance (kips/ft)	(Eq. 6.16) Flow Resistance (kips/ft)	Critical Wedge or Flow Resistance (kips/ft)	(Eq. 6.17) Rankine Passive Resistance (kips/ft)	Passive Resistance (kips/ft)	Failure Mode	Total Passive Force at Given Toe Depth (kips)	Total Active Force on Toe at Given Toe Depth (kips)	Allowance for Disturbance (Eq.6.12 for depth) (kips)	Net Passive Force at a Given Depth (kips)	Factor of Safety	Toe Depth (ft)
0	0.00	-5.88	0.00	6.64	1.29	0.00	0.00	0.00	0.00	0.00	Wedge	0.00	1.12	0	-1.12	-0.07	0
1	0.60	-4.88	0.56	4.31	0.63	0.56	2.80	0.56	2.49	0.56	Wedge	0.28	1.14	0.00	-0.86	-0.06	1
2	1.19	-3.88	1.60	2.48	0.15	1.60	5.61	1.60	4.98	1.60	Wedge	1.36	2.32	0.00	-0.97	-0.06	2
3	1.79	-2.88	3.14	1.13	-0.15	3.14	8.41	3.14	7.47	3.14	Wedge	3.73	3.54	0.00	0.19	0.01	3
4	2.38	-1.88	5.17	0.28	-0.27	5.17	11.22	5.17	9.96	5.17	Wedge	7.88	4.80	0.00	3.09	0.20	4
5	2.98	-0.88	7.69	-0.08	-0.20	7.69	14.02	7.69	12.45	7.69	Wedge	14.31	6.09	0.00	8.22	0.54	5
6	3.57	0.12	10.69	0.04	0.04	10.69	16.82	10.69	14.94	10.69	Wedge	23.50	7.42	0.00	16.08	1.05	6
7	4.17	1.12	14.19	0.66	0.46	14.00	19.63	14.00	17.43	14.00	Interaction	35.84	8.79	0.00	27.06	1.77	7
8	4.76	2.12	18.18	1.77	1.07	17.48	22.43	17.48	19.92	17.48	Interaction	51.58	10.19	0.00	41.39	2.71	8
9	5.36	3.12	22.66	3.36	1.85	21.15	25.24	21.15	22.41	21.15	Interaction	70.90	11.64	0.00	59.26	3.88	9
10	5.96	4.12	27.63	5.45	2.82	25.00	28.04	25.00	24.90	24.90	Rankine	93.92	13.12	0.00	80.81	5.29	10
11	6.55	5.12	33.09	8.03	3.96	29.02	30.85	29.02	27.39	27.39	Rankine	120.07	14.63	0.00	105.44	6.90	11
12	7.15	6.12	39.04	11.10	5.29	33.23	33.65	33.23	29.88	29.88	Rankine	148.71	16.19	0.00	132.52	8.67	12
13	7.74	7.12	45.48	14.66	6.80	37.62	36.45	36.45	32.37	32.37	Rankine	179.83	17.78	0.00	162.05	10.60	13
14	8.34	8.12	52.41	18.71	8.48	42.19	39.26	39.26	34.86	34.86	Rankine	213.45	19.41	0.00	194.04	12.69	14
15	8.93	9.12	59.83	23.25	10.35	46.94	42.06	42.06	37.35	37.35	Rankine	249.56	21.08	0.00	228.48	14.94	15
16	9.53	10.12	67.75	28.28	12.40	51.87	44.87	44.87	39.84	39.84	Rankine	288.15	22.78	0.00	265.37	17.36	16
17	10.13	11.12	76.15	33.80	14.63	56.97	47.67	47.67	42.33	42.33	Rankine	329.24	24.53	0.00	304.71	19.93	17
18	10.72	12.12	85.04	39.81	17.04	62.27	50.47	50.47	44.82	44.82	Rankine	372.82	26.31	0.00	346.51	22.66	18
19	11.32	13.12	94.42	46.31	19.63	67.74	53.28	53.28	47.31	47.31	Rankine	418.88	28.12	0.00	390.76	25.56	19
20	11.91	14.12	104.30	53.30	22.40	73.39	56.08	56.08	49.80	49.80	Rankine	467.44	29.98	0.00	437.46	28.61	20
21	12.51	15.12	114.66	60.79	25.35	79.22	58.89	58.89	52.29	52.29	Rankine	518.49	31.87	0.00	486.62	31.83	21
22	13.10	16.12	125.51	68.76	28.48	85.23	61.69	61.69	54.78	54.78	Rankine	572.02	33.80	0.00	538.22	35.20	22
23	13.70	17.12	136.86	77.22	31.79	91.42	64.50	64.50	57.27	57.27	Rankine	628.05	35.77	0.00	592.28	38.74	23
24	14.29	18.12	148.69	86.18	35.28	97.80	67.30	67.30	59.76	59.76	Rankine	686.57	37.77	0.00	648.80	42.43	24
25	14.89	19.12	161.02	95.62	38.95	104.35	70.10	70.10	62.25	62.25	Rankine	747.58	39.82	0.00	707.76	46.29	25
26	15.49	20.12	173.83	105.55	42.80	111.08	72.91	72.91	64.74	64.74	Rankine	811.07	41.90	0.00	769.18	50.31	26
27	16.08	21.12	187.14	115.98	46.84	118.00	75.71	75.71	67.23	67.23	Rankine	877.06	44.01	0.00	833.05	54.48	27
28	16.68	22.12	200.94	126.89	51.05	125.09	78.52	78.52	69.72	69.72	Rankine	945.54	46.17	0.00	899.37	58.82	28
29	17.27	23.12	215.22	138.30	55.44	132.37	81.32	81.32	72.21	72.21	Rankine	1016.51	48.36	0.00	968.15	63.32	29
30	17.87	24.12	230.00	150.19	60.02	139.82	84.12	84.12	74.70	74.70	Rankine	1089.96	50.59	0.00	1039.38	67.98	30
31	18.46	25.12	245.27	162.58	64.77	147.46	86.93	86.93	77.19	77.19	Rankine	1165.91	52.86	0.00	1113.06	72.80	31
32	19.06	26.12	261.02	175.45	69.71	155.28	89.73	89.73	79.68	79.68	Rankine	1244.35	55.16	0.00	1189.19	77.78	32
33	19.66	27.12	277.27	188.82	74.82	163.27	92.54	92.54	82.17	82.17	Rankine	1325.28	57.50	0.00	1267.78	82.92	33
34	20.25	28.12	294.01	202.68	80.12	171.45	95.34	95.34	84.66	84.66	Rankine	1408.70	59.88	0.00	1348.81	88.22	34
35	20.85	29.12	311.24	217.02	85.60	179.81	98.14	98.14	87.15	87.15	Rankine	1494.61	62.30	0.00	1432.31	93.68	35
36	21.44	30.12	328.96	231.86	91.25	188.35	100.95	100.95	89.64	89.64	Rankine	1583.00	64.75	0.00	1518.25	99.30	36
37	22.04	31.12	347.17	247.19	97.09	197.07	103.75	103.75	92.13	92.13	Rankine	1673.89	67.25	0.00	1606.65	105.08	37
38	22.63	32.12	365.87	263.01	103.11	205.97	106.56	106.56	94.62	94.62	Rankine	1767.27	69.78	0.00	1697.50	111.02	38
39	23.23	33.12	385.06	279.32	109.31	215.05	109.36	109.36	97.11	97.11	Rankine	1863.14	72.34	0.00	1790.80	117.12	39
40	23.82	34.12	404.74	296.12	115.69	224.31	112.17	112.17	99.60	99.60	Rankine	1961.50	74.95	0.00	1886.55	123.38	40
41	24.42	35.12	424.91	313.41	122.25	233.75	114.97	114.97	102.09	102.09	Rankine	2062.35	77.59	0.00	1984.76	129.81	41
42	25.02	36.12	445.57	331.19	128.99	243.37	117.77	117.77	104.58	104.58	Rankine	2165.69	80.27	0.00	2085.42	136.39	42

FIGURE 50
Sample of Spreadsheet for Computing Passive Resistance of Soldier Toes in Sand

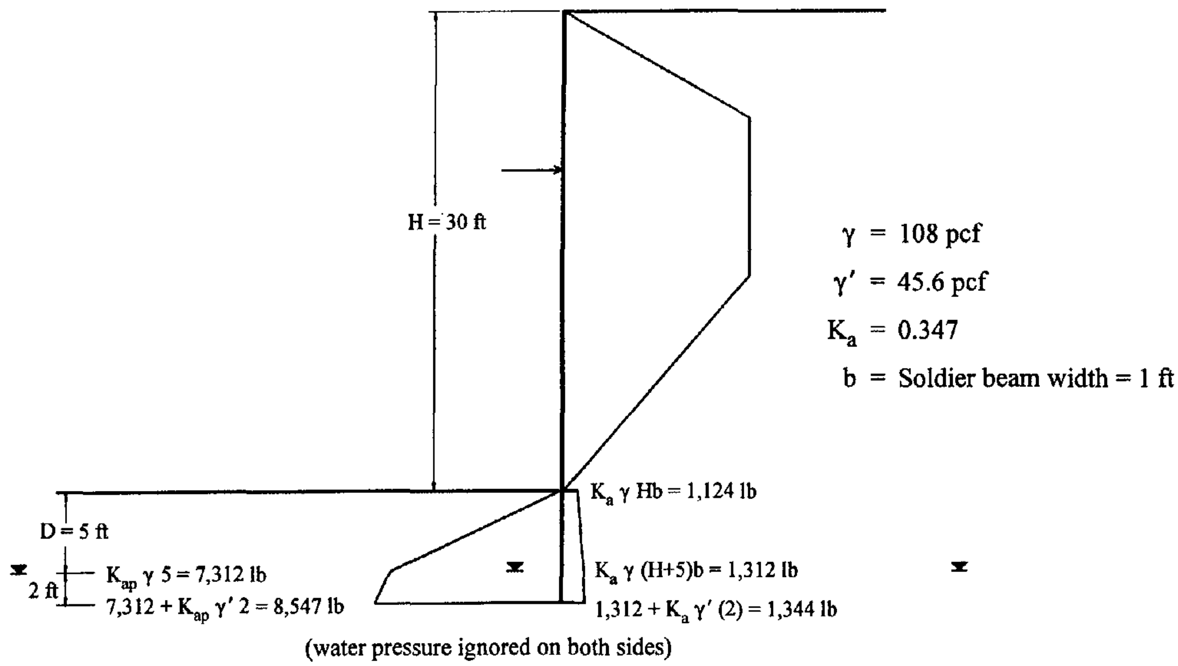


FIGURE 51
Example of Showing Apparent Passive Resistance Factor When Soldier Beam Toe is Partially Submerged

6.2.2 Passive Resistances in Clays

Figure 52 shows the failure wedge for a single soldier beam in clay. Reese (1958) developed the expression for the passive resistance, F_p,

$$F_p = s_u D\,[\tan\theta + (1+K)\cot\theta] + \frac{1}{2}\gamma_{ave}\, b D^2 + s_u D^2 \sec\theta \qquad \text{. . . [6.18]}$$

where:

s_u = average undrained shear strength
K = a reduction factor to apply to s_u to give the adhesion between the soldier beam and the clay
γ_{ave} = average total unit weight of soil
(the other terms are defined in the figure)

Assuming $\theta = 45°$ and the shaft friction, $K = 0$, Equation 6.18 is differentiated to give the ultimate soil resistance at depth, d (Equation 6.19).

$$p = 2s_u b + \gamma b d + 2.83\, s_u d \qquad \text{. . . [6.19]}$$

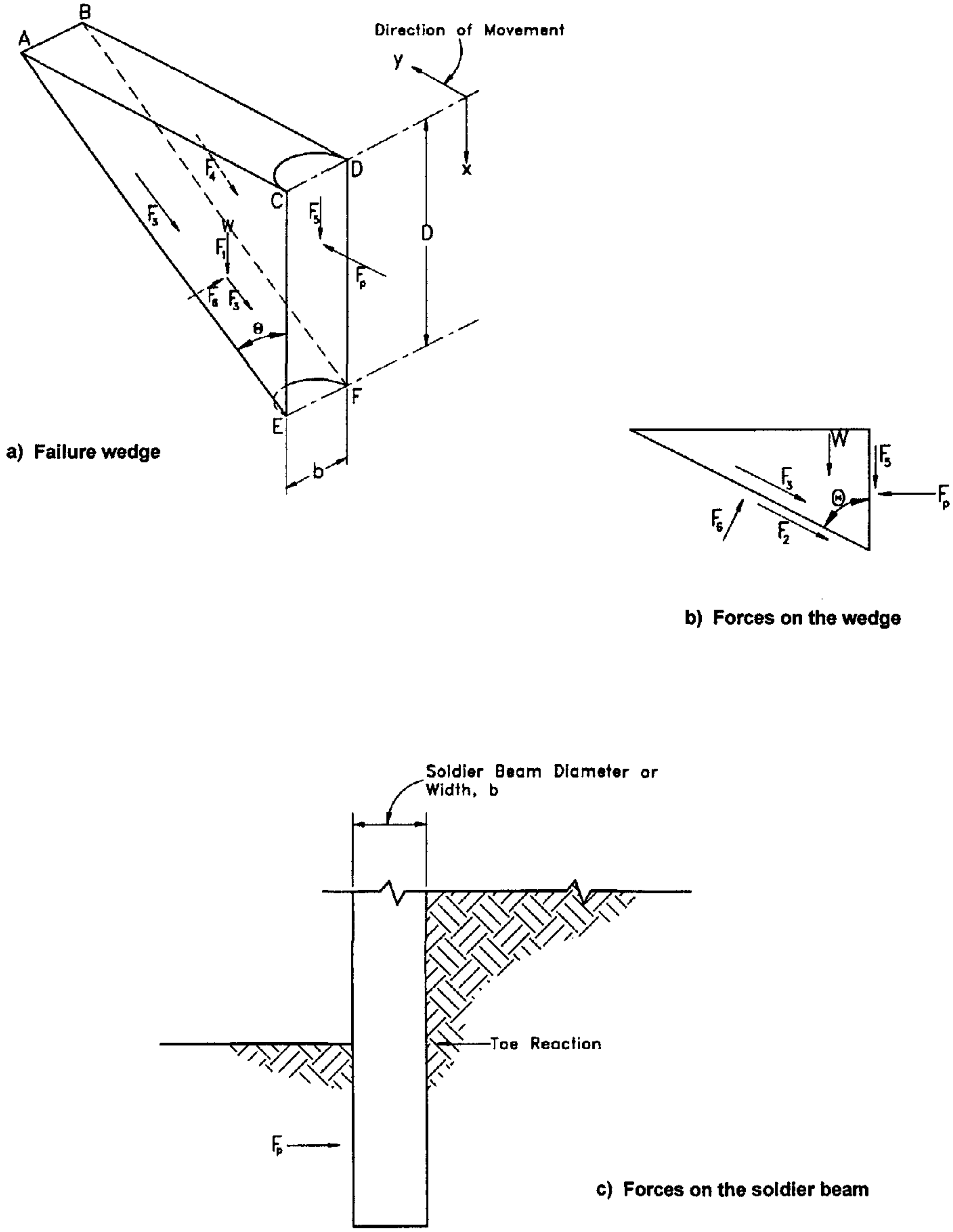


FIGURE 52
Passive Wedge Failure for a Soldier Beam in Clay (after Reese, 1958)

Soldier beams in clay may be close enough that the wedge of soil between the beams is not adequate to develop the full shear resistance (forces F_3 and F_4 in Figure 52) on the sides of the wedge directly in front of the soldier beam. Figure 53 shows the passive wedges in front of each soldier beam and the wedge of soil between the beams (block FDBGHI). If the space between the beams is large, block FDBGHI will be adequate to resist the side shear forces F_3 and F_4 from the wedges in front of the beams. If block FDBGHI is small, then the entire ground in front of the wall will move together and the individual wedges in front of each beam will not develop. Wang and Reese (1986) developed expressions to describe the passive resistance of a row of drilled shafts (soldier beams) in clay. Equation 6.20 gives the critical spacing where the behavior changes from single beam behavior to group behavior.

$$S_{cr} = \frac{2.828\, s_u D}{\gamma_{ave} D + 6 s_u} \qquad \text{. . . [6.20]}$$

Wang and Reese's passive resistance for a soldier beam considering group behavior is given by Equation 6.21.

$$p = 2 s_u (b + s_c) + \gamma_{ave} (b + s_u) d + s_u s_c \qquad \text{. . . [6.21]}$$

If the spacing between soldier beams becomes zero and the soldier beam width is taken as unity, Equation 6.21 becomes Equation 6.22, the passive earth pressure equation for a continuous wall.

$$p = 2 s_u + \gamma_{ave} d \qquad \text{. . . [6.22]}$$

When the toe of the soldier beam extends deep enough below the ground, the soil may flow around the beam as it moves through the soil. The failure is similar to that shown in Figure 47. Wang and Reese (1986) expressed the passive flow resistance in a clay to be approximately (Equation 6.23):

$$p = 11 s_u b \qquad \text{. . . [6.23]}$$

Figure 54 shows the two-dimensional failure wedge that limits the passive resistance that can develop. For a wall in clay the lateral resistance at any depth, d, cannot exceed the value given by Equation 6.24.

$$p = (2 s_u + \gamma_{ave} d)(s_c + b) \qquad \text{. . . [6.24]}$$

Wang and Reese's equations for clays do not include an active pressure term. In stiff clays the active pressure may be negative behind the wall. Considering negative pressures during design is not reasonable since the soldier beam will move away from the soil. A continuous wall will normally be used when the active pressures are positive. Positive active pressures below the bottom of the excavation are given by Equation 6.25.

$$p_{active} = \gamma_{ave} (H + d) - 2 s_u \qquad \text{. . . [6.25]}$$

where:

H = height of wall

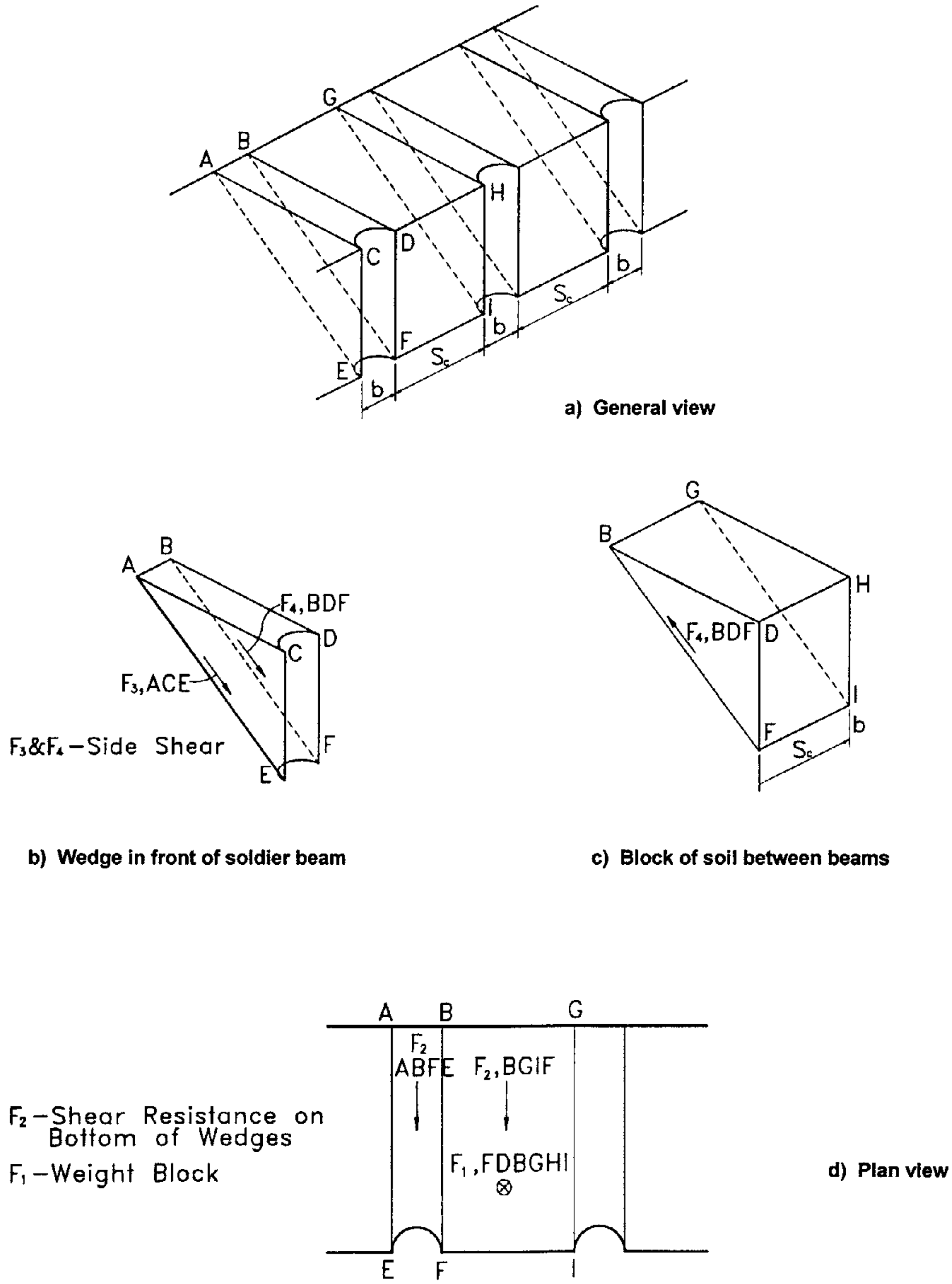


FIGURE 53
Failure Wedges for Adjacent Soldier Beams in Clay (after Wang and Reese, 1986)

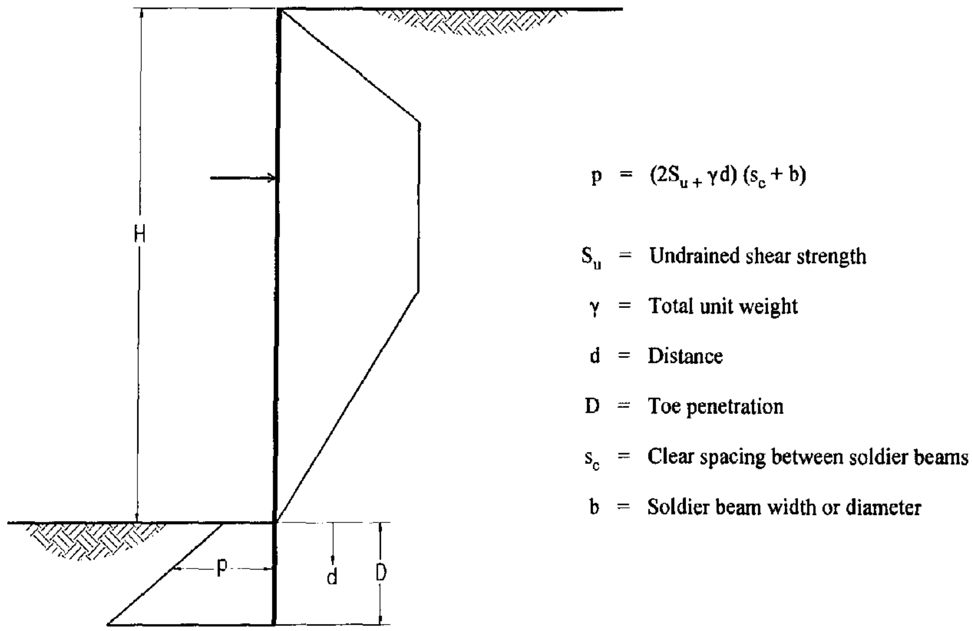


FIGURE 54
Diagram Illustrating the Passive Resistance for a Continuous Wall in Clay

Similar to the soldier beams in sand, soldier beam width is used for drilled shafts backfilled with lean mix, and the drilled shaft diameter is used when the structural concrete is used to backfill the shaft.

Figure 55 shows a spreadsheet, similar to the one for sand, developed to calculate the failure mode that controls the lateral resistance, the resistances for each foot of depth, and the total passive capacity assuming a given toe depth.

γ	120	s/b	4	Toe Reaction =	18.5 (kips) (from AEP calculations)
su	1500	α	1	Depth of disturbance at toe =	1.5 (ft)
b	2				
s	10				
Sc	8				

Toe Depth, d (ft)	(Eq. 6.20) Scr (ft)	Passive Resistance Wedge or Flow Conditions: (Eq. 6.19) Single Beam Wedge Resistance (kips/ft)	(Eq. 6.21) Row of Beams Wedge Resistance (kips/ft)	Critical Wedge Resistance (kips/ft)	(Eq. 6.23) Single Beam Flow Resistance (kips/ft)	Critical Wedge or Flow Resistance (kips/ft)	(Eq. 6.24) Rankine Passive Resistance (kips/ft)	Passive Resistance (kips/ft)	Failure Mode	Passive Capacity: Total Passive Force at Given Toe Depth (kips)	Allowance for Disturbance (Eq. 6.18 for depth) (kips)	Net Passive Force at a Given Depth (kips)	Factor of Safety	Toe Depth (ft)
0	0.00	6.00	42.00	6.00	33.00	6.00	30.00	6.00	Wedge	0.00	14.04	-14.04	-0.76	0
1	0.47	10.49	43.20	10.49	33.00	10.49	31.20	10.49	Wedge	8.24	14.04	-5.80	-0.31	1
2	0.92	14.97	44.40	14.97	33.00	14.97	32.40	14.97	Wedge	20.97	14.04	6.93	0.37	2
3	1.36	19.46	45.60	19.46	33.00	19.46	33.60	19.46	Wedge	38.18	14.04	24.14	1.30	3
4	1.79	23.94	46.80	23.94	33.00	23.94	34.80	23.94	Wedge	59.88	14.04	45.84	2.48	4
5	2.21	28.43	48.00	28.43	33.00	28.43	36.00	28.43	Wedge	86.06	14.04	72.02	3.89	5
6	2.62	32.91	49.20	32.91	33.00	32.91	37.20	32.91	Wedge	116.73	14.04	102.69	5.55	6
7	3.02	37.40	50.40	37.40	33.00	33.00	38.40	33.00	Flow	149.69	14.04	135.64	7.33	7
8	3.41	41.88	51.60	41.88	33.00	33.00	39.60	33.00	Flow	182.69	14.04	168.64	9.12	8
9	3.79	46.37	52.80	46.37	33.00	33.00	40.80	33.00	Flow	215.69	14.04	201.64	10.90	9
10	4.16	50.85	54.00	50.85	33.00	33.00	42.00	33.00	Flow	248.69	14.04	234.64	12.68	10
11	4.52	55.34	55.20	55.34	33.00	33.00	43.20	33.00	Flow	281.69	14.04	267.64	14.47	11
12	4.88	59.82	56.40	59.82	33.00	33.00	44.40	33.00	Flow	314.69	14.04	300.64	16.25	12
13	5.22	64.31	57.60	64.31	33.00	33.00	45.60	33.00	Flow	347.69	14.04	333.64	18.03	13
14	5.56	68.79	58.80	68.79	33.00	33.00	46.80	33.00	Flow	380.69	14.04	366.64	19.82	14
15	5.89	73.28	60.00	73.28	33.00	33.00	48.00	33.00	Flow	413.69	14.04	399.64	21.60	15
16	6.22	77.76	61.20	77.76	33.00	33.00	49.20	33.00	Flow	446.69	14.04	432.64	23.39	16
17	6.53	82.25	62.40	82.25	33.00	33.00	50.40	33.00	Flow	479.69	14.04	465.64	25.17	17
18	6.84	86.73	63.60	86.73	33.00	33.00	51.60	33.00	Flow	512.69	14.04	498.64	26.95	18
19	7.15	91.22	64.80	91.22	33.00	33.00	52.80	33.00	Flow	545.69	14.04	531.64	28.74	19
20	7.44	95.70	66.00	95.70	33.00	33.00	54.00	33.00	Flow	578.69	14.04	564.64	30.52	20
21	7.73	100.19	67.20	100.19	33.00	33.00	55.20	33.00	Flow	611.69	14.04	597.64	32.31	21
22	8.02	104.67	68.40	68.40	33.00	33.00	56.40	33.00	Flow	644.69	14.04	630.64	34.09	22
23	8.30	109.16	69.60	69.60	33.00	33.00	57.60	33.00	Flow	677.69	14.04	663.64	35.87	23
24	8.57	113.64	70.80	70.80	33.00	33.00	58.80	33.00	Flow	710.69	14.04	696.64	37.66	24
25	8.84	118.13	72.00	72.00	33.00	33.00	60.00	33.00	Flow	743.69	14.04	729.64	39.44	25
26	9.10	122.61	73.20	73.20	33.00	33.00	61.20	33.00	Flow	776.69	14.04	762.64	41.22	26
27	9.36	127.10	74.40	74.40	33.00	33.00	62.40	33.00	Flow	809.69	14.04	795.64	43.01	27
28	9.61	131.58	75.60	75.60	33.00	33.00	63.60	33.00	Flow	842.69	14.04	828.64	44.79	28
29	9.86	136.07	76.80	76.80	33.00	33.00	64.80	33.00	Flow	875.69	14.04	861.64	46.58	29
30	10.10	140.55	78.00	78.00	33.00	33.00	66.00	33.00	Flow	908.69	14.04	894.64	48.36	30
31	10.34	145.04	79.20	79.20	33.00	33.00	67.20	33.00	Flow	941.69	14.04	927.64	50.14	31
32	10.57	149.52	80.40	80.40	33.00	33.00	68.40	33.00	Flow	974.69	14.04	960.64	51.93	32
33	10.80	154.01	81.60	81.60	33.00	33.00	69.60	33.00	Flow	1007.69	14.04	993.64	53.71	33
34	11.03	158.49	82.80	82.80	33.00	33.00	70.80	33.00	Flow	1040.69	14.04	1026.64	55.49	34
35	11.25	162.98	84.00	84.00	33.00	33.00	72.00	33.00	Flow	1073.69	14.04	1059.64	57.28	35
36	11.46	167.46	85.20	85.20	33.00	33.00	73.20	33.00	Flow	1106.69	14.04	1092.64	59.06	36
37	11.68	171.95	86.40	86.40	33.00	33.00	74.40	33.00	Flow	1139.69	14.04	1125.64	60.85	37
38	11.89	176.43	87.60	87.60	33.00	33.00	75.60	33.00	Flow	1172.69	14.04	1158.64	62.63	38
39	12.09	180.92	88.80	88.80	33.00	33.00	76.80	33.00	Flow	1205.69	14.04	1191.64	64.41	39
40	12.30	185.40	90.00	90.00	33.00	33.00	78.00	33.00	Flow	1238.69	14.04	1224.64	66.20	40
41	12.49	189.89	91.20	91.20	33.00	33.00	79.20	33.00	Flow	1271.69	14.04	1257.64	67.98	41
42	12.69	194.37	92.40	92.40	33.00	33.00	80.40	33.00	Flow	1304.69	14.04	1290.64	69.76	42
43	12.88	198.86	93.60	93.60	33.00	33.00	81.60	33.00	Flow	1337.69	14.04	1323.64	71.55	43
44	13.07	203.34	94.80	94.80	33.00	33.00	82.80	33.00	Flow	1370.69	14.04	1356.64	73.33	44
45	13.26	207.83	96.00	96.00	33.00	33.00	84.00	33.00	Flow	1403.69	14.04	1389.64	75.12	45
46	13.44	212.31	97.20	97.20	33.00	33.00	85.20	33.00	Flow	1436.69	14.04	1422.64	76.90	46

FIGURE 55
Sample of Spreadsheet for Computing Passive Resistance of Soldier Beam Toes in Clay

CHAPTER 7: SOIL-STRUCTURE INTERACTION ANALYSES

Soil-structure interaction computer codes based on finite difference methods or simple finite element methods are used to analyze structural beam-columns, laterally loaded piles, and beams supported by linear and non-linear springs. These computer codes are called beam-column programs. BMCOL76 (Matlock, et al.,1981), a finite difference program, and CBEAMC (Dawkins, 1994), a finite element program, are two widely used beam-column computer programs. These programs model the earth pressure-deflection behavior of the system identically and give the same results for similar problems.

The programs can be used to analyze anchored soldier beam and sheet pile walls. Above the bottom of the wall, apparent earth pressure diagrams give the load applied to the wall. Below the bottom of the wall, the passive resistance is modeled by a series of non-linear springs. Ground anchors are modeled as non-linear springs, too. Figure 56 illustrates how the apparent earth pressure, toe resistance, and ground anchors are modeled in the analyses. The apparent earth pressures are modeled as a non-uniform distributed load. Ground anchors are modeled as concentrated T-y curves, where T is the anchor load and y is the deflection of the wall at the anchor location. The lateral loads on the toe of the soldier beam are modeled by R-y curves, where R is the resistance and y is the beam deflection. Active R_a-y curves are on the back of the wall and passive R_p-y curves are on the front of the wall. The wall is modeled as a continuous member. As the wall moves, changes in the lateral resistance on each side of the wall are described by the R-y curves. The maximum resistance is related to the passive capacity of the beam and the minimum resistance is related to the active capacity.

7.1 EARTH PRESSURES

Apparent earth pressure diagrams describe the earth load applied to flexible anchored walls in soil-structure interaction analyses better than a series of non-linear soil springs. Research presented in *Summary Report of Research on Permanent Ground Anchor Walls*, "Volume II: Full-scale Wall Tests and a Soil-Structure Interaction Model," (Weatherby, et al., 1998) showed that non-linear soil springs did not accurately model the earth pressures behind flexible walls when the ground anchors were locked-off at loads determined from reasonable apparent earth pressure diagrams. To match predicted behavior with the measured behavior, the active earth pressure used to define the minimum load associated with the springs behind the wall had to be reduced by 50 percent. The active pressures had to be reduced, since they were defined in terms of Rankine or Coulomb coefficients. Active earth pressure coefficients cannot model the redistribution of earth pressures that occurs behind flexible anchored walls. Arching, stressing the ground anchors, construction procedures, and facial stiffness cause the earth pressures on flexible walls to redistribute to the supports. On the other hand, apparent earth pressure diagrams were developed from measured loads and include the effects of arching, soldier beam flexibility, preloading of supports, facial stiffness, and construction procedures.

Earth pressure-deflection curves and non-linear soil springs are used to describe the earth pressures for the design of stiff, structural diaphragm walls. Arching, construction procedures, and pressure redistributions may not affect the earth pressures behind these walls to the same extent as behind an anchored flexible wall. In addition, the assumptions made regarding the soil pressures are not as critical for diaphragm walls since they are normally used as a cut-off wall. Water pressures behind a diaphragm wall are frequently greater than the soil loads. Research reported by Weatherby, et al. (1998) showed that non-linear soil springs could be used to model the earth pressures behind walls when the ground anchors were stressed to loads equal to or greater than those associated with at-rest-pressures. In these cases, the walls distributed the anchor loads to the ground, and they were stiff enough to prevent the development of large zones of active pressures.

Apparent earth pressure diagrams to be used in a soil-structure interaction analysis are the same ones discussed in Chapter 4. The shape of the diagram is not as important as the total earth load to be distributed to the wall. The modified trapezoidal diagram is recommended for the design of temporary and permanent anchored walls in sands and stiff clays. The other diagrams presented in Chapter 4 will give satisfactory results.

7.2 MODELING THE LATERAL RESISTANCE OF THE TOE

Figure 56 shows the nature of the R-y curves used to describe the load-deflection behavior of the toe of the wall. As the wall is loaded by the apparent earth pressures and the ground anchors, the soldier beam will deflect adjusting the load in the soil springs (R-y curves) along the toe and the ground anchor springs (T-y curves) until force equilibrium is satisfied. As the wall deflects outward, load will be mobilized in each spring up to the load associated with the passive resistance of that spring. The deflection required to mobilize the full passive resistance at a point is the passive reference deflection, y_p. While the wall is moving out, the lateral resistance on the other side of the soldier beam will reduce to an active value. The deflection required to reduce the lateral resistance to an active value is the active reference deflection, y_a. When the deflection exceeds either the active or passive reference deflection, the load will no longer change, and the full active or passive resistance will be applied to that side of the wall.

The R_p-y curve is on the excavation side of the wall, and the R_a-y curve is on the opposite side of the wall. To construct an R-y curve, determine the maximum and minimum resistances and the slope of the curve between these two values. Maximum resistances for the R-y curves are computed using the relationships developed by Wang and Reese (1986) (Chapter 6). The minimum resistances are equal to the Rankine active earth pressures over the width of the soldier beam. The slope of the R-y curve between the maximum and minimum resistance is defined by the referenced deflections y_a and y_p. Referenced deflections represent the movements required to develop the active or passive resistance at a location. Defining the slope of the curve in this manner is simple compared with the techniques used for laterally loaded drilled shafts or piles. Parametric studies showed that the bending moments in flexible walls were not very sensitive to the slope of the R-y curves (stiffness of the non-linear spring) (Weatherby, et al., 1998). The parametric studies did show that the moments were sensitive to the values of the maximum and minimum resistance used to develop the R-y curves. Table 14 gives reference deflections developed by Kim and Briaud (1994). Figure 57 illustrates the construction the R_p-y curve for a soldier beam wall, and Figure 58 shows how to construct the R_a-y curve. The resistances and deflections for these curves are inputs in the soil-structure analysis.

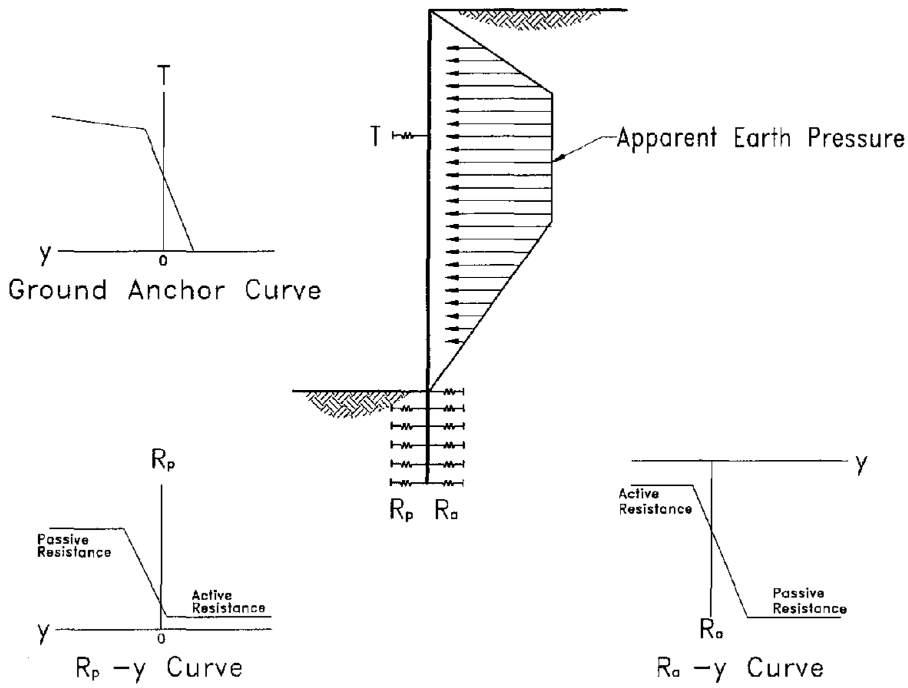
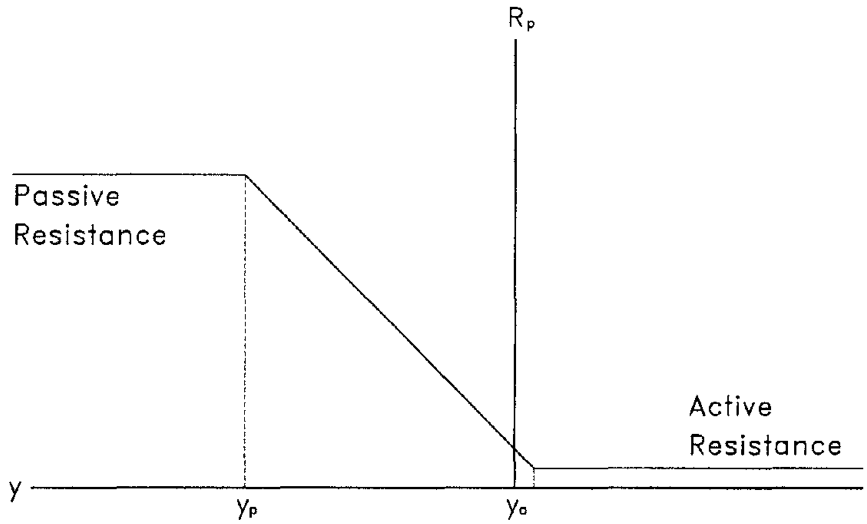


FIGURE 56
Modeling of Earth Pressures, Toe Resistance, and Ground Anchors in a Soil-structure Interaction Analysis

TABLE 14
Reference Deflections for R-y Curves (after Kim and Briaud, 1994)

SOIL TYPE		y_a (in)	y_p (in)
Sand		0.05	0.5
Clay (s_u is the undrained shear strength)	s_u < 4 ksf	0.20	1.0
	4 ksf < s_u < 8 ksf	0.15	0.8
	s_u > 8 ksf	0.12	0.4



① Select Reference Deflections from Table 7.1

② Complete Active Resistance

Sand

$K_a \sigma'_v b$

K_a = Rankine active earth pressure coefficient
σ'_v = Effective stress w/ respect to bottom of excavation.
b = Soldier beam width

Clay

$(\gamma z - s_u)b$ (If negative, use zero)

γ = Total unit weight
z = Depth from bottom of the excavation
s_u = Undrained shear strength

③ Compute Passive Resistance

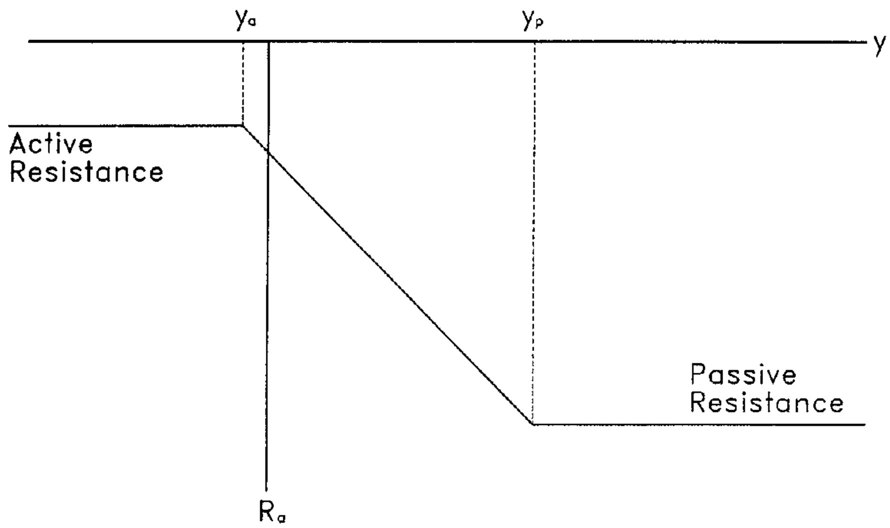
Sand

Equations [6.13],[6.15],[6.16] or [6.17]

Clay

Equations [6.19],[6.21],[6.23] or [6.24]

FIGURE 57
Construction of an R_p-y Curve



① Select Reference Deflections from Table 7.1

② Complete Active Resistance

Sand

$K_a \sigma'_v b$

K_a = Rankine active earth pressure coefficient

σ'_v = Effective stress w/ respect to ground surface.

b = Soldier beam width

Clay

$(\gamma z - s_u)b$ (If negative, use zero)

γ = Total unit weight

z = Depth from ground surface

s_u = Undrained shear strength

③ Compute Passive Resistance

Sand

Equations [6.13],[6.15],[6.16] or [6.17]

Clay

Equations [6.19],[6.21],[6.23] or [6.24]

FIGURE 58
Construction of an R_a-y Curve

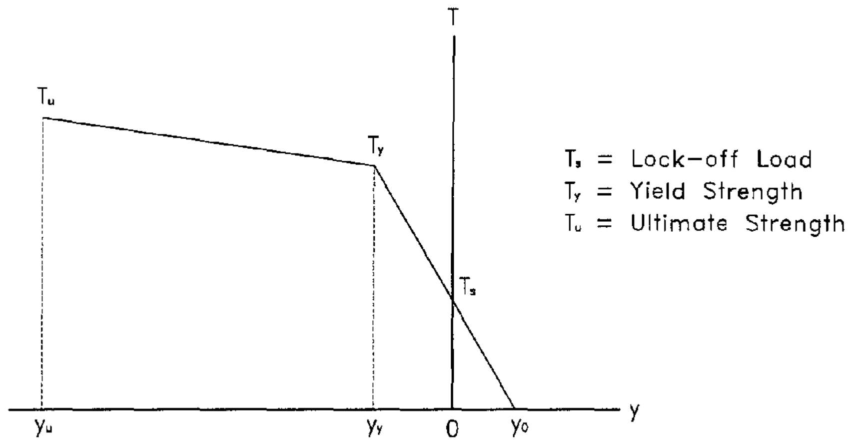
7.3 MODELING THE GROUND ANCHORS

T-y curves are used to model ground anchors after they have been stressed and locked off. Figure 59 shows how a non-linear T-y curve for a soil-structure interaction analysis is developed. Since ground anchors are installed at an angle, the horizontal components of anchor load and tendon elongations are used in developing the T-y curves. The soil-structure interaction analysis described here assumes that the complete structure is "wished" into the ground, and then released to come to equilibrium. The lock-off load is the starting point for the ground anchor in the analysis, and the deflection associated with the lock-off load is zero wall deflection. If the wall moves out, the ground anchor load will increase, and if the wall moves back into the soil, the lock-off load will decrease. High ground anchor loads will move the wall back into the ground. Low anchor loads will result in the wall deflecting outward until the ground anchor load increases. The initial slope of the T-y curve is the horizontal component of the anchor tendon stiffness and it is given by Equation 7.1. In Equation 7.1, the effective unbonded length of the anchor tendon is assumed to be the sum of the unbonded length plus half the tendon bond length. This value is assumed to permit the T-y curve to be constructed, but the actual elastic behavior of the ground anchor will be different. Bending moments are not sensitive to changes in elastic length. If the ground anchor load changes during the analysis, wall deflections will vary depending upon the unbonded length used to construct the T-y curves. At the yield load, the T-y curve changes slope. The second portion of the anchor curve represents the ground anchor behavior between the yield and ultimate tendon strength.

$$k = \frac{A_s E_s}{L_u} \cos\alpha \qquad \text{. . . [7.1]}$$

where

k = anchor tendon stiffness
A_S = area of anchor tendon
E_s = Young's modulus for anchor tendon
L_u = effective unbonded length
α = anchor angle



Deflections

y_o = wall deflection when anchor load = 0

y_s = 0 = wall deflection after anchor stressing (lock-off)

y_y = wall deflection when anchor tendon yields

y_u = wall deflection when anchor tendon ruptures

(α = anchor angle the horizontal)

$y_o + |y_y|$ = horizontal component of tendon elongation @ yield strength $= \dfrac{f_{yield}\, L_u}{A_s\, E_s} \cos\alpha$

(f_{yield} = yield strength of the anchor tendon)

(L_u = effective elastic length of the anchor tendon)

(A_s = tendon area)

(E_s = Young's Modulus of the tendon)

$y_o + |y_u|$ = horizontal component of tendon elongation @ ultimate strength $= L_u\, \epsilon_{rupt} \cos\alpha$

(ϵ_{rupt} = rupture strain)

FIGURE 59
Ground Anchor *T*-*y* Curve

Prestressing steel properties needed to develop T-y curves for ground anchors are given in Table 15.

TABLE 15
Prestressing Steel Properties Required for T-y Curves

TENDON TYPE	MINIMUM YIELD STRESS (ksi)	MINIMUM RUPTURE STRAIN (%)	TYPICAL YOUNG'S MODULUS (ksi)
ASTM A-722 Deformed Bars, Grade 150 ksi	120.0	4.0	30,400
ASTM A-416 Strands, Grade 270 ksi	229.5	3.5	29,400

7.4 EXAMPLE OF ANALYSIS FOR AN ANCHORED SOLDIER BEAM WALL

Soil-structure interaction computer programs require that a structure be determined and the relationships between the spring resistances and the wall deflections be established at each location along the toe so the analysis can be performed. Table 16 summarizes a two-tier, drilled-in soldier beam design for a wall in a loose to medium dense, silty sand with a friction angle of 29° and a unit weight of 108 pcf. The modified trapezoidal apparent earth pressure diagram with a maximum lateral load of 8.86 kips/lf was used in the analysis.

CBEAMC (Dawkins, 1994) used deflections and loads to define the non-linear springs for the T-y, R_p-y and R_a-y curves. A spreadsheet was used to prepare the data for the T-y curves, and Table 17 contains the data that CBEAMC require to develop the non-linear springs for the two ground anchors. Non-linear soil springs were developed from the passive resistance calculations presented in Chapter 6. The spreadsheet shown in Figure 60 contains the ultimate passive resistance calculations for a 30-ft-high wall with the soldier beams 10 ft on center. Passive resistances used to develop the R_p-y and R_a-y curves were taken directly from the spreadsheet. The passive resistances from 0 to 8 ft were the passive resistance for the R_p-y curves. Passive resistances from 30 to 38 ft were the passive resistances for the R_a-y curves. The active resistances for the R_p-y curves were given by the relationship, $K_a\gamma d$, where d ranged from 0 to 8 ft. Active resistances for the R_a-y curves were computed using the relationship, $K_a\gamma(H+d)$, where H = 30 ft and d ranged from zero to 8 ft. Table 18 contains the deflections and the loads used to develop the soil springs on each side of the soldier beam toe for the CBEAMC analysis. In Table 18, the reference deflections y_{a2} and y_{p2} and the loads R_{a2} and R_{p2} were selected to ensure that the solution would converge if the deflections exceeded y_{a1} and y_{p1}.

TABLE 16
Summary of Two-tier, Drilled-in Soldier Beam Design for Example of Soil-structure Interaction Analysis

Wall Height		30 ft
Soldier Beam Size		2 C12×30, Grade 50
Soldier Beam Length		38 ft
Soldier Beam Spacing		10 ft on Center
Drill Hole Size		24 in
Backfill		Lean Mix
Upper Anchor	Tendon	1.25", Grade 150 Bar
	H1	7.5 ft
	Total Length	39 ft
	Unbonded Length	15 ft
	Tendon Bond Length	24 ft
	Design Load	99 kips
	Anchor Angle, α	20°
Lower Anchor	Tendon	1.25", Grade 150 Bar
	H2	11 ft
	Total Length	39 ft
	Unbonded Length	15 ft
	Tendon Bond Length	24 ft
	Design Load	101 kips
	Anchor Angle, α	15°

TABLE 17
Ground Anchor Deflections and Loads for T-y Curves in Example of Soil-structure Analysis

ANCHOR	y_u (in)	y_y (in)	y_s (in)	y_o (in)	T_u (kips)	T_y (kips)	T_s (kips)	T_o (kips)
Upper	-11.39	-0.41	0.00	0.79	176.19	140.95	93.03	0.00
Lower	-11.69	-0.40	0.00	0.83	181.11	144.89	97.56	0.00

uw	108	(pcf)	phi	29	phtn	0.554309	ko	0.52					
ht	30	(ft)	bet	59.5	betn	1.697663	ka	0.347	Toe Reaction =	19.1	(kips)	(from AEP calculations)	
b	1	(ft)	alp	19.33333	altn	0.350848	kp	2.882	Depth of disturbance at toe =	0	(ft)		
s	10	(ft)	btmph	30.500	bmptn	0.589045							
scl	9	(ft)											

Ultimate Passive Resistances and Forces

			Passive Resistance Wedge or Flow Conditions						
Toe Depth (d)	Wing Width	(Eq. 6.14) Intersection Height (hi)	(Eq. 6.13) Wedge Resistance (kips/ft)	Wedge Resistance, d = hi (kips/ft)	Wedge Resistance d = hi, α = 0 (kips/ft)	(Eq. 6.15) Intersecting Wedge Resistance (kips/ft)	(Eq. 6.16) Flow Resistance (kips/ft)	Critical Wedge or Flow Resistance (kips/ft)	(Eq. 6.17) Rankine Passive Resistance (kips/ft)
0	0.00	-7.56	0.00	11.64	2.80	0.00	0.00	0.00	0.00
1	0.60	-6.56	0.56	8.49	1.84	0.56	2.80	0.56	3.11
2	1.19	-5.56	1.60	5.84	1.06	1.60	5.61	1.60	6.23
3	1.79	-4.56	3.14	3.67	0.45	3.14	8.41	3.14	9.34
4	2.38	-3.56	5.17	1.99	0.03	5.17	11.22	5.17	12.45
5	2.98	-2.56	7.69	0.81	-0.21	7.69	14.02	7.69	15.56
6	3.57	-1.56	10.69	0.11	-0.27	10.69	16.82	10.69	18.68
7	4.17	-0.56	14.19	-0.10	-0.14	14.19	19.63	14.19	21.79
8	4.76	0.44	18.18	0.19	0.16	18.15	22.43	18.15	24.90
9	5.36	1.44	22.66	0.96	0.64	22.34	25.24	22.34	28.01
10	5.96	2.44	27.63	2.23	1.30	26.70	28.04	26.70	31.13
11	6.55	3.44	33.09	3.98	2.14	31.25	30.85	30.85	34.24
12	7.15	4.44	39.04	6.23	3.17	35.98	33.65	33.65	37.35
13	7.74	5.44	45.48	8.96	4.37	40.89	36.45	36.45	40.46
14	8.34	6.44	52.41	12.19	5.75	45.98	39.26	39.26	43.58
15	8.93	7.44	59.83	15.91	7.32	51.25	42.06	42.06	46.69
16	9.53	8.44	67.75	20.11	9.06	56.70	44.87	44.87	49.80
17	10.13	9.44	76.15	24.81	10.99	62.33	47.67	47.67	52.91
18	10.72	10.44	85.04	30.00	13.09	68.14	50.47	50.47	56.03
19	11.32	11.44	94.42	35.68	15.38	74.13	53.28	53.28	59.14
20	11.91	12.44	104.30	41.85	17.85	80.30	56.08	56.08	62.25
21	12.51	13.44	114.66	48.50	20.50	86.65	58.89	58.89	65.37
22	13.10	14.44	125.51	55.65	23.32	93.18	61.69	61.69	68.48
23	13.70	15.44	136.86	63.29	26.33	99.90	64.50	64.50	71.59
24	14.29	16.44	148.69	71.42	29.52	106.79	67.30	67.30	74.70
25	14.89	17.44	161.02	80.04	32.89	113.86	70.10	70.10	77.82
26	15.49	18.44	173.83	89.15	36.44	121.12	72.91	72.91	80.93
27	16.08	19.44	187.14	98.76	40.17	128.55	75.71	75.71	84.04
28	16.68	20.44	200.94	108.85	44.08	136.17	78.52	78.52	87.15
29	17.27	21.44	215.22	119.43	48.17	143.96	81.32	81.32	90.27
30	17.87	22.44	230.00	130.50	52.44	151.94	84.12	84.12	93.38
31	18.46	23.44	245.27	142.06	56.89	160.10	86.93	86.93	96.49
32	19.06	24.44	261.02	154.12	61.53	168.43	89.73	89.73	99.60
33	19.66	25.44	277.27	166.66	66.34	176.95	92.54	92.54	102.72
34	20.25	26.44	294.01	179.69	71.33	185.65	95.34	95.34	105.83
35	20.85	27.44	311.24	193.22	76.50	194.53	98.14	98.14	108.94
36	21.44	28.44	328.96	207.23	81.86	203.59	100.95	100.95	112.05
37	22.04	29.44	347.17	221.74	87.39	212.82	103.75	103.75	115.17
38	22.63	30.44	365.87	236.73	93.11	222.24	106.56	106.56	118.28

		Passive Capacity					
Passive Resistance (kips/ft)	Failure Mode	Total Passive Force at Given Toe Depth (kips)	Total Active Force on Toe at Given Toe Depth (kips)	Allowance for Disturbance (Eq.6.12 for depth) (kips)	Net Passive Force at a Given Depth (kips)	Factor of Safety	Toe Depth (ft)
0.00	Wedge	0.00	1.12	0	-1.12	-0.06	0
0.56	Wedge	0.28	1.14	0.00	-0.86	-0.05	1
1.60	Wedge	1.36	2.32	0.00	-0.97	-0.05	2
3.14	Wedge	3.73	3.54	0.00	0.19	0.01	3
5.17	Wedge	7.88	4.80	0.00	3.09	0.16	4
7.69	Wedge	14.31	6.09	0.00	8.22	0.43	5
10.69	Wedge	23.50	7.42	0.00	16.08	0.84	6
14.19	Wedge	35.94	8.79	0.00	27.16	1.42	7
18.15	Interaction	52.12	10.19	0.00	41.92	2.19	8
22.34	Interaction	72.36	11.64	0.00	60.72	3.18	9
26.70	Interaction	96.88	13.12	0.00	83.77	4.39	10
30.85	Flow	125.66	14.63	0.00	111.02	5.81	11
33.65	Flow	157.90	16.19	0.00	141.71	7.42	12
36.45	Flow	192.96	17.78	0.00	175.17	9.17	13
39.26	Flow	230.81	19.41	0.00	211.40	11.07	14
42.06	Flow	271.47	21.08	0.00	250.39	13.11	15
44.87	Flow	314.94	22.78	0.00	292.15	15.30	16
47.67	Flow	361.20	24.53	0.00	336.68	17.63	17
50.47	Flow	410.28	26.31	0.00	383.97	20.10	18
53.28	Flow	462.15	28.12	0.00	434.03	22.72	19
56.08	Flow	516.83	29.98	0.00	486.85	25.49	20
58.89	Flow	574.32	31.87	0.00	542.45	28.40	21
61.69	Flow	634.61	33.80	0.00	600.81	31.46	22
64.50	Flow	697.70	35.77	0.00	661.93	34.66	23
67.30	Flow	763.60	37.77	0.00	725.82	38.00	24
70.10	Flow	832.30	39.82	0.00	792.48	41.49	25
72.91	Flow	903.80	41.90	0.00	861.91	45.13	26
75.71	Flow	978.11	44.01	0.00	934.10	48.91	27
78.52	Flow	1055.23	46.17	0.00	1009.06	52.83	28
81.32	Flow	1135.15	48.36	0.00	1086.79	56.90	29
84.12	Flow	1217.87	50.59	0.00	1167.28	61.11	30
86.93	Flow	1303.39	52.86	0.00	1250.54	65.47	31
89.73	Flow	1391.72	55.16	0.00	1336.56	69.98	32
92.54	Flow	1482.86	57.50	0.00	1425.36	74.63	33
95.34	Flow	1576.80	59.88	0.00	1516.92	79.42	34
98.14	Flow	1673.54	62.30	0.00	1611.24	84.36	35
100.95	Flow	1773.09	64.75	0.00	1708.33	89.44	36
103.75	Flow	1875.44	67.25	0.00	1808.19	94.67	37
106.56	Flow	1980.59	69.78	0.00	1910.82	100.04	38

FIGURE 60
Spreadsheet for Determining Soil Resistance for R_p-y and R_a-y Curves in Example of Soil-structure Interaction Analysis

TABLE 18
Deflections and Soil Resistances for R_p-y and R_a-y Curves in Example of Soil-structure Interaction Analysis

	DEPTH (ft)	y_{p2} (in)	y_{p1} (in)	y_{a1} (in)	y_{a2} (in)	R_{p2} (kips/lf)	R_{p1} (kips/lf)	R_{a1} (kips/lf)	R_{a2} (kips/lf)
R_p-y Curves	0	-2.5	-0.5	0.05	2.5	0	0	0	0
	1	-2.5	-0.5	0.05	2.5	0.57	0.56	0.04	0.03
	2	-2.5	-0.5	0.05	2.5	1.61	1.60	0.07	0.06
	3	-2.5	-0.5	0.05	2.5	3.15	3.14	0.11	0.10
	4	-2.5	-0.5	0.05	2.5	5.18	5.17	0.15	0.14
	5	-2.5	-0.5	0.05	2.5	7.61	7.69	0.19	0.18
	6	-2.5	-0.5	0.05	2.5	10.70	10.69	0.22	0.21
	7	-2.5	-0.5	0.05	2.5	14.20	14.19	0.26	0.25
	8	-2.5	-0.5	0.05	2.5	18.16	18.15	0.30	0.29
	DEPTH (ft)	y_{a2} (in)	y_{a1} (in)	y_{p1} (in)	y_{p2} (in)	R_{a2} (kips/lf)	R_{a1} (kips/lf)	R_{p1} (kips/lf)	R_{p2} (kips/lf)
R_a-y Curves	0	-2.5	-0.05	0.5	2.5	-1.11	-1.12	-84.12	-84.13
	1	-2.5	-0.05	0.5	2.5	-1.15	-1.16	-86.93	-86.94
	2	-2.5	-0.05	0.5	2.5	-1.19	-1.20	-89.73	-89.74
	3	-2.5	-0.05	0.5	2.5	-1.23	-1.24	-92.54	-92.55
	4	-2.5	-0.05	0.5	2.5	-1.26	-1.27	-95.34	-95.35
	5	-2.5	-0.05	0.5	2.5	-1.30	-1.31	-98.14	-98.15
	6	-2.5	-0.05	0.5	2.5	-1.34	-1.35	-100.95	-100.96
	7	-2.5	-0.05	0.5	2.5	-1.38	-1.39	-103.75	-103.76
	8	-2.5	-0.05	0.5	2.5	-1.41	-1.42	-106.56	-106.57

Lateral earth load, shear, and bending moment diagrams from the CBEAMC output are plotted in Figure 61. The soil-structure analysis allowed the wall to be modeled as a continuous vertical member and it computed the bending moments over the full height of the wall. However, it may not reflect reality, and it may not be as good a design tool as apparent earth pressure methods. Table 19 compares the results from the hand calculation and the CBEAMC solution. The tributary area method was used to calculate the ground anchor loads, bending moments, and the toe reactions for the hand solution. Figure 20 describes how the anchor loads, bending moments, and subgrade reactions are calculated using the tributary area method. The ground anchor loads from the CBEAMC analysis are equal to the change in shear at the anchor locations. The toe reaction for the CBEAMC analysis is the shear in the soldier beam at the bottom of the excavation. Bending moments from the computer analysis are shown in Figure 61.

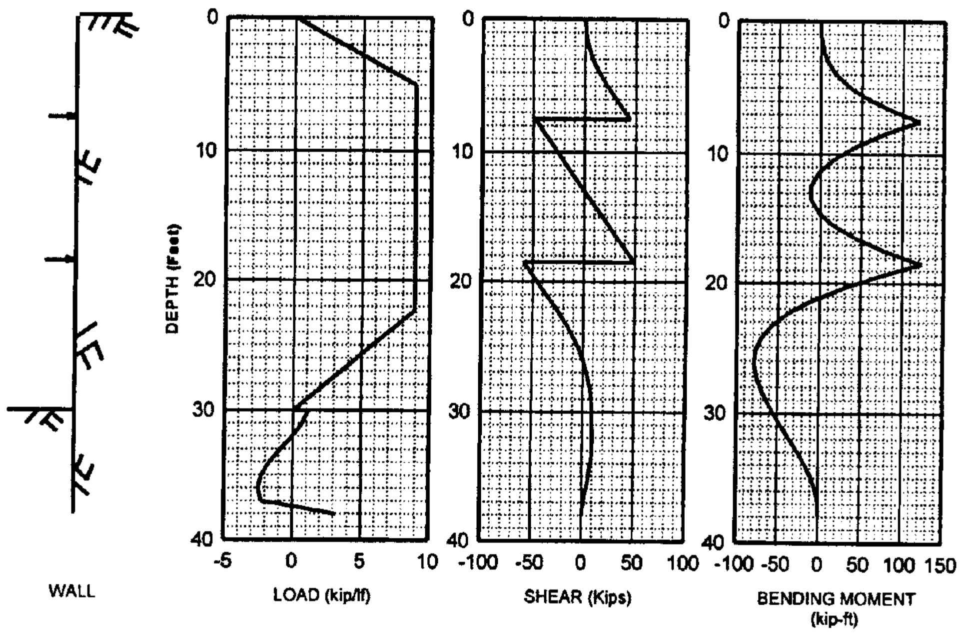


FIGURE 61
CBEAMC Output for Example of Two-tier, 30-ft-high Wall in Loose to Medium-dense Sand

TABLE 19
Comparison of CBEAMC Results and Hand Calculation Results for Example of Wall

	HAND CALCULATION	CBEAMC ANALYSIS
Upper Anchor Load (kips)	93.03	92.76
Moment @ Upper Anchor (k-ft)	119.98	120.00
Maximum Moment Between Anchors (k-ft)	107.21	11.56
Lower Anchor Load (kips)	97.56	107.86
Moment @ Lower Anchor (k-ft)	117.17	123.00
Maximum Moment Between Lower Anchor and Bottom of the Wall (k-ft)	117.17	78.73
Toe Reaction (kips)	19.10	9.07
Passive Resistance Safety Factor	41.92/19.10 = 2.19	41.92/9.07 = 4.62

Table 19 shows the differences between the hand calculation and the soil-structure interaction analysis. The biggest difference between the hand calculation and the computer analysis is the bending moment between the upper and lower anchors. The locations of the bending moments computed by the tributary area method were assumed. Apparent earth pressure methods do a good job of calculating the magnitude of the bending moments but they are not able to predict the location of the moments below the moment at the upper anchor. The bending moments at the second anchor level are about the same, indicating that the maximum bending moments below the upper ground anchor are about $p_a l^2/10$. The soil-structure interaction analysis predicts a higher bottom anchor load and mobilizes lower passive resistance from the toe than the hand solution.

When using a soil-structure interaction analysis as described in this manual, wall deflections can be examined to determine if the analysis seems reasonable. Deflections were 0.415 in at the cantilever, -0.002 in at the upper anchor, 0.087 in at the lower anchor, and 0.442 in at subgrade. These values are reasonable. Deflections at the ground anchor locations should be small. Deflections at subgrade will depend upon the span between the lowest ground anchor and the bottom of the excavation, EI of the beam, and stiffness of the soil springs. Predicted deflections at subgrade should be less than 1.5 in for permanent ground anchor walls. Predicted deflections for temporary walls in soft to medium clays may exceed 6 in if the wall is cantilevered around the lower ground anchor level. Deflection predictions from a soil-structure interaction analysis should not be used to estimate wall deflections. The load-deflection relationships used in these analyses do not consider many of the causes of wall movements.

The soil-structure interaction analysis requires more time than the hand calculation. Table 19 shows that the differences between the two designs do not warrant taking the time to do a computer analysis for most projects. The computer analysis may be warranted when water is behind the wall, a structural diaphragm wall is used, or the soil is weak and the wall will cantilever around the bottom support. Soil-structure analyses can be run to check that the moments computed using the rules associated with the tributary area method or the simple beam method are reasonable.

After the analysis is performed, the mobilized passive resistance is compared with the available passive resistance. The factor of safety is the available passive resistance divided by the mobilized passive resistance. A minimum factor of safety of 1.5 shall be obtained. The available passive capacity of the 8-ft-deep toe is 41.9 kips (Figure 60), giving passive resistance factor of safety of 2.19 for the hand solution and 4.62 for the soil-structure interaction analysis. The soil-structure interaction solution was higher since more load was carried by the lower ground anchor.

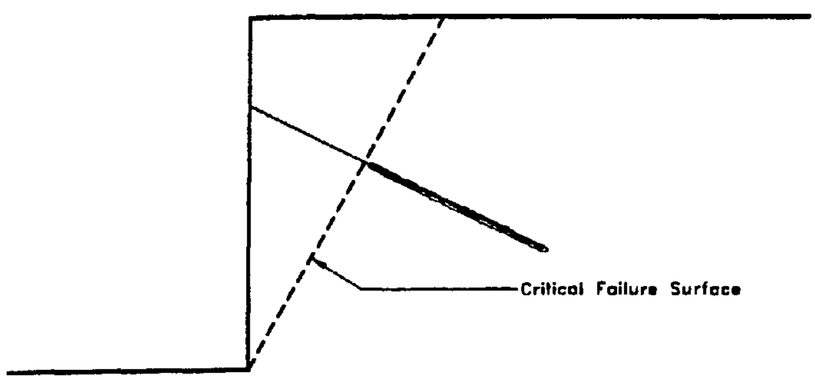
7.5 COMMENTS AND RECOMMENDATIONS

Soil-structure interaction analyses can be done to design permanent and temporary ground anchor walls. These analyses are not necessary for the design of flexible walls constructed with soldier beams and sheet piling. The following comments and recommendations concern the use of soil-structure interaction analyses:

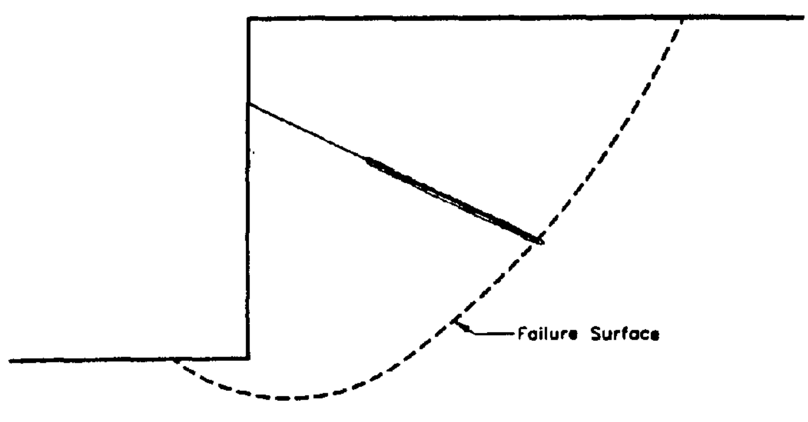
- Use apparent earth pressures for soil-structure interaction analyses for flexible soldier beam and sheet pile walls.
- Use non-linear soil springs to model the lateral resistance of the soil below the bottom of the wall.
- Ultimate lateral resistance at a given location along the toe of the wall is computed using relationships developed by Wang and Reese (1986). These relationships consider four modes of failure and the interaction of adjacent soldier beams.
- Construct the load-deflection curves for the soil-structure interaction analyses, using reference deflections given in Table 14.
- Lateral deformations from soil-structure interaction analyses are not reliable. The load-deflection relationships used in these analyses do not consider many causes of wall movements.
- Apparent earth pressure methods compute similar bending moments and anchor loads as those computed using the soil-structure interaction analyses.
- Apparent earth pressure methods are better than soil-structure interaction methods for flexible walls. Apparent earth pressure methods can be done quickly, and they do not require the selection of a structure to begin the analysis.

CHAPTER 8: ANCHORED WALL STABILITY

Ground anchor walls must be internally and externally stable. Internal stability requires the ground anchors to be located sufficiently behind the wall so that the anchor does not develop load-carrying capacity from the ground supported by the wall. A wall is internally stable when any failure surface that passes between the wall and the top of the anchor bond length will have an adequate factor of safety with the anchor load applied. External stability is satisfied if the ground anchors are long enough so that any failure surface that passes behind the back of the anchor bond zone will have an adequate factor of safety. Internal and external stability are illustrated in Figure 62.



a) Internally stable wall (anchor bond length located behind the critical failure surface)



b) Externally stable wall (anchor extends to or beyond failure surface with adequate *FS*)

FIGURE 62
Internal and External Stability of an Anchor Wall

8.1 INTERNAL STABILITY

A ground anchor wall is internally stable when the anchor bond length is located behind all the failure surfaces that would develop if the wall moved outward. The deepest failure surface is called the critical failure surface. A Rankine failure surface passing through the bottom of the wall and extending upward at an angle of $45 + \phi/2$ is commonly used as the critical failure surface in soils. In sandy ground, the failure surface is inclined upward at an angle between 57° and 68° for most soils. In clays, the failure surface is inclined upward at 45°. If the clay has been sheared, the orientation of the shear zones may determine the location of the critical failure surface. In rock, the critical failure surface is defined by structural features or discontinuities.

Limiting equilibrium was used to create Figure 63, a plot showing the critical failure surfaces for walls with a horizontal back slope in soils having friction angles between 25° and 45°. Distances in Figure 63 are expressed in terms of the excavation height, H. Therefore, the plot can be used to find the critical failure surface for any wall with a horizontal back slope. Critical failure surfaces in the figure were developed by applying a uniform horizontal load to the face of the wall and varying the magnitude of the load until a failure surface with a factor of safety of 1.0 was identified. Failure surfaces for high strength soils coincide with the Rankine failure surface and essentially go through the bottom corner of the excavation. As the soil shear strength decreases, the failure surface drops below the bottom corner of the excavation and the failure surface moves back behind the wall.

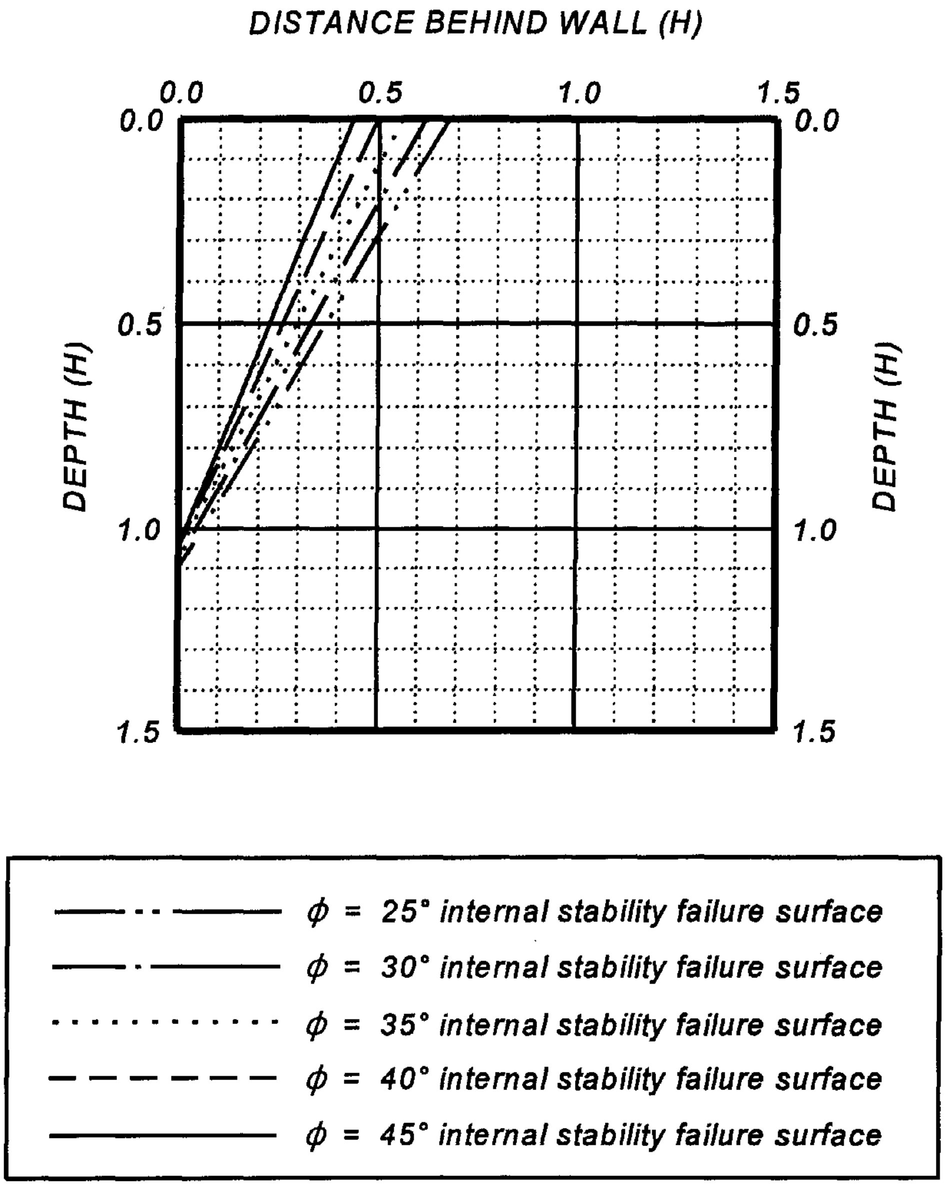


FIGURE 63
Internal Stability Curves for Coarse-grained Soils
(scale: 2 in = *H*)

Deformations and stresses behind a wall in stiff clay correspond to a quasielastic state instead of a limiting equilibrium state. Current practice is to locate the ground anchor bond length behind a plane inclined upward from the bottom of the excavation at an angle of 45°. Figure 64 shows the critical failure surface for a stiff clay.

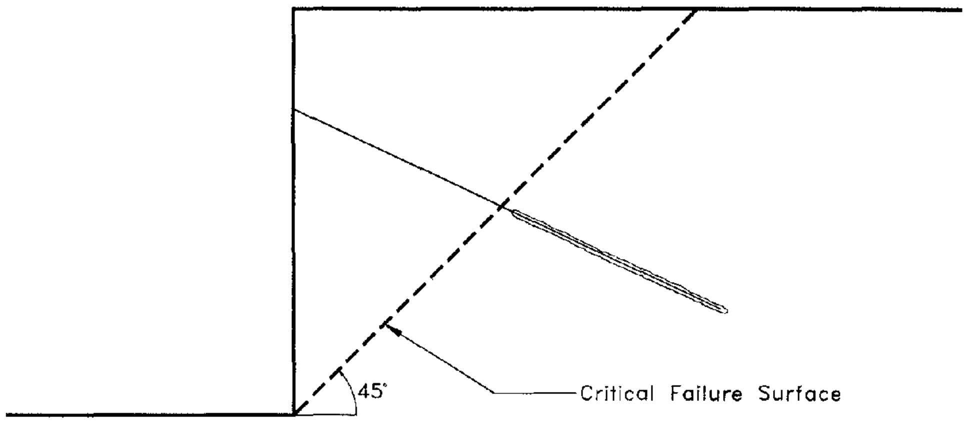


FIGURE 64
Critical Failure Surface for Clay

Deformations and stresses behind a wall in soft to medium clay correspond to a state of limiting equilibrium. If the soft to medium clay does not extend below the bottom of the excavation, the critical failure plane will be inclined upward from the bottom corner of the excavation at an angle of 45°. The critical failure surface in Figure 64 applies to soft to medium clays when competent ground is at the bottom of the excavation. When the soft to medium clay extends below the bottom of the excavation, determine the location of the critical failure surface using limiting equilibrium methods. Figure 65 shows the critical failure surface determined using a limiting equilibrium analysis for a deep, weak clay deposit. Ground anchor load-carrying capacity must be developed behind the critical failure surface.

Ground anchors are frequently used to prevent or correct landslides. For these applications the unbonded tendon length must extend into the stable ground below the potential or existing sliding surface. Usually, general purpose slope stability computer programs are used to determine the ground anchor loads and to define the critical failure surface.

In mixed grounds or when groundwater may affect the stability of the wall, locate the critical failure surface using a general purpose slope stability program. Computer programs can model these conditions satisfactorily. When the groundwater table is behind the wall or near the bottom of the wall, the critical failure surface may move away from the wall (Long, et al., 1998).

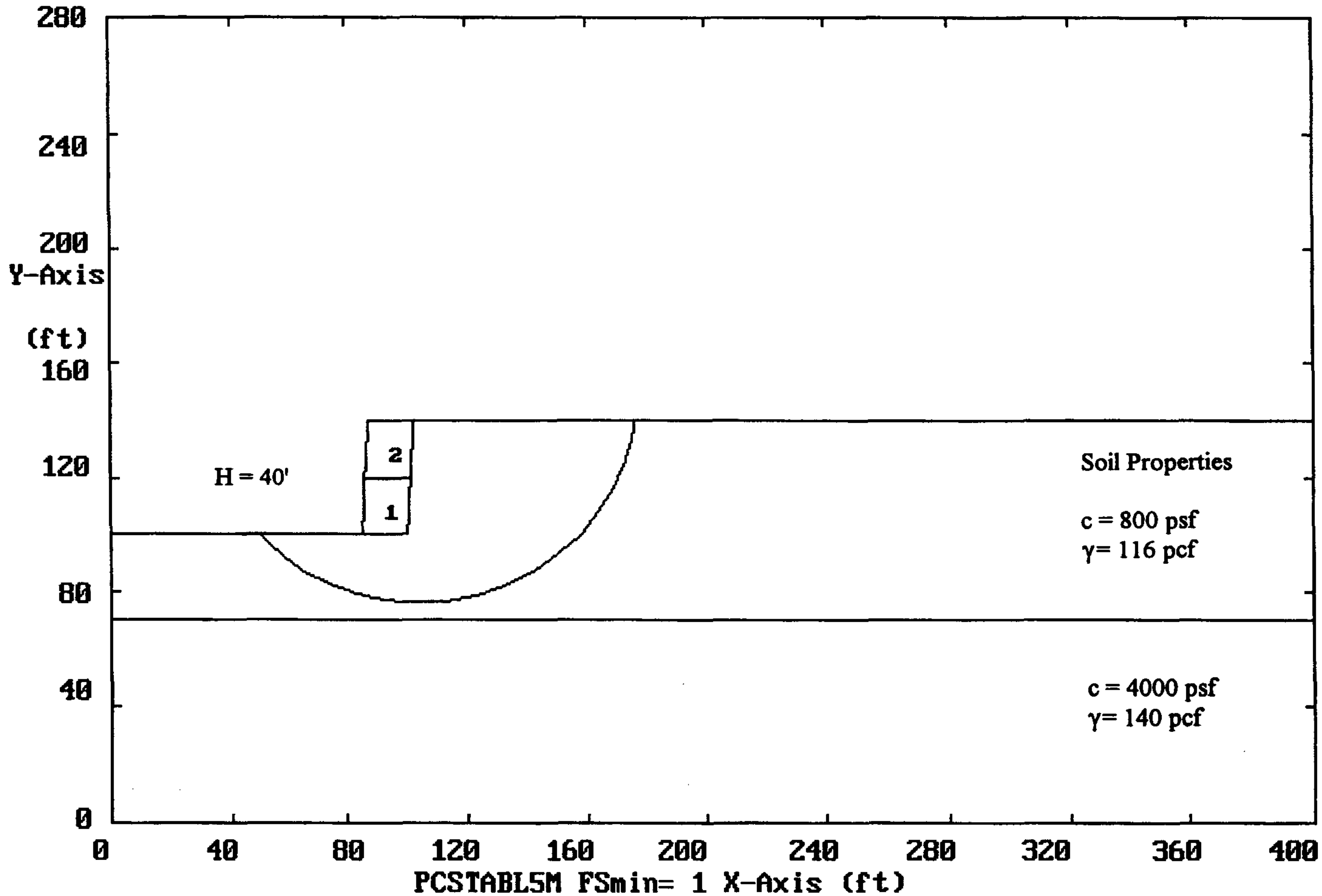


FIGURE 65
Results of Limiting Equilibrium Analysis for Determining the Critical Failure Surface, Deep-seated Failure

The unbonded length of the ground anchor tendon must be long enough to ensure that the anchor will develop its load-carrying capacity behind the critical failure surface. To prevent load transfer above the critical failure surface, some ground anchor standards require that the tendon unbonded length extend a minimum of 5 ft behind the critical failure surface. When the soils at the bottom of the excavation are poor, the actual critical failure surface may extend below the bottom corner of the excavation. To account for a deeper critical failure surface, some standards require that the unbonded length extend a distance of $0.2H$ below the critical failure surface that passes through the bottom corner of the wall. FHWA's practice (Cheney, 1988) has been to use the minimum of the two distances. If the critical failure surface is determined by limiting equilibrium methods, checking that the unbonded length extended $0.2H$ beyond the failure surface is not necessary.

For high-pressure grouted anchors, small-diameter anchors, and rock anchors the tendon bond length and the anchor bond length coincide. Here, the unbonded tendon length is selected so the anchor bond length is behind the critical failure surface a minimum of 5 ft (Figure 66). Large-diameter anchors in fine-grained soils require relatively large movements to mobilize load-carrying capacity. If the unbonded length of these anchor tendons only extends 5 ft behind the back of the critical failure surface, load-carrying capacity may be developed along the unbonded length of the anchor in front of the critical failure surface. To prevent this from happening, the anchors are often grouted in two stages. First, the anchor bond length is grouted and then the anchor is tested. After testing, the unbonded length portion of the drill hole is grouted. This procedure is not desirable. It may compromise the corrosion protection provided by the grout, and it has resulted in settlement when the drill holes cave while waiting to test the anchor.

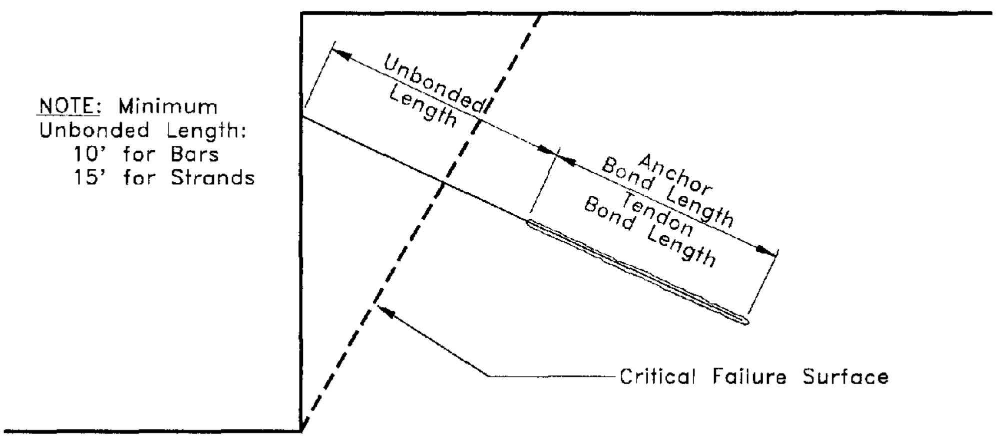


FIGURE 66
Internal Stability Consideration for the Selection of the Unbonded Length of Small-diameter, Pressure Grouted Anchors

Research by Ludwig and Weatherby (1989) and research reported by Mueller, et al. (1998) has shown that large-diameter anchors can be grouted to the surface in one operation, if the unbonded tendon length is extended into the back of the anchor bond length. Extending the unbonded length into the ground anchor bond length separates the anchor bond length from the tendon bond length. Figure 67 shows a ground anchor where the unbonded length was extended into the anchor bond zone. For design purposes, the anchor bond zone is the length of the anchor behind the critical failure surface and the tendon bond length is selected so the load is transferred toward the back of the anchor first. Often the tendon bond length is half the anchor bond length. The minimum tendon bond length is 10 ft for bar tendons and 15 ft for strand tendons (PTI, 1996). Installing large-diameter anchors in this way enables the grout to be placed in one operation, lowers costs, prevents settlement resulting from loss of ground, and improves the tendon corrosion protection.

Besides enabling the ground anchor to develop its load-carrying capacity from behind the critical failure surface, the unbonded length must be long enough to ensure that the anchor can be locked-off properly. A minimum unbonded length is recommended to prevent load losses during seating of the anchorage. The minimum unbonded length for bar tendons is 10 ft, and the minimum unbonded length for strand tendons is 15 ft (PTI, 1996).

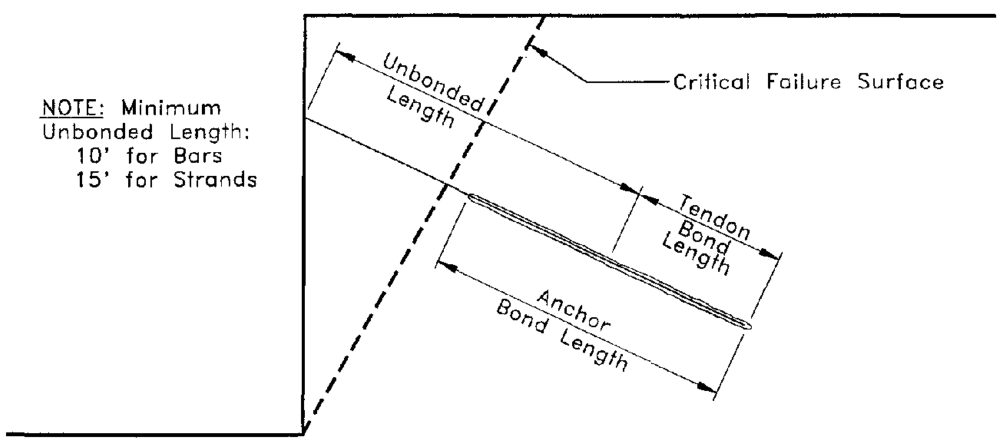


FIGURE 67
Diagram Illustrating the Internal Stability Consideration for Selecting the Unbonded Length of Large-diameter Anchors

8.2 EXTERNAL STABILITY

Anchored walls are externally stable when failure surfaces passing behind the ground anchor have an adequate factor of safety. External stability is sometimes called overall stability. Many ground anchor walls for highway applications are externally stable with 30-ft-long ground anchors (15-ft unbonded length and a 15-ft bond length). External stability is checked for walls constructed in sands and soft to medium clays. External stability is not checked for walls built in stiff clay. Limiting equilibrium analyses will always show that walls in stiff clay are externally stable. External stability is checked if the wall is to be constructed in a stiff clay with shear zones or slickensides oriented in a direction that might affect the stability of the wall. External stability for walls supported by rock anchors is normally adequate. If the rock mass has planes of weakness oriented in a direction that may affect the stability, then the external stability of the wall should be checked.

Limiting equilibrium analyses were used to create Figure 68, a graph containing external stability curves for locating the back of the ground anchors in coarse-grained soils. The curves are for walls having horizontal back slopes in sands having friction angles between 25° and 45°. A factor of safety of 1.3 was applied to the shear strength of the soil. Distances in Figure 68 are expressed in terms of the excavation height, H. To determine if an anchor satisfies external stability, draw the anchor to scale on the graph and ascertain if it intersects or extends beyond the curve for the soil strength. Anchors that extend beyond the curve satisfy external stability. Those that do not intersect the curve must be lengthened. Curves in Figure 68 were developed by doing a series of stability analyses for soils with different friction angles. Failure surfaces were forced to go behind a point representing the back of the ground anchor. The points were at depths of $0.25H$, $0.50H$, $0.75H$, $1.0H$, and $1.25H$ from the ground surface. For each soil strength and depth, the point was moved horizontally in or out until a curve with a factor of safety of 1.3 was obtained. The lateral resistance of the wall was ignored in the analyses.

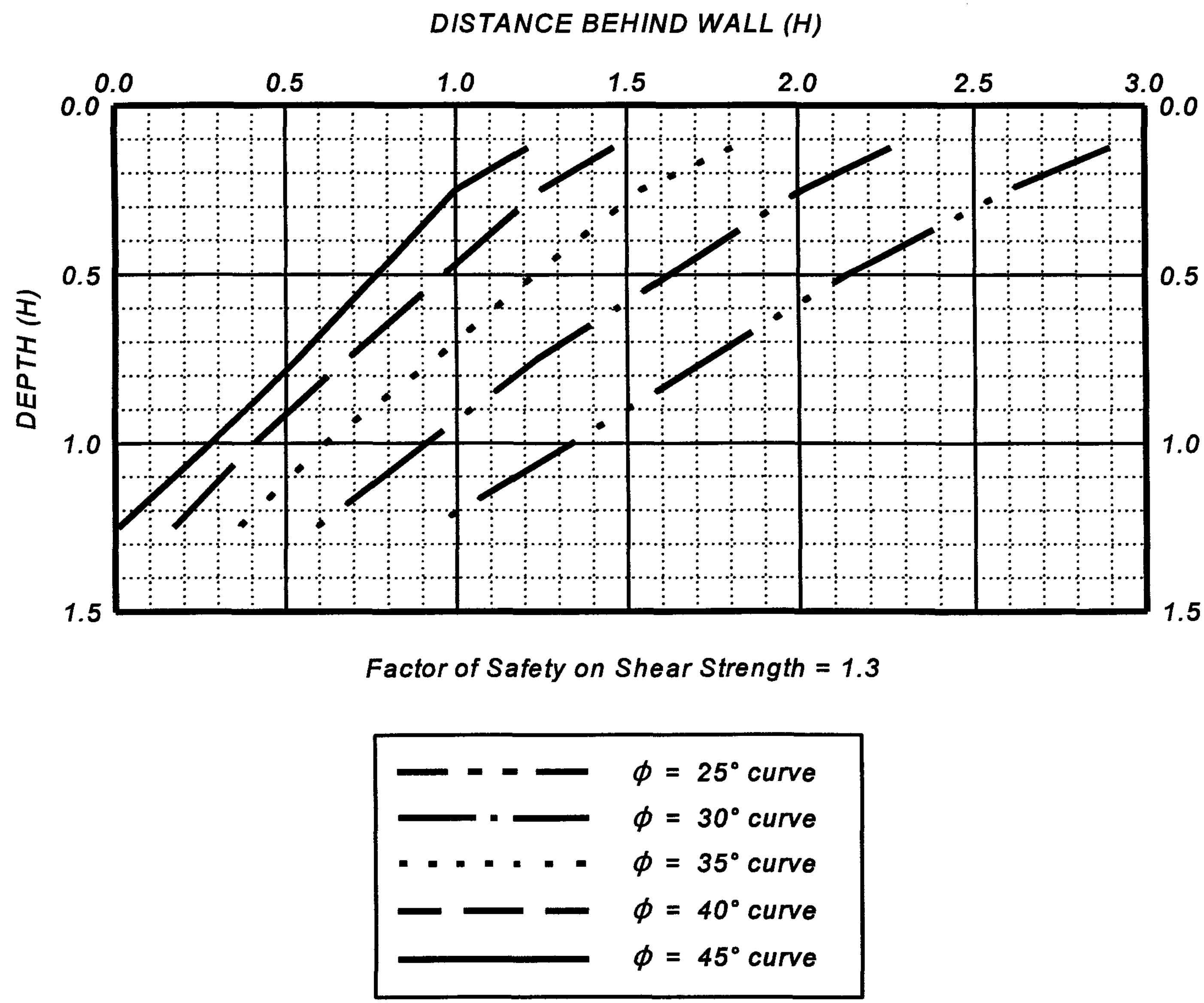


FIGURE 68
External Stability Curves for Coarse-grained Soils (scale: 2 in = *H*)

Deformations and stresses behind a wall in stiff clay correspond to a quasielastic state instead of a limiting equilibrium state. Walls in stiff clay are externally stable unless planes of weakness exist in the deposit. Planes of weakness may be present in heavily overconsolidated clays. Local experience with temporary excavation support systems and open cuts combined with a site-specific geotechnical investigation should identify the presence and orientation of shear zones or slickensides. Laboratory tests and local experience will be used to estimate the shear strength of the soil along the planes of weakness. If the planes of weakness are oriented in a way that may affect the external stability of the wall, then a limiting equilibrium analysis should be done to check wall stability.

External stability of a wall in soft to medium clay should be checked. If the weak clay only extends down to the bottom of the wall, the failure surfaces will start at the bottom corner of the wall. If the weak clay extends below the bottom of the wall, the failure surfaces will go below the bottom of the wall, and the ground anchors may have to be very long to satisfy external stability. Figure 69 shows the external stability failure surface (factor of safety of 1.3) that would develop through the bottom corner of a wall in a weak clay. This analysis shows that the ground anchors must be relatively long or anchored in competent ground to satisfy external stability. Figure 70 shows the external stability failure surface (factor of safety of 1.3) that would develop if the weak clay extends below the bottom of the excavation. In the situation depicted in Figure 70, the ground anchors should be installed into the competent ground. Figures 69 and 70 are at the same scale, and show what happens when the poor ground extends below the bottom of the wall.

In mixed ground or when groundwater may affect wall stability, check external stability using a general purpose slope stability computer program. Longer anchors are usually required if groundwater is present behind the wall. If the ground is primarily sandy, the analysis should use force equilibrium methods and planar failure surfaces. In clayey ground, use circular failure surfaces and moment equilibrium.

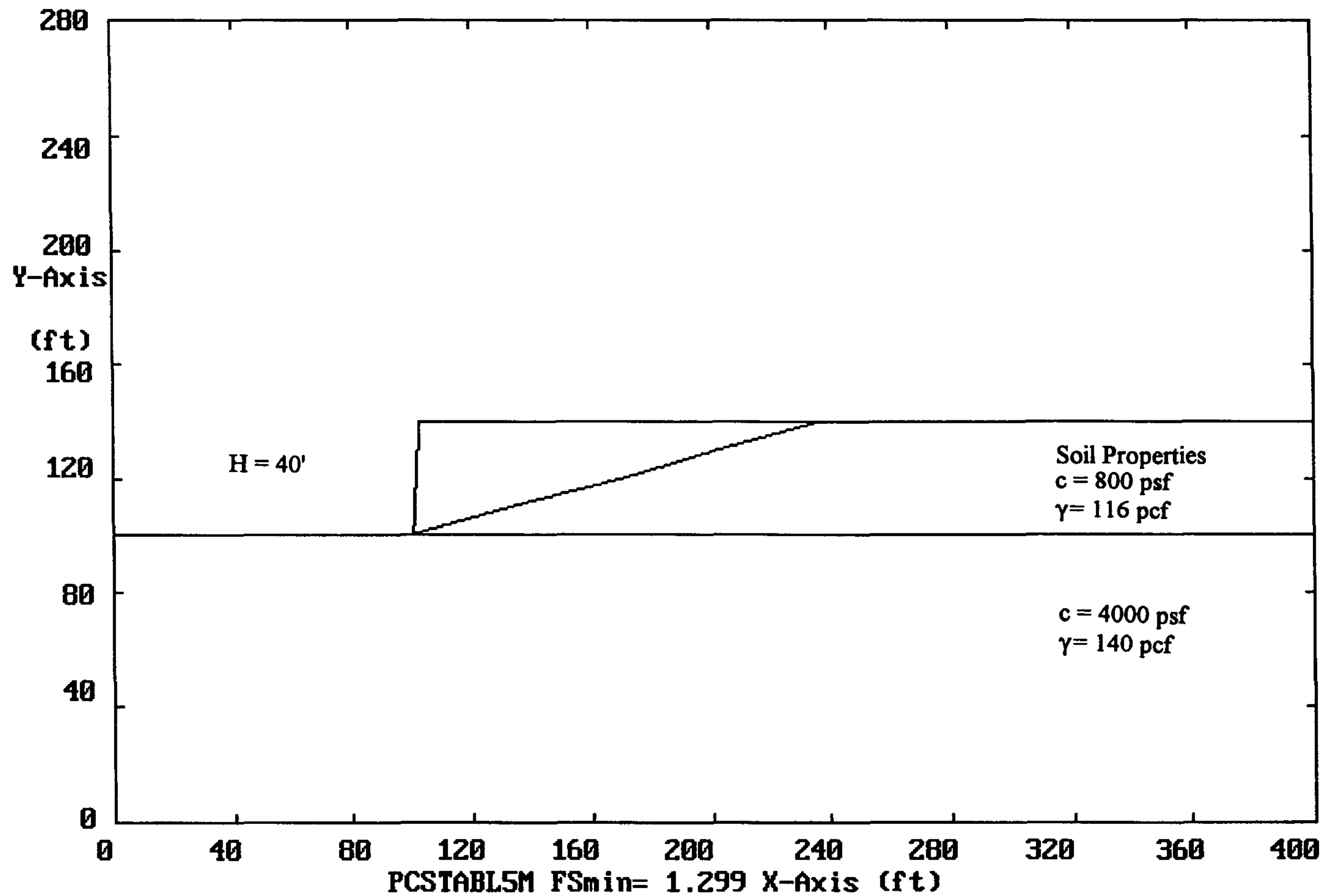


FIGURE 69
Typical External Stability Analysis for a Wall in a Weak Clay Underlain by Competent Clay

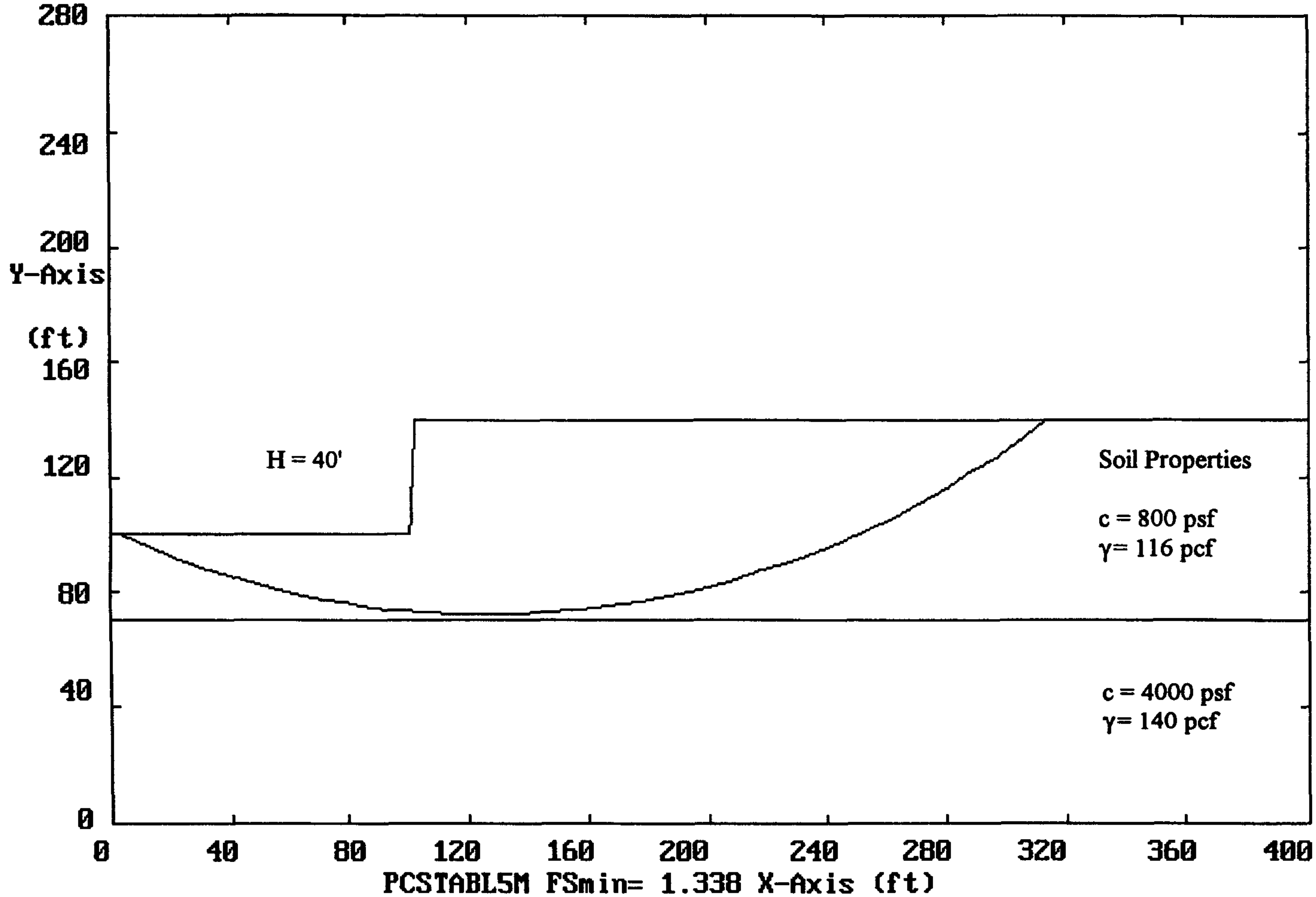


FIGURE 70
Typical External Stability Analysis for a Wall in a Weak Clay, Deep-seated Failure

External stability of walls supported by multiple rows of ground anchors is achieved if the back of each anchor extends beyond the appropriate curve in Figure 68. General purpose slope stability computer programs can be used to check the external stability of a wall support by multiple rows of anchors. Several computer runs will be required. Figure 71 is the graphical output of limit equilibrium analyses done to check the external stability of failure surfaces through the back of a second row of anchors. The lowest factor of safety for these failure surfaces was 1.42, which is adequate. Figure 72 shows failure surfaces passing through the back of the upper row of ground anchors. The analyses done for Figure 72 ignored the lower ground anchor and indicated that the factor of safety is below the minimum of 1.3. To include capacity from the lower ground anchor in the analyses of failure surfaces through the back of the upper ground anchor, determine the lower anchor capacity developed behind the failure surface. Then apply that load to the wall as a surcharge load and check the factor of safety. Figure 73 shows that, when the effect of the lower anchor is included, the factor of safety increases from 1.163 to 1.351.

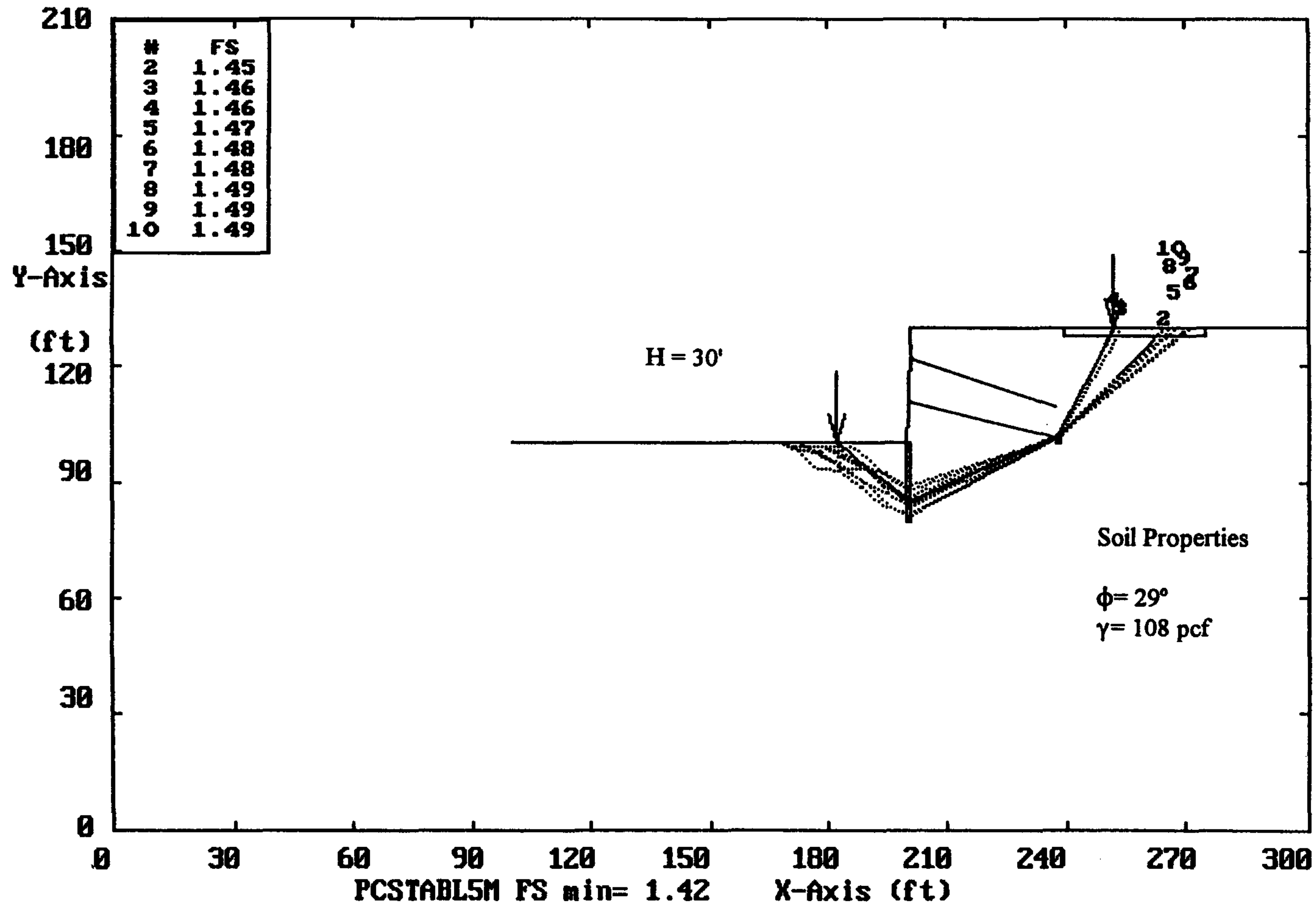


FIGURE 71
Example of External Stability Analysis, Failure Surface Through Back of Lower Anchors

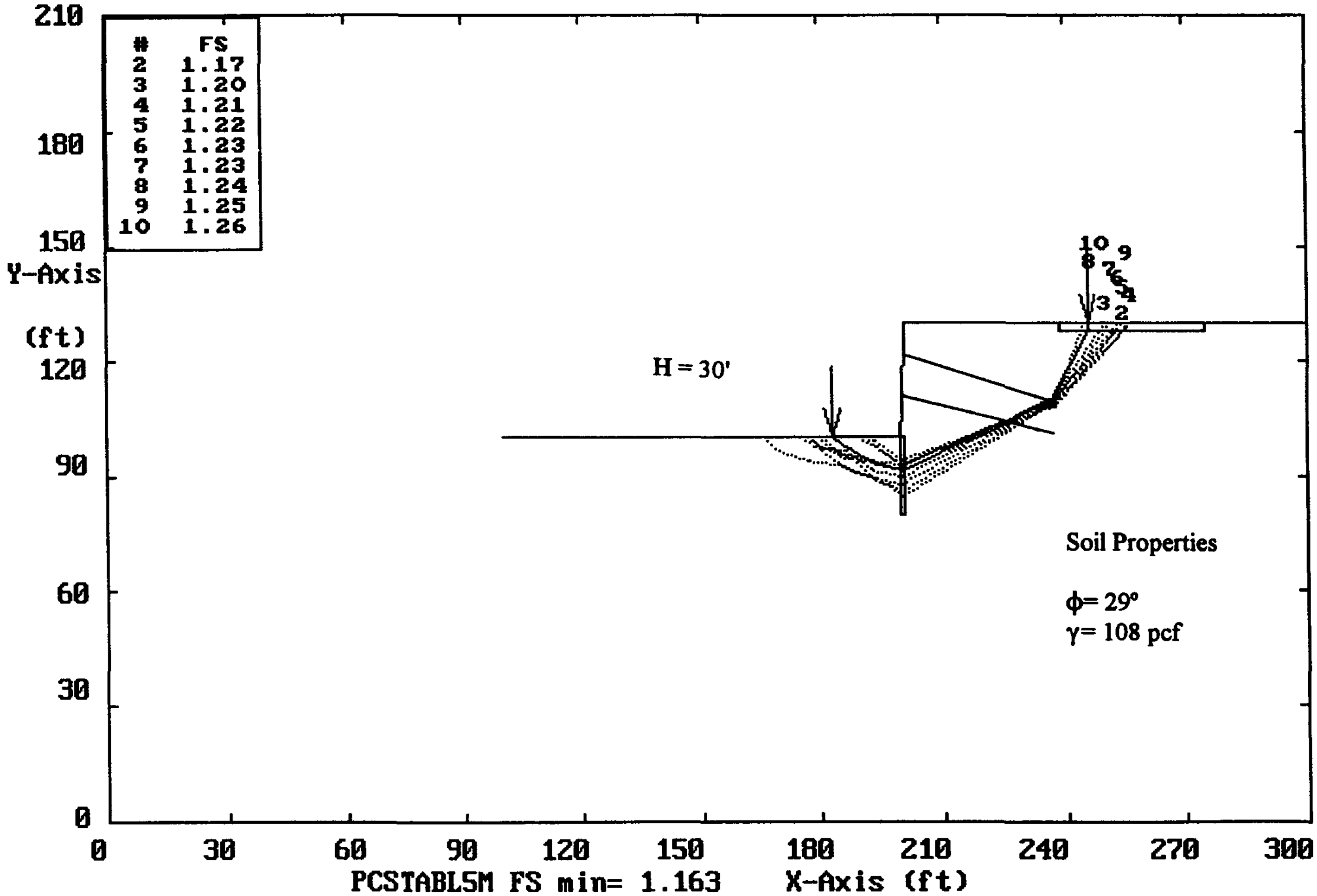


FIGURE 72
Example of External Stability Analysis, Failure Surface Through Back of Upper Anchor, Assuming Lower Anchor Not Installed

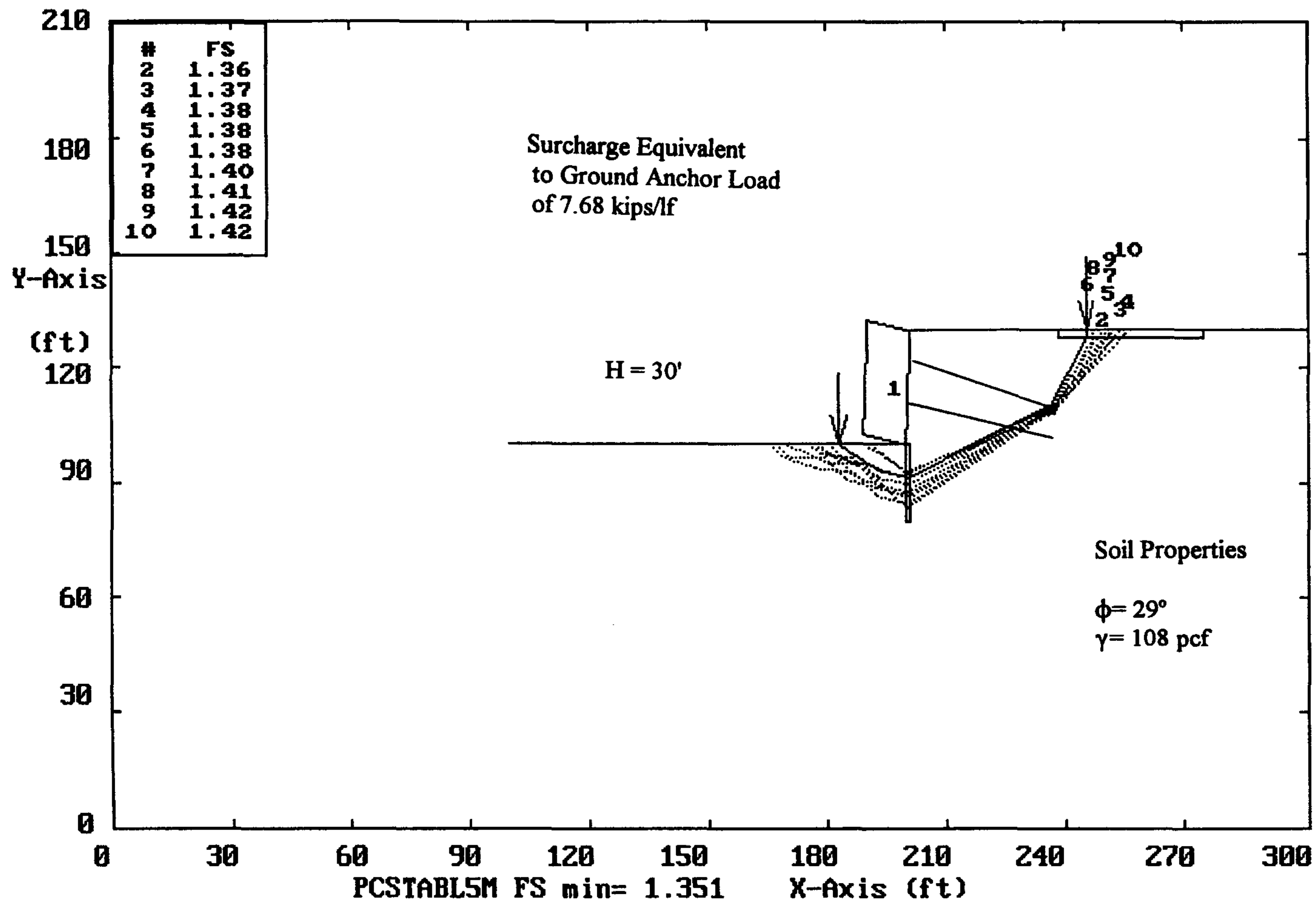


FIGURE 73
Example of External Stability Analysis, Portion of Lower Ground Anchor Load Considered as a Horizontal Surcharge in Analysis of Failure Surface Through Back of Upper Anchor

8.3 RECOMMENDATIONS

Internal and external stability of anchored walls should be checked. On most projects it is only necessary to check one or two critical sections. These sections typically are the highest wall sections.

Figure 63 and 68 are combined to form Figure 74. Figure 74 can be used to check the internal and external stability of walls with horizontal back slopes in course-grained soils. To use Figure 74, plot the ground anchor to scale and select a total anchor length that extends beyond the external stability curve appropriate for the soil at the site. Then check to ensure that the anchor bond length lies behind the critical failure surface. Lengthen the anchor if the anchor bond length is not adequate to develop the required ground anchor load-carrying capacity. Using Figure 74 will provide a factor of safety against an external stability failure of 1.3. Similar plots can be created for different factors of safety or sloping back slopes.

If the wall extends below the bottom of the external stability failure surface, the passive resistance of the wall can be incorporated into the analysis. In general purpose slope stability computer programs, the passive resistance of the wall can be modeled as a small element with a high cohesion. The cohesion used in the analysis will be the smallest of the shear resistance of the wall or the passive capacity of the wall below the failure surface.

When evaluating the internal stability of a ground anchor wall using limiting equilibrium methods, follow the guidelines for modeling the ground anchor loads in Section 4.3. Use the horizontal component of the ground anchor load if the wall extends below the critical failure surface. If the wall does not extend below the critical failure surface, use the total ground anchor load in the analysis.

When using general purpose slope stability computer programs, use planar failure surfaces and force equilibrium methods for sandy soils and circular failure surfaces and moment equilibrium methods in clayey soils.

External stability analyses assume that the ground anchors will develop load-carrying capacity uniformly along bond length. In most ground this is a reasonable assumption. However, in ground that becomes much weaker with depth, the ground anchor may develop most of its load-carrying capacity near the front of the anchor bond length. This anchor could test satisfactorily, but with respect to external stability, it may act like a shorter anchor. Large lateral movements have occurred on temporary excavation support systems where the ground becomes weaker with depth. These movements have not been attributed to this mechanism, but it may explain why they have occurred. In this type of ground, extend the unbonded length into the anchor bond length, and transfer the load to the back of the anchor first. When weak soil underlies good ground and the anchor will develop load-carrying capacity from both layers, design the ground anchors assuming that they will develop their capacity in the poorer soil.

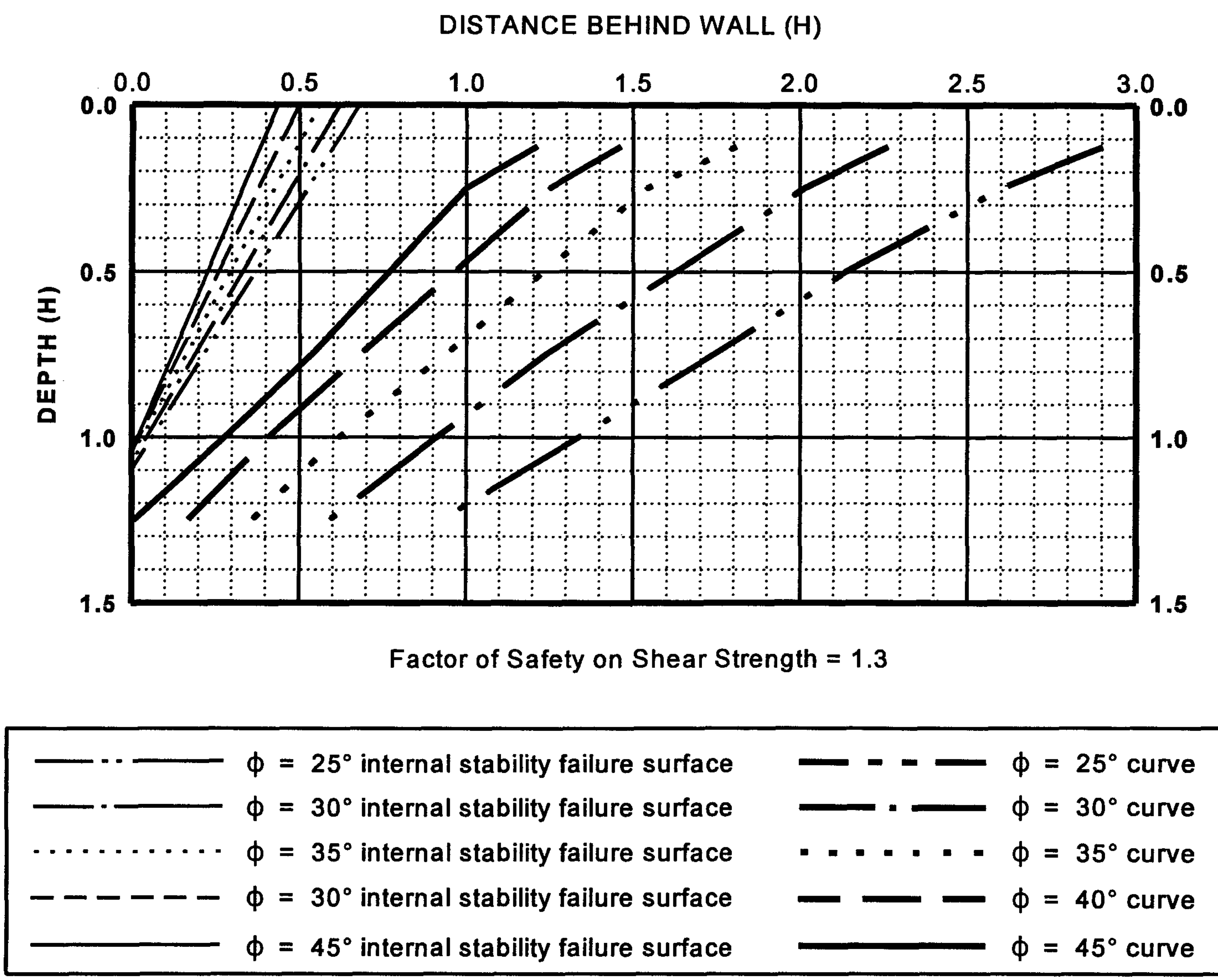


FIGURE 74
Internal and External Stability Curves for Coarse-grained Soils
(scale: 2 in = *H*)

CHAPTER 9: SPECIAL DESIGN CONSIDERATIONS AND LOADS

Besides designing the wall to resist apparent earth pressures and vertical loads, ground anchor wall design may include:

- Estimating wall and ground movements.
- Corrosion protection requirements for the anchor tendon and the soldier beam.
- Prevention of frost loads.
- Landslide loads.
- Surcharge loads.
- Barrier loads.
- Seismic design.
- Facing design.
- Construction stages.

9.1 WALL AND GROUND MOVEMENTS

Lateral wall movements and ground surface settlements behind permanent ground anchor walls will be small. These walls will be constructed in competent ground, and structures will not be nearby. Typically, the maximum lateral wall movements will be about $0.002H$, and maximum vertical soil settlements will be about $0.0015H$, where H is the height of the wall.

Lateral movements can result from bending deformations (cantilever movements and lateral bulging), outward rotation about the toe of the wall, and translation of the wall. Settlement behind the wall is a response to the lateral wall movements or consolidation resulting from lowering the groundwater table. Bending deformations depend upon the height of the wall, stiffness of the wall, the distance to the first anchor, the distance between anchor levels, and the strength of the ground. Outward rotation about the toe is directly related to soldier beam settlement. Translation movements may result from mass movements behind the anchors, re-distribution of load along the anchor bond length, anchor yielding, or elastic elongation of the anchor tendon in response to load increases. In a well-designed wall, most of the deformation will be a result of bending deformations. Rotational and translational movements will be small, with rotational movements larger than translational movements.

Wall and ground movement predictions are based on experience. Typical lateral and horizontal movements for flexible retaining walls have been presented by Peck (1969), Goldberg, et al. (1976), and Clough and O'Rourke (1990). Maximum lateral movements in ground suitable for permanent ground anchor walls are generally less than $0.005H$, with average maximum movements of about $0.002H$. The largest lateral movement occurs at the top of the wall. Maximum vertical soil settlements in ground suitable for permanent ground anchor walls are less than 0.005H, with average maximum settlement tending toward 0.0015H. The maximum settlement occurs near the wall. For a 25-ft-high wall, a maximum lateral movement of 0.6 in and a maximum vertical ground settlement of 0.45 in would represent average performance.

Figure 75, from AASHTO's *The Standard Specifications for Highway Bridges* (1996), gives settlement envelopes for flexible walls in different soils. Curves I or II can be used to estimate the maximum settlements for most permanent ground anchor walls. Curve III can be used to estimate settlements behind a wall in soft to medium clay when competent soil is found at the bottom of the wall. In sands, the settlements decrease linearly from a maximum near the wall to a value near zero at a distance twice the height of the wall. In stiff clays, measured settlements extend back approximately three times the height of the wall.

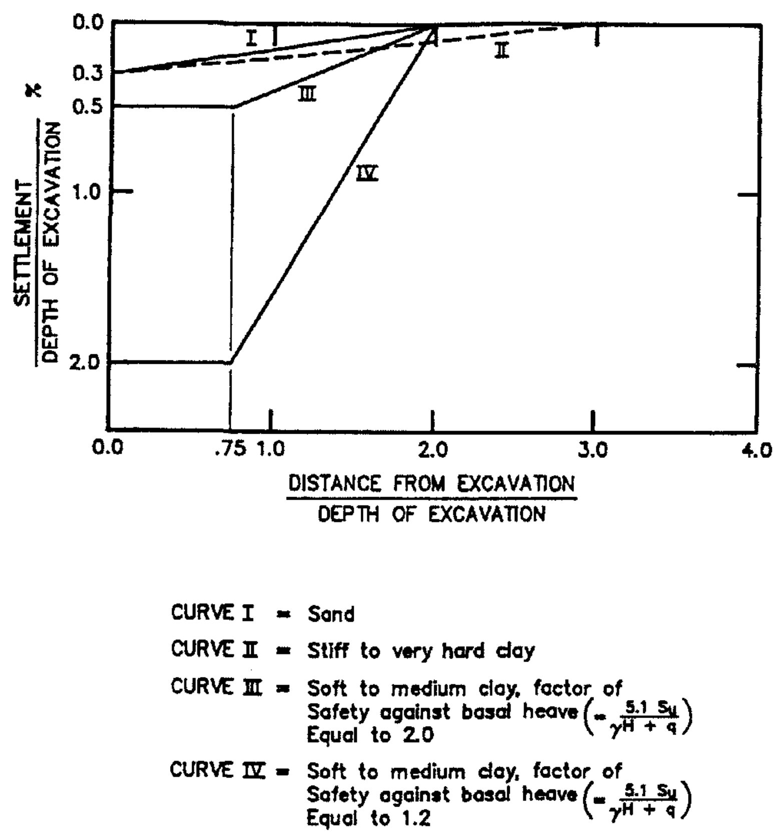


FIGURE 75
Settlement Profile Behind Braced and Anchored Walls
(modified after Clough and O'Rourke, 1990)

Lateral wall movements and ground settlements cannot be eliminated, but they can be reduced by controlling bending deformations and soldier beam settlements. Reducing the distance to the upper ground anchor will reduce the cantilever bending deformations and reducing the span between the ground anchors will reduce the bulging deformations. For flexible walls, the cantilever and bulging deformations can be expressed by Equations 9.1 and 9.2. These relationships were developed by Mueller, et al. (1996).

$$y_c = 4K_o \gamma h_1^2 / E_s \qquad \text{. . . [9.1]}$$

$$y_b = 0.8K_o \gamma h L / E_s \qquad \text{. . . [9.2]}$$

where:

y_c = cantilever deformation
y_b = bulging deformations
K_o = at-rest earth pressure coefficient
γ = total unit weight
h_1 = depth of excavation to allow the installation of the upper ground anchor
E_s = represents a secant modulus on the soil's stress-strain curve (see Table 20)
h = depth of excavation
L = span distance

TABLE 20
Ranges for E_s for Different Soil Types

SOIL		E_s (psi)
Clay	Firm to Stiff	550 - 1150
	Very Stiff	1150 - 2850
Silt		250 - 2850
Loess		2150 - 8550
Fine Sand	Loose	1150 - 1700
	Medium Dense	1700 - 2850
	Dense	2850 - 4250
Sand	Loose	1400 - 4250
	Medium Dense	4250 - 7100
	Dense	7100 - 11400
Gravel	Loose	4250 - 11400
	Medium Dense	11400 - 14200
	Dense	14200 - 28450

The relationships given by Equations 9.1 and 9.2 are for soldier beam walls. When a stiff wall is used, the relationships are not valid. Movements estimated from the equations show trends, and they can be used to evaluate the impact of different ground anchor locations. They represent minimum movements that might be expected. They suggest that cantilever movement varies with the square of the depth of excavation, and bulging deformations are directly related to the distance between the ground anchors or the distance from the lower ground anchor to the bottom of the excavation. For a typical soldier beam wall, where $K_o = 0.4$, $\gamma = 115$ pcf, $h_1 = 9$ ft, $h = 25$ ft, $L = 16$ ft, and $E_s = 6000$ psi the cantilever deformations equal 0.250 in and the bulging deformations equal 0.204 in.

Controlling soldier beam settlements will limit lateral deformations of the wall. Installing ground anchors at flat angles will reduce the downward load applied to the soldier beams and prevent soldier beam settlement. Figure 76 shows how a steep anchor can cause soldier beam settlement and rotation of the wall around the toe if the beam settles. Most ground anchor wall designs use skin friction and end bearing resistances to calculate the ultimate axial load-carrying capacity of the soldier beam. Skin friction is fully mobilized after the beam settles a small amount. Considerably more movement is required to mobilize end bearing fully. Therefore, soldier beam settlements can be limited, by designing the toe to develop most of its axial load-carrying capacity in skin friction.

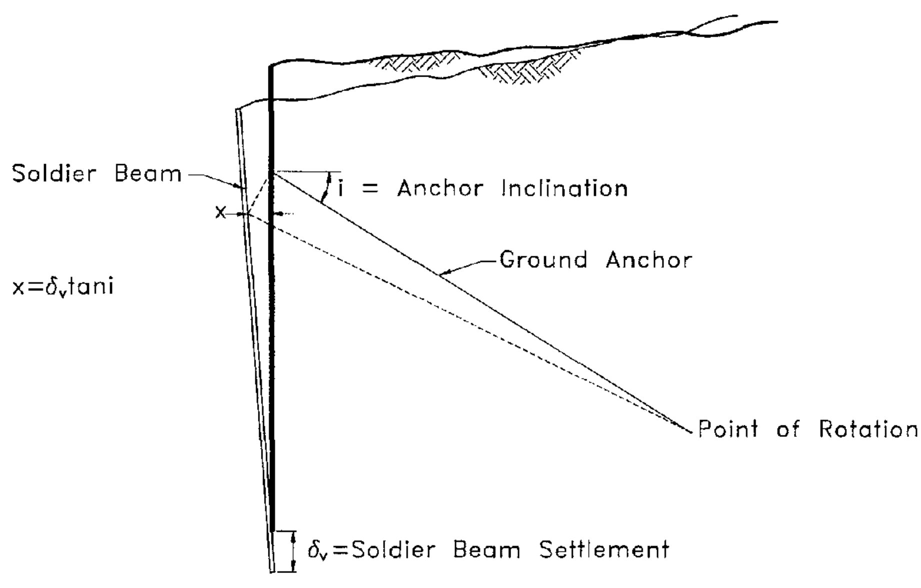


FIGURE 76
Relationship Between Soldier Beam Settlement and Wall Movements

Decreasing the distance to the upper ground anchor, reducing the spans between anchors, and increasing the axial load-carrying capacity of the soldier beam toe will not eliminate lateral movements and ground settlements. These actions should not be taken arbitrarily since they are associated with additional costs. Installing the ground anchor at flat angles generally will not add additional costs and it will reduce the vertical load in the soldier beam. Practically, lateral movements and vertical settlements for most soldier beam walls cannot be reduced below $0.001H$.

Permanent soldier beam walls are not used to support building loads when the building foundation is near the wall. If a line from the bottom of the wall to the edge of the foundation is steeper than a one horizontal to two vertical, then the building is underpinned or supported by a stiff wall (tangent piles, secant piles, or diaphragm wall). Buildings beyond the one horizontal to two vertical line will experience small settlements but no structural damage.

9.2 CORROSION PROTECTION FOR ANCHOR TENDONS

Ground anchor tendon corrosion protection must be designed and constructed to ensure that the ground anchor will reliably support the wall for its design life. Anchor tendons are fabricated using high-strength prestressing steels that are susceptible to embrittlement types of corrosion. When high-strength steels are used, the corrosion protection systems must be designed to prevent corrosion. Estimating design life by predicting metal loss is not valid for prestressing steels. The Post-Tensioning Institute (1996) indicates that two classes of corrosion protection are used in the United States. Figure 77 shows a Class I Protection—Encapsulated Anchor Tendon, and Figure 78 shows a Class II Protection—Grout Protected Anchor. The unbonded length and anchorage area for both classes of protection assume that aggressive conditions exist near the structure. Similar protections are provided for the unbonded length and the anchorage of Class I and Class II protected tendons. Corrosion protections for the tendon bond lengths are different for the different classes of protection. Details about ground anchor corrosion protection can be found in AASHTO's *The Standard Specifications for Highway Bridges* (1996), AASHTO-AGC-ARTBA Task Force 27 Report (1990), PTI's *Recommendations for Prestressed Rock and Soil Anchors* (1996), and *Tiebacks* (Weatherby, 1982).

Corrosion protection for the anchorage area is very important. Most of the ground anchor tendon corrosion failures have occurred near the anchorage. The protection near the anchorage must be designed to protect the tendon where the corrosion protection over the unbonded length is terminated. A grout or corrosion inhibiting compound-filled steel tube (trumpet) attached to the bearing plate provides the protection for the prestressing steel just below the bearing plate. Most trumpets are filled with grout. If a corrosion inhibiting compound is used, then the seal between the trumpet and the unbonded length corrosion protection must function for the life of the structure. If grout is used to fill the trumpet, the seal between the tendon and the trumpet only has to function until the grout has set. The anchorage for most permanent ground anchor walls will be encased in the concrete. If the anchorage remains permanently exposed, it should be protected by a grout or a corrosion inhibiting compound-filled cover. Grout should be used if possible. Good detailing, care during construction, and inspection are necessary to ensure that the anchorage protection is done properly.

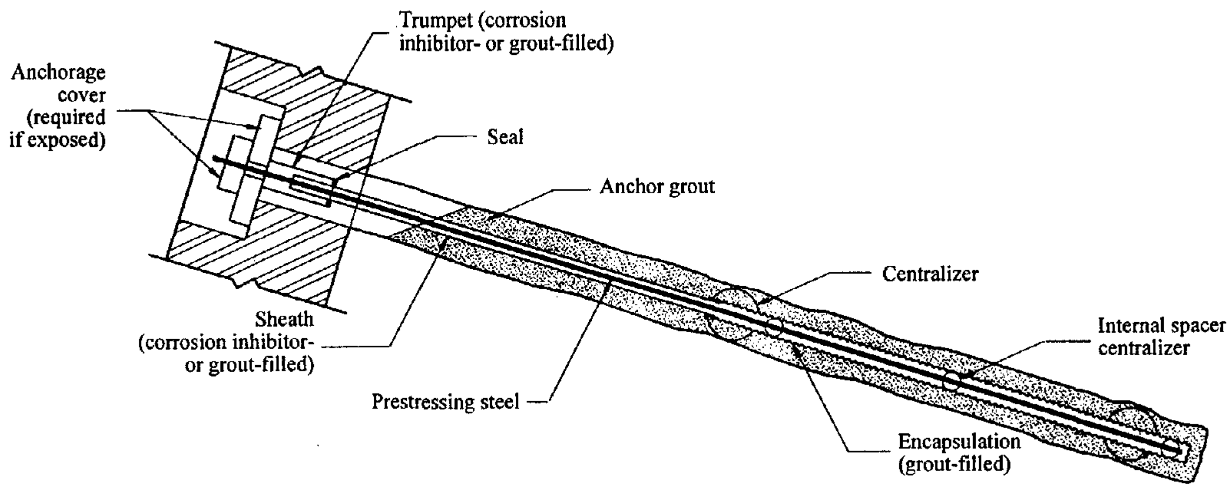


FIGURE 77
Class I Protection — Encapsulation Anchor Tendon

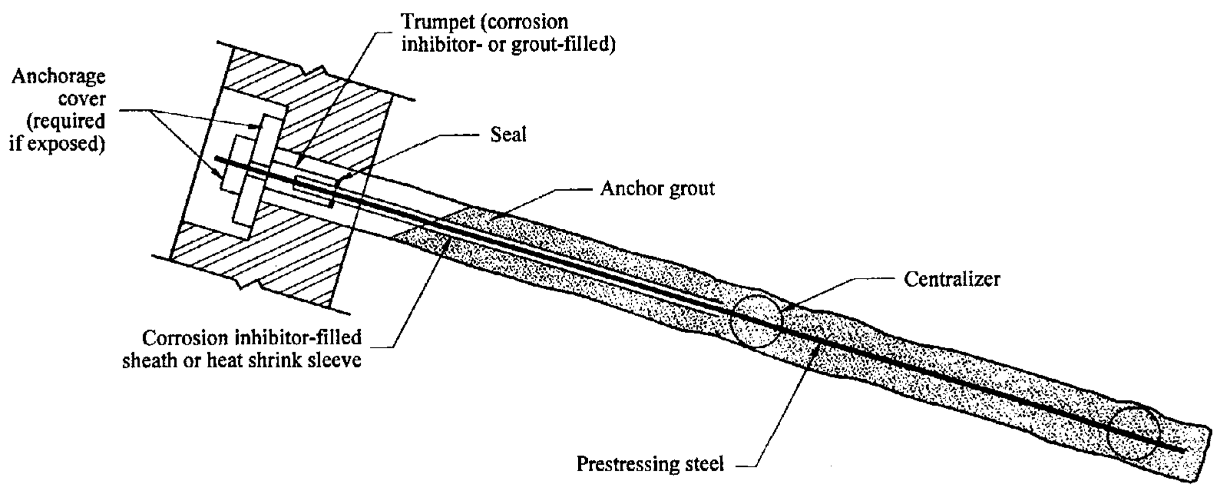


FIGURE 78
Class II Protection — Grout Protected Anchor Tendon

Corrosion protection for the unbonded length is provided by a tube (sheath) filled with a corrosion inhibiting compound or grout, or a heat shrinkable tube internally coated with a mastic coating. Similar protection is provided for both classes of protection.

The difference between a Class I Protection and a Class II Protection is whether the tendon bond length will be grouted inside an encapsulation or protected by anchor grout. The tendon bond length is encapsulated in a corrugated plastic tube or a deformed steel tube when a Class I Protection is used. When a Class II Protection is used, the tendon bond length is protected by the anchor grout. No corrosion failures of a ground anchor tendon along the tendon bond length have been reported when the tendon has been properly grouted. One structural failure is attributed to corrosion along the tendon bond length. When these tendons were unearthed, grout was not present. When a Class II Protection is used, care must be taken to ensure that the anchor grout surrounds the bare prestressing steel along the tendon bond length. The designer or the owner specifies the class of corrosion protection for a project. Figure 79, adapted from the Post-Tensioning Institute (1996), can be used to guide the designer in selecting the class of corrosion protection.

Figure 79 shows that the class of corrosion protection system for a permanent ground anchor is based on aggressivity, consequences of tendon failure, and an evaluation of the extra costs for installing an encapsulated tendon versus the benefits of having the encapsulation.

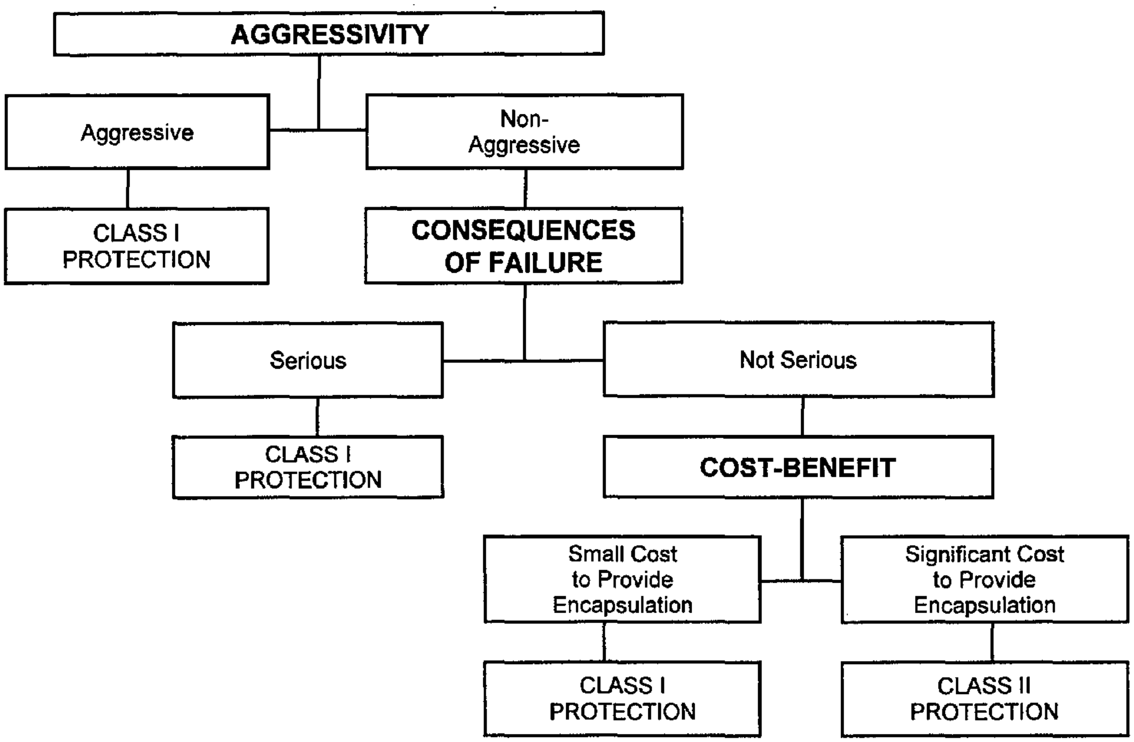


FIGURE 79
Guide for Selecting the Class of Corrosion Protection for a Ground Anchor Tendon

Ground is considered aggressive if it has one or more of the following: a pH value less than 4.5, a resistivity less than 2000 ohm-cm, sulfides present, stray currents present, or caused chemical attack to other buried concrete structures. Tests from a nearby site can be used to evaluate the aggressivity of a site if the designer is confident that the ground conditions are similar. If aggressivity tests are not done, then the ground is assumed to be aggressive. Salt water or tidal marshes, cinder fills, ash or slag fills, organic fills containing humic acid, peat bogs, acid mine wastes, or industrial wastes are considered aggressive ground.

If the corrosion failure of a single ground anchor tendon could result in serious consequences, then a Class I Protection is recommended. A single anchor tendon failure will not have a serious impact on the performance of a permanent ground anchor wall. Ground anchor walls will redistribute load if an anchor tendon fails. To verify this, load was reduced on two ground anchors as part of the research reported by Weatherby, et al. (1998). After the load reduction the wall was monitored for 1 yr. The reduction in load had little effect on the adjacent ground anchors and soldier beam moments. (Adjacent ground anchors were locked-off at a load equal to 75 percent of their design load.) Broms (1988) reported similar results on temporary excavation support systems in weak cohesive soils.

The final criterion for selecting the class of corrosion protection is the incremental cost for changing from a Class II Protection to a Class I Protection. The cost to provide an encapsulated tendon can be more than just the costs of installing the protection. Encapsulating the tendon bond length increases the diameter of the tendon, which may require a more expensive installation method than one suitable for a Class II Protection. In an open drill hole the cost difference can be small, and the designer may elect to use a Class I Protection even though one is not necessary. If the cost of switching from a Class II Protection to a Class I Protection is significant, then the designer may determine that the benefit of encapsulating the tendon is not worth the additional costs associated with a Class I Protection.

9.3 CORROSION PROTECTION FOR SOLDIER BEAMS

Soldier beam corrosion problems have not been reported. Soldier beams are fabricated from Grade 36 or Grade 50 structural steels. These steels are not subject to embrittlement corrosion like the prestressing steel used to fabricate the ground anchor tendon. If corrosion develops on a soldier beam, it will be distributed over a portion of the surface or localized in a pit. Both types of corrosion cause a loss of section, but they do not cause dramatic failure of the member. Unless the environment is acidic, pH less than 4.0, oxygen must be present and the ground must be a good electrolyte for corrosion to continue in the underground. Romanoff (1962 and 1969) presented the results of National Bureau of Standards studies on the corrosion of driven steel piles. Soil conditions at the sites varied widely from well-drained sands to clays. The resistivity of the soils ranged from 300 ohm-cm to 50,200 ohm-cm, and the pH ranged from 2.3 to 8.6. Romanoff found that the steel pilings were not affected by corrosion in undisturbed natural soils regardless of the soil types or properties. He found minor to moderate corrosion in the form of shallow pits on piles driven through fills or in soils above the groundwater table. The average reduction in wall thickness on any of the piles examined was not significant enough to impair the useful life of the structure.

Cheney (1997) reported that recent examinations of driven steel piles exposed during bridge reconstruction operations have revealed severe corrosion. Because of these findings, several State DOT's and the National Cooperative Highway Research Program (NCHRP) are reassessing the corrosion of driven steel piles in non-marine applications. AASHTO is proposing a new standard of recommended practice for corrosion assessment of driven steel piles in non-marine applications. This draft standard is based on NCHRP report 10-46, Corrosion of Steel Piling in Non-marine Applications, which is expected to be published in 1997. The NCHRP report will contain a rational procedure for assessing corrosion, based on testing of the ground and life of the structure.

Corrosion is not a concern for drilled-in soldier beams. Drilled-in soldier beams are surrounded either by lean-mix backfill or structural concrete. Permanent wall facings will be made using cast-in-place concrete or precast concrete panels. When precast concrete panels are used, they will be connected to the soldier beams and the connection should be encased in a cast-in-place concrete closure pour. Lean-mix backfill and structural concrete create a high pH environment for the steel soldier beam. In a high pH environment, a diffusion barrier of hydrous ferrous oxide will develop on the surface of the steel. This barrier will prevent oxygen from reaching the steel surface and keep the rate of corrosion very low.

If buried concrete structures in the vicinity suffer from attack, then drilled-in soldier beams should be coated. Galvanizing, coal-tar epoxy coatings, or fusion-bonded epoxy coatings are suitable.

Recent observations of corrosion on driven foundation piles in natural ground have led FHWA to recommend evaluating the corrosion of driven steel soldier beams in accordance with NCHRP 10-46 when published, or AASHTO *Standard of Recommended Practice* when approved. In the interim before these guidelines are available, if fill soils are present in locations where the driven soldier beam bending moments are expected to be high, the beams should be protected from corrosion. Corrosion protection can consist of increasing the thickness of the member to account for section loss or applying a coating. Increasing the thickness of the flange and web by 1/16 in will allow for a metal loss of 1.25 mils/yr for 50 yr. Instead of increasing web and flange thickness, higher strength steels can be used at lower allowable strengths (using Grade 50 steel at Grade 36 steel allowable stresses would allow a 39 percent loss in section). When the pH is less than 4.0, the soldier beams should be coated or drilled-in. Coal tar epoxy or fusion-bonded epoxy coatings are recommended.

9.4 FREEZING GROUND

Ground anchor loads have increased during the winter because of the ground freezing behind the wall (McRostie and Schriever, 1967, Sandegren, et al., 1972, Stille, 1976, Morgenstern and Sego, 1981, and Eigenbrod and Burak, 1992). The increased loads result from the development of ice lenses in the ground. Three conditions have to exist for the development of ice lenses:

- Ground temperature has to remain below freezing.
- Soils have to be frost susceptible.
- Groundwater or moisture has to be present.

Ground temperature and the depth of frost penetration depend on the duration of the below freezing temperatures. The Freezing Index is a measure of the severity of the winter conditions. It is the cumulative total of the difference between the daily mean air temperature and the freezing point (a Freezing Index of 1 °F day means that the means temperature was 31 °F for one day). Figure 80 shows the distribution of mean Freezing Index values in the continental United States. This figure is used to determine if the temperatures are low enough to cause significant ground freezing, and to help establish insulation requirements for permanent ground anchor walls. Permanent ground anchor walls are not designed for frost pressures. The approach taken is to prevent the ground from freezing.

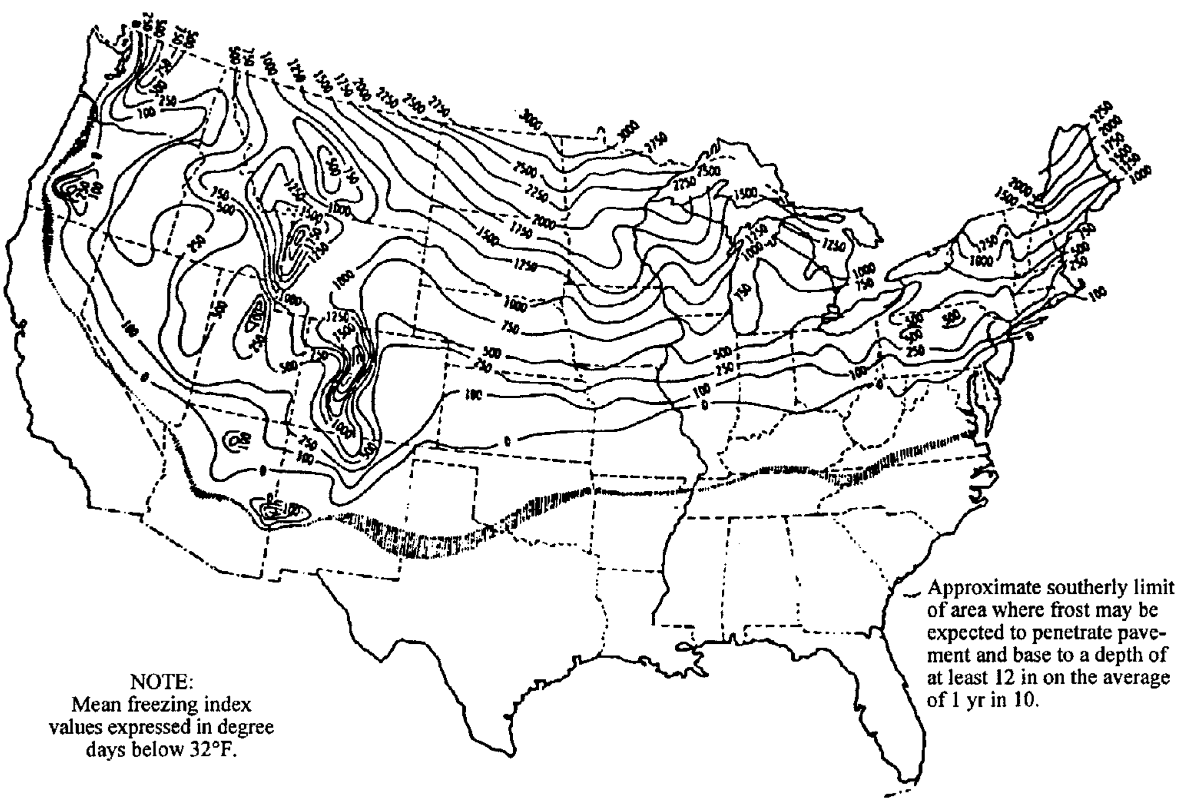


FIGURE 80
Distribution of Mean Freezing Index Values in Continental United States

Inorganic soils with more than 3 percent finer than 0.02 mm are potentially frost susceptible. This means that soils classified as GM, GC, SM, SC, ML and CL are potentially frost susceptible, and Clean GW, GP, SW and SP soils are not frost susceptible. However, soils are frequently interbedded, and the frost susceptibility of the ground cannot be reliably determined based on the soil classifications. Chamberlain (1981) reviewed a variety of index tests for determining the frost susceptibility of a soil.

The amount of frost heave or frost pressures is dependent upon the access to water. Heave will be limited if the only water available is porewater between the soil particles. If groundwater is available, then the ice lenses can continue to grow. Increased pressures on the wall are related to the growth of ice lenses. Potential sources for water are: capillary rise, saturated compressible clays, perched water, groundwater, and surface drainage above the wall.

Schnabel Foundation Company evaluates the need to insulate permanent ground anchor walls when the Freezing Index is greater than 500°F-Days. If the ground is susceptible to the formation of frost lenses and their is a source of groundwater, then we determine the R-Value required for the Freezing Index using Figure 81. After determining the R-Value required, we establish the insulation requirements. R-Values for different insulations are given in Table 21.

TABLE 21
Values for Designing Insulation for Permanent Ground Anchor Walls

MATERIAL	R-VALUE (per in)*
Concrete	0.1
Wood lagging	1.0
Dow styrofoam SM	5.0
Styrene foam-bead board (Zonolite by W.R. Grace)	3.0
Drainage board (Geotech Systems Corp.)	2.5
Enkadrain or Miradrain	1.2
Air	1.2
* Values consider aging and moisture conditions that may exist in the ground	

The R-Value for a permanent wall is computed by multiplying the thickness of the different elements time the values in Table 21. A typical 12-in-thick permanent ground anchor wall will have and R-Value of:

Concrete	12 in x 0.1	=	1.2
Wood lagging	3 in x 1.0	=	3.0
Enkadrain	0.75 in x 1.2	=	0.9
	Total R-Value		5.1

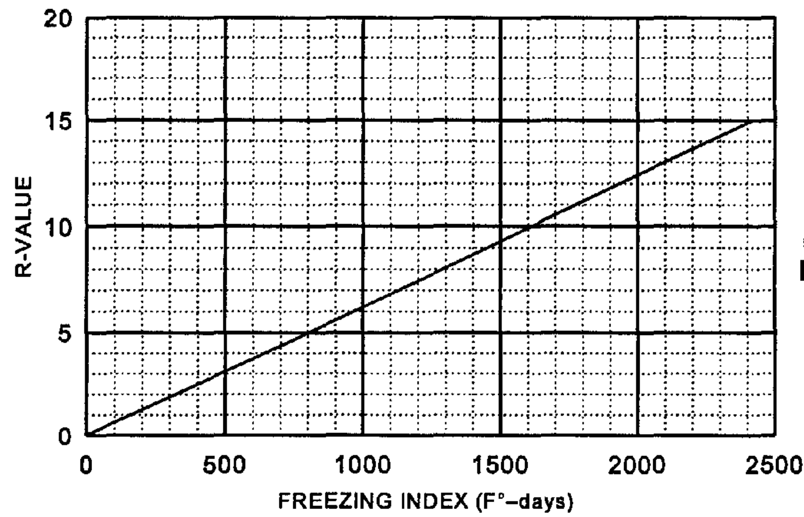


FIGURE 81
R-Value as a Function of Freezing Index (adapted from Robinsky and Bespflug, 1973)

Figure 81 shows that a R-Value of 5.1 is suitable for sites with a Freezing Index of 800°F-Days. Insulation is used when the R-Value of the wall is insufficient. Figure 82 shows insulation board applied to a permanent ground anchor wall before pouring a cast-in-place concrete face.

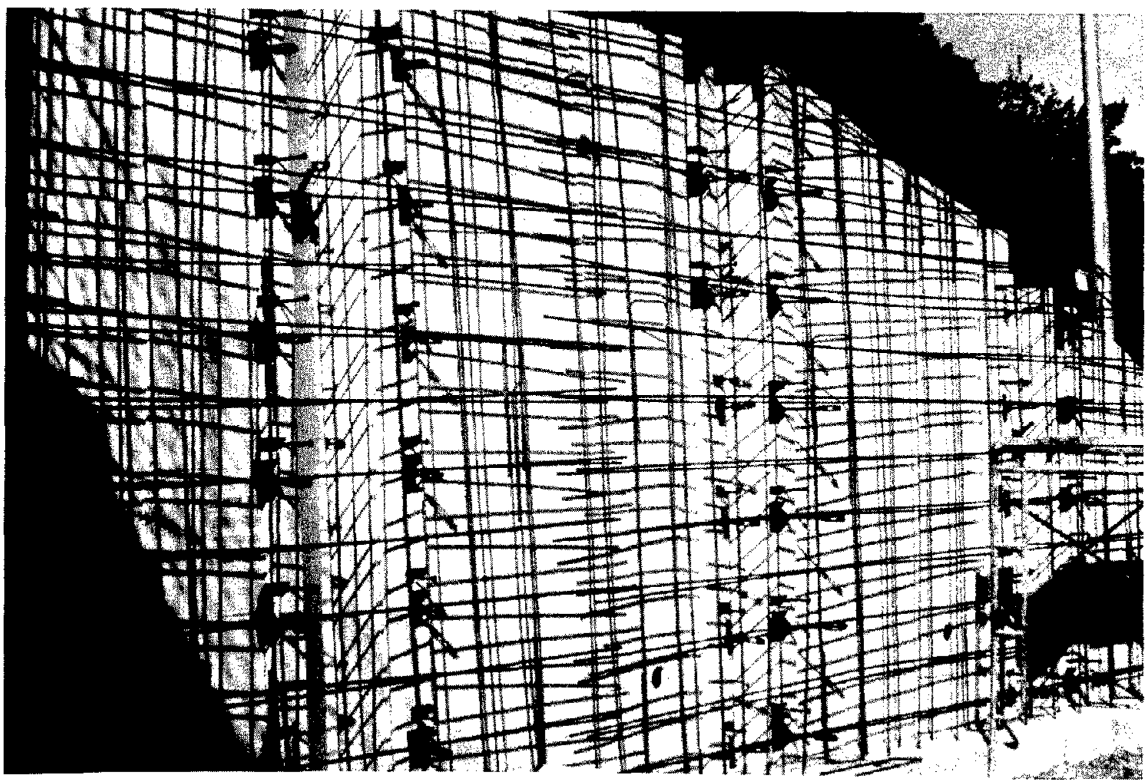


FIGURE 82
Insulation Applied to Permanent Ground Anchor Wall

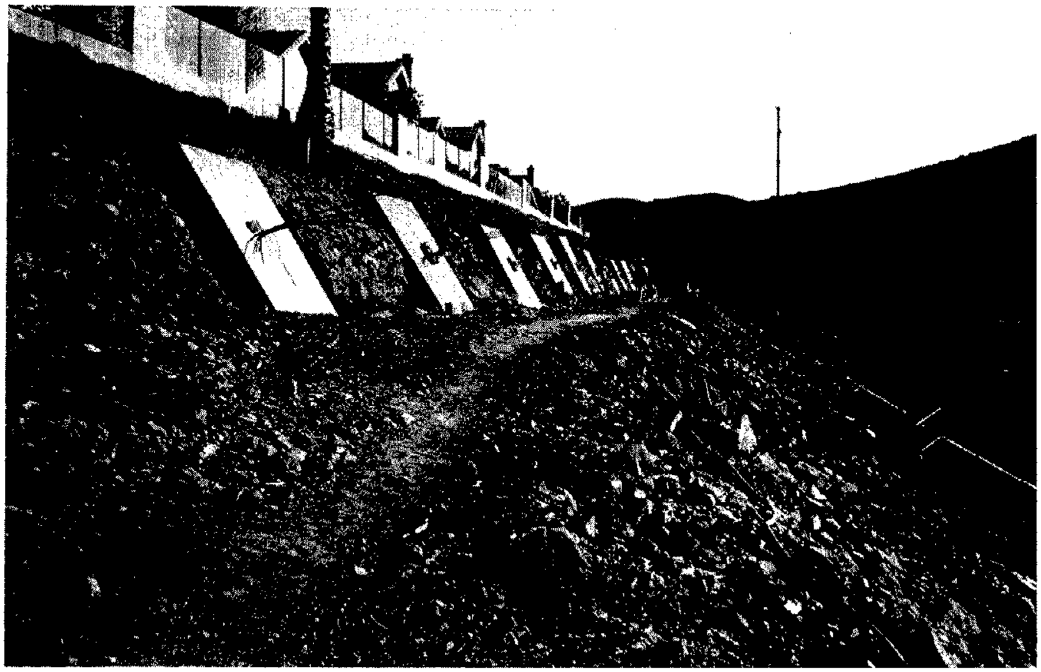
9.5 LANDSLIDE STABILIZATION WALLS

Limiting equilibrium analyses are used to determine the load that must be applied to stabilize a landslide with an adequate factor of safety. A factor of safety of 1.3 is generally used. The analysis is the same as those presented for walls in Section 4.3. Guidelines for modeling the ground anchor load in Section 4.3 apply for landslides too. If the wall does not penetrate the failure surface, then the ground anchor load is used in the limiting equilibrium analysis. If the wall penetrates the failure surface, then the horizontal component of the ground anchor load is used in the analysis. If the wall penetrates the failure surface, the passive capacity of the wall or the shear strength of the wall, whichever is less, may be modeled as an element with cohesion.

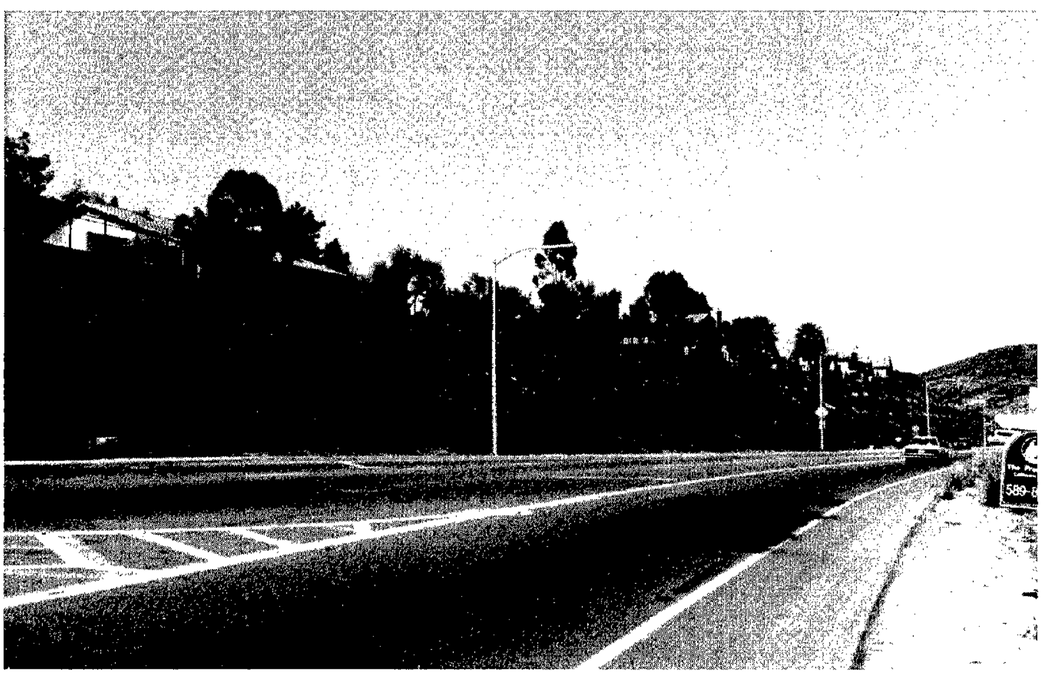
When two or more rows of ground anchors extend beyond the failure surface, the soldier beams do not have to penetrate the failure surface. These walls will be stable if the ground above the failure surface moves as a block. If a landslide has a well-defined failure surface with good ground above and below the failure surface, then anchored elements similar to the ones shown in Figure 83 can be used. Landscaping can conceal the elements completely, making the wall essentially invisible.

When the failure surface is steeply inclined, the load required to stabilize the wall will be small. In this case, load from apparent earth pressures may be greater than the load from the limiting equilibrium analysis. The wall should be designed to resist whichever load is the greatest. If the failure surface is flat, the load required to stabilize the landslide may be large and very sensitive to small changes in the shear strength along the failure surface. It is important to determine realistic shear strength parameters when the failure surface is flat. Small changes in the strength can have a significant impact on the load required to support the landslide.

Soldier beams and ground anchors for a landslide stabilization wall are designed to resist the earth pressure from the limiting equilibrium analysis. If the upper ground anchor load is high, the ground may not be able to develop passive resistance to resist the applied ground anchor test load. A soil-structure interaction analysis using soil response curves (Section 7.2) or the calculation method in Section 9.10 can be used to determine if the ground behind the wall has sufficient passive capacity. Permanent facings for landslide walls are normally placed after the excavation in front of the wall is complete. These facings are designed to resist apparent earth pressures rather than the landslide pressures.



a) During construction



b) After landscaping

FIGURE 83
Anchored Elements Used to Stabilize a Landslide in San Diego, California

9.6 SURCHARGE LOADS

AASHTO's *The Standard Specification for Highway Bridges* (1996) specifies that a live load surcharge equal to 2 ft of earth shall be applied behind the wall when traffic can come within a horizontal distance from the top of the wall equal to one-half the height of the wall. The surcharge is treated as a uniform load on the wall equal to $K_a \gamma 2$. If limiting equilibrium analyses are used to determine the ground anchor loads, the surcharge load is applied as a uniform surcharge load equal to $\gamma 2$.

Concentrated surcharge loads and line loads are distributed to the wall in accordance with Section 5.5.2 of AASHTO's *The Standard Specification for Highway Bridges* (1996).

9.7 BARRIER OR PARAPET LOADS

Walls with vehicle traffic behind them have a concrete barrier placed on top of the wall. The barrier load is a horizontal impact load, and it is considered in the AASHTO Group III loading. AASHTO's *The Standard Specifications for Highway Bridges* (1996) defines the barrier load to be a concentrated force of 10 kips applied a minimum of 2 ft, 8 in above the top of the roadway surface. The barrier load can be resisted by the anchored wall or the barrier can be designed to resist overturning moments by its own mass.

AASHTO allows the horizontal impact load to be distributed to the mechanically stabilized wall, reinforcing strips over a length of 20 ft. This results in a uniformly distributed load of 500 lb/linear ft. Ground anchor walls designed to resist the barrier impact load should be designed to resist a similar uniformly distributed load. The load will be applied a minimum of 2 ft, 8 in above the roadway surface. The barrier impact load can be carried by the soldier beam or a composite section made up of the concrete wall facing and the soldier beam. Since this load is a Group III load, the allowable stresses are increased by 125 percent.

9.8 FACING DESIGN

Permanent wall facings are designed to support the ground between the soldier beams. They are designed to span horizontally between the soldier beams and to resist the apparent earth pressures. Arching reduces the loads below those given by the apparent earth pressure diagram. Normally, the permanent facing is constructed from the bottom up. If the facing is constructed from the bottom up, the facing for landslide stabilization walls is designed for apparent earth pressures rather than pressures computed from the load required to stabilize the landslide.

Precast concrete wall facings are designed as simple spans between soldier beams. Usually the connection between the soldier beam and the facing is a reinforced cast-in-place closure pour. Dowels, welded straps, or bent bars are attached to the panels and extend into the closure pour. Full-height panels or segmental panels are used. Precast lagging placed behind the flanges of the soldier beams has performed poorly. Precast lagging appears to be economical for walls where fill has to be placed behind the upper portions of the wall. However, precast lagging near the ground anchors often is overstressed in shear when the ground anchors are tested. Then the precast has to be replaced, and disputes arise over who should pay to replace it.

Permanent facings are designed to resist bending moments given by equations in Table 22 from AASHTO's *The Standard Specification for Highway Bridges* (1996). Permanent cast-in-place facings are normally 10 to 12 in thick. Thinner sections can develop adequate strength, but experience has shown that a 10- to 12-in-thick face is required to accommodate soldier beam installation tolerances, reinforcement placement, and allow for proper concrete placement.

TABLE 22
Equations for Computing Bending Moments in Permanent Wall Facings

TYPE OF FACING AND SOIL CONDITION	MAXIMUM MOMENT IN A 1-FT HEIGHT
Simple span w/ no soil arching	$p_a l_2/8$
Simple span w/ soil arching	$p_a l_2/12$
Continuous facing w/ no soil arching	$p_a l_2/10$
Continuous facing w/ soil arching	$p_a l_2/12$
p_a= earth pressure, l = effective facing span	

Three-in-thick timber lagging is commonly used for temporary support of the ground between soldier beams. The driven soldier beams are normally installed on 8-ft centers and drilled-in soldier beams installed on 10-ft centers. (The clear span between the drilled shafts is about 8 ft.) Goldberg, et al. (1976) recommended lagging sizes for different soldier beam spacings and soil types, and these recommendations have been adopted by many DOT's. The lagging thicknesses in their report are often greater than 3 in. Goldberg, et al. (1976) stated that the lagging thicknesses in their report were developed for applications where displacements needed to be limited. Limiting displacements is an unnecessary requirement for most permanent ground anchor walls for highway applications. Therefore, thinner lagging boards than those recommended by Goldberg, et al. (1976) can be used for most highway walls. Three-in-thick lagging boards are less expensive and easier to find than thicker boards, and installation costs for 3-in-thick boards are lower since the boards are easier to handle.

Timber lagging is becoming more expensive, and shotcrete construction facings are beginning to be used. Contractors have begun to use 3- to 4-in-thick steel mesh or steel fiber reinforced shotcrete. If shotcrete is used, it should be designed using sound engineering principles. Information regarding construction of the shotcrete facing can be found in FHWA-SA-96-069

(Byrne, et al., 1996). Weep holes through the shotcrete or prefabricated drains placed between the soil and the shotcrete should be used to allow groundwater to drain from the face of the excavation.

9.9 GROUNDWATER CONTROL

Soldier beam walls are free draining, and groundwater is not a serious construction problem or a design issue. When a cast-in-place concrete face is used, groundwater seepage or perched water is collected using prefabricated drains. The most common drain is a 16-in-wide prefabricated drain with a geotextile fabric on one side. The fabric side is installed against the exposed face of the lagging. They are attached to the lagging mid-span between the soldier beams and at construction and expansion joints. Care is required to keep the concrete from flowing around the lagging and choking the fabric. The drains are extended down to the base of the wall as the excavation is made by overlapping them a minimum of 16 in. Prefabricated drains are connected to a footing drain below finished grade or to weep holes that penetrate the finished wall. When shotcrete is used for the temporary construction facing, the geocomposite drains can be placed directly against the soil or attached to the shotcrete. If the drains are placed against the soil, the fabric side of the drains is placed against the ground. If the drains are attached to the shotcrete, vertical columns of weep holes are drilled through the shotcrete and the drains are placed over the weep holes with the fabric side against the shotcrete.

When precast concrete panels are used to face the walls, the space between the panels and the temporary facing may be backfilled with ASTM No. 57 stone. The stone functions as a drain and backfill. Holes are formed in the panels to allow water to escape from behind the panels. The holes can function as weep holes or be tied into the footing drain.

Surface water should be prevented from entering the wall drains. Separate collection systems or grading should route surface water away from the wall.

Occasionally, horizontal drains are used to drain water bearing strata for landslide walls. When used, horizontal drains are connected to the footing drain. Installation of upward sloping drains can be difficult because of vertical equipment clearances needed above the bottom of the excavation and the need to drill between installed anchors.

9.10 SEISMIC DESIGN

Temporary and permanent ground anchor walls performed well during the Northridge and Loma Preita Earthquakes in California. These walls were internally and externally stable. An internally stable wall is strong enough to support seismically induced earth pressures. An externally stable wall is one where the permanent displacements after the earthquake are acceptable.

For internal stability, the seismic loading on an anchored wall is accounted for by applying a pseudo-static inertial force to the wall. The force is determined by multiplying an earthquake acceleration times the mass being supported by the wall. AASHTO's *The Standard Specifications for Highway Bridges* (1996) recommends using an earthquake acceleration equal to 1.5 times the peak acceleration for anchored walls. A map in the Specifications gives peak accelerations for the United States.

Caltrans designs its ground anchor walls in areas with high peak accelerations for lower pseudo-static inertial loads than those recommended by AASHTO. Its walls have performed well. They are designed to support an earth pressure about 25 percent higher than the normal apparent earth pressures.

Caltrans also uses limiting equilibrium to check the internal stability of walls subjected to seismic loading. Ground anchor test loads are used in these analyses since each anchor is tested to an overload of 133 percent, and seismically induced lateral earth pressures are considered in AASHTO Load Combination Group VII. Allowable stresses can be increased 133 percent for Group VII loads. To illustrate this procedure and to determine the pseudo-static acceleration associated with Caltrans' higher earth pressures, a limiting equilibrium analysis was done. Figure 84 shows the results of an analysis on a 30-ft-high wall. The total earth pressure applied to the wall using Caltrans' procedures is $0.8K_a\gamma H^2$. For a soil with a friction angle of 32° and a total unit weight of 115 pcf, the total load would be 25.42 kips. Increasing that load by 133 percent, the anchor test overload, gives a total lateral load of 33.81 kips. Applying a lateral load of 33.81 kips/linear ft to the wall, the limiting equilibrium analysis determined that a horizontal acceleration of 0.386g could be applied to the sliding mass for a factor of safety of 1. AASHTO recommends that anchored walls be designed for an acceleration 1.5 times the peak acceleration. Therefore, the peak acceleration allowed by AASHTO would be 0.257g. A peak acceleration of 0.257g is low compared with those measured in recent seismic events.

Ground anchor walls are similar to other internally reinforced walls. Pseudo-static earthquake coefficients for mechanically stabilized walls may be appropriate for ground anchor walls. The horizontal seismic acceleration coefficient for MSE walls is $K_h = (1.45A)A$, where K_h is the acceleration and A is the peak acceleration. Using this relationship, Caltrans' load would correspond to a peak acceleration of about 0.35g.

Seismically induced earth pressures are applied for a brief time. Only the ground anchor capacity is checked to see if it is adequate to support the additional load. The load-carrying capacities of the soldier beams and wall facings are not increased to resist the seismically induced earth pressures.

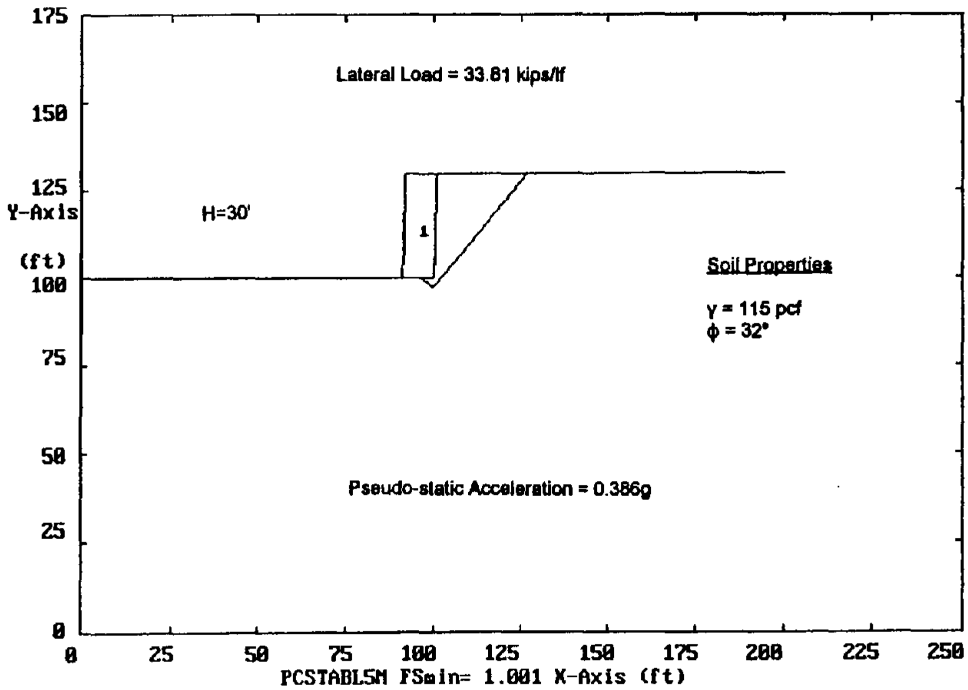
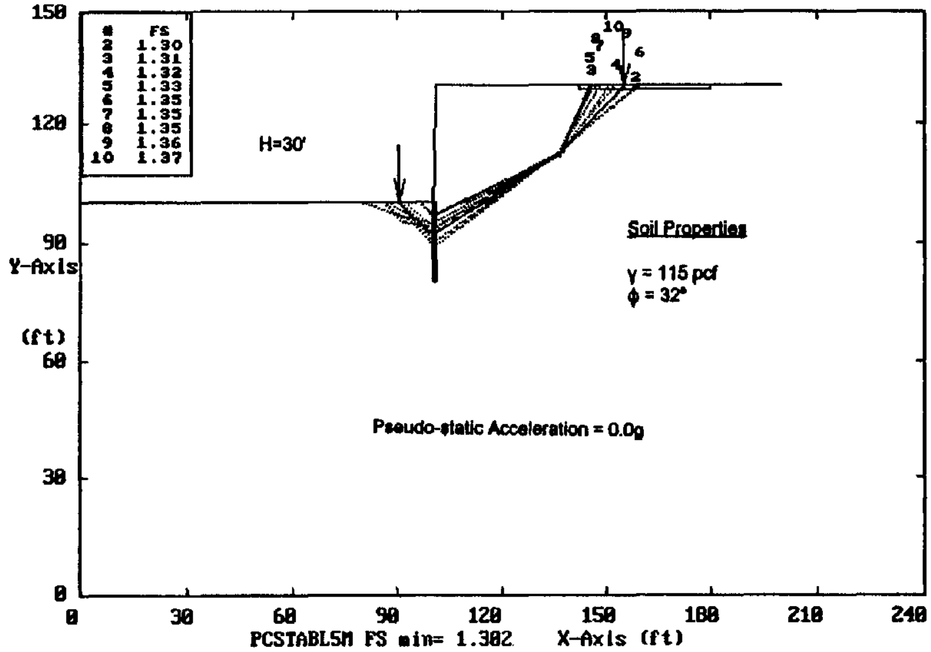
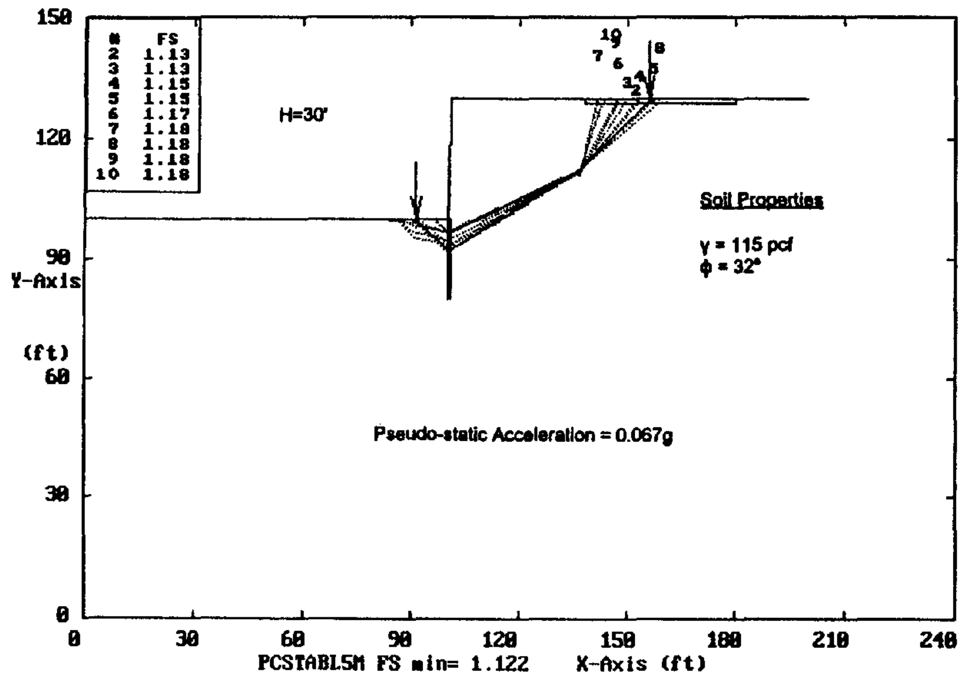


FIGURE 84
Example of Limiting Equilibrium Analysis for Determining the Seismic Acceleration Coefficients, Internal Stability of a Wall

An externally stable wall does not move excessively in response to the ground motion caused by an earthquake. AASHTO's *The Standard Specification for Highway Bridges* (1996) recommends applying a pseudo-static inertial force to the ground mass in an external stability analysis to determine if the factor of safety is adequate. A factor of safety between 1 and 1.1 is used in practice (Norrish and Wyllie, 1996). The Specifications recommend using a pseudo-static acceleration $k_h = 0.5A$, where A is the peak acceleration. Displacements up to $10A$ in are expected when the wall is designed using these procedures. Figure 85 shows an example of a limiting equilibrium analysis used to check the external stability of a wall subjected to a horizontal acceleration of 0.067g. The upper plot shows the results of an analysis performed to locate the back of the ground anchor. A factor of safety of 1.302 was computed when the horizontal acceleration was zero. Then the acceleration was applied to the mass and the factor of safety was determined. The lower plot shows the output from the second analysis. The factor of safety was 1.122 when a horizontal acceleration of 0.067g was applied. This acceleration resulted from a peak acceleration of 0.133g. Permanent displacements for this would be estimated to be about 1.3 in.



a) External stability analysis, acceleration is 0.0g



b) External stability analysis, acceleration is 0.067g

FIGURE 85
Example of Limiting Equilibrium Analyses for Evaluating the External Stability of a Wall Subjected to a Horizontal Acceleration of 0.067g

Caltrans believes that overall stability and displacement predictions are more critical than computation of the internal stability of the wall. Caltrans checks overall stability and displacements using procedures developed by Makdisi and Seed (1978). Their procedure determines a yield acceleration, the acceleration that gives a factor of safety of 1 on the external stability failure surfaces. The yield acceleration is dependent upon the geometry of the wall, the undrained strength of the ground, and the location of the potential failure surfaces. For anchored walls, the external stability failure surfaces are assumed to go behind the back of the ground anchors. After determining the yield acceleration, divide the yield acceleration by the earthquake-induced accelerations to give a ratio, k_y/k_{max}. Figure 86 is used to estimate permanent wall displacements for different magnitude earthquakes and ratios of k_y/k_{max}. The middle range for each magnitude earthquake is used to predict the displacements. If the predicted displacements are too large, then the ground anchors are lengthened, which raises the yield acceleration and the k_y/k_{max} ratio. Smaller displacements are predicted when the k_y/k_{max} ratio increases.

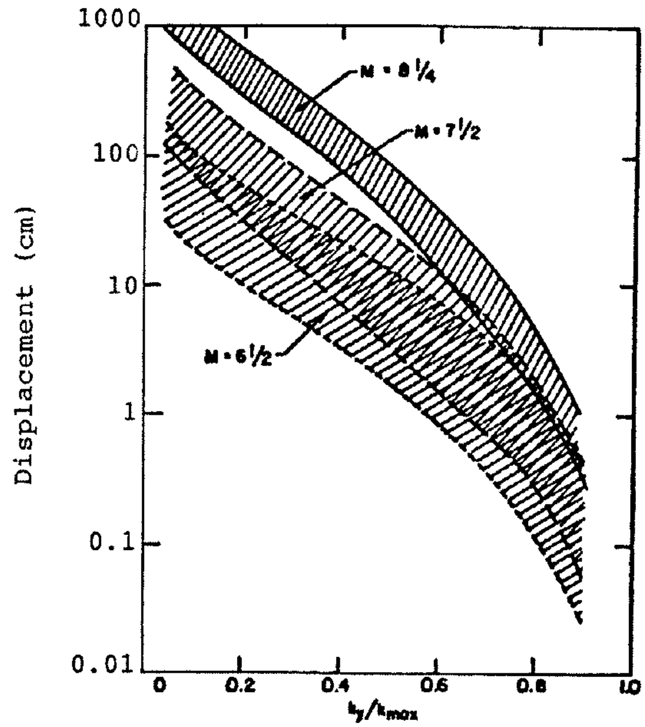


FIGURE 86
Permanent Earthquake-induced Displacements
(from Makdisi and Seed, 1978)

For a peak acceleration of 0.4g and a yield acceleration of 0.2g, the AASHTO method predicts a maximum displacement of 4 in and the Makdisi and Seed (1978) method predicts a displacement of 2 in for a magnitude 6.5 event. The magnitude of acceptable permanent displacements depends upon the nature of the structure and the type of facilities or structures located within or immediately behind the anchored mass. If nothing is behind the wall, large displacements can be tolerated. If utilities are behind the wall, then the movements can be limited to less than 4 in. When structures are within the influence of the wall, then the movements may be restricted further.

Owners should establish the pseudo-static accelerations to use to evaluate the internal and external stability of ground anchor walls. These accelerations can be based on recommendations from AASHTO's *The Standard Specifications for Highway Bridges* (1996), or local experience. Soldier beam and wall facings are not designed to resist seismically induced earth pressures. Ground anchors are designed to provide internal and external stability of the wall under seismic loading. To satisfy internal stability, the ground anchor test load has to be high enough to resist the earthquake loads plus the normal earth pressures. For external stability the ground anchors have to be long enough to create a mass that will not displace excessively when subjected to a pseudo-static acceleration. Ground anchor walls should not be constructed at sites where the soils may be subject to earthquake-induced liquefaction. Geologically young cohesionless sediments below the groundwater table are the most susceptible to liquefaction (Youd and Perkins, 1978). For most of the United States, seismic design will not affect the design of a ground anchor wall.

9.11 RESISTING THE UPPER ANCHOR TEST LOAD

When the ground behind the upper portion of the wall is disturbed or the ground anchor load is high, the soldier beam may deflect excessively during testing of the upper ground anchor. High ground anchor loads result when the anchors are designed to support surcharge, barrier, or landslide loads. To resist the applied test load, the ground behind the soldier beam must develop sufficient passive resistance. If the ground anchor is designed to support loads greater than those given by the apparent earth pressure diagrams, then the passive capacity of the wall should be checked to determine if the ground can resist the upper ground anchor test load. When the ground anchor loads are determined from apparent earth pressure diagrams, checking the passive capacity of the ground is unnecessary unless the ground has been disturbed.

Weatherby, et al. (1998) developed an earth pressure calculation to check the passive capacity of the soldier beam to resist the test load applied to the upper ground anchor. The assumption behind the calculation is that the lateral resistance will be developed over a depth of 1.5 times the distance to the upper ground anchor. Equation 9.3 gives the passive resistance.

$$1.125\, K_p \gamma h_1^2 s \qquad \ldots [9.3]$$

In Equation 9.3, K_p is determined using Figure 24, and h_1 is the depth to the upper ground anchor.

Using Equation 9.3, the lateral capacity of HP12×53 soldier beams on 8-ft centers in a medium dense sand with a friction angle of 32° and a unit weight of 115 pcf is 287 kips, if the ground anchors are installed 6 ft from the top of the soldier beam. A factor of safety of 1.5 is applied to the maximum capacity to obtain a allowable resistance. The allowable resistance should be greater than the upper ground anchor test load. A maximum test load of 191 kips could be applied to the soldier beam in this example.

9.12 CHECKING DIFFERENT CONSTRUCTION STAGES

It is unnecessary to check permanent ground anchor walls for intermediate construction stages if the wall is designed to resist apparent earth pressures and the excavation does not extend too far below the anchor elevation before the anchor is locked-off. The practice of checking bending moments and embedment depths for the "cantilever stage," excavation for the upper row of ground anchors, developed because one-tier walls were designed for triangular earth pressures rather than apparent earth pressure diagrams. When triangular earth pressures are used, the ground anchor can be lower since the earth pressures are much less than the apparent earth pressures. Table 23 illustrates the differences in earth pressures, total load, and bending moments between the Rankine triangular pressure diagram and the modified trapezoidal apparent earth pressure diagram for a 25-ft high wall with one row of anchors at 9 ft. The soil was assumed to have a Rankine active earth pressure coefficient of 0.307 and a total unit weight of 115 pcf. At the ground anchor elevation, the total lateral earth load from the apparent earth pressure diagram is 3.7 times greater than the total load from the Rankine triangular earth pressure diagram, and the bending moment from the apparent earth pressure diagram is 2.7 times greater than the bending moment from the triangular earth pressure diagram.

TABLE 23
Comparison of Earth Pressures and Bending Moments at the Anchor Elevation for Triangular and Apparent Earth Pressure Diagrams

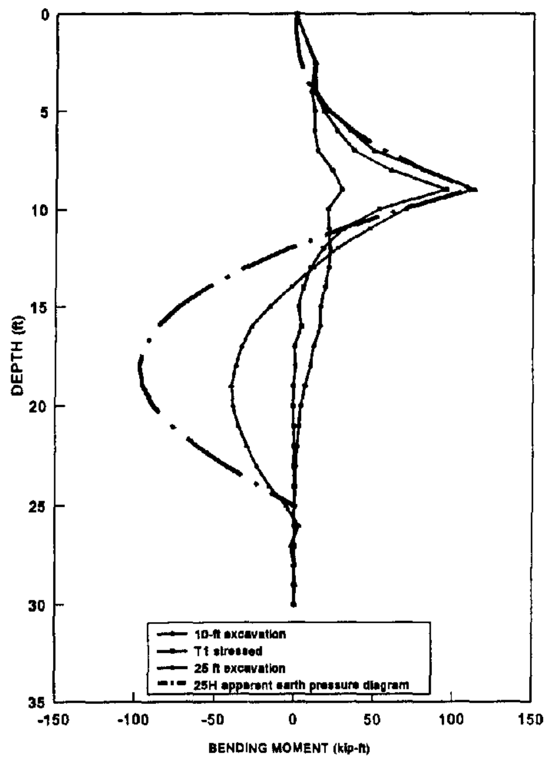
	RESULTS	
	Rankine Triangular Earth Pressure Diagram	**Non-symmetrical Trapezoid Apparent Earth Pressures**
Earth pressures at ground anchor level (ksf)	0.318	0.883
Total lateral load above anchor level (k/lf)	1.431	5.298
Bending moment at ground anchorlLevel (k-ft/lf)	6.4	17.2

Field measurements also show that designing permanent ground anchor walls for intermediate construction stages is unnecessary. Figure 87 shows the measured bending moments for each stage of construction for a wall supported by one row of ground anchors, and a wall supported by two rows of ground anchors. The walls were 25 ft high and built in a medium-dense sand having a friction angle of 32° and a total unit weight of 115 pcf. Table 24 describes the construction stages associated with the moment curves in the figure. Figure 87 shows that the maximum bending moments occurred when the work was completed, and the magnitudes of the bending moments at the final stage were predicted by the apparent earth pressure diagram.

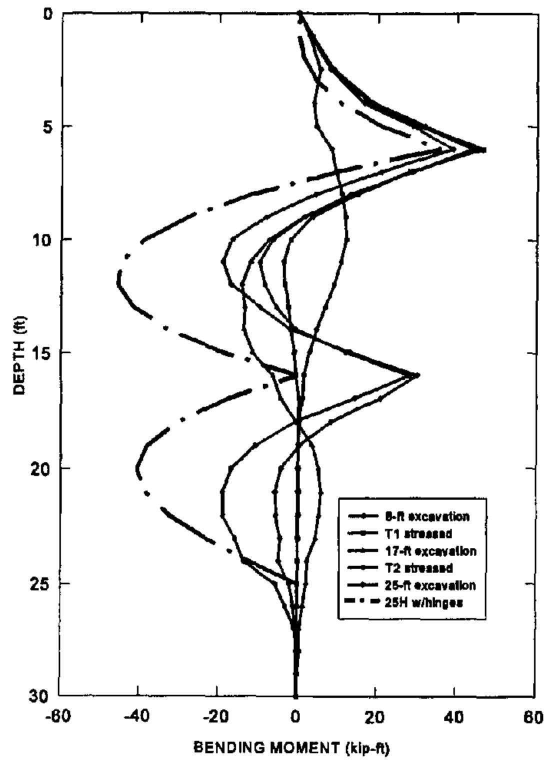
Figure 87 shows maximum cantilever bending moments prior to stressing the ground anchor. The cantilever moment was 29 kip-ft at a depth of 12 to 13 ft for the one-tier wall, and 14 kip-ft at a depth of 10 ft for the two-tier wall. These moments are much less and at different locations than those calculated using procedures recommended by Bowels (1982) and adopted in *Permanent Ground Anchors* (Cheney, 1988) and AASHTO's *The Standard Specification for Highway Bridges* (1996). Table 25 compares the measured bending moments and their locations with those computed using the Bowels' procedure. Computed bending moments were determined using unfactored Rankine passive earth pressure coefficients and coefficients factored by 1.5. The Rankine passive resistances were multiplied by three times the soldier beam width. Table 25 shows that the measured bending moments are about 25 percent of the calculated bending moments.

Measured bending moments are less than calculated moments if the earth pressures are less than those given by Rankine active earth pressure coefficients and/or the passive resistance is greater than assumed. Actual active earth pressures could have been less than the Rankine active pressures, but that would not account for the large difference between the predicted and the measured bending moments. Apparently the passive resistance is greater than that computed assuming Rankine resistances. Ultimate passive resistances computed using relationships developed by Wang and Reese (1986) are approximately twice those given by the Rankine passive coefficient times three soldier beam widths. Rowe (1952) and Bowels (1982) also showed that the actual passive resistance would be mobilized at shallower depths than assumed in the calculations. Mobilization of passive resistance at shallower depths would reduce the bending moments from those predicted. Table 25 shows that the maximum measured moments developed at shallower depths than the locations calculated.

Stage construction analysis may be necessary for temporary walls in soft to medium clay or low-strength soils. When the ground in front of the wall is not adequate to support the toe laterally, the bending moments that develop before the next support is installed may be larger than the bending moments computed for the final construction condition.



a) One-tier wall



b) Two-tier wall

FIGURE 87
Measured Bending Moments for Each Stage of Construction of 25-ft-high Wall

TABLE 24
Construction Stages for One- and Two-tier Walls in Figure 87

	ONE-TIER WALL	TWO-TIER WALL
Excavate to upper anchor	10-ft excavation	8-ft excavation
Install upper anchor	Stress T1 @ 9 ft	Stress T1 @ 6 ft
Excavate to lower anchor	-	17-ft excavation
Install lower anchor	-	Stress T2 @ 16 ft
Final excavation	25-ft excavation	25-ft excavation

TABLE 25
Measured and Predicted Cantilever Bending Moments for One- and Two-tier Wall

WALL	MAXIMUM BENDING MOMENTS (k-ft)			LOCATION OF MAXIMUM MOMENT (ft)		
	Measured	Predicted (K_p/1.0)	Predicted (K_p/1.5)	Measured	Predicted (K_p/1.0)	Predicted (K_p/1.5)
One-tier Wall HP10×57	29	104.5	119.5	12.5	15.9	17.4
Two-tier Wall WF 6×25	14	61.3	70.8	10	14.0	15.5

In conclusion, the maximum bending moments in a permanent ground anchor wall occur at the final stage of construction. The moments are maximum because the anchors are installed without a large amount of overexcavation, the ground has adequate strength to mobilize the passive resistance required to support the wall, and apparent earth pressure diagrams are used to design the walls. It is unnecessary to check different construction stages unless the excavation in front of the wall proceeds more than 3 ft below the anchor elevation without stressing the anchor.

CHAPTER 10:
DESIGN PROCEDURES AND SAMPLE PROBLEMS

This chapter presents a design procedure for permanent ground anchor walls and two design examples. One design example is for a wall in a granular soil deposit, and the other example is for a wall in a cohesive soil deposit. The examples illustrate that there are several satisfactory solutions to each problem, and illustrate how constructability considerations influence the design of a wall.

10.1 GRANULAR SOIL DESIGN EXAMPLE

A 30-ft-high permanent ground anchor wall is to be constructed in the medium dense, granular soil deposit described in Figure 88. The soil has a frictional angle of 29° and a total unit weight of 108 psf. No critical structures or utilities are located near the wall. When construction is completed, a 24-ft-wide service road will be located 10 ft behind the wall. The groundwater table is located 48 ft below the existing ground surface. Soil at the site has a resistivity of 4700 ohm-cm, a pH of 5.5, and no sulfides present. The soil is considered to be nonaggressive. The seismic design condition corresponds to a peak ground acceleration of 0.15g.

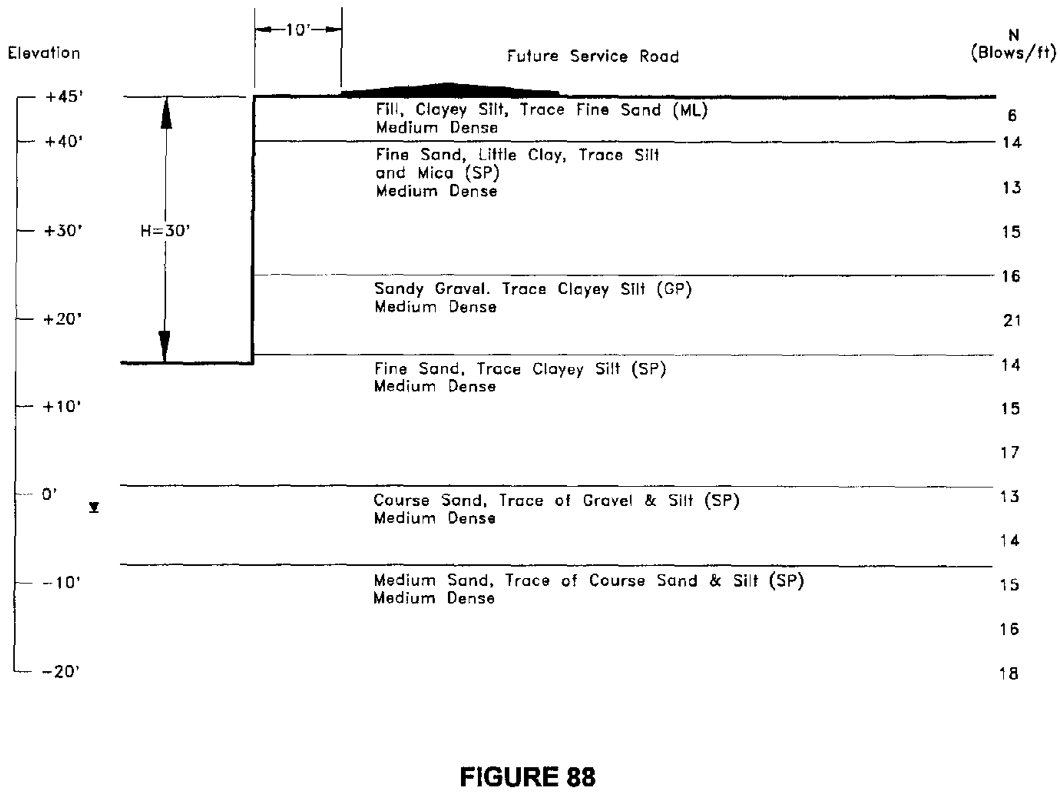


FIGURE 88
Soil Profile for Example No. 1

Bending moment reductions will not be taken. Since bending moment reductions are not taken, soldier beams are not designed for combined axial and bending stresses, and the flanges and the web will not be increased in thickness to account for metal loss resulting from corrosion in the nonaggressive ground.

10.1.1 AASHTO Group I Loading

The Standard Specification for Highway Bridges (1996) Group I Load Combination defines the static loading for the design example. The Group I Load Combination is:

$$\text{Group I} = [D + (L+I) + E + B]$$

where
D = Dead Load
L = Live Load
I = Live Load Impact
E = Earth Pressure
B = Buoyancy

Step 1 — Select the Ground Anchor Type, Estimate the Anchor Capacity, Select the Tendon Type and the Corrosion Protection Requirements.

1. The ground is not aggressive and no critical structures are located behind the wall. Use grout protected ground anchor tendons (see Figure 79).
2. Use pressure-injected ground anchors installed in a driven 3.5-in casing. This method requires a bar tendon to allow a closure point to be driven from the end of the casing.
3. Locate the ground anchor bond length in the 8-ft-thick, medium-dense sandy gravel (GP) layer — Elevation 16 to 24.
4. Estimate that the ultimate load transfer rate for the ground anchor is 10 k/lf.
5. Estimate the maximum feasible ground anchor design load. Allowable load transfer rate equal (10 k/lf)/2 = 5 k/lf. Use anchor bond lengths that are multiples of 8-ft casing lengths.
   a. 24-ft Bond Length (24 ft)(5 k/lf) = 120.0 kips
   b. 32-ft Bond Length (32 ft)(5 k/lf) = 160.0 kips
   c. Allowable tendon capacity for 1⅜-in, Grade 150 bar is 60 percent of the ultimate tendon capacity.
      (0.6)(234.0 kips) = 140.4 kips

   Maximum Ground Anchor Design Load = 140.4 kips

Step 2 — Determine the Total Earth Load.

1. Granular soil has an average standard penetration resistance of 14 blows/ft.

2. Use Figure 30 to determine the Earth Pressure Factor. Figure 89 shows how to use Figure 30. The Earth Pressure Factor for a granular soil with a standard penetration resistance of 14 blows/ft = 23.3 pcf.

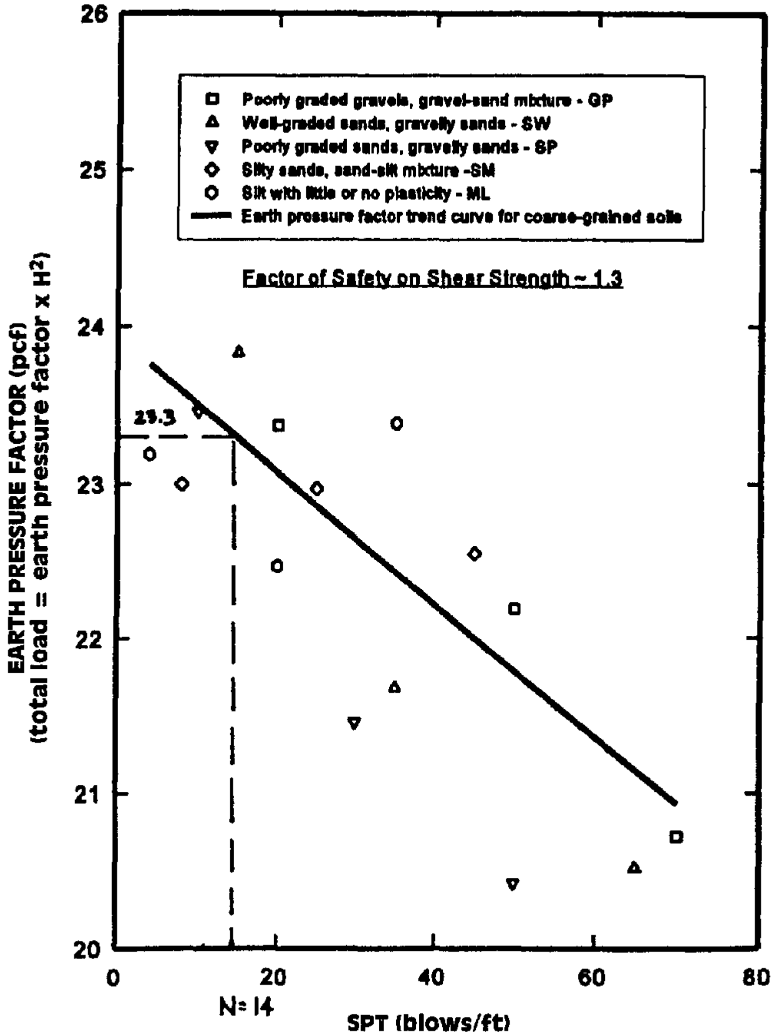


FIGURE 89
Earth Pressure Factor for Example No. 1

Total earth load equal = (23.3 pcf)(30^2 ft²) = 20,970 lb/lf

Step 3 — Determine Earth Pressure Resulting from Traffic Surcharge Pressure.

1. Paragraph 3.20.3 of *The Standard Specification for Highway Bridges* (1996) requires that a surcharge equal to 2 ft of additional soil be applied to a wall if the traffic lanes are within a distance equal to half the height of the wall.

2. Surcharge pressure = (2 ft)(γ) = (2 ft)(108 pcf) = 216 psf

   Lateral surcharge pressure = (K_a)(216 psf) = ($\tan^2(45-\phi/2)$)(216 psf) = 75 psf

Step 4 — One-tier Design—Determine Ground Anchor Load, Soldier Beam Moments, and Subgrade Reaction/Linear Ft of Wall.

1. Earth Pressure Diagram

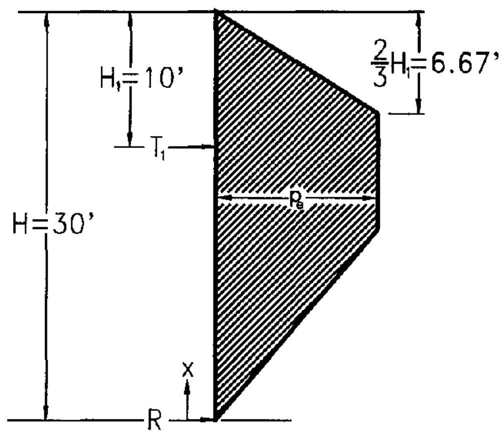


FIGURE 90
Apparent Earth Pressure Diagram for One-tier Wall in Example No. 1

p_e = (23.3 pcf)(H^2 ft^2)/((⅔)(H ft)) = (23.3 pcf)(30^2 ft^2)/((⅔)(30 ft)) = 1048.5 psf

2. Surcharge Pressure Diagram

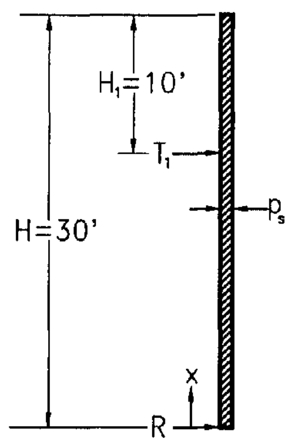


FIGURE 91
Surcharge Pressure Diagram for Example No. 1

p_s = 75 psf

3. Calculate bending moment at the ground anchor, assume H_1 = 10 ft. Figure 28 gives the equations for bending moments and loads for a one-tier wall resulting from the earth pressure, p_e.

   M_1 = $13/54(H_1^2)(p_e) + (p_s)(H_1)(H_1/2)$

   = $(13/54)(10^2)(1048.5) + (75)(10)(10/2)$ = 28,992 lb-ft/lf

4. Calculate ground anchor load (Figure 20 and Figure 28).

$$T_1 = ((23H^2 - 10HH_1)/(54(H-H_1)))(p_e) + (p_s)(H)(H/2)/(H-H_1)$$
$$= (((23)(30^2) - (10)(30)(10))/((54)(30-10)))(1048.5) + (75)(30)(30/2)/(30-10)$$
$$= 18{,}871 \text{ lb/lf}$$

5. Calculate the subgrade reaction (Figure 20 and Figure 28).

$$R = (2/3)(H)(p_e) + (H)(p_s) - T_1 = (2/3)(30)(1048.5) + (30)(75) - 18{,}871 = 4349 \text{ lb/lf}$$

6. Calculate maximum bending moment in span between ground anchor and the bottom of the excavation (Figure 28).

Solve for, x, the location where the shear in the soldier beam is zero. Refer to Figures 90 and 91.

$$R - (p_s)(x) - ((p_e)/((2/3)(H-H_1)))(x)(x/2) = 0$$
$$4349 - 75x - (1048.5/((2/3)(30-10)))(x^2/2) = 0$$
$$39.32x^2 + 75x - 4349 = 0$$
$$x = 9.61 \text{ ft}$$

Compute bending moment, MM_1.

$$MM_1 = Rx - (p_s)(x)(x/2) - ((p_e)/((2/3)(H-H_1)))(x)(x/2)(x/3)$$
$$= (4349)(9.61) - (75)(9.61^2/2) - ((1048.5)/((2/3)(30-10)))(9.61^3/6)$$
$$= 26{,}699 \text{ lb-ft/lf}$$

Since $M_1 \cong MM_1$, the design is balanced.

Step 5 — Ground Anchor Design for One-tier Driven Soldier Beam Wall.

1. Determine ground anchor inclination

   a. Ground anchor elevation = Existing Ground Surface Elevation - H_1
   = 45 - 10 = Elevation 35 ft

   b. Center of anchoring strata = Elevation 20 ft

   c. Install ground anchor at a flat angle to keep downward load on the soldier beam low. Use a 57-ft-long ground anchor.

   d. Assuming a 32-ft-long bond length, calculate the ground anchor inclination.

   Anchor inclination = $\alpha = \sin^{-1}((35-20)/(57-32/2)) = 21.5°$

   Use α = 20° for constructability.

   e. Unbonded length is (57 ft) - (32 ft) = 25 ft > 15 ft minimum recommended by the Post-Tensioning Institute (1996).

2. Determine maximum soldier beam spacing

Soldier beam spacing	=	$((140.4)(\cos 20))/T_1$
	=	$((140{,}400)(\cos 20))/(18{,}871)$
	=	6.99 ft
Use soldier beam spacing	=	**7.0 ft**

3. Anchor Design Load

Ground anchor design load = **140.4 kips**

Step 6 — Soldier Beam Design for the One-tier Driven Soldier Beam Wall.

1. Maximum soldier beam spacing for the one-tier design is 7 ft center to center. Drilled-in soldier beams at this site need to be spaced at 10-ft centers to be economical. Use driven soldier beams for the one-tier design.

2. Determine the size(s) of the driven soldier beams

M_1 = (28,992 lb-ft/lf)(7 ft) = 202,944 lb-ft

$S_{req'd}$ = ((202,944 lb-ft)(12 in/ft))/(20,000 psi) = 121.8 in^3 for Grade 36 steel

HP14×89 Grade 36

$S_{req'd}$ = ((202,944 lb-ft)(12 in/ft))/(27,000 psi) = 90.2 in^3 for Grade 50 steel

HP14×73 Grade 50

3. Determine driven soldier beam toe embedment required to resist the axial load.

   a. Axial load applied to the toe is the vertical component of the ground anchor load plus the deadweight of the wall.
   Vertical component of the ground anchor load = $(140.4)(\sin 20)$ = 48.0 kips

   Dead weight of the wall will be carried by a separate footing since the axial capacity of a driven soldier beam in a medium-dense, granular soil is low.

   Ultimate axial capacity of the soldier beam shall be
   48.0 kips × FS = (48.0 kips)(2) = 96 kips.

   b. Determine the depth of penetration for the driven soldier beam. Refer to section 6.1.2.1.

   Ultimate axial resistance = Skin friction resistance + Tip resistance

   Q_{ult} = $Q_s + Q_t$

   96.0 kips = $f_s A_s + q A_t$

$$= (f_s)((4)(14)/(12))(D) + (q)(14/12)^2$$

$$= (f_s)(4.67)(D) + (q)(1.36)$$

$$= (K)(\gamma)((H+D)/2)(\tan\delta)(4.67)(D) + (1.36)(N_q)\gamma D$$

$$= (1.5)(0.108)((30+D)/2)(\tan(.83\phi))(4.67)(D) + (1.36)(33)(0.108)(D)$$

$$96.0 \text{ kips} = 0.169D^2 + 9.920D$$

$D = 8.5$ ft $\quad Q_{ult} = 96.5 \text{ kips} > 96.0 \text{ kips}$ OK

Use a toe penetration of 8.5 ft.

4. Check the lateral load-carrying capacity of an HP14×73 soldier beam with an 8.5-ft toe penetration, and the beams located on 7-ft centers. Refer to section 6.2.1.

   a. Toe reaction to be resisted is R times the soldier beam spacing.

      (R)(7 ft) = (4349 lb/ft)(7 ft) = 30,443 lb

   b. A spreadsheet incorporating the equations from section 6.2.1 was used to determine the lateral resistance of the soldier beam toe. Figure 92 shows the results of the spreadsheet used to calculate the lateral resistance of an HP14×73 soldier beam in a soil with a friction angle of 29° and a unit weight of 108 pcf. The beams were installed on 7-ft centers. Figure 92 shows that the soldier beam must penetrate to a depth of 9 ft to develop lateral resistance with a FS greater than 1.5.

      (Ultimate Lateral Capacity)/(30.443) = FS

      56.950/30.443 = 1.87 > 1.5 OK

      Use a toe penetration of 9 ft.

Step 7 — Internal and External Stability of the One-tier Driven Soldier Beam Wall.

Internal and external stability of the wall can be checked using Figure 74.

Figure 93 shows how to use Figure 74 to evaluate the internal and external stability of an anchored wall in a granular soil. Draw the anchor to scale on Figure 74, and draw an internal and external stability curve for a soil with a friction angle of 29°. Figure 93 shows that an unbonded length of 25 ft is about twice the unbonded length required for internal stability. A total ground anchor length of 57 ft extends the total anchor length required for external stability by more than 15 ft.

The wall with a ground anchor having an unbonded length of 25 ft and a total anchor length of 57 ft is internally and externally stable.

uw	108	(pcf)	phi	29	phin	0.554309	ko	0.52	Toe Reaction =	30.443	(kips)	(from AEP calculations)		
ht	30	(ft)	bet	59.5	betn	1.697663	ka	0.347	Depth of disturbance at toe =	0	(ft)			
b	1.167	(ft)	alp	19.33333	altn	0.350848	kp	2.882						
s	7	(ft)	btmph	30.500	bmptn	0.589045								
scl	5.833	(ft)												

Ultimate Passive Resistances and Forces

Toe Depth (d)	Wing Width	(Eq. 6.14) Intersection Height (hi)	Passive Resistance Wedge or Flow Conditions: (Eq. 6.13) Wedge Resistance (kips/ft)	Wedge Resistance, d = hi (kips/ft)	Wedge Resistance d = hi, α = 0 (kips/ft)	(Eq. 6.15) Intersecting Wedge Resistance (kips/ft)	(Eq. 6.16) Flow Resistance (kips/ft)	Critical Wedge or Flow Resistance (kips/ft)	(Eq. 6.17) Rankine Passive Resistance (kips/ft)
0	0.00	-4.90	0.00	4.10	0.38	0.00	0.00	0.00	0.00
1	0.60	-3.90	0.61	2.31	-0.05	0.61	3.27	0.61	2.18
2	1.19	-2.90	1.71	1.00	-0.30	1.71	6.54	1.71	4.36
3	1.79	-1.90	3.30	0.19	-0.36	3.30	9.82	3.30	6.54
4	2.38	-0.90	5.38	-0.13	-0.25	5.38	13.09	5.38	8.72
5	2.98	0.10	7.95	0.04	0.04	7.94	16.36	7.94	10.89
6	3.57	1.10	11.01	0.70	0.51	10.82	19.63	10.82	13.07
7	4.17	2.10	14.56	1.85	1.16	13.87	22.91	13.87	15.25
8	4.76	3.10	18.60	3.49	2.00	17.11	26.18	17.11	17.43
9	5.36	4.10	23.13	5.62	3.01	20.52	29.45	20.52	19.61
10	5.96	5.10	28.15	8.24	4.20	24.11	32.72	24.11	21.79
11	6.55	6.10	33.66	11.35	5.58	27.89	36.00	27.89	23.97
12	7.15	7.10	39.66	14.95	7.13	31.85	39.27	31.85	26.15
13	7.74	8.10	46.16	19.04	8.87	35.98	42.54	35.98	28.32
14	8.34	9.10	53.14	23.63	10.78	40.30	45.81	40.30	30.50
15	8.93	10.10	60.61	28.70	12.88	44.80	49.09	44.80	32.68
16	9.53	11.10	68.58	34.26	15.16	49.47	52.36	49.47	34.86
17	10.13	12.10	77.03	40.31	17.61	54.33	55.63	54.33	37.04
18	10.72	13.10	85.98	46.86	20.25	59.37	58.90	58.90	39.22
19	11.32	14.10	95.41	53.89	23.07	64.59	62.18	62.18	41.40
20	11.91	15.10	105.34	61.41	26.07	69.99	65.45	65.45	43.58
21	12.51	16.10	115.75	69.43	29.25	75.57	68.72	68.72	45.76
22	13.10	17.10	126.66	77.93	32.61	81.33	71.99	71.99	47.93
23	13.70	18.10	138.05	86.93	36.15	87.27	75.27	75.27	50.11
24	14.29	19.10	149.94	96.42	39.87	93.39	78.54	78.54	52.29
25	14.89	20.10	162.32	106.39	43.77	99.70	81.81	81.81	54.47
26	15.49	21.10	175.19	116.86	47.85	106.18	85.08	85.08	56.65
27	16.08	22.10	188.54	127.81	52.11	112.84	88.36	88.36	58.83
28	16.68	23.10	202.39	139.26	56.55	119.68	91.63	91.63	61.01
29	17.27	24.10	216.73	151.20	61.18	126.71	94.90	94.90	63.19
30	17.87	25.10	231.56	163.63	65.98	133.91	98.17	98.17	65.37
31	18.46	26.10	246.88	176.54	70.98	141.30	101.45	101.45	67.54
32	19.06	27.10	262.69	189.95	76.13	148.86	104.72	104.72	69.72
33	19.66	28.10	278.99	203.85	81.47	156.61	107.99	107.99	71.90
34	20.25	29.10	295.78	218.24	87.00	164.53	111.26	111.26	74.08
35	20.85	30.10	313.06	233.12	92.70	172.64	114.54	114.54	76.26
36	21.44	31.10	330.83	248.49	98.59	180.93	117.81	117.81	78.44
37	22.04	32.10	349.09	264.35	104.65	189.39	121.08	121.08	80.62
38	22.63	33.10	367.84	280.70	110.90	198.04	124.35	124.35	82.80
39	23.23	34.10	387.08	297.54	117.33	206.87	127.62	127.62	84.97
40	23.82	35.10	406.82	314.87	123.94	215.88	130.90	130.90	87.15
41	24.42	36.10	427.04	332.69	130.72	225.07	134.17	134.17	89.33

Passive Resistance (kips/ft)	Failure Mode	Passive Capacity: Total Passive Force at Given Toe Depth (kips)	Total Active Force on Toe at Given Toe Depth (kips)	Allowance for Disturbance (Eq.6.12 for depth) (kips)	Net Passive Force at a Given Depth (kips)	Factor of Safety	Toe Depth (ft)
0.00	Wedge	0.00	1.31	0	-1.31	-0.04	0
0.61	Wedge	0.30	1.33	0.00	-1.03	-0.03	1
1.71	Wedge	1.46	2.71	0.00	-1.25	-0.04	2
3.30	Wedge	3.96	4.13	0.00	-0.17	-0.01	3
5.38	Wedge	8.30	5.60	0.00	2.70	0.09	4
7.94	Interaction	14.96	7.11	0.00	7.85	0.26	5
10.82	Interaction	24.34	8.66	0.00	15.68	0.52	6
13.87	Interaction	36.68	10.25	0.00	26.43	0.87	7
17.11	Interaction	52.17	11.89	0.00	40.28	1.32	8
19.61	Rankine	70.53	13.58	0.00	56.95	1.87	9
21.79	Rankine	91.23	15.31	0.00	75.92	2.49	10
23.97	Rankine	114.11	17.08	0.00	97.03	3.19	11
26.15	Rankine	139.16	18.89	0.00	120.27	3.95	12
28.32	Rankine	166.40	20.75	0.00	145.65	4.78	13
30.50	Rankine	195.81	22.65	0.00	173.16	5.69	14
32.68	Rankine	227.41	24.60	0.00	202.81	6.66	15
34.86	Rankine	261.18	26.59	0.00	234.59	7.71	16
37.04	Rankine	297.13	28.62	0.00	268.51	8.82	17
39.22	Rankine	335.26	30.70	0.00	304.56	10.00	18
41.40	Rankine	375.57	32.82	0.00	342.75	11.26	19
43.58	Rankine	418.05	34.98	0.00	383.07	12.58	20
45.76	Rankine	462.72	37.19	0.00	425.53	13.98	21
47.93	Rankine	509.57	39.45	0.00	470.12	15.44	22
50.11	Rankine	558.59	41.74	0.00	516.85	16.98	23
52.29	Rankine	609.79	44.08	0.00	565.71	18.58	24
54.47	Rankine	663.17	46.46	0.00	616.71	20.26	25
56.65	Rankine	718.73	48.89	0.00	669.84	22.00	26
58.83	Rankine	776.47	51.36	0.00	725.11	23.82	27
61.01	Rankine	836.39	53.88	0.00	782.51	25.70	28
63.19	Rankine	898.49	56.44	0.00	842.05	27.66	29
65.37	Rankine	962.76	59.04	0.00	903.73	29.69	30
67.54	Rankine	1029.22	61.68	0.00	967.54	31.78	31
69.72	Rankine	1097.85	64.37	0.00	1033.48	33.95	32
71.90	Rankine	1168.66	67.11	0.00	1101.56	36.18	33
74.08	Rankine	1241.65	69.88	0.00	1171.77	38.49	34
76.26	Rankine	1316.82	72.70	0.00	1244.12	40.87	35
78.44	Rankine	1394.17	75.57	0.00	1318.61	43.31	36
80.62	Rankine	1473.70	78.48	0.00	1395.23	45.83	37
82.80	Rankine	1555.41	81.43	0.00	1473.98	48.42	38
84.97	Rankine	1639.29	84.42	0.00	1554.87	51.07	39
87.15	Rankine	1725.36	87.46	0.00	1637.89	53.80	40
89.33	Rankine	1813.60	90.55	0.00	1723.05	56.60	41

FIGURE 92
Spreadsheet for Determining the Lateral Toe Resistance for the One-tier, Driven Soldier Beam Wall in Example No. 1

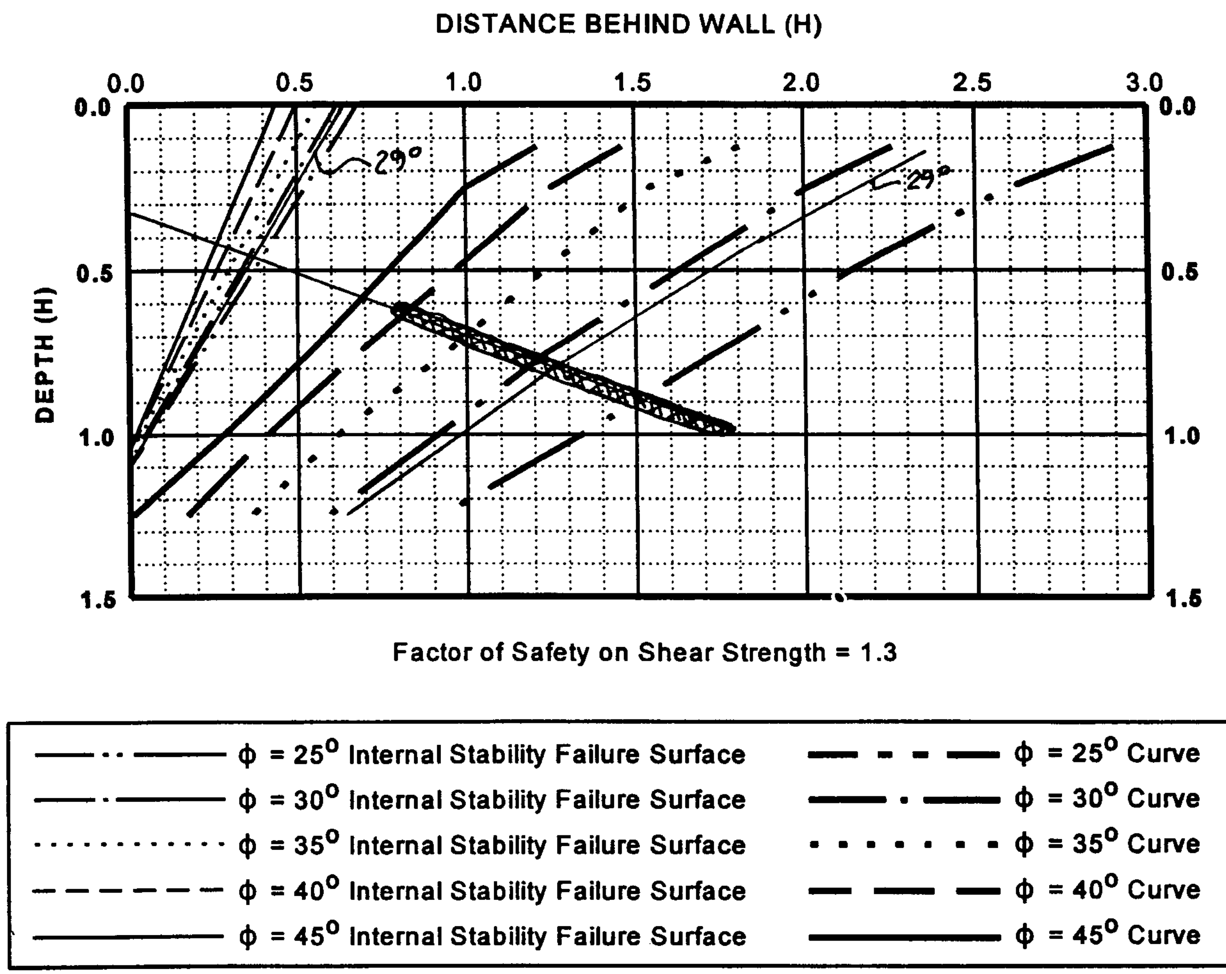


FIGURE 93
Internal and External Stability for the One-tier, Driven Soldier Beam Wall in Example No. 1

Summary of the One-tier, Driven Soldier Beam Design.

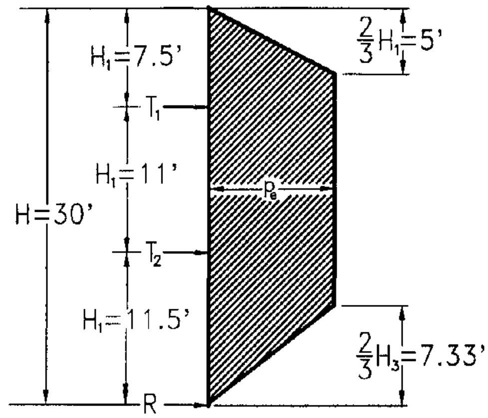
ONE-TIER DRIVEN SOLDIER BEAM DESIGN	
Height of Cut	30 ft
Soldier Beams	HP14×73, Grade 50
	HP14×89, Grade 36
Top Elevation	45 ft
Tip Elevation	6 ft
Soldier Beam Length	39 ft
Soldier Beam Spacing	7 ft center to center
Ground Anchor	1⅜-in Grade 150 Bar
Anchor Design Load	140.4 kips
Anchor Elevation	35 ft
Anchor Inclination	20°
Total Anchor Length	57 ft
Anchor Bond Length	32 ft
Tendon Bond Length	32 ft
Unbonded Length	25 ft

A two-tier wall using driven or drilled-in soldier beams may be more economical to construct than the one-tier wall. Steps 4 through 7 will be repeated for a two-tier driven and drilled-in soldier beam wall. Step 4a is for both the driven and drilled-in soldier beams. Steps 5a through 7a are for the driven soldier beams. Steps 5b through 7b are for the drilled-in soldier beams. The different designs illustrate some of the constructability considerations that should be evaluated when designing a wall.

Step 4a / Two-tier Design — Determine Ground Anchor Load, Soldier Beam Moments, and Subgrade Reaction/Linear Ft of Wall.

1. Earth Pressure Diagram

FIGURE 94
Apparent Earth Pressure Diagram for Two-tier Wall in Example No. 1



p_e = (23.3 pcf)(H^2 ft^2)/($H-H_1/3-H_3/3$) = (23.3)(30^2 /(30-7.5/3-11.5/3) = 886.1 psf

2. Surcharge pressure diagram for the two-tier wall is the same as the one for the one-tier wall (Figure 91).

3. Calculate bending moment at the upper ground anchor, assume H_1 = 7.5 ft. Figure 29 gives the equations for bending moments and loads for a two-tier wall resulting from the earth pressure, p_e.

   M_1 = $(13/54)(H_1^2)(p_e) + (p_s)(H_1)(H_1/2)$
   = $(13/54)(7.5^2)(886.1) + (75)(7.5)(7.5/2)$ = 14,109 lb-ft/lf

4. Calculate the ground anchor loads. Use tributary area method (Figure 20 and Figure 29).

   T_1 = $((⅔)H_1+H_2/2)(p_e) + (H_1+H_2/2)(p_s)$
   = ((⅔)(7.5) + 11/2)(886.1) + (7.5 + 11/2)(75)
   = 10,279 lb/lf

   T_2 = $(H_2/2 + 23H_3/48)(p_e) + (H_2/2 + H_3/2)(p_s)$
   = (11/2 + (23)(11.5)/48)(886.1) + (11/2 + 11.5/2)(75)
   = 10,600 lb/lf

5. Calculate the subgrade reaction. Use tributary area method (Figure 20 and Figure 29).

   R = $(3H_3/16)(p_e) + (H_3/2)(p_s)$
   = ((3)(11.5)/16)(886.1) + (11.5/2)(75) = 2342 lb/lf

6. Calculate maximum bending moment below the upper anchor. Using the tributary area method (Figure 20 and Figure 29), the maximum bending moment is
   $0.1(p_e + p_s)(H_2 \text{ or } H_3\text{, whichever is greater})^2$.

   Determine the maximum moment for an 11.5-ft span.
   $0.1(p_e + p_s)(H_3)^2 = 0.1(886.1 + 75)(11.5)^2 = 12{,}711$ lb-ft/lf

   Since $M_1 \cong$ lower moment, the design is balanced.

Step 5a — Ground Anchor Design for Two-tier Driven Soldier Beam Wall.

1. Determine upper ground anchor inclination
   a. Ground anchor elevation = Ground Surface Elevation - H_1
      = 45 - 7.5 = Elevation 37.5 ft
   b. Center of anchoring strata = 20 ft
   c. Install ground anchor at a flat angle to keep downward load on the soldier beam low. Use a 57-ft-long ground anchor.
   d. Assuming a 24-ft-long bond length, calculate the ground anchor inclination.
      Anchor inclination = $\alpha = \sin^{-1}((37.5 - 20)/(57 - 24/2)) = 22.9°$
      Use α = 20° for constructability.
   e. Unbonded length is (57 ft) - (24 ft) = 33 ft > 15 ft minimum recommended by the Post-Tensioning Institute (1996).

2. Determine upper ground anchor design load. Assume a soldier beam spacing of 8 ft center to center for the driven soldier beams.
   Upper Anchor Design Load = $(T_1)(8)/\cos 20 = (10{,}279)(8)/\cos 20 = 87{,}509$ lb
   Upper Anchor Design Load = 87.5 kips
   Use a 1¼-in Grade 150 bar. Allowable design load is (0.6)(187.5) = 112.5 kips.

3. Determine lower ground anchor inclination
   a. Ground anchor elevation = Ground Surface Elevation - H_1 - H_2
      = 45 - 7.5 - 11 = Elevation 26.5 ft
   b. Center of anchoring strata = Elevation 20 ft
   c. Use a ground anchor tendon with an anchor bond length of 24 ft and an unbonded length of 15 ft. Total ground anchor length = 39 ft.
   d. Assuming a 24-ft-long bond length, calculate the ground anchor inclination.
      Anchor inclination = $\alpha = \sin^{-1}((26.5 - 20)/(39 - 24/2)) = 13.9°$
      Use α = 15° for constructability.
   e. Unbonded length is = 15 ft, the minimum recommended by the Post-Tensioning Institute (1996).

4. Determine lower ground anchor design load. Assume a soldier beam spacing of 8 ft center to center for driven soldier beams.

   Lower Anchor Design Load = $(T_2)(8)/\cos 15$ = $(10{,}600)(8)/\cos 15$ = 87,791 lb

   Lower Anchor Design Load = 87.8 kips

   Use a 1¼-in Grade 150 bar. Allowable design load is (0.6)(187.5) = 112.5 kips.

Step 6a — Soldier Beam Design for the Two-tier, Driven Soldier Beam Wall.

1. Assume driven soldier beams are spaced 8 ft on center.

2. Determine the size(s) of the driven soldier beams. Maximum bending moment is 14,109 lb-ft/lf.

   M = (14,109 lb-ft/lf)(8 ft) = 112,872 lb-ft

   $S_{req'd}$ = ((112,872 lb-ft)(12 in/ft))/(20,000 psi) = 67.7 in³ for Grade 36 steel

   HP14×73 Grade 36

   $S_{req'd}$ = ((112,872 lb-ft)(12 in/ft))/(27,000 psi) = 50.2 in³ for Grade 50 steel

   HP12×53 Grade 50

3. Determine driven soldier beam toe embedment required to resist the axial load.

   a. Axial load applied to the toe is the vertical component of the ground anchor load plus the deadweight of the wall.

      Vertical component of the ground anchor load

      (87.5)(sin20) + (87.8)(sin15) = 52.6 kips

      Dead weight of the wall will be carried by a separate footing since the axial capacity of a driven soldier beam in a medium-dense granular soil is low.

      Ultimate axial capacity of the soldier beam shall be 52.6 kips × FS = (52.6 kips)(2) = 105.2 kips.

   b. Determine the depth of penetration for the HP12×53 driven soldier beam. Refer to Section 6.1.2.1.

      Ultimate axial resistance = Skin friction resistance + Tip resistance

$$
\begin{aligned}
Q_{ult} &= Q_s + Q_t \\
105.2\text{ kips} &= f_s A_s + qA_t \\
&= (f_s)(4)(1)(D) + (N_q)(\gamma)(D)(1)^2 \\
&= (f_s)(4)(D) + (N_q)(\gamma)(D)(1) \\
&= (K)(\gamma)((H+D)/2)(\tan\delta)(4)(D) + (N_q)(\gamma)(D)(1) \\
&= (1.5)(0.108)((30+D)/2)(\tan(.83\phi))(4)(D) + \\
&\quad (33)(0.108)(D)(1)
\end{aligned}
$$

$$= \quad 0.1447D^2 + 7.906D$$

$D = 11.5$ ft $\quad Q_{ult} = 110.1$ kips > 105.2 kips $\quad$ OK

Use a toe penetration of 11.5 ft.

4. Check the lateral load-carrying capacity for a HP12×53 soldier beam with an 11.5-ft toe penetration, and the beams located on 8-ft centers. Refer to Section 6.2.1.

   a. Toe reaction to be resisted is R times the soldier beam spacing.

      (R)(8 ft) = (2342 lb/ft)(8 ft) = 18,736 lb

   b. A spreadsheet incorporating the equations from section 6.2.1 was used to determine the lateral resistance of the soldier beam toe. Figure 95 shows the results of the spreadsheet used to calculate the lateral resistance of an HP12×53 soldier beam in a soil with a friction angle of 29° and a unit weight of 108 pcf. The beams were installed on 8-ft centers. Figure 95 shows that the soldier beam with an 11-ft toe penetration has a lateral resistance of 105.44 kips.

      (Ultimate Lateral Capacity)/(18.736) = FS

      105.44/18.736 = 5.63 > 1.5 $\quad$ OK

<u>Step 7a — Internal and External Stability for a Two-tier Driven Soldier Beam Wall.</u>

Internal and external stability of the wall can be checked using Figure 74.

Figure 96 shows how to use Figure 74 to evaluate the internal and external stability of the wall. Draw the upper and lower anchors to scale on Figure 74 and draw an internal and external stability curve for a soil with a friction angle of 29°. The upper ground anchor has a total anchor length of 57 ft and unbonded length of 33 ft. The lower anchor has a total anchor length of 39 ft and an unbonded length of 15 ft. The wall is externally stable since the back of the ground anchors are well behind the external stability curve for a soil with a friction angle of 29°. The unbonded lengths for the upper and lower ground anchors are more than twice the unbonded length required for internal stability.

The wall with an upper ground anchor having an unbonded length of 33 ft and a total anchor length of 57 ft, and a lower ground anchor having an unbonded length of 15 ft, and a total anchor length of 39 ft is internally and externally stable.

uw	108	(pcf)	phi	29	phtn	0.554309	ko	0.52	Toe Reaction =	18.736	(kips)	(from AEP calculations)
ht	30	(ft)	bet	59.5	betn	1.697663	ka	0.347	Depth of disturbance at toe =	0	(ft)	
b	1	(ft)	alp	19.33333	altn	0.350848	kp	2.882				
s	8	(ft)	btmph	30.500	bmptn	0.589045						
scl	7	(ft)										

Ultimate Passive Resistances and Forces

			Passive Resistance Wedge or Flow Conditions									Passive Capacity					
Toe Depth (d)	Wing Width	(Eq. 6.14) Intersection Height (hi)	(Eq. 6.13) Wedge Resistance (kips/ft)	Wedge Resistance, d = hi (kips/ft)	Wedge Resistance d = hi, α = 0 (kips/ft)	(Eq. 6.15) Intersecting Wedge Resistance (kips/ft)	(Eq. 6.16) Flow Resistance (kips/ft)	Critical Wedge or Flow Resistance (kips/ft)	(Eq. 6.17) Rankine Passive Resistance (kips/ft)	Passive Resistance (kips/ft)	Failure Mode	Total Passive Force at Given Toe Depth (kips)	Total Active Force on Toe at Given Toe Depth (kips)	Allowance for Disturbance (Eq.6.12 for depth) (kips)	Net Passive Force at a Given Depth (kips)	Factor of Safety	Toe Depth (ft)
0	0.00	-5.88	0.00	6.64	1.29	0.00	0.00	0.00	0.00	0.00	Wedge	0.00	1.12	0	-1.12	-0.06	0
1	0.60	-4.88	0.56	4.31	0.63	0.56	2.80	0.56	2.49	0.56	Wedge	0.28	1.14	0.00	-0.86	-0.05	1
2	1.19	-3.88	1.60	2.48	0.15	1.60	5.61	1.60	4.98	1.60	Wedge	1.36	2.32	0.00	-0.97	-0.05	2
3	1.79	-2.88	3.14	1.13	-0.15	3.14	8.41	3.14	7.47	3.14	Wedge	3.73	3.54	0.00	0.19	0.01	3
4	2.38	-1.88	5.17	0.28	-0.27	5.17	11.22	5.17	9.96	5.17	Wedge	7.88	4.80	0.00	3.09	0.16	4
5	2.98	-0.88	7.69	-0.08	-0.20	7.69	14.02	7.69	12.45	7.69	Wedge	14.31	6.09	0.00	8.22	0.44	5
6	3.57	0.12	10.69	0.04	0.04	10.69	16.82	10.69	14.94	10.69	Wedge	23.50	7.42	0.00	16.08	0.86	6
7	4.17	1.12	14.19	0.66	0.46	14.00	19.63	14.00	17.43	14.00	Interaction	35.84	8.79	0.00	27.06	1.44	7
8	4.76	2.12	18.18	1.77	1.07	17.48	22.43	17.48	19.92	17.48	Interaction	51.58	10.19	0.00	41.39	2.21	8
9	5.36	3.12	22.66	3.36	1.85	21.15	25.24	21.15	22.41	21.15	Interaction	70.90	11.64	0.00	59.26	3.16	9
10	5.96	4.12	27.63	5.45	2.82	25.00	28.04	25.00	24.90	24.90	Rankine	93.92	13.12	0.00	80.81	4.31	10
11	6.55	5.12	33.09	8.03	3.96	29.02	30.85	29.02	27.39	27.39	Rankine	120.07	14.63	0.00	105.44	5.63	11
12	7.15	6.12	39.04	11.10	5.29	33.23	33.65	33.23	29.88	29.88	Rankine	148.71	16.19	0.00	132.52	7.07	12
13	7.74	7.12	45.48	14.66	6.80	37.62	36.45	36.45	32.37	32.37	Rankine	179.83	17.78	0.00	162.05	8.65	13
14	8.34	8.12	52.41	18.71	8.48	42.19	39.26	39.26	34.86	34.86	Rankine	213.45	19.41	0.00	194.04	10.36	14
15	8.93	9.12	59.83	23.25	10.35	46.94	42.06	42.06	37.35	37.35	Rankine	249.56	21.08	0.00	228.48	12.19	15
16	9.53	10.12	67.75	28.28	12.40	51.87	44.87	44.87	39.84	39.84	Rankine	288.15	22.78	0.00	265.37	14.16	16
17	10.13	11.12	76.15	33.80	14.63	56.97	47.67	47.67	42.33	42.33	Rankine	329.24	24.53	0.00	304.71	16.26	17
18	10.72	12.12	85.04	39.81	17.04	62.27	50.47	50.47	44.82	44.82	Rankine	372.82	26.31	0.00	346.51	18.49	18
19	11.32	13.12	94.42	46.31	19.63	67.74	53.28	53.28	47.31	47.31	Rankine	418.88	28.12	0.00	390.76	20.86	19
20	11.91	14.12	104.30	53.30	22.40	73.39	56.08	56.08	49.80	49.80	Rankine	467.44	29.98	0.00	437.46	23.35	20
21	12.51	15.12	114.66	60.79	25.35	79.22	58.89	58.89	52.29	52.29	Rankine	518.49	31.87	0.00	486.62	25.97	21
22	13.10	16.12	125.51	68.76	28.48	85.23	61.69	61.69	54.78	54.78	Rankine	572.02	33.80	0.00	538.22	28.73	22
23	13.70	17.12	136.86	77.22	31.79	91.42	64.50	64.50	57.27	57.27	Rankine	628.05	35.77	0.00	592.28	31.61	23
24	14.29	18.12	148.69	86.18	35.28	97.80	67.30	67.30	59.76	59.76	Rankine	686.57	37.77	0.00	648.80	34.63	24
25	14.89	19.12	161.02	95.62	38.95	104.35	70.10	70.10	62.25	62.25	Rankine	747.58	39.82	0.00	707.76	37.78	25
26	15.49	20.12	173.83	105.55	42.80	111.08	72.91	72.91	64.74	64.74	Rankine	811.07	41.90	0.00	769.18	41.05	26
27	16.08	21.12	187.14	115.98	46.84	118.00	75.71	75.71	67.23	67.23	Rankine	877.06	44.01	0.00	833.05	44.46	27
28	16.68	22.12	200.94	126.89	51.05	125.09	78.52	78.52	69.72	69.72	Rankine	945.54	46.17	0.00	899.37	48.00	28
29	17.27	23.12	215.22	138.30	55.44	132.37	81.32	81.32	72.21	72.21	Rankine	1016.51	48.36	0.00	968.15	51.67	29
30	17.87	24.12	230.00	150.19	60.02	139.82	84.12	84.12	74.70	74.70	Rankine	1089.96	50.59	0.00	1039.38	55.47	30
31	18.46	25.12	245.27	162.58	64.77	147.46	86.93	86.93	77.19	77.19	Rankine	1165.91	52.86	0.00	1113.06	59.41	31
32	19.06	26.12	261.02	175.45	69.71	155.28	89.73	89.73	79.68	79.68	Rankine	1244.35	55.16	0.00	1189.19	63.47	32
33	19.66	27.12	277.27	188.82	74.82	163.27	92.54	92.54	82.17	82.17	Rankine	1325.28	57.50	0.00	1267.78	67.67	33
34	20.25	28.12	294.01	202.68	80.12	171.45	95.34	95.34	84.66	84.66	Rankine	1408.70	59.88	0.00	1348.81	71.99	34
35	20.85	29.12	311.24	217.02	85.60	179.81	98.14	98.14	87.15	87.15	Rankine	1494.61	62.30	0.00	1432.31	76.45	35
36	21.44	30.12	328.96	231.86	91.25	188.35	100.95	100.95	89.64	89.64	Rankine	1583.00	64.75	0.00	1518.25	81.03	36
37	22.04	31.12	347.17	247.19	97.09	197.07	103.75	103.75	92.13	92.13	Rankine	1673.89	67.25	0.00	1606.65	85.75	37
38	22.63	32.12	365.87	263.01	103.11	205.97	106.56	106.56	94.62	94.62	Rankine	1767.27	69.78	0.00	1697.50	90.60	38

FIGURE 95

Spreadsheet for Determining the Lateral Toe Resistance for the Two-tier, Driven Soldier Beam Wall in Example No. 1

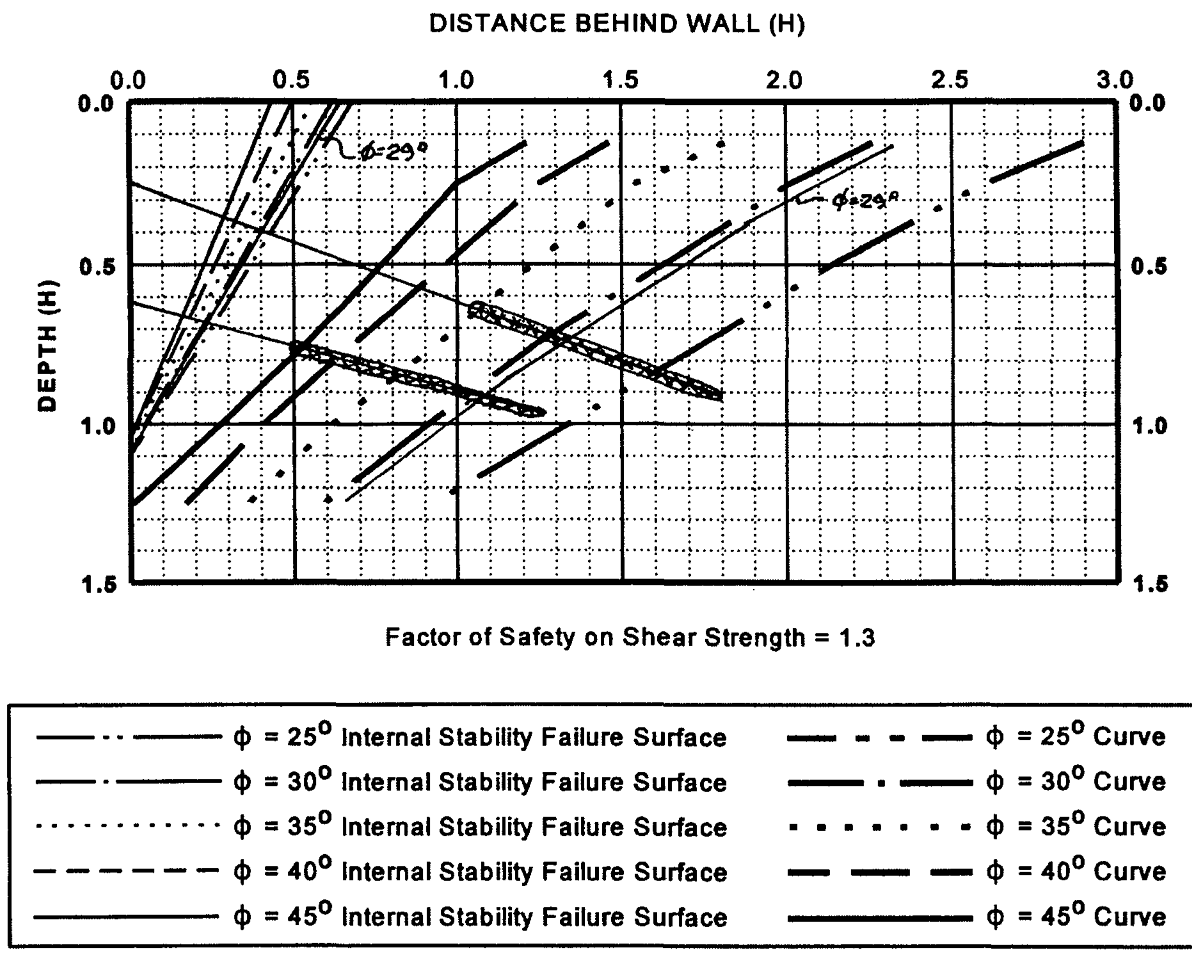


FIGURE 96
Internal and External Stability for Two-tier, Driven Soldier Beam Wall in Example No. 1

Summary of the Two-tier, Driven Soldier Beam Design.

TWO-TIER, DRIVEN SOLDIER BEAM DESIGN	
Height of Cut	30 ft
Soldier Beams	HP12×53, Grade 50
	HP14×73, Grade 36
Top Elevation	45 ft
Tip Elevation	3.5 ft
Soldier Beam Length	41.5 ft
Soldier Beam Spacing	8 ft center to center
Upper Ground Anchor	1¼-in Grade 150 Bar
Anchor Design Load	87.5 kips
Anchor Elevation	37.5 ft
Anchor Inclination	20°
Total Anchor Length	57 ft
Anchor Bond Length	24 ft
Tendon Bond Length	24 ft
Unbonded Length	33 ft
Lower Ground Anchor	1¼-in Grade 150 Bar
Anchor Design Load	87.8 kips
Anchor Elevation	26.5 ft
Anchor Inclination	15°
Total Anchor Length	39 ft
Anchor Bond Length	24 ft
Tendon Bond Length	24 ft
Unbonded Length	15 ft

Step 5b — Ground Anchor Design for Two-tier, Drilled-in Soldier Beam Wall.

1. Determine upper ground anchor inclination
   a. Ground anchor elevation = Ground Surface Elevation - H_1
      = 45 - 7.5 = Elevation 37.5 ft
   b. Center of anchoring strata = Elevation 20 ft
   c. Install ground anchor at a flat angle to keep downward load on the soldier beam low. Use a 57-ft-long ground anchor.
   d. Assuming a 24-ft-long bond length, calculate the ground anchor inclination.
      Anchor inclination = α = $\sin^{-1}((37.5 - 20)/(57 - 24/2))$ = 22.9°
      Use α = 20° for constructability.
   e. Unbonded length is (57 ft) - (24 ft) = 33 ft > 15 ft minimum recommended by the Post-Tensioning Institute (1996).

2. Determine upper ground anchor design load. Assume a soldier beam spacing of 10 ft center to center for drilled-in soldier beams.
   Upper Anchor Design Load = $(T_1)(10)/\cos 20$ = $(10{,}279)(10)/\cos 20$ = 109,387 lb
   Upper Anchor Design Load = 109.4 kips
   Use a 1¼-in Grade 150 bar. Allowable design load is (0.6)(187.5) = 112.5 kips.

3. Determine lower ground anchor inclination
   a. Ground anchor elevation = Ground Surface Elevation - H_1 - H_2
      = 45 - 7.5 - 11 = Elevation 26.5 ft
   b. Center of anchoring strata = Elevation 20 ft
   c. Use a ground anchor tendon with an anchor bond length of 24 ft and an unbonded length of 15 ft. Total ground anchor length = 39 ft.
   d. Assuming a 24-ft-long bond length, calculate the ground anchor inclination.
      Anchor inclination = α = $\sin^{-1}((26.5 - 20)/(39 - 24/2))$ = 13.9°
      Use α = 15° for constructability.
   e. Unbonded length is = 15 ft, the minimum recommended by the Post-Tensioning Institute (1996).

4. Determine lower ground anchor design load. Assume a soldier beam spacing of 10 ft center to center for driven soldier beams.
   Lower Anchor Design Load = $(T_2)(10)/\cos 15$ = $(10{,}600)(10)/\cos 15$ = 109,739 lb
   Lower Anchor Design Load = 109.7 kips
   Use a 1¼-in Grade 150 bar. Allowable design load is (0.6)(187.5) = 112.5 kips.

Step 6b -- Soldier Beam Design for the Two-tier, Drilled-in Soldier Beam Wall.

1. Assumed drilled-in soldier beams are spaced 10 ft on center.

2. Determine the size(s) of the drilled-in soldier beams. Maximum bending moment is 14,109 lb-ft/lf.

   M = (14,109 lb-ft/lf)(10 ft) = 141,090 lb-ft

   $S_{req'd}$ = ((141,090 lb-ft)(12 in/ft))/(20,000 psi) = 84.7 in^3 for Grade 36 steel

   2 C15×40 or 2 S12×40.8 or 2 W12×35 Grade 36 Soldier Beams

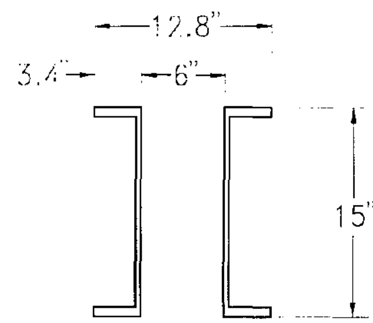
   $S_{req'd}$ = ((141,090 lb-ft)(12 in/ft))/(27,000 psi) = 62.7 in^3 for Grade 50 steel

   2 C15×33.9 or 2 S12×31.8 or 2 W12×26 Grade 50 Soldier Beams

3. Determine drilled-in soldier beam toe embedment required to resist the axial load.

   a. Determine drillhole size for a soldier beam fabricated from a pair of C15×33.9 shapes. Figure 97 shows a cross-section of the soldier beam.

FIGURE 97
Double Channel Soldier Beam for the Two-tier, Drilled-in Soldier Beam Wall in Example No. 1



   Determine diagonal distance from tip of flange to tip of flange.

   $(12.8^2 + 15^2)^{1/2} = 19.7$ in

   Use a 24-in drilled shaft for the soldier beams.

   b. Axial load applied to the toe is the vertical component of the ground anchor load plus the deadweight of the wall.

   Vertical component of the ground anchor load

   (109.4)(sin20) + (109.7)(sin15) = 65.8 kips

   Dead weight of the wall will be carried by a separate footing since the axial capacity of a drilled-in soldier beam in a medium-dense granular soil is low.

   Ultimate axial capacity of the soldier beam shall be 65.8 kips × FS = (65.8 kips)(2) = 131.6 kips.

Determine the toe penetration for a drilled-in soldier using a pair of C15×33.9 shapes in a 24-in-diameter drilled shaft. If lean-mix backfill is used, calculate the penetration using relationships for a driven beam, and calculate the penetration for a drilled shaft. The toe penetration will be the smaller of the two. Calculating the driven pile toe penetration assumes that the soldier beam punches through the lean mix.

c. Determine the axial capacity assuming the pair of C15×33.9 punches through the lean mix. Use the relationships for a driven soldier beam. In the lean mix, use K = 2, γ for the soil, δ = 35°, and N_q for the soil.

Ultimate axial resistance = Skin friction resistance + Tip resistance

$$
\begin{aligned}
Q_{ult} &= Q_s + Q_t \\
131.6 \text{ kips} &= f_s A_s + q A_t \\
&= (f_s)(4.63)(D) + (N_q)(\gamma)(D)(15/12)(12.8/12) \\
&= (f_s)(4.63)(D) + (N_q)(\gamma)(D)(1.33) \\
&= (K)(\gamma)((H+D)/2)(\tan\delta)(4.63)(D) + (N_q)(\gamma)(D)(1.33) \\
&= (2.0)(0.108)((30+D)/2)(\tan(35)(4.63)(D) + \\
&\quad (33)(0.108)(D)(1.33) \\
&= 0.3501D^2 + 15.244D
\end{aligned}
$$

D = 7.5 ft Q_{ult} = 134.0 kips > 131.6 kips OK

A toe penetration of 7.5 ft is adequate to prevent the soldier beam from punching through the lean mix backfill.

d. Determine the axial capacity assuming the pair of C15×33.9 behaves as a 24-in-diameter drilled shaft. Refer to Section 6.1.2.3.

Ultimate axial resistance = Skin friction resistance + Tip resistance

$$
\begin{aligned}
Q_{ult} &= Q_s + Q_t \\
131.6 \text{ kips} &= \int \beta \sigma dA + 1.2 N_{spt} A_t \\
&= (1.5-0.135((H+D)/2)^{1/2})\gamma((H+D)/2)\pi 2rD + \\
&\quad (1.2)(10)(\pi)r^2 \\
&= (1.5-0.135((30+D)/2)^{1/2})(.108)(30+D)\pi D + (1.2)(10)(\pi)
\end{aligned}
$$

D = 8.0 ft Q_{ult} = 131.7 kips > 131.6 kips OK

Use an 8-ft toe penetration and backfill the drilled shaft with lean-mix backfill. Axial capacity determined assuming soldier beam toe behaves as a drilled shaft.

4. Check the lateral load-carrying capacity of a soldier beam fabricated from a pair of C15×33.9 shapes. The soldier beam width will be 12.8 in since the hole is filled with lean-mix backfill. The toe depth is 8 ft. Refer to Section 6.2.1.

a. Toe reaction to be resisted is R times the soldier beam spacing.

$(R)(10 \text{ ft}) = (2342 \text{ lb/ft})(10 \text{ ft}) = 23{,}420 \text{ lb}$

b. A spreadsheet incorporating the equations from section 6.2.1 was used to determine the lateral resistance of the soldier beam toe. Figure 98 shows the results of the spreadsheet used to calculate the lateral resistance of the double channel soldier beam in a soil with a friction angle of 29° and a unit weight of 108 pcf. The beams were installed on 10-ft centers. Figure 98 shows that the soldier beam with an 8-ft toe penetration has a lateral resistance of 41.9 kips.

   (Ultimate Lateral Capacity)/(23.420) = FS

   41.9/23.420 = 1.79 > 1.5 OK

   Use a toe penetration of 8 ft.

<u>Step 7b — Internal and External Stability for a Two-tier, Drilled-in Soldier Beam Wall.</u>

The upper and lower ground anchors for the two-tier, drilled-in soldier beam wall are the same length as those for the two-tier, driven soldier beam wall. Refer to Step 7a for the internal and external stability checks for the walls.

General purpose slope stability computer programs can be used to evaluate the internal stability of an anchor wall. An anchored wall is internally stable if the anchor bond length is located behind the critical failure surface that corresponds to an FS = 1.0. Figure 99 shows the results of a stability analysis to determine the location of the critical failure surface. The ground anchor unbonded lengths are plotted on Figure 99 and they extend well beyond the critical failure surface. The wall is internally stable.

General purpose slope stability computer programs can be used to evaluate the external stability of an anchor wall. An anchored wall is externally stable if the FS for the failure surfaces that pass behind the back of the ground anchors is ≥ 1.3. Figure 100 shows the results of a stability analysis to determine the FS for the failure surfaces that pass behind the back of the lower ground anchor.

a. Determine the coordinates for the back of the lower ground anchor.

   $x = 200 + 39\cos 15 = 237.7$ ft $\qquad y = 111.5 - 39\sin 15 = 101.4$ ft

b. From Figure 100 the FS = 1.42 > 1.3

Figure 101 shows the results of a stability analysis to determine the FS for the failure surfaces that pass behind the back of the upper ground anchor. The analysis ignores he capacity of the lower ground anchor behind the failure surface.

a. Determine the coordinates for the back of the lower ground anchor.

   $x = 200 + 57\cos 20 = 253.6$ ft $\qquad y = 120 - 57\sin 20 = 100.5$ ft

b. From Figure 101 the FS = 1.81 > 1.3

   The wall is internally and externally stable.

uw	108	(pcf)	phi	29	phtn	0.554309	ko	0.52	Toe Reaction =	23.42	(kips)	(from AEP calculations)	
ht	30	(ft)	bet	59.5	betn	1.697663	ka	0.347	Depth of disturbance at toe =	0	(ft)		
b	1.067	(ft)	alp	19.33333	altn	0.350848	kp	2.882					
s	10	(ft)	btmph	30.500	bmptn	0.589045							
scl	8.933	(ft)											

Ultimate Passive Resistances and Forces

			Passive Resistance Wedge or Flow Conditions									Passive Capacity					
Toe Depth (d)	Wing Width	(Eq. 6.14) Intersection Height (hi)	(Eq. 6.13) Wedge Resistance (kips/ft)	Wedge Resistance, d = hi (kips/ft)	Wedge Resistance d = hi, ε = 0 (kips/ft)	(Eq. 6.15) Intersecting Wedge Resistance (kips/ft)	(Eq. 6.16) Flow Resistance (kips/ft)	Critical Wedge or Flow Resistance (kips/ft)	(Eq. 6.17) Rankine Passive Resistance (kips/ft)	Passive Resistance (kips/ft)	Failure Mode	Total Passive Force at Given Toe Depth (kips)	Total Active Force on Toe at Given Toe Depth (kips)	Allowance for Disturbance (Eq.6.12 for depth) (kips)	Net Passive Force at a Given Depth (kips)	Factor of Safety	Toe Depth (ft)
0	0.00	-7.50	0.00	11.30	2.58	0.00	0.00	0.00	0.00	0.00	Wedge	0.00	1.20	0	-1.20	-0.05	0
1	0.60	-6.50	0.58	8.20	1.65	0.58	2.99	0.58	3.11	0.58	Wedge	0.29	1.22	0.00	-0.93	-0.04	1
2	1.19	-5.50	1.64	5.59	0.90	1.64	5.98	1.64	6.23	1.64	Wedge	1.40	2.48	0.00	-1.08	-0.05	2
3	1.79	-4.50	3.20	3.47	0.33	3.20	8.98	3.20	9.34	3.20	Wedge	3.82	3.78	0.00	0.05	0.00	3
4	2.38	-3.50	5.25	1.84	-0.06	5.25	11.97	5.25	12.45	5.25	Wedge	8.05	5.12	0.00	2.93	0.13	4
5	2.98	-2.50	7.79	0.70	-0.27	7.79	14.96	7.79	15.56	7.79	Wedge	14.57	6.50	0.00	8.07	0.34	5
6	3.57	-1.50	10.82	0.05	-0.30	10.82	17.95	10.82	18.68	10.82	Wedge	23.88	7.92	0.00	15.96	0.68	6
7	4.17	-0.50	14.34	-0.10	-0.14	14.34	20.94	14.34	21.79	14.34	Wedge	36.46	9.38	0.00	27.08	1.16	7
8	4.76	0.50	18.35	0.23	0.19	18.31	23.94	18.31	24.90	18.31	Interaction	52.78	10.88	0.00	41.90	1.79	8
9	5.36	1.50	22.85	1.05	0.70	22.50	26.93	22.50	28.01	22.50	Interaction	73.18	12.42	0.00	60.77	2.59	9
10	5.96	2.50	27.84	2.36	1.40	26.87	29.92	26.87	31.13	26.87	Interaction	97.87	13.99	0.00	83.87	3.58	10
11	6.55	3.50	33.32	4.17	2.27	31.42	32.91	31.42	34.24	31.42	Interaction	127.01	15.61	0.00	111.40	4.76	11
12	7.15	4.50	39.29	6.46	3.32	36.15	35.90	35.90	37.35	35.90	Flow	160.68	17.27	0.00	143.40	6.12	12
13	7.74	5.50	45.75	9.25	4.56	41.06	38.90	38.90	40.46	38.90	Flow	198.08	18.97	0.00	179.10	7.65	13
14	8.34	6.50	52.70	12.52	5.97	46.16	41.89	41.89	43.58	41.89	Flow	238.47	20.71	0.00	217.76	9.30	14
15	8.93	7.50	60.15	16.29	7.57	51.43	44.88	44.88	46.69	44.88	Flow	281.85	22.49	0.00	259.36	11.07	15
16	9.53	8.50	68.08	20.54	9.34	56.88	47.87	47.87	49.80	47.87	Flow	328.23	24.31	0.00	303.92	12.98	16
17	10.13	9.50	76.50	25.29	11.30	62.52	50.86	50.86	52.91	50.86	Flow	377.60	26.17	0.00	351.43	15.01	17
18	10.72	10.50	85.42	30.52	13.44	68.33	53.86	53.86	56.03	53.86	Flow	429.96	28.07	0.00	401.89	17.16	18
19	11.32	11.50	94.82	36.25	15.75	74.32	56.85	56.85	59.14	56.85	Flow	485.31	30.01	0.00	455.30	19.44	19
20	11.91	12.50	104.71	42.47	18.25	80.50	59.84	59.84	62.25	59.84	Flow	543.65	31.99	0.00	511.67	21.85	20
21	12.51	13.50	115.10	49.18	20.93	86.85	62.83	62.83	65.37	62.83	Flow	604.99	34.01	0.00	570.98	24.38	21
22	13.10	14.50	125.97	56.37	23.79	93.39	65.82	65.82	68.48	65.82	Flow	669.32	36.07	0.00	633.25	27.04	22
23	13.70	15.50	137.34	64.06	26.83	100.11	68.82	68.82	71.59	68.82	Flow	736.64	38.16	0.00	698.47	29.82	23
24	14.29	16.50	149.19	72.24	30.05	107.00	71.81	71.81	74.70	71.81	Flow	806.95	40.30	0.00	766.65	32.73	24
25	14.89	17.50	161.54	80.91	33.45	114.08	74.80	74.80	77.82	74.80	Flow	880.26	42.48	0.00	837.77	35.77	25
26	15.49	18.50	174.38	90.07	37.03	121.34	77.79	77.79	80.93	77.79	Flow	956.55	44.70	0.00	911.85	38.93	26
27	16.08	19.50	187.70	99.72	40.79	128.78	80.78	80.78	84.04	80.78	Flow	1035.84	46.96	0.00	988.88	42.22	27
28	16.68	20.50	201.52	109.86	44.73	136.39	83.78	83.78	87.15	83.78	Flow	1118.12	49.26	0.00	1068.86	45.64	28
29	17.27	21.50	215.83	120.49	48.85	144.19	86.77	86.77	90.27	86.77	Flow	1203.39	51.60	0.00	1151.79	49.18	29
30	17.87	22.50	230.62	131.61	53.16	152.17	89.76	89.76	93.38	89.76	Flow	1291.66	53.98	0.00	1237.68	52.85	30
31	18.46	23.50	245.91	143.22	57.64	160.33	92.75	92.75	96.49	92.75	Flow	1382.91	56.40	0.00	1326.52	56.64	31
32	19.06	24.50	261.69	155.32	62.30	168.67	95.74	95.74	99.60	95.74	Flow	1477.16	58.86	0.00	1418.31	60.56	32
33	19.66	25.50	277.96	167.91	67.15	177.19	98.74	98.74	102.72	98.74	Flow	1574.40	61.36	0.00	1513.05	64.60	33
34	20.25	26.50	294.72	180.99	72.17	185.90	101.73	101.73	105.83	101.73	Flow	1674.64	63.89	0.00	1610.74	68.78	34
35	20.85	27.50	311.97	194.57	77.37	194.78	104.72	104.72	108.94	104.72	Flow	1777.86	66.47	0.00	1711.39	73.07	35
36	21.44	28.50	329.71	208.63	82.76	203.84	107.71	107.71	112.05	107.71	Flow	1884.08	69.09	0.00	1814.99	77.50	36
37	22.04	29.50	347.94	223.18	88.33	213.08	110.70	110.70	115.17	110.70	Flow	1993.29	71.75	0.00	1921.54	82.05	37
38	22.63	30.50	366.66	238.22	94.07	222.51	113.70	113.70	118.28	113.70	Flow	2105.49	74.45	0.00	2031.04	86.72	38
39	23.23	31.50	385.87	253.76	100.00	232.11	116.69	116.69	121.39	116.69	Flow	2220.68	77.19	0.00	2143.49	91.52	39
40	23.82	32.50	405.57	269.78	106.10	241.89	119.68	119.68	124.50	119.68	Flow	2338.86	79.97	0.00	2258.90	96.45	40
41	24.42	33.50	425.76	286.30	112.39	251.86	122.67	122.67	127.62	122.67	Flow	2460.04	82.79	0.00	2377.25	101.51	41
42	25.02	34.50	446.44	303.30	118.86	262.00	125.66	125.66	130.73	125.66	Flow	2584.21	85.65	0.00	2498.56	106.69	42
43	25.61	35.50	467.62	320.80	125.51	272.33	128.66	128.66	133.84	128.66	Flow	2711.37	88.54	0.00	2622.83	111.99	43
44	[illegible]	36.50	[illegible]	[illegible]	[illegible]	[illegible]	131.65	131.65	136.96	131.65	Flow	2841.52	91.48	0.00	2750.04	117.42	44

FIGURE 98
Spreadsheet for Determining the Lateral Toe Resistance for the Two-tier, Drilled-in Soldier Beam Wall in Example No. 1

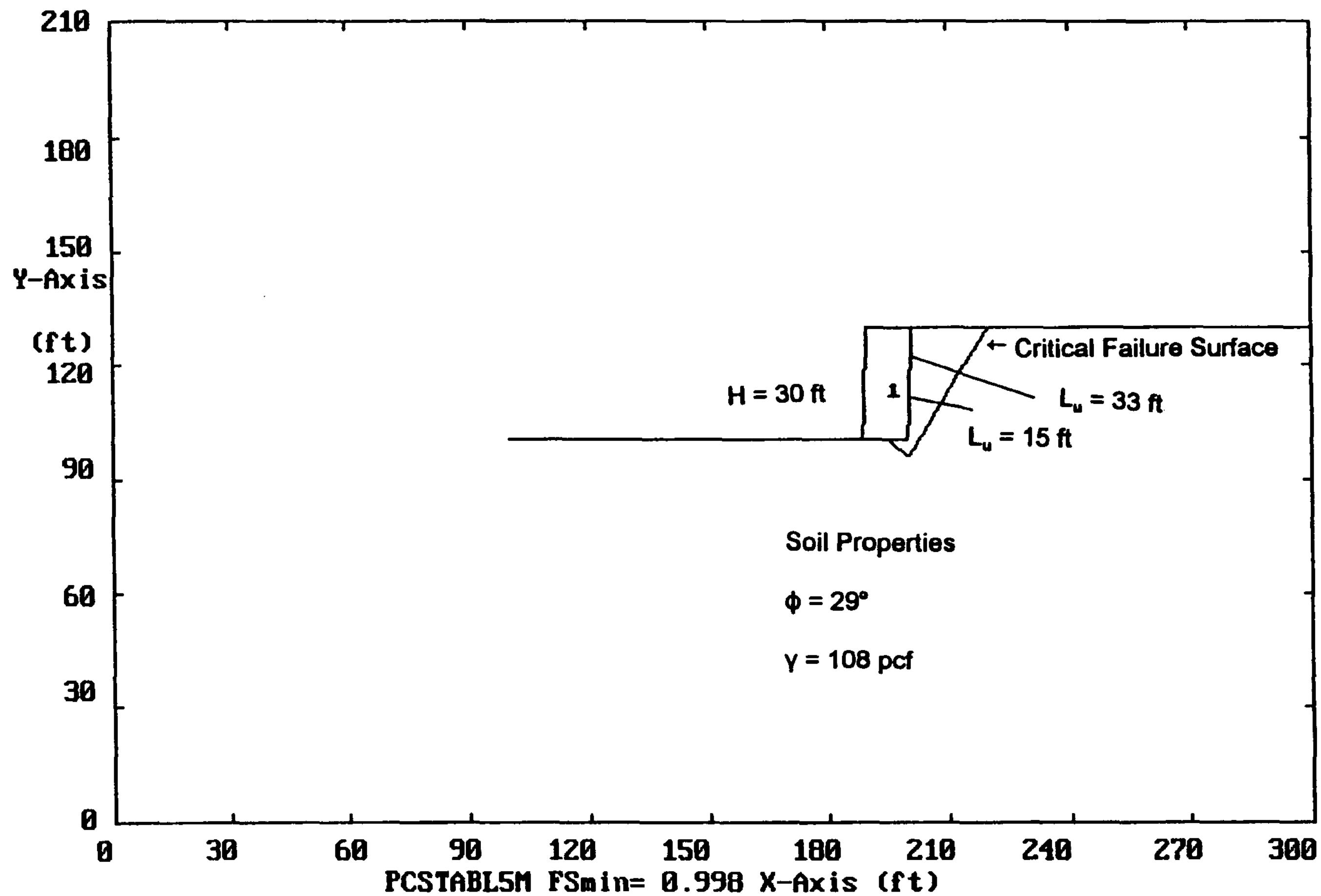


FIGURE 99
Stability Analysis for Determining Location of the Critical Failure Surface for the Wall in Example No. 1

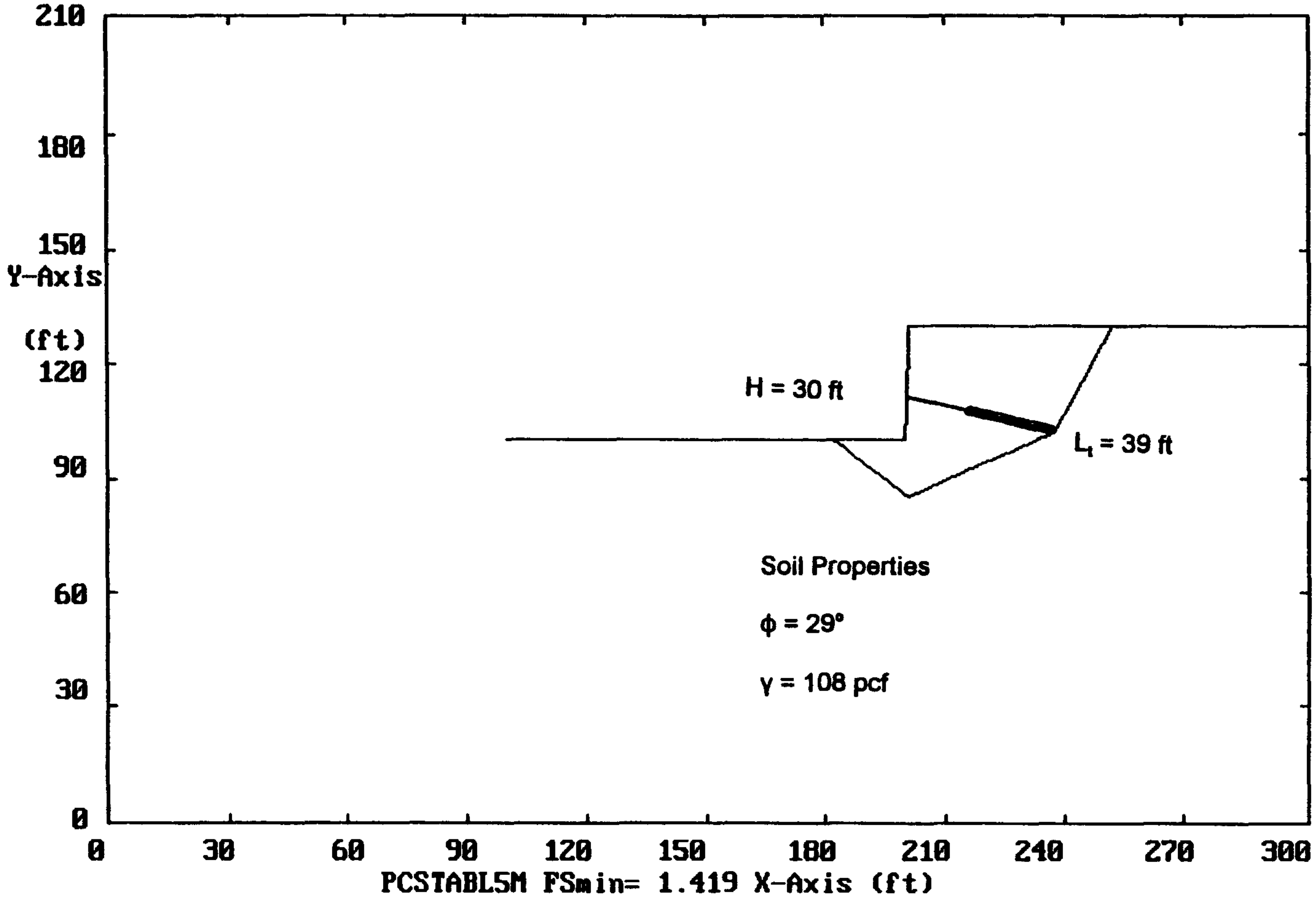


FIGURE 100
External Stability Analysis for the Critical Failure Surface Behind the Back of the Lower Ground Anchor for the Two-tier Walls in Example No. 1

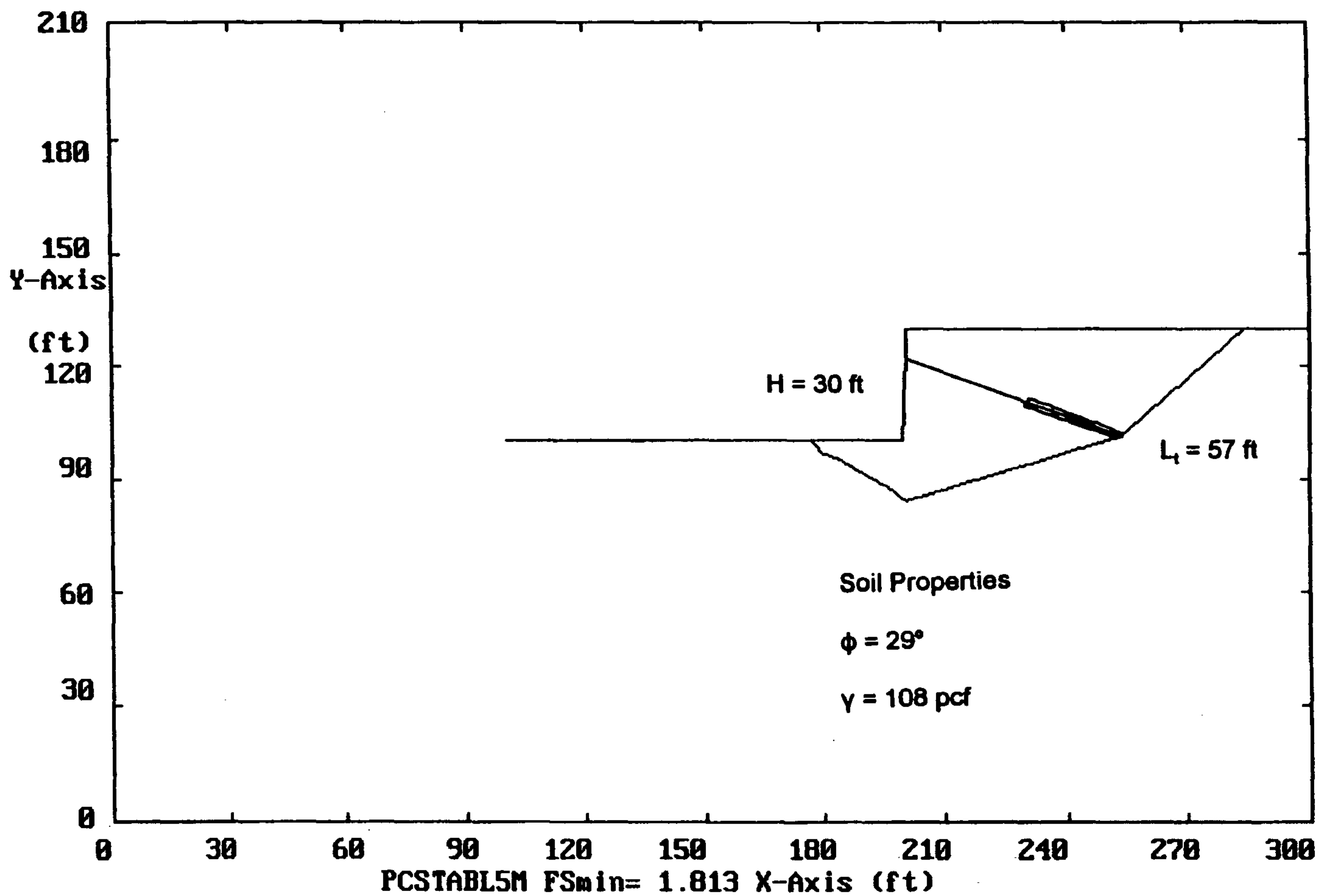


FIGURE 101
External Stability Analysis for Failure Surface Behind the Back of the Upper Ground Anchor for the Two-tier Walls in Example No. 1

Summary of the Two-tier, Drilled-in Soldier Beam Design.

TWO-TIER, DRILLED-IN SOLDIER BEAM DESIGN	
Height of Cut	30 ft
Soldier Beams	2 C15×33.9, Grade 50
Drilled Shaft Diameter	24 in
Top Elevation	45 ft
Tip Elevation	3.5 ft
Soldier Beam Length	41.5 ft
Soldier Beam Spacing	10 ft center to center
Upper Ground Anchor	1¼-in Grade 150 Bar
Anchor Design Load	109.4 kips
Anchor Elevation	37.5 ft
Anchor Inclination	20°
Total Anchor Length	57 ft
Anchor Bond Length	24 ft
Tendon Bond Length	24 ft
Unbonded Length	33 ft
Lower Ground Anchor	1¼-in Grade 150 Bar
Anchor Design Load	109.7 kips
Anchor Elevation	26.5 ft
Anchor Inclination	15°
Total Anchor Length	39 ft
Anchor Bond Length	24 ft
Tendon Bond Length	24 ft
Unbonded Length	15 ft

10.1.2 AASHTO Group VII Loading

AASHTO's *The Standard Specification for Highway Bridges* (1996) Group VII Load Combination defines the seismic loading for the design example. The Group VII Load Combination is:

$$\text{Group VII} = [D+E+B+EQ]$$

where	D	=	Dead Load
	E	=	Earth Pressure
	B	=	Buoyancy
	EQ	=	Earthquake

Allowable unit stresses for the Group VII Load Combination are increased 133 percent.

The wall must be internally and externally stable when subjected to a peak ground acceleration, A, of 0.15g. Internal stability is satisfied if the wall can support the seismically induced earth pressures without reducing the FS determined in a limiting equilibrium analysis below 1.0. When determining the internal stability of the wall, the ground anchor load is increased 133 percent. This increase is equal to the Group VII increase and it is the ratio between the ground anchor test load and the ground anchor design load. A wall is externally stable if the permanent displacements after the earthquake are acceptable.

1. In the internal stability limiting equilibrium analysis, the earthquake load is accounted for by applying a pseudo-static inertial force to the wall. The force is determined by multiplying a horizontal seismic acceleration coefficient times the mass being supported by the wall. The horizontal seismic acceleration coefficient for Mechanical Stabilized Earth (MSE) and soil nailing walls is appropriate for ground anchored walls.

   $k_h = (1.45 - A)A = (1.45\text{-}0.15)(0.15) = 0.195g.$

   where k_h = horizontal seismic acceleration coefficient
   A = peak acceleration

   Use limiting equilibrium to determine the internal stability of the wall subjected to seismic loading. The resistance of the ground anchor for the Group VII loading is 133 percent of the ground anchor load for the Group I loading. The horizontal component of the ground anchor design load is used in the analysis since the soldier beam transfers the vertical component of the ground anchor load below the bottom of the excavation.

   T_1 for the one-tier wall, Group I loading = 18,871 lb/linear ft.
   Ground anchor resistance for the Group VII loading = $18{,}871 \times 1.33 = 25{,}098$ lb.

   The seismic load is equal to the mass of the ground supported by the wall times the acceleration coefficient = 0.195g. STABL5M applies the pseudo-static seismic load to the wall and determines an FS. Figure 102 shows the results of the limiting equilibrium analysis for the wall with the seismic loading applied. The FS = 0.933 indicates that the wall is not internally stable under the seismic loading.

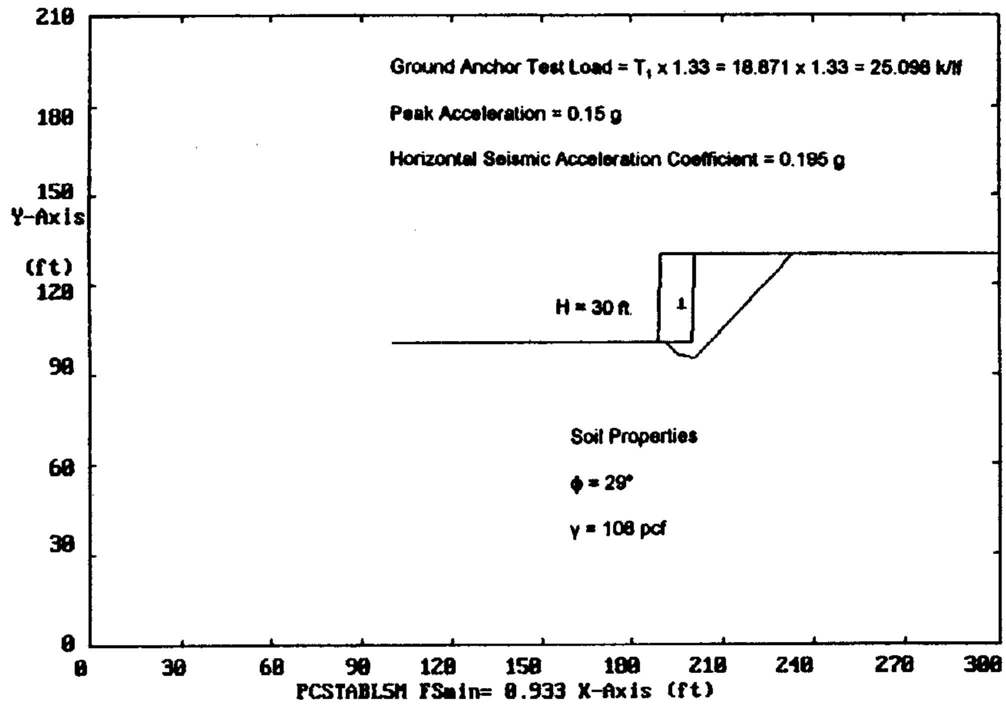


FIGURE 102
Limiting Equilibrium Analysis to Check the Internal Stability of the One-tier Wall in Example No. 1 Under Seismic Loading Associated with a Peak Acceleration of 0.15g

The ground anchor capacity and test load for the one-tier wall has to be increased to make the one-tier wall internally stable. Use limiting equilibrium to determine the ground anchor load required for internal stability of the one-tier wall. Figure 103 shows that a ground anchor test load of 27,550 lb/linear ft is required for an internally stable wall with a horizontal acceleration coefficient of 0.195g.

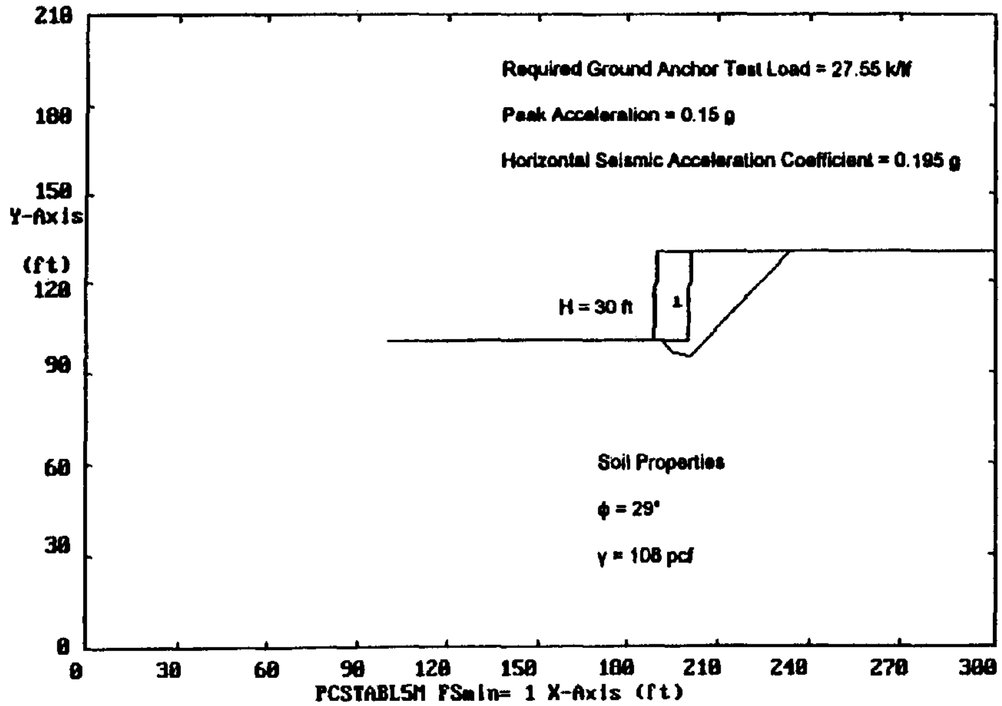


FIGURE 103
Limiting Equilibrium Analysis to Determine the Ground Anchor Load Required for Internal Stability of the One-tier Wall in Example No. 1 Under a Seismic Loading Associated with a Peak Acceleration of 0.15g

T_1 = 27,550/1.33 = 20,714 lb/linear ft.

Ground anchor design load = (20.714 k/lf)(7 ft)/cos20° = 154.3 kips

A load of 154.3 kips is greater than the allowable capacity of a 1⅜-in, Grade 150 or Grade 160 bar. Check the two-tier wall for seismic internal stability.

Determine the ground anchor load for the Group VII loading of the two-tier wall.

T_1 for the two-tier wall, Group I loading = 10,279 lb/linear ft.

T_2 for the two-tier wall, Group I loading = 10,600 lb/linear ft.

Ground anchor resistance for the Group VII loading = (T_1+T_2) (1.33)

(10,279+10,600) (1.33) = 27,769 lb

Figure 104 shows the results of the limiting equilibrium analysis for the two-tier wall with the earthquake load applied to the wall. The FS = 1.007 indicates that the wall is internally stable under the seismic loading.

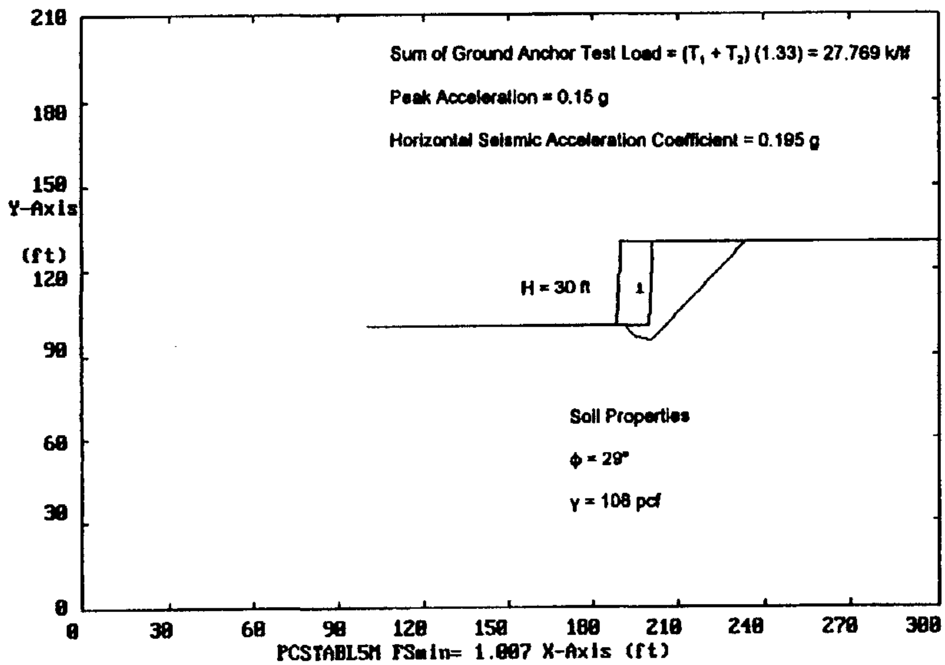


FIGURE 104
Limiting Equilibrium Analysis to Check the Internal Stability of the Two-tier Wall in Example No. 1 Under Seismic Loading Associated with a Peak Acceleration of 0.15g

Two-tier wall is internally stable without increasing the ground anchor loads.

2. External stability is evaluated using limiting equilibrium. Apply a pseudo-static inertial force to the ground mass in an external stability analysis and determine if the FS is greater than 1.0. AASHTO's *The Standard Specification for Highway Bridges* (1996) recommends using a horizontal seismic acceleration coefficient equal to half the peak acceleration for determining the external stability under earthquake loading.

   $k_h = 0.5A = 0.5(0.15) = 0.075g.$

   Figure 105 shows the external stability analysis for the wall. The external stability failure surface goes behind the back of the upper ground anchor in the two-tier wall. The FS = 1.409 with the seismic loading applied. Therefore, the wall is externally stable and the permanent wall displacements will be near zero. (If the FS had been near 1.0, the wall would have been expected to have moved 10A in or about 1.5 in.)

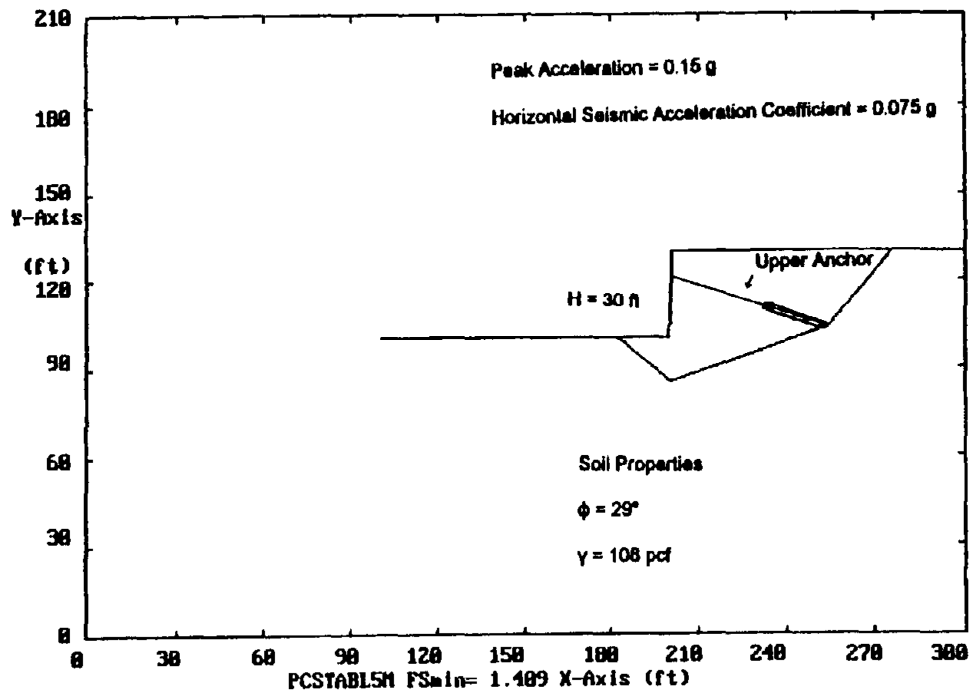


FIGURE 105
Limiting Equilibrium Analysis to Check the External Stability of the Two-tier Wall in Example No. 1 Under Seismic Loading Associated with a Peak Acceleration of 0.15g

10.2 COHESIVE SOIL DESIGN EXAMPLE

A 25-ft-high permanent ground anchor wall is to be constructed in the very stiff, silty clay deposit described in Figure 106. Laboratory tests indicated that the soil has an unconfined compressive strength of 2400 psf, a saturated unit weight of 132 pcf, a liquid limit of 37 percent, a plastic limit of 18 percent, and a natural water content of 19 percent. The overconsolidation ratio for the soil is 3. The silty clay has hydrogen ion concentration, pH, ranging between 5 and 6, resistivities ranging between 2600 and 3000 ohm-cm, and no sulfides present. The natural soil is non-aggressive. The upper 5 ft of the deposit is a fill and it is assumed to be aggressive. The ground watertable is located 10 ft below the bottom of the excavation. Bridge piers supported on spread footing are located 15 ft behind the wall.

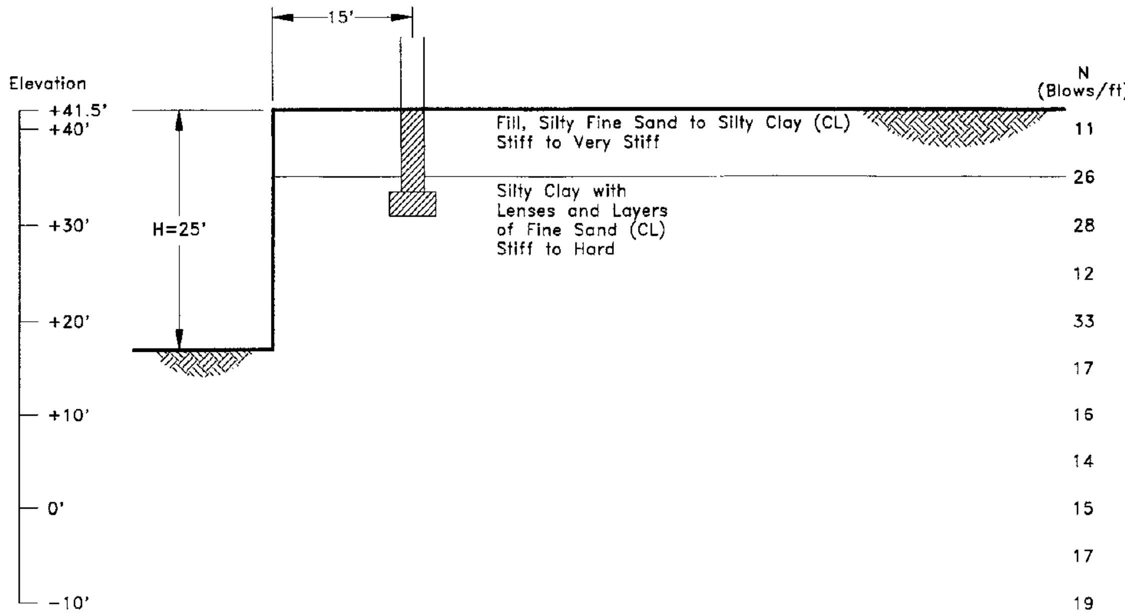


FIGURE 106
Soil Profile for Example No. 2

Bending moment reductions will not be taken. Since bending moment reductions are not taken, soldier beams are not designed for combined axial and bending stresses. The upper portion of the soldier beam is in a fill that is assumed to be aggressive. The lean-mix backfill has a high pH and will satisfactorily protect the soldier beam steel. Bending moments in the portion of the soldier beam in the fill soils are small. Substantial additional steel area is available in the fill soil.

10.2.1 AASHTO Group I Loading

The Standard Specification for Highway Bridges (1996) Group I Load Combination defines the static loading for the design example. The Group I Load Combination is:

$$\text{Group I} = [D+(L+I)+E+B]$$

where
D = Dead Load
L = Live Load
I = Live Load Impact
E = Earth Pressure
B = Buoyancy

Step 1 — Select the Ground Anchor Type, Estimate Anchor Capacity, Select Tendon Type and Corrosion Protection Requirements.

1. The fill soil is assumed to be aggressive and the depth of the fill over the site is uncertain. The silty clay is non-aggressive. Since existing bridge piers are located behind the wall, provide encapsulated anchor tendons for ground anchor corrosion protection.

2. Use hollow-stem-auger anchors installed with a 12-in-diameter auger. Locate the anchor bond length in the stiff to hard silty clay. Use an anchor inclination of 20° for the hollow-stem-augered anchors. A bar or strand tendon may be used.

3. Estimate the ultimate grout to soil bond stress.

   $s_u\alpha$ = (2400psf)(0.725) = 1740 psf

   α = reduction factor determined from load tests performed by Schnabel Foundation Company. Values of α depend upon installation procedures.

4. Allowable load transfer rate for the 12-in hollow-stem-augered anchor.

   $((1740)(1)\pi)/2 = 2733$ p/lf

5. Estimate maximum ground anchor design load. Determine the length required to place the anchor bond length behind the critical failure surface (Figure 107). Assume H_1 = 8 ft.

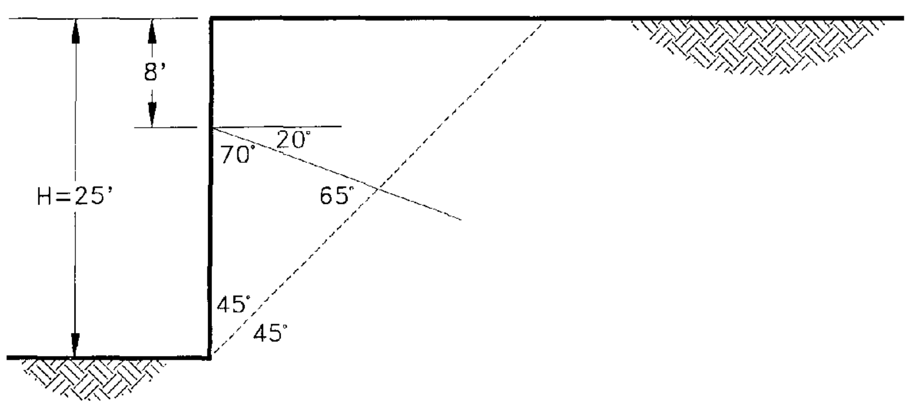


FIGURE 107
Portion of the Ground Anchor Length in Front of the Critical Failure Surface for Example No. 2

$l_{no\ load}$ = ((25-8)(sin(65+70))/(sin65) = 17sin(135)/sin65 = 13.3 ft.

Using a 60-ft-long auger. Determine maximum ground anchor bond length

60 - 13.3 = 46.7 ft.

Determine maximum allowable ground anchor design load

(2733)(46.7)/1000 = 127.6 kips

Maximum Ground Anchor Design Load = 127.6 kips

Step 2 — Determine the Total Earth Load.

1. Soil has an undrained shear strength of 2400 psf and a unit weight of 132 pcf. Use Figure 31 to determine the Earth Pressure Factor. Figure 108 shows how to use Figure 31. The Earth Pressure Factor = 20 for the silty clay. The soil has a plasticity index of 19 and an overconsolidation ratio of 3. Use Figure 38 to determine the drained friction angle for the silty clay. Figure 109 shows that the drained friction angle is approximately 36° for the silty clay. The Earth Pressure Factor for a soil with a drained friction angle of 36° is

   $0.65K_a\gamma = 0.65(\tan^2(45 - \phi/2))(132) = 22.3$ pcf

   Earth pressures will be determined using the drained shear strength of the ground.

2. Determine total earth load.

   Total earth load = (22.3 pcf)(25^2 ft^2) = 13,938 lb/lf

Step 3 — Determine Earth Pressure Resulting from Surcharge.

1. No traffic surcharge.

2. Bridge piers are located 15 ft behind the wall, and behind a one horizontal to two vertical line extending up from the bottom of the wall. Since the piers are beyond the one horizontal to two vertical line, the surcharge load from the abutment is assumed to be zero.

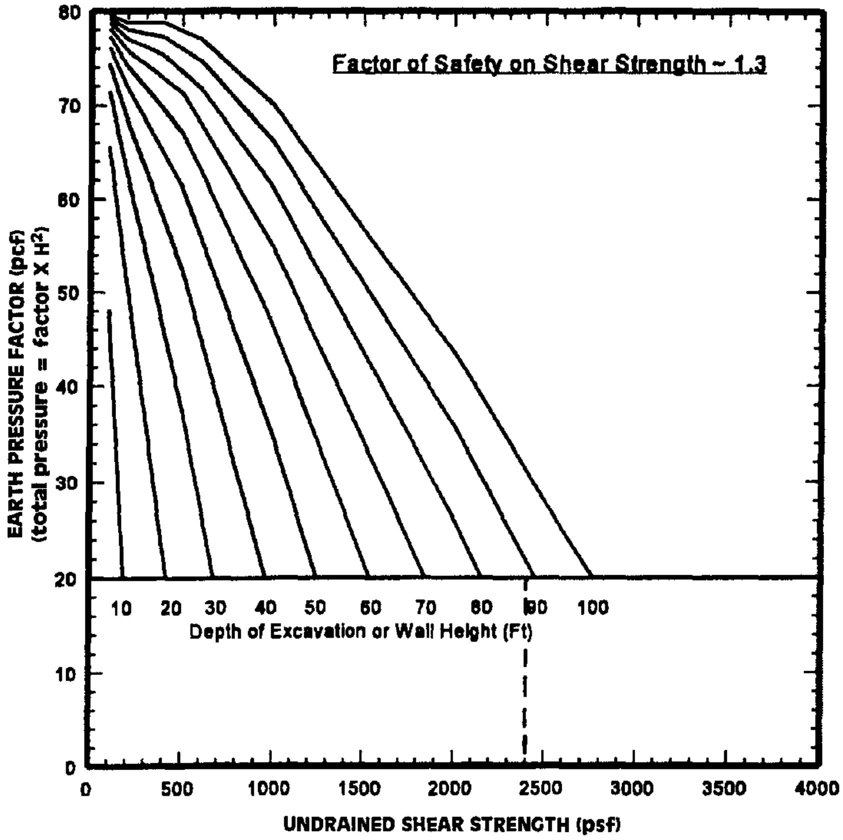


FIGURE 108
Earth Pressure Factor for Example No. 2

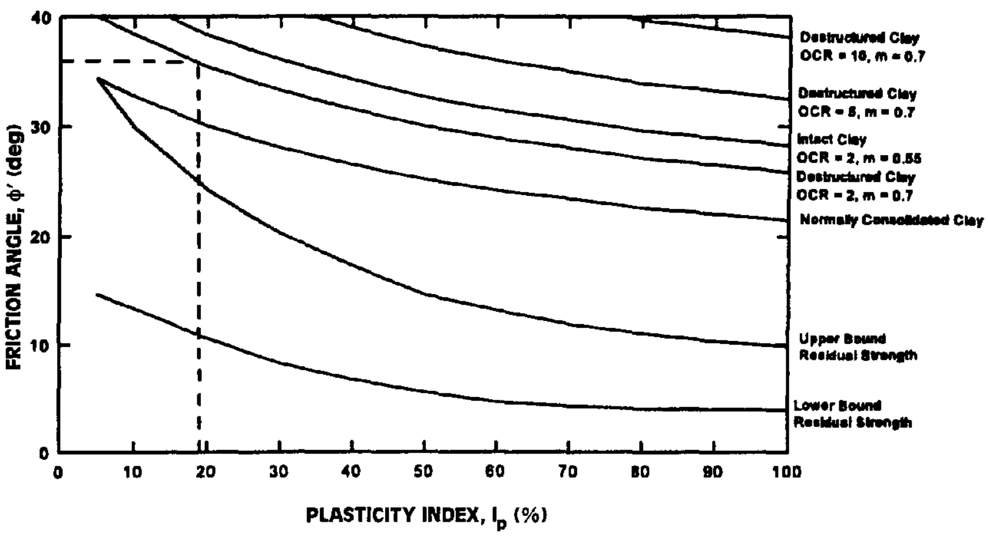
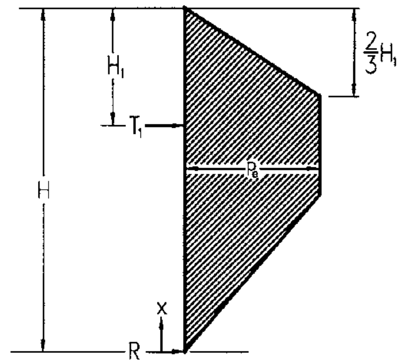


FIGURE 109
Drained Friction Angle for Example No. 2

Step 4 — One-tier Design - Determine Ground Anchor Load, Soldier Beam Moments, and Subgrade Reaction/Linear Ft of Wall.

1. Apparent earth pressure diagram

FIGURE 110
Apparent Earth Pressure Diagram for One-tier Wall in Example No. 2



p_e = (22.3 pcf)(H^2 ft^2)/((⅔)(H ft)) = (22.3 pcf)(25^2 ft^2)/((⅔)(25 ft)) = 836.2 psf/lf

2. Calculate bending moment at the ground anchor, assume H_1 = 8 ft. Figure 28 gives the equations for bending moments and loads for a one-tier wall resulting from the earth pressure, p_e.

   $M_1 = (13/54)(H_1^2)(p_e) = (13/54)(8^2)(836.2)$ = 12,884 lb-ft/lf

3. Calculate ground anchor load (Figure 28).

   $T_1 = ((23H^2 - 10HH_1)/(54(H-H_1)))(p_e) = (((23)(25^2) - (10)(25)(8))/((54)(25-8)))$ (836.2) = 11,272 lb/lf

4. Calculate the subgrade reaction (Figure 28).

   R = (⅔)(H)(p_e) - T_1 = (⅔)(25)(836.2) - 11,272 = 2665 lb/lf

5. Calculate maximum bending moment in span between ground anchor and the bottom of the excavation (Figure 28).

   Solve for, x, the location where the shear in the soldier beam is zero. Refer to Figure 110.

   $x = 1/9(26H^2 - 52HH_1)^{1/2} = 1/9((26)(25^2) - (52)(25)(8))^{1/2}$ = 8.5 ft

   Compute bending moment, MM_1.

   $MM_1 = Rx - ((p_e)(x^3))/(4(H-H_1)) = (2665)(8.5) - ((836.2)(8.5^3))/(4(25-8))$ = 15,101 lb-ft/lf

   Since $M_1 \neq MM_1$, select a new H_1 to achieve a balanced design.

6. Calculate bending moment at the ground anchor, assume $H_1 = 8.25$ ft. Figure 28 gives the equations for bending moments and loads for a one-tier wall resulting from the earth pressure, p_e.

   $M_1 = (13/54)(H_1^2)(p_e) = (13/54)(8.25^2)(836.2) = 13{,}701$ lb-ft/lf

7. Calculate ground anchor load (Figure 28).

   $T_1 = ((23H^2 - 10HH_1)/(54(H-H_1)))\,(p_e) = (((23)(25^2) - (10)(25)(8.25))/((54)(25-8.25)))$ $(836.2) = 11{,}383$ lb/lf

8. Calculate the subgrade reaction (Figure 28).

   $R = (2/3)(H)(p_e) - T_1 = (2/3)(25)(836.2) - 11{,}383 = 2554$ lb/lf

9. Calculate maximum bending moment in span between ground anchor and the bottom of the excavation (Figure 28).

   Solve for, x, the location where the shear in the soldier beam is zero. Refer to Figure 110.

   $x = 1/9(26H^2 - 52HH_1)^{1/2} = 1/9((26)(25^2) - (52)(25)(8.25))^{1/2} = 8.26$ ft

   Compute bending moment, MM_1.

   $MM_1 = Rx - ((p_e)(x^3))/(4(H-H_1)) = (2554)(8.26) - ((836.2)(8.26^3))/(4(25-8.25)) =$ $14{,}062$ lf-ft/lf

   Since $M_1 \cong MM_1$, the design is balanced.

<u>**Step 5 — Ground Anchor Design for One-tier, Drilled-in Soldier Beam Wall.**</u>

1. Use ground anchor inclination of 20° for hollow-stem-augered anchors.

2. Design load assuming drilled-in soldier beams are spaced on 10-ft centers.

   $(11.38)(10)/\cos 20 = 121.1$ kips ≤ 127.6 kips OK

   Use soldier beam spacing = 10 ft

   Use ground anchor design load = 121.1 kips

3. Determine ground anchor bond length

   $121.1/2.733 = 44.3$ ft

   Use 45-ft anchor bond length

4. Determine total anchor length, unbonded length, and tendon bond length.

   $\mathbf{L_a = 45}$ **ft**

   $\mathbf{L_b = L_a/2 = 45/2 = 22.5}$ **ft**

L_u = ((H-H1)(sin135))/sin65+ $L_a/2$ = ((25-8.25)(sin135))/sin65 + 45/2 = 34.6 ft ≅ **35 ft**

L_t = $L_u + L_b$ = **35+22.5 = 57.5 ft**

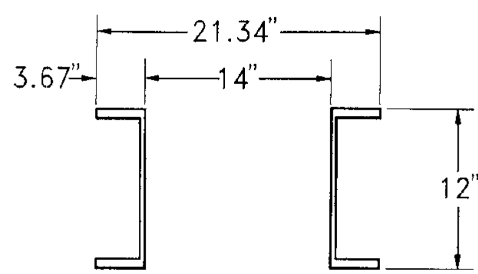
5. Determine number of 0.6-in strands required for the anchor tendon
   (Anchor design load)/(Allowable load per strand) = 121.1/((58.6)(0.6)) = 3.4 strands
   Use four 0.6-in strands for the anchor tendon.

Step 6 — Soldier Beam Design for the One-tier, Drilled-in Soldier Beam Wall.

1. Use a soldier beam spacing of 10 ft.

2. Determine the size(s) of the soldier beams
   M_1 = (14,062 lb-ft/lf)(10 ft) = 140,062 lb-ft
   $S_{req'd}$ = ((140,062 lb-ft)(12 in/ft))/(20,000 psi) = 84.0 in^3 for Grade 36 steel
   2 C15×33.9 Grade 36
   $S_{req'd}$ = ((140,062 lb-ft)(12 in/ft))/(27,000 psi) = 62.2 in^3 for Grade 50 steel
   2 MC12×31 Grade 50
   Use 2 MC12×31 Grade 50
   Determine the size of the drilled shaft for the soldier beam. Figure 111 shows the double channel soldier beam with a clear spacing of 14 in for the 12-in hollow-stem-augered ground anchors.

FIGURE 110
Soldier Beam Dimensions for Drilled-in Soldier Beam in Example No. 2



   Drilled shaft size = $([14+2(3.67)]^2+12^2)^{1/2}$ = 24.5 in
   Use 26-in-diameter drilled shaft.

3. Determine drilled-in soldier beam toe embedment required to resist the axial load.

   a. Determine the axial load that has to be carried by the soldier beam.
      Vertical component of the ground anchor load = (121.1)(sin20)
      = 41.4 kips

Dead weight of the wall will be carried by the embedded portion of the soldier beam.

Weight of the wall = (soldier beam spacing) (height of the wall) (thickness of the wall)(150 pcf)

= (10)(25)(1)(150)/1000 = 37.5 kips

Ultimate axial capacity of the soldier beam must be

(41.4 kips)+(37.5 kips) × FS = (78.9 kips)(2) = 157.8 kips.

b. Determine the axial load transferred from the drilled shaft above the bottom of the wall. Refer to 6.1.1.

$\alpha s_u A_s = (0.25)(2.4)(\pi d/2)(H-H_1) = (0.25)(2.4)(\pi(26/12)/2)(25-8.25) = 34.2$ kips

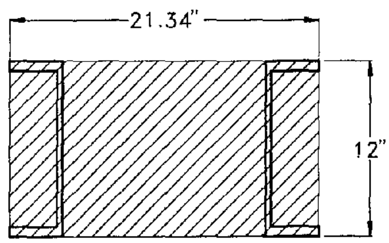
c. Determine axial load transferred to the soldier beam toe.

157.8 - 34.2 = 123.6 kips

d. Determine the toe penetration for a drilled-in soldier using a pair of MC12×31 shapes in a 26-in-diameter drilled shaft. If lean mix backfill is used, calculate the penetration using relationships for a driven beam, and calculate the penetration for a drilled shaft. The toe penetration will be the smaller of the two. Calculating the driven pile toe penetration assumes that the soldier beam punches through the lean mix.

e. Determine the toe depth assuming the pair of MC12×31 punches through the lean mix. Use the relationships for a driven soldier beam. In the lean mix, use K = 2, γ for the soil, δ = 35°, and N_c for a rectangular footing. Figure 112 shows the dimensions used to calculated the surface area and the tip area.

FIGURE 112
Block Area of the Soldier Beam Tip for the One-tier, Drilled-in Soldier Beam Wall in Example No. 1



Ultimate axial resistance = Skin friction resistance + Tip resistance

$$Q_{ult} = Q_s + Q_t$$

$$123.6 \text{ kips} = f_s A_s + q A_t$$

$$= (f_s)(5.56)(D) + (2.4)N_c(1.778)(1)$$

$= (f_s)(5.56)(D) + (2.4)(5)(1+0.2D/1)(1+0.2D/1.778)(1.778)(1)$

$= K\gamma((H+D)/2)(\tan\delta)(5.56)(D) + 21.336 + 6.667D + 0.48D^2$

$= K\gamma((H+D)/2)(\tan 35)(5.56)(D) + 21.336 + 6.667D + 0.48D^2$

$= (2.0)(0.132)((25+D)/2)(\tan(35)(5.56)(D) + 21.336 + 6.667D + 0.48D^2$

$= 12.847D + 0.514D^2 + 21.336 + 6.667D + 0.48D^2$

123.6 kips $= 0.994D^2 + 19.514D + 21.336$

D = 5 ft Q_{ult} = 143.8 kips > 123.6 kips OK

A toe penetration of 5 ft is adequate to prevent the soldier beam from punching through the lean-mix backfill.

f. Determine the axial capacity assuming the pair of MC12×33.9 behaves as a 26-in-diameter drilled shaft. N_c for a drilled shaft. Refer to Section 6.1.2.4.

Ultimate axial resistance = Skin friction resistance + Tip resistance

$Q_{ult} = Q_s + Q_t$

123.6 kips $= f_sA_s + qA_t$

$= \alpha s_uA_s + N_cs_uA_t$

$= (0.55)(2.4)(\pi d)(D) + 6.0[1+0.2(D/d)]2.4\pi d^2/2$

$= (0.55)(2.4)(\pi 26/12)D + 6.0[1+0.2(D/(26/12))]2.4\pi(26/12)^2/2$

$= 8.895D + 53.093 + 4.901D$

123.6 kips $= 13.796D + 53.093$

D=5.11 ft.

Use a 5.25-ft toe penetration for axial load

4. Check the lateral load-carrying capacity of a soldier beam fabricated from a pair of MC12×31 shapes. The soldier beam width will be 21.34 in since the hole is filled with lean-mix backfill. The toe depth is 5.25 ft. Refer to Section 6.2.2.

a. Toe reaction to be resisted is R times the soldier beam spacing.

(R)(10 ft) = (2665 lb/ft)(10 ft) = 26,650 lb

b. A spreadsheet incorporating the equations from section 6.2.2 was used to determine the lateral resistance of the soldier beam toe. Figure 113 shows the results of the spreadsheet used to calculate the lateral resistance of the double channel soldier beam in a soil with an undrained shear strength of 2400 psf and a unit weight of 132 pcf. The beams were installed on 10-ft centers. Figure 113 shows that the soldier beam with a 5-ft toe penetration has a lateral resistance of 130.51 kips.

(Ultimate Lateral Capacity)/(26.65) = FS

130.51/26.65 = 4.90 > 1.5   OK

Use a 5.25-ft toe penetration for lateral load

Step 7 — Internal and External Stability for a One-tier, Drilled-in Soldier Beam Wall.

The soil is a stiff clay. Stresses and deformations correspond to a quasielastic state instead of an limiting equilibrium state.

Practice is to locate the anchor bond length behind a critical failure surface inclined upward at $45° + \phi/2$. In clays the critical failure surface is inclined at an angle of 45°. Calculations in Step 5 selected the ground anchor lengths to satisfy internal stability.

No external stability analysis is performed since limiting equilibrium will indicated a large FS.

The wall is internally and externally stable.

γ	132	s/b	4.624297		
su	2400	α	1	Toe Reaction =	26.65 (kips) (from AEP calculations)
b	1.778			Depth of disturbance at toe =	0 (ft)
s	10				
Sc	8.222				

		Passive Resistance Wedge or Flow Conditions									Passive Capacity				
Toe Depth, d (ft)	(Eq. 6.20) Scr (ft)	(Eq. 6.19) Single Beam Wedge Resistance (kips/ft)	(Eq. 6.21) Row of Beams Wedge Resistance (kips/ft)	Critical Wedge Resistance (kips/ft)	(Eq. 6.23) Single Beam Flow Resistance (kips/ft)	Critical Wedge or Flow Resistance (kips/ft)	(Eq. 6.24) Rankine Passive Resistance (kips/ft)	Passive Resistance (kips/ft)	Failure Mode	Total Passive Force at Given Toe Depth (kips)	Allowance for Disturbance (Eq. 6.18 for depth) (kips)	Net Passive Force at a Given Depth (kips)	Factor of Safety	Toe Depth (ft)
0	0.00	8.53	67.73	8.53	46.94	8.53	48.00	8.53	Wedge	0.00	0.00	0.00	0.00	0
1	0.47	15.56	69.05	15.56	46.94	15.56	49.32	15.56	Wedge	12.05	0.00	12.05	0.45	1
2	0.93	22.59	70.37	22.59	46.94	22.59	50.64	22.59	Wedge	31.12	0.00	31.12	1.17	2
3	1.38	29.61	71.69	29.61	46.94	29.61	51.96	29.61	Wedge	57.22	0.00	57.22	2.15	3
4	1.82	36.64	73.01	36.64	46.94	36.64	53.28	36.64	Wedge	90.35	0.00	90.35	3.39	4
5	2.25	43.67	74.33	43.67	46.94	43.67	54.60	43.67	Wedge	130.51	0.00	130.51	4.90	5
6	2.68	50.69	75.65	50.69	46.94	46.94	55.92	46.94	Wedge	175.81	0.00	175.81	6.60	6
7	3.10	57.72	76.97	57.72	46.94	46.94	57.24	46.94	Flow	222.75	0.00	222.75	8.36	7
8	3.51	64.75	78.29	64.75	46.94	46.94	58.56	46.94	Flow	269.69	0.00	269.69	10.12	8
9	3.92	71.77	79.61	71.77	46.94	46.94	59.88	46.94	Flow	316.63	0.00	316.63	11.88	9
10	4.32	78.80	80.93	78.80	46.94	46.94	61.20	46.94	Flow	363.57	0.00	363.57	13.64	10
11	4.71	85.83	82.25	85.83	46.94	46.94	62.52	46.94	Flow	410.51	0.00	410.51	15.40	11
12	5.10	92.85	83.57	92.85	46.94	46.94	63.84	46.94	Flow	457.44	0.00	457.44	17.16	12
13	5.47	99.88	84.89	99.88	46.94	46.94	65.16	46.94	Flow	504.38	0.00	504.38	18.93	13
14	5.85	106.91	86.21	106.91	46.94	46.94	66.48	46.94	Flow	551.32	0.00	551.32	20.69	14
15	6.22	113.93	87.53	113.93	46.94	46.94	67.80	46.94	Flow	598.26	0.00	598.26	22.45	15
16	6.58	120.96	88.85	120.96	46.94	46.94	69.12	46.94	Flow	645.20	0.00	645.20	24.21	16
17	6.93	127.99	90.17	127.99	46.94	46.94	70.44	46.94	Flow	692.14	0.00	692.14	25.97	17
18	7.28	135.01	91.49	135.01	46.94	46.94	71.76	46.94	Flow	739.08	0.00	739.08	27.73	18
19	7.63	142.04	92.81	142.04	46.94	46.94	73.08	46.94	Flow	786.02	0.00	786.02	29.49	19
20	7.97	149.07	94.13	149.07	46.94	46.94	74.40	46.94	Flow	832.96	0.00	832.96	31.26	20
21	8.30	156.10	95.45	95.45	46.94	46.94	75.72	46.94	Flow	879.90	0.00	879.90	33.02	21
22	8.63	163.12	96.77	96.77	46.94	46.94	77.04	46.94	Flow	926.84	0.00	926.84	34.78	22
23	8.95	170.15	98.09	98.09	46.94	46.94	78.36	46.94	Flow	973.78	0.00	973.78	36.54	23
24	9.27	177.18	99.41	99.41	46.94	46.94	79.68	46.94	Flow	1020.71	0.00	1020.71	38.30	24
25	9.59	184.20	100.73	100.73	46.94	46.94	81.00	46.94	Flow	1067.65	0.00	1067.65	40.06	25
26	9.90	191.23	102.05	102.05	46.94	46.94	82.32	46.94	Flow	1114.59	0.00	1114.59	41.82	26
27	10.20	198.26	103.37	103.37	46.94	46.94	83.64	46.94	Flow	1161.53	0.00	1161.53	43.58	27
28	10.50	205.28	104.69	104.69	46.94	46.94	84.96	46.94	Flow	1208.47	0.00	1208.47	45.35	28
29	10.80	212.31	106.01	106.01	46.94	46.94	86.28	46.94	Flow	1255.41	0.00	1255.41	47.11	29
30	11.09	219.34	107.33	107.33	46.94	46.94	87.60	46.94	Flow	1302.35	0.00	1302.35	48.87	30
31	11.38	226.36	108.65	108.65	46.94	46.94	88.92	46.94	Flow	1349.29	0.00	1349.29	50.63	31
32	11.66	233.39	109.97	109.97	46.94	46.94	90.24	46.94	Flow	1396.23	0.00	1396.23	52.39	32
33	11.94	240.42	111.29	111.29	46.94	46.94	91.56	46.94	Flow	1443.17	0.00	1443.17	54.15	33
34	12.22	247.44	112.61	112.61	46.94	46.94	92.88	46.94	Flow	1490.11	0.00	1490.11	55.91	34
35	12.49	254.47	113.93	113.93	46.94	46.94	94.20	46.94	Flow	1537.05	0.00	1537.05	57.68	35
36	12.76	261.50	115.25	115.25	46.94	46.94	95.52	46.94	Flow	1583.99	0.00	1583.99	59.44	36
37	13.02	268.52	116.57	116.57	46.94	46.94	96.84	46.94	Flow	1630.92	0.00	1630.92	61.20	37
38	13.28	275.55	117.89	117.89	46.94	46.94	98.16	46.94	Flow	1677.86	0.00	1677.86	62.96	38
39	13.54	282.58	119.21	119.21	46.94	46.94	99.48	46.94	Flow	1724.80	0.00	1724.80	64.72	39
40	13.80	289.60	120.53	120.53	46.94	46.94	100.80	46.94	Flow	1771.74	0.00	1771.74	66.48	40
41	14.05	296.63	121.85	121.85	46.94	46.94	102.12	46.94	Flow	1818.68	0.00	1818.68	68.24	41

FIGURE 113
Spreadsheet for Determining the Lateral Toe Resistance for the One-tier, Drilled-in Soldier Beam Wall in Example No. 2

Summary of the One-tier, Drilled-in Soldier Beam Design.

ONE-TIER DRILLED-IN SOLDIER BEAM DESIGN	
Height of Cut	25 ft
Soldier Beams	2 MC12×31, Grade 50
Top Elevation	41.5 ft
Tip Elevation	11.25 ft
Soldier Beam Length	30.25 ft
Soldier Beam Spacing	10 ft center to center
Ground Anchor	Four 0.6-in Grade 270 Strands
Anchor Design Load	121.1 kips
Anchor Elevation	33.25 ft
Anchor Inclination	20°
Total Anchor Length	57.5 ft
Anchor Bond Length	45 ft
Tendon Bond Length	22.5 ft
Unbonded Length	35 ft

CHAPTER 11: SPECIFICATIONS

Permanent ground anchor walls are routinely built for many highway departments. These agencies and the FHWA have developed specifications for different aspects of the work. This chapter is intended to supplement those specifications and identify existing specifications that can be used for different aspects of permanent ground anchor work. Recommended specifications for contractor-prepared designs and soldier beams are included.

11.1 SPECIFYING A CONTRACTOR-DESIGNED PERMANENT GROUND ANCHOR WALL

Many highway departments require the contractor to prepare the working drawings and detailed design calculations for designated walls. A recommended specification for soliciting contractor-prepared wall designs is included in Section 11.2. When using the specification in Section 11.2, the contract documents should include:

- Suggested wall location plans with beginning and end of wall stations, easements, and construction right-of-ways identified.
- Suggested wall elevation.
- Cross-sections defining the surface and subsurface conditions in front and behind the wall.
- Existing and finished grades near the wall.
- Ground surface topography.
- Boring logs.
- Engineering properties of the soil and rock, including unit weight, shear strength parameters, and compressibility test results where appropriate.
- Groundwater conditions.
- Freezing Index expressed in "°F-days."
- Geochemical tests to determine the aggressivity of the ground.
- Surface drainage requirements.
- Barrier, coping, and drainage requirements.
- Apparent earth pressure diagrams or require the contractor to select the diagram using the shear strength parameters provided.
- Surcharge loads.
- Seismic acceleration coefficients.
- Material specification requirements.
- Anchor tendon corrosion protection requirements.

- Wall finish and color requirements.
- Maintenance of traffic requirements that affect wall construction.
- Construction tolerances for wall alignment.
- Locations for abandoned, existing, and future utilities.
- Location of existing and future structures.

Presenting the design requirements in this manner allows the owner control over the finished product and allows the contractor to use his or her experience and specialized knowledge and equipment. A schedule in the contract plans can present many of the design requirements in a clear and concise manner.

11.2 SUGGESTED SPECIFICATION FOR CONTRACTOR-DESIGNED PERMANENT GROUND ANCHOR WALL

The following specification can be used when the department solicits a contractor-designed permanent ground anchor wall as part of a normal construction contract. The specification can be modified if the department wants to obtain pre-bid contractor designs.

CONTRACTOR-DESIGNED PERMANENT GROUND ANCHOR WALL

1.0 DESCRIPTION

This work consists of preparing the design calculations and working drawings for walls identified as potential permanent ground anchor retaining walls on the contract plans. The walls shall meet the geometric requirements shown on the plans.

2.0 CONTRACTOR QUALIFICATIONS

The performance of permanent ground anchored retaining walls is strongly influenced by the experience of the Contractor. A Contractor specializing in the design and construction of permanent ground anchor walls shall prepare the final wall design and the working drawings.

The Contractor (Subcontractor) for the permanent ground anchor walls shall have a minimum of 2 years' experience in the design and construction of permanent ground anchor retaining walls. Submit proof of five permanent ground anchor walls successfully designed and built within the past 2 years. The Contractor's staff shall include a supervising engineer with a minimum of 5 years' experience in the design and construction of permanent ground anchor retaining walls. The engineer shall be a registered professional engineer licensed to perform work in the State of __________. The Contractor's project engineer and superintendent or foreman shall each have a minimum of 1 years' experience in the construction of permanent ground anchor walls.

Within 10 days after award, the Contractor shall submit documentation that the company and the personnel satisfy the qualifications. The Department will approve or reject the submission within 10 days of receipt. The Department may direct that the wall work be discontinued if unqualified personnel are substituted for approved personnel during construction. The Contractor will not be entitled to additional compensation or time if the delays are a result of furnishing unqualified personnel.

3.0 DESIGN CRITERIA

Apparent earth pressure diagrams, soil properties, safety factors, anchor tendon corrosion protection requirements, wall finish and color requirements, and barrier locations are given in the contract plans or specifications.

3.1 Soldier Beams

Determine whether the soldier beams will be driven or drilled-in. Soldier beams shall be steel sections and designed in accordance with the current edition of the AASHTO *Standard Specification for Highway Bridges*. Apparent earth pressure diagrams and surcharge pressures for each wall section are included in the contract plans. Apply barrier loads where shown on the contract plans. Axial loads applied to the soldier beam include the weight of the wall and the vertical components of the ground anchor design loads. If no bending moment reduction is applied, design the soldier beams to resist the bending stresses. If bending moment reductions are taken, design the soldier beams for bending and axial stresses. Use recommended procedures in FHWA-RD-97-130 to compute soldier beam bending moments, ground anchor loads, and toe reactions.

The axial load-carrying capacity of the soldier beam toe shall have a factor of safety of 2. Compute the applied axial load and the axial capacity of the toe in accordance with procedures in FHWA-RD-97-130.

The lateral load-carrying capacity of the soldier beam toe shall have a factor of safety of 1.5. Compute the applied lateral load (toe reaction) and the lateral load-carrying capacity of the soldier beam toe following procedures in FHWA-RD-97-130.

Refer to the contract plans for soldier beam corrosion protection requirements.

3.2 Temporary Construction Facing

Wood lagging or shotcrete can be used to temporarily support the ground between soldier beams. Lagging boards shall be a minimum of 3 in thick. Lagging does not have to be designed. Wire mesh or steel fiber reinforced shotcrete shall be a minimum of 3 in thick unless applied to support broken rock. Shotcrete can be thinner when rock is present. Shotcrete shall be designed using sound engineering principles.

3.3 Ground Anchors

Ultimate ground anchor load-carrying capacity shall be a minimum of twice the ground anchor design load. Ground anchor tendons shall be sized so the design load does not exceed 60 percent of the minimum specified tensile strength of the tendon. Contract plans show right-of-way and easements that may limit the length of the anchor that can be installed. The ground anchor shall develop its load-carrying capacity from behind the internal stability failure surface (critical failure surface). Locate the internal stability failure surface following the recommended procedures in FHWA-RD-97-130. If the contract plans give a seismic acceleration coefficient for internal stability, determine if the wall has a factor of safety of at least 1 when the acceleration and the ground anchor test load are applied to the wall. If the factor of safety is less than 1, the ground anchor load-carrying capacity must be increased. Ground anchor tendon design shall be in accordance with the current edition of the PTI *Recommendations for Prestressed Rock and Soil Anchors*.

Corrosion protection requirements for the ground anchors are given in the contract plans.

3.4 External Stability

The external stability of critical wall sections shall be checked using the procedures recommended in FHWA-RD-97-130. A minimum safety factor of 1.3 is required. If the contract plans give a seismic acceleration coefficient for external stability, determine if the ground anchor length is adequate to limit the permanent displacements to the value specified in the contract plans. Use procedures recommended in the current edition of the AASHTO *Standard Specification for Highway Bridges* or FHWA-RD-97-130.

3.5 Wall Drainage

Prefabricated drains shall be located between each soldier beam. Center drains between beams, and locate them at construction and expansion joints. If precast concrete panels are used, provide for wall drainage using prefabricated drains or backfill between the panel and the temporary construction facing with stone. See contract plans to determine whether drains will be discharged into a footing drain or weep holes.

3.6 Concrete Facing

Determine whether the facing will be precast concrete or cast-in-place concrete. Finish requirements are included in the contract plans. Refer to the contract plans for facing batters. Design the facing for the earth pressures according to the contract plans and in accordance with the current edition of the AASHTO *Standard Specification for Highway Bridges*. Cast-in-place facings shall be a minimum of 10 in thick.

4.0 SUBMITTALS

The Contractor shall prepare and submit detailed design calculations and working drawings signed and stamped by the supervising engineer. The supervising engineer shall be available any time during the life of the Contract to discuss the design with the Department. The submission shall include:

- Soldier beam schedule and design calculations giving the size, steel grade, top and tip elevations for each beam.
- Ground anchor design calculations and a schedule giving the tendon size, design load, range of anchor inclination, total length, unbonded length, and tendon bond length.
- Plan and elevation views of each wall.
- Section views as required to show anchor locations relative to utilities, structures, and right-of-way.
- Notes outlining the sequence of construction and ground anchor testing procedures.
- Design details for soldier beam fabrication, anchor tendon corrosion protection, anchorage corrosion protection, ground anchor to soldier beam connection, wall drains, facing to soldier beam connection, frost protection, and barrier details.
- Design calculations for each wall section.

Submit the detailed design drawings and calculations to the Department at least 30 days before commencement of permanent ground anchor wall work. No work or ordering of materials for the structure will be done until the Department approves the design submittal. The Department will be the sole judge of the submittal. Approval of the design submittal does not relieve the Contractor of responsibility for the successful completion of the work. Delays due to untimely submissions and/or inadequate information or details shall not be grounds for a time extension.

No additional compensation will be made for additional labor, material, or equipment necessary to comply with the project specifications as a result of the Department's review.

5.0 METHOD OF MEASUREMENT

Preparing the working drawings for the ground anchor walls will not be measured for direct payment. The cost of the working drawings will be included in the lump sum cost of the permanent ground anchor walls.

6.0 BASIS OF PAYMENT

The Contractor will be paid for preparing the design calculations and the working drawings as part of the lump sum payment for the soldier beam walls in accordance with an approved schedule of values.

11.3 SPECIFYING SOLDIER BEAMS

Specifications for driven or drilled soldier beams are not common. Soldier beams are different from driven piles or drilled shafts. A soldier beam's primary function is to support the lateral earth pressures and its secondary function is to carry the vertical component of the ground anchor load and other applied loads. When the soldier beams are drilled-in, steel members are normally installed in the drilled shafts backfilled with lean mix. A specification for soldier beams is presented in Section 11.4.

11.4 SUGGESTED SPECIFICATION FOR DRIVEN AND DRILLED-IN SOLDIER BEAMS

A suggested specification for driven and drilled-in soldier beams is presented below.

SOLDIER BEAMS

1.0 DESCRIPTION

This work consists of furnishing and installing steel soldier beams for permanent ground anchor walls in accordance with the plans, specifications, and approved working drawings. Soldier beams may be installed by driving or drilling. Driven soldier beams will be H-pile or steel sheet pile shapes. Any structural shape can be used for drilled-in soldier beams.

2.0 MATERIALS

2.1 Soldier Beam and Structural Steels

2.1.1 Steel H-Piles shall be rolled from steels conforming to AASHTO M 183 or M 223.

2.1.2 Steel Sheet Piles shall conform to AASHTO M 202.

2.1.3 Steel Plate shall conform to AASHTO M 183 or M223.

2.2 Concrete

2.2.1 Lean-mix Backfill

Lean-mix backfill for soldier beams shall consist of a minimum of one sack of Portland cement per cubic yard, fine aggregate and water. Type I or II cement may be used. Mineral or chemical admixtures can be used to improve flowability.

2.2.2 Mineral Admixtures

Mineral admixtures shall conform to requirements of AASHTO M 295.

2.2.3 Chemical Admixtures

Chemical admixtures shall conform to the requirements of AASHTO M 194.

2.2.4 Structural Concrete

Portland cement concrete, if required for soldier beam toes, shall be Class A concrete conforming to the requirements of Section _____. The target slump shall be between 5 and 8 in with the maximum slump of 9 in.

3.0 FABRICATION

Soldier beams may be shop and/or field fabricated. Welding shall conform to the requirements of AWS D1.1. Structural welding and stud welding shall be made by welders certified in accordance with AWS D1.1.

4.0 CONSTRUCTION

Complete excavation to the top of the soldier beams before installation begins. Back slopes shall be graded to final contours unless a fill will be placed behind the wall.

4.1 Driven Soldier Beams

Select a piledriving method and equipment suitable for installing the soldier beams to the tip elevation shown on the approved working drawings without damaging the soldier beam.

Crane supported leads shall support the pile hammer and soldier beam in alignment during driving. Leads shall be constructed in a manner that affords freedom of movement for the hammer and ensures that the hammer's impact energy is properly distributed to the top of the soldier beam. The pile hammers shall be selected so the soldier beam is driven to the desired depth without damaging the beam.

Reinforced points can be used to enable the soldier beams to penetrate to the desired depth.

4.2 Drilled-in Soldier Beams

Select a drilling method and equipment suitable for installing the soldier beams to the elevation shown on the approved working drawings.

4.2.1 Drilling

Uncased shafts can be used where the sides and the bottom of the shaft are stable and may be visually inspected before placing the soldier beam and the lean-mix backfill. Casing or drilling muds shall be used where the sides of the drilled shaft require support. Casing can be installed before drilling using a vibratory hammer or it can be installed using the drill.

4.2.2 Placing the Soldier Beam and Lean-mix Backfill

Lean mix shall be used to backfill the complete drilled shaft unless the working drawings show structural concrete for the soldier beam toes.

Before placing the soldier beam, remove any loose material and accumulated water. Infiltration of groundwater at a rate of less than ½-in/min and a maximum depth of water in the hole less than 2 ft will be considered a dry hole. If it is impractical to dewater the drilled shaft, a concrete pump or tremie will be used to place the lean mix or concrete.

Place the soldier beam in the drilled shaft and align it. Block or clamp the beam into position. Lean mix or concrete shall be placed as soon as possible after the soldier beam is set. If the drilled shaft is dry, place lean mix or concrete by free-falling the material down the drilled shaft and around the soldier beam. If casing is used, commence placing material before the casing is withdrawn. The casing shall be removed while the lean-mix backfill or concrete remains workable. A sufficient head of lean mix or concrete shall be maintained above the bottom of the casing to ensure that the drilled shaft is properly backfilled. Lean mix or concrete to be placed in water or slurry shall be placed through a tremie or concrete pump. The pump hose or tremie shall be withdrawn slowly as the level of the lean mix or concrete rises.

4.3 Construction Tolerances

Soldier beams shall be installed at the locations shown on the approved working drawings. They can be shifted laterally along the wall up to 12 in to avoid utilities or underground obstructions without requiring changes in the design. Soldier beams shall be within 4 in of the planned position in the horizontal plane at the plan elevation. At the bottom of the wall the soldier beam may be embedded up to 4 in into the concrete wall. If the soldier beam is in the wall, the temporary lagging or shotcrete shall be set back to provide the design wall thickness. A 15° twist is allowable in the front flange of a driven soldier beam. The top of the soldier beam shall be no more than 6 in above or 3 in below the elevation given on the approved schedule in the working drawings.

When a soldier beam deviates by more than the tolerances given above, the Contractor shall propose corrective measures to the Engineer. Corrective measures may include:

- Additional ground anchors.
- Redesigning the soldier beams.
- Adding additional soldier beams.
- Building up the soldier beam section.
- Building up the concrete facing.

5.0 METHOD OF MEASUREMENT

Mobilization for soldier beam installation equipment will be measured on a per each basis.

Furnishing and installing soldier beams will not be measured for direct payment. They will be included in the lump sum cost of the permanent ground anchor wall.

6.0 BASIS OF PAYMENT

The Contractor shall be paid for mobilization of the soldier beam installation equipment after the equipment is set up and capable of installing the soldier beams.

The Contractor will be paid for the soldier beam installation as part of the lump sum payment for the soldier beam walls in accordance with an approved schedule of values.

11.5 GROUND ANCHORS

The permanent ground anchor specification contained in AASHTO-AGC-ARTBA Task Force 27 Report (1990) should be used for permanent ground work.

11.6 PRECAST CONCRETE

Precast concrete facing panels shall be specified using Section 613.03, Concrete Face Panels, from *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects* (FHWA, 1985).

11.7 REINFORCING STEEL AND CAST-IN-PLACE CONCRETE

Highway department specifications for reinforcing steel and cast-in-place concrete should be used for the cast-in-place facing. The following specifications can be used instead of departmental specifications:

- Reinforcing steel specified according to Division II, Section 9, Reinforcing Steel, of *Standard Specifications for Highway Bridges* (AASHTO, 1996).
- Concrete specified according to Division II, Section 8, Concrete Structures, of *Standard Specifications for Highway Bridges* (AASHTO, 1996).

CHAPTER 12: A QUALITY WALL

The quality of the wall depends upon a constructible design (Chapter 5) and good construction practices and control. The best inspector is the individual doing the work. If the contractor understands the importance of the specific task and how to do it right, quality can be built into the work and the need for inspection is reduced. Inspection of the work at critical points is still necessary and it will help ensure that the desired quality is achieved. Whether the owner or the contractor inspects a specific aspect of the work will depend upon the type of specification and agency policy. If a prescriptive specification is used, the owner will inspect each aspect of the work. If a performance specification is used, the duties of the owner's inspector will be concerned with checking those aspects of the work that affect the completed wall. When a performance specification is used, the contractor selects the installation methods and the contractor is responsible for ensuring that his or her methods achieve the specified performance. The role of the owner's inspectors will change when the contractor prepares the design and builds the wall. Here the inspector will be verifying that the contractor performs in accordance with the working drawing and notes.

The items that should be checked or inspected to ensure a quality permanent ground anchor wall are the same regardless of whether the contractor or the owner is responsible for the specific aspect of the work. In this chapter, inspection responsibilities will not generally be assigned to the owner or the contractor. Where appropriate, some responsibilities have been assigned. Each owner will determine the type of contract and establish the inspection requirements that fit the contract type, labor availability, and agency policy. Specialty geotechnical contractors are often responsible for the QA/QC functions on anchored wall projects and DOT's have been satisfied with the quality of the work.

12.1 PRE-JOB PREPARATION

The owner's inspector should have an understanding of the wall design and inspection responsibilities. This includes:

- Know the subsurface and groundwater conditions at the wall location.
- Understand the wall and details shown on the working drawings.
- Know the type of specification (prescriptive specification, performance specification—owner designs the wall, performance specification—contractor designs the wall).
- Understand the inspector's and the contractor's responsibilities under the contract.
- Know planned construction sequence.
- Know location of utilities (contractor is responsible for arranging for utility location services).

- Attend a preconstruction meeting with the general contractor, excavator, and wall contractor.

12.2 UTILITY LOCATIONS

The owner will furnish utility location plans to the contractor. The contractor is responsible for contacting the utility location services to verify the location of underground utilities before starting work. Careful excavation may be required to verify the location of utility.

12.3 DRIVEN SOLDIER BEAM INSTALLATION

Driven soldier beams have been used for many permanent ground anchor walls for highways. They can be driven to tight tolerances in many grounds and are often less expensive than drilled-in soldier beams. The contractor normally is responsible for installing them to the required depth. The following actions will help achieve a quality driven soldier beam installation:

- Ensure that a firm level bench for pile driving equipment is available.
- Select a pile hammer with sufficient energy to drive the beams to the desired elevation without damage.
- Verify ground surface elevation.
- Lay out soldier beams with allowance for driving tolerances.
- Verify that soldier beam steel grade and length conform to the schedule on the working drawings.
- Submit mill certificates for the soldier beam steel to the owner.
- Position the soldier beam in the leads and move into position.
- Spot the soldier beam at the desired location and plumb the beam.
- Align the leads so the hammer impacts the beam squarely.
- Check plumbness of the soldier beam and alignment as necessary during the driving of the first few feet.
- Drive the beam to the desired tip elevation.
- Record top and tip elevation.
- Determine if the top of the beam is located within specified tolerances.
- Cut off top to desired elevation and record cut off.

12.4 DRILLED-IN SOLDIER BEAM INSTALLATION

Drilled-in soldier beams are commonly installed on highway projects in uncased drilled shafts. If the drilled shaft has to be cased, the installation becomes more difficult and soldier beam alignment is harder to control. The following actions will help achieve a quality drilled-in soldier beam installation:

- Ensure that a firm level bench for drilling equipment and service crane is available.
- Select drilling equipment suitable for the ground shown on the boring logs.
- Verify ground surface elevation.
- Lay out soldier beams.
- Verify soldier beam steel grade and length conforms to the schedule on the working drawings.
- Submit mill certificates for the soldier beam steel to the owner.
- Establish offsets to be used to locate the center of the drilled shaft.
- Position the drill over the center of the hole and plumb the kelly bar.
- Drill the hole several feet and check alignment and plumbness, and correct if necessary.
- Check for caving and groundwater. Use casing or slurry to support holes that cave.
- Recheck alignment and plumbness, and correct as necessary.
- Complete the hole to the elevation shown on the working drawings.
- Verify drilled shaft depth.
- Verify that the ground encountered is similar to the ground described in the soil borings.
- Clean the bottom of the drilled shaft and remove water if present. Two ft of water can remain in the bottom of the hole.
- If the drilled shaft cannot be dewatered, set up to place the lean mix or concrete using a concrete pump or tremie pipe.
- Place the soldier beam in the drilled shaft and align the beam.
- Secure the soldier beam in position.
- If casing or slurry is not used, pour lean mix into the shaft on each side of the beam. Prevent the beam from shifting during placement of the lean mix.
- If water or slurry is in the drilled shaft, place lean mix or concrete using a concrete pump or tremie pipe. Withdraw pump hose or tremie pipe as the mix is placed, but keep the discharge end below the surface of the mix.
- If a casing is used, fill the casing with lean mix and pull the casing.
- Record the top elevation of the soldier beam.
- Verify that the tolerances for drilled-in soldier beams are satisfied.

12.5 GROUND ANCHOR INSTALLATION

The ground anchor installation will vary from contractor to contractor and from project to project. Small differences in installation techniques can have a large impact on the load-carrying capacity of the ground anchors. The primary objective is to construct an anchor that will carry the ground anchor test load and satisfy the specified load testing acceptance criteria. Since each ground anchor will be load tested, the ultimate check of the working is the test. Unless a prescriptive specification is used, the contractor will be responsible for replacing failed anchors. Therefore, most ground anchor specifications allow the contractor to select the installation method and modify the installation method as necessary. The following will help achieve a quality ground anchor installation:

- Ensure that a firm level bench for ground anchor drill is available.
- Select a ground anchor method that will develop the load-carrying capacities on the working drawings.
- Select a drilling method that will produce a clean hole.
- Verify the location of the underground utilities and adjust ground anchors to miss utilities.
- Verify ground anchor elevation at the wall.
- Locate the drill guide at the anchor location and align the guide at the anchor inclination shown on the working drawings.
- Verify that the anchor alignment satisfies the specified tolerances.
- Drill initial holes observing the rate of penetration, hole cleaning, and characteristics of the ground.
- Verify that the ground is similar to that shown in the borings.
- Adjust drilling rate and volume of drilling fluid (air, water, polymer or slurry) to achieve optimum return of drill cuttings.
- Adjust drilling rate to prevent collaring (blockage) of the drill hole.
- Change drilling method if severe collaring continues.
- Watch for caving of the hole. If caving occurs, modify drilling method. Modifications may include switching drilling fluid or using casing.
- Drill the hole to the length shown on the approved working drawings.
- Insert grout tube or tendon with a grout tube easily to the bottom of the hole.
- Verify that the correct grout mix has been prepared.
- Grout the anchor, and record grout quantity and pressures if pressure grouting techniques are used.
- If anchors are post-grouted, record grout quantity and pressures for each grouting phase.

- After grouting is completed, align the tendon in the connection.
- Allow the tendon to remain undisturbed until the grout sets.
- Check daily to see that drilling and grouting methods do not settle or heave the ground.

12.6 ANCHOR TENDON FABRICATION AND CORROSION PROTECTION

A quality ground anchor installation requires that the anchor tendon be well protected from corrosion. Good detailing and care during construction are necessary to complete the corrosion protection of the tendon. The most important area to protect well is the anchorage area, the area just under the bearing plate. After the anchor is locked-off, it is difficult to determine if the corrosion protection under the bearing plate has been completed satisfactorily. Therefore, each step of the work has to be done well since inspecting quality into the work may not be possible. Additional information on ground anchor corrosion protection systems can be found in *Tiebacks* (Weatherby, 1982) and the *Recommendations for Prestressed Rock and Soil Anchors* (PTI, 1996). The following will help ensure that a well-constructed corrosion protection system is provided:

- Verify that corrosion protection materials conform to requirements of the approved working drawings.
- Ensure the tendon is fabricated in accordance with the approved working drawings.
- Protect the tendon from damage during handling and storage at the site.
- Securely fix the centralizers to the tendons, if centralizers are required.
- If pre-grouted tendons are used, verify that the encapsulation is fully grouted.
- Insert the corrosion protected tendon in the drill hole or casing without damaging the protection.
- Repair damage to the corrosion protection following the supplier's instructions.
- Insert the tendon to the desired depth and align the tendon so the top of the unbonded length corrosion protection is at the correct location.
- Clean the end of the tendon after grouting.
- Insert the bearing plate and trumpet over the tendon before the grout sets up if the ground anchor to soldier beam connection has been prefabricated on the soldier beam.
- Align the tendon and the anchorage before the grout sets up.
- If the connection has not been prefabricated, align the tendon and secure it in position.
- Fabricate the ground anchor to soldier beam connection to fit the installed ground anchor tendon.
- Protect the tendon during welding and burning.

- Slide the trumpet over the tendon and position it so it fits over the unbonded length corrosion protection as shown on the working drawings.
- After locking-off the anchor, fill the trumpet with grout or corrosion inhibiting compound in accordance with the working drawings. Verify that the trumpet is full.
- Cover exposed anchorages with a grout or a corrosion inhibiting compound filled cap. Verify that the cap is full.

12.7 ANCHOR TESTING

Each ground anchor is load tested to verify that it will develop the required load-carrying capacity in accordance with testing procedures on the working drawings. Performance, proof, or creep tests are used. The *Specification for Permanent Ground Anchors* (AASHTO-AGC-ARTBA Task Force 27 Report, 1990) describes each test. Typical testing setups are shown in *Tiebacks* (Weatherby, 1982). Ground anchor failure criteria are based on a creep definition of failure. A creep failure occurs when the anchor movement exceeds a specified amount during a constant load hold period. Creep failure is different from a pullout failure. Creep failure occurs at a lower load than a pullout failure. The test load must be held constant to measure creep movements accurately. Pressure gauges are used to measure anchor loads for all three tests. Load cells are used to monitor the load during the long load hold periods in the creep tests. Accurate pressure gauges are suitable for monitoring load during the load holds required for proof or performance tests. The following will help ensure that the load tests are well run:

- Allow the grout to gain sufficient strength. (Grout strength tests are not performed on most highway work. If a prescriptive specification is used, the owner may want to specify grout strength testing to verify that the contractor has mixed a quality grout.)
- Verify that the jack and pressure gauge have been calibrated in accordance with the specifications.
- Determine the jack pressures that correspond to the test loads.
- Fill out the ground anchor test sheet before starting a test. [The *Specification for Permanent Ground Anchors* (AASHTO-AGC-ARTBA Task Force 27 Report, 1990) contains sample proof, performance, and creep test sheets.]
- Ensure an independent reference point is established to measure ground anchor movements.
- Ensure that the test equipment and dial gauge are aligned.
- Load test the anchors in accordance with the testing procedures on the working drawings.
- Run performance tests on the first anchors installed on the project.
- Plot the anchor movements as the tests are performed. (Unusual behavior or errors in reading the dial gauge will be apparent if the data are plotted as the test is run.)

- Hold the ground anchor load constant during load holds.
- Do not retest ground anchors. [*Recommendations for Prestressed Rock and Soil Anchors* (PTI, 1996) describes a procedure that can be used to allow post-grouted anchors to be retested if they fail the acceptance criteria. The approach taken in the PTI's recommendations is sound, but it has not been verified extensively by experience.]
- Recognize that ground anchor failure will occur. (Failures are most likely to occur at the beginning of the job when the contractor is refining installation techniques. If frequent failures continue, the ground anchor installation methods may have to be modified or changed.)
- Verify that an anchor passes the acceptance criteria when the test is completed.
- Stress the anchors (lock-off) to the specified load. (The load will be between 75 and 100 percent of the design load.)
- Lift-off the anchor and verify that the desired load has been locked-off in the anchor before removing the test jack.

12.8 GENERAL ITEMS

Some general items associated with good practice are listed below.

- Regularly walk the site and check for signs of ground movements. (Some movement of the walls is expected.)
- Monitor the lateral and vertical movements of the soldier beam tops during construction when a structure is within a distance equal to the height of the wall.
- Control drilling fluids so they do not adversely affect areas off site.
- Develop a good working relationship between the contractor and inspector.
- Coordinate the excavation section of the specification with the ground anchor wall section.
- Collect surface water before it reaches the wall during construction.
- Control groundwater so work can continue without interruption.
- Be careful when working next to the active lanes of traffic.

CHAPTER 13: MONITORING AND INSTRUMENTATION

Good performance of anchored walls over the past 10 yr has decreased the need for monitoring. Today, few projects are instrumented. However, the performance of a permanent ground anchor wall can be evaluated by monitoring changes in anchor load and movement of the wall. Monitoring may be done during construction or it can continue for years after the wall is completed. The scope of the monitoring program will depend upon the nature of the project, experience of the department, and risk associated with the installation. A full instrumentation program is different from monitoring. An instrumentation program is undertaken to gather data that can be analyzed to improve the understanding of the behavior of the wall or the anchors.

Routine visual inspections of the wall and/or optical surveys are commonly done on all permanent ground anchor walls. Movements of ground anchor walls are generally small, and most often monitoring consists of establishing baseline lateral and vertical movement readings and routine visual inspections of the work. If the visual inspections show signs of unexpected movements, measurements of the soldier beam tops are resumed. Lateral and horizontal movements of the soldier beam tops should be measured during construction if a structure is near the wall. If unexpected movements are observed, lift-off tests can be used to estimate the ground anchor load. Normally, the load will be less than the lock-off load. A change in a load of ± 10 to 20 percent would be normal.

Monitoring of anchor load and wall movements should be considered when the department undertakes its initial projects in cohesive soils with a liquidity index greater than 0.2. Monitoring for these projects is intended to improve the department's confidence that ground anchors can be made in these soils. Experience has shown that ground anchors installed in these types of soils and tested following the recommendations in *Specification for Permanent Ground Anchors* (AASHTO-AGC-ARTBA Task Force 27, 1990) will perform satisfactorily. Monitoring should be specified when the department wants to be able to verify the long-term, load-carrying capacity of the permanent ground anchors installed on a project.

Ground anchors used for landslide stabilization have on occasion experienced significant load increases, or landslide stabilization walls have moved laterally without large changes in anchor loads. Load increases normally occur when the actual failure surface is different from the failure surface assumed in design, or the resistance along the failure surface is different from that assumed in the design. Resistance along the failure surface is affected by changes in the shear strength of the ground or changes in the location of the groundwater table. When the failure surface is long and flat, the load required to resist a landslide can be very sensitive to small changes in the shear strength of the ground. If the failure surface is behind the back of the anchor, displacements will continue and the wall and the anchors will move together without changes in anchor loads.

Anchor load should be measured with load cells, and wall deformations at the anchor location should be measured using inclinometers or optical surveys. Wall movements must be accurately measured if load measurement data are to be analyzed. Without wall movements, interrupting load cell readings is not possible. Anchor load will decrease if the anchor moves through the ground or the structure is pulled into the ground. Accurate wall movement data are necessary to separate load changes resulting from structural movements from load changes resulting from yielding of the ground anchors. When the anchors are tested according to the procedures in *Specification for Permanent Ground Anchors* (AASHTO-AGC-ARTBA Task Force 27, 1990), load changes are most often the result of structural movements rather than anchor movements.

13.1 MONITORING INSTRUMENTS

The most common monitoring instruments are load cells and inclinometers. Strain gauges are used for research purposes, but seldom for monitoring. Weatherby, et al. (1998) describe soldier beam and ground anchor instrumentation used for a research project.

13.1.1 Load Cells

Vibrating wire load cells are recommended for monitoring ground anchor load. Vibrating wire load cells installed on two Schnabel Foundation Company projects are continuing to perform satisfactorily after more than 5 yr of exposure. Electrical resistance load cells are suitable for short-term monitoring, but they have not performed well over an extended period. Electrical resistance load cell readouts are very sensitive to moisture, which makes them unsuited for typical job applications.

When installing a load cell, align the tendon in the center of the cell. Bearing plates on each side of the load cells should be at least 2 in thick to reduce bending of the plates. Load cells and bearing plates will typically require about 10 to 12 in of space. The connection and the wall near the load cell must be designed to accommodate the extra space required for the load cell. Corrosion protection of the tendon may have to be modified at load cell locations too. Corrosion inhibiting compound should fill the trumpet and the center hole so the anchor tendon is free to move in response to changes in load. The lead wire from the cell must be routed through the wall or anchorage cover and pulled through a conduit to a location where the load cell will be read.

13.1.2 Inclinometers

Inclinometers for monitoring wall movements should be located on the soldier beams and extend below the bottom of the soldier beam into stable ground that will not move in response to the excavation. The bottom of the casing must be fixed since the movements are computed relative to the position of the bottom of the casing. If the casing is too short, then the computed movements will be less than the actual movements. Dunnicliff (1988) provides guidance on the selection of the casing size and installation of the casing.

Inclinometers also can be used to verify the location of the failure surface assumed in the design of a landslide stabilization wall. If the actual failure surface is deeper than that assumed in the design, then the wall and anchors may continue to move as a block. When using inclinometer casings to verify the location of the failure surface, install the inclinometers near the back of the anchors and extend the casing down to stable ground.

13.1.3 Ground Anchor Instrumentation

Strains in the ground anchor tendons can be measured as part of a research program. Strain gauges are attached directly to bar tendons, and vibrating wire strain meters are used with strand tendons. Dunnicliff (1988) describes small, surface-mounted vibrating wire strain gauges that are suitable for attaching to bar tendons. Mueller, et al. (1998) describe vibrating wire strain meters that clamp to a strand in the tendon. When strain gauges are used, the size of the encapsulation may need to be increased, and lead wires will have to be routed through the encapsulation and the anchorage. Strain meters are long instruments and they require careful handling and protection to ensure they will function after installation.

Strains within the anchor grout are measured using vibrating wire concrete embedment gauges. Weatherby, et al. (1998) describe embedment gauges that have been successfully used on more than 50 instrumented ground anchors.

13.1.4 Soldier Beam Strain Gauges

Vibrating wire strain gauges should be used to measure strain in soldier beams. Weatherby, et al. (1998) describe the installation of the gauges. These gauges can withstand pile driving and they are stable in the moist construction environment. The gauges need to be mechanically protected under a sturdy cover if the soldier beams will be driven. Foam insulation inside the cover or other means must be used to secure the lead wires on driven beams. Loose lead wires can damage the strain gauge as they move during pile driving. Avoid welding studs to the front flange of soldier beams after the initial zero reading on the strain gauges have been taken. Welding of pairs of 0.5-in studs every 12 in induced residual compression strains on the front flanges of the instrumented soldier beams (Weatherby, et al., 1998).

13.2 FREQUENCY OF READINGS

Visual observation of the wall should be routinely done during construction so unusual behavior can be identified and corrected before the contractor completes the work. If monitoring is specified, lateral and vertical movement readings, inclinometer readings, and load cell readings are normally taken weekly during construction. After construction is completed, the frequency of the reading is reduced. Typically, readings are taken 3, 6 and 12 months after the work is completed. If no unusual measurements are detected, the readings are discontinued.

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