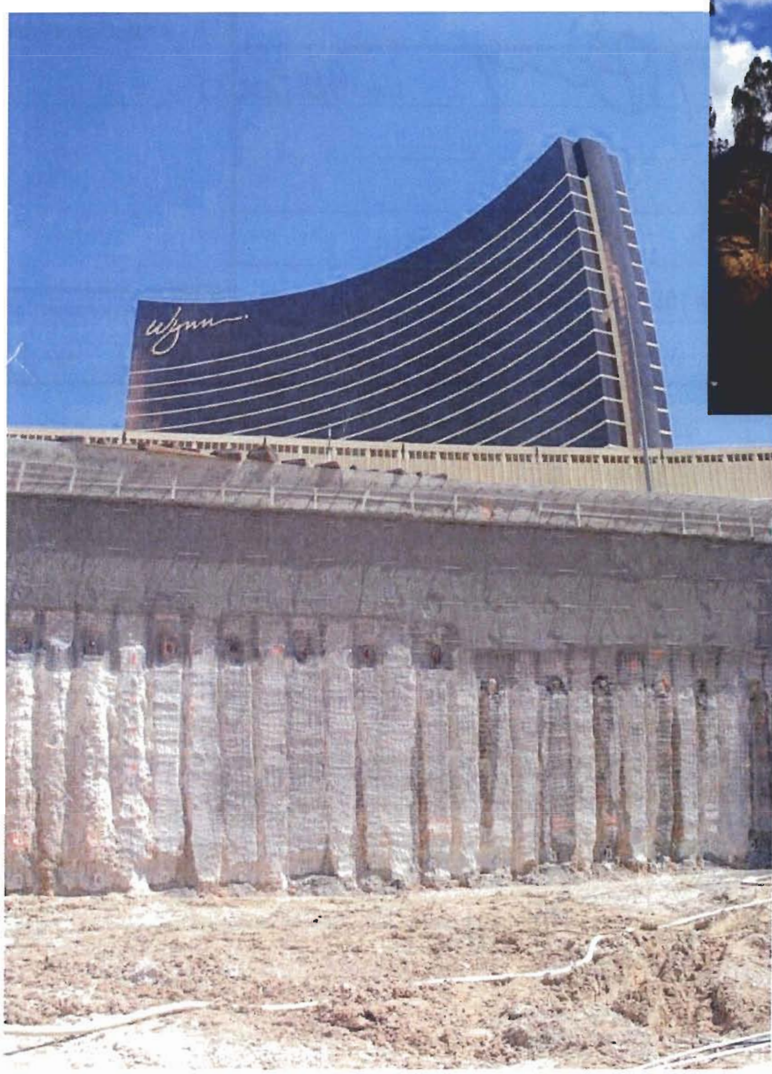
PB&A Inc.
Structural Engineering

STRUCTURAL DESIGN CRITERIA
FOR THE
PERMANENT EARTH RETENTION SYSTEM
GLENDALE ADVENTIST, MEDICAL CENTER
Acute Care Facility, Phase II



Prepared by:

PB&A, Inc.
124 Greenfield Avenue
San Anselmo, CA 94960

For:

ABBOTT

Alternate Method of Compliance
OSHPD Application # IL-040096-19A
Facility ID # 11668

OFFICE OF STATEWIDE HEALTH PLANNING AND DEVELOPMENT
FACILITIES DEVELOPMENT DIVISION www.oshpd.state.ca.gov/fdd
1600 9th Street, Room 420 ~ Sacramento, California 95814 Phone (916) 654-3362 FAX (916) 654-2973
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ALTERNATE METHOD OF COMPLIANCE

OSHPD # IL-040096-19A
Submittal # FL 001
Facility ID # 11668
DATE: August 21, 2006

A

Name of Facility: Glendale Adventist Medical Center
Email:
Address - Street: 1509 East Wilson Terrace
Phone:
FAX#:
City: Glendale County: Los Angeles Zip:
Title of Project: Glendale Adventist Medical Center - Acute Care Facility - Phase II

Name of Facility Representative/Administrator:
Email:
Address - Street:
Phone:
FAX#:
City: State: Zip:

☐ Alt Method of Compliance
☐ Program Flex
☐ Alt Method of Protection
☐ ______

B

***APPLICATION MADE BY* – Name:** Pirooz Barar **Signature:** [signature] Date: 9/27/06
Title: Structural Engineer
Address: 124 Greenfield Avenue
City: San Anselmo State: Ca. Zip: 94960
Phone: (415) 259-0191 FAX: (415) 259-0194
Who is to be known as the:
☐ Legal Owner/Administrator ☒ Agent for the Legal Owner/Administrator/Letter of Authorization must be attached

Local authority approval required.
OK ☐ N/A ☒

C

Type of Facility: ☑ General Acute Care ☐ Skilled Nursing (SNF) and Intermediate Care Facility (ICF)
☐ Psychiatric Hospital ☐ Other

D

Description of proposal : (If more space is needed, please attach a separate sheet.) Applicable Code Section
The proposed Permanent Cantilevered /Tied-back Soldier Beam and Soil Nailed Wall Earth Retention System for the Glendale Adventist Medical Center Phase II is similar to the Earth Retention System for the Phase I of the same facility. PB&A filed an application of Alternate Method of Compliance on 10/15/03for Phase I. Attached to that application was a document titled "Structural Design Criteria for the Permanent Soil Nail and Tied-back and Cantilever Earth Retention System at Glendale Adventist Medical Center. Acute Care Facility Phase I". This document was approved by OSHPD on 4/20/04.

SEE COMMENT ON PAGE #25

Reason:
The elevation of the existing grade verities from the North to the South of the site by approximate the 30 feet. In order to mitigate the effects of the soil pressures, which would be exerted on the Lateral Load Resisting System of the proposed structure, a permanent soil retention system has been proposed.

List of Enclosures:
"Structural Design Criteria for the Permanent Soil Nail and Tied-back and Cantilever Earth Retention System at Glendale Adventist Medical Center. Acute Care Facility Phase I" and revised similar document for Phase II.

E

OSHPD RECOMMENDATIONS	Date	OK	NO	N/A
Architectural Review		☐	☐	☒
Structural Review [signature]	3/6/07	☒	☐	☐
Mechanical Review		☐	☐	☒
Electrical Review		☐	☐	☒
FLSO Review		☐	☐	☒

DHS LICENSING AND CERTIFICATION RECOMMENDATIONS:
☐ OK ☐ NO ☒ N/A
Signature Date

Remarks:

F

☒ Approved ☐ Conditional Approval ☐ Denied
Signature: [signature] Date: 3-19-07

PB&A Inc.

Pirooz Barar & Associates
Structural Engineering

STRUCTURAL DESIGN CRITERIA

Permanent
Earth Retention System

Glendale Adventist
Medical Center - Acute Care Facility Phase II

Glendale, CA.
for
J.R. Abbott Construction, Inc.
IL-040096-19A

REGISTERED PROFESSIONAL ENGINEER
PIROOZ BARAR
No. SE 2502
Exp: 3/31/08
STRUCTURAL
STATE OF CALIFORNIA

Revision -
001 - 12/4/06
002 - 2/14/07

Job No. 060071

August 21, 2006

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Index

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Mr. Pirooz Barar
PB & A, Inc.
124 Greenfield Avenue
San Anselmo, CA 94960

DATE 3/19/2007
OSHPD NO IL040096-19A

PAD FL 001

Facility Name: Glendale Adventist Medical Center - Wilson Terrace-11668
Facility Location: Glendale, CA 91206
Project Description: NEW ACUTE CARE TOWER PHASE II

Re: **Alternate Method of Compliance**

Your Alternate Method of Compliance Request has been reviewed by this office to determine conformance with the standards of the Title 24, California Code of Regulations (C.C.R.).
By the copy of this transmittal we are advising you that the Permanent Cantilevered/Tied-back Solder Beam and Soil Nailed Wall Earth Retention system is found to be in accordance with applicable regulation.

OSHPD advice that this approved Alternate Method of Compliance should be placed on the cover sheet of the Increment #1 construction document.

(X) We have received one copy of the stamped documents.

REVIEWED BY	NAME	PHONE	DATE	RESULT
ARCHITECT:	Gwan Rhee	(213) 897-4094	3/19/2007	APPROVED
STRUCTURAL:	Mohammad Karim		3/6/2007	APPROVED WITH COMMENTS
MECHANICAL:	X			NOT APPLICABLE
ELECTRICAL:				NOT APPLICABLE
FLSO:	X			NOT APPLICABLE

Please contact the appropriate reviewer(s) for any questions that you may have.

Thank You,

Rhee

Gwan Rhee
Senior Architect
Ph. (213) 897-4094

cc: Project File
Administrator
Quality Assurance

Facility Administrator - 11668

OFFICE OF STATEWIDE HEALTH PLANNING AND DEVELOPMENT
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Title of Project: Glendale Adventist Medical Center - Acute Care Facility - Phase II

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☐ ____________

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FLSO Review		☐	☐	☒

DHS LICENSING AND CERTIFICATION RECOMMENDATIONS: ☐ OK ☐ NO ☒ N/A

Signature Date

Remarks:

F

☒ Approved ☐ Conditional Approval ☐ Denied Signature: [signature] Date: 3-19-07

Structural Design Criteria

This document contains design criteria for permanent Soil Nail and permanent Tied-Back and Cantilevered Soldier Beam retaining walls planned to be constructed adjacent to the existing Phase I of the Glendale Adventist Medical Center, Acute Care Facility.

Design, Construction, Quality Control / Quality Assurance shall comply with California Building Code (CCR, Title 24, Part 2), California Building Standards Administrative Code (CCR, Title 24, Part 1) and OSHPD approved Geotechnical/ Geo-hazard reports in addition to requirements of this Design Criteria.

1.0 General Project Description and Types of Earth Retention Systems

1.1 General Description of Building

The Acute Care Center extension is a two level structure below a traffic deck for vehicular/pedestrian access to existing buildings, comprising the second phase of a multi-phase hospital expansion. It is connected directly to existing structures at all levels and houses spaces for radiology, surgical support, and non storage space TBD.

The elevation of the existing grade varies from the northwest corner at about 690.0 ft., to the southwest corner at 661.0 ft. Hence, the existing change in grade is approximately 30 ft. In order to mitigate the lateral soil pressures which would be exerted on the proposed main structure, caused by the difference in grade, a permanent system of Soil Nail, Tied-Back and Cantilevered Soldier Beam retaining walls have is proposed about 3 ft. away from the main structure, freeing the Lateral Load Resisting System of the structure from resisting lateral earth pressures.

1.1a Separation of the Earth Retention System from the Building

The Earth Retention System is separated from the main structure by a gap of about 3 ft. in width. This gap is covered at the top of the wall with a concrete cap beam.

- Seismic separation between the top cap beam of the wall and the main structure is to be calculated in accordance with 2001 CBC 1633A.2.11.

- Concrete cap beam to be designed for concentrated loads in accordance with 2001 CBC 1607A.3.3 including provisions for uncontrolled vehicle access. Minimum uniform live load shall be 100 psf.

1.2 Geotechnical Considerations

The upper natural soils are alluvial deposits of varying depths, underlain by quartz diorite bedrock at depths ranging from El. 660.0 ft. at the North West corner to about 650.0 ft. 60' to the east. At the North East portion of the site there is an existing building with the elevation of the bottom of the footings at approximately 663.0 ft. This building is scheduled for demolition and the area will be backfilled with engineered fill after the soldier beams of the Earth Retention system have been installed, to the approximate elevation of 690.0 ft. which will be the final grade

The site geometry and geotechnical conditions were considered as part of the basis for the use and design of a Soil Nail wall at the North West wall, and Tied-Back and Cantilevered Soldier Beam retaining walls for the balance of the walls of the excavation.

1.2a In the areas where the demolition of the existing building leaves the grade at the bottom of the (E) building footings, (approx. El. 663.0 ft.), the soldier beams may be installed first and backfilled with engineered fill. The strength of the tiebacks may be drawn from dead men installed at appropriate levels as the engineered fill is being placed. (See Sec. 2.2.4)

1.3 Conceptual Description of Permanent Soil Nailed Wall

The basic concept of Soil Nailing, also referred to as "In Situ Reinforced Earth," is to strengthen a slope or excavation wall consisting of existing ground (Foundation Material) by means of installation of steel rods ("Soil Nails"), in grouted holes. The Nails are installed in the pattern of a matrix with a spacing not-to-exceed 6 ft. X 6 ft. The length of the Nails is typically between approximately 80% and 120% of the height of the excavation. When the Nails are installed, it creates a homogeneous and reinforced mass of soil, acting much the same as a gravity earth dam to resist the lateral pressures of the soil behind the boundaries of the Soil Nails. The passive reinforcements develop their strengthening action as the ground deforms during wall construction.

Soil Nailing uses a "top-down" construction technique with one level excavated, a row of Nails installed, and a layer of protective material applied to the face of the excavation (typically shotcrete reinforced by welded wire mesh).

1.4 Description of Tied-Back and Cantilevered Soldier Beam Wall

Tied-Back and Cantilevered Soldier Beam walls consist of discreet flexural steel elements vertically installed in drilled holes at regular intervals, hence the name "Soldier Beam." After excavation on one side of the wall is complete, horizontal timber or concrete lagging spans between the beams and helps deliver some of the horizontal soil and surcharge pressures to the Soldier Beams. The Soldier Beams draw their lateral support above the excavation from Tie-Backs, which consist of steel rods extending into the soil behind the wall, grouted in place and attached to the Soldier Beams, and from the passive resistance of the soil below the bottom of the excavation.

2.0 Method of Analysis

2.1 Soil Nailed Walls

2.1.1 Geotechnical and Structural Engineer of Record must be familiar with Permanent Earth Retention Systems in general and soil nailing in particular, and must be able to provide evidence of having authored at least two Geotechnical Reports, (in case of the Geotechnical Engineer) and be in responsible charge of the design of two projects, (in case of the Structural Engineer) of similar projects in the past five years.

2.1.2 Required Geotechnical Information

a. Suitability of Soil

Based on information provided in the Geotechnical Report, determination is to be made as to the suitability of the soil and foundation material for Soil Nailing. This determination is based on the ability of the soil to withstand an open excavation of short duration (6 to 8 hours) without experiencing extensive localized surface sloughing and instability.

b. Long-term behavior of the soil and the potential for creep in clay soils

c. Unit Weight of the Soil (γ) in pcf

d. Cohesion of the Soil (C) in psf

e. Angle of Soil Friction (φ) in degrees

f. Pull-Out Strength, adhesion (μ) in psf

   Discussion – The pullout strength of the soil and adhesion between grouted Soil Nails and soil is not strictly a geotechnical parameter, although related to the shear strength of the soil, it also depends on the methods of drilling and construction techniques. This value is to be included in the geotechnical report and shall be readily verified in the field. See Section, 3.1.1.3 Soil Nail Wall Testing Requirements.

g. Corrosiveness of the soil

h. Location of the Phreatic Surface

i. Horizontal and vertical Seismic Pseudo-Static Coefficient and the basis for their determination.

2.1.2 Analytical Methods

The basic analysis procedure for Soil Nail walls is based on the assumption that the failure occurs on a bilinear failure surface. This assumption is verified in studies performed at UCLA(1).

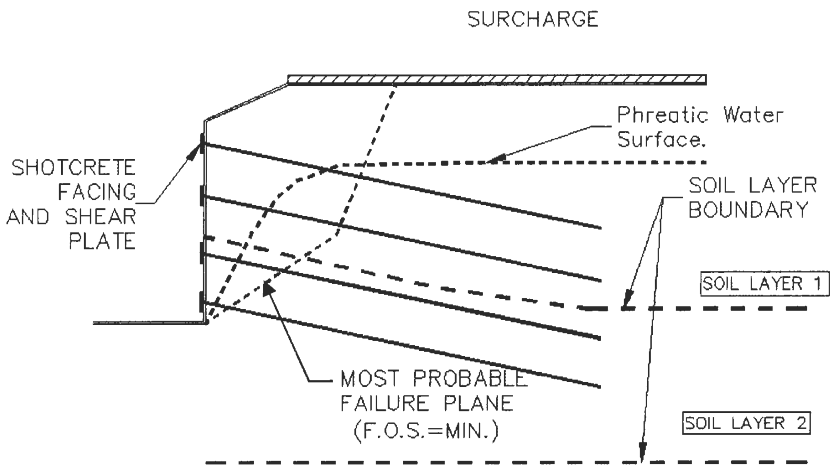


Figure 2.1.2-1: A Typical Soil Nail Wall

The analysis should consider all possible failure surfaces and find the most critical (lowest safety factor condition). A typical Soil Nail wall is shown in Figure 2.1.2-1, above.

The factor of safety is calculated by balancing the driving and resisting forces acting on the wedge of soil in front of the assumed potential failure surface. The forces that are in effect in such an analysis include the weight, the soil resistance from cohesion and internal friction (C and φ) and the forces from the Nail. In addition, the earthquake effect and the effect of ground water are considered. This analysis procedure has been in used in several existing computer programs.

2.1.2.1 Calculation of Safety Factor for a Given Trial Failure Plane

The basic analysis assumption is that the failure condition is made of two wedges. It is assumed that the two wedges (as shown in Figure 2.1.2.1-1) fail with a vertical plane.

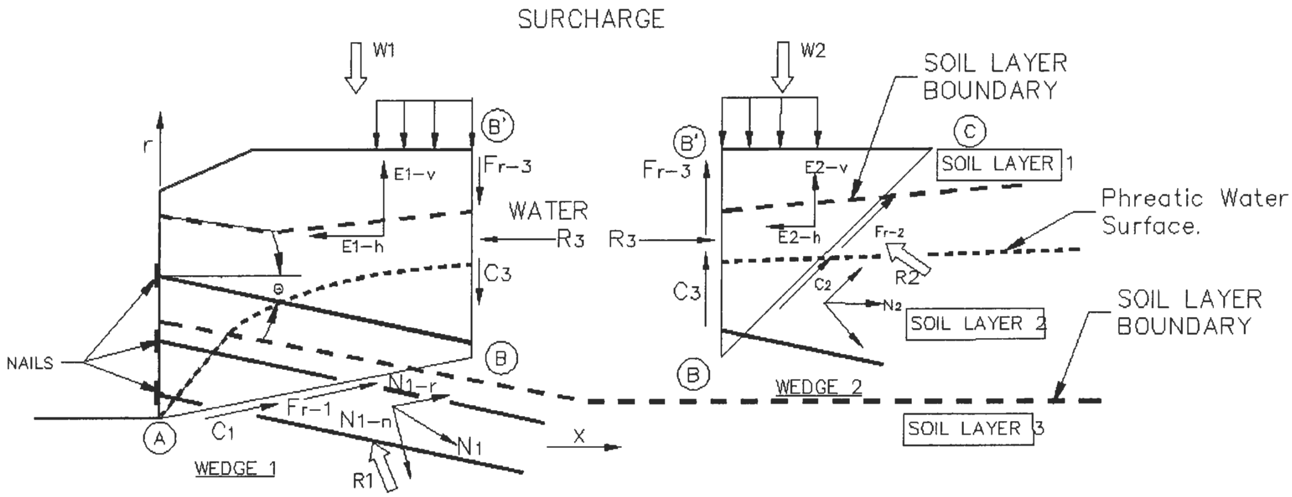


Figure 2.1.2-1-1: FAILURE SURFACE AND FORCES;

The assumption is that the lower wedge (Wedge 1) will move outward and the upper wedge (Wedge 2) will move downward. The forces acting on the total system include weight (W), Nail forces (N), Friction (Fr), Cohesion (C) and Normal force at the intersection (R). Considering Wedge 1 and Wedge 2, the following forces will act on the system, as well as on the interface between the two wedges. Note that the failure surface is called ABC, where A is the toe, C is on the surface, and B is the interface between the two planes. Additionally, the location on the surface directly above point B is called B'.

Note that these forces are all known with the exception of the normal interaction forces (R1, R2 and R3). The safety factor is defined as the reduction factor applied to the resistive forces (Friction, Cohesion and Nail) until the system is in equilibrium, considering the driving forces (Weight, Earthquake and Interaction, R). Considering the three unknown forces and the unknown safety factor, there are a total of four unknowns. Using the equilibrium

equations for forces on each wedge in horizontal and vertical directions, four equations are available to solve for the four unknowns.

Table 2.1.2.1-1: List of Forces Acting on the Slope

Notation	Description	Direction	Act on Wedge	Condition	D/R*
W1	Weight of Wedge 1	Vertical (downward)	1	Known	D
N1-N	Total force from all nail (Component)	Normal to edge AB	1	Known	R
N1-P	Total force from all nail (Component)	Parallel to edge AB	1	Known	R
R1	Normal interaction force along AB	Normal to edge AB	1	Unknown	D
Fr1	Friction force	Parallel to edge AB	1	Known	R
C1	Cohesion force	Parallel to edge AB	1	Known	R
E1-H	Earthquake force	Horizontal	1	Known	D
E1-V	Earthquake force (Uplift)	Vertical (upward)	1	Known	D
R3	Interaction force along BB'	Normal to edge BB'	1&2	Unknown	D
Fr3	Friction force	Parallel to edge BB'	1&2	Known	R
C3	Cohesion	Parallel to BB'	1&2	Known	R
W2	Weight of Wedge 2	Vertical (downward)	2	Known	D
N2-N	Total force from all nail (Component)	Normal to edge BC	2	Known	R
N2-P	Total force from all nail (Component)	Parallel to edge BC	2	Known	R
R2	Normal interaction force along AB	Normal to edge BC	2	Unknown	D
Fr2	Friction force	Parallel to edge BC	2	Known	R
C2	Cohesion force	Parallel to edge BC	2	Known	R
E2-H	Earthquake force	Horizontal	2	Known	D
E2-V	Earthquake force (Uplift)	Vertical (upward)	2	Known	D

*D = Used Directly, R = Reduced by safety factor

The solution scheme considers the equilibrium of each wedge and the fact that it has three unknowns. For example, Wedge 1 has R1, R3 and f (safety factor) as unknowns. Assuming a value for the safety factor (f) the other two can be obtained. Therefore a value for R3 is calculated. The same is repeated for Wedge 2, and another value for R3 is obtained. The correct solution is obtained when the two values of R3 are the same. This is obtained by "trial and error" (iterations), i.e., different values of f are tried until the R3 value from Wedge 1 and Wedge 2 are within an acceptable tolerance.

Note: It is possible that in some cases an acceptable safety factor cannot be obtained due to the numerical nature of the solution. If after a maximum specified number of iterations a solution is not reached, it usually means that the safety factor is too low, and the solution should be revised by providing more resistance, usually achieved by providing more Soil Nails.

Each one of the known forces is calculated based on the geometry and properties of the site, and are explained below:

a. Calculation of Weight

The site may contain several layers of soil, and each layer may have different properties, e.g. unit weight, at each layer. The weight of each wedge is calculated based on the soil that is contained within each wedge. Note that if there is any surcharge force that lies directly above each wedge, then the surcharge force is directly added to the weight of the wedge.

b. Calculation of Nail Force

Each Nail may have its own direction and length. The maximum force that can be developed in each Nail is controlled by three actions. These include the

pull-out from the wall end or the tip, and the tensile yield failure in the bar. The pull-out force is dependent on the maximum or failure bond strength between the soil and the Nail, which is a constant value, multiplied by the total contact area between them. This contact area is the perimeter of the Nail multiplied by the length in contact. The maximum tension is calculated as the yield stress of the rod (reinforcing bar) multiplied by the area of the rod. The force diagram for a typical Nail is shown in Figure 2.1.2.1-2 below.

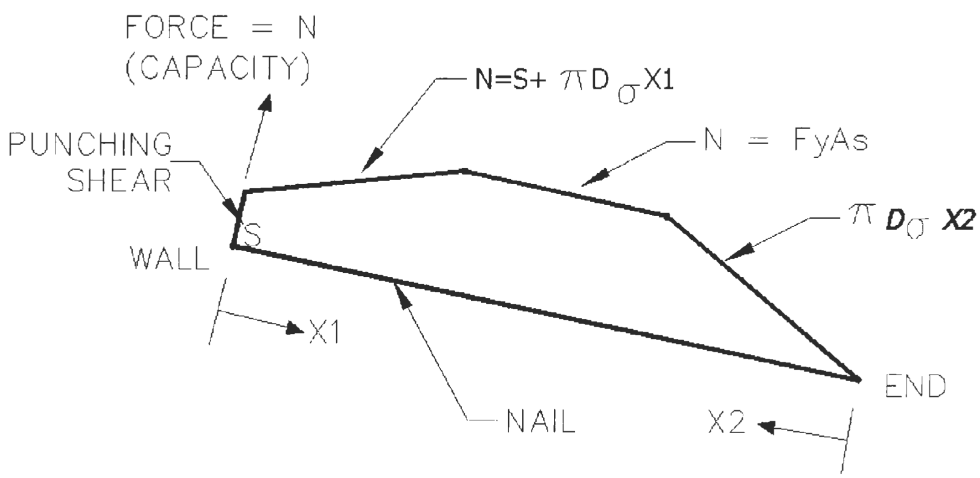


Fig. 2.1.2.1-2: Nail Force Calculation

The pull-out force at the end of the Nail is calculated as the contact area, multiplied by the length of the Nail from the failure plane to the tip of the Nail. This may be shown as: $N_e=\sigma*\pi D*L_{n2}$, where, σ is the bond stress (also sometimes shown as C'), D is diameter of the hole, and L_{n2} is the distance from the point of intersection of the Nail and the failure plane to the end of the Nail. Note that if this portion of the Nail is in contact with more than one soil layer, then the total pull-out force is calculated as the sum of the pull-out forces along the length under consideration.

The pull out force at the wall end of the Nail is calculated in a similar fashion, however, due to the connection of the Nail and wall, the punching shear capacity of the wall can be added to this capacity, i.e. $N_W=\sigma*\pi D*L_{n1}+S$. In this case L_{n1} is the distance from the wall to the point of intersection of the Nail and the failure plane, and S is the punching shear capacity of the wall.

The maximum tensile capacity of the Nail depends on the amount and type of rod contained within the Nail, i.e., $N_T=f_y*\pi r^2$. In this case, f_y is the yield stress of the rod, and r is the radius of the rebar.

The controlling Nail force is then calculated as the minimum of the three forces calculated above: $N=\min(N_e, N_w, N_T)$. This force acts in the direction of the Nail. Therefore, if the Nail has an inclination angle of θ, then the horizontal component of this force is $N*\cos(\theta)$, and the vertical component of this force is $N*\sin(\theta)$. The total Nail force along each failure plane (AB or BC) is the sum of the horizontal or vertical components of all Nails crossing that failure plane.

It should also be noted that the Nail force calculated above is per Nail, therefore it must be divided by the horizontal spacing of the Nails to result in the Nail force per unit width of the wall and to be compatible with the other forces, such as the weight or internal soil resistance forces.

c. Calculation of the Internal Soil Cohesion

The cohesion in the soil depends only on the soil's inherent properties. The internal cohesion within any soil layer is defined in units of stress. Therefore, the total cohesion force per unit of wall width is calculated as the product of the soil cohesion and the length over which it acts, i.e., the wedge plane length AB, BC, or BB'. In the case that more than one soil layer crosses a wedge plane; the total force is calculated as the sum of the forces over each segment of the line. Therefore, if the total force over plane AB is shown as C_1L_1, then this force is calculated as: $C_1L_1 = \Sigma C_{1i}L_{1i}$

The force over planes BC and BB' are calculated in a similar fashion.

d. Calculation of the Internal Soil Friction

The friction force capacity is defined by the friction angle (φ). The maximum internal friction force may be calculated as a fraction of the normal force along the surface (R), i.e., Fr = R*tan (φ).

The friction force along the plane AB is calculated as a fraction of the normal reaction force along this plane. Therefore: $Fr_1 = R_1 * \tan(\varphi_1)$. The force along planes BC and BB' are also calculated in a similar fashion. Note that the normal reaction force, R, is also unknown. Therefore, if a number of soil layers cross the failure plane, then an average friction angle is calculated as the weighted average of the friction angles for the segments of the failure plane, i.e.,: $\tan(\varphi_1) = \Sigma \tan(\varphi_{11})$.

e. Calculation of the Earthquake Force

The earthquake force is calculated as a fraction of the weight of the soil in front of the failure plane. The factor (fraction of the weight) is known as the seismic coefficient (or pseudo-static coefficient) and is determined based on the soil type, site geometry, and the peak ground acceleration. This factor is recommended by the Geotechnical Engineer and is to be included in the Geotechnical Report. The basis on which this factor has been determined is also to be fully expounded upon in the geotechnical report. Therefore, if this ratio is defined as "e", then the earthquake force in the lower wedge is calculated as: $E_{1H}=e*W_1$. Similarly, the force in the upper wedge is: $E_{2H}=e*W_2$. In the case that a vertical seismic coefficient is to be considered, then the force can be defined as the ratio of the vertical to the horizontal force, v, i.e., $E_{1v} = v* E_{1H}$. The force in Wedge 2 is calculated in a similar fashion.

f. Calculation of the Water Effect

Whenever the phreatic surface (water table) crosses the failure planes, i.e., water is present within the wedges, the capacity of the plane is reduced, i.e. the factor of safety is smaller. This effect is included in the calculations in the following ways:

f-1. The weight of the water is included in the weight of the wedge. This weight can be calculated as the void ratio of the soil multiplied by the volume of soil that is submerged in water, or can be specified as a saturated soil unit weight.

f-2. The internal soil friction force is reduced by the effect of the buoyancy force (BF). The buoyancy force is the volume of the submerged soil multiplied by the quantity (1-voidratio), multiplied by unit weight of water. This reduction in the friction force is accomplished by calculating: BF*tan(φ), and applying it is in direction opposite to the internal friction force, Fr.

g. Multi-Linear Failure Surface:

The failure surface may also be defined to be made of a multi-linear curve, instead of a bilinear curve. The critical point for such a surface is the fact that failure surface may run through a weaker soil, and thus result in friction and cohesion values much less than a bilinear path through the soil. If a multi-linear failure surface is defined, the point of intersection of the two wedges (point B) should also be defined. With the two wedges identified, the remainder of the parameter calculations will be carried out in a similar fashion. It is interesting to note that, for example, the resultant of all cohesion forces along a failure plane or curve (say AB) will lie along the straight line AB. The value of the total cohesion force can be found by finding the weighted average value of the cohesion along the curve AB, and multiplying it by the length of the straight line AB. Other parameters such as weight or water effects must be calculated by considering the actual path through the failure surface.

2.1.2.2 Finding the Critical Surface

In order to find the critical failure surface and the lowest factor of safety, various failure planes are examined. This is done by considering a solution area, bound by some assumed distance behind the wall. The length of the space considered within the trial section should be such that all possible conditions are considered. In practice this area should be taken as a large enough area so that the critical surface falls well within it.

Note that the trial surfaces may be examined for various depths of cut. Therefore, the location of point A (in ABC failure surface described earlier) may be specified at various depths. For each depth a sufficiently large number of failure surfaces shall be examined to obtain the most critical surface.

Next, for each location of points A and C, the location of point B should be varied, and tried. It must be noted that some locations of point B are deemed unacceptable by examination. These include:

j. Locations for which a portion of the ABC curve falls outside the soil, such as a stepped wall with point B close to the step, and directly under it.

ii. Locations for which the slope of line BC is flatter than the slope of the line AB.

A typical grid line showing a number of trial surfaces used in design/check trials is shown in Figure 2.1.2.2-1 below.

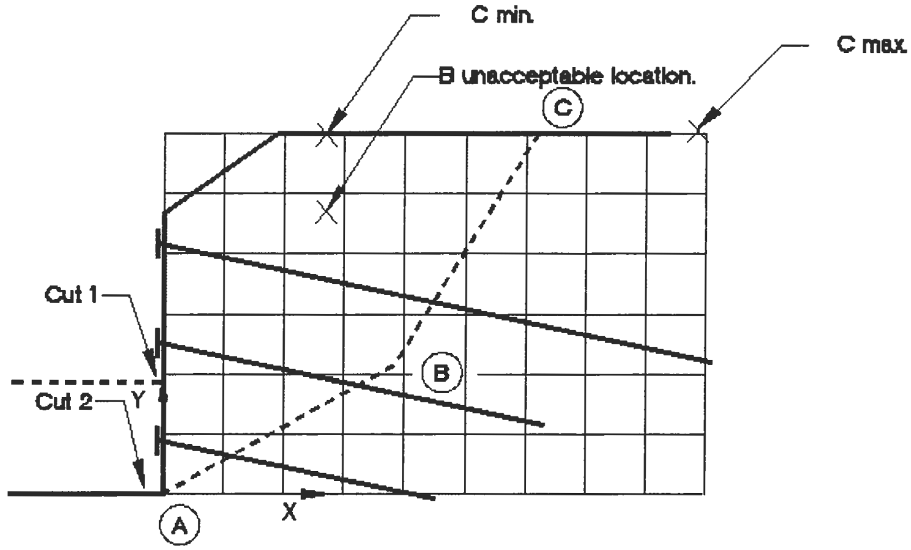


Fig. 2.1.2.2-1a: Grid line of B points for a given A and C

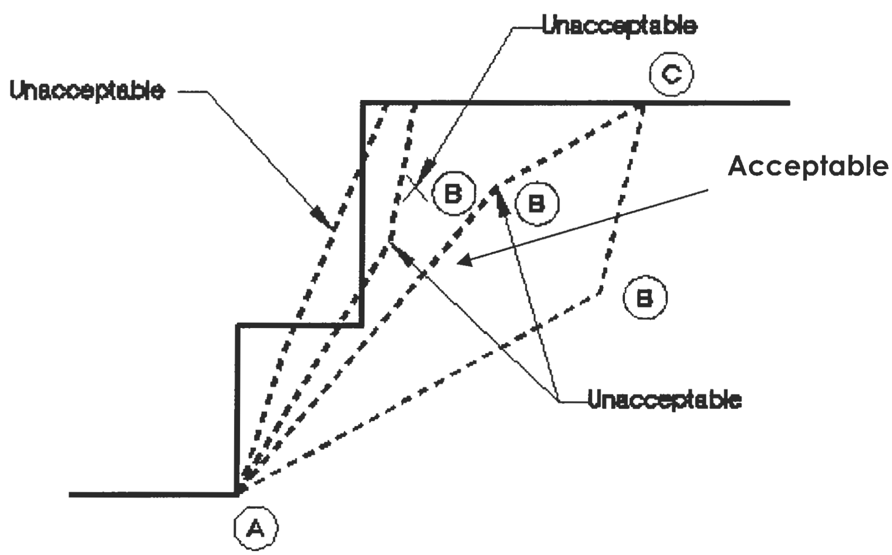


Fig. 2.1.2.2-1b - Example of acceptable and unacceptable intersection (B) Points

Figure 2.1.2.2-2 shows a typical set of safety factors for a 20 ft high wall for various trial failure surfaces.

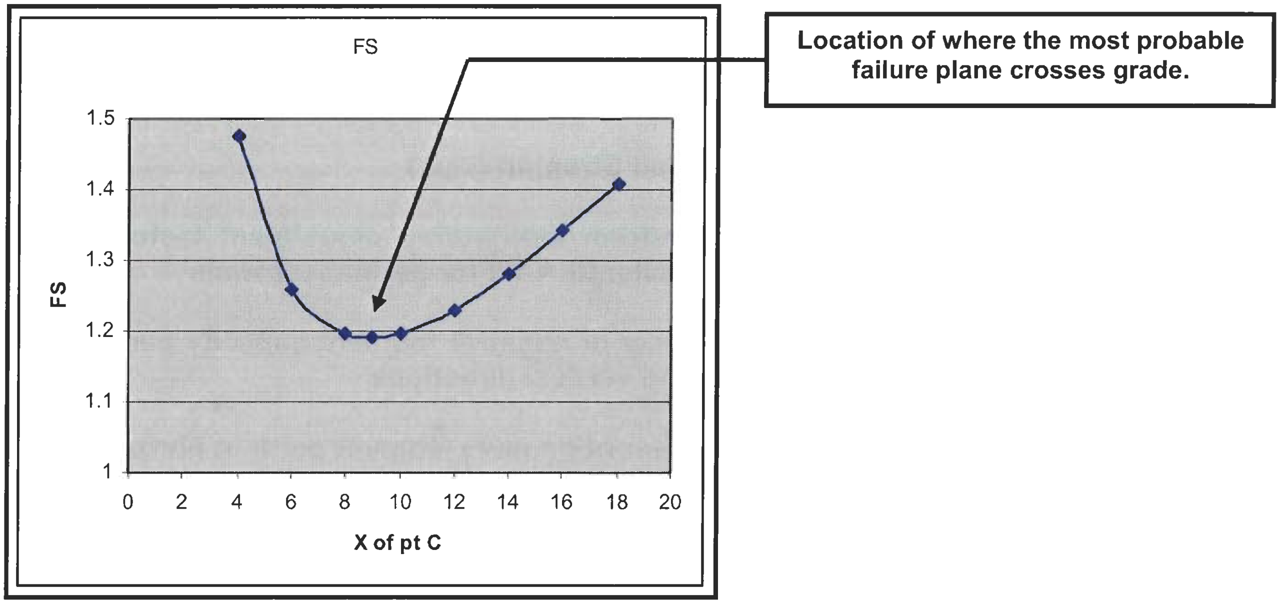


Figure 2.1.2.2-2 Safety Factors for a 20 ft High Wall

2.1.3 Punching shear:

The nail to wall connection shall be designed to carry the force "S" as shown earlier (figure 2.1.2.1.-2), and used in the soil-nail wall calculations. The minimum strength of the connection shall be 25.0 kips. The strength of this connection is referred to as the "nail head strength". Determination of the nail head strength depends on several potential failure modes, i.e., the wall and the nail-wall connection. The critical failure modes are flexure failure of the wall, and punching shear failure. Technical background is available in Reference (3). Figure 2.1.3-1 (4.9 of ref, top figure) shows a typical detail for this type of connection along with forces acting on the connection.

2.1.3.1 Flexural Strength of the Wall

The flexural strength of the wall will develop through the formation of a critical pattern of yield lines throughout the wall. The pressure distribution behind the wall is highly non-uniform in temporary walls, but shall be considered as uniform in permanent walls, see figure 2.1.3-1. The critical nail head strength, T_{FN}, associated with flexural strength of the wall shall be computed as follows:

$$T_{FN} = C_F\,(m_{V,NEG} + m_{V,\ POS})(8S_H/S_V)$$

Where:

T_{FN} = Nail Head Strength (kips)

C_F = Non-uniform distribution adjustment factor for temporary walls; C_F = 1.0 for permanent walls

$m_{V,NEG}$ = Lesser of negative moment capacity per ft in horizontal and vertical directions

$m_{V,POS}$ = Lesser of positive Moment per ft in horizontal and vertical directions

S_H = Horizontal spacing of nails

S_V = Vertical spacing of nails

Note that in this equation it is assumed that horizontal spacing is less than vertical spacing. In the opposite case, the reciprocal of this ratio shall be used. In other words, the ratio: $S_H/S_V \leq 1.0$.

2.1.3.2 Punching Shear Strength of the Wall

Punching shear failure occurs as a punching of a cone-shaped block of shotcrete or concrete, centered about the nail head.

A typical failure mode for a permanent wall connection is shown in figure 2.1.3-1. The punching shear capacity of the wall shall be calculated as follows:

$$V_N = 4\,\phi\,\sqrt{(f'_C)}\,(\pi)(D'_C)(h_C)$$

Where:

$\phi = 0.85$

V_N = Punching Shear Strength of wall

f'_C = Concrete Strength, in psi

D'_C = Average diameter of the punching cone

h_C = Depth of punching cone

The nail head strength associated with punching shear is somewhat higher than the wall punching shear strength due to the contribution from soil. This contribution is included as a multiplier determined from experimental studies [3] and is given as:

$T_{FN} = V_N\{1/[1 - C_S(A_C – A_{GC})/(S_V S_H – A_{GC})]\}$

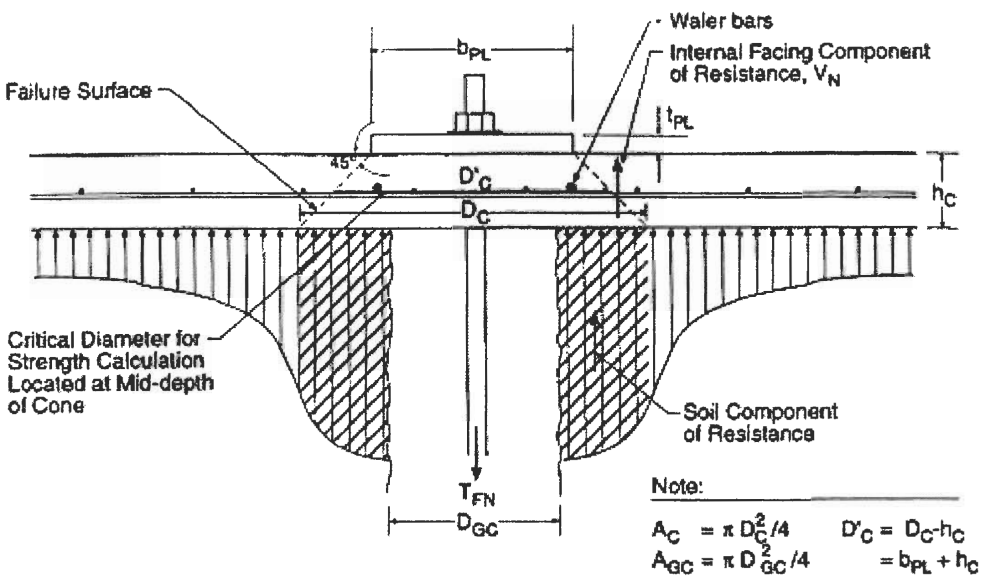
Where; T_{FN} = Nail Head Strength from Punching Shear

C_S = Non-uniform distribution adjustment factor for temporary walls; C_S = 1.0 for permanent walls

A_C = Area of base of cone; see figure

A_{GC} = Area of soil-nail; see figure

S_V, S_H = Vertical and horizontal nail spacing



Bearing-Plate Connection

Figure 2.1.3-1: Bearing Plate Connection

2.1.4 Minimum Factors of Safety

Minimum Factors of Safety for permanent Soil Nailed walls are as follows;

F.O.S. – Static Gravity = 1.50
F.O.S. – Seismic = 1.15

2.1.4.1 Minimum factors of safety shall not be less than recommended values contained in the geotechnical report, using soil strengths recommended therein.

2.1.5 Alternative Methods of Analysis

Finite Element procedure of analysis of geo-structures may be submitted in lieu of the conventional methods of analysis taking into account proper values of the soil moduli provided by the Geotechnical Engineer.

2.2 Tied-Back and Cantilevered Soldier Beam Wall

2.2.1 Required Geotechnical Information

a. General description of soil and foundation material and recommendation for the type of wall to be used; continuous geostructure such as sheet piles or tangent piles or Tied-Back and Cantilevered soldier beam walls with lagging.

b. Information as related to Tied-Back and Cantilevered walls such as stability of drilled holes etc.

c. Active pressure envelope for cantilever walls, and walls with one or multiple tiebacks.

d. Passive pressure envelope.

e. Corrosiveness of the soil.

f. Location of the Phereatic Surface.

g. Horizontal and vertical Seismic Loading.

h. Un-bonded zone envelope for the tiebacks.

i. Surcharge loading.

j. Pressure Diagram for design of the lagging.

k. Passive Arching Capability (A_r).

2.2.2 Analytical Method for Design of Cantilever Soldier beams.

2.2.2-1 Definitions

f = [Passive Arching Capability (A_r) X (Effective Pile Width)] ÷ (Soldier Pile Spacing) Where;

f = Arching Factor < or = 1.0

A_r = 0.08 x φ For granular soils

A_r = 2.0 For Cohesive Soils

K_a = Active Earth Pressure Coefficient

K_p = Passive Earth Pressure Coefficient

C = Undrained Shear Strength

γ = Unit Weight of Soil

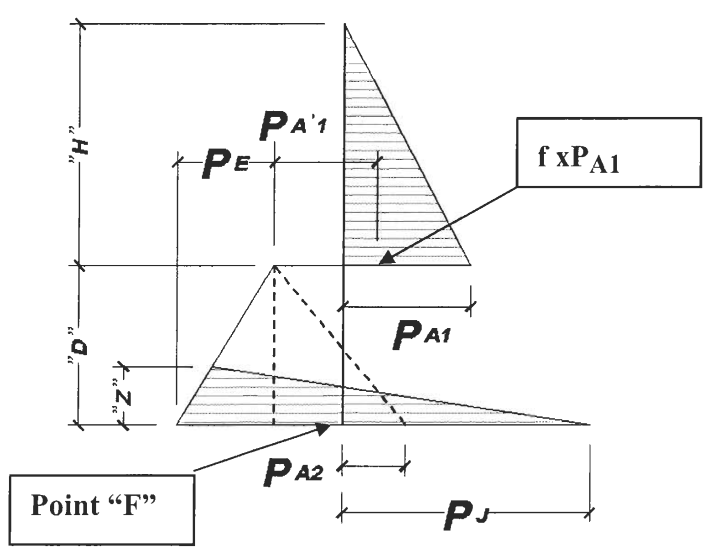


Figure 2.2.1: Force Diagram for Cantilever Soldier beam (φ – C Soil)

2.2.2-2 **Forces**

$$P_{A1} = \gamma\ H\ K_a$$

$$P_{A'1} = f\ (4\ C - P_{A1})$$

$$P_{A2} = f\ \gamma\ D\ K_a + P_{A'1}$$

$$P_E = f\ \gamma\ D\ (K_P - K_a) - P_{A'1}$$

$$P_J = f\ \gamma\ D\ (K_P - K_a) + f\ (4\ C + \gamma\ \ H\ K_P)$$

$$\sum F_H = 0 = [\ (H)\ (P_{A1})\ /\ 2\ ] + [(P_{A'1} + P_{A2})\ (D)\ /2\ ] + [(P_E + P_J)\ (Z)\ /2] - [(P_E + P_{A2})\ (D)\ /2]$$

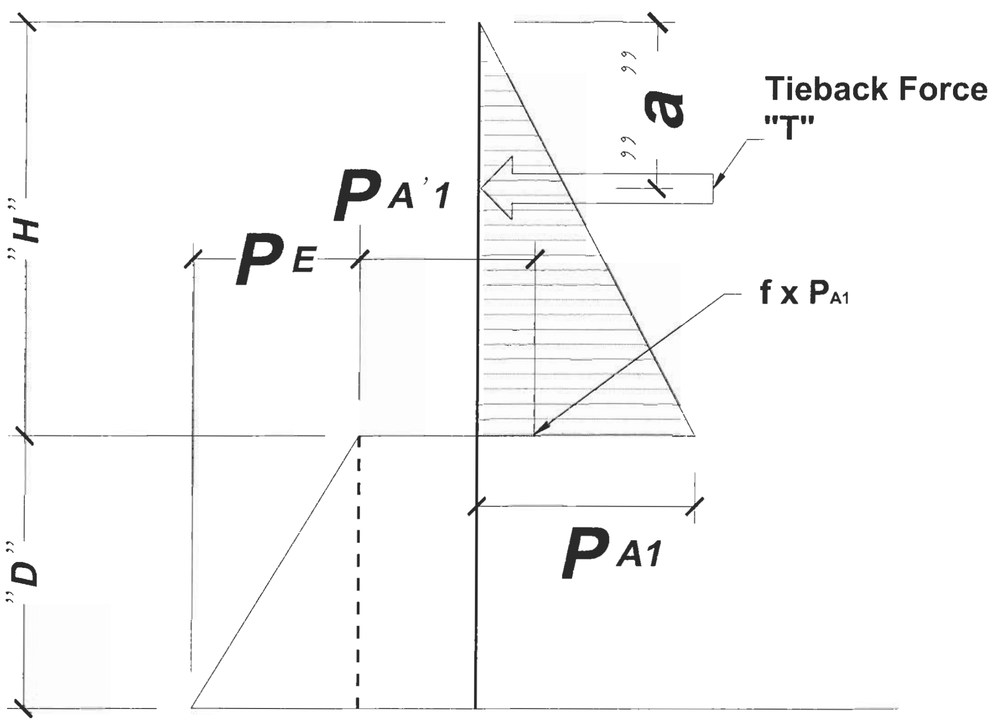
$$\Rightarrow \quad Z = [(P_E - P_{A'1})\ (D) - (H)\ (P_{A1})]\ /\ (P_E + P_J)$$

$$\sum M_F = 0 = [(H)\ (P_{A1})\ /\ 2]\ [\ H/3 + D] + [\ P_{A'1}\ (D)\ (D/2)] + [(P_{A'1} + P_{A2})\ (D)\ /2]\ (D/3) + [(P_{A2} - P_{A'1})\ (D)\ /2]\ [(D/3)] + [(P_E + P_J)\ (Z)\ /2]\ [(Z/3)] - [(P_E + P_{A2})\ (D)\ /2]\ [(D/3)]$$

From the above static equilibrium equation "D" can be found and hence the maximum moment in the soldier beams.

Lateral pressures due to surcharge and seismic loading may easily be incorporated into above equations and would not affect the methodology of the design approach.

2.2.3 Analytical Method for Design of Soldier Beams With One or Multiple Tiebacks.

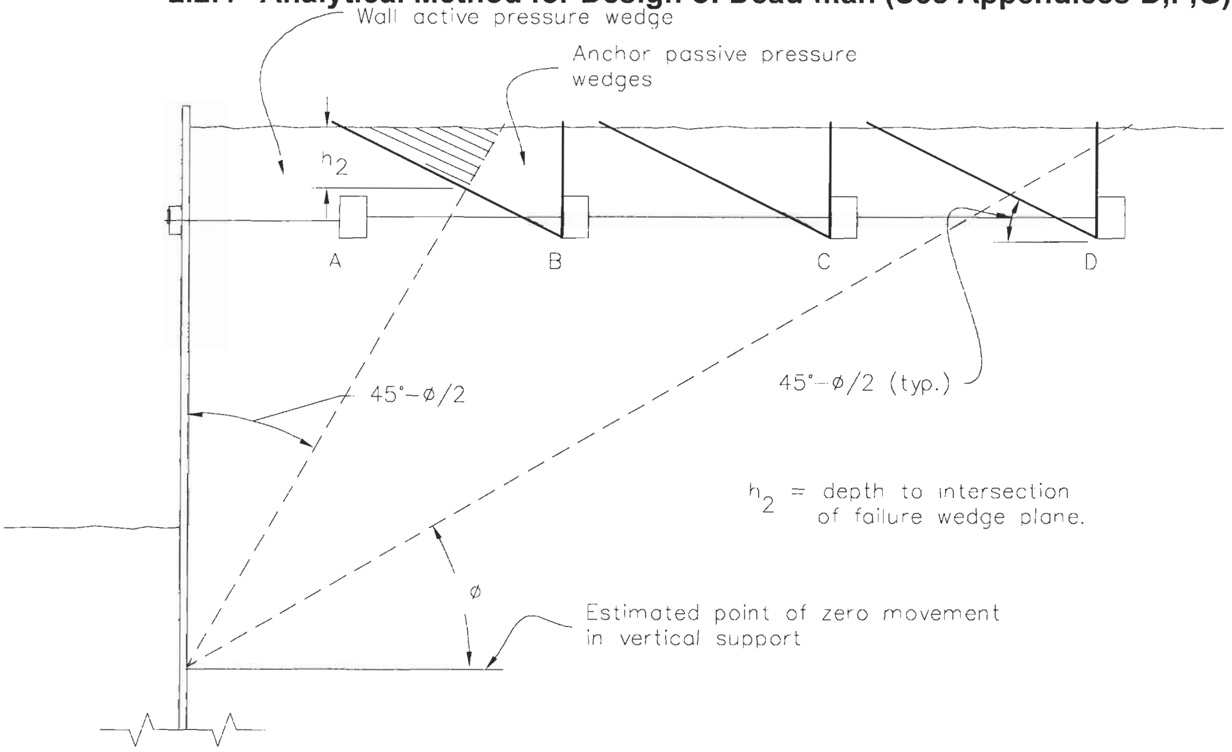


<u>Figure 2.2.2: Force Diagram for Tied Back Soldier beam (φ – C Soil)</u>

2.2.3.1 By applying static equilibrium equations, "D" and "T" can be found and hence the maximum moment in the soldier beams.

2.2.3.2 The pressure diagram for multiple tieback soldier beams is generally trapezoidal or rectangular and because of the fact that they are indeterminate systems, standard structural analysis methods for indeterminate structures is to be employed to determine the maximum moment, depth of embedment and the forces in the tiebacks.

2.2.4 Analytical Method for Design of Dead-man (See Appendices D,F,G).



Deadman A   Located Inside active wedge and offers no resistance.
Deadman B   Resistance is reduced due to overlap of the active wedge (wall) and passive wedge (anchor).

Anchor Reduction: (Granular soils)
$\Delta Pp = (1/2)\ (Kp - Ka) g h_2{}^2$
ΔPp is transferred to the wall.

Deadman C   Develops full capacity but increases pressure on wall
Deadman D   Develops full capacity and has no effect on bulkhead.

Figure 2.2.3: Effect of Dead-man location relative to the shoring wall

In the following sections, analytical design method is only provided for a Dead-man of Type D, where the full capacity can be achieved.

2.2.4.1 Dead-man In Cohesionless Soil Near Ground Surface $d \le H/2$

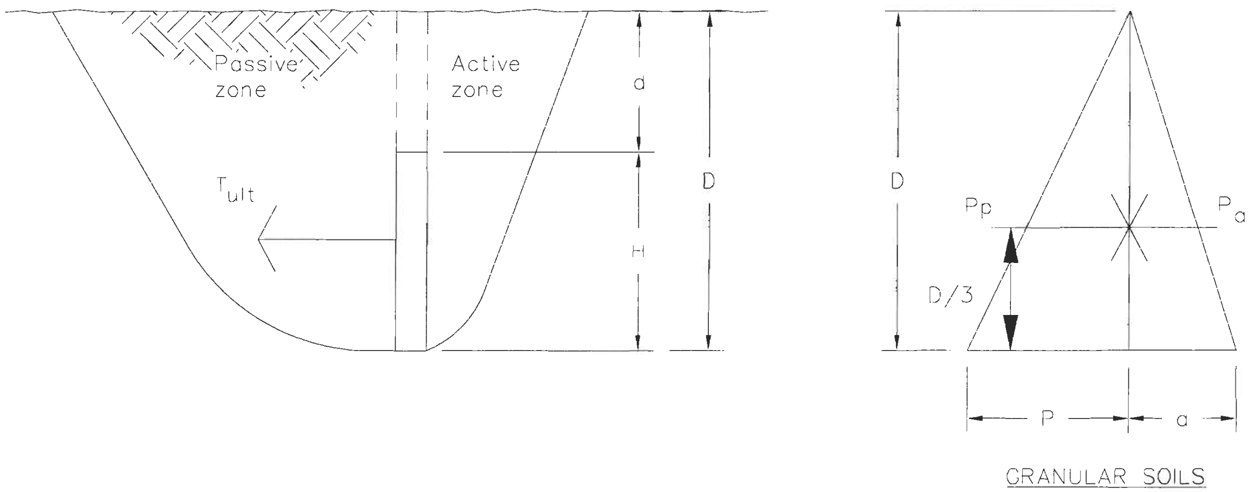


Figure 2.2.4: Diagram of Deadmen in Granular Soil

When a Dead-man is near the ground surface, it is assumed that the Dead-man extends to the ground surface.

As shown in the Figure above, the capacity of a continuous Dead-man can be obtained by,

$$P_{UIT} = L\,(P_P - P_a)$$

Where L is the length of Dead-man, P_P and P_A can be calculated by,

$$P_a = K_a \gamma D^2/2$$

$$P_P = K_P \gamma D^2/2$$

Therefore, the capacity of Deadmen per linear foot is,

$$P_{UIT} = \gamma D^2/2\,(K_P - K_a)$$

2.2.4.2 Dead-man In Cohesionless Soil Where $1.5 \le D/H \le 5.5$

When continuous Dead-men satisfy these criteria, the Ovesen's method, in which two dimensionless anchor resistance factors are introduced, can be employed to find the ultimate anchor resistance.

The first dimensionless resistance factor for the "basic case", R_O, can be determined by,

$R_O = K_\gamma - K_a$

Where the active earth pressure coefficient is given as,
$K_a = \tan^2 (45 - \phi/2)$

Based on the wall friction angle, δ, the earth pressure coefficient, K_γ can be found in the chart shown in Figure 2.2.5.

When ground water table (GWT) is below the base of Dead-man, the hydrostatic earth pressure per foot of Dead-man is,
$P_H = \gamma D^2/2$

The ultimate resistance per foot of Deadman for the "basic case" is,
$T_O = P_H R_O$

The second dimensionless resistance factor for the actual continuous Deadmen dimensions, R, can be calculated by,
$R = R_O (1+ R_O^{2/3}(1.1 (1\text{-}H/D)^4)$

Therefore, the ultimate anchor resistance per foot of Dead-man is,
$T_{ULT} = q_m H R$

Where q_m, the vertical effective stress in the earth at the midpoint of the actual height of the Deadman, is defined as,
$q_m = \gamma (D\text{-}H/2)$

2.2.4.3 Skin Friction of Trench

Additional anchor resistance is generated by the skin friction of the tie-back anchor trench. This resistance can be determined by,

$T_{TRENCH} = q_{trench} A$

Where q_{trench} is the effective shear stress in the earth at the midpoint of the trench, A is the total contact area between the trench and the ~~soil~~ DEADMAN and q_{trench} is given by:

$$q_{TRENCH} = (D - d + (H/2))\,\gamma \tan \phi$$

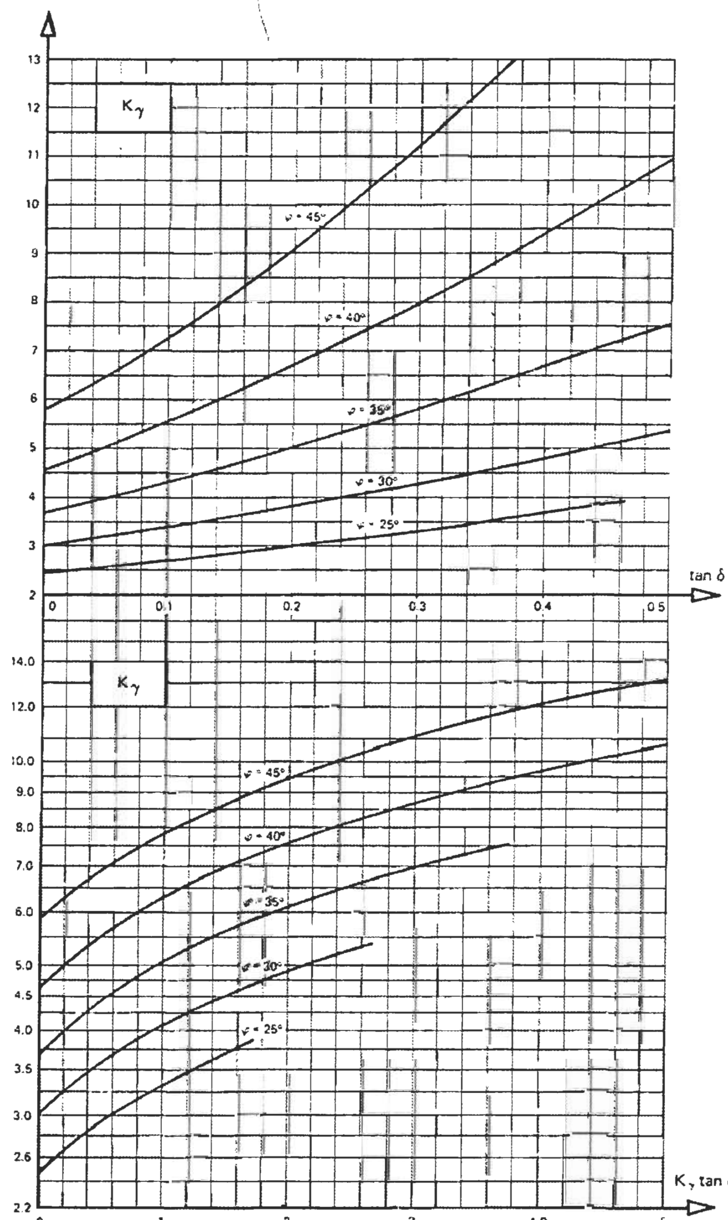


Figure 2.2.5: Earth pressure coefficients for Dead-man (after Ovesen)

2.2.4.4 Minimum Factor of Safety

The minimum factor of safety for a deadman anchor is 2.0

2.2.4.4.1 Minimum factors of safety shall not be less than recommended values contained in the geotechnical report.

2.2.5 Analytical Method for Design of Lagging.

2.2.5.1 The soldier piles and anchors are to be designed for full anticipated lateral pressure due to the retained soils, surcharge pressure and earthquake .

2.2.5.2 Permanent lagging shall be designed for full anticipated lateral pressure due to retained soils, surcharge pressure and earthquake, with due consideration to the arching effect of the soil, as recommended in geotechnical report.

2.2.6 Lateral Earth Pressure Envelope due to Earthquake Force

2.2.6.1 The horizontal pseudo-static seismic force is represented as a lateral earth pressure envelope and is determined based on the soil type, site geometry, and the peak ground acceleration. The configuration and intensity of this pressure envelope is recommended by the Geotechnical Engineer and is to be included in the Geotechnical Report.

2.2.7 Lateral Earth Pressure Envelope due to Surcharge

2.2.7.1 The lateral earth pressure envelope due to surcharge is determined based on the proximity and intensity of the surcharge. The configuration and intensity of the lateral surcharge pressure envelope is recommended by the Geotechnical Engineer and is to be included in the Geotechnical Report.

2.2.7.2 Alternatively, the lateral earth pressures on the retaining wall may be calculated based on the theory of elasticity (Boussinesque Method).

2.2.8 Minimum Factors of Safety

2.2.8.1 Minimum Factors of Safety multiplier to determine the depth of embedment for permanent cantilever and tied-back soldier beams;

F.O.S. – Static = 1.5

F.O.S. – Seismic + Static = 1.15

2.2.8.2 All other elements of the tied-back and cantilever soldier beams shall be designed in accordance with the load combinations of *2001 CBC Section 1603B.6*, Load Factors and Load Combinations.

2.2.9 Alternative Methods of Analysis
See Section 2.1.5.

3.0 General Construction Considerations

3.1 Minimum requirements

In addition to requirements specified in the geotechnical report, minimum requirements specified in this section shall apply.

3.1.1 Soil Nail Walls

3.1.1.1 Soil Nails

a. The diameter of Soil Nails shall be a minimum of 1 in.

b. The diameter of holes shall be a minimum of 4 in. in rock and 6 in. in soil.

c. Spacing of Nails shall be maximum of 6 ft. horizontal and maximum of 6 ft. vertical and a minimum of 3.6 ft. horizontal and vertical

d. Average length of soil nails in any given section would not be less than 80% of the height of excavation

3.1.1.2 Minimum requirements for Soil Nail Wall Facing

The facing of a permanent Soil Nail wall shall consist of one layer of initial shotcrete with 4" minimum thickness and a second layer of final shotcrete or CIP concrete. All wall reinforcing steel shall have minimum cover required by CBC 2001 Section 1907A.7

a.1. The facing layers shall be designed in
1 accordance with Sec. 2.1.3 and CBC 2001 Chapter 19A.

a.2 Force transfer (composite action) between individual facing layers shall be considered. Shotcrete facing layers shall meet, as a minimum, the requirements of 2001 CBC Section 1924A.

3.1.1.3 Soil Nail Wall Testing Requirements

3.1.1.3-1 Proof Test Procedure

a. The purpose of the Proof Test Procedure program for Soil Nails is to verify the assumed pullout strength (adhesion) of the soil (μ).

b. All Test Nails are to be sacrificial, and they are to be in addition to the production nails. The bonded and the unbonded length of the Test Nails each shall be no less than 10 ft., as shown in Fig. 3.1.1.3-1.

c. Elongation of the Test Nails at the time of the testing is to be measured against a fixed point on the wall surface to the accuracy of 0.001 in.

d. **A minimum of one Test Nail per each 10 production nails (approximately 1 per 60 ft.) shall be installed and tested in each row according to Proof Test Procedure. Location of such Test nails shall be determined by the engineer, and indicated on the design drawings.**

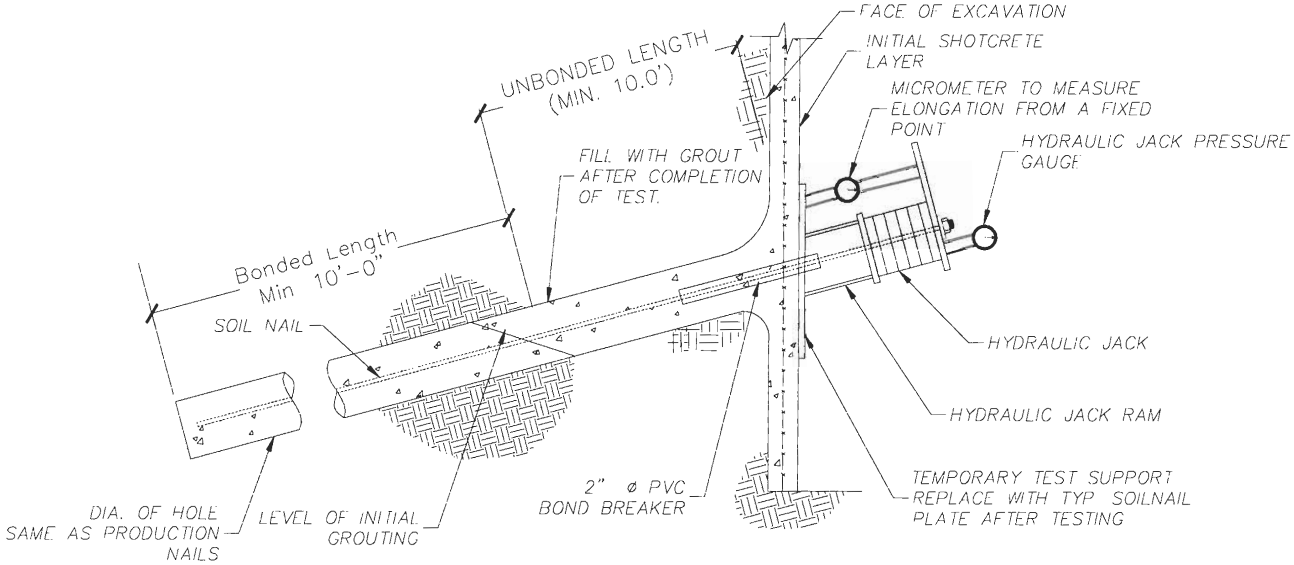


Figure 3.1.1.3-1: Test Nail Detail

e. **The Proof Tests shall be conducted by incrementally loading the Test nails in accordance with the schedule in Table 3.1.1.3-1. At each increment the load is to be held constant for one minute and elongation of the nail is to be recorded. At the maximum Test load, the load shall be maintained constant for 10 minutes and elongation readings shall be recorded at 1, 2, 3, 4, 5, 6, and 10 minutes. If the total elongation during the 10 minutes of the last reading exceeds 1mm (0.040 in.), the Test Load then shall be maintained for an additional 50 minutes and the elongation readings shall be recorded at 20, 30, 40, 50, and 60 minutes. The total elongation at the end of one hour shall not exceed 2 mm (0.080 in.).**

f. During the load hold periods, the hydraulic pressure shall not deviate from the Test Pressure by more than 50 psi.

g. In order to ascertain that the unbonded length of the Test Nails is free to extend, the minimum total elongation of the Test Nails is to be at least 80% of the expected elastic elongation of the unbonded length.

Table 3.1.1.3-1: Rapid Nail Test

Load	Observation Period (Minute)
$0.25M^{+}$	1
0.50M	1
0.75M	1
1.00M	1
1.25M	1
1.50M	10
1.50M	50*

+ $M = \mu\pi DL$, D = nail diameter L = bonded length (Min. 10 Ft.)
* if deflection at 10minute>0.04"

3.1.1.3-2 Extended Test Procedure

a. A minimum of two Test nails for each wall longer than 200 ft, with at least a minimum of two Test nails for the project shall be tested in accordance with the Extended Test Procedure. The Extended Test shall be conducted by incrementally loading the Test nails and measuring elongation according to Table 3.1.1.3-2.

b. At each step the Test nails are to be unloaded completely to Alignment Loading (AL)* prior to applying the incremental loading of the next increment. The elongation of the Test nails is to be measured and recorded at each step similar to section 3.1.1.3-1-e.

   i. (*)AL, Alignment Load = Minimum amount of load necessary to maintain alignment of the jack and the soil nail.

c. The times for reading the elongation shall be at 10, 30, 30, 45, 60, 120, and 210. The total elongation may not exceed 5 mm (0.2 in.) during the 210 minute period in which the the 200 percent load is maintained.

Table 3.1.1.2-1: Extended Nail Test

Load	Observation Period (Minute)
0.25M	10
0.50M	30
0.75M	30
1.00M	45
1.25M	60
1.50M	120
2.00M	210

3.1.1.3-2 Failure of Test Nails

a. In the event that a Test Nail fails to meet the established criteria, the Engineer is to evaluate the failure condition and determine the appropriate remedial action. Any remedial action shall be reviewed and approved by OSHPD.

3.1.2 Tied-Back and Cantilevered Soldier Beam Walls

3.1.2.1 Tied-Back and Cantilevered Soldier Beams

a. Tied-Back Soldier Beams shall consist of two structural steel elements sufficiently spaced to allow the Tie-backs to be placed between the two elements to avoid any eccentricity. Alternatively, the tiebacks may be placed on a waler beam spanning between the soldier beams or in a tieback pocket concentric with the center line of the soldier beam.

b. The lagging between the Soldier Beams shall consist of a minimum of 3x pressure treated lumber, Douglas Fir DF #1. Timber shall be treated so that there will be no decay during the anticipated service life of the lagging.

c. Where the deterioration of the timber lagging over time is a concern, the lagging shall consist of precast concrete planks or shotcrete. Shotcrete or Concrete lagging shall satisfy the coverage requirements of CBC 2001 Section 1907A.7.

d. The design of the initial layer of shotcrete, of wood lagging is in accordance with Sec. 2.2.5.

3.1.2.2 Tied-Back and Cantilevered Soldier Beam Wall Facing

a. The final facing of a Tied-Back and Cantilevered Soldier Beam wall shall consist of minimum 8 in. of shotcrete or a CIP concrete wall.

b. The connection between the final facing and the Soldier Beams is to be secured with Nelson Studs with a minimum of ¾ in. in diameter and 16 in. spacing.

c. A minimum Cap Beam to the dimension of 12 in. wide x 12 in. deep with minimum of reinforcement as described below is to be integrated into the second layer of facing. See 3.1.2.2-a above.

3.1.2.3 Tied-Back and Soldier Beam Testing Requirements

3.1.2.3-1 Proof Test Procedure

a. The purpose of the testing of the tiebacks is to verify the pullout strength of the installed tiebacks.

b. All tie-backs are to be tested to 150% of the design load in the Proof Test Procedure similar to Section 3.1.1.3-1.

3.1.2.3-2 24hr. - Extended Test Procedure

a. A minimum of 3% of all tie-backs shall be tested in a 24hr. - Extended Test Procedure. Location of such tiebacks shall be determined by the Engineer and indicated on the design drawings.

b. Each of these test tiebacks selected for the 24hr. - Extended Test Procedure shall be tested to 200% of its design load (M) by applying the load increments shown in table 3.1.2.3-1 in the following manner. Apply the load increment in each step within one minute, then measure the elongation during the observation period shown, unload the tieback completely to alignment load (AL), then apply the load up to the next increment and measure the elongation in the same way. At the last increment, when 200% of design load is applied, wait 24 hours, and measure the elongation within this period. The elongation must be less than 3/4 inch. If the anchor movement after the 200% load has been applied for 12 hours is less than ½", and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

Table 3.1.2.3-1: 24hr. - Extended Test Procedure

Load	Observation Period
0.25M	10 min
0.50M	30 min
0.75M	30 min
1.00M	45 min
1.25M	60 min
1.50M	120 min
2.00M	24 Hours

3.1.2.3-3 Quick Test Procedure

a. A minimum of an additional 6% of all tie-backs shall be tested in a Quick Test Procedure. The location of such tie-backs shall be determined by the Engineer and indicated on the design drawings. Each of these tie-backs shall be loaded to 200% of its design capacity (M) by applying the load increments shown in table 3.1.2.3-2 in the manner as described in Section 3.1.2.3-2-a. At the last increment, when 200% of the design load is applied, measure the elongation during the last 30 min. of the observation period.

The elongation after the load has been applied should not exceed ¼ inch during the 30 minute period.

b. After a satisfactory test, each production anchor should be locked off at 80% of the design load.

Table 3.1.2.3-2: Quick Test Procedure

Load	Observation Period
0.25M	10 min
0.50M	30 min
0.75M	30 min
1.00M	45 min
1.25M	60 min
1.50M	120 min
2.00M	5 Hours

3.1.2.3-4 Failure of Tiebacks

a. In the event that a Tieback fails to meet the established criteria, the failed tieback is to be replaced. The Engineer is to evaluate the failure condition and determine any required remedial action. Any remedial action shall be reviewed and approved by OSHPD.

3.2 Special Construction Considerations

3.2.1 Soil Nail Wall

Soil Nailing is a top-down construction technique and each sequence of excavation may not exceed the maximum vertical interval between the nails (Max. 6 ft.).

a. The interval of time between when the excavation has taken place, and when the shotcrete facing has been applied and the Nails have been secured, is to be kept to a minimum.

b. Depending on the type of soil, this period ranges from 6 to 48 hours. In certain instances, where surface instability is problematic, a stabilizing berm and/or a shotcrete flash coat may be specified to protect the excavation during this period.

c. During construction, depending on the type of soil encountered, this time may be modified at the discretion of the engineer.

Discussion - During this period most of the deformation of the wall occurs due to relaxation of the internal stresses and loss of surface moisture, etc. It is also during this period that localized surface instabilities occur in soils that have propensity to do so.

3.2.2 Soldier Beams

a. After installation of Soldier Beams, the excavation may not exceed 2 ft. below the level of the Tie-Backs before the Tie-Backs are installed.

b. After installation of the lagging, the cavity between the backside of the lagging and the soil is to be filled with lean mix concrete or grout.

3.3 Monitoring During Construction

3.3.1 During construction, the horizontal and vertical movement of the top of the wall is to be monitored on a weekly basis. Details of the monitoring program shall be submitted for review and approval by the Geotechnical Engineer, the Structural Engineer and OSHPD. If movement exceeds 0.5% of the height of the wall at any given time, the construction shall be stopped and evaluation of the design must be made and remedial action taken.

Note: As mentioned above, movement generally occurs during the time between when excavation has occurred and Soil Nails have been drilled and shotcrete facing applied. Empirically, it has been that decreasing the rate of said period can greatly reduce the rate of deformation of a Soil Nail wall. See section 3.2.1 for further discussion.

4.0 Corrosion

4.1 *General Discussion;*

Corrosion protection shall satisfy the requirements specified in the geotechnical report as a minimum.

There are four degrees of corrosion protection for structural intrusions in soil, for which a brief description of each follows:

4.1.1 Single Corrosion Protection (SCP) fusion bonded epoxy coating ASTM A775 minimum of 0.3 mm thickness, electrically applied or hot dip galvanizing.

Note: Epoxy coating alone is only applicable to Soil Nails or Tie-back rods and will not provide adequate protection for tie-backs consisting of strands.

4.1.2 Double Corrosion Protection, (DCP) Encapsulation; minimum 1.0 mm thick corrugated HDPE 2 conforming to AASHTO M252 or corrugated PCV conforming to ASTM D1784 class 13 464-B. Encapsulation shall provide at least ¼" grout cover over the tie-back bar or strand and be resistant to ultraviolet light degradation.

4.1.3 Triple Corrosion Protection, the combination of the two corrosion protection methods as described above.

4.1.4 System of cathodic protection in conjunction with the above described systems.

4.2 Minimum required corrosion protection for Soil Nailed and Tied-Back/Cantilevered Soldier Beam walls is for a period of 100 years and is as follows:

4.2.1 Soil Nail Walls

a. As a minimum, Soil Nails are to be Single Corrosion Protected (SCP) by epoxy coating or galvanizing.

b. Soil Nail Shear Plates and Nelson Studs are to be protected against corrosion by epoxy coating or galvanizing.

4.2.2 Tied-Back and Cantilevered Soldier Beam Walls

a. Soldier Beams are to be protected, as a minimum by Epoxy or bituminous paint.

b. As a minimum, all Tie-backs are to be double-corrosion protected in High-Density Polyethylene Corrugated tubing.

c. The elements of the tip connection of the Tie-backs shall be protected by Epoxy paint or galvanizing and protective grease cap.

4.3 The plans, specifications and construction procedures are to be reviewed by a duly registered Corrosion Engineer and he or she is to certify in writing that the Earth Retention System, with view to the corrosiveness of the soil and the environment would have a life span of minimum of 100 years. Certification by a Corrosion Engineer shall be submitted to OSHPD along with design calculations.

5.0 <u>Drainage and Waterproofing</u>

5.1 Drainage

Permanent and positive drainage system by use of geo-fabric, both for Soil Nail and Tied-Back or Cantilevered Soldier Beam walls, is to be provided.

5.1.1 Seepage through the lagging will not be considered as an adequate and positive drainage.

5.1.2 Proper allowance needs to be provided in analysis and design according to principles of soil mechanics and structural engineering to allow for variance in ground water table and phereatic surface.

5.2 Waterproofing

5.2.1 Soil Nail Walls

Waterproofing is to be provided per project specifications between the initial layer of shotcrete facing and the final facing of the Soil Nail wall.

5.2.2 Tied-Back or Cantilevered Soldier Beams

Waterproofing per project specification may be provided between temporary lagging and the final facing of the Soldier Beam wall.

6.0 Inspection, Testing and Site Observation Program

6.1 General Discussion

6.1.1 The Structural Engineer of Record for the Earth Retention System shall be responsible for the preparation and submittal as a part of the design documents, of a plan for a Testing, Inspection and Site Observation Program.

6.1.2 The plan submitted for Testing, Inspection and Site observation shall be in compliance with Part 1 & 2, CCR
Title 24 and the geotechnical report. The plan shall be verified for compliance during plan review and shall specifically contain the following detailed requirements:

6.2 Field Quality Control of Materials

6.2.1 Inspection of Manufacturers certification and visual inspection of the following Items:

a. Steel Soldier Beams – Mill Certificates

b. Soil Nails

c. Soil Nail Shear Head connection parts

d. Tie-back tendons

e. Centralizers

f. Drainage material

g. Corrosion protection requirements- epoxy coated or encapsulated tendons

6.2.2 Confirmation of Mix Design and Compression Test for:

a. Grout for Soil Nails or Tie-Backs

b. Shotcrete facing or lagging

6.2.3 Adequacy of Storage and Handling of Materials

6.3 Construction Monitoring and Site Observation

6.3.1 General Installation

a. General conformance with Plans and Specifications

b. Compliance with admissible excavation heights

c. Compliance with specified Bore Hole diameter and length

d. Confirmation that holes have not caved in

e. Observation of grouting and/or tieback installation

f. Verification of Soil Nail and tie-back location / angle

g. Confirmation of placement and number of centralizers

h. Confirmation that corrosion protection system has not been damaged

6.3.2 Grouting

a. Confirmation that the grout is properly gravity injected by tremie pipe.

b. Confirmation that the grout is continually pumped, while auger or casing is removed.

c. Recording volume of grout placed.

d. Verify that grout is batched in accordance with approved mix design.

e. Verify proper placement of post-grouting tubes in tie-backs.

6.3.3 Intermediate Excavation, Structural Facing, Lagging and Drainage

a. Verify proper establishment of drilling benches.

b. Verify establishment of excavation heights.

c. Verify protection of excavated face with proper thickness of “Fat excavation,” per plans.

d. Verify placing of geo-composite drainage strips.

e. Verify the proper installation of reinforcing steel.

f. Record volume of grout placed Verify the proper installation of reinforcing steel.

g. Verify proper establishment of finish line and grade.

h. Verify that application of shotcrete and closure of exposed face of the excavation occurs within the specified time limits.

i. Verify that application of shotcrete and closure of exposed face of the excavation occurs within the specified time limits.

l. In Soldier Beam Walls, verify proper placement of timber and concrete pre-cast/shotcrete lagging.

m. For Soldier Beam Walls with shotcrete lagging, follow procedure as stated above.

n. Verify that shotcrete is batched in accordance with approved mix design.

o. Verify that shotcrete test panels if specified are prepared.

6.3.4 Localized Surface Instability

a. In Soil Nailed Walls, verify that in case of localized sloughing debris is cleaned and procedures in the design drawings are closely followed.

b. In soldier beam walls, verify that cavities behind lagging are filled with sand cement slurry.

6.4 Testing of Soil Nail and Tie-Backs

6.4.1 Testing Program for Soil Nails

a. Verify that all Soil Nail Wall Testing Requirements is met according to Section 3.1.1.3.

b. Verify that the number of test nails is approximately 10% of the total number of nails and that their location is randomly chosen by the engineer.

c. Verify that all test nails are tested.

d. Once the test is successfully performed, verify that the test nails are properly grouted and abandoned.

6.4.2 Testing of Tiebacks

a. Verify that all Tied-Back Wall Testing Requirements are met according to Section 3.1.2.3.

b. Verify that all tiebacks are tested to at least 150% of the design load.

c. Verify that all tiebacks, after testing successfully, are locked at least 80% of the design load.

7.0 Post Construction Monitoring

7.1 The movement of permanent Soil Nailed and Tied-Back or Cantilevered Soldier Beam walls is to be monitored for minimum of 5 years after completion of construction, per project specifications.

7.2 The monitoring of the movement of the Earth Retention System is to achieved by a series of Electronic In-Place Inclinometers that are capable to transmit their data in real time to a data bank within the facility grounds.

7.3 There is to be at least one inclinometer per every 150 linear ft. of the wall with at least one inclinometer per wall placed in an appropriate location behind the wall as per design plans.

7.4 The Owner (building maintenance engineer) of the facility is to manage data collection and reporting system. A summary of the monthly data shall be submitted annually to OSHPD. The Owner shall maintain records of all data at the building site and shall make site records available to OSHPD for inspection at any time during the monitoring period.

7.5 The data is to be collected at a minimum frequency of once every month for the duration for 10 years, and should the movement of the wall exceed 1 in. at the top of the wall over a period of one year, the Owner shall report the findings to OSHPD and the Hospital Administrator.

7.6 Should the movement of the wall at the toe continue to exceed or be equal to 1 in. per year, for three consecutive years, then the Owner shall retain a Geotechnical and Structural Engineer to begin procedures to mitigate the movement of the wall and/or start proceedings to implement the contingency plan as specified in the design drawings.

7.7 Contingency Plan

7.7.1 A contingency plan must be specified on the design drawings in an outline specification form that would describe a structural system and construction procedure which would stop the movement of the Earth Retention System when the above criteria is met.

7.7.2 The contingency plan must contain a feasible structural system that can be constructed to stabilize the earth retention system of the facility and prevent any further movement.

7.7.3 The contingency plan must be constructible without disruption to any of the operations of the facility. Any parts of the contingency plan, i.e. footings, in-beds etc. the construction of which could disrupt the function of the facility during construction of this contingency plan, will need to be implemented as a part of the initial construction of the facility.

8.0 Specialty Contractor Qualifications;

8.1.1 The specialty contractor who is to perform the work must submit documentation for a minimum of 5 years of verifiable experience in construction of permanent Soilnailed and Tied-back/Cantilever Soldier Beam Earth Retention Systems. Further, the specialty contractor is to provide evidence of having completed at least two similar projects in the past five years.

8.1.2 The specialty contractor shall submit documentation for project references. These documentations shall include project descriptions, the specific role of the specialty contractor, and as-built drawings of the referenced shoring systems.

8.1.3 The Structural Engineer of Record shall review and approve the qualifications of the specialty contractor and make a written recommendation to the Owner and OSHPD in addition to the General Contractor.

References

1. Felio, G.Y., Vucetic, M., Hudson, M., Barar, P., and Chapman, R., 1990, "Performance of Soil Nailed Walls During the October 17, 1998 Loma Prieta Earthquake," IN *Proc. Of the 43rd Canadian Geotechnical Conference, Quebec, Canada, pp.165-173.*

2. U.S. Department of Transportation, Federal Highway Administration (FHWA), 1996, "Manual for Design & Construction Monitoring of Soil Nail Walls," FHWA, Publication No. FHWA-SA-96-069

3. U.S. Department of Transportation, Federal Highway Administration (FHWA), 1996, "Soil Nailing Field Inspector's Manual," FHWA, Office of Technology Applications, Publication No. FHWA-SA-93-068

4. Post-Tensioning Institute (PTI), 1996, "Recommendations for Prestressed Rock and Soil Anchors – Third Edition," PTI

5. California Department of Transportation (Caltrans), "Trenching and Shoring Manual" Office of Structure Construction, Caltrans, January 1990; Revision 11, December 1996.

6. USS Steel Sheet Piling Design Manual - Updated and Reprinted by the Federal Highway Administration, July 1984.

7. Das, Braja M. (2004), "Principles of Foundation Engineering – Fifth Edition", Thomson Brooks/Cole.

Appendix "A" Reference 1

ANALYSIS OF SOIL-NAILED EXCAVATIONS STABILITY DURING THE 1989 LOMA PRIETA EARTHQUAKE

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CONTENTS

ABSTRACT

The performance of nine different grouted soil-nailed excavations in the San Francisco Bay area during the Loma Prieta earthquake is analyzed on the basis of postearthquake visual inspections, subsequent stability analyses, and dynamic centrifuge model tests. None of the excavations showed any signs of movements or similar distress, even though one of them was located in the vicinity of the earthquake epicenter where there was strong shaking and important seismic-related damage to other structures. The design and construction practices of grouted soil-nailed excavations in California are discussed. It is concluded that a combination of conservative design and construction is the primary reason for excellent seismic stability. It is also confirmed that the method developed and used by Caltrans for calculating the factor of safety is suitable for the stability analysis of the grouted soil-nailed excavations encompassed by the study. This method is based on a bilinear failure surface and the so-called German mode of failure that considers two sliding blocks.

INTRODUCTION

Soil nailing is an in-situ technique of mechanically stabilizing soil masses which has been used in Europe for more than two decades (Stocker and others, 1979; Chapman and Ludwig, 1993; Federal Highway Administration, 1993). In North America, as well as in Japan (Japan Highway Public Corporation, 1987; Ochiai and others, 1992), soil nailing is steadily gaining popularity because it can be used with conventional shoring equipment, it reduces excavation time, it allows construction-related activities to proceed in restricted space, and it can produce significant savings over conventional shoring techniques in the proper ground conditions.

The main feature of soil nailing is that it is an in-situ method where the existing natural soil is reinforced, as opposed to a backfill reinforcement. As shown in figure 1, the inclusions, commonly called nails, are installed during the excavation using a "top-down" construction procedure, unlike reinforced earth walls which are constructed from the bottom up. This allows soil retention in areas where little space is available for the excavation. The soil-nailing concept is to reinforce the soil with passive inclusions, so that the nailed soil mass behaves as a composite unit, similar to a gravity retaining wall supporting a soil backfill (Juran and Elias, 1991; Mitchell and Villet, 1987). In that sense, soil nailing also differs from the conventional tie-back excavation support since the soil nails are not prestressed; that is, their resistance can be mobilized only by the movement of soil mass or the face of the excavation to which the nails are fixed. Figures 2 to 4 show several soil-nailed retaining structures treated in this paper.

At present, there are three major concerns about soil-nailed excavations: (1) the adequacy of the analysis or design methods, (2) the long-term behavior, and (3) the performance during seismic loading. Items (1) and (2) have been addressed by many researchers, including recently by Gassler (1992), Juran and others (1990), Plumelle and others (1990), and Stocker and Riedinger (1990). With respect to soil-nailing performance during earthquakes, no full-scale field observations were available until the Loma Prieta earthquake. During the earthquake, nine soil-nailed structures were subjected to different levels of shaking, including horizontal ground-surface accelerations probably as high as $0.4\,g$. In spite of such relatively high horizontal accelerations, these structures did not show any visible movements or other signs of distress (Felio and others, 1990; Hudson, 1990). A systematic description of these structures and a discussion of possible reasons why they performed so well are the main purposes of this paper. More details about the corresponding investigation can be found in Tufenkjian and Vucetic (1993).

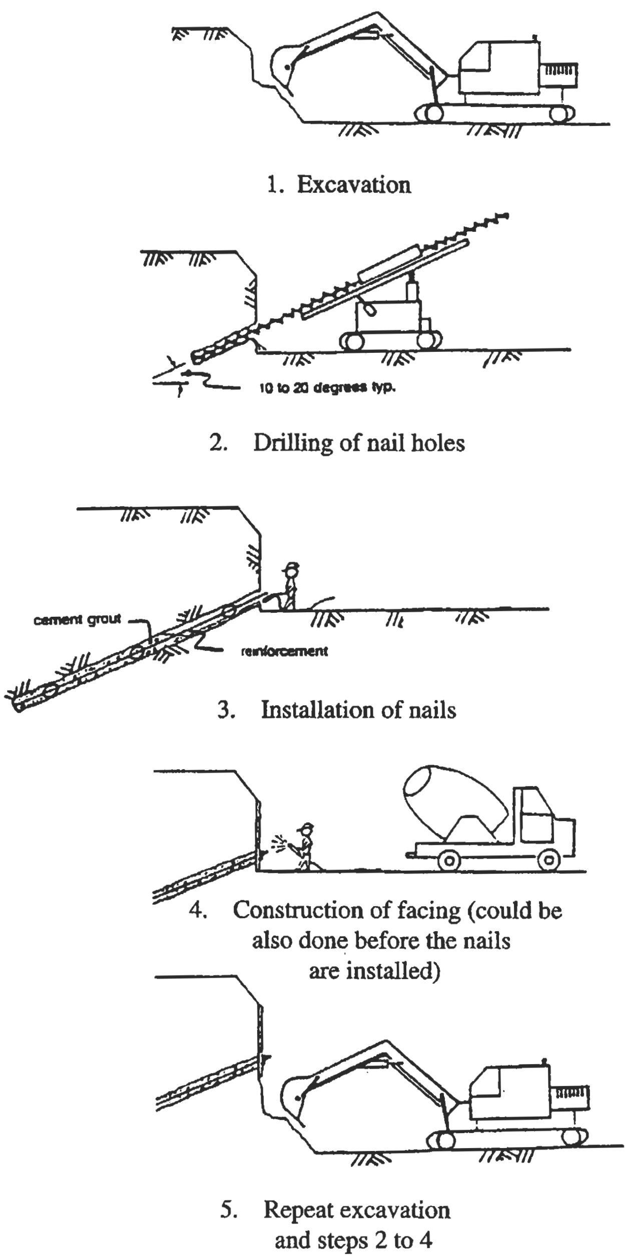

Figure 1.—Steps in the construction of a grouted soil-nailed excavation.

SOIL NAILING PRACTICE IN CALIFORNIA

There are three major steps in the construction of a soil-nailed wall, as illustrated in figure 1. They are (1) excavation, (2) installation of nails, and (3) construction of facing. The excavation generally proceeds in stages ranging from 1.2 to 1.8 m in depth. One of the major requirements for successful soil-nailed systems is that the excavation be capable of self-support for at least a few hours prior to nailing and construction of facing. For the most economical construction, however, the self-support should be able to last 1 to 2 days. As the excavation of each level proceeds, the nails are installed at predetermined locations. These reinforcing elements may be one of several types: driven, grouted, jet-grouted, or even pneumatically propelled into the ground (Myles and Bridle, 1991). However, the vast majority of installations are of the open drilled and grouted type (Chapman and Ludwig, 1993; Federal Highway Administration, 1993).

In California, and North America in general, the most popular type of nails are the grouted nails, such as those shown in figures 2, 3 and 4, since in many locations the soil conditions allow the excavation to stand open long enough. Grouted nails generally consist of Grade 60 mild steel bars (15 to 45 mm in diameter) placed in boreholes of 100 to 250 mm diameter. Plastic centralizers are often used to ensure proper grout cover of the nail. A cement grout is then placed into the boreholes by gravity flow or low pressure. Typical horizontal and vertical spacings range from 1 to 3 m, depending upon the designer's experience and soil conditions. The nails are generally inclined at 10° to 20° from the horizontal.

Either before or after the nails are in place, a facing structure is built. The facing is required to control soil erosion at the excavation face and reduce changes in the moisture content of the soil. The most common type of facing is shotcrete layer, 100 to 250 mm thick, which is usually placed by the shotcrete method and which is reinforced with welded wire mesh. A typical detail of the nail connection to such facing is presented in figure 5. If necessary, a blanket of nonwoven geotexile is placed between the natural soil and the shotcrete to control the drainage. The grouted nail is attached to the facing by bolting the steel bar to a square plate usually 300 to 400 mm wide. For additional reinforcement and strengthening of the facing, horizontal waler bars may be installed to connect the plates. Other methods of attachment are used for driven nails. For permanent walls, the shotcrete facing may not provide for the aesthetic requirements of the project. In such cases, either cast-in-place reinforced concrete facing or prefabricated panels can be used. Figure 6 shows photos of large soil-nailed excavation structures recently completed in California.

THE LOMA PRIETA EARTHQUAKE

The Loma Prieta earthquake (M_s=7.1) was one of the most costly single natural disasters in U.S. history. It caused extensive damage, such as landslides in the epicentral region, liquefaction in various areas of the San Francisco Bay region, structural distress to commercial, industrial, and residential buildings, widespread disruption or total destruction of utility systems, and damage to critical transportation systems. The earthquake has been the subject of a wide range of studies, many of them on geotechnical-related failures, as summarized by Seed and others (1991).

Figure 7 presents an overview of the regional geology and the recorded peak horizontal ground-surface accelerations during the earthquake. The locations of the nine soil-nailed walls considered in this paper are identified on the figure by stars, and the location of the epicenter by a circle. The figure shows that in the epicentral area the measured maximum horizontal ground-surface accelerations, a_{max}, were as high as 0.64 g and the vertical up to 0.60 g. It can be seen that the soil-nailed walls in the northern region (in Richmond, San Francisco, Walnut Creek, and San Ramon) were subjected to seismic forces corresponding to a_{max} of about 0.10 g. In the vicinity of the two walls in Mountain View, an a_{max} of around 0.2 g was measured. In the vicinity of the wall in San Jose, a_{max} was between 0.11 and 0.18 g. The largest a_{max} (0.47 g) recorded near a soil-nailed wall was in Santa Cruz, some 16 km due west of the epicenter.

Most of these locations were visited and inspected 2 days after the earthquake by a team from the University of California, Los Angeles (Felio and others, 1990), and some walls were inspected subsequently by design and construction companies. As stated earlier, no signs of distress or corresponding deformation were found on the walls, indicating excellent performance of such structures during moderate and strong shaking.

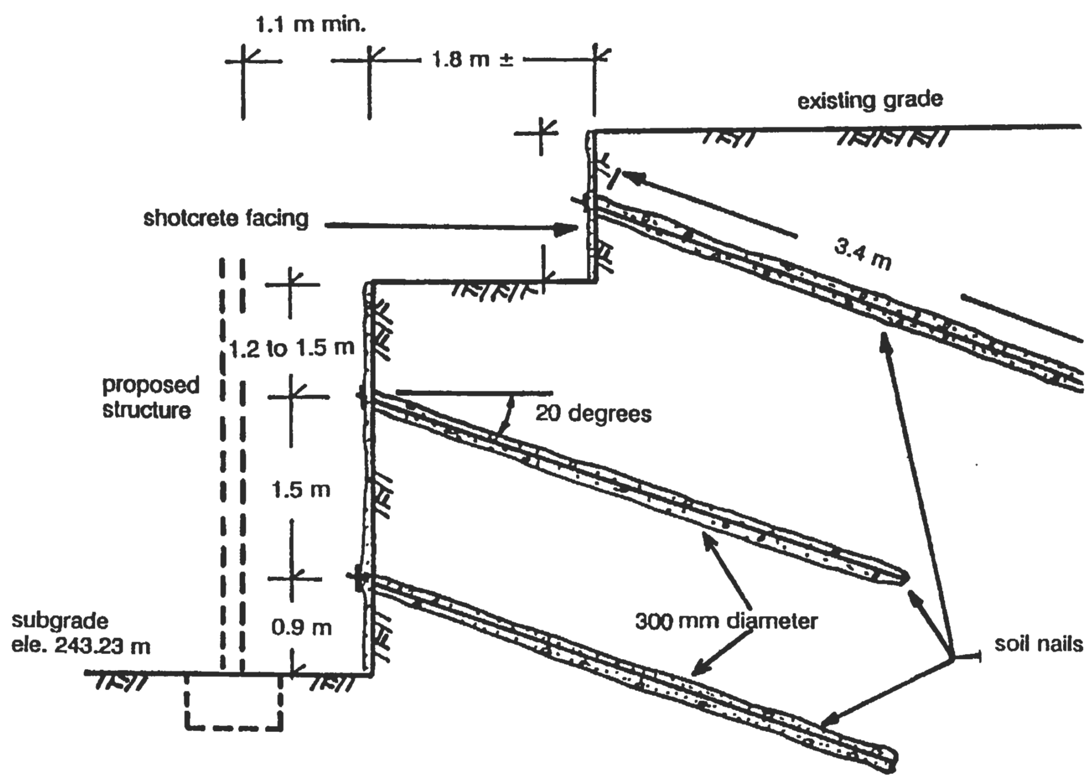

Figure 2.—Cross section of a soil-nailed excavation for a building constructed in Santa Cruz (UCSC wall), Calif. (Felio and others, 1990).

CHARACTERISTICS AND DESCRIPTION OF THE WALLS

The main characteristics of the nine walls inspected after the earthquake are summarized in table 1. In figure 8 the dimensions of the walls are presented in a uniform scale. The variation of the geometry, characteristics of the walls, soil conditions, and estimated ground-surface accelerations are evident. The walls are further characterized in table 2 in terms of the following three dimensionless ratios commonly used as design criteria (Bruce and Jewell, 1987):

$$\text{Length ratio} = \frac{\text{maximum nail length}}{\text{excavation height}} \qquad (1)$$

$$\text{Bond ratio} = \frac{\text{hole diameter} \times \text{nail length}}{\text{horizontal spacing} \times \text{vertical spacing}} \qquad (2)$$

$$\text{Strength ratio} = \frac{(\text{nail diameter})^2}{\text{horizontal spacing} \times \text{vertical spacing}} \qquad (3)$$

Table 3 further compares the three dimensionless ratios computed for the nine San Francisco walls, with the values computed by Bruce and Jewell (1986, 1987) for soil-nailed structures with drilled and grouted nails constructed all over the world. The bond and strength ratios generally fall within the range of other soil-nailed retaining structures. However, the length ratios for the San Francisco walls are generally much higher than those calculated from other sites, suggesting that the San Francisco walls are more conservatively designed. Note from table 2 that the length ratio for the UC Santa Cruz wall is the smallest. This wall is apparently the least conservatively designed of all of the walls, yet it is located in the vicinity of the highest estimated peak horizontal ground-surface acceleration. In spite of these facts, no ob-

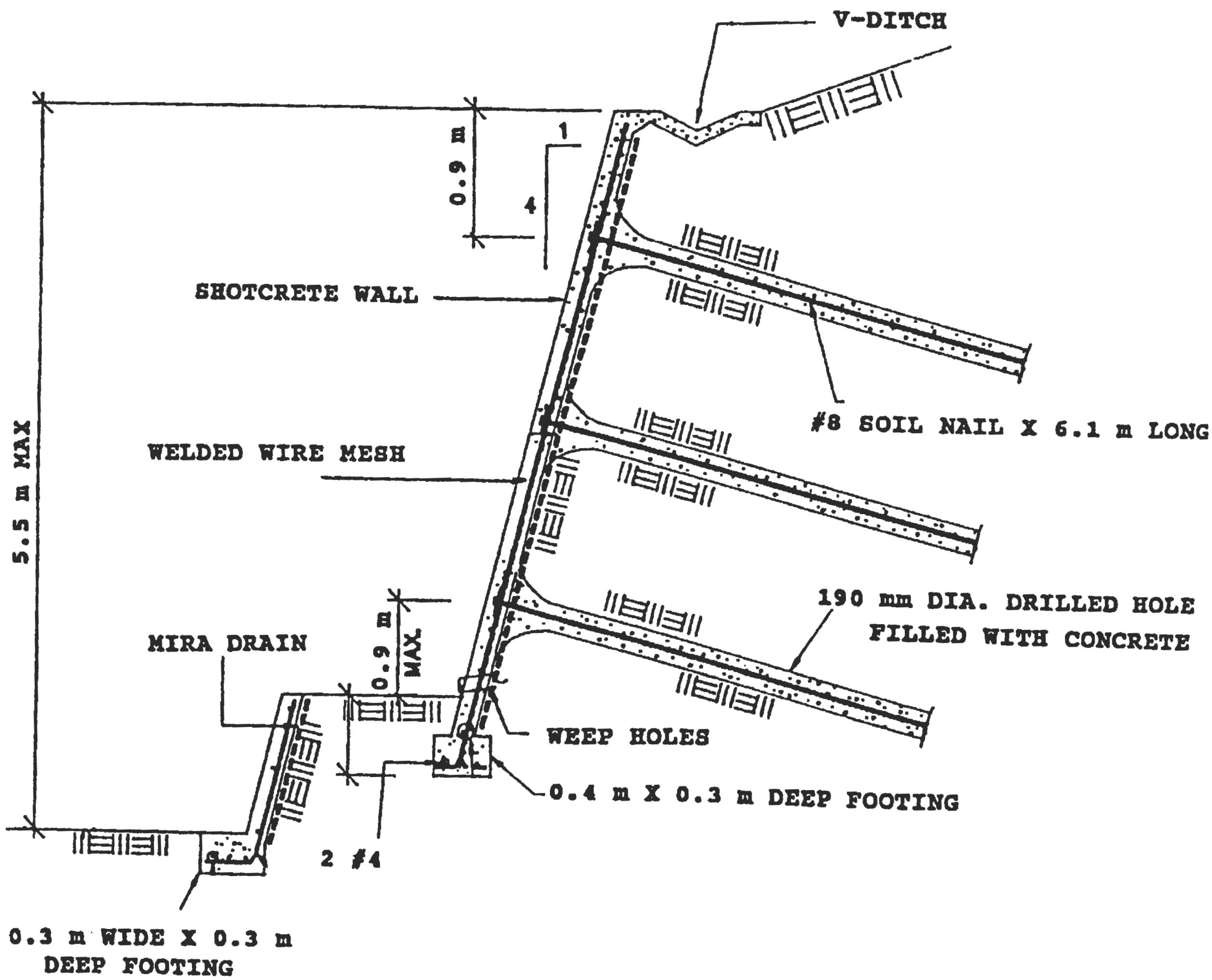

Figure 3.—Cross section of a permanent soil-nailed retaining structure located in San Ramon (NME wall), Calif. (Barar, 1990).

servable damage was noted on the Santa Cruz structure after the earthquake. The walls are described in greater detail below.

MOUNTAIN VIEW, 2350 EL CAMINO REAL (ECR WALL)

Nearly 280 m^2 of soil-nailing construction was used to provide temporary shoring of an excavation for an office building. The concrete wall for the new structure was to be poured in front of the soil-nailed concrete facing. The subsurface soil consisted of gravelly and clayey sand. The shear-strength parameters used in design were c = 9.6 kN/m^2 and f = 30°, while the soil unit weight was assumed to be 17.3 kN/m^3. The soil-nailed wall was completed by May of 1989 and the excavation was still open when the earthquake struck. Postearthquake observations revealed only a few shallow hairline cracks in the concrete facing, typical of flexural cracking if the facing is considered as a vertical slab with the nails acting as reaction points. Note in table 1 that the facing of this wall was relatively thin (100 mm), while the estimated horizontal acceleration was considerable (0.21 to 0.27 g).

MOUNTAIN VIEW, KAISER PERMANENTE PARKING GARAGE (KPG WALL)

Approximately 380 m^2 of shoring was provided for the construction of a parking garage. Soil nailing was used only on one side of the excavation, while a combination of other shoring techniques were used on the remaining sides. The soil conditions at the site consisted of stiff sandy to clayey

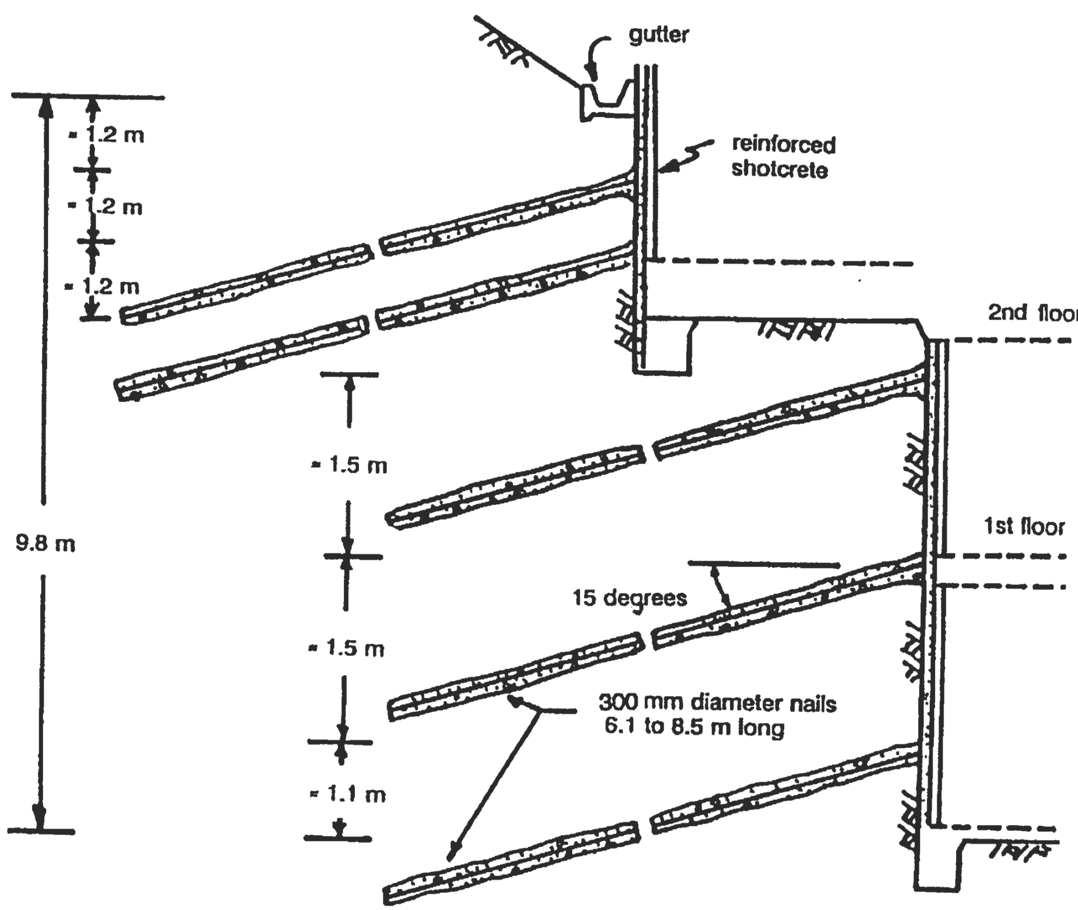

Figure 4.—Cross section of a soil-nailed retaining wall constructed on a steep hill in San Francisco (CVA wall), Calif., to provide adequate space for an apartment complex (Felio and others, 1990).

silt overlying silty to sandy clay. The shear strength parameters used in design were $c = 23.9$ kN/m^2 and $f = 14°$, while the soil unit weight was assumed to be 18.8 kN/m^3. The construction of the shoring was completed just 8 days before the earthquake. The postearthquake observations revealed no visible distress to the soil-nailed wall, while the opposite side

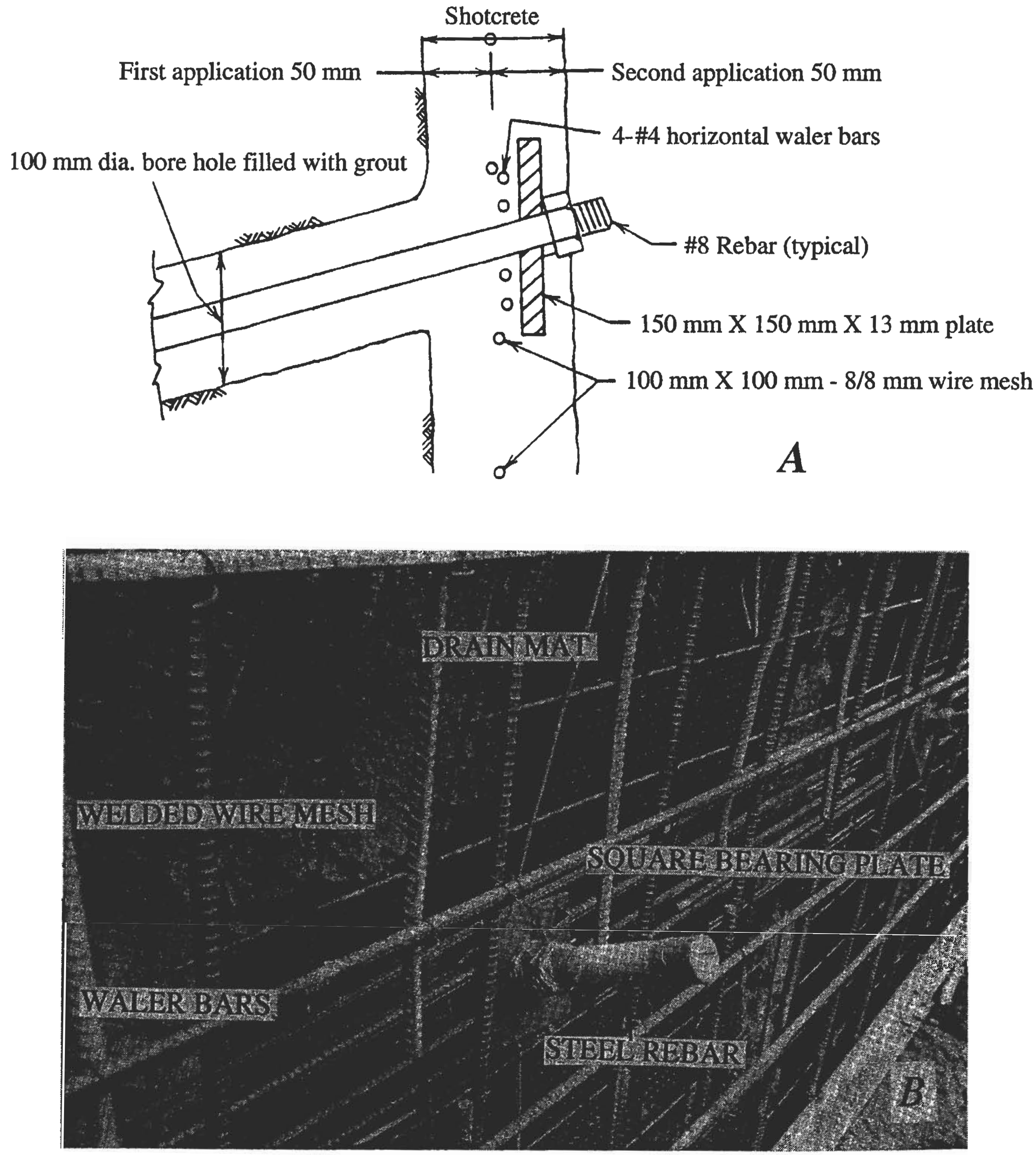

Figure 5.—Connection between grouted nail and facing. *A*, Cross section of a typical connection (from Koerner, 1984). *B*, Strong reinforcement around the nail tip.

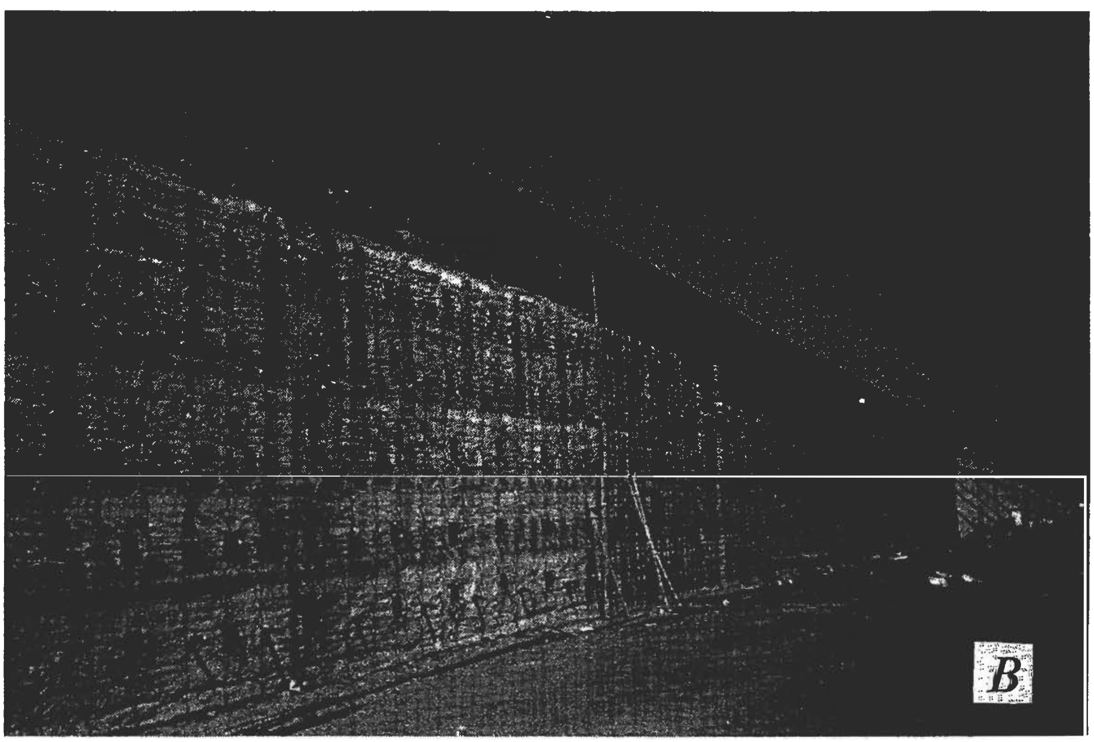

Figure 6.—Soil-nailed structures recently completed in California. *A*, Soil-nailed excavation for an underground structure of a building. *B*, A highway retaining soil-nailed wall showing different stages of the construction of permanent facing.

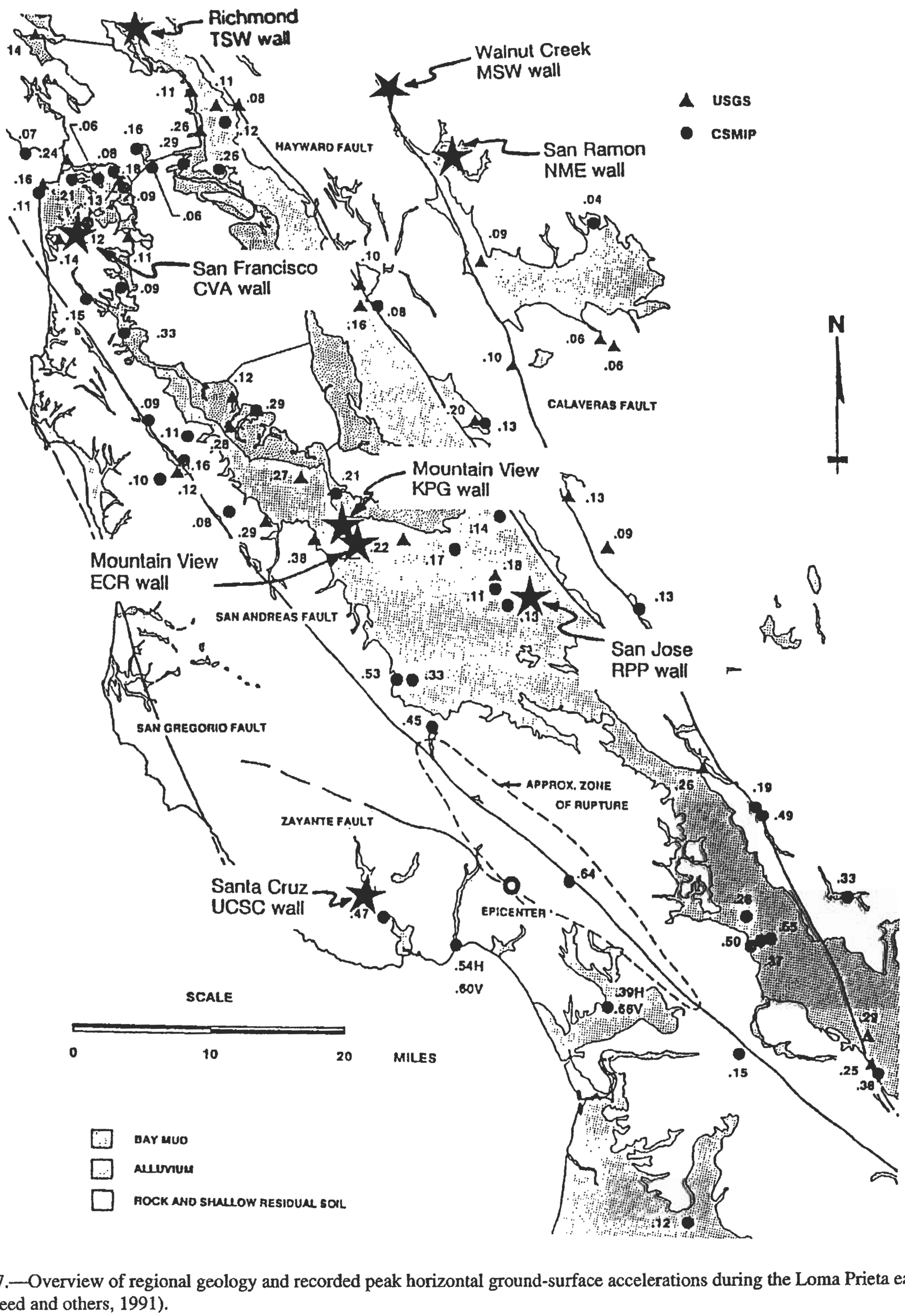

Figure 7.—Overview of regional geology and recorded peak horizontal ground-surface accelerations during the Loma Prieta earthquake (from Seed and others, 1991).

of the excavation, which used cantilevered soldier beams with a concrete facing between the beams, revealed some vertical hairline cracks in the facing.

SANTA CRUZ, UNIVERSITY OF CALIFORNIA AT SANTA CRUZ (UCSC WALL)

Approximately 350 m^2 of shoring was required to construct a new science library on the UCSC campus. The soil conditions at the site consisted of sandy silt to sandy clayey silt extending from the ground surface to a depth of approximately 6.4 m. The soil has an average dry unit weight of 13.8 kN/m^3 and a moisture content ranging from 26.2 percent in the clayey silt near the surface to 13.9 percent in the sandy silt at 6.4 m. Shear strength properties used in design were c = 23.9 kN/m^2 and f = 25°. The cross section at the highest location of the wall is shown in figure 2. Construction of the wall was completed on September 28, 1989, less than 3 weeks before the earthquake. Since three sides of the excavation were soil nailed, at least one side may have been subjected to the full strength of the earthquake, which produced in the vicinity of the excavation peak horizontal ground-surface accelerations of about 0.47 g. It should be noted that this wall was located closest to the epicenter and presumably was subjected to the strongest shaking, while at the same time it had the smallest length ratio and thinnest facing among the nine walls examined (see tables 1 and 2). Prior to the earthquake, some wall and column spread footings had been poured (see fig. 2). A postearthquake inspection revealed significant cracking in the concrete of the footings. This cracking was not attributed to shrinkage since foundations constructed after October 17 showed fewer cracks. As opposed to that, the inspection of the soil-nailed wall after the earthquake revealed no cracking. A week after the earthquake, nine nails were tested to 150 percent of their design pull-out load. The tests showed no loss in the carrying capacity of the nails due to the seismic activity.

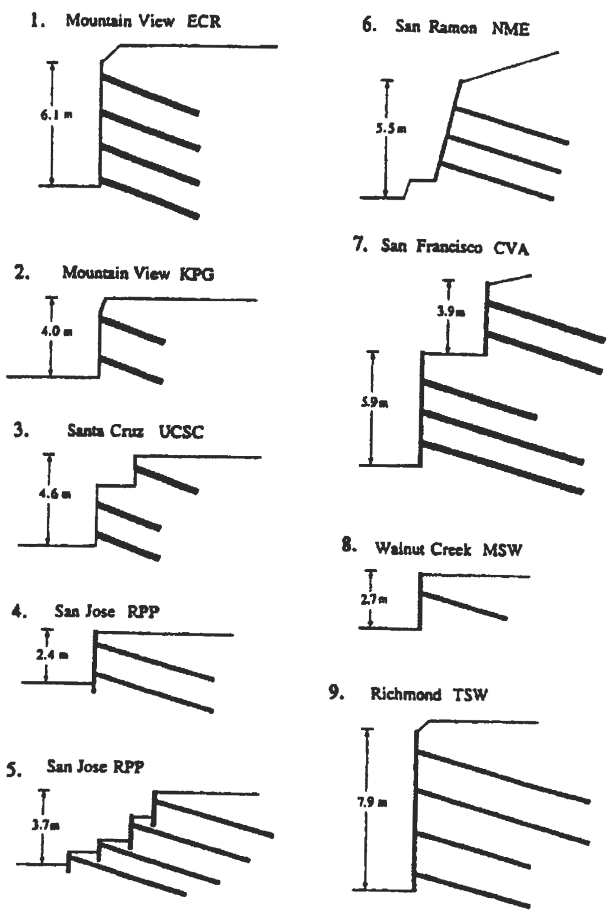

Figure 8.—Dimensions of the investigated soil-nailed walls.

SAN JOSE, RIVERPARK PROJECT (2 RPP WALLS)

These two retaining walls were designed and built as permanent structures along the Guadalupe River in San Jose, approximately 40 km north of the epicenter. The subsurface soil consists of silty and sandy clays to a depth of about 4.5 to 6 m. According to the geotechnical report, these clays have an intermediate to high plasticity with an approximate average dry unit weight and moisture content of 14.1 kN/m^3 and 22 perecent, respectively, and an undrained shear strength ranging from 72 to 240 kN/m^2, as interpolated from static cone penetration tests. The clays are underlain by a 3 m zone of dense, clayey, silty, gravelly sand with an average dry unit weight and moisture content of about 17.3 kN/m^3 and 15 percent, respectively. The shear strength parameters used in design were c = 23.9 kN/m^2 and f = 0°, while the total unit weight was assumed to be 19.6 kN/m^3. Since these are permanent walls, the concrete surface was finished off with architectural concrete and in some places clad with granite. The postearthquake observations revealed no signs of distress.

SAN RAMON, NATIONAL MEDICAL ENTERPRISES COMMUNITY HOSPITAL (NME WALL)

This soil-nailed retaining structure forms a part of a permanent retaining wall used for the roads and landscape that surround the medical center. The wall cross section is shown in figure 3. According to the geotechnical report, the soil conditions consist mainly of engineered fill up to a maximum depth of 24 m, generated from cut-and-fill operations performed previously. Therefore, the soil-nailed retaining structure was built in fill material. The fill consists of sandy and silty clay of moderate to high plasticity, with an average dry unit weight and moisture content of approximately 17.1 kN/m^3 and 18 percent, respectively. The shear strength prop-

Table 1.—*Summary of soil-nailed walls investigated*

Project No.	Location (see fig. 7)	Height of wall (m)	Nail details: Spacing (m)	Nail details: Length (m)	Nail details: Diameter: Grout (mm)	Nail details: Diameter: Rebar (mm)	Nail details: Inclination (degrees)	Facing thickness (mm)	General soil Type	Soil unit weight (KN/m^3)	Shear strength design parameters: c (KN/m^2)	Shear strength design parameters: ϕ (degrees)	Estimated horizontal ground surface acceleration (g)
1	Mountain View ECR	6.1	1.7V 1.5H	5.2	300	32	20	100	clayey sand	17.3	9.6	30	0.21 to 0.27
2	Mountain View KPG	4.0	2.0V 1.5H	3.4	300	25	20	100	sandy to clayey silt	18.8	23.9	14	0.21 to 0.27
3	Santa Cruz UCSC	4.6 2-levels	1.5V 1.8H	3.4	300	32	20	75	sandy to clayey silt	18.1	23.9	25	0.47
4	San Jose RPP1	3.7 4-levels	1.8V 1.8H	6.1	180	25	15	200	alluvial clay, silt and sand	19.6	23.9	0	0.10 to 0.15
5	San Jose RPP2	2.4	1.8V 1.8H	6.1	180	25	15	200	alluvial clay, silt and sand	19.6	23.9	0	0.10 to 0.15
6	San Ramon NME	5.5	1.8V 1.8H	6.1	190	25	15	150	engineered fill	18.8	47.9	0	0.05 to 0.10
7	San Francisco CVA	9.8 2-levels	1.8V 1.5H	6.1 to 8.5	300	25	15	200	fill over silty clay and highly weathered siltstone	19.6	9.6	35	0.10 to 0.15
8	Walnut Creek MSW	2.7	(one row) 1.8H	4.6	190	25	15	200	fill over medium stiff to stiff clay	18.8	14.4	28	0.01 to 0.10
9	Richmond TSW	7.9	1.8V 1.8H	6.1 to 9.1	190	25	15	200	alluvium deposits	18.8	19.1	28	0.05 to 0.15

erties used in design were $c = 47.9$ kN/m^2 and f = 0°. Since this is a permanent structure, the facing of the soil-nailed wall was finished with a colored architectural concrete finish. The postearthquake walk-through revealed that the surface of the concrete remained smooth and free of cracks.

SAN FRANCISCO, CRESTA VISTA APARTMENTS (CVA WALL)

This wall demonstrates the unique concept of using soil nailing on a permanent basis to retain the slope and cut on a

Table 2.—*Dimensionless ratios for San Francisco area soil-nailed walls*

Project No.	Location	Length ratio $\left(\frac{\text{max. nail length}}{\text{excav. height}}\right)$	Bond ratio $\left(\frac{\text{hole dia. x nail length}}{\text{H. spacing x V. spacing}}\right)$	Strength ratio (10^{-3}) $\left(\frac{(\text{nail dia.})^2}{\text{H. spacing x V. spacing}}\right)$
1	Mountain View ECR	0.85	0.61	0.40
2	Mountain View KPG	0.85	0.34	0.21
3	Santa Cruz UCSC	0.74	0.38	0.38
4	San Jose RPP	1.6	0.34	0.19
5	San Jose RPP	2.5	0.34	0.19
6	San Ramon NME	1.1	0.36	0.19
7	San Francisco CVA	1.5 to 1.7	0.68 to 0.94	0.28
8	Walnut Creek MSW	1.7	not applicable	not applicable
9	Richmond TSW	1.2	0.36 to 0.53	0.19

Table 3.—*Dimensionless ratios for soil-nailed walls*

	Drilled and grouted in granular soils (Bruce and Jewell, 1986, 1987)	Drilled and grouted in morraine and marl (Bruce and Jewell, 1986, 1987)	San Francisco walls
Length ratio	0.5 to 0.8	0.5 to 1.0	0.7 to 0.25
Bond ratio	0.3 to 0.8	0.15 to 0.20	0.34 to 0.94
Strength ratio (10^{-3})	0.4 to 0.8	0.1 to 0.25	0.19 to 0.40

steep hill to make room for the development of a housing project. The wall cross section is shown in figure 4. The 9.8-m-high soil-nailed structure was constructed at the toe of a 45.7-m-high slope to allow for the construction of apartment units. The wall is about 90 m long and consists of two levels. Due to the permanent nature of the structure, a 200 mm reinforced concrete facing and a small footing at the base were used. The soil conditions at the site can be described as colluvium and residual soil deposits. The design parameters used were cohesion c = 9.6 kN/m^2 and an angle of internal friction of f = 35°. The inspection that took place 3 days after the earthquake showed no signs of distress to the wall and no indications of lateral movements or tension cracks in the hill behind the wall.

WALNUT CREEK, MINI STORAGE FACILITY (MSW WALL)

The project consisted of a three-story building with two levels above grade and one level of basement below The soil-nailed wall was integrated into the final basement wall. The soil at the site consists mainly of fill material up to 3 m depth, including a nonuniform mixture of gravel, sand, and clay. The underlying soil consists of stiff silty clay. The average dry unit weight and moisture content of the fill is 16.5 kN/m^3 and 20 percent, respectively. The shear strength parameters assumed in design were c = 14.4 kN/m^2 and f = 28°, while the total unit weight of the soil was assumed to be 18.8 kN/m^3. The postearthquake observations revealed no signs of distress on the surface of the wall or at grade behind the wall.

RICHMOND, TEMPORARY SHORING WALL (TSW WALL)

Soil nailing was used here to construct a temporary shoring wall which has the tallest single-level vertical face of any of the walls examined in this paper. A permanent retaining wall was eventually built in front of the soil-nailed wall. Unfortunately, soil stratigraphy data is not available for this site. However, the shear strength parameters used in design were c = 19.1 kN/m^2 and f = 28°, while the unit weight of the soil was assumed to be 18.8 kN/m^3. A walk-through of the site following the earthquake did not reveal any signs of distress attributable to seismic activity.

METHODS OF ANALYSIS

Most of the current design methods for soil-nailed retaining structures under static loads are derived from classical slope-stability analyses, which incorporate a limit equilibrium approach. Accordingly, they evaluate global factors of safety along assumed failure surfaces such as those shown in figure 9. They are usually referred to as the German method (Stocker and others, 1979; Gassler and Gudehus, 1981; Lambe and Jayaratne, 1987), Davis method (Shen and others, 1981; Bang and others, 1992), French method (Schlosser and others, 1983), and Caltrans method (computer program SNAIL: Caltrans, 1993). The differences in the methods result from the definition of the factor of safety, assumed failure surface shape, and the assumed contribution of the soil nails to the stability. In that respect, the methods are contradictory, and because of the lack of full-scale observations of actual failure mechanisms, different points of view about their applicability have emerged.

The German method (fig. 9A) assumes a bilinear failure surface passing through the toe of the excavation. The failing soil mass is broken into two parts. The first part contains most of the nailed soil mass, while the second part forms the active earth pressure wedge behind it—behind the "soil-nailed gravity wall." The analysis considers the tensile and pull-out resistance of the nails crossing the failure surface and, of course, the forces of interaction between the nailed mass and active wedge behind it. The assumed failure surfaces are consistent with the concept of soil nailing, that is, the nailed soil mass behaves like a reinforced block.

The Davis method incorporates a parabolic failure surface that also passes through the toe, as shown in figure 9B. The sliding surface either passes entirely through the nails or intersects the ground surface somewhere beyond the reinforced zone. In the analysis, the tensile and pull-out resistance of the nails crossing the failure surface are considered the governing stabilizing forces. Because of its successful track record and easy implementation, it has been a popular design method in the United States. This has been the case in spite of the fact that the assumption of a parabolic slip surface (which does not change slope when crossing from the nonreinforced to the reinforced zone) has not been adequately verified by laboratory or field tests.

The French method follows procedures similar to the Davis method, but assumes a circular failure surface passing entirely through the nails, as shown in figure 9*C*. But, unlike the previous two methods, this method considers the shear and bending of the nails, which adds to the complexity of the analysis.

The Caltrans method also assumes a bilinear failure surface, just like the German method. However, unlike in the German method, the bilinear failure surface may pass entirely through the nails (see fig. 9*D*).

More recently, a kinematical limit analysis approach has been proposed for the design of soil-nailed retaining structures (Juran and others, 1990). It differs from the other analysis procedures in that it suggests a method for estimating nail forces. In this way, it may provide a check on local stability at each level of nail reinforcement. The method assumes that the failure surface is defined by a log-spiral passing partially through the nails and that the failure occurs by rotation of a quasi-rigid body along this surface.

All of the San Francisco walls examined in this study were designed using a modified version of the Davis method (Barar, 1990; Felio and others, 1990). Seismic forces were accounted for by using an equivalent static horizontal force $H = W \times k_h$, applied at the center of gravity of the potentially unstable soil nailed mass, where W is the weight of the moving soil mass and k_h is the horizontal seismic coefficient.

It should be mentioned at this point that the Davis method, as well as the German and Caltrans methods, has a certain degree of the inherent conservatism in that the potential stabilizing effects of the shear and bending resistances of the nails are ignored. New studies (Jewell and Pedley, 1992; Federal Highway Administration, 1993) show, however, that the effects of bending stiffness are small. Also, the contribution of the steel reinforced facing to the strength of the system is unaccounted for. The lack of full understanding of the role of facing in the global and local stability apparently led to the difference by a factor of 3 (75 mm vs. 200 mm) in the thicknesses of the facing among the nine walls under consideration. Some designers and construction companies feel comfortable with thinner facing, while some prefer more conservative thicker facing. Figure 5, for example, illustrates a rather heavily reinforced facing with a sturdy nail contact. The role of the facing in soil reinforcing stability is just beginning to be studied as a separate issue (Tatsuoka, 1992), and it should definitely be given more attention in the future.

The factors of safety for the CVA and TSW soil-nailed walls obtained by the Davis method, modified to account for earthquake forces by the pseudostatic technique, are presented in table 4. The location of the assumed failure surfaces that yield minimum factors of safety for the TSW wall are shown in figure 10. In general, the factors of safety are relatively low, especially for the range of estimated peak horizontal ground accelerations during the Loma Prieta earthquake. According to such low factors of safety, and given the fact that some soil-nailed structures were probably subjected to much larger horizontal forces, some visible damage should have occurred during the earthquake. This should have been expected in particular for the UCSC Wall in Santa Cruz, which had the smallest length ratio and facing thickness, and yet is likely to have undergone horizontal seismic forces as large as 0.4 *g*. The lack of visible damage on any of the walls, except very thin cracks on the ECR wall, suggests that either the design, analysis, or construction, or most likely their combination, may have been more conservative than necessary. The lack of damage also indicates that the assumed failure surface and mechanism of failure of the Davis method may not be fully appropriate for the nine walls treated here. In the following section, the components of the analysis, design, and construction that appear to be on the conservative side, and therefore could be responsible for such excellent seismic performance, are discussed.

POSSIBLE REASONS FOR THE OBSERVED BEHAVIOR

Since soil nailing is a relatively new soil-stabilization technique, with very little practical experience of full-scale static failures and practically no experience of seismic failures, the design and construction are usually quite conservative. The preliminary design of a soil-nailed retaining structure proceeds much like that of retaining walls, by trial and error. Based mainly on the expected excavation height and the soil strength properties, tentative characteristics of nails and facing (length, diameter, horizontal, and vertical spacings of nails, and the thickness and reinforcement of facing, etc.) can be assumed and some sort of stability analysis performed. The assumed values and characteristics depend primarily on the designer's experience with other satisfactorily constructed soil-nailed walls, which may lead to an overly conservative design, and to a lesser extent on charts and dimensionless parameters derived by others, such as those by Bruce and Jewell (1986, 1987) and Guilloux and Schlosser (1982). Table 3 shows, for example, that the length ratios and bond ratios for the nine walls considered here are on the conservative side in comparison with the values suggested by Bruce and Jewell (1986, 1987).

The main components of the conservative design and construction for seismic loads include (1) conservative and most probably unrealistic assumption of the failure mechanism, (2) no consideration of the contribution of the facing in the stability analysis, and (3) conservative construction due to the lack of field experience and understanding of the various aspects of soil-nailed excavation seismic response. The first two components are discussed below.

FAILURE-MECHANISM ASSUMPTION

Due to a lack of full-scale observations of failures and corresponding failure mechanisms under both static and seismic loads, there is currently no consensus among designers on which failure mode is the most realistic among the four basic modes presented in figure 9. To cast more light on possible modes of failures under dynamic loads, two series of dynamic centrifuge tests were conducted, one in 1991 (Tufenkjian and others, 1991; Tufenkjian and Vucetic, 1992; Vucetic and others, 1993) and the other in 1996 (Vucetic and others, 1996). Figures 11 and 12 show the main features of the models tested and results obtained in 1991.

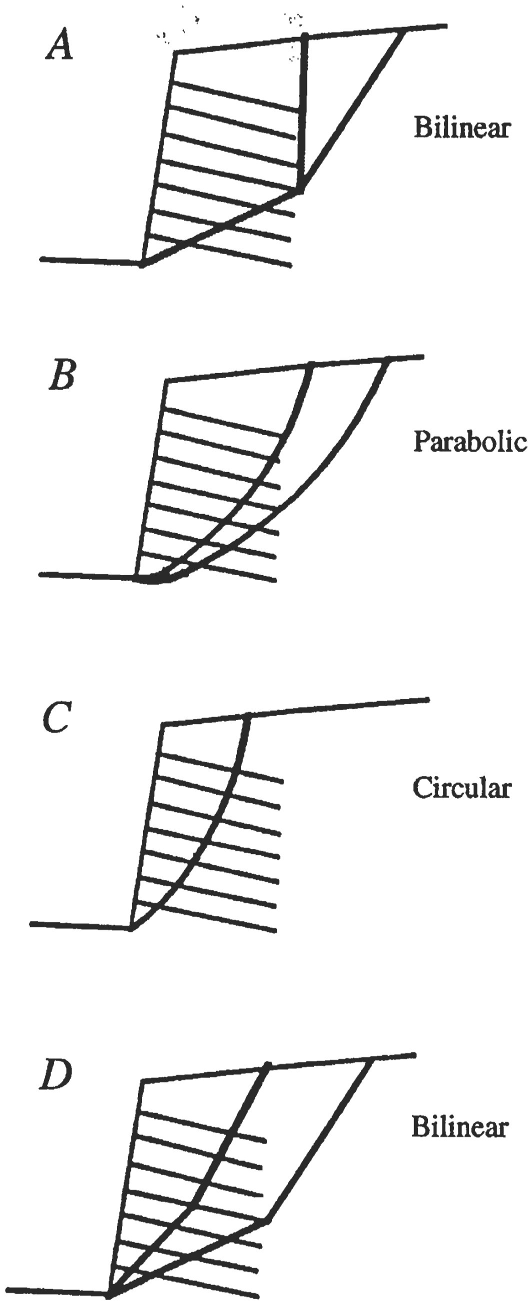

Figure 9.—Assumed failure surfaces used in analyses. *A*, German Method. *B*, Davis Method. *C*, French Method. *D*, Caltrans Method.

The centrifuge tests were performed at the Rensselaer Polytechnic Institute (RPI) Geotechnical Centrifuge Research Center on a 3-m radius Accutronic 665-1 centrifuge (Elgamal and others, 1991). The scale factor was 50 in all of the tests. Accordingly, to simulate prototype geostatic stresses, the models had to undergo a centrifugal acceleration of 50 *g*. For dynamic testing, a servo-hydraulic earthquake simulation shaker mounted on the centrifuge platform was used. Four models were tested in 1991.

They represented 7.6-m-high soil-nailed excavations with grouted nails, corresponding roughly to an excavation height of a two- to three-story underground garage. The effects of two important characteristics of soil-nailed structures were tested: the length of nails (expressed in terms of the length ratio), and the axial and flexural rigidities of the nails.

Three length ratios were tested, 0.33, 0.67, and 1.0, which could be characterized as the ratios corresponding to short, medium, and long nails (see table 3). These ratios cover approximately five out of the nine walls listed in table 2. The four other walls have very large length ratios between 1.5 and 2.5. Two axial and flexural rigidities of the nails were used, one that can be considered regular and the other than can be considered small. By varying the axial and flexural rigidities of the soil nails, their effect on the failure surface geometry and stability could be assessed. As shown in figures 11*A* and 11*C*, three displacement transducers (LVDT's) were used to record the lateral movements of the facing and the vertical soil settlement behind the facing. During dynamic loading, four accelerometers were utilized to measure the accelerations of the model box and in various locations within the model box. The soil used in the experiments was fine sand. The sand was partially saturated to generate an apparent cohesion, necessary for a rough simulation of in-situ cohesion and cementation. Other details of the 1991 testing are described by Vucetic and others (1993).

Figures 11*B* and 12 show a typical failure mechanism obtained in the tests under horizontal dynamic loads. In all four tests the failure surface never started at the ground surface above the nails. Instead, it started at the ground surface behind the ends of the nails. Figure 11*B* reveals that the failure mechanism involves three soil "zones" and two soil "blocks," with two failure surfaces, one of which consists of two parts. The primary failure surface extends from behind the nails at the ground surface down to the end of the second row of nails, at which point it changes curvature and continues down to the bottom of the excavation through the toe. The secondary failure surface develops within the sliding soil mass and divides zones 1 and 3. Such deformation patterns after the tests point to the following failure mechanism. The soil above the second row of nails in zone 1 moves horizontally under

Table 4.—*Calculated factors of safety using the Davis method (see also Hudson, 1990)*

Horizontal acceleration coefficient, k_h	Cresta Vista apartments CVA	Temporary shoring wall TSW
0	1.31	1.19
0.1	1.14	1.06
0.2	1.00	0.94

Indicates the factors of safety corresponding to the range of estimated horizontal peak ground-surface accelerations near the site.

large inertial forces as a relatively rigid block held together by the nails. Consequently, the soil in zone 2 is pushed outward by the horizontal friction along the interface between the upper zone 1 and the lower zone 2. Accordingly, the failure surface passes through the bottom row of nails. In such a mechanism, the bottom nails obviously act as anchors between the back soil and the facing, while the top nails hold the soil together in the upper part of the excavation. As zones 1 and 2 move horizontally outward during seismic shaking, the lateral stresses in zone 3 are greatly reduced. Consequently, zone 3 represents a typical failure wedge behind a retaining wall, the retaining wall being zone 1. This mechanism and kinematics of the soil movement resemble the geometry of German method for static stability evaluation, shown in figure 9*A* (Gassler and Gudehus, 1981), while they contradict the assumption that rotation of one monolith occurs along a continuous circular or parabolic failure surface.

To examine the factors governing the failure corresponding to the above mechanism, the forces and factors of safety for the TSW Wall in Richmond (see fig. 10 and table 4) are reevaluated. Figure 13 shows the assumed failure surfaces and governing forces, while figure 14 shows the corresponding polygons of forces. To account for the effects of dynamic horizontal forces the pseudostatic method of analysis is used again, where the dynamic action is represented by the static horizontal force $H = W \times k_h$. Two definitions of the factor of safety, *FS*, based on the German

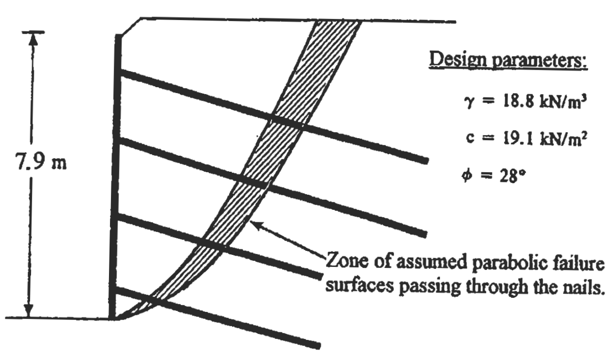

Figure 10.—Assumed failure surfaces passing through the nails for the factor of safety evaluation of the TSW wall in Richmond using Davis method and its modifications.

type of failure mechanism are considered below. First is the definition for static stability proposed by Stocker, Korber, Gassler and Gudehus (1979), which is adapted here for the dynamic stability by adding to the poligon of forces a horizontal force $H = W \times k_h$. The second is the definition proposed and used by Caltrans (1993). Accordingly, the two methods for the calculation of *FS* are called here the SKGG method and the Caltrans method.

According to the SKGG method, the factor of safety is calculated as

$$FS = \frac{Z_a}{Z_e}, \tag{4}$$

where Z_a = cumulative axial pull-out force of the nails beyond the failure surface and Z_e = mobilized cumulative axial force of the nails beyond the failure surface. Therefore, the entire factor of safety is based on the pullout of the nails. For the TSW wall, *FS* for different seismic coefficients, k_h, was calculated. The results of this calculation are presented in terms of the *FS* vs. k_h relationship in figure 15. On the same figure the equivalent relationship between *FS* and k_h obtained by the Caltrans method is presented as well.

The bilinear failure surface assumed in the Caltrans method is similar to the failure surfaces assumed in the SKGG method and thus to the deformation patterns and failure surfaces observed in the centrifuge tests. In fact, as indicated in figure 13, the forces and their positions relative to the free bodies for the TSW wall are the same for the SKGG and Caltrans methods. However, the methods differ fundamentally in their definitions of the factor of safety. The Caltrans method applies a unique factor of safety to the soil cohesion, *c*, soil friction angle, ϕ, and the cumulative nail pullout force, Z_a :

$c' = c/FS$ = mobilized cohesion,
$\phi' = \tan^{-1}[(\tan\phi)/FS]$ = mobilized friction angle, and
$Z_e = Z_a/FS$ = mobilized pullout force.

The method then utilizes these "mobilized" parameters in the force equilibrium equations to solve for the interwedge forces, *F* (see figs. 13 and 14). Since these forces must be equal in magnitude and opposite in direction, assumed value of *FS* is systematically varied until this condition is fulfilled, which then yields the corresponding *FS* used in design. The Caltrans method has been coded into a computer programming language and can be run on personal computers. The computer program is called SNAIL (Caltrans, 1993). By inputting the geometry of the slope and details of the soil strength and nail properties, the program can systematically vary the location of the bilinear failure surfaces, until the one producing the lowest factor of safety is found. The program also has an option to calculate the *FS* for a specified surface, as well as for considering seismic forces by the pseudostatic technique.

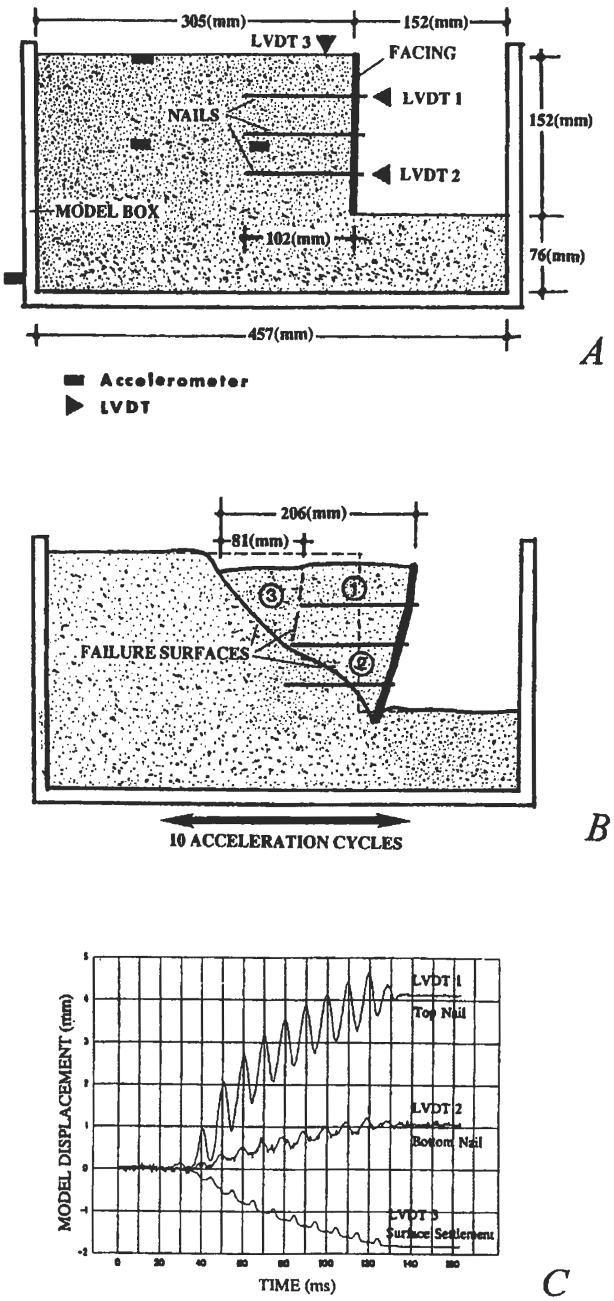

Figure 11.—Features of the typical centrifuge model test with length ratio of 0.67 (Tufenkjian and others, 1991; Vucetic and others, 1993). *A*, Longitudinal cross section of the soil-nailed excavation centrifuge model box. *B*, Failure mechanism obtained in the centrifuge due to strong horizontal shaking. *C*, Typical records of soil mass movements during shaking with 10 cycles of 0.27 *g* cyclic acceleration amplitude.

Several interesting conclusions can be derived from figure 15. First is that for $FS=1$ (the conditions of the failure of the wall), $k_h \approx 0.37$ is obtained by both methods. Second, this k_h value is much larger than $k_h \approx 0.1$ to 0.20 corresponding to $FS = 1$ calculated according to the Davis method (see table 4). Third, $k_h \approx 0.37$ is in relatively good agreement with the amplitude of the cyclic acceleration of 0.45 g that was required in the centrifuge testing for the failure of the soil-nailed excavation model of similar length ratio (see Vucetic and others, 1993). And fourth, the *FS* versus k_h relationship for the SKGG method has a singularity point, while the same relationship for the Caltrans method does not.

Based on these observations it can be concluded that the failure mechanism according to the German method seems to be more appropriate than that of the Davis method. However, figure 15 also shows that using the SKGG method to calculate the factor of safety may not be suitable for the calculation of stability involving the horizontal forces ($W \times k_h$), because it is too sensitive to the variation of k_h. By varying k_h from 0.3 to 0.4, *FS* varies from -1 to $-\infty$ and then from $+\infty$ to 0.7—that is, as noted above, the function $FS = f(k_h)$ has a singularity point. The reason for such sensitivity of *FS* with respect to k_h can be easily understood from the polygon of forces in figure 14*B*. For example, if the force $H_1=W_1 \times k_h$ is increased by only 15 percent, the Z_e force will double—that is, change by 100 percent. Consequently, the $FS=Z_a/Z_e$ will change dramatically too. Such sensitivity of *FS* comes from the fact that *FS* is defined on the basis of forces which are of secondary importance for the stability of the structure. In other words, force Z_e is relatively small compared to the other forces in the polygon. More dominant forces are apparently the reaction force Q_1 and cohesion force C_1 mobilized along the failure surface.

In the configuration of forces such as shown in the polygon in figure 14*B*, corresponding to the German method type of failure mechanism, the role of the nails is predominantly to interact with the soil and form the nailed block. Such a large soil block is evidently seismically very stable, and its stability is governed by the large forces of friction and cohesion at the interfaces with the surrounding soil, not by the small forces such as Z_e. This, of course, would change if Z_e is relatively large, that is, corresponding to very long nails installed deep beyond the failure surface. In such a case the kinematics of the failure would be different too. Instead of predominantly sliding along the failure surfaces, the facing and thus the soil mass would be forced to rotate around the bottom row of nails which are anchored beyond the failure surface.

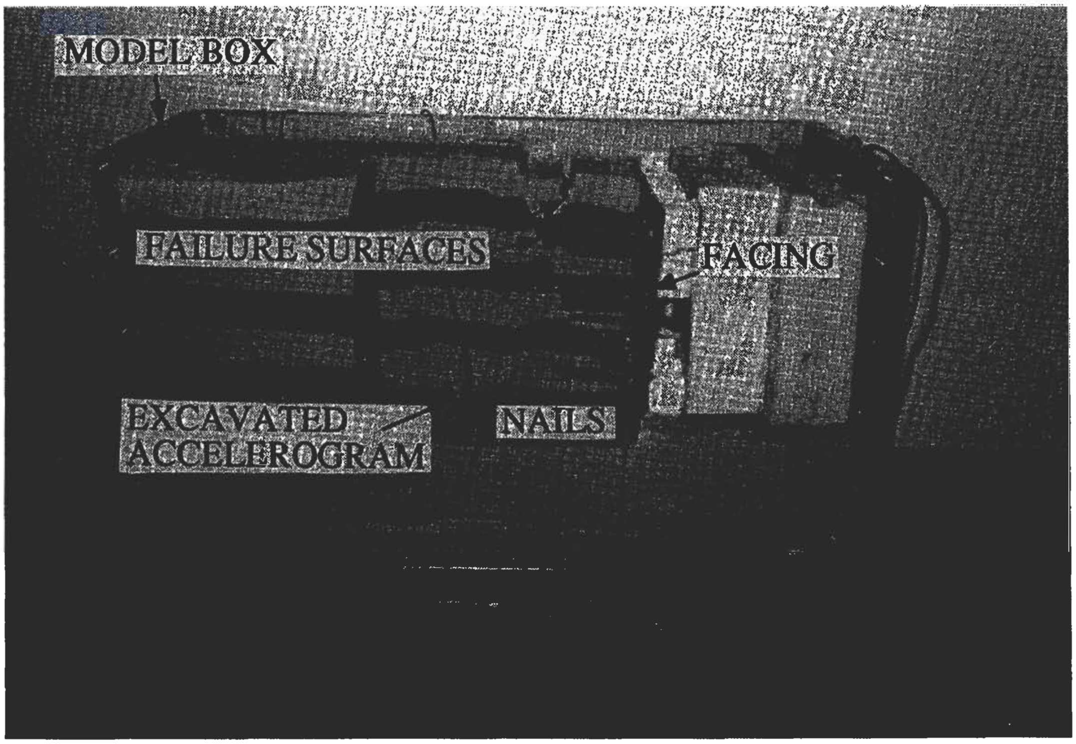

Figure 12.—Excavated model of a test with the length ratio of 0.33 (short nails) shaken by 10 cycles of the uniform acceleration amplitude of 0.10 g.

As opposed to the SKGG method of stability evaluation, the Caltrans method does incorporate all significant forces in the definition of FS and consequently yields a more meaningful relationship between FS and k_h. This, along with the above discussion, leads to the following conclusions: (1) the German-method type of failure mechanism seems to be appropriate, (2) the Caltrans method for calculating FS seems to yield appropriate and meaningful results, and (3) the Davis method used in the design of the San Francisco Bay area soil-nailed walls seems to be overly conservative, apparently because it employs an unlikely failure mechanism.

To confirm the above conclusions, the factors of safety of the eight soil-nailed walls subjected to the Loma Prieta earthquake were calculated by the Caltrans method. The four-level RPP wall located in San Jose could not be accurately reproduced using the SNAIL program. The wall geometries were scaled from figure 8 and the soil design parameters were taken from table 1. The calculated factors of safety are shown in table 5, where those corresponding to the range of estimated horizontal ground-surface accelerations near each soil-nailed wall are indicated by double-headed arrows. Note that the factors of safety are generally much greater than unity for the range of estimated ground-surface accelerations, even for the UCSC wall which was subjected to horizontal accelerations in the range of 0.4 to 0.5 g. Recall from Table 4, for example, that the Davis method predicted failure ($FS = 1.0$) for the TSW wall for the acceleration coefficient between

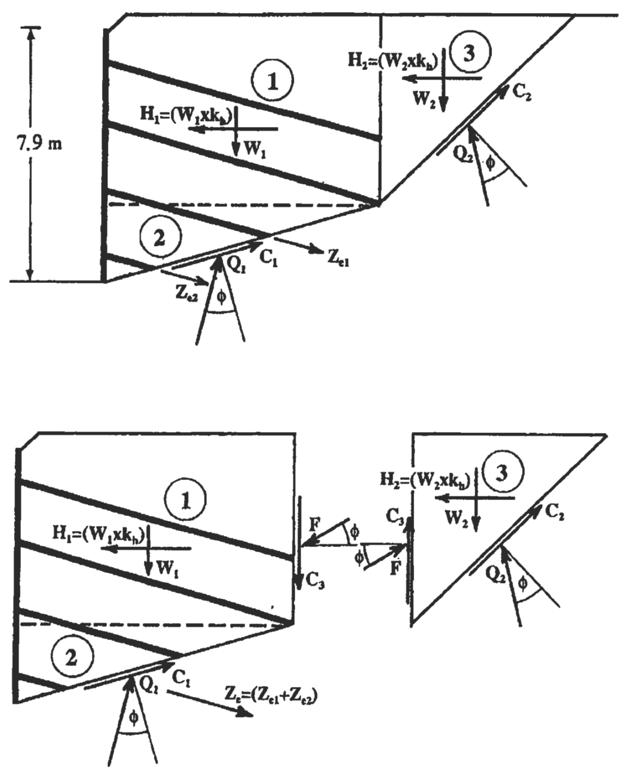

Figure 13.—Failure surfaces and forces for the TSW wall employed in both SKGG and CALTRANS methods for the calculation of the factor of safety according to the German type of failure.

0.10 and 0.20, while the Caltrans method required an acceleration coefficient between 0.4 and 0.5. Such large values of *FS* obtained by Caltrans method are in agreement with the excellent performance of the walls during the earthquake.

ROLE OF FACING

None of the methods discussed above account explicitly for the contribution of the facing in the evaluation of the global factor of safety, although they do incorporate the evaluation of punching shear around the nail connection. In other words, the factor of safety is calculated without considering axial and flexural rigidities of the facing. In that respect, there is no consensus on what the contribution of the facing to the global stability of soil nailed structure really is. However, it is obvious that stronger facing and stronger contact between the facing and the nails will make the nailed soil mass more coherent. The failure mechanism of such a coherent soil mass is likely to be of the German type—that is, behaving as a large seismically stable block. In addition, the inability of the nails (which are firmly fixed to the facing) to move freely decreases the likelihood of local failures, especially in the zones most critically stressed during construction and seismic loading. As suggested earlier, the lack of full understanding of the role of facing in the global and local stability apparently led to the difference by a factor of 3 (75 mm vs. 200 mm) in the thicknesses of the facing between the nine walls considered here.

CONCLUSIONS

Postearthquake inspections of nine soil-nailed walls following the Loma Prieta earthquake indicated superior performance and no signs of distress, even though one of the walls was subjected to horizontal accelerations probably as high as 0.4 *g*. It was shown that the excellent performance may be attributed to a conservative design, generally conservative stability analysis which is mainly the result of an

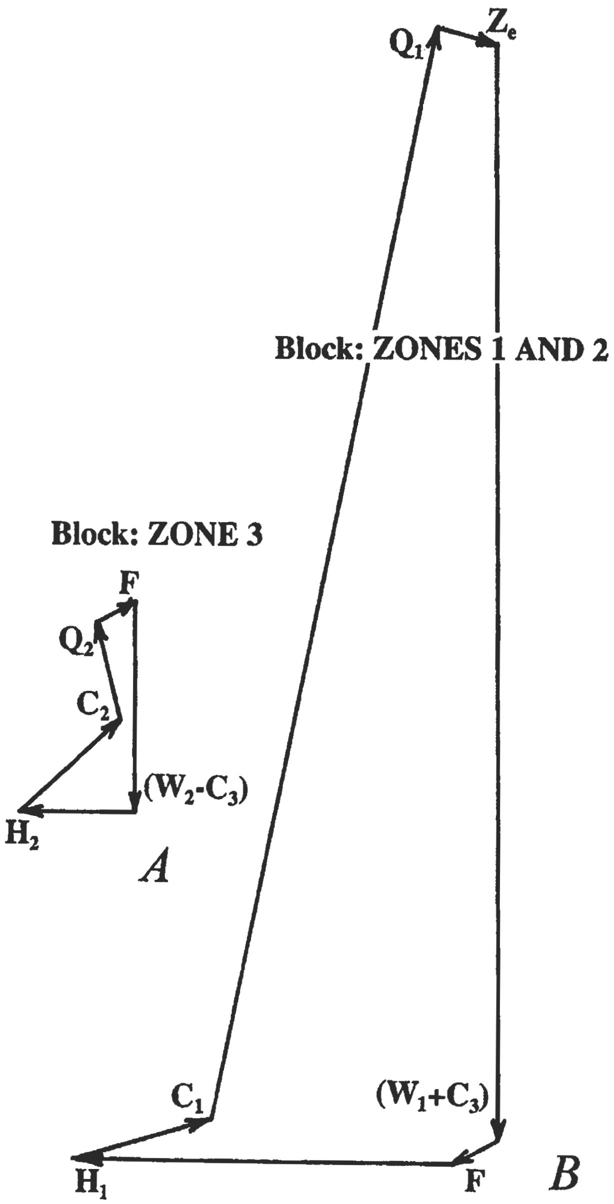

Figure 14.—Polygons of forces identified in figure 13. *A*, Polygon for block comprising zone 3. *B*, Polygon for block comprising zones 1 and 2.

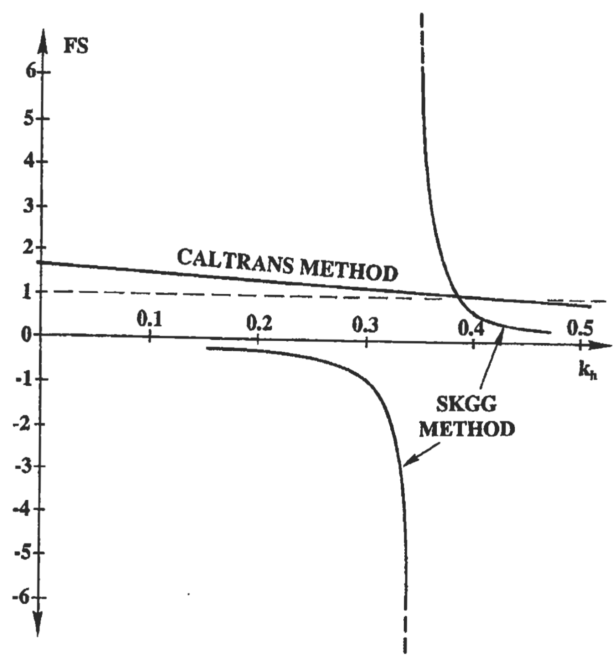

Figure 15.—Variation of the factors of safety, *FS*, with the seismic coefficient, k_h, for the TSW wall.

Table 5.—Calculated factors of safety using the Caltrans method for San Francisco area soil-nailed walls

Horizontal acceleration coefficient, k_h	Soil-nailed wall							
	ECR	KPG	UCSC	RPP (single-level)	NME	CVA	MSW	TSW
0.0	2.28	2.53	3.30	5.15	1.88	1.59	3.07	1.64
0.1	2.00	2.26	2.75	4.44	1.52	1.35	2.68	1.51
0.2	1.74	1.96	2.32	2.80	1.27	1.17	2.35	1.39
0.3	1.51	1.71	2.01	1.99	1.10	1.02	1.97	1.19
0.4	1.31	1.43	1.76	1.54	0.97	0.89	1.64	1.03
0.5	1.16	1.18	1.53	1.26	0.86	0.78	1.39	0.91

Indicates the factors of safety corresponding to the range of estimated horizontal peak ground-surface accelerations near the site.

unlikely mechanism and geometry of failure, and conservative construction.

Because seismic failures of soil-nailed excavations have not occurred in the past and are therefore absent from the literature, dynamic centrifuge testing was performed to provide evidence of the most probable failure mechanism. The centrifuge testing revealed that the most likely failure mechanism is the German type of failure mechanism. Furthermore, a simple analysis of the dynamic centrifuge test results and field observations showed that the Caltrans method for calculating the factor of safety, which also incorporates the German type of failure mechanism, yields very consistent and logical results. Accordingly, the Caltrans method implemented by the computer program SNAIL seems to be an appropriate method for calculating the static and dynamic stability of grouted soil-nailed excavations of the type discussed in this paper.

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REFERENCES CITED

Bang, S., Kroetch, P.P., and Shen, C.K., 1992, Analysis of soil nailing system, *in* Proceedings of the International Symposium on Earth Reinforcement Practice, Fukuoka, Kyushu, Japan: Balkema, Rotterdam, v. 1, p. 457-462.

Barar, P., 1990, The behavior of five soil nailed earth retaining structures during the Loma Prieta earthquake of October 17, 1989: Report prepared for the Department of Civil Engineering, University of California, Los Angeles, 101 p.

Bruce, D.A., and Jewell, R.A., 1986, Soil nailing—Application and practice, part 1: Ground Engineering, November, p. 10-15.

———1987, Soil nailing—Application and practice, part 2: Ground Engineering, January, p. 21-33.

Caltrans, 1993, California Department of Transportation—SNAIL Computer Program, Version 2.06: California Department of Transportation, Office of Geotechnical Engineering, Sacramento, California.

Chapman, K.R., and Ludwig, C., 1993, ADSC participates in FHWA international geotechnology tour: Drilling Foundation, December/January, p. 17-19.

Elgamal, A.-W., Dobry, R., and Van Laak, P., 1991, Design construction and operation of 100 g-ton centrifuge at RPI, in Proceedings of the International Conference Centrifuge 1991, Boulder: Balkema, Rotterdam, p. 27-34.

Felio, G.Y., Vucetic, M., Hudson, M. Barar, P., and Chapman, R., 1990, Performance of soil nailed walls during the October 17, 1989 Loma Prieta Earthquake, *in* Proceedings of the Forty-Third Canadian Geotechnical Conference, Quebec: Canadian Geotechnical Society, p. 165-173.

Federal Highway Administration, 1993, FHWA Tour for Geotechnology—Soil Nailing: Federal Highway Administration Report No. FHWA-PL-93-020, 84 p.

Gassler, G., 1992, Full scale test on a nailed wall in consolidated clay, in Proceedings of the International Symposium on Earth Reinforcement Practice, Fukuoka, Kyushu, Japan: Balkema, Rotterdam, v. 1, p. 475-480.

Gassler, G., and Gudehus, G., 1981, Soil nailing—some aspects of a new technique, in Proceedings of the 10th International Conference Soil Mechanics and Foundation Engineering: Balkema, Rotterdam, v. 3, p. 665-670.

Guilloux, A., and Schlosser, F., 1982, Soil nailing practical applications, in Proceedings of the Symposium on Soil/Rock Improvement Technique: A.I.T.

Hudson, M.B., 1990, The effects of the Loma Prieta earthquake on soil-nailed earth retaining structures in the San Francisco Bay area: Civil Engineering Department, University of California, Los Angeles, M.S. thesis, 108 p.

Japan Highway Public Corporation, 1987, Guide for design and construction on reinforced slope with steel bars: September, 33 p.

Jewell, R.A., and Pedley, M.J., 1992,Analysis for soil reinforcement with bending stiffness: Journal of Geotechnical Engineering, v. 118, no. 10, p. 1505-1528.

Juran, I., Baudrand, G., Farrag, K., and Elias,V., 1990, Kinematical limit analysis for soil nailed structures: Journal of Geotechnical Engineering, v. 116, no. 1, p. 54-72.

Juran, I., and Elias, V., 1991, Ground anchors and soil nails retaining structures, in Foundation Engineering Handbook (2d ed.): Van Nostrand Reinhold, New York, p. 868-905.

Koerner, R.M., 1984, Construction and geotechnical methods in foundation engineering: McGraw-Hill, Inc.

Lambe, P.C., and Jayaratne, N.N., 1987, Construction of retaining walls from the top down: Center forTransportation Engineering Studies, Department of Civil Engineering, North Carolina State University at Raleigh, Research Report No. 23241-86-6, 183 p.

Mitchell, J.K., and Villet, W.C.B., 1987, Reinforcement of Earth Slopes and Embankments,Appendix C, Soil Nailing: National Cooperative Highway Research Program, Report 290,Washington, D.C., June, p. 258-323.

Myles, B., and Bridle, R.J., 1991, Fired soil nails—the machine: Ground Engineering, v. 24, no. 6, p. 38-39.

Ochiai, H., Hayashi, S., and Otani, J., eds, 1992, Proceedings of the International Symposium on Earth Reinforcement Practice, IS Kyusha '92, v. 1 and 2. Fukuoka, Japan: Balkema, Rotterdam.

Plumelle, C., Schlosser, F., Delage, P., and Knochenmus, G., 1990, French national research project on soil nailing Clouterre: American Society of Civil Engineers SpecialTechnical Publication No. 25, p. 660-675.

Schlosser, F., Jacobsen, H.M., and Juran, I., 1983, Soil reinforcement—general report, *in* Proceedings of the Eighth European Conference on Soil Mechanics and Foundation Engineering: Balkema, Rotterdam, v. 3, p. 1159-1180.

Seed, R.B., Dickenson, S.E., and Idriss, I.M., 1991, Principal geotechnical aspects of the 1989 Loma Prieta earthquake: Soils and Foundation, v. 31, no. 1, p. 1-26.

Shen, C.K., Herrmann, L.R., Romstad, K.M., Bang, S., Kim,Y.S., and DeNatale, J.S., 1981, An in situ earth reinforcement lateral support system: Department of Civil Engineering, University of California, Davis, Report No. 81-03, 188 p.

Stocker, M.F., Korber, G.W., Gassler, G., and Gudehus, G., 1979, Soil nailing, in Proceedings of the International Conference on Soil Reinforcement, Paris, v. 2, p. 469-474.

Stocker, M.F., and Riedinger, G., 1990, The bearing behavior of nailed retaining structures: American Society of Civil Engineers Special Technical Publication No. 25, p. 612-628.

Tatsuoka, F., 1992, Keynote lecture—roles of facing rigidity in soil reinforcing, *in* Preprints of Special and Keynote Lectures of the International Symposium on Earth Reinforcement Practice Kyusha '92, Fukuoka, Japan: Institute of Industrial Science, Tokyo, p. 77-115.

Tufenkjian, M.R., and Vucetic, M., 1992, Seismic stability of soil nailed excavations, *in* Proceedings of the International Symposium on Earth Reinforcement Practice, Fukuoka, Kyushu, Japan: Balkema, Rotterdam, p. 573-578.

———1993, Seismic stability of soil nailed excavations: Civil and Environmental Engineering Department, University of California, Los Angeles, Research Report ENG-97-169, June, 298 p.

Tufenkjian, M.R., Vucetic, M., and Doroudian, M., 1991, Stability of soil nailed excavations, in Proceedings of the International Workshop on Technology for Hong Kong's Infrastructure Development: Commercial Press Ltd., Hong Kong, p. 751-762.

Vucetic, M., Iskandar, V.E., Doroudian, M., and Luccioni, L., 1996, Dynamic failure of soil-nailed excavations in centrifuge, *in* Proceedings of the International Symposium on Earth Reinforcement Practice, Fukuoka, Kyushu, Japan: Balkema, Rotterdam, v. 1, p. 829-834.

Vucetic, M., Tufenkjian, M.R., and Doroudian, M., 1993, Dynamic centrifuge testing of soil nailed excavations: Geotechnical Testing Journal, v. 16, no. 2, p. 172-187.

Appendix "B" Excerpts from Reference 5 – Soldier Piles

SOLDIER PILES

SOLDIER PILE SYSTEMS

Soldier piles of varying materials and sections are used, often in conjunction with some form of lagging to support soils as a continuous wall above the depth of excavation. Soldier piling elements may consist of HP or wide flange sections, sheet pile sections, or CIDH piles. Lagging may consist of horizontally placed wood members, steel plates, or concrete sections.

Soil loads are transferred to soldier piles partially through the lagging and partially through soil arching. A semi-circular section of soil immediately behind the lagging may represent all the load that gets transmitted to the lagging. When the soil between soldier piles is capable of self support the soil loads will transfer to the adjacent soldier piles, and no lagging will be needed. This soil load transfer is referred to as soil arching. Compact or cohesive soils will demonstrate a greater ability for soil arching than will loose and cohesionless soils. However, the looser soils will tend to load the lagging more.

Stiff soils exhibit an ability to stand unsupported for some height for some period of time. This is evident by comparison to relatively small square or rectangular excavations where no shoring is used. The soil behind and along the cut faces transmit the lateral forces to the vertical corners through soil arching. Soldier piles act in the same manner as the vertical corners.

The general design procedure for soldier pile walls is to assume one half the pile spacing either side of the pile acts as a panel loaded with active soil pressures and surcharge loadings above the depth of excavation. The portion of soldier pile below the depth of excavation is likewise loaded with both active soil pressure and surcharge loadings.

Resistance to lateral movement or overturning (about any point) of the soldier pile is furnished by the passive resistance of the soil below the depth of excavation. The depth of pile penetration must be sufficient to prevent lateral movement or tip over (about the base) of the soldier pile system. To account for soil disturbance at the excavation elevation AASHTO recommends that any passive resistance be ignored or discounted for a distance equal to 1.5 times the effective pile diameter immediately below the depth of the excavation.

Soldier piles may be driven or they may be installed in drilled holes. Drilled holes may be backfilled with concrete, slurry, sand, pea-gravel or similar material after the soldier pile has been installed in the hole. Some soldier pile drilled holes are backfilled with concrete to the depth of excavation and then the remainder of the hole is filled with slurry to ground level. Slurry is generally considered to be a sand-cement mix placed wet enough to fill all voids. Backfill materials other than concrete are used when it is desirable to extract the soldier piles. When materials other than concrete are used to backfill drilled soldier pile holes some vibratory methods and/or jetting procedures need to be used. The backfill should be as compact as the native soil into which the pile is set.

The use of pea gravel backfill in lieu of concrete substantially decreases the passive resistance on all sides of the soldier pile. The pea gravel does not permit the soldier pile to act as a unit until sufficient soldier pile deflection compacts the pea gravel against the soil. Similar reasoning is true for compacted sand backfill.

Effective Width:

The effective width of a soldier pile is considered to be the dimension of the soldier pile taken parallel to the line of the wall for piles either driven, or placed in drilled holes backfilled with materials other than concrete. When hard rock concrete is used for the backfill of drilled holes the effective width of the soldier pile is the diameter of the drilled hole. Structural concrete is generally considered to be a 4 sack or better concrete mix. Properly placed lean concrete can also be effective. However, lean concrete must be sufficiently strong to prevent collapse of the hole, yet weak enough to be excavated easily. A lean concrete mix is normally about 1 to 2 sacks of cement per cubic yard with a minimum specified strength of 2,400 psi.

Experimentation (1970's and later) has determined that the passive resistance of cohesionless soils acts over a width greater than the effective width of the soldier pile. The pressure exerted by the laterally pulled soldier pile produces what amounts to a wedge shaped resisting soil configuration. This soil failure configuration offers a resistance similar to the resistance that would act over something wider than the effective pile width.

Because of the apparent increase in passive resistance previously mentioned, the effective widths of soldier piles installed in cohesionless soils may be increased by an adjustment factor (passive arching capability) of 0.08 times the internal friction angle of the soil **(0.08ϕ)** , but not to exceed a value of 3.00.

> This means for example, that the final adjusted design width for a soldier pile with a flange width of 14" installed in soil which has a ϕ angle of 38°, not installed in hard rock concrete, would be equivalent to [(0.08)(38)](14/12) = [3.00 max] (14/12) = 3.5 feet. The full value of 3.5 feet per soldier pile could be used provided the pile spacing is greater than 3' - 6". Care will have to be exercised to be sure that no more than 1 pile spacing is used for the final adjusted widths so that none of the widths overlap.

SOLDIER PILES

Adjustments:

Adjustment Factor = Arching Capability = 0.08ϕ (≤ 3.00)

Adjusted Pile Width = (Effective Width)(0.08ϕ) $\leq$ 1 Pile Spacing

For cohesive soils the adjustment factor for increasing the effective soldier pile width ranges between 1 to 2. Permissible adjustment factors are listed in Table 104.

For excavations adjacent to railroad tracks the AREA recommendations specify that ϕ and C values be reduced 15% for the effects of dynamic loading when these values are determined by a qualified soils analysis laboratory. See the railroad requirements in the appendix.

Below the excavation depth the final adjusted width may be used for the passively loaded side of the pile. The same final adjusted width may be used for the active and surcharge loading also. Some consultants have used only the effective width of the piling for the active and surcharge loaded width (not advisable).

Soldier Piles As Sheet Piling:

Soldier piling can be analyzed in the same manner as sheet piling when the active loaded width below the depth of excavation is assumed to be the same as the passive loaded width.

When soldier piling is analyzed in the same manner as sheet piling either the loaded panel width (the pile spacing) or the effective soldier pile width must be adjusted. Proportioning the soldier pile effective width to the pile spacing permits analysis on a per foot basis of wall as is done for sheet pile analysis. For example, a soldier pile spaced on 8'-0" centers having a final effective pile width of 2'- 0" has an equivalent sheet pile width of 1'-0" above the excavation line and 2'/8' = 0.25 foot width below that line. Completion of the soldier pile computations using sheet pile analysis is accomplished by increasing all answers for moments and shears by a factor equal to the soldier pile spacing, in this case 8 feet.

An easy method for converting from soldier pile to sheet pile analysis involves determining an Arching Factor (f). The value of f is determined by multiplying the adjustment factor (Passive Arching Capability listed in Table 10-1), by the effective pile width then dividing that result by the soldier pile spacing.

f = Arching Factor

$$f = \frac{\text{(Passive Arching Capability) (Effective Pile Width)}}{\text{Soldier Pile Spacing}}$$

Where: $f \leq 1.0$

The value f must be equal to or less than 1.0 to prevent overlap of the passive resisting lengths.

Assume the same values previously cited where the soldier pile spacing is 8'- 0", the pile is not encased in hard rock concrete, the pile flange width is 14", and the internal friction angle ϕ of the cohesionless soil is 38°.

$$f = \frac{[0.08(38)](14/12)}{8.0} = 0.438$$

A sheet pile analysis could then be made for the soldier piling as long as all equations used below the excavation line are factored by 0.438, and the final answers multiplied by 8 which equals the pile spacing of 8'- 0".

AASHTO Methodology:

The 1992 publication, Standard Specifications for Highway Bridges by AASHTO contains a simplified method for designing cantilever soldier piling in cohesionless soils. The methodology along with a sample problem is included near the end of this chapter. The AASHTO method permits the inclusion of surcharges. This design method requires that no passive resistance be counted within 1.5 times the effective pile width below the depth of excavation, The method also provides that the computed pile depth (D) be increased by 30% for temporary work.

The AASHTO method indicates that the adjusted pile width may be up to 3 times the effective pile width provided that the soldier pile spacing is equal to or greater than 5 times the effective pile width. Structures policy will be to use an adjusted pile width of **0.08ϕ (≤ 3.00)** times the effective pile width provided this width does not exceed the soldier pile spacing.

GUIDELINES FOR REVIEW OF SOLDIER PILE

PASSIVE ARCHING CAPABILITIES

GRANULAR SOILS

COMPACTNESS	VERY LOOSE		LOOSE		MEDIUM		DENSE		VERY DENSE
Relative Density, D_d		15%		35%		65%		85%	
Standard Penetration Resistance, N = Blows/ft		4		10		30		50	
Angle of Internal Friction, ϕ		28		30		36		41	
Unit Weight (PCF)									
Moist	100		95-125		110-130		110-140		130+
Submerged	60		55-65		60-70		65-85		75+
Arching Capability	0.08ϕ		0.08ϕ		0.08ϕ		0.08ϕ		0.08ϕ

COHESIVE SOILS

CONSISTENCY	VERY SOFT		SOFT		MEDIUM		STIFF		VERY STIFF		HARD
q_u = unconfined comp. strength (PSF)		500		1000		2000		4000		8000	
Standard Penetration Resistance, N = Blows/Ft		2		4		8		16		32	
Unit Weight (PCF) Saturated		100-120			110-130			120-140			130+
Arching Capability		1 to 2		1 to 2		2		2		2	

TABLE 10 - 1 (TABLE 21)

LAGGING

Wood Jagging is commonly installed in front of, or behind the front flange of wide flange beam soldier piles. The procedure of installing lagging behind the back flange of the soldier piling is not recommended because the potential arching action of the supported soil will be destroyed. Lagging placed behind the front flange may be wedged back to provide tight soil, to lagging contact. Voids behind lagging should be filled with compacted material. Lagging may be installed with a maximum spacing up to 1 1/2" to permit seepage of moisture through the wall system. Movement of soil through the lagging spaces an be prevented by packing straw or hay in the spaces.

Construction grade lumber is the most common material used for lagging. Treated lumber is used when it is expected that the lagging will remain in place for a longer period of time or permanently.

Soil arching behind lagging is induced by lateral soil movement within the failure wedge. This soil movement causes the lagging to flex outward. The arching process induces a redistribution of soil pressure away from the center of the lagging toward the much stiffer soldier pile support. Because of this, the design load on the lagging may be taken as 0.6 times the theoretical or calculated pressure based on a simple span. Studies have shown that a maximum lagging pressure of 400 psf should be expected when surcharges are not affecting the system. Without soil arching, the pressure redistribution would not occur and reduced lagging loads should not be considered. For the arching effect to occur the back side of the soldier pile must bear against the soil.

- **Lagging design load = 0.6(shoring design load)**
- **Maximum lagging load may be 400 psf without surcharges**

Table 10-2 lists FHWA recommended minimum timber thickness for construction grade douglas fir lagging for a variety of soil classifications.

- <u>**Competent Soils:**</u> **These soils include high internal friction angle sand or granular material or stiff to very stiff clays.**
- <u>**Difficult Soils:**</u> **These soils consist of loose or low friction angle cohesionless material, silty sands, and over consolidated clays which may expand laterally, especially in deep excavations.**
- <u>**Potentially Dangerous Soils:**</u> **The use of lagging with potentially dangerous soils is questionable.**

The tabular values may be used for lagging where soil arching behind the lagging can develop. Tabular values should not be used for excavations adjacent to existing facilities including railroads. Lagging used in conjunction with surcharges should be analyzed separately.

RECOMMENDED THICKNESS OF WOOD LAGGING

WHEN SOIL ARCHING WILL BE DEVELOPED

(FOR LOCATIONS WITHOUT SURCHARGE LOADINGS)

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

*Adapted and revised from the April 1976 Federal Highway Administration Report No. FHWA-RD-75-130.

TABLE 10- 2 (TABLE 22)

CANTILEVER SOLDIER PILES - GRANULAR SOIL

Basic soil - no surcharge

f = Arching factor
(See page 10-3)

$P_{A1} = \gamma H K_a = KwH$

$P_{A'1} = fP_{A1}$

$P_{A2} = f\gamma D K_a + P_{A'1}$

$P_E = f\gamma D(K_p - K_a) - P_{A'1}$

$P_J = f\gamma D(K_p - K_a) + f\gamma H K_p$

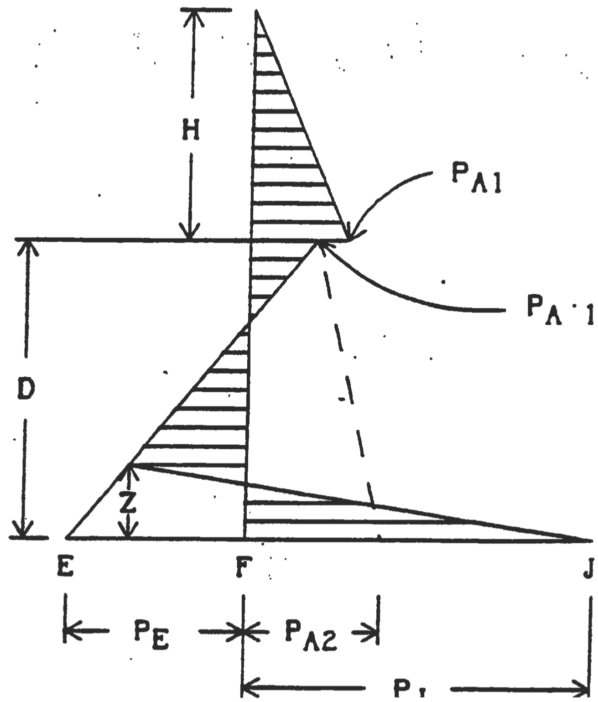


FIGURE 10-1

$$\Sigma F_H = 0 = (H)(P_{A1})/2 + (P_{A'1} + P_{A2})(D)/2 + (P_E + P_J)(Z)/2 - (P_E + P_{A2})(D)/2$$

$$\therefore Z = \{(P_E + P_{A2})(D) - (H)(P_{A1}) - (P_{A'1} + P_{A2})(D)\}/(P_E + P_J)$$
$$= \{P_E - P_{A'1})(D) - (H)(P_{A1})\}/(P_E + P_J)$$

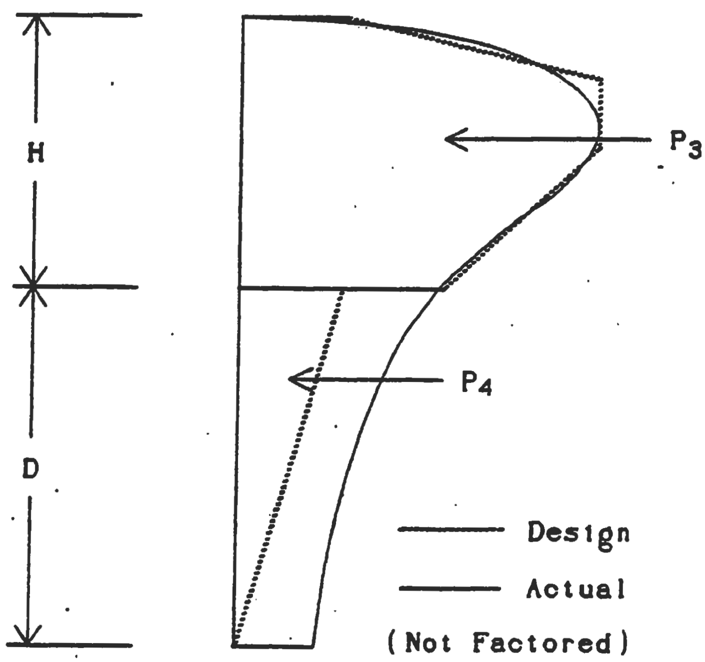
$$\Sigma M_F = 0 = \{(H)(P_{A1})/2\}[H/3 + D] + (P_{A'1}(D)[D/2\} + \{(P_{A2} - P_{A'1})(D)/2\}[D/3\} + \{(P_E + P_J)(Z)/2\}[Z/3] - \{(P_E + P_{A2})(D)/2\}[D/3]$$

Surcharge Considerations:

In most real situations there will be some sort of surcharge present. The following pages demonstrate how two types of surcharges may be handled. Simplifying the pressure diagrams (using sound engineering judgement) should not alter the results significantly and may make the problem much easier to solve. The surcharge pressures can be added directly to the soil pressure diagram, or may be sketched separately. It will often be convenient to convert a long uniform surcharge to an equivalent height of soil, increase H by that amount, and then analyze the shoring with one pressure diagram.

SOLDIER PILES

For Boussinesq surcharges, the pressure diagram above the excavation depth is simplified so as to match the area as close as possible, while still allowing for ease of computation. The surcharge pressure immediately below the depth of excavation is adjusted by the Passive Arching Capability factor and then may be tapered to zero (for small surcharge) at the bottom end of the soldier pile.



BOUSSINESQ SURCHARGE

FIGURE 10 - 2

P_3 = Area under dashed line above the excavation depth.

P_4 = Area under dashed line below the excavation depth.

The forces and moments are then added to the equations on the previous page to solve for the total horizontal forces and moments and to arrive at the required depth (D).

To allow for a Safety Factor and sufficient embedment increase D by 20% - 40% or initially adjust K_p by using $K_p/1.5$ to $K_p/1.75$.

SAMPLE PROBLEM 10-1: CANTILEVER SOLDIER PILE

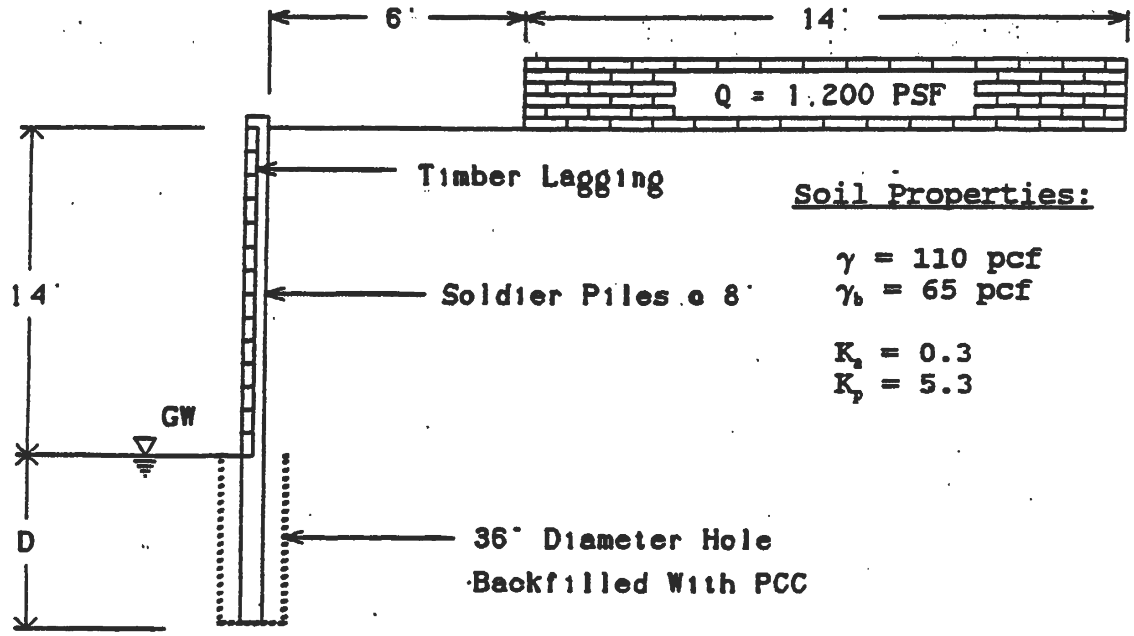


FIGURE 10 - 3

Determine Lateral Pressures: Soil parameters are arbitrary values chosen for simplicity

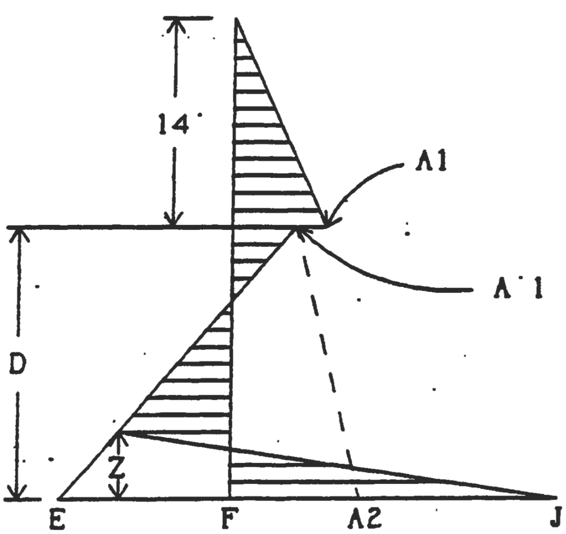


FIGURE 10 - 4

Soil pressure only.

Arching capability = 2

$f = (2)(36/12)/8 = 0.75$

$P_{A1} = \gamma H K_a = (110)(14)(0.3)$
$= 462$ psf

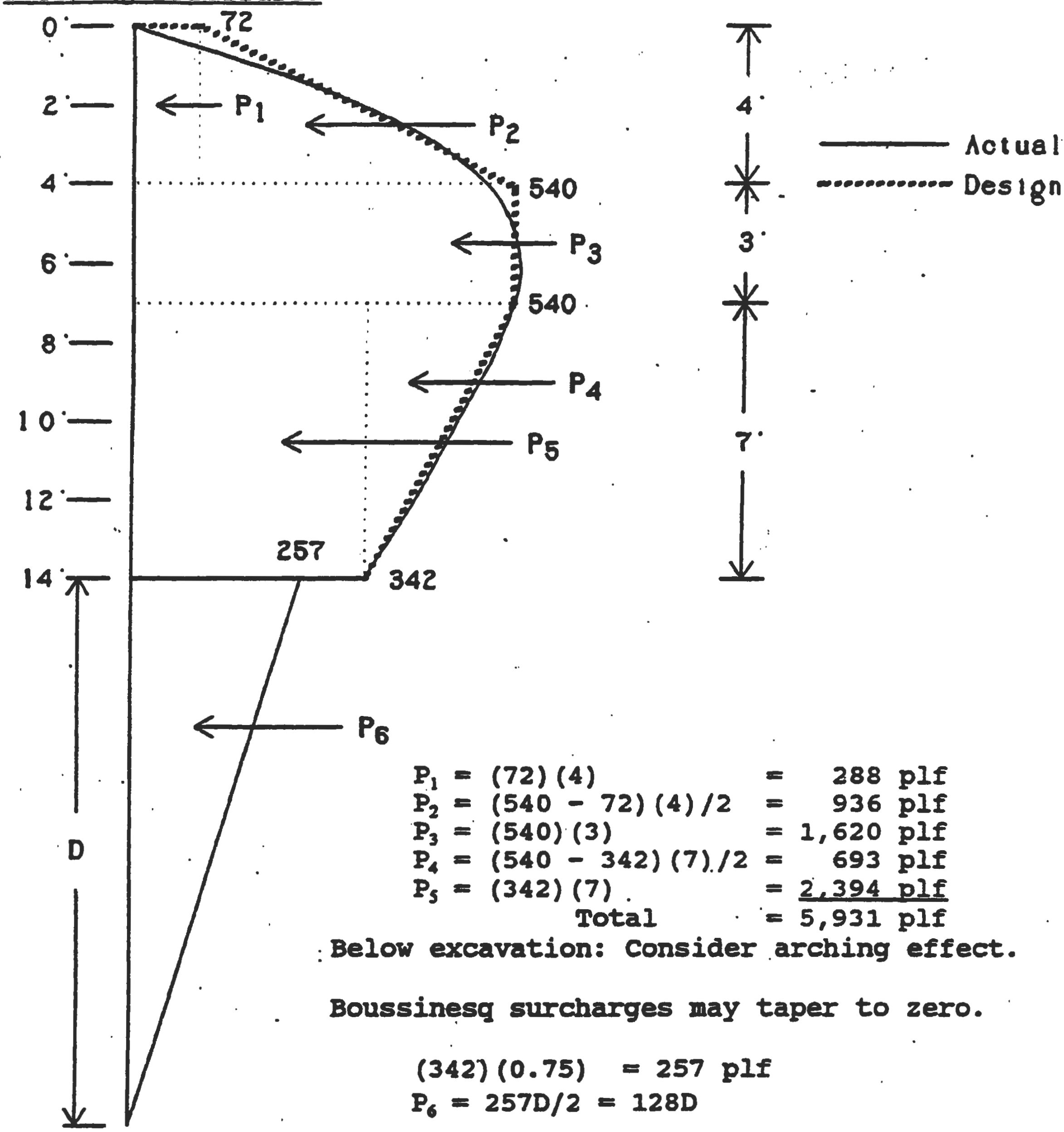
$P_{A'1} = f P_{A1} = (0.75)(462)$
$= 347$ psf

$P_{A2} = f\gamma_b D K_a + P_{A'1}$
$= (0.75)(65)(0.3)D + 347$
$= 15D + 347$

$P_E = f\gamma_b D(K_p - K_a) - P_{A'1} = (0.75)(65)(5.3 - 0.3)D - 347$
$= 244D - 347$

$P_J = f\gamma_b D(K_p - K_a) + f\gamma H K_p$
$= (0.75)(65)(D)(5.0) + (0.75)(110)(14)(5.3) = 244D + 6{,}122$

Surcharge Pressure:



P_1 = (72)(4) = 288 plf
P_2 = (540 - 72)(4)/2 = 936 plf
P_3 = (540)(3) = 1,620 plf
P_4 = (540 - 342)(7)/2 = 693 plf
P_5 = (342)(7) = 2,394 plf
Total = 5,931 plf

Below excavation: Consider arching effect.

Boussinesq surcharges may taper to zero.

(342)(0.75) = 257 plf
P_6 = 257D/2 = 128D

FIGURE 10 - 5

Determine D:

$\Sigma F_H = 0$

$= (14)(462)/2 + (347 + 15D + 347)(D)/2$
$+ (244D - 347 + 244D + 6{,}122)(Z)/2 + 5{,}931 + 128D$
$-(244D - 347 + 15D + 347)(D)/2$
$= -122D^2 + 475D + 244DZ + 2{,}888Z + 9{,}165$

$\therefore Z = (D^2 - 3.9D - 75.1)/(2D + 23.7)$

$$
\begin{aligned}
\Sigma M_F &= 0 \\
&= \{(14)(462)/2\}[D + 14/3] + 347D[D/2] + \{(15D)(D)/2\}[D/3] \\
&\quad + \{(244D - 347 + 244D + 6{,}122)(Z)/2\}[Z/3] + 288[D + 12] \\
&\quad + 936[D + 11.33] + 1{,}620[D + 8.5] + 693[D + 4.67] \\
&\quad + 2{,}394[D + 3.5] + 128D[2D/3] \\
&\quad - \{(244D - 347 + 15D + 347)(D)/2\}[D/3] \\
&= -41D^3 + 259D^2 + 9{,}165D + 81DZ^2 + 963Z^2 + 54{,}538 = 0
\end{aligned}
$$

$$\therefore Z^2 = (D^3 - 6.3D^2 - 223.5D - 1{,}330.2)/(2.0D + 23.5)$$

By trial and error or other means $D = 22.30'$ & $Z = 4.91'$
Use a safety factor of 30%: Use $D = 1.3(22.3) = 29.0'$

<u>Find Maximum Moment:</u>

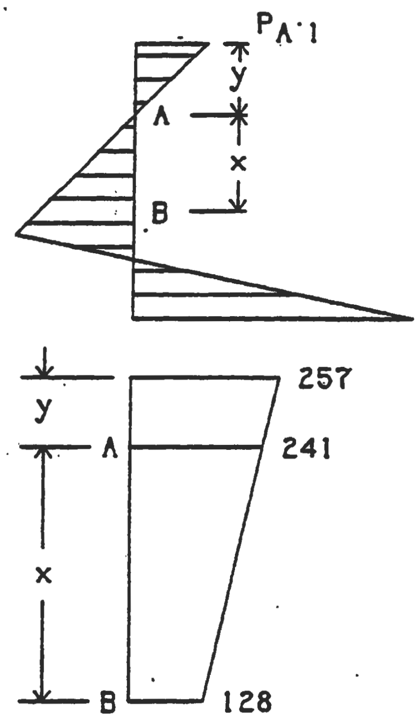


FIGURE 10 - 6

(Composite section properties ignored)

Locate plane of zero shear (B).

$$y = P_{A'1} / f\gamma_b(K_p - K_a)$$

$$= 347/(0.75)(65)(5.3 - 0.3) = 1.42'$$

Surcharge pressure at A:

$$257(22.30 - 1.42)/22.30 = 241 \text{ psf}$$

Shear due to surcharge at A:

$$1.42(257 + 241)/2 + 5{,}931 = 6{,}285$$

Total shear at A:

$$14(462)/2 + 347(1.42)/2 + 6{,}285$$
$$= 9{,}765 \text{ Lb/LF}$$

Shear for area between A & B = 9,765

$$\{f\gamma_b(K_p - K_a)x^2/2\} - (\{241 + (257)(22.30 - 1.42 - x)/22.30\}/2)x = 9{,}765$$

Substituting: $0.75(65)(5.3 - 0.3)x^2 - 241x + 5.76x^2 = 9{,}765$
Simplifying: $127.6x^2 - 241x - 9{,}765 = 0$

Solving for x: $x = 9.74'$

<u>Find Moment at B:</u>

M due to soil pressure above A.

$$\{14(462)/2\}[15.83] + \{347(1.42)/2\}[10.69] = 53{,}828 \text{ Ft-Lb/LF}$$

M due to soil pressure between A & B:

0.75(65)(5.3 - 0.3)(9.74)(9.74/2)[9.74/3] = 37,538 Ft-Lb/LF

M due to surcharge above excavation:

288[23.16] + 936[22.49] + 1,620[19.66] + 693[15.83] + 2,394[14.66] = 105,636 Ft-Lb/LF

Surcharge pressure at B:

257(22.30 - 1.42 - 9.74)/22.30 = 128 psf

M due to surcharge below excavation:

128(11.16)[11.16/2] + {(257 - 128)(11.16)/2}[11.16)(2)/3] = 13,326 Ft-Lb/LF

M (Total) = 8(53,828 - 37,538 + 105,636 + 13,326) = 1,082,016 Ft-Lb

S Required = (1,082,016)(12)/22,000 = 590.2 in^3

Use W30 x 191, S = 598 in^3

Determine Lagging Needed:

By inspection, maximum load occurs at the depth of excavation.
$M_{max} = wL^2/8 = (342 \quad 462)(8)^2/8 = 6,432$ Ft-Lb
(With total soil arching $M_{max} = 400(8)^2/8 = 3,200$ Ft-Lb)

S Required = 6,432(12)(0.6)/(1,500)(1.0)* = 30.9 in^3
*** 1.0 In lieu of 1.33 duration factor due to high risk building**

Use 4 x 12's (rough lumber): S = 32 in^3

This answer does not agree with values in Table 22 because the table does not provide for surcharge loadings.

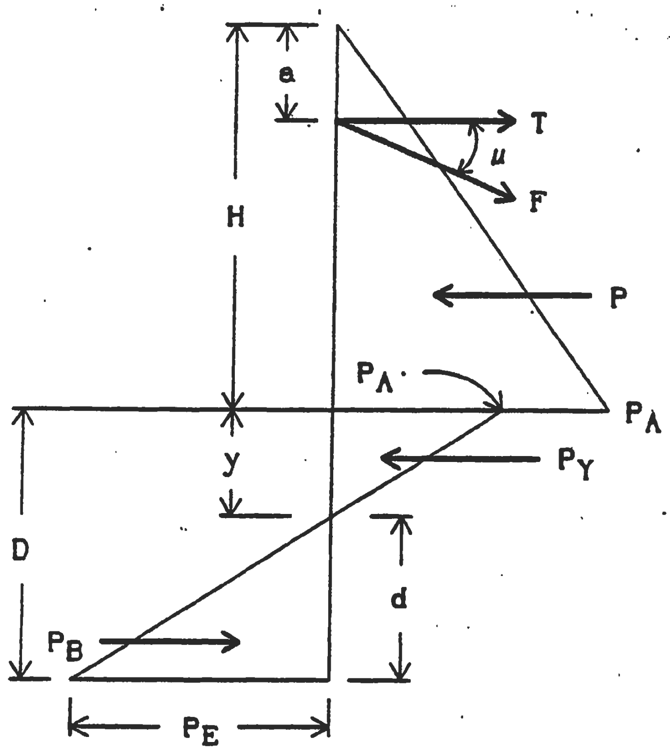
Summary:

Use W30 X 191's - minimum length of 42' 8", placed in 36" diameter holes and backfilled to bottom of excavation with concrete.

Use 4 x 12's (Rough lumber) for lagging.

The size of the wide flange beam and the diameter of the drilled hole indicate that cantilevered soldier piles would not be the correct type of shoring for the conditions given.

SOLDIER PILES W/SINGLE TIEBACK - GRANULAR SOIL



Soil only (No surcharge)
$a \approx 0.2H$ to $0.4H$

S = Soldier pile spacing

f = Arching factor

$P_A = \gamma H K_a = KwH$

$P = HP_A/2 = \gamma H^2 K_a/2$

$= KwH^2/2$

$P_{A'} = fP_A$

$y = P_{A'}/f\gamma(K_p - K_a)$

$P_Y = yP_{A'}/2$

$P_E = f\gamma d(K_p - K_a)$

$P_B = dP_E/2$

FIGURE 10 - 7

Determine D:

$$\Sigma M_T = 0 = P[2H/3 - a] + P_Y[H-a + y/3] - P_B[H - a + y + 2d/3]$$

Solve for d by trial and error (or other means). For the first approximation try $d = H/4.5$.

$D = y + d$. Adjust the value of D or K_p for a safety factor.

Determine T: $T = S(P + P_Y - P_B)$ and $F = T/\cos\mu$

Find the maximum moment. This will generally be either the cantilever section above T or somewhere between T and the excavation level. Determine this second point by locating the point of zero shear.

Determine soldier pile section modulus required, size of lagging needed, and tieback requirements.

Note: A surcharge will normally be present (or the minimum surcharge or 72 psf will be used) and should be added to the equations shown above.

SAMPLE PROBLEM 10-2: SOLDIER PILE WITH SINGLE TIEBACK

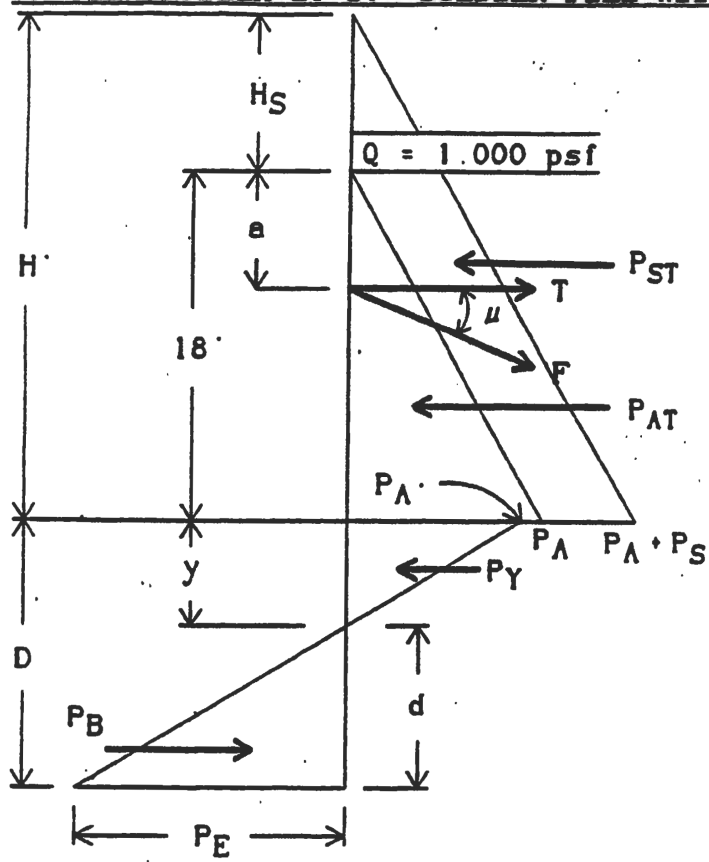


FIGURE 10 - 8

Given:

$K_w = 35$ pcf
$\gamma = 115$ pcf $\quad \mu = 20°$

$K_a = K_w/\gamma = 0.30$
$K_p = \gamma/K_w = 3.29$

$H = 18' \quad a = 6'$

Solution:

Q = Uniform surcharge
$H_S = 1{,}000/115 = 8.7'$
$H' = 18 + 8.7 = 26.7'$

Pile spacing (S) = 6'
Pile Hole Diameter = 24"
Arching capability = 2.0
$f = (2)(24/12)/6 = 0.67$

$P_A = K_wH = 35(18) = 630$

$P_A + P_S = K_wH' = 35(26.7)$
$= 935$ psf
$P_S = 935 - 630 = 305$ psf

$P_{AT} = 630(18/2) = 5670$ Lb/LF

$P_{ST} = 305(H) = 305(18) = 5490$ Lb/LF

$P_{A'} = fP_A = 0.67(630) = 422$ psf

$y = P_{A'}/f\gamma(K_p - K_a) = 422/[(0.67)(115)(3.29 - 0.30)] = 1.83$

$P_Y = yP_{A'}/2 = 1.83(422)/2 = 386$ Lb/LF

$P_E = f\gamma(d)(K_p - K_a) = 0.67(115)(d)(3.29 - 0.30) = 230d$

$P_B = dP_E/2 = d(230d)/2 = 115d^2$

Determine D:

$$\Sigma M_T = 0 = P_{AT}[2H/3 - a] + P_y[H - a + y/3] + P_{ST}[H/2] - P_B[H - a + y + 2d/3]$$

$$= 5{,}670[2(18.0)/3 - 6] + 386[18 - 6 + 1.83/3] + 5490[18/2 - 6] - (115d^2)[18 - 6 + 1.83 + 2d/3]$$
$$= 55{,}357.5 - 1590.5d^2 - 76.7d^3 = d^3 + 20.7\,d^2 - 721.7$$

By trial and error, or by other means, $d = 5.27'$

$D = d + y = 5.27 + 1.83 = 7.10'$
Increase D by 30% for safety factor: $D = 1.3(7.1) = 9.2'$

<u>Determine T:</u>

$P_B = 115(5.27)^2 = 3{,}194$ Lb/LF

$T = P_{AT} + P_{ST} + P_y - P_B = 5{,}670 + 5{,}490 + 386 - 3{,}194 = 8{,}352$ Lb/LF
$F = T/\cos\mu = 8{,}352/\cos 20° = 8{,}888$ Lb/LF
Total $F = 6(8{,}888) = 53{,}328$

Find Maximum Moment (See Note)

Locate point of zero shear (x).

$8{,}352 = P_A(x/18)(x/2) + P_S x$

$8{,}352 = 17.5x^2 + 305x$

$x = 14.8'$

$M = -(630)(14.8/18)(14.8/2)[14.8/3]$

$- (305)(14.8)[14.8/2]$

$+ (8{,}352)[14.8 - 6.0]$

$= 21{,}184$ Ft Lb/LF

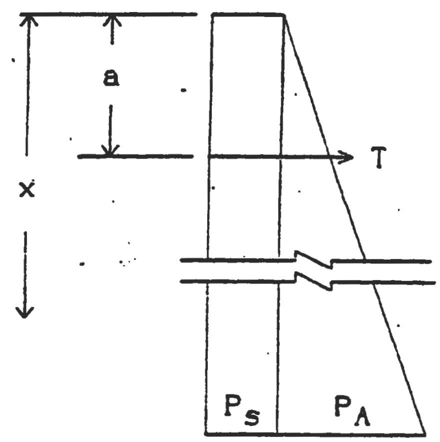


FIGURE 10 - 9

<u>Check Cantilever:</u>

$M = \{(630)(6.0/18)(6.0)/2\}[6.0/3] = 6{,}750$ Ft-Lb/LF
This does not control

S Required $= 21{,}184(6)(12)/22{,}000 = 69.3$ in^3

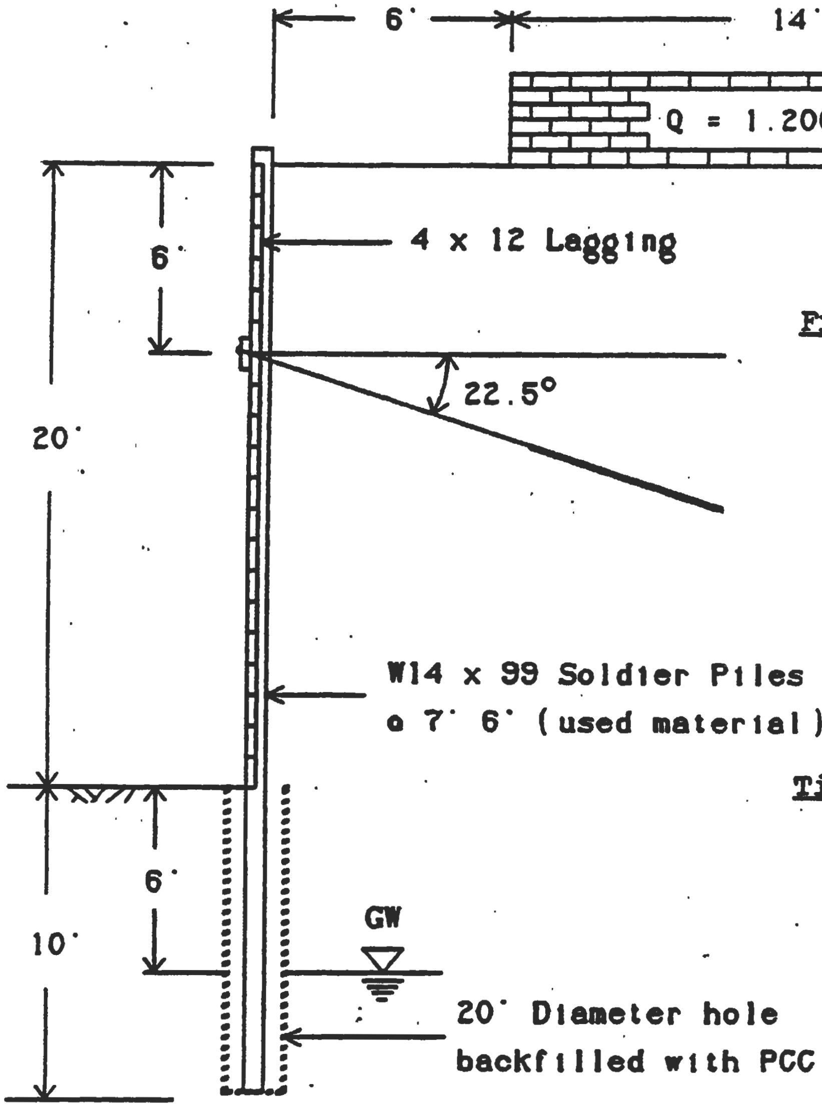
Use W14 x 53, $S = 77.8$ in^3

Determine lagging and tieback requirements.

See alternate analysis using AISC specifications at end of Chapter.

Note: When the soldier pile is encased in 4 sack or better concrete, the buried portion of the pile acts as a composite section which will have a large section modulus. When this is the case the moment at the excavation line may often be controlling.

SAMPLE PROBLEM 10-3: SOLDIER PILE WITH SINGLE TIEBACK



From soils Report

γ = 115 pcf
γ_b = 70 pcf
ϕ = 34°
δ = 0
K_a = 0.28
K_p = 4.30

Tieback Data

Two 1" tierods
at 7' - 6"
F_u = 150 ksi
16" Diameter
shaft.
Bond capacity is
600 psf.
Unbonded L = 7'
Bonded L = 42'

FIGURE 10 - 10

Determine Lateral Pressures:

FHWA recommends adjusting K_p by division with 1.5 in lieu of increasing D by 20% - 40%. (This is conservative)

Adjust K_p by 1.5. For this example $K_p = 4.3/1.5 = 2.87$

$K_p - K_a = 2.87 - 0.28 = 2.59$

Arching capability = 0.08(34°) = 2.7 Use 1.5 due to wet condition (Need to be conservative next to building).
f = 1.5(20/12)/7.5 = 0.33

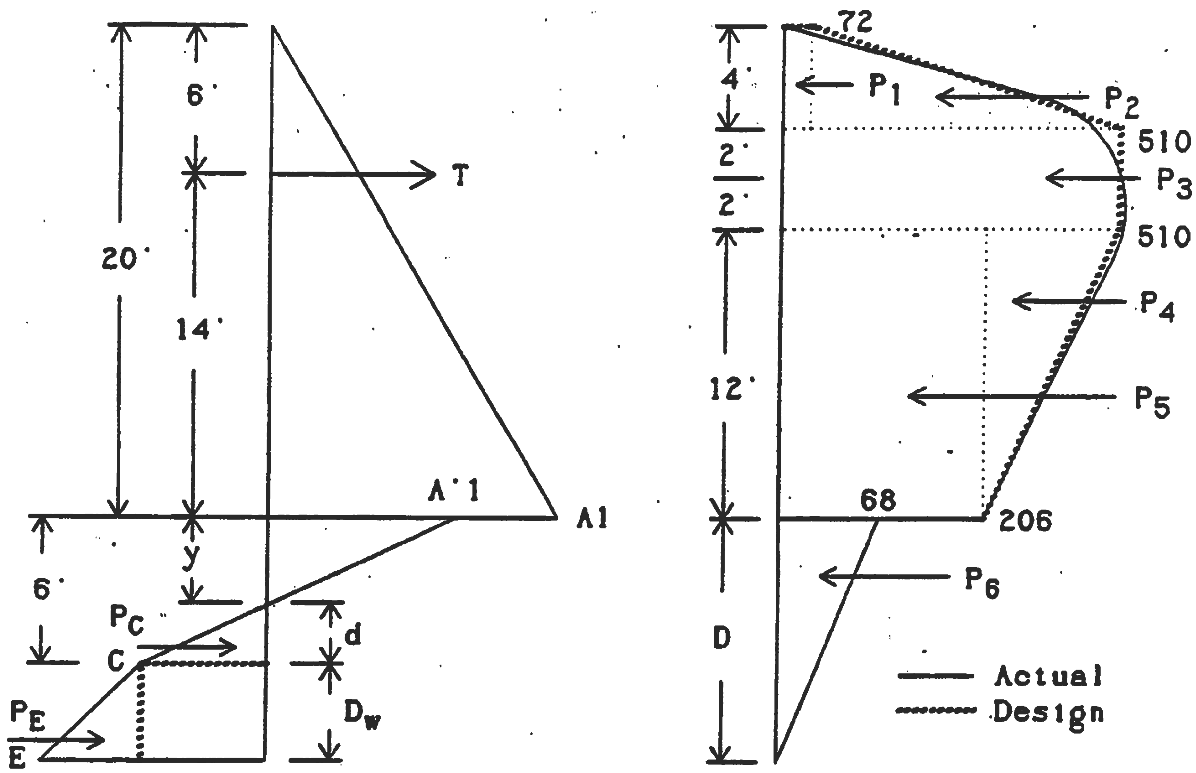


FIGURE 10 - 11

$P_{A1} = \gamma H K_a = 115(20)(0.28) = 644$ psf

$P_{A'1} = fP_{A1} = 0.33(644) = 213$ psf

$y = P_{A'1}/f\gamma(K_p - K_a) = 213/[(0.33)(115)(2.59)] = 2.17'$

$d = 6 - 2.17 = 3.83'$

$P_C = f\gamma d(K_p - K_a) = 0.33(115)(3.83)(2.59) = 376$ psf

$P_E = P_C + f\gamma_b D_w(K_p - K_a) = 376 + 0.33(70)(2.59)D_w = 376 + 60D_w$

$P_1 = 72(4) = 288$ psf
$P_2 = (510 - 72)(4)/2 = 876$ psf
$P_3 = 510(4) = 2{,}040$ psf
$P_4 = (510 - 206)(12)/2 = 1{,}824$ psf
$P_5 = 206(12) = 2{,}472$ psf

Below excavation: Consider arching effect. Boussinesq surcharges may be tapered to zero at the calculated depth D.

$206(0.33) = 68$ psf
$P_6 = 68(6 + D_w)/2 = 204 + 34D_w$

Determine D:

$\Sigma m_T = 0$

Moment due to surcharge:

$$(204 + 34D_w)[14 + (6 + D_w)/3] + 2{,}472[8] + 1{,}824[6] + 2{,}040[0] - 876[3.33] - 288[4]$$
$$= 11D_w^2 + 612D_w + 29{,}915$$

Moment due to soil:

$$\{644(20)/2\}[7.33] + \{213(2.17)/2\}[14.72] - \{376(3.83)/2\}[18.72] - 376(D_w)[20 + D_w/2] - \{60D_w)(D_w)/2\}[20 + (2/3)D_w]$$
$$= -20D_w^3 - 788D_w^2 - 7{,}520D_w + 37{,}128$$

Combined moment:

$$20D_w^3 + 777D_w^2 + 6{,}908D_w - 67{,}043 = 0$$
$$\text{or } D_w^3 + 39D_w^2 + 345D_w - 3{,}352 = 0$$

From which $D_w = 5.62'$

$D = D_w + 6 = 5.62 + 6 = 11.62'$ **(Use D = 11'-8")**

Determine T:

$$\Sigma F_H = 0$$
$$= 288 + 876 + 2{,}040 + 1{,}824 + 2{,}472 + \{204 + 34(5.62)\} + 644(20)/2 + 213(2.17)/2 - 376(3.83)/2 - 376(5.62) - 60(5.62)(5.62/2) - T$$

$\therefore T = 10{,}785$ **Lb/LF**

Total T = 10,785(7.5) = 80,888 Lb

Find Maximum Moment:

Check cantilever moment at T:

$$M = \{644(6/20)(6/2)\}[6/3] + 288[2 + 4/2] + 876[2 + 4/3] + 510(2)[2/2]$$
$$= 6{,}251 \text{ Ft-Lb/LF}$$

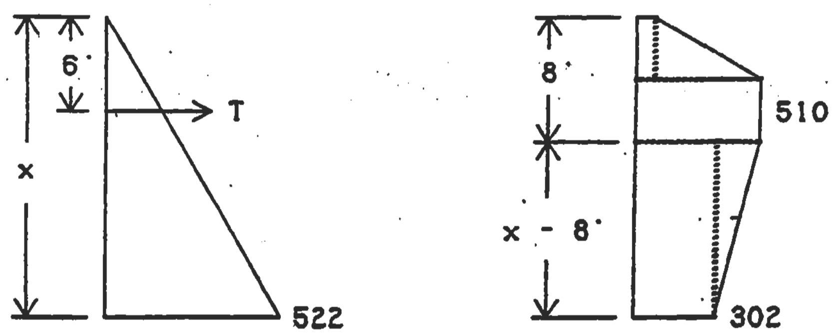


FIGURE 10 - 12

Locate point of zero shear:

$$10{,}785 - 288 - 876 - 2{,}040 - 644(x/20)(x/2)$$
$$- \{x - 8\}\{510 + [206 + (510 - 206)(20 - x)/12]\}/2 = 0$$

$$x^2 + 207.6x - 3{,}632.5 = 0$$
$$\therefore x = 16.23'$$

Pressure at point of zero shear due to soil:

$644(16.2/20) = 522$ psf

Pressure at point of zero shear due to surcharge:

$206 + (510 - 206)(20 - 16.2)/12 = 302$ psf

Moment due to tieback and soil:

$10{,}785[16.2 - 6] - \{(522)(16.2/2)\}[16.2/3] = 87{,}175$ Ft-Lb/LF

Moment due to surcharge:

$$M = - 288[16.2 - 4/2] - 876[16.2 - 4(2/3)]$$
$$- 2{,}040[16.2 - 4 - 4/2] - 302(16.2 - 8)[(16.2 - 8)/2]$$
$$- \{(510 - 302)(16.2 - 8)/2\}[(16.2 - 8)(2/3)]$$
$$= - 51{,}568 \text{ Ft-Lb/LF}$$

Combined moment (assuming non-compact section):

$87{,}175 - 51{,}568 = 35{,}607$ Ft-Lb/LF   This controls.

S Required = $35{,}607(7.5)(12)/22{,}000 = 145.67$ in^3

S furnished = 157 in^3 > 145.67 in^3 $\therefore$ O.K.

See alternate analysis using AISC specifications on page 10-21.

Check Lagging:

Consider arching effect on lagging. Multiply all pressure results by 0.6. By inspection, maximum moment occurs at the depth of excavation.

$M_{max} = wL^2/8 = (644 + 206)(7.5)^2/8 = 5{,}977$ Ft-Lb

S Required $= 5{,}977(12))(0.6)/1{,}500(1.0)* = 28.7\ in^3$
*** Load duration factor due to high risk building.**

Use 4 x 12's (Rough lumber). $S = 32\ in^3$

Check Shear:

$V = (7.5/2 - 0.33)(850)(0.6) = 1{,}744$ Lb
$v = 3V/2A = 3(1{,}744)/2(4)(12) = 54.5$ psi < 140 psi $\therefore$ O.K.

Tiebacks:

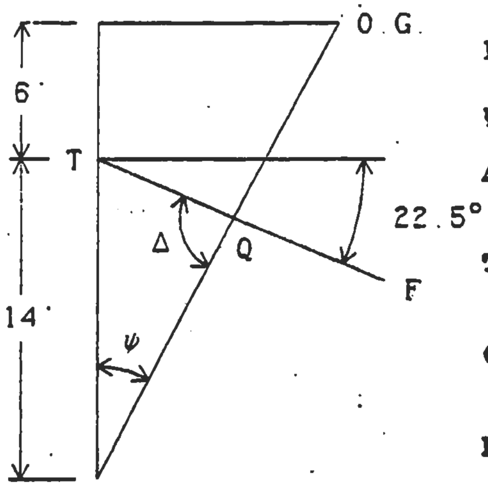


FIGURE 10 - 13

$F = 80{,}888/\cos 22.5° = 87{,}553$ Lb

$\psi \approx 45° - \phi/2 \approx 45° - 34°/2 \approx 28°$

$\Delta = 180° - (90° - 22.5°) - \psi = 84.5°$

TQ = Unbonded length
$= (14 \sin 28°)/\sin \Delta = 6.6'$

6.6' < 7.0' provided $\therefore$ O.K.

Bond capacity given as 600 psf

Bonded frictional resistance (per foot) $= \pi(16/12)(600)(L)$
$= 2{,}513(L)$ Lb/LF

Length needed = 87,553/2,513 = 34.8' < 42' $\therefore$ O.K.

Safety Factor = 42/34.8 = 1.21 = (21%)

Note: Vertical downward component of tie may be used in conjunction with wedge weights in stability analysis to counteract slip circle failure.

Vertical component = 87,553 sin 22.5° = 33,505 Lb

SOLDIER PILE WITH SINGLE TIEBACK

Soldier piles in the two previous problems were not checked for compressive stress or for the combined stresses due to the vertical component of the tieback force. AISC criteria may be used to check combined stresses. For Sample Problems 15 and 16 assume maximum unbraced length is the cantilever or the length between the tie and the point where the passive soil resistance becomes effective.

SAMPLE PROBLEM 10 - 2 - SOLDIER PILE: (ALTERNATE ANALYSIS)

Pile properties: $A = 15.6 \text{ in}^2$ $r_x = 5.89 \text{ in}$

Assume $k = 1$, **then** $KL/r = 15.8(12)/5.89 = 32.19$

From AISC: $M_r = 142.6$ **Ft-k** **and** $F_a = 19.79$ **ksi**

Downward tie force $= F \sin \omega = 53.3 \sin 20° = 18$ **k**

$M_{max} = 6(21.18) = 127.1$ **Ft-k**

$f_a/F_a + f_b/F_b = f_a/F_a + M_{max}/M_r \leq 1.0$

$= (18/15.6)/19.79 + 127.1/142.6$

$= 0.06 + 0.89 = 0.95 \leq 1.0$ **OK**

SAMPLE PROBLEM 10 - 3 - SOLDIER PILE: (ALTERNATE ANALYSIS)

Pile properties: $A = 29.1 \text{ in}^2$ $r_x = 6.14 \text{ in}$

Assume $k = 1$; **then** $KL/r = 16.2(12)/6.14 = 31.7$

From AISC: $M_r = 288$ **Ft-k** **and** $F_a = 19.82$ **ksi**

Downward tie force $= F \sin \omega = 87.6 \sin 22.5° = 33.5$ **k**

$M_{max} = 7.5(35.6) = 267$ **Ft-k**

$f_a/F_a + f_b/F_b = f_a/F_a + M_{max}/M_r \leq 1.0$

$= (33.5/29.1)/19.82 + 267/288$

$= 0.06 + 0.93 = 0.99 \leq 1.0$ **OK**

SAMPLE PROBLEM 10-4: SOLDIER PILE WITH RAKER

Given:

From Soils Report:

γ = 115 pcf
ϕ = 35°
δ = 0

Allowable Soil Bearing = 2 ksf

Loose Ballast:

γ = 115 pcf

Rough lumber is to be used

$\beta = \tan^{-1}(5.2/8)$
$= 33°$

$\beta/\phi = 33°/35°$
$= 0.94$

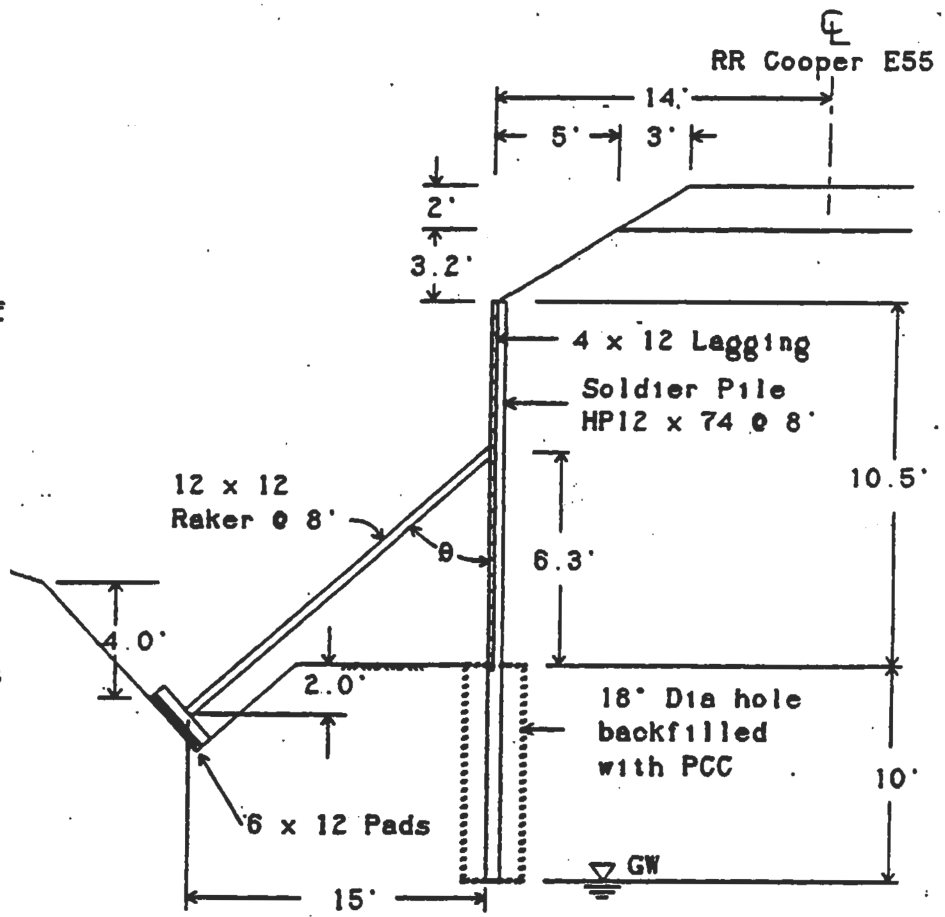


FIGURE 10-14

Solution:

NOTE: Assume that the back slope and ballast are one unit since their densities are equal and their angles are practically the same l If this was not the case, or if $\beta/\phi >$ then Trial Wedge or similar analysis would have to be used to solve the problem.

From Figure 8, $K_a = 0.65$; $K_p = \tan^2(45° + 35°/2) = 3.69$

$P_A = K_a\gamma H = 0.65(115)(10.5) = 785$ psf

Adjust K_p per FHWA recommendation in lieu of increasing D 20%-40%.

Adjust K_p by 1.5: $K_p = 3.69/1.5 = 2.46$

$K_p - K_a = 2.46 - 0.65 = 1.8$

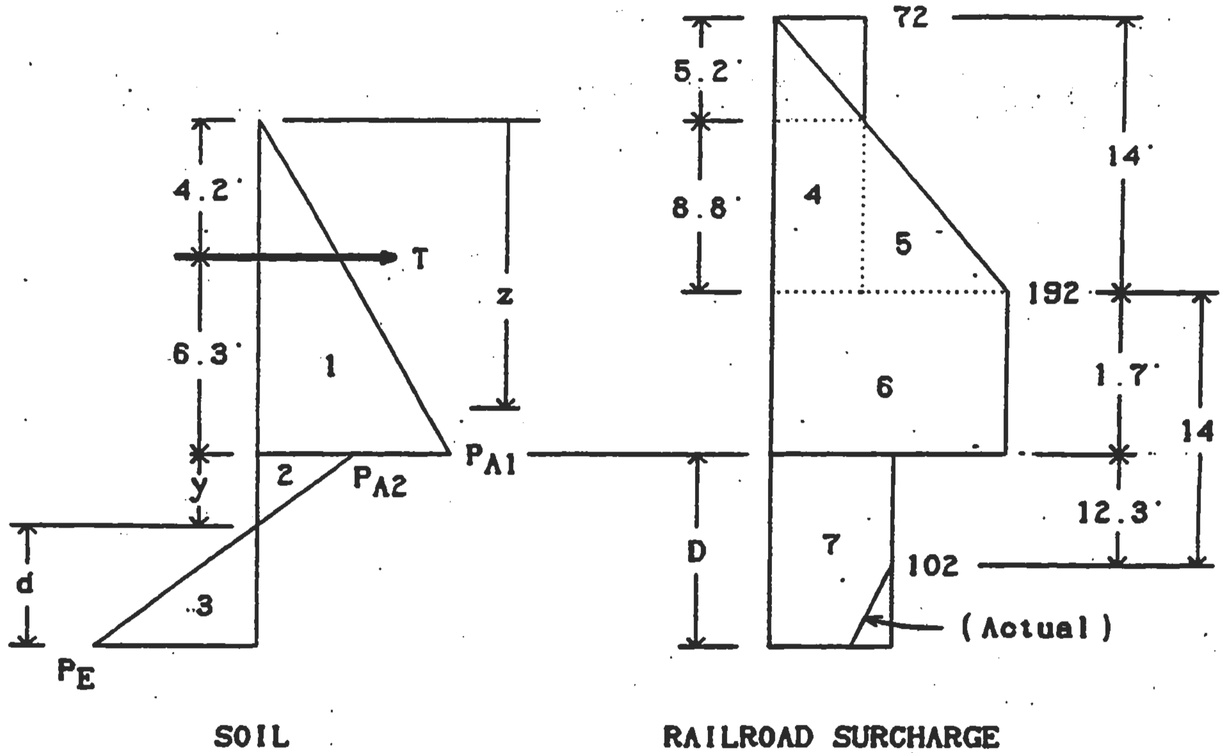


FIGURE 10 - 15

NOTE:- For ease of computation assume that the railroad surcharge has a rectangular shape below the excavation. The railroad sucharge is prorated from "CHART 3.6, LATERAL PRESSURE FOR COOPER RAILROAD LIVE LOAD": 279(E55/E80) = 192 psf. Note that the top of the railroad surcharge diagram is always located at the elevation of the top of the rail.

Soil and surcharge act on adjusted width of soldier pile. Determine f and multiply all pressures below. the excavation line by this factor.

Passive arching capability = 0.08(35°) = 2.8
Diameter of drilled hole = 18/12 = 1.5'
Arching factor (f) = 2.8(1.5)/8 = 0.53

$P_{A2} = 0.53P_{A1} = 0.53(785) = 416$ psf
Surcharge at excavation line = 0.53(192) = 102 psf

$y = P_{A2}/f\gamma(K_p - K_a) = 416/[0.53(115)(1.8)] = 3.79'$
$P_E = f\gamma d(K_p - K_a) = 0.53(115)(1.8)d = 110d$

Determine D:

$D = d + y$

$\Sigma M_T = 0$

	AREA		ARM		MOMENT
1	785(10.5)/2	= 4,121	2/3(10.5) - 4.2	= 2.80	11,539
2	416(3.79)/2	= 788	6.3 + 3.79/3	= 7.56	5,957
4	72(8.8)	= 634	8.8/2 - 4.2	= 0.20	127
5	(192 -72)(8.8)/2	= 528	2/3(8.8) -4.2	= 1.66	876
6	192(1.7)	= 326	6.3 - 1.7/2	= 5.45	1,777
		6,397			20,276

3 **AREA** = $-110d(d/2) = -55d^2$
ARM = $6.3 + 3.79 + 2/3(d) = 10.09 + 0.67d$
MOMENT = $-554.95d^2 - 36.85d^3$

7 **AREA** = $102(3.79 + d) = 386.58 + 102d$
ARM = $6.3 + (3.79 + d)/2 = 8.20 + 0.5d$
MOMENT = $3{,}169.96 + 1{,}029.69d + 51d^2$

$\therefore 0 = 23{,}445.96 + 1{,}029.69d - 503.95d^2 - 36.85d^3$

Solve for d and D: $d = 6.37'$
$D = d + y = 6.37 + 3.79 = 10.16'$

Determine T:

T $= \Sigma$ Active areas $- \Sigma$ passive areas
$= 6{,}397 + 102(6.37 + 3.79) - (6.37)^2(110)/2 = 5{,}202$ Lb/LF

Check Soldier Pile: Find M_{max}

By inspection, the point of zero shear will be above the excavation.

$0 = 785z/10.5(z/2) + 634 + 528 + 192(z - 8.8) - 5{,}202$
$= 37.38z^2 - 4{,}040 + 192z - 1{,}689.60 = z^2 + 5.14z - 153.28$
$\therefore z = 10.08' \approx 10.1'$

$M_{max} = 37.38(10.1)^2[10.1/3] + 634[10.1 - 8.8/2]$
$+ 528[10.1 - 2/3(8.8)] + 192(10.1 - 8.8)[(10.1 - 8.8)/2]$
$- 5{,}202[10.1 - 4.2]$
$= 11{,}843$ Ft-Lb/LF

Total Moment = 8(11,843) = 94,744 Ft-Lb

S Required = M/F_b = 94,744(12)/22,0000 = 51.68 in^3
S Furnished = 93.8 in^3 > 51.68 in^3

Check Lagging:

Pressure is greatest at the bottom of the excavation:
785 + 192 = 977 psf
$M = wL^2/8 = 977(12)(8)^2/8 = 93{,}792$ In-Lb

S Required = $93{,}792(0.6)/(1{,}500)(1.0)^* = 37.52$ in^3
*** No load duration factor used when adjacent to railroads.**

S for a 4 x 12 = $12(4)^2/6 = 32$ in^2 < 37.52 in^3
S for a 6 x 12 = $12(6)^2/6 = 72$ in^3 Use 6 x 12 lagging

$V = (8/2 - 0.33)(977)(0.6) = 2{,}151$ Lb

$f_v = 3V/2A = 3(2{,}151)/[2(6)(12)] = 44.8$ psi < 140 psi $\therefore$ OK

Check Raker:

$T = 8(5{,}202) = 41{,}616$ Lb

Angle $\theta = \tan^{-1} 15/8.3 = 61°$

$L = [(15)^2 + (8.3)^2]^{1/2} = 17.14$ Ft

Axial load = $41{,}616(17.14/15) = 47{,}553$ Lb

$P/A = 47{,}553/(12)(12) = 330$ psi

Allowable $F_c = 480{,}000/(L/d)^2 = 480{,}000/[(17.14)(12)/12]^2 = 1{,}634$
= 1,600 psi max

330 < 1,600 $\therefore$ OK

Check Pad:

Determine allowable soil pressure under 6 x 12 pads:

(Use NAVFAC inclined load on inclined footing - See Appendix B)

Angle of pad to horizontal = 61°

$D/B = (4.0/5.0) = 0.8$ $N_{\gamma q}$ From graph = 20

$q_{ult} = CN_{cq} + 1/2(\gamma B)N_{\gamma q} = 0 + 1/2(115)(5)(20) = 5{,}750$ psf

$q_{Allowable}/FS = 5{,}750/2 = 2{,}875 > 2{,}000$ psf Use 2,000 psf

Pad bearing area needed = $47{,}553/2{,}000 = 23.78$ Ft^2

Pad length needed = 23.78/5.00 = 4.76 Ft

Pad cantiliver length = (4.76 - 1.00)/2 = 1.88 Ft

M (for 1 pad) = $wL^2/2 = 2,000(1.88)^2/2 = 3,534$ Ft-Lb

$f_b = M/S = 3,534(12)/[12(6)^2/6] = 589 < 1,500$ psi

Shear $V = 2,000(1.88 - 0.5) = 2,760$ Lb

$f_v = 3V/2A = 3(2,760)/2(6 \times 12) = 58 < 140$ psi

Check Corbel:

Raker to corbel crushing = $47,553/12 \times 12 = 330 < 450$ psi

Length for flexure = $(5.00 - 1.00)/2 = 2.00$

Load per foot of corbel = $2,000(4.76) = 9,520$ Lb/Et

$M = wL^2/2 = 9,520(2)^2/2 = 19,040$ Ft-Lb

$f_b = M/S = 19,040(12)/[12(12)^2/6] = 793 < 1,800$ psi

Length for shear = $5.00/2 - 1.00/2 - 1.00 = 1.00$ Ft

Shear $V = 2,000(4.76)(1.00) = 9,520$ Lb

$f_v = 3(9,520)/2(12 \times 12) = 99 < 144$ psi

Summary:

Use HP12 x 74 (equivalent or larger) soldier pile (D = 10'- 3").
Use 6 x 12's for lagging (may use 4 x 12 for upper half).
Use 12 X 12 raker.
Provide at least 5 x 5 pad.

SAMPLE PROBLEM 10-5: PREVIOUS PROBLEM WITH NO RAKER

Given:
Analyze previous problem using an H of 8 feet and no raker.

Solution:
Instead of adjusting K_p per FHWA recommendation, use a safety factor of 30% for D.

$K_a = 0.65 \quad K_p = 3.69$ (Arbitrary values)

$K_p - K_a = 3.69 - 0.65 = 3.04$

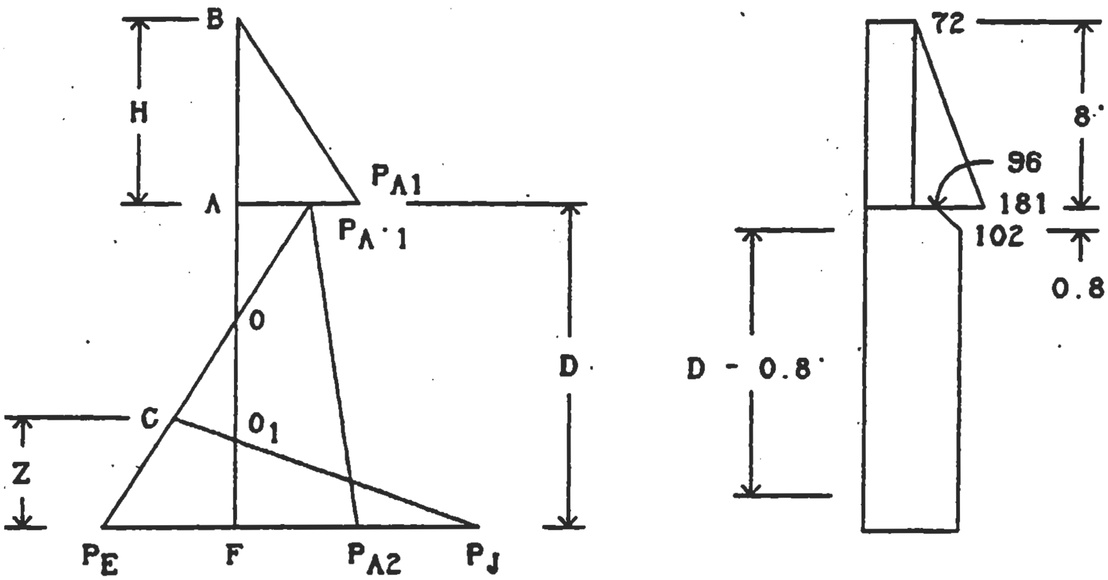


FIGURE 10 - 16

$$P_{A1} = K_a \gamma H = (0.65)(115)(8)$$
$$= 598 \text{ psf}$$

$$P_{A'1} = (0.53)(598)$$
$$= 317 \text{ psf}$$

$$P_{A2} = P_{A'1} + f\gamma D K_a = 317 + (0.53)(115)(0.65)D - 317$$
$$= 317 + 40D$$

$$P_E = f\gamma D(K_p - K_a) - P_{A'1} = (0.53)(115)(3.04)D - 317$$
$$= 185D - 317$$

$$P_J = f\gamma D(K_p - K_a) + f\gamma H K_p$$
$$= (0.53)(115)(3.04)D + (0.53)(115)(8)(3.69)$$
$$= 1{,}799 + 185D$$

Areas:

ABA_1 $598(8)/2 = 2,392$

$AA'_1A_2F = (317 + 317 + 40D)D/2 = 317D + 20D^2$

$ECJ = (185D - 317 + 1,779 + 185D)Z/2 = 185DZ + 731Z$

$EA'_1A_2 = -(185D - 317 + 317 + 40D)D/2 = -113D^2$ **(negative area)**

Surcharge area (for ease of computation assume surcharge uniform below excavation):

$72(8) + (181 - 72)(8)/2 + (102)D = 1,012 + 102D$

$\Sigma F_H = 0 \quad \therefore ABA_1 + AA'_1A_2f + ECJ - EA'_1A_2 + \text{surcharge} = 0$

$2,392 + 317D + 20D^2 + 185DZ + 731Z - 113D^2 + 1,012 + 102D = 0$

$z = (D^2 - 4.51D - 36.60)/(1.99D + 7.86)$

$\Sigma M_F = 0$

$$= ABA_1[D + 8/3] + (P_{A'1})(D)[D/2] + (P_{A2} - P_{A'1})(D/2)[D/3] + ECJ[Z/3] - EA'_1A_2[D/3] + 576[8/2 + D] + 436[8/3 + D] + 102D[D/2]$$

$$2,392[D + 2.7] + 158D^2 + 6.67D^3 + (185DZ + 731Z)[Z/3] - 37.67D^3 + (2,304 + 576D) + (1,163 + 436D) + 51D^2 = 0$$

$Z^2 = (D^3 - 6.74D^2 - 109.81D - 320.17)/(1.99D + 7.86)$

By trial and error, or by other means: Z = 4.15' and D = 16.88'
Increase D by 30% D = 1.30(16.88) = 21.9 Ft
Use D = 22 Ft

<u>Check Soldier Pile:</u> $Find_{max}$

Locate point of zero shear.
Assume point between 0 & G).

$AO/P_{A'1} = D/(P_{A'1} + P_E)$
$AO = 1.71'$

$OG = D - AO - z$
$= 16.88 - 1.71 - 4.15 = 11.02'$

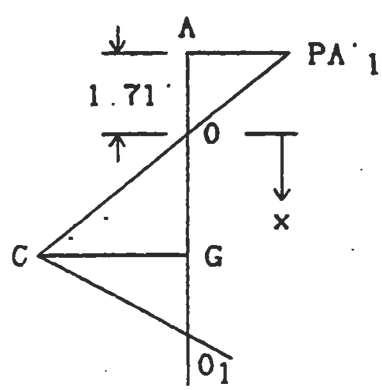


FIGURE 10 - 17

Shear at point 0 = 2,392 + 576 + 436 + 317(1.71)/2 + (102)(1.71)

= 3,849 Lb/LF

$3,849 + 102x = 185x(x/2)$

$92.5x^2 - 102x - 3,849 = 0$

$x^2 - 1.10x - 41.61 = 0 \quad \therefore x = 7.02'$ (and assumption is correct)

$AO + x = 1.71 + 7.02 = 8.73'$

$$\begin{aligned} M_{max} &= 2,392[8/3 + 8.73] + 576[8/2 + 8.73] + 436[8/3 + 8.73] \\ &\quad 271.0[(1.71)(2/3) + 7.02] + 102(8.73)[8.73/2] \\ &\quad -(185)(7.02)(7.02/2)[7.02/3] \\ &= 34,994 \text{ Ft-Lb/LF} \end{aligned}$$

Total soldier pile moment = moment per foot times pile spacing:

= 8(34,994) = 279,952 Ft-Lb

S required = M/F_b = 279,952(12)/22,000 = 152.7 in^3

S furnished with HP12 x 84 = 106 in^3 < 152.7 in^3

Use HP14 x 117, S = 172 in^3

A second point of zero shear occurs near the computed depth, but this point is not normally used for maximum moment.

Compare the moment computed above to the moment at the depth of excavation:

M = 2,392[8/3] + 576[8/2] + 436[8/3] = 9,846 < 34,994 Ft-Lb/Lf

The portion of piling encased in sound concrete (generally, four sack or better) comprises a composite section usually having a large section modulus. If this is the case, the moment at or above the excavation elevation may be controlling to determine the critical section modulus.

CANTILEVER SOLDIER PILE - COHESIVE SOIL (ϕ = 0 METHOD)

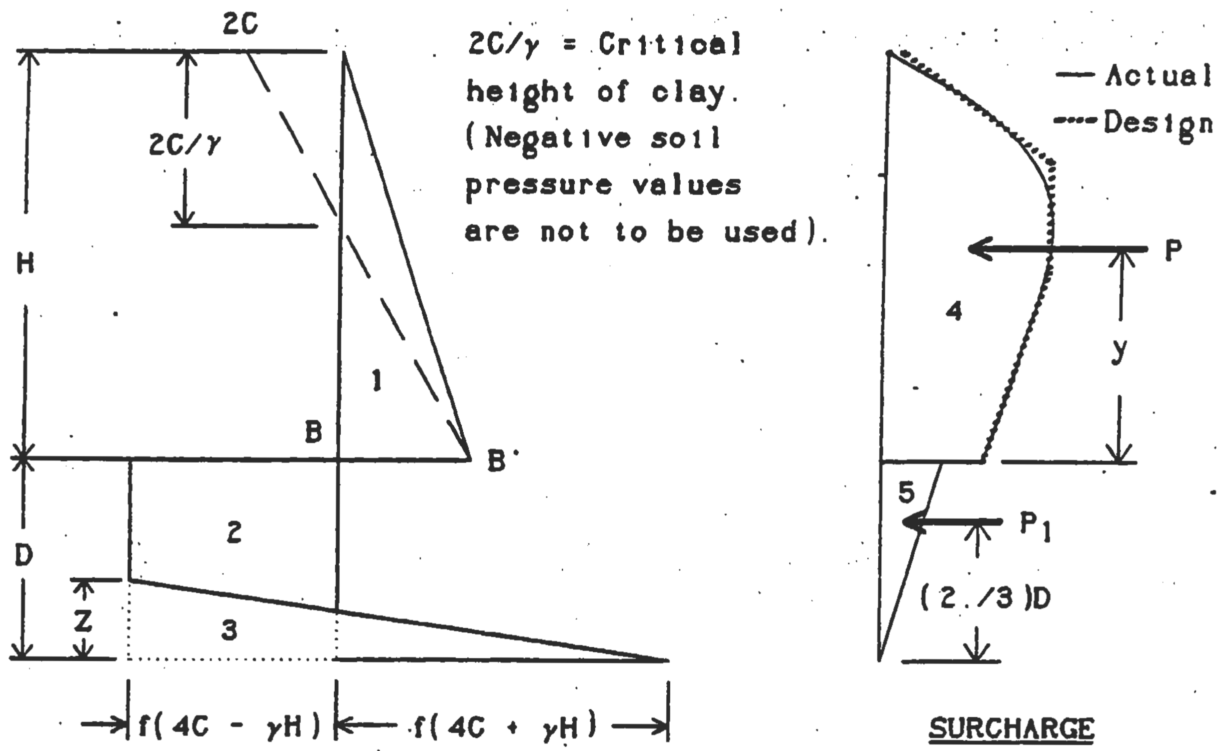


FIGURE 10-38

Use a safety factor of 50% - 70% with the clay or increase D by 20% - 40%. Critical height of wall = H_c = $4C/\gamma$. Theoretically the wall will fail if $\gamma H_c > 4$.

BB' = $\gamma H - 2C \geq 0$. (If not, see note below)
f = Arching factor

1 = BB'(H/2)
2 = f(4C − γH)D
3 = {f(4C − γH) + f(4C + γH)}{Z/2} = 4fCZ
4 = P = Area under dashed line above the excavation depth.
5 = P_1 = Area below the excavation depth (when used).

1) $\Sigma F_H = 0 = 1 - 2 + 3 + 4 + 5$ and $Z = (2 - 1 - 4 - 5)/4fC$

2) $\Sigma M_{base} = 0 = 1[H + H/3] - 2[D/2] + 3[Z/3] + 4[D + y] + 5[2D/3]$

Solve equations 1) and 2) simultaneously for D and Z.
Determine maximum moment and section modulus required.
Determine. lagging requirements.

*Note : **If ϕ does not = 0, or if BB' < 0, see next page.**

CANTILEVER SOLDIER PILE - COHESIVE SOIL (ALTERNATE METHOD)

This approach should be used only when $\phi \neq 0$, or when $BB' \leq 0$.

If $\phi \neq 0$, then $BB' = \gamma K_a H$ where $K_a = \tan^2(45° - \phi/2)$

If $BB' < 0$, then assume $C = 0$ and $\phi = 20°$ to $30°$.
$BB' = \gamma H K_a$ where $K_a = \tan^2(45° - \phi/2)$

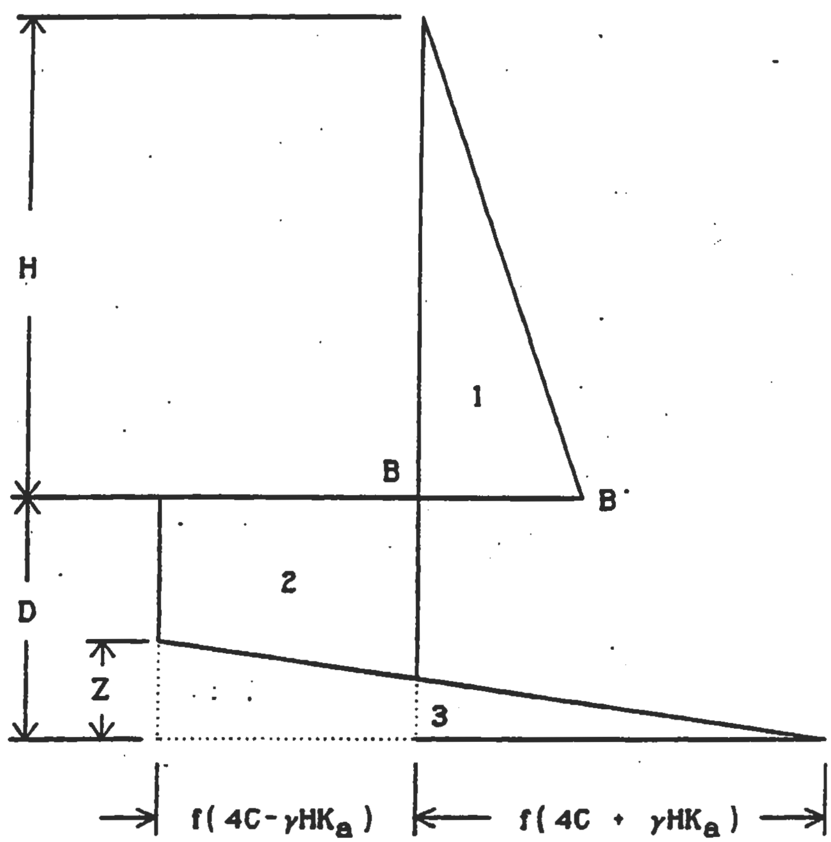
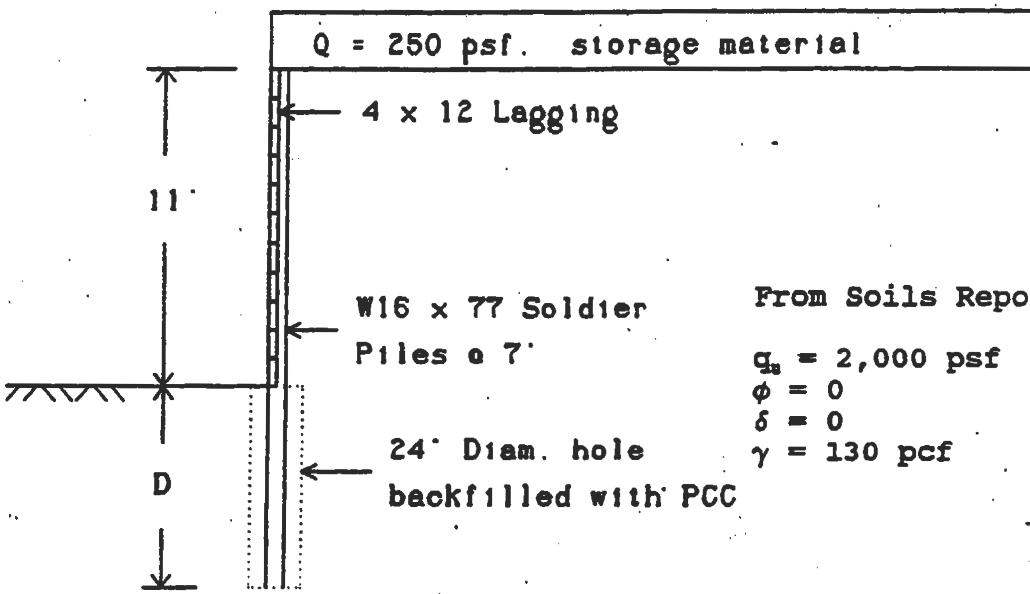


FIGURE 10 - 19

The procedure from this point on, including the addition of any surcharges, is identical to the "$\phi = 0$ method" outlined on the previous page.

SAMPLE PROBLEM 10-6: CANTILEVER SOLDIER PILE: COHESIVE SOIL

Given:



From Soils Report:

q_u = 2,000 psf
ϕ = 0
δ = 0
γ = 130 pcf

FIGURE 10 - 20

Solution:

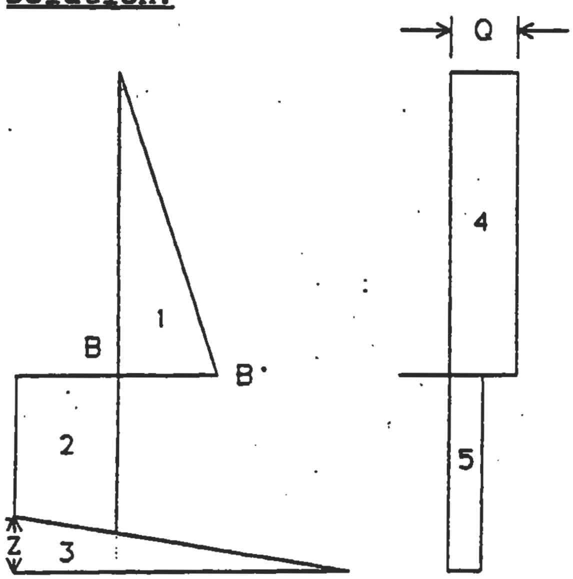


FIGURE 10 - 21

Use a safety factor of 1.6 for clay.

$q_u/1.6 = 2{,}000/1.6 = 1{,}250$ psf
$C = q_u/2 = 1{,}250/2 = 625$ psf

Arching capability = 2.0
$f = 2(24/12)/7 = 0.57$

Check critical height of wall:
$H_c = 4C/\gamma$
$= 4(625)/130 = 19.2' > 11'$

$BB' = \gamma H - 2C$
$= 130(11) - 2(625)$
$= 180$ psf

AREAS:
1 = $bb'h/2 = 180(11)/2 = 990$
2 = $F(4c - \gamma h)d = [(0.57)(4)(625) - 130(11)]D = 610D$
3 = $4fCZ = 4(0.57)(625)Z = 1{,}425Z$
4 = $QH = 250(11) = 2{,}750$
5 = $fQD \doteq 0.57(250)D = 143D$

Determine D:

$\Sigma F_H = 0$

$= 990 - 610D + 1{,}425Z + 2{,}750 + 143D = 0$

$3{,}740 + 1{,}425Z - 467D = 0$

1) $Z = 0.33D - 2.62$

$\Sigma M_{base} = 0$

$= 990[D + 11/3] - 610D[D/2] + 1{,}425Z[Z/3] + 2{,}750[D + 11/2]$
$+143D[D/2] = 3{,}740D + 18{,}755 + 475Z^2 - 233D^2$

2) $Z^2 = 0.49D^2 - 7.87D - 39.48$

By trial and error, or by other means, solve equations 1) and 2):

There is no need to increase D since a safety factor has already been applied to the clay.

<u>Find Maximum Moment:</u> (Assuming non-composite section):

Locate point of zero shear.

Shear at excavation elevation.
990 + 250(11) = 3,740 psf/LF

Determine distance below excavation level to obtain shear. equality:

$610x - 143x = 467x$ psf/LF
$467x = 3{,}740 \quad \therefore x = 3{,}740/467$

$M_{max} = 990[8.01 + 11/3] + 250(11)[8.01 + 11/2]$
$+143(8.01)[8.01/2] - 610(8.01)[8.01/2]$
$= 33{,}731$ Ft-Lb/LF

S required = $33{,}731(7)(12)/22{,}000 = 128.8\ in^3$
S furnished = $134\ in^3 > 128.8\ in^3$ OK

Often, with sound concrete below the depth of excavation, the moment occurring at that elevation may be used to determine the critical section modulus.

Check Lagging:

Consider arching effect on lagging. Multiply all loads by 0.6. By inspection, maximum moment occurs at the depth of excavation.

$M_{max} = wL^2/8 = (180 + 250)(7)^2/8 = 2{,}634$ Ft-Lb

S required = $2{,}634(12)(0.6)/(1{,}500) = 12.43$ in^3.

S furnished (rough lumber) = 32 in^3 > 12.43 in^3 OK

$V = (7/2 - 0.33)(180 + 250)(0.6) = 818$ Lb

$v = 3V/2A = 3(818)/[2(4)(12)] = 25.6$ psi < 140 psi OK

EFFECT OF SURCHARGE BELOW DEPTH OF EXCAVATION

Sample problems 10-1, 10-3, 10-6 and 22 (Appendix F) were recomputed using no surcharge below the depth of excavation to demonstrate the negligible difference in answers. A comparison of answers follows:

SAMPLE PROBLEM 10-1	WITH SURCHARGE	WITHOUT SURCHARGE
D	22.3′	21.1′
130%(D)	29.0′	27.4′
Z	4.91′	4.7′
M_{total}	1,082,016 Ft-Lb	1,009,632 Ft-Lb
S Required	590.2 in^3	550.7 in^3

SAMPLE PROBLEM 10-3		
D_w	5.62′	5.2′
D	11.62′ Use 11′- 8"	11.2′ Use 11′-3"
T	10,785 Lb/LF	10,685 Lb/LF
Total T	80,888 Lb	80,138 Lb
Combined Moment	35,607 Ft-Lb/LF	34,587 Ft-Lb/Lf
S Required	145.67 in^3	141.5 in^3

SAMPLE PROBLEM 10-6		
Z	4.54′	5.1′
D	21.71′	18.0′
M_{max}	33,731 Ft-Lb/LF	30,220 Ft-Lb/LF
S Required	128.8 in^3	115.4 in^3

SAMPLE PROBLEM 22 (Appendix F)		
Y	20.45 ′	19.77′
D	24.26′	23.63′

ACCEPTABLE ALTERNATE DESIGN METHODS WHICH HAVE BEEN USED

Cantilever System or Single Tie (Or Strutted) System:

Surcharge (S) may be limited to a depth of 10 feet or more, or to the elevation of the upper tie or strut (depicted as force T).

Dimension A is designers choice.

The location of point M is the designers choice. M may be located anywhere between points L to N. Point N is used for cantilever sheetpile or continuous walls.

Forces above L-N represent active loads on the soldier pile.

Passive forces are based on the effective pile diameter which includes the unitless number 3 (divided by an appropriate safety factor), times the pile dimension or drilled hole diameter, times K_p, times the unit weight of the soil.

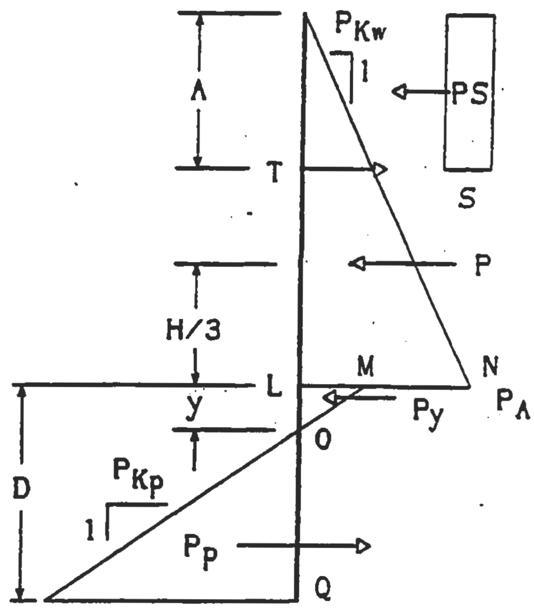


FIGURE 10 - 22

$y = LM/[3dK_p\gamma/S.F.]$ (May not include contribution of surcharge).

Cantilever Systems:

D is generally determined from moments taken about point Q. A safety factor against overturning should be included so that passive moments exceed active moments, all taken above point Q.

The section modulus of the soldier pile may be determined from moments taken about the point of zero shear.

Single Tie Or Strut System: (Only one method described below.)

T may be determined from moments of the forces above point 0.

D may be determined from moments of the forces about T.

Check stability against overturning by taking moments about Q.

The section modulus required for the soldier pile is determined from the larger of the cantilever moment for the forces above point T, or from moments taken about the plane of zero shear.

AASHTO Soldier Pile Method

The figure below represents the soil pressures that may be used for cantilever soldier piles cohesionless soil This figure (excluding surcharge) is an adaptation from the figure titled, Simplified Earth Pressure Distributions for Permanent Flexible Cantilevered Walls with Discrete Vertical Wall Elements, in AASHTO Standard Specifications For Highway Bridges.

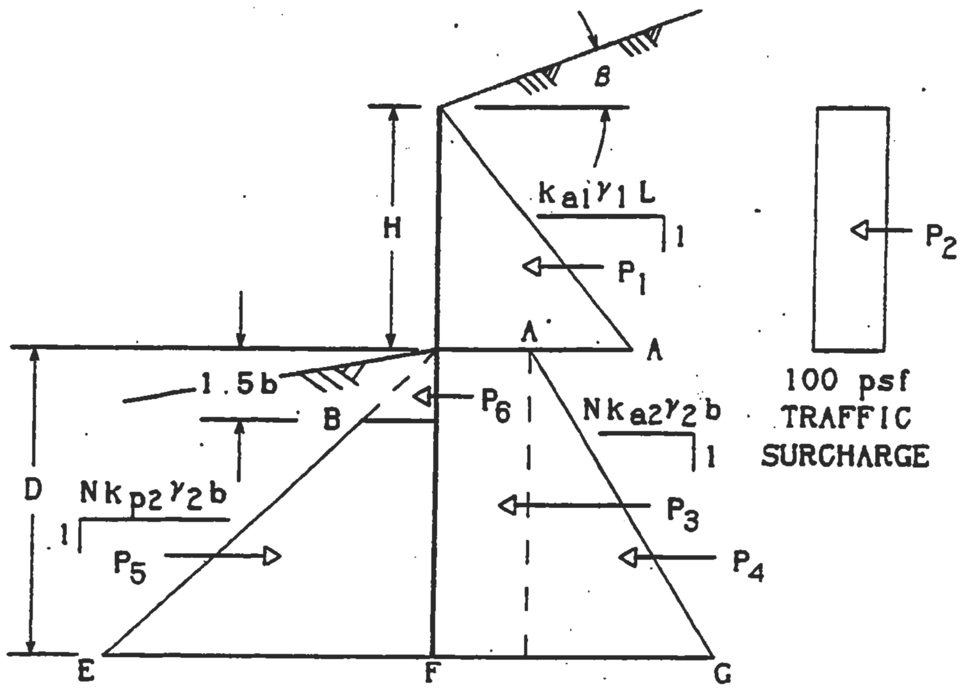


FIGURE 10 - 23

L = Soldier Pile Spacing
N = Increase in effective pile width. AASHTO uses N = 3, (Caltrans uses N = **0.08ϕ**). The value of Nb cannot exceed the soldier pile spacing "L" (AASHTO uses L $\leq$ 5b).
b = Effective pile width: pile width or width of drilled hole backfilled with hard rock concrete.

PROJECT USE:

A-minimum safety factor of 50% is to be added to the computed embedment depth (D) for permanent flexible cantilever walls, otherwise 30% should be added for temporary construction.

For temporary construction the forces P_3 and P_4 may be ignored.

GENERAL PRESSURE EQUATIONS:

$P_A = K_{a1}\gamma_1 H(L)$ $P_B = K_{p2}\gamma_2(1.5b)(Nb)$ $Schrg = P_S(L)$

$P_{A'} = K_{a2}\gamma_2 H(Nb)$ $P_G = K_{a2}\gamma_2 D(Nb)$ $P_E = K_{p2}\gamma_2 D(Nb)$

SAMPLE PROBLEM 10 - 7: USING THE AASHTO METHODOLOGY

ϕ = 32° $\beta = \phi$

$\gamma_1 = \gamma_2$ = 110 pcf

L = 7 Ft

b = 1.33 Ft

N = 0.08(32)
= 2.56

Nb = 2.56(1.33)
= 3.40

From Log-Spiral:
K_a = 0.80
K_p = 8.0(0.425)
= 3.4

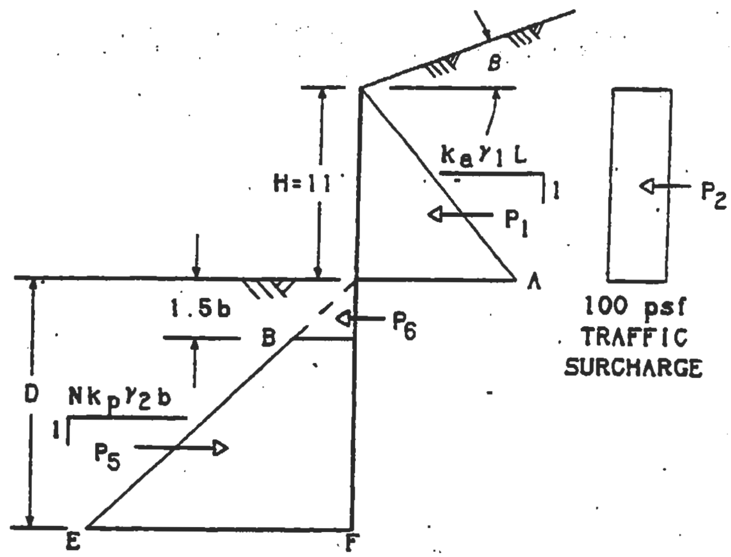


FIGURE 10-24

General Equations:

$P_A = K_a\gamma_1H(L)$ = 0.8(110)(11)(7) = 6,776 Lb/Ft

P_S = 100(L) = 100(7) = 700

$P_E = K_p\gamma_2D(Nb)$ = 3.4(110(3.4)D = 1,271.6D

$P_B = K_p\gamma_2(1.5b)(Nb)$ = 3.4(110)(1.5)(1.33)(3.40) = 2,536.8

$P_1 = P_A(H/2)$ = 37,268 Lb

$P_2 = P_SH$ = 7,700

$P_6 = P_B(1.5b/2)$ = 2,530.5

$P_5 = P_E(D/2)$ = $- 635.8D^2$

Determine D by Taking Moments About F:

$0 = P_1[D + H/3] + P_2[D + H/2] + P_6[D - 2/3(1.5b)] - P_5[D/3]$

$= 37{,}268D + 136{,}649.3 + 7{,}700D + 42{,}350 + 2{,}530.5D - 3{,}365.6 - 211.9D^3$

$= 211.9D^3 - 47{,}498.5D - 175{,}633.7$ From which D = 16.56′

Use D = 1.30(16.56) = 21.5 Ft

Locate Depth To Plane Of Zero Shear:

(Use x in lieu of D in Figure 10 - 25)

$$P_1 + P_2 + P_6 = P_5$$

$$37{,}268 + 7{,}700 + 2{,}530.5 = 635.8(x^2)$$

$$x^2 = 74.71' \qquad x = 8.64'$$

Determine Moment At Plane Of Zero Shear:

$$M = P_1[8.64 + H/3] + P_2[8.64 + H/2] + P_6[8.64 - 2/3(1.5b)] - P_5[8.64/3]$$

$$= 37{,}268[12.31] + 7{,}700[14.14] + 2.530.5[7.31] - 635.8(8.64)^2[2.88]$$

$$= 458{,}769 + 108{,}878 + 18{,}498 - 136{,}691$$

$$= 449{,}454 \text{ Ft-Lb}$$

Determine Section Modulus Required:

$S = M(12)/22{,}000 = 449{,}454(12)/22{,}000 = 245\ in^3$

W12 X 190 ($S = 263\ in^3$) Could be used (providing deflection is not a consideration).

Appendix "C" Write up for program "Winslope"

"Winslope" © (1) Analysis Procedures
By: Toorak Zokaie, Ph.D.

The software program Winslope is used to perform soil nailing analysis on the ultimate strength basis. The basic failures plane is a bilinear one, and the factor of safety is calculated by balancing the driving and resisting forces. The forces that are in effect in such an analysis include the weight, the soil resistance from cohesion and internal friction (C and Φ), and the forces from the nail. In addition, the earthquake effect and the effect of ground water are considered. A typical soil nailed wall is shown in the following figure.

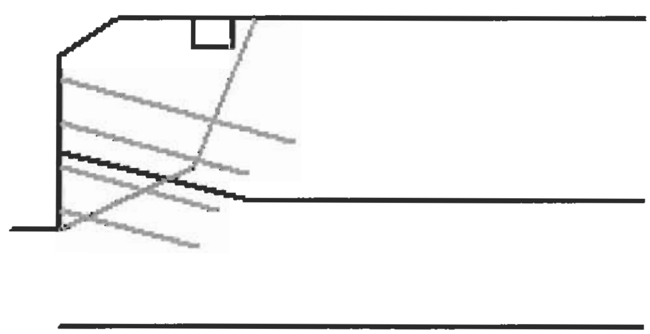


Figure 1: A Typical Wall

The overall analysis process is explained in the following sections as follows:

1. Analysis Procedure
2. Design/Check Procedure
3. User Interface
4. Definition of Output Parameters
5. Verification
6. Modeling Techniques

1. Analysis Procedure

The basic analysis assumption is that the failure condition is made of two wedges. It is assumed that the two wedges (as shown in Figure 2) fail with a vertical plane. The assumption is that the lower wedge (wedge 1) will move toward outside and the upper wedge (wedge 2) will move downward. The forces acting on the total system include weight (W), Nail forces (N), Friction (Fr), Cohesion (C), and Normal force at the intersection (R). Considering Wedge 1 and Wedge 1, the following forces will act on the system, as well as on the interface between the two edges. Note that the failure surface is called ABC, where A is the toe, C is on the surface, and B is the interface between the two surfaces. Additionally, the location on the surface directly above point B is called B'.

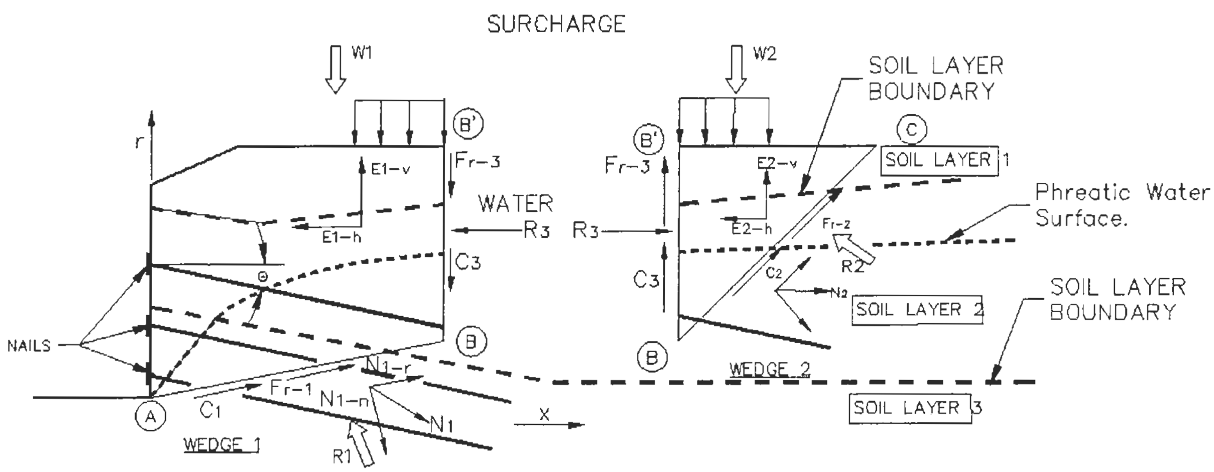


Figure 2: Failure Surface and Forces

Table 1: List of Forces acting on the Slope

Notation	Description	Direction	Act on Wedge	Condition	D/R*
W1	Weight of Wedge 1	Vertical (downward)	1	Known	D
N1-N	Total force from all nail (Component)	Normal to edge AB	1	Known	R
N1-P	Total force from all nail (Component)	Parallel to edge AB	1	Known	R
R1	Normal interaction force along AB	Normal to edge AB	1	Unknown	D
Fr1	Friction force	Parallel to edge AB	1	Known	R
C1	Cohesion force	Parallel to edge AB	1	Known	R
E1-H	Earthquake force	Horizontal	1	known	D
E1-V	Earthquake force (Uplift)	Vertical (upward)	1	known	D
R3	Interaction force along BB'	Normal to edge BB'	1&2	Unknown	D
Fr3	Friction force	Parallel to edge BB'	1&2	Known	R
C3	Cohesion	Parallel to BB'	1&2	Known	R
W2	Weight of Wedge 2	Vertical (downward)	2	Known	D
N2-N	Total force from all nail (Component)	Normal to edge BC	2	Known	R
N2-P	Total force from all nail (Component)	Parallel to edge BC	2	Known	R
R2	Normal interaction force along AB	Normal to edge BC	2	Unknown	D
Fr2	Friction force	Parallel to edge BC	2	Known	R
C2	Cohesion force	Parallel to edge BC	2	Known	R
E2-H	Earthquake force	Horizontal	2	known	D
E2-V	Earthquake force (Uplift)	Vertical (upward)	2	known	D

D = Used Directly, R = Reduced by safety factor

Note that these forces are all known with the exception of the interaction forces (R1, R2, and R3). The safety factor is defined as the factor that can be used to reduce the resistive forces (Friction, Cohesion, and Nail) to equate the driving forces (Weight, Earthquake, and Interaction, R). Considering the three unknown forces and the unknown safety factor, there are a total of four unknowns. Using the equilibrium equations for forces on each wedge in horizontal and vertical directions, four equations are available to solve for the four unknowns.

The solution scheme considers the equilibrium of each wedge, and the fact that it has three unknowns. For example, Wedge 1 has R1, R3, and f (safety factor) as unknowns. Assuming a value for the safety factor (f) the other two can be obtained. Therefore a value for R3 is calculated. The same is repeated for Wedge 2, and another value for R3 is obtained. The correct solution is obtained when the two values of R3 are the same. This is obtained by trial and error (iterations); i.e., different values of f are tried until the R3 value from Wedge 1 and Wedge2 are within an acceptable tolerance. Note that it is possible that in some cases an acceptable safety factor cannot be obtained due to numerical nature of the solution. If, after a maximum specified number of iterations, a solution is not reached, it usually means that the safety factor is too low and the solution should be revised, by providing more resistance, usually achieved by providing more nails.

Each one of the known forces is calculated based on the geometry and properties of the site, and are explained below:

Calculation of Weight:

The site may contain several layers of soil, and each layer may have different properties, e.g, unit weight, at each layer. The weight of each wedge is calculated based on the soil that is contained within each wedge. Note that if there is any surcharge force that lies directly above each wedge, then this surcharge force is directly added to the weight of the wedge.

Calculation of Nail Force:

Each nail may have its own direction and length. The maximum force that can be developed in each nail is controlled by three actions. These include the pull out from the wall end or the tip and the tensile yield failure in the bar. The pull out force is dependent on the bond between the soil and the nail, which is a constant value, multiplied by the total contact area between them. This contact area is the perimeter of the nail multiplied by the length in contact. The maximum tension is calculated as the yield stress of the reinforcing bar multiplied by the area of the rebar. The force diagram for a typical nail is shown in Figure 3.

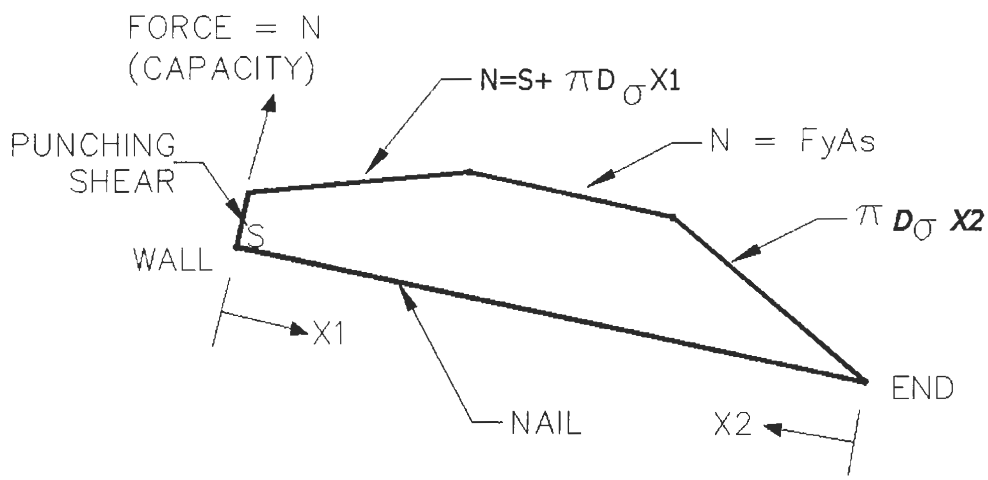


Figure 3: Nail Force Calculation

The pull out force at the end of the nail is calculated as the contact area, multiplied by the length of the nail from the failure plane to the tip of the nail. This may be shown as: Ne=σ*πD*Ln2, where, σ is the bond stress (also sometimes shown as C'), D is diameter of the whole, and Ln2 is the distance from the point of intersection of the nail and the failure plane to the end of the nail. Note that if this portion of the nail is in contact with more than one soil layer, then the total pull out force is calculated as the sum of the pull out forces along the length under consideration.

The pull out force at the wall end of the nail is calculated in a similar fashion, however, due to the connection of the nail and wall, the shear capacity of the wall can be added to this capacity, i.e., NW= σ *πD *Ln1+S. In this case Ln1 is the distance from the wall to the point of intersection of the nail and the failure plane, and S is shear capacity of the wall.

The maximum tensile capacity of the nail depends on the amount and type of rebar contained within the nail, i.e., NT=fy*πr2. In this case, fy is the yield stress of the rebar, and r is the radius of the rebar.

The controlling nail force is then calculated as the minimum of the three forces calculated above: N=min(Ne, Nw, NT). This force acts in the direction of the nail. Therefore, if the nail has an inclination angle of θ, then the horizontal component of this force is N*cos(θ), and the vertical component of this force is N*sin(θ). The total nail force along each failure plane (AB or BC) is the sum the horizontal or vertical components of all nails crossing that failure plane.

It should also be noted that the nail force calculated above is per nail; therefore it must be divided by the horizontal spacing of the nails to result in the nail force per unit width of the soil, and be compatible with the other forces such as the weight or internal soil resistance forces.

Calculation of the Internal Soil Cohesion:

The cohesion in the soil depends only on the soil inherent properties. The internal cohesion within any soil layer is defied in units of pressure. Therefore the total cohesion force per unit of width is calculated as the product of the soil cohesion and the length over which it acts, i.e., the wedge plane AB, BC, or BB'. In the case that more than one soil layer crosses a wedge plane, the total force is calculated as the sum of the forces over each segment of the line. Therefore if the total force over plane AB is shown as C1L1, then this force is calculated as:

$C1L1 = \sum C1iL1i$

Similarly, the force over planes BC and BB' are calculated in a similar fashion.

Calculation of the Internal Soil Friction:

The friction force capacity is defined by the friction angle (φ). The maximum internal friction force may be calculated as the a fraction of the normal force along the surface (R), i.e., $Fr = R*\tan(\varphi)$.

Therefore, the friction force along the plane AB is calculated as a fraction of the normal reaction force along this plane. Therefore: $Fr1 = R1*\tan(\Phi 1)$. The force along planes BC and BB' are also calculated in a similar fashion. Note that the normal reaction force, R, is also unknown. Therefore, if a number of soil layers cross the failure plane, then an average friction angle is calculated as the weighted average of the friction angles for the segments of the failure plane, i.e.,: $\tan(\varphi\ 1) = \sum\tan(\varphi\ 1i)$.

Calculation of the Earthquake Force:

The earthquake force is calculated as a fraction of the weight. The factor (fraction of the weight) is determined as a typical value based on other soil and overall site properties. Therefore, if this ratio is defined as "e", then the earthquake force in the lower Wedge is calculated as: E1H=e*W1. Similarly, the force in the upper wedge is: E2H=e*W2. In the case that vertical acceleration for the earthquake force is to be considered, then this force is defined as the ratio of the vertical to the horizontal force, v, i.e., E1v = v* E1H. The force in wedge 2 is calculated in a similar fashion.

Calculation of the Water Effect:

Whenever the water table crosses the failure planes, i.e., water is present within the wedges, the capacity of the plane is reduces, i.e., the factor of safety is smaller. This effect is included in the calculations in the following ways:

1. The weight of the water is included in the weight of the wedge. This weight is calculated as the void ratio of the soil multiplied by the volume of soil that is submerged in water.

2. The internal soil friction force is reduced by the effect of the buoyancy force (BF). The buoyancy force is the volume of the submerged soil multiplied by the quantity (1-voidratio), multiplied by unit weight of water. This reduction in the friction force is accomplished by calculating: BF*tan(φ), and applying it is the direction opposite to the internal friction force, Fr.

Multi-Linear Failure Surface:

The failure surface may also be defined to be made of a multi-linear curve, instead of a bilinear curve. The critical point for such a surface is the fact that failure surface may run through a weaker soil, and thus result in friction and cohesion values much less than a bilinear path through the soil. If a multi-linear failure surface is defined, the point of intersection of the two wedges (point B) should also be defined. With the two wedges identified, the remainder of the parameter calculations will be carried out in a similar fashion. It is interesting to note that, for example the resultant of all cohesion forces along a failure plane or curve (say AB) will lie along the straight line AB. The value of the total cohesion force can be found by finding the weighted average value of the cohesion along the curve AB, and multiplying it by the length of the straight line AB. Other parameters such as weight or water effects must be calculated by considering the actual path through the failure surface.

2. Design/Check Procedure

In order to find the critical failure surface and the lowest factor of safety, various failure planes are examined. This is done by considering a solution area, bound by a distance behind the wall. For example, we may consider that the solution area is contained within a space up too 100 ft behind the wall. In practice, this area should be taken as a large enough area so that the critical surface falls well within it.

Note that the trial surfaces may be examined for various depths of cut. Therefore, the location of point A (in ABC failure surface described earlier) may be specified at various depths. In Winslope, the Y coordinate is measured from the toe of wall upward. Therefore, higher values of Y correspond to lower depths of cut. For each depth a number of failure surfaces can be examined.

Secondly, a number of trial bilinear surfaces should be tried. This is accomplished by first locating all the surfaces that end at specific locations on top of the soil. This is done by specifying a start and an end location, and specifying the number of locations. For instance we may specify that the trial curves are contained from 4 ft to 100 ft behind the wall, and we would like to try all locations in between in 2 ft increments. This means that the location of point C in the ABC failure surfaces will be set to 4, 6, 8, …, and 100 ft behind the wall in various trials. Note that if the starting location is before the X coordinate of point A, then the coordinate of point A is used as the starting point.

Next, for each location of points A and C, the location of point B should be varied, and tried. This is accomplished by dividing the horizontal and vertical distance between points A and C in a number of divisions, N, effectively creating a grid of intersection points. Each point in this grid specifies a trial location for point B, and thus a trail failure surface. Winslope currently uses a 10 by 10 grid.

Next, it must be noted that some locations of point B are deemed unacceptable by examination. These include: 1. Locations for which a portion of the ABC curve falls outside the soil, such as a stepped wall with point B close to the step, and directly under it.

2. Locations for which the slope of line BC is flatter than the slope of the line AB.

The grid line used in design/check trials is shown in Figure 4.

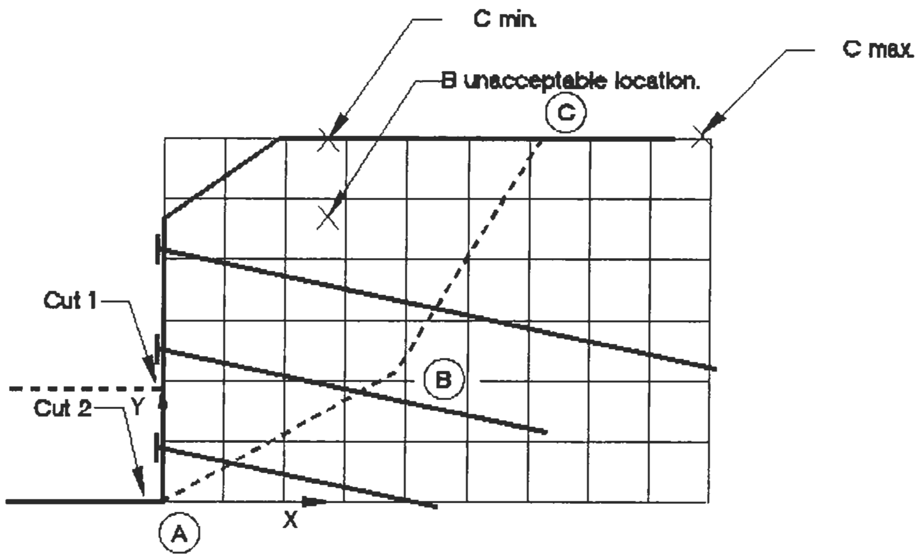


Figure 4a: Grid line of B points for a Given A and C

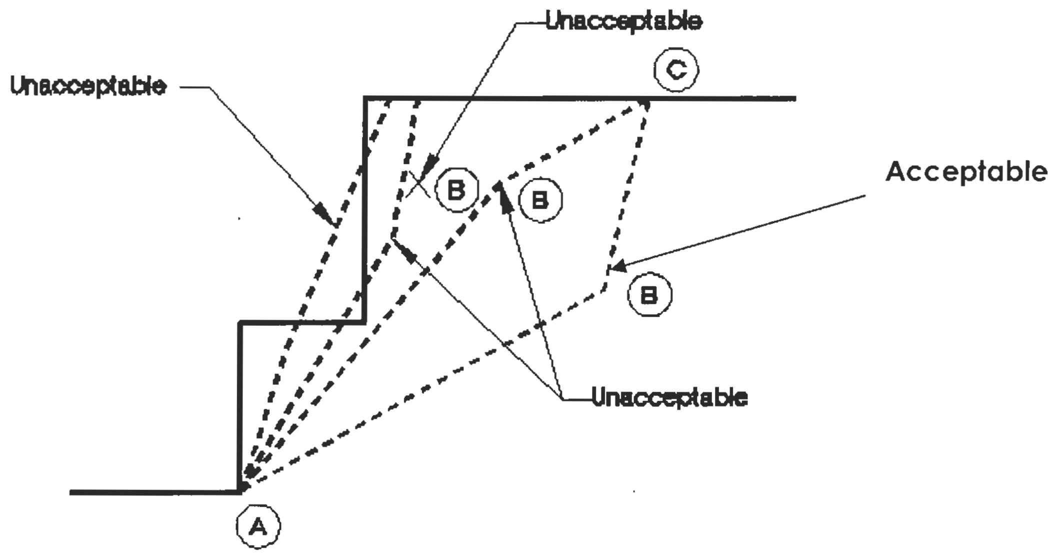


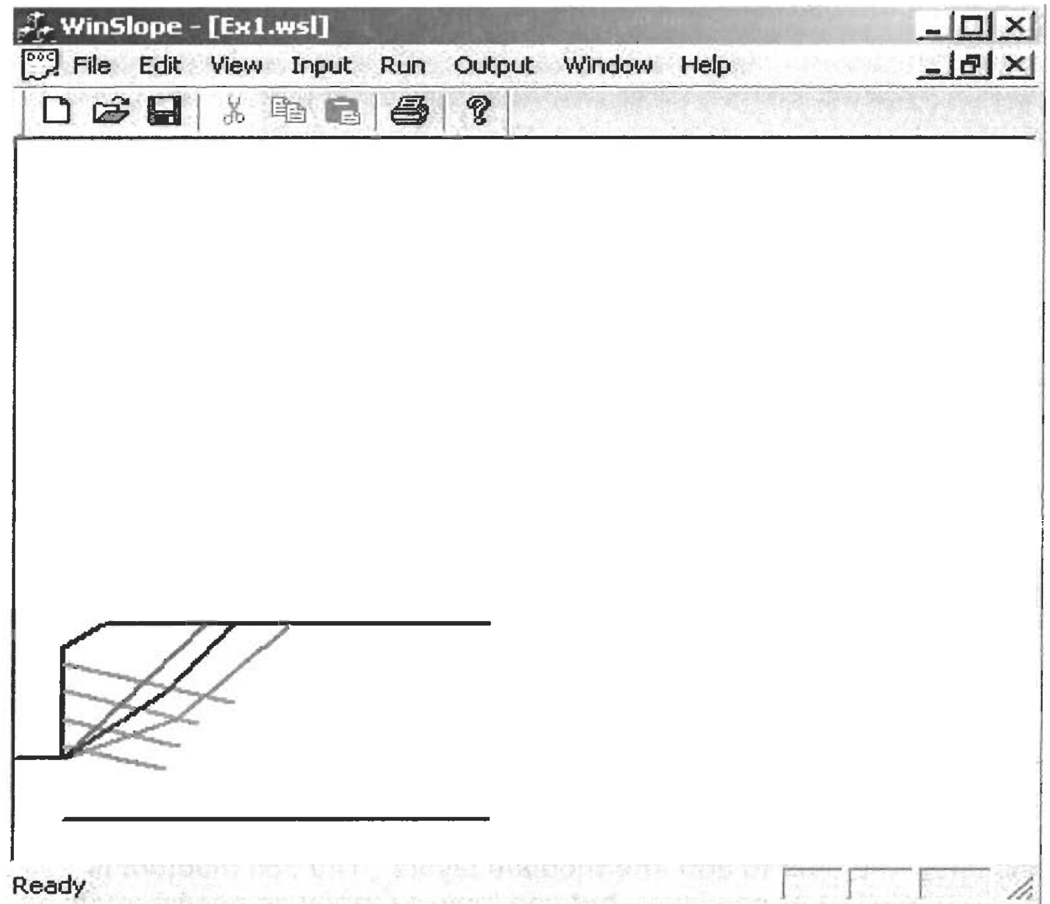
Figure 4b: Example of acceptable and unacceptable intersection (B) Points

3. User Interface:

The computer program Winslope performs a soil-nail analysis based on bilinear critical plane assumption. This program can work in two modes, one as an analysis program with given failure surface, and another as a check program to determine the most critical failure surface. For detailed description of the methodology, refer to the previous sections. The following sections provide a description of the input data and user interaction.

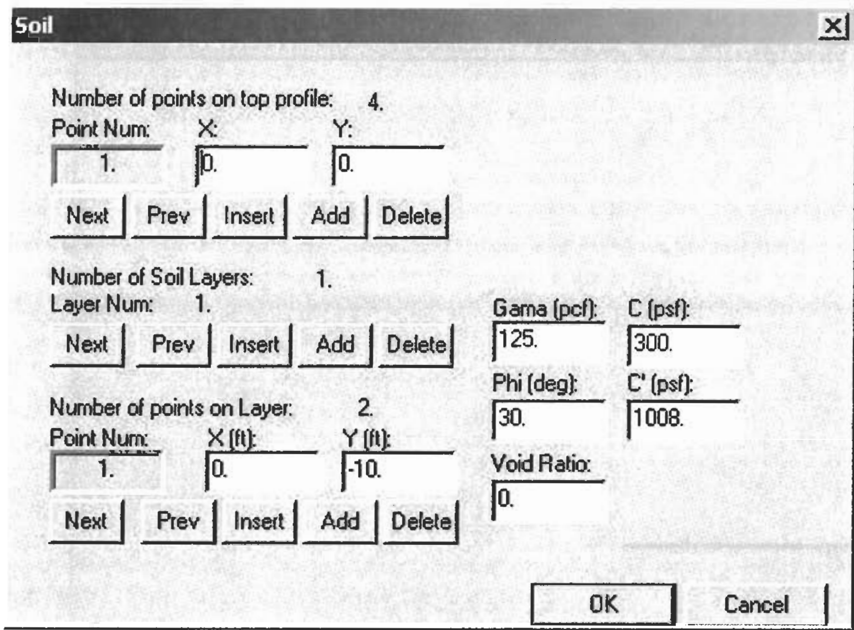
The input data is organized in a number of sets present logical groupings of the input data. The user will enter the data for each set in a dialog box. The dialog boxes are shown below, along with a description of their data items.

Appendix "C"


WinSlope - [Ex1.wsl]
File Edit View Input Run Output Window Help
Ready

Appendix "C"

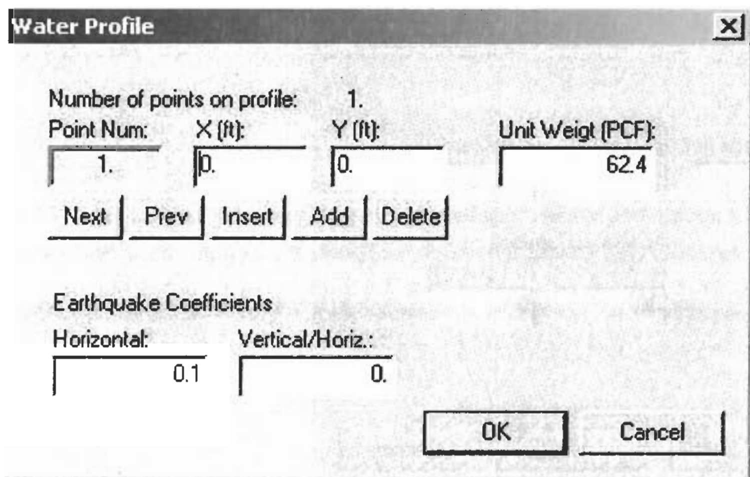
Soil Dialog:



The soil data includes definition of a curve that signifies the wall and the top profile of the soil. This profile is entered as a number of X and Y coordinates on the top part of the dialog box. A number of command buttons allow the user to add intermediate layers. The soil medium is made of a minimum of one layer, but can have as many as necessary to capture the properties of the site. Once a soil layer is selected (or added) the profile of the line defining its base (bottom) is defined via X and Y coordinates given at the lower left corner of the dialog. Each layer also has its distinct properties defined on the right side of the dialog. These properties include:

γ (pcf): Unit weight of soil layer (Dry)
C (psf): The cohesion of the soil layer
φ (deg): The friction angle of the soil
C' (psf): The concrete to soil bond strength
Void Ratio: Void ratio used to calculate the saturated weight of the soil

Water Profile Dialog:



This dialog box is used to define the water profile and the earthquake coefficients. The water profile is entered by giving the X and Y coordinates of points along the profile. As many points as needed may be entered to define the profile. If only one point is defined (0, 0) then the program takes the entire site as being dry. The unit weight of water is also entered in this dialog, but should always be entered as 62.4 PCF.

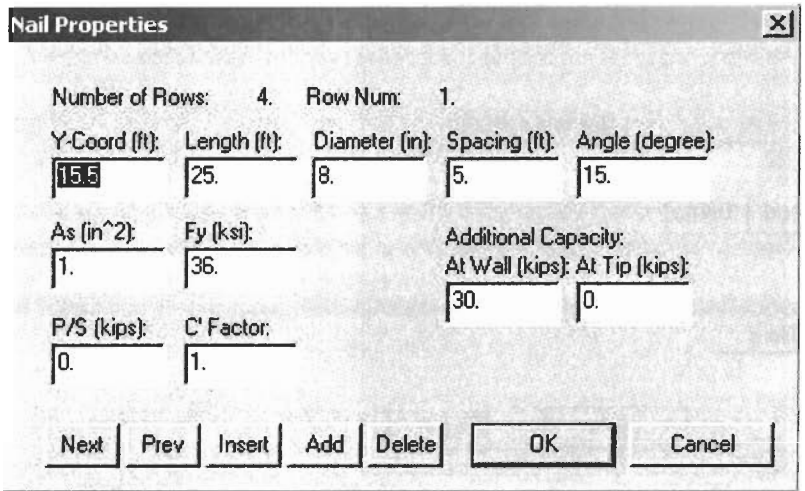
The earthquake coefficients include the following:

Horizontal: The ratio of the weight of the soil to be used as horizontal force acting away from the failure plane.

Vertical/Horiz.: The ratio of vertical to horizontal earthquake force (vertical to horizontal acceleration). The horizontal force found above is multiplied by this ratio, then applied as vertical upward force.

Appendix "C"

Nail Properties Dialog:

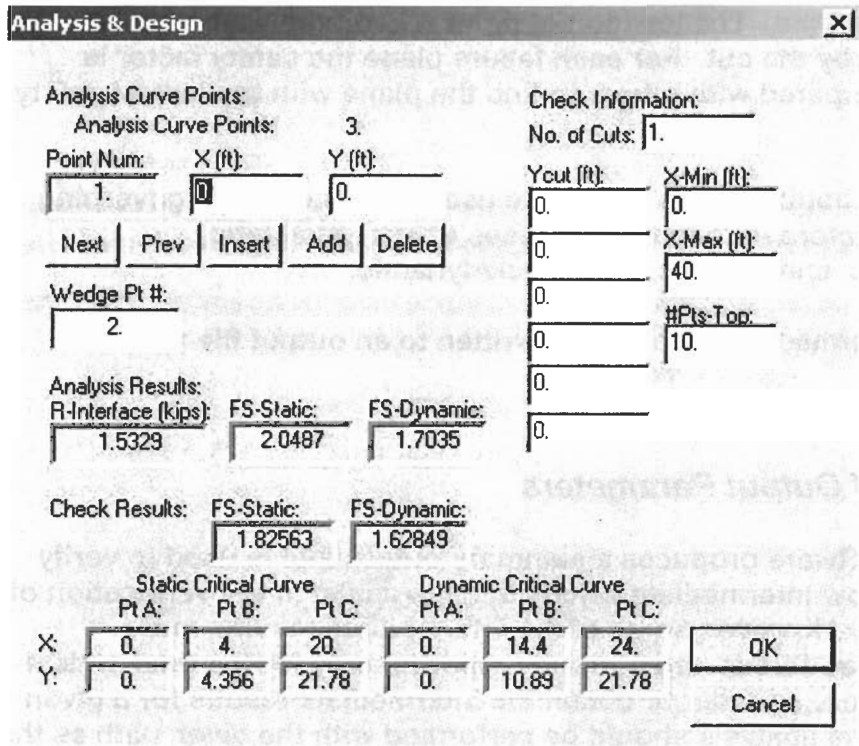


Nail properties are input for each row of nails individually. The data for each row of nails includes the following:

Y-Coord (ft): The Y coordinate at which the nails in this row intersect the wall defined by top soil profile
Length (ft): The length of each nail in this row
Diameter (in): Diameter of the nail in this row
Spacing (ft): The horizontal spacing of the nails in this row
Angle (degree): The inclination of the nails in this row measured from horizontal plane, downward positive.
As (in^2): The area of steel in each nail in this row
Fy (ksi): The yield strength of steel in each nail in this row
Additional Capacity, At Wall (kips): The nail/Wall connection capacity for permanent wall condition
Additional Capacity, At Tip (kips): Optional additional capacity at end of nail (usually 0)
P/S (kips): Amount of post-tensioning force, if applied
C' Factor: A factor to that is applied to C' for increased bond stress within the post-tensioned length. This factor is usually taken as 1.0; i.e., no increase in C' unless recommended by the Geotechnical engineer

Control buttons at the bottom of dialog are for navigation, and adding or deleting nails. Any number of rows of nails may be defined as needed.

Analysis and Design Dialog:



This dialog is used to define a critical surface for analysis, and the depths of cuts for checking (finding) critical surfaces. The final results are also reported in this dialog.

In an analysis case, a pre-defined failure surface is used. In this case a multi-linear surface can be defined by entering the X and Y coordinates of the surface at the top-left corner of the dialog. The first point shall be on the wall (top profile curve) at the bottom of the assumed surface, and the last point shall be beyond the top profile, to define an enclosed area for failure. The failure mechanism in made of two segments much like the bilinear case, and therefore, the point separating the two wedges is defined below the X and Y coordinates. This option is usually used for academic purposes (verification), or when a weak layer of the soil is known to exist in the soil medium.

In the check case, Winslope can try critical planes at up to six cut depths. Each cut depth is defined by its Y coordinate at the bottom the cut given as "Ycut (ft)" at the top-right side of the dialog. In addition, the number of trial grids for critical planes is given by range of X coordinates for point C (top

profile intersection) and the number of C locations to be tried. Therefore, for each depth of cut, a number of C locations are set, and then a number of B locations are tried. The location of point A is on the wall at the Y coordinate given by the cut. For each failure plane the safety factor is obtained and compared with others to find the plane with the lowest safety factor.

The left side and bottom of the dialog are used to present final governing (lowest) safety factors for each of the cases of analysis/static, analysis/dynamic, check/static, and check/dynamic.

The final and intermediate data can be written to an output file via menu options.

4. Definition of Output Parameters

The Winslope software produces a summary output that is used to verify the input and show intermediate calculations to assist in the verification of calculate results. However, some of the intermediate results, more specifically the nail forces, are currently reported only for the final critical surface calculation. In order to obtain the intermediate results for a given failure surface, the analysis should be performed with the given path as the last analysis. The output of the Winslope program is composed of the following data:

A. Echo of Input Data:

1. Site Properties: The top layer profile is the profile of the soil surface after all cuts are made. The X and Y coordinates are measured from an arbitrary origin, usually, the tip of the wall X is measured to the right (into the soil), and Y is measured upward.

Top Layer Profile:

X	Y
0	0
0	18
5.89	21.781
60	21.781

2. Soil Properties: These are reported next for each soil layer. These properties include the profile layer, which is the coordinates of the multi-

linear curve that defines the bottom of the soil layer, followed by the physical properties of the layer. These properties are:

δ = unit weight, C = Cohesion, Phi = Friction angle, Void ratio, and C' = bond stress.

Soil Properties:
Layer Number: 1
Layer Profile:

X	Y
0	-10
60	-10

δ (PCF)	C(PSF)	Phi(Degree)	Void Ratio	C'(PSF)
125	300	30	0	1008

3. Water Properties: This includes the unit weight of water (Gama), and the X and Y coordinates of the multi-linear curves that defined the top of the water line.
Water Profile: = δw =62.4
X Y

4. Nail information: This includes the nail geometry and properties, reported for each row in the order of input data. The nail geometry includes: Row number (sequence), Y=coordinate (height) at which this nail row intersects the wall (Top surface), L=length of nail, d=diameter of the whole, S=horizontal spacing at this row, Theta=inclination angle of the nail, measured clockwise from the horizontal line, As=area of the rebar within the nail, Fy=yield stress of the rebar.

The nail properties include: Row number (sequence), F-Start=shear strength of the wall, F-End=additional capacity that may be added to the end, F-P/S=prestressing force for pre-stressed nails, PS-C' factor=an optional factor to increase the bond stress for pre-stressed nails.

Nail Geometry:

ROW NUMBER	Y(FT)	L(FT)	D(IN)	S(FT)	Θ (DEG)	AS(IN^2) FY(KSI)	
1	15.5	25	8	5	15	1	36
2	11	20	8	5	15	1	36
3	6.5	17	8	5	15	1	36
4	2	15	8	5	15	1	36

Nail Properties:

ROW NUMBER	F-START(KIPS)	F-END (KIPS)	F-P/S (KIPS)	PS-C'FACTOR
1	30	0	0	1
2	30	0	0	1
3	30	0	0	1.25
4	30	0	0	1.25

7. **Predefined Analysis Curve: If an analysis surface is defined (multi-linear), it is echoed in this section. The location of the intersection of the two wedges (point B) is identified next to its coordinates.**

Given Analysis Curves

X	Y
0	0
16	6.52 <-- B
32	21.782

B. Results of Analysis

6. **The critical failure surface that was obtained for each of static and dynamic analyses is reported in this section. The notation Ax, Ay, Bx, By, Cx and Cy refer to the X and Y coordinates of points A, B, and C for each of the two critical surfaces.**

Calculated Failure Surface

Static

Ax	Ay	Bx	By	Cx	Cy
0	0	4	4.356	20	21.78

Dynamic

Ax	Ay	Bx	By	Cx	Cy
0	0	14.4	10.89	24	21.78

C. Intermediate Data: This includes the selected intermediate results to assist in verifying the calculations:

7. Wedge Data: This data includes the weight of each wedge, and its geometry. The wedge weights are reported as saturated weight, buoyant weight, and the dry weight. The angle at the bottom each wedge (slope of line AB or BC) measured from horizontal is also reported for verification.

Wedge #	Sat. Wt	Buoyant Wt	Dry wt (lbs)	Angle(deg)
1	35648.5	35648.5	35648.5	22.1709
2	15260	15260	15260	43.6439

The geometry of each wedge is shown in the following tables. This geometry is simply the coordinates of the top profile augmented with the coordinates of point A, B, B', and/or C as appropriate.

Points for: Wedge 1, 6 Points

X	Y (ft)
0	0
16	6.52
16	21.78
5.89	21.78
0	18
0	0

Points for: Wedge 2, 4 Points

X	Y (ft)
16	6.52
32	21.78
32	21.78
16	21.78

8. Average Soil Properties: This includes the weighted average properties that are used for calculations. The average properties include C (Cohesion), C'(bond stress), and Phi (friction angle) for each of the failure planes: Under Wedge 1 (AB), under Wedge 2 (BC), and Vertical plane (BB').

Under Wedge 1:

C (psf)	C' (psf)	Phi(deg)
300	1008	30

Under Wedge 2:

C (psf)	C' (psf)	Phi(deg)
300	1008	30

On Vertical Plane:

C (psf)	C' (psf)	Phi(deg)
300	1008	30

9. Nail Information:

Nail Intersection and Properties (F = capacity after PS): This includes the nail intersection point with the failure plane, and the calculated nail force. The parameters are: Row number (sequence), X-int & Y-int=X and Y coordinates of the point of intersection of each nail with the failure plane (ABC), Wedge-int=Wedge number intersected by the nail (1=lower, 2=upper), F-Left=Pull-out force for the wall-end, including the wall shear capacity (Per nail), F-Right=Pull out capacity for the tip of the nail (Per nail), F-yield=Tensile strength of the nail rebar (Per nail), and F-control=lowest of the three nail forces divided by the nail spacing (Per unit width).

Row #	X-int.	Y-int.	Wedge-int.	F-left(lb)	F-right(lb)	F-yield(lb)	F-control(lb/ft)
1	19.8412	10.1836	2	73365.4	9413.32	36000	1882.66
2	16.1578	6.67052	2	65314.9	6908.09	36000	1381.62
3	9.62323	3.92146	1	51032.7	14856.8	36000	2971.36
4	2.96099	1.2066	1	36471.6	25195.6	36000	5039.13

This data fallowed by the data related to the pre-stressing of the nails, namely the effective pre-stressing force per unit width, and the anchor length required to achieve the pre-stressing.

Row #	F-PSeff(lb/ft)	Anchor Len(ft)
1	0	0
2	0	0
3	0	0
4	0	0

Next the nail force used in the wedge calculation is reported. This force is reported as the total sum of the nail forces effective within each wedge. The force components are reported along the failure plane, and perpendicular to it. These force components are separated into the forces from the pre-stressing, and the remaining capacity after pre-stressing.

Total Nail Force in wedge

Wedge #	T*Sin(Theta+Alfa)	T*Cos(Theta+Alfa)	PSeff*Sin()	PSeff*Cos()
1	4839.89	6383.05	0	0
2	2787.53	1698.59	0	0

D. Final Results:

10. R3 and Safety Factor: The final results include the interface force, i.e., the reaction force along the vertical plane BB', or R3, which is the value that is obtained by convergence to be the same from the two wedges. If dynamic loads were included, then this R3 value is for the dynamic calculations. Next, the static and dynamic safety factors are reported for the last analysis.

Interface Force = 1532.91 lbs
Safety Factor = 2.04875 (Static), 1.70346 (Dynamic)

5. Verification:

In order to verify the solution, a number of simple geometry cases are solved by long-hand solution. This is performed by performing the trial and errors manually in a spread sheet, until values of R3 match from the two wedges. Hand solutions are shown below. The site can be described as follows.

1. The site is composed of one type of soil. The cut profile is similar to what is shown in Figure 1, with a vertical depth of 18 ft, and top corner dimensions of 3.78' vertical and 5.89' horizontal. Therefore, the X and Y coordinates that define the surface are as follows:

X(ft)	Y(ft)
0.00	0.00
0.00	18.00
5.89	21.78
60.00	21.78

The soil properties for this example are as follows:

Unit Weight = 125 pcf, Cohesion=300 psf, Friction angle (Φ)=30 degrees, Void Ratio=0, Bond Stress=7 psi =1008 psf

This site is stabilized by four nails, with the following properties:

Row Number	Y (ft)	L (ft)	D (in)	S (ft)	Theta (deg)	As (in^2)	Fy (ksi)
1(top)	15.5	25.0	8.0	5.0	15.0	10.	36.0
2	11.0	20.0	8.0	5.0	15.0	1.0	36.0
3	6.5	17.0	8.0	5.0	15.0	1.0	36.0
4(bot)	2.0	15.0	8.0	5.0	15.0	1.0	36.0

In addition, the shear capacity of the wall is 30 kips.

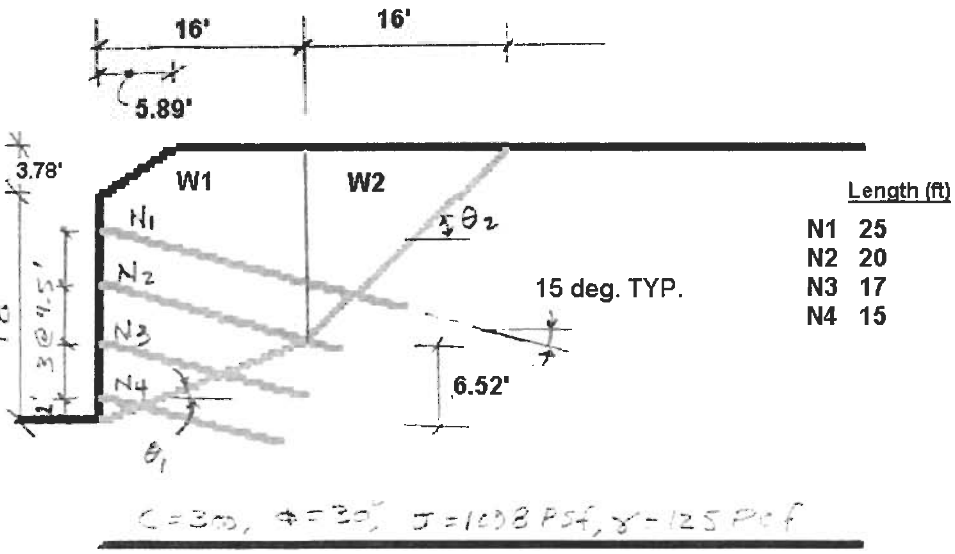
The failure surface ABC is defined as follows:

Point	X	Y
A	0.00	0.00
B	16.00	6.52
C	32.00	21.78

This case is verified via hand solutions and a safety factor of 2.05 is obtained as the solution for this specific failure surface. Note that the given safety factor is not the critical one, and a lower safety factor may be obtained for a different failure surface.

Verification Calculation

The example shown above has been carried out in detailed hand solution as shown below. The problem description can be shown in the figure below. Hand solution in accordance with the theoretical approach is shown in the following pages. The first page of calculations show the calculation of weight of soil wedges and governing nail forces. These values are used in the trial and error calculations in a spreadsheet to verify the safety facto calculated by the program. The verification includes carrying out calculation of R3 from the two wedges based on an assumed safety factor, and comparing their difference with their absolute value, as Epsilon. When the two values are found to be close enough, an acceptable safety factor has been obtained. Plugging this values back into the equations for each wedge, and calculating the normal force on its face from summation of forces in X and Y directions should result in the same normal force value (N). This is used to back check the calculated safety factor and R3 force values.



Verification Example: 18' high wall with 3.78' berm

Wedge Number 1

Area:	16*21.78-16*6.52/2-3.78*5.89/2)=	285.19	ft^2		
Weight:	125*Area=	35,648	lb		
Theta:	Atan(6.52/16)=	22.2	deg.	X	Y
Line eq.:	Y = X*(6.52/16)			9.62	3.92
				2.96	1.21

Wedge Number 2

Area:	16*(21.78-6.52)/2=	122.08	ft^2		
Weight:	125*Area	15,260	lb		
Theta:	Atan[(21.78-6.52)/16]=	43.6	deg.	X	Y
Line eq.:	Y = 6.52 + (X-16)*[(21.78-6.52)/16]			19.84	10.18
				16.16	6.67

Nail intersection

	Equation	X-Intersect	Y-Intersect	With
N1:	Y = 15.5 - X*tan(15)	19.84	10.18	Wedge2
N2:	Y = 11.0 - X*tan(15)	16.16	6.67	Wedge2
N3:	Y = 6.5 - X*tan(15)	9.62	3.92	Wedge1
N4:	Y = 2.0 - X*tan(15)	2.96	1.21	Wedge1

N1:

Length-L:	sqrt(19.84^2+(15.5-10.18)^2)=	20.54	ft
Length-R:	25 - Length-L=	4.46	ft
Force-L:	30000 + Length-L*(pi*8/12)*1008=	73,363	lb
Force-R:	Length-R*(pi*8/12)*1008=	9,416	lb
Force-Y:	As*Fy = 1*36000=	36000	lb
F-Cont:	min(F-L, F-R, F-Y)=	9,416	lb
Force/ft:	F-Cont./H-Spac = F-Cont/5 =	1883	lb/ft

N2:

Length-L:	sqrt(16.6^2+(11.0-6.67)^2)=	16.73	ft
Length-R:	20 - Length-L=	3.27	ft
Force-L:	30000 + Length-L*(pi*8/12)*1008=	65,320	lb
Force-R:	Length-R*(pi*8/12)*1008=	6,903	lb
Force-Y:	As*Fy = 1*36000=	36000	lb
F-Cont:	min(F-L, F-R, F-Y)=	6,903	lb
Force/ft:	F-Cont./H-Spac = F-Cont/5 =	1381	lb/ft

N3:

Length-L:	sqrt(9.62^2+(6.5-3.92)^2)=	9.96	ft
Length-R:	17 - Length-L=	7.04	ft
Force-L:	30000 + Length-L*(pi*8/12)*1008=	51,026	lb
Force-R:	Length-R*(pi*8/12)*1008=	14,864	lb
Force-Y:	As*Fy = 1*36000=	36000	lb
F-Cont:	min(F-L, F-R, F-Y)=	14,864	lb
Force/ft:	F-Cont./H-Spac = F-Cont/5 =	2973	lb/ft

Appendix "C"

N4:

Length-L:	sqrt(2.96^2+(2.0-1.21)^2)=	3.06	ft
Length-R:	15 - Length-L=	11.94	ft
Force-L:	30000 + Length-L*(pi*8/12)*1008=	36,469	lb
Force-R:	Length-R*(pi*8/12)*1008=	25,198	lb
Force-Y:	As*Fy = 1*36000=	36000	lb
F-Cont:	min(F-L, F-R, F-Y)=	25,198	lb
Force/ft:	F-Cont./H-Spac = F-Cont/5 =	5040	lb/ft

Verification Example: 18' high wall with 3.78' berm

S.F. =	2.05				Phi (rad)	F3	F3+C3L3
R3	Phi3	L3	C3				
1835	30	15.28	300		0.523599	1059.438	5643.438
			146.3415		0.274524		

Wedge Number 1

Nail Force	Nail Angle	Nail Angle	Nail Fx	Nail Fy
5039	15	0.261799	4867.30	1304.19
2971	15	0.261799	2869.77	768.95
0	0	0	0.00	0.00
8010			7737.07	2073.14

A1	B1	A2	B2	f	R3	
0.117084	6104.662	1.032284	37946.33	0.281634	1744.964	1812.848
					1812.848	

Theta	Phi	L	C	W
22.2	30	17.2	300	35650
0.387463	0.523599		146.3415	
	0.274524			

N-(Fx=0) =	36466.69	37235.68
N-(Fy=0) =	37260.24	

Wedge Number 2

Nail Force	Nail Angle	Nail Angle	Nail Fx	Nail Fy
1382	15	0.261799	1334.91	357.69
1883	15	0.261799	1818.84	487.36
0	0	0	0.00	0.00
3265			3153.75	845.04

A1	B1	A2	B2	f	R3	
0.485668	3880.491	0.918392	11205.79	0.281634	1780.256	1849.512
					1849.512	

Theta	Phi	L	C	W
43.6	30	22.1	300	15260
0.760964	0.523599		146.3415	
	0.274524			

N-(Fx=0) =	11768.31	11655.59
N-(Fy=0) =	11638.81	

Epsilon= -0.01001

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Pirooz Barar & Associates
Structural Engineering

6. Modeling Techniques

Care must be exercised in preparing input, as modeling inaccuracies can lead to non-convergence and bad results. Some of the points to keep in mind are shown below:

1. Surcharge loads cannot be input in Winslope directly at this time. However, to model the surcharge forces, it is possible to add a fictitious layer with minimal strength, but just weight to the top of the top of the profile. This layer should be kept small, say less than a foot thick, to not allow an intersection point (B) fall within it. This layer can have a variable thickness as needed to model various surcharge loads.

2. When stepped walls are modeled, the wall profile (top profile) should have a positive slope in all its segments. Although it is possible to model flat surfaces, if a cut depth falls at the height of a step, then the location of point A is not well-defined. In this case if the Y-coordinate of the start and the end of a step are different by a fraction of a foot (say 0.01 ft) all points can be calculated accurately.

3. When stepped walls are considered, it is possible that a local failure surface becomes the critical one. If it is desired to obtain the critical safety factor for a critical surface that extends to the top of the wall, then the starting location for point C should be specified at a coordinate larger than the width of the step.

4. If after an analysis, the critical failure surface is located at the maximum coordinate for point C, then the trial space should be expanded to make sure that the critical surface is not at a farther location and to obtain the true critical failure surface.

5. Care must be exercised to make sure that the soil layers do not cross each other.

6. Care must be exercised to make sure that nails do not cross each other.

7. Care must be exercised to make sure the soil profiles extend to the end of the solution space.

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Appendix "D" Excerpts from Reference 6

where R_b = pull in pounds per bolt
ℓ = the driving distance of a single sheet pile (*if* each section is bolted)
F.S. = a desired safety factor to cover stresses induced during bolting (between 1.2 and 1.5)

The fixing plate (as shown in Figure 39, Section A-A) may be designed as a beam simply supported at two points (the longitudinal webs of the wale) and bearing a single load, R_b, in the center.

The wales are field bolted at joints known as fish plates or splices, as shown in Figure 39, Section C-C. It is preferable to splice both channels at the same point and place the joint at a recess in the double piling element. Splices should be designed for the transmission of the bending moment. The design of tie rods and wales is illustrated in Problem No. 4a (pages 107-110).

ANCHORS

The stability of an anchored sheet pile bulkhead depends mainly on the stability of the anchor device to which the wall is fastened. The reaction of the tie rods may be carried by any one of the types of anchorages shown in Figure 40.

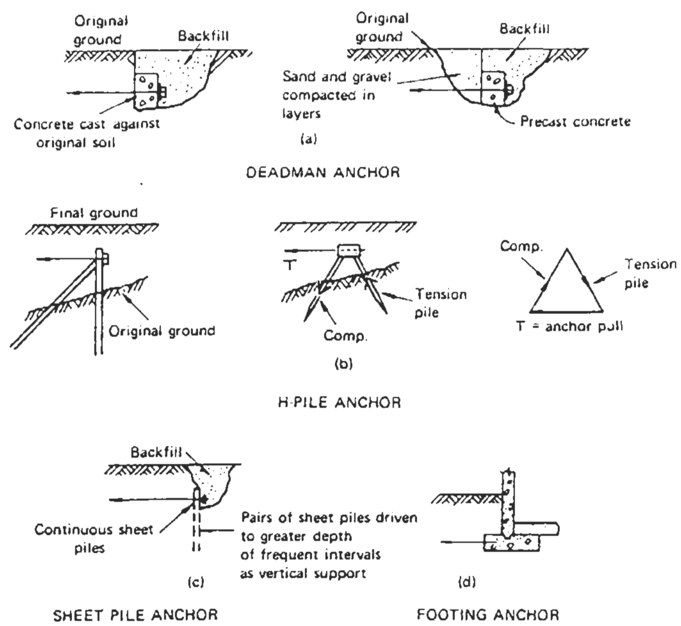

Fig. 40 — Types of anchorage systems (after Teng[1])

Location of Anchorage — In order for an anchorage system to be effective it must be located outside the potential active failure zone developed behind a sheet pile wall. Its capacity is also impaired if it is located in unstable ground or if the active failure zone prevents the development of full passive resistance of the system. Figure 41 shows several installations that will not provide the full anchorage capacity required because of failure to recognize the above considerations.

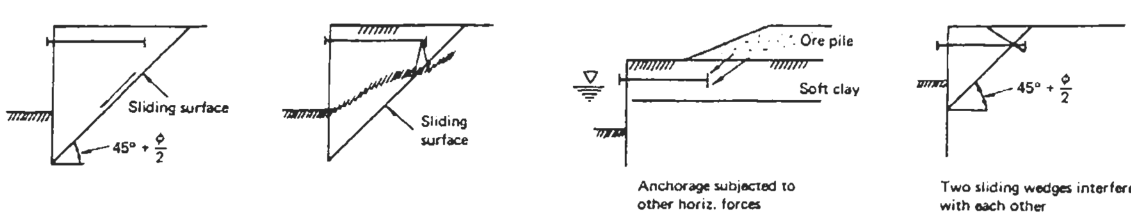

Fig. 41 — Installations having reduced anchorage capacity (after Teng[1])

Figure 42 shows the effect of anchorage location on the resistance developed.

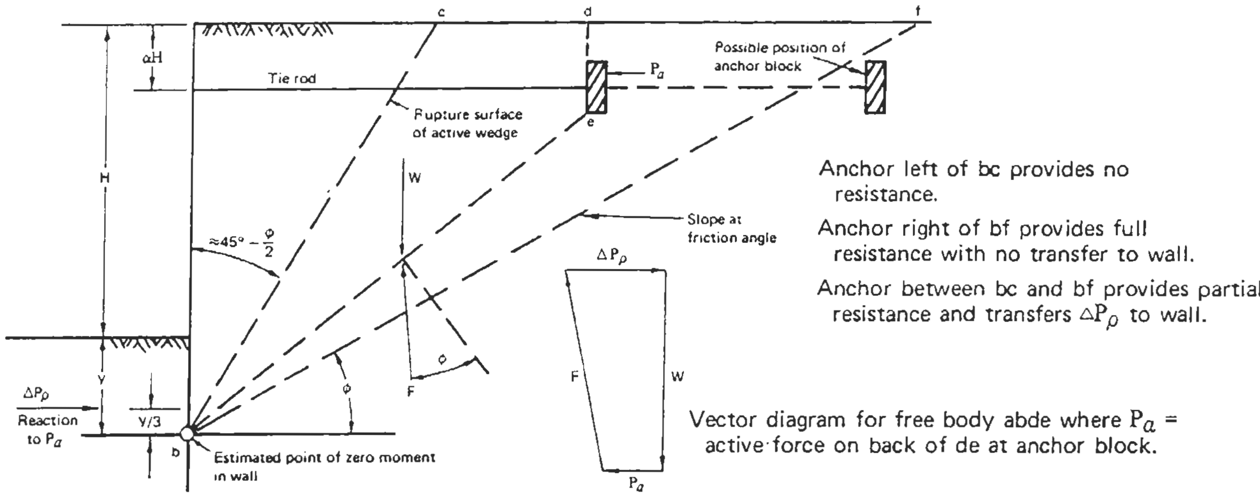

Fig. 42 — Effects of anchor location relative to the wall (after Navdocks[11], Terzaghi[13])

If the anchorage is located between bc and bf, only partial resistance is developed due to the intersection of the active and passive failure wedges. However, the theoretical reduction in anchor capacity may be analytically determined (see *Theoretical Soil Mechanics* by K. Terzaghi,[13] p. 232.)

Sheet Pile Anchor Walls — Short steel sheet piles driven in the form of a continuous wall may be used to anchor tie rods. The tie rods are connected with a waling system similar to that for the "parent" wall, and resistance is derived from passive pressure developed as the tie rod pulls against the anchor wall. To provide some stability during installation of the piling and the wales, pairs of the piling should be driven to a greater depth at frequent intervals. The anchor wall is analyzed by conventional means considering full passive pressure developed only if the active and passive failure zones do not intersect. However, if the failure wedges do intersect, the total passive resistance of the anchor wall will be reduced by the amount

$$\Delta P_p = (K_p - K_a)\,\frac{\gamma (h_2)^2}{2} \qquad \text{(for granular soils)}$$

where h_2 = depth to the point of intersection of the failure wedges as shown in Figure 43.

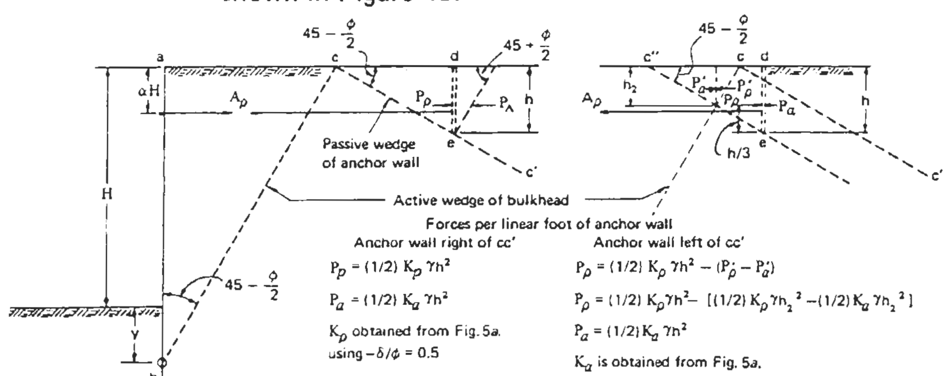

Fig. 43 — Continuous anchor wall (after Navdocks[11])

The tie rod connection to the anchorage should be ideally located at the point of the resultant earth pressures acting on the anchorage. Problem No. 4a (pages 107-110) illustrates the design of sheet pile anchor walls.

Deadmen Anchors — The effects of interaction of the active and passive failure surfaces, as mentioned above, also apply to the design of deadmen anchors.

Care must be exercised to see that the anchor block or deadman does not settle after construction. This is generally not a problem in undisturbed soils, however, where the anchorage must be located in unconsolidated fill, piles may be needed to support the blocks. Also, the soil within the passive wedge of the anchorage should be compacted to at least 90 per cent of maximum density unless the deadman is forced against firm natural soil.

Continuous Deadmen Near Ground Surface — A continuous deadman is shown in Figure 44.

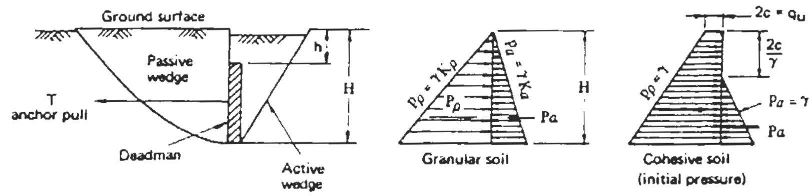

Fig. 44 — Continuous deadmen near ground surface (after Teng[1])

If $\frac{1}{2}H > h$, assume deadman extends to ground surface and the ultimate capacity of the deadman is

$$T_{ult} = P_p - P_a$$

where T_{ult} = ultimate capacity of the deadman, pounds per linear foot
P_p = total passive earth pressure, pounds per linear foot
P_a = total active earth pressure, pounds per linear foot

The active and passive pressure distributions for granular and cohesive soils are also shown in Figure 44. For design in cohesive soils, both the immediate and the long-term pressure conditions should be checked to determine the critical case. A safety factor of two against failure is recommended; i.e.,

$$T \leq T_{ult}/2$$

Short Deadmen Near Ground Surface — Figure 45 shows a deadman of length, L, located near the ground surface, subjected to an anchor pull, T. Experiments have indicated that at the time of failure, due to edge effects, the heave of the ground surfaces takes place in an area as shown. The surface of sliding at both ends is curved.

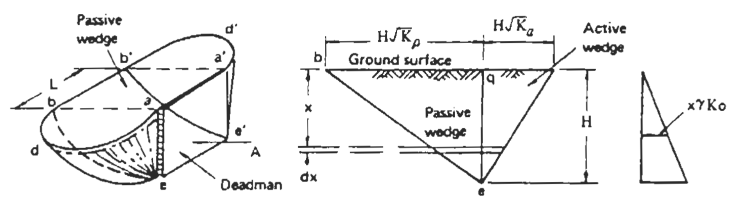

Fig. 45 — Short deadmen near ground surface (after Teng[1])

Integration of the resistance along these curved sliding surfaces results in the following expression for the ultimate capacity of short deadmen in granular soils

$$A_{ult} \leq L\,(P_p - P_a) + \frac{1}{3} K_o \gamma \,(\sqrt{K_p} + \sqrt{K_a})\, H^3 \tan\phi$$

where A_{ult} = ultimate capacity of the deadman, pounds
L = length of the deadman, feet
P_p, P_a = total passive and active pressure, pounds per lineal foot
K_o = coefficient of earth pressure at rest. (It may be taken as 0.4 for design of deadman)
γ = unit weight of soil, pounds per cubic foot.
K_p, K_a = coefficients of passive and active earth pressure.
H = height of deadman, feet.
ϕ = angle of internal friction.

For cohesive soils, the second term in the above expression should be replaced by the cohesive resistance, thus

$$A_{ult} \leq L\,(P_p - P_a) + 2cH^2$$

where c = the cohesion of the soil, pounds per square foot.

ANCHOR SLAB DESIGN BASED ON MODEL TESTS

General Case in Granular Soils

N. K. Ovesen[50] conducted 32 different model tests in granular soil and developed a procedure for designing anchor slabs located in a zone where the anchor resistance can be fully mobilized. The proposed method considers that the earth pressure in front of the slab is calculated on the basis of a rupture surface corresponding to a translation of the slab. This method can be used to solve the general case in Figure 46 (a) for rectangular anchors of limited height and length located at any depth as shown in Figure 46 (b). Surface loads behind the anchor slab are not included in this publication since their influence is small on the anchor resistance for granular soils with an angle of internal friction equal to or greater than 30 degrees.

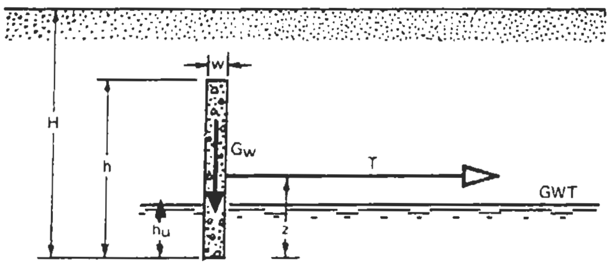

Fig. 46 (a) — Geometrical parameters for an anchor slab.

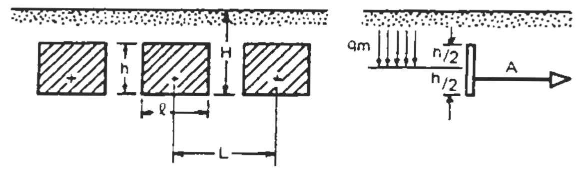

Fig. 46 (b) — Geometrical parameters for anchor slabs with limited height and length.

Where

A	=	resultant anchor force per slab, lbs.
GWT	=	ground water table
G_w	=	weight per foot of wall of the anchor plus the soil on top of the slab, lbs. per foot
H	=	distance from base of slab to ground surface, ft.
L	=	distance between centers of two consecutive slabs, ft.
T	=	resultant anchor force, lbs. per foot
W	=	thickness of anchor slab, ft.
Z	=	distance from base of slab to resultant anchor force, ft.
h_u	=	distance from base of slab to ground water table, ft.
h	=	actual height of anchor slab, ft.
ℓ	=	actual length of anchor slab, ft.
q_m	=	vertical effective stress in earth at midpoint of actual height of anchor slab, lbs. per square foot
γ	=	unit weight of soil, lbs. per cubic foot
γ'	=	submerged unit weight of soil, lbs. per cubic foot

Ovesen suggests that a two-step procedure be used to find the ultimate resistance of the anchor per slab A_{ult} which equals $q_m h \ell R$. First the dimensionless anchor resistance factor, R_o, is determined for the "basic case". The basic case is a continuous strip, $\ell = L$, extending the full height, $h = H$, of the anchor. Next, the dimensionless anchor resistance factor, R, which is dependent upon R_o is calculated for the actual anchor dimensions under consideration. Knowing R, the ultimate resistance of the anchor slab A_{ult} can be calculated. A similar two-step procedure is used to find Z, the location of the line of action of the anchor tie-rod force. The application of Ovesen's method is described below and illustrated in Problem No. 4b (pages 111-113).

1. Determine the dimensionless anchor resistance factor, R_o, for the "basic case". For a given angle of internal friction, ϕ, and angle of wall friction, δ, calculate $\tan \delta$, and use Figure 46 (c) to obtain the earth pressure coefficient, $K\gamma$. Calculate the Rankine active earth pressure coefficient K_a, and then solve for R_o.

$$K_a = \tan^2 (45 - \phi/2)$$
$$R_o = K\gamma - K_a$$

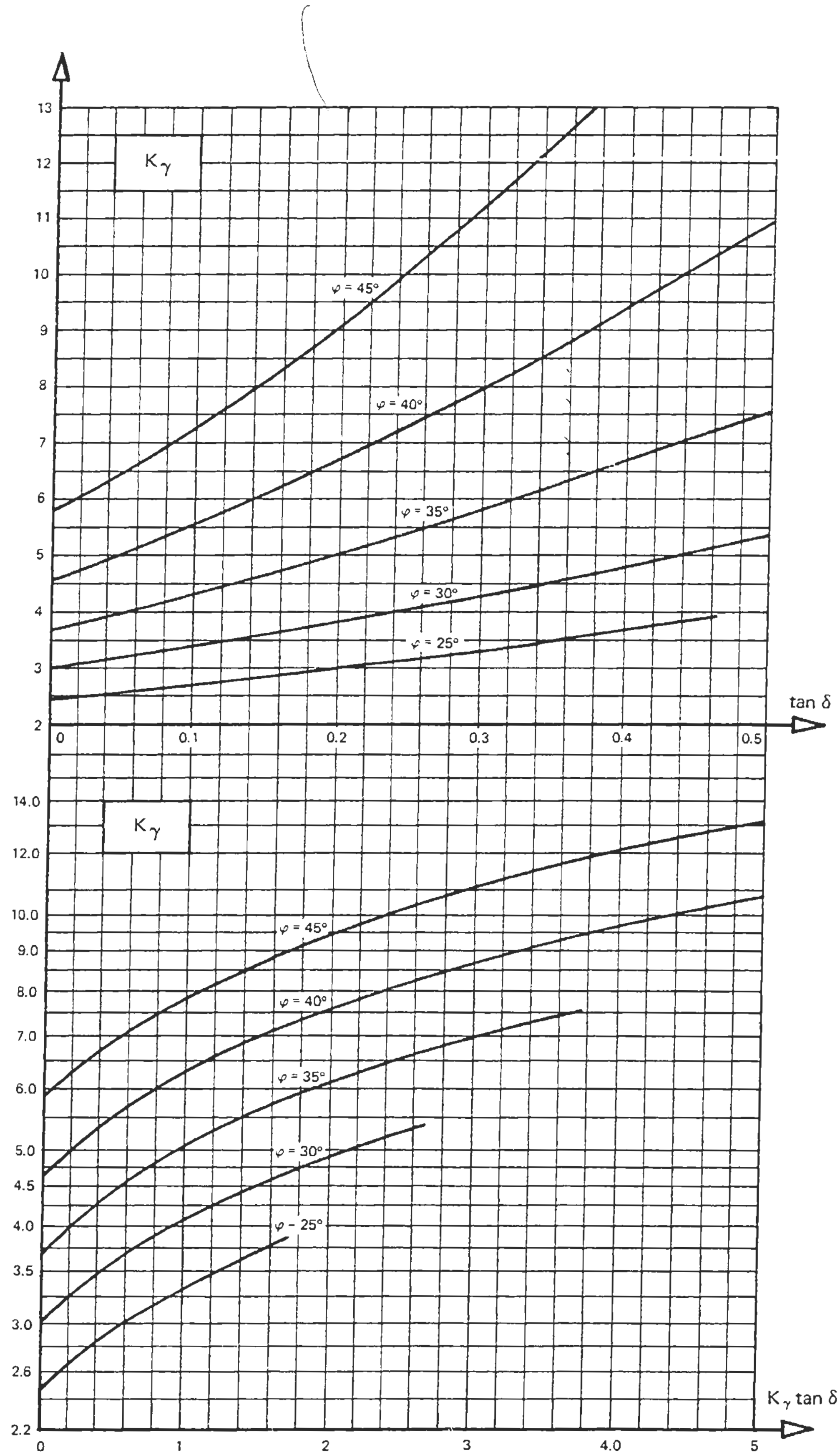

Fig. 46 (c) — Earth pressure coefficients for the normal earth pressure in front of an anchor slab. (after Ovesen[50])

Calculate the hydrostatic earth pressure per foot of wall,

$$P_H = \frac{1}{2}\gamma H^2 - \frac{1}{2}(\gamma - \gamma')\,h_u^2$$

Calculate, T_o, the ultimate anchor resistance per foot of wall for the "basic case",

$$T_o = P_H R_o$$

The following method is recommended for obtaining $K\gamma$ for those cases where the tangent of the angle of wall friction, tan δ, is not known:

Calculate the normal and tangential active earth pressure per foot of wall on the back of the slab,

$$P_A = P_H K a$$

$$F_A = -P_A \tan\phi$$

Calculate G_W, which is the weight per foot of wall of the anchor plus the soil on top of the slab, then

$$K\gamma \tan\delta = \frac{G_W - F_A}{P_H}$$

Use Figure 46 (c) to obtain $K\gamma$.

2. The dimensionless resistance factor, R, for the actual anchor slab dimensions is then calculated by the formula[51] below or by the use of Figure 46 (d), which is the below equation plotted for values of ℓ/L, ℓ/h, and h/H.

$$R/R_o = 1 + R_o^{2/3}\left(1.1E^4 + \frac{1.6B}{1+5\ell/h} + \frac{0.4\,R_o\,E^3\,B^2}{1+0.05\ell/h}\right)$$

where $E = (1 - h/H)$ and $B = 1 - (\ell/L)^2$

3. The ultimate anchor resistance per slab, A_{ult}, and the ultimate anchor resistance per foot of wall, T_{ult}, are equal to,

$$A_{ult} = q_m\,h\ell R$$

$$T_{ult} = A_{ult}/L$$

where q_m is the vertical effective stress in the earth at the midpoint of the actual height of the anchor slab, $q_m = \gamma(H - 1/2\,h)$.

4. The location of Z shown in Figure 46 (a), which is the line of action of the anchor tie-rod force, can be obtained directly from Figure 46 (e) when the ground water table is at or below the anchor slab base ($h_u = 0$).

5. Use the following method to find Z when the ground water table is above the anchor slab base ($h_u \neq 0$). Calculate M_H, the hydrostatic earth pressure moment, about the base of the anchor (Figure 46 (a)).

$$M_H = \frac{1}{6}\gamma H^3 - \frac{1}{6}(\gamma - \gamma')\,h_u^3$$

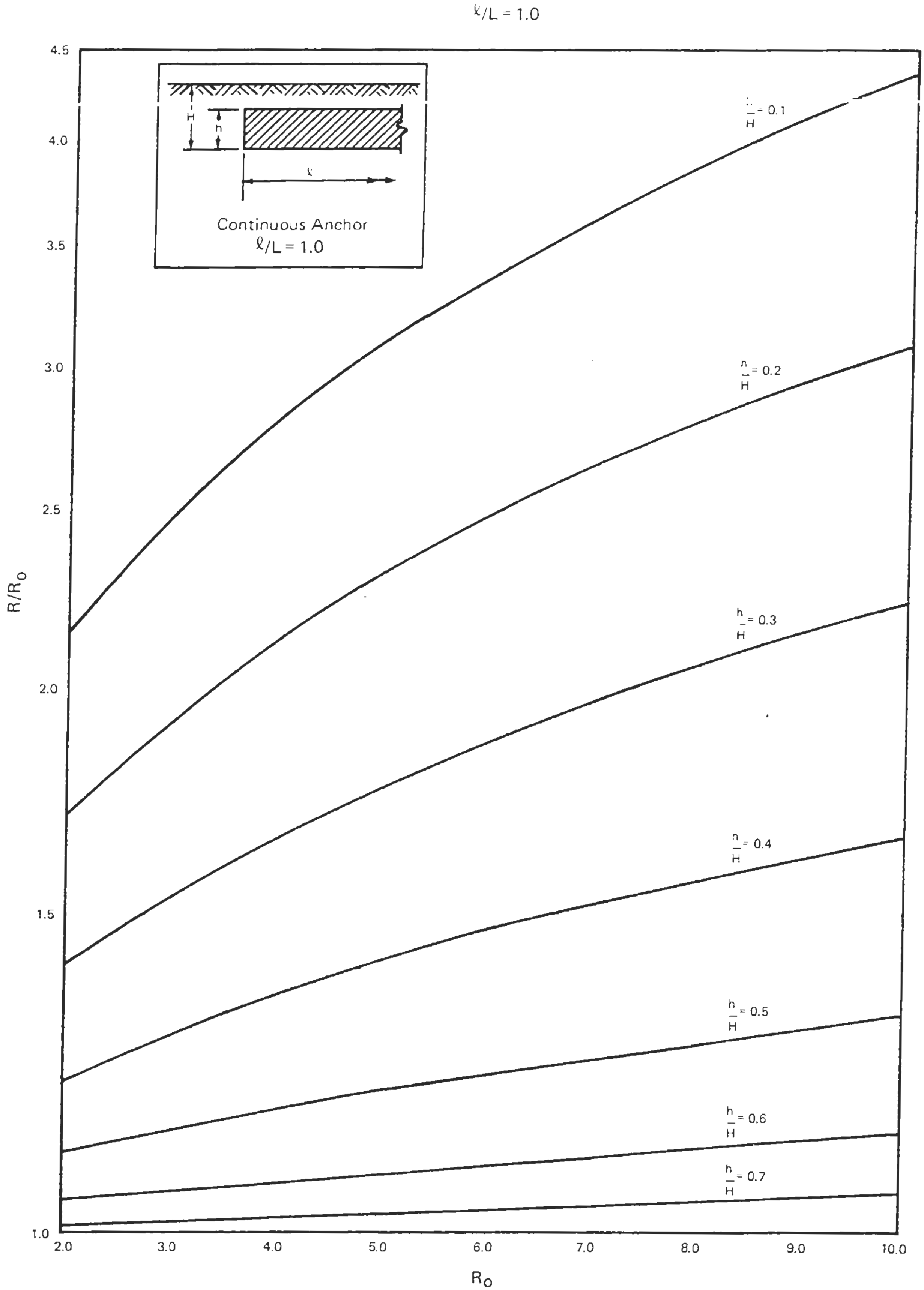

Fig. 46 (d) — Dimensionless resistance factor ratio for continuous anchor slab, ℓ/L = 1.0

Appendix "E"

Structural Design Criteria by PB&A, Inc.
Prepared for :
Glendale Adventist Medical Center
Acute Care Facility, Phase I / Alternate Method Of Compliance OSHPD Application #IL-021538

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